



MDOT RC-1629

Remote Monitoring of Fatigue-sensitive Details on Bridges

MARCH 2015



Department of Civil & Construction Engineering
College of Engineering and Applied Sciences
Western Michigan University

RESEARCH

Intentionally left blank

1. Report No. RC-1629	2. Government Accession No. N/A	3. MDOT Project Manager Steve Kahl, P.E.	
4. Title and Subtitle Remote Monitoring of Fatigue-sensitive Details on Bridges		5. Report Date 03/30/2015	
		6. Performing Organization Code N/A	
7. Author(s) Upul Attanayake, Ph.D., P.E. and Haluk Aktan, Ph.D., P.E., Western Michigan University. Robert Hay, Ph.D., TISEC Inc. Necati Catbas, Ph.D., P.E., Transtek International Group LLC.		8. Performing Org. Report No. N/A	
		9. Performing Organization Name and Address Western Michigan University 1903 West Michigan Avenue Kalamazoo, Michigan 49008	
12. Sponsoring Agency Name and Address Michigan Department of Transportation Research Administration 8885 Ricks Road P.O. Box 30049 Lansing, MI 48909		10. Work Unit No. (TRAIS) N/A	
		11. Contract No. 2010-0297	
		11(a). Authorization No. Z7	
15. Supplementary Notes		13. Type of Report & Period Covered Final Report 10/01/2012 - 03/30/2015	
		14. Sponsoring Agency Code N/A	
16. Abstract Fatigue is one of the most critical problems for steel bridges as well as for any steel structures that needs to be considered during design and operation. The objectives of this study are to explore monitoring technologies, and to develop effective structural and data analysis strategies as well as implementation recommendations for evaluating performance of fatigue-sensitive details and retrofits in steel bridges. Acoustic emission (AE) was selected as a candidate inspection technology, and a monitoring system was installed on a bridge. In general, the performance of the monitoring system and associated software is satisfactory. The majority of AE monitoring challenges are associated with AE data analysis and interpretation of results. In this study, cluster analysis and non-linear mapping signal analysis techniques are used to group AE data with similar waveform characteristics. The presence of the signals that resemble the characteristics of crack opening signals, noise, and structural resonance is identified through waveform analysis. Once the presence of crack opening signals is confirmed, the source location plots are utilized to assess the concentration and the level of activity at the locations of interest. A significant difference is observed in the fatigue life calculated using measured stress, and the stresses calculated using finite element models loaded with a fatigue truck. Hence, a two-tier implementation process is recommended. Tier I process includes the assessment of bridges with repaired details. Tier II process recommends evaluating the entire bridge population with fatigue-sensitive details. Additional recommendations include implementing AE data interpretation capability in an on-line system to provide reliable input with minimal interpretation requirements for inspection-based maintenance management.			
17. Key Words Acoustic emission, bridge, fatigue, fatigue life, hot spot stress, monitoring		18. Distribution Statement No restrictions. This document is available to the public through the Michigan Department of Transportation.	
19. Security Classification - report Unclassified	20. Security Classification - page Unclassified	21. No. of Pages 179 (excluding appendices)	22. Price N/A

Intentionally left blank

Remote Monitoring of Fatigue-sensitive Details on Bridges

Project Manager: Steve Kahl, P.E.

Submitted to:



Submitted by

Upul Attanayake, Ph.D., P.E.
Associate Professor
Western Michigan University
(269) 276 – 3217
upul.attanayake@wmich.edu

Haluk Aktan, Ph.D., P.E.
Professor
Western Michigan University
(269) 276 – 3206
haluk.aktan@wmich.edu

Robert Hay, Ph.D.
TISEC Inc.
Morin Heights, QC, Canada
(450) 226 - 6804
bobhay@structuralinsights.com

Necati Catbas, Ph.D., P.E.
Transtek International Group LLC
fncatbas@gmail.com
(321) 945-1786



Western Michigan University
Department of Civil & Construction Engineering
College of Engineering and Applied Sciences
Kalamazoo, MI 49008-5316
Fax: (269) 276 – 3211

Intentionally left blank

DISCLAIMER

“This publication is disseminated in the interest of information exchange. The Michigan Department of Transportation (hereinafter referred to as MDOT) expressly disclaims any liability, of any kind, or for any reason, that might otherwise arise out of any use of this publication or the information or data provided in the publication. MDOT further disclaims any responsibility for typographical errors or accuracy of the information provided or contained within this information. MDOT makes no warranties or representations whatsoever regarding the quality, content, completeness, suitability, adequacy, sequence, accuracy or timeliness of the information and data provided, or that the contents represent standards, specifications, or regulations.”

Intentionally left blank

ACKNOWLEDGEMENTS

This project was funded by the Michigan Department of Transportation. The authors would like to acknowledge the support and effort of Mr. Steve Kahl for initiating this research. The authors also wish to acknowledge the continuing assistance of the Research Advisory Panel (RAP) members in contributing to the advancement of this study. Continuous technical support provided by Mr. Juan Mejia, Vice President, Engineering & Inspections, TISEC Inc., is greatly appreciated. Support and efforts of graduate research assistant Lizmert Lopez and undergraduate research assistants, Timothy Schnell and Simon Matar, are acknowledged. Authors also would like to acknowledge the support of Dr. Diana Prieto. Last but not least, the support of the department technician, John Cernius, is greatly appreciated.

Intentionally left blank

EXECUTIVE SUMMARY

INTRODUCTION

Fatigue is one of the most critical problems for steel bridges as well as for any steel structures that needs to be considered during design and operation. The objectives of this study are to explore monitoring technologies, and to develop effective structural and data analysis strategies as well as implementation recommendations for evaluating performance of fatigue-sensitive details and retrofits in steel bridges.

Fatigue cracking is developed at certain steel bridge details due to a direct result of the loads (load-induced fatigue) or a deformation that is not accounted for during design (distortion-induced fatigue). The details that are prone to load-induced fatigue can be identified using the detail categories presented in the American Association of State Highway Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications Table 6.6.1.2.3-1. The secondary and/or distortion-induced stresses that are not typically used in design are the most common reasons for fatigue cracking developed in bridges. The existence and the need to evaluate distortion-induced fatigue are acknowledged in the Manual for Bridge Evaluation (MBE) as well as in the National Cooperative Highway Research Program (NCHRP) Project 12-81 proposed revisions; yet, the scope of the manual as well as the proposed revisions are limited to assessing the cumulative fatigue damage of uncracked members subjected to load-induced stresses. Therefore, identifying the details that are prone to distortion-induced fatigue cracking requires utilizing information in bridge files, refined analysis capabilities, and experience. Developing high-fidelity analysis models and/or effective monitoring systems is vital to evaluate the causes of cracking or potential for cracking as well as to estimate remaining fatigue life, especially when evaluating complicated details or the effect of secondary stresses. A majority of the fatigue problems are associated with weld terminations and weld defects. The hot spot stress (HSS) method is widely used in other disciplines such as ship building and offshore structures for the assessment of weldments. However, the HSS method applications in bridge engineering are rarely documented.

The Michigan Department of Transportation (MDOT) performs inspections of over 200 bridges with fatigue-sensitive details. MDOT is interested in identifying technology that can be implemented on bridges with fatigue-sensitive details as well as to learn the recent advances in

fatigue-sensitive detail assessment and retrofits. Hence, this study is completed with five tasks: (1) review the state-of-the-art and practice literature to identify technologies for a structural health monitoring (SHM) system, (2) select a bridge and perform structural analysis to identify details for monitoring, (3) procure an SHM system, (4) install and calibrate the system, and analyze data, and (5) develop recommendations for technology integration into MDOT practice.

LITERATURE REVIEW

Fatigue-sensitive detail assessment methods, technology for fatigue-sensitive detail monitoring, technology implementation considerations, and retrofit methods for fatigue-sensitive details were reviewed within the scope of this project. A summary of the key findings from the literature review is given below:

- The AASHTO MBE Section 7 presents a procedure to assess the cumulative fatigue damage of uncracked members subjected to load-induced stresses. The outcome of this process has been in much debate because the calculation process, in many cases, resulted in a negative remaining fatigue life indicating that the detail had expired several years before the assessment was undertaken even when the in-service bridge in good condition. Hence, the NCHRP Project 12-81 proposed revisions to Section 7 and presented a process to calculate a non-dimensional parameter (the fatigue serviceability index - Q), fatigue rating, and guidance for using the fatigue serviceability index in bridge management decision-making.
- A combined effort of utilizing information in bridge files, refined analysis capabilities, and experience is needed to identify the details that are prone to distortion-induced fatigue cracking. The HSS method is widely used for the assessment of weldments. The International Institute of Welding (IIW) presents guidelines for HSS calculation as well as for using S-N curves to assess weldments.
- The fracture mechanics approach with an assumed initial crack size can be used to estimate crack growth rate; thus, to determine the inspection frequency. When such an approach is implemented, an accurate estimation of the initial crack size is important. The suggested approach is inspecting the detail using a nondestructive testing technique and selecting the largest non-detectable crack as the initial crack size.
- When a large number of bridges with fatigue-sensitive details or fracture-critical members (FCM) are present, bridges that require detailed evaluation and implementation of structural

health monitoring systems can be identified by grouping the bridges based on inventory data and performing detailed structural analysis of representative bridges from each group.

- An acoustic emission (AE) monitoring system with strain gages is one of the most effective technologies for fatigue event detection (i.e., crack initiation or crack growth monitoring). AE has been successfully implemented in the field and evaluated for continuous monitoring of fatigue-sensitive details. At this time, AE is the only technology that is capable of real-time monitoring of fatigue events and providing data for damage location detection. In addition to the AE sensors, strain gauges are required to evaluate the stress state to calculate remaining fatigue life, and to support AE data analysis by developing a load matrix.
- An extensive list of AE technology implementation challenges is prepared within the scope of this project. The majority of the challenges are associated with noise elimination, AE signal analysis, and interpretation of the results.
- AE technology can be implemented for global, semi-global, or local monitoring. Global monitoring is to identify potential source locations for planning or asset management purposes. Semi-global monitoring is implemented to identify individual source locations to assess, evaluate, and rank. The ranking is based on the source characteristics and number of emissions. The ranking system is used for planning detailed investigations through local monitoring. The local monitoring is implemented to identify the source location and to characterize the AE events. The sensor array for local monitoring is designed based on (a) the classification of the detail to be monitored, (b) prior experience with similar details, (c) inspection and maintenance records, (d) an attenuation survey, and (e) finite element analysis or a combination thereof.
- Monitoring duration is defined as short-term, loading pattern dependent, and long-term or continuous. Short-term monitoring is suitable when the causes and damage mechanisms are well understood. Short-term monitoring is also used as a means of repair/retrofit quality control. The duration for short-term as well as loading pattern dependent monitoring needs to be selected based on the traffic patterns. Long-term or continuous monitoring is primarily implemented when fatigue-sensitive or fracture-critical details are present, and the bridge is a critical node of a road network.
- Several repair and retrofit methods are presented in the report. The fatigue resistance of welds can be enhanced with the ultrasonic impact treatment (UIT). This method is relatively new,

and following manufacturer specifications is recommended. Repair or retrofit methods for out-of-plane distortion are presented. A majority of these methods requires evaluating alternatives through refined structural analysis and developing implementation recommendations.

STRUCTURAL MODELING AND HOT SPOT STRESS ANALYSIS

The bridge (S16 of 11015) that carries I-94 EB over Puetz Road, located in Stevensville, Michigan was selected for monitoring system implementation and performance evaluation. The bridge consists of category C' fatigue-sensitive partial depth diaphragm details and category E fatigue-sensitive welded cover plate detail. In addition to having fatigue-sensitive details, the other reasons for selecting the bridge include heavy truck traffic and easy access to the bridge for instrumentation and system maintenance. The primary objective of this study was to evaluate the state-of-the-art technology; hence, bridge access was an important parameter when selecting the site.

A 3D finite element (FE) model of the bridge was developed, and a submodeling concept was implemented to calculate hot spot stresses using the quadratic extrapolation equation recommended by the International Institute of Welding. The permanent stresses developed at the weld toe were calculated using deck dead load as a construction load. The fatigue truck given in AASHTO LRFD is a notional truck. Hence, Michigan legal load configurations were used to evaluate the stress state at the weld toe, representing more realistic loads on the bridge. Based on the analysis results, two web gap details located underneath the truck lane were selected for instrumentation.

STRUCTURAL HEALTH MONITORING (SHM) SYSTEM IMPLEMENTATION

The SABRE™ (Structural Acoustics for Bridge Reliability Evaluation) with monitoring and interpretation capabilities was developed by TISEC Inc. as a result of over two decades of collaboration and inspection of over 500 bridges with the railroad industry. As a commercially available implementation-ready SHM system, the SABRE™ system comprising the Mistras Group Inc. Sensor Highway™ II System instrumentation and a set of TISEC software post processing modules were selected as the basic AE system. One web gap detail was instrumented with 4 AE sensors and a strain gage. The other detail was instrumented with 3 strain gages to calculate hot spot stress. As per the International Institute of Welding recommendations, strain

gages were mounted at $0.4t$, $0.9t$, and $1.4t$ distance from the weld toe; where t is the web thickness. As per the American Society for Testing and Materials (ASTM) E976-10, pencil lead breaks (PLB) were performed, the system was calibrated, and a region was demarcated for monitoring.

AE Win was the primary software used to set up the data acquisition system and parameters, and acquire and present AE and strain data. *Omega USB* was used to configure the temperature sensor and data logger. *LogMe In* was used for remote access and data transfer. Also, *Remote Desktop Connection* was used to access the remote computer in the SHM system. In addition, the *MS View* software program that was provided with the SunSaver MPPT charge controller was used to program the charge controller and record solar power system parameters.

DATA ANALYSIS AND SYSTEM PERFORMANCE

The sub-tasks performed include (a) analysis of a set of acoustic emission (AE) data recorded by the monitoring system using ICEPAKTM (Intelligent Classifier Engineering Package), (b) calculation of effective stress at a web gap using a refined finite element model loaded with a fatigue truck, (c) calculation of effective stress at a web gap using strain data from field monitoring, and (d) documenting monitoring system performance. A summary of key findings of AE data analysis is given below:

- ICEPAKTM can perform cluster analysis and non-linear mapping (NLM) of data in the time, power, phase, cepstral and auto-correlation domains independently. The AE data collected from the bridge was examined directly to identify any significant similar AE activity formations. NLM with the spectral power domain produced three significant concentrations. Clustering was performed using the same spectral power domain features, and the clusters were well aligned with the visual presentation of the NLM result. NLM and cluster analysis demonstrated the usefulness of such techniques for understanding the AE data.
- PLB data and the AE data in three clusters were evaluated. Waveform characteristics were evaluated using PRISMTM software developed by TISEC Inc. The dominant frequency ranges of each cluster were calculated. The results were used to identify the relationship of each cluster to the characteristics of crack opening signals, background noise, and structural resonance.

- When waveform characteristics in any of the signal clusters resemble the crack opening signal characteristics, location plots can be reviewed to verify the presence of sources within the zone of interest.

The MBE procedure was used to assess fatigue performance. The findings are given below:

- Weigh-in-motion data yielded the gross fatigue truck weight of 57 kips.
- The maximum stress range calculated using refined FE models loaded with a fatigue truck can be as high as 10.88 ksi. When combined with the effects of other loads, the stress range well exceeded the constant-amplitude fatigue threshold (CAFT) indicating a finite fatigue life for the detail.
- The detail was instrumented with strain gages, and hot spot stress was calculated. The maximum stress range of 1.6 ksi was calculated using the measured strain under ambient traffic. When combined with the effects of other loads, the maximum stress range barely reaches the CAFT indicating an infinite fatigue life.

The monitoring system's ruggedness and reliability was evaluated by installing it on a bridge with fatigue-sensitive details and subjecting it to an outdoor environment. A summary of key findings is given below:

- One of the AE board channels malfunctioned and needed to be repaired. This highlights the need of using rugged electronics to develop reliable monitoring systems for outdoor applications. System reliability and ruggedness are very important for monitoring critical bridge details or hard to reach details.
- When dealing with software from different vendors, compatibility issues are common. However, within the data acquisition, interpretation, and presentation stream, these issues were resolved for this application by using the integrated SABRE™ system.
- A standalone solar power system is ideal for short-term or as needed monitoring. However, the system needs to be designed by considering the monitoring system power requirement, solar irradiance at the site, type and quality of the solar array and the charge controller, along with the required power storage (battery bank capacity).
- In general, the performance of the monitoring system and associated software can be considered as satisfactory.

IMPLEMENTATION RECOMMENDATIONS

A two-tier implementation process is recommended. Tier I includes assessment of bridges with repaired details. Tier II includes grouping bridges with fatigue-sensitive details based on a defined set of attributes, and selecting a representative bridge from each group for further analysis and assessment. Additional recommendations to enhance the two-tier implementation process as well as for future research are listed below:

1. Implement AE data interpretation capability demonstrated in this study with an on-line system to provide reliable input with minimal interpretation requirements for inspection-based maintenance management.
2. Develop a fatigue cracking signal characteristic database using typical steel and welds used in Michigan bridges. This database would allow further refining of the AE data interpretation and more accurately detecting the critical events.
3. Install the AE monitoring system with the on-line signal classification system at a half dozen sites to gather data from typical fatigue-sensitive details to identify optimal settings and to optimize other deployment issues.

Intentionally left blank

TABLE OF CONTENTS

ACKNOWLEDGEMENTS	ix
EXECUTIVE SUMMARY	xi
TABLE OF CONTENTS	xix
LIST OF TABLES	xxiii
LIST OF FIGURES	xxv
1 Introduction.....	1
1.1 Overview.....	1
1.2 Objectives and Tasks	2
1.3 Report Organization.....	3
2 State-of-the-Art and Practice Review	5
2.1 Overview.....	5
2.2 Fatigue-sensitive Detail Categories	5
2.3 Evaluation of Fatigue-sensitive Details	6
2.3.1 Linear Palmgren-Miner Damage Rule.....	8
2.3.2 AASHTO MBE Procedure	10
2.3.3 NCHRP 271 Procedure	11
2.3.4 Hot Spot Stress Method	14
2.3.5 Fracture Mechanics Based Assessment	21
2.4 Fatigue-sensitive Detail Monitoring	24
2.5 Technology for Fatigue-sensitive Detail Monitoring	25
2.5.1 Overview and Technology Selection.....	25
2.6 Technology Implementation Considerations	34
2.6.1 Level of Monitoring.....	35
2.6.2 Monitoring Duration	35
2.6.3 Inspection Frequency	36
2.7 Repair and Retrofit Methods.....	37
2.7.1 Surface Treatments	41
2.7.2 Repair of Through-Thickness Cracks	46
2.7.3 Connection(s) or Global Structure Modification	50

2.7.4	Repair Methods to Eliminate Out-of-Plane Distortion.....	51
2.8	Summary	57
3	Structural Modeling and Hot Spot Stress Analysis.....	59
3.1	Overview.....	59
3.2	Bridge Location and Details	59
3.3	Site Selection	62
3.4	As-Built Detail Documentation	62
3.4.1	Comparison of As-Built and As-Designed Details.....	63
3.5	Element Types and Mesh Parameters for Hot Spot Stress Calculation.....	66
3.6	Bridge Structural Modeling and Hot Spot Stress Analysis.....	73
3.6.1	Material Properties.....	73
3.6.2	Loads.....	73
3.6.3	Boundary Conditions	75
3.6.4	Finite Element Representation of the Bridge.....	75
3.6.5	Analysis Results.....	80
3.7	Summary	87
4	Structural Health Monitoring (SHM) System Implementation.....	89
4.1	Implementation and Calibration	89
4.1.1	Monitoring System Components	89
4.1.2	Instrumentation	91
4.1.3	Power Management for Field Monitoring	93
4.1.4	Data Acquisition and AE Sensor Calibration	97
4.1.5	Software for System Configuration, Data Acquisition, Data Display, Remote Access, and Data Transfer	99
4.2	Summary	103
5	Data Analysis and System Performance.....	105
5.1	Acoustic Emission Data Analysis.....	105
5.1.1	Data Acquisition	105
5.1.2	Data Preprocessing.....	105
5.1.3	Pencil Lead Break (PLB) Signals	106
5.1.4	Unsupervised Learning via Clustering.....	107

5.1.5	AE Data Analysis Observations.....	117
5.2	Fatigue Performance Evaluation.....	119
5.2.1	WIM Data and Hot Spot Stresses	119
5.2.2	Hot Spot Stresses from Field Measured Strain Data	120
5.3	Reliability Performance of the Monitoring System.....	123
5.3.1	AE and Strain Data Acquisition.....	123
5.3.2	Remote Access.....	124
5.3.3	Data Replay.....	124
5.3.4	System Reboot	125
5.3.5	MS View	125
5.3.6	Solar Array.....	126
5.4	Summary	132
6	Summary, Conclusions, and Implementation Recommendations	133
6.1	Summary and Conclusions	133
6.2	Implementation Recommendations	136
7	References.....	139

APPENDIX A: Abbreviations

APPENDIX B: Notations

APPENDIX C: Technology for fatigue-sensitive detail monitoring

Intentionally left blank

LIST OF TABLES

Table 2-1. Fatigue Rating and Assessment Outcomes (Bowman et al. 2012).....	12
Table 2-2. Extrapolation Methods for HSS Calculation using Solid Element FE Models (Hobbacher 2008).....	17
Table 2-3. FE Pre/Post-Processing and HSS Calculation Guidelines.....	18
Table 2-4. FE Discretization Guidelines (Hobbacher 2008)	19
Table 2-5. Sensor Types and Implementation Objectives	26
Table 2-6. Crack Initiation Detection Technology	27
Table 2-7. Crack Location Detection Technology.....	27
Table 2-8. Crack Growth Monitoring Technology	28
Table 2-9. Technology for Fatigue Crack Characterization	29
Table 2-10. Strain Measurement Technology.....	29
Table 2-11. Technology for Measuring Residual Stress and Temperature	30
Table 2-12. AE Implementation Challenges and Mitigation Strategies	32
Table 2-13. Crack Stop Hole Diameter Recommendation Based on Plate Thickness (Gregory et al. 1989).....	48
Table 3-1. As-Designed and As-Built Girder Spacing	63
Table 3-2. Web Gap Dimensions.....	66
Table 3-3. HSS Calculated Using FE Models of Type 1 and 2 Specimens.....	72
Table 3-4. HSS Calculated Using Bhargava (2010) Recommendations - Type 1 and 2 Specimens under Tension.....	73
Table 3-5. Bridge Material Properties	73
Table 3-6. HSS at L_1 and L_3 Web Gap Weld Toe Due to Truck #17	81
Table 3-7. HSS at L_1 and L_3 Web Gap Weld Toe Due to Deck Weight	84
Table 3-8. HSS at L_1 and L_3 Web Gap Weld Toe Due to Deck Weight and Truck #17	87
Table 5-1. HSS at L_1 Weld Toe due to Fatigue Truck Load.....	119

Intentionally left blank

LIST OF FIGURES

Figure 2-1. Stress range (S) versus number of cycles (N) (AASHTO LRFD 2013).....	6
Figure 2-2. Nominal, hot spot, and notch stress definitions	7
Figure 2-3. Fatigue assessment procedure for steel bridges (Kühn et al. 2008).....	10
Figure 2-4. AASHTO Manual for Bridge Evaluation – Fatigue life calculation procedure..	11
Figure 2-5. NCHRP 721 – Fatigue life and serviceability index calculation procedure	13
Figure 2-6. (a) Type A and B HS locations and (b) cross-section of specimen showing stress distribution through plate thickness and along the surface close to the weld (Hobbacher 2008).....	16
Figure 2-7. S-N curve definitions for fatigue assessment using hot spot stresses	21
Figure 2-8. Fracture mechanics based fatigue life calculation for a riveted detail (Kühn et al. 2008).....	22
Figure 2-9. Details of an example fatigue inspection cycle (Lovejoy 2003).....	23
Figure 2-10. Fatigue inspection periods for a riveted section crack model at various stress ranges and traffic volumes (Lovejoy 2003).....	23
Figure 2-11. Repair and retrofit methods.....	39
Figure 2-12. Repair of out-of-plane distortion	40
Figure 2-13. Position of the grinding disk with respect to the material surface (Gregory et al. 1989).....	42
Figure 2-14. Typical disc grinders (Dexter and Ocel 2013)	42
Figure 2-15. Position of the burr tip relative to material surface (Gregory et al. 1989).....	43
Figure 2-16. Burr grinder and carbide burr tips (Dexter and Ocel 2013).....	43
Figure 2-17. Hammer positioning and the depth of penetration (Hausammann et al. 1983)	44
Figure 2-18. (a) Pneumatic hammer/chisel and (b) close-up view of the peening tip (Hausammann et al. 1983).....	45
Figure 2-19. UIT equipment (GÜNTHER et al. 2005).....	46
Figure 2-20. UIT (a) deformed shape of a treated surface and (b) application at a weld (Applied Ultrasonics 2014)	46
Figure 2-21. Crack arrest holes drilled at crack tips (Roddis et al. 2001)	47
Figure 2-22. Drilled holes covered by bolts and nuts (Courtesy: Michigan DOT)	48
Figure 2-23. Vee-and-weld repair process.....	49

Figure 2-24. Bolted doubler plate retrofit (the dashed line and the circle represent a crack and a drilled hole to remove the crack tip) (Dexter and Ocel 2013)	51
Figure 2-25. A typical hybrid connection detail (Dexter and Ocel 2013)	55
Figure 2-26. Proposed retrofit scheme with adhesives (Hu et al. 2006).....	56
Figure 2-27. A large hole retrofit example (Dexter and Ocel 2013)	57
Figure 3-1. (a) Bridge location and (b) aerial view of the EB and WB bridges (Google).....	60
Figure 3-2. Girder and diaphragm layout of the EB bridge span 3.....	61
Figure 3-3. Typical girder elevation showing the partial depth diaphragms	61
Figure 3-4. Typical diaphragm detail.....	61
Figure 3-5. (a) Aerial view and (b) span 3 of the I-94 EB bridge.....	62
Figure 3-6. Girders and intermediate diaphragms	63
Figure 3-7. Diaphragm spacing calculated from the model.....	64
Figure 3-8. Intermediate diaphragm detail and elevations at girder F and G	65
Figure 3-9. Specimen 1 and 2 geometry	68
Figure 3-10. Specimen loading and boundary conditions	69
Figure 3-11. The maximum principal stress contours under tension and flexure.....	70
Figure 3-12. Typical truck traffic on the bridge	74
Figure 3-13. Girder layout and outer lane position.....	74
Figure 3-14. Michigan legal load: truck no. 17 configuration with normal axle loads	74
Figure 3-15. Girder end boundary conditions.....	75
Figure 3-16. Girder and diaphragm layout, and the three web gap locations selected for detailed stress analysis.....	77
Figure 3-17. A typical web gap fatigue-sensitive detail	78
Figure 3-18. 3D view of the bridge model.....	78
Figure 3-19. Detailed view of web gaps	79
Figure 3-20. Web gap at L_1 and the FE representation using a submodel.....	79
Figure 3-21. Web gap at L_1 and stresses around weld toe due to truck #17	81
Figure 3-22. Hot spot stress (S_{YY}) at L_1 using quadratic extrapolation	81
Figure 3-23. Web gap at L_3 and stresses around weld toe due to truck #17	82
Figure 3-24. Hot spot stress (S_{YY}) at L_3 using quadratic extrapolation	82
Figure 3-25. Girder and diaphragm deformation under truck #17.....	83

Figure 3-26. Deformation of girder E and the web gap (L_1) due to truck # 17	83
Figure 3-27. Deformation of girder D and the web gap (L_3) due to truck # 17	84
Figure 3-28. Hot spot stress (S_{YY}) at L_1 using quadratic extrapolation (deck dead load)	85
Figure 3-29. Hot spot stress (S_{YY}) at L_3 using quadratic extrapolation (deck dead load)	85
Figure 3-30. Deformation of girder E and the web gap (L_1) due to deck weight	86
Figure 3-31. Deformation of girder D and the web gap (L_3) due to deck weight.....	86
Figure 4-1. Monitoring system components in the enclosure.....	90
Figure 4-2. (a) An acoustic emission sensor, (b) a spring –loaded magnetic holder, and (c) AE sensors mounted on a steel girder	91
Figure 4-3. AE sensors and a strain gauge at the partial depth diaphragm connection, L_3 ...	92
Figure 4-4. AE sensors around the web gap and a strain gage at girder bottom flange	92
Figure 4-5. Strain gages at the webgap.....	93
Figure 4-6. Basic components of a solar powered monitoring system.....	94
Figure 4-7. Basic components of a solar power system.....	95
Figure 4-8. The source location page in <i>AE Win</i> showing several AE source locations	98
Figure 4-9. AE source locations generated through pencil lead breaks.....	99
Figure 4-10. Activity screens.....	100
Figure 4-11. Parametric data display (microstrain vs. time in seconds).....	100
Figure 4-12. Opening an existing sensor layout	101
Figure 4-13. Accessing an existing data file for replaying	101
Figure 4-14. <i>LogMe In</i> interface	102
Figure 4-15. Accessing the SHM system using a mobile device.....	102
Figure 5-1. Typical signals present in a data file.....	106
Figure 5-2. NLM Time Domain.....	107
Figure 5-3. NLM Power Domain.....	107
Figure 5-4. NLM Phase Domain.....	108
Figure 5-5. NLM Cepstral Domain.....	108
Figure 5-6. NLM Auto-Correlation Domain	108
Figure 5-7. Clustering Power Domain.....	108
Figure 5-8. NLM Power Domain.....	108
Figure 5-9. Clustering Power Domain.....	108

Figure 5-10. Individual data clusters	109
Figure 5-11. Linear Discriminant	110
Figure 5-12. K-Nearest Neighbors.....	110
Figure 5-13. Empirical Bayesian	110
Figure 5-14. Minimum Distance.....	110
Figure 5-15. Neural Network.....	110
Figure 5-16. A sample PLB waveform in class 1	111
Figure 5-17. A sample PLB waveform in class 2	111
Figure 5-18. A sample rejected PLB waveform	111
Figure 5-19. A sample class 1 waveform and its power spectrum	113
Figure 5-20. A sample class 2 waveform and its power spectrum	114
Figure 5-21. A sample class 3 waveform and its power spectrum	115
Figure 5-22. A sample “rejected” waveform and its power spectrum.....	116
Figure 5-23. AE source location plot.....	118
Figure 5-24. Waveform observation	118
Figure 5-25. A sample strain profile collected at 0.4t.....	121
Figure 5-26. A sample strain profile collected at 0.9t.....	122
Figure 5-27. A sample strain profile collected at 1.4t.....	122
Figure 5-28. Menu options available in <i>LogMe In Pro</i>	124
Figure 5-29. A warning message that appears when data replay program is opened.....	125
Figure 5-30. Program closure warning	125
Figure 5-31. Battery charging and discharging evaluation under laboratory conditions.....	127
Figure 5-32. Theoretical battery discharge and recharge cycle	127
Figure 5-33. Variation of array voltage, charge voltage, array current, and charge current during battery charging period	130
Figure 5-34. Variation of battery voltage and array voltage during a typical day.....	131
Figure 5-35. Variation of battery voltage and array voltage during continuous operation of a load from June 30 (12:21:38 hr) to July 5 (06:09:58 hr), 2014.....	131
Figure 6-1. Two-tier implementation process.....	137

1 INTRODUCTION

1.1 OVERVIEW

Fatigue is one of the most critical problems for steel bridges as well as for any steel structures that needs to be considered during design and operation. Fatigue cracking is developed at certain steel bridge details due to a direct result of the loads (load-induced fatigue) or a deformation that is not accounted for during design (distortion-induced fatigue). The details that are prone to load-induced fatigue can be identified using the detail categories presented in the AASHTO LRFD (2013) specifications Table 6.6.1.2.3-1. These details are grouped into 8 different categories such as A, B, B', C, C', D, E, and E'. Of these categories, Category A has the greatest fatigue resistance while Category E' has the least. According to the AASHTO Manual for Bridge Evaluation (MBE 2011) a majority of the fatigue problems are associated with weld terminations and weld defects. In addition to weld locations, fatigue cracks can initiate at a material flaw, changes in member cross-sections, or some combinations thereof. The MBE (2011) section 7 presents a procedure to assess the cumulative fatigue damage of uncracked members subjected to load-induced stresses. Recently, Bowman et al. (2012) proposed revisions to MBE (2011) section 7.

The secondary and/or distortion-induced stresses that are not typically used in design are the most common reasons for fatigue cracking developed in bridges (Fisher 1984). The existence and the need for evaluating distortion-induced fatigue are acknowledged in the MBE (2011) as well as in the proposed revisions by Bowman et al. (2012). However, the scope of the MBE (2011) section 7 as well as the proposed revisions is limited to the load-induced fatigue evaluation. Therefore, identifying the details that are prone to distortion-induced fatigue cracking requires a combined effort of utilizing information in bridge files, refined analysis, and experience. Developing high fidelity analysis models and effective monitoring systems is vital to evaluate the causes of cracking or potential for cracking as well as to estimate remaining fatigue life, especially when evaluating weldments, complicated details, or the effect of secondary stresses. According to the MBE (2011), a majority of the fatigue problems are associated with weld terminations and weld defects. The hot spot stress (HSS) method is widely used in other disciplines such as ship building and offshore structures for the assessment of weldments (Fricke and Kahl 2005; Chakaray et al. 2008; Fricke et al. 1998; Nguyen et al. 2012).

However, the HSS method applications in bridge engineering are rarely documented (Alemdar et al. 2014b, Hassel et al. 2010; Akhlaghi et al. 2009).

MDOT performs inspections of over 200 bridges with fatigue-sensitive details. Once the fatigue-sensitive details are identified, a structural health monitoring (SHM) system can be implemented to achieve the following objectives:

- Alert bridge managers when a crack initiates so that the crack growth is monitored to schedule retrofits.
- Identify details with active cracks so that the growth is monitored to schedule retrofits.
- Evaluate effectiveness of repairs or retrofits.
- Evaluate the response of a detail under permit loads.
- Evaluate the response of a detail due to change in service conditions.
- Monitor stress flow to calculate remaining fatigue life (or the fatigue serviceability index) to support bridge management decisions.

The development of a reliable SHM system requires implementing technologies with proven records of field performance. Further, the system needs to be robust and portable enough to be used at multiple sites as needed.

1.2 OBJECTIVES AND TASKS

As presented in the overview, fatigue is one of the most critical problems for steel bridges as well as for any steel structures that needs to be considered during design and operation. The objectives of this study are to explore fatigue monitoring technologies, and to develop effective structural and data analysis strategies as well as implementation recommendations for bridge engineering. The scope of this study is to:

1. Prepare a synthesized report from a literature search and provide recommendations for the appropriate technology to use for remote monitoring steel bridges having fatigue-sensitive details.
2. Select an appropriate SHM system using commercially available equipment to the extent possible, and perform an installation on a selected bridge with fatigue-sensitive details.
3. Conduct field monitoring to validate the system's accuracy and robustness.
4. Provide recommendations for implementation of the SHM system and further research.

To achieve the project objectives with the given scope, this project is organized into five tasks: (1) review the state-of-the-art and practice literature to identify technologies for a SHM system and to develop implementation recommendations, (2) select a bridge and perform structural analysis to identify details for monitoring, (3) procure an SHM system, (4) install and calibrate the system, and analyze data, and (5) develop recommendations for technology integration into MDOT practice and further research.

1.3 REPORT ORGANIZATION

The report is organized into 7 chapters.

Chapter 1, (this chapter) includes the introduction and overview of the study.

Chapter 2, literature review, presents the fatigue-sensitive detail categories, methodologies to establish the needs for monitoring fatigue-sensitive details, technology for fatigue-sensitive detail monitoring, and technology implementation considerations. In addition, retrofit methods for fatigue-sensitive details are also presented.

Chapter 3 provides a description of the bridge with partial depth diaphragms and welded details. Evaluation of the welded details requires calculating hot spot stresses. Hot spot stress calculation requires selecting appropriate element types, mesh parameters, and reference points for stress calculation. Therefore, this chapter also presents (a) element types and mesh parameters for hot spot stress calculation, and (b) bridge structural analysis results and recommended details for instrumentation and monitoring.

Chapter 4 provides information related to field implementation, SHM system calibration, and power supply. This chapter also presents the system components and software used for system configuration, along with data acquisition, data display, remote access, and data transfer.

Chapter 5 presents (a) a set of acoustic emission (AE) data recorded by the monitoring system and data analysis using ICEPAKTM, pattern classifiers for use in real-time structural health and reliability monitoring, (b) calculation of effective stress at a web gap detail using WIM data and refined finite element analysis, (c) calculation of effective stress at a web gap detail using strain data from field monitoring, and (d) findings from the monitoring system performance evaluation.

Chapter 6 provides a summary, along with the conclusions, and recommendations.

Chapter 7 includes the list of cited references.

Intentionally left blank

2 STATE-OF-THE-ART AND PRACTICE REVIEW

2.1 OVERVIEW

To develop a successful program for monitoring fatigue-sensitive detail, it is required to (i) identify fatigue-sensitive details, (ii) establish monitoring objectives, (iii) select technology suitable for achieving the monitoring objectives, and (iv) decide on the level and duration of monitoring.

The following sections of this chapter present fatigue-sensitive detail categories, methodologies to establish the needs for monitoring the fatigue-sensitive details, technology for monitoring the fatigue-sensitive detail, and technology implementation considerations. In addition, retrofit methods for the fatigue-sensitive details are also presented.

2.2 FATIGUE-SENSITIVE DETAIL CATEGORIES

The AASHTO LRFD Bridge Design Specifications (2013), Table 6.6.1.2.3-1, present detail categories for load-induced fatigue. These details are grouped into 8 different categories such as A, B, B', C, C', D, E, and E'. Of these categories, Category A has the greatest fatigue resistance while Category E' has the least. Figure 2-1 is a graphical representation of the nominal fatigue resistance for these details. The horizontal broken lines represent the constant amplitude fatigue threshold (CAFT). According to Section 7 of the Manual for Bridge Evaluation (MBE 2011), fatigue life of the load-induced fatigue-sensitive details is evaluated when two times the effective stress is greater than the CAFT.

The secondary and/or distortion-induced stresses that are not typically used in design are the most common reasons for fatigue cracking developed in bridges (Fisher 1984). The existence and the need of evaluating distortion-induced fatigue are acknowledged in the MBE (2011); yet, the scope of the Section 7 of the MBE is limited to the load-induced fatigue evaluation. Therefore, the details prone to distortion-induced fatigue cracking need to be identified with prior experience with similar details, by reviewing inspection and maintenance records, and through refined analysis. Also, the assessment procedures of such details need to be identified and documented from literature.

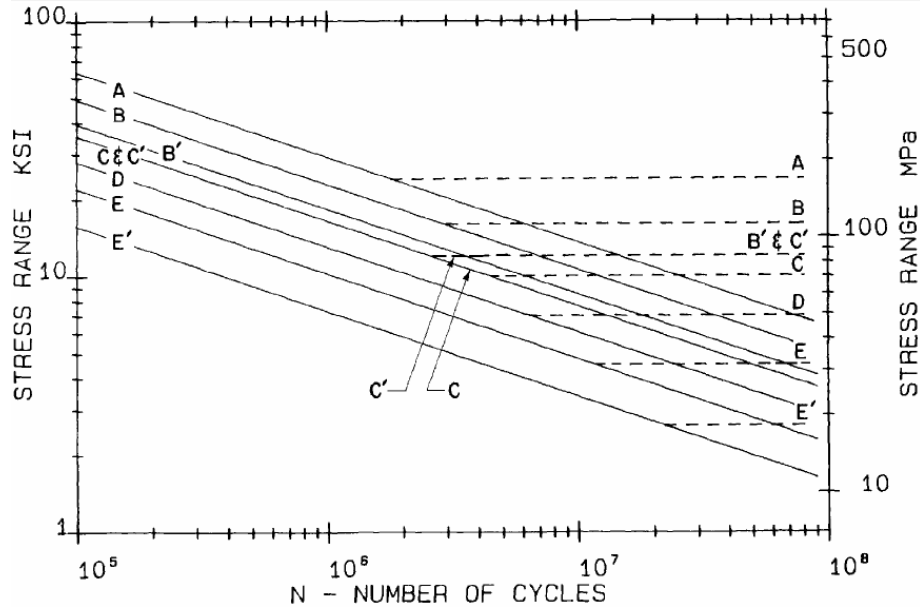


Figure 2-1. Stress range (S) versus number of cycles (N) (AASHTO LRFD 2013)

2.3 EVALUATION OF FATIGUE-SENSITIVE DETAILS

Kühn et al. (2008) provide a technical background for further development of the Eurocode for assessing existing steel structures and estimating remaining fatigue life through a four-phase process to estimate the remaining fatigue life of existing steel structures. These phases include (i) a preliminary evaluation by an engineer in accordance with the current codes, (ii) detailed investigation by an engineer with the help of an expert and supported with data collected using low-tech nondestructive testing (NDT) methods, (iii) expert investigations that may be supported with data collected using high tech NDT, and (iv) evaluation of fatigue performance after developing remedial measures, such as enforcing load restrictions, or performing repairs or retrofits. This monitoring is used to support detailed investigation by experts. Another aspect of monitoring is to evaluate the effectiveness of remedial measures.

The key step in the evaluation of fatigue-sensitive details is the calculation of remaining fatigue life. Nominal stress, hot spot stress, effective notch stress, fracture mechanics, and component testing are the methods used for assessing fatigue life (Hobbacher 2008). The most common fatigue assessment method uses S-N curves (Figure 2-1). This approach is based on the nominal stress. In addition, hot spot and notch stresses are also used for fatigue assessment of welded details. The fracture mechanics based approach can be used irrespective of the detail configuration. Hobbacher (2008) developed a report, titled *Recommendations for Fatigue*

Design of Welded Joints and Components, for the International Institute of Welding (IIW). The guidelines presented in this report are commonly cited in literature as IIW guidelines. Hobbacher (2008) defines the nominal stress, hot spot stress, and notch stress as follows:

- Nominal stress – *stress calculated in the sectional area under consideration.*
- Hot spot stress – *also known as the structural or geometric stress at the hot spot includes all the stress raising effects of a structural detail excluding that due to the local weld profile itself.*
- Notch stress – *total stress at the root of a notch taking into account the stress concentration caused by the local notch, consisting of the sum of structural stress and nonlinear stress peak.*

A graphical representation of the above definitions are shown in Figure 2-2.

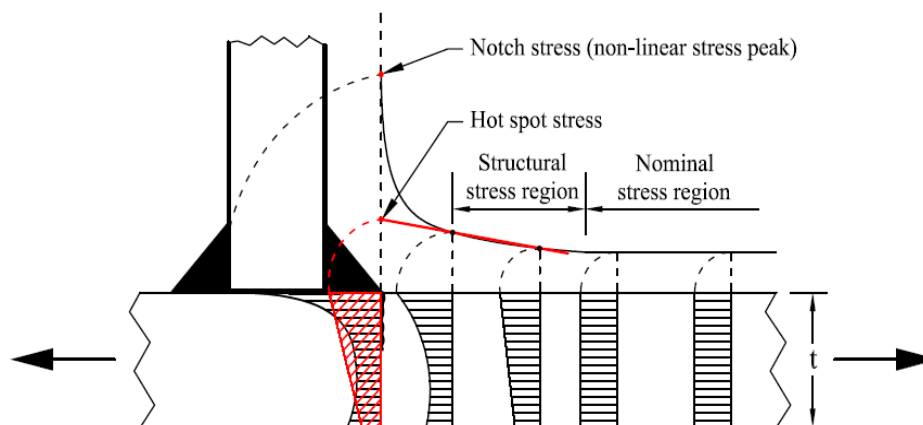


Figure 2-2. Nominal, hot spot, and notch stress definitions

Kühn et al. (2008) proposed using the linear Palmgren-Miner damage rule based approach. The fatigue assessment procedure for existing steel bridges proposed by Kühn et al. (2008) is presented in section 2.3.1.

At present, highway agencies in the U.S. follow the provisions of Article 7.2 of the AASHTO MBE (2011) to assess the cumulative fatigue damage of uncracked members subjected to load-induced stresses (See Section 2.3.2 for more details.) The NCHRP Project 12-81, “Evaluation of Fatigue on the Serviceability of Highway Bridges” was initiated to address the shortcomings in the MBE (2011) procedure (Bowman et al. 2012). Instead of calculating a remaining fatigue life, calculation of a nondimensional parameter (the fatigue serviceability index - Q) was presented by Bowman et al. (2012) to evaluate the condition and the assessment outcome with respect to

fatigue. Section 2.3.3 presents calculation process of the fatigue serviceability index (Q), fatigue rating, and guidance for using Q in bridge management decisions.

Hot spot stress is commonly used for fatigue life assessment of welded details. Hot spot stress is calculated by extrapolating structural stress using linear or cubic extrapolation equations. Structural stress for extrapolation equations is calculated using refined finite element models. Section 2.3.4 presents finite element guidelines, extrapolation equations, and the procedure for using hot spot stresses with a reference detail to calculate fatigue life.

The effective notch stress method has not been widely used in assessing bridges. Hence, that method is not discussed in this report. The fracture mechanics based approach is used to calculate the rate or duration of a crack growth under fatigue loading. This method has been used to formulate inspection frequency. Section 2.3.5 presents two case studies where the fracture mechanics based approach was used to assess fatigue-sensitive details or fracture-critical members, and to determine the inspection frequency.

2.3.1 Linear Palmgren-Miner Damage Rule

Kühn et al. (2008) presented a four-phase process to estimate the remaining fatigue life of existing steel structures. As per the phase I investigation, the preliminary assessment is carried out by an engineer to identify the presence of fatigue-sensitive details and critical members. This level of assessment is carried out by studying the documents in the bridge file (such as plans, design calculations, and inspection and maintenance reports) and data collected by performing an inspection of the bridge. In general, highway agencies have performed such investigations and have identified the need for detailed assessment.

Presence of fatigue-sensitive details and fracture-critical members can be identified using an agency database or, in the U.S., by using the National Bridge Inventory (NBI) database. Because of this reason, only phase II to IV presented by Kühn et al. (2008) are discussed here. The relation between three assessment phases and decision-making involved throughout the processes are depicted in Figure 2-3. The variables used in the figure are defined in Appendix B. As shown in Figure 2-3, phase II investigation is carried out by an engineer alone. The linear Palmgren-Miner damage rule (Eq. 2-1) is used to calculate the damage sum (D). If the calculation shows sufficient safety (i.e., $D < 1$), the remaining fatigue life (T_{fat}) is calculated. If

$T_{\text{fat}} > 10$ years, the standard inspection is performed. When $T_{\text{fat}} \leq 10$ years, phase IV activities are performed. Phase IV activities include intensified monitoring of the detail of concern and other decisions such as repair, rehabilitation, imposing load restrictions, or structure demolition. Once repair, rehabilitation, or load restriction decisions are implemented, new residual life is calculated, and the bridge file is updated.

$$D = \sum \frac{n_i}{N_i} \quad (2-1)$$

where, n_i is the actual number of cycles at stress range i , and N_i is the theoretical fatigue life at stress range i .

When $D \geq 1$, the following steps can be used to update the calculations:

- Use site-specific traffic and material data, and an accurate estimation of the variable loads and dead or permanent loads.
- Use detail focused refined and advanced analysis techniques such as submodeling.
- Acquire real-time data such as stresses under service loads. The measured data can help improve understanding of the structure such as unintended composite behavior, unforeseen load distribution, effect of non-structural elements, and boundary condition effects (such as partial fixity, frozen bearings, and unintended continuity at intermediate supports). Hence, strain at the detail of interest can be measured to perform the rainflow counting as per the ASTM E1049-85. Once the rainflow counting is performed and the data is placed in bins to develop a stress histogram, Miner's rule can be applied to calculate an accurate damage sum.

When the refinements do not improve the calculations to yield $D < 1$, the consequences of failure and the cost of repair/retrofits need to be evaluated. If the consequences or the repair/retrofit costs justify, phase III investigations, with the help of a team of experts, are initiated. The phase III investigations include use of advanced monitoring, quantitative advanced NDT, material property evaluation, and fracture mechanics and/or probabilistic assessment methods. Based on the phase III outcome, remedial measures can be developed under phase IV. Monitoring technologies such as AE can be used to evaluate the effectiveness of the remedial measures.

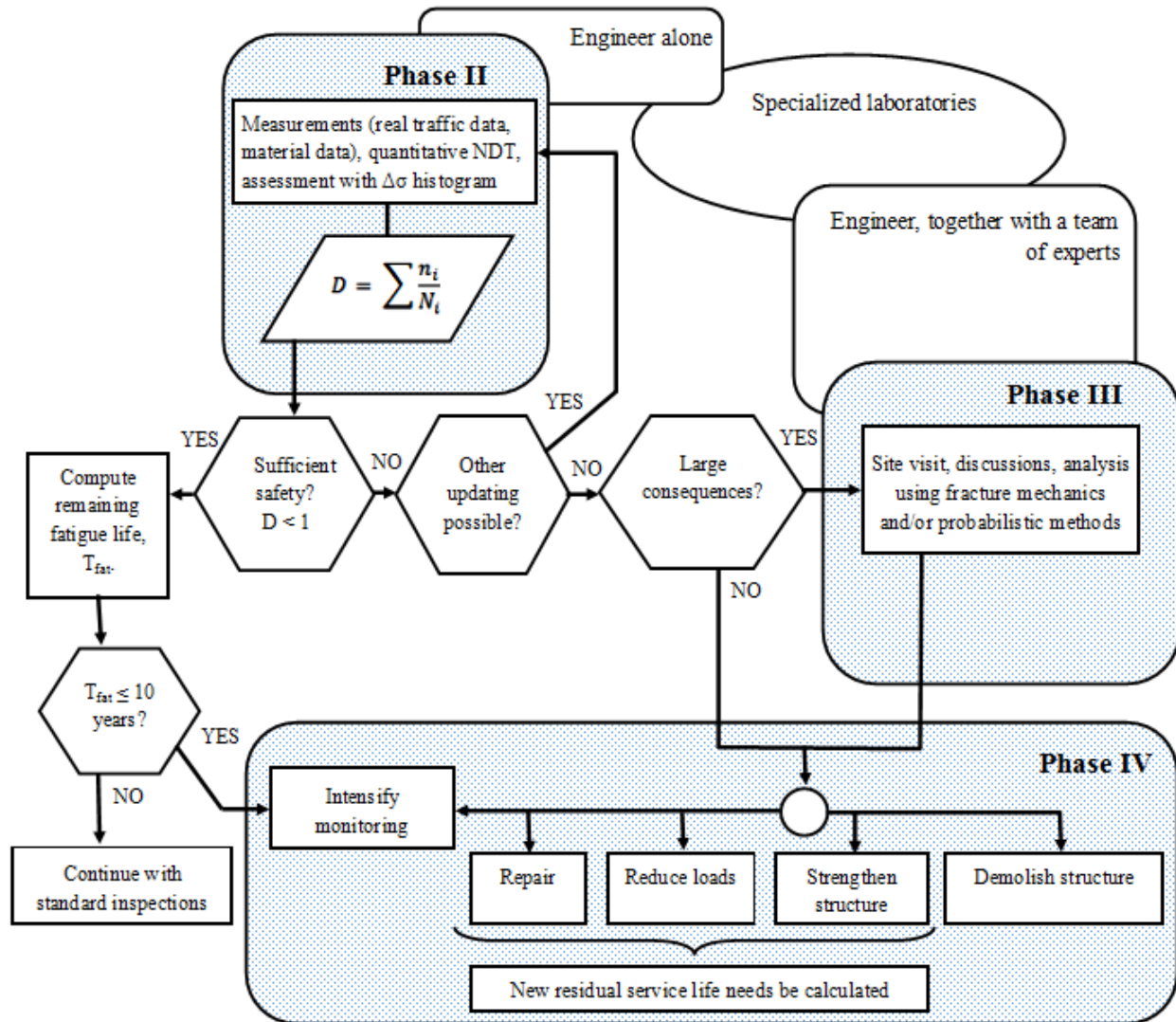


Figure 2-3. Fatigue assessment procedure for steel bridges (Kühn et al. 2008)

2.3.2 AASHTO MBE Procedure

Section 7 of the AASHTO Manual for Bridge Evaluation (MBE 2011) provides a procedure for evaluating load-induced fatigue. As depicted in Figure 2-4, the first step is to identify the fatigue-sensitive details. The next step is to evaluate the need for fatigue assessment. Once the need is identified, required data for an accurate assessment of a detail such as traffic data and strain is acquired. The variables used in the figure are defined in Appendix B. The process depicted in Figure 2-4 is based on measured strains. Hence, strain at the detail of interest is measured, and the rainflow counting is performed as per the ASTM E1049-85. Once the rainflow counting is performed and the data is placed in bins to develop a stress histogram, the effective stress is calculated. As the first step in evaluation, an infinite-life check is performed.

The AASHTO LRFD Bridge Design Specifications (2013) in Table 6.6.1.2.3-1 provide the threshold values, $(\Delta f)_{TH}$. In general, the maximum effective stress, $(\Delta f)_{max}$, is taken as twice the effective stress, $(\Delta f)_{eff}$. If the check fails, finite fatigue life (Y) is calculated. Once the present age of the detail (A) is subtracted from the finite fatigue life, the remaining fatigue life can be calculated. Remedial measures are suggested in the MBE (2011) when an inadequate or negative remaining life is yielded through this process.

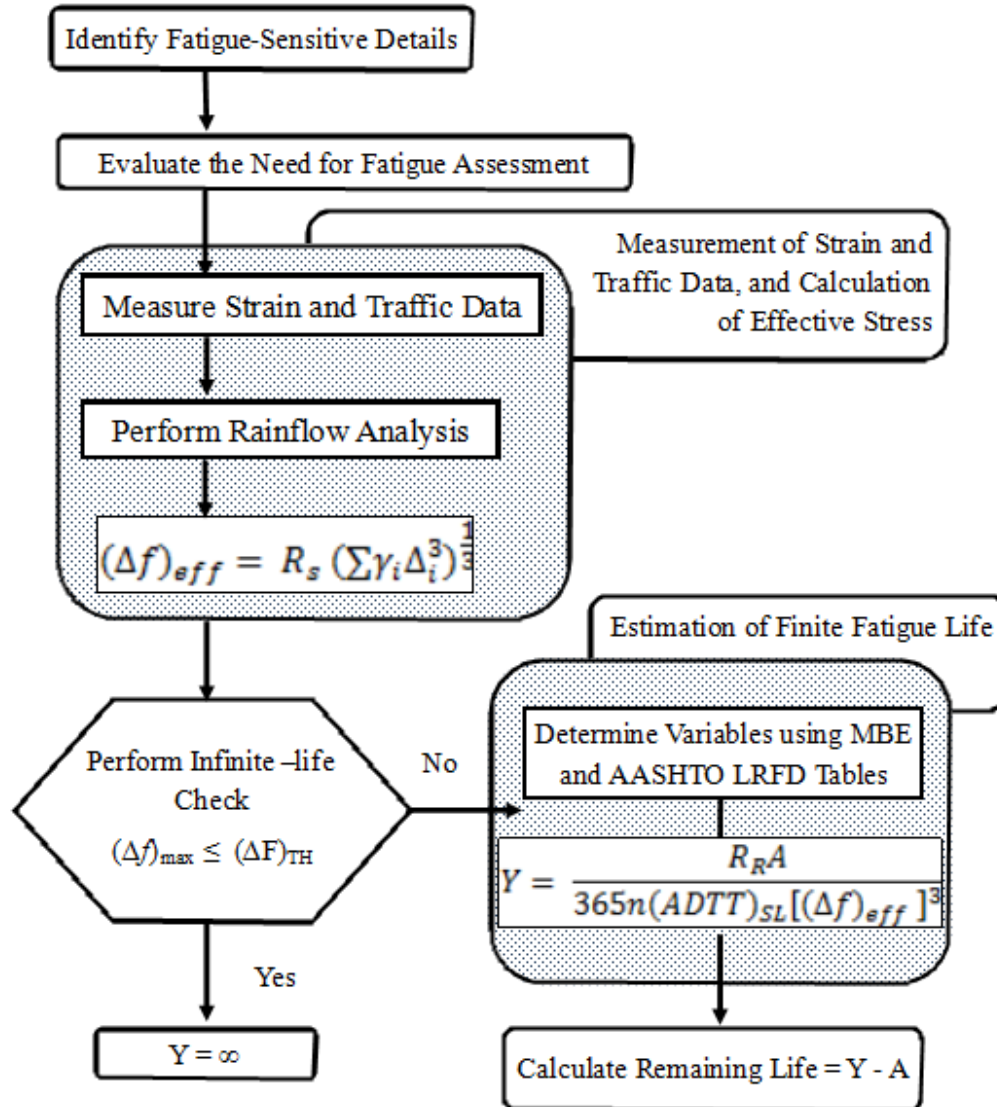


Figure 2-4. AASHTO Manual for Bridge Evaluation – Fatigue life calculation procedure

2.3.3 NCHRP 271 Procedure

As depicted in Figure 2-5, the process up to performing the infinite-life check is the same as the process presented in section 2.3.2. If the infinite-life check fails, finite life is calculated using a

much more elaborate equation than the equation presented in Figure 2-4. The variables used in Figure 2-5 are defined in Appendix B. Using the finite fatigue life and the current age of the detail, fatigue serviceability index (Q) is calculated. Revisions to the calculation of Q are needed when the values are negative. If the revised calculation yields a negative value for Q, the fatigue rating of the detail is determined as “critical” fatigue rating (Table 2-1). This requires considering retrofit, replacement, or reassessing the detail. When Q is equal or greater than zero, the detail is assigned a fatigue rating and corresponding management decisions are taken. When the fatigue rating of a detail is determined as either Fair or Poor, the detail needs to be assessed frequently (Table 2-1).

Table 2-1. Fatigue Rating and Assessment Outcomes (Bowman et al. 2012)

Fatigue Serviceability Index, Q	Fatigue Rating	Assessment Outcome
1.00 to 0.50	Excellent	Continue Regular Inspection
0.50 to 0.35	Good	Continue Regular Inspection
0.35 to 0.20	Moderate	Continue Regular Inspection
0.20 to 0.10	Fair	Increase Inspection Frequency
0.10 to 0.00	Poor	Assess Frequently
< 0.00	Critical	Consider Retrofit, Replacement or Reassessment

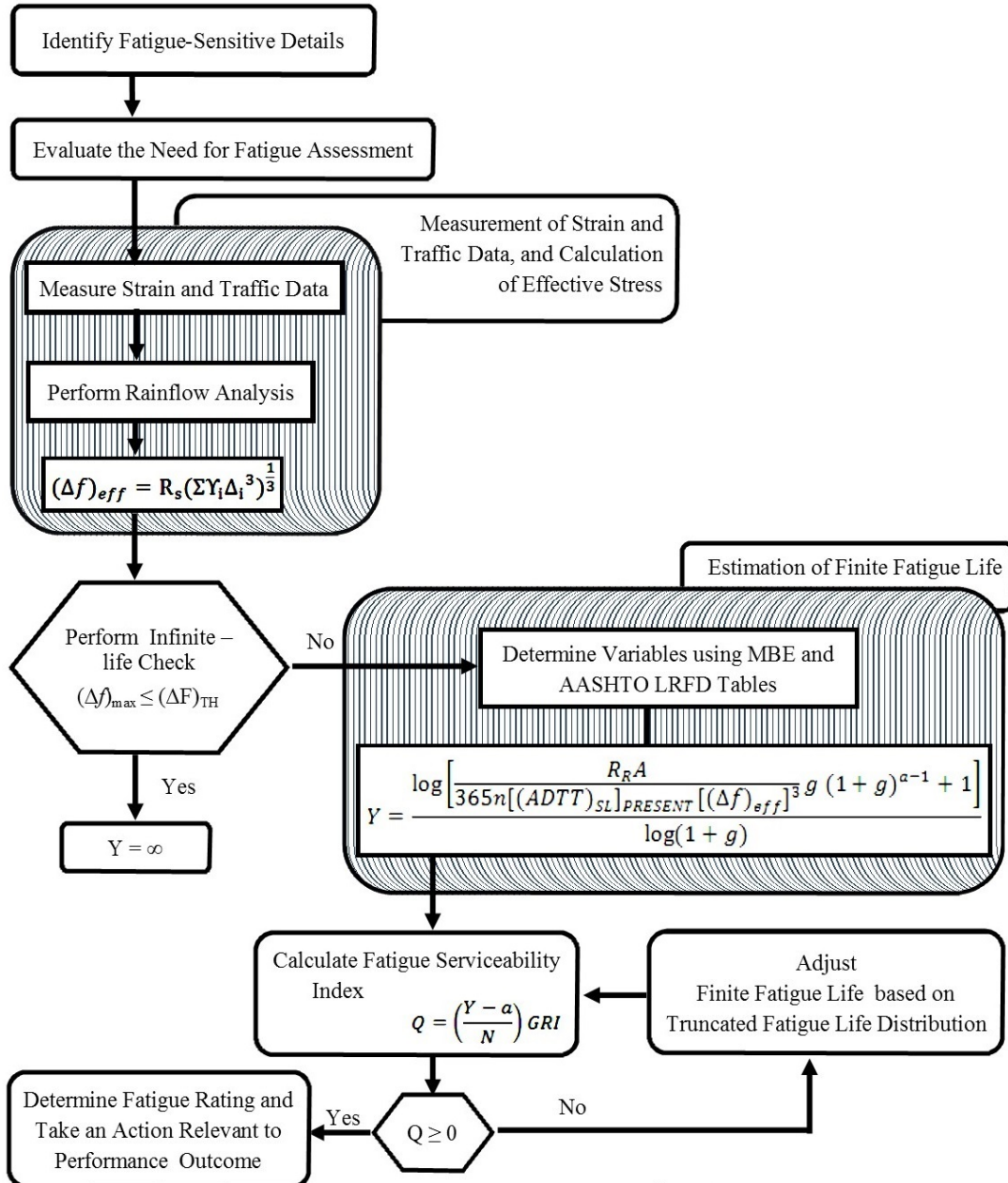


Figure 2-5. NCHRP 721 – Fatigue life and serviceability index calculation procedure

2.3.4 Hot Spot Stress Method

2.3.4.1 Hot Spot Stress Analysis

The hot spot stress (HSS) method is widely used for the assessment of weldments. Causes of cracking or potential for cracking is investigated through analysis with the support of experimental data. Developing high fidelity analysis models is vital to understand the behavior of fatigue-sensitive details in bridges for estimating remaining fatigue life, especially when evaluating complicated details or secondary stresses (Alemdar et al. 2014a; Aygul et al. 2012; Lee et al. 2010; Bhargava 2010; Akhlaghi et al. 2009; Hobbacher 2008). Even though the use of the HSS method for calculating secondary stresses is briefly discussed in design specifications, manuals, and many publications such as the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD 2013) and the AASHTO Manual for Bridge Evaluations (MBE 2011), there are no clear guidelines provided in those documents for selecting model parameters and interpretation of results. Also, the recently proposed updates to the Manual for Bridge Evaluation Section 7 by Bowman et al. (2012) do not include guidelines for hot spot stress analysis.

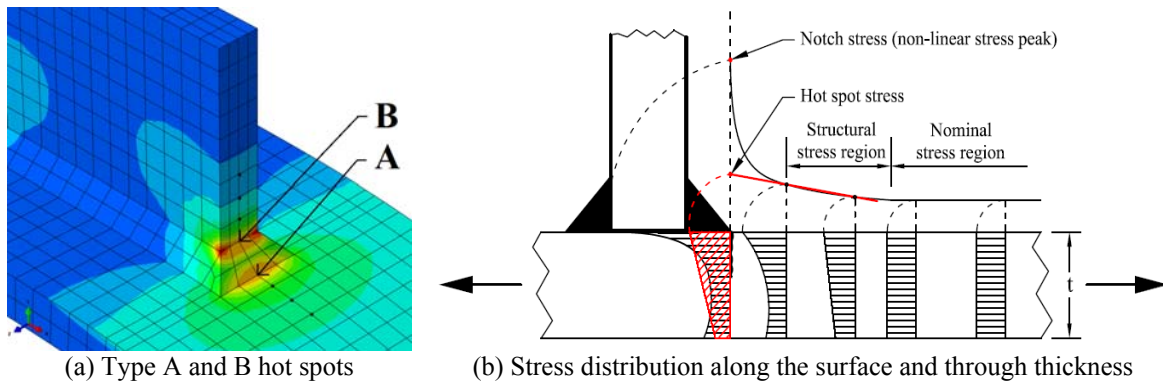
Fatigue cracks initiate at weld toes; thus, it is critical to quantify stresses due to load effects at such a location (Hobbacher 2008). According to Lee et al. (2010) the HSS method is also useful for fatigue resistant design. The HSS method has been extensively used in other disciplines such as ship building and offshore structures (Fricke and Kahl 2005; Chakaray et al. 2008; Fricke et al. 1998; Nguyen et al. 2012). More recently, the HSS method is used in investigating bridge details (Alemdar et al. 2014b, Hassel et al. 2010; Akhlaghi et al. 2009).

Linear elastic theory or numerical methods (e.g., FE) can be used in conjunction with S-N curve classification to estimate the fatigue life (Aygul 2012). State-of-the-art technology allows accurate representation of geometries and complex details using refined FE models. However, calculating HSS using FE is a challenge due to the sensitivity of analysis results to element types and mesh discretization within the region of high strain gradient. FE analysis results depend on (1) element type, (2) mesh discretization parameters such as size, Jacobian, skew, and aspect ratio, (3) boundary conditions, (4) loads and load application procedures, and (5) postprocessor capabilities. Therefore, various guidelines and recommendations for calculating HSS using FE

are presented in literature (Bhargava 2010; Hobbacher 2008; Aygul 2012; Niemi 1993; Niemi et al. 2006). This section of the report presents a comprehensive summary of FE modeling, analysis, and results interpretation guidelines for the HSS method documented in literature.

HSS is a combined effect of membrane and bending stresses, and follows a linear stress distribution in steel welded structures along the surface. Analysis results are expected to be ‘mesh size insensitive’ (Akhlaghi et al. 2009). Hence, an expert panel led by Hobbacher representing the International Institute of Welding Commissions XIII and XV developed recommendations for fatigue design of welded joints and components (Hobbacher 2008); this document is commonly stated in literature as the IIW guidelines. The recommendations are for element types and sizes as well as type of extrapolation and location of extrapolation points. According to Hobbacher (2008), there are two types of hot spots, Type A and Type B, based on their location on the plate and orientation with respect to the weld toe (Figure 2-6a).

When calculating Type A HSS, reference points are defined at specific distances from the weld toe based on the plate thickness. The reference point stresses are calculated from the plate surface. For Type B HSS, the reference points are defined at absolute distances from the weld toe, regardless of plate thickness. Then, the stresses at the weld toe are calculated by extrapolating the reference point stresses. According to Hobbacher (2008), reference points are located within the structural stress region (Figure 2-6b). With the above stated process, nonlinear stress peak at the weld toe due to notch effect is excluded from calculation. Hence, HSS is dependent on global geometry and loading parameters, and quantified using HSS equations presented in Table 2-2. Columns 1 and 2 of Table 2-2 list hot spot (HS) type and mesh type, respectively. Columns 3 and 4 list the element length and the location of reference points, respectively. Column 5 shows the order of the stress extrapolation equation as well as the extrapolation equations.



(a) Type A and B hot spots (b) Stress distribution along the surface and through thickness
Figure 2-6. (a) Type A and B HS locations and (b) cross-section of specimen showing stress distribution through plate thickness and along the surface close to the weld (Hobbacher 2008).

Figure 2-6a shows a finite element model of a welded steel plate assembly using solid elements. The recommended extrapolation path is shown along the plate surface for Type A and along the edge for Type B HSS calculation methods. Probing HSS from the FE model at the weld toe is misleading, because nodal stresses at the weld toe include non-linear notch stress developed due to weld geometry. Notch stress is included in S-N design curves based on experimental results (Lee et al. 2010). Hence, the elements that are located in front of the weld toe and within the structural stress region are of interest. In order to eliminate the influence of notch stress on HSS calculation, Hobbacher (2008) recommended using an extrapolation technique with the stresses calculated at least at $0.4t$ away from weld toe for Type A detail. Hobbacher (2008) presented two extrapolation equations to calculate HSS for Type A and B locations when a weld detail is modeled with solid elements: linear with 2 reference points and quadratic with 3 reference points (Table 2-2). With the recent advances in FE pre- and post-processing capabilities, stress at specified reference points can be probed and used with the equations presented in Table 2-2 to calculate HSS.

Table 2-2. Extrapolation Methods for HSS Calculation using Solid Element FE Models (Hobbacher 2008)

HSS Type	Mesh Type	Element length at HS		Reference points	Extrapolation Method and Equation	
A	Fine	Based on Plate Thickness	$\leq 0.4 t \times t^*$ or $\leq 0.4 t \times w/2$	Located at Plate Surface	0.4 t and 1.0 t Nodal Points	Linear $1.67 \sigma_{0.4t} - 0.67 \sigma_{1.0t}$
					0.4 t, 0.9 t and 1.4 t Nodal Points	Quadratic** $2.52 \sigma_{0.4t} - 2.24 \sigma_{0.9t} + 0.72 \sigma_{1.4t}$
	Coarse		$t \times t$ or Max $t \times w^{***}$ (Use higher-order elements)	0.5 t and 1.5 t Surface Center	Linear $1.50 \sigma_{0.5t} - 0.50 \sigma_{1.5t}$	
B	Fine	Based on Absolute Distance	$\leq 4 \times 4$ mm	Located at Plate Edge Surface	4, 8 and 12 mm Nodal Points	Quadratic* $3 \sigma_{4mm} - 3 \sigma_{8mm} + \sigma_{12mm}$
	Coarse		10×10 mm		5 and 15 mm Surface Center	Linear $1.5 \sigma_{5mm} - 0.5 \sigma_{15mm}$

* t = plate thickness

** Quadratic extrapolation is recommended when non-linear structural stress increase is expected towards the hot spot.

*** w = longitudinal attachment thickness + 2* weld leg length

Note: 1 in. = 25.4 mm

Different FE modeling guidelines and recommendations for welded details are presented in literature. Table 2-3 summarizes FE pre/post processing and HSS calculation guidelines from literature and available specifications since 1992 to 2012. Typical guidelines include model type, mesh, element size, element shape, HSS calculation method, distance to reference points from weld toe, type of stress used for calculating HSS, and inclusions/exclusions of the model. The majority of sources listed in Table 2-3 has followed the guidelines and recommendation presented by Hobbacher (2008).

When preprocessing a FE model, in order to have nodes at each reference point, it is prudent to define element length near a weld toe based on the distance to the reference points measured from the weld toe. As an example, if the quadratic extrapolation recommended by Hobbacher (2008) is used to calculate HSS at Type A detail, sizes of the first three elements located in front of the weld toe need to be defined as shown in Table 2-4c and e. The other option is to use higher order elements and the stress values calculated at the center of the element top surface with the extrapolation equations to calculate HSS (Table 2-4a and d).

Table 2-3. FE Pre/Post-Processing and HSS Calculation Guidelines

Year	Reference	FE Pre/Post-Processing and HSS Calculation Guidelines
1992	ABS (1992)	<ul style="list-style-type: none"> • Model with solid elements: 20-node solid elements • Model with shell elements: 8-node thick shell elements • Element size: $t \times t$ • HSS calculation method: Linear extrapolation • Distance to reference points from weld toe: 0.5t and 1.5t • Stress used for HSS calculation: Maximum Principal Stress
1998	Fricke et al. (1998)	<ul style="list-style-type: none"> • Model with solid elements: 20-node solid elements • Element size: $t \times t$ • Mesh guidelines: Use at least three elements of equal length within the area of high stress gradient • HSS calculation method: Quadratic extrapolation from the 3 equal length elements. • Recommendation: Include weld in model
1998	Eurocode 9 (1998)	<ul style="list-style-type: none"> • Model with shell or solid elements • Element size: Increase fineness till stress converge 0.25 ($t \times t$) • Stress used for HSS calculation: Structural stress is defined as the greatest value of the component stress extrapolated in the normal direction to the weld
2002	Reijmers (2002)	<ul style="list-style-type: none"> • Model with solid elements: 8 or 20 node with element size: $t/2 \times t/2 \times t/2$ • Model with shell elements: 4 or 8 node with element size $t \times t$ • Recommendation: Increase the number of solid elements through plate thickness until solution converges.
2008	Hobbacher (2008)	<ul style="list-style-type: none"> • Refer to Table 2-2 and Table 2-3 • Include weld in solid element models • Do not include weld in shell element models
2010	Lee et al. (2010)	<ul style="list-style-type: none"> • Tension: No significant differences between linear and quadratic extrapolation presented in Hobbacher (2008) • Bending: The effect of higher order elements and number of node may be sufficiently small to ensure convergence. • Solid elements is recommended when dominant loading pattern is out-of-plane bending. • Recommend quadratic extrapolation for calculating stress under bending.
2010	Bhargava (2010)	<ul style="list-style-type: none"> • Model with solid elements: 8 or 20-node solid elements with element size 0.25 ($t \times t$) 20-node reduced integration solid element with element size $t \times t$ • Model with shell elements: 4-node thick shell elements • HSS calculation method: Linear extrapolation (Note: same as Hobbacher 2008) • Recommendation: Include weld in model • Stress used for HSS calculation: Maximum Principal Stress
2010	ABS (2010)	<ul style="list-style-type: none"> • Model with solid (Quadrilateral shape element) • Mesh guidelines: element size $t \times t$ • Aspect Ratio: 1:1 near HS, 1:3 to 1:5 away from HS • HSS calculation method: Linear extrapolation (Note: same as Hobbacher 2008) • Stress used for HSS calculation: Maximum Principal Stress
2011	DNV (2011)	<ul style="list-style-type: none"> • Model with solid elements: 20-node solid elements with element size: $t/2 \times t/2$ • Model with shell elements: 8-node shell elements with a size of $t \times t$ • HSS calculation method: Linear extrapolation 0.5t and 1.5t (Note: same as Hobbacher 2008) • Stress used for HSS calculation: Maximum Principal Stress
2012	Aygul (2012)	<ul style="list-style-type: none"> • Model with solid elements: 20-node solid element • Element size: 1 mm^3 (note: 1 in. = 25.4 mm) • Mesh guidelines: Fine (1mm, 2mm, 4mm), and Coarse (10mm) • HSS calculation method: Use quadratic extrapolation for Type B HSS (Note: same as Hobbacher 2008) • Recommendation: Include weld in model

Table 2-3. FE Pre/Post-Processing and HSS Calculation Guidelines (Contd.)

Year	Reference	FE Pre/Post Processing and HSS Calculation Guidelines
2012	Aygul (2012)	<ul style="list-style-type: none"> Model with shell elements: 8-node shell element (Quadrilateral element) Element size: 1 mm (note: 1 in. = 25.4 mm) Mesh guidelines: Fine (1mm, 2mm, 4mm), and Coarse (10mm) Mesh guidelines: Fine or Coarse (Note: same as Hobbacher 2008) Recommendation: Do not include weld in model
2012	Aygul (2012)	<ul style="list-style-type: none"> Model with shell elements: 8-node thick shell elements Element size: $\leq 4\text{mm}$ (quadratic), $\leq 10\text{mm}$ (linear) (Note: extrapolation equations are same as in Hobbacher (2008); 1 in. = 25.4 mm) HSS calculation method: Use linear or quadratic extrapolation for Type B Distance to reference points from weld toe: 4, 8, and 12 mm for quadratic 5 and 15mm for linear Recommendation: Include weld in the model at 45°

Table 2-4. FE Discretization Guidelines (Hobbacher 2008)

		HSS Type A			
		Coarse Mesh	Fine Mesh		
Reference Point Locations with Respect to Weld Toe					
		(a) Linear	(b) Linear	(c) Quadratic	
			HSS Type B		
			Coarse Mesh	Fine Mesh	
	(d) Linear	(e) Quadratic			

2.3.4.2 Submodeling

Berglund and Schultz (2006) used linear finite element (FE) models to understand the behavior of web gap distortion and correlate the fatigue stresses in the web gap region to differential vertical deflection between girders for skewed multi-girder steel bridges. However, the global structural FE models are not effective in accurately predicting the local behavior of the fatigue-sensitive detail or the web gap distortion. Also, use of a densely meshed model representing the entire bridge is not justifiable. Hence, the submodeling approach is widely used to understand the local behavior and to quantify the strains developed at critical details (Shifferaw and Fanous 2013; Bowman et al. 2012; Bhargava 2010; Ju and Tateishi 2012; Hassel 2011). The

submodeling approach reduces the time and effort required for refined FE analysis of such details.

A recent study conducted by Shifferaw and Fanous (2013), through field testing and FE analysis of a multi-girder steel bridge with web gap fatigue-sensitive detail, demonstrated the use of the submodeling approach with refined FE models to simulate out-of-plane displacement or distortion and the strains induced in the web gap region. Also, Bhargava and Roddis (2007) used a refined finite element analysis and field investigation to identify the source of distortion induced fatigue cracking at a cross-frame to girder connection. The state-of-the-art research performed on fatigue-sensitive details shows the usefulness of refined FE modeling and submodeling approaches for evaluating distortion induced fatigue details.

2.3.4.3 Fatigue Resistance Using a Reference Detail

Hobbacher (2008) presents structural details and detail specific S-N curves for fatigue assessment using hot spot stresses. When the detail to be assessed does not exactly represent the reference detail for a given S-N curve, Hobbacher (2008) recommends implementing the following steps for fatigue assessment using hot spot stresses:

- i. Select a reference detail with known fatigue resistance. The reference detail should represent the detail to be assessed as closely as possible in terms of geometry and loading.
- ii. Identify the type of stress used in the S-N curve for the reference detail. In a majority of the guides or specifications, S-N curves are presented in terms of nominal stress.
- iii. Develop finite element models of both details using the same element type and mesh discretization parameters.
- iv. Load both models to yield the stress identified in step (ii).
- v. Calculate hot spot stresses for both details. Label hot spot stress of reference detail and the detail to be assessed as $\sigma_{hs, ref}$ and $\sigma_{hs, assess}$, respectively.
- vi. Use Eq. 2-2 to calculate the constant amplitude fatigue threshold of the detail to be assessed: $CAFT_{assess}$. $CAFT_{ref}$ is the constant amplitude fatigue threshold of the reference detail.

$$CAFT_{assess} = k \cdot CAFT_{ref} \quad (2-2)$$

where, $k = (\sigma_{hs,ref}) / (\sigma_{hs,assess})$

Figure 2-7 shows the positions of reference S-N curve and the S-N curves used for assessment depending on the k value. As per Eq. 2-2, the $CAFT_{ref}$ value is adjusted using the hot spot stress ratio (k). When $k > 1$, the $CAFT_{ref}$ value is increased, and the $CAFT_{assess}$ and $S-N$ curve $_{assess}$ are located above the references. Similarly, when $k < 1$, the $CAFT_{ref}$ value is decreased, and the $CAFT_{assess}$ and $S-N$ curve $_{assess}$ are located below the references. Hence, depending on the hot spot stress ratio, the adjusted $CAFT_{ref}$ and $S-N$ curve $_{ref}$ can be used to assess fatigue-sensitive details using hot spot stresses.

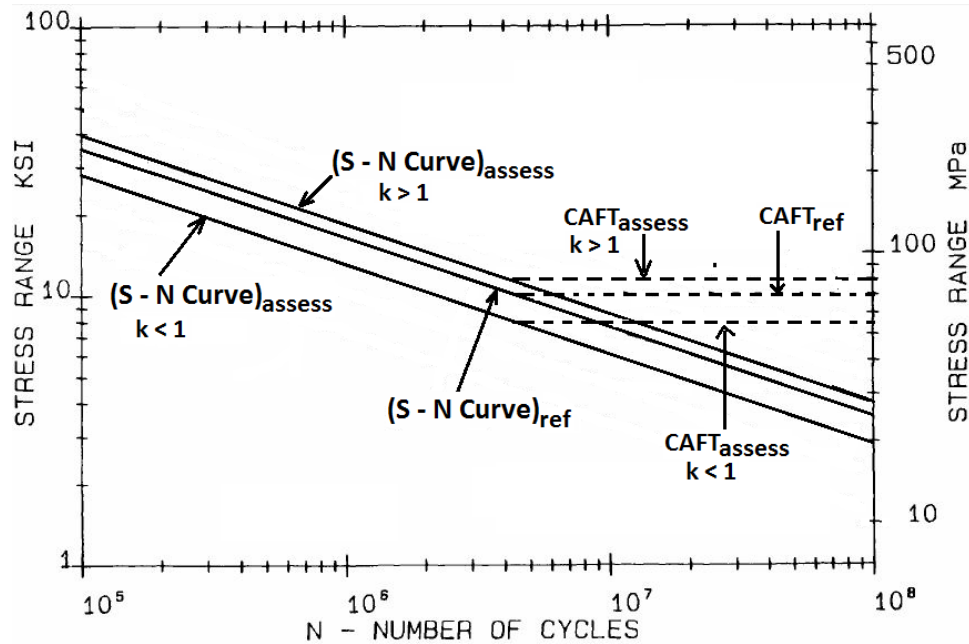


Figure 2-7. S-N curve definitions for fatigue assessment using hot spot stresses

2.3.5 Fracture Mechanics Based Assessment

The fracture mechanics approach with an assumed initial crack size can be used to estimate crack growth rate (Kühn et al. 2008; Lovejoy 2003). Once the crack growth rate is calculated, inspection frequency can be determined. As an example, Kühn et al. (2008) presented a case study for a railway bridge. In this example, a 0.2 in. (5 mm) visible crack at a rivet hole was considered, and the remaining fatigue life was calculated as 3.6 years (Figure 2-8). If the bridge owner decides to make three inspections, the detail needs to be inspected at least every 1.2 years. Using the inspection data, the calculated fatigue life can be revised, and the inspection or repair/retrofit/maintenance decisions can be taken accordingly. Even if no cracks are identified, the process can be implemented with an assumed crack size. Later, the inspection frequency can be revised based on the findings from the subsequent inspections.

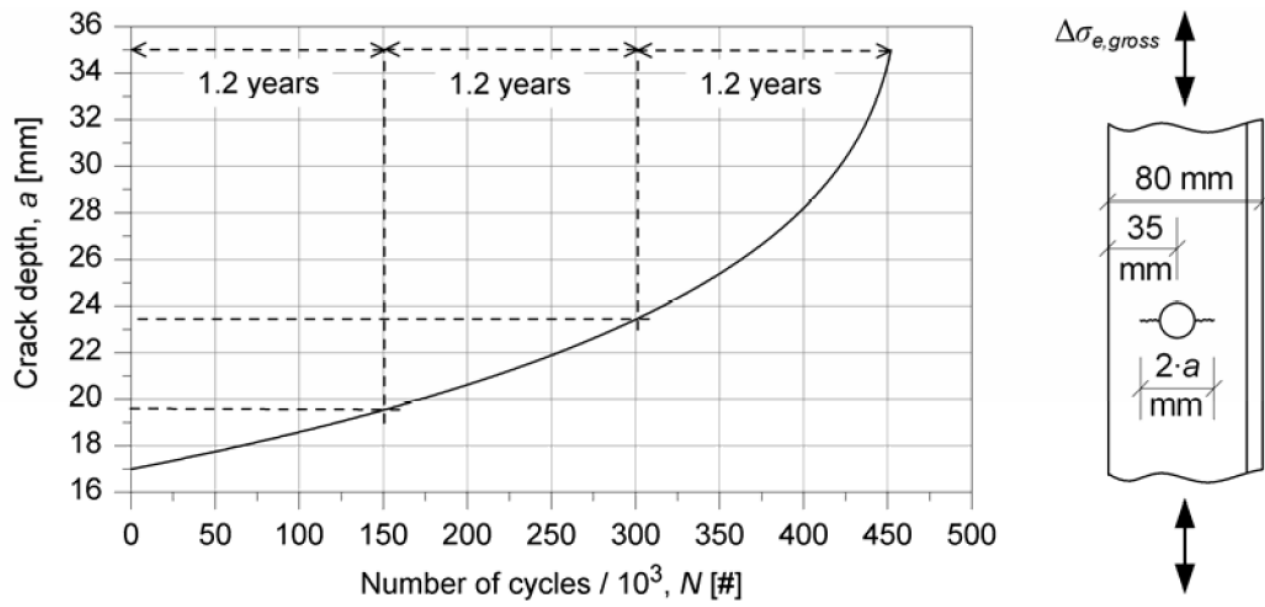


Figure 2-8. Fracture mechanics based fatigue life calculation for a riveted detail (Kühn et al. 2008)

A linear elastic fracture mechanics (LEFM) based inspection decision-making process was developed for Montana DOT fracture-critical bridges (Lovejoy 2003). This process was presented to the Federal Highway Administration (FHWA) and received the approval to be executed to assure safety of nonredundant bridges in lieu of implementing procedures given in the FHWA publication “Inspection of Fracture-critical Bridge Members”. With the ODOT process, a bridge specific inspection program was developed and, at that time, the expected cost saving from 1995 to 2005 was over \$6 million. An example fatigue inspection frequency decision-making diagram that was developed for a specific bridge in Oregon is shown in Figure 2-9. The figure was developed with an assumed initial crack size of ($2a_i = 38$ mm), a critical crack length of 6 in. ($2a_{crit} = 150$ mm), and the maximum allowable final crack length of 2 in. ($2a_f = 50$ mm). A safety margin of 3.5 has been achieved by setting the number of stress cycles between inspections to 500,000. Further, by knowing the number of cycles to grow a crack from a_i to a_f , the number of significant stress cycles per truck, and the ADTT at the site, a family of curves can be developed to help bridge managers schedule inspection intervals (Figure 2-10). By the year 2003, Oregon DOT has calculated the fracture-critical inspection period of 76 bridges in their inventory. The inspection periods of 76 bridges ranged from 2 to 10 years with an average of 8.4 years and a standard deviation of 2.7 years.

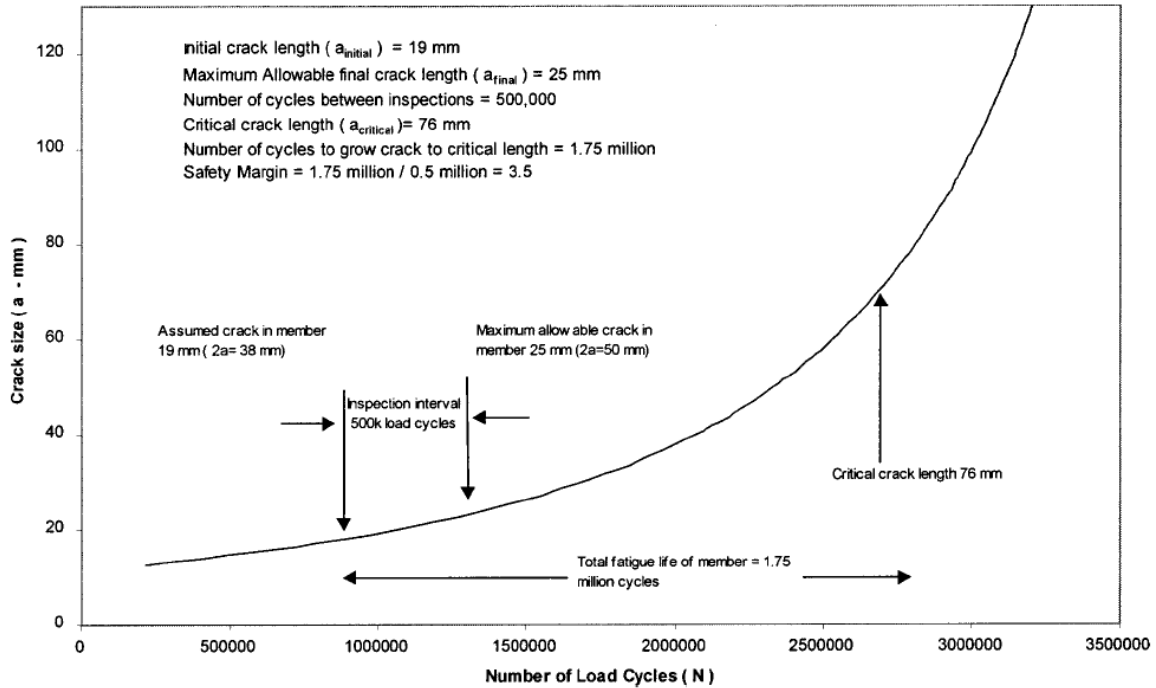


Figure 2-9. Details of an example fatigue inspection cycle (Lovejoy 2003)

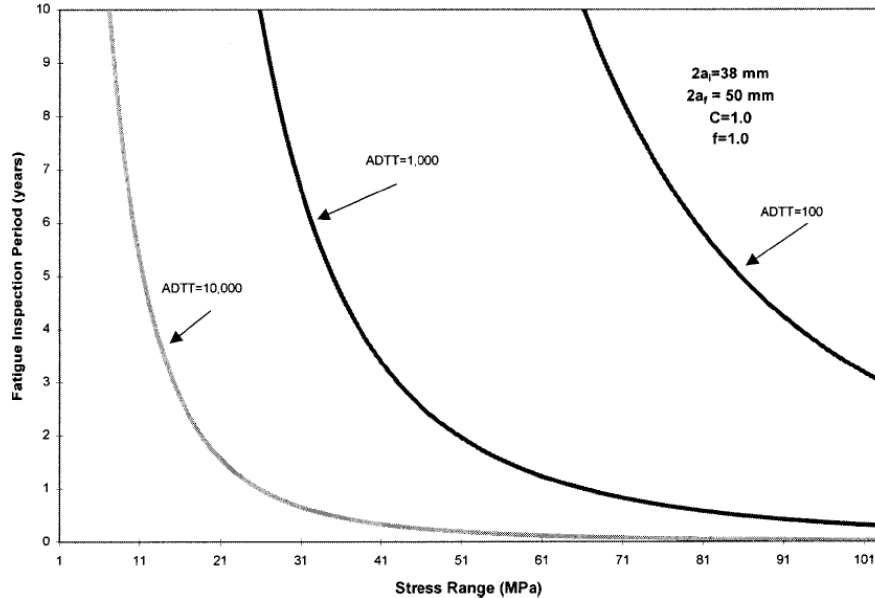


Figure 2-10. Fatigue inspection periods for a riveted section crack model at various stress ranges and traffic volumes (Lovejoy 2003)

When fracture mechanics analysis is implemented, establishing an accurate initial crack size is important. Åkesson (2010) suggests inspecting the detail using a nondestructive testing technique and selecting the largest non-detectable crack as the initial crack size. As an example, when inspected using ultrasonic testing, the largest non-detectable crack at a rivet hole is 0.08 in. (2 mm) and can be taken as the initial crack size (Åkesson 2010). However, if a crack is

detected, the practice is to immediately stop the growth (Åkesson 2010). In addition to visual inspection, ultrasonic testing and acoustic emission can be used to determine the presence of fatigue cracks. Ultrasonic testing is more suitable for rivet connections because of the challenges in identifying AE signals that arise from friction slip and crack propagation. AE is a more suitable method for welded structures to monitor crack growth (Åkesson 2010).

2.4 FATIGUE-SENSITIVE DETAIL MONITORING

Kühn et al. (2008) presented a four phase process to assess a structure and to develop remedial measures. These phases include (i) a preliminary evaluation by an engineer in accordance with the current codes, (ii) detailed investigation by an engineer with the help of an expert and supported with data collected using low tech NDT methods, (iii) expert investigations that may be supported with data collected using high tech NDT, and (iv) development of remedial measures such as structural response monitoring, enforcing load restrictions, or performing strengthening, repair or rehabilitation. Monitoring is used to support detailed investigation by experts as well as to evaluate the effectiveness of remedial measures.

When a large number of bridges with fatigue-sensitive details or fracture-critical members (FCM) are present, it is very difficult to evaluate fatigue performance of each and every detail of every bridge. Hence, Fu et al. (2003) suggested clustering bridges with fatigue-sensitive details (both load-induced as well as distortion-induced) based on the following attributes:

- Jurisdiction (state vs. local agency)
- Functional class of the roadway
- Type of construction (plate girders, rolled beams, trusses, etc.)
- Type of span (simple or continuous)
- Span length
- The year of original construction
- Any other parameter deemed necessary.

Once the clustering is completed, the presence of targeted fatigue-sensitive details, as well as the condition, can be verified by reviewing bridge plans, inspection records, and maintenance records. After that, a representative bridge from each cluster is selected to calculate remaining fatigue life or fatigue serviceability index. If the analysis results call for additional

investigations, the bridge can be monitored to evaluate stresses under ambient loads as well as to verify the presence of active cracking.

2.5 TECHNOLOGY FOR FATIGUE-SENSITIVE DETAIL MONITORING

2.5.1 Overview and Technology Selection

The mature as well as emerging technologies for fatigue-sensitive detail monitoring were synthesized and presented in Appendix C. The nondestructive evaluation (NDE) technology, which is currently integrated into the visual inspection for crack detection and characterization, was not included in the review. Table 2-5 lists the sensor types and implementation objectives. The information presented in Appendix C was used to develop Table 2-6 to Table 2-11 that list the implementation objective, technology, and technology ranking based on their capabilities and status (i.e., their ability to address the implementation objective and readiness for field implementation). The tables also include a few remarks to explain the ranking decisions. The technologies listed in the tables are: AE- acoustic emission, DIC- digital image correlation, EFS- electrochemical fatigue sensor, FD - Fatigue damage sensor, FF - fatigue fuse, PE- photoelasticity, RFID- radio frequency identification, SG- strain gauges, TS- temperature sensor, USGW- ultrasonic guided wave, and XRD- X-ray diffraction. The rank 1 to 4 indicates the most to least appropriate technology for achieving the implementation objectives.

Table 2-5. Sensor Types and Implementation Objectives

Sensor Type	Implementation Objectives
<i>Fatigue event detection and characterization of cracks</i>	
Acoustic emission (AE) sensors	detect crack initiation detect crack growth or the status of a crack (i.e., an active or a dormant crack) detect potential for cracking (material plasticity)
Electrochemical fatigue sensors (EFS)	detect crack growth or the status of a crack detect potential for cracking (monitoring strain localization and/or micro plasticity)
Fatigue damage (FD) sensor and fatigue fuse (FF)	measure crack growth rate predict remaining fatigue life
Radiofrequency identification (RFID) chips	measure strain detect crack initiation detect crack growth
Ultrasonic guided wave (USGW) sensors	detect presence of a crack detect crack growth
<i>Complementary technologies</i>	
Accelerometers	measure dynamic response
Displacement transducers, laser distance measurement sensors	measure differential deflection measure component deformation
Digital image correlation (DIC)	measure displacement and strain
Strain gages (SG) and strain transducers (ST)	measure strain (with many objectives including fatigue life calculation, crack growth potential evaluation, and development of a load matrix for evaluating AE data)
Thermocouples (TC)	measure temperature
Laser point tracking	measure displacement
Photoelastic (PE) imaging	measure strain
X-ray diffraction (XRD)	measure strain (to determine in-service dead load, load path determination, crack-stop hole validation by checking the stresses around it, baseline stress measurement for enhanced structural monitoring, and residual stress measurement before and after a retrofit).

Table 2-6. Crack Initiation Detection Technology

Objective	Tech	Rank	Remarks
Crack initiation detection	AE	1	This technology has been widely implemented in the field. It can detect crack initiation based on the energy release from the source. Also, the technology is capable of detecting material plasticity by monitoring burst signals that arise due to dislocation or slipping of portions of the crystal over on another near the yield stress. However, when the signal strength is weak, it requires implementation of filtering techniques and advanced analysis tools to differentiate between crack-relevant emissions from ambient noise. The rank is assigned considering its extensive use in the field, suitability for long-term as well as short-term monitoring, capability to identify early signs to crack initiation, capability to identify the crack location, and mainly not requiring mounting the sensors over a potential crack location.
	EFS	3	Limited use under field conditions. Sensors can be used only for short-term monitoring due to evaporation of the electrolyte. Primarily one group has been involved in technology implementation so far; hence, limited experience is documented. Technology has the capability to capture early signs to crack initiation and growth in sensitive regions. It is required to mount the sensor over or very close to a crack tip to monitor growth. The ranking is assigned because there are limited field implementations; as well it requires identifying potential crack initiation locations. Also, the technology, at the current state, is not suitable for long-term monitoring due to the need of refilling electrolyte.
	RFID	4	RFID tags are like EFS that need to be placed on top of the potential crack location. However, technology is still under development.
<i>Rank 1 to 4 defines most to least appropriateness of the technology for the stated purpose</i>			
<i>Abbreviations: AE- Acoustic emission, EFS- Electrochemical fatigue sensor, RFID- Radio frequency identification.</i>			

Table 2-7. Crack Location Detection Technology

Objective	Tech	Rank	Remarks
Crack location detection	AE	1	Crack source location can be detected in real-time with the use of sensor arrays mounted around the area of concern and the time of arrival of the emissions at sensors within the array. The theory behind crack location measurement is well developed. The rank is assigned because of availability of successful implementation records and well developed procedures in establishing crack location.
	USGW	3	Sensor data need to be transferred from the field and need to be mapped using tomography approach to locate the cracks. The rank is assigned because it is an active sensor system and requires triggering to detect crack growth. Very limited use in bridges and details with multiple plates and welds. Also, pulsers and receivers need to be mounted on either side of a potential crack growth location or an existing crack to detect the crack location.
	DIC	4	These technologies are only evaluated under laboratory conditions. Sensitivity of the technology is not evaluated under laboratory and field conditions. The noise levels of DIC and PE methods are around +/-150 µε and +/- 50 µε, respectively, and may lead to greater inaccuracies under outdoor conditions. Technology is expensive and not suitable to monitor hidden cracks. However, they have the potential of monitoring crack growth over a large area within the camera range.
	PE	4	
<i>Rank: 1 to 4 defines most to least appropriateness of the technology for the stated purpose</i>			
<i>Abbreviations: AE- Acoustic emission, DIC- Digital image correlation, PE- Photo elastic, USGW- Ultrasonic guided wave.</i>			

Table 2-8. Crack Growth Monitoring Technology

Objective	Tech	Rank	Remarks
Crack growth monitoring	AE	1	AE is capable of providing indication of crack growth activity. When a sensor array is used, the instantaneous location of the crack activities can be continuously recorded to give the crack growth path. AE can detect material plasticity by monitoring burst signals that arise due to dislocation or slipping of portions of the crystal over on another near the yield stress. However, the signal strength is weak and requires implementation of filtering techniques and advanced analysis tools. AE can detect and monitor internal cracks that are not visible on the surface. The rank is assigned because the technology is good for long-term monitoring, capable of identifying early signs to crack growth, supported with advanced data analysis tools, and has a long history of field implementation. Also, the sensors can be mounted in different configurations based on the monitoring objectives.
	EFS	2	The crack sensor needs to be mounted on top of an existing crack tip and also along the predicted crack growing path to monitor the crack growth. It requires mounting a reference sensor in a place with similar stress intensity. A rapidly growing crack may grow beyond the sensors. Rank is assigned because the technology, at the current state, is not suitable for long-term monitoring due to the need of refilling electrolyte as well the need for mounting the sensor over the identified crack tip. However, EFS are capable of identifying early signs to crack growth. After identifying an existing crack, growth can be monitored.
	USGW	2	USGW sensors need to be mounted in an array such that the detectable path of the actuators and sensors is intercepted by the growing crack. The rank is assigned because the sensors can be triggered automatically at regular intervals or when a crack activity is detected by other sensors such as AE. However, the sensors need to be mounted around an existing crack or a potential crack location. Further, the technology is still under evaluation stages for fatigue crack monitoring of multi girder bridges.
	FD and FF	3	Technology is still under development with limited applications. Data interpretation is simple; however, sensor calibration can be a challenge depending on the complexity of the detail configuration, boundary conditions, and loading. Crack growth can be determined based on the number of fuses broken.
	DIC	4	These technologies are only used under laboratory conditions. Sensitivity of the technology is not evaluated under laboratory and field conditions. The noise levels of DIC and PE methods are around +/-150 µε and +/- 50 µε, respectively, and may lead to greater inaccuracies under outdoor conditions. Technology is expensive. It may not be suitable for monitoring internal cracks. However, the technology has the potential for monitoring crack growth over a large area within the camera ranges.
	PE	4	
	SG	3	Technology is primarily used for developing a load matrix rather than crack growth monitoring. However, multiple strain gages can be placed across the potential crack growth path to detect crack growth similar to FD or FF.
	RFID	4	RFID tags are like EFS that need to be placed on top of the growing crack. However, technology is still under development.
<i>Rank: 1 to 4 defines most to least appropriateness of the technology for the stated purpose</i>			
<i>Abbreviations: AE- Acoustic emission, EFS- Electrochemical fatigue sensor, FD- Fatigue damage sensor, FF – Fatigue fuse, PE- Photo elastic, SG- Strain gauges, RFID- Radio frequency identification, USGW- Ultrasonic guided wave.</i>			

Table 2-9. Technology for Fatigue Crack Characterization

Objective	Tech	Rank	Remarks
Crack characterization	AE	2	AE can be used to characterize cracks; however, it requires access to a database of signals with characteristics specific to the detail being investigated. Hence, the signal characteristics need to be specific to the material, component thickness, surface finish, temperature, etc. Further, having access to an advanced data analysis tool is important to evaluate signal characteristics.
	EFS	2	The crack sensor needs to be mounted on top of the existing crack tip and also along the predicted crack growing path to monitor the crack growth. The system requires mounting the reference sensor in a place with comparable stress intensity. A rapidly growing crack may grow beyond the sensors. The technology, at the current state, is not suitable for long-term monitoring due to the need of refilling electrolyte.
	USGW	2	Sensors need to be mounted around an existing crack or in an array with enough sensors to capture sufficient wave propagation data around the potential damage area. Data can be analyzed and interpreted for crack characterization using tomography approach. Technology is not widely used and needs additional research to integrate this technology in bridge monitoring.
<i>Rank: 1 to 4 defines most to least appropriateness of the technology for the stated purpose</i>			
<i>Abbreviations: AE- Acoustic emission, EFS- Electrochemical fatigue sensor, USGW- Ultrasonic guided wave.</i>			

Table 2-10. Strain Measurement Technology

Objective	Tech	Rank	Remarks
Strain measurement	SG	1	Strain gauges are a proven technology for strain measurement. Real-time strain measurement data can be obtained. Single or multiple gauges can be used to measure the strain field distribution of a global structure or local component. However, the measurements are made at discrete locations.
	FD and FF	2	Technology is still under development with limited applications. Strain is recorded with the aim of calculating remaining fatigue life. Hence, the technology is more suitable for remaining fatigue life calculation rather than just the strain monitoring. Data interpretation is simple; however, sensor calibration can be a challenge depending on the complexity of the detail configuration, boundary conditions, and loading. Ranking is primarily assigned due to lack of implementation records in the field of bridge engineering.
	DIC	4	These technologies are mainly used under laboratory conditions. Very limited field applications for displacement and strain measurement of highway bridges. Suitable for measuring strain field distribution of a larger area within the range of the camera. Measurement accuracy depends on many factors including exposure and ambient conditions. Ranking is primarily assigned due to lack of implementation records in the field of bridge engineering.
	PE	4	
	XRD	4	A newly developed technology to measure in-service strain using x-ray diffraction method. It requires point to point measurement. It is not suitable for continuous monitoring at multiple locations simultaneously. Technology has been use in very limited highway bridge applications. Ranking is assigned considering the inability to incorporate into remote monitoring at the current state of technology.
<i>Rank: 1 to 4 defines most to least appropriateness of the technology for the stated purpose</i>			
<i>Abbreviations: DIC- Digital image correlation, FD- Fatigue damage sensor, FF – Fatigue fuse, PE- Photo elastic, SG- Strain gauges, XRD- X-ray diffraction.</i>			

Table 2-11. Technology for Measuring Residual Stress and Temperature

Objective	Tech	Rank	Remarks
Residual stress measurement	XRD	1	The only technology currently implemented in the field. Uses x-ray diffraction for measuring residual stress of a component. The technology has been used at a few bridge sites. Ranking is assigned considering the state-of-the art. Further, measuring residual stress is required only at discrete times, mainly to calculate remaining fatigue life.
Temperature	TS	1	Continuous measurement can be recorded for parametric input required for other technologies such as AE and strain measurement.
<i>Rank: 1 to 4 defines most to least appropriateness of the technology for the stated purpose</i>			
<i>Abbreviations: TS- Temperature sensor, XRD- X-ray diffraction.</i>			

Section 1.1 lists the SHM system implementation objectives. As per the listed objectives, technology for detecting crack initiation and growth needs to be a major component in any fatigue-sensitive detail monitoring system. AE is the most effective technology for fatigue event detection (i.e., crack initiation or crack growth monitoring). AE has been successfully implemented in the field and evaluated for continuous monitoring of fatigue-sensitive details. At this time, AE is the only technology that is capable of real-time monitoring of fatigue events and providing data for damage location detection. In addition, strain gauges are required to validate FE models, evaluate stress state to calculate remaining fatigue life, and support AE data analysis by developing the load matrix. Even though Table 2-6 to Table 2-9 list capabilities of AE justifying the selection of technology, reasons for widespread use of the technology is listed below;

- AE can be used as a local as well as global crack growth monitoring tool (FHWA 2012; DFT UK 2006; Nair and Cai 2010).
- Very minimal surface preparation is required to mount AE sensors (FHWA 2012).
- Frequent access to a detail is not required once the sensors are installed (FHWA 2012; DFT UK 2006).
- Portable systems with permanently attached transducers to bridge structures offer cost effective means of as-needed remote monitoring (McCrea et al. 2002; DFT UK 2006).
- AE is capable of locating the source of failure (McCrea et al. 2002; DFT UK 2006).
- AE is capable of detecting and locating defects in areas obscured from view or in areas that are difficult to inspect (e.g. weld defects, material imperfections, etc.) (Kaphle et al. 2012; DFT UK 2006).

- The data from an AE monitoring system can be used to track the history of crack growth activity (FHWA 2012).
- Parametric data (strain, displacement, temperature, etc.) can be used to correlate AE events to improve the accuracy of data analysis results (DFT UK 2006).
- Real-time data display tools are available (Wang et al. 2012).
- AE signal characteristics and source location plots can be used to help with management decisions (DFT UK 2006).
- Technology has been used for decades in many disciplines and the experience is well documented.

With any technology there are advantages as well as implementation challenges. This is no different to AE technology implementation and data interpretation. Table 2-12 lists the challenges and potential solutions.

Table 2-12. AE Implementation Challenges and Mitigation Strategies

AE Implementation Challenges	Mitigation Strategies
In many instances, AE can only qualitatively gauge the extent of damage.	In order to obtain quantitative results about size, depth, and overall acceptability of a part, other NDT methods are necessary. Source: FHWA 2012; DFT UK 2006; Huang et al. 1998; Nair and Cai 2010.
Extensive expertise is required to plan, set up the sensors, test, and interpret results (McCrea et al. 2002)	
Standardized procedures are not available for all types of bridges, as most recommendations cater for bridges under unique conditions of loading, materials, etc. (Nair and Cai 2010)	Volume 3 section 1 of the Design Manual for Roads and Bridges (DMRB), Department for Transport, UK, Advice Notes on the Non-Destructive Testing of Highway Structures, provides guidelines up to a certain extent to help the technology implementation. Source: DFT UK 2006
Impact of the service environments which contribute extraneous noise to the signals. (FHWA 2012; DFT UK 2006; Huang et al. 1998)	<ul style="list-style-type: none"> • In order to eliminate receiving background noise in AE data acquisition, a lower bound threshold can be defined. Any signal below a user defined signal voltage or signal in the frequency range will be ignored by the AE sensor. • Parametric data (e.g., strain) can be used to distinguish crack related AE from noise that occurred from mechanical rubbing and crack closure. • Waveforms and their characteristics obtained from Pencil Lead Break (PLB) tests are used as a frame of reference to indicate genuine hits. Typical parameters of AE waveforms include peak amplitude, rise time, duration and emission counts. The waveforms caused by ‘false’ hits generally have long rise time, high counts, and poorly defined peak amplitudes. In PLB tests, the signal rapidly attains the peak amplitude and then fades gradually. The shorter rise time, lower counts, and higher peak amplitude are indicative of genuine AE, as is the clean front-end of the waveform. <p>Source: Yu et al. 2011; Shield and McKeefry 1999; Schultz and Thompson 2010; Wang et al. 2012.</p>
Background noise due to fretting bolts (Shield and McKeefry 1999)	<ul style="list-style-type: none"> • A method for eliminating this type of background noise is placing an AE sensor local to the fretting bolt, and then all AE signals this sensor receives will be ignored by sensors located at other areas of interest. • The noise generated from fretting of bolted connections can be filtered by selecting a lower bound threshold value. <p>Source: Schultz and Thompson 2010</p>
Background noise from grating	AE from grating can be minimized by collecting and analyzing signals in the vicinity of the peak load. Source: Yu et al. 2011
Acoustic coupling requires clean smooth flat surface and the removal of thick coating (McCrea et al. 2002)	

Table 2-12. AE Implementation Challenges and Mitigation Strategies (Contd.)

AE Implementation Challenges	Mitigation Strategies
Transducer arrangement is critical to the results (McCrea et al. 2002)	<ul style="list-style-type: none"> • The effect of geometric dispersion of signals can be limited by an understanding of the structure e.g. the hidden structure of box girders, and by placing AE sensors accordingly. When high accuracy is required for complex structures the number of sensors is generally increased. • Attenuation survey is performed to identify the optimal sensor placement. • Resonant AE transducers are used in linear or planar arrays to detect the presence and the location of defects and to monitor their activity under normal service loads • Areas of interest, such as long welds, can be monitored using linear arrays. <p>Source: DFT UK 2006; Hay et al. 2009.</p>
Lack of possibilities to intensify the elastic waves field (McCrea et al. 2002)	<ul style="list-style-type: none"> • Perform attenuation survey to identify the optimal sensor placement. <p>Source: DFT UK 2006</p>
Measurements cannot be repeated (McCrea et al. 2002)	<ul style="list-style-type: none"> • The data from an AE monitoring system can be continuously recorded and used to track the history of crack growth activity. • Data replay tools can be used to backtrack the activities. <p>Source: FHWA 2012</p>
Signals are transient and random in time (standard noise reduction methods cannot be used) (McCrea et al. 2002)	<ul style="list-style-type: none"> • Perform attenuation survey to identify the optimal sensor placement. • Implement advanced analysis techniques such as supervised and un-supervised learning techniques and pattern recognition classification. <p>Source: DFT UK 2006; ICEPak® 2014</p>
Several simultaneous measurements are required for verification and orientation and determination (McCrea et al. 2002)	Pencil lead break should be conducted before each fatigue test to check the condition of each sensor and the location capability of the sensor arrangement.
It does not detect defects that are either no longer propagating or not growing during the monitoring period (e.g., dormant cracks) (DFT UK 2006)	Structure should be monitored under representative peak loading
It may not be applicable where the critical crack size is very small and failure is sudden (DFT UK 2006)	
Generation of large amount of data during the testing; hence an effective data analysis and management is necessary, especially for long term monitoring (Kaphle et al. 2012)	Use of pencil lead break calibration and setting up the thresholds and gains will help eliminate most of the noises; thus, reducing the amount of data collected by the system.
Quantifying the level of damage to assess severity of source (Kaphle et al. 2012)	Develop a database of signals through laboratory studies to correlate.

Table 2-12. AE Implementation Challenges and Mitigation Strategies (Contd.)

AE Implementation Challenges	Mitigation Strategies
<p>AE monitoring of bridges has generally been limited to short-term tests contingent on either fair weather or availability of some shelter on site.</p>	<p>A weatherproof enclosure could be developed to protect the AE hardware and connect it to a rugged laptop computer and a battery-backed power supply. This enclosure could be clamped to the bridge near the area of interest making long noise-prone cable runs unnecessary. To reduce electrical noise and spurious AE hits from the enclosure itself, the enclosure could be mounted on rubber feet and placed well outside the AE arrays so any events would be rejected by the AE processing filters</p> <p>Source: Kosnik 2008</p>
<p>The noise levels are greater than the threshold</p>	<p>In order to filter out the noises, the following approaches can be implemented.</p> <ul style="list-style-type: none"> • The first-hit channel (FHC) analysis. • Use average frequency spectrums of triggered signals from multiple (e.g., three) AE sensors. • Use a guard sensor. • Use a band-pass filter. <p>Source: Kosnik 2008; Zhang et al. 2013; Schultz and Thompson 2010</p>
<p>Complicated geometry and low strength signals make tasks unattainable.</p>	<ul style="list-style-type: none"> • Implement advanced signal enhancing and processing techniques to minimize the noise effects. • Monitor the structure for an adequately long duration to identify the noise sources and establish thresholds to capture fatigue events. • Use wave form parameters such as amplitude, energy, signal count, signal rise time, and signal duration for event characterization. • Use Artificial Intelligence Software such as ICEPAK™ for development and implementation of pattern classifiers. <p>Source: Kosnik 2008; Schultz and Thompson 2010; Hay et al. 2000.</p>

2.6 TECHNOLOGY IMPLEMENTATION CONSIDERATIONS

Kühn et al. (2008) presented a four-phase process to assess a structure. According to the four-phase procedure, AE is implemented during the 3rd and 4th phases. Fu et al. (2003) conducted a study to evaluate the effect of truck weight on bridge network cost. As part of the study, a methodology was developed to calculate the network level maintenance, repair, and replacement costs due to fatigue damage. The first step in the evaluation starts with clustering bridges based on the jurisdiction, functional class, type of construction, type of span, span length, and the year of construction. Then the fatigue life of selected bridge details is calculated to make repair, retrofit, or replacement decisions. A similar bridge clustering approach as well as the DOT experience with specific structures in the inventory can be used to identify bridges and/or details for monitoring stress state and AE events.

Implementation of structural response or health monitoring systems requires a thorough understanding of the structure, careful planning, development of strategies based on the level of

monitoring and the duration of monitoring, access to advanced data analysis tools, and experience in understanding data and results interpretation (DFT UK 2006).

2.6.1 Level of Monitoring

Global, semi-global, and local monitoring are the three levels (DFT UK 2006). The purpose of global monitoring is to identify potential source locations for planning or asset management purposes. Semi-global monitoring is implemented to identify individual source locations to assess, evaluate, and rank. The ranking is based on the source characteristics and number of emissions. The ranking system is used for planning detailed investigations through local monitoring. The local monitoring is implemented to identify the source location and to characterize the AE events. The AE event characteristics help to identify if the events are due to fatigue crack growth or due to fretting, crack opening or closure, or grinding of debris between cracked surfaces. The local monitoring can also be implemented as a means of quality control. As an example, a retrofit can be monitored using AE to evaluate if the defect's growth has been arrested. Local monitoring can be planned based on global/semi-global monitoring results, inspection records, or prior experience with similar details. The sensor array for local monitoring is designed based on (a) the classification of the detail to be monitored, (b) prior experience with similar details, (c) inspection and maintenance records, (d) an attenuation survey, (e) the finite element (FE) analysis, or a combination thereof.

2.6.2 Monitoring Duration

When an AE monitoring system is installed, it is crucial that the correct monitoring duration is selected so that active defects may be detected. The key to successful implementation of an AE monitoring system is an understanding of the causes and damage mechanisms. Knowing the stimulus of cracking, the monitoring duration can be selected. Otherwise, a short monitoring duration may miss defects that are inactive under prevalent conditions; whereas an extended monitoring may provide no further new information while being carried out at an unnecessary cost. Volume 3 section 1 of the Design Manual for Roads and Bridges, Department for Transport, UK, Advice Notes on the Non-Destructive Testing of Highway Structures, defines monitoring duration as short-term, loading pattern dependent, and permanent (DFT UK 2006).

2.6.2.1 Short-Term Monitoring

Short-term monitoring is suitable when the causes and damage mechanisms are well understood. The monitoring duration should be sufficiently long to acquire a representative amount of AE data to locate the sources and characterize the cracks. Short-term monitoring is also used as a means of quality control, such as an evaluation of effectiveness of a retrofit (i.e., use of crack stop hole). Even though it is not related to AE monitoring, Zhou (2005) suggested monitoring a structure for at least seven continuous days to capture the structural response to a representative unit of vehicle patterns. Monitoring duration is primarily based on the traffic patterns. On certain roads, a uniform traffic volume can be observed. Different monitoring durations and use of data for fatigue evaluation are documented by Li et al. (2001), Leander et al. (2009), and Fasl et al. (2013).

2.6.2.2 Loading Pattern Dependent Monitoring

When AE monitoring is used as a means of quality control, such as an evaluation of effectiveness of a retrofit (i.e., use of crack stop hole), loading pattern dependent monitoring is more effective than short-term monitoring. When loading pattern dependent monitoring is implemented, the monitoring duration decision needs to be made with a clear understanding of the traffic patterns. In such cases, it is vital to have access to weigh-in-motion (WIM) data or information about a passage of a permit load (a large load). According to DFT UK (2006), monitoring AE activities should take place for one day on a heavily travelled road bridge with uniform traffic patterns and for two or more days for less frequently trafficked bridges.

2.6.2.3 Long-Term or Continuous Monitoring

Long-term or continuous monitoring is primarily recommended when fatigue-sensitive or fracture-critical details are present, and the bridge is a critical node of a road network. Even if data is collected sporadically, it is cost effective to have the sensors and the cables mounted on a bridge when it is challenging to access the bridge due to traffic or site conditions.

2.6.3 Inspection Frequency

Stresses calculated using refined models and accurate traffic data can be used as the first step to make a decision on the level of inspection or monitoring. Kühn et al. (2008) recommended using

Miner's rule to calculate fatigue life. As the initial step, it is recommended to calculate stresses using codified loads, if possible, using more realistic loads calculated from WIM data. As shown in Figure 2-3, when the remaining fatigue life is greater than 10 years, performing biennial inspection is adequate.

If the calculation yields a negative life, it is recommended to inspect the detail for presence of cracking. Cracks can be present in two forms: active and dormant. If the detail is accessible for close inspection, traditional NDE can be used to locate cracks and measure crack size. If no cracks are identified using traditional NDE or all the surfaces are not accessible for inspection, short-term or loading pattern dependent monitoring can be performed using a system with AE and strain sensors. Strain data can be used to recalculate the fatigue life and provide parametric input for AE data analysis while AE data can be used to identify the presence of active cracking.

If no cracks are found, the inspection frequency is normally decided based on the codes and/or experience. However, inspection frequency can be determined based on an estimated crack growth rate. The fracture mechanics approach with an assumed initial crack size can be used to estimate crack growth rate (Kühn et al. 2008; Lovejoy 2003). Once the crack growth rate is calculated, inspection frequency can be determined. Two such examples are presented in section 2.3.5. Establishing an accurate initial crack size is very important when the fracture mechanics based approach is used to determine inspection frequency. As suggested by Åkesson (2010), the largest non-detectable crack size, determined by applicable nondestructive testing techniques, can be used as the initial crack size. However, if a crack is detected, the practice is to immediately stop the growth (Åkesson 2010). The AE system can be used to monitor crack growth or effectiveness of a repair.

2.7 REPAIR AND RETROFIT METHODS

Repair of a fatigue-sensitive detail is performed to arrest the propagation of a crack, while retrofitting is primarily meant to upgrade the fatigue resistance of a detail with the goal of prevention of cracks (Dexter and Ocel 2013). According to Dexter and Ocel (2013), the repair and retrofit methods can be broadly grouped into surface treatments, repair of through-thickness cracks, modification of connections or the global structure to reduce the cracking potential and repair of out-of-plane distortion (Figure 2-11). Repair methods for out-of-plane distortion can be

grouped into hole drilling, diaphragm or cross-frame removal, diaphragm repositioning, bolt loosening, web-gap stiffening and web-gap softening (Figure 2-12). In addition to using doubler plates, several methods that are listed in Figure 2-12 such as diaphragm or cross-frame removal, diaphragm repositioning, and bolt loosening methods can be used to modify the global structure response. Also, a connection can be modified by loosening the bolts.

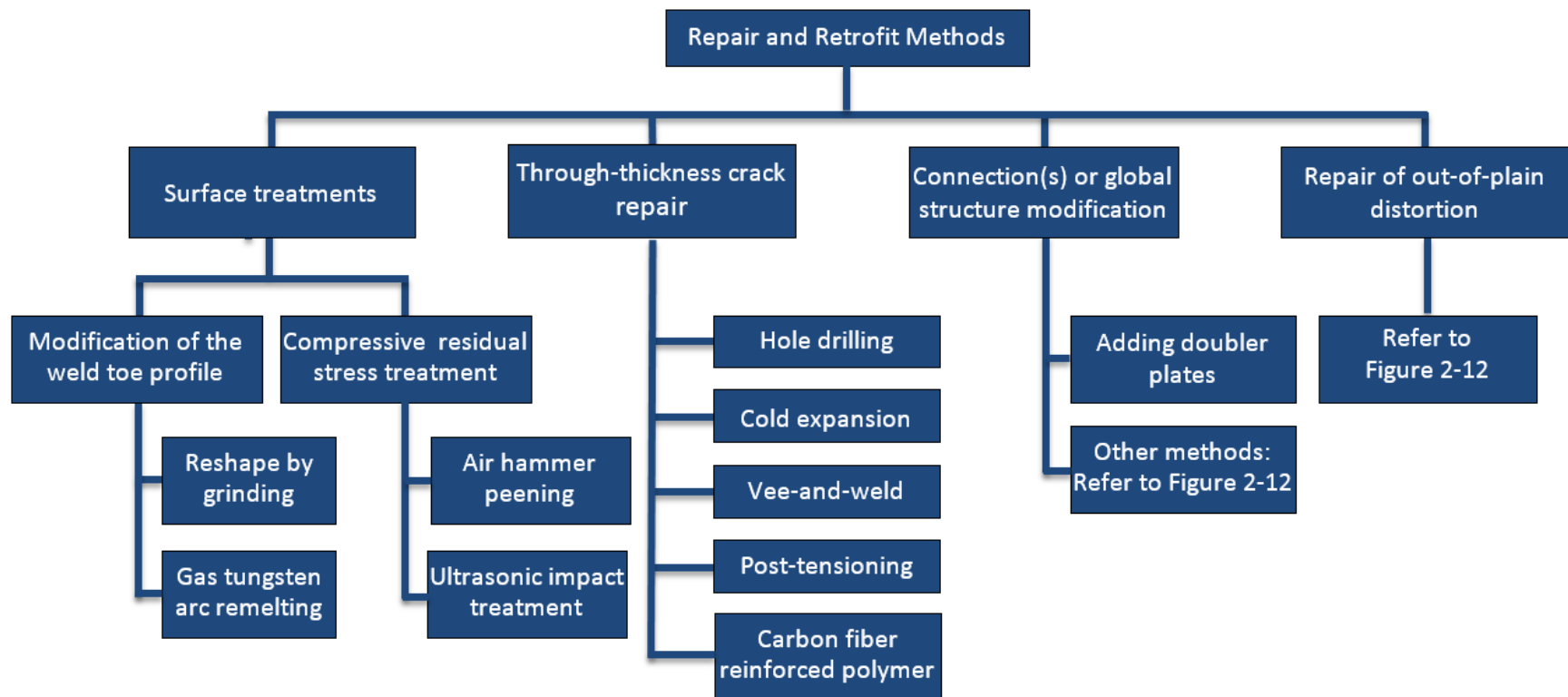


Figure 2-11. Repair and retrofit methods

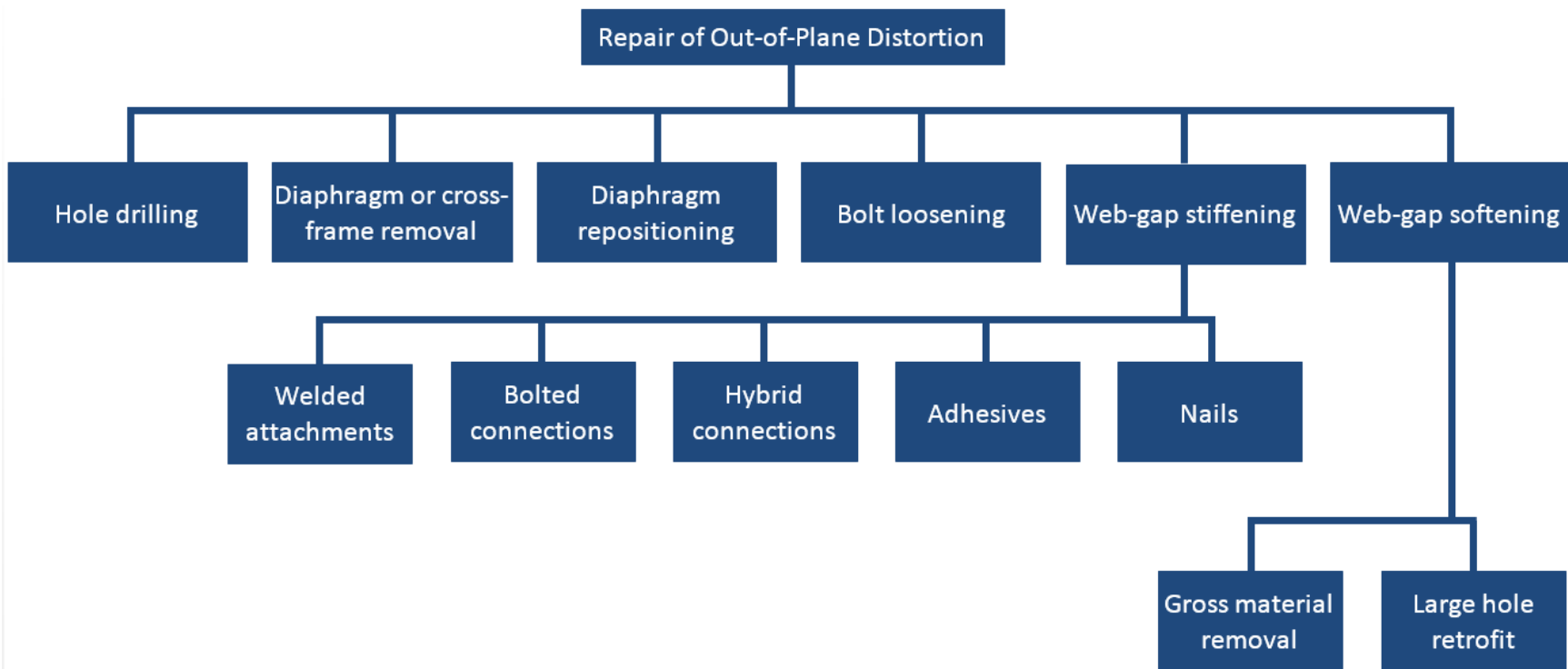


Figure 2-12. Repair of out-of-plane distortion

2.7.1 Surface Treatments

The weld toe surface treatment methods can be divided into two major groups: modification of the weld toe profile and compressive residual stress treatments. These methods can be used as retrofit techniques to enhance fatigue durability of uncracked welds (Dexter and Ocel 2013).

2.7.1.1 *Modification of a Weld Toe Profile*

The objective of modifying a weld toe profile is to remove the intrusions at the weld and obtain a continuous connection between the weld and the plate. This can be achieved by reshaping a weld toe using a grinder or remelting a weld using gas tungsten arc welding equipment.

2.7.1.1.1 Reshape by Grinding

Disc and burr grinders can be used to reshape a weld toe (Figure 2-13, Figure 2-14, and Figure 2-16). Small cracks at the edges of flanges or other plate like components can be effectively removed by grinding. However, grinding is not an effective or an economical method for removing microcracks in welds; it is also not the best on-site process for reshaping welds at bridge details. Grinders are commonly used in the field as finishing tools during repair or retrofit of bridge details (as an example, burr removal during hole drilling or toe finishing during welding).

Only the required amount of material is removed by grinding. If extra material is lost, the surface may be gouged (Dexter and Ocel 2013). To control the amount of material removed, the angle that the equipment must be held is important. When a disc is utilized, the disc must be held at an angle between 30° to 45° to the material being ground (Figure 2-13). When a burr is preferred, it must be held only at 45° as shown in Figure 2-15. In more confined regions, a burr is recommended over a grinding disc (Gregory et al. 1989).

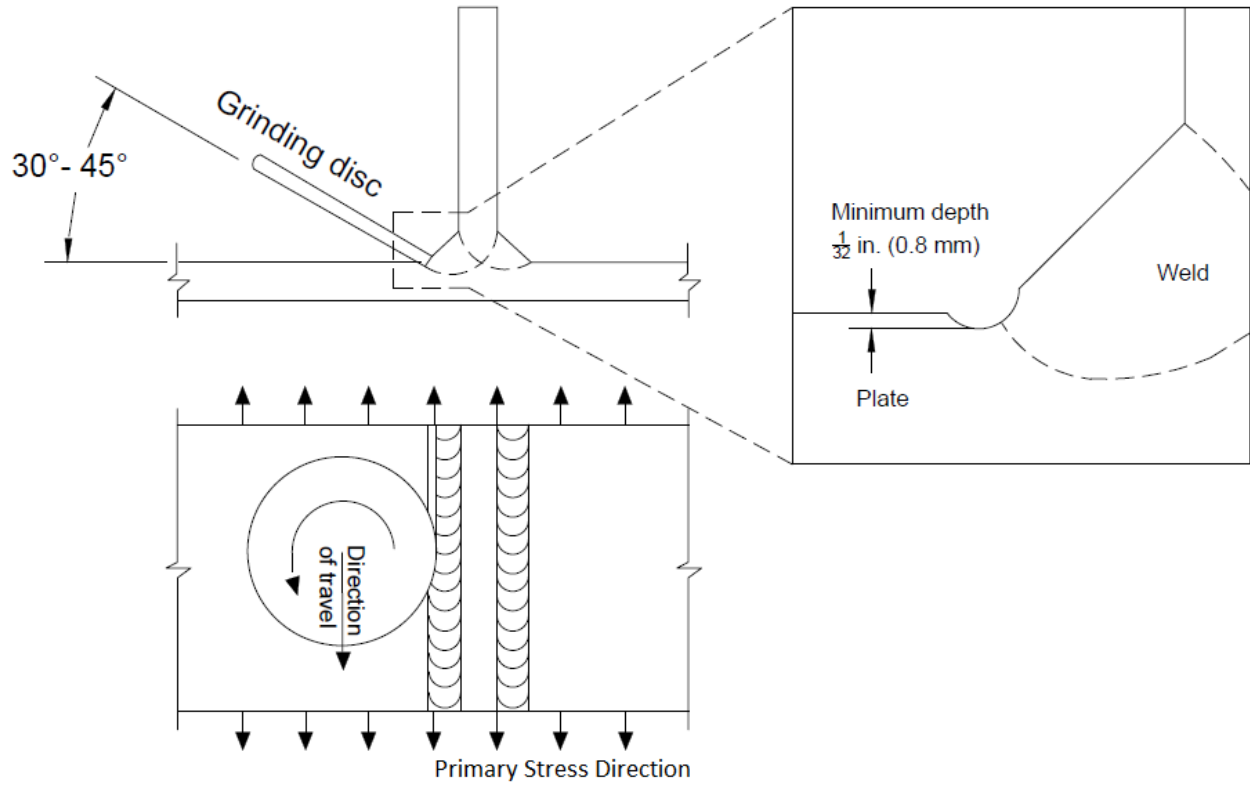


Figure 2-13. Position of the grinding disc with respect to the material surface (Gregory et al. 1989)



Figure 2-14. Typical disc grinders (Dexter and Ocel 2013)

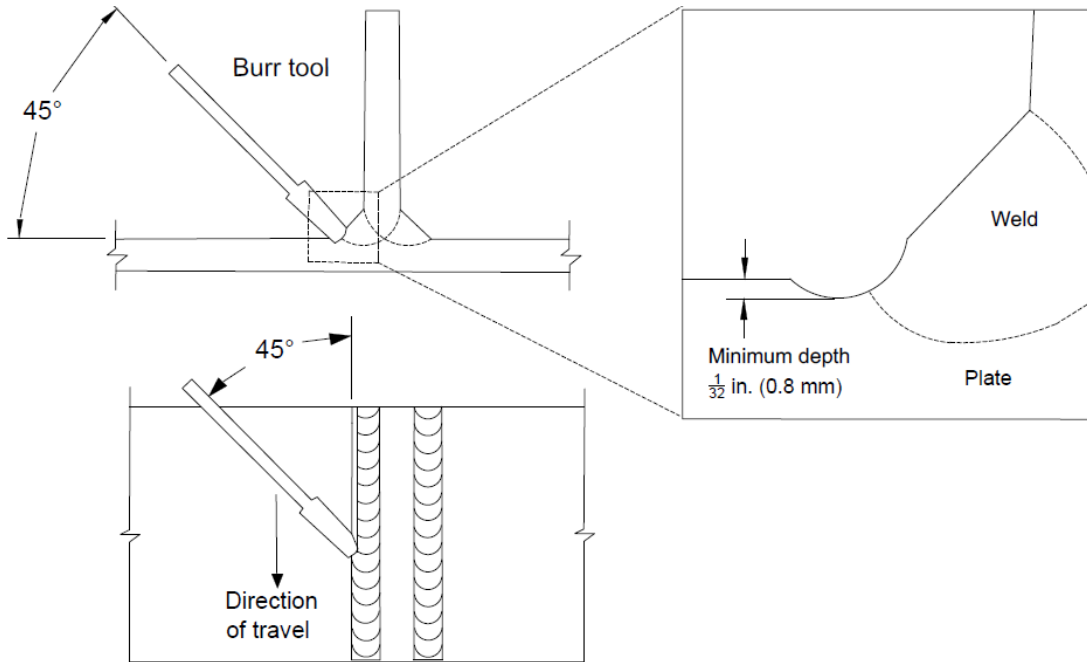


Figure 2-15. Position of the burr tip relative to material surface (Gregory et al. 1989)



Figure 2-16. Burr grinder and carbide burr tips (Dexter and Ocel 2013)

2.7.1.1.2 Gas Tungsten Arc (GTA) Remelting

GTA remelting helps reducing stress concentration at the weld toe. The GTA remelting is used for removing slag intrusions as well as to repair cracks up to approximately 3/16 in. deep (Dexter and Ocel 2013). During the process, a gas shielded tungsten electrode is moved along the weld toe at a steady speed in order to melt the targeted area of the weld without adding a new filler material. This retrofit technique requires highly skilled operators; hence, it is challenging and cost prohibitive to implement in the field.

2.7.1.2 Compressive Residual Stress Treatment

Impact treatment methods can be used to induce compressive residual stresses near a weld toe. Induced residual stresses are most effective when a weld toe is treated under dead load. Air hammer peening and ultrasonic impact treatment are the two methods that are currently used in the industry (Dexter and Ocel 2013).

2.7.1.2.1 Air Hammer Peening

In addition to treating weld toes, air hammer peening can be used to repair up to 1/8 in. deep surface cracks (Hausammann et al. 1983; Dexter and Ocel 2003). Figure 2-17 shows the positioning of the hammer for weld toe treatment and penetration depth range. Figure 2-18 shows a pneumatic hammer and a close-up view of the peening tip.

The recommended pressure for air hammer peening is 40 psi, and the number of passes is up to 6. The depth of penetration may range from 0.03 in. to 0.06 in. (Figure 2-17), where thinner sections may require less penetration. Light surface grinding to remove lap-type defects (that are introduced by the air hammer peening operation) is required for enhancing the performance of treated welds. Maintaining quality (or consistency of the depth of penetration), requires several passes to achieve the required penetration depth; while the noise produced by the hammer is one of a few concerns (Hacini et al. 2008).

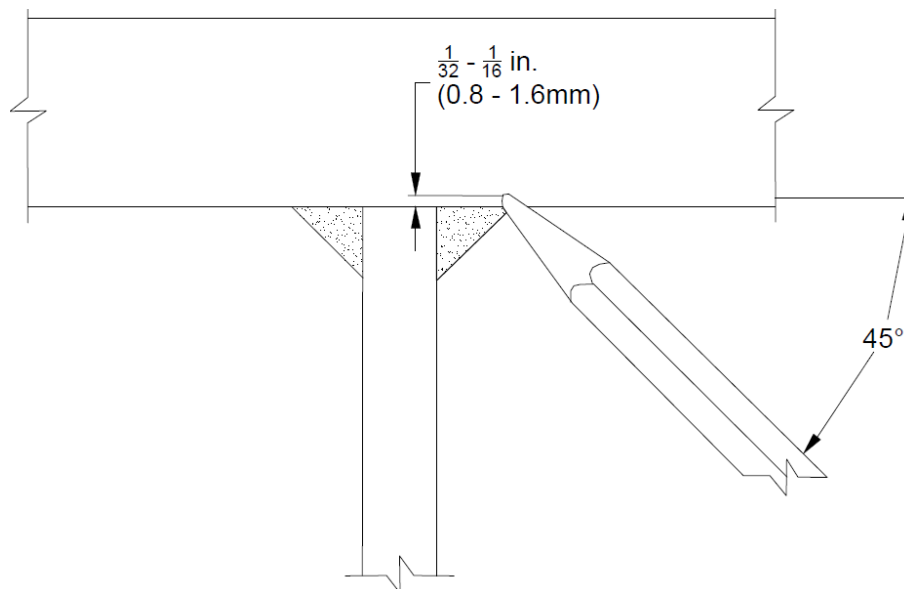
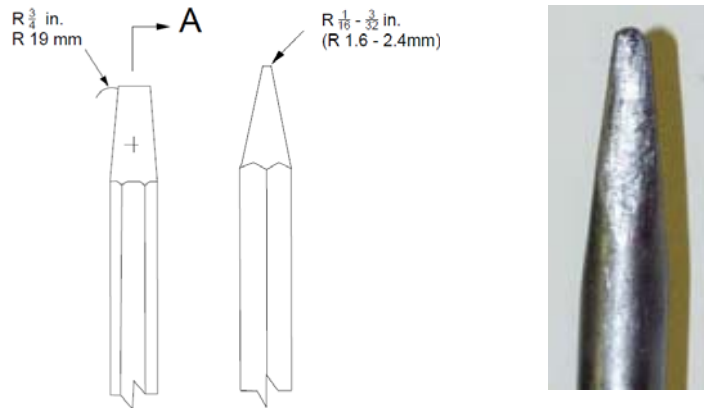


Figure 2-17. Hammer positioning and the depth of penetration (Hausammann et al. 1983)



(a) Pneumatic hammer/chisel



(b) Close-up view of the peening tip

Figure 2-18. (a) Pneumatic hammer/chisel and (b) close-up view of the peening tip (Hausammann et al. 1983)

2.7.1.2.2 Ultrasonic Impact Treatment (UIT)

Another method to induce compressive stresses by improving the weld toe profile is known as Ultrasonic Impact Treatment (UIT). It is commonly used as an alternative to hammer peening. According to several recent studies, this method is more effective, relatively quiet, and easier to handle (Fisher et al. 2001; Roy et al. 2003; GÜNTHER et al. 2005). Roy et al. (2003) show that the UIT can increase the fatigue strength of cover plate details and transverse stiffeners by at least one detail category. After compiling available test results, Yildirim and Marquis (2011) concluded that the welds treated with high frequency treatment methods (such as UIT) have slightly improved performance compared to the welds treated with traditional hammer peening. A detailed study conducted by Yekta et al. (2013) shows that the fatigue life of treated welds is improved regardless of under- or over-treating as per the current guidelines. The 2008 interim of AASHTO LRFD Construction Specifications (AASHTO 2008) section 11.9 included a detailed UIT procedure for weld toe treatment. UIT is a proprietary method; hence, it is prudent to follow manufacturer recommendations during special provisions development. Further, the UIT application will be more expensive compared to traditional methods, and may require a licensing agreement (Dexter and Ocel 2013).

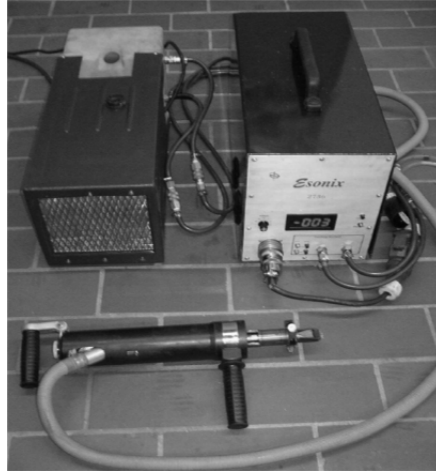
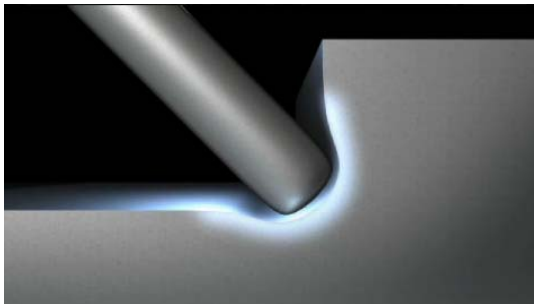


Figure 2-19. UIT equipment (GÜNTHER et al. 2005)



(a) Deformed shape of a treated surface



(b) Application at a weld

Figure 2-20. UIT (a) deformed shape of a treated surface and (b) application at a weld (Applied Ultrasonics 2014)

2.7.2 Repair of Through-Thickness Cracks

Through-thickness crack repair methods include hole drilling, cold expansion, vee-and-weld, post-tensioning and use of carbon fiber reinforced polymers.

2.7.2.1 Hole Drilling

The objective is to remove the sharp notch at a crack tip by placing a hole as shown in Figure 2-21; thus, it removes the stress concentration points. Effective implementation requires complete removal of a crack tip. The major challenge in successful implementation of this method is identifying the correct location of the crack tip prior to drilling (Li and Schultz 2005).

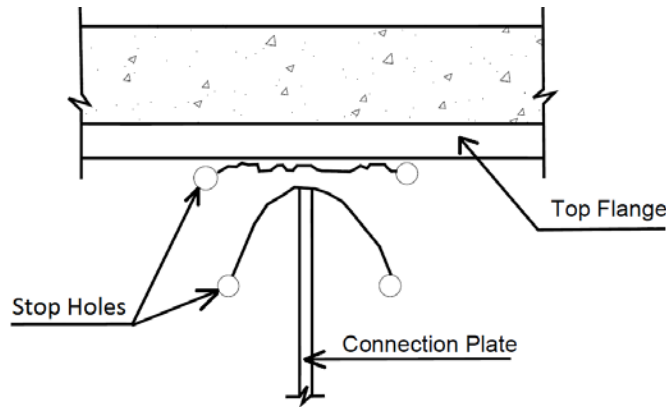


Figure 2-21. Crack arrest holes drilled at crack tips (Roddis et al. 2001)

Fisher et al. (1990) recommended the following equations for calculating crack-stop hole diameters (2ρ) using the stress intensity factor range (ΔK) and the yield strength (σ_y).

$$\frac{\Delta K}{\sqrt{\rho}} < 10.5 \sqrt{\sigma_y} \quad (\text{for } \sigma_y \text{ in MPa}) \quad (\text{Eq. 1})$$

$$\frac{\Delta K}{\sqrt{\rho}} < 4 \sqrt{\sigma_y} \quad (\text{for } \sigma_y \text{ in ksi})$$

For certain details, Eq. 1 may yield a hole diameter less than 1 in. In such cases, Dexter and Ocel (2013) recommend increasing the diameter by 25%. However, according to Gregory et al. (1989) the hole diameter depends on the plate thickness (Table 2-13). Satisfactory performance of hole diameters ranging from of 2 to 4 in. (50.8 to 101.6 mm) has been recorded (Dexter and Ocel 2013). For a hole size ranging from 1 to 1.5 in., the performance can be improved by introducing compressive stresses around the crack stop hole. This can be accomplished by placing a fully-tensioned, high strength bolt with hardened washers on both sides (Figure 2-22). Bolts and nuts are also used for aesthetics purposes. However, the use of bolts and nuts might hinder the crack growth inspection using traditional methods. In such instances, acoustic emission (AE) technology can be an alternative for monitoring performance of the repair. When bolts and nuts are not used, plugging crack stop holes is also recommended to avoid fecal or nesting material left from birds as well as to avoid unwarranted public attention to perforated bridge details. It is advised to drill crack stop holes and never to flame-cut (Dexter and Ocel 2013).

As an alternative to using bolts and nuts, a tapered mandrel that is slightly larger than a crack stop hole diameter can be forced through to develop residual stresses around the hole. This process is known as the cold expansion of a drilled hole and will be discussed in section 2.7.2.2.

Table 2-13. Crack Stop Hole Diameter Recommendation Based on Plate Thickness (Gregory et al. 1989)

Plate Thickness (in.)	Hole Diameter (in.)
up to $\frac{3}{8}$	$\frac{5}{8}$
$> \frac{3}{8} - \frac{3}{4}$	$\frac{3}{4}$
$> \frac{3}{4}$	$1\frac{3}{4}$



Figure 2-22. Drilled holes covered by bolts and nuts (Courtesy: Michigan DOT)

2.7.2.2 Cold Expansion

The standard procedure includes either pushing or pulling hardened components (e.g., mandrels, bearings, etc.) through a drilled hole (Leon 1998). The procedure initiates with an expansion of an undersized hole generating a radial plastic flow of material that introduces residual compressive stresses around the hole (Ozelton and Coyle 1986; Adams 2010; Stefanescu et al. 2004). The induced compressive stresses retard crack initiation and growth. Cold expansion is used as a complement with hole drilling to undersize hole diameter when a sufficient diameter cannot be achieved due to space limitations (Adams 2010).

2.7.2.3 Vee-and-Weld

The objective is to remove the entire crack and provide continuity using a weld. The procedure is presented in Figure 2-23.

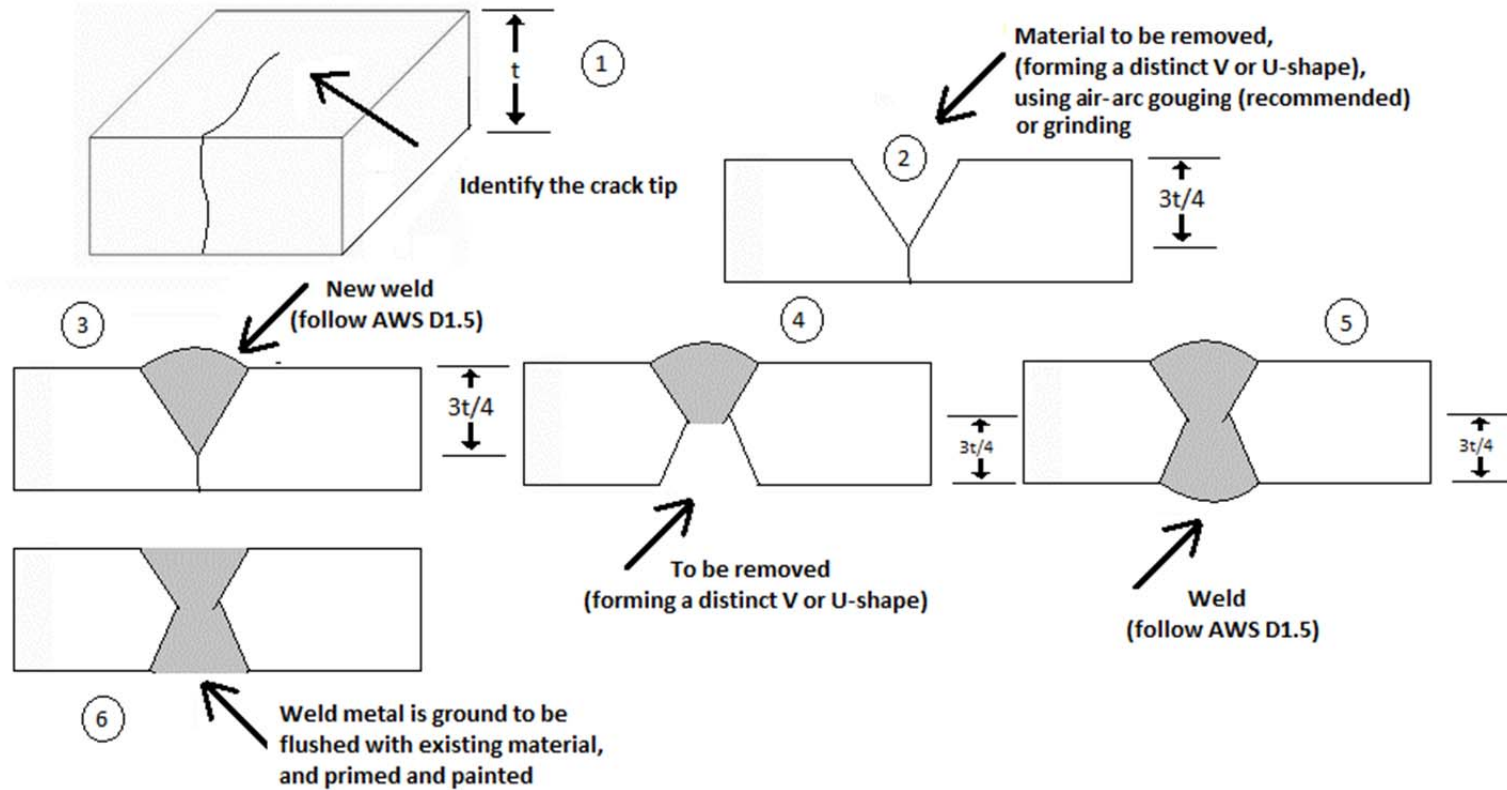


Figure 2-23. Vee-and-weld repair process

2.7.2.4 *Post-tensioning*

The objective is to keep the cracks closed while inducing compressive stress to abate fatigue crack growth. However, this method is not widely used and needs additional research.

2.7.2.5 *Carbon Fiber Reinforced Polymer Application*

The application of Carbon Fiber Reinforced Polymer (CFRP) patches for improving fatigue performance is new to the steel bridge industry (Bassetti et al. 2000). Research has shown that the CFRP patches can be used over existing cracks to reduce crack growth rate (Bassetti et al. 2000). The effectiveness of this retrofit method greatly depends on the quality and strength of the bond between patch and steel substrate. According to recent research, an adequate bond was developed by attaching the patch using a layer of epoxy with embedded breather cloth with a thickness of 1/8 in. (Alemdar et al. 2011). However, this method is not widely used and needs additional research.

2.7.3 Connection(s) or Global Structure Modification

The objective is to reduce the effective stress range in order to minimize the potential for fatigue cracking. This is achieved by adding doubler plates, smoothing out sharp corners, or modifying selected details such as diaphragm or cross-frame removal, diaphragm repositioning, and bolt loosening. A connection also can be modified by loosening the bolts. These modifications can decrease the stress ranges, but the connection or structural detail modification decision has to be well supported by structural analysis. Only the use of doubler/splice plates is discussed in this section. The other methods are discussed in section 2.7.4.

2.7.3.1 *Adding Doubler/Splice Plates*

The objective is to increase the cross-section in order to reduce stresses, provide continuity across a crack, or a combination thereof. It is often used as a complement to other repair methods to ensure adequate fatigue resistance to the stresses that contributed to the original cracking. Doublers can be attached by welding or using high-strength bolts. Use of bolted doublers, as shown in Figure 2-24, is recommended to enhance fatigue resistance. When the doublers are used to provide continuity across a crack, crack tips need to be removed by drilling holes – consider the information provided in section 2.7.2.1 on sizing a hole, protecting, and

monitoring performance. When welded doublers are used, a challenge often observed is to maintain the alignment of the cracked section before welding the plates. Another application of doublers is to restore the properties of a severely corroded section (Dexter and Ocel 2013).



Figure 2-24. Bolted doubler plate retrofit (the dashed line and the circle represent a crack and a drilled hole to remove the crack tip) (Dexter and Ocel 2013)

2.7.4 Repair Methods to Eliminate Out-of-Plane Distortion

Out-of-plane distortion at web gaps has led to fatigue cracking in steel bridges. The web gap is the unstiffened portion between the girder flange and the end of the stiffener that is connected to a cross-frame or a beam-type diaphragm. Design codes and specifications provide assumptions and do not describe the real behavior of the bridge or its components. The secondary stresses that are developed within the web gap due to out-of-plane distortion are often not accounted for during analysis and design. These secondary stresses are the primary cause of fatigue cracking. Hole drilling, diaphragm or cross-frame removal, diaphragm repositioning, bolt loosening, and web gap stiffening are the documented approaches for repair or retrofit of details that are susceptible to fatigue due to out-of-plane distortion.

2.7.4.1 Hole Drilling

The objective is to remove the crack tip; thus to eliminate or lower the crack propagation potential. A detailed discussion on hole drilling is presented in section 2.7.2.1. In a straight bridge, hole drilling is sufficient when the following conditions are satisfied (Li and Schultz 2005; Fisher et al. 1990);

- In-plane bending stress < 6 ksi (42 MPa)
- Out-of-plane stress range < 15 ksi (105 MPa)

At certain details, even if the aforementioned criterion is met, drilled holes alone are inadequate to arrest crack propagation. Hence, other retrofit methods need to be implemented in conjunction with the crack stop holes (Dexter and Ocel 2013).

Cracks due to out-of-plane distortion initiate from the welds. However, quantifying stress range at the weld toe has been a challenge. Section 2.3.4 presents a detailed discussion as well recommendations for calculating weld toe stress range due to out-of-plane distortion.

2.7.4.2 Diaphragm or Cross-Frame Removal

The objective is to remove or minimize the out-of-plane bending stress. The decision to remove diaphragms or cross-frames needs to be supported by detailed analyses to evaluate girder capacities, adequacy of alternate load paths, structural redundancy, and girder stability under all applicable loads such as live, wind, temperature gradient, and potential constraints to bridge expansion and contraction under uniform thermal loads. In addition, the effects of member removal on lateral-torsional buckling of compression flanges, girder stability during deck replacement, the additional cost to provide temporary bracings, and load distribution during a high load hit need to be evaluated. Further, it is vital to maintain an accurate record of any modification to the original as-built details within the current bridge management system.

2.7.4.3 Diaphragm Repositioning

The objective is to increase the flexibility of the detail. Limited applications of this approach are documented in literature. The diaphragm repositioning design needs to be adequately supported by detailed analyses. Any modification to the original as-built details needs to be accurately documented within the current bridge management system.

2.7.4.4 Bolt Loosening

Bolt loosening is a retrofit method that can be implemented as a stand-alone method or in conjunction with other methods. It is performed on the bolts that connect the bracing elements to the connection plates in the negative moment area. The method has been implemented on bridges with I-beam diaphragms, channel-type diaphragms, X-type cross-frames, and K-type cross-frames. The implementation records indicate a reduction of strain in or around the web gap by approximately 30-80% (Dexter and Ocel 2013; Tarries et al. 2002; Wipf et al. 2003).

However, the results depend on the bracing configuration, the number of loosened bolts, and the degree of restraint provided by each bolt. Hence, the decision needs to be supported by detailed analyses. A few recommendations are as follows:

- Evaluate bridge structural stability. When needed, provide bracings to prevent lateral torsional buckling (Tarries et al. 2002).
- Loosen the bolts sequentially until the desired strain reduction is obtained (Dexter and Ocel 2013).
- Consider lateral load distribution (Tarries et al. 2002).
- Secure bolts in place using lock nuts or double nuts to prevent from fall out due to vibration (Tarries et al. 2002; Dexter and Ocel 2013).

As documented in Dexter and Ocel (2013), the relative displacement between a girder flange and a connecting plate is small and ranges between 0.001 - 0.03 in. The bolt or rivet holes can be drilled to a size larger than the fasteners to be inserted in order to provide adequate flexibility to the web gap region; thus, leading to lower stresses.

2.7.4.5 Web-Gap Stiffening

The objective is to minimize the relative displacement between the connection plate and the girder flange in order to lower the secondary stresses due to out-of-plane distortion. Web-gap stiffening retrofit uses positive attachments (plates, angles or tee sections) connected by welds, bolts, a combination of welds and bolts (i.e., hybrid connection), adhesives or nails.

2.7.4.5.1 Welded Attachment

Connection plates that are welded to the top and bottom flanges are used to stop out-of-plane distortion between the girder flange and connection plate. The AASHTO LRFD section 6.6.1.3.1 recommends providing transverse welds on both the tension and compression flanges to mitigate out-of-plane distortion (AASHTO LRFD 2013). However, fatigue cracks have been developed at the new field welds leading to greater challenges in preventing fatigue cracks. This is mainly due to the challenges associated with maintaining weld quality in the field while providing an overhead weld at the plate to the top flange connection. The additional challenges to maintain field weld quality as well as to execute this retrofit method are listed below:

- High-strength steel is difficult to weld. If the girder flanges are made from A514 or T1 steel, welding is a challenge and this retrofit approach is undesirable.
- When the web gap is at the top girder flange that is embedded in a concrete deck, this retrofit method is undesirable because the concrete deck works as a heat sink making it difficult to maintain the proper preheat and interpass temperatures.
- Due to impact of live traffic on weld integrity, especially at the root, closure of lanes over and adjacent to the girder for root pass must be considered.
- The effect of heating girder flanges that are carrying deck dead load needs to be evaluated.

A few recommendations related to this retrofit method are listed below:

- Remove all contaminants and paint from the area before welding.
- If the dead load is considerably large, temporary shoring is needed to maintain bridge stability with the soften girder flanges under high temperatures.
- Welds need to be designed as per the current specifications. Also, the interaction of new welds with the existing welds needs to be avoided.
- Welds should have the same length as the length of the plate-flange connection. However, the weld should not wrap around the connection plate, nor intersect the longitudinal flange-web weld.
- In the case where there is a slide gap between the connection plate and flange, additional plates or spaces will be needed. The width of the spacer block should not be wider than 1.5 in. (38.1 mm).

2.7.4.5.2 Bolted Connection

Angles or tee sections are used for bolted connections between girder flanges and connection plates. This method is only applicable if the net section fracture does not control the strength of the member (Dexter and Ocel 2013). A bolted connection does not provide the same stiffness as a welded connection. The following minimum requirements must be met when bolted connections are established:

- A total of four bolts must be utilized across the flange when connecting angles or tee sections. Bolts need to be placed in two rows and two columns. The minimum thickness of the angles or tees needs to be 0.75 in. (19.1 mm).

- Tees need to be rolled sections because built-up sections have a tendency to crack.
- The haunch or a portion of the deck needs to be removed for bolting tees or angles to the flange when it is embedded into a concrete deck.
- Partial deck removal requires maintenance of traffic. Use of tapped holes into the girder flange is an option to prevent traffic disruption. However, it is recommended to give at least 1 in. embedment length to the bolt or stud in a tapped hole. Hence, this is only possible in girders with a thick flange.

2.7.4.5.3 Hybrid Connection

Hybrid connection, as the name implies, utilizes both welding and bolting to establish the connection. It includes bolting a thick plate to the girder flange and welding the connection plate to the already bolted thick plate (Figure 2-25). The plate must be bolted as specified in section 2.7.4.5.2 and have a minimum thickness of 0.875 in. (22.2 mm). The connection plate welding to the thick plate needs to be performed as specified in section 2.7.4.5.1.

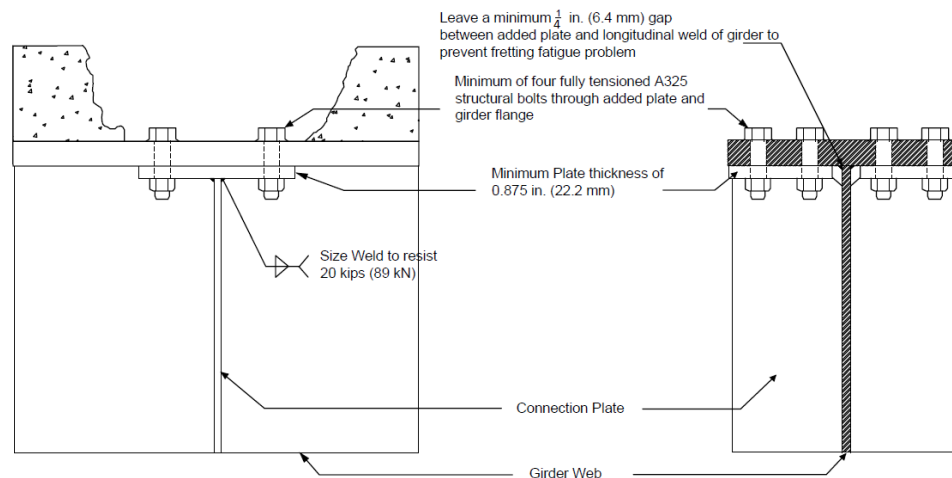


Figure 2-25. A typical hybrid connection detail (Dexter and Ocel 2013)

2.7.4.5.4 Adhesives

The use of adhesives is an option when partial deck removal or access to haunch is limited. Use of adhesives is currently at the experimental level. Hence, this method is not recommended as a long-term solution. Service life of an adhesive connection may be adequate for a year (Dexter and Ocel 2013). This method can be implemented as a temporary fix for a bridge with a deteriorated deck which is already in a program to be replaced. The adhesives used in such applications need to be suitable for field application, compatible with the substrate, and durable

under the specific exposure conditions. Further, small bolts can be used to prevent the angles or tees falling onto traffic, and also to hold the angles or tees at the final position until the adhesives are hardened (See Figure 2-26).

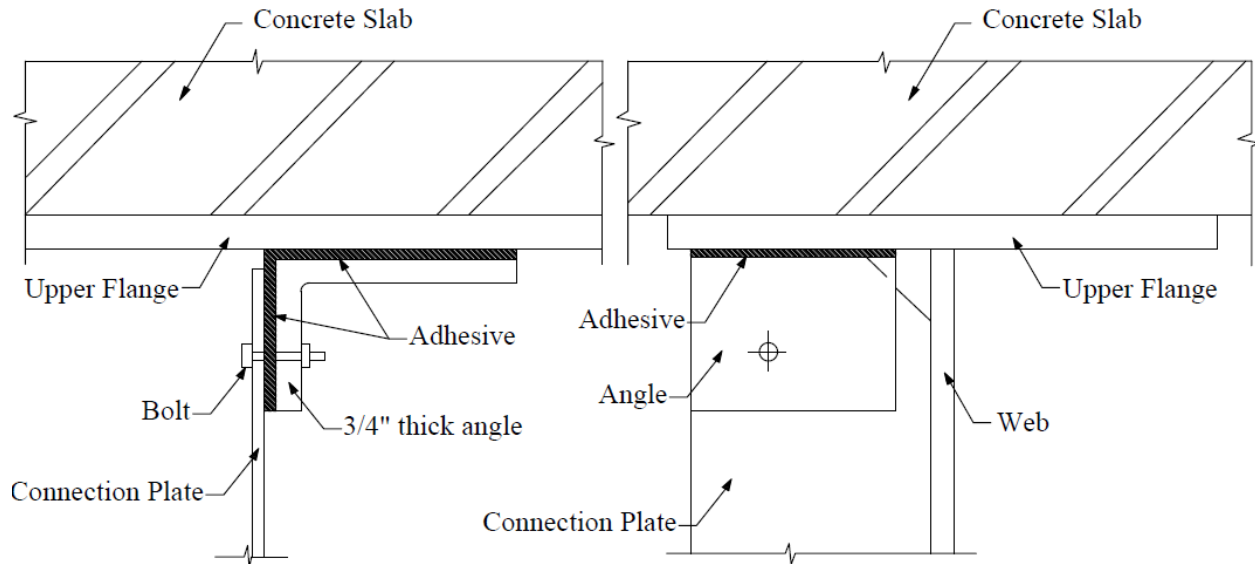


Figure 2-26. Proposed retrofit scheme with adhesives (Hu et al. 2006)

2.7.4.5.5 Nails

Use of nailed connections is also a new retrofit technique that requires additional investigations. The use of nails for this purpose has been experimentally tested, but not to failure (Dexter and Ocel 2013). It can be performed under traffic and does not require deck concrete removal. Further, this technique allows developing permanent connections. However, the use of nails with tee connections is recommended, but not with angles because of a possible misalignment on the opposite side (Dexter and Ocel 2013).

2.7.4.6 Web-Gap Softening

The objective is to increase flexibility at the web gap. This retrofit method includes removal of a connection plate section (which is stated in literature as gross material removal) or drilling large diameter holes.

2.7.4.6.1 Gross Material Removal

The process involves removal of a portion of the connection plate by flame cutting or any other means to increase web gap in order to reduce secondary stresses. All residual weld material from the girder web must be removed, ground smooth, and painted. It is recommended to limit

the new web gap dimension to the smallest of 15 in. or $1/6^{\text{th}}$ of the web depth. A limited number of applications, with inconsistent performance records, are documented in literature.

2.7.4.6.2 Large Hole Retrofit

This method includes drilling large diameter holes close to the web-gap area. The diameter equal to or greater than 3 in. (76 mm) is defined as a large hole. An example is shown in Figure 2-27. A few implementation examples are documented in literature. The effect of such a retrofit method on secondary stress reduction needs to be evaluated through refined analysis before implementation.

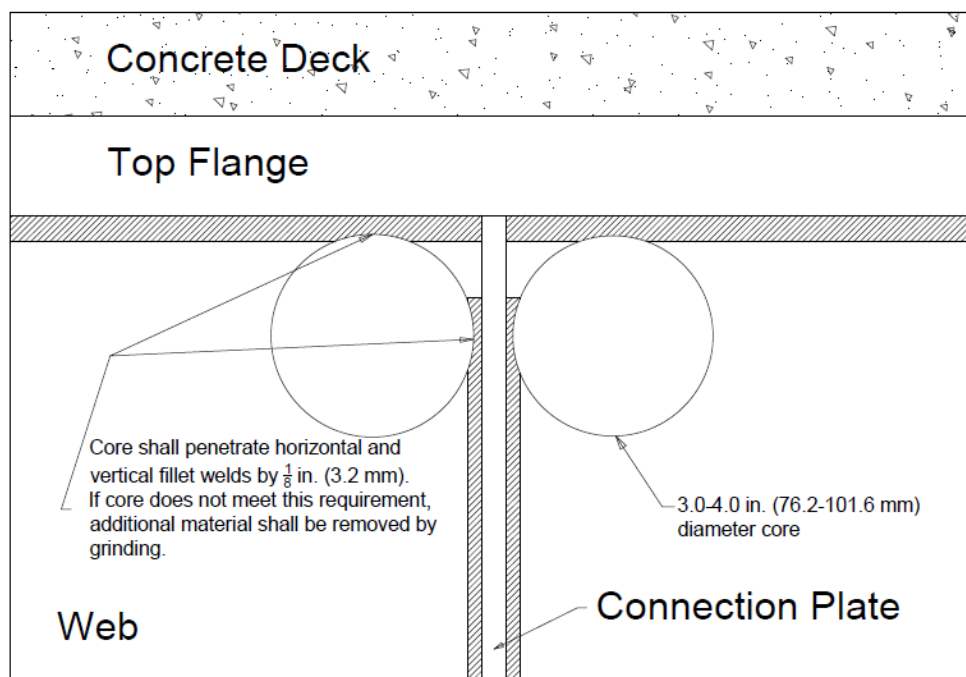


Figure 2-27. A large hole retrofit example (Dexter and Ocel 2013)

2.8 SUMMARY

To develop a successful program for monitoring the fatigue-sensitive detail, it is required to (i) accurately identify the fatigue-sensitive details, (ii) establish monitoring objectives, (iii) select technology suitable for achieving the monitoring objectives, and (iv) decide on the level and the monitoring duration. This chapter presents the fatigue-sensitive detail categories, methodologies to establish the needs for monitoring fatigue-sensitive details, technology for monitoring the fatigue-sensitive detail, and technology implementation considerations. A good understanding of the retrofit methods for fatigue-sensitive details is required to develop an appropriate monitoring

program for evaluating the effectiveness of the retrofits. Hence, retrofit methods for fatigue-sensitive details are also presented in this chapter.

An acoustic emission (AE) monitoring system with strain gages was recommended as one of the most effective technology for monitoring fatigue-sensitive details. AE has been successfully implemented in the field and evaluated for continuous monitoring of fatigue-sensitive details. At this time, AE is the only technology that is capable of real-time monitoring of fatigue events and providing data for damage location detection. In addition to the AE sensors, strain gauges are required to evaluate the stress state to calculate remaining fatigue life. Strain data is also needed to support AE data analysis by developing the load matrix.

This chapter presents a cluster analysis approach for developing a monitoring program for bridges with fatigue-sensitive details. Fatigue-sensitive details can be identified using the detail categories presented in the AASHTO LRFD Bridge Design Specifications (2013) Table 6.6.1.2.3-1 or through a combined effort of utilizing information in bridge files, refined analysis, and experience. The weld toe stresses are calculated using the hot spot stress (HSS) method. Structural modeling and HSS calculation guidelines are presented in this chapter to help evaluating the fatigue-sensitive welded details. Finally, the technology implementation considerations such as level of monitoring, duration of monitoring, and inspection frequency are presented.

3 STRUCTURAL MODELING AND HOT SPOT STRESS ANALYSIS

3.1 OVERVIEW

The scope of the work documented in this chapter includes selecting a bridge with fatigue-sensitive details, documenting as-built details to develop analysis models, evaluating the structural behavior and girder and diaphragm deformation under specific loads and load patterns, and selecting details for instrumentation and monitoring.

The bridge selected for this study contained partial depth diaphragms with welded details. Hot spot stresses were calculated in order to identify the critical details for monitoring. Hot spot stress calculation requires selecting appropriate element types, mesh parameters, and reference points for stress calculation. Therefore, this chapter presents information on (a) element types and mesh parameters for hot spot stress calculation, and (b) bridge structural analysis for identification of details for instrumentation and monitoring.

3.2 BRIDGE LOCATION AND DETAILS

The bridge (S16 of 11015) is located in Stevensville, Michigan, and carries I-94 over Puetz Road (Figure 3-1). The bridge was built in 1961. In 1997, MDOT performed girder end retrofits. There are two parallel bridges to carry westbound (WB) and eastbound (EB) traffic on I-94. After reviewing MDOT biennial inspection reports and conducting a field visit to document the bridge superstructure and substructure condition, the I-94 EB bridge was selected. The longest span (span 3) of the EB Bridge is 56 ft - 6 in. and has a 54.5° skew (Figure 3-2). The span is supported on an integral abutment and a pier with expansion bearings (Figure 3-3). The superstructure consists of 12 steel I-girders (10-W30×108 and 2-W30×99) and a 9 in. thick cast-in-place concrete deck. The girders are connected transversely using intermediate and end diaphragms (Figure 3-4). The partial depth diaphragm connection detail is classified as a category C' fatigue-sensitive detail (AASHTO LRFD 2013). As shown in Figure 3-3, partial length welded cover plates are attached to girder bottom flanges. This detail is classified as a category E fatigue-sensitive detail because the girder flange width is less than 0.8 in. (AASHTO LRFD 2013).

The bridge has been designed following the 1958 Edition of the Specifications for Design of Highway Bridges, Michigan State Highway Department (MDOT 1958). This is several years earlier than the first fatigue design provisions of the AASHTO Standard Specifications for Highway Bridges appeared; that is in 1965. At that time, details were not specifically designed for fatigue. However, Article 8.02h of MDOT (1958), acknowledges developing alternating stress due to the combined effect of dead, live, impact, and centrifugal stresses. In order to accommodate the effect of cyclic nature of stresses, the specifications states “*If alternating stresses occur in succession during one passage of the live load, each shall be increased by 50% of the smaller.*”



Figure 3-1. (a) Bridge location and (b) aerial view of the EB and WB bridges (Google)

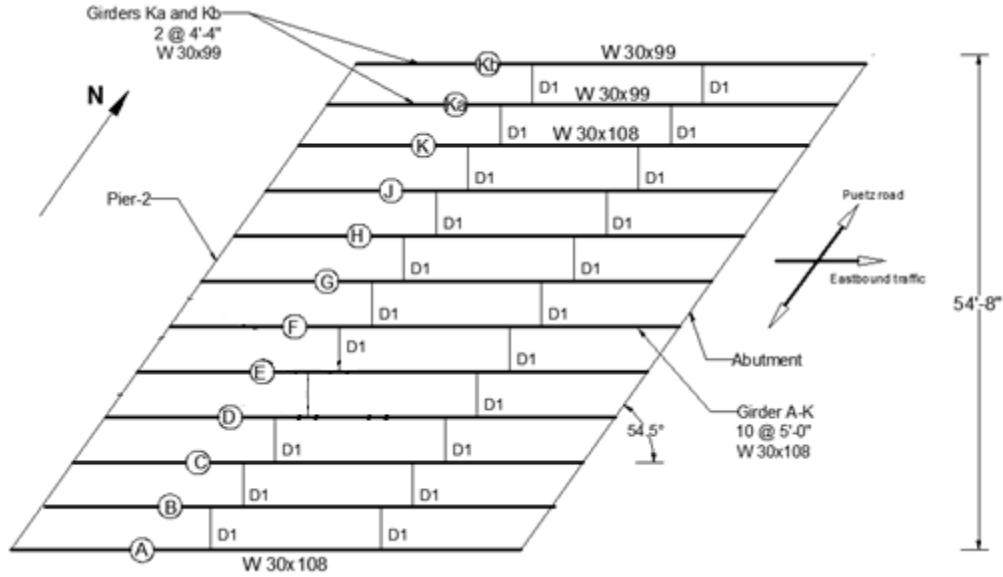


Figure 3-2. Girder and diaphragm layout of the EB bridge span 3

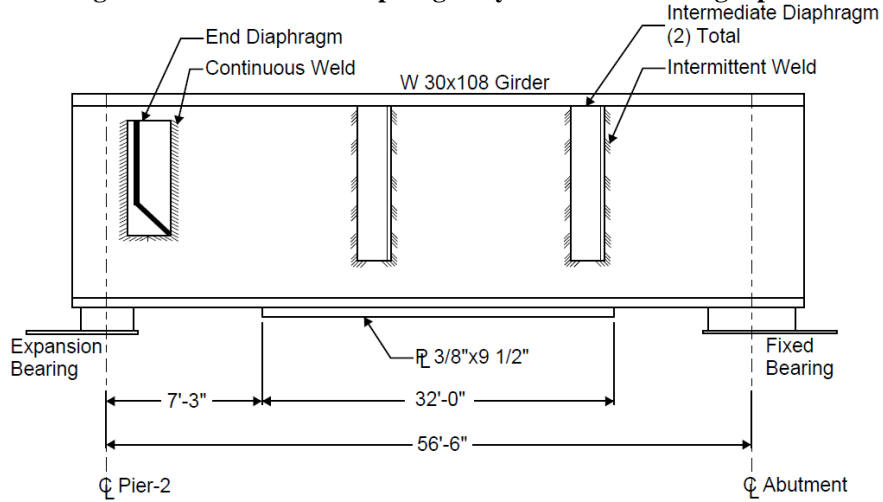


Figure 3-3. Typical girder elevation showing the partial depth diaphragms

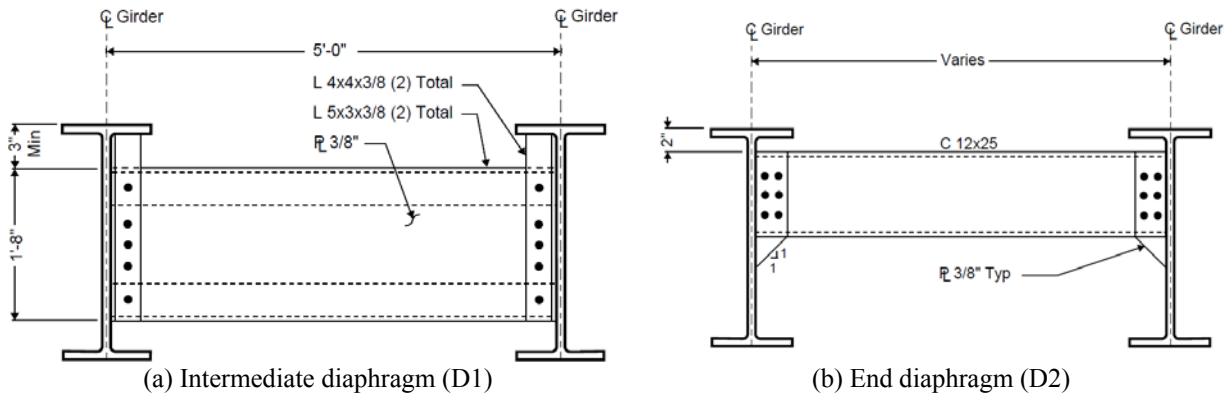


Figure 3-4. Typical diaphragm detail

3.3 SITE SELECTION

This I-94 EB bridge was selected for structural health monitoring (SHM) system installation. In addition to having fatigue-sensitive details and staggered diaphragms, the other reasons for selecting the bridge include heavy truck traffic and easy access to the bridge for instrumentation and system maintenance. Since the primary objective of this research project is to evaluate technology for fatigue-sensitive detail monitoring, and this is the first time such a monitoring system is implemented in Michigan and powered by solar panels, bridge access was a major consideration when selecting the bridge.

3.4 AS-BUILT DETAIL DOCUMENTATION

The EB and WB bridges were labeled as *South Bridge* and *North Bridge*, respectively. After reviewing maintenance records, *span 3* of the I-94 EB bridge was selected for documentation of as-built details and in-service condition using Leica C-10 laser scanner. Figure 3-5 shows an aerial view and a snap shot of *span 3*. Visual inspection was performed, and digital images were also taken. The information collected through visual inspection and digital images was necessary for the reverse engineering process of the bridge details. Figure 3-6 shows the reverse engineered model.



(a) Aerial view of I-94 over Puetz Road in Stevensville, Michigan



(b) Span 3 of I-94 EB over Puetz Road

Figure 3-5. (a) Aerial view and (b) span 3 of the I-94 EB bridge



(a) Digital image (b) Modeled geometry
Figure 3-6. Girders and intermediate diaphragms

3.4.1 Comparison of As-Built and As-Designed Details

One of the primary objectives of this work was to compare the as-built details with the as-built plan dimensions. The girder and diaphragm spacing were calculated from the modeled bridge geometry and compared to the plan dimensions. Also, web gap dimensions were calculated from the model.

3.4.1.1 Girder Spacing

The bridge was originally constructed with only 10 girders at 5 ft spacing and was later widened by adding two girders with a spacing of 4 ft 4 in. (Figure 3-2). Using the scanned geometry and *Measurement* tool in *Cyclone*, as-built spacing of the girders was calculated. Table 3-1 lists the as-designed and as-built spacing dimensions. As shown in Table 3-1, girder spacing matches with as-designed spacing except at one location where a maximum of 0.6 in. deviation was observed.

Table 3-1. As-Designed and As-Built Girder Spacing

Girder Spacing	As-Design Spacing (ft:in)	As-Built Spacing (ft:in)
G1	4 ft 4 in	4 ft 3.8 in
G2	4 ft 4 in	4 ft 4.0 in
G3	5 ft 0 in	4 ft 11.9 in
G4	5 ft 0 in	4 ft 11.9 in
G5	5 ft 0 in	4 ft 11.8 in
G6	5 ft 0 in	4 ft 11.6 in
G7	5 ft 0 in	4 ft 11.9 in
G8	5 ft 0 in	4 ft 11.8 in
G9	5 ft 0 in	5 ft 0.0 in
G10	5 ft 0 in	5 ft 0.1 in
G11	5 ft 0 in	4 ft 11.4 in

3.4.1.2 Intermediate Diaphragm Spacing

The intermediate diaphragm spacing calculated from the model was 18 ft 3.8 in. (Figure 3-7). The diaphragm spacing from the design plans were not specified, but the plans stated that the diaphragm spacing is $\frac{1}{3}$ the span length. The span was given as 55 ft $4\frac{5}{8}$ in., which gives a diaphragm spacing of 18 ft. 5.5 in. Hence, the difference between as-designed and as-built diaphragm spacing is 1.7 in.

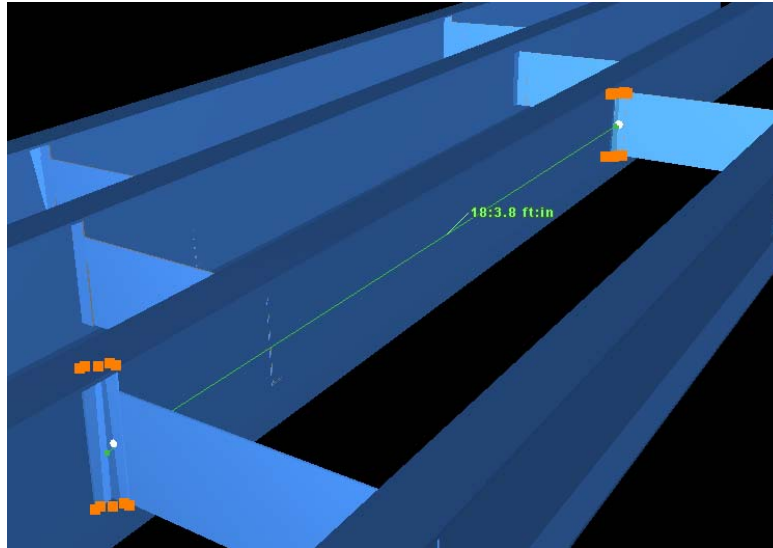
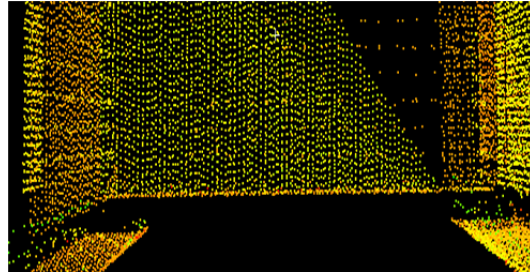
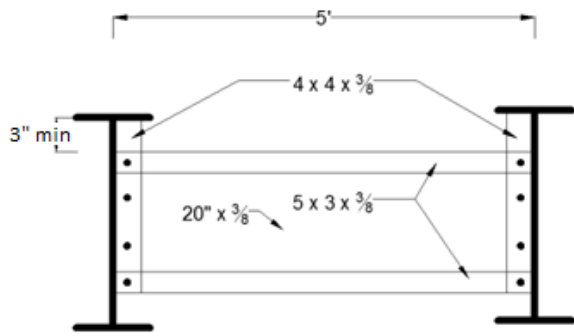


Figure 3-7. Diaphragm spacing calculated from the model

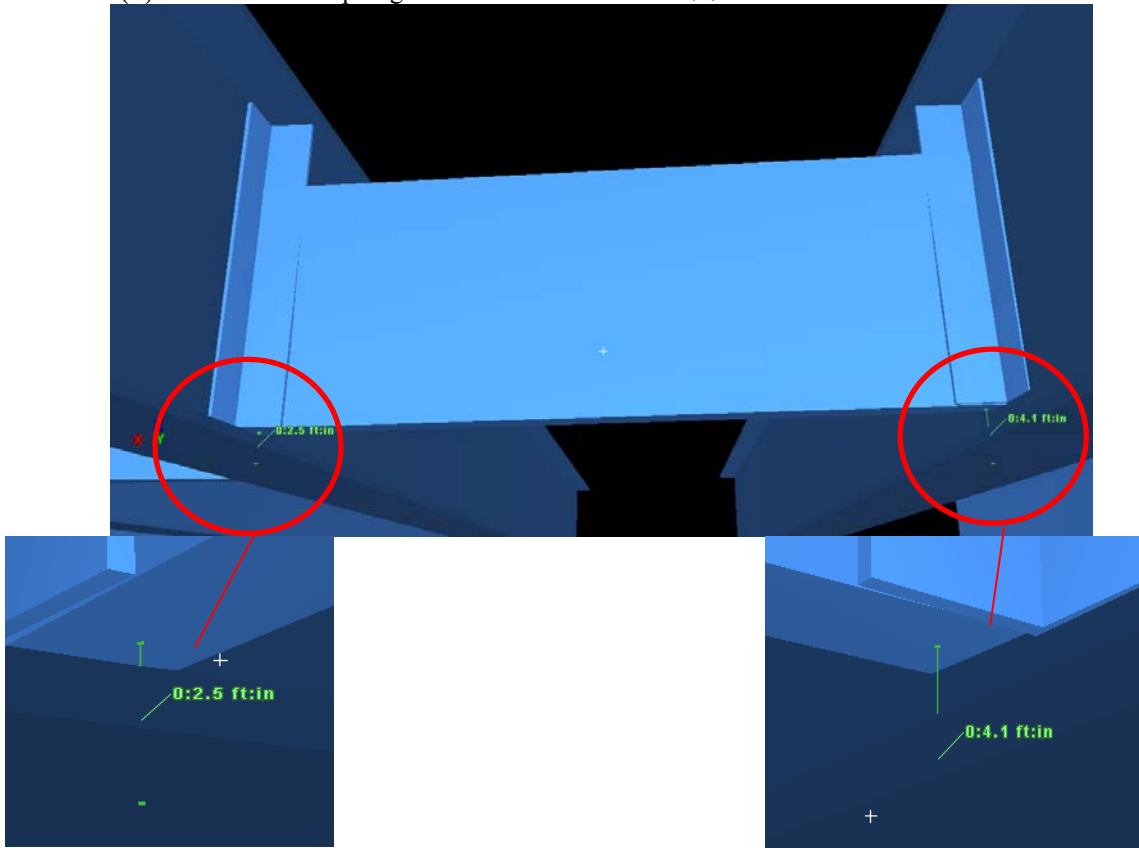
3.4.1.3 Diaphragm Elevation

As per the bridge plans, diaphragms were supposed to be located at a minimum of 3 in. below the girder top flange (Figure 3-8a). Knowing exact dimensions at the web gaps is important for structural modeling and analysis. Hence, the diaphragm elevations, with respect to the bottom surface of girder bottom flange, were measured. As shown in Figure 3-8c, the diaphragm is located 2.5 in. and 4.1 in. from girder 6 and 7 bottom surfaces, respectively. The bottom flange thickness of girders A to K (W30×108) is 0.76 in. In addition, these girders have $\frac{3}{8}$ in. cover plates. Hence, the web gap was calculated by subtracting 1.135 in. from the distances calculated using the point cloud data (Table 3-2). Girders K_a and K_b are W30×99. Hence, appropriate flange thicknesses and cover plate thicknesses were considered in calculating the web gap.



(a) Intermediate Diaphragm Detail

(b) Intermediate diaphragm point cloud



(c) Diaphragm elevation measured from girder bottom flange

Figure 3-8. Intermediate diaphragm detail and elevations at girder F and G

Table 3-2. Web Gap Dimensions

Diaphragm	Girder	Web gap (in.)	
		Diaphragm closer to the abutment	Diaphragm closer to the pier
1	A	3.065	3.765
	B	1.065	1.865
2	B	3.065	3.765
	C	0.565	1.165
3	C	3.065	3.665
	D	0.465	1.265
4	D	3.165	3.765
	E	0.465	1.665
5	E	3.065	3.665
	F	1.165	1.165
6	F	2.965	3.565
	G	1.365	2.065
7	G	3.265	3.565
	H	2.065	2.765
8	H	2.865	3.465
	J	2.665	3.165
9	J	3.165	3.465
	K	2.665	3.265
10	K	2.865	3.465
	K _a	3.290	3.490
11	K _a	3.390	3.090
	K _b	3.390	3.490

3.5 ELEMENT TYPES AND MESH PARAMETERS FOR HOT SPOT STRESS CALCULATION

Section 2.3.4 presents finite element modeling guidelines and hot spot stress calculation methods. This section details the evaluation process and results of those guidelines as well as the recommendations for modeling, analysis, and results interpretation with regards to hot spot stress calculation.

Lee et al. (2010) used two specimens to perform tension and flexural tests. Bhargava (2010) also performed a tension test evaluating various parameters such as weld geometries and longitudinal length of gusset plates. The specimen geometry, loading, boundary conditions, and experimental results presented in Lee et al. (2010) were used to evaluate the guidelines and recommendations given in literature for FE modeling, analysis, and calculation of HSS at a weld toe. In this evaluation, Abaqus CAE was used as the pre- and post-processor while Abaqus standard was the FE solver. 3D FE models of both specimens were developed using solid elements.

Figure 3-9 shows the specimen geometry. Both specimens represent two welded plates. The Type 2 detail represents a typical partial depth diaphragm. Plates were made from ship structural steel grade A with a modulus of elasticity of 29,000 ksi (200 GPa) and Poisson's ratio of 0.3. Plate thickness is 0.4 in. (10 mm). A forty five (45) degree fillet weld with a leg length of 0.2 in. (5 mm) is used in the model. Plate length and width are 11.8 in. (300 mm) and 2.8 in. (70 mm), respectively. Specimen grip width is 3.5 in. (90 mm). Plate height is 2 in. (50 mm). Figure 3-10 shows the loading and boundary conditions. Figure 3-10a and b show the boundary conditions and loads applied on specimen types 1 and 2 during the tension test. Figure 3-10c and d show the boundary conditions and loads applied on specimen types 1 and 2 during flexure test.

Specimens with fine and coarse meshes were used. Lee et al. (2010) used an element size of $0.5t$ as a fine mesh and t as a coarse mesh; where t is the plate thickness. According to Hobbacher (2008), $0.4t$ is considered as a fine mesh while $0.5t$ and larger are considered as coarse meshes. As stated in Section 2.3.4, FE analysis results are sensitive to element types and mesh discretization within regions of high strain gradient. Element type and mesh recommendations in Lee et al. (2010) and Bhargava (2010) were evaluated because these two references encompass most of the recommendations given in literature for HSS analysis (refer to Table 2-3 in chapter 2).

Lee et al. (2010) suggested element types of C3D8 and C3D20 (Abaqus 2014) and mesh configurations with $0.5(t \times t)$ and $t \times t$ element sizes. Following the recommendations by Bhargava (2010), C3D20R was also used with an element size of $(t \times t)$. Linear and quadratic extrapolation techniques were utilized following the guidelines for the Type A HSS method presented in Table 2-2. The maximum principal stress was probed from nodes at exact reference points. The maximum principal stress contours and deformed shapes of the specimen type 1 and 2 are shown in Figure 3-11a-d.

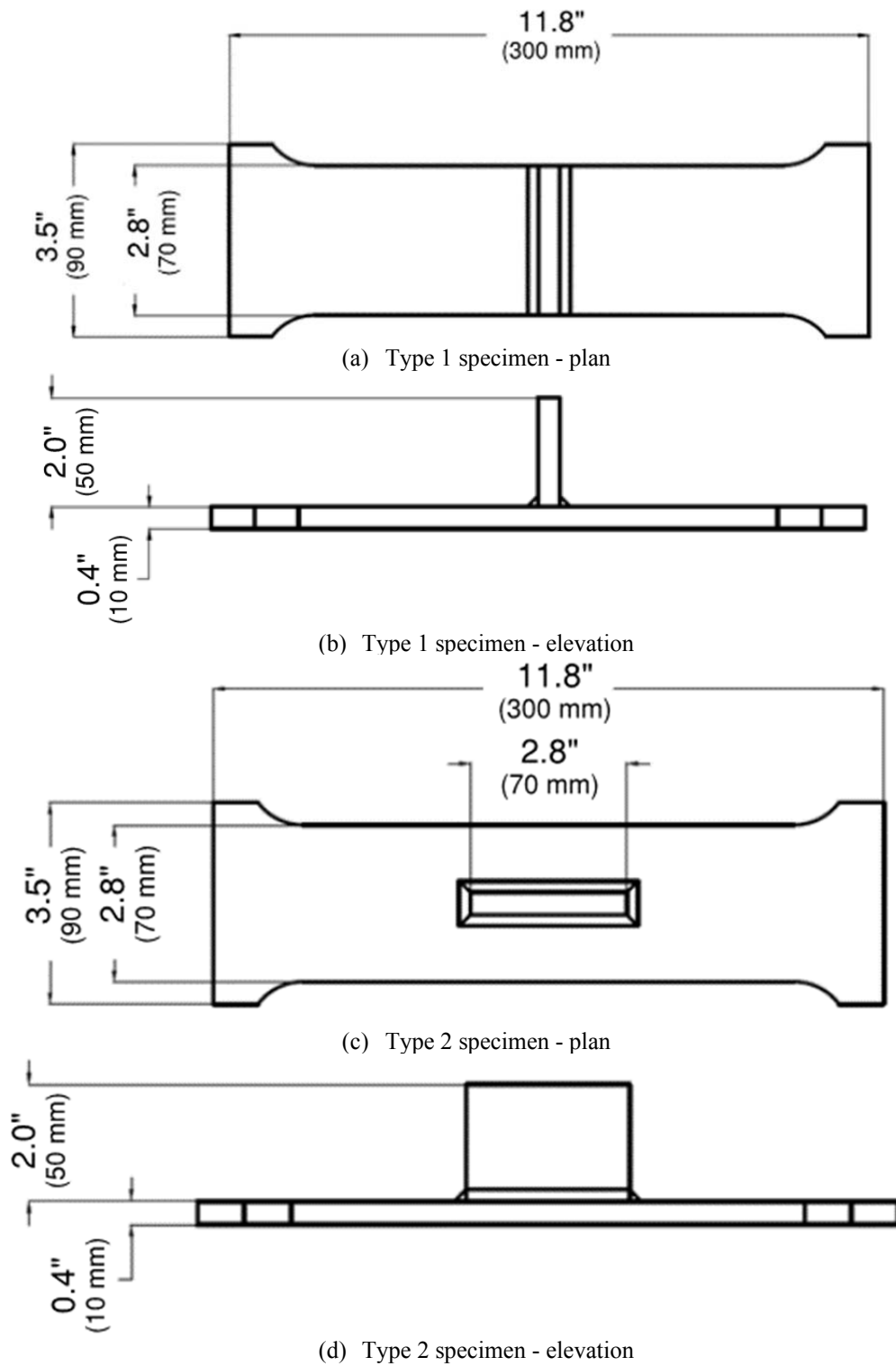


Figure 3-9. Specimen 1 and 2 geometry

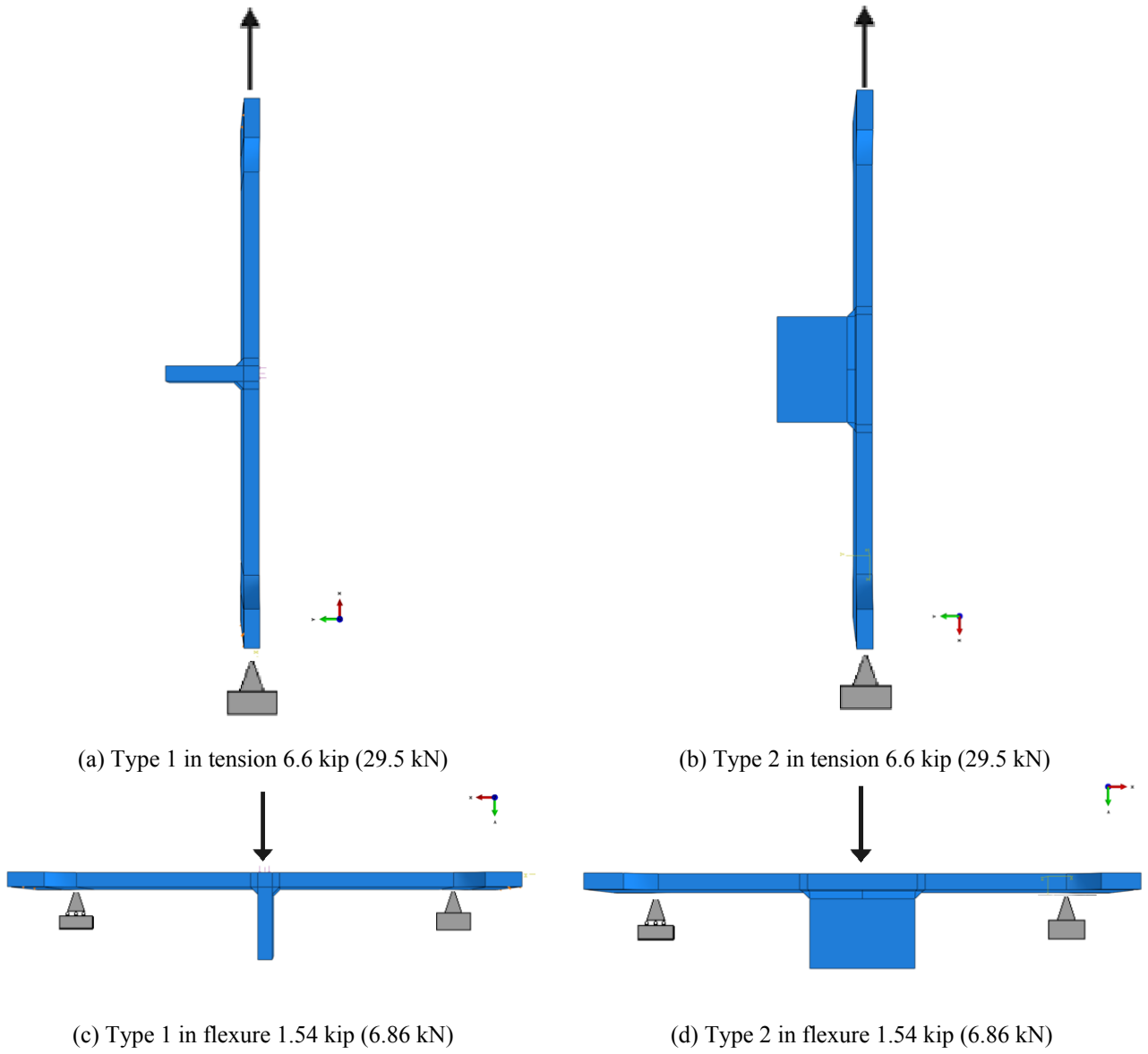


Figure 3-10. Specimen loading and boundary conditions

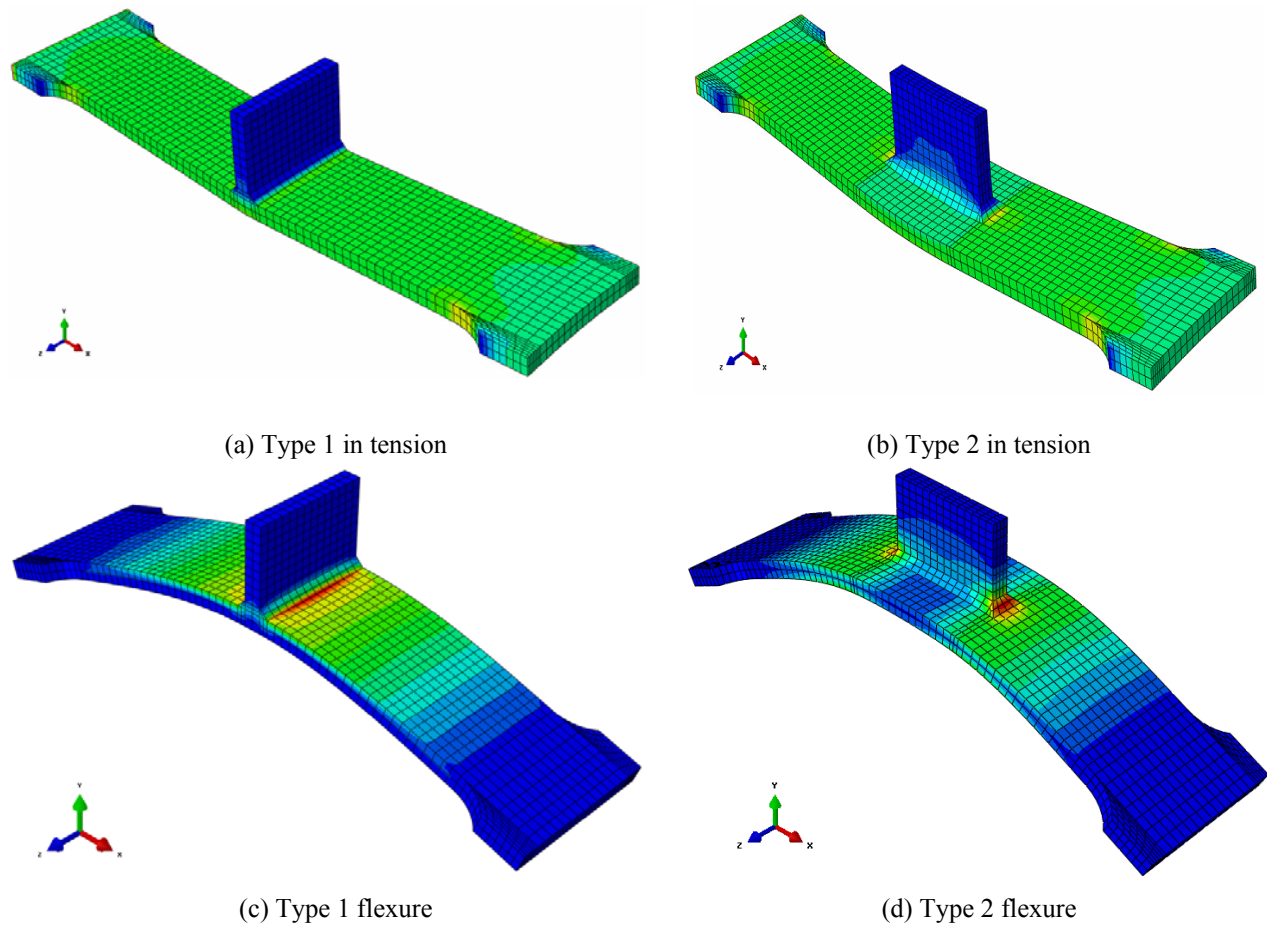


Figure 3-11. The maximum principal stress contours under tension and flexure

Hot spot stress (HSS) calculated from FE analysis was compared with experimental and other published results in literature (Table 3-3). The first column of the table specifies the specimen type. Column 2 presents FE model parameters such as mesh (F-fine or C-coarse), nodes per element, and element size as a fraction of plate thickness (t). The next two columns include HSS calculated using linear and quadratic equations presented in Hobbacher (2008) (Refer to Table 2-2 for details.) as well as the experimental results documented in Lee et al. (2010). The last column shows percentage difference between experimental and FE analysis results. The comparison was made using only the linear extrapolation results because the quadratic extrapolation results of experimental data were not available in Lee et al. (2010).

Bhargava (2010) recommended FE modeling, analysis, and results interpretation guidelines based on experimental and finite element analysis results of specimens that were subjected to axial tension. These recommendations were evaluated using the specimen geometry, loads,

boundary conditions, and the experimental results presented in Lee et al. (2010). Table 3-4 presents an evaluation of both specimens in tension. The load applied on the specimen is 6.6 kips (29.4 kN). The first column shows the specimen type while the second column shows mesh refinement, number of nodes per element and the element size. The third, fourth, and fifth columns show the FE analysis results, experimental results, and the percent difference between FE and experimental results, respectively. As per the results shown in Table 3-3 and Table 3-4, the following conclusions can be derived;

- 1) Use of finer mesh, $0.25(t \times t)$, compared to $0.5(t \times t)$ mesh with 8-node linear brick elements and linear extrapolation to calculate HSS does not increase the accuracy of the analysis results of Type 1 and 2 details under tension.
- 2) When Type 1 and 2 details are simulated under tension, the use of $0.25(t \times t)$ mesh with 20-node quadratic brick elements and linear extrapolation to calculate HSS increases error by at least 10% compared to the results obtained from a $0.5(t \times t)$ mesh.
- 3) When the Type 1 detail is simulated under tension, use of a $0.25(t \times t)$ mesh with 8-node linear brick, 20-node quadratic brick, or a $t \times t$ mesh with 20-node quadratic brick reduced integration elements and linear extrapolation to calculate HSS leads to about a 20% error compared to the experimental results.
- 4) When Type 2 detail is simulated under tension, the use of a $0.25(t \times t)$ mesh with 8-node linear brick, a 20-node quadratic brick, or a $t \times t$ mesh with 20-node quadratic brick reduced integration elements and linear extrapolation to calculate HSS leads to an about 12% error compared to the experimental results.
- 5) When Type 1 and 2 details are simulated under tension, the use of a $0.5(t \times t)$ mesh with 20-node quadratic brick elements gives the least amount of errors (about 12% and 2%, respectively) compared to other mesh configurations and element types. Hence, the use of a $0.5(t \times t)$ mesh with 20-node quadratic brick elements is recommended for simulating Type 1 and 2 details under tension.
- 6) When Type 1 detail is simulated under flexure, the use of a $t \times t$ mesh with 20-node quadratic brick elements or a $0.5(t \times t)$ mesh with 20-node quadratic brick elements leads to almost identical error percentages (i.e., about 2% and 3%, respectively). However, the stress

calculated with a 0.5(t×t) mesh with 20-node quadratic brick elements is 3% greater than the experimental results, and it is recommended for fatigue life calculation.

- 7) When Type 2 detail is simulated under flexure, the use of a t×t mesh with 20-node quadratic brick elements or a 0.5(t×t) mesh with 20-node quadratic brick elements leads to identical results (i.e., about 0% and 1% error, respectively). Both approaches give higher stresses than the experimental values. Comparing simulation results of Type 1 and 2 details, use of a 0.5(t×t) mesh with 20-node quadratic brick elements is recommended for fatigue life calculation.
- 8) As shown in Table 3-3, the use of 0.5 (t×t) mesh with 20-node quadratic brick elements and quadratic extrapolation to calculate HSS yields about 10 to 15% greater stresses at the hot spot compared to using the linear extrapolation function. These estimates are conservative for fatigue evaluation and recommended to use until additional experimental data proves otherwise.

Table 3-3. HSS Calculated Using FE Models of Type 1 and 2 Specimens

Specimen Type	FE Model - Solid Element			HSS, ksi (MPa)			Percent Difference $ (b-a_1)/b \times 100 $
				Extrapolated Analysis Results (a)		Extrapolated Experimental Results (b)	
	Mesh	Nodes	Size	Linear (a ₁)	Quad. (a ₂)	Linear	
Tension Test with a 6.6 kip (29.4 kN) Force							
1	F	8	0.5(t×t×t)	6.3 (44)	6.5 (45)	7.6 (52)	17%
	F	20	0.5(t×t×t)	6.7 (46)	7.1 (49)		12%
	C	20	t×t×t	6.5 (45)	-		15%
2	F	8	0.5(t×t×t)	7.9 (54)	8.3 (57)	8.8 (60)	10%
	F	20	0.5(t×t×t)	8.6 (59)	9.3 (64)		2%
	C	20	t×t×t	8.1 (56)	-		8%
3-Point Bending with a 1.54 kip (6.86 kN) Force							
1	F	8	0.5(t×t×t)	45.0 (310)	47.4 (327)	59.5 (411)	24%
	F	20	0.5(t×t×t)	61.2 (422)	69.9 (482)		3%
	C	20	t×t×t	58.5 (403)	-		2%
2	F	8	0.5(t×t×t)	50.3 (347)	56.1 (387)	63.1 (435)	20%
	F	20	0.5(t×t×t)	63.4(437)	70.9 (489)		0%
	C	20	t×t×t	63.7 (439)	-		1%

Table 3-4. HSS Calculated Using Bhargava (2010) Recommendations - Type 1 and 2 Specimens under Tension

Specimen Type	FE Model - Solid Element			HSS, ksi (MPa)		Percent Difference $ (b-a)/b \times 100 $
				Extrapolated Analysis Results (a)	Extrapolated Experimental Results (b)	
	Mesh	Nodes	Size	Linear	Linear	
1	F	8	0.25(t×t)	6.2 (43)	7.6 (52)	18%
	F	20	0.25(t×t)	5.9 (41)		22%
	C	20R	t×t	6.1 (42)		20%
2	F	8	0.25(t×t)	7.7 (53)	8.8 (60)	12%
	F	20	0.25(t×t)	7.5 (52)		14%
	C	20R	t×t	7.8 (54)		11%

3.6 BRIDGE STRUCTURAL MODELING AND HOT SPOT STRESS ANALYSIS

The girder and diaphragm layout, a girder geometry, and diaphragm configurations are shown in section 3.2. The objective of this section is to identify a few critical details for instrumentation. This was accomplished by evaluating stresses at cover plate ends and partial depth diaphragms. A finite element model of the bridge was developed using the dimensions documented in section 3.2 and 3.3 and the as-built plans.

3.6.1 Material Properties

Table 3-5 shows the material properties used in the finite element model. The bridge deck concrete modulus was reduced to account the current deck condition.

Table 3-5. Bridge Material Properties

Material	Material Property		
	Unit Weight (lbs/ft ³)	Modulus of Elasticity (ksi)	Poisson's Ratio
Concrete	150	4,000	0.2
Steel	490	29,000	0.3

3.6.2 Loads

Analysis was performed under deck and barrier self weight and live loads. The fatigue truck given in the MBE (2011) is a notional configuration. It was decided to use more realistic loads and vehicle configurations for the analysis to identify the fatigue-sensitive details for instrumentation. Hence, Michigan Legal Vehicle configurations with designated loads and an impact factor of 0.3 were used as live loads. These vehicle configurations are provided in the

Michigan Bridge Analysis Guide (MDOT 2009) and represent the typical trucks travelling over the bridge. The bridge carries 48,500 vehicles daily, with 30% of the vehicles being trucks. Of the trucks, 84.6% travel in the outer lane daily (Figure 3-12). Therefore, preliminary analysis was performed by placing the legal loads on the outer lane (Figure 3-13). After the preliminary analysis, truck number 17 (Figure 3-14) was identified as the critical load for HSS at the web gap welds.



Figure 3-12. Typical truck traffic on the bridge

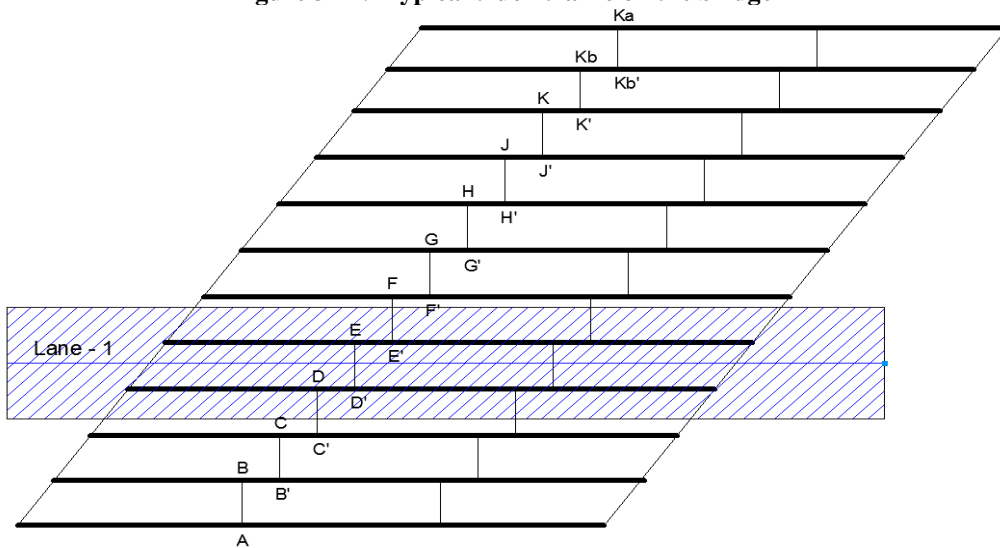


Figure 3-13. Girder layout and outer lane position

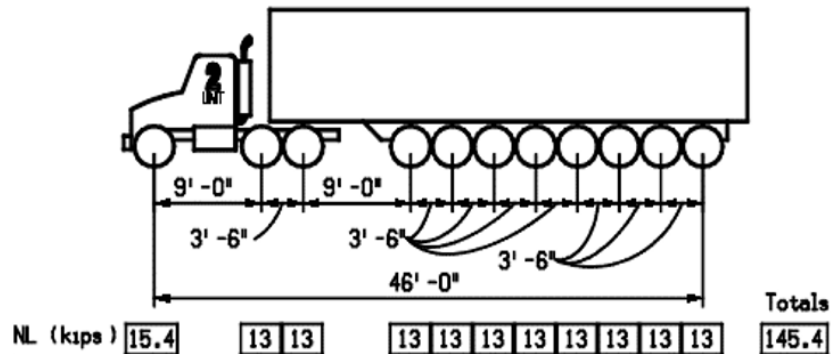


Figure 3-14. Michigan legal load: truck no. 17 configuration with normal axle loads

3.6.3 Boundary Conditions

Girder ends over the pier are supported on expansion bearings. Hence, roller boundary conditions were used over the pier. The bridge has an integral abutment where the girder ends are embedded in the backwall. Girder end fixity over the abutment is not easy to quantify. Hence, weld toe stresses were calculated using pin and fixed support conditions to develop a stress envelope. Since the girder end is embedded into the backwall, girder ends were restrained for torsion.

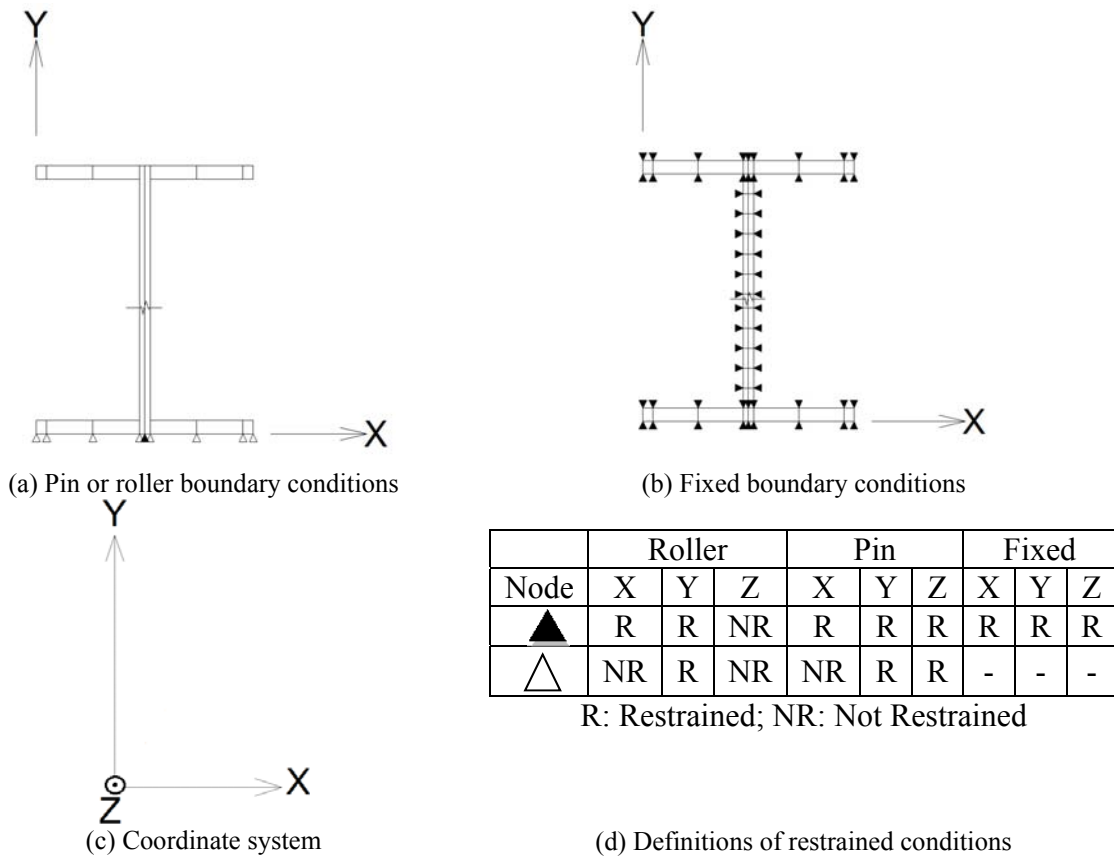


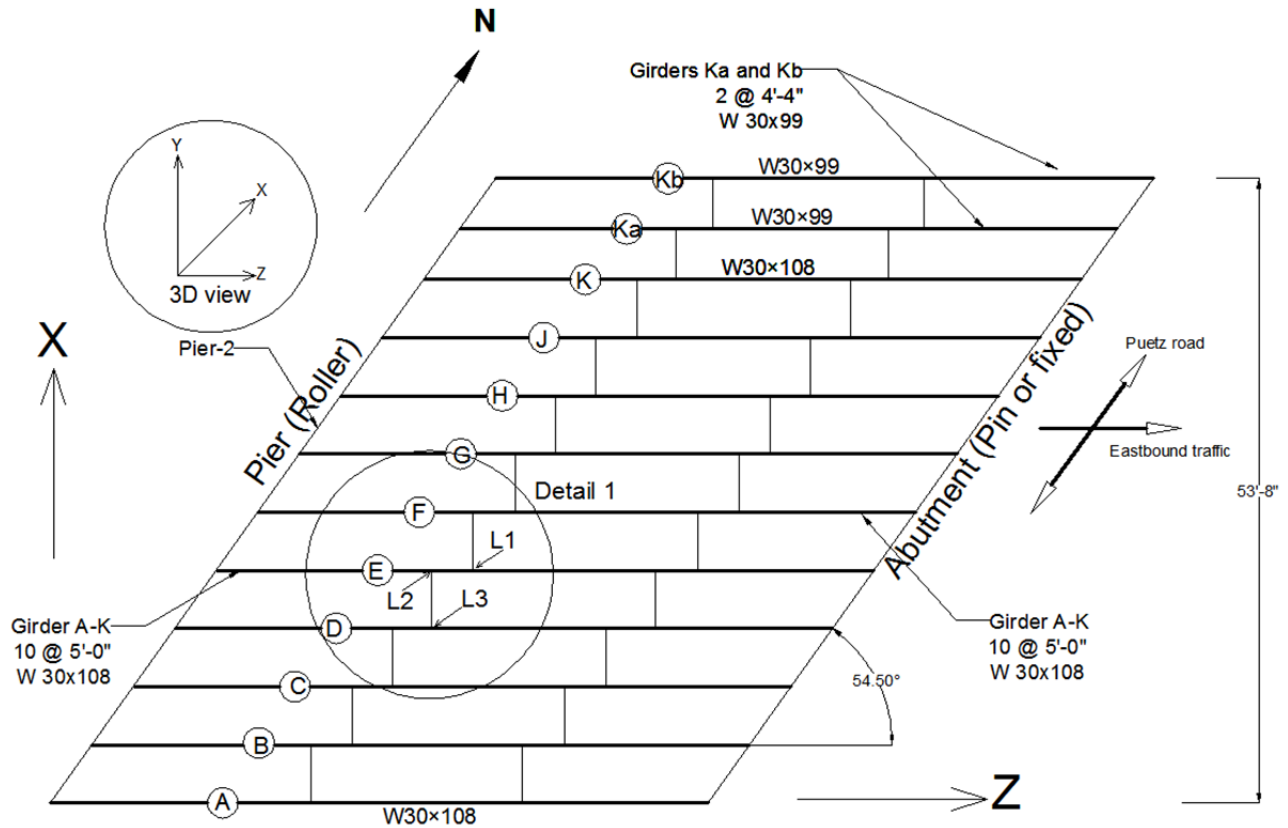
Figure 3-15. Girder end boundary conditions

3.6.4 Finite Element Representation of the Bridge

The reverse engineered model using point cloud data from the laser scanner was instrumental in measuring diaphragm positions and web gap space as well as verifying dimensions of the rest of the bridge components. The information collected through field measurements (such as weld length), laser scanning, and bridge plans was used to develop a 3D finite element model of the bridge. The welds, at web gap and coverplate ends, were identified as fatigue-sensitive details. Preliminary analysis under Michigan legal loads indicated that stresses at partial depth

diaphragms located below the outer lane are the greatest. Based on the outer lane position, three web gap locations were identified for detailed investigation of the weld toe stresses and labeled as L_1 , L_2 , and L_3 (Figure 3-16 and Figure 3-17). The web gap dimensions of L_1 , L_2 , and L_3 are 3.765 in., 1.665 in., and 3.665 in., respectively. According to Zhou (2005), three conditions must be simultaneously present for the development of weld toe cracking due to out-of-plane deformation at a web gap. These conditions are (1) an unstiffened web gap, (2) out-of-plane deformation, and (3) constraint at the web gap boundaries. Even though the web gaps are located above the girder bottom flange, which provides less restraint than the top flange with a deck, large skew and staggered diaphragm positions are other reasons for the large stresses observed. Hence, a detailed analysis was performed using truck #17, the most critical legal load, to calculate stresses at L_1 , L_2 , and L_3 locations.

The entire bridge superstructure was modeled using solid elements (Figure 3-18). Figure 3-19 shows a close up view of the web gaps. A submodeling approach was implemented to develop refined FE models for weld toe stress calculation (Figure 3-20). Section 3.5 presents recommendations for element types, mesh configuration, and stress extrapolation methods for HSS calculation. As per the recommendations, C3D20 elements with an average element size of $0.5t$ were selected for the submodels. Mesh size close to the weld toe was modified based on the stress extrapolation technique to define reference points for stress calculation.





(a) Back side of the connection
(view from abutment towards pier)



(b) Front side of the connection
(view from pier towards abutment)



(c) Opposite face of the connection



(d) Close-up view of the web gap

Figure 3-17. A typical web gap fatigue-sensitive detail

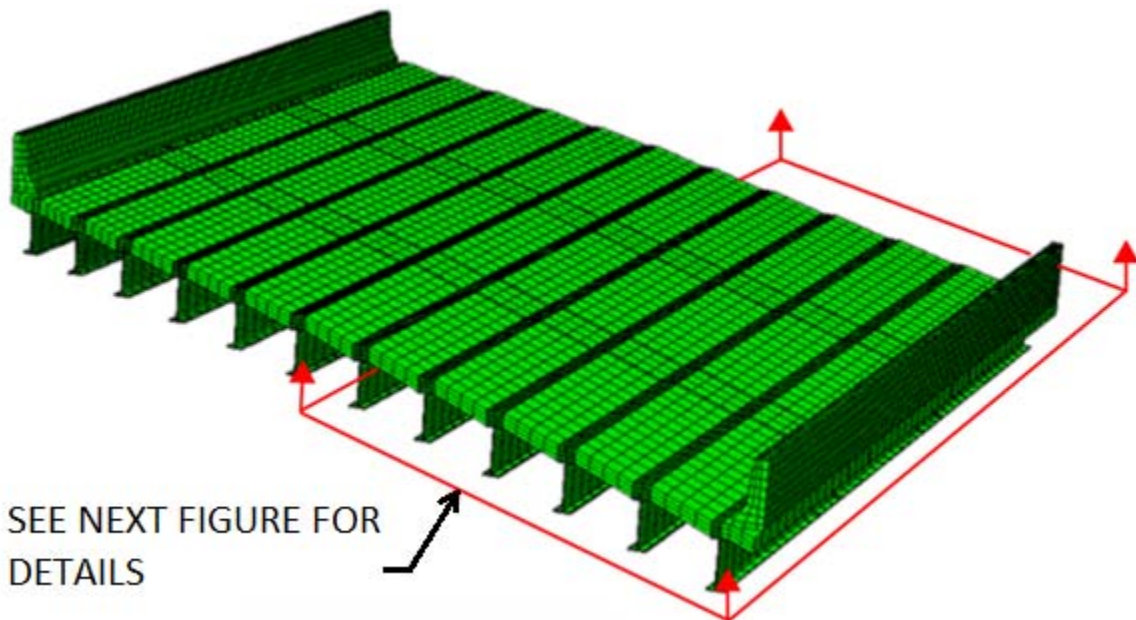


Figure 3-18. 3D view of the bridge model

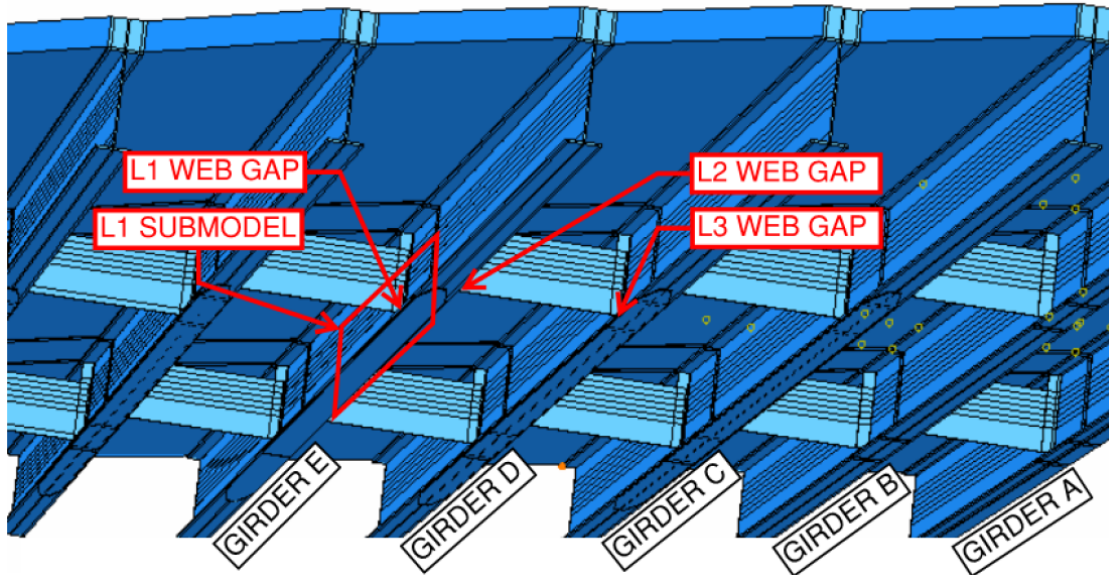
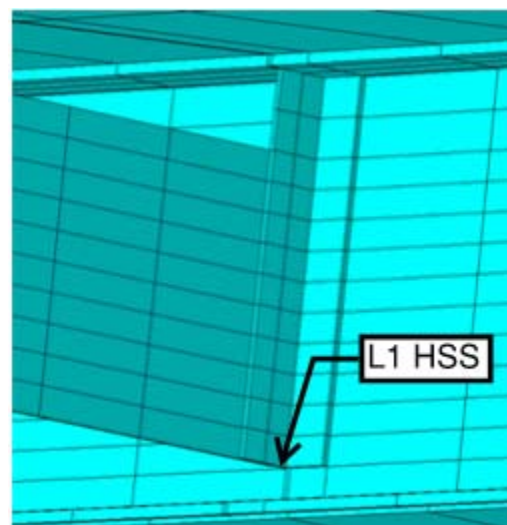


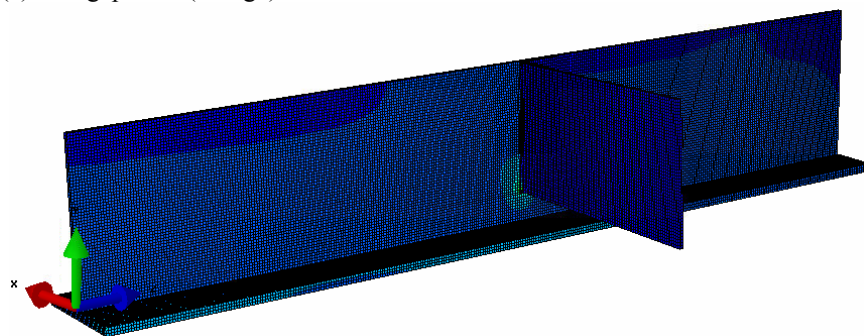
Figure 3-19. Detailed view of web gaps



(a) Webgap at L_1 (Bridge)



(b) Webgap at L_1 (FE Model)



(c) Submodel L_1

Figure 3-20. Web gap at L_1 and the FE representation using a submodel

3.6.5 Analysis Results

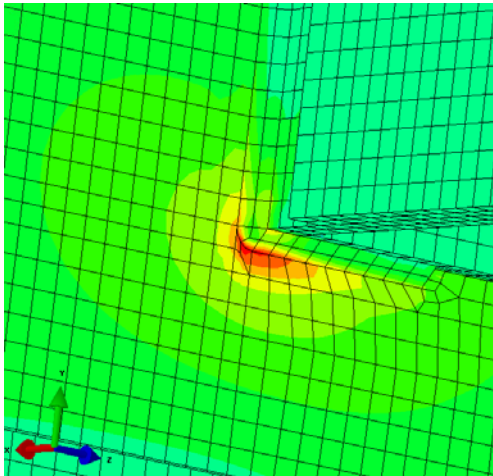
3.6.5.1 Hot Spot Stresses under Michigan Legal Loads

Abaqus CAE (Abaqus 2014) was used as the pre/post-processor. Analysis was performed using the Abaqus standard solver. Based on the recommendations presented in section 3.5, the HSS calculation method recommended by Hobbacher (2008), also known as the International Institute of Welding (IIW) guidelines, was utilized. Note that the cracks at the transverse connection plate weld toe develop parallel to the girder flange or the primary bending stresses in the web plate (Fisher et al. 1990). Hence, instead of using principal stresses, the stresses perpendicular to cracking (S_{YY}) were calculated at reference points and presented in Table 3-6. Then, the HSS at L_1 and L_3 was calculated using Eq. 4-1 (Table 3-6). The quadratic extrapolation equation (Eq. 4-1) is not applicable for the L_2 web gap because the stress pattern around the weld toe does not represent the experimental stress pattern that was used to derive the equation. The web gap space (1.665in.) is smaller than the gaps at other two locations. Also, the stresses below the weld changes from compression to tension under truck #17 load. Linear extrapolation uses two reference points that are located at 0.5t and 1.5t from the weld toe. However, this method is also not applicable because the reference points are located within tension and compression zones. Further investigations of weld toe stresses at L_2 were not conducted because the stresses at this location are much smaller compared to the stresses at L_1 and L_3 . Hence, location L_1 and L_3 were selected for instrumentation. Additional details related to instrumentation are provided in chapter 4. Figure 3-25 shows the girder and diaphragm deformation under truck #17. Figure 3-26 and Figure 3-27 show girder and web gap deformation under truck # 17. According to Figure 3-26 and Figure 3-27, the web gap deformation under truck # 17, with fixed or pinned abutment conditions, shows a single curvature. This deformation validates the application of linear or quadratic stress extrapolation methods to calculate HSS at L_1 and L_3 weld toes.

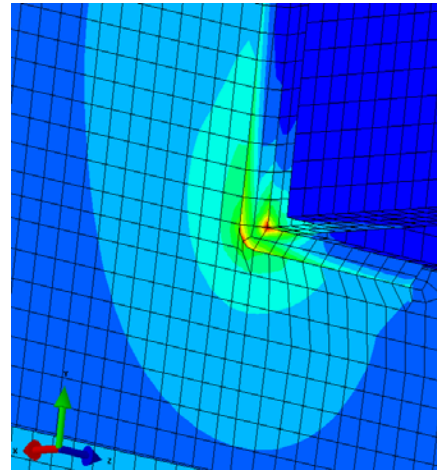
$$\mathbf{HSS} = 2.52 \sigma_{0.4t} - 2.24 \sigma_{0.9t} + 0.72 \sigma_{1.4t} \quad (4-1)$$

Table 3-6. HSS at L₁ and L₃ Web Gap Weld Toe Due to Truck #17

Reference point location from weld toe	Distance from weld toe with $t = 0.545$ in. (in.)	Stress in Y-direction (S_{YY}), (ksi)			
		Fixed support over the abutment		Pinned support over the abutment	
		L ₁	L ₃	L ₁	L ₃
1.4t*	0.218	1.96	2.90	4.40	2.90
0.9t	0.491	3.31	3.90	5.46	3.86
0.4t	0.763	5.12	5.70	8.14	5.71
*t = web thickness	Hot Spot Stress, (ksi)	6.90	7.72	11.45	7.83



S_{YY} at L₁



Misses stress at L₁

Figure 3-21. Web gap at L₁ and stresses around weld toe due to truck #17

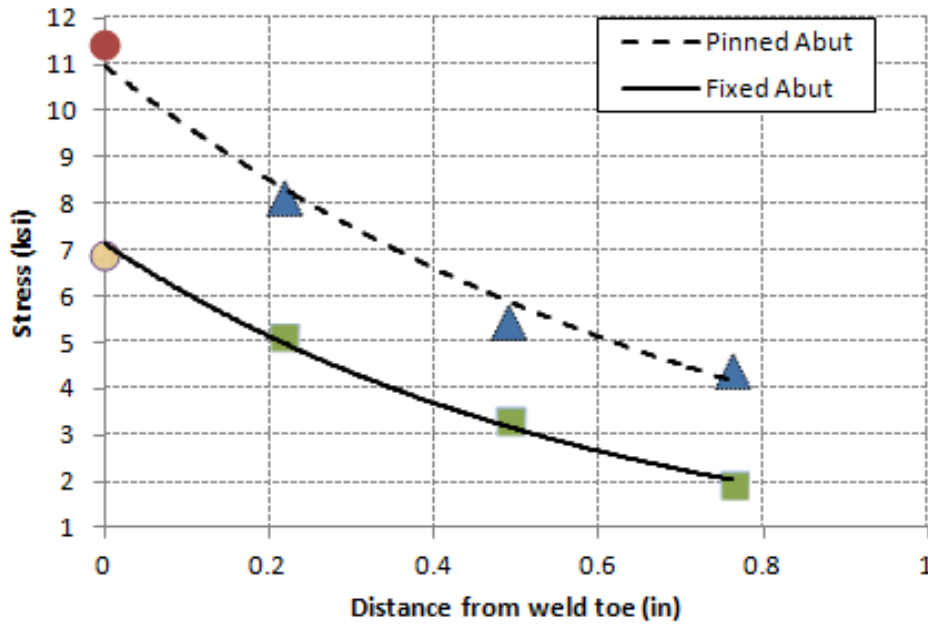
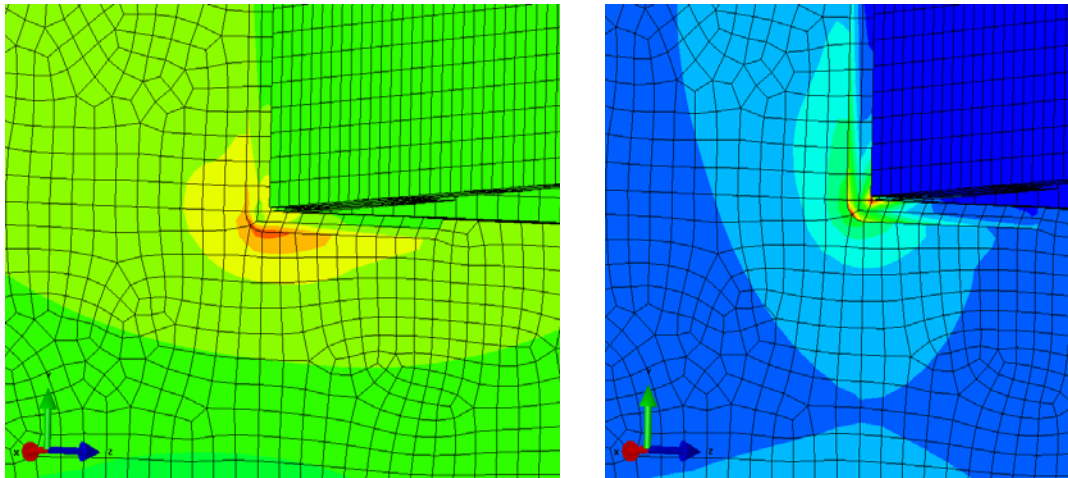


Figure 3-22. Hot spot stress (S_{YY}) at L₁ using quadratic extrapolation



S_{YY} at L_3 Misses stress at L_3
Figure 3-23. Web gap at L_3 and stresses around weld toe due to truck #17

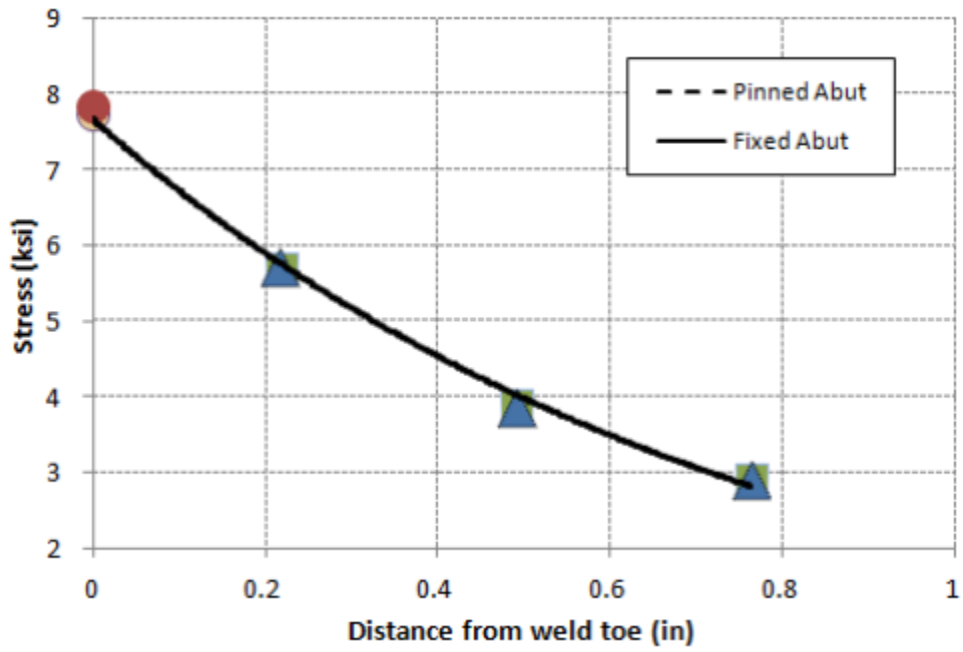


Figure 3-24. Hot spot stress (S_{YY}) at L_3 using quadratic extrapolation

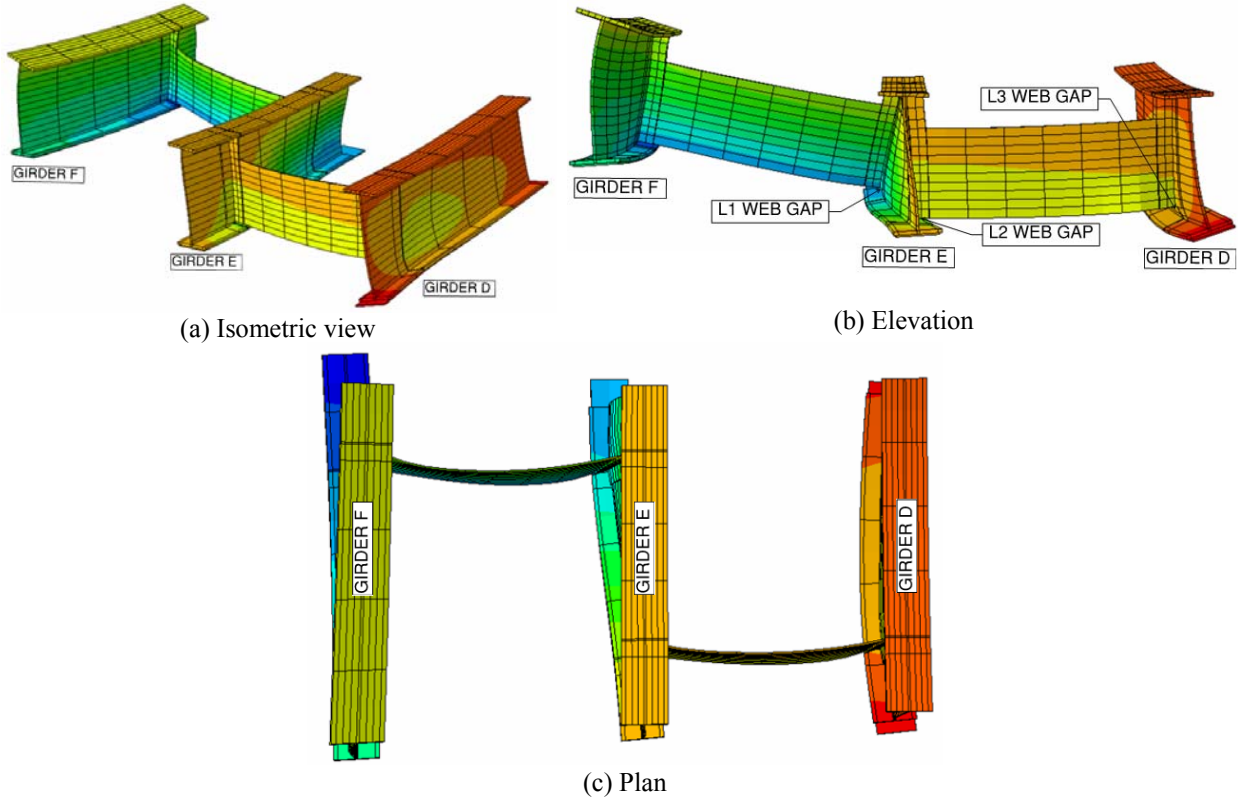


Figure 3-25. Girder and diaphragm deformation under truck #17

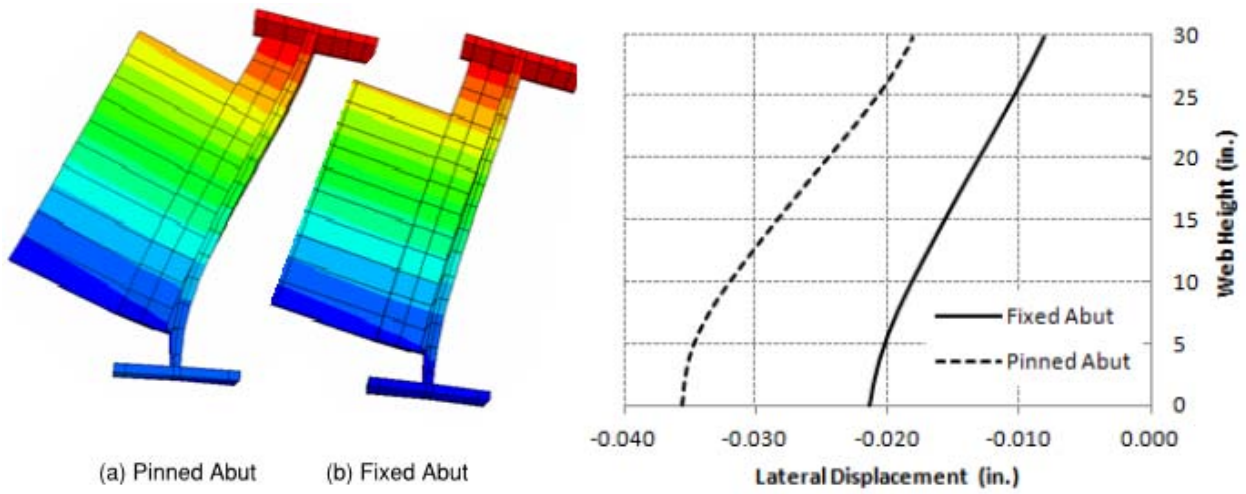


Figure 3-26. Deformation of girder E and the web gap (L_1) due to truck # 17

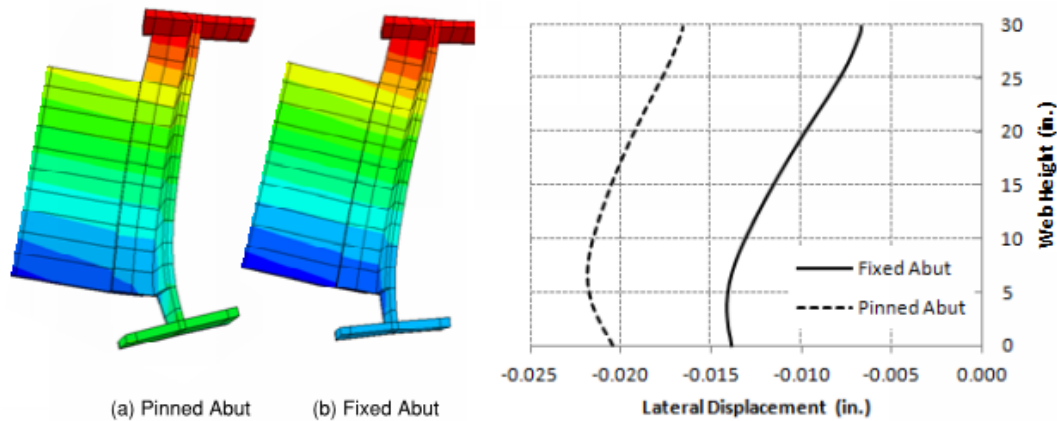


Figure 3-27. Deformation of girder D and the web gap (L_3) due to truck # 17

3.6.5.2 Hot Spot Stresses under Deck Dead Load

Plastic concrete weight during bridge deck placement is carried by the girder-diaphragm assembly. Considering this scenario, analysis was performed to calculate weld toe stresses due to the weight of plastic concrete. In order to establish a stress envelope with lower and upper bound values, two different boundary conditions were considered over the abutment: fixed and pinned. Table 3-7 presents hot spot stresses. Figure 3-28 and Figure 3-29 show stresses at reference points and weld toe. According to the Figure 3-30 and Figure 3-31, web gap deformation under deck weight with fixed or pinned abutment condition shows a single curvature. This deformation validates the application of liner or quadratic stress extrapolation methods to calculate HSS at L_1 and L_3 weld toes.

Table 3-7. HSS at L_1 and L_3 Web Gap Weld Toe Due to Deck Weight

Reference point location from weld toe	Distance from weld toe with $t = 0.545$ in. (in.)	Stress in Y-direction (S_{YY}), (ksi)			
		Fixed support over the abutment		Pinned support over the abutment	
		L_1	L_3	L_1	L_3
1.4t*	0.218	2.41	0.99	3.35	2.42
0.9t	0.491	3.05	2.41	4.24	3.19
0.4t	0.763	4.63	3.55	6.44	4.68
*t = web thickness	Hot Spot Stress, (ksi)	6.57	4.26	9.14	6.39

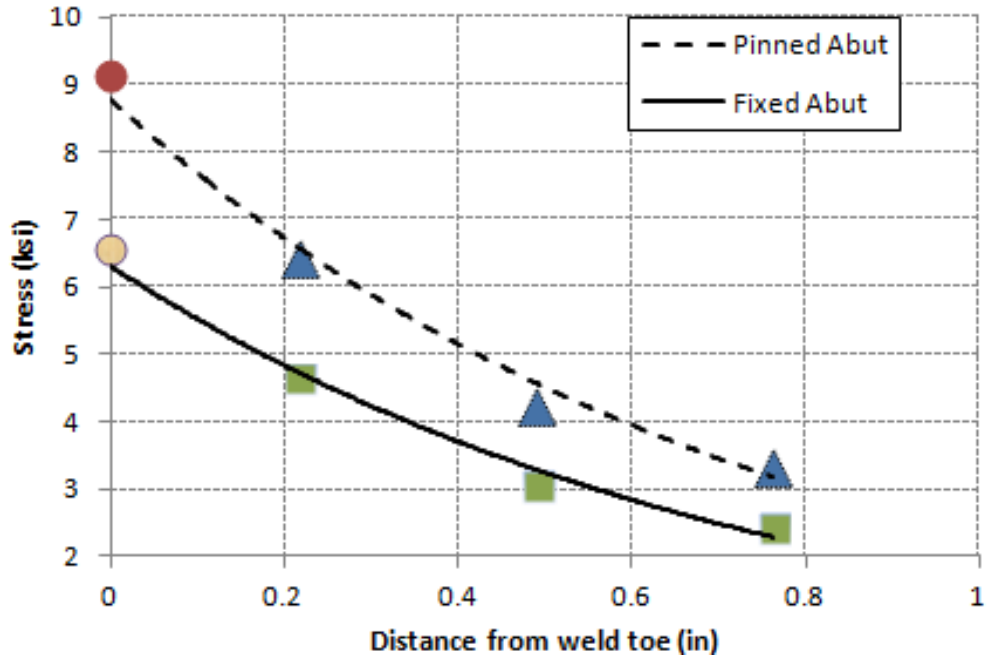


Figure 3-28. Hot spot stress (S_{YY}) at L_1 using quadratic extrapolation (deck dead load)

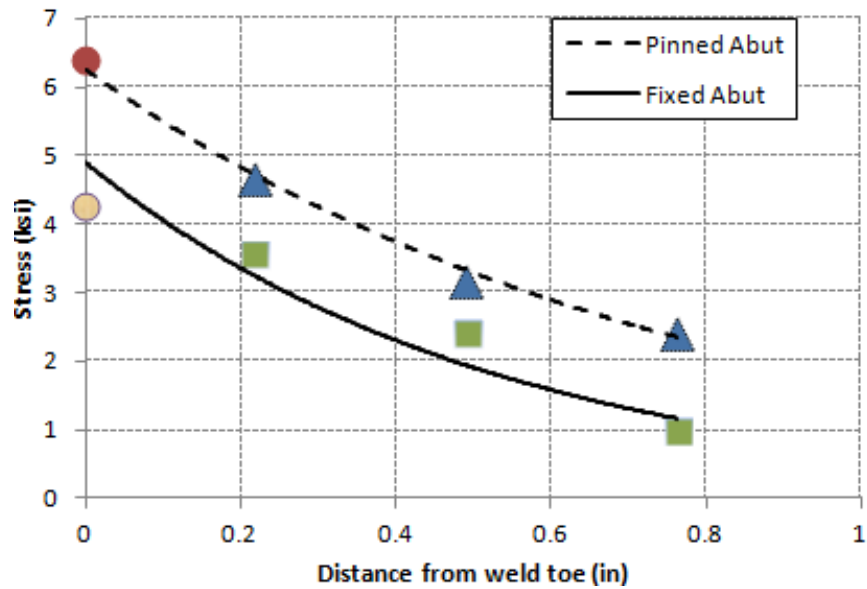


Figure 3-29. Hot spot stress (S_{YY}) at L_3 using quadratic extrapolation (deck dead load)

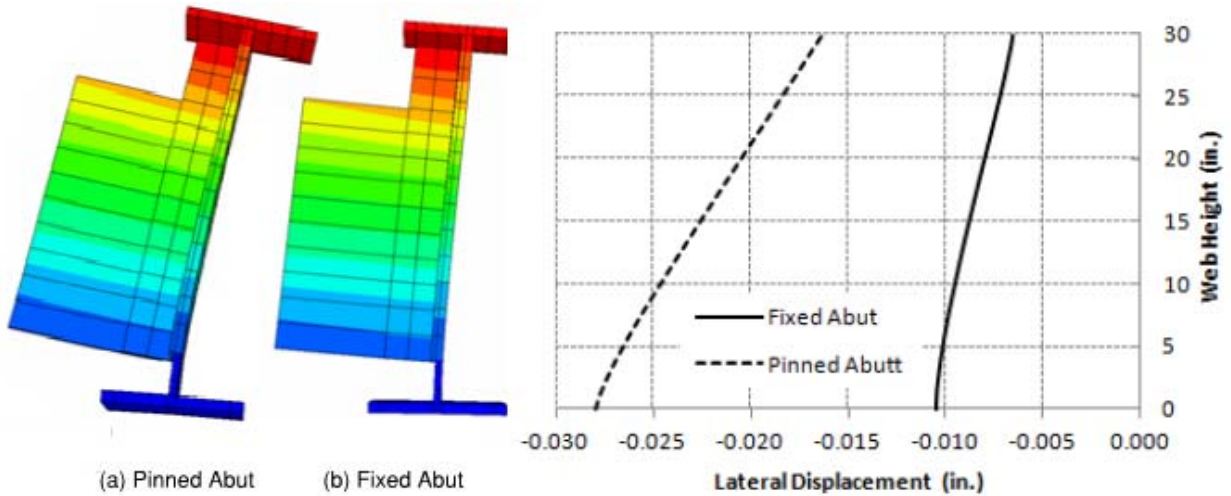


Figure 3-30. Deformation of girder E and the web gap (L_1) due to deck weight

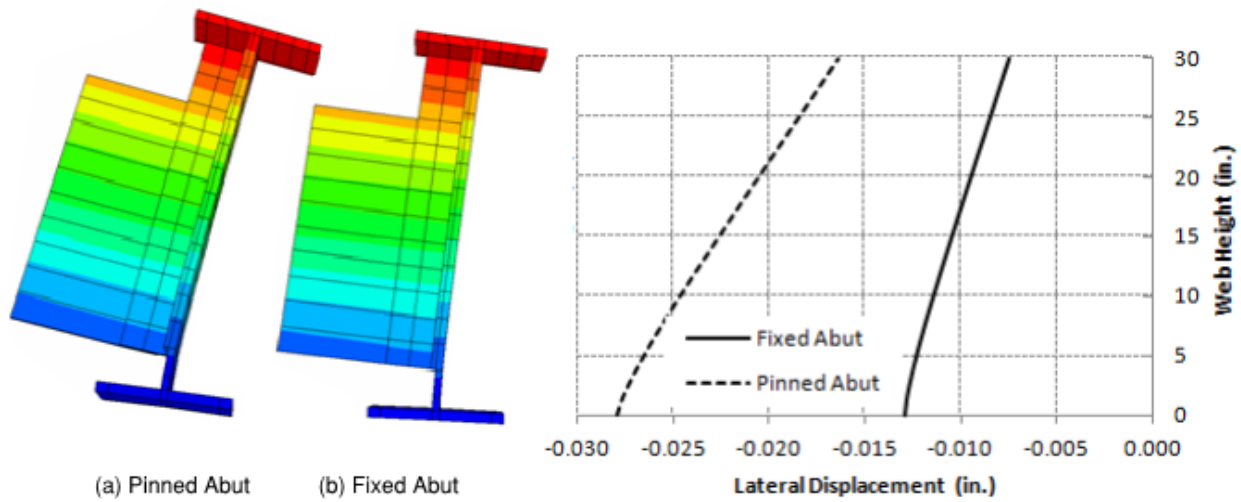


Figure 3-31. Deformation of girder D and the web gap (L_3) due to deck weight

A New Jersey Type-4 safety barrier weight was applied on the deck-girder assembly. Barrier weight develops very small compressive stresses (< 0.9 ksi) at the weld toes. According to Hobbacher (2008) stresses developed from all the loads, including transient temperature changes, need to be considered. Residual stresses due to welding and manufacturing need to be considered as the tensile residual stresses decrease the fatigue resistance. However, fatigue data that is commonly presented in S-N curve format includes the effect of residual stresses. Following a conservative approach, the stresses developed under barrier weight were neglected; only truck #17 and the deck dead load were considered. Table 3-8 shows hot spot stress at weld toes due to the deck dead load and Truck #17. As discussed previously, girder end fixity over the abutment is not easy to quantify. Hence, weld toe stresses were calculated using pin and

fixed support conditions to develop a stress envelope. Since the girder end is embedded into the backwall, girder ends were restrained for torsion when the pinned supports were used for analysis. Yield strength of steel girders (f_y) is 36 ksi. Hence, the hot spot stress at L_1 and L_3 weld toes ranges from $0.37f_y - 0.57f_y$ and $0.33f_y - 0.40f_y$, respectively. Due to these very high stresses, the details were selected for instrumentation and monitoring. Further, the analysis results show the importance of quantifying residual stresses and the stresses due to permanent loads using technologies such as XRD.

Table 3-8. HSS at L_1 and L_3 Web Gap Weld Toe Due to Deck Weight and Truck #17

Reference point location from weld toe	Distance from weld toe with $t = 0.545$ in. (in.)	Stress in Y-direction (S_{YY}), (ksi)			
		Fixed support over the abutment		Pinned support over the abutment	
		L_1	L_3	L_1	L_3
1.4t*	0.218	9.75	9.25	14.58	10.39
0.9t	0.491	6.36	6.31	9.70	7.05
0.4t	0.763	4.37	3.89	7.75	5.32
*t = web thickness	Hot Spot Stress, (ksi)	13.47	11.98	20.59	14.22

3.7 SUMMARY

A bridge with fatigue-sensitive details was selected, and as-built details and in-service condition were documented. A three-dimensional (3D) finite element model of the bridge was developed and used to evaluate the structural behavior and girder and diaphragm deformation under specific loads and load patterns. Finally, two details were selected for instrumentation and monitoring.

The bridge selected for this study contained partial depth diaphragms with welded details. Evaluation of the fatigue-sensitive welded details requires calculating hot spot stress (HSS). HSS calculation requires selecting appropriate element types, mesh parameters, and stress extrapolation methods. The finite element modeling guidelines and HSS calculation methods presented in Section 2.3.4 were evaluated by simulating tension and flexure tests conducted by Lee et al. (2010). These simulations were helpful in selecting C3D20 elements with an average element size of $0.5t$ for the submodels used in the subsequent analysis. The quadratic extrapolation technique was recommended to calculate HSS.

Based on the recommendations, a 3D finite element model of the bridge was developed and HSS were calculated under Michigan legal loads and deck dead load. In order to develop a stress

envelope for each weld, girder end boundary conditions over the abutment were considered as fixed and pin. The HSS at two web gap weld toes (L_1 and L_3) ranged from $0.37f_y - 0.57f_y$ and $0.33f_y - 0.40f_y$, respectively. Due to these very high stresses, the details were selected for instrumentation and monitoring. Furthermore, the analysis results show the importance of quantifying residual stresses and the stresses developed under permanent loads for accurate evaluation of the fatigue-sensitive details.

4 STRUCTURAL HEALTH MONITORING (SHM) SYSTEM IMPLEMENTATION

As presented in Chapter 2, after an extensive review of successes and challenges of implementing technologies for fatigue event detection and crack growth monitoring, a monitoring system with acoustic emission (AE) sensors was selected. Chapter 3 presents the process that was followed for selecting bridge details for instrumentation and monitoring. This chapter presents information related to field implementation, calibration of the SHM system, and power supply. In addition, a brief description of the system components and software used for system configuration, data acquisition, data display, remote access, and data transfer is presented.

4.1 IMPLEMENTATION AND CALIBRATION

4.1.1 Monitoring System Components

The Sensor Highway™ II System selected for this project has a low-power computer. The operating system and essential software are installed in a 2 GB hard drive. The supplemental software and sensor data are stored in a 110 GB drive. The system for bridge monitoring was integrated for deployment by TISEC Inc. in its SABRE™ system to best suit it for the specific application. The monitoring system components are shown in Figure 4-1.

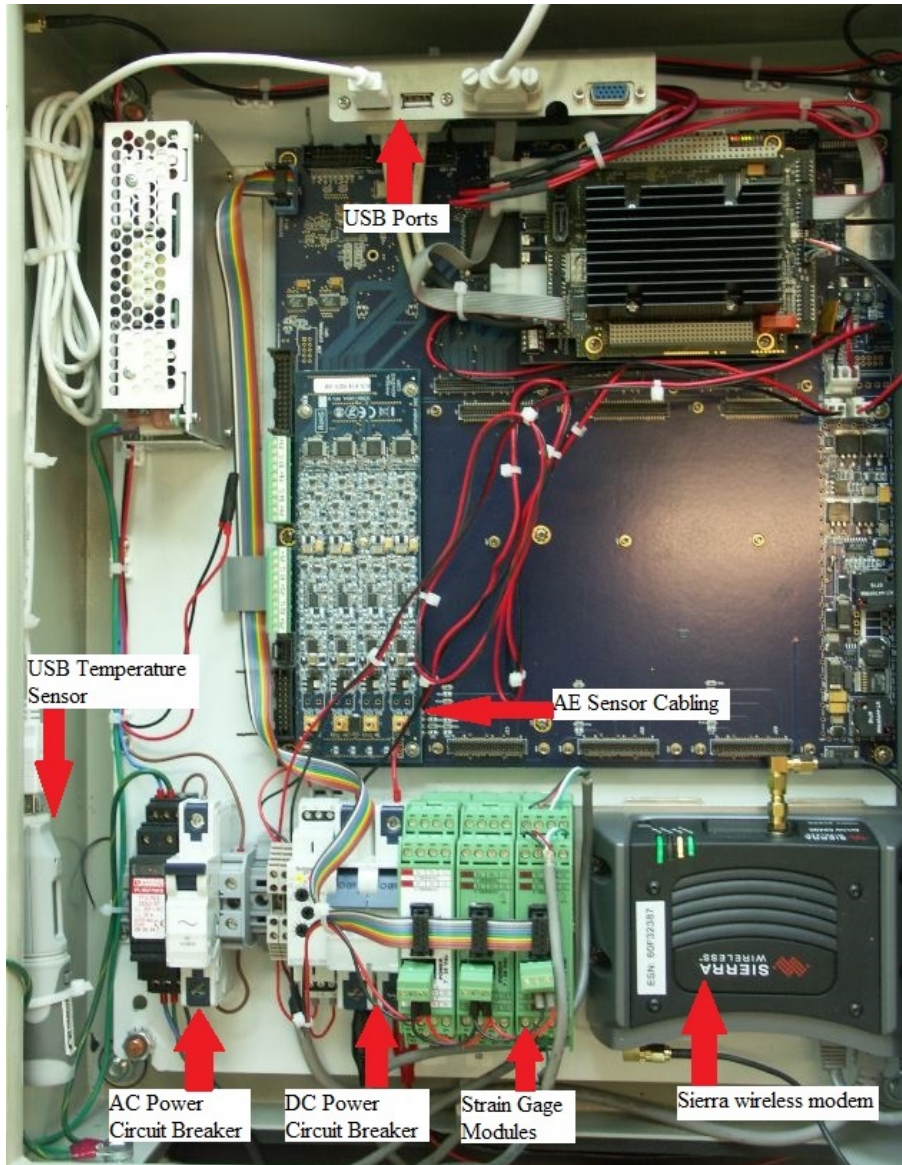


Figure 4-1. Monitoring system components in the enclosure

As shown in Figure 4-1, the monitoring system used in the project consists of only one Mistras AE board (PCI/DSP-4) with four-channels. The system capability can be extended to accommodate 3 AE boards with a total of 12 AE sensors. The monitoring system included Mistras 4 PK30I narrow band sensors (Figure 4-2a). The sensor has an integral, ultra-low noise, low power preamplifier. The frequency range of the sensor is 200 – 450 kHz with a resonance frequency of 300 kHz. Advice issued by the Department for Transport, the United Kingdom, recommends using a magnetic clamp to hold an AE sensor in place on steel structures (Figure 4-2b) (DFT UK 2006). The clamp design should include a spring to exert an adequate force on the sensor. The spring should be carefully selected not to deform the piezoelectric sensor

permanently but to ensure a constant contact pressure between the sensor and the structure. Magnetic holders, which comply with the requirements, were used to mount the AE sensors (Figure 4-2c). One of the objectives of the project was to identify a monitoring system that could be moved from site to site, as needed. Hence, using magnetic holders allows removing and remounting the sensors. Two handles, located at the top of the holder, help remove the strong magnets from the mounting surface. The holders are made of an electrical insulating material with a high acoustic attenuation.

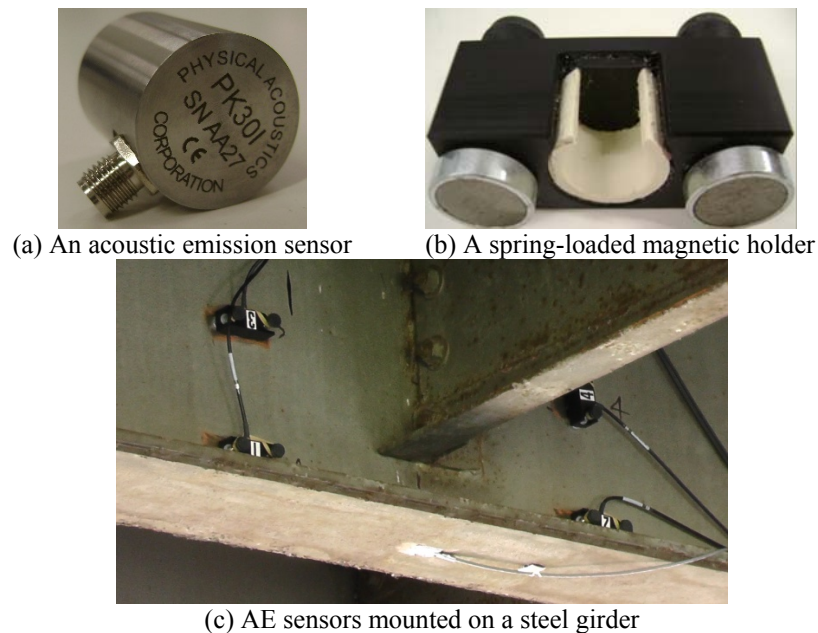


Figure 4-2. (a) An acoustic emission sensor, (b) a spring –loaded magnetic holder, and (c) AE sensors mounted on a steel girder

4.1.2 Instrumentation

Partial depth diaphragm connections at L_1 and L_3 locations, shown in Figure 3-16, were selected for instrumentation. Figure 3-17 shows the web gap detail at L_3 . Four AE sensors were mounted around the web gap, and a strain gauge was mounted at the bottom flange of Girder D to establish the load matrix (Figure 4-3 and Figure 4-4). In general, an attenuation survey needs to be performed to determine the optimum sensor spacing and arrangement. Data from the survey helps in setting up the signal threshold so that signals arising from fatigue cracks do not fall below the threshold before arriving at a sensor. Prior experience with similar details can play a significant role when designing sensor configurations and setting up signal thresholds. During this project, the sensor configuration was developed based on prior experience. The process that

was followed for sensor calibration and setting up of signal thresholds is discussed in Section 4.1.4. Three metal foil strain gauges were mounted at the web gap of the connection detail at Girder E (L_1) to calculate hot spot stresses under ambient traffic. Following the recommendations given in Hobbacher (2008) for calculating hot spot stress, three strain gages were mounted at $0.4t$, $0.9t$, and $1.4t$ distances from the weld toe; where t is the web thickness. Figure 4-5 shows the strain gages mounted at the web gap.

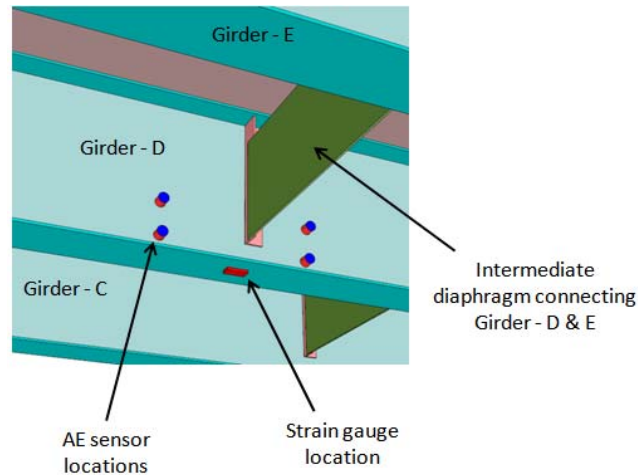


Figure 4-3. AE sensors and a strain gauge at the partial depth diaphragm connection, L_3



(a) AE sensors are mounted using magnet holders



(b) a strain gauge at girder bottom flange

Figure 4-4. AE sensors around the web gap and a strain gauge at girder bottom flange

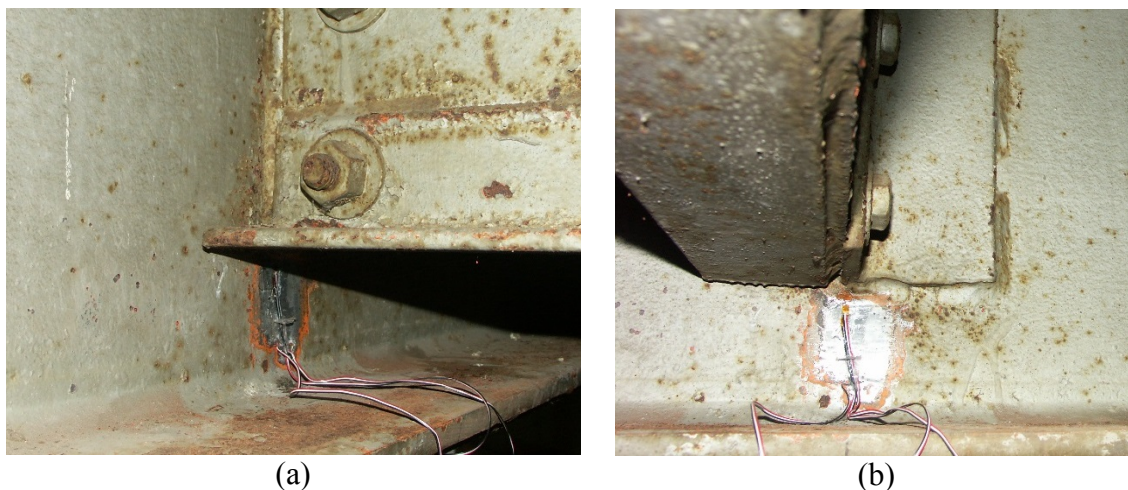


Figure 4-5. Strain gages at the webgap

4.1.3 Power Management for Filed Monitoring

One of the single most important factors in regard to as-needed or short-term monitoring is a reliable and cost-effective power supply. Grid power, while being extremely reliable and low cost for long-term monitoring, is not always readily accessible or economical for as-needed or short-term monitoring. Most power companies throughout the United States are willing to drop and run a line up to 300 ft for little to no cost; however, the costs of additional materials required for system operation including the transformer and power meter, often are in the \$3,000 to \$5,000 range. Because these associated costs are high, and such materials cannot be easily transferred to new locations, grid power is not feasible for short-term monitoring. The most significant obstacle is that the power supply from the grid will not be available when needed.

The use of energy harvested from sources such solar, structural vibration, sound, wind or a combination thereof is one of the fastest growing areas in the field of remote monitoring and sensing (Shirahata et al. 2014). Solar power systems or photovoltaic (PV) systems are devices that use sunlight to generate electricity (Ramchandra and Boucar 2011). Sunlight provides energy in the form of radiation. The amount of solar irradiance directly corresponds to the amount of energy that can be produced from a solar panel, and is the single largest factor effecting the production of electricity. Solar irradiance is the term used to denote the amount of solar radiation that reaches a surface or an area over a certain period of time, and it is often expressed in units of Langley (Ly) or kilowatts per square meter per day ($\text{kW}/\text{m}^2/\text{day}$). Due to the extreme importance of solar insolation in the production of reliable power, it is important to

determine the solar insolation instead of the number of sunshine hours. This is because the sunshine hours provide only the number of hours that sunlight is available at a site, but not the intensity of sunlight (Shirahata et al. 2014). Solar insolation data can be obtained from the *Atmospheric Science Data Center* at the *NASA Langley Research Center* (NASA 2014).

In addition to solar insolation, exposure conditions and type of solar photovoltaic affect the solar power system performance. Cloud cover, temperature, humidity, wind, and dust effect the production of solar power from a panel. Cloud cover affects energy production by way of reducing the amount of solar irradiance that comes into contact with the panel. Similarly, dust particles and shadows from nearby structures or trees can have a large impact on the electricity generation. Higher temperature also reduces the amount of energy being produced by a solar panel. In addition, humidity levels in excess of 75% can lead to a decrease in solar panel efficiency (Ettah et al. 2012; Kazem et al. 2012). Kazem et al. (2012) studied the effect of humidity on the performance of three types of solar photovoltaic: Monocrystalline, Polycrystalline and Amorphous silicon. Results show that the reduction in relative humidity increases the voltage, current and efficiency. Further, the Monocrystalline panel has the highest efficiency when relative humidity is decreased with respect to other technologies.

4.1.3.1 A Standalone Solar Power System

The standalone, hybrid, and grid tie systems are the three basic solar power systems (Hee and Isa 2009). Three main components of a standalone solar power system are (a) a solar panel (or an array), (b) a power control equipment such as a charge controller, and (c) batteries or other forms of energy storage devices (Figure 4-6 and Figure 4-7) (Tianze et al. 2011). A standalone system is ideal for short-term or as needed monitoring.

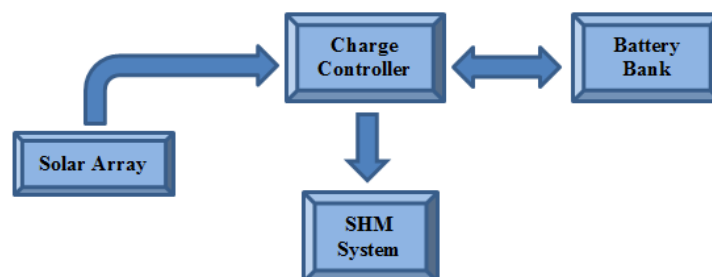
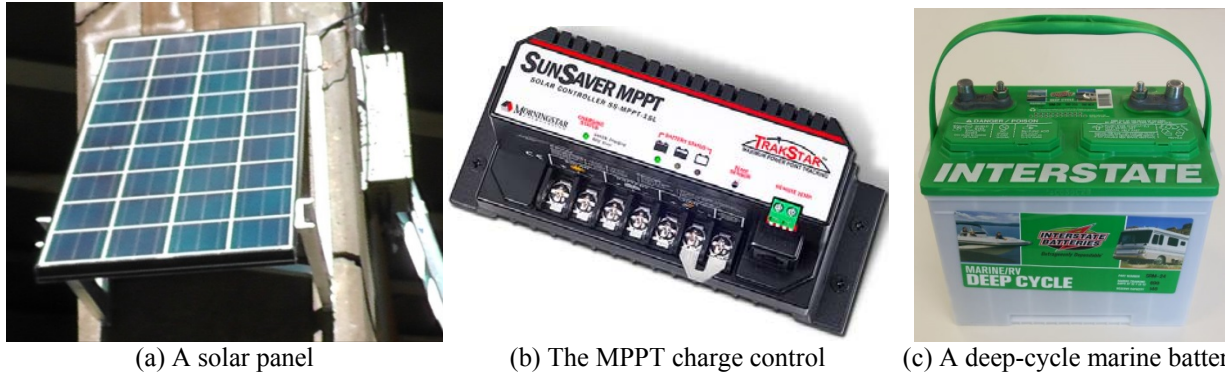


Figure 4-6. Basic components of a solar powered monitoring system



(a) A solar panel

(b) The MPPT charge control

(c) A deep-cycle marine battery

Figure 4-7. Basic components of a solar power system

As shown in Figure 4-6, an advanced charge controller can manage the entire solar power system. It has the ability to manage the power to equipment, and to protect the battery bank from overcharging and over-discharging and the solar array from reverse current (Hee and Isa 2009). Where needed, inverters can be used to convert direct current (DC) power into alternating current (AC) power. However, additional losses due to an inverter need to be considered when sizing the solar array and the storage. Some charge controllers have load control options for the purpose of setting up timers as well as advanced programming to improve charging efficiency. One such example is the Maximum Power Point Tracking (MPPT) charge controller. The MPPT charge controller uses DC to DC conversion to convert the array's operational voltage to the battery charging voltage. The conversion works on the principle that power into the MPPT is equal to the power out of the MPPT (i.e. $\text{Volts In} \times \text{Amps In} = \text{Volts Out} \times \text{Amps Out}$). The efficiency of the power conversion ranges from 92% to 95% (Sunsaver 2014). With the power conversion algorithm used in the MPPT controller, the charge (amperage) supplied to the battery can be increased by about 10% to 35% (Sunsaver 2014). The advantage of using an MPPT charge controller is that it allows the solar array to operate at its maximum power point voltage (V_{mp}); whereas, a typical charge controller supplies a charge to the battery in proportion to the power produced by the array.

The battery provides needed energy storage. Deep discharge lead-acid batteries are good for storing solar energy that can later be delivered with minimal damage to the battery cells. Battery storage or capacity is often expressed in amp-hours. The capacity stated on the battery is the total amount of energy that can be withdrawn from the battery before complete discharge. However, the total capacity of a battery can be affected by the rate of discharge and operating environment conditions (e.g., battery capacity drops to about 60% under below freezing

temperatures). This effect on the battery's capacity can be determined by using Peukert's equation or through C-rate tables available on most battery datasheets from the manufacturer of the battery. Inefficiency also exists in the process of discharging and recharging a battery; for a typical lead-acid battery, this loss can be in the range of 10%-30% (Shirahata et al. 2014). Batteries have a cycle life, or an estimated number of times that they can be discharged and recharged before the battery begins to lose its ability to maintain a charge. A cycle is the period over which the battery is discharged and recharged once; thus in a solar system a cycle would occur every day. The depth of discharge (DoD) is a term that refers to the amount of energy discharged from a battery, before it begins a new cycle. The DoD can affect the life of a battery, the majority of battery manufacturers recommend not exceeding 50%-60% discharge.

4.1.3.2 Solar Power System Implementation

The monitoring system was configured to run on either AC or DC by providing two circuit breakers (Figure 4-1). Also, the system can be connected simultaneously to an AC as well as a DC power source. This feature allows uninterrupted operation of the system by switching between the power sources seamlessly. Nevertheless, the monitoring system was powered by only a solar power system that comprised of a solar panel, a Maximum Power Point Tracking (MPPT) charge controller, and a battery bank with deep cycle marine batteries (Figure 4-7). The solar panel was rated at 140 W, with a peak production of 8.1 A and 17.2 V. The battery bank included two deep discharge lead-acid, marine batteries connected in parallel: one providing 140 Ah capacity and the other 160 Ah capacity, providing a total of 300 Ah. The monitoring system draws about 40 W. With a 12V battery bank, about 3.33 A current was required to operate the monitoring system.

The monitoring system power requirement, system losses, and solar insolation need to be considered for solar power system design. It is typical to select the month with the lowest amount of solar insolation as the 'design month.' However, with access to adequate historical data, the system can be configured for the specific month(s) that the monitoring system will be operated. Nevertheless, the solar array used in this study was undersized. This required using advanced programming features in the MPPT charge controller to operate the monitoring system based on the availability of adequate charge from the battery bank. To control power supply to a load, the MPPT charge controller has an option to define voltage thresholds: low voltage

disconnect (LVD) and low voltage reconnect (LVR). This is mainly to control the depth of discharge (DoD) to prevent battery damage as well as to maintain the longest possible SHM system operation duration. Considering the fluctuations in battery voltage at the start and end of a battery charge cycle, LVR and LVD thresholds were set to 13.6 V and 11 V, respectively. The targeted maximum battery voltage was set to 14 V.

4.1.4 Data Acquisition and AE Sensor Calibration

After AE sensors are mounted and the data acquisition system and data processing software are ready, AE signals generated by pencil lead breaks (PLB) are used to evaluate the sensitivity of sensors. The AE signals generated by PLB represent the AE signals from fracture events (ASTM E976-10, Standard Guide for Determining the Reproducibility of Acoustic Emission Sensor Response). A well-mounted sensor is expected to record a signal between 95 and 97 dB when a PLB is performed at a distance of 2 in. from the sensor (DFT UK 2006). Source location calculation requires knowing wave velocity through the material. Wave velocity can be calculated by performing PLB outside the sensor array and recording the arrival times at two sensors. The wave velocity is calculated using the distance between two sensors and the difference in the signal arrival times. Also, the noise sources are identified prior to monitoring in order to set up appropriate thresholds and determine the need for guard sensors.

The AE monitoring system implementation discussed in this study did not require using guard sensors. Wave velocity was determined knowing the girder material, A36 structural steel. Wave velocity and the signal arrival time are used to calculate the distance to an AE signal source location from an AE sensor. At least three AE sensors are needed to accurately locate the source of an AE signal on a plane. The other settings of an AE data acquisition system are signal threshold, preamplifier gain, analog filter range, and waveform parameters. The PK 301 AE sensor used in this study has an integral preamplifier with a 26 dB voltage gain. In addition, the data acquisition settings included a 40 dB preamplifier voltage gain, 45 dB threshold, 1 kHz to 1 MHz analog filter range, and waveform settings of 1MSPS sample rate, 256 μ s pre-trigger, and 1k waveform length.

Once the AE sensors were mounted and the data acquisition started, AE source locations appeared on the *AE Win* source location page (Figure 4-8). The numbers 1 to 4 in blue show the

AE sensor positions while the red dots indicate the source locations. PLB signals are used to demarcate the area of interest as well as to fine-tune the data acquisition settings: preamplifier voltage gain and the signal threshold. Pencil lead breaks were performed and the waveforms were recorded. Figure 4-9 shows the source locations calculated based on the PLB signal arrival time. These source locations mark the boundaries of the area of interest for continuous monitoring. If AE activities are documented within this region, advanced AE data analysis techniques could be implemented to understand the signal characteristics to evaluate the potential signal sources. The findings can be confirmed by conducting a detailed inspection of the detail using traditional NDE techniques. This process will verify the system performance and help fine-tune the data acquisition system variables for the specific detail.

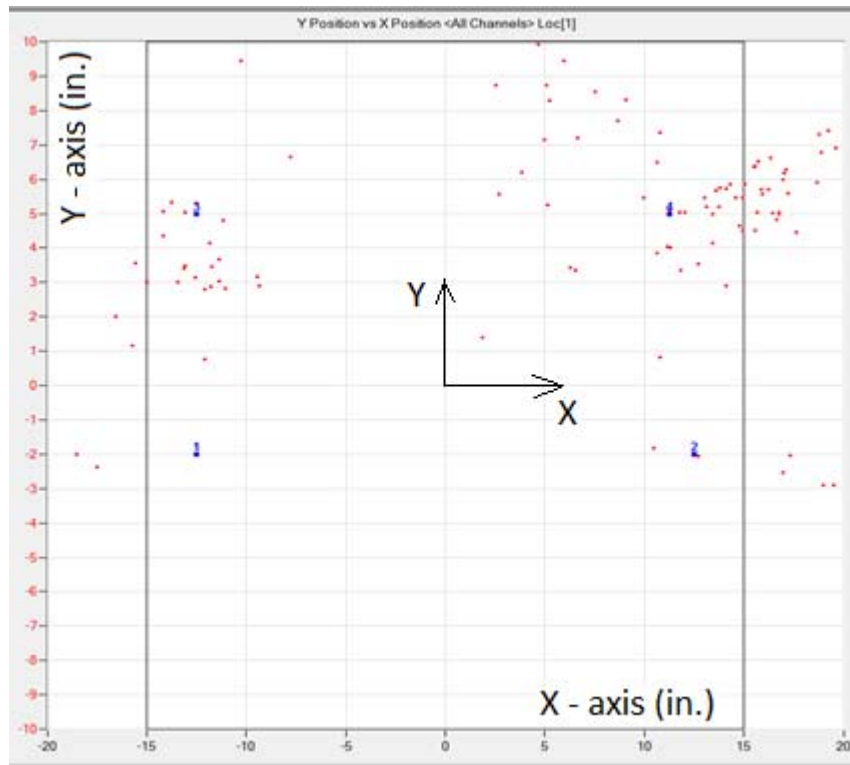


Figure 4-8. The source location page in *AE Win* showing several AE source locations

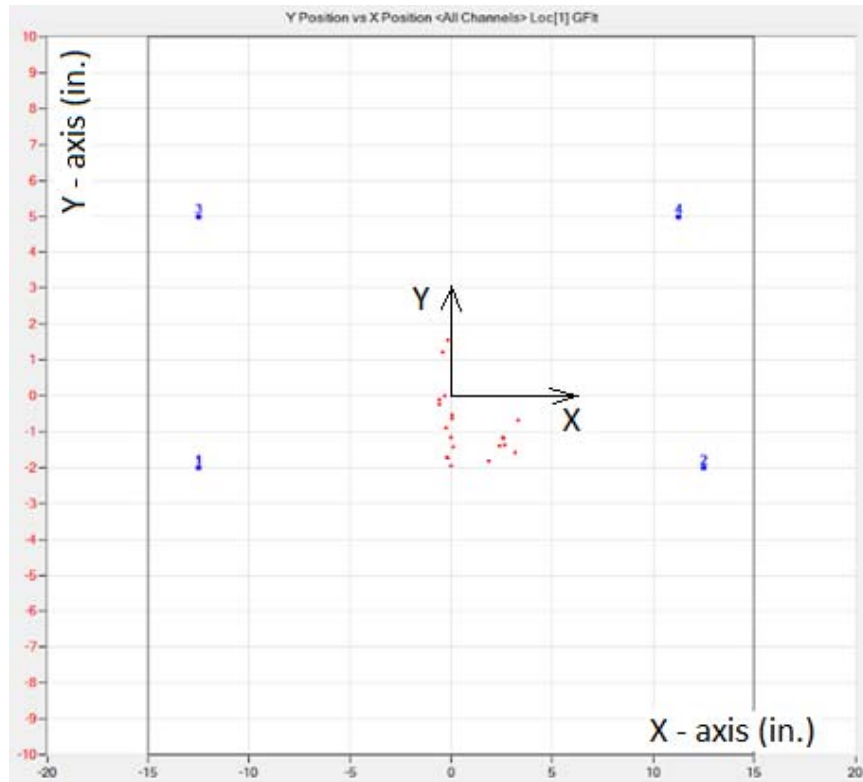


Figure 4-9. AE source locations generated through pencil lead breaks

4.1.5 Software for System Configuration, Data Acquisition, Data Display, Remote Access, and Data Transfer

AE Win is the primary software used to set up the data acquisition layout and parameters, and acquire and present AE and strain data. *Omega USB* is used to configure the temperature sensor and record data. *LogMe In* allows remote access and data transfer. Also, the *Remote Desktop Connection* can be used to access the remote computer in the SHM system. In addition, the *MS View* software program provides the SunSaver MPPT charge controller to program the charge controller and record solar power system parameters.

4.1.5.1 *AE Win*

AE Win allows system configuration, data acquisition, and real-time data display or data replay. A data acquisition setup layout was developed for the monitoring system based on the sensor layout that was designed based on the fatigue-sensitive detail configuration. A typical layout includes a source location display, an activity screen, and parametric data display (Figure 4-8, Figure 4-10, and Figure 4-11). *AE Win* can be used to replay recorded data. The process is depicted in Figure 4-12 and Figure 4-13.

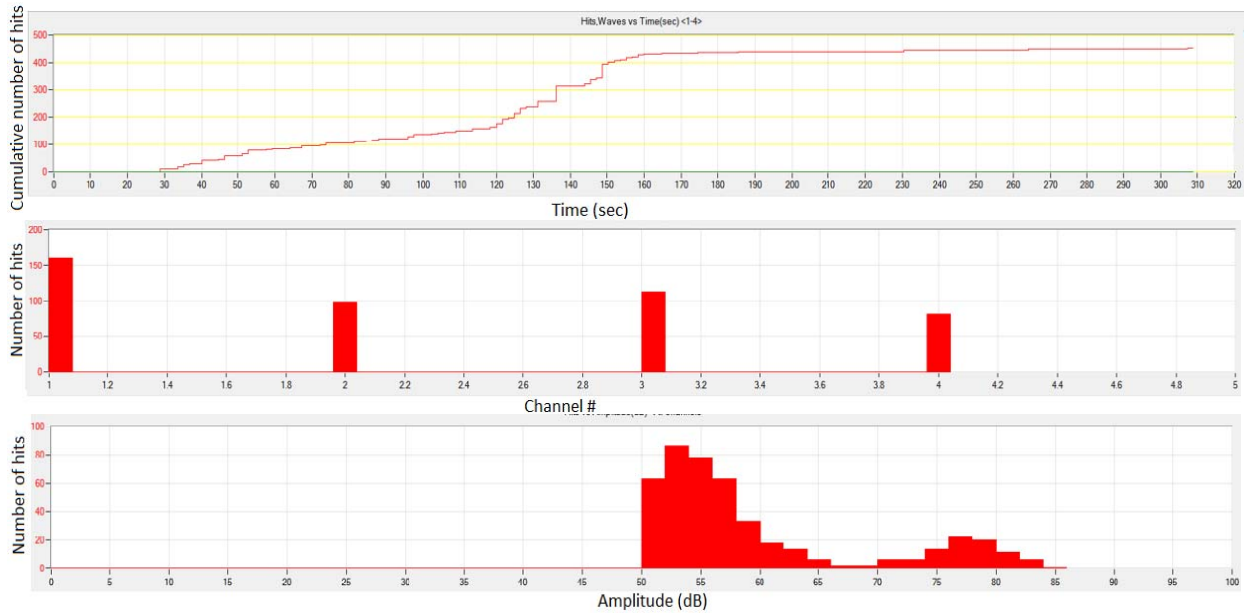
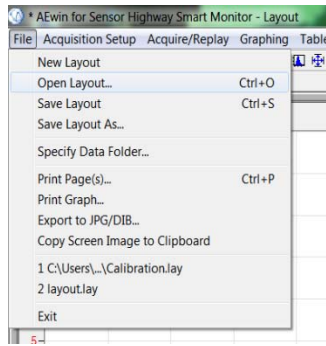


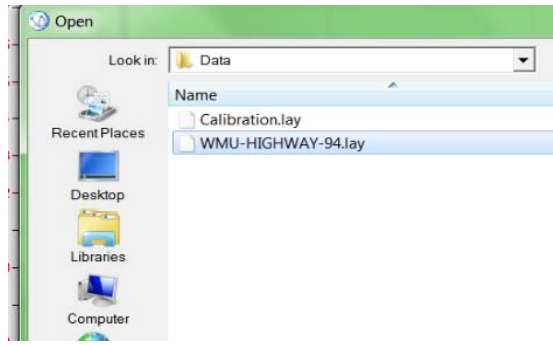
Figure 4-10. Activity screens



Figure 4-11. Parametric data display (microstrain vs. time in seconds)

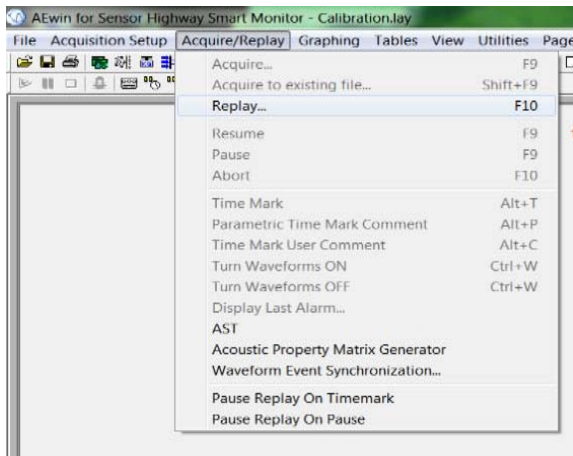


(a) Open an existing layout

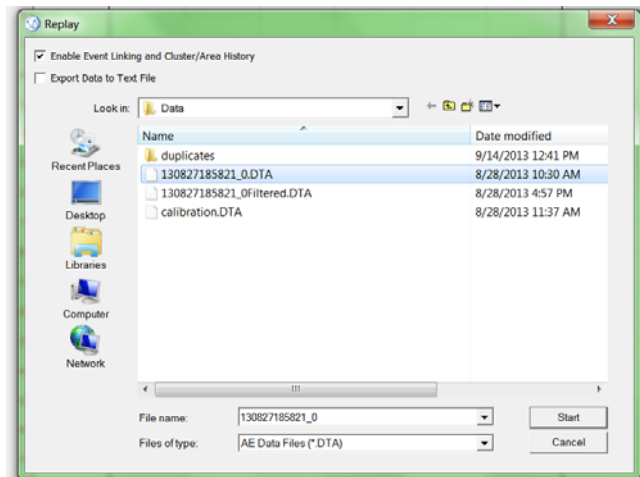


(b) Selection of the predefined layout

Figure 4-12. Opening an existing sensor layout



(a) Selection of replay command



(b) Selection of an existing data file

Figure 4-13. Accessing an existing data file for replaying

4.1.5.2 LogMe In

LogMe In is used to access a remote computer and transfer data using a web browser (Figure 4-14).

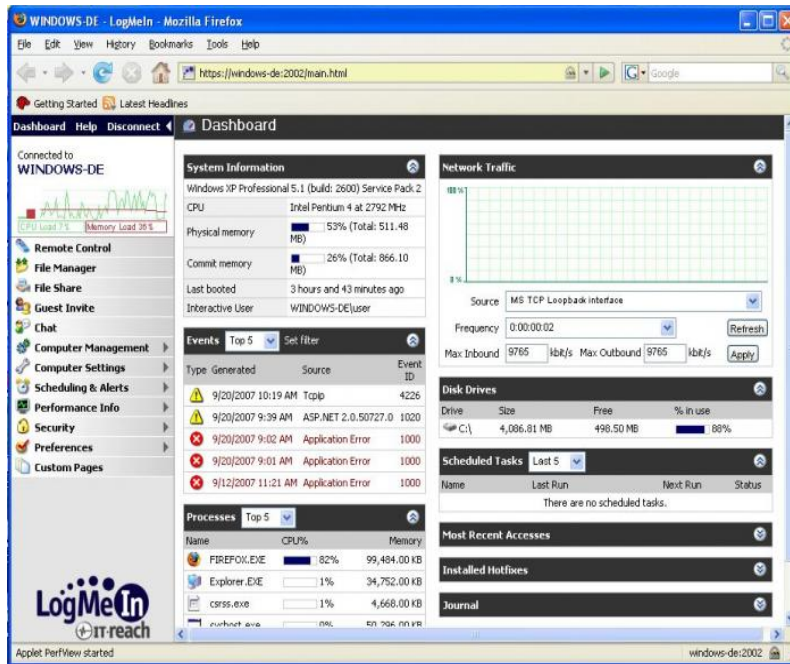


Figure 4-14. LogMe In interface

In addition, there is a mobile application that allows remote access and data visualization (Figure 4-15).



LogMe In logo

(a) Application on a mobile device

(b) AE activity screen on a mobile device

Figure 4-15. Accessing the SHM system using a mobile device

4.1.5.3 MS View

The MS View software program allows users to program the charge controller and record solar power system parameters such as battery voltage, array voltage, charge current, load voltage,

load current, etc. A SunSaver MPPT can be connected to a computer in the SHM system using a PC MeterBus Adapter, RJ-11-6P6C and RS-232 cables. Using MS View software, the low-voltage disconnect (LVD) and low-voltage reconnect (LVR) thresholds can be specified, and several timers can be added to control system operation duration.

4.2 SUMMARY

A monitoring system with acoustic emission (AE) sensors was selected. Moreover, the SHM system components, field implementation, power management for field monitoring, AE monitoring system calibration, and software used for system configuration, data acquisition, data display, remote access, and data transfer were presented for an integrated implementation with most relevant considerations.