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16. Abstract This report documents the results of a survey of current practices and technology for the evaluation and rehabilitation of historic metal truss bridges. It is intended to aid engineers in identifying solutions to typical problems found in older metal truss bridges, including inadequate strength, damage, deterioration, and inadequate height and width. This study included a literature search as well as a survey of Departments of Transportation. The literature search yielded approximately 150 documents related to the evaluation and rehabilitation of metal truss bridges, which are cataloged in this report in the form of an Annotated Bibliography. The survey of DOTs was sent to 60 agencies to solicit information on current engineering trends in addressing problems associated with historic metal truss bridges. Thirty-nine responses were received and complied. The results of the literature search and the DOT survey are reported and combined in a broad summary and synthesis of topics related to historic metal truss bridges.					
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**EVALUATION AND REHABILITATION
OF HISTORIC METAL TRUSS BRIDGES:
SURVEY OF LITERATURE AND CURRENT PRACTICES**

by

M. E. Thiel, K. Zulfqar, and M. D. Engelhardt

Research Report 1741-1

Research Project 0-1741

*PRESERVATION ALTERNATIVES
FOR HISTORIC TRUSS BRIDGES*

Conducted for the
Texas Department of Transportation

In cooperation with the
U.S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

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DISCLAIMER

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

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SUMMARY

This report documents the results of a survey of current practices and technology for the evaluation and rehabilitation of historic metal truss bridges. It is intended to aid engineers in identifying solutions to typical problems found in older metal truss bridges, including inadequate strength, damage, deterioration, and inadequate height and width. This study included a literature search as well as a survey of Departments of Transportation (DOTs). The literature search yielded approximately 150 documents related to the evaluation and rehabilitation of metal truss bridges, which are cataloged in this report in the form of an Annotated Bibliography. The survey of DOTs was sent to 60 agencies to solicit information on current engineering trends in addressing problems associated with historic metal truss bridges. Thirty-nine responses were received and complied. The results of the literature search and the DOT survey are reported and combined in a broad summary and synthesis of topics related to historic metal truss bridges.

Chapter 1: Introduction

1.1 BACKGROUND

Nearly 40% of the nations' bridges are structurally and/or geometrically deficient [1.15]. Some of the deficient bridges are in service with speed and/or load restrictions and some are out of service. Deficiencies that may be found in bridges are numerous, including uncertainty in load carrying capacity, damage to bridge members due to accidents, excessive loss of the member cross-sectional area due to corrosion, inadequate geometrical clearances, foundation deficiencies, etc.

Options available for addressing the problems associated with a deficient bridge include both rehabilitation and replacement. Many issues are involved in the decision of whether to rehabilitate or to replace a deficient bridge. The decision becomes even more complex when the bridge in question is of historic interest. Engineering, social and political factors may all play a role when addressing such a bridge. When the decision is made to rehabilitate a bridge, further questions arise as to the most cost effective rehabilitation options that maintain the historical integrity of the bridge and that address the various engineering, social and political constraints. One class of historic bridge that is frequently found to be either structurally or geometrically deficient is historic metal truss bridges. Rehabilitation of historic metal truss bridges is the subject of this report.

1.2 PROJECT DESCRIPTION

The work reported herein is part of a larger project conducted for the Texas Department of Transportation (TxDOT) entitled: "Preservation Alternatives for Historic Metal Truss Bridges." (TxDOT Project 0-1741). The overall objectives of this larger project were to develop information and tools to aid engineers and decision-makers involved with historic metal truss bridges and to help maintain these bridges in vehicular service.

Historic metal truss bridges in Texas can be divided into two broad categories: "on-system" bridges and "off-system" bridges. On-system bridges are those on the state highway system, and are found on state highways, US highways, farm-to-market routes, ranch-to-market routes, interstate frontage roads, etc. The surviving on-system historic trusses were typically constructed in the 1920s and 1930s, and were designed by TxDOT, for H10 to H15 loads.

The "off-system" bridges are those not on the state highway system, and are typically found on county roads or city streets. Many of the off-system historic truss bridges in Texas were constructed in the late 1800s or early 1900s. These bridges were often designed and erected by private bridge companies. The off-system bridges are typically constructed of light steel, wrought iron or cast iron components and have timber decks. Many of the off-system trusses pre-date the automobile, and originally carried horse traffic and livestock.

Research for Project 0-1741 was organized into the following tasks:

1. Conduct a survey of literature and of the practices of other Departments of Transportation (DOTs) on metal truss bridge evaluation and rehabilitation.
2. Conduct a case study of an off-system historic metal truss bridge.

The bridge selected for this case study is located on County Road 188 in Shackelford County, Texas. Constructed in 1885, this bridge crosses the Clear Fork of the Brazos River, and is an example of a Pratt through-truss.

3. Conduct a case study of an on-system historic metal truss bridge.

The bridge selected for this case study is located on State Highway 16 in Llano, Texas and crosses the Llano River. Also known as the Roy Inks Bridge, it was constructed in 1936 and consists of four main spans. Each span is a 198 ft. long Parker through-truss.

4. Conduct laboratory studies on floor systems representative of on-system historic truss bridges in Texas.
5. Conduct studies on the application of historic preservation principles to projects involving historic metal truss bridges.

1.3 SCOPE OF REPORT

This research report represents the completion of the first task identified above. To successfully carry out this step, the task was separated into three segments. The first step was a survey of DOTs to investigate current engineering practices for addressing problems associated with older metal truss bridges. The second step involved assembling a collection of articles and documents related to truss bridge evaluation and rehabilitation. Thirdly, the elements from the first two steps were synthesized to provide a summary of current rehabilitation techniques, supported with relevant literature and the experiences of other transportation agencies. In the following paragraphs, a short introduction to each of these segments is provided.

1.3.1 Survey of DOTs

This task involved sending mail questionnaires to other DOTs on engineering practices and experience with older metal truss bridge evaluation and rehabilitation. The first step in completing this task involved the development of the survey. Through discussions with TxDOT officials and other researchers at the University of Texas at Austin, a collection of nine questions was assembled. This survey was mailed to sixty transportation agencies throughout the United States and Canada. Thirty-nine responses were received and transcribed into a computer database. A complete summary of the survey including the methodology and techniques utilized may be found in Chapter 3. A copy of the survey is located in Appendix A along with a complete listing of the responses from the DOTs in Appendix B.

1.3.2 Literature Review

The literature review sought to collect as much relevant information concerning metal truss bridge rehabilitation as possible. The documents recovered include journal articles, books, manuals, and product information. The results of this literature review are presented in the form of an Annotated Bibliography in Appendix C.

1.3.3 Summary of Literature Review and DOT Survey

To collect the information gathered in the first two parts of the project into a useful format, a synthesis of the information was undertaken. A collection of common rehabilitation topics related to metal truss bridges was assembled. Materials from the survey of DOTs as well as from relevant documents discovered in the literature search were assembled for each topic and presented in Chapter 4. General topics such as the Analysis and Testing of Bridges, Structural and Geometric Deficiencies, and Damage Repair are included in this chapter.

Chapter 2: Literature Search

2.1 GOALS OF LITERATURE REVIEW

A major component of this study consisted of an in-depth literature search. The purpose of this search was to collect, catalog, and summarize information related to metal truss bridge evaluation and rehabilitation. This database of literature is intended to serve as a resource to engineers involved with truss rehabilitation projects, providing sources of information on technical issues pertinent to older truss bridges. The database also provides information on rehabilitation techniques that have been successfully implemented in other states, and for which experience and precedence of use already exist. Finally, the literature search served as a resource for the other portions of this research study.

This chapter describes the methods used to conduct the literature review, provides a summary of topics covered in the review, and provides guidance on methods to conduct more detailed searches to obtain publications. The results of the literature survey are summarized in the form of an Annotated Bibliography presented in Appendix C of this report. A synthesis and discussion of information found in the literature is provided in Chapter 4.

2.2 METHODOLOGY

The methodology used during this phase of the project involved several steps to collect the desired information. First, a list of topics and keywords related to steel truss bridge rehabilitation was identified. The next step was to locate publications related to these topics.

The preliminary literature search was conducted using the University of Texas at Austin library database (UTCAT) which is a computerized listing of books and articles available at the university. A variety of publications were found using the UTCAT system and retrieved for the database. A second database, namely the Engineering Index (EiCPX), was also extensively utilized. This database, available over the world-wide-web, lists articles published related to engineering topics from the 1970s to the present.

Pertinent articles located in the literature search are cataloged in the Annotated Bibliography in Appendix C. Complete copies of most of the articles in the Annotated Bibliography were also obtained and read. A summary of the article was then prepared for inclusion in the Annotated Bibliography and for use in the discussion in Chapter 4. For several articles, a copy of the complete article was not obtained. For these cases, the abstract of the article available from the database was included in the Annotated Bibliography in Appendix C.

2.3 SUMMARY OF THE DATABASE

In order to facilitate the use of the literature database, the publications have been categorized and cataloged into the following four major sections:

1. General References
2. Rehabilitation Techniques
3. Evaluation
4. Testing

Within each major section, articles were then further categorized according to topic areas. To assist in locating articles in the database, a sequential numbering system is used. For example, an article dealing with the evaluation of fatigue and fracture in a bridge can be found at 3.3.X referring to Section 3 (Evaluation), Topic 2 (Fracture and Fatigue). Many articles include information pertinent to more than one topic area. For these, the article was placed under the topic heading that most closely matched the contents of the article. However, the reader is encouraged to review the entire Annotated Bibliography to locate articles of interest.

2.3.1 General References

The articles in this section provide an introductory or general presentation of bridge preservation, problems present in metal truss bridges, and solutions to some common deficiencies. Included are articles or books of a rather broad and general nature, or articles that did not fit within the other topic categories.

2.3.2 Rehabilitation Techniques

This section contains articles that focus on individual rehabilitation techniques. Many different techniques are included with a wide range of applications. Articles covering simple rehabilitation solutions such as the addition of coverplates, to complex rehabilitation efforts involving the replacement of pins in a truss are included. These articles should aid in considering the full range of available options for truss rehabilitation, as well as provide details of the various techniques. Topics in the Rehabilitation Techniques Section include:

1. General
2. Post Tensioning
3. Superimposed Truss
4. Coverplating
5. Rivet Replacement

6. Additional Members
7. Pin Replacement
8. Deck Replacement
9. Flame Straightening

2.3.3 Evaluation

The evaluation section is comprised of references dealing with the assessment of the bridge structure. In a rehabilitation project, the key first steps include structural analysis and load rating, as well as an inspection of the bridge. Articles in this section relate to appropriate methods for structural analysis of truss bridges, as well as introductory information on non-destructive inspection and evaluation techniques. Also included are articles on fatigue and fracture concerns. Topics in the Evaluation Section include:

1. General
2. Corrosion
3. Fatigue and Fracture
4. Stability
5. Structural Analysis
6. Non-Destructive Testing
7. Reliability Analysis

2.3.4 Testing

The testing section of the catalog includes articles concerning either small or large-scale load testing of the bridge structure. To fully understand the response of a truss bridge it may be beneficial to load test either the entire structure or certain members. Articles in this section refer to some of the situations that might be presented to an engineer who would like to test a bridge, or individual members of a bridge. Topics contained in the Testing Section include:

1. Structure
2. Members
3. Connections
4. Deck

2.4 SEARCHING TIPS

The literature search required both finding pertinent references, as well as retrieving these documents. This section provides a brief discussion of literature searching techniques that may be useful to engineers that would like to search further in the literature for pertinent articles on specific topics not included in Appendix C.

2.4.1 Finding Articles

As previously mentioned, the two main search engines used were the University of Texas library system and the Engineering Index. Hundreds of article citations were identified and examined to determine which would be most applicable to the research. The University of Texas library resources may be accessed via the world-wide-web at “www.utexas.edu”. By following the links to the library, the UTNetCAT system may be referenced. More in-depth article listings may be searched by linking to the “Indexes & Abstracts” then linking to the “Science/ Technology/ Health” page. The EiCPX may be accessed, as well as other article search engines including.

- Applied Science and Technology Abstracts
- ArticleFirst
- OCLC WorldCat
- CARL UnCover

Unfortunately, the resources mentioned in the previous paragraph are available to university students, faculty, or at the University of Texas Library. Therefore, engineers outside the university must find alternate means of document location. For engineers within close proximity of the University of Texas at Austin, access to many of these databases may be gained by visiting the library and using the computer terminals inside the library. If the engineer is not located near a library facility with search capabilities, another option is to utilize various state databases and search engines. Engineers should check within the office for access to resources such as TRIS and other transportation information databases available to state agencies. If these services are not available, some databases are accessible for a charge, such as EiCPX.

2.4.2 Document Procurement

Once pertinent article or book citations were identified, it was necessary to obtain a hard copy of the document. It was determined whether the article was available in The University of Texas at Austin using the electronic card catalog. If an article was not at the university, a document delivery service was employed. By completing a request form through the on-line library system at the university, the articles were requested. If a book was not contained in the university stacks, an Interlibrary Loan (ILL) form was completed. The ILL program lends books between member libraries at little or no cost to the patrons.

The document delivery services might not be available to practicing engineers. To collect articles, the following steps are suggested. The first task involves collecting a list of citations that are relevant to the project at hand. Information such as journal name, issue, number, and pages should be listed for each article. The second step would involve checking the available sources for the articles. The University of Texas at Austin system, or other local universities may house the articles. Typically the university catalogs may be accessed on-line, saving a trip to the library. The articles, which cannot be found locally, may be retrieved using a document delivery service. Many of the search engines listed in the previous section, including the Engineering Index and ArticleFirst, provide article reprints. Usually a fee is charged for the delivery of these documents.

To procure a copy of a specific book, the engineer may investigate ILL services at their local library. A copy of the book might also be available through the publisher, provided that the book is still in print.

A wealth of information is available within the published literature that can assist engineers with solving problems associated with truss bridge evaluation and rehabilitation. While much of this has been located and is cataloged in Appendix C, there are likely many additional articles that may be helpful for addressing specific problem areas. In the past, locating pertinent information was often quite difficult without access to a large university library. However, useful articles can now be identified and obtained with much greater convenience using on-line resources. Engineers are encouraged to become familiar with these resources as a useful tool for addressing truss bridge rehabilitation problems.

Chapter 3:

Survey of Departments of Transportation

3.1 SURVEY OBJECTIVES

A mail survey was conducted of Departments of Transportation (DOTs) and other agencies on their experiences with historic steel truss bridges. The main objective of the survey was to gather additional information on a variety of topics related to truss bridge evaluation and rehabilitation. The survey was intended to document current trends and attitudes concerning truss bridge rehabilitation, to identify practical application of rehabilitation techniques documented in the literature, and to identify new or innovative rehabilitation techniques that have not yet been documented in the literature. This chapter provides a summary of the responses received in the survey. A copy of the actual survey is included in Appendix A, and a listing of all responses is provided in Appendix B.

3.2 DESCRIPTION OF SURVEY

In assembling the survey, consideration was given to accommodate both the goals of this study as well as the convenience of the survey recipients. The goal was to make the survey sufficiently in-depth to be useful, but at the same time, brief enough such that the survey recipients would not be burdened by a lengthy document. To this end, a short discussion will be included of the considerations taken to develop the survey.

3.2.1 Survey Overview

The two components of the survey were the cover letter and the main body of the survey. The cover letter summarized the goals of this research project and objectives of the survey. The survey itself was designed to be easy to comprehend, and complete, but technically relevant to the task at hand. Both the cover letter and the survey are reproduced in Appendix A.

3.2.1.1 Cover Letter

The cover letter, which accompanied each survey packet, served as an introduction of the research to the surveyed DOTs. An important primary issue involved contacting the appropriate individual at the various agencies. A book containing a listing of American Association of State Highway and Transportation Officials (AASHTO) members working at DOTs was invaluable in the creation of a mailing list for the survey [Ref. 1.25]. The survey was sent to individuals with titles related to bridge design or repair. A total of 60 surveys were sent, including the 49 other states, the District of Columbia, Puerto Rico, and 9 provinces in Canada.

The cover letter gave a short introduction to the research project, including its goals and how the survey would aid in the successful completion of the project. The cover letter also included contact persons at both The University of Texas at Austin and the Texas Department of

Transportation. Inquiries about the survey could be made via phone, mail, or email to accommodate as many people as possible.

3.2.1.2 Survey

A major consideration in designing the survey was to limit the number and complexity of the questions so as to encourage the recipients to actually complete the survey. The questions were written to allow for simple answers. However, adequate space was also included for a more involved discussion.

Questions asked in the survey related to many facets of bridge rehabilitation. Topics such as analysis techniques, non-destructive testing (NDT), railings, as well as general questions related to geometric clearances and structural strengthening were included

3.3 RESULTS OF THE SURVEY

Responses to the survey were mailed to the researchers at the University of Texas at Austin. The responses to the questions were compiled and may be found in Appendix B. Of the 60 surveys mailed, 39 responses were received, representing a 65% return rate. The responses collected from the DOTs varied in content and substance. Some respondents gave brief answers only consisting of checkmarks without elaborating on the answers. Other engineers thoroughly discussed individual questions by introducing examples and possible contacts. The graphs and charts in the following pages summarize the survey responses.

3.3.1 Question 1

Has your state developed any reports, guidelines, or other documents addressing the evaluation or rehabilitation of steel truss bridges?

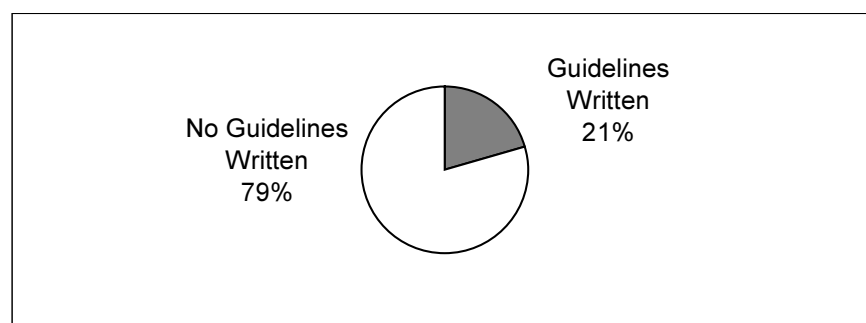


Figure 3.1: Response to Question 1

Figure 3.1 shows that a majority of DOTs have not developed standards or guidelines related to historic metal truss bridges. The state with the most published work concerning truss bridges was Iowa. They have documented experience in load testing, as well as a research project produced by Iowa State University concerning the rehabilitation of truss bridges, [Ref.

1.15]. Other states have produced reports, but on smaller levels such as Washington’s “Report on Steel Bridge Cracking” or Minnesota’s “Bridge 4174 – Summary of Inspection for Reuse as a Pedestrian Bridge”.

3.3.2 Question 2

Have you used advanced structural analysis techniques to provide improved estimates of the structural capacity of steel truss bridges?

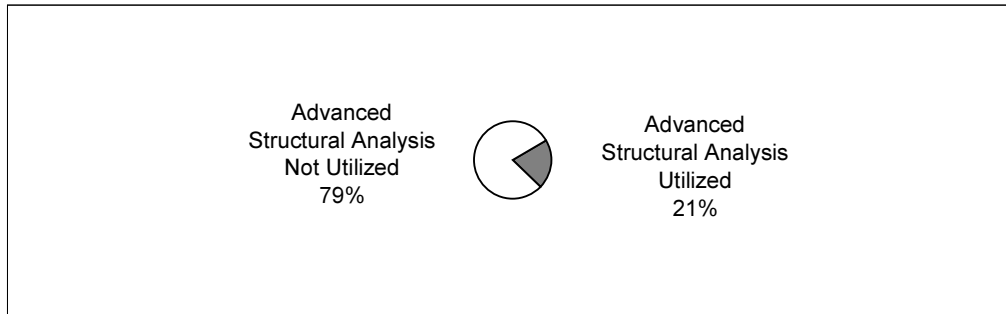


Figure 3.2: Response to Question 2

The most common structural analysis technique reported by agencies was two-dimensional analysis. Only a few agencies, such as Connecticut, Arizona, and Newfoundland, indicated that more advanced, three-dimensional analyses have been used. Analysis programs used by these DOTs include GTSTRUDL, SAP90, and BRUFEM. Based on the survey responses, conventional frame analysis, using either hand methods, or commercial structural analysis programs, is the most common technique for analyzing truss bridges. A few agencies have employed more advanced finite element programs or other advanced analysis techniques for truss bridges.

3.3.3 Question 3

Have you used advanced non-destructive evaluation techniques (e.g. acoustic emission monitoring) to assist in evaluating the condition of steel truss bridges?

Figure 3.3 shows that, most agencies have not conducted in-depth investigations by non-destructive methods. The most common NDT method indicated in the survey was the use of ultrasonic evaluation to test pins for flaws. Further discussion of this topic may be found in Section 4.2.2.2.

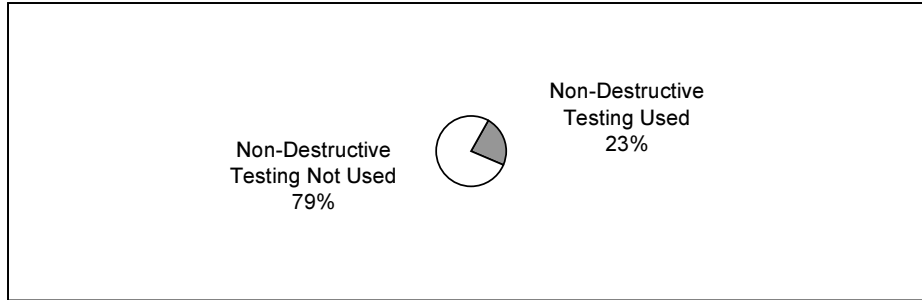


Figure 3.3: Response to Question 3

3.3.4 Question 4

Have you used load testing to assist in the evaluation of the structural capacity of steel truss bridges?

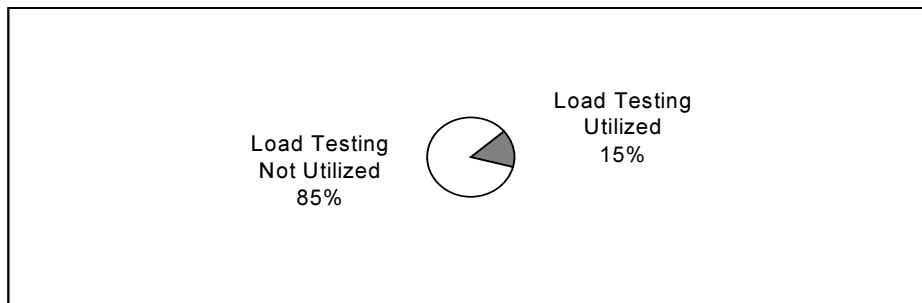


Figure 3.4: Response to Question 4

Load testing as a method of bridge evaluation is a very time-consuming and expensive endeavor. Figure 3.4 demonstrates that very few agencies employ load testing to aid in the evaluation of a bridge. A majority of load tests conducted in the United States were conducted as part of research projects. Overall though, most DOTs in the United States do not appear to perform load testing on a routine basis. However, most provinces in Canada use load testing to rate their bridges. Many examples were found in the literature of “proof loading” of truss bridges. Further discussion of this topic can be found in Section 4.2.1.2.

3.3.5 Question 5

What are the most common structural strengthening techniques your department has used in rehabilitating steel truss bridges?

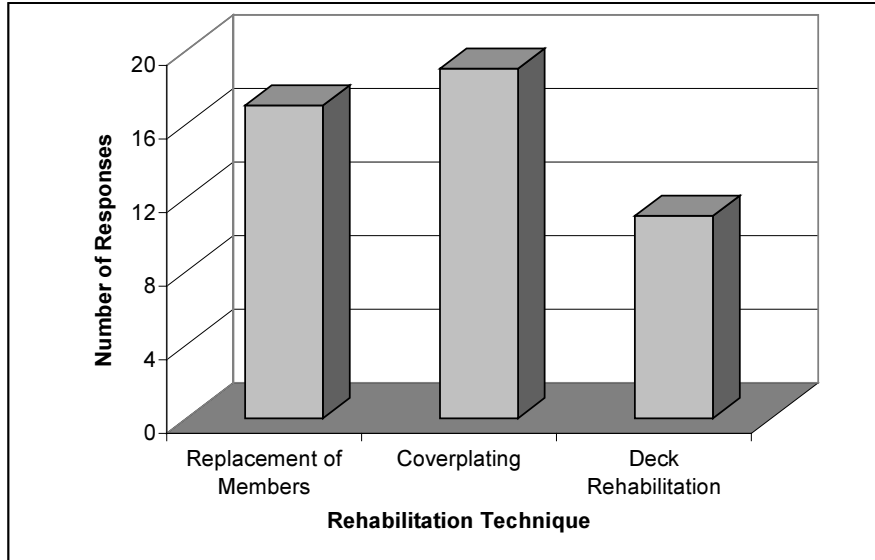


Figure 3.5: Response to Question 5

These responses indicate that most rehabilitation efforts focus on individual members instead of the whole structure. In rehabilitating a structure, the most common techniques involve repairs to the critical portions of the system. Very few responses indicated rehabilitation of an entire system as the most common solution. This can likely be attributed to the fact that most bridges are deficient in only a few locations, while other elements may be substantially overstrength and do not require repair or strengthening. Most main truss members generally appear to have adequate strength, while floorbeams and decks are frequently found deficient. The rehabilitation of these members usually involves techniques such as replacement of members, coverplating, or deck replacement.

3.3.6 Question 6

Please check any other structural strengthening techniques you have used.

- | | |
|--|--|
| <input type="checkbox"/> Superimposed trusses | <input type="checkbox"/> Addition of longitudinal beams |
| <input type="checkbox"/> Post-tensioning bottom chord | <input type="checkbox"/> Providing additional supports |
| <input type="checkbox"/> Joining simple spans into continuous span | <input type="checkbox"/> Adding king or queen posts post-tensioned tendons |
| <input type="checkbox"/> Replace floor deck with a lighter system | <input type="checkbox"/> Pin replacement |
| <input type="checkbox"/> Attach cover plates to members | <input type="checkbox"/> Other (please explain) |

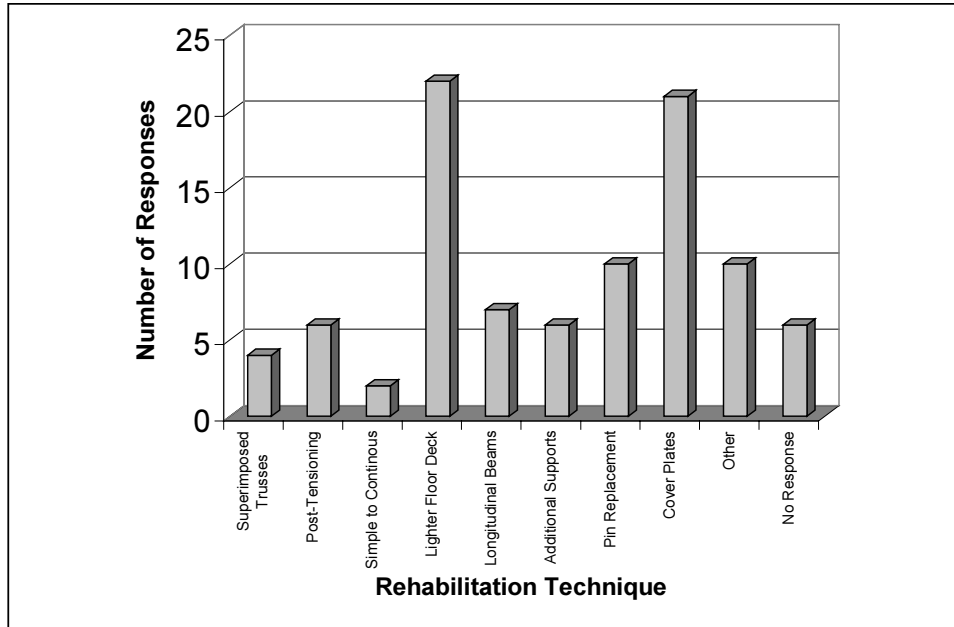


Figure 3.6: Response to Question 6

Figure 3.6 shows that a wide variety of techniques have been used successfully. Coverplating and deck replacement are the two most common rehabilitation techniques. The coverplating technique is useful when a few members are understrength. By utilizing a lighter deck system, the dead load of the structure is reduced and the live load capacity is increased. However, deck replacement might also be warranted to repair a deficient deck.

3.3.7 Question 7

For bridges with geometric deficiencies, either inadequate height or width, please check any solutions you have used:

- Relaxing geometric standards for historic bridges
- Widening bridge
- Increasing portal height by removing or altering overhead members
- Convert bridge to one-way traffic
- Other (please explain)

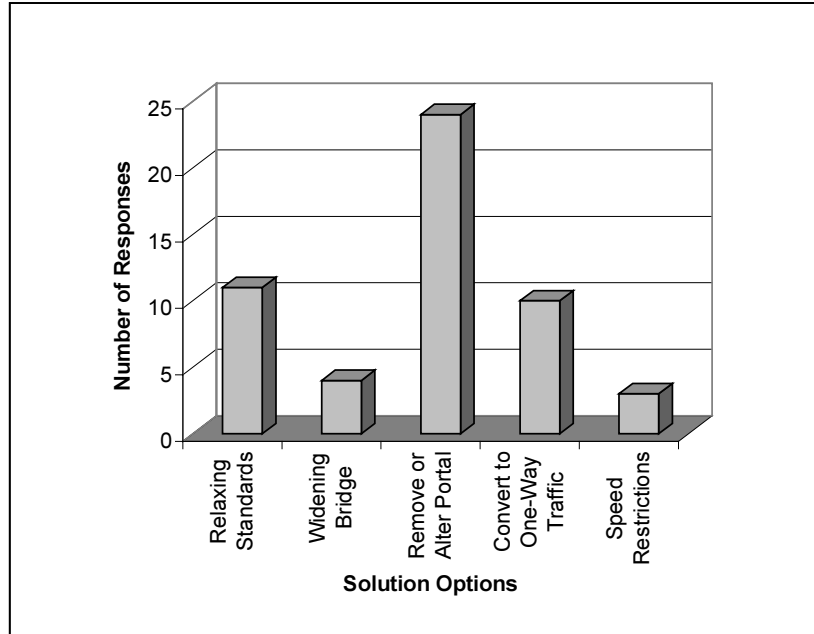


Figure 3.7: Response to Question 7

The issue of geometric deficiencies, as they relate to historic truss bridges, is highly problematic issue. Figure 3.7 shows that many agencies have altered the portal dimensions to address height clearance problems. A larger problem is the issue of bridge width. Figure 3.7 also shows that conversion to one-way traffic and relaxing geometric standards are the most common solutions for width problems. Some states such as Arizona, Nevada, and Oklahoma, have widened truss bridges to meet with current requirements. Further discussion of this topic can be found in Sections 4.3.1.1 and 4.3.1.2.

3.3.8 Question 8

What methods, if any, have you used to improve railings on historic steel truss bridges? We are particularly interested in information on crash tested railings which have been added to historic steel truss bridge.

The responses to this question were varied. Sixteen of the DOTs that returned their survey did not respond to this question. In comparison, for questions 5, 6, and 7 no response was given by only 6, 5, and 5 DOTs respectively. This suggests that the issue of railings is a difficult rehabilitation topic. Of the 23 DOTs which did respond, no clear solution was evident. Figure 3.8 indicates that the W-shape was the most popular retrofit, but not a majority solution. This topic will be discussed further in Section 4.2.1.3.

After the results of the survey were compiled some of the respondents, who indicated the use of new railing techniques in their survey response, were contacted. From these follow-up calls, some innovative solutions were discovered; a hollow tube railing used in Vermont, insight on methods to attach the bridge railings to the deck, and a method of utilizing a concrete barrier with a simulated rail. These solutions are discussed further in Section 4.3.1.3.

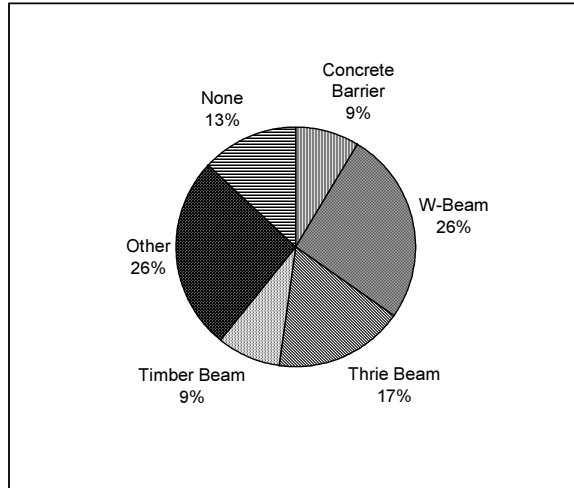


Figure 3.8: Response to Question 8

3.3.9 Question 9

What methods have you used to deal with the presence of lead based paints on historic steel truss bridges:

- _____ Remove old lead paint (with appropriate disposal techniques) and repaint bridge
- _____ Apply sealer to encapsulate lead based paint
- _____ Other (please explain)

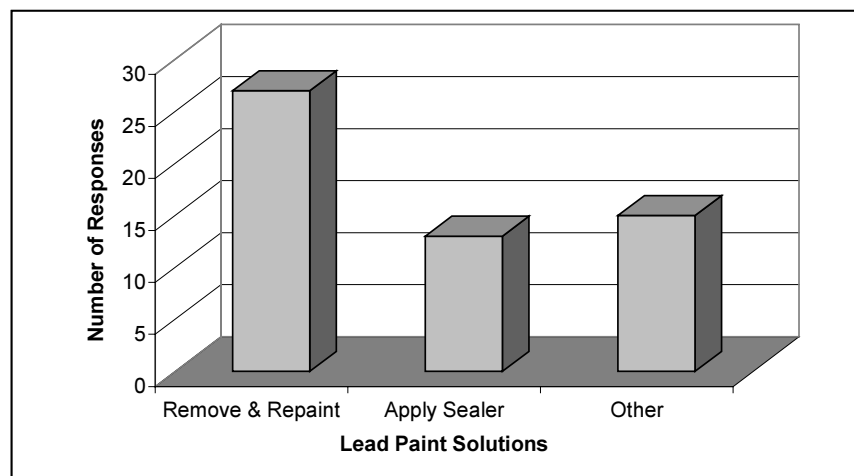


Figure 3.9: Response to Question 9

Lead paints are often a problem for older metal bridges. Figure 3.9 shows that the most common solution involves removing the existing paint and repainting the structure. The responders who indicated “Other” included comments concerning the first two options and other suggestions such as spot painting.

3.4 FINAL COMMENTS

A wide range of information and examples were gathered on a variety of topics related to metal truss bridge rehabilitation. Information collected in the survey is also used in Chapter 4, to provide practical applications of the rehabilitation techniques. The survey also uncovered new rehabilitation techniques that are also discussed in Chapter 4.

Chapter 4:

Summary of Literature Review and DOT Survey

4.1 INTRODUCTION

The objective of this chapter is to summarize and synthesize the information obtained in the survey of Departments of Transportation (DOTs) with information found in the literature review. The discussion in this chapter will involve the division of the problems present in truss bridge rehabilitation into discrete topics. Each topic will consist of an explanation of the problem, pertinent references in the literature, experience of other DOTs from the survey, and a discussion. The topics presented in this section are presented from a structural engineer's point of view. It is important to note that other factors might determine a suitable rehabilitation technique besides structural engineering issues, including cost, historical integrity, and community acceptance. This chapter presents a variety of options for truss bridge rehabilitation so that engineers can be informed of a full range of possible rehabilitation alternatives. The topics covered in this chapter are divided into two sections; Analysis and Testing of Structures, and Deficiencies in Structures.

4.2 ANALYSIS AND TESTING OF STRUCTURES

A primary step in the rehabilitation of structures involves an investigation, evaluation, and understanding of the entire structure. This calls for an understanding of both the structural system and an understanding of material properties. This section will discuss the analysis and testing of both the structural system and individual components of the bridge. A rehabilitation project might be completed by simply performing a more exact analysis or by using an actual yield stress derived from a material test.

4.2.1 Structure

There are several levels of structural analysis and evaluation that may be used for historic truss bridges, including an elementary structural analysis, a more advanced structural analysis, and a load test. Each of these options has benefits and disadvantages. For example, older bridges may have reserve capacity beyond that indicated by an elementary analysis. The source and reliability of factors providing additional strength must be identified. The following section will provide an introduction into analysis techniques for truss bridges and the load testing of bridges. Information from related articles, as well as, responses to questions 2, 3, and 4 of the survey will provide the experiences of DOTs and other researchers.

4.2.1.1 Structural Analysis

Structural analysis of historic truss bridges must not only consider the normal limitations of analysis, but also the added uncertainty of a structure that can be over one hundred years old. The goal is to predict the actual response, and thereby account for it in the rehabilitation process.

Phenomenon must be considered which may not have been originally considered in the design, such as composite action and the stability of the structure. Therefore the calculation of a reasonable but conservative load rating of the structure may be a difficult task.

There are a number of issues that must be addressed related to the analysis of an older truss bridge. There has been much discussion whether a bridge should be analyzed as a two or three-dimensional structure. Stability analysis of the structure and fatigue analysis are other topics that should also be addressed. This section aims to provide background information on the topics of analysis, stability, and fatigue.

4.2.1.1.1 Analysis

When these trusses were designed, it is assumed that only two-dimensional analysis was used. However, gusseted connections do represent a degree of fixity in the joints of these structures in more than two dimensions. A three-dimensional analysis will predict some of the interaction between orthogonal members such as the wind bracing or deck, and the main truss members. Whether or not the extra effort to conduct a three-dimensional analysis is warranted must be established.

The survey responses provided some examples of common structural analysis methods for historic truss bridges. The Kentucky DOT uses the computer program, STRUDL, to analyze their bridges. The Florida DOT has developed their own program, for deck and girder analysis, called BRUFEM. Six other DOTs stated that they had used advanced structural analysis which may be interpreted as three-dimensional analysis. The literature search uncovered articles related to this topic. Reference [3.5.3] presents the analysis of a truss bridge, modeled using 2-D truss, 2-D truss and beam, 3-D truss and beam, and 3-D beam, truss, and deck elements. A discussion of the results is included for the first two models. Article [3.5.4] describes a similar theoretical research effort and concluded that 3-D frame elements predict the behavior of truss bridges most accurately compared to field measurements. Reference [3.3.4] conclusions agree that 3-D analysis provides the best representation of bridge behavior.

4.2.1.1.2 Stability

The issue of stability relates to the capacity of compression members, both individual and global, and their response to load. Compression members are rated with respect to their slenderness ratio, which is calculated as the unbraced length divided by the radius of gyration. Reducing the unbraced length or increasing the radius of gyration of the member increases the compressive strength of a member. The global stability of the structure should also be considered. Because the upper chords of pony trusses do not have out-of-plane bracing, the effective buckling length extends beyond the length of the individual member. The buckling resistance of the overall system must therefore be considered. The stability of through trusses is dependent on the portal bracing connecting the two trusses. Any modifications to the portal bracing (i.e. to increase portal height) should take into account the global stability of the structure.

The survey of DOTs produced few comments related to the stability of truss structures. The Iowa DOT indicated they had added new bracing to decrease the slenderness ratio of

compressive members. The related literature provides some guidance in stability issues. Article [2.4.1] examines possible modifications to a pony truss, and the corresponding effect on the stability of the structure. Reference [3.4.1] also provides an introduction to stability considerations. The calculation of buckling loads of members with uncertain end restraints is examined in reference [3.4.3]. This reference, along with articles [3.4.4] and [3.4.5], describe more complicated analysis for stability situations. Discussion of methods to improve the compressive capacity of truss members may be found in section 4.3.3.3.

4.2.1.1.3 Fatigue & Fracture

Phenomena that may not have been included in the original design of older metal truss bridges include fatigue and fracture. In elementary terms, fatigue refers to damage that arises in a structure when repeated low levels of stress are applied. The fatigue and fracture resistance characteristics of older bridge steels are often much different than their modern counterparts, which were designed with fatigue and fracture in mind. To address this problem, supplementary members are often added to reduce the stresses in a particular member. Bridge members may also experience premature failure due to poor fracture details. Certain members and connections are susceptible to fracture due to poor geometry and low-toughness material. Since the tragic collapse of the Point Pleasant Bridge, much research has been completed in this area. Section 4.3.3.2 provides further discussion of fatigue and fracture with respect to the rehabilitation of truss members.

The responses of DOTs were limited in this area. The Washington DOT stated that they had rehabilitated fracture critical hanger connections by adding a second hanger. The Minnesota DOT commissioned the Sverdrup Corporation to investigate the fatigue capacity of the Wabasha Street Bridge. A copy of this report may be found with the survey responses. The literature search yielded many references related to this subject. Articles [3.3.1], [3.3.4], and [3.3.2] discuss the evaluation of historic railroad bridges related to fatigue and fracture concerns. References [3.3.3] and [3.3.5] both discuss the fatigue evaluation of highway bridge structures.

4.2.1.2 Load Testing

An alternative approach to aid in load rating a bridge structure is the load test. Simply stated, strain gauges and other instruments are attached to a bridge and loads are applied to induce stresses and deformations in the instrumented members. A load rating is then extrapolated from these measurements. If it can be shown through load testing that the bridge is much stronger and stiffer than calculated by structural analysis, then using ratings based on load tests may allow the bridge to remain in service. There are three general levels of loads that may be applied in a test including diagnostic, proof, and ultimate. Diagnostic loading relates to low applied loads, which induce small stresses in the members, to measure the elastic response of the bridge. Proof loading involves loading the bridge to first yield, and thereby establishing a maximum load that can be applied to the bridge. Ultimate loading is used to examine failure mechanisms in the bridge under extreme loads. Ultimate load testing is reserved for bridges which are to be decommissioned. However, load testing of bridges is expensive and the results are often difficult to interpret.

The survey of DOTs revealed that a number of agencies have utilized load testing to assess their bridges. The Minnesota and Nebraska DOTs have sponsored investigations of bridges using load testing. The Arizona DOT stated that they had “used observation of behavior under load, but no formal testing procedure” on bridges in their state. The DOTs of Maryland and Ohio also indicated the use of load testing to rate their bridges. During the literature search, many references related to load testing were found. Article [4.1.6] describes diagnostic testing on a truss bridge. Reference [4.1.5] provides general guidance on load testing of truss bridges and common characteristics of truss bridges including unequal sharing of load by eyebar tensile members, and the rigidity of flooring systems. Article [4.1.7] describes the proof loading of a pin-connected truss. Articles [4.1.1] and [4.1.3] provide further examples of load testing bridges.

4.2.2 Material Testing and Non-Destructive Testing

An alternative to testing the entire structure is the testing of individual members of the system. Two classes of testing include physical and non-destructive testing. Material testing can determine properties such as yield and ultimate strength, toughness, and weldability. When information on the specifications of steel is not available, AASHTO provides assumed allowable stresses based on date of construction that may be used for load rating a bridge. Testing of a metal sample, or coupon, may reveal a higher allowable stress usable in the analysis of a structure. Chemical tests can provide information on other properties such as weldability. Non-destructive techniques may also be used to estimate corrosion loss of a member or to identify cracks or other discontinuities. Discussions of both types of testing are included in the following pages.

4.2.2.1 Material Testing

To accurately assess the capacity of a truss bridge, it is necessary to begin the analysis with accurate estimates of the physical properties of the metal. Taking coupon specimens from the bridge can aid in better characterizing in-situ material properties. Values of yield strength, toughness, ductility, and ultimate strength may be found from a sample of the material.

The survey of DOTs only provided one example of physical member testing. The Maryland DOT indicated that they had “tak[en] coupons from batten plates and have then tested for yield strength”. The literature search, however, provided many examples of both material testing and member testing. Articles [4.2.2] and [2.4.2] describe projects which investigated the material properties of older bridge metals. References [4.1.2] and [4.2.3] discuss the testing of truss bridge stringers and girders. Articles [4.2.1] and [4.3.3] both investigate riveted connection behavior as compared to bolted.

4.2.2.2 Non-Destructive Testing

Another option for investigating the integrity of truss members is non-destructive testing (NDT). For bridge structures, NDT methods must be transportable, cost-effective, and easy to complete. This section will discuss some of the NDT methods that are applicable for truss bridge inspection and evaluation. Two categories of testing are used in bridge inspection including geometry and integrity. Geometric testing involve tasks such as measuring thickness

or corrosion loss of sections. NDT techniques that measure quantities such as crack length and detect defects may be classified as integrity tests. In the following sections, six NDT methods will be briefly discussed along with common applications, advantages, and disadvantages.

4.2.2.2.1 Dye-Penetrant

This technique is used to verify and measure existing surface cracks on a member. The surface to be examined is first cleaned and a dye is applied to the member. A developer is administered which reveals the cracks on the exterior of the member. Dye-penetrant is an inexpensive NDT that does not require a trained professional to successfully examine the member. For these reasons, it is a common method used in bridge inspection. However, a time consuming visual inspection must first be completed to identify the potential cracked areas. Dye-penetrant also does not reveal cracks or flaws in the interior of the member.

4.2.2.2.2 Magnetic Particle

Induced magnetic fields may also be used to examine the integrity of truss bridge members. Applying a current to the member, through the area to be examined, magnetizes the member. Fine, iron powder flakes are applied to the magnetized zone and affix themselves to the member. The iron powder accumulates along flux lines and at locations where discontinuities in the material prevent the smooth flow of current. This method can be used to identify surface flaws, similar to dye-penetrant, but does not locate internal flaws. This method may require professional assistance, raising the cost of the procedure.

4.2.2.2.3 Ultrasonic Testing

The use of sound waves to examine the integrity of the material has become well-established practice. The sound waves are passed through the material and recorded using transducers on the opposite side. Discontinuities, both internal and external, can be identified by variations in the transducer signal. Ultrasonic testing can also be used to measure section thickness or corrosion loss of members. Many inspections of truss pins have been completed using ultrasonic testing. This method is a powerful inspection tool but requires trained professionals to perform the testing.

4.2.2.2.4 Radiography

Radiography operates on a similar concept to ultrasonic testing. Instead of sound waves, X-rays are passed through the member. Film is placed on the opposite side and developed by the X-rays striking the film. Examining the “photograph” can identify discontinuities, since the X-rays are absorbed at different rates in flaws and sound material. Radiography is a powerful tool however it does not reveal the depth of the flaw, and special care must be given when using radioactive materials, especially while examining thick metal plates.

4.2.2.2.5 Magnetic Flux Leakage (MFL)

This method uses the principles, relied upon in Magnetic Particle examination, taken to another level. As discussed previously, discontinuities in the material cause distortions in the flux lines of a magnetized member. The Hall Effect sensor, an electronic device, can be used to identify these discontinuities. MFL has the added advantage of revealing both internal and external flaws in the material. However, the special equipment involved and interpretation of the data tend to elevate the cost of this method.

4.2.2.2.6 Acoustic Emission (AE)

Acoustic Emission monitors the release of energy caused by events occurring in the member. Yielding of the metal, micro-cracking, and other phenomena can be recorded by sensors attached to the material. Noise, such as paint flaking, traffic, and rainfall, is also recorded by the sensors causing difficulties in data interpretation. Filtering programs have been developed to remove much of this noise. The author suggests that this method might be used in monitoring critical connections or members and warning of possible failures.

4.2.2.2.7 Summary

Of the responding agencies, nine stated that they had used NDT in the evaluation of their bridges. A majority of the departments, six, have used ultrasonic testing to verify the condition of pins. The New York DOT has used X-ray diffraction to investigate the stresses present in some members. The Arizona DOT has used hardness testing to verify the metal present in the bridge structure. From the responses, it was inferred that NDT has been reserved for special circumstances and larger projects. As the cost of NDT decreases, more widespread utilization of these techniques will probably occur.

The literature search uncovered some introductory materials related to common NDT practices and applications. Article [3.6.1] provides a thorough introduction to the testing methods as well as a short discussion of the advantages and limitations. Reference [2.1.1] discusses NDT methods such as ultrasonics, magnetic particle, and dye penetration. Reference [3.6.2] gives an example of ultrasonics used to locate lamellar tears in weldments. Reference [3.6.3] is a manual compiled by the National Research Council of Canada to introduce non-destructive methods for the evaluation of structures. Although this document is geared towards building structures, it provides a very thorough discussion of current NDT methods which may also be applied to bridge structures.

4.2.3 Discussion of Analysis and Testing of Structures

Understanding the structural system of a bridge is a key element of a rehabilitation effort. The structure must be closely examined to uncover distressed members, relevant load paths, and how the entire structure will respond to the applied loads. In the structural analysis, the assumptions made in the original design should be checked, as well as assumptions used in modern computer analysis of the structure. Special attention should be given to phenomena such as lateral stability, fatigue, and fracture details which may not have been considered in the original

design. Taking coupon samples can be a valuable aid in characterizing material properties. Before any welding occurs on the structure, a chemical analysis should be performed to establish the material's weldability. NDT should be utilized in critical areas such as non-redundant pin connections or fracture critical tensile members.

4.3 DEFICIENCIES IN STRUCTURES

The repair of historic bridges requires knowledge of many types of retrofit techniques and considerations. This section contains information related to functional deficiencies, damage to structures, and structural deficiencies. These topics cover the majority of difficulties inherent in bridge rehabilitation. The contents of this section include:

Functional Deficiencies

1. Width
2. Height
3. Railings

Damage

1. Corrosion
2. Impact
3. Fire

Structural Members

1. Floor-beams, Girders, and Stringers
2. Tension Members
3. Compression Members
4. Pin Connections
5. Riveted Connections

Structural System

1. Deck
2. Post-Tensioning
3. Additional Load Bearing System
4. Additional Continuity

4.3.1 Functional Adequacy

Functional adequacy deals with the prevention of accidents and fatalities by providing adequate geometric and safety features on a bridge. Functional requirements are thus somewhat different from structural requirements, which deal with strength, load carrying capacity, and the potential for catastrophic collapse. Modification of the metal truss superstructure to correct functional deficiencies such as inadequate horizontal and vertical clearance, or obsolete bridge railings may present greater difficulty than remediation of structural deficiencies. Obstacles to clear in geometric issues might be more complicated than other bridge rehabilitation problems, primarily since changing the shape of a bridge is complicated in a construction sense and may pose difficulties in maintaining the historical integrity of the bridge. A good reference in the thought process involved in solving geometric problems can be found in the report conducted for the Vermont DOT by A.G. Lichtenstein. This report may be found in the collection of DOTs responses. Railing rehabilitation is also frequently a major difficulty in historic bridges. This section will discuss common functional deficiencies such as geometric clearances, both height and width, and railing issues. Information gathered from the survey questions seven and eight as well as literature related to these topics will be presented in each section.

4.3.1.1 Width

Truss bridges frequently do not meet current ASSHTO requirements for width. Solution options include:

- Relaxing Standards
- Widening
- Convert to One-Way Traffic
- Speed Restrictions

4.3.1.1.1 Relaxing Standards

Truss bridges that are not wide enough, according to current standards, are often located on roads with very low traffic volumes. Of the DOTs surveyed, ten have used the option of relaxing standards to deal with width deficiencies. No examples could be found in the literature related to this topic.

4.3.1.1.2 Widening

An alternative solution to width deficiencies involves widening the bridge to comply with current standards. This is primarily done on pony truss bridges rather than through truss bridges whose upper bracing would present further difficulties. States which have successfully widened a bridge include Arizona, Nevada, and Oklahoma. A research project conducted at the University of Nebraska-Lincoln studied the effects of widening of a pony truss. The Virginia Department of Transportation has also considered widening a truss bridge to comply with current

standards [1.10]. Although it is not common practice, an example of widening a through truss bridge has been found [1.6]. The question of lateral stability in a widened truss bridge is examined in reference [2.4.1]. The rehabilitation of a cast iron bridge in England involved widening the walkways and parapets using “hidden portals” [2.6.2]. The figures below show two of the options in widening a bridge. The procedure shown in Figure 4.1 uses supplementary girders to aid in the distribution of loads. Figure 4.2 relies on strengthening of floor beams to distribute the loads to the trusses.

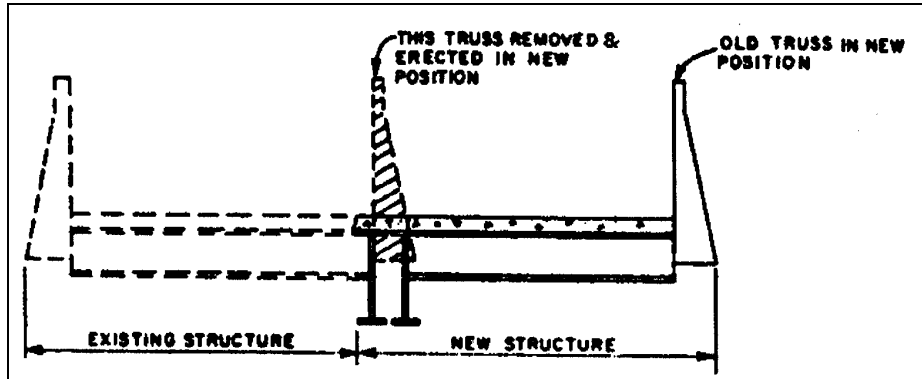


Figure 4.1: Widening of Pony Truss [Ref. 2.4.1]

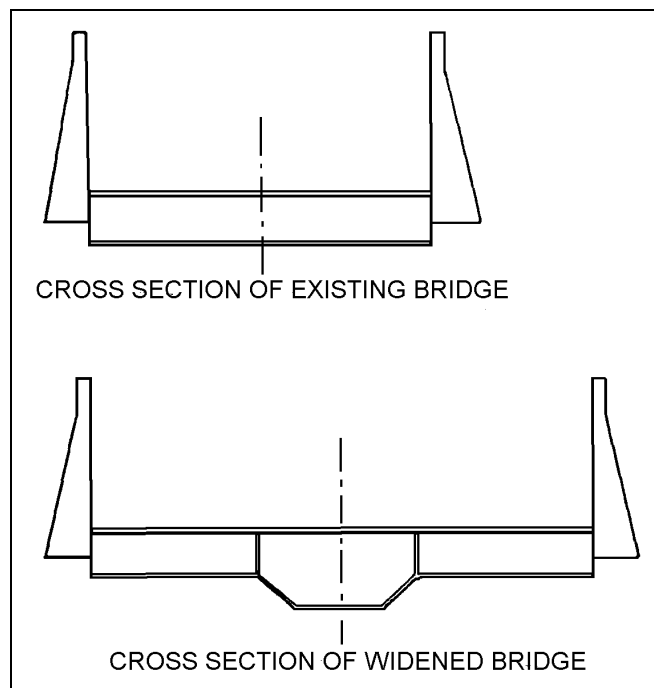


Figure 4.2: Widening Floorbeams of a Pony Truss [Ref. 2.4.1]

4.3.1.1.3 Convert to One-Way

Another common solution involves converting the bridge to one-way traffic. This might involve using alternating one-way traffic, or dividing the road and building a second bridge. In response to the survey, ten states indicated the use of this technique. The literature also supports this option. A bridge in Hawaii was converted to alternating one-way traffic [1.6]. A train bridge in Europe was reduced to one lane of traffic to accommodate future travel [3.3.1]. In the report by Virginia University, conversion to one-way traffic was given a favorable recommendation [1.10].

4.3.1.1.4 Speed Restrictions

A technique for addressing inadequate width is to post a lower speed limit on the bridge. In the survey, California and Oklahoma indicated the use of this method. A reduction in the posted speed limit on a bridge may provide the basis for granting design exceptions for certain functional deficiencies. The engineer may find more information on current AASHTO standards in “A Policy on Geometric Design of Highways and Streets” AASHTO 1990.

4.3.1.1.5 Discussion of Width Deficiencies

From a historic preservation perspective, altering the overall appearance of the bridge should be a last resort. To this end, the three options of relaxing geometric standards, conversion to one-way traffic, or speed restrictions are preferred. Although width requirements do not change with conversion to one-way operations, or with reduction of the posted speed limit, the inherent improvement in safety provided by these measures may form the justification for granting design exceptions for width deficiencies.

4.3.1.2 Height Deficiencies

The portal bracing on the bridge may not provide adequate height clearance according to current standards. Solution options include:

- Remove or Alter Bracing
- Relax Current Standards
- Post height clearance
- Install electronic sensors
- Install warning system

4.3.1.2.1 Remove or alter bracing

The most common solution to height deficiencies is removing or altering the portal bracing. In the survey, twenty-four of the responding states indicated use of this method. NCHRP Report 222, "Bridges on Secondary Highways and Local Roads", provides good examples for common portal modifications, reference [1.14]. Figures 4.3, 4.4, and 4.5 , represent three of the options available in removing or altering portal bracing. Since the bracing members serve to stabilize the frame, allowances should be made to provide adequate stiffness and strength to the altered system. Changes to a truss bridge usually involve altering only a few portals which make stability concerns less of an issue. The report also recommends the use of bolts to replace rivets.

Option 1: Remove all bracing and install new bracing

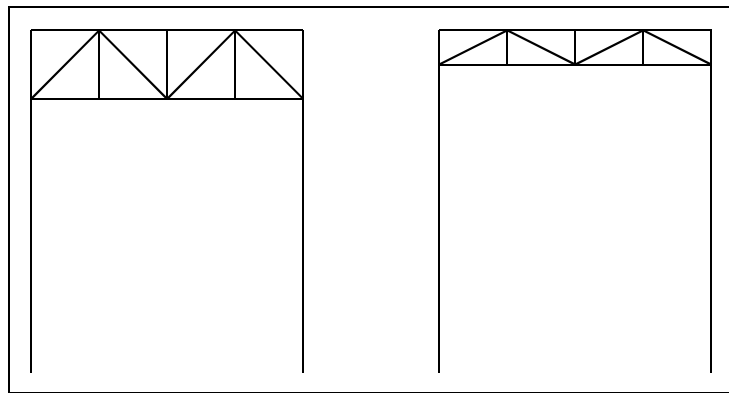


Figure 4.3: Alteration of Portal Bracing 1

Option 2: Shorten existing bracing and utilize existing horizontal member

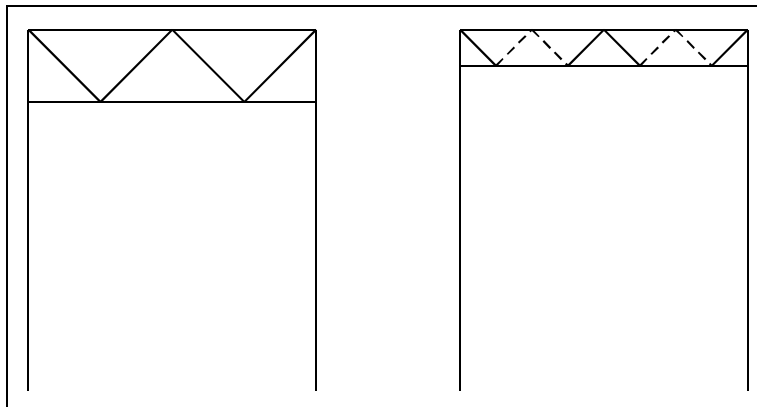


Figure 4.4: Alteration of Portal Bracing 2

Option 3: Removal of knee bracing

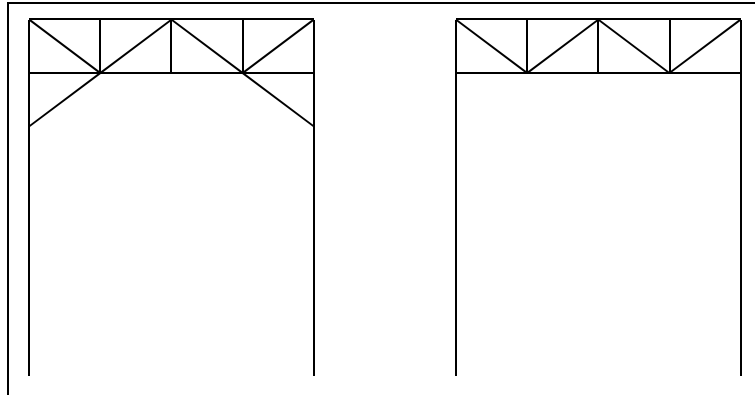


Figure 4.5: Alteration of Portal Bracing 3

4.3.1.2.2 Relaxing Standards

Another option in dealing with overheight deficiencies involves relaxing the standards by the DOT. Eleven of the states surveyed have granted such exceptions. This solution should not be a problem if the bridge has a low daily traffic volume. The bridge should also be posted and/or a warning system implemented to alert drivers if their vehicle might collide with the bridge members.

4.3.1.2.3 Posting

A third option to address an insufficient height clearance is to post a clearance warning sign. The Georgia DOT has used this technique to reduce the chance of overheight collisions on their bridges. It should be noted that often bridge wearing surfaces are repaired by applying a new layer of asphalt or concrete without removing the existing surface. Care should be taken to verify any existing height postings as they might have been placed prior to re-decking.

4.3.1.2.4 Other Warning Systems

The California DOT indicated in their response to the survey the use of electronic sensors placed prior to the bridge to warn any overheight vehicles. Another warning system could involve a physical system placed prior to the bridge. Structures such as parking garages have utilized systems consisting of a sign or chains hanging prior to the entrance, which replicate the maximum height allowable in the structure. A similar system, shown in Figure 4.6, constructed of a simple frame placed before the bridge, would warn drivers if their vehicle has a risk of striking the portal bracing.

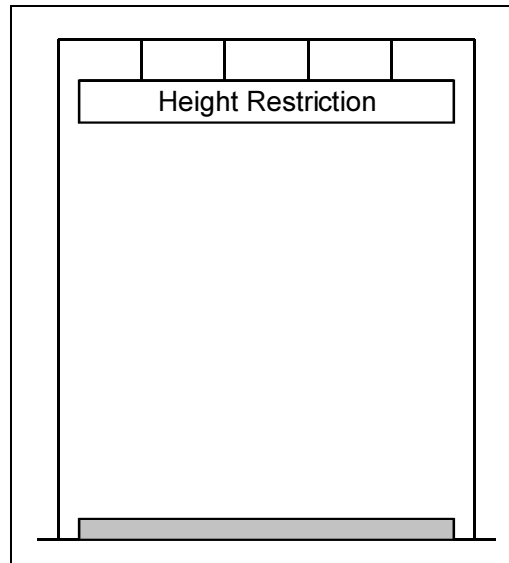


Figure 4.6: Portal Height Warning System

4.3.1.2.5 Discussion of Height Deficiencies

Height clearances should be addressed in bridge rehabilitation projects. Failure to do so might lead to a major accident as demonstrated in reference [2.4.2]. The options for solving height problems fall under two categories; active and passive. The active approach involves changing the bridge layout, while the passive approach keeps the bridge in its original state and warns the approaching vehicles of possible collisions. A method to increase the portal height without altering the bracing might involve replacing the thick concrete or timber deck with a thinner, lightweight concrete or orthotropic deck. This solution may only yield an increase of a few inches, but could prevent a future collision.

4.3.1.3 Railing

A problem that plagues many historic bridges is an inadequate railing system. Often, existing railing systems consist of a negligible member, sometimes only an angle carried across the structure, to serve as a railing. Acceptance criteria for bridge railings are established in NCHRP 350 “Recommended Procedures for the Safety Performance Evaluation of Highway Features” published in 1993. Current FHWA and TxDOT policy with respect to bridge railings are guided by this document. Installing a retrofit railing, which meets current standards but does not interfere with the historic integrity of the bridge, may be difficult. The inclusion of a new railing system often decreases the existing width of the portal, another common deficiency of older truss bridges. The attachment between the new railing system and the existing bridge must be carefully considered. The railing might be fixed to the bridge superstructure, or the deck. There has been very little published material concerning this subject, and even less agreement between DOTs concerning their handling of the situation. During the literature search, a research report was identified titled “Retrofit Railings for Narrow Through Truss and Other Obsolete Bridge Structures” sponsored by the Federal Highway Administration (FHWA) [2.1.5].

This was the only substantial document uncovered, which dealt with the issue of retrofit railings as applied to truss bridges.

The issue of retrofit railings has been broken down into two approaches, including granting exceptions and applying conventional railing systems to the bridge. The railing systems developed in the FHWA research should also be considered. Following the discussion of the two classes of railing repair, a summary of the findings from the FHWA research will be presented.

4.3.1.3.1 Granting Exceptions

One of the options, which appears to be frequently applied, is granting design exceptions for the bridge. This might be justified if very little traffic crosses the bridge each day (e.g. VPD of 50 or less), or is not used by large vehicles such as trucks or school buses. Preservation officials might also object to the use of a standard railing system due to the alteration of the historical characteristics of the bridge. The survey revealed that the Oklahoma and Newfoundland DOTs have exercised the option of not rehabilitating the railings on a truss bridge.

4.3.1.3.2 Conventional Railing

A popular alternative is to apply a conventional railing system, used in new bridge construction, to the rehabilitation of a truss bridge. During the research project, four such railing systems were identified including:

- Thrie Beam
- Concrete Jersey Barrier
- Box Beam
- Timber

In the following paragraphs, each system will be discussed and applications of each will be presented.

4.3.1.3.2.1 Thrie Beam

The thrie beam, also called a W-beam guardrail, is the most popular railing system used in rehabilitating truss bridges. This system has been used in many forms, from attaching the thrie beam to the truss members, to welding reinforcing plates to the back of the plate, and fixing it to the deck. Special consideration must be given to the attachment of the system to the bridge, and how this will affect the bridge in the event of a collision. Attachment to the deck will allow the railing to deform in the event of a collision and may reduce the damage incurred by the truss members. However, attachment of the railing to the deck may decrease the travelable width. The literature search provided a few examples of thrie beam railing systems and their application to truss bridges. NCHRP 222 presents two guidelines for this type of system, reference [1.14].

Repair R-1 titled “Addition of Bridge Rails to Through Truss”, shown in Figure 4.7, describes a procedure in which a supplementary frame is constructed below the floor beams to channel the impact forces into the deck system, instead of the superstructure. However, the report cautions that this system has not been crash tested.

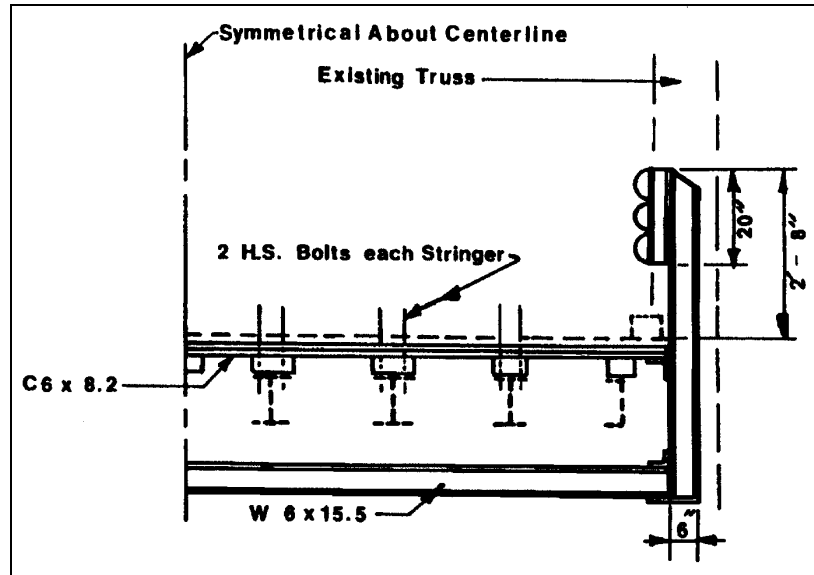


Figure 4.7: NCHRP 222, System R-1 Retrofit Railing [Ref. 1.14]

A second option presented in NCHRP 222, System M-5, is connected to the bridge at the deck and the first two longitudinal beams, reference [1.14]. Figure 4.8 shows this system as applied to an existing bridge. There has been little guidance provided on the applicability of these systems and their responses to crash scenarios.

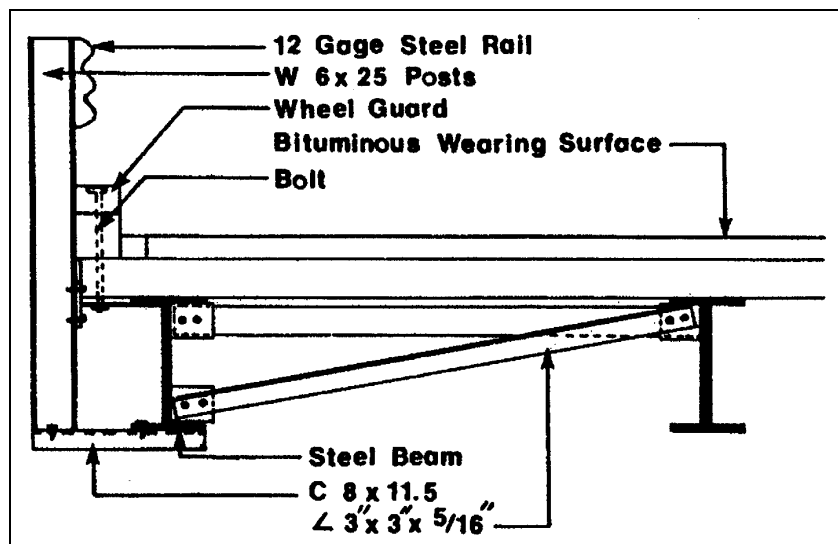


Figure 4.8: NCHRP 222, System M-5 Retrofit Railing [Ref. 1.14]

4.3.1.3.2.2 Concrete Jersey Barrier

This common railing system has been applied to truss bridges to attain a crash tested system. The author assumes that the barrier is typically attached to the deck of the truss similar to new bridge construction. The concrete barrier reduces the clear width of the bridge and significantly impacts the appearance of the bridge. However, the concrete barrier should provide excellent protection to the truss structure. In response to the survey, the Alaska, Arizona, and Connecticut DOTs indicated their agencies had used this system on truss bridges. A design guideline in NCHRP 222 System R-5 describes the application of a concrete barrier to an existing bridge deck, reference [1.14]. Figure 4.9 displays the implementation of a concrete barrier to an existing bridge deck.

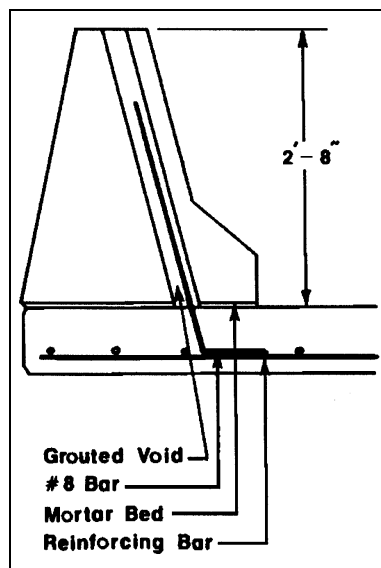


Figure 4.9: NCHRP 222, System R-5 Retrofit Concrete Railing [Ref. 1.14]

4.3.1.3.2.3 Box Beam

The use of box beam members as a railing system is another option in the rehabilitation of truss bridges. The system is similar to the thrie beam, but instead uses a tubular steel member to span between the posts. This railing system may be more acceptable from a historic preservation perspective, since the box section would merge better with the existing structure. The survey of DOTs revealed that three agencies including Montana, Vermont, and Rhode Island have used box beam railing on truss bridges. Figures 4.10 and 4.11 show box beam railing systems used by Vermont and Rhode Island, respectively, in the rehabilitation of a pony truss.

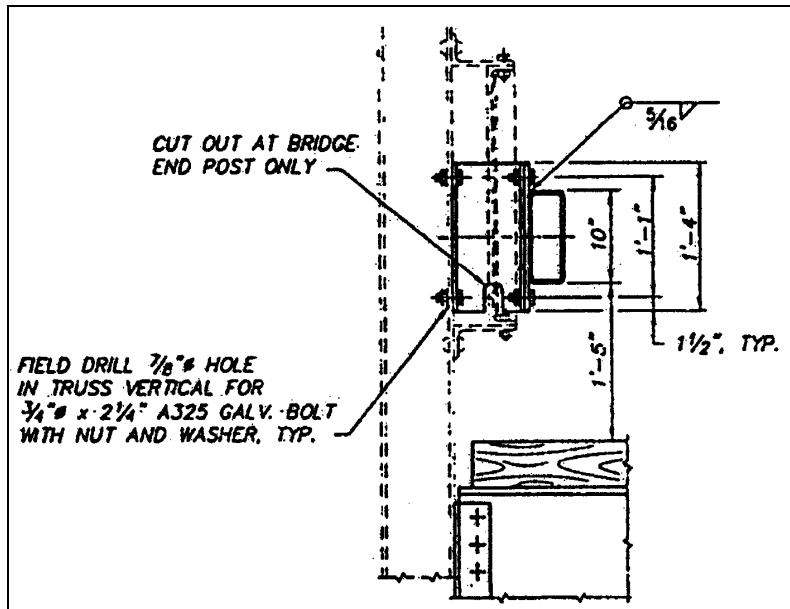


Figure 4.10: Vermont Box Beam Railing [Vermont DOT]

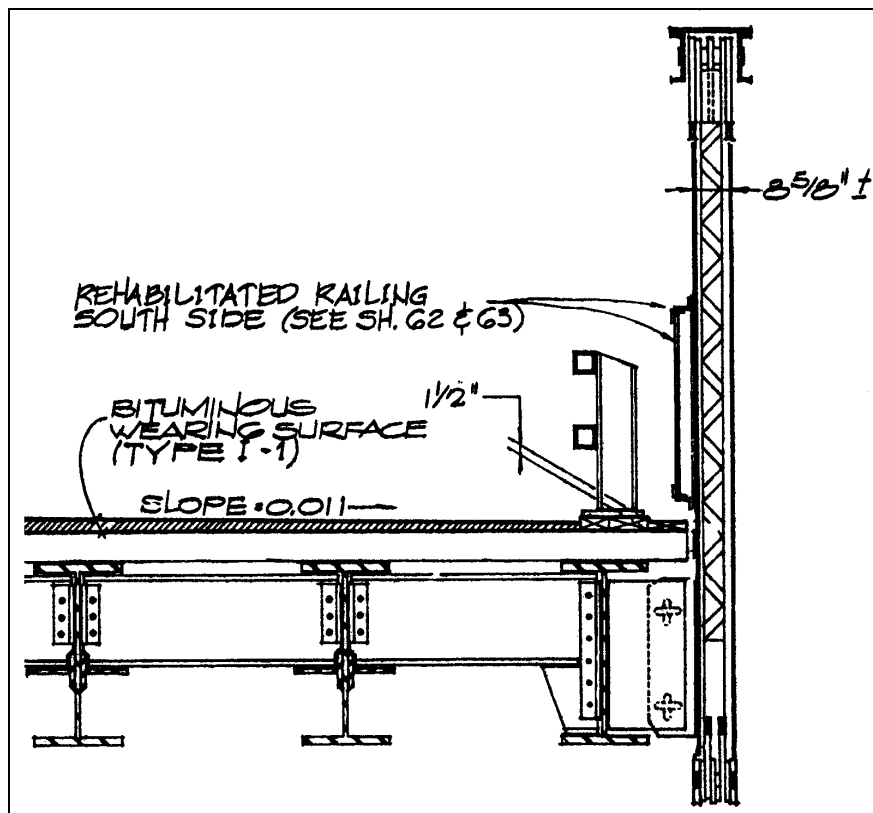


Figure 4.11: Rhode Island Box Beam Railing [Rhode Island DOT]

4.3.1.3.2.4 Timber

Railing systems using timber products are not as popular as other systems, but may provide a more aesthetically pleasing solution. The Delaware DOT stated that their state has used timber railings in the rehabilitation of their truss bridges. The Maryland DOT indicated their agency has used a timber rubrail to “take the brunt of the wheel load and deflect the vehicle back into the roadway before striking truss members or hand railing.” No crash tested timber systems could be found. However NCHRP 222 System M-5 provides an example of this system applied to a bridge structure as seen in Figure 4.12, reference [1.14].

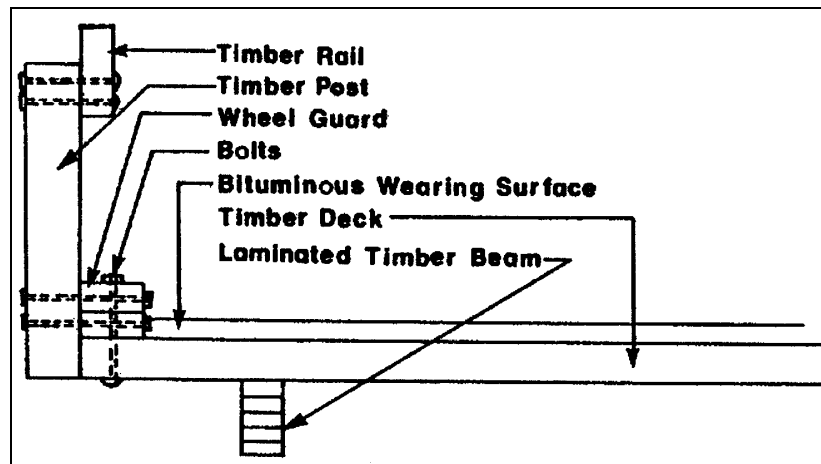


Figure 4.12: Timber Railing System [Ref. 1.14]

4.3.1.3.3 Findings of FHWA RD-82/099 [2.1.9]

The Southwest Research Institute investigated two railings. The first, a high performance railing (HP), was to be designed so a 20,000 lb. vehicle traveling at 55 mph, impacting the railing at 15°, would not damage the truss. The second, a low performance system (LP), was designed for a 4,500 lb. vehicle traveling at 60mph with an incident angle of 15°. The objective was to design a rigid system to reduce deflections, and therefore clearance requirements, and one that weighed less than a standard concrete barrier (300 lb./ft). The connection between the railing and the bridge deck was found to be very important. The research report provides further details concerning the crash testing.

The HP railing consisted of a lower and upper barrier that would both deflect the vehicle and prevent overturning. A battery of tests was completed on the HP railing system. A modification of the original design introduced a self-restoring lower rail as shown in Figure 4.13. The modified HP railing performed very well (under previous standards of acceptance for bridge railing) under a series of crash scenarios.

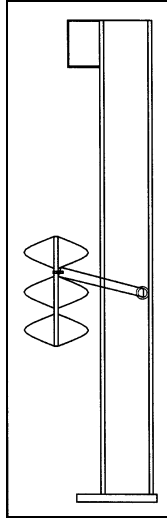


Figure 4.13: High Performance Railing System [Ref. 2.1.9]

The LP railing, shown in Figure 4.14, was comprised of a thrie beam attached to a W6x8.5 section and fixed to the deck with a baseplate. After extensive testing, it was concluded that the LP railing also responded satisfactorily (under previous standards of acceptance for bridge railing) to various crash situations. It was recommended that the LP railing be used only in cases of:

- Single lane bridges
- Narrow two-way bridges (less than 20 ft.)
- Bridges with only automobile traffic
- Bridges posted for less than 35mph with truck or bus traffic

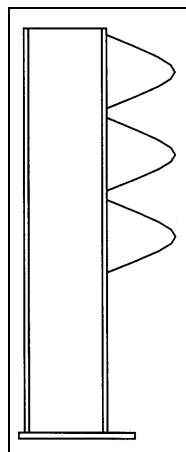


Figure 4.14: Low Performance Railing [Ref. 2.1.9]

4.3.2 Damage

A phenomenon present in older bridges is damage that has occurred throughout the life of the structure. This damage might be due to collisions with vehicles, acts of nature, or other incidents. Collisions with vehicles might have resulted in fractured or severely deformed members. Corrosion might have occurred on the bridge due to harsh environmental conditions, or the bridge's age. There have even been instances in which fire has damaged the bridge structure. In all of these cases, it is necessary to have the tools to assess the damage as well as rehabilitation techniques to repair the damaged bridge members.

In addressing any damage scenario three steps must be completed prior to undertaking rehabilitation:

1. Identification of member damaged (including metal type and properties)
2. Determination of extent of damage
3. Choice of rehabilitation technique

The first step involves deciding what type of member is damaged (e.g. tension, compression, fracture critical, etc.). The steel type and properties should also be quantified. This might be as simple as examining the construction records for the bridge, or as complicated as taking samples for chemical analysis. The use of non-destructive testing could also aid in this step. Further discussion of these techniques can be found in section 4.2.2.2.

The determination of the extent of damage is dependent upon the type of damage. Section loss should be evaluated on corroded members. Strain calculations may be necessary in impact or fire scenarios. A discussion of evaluation methods specific to each type of damage is included in the following sections.

The choice of rehabilitation technique is also damage type dependent. To resolve a damage situation there are three options to consider including:

- Do nothing
- Repair member
- Replace member

The solution might be as simple as a more in-depth analysis, or as complicated as providing temporary support and replacement of a member. In the subsequent sections, discussions of rehabilitation techniques will be included for each damage type.

4.3.2.1 Corrosion Damage

Extensive corrosion might have occurred to one or many members due to environmental conditions or age of the structure. Corrosion, by definition, involves the oxidation of a metal, or in other words, the conversion of a solid metal into ions of the metal, which in turn combine with

other elements to form rust. A more complete discussion of the mechanisms involved in corrosion may be found in reference [3.2.1]. There are many types of corrosion that might be present in bridge members including galvanic, pitting, and crevice. The mechanisms for these varieties of corrosion may differ. However, the end result is identical, a net loss in available section to carry the applied loads. Common areas in which corrosion can be exacerbated include:

- Top flange of floor-beams due to roadway leakage
- Bottom flanges of members due to debris build-up
- Lacing bars
- Pinned connections where small relative movements might trap moisture

4.3.2.1.1 Evaluation

The first step in dealing with corrosion is to measure the section loss in the member. The member may be measured using calipers or other appropriate methods for totally exposed sections. If in the case of floor-beams, in which a portion of the member might be imbedded in concrete, NDT methods such as ultrasonic inspection might be employed to evaluate the remaining section. An excellent reference for the evaluation of corroded members can be found in reference [3.2.1]. In this article, a series of tests were completed to quantify the performance and remaining strength of corroded bridge members. This research confirmed the common approximation that the strength of a member can be determined by multiplying the remaining area by the ultimate stress. The researchers also found that corrosion did not affect ductility.

Fatigue tests were also completed to quantify the influence of corrosion on fatigue strength. It was found that if the corrosion loss was less than 50%, notch effects rather than section loss governed fatigue capacity. Other articles, which discuss properties and evaluation of corroded members, include [4.1.2], [4.1.3], and [4.2.1].

4.3.2.1.2 Solution – Do Nothing [Section loss < 15%]

Although this option might seem elementary, a common solution to corrosion problems is to leave the situation alone. It might be advantageous to take a sample of the steel to confirm the strength of the material in question. Often, the tensile tests will produce yield strengths higher than assumed, which in turn provides extra confidence in the existing structure, and justifies the “Do Nothing” option.

4.3.2.1.3 Solution – Repair Member [Section loss < 40%]

If the corroded member is moderately deteriorated, a repair of the member might be warranted. The type of member also plays a part in the determination of rehabilitation technique. For example, adding new steel plates can strengthen a corroded tension member. Conversely, a compression member can be reinforced using coverplates, or by post-tensioning the system.

Although repair of corroded members was not explicitly asked in the survey, two states, Connecticut and New York indicated they had repaired corroded members. Corroded connections may also be strengthened by member repair. Examples of these techniques may be found in NCHRP 222, Repairs S-4 and S-3, reference [1.14].

Repair S-4 provides an example of the repair of an end of a beam member deteriorated by corrosion. The method involves cutting out the corroded area and welding new material in its place. It should be noted that welding of older steels is a complicated issue. The older metals such as wrought iron and pre-A7 steel were produced before welding became commonplace and may not be as weldable as A36 and other current metals.

Repair S-3 documents a rehabilitation of one channel in a two-channel built-up bridge member, a common occurrence in older bridges. The weakened area is bridged with a splice plate and attached to existing members with high strength bolts which replace the existing rivets. Reference [2.4.3] describes the repair of the historic Eads Bridge in St. Louis whose floor system had been badly deteriorated by corrosion. A feasibility study was conducted on different rehabilitation techniques. The final solution involved a combination of replacement of the bottom flange and coverplating less corroded flanges. Some examples of coverplated, bolted repairs for bridge members are shown in Figure 4.15.

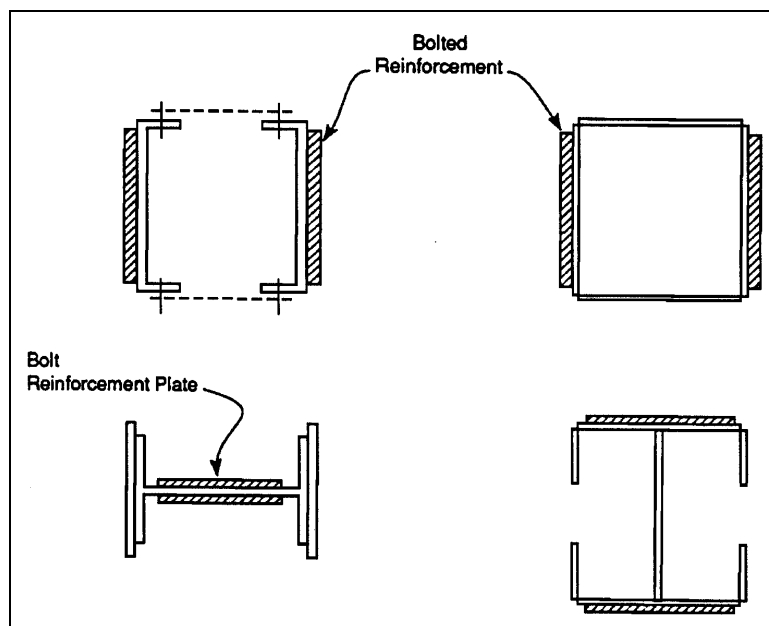


Figure 4.15: Bolted Repairs for Corroded Members [Ref. 1.12]

4.3.2.1.4 Solution – Replace Member [Section loss > 40%]

If the member has been severely corroded, replacement might be the in order. Steps in replacing a member include:

1. Support structural system before removing member
2. Remove damaged member
3. Add replacement member
4. Remove supporting system

Examples of these steps can be found in NCHRP 222, reference [1.14]. These rehabilitation schemes could also be used for severely impacted or fire damaged members. Repair S-1 outlines the replacement of a diagonal tension member in a truss shown in Figure 4.16. Repair S-2 provides an example of a replacement scheme for a vertical tension member. If a floor beam has been attacked by corrosion and requires replacement, repair S-5 furnishes an example of such a repair.

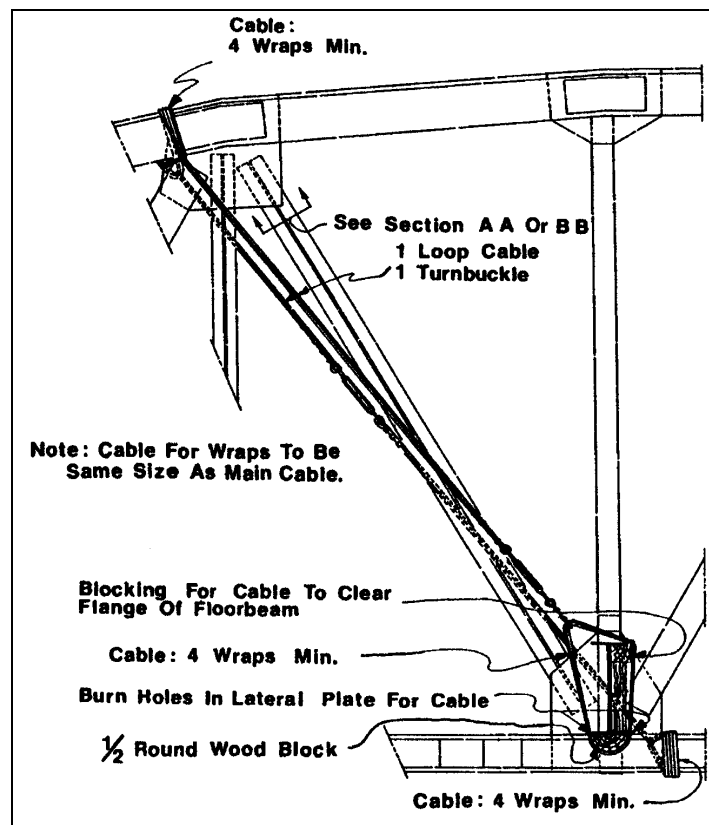


Figure 4.16: Replacement of Diagonal Tension Member [Ref. 1.14]

4.3.2.2 Impact Damage

Bridge member has been struck and damaged by a vehicle. An example of a impact damaged bridge is shown below in Figure 4.17.

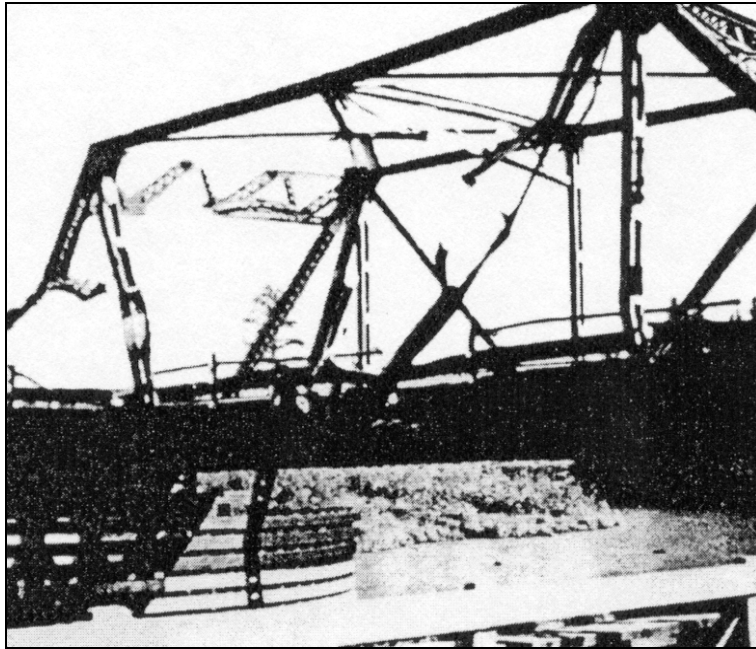


Figure 4.17: Impact Damage to Truss Bridge [Ref. 1.13]

4.3.2.2.1 Evaluation

In approaching an impact-damaged member, the steps outlined in section 4.3.2 should be followed. After the type and properties of the member have been established, an estimate of the damage suffered by the member should be calculated. NCHRP 271 titled “Guidelines for Evaluation and Repair of Damaged Steel Bridge Members” provides a detailed discussion of methods to determine strains associated with impacts, reference [1.13]. These calculations provide information which aids in choosing a suitable repair method. Once the damage has been assessed, the solution to the damaged situation must be decided upon. The three options of “Do nothing”, “Repair member”, or “Replace member”, must again be considered.

4.3.2.2.2 Do Nothing

As in the corrosion case, there are certain situations in which it is acceptable to not repair the damaged member. For example, a tension member which has been bent, but not cracked, would probably present no threat to the continued service of the bridge structure. Minor impact damage to secondary members or portal bracing might also not warrant repair or replacement.

4.3.2.2.3 Repair member

If the damage is judged severe, but repairable, an appropriate method must be chosen. Possible techniques to repair impact damaged members include:

- Flame Straightening
- Cold Mechanical Straightening
- Welding
- Bolting

In the following pages, an explanation of each technique will be given. Since some of these methods are only recommended for certain members, a discussion of limitations and examples of each will also be provided.

4.3.2.2.3.1 Flame Straightening

This repair method utilizes the property of thermal expansion to straighten a member. If the member is heated, using a torch, to temperatures below 1200 °F, the member may be repaired without degrading the physical properties of the steel. The restraint provided by the unheated portion of the member, coupled with the number and pattern of heating cycles, work to straighten the member. In general all types of truss members may be repaired using flame straightening. The literature suggests a few limitations on the use of flame straightening including:

- Do not apply to fracture critical members
- Use caution with primary tension members
- Do not use in cases of excessive curvature, wrinkling, or kinking
- A514 and A517 steels should not be flame straightened

This technique is supported both in the literature and by the DOTs surveyed. Reference [2.9.2] describes an application of flame straightening on a damaged bridge member in Boston. In chapter 5 of NCHRP 271 a detailed description of possible repair scenarios is presented, reference [1.13]. Examples of repairs to three impacted bridges are given in the chapter. All three examples stem from the experiences of the Washington DOT. One repair involved a deck truss that had been impacted by a barge-mounted crane passing beneath the bridge. The repair was completed in three weeks at substantial savings as compared to a more conventional repair. In the survey of DOTs, there was no explicit question concerning flame straightening. However the Rhode Island DOT indicated the use of flame straightening in the past. NCHRP 271 also indicated that the Alaska DOT had successfully used flame straightening in repairs of their bridges, reference [1.13].

4.3.2.2.3.2 Cold Mechanical Straightening

An applied force, without the inclusion of heat, can be used to straighten a damaged member. Research has been conducted which indicates that a member may be repaired once using this technique without degrading the physical properties of the steel, see reference [2.1.4].

NCHRP 271, reference [1.13], provides guidelines in the use cold straightening including recommendations of damage limits which can be summarized in the following manner:

There is no damage limitation on the use of cold straightening in:

- Compression members
- Secondary members

If the plastic strain is estimated to be less than 5% of the normal strain, than cold straightening can be used in:

- Tension members
- Fracture Critical members

Cold Mechanical Straightening should not be used in the cases of:

- Cracks, nicks, or gouges in the member
- Steels with low Charpy values
- Twisted or rotated members

The Texas DOT has performed cold straightening on damaged members. NCHRP 271 indicated that the Transportation Departments of Illinois, New York, and Washington have also used cold mechanical straightening to repair damaged members, reference [1.13]. No other responses were found in the survey of applications of this technique.

4.3.2.2.3.3 Welding

The use of welded plates to repair damaged bridge members is a complicated issue. Older bridges were often constructed of pre-A7 steels or possibly wrought iron. These metals were created before the advent of welding technology and therefore the materials were not designed specifically for welding. There has been much discussion concerning appropriate situations for the use of welding repairs. Reference [2.1.1] discusses the possible problem associated with welding older steels. The author states that the intense heat present in welding might cause cracking in the lamellar structure of older steels. This could lead to fracture of the member, eclipsing the benefits of the welded plate. It is the recommended that the following guidelines be followed in the welding of older steel structures:

- A36, and other current weldable steels may be repaired using welding
- A7 compressive members may be repaired but only by experienced individuals

Welding should be approached with caution for:

- Wrought iron or pre A7 steels

- Steels with low Charpy values

Welding of plates to damaged or understrength members is a common practice as demonstrated by articles and the responses of the DOTs. NCHRP 271 indicated that Illinois had successfully removed a damaged portion of a girder and welded a replacement plate in its place, reference [1.13]. In the survey, the Georgia DOT explicitly stated that they had used “coverplates for damaged areas”. Four other DOTs indicated they had welded plates in strengthening or repairing damaged members.

4.3.2.2.3.4 Bolting

The repair of impact damaged members using bolted splices is a common technique. Bolting may be used on any member provided that both net and gross section failure mechanisms are checked. Many instances of bolting repair of impacted members were found in the literature. Reference [2.4.2] describes a Warren truss whose bottom tension chord fractured due to a progressive failure initiated by an impact. The bottom chord was repaired using a bolted splice across the fracture. Reference [2.4.4] is an account of a girder which cracked and was repaired using bolting. Reference [2.4.5] is another example of a bolted repair of an impact damaged member. Figures 4.18 and 4.19 show the member before and after the bolting repair. The survey of DOTs revealed that many of the agencies have used bolted coverplates for member repair. While only four DOTs stated specifically the use of bolting, many others indicated use of coverplates to strengthen members. Bolting additional plates to a member is also a technique often used in strengthening inadequate members. Further discussion of bolting repairs and their relation to strengthening can be found in section 4.3.3.1-3.

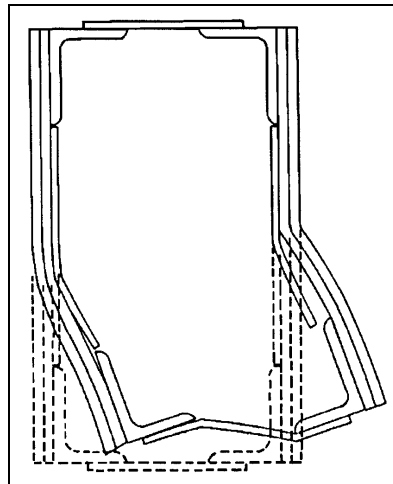


Figure 4.18: Impact Damaged Member [Ref. 2.4.5]

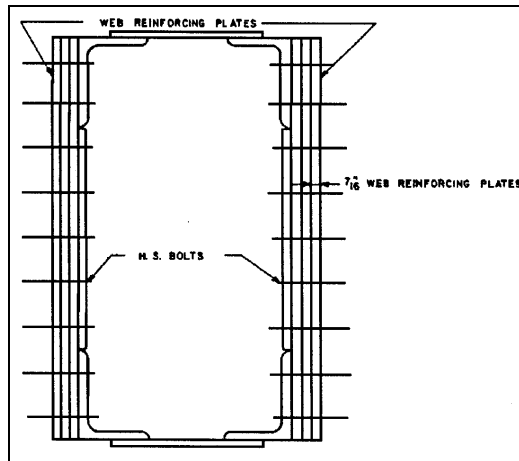


Figure 4.19: Bolted Repair of Impact Damaged Member [Ref. 2.4.5]

4.3.2.2.4 Replace Member

As in the case of severely corroded members, replacement might be the only viable option. The replacement of impact damaged members is no different than corroded members. Section 4.3.2.1 provides guidance for these circumstances.

4.3.2.3 Fire Damage

A third form of damage, which occurs to bridge members, is fire. If steel members have been exposed to temperatures greater than 1200 °F, the properties of the metal might be altered. Plastic deformations in tensile members and premature buckling in compression members can result.

4.3.2.3.1 Evaluation

Fire damage results in deformations of the member due to the intense heat. An evaluation of fire damage should determine which section of the member has experienced high temperatures. The literature suggests sampling the area in question to determine if the material properties have been affected. After an assessment of the damage, one of the three options discussed previously should be chosen.

4.3.2.3.2 Do Nothing

If coupon tests indicate that no significant loss in strength is present, then the member may be left in place.

4.3.2.3.3 Repair Member

More serious damage might require a utilization of one of the repair techniques discussed in the impact repair section. Ironically, flame straightening has been used, with success, to repair fire damaged members, as described in reference [2.9.1]. Partial replacement has also been used to rehabilitate fire-damaged members as shown in reference [2.1.3]. In this rehabilitation, the fire-damaged area of the bridge was removed and new material was welded in its place.

4.3.2.3.4 Replace Member

As a final option, total replacement of the member might be necessary. Replacement methods described in Section 4.3.2.2 are applicable to fire damaged members.

4.3.3 Strengthening Structural Members

It is often the case that historic bridges are understrength, relative to the current standards. These bridges were often constructed prior to development of national standards, or designed using outdated capacities such as H-10 or H-15 loadings. In addressing structurally deficient bridges either a system or member approach may be taken. If the entire structure is inadequate, altering the entire structural system might be in order. On the other hand, if one or a few members are understrength, then rehabilitation might address only those members. The three options of do nothing, strengthen member, and replace member, are available in dealing with structural deficiencies. For the purposes of this discussion, options to strengthen the deficient bridge members will be presented, as replacement should be the last option.

Another decision necessary is the extent of the structural rehabilitation. Three strengthening levels are available including original design, better than original design, or current standards. The decision of the extent of the rehabilitation is dependent on location of the bridge, use of the bridge, and available funds. If the bridge is on a major highway, the rehabilitation should likely increase the capacity to current standards. Strengthening to a lower capacity, appropriate to its usage, may be in order for a truss bridge on a county road.

In the following pages, strengthening techniques applicable to different truss members are presented. Examples of these techniques in practice will be supported both in the literature and from responses to the survey. The techniques presented are intended to provide a starting point for an engineer approaching a rehabilitation project. The historic preservation ramifications of each technique must also be addressed. The types of structural members and connections that will be discussed include:

1. Floor Beams, Girders, and Stringers
2. Tension Members
3. Compression Members
4. Pinned Connections
5. Riveted Connections

4.3.3.1 Floor Beams, Girders, and Stringers

Possibly the most common structural deficiency in older truss bridges are floor beams and other flexural members with inadequate strength. These members were primarily designed as simply supported members to accept distributed loads and transfer them to the main truss members. Loading requirements now call for heavier loads, more complicated loading patterns, and more severe point loads. It is under these point loads that flexural members frequently do not exhibit adequate strength. There has been some disagreement on the behavior of these flexural members pertaining to their apparent fixity with the main members of the truss. One of the references investigated this phenomenon and concluded that though some fixity is present, the overall behavior should be categorized as simply supported [4.1.6]. To strengthen existing flexural members, there are many options available. An important question for all strengthening techniques is, if dead, live, or both types of stress will be reduced. The interaction of the flexural members with other elements in the structure should also be considered. Five strengthening methods will be discussed in the following sections including:

- Coverplating
- Post-Tensioning
- King Post
- Composite Action
- Additional Members

4.3.3.1.1 Coverplating

Coverplating is a well-recognized method of increasing the flexural capacity of a member. The coverplates are either bolted or welded to the existing member to increase the section modulus of the member, thereby decreasing the stresses present in the member. Again, welding of older steels must be approached with caution. Welding might cause delamination of wrought iron or fatigue cracking at the ends of coverplates. In the survey of DOTs, twenty of the agencies indicated that they had used coverplating in the strengthening of flexural members. Of the DOTs, three specifically indicated using welding of the plates, while two designated bolting. The Maryland DOT used channels bolted to the web of the flexural member to increase its capacity.

The literature search uncovered many examples of coverplating weak flexural members. NCHRP 293 provides a general overview of coverplating techniques and examples of these methods, reference [1.15]. Included in this report are examples from Iowa of a bolted coverplate repair using angles. Article [2.4.3] described a rehabilitation effort in which the bottom flanges of floor-beams were strengthened using coverplates. Reference [4.1.3] related a repair in which coverplates were welded onto existing members. Although the steel was pre-A7, the effort was successfully completed. In a report published by the Virginia DOT, bolted coverplates were recommended, reference [1.16]. An actual repair guideline is available in NCHRP 222 Repair R-2, reference [1.14]. This guideline utilizes welding. However the report specifically requires only applying this technique to weldable steels.

4.3.3.1.2 Post-Tensioning

This technique relies on a supplementary element to apply a negative moment thereby reducing the flexural stresses in the member. Post-tensioning bars or pre-stressing tendons are often used to apply the tensile forces to the system. These systems increase the allowable service loads in the member, but do not significantly affect the ultimate capacity of the element since there is a small amount of added material. A simple diagram of a typical post-tensioning system applied to a flexural member can be seen in Figure 4.20. Special consideration should be given to the concentrated forces present at the anchorage locations and how these forces might affect the member. In response to the survey only one agency, the New York DOT, indicated the use of post-tensioning for floor beams.

The literature search exposed more examples of the use of post-tensioning for floor beams. In NCHRP 293, examples of different tendon patterns and a design example for a typical retrofit are presented, reference [1.15]. Article [2.2.4] presents an investigation into stress calculations for a variety of post-tensioned floor beam systems. An increase of capacity of up to 90% was realized in their calculations.

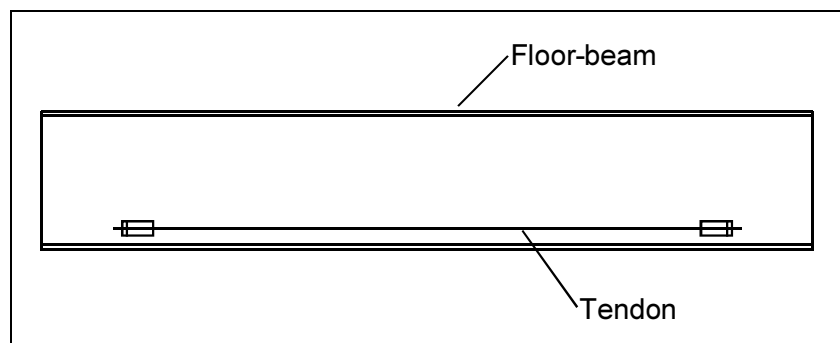


Figure 4.20: Post-tensioned Floorbeam

4.3.3.1.3 King Post

The application of a king post utilizes the principles of post-tensioning, but applied in a different geometry. King posts form a triangular shape stemming from a bracket located at midspan of the beam separating the tendon from the flexural member. The primary benefit of the king post is the small axial force in the tendon, relative to the high negative moment applied to the beam. The king post method has been used on timber bridges since the 19th century. A limitation of the king post technique is the reduction of clearance under the beam. None of the responding DOTs had used king posts to strengthen their members.

The literature provides some credence to the application of king posts to flexural members. In a research report by the Virginia DOT, reference [1.16], this method was suggested as an option and references to king posts being used in practice were included. In NCHRP 222 Repair R-5, a guideline for king posts applied to flexural members is presented, reference [1.14]. Figure 4.21 shows a king post arrangement used in strengthening a floorbeam.

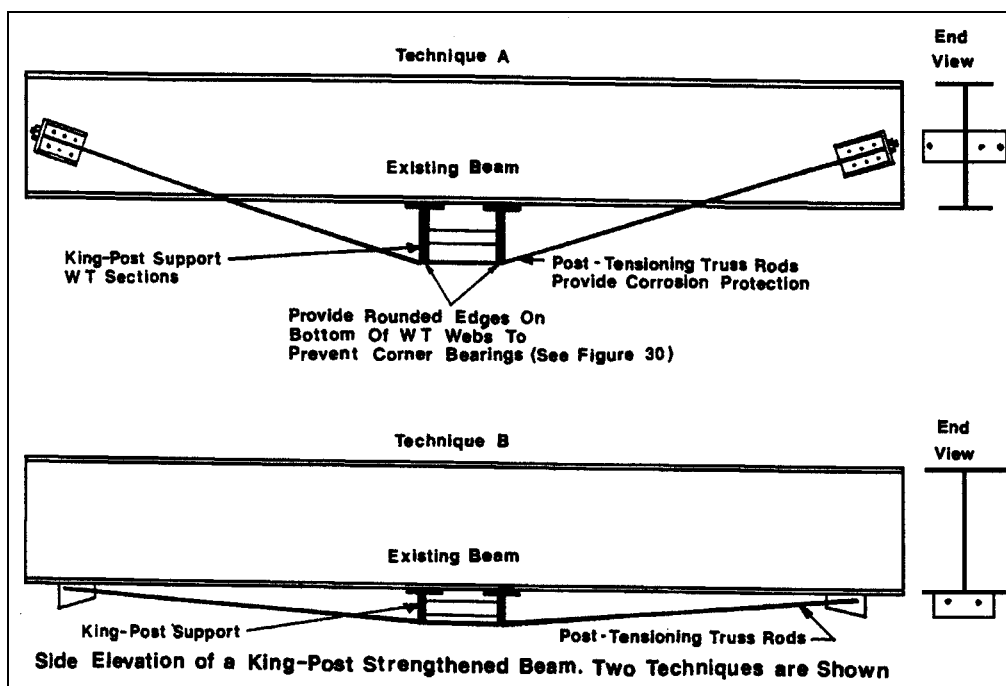


Figure 4.21: King Post Arrangements for Floorbeams [Ref. 1.13]

4.3.3.1.4 Composite Action

The use of composite construction was not developed until the middle of the 20th century. For this reason, almost every historic truss bridge was not designed with composite action in mind. In elementary terms, composite action is present when shear is transferred between a concrete slab and steel beams. Shear is transferred using a mechanical system, usually welded studs. The composite beam is much stiffer and the resultant flexural stresses in the steel are less than a non-composite beam. There has been much discussion on the presence, or lack, of composite action in older bridges due to the apparent bond between the concrete and steel girders. A research project conducted in Canada addressed this issue as well as other topics in bridge testing [4.1.5]. The researchers found that composite action was present at service level loads, while the bond between the concrete and steel was complete. However, as more severe loadings were applied (e.g. ultimate loads), the bond was compromised, and the system behaved as non-composite construction. Therefore, to ensure composite action in rehabilitating older bridges throughout the loading spectrum, a shear transfer mechanism must be provided. The most common way to accomplish this requires removing the existing deck, attaching shear studs to the flexural members, and placing a new concrete deck. It should be noted that welding of older bridge steels might deteriorate the material. However, welding of old steel, in this case, is not as critical since the welded area is in compression. Of the DOTs surveyed, four indicated that they had used composite action to strengthen their truss bridges.

The literature also provides examples of this method in practice. NCHRP 222 discusses providing composite action, and gives a procedure for removing an existing deck, attaching

studs, and placing a new deck (see Repair S-3), reference [1.13]. The report states that a reduction of 15-30% might be realized in the flexural stresses of the member. NCHRP 293 also reviews the procedure involved in making a deck composite, reference [1.15]. The report also outlines two additional methods in achieving composite action. The first involved removing the existing deck, attaching shear studs, and placing pre-cast panels with holes in them onto the girders. The holes are then grouted to complete the process. Figure 4.22 displays this technique as applied to existing girders. The second technique requires holes to be drilled through the existing concrete deck to expose the girder. Shear studs are then welded to the girder and grout is placed in the holes.

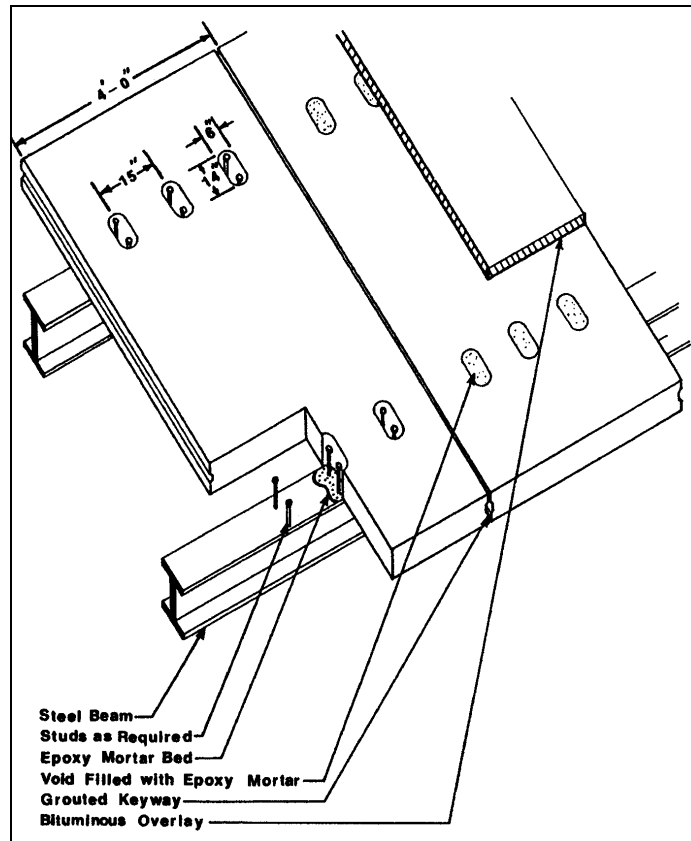


Figure 4.22: Composite Action Using Pre-cast Concrete Panels [Ref. 1.14]

4.3.3.1.5 Additional Flexural Members

Another option in reducing stresses present in flexural members is the addition of new members. The new members act to reduce the applied loads in the existing members and also will distribute the forces more evenly to other bridge elements. An important consideration is the stiffness of the new members. Similar stiffness should be present in the old and new members so that the applied loads will be resisted equally. The addition of new members is most commonly undertaken during a deck replacement. With the deck removed, easier access is available to the existing flexural members. It is recommended that if additional stringers are added as a part of a deck replacement, they should be made composite with the new deck.

The survey of DOTs revealed that the technique of adding flexural members is widely accepted. Ten agencies indicated the use of additional flexural members in the rehabilitation of bridges. The Saskatchewan, North Carolina and California DOTs specifically stated that they had installed additional stringers to bridges, while the other seven DOTs indicated the use of additional longitudinal beams. The literature provides added information about the use of additional flexural members. NCHRP 293 presents a thorough discussion of the advantages and difficulties in adding flexural members, reference [1.15]. The Virginia DOT, in their research report, reference [1.16], also recommends this technique as a viable option. Reference [2.6.2] describes a bridge in England whose sidewalks and parapets were strengthened using additional flexural members.

4.3.3.2 Tension and other Fracture Critical Members

The capacity and reliability of tension members in a truss is essential in the continued service of the structure. The repair of these members should be undertaken with special care and consideration. In older truss structures very little redundancy was typically included in their design. Many of the truss bridges rely on eye-bar tension members that require strengthening to comply with current standards. From investigations of the literature and current trends of DOTs, two rehabilitation techniques are recommended. The most popular decreases the stress in the tension members with the use of supplementary post-tensioning cables or rods. A second option is coverplating.

4.3.3.2.1 Adding Post-tensioning Cables or Rods, or Supplementary Members

The addition of post-tensioned cables or rods can be used to reduce stresses in tension members of a truss. From the data collected in the survey of DOTs, nine agencies indicated the use of post-tensioning to reduce stresses. For example, the Kentucky DOT stated that they had used “tension tighteners on eyebars”. The Alberta DOT responded that their agency had used Dywidag rods to post-tension the bottom chord on a truss bridge. The available literature also strongly supports this technique. NCHRP 293 [1.15] provides a detailed discussion of advantages and disadvantages of this system, reference [1.15].

Reference [2.6.3] describes a rehabilitation of a truss bridge in which a third member was added to a two eye-bar system to reduce the tensile stresses and provide added redundancy. Figure 4.23 shows the supplementary members added to the tension chords.

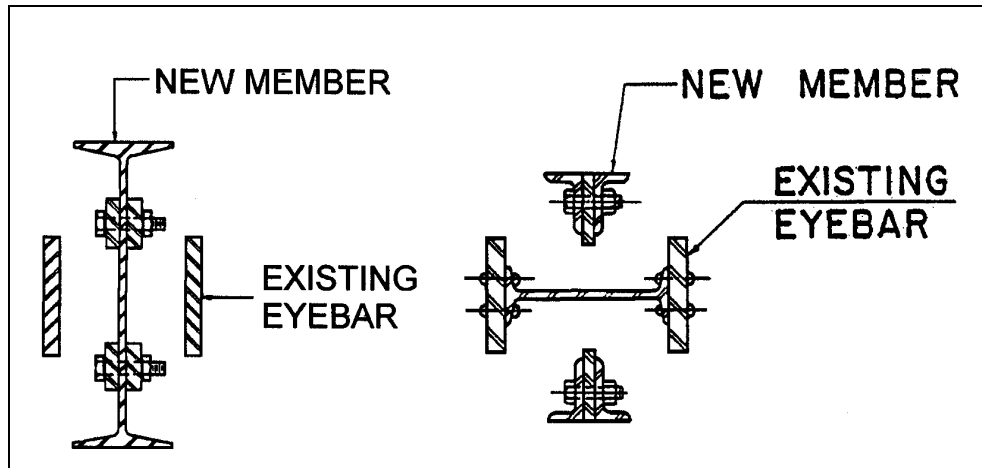


Figure 4.23: New Members Added to Existing Tension Members [Ref. 2.6.3]

4.3.3.2 Coverplates

The use of coverplates on tension members requires special considerations. As in coverplating compressive members, it may be desirable to partially induce the dead loads present in the existing structure into the new plates. Therefore, it may be necessary to utilize a jacking system that can apply a compressive force to the existing member while the new member is bolted in place. It should be noted that the stability of the tension member should be considered if a compressive stress is to be applied.

4.3.3.3 Compression Members

The two most common strengthening techniques for compression members involve coverplating or adding supplementary bracing to reduce the slenderness ratio. In the following paragraphs, a discussion of each method, and examples of each will be presented.

4.3.3.3.1 Coverplating

The added material of coverplates reduces the stresses in the compressive member. To add coverplates to a built up member, the rivets may be removed and replaced with high strength bolts. In a coverplating repair, the dead load stresses present in the structure should be considered. If the dead load stresses in the existing member are to be relieved, a load transfer mechanism must be utilized. In the literature two methods of transferring dead load stresses were discussed. The first option requires the stress in the member to be removed using a jacking system as a primary step. The coverplate is attached to the existing member in the elongated position, and the jack removed. As a result, both the original member, and the new coverplates resist the dead loads in the structure. The second option involves offsetting the bolt-holes of the coverplate and existing member in such a manner that as the coverplate is bolted into place, a compressive stress is induced in the coverplate. Common arrangements for coverplates applied to existing compressive members can be seen in Figure 4.24.

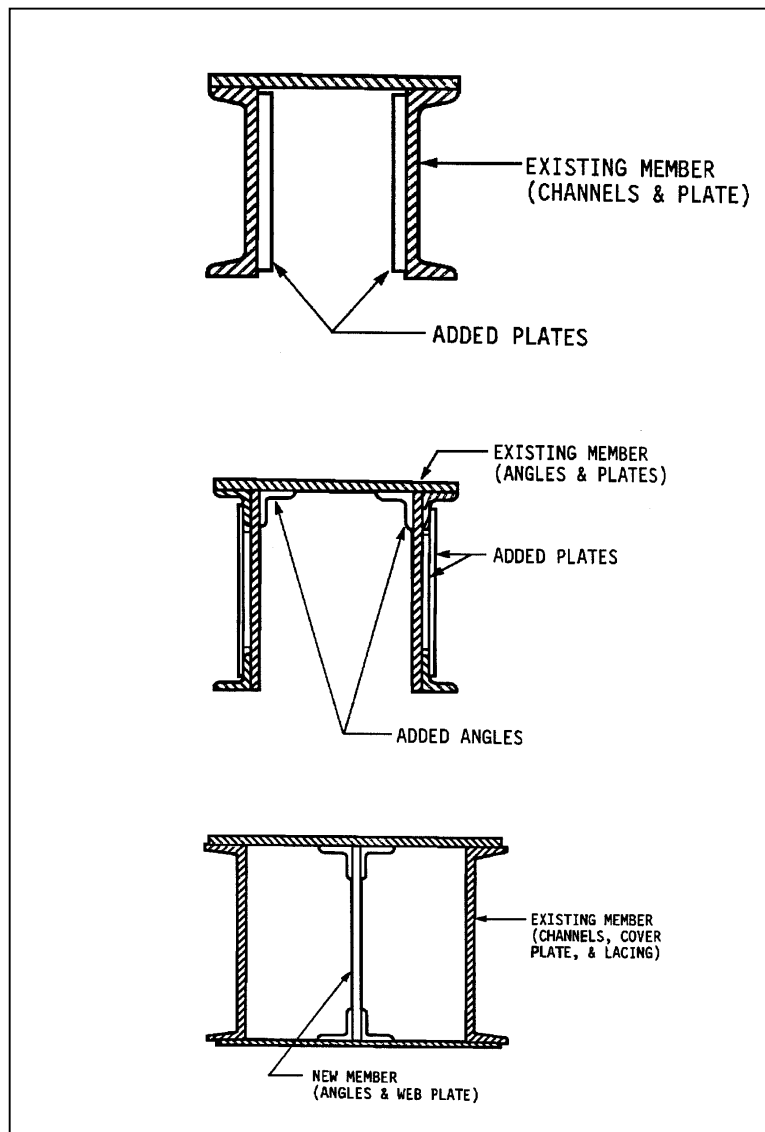


Figure 4.24: Coverplate Options for Compressive Members [Ref. 1.15]

In response to the survey of DOTs, twenty agencies indicated a use of coverplating to increase the strength of members in general. The Maryland DOT explicitly stated the use of bolting plates or rolled shapes to the webs of understrength compressive members. A similar application was found in the literature, in which hidden members were placed in the compressive chord of a truss [1.6]. Reference [2.4.1] describes another strengthening effort which added coverplates to the top chord of a pony truss. NCHRP 293 provides a step by step design example of this technique including outlining the calculations involved in applying this method, reference [1.15].

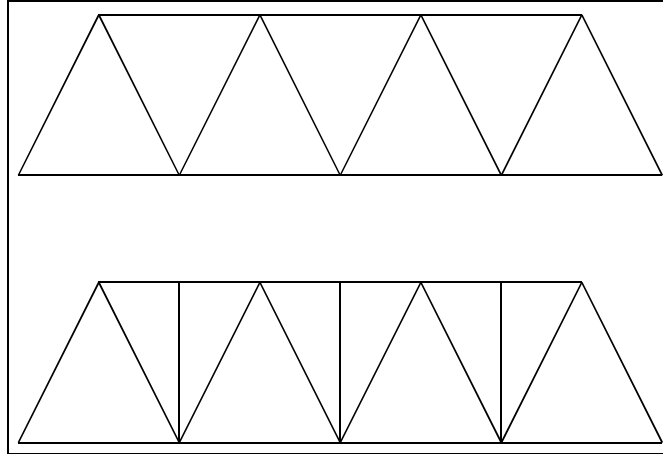


Figure 4.25: Bracing Compression Chord of Truss

4.3.3.4 Pinned Connections

Pin-connections are common in historic truss bridges. Because of the non-redundant nature of pin-connected bridges, caution should be taken in the rehabilitation of these members. A primary step involves determining the material properties of the pin. The Maryland DOT stated “We typically increase the allowable stress by 50% over what was determined by testing batten plate material.” The calculation of stresses in the pin must also be carefully examined since they are usually based on beam theory, which might not be valid for the short cylindrical pin members.

Two common problems associated with pinned connections are corrosion and cracking. Corrosion damage might be critical due to the loss of section or the freezing of the pin to connecting members inducing bending stresses. It may be necessary to apply chemicals to loosen the seizing between the pin and eye-bar. If excessive corrosion has damaged the pin, replacement might be warranted. The new pin can be inserted, with appropriate lubrication, to protect the joint from debris and future corrosion.

Cracks may occur in the pins due to a combination of many factors including excessive loads, corrosion, or poor material. To evaluate the integrity of the pins, a common technique involves using non-destructive testing methods. In response to the survey, seven of the DOTs indicated the use of ultrasonic testing to check the pins of trusses.

If the pin is severely corroded or cracked it should be replaced. The stresses must be relieved usually with the use of false work or other load transferring mechanism. The survey revealed that ten of the DOTs had replaced pins in trusses. Many examples of pin replacement were also found in the literature. References [1.15] and [1.12] both provide introductory material on this topic. Reference [2.7.1] describes a rehabilitation that involved dismantling the truss and replacing the pins. Article [2.7.2] relates a pin replacement effort of a wrought iron bridge built in 1884. A detailed description of the procedure is included. The new pins were longer than the originals, to accommodate supplementary members. Articles [4.1.5] and [4.1.6] discuss the holistic behavior of pin connected bridges. Reference [4.1.7] investigates the phenomenon of

unequal sharing of tensile loads in eye-bar members and reemphasizes the non-redundancy of pin connected trusses.

4.3.3.5 Riveted Connections

The connections of older truss bridges are often susceptible to corrosion due to the built up nature of the elements. Large areas of rivets and the intersection of plate elements often collect debris and exacerbate corrosion problems. Approaching a connection repair requires attention to many details. The repair should allow for drainage and reduce the presence of built up debris. When the connection was originally designed, fatigue may not have been considered. A connection repair should address fatigue and be designed accordingly. Common methods include bolting or welding additional plates to the existing connection. Welding should only be considered if the material is weldable. Often, high strength bolts are used to replace rivets in rehabilitated connections. The clamping force imposed by the bolts act to increase the shear capacity of the connection and the fatigue life.

The survey of DOTs revealed that some of the agencies had strengthened connections on their truss bridges. The Georgia DOT indicated the use of bolts to replace rivets. The Saskatchewan DOT stated that one of their repairs has “..involved replacement of rivets with longer high strength bolts to accommodate the additional members”. The Nova Scotia DOT echoed the other two DOTs by reporting they had strengthened existing connections. The literature search produced many articles related to this topic. References [1.15] and [1.12] provide introductory information on the rehabilitation of connections. Reference [2.1.1] presents a chart of common connection deficiencies and possible solutions. The article also recommends bolting for older steels and discusses the possibility of using composite laminates to repair connections. Articles were also found which investigated the strength and fatigue life of riveted or rehabilitated connections. Reference [2.5.3] investigated the behavior of manufactured riveted and bolted connections as well as a sample connection from a bridge. Article [2.5.2] examined the fatigue life of connections whose rivets were replaced with bolts. This article also investigates repairing connections with small cracks already present. Other articles, which could be helpful, include [4.1.2], [2.4.2], [4.3.1], [4.3.2], and [4.3.4].

4.3.4 Structural System

4.3.4.1 Deck Rehabilitation

The replacement of the deck system presents many options and considerations. Replacement of a deck is often an attractive proposal as the high dead load associated with a concrete deck can be replaced with a much lighter system. In the survey of DOTs, twenty-one of the agencies indicated the use of a lighter deck system in the rehabilitation of their truss bridges. This reduction in dead load increases the bridge’s live load capacity and often significantly increases the load rating of the bridge. Deck replacement is a good option when the existing floor beams and/or stringers are in good repair, but understrength.

A primary concern is the interaction of the bridge with the new deck. If the deck system is stiff, compared to the truss, forces in the structure may be resisted by the deck. It is of utmost

importance to check the connections of the deck to the floor-beams, stringers, and other elements. The new deck may be made composite with the structure, which will also alter stresses in members. A general design guideline for deck replacement is presented in NCHRP 293, reference [1.15].

Six classes of deck replacement systems were identified and are presented in the following pages. It should be noted that it is possible to combine systems, such as the use of lightweight concrete and aluminum extruded deck. The six major system types discussed include:

- Cast-in-place Concrete
- Pre-cast Concrete
- Open Grid
- Filled Grid
- Plate Decks
- Timber

4.3.4.1.1 Cast-in-place Concrete

The most common replacement option, for concrete decks, is to replicate the existing system of cast-in-place concrete. Lightweight concrete that weighs approximately 20% less than standard concrete might be chosen. Another option is to create composite action between the existing steel members and the new concrete deck. This may be accomplished by attaching shear studs to the steel floor-beams and stringers. These studs may be welded, if the steel is weldable, or bolted through the top flange of the member. An advantage of cast-in-place concrete is the widespread use of the technique. This translates into ease of construction and a lower cost of the system. Disadvantages of this system include the high unit weight of concrete and the delay involved with the curing time.

In response to the survey, many of the DOTs indicated that they had used this technique. Four specifically stated the use of cast-in-place concrete. In the literature many examples of cast-in-place concrete decks were found. Reference [1.12] presented four examples of this technique. NCHRP 222 describes a procedure for deck rehabilitation titled "Replacement S-9 Site-Cast Deck on Steel Beams", reference [1.14]. The Virginia DOT also supported this deck replacement option in their report, reference [1.16].

4.3.4.1.2 Pre-Cast Concrete

The use of pre-cast elements in structural engineering has become a well-established technique. In the bridge rehabilitation area, this method was first implemented in the late sixties and continues to be popular. As in cast-in-place concrete, the new system may be made composite with the existing structure to further increase the performance of the system. The

quality of the pre-cast elements can be closely controlled during the manufacturing process, resulting in quality concrete members. Other advantages include ease of construction and the wide use of the technique. Disadvantages of this system include the high unit weight of concrete, increased costs as compared to conventional concrete, and more detailing. The weight issue may be diminished by the use of lightweight concrete, but even this might not provide an adequate decrease in dead load.

Only one DOT indicated the use of pre-cast panels in truss bridge rehabilitation. The Connecticut DOT stated that their agency had used concrete panels during off-peak hours to expedite the replacement of an existing structure. The literature search uncovered more examples of this method. Reference [1.12] relayed examples of pre-cast deck replacement in New York, Pennsylvania, and the District of Columbia. NCHRP 222 “Replacement S-3 Pre-Cast Deck Slabs on Steel Beams” describes many different design scenarios and solutions to each, reference [1.14]. The report illustrates three options in the use of pre-cast panels. The first uses panels with stringers cast into the concrete that are attached to the floor-beams. The second replacement utilizes clamps to attach the pre-cast panels to the existing stringers. The final procedure employs panels with holes formed in them that are grouted when placed over the existing stringers which have shear studs attached to the top flange. Section of 4.3.3.1, Composite Action, provides further discussion of this system.

4.3.4.1.3 Open Grid

Open metal deck is another option in replacing an existing deck. The replacement system rests on the existing stringers and floor-beams. The deck is usually welded to the elements, but this should only be done if the bridge metal has been approved for welding. If welding is not viable, then the deck may be bolted to the structural system. Special consideration must be given to construction details to ensure adequate continuity between the deck and floor system. These systems are very lightweight and will reduce the dead loads on the structure substantially. Another advantage in this technique is the speed of construction. The metal decks are modular in nature, expediting their installation. Water drainage is very good in these systems because of the openings in the deck. However, the good drainage properties do not protect the supporting elements from corrosive attack. The skid resistance of open grid decks is low. Open decks have also been subject to weld failures in some circumstances. A typical open grid deck panel can be seen in Figure 4.26.

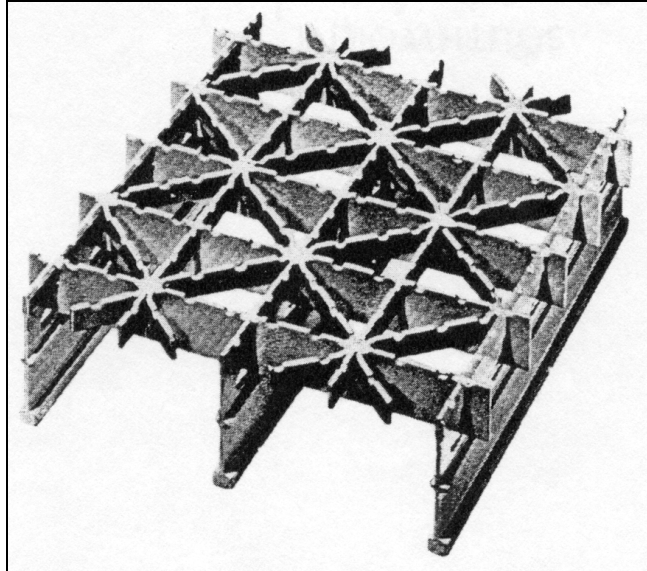


Figure 4.26: Open Grid Deck [Ref. 1.15]

From the responses of the surveys, it was discovered that this method has been implemented in some states. The Iowa DOT stated that their agency had used a steel grid system to replace a concrete deck. The Kansas and Ohio DOTs also indicated the use of metal gridding in the rehabilitation of their truss bridges. The literature also provided examples of this technique in practice. Reference [1.12] provides an overview of the technique. NCHRP 293 gives examples of open decking used in West Virginia, Illinois, and New Jersey, reference [1.15]. The report also states that some agencies placed concrete in the grids to improve the skid resistance of the deck. Replacement S-6 in NCHRP 222 is a description of metal grid decking being placed on existing stringers and floor-beams, reference [1.14].

4.3.4.1.4 Filled Grids

Filled grid systems usually involve a steel gridding system filled with concrete. The concrete provides a better wearing surface but increases the dead weight of the system. Typical concrete filled steel grids weigh 75-80 psf. These systems have most of the advantages of open steel grids. However, the curing time of concrete will extend the construction time. Another disadvantage is a phenomenon called deck growth. Deck growth occurs due to corrosion of the steel gridding causing the system to expand. This can result in connection failures and unnecessary stresses formed in the concrete. To alleviate this problem, an overlayment or other protection for the concrete may be employed. A picture of a filled grid deck can be seen in Figure 4.27.

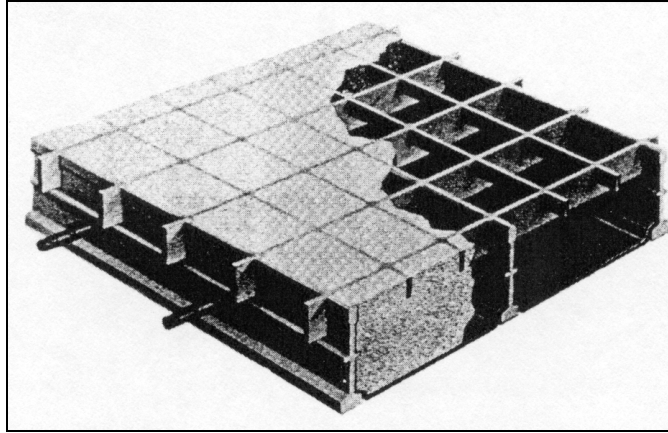


Figure 4.27: Concrete Filled Deck [Ref. 1.15]

Another type of filled grid is the exodermic grid. This system uses a steel gridding with a thinner concrete topping rather than a filled grid. These systems are lighter, from 40-60 psf, and also have better skidding resistance, similar to filled systems. Exodermic grids do not experience deck growth like filled grids since the concrete topping protects the steel components. Exodermic panels can also be pre-fabricated providing better quality control and quicker installation.

In response to the survey, no DOTs stated that they had used steel filled decks. The literature, however, provided some examples of this technique in practice. NCHRP 222 Replacement S-7 outlines a procedure for using concrete filled steel decking, reference [1.14]. The report indicated that Ohio, Colorado, and Virginia had used this method with success. A report from the Exodermic Bridge Deck Institute, reference [2.8.4], provides additional information on this technique.

The literature uncovered another form of filled deck, which utilizes extruded aluminum panels, upon which the concrete is placed. Reference [2.8.1] provides an example of this system on a truss bridge. In response to the survey, the Connecticut DOT stated that their agency had used “Alumadeck” which the appears to be similar to the system described in reference [2.8.1].

4.3.4.1.5 Plate Decks

Another modular decking system, which has become popular in the last few decades, is the plate deck. Also called orthotropic systems, these decking schemes are designed to distribute loads according to plate theory instead of isotropic behavior typical of most deck systems. Orthotropic systems often replace the stringers as well as the deck, which further lowers the dead loads. Span lengths may vary from 15-40 feet, which make orthotropic plates practical for many applications. The weight of this system may vary from 45-130 psf depending on the span. The deck is typically covered with a bituminous surface to provide adequate skid protection. Figure 4.28 shows a plate deck system.

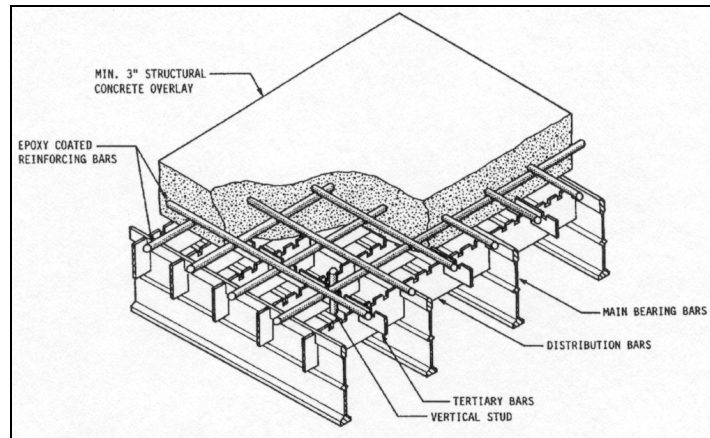


Figure 4.28: Plate Deck [Ref. 1.15]

Although none of the responses to the survey indicated the use of orthotropic systems, examples of the method were found in the literature. NCHRP 293 provides an introductory discussion of this system, reference [1.15]. Reference [1.12] further discusses the technique and described applications in Vancouver, New York, and on the Golden Gate Bridge. Information from a company named BGFMA outlines examples of orthotropic systems in practice and design specifications for their product in reference [2.8.8]. NCHRP 222 Replacement S-8 presents the use of orthotropic plates in deck rehabilitation and cites the Maryland DOT as a case study for this method, reference [1.14].

4.3.4.1.6 Timber

The use of timber to replace existing decks has undergone a very beneficial change over the past years, with the advent of laminated wood technology. Laminated timber utilizes normal dimensional lumber which is glued together to form a stronger system. The result is a material with more continuous properties than normal lumber, which translates into higher allowable stresses. Laminated material may take the form of beams, columns, or decking systems. Timber decks made of laminated wood consist of panels, which are joined with dowels to form a continuous deck system. The panels speed construction time, making this system very practical. These decks may also be made composite with the existing steel girders. Another advantage is the weight of laminated decks, which average between 10-25 psf. Deicing chemicals do not harm the decking materials and laminated decks may be treated to resist fire damage. Like other timber products laminated decking is subject to decay, however this can be reduced with the use of pressure treated wood.

Two states responded favorably to the use of timber decking in the rehabilitation of truss bridges. The Vermont DOT stated their agency had replaced an existing concrete deck with a timber deck. The Maryland DOT has considered the use of these systems in the restoration of their bridges. Many examples of this technique were also found in the literature. An introductory discussion of timber decking can be found in NCHRP 293, reference [1.15]. NCHRP 222 provides two replacement examples, reference [1.14]. Replacement S-4 describes laminated decking on existing stringers with case examples from Alaska and Virginia. Replacement S-1 offers the use of laminated deck and beams to replace the deck and existing

stringers. Additional information on laminated bridge decking may be found in references [2.8.7]. An example of a typical laminated timber deck, applied to an existing structure can be seen below in Figure 4.29.

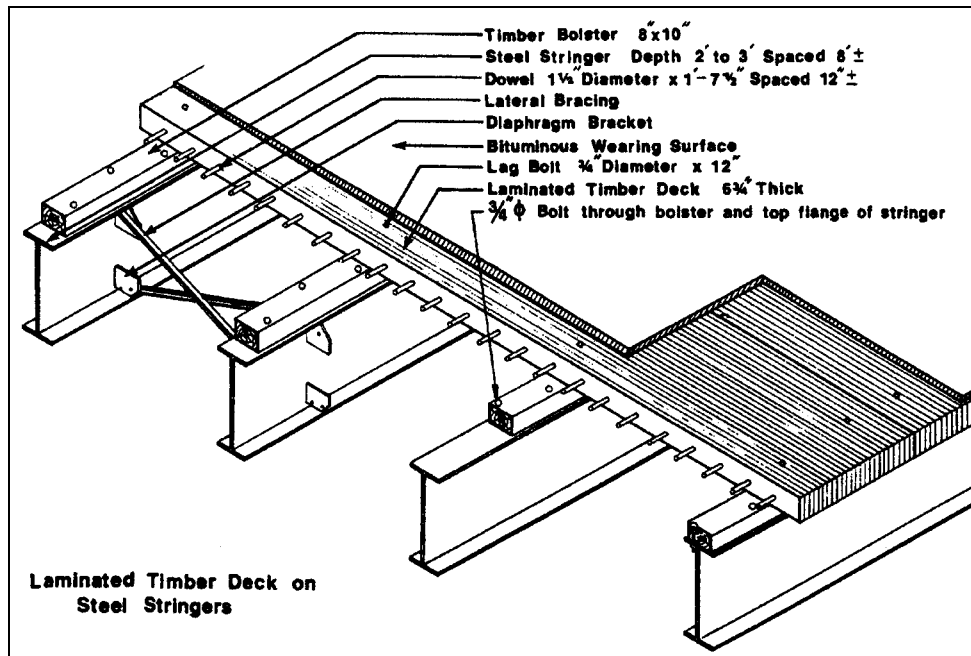


Figure 4.29: Laminated Timber [Ref. 1.15]

4.3.4.2 Post-Tensioning

Post-tensioning, as a system strengthening technique, has been used for approximately thirty years. A tendon or tendons are placed on the structure and loaded in tension to reduce the stresses in the members. There are four different post-tensioning arrangements, which can be seen in Figure 4.30.

- Tendons concentric with members
- Tendons not concentric with members
- Tendon with compression strut
- King post

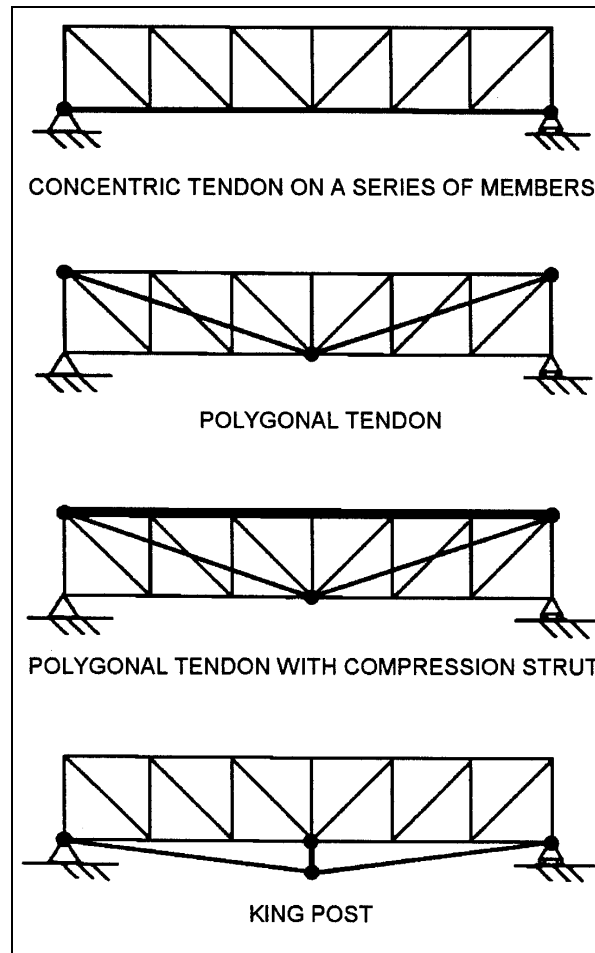


Figure 4.30: Post-Tensioning Options for Truss Structure [Ref. 1.15]

Concentric tendon layouts reduce the stresses in the members with tendons. This approach could be used if a few members are understrength, and require rehabilitation. Non-concentric tendons improve the performance of members close to the tendons. The final two arrangements apply a negative moment to the entire structure, thereby reducing the stresses. Post-tension repairs often require few traffic disruptions and allow for fabrication off-site. The tendons also increase the allowable stress range of the system. This method can also reduce the displacements in the structure and add a level of redundancy to the structure.

There are a few disadvantages to this method. A primary one is the high local stresses induced, where the tendons are attached to the structure. Careful consideration must be given to the resistance of these local stresses. Strengthening of the joints might be necessary to accommodate for the higher stresses. Another concern with these systems is the corrosion of the tendons. The tendons are often hidden inside the tension chord of the structure, a prime location for the build up of debris and other corrosive agents. A final concern is the reduction in safety factor of the bridge. The tendons increase the allowable stress range of the elements, but do not add a proportional increase in ultimate strength of the members. This leads to a less ductile response.

This technique has been embraced by many agencies across the world. The survey responses reinforce this statement. Seven of the agencies indicated the use of this method. The Alberta and Oklahoma DOTs stated that their department had used a threaded bar system (i.e. Dwyidag) to relieve the stresses in the bottom chord of bridges. The literature also provides many examples of this practice. NCHRP 293 describes six cases of this technique in locations such as England, Switzerland, and Italy, reference [1.15]. The report also provides an example of beam strengthening using post-tensioning and a general discussion of the entire topic. Reference [2.2.1], describes a research project dealing with the analysis of post-tensioned systems. A discussion of concentric and non-concentric tendon layouts is discussed, as well as, structural response to their inclusion. Article [2.2.2] relates a rehabilitation in which a concentric post-tensioned system provided the safe rehabilitation of the truss bridge and reduced the budget of the repair as compared to member replacement. Reference [2.2.3] discussed the added redundancy of the post-tensioned method. Articles [2.1.1], [2.7.1], and [1.10] support this technique as a viable option in the strengthening of a truss.

4.3.4.3 Additional Load Bearing System

If the load capacity of the truss bridge requires a major increase, a superimposed system might be required. The new system usually accepts the entire live load once applied to the truss, and requires the existing truss to carry its own weight and aid in the bracing of the new structure. In the discussion of this method, three supplementary systems will be described including:

- Superposition of an arch
- Bailey Truss
- Hidden Beams

4.3.4.3.1 Superposition of an arch

This technique uses the advantages of an arch as a load bearing structure in the rehabilitation of a truss bridge. An arch is constructed on the exterior of the existing bridge and is attached to the bridge. The pre-existing truss provides lateral stability to the arch. Floor-beams connect the bottom chords of the arches with the exiting deck system. This method has the advantages of simple construction where no jacking or shoring is usually necessary. Traffic disruption is minimal since most of the work is on the exterior of the structure. Two main difficulties include support conditions and architectural concerns. The substructure might require widening to support the new system, and should be checked for the thrusting forces that are associated with an arch system. Figure 4.31 shows an arch system attached to an existing through truss.

The survey of DOTs revealed that this rehabilitation technique has been put into practice. Four of the agencies, including New York, Hawaii, California, and Alberta, stated that they had used a superimposed system. The literature also provided examples of this method. References [1.12], [1.15], and [1.16] all provide introductory information on this topic. The NCHRP report, reference [1.15], provides examples in Europe dating back to the 19th century. Article [2.3.1] also gives a general overview of this technique. References [2.3.2] and [2.3.4] both discuss the

rehabilitation of the same truss bridge using an arch system. Article [2.3.3] describes a research project that investigated superimposed arches attached to a metal truss.

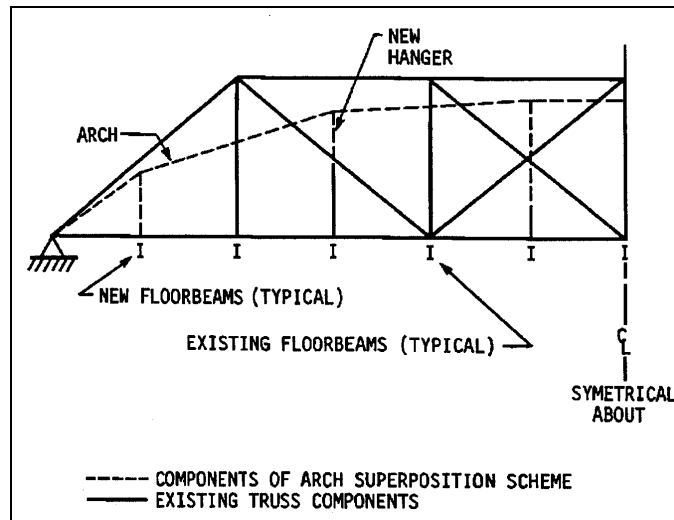


Figure 4.31: Superimposed Arch Applied to Truss Structure [Ref. 1.15]

4.3.4.3.2 Bailey Truss

A Bailey truss is a pre-fabricated truss element that is typically used in the repair of truss bridges. Bailey trusses have been used extensively in the military as a quick method of bridging spans. The truss is attached to the existing floor beams with hangers and is braced by the existing truss. These systems have the advantages of easy construction and wide availability. However, the Bailey truss is often placed on the interior of the existing bridge, reducing the width of the bridge. Figure 4.32 shows a Bailey truss attached to the interior of a pony truss to increase the capacity of the bridge.

None of the responding DOTs indicated the use of this technique in their jurisdictions. The few references found in the literature include NCHRP and article [2.4.1]. The NCHRP report provides background information on this system. Reference [2.4.1] refers to the Bailey truss as a possible solution to the strengthening of a pony truss bridge. However, the reduction in width eliminated the Bailey truss from construction.

4.3.4.3.3 Hidden Beams

An innovative technique to repair truss bridges is the replacement of the existing support system with longitudinal beams. With the advances in technology, higher strength steels are able to span further than previously. The new support system must be independent of the original trusses, to accept the entire live load applied to the structure. The truss remains to aid in bracing the longitudinal beams. The existing substructure might have to be altered, as the existing system might have only been designed to support the truss.

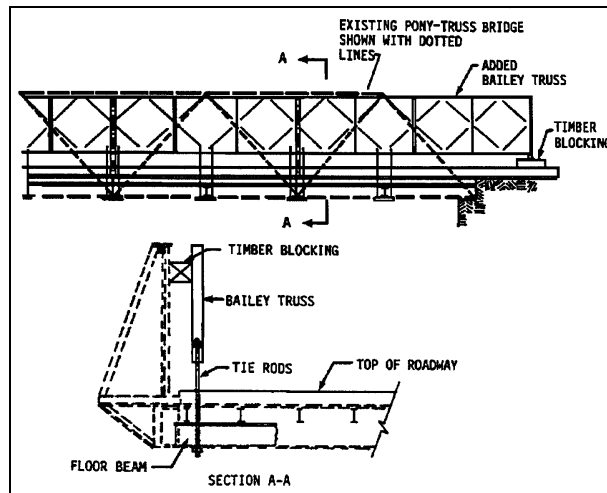


Figure 4.32: Bailey Truss Applied to Pony Truss Bridge [Ref. 1.15]

The survey revealed that seven DOTs had used longitudinal beams in the rehabilitation of their trusses. It is unknown if these beams act as the entire load bearing system, or aid the truss in resisting loads. The literature search provided additional information on the use of this method. Article [2.6.1] described the rehabilitation of two truss bridges, in Rhode Island, with spans of 58 and 98 feet. Reference [2.6.2] used “hidden portals” to strengthen the existing system. The author of these articles commented on the necessity of considering the stability of the additional beams.

4.3.4.4 Additional Continuity and Support

This method of rehabilitation includes two options, adding additional supports and converting simply supported systems into continuous systems. Both techniques reduce the stresses in the system. However, extensive analysis should accompany the consideration of these methods.

4.3.4.4.1 Additional supports

This method reduces the span length, drastically reducing the stresses in the bottom chord. However, members that acted in tension before the new supports, may undergo stress reversals and be placed in compression. Careful consideration should also be given to the placement of the new substructure elements. The type of soil present, as well as flow characteristics of any waterways should also be considered. A simple example of constructing additional supports can be seen in Figure 4.33 below.

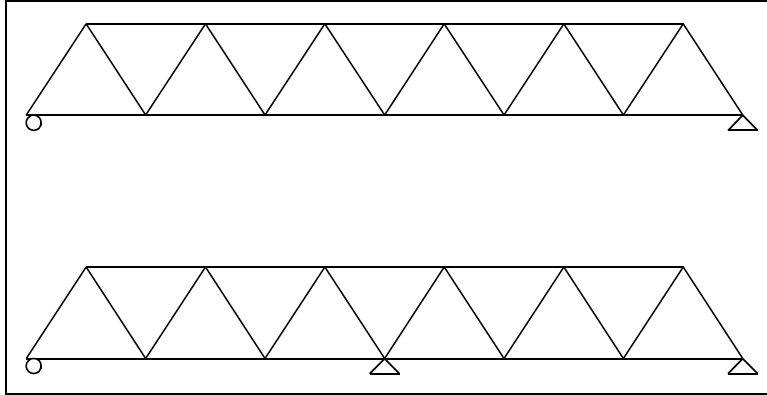


Figure 4.33: Additional Supports Added to Truss Bridge

Of the responding DOTs, six indicated they had used additional supports in the rehabilitation of their bridges. In the literature search, little information could be found pertaining to this topic. NCHRP 293 provides a discussion of this method and provides a few examples of this option in practice, reference [1.15]. Reference [1.16] analyzed this option and its effects on a truss bridge. The researchers reported a general lowering of the stresses but noted that some members would require additional strengthening. Article [1.10] also suggests this option in the rehabilitation of bridges.

4.3.4.4.2 Simply Supported to Continuous

This technique requires the addition of struts on the upper chord of the bridge to form a continuous member. Again, careful consideration to stress reversals should be realized. Strengthening of tensile members might be necessary to accommodate for these reversals.

Two of the responding DOTs indicated the use of this technique. Only two references were found in the literature relative to this method. The NCHRP 293 report discusses this option, but suggests the use of it be reserved for steel girder bridges or concrete bridges, reference [1.15]. The report published by the Virginia DOT, reference [1.16], attempted this technique on a bridge without success. The bridge did not respond as expected, and stress reductions were minimal.

Chapter 5:

Conclusions

This report has documented the results of a study on current practices and available literature on the evaluation and rehabilitation of historic metal truss bridges. This study included a literature search; a survey of Departments of Transportation (DOT)s; and the development of a summary and synthesis of information found in the literature review and DOT survey.

The literature search yielded approximately 150 documents related to the evaluation and rehabilitation of metal truss bridges. The literature was cataloged in the form of an Annotated Bibliography and is organized so as to facilitate inclusion of additional references in the future.

The survey of DOTs was sent to 60 agencies to solicit information on current engineering trends in addressing problems associated with historic metal truss bridges. Thirty-nine responses were received and complied. This information was used to provide practical examples of rehabilitation techniques documented in the literature. The survey also uncovered some rehabilitation techniques that have not yet been documented in the literature.

The results of the literature search and the DOT survey were combined in a broad summary and synthesis of topics related to historic metal truss bridges. This synthesis should prove useful to engineers involved in metal truss bridge evaluation or rehabilitation projects by providing information on a wide variety of engineering approaches and solutions.

Appendix A: Survey of Departments of Transportation

The following letter was mailed to 49 state Departments of Transportation, as well as, Washington D.C, and 10 Canadian provinces. The words enclosed in double arrows are fields from a mail merge that included the names and addresses of the parties to be contacted at the transportation agency. A copy of the survey sent to each agency follows the letter. Survey responses are summarized in Appendix B. Further discussion of the survey can be found in Chapter 3.

April 27, 1997

«FirstName» «LastName»
«JobTitle»
«Company»
«Address1»
«Address2»
«City_State»
«PostalCode»

Dear «FirstName» «LastName»:

The Texas Department of Transportation (TxDOT) is currently sponsoring a research project through the University of Texas at Austin to investigate preservation alternatives for historic metal truss bridges. The bridges under consideration were constructed in the late 19th to early 20th centuries, and often suffer from structural and geometric deficiencies. The goal of this project is to research typical historic metal truss bridges in Texas and provide guidance for future rehabilitation efforts undertaken by TxDOT that will permit these bridges to remain in vehicular service.

As a preliminary step in this project, our team is currently collecting information concerning all aspects of metal truss bridge investigation and rehabilitation projects in other states. Any information you can provide on this subject would greatly help our project. This information will aid our research team to create a database of knowledge to be used in rehabilitating historic metal truss bridges.

If you would please fill out the enclosed survey and return it to us by June 30,1997, it would be of tremendous help to our project. If you do not have the information or time to complete all portions of the survey, please feel free to return the form only partially completed. We would prefer to have a partial response, rather than none at all.

If you have any questions, please feel free to call Matthew Thiel, research assistant at (512) 323-5934; Dr. Michael Engelhardt, research supervisor at (512) 471-0837; or Barbara Stocklin, TxDOT point of contact at (512) 416-2628. The researchers may also be reached via email at mthiel@mail.utexas.edu and mde@uts.cc.utexas.edu respectively. We would be happy to send you a copy of our final report at the completion of our project.

Thank you for your attention.

Sincerely,

Dianna F. Noble, P.E.
Director of Environmental Affairs

Preservation of Historic Metal Truss Bridges Survey

The University of Texas at Austin is currently investigating preservation of steel truss bridges under a project sponsored by the Texas Department of Transportation. To this end, we are asking your assistance in providing information which will aid our research team, and provide guidance for future rehabilitation efforts undertaken by TxDOT. A variety of issues have been identified as particularly relevant to our investigation including: 1) structural evaluation, repair, and strengthening techniques 2) dealing with geometric deficiencies 3) funding of rehabilitation projects 4) obtaining design exceptions for rehabilitation efforts.

As a preliminary step in this project, our team is currently collecting information concerning all aspects of steel truss preservation underway in other states. As such, we would appreciate your time and effort in filling out this survey. If you do not have the information or time to complete all portions of the survey, please feel free to return the form only partially completed. We would prefer to have a partial response, rather than none at all. In exchange for your assistance, we would be happy to send you a copy of our final report at the completion of our project.

Note: For any question which falls outside of your specialty, instead of providing an answer, would you please indicate the name and phone number of an individual we can contact for further information.

1. Has your state developed any reports, guidelines, or other documents addressing the evaluation or rehabilitation of steel truss bridges?

Yes _____ No _____

If yes, we would greatly appreciate receiving a copy of any pertinent reports returned with this survey.

Report Name & Date	Name, address or phone number of individual to contact to obtain a copy of the report

2. Have you used advanced structural analysis techniques to provide improved estimates of the structural capacity of steel truss bridges?

Yes_____ No_____

If yes, please explain or provide contact for additional information.

3. Have you used advanced non-destructive evaluation techniques (e.g. acoustic emission monitoring) to assist in evaluating the condition of steel truss bridges?

Yes_____ No_____

If yes, please explain or provide contact for additional information.

4. Have you used load testing to assist in evaluation the structural capacity of steel truss bridges?

Yes_____ No_____

If yes, please explain or provide contact for additional information.

5. What are the most common structural strengthening techniques your department has used in rehabilitating steel truss bridges?

- 6.** Please check any other structural strengthening techniques you have used.
- | | |
|--|--|
| <input type="checkbox"/> Superimposed trusses | <input type="checkbox"/> Addition of longitudinal beams |
| <input type="checkbox"/> Post-tensioning bottom chord | <input type="checkbox"/> Providing additional supports |
| <input type="checkbox"/> Joining simple spans into continuous span | <input type="checkbox"/> Adding king or queen posts and post-tensioned tendons |
| <input type="checkbox"/> Replace floor deck with a lighter system | <input type="checkbox"/> Pin replacement |
| <input type="checkbox"/> Other (please explain) | <input type="checkbox"/> Attach cover plates to members |

7. For bridges with geometric deficiencies, either inadequate height or width, please check any solutions you have used:

- | |
|--|
| <input type="checkbox"/> Relaxing geometric standards for historic bridges |
| <input type="checkbox"/> Widening bridge |
| <input type="checkbox"/> Increasing portal height by removing or altering overhead members |
| <input type="checkbox"/> Convert bridge to one-way traffic |
| <input type="checkbox"/> Other (please explain) |

8. What methods, if any, have you used to improve railings on historic steel truss bridges? We are particularly interested in information on crash tested railings which have been added to historic steel truss bridges?

9. What methods have you used to deal with the presence of lead based paints on historic steel truss bridges:

- Remove old lead paint (with appropriate disposal techniques) and repaint bridge.
- Apply sealer to encapsulate lead based paint
- Other (please explain)

10. Has your department been involved with the rehabilitation of a historic steel truss bridge that has involved a particularly interesting, unique, or innovative approach?

Yes _____ No _____

If yes, please state the name or location of the bridge, and the name and phone number of an individual that can provide additional information.

11. At the completion of our project on the rehabilitation of historic steel truss bridges for TxDOT, would you like to receive a copy of the reports?

Yes _____ No _____

If yes, please provide a name and address to which the reports should be sent.

12. Additional Comments:

13. Please provide your name, address, and phone number

Appendix B:

Responses to Survey of Departments of Transportation

This appendix contains the responses received from the survey. The responses for each question have been grouped together for convenience. Any phrase or word surrounded with square brackets [] indicate notes or modifications made by the authors. Summaries and discussion of these survey responses is provided in Chapter 3.

QUESTION 1:

Has your state developed any reports, guidelines, or other documents addressing the evaluation or rehabilitation of steel truss bridges?

Yes:

AZ : In-Depth Steel Bridge Inspection Program, July 20, 1996

IA : Ultimate Load Behavior of Full Scale Highway Truss Bridges, Aug. 1975, Sept. 1975

MN : Bridge 4175 - Summary of Inspection for Reuse as a Pedestrian Bridge, 4/97 Wabasha Street Bridge Fatigue Analysis, 8/89

NE: ["Evaluation and Retrofitting of Historic Truss Bridge", University of Nebraska at Lincoln, 1996]

VT : [see Additional Comments]

WA : Research project w/ U. Of W. "Steel Bridge Cracking" Report due Phase I August, 1997, Phase II, 1999; Contact Harvey Coffman (360) 753-6076

[Alberta] AB : Bridge Truss Rating System - A computer system for load capacity rating of truss bridges; Contact Raymond Yu (403) 415-1016 email: ryu@tu.gov.ab.ca

[Nova Scotia] NS : We use Clause 12 of the Canadian Highway Bridge Design Code to evaluate our structures.

No (with note):

MD : We haven't developed any official guidelines pertaining to truss rehabilitation since the State of Maryland doesn't have any pin connected, iron or steel trusses, however several counties in Maryland do have pin connected trusses. We get involved in the review of ISTEPA funded, local government projects and suggest, on a case by case basis, a similar theme. The major points that we suggest are as follows: [see question 12 additional comments]

No: AK, AL, AR, CA, CT, DE, DC, FL, GA, HI, IN, KS, KY, MS, MO, MT, NV, NH, NJ, NM, NY, NC, OH, OK, RI, TN, [Manitoba] MB, [New Brunswick] NB, [Newfoundland] NF, [Saskatchewan] SK

QUESTION 2:

Have you used advanced structural analysis techniques to provide improved estimates of the structural capacity of steel truss bridges?

Yes:

AL: Developed a truss analysis program that used the stiffness method of analysis. Accounts for stiffness at each joint and performs analysis

CT: BAR 7 - Analysis for rating; On occasion GTSTRUDL has been used for 3-D finite element modeling

FL: Bridge Rating of Girder - Slab Bridger Using Automated Finite Element Technology (BRUFEM) was used to analyze the deck girder portion of the bridge. Space frame analysis was used to determine forces in secondary and primary members.

KY: We have used STRUDL and analyzed trusses as a space frame for LL distribution.

NE: [BARS, SAP 90 utilized in research project]

No (with note):

AZ: Used conventional 2D & 3D elastic analysis methods to verify behavior and load distribution

OH: ODOT has accepted finite element from a consultant for truss rating. We don't normally do it.

NF: Normal methods. M-STRUDL.

No: AK, AR, CA, DE, DC, GA, HI, IN, IA, KS, MD, MN, MS, MO, MT, NV, NH, NJ, NM, NY, NC, OK, RI, TN, VT, WA, AB, MB, NB, NS, SK

QUESTION 3:

Have you used advanced non-destructive evaluation techniques (e.g. acoustic emission monitoring) to assist in evaluating the condition of steel truss bridges?

Yes:

AK: We use ultrasonic testing equipment to check the pins at connections

CT: Ultrasonic testing of pins for a truss bridge carrying metro-north railroad over Washington and Main Street in Norwalk Conn. Was performed in 1996. Contact Mr. Robert Brown at ConnDOT (860) 594-3207.

Eye bars have also been tested. Contact Mr. Richard Van Allen (860) 594-3172.

MD: We use ultrasonic testing to determine if defects are present in pins.

MN: Ultrasonic testing of pins and welds.

NJ: Non-destructive testing of pins.

NY: New York City DOT did use non-destructive "X-ray Diffraction Technique" to determine the load distribution in the eyebars at specified pin locations. The testing was done by PROTO Manufacturing Limited, 2175 Solar Crescent, Oldcastle, Ontario, NOR 1L0 Canada. The contact person is R. Mayrbaur, Weidlinger Associates, 375 Hudson St., New York, N.Y. 10014-3656.

OH: No, specifically on a Ohio DOT bridge but did assist a county (Sandusky) with a truss bridge. This structure was load tested to validate the finite element results; the finite element revised; recommendations made for strengthening; and a final rating defined for the structure.

OK: We use Ultrasonic Testing to inspect the pins of pin-connected trusses.

No (with note):

AZ: Used conventional non-destructive testing techniques such as ultrasonic testing of pins, eyebars and impacted members; ultrasonic techniques for determining member thickness; and electronic in-situ hardness testing for material confirmation. Also used pachometer testing and coring for evaluation of concrete substructure, and seismic refraction methods and geotechnical borings to evaluate subgrade conditions for seismic and scour vulnerability. Contact : Rob Turton at Cannon & Associates (602) 470-8477

No: AL, AR, CA, DE, DC, FL, GA, HI, IN, IA, KS, KY, MS, MO, MT, NV, NH, NM, NC, RI, TN, VT, WA, AB, MB, NB, NF, NS, SK

QUESTION 4:

Have you used load testing to assist in evaluation the structural capacity of steel truss bridges?

Yes:

MD: We used load testing to evaluate the capacity of a gusset connected, steel truss in addition to concrete girder and slab bridges and an open spandrel, concrete arch. URS Greiner of Baltimore, MD completed the study in 1996.

MN: The Wabasha Bridge (see report) was instrumented and load tested to determine stress ranges.

NE: [see research report]

OH: [See answer for question 3]

No (with note):

AZ: Used observation of behavior under load, but no formal testing procedure

MO: Years ago a heavily used truss bridge was load tested with strain gauges on the floor system. (The floor system nearly always control our ratings) As I recall the capacity was determined to be appreciably higher than the theoretical values. It was thought at the time that the concrete slab which replaced the timber floor was acting compositely even though shear connectors were not present.

No: AK, AL, AR, CA, CT, DE, DC, FL, GA, HI, IN, IA, KS, KY, MS, MT, NV, NH, NJ, NM, NY, NC, OK, RI, TN, VT, WA, AB, MB, NB, NF, NS, SK

QUESTION 5:

What are the most common structural strengthening techniques your department has used in rehabilitating steel truss bridges?

AK: Replacement in kind of damaged or corroded members

AL: Addition of cover plates. Attached by longitudinal welds to increase the section.

AZ: Old Colorado River Bridge in Yuma County (SN 08533) [Report included]
 Airport Road Wash Bridge in Cochise County (SN)08116) [Report included]
 Cedar Canyon Bridge in Navajo County (SN 00215) - Though an arch bridge an identical historic arch (Corduoy Creek) was disassembled and reassembled to create a stronger wider bridge.

CA: Cover Plates

CT: Plating to replace lost section. Member replacement. Bearing replacement to decrease bottom chord longitudinal stresses due to thermal forces. Reinforcing eye bars. Light weight deck replacement to reduce dead load.

DE: Use of heavier or higher strength steel components.

FL: Replacing members and adding section as needed.

GA: Member replacement for damaged members, cover plates for damaged areas, etc. High strength bolts to replace rivets, etc.

IA: Bolting/Welding new material to existing members. Replace concrete deck with lighter weight steel grid deck. Add new bracing to reduce L/R of compression members. Add wire rope and turnbuckles to strengthen pin-connected eye-bar tension members.

KS: Lighten load by replacing concrete deck with metal grid deck.

KY: Tension tighteners on eye bar members. Additional members added. Building up of members using plating.

MD: Frequently, floorbeams and compression members in the truss have been strengthened by bolting plates or rolled shapes to their webs.

MN: Replacing members. Reinforcing members with additional plates and angles

MO: Most truss bridges in Missouri are functionally obsolete (too narrow, low overhead clearance) and many are posted. General practice is to replace these structures. We have a major river crossing currently being rehabbed and redecked. The longitudinal stringers are being made composite to increase the load rating.

MT: replacing deck, stringer and floorbeams

NE: [post-tensioning used in research project]

NV: We have manually rated a metal truss bridge and then widened it to one side using different truss members but the same truss configuration.

NH: Replace deck and deteriorated members.

NJ: Replace deck system with a lighter system. Install coverplates to strengthen members.

NM: We have done very little work in strengthening old trusses.

NY: An individual evaluation of each structure must be made to determine if one or more of the following techniques is (are) appropriate.

- Decrease dead load to provide additional live load capacity
- Repair or replace deteriorated material
- Post-tensioning elements which have low load capacity (i.e. floor beams)
- Adding a superimposed load carrying system (i.e. steel arches)

Generally these techniques are used to restore lost load carrying capability rather than add additional capability to the original design.

NC: Bridge replacement.

OH: Add plates, reconstruct, when required. Have not used composites or post tensioning. Have pulled a concrete deck to use open grid for increasing live load capacity.

OK: Redecking with a composite concrete deck. Welding steel plates to truss chords.

RI: Heat Straightening. Replace Members and Pins.

TN: Replace members. Add section to members for strengthening. Replace gusset plates.

VT: Replace weak or deteriorated members. Weld additional metal to weak members. Replace existing concrete decks with lighter timber decks.

WA: Replacement of decks (making them composite in some cases)

AB: Cover plates. Post-tension bottom chord with Dywidag rods. Member replacement.

MB: Replace deficient or damaged members. Add cover plates.

NB: Lightweight concrete deck. Composite floor action with concrete decks. Post-tensioning bottom chords. Member replacement, strengthening.

NS: Replacement or strengthening of members and connections is commonly used.

SK: installed additional longitudinal stringers. Installed additional members to lower chord and verticals/diagonals to strengthen deficient members. Gusset plates have generally been adequate, so involved replacement of rivets with longer high strength bolts to accommodate the additional members.

No Response: AR, DC, HI, IN, MS, NF

QUESTION 6:

Please check any other structural strengthening techniques you have used.

Superimposed trusses: CA, HI, NY*, AB

* Superimposed arch/hanger/transverse floorbeam system

Post-tensioning bottom chord: CT, KS*, [NE], VT, AB, NB

*This method was considered and analysis was done but final decision was to replace structure.

Joining simple spans into continuous span: KY, NY

Replace floor deck with a lighter system: AR, AZ, CA, CT, DE, FL, IA, KS, KY, MD, MT, [NE], NH, NJ, NY, NC, OH, TN, AB, NB, NF, NS

Addition of longitudinal beams: AZ, CT, NJ, RI, TN, AB, SK

Providing additional supports: CA, CT, NJ, RI, TN, NB

Adding king or queen posts and post-tensioned tendons:

Pin replacement : CT, FL, GA, KY, MD, MN, NJ, NY, RI, NS

Attach cover plates to members : AL, AZ, CA, CT, FL, GA, IA, KY, MD, MN, [NE], NH, NJ, OH, OK, RI, TN, AB, MB, NS, SK

Other:

AZ : Considered or proposed: Airport Wash Bridge draft documentation attached for reference but County chose to replace bridge due to cost of rehabilitation. Old Colorado River Bridge structural rehabilitation recommendations are presently being considered. (Drafts are attached

for reference) Other techniques include strengthening existing deck by removing existing AC and providing structural concrete (reinforced) overlay and seismic retrofitting (bearing anchorages and pier strengthening).

CA: The strengthening techniques used above have mostly been used on local agency bridges.

MD: It has been our goal in working with the Maryland Historical Trust that we do as little modification to the truss as possible. When needed we will strengthen or replace members, use a lighter floor system, and replace pins but shy away from superimposed arches and trusses and post-tensioning.

MO: Although I am unaware of any trusses rehabbed with cover plates we have strengthened a number of beam bridges using this method.

NM: We've replaced floor decks but not to add strength.

NY: Post-tensioning of floorbeam.

NC: Additional stringers.

OK: Attaching threaded rebar to lower chord. Placing shims under floorbeams at abutments.

WA: Elimination of fracture critical hangers by adding secondary hangers.

NB: Composite stringers with concrete deck.

No Response: AK, DC, IN, MS, NV

QUESTION 7:

For bridges with geometric deficiencies, either inadequate height or width, please check any solutions you have used:

Relaxing geometric standards for historic bridges: CT, FL, IN, KY, [MD (see other)], NH, NJ, NY, OH, VT, NF

Widening bridge: AZ, [NE (tested in research)], NV, OK

Increasing portal height by removing or altering overhead members: AK, CA, DE, FL, GA, IA, KS, KY, MN, MO, MT, NH, NC, OH, OK, TN, VT, WA, AB, MB, NB, NF, NS, SK

Convert bridge to one-way traffic: AZ (Considered or proposed on Old Colorado bridge), [CT (see other)], FL, [MO (see other)], NJ, OK, RI, WA, AB, NB

Other:

AK: Replaced highway bridge and retained truss bridge for pedestrian use

CA: Placing speed restrictions. Placing electronic sensing devices prior to bridges to prevent overheight loads from entering bridge

CT: Convert to pedestrian traffic. Alternating one-way traffic.

GA: Posted low clearances. Relocation of truss.

MD: We believe that narrow structures having low speed limits are not vulnerable to the railing loads prescribed in AASHTO, therefore we grant design exceptions on those trusses having low incidence of accidents.

MO: On some low traffic roadways we have limited some structures to one lane. These bridges are usually narrow and re-stripped to direct traffic to the center of the structure.

NM: Build an adjacent bridge and preserve the existing structure. This is what we usually do.

NY: Convert the bridge to alternate uses such as cars only, pedestrian and/or bicycle use.
OK: Post a reduced speed limit.

No Response: AL, AR, DC, HI, MS

QUESTION 8:

What methods, if any, have you used to improve railings on historic steel truss bridges? We are particularly interested in information on crash tested railings which have been added to historic steel truss bridges?

AK: We have added the “f” shape concrete barrier

AL: Have added 12” metal W-beam guardrail on inside

AZ: On Cedar Canyon used concrete jersey shaped barrier with architectural treatment on the outside to simulate metal rail

Considered or proposed: Extension of existing concrete curbs; concrete jersey-type barriers where feasible from loading standpoint (and acceptable to SHPO); crash-tested open metal railing system; non-standard open metal railing systems that may suit existing framing; and strengthening or providing additional members in existing system where operations permit (such as with one-way traffic of low volume/low speed/no truck facilities.

CA: We have used thrie-beam railing. Some 2 members high and blocked out a small amount.

CT: We have used W-beam rail systems with backing plates and rub rails and concrete AASHTO safety shapes.

DE: We have used glue-laminated timber rails as shown on the enclosed drawing. This detail is not crash tested. [Drawing can be found in collection of responses]

FL: Replace rail and post with Iowa Block railing.

KY: We use Ohio’s curb & guardrail details when they will fit.

MD: We feel that the railing should be in harmony with the truss’ appearance, however we recommend adequate protection for the endposts and smooth transition between approach barrier and the railing on the structure. We also recommend a heavy 12 inch tall timber rubrail attached to the deck to take the brunt of the wheel load and deflect the vehicle back into the roadway before striking truss members or hand railing.

MN: We added thrie beam rail to one truss to achieve a crash tested design.

MO: We have used thrie beams on truss bridges

MT: We have used the Texas T101 and Wyoming box beam.

NH: No crash tested rails used.

NJ: Guide rail carried across structure.

NY: No new crash tested railing systems have been used. However, by reducing speed limits, introducing higher curb/barrier curb lines and eliminating all but delivery and necessary access vehicles to a historic area, new and similar to original steel railing and parapet details have been used.

NC: Add 12” guardrail to pony truss.

OK: We have not improved the railings on historic metal trusses.

RI: Bolt guardrail thru deck.

TN: If we repair the structure by contract, we will specify the 10 gauge W shape guardrail be mounted to the lattice rail in the truss section of the structure. To my knowledge this application has not been crash tested. However, we feel that it is better than the existing condition.

VT: We have used steel box beam tube rails and also W-beam guard rail sections to supplement existing rails. None have been crash tested.

AB: HSS 6" x 8" rail.

NF: None. We use steel beam W guide rail or none. We do have some damaged truss members.

SK: We do not have any historic metal truss bridges. On some older bridges we have installed a heavy angle along the traffic face of lattice type railing, and then installed W. Beam in front of the angle.

No Response: AR, DC, GA, HI, IN, IA, KS, MS, [NE], NV, NM, OH, WA, MB, NB, NS

QUESTION 9:

What methods have you used to deal with the presence of lead based paints on historic steel truss bridges:

Remove old lead paint (with appropriate disposal techniques) and repaint bridge: AK, AL, AZ, CA, CT, DE, FL, GA, IN, IA, KY, MN, MO, NH, NJ, NM, NY, OH, OK, RI, TN, VT, WA, AB, MB, NB, NS

Apply sealer to encapsulate lead based paint: AZ, CT, IN, KY, MO, NH, NM, OH, OK, RI, TN, WA, AB

Other:

AZ: Considered or proposed. Also considered scrape bad areas with proper containment (partial removal) and overcoat.

Cedar Canyon Bridge - existing members were steam cleaned and painted over.

CA: Preventive Maintenance painting is also used to overcoat existing coating.

CT: Sealers are not used anymore

KS: We have no trusses on the State Highway System now classified as historic, but if we did we would recommend removal with appropriate disposal. The county system has some historic trusses, but they rarely get painted. We would recommend paint removal and appropriate disposal also.

KY: Currently we are hand cleaning loose paint & rust and encapsulating most of our bridges.

MD: We have been successfully using a moisture cured urethane coating that does not require 100% removal or near white cleaning. We remove the existing coating down to sound paint and tight mill scale. Lead abatement, containment, and worker protection is our highest priority.

MN: We have spot painted with primer and top coat.

MO: We have used both systems depending on life expectancy of bridge. Aluminum or calcium sulfanate overcoats are used for short term bridges.

MT: We tried to let one contract to fully remove the lead paint. The cost was so high that we rejected all bids. We have one more project that we will try full removal on. This will go to contract in early 1998.

NV: Have not yet had to repaint a metal truss bridge.

NM: We have done both. Encapsulation is the most common.

NC: Spot clean (hand tools) and paint as necessary.

OK: A combination of removal and encapsulation. We did removal at the joint regions for the stringers and floor beams and we did full removal for the lower portion of the truss (bottom 10') and encapsulation every where else.

TN: Removal & Repaint: Normal Abrasive blast (SSPC SP-10) of steel followed with an inorganic zinc, epoxy tie coat and a urethane topcoat. Total containment with negative pressure.

Encapsulation: Surface prep. Include pressure washing (3000-4000 psi) existing steel and grinding rusted areas. The waste is collected, tested and disposed according to EPA standards. Overcoating is done with universal primers or epoxy mastics based on existing paint system.

SK: No recent projects. Some repainting projects in the past involved removal of old paint and repainting but without containment. We would have to use different procedures if we were to handle these projects today.

No Response: AR, DC, HI, MS, [NE], NF

QUESTION 10:

Has your department been involved with the rehabilitation of a historic steel truss bridge that has involved a particularly interesting, unique, or innovative approach?

Yes:

CA: Our department administered H.B.R.R. funds on a local agency bridge project where the structure was historic; it was unique in that the bridge was essentially replaced in kind, element by element.

Bridge Name : Deer Creek (#17C-0001) @ Pine St., Nevada City, CA. (Gault Bridge)

Type : Three Hinged Deck Truss Arch (150' Main span)

Date of Construction : 1903

Owner : City of Nevada City

Contact : William Falconi, City Engineer & Project Resident Engineer

Ph. (916) 265-2496

317 Broad St.

Nevada City, CA 95959

CT:

- We replaced the deck of bridge #1487 with precast concrete panels during off peak hours (nightly) to accommodate high traffic flows.
- Some have been left in place but no longer support traffic.
- Route 1?Patchogue River, Westbrook #349. This truss was scheduled for rehabilitation under a painting project. Department of Environmental Planning regulations made the painting cost prohibitive so a larger truss will be built and swapped with the existing. Bob Zaffetti or Sowatei Lomotey (860) 594-3402.
- East Haddam swing bridge #1138 - proposed deck replacement on a 456' swing span. Replacing the existing light-weight concrete filled steel grid with "Alumadeck" (A light-weight extruded aluminum decking system).

FL: Yes, but simply utilizing more advanced analyses.

NM:

- NM -502/ Rio Grande

- San Juan Pueblo Rio Grande Bridge
- Montezuma Bridge
- Old US-66/ Rio Puerco Bridge
Jose Rojas NMSH&TD Bridge Engineer
(505) 827-5465

TN: Walnut Street Bridge over Tennessee Rive, Chattanooga, TN
A.G. Lichtenstein & Assoc., Inc.
45 Eisenhower Drive
Paramus, NJ 07652

No (with comments):

AL: Just deck replacement, member replacement and cleaning and painting

AZ: Cedar Canyon Bridge was an innovative solution but was a steel arch not a truss bridge. The bridge is located on US 60 at mile post 323.44 south of ShowLow. Contact James R. Pyne (602) 255-8601

MD: Frederick County in Maryland has performed many ground up restorations of historic trusses. Contact Mr. Tom Meunier, Division Chief at (301) 696-2950. Also, Baltimore County, Mayland has rehabbed several trusses. Contact Mr. James Arford, Division Chief at (410) 887-3764. These gentlemen, being directly responsible for the projects should provide you with the information desired. We consult frequently with Mr. Aba G. Lichtenstein of Tenafly, NJ. He can be reached at (201) 567-7381

NY: See the attached article on Stuyvesant Falls Bridge.

Ryan - Biggs Associates P.C.
291 River Street
Troy, New York 12180
Contact : H. Daniel Rogers
Phone: (518) 272-6266

Jai B. Kim, P.E., PhD
Bucknell University
Department of Civil Engineering
Lewisburg, Pennsylvania 17837
Phone: (717) 524-1112

No: AK, AR, DE, DC, GA, HI, IN, IA, KS, KY, MN, MS, MO, MT, [NE], NV, NH, NJ, NC, OH, OK, RI, VT, WA, AB, MB, NB, NF, NS, SK

QUESTION 12:

Additional Comments:

AR: In general, when metal truss bridges are retained for historical purposes vehicular traffic is prohibited.

CA: The most extensive work Caltrans has done was by the Toll Bridge Unit on the San Francisco-Oakland Bay Bridge in 1960-61. The upper deck was designed for H10 with trucks and rail transit on the lower deck. The rails were removed and the decks were converted to 5 lanes each direction. The upper deck was strengthened by adding high strength cover plates to the floor beams and adding stringers between the existing stringers. The decks are lightweight concrete.

IN: Indiana has on the state highway system: 64 steel thru trusses, 22 steel pony truss, 3 deck trusses, 1 Bailey truss. This does not include metal truss bridges on city or county roadways.

KS: The State Highway System had one bridge removed several years ago and had to document, photograph and preserve it in the records. The counties have done this a few times.

MD:

- Replace only those members that do not rate out to desired load. Since most trusses were designed for 100 psf of deck, the top and bottom chords almost always rate out at or higher than H15 in the inventory stress range. Intermediate vertical members typically rate out well above H15 also. Take coupons from batten plates and have them tested for yield strength. The allowable stress derived from the yield values are typically higher than those recommended in AASHTO and other texts. Diagonals, hip verticals and pins are occasionally under capacity as joint loads from concentrated axle loads often exceed those resulting from the original, uniform design load even when using higher allowable values.
- Consider using glue-laminated timber decking in the replacement deck. This type of deck matches or exceeds the life span of plank decking, is typically less thick and therefore lighter than plank decking and virtually eliminates debris and moisture build-up on the steel framing below.
- When replacing the existing stringers, design the new ones as continuous. The design is slightly more economical and adds an additional level of redundancy to the bottom chord.
- Reuse the floorbeams by bolting channels to their webs provided they are in good condition. Floor beams will typically fail in bending due to axle loads not originally designed for. However, when you decide to replace the floorbeams, it becomes necessary to dismantle the bottom diagonal bracing and the U-hangers to the pins. By salvaging the floorbeams the overall project cost can be reduced and the number of original members to be incorporated in the rehabilitated structure increased.
- Try to salvage pins. Ultrasonically test them beforehand to verify that excessive wear or grooving is not a problem. Rate them for bending and shear. We typically increase the allowable stress by 50% over what was determined by testing batten plate material.
- Avoid truss disassembly as much as possible. Many trusses are especially unique as a result of odd details which are often destroyed in the dismantling process. The contractor's methods must be carefully reviewed to ensure that in the process of completing necessary repairs, the integrity of these details is not compromised.

MS: Sorry that we could not be of help.

MT: We have rehabilitated 2 truss bridges but have several more in the future. We look forward to your final report.

NM: We've had a fair amount of discussion about these bridges lately. We've mainly been building a parallel new bridge & rehabilitating the existing bridge for pedestrian and horses. We've tried to build up a large enough bank of this type of bridge so that we can remove & destroy the existing bridge where building a parallel bridge isn't possible.

NY: A list of persons that may provide additional information follows:

Abba Lichtenstein	William P. Chamberlain
26 Trafalgar St.	1046 Shave Court
Tenafly, New Jersey	Schenectady, New York 12303
07670	Project Manager NCHRP Topics 28-08
	"Historic Highway Bridge Preservation Practices"

NC: There are approximately 100 truss bridges in North Carolina most of which are small 1 lane bridges. Fifty of these are scheduled to be replaced within the next 5 or 6 years. Only 5 truss bridges are of any size (i.e. large), and are not scheduled for replacement or repair.

VT: We have had a consultant study done on approximately 110 truss bridges. This study, at a cost of approx. \$10,000 per bridge, did not include a detailed structural capacity study of each

bridge but did give overall recommendations for each site. Enclosed is a copy of a draft of one report. Obviously, it is not practical to send you a copy of all 110 reports, but this one will give you an idea of what was done.

WA: [on enclosed letter]

Our Department commissioned a multi-discipline independent team (HAER-Historic American Engineering Record) to conduct a Historic Washington Bridge Recording project in 1993 that documented 30 of the most historically significant bridges in the state. The majority of these historic bridges have steel truss construction and pre-date 1940. While “historic” rehabilitation has not been commonplace, timely maintenance and painting of mainspans to extend bridge service life; such a project is currently under development for the 1911 City Waterway Bridge in the city of Tacoma, Washington.

Aside from providing redundancy to certain fracture critical bridge elements, and replacement of deteriorated decks (and some steel members where section loss due to corrosion warrants), our Department does not have a policy or program to perform bridge strengthening to improve live load capacity. The Department does not have guidelines regarding the preservation of historically aesthetic features of these older steel structures (such as ornamental rails).

NF: We have no historic steel trusses on the present Highway System - just some trusses built in the sixties which are in the main galvanized and have served us well. However we have a now deficient railway with many trusses. These are now the responsibility of the Province and this is why we have an interest in rehabilitation of old trusses. Some of these trusses could be classed as historic.

Appendix C: Annotated Bibliography

1. GENERAL REFERENCES

General Reference [1.1]

Burke, Martin P., Jr. "Enduring Symbol of American Endeavor." Transportation Research News, March-April 1989, 3-8.

This article describes the first cast iron bridge built in America. The bridge was built on the National Road (Cumberland), originally built to encourage settlement to the west. The author tells the history of the design and construction of the bridge.

General Reference [1.2]

Nichols, G., and R. McGee. "A Management Plan for Richmond Bridge, Tasmania." Road & Transport Research 4, No. 2, June 1995, 4-14.

Summary of the management plan developed for Australia's historic bridges. This article outlines the following topics which are included in a typical historical report: 1) Historical record: includes construction, repairs, and major events 2) Statement of Significance: significance of bridge to national and local history 3) Condition of bridge: material conditions and properties 4) Terrestrial Photogrammetry: pictures to measure quantities on and around bridge 5) Hydraulic Analysis: assessing flood risks at bridge 6) Structural Analysis: bridge analyzed for different types of loading on superstructure and substructure 7) Review Process: request for outside agencies or committees to comment on findings and make recommendations 8) Recommendations: recommended immediate, continual, and future work on the bridge.

General Reference [1.3]

Bigelow, Lawrence N. "Fifty-Year Development-Construction of Steel Truss Bridges." Journal of the Construction Division, Proceedings of American Society of Civil Engineers 101, No. C02, June 1975, 239-257.

This article provides general background information for the history of metal truss bridges in America. The author discusses many bridge types and construction techniques. Simple, Cantilever, and Continuous truss bridges are also described in this article.

General Reference [1.4]

Lichtenstein, Abba G., and Mary Elizabeth McCahon. "Historic American Bridges." In Structural Preservation of the Architectural Heritage: Proceedings of the Symposium in Rome, Italy, 1993, by the International Association of Bridge and Structural Engineering. Italy, 1993 573-80.

In this paper, the author gives a brief overview of bridge preservation in America. Firstly, a discussion of what constitutes a historic bridge is presented along with examples of historic bridges. Secondly, examples of bridges that were repaired and some that were destroyed are given. In conclusion, the author states that the preservation of bridges should be dealt with on a case-by-case basis.

General Reference [1.5]

Watson, Sarah Ruth. "Some Historic Bridges in the United States." Journal of Professional Activities, Proceedings of the American Society of Civil Engineering 101, no. 3, July 1975, 383-390.

This article is a brief checklist of famous bridges in America. The author discusses the requirements for a bridge to be deemed historic. In eloquent language, the author describes the significance of many bridges that relate to not only engineering feats, but also significant times in U.S. history. The author also focuses on the symbol of a bridge being used in many cultures, folklore, and history.

General Reference [1.6]

Lichtenstein, Abba G. "Historic Bridges: Conflict Ahead." Civil Engineering 7, no. 5, May 1987, 64-6.

This article briefs a few preservation conflicts/solutions the author has been involved with. A 75 year old steel truss in Hawaii, after much input from the public, was converted to a one-way bridge, two-way traffic pattern. A 90 year old lenticular truss in Somerset County New Jersey was rehabbed by replacing some deteriorated members and adding high strength beams inside existing built up members. A bridge in Califon, New Jersey was widened by cutting it in half and new floorbeams were added. A bridge in West Virginia was dismantled and moved to a golf course. The worst case was a chain link suspension bridge in New York which was judged structurally deficient and was removed for fear of sudden collapse due to flooding. Detailed drawings and certain details were saved for future reference. A list of "rules of thumb" of bridge preservation are given.

General Reference [1.7]

Lichtenstein, Abba. "Impact Fracture in Historic Bridges." In Structures Congress XIII: Proceedings of papers presented at the Structures Congress '94, Atlanta, Georgia, April 24-28, 1994, sponsored by the Structural Division of the American Society of Civil Engineers. Atlanta, Georgia, 1994, 1289-1292.

This article outlines failures that have occurred to historic truss bridges and the outcome of each situation. The first bridge discussed was a suspension truss bridge that collapsed after a car impact. Pieces of the bridge were salvaged from the river and an investigation into reconstructing the bridge was undertaken. Because of monetary constraints, the bridge was not restored. The second bridge, a wrought iron Phoenix truss, suffered extensive damage to one truss but did not collapse due to the other truss accepting the additional dead load. The bridge was repaired at a minimal cost compared to a new structure. The last bridge suffered a partial collapse due to an overload caused by a power generator placed on the bridge during a repair. The rehabilitation involved hiding new members inside the old members to reduce the stresses.

General Reference [1.8]

Report of the Secretary of Transportation to the United States Congress. The Status of the Nation's Highways and Bridges: Condition and Performance and Highway Replacement and Rehabilitation Program 1989. Washington D.C.: Government Printing Office, June 1989.

This report outlines many factors which effect the nations bridges. Chapters on Highway Finance, Conditions and Performance, and Highway and Bridge Needs for the Future are presented. These chapters detail the inner workings of how the national government views the status of the bridges on the highway system. A second part of the report focuses on the bridge replacement and rehabilitation program. An explanation of the goals and methodology of the replacement program is included. The report also describes the various funding types available for bridge rehabilitation or replacement. This report was very helpful in explaining how the government is dealing with the problem of deficient bridges.

General Reference [1.9]

Sanders, Wallace W. "Bridge Repair and Rehabilitation, North American Codes and Practice." In Structural Faults and Repairs: Proceedings of the 5th International Conference on Structural Faults and Repairs at the University of Edinburgh, July 1, 1993, by the International Conference on Structural Faults and Repairs, 1993, 9-12.

This article provides an introduction to the codes that govern or provide guidance for bridge repair. References to strengthening both concrete and steel bridges are given. A discussion of whether existing codes should apply to old structures is presented. An extensive bibliography lists research projects relating to various topics in bridge repair and rehabilitation.

General Reference [1.10]

Zuk, William, and Wallace T. McKeel, Jr. "Adaptive Use of Historic Metal Truss Bridges." Transportation Research Record 834, 1-6.

Twenty bridges in Virginia were surveyed for different methods of preservation. Two major options were proposed: (a) continued vehicular use (b) convert to non-vehicular use. Four sub-options for continued use include: (1) upgrade by strengthening (2) widening (3) convert to on way traffic and build secondary bridge (4) move bridge to less demanding traffic location. Article focused on strengthening techniques which included: (1) join simple spans to form

continuous span (2) add pylons and cable stays to bridge (3) posttension bottom chord of individual trusses (4) add queen post under individual trusses (5) place additional supports under trusses (6) add longitudinal beams under trusses (7) add an additional truss on the outside of old truss. A discussion of non-vehicular options such as conversion to a footbridge, restaurant, museum, etc. was offered in the article.

General Reference [1.11]

NATO Advanced Research Workshop on Bridge Evaluation, Repair, and Rehabilitation; A.S. Nowa, ed. Bridge Evaluation, Repair and Rehabilitation. Dordrecht, Boston: Kluwer Academic, 1990.

This book presents articles related to different topics in bridge repair. Major issues covered include: Bridge Management, Diagnostics and Monitoring, Loads and Analysis, Evaluation and Tests, and Repair and Rehabilitation. Only a few of the articles use metal truss bridges as case studies. The most informative article related to truss bridges is titled "Rehabilitation of Steel Truss Bridges in Ontario". The article outlines four bridges and the solution to each of the bridges' deficiencies. The Burlington Skyway Bridge's floorbeam trusses were strengthened using Dywidag bars. The South Muskota River Bridge was rehabilitated by adding a new deck truss to halve the existing span. The Buskegau River Bridge was replaced after extensive cost analysis was completed. A new bridge was found to be much more economical. The Confederation Drive Bridge was rehabilitated by removing the existing deck and replacing it with a lightweight, pre-stressed timber bridge.

General Reference [1.12]

Silano, Louis G., ed. Bridge Inspection and Rehabilitation: A Practical Guide. New York: John Wiley & Sons Inc., 1993.

This book discussed many topics concerning the repair of concrete and steel bridges. Chapters on Bridge Inspection, Steel Structures, and Deck Reconstruction, would be of particular interest for an engineer working on a rehabilitation project. The Steel Structures chapter provides an introductory discussion of topics including damage and strengthening, as related to bridges. This book is an excellent reference for an explanation of repair schemes used in steel bridges.

General Reference [1.13]

Horn, W. B., G. O. Shanafelt. Guidelines for Evaluation and Repair of Damaged Steel Bridge Members. Washington, D.C.: Transportation Research Board, National Cooperative Highway Research Program 271, June 1984.

This report is a very good review of repair methods for members subject to impact, fire, or other damage. Chapters dealing with inspection, assessment, repair selection, and guidelines of repair methods, are included. Repair techniques such as flame straightening, welding, hot mechanical straightening, and bolting are covered as repair techniques. This document was referred to extensively during the writing of this thesis, and the author would strongly recommend any engineer approaching a rehabilitation to have a copy of this document on hand.

General Reference [1.14]

University of Virginia Civil Engineering Department, Virginia Highway and Transportation Research Council, and the Virginia Department of Highways and Transportation. Bridges on Secondary Highways and Local Roads. Washington, D.C.: Transportation Research Board, National Cooperative Highway Research Program 222, 1980.

This report deals with common bridge deficiencies on secondary roads or highways. This report included repair and replacement procedures for concrete, steel, and timber bridges. Topics such as railing retrofits, and geometric clearances are discussed. Information is also given on the repair of bridge substructures. A complete section of replacement schemes is also included for a variety of bridge members. During the writing of this thesis, many examples, and figures were taken from this report. The author would strongly recommend any engineer approaching a rehabilitation to have a copy of this document on hand.

General Reference [1.15]

Klaiber, F. W., K. F. Dunker, T. J. Wipf, and W. W. Sanders, Jr. Methods of Strengthening Existing Highway Bridges. Washington, D. C.: Transportation Research Board, National Cooperative Highway Research Program 293, 1987.

This report is an excellent reference for any engineer seeking information on a wide range of rehabilitations. The authors examined over 300 references related to bridge repair. The referenced were grouped into general classifications including member replacement, stiffness modification, member additions, and post-stressing. The authors also discuss economic analysis as related to bridge replacement versus repair. A bibliography is included of references used, and would be helpful to an engineer.

General Reference [1.16]

Zuk, William, Howard Newlon, Jr., and Wallace T. McKeel, Jr. Methods of Modifying Historic Bridges for Contemporary Use. Charlottesville, Virginia: Virginia Highways & Transportation Research Council, 1980.

This report focuses on the rehabilitation options for 21 bridges in Virginia. A detailed investigation into each bridge structure was undertaken including history, architecture, and structural aspects. Continued vehicular service as well as conversion to non-vehicular uses were investigated. A bibliography of related articles is also included. This report provides a good introduction to bridge preservation.

General Reference [1.17]

Xanthakos, Petros. Bridge Strengthening and Rehabilitation. Upper Saddle River, New Jersey: Prentice Hall PTR, 1996.

This book provides a wide variety of information on bridge strengthening, rehabilitation and replacement, for a number of different types of bridges, including trusses.

General Reference [1.18]

Farago, B. "Rehabilitation of Steel Bridges in Ontario," in Bridge Evaluation, Repair and Rehabilitation. Dordrecht, Boston: Kluwer Academic, 1990, p.555-566 [Ref. 1.11]

The paper deals with the rehabilitation of truss bridges, which often involves strengthening as well. The key question, whether to rehabilitate or replace a bridge is complex. The engineer must have a good knowledge of the condition of the bridge, the actual load capacity and reliable cost estimates for rehabilitation. Increasingly present value based on life cycle costing is used in the decision making. Alternative methods of strengthening and the decision process followed in Ontario are illustrated through case histories.

General Reference [1.19]

Danko, George Michael "The Evolution of the Simple Truss Bridge 1790-1850: From Empiricism to Scientific Construction" University of Pennsylvania, Ph.D. Dissertation, 1979.

This work has placed the evaluation of simple truss bridge within the context of the social, economic, technical, and scientific changes which were occurring in America during the late eighteenth century and first half of the nineteenth. In choosing this approach, it was recognised that this type of bridge and its antecedents are more than their mere structural entities. On the contrary, because the early development of the simple truss spans the breath of one of the important periods of American history, it can be used as a vehicle to explore those changes, which influenced its growth.

General Reference [1.20]

Bucak, Omer; Mang, Friedrich. "Erfahrungen mit alten stahlkonstruktionen (Experiences with old steel structures)" Stahlbau 67, p. 46-60, 1998.

Since their development, numerous steel structures of the 19th century, especially railroad bridges are continuously in operation for some decades partly for reasons of monument preservation, partly for economical reasons. Nowadays, the knowledge of steel applied at that time and of rivet technology as most important connections is mastered only by a few experts. In the first part a report about rivet technology and some information about some new knowledge are given. In the second part it will be reported about the fatigue behaviour, strengthening measures and also about corrosion state of old structures.

General Reference [1.21]

Ingram, Michael N.; Hill, Sean M. "Rehabilitating a Steel Truss Bridge for under \$50, 000" Public Works v.124, May 1993, p. 60-61.

Inexpensive rehabilitation of a steel truss bridge owned by Atlantic County, New Jersey, has kept the county bridge in service. The bridge had a previous sufficiency rating of 28 percent and was not in good overall condition. It was decided to rehabilitate the bridge by converting it from a single –span truss to a 4 –span trestle.

General Reference [1.22]

Wallsgrove; J. J.; “Aesthetic Aspect of Widening and Rehabilitating Historic Bridges” in: Bridge Management 3 : Inspection, Maintenance, Assessment, and Repair, edited by J.E. Harding, G.E.R. Parke and M.J. Ryall, Published by E & FN Spon, 1996.

This paper gives basic principles and detailed advises on aspects of the appearance of bridges when they are being altered or refurbished. It applies particularly to historic bridges, but the advice should be useful to designers working on the refurbishment of all structures.

General Reference [1.23]

Gasparini, Dario, Simmons, David “American Truss Bridge Connections in the 19th Century II: 1850-1900” Journal of Performance of Constructed Facilities, ASCE Vol. 11, No. 3, August 1997, 130-140.

Following the introduction of iron in the 1840s, bridge companies came to dominate bridge procurement. Their designs featured proprietary compressive elements and joint details. As analysis of statically determinate trusses evolved into an ordinary skill, statically indeterminate forms and prestressing fell into disuse. The statically determinate “American pinned truss” with built-up compressive elements and eyebars became the mainstream design. American fabricators produce ductile eyebars with extraordinary dimensional accuracy. However, all-riveted lattice trusses, principally designed by New York Central Railroad engineers, provided competition. In general, such designs performed satisfactorily and helped to calibrate approximate design procedures for riveted joints. The development of shop and field riveting equipment finally made riveted, gusset-plate connections practical by the turn of the century.

General Reference [1.24]

“Historic Highway Bridge Preservation Practices,” Synthesis of Highway Practice 275, Transportation Research Board, 1999.

This synthesis report will be of interest to state highway design engineers and structural engineers, as well as environmental and historic preservation personnel in transportation agencies. It will also be of interest to state historic preservation offices, federal historic preservation agencies, and engineering preservation consultants. It describes the current state of the practice and experience of state and local transportation agencies dealing with the preservation of historic bridges in their jurisdictions. Information for the synthesis was collected by surveying U.S. state and federal transportation agencies and by conducting a literature search.) The synthesis is intended to complement NCHRP Synthesis 101, Historic Bridges - Criteria for Decision Making (1983), and draws on that work by reference. This report of the Transportation Research Board provides information on the policies, decision criteria (or models), and administrative practices used to determine which historic bridges to preserve and the specific preservation option to be employed with each. It also emphasizes the experience of highway agencies in administering these policies and practices, describes a number of successful examples, and identifies some unresolved issues. Several case studies are included to highlight

the differences between the policies and practices of state transportation agencies and preservation agencies.

General Reference [1.25]

American Association of State Highway and Transportation Officials. AASHTO Reference Book of Member Department Personnel and Committees. Washington, D.C., 1996-1997.

This book contains names and addresses of individuals who work at transportation departments. Information is also included for transportation departments in Canada. This document was used to compile a mailing list for the survey.

2. REHABILITATION TECHNIQUES

Rehabilitation: General [2.1.1]

Morf, Ulrich. "Investigation of Obsolete Structural Elements and Retrofit of Old Steel Structures." International Association of Bridge and Structural Engineering: 547-52.

The characteristics of old steel structures are different than modern steels making evaluation of such structures difficult. This article suggests methods of repairing a variety of joints using high strength bolts, welding, and external prestressing. However, first an evaluation of the material must be completed to determine its properties. The author suggests ultrasonic, magnetic particle or penetration testing. A discussion of the fracture characteristics of old bridge steel is included with formulae to quantify Charpy tests.

Rehabilitation: General [2.1.2]

"Bridge Rehabilitation Saves a Lost Art." Modern Steel Construction, October 1997, 58-62.

This article describes a rehabilitation effort completed on a lenticular truss built in 1886. The wrought iron bridge had been closed to vehicles in 1969 and converted into a pedestrian bridge. Repairs to the bridge included: replacement of fracture critical elements with high strength steel, replacement of bearings at abutments, substructure repair, removal of existing concrete deck and replacement with concrete filled steel grid deck, floorbeam repairs, and repainting.

Rehabilitation: General [2.1.3]

Stolldorf, Dennis W. "Fire Damaged Bridge Requires Major Repair." Public Works 121, no. 12, December 1990, 32-3.

A bridge in Washington D.C. was badly damaged due to a ruptured fuel tanker below the bridge. Concrete of the piers was spalled, as well as severe damage to many of the steel components. The repair consisted of demolishing the damaged concrete and replacing it. Damaged portions of the steel girder were removed and new plates welded in their place. It

should be noted that the bridge was not historic, therefore welding was not difficult on the modern materials.

Rehabilitation: General [2.1.4]

Mishler, H. W., and B. N. Leis. Evaluation of Repair Techniques for Damaged Steel Bridge Members. Washington, D. C.: Transportation Research Board, National Cooperative Highway Research Program 12-17, 1981.

This report provides information on four common techniques of repairing damaged bridge members including welding, cold mechanical straightening, hot mechanical straightening, and flame straightening. A majority of the document investigates hot mechanical straightening and flame straightening. The authors concluded that the effects of these procedures are very variable and rely primarily on the skill and expertise of the person using the technique.

Rehabilitation: General [2.1.5]

Griggs, Francis E. “1864 Moseley Wrought-Iron Arch bridge: Its Rehabilitation” Practice Periodical on Structural Design and Construction, May 1997, 61-72.

The rehabilitation of a Moseley wrought iron arch is described along with a history of the development of the arch and its builder. The bridge, originally built in 1864 over the North Canal in Lawrence, Mass., is the last bridge of its type that is currently carrying load in the United States. It is the oldest iron bridge in Massachusetts and one of, if not the oldest, riveted wrought iron bridges in the country.

Rehabilitation: General [2.1.6]

Gregory, E. N.; Slater, G.; Woodley, C.C. “Welded Repair of Cracks in Steel Bridge Members” TRB Record 321, Transportation Research Board, October, 1989.

This report contains findings of a study that was performed to identify and evaluate welding methods for repair of cracked steel bridge members in order to restore their load carrying capacity and fatigue life. The report provides a manual of recommended practice for steel bridge member repair through field welding that if, followed, should lead to good quality repair welds. The manual is based on an examination of bridge repair techniques previously used in the field and typical bridge member repaired performed during course of this study.

Rehabilitation: General [2.1.7]

Brustad; Vik, A.; Engesvik, K. “Assessment and Rehabilitation of a Fracture Bascule Bridge” in: Bridge Management 3 : Inspection, Maintenance, Assessment, and Repair, edited by J.E. Harding, G.E.R. Parke and M.J. Ryall, Published by E & FN Spon, 1996.

This paper describes the main events in a rehabilitation process following the fracture of a bridge member. Some topics of general relevance to old steel bridges are briefly commented on.

Rehabilitation: General [2.1.8]

Oosthuizen, A. P.; Varkevisser, J. H. "Durability Enhancement of Half Joints" in: Bridge Management 3 : Inspection, Maintenance, Assessment, and Repair, edited by J.E. Harding, G.E.R. Parke and M.J. Ryall, Published by E & FN Spon, 1996.

The aim of the rehabilitation of a 40-year-old major riveted bridge, which is in serious jeopardy due to the deterioration of 8 half joints, is to reduce the existing stress concentration at the bearing location. Shear sections at the half joints are doubled and partial continuity is introduced. It introduces even greater load carrying capacity and enhanced durability with half the cost of rebuilding half joints.

Rehabilitation: General [2.1.9]

U.S. Department of Transportation. Federal Highway Administration. Retrofit Railings for Narrow Through Truss and other Obsolete Bridge Structures. Report FHWA RD-82/099, by M.E. Bronstad, L. R. Calcote, C. E. Kimball, Jr., of the Southwest Research Institute. San Antonio, Texas, 1986.

Rehabilitation: Post-Tensioning [2.2.1]

Ayyub, Bilal M., Ahmed Ibrahim, and David Schelling. "Post-tensioned Trusses: Analysis and Design." Journal of Structural Engineering 116, no. 6, June 1990, 1491-1506.

This article describes a structural stiffness analysis method of evaluating post-tensioned trusses. Post-tensioning allows a gain of strength, as well as, introducing redundancy into the design. Post-tensioning increases the elastic range and reduces the force on members. Stiffness matrices are developed for straight, one-drape, and two drape configurations. It is possible to use either internal or external tendons. Internal tendons, as the name implies, are contained inside the truss system. Internal tendons lessen tension member forces with no, or slight increase, in compression member stresses. External tendons are placed on the exterior of the truss system usually below the bottom chord. These tendons are more effective in reducing both tensile and compressive forces, but might unacceptable due to geometric clearances. A statically determinate and indeterminate bridge with internal tendons and a statically determinate truss with external tendons were both analyzed in this study.

Truss 1: Statically determinate truss with (a) a straight tendon (b) a one-drape tendon (c) a two-drape tendon. The straight and two-drape tendons reduced the tensile stresses on members which coincided with the tendon. The one-drape tendon, while reducing all tension stresses, caused increases in some compression members.

Truss 2: Statically indeterminate truss with (a) two straight tendons, one two-drape tendon (b) a two-drape tendon (c) a straight tendon. Again, all three cases saw a reduction in tension members which coincided with the cable layout. The redundant members of the truss experienced a reversal of stresses which should be accounted for in the design.

Truss 3: Statically determinate truss with external two-drape tendons. Three cases were investigated for different distance (h) between the bottom chord of the truss and the post-

tensioned tendon. In all three cases both compression and tension members are relieved of some of their stress. With increasing (h), the reduction in stresses increased.

Conclusions: Tension members can be strengthened by using internal coincidental tendons. Both tension and compression members may be strengthened by using external tendons. This study was theoretical and dependent upon assumptions such as: a) linear elastic materials b) frictionless joints c) constant tendon forces throughout the member d) 2-D geometry.

Rehabilitation: Post-Tensioning [2.2.2]

“Cables Rejuvenate Old Truss Span.” Engineering News Record 223, September 7, 1989, 21.

A pin-connected, camelback through truss in Tennessee was closed in 1978 and saved by local historical group. The local DOT proposed replacing deteriorated members for a cost of \$8 million. A.G. Lichtenstein & Assoc. Inc. Proposed \$4 million post-tensioned cable solution. Pairs of 0.6 inch tendons were placed coincidentally to existing diagonal members and the bottom chord to relieve dead load and account for the live load of proposed trolley and pedestrian traffic.

Rehabilitation: Post-Tensioning [2.2.3]

Ayyub, Bilal M., and Ahmed Ibrahim. “Post-Tensioned Trusses: Reliability and Redundancy.” Journal of Structural Engineering 116, no. 6, June 1990, 1507-1520.

The concept of post-tensioned trusses has been used in many rehabilitation efforts. Most studies and experiments focus on stress level reduction or increases in stiffness and fatigue resistance. This paper focuses on reliability and redundancy changes due to post-tensioning a bridge. Three tendon layouts including one-drape, two-drape, and straight were investigated. It was confirmed that the tendons work to reduce stresses in coincidental members. External tendon layouts (e.g. king post) reduce stress levels in most of the members in the structure.

System reliability was determined by using an event-tree analysis. The study demonstrated that a post tensioned structure with a classic definition of redundancy of one, actually has a higher level of redundancy dependent upon the tendon layout. For example, a redundancy factor of over three was calculated for the straight tendon layout.

Rehabilitation: Post-Tensioning [2.2.4]

Belenya, E. I., and D. M. Gorovskii. “The Analysis of Steel Beams Strengthened by a Tie Rod.” International Civil Engineering Monthly 2, no. 9, 1971/72, 412-419.

This article presents a straightforward analysis of steel beams strengthened by prestressing rods. The authors describe very clearly their analysis methods. Equations are developed which relate tie rod placement, tension in the rod, length of rod, and number of rods. Using these equations the stress resultants for different configuration may be found. The authors also discuss deflection calculations. An example problem is worked out to show these techniques in practice. General conclusions include: (1) the load carrying capacity of a rolled section may be increased by 80-90% (2) the length of the tie should be between 0.5-0.7 the

length of the beam (3) the initial deformations of the beam should be taken in to account if h/l is less than $1/20$ and if the length of the tie is to be between 0.8-1.0 the length of the beam.

Rehabilitation: Post-Tensioning [2.2.5]

Li, Wulin; Albrect, Pedro; Saadatmanesh, Hamid “Strengthening of Composite Steel – Concrete Bridges” Journal of Structural Engineering, Vol. 121 no. 12, December 1995, 1842-1849.

The concept of prestressing has been used as an effective technique to strengthen composite steel-concrete girders in bridges and buildings. Although several researchers have investigated the behaviour of this structural system under static loading, no one has addressed the fatigue behaviour. This paper analyses and summarises the fatigue test data for the main constituents of prestressed composite steel concrete girders- strands, shear studs, and cover plate. It appears that the end anchored strands for unbonded, prestressed structures can be designed for fatigue to the AASHTO allowable stress range for category C. AASHTO’s approach to designing shear studs and cover plates seems reasonable for prestressed composite girders.

Rehabilitation: Superimposed Truss [2.3.1]

Kim, Jai B., Robert J. Brungraber, and Robert H. Kim. “Recycling Bridges.” Civil Engineering, November 1988, 58-9.

This brief article describes the use of a superimposed arch to strengthen an existing bridge. It provides general information on the concerns and advantages of this system.

Rehabilitation: Superimposed Truss [2.3.2]

“Old Truss Bridge Rehabilitated.” Highway and Heavy Construction 128, Feb 1985, 74-5.

A truss bridge with buckled floor beams and a posted 3 ton load limit was strengthened to a 20 ton limit. Superimposed steel arches and floor beams of A572-50 steel were added to the old truss. Cost of rehab approximately \$62,000 compared to estimated cost of new bridge (about \$200,000). “Underlying concept of their technique...the combination of a reinforcing arch with an existing truss system can carry a significant extra load if it is well supported laterally”. This rehabilitation solution was proposed by Brungraber and Kim of Bucknell.

Rehabilitation: Superimposed Truss [2.3.3]

Kim, Jai B., Robert J. Brungraber, and John M. Yadlosky. “Truss Bridge Rehabilitation Using Steel Arches.” Journal of Structural Engineering 110, no. 7, July 1984, 1589-97.

1:7 scale model of a truss bridge was fitted with steel arches to investigate this type of strengthening technique. The model consisted of steel tubes, eyebars, and rods while actual bridge was channels, laced members, etc. Connections of bridge were represented in the model with a combination of pins and welds. Two arches made of channels back-to-back were placed on the outside of the truss to carry total dead and live load. This apparatus was subjected to four testing stages: (1) model without arch (2) model with arch and ends restrained (3) #2 with one member of bottom chord of original truss removed (4) #2 with two bottom chord members

removed. Load was applied to different panel points and deflections were measured. Results yielded that deflections were decreased by 30-40% on average. When members were removed from the truss (tests (3) & (4)) arches prevented collapse of system. Discussion of benefits of superimposed arches. Written by Brungraber and Kim of Bucknell.

Rehabilitation: Superimposed Truss [2.3.4]

Brungraber, Robert J., and Jai B. Kim. "Rehabilitation of Steel Truss Bridges Using a Superimposed Arch System." Transportation Research Record 950, 1984, 113-120.

The article described a rehabilitation project for a 74 ft. Pratt truss in Pennsylvania. The superimposed arch method was used, which acted to accept all live loads of the bridge and increased the rating to HS-20. The project saved the historic bridge and was completed in only 3 weeks at a cost of \$62,000. New floor beams were added which aided in reducing the load resisted by the pre-existing floor beams. Floor beam were repaired while the bridge was still in service. After the completion of the project, a load test of the repaired bridge was conducted by running a 22.5 ton truck across the bridge. Midspan deflections were measured to be 0.2 inches and deemed adequate.

Rehabilitation: Coverplating [2.4.1]

Bakht, Baidar, and Paul F. Csagoly. "Strengthening and Widening of Steel Pony Truss Bridges." Canadian Journal of Civil Engineering 4, 1977, 214-225.

Lateral buckling behavior of pony trusses and repair options are discussed in this article. The authors give a short history of the analysis of pony trusses and then present a modified solution to the analysis. A computer program was developed to predict the capacity of pony trusses. Two full scale tests were run to check the accuracy of the program. The failure loads of the bridges were within 10% of the computer prediction.

To increase the portal rigidity of a pony truss, intuitively, either, or both, the floor beams or vertical members may be strengthened. It was shown that by only increasing the rigidity of the vertical members, the portal rigidity was actually decreased. The authors, therefore, recommend strengthening only the floor beams. Several rehabilitation techniques are discussed and their effects on portal stability are presented. The addition of longitudinal plate girders to widen the bridge had little effect on stability. If only the floor beams are lengthened, the article states that the variation in load carrying capacity is usually detrimental, and therefore should be avoided. A third option is adding a Bailey truss to the interior of the pony truss. This solution increased the load carrying capacity, but does decrease the width of the bridge. A final technique involves the addition of cover plates to the existing top chord. This method is presented as a good solution for bridges with stability plane concerns.

Rehabilitation: Coverplating [2.4.2]

Beauchamp, J. C., M. Y. T. Chan, and R. H. Pion. "Repair and Evaluation of a Damaged Truss Bridge-Lewes, Yukon River." Canadian Journal of Civil Engineering 11, no. 3, September 1984, 494-504.

This article describes a series of repairs, evaluations, and tests completed on a Warren truss that had been damaged due to an overheight collision. The bridge, built in 1955 using A7 steel, consists of two 250-foot spans and was designed for HS20-S16 loadings. A flat bed trailer carrying a backhoe struck the bridge in January of 1982. All bottom struts of the sway frames were severed which lead to progressive failures including the brittle fracture of the bottom chord. The bridge did not collapse completely, but was closed to traffic immediately. The article describes the temporary repair and subsequent permanent repair and evaluation. The repair of the bottom chord involved attaching four Dywidag bars and pre-stressing them to transfer the stresses back to the bottom chord. A splice was installed while the Dywidag bars held the bottom chord in place. The bridge was instrumented and computer models were used in verifying the success of the repair. After the bottom chord was repaired, the bridge was opened to one lane of traffic while the sway frames and other repairs were completed. Coverplates were added to strengthen the bottom chord and improve fatigue capacity.

Physical and chemical tests were also conducted on materials taken from the bridge. The tests revealed low Charpy values and good yield and elongation characteristics. A proof load test of the repaired bridge confirmed that the rehabilitation was a success. Conclusions from the study included the restatement of the importance of the integrity of the bottom chord in a truss bridge. Response of the bridge deck, in redistributing and relieving some of the stresses lost in the failure of the bottom chord, should also be noted. The authors state that older structures are just as susceptible to overheight damage as overweight damage due to the general trend of low toughness in older steels.

Rehabilitation: Coverplating [2.4.3]

Billar, Benjamin J. "Economical Flange Replacement for Built-up Steel Sections." In Structures Congress XIII: Proceedings of papers presented at the Structures Congress '94, Atlanta, Georgia, April 24-28, 1994, sponsored by the Structural Division of the American Society of Civil Engineers. Atlanta, Georgia, 1994, 811-814.

This article describes four rehabilitation alternatives investigated for the strengthening of deteriorated floor beam members of the Eads bridge in St. Louis. The 120-year-old structure was being upgraded to allow a light rail system to pass over the bridge. The floorbeams were determined to be a critical component in the rehabilitation. Four schemes were investigated including: (1) total floorbeam replacement (2) full bottom flange replacement (3) partial bottom flange coverplating (4) partial bottom flange coverplating and angle replacement. Cost estimates for each type of repair were determined. It was found that the full bottom flange replacement and partial coverplating were the most economical solutions depending on the level of deterioration in the beams. Over 200 floor beams were replaced using one of these two methods saving over \$200,000 compared to total floorbeam replacement.

Rehabilitation: Coverplating [2.4.4]

Schwendeman, Louis P., and Arthur W. Hedgren, Jr. "Bolted Repair of Fractured I-79 Girder." Journal of Structural Engineering 104, no. ST10, October 1978, 1657-1670.

A brittle fracture occurred at the midspan of a 350 ft. steel girder over the Ohio River in Pennsylvania. The girder was cracked through its entire 11 ft height causing a deflection of 5

inches in the roadway. The bridge was immediately closed to traffic. Several repair schemes were investigated but the final solution involved using a floating barge, jacking the girder back together, and splicing the fractured girder. Jacks were used to reintroduce the dead loading of the bridge into the girder. Detailed analysis of stresses on the splice and bolting ensured a conservative rehabilitation design. The girder was strain gauged to check the stresses in the girder throughout the operation. A detailed description of the entire process is included in the paper. No rehabilitation was needed for the deck or parapets of the bridge. At the conclusion of the repair, elevations were shot of the roadway and showed that the slab was only $\frac{1}{4}$ of an inch below the original alignment. The entire process from closure of the bridge, to being reopened for traffic, took slightly over two months.

Rehabilitation: Coverplating [2.4.5]

Martin, Robert A., and Jerome S. B. Iffland. "Marine Parkway Bridge Truss Member Replacement." Journal of Structural Engineering 109, no. 7, July 1983, 1603-1616.

This article gives a detailed description of repair efforts on a damaged tension chord, and the replacement of vertical hangers in a truss. The structure is a lift-span truss with a clear span of 500 ft. A barge travelling underneath the bridge struck the tension chord, badly damaging it. The engineers decided that flame straightening would not be a viable option due to the severe damage to the member. The solution consisted of affixing a temporary load carrying assembly, consisting of stressable bars, to the bottom chord. Once the bars were in place, the damaged section was removed and new material was spliced to the tension chord. A similar repair was used in replacing the corroded vertical hangers. Tension rods were attached to the truss to accept the load of the hangers. The vertical hangers were removed, and new hangers were installed.

Rehabilitation: Rivet Replacement [2.5.1]

"Riveting Experience." Modern Steel Construction, Sep 1993, 34-37.

Description of recreation of riveted truss bridge. Original structure was fabricated in the shop then shipped to the site and assembled. References are included to riveting processes and suppliers who completed work. The new bridge met HS20 loading requirements.

Rehabilitation: Rivet Replacement [2.5.2]

Reemsnyder, Harold S. "Fatigue Life Extension of Riveted Connections." Journal of Structural Engineering 101, no. ST12, December 1975, 2591-2608.

To extend the fatigue life of riveted connections, this article describes a study in which rivets were replaced with high strength bolts. Two connections were taken from an actual bridge, while 16 other specimens were modeled using comparable materials. A series of constant and variable amplitude fatigue tests were run on the specimens. When damage (cracking) was observed in the specimen, the rivet was removed using field techniques, and a high strength bolt was installed. The benefit of the bolt was the increased clamping force between the members. From the experiments, it was determined that this technique increased the fatigue life from two to six times the base life estimated from a non-rehabilitated specimen. Crack lengths of less than one inch, prior to rehabilitation, showed the best performance. This

confirms that early detection and rehabilitation are paramount to extending the service life of structures.

Rehabilitation: Rivet Replacement [2.5.3]

Baker, K. A., and G. L. Kulak. "Fatigue of Riveted Connections." Canadian Journal Civil Engineering 12, 1985, 184-191.

This article summarizes previous work on the fatigue of riveted connections, as well as, adds to the available data with additional tests. The authors have compiled the results from many researchers and provide a short commentary on each study. In their study, ten specimens were examined. Three of which were models of completely loose rivets. Another three were simulated using high strength bolts to replace rivets. The final four specimens were taken from a riveted bridge structure to examine the fatigue safety of the members. The loose rivet specimens were prepared by punching holes in a steel beam. This was to simulate a lower bound of riveted connections. The bolted specimens used $\frac{3}{4}$ inch A325 bolts. The actual bridge members were comprised of pairs of angles with lacing between them, forming an I shaped member.

The three "loose rivet" beams failed below AASHTO Category D loading. The bolted beams failed between Categories A and B. The in-service specimens were highly scattered but all tested higher than Category D. A duty spectrum was established, and it was determined that previous loadings on the bridge had minimal, if any, effect on the results. In conclusion, by comparing the new test results with previous studies, it was determined that AASHTO Category D rating is a conservative designation for riveted structures. The study confirmed that the replacement of rivets with high strength bolts greatly improves the fatigue rating.

Rehabilitation: Additional Members [2.6.1]

"Rehabbed Trusses Shed Load." Engineering News Record, November 6, 1995, 21.

Two 110 year old Pratt pony trusses were saved by supporting the bridge using hidden steel girders. The bridges consisted of a single 58' span and two 98' spans. The timber decks in both had deteriorated causing a significant loss in capacity. Approximately \$3 million was spent to install the new steel girders and decks for both bridges. The steel girders are shallow enough to be hidden by the bottom members of the truss. Vertical slip connections were added to allow the deflection of the new girders without transferring force to the trusses. The beams are braced against each other and by the truss superstructure.

Rehabilitation: Additional Members [2.6.2]

Dodds, N. M. S., L. A. Locke, and R. N. Welsford. "Recent Strengthening Work to Cleveland Bridge, Bath." The Structural Engineer 73, no. 5, 7 March 1995, 69-75.

This article describes a cast iron bridge built in 1827 and strengthened in the 1920's. The new work included the addition of pedestrian sidewalks. The span of the bridge is 30.4 m. The main structural system is comprised of seven arch ribs. In 1925 it was determined that the 5 ton load limit was unsafe and the bridge should be strengthened. Four reinforced concrete trusses were added between the arched ribs as well as a new reinforced concrete deck. An examination

of the structure before the present rehabilitation determined that while the load capacity of the bridge was still fairly good, the footpaths and parapets were unsafe and the concrete trusses impeded the view of the original structure. Beginning in 1988, an assessment and feasibility study for the repair of the bridge were undertaken. A load assessment yielded that the combined structure could handle a 40t C&U load (two loads of 205 and 236t). A detailed description of the analysis techniques can be found in the article. From the analysis, it was determined that the main portion of the bridge was adequate but the footways and parapets were in need of strengthening. A solution of “hidden portals” was adopted to strengthen the footways, but remain out of view. The added portals bear on original spreader plates and are tied into rock abutments. The footway slab is reinforced concrete able to resist all loading conditions including impact forces. In placing the new girders between the two pre-existing cast iron members, the cross bracing and attachment to the footpath was lost. Bracing was added between the old and new girders to provide adequate lateral support. The original parapets were left in place, but strengthened to provide safety.

Rehabilitation: Additional Members [2.6.3]

Bondi, Robert W. “Adding Redundancy to Fracture Critical 2-Eyebars Members in a Cantilever Truss Bridge.” In Proceedings 2nd Annual International Bridge Conference in Pittsburgh, Pennsylvania, June 17-19, 1985, by the Engineers’ Society of Western Pennsylvania, 1985, 47-53.

A truss bridge built in 1926-27 connecting Ambridge and Aliquippa, PA was investigated and repaired for fracture critical concerns. Following the Point Pleasant Bridge collapse, investigation of non-redundant tension eye-bar members for high strength steel truss bridges became a topic of much discussion. The high strength steels of the 1920s prove susceptible to stress corrosion and corrosion fatigue cracking. An in-depth analysis of this bridge revealed 72 locations at which the failure of one member would lead to a catastrophic collapse. It was also recommended, that the bridge should be strengthened from a 10 ton rating, to HS20. A detailed description of the failure investigation and solution are presented in this paper. A third member was added to strengthen the existing 2 eye-bar members. A new deck of pre-cast, post-tensioned concrete was implemented which reduced the dead loads on the bridge. A railing system was also added to meet specifications. The rehabilitation, costing \$2.3 million was completed in 1983.

Rehabilitation: Pin Replacement [2.7.1]

Taavoni, Shahin. “Upgrading and Recycling of Pin-Connected Truss Bridges by Pin Replacement.” Transportation Research Record 1465, 1994, 16-21.

This article describes the rehabilitation of a 113 year old wrought iron Pratt through truss in Maryland. All members, connections, and pins were evaluated to determine if the bridge could take additional loads. From site visits some tension members were found to be unsymmetrical due to dynamic effects, repairs, fatigue, or corrosion. It was determined that the pins were the critical members in the repair. Four rehab alternatives were examined: (1) superposition of an arch (2) supplementary girders under the truss (3) pre-stressing the bottom chord (4) replacing pins. The solution included dismantling each truss and moving it to a

working space, to replace pins. Material tests were done on removed pins $f_y \approx 26$ ksi. The total cost of the repairs was \$300,000. The rating of the bridge was increased to HS23.

Rehabilitation: Pin Replacement [2.7.2]

Bondi, Robert W. "Pin Replacement on a 100 Year Old Whipple Truss Bridge." In Proceedings 2nd Annual International Bridge Conference in Pittsburgh, Pennsylvania, June 17-19, 1985, by the Engineers' Society of Western Pennsylvania, 1985, 218-223.

The author describes the procedure used in the rehabilitation of a wrought iron truss bridge built in 1884. The bridge consists of two 200 ft spans with an 18.3 ft roadway and a six foot sidewalk on one side. An inspection of the bridge in 1977 identified corrosion in the diagonals, verticals, floor beam hangers, and stringers. As a result of the investigation, repairs including adding redundancy to eye-bar members, U-bolt hanger replacement, frame repairs, addition of a steel railing, and repairs to abutments. Five pins were replaced in the structure. The author outlines the repair procedure for the pins in nine steps. The new pins were longer to accommodate new vertical and horizontal strengthening members. The U-bolt hangers were replaced with a more reliable floor beam hanger system. The replaced pins showed minor wear and corrosion damage. It was speculated that the damage occurred due to the eccentricity induced by the single U-bolt hanger type connection. The pins on the sidewalk side were not replaced due to the fact a double U-bolt connection on that side of the structure did not have the eccentricity associated with the single U-bolt connection.

Rehabilitation: Deck Replacement [2.8.1]

"New Aluminum Decks Cut Loads, Add Life." Civil Engineering, August 1996, 12.

Two bridges, one in Pennsylvania, another in Virginia have used pre-cast aluminum panels to replace the decks. The dead load of the 55 ft. span in Virginia was reduced between 30-40%. A cost analysis between replacing the bridge with a new steel/concrete equivalent and adding a new aluminum deck were comparable. On the Pennsylvania project, a 320 ft suspension bridge built in the 1930's, new aluminum panels reduced the deck weight by 50%. An increase of load rating from 7 tons to 24 was realized. Construction time was reduced, as well as keeping traffic disruption to a minimum.

Rehabilitation: Deck Replacement [2.8.2]

Mangone, Ronald W. "Weldless Decking Expands Bridge Options." Modern Steel Construction 12, November 1997, 14-17.

This article provides a discussion of a deck product which has been utilized in West Virginia. A truss span which has been in service since the 1920's was rated poor to critical for the bridge deck. The new decking material, which was attached to the floorbeams and stringers using Nelson studs, achieved a lighter construction than conventional methods. The rehabilitated bridge rated at HS23 for the truss and the new deck system. The project also came in under cost.

Rehabilitation: Deck Replacement [2.8.3]

Holm. "Structural Lightweight Concrete for Bridge Redecking." Concrete Construction 30, August 1985, 668-672.

This article provides introductory information on lightweight concrete. The author uses a case study bridge in New York which was widened from two lanes to three. Only 20% of the steel framing was strengthened due to the lowered dead weight of the lightweight concrete. The author further praises lightweight concrete for its durability and ease of placement. The durability of lightweight concrete is attributed to the similar moduli of the cement grout and the lightweight aggregate. Less cracking is observed since freeze/thaw cycles induce smaller stresses between grout and aggregate as compared to normal aggregate. Some simple guidelines are presented which discuss topics such as changes in water content due to a change in entrained air or fine aggregate. The author discusses placing techniques and states the lightweight concrete is, in fact, easier to place than normal weight concrete. An example mix design is also included.

Rehabilitation: Deck Replacement [2.8.4]

Exodermic Bridge Deck Institute, Inc. "Exodermic Bridge Deck Handbook." Scarsdale, New York: Exodermic Bridge Deck Institute, Inc., 1996.

This handbook provides an overview of exodermic bridge decking provided by the Exodermic Bridge Deck Institute. A discussion of the theory, design, and construction of this system is included. A presentation of related laboratory tests completed on exodermic systems is offered. A listing of design criteria and assumptions is included to provide guidance for design engineers. Design examples using both pre-cast and cast-in-place concrete are provided. A tabular listing of calculations provide quick reference for the capacities of various configurations of bridge decks. Finally, structural details are presented to aid in the completion of design drawings.

Rehabilitation: Deck Replacement [2.8.5]

Buffalo Specialty Products, Inc. "Bridge Flooring." Allentown, Pennsylvania: Buffalo Specialty Products, Inc.

This document is a product guide for a galvanized decking system. The deck is comprised of a saw-tooth floor of either 10 or 12 gauge material with a bituminous wearing surface on top. The weight of the floor system varies from 35-52 psf dependent upon the depth of wearing surface applied. Typical attachment details to existing bridge members are included to aid the engineer in design drawings.

Rehabilitation: Deck Replacement [2.8.6]

Reynolds Metals Company. "Reynolds Aluminum Bridge Decks." Richmond, Virginia: Reynolds Metals Company, 1996.

This decking system consists of hollow aluminum sections welded together to form panels. The panels are topped with a thin epoxy wearing surface to provide the necessary skid

resistance. The system weighs approximately 22 psf, significantly reducing the dead loads on the bridge. A discussion of design for the decking is included, as well as, material properties. The document also discusses corrosion, thermal expansion, and costs relative to other decking systems.

Rehabilitation: Deck Replacement [2.8.7]

Laminated Concepts Inc. “Longitudinal Truss Concept.” Elmira, New York: Laminated Concepts Inc.

This document is introduction to a glue-laminated deck system. The paper presents various deck thickness for different spans which will meet current AASHTO standards. Sample drawings are included to aid in the visualization of the system. The paper also includes a timber railing detail, which may be fixed to the new deck. No indication is given whether the railing system was crash tested.

Rehabilitation: Deck Replacement [2.8.8]

Bridge Grid Flooring Manufacturers Association (BGFMA). Collected Articles, Mount Pleasant, Pennsylvania.

“Design of Grid Reinforcement Concrete Bridge Decks.” March 1997.

This technical brief outlines the analysis of concrete grid decks as orthotropic systems, instead of the conventional beam strip method of analysis. The result is better live load distribution and thereby lower stresses.

“Enhancing Grid Performance & Service Life through Integral Overfills.” Issue 13, Spring 1997.

This paper discusses the use of concrete overfills as opposed to flush filled grids in bridge deck rehabilitation. Primary advantages include better riding surfaces and protection for the gridding. A list of ten bridge rehabilitation projects which used metal grid, with concrete overfills, is included along with comments on each repair.

“Design of Grid Reinforcement Concrete Bridge Decks Using AASHTO’s 16th Edition.” Spring 1997.

This short brief contains a typical design example for grid decking. The example is of a 4.25 inch metal grid with stringer spacing of twelve feet.

“Corrosion Protection for Grid Reinforced Concrete Bridge Decks” Issue 14, Summer 1997.

This article discusses the current research on corrosion resistance for steel gridding. The paper presents the findings of Donald Timmer, an engineer in Ohio, who has specified different types of corrosion protection for metal grids. Four case studies are presented in which either galvanized, or epoxy coated gridding was used in the replacement deck. The article concludes

that galvanizing, while slightly more expensive than painting, provides very good resistance to corrosion of steel gridding.

Rehabilitation: Deck Replacement [2.8.9]

Crews, Keith; Taylor, Raymond; Carter, Donald “History, Development and Application of Stressed Laminated Timber Systems for the Repair and Rehabilitation of Bridge Decks” Proceedings Of Fifth International Conference On Structural Repair, June 1993, 205-216.

The concept of prestressed laminated timber bridge decks was conceived in Ontario in 1976 as a method of upgrading existing deteriorated nailed timber decks. This paper presents an overview of stress laminated technology with an emphasis on its applicability for both rehabilitation and new construction, and its particular relevance to bridge “Asset Management” in Australia. An overview of structural concepts is presented, along with a number of specific field applications covering both rehabilitation and new construction.

Rehabilitation: Flame Straightening [2.9.1]

Holt, Richard E. “Primary Concepts for Flame Bending.” Welding Journal 50, June 1971, 416-24.

This article provides a very thorough introduction to flame bending. A discussion of the theory of flame bending is included. The author discusses spot, vee, line, and strip heats. Each of these discussions included theory and applications of the various heats to bridge members. The author also describes heating processes for plate girders and other larger built up members. The author provides examples of repairs using heat straightening for tension and compression members. The author also discusses damage to joints and the interaction of truss members with one another. I would strongly recommend reading this article prior to utilizing flame bending in practice.

Rehabilitation: Flame Straightening [2.9.2]

Post, Jeffrey, W. “Flame Straightening Repairs Bridge.” Welding Innovation Quarterly Vol. 13, No. 3, 1996, 4-6.

This article gives a brief overview of flame straightening, as well as, an example of its use in the field. A compression member on the I-93 Bridge crossing the Charles River was damaged due to an impact of wood which fell off of a truck. The column had an 18-inch bend in a 25-foot length. The options in repairing the bridge consisted of flame straightening or replacement. A replacement member would take three weeks to fabricate; therefore the decision was made to use flame straightening. For 40 hours, people were working on the member with torches to remove the damage. At the end of the repair, over one-million dollars was saved in material costs and the bridge was back in service in less than a week after the collision.

Rehabilitation: Flame Straightening [2.9.3]

Avent, R. Richard. "Heat-Straightening of Steel: Fact and Fable" Journal of Structural Engineering, Vol. 115 no. 11, November 1989, 2773-2793.

Although heat-straightening technique has been used for the repair of damaged steel structure for over 40 yr., the process is more of an art than a science. Since the effects of heat-straightening steel structures are not well understood by the engineering community, a number of undocumented opinions have appeared in the literature that is false or misleading. Presented here is a state of art review in the form of popular fables followed by the documented facts related to each one. Sixteen fables are listed in two categories: basic material properties and behaviour of structural systems. The reference cited form a comprehensive bibliography on the subject of heat straightening effects on steel. The selection of facts and fables has been chosen to provide the reader with an understanding of the process. Important factors influencing heat-straightening of rolled and built up members, a working knowledge of its effect on the material properties of steel and the sources for additional information. Heat straightening has produced good track record that illustrates the potential of the method for providing safe and economical repairs.

Rehabilitation: Flame Straightening [2.9.4]

Zobel, Henryk "Determination of Heat-Straightening Parameters for Repair of Steel Pedestrian Bridge" Proceedings of the Fourth International Bridge Engineering Conference, TRB-Washington DC, 1995, 26-32.

The application of the thermomechanical method of repair to a pedestrian bridge in Warsaw, Poland, over one of the busiest expressways is presented. The comparison of three kinds of straightening is shown. The thermomechanical procedure was the most effective, but needs still more theoretical and experimental research.

3. EVALUATION

Evaluation: General [3.1.1]

White, Kenneth R. Bridge Maintenance Inspection and Evaluation. New York: M. Dekker, 1981.

This book provides an introductory description of bridge inspection and evaluation. It covers timber, steel, and concrete bridges as well as documentation techniques and equipment needed. A very short discussion is included concerning bridge maintenance and rehabilitation. Calculation examples are included for stress determination in various bridge types. However, this reference is very general and only provides the introductory information in the repair and rehabilitation arena.

Evaluation: General [3.1.2]

Skallerud, B.; Eberg, E.M.; Fergestad, D. “Ultimate Capacity Analysis of Bridges With Deteriorated Connections or Members” in: Bridge Management 3 : Inspection, Maintenance, Assessment, and Repair, edited by J.E. Harding, G.E.R. Parke and M.J. Ryall, Published by E & FN Spon, 1996.

In this study, two bridges are analysed by means of a non-linear 3D-frame finite element program. One bridge represents a railway bridge of trusswork type. The other is a cable-stayed suspension bridge with a trusswork bridge deck.

Evaluation: General [3.1.3]

Rabbat, B. G.; Russell, H. G. “Friction Coefficient of Steel on Concrete or Grout” *Journal of Structural Engineering* v. 111, no.3, March 1985, 505-515.

An experimental investigation was conducted to determine the coefficient of static friction between rolled steel plate and cast-in-place concrete or grout. Fifteen tests were performed under conditions that represented the interior and exterior bearing surfaces of a containment vessel. Test parameters included concrete blocks or grout blocks, wet or dry interface, and level of normal compressive stress. For conditions tested, the average effective coefficient of static friction varied between 0.57 and 0.70 it is recommended that the coefficient of static friction for concrete cast on steel plate and grout cast below steel plate should be taken as 0.65 for a wet interface with normal compressive stress levels between 20 and 100 psi (0.14 and 0.69 MPa). For dry interface, the coefficient of static friction should be taken as 0.57.

Evaluation: Corrosion [3.2.1]

Fisher, John W., Ben T. Yen, and Dayi Wang. “Corrosion and Its Influence on Strength of Steel Bridge Members.” Transportation Research Record 1290, 1991, 21-219.

This paper reports on a study which tested the strength and performance of corroded, riveted bridge members. A discussion of the causes of galvanic and pitting corrosion is included. The first part of the study involved the fatigue strength of corroded notched members. Eight girders with area losses ranging from 5-40% failed at equivalent AASHTO fatigue categories C, D, and E. The second set of tests concerned the strength of corroded hangers. Two specimens (losses of 39-41% area) were loaded to a failure load which was about 5% less than that calculated using net areas and ultimate strength. This small difference was attributed to shear lag. The hangers did not fail by sudden fracture, rather by ductile elongation. Conclusions arrived at by this paper include: (1) corrosion notch effects relate to AASHTO fatigue categories ranging from C to E depending on the severity of the corrosion (2) the ultimate strength of corroded tension members is about equal to the tensile strength multiplied by the remaining net area (corrosion did not affect ductility).

Evaluation: Corrosion [3.2.2]

Cywinski, Z. “Simplified evaluation of wrought iron bridges” Stahlbau 4, 1985, 103-106.

Wrought iron bridges are in service more than a century and there is an urgent request for updating their load capacities. Within an approximate approach the decrease in strength of questioned bridges is assumed to be caused only by corrosion and fatigue, whereby their interdependence enables to consider corrosion effect alone. For needs of this approach standards of corrosion rate have been suggested, taking into account the bridge history and differentiating between favourable and bad environmental condition, periods of good and insufficient bridge maintenance, low and high daily moving load frequencies. Proper analysis shows that in given circumstances the present allowable stresses should not exceed the values of 85 N/mm² for short and 100 N/mm² for long span constructions.

Evaluation: Corrosion [3.2.3]

Tam, C. K., Stiemer, S. F “Development of Bridge Corrosion Cost Model for Coating Maintenance” Journal of Performance of Constructed Facilities, May 1996.

This paper deals with the minimising the cost of protective coating maintenance for steel bridges. Coating maintenance for steel bridges is a major significance and is revived here in some detail with respect to problems, causes, remedial techniques and associated costs. One of two analysis methods under investigation to minimise the cost of coating maintenance for steel bridges is presented in this paper. This first method featured in the bridge corrosion cost model, performs a life cycle cost analysis using equivalent annual costs to compare the three maintenance strategies: spot repair, overcoat and recoat. The strategy that provides the minimum equivalent annual cost is considered to be optimal. A computer application, which will be presented in a larger paper, has been developed to reduce the manual work required to implement the procedures described in bridge corrosion cost model.

Evaluation: Fatigue & Fracture [3.3.1]

Keller, Andreas, Eugen Brühwiler, and Manfred A. Hirt. “Assessment of a 135 Year Old Riveted Railway Bridge.” International Association of Bridge and Structural Engineers 1029-1034.

A wrought iron railroad bridge connecting Switzerland and Germany was assessed for continuing service of passenger trains. The structure, built in 1859, was designed to carry two trains, but has only carried one line throughout its life. Three stages of analysis were completed. The first involved checking the structural safety, fatigue safety, and serviceability of the structure. All the members were determined to be adequate. Secondly, a fatigue analysis using the approximated loadings was completed. Stress ranges for the freight and passenger trains were calculated and compared to the damage limit stress range. It was found that some damage has occurred due to freight trains, but the effects were small. By comparing the stress range for the proposed new passenger trains, to the damage limit stress range, it was determined that the bridge would not incur more damage in the future. As a final step, the critical crack size prior to failure was determined using fracture mechanics methods. This value provides guidance for inspection procedures in the future. Fatigue crack propagation was investigated and also found not to be critical. In conclusion, the author states that the bridge can continue in service, with inspection and normal maintenance, for many years to come.

Evaluation: Fatigue & Fracture [3.3.2]

Szeliski, Z. L., I. A. Elkholy. "Fatigue Investigation of a Railway Truss Bridge." Canadian Journal of Civil Engineering 11, 1984, 625-631.

A thorough investigation of a railway truss bridge was conducted to determine the fatigue life, as well as, if any rehabilitation effort were needed. The bridge, built in 1906, was designed for an E-48 Cooper train. Increased traffic raised concerns of the bridge's safety. Strain gauges were placed on the bridge members and data was taken for several days of normal operation. Computer analysis was also run to determine the best modeling technique for the structure. Conclusions included: the stringers and floor beams behaved like simple beams, hangers behaved somewhat like a plane truss and somewhat like a space frame.

An estimate of the past, present, and future traffic was compiled and a fatigue damage model was assembled. A root-mean-square model was used to estimate the damage the bridge had sustained. Field investigation revealed small cracks at some rivet holes, however, these cracks were not considered critical. The investigators concluded that if critical rivet areas were replaced with high strength bolts, the fatigue life of the structure could be extended past 1998. A similar conclusion, that of replacing the rivets, was drawn for floor beams and stringers. As a result of the investigation, all critical rivets were replaced with high strength bolts.

Evaluation: Fatigue & Fracture [3.3.3]

Idriss, R. L., K. R. White, C. B. Woodward, and D. V. Jauregui. "Evaluation and Testing of a Fracture Critical Bridge." Non-Destructive Testing & Evaluation International 28, no. 6, 1995, 339-347.

This article reports on a series of tests of a two girder, steel bridge. This fracture critical structure, as defined by AASHTO, was tested with a man-made flaw, to examine the post-fracture response. The bridge showed very good reserve strength with moment redistribution occurring in the damaged span. The rigid concrete floor system, floor beams, and stringers contributed in a secondary manner. The important point of hidden redundancy in "fracture critical" structures (e.g. truss bridges) was reinforced in the article.

Evaluation: Fatigue & Fracture [3.3.4]

Fisher, John W., and J. Hartley Daniels. "An Investigation of the Estimated Fatigue Damage in Members of the 380-ft Main Span, Fraser River Bridge." American Railway Engineering Association Bulletin 658 577-597.

This article investigates the remaining fatigue life of a railroad truss bridge. The bridge, constructed in 1904, is along an important route carrying traffic to and from Vancouver. The critical member investigated was a vertical hanger comprised of four angles with a filler plate forming an I-section. The web plate was replaced with laticing in 1923. To analyse the structure, a simple truss model and 3-D model were both used. The pin-plates, which connected the hanger to the floor beam were also investigated using simple analysis and a more complicated 3-D model. The 3-D model predicted that the stresses in the plates were unequal, which agreed with field tests on the pin-plates.

For fatigue calculations, the capacity of the riveted members was taken to be AASHTO Category D. This conclusion stemmed from previous work done by others. The fatigue strength of the pin-plates were determined using fracture mechanics and a finite element analysis. It was found that AASHTO Category E would predict the fatigue capacity of the pin-plates with a reasonable initial flaw size.

The damage assessment was determined using both Miner's solution and RMS and the loading spectrum determined by Sweeny in previous work. The hangers were found to be adequate until the year 2000. The pin-plates, due to their unequal sharing of the load, were determined to be critical by 1985. The stringers were found to be critical by 1979. In conclusion, the authors point out that the field investigation lead to a stress range reduction of between 15-20%. This translated into over a 33% change in the allowable stress for future calculations.

Evaluation: Fatigue & Fracture [3.3.5]

Kunz, Peter M., Eugene Brühwiler, and Manfred A. Hirt. "Evaluation of the Remaining Fatigue Life of Steel Bridges." In Developments in Short and Medium Span Bridges Engineering 1994 edited by Aftab A. Mufti, Baidar Bakht, and Leslie Jaeger, August 1994, 1219-1230.

In this paper, the authors discuss the calculation of the fatigue life of a bridge. Critical details must first be assessed to determine the members which are subjected to a maximum stress. A fatigue calculation can then be performed using past, present, and anticipated loadings. Finally, the bridge should be monitored to verify calculated results. The authors concern themselves with the second step in the process, the evaluation of fatigue life, and developed a computer program to aid in damage accumulation calculations. Damage accumulation using fatigue strength curves is a widely accepted method employed by the European and US codes. Damage accumulation utilizing crack propagation calculations based on fracture mechanics, is a less used procedure, but yields longer remaining life. Using the American, European, and fracture mechanics methods, the authors compare the predicted remaining fatigue life of a railway girder bridge built in 1900. As predicted, the fracture mechanics method yielded the longest remaining fatigue life. A sensitivity study was performed to determine the parameters which effect the various methods. It was found that the fracture mechanics method displayed vastly different results from the fatigue strength curves in low fatigue loading, but similar results in high fatigue loading.

Evaluation: Fatigue & Fracture [3.3.6]

Schindler, Hans-Jakob and Ulrich Morf. "Toughness and Fracture Behavior of Obsolete Wrought Bridge Steel." International Association of Bridge and Structural Engineers Workshop, Lusanne, Switzerland, 1990, 85-93.

In this article, the authors discuss the performance of wrought iron in fracture situations. The fibrous nature of wrought iron contains many impurities and any type of welding repair could change the composition of the material possibly leading to a more brittle structure. To determine the fracture mechanics properties, a series of compact tension tests were performed and the results analyzed. Dynamic fracture toughness was also measured using pre-cracked

Charpy specimens. It was concluded that the fracture toughness of the material was approximately $5000 \text{ N/mm}^{3/2}$ for in-service cases. A series of experiments were performed to verify these conclusions. A failure assessment diagram was constructed using the R6 procedure and an assumed toughness of $4000 \text{ N/mm}^{3/2}$. The failures of the specimens corresponded to points outside, near the R6 curve which verifies the predictions of the tests. In conclusion, the authors state that wrought iron is not as crack-sensitive as previously thought, and members with small cracks (e.g. hidden by rivet heads) should not be considered critical.

Evaluation: Fatigue & Fracture [3.3.7]

Fisher, John W., Menzemer, Craig , in: Bridge Evaluation, Repair and Rehabilitation. Dordrecht, Boston: Kluwer Academic, 1990, p.495-512 [Ref. 1.11]

Nearly half of the highway bridges in the United States have been classified as structurally deficient or functionally obsolete. The question of functional obsolescence will not be dealt with. Reasons for the structural deficiencies are numerous, as a number of the bridges are over fifty years old and have not been properly maintained. Over a period time, environmental corrosion has accumulated. In addition, increase in the numbers of trucks using the highways as well as the rise in allowable truck weights has led to increasing amounts of fatigue cracking. Many structures have welded details that have been identified as susceptible to fatigue cracking only after they were built. Rules used to design some structures oversimplify member interaction and connection behaviour, resulting in distortion-induced fatigue cracking in a large number of different types of bridge structures.

Due to magnitude and breath of the bridge deficiency problem, and the prohibitive costs of new construction, a large effort in the repair and rehabilitation of existing bridges is required. Careful examination of the causes of fatigue cracking and corrosion accumulation, coupled with effective technology transfer, will help to prevent the development of similar situation in future construction. This paper briefly reviews the causes of some of the fatigue cracking being experienced in bridge structures. Various repair and retrofit procedures are reviewed, and examples of typical application of these procedures are examined. It also examines examples of severe corrosion and the inability to detect active corrosion cell activity.

Evaluation: Fatigue & Fracture [3.3.8]

Sedlacek, G.; Hensen, W. “New Design Methods for the Rehabilitation of Old Steel Bridges” 3rd International Workshop on Bridge Rehabilitation, June 1992.

This paper presents new methods for the identification and toughness related material checks of old steel bridges that allows to get a complete view on the residual safety and service life of such bridges and also permit to determine measures for strength and toughness related strengthening. The fracture mechanic based verification procedure have been simplified and presented in such a form that the assessment can be carried out as easily as the conventional strength verification. The application of the simplified procedure has been demonstrated by an example. Guidelines for the design of strengthening are given.

Evaluation: Fatigue & Fracture [3.3.9]

Byers, William G.; Marley, Mark J.; Mohammadi, Jamshed “Fatigue reliability reassessment applications: State of the art paper” Journal of Structural Engineering v. 123, March 1997, 277-85.

The authors investigate the currently accepted procedures for applying fatigue reliability analysis to railroad bridges, Highway bridges, and offshore structures. Fatigue reliability estimate for both railroad and highway structures are typically calculated on the basis of fatigue life predictions. For offshore structure, it is feasible to develop more complicated fracture mechanics model that explicitly allow updated estimates of fatigue reliability based on inspection results.

Evaluation: Fatigue & Fracture [3.3.10]

Oka, V. G.; Hopwood, T., II; Harik, I. E “Fatigue analysis of steel bridges using a portable microcomputer based strain gage system” Computers and Structures, vol. 31, no. 2, p. 151-86.

Steel bridges show increasing susceptibility to fatigue cracking with increasing age due to accumulative damage caused by cyclic live loading. Fatigue cracking of the non-redundant structural members can cause the failure of a bridge. A portable microcomputer based strain gage system, capable of on-line data acquisition and subsequent data analysis, has been devised in this study. The system can be used to determine if the live load stress cycles experienced by a particular bridge member are sufficient to cause fatigue crack initiation or the fatigue growth of the potential pre existing cracks. The relatively low cost strain gage system devised in this study can thus be used as a precursor to the expensive non destructive testing by prioritising inspection among a large family of bridges.

Evaluation: Fatigue & Fracture [3.3.11]

Fisher, John W.; Yen, Ben T.; Wang, Dayi. “Fatigue Strength of Riveted Bridge Members” Journal of Structural Engineering, Vol. 116 no. 11, November 1990, 2968-2981.

Available experimental data from more than 1,200 previous fatigue tests and some fatigue and fracture tests on full scale members are compiled and examined in conjunction with the details and measured live load stresses in riveted bridges. It is found that the type of riveted detail does not significantly affect fatigue resistance, that AASHTO fatigue strength category D is a reasonable lower bound for the initial fatigue crack development, and that the fatigue strength of a riveted built up members effectively exceeds the category C resistance curve. It is also found that the primary members in riveted highway bridges seldom are subjected to stress ranges that exceeds the category D fatigue limit, and thus are not likely to develop cracks. The riveted details most likely to develop cracks are those subjected to distortion or displacement-induced stresses at connections. Corrosion can reduce the fatigue resistance, but the effect is not critical if the minimum remaining thickness is not less than one-half of the original thickness. Recommendations are made for rating riveted highway bridges for fatigue damage.

Evaluation: Fatigue & Fracture [3.3.12]

Nyman, Williams E.; Moses, Fred. "Calibration of Bridge Fatigue Design Model" Journal of Structural Engineering, Vol. 111 no. 6, June 1985, 1251-1266.

A structural reliability evaluation is performed of the current AASHTO fatigue specification for steel bridges. The reliability model incorporate uncertainties in vehicular loading, analysis and fatigue life. Field data is obtained from a weigh-in-motion system, which utilises existing bridges as equivalent static scales. The load data include truck axle and gross weights, headway's, impact, and stress range, girder distribution, and volume. A fatigue life model is formulated in terms of a fatigue function. Using the loading and fatigue life data a safety index is calculated using a level II reliability program. The study reviews the current specifications in order to drive uniform reliability levels over a range of typical designs. The proposed revisions includes 1) A design vehicle model more representative of the current US truck population. 2) Changes in allowable stress ranges to eliminate variations in safety index and 3) a range of load factors to represent site specific truck volume and loadmeter values. These recommended changes lead to more uniform safety levels and fatigue lives for steel bridges.

Evaluation: Fatigue & Fracture [3.3.13]

Yazdani, Nur; Albrecht, Pedro. "Risk Analysis of fatigue failure of highway steel bridges" Journal of Structural Engineering, Vol. 113 no. 3, March 1987, 483-501.

A probabilistic fracture mechanics model was developed for determining the risk of fatigue failure of steel highway bridges. The model consists of 1) Stochastic inputs of crack growth rate, fracture toughness, initial crack size, and load history; 2) a deterministic fracture mechanics calculation of crack growth; and 3) a Monte Carlo simulation to obtain the out put variable, i.e., the fatigue life. The following effects on the risk of failure were examined: inspection interval, truck weight, truck traffic, system versus detail reliability, and length of service life extension. The model was applied to three bridges with cover plated girders. It was found to predict well the short service life of the Yellow Mill Pond Bridge, Connecticut, in which the cover plate end details began to fail 12 yr. after bridge opening. The model is particularly useful in determining the risk of extending the service life of a bridge beyond its intended design life.

Evaluation: Fatigue & Fracture [3.3.14]

Wilson, Wilbur M. "Flexural Fatigue Strength of Steel Beams" University of Illinois Bulletin-377, Vol.45, no. 33, January 22, 1948.

The principle purpose of the investigation described in this bulletin was to determine the relative fatigue strengths of various kinds of flexural members. The fatigue tests were supplemented by static tests of similar specimens. The specimens tested include: (1) Rolled beams without reinforcement and without lateral plates; (2) Rolled beams without reinforcement and with lateral plates, some attached with rivets and others attached with welds; (3) Rolled beams reinforced with full length cover plates, some attached with rivets and others attached with welds; (4) Rolled beams reinforced with partial length cover plates attached with welds; (5) Fabricated beams made of web and flange plates attached with welds; (6) Beams with

intermediate stiffeners, some welded to the web and both flanges, others welded to the compression flange and to the compression portion of the web, but not welded to the tension flange. Some reinforcing cover plates were attached with continuous fillet welds; others were attached with intermittent fillet welds.

Evaluation: Fatigue & Fracture [3.3.15]

Wilson, Wilbur M., Munse, William H. “Fatigue Strength of Various Details Used for the Repair of Bridge Members” University of Illinois Bulletin-382, Vol.47, no. 30, December, 1949.

Various means has been used to strengthen or repair members of bridges in service. Some of these methods have been in use for many years, yet very little information is available as to either the static or the fatigue strength of the modified members. The most important factors for determination of fatigue strength of the members are: (1) The magnitude of the strain raising factor; (2) the ratio of the minimum to the maximum stress in the cycle; and (3) the number of cycles of near-maximum stress to which the member is subjected during its life. The tests reported herein are planned to determine the fatigue strength of the specimens that incorporated expedients which have been used to strengthen or repair the members of old bridges. The specimens for some tests are the product of methods that have been used to shorten eyebars, which, due to wear on the pins and pinholes, have become so loose that it is necessary that they be tightened by shortening. The specimen for other tests represent reinforced bridge members for which the increased area has become necessary because of an increase in the loads to which the structure is subjected. Other specimens represent expedients that have been used to splice members in service. The object of the tests are twofold: (1) to determine the relative fatigue strength of various devices that have been used or proposed to strengthen or repair old bridge members in order to eliminate those methods that involve details with a low fatigue strength; and (2) to determine quantitatively the fatigue strength of the members that have been strengthened or repaired by various methods.

Evaluation: Fatigue & Fracture [3.3.16]

Fisher, J.W., Yen, B. T., Wang, D., Mann, J.E. “Fatigue and Fracture Evaluation for Rating Riveted Bridges” TRB Record 302, Transportation Research Board, December 1987.

The criteria often adopted for control of fatigue and fractures in new bridge structures are based on studies of modern welded construction and ongoing laboratory research on welded members. Most old bridges are constructed of riveted built up members. The research was conducted to review existing data and results of analysis, review current practice of rating and retrofitting riveted bridge members components; conduct full –scale bridge members tests to observe the behaviour of riveted bridge component in fatigue and fracture; and to correlate all information to develop the rating procedure.

Evaluation: Fatigue & Fracture [3.3.17]

Morre, Herbert F., Lyon, Stuart W., Inglis, Norman P. “Tests of the Fatigue Strength of Cast Iron” University of Illinois Bulletin-164, June 1927.

The fatigue strength of gray cast iron is investigated at the University of Illinois by performing a series of test with corporation from Allis-Chalmers Manufacturing Company, Wisconsin.

Evaluation: Fatigue & Fracture [3.3.18]

Badoux, Marc., Sparks, Patrick “Fracture Critical Study of a Historic Wrought Iron Bridge” Structural Engineering International, February 1998, 136-139.

This project is an example of a historic restoration project governed primarily by structural engineering consideration. The fact that no expensive pin or bottom chord eyebar replacement was required was very significant for the financial feasibility of the rehabilitation project currently underway. The modern analysis of the Faust Street Bridge shows that the bridge engineers developed highly efficient wrought iron bridges in the 19th century and their structural design was optimised in respect of material.

Evaluation: Fatigue & Fracture [3.3.19]

Seong, Chun Kyung “Fatigue Resistance of Riveted Steel Truss Bridge Members and Joints” Lehigh University, Ph.D. Dissertation, 1984.

The fatigue resistance of old riveted truss bridge members and joints were investigated in this dissertation. Emphasis was placed on developing the analytical model for estimating the fatigue crack. The analysis of the member was conducted by FEM of a bridge span as three-dimensional space frame. A more appropriate estimate of fatigue resistance was established through consideration of crack initiation and propagation. The estimate fatigue life of riveted details compared well with test results of previous studies.

Evaluation: Fatigue & Fracture [3.3.20]

Fisher, John, W. “Fatigue and Fracture in Steel Bridges” John-Wiley & Sons, 1984.

A number of highway bridges structures in the United States and Canada have experienced fatigue cracking which sometimes resulted in brittle fracture as a result of service loading. This book provides a detailed review of 22 case studies of bridges that have experienced crack growth. The first part deals with the cracks that formed as a result of low fatigue resistant details or large initial discontinuities. The second part deals with fatigue cracks that form as a result of unanticipated secondary or displacement induced stresses.

Evaluation: Fatigue & Fracture [3.3.21]

Moses, F.; Schilling, C. G., Raju, K.S. “Fatigue Evaluation Procedures for Steel Bridges” TRB Record 299, Transportation Research Board, November 1987.

This report contains recommended revision to the fatigue evaluation requirements in the AASHTO Manual for Maintenance Inspection of Bridges, along with a companion commentary. Probabilistic limit state concepts are used in the development of the recommended procedures. In

addition, fatigue evaluation procedures are also presented in the report. The content of this report will be of immediate interest and are useful to bridge engineers and researchers.

Evaluation: Fatigue & Fracture [3.3.22]

Zuraski, Patrick D.; Johnson, John F, Johnson “ Fatigue Strength of Deteriorated Steel Highway Bridges” Journal of Structural Engineering, v. 116, no.10, October 1990, 2671-2690.

The effect of long-term environmental exposure and an extended history of highway loading were quantitatively determined for steel bridge beams that had been in service more than 50 years. Fatigue tests were conducted on specimens removed from the tension flange of truss stringers salvaged from four bridge re- placement projects. Based on the fatigue strength reduction factor K, determined on a basis of mean fatigue life for all specimens in one bridge, the fatigue sensitivities encountered were less than that associated with an AASHTO Category C welded connection detail. For individual specimens, the maximum K, value determined was 3.0. At 36-ksi (248-Mpa) stress range, average specimen lives exceeded 300.000 cycles for all bridges, for both centre-span (maximum prior stress history) and near-support (insignificant prior stress history) specimens. The corrosion deterioration experienced by the plain (non-welded) simple-span steel bridge beams in this study caused the average fatigue strength to be reduced by approximately one AASHTO fatigue category.

Evaluation: Fatigue & Fracture [3.3.23]

Brandes, K. “Experimentelle Absicherung von Untersuchungen zur Restnutzungsdauer alterer Stahlbrücken (Experimental corroboration of remaining fatigue life investigations of old steel bridges)” p. 273-278, Ernst & Sons. Stahlbau 65, 1996.

The rating of old steel bridges for their future use are a task, which far exceeds the straightforward procedure of designing a new one. Material, joining technology and respectively designed constructional details demands an intensive pursuit with the arts of engineering at that time. Experimental investigations are sometimes helpful, possibly necessary which concern the material as well as the real distribution of forces in the construction. Some investigations of this type are subject of the report as well as fatigue tests on complete decommissioned bridges or large parts of them in order to confirm statements regarding the remaining fatigue life of these bridges.

Evaluation: Fatigue & Fracture [3.3.24]

Akesson, Bjorn; Edlund, Bo “Remaining Fatigue Life of Riveted Railway Bridges” p.429-436, Ernst & Sons. Stahlbau 65, 1996.

This article presents the results of field investigations of riveted steel railway bridges and a full-scale fatigue test series of riveted members from an old bridge. The main finding is that there might be a substantial remaining fatigue life of riveted railway bridges still in use today.

Evaluation: Fatigue & Fracture [3.3.25]

Kulak, Geoffrey L. "Fatigue Strength of Riveted Connections" p.445-451, Ernst & Sons. Stahlbau 65, 1996.

New riveted structures generally have not been built in the past several decades. As a consequence, less attention has been paid to their fatigue life behavior than to structures containing contemporary fastening elements such as bolts or welds. The behaviour of riveted members is a matter of considerable economic importance to owners and regulatory authorities, however. For example, it is obvious that the life of the large stock of riveted bridges that still exist must be extended, at the same time maintaining a satisfactory level of safety. Evaluation of the remaining fatigue life of a riveted structure has been impeded by the lack of a reasonable database of test results of full-size riveted members. Recent work in both North America and Europe has addressed the need for better fatigue life data for riveted members. The paper reports on the author's research and that of others into the fatigue behavior of riveted members. The results of tests on full-size flexural members and axially loaded members are reported, and these test results are compared with the design recommendations of several European and North American standards. Reassessment of riveted shear splices.

Evaluation: Fatigue & Fracture [3.3.26]

Hirt, Manfred A.; Kunz, Peter M "Ermudungssicherheit bestehender Brucken aus Stahl (Fatigue safety of existing steel bridges)" p.399-406, Ernst & Sons. Stahlbau 65, 1996.

All over the world many railway bridges were built around the turn of the century. These bridges are riveted and are now 100 years old or more. Owners and regulatory authorities must decide whether and under what conditions these bridges can still be used in the future. A similar problem exists for highway bridges constructed in the 1950's and 1960's. These bridges are often welded, but detailing suitable for fatigue was relatively unknown at the time. Furthermore, traffic on highway and railway bridges has become heavier and traffic volumes have increased significantly. Therefore, in both cases these bridges must be re-assessed in order to establish whether fatigue is critical. A simple and reliable procedure will be presented in order to assess the fatigue safety.

Evaluation: Stability [3.4.1]

Csagoly, Paul F. And Baidar Bakht. "In-Plane Buckling of Steel Trusses." Canadian Journal of Civil Engineering 5, no. 4, December 1978, 533-541.

This paper discusses buckling phenomena associated with truss structures. The authors state that buckling in a truss might occur due to one member failing, or an instability of a joint. A computer program was developed which takes into account the change in rotational stiffness of a member due to an applied load. A thorough explanation of the mechanics behind the methods is stated in the paper. Two frames were fabricated to test the results of the analysis using this program. The results of the tests were in agreement with the predictions of the

computer program. Estimates of failure loads were also computed using other published results. Again, the computer predictions were similar to the test results.

Evaluation: Stability [3.4.2]

Bober, Marlene N., ed. "Investigation of Historic Bridges." Forensic Engineering in Construction By Zallen Engineering 3, No 2.

This article gives a brief description of the investigation of a 71' Pratt pony truss. Field observation was used to determine member sizes due to the lack of construction drawings. A discussion of the lateral resisting system for a pony truss is included. A rating of 2.5 tons was given to the bridge. Since there is only light traffic on the bridge, removal was not warranted.

Evaluation: Stability [3.4.3]

Alibe, Bunu. "Characteristics of Columns with Uncertain End Restraint." Journal of Structural Engineering 116, no. 6, June 1990, 1522-1534.

This article examines a probabilistic approach of determining the buckling load of a column with uncertain restraints. The author summarizes other studies which use deterministic methods to solve similar problems. This paper proceeds to derive equations for partially end-restrained columns. This paper might be helpful if a detailed analysis of members is desired.

Evaluation: Stability [3.4.4]

Tarnai, T. "Lateral Buckling of Plane Trusses with Parallel Chords and Hinged Joints." Acta Technica Academiae Scientiarum Hungaricae 85, 1977, 179-196.

In the analysis of trusses, the buckling behavior of the structure is often an intricate, difficult problem. The two widely accepted continuum models for trusses include a cross-section with a deformable web, and a hinged cross-section with non-torsional flanges. The author, using the second method, derives a series of differential equations for the laterally buckled shape. Boundary conditions for various truss layouts are discussed. This article is very in depth as far as mathematical computations, and probably would not be helpful in the design arena.

Evaluation: Stability [3.4.5]

Machaly, El-Sayad Bahaa. "Buckling Contribution to the Analysis of Steel Trusses." Computers and Structures 22, no. 3, 1986, 445-458.

This article deals with the stability of semi-rigid steel trusses in flexure. The author outlines previous work related to the topic and suggests that the effects due to partial rigidity have not been investigated. A series of differential equations and stability functions are used to model trusses with one to six panels. For practical cases it was found that stresses due to moment were approximately 12% that of normal stresses in the web members. This article would probably not be helpful in the design arena.

Evaluation: Stability [3.4.6]

Webb, Stuart and Yura, Joseph, "Evaluation of Bridge Decks as Lateral Bracing for Supporting Steel Stringers," Research Report 1239-3, Center for Transportation Research, University of Texas at Austin, May 1992.

A full-size test on a wooden bridge deck showed that the deck nailers and the connection between the nailers and planks were the main contributors to the lateral stiffness of the deck. The measured coefficient of friction between the steel stringers and the wooden deck was 0.25. Design methods are presented for the bracing requirements for steel stringers including the required coefficient of friction.

Various typical bridge decks were evaluated, and it was found that the decks had sufficient stiffness to force the supporting stringers to yield before buckling. The required coefficient of friction for stringers with span-to-depth ratios of 40 is 0.08. Less friction is required for smaller span-to-depth ratios.

Evaluation: Stability [3.4.7]

Galambos, T.V. (Ed), "Guide to Stability Design Criteria for Metal Structures," 5th Edition, John Wiley and Sons, Inc., 1998.

This book provides extensive summaries and state-of-the-art reviews of numerous topics related to stability of metal structures. Topics of particular interest in evaluation of older bridge structures include built-up compression members (columns with lacing or battens) and out-of-plane stability analysis of the compression chord of pony trusses. Numerous references are provided.

Evaluation: Stability [3.4.8]

Kleiser, M. and C.M. Uang. "Steel Latticed Members under Cyclic Axial and Flexural Actions," Journal of Structural Engineering, ASCE, Volume 125, No. 4, April 1999, pp. 393-400.

To evaluate the cyclic behavior of a type of as-built steel latticed member for the seismic retrofit of the San Francisco–Oakland Bay Bridge, cyclic testing of three half-scale specimens was conducted. For eccentrically loaded specimens, local buckling in the end panel caused significant degradation in compressive strength; net section fracture of flange angles near rivet holes where lacing bars were connected was also observed. Bifurcation type global buckling was observed for the concentrically loaded specimen, causing a drastic drop in compressive strength. The buckling strength can be reliably predicted when the shearing effect and the appropriate effective length were considered. All three specimens were able to reach and exceed the strength predicted by the Load and Resistance Factor Design interaction formulas by about 6%, but this overstrength was slightly lower than that typically found in wide-flange beam-columns. Based on the test results, a modified axial force-moment interaction relationship was established. The ductility capacity ranged from 2.0 to 2.4.

Evaluation: Stability [3.4.9]

Duan, L., Reno, M. and C.M. Uang. "Effect of Compound Buckling on Compression Strength of Built-Up Members." Engineering Journal, American Institute of Steel Construction, Vol. 39, No. 1, First Quarter, 2002

Two types of built-up members are commonly used for steel construction. Laced or battened members with widely spaced flange components fall in the first type, and closely spaced steel shapes interconnected at intervals by welds or connectors form the second type. The compressive strength of both types of members is affected by the shearing effect. The compressive strength of built-up members may also be affected by the "compound" buckling due to the interaction between the global buckling mode of the member and the localized flange buckling mode between lacing points or intermediate connectors. In this paper, a factor b was developed to consider the effect of compound buckling. Numerical values of b , that are a function of the global slenderness ratio, local slenderness ratio of flange components, out-of-straightness ratio, and separation factor, are presented in charts. A proposed revision to the 1999 AISC LRFD Specification is also presented.

Evaluation: Structural Analysis [3.5.1]

Adeli, H. and K. V. Balasubramanyam. "A Heuristic Approach for Interactive Analysis of Bridge Trusses Under Moving Loads." Microcomputers in Civil Engineering 2, March 1987, 1-18.

The authors have developed a method for determining the maximum stresses using influence line diagrams (ILD). ILD's are generated for each member and like shapes are grouped together. A logic tree for calculating maximum stresses due to AASHTO loadings of uniform lane loading, two axle, and two axle plus one axle are presented. By following the steps in the procedures, maximum forces can be found. However, this procedure seems catered to an expert system which can analyze many parameters to recommend an optimal structure. The procedure noted is valid for only Pratt trusses.

Evaluation: Structural Analysis [3.5.2]

Adeli, H. and K. V. Balasubramanyam. "Heuristic Analysis of Bridge Trusses Under AASHTO Live Loads." Microcomputers in Civil Engineering 2, June 1987, 1-18.

This article expands upon the study presented in the previous work. The analysis is extended to include determinate K-trusses and indeterminate Parker trusses. Generalized ILD types are recommended for various members of a truss. Maximum forces due to uniform lane loading, two axle loading, and two axle plus one axle can be calculated by following the flowcharts created by the authors. This method lends itself to new construction and requires the use of a computer.

Evaluation: Structural Analysis [3.5.3]

Trautner, Janice J. and Dan M. Frangopol. "Computer Modeling and Reliability Evaluation of Steel Through Truss Bridges." Structural Safety 7, March 1990, 255-267.

Many forms of analysis have been used in evaluating old truss structures including 2D, 3D, and Finite Element methods. This paper analyses a through truss bridge built in 1935 to determine which modeling technique is the most applicable. The bridge is a 150' Pratt through truss connected by riveted gusset plates. The steel used in the truss is A7 with a f_y of 30ksi with a reinforced concrete deck with f'_c of 2 ksi. Live loads of a HS20-44 trucks were applied asymmetrically to produce a worst case scenario, as well as, torsion on the bridge. Four modeling methods were run: (1) 2D all truss elements (2) 2D beam and truss elements (3) 3D beam and truss elements (4) 3D beam and truss elements including the deck. Corrosion effects were analyzed with damage factors.

A detailed probabilistic analysis identified each member as either redundant or non-redundant. 2D truss elements: A table of maximum load carried by the system under each loading is presented. Ductile and brittle scenarios were considered.

2D beam and truss: Top and bottom chord and posts were modeled as Euler-Bernoulli beams. All other members were identified as truss elements. It was found in general that the beam model induced slightly smaller stresses in the diagonal members than the truss model. However, post and chord members saw an increase in stresses, some up to 15%. The load magnification factor at failure was larger than the truss model, but not substantially. The paper did not discuss the 3D models.

Evaluation: Structural Analysis [3.5.4]

Seong, C. K., B. A. Ward, B. T. Yen, and J. W. Fisher. "Behavior of Truss Bridges as Three Dimensional Structures." In Proceedings 1st Annual International Bridge Conference in Pittsburgh, Pennsylvania, 1984, by the Engineers' Society of Western Pennsylvania, 1984, 203-207.

In this article, the authors investigate the modeling techniques which can be used in analyzing truss bridges and compare these results to actual tests. In the study three analysis methods including 2D truss, 2D frame, and 3D frame were compared with the actual readings on the bridge. The authors conclude that the 3D frame model is the most accurate predictor of the actual stresses in the members. A second part of the investigation looked at the effects of a damaged member on the stresses in the other members. It was found that members near the damage experienced large changes in stress, while distant members saw little change.

Evaluation: Structural Analysis [3.5.5]

Korol, R. M., A. Rutenberg, and D. Banariol. "On Primary and Secondary Stresses in Triangulated Trusses." Journal of Construction Steel Research 6, 1986, 123-142.

This article investigate the contribution of secondary stresses in the failure of a truss structure. The authors provide an informative history of the discussion on secondary stresses

which covers the entire 20th century. The authors state that secondary stresses may be disregarded if: (1) the joints are stronger than the members and allow for moment redistribution (2) the joints are weaker than the members than they must be ductile enough to allow redistribution.

In their investigation, two Pratt trusses with welded connections were loaded to failure. An analytical model which accounted for secondary stresses was used to predict the failure loads. End restraints of pin-pin and pin-roller were investigated and compared with actual results. The models also analyzed the structure using the average yield strength and lowest yield strength of the members. From the test results, it was concluded that the trusses did experience secondary stresses and responded between pin-pin and pin-roller end restraint conditions. The lowest yield strength predicted the collapse of the truss more accurately than the average value. The fixity of the joint provided approximately 5% more capacity than the simple truss model.

Evaluation: Structural Analysis [3.5.6]

Tabsh, Sami W. "Simple Live Load Factors for Girder Bridges." In Structures Congress XII: Proceedings of the Papers Presented at the Structures Congress '94 Held in Atlanta, Georgia 24-28 April 1994, edited by N. C. Baker and B. J. Goodno, 497-502. New York: American Society of Civil Engineers, 1994.

In the analysis of a bridge with concrete on steel girders, distribution factors are often used. The current AASHTO factors do not account for such factors as span length, number of girders, and the stiffness of the girders. The author has developed an approximate method for distributing live load to the girders. An example for composite and non-composite girders is include in the article. The results were checked by performing similar calculations using the OHBDC code, a method developed using more exact techniques. The simple method developed by the author mimicked the complex calculations for the cases shown.

Evaluation: Structural Analysis [3.5.7]

Philpot, Timothy A.; Rosowsky, David V.; Fridley, Kenneth J. "Reliability of Wood Joist Floor Systems With Creep" Journal of Structural Engineering v. 121, no.6, June 1995, 946-954.

Light-frame wood floor are constructed from a number of parallel flexural member (joists) which provide the primary structural support. Sheathing material is attached across the top edges of these joists to form the floor surface. Because the joists are connected in this manner, load sharing occurs, which enhances the performance of the system as a whole. Current design procedures focus on the behaviour of single members with only limited provisions that account for the improved performance of systems. In this study, the reliability of wood joist floors is investigated considering both strength and serviceability limit states. The objective of the system reliability analyses is to determine appropriate system factors for use in single member load and resistance factor design checking equations. These factors are included to account for the beneficial effects associated with systems of parallel members. This study considers factors such as lumber species and grade, floor size, and the effects of creep deformations. A stochastic visco-elastic stress-strain relationship is used to model the time-dependent behaviour of wood, and stochastic pulse process models are used to account for the temporal variability of the loads. Cumulative damage analysis using two common damage

accumulation models is used to account for the time-dependent strength of wood flexural members.

Evaluation: Structural Analysis [3.5.8]

Davalos, J. F, Salim, H. A. “Effective Flange Width for Stress – Laminated T-System Timber Bridges” Journal of Structural Engineering, Vol. 119 no. 3, March 1993, 938-953.

Stress-laminated T system timber bridges consist has laminated deck sections combined with glued-laminated timber beams compressed transversely with high strength steel bars. In the design of these structures, a deck-and-beam T-section is isolated and analysed as a T-beam. This paper presents a regression equation for the computation of effective flange width for stress-laminated T-system timber bridge; a width over which a normal stress can be assumed constant. Using actual dimension, orthotropic material properties, and AASHTO truckloads, 125 bridge models were analysed by a special finite element program for general anisotropic shell and beam type structures. The analyses of the result were used to conduct a parametric study followed by a regression analysis to develop a prediction equation for the computation of effective flange width. Simplified equations for the computation for the computation of flange width, which are within current upper and lower bound elastic solution, are proposed for use in design.

Evaluation: Structural Analysis [3.5.9]

Trautner, Janice J. “Computer Modelling and Reliability Evaluation of Truss Bridges” University of Colorado, Ph.D. Dissertation, 1989.

In this study an investigation is conducted on the methodology used to analytically evaluate the safety of one of the oldest types of bridges in this country, truss bridges. This methodology is presented and applied to a representative, existing through -truss bridge. A detailed description and interpretation of the AASHTO loading for the bridge is presented. Bridge finite element models are identified and described. Both deterministic and probabilistic bridge evaluation methods are used. Result of four finite element models are presented and discussed: two two–dimensional models, one composed entirely of truss elements; and two three–dimensional models, one composed of beam and truss elements only and one including beam elements, truss elements, and a concrete deck. An introduction to bridge redundancy evaluation by using damage scenarios is presented. Recommendation with regards to truss bridge, computer modelling and modal size comparison are given. Finally further research needs are stated.

Evaluation: Non-Destructive Testing [3.6.1]

Clemena, Gerardo G. and Wallace T. McKeel, Jr. “Non-Destructive Inspection of Steel Bridge Members.” Nondestructive Inspection 1014-1019.

This article is a thorough list of current non-destructive inspection techniques. The author briefly describes each method. Comments on cost, difficulties, and advantages of each method are included. Techniques discussed include visual inspection, Dye-Penetrant, Magnetic Particle, Ultrasonic, Radiographic, Magnetic Flux Leakage, and Acoustic Emission. This article contains good basic information on the methods available for bridge engineers.

Evaluation: Non-Destructive Testing [3.6.2]

Pope, C. W. and K. J. Card. "The Detection of Lamellar Tearing by Ultrasonic Testing." Non-Destructive Testing-Australia. November/December 1976, 19-22.

This article describes some techniques in using ultrasonics to find lamellar tearing damage in welds. Examples of compression and shear wave tests are described along with the necessary considerations and possible complications associated with each. Although it is not discussed in the article, I believe that ultrasonic testing could be used in damage detection in bridge members. More particularly wrought iron, which has a fibrous nature which leads to lamellar tearing. This paper could provide valuable background information if ultrasonic testing was to be used in examining wrought iron bridge members.

Evaluation: Non-Destructive Testing [3.6.3]

Maurenbrecher, A.H.P., and G. Pernica. Review of Non-Destructive Test Methods for Assessing Strength, Serviceability and Deterioration in Buildings. Canada: National Research Council, March 1993.

This report is a synopsis of NDT methods which are applicable to steel, concrete, and masonry. The authors present a very sound introduction to ND methods by first breaking them down into visual, mechanical, electromagnetic, sonic, and radiographic categories. The following chapters contain summaries of NDT methods for different situations. Although this document is geared towards buildings, many of the NDT method are also used in bridge inspection. Sections discussing methods for investigating geometrical, strength and physical, and moisture properties are included. Manufacturers, advantages, disadvantages, applications, and required equipment are listed for each method. This report is a very good introductory source for information on NDT.

Evaluation: Non-Destructive Testing [3.6.4]

Ghorbanpor, Al. "Evaluation of Fatigue Cracking in Steel Bridges by Acoustic Emission" Proceedings Of Fifth International Conference On Structural Repair, June 1993, 231-235.

In this study acoustic emission technique was used to investigate the fatigue cracking activities during a series of laboratory tests of various bridge steels and during field testing of an in-service steel highway bridge. Loaded test trucks were placed on the bridge during the field-testing. It was found that AE signal from growing fatigue cracks contains distinct characteristic features which could be used to perform testing in history in noisy environment such as the in case of bridge structures. Good correlation was established between the AE activities observed during the field-testing and the crack growth that had been evaluated over the previous five years. It was shown that AE may be used as an effective tool to determine fatigue crack activities at a structural detail in a steel bridge.

Evaluation: Reliability Analysis [3.7.1]

Ebrahimpour, A., E. A. Maragakis, and S. Ismail. "Point Distribution Methods for Bridge Reliability Analysis." Forensic Engineering 3, no 2/3, 1991, 137-145.

This article describes a complicated method to analyze bridges using probabilistic methods. The author describes the use of 2n point estimates and three point estimates. Applications on continuous beams, steel girders, reinforced concrete and pre-stressed concrete are presented. However, this article is very in depth and probably out of the scope of normal bridge analysis.

Evaluation: Reliability Analysis [3.7.2]

"A Fatigue Reliability Model for Railway Bridges" Proceedings of the Specialty Conference, Proceedings of the 6th ASCE Specialty Conference on Probabilistic Mechanics, and Structural and Geotechnical Reliability, Jul 8-10, 1992, Pg. 320-323.

An equation is developed utilizing the Miner damage theory to quantify the remaining fatigue life of a railway bridge. The bridges examined were riveted truss bridges built near the turn of the century. The equation contains six variables that represent material and loading variability and analysis uncertainty. A random number generator was used to input various distribution factors for the variables. A sensitivity analysis was conducted to determine which variables affect the computations significantly. It was found that traffic volume and the fatigue linear model did not significantly affect the results.

Evaluation: Reliability Analysis [3.7.3]

Frangopol, Dan M., and Rachid Nakib. "Effects of Damage and Redundancy on the Safety of Existing Bridges." Transportation Research Record 1290, 1991, 9-15.

The authors provide a summary of methods and theories used in the analysis of redundancy in bridges. Both deterministic and probabilistic methods are discussed. Terms such as redundancy factor, reserve strength factor, and redundancy factor with respect to a given damaged state of the system are well defined. A composite steel girder/concrete deck bridge was modeled using a finite element analysis program named ABAQUS. Corrosion and accidental damage were introduced into the model and redundancy factors were calculated using deterministic and probabilistic methods.

Evaluation: Reliability Analysis [3.7.4]

Geibler, Karsten; Grabe, Wolfgang "Probabilistisch abgesicherter Ermudungsfestigkeitsnachweis für bestehende Straßenbrücken (Probabilistic based evaluation of fatigue safety of existing highway bridges)" p.437-444, Ernst & Sons. Stahlbau 65.

In the presented paper a newly developed multi step method for the evaluation of the remaining fatigue life of steel bridges is discussed. This method is based on the (traditional) damage accumulation concept and the fracture mechanics. The application on existing highway bridges is shown. Basic information about safety theory, in order to define the partial safety

factors and the design values, is briefly described. Furthermore, the decreasing value of the reliability index B in function of time will be shown on two examples, a new steel-concrete composite bridge and an old riveted steel bridge.

4. TESTING

Testing: Structure [4.1.1]

Choros, John, and Vinaya Shama. "Testing Railway Bridges for Increasing Life and Service Loads." In Structural Faults and Repairs: Proceedings of the 5th International Conference on Structural Faults and Repairs at the University of Edinburgh, July 1, 1993, by the International Conference on Structural Faults and Repairs, 1993, 35-42.

This article describes a large collection of tests conducted on various types of railroad bridges. Both concrete and steel bridges were evaluated in this study. Included in the study were the investigation of a riveted truss and a pin-connected truss. Both bridges spanned approximately 150 feet and were built in the early part of the 20th century. The article describes in detail, the monitoring equipment used in the study including strain gauges, data acquisition techniques, and various non-destructive methods. The results of the investigations are printed in separate reports referenced in the bibliography.

Testing: Structure [4.1.2]

Grundy, P. "Capacity of a Wrought Iron Lattice Girder Bridge After 117 Years in Service." Civil Engineering Transactions 28, 1986, 195-200.

A railroad bridge built in 1860 of iron lattice girders was statically tested and described in this article. The iron was corroded in many places including flanges and connections. Four tests were conducted (1) Static test of girder (2) Tensile coupon tests (3) Riveted splice tests (4) Fatigue tests. Results: (1) Girder Test: A bending moment equivalent to Cooper train was applied to the girder and failed at Cooper M218, without impact loads. Failure mechanism was rupture of tension flange which was not corroded. Compression flange which was corroded did not buckle prematurely as was expected. (2) Tensile test: elongation of 10%; mean yield stress 231 Mpa; 14 specimens were cut from the tension flange. (3) Riveted Splice Test: Two splices were tested. Tensile load was applied which placed the splices in double shear. Failure occurred on the net section in one section and an "endsplit" failure in the second. It was concluded that the capacity of the rivets were not less than 153 kN in double shear (7/8" rivets). (4) Fatigue test: 19 specimens. Failures occurred by fracturing of laminae. Test results lie above fatigue category C (AISC). The conclusion was made that the fatigue life of these specimens is comparable to mild steel.

Testing: Structure [4.1.3]

Aktan, A. E., K. L. Lee, R. Naghavi, and K. Hebber. "Destructive Testing of Two 80-Year-Old Truss Bridges." Transportation Research Record 1460 62-72.

Two decommissioned bridges were tested using non-destructive methods. An investigation of connection retrofitting by welding plates to critical connections was also undertaken. Objectives of this project included: (1) survey of existing similar bridges (2) investigation of simple methods to retrofit bridges (3) better understanding of condition assessments due to two main problems (a) inability to locate damage in hidden or hard to see members (b) a lack of reliable analytical modeling.

Bridges: Pratt through truss span of 46.4 m; members were built up riveted. The second bridge was a Camelback through truss with a span of 76.2 m. Both bridges were inspected on site to check construction drawings, locate deteriorated members, connections, etc., and to identify any pre-existing repairs. Both bridges were judged to be in fair to poor repair. Extensive explanation of these investigations are found in the article. Deteriorated members which would not affect vertical loading (e.g. wind bracing) were removed to conduct material testing. An account of the loading frames and testing methods, as well as, a list of instruments used can be found in the article. The bridges were tested at service loads. Static tests used the shakedown technique with peak loads of 12.8 HS20-44 and 20 HS20-44 trucks for the Pratt and Camelback bridges respectively. Graphs of load/deflection, strain histories, and failure patterns are given for each truss.

Conclusion: If bearings, abutments, and floor members are adequate, deterioration in some truss members or connections did not affect the load capacity of the bridge. This conclusion may not hold true for non-gusseted bridges (e.g. eye-bar connections). Frozen rollers may cause the bridge to act in an arch manner until significant load frees the joints. Retrofit of welded plates proved very successful.

Testing: Structure [4.1.4]

Nowak, Andrzej S., and T. Tharmabala. "Bridge Reliability Evaluation Using Load Tests." Journal of Structural Engineering 114, no. 10, October 1988, 2268-79.

This article investigates the evaluation of existing bridges using a reliability analysis. A reliability study was conducted along with load testing on a 110 ft. steel truss bridge built in 1948. A discussion of bridge reliability methods is included with a description of series and parallel systems, correlated and uncorrelated, and the probability equations associated with each. In testing a bridge behavior, proof, and ultimate loading methods are described with a discussion of the useful data obtainable with each type of test. The researchers proposed to use a proof test to modify the distributions associated with the bridge properties, to come to a better understanding of the bridge's capacity. To this end, the bridge was instrumented and loaded to determine the member forces and these values were compared to theoretical values. Reliability indices were modified from published values using the data previously obtained, and the reliability analysis was re-run. The load carrying capacity of the bridge was more accurately evaluated using the modified approach. By justifying some correlation (or load sharing) by bridge members, the capacity of the members could be enhanced by 30-55% thereby increasing the allowable loads on the bridge in the future.

Testing: Structure [4.1.5]

Bakht, Baidar, and Leslie Jaeger. "Bridge Testing-A Surprise Every Time." Journal of Structural Engineering 116 no. 5, May 1990, 1370-1383.

This paper draws on the experiences in testing a variety of bridges in Canada. The authors consider slab on girder, steel truss, and concrete bridges. Five unusual characteristics are documented in the article pertaining to steel truss bridges. These qualities include:

The various components of the tension chords of pin-connected trusses share loads so unevenly that only one component should be considered in the calculations for bridge evaluation.

In pony-truss and through-truss bridges, the floor system participates with the bottom chords of the trusses only if the stringers are connected effectively with all the nodes of the trusses.

In calculating the capacity of a compression chord, account should be taken of the potential sources of weakness introduced by the rippling of the cover plate between rivets, which may be caused by the buildup of rust between the cover plate and other components of the compression chord.

In some bridges advantage can be taken of the floor system of substantial flexural rigidity that can themselves take a sizable portion of the load directly spanning the truss supports

Component interaction can, in certain cases, be used to advantage in upgrading the analytical bridge capacity.

The authors suggest that the best way to determine the capacity of the bridge is by applying a proof test load.

Testing: Structure [4.1.6]

Elleby, Hotten A., Wallace W. Sanders, Jr., F. Wayne Klaiber, and M. Douglas Reeves. "Service Load and Fatigue Tests on Truss Bridges." Journal of Structural Engineering 102, no. ST12, December 1976, 2285-2300.

Two bridges in Iowa, constructed near the turn of the century were field tested. The bridge was comprised of eyebar tension members, steel posts, and built up members. The first part of the test involved a service load testing on the truss components. It was found that the pin connected analysis conservatively predicted the response of the bridge. Secondly, a service load test was performed on the floor beams. It was found that the deflections of the floor beams fell in between the predictions of pinned and totally fixed, determined by theoretical calculations. A third test was a service load test on the timber decking. In this case, the pin ended calculation provided a good estimate of the experimental results.

Eyebars were taken from the bridge to perform fatigue tests. A total of 30 eyebars were tested. Some of these were intentionally damaged and repaired. Highly scattered results were obtained from these tests, however, the stress ranges that these bridges were subjected to, would

not conceivably cause problems in the future. Static tests were also performed on 17 eyebars. Yield stresses were about 30 ksi, while ultimate strengths were consistently over 40 ksi.

Testing: Structure [4.1.7]

Bakht, Baidar, and Leslie G. Jaeger. "Behavior and evaluation of pin-connected steel truss bridges." Canadian Journal of Civil Engineering 14, 1987, 327-335.

Two pin-connected bridges in Ontario were proof tested to investigate the behavior of these bridges to loading. Both bridges were subjected to a load of 300 kN and monitored. The most unusual behavior observed was the unequal sharing of load in the bottom tension chords pairs. The chord forces were influenced by the floor system accepting a substantial portion of the load. However, in the end panel, the floor system did not participate in accepting part of the load. This was attributed to the lack of a floor beam at the truss support. From these tests, it was also concluded that the reserve strength usually associated with truss bridges, cannot be counted on in pin-connected systems. A technique for determining the dead load supported by the pairs of members is outlined. The frequency of the two bars are measured, and from the differences in responses, a ratio of force can be found. A conversion of proof test results to recommended load postings is also provided. However, these calculations only apply to Canadian codes, but possibly could be adjusted for AASHTO specifications.

Testing: Structure [4.1.8]

Szabó Gy. "Examination for the determination of load-carrying capacity of the bridge over the Sebes-Körös at Vészto." Periodica Polytechnica Ser. Civil Engineering 35, no. 3-4, 1991, 361-388.

This article describes the research conducted on a 100 year old wrought iron railway bridge. The history of the bridge includes a renovation 30 years ago to strengthen the bridge. It was necessary to examine the bridge to determine whether further service should be allowed. Material from the rivet plates and stringers was removed and tested statically and dynamically. The average yield and maximum stress observed were approximately 230 and 325 N/mm² respectively. Impact values were also presented. A fatigue test was conducted on samples with conclusions including: 1) longitudinal and transverse direction responses were similar 2) previous loading did not affect the tests 3) a larger standard deviation was noted than current materials. A fatigue test was also completed on a stringer from the trussed section of the bridge. Failure was initiated at the rivet holes, one of which had been damaged by the torching process to remove the rivet. A computer analysis using second order theory noted that stress levels in members are significantly lower than an analysis by a simple truss model.

The bridge was instrumented and a series of locomotives were passed over the bridge. Stress levels from the tests are included in the paper. The conclusions of the report include: no damage during its 100 years of previous service, the stringers could withstand very severe fatigue damage, including partial failure, and still carry service loads for long periods of time, the yield properties of the wrought iron were somewhat lower than current materials and variability was more pronounced.

A series of recommendations were made including:

A reduction of maximum speed to limit dynamic effects on the bridge

Inspection of the bridge every six months to identify possible areas of distress

Repairs should be made with care to prevent damage such as flame-widening a rivet hole

Testing: Structure [4.1.9]

Heins, Jr., Conrad P, and Charles F. Galambos. "Highway Bridge Field Tests in the United States, 1948-1970." Public Roads 36, no. 12, February 1972, 271-291.

This article lists tests conducted on various types of bridges over a span of 22 years. Two truss bridges, one in Maryland, the other in Indiana, are documented. References to available materials concerning each bridge are listed in the bibliography.

Testing: Structure [4.1.10]

Brandes, Klaus; Knapp, Jurgen; Herter, Jurgen "Messungen an alteren Stahl-brucken zur Zustandsanalyse und Ermudungsbeurteilung (Rating of structural and fatigue safety of old steel bridges by measurement)" p. 421-427, Ernst & Sons. Stahlbau 65, 1996.

When we evaluate old steel bridges, very often we discover considerable reserves of the load bearing capacity. Referring to three examples - a beam grillage of a truss girder bridge, a beam girder bridge and an arch-bridge - we present how to solve problems occurring on old steel bridges by measurement. It is an additional aim of the contribution to give attention to special structural effects which can only be discovered by measurement.

Testing: Structure [4.1.11]

Salane, H.J.; Baldwin, J. W. "Identification of Modal Properties of Bridges" Journal of Structural Engineering, v. 116, no.7 July 1990, 2008-2021.

Modal stiffness and damping are determined from experimental test data for a single-span laboratory bridge model and a full-scale three-span highway bridge. Steady-state vibration tests are conducted on both structures using an Electro-hydraulic actuator. Accelerations are recorded at selected locations on the structures. For the laboratory model, the modal properties are calculated from experimental data for the as-built model and after one flange is cut. Modal properties for the highway bridge are determined during a 40,000-cycle fatigue test of the in situ structure. Changes in stiffness and mode shapes are related to the observed deterioration of the structures. Mode shapes are the best indicators of the deterioration.

Testing: Structure [4.1.12]

Liu, Pei-Ling Liu "Identification and Damage Detection of Truss Using Modal Data" Journal of Structural Engineering, v. 121, no.4, April 1995, 599-608.

A method for identifying the element properties of a truss is developed in this paper. The measured natural frequencies and modal shapes of the truss are used in the identification process.

The finite-element method is used to formulate the eigenequation of the truss. The identification problem is then formulated as an optimisation program in which the error norm of the eigenequation is minimised. It is shown that given sufficient modal data, the element properties can be directly attained without iterations. Further, the solution is unique and global minimal. The identifiability of the inverse problem is then studied in depth. The perturbation method is adapted to investigate the influence of measurement errors on the identification results. A numerical example is presented to demonstrate the effectiveness of the proposed method in property identification and damage detection.

Testing: Structure [4.1.13]

Mazurek, David F.; Dewolf, John T. "Experimental Study of Bridge Monitoring Technique" Journal of Structural Engineering, v. 116, no.9, September 1990, 2532-2548.

Laboratory model tests were conducted to examine the feasibility of detecting structural deterioration in highway bridges by vibrational signature analysis. The model is a two-span aluminium plate girder bridge that permits vibrations to be induced using vehicular excitation. The ambient vibration method was used to obtain vibrational signature elements. Data was processed both by curve fitting and by using a more automatable analytical approach. Using low-mass vehicular excitation, ambient vibration results compare well with conventional modal analyses for resonant frequencies and mode shapes, but damping is overestimated. Roadway roughness and vehicle velocity do not influence resonant frequencies or mode shapes, although variable mass can have a significant impact of resonant frequencies. Vehicular mass influences on mode shapes appear to be minimal. Major structural degradation can cause significant changes to both resonant frequencies and mode shapes. Degradation is detectable using a readily automatable analytical approach. Preliminary full-scale tests suggest that vibrational signatures are obtainable in the field using the same methodology employed in the laboratory.

Testing: Structure [4.1.14]

Paultre, Patrick, Proulx, Jean, Talbot, Martin "Dynamic Testing Procedures for Highway Bridges Using Traffic Loads" Journal of Structural Engineering, Vol. 121 no. 2, February 1995, 362-376.

Dynamic bridge testing techniques used at the University of Sherbrooke on a series of highway bridges in the province of Quebec, Canada, are presented. These procedures were developed to obtain a reliable evaluation of the dynamic amplification factor for existing bridges, as part of an ongoing rehabilitation program of the province's road network. Vertical acceleration responses are obtained under normal or controlled traffic using different test vehicles and loading patterns. The vibration frequencies and mode shapes are calculated from a frequency analysis of the measured data, and used to calibrate finite-element models for each structure. The experimental methods and data-processing techniques are described. The need to establish standard testing procedure is discussed and some recommendations are presented. Examples of test results and comparison with finite –element eigenvalue analyses are given for three different bridges.

Testing: Structure [4.1.15]

Fu, Gongkang, Tang, Jianguo “Risk-Based Proof Load Requirement for Bridge Evaluation” Journal of Structural Engineering, Vol. 121 no. 3, March 1995, 542-556.

Proof load testing is considered a reliable approach to evaluation of an existing structure’s load carrying capacity. It can be effectively used for highway bridge evaluation, to either enhance an existing rating obtained by an analytical method or establish a rating for bridges unsuitable for analytical rating. Proof-load factors in design and evaluation codes vary, and their basis has not been well documented. For highway bridge evaluation in U.S. this paper proposes proof-load requirement in the format of load and resistance factors, to determine target proof loads and resulting load ratings. They derived to reach target structural reliability levels consistent with current practice by analytical methods, and to ensure relatively uniform safety of bridges. These requirements may be included in specification for highway bridge evaluation by proof-load testing in the U.S., and their application requires little knowledge of structural reliability theory.

Testing: Structure [4.1.16]

Pietraszek, T. T. “Full Scale Testing of Steel Railway Bridges” in: Bridge Evaluation, Repair and Rehabilitation. Dordrecht, Boston: Kluwer Academic, 1990, p.463-478 [Ref. 1.11].

Fundamental purposes of full-scale bridge tests conducted by the Canadian National Railways (CNR) are bridge safety and serviceability. Results collected during these tests were of a broader sense and allowed the study of more general aspects of bridge behaviour under moving loads as static and dynamic stresses and displacements, Impact effects, vibration and fatigue. This paper concentrates on major technical aspects of load, strain and displacement measurements, recording apparatus, and signal processing techniques and evaluation of results. Samples of obtained results concerning three different bridge structures tested recently are major conclusion discussed.

Testing: Structure [4.1.17]

Agarwal, Akhilesh C. “ Testing and Evaluation of a Steel Pony Truss Bridge” Proceedings of the 1988 annual conference of Road and Transportation Association of Canada (RTAC).

This paper contains results of proof load testing of Priebe Bridge, which is a single lane steel pony truss bridge with a non composite deck, and fixed bearings at both abutments. The structure was built in 1938 and is currently posted for 10 tonnes. The bridge was tested with a vehicle slightly higher than the load limit determined by lateral stability considerations using the LATBUCK computer program. The stability of similar bridges can be improved by providing this lateral restraint at the elbows.

Testing: Structure [4.1.18]

“Manual for Bridge Rating Through Load Testing” Research Result Digest HDR234, Transportation research Board, 1998.

This digest describes the research findings from National Cooperative Highway Research Program (NCHRP) Project 12-28(13)A, Bridge Rating Through Nondestructive Load Testing. The project developed and documented processes for performing load tests and using the test results to calculate bridge ratings. The research results are presented in the form of a manual, which provides guidelines for integrating the load testing of bridges with their load rating. The manual is supplemented with a technical report, which presents detailed data on two major technical areas - evaluating unintended composite action and establishing target proof load levels.

The contents of the manual are as follows: (1) Introduction; (2) General Considerations; (3) Factors Which Influence the Load-Carrying Capacity of Bridges; (4) General Load Testing Procedures; (5) Load Test Equipment and Measurements; (6) Diagnostic Load Tests; (7) Proof Load Tests; (8) Posting and Permit Considerations; (9) Illustrative Examples; Appendix A - Review of Bridge Load Tests; Appendix B - Procedures for Field Evaluation of Live Load Impact; and Appendix C - Fatigue Life Testing for Steel Bridges.

Testing: Members [4.2.1]

Brühwiler, E., I. F. C. Smith, and M. A. Hirt. "Fatigue and Fracture of Riveted Bridge Members." Journal of Structural Engineering 116, no. 1, January 1990, 198-214.

The purpose of the paper is to provide fatigue data on riveted connections. Almost all fatigue studies are done on bolted or welded connections. Three full scale bridge girders were tested in the investigation. One was a mild-steel girder that was built as a temporary structure but never loaded. Six wrought-iron girders of which two had experienced corrosion damage formed the second test group. It was estimated that the girders had experienced approximately 100 million cycles in its 100 years of service. The third set of test specimens consisted of three latticed, wrought-iron girders built in 1891. 20 million cycles of constant amplitude loading were applied to all specimens.

Results: Yielding and tensile strengths were about equal for wrought-iron and mild steel specimens. Wrought-iron had a fracture strain of 15% compared to 40% for the mild steel. The Young's Modulus of the wrought-iron was about 15% lower than for mild steel. It was noted that by observing polished macro sections mild steel had a homogenous nature while wrought-iron was lamellar. Fatigue results: Steel girders (test set 1) all but one failed above ECCS 90 (AASHTO C) levels. Wrought iron girders and latticed members (sets 2 & 3) all but two failed above AASHTO C. Corroded members did not have a lower fatigue results due to the fact that most failures occurred in the rivet holes. Rivets in corroded members were fine. The author concluded that wrought-iron elements have similar fatigue strengths to mild steel members.

Fatigue Strength under shear loads was tested and a conservative constant amplitude limit of 15ksi was estimated.

Fracture strength: High variability was found in tests. Table of results show that Charpy test results do not undergo drastic changes from -20 to 20 degrees Celsius. A discussion of determining critical crack length follows.

Other conclusions included using AASHTO D as a conservative estimate of fatigue strength and the critical crack length is smaller in wrought iron than mild steel.

Testing: Members [4.2.2]

Schindler, Hans-Jakob. "Toughness Evaluation and Assessment of Old Bridge Steel." International Association of Bridge and Structural Engineers (1313-1318).

The fracture toughness of wrought iron is very low. Evaluations of structures made of these steels often use standard Charpy V-notch tests which may be misleading in the case of wrought iron. Wrought iron, due to its anisotropic structure, tends to produce a wide scatter in V-notch tests, but also resists crack propagation. This article seeks to reach a better understanding of these two phenomena. Wrought iron's fracture behavior differs markedly from that of normal steel. Wrought iron has a large brittle to ductile transition temperature range and low upper shelf energies which indicate stable crack growth behavior. This behavior is due to the lamellar structure of wrought iron. The fibrous nature of wrought iron tends to arrest local cracking and allow for delamination and crack branching.

The author presents equations to relate standard V-notch quantities to an approximation of toughness. An equation is also presented to correct the impact testing values to lower loading rates, which the structure is subjected to. Finally, required toughness equations are presented which provide general yielding before fracture.

Testing: Members [4.2.3]

Szittner, A., L. Kristóf, and L. Kaltenbach. "Fatigue tests on the old stringers of the railway bridges." Periodica Polytechnica Ser. Civil Engineering 35, no. 3-4, 1991, 345-360.

Members from the bridges in Hungary were tested to examine their fatigue capacity. The bridges examined included a continuous multi-span girder built in 1948 and a trussed bridge built in 1911-12. The truss bridge had experienced some damage during WWII, but it was unknown which members were replaced. From the stringers in the bridges, four types of samples were cut. One being material from the flange plates and web plates which included the edge of the rivet holes (A). A second type of sample contained a rivet hole in the center of the specimen (B). The third type of specimen consisted of undisturbed material from the middle of the web plate with new holes drilled in the middle (C). Charpy specimens were removed from the flange plates and web plates both near rivet holes and undisturbed material (D).

Charpy tests of the 1948 bridge showed poor results with brittle failure possible. The trussed bridge showed better results in Charpy testing. A series of fatigue tests were run on the specimens. Results showed that (A) samples performed the best (180-200 N/mm²), (C) samples averaged between 130-150 N/mm², (B) samples performed the worst at 100-130 N/mm². A fatigue test was also run on a full stringer. The testing machine's capacity was reached before any sign of fatigue distress was observed. Cumulative damage theory was applied and determined that both bridges could remain in service with a closely monitored schedule of replacement.

Testing: Members [4.2.4]

Keating, Peter B., John W. Fisher, Ben T. Yen, and William J. Frank. "Fatigue Behavior of Welded Wrought-Iron Bridge Hangers." Transportation Research Record 950, 1984, 113-120.

This paper deals with the fatigue evaluation of welded wrought iron bridge components. The bridge investigated was a railroad bridge crossing the Mississippi built in 1888. To correct the problem of eyebar loosening, a practice of removing part of the eyebar and welding it back together with lap splices had been performed. This repair was undertaken in 1937 when welding techniques were first being developed. The primary question facing the investigators was, if the bridges could continue in service safely, or would fatigue concerns warrant closing the bridge. A series of field inspections and measurements were conducted to give preliminary information. To test the repair, three specimens were fabricated to replicate the conditions on the bridge. Actual specimens were also taken from a decommissioned truss span, replaced years ago due to a barge accident. All tests indicated that the welded joint was adequate for AASHTO fatigue category D and E loadings. To assess the likelihood of a future fracture, a traffic study was completed to estimate the damage which has, and will occur to the bridge. Predicted stress ranges showed that the bridge will be able to stay in service for many years to come. Conclusions arrived at due to this study include:

Cracks at the toe of the splice plate were arrested by the slag inclusions inherent in the wrought iron matrix.

Splice connection was determined to be a category D detail, but category E should be used for evaluation.

Fatigue crack growth could not be arrested in the edge welded wrought iron splices.

Testing: Members [4.2.5]

Hahin, C.; South, J. M.; Mohammadi, J.; Polepeddi, R. K. "Accurate and Rapid Determination of Fatigue Damage in Steel Bridges" Journal of Structural Engineering v. 119, no.1, January 1993, 150-168.

Fifteen representative bridges throughout the state of Illinois were instrumented with foil strain gages to determine their frequencies of loading and the magnitudes of stresses induced by traffic over a 3- to-8-hour period, depending on traffic volume. Fatigue prone details, such as cover-plated wide flanges, were instrumented. For each stress range increment gathered by the data-acquisition system, the cumulative damage sustained over an extended number of years is compared with the number of available fatigue cycles for that stress range using published $S = f(N)$ data for various details and the Palmgren-Miner linear damage rule. A new equation for factor of safety for welded structures subject to fatigue is proposed, taking dead load, live load, and bridge detail fatigue strengths into account. A new method of assessing future fatigue damage in bridges that takes traffic growth and increased truck weights into account is also proposed.

Testing: Members [4.2.6]

Gillespie, John W. Jr.; Mertz, Dennis R.; Edberg, William M.; Ammar, Nouredine, "Steel Girder Rehabilitation Through Adhesive Bonding Of Composite Materials," Proceedings of the 1997 55th Annual Technical Conference, ANTEC. Part 1 Apr 27-May 2 1997 v 1 1997 Toronto., Soc of Plastics Engineers Brookfield, 1171-1175.

A complete system approach to the rehabilitation of degraded steel bridge girders through the adhesive bonding of composite materials is presented and demonstrated. The flexural characteristics of steel bridge girders are shown to be improved through testing of various reinforcement schemes on scale steel sections and testing of a selected scheme on corroded bridge girders taken out of service. Applications to bridge girders with corrosion damage and fatigue crack damage are demonstrated.

Testing: Connections [4.3.1]

Yamamoto, Kazuyuki, Narioki Akiyama, and Toshie Okumura. "Buckling Strength of Gusseted Truss Joints." Journal of Structural Engineering 114, no. 3, March 1988, 575-590.

Investigation of the buckling modes of eight gusseted joints is presented. A detailed account of the buckling phenomena is documented for each test. A proposed design formula is also recommended. This paper might only be useful if the case study bridge has gusseted joints.

Testing: Connections [4.3.2]

Yamamoto, Kazuyuki, Narioki Akiyama, and Toshie Okumura. "Elastic Analysis of Gusseted Truss Joints." Journal of Structural Engineering 111, no. 12, December 1985, 2545-2563.

Experiments were performed to find the stress distributions of gusseted plates. Design formulae for finding required plate thickness are derived by the authors. This paper might only be useful if the case study bridge has gusseted joints.

Testing: Connections [4.3.3]

Leon, Roberto, Roeder, Charles W., and F. Robert Preese. "A Comparison of the Cyclic Performance of Bolted and Riveted Connections." In Proceedings of the Symposium on Structural Engineering in Natural Hazards Mitigation, Apr 19-21, 1993, 415-420.

This article reports the results of tests that compared three connections including A307 bolts, A325 bolts, and A502 rivets. Connection details were replicated using a T-stub connection common in building construction in the 1920's. Results showed poor performance by the riveted connection and good performance from the A325 bolts. It is concluded that under severe cyclic conditions, riveted connections lose their clamping force and therefore their dissipative capacity.

Testing: Connections [4.3.4]

van Maarchalkerwaart, H. M. C. M. "Fatigue Behavior of Riveted Joints." International Association of Bridge and Structural Engineers 37 691-698.

In this article, the author discusses a few variables which influence the fatigue characteristics of riveted joints. A variety of type of steel are tested with varying clamping forces, bearing-tension ratios, and shear tension ratios on double angle lap joints. Data was compiled from a variety of tests performed all over the world. A scatterband suggested by W.H. Munse was used in comparing the different tests. The clamping force results showed that as the grip of the rivet increases, the clamping force increases, and therefore the fatigue resistance increases. Other conclusions of this work suggest that as the ratio of minimum stress to maximum stress approaches one, the fatigue capacity decreases. Single lap joints with eccentric connection were tested with the largest eccentricity displaying the poorest performance. The author suggests a series of S-N curves to be used in the prediction of fatigue life for various combinations of single and double splices with different stress range factors. A discussion of riveted wrought iron connection is also presented. The wrought iron displays poorer fatigue resistance than steel. A S-N curve for wrought iron is also proposed.

Testing: Deck [4.4.1]

Sanders, Wallace W., F. Wayne Klaiber, Hotten A. Elleby, and Leonard W. Timm. "Ultimate Load Test of Truss Bridge Floor System." Journal of Structural Engineering 102, no. ST7, July 1976, 1383-1397.

A timber deck of a truss bridge built in 1909 was tested to failure. This article compares the test results to the theoretical capacity of the deck. A test set-up which loaded the bridge at four points was used. The first test positioned the load cell in the center of the deck. The second test eccentrically loaded the structure. It was found that the deck behaved elastically to approximately half its ultimate load. The theoretical capacity was very near the experimental in both tests. An AASHTO rating of H32 was determined from the ultimate load. However, this rating applied only to the deck and not the entire bridge.

Testing: Deck [4.4.2]

Avnet, R. Richard "Decay, Weathering and Epoxy Repair of Timber" Journal of Structural Engineering v. 111, no.2, February 1985, 328-342.

Wood used in structural applications can be a durable material. However, conditions in the surrounding environment can cause premature deterioration. Two common and related conditions are decay and weathering. An experimental investigation was conducted to evaluate the effects of weathering and decay on the epoxy repair of timber. In one phase, unprotected epoxy repaired joints were exposed to natural weathering in the southeastern United States. Joints were periodically loaded tested over a 4-1/2 yr. period and correlated to accelerate weathering tests on small epoxy bonded shear block samples. It is recommended that the dry condition shear strength of epoxy repaired southern pine be reduced by one-third when the repaired member is exposed to natural weathering conditions. Also, normal precautions (including preservative treatments) should be taken if the repair is exposed to weathering over a

multi-year period. A second phase consisted of epoxy repairing both lightly and heavily damaged weathered and decayed material. The strength of the repaired joints compared well with that of undamaged material. However, caution is advised in initiating such repairs without careful evaluation.

Testing: Deck [4.4.3]

Darlow, Mark S., Bettigole, Neal H. "Instrumentation and Testing of Bridge Rehabilitated with Exodermic Deck" Journal of Structural Engineering, Vol. 115 no.10, October 1989, 2461-2481.

An exodermic deck was used to replace a standard, full-scale depth cast-in-place composite reinforced concrete deck on a bridge over the New York Thruway. The exodermic deck, consist of a layer of reinforced concrete poured over a steel grid in prefabricated panels, is much lighter and simpler to install then standard composite decking. Thus, it can be used for redecking existing bridges, or for decking new bridges, much more quickly, and in a less expense, then the current standard procedure. This exodermic deck is the first to install in New York State. In order to verify the load carrying capacity, a series of live load tests were conducted on this bridge before and after redecking. The result of these tests compares favourably to analytical predictions. While the live load strain for the exodermic deck is somewhat higher than the for the original deck, as expected, the dead load strain is considerably less and the overall strain is well within original design limits.