
J. H. Daniels
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GUIDELINES FOR DETERMINING REDUNDANCY IN TWO-GIRDER STEEL BRIDGES

INTERIM REPORT

Prepared For
National Cooperative Highway Research Program
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J. Hartley Daniels, Professor of Civil Engineering, Lehigh University, was the principal investigator. John L. Wilson, Associate Professor of Civil Engineering, Lehigh University, was the co-principal investigator for this study. Wonki Kim was the Research Assistant and Ph.D. Candidate in the Department of Civil Engineering, Lehigh University.

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ABSTRACT

Article 10.3.1 of the AASHTO bridge specifications (13th Ed.) requires the use of reduced allowable fatigue stress ranges for nonredundant load path structures. As part of the AASHTO fracture control criteria this provision is intended to reduce the probability of fracture arising from undetected fatigue cracking. It is not clear, however, what constitutes a nonredundant as distinct from a redundant load path structure. This interim report describes the initial 12 month phase of a 30 month investigation undertaken to develop guidelines for determining redundancy in steel bridges. Case studies of 17 fatigue damaged and fractured steel girder bridges are presented. Previous research into redundancy is discussed. A new definition of redundancy is formulated which introduces the concept of serviceability as well as strength. Four worked examples are presented which illustrate the design procedures and guidelines which can be used to ensure the redundancy of simple span steel girder bridges which suffer near full-depth midspan fracture of one girder.
SUMMARY OF FINDINGS

This interim report describes the initial 12 month phase of a 30 month investigation undertaken to develop guidelines for determining redundancy in steel girder bridges. In this phase the redundancy of two-girder and multi-girder steel bridges are compared.

Case studies of 17 fatigue damaged and fractured steel girder bridges are presented. Of these 17 bridges, one is located in Ontario, Canada, one in Texas and the remaining 15 are located in the U.S. above the 40-th degree of latitude. The number of girders varied from two to nine. Spans varied from a single span up to 6-span continuous. The majority are 3-span continuous. The length of time the bridges were in service before damage was discovered varied from under 1 year to more than 16 years. Complete girder fracture occurred in 4 of the 17 bridges. In each case, however, collapse of the bridges did not occur, and each remained relatively serviceable under normal highway traffic. A summary of these 17 bridges and associated damage is shown in Table 1, pages 12 and 13.

It is pointed out that research into redundancy as defined by Art. 10.3.1 of the AASHTO Bridge Specifications (13-th Ed.) can date only from the late 1970's. An examination of the available research indicates that studies of the redundancy of two-girder bridges should include the investigation of the role of the bottom lateral and cross bracing systems as a means of efficiently and economically providing redundancy. Such an approach was taken by Daniels and Wilson in the
1986 FHWA/PADOT investigation of the redundancy of simple span two-girder steel bridges.

Alternate definitions of redundant and nonredundant load path structures are proposed herein which could replace the current definitions in Art. 10.3.1 of the AASHTO Bridge Specifications. These definitions are written in terms of parameters such as load levels, serviceability, and fracture scenarios of which require further definition. The definition of these and several other concepts such as alternate load path survivability and bridge rating can only be defined through appropriate research. This interim report presents the initial results of research into some of these concepts and parameters.

The concept of design for redundancy of two-girder bridges is illustrated by worked examples of redundant designs for two USDOT/FHWA standard two-girder steel bridges. Each design considers only a single fracture scenario consisting of a near full-depth midspan fracture of one girder. It is found that efficient and economical redundancy can be provided by designing an alternate load path consisting of the bottom lateral bracing, cross bracing, drag strut (floor beam) and deck which is subjected to cross bending.

It is found that the above concept is also applicable to multi-girder bridges. This is demonstrated by a worked example of a redundant design for a USDOT/FHWA standard four-girder steel bridge.
CHAPTER ONE
INTRODUCTION AND RESEARCH APPROACH

PROBLEM STATEMENT AND RESEARCH OBJECTIVES

Problem Statement

The design of steel bridges in the United States requires design against fatigue resulting from repetitive live loads \(^{(1)}\). The allowable stress ranges which are used in design depend upon whether the bridge is considered to be a redundant or nonredundant load path structure. Article 10.3.1 of the AASHTO Bridge Specifications \(^{(1)}\) defines redundant load path structures as "structure types with multi-load paths where a single fracture in a member cannot lead to the collapse". Nonredundant load path structures are defined as structure types "where failure of a single element could cause collapse". The "element" referred to is defined as a "main load carrying component subject to tensile stress".

The allowable fatigue stress ranges for redundant load path structures which are provided in Table 10.3.1A of Art. 10.3.1 result primarily from research by J.W. Fisher, et al, at Lehigh University over the past 25 years, much of it sponsored by the NCHRP \(^{(2} \text{ to } 8)\). The allowable fatigue stress ranges for nonredundant load path structures which are also provided in Table 10.3.1A are empirical and are not based on research results. These reduced stress ranges were determined simply by shifting the values for redundant load path

*References begin on page 52 of this report.
structures one column to the left and introducing additional values for over 2,000,000 cycles as evidenced by examining Table 10.3.1A in Ref. 1.

Design against fatigue by the use of the allowable stress ranges in Table 10.3.1A for either redundant or nonredundant load path structures does not guarantee that fracture of a welded steel bridge member cannot occur. Fracture is one possible outcome of undetected fatigue crack growth in any welded steel bridge structure. AASHTO assumes, however, that the consequences of fracture of a redundant load path structure are not severe in that total collapse is not likely to occur. Whether or not the bridge remains serviceable, after fracture occurs, for traffic travelling at normal highway speeds is not considered, however, and has not so far been addressed by AASHTO.

The consequences of fracture of nonredundant load path structures are assumed to be severe. It is, in fact, assumed by AASHTO that total collapse of the superstructure will occur. Therefore, the reduced allowable stress ranges provided in Table 10.3.1A, are intended to reduce the probability of a nonredundant load path structure developing undetected fatigue crack growth which could lead to fracture.

As a guide to bridge engineers, AASHTO classifies, by example, structures which are to be considered either redundant or nonredundant. These examples appear in Art. 10.3.1 including a footnote to Table 10.3.1A in Ref. 1. For example, AASHTO classifies multi-girder bridges as redundant and two-girder bridges as nonredundant. Such classifications are based on unrealistic beliefs widely held by bridge engineers on the behavior of bridges under dead and live loads. These
beliefs in turn are based on the usual over-simplified assumptions used in the design of steel girder bridges.

For example, in the design of straight two-girder bridges, the two girders alone (or the two composite Tee girders in composite construction) are considered to be the only design load paths available for transmitting all dead, live and impact loads to the substructure. The deck, stringers and floor beams are considered only to transmit the vertical loads to the two girders. The bottom lateral bracing (and top lateral bracing, if any) as well as the cross bracing (or cross frames or diaphragms) are assumed to play no part in sharing the vertical loads with the two girders. For noncomposite construction the flexural and torsional strength of the deck is not considered. For composite construction the torsional strength of the composite deck/girder system is not considered. In short, the three-dimensional behavior of all components of the superstructure acting together to share the vertical loads, especially when unsymmetrical vertical loads exist, is not considered.

Although the elementary design model of the two-girder superstructure greatly simplifies the design of the two girders (and for static loads can be shown to be safe) it fosters the erroneous idea that if one of the two girders suffers a nearly full-depth fracture, say at midspan, then all resistance to vertical loads vanishes and that the superstructure becomes geometrically unstable and collapse follows. The considerable amount of research into stress history studies of in-service bridges as well as full scale laboratory tests of three-dimensional bridge superstructure components indicates that the real
The performance of steel bridges is significantly different from that assumed in design (2 to 27).

The results of recent theoretical research into the behavior of two-girder steel bridges following a nearly full-depth midspan fracture of one of the girders indicates that a simplistic view of the structural behavior is not always justified in reality (28). That study shows that as the fractured girder deflects under dead and live loads, forces are introduced into the bottom lateral bracing system. In turn these forces are transmitted through the cross bracing to the deck which is subjected to in-plane (cross) bending. Differential displacement between the two girders also subjects the deck to torsion. The study shows that the after-fracture behavior of the superstructure is quite complex and may involve nonlinear as well as linear three-dimensional interaction between all the components making up the superstructure.

The study reported in Ref. 28 went on to develop a reasonably simple and straightforward linear elastic analytical procedure which can be used by bridge engineers to proportion the bottom lateral and cross bracing systems as the first step to ensuring redundancy of a two-girder bridge in the event of a near full-depth midspan fracture of one girder. The design example provided in the study showed that for simple span two-girder bridges a relatively small increase in sizes of the bottom lateral bracing members can provide the required redundancy.

Of major significance in that study is the direct provision for an after-fracture serviceability limit in the design procedure for
redundancy. The serviceability limit is expressed in terms of a deflection-to-span length ratio. Although the design example uses a limiting deflection equal to the span length over 300, it was beyond the scope of that study to define serviceability or to suggest what the proper value of this ratio should be. Realistic ratios would have to take into account variables such as vehicle speed and the effect of excessive deflection on the stability of the vehicle and safety of the occupants. That study did, however, suggest levels of dead, live and impact loading for use in design for redundancy and used those load levels in the design examples presented.

This interim report presents the results of the initial 12 month phase of this investigation into the development of guidelines for determining redundancy in specific types of steel bridges and is a follow up to the research results presented in Ref. 28. The March 1, 1986 start date for this investigation was selected so that part of this 12 month phase would overlap with the final phase of the PADOT study reported in Ref. 28. That final phase dealt with the development of a basic framework for the application of practical and economical design procedures and initial guidelines to ensure redundancy and after-fracture serviceability of simple span and two-span continuous two-girder steel bridges. As expected, the results of the PADOT study contributed significantly to the work of this investigation through the collaboration of the researchers during the final 5 months of the PADOT study. Although that study showed that it is possible to design two-girder bridges to ensure redundancy and after-fracture serviceability, much more remains to be done to complete the design procedures and
guidelines for two-girder bridges. The PADOT study also proposed practical approaches to ensure redundancy of multi-girder steel bridges as well.

**Research Objectives**

The overall objective of the 30-month NCHRP investigation is to go as far as possible within the available time and resource constraints to complete the work for two-girder and multi-girder bridges and, if feasible, go beyond that to provide guidelines for determining redundancy in other steel bridge configurations as well. The detailed plan of work for the remaining 18 month phase of this investigation is proposed in Chapter 4.

The following are the specific objectives of the initial 12 month phase of this investigation the results of which are reported herein:

1. To develop a better understanding and definition mainly of the redundancy of two-girder bridges.
2. To propose practical and economical design procedures and guidelines to ensure redundancy and after-fracture serviceability.
3. To develop a framework for a better understanding and definition of redundancy classifications.
4. To illustrate the concepts of design for redundancy by developing worked examples.
SCOPE OF STUDY

The scope of the initial 12 month phase of this investigation is as follows:

1. Collect and review the performance of a number of U.S. and foreign steel girder bridges in which fractures were observed.
2. Analyze and evaluate the information collected in item 1 above with respect to the observed after-fracture performance of the bridge.
3. Establish a general definition of redundancy in steel girder bridges.
4. Using the definition established in item 3 above, continue, as appropriate, the development of the methodology begun in the PADOT study (28) for application to two-girder steel bridges with near full-depth midspan fracture of one girder.
5. Extend the methodology to the design for redundancy of multi-girder steel bridges.
6. Perform worked examples illustrating the application of the methodology to steel girder bridges with near full-depth midspan fracture of one girder.
7. Prepare of an updated working plan for the final 18 month phase of the investigation including a description of the framework for the guidelines to be developed in that phase.

Research into the appropriateness of the reduced stress ranges in Table 10.3.1A for use in the design of nonredundant load path structures is not part of this investigation. Although this investigation includes consideration of after-fracture serviceability of a steel bridge no attempt is made to define what constitutes an acceptable level of serviceability. However, in order to show the method of accounting for serviceability in the worked examples in item 6 above, specific levels of serviceability are selected for
illustration only. Similarly, although the establishment of load factors for dead and live load and impact factors are beyond the scope of this investigation, specific levels are suggested and used in the worked examples – again illustrative purposes only.
CHAPTER TWO

FINDINGS

STEEL GIRDER BRIDGE DAMAGE - CASE STUDIES

Information was obtained (from J.W. Fisher's files at Lehigh University) and reviewed for 137 fatigue and/or fracture damaged steel bridges in the U.S., Canada and Japan. Of these, 17 were either two-girder or multi-girder steel highway bridges, having fatigue and/or fracture damage of the girders, and within the scope of this interim report. It is hoped that information on damaged European steel highway bridges can be found and included in the final report.

Appendix A of this interim report provides detailed case studies of the steel girder bridge damage for these 17 bridges. One bridge is located in Ontario, Canada (No. 1 Aquasabon Bridge) one bridge is located in Texas (No. 2 Atascocita Road Bridge) and the remaining 15 bridges are located in the U.S. at or above the 40-th degree of latitude.

The number of girders in the bridge cross sections varied from two (8 bridges) to nine (1 bridge). Spans varied from a single span (1 bridge) to six span continuous (1 bridge) with seven bridges having three-span continuous girders.

The length of time the bridges were in service before damage (substantial fatigue cracking or fracture) was discovered varied from under 1 year (1 bridge) to 5 to 9 years (3 bridges) to 10 to 15 years (4 bridges) to more than 16 years (2 bridges). No data was available for the other 7 bridges.
Table 1 Summary of Steel Girder Bridge Damage - Case Studies

<table>
<thead>
<tr>
<th>Bridge No.</th>
<th>No. of Girders</th>
<th>Girder Profile</th>
<th>Type of Detail</th>
<th>Extent of Cracking and/or Fracture</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4</td>
<td></td>
<td>Web transverse groove weld</td>
<td>85% of web, 65% of bottom flange</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td></td>
<td>Lateral bracing connection to bottom flange</td>
<td>2/3 of web, full bottom flange</td>
</tr>
<tr>
<td>3</td>
<td>7</td>
<td></td>
<td>Lateral connection plate web gaps</td>
<td>1/3 of web</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td></td>
<td>Lateral bracing connection to bottom flange</td>
<td>Fracture: full web, full bottom flange, part of top flange</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Web transverse groove weld</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td></td>
<td>Gusset plate welds to bottom flange</td>
<td>Substantial fracture of main girder</td>
</tr>
<tr>
<td>6</td>
<td>2</td>
<td></td>
<td>Floorbeam - girder web gap</td>
<td>Horizontal crack, top of web</td>
</tr>
<tr>
<td>7</td>
<td>2</td>
<td></td>
<td>Floorbeam - girder web gap</td>
<td>Vertical crack, top of web</td>
</tr>
<tr>
<td>8</td>
<td>2</td>
<td></td>
<td>Lateral bracing connection plate</td>
<td>15% of web</td>
</tr>
<tr>
<td>Bridge No.</td>
<td>No. of Girders</td>
<td>Girder Profile</td>
<td>Type of Detail</td>
<td>Extent of Cracking and/or Fracture</td>
</tr>
<tr>
<td>------------</td>
<td>---------------</td>
<td>----------------</td>
<td>----------------</td>
<td>-----------------------------------</td>
</tr>
<tr>
<td>9</td>
<td>2</td>
<td></td>
<td>Floorbeam - girder web gap</td>
<td>2.5&quot; to 10.5&quot; horizontal crack, top of web</td>
</tr>
<tr>
<td>10</td>
<td>6</td>
<td>(No recorded info.)</td>
<td>Cross bracing web gaps</td>
<td>Horizontal and diagonal crack, bottom of web</td>
</tr>
<tr>
<td>11</td>
<td>6</td>
<td></td>
<td>Lower longitudinal stiffener groove weld</td>
<td>3/4 of web</td>
</tr>
<tr>
<td>12</td>
<td>2</td>
<td></td>
<td>Bottom flange butt weld - electroslag weld</td>
<td>Fracture: full web, full bottom flange</td>
</tr>
<tr>
<td>13</td>
<td>4</td>
<td></td>
<td>Floorbeam - girder web gap</td>
<td>0.5&quot; to 6&quot; horizontal and diagonal cracks, bottom of web</td>
</tr>
<tr>
<td>14</td>
<td>2</td>
<td></td>
<td>Lateral gusset plate.</td>
<td>Fracture: 95 % of web, full bottom flange</td>
</tr>
<tr>
<td>15</td>
<td>2</td>
<td>(No recorded info.)</td>
<td>Floorbeam - girder web gap</td>
<td>19&quot; horizontal cracks, bottom of web</td>
</tr>
<tr>
<td>16</td>
<td>5</td>
<td></td>
<td>Lower longitudinal stiffener groove weld</td>
<td>Full depth web</td>
</tr>
<tr>
<td>17</td>
<td>9</td>
<td></td>
<td>Bottom flange butt weld</td>
<td>Fracture: full web, full bottom flange</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Lower longitudinal stiffener groove weld</td>
<td>1/2 of web, 1/2 of bottom flange</td>
</tr>
</tbody>
</table>
Although moderate to severe cracking had developed in 13 bridges and near full-depth fracture of a girder had occurred in 4 bridges no collapse of a span occurred. All case study bridges remained relatively serviceable following fracture. That is, it appears that all the bridges remained in service, without undue problems encountered by the bridge traffic until the damage was discovered.

Table 1 provides a summary of steel girder bridge damage for the 17 case studies reviewed in Appendix A. The bridge number in the first column of the table corresponds to the bridge number in Appendix A. The second column of the table shows the number of girders in the cross section. The third column shows schematic views of the steel girder profile together with the relative positions of cracks along the girder, the crack direction (vertical or horizontal) and the relative lengths of the girder cracks or fractures. The fourth column describes the type of detail involved. The fifth column provides a description of the extent of the observed cracking and/or fracture.

PREVIOUS RESEARCH

In 1978, the AASHTO Guide Specification for Fracture Critical Nonredundant Steel Bridge Members was introduced (29). Allowable stress ranges for nonredundant load path structures and examples of redundant and nonredundant load path structures were introduced into the 12-th Ed. of the AASHTO Bridge Specifications with the 1979 Interim Specification. Neither the allowable stress ranges for nonredundant load path structures nor the examples for redundant and nonredundant load path structures were determined by rational research. Thus
previous research into redundancy as presently defined by the 13-th edition of AASHTO can date only from the late 1970's.

Sweeney investigated the importance of redundancy in riveted and welded steel bridges (30). It is shown that fatigue and fracture are much more critical problems in welded structures than in riveted structures. This is because riveted structures have an inherent component redundancy and lower rigidity. Therefore, riveted structures tend to be fail-safe while welded structures are generally not component fail-safe. The study concludes that designers, fabricators, and inspectors must ensure that welded structures will not develop large cracks because they do not have the inherent crack stoppers which riveted structures have. This is absolutely critical for nonredundant load-path welded structures. The importance of steel bridge repairs are also discussed. If welded repairs are to be used, it is shown that they must be of American Welding Society (AWS) quality. Otherwise, the weld may destroy the initial component redundancy of the structure. It is concluded that the safe-life approach is an absolute requirement for nonredundant load-path structures.

Haaijer, Schilling, and Carskaddan introduced four new design procedures which handle redundancy and fatigue more directly (31). These methods somewhat close the gap between design approximations and actual conditions. Each design procedure is based on a load level and a primary performance requirement at that load level. The study presents an investigation into fail-safe analysis. The structural performance requirement at the fail-safe load is to provide adequate load-carrying capacity when a bridge has one separated component. It is
noted that a fail-safe load need only be considered when the design of a member is governed by fatigue. Questions such as: load level, components to be considered, and acceptable level of damage are introduced. The study concludes that a great deal of research is needed before fail-safe analysis becomes a realistic design tool. These new methods call for the use of redundant structures which are more rationally designed.

Csagoly and Jaegar established, by providing proper definitions, a framework of reference for further discussion into the merits of excluding single-load-path structures from future designs (32). Historical background with six cases of bridge collapses including the Silver Bridge, Lafayette Street and I-79 Bridges, Ontario-35 Bridge, Ontario-33 Bridge, truss bridges, and excessive movement of pier are presented. The six cases show that many existing bridges are unintentionally of the multi-load-path type. The study introduces key definitions of Collapse, Component, Failure, and Multi-Load-Path Structure which were provided by the 1979 Ontario Highway Bridge Design (OHBD) code. This is the first design specification attempting to deal with the issue of bridge design for redundancy. The study concludes that a mandatory backup system should be made a part of the design process. According to this study, the cost of the extra design work is only a small fraction of the potential savings. It is claimed that the introduction of compulsory backup systems will reduce the probability of collapse to nearly zero. This is shown with a simple calculation of probability of collapse by comparing failure of a primary member in a
single-load-path structure to the failure of both the primary and backup systems in a multi-load-path structure.

Heins and Hou studied the effects of cross bracing (diaphragms) and bottom lateral bracing on bridge redundancy (33). The study focused on two and three-girder bridges where one or both flanges of one of the girders is assumed to be cracked. For the two-girder bridge only the bottom flange is assumed cracked. The study apparently is conducted only in the elastic range. It is shown that bracing can effectively reduce the deformations in the girders. The study indicates that the effect of flange cracking on the three-girder bridge is negligible but quite important for the two-girder system. It concludes that if bracing is utilized, the two-girder bridge behaves similar to the three-girder bridge.

Heins and Kato followed up on the above study with an investigation of load redistribution in cracked girders (34). The study focused on two-girder bridges where one girder is assumed to be fractured near midspan. It is concluded that the influence of the bottom lateral bracing on load redistribution is significant. Further, the study concludes that utilization of the secondary members (cross bracing and bottom lateral bracing) effectively creates redundancy in two-girder bridges. Unfortunately, specific design procedures and guidelines for the design of the bracing members to ensure redundancy are absent. This study also appears to have been conducted only in the elastic range.

Sangare and Daniels conducted a computer study of the redundancy of a steel deck truss bridge (35). Post-elastic member behavior is
considered. In the investigation, one of the 340 ft. suspended spans of the Newburgh-Beacon Bridge No. 2 over the Hudson River at Newburgh, N.Y. was modeled for finite element analysis. The bridge, designed by Modjeski and Masters Consulting Engineers, Harrisburg, PA., is a deck type cantilever truss bridge carrying four design traffic lanes supported by two steel trusses. Each truss is 48-ft. deep. The two trusses are spaced 33 ft. apart. Each truss contains ten panels 34-ft. in length. In the redundancy investigation the tension (bottom) chord of one truss is assumed to be completely fractured at midspan. The analytical results (elastic and inelastic ranges) show that although the span would be considered nonredundant by most bridge engineers it carries at least full calculated dead load (load factor of 1.0) plus four lanes of HS20 lane loading (load factor of 1.0) plus AASHTO impact in all four lanes. Even in the fractured condition all members of both main trusses remain elastic. Redundancy is provided by the cross bracing system and top and bottom lateral bracing systems even after many members of these bracing systems have yielded in tension or buckled in compression.

Reference 36 reviews the state-of-the-art on redundant bridge systems as of 1985. Of the 51 references listed only 8 are dated since 1980. Of these, two appear in this interim report as Ref's. 33 and 34. The other 6 (as well as all those prior to 1980) do not address research on redundancy as defined by Art. 10.3.1 of the AASHTO Specifications. Among the conclusions in the review are the following statements:

1. Little work has been done on quantifying the degree of redundancy that is needed in bridges.
2. It is hoped that further research into structural redundancy in bridge systems will be conducted.

3. Computer speed and available software has made evaluation of redundancy more quantifiable than previously possible.

It is interesting to note in reading Ref. 36, which is generated by several individuals, that their use of the term "redundant" does not appear to be consistent. The early and latter parts of the paper use the term mainly in the context of AASHTO Art. 10.3.1, as is the use of the term throughout this interim report. The middle parts of the paper, those dealing with analysis of redundancy, types of analysis and modeling for analysis, appear to refer to "redundancy" as the excess capacity inherent in a normally designed and undamaged structure. For example, the use of the term "overload" must refer to the latter definition of redundancy since one would not likely be investigating the overload capacity of a fractured structure if the term overload is used in its normal context to mean over the AASHTO design load. Rather, in a fractured bridge the designer should be content to design for a specified "underload" (i.e.: under the AASHTO design load) to ensure redundancy as defined in Art. 10.3.1 of AASHTO. This "underload" concept is of importance and is mentioned later in this Chapter during the discussion of load levels. (See Definition of Redundancy)

Daniels, Wilson and Chen recently conducted a detailed research investigation into the redundancy of simple span and two-span continuous welded steel two-girder bridges as reported in Ref. 28. The research was sponsored by PADOT and USDOT/FHWA. The purpose of the
investigation was to study the behavior of three real two-girder steel bridge spans selected jointly by PADOT and the research investigators, to determine whether or not redundant load paths exist, and if so, to suggest design procedures and guidelines for ensuring redundancy of the case study two-girder bridges.

The three bridge spans selected for the investigation were:

1. Simple-span right, 90-ft. span, two lane, 32 ft. clear roadway.
2. Simple-span skew, 90-ft. span, 45° skew, two lane, 32-ft. clear roadway.
3. Two-span continuous right, two 90-ft. spans, two lane, 32-ft. clear roadway.

All three spans were taken from the Betzwood Bridge carrying LR 10461 over the Schuylkill River and Reading Railroad in Montgomery County, Pennsylvania, and were designed as noncomposite and to HS20 truck loading. Since all spans of this bridge are right spans, the 90-ft. skew span was obtained by modifying the 90-ft. right span. The three bridges were assumed in the investigation to be composite so that cross bending of the deck would provide the required transverse stiffness of the span following fracture of one girder.

In the investigation both upper and lower bound analyses were performed on the three-dimensional, welded steel two-girder composite bridges including deck, girders, floor beams, stringers, cross bracing and bottom lateral bracing systems. Upper bound analyses of all three bridges provided estimates of the stability limit loads under dead, live, and impact loads. Lower bound analyses of the simple span right
and the two-span right bridge provided elastic-plastic load-deflection curves up to near the respective stability limit loads. Excellent agreement is achieved between the upper bound and lower bound stability limit loads.

These analyses led to an understanding of load redistribution in the three composite bridges, to the identification of the alternate load paths that develop and to the formulation of specific design procedures and guidelines to ensure both redundancy (strength) and deflection control (serviceability) of the study bridges and similar bridges.

The following conclusions are contained in Ref. 28 and are based on the results of the investigation:

Simple Span Two-Girder Steel Bridges

1. Studies of alternate load paths and after fracture serviceability of two-girder bridges require the use of three-dimensional analytical models in order to simulate the role each bridge member and component plays during load redistribution.

2. For the two-girder study bridges the bottom lateral bracing system is the primary alternate load path following near full depth midspan fracture of one girder.

3. The cross bracing system acts together with the deck to resist the forces which develop in the bottom lateral system.

4. The cross bracing system is also required to provide sufficient stiffness to prevent significant distortion of the cross section.

5. The bottom lateral system can be easily and economically designed to provide both redundancy (strength) and deflection control (serviceability) following near full depth midspan fracture of one girder.
6. Design procedures and guidelines were developed for the design of the bottom lateral and cross bracing systems.

7. Reframing and redesign of the simple span study bridges were performed to demonstrate the validity of the proposed design procedures and guidelines for ensuring redundancy and after-fracture serviceability.

8. The redundant designs of the simple span study bridges were verified by finite element modeling and analyses of the redesigned three-dimensional bridges.

9. It is suggested that the redundant design procedures developed for the simple span study bridges are applicable to other similar two-girder bridges as well as to simple span multi-girder bridges.

Two-Span Continuous Two-Girder Steel Bridges

1. The two-span study bridge developed a reduced level of redundancy similar to the simple span study bridges and was not automatically more redundant than a simple span bridge. (Bridge engineers usually assume that continuous bridges are automatically more redundant than simple span bridges which was not the case in this study).

2. The major weakness of the two-span study bridge, from a redundancy point of view, was the reduced cross section at the point (region) of inflection and at midspan, which is a normal situation in traditionally designed continuous girders.

3. The two-span study bridge can easily be redesigned for redundancy using the bottom lateral system as the redundant load path, similar to the simple span study bridges, although a more economical redundant design, making use of the strength of the cantilever girder, was selected in the investigation.

4. In the investigation the two-span study bridge was designed for redundancy and serviceability after midspan fracture by redesigning the continuous girder over the negative moment region to carry the increased negative moment which follows the girder fracture.

General Conclusions

1. Girder redesign for continuous steel girder bridges is a relatively simple procedure to ensure redundancy and serviceability after midspan girder fracture.
2. The design procedures and guidelines developed in the investigation for the study bridges suggest themselves for application to the redundant design of other two-girder and multi-girder steel bridges with different configurations and with either the same or different girder fracture conditions.

3. The design procedures and guidelines developed in the investigation were demonstrated by means of hand calculations to show that they are relatively simple and easy to apply, and could readily be computerized.

4. The design procedures and guidelines developed in the investigation produce lower bound, safe, redundant designs since the strength of other components such as the torsional and flexural strength of the deck are ignored.

5. The design procedures may be performed by computer, but will require a three-dimensional discretization of simple span two-girder bridges in order to develop accurate forces in the bottom lateral and cross bracing systems.

6. If desired, hand calculations can be performed following the procedures developed in the investigation to provide a preliminary design of the bottom lateral and cross bracing systems for input to a finite element model for final analysis and design.

Parmelee and Sandberg presented the design of an actual three span continuous bridge for a given level of redundancy by Alfred Benesch and Company (37). It was decided to use a three girder design. For the study, failure was defined as the placement of a hinge at any point in one of the girders. Redundancy was provided by designing the cross bracing to carry the necessary transverse loads. The typical cross bracing was designed to yield under the application of the redundant load, while functioning normally under service loads. Redundant, or stiffened, cross bracing was placed at the field splices. A computer model showed that redundancy was provided by the interaction of the stiffened cross bracings and the failed girder. It is concluded that
redundancy is more than a question of having three or more main longitudinal members. It is also necessary to have "reliable" redundancy. Redundant paths must give visual signs of distress before they fail. The "warning system" is needed so that it is clear that the bridge is in need of repair after the fracture occurs. The study points out the need to be aware of the possibility of failure in members along the redundant path that were not designed to function as they actually did. The designer must investigate weak links along the redundant path that may prevent its use. It is emphasized that criteria need to be established for live load levels, permissible allowable stresses, load factors, deflection limits, and critical fracture scenarios.

Seim investigated economical ways in which redundancy can be achieved in steel bridges (38). The study recommends using parallel structural elements in the form of cables. The placement of cables across critical tension areas such as ties of arches or flanges of girders is suggested. If the steel develops a crack, the stress has the alternate load path of the cables available. The design of the Coushatta Bridge crossing the Red River in Louisiana is presented. This bridge consists of a 40 ft. wide concrete deck and is supported by two girders. This structure is considered nonredundant by current AASHTO Specifications. Computer simulation of a tension flange fracture in five different locations was examined. It is shown that the structure would survive carrying one lane of HS20 truck for all five fracture scenarios if reinforced cross-frames and lateral bracing were supplied. The study concludes that the cost of adding the bracing
was far less than the cost of adding a girder. It is emphasized that further research is needed to develop rules and proper factors of safety. Questions are introduced, such as, "What role does a concrete deck play?" and "What is the most effective way to develop a torsion tube without adding a lot of costly bracing?"

Probabilistic/Reliability Based Research

Galambos examined the use of a simple first-order probabilistic method to assess the reliability of the 1977 AASHTO Specifications for the design of steel bridges (39). It is demonstrated that the AASHTO LFD method provides a consistent reliability index but that the AASHTO ASD method does not. The study also investigated load- and resistance-factor design methods. These methods use multiple load factors and multiple resistance factors. It is concluded that load- and resistance-factor methods are shown to be most reliable and economical. Uniform reliability can be achieved through the judicious choice of load and resistance factors. The study concludes that there is sufficient statistical information on steel structures available to allow a probability-based design method to be developed.

Gorman investigated the interaction between structural redundancy and system reliability (40). Structural redundancy is defined as the degree of static indeterminacy. Increasing structural redundancy tends to increase the number of members that must fail before the system fails. However, increasing structural redundancy also increases the number of failure modes. The effect of these two different influences on system reliability is examined for a series of optimal trusses with varying structural redundancy. The study concludes for the truss
examples that increasing structural redundancy increased system reliability. The greatest benefit was achieved in going from statically determinate to two or three times indeterminate. It is shown that for highly redundant structures system reliability is only slightly improved, or even slightly reduced.

Moses and Verma in a recent NCHRP project have implemented a reliability-based strategy for evaluating bridge components (41). The application is not intended to predict the probability of structural failure but rather attempts to evaluate and adjust the safety factors in an evaluation code. The LRFD format was adopted for flexibility in dealing with different bridge components. The reliability of the partial safety factors is transparent to the code user and the designer would apply the LRFD check in a deterministic fashion. Strength rather than serviceability limit states are discussed. Safety is expressed in terms of a measure of the probability that the capacity will exceed the extreme load that may occur during the inspection interval. Data for the loading model have been assembled using Moses' WIM data. Load and resistance factors have been recommended leading to reliability levels. Numerous comparisons illustrate the effects on rating for different factors and options contained in the proposed rating guidelines. According to Moses these guidelines are suitable for inclusion in the AASHTO Maintenance-Inspection Manual.

General Conclusions: There is much useful literature and, of course, considerable difference of opinion about redundancy; and, in particular, which types of steel bridges can be defined as redundant.
Various tools for safety evaluation have been proposed which are at different stages of development. Topics include risk analysis; failure scenarios; progressive collapse; Bayesian uncertainty propagation models; strategies for ratings, inspections, and maintenance; knowledge-based expert systems, with fuzzy logic. Although many interesting results are available, the behavior and reliability aspects of the structural systems, which are the central focus of this research project, remain to be studied further (42) (43).

Though the further development of tools using probabilistic and reliability techniques for failure analysis, risk analysis and evaluation, and decision analysis are highly desirable, much more study is warranted. For example, more data needs to be collected, compiled, and exercised for model verification. In addition, the expert systems approach to damage assessment and decision support such as SPERIL-1 (44), although extremely useful in earthquake situations, is not yet appropriate for this project. The basic rationale behind expert systems, however, strongly suggest the potential for additional research and use in bridge safety and rating areas.

Summarizing, the probabilistic and reliability-based techniques, though innovative and important for safety analysis in general, are not considered to fit within the scope defined by the objectives of this project.
FACTORS INVOLVED IN REDUNDANCY

The term redundant and the associated terms nonredundant and redundancy have at least three different structural engineering definitions in bridge engineering. So that there is no confusion as to which definition is dealt with in this interim report, each of the three are briefly defined as follows:

1. **Statically Indeterminate Structure:** A statically indeterminate structure is frequently referred to as a redundant structure, or a structure containing redundancies, since removal of the redundant members or supports, for example, will result in a statically determinate structure. This investigation and this interim report is not concerned with this definition of redundancy.

2. **Overdesigned Structure:** Statically determinate or statically indeterminate structures may be inherently overdesigned as a result, for example, of following the provisions of a code or specification. The as-built structure is then considered to have excess capacity or redundancy with respect to the actual loads to which the structure is subjected. Although this use of the term redundancy may well apply to steel bridges designed by the AASHTO bridge specifications and is of considerable research interest, this investigation and this interim report does not address this aspect of redundancy either.
3. Definition by AASHTO Article 10.3.1: The term redundancy used in this investigation and interim report refers to redundancy within the context of the definitions of redundant and nonredundant load path structures as defined in the AASHTO bridge specifications, Art. 10.3.1 including the footnote to Table 10.3.1A in Ref. 1.

It is useful in the discussion of the factors involved in redundancy to reprint from Ref. 1 the full text of the AASHTO definitions of redundant and nonredundant load path structures:

**Redundant Load Path Structures:**

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

**Nonredundant Load Path Structures:**

Main load carrying components subjected to tensile stresses that may be considered nonredundant load path members - that is, where failure of a single element could cause collapse - shall be designed for the allowable stress ranges in Table 10.3.1A for Nonredundant Load Path Structures. Examples of nonredundant load path members are flange and web plates in one or two girder bridges, main one-element truss members, hanger plates, and caps at single or two-column bents.

Leaving aside the question of whether or not the examples cited by AASHTO are well founded, the definitions appear, at first reading, to be satisfactory although the definition of nonredundant load path
structures seems a bit awkward. To be more consistent with that for redundant load path structures the definition could be simplified to:

"Structure types with a single load path where a single fracture in a member can lead to the collapse".

On further reading of both definitions, including the suggested simplification for nonredundant load path structures they appear to require clarification and reformulation. Both definitions pivot on the word "collapse". But collapse is not defined. The AASHTO definition of redundant load path structures implies that if multi-load paths exist, collapse cannot occur. This is not necessarily true. After fracture occurs, all potential redundant load paths must not only exist but also be capable of resisting the redistributed loads. Otherwise, even with multi-load paths the structure still may be nonredundant. The phrases "multi-load paths" and "single load path" are somewhat misleading. They may have easy to visualize meanings in terms of the simplistic analytical models employed in structural design but not so easy to visualize meanings with respect to a real as-built three-dimensional steel bridge superstructure and the way the as-built structure redistributes loads after fracture of a fracture critical member (FCM). The definitions of redundant and nonredundant load path structures contained in the AASHTO provisions concern repetitive loading, toughness considerations and allowable fatigue stress ranges for steel bridges. However redundancy is also important in the context of other damage scenarios such as collision and corrosion damage of bridges for example. Also, absent from these definitions is a reference to the after-fracture serviceability of the steel bridge.
That is, immediately following fracture and up until the damage is discovered, hopefully a relatively short period of time, can all highway traffic using the damaged bridge continue to do so safely at normal highway speeds?

Before suggesting a general definition of redundancy this question and other significant factors involved in redundancy require further examination:

**Collapse:** The term collapse should be viewed not only with respect to behavior of the bridge or span following fracture but also with respect to the effect on the vehicles and occupants using the span. If the entire bridge or span collapses then all the vehicles on the span fall with it as well as those entering the span until the traffic can be stopped. There should be no question that in this case the span is nonredundant. On the other hand, suppose that only one girder of a multi-girder bridge fractures at midspan. Perhaps only the local area of the deck which spans the fractured girder collapses after the passage of a heavy vehicle, leaving a substantial void in the deck. Although this multi-girder bridge is considered redundant by AASHTO all vehicles entering the area of the local deck collapse suffer the same fate as though the entire span had collapsed. Still further, suppose that in the first case above, the span does not entirely collapse but deflects and twists substantially such as one might expect if one girder of a two-girder bridge fractures at midspan as reported in Ref. 28. These deformations may be of sufficient magnitude that vehicles travelling at normal highway speeds would not be able to
safely cross the span. In the second case, a similar situation occurs if the local area of deck does not collapse but suffers large deflections. It appears from this brief discussion that the term redundancy is not so much dependent on the term collapse as it is on the after-fracture serviceability of the bridge or span. In turn, this implies an appropriate after-fracture serviceability of the bridge deck or portion of the deck in the travelling lanes. Thus, alternate definitions of redundant and nonredundant load path structures should avoid the word collapse, which does not convey the full implications of redundancy.

After-Fracture Serviceability: One expects that after fracture of a member or component carrying significant vertical loads some adverse deformations of the bridge deck would occur. Members and components which carry significant vertical loads include the deck, stringers, floor beams, girders and bearings. The after-fracture deformation consists of two parts. One part is the deformation under dead loads. The other is the deformation under live and impact loads. The ratio between these two parts is not constant for a given bridge or span but is a function of the dead to live load ratio for the particular member or component under consideration. Serviceability of the bridge deck therefore, is also a function of the dead to live load ratio.

As an example, consider the near full-depth midspan fracture of one girder of a simple span steel two-girder bridge as was reported in Ref. 28. Assume that the dead to live load ratio for the girder is large and that collapse of the span does not occur. In this case most
of the after-fracture deformation of the bridge deck (vertical deflection plus twisting) occurs under dead load alone. The additional deformation due to vehicles, even heavy trucks, crossing the span is not so large. The total deformation, however, may or may not be tolerable for vehicles crossing the span at normal highway speeds.

As another example, consider the midspan fracture of one stringer spanning between floorbeams of a two-girder or multi-girder bridge. In this case the dead load to live load ratio for the stringer is probably quite small. Under dead load alone the after-fracture deflection of the bridge deck over the stringer is also probably very small since the deck may be quite capable of spanning the fractured stringer under the dead load alone. However, under heavy truck wheel loading the deflection of the deck over the fractured stringer may be very large. Again the total deflection may or may not be tolerable for vehicles crossing the span.

After-fracture deformations produced by dead and live loads are important from the point of view of after-fracture serviceability. In all cases, for the span as a whole or a local area of the bridge deck, the criterion for serviceability should be formulated on the basis of the maximum total dead and live load deformations which a vehicle and its occupants can safely tolerate. It is desirable, however, that some noticeable deformations occur following a fracture so that there is adequate warning to vehicles, either during their approach to a span or during their crossing of a span, that the bridge has suffered damage and should be repaired.
Load Paths: The AASHTO concept of redundancy in terms of single versus multi-load paths seems logical but in reality is difficult to define for complex structures such as bridges. Even for behavioral models of simple structures this concept can be misleading. Consider, for example, a load supported by a structure consisting of a single tension rod. Obviously this is a nonredundant structure since it consists of only a single load path. Now consider the load to be supported by two rods. One might logically classify this structure as redundant since two load paths exist. However, if the two rods do not share the load equally, and the rod carrying the larger part of the load fractures the other rod may not be capable of carrying the full load. Thus, although multi-load paths exist, the structure is still nonredundant. This argument can be extended to any number of rods, where upon fracture of the critical rod (FCM), an unbuttoning effect occurs which can lead to progressive failure of all the rods. Therefore, to ensure redundancy the relative strengths of all the rods is important.

Consider, as another example, a structure consisting of a single simple beam or plate girder. For midspan fracture of the girder, this structure is nonredundant. However, for a two-span continuous plate girder one might consider this to be a redundant load path structure since for midspan fracture in one span the loads might be carried by increased negative bending moment over the interior support. However, if the increased negative moment exceeds the capacity of the girder the structure again is nonredundant.

For complex three-dimensional structures such as steel girder bridges the concept of redundancy in terms of alternate load-paths
becomes much more involved. After fracture of a main vertical load carrying member such as a plate girder, the originally designed structure is transformed into an altogether different structure. Redundancy of this new structure can best be determined through a complex incremental load-deflection computer analysis considering nonlinear elastic-plastic and instability behavior of all the members of the structure as was done for the bridges reported in Ref. 28. Obviously this is not a viable analytical procedure for use in the routine design of steel bridges.

In view of the above brief discussion it should be obvious that the examples of redundant and nonredundant bridges presented in Art. 10.3.1 of the AASHTO bridge specifications are not so clear cut as they appear. The results of the extensive computer study reported in Ref. 28, for example, showed that the simple span two-girder study bridges were really more redundant than AASHTO would assume. Alternate load paths do exist in simple span two-girder bridges and the study bridges are not expected to collapse under a reasonable level of dead plus live loading selected in Ref. 28. However, since fairly large deformations also occurred the study bridges may not be serviceable. The computer study also showed that the two-span two-girder study bridge of Ref. 28 was no more redundant than the simple span study bridges since the reduced flexural strength of the fractured girder at the inflection point (region) did not allow after-fracture bending moments to be fully redistributed to the negative moment region.
DEFINITION OF REDUNDANCY

The apparent difficulty with the interpretation and use of the AASHTO definitions of redundancy and the associated examples can be substantially reduced if a positive, rather than the present passive, approach is taken in defining redundancy. Such an approach, which addresses the factors discussed above is illustrated in the following alternate definitions of redundancy.

Redundant Load Path Structure: New or rehabilitated steel bridges where at least one alternate load path is defined and designed to support the specified dead and live loads and to ensure serviceability of the deck following the fracture of a FCM.

Nonredundant Load Path Structure: All steel bridges not classified as redundant load path structures.

The above definitions (like the existing definitions) are still of little help to bridge engineers without additional guidelines, design procedures and specification provisions which are not presently available but which can and are being developed through research. Examples of the additional information required includes the following:

1. Load Levels: Load levels appropriate for use in designing the alternate load path(s) are required. These should be less than the AASHTO design load levels (ie: underload). On a load factor basis the load factors selected in Ref. 28 were 1.1 for dead load, 1.3 for live load and 30% impact. All of the other requirements of AASHTO were followed without change.
2. **Serviceability**: Examples of information required to establish serviceability of the bridge deck include: (1) the permissible sudden change in elevation of the bridge deck at an expansion or construction joint or at a local punching shear of the deck, (2) the permissible overall deflection of the deck under **dead and live** loads (span length over 300 was used in Ref. 28, and (3) the permissible rotation or lateral slope of the deck (in Ref. 28 it was assumed that if the lateral slope of the deck under **dead and live** loads exceeded about 15 degrees this would be sufficient to cause a truck to roll over).

3. **Fracture Scenarios**: For steel girder bridges there are a large number of possible fracture scenarios. Examples would include: (1) potential fractures arising from all possible fatigue crack locations along the length of each girder, (2) potential fractures arising from all possible fatigue cracks growing out of lateral and cross bracing connections into the main girders, and (3) fractures from fatigue cracking along the length of every stringer and every floor beam. Obviously, the bridge engineer cannot perform redundancy analyses considering all of these possibilities. It is the function of research to isolate from these possibilities the few critical fracture scenarios, to suggest simplified analytical models incorporating these few fracture scenarios into the design of alternate load
paths and to provide suitable guidelines and proposed specifications for consideration by AASHTO. The work reported in Ref. 28 provides an example of this approach for one fracture scenario for the study bridges.

4. **Alternate Load Path Survivability**: The alternate load path is to be identified and designed into the bridge for later use; that is, for use at some time in the future when it may be needed to provide strength and serviceability after fracture of a FCM. At that time all members and components of the alternate load path also become FCM's. Thus it is important that a fatigue analysis of all members and connections of the alternate load path also be performed for the unfractured bridge and for the AASHTO design load levels. Such a task may present a substantial increase in the level of analysis currently employed for many steel bridges. For example, a live load analysis of the bottom lateral and cross bracing members of a steel girder bridge is not required by AASHTO. This is not because AASHTO does not believe live load stresses and stress ranges exist in these members. It is because the AASHTO distribution of loads requirements (Section 3, Part C, Ref. 1) evolved from the concept that for static loading (AASHTO converts live load dynamic effects into "equivalent" static load effects) it is safe to ignore the contribution of these
members in the design of a girder. Thus, for vertical dead and live loads the girders alone become the design load path. For static loading it can be shown that this is a safe approach. For dynamic (cyclic) live loading it is not necessarily safe. Thus the analysis of an appropriate three-dimensional model of the unfractured steel girder bridge is required in order to provide the required fatigue strength of the alternate load path as well. This analysis is to be performed for the normal AASHTO specifications, not for the proposed specifications for design for redundancy. The new design load path then consists of the girders all plus the members and components along the alternate load path(s).

5. **Bridge Rating:** The results of research into redundancy of new and rehabilitated steel bridges are of considerable interest and importance in the rating of existing bridges. In this case the bridge engineer is required, in effect, to work backwards. The existing viable alternate load path(s) for the existing bridge are defined in terms of one or more critical fracture scenarios. For each of these load paths the live load rating is calculated using the same philosophy contained in the present AASHTO Manual (45). The smallest live load rating then governs the rating of the bridge in terms of its expected redundancy. The load
levels used in the redundancy calculations would likely be the same "underload" values as those used in the design of new and rehabilitated bridges for redundancy. The live load rating calculations should also consider after-fracture serviceability of the bridge deck. As for new and rehabilitated bridges there is not much point in providing redundancy if the deck slope or condition is such that vehicles cannot safely use the bridge after a fracture occurs.

The analysis of existing steel bridges to determine whether or not they have adequate alternate load paths is likely to be a much more difficult task, however, than the design of new and rehabilitated bridges for redundancy. In the latter situation the bridge engineer is free to frame the structure and proportion members and components comprising the alternate load path(s) to suit the given load levels and other design conditions. In the former case, alternate load path(s) may not be complete or may have severe load level or fatigue restrictions, primarily because they were not originally designed for the purpose of providing redundancy. Two examples will illustrate:

- Stress resultants (axial force, bending moment and shear) developed in a potential alternate load path consisting of, for example, the bottom lateral/cross...
bracing/drag strut (floor beam)/deck (cross bending) may be severely limited by member and connection strengths as well as by missing links in the load path. Missing links may consist of such items as the lack of connectors to transfer force from the drag strut to the deck, inability of the deck to carry cross bending due to lack of sufficient reinforcement, and placement of the bottom lateral bracing in an ineffective location such as above the bottom flange of the plate girders. (See, for example, Ref. 28 and App. B, C and D of this interim report for proposed designs of alternate load paths for new and rehabilitated bridges).

• Even if the strength calculation for an existing bridge indicates that an alternate load path exists and provides an adequate after-fracture load level, an unknown amount of existing and future fatigue cracking of some of the members and components of the load path may not allow that load level to be achieved. Thus the present condition of the potential alternate load path and its survivability
until fracture of a FCM occurs is of major importance. Suppose, for example, that the existing bottom lateral and cross bracing members and connections are part of the alternate load path. In the design of the bridge AASHTO did not require a calculation of stress ranges in these members and connections under live loading. The potential for existing as well as new fatigue cracking must be determined not only by inspection of the existing bridge but also by analysis of the existing unfractured bridge where the analysis is performed on an appropriate three-dimensional model of the structure, and under normal AASHTO design load levels as discussed in item 4 above.

6. **Allowable Fatigue Stress Ranges:** The allowable fatigue stress ranges provided in Table 10.3.1A of the AASHTO bridge specifications for redundant load path structures should be applicable for all steel girder bridges designed within the alternate definitions of redundancy stated above for redundant load path structures and as discussed in items 1 through 5. However, it should be noted that design against fatigue should apply not only
to main members and connections such as for the plate girders, but also to all members and connections comprising the design alternate load path(s). As mentioned above, this requires the calculation of live load stress ranges and displacement induced stress ranges in these components of the unfractured bridges under normal AASHTO design load levels.

The reduced allowable fatigue stress ranges provided in Table 10.3.1A for nonredundant load path structures should be applicable for all steel girder bridges defined within the alternate definitions of redundancy stated above for nonredundant load path structures.

The previous six examples illustrate the information needed to supplement the alternate definitions of redundancy. They also illustrate the complexity of the concept of design for redundancy. It is envisioned that this information, some of which will be developed in this NCHRP investigation, will be suggested to AASHTO for inclusion in the Bridge Specifications in the form of guidelines and design procedures. Chapter 4 of this interim report suggests a proposed framework for these guidelines.
EXAMPLES OF REDUNDANT LOAD PATH CALCULATIONS

Three examples of redundant load path calculations are provided in Appendix B, C and D of this report. All three examples use simple span steel girder bridges taken from recent USDOT/FHWA Standard Steel Bridge plans (46), as follows:


APPENDIX C: Welded Two-Girder Superstructure, 180-ft. simple span, 44-ft. roadway, designed to HS20-44 AASHTO lane loading.

APPENDIX D: Welded Four-Girder Superstructure, 180-ft. simple span, 28-ft. roadway, designed to HS20-44 AASHTO lane loading.

Examples of redundant load path calculations, to be realistic and to have any practical value, must employ steel girder bridges designed in accordance with the AASHTO bridge specifications. The USDOT/FHWA Standard Steel Bridge plans are used in this interim report because they represent standard designs and are widely available to bridge engineers. The 180-ft. span lengths, the different roadway widths and the alternate design live load levels were selected to provide a range of application.

In each example the alternate load path considered consists of the bottom lateral bracing/cross bracing/drag strut/deck system since the research reported in Ref. 28 and in this interim report was based on
these concepts. In Ref. 28, only the bottom lateral and cross bracing systems were considered and applied to two-girder bridges. In this NCHRP investigation these concepts were extended to include the drag strut and deck systems in some of the examples as necessary parts of the alternate load path and extended the application of these concepts to multi-girder bridges as well.

The examples presented in the Appendices of this interim report are intended to illustrate the development so far of design procedures and guidelines to ensure the redundancy and serviceability of two-girder and multi-girder steel bridges, but only for near full-depth midspan fracture of one girder. In order to design viable alternate load paths the example bridges required minor changes in framing. These changes are explained in the design examples.

Much more remains to be accomplished. In the remaining 18 month phase of this NCHRP investigation alternate fracture scenarios will be studied. Other alternate load paths will be investigated and guidelines and design procedures developed. A wider range of examples will be selected for application of these guidelines and design procedures.
CONCLUSIONS

This interim report describes the initial 12 month phase of a 30 month investigation undertaken to develop guidelines for determining redundancy in steel bridges. In this phase only two-girder and multi-girder steel bridges are compared.

The following conclusions are applicable to the studies reported herein:

1. Case studies of 17 fatigue damaged steel bridges are presented. Of these, 4 suffered major fracture of one girder. Of these 4 bridges, two are two-girder and two are five-girder bridges. No collapses occurred and all 17 bridges remained relatively serviceable under normal highway traffic.

2. Previous research, dating from the late 1970's was reviewed. It was concluded that research into the redundancy of two-girder bridge should concentrate on the investigation of the alternate load path containing the bottom lateral and cross bracing systems. This was investigated by Daniels and Wilson in the 1986 FHWA/PADOT study of the redundancy of simple span two-girder steel bridges.

3. It is concluded in this NCHRP investigation that the existing definitions of redundancy contained in Art. 10.3.1 of the AASHTO Bridge Specifications (13-th Ed.) are awkward and require clarification and reformulation. Alternate definitions of redundant and nonredundant load path structures are proposed. These definitions (like the
existing AASHTO definitions) are still of little help to bridge engineers without additional guidelines, design procedures and specification provisions. It is concluded that these can and are being provided through the ongoing research reported herein. This interim report presents the initial results of some of the research undertaken and proposes additional studies that are needed (some beyond the scope of this NCHRP investigation) to develop design for redundancy of steel plate girder bridges.

4. The concept of design for redundancy is illustrated by worked examples of redundant designs for two USDOT/FHWA standard two-girder bridges. Each design considers a single fracture scenario consisting of near full depth midspan fracture of one-girder. It is concluded that efficient and economical redundancy can be provided by designing an alternate load path consisting of the bottom lateral bracing, cross bracing, drag strut (floor beam) and deck (cross bending).

5. It is concluded that the above concept is also applicable to multi-girder bridges. This is illustrated by a worked example of a redundant design for a four-girder USDOT/FHWA standard bridge.

UPDATED WORK PLAN

The following studies are proposed for the updated work plan covering the final 18 month phase of this NCHRP investigation. All studies concern the after-fracture live load rating (AFLR rating) of various types of two-girder steel plate girder bridges unless otherwise mentioned.
Case Studies

Additional fatigue damaged bridges are to be examined and case studies formulated. It is hoped that information on fatigue damaged European highway bridges can be found.

Also, various types of existing undamaged two-girder bridges are to be collected and classified according to their structural configuration and redundant load path(s).

Simple Span Bridges

1. AFLL rating procedures will be formulated for alternate fracture scenarios selected on the basis of the case studies presented in Appendix A as well as any additional case studies.

2. Validation studies for evaluation of redundancy load level are to be conducted by computer using the facilities of the Computer-Aided Engineering Laboratory of the Department of Civil Engineering, Lehigh University for selected redundant designs.

3. Guidelines and AASHTO specification provisions will be proposed for the AFLL rating of various types of two-girder simple span bridges.

Continuous Girder Bridges

1. AFLL rating procedures will be developed for continuous steel girder bridges based on the design example shown in Ref. 28, for various fracture scenarios. This procedure makes use of the negative moment capacity and stiffness of the fractured girder.

2. Validation studies for evaluation of redundancy load level are conducted by computer for selected existing bridges.
3. Guidelines and AASHTO specification provisions will be proposed for AFLL rating of various types of continuous two-girder bridges.

Load Sharing

The investigation of load sharing between two possible alternate load paths will be studied. This is the approach taken by Sandberg and Parmelee (47). This approach may require nonlinear (inelastic or buckling) behavior of some of the components of the alternate load paths. If so, it presents a much more difficult problem than relying on a single load path. Bridge engineers are also not presently familiar, in general, with nonlinear approaches to design as well as live load rating.

Redundancy Classification

This research primarily focuses on determining redundant load path(s) and AFLL rating of various types of two-girder plate girder bridges. The bridges are to be classified according to their structural configuration and redundant load path(s) for the following detailed studies.

1. Determination of the critical fracture scenario.
2. Identification of alternate load path(s).
3. Development of methodology for evaluation of redundancy load level or AFLL rating factor.
4. Identification of incomplete load paths (missing and/or weak links).
5. Suggestions for upgrading weak links or adding missing links.

7. Consideration of load sharing.

8. Establishment of bridge inspection and replacement priorities.

FRAMEWORK FOR GUIDELINES

The alternate definitions of redundancy proposed on page 36 of this interim report were accompanied by a statement that these definitions require additional guidelines, design procedures and specification provisions for their implements. For the purpose of AFLL rating these definitions also require additional rating guidelines, rating procedures and specifications. It is anticipated that specific guidelines and specification provisions of AFLL rating can be proposed for various types of two-girder steel plate girder bridges. These guidelines and provisions would be part of a suggested larger framework which could incorporate future results of research into redundancy.

For example, consider the AFLL rating for redundancy of two-girder plate girder bridges. The AASHTO Manual for Maintenance Inspection of Bridges of Ref. 45 could contain a separate section on after-fracture live load rating for redundancy. In that section would be provisions for various types of two-girder plate girder bridges. These provisions would identify the alternate load path and its members and components which are included in the AFLL rating. These provisions would also indicate guidelines for retrofit and corresponding after-retrofit AFLL rating.

The AASHTO Manual for Maintenance Inspection of Bridges should also include provisions for load factors and serviceability criteria to be used in the AFLL ratings for redundancy. Although it is beyond the
scope of this investigation to propose specific load factors and serviceability criteria, the proposed guidelines should include a framework suitable for incorporate specific recommendations. The recommendations on load factors would most likely be incorporated into Section 3 of the AASHTO Bridge Specifications. The recommendations for after-fracture serviceability of steel bridges might be incorporated into Art. 10.6.

Supplementing the guidelines and specification provisions would be a number of worked examples illustrating the AFLL ratings of various types of two-girder steel plate girder bridges for redundancy.
REFERENCES


INTRODUCTION

Information was obtained (from J. W. Fisher's files at Lehigh University) and reviewed for 137 fatigue and/or fracture damaged bridges in the U.S., Canada and Japan. Of these, 17 were either two-girder or multi-girder steel highway bridges, having fatigue and/or fracture damage of the girders, and within the scope of this research. The following provides detailed case studies of the steel girder bridge damage for these 17 bridges.

1. AQUASABON BRIDGE

Location: The bridge is located on the north shore of Lake Superior on Highway 17 (Trans-Canada Highway) 130 miles east of Thunderbay, Ontario.

Date Opened: 1948

Description of Bridge: The bridge is a three-span continuous composite four-girder bridge as shown in Fig. A-1. Each girder is fabricated from W33x141 rolled beam with haunches at the piers and abutments. These haunches were fabricated by cutting the bottom flange from the web fillet and welding a 5/8 in. parabolic insert plate into the web which result in a
51.25 in. deep section as shown in the figure. The main girders are field spliced at two points in the center span 22 ft. from each pier. The splice points are placed at the points of dead load contraflection. The reveted splice consists of 5/8 in. flange plates on the interior and 1/2 in. plates on the exterior girders. All have two 3/8 in. web plates for shear splices. The four girders are composite section with a 7 in. reinforced concrete deck. The deck is connected to the girders by channel-type shear connectors.

Live Loading: H20 truck load

Date Damage Discovered: 1963 and 1973

Description of Damage: In 1963 cracks were discovered at the vertical butt weld detail in three of the six haunch inserts of the north interior main girder. One of these cracks extended 44 in. from the bottom flange into the girder web along a diagonal line starting from the vertical butt weld detail. In 1973 four other weld cracks were discovered in the main girders. These cracks propagated from large initial weld imperfections or inclusions in the short transverse groove welds at the ends of the parabolic haunch inserts in the main girders. One of the cracks penetrated 7 in. up the web beyond the 2.5 in. transverse weld and had cracked about 65% of the bottom flange. With large
imperfections residing near the bottom of the web, crack growth developed in the web and then into the bottom flange as a part circular crack. An enlargement of this crack extended it into the flange and up the web. After the crack penetrated through the flange, most of the fatigue resistance was exhausted. All cracks were discovered before the flanges fractured because the details were located near the contraflexure points where the dead load stress was small. Hence large fatigue cracks were able to develop from repeated live loads without brittle fracture of the remaining section. Of 24 welded details, 7 had cracked by 1973. Additional cracks have been detected since. The cracking that developed was caused by large imperfections that were fabricated in short transverse groove welds. The lengths of these welds were insufficient to produce sound connection.

Consequences of Fracture: No recorded information.

Repairs: The fatigue cracks discovered in 1963 in the transverse weld detail in three of the six haunch inserts of the north interior girder were repaired by welding cover plates or insert plates into the cutout hole. In 1973 the transverse weld area was cut out in a circular shape, and an insert was welded in its place. Where the crack penetrated the bottom flange,
it was gouged out and filled with weld material at a slow rate of deposit, by using low-hydrogen electrodes. All repaired surfaces were subsequently ground smooth and flush to eliminate stress concentrations. The probable adverse effects of the original repairs might be the possibility of new imperfections or inclusions fabricated into the repair welds. This was prevented by the installation of bolted flange splices to the entire haunch, as cracks were later observed in the groove welds that connected the flange to the web plate insert.
Fractures

W. Abut.  Pier 1  Pier 2  E. Abut

(a) Elevation

7" Slab

S 15 x 42.9

10'-4"  5'-2"

Secondary members not identified

(b) Half Cross Section

Fig. A-1 Elevation and Cross Section - Aquasabon Bridge
2. ATASCOCITA ROAD BRIDGE

Location: This bridge is Bridge "B" located on the Atascocita road in Texas.

Date Opened: Early 1950's

Description of Bridge: The bridge is a three-span continuous multi-girder bridge as shown in Fig. A-2. The superstructure consists of four longitudinal plate girders at a spacing of 14'-8". The girders are haunched at intermediate piers. The W21x68 floor beams, spaced at 20 ft, support a W21x62 stringer at midspan. The cross bracing consists of K-bracing.

Live Loading: H20 loading, 1949 AASHO Specification

Date Damage Discovered: 1977-1978

Description of Damage: A fracture was discovered at the lateral bracing welded to bottom flange of inside girder. It is located 25 ft. from the abutment, near the first interior diaphragm. A fatigue crack initiated at the lateral bracing splice weld to bottom flange. Initial fatigue cracking propagated through the flange and resulted in a fracture of 2/3 of girder depth. The fractured girder opened to a 2 in. gap at the bottom flange.

Consequences of Fracture: Noticeable sag on road.

Repairs: Rewelded, plate attached.
a) Elevation

b) Half Cross Section

Figure A-2 Elevation and Cross Section - Atascocita Road Bridge
3. CROMWELL BRIDGE

Location: The bridge is located on Route 3 over Route 9 in Cromwell Town, Connecticut.

Date opened: Late 1960's

Description of Bridge: The bridge is a 55 degree skewed, 5-span continuous, seven-girder bridge as shown in Fig. A-3. Each girder is haunched at intermediate piers 2 and 3. The 3/8 in. thick web varies in depth from 3 ft. 8 in. to 6 ft. 4 in. All the girders are connected to X-type cross bracing spaced at 22 ft. 4-1/2 in. Bottom lateral bracing is installed to connect all the joints between the seven girders and the cross bracing.

Live Loading: No recorded information.

Date Damage Discovered: 1973

Description of Damage: Three cracks were discovered at the midspans of three girders between piers 2 and 3, and other three cracks at the midspans between piers 3 and 4. A fatigue crack near the bottom of the web in the main girders initiated from the weld termination in a connection plate joining horizontal diagonal bracing to the main girders. The two cracks between piers 2 and 3 had propagated to 5 in. below and 4 in. above the connection plates, and were fairly recent because of their shiny surface. The other two cracks between piers 3 and 4 had propagated to 5 in. below and 10 in.
above the connection plates, and showing rust at time of discovery.

Repairs: No recorded information.
Fractures

59'-3"  124'-9"  147'-4"  124'-9"  56'-3"
S. Abut. Pier 1  Pier 2  Pier 3  Pier 4  N. Abut.

a) Elevation

49'-0" Roadway

7'-10"
6 spaces @ 7'-10" = 47'-0"

b) Cross Section

Figure A-3 Elevation and Cross Section - Cromwell Bridge

A-10
4. DECATUR BRIDGE

Location: The bridge is located on I-35 over the Grand River in Decatur County, Iowa.

Date Opened: No recorded information.

Description of Bridge: The bridge is an 8 degree skewed, three-span continuous, composite, multi-girder bridge as shown in Fig. A-4. The five girders are spaced 9' 7-1/2" apart and all girder webs are stiffened. All the girders are connected to X-type cross bracing spaced approximately 24 ft. apart. Only the center span has a conventional bottom lateral bracing system between the fascia and first interior girders.

Live Loading: No recorded information.

Date Damage Discovered: August 2, 1979

Description of Damage: There was a crack in the west exterior girder of the north bound bridge in the center span, at the first lateral bracing gusset plate from the north pier splice, 7'-5 toward the center of the bridge. The crack extends through the bottom flange and web and into the top flange. At this location the girder has a 16 x 1 bottom flange plate. On the inside of the girder at this location the 1/2 inch thick bottom lateral gusset plate is notched to frame in around a 7 x 1/2 stiffener about five inches above the bottom flange. The gusset plate is welded to the
stiffener and web plate with a downhand 7/16 inch groove weld. The crack was rusted through its entire length.

It appears that the crack originated in the gusset plate to stiffener groove weld and then propagated slowly along the stiffener face of the weld until contact was made with the web. The crack then probably slowly lengthened toward the flanges until at some point an abrupt brittle fracture occurred through the rest of the web, bottom flange and part of the top flange. The web crack shifts about 1-1/8 inches away from the south face of the stiffener at the bottom flange and the flange crack is perpendicular to the web. As the web crack approaches the top flange it shifts to a point 17-3/4 inches away from the stiffener and 5-1/2 inches below the top flange. At this point on the web the crack abruptly veers away from the stiffener with the crack in the top flange being about 36 inches south of the stiffener. The design plan showed that the lateral gusset plate was to be coped one inch on either side of the transverse stiffener slot so that the web to stiffener, gusset plate to web, and gusset plate to stiffener welds could not intersect. The bridge had been fabricated without this cope and as a result these welds did intersect. The
cause of the crack was probably the intersecting welds, combined with a roughly flame cut transverse stiffener slot and the use of partial penetration groove welds to connect the transverse stiffener to the lateral bracing gusset plate.

Consequences of Fracture: The section of the fractured girder towards pier had dropped about 7/16 inch measured on the outside edge of the flange and 3/16 inch on the inside, and had been displaced about 1/8 in. to the outside of the bridge. There is also a small diagonal crack in the slab overhang.

Repairs: The bridge was repaired by removing and replacing about 10'-3" section of the girder, extending from the north field splice to a couple of inches past the crack in the top flange. The new section of girder was spliced into the existing girder by using high strength bolted splices at both ends of the girder. Falsework was used to support the existing girder near the fracture before the floor could be removed and fractured girder section flame cut.
Fracture

S. Abut.   Pier 1   Pier 2   N. Abut.

110'     43'-5"     140'     110'

a) Elevation

40'-0" Roadway

8" Slab

5'-0"

LST 5' x 10.5

L's 3'2" x 2'2" x 5/6

W 36 x 135

Intermediate Section   Section Near Pier

4 Spaces @ 9'-7½" = 38'-6"

b) Cross Section

Figure A-4 Elevation and Cross Section - Decatur Bridge
5. DEKORRA BRIDGE

Location: The bridge is located in the City of Madison, Wisconsin.

Date Opened: No recorded information

Description of Bridge: The bridge is a three-span continuous welded two-girder bridge as shown in Fig. A-5. The two main plate girders, 21 feet apart, are connected to W21x62 floor beam and cross bracings. Each floor beam supports three W18x50 stringers. All the spans have conventional lateral bracing spaced at 20 ft. The gusset plate is welded to the top of the bottom flange rather than to the edge of the flange edge.

Live Loading: No recorded information.

Date Damage Discovered: around 1975 (10 years after construction)

Description of Damage: Cracking occurred first in the weld at the edge of the flange, which is in the center span and 60 ft. from the support, and then proceeded into the transverse welds. Another crack occurred at vertical web splice butt weld, which is 36 ft. 2 in. from the abutment. It appears that fatigue cracks developed in gusset plate welds joining the horizontal bracing to the upper surface of the bottom flange of the main girder. Another fatigue crack occurred in the vertical
web splice weld.

Consequences of Fracture: No recorded information.

Repairs: In an attempt to stop further fatigue cracking at the ends of the horizontal cracking, hangers were installed at the midway between the girders to reduce vertical deflection of the bracing. However, fatigue cracking continued which eventually led to a substantial fracture of the main girder.
a) Elevation

b) Cross Section

Figure A-5  Elevation and Cross Section - Dekorra Bridge

A-17
6. DES MOINES BRIDGE

Location: The bridge carries Iowa 163 over the Rock Island RR and East Four Mile Creek, located at the east edge of Des Moines, Iowa.

Date Opened: 1960's

Description of Bridge: The bridge is a 37 degree skewed, five-span continuous two-girder bridge as shown in Fig. A-6. The main two girders, 26 ft. apart, are connected to the plate girder floor beams. The floor beams do not frame into the bearing stiffeners but into an intermediate stiffener that is nine inches from the bearing stiffener. This is a single stiffener on the inside of the girder and it is not welded to the top flange. Each floor beam supports two W18x45 stringers.


Date Damage Discovered: September 17, 1979

Description of Damage: At some locations there is a horizontal crack in the girder web along the edge of, and parallel to, the web to top flange weld. This crack is visible from the inside and outside of the girder. At several locations there is also a 2 in. diagonal crack in the web that is visible from the outside. At all locations there is a crack in the top of the weld for the stiffener to web connection. Several of these had propagated into the web plate.
Repairs: 3/4 in. diameter holes were drilled at various locations on the main girders of this bridge.
Figure A-6  Elevation and Cross Section - Des Moines Bridge

a) Elevation

b) Cross Section
7. DES PLAINES RIVER BRIDGE

Location: The bridge is located on I-55 over Des Plaines River in Cook county, Illinois.

Date Opened: Fall of 1964

Description of Bridge: The bridge is a four-span continuous, 50 to 65 degree skewed, curved two-girder bridge as shown in Fig. A-7. The plate girder floor beams connecting the two main girders are spaced at 20 ft, and support 6 stringers. Main girder webs are 120" x 7/16", and flanges are 30" x 1-1/4" - 3-1/2". Floor beam webs are 60" x 3/8" with knee struts to bottom and top of girder connection plates.

Live Loading: No recorded information.

Date Damage Discovered: August 19, 1975

Description of Damage: Twenty web cracks in the main girders were reported at upper ends of floor beam connection stiffeners in 1975. In February 1977 some cracks progressed downward along welds or into web as much as 2-5/8 in. Two to four cracks occurred at each of 31 locations. It appears that differential deflection of skewed bridge girders induced out-of-plane bending into floor beam connections near the piers. High stress ranges resulted at the upper ends of the floor beam to web connection plates.

Repairs: Temporary repairs, August and September 1975; 1/2
in. holes drilled at crack ends. Permanent repairs, February 1977 included installation of reinforcement plates to resist twisting in web to flange connections. No additional crack growth observed during a May 1981 inspection.
Figure A-7  Elevation and Cross Section - Des Plaines River Bridge
8. DRESBACH BRIDGE

Location: The bridge is located on I-90 over the Mississippi River in LaCrosse County, Campbell City, Wisconsin.

Date Opened: 1960's

Description of Bridge: The bridge is a four-span continuous two-girder structure as shown in Fig. A-8, which is pin-connected to approach spans. The approach spans have two haunched welded plate girders spaced 24 ft. apart. The girders are connected to the floor beam and K-type cross bracing.

Live Loading: No recorded information.

Date Damage Discovered: No recorded information.

Description of Damage: The bridge experienced a web crack about 17 in. long that originated in a gusset plate to stiffener weld. The crack grew both up and down the web from the origin and appeared to terminate in the web to the bottom flange weld.

Repairs: Holes drilled in corners of gusset to stiffener welds. Crack arrester hole drilled at end of web crack and bolted splices replaced on flange and web.
Figure A-8 Elevation and Cross Section - Dresbach Bridge
9. IOWA CITY BRIDGE

Location: The bridge is located on I-80 over US 6 in Johnson County west of Iowa City, Iowa.

Date Opened: 1960's

Description of Bridge: The bridge is a four-span continuous two-girder bridge as shown in Fig. A-9. This bridge is a welded plate girder structure with plate girder floor beams and W18x45 stringers. There are bottom lateral bracings only between the piers and the first floor beam on each side of the pier, and from the abutments to the first floor beam. In the positive moment areas the floor beam stiffeners are welded to the top flange, and in the negative moment areas they are welded to the bottom flange. The bottom lateral bracing gusset plates are welded to the floor beam stiffeners and to the girder webs.


Date Damage Discovered: August 30, 1979

Description of Damage: There were 24 locations at which cracks were found. All of these locations were at the first floor beam from the pier and near the top flange where the floor beam frames into the main girder. At each of these 24 locations there is a horizontal crack, from 2-1/2 to 10-3/4 in. long, in the girder web along the bottom of the flange to web weld. At about six of
these locations there is an additional crack or cracks that occur in the web between 2 and 3 inches below the top flange. Some of these cracks are parallel to the top flange while other angle downward. At most of these 24 locations there are vertical cracks in the web to stiffener weld at the top of the stiffener and some of these cracks angle outward into the web.

It appears that these cracks are caused by out of plane deformation of the girder web in the small gaps left by coping the floor beam stiffener. At these locations the stiffener is not welded to the top flange and therefore any deformation at this location has to be taken by the girder web. These cracks are fatigue type cracks.

Repairs: In order to prevent these cracks from propagating, a 3/4 in. dia. hole was drilled about 1/2 in. farther along the end of each crack. Permanent repairs for these locations consisted of either removing the upper part of the floor beam stiffener and part of the floor beam or cutting large oblong cut-outs in the web of the girder at these locations.
Figure A-9 Elevation and Cross Section - Iowa City Bridge
10. I-70 FAYETTE COUNTY BRIDGE

Location: The bridge is located on I-70 over Ill. 140 and Conrail at Mulberry Grove, Illinois.

Date Opened: 1966

Description of Bridge: The bridge is a three-span continuous, 22 degree skewed, six welded plate girder bridge as shown in Fig. A-10. Five X-type cross bracings are spaced at 24 ft. centers between piers. Connection gussets bolted to stiffeners and connection plates, 3 in. to 5 in. clear of flanges.

Live Loading: No recorded information.

Date Damage Discovered: April 20, 1976

Description of Damage: Cracks initiated in weldments at lower ends of connection plates and progressed into web, horizontally and/or diagonally upward. Weldment cracking occurred between supports at lower ends of connection plates, at 81% of interior girder connections and 32% of west fascia girder connections. Similar cracking occurred at 25% of pier connections. The fatigue cracks resulted from differential deflection out of plane bending due to thrust of cross bracing.

Repairs: 5/8 in. dia. holes drilled at 63 crack ends. On east bridge 3 bolts of all bottom cross frames gussets were replaced with single bolt snug fit.
Figure A-10  Elevation and Cross Section - I-70 Fayette County Bridge
11. I-78 E.B. over Ramp "A" BRIDGE

Location: The bridge is located on Route I-78, Bedminster Township, Somerset county, New Jersey.

Date Opened: 1960's

Description of Bridge: The bridge is a simple-span, 69 degree skewed, Composite, six-girder bridge as shown in Fig. A-11. The main plate girders are stiffened with vertical stiffeners inside of the girders, and upper and lower longitudinal stiffeners outside of the facia girders. Each girder is connected to X-type cross bracing.

Live Loading: H20-S16-44 and two 24 kips axles spaced 4' apart were investigated based on 1957 AASHO Specification.

Date Damage Discovered: March 1970

Description of Damage: A vertical fracture was discovered in the fascia girder. It appears that a fatigue crack initiated at discontinuity in butt weld of the lower longitudinal stiffener of the fascia girder. The lower longitudinal stiffener was placed for architectural reasons. The crack propagated into the web, and the fracture extended 42.5 inches upward from the bottom flange and into the full bottom flange of the fascia girder.

Consequences of Fracture: No recorded information.
Repairs: A bolted field splice was placed over the web and bottom flange.
a) Main Stringer Elevation

b) Cross Section

Figure A-11 Elevation and Cross Section - I-78 E.B. over Ramp "A" Bridge
12. I-79 BACK CHANNEL BRIDGE

Location: The Interstate 79 Bridge carries I-79 northbound over the Ohio River backchannel between Moon Township and Neville Island, approximately eight miles downstream from Pittsburgh, Pennsylvania.

Date Opened: Approximately September 1976

Description of Bridge: Three-span continuous haunched two-girder bridges for each of the northbound and southbound roadways as shown in Fig. A-12. The pairs of main girders under each roadway are framed together at 25 ft. intervals with deep trussed or plate girder type floor beams which in turn support longitudinal rolled interior or filler stringers at 8 ft. centers. The adjacent interior main girders for the two roadways are joined together at 50 ft. intervals with trussed diaphragms designed to transfer one half of the maximum live load and impact from one parallel bridge to the other.

Live Loading: No recorded information.

Date Damage Discovered: Jan. 28 1977

Description of Fracture: A fracture was discovered through the bottom flange and the full depth of the web plate and. The fractured bottom flange is 3-1/2 in. x 30 in. plate, and the web is 1/2 in. x 132 in. plate. The fracture ended at the underside of the top flange.
plate. The fracture was located at an electro-slag welded shop splice in the bottom flange at the mid-point of the center span. The fracture was of a brittle nature with little or no apparent plastic deformation of the steel. The crack in the bottom flange plate on the girder opened approximately 1-3/4 in.

Consequences of Fracture: Field surveys showed that the crack in the bottom flange plate on the girder opened approximately 1-3/4 in. The concrete bridge deck slab sagged 5 in. below the theoretical elevation directly above the fracture. The top flange of the girder separated from the underside of the concrete deck over a length of approximately 50 ft. and dropped an additional 5/8 in. vertically at the point of failure. The total deflection is approximately 1/750 of the bridge span. The undamaged top flange of the fractured girder rotated outward approximately 3/8 in. with respect to the concrete deck and sheared off approximately 30 ft. of the outside portion of the concrete haunch. No cracks were discovered in the concrete deck, parapets and barriers. The bridge was closed to all traffic for about two months before reopened to full traffic.

Repairs: Repairing work was performed from a barge mounted
platform moored in the Ohio river. Splice plates were placed across the fractured web and bottom flange and drilled and bolted on both sides.
Figure A-12  Elevation and Cross Section - I-79 Back Channel Bridge
13. I-229 BIG SIOUX RIVER BRIDGE

Location: The bridge is located on I-229 over Big Sioux River in Sioux Falls, Minnehaha County, South Dakota.

Date Opened: 1960's

Description of Bridge: The bridge is a four-span continuous, 45 degree skewed, haunched multi-girder bridge as shown in Fig A-13. It has four main girders 8 ft. 10 in. apart. Web depths vary from 4 ft. to 6 ft. 8 in.


Date Damage discovered: Feb. 1977 and Nov. 1977

Description of Damage: Except for cracks at the top of the girder in one location, all cracks were discovered at or near the bottom of the girder in February 1977. A bottom flange to web weld was cracked about 5 in. In November 1977 new cracks were discovered at the top of the girder in one location, and two cracks extended about 1/2 to 1 inch further than it was marked in February 1977.

Repairs: No recorded information.
Figure A-13  Elevation and Cross Section - I-229 Big Sioux River Bridge

A-39
14. LAFAYETTE STREET BRIDGE

Location: The Lafayette Street Bridge spans the Mississippi River at St. Paul, Minnesota.

Date Opened: November 13, 1968

Description of Bridge: The bridge is a three-span continuous haunched two-girder bridge as shown in Fig A-14. The cross section consists of two main plate girders with floor beam, two stringers, cross bracing, deck, and gusset plates for bottom lateral bracing.

Live Loading: No recorded information.

Date Damage Discovered: May 7, 1975

Description of Damage: A fracture was discovered in the bottom flange and web of the main girder in the center span, 118 ft. 8 in. from pier 10 as shown in the figure. The web crack had propagated to within 7.5 in. of the top flange when it was discovered. The entire bottom flange was fractured. It appears that a fatigue crack growth originated in the weld between the gusset plate and the transverse stiffener as a consequence of a large lack of fusion discontinuity in this location. A brittle or cleavage fracture occurred after the fatigue crack propagated into the web through the gusset plate-stiffener weld. The cleavage fracture of web continued and also extended down into the bottom flange, and consequently the entire bottom flange was
broken. The cleavage fracture in the web was arrested 4 to 6 in. above the gusset. The web fracture surface was reported as shiny metal without a significant oxide coating from a point 4 to 6 in. above the gusset to the end of the crack near the top flange. The balance of the cracked section in the web and the flange was coated with product of corrosion. The crack appears to have arrested because the lateral gusset plate prevented excessive crack opening and the crack tip was far removed from the residual tensile stress field at the level of the gusset plate welded connection.

Consequences of Fracture: No recorded information.

Repairs: All gussets located in the regions of cyclic stress range and tensile stress were retrofitted to prevent other fatigue crack growth into the girder webs. The original fracture was bolt spliced after the cracked girder was jacked up from the adjacent bridge.
Figure A-14  Elevation and Cross Section - Lafayette Street Bridge
15. POPLAR STREET BRIDGE

Location: The bridge approaches are located on the bank of the Mississippi River in East St. Louis, Illinois.

Date Opened: 1971

Description of Bridge: The bridge is a six-span continuous bridge as shown in Fig. A-15. The majority of the bridges in the complex are on horizontal curves with approximately 1800 ft. radii of curvature. The torsional and side-sway (transverse) rigidity of the system are provided by W36x170 floor beams. The floor beams support four W18x14 stringers.

Live Loading: No recorded information.

Date Damage Discovered: 1973, 1975 and January 1978

Description of Damage: In 1973 fatigue cracks were discovered at the gap between the lower end of the floorbeam-main girder connecting plate and the bottom flange of the main girder near the end support. The cracks were first observed near the end of the girder under the end floorbeam connecting plate which was positioned 7 in. toward the center of the span from the bearing stiffeners. Most of these cracks started near the lower end of the vertical connection plate and extended in both directions along the web-flange weld. The longest crack was 19 in. long. Occasionally two cracks were observed at the same location one
immediately under the end of the connecting plate, and
the other along the web-to-flange weld.

Other fatigue cracks were discovered in the upper "web
gap" region of main girders at floorbeam connection
plates adjacent to end-bearing stiffeners which is
located at the ends of the continuous main girders.

In 1975 some other fatigue cracks were discovered in
the region between the top end of the floorbeam-girder
connecting plate and the top flange of the main girder
in the negative moment regions. One interior support
developed a full depth cleavage fracture. The fracture
likely occurred at a reduced temperature near the time
of its discovery in January 1978. The crack origin
was the web gap at the upper end adjacent to the
tension flange. The deformation out-of-plane indicated
that some difference existed in the elevation between
the two ends of the beam. This resulted in a locked in
out-of-plane force and eventually resulted in crack
instability.

Consequences of Fracture: No recorded information.

Repairs: For the girder end cracks, the floorbeam
connection plates were welded to the top and bottom
flanges in order to prevent relative displacement
between the ends of the connection plates and the
girder flanges. One-half in. holes were drilled
through the web at the ends of the existing web-to-flange connections. Drilled holes were also placed at the ends of the web cracks at the ends of connection plates or stiffener connection plates. The cracks were gouged out and welded with a full-penetration groove weld up to the hole.

For the cracks at negative moment regions, holes were drilled at each end of the cracks. Holes were drilled near the ends of the crack along the web-flange weld and on each side of the stiffener. This procedure permitted the crack to develop between the holes and thus softened the connection to accommodate the out-of-plane displacements.
Figure A-15 Elevation and Cross Section - Poplar Street Bridge
16. QUINEBAUG BRIDGE

Location: The bridge is located on Route 14 over the Quinebaug River in Canterbury, Connecticut.

Date Opened: 1949

Description of Bridge: The bridge is a two-span continuous, 35 degree skewed, non-composite, five-girder bridge as shown in Fig. A-16. The girders are haunched at the center support over a length of forty feet in each span. The depth of the girders at the location of the crack is approximately 64 inches, and approximately 94 inches at the center support.

Live Loading: No recorded information.

Date Damage Discovered: April 6, 1981

Description of Damage: A fracture was discovered at the flange shop weld on the north fascia girder. The fracture was located approximately at the center of the span, 75 feet from the east end of the bridge. Deficiencies in 19 other locations were indicated at bottom flange welds. The lengths of discontinuities vary from total flange width to three intermittent lengths of approximately 1-1/2 inches across the flange. It appears that initial cracks existed in the tension flange butt welds and crack propagation occurred from the embedded flaws in the transverse butt weld of the lower flange due to the cyclic (fatigue) loading of
the bridge. The crack propagated through the flange, into the web and almost through the entire depth of the girder before it was discovered.

Consequences of Fracture: No recorded information.

Repairs: A determination as to the location of the crack tip was made and a hole was drilled at that location as a temporary measure to arrest further propagation of the crack.
Figure A-16 Elevation and Cross Section - Quinebaug Bridge
17. QUINNIPIAC BRIDGE

Location: The bridge is located over the Quinnipiac River on I-91 near New Haven, Connecticut.

Date Opened: 1964

Description of Bridge: The bridge is a four-span continuous nine-girder bridge as shown in Fig. A-17. Span 1 is of composite construction with wide flange beams and welded girders. Spans 2, 3, and 4 are noncomposite welded girders of a cantilever type with a suspended center span. The entire structure is on a skew. The cross section of spans 2, 3, and 4 are composed of nine parallel plate girders supporting each roadway in between metal beam-type guard rails separating the traffic. The main girders have transverse X-type bracing and longitudinal stiffeners. The roadway is a 7-3/4 in. thick reinforced concrete deck.

Live Loading: No recorded information.

Date Damage Discovered: November 1973

Description of Damage: A large crack was discovered in the fascia girder of the suspended span in the center portion of the bridge. The crack propagated approximately to center of the girder web and had penetrated into the bottom flange of the girder at the time it was discovered. The location of the crack was...
approximately 59 ft. 6 in. from the left support of the noncomposite suspended span. A second crack was detected toward the midspan about 29 ft. from the cracked section. This second crack had severed the stiffener but had not propagated through the web. The fracture had initiated at the unfused butt welds in the longitudinal stiffener. The portions of the fracture surfaces in the vicinity of the stiffener groove weld were severely corroded from exposure to the environmental effects. The crack surface indicated that some crack extension probably developed from the unfused section of the butt weld across the thickness of the stiffener during transport and erection. It appears that the final brittle fracture and failure of the girder web resulted from an initial crack which started from an unfused butt weld in a longitudinal stiffener-girder web interface and enlarged from fatigue crack propagation. The crack condition at discovery in March 1973 indicated that the crack developed in several stages. It seems probable that the web crack instability occurred during the period of December 1972 to March 1973.

Consequences of Fracture: No recorded information.

Repairs: The cracked fascia girder was repaired by using bolt splice plates following the removal of the crack.
segments. In addition holes were drilled in the web in order to isolate the crack.
Figure A-17 Elevation and Cross Section - Quinnipiac Bridge
INTRODUCTION

Figures B-1 and B-2 show schematic views of the steel structure for the simple span two-girder noncomposite right bridge selected for design for redundancy. The bridge is contained in the 1982 USDOT/FHWA Standard Plans For Highway Bridges (35)*. The bridge selected has a 180-ft. simple span, a 28 ft. wide roadway and is designed for the 1965 AASHTO H15-44 lane loading. The girders are fabricated from A441 steel. All other steel is A36. A 7-in. noncomposite reinforced concrete deck carries two lanes of traffic.

Figure B-1(a) shows a plan view of the steel superstructure. Stringers and girders are spaced at 7'-8". Floor beams and cross frames are spaced at 20-ft. X-type top lateral bracing is also shown in the figure. Figure B-1(b) shows an elevation view of one of the two girders. The girder is symmetrical about midspan.

Figure B-2 shows the half cross-sections near midspan and near the bearings. At the interior floor beam locations the diaphragms consist of cross frames as shown

* References are presented on page 45 of this report.
in the figure. Cross bracing is used at the end diaphragm locations. Top lateral bracing frames into the gusset plates shown in the figure, which are located at the floor beam elevations.

DESIGN FOR REDUNDANCY

Redundancy is provided by an alternate load path consisting of bottom lateral bracing, cross bracing, floor beams, drag struts and deck (cross bending). In this example the bottom lateral and cross bracing systems plus the drag struts will be designed for a fracture scenario consisting of near full depth midspan fracture of one of the two girders. Design of the deck for the resulting cross bracing forces has not been fully developed and will not be included in this example.

The existing steel superstructure shown in Figures B-1 and B-2 contains top lateral bracing and interior cross frames. For this design example the top lateral bracing will be removed and lowered to the level of the bottom flanges of the girders. Cross bracings rather than cross frames will be assumed at all interior locations.

The cross bracings transferring bottom lateral bracing forces into concrete deck will produce downward forces on the unfractured girder and upward forces on the other fractured girder. When bottom lateral forces due to
midspan fracture are calculated in this design example, the upward forces on the fractured girder are neglected conservatively.

Load levels appropriate for use in designing the alternate load path(s) are required. These should be less than the AASHTO design load levels. On a load factor basis this bridge is subjected to loading consisting of 1.1 D plus two lanes of 1.3 (L+I) where the truck loading is H 15 lane loading with assumed 30% impact. An impact factor of 30% is assumed to account for the effect of increased deck deflections with traffic maintaining normal highway speeds.

The permissible overall deflection of the deck under factored live loads is assumed as span length divided by 300 to establish serviceability of the bridge deck.

In the following example, an iterative approach is used to obtain the forces in the bottom lateral members. This approach will yield answers as accurate as deemed necessary by the bridge design. Although hand computations are used, the procedure can be readily adopted to computer solution.

DESIGN OF BOTTOM LATERAL BRACING SYSTEM

Figure B-3 shows the analytical model used to calculate the bottom lateral bracing forces. A free body of the fractured girder is shown in Fig. B-3(a).
girder is shown with a midspan fracture. The factored uniform dead load, including 22 pounds per square foot for future wearing surface on the roadway slab, is calculated as 2.94 k/ft. The factored live loads, obtained by positioning two lanes of trucks towards the fractured girder for maximum midspan bending moment, are calculated as 1.02 k/ft uniform load and 28.8 kips concentrated load above the girder fracture.

The bottom flange is subjected to horizontal forces $F_1$ through $F_5$ as shown in the figure which are imposed by the lateral bracing system after the fracture occurs. The vertical reactions are also shown as 370.3 kips at each support.

The total force $F_1+F_2+F_3+F_4+F_5$ acting at the level of the bottom flange on half the span can be calculated on the condition of zero bending moment at mid span. The total applied moments are divided by resisting moment arm, 9.75 ft., which is the girder depth as shown in the figure.

$$F = \frac{1}{9.75} \left[ \frac{1}{8} (2.94+1.02)(180)^2 + \frac{1}{4} (28.8)(180) \right]$$

or $F = 1777.8$ kips \hspace{1cm} (B-1)

The arrangement of bottom lateral system is shown in Fig. B-3(b). The spacing center-to-center of the girder webs is 23 ft. The forces $F_1$ through $F_4$ are each developed
by two diagonal members framing into the girder flange. The force $F_5$ is developed by only one diagonal member. It is assumed that all the diagonal members of the bottom lateral system are identical, having equal cross section areas, $A_b$, and equal properties.

Figure B-4 shows the displacements of the fractured girder and the bottom lateral system after fracture. In Fig. B-4(a) the fractured girder is shown in its deflected position. If the horizontal displacement of the bottom flange at the fracture is assumed as $d$ as shown in the figure then the vertical displacement at the fracture, $v$, is $90d/9.75 = 9.23d$. In Fig. B-4(b) the displacements of the bottom lateral system are shown. Both of the diagonals in Bay 5 are in tension. In Bays 1 through 4 one diagonal is in tension, the other in compression as shown. The dashed lines show the original position of the bottom lateral and cross bracing members. The solid lines show the positions after fracture. The horizontal displacement of joint A is $d_1 = d$. Since no girder shortening occurs between joint A and the fracture, $s_1 = 0$. The displacement, $d$, is entirely controlled by the level of stress, selected by the designer, in the tension diagonals in Bay 5. The horizontal displacement $d_2$ of joint B is less than $d$ by the amount of girder shortening, $s_2$, between A and B. Similarly the horizontal displacement of joints C, D and E...
is \( d-s_3 \), \( d-s_4 \) and \( d-s_5 \) respectively. The horizontal displacements of joints \( F, G, H, I \) and \( J \) on the unfractured girder due to girder elongation are \( e_1 \), \( e_2 \), \( e_3 \), \( e_4 \) and \( e_5 \) respectively.

It is assumed in the design of the lateral bracing system that no relative displacement occurs between the girders. That is, the cross bracing horizontal members connecting the bottom flanges of the girders are assumed to be axially rigid.

Since the forces in the diagonal member can be obtained from a trial and error approach, the initial force distribution ratio of the diagonal members in Bay 5 through Bay 1 is assumed as 5:4:3:2:1 as shown in Fig. B-4(c). The resulting forces acting on both girders are determined with using \( F = 1777.8 \) kips.

Consider, for example, the segment of the fractured girder from \( A \) to \( B \). If \( N \) is the sum of the forces applied at joints \( B \) through \( E \), then the displacement, \( u \), of joint \( B \) relative to joint \( A \) is,

\[
\begin{align*}
  u &= \frac{NL}{A_0E} + \frac{NL}{EI_y} \frac{h^2}{4} \\
  \text{(B-2)}
\end{align*}
\]

where \( E = 29,000 \) ksi (Young's modulus)

\( L = 20 \) ft. (bay length)

\( h = \) overall girder depth (See Table B-1)
\[ A_g = \text{area of girder} \quad (\text{See Table B-1}) \]
\[ I_g = \text{moment of inertia of girder} \quad (\text{See Table B-1}) \]

Equation (B-2) can be used to calculate the relative displacement between any two joints on the fractured and unfractured girders except, of course, in Bay 5 of the fractured girder.

Equation (B-2) is used to calculate the following values of the displacements \( s \) and \( e \) shown in Fig. B-4(b).

\[
\begin{align*}
    s_1 &= 0 & e_1 &= +0.0616 \text{ in.} \\
    s_2 &= -0.1314 \text{ in.} & e_2 &= +0.1930 \text{ in.} \\
    s_3 &= -0.2115 \text{ in.} & e_3 &= +0.2730 \text{ in.} \quad (B-3) \\
    s_4 &= -0.2592 \text{ in.} & e_4 &= +0.3208 \text{ in.} \\
    s_5 &= -0.2757 \text{ in.} & e_5 &= +0.3373 \text{ in.}
\end{align*}
\]

If \( k \) is the axial stiffness of a diagonal member, then

\[
    k = \frac{29,000 A_b}{30.48 \times 12} = 79.287 A_b \quad \text{(kips/in.)} \quad (B-4)
\]

where \( E = 29,000 \text{ ksi} \); \( A_b \) = area of the diagonal member, and the length of the member is 30.48 ft. The resulting force, \( P \), in the diagonal member is

\[
    P = 79.287 A_b \frac{20}{30.48} (d - s \pm e)
\]
or \[ P = 52.026 A_b (d - s \pm e) \] (B-5)

where the values of \( s \) and \( e \) at the ends of the diagonal are provided in Eq's. (B-3). The component, \( P_h \), of \( P \) in the direction of the girder is

\[ P_h = 34.14 A_b (d - s \pm e) \] (B-6)

The forces \( F_1 \) through \( F_5 \) acting on the fractured girder as shown in Fig. B-3(a) can now be calculated in terms of \( A_b \) and \( d \). For example, at joint A, \( F_1 = F_{11} + F_{12} \) where \( F_{11} \) refers to Bay 5, \( F_{12} \) to Bay 4, and

\[ F_{11} = 34.14 A_b (d + 0.0616) \] (B-7)
\[ F_{12} = 34.14 A_b (d - 0.1930) \] (B-8)

and \( F_1 = 34.14 A_b (2d - 0.1314) \) (B-9)

Similarly at the other joints, B, C, D, and E,

\[ F_{21} = 34.14 A_b (d - 0.1314 - 0.0616) \] (B-10)
\[ F_{22} = 34.14 A_b (d - 0.1314 - 0.2730) \] (B-11)

and \( F_2 = 34.14 A_b (2d - 0.5974) \) (B-12)

\[ F_{31} = 34.14 A_b (d - 0.2115 - 0.1930) \] (B-13)
\[ F_{32} = 34.14 \, A_b \, (d - 0.2115 - 0.3208) \quad (B-14) \]

and
\[ F_3 = 34.14 \, A_b \, (2d - 0.9368) \quad (B-15) \]

\[ F_{41} = 34.14 \, A_b \, (d - 0.2592 - 0.2730) \quad (B-16) \]
\[ F_{42} = 34.14 \, A_b \, (d - 0.2592 - 0.3373) \quad (B-17) \]

and
\[ F_4 = 34.14 \, A_b \, (2d - 1.1287) \quad (B-18) \]

\[ F_{51} = 34.14 \, A_b \, (d - 0.2757 - 0.3208) \quad (B-19) \]

and
\[ F_5 = 34.14 \, A_b \, (d - 0.5965) \quad (B-20) \]

The total horizontal force, \( F \), can be now obtained by adding Eq's. (B-9), (B-12), (B-15), (B-18) and (B-20).

\[ F = 34.14 \, A_b \, (9\,d - 3.3908) \text{ kips} \quad (B-21) \]

Since the permissible midspan deflection of the fractured girder, \( v \), is assumed as the span length over 300,

\[ v = \frac{180 \times 12}{300} = 7.2 \text{ in.} \quad (B-22) \]

and
\[ d = \frac{v \times 9.75}{90} = 0.78 \text{ in.} \quad (B-23) \]

The highest tensile stress of the diagonal member in Bay 5, \( S_t \), can be determined by using Eq's. (B-5) and (B-7).
\[ S_t = 52.026 \left( d + 0.0616 \right) \text{ ksi} \]  
\text{(B-24)}

By substituting \( d \) in Eq (B-23) into Eq. (B-24),

\[ S_t = 43.79 \text{ ksi} \]  
\text{(B-25)}

Since the highest tensile stress is less than the yield stress, \( F_y = 50 \text{ ksi} \), the limited horizontal displacement of the bottom flange at the fracture, \( d \) in Eq. (B-23), governs in determining the diagonal member forces. The required area for all the diagonal members, \( A_b \), is found from Eq's. (B-1) and (B-21) by substituting \( d = 0.78 \text{ in.} \)

\[ A_b = 14.35 \text{ in}^2 \]  
\text{(B-26)}

The member forces \( P_{11} \) in Bay 5, \( P_{12} \) and \( P_{21} \) in Bay 4, \( P_{22} \) and \( P_{31} \) in Bay 3, \( P_{32} \) and \( P_{41} \) in Bay 2, and \( P_{42} \) and \( P_{51} \) in Bay 1 are now calculated from Eq's. (B-7), (B-8), (B-10), (B-11), (B-13), (B-14), (B-16), (B-17) and (B-19) respectively, as follows:

\[
\begin{align*}
P_{11} & = + 628 \text{ kips} \\
P_{12} & = - 438 \text{ kips} \\
P_{22} & = - 280 \text{ kips} \\
P_{32} & = - 185 \text{ kips} \\
P_{21} & = + 438 \text{ kips} \\
P_{31} & = + 280 \text{ kips} \\
P_{41} & = + 185 \text{ kips}
\end{align*}
\]  
\text{(B-27)}
\[ P_{42} = -137 \text{ kips} \quad P_{51} = +137 \text{ kips} \]

These forces are shown in Fig. B-5(a). The assumed distribution of forces is also shown in parentheses in Fig. B-5(a). The sum of all these forces over a half span must be constant and equal to 1,777.8 x 30.48 / 20 = 2,709 kips.

The distribution of member forces based on the calculated values is somewhat different from the assumed distribution. Rather than the assumed distribution of 5 : 4 : 3 : 2 : 1, the resulting distribution is 6.73 : 4.69 : 3 : 1.98 : 1.47, where the distribution ratio is calculated based on the fixed value 3 of Bay 3.

To obtain more accurate results, the member forces are recalculated using a distribution of 5.87 : 4.35 : 3 : 1.99 : 1.24 which is an average of the above two distributions. Since the center value 3 of Bay 3 is fixed, minimized error between the distribution factors of Bay 1 and 5 will result in a precise ratio in least number of trials. The revised \( S_t \), \( A_b \), and member forces \( P \)'s corresponding to this distribution are as follows:

\[ S_t = 43.60 \text{ ksi} \quad (B-29) \]
\[ A_b = 14.02 \text{ in}^2 \quad (B-30) \]
\[ P_{11} = +611 \text{ kips} \ ]
The resulted distribution of member forces becomes 6.45 : 4.57 : 3 : 2.02 : 1.49. Since the resulted distribution is still somewhat different from the assumed distribution, the member forces are recalculated using a distribution of 6.16 : 4.46 : 3 : 2.01 : 1.37 which is an average of the latest assumed and resulted distributions. These values are shown in parentheses in Fig. B-5(b). The revised $S_t$, $A_b$, and member forces $P$'s corresponding to this distribution are as follows:

\[
S_t = 43.56 \text{ ksi} \quad \text{(B-32)}
\]

\[
A_b = 14.01 \text{ in}^2 \quad \text{(B-33)}
\]

\[
P_{11} = +610 \text{ kips}
\]

\[
P_{12} = -434 \text{ kips} \quad P_{21} = +434 \text{ kips} \quad \text{(B-34)}
\]

\[
P_{22} = -285 \text{ kips} \quad P_{31} = +285 \text{ kips}
\]

\[
P_{32} = -191 \text{ kips} \quad P_{41} = +191 \text{ kips}
\]

\[
P_{42} = -139 \text{ kips} \quad P_{51} = +139 \text{ kips}
\]

The resulting distribution is 6.42 : 4.57 : 3 : 2.01

B-12
1.46. Further refinement is not necessary because design loads for the bottom lateral, \( P_{11} \) and \( P_{12} \), have nearly converged. During this latest calculation, the maximum compressive stress occurs in the diagonal member in Bay 4. The compressive stress is limited by the AASHTO critical stress, 0.85 \( F_{cr} \).

\[
0.85 F_{cr} = 52.026 (d - 0.1842) \quad \text{(B-35)}
\]

Since this analysis is based on the limited horizontal deflection \( d = 0.78 \) in. of Eq. (B-23),

\[
F_{cr} = 36.50 \text{ ksi} \quad \text{(B-36)}
\]

Article 10.54.1.1 of the AASHTO Bridge Specification (1) defines \( F_{cr} \) in the inelastic and elastic ranges of buckling as follows:

\[
\left( \frac{KL}{r} \right)^2 = \frac{4t^2E}{F_Y^2} \left( F_Y - F_{cr} \right) \quad \text{(B-37)}
\]

when \( KL/r \) is less than 107 for \( F_Y = 50 \) ksi

and

\[
\left( \frac{KL}{r} \right)^2 = \frac{t^2E}{F_{cr}} \quad \text{(B-38)}
\]

when \( KL/r \) is greater than 107 for \( F_Y = 50 \) ksi

By substituting Eq (B-36) into Eq's. (B-37) and (B-38), \( KL/r \)
is determined in the inelastic range of buckling using Eq. (B-37).

\[ KL/r = 78.7 \]  \hspace{1cm} (B-39)

The revised member forces corresponding to this distribution are also shown in Fig. B-5(b), together with the computed values of \( A_b \), \( S_t \), and KL/r.

A suitable steel shape can now be selected for the diagonal members of the bottom lateral bracing system based on the conditions shown in Fig. B-5(b). The diagonal members are laterally supported in both directions at the ends and the compression member is assumed to be braced only in the horizontally plane at mid-length, so that \( K = 0.75 \), \( L_x = 366 \) in. and \( L_y = 183 \) in.

Try WT 13.5x51

\[ A = 15.0 \text{ in}^2 \hspace{1cm} r_x = 4.14 \text{ in.} \hspace{1cm} r_y = 2.15 \text{ in.} \]

\[ KL / r_x = 66.3 \hspace{1cm} KL / r_y = 63.8 \]

\[ S_t = 42.15 \text{ ksi} \hspace{1cm} d = 0.753 \text{ in.} \hspace{1cm} v = 6.95 \text{ in.} \]

The WT 13.5x51 meets all the design requirements. The
member forces corresponding to the area provided by this member are shown without parentheses in Fig. B-5(c).

**DESIGN OF CROSS BRACING SYSTEM**

In design for redundancy the cross bracing system acting with the floor beams transfers the bottom lateral bracing forces into the deck. All ten end and interior cross bracing designs are assumed to be identical. The design is based on the maximum force condition which occurs for the cross bracing between joints A and F. The design forces at joints A and F are shown in Fig. B-6(a). These forces are the components of the forces in the diagonal members of the lateral bracing system at the joints. The configuration of K bracing is also shown in Fig. B-6(a).

Figure B-6(b) shows the member forces for the K bracing and the sections selected for the members based on $F_y = 50$ ksi. The WT 12x81 spans between joints A and F, and is assumed not to be braced between joints A and F. The WT 9x38 is used for both sloping members.

An alternative design for Reversed-K bracing is shown in Fig. B-6(c). The WT 12x52 is designed to resist compressive force 477 kips. The WT 9x38 is also used for both sloping members. A comparison of Figs. (b) and (c) indicates that the Reversed-K bracing is more efficient.
DESIGN OF DRAG STRUT

Since the webs of main girders and stringers are too flexible to carry the horizontal reaction, 670 kips, which is carried from the cross bracing as shown in Fig. B-6(a), three drag struts are introduced between the top flange of floor beams and the reinforced concrete deck as shown in Fig B-7(a). Figure B-7(b) shows a detail of the drag strut between two stringers. The cross section of the drag strut is identical to the stringer. The required length to develop sufficient shear stress is specified in Article 10.48.1.1(e) of the AASHTO Bridge Specification (1).

\[ L_{ds} = \frac{670}{3} \times 0.55 \times 50 \times 0.36 = 22.56 \text{ in.} \] (B-40)

where the stress due to bending moment is small enough to be negligible.

The drag strut is welded or bolted to the floor beam, and is connected to the reinforced concrete deck through shear studs. Since the flange thickness of the drag strut is 0.605", 3/4" dia. shear studs are used in a row above web. Based on Article 10.38.5.1.2 of the AASHTO Bridge Specification (1) the required number of studs is:

\[ N_1 = \frac{P}{\phi \text{ Su}} = \frac{670}{3} \times 0.85 \times 21.775 = 12.07 \] (B-41)

where,
\[ S_u = 0.4 \, d^2 \left[ f'_c \, E_c \right]^{0.5} = 21.775 \text{kips} \]

Figure B-7(b) shows the arrangement of 13 welded studs, 3/4" dia., spacing 6 in. on the 6'-8" long W18x45 drag strut. This drag strut transfers the horizontal force to the reinforced concrete deck through those welded struts.
Table B-1 Section Properties of Main Girders

<table>
<thead>
<tr>
<th>Location</th>
<th>h, inch</th>
<th>Ag, in.²</th>
<th>Ig, in.⁴</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bay 1</td>
<td>119.0</td>
<td>93.19</td>
<td>204,598</td>
</tr>
<tr>
<td>Bay 2</td>
<td>120.0</td>
<td>120.19</td>
<td>300,634</td>
</tr>
<tr>
<td>Bay 3</td>
<td>121.5</td>
<td>154.69</td>
<td>426,393</td>
</tr>
<tr>
<td>Bays 4 and 5</td>
<td>122.0</td>
<td>166.19</td>
<td>469,009</td>
</tr>
</tbody>
</table>
Figure B-1  Plan and Elevation Views of Steel Superstructure
Figure B-2 Typical Cross Section
Figure B-3 Analytical Model for Calculating Bottom Lateral Forces

(a) Free Body of Fractured Girder

(b) Bottom Lateral System
Figure B-4  Displacements of Girder and Bottom Lateral Bracing System after Fracture
(a) \( Ab = 14.35 \text{ in}^2 \quad St = 43.79 \text{ ksi} \)

(b) \( Ab = 14.01 \text{ in}^2 \quad St = 43.46 \text{ ksi} \quad KL/\gamma = 78.7 \)

(c) WT 13.5 x 51 \( Ab = 15.0 \text{ in}^2 \quad St = 42.15 \text{ ksi} \quad KL/\gamma = 66.3 \)

Figure B-5 Design of Bottom Lateral System
(a) Design Forces

(b) Member Forces and Sections Designed

(c) Alternative Design

Figure B-6 Design of Cross Bracing System
a) Design Forces of Drag Strut

b) Connection Detail

Figure B-7 Design of Drag Strut and Connection
INTRODUCTION

Figures C-1 and C-2 show schematic views of the steel structure for the simple span two-girder noncomposite right bridge selected for design for redundancy. The bridge is contained in the 1982 USDOT/FHWA Standard Plans For Highway Bridges (35)*. The bridge selected has a 180-ft. simple span, a 44 ft. wide roadway and is designed for the 1965 AASHTO HS20-44 lane loading. The girders are fabricated from A441 steel. All other steel is A36. A 7-1/2 in. noncomposite reinforced concrete deck carries three lanes of traffic.

Figure C-1(a) shows a plan view of the steel superstructure. Stringers and girders are spaced at 7'-6". Floor beams and cross frames are spaced at 20-ft. K-type bottom lateral bracing is also shown in the figure. Figure C-1(b) shows an elevation view of one of the two girders. The girder is symmetrical about midspan.

Figure C-2 shows the half cross-sections near midspan and near the bearings. At the floor beam locations the diaphragms consist of cross frames as shown in the figure.

* References are presented on page 45 of this report.
Connection beams are used between stringers at the end diaphragm locations. Bottom lateral bracing frames into the gusset plates shown in the figure, which are located at the level of floor beam bottom flange.

DESIGN FOR REDUNDANCY

Redundancy is provided by an alternate load path consisting of bottom lateral bracing, cross bracing, floor beams, drag struts and deck (cross bending). In this example the bottom lateral and cross bracing systems plus the drag struts will be designed for a fracture scenario consisting of near full depth midspan fracture of one of the two girders. Design of the deck for the resulting cross bracing forces has not been fully developed and will not be included in this example.

The existing steel superstructure shown in Figures C-1 and C-2 contains cross frames and top lateral bracing at the floor beam bottom flange level. For this design example the bottom lateral bracing will be lowered to the level of the bottom flanges of the girders. Cross bracings rather than cross frames will be assumed at all interior locations.

The cross bracings transferring bottom lateral bracing forces into concrete deck will produce downward forces on the unfractured girder and upward forces on the other
fractured girder. When bottom lateral forces due to midspan fracture are calculated in this design example, the upward forces on the fractured girder are neglected conservatively.

Load levels appropriate for use in designing the alternate load path(s) are required. These should be less than the AASHTO design load levels. On a load factor basis this bridge is subjected to loading consisting of 1.1 D plus three lanes of 1.3 (L+I) where the truck loading is HS20 lane loading with assumed 30 % impact. An impact factor of 30 % is assumed to account for the effect of increased deck deflections with traffic maintaining normal highway speeds.

The permissible overall deflection of the deck under factored live loads is assumed as span length divided by 300 to establish serviceability of the bridge deck.

In the following example, an iterative approach is used to obtain the forces in the bottom lateral members. This approach will yield answers as accurate as deemed necessary by the bridge design. Although hand computations are used, the procedure can be readily adopted to computer solution.
DESIGN OF BOTTOM LATERAL BRACING SYSTEM

Figure C-3 shows the analytical model used to calculate the bottom lateral bracing forces. A free body of the fractured girder is shown in Fig. C-3(a). The girder is shown with a midspan fracture. The factored uniform dead load, including 22 pounds per square foot for future wearing surface on the roadway slab, is calculated as 4.73 k/ft. The factored live loads, obtained by positioning three lanes of trucks towards the fractured girder for maximum midspan bending moment, are calculated as 2.06 k/ft uniform load and 57.8 kips concentrated load above the girder fracture.

The bottom flange is subjected to horizontal forces $F_1$ through $F_5$ as shown in the figure which are imposed by the lateral bracing system after the fracture occurs. The vertical reactions are also shown as 640 kips at each support. The total force $F_1 + F_2 + F_3 + F_4$ acting at the level of the bottom flange on half the span can be calculated on the condition of zero bending moment at mid span. The total applied moments are divided by resisting moment arm, 11.25 ft., which is the girder depth as shown in the figure.

$$ F = \frac{1}{11.25} \left[ \frac{1}{8} (4.73 + 2.06)(180)^2 + \frac{1}{4} (57.8)(180) \right] $$

or $F = 2675.6$ kips

(C-1)
The arrangement of the bottom lateral system is shown in Fig. C-3(b). The spacing center-to-center of the girder webs is 37.5 ft. The forces $F_1$, $F_3$, and $F_4$ are each developed by only one diagonal member framing into the girder flange. The force $F_2$ is developed by two diagonal members. It is assumed that all the diagonal members of the bottom lateral system are identical, having equal cross section areas, $A_b$, and equal properties.

Figure C-4 shows the displacements of the fractured girder and the bottom lateral system after fracture. In Fig. C-4(a) the fractured girder is shown in its deflected position. If the horizontal displacement of the bottom flange at the fracture is assumed as $d$ as shown in the figure then the vertical displacement at the fracture, $v$, is $90d/11.25 = 8.0d$. In Fig. C-4(b) the displacements of the bottom lateral system are shown. The four diagonals in Bays 4, 5, and 6 are in tension. In Bays 2, and 3 one diagonal is in tension, the other in compression as shown in the figure. In Bay 1 the diagonal near the fractured girder is in tension, the other in compression as shown. The dashed lines show the original position of the bottom lateral and cross bracing members. The solid lines show the positions after fracture. The horizontal displacement of joint A is $d_1=d$. Since no girder shortening occurs between joint A and the fracture, $s_1=0$. The displacement,
d, is entirely controlled by the level of stress, selected by the designer, in the tension diagonals in Bay 5. The horizontal displacement $d_2$ of joint B is less than $d$ by the amount of girder shortening, $s_2$, between A and B. Similarly, the horizontal displacement of joints D, and E is $d-s_3$, and $d-s_4$ respectively. The horizontal displacements of joints F, G, I, and J on the unfractured girder due to girder elongation are $e_1$, $e_2$, $e_3$, and $e_4$ respectively.

It is assumed in the design of the lateral bracing system that no relative displacement occurs between the girders. That is, the cross bracing horizontal members connecting the bottom flanges of the girders are assumed to be axially rigid.

Since the forces in the diagonal member can be obtained from a trial and error approach, the initial force distribution ratio of the diagonal members in Bay 5 through Bay 1 is assumed as 5:4:3:3:2 as shown in Fig. C-4(c). The resulting forces acting on both girders are determined with using $F = 2675.6$ kips.

Consider, for example, the segment of the fractured girder from A to B. If $N$ is the sum of the forces applied at joints B through E, then the displacement, $u$, of joint B relative to joint A is,

$$u = \frac{NL}{A_E} + \frac{NL}{EI_g} \frac{h^2}{4}$$  \hspace{1cm} (C-2)

\[C-6\]
where \( E = 29,000 \text{ ksi} \) (Young’s modulus)

\[ L = 20 \text{ ft.} \] (bay length)

\[ h = \text{overall girder depth (See Table C-1)} \]

\[ A_g = \text{area of girder (See Table C-1)} \]

\[ I_g = \text{moment of inertia of girder (See Table C-1)} \]

Equation (C-2) can be used to calculate the relative displacement between any two joints on the fractured and unfractured girders except, of course, in Bay 5 of the fractured girder.

Equation (C-2) is used to calculate the following values of the displacements \( s \) and \( e \) shown in Fig. C.2.2(b).

\[
\begin{align*}
    s_1 &= 0 & e_1 &= -0.0064 \text{ in.} \\
    s_2 &= -0.1540 \text{ in.} & e_2 &= +0.0321 \text{ in.} \\
    s_3 &= -0.3245 \text{ in.} & e_3 &= +0.2025 \text{ in.} \quad (C-3) \\
    s_4 &= -0.3821 \text{ in.} & e_4 &= +0.2602 \text{ in.} \\
\end{align*}
\]

If \( k \) is the axial stiffness of a diagonal member, then

\[
k = \frac{29,000 A_b}{54.83 \times 12} = 44.076 A_b \quad (\text{kips/in.}) \quad (C-4)
\]

where \( E = 29,000 \text{ ksi} \); \( A_b = \text{area of the diagonal member} \), and the length of the member is 54.83 ft. The resulting force, \( P \), in the diagonal member is
\[ P = 44.076 \ A_b \frac{40}{54.83} (d - s \pm e) \]

or

\[ P = 32.155 \ A_b \ (d - s \pm e) \quad \text{(C-5)} \]

where the values of \( s \) and \( e \) at the ends of the diagonal are provided in Eq's. (C-3). The horizontal component, \( P_h \), of \( P \) in the direction of the girder is

\[ P_h = 23.458 \ A_b \ (d - s \pm e) \quad \text{(C-6)} \]

The forces \( F_1 \) through \( F_4 \) acting on the fractured girder as shown in Fig. C-3(a) can now be calculated in terms of \( A_b \) and \( d \). For example, at joint A, \( F_1 = F_{11} \) where \( F_{11} \) refers to Bay 5, and

\[ F_{11} = 23.458 \ A_b \ (d + 0.0321) \quad \text{(C-7)} \]

and

\[ F_1 = 23.458 \ A_b \ (d + 0.0321) \quad \text{(C-8)} \]

Similarly at the other joints, B, D, and E,

\[ F_{21} = 23.458 \ A_b \ (d - 0.1540 - 0.0064) \quad \text{(C-9)} \]

\[ F_{22} = 23.458 \ A_b \ (d - 0.1540 - 0.2025) \quad \text{(C-10)} \]

and

\[ F_2 = 23.458 \ A_b \ (2d - 0.5169) \quad \text{(C-11)} \]

\[ F_{31} = 23.458 \ A_b \ (d - 0.3245 - 0.0321) \quad \text{(C-12)} \]
and \[ F_3 = 23.458 A_b (d - 0.3566) \] (C-13)

\[ F_{41} = 23.458 A_b (d - 0.3821 - 0.2602) \] (C-14)

and \[ F_4 = 23.458 A_b (d - 0.6423) \] (C-15)

The total horizontal force, \( F \), can be now obtained by adding Eq's. (C-8), (C-11), (C-13), and (C-15).

\[ F = 23.458 A_b (5d - 1.4837) \text{ kips} \] (C-16)

Since the permissible midspan deflection of the fractured girder, \( v \), is assumed as the span length divided by 300,

\[ v = 180 \times 12 / 300 = 7.2 \text{ in.} \] (C-17)

and \[ d = v \times 11.25 / 90 = 0.90 \text{ in.} \] (C-18)

The highest tensile stress of the diagonal member in Bay 5, \( S_t \), can be determined by using Eq's. (C-5) and (C-7).

\[ S_t = 32.155 (d + 0.0321) \text{ ksi} \] (C-19)

By substituting \( d \) in Eq (C-18) into Eq. (C-19),

\[ S_t = 29.97 \text{ ksi} \] (C-20)
Since the highest tensile stress is less than the yield stress, \( F_y = 50 \) ksi, the limited horizontal displacement of the bottom flange at the fracture, \( d \) in Eq. (C-18), governs in determining the diagonal member forces.

The required area for all the diagonal members, \( A_b \), is found from Eq's. (C-1) and (C-16) by substituting \( d = 0.90 \) in.

\[
A_b = 37.81 \text{ in}^2 \tag{C-21}
\]

The member forces \( P_{11} \) in Bay 5, \( P_{21} \) in Bay 4, \( P_{22} \) in Bay 3, \( P_{31} \) in Bay 2, and \( P_{51} \) in Bay 1 are now calculated from Eq's. (C-7), (C-9), (C-10), (C-12), and (C-14) respectively, as follows:

\[
\begin{align*}
P_{11} & = +1133 \text{ kips} \\
P_{21} & = +899 \text{ kips} \\
P_{22} & = -661 \text{ kips} \\
P_{31} & = +661 \text{ kips} \\
P_{41} & = +313 \text{ kips}
\end{align*}
\tag{C-22}
\]

These forces are shown in Fig. C-5(a). The assumed distribution of forces is also shown in parentheses in Fig. C-5(a). The sum of all these forces over a half span must be constant and equal to \( 2,675.6 \times 54.83 / 40 = 3,668 \) kips.
The distribution of member forces based on the calculated values is somewhat different from the assumed distribution. Rather than the assumed distribution of 5 : 4 : 3 : 3 : 2, the resulting distribution is 5.14 : 4.08 : 3 : 3 : 1.42, where the distribution ratio is calculated based on the fixed value 3 of Bay 3.

To obtain more accurate results, the member forces are recalculated using a distribution of 5.07 : 4.04 : 3 : 3 : 1.71 which is an average of the above two distributions. Since the center value 3 of Bay 3 is fixed, minimized error between the distribution factors of Bay 1 and 5 will result in a precise ratio in least number of trials. The revised $S_t$, $A_b$, and member forces $P's$ corresponding to this distribution are as follows:

\[
\begin{align*}
S_t &= 29.75 \text{ ksi} \\
A_b &= 37.01 \text{ in}^2 \\
P_{11} &= +1101 \text{ kips} \\
P_{21} &= +879 \text{ kips} \\
P_{22} &= -667 \text{ kips} \\
P_{31} &= +667 \text{ kips} \\
P_{41} &= +355 \text{ kips}
\end{align*}
\]
The resulting distribution of member forces becomes 4.95 : 3.95 : 3 : 1.60. Further refinement is not necessary because design loads for the bottom laterals, \( P_{11} \) and \( P_{12} \), have nearly converged. During this latest calculation, the maximum compressive stress occurs in the diagonal member in Bay 3. The compressive stress is limited by the AASHTO critical stress, 0.85 \( F_{cr} \).

\[
0.85 F_{cr} = 32.155 (d - 0.3399) \tag{C-26}
\]

Since this analysis is based on the limited horizontal deflection, \( d = 0.90 \) in. of Eq. (C-18),

\[
F_{cr} = 21.19 \text{ ksi} \tag{C-27}
\]

Article 10.54.1.1 of the AASHTO Bridge Specification defines \( F_{cr} \) in the inelastic and elastic ranges of buckling as follows:

\[
\frac{KL}{r} \left( \frac{1}{F} \right)^2 = \frac{4v^2E}{F_Y} \left( F_Y - F_{cr} \right) \tag{C-28}
\]

when \( KL/r \) is less than 107 for \( F_Y = 50 \) ksi

and

\[
\frac{KL}{r} \left( \frac{1}{F} \right)^2 = \frac{v^2E}{F_{cr}} \tag{C-29}
\]

when \( KL/r \) is greater than 107 for \( F_Y = 50 \) ksi
By substituting Eq (C-27) into Eq's. (C-28) and (C-29), KL/r is determined in the elastic range of buckling using Eq. (C-29).

\[ \frac{KL}{r} = 116.2 \]  

(C-30)

The revised member forces corresponding to this distribution are also shown in Fig. C-5(b), together with the computed values of \( A_b \), \( S_t \), and KL/r.

A suitable steel shape can now be selected for the diagonal members of the bottom lateral bracing system based on the conditions shown in Fig. C-5(b). The diagonal members are laterally supported in both directions at the ends and the compression member is assumed to be braced only in the horizontally plane at mid-length, so that \( K = 0.75 \), \( L_x = 658 \) in. and \( L_y = 329 \) in.

Try WT 18x130

\[ A = 38.2 \text{ in}^2 \quad r_x = 5.26 \text{ in.} \quad r_y = 3.78 \text{ in.} \]

\[ \frac{KL}{r_x} = 93.82 \quad \frac{KL}{r_y} = 65.28 \]

\[ S_t = 28.93 \text{ ksi} \quad d = 0.877 \text{ in.} \quad v = 7.2 \text{ in.} \]
The WT 18x130 meets all the design requirements. The member forces corresponding to the area provided by this member are shown without parentheses in Fig. C-5(c).

**DESIGN OF CROSS BRACING SYSTEM**

In design for redundancy the cross bracing system acting with the floor beams transfers the bottom lateral bracing forces into the deck. All ten end and interior cross bracing designs are assumed to be identical. The design is based on the maximum force condition which occurs for the cross bracing between joints B and G. The design forces at joints B and G are shown in Fig. C-6(a). These forces are the components of the forces in the diagonal members of the lateral bracing system at the joints. The configuration of K bracing is also shown in Fig. C-6(a).

Figure C-6(b) shows the member forces for the K bracing and the sections selected for the members based on $F_y = 50$ ksi. The WT 15x95.5 spans between joints B and G, and is assumed not to be braced between joints B and G. The WT 12x52 is used for both sloping members.

An alternative design of cross bracing without cross frame horizontal is shown in Fig. C-6(c). The sloping members are enlarged to WT 18x115.

**DESIGN OF DRAG STRUT**
Since the webs of main girders and stringers are too flexible to carry the horizontal reaction, 1065 kips, which is carried from the cross bracing as shown in Fig. C-6(a), five drag struts are introduced between the top flange of floor beams and the reinforced concrete deck as shown in Fig C-7(a). Figure C-7(b) shows a detail of the drag strut between two stringers. The cross section of the drag strut is identical to the stringer. The required length to develop sufficient shear stress is specified in Article 10.48.1.1(e) of the AASHTO Bridge Specification (1).

\[
L_{ds} = \frac{1065 / 5}{0.55 \times 50 \times 0.40} = 19.36 \text{ in.} \tag{C-31}
\]

where the stress due to bending moment is small enough to be negligible.

The drag strut is welded or bolted to the floor beam, and is connected to the reinforced concrete deck through shear studs. Since the flange thickness of the drag strut is 0.65", 3/4" dia. shear studs are used in a row above web line. Based on Article 10.38.5.1.2 of the AASHTO Bridge Specification (1) the required number of studs is:

\[
N_1 = \frac{P}{\phi Su} = \frac{1065 / 5}{0.85 \times 21.775} = 11.51 \tag{C-32}
\]

where,
\[ S_u = 0.4 \, d^2 \left[ f'_c \, E_c \right]^{0.5} = 21.775 \text{ kips} \]

Figure C-7(b) shows the arrangement of 13 welded studs, 3/4" dia., spacing 6 in. on the 6'-6" long W21x55 drag strut. This drag strut transfers the horizontal force to the reinforced concrete deck through those welded struts.
Table C-1 Section Properties of Main Girders

<table>
<thead>
<tr>
<th>Location</th>
<th>h, inch</th>
<th>Ag, in.²</th>
<th>Ig, in.⁴</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bay 1</td>
<td>137.25</td>
<td>119.25</td>
<td>342,253</td>
</tr>
<tr>
<td>Bay 2</td>
<td>138.00</td>
<td>157.50</td>
<td>521,758</td>
</tr>
<tr>
<td>Bay 3</td>
<td>139.50</td>
<td>216.00</td>
<td>801,923</td>
</tr>
<tr>
<td>Bays 4 and 5</td>
<td>140.00</td>
<td>232.50</td>
<td>882,484</td>
</tr>
</tbody>
</table>
Figure C-1 Plan and Elevation Views of Steel Superstructure
Half Section near Midspan  

Half Section near Bearings

Figure C-2 Typical Cross Sections
Figure C-3 Analytical Model for Calculating Bottom Lateral Forces

(a) Free Body of Fractured Girder

(b) Bottom Lateral System
Figure C-4  Displacements of Girder and Bottom Lateral Bracing System after Fracture
Figure C-5  Design of Bottom Lateral System

C-22
a) Design Forces

b) Member Forces and Sections Designed

c) Alternative Design

Figure C-6  Design of Cross Bracing System
a) Design Forces of Drag Strut

b) Connection Detail

Figure C-7 Design of Drag Strut and Connection
INTRODUCTION

Figures D-1 and D-2 show schematic views of the steel structure for the simple span four-girder composite right bridge selected for design for redundancy. The bridge is contained in the 1982 USDOT/FHWA Standard Plans For Highway Bridges (35)*. The bridge selected has a 180-ft. simple span, a 28 ft. wide roadway and is designed for the 1977 AASHTO HS20-44 lane loading. The girders are fabricated from A572 steel. All other steel is A36. A 8-in. composite reinforced concrete deck carries two lanes of traffic.

Figure D-1(a) shows a plan view of the steel superstructure. Girders are spaced at 8 ft. Cross bracings are spaced at 20'-6". W-type bottom lateral bracing is also shown in the figure. Figure D-1(b) shows an elevation view of one of the four girders. The girder is symmetrical about midspan. As a composite girder, the top flange is smaller than the bottom one.

Figure D-2 shows the half cross-sections near midspan and near the bearings. The diaphragms consist of X-type

*References are presented on page 44 of this report.
cross bracings as shown in the figure. Connection beams are used between top of main girders at the end diaphragm locations. Bottom lateral bracing frames into the T-gusset as shown in the figure.

DESIGN FOR REDUNDANCY

Redundancy is provided by an alternate load path consisting of bottom lateral bracing, cross bracing, shear connection and deck (cross bending). In this example the bottom lateral and cross bracing systems, shear connection, and reinforced concrete deck will be designed for a fracture scenarios consisting of near full depth midspan fracture of one of the four girders.

The existing steel superstructure shown in Figures D-1 and D-2 contains bottom lateral bracing at somewhat upper level than the girder bottom flange. For this design example the bottom lateral bracing will be lowered to the level of the bottom flanges of the girders. The W-type bottom lateral bracing will be modified as a V-type one.

The cross bracings transfering bottom lateral bracing forces into concrete deck will produce downward forces on an unfractured girder and upward forces on the other fractured girder. When bottom lateral forces due to midspan fracture are calculated in this design example, the upward forces on the fractured girder are neglected.
conservatively.

Load levels appropriate for use in designing the alternate load path(s) are required. These should be less than the AASHTO design load levels. On a load factor basis this bridge is subjected to loading consisting of 1.1 D plus two lanes of 1.3(L+I) where the truck loading is HS 20 lane loading with assumed 30% impact. An impact factor of 30% is assumed to account for the effect of increased deck deflections with traffic maintaining normal highway speeds.

The permissible overall deflection of the deck under factored live loads is assumed as span length divided by 300 to establish serviceability of the bridge deck.

In the following example, an iterative approach is used to obtain the forces in the bottom lateral members. This approach will yield answers as accurate as deemed necessary by the bridge design. Although hand computations are used, the procedure can be readily adopted to computer solution.

**DESIGN OF BOTTOM LATERAL BRACING SYSTEM**

Figure D-3 shows the analytical model used to calculate the bottom lateral bracing forces. A free body of the fractured girder is shown in Fig. D-3(a). The girder is shown with a midspan fracture. The factored uniform dead load, including 25 pounds per square foot for
future wearing surface on the roadway slab, is calculated as 1.78 k/ft approximately for each interior and exterior girders. The factored live loads, based on Article 3.23.2 of the AASHTO Bridge Specification, are calculated as 0.79 k/ft uniform load and 22.14 kips concentrated load above the girder fracture for the interior girders and conservatively for the exterior girders.

Figure D-3(b) shows a modified bottom lateral system which is capable of efficiently transmitting the load induced by fracture to the cross bracing system. The three diagonals in a bay as shown in Fig. D-1(a) are modified as the two diagonals. It is assumed that all the diagonal members of the bottom lateral system are identical, having equal cross section areas, \( A_b \), and equal properties. The fracture "A" or "B" is assumed to occur near midspan of the interior girder or the exterior girder, respectively.

Those two fracture types are compared together with resisting forces as shown in Fig. D-4. The load induced by the fracture "A" as shown in Fig. D-4(a) is carried by 6 diagonals as the forces \( F_1 \) through \( F_7 \), while the load induced by the fracture "B" as shown in Fig. D-4(b) is carried by 8 diagonals as the forces \( F_1 \) through \( F_8 \). Since the critical case for bottom lateral system results from the fracture "A", all the following designs are based on the fracture "A" as shown in Fig D-4(a).
The bottom flange is subjected to horizontal forces \( F_1 \) through \( F_4 \) as shown in the figure which are imposed by the lateral bracing system after the fracture occurs. The total force \( F_1 + F_2 + F_3 + F_4 \) acting at the level of the bottom flange on half the span can be calculated on the condition of zero bending moment at mid span.

The total applied moments are divided by resisting moment arm, 10.52 ft., which is the girder depth as shown in the figure.

\[
F = \frac{1}{10.52} \left[ \frac{1}{8} (1.78 + 0.79)(180)^2 + \frac{1}{4} (22.14)(180) \right]
\]

or \( F = 1084.1 \) kips \( \text{(D-1)} \)

Figure D-5 shows the displacements of the fractured girder and the bottom lateral system after fracture. In Fig. D-5(a) the fractured girder is shown in its deflected position. If the horizontal displacement of the bottom flange at the fracture is assumed as \( d \) as shown in the figure then the vertical displacement at the fracture, \( v \), is \( 90d/10.52 = 8.555d \).

In Fig. D-5(b) the displacements of the bottom lateral system are shown. The two diagonals in Bay 4 have no force to satisfy equilibrium condition. In Bays 1 through 3 one diagonal is in tension, the other in compression as shown. The dashed lines show the original
position of the bottom lateral and cross bracing members. The solid lines show the positions after fracture. The horizontal displacement of joint A is \( d_1 = d \). Since no girder shortening occurs between joint A and the fracture, \( s_1 = 0 \). The horizontal displacement \( d_2 \) of joint B is less than \( d \) by the amount of girder shortening, \( s_2 \), between A and B. Similarly, the horizontal displacement of joints C, and D is \( d - s_3 \), and \( d - s_4 \) respectively. The horizontal displacements of joints G, I, and K on the unfractured girder due to girder elongation are \( e_1 \), \( e_2 \), and \( e_3 \) respectively under the assumption of no horizontal displacement at joint E.

It is assumed in the design of the lateral bracing system that no relative displacement occurs between the joints connecting cross bracing and girder. That is, the cross bracing horizontal members connecting the bottom flanges of the girders are assumed to be axially rigid. Consequently, only the joints E, G, I, and K have transverse displacement to satisfy equilibrium condition that results in identical compressive and tensile forces in a bay.

Since the forces in the diagonal member can be obtained from a trial and error approach, the initial force distribution ratio of the diagonal members in Bay 3 through Bay 1 is assumed as 10:3:1 as shown in Fig. D-5(c). The resulting forces acting on both girders are determined with
using $F = 1084.1$ kips.

Consider, for example, the segment of the fractured girder from A to B. If $N$ is the sum of the forces applied at joints B, C and D, then the displacement, $u$, of joint B relative to joint A is,

$$u = \frac{NL}{AgE} + \frac{NL}{EIg} \frac{h^2}{4} \quad (D-2)$$

where $E = 29,000$ ksi (Young's modulus)

$L = 22.5$ ft. (bay length)

$h = $ overall girder depth (See Table D-1)

$Ag = $ area of girder (See Table D-1)

$Ig = $ moment of inertia of girder (See Table D-1)

In calculating the section properties of the main girders as shown in Table D-1, the composite sections are conservatively assumed as steel sections which have the same top flange section as the bottom flange.

Equation (D-2) can be used to calculate the relative displacement between any two joints on the fractured and unfractured girders except, of course, in Bay 4 of the fractured girder. Equation (D-2) is used to calculate the following values of the displacements $s$ and $e$ shown in Fig. D-5(b).
\( s_1 = 0 \quad e_1 = +0.2482 \text{ \text{in.}} \)
\( s_2 = -0.1732 \text{ \text{in.}} \quad e_2 = +0.3313 \text{ \text{in.}} \) \hspace{1cm} (D-3)
\( s_3 = -0.2252 \text{ \text{in.}} \quad e_3 = +0.3520 \text{ \text{in.}} \)
\( s_4 = -0.2356 \text{ \text{in.}} \)

If \( k \) is the axial stiffness of a diagonal member, then

\[
k = \frac{29,000 \ A_b}{13.80 \times 12} = 175.06 \ A_b \quad \text{(kips/in.)} \quad (D-4)
\]

where \( E = 29,000 \text{ \text{ksi}} \); \( A_b = \text{area of the diagonal member} \), and the length of the member is 13.80 ft. The resulting force, \( P \), in the diagonal member is

\[
P = 175.06 \ A_b \frac{11.25 \ (d - s_i + d - s_{i+1})}{13.80 \times 2} \quad -e_i
\]

or

\[
P = 142.72 \ A_b \ (d - 0.5 \ s_i - 0.5 \ s_{i+1} - e_i) \quad (D-5)
\]

where the values of \( s \) and \( e \) at the ends of the diagonal are provided in Eq's. (D-3). The horizontal component, \( P_h \), of \( P \) in the direction of the girder is

\[
P_h = 116.34 \ A_b \ (d - 0.5 \ s_i - 0.5 \ s_{i+1} - e_i) \quad (D-6)
\]

The forces \( F_1 \) through \( F_4 \) acting on the fractured girder as shown in Fig. D-4(a) can now be calculated in terms of \( A_b \)
and d. For example, at joint B, \( F_2 = F_{12} + F_{21} \) where \( F_{12} \) refers to Bay 3, \( F_{21} \) to Bay 2, and

\[
F_{11} = F_{12} = 116.34 A_b (d - 0.3348) \quad \text{(D-7)}
\]
\[
F_{21} = F_{22} = 116.34 A_b (d - 0.5305) \quad \text{(D-8)}
\]
\[
F_{31} = F_{32} = 116.34 A_b (d - 0.5824) \quad \text{(D-9)}
\]

and

\[
F_1 = 116.34 A_b (d - 0.3348) \quad \text{(D-10)}
\]
\[
F_2 = 116.34 A_b (2d - 0.8653) \quad \text{(D-11)}
\]
\[
F_3 = 116.34 A_b (2d - 1.1129) \quad \text{(D-12)}
\]
\[
F_4 = 116.34 A_b (d - 0.5824) \quad \text{(D-13)}
\]

The total horizontal force, \( F \), can be now obtained by adding Eq's. (D-10) through (D-13).

\[
F = 116.34 A_b (6d - 2.8954) \text{ kips} \quad \text{(D-14)}
\]

Since the permissible midspan deflection of the fractured girder, \( v \), is assumed as the span length divided by 300,

\[
v = 180 \times 12 / 300 = 7.2 \text{ in.} \quad \text{(D-15)}
\]

and

\[
d = v \times 10.52 / 90 = 0.842 \text{ in.} \quad \text{(D-16)}
\]

The highest compressive stress, which is identical to the
highest tensile stress, of the diagonal member in Bay 3, $S_t$, can be determined by using Eq's. (D-5) and (D-7).

$$S_t = 142.72 \left( d - 0.3348 \right) \text{ksi} \quad (D-17)$$

By substituting $d$ in Eq (D-16) into Eq. (D-17),

$$S_t = 72.39 \text{ksi} \quad (D-18)$$

Since the highest compressive stress is larger than the yield stress, $F_y = 50 \text{ ksi}$, buckling stress of the compressive member in Bay 3 governs in determining the diagonal member forces. The compressive stress is limited by the AASHTO critical stress, $0.85 F_{cr}$.

$$0.85 F_{cr} = 142.72 \left( d - 0.3348 \right) \text{ksi} \quad (D-19)$$

If $0.85 F_{cr}$ is assumed to be 38 ksi,

$$d = 0.6011 \text{ in.} \quad (D-20)$$

The required area for all the diagonal members, $A_b$, is calculated from Eq's. (D-1) and (D-14) by substituting $d = 0.6011 \text{ in.}$
The member forces $P_{11}$ and $P_{12}$ in Bay 3, $P_{21}$ and $P_{22}$ in Bay 2, and $P_{31}$ and $P_{32}$ in Bay 1 are now calculated from Eq's. (D-7), (D-8), and (D-9) respectively, as follows:

\[
\begin{align*}
P_{11} &= -498 \text{ kips} \\
P_{21} &= -132 \text{ kips} \\
P_{31} &= -35 \text{ kips} \\
P_{12} &= +498 \text{ kips} \\
P_{22} &= +132 \text{ kips} \\
P_{32} &= +35 \text{ kips}
\end{align*}
\] (D-22)

These forces are shown in Fig. D-6(a). The assumed distribution of forces is also shown in parentheses in Fig. D-6(a). The sum of all these forces over a half span must be constant and equal to $1,084.1 \times 13.80 / 11.25 = 1,330 \text{ kips}$.

The distribution of member forces based on the calculated values is somewhat different from the assumed distribution. Rather than the assumed distribution of 10 : 3 : 1, the resulting distribution is 11.31 : 3 : 0.80, where the distribution ratio is calculated based on the fixed value 3 of Bay 2.

To obtain more accurate results, the member forces are recalculated using a distribution of 10.65 : 3 : 0.90 which is an average of the above two distributions. Since the center value 3 of Bay 2 is fixed, minimized error
between the distribution factors of Bays 1 and 3 will result in a precise ratio in a least number of trials. The revised $d$, $A_b$, and member forces $P$'s corresponding to this distribution are as follows:

\[
\begin{align*}
  d &= 0.60 \text{ in.} \\
  A_b &= 12.33 \text{ in}^2
\end{align*}
\]

\[
\begin{align*}
  P_{11} &= -469 \text{ kips} \\
  P_{21} &= -139 \text{ kips} \\
  P_{31} &= +57 \text{ kips} \\
  P_{12} &= +469 \text{ kips} \\
  P_{22} &= +139 \text{ kips} \\
  P_{32} &= +57 \text{ kips}
\end{align*}
\]

The resulted distribution of member forces becomes $10.12 : 3 : 1.23$. Since the resulted distribution is still somewhat different from the assumed distribution, the member forces are recalculated using a distribution of $10.39 : 3 : 1.07$ which is an average of the latest assumed and resulted distributions. These values are shown in parentheses in Fig. D-6(b). The revised $d$, $A_b$, and member forces $P$'s corresponding to this distribution are as follows:

\[
\begin{align*}
  d &= 0.601 \text{ in.} \\
  A_b &= 13.01 \text{ in}^2
\end{align*}
\]
\[ P_{11} = -496 \text{ kips} \quad P_{12} = +496 \text{ kips} \]
\[ P_{21} = -134 \text{ kips} \quad P_{22} = +134 \text{ kips} \]  \hspace{1cm} (D-28)
\[ P_{31} = -36 \text{ kips} \quad P_{32} = +36 \text{ kips} \]

The resulting distribution is 11.1 : 3 : 0.8. Further refinement is not necessary, since the maximum compressive force \( P_{11} \), which governs this design for redundancy, has nearly converged. The revised member forces corresponding to this distribution are shown in Fig. D-6(b), together with the computed \( d \), and \( A_p \).

Article 10.54.1.1 of the AASHTO Bridge Specification defines \( F_{cr} \) in the inelastic and elastic ranges of buckling as follows:

\[
\left( \frac{KL}{r} \right)^2 = \frac{4\pi^2E}{F_y^2} \left( F_y - F_{cr} \right) \]  \hspace{1cm} (D-29)

when \( KL/r \) is less than 107 for \( F_y = 50 \text{ ksi} \)

and

\[
\left( \frac{KL}{r} \right)^2 = \frac{\pi^2E}{F_{cr}} \]  \hspace{1cm} (D-30)

when \( KL/r \) is greater than 107 for \( F_y = 50 \text{ ksi} \)

A suitable steel shape can now be selected for the diagonal members of the bottom lateral bracing system based on the conditions shown in Fig. D-6(b). The diagonal members are laterally supported in both directions at the ends, so that \( K = 0.75, L_x = L_y = 165.6 \text{ in.} \)
Try WT 9x48.5

\[ A_b = 14.3 \text{ in}^2 \quad r_x = 2.56 \text{ in.} \quad r_y = 2.65 \text{ in.} \]

\[ \frac{KL}{r_x} = 48.52 \]

\[ F_{cr} = 44.86 \text{ ksi} \]

Since the assumed 0.85 \( F_{cr} \), 38 ksi, is less than the obtained value, 38.13 ksi, the WT 9x48.5 meets all the design requirements. The member forces corresponding to the area provided by this member are shown without parentheses in Fig. D-6(c).

**DESIGN OF CROSS BRACING SYSTEM**

In design for redundancy the cross bracing system acting with the main girders transfers the bottom lateral bracing forces into the deck. All nine end and interior cross bracing designs are assumed to be identical. The design is based on the maximum force condition which occurs for the cross bracing between joints A and F. The design force at joint A is shown in Fig. D-7(a). This force is the component of the force in the diagonal member of the lateral bracing system at the joint. The configuration of X bracing is also shown in Fig. D-7(a).
Figure D-7(b) shows the member forces for the X bracing and the sections designed for the members based on $F_y = 50$ ksi. The WT 5x15 spans between joints A and F. The L 8x6x7/16 is used for all diagonals. The fracture "B" case discussed in Fig. D-3(b) is separately studied to check whether or not a greater force on the cross bracing occurs, and results in a lesser force.

The corresponding horizontal reactions at the main girder top flanges, 50.3 kips at the exterior ones and 100.7 kips the interior ones, will be transferred to the deck through welded studs on the main girders.

**DESIGN OF SHEAR CONNECTION**

The original FHWA drawing (35) is designed with four 3/4" diameter welded studs on each girder top flange to be a composite section as shown in Fig. D-2. The required number of 3/4" diameter welded studs on each girder are computed with the maximum shear force 100.7 kips based on Article 10.38.5.1.2 of the AASHTO Bridge Specification (1).

$$N_1 = \frac{P}{\phi S_u} = \frac{100.7}{0.85 \times 29.52} = 4.01 \text{ EA} \quad (D-31)$$

where,

$$S_u = 0.4 d^2 \left[ f_c E_c \right]^{0.5} = 29.52 \text{ kips}$$

D-15
The required amounts of shear connectors are already arranged above the cross frames for the composite section of the main girders as shown in Fig. D-2.

**DESIGN OF REINFORCED CONCRETE DECK**

The design loads of reinforced concrete deck is shown in Fig. D-8(a). Instead of the resulting forces from the welded stud group on each girder, the resulting horizontal components of the bottom lateral bracing system at the points of each cross bracing are indicated for convenience. A self-equilibrium shall exist among the applied forces without any reaction. The maximum bending moment at the midspan computed from the part of applied loads at the right cross bracings will be identical to the maximum moment computed from the loads at the left cross bracing.

Figure D-8(b) shows the corresponding moment diagram. The maximum bending moment is 8,640 k-ft at the midspan. The maximum shear force is 302 kips also near the midspan. Figure D-8(c) shows the reinforcement design of the deck. The longitudinal reinforcements, # 4 bars spacing 10 in. at top and 5 in. at bottom of the deck, are already designed in the FHWA drawing. The reinforcements are redesigned as # 4 bars spacing 5 in. at both top and bottom of the deck.

Considering equivalent concentrated reinforcements, $A_{st} = 6.2 \text{ in}^2$, at the point 39 in. from the edge of deck,
the bending moment capacity can be calculated as follows:

\[ a = \frac{A_s f_y}{0.85 f'_c b} = \frac{6.2 \times 60}{0.85 \times 4.5 \times 8} = 12.16 \text{ in.} \]

\[ \varphi M_n = \varphi \left[ A_s f_y \left( d - \frac{a}{2} \right) \right] \]
\[ = 0.9 \left[ 6.2 \times 60 \times \left( 327 - \frac{12.16}{2} \right) \right] \]
\[ = 107,444 \text{ k-in.} = 8,954 \text{ k-ft} \]

which is larger than the required bending moment 8,640 k-ft. Also the top flanges of the composite girders would contribute to carry the deck bending moments.
Table D-1 Section properties of main girders

<table>
<thead>
<tr>
<th>Location</th>
<th>h, inch</th>
<th>Ag, in.²</th>
<th>Ig, in.⁴</th>
</tr>
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<tbody>
<tr>
<td>Bays 1 and 2</td>
<td>125.94</td>
<td>96.25</td>
<td>215,200</td>
</tr>
<tr>
<td>Bay 3</td>
<td>Between the properties of Bays 2 and 4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bay 4</td>
<td>126.25</td>
<td>107.50</td>
<td>260,700</td>
</tr>
</tbody>
</table>
Figure D-1  Plan and Elevation Views of Steel Superstructure
Figure D-2  Typical Cross Sections
Figure D-3 Analytical Model for Calculating Bottom Lateral Forces
Figure D-4 Comparison of Resisting Forces for Two Fracture Conditions

a) Fracture at Interior Girder

b) Fracture at Exterior Girder
(a) Displacement of fractured Girder

(b) Displacement of Bottom Lateral Bracing System

(c) Assumed Force Distribution

Figure D-5  Displacements of Girder and Bottom Lateral Bracing System after Fracture
(a) \( A_b = 13.11 \text{ in}^2 \quad 0.85 \sigma_{cr} = 38 \text{ ksi} \quad d = 0.60 \text{ in} \)

(b) \( A_b = 13.01 \text{ in}^2 \quad 0.85 \sigma_{cr} = 38 \text{ ksi} \quad d = 0.60 \text{ in} \)

(c) \( W T 9 \times 48.5 \quad A_b = 14.3 \text{ in}^2 \quad \sigma_t = 36.45 \text{ ksi} \quad K_{fr} = 48.5 \)

Figure D-6  Design of Bottom Lateral System
(a) Design Forces

(b) Member Forces and Sections Designed

Figure D-7 Design of Cross Bracing System
(a) Design Loads of Concrete Deck

(b) Bending Moment Diagram

(c) Reinforcement Design

Figure D-8 Design of Reinforced Concrete Deck