NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Report 424

Improved Design Specifications for Horizontally Curved Steel Girder Highway Bridges

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Staff
Transportation Research Board

This report contains the findings of a study undertaken to develop revised guide specifications for horizontally curved highway bridges on the basis of current practice and technology. The proposed specifications resulting from this work have been provided to AASHTO. The contents of this report will be of immediate use to bridge designers interested in understanding the background of the specification development.

Horizontally curved-girder bridges are a structurally efficient and aesthetically pleasing method of supporting curved roadways. Nevertheless, because of the curvature, torsional effects are added to bending effects. Also, there is far more interaction among curved girders than among straight girders in the bridge cross section. These considerations make design and construction of horizontally curved-girder bridges considerably more challenging than design and construction of straight bridges.

The AASHTO *Guide Specifications for Horizontally Curved Highway Bridges* was issued in 1980 and is based on research performed in the early 1970s. The *Guide Specifications* include design provisions for curved steel I-girder and composite box-girder bridges. Design and construction experience with the *Guide Specifications* has demonstrated some major deficiencies, which can result in overly conservative designs. Fabrication and erection provisions are also missing, and there is insufficient guidance on analytical procedures for both preliminary and final design.

The objective of NCHRP Project 12-38 was to develop revised guide specifications for horizontally curved highway bridges, on the basis of current practice and technology, that can be recommended to AASHTO for consideration for adoption. The revised guide specifications are applicable to the load factor design, fabrication, and erection of steel I- and box-girder bridges. A full commentary, which clarifies the application of the specification provisions, is included.

This report provides a full description of the research performed to develop the improved specifications. The report is particularly noteworthy because it identifies major departures from the current specifications and explains the reasoning behind these departures. Thus, the report should prove to be a valuable resource document for design engineers interested in understanding the theoretical and historical basis of the specification provisions.

CONTENTS

AUTHOR ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 12-38 by the Highway Research Center, Auburn University, and by Bridge Software Development International, Ltd., of Coopersburg, PA. Auburn University was the contractor for this study. The work undertaken at Bridge Software Development International, Ltd., was under a subcontract with Auburn University.

Chai H. Yoo, Huff Professor of Civil Engineering, Auburn University and Dann H. Hall, principal, Bridge Software Development International, Ltd., were co-principal investigators.

IMPROVED DESIGN SPECIFICATIONS FOR HORIZONTALLY CURVED STEEL GIRDER HIGHWAY BRIDGES

SUMMARY

The primary focus of NCHRP Project 12-38 was to develop revised *Guide Specifications for Horizontally Curved Highway Bridges*, based on current practice and technology, that could be recommended to AASHTO for possible adoption. The revised *Guide Specifications* were to be applicable to the design, fabrication, and erection of horizontally curved steel I- and box-girder bridges.

The research team was to complete ten tasks in this project. Chapter 1 provides a detailed description of each task. Six appendixes were also prepared and submitted: Appendix A: I-Girder Curvature Study; Appendix B: Curved-Girder Design and Construction, Current Practice; Appendix C: A Unified Approach for Designing and Constructing Horizontally Curved Girder Bridges, Proposed AASHTO Specifications (Highlights of Major Changes); Appendix D: Recommended Specifications for Steel Curved-Girder Bridges and Commentary; Appendix E: Design Example, Horizontally Curved Steel I-Girder Bridge; and Appendix F: Design Example, Horizontally Curved Steel Box-Girder Bridge.

This report summarizes the development of the revised *Guide Specifications* and the appendixes.

CHAPTER 1

INTRODUCTION

A. PROBLEM STATEMENT

Modern highway construction often requires bridges with horizontally curved alignments. According to a 1991 survey (*1*), curved bridges represent 20 to 25 percent of the market for new steel bridge construction each year and this trend is likely to increase in the future. Straight steel or concrete girders may be placed on the chords of the roadway curve between supports or field splices to approximate the required curvature. This approach, however, has several disadvantages. In general, bridges constructed in this fashion are not aesthetically pleasing and the advantages of structural continuity are also lost. Therefore, most curved bridge alignments today use continuous horizontally curved steel girders.

Curved bridges using curved girders are usually characterized by simpler and more uniform construction details, because the girder spacing and the concrete deck overhangs are generally constant along the length of the structure. Curved girders allow for the use of longer spans, which reduces substructure costs and the required number of expansion joints and bearing details. Deterioration around joints and bearings due to leakage is a significant maintenance issue associated with these details. Curved girders also more easily satisfy the demand placed on highway structures by predetermined roadway alignments and tight geometric restrictions presented by today's increasing right-of-way restrictions.

Curved girders are fabricated by cutting the flanges to fit the curve, either by heat curving or cold bending. The introduction of heat curving, along with associated fabrication aids, helped to increase the use of horizontally curved girders. Two types of cross sections are used for horizontally curved steel girders: I-girders (both noncomposite and composite) or composite box-shaped tub girders.

Despite all the inherent advantages of curved girders, their behavior is generally more complex than that of straight girders. Curved girders are subject to torsion as well as vertical bending. The interaction between adjacent girders is also more important in curved bridges than in straight bridges without skew. To assist engineers in dealing with the complexities of curved girders, several practical approximate analysis methods, such as the V-load method and the M/R method, were developed in the 1960s. More recently, refined three-dimensional finite element analysis methods and grillage analysis methods have been incorporated into commercially available computer programs for curved bridge analysis and design.

Construction of horizontally curved steel bridges is also generally more complex than construction of straight-girder bridges of similar span. Curved-girder bridges, once completed, have generally performed as intended. Successful completion of a curved structure, however, often requires that critical phases of construction be engineered so that the final structure has the proper geometry and dead load stresses assumed in the design. Curved girders are often erected based solely on experience, which has sometimes led to unanticipated problems.

The *Guide Specifications for Horizontally Curved Highway Bridges* (*2*), hereafter referred to as the Guide Spec, were originally published in 1980 by AASHTO. Since its publication, the Guide Spec has provided guidance in designing horizontally curved girder bridges to engineers in the United States and Canada. There are, however, some significant deficiencies with the Guide Spec. The Research Problem Statement for NCHRP Project 12-38 states, "More than 12 years of design and construction experience with the *Guide Specifications* has demonstrated some major deficiencies. In its present form, the Guide is disjointed and difficult to follow. The commentary is incomplete and lacks needed detailed explanation regarding the development of many of the provisions. Many provisions are overly conservative, and many are difficult to implement. Other provisions lend themselves to misinterpretation, which may lead to uneconomical structures or structures with factors of safety less than intended. Other critical deficiencies include lack of fabrication and erection provisions, insufficient guidance on analytical procedures for both preliminary and final design, lack of guidance on which curvature effects can be safely ignored in bridges with large radii of curvature, and design criteria apparently inconsistent with that for current straight-girder bridges."

In addition to deficiencies in the design criteria, the RFP also mentions a lack of provisions in the Guide Spec related directly to the construction of curved bridges. Designers, builders, and owners have expressed disappointment with the Guide Spec because it does not specifically address several important construction and erection issues. Several states have modified the existing provisions because of expensive claims and several lawsuits involving construction of curved bridges. Other states have limited the application of curved girders to specified spans or methods of erection. Fewer problems exist with respect to the fabrication of horizontally curved bridges as a result of the introduction of more modern fabrication equipment and techniques and the fact that fabricators have gradually gained more experience with curved girders.

There have been a few attempts to improve on some of the deficiencies in the Guide Spec; however, most of these attempts have been primarily editorial in nature. Without a major overhaul, the primary deficiencies would have most likely remained intact.

B. RESEARCH SCOPE

The scope of NCHRP Project 12-38 was, therefore, to develop completely revised *Guide Specifications for Horizontally Curved Highway Bridges,* hereafter referred to as the Recommended Specifications, and an accompanying commentary that could be recommended for adoption to AASHTO. The specifications were to be based on current practice and technology, were to be applicable to the design, fabrication, and erection of steel I- and box-girder bridges, and were to use load factor design methods. In addition, design examples were to be developed illustrating the application of the more significant provisions in the Recommended Specifications.

As detailed in the research proposal and in the work plan for the project, the Project Panel defined 10 specific tasks to accomplish the project objectives. These tasks are summarized below:

- **Task 1.** Collect and review relevant domestic and foreign literature, research findings, and current practices related to the design, fabrication, and erection of horizontally curved steel I- and box-girder bridges. Much of this material will be compiled under an existing FHWA project and will be available at the start of this project.
- **Task 2.** Survey owners, designers, fabricators, and erectors of curved steel bridges are to identify successful practices and problems that have occurred in design, fabrication, erection, and service.
- **Task 3.** Evaluate the information assembled in Tasks 1 and 2, and prepare a comprehensive list of, and rationale for, revisions that can be recommended based on current practice and technology. Discuss any remaining limitations, which cannot be adequately addressed without further analytical and experimental work, noting specific research needs to better model horizontally curved girder bridge behavior and to define improved limit-state design criteria.
- **Task 4.** Prepare a detailed outline for the Recommended Specifications, which includes a discussion of the intent and contents of each section.
- **Task 5.** Document the findings of Tasks 1 through 4 in an interim report.
- **Task 6.** Prepare a first draft of the specifications and commentary in a format suitable for consideration by the AASHTO Highway Subcommittee on Bridges and Structures.
- **Task 7.** Demonstrate the use of the proposed specifications with practical design examples. The examples shall be a fully explanatory application of the proposed code, demonstrating load generation, computation of moments, shears, torsional effects, reactions, and application of all provisions at critical sections throughout the bridge. Example problems shall cover I- and box-girder multispan continuous bridges with radial supports.
- **Task 8.** Submit the draft specifications and example problems.
- **Task 9.** Prepare and submit a second draft of the specifications and example problems revised, as necessary, based on the NCHRP and AASHTO review comments.
- **Task 10.** Prepare and submit a final report containing: the research findings; the proposed Guide Specifications, including commentary and design examples, revised in accordance with the review comments generated after Task 9; and recommendations for further research.

An addendum to the project proposal clarified the main roles of each investigator and the scope and range of the design examples to be prepared.

CHAPTER 2
PRIOR WORK AND ONGOING RESEARCH

A. CURT PROJECT

Although there was a considerable amount of fragmented information available dealing with the structural analysis of horizontally curved members in the United States, an organized research effort on curved bridges had not been undertaken until the Pennsylvania DOT initiated an analytical study at Carnegie Mellon University in 1969 (*3*). The study was aimed at collecting and generating a broad spectrum of information on curved-girder highway bridge design and construction. At that time, horizontally curved girder bridges were designed in the United States without the aid of any official specifications related specifically to curved-bridge design. In 1967, one of the first curved bridges was built in Springfield, Massachusetts. This sharply curved bridge has two closed box girders in the cross section. The bridge was inspected recently and was found to be in excellent condition after nearly 30 years of service. By present standards, however, the steel weight (approximately 90 lb/ft^2) is considered excessive.

Because there were no officially accepted design specifications for curved structures at the time, a comprehensive research project was initiated in October 1969. The CURT (Consortium of University Research Team) project was conducted under the auspices of FHWA with financial support from 25 participating state highway departments.

The consortium included researchers from Carnegie Mellon University, the University of Pennsylvania, the University of Rhode Island, and Syracuse University. The charge of the consortium was to (1) review all the published information on the subject of curved bridges; (2) conduct analytical and experimental studies to confirm or supplement the published information and assimilate information from related research programs sponsored by state highway departments; (3) develop simplified analysis and design methods along with supporting computer programs and design aids; and (4) correlate the developed analysis and design methods with analytical and experimental data. Design recommendations (*3*) and *Tentative Design Specifications for Horizontally Curved Highway Bridges* (*4*) in an allowable stress format were developed by the CURT team, which eventually formed the basis for the Guide Spec.

As part of the CURT project, Culver and his coworkers at Carnegie Mellon University studied the nominal bending strength, lateral stability, and local buckling of curved flanges and webs. Culver led the research on the stability of curved plate girders. McManus' Ph.D. dissertation (*5*) became the basis for the strength predictor equation (*6*) in the Guide Spec. Nasir (*7*) studied local buckling of curved plate elements, and Brogan (*8*) studied the bending behavior of cylindrical web panels. These researchers also performed tests on single curved box and I-girders and on a two-girder arrangement of doubly symmetric I-girders.

Although not members of the CURT research team, Heins at the University of Maryland (*9–11*) and Powell at the University of California, Berkeley (*12*), also made significant contributions that were included in the Guide Spec and its commentary.

B. GUIDE SPEC

Based primarily on the work from the CURT project, suggested tentative design criteria were assembled for both curved I- and box-girder bridges by the Task Committee on Curved Girders of the ASCE-AASHTO Committee on Flexural Members. The tentative criteria were adopted by AASHTO as a guide specification in 1976. At the time, the criteria in the curved girder guide specification were allowable stress design criteria only.

In 1975, the American Iron and Steel Institute initiated Project 190 to develop load factor design criteria for steel curved-girder bridges. The criteria were developed by Galambos and Stegmann at Washington University (*13,14*), based on the work of Culver and McManus (*6*) and Mozer and Culver (*15*) at Carnegie Mellon University on the stability of horizontally curved members. The tentative load factor design criteria for curved I- and box-girder bridges were adopted by AASHTO and incorporated in the Guide Specifications issued in 1979. The first printing of the Guide Specifications, containing both the allowable stress and load factor criteria together, was issued by AASHTO in 1980.

The allowable stress design criteria are contained in Part I of the Guide Spec and include provisions for curved steel, composite, and hybrid I-girder bridges and curved composite box-girder bridges. The I- and box-girder provisions are separated into two parts with a commentary for each part. A brief review of the current Guide Spec allowable stress design provisions (from Article 1.1, *General,* to Article 1.29, *Cross Frames, Diaphragms and Lateral Bracing*) is presented in Appendix B.

The load factor design criteria are contained in Part II of the Guide Spec and also include provisions for curved steel, composite, and hybrid I-girder bridges and curved composite box-girder bridges. The I- and box-girder provisions are combined, and one commentary on both the I- and box-girder provisions is presented at the end of Part II. A brief review of the current Guide Spec load factor design provisions (from Article 2.1, *General,* to Article 2.28, *Diaphragms within the Box*) is also presented in Appendix B.

C. JAPANESE TESTS

In Japan, Nakai et al. (*16–18*) performed tests on nine doubly symmetric M-series, I-shaped specimens. As in the Mozer-Culver single-specimen tests, Nakai's M-series tests employed significantly more torsional fixity than exists in typical curved bridges. The M-series tests were performed in almost pure negative bending with very rigid rotational restraints at the ends. Fukumoto (*19*) tested simple-span specimens with a single concentrated load applied through a gravity simulator mechanism at the center of the span on six AR-series, I-girder specimens. Appendix B provides additional details on these Japanese tests.

D. HANSHIN GUIDELINES

In addition to the Guide Spec, there is only one other known design specification for horizontally curved steel bridges worldwide and that specification is the *Guidelines for the Design of Horizontally Curved Girder Bridges (Draft)* (*20*). This specification was developed by the Hanshin Expressway Public Corporation in Japan (hereafter referred to as the Hanshin Guidelines). A translation of the Hanshin Guidelines was obtained in 1993 with the assistance of the FHWA.

The Hanshin Guidelines refer to the Japanese Road Association *Specifications for Highway Bridges* (*21*) for the basic requirements and primarily contain only the provisions that are directly influenced by the effects of curvature. The provisions in the guidelines are presented in an allowable stress design format, as are the Japanese straight-girder design provisions. Some of the provisions, however, apply the factor of safety to the ultimate strength of the element rather than the yield stress or buckling stress, as is typically done in an allowable stress design format. A detailed discussion of the research that resulted in the Hanshin Guidelines is given by Nakai and Yoo (*22*).

The Hanshin Guidelines do not distinguish between specification language and commentary. The guidelines, however, do suggest when to select either I-girders or box girders. Box girders are divided into three major cross-section types: single-cell mono-box with two webs, multicell mono-box with more than two webs, and multiple single-cell box. Webs are typically oriented vertically; inclined webs are discouraged. Aesthetics is discussed but is not specified.

The guidelines discuss framing with regard to economics. The use of straight interior girders with curved exterior girders is recommended. The use of top- and bottom-flange lateral bracing in certain bays is recommended for curved I-girder bridges with spans above approximately 80 ft. The need to keep the lateral bracing in or near the plane of the flange is discussed. When this is not practical, the provisions require that transfer forces from the bracing to the flanges be considered.

Effective deck width is discussed. Generally, more effective deck is used in Japanese composite bridges than is provided for in the current AASHTO specifications (*2,23*).

Analysis is discussed at some length, but specific analytical techniques are not designated. Strength provisions for the components are derived from inelastic finite element analyses and curve fitting.

An extended discussion of thermal movement is presented. The provisions require bearings to be oriented considering the effects of restraint and friction forces. The provisions differentiate between sharp and shallow curvatures. Sharp curves may require that bearings be oriented to permit transverse thermal movement.

Expansion joints are designed to be consistent with the movement permitted by the bearings. If a finger joint is used, the fingers may be lined up to be parallel to the girder rather than perpendicular to a joint. If the bearings permit the bridge to expand transversely, then the fingers should be oriented perpendicular to the skewed joint.

1. Stress Components

The resisting normal stresses due to load effects in a curved-girder bridge are divided into four basic components in the Hanshin Guidelines. The characteristics of these four stress components are shown in Figure 1. The first three components are due to vertical bending effects, and the fourth component is due to lateral bending caused by nonuniform torsion. Further discussions on the four stress components are presented in Appendix A.

2. Strength Equations

The I-girder strength predictor equation in the Hanshin Guidelines is a linear interaction equation relating the vertical bending stress and the lateral bending stress, as shown in Equation 1:

$$\frac{f_b}{F_b} + \frac{f_l}{F_l} \le 1.0$$

where

f_b = vertical bending stress,
f_l = lateral flange bending stress,
F_b = allowable vertical bending stress considering any potential reduction due to lateral torsional buckling, and
F_l = allowable lateral flange bending stress.

F_l is simply taken as the yield stress divided by a basic factor of safety, disregarding any concern for buckling. F_b is determined from the straight-girder bridge specifications.

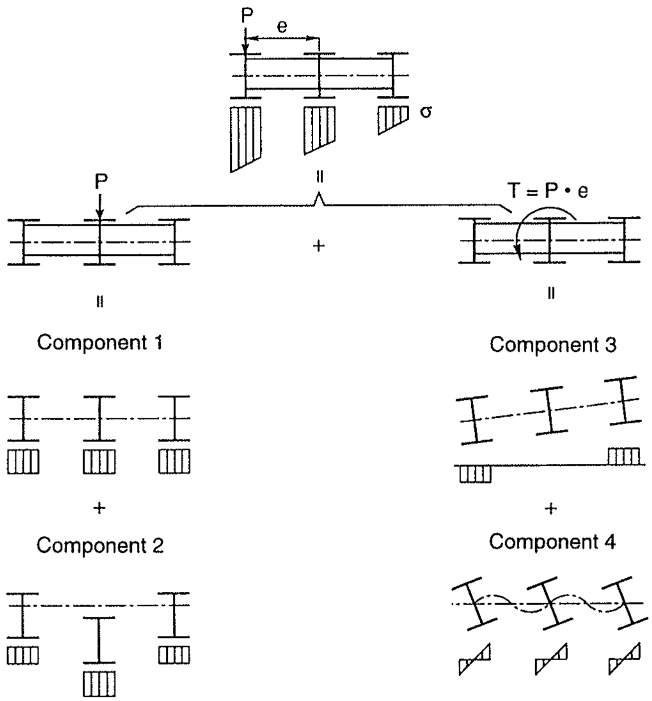

Figure 1. Four normal stress components.

The Hanshin Guidelines use a quadratic interaction relationship relating the normal stress and the shear stress for the box-girder strength predictor, as shown in Equation 2:

$$\left(\frac{f_b}{F_b}\right)^2 + \left(\frac{f_v}{F_v}\right)^2 \leq 1.2$$

where

- f_b = total normal stress including stresses due to the vertical bending, lateral bending, and cross-sectional distortion,
- f_v = shear stress, and
- F_v = allowable shear stress.

Although both (f_b/F_b) and (f_v/F_v) are not permitted to be greater than 1.0, the sum of the square of these ratios may be taken as high as 1.2. Both the I- and box-girder design examples presented in the Hanshin Guidelines do not use composite girders.

3. Stiffeners

The Hanshin Guidelines use the dimensionless parameter, Z, to modify the required rigidity of transverse stiffeners on curved web panels. The increase in rigidity is required to overcome the tendency of the web to bow. This same relationship is included in the Guide Spec and has also been included in the Recommended Specifications.

The Hanshin Guidelines use a similar but more complex parameter, β, to modify the required rigidity of longitudinal web stiffeners on curved girders. This parameter was developed from a regression analysis of data generated from three-dimensional, inelastic finite-element analyses. This factor has been further simplified and included in the Recommended Specifications.

E. FHWA CURVED STEEL BRIDGE RESEARCH PROJECT

At about the same time as the inception of NCHRP Project 12-38, FHWA initiated a comprehensive multiyear research program on horizontally curved steel I-girder highway bridges, "Curved Steel Bridge Research Project." The work in this ongoing project includes both analytical and laboratory studies. The basic objectives of this FHWA project are to conduct fundamental research on the behavior of curved I-girders in bending and shear and to directly address significant constructibility issues related to curved I-girder bridges.

The initial task in the FHWA project was to develop a synthesis of all the available published research work on horizontally curved girders. As detailed in Zureick et al. (*23*), approximately 750 references were collected and stored in a database. Approximately 540 of these 750 references were considered significant and were briefly reviewed by the researchers to document the most interesting highlights. When sorted according to subject area, the more significant references on curved I-girders reveals the following distribution: compression flange stability, 9; web buckling, 12; overall buckling, 16; ultimate strength, 10; and cross bracing, 3. A significant number of these references have an academic rather than a practical approach. Only one of the references presents any discussion related to cross bracing requirements for construction. Similar information on curved box girders is even harder to find.

A majority of the references deal with the static analysis of curved bridges. There are even fewer references available on loads, stability, design recommendations, fatigue details, fabrication, and construction. The numerous references on curved bridge analysis discuss both approximate methods (plane grid method, space frame method, and V-load method) and refined methods (finite element method, finite strip method, finite difference method, analytical solution to differential equations, and slope deflection method).

The synthesis by Zureick et al. (*24*), which is probably the most comprehensive literature survey available on horizontally curved girders, was reviewed at length by the research team before developing the Recommended Specifications.

To properly examine experimentally the fundamental strength behavior of horizontally curved girders, it is desirable to test individual components that are part of a complete bridge structure that resists vertical loads and torsion as a system. After significant studies of alternative approaches to the testing of horizontally curved I-girders, the FHWA project team decided that the testing of I-girder component specimens within a full-scale, curved three-girder test frame was the most rational and safest approach. All previous tests on horizontally curved component specimens in the United States and Japan, except for Culver's two-girder tests, did not accurately represent the behavior of the component within an actual bridge system. Previous tests were also not conducted on full-scale specimens. The use of full-scale specimens eliminates modeling concerns related to fabrication accuracy so that the results are considered to be more directly applicable to real bridges. Full-scale tests also eliminate the need for the addition of compensatory load to adjust for the lack of modeling similitude. The use of a test frame that looks and functions like a real bridge provides two other significant advantages: (1) more realistic boundary conditions are ensured for the component specimen testing, and (2) for economy, parts of the test frame can be reused in testing the bridge as a full-scale prototype.

In the first phase of the component test program, six noncomposite I-girder component specimens will be inserted one at a time in the center of the convex outermost girder of the test frame and will be subjected to nearly pure bending up to their ultimate capacity. In a second phase, the research team plans to subject a different set of specimens to various ratios of moment and shear. Finally, a concrete deck will be cast onto the frame and an attempt will be made to test the composite bridge system to failure.

Because the members of the NCHRP Project 12-38 research team also serve on the FHWA project research team, coordination between the two projects is maximized. As a result, certain identified gaps in the Recommended Specifications can potentially be addressed in the FHWA project. Also, the research team anticipates that portions of the Recommended Specifications developed in the NCHRP project will be updated based on data obtained from the various analytical and experimental studies being conducted in the FHWA project.

CHAPTER 3

REVIEW OF NCHRP PROJECT 12-38 WORK

A. SURVEYS OF OWNERS AND FABRICATORS

One of the 10 tasks identified by the Project Panel was to conduct a survey of owners, designers, fabricators, and erectors of curved steel bridges to identify successful practices and problems that have occurred in design, fabrication, erection, and service.

In response to this request, the research team prepared a questionnaire that was sent to selected state bridge engineers. Only a few responses with sketchy information were received. The requested information was either too time consuming to collect or simply not available.

As a result, it was decided to prepare a second questionnaire that would only be distributed to five leading fabricators of curved steel I- and box-girder bridges: (1) High Steel; (2) Trinity Industries; (3) PDM-Bridge; (4) Carolina Steel; and (5) Grand Junction. This survey asked for specific information on design, fabrication, and erection related to actual curved bridge projects that the individual fabricator had been involved with. Responses for approximately 20 bridges were received. Once again, the information was sketchy and there was little information provided regarding erection experiences or problems. Thus, as directed by the Senior Program Officer for NCHRP, the research team abandoned this task.

B. I-GIRDER CURVATURE STUDY

Although not included in the 10 original tasks, a need was identified to conduct a parametric study to determine appropriate rules regarding approximate analyses of curved I-girder bridges. The Guide Spec permits curvature effects on the vertical bending moments in I-girders to be ignored if the subtended angle of a span is less than that given by Table 1.4A in the Guide Spec. Lateral flange bending effects, however, must still be considered regardless of the curvature. The Hanshin Guidelines allow bridges with a subtended angle (calculated based on an effective span) of 5 deg or less to be exempt from analysis for curvature effects. The Hanshin Guidelines define the effective span as 0.6 times the average distance between bearings for interior spans and 0.8 times the average distance for end spans.

The background parametric study by Lavell and Laska (*25*) leading to the development of Table 1.4A in the Guide Spec could not be located and is believed to be lost. Also, AASHTO recently adopted new wheel load distribution factors for application to straight-girder analyses in a separate guide specification (*26*) and in the new load and resistance factor design (LRFD) specifications (*27*).

Thus, it was felt that another look at the parameters in Table 1.4A was justified to determine if the parameters were still applicable to modern designs and if additional conditions or limitations were needed.

The results of this study are reflected in Article 4.2.2 of the Recommended Specifications. A single limiting subtended angle, calculated based on an effective span in a fashion similar to the Hanshin Guidelines, is defined. In addition, the girders must be concentric and the bearings must not be skewed more than 10 deg from radial. For girders satisfying these limiting conditions, live load must be applied to each developed straight I-girder (with a span length equal to the individual girder arc length) by using a wheel-load distribution factor of S/5.5 when computing the vertical bending moments, shears, and deflections. A more detailed description of this study is in Appendix A.

C. CURVED-GIRDER DESIGN AND CONSTRUCTION, CURRENT PRACTICE

One of the earliest curved I-girder bridges was designed and constructed in Pittsburgh. The bridge is a three-span, two-girder structure with a radius of curvature less than 500 ft, carrying California Avenue over Ohio River Boulevard. An early formulation of the V-load method (*28*) was used in the analysis. The bridge, which was designed in 1967 by Richardson and Gordon (presently HDR Engineering, Inc.), is still carrying traffic with little remedial work required. Early curved steel bridge designs were mostly short, simple-span girders. The simple-span Hull's Falls Bridge in New York, carrying traffic over the Ausable River was designed with three I-girders on a 123-ft span and a radius of 477 ft. An I-girder bridge, carrying Route 65 over Pigeon Creek in Dalbarton, West Virginia, was designed with spans of 65 and 72 ft and a radius of 127 ft. A simple-span I-girder bridge in Minnesota was designed with a span of 104 ft and radius of 106 ft, with one abutment skewed approximately 45 deg.

The economic benefits of horizontally curved girders became even more apparent when continuous multispan curved structures became more common. Continuous curved-girder construction has evolved to the point where continuous curved steel structures over 1,900-ft long have been built with expansion joints only at the ends, such as the curved box-girder viaduct carrying Eighth Avenue in Denver, Colorado (built in 1983). Curved bridges have also been built with spans greater than 300 ft and radii less than 1,000 ft. One unit of the Little Falls Bridge in New York has spans of 234-329-365-234 ft with a radius of 750 ft. This bridge has seven I-girders in the cross section that vary in depth from 7 to 12 ft. One of the Shosone Dam Bridges in Glenwood Canyon, Colorado (completed in 1992), has spans of 219-274-216 ft and a radius of 955 ft. The bridge has four I-girders, varying in depth from 7 to 15 ft.

The cost to fabricate curved girders is greater than the cost to fabricate straight girders of similar size. The increased cost is usually related to the process of heat curving the girders or to the additional scrap that is generated when the flanges are cut-curved, and to the handling of the girders, both during fabrication and shipping. The erection of curved girders creates a separate set of problems. Often additional falsework or temporary shoring is needed. In the case of cantilever construction on a multispan alignment, insertion of the suspended span becomes problematic as the vertical camber and the rotation are always coupled.

The above information is only a sample of the current state of the art in curved-girder design and construction that has been assembled in Appendix B. This report summarizes the current practice related to design and construction and also makes comparisons of known relevant test data to the predicted strength based on several formulations, including those found in the Guide Spec and in the Hanshin Guidelines. The report also summarizes all the past research on several aspects of curved-girder behavior, reviews the background of the design criteria contained in the current Guide Spec, and discusses future research needs.

D. A UNIFIED APPROACH

Appendix C summarizes the overall philosophy that the research team adopted in the development of the Recommended Specifications and highlights the most significant changes that were introduced into the specifications. Appendix C provides the philosophy behind different specification formats and the development of the following basic specification components: loads, load application, structural analysis, load effects, and limit states (strength and stability, fatigue, serviceability and constructibility). The development of each of these components in the Recommended Specifications is also summarized. The appendix also contains brief discussions on the development of the provisions in Division II, Construction, and on the effect of design specifications on economics.

The Recommended Specifications are presented, for the most part, in a load factor design format. Several of the provisions, however, draw on other formats. For example, fatigue provisions are based on AASHTO's *LRFD Bridge Design Specifications* (*27*), as requested by the Project Panel. An attempt was made in the development of the Recommended Specifications to provide a unified methodology for the analysis and design of horizontally curved bridges based on sound engineering principles with minimal reliance on rules of thumb. The engineer is called on to broadly apply the principles when the provisions have been exhausted. The research team felt that this was the only way in which the wide variety of complex curved bridges demanded by owners can be designed and built, while still providing the desired uniformity of capacity and performance. In addition, a commentary is provided with the specifications to explain the reasoning behind the provisions and to help ensure their proper application. Both the specification provisions and the commentary are presented in a two-column format similar to the one in the AASHTO LRFD specifications.

E. RECOMMENDED SPECIFICATIONS: DIVISION I, DESIGN

The following discussion highlights most of the significant sections of the Recommended Specifications. Major deviations from the current specifications, where they occur, are identified and the reasons behind these deviations are presented. The numbered sections follow the current AASHTO Guide Spec. Sections are included only if there are changes.

1. General

1.1 Scope

The provisions apply to the design and construction of horizontally curved steel I-shaped and box-shaped girders with spans up to 300 ft and with radii greater than 100 ft. The provisions are limited to 300-ft spans because of the history of construction problems associated with curved bridges with spans exceeding 300 ft. Also, the approximations made in several of the formulas defining I-girder strength and impact have not been studied for radii smaller than 100 ft. Bridges beyond these limitations can be designed, however, by following the basic principles referred to in Article 1.2. The provisions state that any bridge superstructure containing a curved-girder segment shall be designed according to the provisions. This requirement is included because the effects of curvature usually extend beyond the curved segment, particularly the effects of lateral flange bending and of curvature on the reactions. This requirement also ensures that the bridge is designed according to the provisions of a single specification, which simplifies rating or evaluation at a later time. The provisions are to be used in conjunction with AASHTO's *Standard Specifications for Highway Bridges, Divisions I and II,* and AASHTO's *LRFD Bridge Design Specifications.* When there is a conflict between the provisions in the AASHTO standard and LRFD specifications and the provisions in the Recommended Specifications, the provisions in the Recommended Specifications should prevail.

A minimum of two I-girders or one box girder is required in the bridge cross section to provide static equilibrium. Both closed box and tub girders are permitted. The earliest steel box-girder bridges in the United States were closed sections, and the use of closed sections is not forbidden in the current Guide Spec. The use of closed box sections is still common in Europe and Asia. Special provisions are provided in Article 10.4.3 of the Recommended Specifications to design the top composite box flange. Composite box flanges are also permitted for use as bottom flanges of box sections. Since Occupational Safety and Health Administration (OSHA) requirements make it expensive to fabricate closed box sections, this type of section should probably only be considered for larger box girders. Although not explicitly prohibited, the provisions do not cover the design of multicell box girders because there is only a small amount of published research in the United States regarding the design of these members. Multicell box girders have been used in the United States in the past but are now uncommon. Although many different framing arrangements are permitted, girder-stringer-floor beam framing and a mix of box and I-girders in the cross section are not covered. These systems, however, are not explicitly forbidden.

Hybrid girders are not permitted in the Recommended Specifications because there has been limited research on curved hybrid girders. Girders with kinked alignments exhibit the same behavior as horizontally curved girders except that the noncollinearity of the flanges is concentrated at the kinks; therefore, kinked girders are covered by the provisions. Cast-in-place or precast concrete decks of constant or variable thickness are permitted. The decks may be longitudinally and/or transversely prestressed. The provisions do not apply to curved superstructures with steel orthotropic decks.

1.2 Principles

To ensure that the provisions can be applied on a uniform basis to a wide variety of complex structures, including structures beyond the scope of those covered by the provisions, the Recommended Specifications require the engineer to meet basic structural engineering principles. These principles, which include satisfying static equilibrium, ensuring adequate stability, and satisfying normal strength of materials assumptions, overrule the provisions if the provisions are inconsistent with the principles. Although not specifically stated in AASHTO, such a requirement is implied. When bridges are analyzed by considering the entire superstructure as required by both the Guide Spec and the Recommended Specifications, basic engineering principles should be considered to ensure that a proper analysis is made for each structure. For example, static equilibrium between the transverse deck and cross frame forces must be ensured at each cross frame line.

2. Limit States

2.2 Strength

Load combinations for the strength limit state are the same as those specified in Table 3.22.1A of the AASHTO Standard Specifications. The strength limit state considers stability and/or yielding of each structural element. If the strength of any element, including splices and connections, is exceeded, it is assumed that the bridge has exceeded its capacity.

2.3 Fatigue

At the request of the Project Panel, the provisions given in AASHTO LRFD Article 6.6.1 are applied at the fatigue limit state, except as modified in the Recommended Specifications. The LRFD specifications specify that a single HS20 truck (with a constant rear-axle spacing of 30 ft) be multiplied by a load factor of 0.75 when checking fatigue. An impact factor of 1.15 is applied to the live load.

Moses et al. (*29*) suggested that more work should be done regarding metal fatigue in longer span bridges. The report mentions that multiple presence of trucks becomes more important with longer spans and that the number of cycles of this loading is dramatically reduced from the number of cycles of a single truck. The effect of span continuity is not examined very extensively. A critical stress range may be caused by multiple presence trucks. Moses et al. did not attempt to quantify fatigue in long spans or continuous spans near inflection points, or the importance of multiple trucks in adjacent lanes.

Snyder et al. (*30*) addressed truck weights measured by weigh-in-motion and not stress spectra causing fatigue damage. Thus, short spans were selected to ensure a larger signal. Data were only recorded when a single truck was on the bridge so the truck weight could be ascertained.

Fatigue in long-span bridges is not addressed in Moses et al., Snyder et al., or the AASHTO LRFD fatigue provisions. The present AASHTO fatigue provisions require a check at 500,000 cycles of lane load with multiple lanes loaded for Case I roadways. This check is the same as checking a stress range due to two times one lane of live load for 100,000 cycles if the wheel load distribution factor for multiple lanes loaded is S/5.5 and for a single lane loaded is S/7.

A fatigue truck for the stress range due to two times HS20 truck or lane load in one traffic lane at 100,000 cycles is equivalent to the requirement in the present AASHTO Standard Specifications and would address longer spans and other situations where multiple presence is important. In addition to a fatigue check for a lower number of stress cycles, the loading could be used to check for the maximum tensile stress required by the AASHTO LRFD.

Moses et al. (*29*) addressed a factor for trucks side by side. The outside girders of most bridges experience the larger live-load stress range when cross frames are present. The effect of trucks in the outside lane is most critical for these girders. Trucks in the adjacent lane cause additional stress in the exterior girder, which can be referred to as the "bunching effect." Moses et al. computed the bunching effect to be approximately 1.15 times the single truck effect on a straight bridge. This value is computed assuming an 80 percent effect of the second truck for 10 percent of the stress range occurrences.

The 80 percent value for the second truck is too high for the exterior girders in most straight bridges, but it is not too high for sharply curved I-girder bridges. The researchers suggest a factor to account for the bunching effect on curved I-girder bridges to vary from 1.15 for bridges with a radius of 100 ft to 1.05 for tangent bridges. This value would be applied to I-girder bridges with large traffic volumes having two or more striped lanes in one direction.

2.4 Serviceability

Safety margins are not directly applicable to serviceability. Serviceability limits are, to a large extent, based on experience. To satisfy performance requirements for serviceability, the Recommended Specifications limit average flange stresses and local web buckling to control permanent set, vertical live-load deflections, and concrete deck cracking through proper placement of the longitudinal tensile reinforcing steel.

2.4.2 Deflection. As in the current AASHTO Standard Specifications, permanent deflections under overload (dead load plus factored live load plus impact applied in a single lane) are indirectly controlled in the Recommended Specifications by limiting the factored average flange stresses due to overload to $0.80F_y$ for noncomposite girders and $0.95F_y$ for composite girders.

At the request of the Project Panel, live-load deflections should be checked in the Recommended Specifications by using the specified service live load plus impact applied in all lanes. This live load is to be placed at whatever location produces the maximum deflection in each girder.

2.4.3 Concrete Crack Control. Article 10.38.4.3 of the Standard Specifications requires that longitudinal reinforcement equal to a minimum of 1 percent of the total deck area be placed in negative-moment regions of continuous spans for crack control. Two-thirds of the required reinforcement is to be placed in the top layer of the slab within the effective width. The negative-moment region in a continuous span is often implicitly taken as the region between points of dead-load contraflexure. Under moving live loads, the concrete deck can experience significant tensile stresses outside the points of dead-load contraflexure. Placement of the concrete deck in stages can also produce negative moments in regions that are primarily subject to positive moments in the final condition. Thermal and shrinkage effects can also result in tensile stresses in the deck in regions where such stresses might not be anticipated. Krauss and Rogalla (*31*) discussed the effects of design details, concrete materials, thermal and shrinkage effects, and construction practices on the potential for early-age transverse deck cracking. In addition, the current Guide Spec and Standard Specifications do not recognize the state of stress in the concrete deck when determining the placement of the longitudinal reinforcement; the tensile strength of the concrete is ignored.

To address some of these issues, Article 2.4.3 of the Recommended Specifications requires that the 1-percent longitudinal reinforcement be placed wherever the longitudinal tensile stress in the deck due to the factored construction loads or due to the overload exceeds the modulus of rupture of the concrete. Thermal and shrinkage effects were considered outside the scope of this effort and were not specifically addressed. By controlling the crack size under the overload, the concrete deck can be considered to be effective in tension at overload and at all loadings below overload. The modulus of rupture is computed from AASHTO Article 8.15.2.1 in the allowable stress design portion of the concrete design specifications; the tensile strength of the concrete is ignored for flexural calculations in the load factor design portion of the concrete design specifications. A resistance factor of 0.7 is applied to the modulus of rupture in the Recommended Specifications, rather than the 0.21 factor used in conjunction with a load factor of 1.0 in AASHTO Article 8.15.2.1, to provide additional assurance against cracking. Article 2.4.3 also specifies that the required reinforcement be No. 6 bars or smaller, spaced at no more than 12 in., to ensure adequate distribution of the reinforcement to control the crack size. Adequate shear capacity of the concrete deck is considered to be important in bridges subject to torsion, such as curved bridges. Analysis of the bridge assuming the deck to act as a continuum implies adequate horizontal shear strength in the concrete deck.

2.5 Constructibility

2.5.1 General. Curved-girder bridges, once completed, have generally performed as intended. The majority of problems with curved-girder bridges have typically occurred during construction. Construction of horizontally curved steel bridges is generally more complex than construction of comparable tangent bridges of similar span. Curved-girder bridges are typically less stable during erection than tangent bridges. Curved bridges often require additional support by using either cranes or falsework. Successful completion of a curved structure requires that each phase of construction proceed as anticipated to ensure that the final structure is at its proper elevation. Like segmental concrete bridges, construction issues can often drive design decisions.

Thus, the Recommended Specifications include a constructibility limit state to ensure that critical construction issues are properly engineered in the design. This article requires that one construction scheme be shown on the design plans. Some precedent for this requirement does exist given that deck casting sequences are often specified in the design plans of many states. Such a requirement does not relieve the contractor of the responsibility to provide a final construction plan and to execute the construction successfully according to that plan. Division II of the Recommended Specifications allows the contractor to consider and engineer construction schemes other than the scheme provided in the design plans.

2.5.2 Stresses. Factored stresses in the steel due to construction loads are limited to the yield stress at each critical stage of erection to ensure that any permanent deformations are controlled. For similar reasons, high-strength bolts in connections of primary load-carrying members are to be designed as slip-critical connections at the constructibility limit state.

2.5.3 Deflections. Fit-up of girders in the fabrication shop is often done with the girders blocked into their cambered positions so that they are in a "no-load" or "zero-stress" state. In the design of the bridge, it is typically assumed that the girders will be erected under no-load conditions, or assuming gravity is turned off, when the connections are made and that the self-weight of the girders will be applied only to the completely erected steel structure. Dead-load cambers are typically computed based on this assumption and are introduced into the girders accordingly.

If curved girders are not fully shored to match the conditions assumed in the fabrication shop, then the girders will begin to deflect and rotate immediately due to their own self-weight as they are erected until they are restrained by cross frames attached to adjacent girders. These temporary deflections and bearing rotations during construction may actually exceed the dead-load deflections and rotations computed for the completed bridge. In particular, for longer spans where the self-weight of the girders can result in significant distortions, the resulting deflections and rotations of one or two girders—superimposed on the initial assumed camber geometry—may make it difficult to connect cross frames and field splices that were originally detailed for the no-load condition. Relocation of an erected system of girders to ensure proper fit-up and girder elevations may require more crane capacity than might be available. Lifting of connected girders to make adjustments becomes particularly difficult when dealing with curved bridges and bridges with skewed supports.

To ensure proper fit-up of steel sections during erection and to prevent damage to bearings or expansion devices in these situations, the Recommended Specifications require that computed girder rotations at bearings and vertical girder deflections be accumulated over the erection sequence. The computed rotations must not exceed the specified capacity of the bearings for the accumulated factored loads at any stage of the erection. Refined methods of analysis are now available to allow the engineer to analyze a curved bridge for a given erection sequence and detail the bridge to accommodate the accumulated deflections and rotations. As a result, these provisions do not implicitly require that girders be fabricated in the no-load condition.

The provisions also require the computed deflections for the specified deck casting sequence to be accumulated. If the accumulated deflections differ significantly from the dead-load deflections computed for the final condition, then the engineer could have to make a judgment on how to camber the girders to achieve the final desired profile grade.

3. Loads

3.1 General

Loads are factored and combined as prescribed in AASHTO Table 3.22.1A, except as modified in the Recommended Specifications.

3.2 Dead Loads

This article specifies that the engineer should consider the sequence of application of dead load in the analysis. The stiffness of the structure changes as subsequent girder sections are erected. For example, an interior girder may actually be an exterior girder at some phase of the erection, which may result in a larger moment in the girder than after the actual exterior girder is properly erected. Also, during the deck casting, portions of the deck may cure before subsequent casts are made. The resulting change in stiffness of the structure will affect the computed moments and deflections for subsequent casts. Article 4.6 states that when checking deck stresses during deck casting, the uncracked section computed with a modular ratio of n is to be used. Thus, the effects of creep should be ignored for this relatively short-term condition.

The Project Panel requested that, for consistency, Article 3.23.2.3.1.1 of the Standard Specifications be referred to for application of superimposed dead loads. This article permits lighter loads, including curbs and railings, to be distributed uniformly to each girder in the cross section. Some have interpreted this clause to apply to heavier loads such as parapets and barriers as well, while others have instead assigned a larger percentage of these loads to the exterior girders when line-girder analyses are employed. Article 3.2 permits heavier superimposed line loads, such as parapets, sidewalks, and barriers, to be applied at their actual locations in the analysis. The behavior of curved bridges is unsymmetrical by nature. As a result, it does not seem reasonable to apply heavy line loads uniformly to all girders in a curved bridge. Application of these loads at or near their actual locations can easily be accommodated when refined methods of analysis are employed, which are recommended for curved bridges. Typically, wearing surface loads and other similar loads would be uniformly distributed to each girder.

3.3 Construction Loads

The Recommended Specifications specify that a load factor of 1.4 should be applied to all construction loads (e.g., self-weight of the steel, deck forms, deck, haunches, parapets, and construction equipment) when computing girder actions to be applied in checking the constructibility limit state. This load factor is approximately equal to the average of the dead-load factors of 1.25 and 1.5 applied in the Strength I and Strength IV load combinations in the AASHTO LRFD Specifications. The specified factor is reduced to 1.0 when computing deflections due to the construction loads. Although the construction loads are temporary and are often not accurately known at design time, the magnitude and location of these loads should be noted on the design plans.

For construction loads that resist uplift, a reduced load factor of 0.9 is conservatively applied. For construction loads that cause uplift, a load factor of 1.2 should be applied. This factor is slightly less than 1.4 because a slight uplift is generally not critical to most structures. For cases where uplift might result in instability or excessive deflections, the engineer may wish to consider applying a larger factor to these loads.

3.4 Wind Loads

According to this article, wind load is to be applied unidirectionally to the projected vertical area of the bridge, including barriers and sound walls. According to Article 3.15 of the Standard Specifications, wind should be applied either perpendicular or longitudinal to the bridge. For a curved bridge, this is not practical, nor is it reasonable to apply wind as a circular wind force radial to the bridge. Applying the wind unidirectionally to the projected vertical area of the curved bridge accomplishes the same objective as the provisions specified in AASHTO Article 3.15, although the total wind load may be less than if the provisions of Article 3.15 were applied literally.

The Recommended Specifications also require the engineer to determine the direction(s) of the wind that causes the critical load effect in various components of the bridge. A load path must be identified through which the wind loads are assumed to be transmitted to the substructure. All members and connections along that path should be designed for the effect of the wind loads in combination with other loads. Wind load applied to the top half of the exposed area is mainly resisted through horizontal deck shear, assuming the deck is capable of providing horizontal diaphragm action. The majority of the wind load applied to the bottom half of the exposed area is typically transferred from the bottom flange through diaphragms or vertically inclined cross bracing up to the deck and then down to the bearings and substructure through end cross frames or diaphragms.

3.5 Live Loads

3.5.3 Permit Loads. As an option to placing permit loads in all traffic lanes, this article allows the engineer to consider a permit vehicle in one lane in combination with the factored design vehicular live load applied in the remaining lanes, if permitted by the owner. Such an option, which is currently allowed in some states, can be investigated more easily when refined methods of analysis are employed. However, consideration of permit loads in all loaded lanes is also allowed.

3.5.4 Overload. Overload is defined as the dead load plus the maximum expected live load to occur a relatively infrequent number of times. Infrequent implies that overload stresses need not be considered for fatigue. Overload is used to determine if the concrete deck is subject to a critical tensile stress for locating the additional 1-percent longitudinal reinforcement (Article 2.4.3) and for checking slip in high-strength bolted connections (Article 11.2). Finally, the overload is used to compute and ensure control of permanent deflections (Articles 9.5 and 10.5).

The overload was developed from the AASHTO Road Tests (*32*) in Ottawa, Illinois, in the late 1950s. Researchers observed that single heavy trucks used in the tests could cause permanent set in steel girder bridges after several hundred thousand cycles. When the load factor design provisions were developed, the overload was incorporated to address such issues as permanent set and slip of bolted connections.

AASHTO Article 10.57 defines overload as $D + 5/3(LL + I)$ for bridges designed for live-load H20 and greater. The provisions imply that multiple lanes of traffic are considered. AASHTO Article 3.5 requires that for bridges designed for less than H20, a single lane of H or HS truck load be considered and that loading combination IA in AASHTO Article 3.22, which imposes a live-load factor of 2.2, be applied. Although the provision is somewhat ambiguous, it clearly implies consideration of a single infrequent heavy truck. Using the standard wheel load distribution factors, two times a single lane loaded is approximately equal to 5/3 times multiple lanes loaded as follows:

Multiple lanes loaded with the normal live load:

$$\frac{5}{3} \times \frac{S}{5.5} \times \text{one lane moment}$$

$$= \frac{S}{3.3} \times \text{moment due to one lane}$$

Single lane loaded with two times the normal live load:

$$2 \times \frac{S}{7} \times \text{one lane moment}$$

$$= \frac{S}{3.5} \times \text{moment due to one lane}$$

Thus, the present overload is approximately equivalent to a single lane of two times the normal live load. When the load factor design provisions were written, HS20 was the standard live load used in AASHTO. Thus, the intent of the original overload provisions was to apply two times a single lane of HS20 live load. This concept is similar to the use of two times a single fatigue vehicle used in the AASHTO LRFD provisions.

The researchers suggest the application of two times a single lane of HS20, rather than the application of the overload live load to several lanes. A heavy live load over the outside girder of a bridge may be more critical on a curved-girder bridge than on a tangent one. A heavy lane also produces more torque in box girders than do multiple lighter lanes of live load.

3.5.5 Sidewalk Load. The sidewalk live load should be taken from AASHTO Article 3.14 unless overridden by the owner. A footnote to Table 3.22.1A in the Standard Specifications states that a reduced load factor of 1.25 should be applied to the load effects caused by the combination of sidewalk live load and the vehicular live load plus impact (in lieu of the typical load factor of 1.67 applied to the vehicular live load plus impact acting alone). In computing the load effects due to the vehicular live load, according to the Standard Specifications, a lateral distribution factor is typically applied, which assumes the most critical placement of the design lanes over the girder nearest the sidewalk ignoring the presence of the sidewalk. The reduced load factor accounts for the low probability of the vehicular live load being located at this critical position when the sidewalk live load is also located at its most critical position.

When refined methods of analysis are employed, the actual transverse position of the loads is recognized considering the presence of the sidewalk loading, which typically results in the loaded lanes being further from the girder nearest the sidewalk. As such, the live-load effects in the girder are automatically reduced. Thus, the Recommended Specifications state that the maximum load factor applied to the vehicular live load should also be applied to the sidewalk load when the two loads are applied in combination (i.e., when the vehicular live load is not permitted on the sidewalk).

If the assumption is made that the vehicular live load is able to mount the sidewalk, the Recommended Specifications state that the sidewalk live loading should not be considered concurrently. Also, the Recommended Specifications state that an impact allowance need not be applied to the sidewalk live load; the current AASHTO Article 3.14.1 does not address this issue.

3.5.6 Impact. Impact factors that are applied to vehicular live loads to account for dynamic amplification are specified in this article. The specified factors that are applied to I- and box girders at the strength and serviceability limit states (Articles 3.5.6.1 and 3.5.6.2, respectfully) are simplifications of the factors provided in the current Guide Spec. Separate factors are specified for truck and lane loading in each case, with a slightly higher factor to be applied to truck loading. The impact factors for box girders are higher than for I-girders. Article 3.5.6.3 specifies a reduced impact factor of 15 percent to be applied to the vehicular fatigue load, which corresponds to the reduced impact factor applied to the fatigue live load in the AASHTO LRFD Specifications.

3.5.7 Fatigue. 3.5.7.1 General. The development of the fatigue live load given in the Recommended Specifications was covered previously under the discussion of Article 2.3.

3.5.7.2 Application. For transverse members (e.g. cross frames and diaphragms), the Recommended Specifications state that one cycle of stress be defined as 75 percent of the stress range determined by the passage of the fatigue live load in two different transverse positions, but not to be less than the stress range due to a single passage of the fatigue live load. The use of two transverse positions of the truck is synonymous with the assumption that the stress range is determined by the separate passage of two trucks rather than one. The AASHTO Standard Specifications and LRFD Specifications do not address stress range in transverse members. Two transverse positions of the fatigue live load are considered because the sense of stress in a transverse member is reversed when the live load is placed over an adjacent girder or web connected by the cross member. Two transverse positions of the live load, therefore, usually create the largest stress range in radially oriented members. Of course, the two transverse positions of the load are to be governed by the rules for the placement of design lanes within the roadway width.

Consider a tangent single box girder with the deck placed centrally on the box. The cross frame forces inside the box caused by placing a truck at its extreme permitted position on the left side of the deck will be doubled by placing the same truck at its extreme permitted position on the right side of the deck. Thus, the full range of stress in the cross frame members is caused by a fatigue truck in each lane, with one truck being led by the other. The range of stress in the cross frame members would be twice the range of stress computed for a single passage of the fatigue truck. To account for the reduced probability of two vehicles being in their critical relative positions, a factor of 0.75 was arbitrarily chosen to be applied to the computed stress range. At this time, there is no known data suggesting what an appropriate value should be, so a conservative value of 0.75 is reasonable until further research is done.

In addition, there is currently no allowance for the fact that two trucks are required to compute the maximum stress range in these members. In many instances, the specific number of cycles will not matter because fatigue details will be designed for an infinite life beyond the fatigue limit. However, if the design life of the detail is less than the fatigue limit, the engineer may wish to consider a reduction in the number of cycles for this case. This is similar to the approach currently given in the AASHTO LRFD Specifications for the fatigue loading of floor beams.

3.6 *Thermal Loads*

According to the Recommended Specifications, curved bridges should be designed for the assumed uniform temperature change specified in AASHTO Article 3.16. The orientation of bearing guides and the freedom of bearing movement is extremely important in determining the magnitude and direction of thermal forces that can be generated. For example, sharply skewed supports and sharp curvature can cause very large lateral thermal forces at supports if tangential movements are permitted and radial movements are not permitted. Under a uniform temperature change, orienting the bearing guides toward a fixed point and allowing the bridge to move freely along rays emanating from the fixed point will theoretically result in zero thermal forces. Other load conditions, however, can dictate the bearing orientation. The bearing restraints and orientation, as well as the lateral stiffness of the substructure, must be considered in a thermal analysis.

In addition, a uniform 25°F temperature difference between the deck and the girders should be considered when the width of the deck is less than one-fifth of the longest span. A temperature gradient is also specified in the AASHTO LRFD Specifications and in the AASHTO *Guide Specifications for Design and Construction of Segmental Concrete Bridges* (*33*). Krauss and Rogalla (*31*) discussed the effect of such temperature differences. The difference is important in narrow bridges where uplift at bearings might occur due to induced bending. Designers may wish to check deck stresses with this load, but such a check is not required.

4. Structural Analysis

4.1 *General*

As specified in the Guide Spec, this article requires that the determination of moments, shears, and other load effects for the design of the superstructure and its components be based on a rational analysis of the entire superstructure. The analysis may be based on elastic small-deflection theory, unless the engineer believes that a more rigorous approach is necessary.

4.2 *Neglect of Curvature Effects*

Under certain specified conditions, the effects of curvature may be neglected in the computation of vertical bending moments in I- and box-girder bridges. However, the effects of curvature must still be considered in all checks related to strength and stability. The development of the specific conditions in Article 4.2.1 when curvature effects may be neglected in determining the vertical bending moments in I-girders is discussed at length in Appendix A. The conditions in Article 4.2.2 for closed box and tub girders are taken from Tung and Fountain (*34*).

A convenient approximate equation for determining the lateral bending moment in I-girder flanges resulting only from the effects of curvature when the cross frame spacing is relatively uniform is given in Article 4.2.1.

4.3 *Methods*

As specified in Article 4.3.1, approximate analysis methods may be used when the engineer confirms that results from such an analysis are consistent with the principles defined in Article 1.2. Although not specified, such checks include global equilibrium checks of dead-load reactions and static equilibrium checks at various cross sections. If these checks are not satisfactory, refined methods of analysis must be used according to the provisions of Article 4.3.2.

Two of the most commonly used approximate methods for curved-bridge analysis are the V-load method for I-girder bridges and the M/R method for box-girder bridges. Strict rules and limitations on the applicability of these approximate methods of analysis do not exist. The engineer must determine when approximate methods of analysis are appropriate for application to a given case. A brief discussion of some of the inherent limitations of these methods is given in the commentary to Article 4.3.1. An additional limitation of these methods that is not discussed is that vertical bearing reactions at interior supports on the inside (concave side) of continuous-span curved bridges are often significantly underestimated by these methods.

4.4 *Uplift*

When lift-off is indicated in the analysis, this article requires that the analysis be modified to recognize the absence of vertical restraint at that particular support and the freedom of the girder to move at that bearing. Lift-off is most likely to be permitted to occur during sequential placement of the concrete deck.

4.5 *Concrete*

4.5.1 General. Concrete is assumed to be effective in compression, tension, flexural shear, and in-plane shear for determining the global stiffness used to generate moments, shears, reactions, and torsion. Such an assumption simplifies the analysis and is consistent with the assumption made in the analysis of reinforced concrete structures where the concrete is generally considered to be effective in tension. Field measurements (*35,36*) also confirm that the concrete is effective in tension for typical magnitudes of live load. For extremely large girder spacings, such as in girder floor-beam bridges, shear lag should be considered in determining the amount of deck to be considered effective. The AASHTO *Guide Specifications for Design and Construction of Segmental Concrete Bridges* (*33*) can be used to determine the effective width adjusted for shear lag.

4.5.2 Stresses in Composite Sections. This article provides the computation of stresses in composite sections, which is a subject given limited consideration in the current AASHTO Standard Specifications. This article is intended to provide consistency in determining bridge capacity from these provisions by ensuring a unified approach in computing the stresses in composite sections. The economics of many composite girders are affected by decisions related to the computation of section properties.

Because of the effects of moving live loads, points of dead-load contraflexure have little meaning in continuous bridges. Both positive and negative live-load moments are applied at nearly all points along the bridge. The Recommended Specifications state that positive live-load moments should be applied to the uncracked composite section at all limit states.

At the strength limit state only, negative live-load moments should be applied to the cracked composite section (ignoring the concrete) at all locations regardless of the magnitude of the net concrete tensile stress. Positive superimposed dead-load moments should be applied to the uncracked composite section at the strength limit state unless the net stress in the concrete deck due to the superimposed dead-load moment plus the negative live-load moment is tensile.

At the fatigue, serviceability, and constructibility limit states, the uncracked section should be used for computing stresses due to both positive and negative superimposed dead- and live-load moments. This assumption will have a significant beneficial effect on the computation of fatigue stress ranges in regions of stress reversal and in negative moment regions. Researchers have established that concrete provides significant resistance to tensile stress at service load levels. Article 2.4.3 of the Recommended Specifications provides a proper amount, size, and spacing of longitudinal deck reinforcement in regions where the longitudinal tensile stress in the deck due to factored construction loads or due to overload exceeds the modulus of rupture times a resistance factor. By using properly spaced longitudinal reinforcement, crack length and width can be controlled so that full-depth cracks do not occur across the entire deck. When a crack does occur, the stress in the longitudinal reinforcement increases while the crack grows. Ultimately, the cracked concrete and the reinforcement reach equilibrium and the crack is arrested. Thus, the width of deck that is effective with the girder could contain a small number of cracks that are staggered at any given section of the deck. However, properly placed longitudinal reinforcement does not allow these cracks to coalesce. Brazegar and Maddipudi (*37*) addressed the effects of slip and crack size on both the strength and stiffness of concrete in tension. Even at the overload level, it is unlikely that the net tensile stresses in the concrete deck will be far enough above the modulus of rupture to cause significant deck cracking. Field data substantiates that stresses in the composite section are best predicted based on section properties computed assuming an uncracked section up to the overload level (*35*).

In addition to considering that the concrete deck is effective for computing flexural stresses at selected limit states, the provisions also consider that the concrete deck is fully effective with the top flange bracing at all locations in resisting the torsional shear in tub girders. In this instance, the deck is considered to be fully effective in horizontal shear at all limit states. Although not discussed, orthogonal reinforcement in the deck of I-girders also helps ensure adequate shear capacity for torsional loads.

This article also states that the effective width of the deck shall be the full deck width over each girder or web when computing the composite section properties. Current AASHTO Standard Specifications limit the effective width of the concrete deck over interior girders to the lesser of (1) 12 times the least thickness of the deck (12t), (2) one-fourth the span length of the girder, and (3) the girder spacing. At a time when girder spacings were typically 8 ft or less, the effective width computed according to this requirement most always included all of the deck. With the increasing use of wider girder spacings, recognition of more of the concrete deck is necessary to better predict composite dead- and live-load deflections. Recent field measurements reported that an entire 9.5-in. thick deck was fully effective in both positive and negative moment regions on a bridge with a 15-ft girder spacing (*35*). The AASHTO *Guide Specifications for Design and Construction of Segmental Concrete Bridges* (*33*) recognize the entire deck width to be effective unless shear lag adjustments become necessary.

Article 4.5.2 also requires that girder stresses due to vertical bending and lateral flange bending acting on the noncomposite and composite sections be accumulated for checking all limit states. Thus, total stresses cannot be determined by applying the sum of the noncomposite and composite moments to the composite section. When computing the noncomposite dead-load stresses to be added to the composite dead- and live-load stresses at the strength limit state, the provisions allow the engineer to assume that the deck is placed at one time rather than sequentially. Over time, creep of the concrete will result in stresses corresponding to this assumption.

Lateral flange bending stresses do not need to be considered in the concrete deck. When a cast-in-place deck is used, lateral bending stresses in the top flange (including the locked-in stresses due to the noncomposite loads) can be ignored after the deck has hardened. The combined lateral section modulus of the top flange and concrete slab is so large that the lateral flange bending stresses due to loads applied to the composite section are negligible. Lateral flange bending stresses induced in the noncomposite flange prior to its encasement in concrete remain in the flange. The provisions permit these stresses to also be ignored after the deck has hardened because these stresses do not contribute to lateral failure of the composite flange. Although these stresses do theoretically contribute to the potential for premature failure of the steel flange, the deck should provide adequate additional capacity at the strength limit state. If precast deck panels are used, then the engineer may choose not to ignore the noncomposite lateral flange stresses in the top flange, particularly if the flange is not continuously supported by the deck panels.

4.5.3 Prestressed Concrete. Prestressed concrete decks and integral post-tensioned pier caps are explicitly permitted by the Recommended Specifications. Although the AASHTO Standard Specifications do not forbid these forms of construction, they do not cover them either. The intent of this article is to enumerate several issues that should be considered when these forms of construction are used. These issues include friction forces, sequencing of prestressing, forces induced in the girders and their components due to the prestressing, and transverse tensile stresses resulting from the Poisson effect. In an integral post-tensioned pier cap, the transverse Poisson tensile stress induced in the concrete by the post-tensioning also induces a longitudinal stress in the steel girders passing through the cap.

5. Flanges with One Web

5.1 General

The provisions in this article apply to a single horizontally curved flange attached at midwidth to a vertical or inclined web. The flange can be either continuously or partially braced and either compact or noncompact. The strength equations in this article, as is the case in the current AASHTO Standard Specifications, were derived assuming a prismatic flange between brace points. Thus, changes in flange size should be made at or near a panel brace point. When changes in flange size are made away from a brace point, the flange size should still be assumed to be constant using the properties (width and radius) of the smaller flange within the panel. The procedures for members with a variable moment of inertia between brace points that are referred to in the AASHTO Standard Specifications (Table 10.32.1A, footnote c) should not be applied to the strength equations given in the Recommended Specifications.

The stress, f_b, in the strength equations should be taken as the largest average factored flange stress at either brace point. The total lateral flange bending stress, f_l, due to both curvature and other effects should be taken as that stress at the critical brace point. Sign conventions for both stresses are given in this article. In general, when the lateral flange moment is a restoring force at the cross frame, the ratio of the lateral flange bending stress to the average flange stress is positive. Thus, when the lateral flange bending is due primarily to curvature, the ratio will be positive at the cross frame in both top and bottom flanges.

The limitations on the unbraced length and the lateral stress-to-bending stress ratio at brace points are retained from the current Guide Spec. In the Recommended Specifications, however, the bending stress ratio should be checked only when the average flange stress is greater than or equal to 0.5 times the critical flange stress because there is no need to limit this ratio when the average flange stress is small. The lateral flange bending stress is also limited to $0.5F_y$ in the Recommended Specifications, regardless of the level of vertical bending stress. Other parameters that were limited in the research that led to the development of the strength equations are listed in the commentary to this article. An extreme deviation from any of these parameters in the design is not recommended.

5.2 Partially Braced Compression Flanges

Partially braced compression flanges are defined in the Recommended Specifications as flanges that are not braced continuously by the hardened concrete deck. Partially braced compression flanges qualify as either compact or noncompact depending on the width-to-thickness ratio and yield stress of the flange. In the Recommended Specifications, flanges rather than sections are defined as compact and noncompact because curved girders fail due to lateral bending and are not able to develop the full plastic-moment capacity, M_p. Thus, a compactness requirement for the web has little significance. This assumption is consistent with the work done by Culver and McManus (*6*), which is reflected by the absence of a web compactness requirement in the current Guide Spec.

5.2.1 Compact Flanges. Partially braced compression flanges qualify as compact when the yield stress does not exceed 50 ksi and the width-to-thickness ratio of the flange does not exceed 18. The specified yield stress limit of 50 ksi was also recognized by Culver and McManus. The maximum width-to-thickness ratio of 18 specified for compact flanges has been liberalized from the current value in the Guide Spec. The value in the Recommended Specifications corresponds approximately to the maximum permissible flange slenderness for compact sections in tangent girders fabricated from 50-ksi steel, as given in the AASHTO LRFD Specifications. During construction conditions, a partially braced flange should not be considered compact when determining the critical flange stress.

Proposed Equation (5-4) for the critical flange stress is almost identical to the equation from the Guide Spec. There have been some adjustments made to the equation based on further study of the work by Culver and McManus (*6*) under the CURT project. In the equations for ρ_b and ρ_w, f_w has been replaced with f_m, which is defined as the factored lateral flange bending stress at the critical brace point *due to effects other than curvature.* These effects might include torsion resulting from the effect of skewed supports or dead loads acting on overhang brackets. If torsion is due only to the effects of curvature, which is often the case, f_m will equal zero. Lateral flange bending stresses due to curvature are not included in f_m because such an assumption was made in the original development of the ρ terms by Culver and McManus. This fact was not made evident in the current Guide Spec. By setting f_m to zero, Equation (5-4) predicts lower strengths. This change causes the proposed Equation (5-4) to better predict the ultimate strength of curved test specimens. A more detailed discussion is found in Appendix B. The equations for the ρ factors were also modified slightly in the Recommended Specifications to allow the length between brace points and the minimum radius in the panel to be input in units of feet rather than inches.

The proposed Equation (5-4) recognizes the ability of sections with compact flanges to achieve full plastification under the combination of vertical and lateral bending. Because the product of the two ρ factors is limited to 1.0, the maximum critical flange stress is conservatively limited to the lateral buckling stress given by the proposed Equation (5-5).

The proposed Equation (5-5) is retained from the current Guide Spec with one exception. For singly symmetric sections, the flange width, b_f, should be taken as $0.9b_f$ in the computation of the term λ. Equation (5-5) was the lateral torsional buckling equation for tangent-girder compression flanges that existed in the AASHO Specifications at the time of the Culver and McManus work. The factor of 0.9 applied to the flange width was prescribed for singly symmetric sections in those AASHO provisions but was omitted in the Guide Spec. The decision was made to restore this factor in the Recommended Specifications. Strictly speaking, Equation (5-5) is only applicable when the critical stress computed from the equation is greater than $0.5F_y$, which is the typical case. For critical stresses less than $0.5F_y$, the engineer may wish to consider using the classical Euler hyperbola to determine the critical stress because the parabolic shape of Equation (5-5) is too conservative for predicting the critical stress at higher values of the flange slenderness.

The current Guide Spec does not prescribe a limit on the flange tip stress for compact flanges in the load factor design provisions. If a compact flange has an average stress equal to the yield stress, there is theoretically no capacity remaining in the flange to resist the lateral bending. Thus, the proposed Equation (5-6) was developed to address this shortcoming in a rational fashion. The limit on the combined vertical and lateral bending stress was derived on the basis of the assumption that the flange is fully yielded under the combination of the vertical and lateral flange bending moments. The derivation of the proposed Equation (5-6) is discussed in greater detail in Appendix A. Based on this derivation, a small portion of the web is permitted to yield. If the engineer wishes to prevent any yielding of the web, then the factor of ⅓ in the proposed Equation (5-6) can be replaced by a factor of ½ (*38*). The critical average stress for a compact flange is taken as the smaller value from either the proposed Equation (5-4) or (5-6).

5.2.2 Noncompact Flanges. Noncompact compression flanges are partially braced flanges that are permitted to reach the yield stress at the flange tip without local buckling. Noncompact flanges must satisfy the width-to-thickness limitation given by the proposed Equation (5-7). The actual tip stress is included in the denominator of the width-to-thickness requirement, which provides some relief at more lightly stressed sections. Research to date has shown that local buckling of flanges is critical when the flange slenderness exceeds 23 (*15,39*). During erection and deck placement when the compression flange is partially braced, the flange slenderness must satisfy Equation (5-7).

The proposed Equation (5-8) is close to the current equation given in the Guide Spec, except for an adjustment in the equations for the ρ factors that allows the length between brace points and the minimum radius in the panel to be in units of feet rather than inches. Also, f_w has been replaced by f_m in the equations for ρ_w. As shown in Appendix B, this change increases the predicted strength of curved girders with noncompact flanges and results in better correlation with the onset of initial nonlinear behavior in curved test specimens. The flange tip stress is limited to the yield stress according to the proposed Equation (5-9). The critical average stress for a noncompact flange is taken as the smaller value from either the proposed Equation (5-8) or (5-9).

5.3 Partially Braced Tension Flanges

In the current Guide Spec, the average stress in tension flanges is limited to F_y, without any consideration of the ρ factors. Tension flanges were not explicitly considered in the CURT project because only doubly symmetric sections were studied. As a result, McManus (*5*) had originally suggested that ρ factors be applied to tension flanges of singly symmetric sections. In earlier versions of the Guide Spec, ρ factors were therefore applied in determining the critical stress for tension flanges. Then, in the early 1990s, a rather arbitrary decision was made to remove these factors. After further study of the McManus work, it was decided to conservatively reinstate the application of the ρ factors to F_y in the proposed Equation (5-10) of the Recommended Specifications when determining the critical average stress for *partially braced* tension flanges. The ρ factors for compact flanges were used. It was also decided to limit the flange tip stress according to the proposed Equation (5-11), which is the same equation that is applied to compact compression flanges.

Finally, partially braced tension flanges should meet the width-to-thickness requirement for noncompact compression flanges given by the proposed Equation (5-7) to ensure that the flange will not distort excessively when welded to the web.

5.4 Continuously Braced Flanges

Continuously braced flanges typically represent either compression or tension flanges that are encased in concrete. In the Recommended Specifications, the critical average flange stress for a continuously braced flange is limited to F_y.

When the flange is partially braced, it must satisfy the width-to-thickness limitation given by the proposed Equation (5-7) for the conditions that exist when the flange is partially braced. As discussed earlier under Article 4.5.2, lateral flange bending stresses do not need to be considered once a flange is continuously braced.

6. Webs

6.2 Unstiffened Webs

To date, there have been no tests of horizontally curved unstiffened webs with a slenderness greater than about 70. Therefore, for girders with unstiffened webs and a radius less than 700 ft, the maximum allowable web slenderness is limited to 100 in the Recommended Specifications. The web slenderness limit of 100 is approximately equal to the maximum allowable web slenderness for compact webs of up to 50-ksi steel given in the AASHTO LRFD Specifications. According to Equation (6-2), the permitted web slenderness increases linearly with the minimum radius in the panel, up to a maximum slenderness of 150 at a minimum radius greater than or equal to 2,000 ft. The slenderness limit of 150 is the maximum allowable slenderness for tangent girders with unstiffened webs given in the AASHTO Standard Specifications.

6.2.1 Bending Stresses. Longitudinal compressive stresses in both unstiffened and stiffened curved web panels are limited to the smaller of the elastic bend-buckling stress or the yield stress at each limit state, including constructibility, in the Recommended Specifications. Equation (6-3) represents the typical form of the elastic bend-buckling stress in these provisions. The AASHTO Guide Spec and the Standard Specifications indirectly control the compressive stress (and therefore bend-buckling) in the web by providing explicit limits on the overall web slenderness, D/t_w, for both unstiffened and stiffened webs. In load factor design, these limits are primarily expressed as a function of the yield stress, F_y. Limiting the slenderness as a function of F_y assumes that the factored stress at all sections is equal to F_y. At sections where the factored stress is below F_y, a higher slenderness should be permitted.

In the Recommended Specifications, absolute limits on the overall web slenderness, D/t_w, are also provided for both stiffened and unstiffened webs; however, these limits are independent of F_y. The maximum longitudinal compressive stresses in the web are then limited separately to prevent elastic bend-buckling, which is logical since bend-buckling is an elastic phenomenon not related to the yield stress of the material. The permissible bend-buckling stress is a function of the depth of the web in compression, D_c, which, in turn, is related to the actual factored accumulated stress in the web. As in the current Guide Spec, post bend-buckling resistance is not recognized in curved web panels. Factored longitudinal tensile stresses in the web are limited to F_y.

For curved unstiffened webs, the bend-buckling coefficient, k, in the Recommended Specifications is conservatively limited to a value of $7.2(D/D_c)^2$, which is approximately 20 percent lower than the value of $9.0(D/D_c)^2$, given in Article 10.61.1 of the AASHTO Standard Specifications for flat webs. The bend-buckling coefficient for a flat plate subject to a symmetrical bending stress pattern and with clamped edges at the top and bottom of the plate is equal to 39.6 (*40*). The value of k in the current Standard Specifications assumes that the flanges offer only some partial degree of rotational restraint. For a symmetrical section, the resulting bend-buckling coefficient is 36.0. Both cases assume pure bending with no shear present. A moment-shear interaction relationship is currently not provided in the Guide Spec or the Standard Specifications to handle the case of unstiffened webs. Therefore, to provide some conservatism for cases where an unstiffened web panel is subjected to both high shear and high moment, the bend-buckling coefficient for unstiffened webs has been lowered in the Recommended Specifications. For a symmetrical section, the resulting bend-buckling coefficient is 28.8. Both the Guide Spec prior to 1990 and the Standard Specifications prior to 1987 followed a different tact by instead lowering the shear-buckling strength of unstiffened web panels. In both specifications, the shear strength of unstiffened web panels was limited to a strength approximately 30 percent below the theoretical elastic shear-buckling strength, presumably to account for possible moment-shear interaction effects. The only known tests of flat unstiffened web panels subjected to high shear and high moment did show some deterioration of strength (*41*). There are no known similar tests on curved unstiffened web panels. The paucity of tests on curved unstiffened web panels up to a slenderness of 150 subjected to various ratios of shear and moment is one significant argument for the suggested conservatism.

Near points of dead-load contraflexure, both edges of the web may be in compression when stresses in the steel and composite sections due to moments of opposite sign are accumulated. In this case, the neutral axis lies outside the web. The equation given for the bend-buckling coefficient does not recognize this situation and gives an exceedingly low coefficient for this case (i.e., D_c greater than D). Thus, the provisions also limit the minimum value of the bend-buckling coefficient for both unstiffened and stiffened webs to a value of 7.2, which is approximately equal to the theoretical bend-buckling coefficient for a web plate under uniform compression assuming clamped boundary conditions at the flanges (*40*).

6.2.2 Shear Strength. In the Recommended Specifications, the shear strength of unstiffened curved web panels is limited to the elastic shear buckling strength given by the proposed Equation (6-4). Because the decision was made to lower the bend-buckling strength to account for the effect of coincident shear, the shear strength does not need to be adjusted for coincident bending. The equations for the constant, C, are equivalent to the equations given in the current Standard Specifications. These equations are expressed in a nondimensional format in the Recommended Specifications. The shear-buckling coefficient, k_w, is taken as 5.0 for unstiffened web panels.

6.3 Transversely Stiffened Webs

The maximum allowable web slenderness, D/t_w, of transversely stiffened curved web panels is specified to be 150 in the Recommended Specifications. This web slenderness limit is independent of the yield stress of the web plate. For inclined webs, the distance along the web is used for D rather than the vertical distance. The maximum spacing of transverse stiffeners is limited to the web depth, D, which is retained from both the Guide Spec and the Hanshin Guidelines. There has been no research to date to indicate that a larger maximum spacing should be allowed. At simple supports, the maximum stiffener spacing is limited to $0.5D$ in order to anchor the panel against buckling.

6.3.1 Bending Stresses. The maximum longitudinal bending stress in a transversely stiffened curved web panel is limited according to the proposed Equation (6-8). The bend-buckling coefficient is increased from a value of $7.2(D/D_c)^2$ for unstiffened webs to a value of $9.0(D/D_c)^2$ because transverse stiffeners tend to strengthen curved web panels and because the post-buckling strength of the panel in bending and shear is currently ignored. A minimum bend-buckling coefficient of 7.2 is also specified for reasons discussed previously.

6.3.2 Shear Strength. The shear strength of transversely stiffened curved web panels is also taken as the elastic shear-buckling strength. As in the current Guide Spec, the post-buckling shear strength of stiffened curved web panels is ignored. The shear-buckling coefficient, k_w, is calculated according to the proposed Equation (6-9). The calculated stiffener spacing, d, that is required to provide the necessary shear strength should be used in computing k_w, because the actual stiffener spacing, d_o, may be less than d for practical reasons. A similar requirement appeared in earlier versions of the AASHTO Standard Specifications.

6.4 Longitudinally and Transversely Stiffened Webs

When the web slenderness of a transversely stiffened curved web panel exceeds 150, one or two longitudinal web stiffeners may be used in conjunction with the transverse stiffeners. The maximum web slenderness of a longitudinally and transversely stiffened web may not exceed 300, which is equivalent to the limit given in the Hanshin Guidelines for a web with two longitudinal stiffeners (*20*). There has been limited research on the behavior of longitudinally and transversely stiffened curved webs in Japan. No research on longitudinally and transversely stiffened curved webs has been conducted in the United States to date.

6.4.1 Bending Stresses. The maximum longitudinal compressive stress in a longitudinally and transversely stiffened curved web panel is limited according to the proposed Equation (6-10). The bend-buckling coefficient is given by either proposed Equation (6-11) or (6-12), depending on the location of the critical longitudinal stiffener (adjacent to the compression flange) with respect to a theoretical optimum location of 0.4 D_c from the compression flange. Equations (6-11) and (6-12) appear in the current AASHTO Standard Specifications and were developed assuming simply supported boundary conditions at the flanges (*42*). Changes in flange size along the girder cause D_c to vary along the length of the girder. If the longitudinal stiffener is located a fixed distance from the compression flange, which is normally the case, the stiffener cannot be located at its optimum location all along the girder. Also, the position of the longitudinal stiffener relative to D_c in a composite girder changes due to the shift in the location of the neutral axis after the concrete slab hardens. This shift in the neutral axis is particularly evident in regions of positive flexure. Thus, Equations (6-11) and (6-12) were developed to compute the web bend-buckling capacity for any position of the longitudinal stiffener with respect to D_c. D_c should be computed based on the sum of the accumulated stresses due to the factored loads for the limit state under consideration. In regions of negative bending at the strength limit state, D_c may conservatively be taken as the value of D_c computed for the section consisting of the steel girder plus the longitudinal reinforcement, since the distance between the neutral axis locations for the steel and composite sections is typically not large in negative-bending regions.

Because a longitudinally and transversely stiffened web must be investigated for the stress conditions at different limit states and at various locations along the girder, it is possible that the stiffener might be located at an inefficient location for a particular condition resulting in a very low bend-buckling coefficient from Equation (6-11) or (6-12). Because simply supported boundary conditions were assumed in the development of these equations, the computed web bend-buckling resistance for a longitudinally and transversely stiffened web may be less than that computed for a web without longitudinal stiffeners where some rotational restraint from the flanges has been assumed. To prevent such an anomaly, the provisions state that the k value for a longitudinally stiffened web must equal or exceed the k value for a web without longitudinal stiffeners, which is calculated assuming partial rotational restraint from the flanges.

In regions where the web undergoes stress reversal, it may be necessary to use two longitudinal stiffeners on the web. Alternatively, it may be possible to place one stiffener on the web such that the limit states are adequately satisfied with either edge of the web in compression.

6.4.2 Shear Strength. The shear strength of a longitudinally and transversely stiffened curved web panel should be taken the same as the shear strength of a web panel without longitudinal stiffeners. As in the current specifications, the contribution of the longitudinal stiffener to the shear strength of the panel is ignored.

6.5 Transverse Web Stiffeners

The provisions for the design of transverse web stiffeners are essentially the same as the requirements in the load factor design portion of the current Guide Spec. The curvature parameter, Z, that is applied in determining the required rigidity of the stiffener accounts for the radial force that develops in curved girder webs. The required moment of inertia of the stiffener is determined by using the required stiffener spacing, d, in the panel, because d is also used to determine the panel shear strength. However, the actual stiffener spacing, d_o, is used to compute the curvature parameter, Z, because this parameter is actually dependent on the restraint provided by the transverse stiffeners.

The maximum permissible width-to-thickness ratio for the stiffener given by the proposed Equation (6-13) is essentially the same as the current requirement in the Guide Spec, only expressed in a nondimensional format.

The provisions do recommend that intermediate transverse stiffeners be attached to both flanges when single stiffeners are used on only one side of the web to help retain the cross-sectional configuration of the web panel under nonuniform torsion and to help prevent torsional buckling of the flanges. When pairs of transverse stiffeners are used, the provisions require that the stiffeners fit tightly against both flanges. Stiffeners serving as cross frame connection plates must be rigidly attached to both flanges to transfer the horizontal force in the cross members to the flanges. The stiffeners must also be of the same or lesser material grade as the web to which the stiffeners are attached.

6.6 Longitudinal Web Stiffeners

For composite sections, the vertical position of a longitudinal web stiffener relative to D_c changes after the concrete slab hardens. This relative position can also change at various stages of the deck casting sequence and as the web undergoes stress reversal. Thus, the computed web bend-buckling resistance given by the proposed Equation (6-10) is different for different conditions before and after the slab hardens. As a result, the determination of the optimum location should be left up to the engineer. Several trial vertical locations of the stiffener may need to be investigated to determine the optimal location of the stiffener to provide adequate elastic web bend-buckling resistance at each limit state, particularly in positive bending regions. Because the neutral axis location does not vary greatly in negative bending regions before and after the slab hardens, an initial trial location of 0.4 D_c from the compression flange should be examined in these regions, where D_c is the depth of the web in compression at the section with the maximum compressive stress due to the factored loads. As discussed previously, D_c, for this case, should be computed for the section consisting of the steel girder plus the longitudinal reinforcement rather than from the accumulation of the factored stresses. Because D_c may vary along the span, the stiffener may not be located at its optimum location at other sections with a lower stress and a different D_c. These sections should also be examined to ensure that they satisfy the specified limit states (see Appendix E).

Longitudinal stiffeners placed on the opposite side of the web from transverse stiffeners are preferred. At bearing stiffeners and connection plates, where the longitudinal stiffeners and transverse web elements must intersect, the longitudinal stiffeners must be made continuous to satisfy the analytical assumptions made in the theoretical development of the web bend-buckling strength given by the proposed Equation (6-10) and in the theoretical development of the rigidity requirement given by the proposed Equation (6-19), in which the longitudinal stiffeners were assumed to be restrained at their ends by the transverse stiffeners (*20,42*). In the case of curved webs, the required longitudinal stiffener is typically larger and is also subject to lateral bending, which makes it even more imperative that the detailing be consistent with the theoretical design assumptions. The engineer may interrupt either the longitudinal stiffener or the transverse web element. However, the discontinued element must be fitted and attached to both sides of the continuous element with connections sufficient to develop the flexural and axial resistance of the discontinued element. If the longitudinal stiffener is interrupted, however, the interruptions must be carefully designed with respect to fatigue. Copes must also be provided in the stiffener plates in either case to avoid intersecting welds.

The development of the proposed Equation (6-19) for the required moment of inertia of a longitudinal stiffener on a curved web panel was covered in the section on the Hanshin Guidelines. The factor, β, in the proposed Equation (6-19) was developed from a linear fit of a more complex requirement given in the Hanshin Guidelines. The required moment of inertia for a longitudinal stiffener on a curved web panel will typically be larger than the required moment of inertia for a longitudinal stiffener on an equivalent flat web panel because of the tendency of the curved web to bow.

The Recommended Specifications also require that the width-to-thickness ratio of the outstanding longitudinal web stiffener element satisfy the proposed Equation (6-13), which is the same requirement that is specified for transverse web stiffeners. The provisions also require that the material grade of the longitudinal stiffener be the same as for the web to which the stiffener is attached to prevent the possibility of a premature failure of the longitudinal stiffener.

6.7 Bearing Stiffeners

The provisions for the design of bearing stiffeners in the Recommended Specifications cover the design of both concentrically and eccentrically loaded bearing stiffeners. The provisions for the design of bearing stiffeners given in Article 10.34.6 of the Standard Specifications were written at a time when rocker-type steel bearings were the most common type of bearing used. Thus, eccentricities due to thermal movements did not generally occur. With the use of more modern sliding bearings, larger eccentricities of the vertical reactions on bearing stiffeners can occur, in particular, when the bearing stiffeners are attached to solid-plate diaphragms that are perpendicular to the direction of the primary thermal movements.

For concentrically loaded bearing stiffeners, the Recommended Specifications state that the stiffeners should be designed as centrally loaded compression members. The AASHTO Standard Specifications do not currently contain specific load factor design provisions for bearing stiffeners. Therefore, the Recommended Specifications refer directly to the load factor design provisions for centrally loaded columns given in Article 10.54.1 to design the stiffeners. An effective length factor, K, of 0.75 should be used. For bearing stiffeners subject to significant eccentricities, the stiffeners should preferably be designed as beam-columns, according to the provisions of AASHTO Article 10.54.2. These provisions provide interaction equations for the design of beam-columns under combined bending and axial load. Again, K should equal 0.75. The equivalent moment factor, C, is to be conservatively taken as 1.0. In both cases, a centrally located portion of the web or diaphragm equal to $18t_w$ can be considered effective with the pair of bearing stiffeners. As referred to previously, bearing stiffeners can be used in pairs on either the girder or diaphragm web at locations that receive girder reactions or other concentrated loads. For box girders, bearing stiffeners should preferably not be used on inclined webs. Rather, bearing stiffeners for box girders should be placed on the diaphragms so that the stiffeners are perpendicular to the sole plate. If the web or diaphragm should be restrained by concrete, bearing stiffeners may not even be required as long as adequate shear connection is provided to transfer the reaction or concentrated load between the concrete and steel elements. In this case, however, another load path should be provided through the steel because the concrete is likely to creep around the shear connectors.

The bearing stiffeners can either be milled-to-bear or connected to the flange with full-penetration groove welds. When the stiffener plates are milled to fit tightly against the flange, fillet welds are normally used. The welds must be designed to transmit the entire end reaction to the bearing in shear. For the case where the stiffeners are milled-to-bear, the calculated bearing stress due to the factored loads at the ends of the bearing stiffeners is not to exceed $1.35F_y$, which is the bearing strength specified for milled surfaces in the AISC LRFD specifications (*43*). In all cases, the provisions require the final attitude of the bearing stiffeners to be vertical. Rotation of the girder due to dead load about either axis at the bearing may cause the bearing stiffeners to be loaded unevenly. When these rotations become significant, the girder can be cambered for rotation about one or both axes to help ensure more uniform bearing.

The Recommended Specifications also require that the width-to-thickness ratio of the bearing stiffeners satisfy the proposed Equation (6-13), which is the same requirement that is specified for transverse and longitudinal web stiffeners. Finally, the provisions also require the material grade of bearing stiffeners to be the same as or less than for the web to which the stiffeners are attached.

7. Shear Connectors

7.1 General

In the Recommended Specifications, the provisions for the design of shear connectors for ultimate strength are based on the provisions given in the AASHTO Standard Specifications. The provisions for the design of shear connectors for fatigue are based on the provisions given in the AASHTO LRFD Specifications. In each case, however, modifications are made to consider the radial forces that develop on shear connectors in curved bridges.

According to the Recommended Specifications, shear connectors must be provided in negative moment regions. Current AASHTO provisions do not require the use of shear connectors in negative moment regions unless the longitudinal reinforcement is considered when computing the section properties. If the longitudinal reinforcement is not considered, shear connectors may be omitted in the negative moment regions. In this case, a defined number of additional anchorage connectors must be provided at points of dead-load contraflexure and the longitudinal reinforcement must be extended a given distance beyond the anchorage connectors. The anchorage connectors should be placed within a distance equal to one-third of the effective deck width on either side or centered about each point of dead-load contraflexure.

When moving loads are considered, the points of dead-load contraflexure have little meaning. Also, when the shear connectors are discontinued, all of the force in the deck is transferred into the steel at one location, causing large forces in the anchorage connectors and an increasing tendency for the deck to crack at the point where the shear connectors are discontinued (unless the bond between the top flange and deck is adequate). There are no known failures of shear connector welds to tension flanges in cases where studs have been provided along the entire length of the girder. However, fatigue failures at shear connector welds have been reported at locations where the shear connectors have been terminated at the dead-load points of contraflexure. Shear connectors can help to control the cracking in concrete subject to a net tensile stress when longitudinal reinforcement is also present. Finally, shear connectors must be present along the entire span to resist the torsional shear that exists in both positive and negative moment regions of curved girder flanges in order to avoid possible debonding of the deck.

7.2 I-Girders

7.2.1 Strength. The proposed Equation (7-1) defines the minimum number of shear connectors required for strength between the point of maximum positive live-load moment and the adjacent end of the girder. The maximum point of *live-load* moment is used because it is easier to locate than the maximum of the sum of the dead- and live-load moments acting on the composite section and because the live-load moment applies directly to the composite section. The pitch of the shear connectors should be uniform in this region. To satisfy equilibrium, the force, P, in the slab at the point of maximum live-load moment should be computed according to the proposed Equation (7-2) as the vector sum of the longitudinal and radial components of the slab force. Equation (7-2) supplants a more complex empirical approach given in the current Guide Spec for designing the shear connectors for strength. The longitudinal slab force is computed as the smaller of the longitudinal forces in the girder and the deck computed from proposed Equations (7-3) and (7-4), respectively. The deck force given by Equation (7-4) is computed by using the full effective width of the concrete defined in Article 4.5.2. The resulting longitudinal force is conservatively assumed to be constant over the entire arc length within this region. The radial force in the slab for this region is computed from the proposed Equation (7-5), which is simply taken as the radial component of the longitudinal force.

The minimum number of shear connectors required for strength between the point of maximum positive live-load moment and adjacent points of greatest negative live-load moment is also given by Equation (7-1). In this case, the longitudinal slab force that should be used in determining the resultant force, P, according to the proposed Equation (7-6), should be the *sum* of the critical longitudinal slab forces at the maximum positive and negative moment locations. In positive moment regions, the longitudinal forces on the shear connectors act in one direction. In negative moment regions, the forces act in the opposite direction. In areas of stress reversal, which are not well defined, the shear connectors must resist longitudinal forces acting in both directions. To ensure adequate capacity for any possible live-load position, the shear connectors are conservatively designed for the sum of the critical slab forces at the point of maximum positive live-load moment and at the point of greatest negative live-load moment according to the proposed Equation (7-7).

The critical longitudinal slab force at the point of greatest negative live-load moment is taken as the smaller of the longitudinal forces in the girder and the deck given by proposed Equations (7-8) and (7-9), respectively. In the Standard Specifications, only the longitudinal reinforcement is considered in computing the critical longitudinal deck force at this point. The contribution of the concrete that is effective in tension is ignored completely. Here, the longitudinal tension force in the deck is defined in Equation (7-9) as 45 percent of the specified minimum 28-day compressive strength of the concrete multiplied by the full effective concrete width given in Article 4.5.2 multiplied by the average concrete thickness. Equation (7-9) is a somewhat conservative approximation to account for the combined contribution of both the longitudinal reinforcing steel and the concrete that remains effective in tension based on its modulus of rupture. It was desired to be conservative in this region to ensure that sufficient shear connectors are provided to resist both the torsional shears, which develop in the full composite section along the entire span, and the horizontal shears in curved girder flanges. The radial force in the slab for this region is computed from the proposed Equation (7-10).

7.2.2 Fatigue. The required shear connector pitch for fatigue should be computed according to the provisions of AASHTO LRFD Article 6.10.7.4.1b. According to the Recommended Specifications, the required pitch should be computed at all supports and at each cross frame. The horizontal shear range in the flange, V_{sr}, to be used in determining the required pitch according to AASHTO LRFD Equation (6.10.7.4.1b-1) should be computed from the proposed Equation (7-11) in the Recommended Specifications. The resultant horizontal shear range computed from Equation (7-11) is the vector sum of the longitudinal and radial fatigue shear ranges per unit length of flange.

The longitudinal fatigue shear range is the flexural shear due to the fatigue truck defined in Article 3.5.7.1. The maximum longitudinal fatigue shear range is produced by placing the fatigue truck immediately to the left and to the right of the point under consideration. The radial fatigue shear range results from the effects of curvature, skew and other conditions that may cause torsion. The maximum radial fatigue shear range is usually produced by placing the fatigue truck at the position to produce the greatest positive or negative vertical bending moment in the girder. Therefore, a vector addition of the two shear range values is conservative because the maximum longitudinal and radial shear ranges are not usually produced by concurrent loads.

Article 4.5.2 of the Recommended Specifications requires that the uncracked section be used at the fatigue limit state in both positive and negative moment regions. Some of the primary reasoning behind this assumption was discussed earlier. In addition, the horizontal shear force in the deck is considered to be effective in the analysis. The deck must be positively connected to the steel girders to satisfy this assumption. Therefore, in determining the required pitch of the shear connectors for fatigue, the uncracked section should be used in determining the shear force ranges along the entire span. The full effective deck width is also used according to the provisions of Article 4.5.2. Shear connectors are designed for shear and not moment. As mentioned, the maximum longitudinal fatigue shear range is produced by placing the fatigue truck immediately to the left and to the right of the point under consideration. The influence line for moment shows that for the load in this position, positive moments are produced over much of the girder length. As a result, the concrete deck is in compression over much of the girder length for this shear loading and the use of the uncracked section along the entire span is deemed to be reasonable.

The radial fatigue shear range in the flange due to the effects of curvature between brace points can be computed from the proposed Equation (7-12). The force is taken as the radial component of the maximum longitudinal range of force in the flange between brace points. The resulting radial force is then distributed over an assumed effective length of deck, w, on either side of the cross frame. The effective length of deck is arbitrarily taken as 48 in., except at simple supports where the value of w is halved. The proposed Equation (7-13) is also provided to compute the radial fatigue shear range in the flange. In this case, the radial shear range is determined as the net range of radial force in the cross frame at the top flange divided by the effective length of flange, w. The larger value from either Equation (7-12) or (7-13) should be used. The two equations yield approximately the same results when there are no other sources of torsion besides curvature. When other sources of torsion, such as skew, start to predominate, Equation (7-13) will typically govern. These equations are included to ensure that a load path is provided through the shear connectors to satisfy equilibrium at a transverse section through the girders, deck, and cross frames.

7.3 Closed Box and Tub Girders

7.3.1 General. Shear connectors for top flanges of closed box and tub girders should be designed for both strength and fatigue according to the provisions for I-girders discussed previously, except as detailed below. The shear connectors for composite bottom flanges of closed box and tub girders must be sufficient to develop the flexural and torsional shears used to design both the steel bottom flange and the flange concrete. The connectors should be distributed uniformly across the box flanges, as specified in Article 10.4.3.5. The shear connectors should also be spaced to help prevent local buckling of the flange plate when it is in compression after the deck has hardened.

7.3.3 Fatigue. In computing the required pitch of the shear connectors on the top flange of a closed box or tub girder for fatigue, the longitudinal fatigue shear range is to be computed for the flange over the web subjected to additive flexural and torsional shears due to the fatigue truck. The maximum flexural and torsional shears are typically not produced by concurrent loads. For top flanges of closed box sections, one-half of the flange width should be considered to be acting with the critical web. The top flange over the other web (or the other half of the flange for a closed box section) is to contain an equal number of shear connectors. Because of the conservatism inherent in these requirements, the radial fatigue stress range given by either Equation (7-12) or (7-13) does not need to be included in computing the horizontal shear range for closed box and tub girders.

8. Bearings

8.1 General

According to the Recommended Specifications, the effects of support restraint should be recognized in the analysis. Modeling of the actual support restraints is particularly important for thermal analyses. When two bearings are used under a box girder, the two vertical reactions together provide a resisting torque, which should be comprehended in the analysis. The horizontal forces that must be transmitted to the substructure due to the effects of friction within bearing devices and all other sources should also be considered in the design of the bearing device and superstructure. The provisions permit the superstructure to be supported either on bearing devices or through direct attachment to the substructure or pier cap beam as an integral connection.

8.2 Forces

Bearing devices should be designed to transmit vertical forces and horizontal (both longitudinal and transverse) forces due to factored design loads and factored construction loads. Horizontal forces on the bearings are often created by the lateral and tangential restraints provided to the structure. Bridge structures, especially curved bridge structures, tend to move transversely as well as longitudinally. Relatively wide curved and skewed bridges can undergo significant diagonal thermal movements that can result in either large transverse movements or large transverse forces if the movement is restrained.

At integral pier caps and abutments, girder rotations are restrained. Thus, adequate shear connection must be provided between the girders and the substructure to transmit the shear forces that are developed from the girder to the integral pier or abutment.

Transverse or longitudinal prestressing of concrete decks or steel girders can significantly affect the vertical reactions because of the eccentricity of the prestressing forces, which creates restoring forces. Box girders resting on two bearings are particularly sensitive to the effects of transverse prestressing of concrete decks; the vertical reactions are significantly affected by transverse stressing of the deck.

8.3 Movements

Bearing devices should be designed to accommodate movements due to temperature changes in the superstructure and to accommodate rotations about the tangential and radial axes of the girder. Thermal forces due to a uniform temperature change will not be generated if the bearings are oriented to permit free translation along rays emanating from a single point. Any other orientation will induce thermal forces into both the superstructure and substructure. Practical considerations, however, often do not allow the bearings to be oriented in such a manner.

Girder rotations can occur about both the tangential and radial axes, particularly in skewed bridges. Rotation about the tangential axis of the girder is usually minimal and can often be ignored in the design of the bearing, unless the bridge is heavily skewed. Rotations that occur during construction should be considered when determining the design rotation capacity of the bearing because unanticipated rotations or eccentric loading of the bearings during construction may lead to premature bearing failure.

8.4 Bearing Replacement

According to the Recommended Specifications, a provision should be made for future jacking of the structure to accommodate replacement of the bearings, as directed by the owner. The owner may feel that a provision for future jacking during the initial design process is more efficient and cost-effective than retrofitting the structure at some future time to accommodate jacking. The design should ensure that excessive torsion and uplift is avoided when the structure is jacked.

9. I-Girders

9.1 General

According to the Recommended Specifications, horizontally curved I-girders can be either welded or rolled shapes. The webs must be attached at the midwidth of the flanges and the flanges can be either compact or noncompact.

This article also specifies recommended minimum flange proportions, which are not given in the current AASHTO specifications. Compression-flange widths should not be less than 0.2 times the girder depth and never less than 0.15 times the girder depth. Flange thicknesses should preferably not be less than 1.5 times the web thickness. If the compression flange of the girder is smaller than the tension flange, the minimum flange width may be based on a web depth of $2D_c$ rather than the girder depth. These proportions are recommended to help ensure that the web is adequately restrained by the flanges to control web bend-buckling. These proportions were developed based on a study by Zureick and Shih (*44*) on doubly symmetric straight girders, which clearly showed that the web buckling capacity was dramatically reduced when the compression flange buckled prior to the web. Although Zureick and Shih's study was limited to doubly symmetric girders, the recommended minimum flange proportions are considered adequate for reasonably proportioned singly symmetric I-girders. The advent of composite design led to a significant reduction in the size of compression flanges in positive moment regions. These smaller flanges are the most likely to be governed by the recommended limits. Providing minimum compression flange widths that satisfy the recommended limit in these regions will help to ensure a more stable girder that is easier to handle.

9.2 Variable Depth Girders

Variable depth I-girders were not directly considered in the development of the curved-girder strength equations in the Guide Spec or in the Recommended Specifications. If only the flexural stiffness of a variable depth I-girder is input into the analysis, the vertical shear component of the inclined bottom flange force is not computed. As a result, the actual force (and stress) in the inclined flange is underestimated and the web shear is overestimated. With the advent of refined analysis techniques, variable depth I-girders can be modeled more accurately and the effects of the vertical flange shear force can be considered.

Therefore, the provisions require the engineer to consider the vertical component of the force in the inclined bottom flange of variable depth girders. If the flange inclination is large, the resultant force along the longitudinal axis of the flange can become significant. The horizontal component of the resultant force balances the horizontal force in the top flange. Although the vertical component of the flange force reduces the web shear, this benefit is conservatively not recognized in the provisions.

Where the inclined flange becomes horizontal, the vertical shear component in the flange is transferred from the bottom flange back into the web, which may require additional local stiffening of the web at that location and which may also cause an increased stress in the flange-to-web welds (*45*). The provisions require that the vertical shear be considered in the design of the web and the web-to-flange welds at changes in the inclination of the bottom flange.

9.3 Cross Frames and Diaphragms

9.3.1 General. Cross frames and diaphragms often transfer significant loads in horizontally curved I-girder bridges.

Thus, cross frames and diaphragms in curved I-girder bridges should be considered primary members, should contain both top and bottom chords, and should be as close as practical to full-depth of the girders. Eccentricities between the cross frame or diaphragm and the girder flanges are to be recognized in both the analysis and the design. A provision must be made to transfer the net horizontal force in the cross frame or diaphragm to the girder flanges through the connection plates.

The effective length factor, K, for cross frame members in compression should be 1.0 for single-angle members and 0.9 for all other types of members. Article 10.54.1.2 of the Standard Specifications currently specifies a K factor of 0.75 for compression members with bolted or welded end connections that are laterally supported in both directions at their ends. Bracing members are different from typical compression members in that their ends are likely to translate and rotate with respect to each other causing an increase in K. Single angles, in particular, are loaded through one leg and are subject to eccentricity and twist, which is typically not recognized in the design. The most realistic tests available on the behavior of single-angle members have been transmission tower tests. The results of these tests are in an ASCE manual (*46*), which covers the design of these structures. This document contains a complex approach for determining the K factor for single-angle members. A K value of 1.0 was deemed to be a reasonable conservative approximation of the effective length factor calculated for these members by using the more complex procedures. For other types of members that have a more predictable behavior, a K value of 0.9 is recommended in the provisions. Bracing members in compression should be designed according to the load factor design provisions of Article 10.54 of the Standard Specifications. Bracing members in tension should be designed according to the provisions of Article 10.18.4 of the Standard Specifications by using the effective net area defined in Article 10.9.

Solid-plate diaphragms with span-to-depth ratios greater than 4 can be designed by using the elementary beam theory. According to the ACI Code for reinforced concrete members, beams deeper than one-fourth of their length must be designed as deep beams. In deep beams, shear deformations must be considered. Finite-element studies have confirmed that shear deformations are important for diaphragms with a depth greater than one-fourth of the span. Thus, the Recommended Specifications require shear deformations to be considered when the span-to-depth ratio of a diaphragm is equal to 4 or less.

9.3.2 Arrangement. Intermediate cross frames or diaphragms in curved bridges should be spaced as nearly uniform as possible to ensure that the flange strength equations given in Section 5 of the Recommended Specifications are appropriate. The research that led to the development of these equations assumed a uniform spacing of cross frames. Intermediate cross frames or diaphragms should also be spaced to control lateral flange bending stresses and to provide stability to the girder. Equation (C9-1) is provided in the commentary to this article to assist in determining a preliminary cross frame or diaphragm spacing to limit lateral flange bending stresses to a desired level. The spacing of intermediate cross frames or diaphragms should not exceed 30 ft. This limit is reduced to 25 ft for bridges where the effects of curvature can be neglected in determining the vertical bending moments, according to Article 4.2 of the Recommended Specifications. The preceding requirement ensures that the maximum spacing does not exceed the maximum spacing limit of 25 ft specified for tangent bridges in the Standard Specifications.

For bridges with radial supports, the intermediate cross frames should preferably be placed in contiguous radial lines. When the supports are skewed not more than 20 deg from radial, the intermediate cross frames or diaphragms may be placed in contiguous skewed lines parallel to the skewed supports, which is consistent with Article 10.20.1 of the Standard Specifications. When cross frames or diaphragms are skewed, the connection plates should preferably be oriented in the plane of the cross frame or diaphragm to prevent undue distortion of the connection plate. When the supports are skewed more than 20 deg from radial, the intermediate cross frames or diaphragms may be placed in radially staggered patterns. Skewed cross frames become more difficult to fabricate when the angle from radial exceeds 20 deg. Welding of skewed connection plates becomes more difficult at locations where the plate forms an acute angle with the girder. As the skew angle increases, the components of the cross frame forces tangent to the girder also become more significant. The use of staggered cross frames generally results in lower cross frame forces than if a contiguous arrangement is used. There is, however, a concomitant increase in the lateral bending of the girder flanges with the use of staggered cross frames. The determination of the resulting lateral flange bending moments must be examined with a refined analysis in this case. The lateral flange bending moment that is determined from the approximate Equation (4-1) in Article 4.2.1 of the Recommended Specifications reflects only curvature effects on flanges supported by uniformly spaced braces arranged in a contiguous pattern.

At the ends of the bridge, cross frames or diaphragms should be provided to support the deck and expansion devices. Along skewed interior support lines, cross frames or diaphragms need not be used, which differs from the provisions in the current Guide Spec. The Guide Spec requires contiguous rows of diaphragms or cross frames because the analysis tools available at that time were based on that assumption. Large skews were not properly treated at the time. Allowing the elimination of a row of cross frames along an interior skewed support line is a reflection of modern practice and simplifies detailing because radial cross frames will not have to intersect the skewed row. Although cross frames are not present along the support line, cross frames or diaphragms are required to transfer radial forces from the superstructure to bearings that are provided with radial restraint.

9.3.3 Load Effects. As primary members in horizontally curved I-girder bridges, cross frames and diaphragms should be designed for the load effects due to the factored loads as determined from an appropriate analysis on the system behavior of the bridge. For bridges where the effects of curvature can be neglected in determining the vertical bending moments, according to the provisions of Article 4.2 of the Recommended Specifications, the cross frame or diaphragm forces can be estimated from a V-load analysis of the bridge or by other rational means.

The provisions also state that an effective length, w, of hardened concrete deck defined in Article 7.2.2 may be considered to act compositely with the cross frame or diaphragm for loads applied after the deck hardens. As discussed previously, w is arbitrarily specified as 48 in., except at simple supports where w is 24 in. To satisfy equilibrium, the entire transverse section, including the deck and cross frame, acts together to transfer the loads between composite girders. The composite action of the deck increases the effective depth of the cross frame and, therefore, its effective stiffness. In curved bridges without skewed supports, large transverse shear forces between the deck and steel girder (that are transferred through the shear connectors) do not typically occur. When the skew is large, however, the transverse shears may become significant and both the transverse reinforcing and shear connectors should be checked for this additional force.

9.4 Flange Lateral Bracing

Bottom flange bracing creates a pseudo-closed section formed by the I-girders, bracing, and hardened concrete deck. When the bracing is added, cross frame forces tend to increase because the cross frames act to retain the shape of the pseudo-closed section. The bending moments in the girders are also affected because the outside girder moment and deflection are reduced, while the moment and deflection in the adjacent interior girder are typically increased with the addition of bottom flange bracing. A refined analysis must be used to recognize these effects. The bracing members carry significant forces and must be designed as primary members for the load effects determined from the analysis. The bracing is preferably placed in the plane of the girder flanges.

In cases where the curvature of the bridge is very sharp and the use of temporary shoring is not practical, use both top and bottom flange lateral bracing to provide greater stability to curved I-girder pairs during construction. When both top and bottom flange bracing is provided in this manner, it is not necessary to provide the bracing over the entire girder length.

9.5 Permanent Deflection

To control permanent deflections, Article 10.57 of the AASHTO Standard Specifications limits flange stresses under $D + 5/3(L + I)$ (overload) to $0.95F_y$ and $0.80F_y$ at composite and noncomposite sections, respectively. The live load is assumed to be placed in all design lanes in this check in the Standard Specifications. These limits are based on the results of the AASHO Road Test (*32*), in which permanent sets in composite and noncomposite bridges were measured after several hundred thousand passages of a single lane of heavy vehicles.

In the Recommended Specifications, average flange stresses in I-girders under overload are subject to the same limits. Unlike the current Guide Spec, lateral flange bending stresses at brace points are not included in this check because these stresses occur over only a small portion of the flange and should not significantly contribute to the permanent set. However, for partially braced compression flanges, the average flange stress at overload is limited to the critical flange stress for noncompact flanges given by the proposed Equation (5-8) in Article 5.2.2 to prevent the possibility of lateral buckling of the partially braced flange at overload. Also, the overload stress in other primary steel members is limited to the specified minimum yield stress of the member.

Because these checks fall under the serviceability limit state, the Recommended Specifications state that the uncracked section should be used to compute the bending stresses in composite sections. When the uncracked section is used in negative moment regions, the neutral axis is typically nearer to the tension flange causing more than half of the web to be in compression and increasing the possibility of web bend-buckling. Therefore, the Recommended Specifications also limit the maximum longitudinal compressive stress in the web at overload to the web bend-buckling stress according to the provisions of Article 6.

9.6 Fatigue

According to the Recommended Specifications, base metal and details subjected to a net tensile stress under dead load plus two times the fatigue truck are to be designed for fatigue. The provisions of AASHTO LRFD Article 6.6.1 should apply, except as modified in Articles 2.3 and 3.5.7 of the Recommended Specifications. The uncracked section should be used to compute bending stresses in composite girders due to the fatigue loading for reasons discussed previously. At points where attachments are welded to I-girder flanges, such as cross frame connection plates and possibly gusset plates for lateral bracing, the flange must be checked for the average stress range plus the lateral flange bending stress range at the critical transverse location on the flange. Flange-to-web welds, obviously, need only be checked for the average stress range because the welds are near the midwidth of the flange. Flange butt welds need to be checked for lateral bending in addition to the average stress range.

Distortion-induced fatigue resulting from through-thickness bending at the termination of cross frame connection plate welds to the web is controlled by rigidly attaching the connection plate to both flanges and designing the connection to transfer the net horizontal cross frame force from the connection plate to the flanges. At connections of cross frames, diaphragms, and lateral bracing, the stress range in the member due to the axial force plus any bending moments induced by eccentricities in the connection is to be checked.

10. Closed Box and Tub Girders

10.1 General

Closed box and tub girders provide a more efficient cross section for resisting torsion than I-girders. As such, box girders are particularly advantageous for curved structures because of the high torsional resistance and their ability to resist the applied torsion, often without the extensive use of diaphragms between the girders and without the creation of significant longitudinal warping stresses. Box girders provide a continuous uninterrupted exterior surface, which is easier to paint and maintain and less susceptible to corrosion. The increased torsional resistance of a closed composite steel box girder also results in an improved lateral distribution of loads in much the same manner as when bottom flange bracing is used in conjunction with I-girders.

For curved structures, warping or lateral flange bending stresses are lower in box girders once the section is closed because the warping torsional stiffness of a closed box girder is much smaller than the warping torsional stiffness of an I-girder. Distortional warping stresses can occur in box girders; however, these stresses can be controlled with interior cross frames spaced to reduce the distortion. Overall, less material needs to be added to box girders to resist torsional effects. Erection costs for box girders may be less than for I-girders because erection of one box in a single lift is equivalent to placement and connection of two I-girders, which may require additional bracing or temporary supports to control stresses and deflections. Box girders are also inherently more stable during erection, which makes them easier to erect under difficult conditions.

In the Standard Specifications, Article 10.51 applies rather restrictive cross-sectional limitations to tangent box-girder bridges, including the limitation that two or more single-cell boxes must be provided. These limitations resulted from research that developed the empirically based formula for live-load distribution for bending moment in tangent multiple-box sections given in Article 10.39.2 (*47*). For tangent multiple-box sections falling within these limitations, torsional effects are ignored. Because the Recommended Specifications require a rational analysis of an entire curved superstructure that comprehends torsional effects, the use of live-load distribution factors is not required and similar cross-sectional limitations need not be applied. As a result, greater flexibility is provided in developing more economical cross-sectional configurations. For example, the use of single, or widely spaced, box girders, which are often found to be very economical structures, is implicitly permitted by the provisions.

Webs of closed box and tub girders should be designed according to the provisions of Article 6 of the Recommended Specifications. At all limit states, the web shear should be the sum of the flexural and torsional shears. Because cross-sectional distortion is limited by the use of interior cross frames or diaphragms, torsion is mainly resisted by St. Venant torsional shear flow. In a single box section, the total shear in one web is greater than the total shear in the other web because the St. Venant torsional shears are of opposite sign in the two webs. As a practical matter, both webs should be designed for the critical shear. Although shear and longitudinal stresses in the section due to warping are not zero, these effects are quite small and can be ignored in the design of the webs.

The use of inclined webs reduces the width of the bottom flange to provide for greater efficiency. The inclination of the webs relative to a plane normal to the bottom flange is not to exceed 1 to 4. For inclined webs, the web must be designed for the component of the vertical shear force in the plane of the web. When the deck is super-elevated, it is preferable to rotate the entire cross section into the super-elevated position to match the deck slope to simplify the fabrication by maintaining symmetry of the girder sections. When box webs are inclined, a transverse force is induced into the flanges equal to the shear in the web times the sine of the angle the web makes with an angle perpendicular to the flange. The change in this transverse force plus the change in the St. Venant torsional shear per unit length along the girder act as a uniformly distributed transverse load on the girder flanges and should be considered.

On tub girders, top flanges should be braced with lateral bracing that is capable of resisting the torsional shear flow that exists prior to hardening of the deck. According to the Recommended Specifications, top flanges of tub girders should be designed at the constructibility limit state, according to the provisions of Article 5, Flanges with One Web, for the factored dead and construction loads prior to hardening of the deck. The panel length of the top flanges should be defined as the distance between interior cross frames or diaphragms. Top flange bracing attached at points other than where cross frames or diaphragms exist is not considered to be an effective brace for the top flange because this connection does not adequately restrain deflection of the flange.

10.2 Framing

10.2.1 Bearings. The bearing arrangement dictates how the torsion is to be resisted at the supports of closed box and tub girders. In multiple-box sections, when the boxes are each supported on single bearings, the torque is typically removed from each box through diaphragms between the boxes. Two bearings under each box provide a couple to resist the torque in the box. Double bearings can be located between the box webs or outboard of the box if uplift is a problem. Integral steel or concrete cap beams have been used with box girders in lieu of bearings.

10.2.2 Internal Bracing and Diaphragms. 10.2.2.1 General. The provisions of Articles 9.3 and 9.4 for the design of cross frames, diaphragms, and flange bracing for I-girders should be used to design these elements for the interior of closed box and tub girders, except as modified in Article 10.2.2. A minimum of two bolts should be used in each connection of the bracing members.

10.2.2.2 Internal Diaphragms at Supports. An internal diaphragm is required inside box girders at each support. The connection of the diaphragm to the box must be adequate to transmit the computed bending and torsional shears between the diaphragm and the box. Support diaphragms are subject to bending stress and torsional shear stress. Therefore, the principal stresses in the diaphragm must be computed and should not exceed the critical stress given by the proposed Equation (10-2), which is based on the von Mises yield criterion for combined normal and shear stress and is discussed in more detail later.

The diaphragms may be made of steel or concrete. The potential use of concrete diaphragms is currently not recognized by AASHTO. The design of concrete diaphragms has been performed in the past, but as of this writing, it is believed that box-girder bridges using concrete diaphragms have not been constructed in the United States. The diaphragms should preferably bear against the bottom flange, unless composite box flanges are used (discussed later); in which case, the diaphragms should preferably permit the concrete to be contiguous through the diaphragm for a distance not less than one-half the bottom flange width. When access holes are provided in the diaphragm, a refined analysis of the diaphragm is desirable because of the complex stress state around the hole. Finally, as discussed previously, thermal movements of the bridge may cause the diaphragm to be eccentric to the reaction at the bearing. This eccentricity should be considered when designing the diaphragm and the bearing stiffeners.

10.2.2.3 Intermediate Internal Bracing. Tests have shown that the cross section of steel box girders distorts when subjected to torsional loads (*48*). The distortion of the box cross sections results in transverse bending stresses in box flanges and webs due to changes in the direction of the shear flow vector in addition to longitudinal warping stresses resulting from the resistance to out-of-plane warping of the section. The transverse bending stiffness of the flanges and webs alone is not sufficient to retain the shape of the box. Therefore, intermediate internal bracing consisting of cross frames or diaphragms is necessary to maintain the box shape and to reduce the transverse bending and longitudinal warping stresses in the box girder. Intermediate internal bracing also helps to control lateral flange bending stresses in the top flanges of tub girders prior to hardening of the deck.

In the Recommended Specifications, the spacing of the internal bracing is determined so that the total longitudinal warping stress in the box (caused by both torsion and distortion) does not exceed 10 percent of the longitudinal stress due to vertical bending at the strength limit state. The spacing, therefore, is not to exceed 30 ft. By limiting the longitudinal warping stress to 10 percent of the longitudinal stress due to vertical bending, warping does not need to be considered in the design of the flanges at the strength limit state. Also, the full torsional stiffness of the box is adequate to permit classical strength of materials assumptions to be used to compute the distribution of loads within the box. The transverse bending stresses and longitudinal warping stresses in the box flanges and web must be computed by a rational approach that considers the reduction in torsional stiffness due to the distortion of the box cross section. The beam-on-elastic foundation (BEF) approach (*49,50*) is most commonly used. The use of finite-element analysis is quite problematic for determining through-thickness transverse bending stresses. Transverse stiffeners that are attached to the web or box flange are effective in resisting the transverse bending. The transverse bending stresses are not to exceed 20 ksi at the strength limit state, which is a limit retained from the Guide Spec. The engineer should be aware that through-thickness transverse bending stresses resulting from cross-sectional distortion must also be considered in checking flange-to-web fillet welds on box girders for fatigue according to Article 10.6.1. Longitudinal warping stresses also must be considered when checking fatigue. Therefore, a rational analysis to determine these stresses should also be conducted for checking the fatigue limit state.

As part of the internal bracing, transverse bracing members are required across the bottom and top of the box or tub at internal cross frame locations to assist in retaining the shape of the box. The cross-sectional area and stiffness of these members should not be smaller than the area and stiffness of the diagonal members. The transverse bracing member must be attached to box flanges (top or bottom) to better control the transverse distortion of the flange, unless longitudinal flange stiffeners are used or the flange is composite. If longitudinal flange stiffeners are used, the transverse member must be attached to the stiffener(s) by bolting to ensure that fatigue cracks do not originate at the discontinued fillet welds. Reasonable estimates of stresses at this detail are difficult to predict accurately. If the bottom flange is composite, the transverse bracing member is to be located above the concrete.

10.2.3 External Bracing. External cross frames or diaphragms are required between curved boxes at end supports to resist torsion and twist. Diaphragms do not need to be continuous through the box webs. At internal supports and at intermediate locations between supports, external cross frames or diaphragms can also be used. At all external brace locations, there must be opposing bracing placed on the inside of the box to receive the forces.

At locations other than support points, permanent external bracing is usually unnecessary. If an analysis indicates that the boxes will twist excessively when the deck is cast, temporary external bracing may be desirable. The effects of removal of the temporary bracing, however, should be considered. These effects include an increase in the deck stresses.

If the span-to-depth ratio of an external diaphragm is less than or equal to 4.0, the principal stresses in the diaphragm should be computed and should not be less than the critical stress given by the proposed Equation (10-2), which is discussed later.

10.2.4 Bracing of Tub Flanges. Top flange bracing internal to tub girders must be provided to help control twist and distortion. A Warren-type configuration is preferred for the bracing. The bracing must have adequate capacity to resist the St. Venant torsional shear flow in the noncomposite section at the constructibility limit state. Once the concrete deck hardens, the bracing is less effective because the deck usually has a much greater shear stiffness.

The bracing should preferably be connected to the top flanges or located as close as practical to the plane of the top flanges with provision made to transfer the bracing forces to the flanges. The bracing should also be continuous across field splice locations. The torsional analysis of a noncomposite tub girder should recognize the true location of the bracing in computing the enclosed area of the box section.

10.2.5 Access Holes. Outside access holes in box sections should preferably be located in the bottom flange in areas of low stress. The flange at access holes should be reinforced as necessary. Covers or screens should be provided over outside access holes to prevent entry by birds, animals, and unauthorized persons.

10.3 Stress Determinations

10.3.1 General. The longitudinal stresses in box flanges can be assumed to be uniform across the flange for the strength, serviceability, and constructibility limit states because the flange warping stress is limited to 10 percent of the vertical bending stress at the strength limit state, according to the provisions of Article 10.2.2.3. Longitudinal warping stresses must be considered, however, at the fatigue limit state as presently required in the Guide Spec. Shear due to vertical bending and torsion should be considered at all limit states.

For box flanges (in tension or compression) wider than one-fifth of the distance between points of contraflexure or between a simple support and a point of contraflexure, shear lag should be considered. The Standard Specifications currently require that shear lag only be considered for tension flanges of box girders. Box flanges in compression are susceptible to the effects of shear lag as well.

10.3.2 Variable Depth Girders. Variable depth box and tub girders are permitted by the Recommended Specifications. As discussed earlier for variable depth I-girders (Article 9.2), the provisions require the engineer to consider the vertical component of the force in the inclined bottom flange of variable depth girders. The force induced into the web and the flange-to-web welds at changes in the inclination of the flange should also be considered. The inclination of the webs should remain constant, which requires that the width of the bottom flange vary along the length, that the distance between the webs at the top of the box or tub be kept constant, and that the web heights be kept equal to simplify the fabrication and the analysis.

10.4 Strength of Box Flanges

10.4.1 General. This article defines the critical stress for noncomposite and composite box flanges. A box flange is a flange that is connected to two webs. A box flange can be either a flat unstiffened plate, a flat stiffened plate, or a flat plate with reinforced concrete attached to the plate by shear connectors. A closed box girder has a box flange on the top and the bottom of the section. A tub girder has a box flange on the bottom of the section only. According to the Recommended Specifications, the computed average longitudinal stress in a box flange should not exceed the critical stress defined in this article, unless shear lag is considered, in which case the longitudinal stress at the webs should not exceed the critical stress.

10.4.2 Noncomposite Box Flanges. 10.4.2.1 General. Noncomposite top flanges of closed box sections should be designed for the weight of the fresh deck concrete and any other temporary or permanent loads placed on the flange by assuming that the flange acts as a simple span between the two webs. The maximum vertical deflection of noncomposite top or bottom box flanges under self-weight and applied permanent loads should not exceed $\frac{1}{360}$ of the span. This limit, which has its origins in building construction in an attempt to keep plaster from cracking, is specified in these provisions to minimize the distortional strains in the flange for both aesthetic and structural reasons. At the constructibility limit state, it is also specified that the transverse through-thickness bending stress in noncomposite box flanges not exceed 20 ksi, which is a limit retained from Article 1.27(B) of the Guide Spec. To satisfy these deflection and stress limits, transverse and/or longitudinal stiffening of the box flange may be required.

At support diaphragms, a width of the box flange equal to 18 times its thickness can be considered to be effective with the diaphragm. This requirement is similar to the provision for the width of web or diaphragm that can be considered to be effective with bearing stiffeners. Box flanges at interior supports are subject to a biaxial stress due to vertical bending in the girder and vertical bending in the diaphragm over the bearing sole plate. Torsional shear is also present and its effect must be considered. Therefore, the provisions require that the calculated principal stress in the box flange due to vertical bending in the girder and diaphragm plus the torsional shear not exceed the critical flange stress given by Equation (10-2) at the strength limit state.

10.4.2.2 Shear. The effects of torsional shear must be considered in the design of box flanges for curved box girders or for box girders with skewed supports when torsional shear stresses are large. The St. Venant torsional shear flow in the box can be calculated from the torque using the classical Equation (C10-1) for torsion of thin-walled cross sections given in the commentary to this article in the Recommended Specifications. This equation assumes that cross-sectional deformations are limited by the provision of sufficient numbers of internal cross frames or diaphragms along the span. The torsional shear force computed from the shear flow should be combined with the shear force due to bending. The critical shear stress, F_v, for a noncomposite box flange is given by the proposed Equation (10-1). In the Guide Spec, a critical shear stress equal to the shear yield stress, or $F_y/\sqrt{3}$ is permitted. At this level of shear stress, however, there is also a significant reduction in the permitted average longitudinal flange stress. To prevent this reduction and also simplify the provisions, the critical shear stress is limited to $0.75F_y/\sqrt{3}$ in the Recommended Specifications. Such a level of torsional shear stress is rarely, if ever, encountered in practical box-girder designs.

10.4.2.3 Tension Flanges. The critical stress for noncomposite box flanges subject to tension is given by Equation (10-2) in the Recommended Specifications, which is the same equation given in the load factor design portion of the Guide Spec. Because instability or buckling is not a concern for tension flanges, the critical stress can be determined on the basis of yielding. Also, because the span length is typically large with respect to the width of the flange, it can be assumed that the shear and normal stresses are reasonably uniform across the width of the flange. Under combined uniform shear and normal stress, the von Mises yield criterion can be used to determine the stress combination required to cause first yield in the flange and is the basis for Equation (10-2). The effects of residual stress are ignored due to the fact that strain hardening can occur.

10.4.2.4 Compression Flanges. 10.4.2.4.1 Unstiffened flanges. The critical compressive stress specified for noncomposite curved box flanges in the Recommended Specifications is identical to the critical compressive stress specified for noncomposite tangent box flanges in the Standard Specifications, except that the effect of the St. Venant torsional shear stress is included for curved flanges. In fact, the equations specified for curved compression flanges in the Recommended Specifications converge to the equations specified for tangent compression flanges in the Standard Specifications as the torsional shear stress goes to zero. The equations given for unstiffened compression flanges in the Recommended Specifications are identical to the equations given for unstiffened compression flanges in the load factor design portion of the Guide Spec, except that the normal and shear stresses are in units of ksi rather than psi.

As for tangent box flanges in the Standard Specifications (where the effects of torsion are ignored), the critical compressive stress for curved box flanges is defined for three distinct regions based on the slenderness, b_f/t_f, of the plate. For unstiffened flanges, b_f is the full flange width between webs and t_f is the flange thickness. For the most slender plates, elastic buckling represented by the classical Euler hyperbola governs the behavior. For flanges under combined normal and torsional shear stress, a nonlinear interaction curve is used to derive the critical flange stress in this region. The interaction curve relates the theoretical elastic Euler buckling equations for an infinitely long plate under a uniform normal stress and under shear stress (*4*). The critical longitudinal flange stress resulting from this interaction curve is given by the proposed Equation (10-8). For stocky plates, full yielding of the plate, as defined by the von Mises yield criterion for combined normal and shear stress, can be achieved. The critical compressive stress for plates in this region is given by the proposed Equation (10-4). In between these two regions is a transition region that reflects the fact that partial yielding due to residual stresses and initial imperfections does not permit the attainment of the elastic buckling stress. As in the Standard Specifications and the Guide Spec, the critical flange compressive stress in this region is arbitrarily defined by a sine curve. This curve is given by the proposed Equation (10-6) in the Recommended Specifications. In the derivation of Equation (10-6), a residual stress level of $0.4F_y$ is assumed (*4*). The plate-buckling coefficient, k, for uniform normal stress is 4.0 and the plate-buckling coefficient, k_s, for shear stress is 5.34 for unstiffened flanges in these equations. These buckling coefficients assume simply supported boundary conditions at the edges of the flanges (*40*).

As mentioned previously, the equation to use depends on the slenderness of the flange. The limiting flange slenderness defining whether to use Equation (10-4) or (10-6) is based on the factor, R_1, given by Equation (10-5). $R_1/\sqrt{F_y}$ is defined as 0.6 times the flange slenderness where the critical elastic buckling stress given by Equation (10-8) equals the critical stress for yielding under combined normal and shear stress, as given by Equation (10-4). The limiting slenderness defining whether to use Equation (10-6) or (10-8) is based on the factor, R_2, given by Equation (10-7). $R_2/\sqrt{F_y}$ is the flange slenderness where the critical elastic buckling stress given by Equation (10-8) equals the critical stress for yielding under combined normal and shear stress given by Equation (10-4) minus the assumed residual stress level of $0.4F_y$.

10.4.2.4.2 Longitudinally stiffened flanges. As a noncomposite unstiffened box flange in compression becomes wider, the critical flange stress decreases to a level that soon becomes impractical. At some point, it is beneficial to add longitudinal flange stiffeners. The critical compressive stress for a noncomposite longitudinally stiffened curved box flange is given by the same basic equations discussed previously for unstiffened flanges. However, the spacing between the longitudinal stiffeners, b_s, must be substituted for the flange width between webs, b_f, in the equations. Different plate-buckling coefficients, k and k_s, must also be used in the equations. The plate-buckling coefficient for shear stress, k_s, for a longitudinally stiffened flange is given by the proposed Equation (10-9) (*4*). The plate-buckling coefficient for uniform normal stress, k, is related to the stiffness of the longitudinal stiffeners as defined by the proposed Equation (10-10) and can take any value between 2 and 4. If the longitudinal stiffeners are very rigid, nodes are formed at the longitudinal stiffeners and k will be at or near a value of 4. Less rigid stiffeners will yield a lower value of k and a corresponding lower value of the critical flange stress. By recognizing this phenomenon, the engineer is able to match the stiffener size to the required buckling stress rather than always providing the largest stiffener(s) required to obtain a k value of 4.

Equation (10-10) is an approximation of the theoretical solution for the minimum required buckling coefficient, k, which will ensure that the longitudinal stiffeners on a simply supported flange plate under uniform compression will remain straight when the plate buckles (*40*). This solution assumes that the plate and stiffeners are infinitely long and ignores the effect of any transverse bracing or stiffening. As a result, when the number of stiffeners exceeds two, the moment of inertia of the stiffeners required to achieve the desired k value according to the approximate Equation (10-10) increases dramatically so as to become nearly impractical. Therefore, the provisions state that the number of longitudinal flange stiffeners should preferably not exceed two. Current AASHTO specifications permit up to five longitudinal stiffeners to be used in conjunction with Equation (10-10).

In new designs where an exceptionally wide box flange is required, it could become necessary to provide more than two longitudinal flange stiffeners. There have also been bridges constructed with more than two longitudinal stiffeners. Load factor rating of these bridges becomes problematic if Equation (10-10) is employed because the longitudinal stiffeners are not likely to provide enough moment of inertia to satisfy the requirement. Thus, the commentary to this article suggests that transverse flange stiffeners be added in this case to reduce the required size of the longitudinal flange stiffeners to a more practical value. Article 10.39.4.4 of the Standard Specifications gives provisions for the allowable stress design of box flanges that are stiffened both transversely and longitudinally. These provisions are presented in an allowable stress design format and do not recognize the effect of the torsional shear. The provisions can be modified for application to curved box flanges designed according to the Recommended Specifications by modifying the equations to include the torsional shear and by dividing out the factor of safety to cast the equations in a load factor design format. Usually, the torsional shear is insignificant and can be ignored. It was decided not to perform these modifications and include these provisions in the Recommended Specifications because of their relative complexity and because the need for more than two longitudinal stiffeners on a box flange is rare.

If transverse flange stiffeners are to be added to reduce the size of the longitudinal flange stiffeners, the spacing of the transverse stiffeners should not exceed 3 times the width of the box flange in order for the transverse stiffeners to be effective. The bottom strut of the transverse interior bracing can be considered to act as a transverse stiffener if the strut satisfies the transverse stiffening requirements of Article 10.2.2.3 of the Recommended Specifications and if the stiffness of the strut satisfies Equation (10-81) in the Standard Specifications. The required moment of inertia of the longitudinal stiffeners in this case, given by Equation (10-80) in the Standard Specifications, is equivalent to Equation (10-10) of the Recommended Specifications when the number of stiffeners, n, is equal to 1 and k is equal to 4.

The Recommended Specifications require transverse stiffening of the flange at the point of maximum compressive flexural stress in the flange (regardless of the number of longitudinal stiffeners) in order to provide support to the longitudinal stiffener(s). When only one or two longitudinal flange stiffeners are provided, the proportions of the transverse stiffening need only satisfy the requirements of Article 10.2.2.3.

Longitudinal flange stiffeners must be equally spaced and must have the same yield stress as the flange to which the stiffeners are attached. The stiffeners should be included when computing the section properties of the box or tub girder. The stiffeners are best terminated at field splice locations at the free edge of the flange where the flange stress is zero. To accomplish this successfully, the splice plates must be split to allow the stiffener to be taken to the free edge of the flange where the vertical bending stress is zero. Otherwise, if the stiffener must be terminated in a region subject to a net tensile stress, then the base metal at the termination of the stiffener-to-flange weld must be checked for fatigue according to the terminus detail.

10.4.3 Composite Box Flanges. 10.4.3.1 General. This article covers the design of composite box flanges, which are defined as box flanges composed of steel with concrete attached to the steel by the use of shear connectors that are designed for flexural and torsional shear. Composite box flanges can be either the top flange of a closed box girder or the bottom flange of a tub or closed box girder. The concrete contributes to the section properties of the box and stiffens the compression flange allowing for the use of thinner steel plate.

The design of composite box flanges is currently not covered in the Guide Spec or the Standard Specifications. Top composite box flanges are rare in the United States because the cost of meeting the necessary safety requirements to work inside closed box sections has been prohibitive. A bridge with a bottom composite box flange was built in Toronto several years ago. Recently, some box-girder bridges in Colorado were designed with bottom composite box flanges, but these bridges were not constructed. Even though the use of composite box flanges has been limited, it was felt that this concept could be economically viable in certain situations; therefore, provisions were included in the Recommended Specifications to permit its use and to ensure the proper design of the flanges.

In the design of a composite box flange, torsion and torsional shear stresses should be computed using the uncracked section at all limit states. Flexural and shear forces should be proportioned to the steel and concrete by using the appropriate section properties. Creep should be conservatively ignored in determining the dead- and live-load flexural and shear stresses in the concrete but should be considered when checking the flexural compressive stresses in the steel. For all loads applied to the flange prior to the hardening of the concrete, the flange should be designed as a noncomposite box flange according to the provisions of Article 10.4.2.

10.4.3.2 Tension Flanges. Longitudinal tensile stresses in the steel should be computed by using the uncracked section at the fatigue, constructibility, and serviceability limit states. At the strength limit state, the tensile stresses in the steel should be computed by using the cracked section. The computed tensile stresses should not exceed the critical longitudinal flange stress given in Equation (10-2) based on the von Mises yield criterion.

10.4.3.3 Compression Flanges. At the strength limit state, the longitudinal compressive stresses in the steel should not exceed the critical longitudinal flange stress given in Equation (10-4) based on the von Mises yield criterion. The hardened concrete prevents local and overall bend and shear buckling of the steel flange at the strength limit state. At the strength and constructibility limit states, the compressive stress in the hardened concrete on the box flange should not exceed $0.85f'_c$.

10.4.3.4 Shear. The steel flange should be designed to carry the total torsional shear at the strength limit state. The shear stress in the flange can be computed assuming that the steel acts compositely with the concrete. The computed shear stress is not to exceed the critical flange shear stress given by Equation (10-1).

The concrete must be provided with adequate orthogonal reinforcement to resist the computed shear in the concrete and to satisfy the requirements of Article 2.4.3 at the serviceability, constructibility, and strength limit states.

10.4.3.5 Shear Connectors. Shear connectors for composite box flanges should be designed to resist the effects of the flexural and torsional shear used to design both the steel and concrete. The number of shear connectors on composite bottom box flanges between the point of greatest negative moment and the terminus of the concrete must satisfy the shear-connector strength requirements of Article 7.2.1, ignoring the term, P_p. The radial shear due to curvature should also be ignored. Instead, the torsional shear force in the concrete should be vectorially added to the longitudinal shear connector force when checking the number of shear connectors required at the strength limit state. The number of shear connectors provided at the terminus of the concrete should also be increased to satisfy the requirements of Article 10.38.5.1.3 of the Standard Specifications. Finally, the shear connectors should be checked for fatigue according to the proposed Equation (7-11), where F_{fat} is the torsional shear force in the box flange concrete.

The shear connectors are best distributed uniformly across the box flange width to ensure adequate composite action of the entire flange with the concrete. To help prevent local buckling of the flange plate after the deck has hardened when the flange is subject to compression, the transverse spacing of the shear connectors is limited by the slenderness requirement accompanying Equation (10-4), with b_f in the slenderness limit taken as the transverse distance between the shear connectors.

10.5 Permanent Deflection

For closed box and tub girders, the checks for control of permanent deformations at overload are similar to the checks described earlier for I-girders under Article 9.5. The only exception is that the longitudinal flange stress at overload in a noncomposite box flange subject to compression is not to exceed the critical longitudinal flange stress for such a flange given by either Equation (10-4), (10-6), or (10-8), as applicable. As for I-girder flanges, the lateral flange bending stresses at brace points need not be included when checking the overload stress in top flanges of tub girders because these stresses do not significantly contribute to the permanent set.

10.6 Fatigue

When checking fatigue in closed box and tub girders, the longitudinal fatigue stress range should be computed as the sum of the stress ranges due to vertical bending and warping. Although warping stresses are generally small in box sections if sufficient internal bracing is provided to control warping stresses due to cross-sectional distortion, these stresses still need to be considered for fatigue because they are highest at the corner of the box where critical welding details are located. The transverse through-thickness bending stress range at flange-to-web fillet welds due to cross-sectional distortion must also be checked. A rational method of estimating these distortion-induced warping and through-thickness bending stresses was discussed in Article 10.2.2.3.

Cross bracing and diaphragm members and their connections must also be checked for fatigue. The fatigue loading should be applied in accordance with the provisions of Article 3.5.7.2 for determining the stress range in transverse members (discussed previously). Top flange bracing is not fatigue critical because the deck is much stiffer than the bracing, and therefore, the bracing members resist relatively little live load.

11. Splices and Connections

11.1 General

Splices and connections for curved bridges should be designed according to the provisions given in Articles 10.18, 10.19, and 10.56 of the Standard Specifications for the design of connections and splices for tangent bridges, with the exceptions noted in this article.

According to the Recommended Specifications, connections of bracing members and diaphragms, which are considered to be primary members in curved bridges, should be designed for the computed factored actions in the members. Article 10.19.3.1 of the Standard Specifications currently permits end connections for cross frames and diaphragms in tangent bridges to be designed for the calculated factored member actions with no consideration of the "75 percent" rule or the "average" rule given in Article 10.19.1.1. The Recommended Specifications are simply applying similar logic to the design of these connections in curved bridges. Article 10.19.1.1 of the Standard Specifications currently states that connections for main members should be designed for not less than the average of the required strength at the point of connection and the strength of the member at the same point, but in any event, not less than 75 percent of the member's strength. In many cases, slip of a bolted connection at overload will govern the design of the connection versus strength. Also, with the increasing use of refined methods of analysis, the actions in cross frame and diaphragm members are more readily and easily determined. Designing these connections according to the 75 percent or average rules can result in excessively large connections, which, in turn, can create eccentricities that cause higher member stresses leading to the need for even larger connections. Assuming that the factored member actions have been computed by a rational approach that considers the system behavior of the entire superstructure, as required by the provisions, designing the connections of these members for those actions is reasonable and sufficient.

Girder splices in curved girders should be designed according to the latest provisions given in Article 10.18 of the Standard Specifications for the design of girder splices in tangent girders. In addition, I-girder flange splices should be designed considering the effects of lateral bending. An approach similar to the traditional approach used to design web splices for eccentric shear can be used. Box flange splices and web splices should be designed by considering the additional effects of torsional shear. Warping shall also be considered in the design of a box flange splice at the fatigue limit state and when checking for slip of bolted connections in a box flange splice at overload. Navier's hypothesis shall be assumed to determine the forces in the bolt and welds used in girder splices at all limit states. Forces in the splice plates and their connections at composite sections should be computed from the accumulated stresses according to the provisions of Article 4.5.2.

11.2 Bolted Connections

Slip of high-strength bolted connections should be checked at the overload and constructibility limit states according to the provisions of AASHTO Article 10.57.3 of the Standard Specifications. In checking the slip resistance of the connection, consideration should be given to the use of a Class B surface condition for determining the slip resistance of the faying surface wherever possible. A Class B surface condition refers to an unpainted faying surface that has been blast cleaned or to a surface that has been blast cleaned and painted with a Class B coating. Provided that the coatings have been qualified by test as required in the Standard Specifications, many commercially available primers satisfy the requirements for Class B coatings. Unpainted faying surfaces on weathering steel that have been blast cleaned also qualify as Class B surfaces.

Base metal at the gross section of slip-critical connections made with properly tightened high-strength bolts is designed for fatigue Category B. High-strength bolts subject to tension must be designed for fatigue according to the provisions of AASHTO LRFD Article 6.13.2.10.3.

In girder splices, standard-sized bolt holes must be used according to the Recommended Specifications, which is consistent with the latest provisions for splice design given in the Standard Specifications. In the connections of primary members, standard-sized bolt holes should preferably be used to ensure that the steel will fit together in the field. Oversized or slotted holes are permitted for these connections if it is ascertained by the engineer that the correct geometry of the erected steel can still be obtained. It is strongly recommended that the use of cross-section or slotted holes be limited to connections of bracing members that are not required to maintain the geometry of the erected steel.

11.3 Welded Connections

According to the Recommended Specifications, fatigue of fillet welds should be checked according to the provisions of AASHTO LRFD Article 6.6.1, using Category E for the weld material.

12. Deflections

12.1 General

All deflections should be computed using unfactored loads, unless otherwise noted, according to the Recommended Specifications.

12.2 Span-to-Depth Ratio

For girders having a specified minimum yield stress of 50 ksi or less, the preferred span-to-depth ratio of the steel girder is not to exceed 25. The span to be used in determining this ratio is defined as a girder arc length, L_{as}, equal to the following: (1) the arc span length for simple spans, (2) 0.9 times the arc span length for continuous end spans, and (3) 0.8 times the arc span length for continuous interior spans.

The earliest preferred span-to-depth ratio of 25 given for steel beams and girders in the AASHTO Specifications was developed for steels with a specified minimum yield stress of 33 ksi and for noncomposite design. With the adoption of composite design, the preferred span-to-depth ratio for the steel girder was relaxed to 30. The span-to-depth ratio for the composite girder (concrete deck plus steel girder) was set at 25.

In the Recommended Specifications, the preferred steel girder depth for curved composite girders is increased back to 25. The same limit would also apply to curved noncomposite girders. This change is introduced to reflect the fact that the outside curved girder receives a disproportionate share of the load and needs to be stiffer. Cross frame forces in curved skewed bridges are directly related to the relative girder deflections. Increasing the depth (and stiffness) of all the girders in a curved skewed bridge leads to smaller relative differences in the deflections and smaller cross frame forces. Deeper girders also result in reduced girder out-of-plane rotations, which may make the bridge easier to erect. A girder shallower than the recommended limit might be used if the engineer evaluates the cross frame forces, bridge deformations, and live-load deflections and finds them to be within tolerable limits.

According to the proposed Equation (12-1) in the Recommended Specifications, an increase in the girder depth is suggested for bridges with girders having a yield stress greater than 50 ksi. As mentioned previously, the span-to-depth limits of 25 and 30 evolved from a history of noncomposite bridges constructed by using steels with a specified minimum yield stress of 50 ksi or less. These noncomposite bridges actually exhibited composite behavior at service loads. Without an increase in the depth of girders fabricated from higher strength steels, the girders will be much more flexible and deflections will increase.

12.3 Dead-Load Deflections

According to the Recommended Specifications, deflections due to the steel and concrete weights should be reported separately, as is typically required. The specification of cambers for the steel weight alone allows for checking of the geometry during the steel erection. Deflections due to future wearing surfaces or other loads not applied during construction are also to be reported separately. Vertical cambers should be specified to compensate for the computed dead-load deflections. Lateral cambers may also be specified to compensate for the girder rotations to ensure proper seating of the bearings and/or the proper lateral bridge geometry. Explicit computation of the girder rotations at the supports may indicate that a properly designed elastomeric bearing pad can be substituted for a more expensive pot bearing.

12.4 Live-Load Deflections

At the direction of the Project Panel, live-load deflection limits are the same as the present AASHTO. The maximum computed live-load deflection of each girder in the cross section is preferably limited to $L/800$, except for girders under a sidewalk for which the live-load deflection is preferably limited to $L/1,000$. L is taken as the girder arc length between bearings. However, the maximum live-load deflection in each girder in each span must be checked. The loading is as specified in AASHTO. The reduction due to multiple presence is permitted as specified in AASHTO Article 3.12.1. For girders under a sidewalk, the pedestrian live load should not be included when making this deflection check. The deflection limit should be applied to each individual girder in the cross section because the curvature causes each girder to deflect differently than the adjacent girder so that an average deflection has no meaning.

Acceleration related to the frequency and amplitude of vibration is the primary cause of discomfort to pedestrians or to vehicle passengers in stopped traffic on a bridge. In fact, Canadian specifications limit the computed static live-load deflections as a function of the first natural frequency of the bridge superstructure based on different degrees of pedestrian use. The Recommended Specifications, however, conform to the current accepted practice in U.S. specifications of controlling vibrations and deflections indirectly using the "L-over" ratios.

The researchers originally recommended consideration of the use of a single lane loaded to compute maximum live-load deflections. The use of two times one lane of HS20 truck or lane load was suggested. Two times an HS20 truck is approximately the weight of the vehicle being considered in the new Canadian specification for live-load deflection checks using an acceleration limit derived from the natural frequency of the bridge. The approach of limiting deflection based on an acceleration limit has also been adopted by AASHTO for the *Guide Specifications for Pedestrian Bridges,* although the live loading is different. Again, the use of a single loaded lane with a heavy load on curved bridges is more meaningful than loading multiple lanes with a lighter loading. Live load for deflection is not related to live loading for strength, which may change over time.

In a curved bridge, each girder in the cross section is likely to deflect a different amount even if all lanes are simultaneously loaded, as assumed in the straight-girder specifications. In wider curved-girder bridges, the outside girder is more heavily loaded than the outside girder of a narrow bridge with only one or two lanes. Thus, by using a single lane of load, bridges of various widths are treated more uniformly than by assuming that all lanes are loaded simultaneously. For tangent bridges, assuming that all girders act together and deflect equally, as permitted in Article 10.6.4 of the Standard Specifications when full-depth cross bracing is provided, gives about the same results as two times one lane of HS20 loading (plus impact). For curved bridges, such an assumption does not yield the same results.

To compare the effect of using two times HS20 in one lane to the effect of using HS25 in multiple lanes for checking live-load deflections, deflections for several actual bridge designs were computed using a refined analysis. For each girder, the live load was placed in the position to cause the largest deflection for that girder. For the latter case, where the deflection was governed by three-lanes loaded, the multiple presence factor of 0.90 specified in Article 3.12 of the Standard Specifications has been included in the reported values. Where the deflection was governed by four-lanes loaded, the specified multiple presence factor of 0.75 has been included in the reported values. The results of these comparisons are summarized below. The governing number of loaded lanes for the case of the actual design live loading is indicated either in the text or in the table.

Bridge 1 is a three-span continuous bridge with radial supports and average span lengths of 321-416-321 ft. Span 1 is curved with a radius of 500 ft measured at the longitudinal centerline. Spans 2 and 3 are tangent. The out-to-out width of the concrete deck is 48 ft-3 in. There are four girders in the cross section spaced at 13 ft-6 in. A 5 ft-0 in.-wide sidewalk is on the outside of the curve. The girders are 14 ft-0 in. deep and the design live load is HS25. For this study, the bridge was examined both with and without the sidewalk. The results for this bridge are summarized in Tables 1 and 2. G4 is the outermost convex girder. The governing deflections that are reported for Bridge 1 for the case of the design live load are for three lanes loaded.

In this particular case, the HS25 loading generally results in slightly larger deflections than the proposed loading in the Recommended Specifications. Note the significant difference in the computed deflections for each girder in the curved span 1, which indicates that the use of an average deflection would be inappropriate for curved girders.

In the tangent span 3, which is furthest from the curved span, the deflections are more equal. Thus, the use of an average deflection, or considering all girders to act together and deflect equally, seems more reasonable for tangent spans. Although span 2 is tangent, the deflections still appear to be affected by the curvature in span 1. Such behavior is indicative of why Article 1.1 of the Recommended Specifications states that any bridge superstructure containing a curved segment shall be designed as a curved bridge according to the provisions.

Bridge 2 is a five-span continuous horizontally curved bridge with spans of 180-229-164-164-131 ft and radial supports. The radius is 886 ft, measured at the longitudinal centerline. The out-to-out deck width is 30 ft-6 in. with the girders spaced at 11 ft-6 in. There are no sidewalks and the design live load is HS25. The girders are 7 ft-6 in. deep. A comparison of the computed live-load deflections in the first three spans is given in Table 3. In this case, girder G1 is defined as the outermost convex girder. The governing deflections reported for Bridge 2 for the case of the design live load are for two-lanes loaded.

Bridge 3 is a horizontally curved simple-span bridge with a span of 169 ft and a radius of 275 ft measured at the longitudinal centerline and skewed supports. The out-to-out width of the concrete deck is 36 ft-4 in. There are six girders in the cross section spaced at 6 ft-0 in. The girders are 5 ft-8 in. deep and the design live load is HS25. Girder G6 is defined as the outermost convex girder. The governing deflections reported for Bridge 3 for the case of the design live load are for two-lanes loaded. The comparison of the computed deflections is given in Table 4.

Bridge 4 is a four-span continuous horizontally curved bridge with spans of 106-138-138-106 ft and a radius of 1,300 ft measured at the longitudinal centerline. The out-to-out deck width is approximately 83 ft-8 in., and there are seven girders in the cross section spaced at approximately 12 ft-8 in. The girders are 6 ft-5 in. deep and there are no sidewalks. In this case, the design live loading is HS20. Girder G1 is defined as the outermost convex girder. The comparison of the computed deflections for the first two spans (the bridge is symmetrical) is summarized in Table 5. In this case, the number of loaded lanes that governed for each girder for the case of the design live load is indicated in Table 5.

Bridge 5 is a two-span continuous horizontally curved bridge with spans of 162-162 ft and a radius of 642 ft, mea-

TABLE 1 Bridge 1 with sidewalk, maximum live-load deflections (in.)

Girder	Span 1		Span 2		Span 3	
	HS25 Design load	2xHS20 Single lane	HS25 Design load	2xHS20 Single lane	HS25 Design load	2xHS20 Single lane
G4	4.79	4.50	4.71	4.84	2.97	3.16
G3	3.65	3.17	4.18	3.30	2.69	2.12
G2	2.53	1.84	4.26	2.00	2.99	2.25
G1	1.62	1.21	4.96	4.72	3.61	3.46

TABLE 2 Bridge 1 without sidewalk, maximum live-load deflections (in.)

Girder	Span 1		Span 2		Span 3	
	HS25 Design load	2xHS20 Single Lane	HS25 Design load	2xHS20 Single Lane	HS25 Design load	2xHS20 Single Lane
G4	6.33	5.33	6.33	5.96	3.99	3.88
G3	4.58	3.65	4.88	3.74	3.14	2.37
G2	2.85	2.01	4.26	3.01	3.04	2.28
G1	1.58	1.21	5.06	4.86	3.67	3.56

sured at the longitudinal centerline. The out-to-out width of the concrete deck is 32 ft and there are four 7 ft-6 in. deep girders in the cross section. The design live load is HS20. The comparison of the computed deflections for this case is given in Table 6. Girder G4 is defined as the outermost convex girder. The governing deflections reported for Bridge 5 for the case of the design live load are for two-lanes loaded.

13. Constructibility

13.1 General

As discussed earlier under Article 2.5, a construction plan must be provided on the design plans indicating one possible scheme to construct the bridge, including the sequence of girder and deck placement and the location of any temporary supports. This construction plan does not relieve the contractor of any of the responsibility for the fabrication, erection, or construction of any part of the bridge. Construction loads and wind loads, specified in Articles 3.3 and 3.4, respectively, must be considered at each critical stage of construction, which is defined as any condition during the construction of the bridge that might cause instability or yielding. The construction plan must also show all assumed construction loads in addition to the computed dead loads. Any computed deflections shown on the construction plan are to be unfactored.

13.2 Steel

The stresses in primary steel members due to the factored construction loads are not to exceed the specified minimum yield stress in any element, or, the critical buckling stress for any steel element subjected to compression during construction. As specified in Article 3.3, a load factor of 1.4 should be applied to the construction loads. The critical compressive stress in webs is limited to the appropriate web bend-buckling stress, as defined in Article 6. The critical tensile stress in webs is limited to the yield stress according to Article 6. The critical stress in flanges having one web is given in Article 5. For partially braced compression flanges during erection and deck placement prior to deck hardening, the critical compressive stress for noncompact flanges specified in Article 5.2.2 is to be used. The compression-flange slenderness during erection and deck placement is also limited to the noncompact flange slenderness limit given by Equation (5-4). The critical stresses for tension flanges and continuously braced flanges having one web during construction are given in Articles 5.3 and 5.4, respectively. Critical stresses for noncomposite box flanges during construction are given in Article 10.4.1. Finally, as mentioned previously, the slip resistance of bolted joints should be checked for the factored loads at each critical construction stage.

13.3 Concrete

The compressive concrete stress due to factored construction loads during any stage of construction is not to exceed $0.85f'_c$ where f'_c is the compressive strength for the expected age of the concrete at the time the construction loads are to be applied. Tensile stresses in the concrete at all stages of construction must also be checked. For example, when the concrete deck is cast in a span adjacent to a span where the concrete deck has already hardened, the tensile stress in the hardened concrete may exceed the modulus of rupture. According to Article 2.4.3 of the Recom-

TABLE 3 Bridge 2, maximum live-load deflections in spans 1, 2, and 3 (in.)

Girder	Span 1		Span 2		Span 3	
	HS25 Design load	2xHS20 Single Lane	HS25 Design load	2xHS20 Single Lane	HS25 Design load	2xHS20 Single Lane
G1	2.25	2.40	2.86	3.00	1.35	1.45
G2	1.46	1.30	1.87	1.67	0.87	0.77
G3	1.25	1.32	1.54	1.60	0.74	0.79

TABLE 4 Bridge 3, maximum live-load deflections (in.)

Girder	Span 1	
	HS25 Design load	2xHS20 Single lane
G6	1.53	1.90
G5	1.51	1.50
G4	1.20	1.11
G3	0.91	0.76
G2	0.81	0.74
G1	0.80	0.86

mended Specifications, if the tensile stress in the concrete at the constructibility limit state exceeds 0.7 times the modulus of rupture, a minimum of 1-percent longitudinal reinforcement must be provided in the deck at that location for crack control. Because tensile stresses are limited and construction loads are relatively short term, stresses in composite sections at the constructibility limit state should be computed assuming an uncracked section based on the modular ratio of n.

13.4 Deflection

For adequate rideability and drainage, vertical and lateral deflections of the girders through the construction sequence must be evaluated to ensure that the final position of the girders is as close as possible to the final position anticipated in the design. To ensure that the rotational capacities of the bearings are not exceeded during construction, rotations of the girders about the longitudinal and transverse axes should also be determined for each critical stage of the construction.

13.5 Shipping

Field sections should be limited in weight, height, and width so that the sections can be delivered to the site in a practical manner. For loads that exceed the normal load size and weight permitted on the highway system, escorts or special permits are required. Shipments by rail are generally limited only by the geometry of the sections.

TABLE 5 Bridge 4, maximum live-load deflections in spans 1 and 2 (in.)

Girder	Span 1		Span 2	
	HS20 Design load	2xHS20 Single lane	HS20 Design load	2xHS20 Single lane
G1	0.58(3L)	0.69	0.92(3L)	1.07
G2	0.46(3L)	0.42	0.73(4L)	0.68
G3	0.41(4L)	0.30	0.64(4L)	0.44
G4	0.36(4L)	0.27	0.55(4L)	0.38
G5	0.37(4L)	0.28	0.57(4L)	0.40
G6	0.39(3L)	0.36	0.61(4L)	0.55
G7	0.45(3L)	0.55	0.73(3L)	0.86

TABLE 6 Bridge 5, maximum live-load deflections in spans 1 and 2 (in.)

Girder	Span 1		Span 2	
	HS20 Design load	2xHS20 Single lane	HS20 Design load	2xHS20 Single lane
G4	0.85	1.19	0.85	1.19
G3	0.68	0.83	0.68	0.83
G2	0.55	0.58	0.55	0.58
G1	0.56	0.78	0.56	0.78

13.6 Steel Erection

The erection scheme provided on the construction plan is to denote a sequence of erection, which will ensure that computed stresses do not exceed the critical stresses during any stage of erection and that the final steel deflections are as close as possible to those determined from the analysis.

Temporary supports are often required to support curved girders during erection. Should temporary supports be required, the construction plan must instruct the contractor where the supports can be located and the magnitude of the reactions at the supports. The final elevation of the steel at the location of the temporary supports must also be provided so that when the supports are removed, the correct girder elevation will be obtained. In addition, the amount of girder deflection that will occur on removal of the supports must be furnished so that the proper stroke is provided to permit removal of the jacks at the supports. Provisions must also be made to receive any temporary reactions acting on the girders at the supports.

The addition of any bracing deemed to be necessary in the erection process should be sized and the design of the required connections should be provided. If temporary bracing is required, its addition and removal should be specified in the erection sequence. In many instances, cranes are used to support individual curved girders until they are braced to adjacent girders. Curved I-girders are sometimes erected in pairs to increase stability. Temporary top and/or bottom flange bracing can help to improve the stability of girder pairs.

13.7 Deck Placement

The deck placement sequence, time between casts, and the assumed concrete strength at the time of subsequent casts must be specified in the construction plan. The factored steel stresses due to the sequential placement of the deck concrete should not exceed the appropriate critical stresses. Uplift of the girders at the bearings during any stage of the deck-placement sequence is not permitted. If uplift is indicated, temporary loads can be placed on the structure as necessary to prevent lift-off. The magnitude and position of the required temporary loads must be given. If the bridge is to be redecked in the future under traffic, the girder and deck stresses and cross frame forces should be evaluated for the anticipated redecking sequence (i.e., assuming that selected portions of the deck are removed and that live load is positioned in the appropriate lanes on the remaining deck).

13.8 Deck Overhangs

During the construction of steel girder bridges, concrete deck overhang loads are usually supported by brackets attached to the top flange and braced against either the web or bottom flange. The brackets are typically spaced longitudinally at approximately 3- to 4-ft intervals along the exterior girders. As overhangs become larger, the effects of these loads on the exterior girders become more significant. Therefore, this article requires that these effects be considered in the design of the exterior girder flanges. The overhang loads include the weight of the wet concrete in the deck overhang, the weight of the overhang deck forms, the weight of the concrete finishing machine and screed rails, and any miscellaneous construction loads. The weight of the concrete finishing machine is usually not known at design time and must typically be assumed. The engineer may wish to provide the assumed weight used in the calculations with the construction plan. Additional guidance may be found in work by Grubb (*51*).

The individual bracket loads applied eccentrically to the exterior girders create applied torsional moments to the girders, which tend to twist the girder top flanges outward and may twist the girder bottom flanges inward. Additional lateral bending moments in the flanges result, which produce tension in the top flange at brace points on the side of the flange opposite from the bracket. The proposed Equations (C13-1) and (C13-2) are provided in the commentary to this article to estimate the lateral bending moments in the top flange due to both a uniformly distributed and a concentrated bracket force. These approximate equations assume that panel lengths between brace points are equal and that distributed lateral bracket loads on the flanges are uniform. Lateral flange bending stresses resulting from these moments should be added algebraically to any additional lateral flange bending stresses resulting from effects other than curvature in the term, f_m, defined in Article 5.2.1, which is then to be used in the proposed Equations (5-8) and (5-10) to determine the critical top flange stresses. When combining the lateral flange stresses, careful consideration must be given to the sign of the stresses. The critical flange stresses are to be determined for the combination of the effects due to the factored construction loads and the factored overhang loads.

The effect of the reactions in the top strut of the overhang brackets should be considered in the design of the cross frames. In addition, the horizontal components of the resultant loads in the inclined members of the overhang brackets are often transmitted directly onto the exterior girder webs. The girder webs may deflect laterally due to these applied loads. Precautions should be provided to ensure that the webs are not damaged. Alternatively, where practical, the overhang brackets should preferably be extended to the intersection of the bottom flange and the web. Rotations and vertical deflections of the exterior girders due to the overhang loads should also be checked to ensure that the proper deck thickness is obtained.

F. RECOMMENDED SPECIFICATIONS: DIVISION II, CONSTRUCTION

1. General

The Division II requirements for construction in the Recommended Specifications should be considered in addition to the Division II requirements in the AASHTO Standard Specifications. However, if there is a conflict between similar provisions in the two specifications, the provisions in the Recommended Specifications are to prevail. In some instances, the requirements in the Recommended Specifications are more restrictive. The provisions in Division II apply specifically to the fabrication, shipping, and erection of the steel superstructure and to the placement of the concrete deck and should be used by the contractor to prepare a construction plan.

The construction plan shown in the design plans must indicate one possible way of constructing the bridge and also must indicate the specific considerations related to the construction of the bridge that were made to accommodate that plan in the design by the engineer. The creation of this plan does not in any way relieve the contractor of full responsibility for the fabrication, erection, or construction of any part of the bridge, even if there are no modifications to this plan in the construction plan to be prepared by the contractor. The contractor, fabricator, and erector are not required to build the bridge according to the construction plan given in the design plans. However, in most instances, the construction plan prepared by the contractor should not differ significantly enough from the construction plan shown in the design plans that a redesign of the bridge would be required.

1.2 Construction Plan

This article requires the contractor to produce a detailed construction plan that is stamped by a registered professional engineer and approved by the owner's engineer. The plan may be based on the one in the design plans or may be developed independently by the contractor. This construction plan should include the following: (1) fabrication procedures, including the method used to curve the girders; (2) shipping weights, lengths, widths, and heights and means of shipping; (3) an erection plan showing the sequence of erection, crane capacities and positions, and the location, capacity, and elevation of any required temporary supports; and (4) computations showing a check of the factored construction stresses according to the requirements of Article 13 in Division I of the Recommended Specifications. If the contractor's plan results in a change to the dead-load camber of the girders from that presented in the design plans, then approval must be obtained from the owner's engineer prior to the start of fabrication. This timing is required to ensure that any changes in the camber, or other changes, will be accounted for at that time.

Although some state DOTs have requirements similar to the above, such requirements are not currently included in the AASHTO Standard Specifications, LRFD Specifications, or the Guide Spec. The intent of these requirements is to ensure that a safe method of construction is provided and that the contractor plan for the implementation of such a construction procedure when preparing the bid. The development of an acceptable construction plan by the contractor prior to the bid should reduce the number of claims and extras submitted by contractors who submit too low a bid based on poor construction plans or schemes. Because of the complexity of curved bridges, the construction plan prepared by the contractor is required to be stamped by a registered professional engineer.

2. Fabrication

2.1 General

Unless otherwise specified in the construction plan, the fabricator should ensure that the steel can be fit up in the no-load condition. The intent of this provision is to ensure that the steel is erected in a manner consistent with the analysis for the steel self-weight. The steel may be instead erected in stages and connected without adjustments to the camber if the analysis reflects the planned erection sequence.

2.2 Handling

Care should be exercised in handling rolled shapes and plates for flanges and webs to ensure that visible damage or other incidental damage is prevented.

2.3 Girders

2.3.1 Rolled I-Beams. Horizontal curvature of rolled I-beams may be obtained either by heat curving or cold bending and only when the specified minimum yield stress of the steel does not exceed 50 ksi.

Heat curving is typically accomplished by fabricating a straight girder in a conventional manner and then introducing thermal stresses and yielding in the top and bottom flange edges, approximately simultaneously along one side of the girder to induce a residual curvature after cooling. As the top and bottom flanges on the side of the girder to be shortened are heated, the edges attempt to elongate. However, the edges are restrained by the unheated portions of the flange and the unheated web causing the flanges to upset, or increase in thickness, to relieve the compressive stresses. Assuming the temperature is high enough, the heated edges will yield resulting in residual stresses and strains that remain after the flanges cool. The magnitude of the residual strains determines the amount of net shortening, which produces a concave horizontal curvature in the girder after cooling. Heat curving can be accomplished by heating the flange edges continuously along their length or by heating the flanges in evenly spaced triangular or wedge-shaped spots—a process known as spot or V-heating. Generally, V-heating is used for longer radius curves. Prior to 1968, heat curving of bridge girders was not accepted on a national basis. Research by Brockenbrough (*52–56*) eventually led to the development of specification criteria for heat curving.

Quenched and tempered steels may not be heat curved according to the Recommended Specifications. Heat curving should be performed in accordance with the provisions of AASHTO Articles 10.15.2 in Division I and Article 11.4.12.2 in Division II of the Standard Specifications. Article 10.15.2 in Division I specifies a minimum allowable radius of curvature for the heat curving of a straight girder.

Because of the increased residual stresses due to heat curving, slight inelastic action may occur under initial loadings. As a result, a vertical camber increase was introduced in Article 10.15.3 of the AASHTO Standard Specifications for girders with radii less than 1,000 ft to negate a possible loss in camber as the residual stresses dissipate (*52*). A modification factor was added to the equation for the vertical camber increase in 1988 to bring the predicted camber loss more in line with measurements made in the field (*57*), which indicated little or no observed camber loss. The modification factor has received no additional verification. Therefore, the Recommended Specifications state that the owner should determine the necessity for providing the additional camber given in AASHTO Article 10.15.3.

Research has indicated that heat curving has no deleterious effect on the fatigue strength of horizontally curved girders (*58*).

Cold bending, although not dealt with explicitly in the AASHTO Specifications in the past, is permitted by the provisions. The cold bending may be performed by using either a press or a three-roll bender and should be controlled so that the flanges and web of the beam are not buckled or twisted out-of-plane. It is up to the owner to specify the limits on the cold-working strain such that the metallurgy of the steel is not affected.

2.3.2 Welded I-Girders. In addition to heat curving and cold bending, welded girders may be fabricated from cut-curved flanges. An advantage of cut curving is that there is no limit on the radius of curvature that may be obtained. Disadvantages are that special fixtures are usually required for fit-up and significant amounts of scrap can be generated without careful planning. The flanges may be cut either from a single wide plate, or more desirably, nested for multiple cutting from a single plate to minimize the scrap. When the latter practice is used, it is necessary to heat curve the girders to the final radius because adjacent flanges would have nearly the same radius. Otherwise, it would be necessary to re-burn each flange with concomitant material loss. Vertical camber should be obtained by cutting the web plate to the necessary contour.

2.3.3 Welded Box and Tub Girders. Box flanges must be cut curved according to the provisions. Top flanges of tub girders, however, may be curved in the same fashion as the flanges of welded I-girders. Heat curving of these flanges may be done after the flanges are welded to the webs.

2.4 Web Attachments

In this article, connection-detail requirements for web attachments, including transverse stiffeners, connection plates, and longitudinal stiffeners, are given. Many of the provisions re-emphasize the provisions given in Division I of the Recommended Specifications regarding the detailing of these elements. The continuous fillet welds connecting transverse stiffeners to the web must be terminated between $4t_w$ and $6t_w$ from a flange or longitudinal web stiffener, where t_w is the thickness of the web, to avoid intersecting welds and to prevent high restraint stresses from developing due to weld shrinkage. To minimize through-thickness bending stresses in the web, prevent flange rotation or raking relative to the web, and transfer lateral cross frame forces directly to the flanges, connection plates must be attached to both flanges by welding or bolting. A welded connection should not be substituted for a bolted connection on the design plans without the permission of the designer-of-record. Cross frames or diaphragms to be attached to the web are to be fit under the no-load condition, unless otherwise specified.

2.5 Bolt Holes

Standard size bolt holes are to be used in girder splices and in primary load-carrying members to control the geometry during erection, unless otherwise approved by the owner's engineer and the engineer of record.

2.6 Tolerances

2.6.1 Welded Web Flatness. Webs are to meet the dimensional tolerances specified in Articles 3.5 and 9.19 of the *ANSI/AASHTO/AWS D1.5 Bridge Welding Code* (*59*). The flatness can be measured using a straightedge oriented along the shortest line between the flange-to-web welds. The tolerance for stiffened webs is determined using the intermediate transverse stiffener spacing as the greatest dimension of the panel, since the maximum spacing of transverse stiffeners on curved girder webs is limited to the web depth. For unstiffened webs, the tolerance is determined using the web depth.

2.6.2 Camber. Vertical camber of the girders should be provided to allow for dead-load deflections and support rotations about the axis radial to the girder. Twist camber may be provided to allow for rotation of the girder about the axis tangential to the girder. Because torsion can twist the girders, it may be necessary to camber the girders to account for the rotations due to the twist to assist with fit-up. However, twist camber will not likely be required unless skews are significant. Vertical cambers are difficult to measure for curved girders. If twist camber is not provided, the camber of curved I-girders can be measured by laying the girder units on their sides.

2.6.3 Sweep. Tolerances for girder sweep are given in Article 3.5.1.4 of the *ANSI/AASHTO/AWS D1.5 Bridge Welding Code* (*59*). Sweep should be measured radially from the theoretical curve of the girder. The theoretical curve may be a constant radius, a compound radius, or a spiral. To check the sweep tolerance for a curved girder, theoretical offsets from a chord can be computed. Deviations from the theoretical offsets can then be compared to the AWS (American Welding Society) sweep tolerance values given for tangent girders. Offset measurements should be made with the girder vertical and in the no-load condition.

2.6.4 Girder Lengths. Accurate measurement of girder lengths is important to ensure the proper location of anchor bolts. According to the Recommended Specifications, girder lengths should be measured along the arc based on an ambient temperature of 68°F. If girder lengths or anchor bolt locations are surveyed with a laser instrument, make compensations for the actual temperature of the girder.

2.7 Fit-Up

2.7.1 General. Fit-up of girder sections prior to erection is to satisfy the requirements of Article 11.5.3 of Division II of the AASHTO Standard Specifications. As a minimum, three contiguous sections adjusted for line and camber should be preassembled. For multistringer bridges, such preassemblies are normally used to check the fit-up of field splices. For structures longer than 150 ft, the assemblies should not be less than 150-ft long.

When numerically controlled drilling is employed, the Recommended Specifications require check assemblies of cross frames or diaphragms to be made between properly positioned girder sections, as prescribed in Article 11.5.3.3 of AASHTO Division II. If numerically controlled drilling is not employed, shop fit-up of bolted cross frame or diaphragm connections may still be required for structures with particularly rigid or complex framing. Full shop assembly of the bridge is not required unless specified in the construction plan. Also, fit-up shall be assumed to be performed in the no-load condition unless otherwise specified in the construction plan.

2.7.2 Girder Section Field Splices. Girder section field splices may be fit-up with the flange and web plates in either the vertical or horizontal position. If the flanges are cut curved, fit-up of I-girder flange splices is usually performed with the flanges horizontal either before or after welding the flanges to the web. If the girder is to be heat curved, splices may be fit-up prior to heat curving with the web in either a vertical or horizontal orientation. The design vertical camber must be in the girders prior to the fit-up of girder splices.

3. Transportation Plan

A transportation plan may be required by the owner for complex or large structure components that are wider, heavier, deeper, or longer than normally permitted for the selected mode of transportation. As a minimum, the transportation plan should include (1) the type of girder supports required and their location; (2) the type, size, and locations of tie-downs; and (3) any temporary stiffening trusses or beams that would be required to meet the requirements of this article.

During transportation, girders should not be subjected to large stresses or possible fatigue damage. According to the Recommended Specifications, girder stresses during shipment due to self-weight are to be computed with an impact allowance of 100 percent and are not to exceed the critical stresses specified in the applicable section of Division I. A 100 percent impact allowance is specified to account for the dropping of the girders on rigid supports before and after shipment. Fatigue during shipment can be caused by longitudinal stresses in the girders or by through-thickness stresses resulting from repeated raking of the section. Girder sections are preferably to be shipped in the same orientation as in the completed structure, and are to be supported so that their cross-section shape is maintained and the through-thickness stresses are minimized. The Recommended Specifications also conservatively limit the fatigue stress ranges in the supported girder to two times the nominal fatigue resistance specified in Article 6.6.1 of the AASHTO LRFD Specifications.

The girder should also be supported during shipment so that dynamic lateral flange bending stresses are controlled. To control vibrations during shipment, single unbraced I-girders should preferably not be cantilevered more than the length, L_c, given by Equation (3-1). Equation (3-1) was derived from the equation for the fundamental natural frequency of a cantilever beam subject to free vibration. The equation ensures that the first mode of vibration of the fixed-end cantilever will be greater than 5 Hz. Critical lengths for other frequencies can be determined by multiplying 43 times the desired frequency divided by 5.

4. Steel Erection

4.1 General

Steel erection should be performed in accordance with the construction plan prepared by the contractor and approved by the owner's engineer. Article 11.2.2 in Division II of the AASHTO Standard Specifications requires the contractor to supply erection drawings indicating the step-by-step plan of erection, including the details of any required falsework or bracing. The article goes on to state that calculations may be required to demonstrate that member capacities are not exceeded during any step in the erection sequence and that the final geometry of the erected steel will be correct.

The Recommended Specifications *require* that the factored stresses due to the self-weight of the steel and due to wind loading at each stage of erection be computed and checked according to Article 2.5.2 in Division I. Steel that does not fit together in the field is indicative that the stresses in the steel are not consistent with the stresses computed in the design and in the subsequent detailing. Excessive stresses can be relieved through the proper use of temporary supports. Reaming of bolt holes during erection to permit fit-up is only to be allowed with the approval of the owner's engineer, and only after the resulting stress state and deflections have been investigated. Any erection stresses induced in the structure as a result of using a different method of erection from that in the construction plan should be accounted for by the contractor, according to the provisions of Article 11.6.4.2 in Division II of the Standard Specifications. This is to ensure that the integrity and load capacity of the completed bridge is maintained.

Bolted girder splices should be field assembled according to the provisions of Article 11.6.5 in Division II of the Standard Specifications, which ensures that an adequate number of pins and bolts are provided during the assembly.

4.2 Falsework

Temporary falsework bents are often required to provide stability to curved girders during the erection. Where conditions permit, cranes may be substituted for falsework. Falsework, when required, should be designed for the vertical and lateral loads specified in the construction plan. The elevation of the falsework should be set to support the girders at their cambered no-load elevations and to allow for the deflection of the erected steel after the temporary supports are removed. The jacks used in conjunction with the falsework should have a stroke adequate to permit full unloading of the steel and should preferably be arranged to allow uniform unloading of all temporary supports at each cross section to minimize twisting of the steel and any damaging stresses.

4.3 Bearings

Bearings should be installed such that sufficient rotation capacity remains to accommodate rotations due to live loads and environmental loads after all dead load has been applied. Skewed structures are particularly susceptible to twisting about the longitudinal axis of the girders. As a result, significant bearing rotations can occur in these structures during construction.

4.4 *I-Girders*

Extreme care must be exercised to ensure that curved I-girders are stable during all stages of the erection. Adequate torsional restraint of curved I-girders is required at all times. The stability of curved I-girders with large unbraced lengths is not well predicted by current theories. Stability is best determined by performing small test lifts of the girder. Unbraced lengths of curved I-girders should be kept within the limits prescribed in Articles 5 and 9.3.2 in Division I, where practical. Also, when evaluating the strength and stability of curved I-girders during erection, the stage of completeness of all bolted connections must be considered.

4.5 *Closed Box and Tub Girders*

The erection of box girders, particularly skewed box girders, is complicated by their large torsional stiffness. Shop fit-up of cross frames and external diaphragms is extremely important because the torsional stiffness of the box makes field adjustments difficult. Care should be taken to ensure that the cross-section shape of each box is maintained during the erection by using additional cranes or temporary supports to restrain the rotations until the diaphragms or cross frames between the girders are adequately connected.

5. Deck

5.1 *Forms*

5.1.1 General. Deck forms may be either plywood, permanent metal forms, or concrete panels, as approved by the owner. Proprietary deck forms must be placed according to the manufacturer's specifications, which must be approved by the owner's engineer. The deck forms must be firmly attached to the top flange and may not be assumed to provide lateral support to the top flanges. Tests have shown that the commonly used connections for attaching the deck forms to the top flange do not have adequate strength to transfer the torsional shear forces normally encountered *(60)*.

5.1.2 Overhangs. Loads on deck overhang brackets—and their effects on the girders and cross frames—should be considered as discussed previously under Article 13.8 of Division I. If the loads should differ significantly from those given in the design plans, the contractor has to provide an additional analysis for the new overhang loads to be reviewed and approved by the owner's engineer. Particular attention needs to be paid to the compression flange of the exterior girders.

The brackets should preferably be attached to the top flanges and bear near the bottom flange. If it is necessary for the overhang brackets to bear directly on the web, then the contractor's engineer must take adequate precautions to prevent permanent deformations of the web.

5.1.3 Tub Girders. Permanent deck forms are highly desirable for use between the flanges of a tub girder because of the difficulty in removing deck forms from inside the tub. The forms should not be supported at locations other than the girder flanges unless specifically considered in the design. Debris should not be left inside the box because it interferes with subsequent inspections.

5.2 *Placement of Concrete*

Concrete casts should be made in the sequence specified in the approved construction plan. The duration of each cast is also to be specified in the construction plan. The time between casts should be such that the concrete in the previous casts has reached the strength specified in the construction plan. Any retarding or accelerating agents to be used in the concrete mix should also be specified.

Casts that include both a positive and negative moment region should preferably be cast so that the positive moment region is cast first. In a long cast, if the negative moment region is cast first, it is possible that this region will harden and be subject to tensile stresses during the remainder of the cast, which may result in early age cracking of the deck. When concrete is cast in a span adjacent to a span that already has a hardened deck, induced negative moments in the adjacent span will also cause tensile stresses and torsional shear stresses that may result in transverse deck cracking.

6. Reports

Modifications in the field from any of the original plans must be documented with the appropriate approvals noted. The complete Recommended Specifications and commentary are presented in Appendix D.

G. DESIGN EXAMPLE, HORIZONTALLY CURVED STEEL I-GIRDER BRIDGE

A comprehensive example, illustrating the application of the Recommended Specifications to the design of a three-span horizontally curved steel I-girder bridge with four girders in the cross section, is presented in Appendix E. The bridge has spans of 160-210-160 ft, measured along the centerline of the bridge. The radius of the bridge is 700 ft at the center of the roadway. The supports are radial with respect to the roadway. The out-to-out deck width is 40.5 ft with the four girders in the cross section spaced at 11 ft. Live load is HS25 for the strength limit state and structural steel with a specified minimum yield stress of 50 ksi used throughout. The deck is conventional cast-in-place concrete with a specified minimum 28-day compressive strength of 4,000 psi. The bridge is designed for a 75-year fatigue life.

Five different framing plans considering different girder depths, cross frame spacings, and with and without lateral flange bracing are investigated. Three options are investigated for web design: an unstiffened web, a transversely stiffened web, and a longitudinally and transversely stiffened web. The bridge is investigated for wind and thermal loadings. Steel erection is also examined, including the need for temporary supports. Sequential placement of the concrete deck is considered. A complete field splice design is provided. Analyses for the example are performed using a three-dimensional finite element program. Comparisons are made with the results from a two-dimensional grid analysis and an approximate one-dimensional V-load analysis.

H. DESIGN EXAMPLE, HORIZONTALLY CURVED, STEEL BOX-GIRDER BRIDGE

A comprehensive example, illustrating the application of the Recommended Specifications to the design of a three-span horizontally curved, steel box-girder bridge with two tub girders in the cross section, is presented in Appendix F. Like the preceding I-girder example, the bridge has spans of 160-210-160 ft, measured along the centerline of the bridge. The radius of the bridge is 700 ft at the center of the roadway. The supports are radial with respect to the roadway. The out-to-out deck width is 40.5 ft, with the webs of each trapezoidal tub girder spaced 10 ft apart and with a clear deck span of 12.5 ft between the interior webs of each tub. Live load is HS25 for the strength limit state and structural steel with a specified minimum yield stress of 50 ksi used throughout. The deck is conventional cast-in-place concrete with a specified minimum 28-day compressive strength of 4,000 psi. The bridge is designed for a 75-year fatigue life.

The slope of the webs is one-on-four, which results in a bottom flange width of 81 in. between the webs of each tub. A single longitudinal flange stiffener is used on the box flanges in negative moment regions. The boxes are braced internally with K-frames in between supports. At the support lines, full-depth internal and external diaphragms are provided. There are no other external braces provided between the boxes. The top flanges of the tubs are braced with single members placed diagonally between the tub flanges and connected directly to those flanges. The example illustrates the computation of transverse bending stresses using the BEF analogy and also investigates a composite box flange option in negative moment regions. The constructibility limit state is examined, including consideration of sequential deck placement and the effect of deck overhang loads. A complete field splice design is also provided. Analyses for the example are performed using a three-dimensional finite element program. Comparisons are made with the results from a two-dimensional grid analysis and an approximate one-dimensional M/R analysis.

CHAPTER 4

CONCLUSIONS AND RECOMMENDED RESEARCH

A. CONCLUSIONS

This project was initiated by NCHRP to update and clarify the provisions of the existing Guide Spec, which are used in conjunction with the AASHTO Bridge Specifications. The work led to the development of the recommended specification for the design and construction of horizontally curved steel girder bridges. The proposed provisions are based on a thorough review of the literature, existing guidelines, laboratory testing, analytical studies, and experience with state-of-the-art design and construction of these structures in the United States. No fundamental research was undertaken.

The proposed provisions are presented in a format similar to the AASHTO LRFD specifications with a parallel commentary that explains and discusses application of the provisions. Separate reports presenting design examples for I- and box-girder bridges further demonstrate the application of the provisions.

Experience has shown that erection and construction of curved-girder bridges is often much more complex than that of tangent girder bridges of similar span. A constructibility limit state is introduced in the proposed provisions to ensure that the bridge can be constructed in a practical manner. Additionally, a scheme to build the bridge is required to be developed by the designer and shown on the design plans. The contractor is not obligated to use the scheme shown in the plans. In fact, the provisions of Division II, Construction, require that the contractor develop a more detailed construction plan that may, or may not, be based on the scheme shown on the plans.

Serviceability is extended to apply to the control of concrete deck cracking. Although this feature has not been provided for in the AASHTO specifications for composite bridges, the research team believes that serviceability should be covered for curved-girder and skewed bridges, which are subjected to higher torsional shears than are tangent nonskewed girder bridges. The provisions require that longitudinal reinforcing steel be placed in the deck wherever the computed concrete tensile stresses during sequential deck placement or due to overload may cause concrete cracking. AASHTO requires that longitudinal reinforcing steel only be placed in the negative moment regions between points of dead-load contraflexure. The proposed provisions recognize that construction sequencing and moving loads require a broader definition of negative moment regions.

The researchers have proposed provisions that define the overload live load as two times an HS20 truck or lane loading placed in the critical location in a single lane. This approach is consistent with the original overload live load provided in the AASHTO load factor design provisions. The effect of a single heavy truck(s) is more critical and should be considered in the design of curved-girder bridges, which are subject to torsion. The proposed overload would be used to check permanent set, live-load deflection, concrete cracking, and net tensile stress for application of the fatigue provisions. The loading intended for each of these conditions was developed using similar logic. Also, by using the same loading for each of these cases, the analytical work required for the design is reduced. However, at the request of the NCHRP Project Panel, the current AASHTO overload is implemented instead of the proposed overload in the Recommended Specifications.

Other modifications were made, however, to the current provisions related to loads to ensure that the application of loads to curved bridges is logical. For example, wind loads are defined in the provisions as unidirectional whereas they are specified in AASHTO as either perpendicular or longitudinal to the bridge, which obviously applies to a tangent bridge.

The provisions for compact I-sections in the Guide Spec were not conservative with respect to some physical tests. However, a change in the application of the corresponding strength equations was identified that brings the tests into a conservative position with respect to the proposed provisions. Otherwise, the strength equations in the Guide Spec for girders either were not changed or were made slightly more liberal.

The Recommended Specifications should provide for the design of curved-girder steel bridges that have a load capacity comparable to designs prepared according to the AASHTO load factor design provisions. The designs resulting from the proposed provisions should be more easily buildable. The requirement of a construction plan to be shown on the design plans tends to level the bidding field by ensuring that all potential builders know how construction was considered in the design. This information should lead to less divergent bidding and fewer opportunities for misunderstandings, extras, or claims.

B. RECOMMENDED RESEARCH

There is every indication that the proposed specifications are safe. There are many areas of potential savings to be gained from further research. Cost savings are likely to be gained in at least two areas. Reduction of required web stiffening is one area where gains are possible. Better understanding of the behavior of curved girders during fabrication, shipping, and erection should yield economic benefits through reduced uncertainty at bidding and fewer field problems. Greater confidence in the girder behavior during construction should also lead to bolder designs that can be applied to a wider variety of bridge sites. For example, the provisions are limited to a maximum span of 300 ft. Some states presently limit the span of curved girders to even shorter spans due to bad experiences. Better understanding of curved-girder behavior would permit more confident use of curved girders in even longer spans.

The strength predictor equations are empirically based on analytical work, which was limited to doubly symmetric I-sections without longitudinal stiffeners and with a yield stress of 50 ksi. There is a need to improve the understanding of these members with concomitant application to a broader range of sections, particularly singly symmetric and composite I-sections. The strength predictor equations for tangent I-girders are theoretical, whereas those for curved I-girders are empirical. A more unified approach to I-girder strength should be provided in AASHTO.

The proposed provisions limit the maximum spacing of transverse web stiffeners to the girder depth. This limit has been retained from the Guide Spec and is the same as that used in the Hanshin Guidelines. Relief from this requirement for some curvatures can be justified with additional testing. Neither fatigue behavior nor strength of curved-girder webs is well understood at this time, and it would be risky to reduce the stiffening requirements without further analytical and experimental research. Longitudinally stiffened webs have not been investigated in the United States, although they have been studied analytically in Japan. Although the recent modifications to the longitudinal web stiffening provisions in AASHTO for tangent girders (in the 1997 Interims) have been modified and included in the proposed provisions, there is a need to improve the understanding of this type of curved web.

The behavior of curved composite I-girder bridges at ultimate load have not been investigated analytically or experimentally. The reserve capacity of these bridges is unknown. It is likely that curved-girder bridges fail in a manner different from tangent girder bridges due to their larger torsion and the associated load shifting upon yielding. The bracing members in curved-girder bridges are treated as primary members because of their importance. However, the true behavior of these members in curved composite bridges has not been adequately investigated. For example, there has been no analytical consideration of fatigue design in these members although the proposed provisions do address the issue with little experimental support.

Curved girders remain one of the least understood structural elements in common use in bridge construction. The future for the use of these elements in U.S. bridge construction is very promising as curved steel girder bridges continue to make up an increasing portion of the American steel bridge market. Thus, improvements in the understanding of their structural behavior as individual members and as components in the completed bridge will pay ever-increasing dividends.

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APPENDIX A

I-GIRDER CURVATURE STUDY

PREFACE

AASHTO first published *Guide Specifications for Horizontally Curved Highway Bridges* in 1980. These guide specifications included allowable stress design (ASD) provisions developed by the consortium of University Research Teams (CURT) and approved by ballot of the AASHTO Highway Subcommittee on Bridges and Structures in November 1976. CURT consisted of Carnegie Mellon University, the University of Pennsylvania, the University of Rhode Island, and Syracuse University. The 1980 guide specifications also included load factor design (LFD) provisions developed in American Iron and Steel Institute (AISI) Project 190 and approved by ballot of the AASHTO Highway Subcommittee on Bridges and Structures in October 1979. The guide specifications covered both box and I-girders.

Changes to the 1980 guide specifications were included in the AASHTO *Interim Specifications - Bridges* for the years 1981, 1982, 1984, 1985, 1986, and 1990. A new version of the *Guide Specifications for Horizontally Curved Highway Bridges* was published in 1993. It included these interim changes and additional changes but did not reflect the extensive research on curved-girder bridges that has been conducted since 1980 or many important changes in related provisions of the straight-girder specifications.

The I-girder curvature study of the design and construction of horizontally curved girder highway bridges reflects the state of the art, both in the United States and Japan. These are the only two countries that have issued an official or semi-official guide specification for the design and construction of such bridges. The I-girder curvature study was completed as part of NCHRP Project 12-38. The following terms are used in this appendix:

- "Previous curved-girder specifications" refers to the 1993 AASHTO *Guide Specifications for Horizontally Curved Highway Bridges;*
- LFD/ASD refers to the 1992 AASHTO *Standard Specifications for Highway Bridges,* and
- LRFD refers to the 1994 AASHTO *LRFD Bridge Design Specifications.*

I. INTRODUCTION

The AASHTO *Guide Specifications for Horizontally Curved Highway Bridges* (Guide Spec) (*1*) permit curvature effect on vertical bending moments in the girders to be ignored when the included angle of a span is less than that given in Table 1.4A, shown here as Table A-1. The curvature effect on lateral flange bending, however, must be evaluated for all curved bridges regardless of their curvature. The research upon which Table A-1 is based has been lost.

Guidelines for the Design of Horizontally Curved Girder Bridges (Draft) (*2*) by Hanshin has a similar provision permitting curvature to be ignored if the bridge is only slightly curved. Hanshin allows bridges with effective spans of 5 deg, or less, to be exempt from analysis for curvature effects. The effective length of interior spans is defined as 0.6 times the average distance between bearings for each girder and 0.8 times that length for end spans of continuous bridges. The multiplier is 1.0 for simple spans.

This study was undertaken to determine appropriate rules regarding approximate analyses of curved I-girder bridges. A second purpose was to investigate guidelines regarding load application when a straight-line girder analysis is used for a curved I-girder bridge.

Table A-2 gives the number of I-girders, span lengths, and radii of the six bridges studied. The bridges are all I-girder type with flanges composed of flat plates with the web welded along their centers. Analyses have been performed using a three-dimensional (3D) finite element analysis (BSDI 3D System; Bridge Software Development International, Ltd.) and a line-girder analysis (BSDI LGS). The BSDI 3D System accounts for curvature while the BSDI LGS is based on assumptions of a lone, straight girder. Girder moments, deflections, and reactions due to dead and live load for each of two analyses are examined in each of the bridges.

II. LONGITUDINAL FLANGE STRESS COMPONENTS

A. Vertical Bending Effects

The vertical bending moment in a curved I-girder can be divided into three components. Component 1 is the moment due to load applied directly through the girder's shear center. It is computed by assuming the girder is independent from the rest of the structure.

Component 2 is the moment due to restoring forces in connecting members between girders. It is created when adjacent girders have either different stiffnesses and/or different loads, causing the deflections due to Component 1 moments in adjacent girders to be different. The connecting members between girders impose restoring forces that tend to mitigate the difference in these deflections. Effectively, load is shifted from

TABLE A-1 AASHTO Guide Specifications, Table 1.4A

Number of girders	Angle for 1 Span	Angle for 2 or more spans
2	2°	3°
3 or 4	3°	4°
5 or more	4°	5°

the girder with the larger deflection to adjacent girders. The differences in girder moments computed by the BSDI LGS and the BSDI 3D for Bridge 1 are due to Component 2 effects. Equal distribution of superimposed dead load to all stringers and the use of wheel load distribution factors for live load are examples of assumptions often used to compensate for Component 2 effects when using a single girder analysis.

Component 3 is the portion of vertical bending moment due to horizontal curvature. It is caused by the forces in connecting members between girders required for equilibrium of the bridge due to curvature. These forces provide statical equilibrium of the curved girders. Normally, curved I-girders are braced intermittently against rotation by cross frames and continuously along the top flanges by the deck. The restraining forces cause vertical force components in these members. The vertical forces cause load to be shifted between girders. Component 3 is generally characterized by a shift of load to the exterior girder and away from the interior girder.

B. Lateral Flange Bending Effects

Component 4 is lateral flange bending that causes longitudinal flange stress when the girders are subjected to nonuniform torsion. This action also occurs when a load is applied off the shear center of a girder.

Lateral flange bending causes longitudinal stresses in the flanges which vary linearly from one flange tip to an equal and opposite value across the flange. Lateral flange bending moment due to nonuniform torsion typically varies along the flange as shown in Figure A-1. Radial forces shown in the figure are from the cross frames. Axial force in the flange is due to vertical bending.

McManus (*3*) divides lateral flange bending into three effects:

- Warping stress due to nonuniform torsion,
- Radial bending due to curvature, and
- Amplification due to second order effects.

Nonuniform torsion resisted by an open section causes torsional warping stresses. It is a function of the third derivative of the cross-section rotation and the section's warping constant (*4–6*). Equilibrium requires equal and opposite action on the top and bottom flanges called the bimoment. Lateral flange bending moments at brace points can be estimated with the well known V-load equation below (*7*):

$$M_{lat} = \frac{M_v l^2}{10DR}$$

where

M_{lat} = lateral flange bending moment at brace point,
M_v = vertical bending moment,
l = distance between brace points,
D = girder depth, and
R = girder radius.

The V-load equation for lateral flange moment is derived by assuming a radial load on the flange equal to the force in the flange divided by the radius of the girder and analyzing the flange as a continuous beam on rigid intermediate supports. The direction of force is away from the center of curvature on the compression flange and toward the center of curvature on the tension flange. Although the bimoment and the lateral flange bending moments result in a similar normal stress distribution in the top and bottom flanges of a curved I-girder, they are derived based on different modelings of girders. Their units are different.

TABLE A-2 Bridge parameters

Bridge	Span(s) (ft)	Girder Depth (In.)	Number of Girders	Cntrline Radius (ft)
Bridge 1	160-210-160	84	4	Straight
Bridge 2	160-210-160	84	4	2,400
Bridge 3	160-210-160	84	4	700
Bridge 4	208	96	4	2,400
Bridge 5	128	72	4	2,400
Bridge 6	128	72	2	2,400

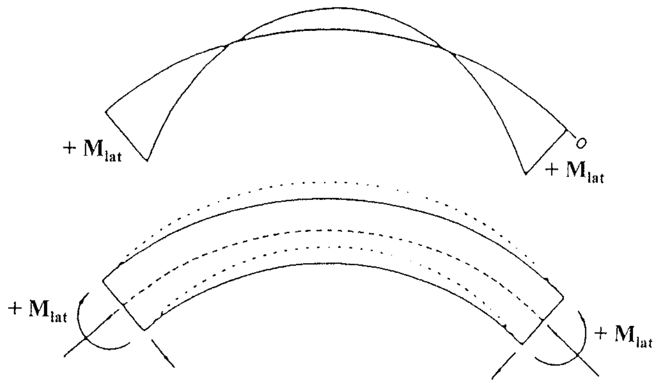

Figure A-1. Lateral flange bending diagram.

The V-load equation for lateral flange moment is appropriate when cross frames are spaced nearly equally and the cross frames are essentially full depth of the girders. It tends to be less accurate when the girder stiffnesses in the bridge cross section differ and when the bridge has skewed supports.

Implicit in application of the V-load equation for lateral flange bending is the assumption that the section is open and the shear center is within the beam cross section. If girders are attached to a deck, the shear center is above the deck. Thus, the V-load equation is not as appropriate.

A secondary effect causing lateral flange bending is the radial effect. It can be thought of as resulting from a uniform load applied to the flanges in a radial direction (*3*). The lateral flange moment varies between brace points as shown in Figure A-1. Unlike the nonuniform torsion load effect, the lateral moments in the tension and compression flanges have the same sign. These two effects can be estimated using small-deflection elastic finite elements if the flanges and web are modeled discretely.

A third load effect causing lateral flange bending is due to the amplification effect, which is due to lateral deflection of the compression flange. As the compression flange curvature increases due to load, additional bending moment is introduced by a mechanism similar to the P-delta effect. The amplification effect can be considered only by using large-deflection theory that considers the effect of the deflected shape of the structure with regard to application of force.

After a portion of the flange section commences to yield, an inelastic finite element solution is required. There are other related problems in such solutions that make application of inelastic, large-deflection finite element computer programs to curved girders problematic. Discussion of this topic is beyond the scope of this report.

C. Summary of Component Effects

Components 1 and 2 exist in virtually all girder bridges. Component 2 for dead load is small in a straight bridge where supports are not skewed and equal load is applied to each girder and girders have the same stiffness. However, vagaries, such as deck overhang, result in different dead loads applied to the girders, resulting in Component 2 moments in most practical straight bridges.

Components 3 and 4 make curved girders special and the prediction of their strength extremely difficult. Component 3 is related to the statical behavior of curved bridges. The center of gravity of the bridge is eccentric with respect to a line drawn between the center of cross section at the ends shown in Figure A-2 for the 208-ft simple span. This eccentric effect causes the entire structure to rotate about this line. The eccentricity is mitigated for continuous span bridges as shown in Figure A-3. However, when curvature is increased as shown in Figure A-4, or the spans are increased, the eccentricity increases and Component 3 effects increase. Studies have demonstrated that cross frame spacing and different girder stiffnesses do not affect stress Component 3 significantly in I-girder bridges because the torsional stiffness of an I-girder is small with respect to its vertical stiffness (*8*). Hanshin (*2*) also defines four normal stress components similar to those shown in Figure A-5.

III. ANALYSIS METHODS

A. General

Applicability of different analysis techniques can be separated by the components that each is capable of addressing. Different analysis methods are capable of determining various combinations of the load effects. However, the reliability of results from an analysis often is dependent more on appropriate application of the method rather than on the technique itself. Judgment is almost always required in applying any technique to the design of curved bridges.

B. Line Girder

The line-girder analysis is the simplest technique because it assumes vertical loads are applied to one girder. Any redistribution of load between girders must be recognized by the engineer's assumptions in determining the amount of load to be applied to the girder.

When a line-girder analysis is used for a curved bridge, each girder should be analyzed and the actual length of each girder should be recognized. Different girder lengths receiving the same load intensities result in different total load applied to each girder.

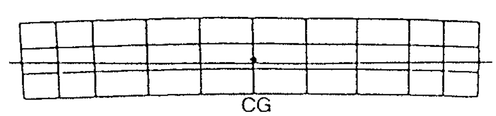

Figure A-2. Plan, Bridge 4.

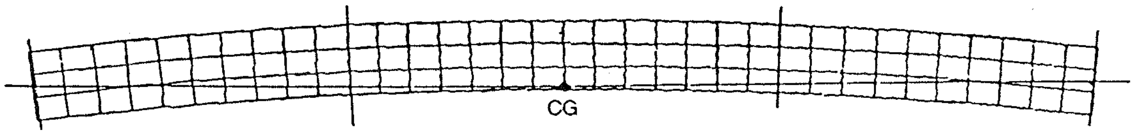

Figure A-3. Plan, Bridge 2.

Line-girder analyses recognize only Component 1 effects. Component 2 load effects cannot be considered. Application of the V-load method, however, permits extension of the line-girder method to consider Component 3 in certain circumstances. Component 4 load effects can be recognized partially by the V-load method equation given earlier.

C. Grid

Most grid analyses assume that girder cross sections remain plane so only St. Venant torsion is recognized. If an open section is used, the torsional stiffness is usually negligible. Bearings are assumed at the neutral axis because the grid model generally recognizes no section depth. For the noncomposite condition, cross frames are modeled by single beam elements. The transverse deck bending stiffness can also be recognized by the same transverse beam elements using modified properties.

Grid analyses recognize Components 1, 2, and 3. The warping portion of Component 4 is recognized when the warping degree of freedom is considered. Some grid analyses (*9–11*) account for warping by introducing an explicit kinematic degree of freedom associated with warping in the element stiffness matrix. Horizontal shear stiffness of the deck is not recognized. Lateral flange bending can be considered indirectly by the V-load method.

Live load may be treated either by applying load directly to the model or by creating influence lines or surfaces with unit loads. If live loads are to be applied directly to the model, a wheel load distribution factor may be used to determine how much load is to be applied to each girder. Alternatively, loads may be applied directly to the grid as either a series of concentrated loads and/or uniform loads. If influence surfaces, or influence lines are used, live loads are applied to the influence surfaces or lines rather than to the model. Usually some method of applying live load to the deck overhangs is needed such as "phantom" girders.

D. Finite Strip

One girder is modeled with the finite strip method. Section properties usually include warping. The method recognizes the girder connection to cross frames and the deck, which permits recognition of the contribution of the these elements to enforce structural compatibility of the girder. Thus, Components 2 and 3 are considered. The finite strip method is able to treat the first two load effects of Component 4.

Proper determination of the portion of dead load to be applied and a proper live-load wheel load distribution factor are important in the application of this method.

E. Finite Element

The 3D finite element method is capable of determining Components 1, 2, and 3, if the entire superstructure is well modeled. The method generally recognizes distortion of the girder cross section. However, recognition of distortion prevents determination of warping stresses. The four component load effects can be determined with the exception of warping stress. The amplification component cannot be determined with the finite element method unless a large-deflection solution is employed.

Benefits of the 3D finite element method include recognition of lateral forces at reactions and modeling the deck as a continuum so that horizontal shear stiffness of the deck, including shear lag, is considered. Nevertheless, stress components are generated rather than girder moments, shears, and torques. Because AASHTO bridge design specifications are written in terms of girder responses, these responses must be derived from stresses.

A 3D finite element analysis permits influence surfaces to be defined by placing unit loads on the deck nodes, including deck overhangs. Influence surface generation and subsequent loading generally requires computer post-processing because of the large amount of data generated.

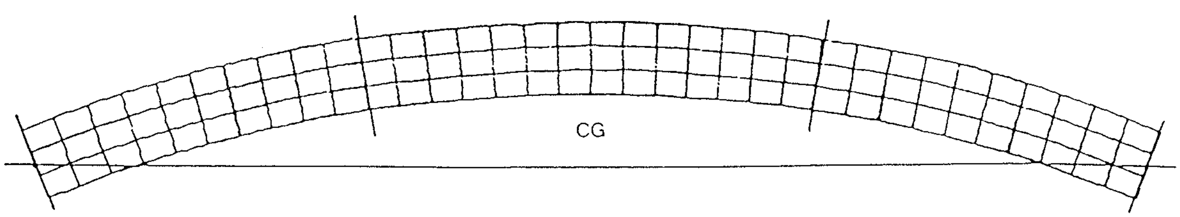

Figure A-4. Plan, Bridge 3.

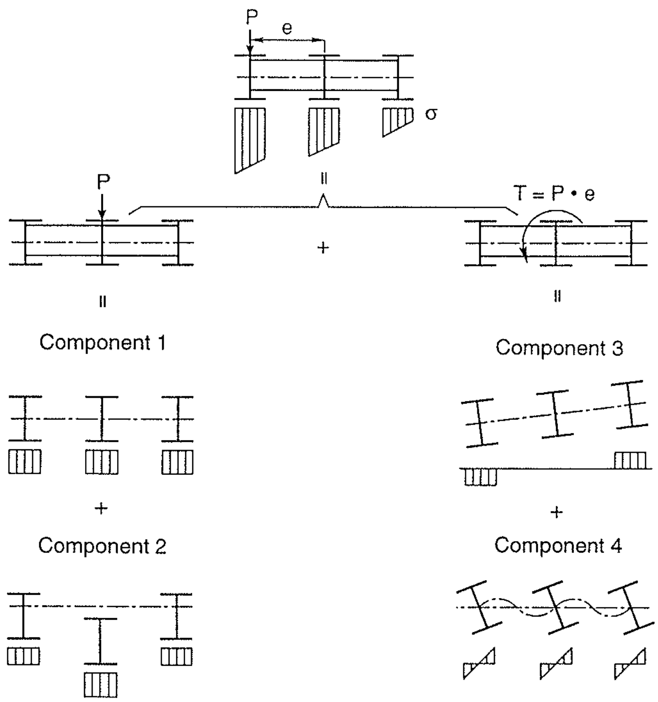

Figure A-5. Four normal stress components.

This method is used for the 3D analyses presented in this study.

F. Orthotropic Sector

The orthotropic sector method represents the bridge as a curved sector with the girders modeled as circumferential stiffeners on a flat, curved plate. This method recognizes the horizontal shear stiffness of the deck.

G. Summary of Analysis Techniques

This discussion is not an exhaustive examination of analysis techniques. There are many variations of these techniques that allow consideration of the four components in addition to those acknowledged. The engineer, however, should have a basic understanding of the four components and the analysis method's means of considering them. Each bridge is unique so the relative importance of each of the components varies greatly from bridge to bridge.

IV. STUDY DESCRIPTION

A. General

Components 1, 2, and 3 of the girders in six bridges are reported for dead and live load. Selected Component 4 results are also presented. Based on these results, recommendations are made regarding simplifying assumptions of mildly curved I-girder bridges.

B. Analyses

1. General

Two types of analyses are used in this study: line-girder method and 3D finite element method. In both types of analyses, the deck is assumed to be composite and to be effective in compression and tension. Its modulus of elasticity is computed by using the equation given in AASHTO Article 8.7.1 (*12*). A value of 3n is assumed to allow for creep when analyzing the structures for superimposed dead load.

2. Line-Girder Analysis

The line-girder analysis is made by using a series of beam finite elements with two degrees of freedom per node, which account for vertical bending and shear. Individual girders of each cross section are considered. The actual span length of each girder is used. The entire deck area is used, rather than 12 times the thickness, to compute stiffnesses used in the analyses.

3. 3D Finite Element Analysis

Girders are modeled in the 3D analyses with one shell element representing the entire web depth and one beam element representing each flange. Flange stiffness about all three axes is recognized with the beam elements. All elements are planar so curvature is approximated by kinks at each node. Cross frames are modeled as truss elements connected to the top and bottom web and flange nodes. The deck, which is modeled with eight-node solid elements, is connected to the girders with beam elements of length equal to the haunch height. Figure A-6 shows a schematic of a portion of the 3D model.

The deck is effective in horizontal shear and shear lag is considered. Poisson's ratio for concrete is assumed to be 0.15.

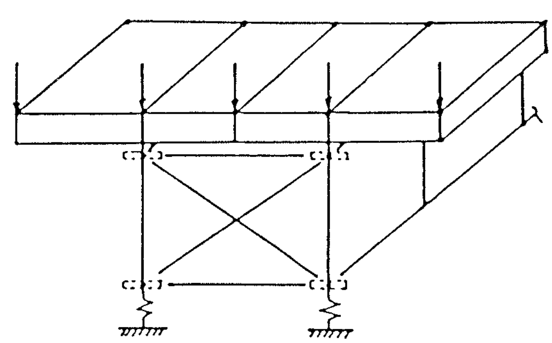

Figure A-6. 3D partial schematic.

C. Bridge Description

The six I-girder bridges in Table A-2 cover a rather wide range of curvature and framing. Bridges 1, 2, and 3 have three spans. Bridges 4, 5, and 6 have a single span. Girders are spaced at 11 ft in all of the bridges and support lines are radial. A typical cross section of Bridges 1 through 5 is shown in Figure A-7.

Bridge 1 acts as a control giving responses with no curvature effect (i.e., Components 3 and 4 are zero, so Component 2 can be isolated). The introduction of curvature in Bridges 2 and 3 permits the study of Components 3 and 4.

Bridges 4 through 6 have the same 2,400-ft radius as Bridge 2. Bridge 4 has approximately the same span as the interior span of Bridges 1 through 3; however, its girder depth has been increased. Bridges 5 and 6 have spans equal to 0.8 times the end span of Bridges 1, 2, and 3, but their girder depths have been decreased. Bridge 6 has two I-girders to investigate bridge width.

Figure A-7 shows the framing plan of Bridge 3 (radius = 700 ft) with cross frames spaced at approximately 16 ft. Cross frames are spaced at approximately 24 ft in the other bridges. All intermediate cross frames in the study are "X" type with top and bottom chord members. The area of each cross frame member is 5 in^2.

All girders within any cross section have the same size flanges and webs. The specified minimum yield stress of all girder steel is 50 ksi. The deck is assumed to be cast at one time (i.e., not staged).

All analyses are made with the assumption that the negative moment regions are fully composite. This assumption is consistent with test data regarding service loads and is the assumption used in the PennDOT *Design Manual* (*13*) and encouraged in the AASHTO LRFD (*14*). However, the deck is assumed cracked in the negative moment regions and only the reinforcing steel is assumed effective when sizing girders for strength.

Tables A-3 through A-7 shows the individual girder lengths for Bridges 2 through 6 used in the LGS analyses.

D. Deck

The deck of each bridge is 4,000 psi normal weight concrete (9 in. thick). The weight of permanent steel deck forms is considered. The haunch height is assumed to be 4 in. from the top of the web to the bottom of the deck. Haunch width is assumed to be 20 in.

E. Bearings

Supports are assumed free to translate and rotate about all axes but rigid in the vertical direction in the analyses. Thus, girders are subjected to vertical and lateral bending moments but not thrust. In the 3D analysis, stability is accomplished by restraining rotation about the vertical axis of one bearing while leaving the others free to translate in both horizontal directions and to rotate about all axes. Only vertical loads are applied.

F. Load Application

1. Steel

In the BSDI 3D System, steel weight includes the girders and cross frames, which are considered effective in the 3D System analyses. Thus, Component 2 effects are evident.

In the line girder analyses, additional load to compensate for the cross frames is added to the girder weights. Component 2 effects are not recognized in these analyses because

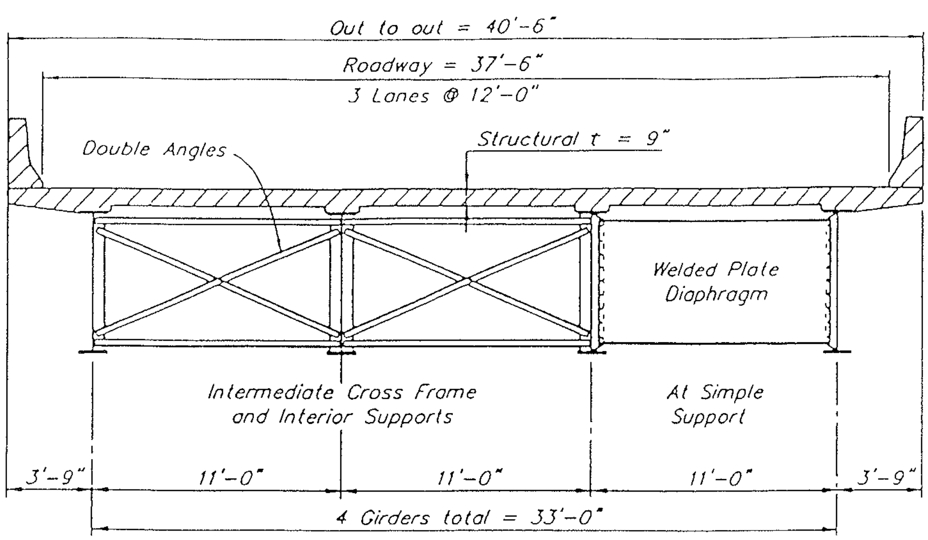

Figure A-7. Four-girder cross section.

TABLE A-3 Girder lengths for Bridge 2

Girder No.	Spans 1, 3 (ft)	Span 2 (ft)	Total (ft)	Gir. Length / Avg. Length
G1	158.90	208.56	526.36	0.993
G2	159.63	209.52	528.78	0.998
G3	160.34	210.48	531.16	1.002
G4	161.10	211.44	533.64	1.007
Average	160.00	210.00	530.00	1.000
Arc Angle Degrees	3.1 0.8 span	3.0 0.6 span		

responses are due purely to loads applied to the individual girder. Steel girder weight is recognized according to its actual location along the girders in both analysis types.

2. Deck

The deck weight is applied as a series of concentrated loads to the girders in both analysis methods. The deck load is defined as the weight per foot of contributory area of deck. On the four-girder bridges, the load applied to the exterior girders is less than the load applied to the interior girders because the weight of deck on the overhang is less than half of the weight of the deck between girders.

3. Superimposed Dead Load

Superimposed dead load includes only the two parapets. In the line-girder analyses, 320 lb/ft (64 percent) is assigned to the exterior girders and 180 lb/ft (36 percent) is assigned to the interior girders of the four-girder bridges in an attempt to represent Component 2 effects. All of the parapet load is actually applied to the exterior girders. The load transfer between girders occurs through the cross frames and the deck. In the 3D analyses, the weight of the parapets is applied to the overhanging edges of the deck.

4. Live Load

The live load is HS25 made up of a uniform load with appropriate concentrated loads, or a vehicle, whichever is greater. There are three 12-ft-wide traffic lanes on the four-girder bridges. Because the deck is 40-ft wide, each of the three 12-ft-wide lanes can be shifted laterally up to 4 ft. Thus, the lateral position of live load to produce maximum moments in the right-most girder occurs when the traffic lanes are as close to the right curb as possible. The opposite is true for the left-most girder. There is only one traffic lane on the two-girder bridge.

Wheel load distribution factors are not used in the 3D analyses. Instead, individual unit loads are applied downward to the deck surface so that influence surfaces can be produced for the necessary responses. Live loads are applied to these influence surfaces with a special-purpose computer program. The live load is 6-ft wide and is allowed to move laterally to within 2 ft of the edge of its lane. Live-load responses due to three lanes of traffic are reduced to 90 percent for multiple-presence allowance according to AASHTO (*12*).

In the line-girder analyses, a wheel load distribution factor (WLDF) of 2.0 (11 ft/5.5) is used for the four-girder bridges. A wheel load of 1.57 (11 ft/7.0) is used for the single lane of traffic on Bridge 6. The WLDFs in AASHTO are an attempt to account for the effect of Component 2 with

TABLE A-4 Girder lengths for Bridge 3

Girder No.	Spans 1 & 3 (ft)	Span 2 (ft)	Total (ft)	Gir. Length / Avg. Length
G1	156.23	205.05	517.51	0.976
G2	158.74	208.35	525.83	0.992
G3	161.26	211.65	534.17	1.008
G4	165.77	214.95	542.49	1.024
Average	160.00	210.00	530.00	1.000
Arc Angle Degrees	10.9 0.8 span	10.6 0.6 span		

TABLE A-5 Girder lengths for Bridge 4

Girder Number	Span 1 (ft)	Gir. Length / Avg. Length
G1	206.57	0.993
G2	207.52	0.998
G3	208.48	1.002
G4	209.43	1.007
Average	208.00	1.000
Arc angle Degrees	5.0 1.0 span	

regard to live load. The S/5.5 WLDF is developed assuming that two lanes of traffic are placed in the critical lateral position with respect to each girder. Similar girder stiffnesses and spacing are assumed. If stiffnesses of the girders in the cross section vary, the WLDFs give results too small for the stiffer girders and too large for the less stiff girders. Typically, exterior girders that are deeper than interior girders receive a much larger portion of load than determined using the AASHTO WLDFs. In curved-girder bridges, it is not unusual to find the exterior girder on the outside of the curve to be heavier than the others, which will cause that girder to attract more load. Although AASHTO differentiates between exterior and interior girders when specifying WLDFs, this distinction has not been considered.

In the line-girder analyses, WLDF for live-load deflection of 1.50 (6 wheels/4 girders) is used for the four-girder bridges and a WLDF of 1.0 is used for the two-girder bridge. Thus, live-load deflections are assumed to be equal for all girders.

The designs are made assuming the AASHTO WLDF, S/5.5. AASHTO has recently introduced new WLDFs in the *Guide Specifications for Distribution of Loads for Highway Bridges* (*15*). The AASHTO LRFD has introduced a variation of these factors (*14*). The LRFD provisions use the "lever" method for certain girder arrangements including exterior girders. Use of WLDFs different from S/5.5 and S/7.0 would obviously affect the live-load comparisons here. The new WLDFs (except for the lever rule) are developed for bridges with no cross frames (*15*). This assumption is not appropriate for curved girders and is of questionable validity for almost any steel girder bridge. In the other extreme, the lever method assumes rigid behavior of the bridge cross section.

TABLE A-6 Girder lengths for Bridge 5

Girder No.	Span 1 (Feet)	Ratio
G1	127.12	0.993
G2	127.71	0.998
G3	128.29	1.002
G4	128.88	1.007
Average	128.00	1.000
Arc angle Degrees	3.1 1.0 span	

TABLE A-7 Girder lengths for Bridge 6

Girder No.	Span 1 (ft)	Ratio
G1	127.71	0.998
G2	128.29	1.002
Average	128.00	1.000
Arc angle Degrees	3.1 1.0 span	

Tables A-8 and A-9 show a comparison of the new AASHTO *Guide Specifications for Distribution of Loads for Highway Bridges,* the new AASHTO LRFD, and the AASHTO classical S-over methods to the equivalent WLDFs developed from the 3D analyses for Bridge 1. For example, to determine the 3D equivalent WLDF for G4 at the pier, the following procedure is used. From Table I-8 (Annex I), the 3D live-load moment is –4,630 k-ft, while the LGS moment is –4,856 k-ft. The LGS moment is determined from a WLDF of 2.0, which means that one lane of traffic (2 wheels) is applied to the girder. It is possible to determine the 3D equivalent WLDF by multiplying the ratio of the 3D moment divided by the LGS moment by 2.0. Using this approach, the equivalent 3D WLDF equals $2.0 \times (-4{,}630/-4{,}856) = 1.91$. Equivalent WLDFs for the single-lane condition are developed using values for one lane loaded. WLDFs for G3 are developed in a similar manner.

Because G4 is the critical exterior girder, it is used in determining the 3D WLDFs for the exterior girder. The WLDFs for G3 and G4, Bridge 1, are shown in Tables A-8 and A-9, respectively. The lines denoted by "(M)" are for the multiple-lane condition while those denoted by "(S)" are for the single-lane condition. Moments for a single-lane straight bridge are not reported.

AASHTO Article 3.23.2.3 (*12*) specifies that the simple beam method of distribution be used for the exterior girders; however, the exterior stringer is required to have not less capacity than an interior stringer. Therefore, a WLDF of S/5.5 is used for the exterior girders. Generally, S/5.5 in the LGS analysis gives G4 moment closer to the 3D analysis for Bridge 1 than does either the new Guide Specification for Load Distribution or the AASHTO LRFD. Ratios of the WLDFs by the three methods are divided by the equivalent 3D WLDFs as shown in Tables A-8 and A-9. A ratio less than 1.0 indicates that the approximate method gives unconservative live-load moments compared to the 3D analysis.

TABLE A-8 Wheel load distribution factors for G3, Bridge 1 (interior girder)

Case	AASHTO Guide (1)	AASHTO LRFD (2)	AASHTO S/Over (3)	3D Equiv (4)	Ratio (1)/(4)	Ratio (2)/(4)	Ratio (3)/(4)
160(M)	1.49	1.42	2.00	1.45	1.03	0.98	1.38
(S)	0.98	0.93	1.57	0.83	1.18	1.12	1.89
210(M)	1.43	1.36	2.00	1.44	0.99	0.94	1.39
(S)	0.91	0.87	1.57	0.72	1.26	1.21	2.18
Pier(M)	1.53	1.39	2.00	1.56	0.98	0.89	1.28
(S)	0.99	0.90	1.57	0.81	1.22	1.11	1.94

V. RESULTS

A. General

Reactions, maximum moments, and maximum deflections for each girder of each bridge are reported. Results are presented separately for steel, deck, parapet, and live load. Results by both LGS and 3D analyses are presented.

Dead- and live-load reactions for the 3D and line-girder analyses are compared in Tables I-1 through I-4. End reactions are identified as R1; and interior reactions on the continuous span bridges are identified as R2. The ratio computed by dividing each 3D value by the corresponding LGS value is shown for each pair of data. The sums of the reactions for each girder also are presented. A comparison of the sums of dead-load reactions for the two types of analyses shows that the dead loads applied in the two analyses are essentially equal. Thus, differences in responses are due to different assumptions in loading and to different analytical considerations, not different dead loads.

Reactions of Bridge 3 demonstrate the effect of curvature on multispan girder bridges. Interior reactions, R2, due to deck weight on G1 are larger than the comparable LGS reactions, while the 3D end reactions, R1, on G1 are smaller than the comparable LGS reactions. Girder G4 exhibits the reverse behavior. This effect is due to the bridge attempting to rotate about a longitudinal axis through its center of gravity. The center of weight of the end spans times their distance from this axis counters the center of weight of the center span times its distance from the same axis. These differences in reactions are manifest in the 3D analysis because static equilibrium of the entire bridge must be satisfied. This effect is typical of multispan curved-girder bridges.

Maximum moments in each span and at the piers of the continuous bridges are reported for steel, deck, parapet, and live load in Tables I-5 through I-16.

By comparing the sums of responses for dead-load moments in the four girders for the 3D and LGS analyses at a cross section, the accuracy of the analyses can be ascertained. The statical moment at any section in a structure is a constant, regardless of the analysis. It equals the sum of the LGS dead-load moments in the girders at a cross section. The sum of the 3D moments should equal the comparable LGS sum. The sums of the 3D and LGS dead-load moments are, in fact, very nearly equal. It is not possible to make a similar comparison for live-load moments because the critical moment for each girder occurs from a different live-load position.

Maximum deflections in each span for steel, deck, and parapet loads are presented in Tables I-17 through I-20. Deflection is particularly important in curved bridges because unpredicted deflections have been the source of many problems during construction. One of the significant differences between curved and straight bridges is the difference in deflection between the inside and outside girders.

Live-load deflections by the LGS are made assuming that the girders participate uniformly for all three traffic lanes loaded. On Bridges 1 through 5, the WLDF for deflection equals 1.5 (6 wheels/4 girders). For Bridge 6, the factor is

TABLE A-9 Wheel load distribution factors for G4, Bridge 1 (exterior girder)

Case	AASHTO Guide (1)	AASHTO LRFD (2)	AASHTO S/Over (3)	3D Equiv (4)	Ratio (1)/(4)	Ratio (2)/(4)	Ratio (3)/(4)
160(M)	1.58	1.45	2.00	1.78	0.89	0.81	1.12
(S)	1.31	1.25	1.57	1.19	1.10	1.05	1.32
210(M)	1.58	1.39	2.00	1.78	0.89	0.78	1.12
(S)	1.31	1.25	1.57	1.19	1.10	1.05	1.32
Pier(M)	1.65	1.42	2.00	1.91	0.86	0.74	1.05
(S)	1.38	1.25	1.57	1.32	1.05	0.95	1.19

computed for one lane on two girders, so the factor is 1.0. The 3D analysis uses influence surfaces to determine maximum live-load deflections. The 3D live-load deflections given are due to 90 percent of three lanes loaded, plus impact.

Table A-10 gives allowable live-load deflections based on span over 800 and span over 1,000. Comparison of permitted values with 3D tabulated values shows that G4 deflections exceed the limit ocassionally, while the other girders do not.

B. Component 1

Component 1 is simply the result of the LGS analyses of Bridge 1, which is the result for loads assigned to the individual girders. In the cases of steel weight and deck weight, the applied loads are computed with no distribution. Parapet weight is assigned in the LGS analyses as discussed earlier. Thus, parapet moments from the LGS are not truly Component 1, because all of the parapet load is actually applied to the exterior girders. Moments due to one parapet can be estimated in Bridge 1 as the sum of parapet moments in G1 and G2. Live-load results from the LGS are based on a WLDF that also considers distribution, thus LGS responses for parapets and live load are not true Component 1 values.

C. Component 2

Component 2 can be determined from the differences between the LGS and the 3D responses for Bridge 1. If loads are assigned perfectly in the LGS analyses to account for load transfer, Component 2 would appear to be zero by this method. In the case of steel and deck loads, the difference in LGS and 3D results gives a pure Component 2. Because an assumption of parapet load distribution is made in anticipation of Component 2, the true Component 2 is larger than reported.

The sums of LGS reactions for each dead load for a girder give the total dead load applied to the individual girders. Table I-2a shows that the deck load applied to G1 in Bridge 1 equals 702 kips and the deck load applied to G2 is 784 kips. The difference in load between G1 and G2 is 82 kips. This difference occurs because the deck overhang is less than half of the girder spacing. In the 3D analysis, the difference in load applied to each girder is the same; however, the difference in total reactions of G1 and G2 is only 18 kips, or 2 percent. The change is due to the action of the cross frames that is recognized in the 3D analysis. The cross frames transfer 64 kips (82 − 18) of dead load from each interior girder to each exterior girder. Thus, the total vertical component of the force in the cross frames in the bay between G1 and G2 is 64 kips for the dead load. This shift of load is the cause of Component 2 moments. Similar comparisons of the data from Bridge 1 can be made.

Tabulations of differences of the 3D to LGS ratios for reactions, moments, and deflections for G4, Bridge 1, are presented in Tables I-21 through I-24. These values represent Component 2 as a portion of Component 1. Because the cross section of the four-girder bridges in this study are the same, Component 2 is assumed to be the same for the other bridges except for Bridge 6. It is reasonable to assume that half of the dead load in each case is assigned to the two girders in Bridge 6. Live-load Component 2 cannot be evaluated for Bridge 6.

Table I-10a shows that for Bridge 1, the difference between G1 and G2 LGS deck moments in the 160-ft span is 236 k-ft. The comparable difference in 3D moments in G1 and G2 is nearly zero, which indicates that a shift of approximately 115 k-ft from G2 to G1 occurred. The shifted moment is Component 2. The 3D analysis shows that a reasonable distribution of deck load for Bridge 1 would have been to distribute the total deck weight per unit length, equally to the girders.

The loading used in the LGS analyses represents what would happen if no cross frames were effective when the deck is cast. The actual deflections of the girders need to be considered when specifying their camber. In this case, there is essentially no difference in deflection of the girders of Bridge 1 due to deck load when cross frames are acting.

D. Component 3

Comparisons of the LGS analyses with the 3D analyses for the curved girders show a combined effect of Components 2 and 3, so it is difficult to identify Component 3 directly. Because Component 2 has already been defined, Component 3 can be determined by subtracting Component 2 from the total difference between the 3D results and the LGS results for the curved bridges.

Using the deck moments in Table I-10a, Component 3 moments can be determined for the curved four-girder bridges. It is assumed that Component 2 remains 0.06 times Component 1 in G4 for Bridges 2 through 5. For G1, Bridge 2, Component 3 equals −0.04 (1.02 − 1.06) times Component 1. For G1, Bridge 3, Component 3 equals −0.21 (0.85 − 1.06) times Component 1. For G4, Bridge 2, a comparable value is: 1.11 − 1.06 = 0.05. For G4, Bridge 3, a comparable value is: 1.23 − 1.06 = 0.17. Moments in G2 and G3 computed by 3D analysis for Bridges 2 and 3 remain nearly the same percent of the LGS moment as for Bridge 1. Thus, Component 3 moment for these girders is essentially zero. Other comparisons of Components 2 and 3 can be made from the data reported.

TABLE A-10 Allowable live-load deflections (in.)

Limit	Span (ft)		
	128	160	210
Span/800	1.92	2.40	3.15
Span/1,000	1.54	1.92	2.52

Comparison of live-load moments involves different assumptions. Because the same WLDF was used for G1 and G2, their LGS live-load moments are the same for Bridge 1. The Bridge 1 3D live-load moment of G1 at the pier is 0.93 times the LGS value in Bridge 1 as seen in Table I-8. In the interior girders, the LGS live-load moments are larger than the comparable 3D moments in Bridge 1. The reason for this reversal is that the WLDF underestimates the ability of the cross frames and deck to transfer live load from the interior girders.

E. Component 4

Bottom flange lateral bending moments by the V-load equation and the 3D analyses are reported for dead and live loads at the center of the 210-ft span and at the piers for Bridges 1, 2, and 3 in Tables I-24 and I-25. The ratio of the 3D value divided by the V-load value are presented. The 3D values for Bridge 1 are 0 for dead load as expected, however, live load produces some lateral bending. Lateral flange bending is typically ignored in design of straight girders.

The V-load method provides a reasonable estimate of the lateral bending moment in these examples where lateral bending is due to only curvature. Even in Bridge 2 with a radius of 2,400 ft, the lateral flange moments are of magnitude that should not be ignored in design.

The lateral flange bending moment, M_{lat}, can be added to the vertical bending stress to cause full yielding of the flange as shown in Figure A-8. This situation is normally assumed to occur at cross frame locations (*1*).

The elastic lateral flange moment equals $M_{lat} = f_w \dfrac{B^2}{6}$, where B equals the flange width. From Figure A-8, the plastic lateral flange moment can be written as $M_{lat} = f_y C(B - C)$. The thickness of the flange has been ignored.

But the elastic and plastic lateral flange moments are equal. The portion of the flange required for the lateral moment, C, can be determined by solving the quadratic equation:

$$f_y C(B-C) = f_w \frac{B^2}{6}$$

Rewriting:

$$C^2 - BC + \frac{f_w}{f_y}\frac{B^2}{6} = 0$$

Solving for C:

$$C = \frac{B}{2} \pm \frac{1}{2}\sqrt{B^2 - \frac{2 f_w B^2}{3 f_y}}$$

The vertical bending stress, f_b, permitted with the plastic lateral moment can be computed as follows:

$$f_b = f_y\left(\frac{B-2C}{B}\right)$$

If the vertical bending stress, f_b, plus one-third of the lateral bending stress, f_w, is less than the specified minimum yield stress of the flange, there is adequate flange capacity to develop simultaneously the plastic lateral flange moment and the axial force due to vertical bending.

F. Cross Frames

Cross frames are an important element in curved I-girder bridges because they transmit loads between the girders required for equilibrium. If a straight-girder analysis is to suffice for a curved-girder bridge, then the cross frame members should also be adequate. Without an analysis that considers these members, there are two possible approaches. It can be assumed that the nominal strength of these members is adequate. Alternatively, it is possible to assume some arbitrary strength be used to ensure their adequacy.

To evaluate these alternatives, the forces in the critical cross frame diagonal member for the six bridges from the 3D analyses are summarized in Table A-11. The members are from the bay between G4 and G3 at the piers in the three-span bridges and from the bay near midspan of the simple-span bridges. It is apparent that as curvature increases, the cross frame forces increase.

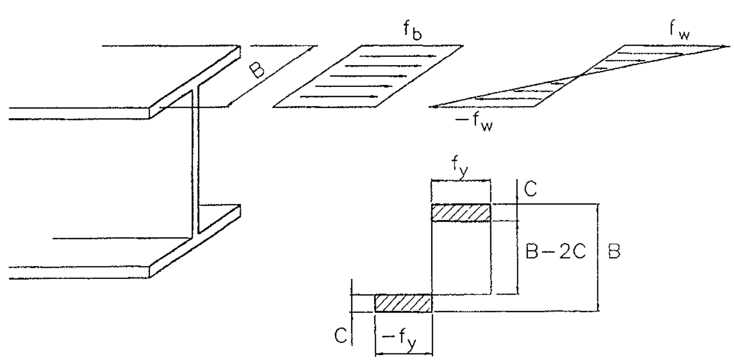

Figure A-8. Combined lateral bending and vertical bending stresses.

TABLE A-11 Critical diagonal cross frame unfactored forces (kips)

Bridge	Steel	Deck	Para-pet	Live Load	Total
Bridge 1	-1	-4	-7	-4	-16
Bridge 2	-2	-8	-8	-6	-24
Bridge 3	-5	-19	-11	-12	-47
Bridge 4	3	9	-10	13	15
Bridge 5	1	5	-12	-10	-16
Bridge 6	1	4	-3	4	6

As with moments, the results from Bridge 1 can be thought of as Component 2 in the cross frames. The difference between the cross frame forces in Bridges 2 and 3 and those in Bridge 1 are due to Component 3. In Bridges 4 through 6, the reported forces at midspan are not comparable to those in Bridge 1.

These results are anecdotal because other parameters may affect their magnitude. If the girder spacing were wide and the overhang smaller, cross frame forces due to deck load would be larger because the relative difference in loads applied to the girders would be greater. If the deck overhang were larger, live-load force would be larger. If the girder flexibility is increased, cross frame forces increase because relative deflection between the girders increases. Larger cross frame spacing would also cause an increase in their forces.

The Component 3 forces in Bridge 2 are 50 percent greater than Component 2 forces, but increase 300 percent over Component 2 in Bridge 3. The Component 3 forces in the simple-span bridges appear to be less critical than those at the piers of continuous spans.

G. Summary

The responses from analyses of curved-girder bridges can be divided into components. Component 2 occurs when the energy in a straight-girder bridge is distributed according to the minimum energy theorem. This component causes changes in the girder reaction, moments, and deflections. It does not, however, cause an increase in the total moment in the bridge at a cross section. These changes are manifest in forces in the cross frames or diaphragms and the deck connecting adjacent girders. In addition, Components 3 and 4 exist in curved-girder bridges.

Results from this study show that in normal I-girder bridges, Component 2 moments vary between plus and minus 10 percent of Component 1, except for parapet loading, which may have a larger variation. However, the parapet load is usually small so the larger variation is not significant. There may be exceptions that deserve special consideration such as sound barriers.

Component 3, as a percent of Component 1, varies much more widely. When the curve within a span is greater than 3 deg, Component 3 usually exceeds 10 percent. Because Component 2 ranges up to 10 percent and is ignored in most designs, it is reasonable to ignore Component 3 when it is less than 10 percent. Therefore, Component 3 does not need to be considered for I-girder bridges with curvature within an effective simple span less than 3 deg.

Although Component 4 is small in many curved bridges, the research team recommends that it always be considered.

Nominal cross frame designs suffice for Component 2 forces in normal straight bridges. In curved bridges of moderate curvature, these members appear to have relatively small Component 3 forces, although, as a portion of Component 2, they appear to be significant. Perhaps there could be specific cross frame details for curved bridges that provide minimal strength and proper connections. Cross frames on curved-girder bridges should be required to have a top chord because the noncomposite girder causes pronounced moments in the cross frames.

The bridge capacity at any cross section is of interest. Review of the moment tables shows that the total moment capacities of the four-girder bridges are nearly constant at any comparable cross section, regardless of curvature. Thus, if the bridge is designed for the moments in the girders, the total moment capacity of the four girders in the bridge is essentially the same regardless of the curvature. However, the method of computing the live load gives highly variable moments.

In the four-girder bridges, the WLDF is 2.0, which results in one lane of traffic per girder, for a total capacity of four lanes. However, only three 12-ft traffic lanes fit on the deck cross section. Applying the AASHTO multiple presence factor of 0.90 yields a maximum design condition of 2.7 lanes. According to the LGS analysis of Bridges 1, 2, and 3, the total live-load pier moment only varies from −19,300 to −19,307 k-ft as seen in Table I-8. The 3D results have a similar range, although the total is less. Since the WLDF is 2.0, the required live load for 2.7 lanes of traffic capacity can be computed as:

$$-19{,}300 \times \frac{6 \text{ wheels} \times 0.90}{2 \text{ wheels/girder}} = -13{,}032 \text{ k-ft}$$

The total 3D live-load moment is about −16,300 k-ft at the piers. This value is greater than the 13,000 k-ft required for 2.7 lanes. In G4, Bridge 3, the 3D and LGS live-load moments are essentially equal. In G4, Bridge 2, the difference is only 5 percent. Table A-9 shows that the LRFD WLDF would give a live-load moment in G4 that is only 74 percent of that from the 3D analysis. Thus, an approximate design of Bridge 2 using the LRFD WLDFs would cause an unconservative prediction of live load in G4.

VI. CONCLUSIONS

These results indicate that a line-girder analysis of curved bridges with 3 deg of curvature between points of zero moment provides dead- and live-load reactions, moments, and deflections within 10 percent of those from a refined 3D finite element analysis if S/5.5 is used for live load and a reasonable estimate of superimposed dead load is used. Therefore, the proposed curved-girder specification should allow a straight-girder analysis to be used for vertical bending effects for bridges with no skew and spans with curvature of 3 deg or less if an S/5.5 WLDF is used for all girders. This WLDF should also be used for live-load deflection of the outside girder rather than uniform participation. The straight line analysis appears to be appropriate for the first three components of stress discussed earlier.

The effect of curvature on the fourth stress component, however, must be considered in all curved girders. This is the effect of nonuniform torsion that is manifest in lateral flange bending stress. All bridges with curved girders must be designed by using the curved-girder provisions. The V-load equation used in this report is adequate to predict lateral flange bending for curved bridges.

ANNEX I

ANALYSIS RESULTS

TABLE I-1a Steel reactions (kips)

Girder Reaction		Bridge 1 Straight			Bridge 2 R = 2,400 ft			Bridge 3 R = 700 ft		
		LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	R1	18	20	1.11	18	19	1.06	18	16	0.89
	R2	78	80	1.03	78	81	1.04	77	81	1.05
	Total	192	200	1.04	192	200	1.04	190	194	1.02
G2	R1	20	21	1.05	20	21	1.05	20	20	1.00
	R2	85	82	0.96	85	81	0.95	84	80	0.95
	Total	210	206	0.98	210	202	0.96	208	200	0.96
G3	R1	20	21	1.05	20	22	1.10	20	22	1.10
	R2	85	82	0.96	85	82	0.96	85	86	1.01
	Total	210	206	0.98	210	208	0.99	210	216	1.03
G4	R1	18	20	1.11	18	21	1.17	19	23	1.21
	R2	78	80	1.03	78	80	1.03	80	78	0.98
	Total	192	200	1.04	192	202	1.05	198	202	1.02
Total		804	812	1.01	804	812	1.01	806	812	1.01

TABLE I-1b Steel reactions (kips)

Girder	Bridge 4			Bridge 5			Bridge 6		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	58	48	0.83	20	19	0.95	20	18	0.90
G2	61	59	0.97	22	22	1.00	--	--	--
G3	62	65	1.05	22	23	1.05	--	--	--
G4	59	70	1.19	20	22	1.10	20	23	1.15
Total	240	242	1.01	84	86	1.02	40	41	1.03

TABLE I-2a Deck reactions (kips)

Girder Reaction		Bridge 1 Straight LGS	Bridge 1 Straight 3D	Bridge 1 Straight Rat	Bridge 2 R = 2,400 ft LGS	Bridge 2 R = 2,400 ft 3D	Bridge 2 R = 2,400 ft Rat	Bridge 3 R = 700 ft LGS	Bridge 3 R = 700 ft 3D	Bridge 3 R = 700 ft Rat
G1	R1	74	78	1.05	73	74	1.01	72	63	0.88
	R2	277	290	1.05	275	291	1.06	271	292	1.08
	Total	702	736	1.05	696	730	1.05	686	710	1.03
G2	R1	83	80	0.96	83	79	0.95	82	76	0.93
	R2	309	297	0.96	308	294	0.95	307	291	0.95
	Total	784	754	0.96	782	746	0.95	778	734	0.94
G3	R1	83	80	0.96	83	81	0.98	83	84	1.01
	R2	309	297	0.96	310	301	0.97	312	313	1.00
	Total	784	754	0.96	786	764	0.97	790	794	1.01
G4	R1	74	78	1.05	75	82	1.09	76	91	1.20
	R2	277	290	1.05	279	288	1.03	284	279	0.98
	Total	702	736	1.05	708	740	1.05	720	740	1.03
Total		2,972	2,980	1.00	2,972	2,980	1.00	2,974	2,978	1.00

TABLE I-2b Deck reactions (kips)

Girder	Bridge 4 LGS	Bridge 4 3D	Bridge 4 Rat	Bridge 5 LGS	Bridge 5 3D	Bridge 5 Rat	Bridge 6 LGS	Bridge 6 3D	Bridge 6 Rat
G1	137	118	0.86	84	82	0.98	85	76	0.89
G2	153	142	0.93	94	90	0.96	--	--	--
G3	154	156	1.01	95	93	0.98	--	--	--
G4	139	169	1.22	85	95	1.12	85	94	1.11
Total	583	585	1.00	358	360	1.01	170	170	1.00

TABLE I-3a Parapet reactions (kips)

Girder/ Reaction		Bridge 1 Straight			Bridge 2 R = 2,400 ft			Bridge 3 R = 700 ft		
		LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	R1	18	18	1.00	18	17	0.94	17	14	0.82
	R2	66	66	1.00	66	66	1.00	65	64	0.98
	Total	168	168	1.00	168	166	0.99	164	156	0.95
G2	R1	10	11	1.10	10	10	1.00	10	10	1.00
	R2	37	37	1.00	37	36	0.97	37	36	0.97
	Total	94	96	1.02	94	92	0.98	94	92	0.98
G3	R1	10	11	1.10	10	11	1.10	10	11	1.10
	R2	37	37	1.00	37	38	1.03	37	40	1.08
	Total	94	96	1.02	94	98	1.04	94	102	1.09
G4	R1	18	18	1.00	18	18	1.00	18	20	1.11
	R2	66	66	1.00	66	67	1.02	68	67	0.99
	Total	168	168	1.00	168	170	1.01	172	174	1.01
Total		524	528	1.01	524	526	1.00	524	524	1.00

TABLE I-3b Parapet reactions (kips)

Girder	Bridge 4			Bridge 5			Bridge 6		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	33	24	0.73	20	19	0.95	32	28	0.88
G2	18	22	1.22	11	11	1.00	--	--	--
G3	19	25	1.39	11	12	1.10	--	--	--
G4	33	33	1.00	20	22	1.10	32	35	1.09
Total	102	104	1.02	62	64	1.03	64	63	0.98

TABLE I-4a Live-load reactions (kips)

Girder/ Reaction		Bridge 1 Straight			Bridge 2 R = 2,400 ft			Bridge 3 R = 700 ft		
		LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	R1	107	96	0.90	107	94	0.88	105	87	0.83
	R2	244	218	0.89	243	218	0.90	239	216	0.90
	Total	702	628	0.89	700	624	0.89	688	606	0.88
G2	R1	107	83	0.78	107	82	0.77	106	82	0.77
	R2	244	193	0.79	244	190	0.78	242	188	0.78
	Total	702	552	0.79	702	544	0.77	696	540	0.78
G3	R1	107	83	0.78	108	85	0.79	107	89	0.83
	R2	244	193	0.79	245	194	0.79	245	201	0.82
	Total	702	552	0.79	706	558	0.79	704	580	0.82
G4	R1	107	96	0.90	107	100	0.93	108	108	1.00
	R2	244	218	0.89	246	217	0.88	248	209	0.84
	Total	702	628	0.89	742	634	0.85	712	634	0.89
Total		2,808	2,360	0.84	2,850	2,360	0.83	2,800	2,360	0.84

TABLE I-4b Live-load reactions (kips)

Girder	Bridge 4			Bridge 5			Bridge 6		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	132	110	0.83	99	86	0.87	79	72	0.91
G2	133	101	0.76	100	80	0.80	--	--	--
G3	133	109	0.82	100	81	0.81	--	--	--
G4	133	128	0.96	101	92	0.91	79	78	0.99
Total	531	448	0.84	401	339	0.85	158	150	0.95

TABLE I-5 Steel moments (k-ft), pier

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	-1,349	-1,404	1.04	-1,340	-1,353	1.01	-1,286	-1,244	0.97
G2	-1,464	-1,412	0.96	-1,460	-1,395	0.96	-1,441	-1,375	0.95
G3	-1,464	-1,412	0.96	-1,470	-1,435	0.98	-1,487	-1,509	1.01
G4	-1,349	-1,404	1.04	-1,361	-1,457	1.07	-1,413	-1,586	1.12
Total	-5,626	-5,632	1.00	-5,631	-5,640	1.00	-5,627	-5,714	1.02

TABLE I-6 Deck moments (k-ft), pier

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	-5,096	-5,457	1.07	-5,080	-5,255	1.03	-4,870	-4,829	0.99
G2	-5,693	-5,506	0.97	-5,682	-5,436	0.96	-5,594	-5,357	0.96
G3	-5,693	-5,506	0.97	-5,754	-5,596	0.97	-5,773	-5,882	1.02
G4	-5,096	-5,457	1.07	-5,131	-5,665	1.10	-5,340	-6,164	1.15
Total	-21,578	-21,926	1.02	-21,547	-21,952	1.01	-21,577	-22,232	1.03

TABLE I-7 Parapet moments (k-ft), pier

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	-1,196	-1,106	0.93	-1,180	-1,057	0.90	-1,140	-953	0.84
G2	-671	-860	1.28	-661	-843	1.28	-661	-824	1.25
G3	-671	-860	1.28	-678	-880	1.30	-682	-946	1.39
G4	-1,196	-1,106	0.93	-1,230	-1,158	0.94	-1,253	-1,282	1.02
Total	-3,734	-3,932	1.05	-3,749	-3,938	1.05	-3,736	-4,006	1.07

TABLE I-8 Live-load moments (k-ft), pier

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	-4,825	-4,476	0.93	-4,795	-4,317	0.90	-4,632	-3,927	0.85
G2	-4,825	-3,663	0.76	-4,803	-3,594	0.75	-4,759	-3,513	0.74
G3	-4,825	-3,663	0.76	-4,849	-3,751	0.77	-4,893	-4,029	0.82
G4	-4,825	-4,476	0.93	-4,856	-4,630	0.95	-5,023	-4,961	0.99
Total	-19,300	-16,278	0.84	-19,303	-16,292	0.84	-19,307	-16,430	0.85

TABLE I-9a Steel moments (k-ft), 160-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	509	521	1.02	492	489	0.99	483	405	0.84
G2	556	522	0.94	546	510	0.93	544	482	0.89
G3	556	522	0.94	567	532	0.94	582	543	0.93
G4	509	521	1.02	534	553	1.04	531	625	1.18
Total	2,130	2,086	0.98	2,139	2,084	0.97	2,120	2,055	0.97

TABLE I-9b Steel moments (k-ft)

Girder	Bridge 4			Bridge 5			Bridge 6		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	3,260	2,706	0.83	683	645	0.94	105	87	0.83
G2	3,452	3,167	0.92	752	687	0.91	--	--	--
G3	3,484	3,617	1.04	754	728	0.97	--	--	--
G4	3,351	4,067	1.21	702	768	1.09	108	108	1.00
Total	13,547	13,557	1.00	2,891	2,818	0.97	213	195	0.92

TABLE I-10a Deck moments (k-ft), 160-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	2,043	2,170	1.06	1,994	2,025	1.02	1,953	1,656	0.85
G2	2,279	2,174	0.95	2,268	2,126	0.94	2,243	2,003	0.89
G3	2,279	2,174	0.95	2,287	2,220	0.97	2,382	2,322	0.97
G4	2,043	2,170	1.06	2,083	2,310	1.11	2,141	2,633	1.23
Total	8,644	8,688	1.01	8,632	8,681	1.01	8,719	8,615	0.99

TABLE I-10b Deck moments (k-ft)

Girder	Bridge 4			Bridge 5			Bridge 6		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	7,104	6,245	0.88	2,690	2,692	1.00	2,715	2,474	0.91
G2	7,977	7,316	0.92	3,031	2,872	0.95	--	--	--
G3	8,051	8,365	1.04	3,035	3,043	1.00	--	--	--
G4	7,301	9,408	1.29	2,765	3,210	1.16	2,740	3,114	1.14
Total	30,433	31,334	1.03	11,521	11,817	1.03	5,455	5,588	1.02

TABLE I-11a Parapet moments (k-ft), 160-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	503	405	0.81	490	375	0.77	481	298	0.62
G2	284	375	1.32	277	366	1.32	279	342	1.23
G3	284	375	1.32	291	385	1.32	288	405	1.41
G4	503	405	0.81	524	434	0.83	528	499	0.95
Total	1,574	1,560	0.99	1,582	1,560	0.99	1,576	1,544	0.98

TABLE I-11b Parapet moments (k-ft)

Girder	Bridge 4			Bridge 5			Bridge 6		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	1,706	1,107	0.65	646	491	0.76	1,586	1,395	0.88
G2	968	1,289	1.33	366	485	1.33	--	--	--
G3	977	1,470	1.50	368	517	1.40	--	--	--
G4	1,754	1,669	0.95	664	590	0.89	1,600	1,703	1.06
Total	5,405	5,535	1.02	2,044	2,083	1.02	3,186	3,098	0.97

TABLE I-12a Live-load moments (k-ft), 160-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	3,362	2,850	0.85	3,308	2,716	0.82	3,235	2,404	0.74
G2	3,362	2,356	0.70	3,343	2,282	0.68	3,319	2,132	0.64
G3	3,362	2,356	0.70	3,385	2,439	0.72	3,409	2,667	0.78
G4	3,362	2,850	0.85	3,411	2,989	0.88	3,496	3,411	0.98
Total	13,448	10,412	0.77	13,447	10,426	0.78	13,459	10,614	0.98

TABLE I-12b Live-load moments (k-ft)

Girder	Bridge 4			Bridge 5			Bridge 6		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	6,248	4,949	0.79	2,792	2,380	0.85	2,209	1,777	0.80
G2	6,295	4,335	0.69	2,813	2,033	0.72	--	--	--
G3	6,346	4,872	0.77	2,834	2,134	0.75	--	--	--
G4	6,397	6,127	0.96	2,856	2,608	0.91	2,223	2,001	0.90
Total	25,286	20,283	0.80	11,295	9,155	0.81	4,432	3,778	0.85

TABLE I-13 Steel moments (k-ft), 210-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	599	616	1.03	590	571	0.97	571	453	0.79
G2	649	616	0.95	644	601	0.93	639	554	0.87
G3	649	616	0.95	654	630	0.96	659	649	0.98
G4	599	616	1.03	608	658	1.08	627	741	1.18
Total	2,496	2,464	0.99	2,496	2,460	0.99	2,496	2,397	0.96

TABLE I-14 Deck moments (k-ft), 210-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	2,229	2,345	1.05	2,220	2,181	0.98	2,129	1,753	0.82
G2	2,485	2,345	0.94	2,480	2,289	0.92	2,446	2,118	0.87
G3	2,485	2,345	0.94	2,492	2,391	0.96	2,525	2,460	0.97
G4	2,229	2,345	1.05	2,238	2,496	1.12	2,335	2,795	1.20
Total	9,428	9,380	0.99	9,430	9,357	0.99	9,435	9,126	0.97

TABLE I-15 Parapet moments (k-ft), 210-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	567	436	0.77	558	400	0.72	541	306	0.57
G2	320	434	1.36	318	423	1.33	315	387	1.23
G3	320	434	1.36	323	445	1.38	325	457	1.41
G4	567	436	0.77	576	469	0.81	594	530	0.89
Total	1,774	1,740	0.98	1,775	1,737	0.98	1,775	1,680	0.95

TABLE I-16 Live-load moments (k-ft), 210-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	3,606	3,080	0.85	3,591	2,990	0.83	3,465	2,675	0.77
G2	3,606	2,526	0.70	3,600	2,442	0.68	3,560	2,204	0.62
G3	3,606	2,526	0.70	3,617	2,603	0.72	3,657	2,745	0.75
G4	3,606	3,080	0.85	3,622	3,207	0.89	3,751	3,509	0.94
Total	14,424	11,212	0.78	14,430	11,242	0.78	14,433	11,133	0.77

TABLE I-17a Steel deflections (in.), 160-ft span

Girder	Bridge 1 Straight			Bridge 2 R = 2,400 ft			Bridge 3 R = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	0.50	0.53	1.06	0.49	0.49	1.00	0.46	0.42	0.91
G2	0.55	0.53	0.96	0.54	0.52	0.96	0.53	0.50	0.94
G3	0.55	0.53	0.96	0.56	0.54	0.96	0.57	0.58	1.02
G4	0.50	0.53	1.06	0.52	0.56	1.08	0.55	0.66	1.20
Total	2.10	2.12	1.01	2.11	2.11	1.00	2.11	2.16	1.02

TABLE I-17b Steel deflections (in.)

Girder	Bridge 4			Bridge 5			Bridge 6		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	2.84	2.42	0.85	0.77	0.72	0.94	0.79	0.70	0.89
G2	3.04	2.84	0.93	0.85	0.78	0.92	--	--	--
G3	3.10	3.26	1.05	0.87	0.83	0.95	--	--	--
G4	3.00	3.68	1.23	0.81	0.88	1.09	0.80	0.88	1.10
Total	11.98	12.20	1.02	3.30	3.21	0.97	1.59	1.58	0.99

TABLE I-18a Deck deflections (in.), 160-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	2.07	2.24	1.08	2.00	2.09	1.04	1.89	1.76	0.93
G2	2.31	2.25	0.97	2.29	2.20	0.96	2.24	2.14	0.96
G3	2.31	2.25	0.97	2.33	2.31	0.99	2.38	2.51	1.05
G4	2.07	2.24	1.08	2.15	2.41	1.12	2.27	2.88	1.27
Total	8.76	8.98	1.03	8.77	9.01	1.03	8.78	9.29	1.06

TABLE I-18b Deck deflections (in.)

Girder	Bridge 4			Bridge 5			Bridge 6		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	6.26	5.65	0.90	3.06	3.03	0.99	3.11	2.85	0.92
G2	7.09	6.62	0.93	3.47	3.26	0.94	--	--	--
G3	7.22	7.59	1.05	3.51	3.49	0.99	--	--	--
G4	6.61	8.57	1.30	3.23	3.70	1.15	3.17	3.59	1.13
Total	27.18	28.43	1.05	13.27	13.48	1.02	6.28	6.44	1.03

TABLE I-19a Parapet deflections (in.), 160-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	0.28	0.23	0.82	0.27	0.21	0.78	0.25	0.17	0.68
G2	0.15	0.21	1.40	0.14	0.21	1.50	0.14	0.20	1.42
G3	0.15	0.21	1.40	0.16	0.22	1.38	0.15	0.24	1.60
G4	0.28	0.23	0.82	0.30	0.23	0.80	0.30	0.30	1.00
Total	0.86	0.88	1.02	0.87	0.87	1.00	0.84	0.91	1.08

TABLE I-19b Parapet deflections (in.)

Girder	Bridge 4			Bridge 5			Bridge 6		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	0.99	0.67	0.68	0.40	0.30	0.75	0.67	0.60	0.90
G2	0.54	0.75	1.39	0.22	0.30	1.36	--	--	--
G3	0.55	0.84	1.53	0.22	0.32	1.45	--	--	--
G4	1.04	0.96	0.92	0.43	0.37	0.86	0.69	0.72	1.04
Total	3.12	3.22	1.03	1.27	1.29	1.02	1.36	1.32	0.97

TABLE I-20a Live-load deflections (in.), 160-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	1.33	1.54	1.16	1.29	1.39	1.08	1.22	1.09	0.89
G2	1.27	1.23	0.97	1.26	1.20	0.95	1.24	1.19	0.96
G3	1.27	1.23	0.97	1.28	1.33	1.04	1.31	1.63	1.24
G4	1.33	1.54	1.16	1.36	1.70	1.25	1.45	2.19	1.51
Total	5.20	5.54	1.07	5.19	5.62	1.08	5.22	6.10	1.17

TABLE I-20b Live-load deflections (in.)

Girder	Bridge 4			Bridge 5			Bridge 6		
	LGS	3D	Rat	LGS	3D	Rat	LGS	3D	Rat
G1	1.86	1.83	0.98	0.90	0.99	1.10	0.60	0.73	1.22
G2	1.80	1.72	0.96	0.87	0.85	0.98	--	--	--
G3	1.83	1.99	1.09	0.88	0.91	1.03	--	--	--
G4	1.96	2.52	1.29	0.93	1.15	1.24	0.61	0.88	1.44
Total	7.45	8.06	1.08	3.58	3.90	1.09	1.21	1.61	1.33

TABLE I-21a Components 2 and 3, end reactions (kips), G4

Load	Bridge 1 Straight Comp 2	Bridge 2 3 Deg Comp 3	Bridge 3 10.5 Deg Comp 3	Bridge 4 5.0 Deg Comp 3	Bridge 5 3.1 Deg Comp 3	Bridge 6 3.1 Deg Comp 3
Steel	0.11	0.06	0.10	0.08	-0.01	0.15
Deck	0.05	0.04	0.15	0.17	0.07	0.11
Parapet	0.00	0.00	0.11	0.00	0.10	0.09
Live	-0.10	0.03	0.10	0.06	0.01	-0.01

TABLE I-21b Components 2 and 3, interior reactions (kips), G4

Load	Bridge 1 Straight Comp 2	Bridge 2 3 Deg Comp 3	Bridge 3 10.5 Deg Comp 3
Steel	0.03	0.00	-0.05
Deck	0.05	-0.02	-0.07
Parapet	0.00	0.02	-0.01
Live	-0.11	-0.01	-0.05

TABLE I-22a Components 2 and 3, pier moments (k-ft), G4

Load	Bridge 1 Straight Comp 2	Bridge 2 3 Deg Comp 3	Bridge 3 10.5 Deg Comp 3
Steel	0.04	0.03	0.08
Deck	0.07	0.03	0.08
Parapet	-0.07	0.01	0.09
Live	-0.07	0.02	0.06

TABLE I-22b Components 2 and 3, 160-ft span moments (k-ft), G4

Load	Bridge 1 Straight Comp 2	Bridge 2 3 Deg Comp 3	Bridge 3 10.5 Deg Comp 3	Bridge 4 5.0 Deg Comp 3	Bridge 5 3.1 Deg Comp 3	Bridge 6 3.1 Deg Comp 3
Steel	0.02	0.02	0.16	0.19	0.07	0.00
Deck	0.06	0.05	0.17	0.23	0.10	0.14
Parapet	-0.19	0.02	0.14	0.14	0.08	0.06
Live	-0.15	0.03	0.13	0.11	0.06	-0.10

TABLE I-22c Components 2 and 3, 210-ft span moments (k-ft), G4

Load	Bridge 1 Straight Comp 2	Bridge 2 3 Deg Comp 3	Bridge 3 10.5 Deg Comp 3
Steel	0.03	0.05	0.15
Deck	0.05	0.07	0.15
Parapet	-0.23	0.04	0.12
Live	-0.15	0.04	0.09

TABLE I-23 Components 2 and 3, 160-ft span deflections (in.), G4

Load	Bridge 1 Straight Comp 2	Bridge 2 3 Deg Comp 3	Bridge 3 10.5 Deg Comp 3	Bridge 4 5.0 Deg Comp 3	Bridge 5 3.1 Deg Comp 3	Bridge 6 3.1 Deg Comp 3
Steel	0.06	0.02	0.14	0.17	0.03	0.10
Deck	0.08	0.04	0.19	0.22	0.07	0.13
Parapet	-0.18	-0.02	0.18	0.10	0.04	0.04
Live	0.16	0.09	0.35	0.13	0.08	0.44

TABLE I-24a Steel lateral flange moments (k-ft) G4, 210-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	Vload	3D	Rat	Vload	3D	Rat	Vload	3D	Rat
G1	0	0	--	0.8	0	--	2.0	2.2	1.10
G2	0	0	--	0.8	0	--	2.7	3.1	1.15
G3	0	0	--	0.8	0	--	3.2	3.7	1.16
G4	0	0	--	0.8	0	--	3.7	4.1	1.11

TABLE I-24b Deck lateral flange moments (k-ft) G4, 210-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	Vload	3D	Rat	Vload	3D	Rat	Vload	3D	Rat
G1	0	0	--	3.0	2.3	0.77	8.0	11.0	1.38
G2	0	0	--	3.0	2.2	0.73	10.1	11.2	1.11
G3	0	0	--	3.0	2.0	0.73	12.0	12.2	1.02
G4	0	0	--	3.4	2.0	0.59	13.8	16.4	1.19

TABLE I-24c Parapet lateral flange moments (k-ft) G4, 210-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	Vload	3D	Rat	Vload	3D	Rat	Vload	3D	Rat
G1	0	0	--	1.1	0.0	0.00	3.2	4.1	1.28
G2	0	0	--	1.2	1.0	0.83	4.0	4.1	1.03
G3	0	0	--	1.4	1.2	0.86	4.8	4.2	0.88
G4	0	0	--	1.5	1.1	0.73	5.6	6.1	1.09

TABLE I-24d Live-load lateral flange moments (k-ft) G4, 210-ft span

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	Vload	3D	Rat	Vload	3D	Rat	Vload	3D	Rat
G1	0	2.3	--	4.2	4.6	1.10	12.6	16.0	1.27
G2	0	2.3	--	4.3	3.5	0.81	10.6	11.5	1.08
G3	0	2.3	--	4.5	3.3	0.73	13.4	11.5	0.86
G4	0	2.3	--	4.6	2.3	0.50	17.4	19.5	1.12

TABLE I-25a Steel lateral flange moments (k-ft) G4, pier

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	Vload	3D	Rat	Vload	3D	Rat	Vload	3D	Rat
G1	0	0	--	-2.2	-1.0	0.45	-6.7	-6.0	0.90
G2	0	0	--	-2.2	-1.0	0.45	-7.5	-6.0	0.80
G3	0	0	--	-2.3	-1.0	0.43	-8.4	-6.0	0.71
G4	0	0	--	-2.4	0.0	--	-8.9	-7.1	0.80

TABLE I-25b Deck lateral flange moments (k-ft) G4, pier

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	Vload	3D	Rat	Vload	3D	Rat	Vload	3D	Rat
G1	0	-2	--	-8.4	-5.0	0.60	-25.9	-27.1	1.05
G2	0	0	--	-8.7	-4.0	0.46	-29.2	-26.2	0.90
G3	0	0	--	-9.0	-3.0	0.33	-32.6	-27.0	0.83
G4	0	0	--	-9.1	-2.0	0.22	-34.7	-30.1	0.87

TABLE I-25c Parapet lateral flange moments (k-ft) G4, pier

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	Vload	3D	Rat	Vload	3D	Rat	Vload	3D	Rat
G1	0	-1	--	-3.4	-2.0	0.59	-10.5	-9	0.86
G2	0	0	--	-3.2	-1.1	0.34	-10.6	-8	0.75
G3	0	0	--	-3.3	0.0	0.00	-12.0	-8	0.67
G4	0	1	--	-3.8	0.0	0.00	-14.4	-6	0.42

TABLE I-25d Live-load lateral flange moments (k-ft) G4, pier

Girder	Bridge 1 Straight			Bridge 2 Radius = 2,400 ft			Bridge 3 Radius = 700 ft		
	Vload	3D	Rat	Vload	3D	Rat	Vload	3D	Rat
G1	0	-8.1	--	-6.9	-10.4	1.51	-21.0	-22	1.05
G2	0	-8.1	--	-5.7	-9.3	1.63	-19.1	-16	0.84
G3	0	-8.1	--	-6.0	-8.1	1.35	-22.3	-14	0.63
G4	0	-8.1	--	-7.5	-7.0	0.93	-27.9	-15	0.54

ANNEX II

COMPUTATION OF WHEEL LOAD DISTRIBUTION FACTORS FOR MOMENT-BRIDGE 1

A. LRFD (*14*) - G3 (Interior Girder)

A.1 Multiple Lanes, $S = 11$ ft

$$g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}(1) \text{ lanes}$$

160-ft span

$$0.075 + \left(\frac{11}{9.5}\right)^{0.6}\left(\frac{11}{160}\right)^{0.2}(1)$$

$$0.075 + 1.09(0.585)(1) = 0.713 \text{ lanes}$$

$$0.713 \times 2 = 1.42 \text{ wheels}$$

210-ft span

$$0.075 + 1.09\left(\frac{11}{210}\right)^{0.2}(1) = 0.679 \text{ lanes}$$

$$0.679 \times 2 = 1.36 \text{ wheels}$$

Pier, $L = (160 + 210)/2 = 185$ ft

$$0.075 + 1.09\left(\frac{11}{185}\right)^{0.2}(1) = 0.695 \text{ lanes}$$

$$0.695 \times 2 = 1.39 \text{ wheels}$$

A.2 Single Lane, $S = 11$ ft

$$g = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}(1) \text{ lanes}$$

160-ft span

$$0.06 + \left(\frac{11}{14}\right)^{0.4}\left(\frac{11}{160}\right)^{0.3}(1)$$

$$0.06 + 0.908(0.448)(1) = 0.467 \text{ lanes}$$

$$0.467 \times 2 = 0.93 \text{ wheels}$$

210-ft span

$$0.06 + (0.908)\left(\frac{11}{210}\right)^{03}(1) = 0.435 \text{ lanes}$$

$$0.435 \times 2 = 0.87 \text{ wheels}$$

Pier, $L = \dfrac{(160 + 210)}{2} = 185$ ft

$$0.06 + (0.908)\left(\frac{11}{185}\right)^{0.3}(1) = 0.449 \text{ lanes}$$

$$0.449 \times 2 = 0.90 \text{ wheels}$$

B. LRFD - G4 (Exterior Girder)

B.1 Multiple Lanes

$$g_{ext} = e g_{interior}$$

$$e = 0.77 + d_e/9.1 \geq 1.0$$

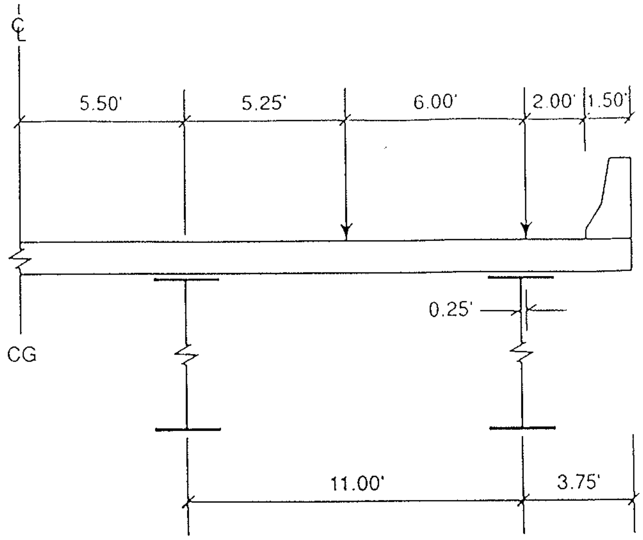

Figure II-1. Schematic of simple-span method.

where

d_e = Distance from center of exterior beam and edge of roadway (See Figure II-1)

$d_e = 3.75 - 1.5 = 2.25$ ft

$e = 0.77 + 2.25/9.1 = 1.02$

160-ft span

From Table A-8, $g_{interior} = 1.42$

$g_{ext} = 1.02 \times 1.42 = 1.45$ wheels

210-ft span

From Table A-8, $g_{interior} = 1.36$

$g_{ext} = 1.02 \times 1.36 = 1.39$ wheels

Pier

$g_{interior} = 1.39$

$g_{ext} = 1.02 \times 1.39 = 1.42$ wheels

B.2 Single Lane

$$R = \frac{N_L}{N_b} + \frac{X_{ext}\sum^{N_L} e}{\sum^{N_b} X^2}$$

where

N_L = Number of lanes considered = 1

N_b = Number of beams = 4

X_{ext} = Distance from the center of gravity of beams to the exterior beam

$X_{ext} = 11 + 5.5 = 16.5$ ft

X = Distance from center of gravity of beams to each beam

e = Distance from center of gravity of beams to center of gravity of lanes

Axle = 6 ft; 2 ft from curb to first wheel; curb is 1.5-ft wide

$6/2 + 2 + 1.5 = 6.5$ ft from edge

$e = 40.5/2 - 6.5 = 13.75$ ft

$X = 16.5$ ft; 5.5 ft; –5.5 ft; –16.5 ft

$$R = \frac{1}{4} + \frac{(16.5)(13.75)}{(2)(16.5)^2 + (2)(5.5)^2} = \frac{1}{4} + \frac{226.875}{605}$$

$R = 0.625$ lanes

$R = 0.625 \times 2 = 1.25$ wheels

C. Guide Spec (*15*) - G3 (Interior Girder)

C.1 Multiple Lanes: Same as LRFD except in wheels instead of lanes.

$$g = 0.15 + \left(\frac{S}{3}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}(1)$$

Continuity factor for positive moment = 1.05

Continuity factor for negative moment = 1.10

160-ft span, $S = 11$ ft

$$0.15 + \left(\frac{11}{3}\right)^{0.6}\left(\frac{11}{160}\right)^{0.2}(1) = 1.43 \text{ wheels}$$

Modifying for continuity,

$g = 1.43 \times 1.05 = 1.50$

210-ft span, $S = 11$ ft

$$g = 0.15 + \left(\frac{11}{3}\right)^{0.6}\left(\frac{11}{210}\right)^{0.2}(1) = 1.36$$

Modifying for continuity,

$g = 1.36 \times 1.05 = 1.43$

Pier, $L = (160 + 210)/2 = 185$ ft, $S = 11$ ft

$$g = 0.15 + \left(\frac{11}{3}\right)^{0.6}\left(\frac{11}{185}\right)^{0.2}(1) = 1.39$$

Modifying for continuity,

$g = 1.39 \times 1.10 = 1.53$

C.2 Single Lane: Same as LRFD except in wheels instead of lanes.

Continuity factor for positive moment = 1.05

Continuity factor for negative moment = 1.10

D: Guide Spec (*15*) - G4 (Exterior Girder)

D.1 Multiple Lanes

For exterior girder, use simple-span distribution as shown in Figure II-1, since $S = 11$ ft > 10.5 ft.

Continuity factor for positive moment = 1.05

Continuity factor for negative moment = 1.10

$R = [(5.25)(1) + (11.25)(1)]/11 = 1.50$ wheels

D.2 Single Lane: Same as LRFD except continuity factor.

Continuity factor for positive moment = 1.05

Continuity factor for negative moment = 1.10

For positive moments, 160-ft and 210-ft spans,

$1.50 \times 1.05 = 1.58$

$1.25 \times 1.05 = 1.31$

For negative moment at pier,

$1.50 \times 1.10 = 1.65$

$1.25 \times 1.10 = 1.38$

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APPENDIX B

CURVED-GIRDER DESIGN AND CONSTRUCTION, CURRENT PRACTICE

PREFACE

AASHTO first published *Guide Specifications for Horizontally Curved Highway Bridges* in 1980. These guide specifications included allowable stress design (ASD) provisions developed by the Consortium of University Research Teams (CURT) and approved by ballot of the AASHTO Highway Subcommittee on Bridges and Structures in November 1976. CURT consisted of Carnegie Mellon University, the University of Pennsylvania, the University of Rhode Island, and Syracuse University. The 1980 guide specifications also included Load Factor Design (LFD) provisions developed in American Iron and Steel Institute (AISI) Project 190 and approved by ballot of the AASHTO Highway Subcommittee on Bridges and Structures in October 1979. The guide specifications covered both box and I-girders.

Changes to the 1980 guide specifications were included in the AASHTO *Interim Specifications - Bridges* for the years 1981, 1982, 1984, 1985, 1986, and 1990. A new version of the *Guide Specifications for Horizontally Curved Highway Bridges* was published in 1993. It included these interim changes and additional changes, but did not reflect the extensive research on curved-girder bridges that has been conducted since 1980 or many important changes in related provisions of the straight-girder specifications.

The current practice in the design and construction of horizontally curved girder highway bridges is written to reflect the current state of the art both in the United States and Japan. These are the only two countries in the world today that have issued an official or semi-official guide specification for the design and construction of such bridges. Appendix B was compiled as a part of NCHRP Project 12-38.

- AWS refers to the 1991 edition of ANSI/AASHTO/AWS D1.5-88 *Bridge Welding Code,* American Welding Society;
- "Previous curved-girder specifications" or Guide Spec refers to the 1993 AASHTO *Guide Specifications for Horizontally Curved Highway Bridges;*
- LFD/ASD refers to the 1992 AASHTO *Standard Specifications for Highway Bridges;* and
- LRFD refers to the 1994 AASHTO *LRFD Bridge Design Specifications.*

I. INTRODUCTION

A. Need

Curved bridges were once rare. Streams were usually crossed perpendicular to their flow, and adjustments in road alignment often were made by simply turning onto the bridge. As highway speeds increased, alignments with sharp turns became unacceptable, and grade separation structures at interchanges became commonplace. Both of these events increased the need for horizontally curved bridges.

Prior to the use of curved girders, curved alignments were usually accomplished with simple spans having girders "kinked" at the supports. These designs resulted in skewed supports, decks with varying overhang widths, and shorter than optimum span lengths. A typical example of this application is the riveted girder spans approaching the Fort Pitt Bridge in Pittsburgh. Grade separation bridges with sharper curvatures required ever shorter spans, which led to poor visibility around numerous pier columns, poor bridge aesthetics, an excessive number of joints, and increased bridge cost. Clearly, longer continuous spans would reduce the cost of curved alignments, eliminate many troublesome bridge joints, and improve safety and aesthetics.

B. Description

Curved beams need lateral restraint for rigid body stability, whereas straight beams need only lateral support against buckling. The curvature introduces torsion into a curved girder, and the lateral restraint introduces nonuniform torsion. In typical bridges, lateral restraint is provided by intermediate cross frames and the deck. An I-beam is called an "open section." These members have little resistance to torsion and must be laterally supported at relatively frequent intervals. The nonuniform torsion in these beams is manifest in the lateral bending moments in the flanges.

A box girder is called a "closed section." These members are capable of resisting torsion with minimal warping if the cross section is adequately restrained against deformation by bracing members within the box. The bending components of a box beam are webs, flanges and, perhaps, longitudinal flange stiffeners. Box beams may be built with open tops that are called "trough" or "tub" girders, or they may be built with four plates. When curved trough girders are used, the top flange must be braced with diagonal members to provide torsional rigidity. Box flanges are often stiffened in the compression regions with longitudinal stiffeners. Webs may be inclined to minimize the bottom flange width.

C. History

The earliest horizontally curved girder bridges in the United States were designed by using box sections composed of four plates. One of the first curved-girder bridges was built in Springfield, Massachusetts, in 1967. It had two simple spans approximately 88 and 130 ft, with a radius of 155 ft, and two boxes in the cross section. It was designed by Goodkind, Goodkind and O'Dea (presently, Dewberry & Davis). This bridge, which was inspected recently, was found to be in excellent condition after almost 30 years. By today's standards, however, the steel weight (approximately 90 lb/ft^2) was excessive.

One of the earliest applications of curved I-girders was in Pittsburgh. It is a three-span, two-girder structure with a radius of curvature less than 500 ft, carrying California Avenue over Ohio Boulevard. An early formulation of the V-load method was used in its analysis (*1*). This bridge, which was designed in 1967 by Richardson and Gordon (presently HDR), is still carrying traffic, with little remedial work required.

D. Alignment

Curvature may be measured in degrees, radians, or *L/R,* where *L* is span length and R is radius. Roadway curvature can be accomplished by curving of the girders or by kinks in an otherwise straight girder. If the girder is kinked at a location other than at simple supports, sharp discontinuities in flange force may cause extremely large cross frame forces and lateral flange bending in the vicinity of the kink. Curvature often varies along the bridge. In many states, curvature changes are accomplished with compound curves, each with a constant radius. Other states accomplished the same changes with a spiral. Most bridge fabricators are now able to fabricate girders having spiral curves with few problems.

The economic benefits of horizontally curved girders become apparent when multiple-span curved alignments are required. Continuous curved steel structures over 1,900 ft long have been built with expansion joints only at the ends, such as the curved box-girder viaduct carrying Eighth Avenue in Denver (built in 1983). Wherever possible, continuous spans are used in lieu of simple spans to minimize the number of deck joints and to improve overall economy.

One early (1969) example of a short curved bridge is the Hull's Falls Bridge in New York State. This bridge was designed by New York DOT to carry traffic over the Ausable River (*2*). Three I-girders were used with 123-ft span and a radius of 477 ft. A straight bridge could have been used, but the roadway alignment would have been compromised.

An I-girder bridge carrying Route 65 over Pigeon Creek in Dalbarton, West Virginia, has spans of 65–72 ft and a radius of 127 ft. A simple-span, I-girder bridge in Minnesota has a span of 104 ft and radius of 106 ft, with one abutment skewed approximately 45 deg. Temporary supports were required to erect the steel for this relatively short span. It frequently becomes difficult to erect single-span curved bridges having skewed support lines. Although not record structures, the West Virginia and Minnesota highway bridges are examples of extreme curvature.

Curved-girder bridges have been built with spans greater than 300 ft and radii less than 1,000 ft. One unit of the Little Falls Bridge, New York State, has spans of 234-329-365-234 ft with a radius of 750 ft. This bridge has seven I-girders in the cross section, varying in depth from 7 to 12 ft. One of the Shoshone Dam Bridges in Glenwood Canyon, Colorado, has spans of 219-274-216 ft and a radius of 955 ft. This bridge has four I-girders, varying in depth from 7 to 15 ft.

Integral pier caps have been used with curved girders. The cap beams have been made of steel box- and I-shapes and reinforced or post-tensioned concrete. There are many instances where a pier cap below the girders will not provide adequate clearance, so an integral pier cap is used to gain the needed clearance. Integral pier caps also have been used for aesthetic reasons or to eliminate the special design problems and cost of skewed piers.

Skewed support lines present additional consideration in design and construction of curved girders due to the increased torsion in the girders. These load effects may lead to excessive girder rotations, larger than normal cross frame forces, unusual thermal movements, and large differential deflections between girders during construction.

Due to alignment constraints, span ratios can vary greatly from efficient values. Poor span ratios can lead to uplift problems during construction and, in some cases, due to live load, requiring an anchorage system. Poor span ratios can significantly increase the cost of steel framing. Desirable span ratios for continuous steel structures provide spans with approximately equal positive dead-load moments in each span. This usually can be accomplished with end spans that are approximately 75 percent of the adjacent interior span length. This ratio is different from the optimal span arrangement for concrete bridges. If cantilever segmental concrete construction is used, the optimal span arrangement employs end-span lengths of approximately half the interior span length. Equal spans usually are optimal when simple-span prestressed concrete girders are used.

Another increasingly common alignment is the bifurcated structure. This situation occurs when a ramp exits or enters a street on a single bridge structure. It can be accomplished with girders simply peeling off in two directions. However, roadway width often dictates that the total number of girders in the two branches be greater than the number required in the stem, necessitating some arrangement for discontinuing the girders. This is often done by placing a bulkhead near a point of contraflexure of two continuing girders. Another, less desirable, arrangement is to attach girders from one branch of the wye (Y) to one stem girder, usually resulting in large eccentric loading on the stem girder and very complex connections.

E. Researchers

The earliest theoretical work on open and closed curved beams was performed by St. Venant over 150 years ago (*3*). In the early 1960s, Dabrowski (*4–6*) and Vlasov (*7*) presented rather complete strength of materials solutions to the curved beam problem for stresses and deflections.

PennDOT funded some of the earliest research on curved highway girders at Carnegie Mellon University, but the first extensive study of analysis and design techniques for curved girders was the CURT project started in the 1960s and completed in the early 1970s. This work was funded by the Federal Highway Administration. The team included researchers from the following universities: Syracuse, Rhode Island, Pennsylvania, and Carnegie Mellon.

The CURT project encompassed analytical study and laboratory tests of scaled steel box and I-girders. A draft specification (*8*) was developed by CURT which evolved into the *AASHTO Guide Specifications for Horizontally Curved Highway Bridges* (Guide Spec) (*9*).

Experimental work also was performed at Lehigh University, including the only fatigue tests of curved girders in the United States. The specimens were first tested statically in the elastic range. One test was performed on a full-scale, two I-girder simple-span unit without deck (*10–12*). The span was then subjected to cyclic loading until fatigue cracks were observed. The results indicated no significant differences between the fatigue behavior of straight and curved girders. A concrete deck was added and the unit was tested to failure (*13*). The specimen failed due to crushing of the concrete deck rather than plastification of the steel sections. A similar test of a near full-scale single box girder was also performed as part of the same project (*14*). The Lehigh University work (*15*), completed in the 1970s, formed the basis of the Guide Spec ASD provisions.

Later, the American Iron and Steel Institute (AISI) funded research resulting in load factor design (LFD) provisions (*16,17*) for the Guide Spec (*9*). This was simply an extension of work performed in the CURT project at Carnegie Mellon University.

At the University of Maryland, Heins et al. (*18–20*) also contributed to the knowledge of curved box and I-girders. Many of these contributions regarding approximations rather than refined analyses are included as part of the Guide Spec and its commentary.

Yoo et al. (*21–27*) studied the basic buckling and dynamic behavior of curved I-girders, including development of a finite element that has a warping degree of freedom.

Significant work on curved girders also has been performed in Japan. Fukumoto developed an equation to predict the inelastic strength of I-girders (*28*) and conducted confirming tests on single I-beams (*29*). Nakai et al. also performed numerous component tests of curved beams (*30–35*). From this work, an empirical equation to predict the strength of curved I-girders was developed (*36*). Nakai and Kitada and others have performed inelastic finite element studies, but this work has not been translated in English (*37*).

The Japanese research culminated in the *Hanshin Guidelines for the Design of Horizontally Curved Girder Bridges* (draft) by the Hanshin Expressway Public Corporation (*38*). These design provisions, for box and I-girders, are ASD-based. Like their U.S. counterpart, many references are made to the Japanese Bridge Specifications for straight girders (*39*). The theoretical background and experimental research leading to the Hanshin provisions are presented in a text by Nakai and Yoo (*40*). The AASHTO Guide Spec and Hanshin Guidelines represent the only known design provisions for horizontally curved steel girders.

Although never receiving the status of full AASHTO specifications (*41*), the Guide Spec has been used to design hundreds of curved-girder bridges in the United States and Canada. Recent contributions have been made by Yang and Kuo (*42–44*) and Tan et al. (*45*). Zureick et al. have provided the most recent literature survey (*46*).

II. RESEARCH

A. I-Beams

A.1 Bending - Analytical

A.1.a Static Equilibrium. Horizontally curved I-beams must be braced laterally to be statically stable because applied vertical loads produce twist as well as bending. Internal vertical bending and lateral flange bending moments combine to provide equilibrium. Lateral flange moments are computed from the bimoment, *BM,* from Equation (B-1):

$$BM = EI_{\omega}\phi'' \tag{B-1}$$

where I_{ω} = warping constant and ϕ = angle of twist.

Warping stress, σ_{ω} can be computed from Equation (B-2) below:

$$\sigma_{\omega}\omega = \frac{BM\omega_n}{I_{\omega}} \tag{B-2}$$

where ω_n = normalized unit warping of cross section.

To mobilize the bimoment, the member twists. The distribution and magnitude of the resulting internal stresses are functions of the spacing and stiffness of the bracing, as well as of cross-sectional properties and curvature of the girder. Vertical bending in an unbraced curved I-beam is slightly different than for a straight beam. Dabrowski (*6*) gives the equation, based on strength of materials, for the statical girder moment due to a concentrated vertical load, P, placed in the center of a simple-span beam with a span, L, measured along the arc and radius, R. The beam is torsionally pinned at its ends with no intermediate restraints.

$$M = \frac{PR}{2}\left(1 - \frac{e}{R}\right)\frac{\sin\frac{z}{R}}{\cos\frac{L}{2R}} \qquad \text{(B-3)}$$

where e = eccentricity of applied load measured with respect to the vertical axis through the shear center and z = arc distance from support.

The difference between moment in a curved beam and a straight beam is insignificant for the relatively short, unbraced lengths typically found in erected curved bridges. For long unbraced lengths, however, such as those found in girders during erection and handling, consideration of curvature may be necessary to accurately compute the vertical bending moment, as well as the effects of nonuniform torsion.

A.1.b Strength of Materials Behavior. Classical strength of materials assumptions requires that the cross section does not deform, that Hooke's law applies, that the member geometry does not change, and that small deflection theory applies. Elastic buckling is assumed to occur at a load determined as bifurcation. Dabrowski has solved the single curved-girder problem by using these assumptions (*6*).

Unlike a straight beam, which does not deflect laterally until failure is imminent (bifurcation), lateral deflection and twist commence with initial application of vertical load to a curved beam. Therefore, lateral deflection of the girder compression flange increases with a concomitant increase in lateral flange bending until, due to increasing load, failure as a result of yielding occurs.

A.1.c Inelastic Behavior. Amplification of lateral deflection of the beam away from the center of curvature causes additional lateral bending. Although lateral moments are determined by the original curvature according to strength of materials, the actual lateral moments are greater because of the increase in curvature due to lateral deflection. This behavior is similar to the amplification effect evident in a beam-column. McManus found that the lateral deflection is approximately linear up to about 30 percent of the elastic buckling load for the equivalent straight beam; above 30 percent of the buckling load, the lateral deflection increases nonlinearly to failure (*47*).

Further, the cross section of a curved beam deforms when subjected to bending and torsion. Basler and Thurlimann recognized that the compression portion of the web of a straight beam in pure bending buckles, shedding some of its load to the compression flange (*48*). This shedding causes an increase in the compression flange force compared to that predicted from Navier's hypothesis. Mozer and Culver observed this behavior in curved beam tests. The compression bending stress was 10 percent greater than the tension stress in a doubly symmetric beam having a web slenderness of 156 (*49*). It was suggested that this action may be more critical in curved beams because the web is curved and more susceptible to bend-buckling (*49*).

An additional type of cross-section deformation occurs in curved I-girders. The flanges of a curved I-beam rake with respect to the web. Raking occurs because the web is not stiff enough to retain the cross-section shape when the flanges are subjected to lateral moments. The web takes an S-shape; the girder flanges rotate and are no longer parallel with respect to each other; and the web is no longer perpendicular to the flanges.

A.1.d Failure of Curved Beams. Failure of a curved beam may occur by either local buckling or yielding in a compression flange. Either phenomenon can occur at a brace point or at midspan between brace points. The location depends on curvature, section properties, and boundary conditions. Local buckling of the flange or web will not occur if the element slenderness is adequate. Such sections are called compact; they fail by yielding. Plastification or yielding of curved compact sections occurs under a combination of lateral and vertical bending.

It is possible for a curved beam to fail laterally prior to plastification of the entire web. At this time, research of curved-girder behavior has not dealt with redistribution of loads due to plastification. Instead, plastification is dealt with at one cross section. When any cross section yields enough such that additional load cannot be applied, it is assumed that the member has failed (*47*). The contribution of lateral bending to plastification is complex due to the amplification of lateral deflections and cross-section deformation.

A.1.e Analysis. Analysis of curved bridge superstructures can be performed with different techniques that address various aspects of the problem. Originally, Dabrowski (*6*) and Vlasov (*7*) developed equations defining stresses based on strength of materials methods that permit computation of vertical bending and lateral flange bending stresses. Dabrowski presents the equations for a singly symmetric, single I-girder with rigid and flexible supports (*6*). Subsequent discussions by Dabrowski reflect how the method can be extended to an actual bridge where the girders act with the deck (*6*). This work has been extended by Yoo and Heins to allow solutions to be obtained using the finite difference technique (*50*). The method does not treat stability of curved beams because classical buckling techniques cannot be applied to a curved beam.

The two-dimensional finite element method, sometimes called the grillage method, can be used to obtain a proper strength of materials solution when the warping degree of freedom is considered in the beam element (*22*). Most two-dimensional finite element programs do not provide this capability. Thus, grillage solutions often recognize only St. Venant torsional stiffness to establish equilibrium. Additionally, by not considering warping, the distribution of load is in error by the amount of the warping stiffness. Various techniques have been proposed to correct for the absence of warping stress. For example, the V-load method can be used to generate lateral flange moments from girder moments determined with a grillage analysis (*1*).

The three-dimensional finite element method can be used to model the beam cross section with a series of elements (i.e., the web is modeled with a series of planar shell elements; flanges are modeled with either shell or beam elements; and bracing and bearings must also be represented appropriately). This type of three-dimensional finite element modeling considers warping. The radial component and deformation of the cross section are recognized in the first-order assumption, but the effect of amplification of lateral deflections is recognized only if geometric nonlinear behavior is considered. Deformation of the cross section is also considered with this method. The lateral flange moments cannot exceed those required for equilibrium; thus, the warping bimoment will yield lateral stresses that put the section in equilibrium. Lateral flange moments due to raking simply displace some of the warping bimoment lateral bending based on a strength of materials analysis.

Large deflection finite element codes can be used to recognize amplification and cross-sectional deformation effects. Lateral flange bending is modified to correspond to the change in curvature. Thus, the lateral flange bending moments may increase beyond those from a strength of materials-based analysis.

An inelastic finite element code is a further refinement of the finite element method that recognizes the plastification of the section and its effects, including changes in shape of the structure. Such analyses are quite expensive, and the results depend on assumed residual stress distributions and material properties beyond the elastic regime. Superposition does not apply to analyses that consider large deflections or inelastic behavior; therefore, design techniques, such as using influence surfaces do not apply. Although nonlinear finite element analyses are not frequently used in bridge design, they are important research tools necessary to develop an understanding of curved-girder behavior.

A.1.f Culver et al. McManus examines doubly symmetric, curved I-beams in pure bending (*47*). These beams have coincident shear center and center of gravity and are assumed to be prismatic with equal vertical moments and lateral moments applied at their ends. Singly symmetric and composite I-sections are discussed but not investigated. Warping due to nonuniform torsion, cross-section deformation, and radial effects are considered. Amplification of deformations due to warping and twist are also addressed. Cross sections are separated into compact or noncompact types. The failure load for a noncompact section is defined as that load at which a flange tip reaches first yield (*47*). Failure of a compact section is defined as that load at which plastification of the entire cross section occurs due to a combination of vertical and lateral bending (*47*).

Buckling of curved flanges is examined, considering the flange on each side of the web separately (*51,52*). The side of the flange on the inside is restrained against buckling by the web on the outside of its curvature. The other side of the flange is restrained on its inside by the web. The problem is further complicated by the stress gradient across the flange that can be oriented in either direction. Nasir found that if the flange slenderness, b/t, is not greater than, $3,200/\sqrt{F_y}$, local buckling does not occur at reasonable radii for steels with a yield stress up to 50 ksi (*51*). Steels with a yield stress greater than 50 ksi are not examined. In the case of doubly symmetric sections, the yield stress is reached at approximately the same time in tension and compression flanges, but on opposite flange tips. Therefore, no investigation of slenderness is required for tension flanges.

McManus defines a compact section as one with flanges having b/t not greater than $3,200/\sqrt{F_y}$ (*47*). The web need not be compact because it is assumed that load is not transferred to the compression flange when the web buckles. This assumption is acknowledged to be unconservative, but other conservative assumptions are thought to compensate (*47*). Sections with web slenderness only up to 150 are investigated.

The research assumes a strength of materials analysis of the member that provides warping and bending stresses at brace points (*47*). Any strength of materials analysis which considers warping due to nonuniform torsion is appropriate. It is assumed that the warping stress and bending moment at brace points are provided. There is no relationship assumed between bending stress and warping stress at the ends of the member; the change in warping stress over the member length is a function of curvature.

The tendency of the girder to arch, or change curvature, is called the radial effect. This effect is approximated by assuming a uniform radial load as computed in Equation (B-4) (*47*):

$$q_{x_{SM}} = EI_y\left(u_2^{iv} + \frac{u_2''}{R^2}\right) \tag{B-4}$$

where

u_2 = radial deflection in state 2,
I_y = moment of inertia of the beam about vertical axis, and
R = radius of the beam.

State 2 is defined as the condition that exists after translation and rotation of the section occurs. This load is applied radially in the plane of the beam at the shear center and center of gravity away from the center of curvature. This loading produces an equal lateral moment in both flanges whose sense is the same as the lateral flange bending stress in the compression flange due to curvature. The result is that the magnitude of lateral flange bending in the top and bottom flanges is no longer equal and opposite. These radial bending stresses are added to those due to warping.

Amplification of the radial deflection and twist is found to be important at loads greater than 30 percent of the critical buckling load. An approximate amplification term given in Equation (B-5) is developed to represent the change in curvature:

$$\text{Amplication Factor} = \frac{1-0.86M^{*}+0.4M^{*2}}{1-M^{*}} \tag{B-5}$$

where

$$M^{*} = \frac{\text{applied moment}}{\text{buckling moment}}$$

The bimoment and the portion of the radial moment due to twist at midspan in both elastic and inelastic regimes are modified by this factor.

The effect of raking of the flanges is approximated by applying a pair of uniform radial forces given in Equation (B-6) to the flanges in the plane of curvature. These forces equal the force in the plane divided by the radius of the girder (*47*):

$$q = \frac{f_b A_f}{R} \tag{B-6}$$

The force on the compression flange is applied away from the center of curvature. It is applied toward the center of curvature on the tension flange. This results in lateral flange bending stresses having the same pattern and magnitude as those due to warping resulting from curvature. The lateral flange bending stresses due to flange raking are added to the other lateral flange bending stresses. The term "flange raking" means that the flange on one side of the web deforms more (somewhat independently) than the flange on the other side, thereby inducing deformation of the web. In fact, it is suggested that the raking distortional stress is the V-load stress, or the warping stress. If this is the case, it appears to be accounted for twice by McManus.

Figure B-1 (from Reference 47) shows the percent of flange tip stress due to bending, warping, radial, and deformation for two cases at the fully plastic stress state. These plots indicate stress at midspan, which is the critical location. The girders have a fixed ratio of warping-to-bending stress at brace points (0.50). The ratio of radius to unbraced length is also constant, as is the ratio of web-to-flange area (0.50). The ratio of unbraced length to flange width is the independent variable. The stress ratio and curvature ratio are at the upper limits studied by McManus.

McManus uses a sign convention for warping stress different from that used by Dabrowski and the Guide Spec. In order to have a positive f_w/f_b ratio, according to McManus, the torque at the brace points is opposite to what would be expected if it were due to only curvature. To obtain a 0.50 stress ratio, a torque must be applied to the beam in the opposite sense of that due to curvature and have sufficient magnitude to overcome the end torque.

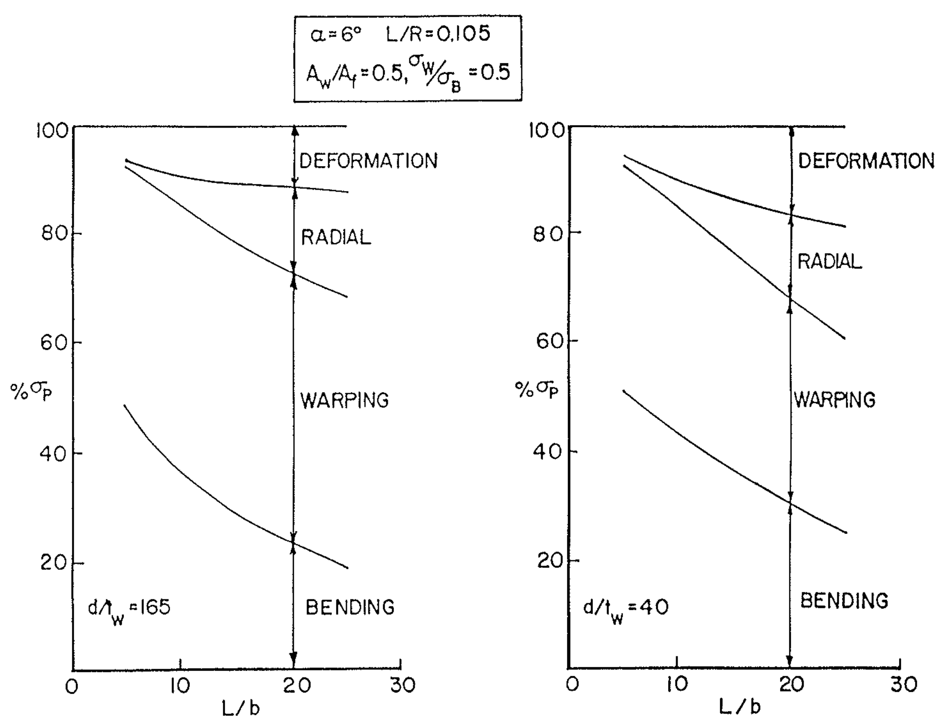


Figure B-1. Contribution of various effects to ultimate strength (47).

The nondimensional constants used present a special consideration. How these ratios were manipulated was not revealed. Because L/R is constant and L/b varies, it is possible to create this situation by either holding L and R constant and varying b, or by varying L and R while holding b constant. Of course, the variation could be a combination of the two. If b varies, then flange thickness must vary because the ratio of flange-to-web area is constant and the web depth-to-thickness ratio cannot vary. If b varies, then the bimoment at the brace points must vary since the warping stress-to-bending stress is constant.

In Figure B-1, at midspan, the lateral flange bending due to curvature is of the same sign as the warping stress at brace points. Thus, the magnitude of warping stress at midspan is the sum of these stresses. If curvature alone caused the torsion, the warping stress at midspan would be of the opposite sign as that at the brace points. The result would be that the warping stress at midspan is approximately equal and opposite to that at brace points.

It seems unusual that the distortional stress component is greater for the stocky web; the probable reason is that the ratio of web-to-flange area is kept constant, causing the flange size to change.

The radial and deformation stresses in the elastic range should be smaller than those shown in Figure B-1. The total lateral flange bending stresses appear to have been determined by adding the radial and deformation stresses to the warping stresses. As discussed earlier, it appears that the deformation stress is equal to the warping stress due to curvature.

Using approximate solutions to the differential equations, McManus developed families of strength curves for the first-yield and plastic conditions (*47*). These strengths, in terms of moments, were divided by the critical lateral torsional buckling moment for an equivalent straight girder. Equation (B-7) was used to determine the critical straight-girder moment. This equation was taken from the AASHO provisions in effect at that time:

$$F_{cr} = F_y\left(1 - 3\lambda^2\right) \qquad \text{(B-7)}$$

where

$$\lambda = \frac{(l/r)F_y}{4\pi^2 E}$$

Equation (B-7) has been replaced in the present AASHTO Standard Specifications (*41*) with an equation for lateral torsional buckling that considers the entire undeformed cross section. Equation (B-7), however, must be used since it was used in the derivation.

The resulting nondimensional ratios were defined as the product of two ρ factors. These factors were defined in terms of nondimensional parameters; warping stress/bending stress at brace points (f_w/f_b),l/R, and l/b_f.

The ρs are applicable only within defined limits of the parameters as stipulated in the Guide Spec: l/b_f, l/R, and f_w/f_b. Bending and warping stresses, at brace points only, need be computed for these equations because the stress condition at the midspan location is considered implicitly in the derivation of the equations.

Warping stresses, f_w, used in determining ρ_w are assumed to be from a strength of materials analysis that ignores changes in the geometry of the structure or cross-section deformation. Therefore, these warping stresses do not include radial or amplification effects. As lateral bending stresses due to distortion and radial effects occur, they must displace the warping stresses such that the total lateral flange bending stresses satisfy equilibrium. If lateral flange bending stress due to cross-section distortion is found to equal the original warping stress, the f_w to be used in determination of ρ_w should be zero.

Examination of Figure B-1 further supports this interpretation. The McManus approach denotes very high distortion and amplification stresses, so that their sum is almost equal to the warping stress. The distortion and amplification stresses cannot be so large for a girder segment that is braced according to the AASHTO Guide Spec, even in the inelastic range. This is demonstrated in Annex I by example calculations. Figure B-1 was prepared for the case with a warping-to-bending stress ratio of 0.50 at the brace points. For the two web slenderness ratios shown, the contribution of the warping stress to the yield stress is greater than that of the bending stress. Calculations for the example in Annex I demonstrate that such large warping stresses are unlikely.

Compact sections are assumed to be plastic over zero length, and redistribution of moment is assumed not to occur. For short unbraced lengths with small curvature, the equation gives bending strengths greater than the yield moment. McManus predicts that bracing forces are greater than those computed using a first-order elastic analysis for the noncompact case because of amplification effects (*47*). The bracing forces are greater, again, for the plastic case because plastification requires greater lateral restraint to develop plastic lateral bending moments. To create the plastic moment, the brace forces can be as much as 8 percent of the flange force (*47*). The amount of additional bracing force beyond that determined from a first-order analysis of a curved I-girder is not discussed.

A.1.g Fukumoto. Fukumoto and Nishida developed a quartic equation to predict the ultimate strength of a curved compact I-beam that must be solved using an iterative process (*28*). This predictor equation is unique in that it is not related to an existing lateral torsional buckling equation for a straight girder. The theoretical work was also limited to doubly symmetric sections. Whether shear was considered is unclear.

A.1.h Hanshin Design Provisions. The Hanshin design provisions are based on working stress design (*38*). In order to compare them to the strength predictor equations of Culver and Fukumoto, factors of safety were removed. These provisions use a two-ratio interaction equation relating vertical and lateral bending stresses:

$$\frac{f_c}{F_c} + \frac{f_l}{F_y} \leq 1.0 \qquad \text{(B-8)}$$

where

f_c = vertical bending stress,
f_l = lateral bending stress,
F_c = allowable vertical bending stress, and
F_y = minimum specified yield stress.

F_c is the curved-girder allowable stress that is determined from modifications of the straight-girder allowable stress in the Japanese Bridge Specifications (*39*). The V-load method is used to compute the lateral flange bending moment to determine f_l. The Hanshin provisions appear to be applicable to singly symmetric sections; however, their development is not known. Probably they have been developed from curve fitting to finite element results and confirmed by tests.

A.1.i Finite Element Studies. Recently, inelastic large deflection finite element modeling has been used, with varying degrees of success, in predicting the behavior of test girders by several researchers (*45,53,54*). A single equation is being developed for predicting the ultimate strength of I-sections by Yoo using finite element results (*55*).

A.1.j Unsymmetrical Sections. McManus assumes coincident centers of gravity and shear, as well as inextensibility of these centers. The theoretical solution is somewhat more complex when these assumptions cannot be made.

Horizontally curved unsymmetrical sections have not been studied, although there are two situations where they are important. First, in the positive moment region of noncomposite girders, the top compression flange is often smaller than the bottom tension flange. Second, singly symmetric sections always exist in composite bridges after the deck has cured. The bottom flange must be designed considering stresses due to lateral bending and vertical bending in the noncomposite section, plus similar stresses in the composite section. There has been little theoretical or experimental research relating to either condition.

A.2 Bending - Experimental

Mozer et al. (*49,56,57*), Fukumoto and Nishida (*28*), and Nakai et al. (*31,32*) performed tests of single curved, doubly symmetric I-girders. Mozer et al. also tested a pair of I-girders (*57*). Daniels et al. (*10,13*) and Nakai and Kotoguchi (*35*) have tested pairs of I-girders not reported here. Consistent with the theoretical models, all component tests were doubly symmetric. Plate sizes and their reported yield stresses for 27 curved I-girder specimens are presented in Tables B-1 and B-2.

Figures B-2 through B-4 show the schematic testing arrangement for the Mozer-Culver tests (*49,56*). A concentrated load was applied at midspan in Specimens C8-2, C9-2, D13, and D14 (*49*). The ends of the specimens were braced against rotation about their longitudinal axes. The concentrated load was applied through a steel ball some transverse distance off the shear center of the section to create additional torque. The lateral position of the load was set to provide a warping stress in the center of the test panel equal to 0.3 times the vertical bending stress at the same longitudinal

TABLE B-1 Mozer-Culver test specimens: I-girder (*49,56,57*)

Specimen	Unbraced Length (in)	Radius (in)	Web Depth (in)	Web Thick. (in)	Web Area (in^2)	F_{yw} (ksi)	Flange Width (in)	Flange Thick. (in)	Flange Area (in^2)	F_{yc} (ksi)
C8-2	60	1113	17.95	0.127	2.28	31.5	7.52	0.371	2.79	41.3
C9-2	60	378	18.00	0.120	2.16	35.1	7.47	0.368	2.75	34.6
D13	60	443	17.94	0.120	2.15	35.1	8.60	0.368	3.16	34.6
D14	60	442	18.00	0.120	2.16	35.1	8.48	0.368	3.12	34.6
L1-A	60	595	17.87	0.120	2.14	29.7	5.94	0.390	2.32	41.1
L2-A	60	607	17.93	0.119	2.13	29.7	6.00	0.390	2.34	41.1
L2-B	60	607	17.93	0.119	2.13	29.7	6.00	0.390	2.34	41.1
L2-C	60	607	17.93	0.119	2.13	29.7	6.00	0.390	2.34	41.1
GI-3	60	600	17.98	0.120	2.16	40.5	3.92	0.501	1.96	33.6
GI-4	60	600	17.98	0.120	2.16	40.5	3.92	0.501	1.96	33.6
GI-5	60	600	17.98	0.120	2.16	40.5	3.92	0.501	1.96	33.6
GO-8	63.6	636	18.01	0.312	5.62	38.7	5.99	0.380	2.27	37.6

TABLE B-2 Nakai and Fukumoto test specimens: I-girder (*28,31*)

Specimen	Unbraced Length (in)	Radius (in)	Web Depth (in)	Web Thick. (in)	Web Area (in^2)	F_{yw} (ksi)	Flange Width (in)	Flange Thick. (in)	Flange Area (in^2)	F_{yc} (ksi)
M1	78.7	∞	31.5	0.177	5.57	46.2	7.09	0.472	3.35	56.1
M2	78.7	1157	31.4	0.177	5.56	46.2	7.16	0.472	3.38	56.1
M3	94.3	1157	31.4	0.176	5.53	42.2	7.09	0.474	3.36	49.2
M4	78.7	409	31.4	0.177	5.56	46.2	7.09	0.473	3.35	56.1
M5	78.7	420	31.5	0.127	4.00	39.1	7.09	0.471	3.34	49.2
M6	78.7	425	31.4	0.122	3.83	36.8	7.16	0.469	3.36	49.2
M7	78.6	420	31.5	0.177	5.57	37.5	7.14	0.472	3.36	49.2
M8	78.5	366	31.5	0.180	5.67	37.5	3.56	0.469	1.67	49.2
M9	94.2	419	31.5	0.180	5.67	42.2	7.09	0.473	3.35	49.2
AR-1	66.9	912	9.90	0.221	2.19	34.1	4.00	0.331	1.32	34.1
AR-2	66.9	1991	9.92	0.224	2.22	34.1	3.98	0.327	1.30	34.1
AR-3	66.9	6676	9.92	0.228	2.26	34.1	3.97	0.327	1.30	34.1
BR-1	110.2	1337	9.85	0.217	2.14	34.1	3.96	0.331	1.31	34.1
BR-2	110.2	2839	9.91	0.224	2.22	34.1	3.96	0.327	1.29	34.1
BR-3	110.2	19002	9.86	0.221	2.18	34.1	3.94	0.327	1.29	34.1

location. Stresses in the specimens were computed using Dabrowski (*6*) strength of materials methods. Shear in the test panels was equal to half of the applied load.

Specimen L1-A had double-sided transverse stiffeners cut short of the tension flange but fitted to the compression flange. Specimen L2-A had full-depth, double-sided transverse stiffeners fitted against both flanges. Specimens L1-A and L2-A were tested with two concentrated loads applied over the web at third-points, creating a nearly pure bending region over the center-third of the specimen. These specimens were restrained against rotation at their ends and at the load points, as shown in Figure B-3. These two tests were the only single-girder tests by Mozer et al. (*56*) that matched the analytical work by Culver and McManus. The concentrated loads were arranged such that the computed warping stress at center brace points was approximately one-third of the vertical bending stress in Specimens L1-A and L2-A.

Specimens L2-B and L2-C were tested with a single concentrated load placed at midspan as shown in Figure B-3. This arrangement produced rather steep moment gradients in these specimens. The loading arrangement itself provided lateral restraint at the point of application. The ratio of warping stress to bending stress at the center of the test panel, which was near the load point, was 0.50 in L2-B and 0.25 in L2-C. These stress ratios were accomplished by applying the concentrated load eccentric with respect to the girder web, creating a torque in addition to that due to curvature. These specimens were braced laterally at the ends, possibly causing some bending restraint.

Specimens GI and GO were connected by intermediate full-depth cross frames and near full-depth diaphragms at the bearings, as shown in Figure B-4 (*57*). The girders were spaced 3 ft apart. This arrangement represented behavior of curved girders before the deck cures. GI was the inside girder; GO was the outside girder. Cross frames could be removed to change the unbraced length for different tests. The center cross frame was removed in Tests 5 and 8, which were pure bending tests. In Test 5, two concentrated loads were applied at third-points of the span over GI, as shown in Figure B-4, and GI was tested to failure. In Test 8, two concentrated loads were applied at third-points of the span, but they were applied to the cross frames three-quarters of the distance from GI toward GO to ensure uplift at GI did not occur. Girder GO was tested to failure in this test.

These two tests modeled the assumptions of the McManus-Culver analytical model. The GI computed warping stress at the brace points in Test 5 was approximately 0.75 of the vertical bending stress. The comparable ratio in girder GO was 0.57 in Test 8. Tests G3 and G4 were combined bending and shear tests, with only one concentrated load applied at either the one-third or two-third point. The test arrangement was similar to that shown in Figure B-4. The G-test series most closely represented actual bridges of the tests by Mozer-Culver.

Mozer-Culver single specimen tests and Nakai's M-test series employed significantly more torsional fixity than exists in typical curved bridges. The M-test series was performed in nearly pure negative bending with very rigid rotational restraints at the ends, as shown in Figure B-5.

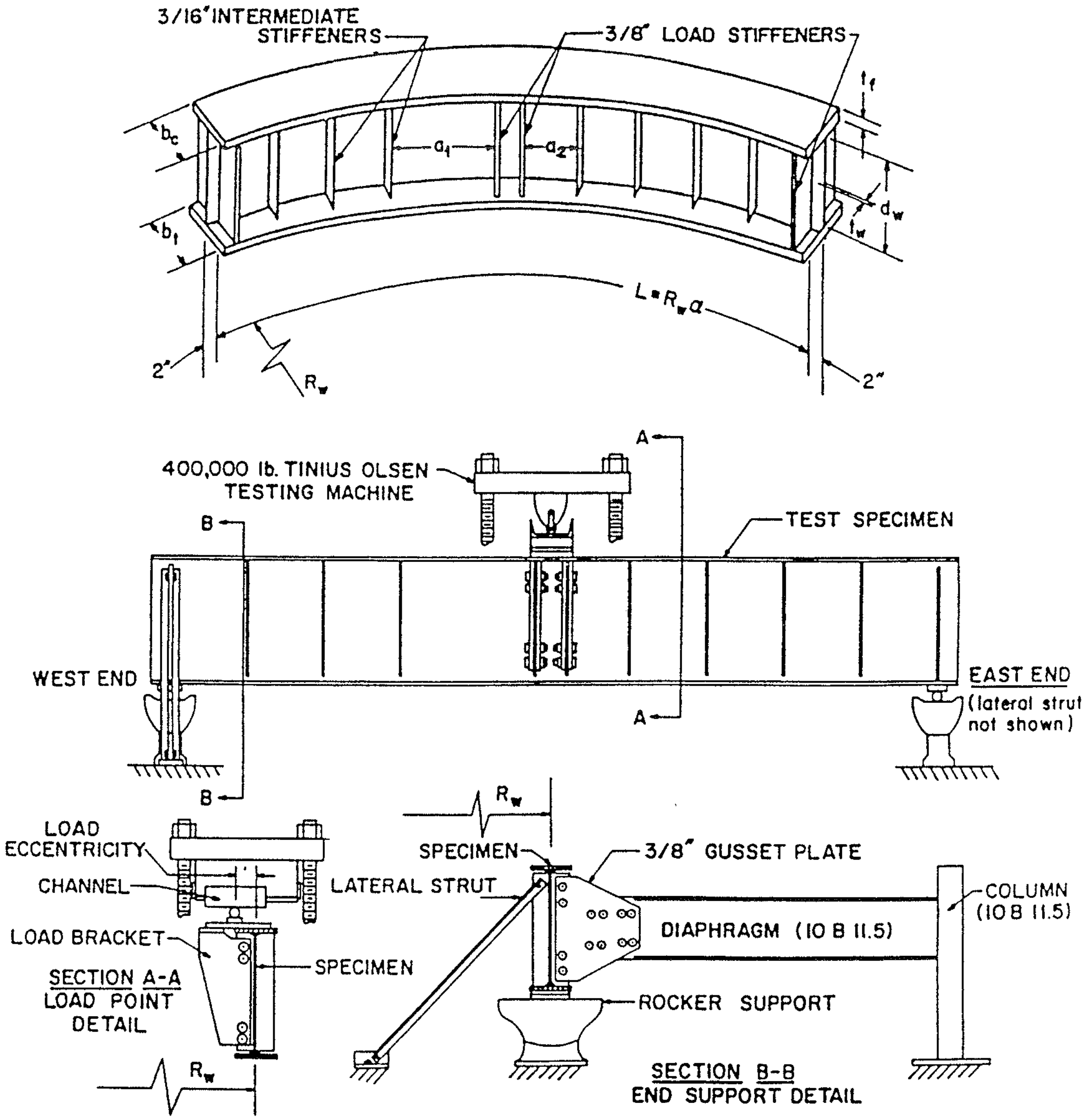


Figure B-2. Mozer-Culver tests C8-2, C9-2, D13, D14 (49).

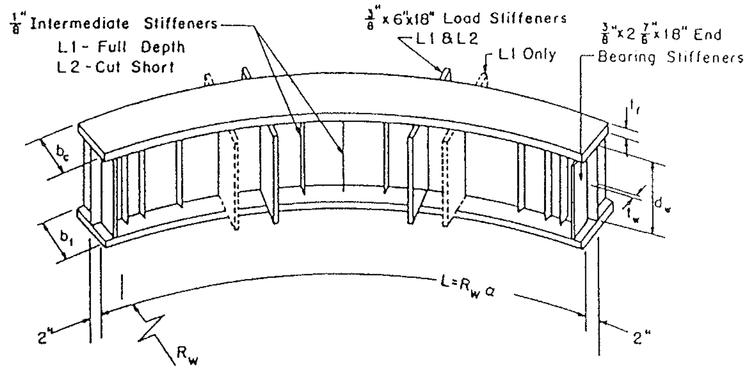
Fukumoto tested simple-span specimens with a single concentrated load applied through a gravity simulator mechanism at the center of the span (*29*). The top and bottom of the girders were laterally restrained at the ends of the span, similar to the Mozer-Culver tests C8-2, C9-2, D13, and D14. This test arrangement resembles behavior of a simple-span single girder with end tie-downs during erection, except shear is nearly constant in each half of the span and the moment varies in a nearly linear manner.

Geometric ratios of 27 curved-girder test specimens are presented in Tables B-3 and B-4. Web slenderness of the twenty-seven specimens varies from 58 to 257. There are two specimens with web slenderness over 225; one of these has a longitudinal stiffener. Nakai et al. noted that transverse stiffeners provide additional strength to curved girders tested in flexure (*31,32,37*).

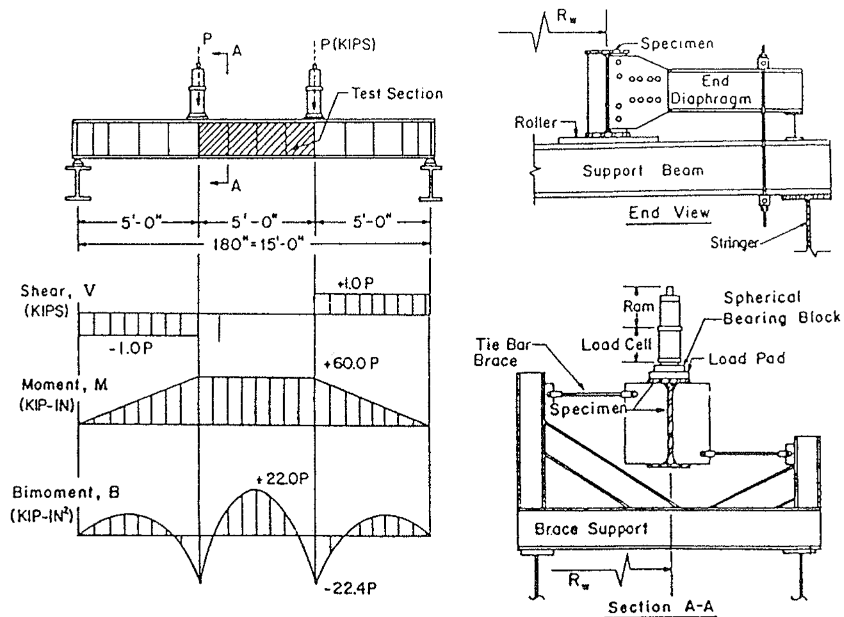
The ratio of flange-to-web yield stress for the specimens listed in Tables B-3 and B-4 ranges between 0.83 and 1.38. A typical hybrid girder exists when this ratio is greater than 1.0. The reported yield stresses were used to compute limiting flange and web slenderness based on the McManus-Culver analytical model, as shown. Measured flange slenderness values are also presented for the specimens.

A.3 Test Moments versus Predicted Moments

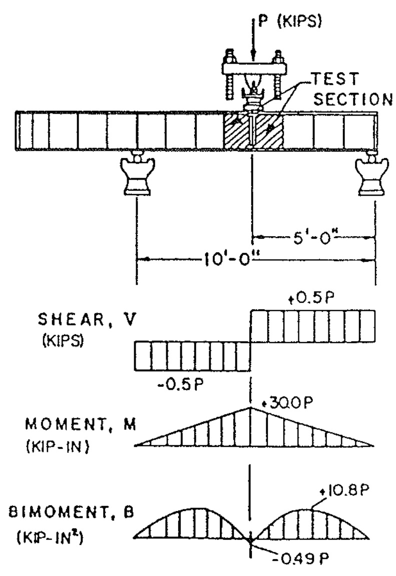
A computer program was written to predict curved-girder moment capacities using the McManus-Culver, Fukumoto, and Hanshin bending strength predictor equations. The McManus-Culver equations and the Hanshin provisions require input of the warping stresses at brace points. The lat-



Specimens L1, L2



Tests L1-A, L2-A



Tests L2-B, L2-C

Figure B-3. Mozer-Culver tests L1-A, L2-A, -B, -C (56).

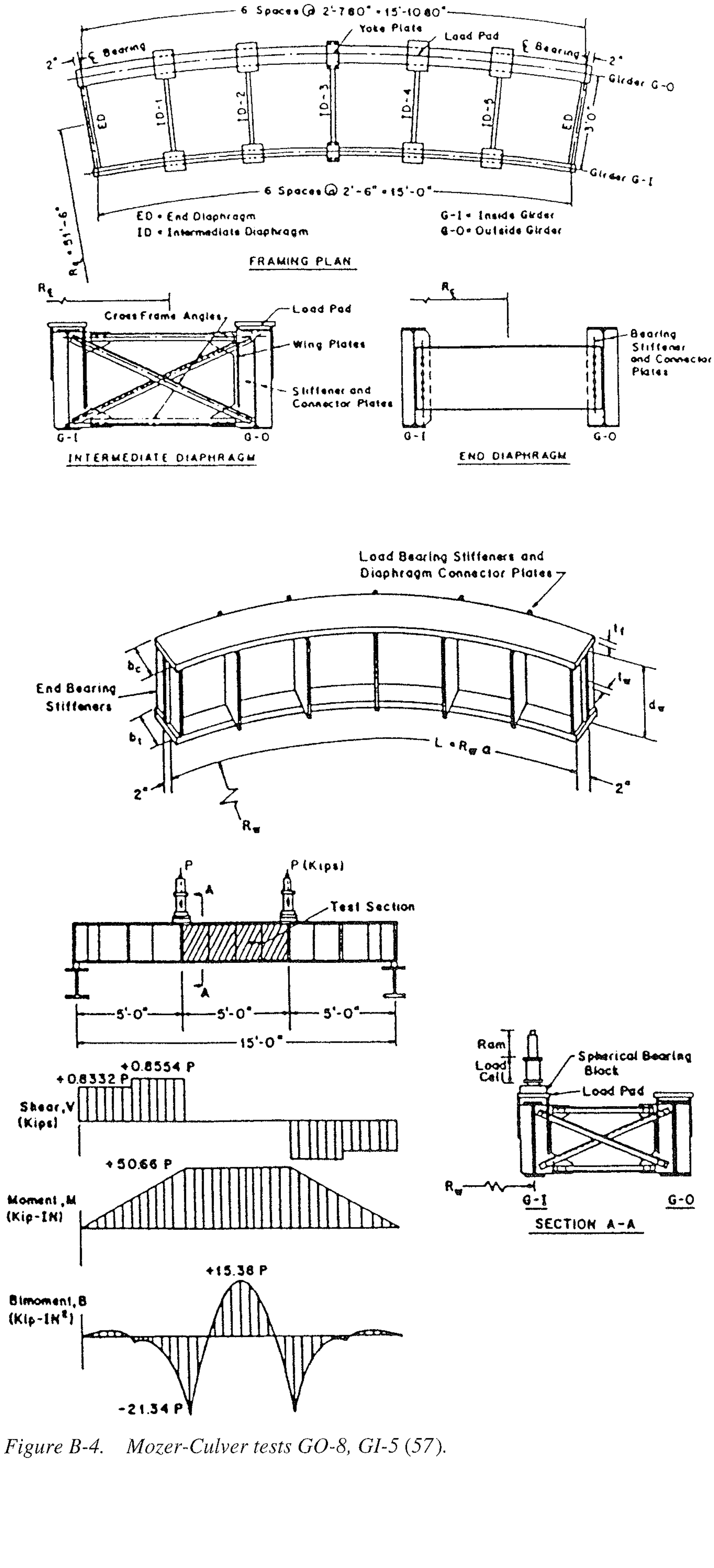
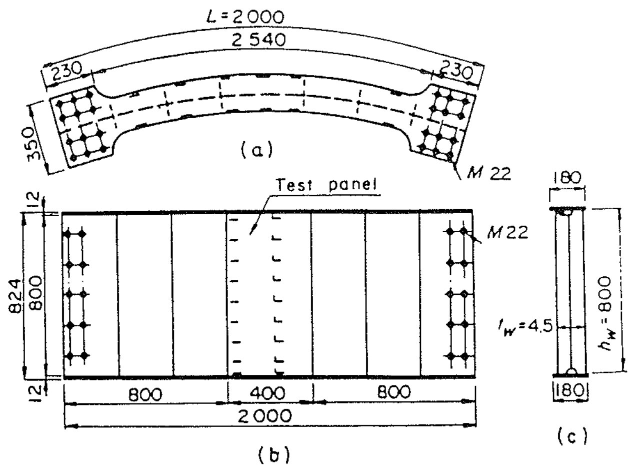


Figure B-4. Mozer-Culver tests GO-8, GI-5 (57).



Detail of a test girder under bending (in mm): (*a*) plan, (*b*) side elevation, and (*c*) cross section.

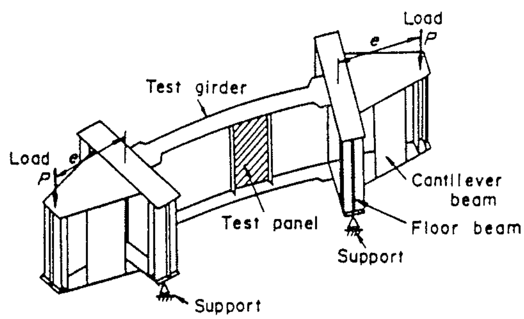


Figure B-5. Nakai tests M1-M9 (31).

eral bending stress values used in the predictions were determined using the V-load method. The lateral bending stress and warping stress values for the specimens are given in Tables B-3 and B-4.

McManus-Culver equations are applicable when l/b_f is not greater than 25; *R/l* is greater than 10; and the absolute value of f_w/f_b is not greater than 0.5. These limitations are used in the Guide Spec (*9*). The equations were used to predict some test values that exceeded these limits. Predicted moments are based on the assumption that the beam is torsionally restrained and that the bending moment is uniform over the braced length. The Hanshin provisions have limits of applicability similar to the McManus-Culver equations. Again, these limits were ignored when predicting the ultimate moment of the test specimens.

Failure of a compact beam requires the formation of a plastic mechanism. To obtain a mechanism, a hinge must occur at each brace point and at midspan. The McManus-Culver model predicts only the first hinge, assuming that the remainder of the beam remains elastic. Failure of the beam is expected to occur at a load greater than that causing the first hinge. When the lateral flange bending stress is small, that is, *R/l* is large, the beam should fail predominantly in vertical bending. Torsionally pinned simple-span specimens need only a hinge at midspan to form a mechanism.

The lateral flange bending stress computed from the virtual "q" load in Equation (B-6) in the V-load method (*1*) is used to compute both the compact plastic and noncompact first yield moment values in Tables B-5 and B-6 for the Culver and Hanshin cases.

To investigate the assumption that the McManus-Culver equation may double count lateral bending stress due to curvature if the V-load method is used to determine f_w, the

TABLE B-3 Mozer-Culver test specimens: I-girder selected ratios (*56,57*)

Specimen	ℓ/b_f	R/ℓ	Compact D/t_w	D/t_w	$3{,}200/\sqrt{F_y}$ b_f/t_f	b_f/t_f	$4{,}400/\sqrt{F_y}$ b_f/t_f	F_{yc}/F_{yw}	A_w/A_f	a d_o/D	Long. Stiff.	f_w/f_b	f_m/f_b
C8-2	8.0	18.6	108	141	15.8	20.3	21.7	1.31	0.80	0.67	no	.29	-.29
C9-2	8.0	6.3	103	150	17.2	20.3	23.7	0.97	0.77	1.33	no	.84	-.84
D13	7.0	7.4	103	150	17.2	23.4	23.7	0.97	0.68	0.67	no	.62	-.62
D14	7.0	7.4	103	149	17.2	23.0	23.7	0.97	0.74	0.67	no	.63	-.63
L1-A	10.0	9.9	112	149	15.8	15.2	21.7	1.38	0.92	0.83	no	.69	-.69
L2-A	10.0	10.1	112	151	15.8	15.4	21.7	1.38	0.91	0.83	no	.67	-.67
L2-B	10.0	10.1	112	151	15.8	15.4	21.7	1.38	0.91	0.83	no	.67	-.67
L2-C	10.0	10.1	112	151	15.8	15.4	21.7	1.38	0.91	0.83	no	.67	-.67
GI-3	15.3	10.0	95	150	17.5	7.82	24.0	0.83	1.10	1.00	no	1.05	-1.05
GI-4	15.3	10.0	95	150	17.5	7.82	24.0	0.83	1.10	0.83	no	1.05	-1.05
GI-5	15.3	10.0	95	150	17.5	7.82	24.0	0.83	1.10	0.83	no	1.05	-1.05
GO-8	10.0	10.6	98	58	16.5	15.7	22.7	0.97	2.48	none	no	.77	-.77

TABLE B-4 Nakai and Fukumoto test specimens: I-girder selected ratios (*28,31*)

Specimen	ℓ/b_f	R/ℓ	Compact D/t_w	D/t_w	$3{,}200/\sqrt{F_y}$ b_f/t_f	b_f/t_f	$4{,}400/\sqrt{F_y}$ b_f/t_f	F_{yc}/F_{yw}	A_w/A_f	Aspect a	Long. Stiff.	f_w/f_b Brace	f_m/f_b Midspan
M1	11.1	∞	89.5	178	13.5	15.0	18.6	1.21	1.66	0.5	no	0	0
M2	11.0	14.7	89.5	177	13.5	15.2	18.6	1.21	1.64	0.5	no	.56	-.56
M3	13.3	12.3	93.6	178	14.4	15.0	19.8	1.17	1.65	1.0	no	.81	-.81
M4	11.1	5.2	89.5	177	13.5	15.0	18.6	1.21	1.66	0.5	no	1.60	-1.60
M5	11.1	5.3	97.3	248	14.4	15.1	19.8	1.26	1.20	0.5	no	1.47	-1.47
M6	11.0	5.4	100.2	257	14.4	15.3	19.8	1.34	1.14	0.5	yes	1.43	-1.43
M7	11.0	5.3	99.3	178	14.4	15.1	19.8	1.31	1.65	0.5	no	1.55	-1.55
M8	22.1	4.7	99.3	175	14.4	7.6	19.8	1.31	3.40	0.5	no	4.30	-4.30
M9	13.3	4.5	93.6	175	14.4	15.0	19.8	1.17	1.69	1.0	no	2.20	-2.20
AR-1	16.7	13.6	104.1	45	17.3	12.1	23.8	1.0	1.66	none	no	.90/0	0
AR-2	16.8	29.8	104.1	44	17.3	12.2	23.8	1.0	1.71	none	no	.42/0	0
AR-3	16.9	99.8	104.1	44	17.3	12.1	23.8	1.0	1.74	none	no	.12/0	0
BR-1	27.8	12.1	104.1	45	17.3	12.0	23.8	1.0	1.63	none	no	1.67/0	0
BR-2	27.8	25.8	104.1	44	17.3	12.1	23.8	1.0	1.72	none	no	.80/0	0
BR-3	28.0	172	104.1	45	17.3	12.0	23.8	1.0	1.69	none	no	.12/0	0

McManus-Culver predictor equations were used again, assuming f_w equals zero. The comparison of test results to the new moments is shown in Tables B-7 and B-8. Assuming the warping stress is zero at brace points is different from assuming there is no lateral flange bending stress at these points in the McManus-Culver predictor equations. Lateral bending stress due to radial and amplification effects is tacitly considered, as discussed earlier.

The effect of assuming the warping stress is zero on plastic moment based on combined vertical and lateral moments, $M_{compact}$, is to decrease the predicted moment capacity. The effect on the first yield moment M_y is to increase the predicted moment capacity. Generally, the strength predictions compared to test values are thought to be improved over those in Tables B-5 and B-6 where double counting of warping stress is thought to have occurred.

Measured lateral flange bending stresses reported by Mozer et al. (*56,57*) were significantly different from those computed using the strength of materials method. Lateral flange bending stresses were not reported by the other researchers. Although

TABLE B-5 Mozer-Culver I-girder moment test data compared to strength equations

Specimen	M_{TEST} (k-in)	Culver				Hanshin		Fukumoto	
		Compact		Noncompact					
		M_c (k-in)	Ratio	M_n (k-in)	Ratio	M_u (k-in)	Ratio	M_u (k-in)	Ratio
C8-2	2241	2342	1.05	1521	0.68	1778	0.79	2190	0.98
C9-2	1794	1627	0.91	598	0.33	976	0.54	1453	0.81
D13	2046	1958	0.96	903	0.44	1274	0.62	1824	0.89
D14	1938	1935	1.00	885	0.46	1254	0.65	1803	0.93
L1-A	1830	1716	0.94	843	0.46	1107	0.60	1509	0.82
L2-A	1830	1749	0.96	872	0.48	1136	0.62	1540	0.84
L2-B	1833	1749	0.95	872	0.48	1136	0.62	1540	0.84
L2-C	1944	1749	0.90	872	0.45	1136	0.58	1540	0.79
GI-3	1636	1058	0.65	453	0.28	651	0.40	991	0.61
GI-4	1674	1058	0.63	453	0.27	651	0.39	991	0.59
GI-5	1377	1058	0.77	453	0.33	651	0.47	991	0.72
GO-8	2120	1995	0.94	931	0.44	1167	0.55	1923	0.91

TABLE B-6 Nakai and Fukumoto I-girder moment test data compared to strength equations

Specimen	M_{TEST} (k-in)	Culver				Hanshin		Fukumoto	
		Compact		Noncompact					
		M_c (k-in)	Ratio	M_n (k-in)	Ratio	M_u (k-in)	Ratio	M_u (k-in)	Ratio
M1	8098	7844	0.97	6930	0.85	7428	0.92	7989	0.99
M2	7752	7062	0.91	3692	0.48	4653	0.60	6169	0.80
M3	6131	5718	0.93	2620	0.43	3498	0.57	5145	0.84
M4	7203	4774	0.66			2618	0.36	4261	0.59
M5	5902	3995	0.68			2291	0.39	3589	0.61
M6	6287	4024	0.64			2311	0.37	3558	0.57
M7	6547	4338	0.66			2366	0.36	3827	0.58
M8	2935							1069	0.36
M9	5548	2575	0.46					3126	0.56
AR-1	442					287	0.65	420	0.95
AR-2	463					387	0.84	510	1.10
AR-3	548					498	0.91	599	1.09
BR-1	270					199	0.74	260	0.96
BR-2	335					303	0.90	342	1.02
BR-3	387					443	1.27	474	1.22

no longitudinal forces at bearings were reported, it is likely that significant lateral forces occurred at the bearings in all tests. When restraining forces at bearings are in line with the girder, they produce thrust and negative bending in the specimen.

The flanges of the first four specimens in Table B-3 are noncompact. Specimens C9-2, D13, and D14 failed by local flange buckling (*49*). Specimens L2-B and L2-C failed by web buckling. All five of these specimens were subjected to shear in the test panels. They failed in modes other than bending and/or lateral bending; thus, they did not test the theoretical models for pure bending.

When the warping stress is that determined by the V-load method, the ultimate moment capacity predicted by McManus-Culver is unconservative by 6 to 27 percent, as shown in Tables B-5 and B-6. When the warping stress is assumed to equal zero, the predicted plastic moments are reduced, as shown in Tables B-7 and B-8. Predicted moments for four Fukumoto specimens exceed the test moments. The specimens are subjected to high shear in these cases.

TABLE B-7 Test moment versus compact and noncompact moments with $f_w = 0$

Test	M_{TEST} (k-in)	Compact		Noncompact		M_n / M_c
		M_c (k-in)	M_c / M_{TEST}	M_n (k-in)	M_n / M_{TEST}	
C8-2	2241	2171	0.97	1591	0.71	0.73
C9-2	1794	1346	0.75	828	0.46	0.62
D13	2046	1683	0.82	1101	0.54	0.65
D14	1938	1659	0.86	1082	0.56	0.65
L1-A	1830	1450	0.79	925	0.51	0.64
L2-A	1830	1484	0.81	951	0.52	0.64
L2-B	1833	1484	0.81	951	0.52	0.64
L2-C	1944	1484	0.76	951	0.49	0.64
GI-3	1636	843	0.52	505	0.31	0.60
GI-4	1674	843	0.50	505	0.30	0.60
GI-5	1377	843	0.61	505	0.37	0.60
GO-8	2120	1580	0.75	999	0.47	0.63

TABLE B-8 Test moment versus compact and noncompact moments with $f_w = 0$

Test	M_{TEST} (k-in)	Compact		Noncompact		M_n / M_c
		M_c (k-in)	M_c / M_{TEST}	M_n (k-in)	M_n / M_{TEST}	
M1	8098	7819	0.97	6865	0.85	0.88
M2	7752	6112	0.79	3986	0.51	0.65
M3	6131	4695	0.77	2871	0.47	0.61
M4	7203	3737	0.52	2216	0.31	0.59
M5	5902	3160	0.54	1882	0.32	0.60
M6	6287	3180	0.51	1895	0.30	0.60
M7	6547	3384	0.52	2015	0.31	0.78
M8	2935	677	0.23	528	0.18	0.60
M9	5548	2530	0.46	1529	0.28	0.60
AR1	442	389	0.88	227	0.51	0.58
AR2	463	498	1.07	320	0.69	0.64
AR3	548	551	1.01	430	0.81	0.78
BR1	270	209	0.77	121	0.45	0.58
BR2	335	362	1.08	193	0.58	0.53
BR3	387	442	1.14	344	0.89	0.78

The predicted moments at first-yield increase when the warping stress is assumed equal to zero, as seen in Tables B-7 and B-8. The predicted values are lower, compared to observed ultimate moments.

Nonlinear behavior was visually identified from plots of vertical deflection versus concentrated load for the Mozer pure bending tests. These values, as a percent of ultimate loads, are compared to the computed first yield moment divided by the observed ultimate moment in Table B-9. Predicted moment values assuming a warping stress value based on the V-load method and assuming warping stress equal to zero are compared to the moment at which departure from linear behavior was observed. The ratio of the observed nonlinear behavior value divided by predicted first yield is presented. The first yield moment is a conservative predictor of the onset of nonlinear behavior. Table B-10 gives similar values for the test specimens loaded in bending and shear.

Nonlinear behavior is related to yielding and deflection amplification. Yielding occurs prior to computed first yield values when residual stresses are present. Heat-curving of girder flanges can increase residual stresses significantly; however, only flanges of Specimen C8 of the Mozer tests were heat-curved. Flanges of the other Mozer specimens were cut-curved, which tends to produce lower residual stresses in the flanges.

The specimens under pure moment deviate from linear behavior at higher moments than those predicted using McManus' first yield. The specimens under combined shear and bending tend to deviate from linear behavior at moments less than those predicted with the first yield criterion. The largest error in predicting nonlinear behavior using first yield occurred for Specimen C8, which was heat-curved. However, Specimen C8 also has the largest radius of any specimen so lateral bending was smaller and the predicted moment was relatively large. It appears that first yield is a

TABLE B-9 Nonlinear deflection comparison, pure moment case

Specimen	P_n/P_{ut} at First Nonlinear Defl (1)	M_n / M_{TEST} f_w by V-load (2)	Predicted/ Observed (2)/(1)	M_n / M_{TEST} $f_w = 0$ (3)	Predicted/ Observed (3)/(1)
L1-A	0.70	0.46	0.66	0.51	0.73
L2-A	0.55	0.48	0.87	0.52	0.95
GI-5	0.55	0.33	0.60	0.37	0.67
GO-8	0.90	0.44	0.49	0.47	0.52

TABLE B-10 Nonlinear response compared to first yield, single load case

Specimen	P_n/P_{ul} at First Nonlinear Defl (1)	M_n/M_{TEST} f_w by V-load (2)	Predicted/ Observed (2)/(1)	M_n/M_{TEST} $f_w = 0$ (3)	Predicted/ Observed (3)/(1)
L2-B	0.55	0.48	0.87	0.52	0.95
L2-C	0.55	0.45	0.82	0.49	0.89
C8*	0.65	0.68	1.05	0.71	1.09
C9	0.45	0.33	0.73	0.46	1.00
D13	0.45	0.44	0.98	0.54	1.20
D14	0.50	0.46	0.92	0.56	1.12

** Heat-curved*

reasonable estimate of the onset of nonlinear behavior for the curved I-girders tested.

The Hanshin criterion gives moments lower than Fukumoto's equation in most cases, higher than Culver's first yield moments, and lower than Culver's plastic moment values.

A.4 Shear Strength

Shear buckling strength of a curved web is higher than the shear buckling strength of the same flat web panel (*58*). However, it does not follow that the ultimate strength of a curved web is higher than that of a flat web. There have been numerous tests of curved I-beams subjected to flexural shear, but these are not reported herein because no new strength models have been presented for these tests. Shear-moment tests were performed by Mozer et al., Nakai et al., Brogan, and Culver. These tests indicated the ultimate shear strength of curved webs may be somewhat less than that of flat panels of the same size; however, post buckling strength was still evident (*8,33,58–60*). Stiffened web panels with aspect ratios near 1.0 were tested, along with unstiffened stocky webs (D/t_w<70).

Culver et al. (*60*) proposed that the elastic buckling strength of an equivalent stiffened flat plate be used for a conservative prediction of shear strength of a curved I-girder web. Shear and moment interaction of curved girders has been found to be well predicted by Rocky's model used in BS5400 (*61*). The strength of a curved I-girder web under combined shear and moment is somewhat less than the strength predicted by Basler's model (*33*).

A.5 Bracing Forces

Bracing members transfer load between the curved girders and provide for statical equilibrium; therefore, bracing forces must satisfy equilibrium between applied loads and action of the girders. The vectorial sum of horizontal cross frame forces at a point on a flange must correspond with the lateral bending in the flange. The cross frame forces cause nonuniform torsion in the girder, resulting in either warping stress or lateral flange bending due to distortion and amplification.

Forces reported by Mozer et al. in the cross frame members in Tests 5 and 8 were not in equilibrium based on first order analyses; the top-chord force was larger than the bottom-chord force in several locations (*57*). Reported compression forces were higher than predicted values, and the tension forces were lower than predicted (*57*). Two strain gages were used on each cross frame member, and the strain readings were averaged. It is possible that the discrepancies resulted from experimental error in measuring the small cross frame forces or that the bolted connections caused eccentricities not considered. However, the measured stresses due to lateral flange bending moment correlated quite well with corresponding cross frame forces. For example, where the top chord of a cross frame had a larger force than the bottom chord, there was a similar difference in lateral flange bending stress at the same point. This correlation leads to the tentative conclusion that the reported cross frame forces are reasonably correct. The difference between top and bottom flange lateral bending moments may be due to radial effect and amplification (i.e., the deformation of the structure such that a first-order analysis is not accurate). As shown in Figure B-6, the top-chord force is nearly twice the computed value based on small deflection theory in some locations.

Chord forces and end diaphragm stresses in Test 8 were nonlinear with increasing load. This indicates load shifting from girder GO to girder GI as girder GO yielded. When the applied load increased, girder GO probably became less stiff due to yielding, resulting in the girder carrying a smaller portion of the additional load. Girder yielding tends to reverse the sense of force in the cross frames. Because the strain gages on girder GO were not measuring strains beyond yielding, the failure moment was determined by extrapolating the applied load. But the loss of stiffness in GO causes an extrapolation of the elastic moment to overestimate its true moment. This is consistent with observed and computed moments in Table B-8 where the predicted moment capacity of GO is less than the observed moment capacity. This is due, in part, to the fact that GO received less load at failure than estimated by using a linear extrapolation. Confirmation of this was not possible because the corresponding moment in GI was not measured.

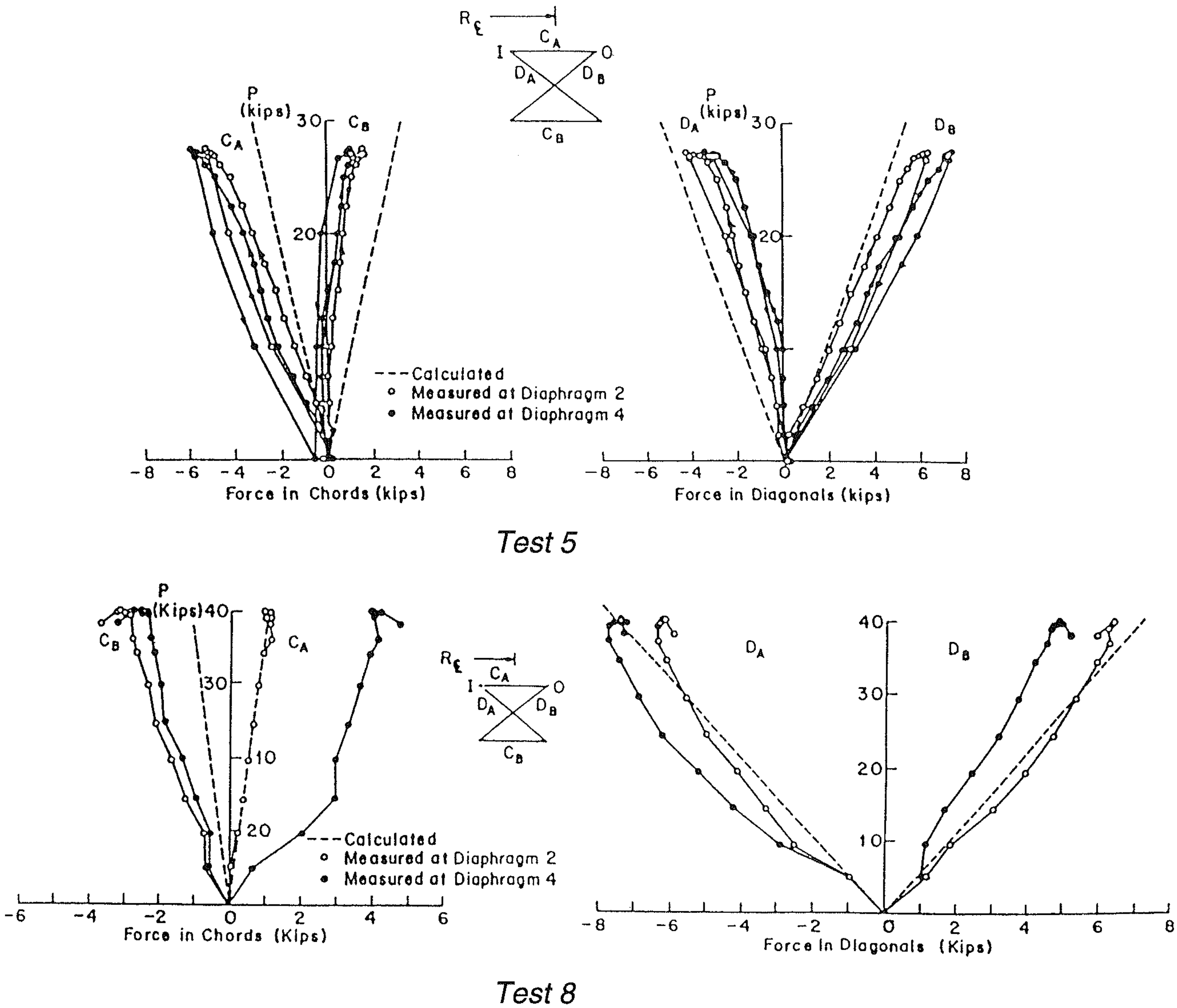


Figure B-6. Comparison of cross frame forces (57).

Due to this shift of load between girders as they yield and the change in their stiffnesses, inelastic moment in curved girders cannot be predicted from elastic analyses. A similar effect can be expected in straight-girder bridges. Due to the shift of load between girders, plastic behavior of curved girders cannot be predicted from elastic analyses.

B. Box Beams

B.1 Strength - Analytical

Behavior of curved box beams is complex because of torsion. Nasir found that torsional shear in box flanges reduces the bending tensile stress required to cause yielding (*51*). Premature yielding of the flange has, in turn, a deleterious effect on flange buckling (*51*).

A curved compression flange subjected to bending and torsion begins to deform out-of-plane as soon as torsion is applied (*51*). Longitudinal stiffeners, if present, tend to twist very early under these loading conditions (*51,62*). This behavior is different from that observed in box beams not subjected to torsion. The addition of transverse flange stiffeners increases the buckling strength of flanges subjected to torsional shear. Otherwise, flexural buckling of the flange plates is similar to flanges of straight box girders not subjected to torsional shear.

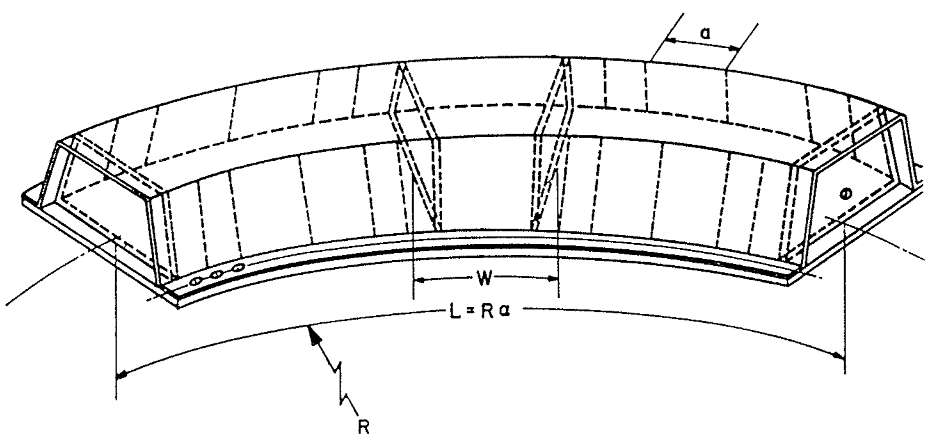
B.2 Tests

Even fewer tests were conducted on curved steel box beams than on curved steel I-beams. Box beam tests performed during the CURT project were limited to six small specimens and two larger ones. Figure B-7 describes the smaller box Specimens B1 and B2. Figure B-8 describes box Specimens B3 through B6 (*63*). All of these specimens had transverse web stiffeners spaced from 0.3 to 1.0 times the web depth. The intermediate diaphragms were ¼-in. thick plates. Figure B-9 is a schematic of the test arrangement for box Specimens B1 through B6 (*63*). Cross sections of Specimens B7 and B8 are shown in Figures B-10 and B-11, respectively (*64*). A schematic of the test arrangement for Specimens B7 and B8 is shown in Figure B-12 (*64*). Table B-11 gives the dimensions of the curved box beam Specimens B1 through B8.

A plate was used as a tension flange for each pair of Specimens B1–B2, B3–B4, and B5–B6. These tension flange plates represented an uncracked concrete deck. The yield stress of steel in these boxes was approximately 40 ksi. The boxes were tested as simple spans. Torque was applied at one end through an eccentric ram, in addition to the torque due to curvature. The two concentrated loads provided a center region of low bending shear. Bending and end torque were increased monotonically. Because torque and bending loads were applied separately, different ratios of vertical and torsional loads could be applied. Eight tests were performed on the six specimens.

Torque due to curvature associated with bending changes from negative to positive along the span, while torque due to the applied eccentric load is constant along the span. This means that the sum of torsional and bending shear stresses is greatest at the supports where the bending stress is small, and the bending stress is greatest in the middle of the specimens. Maximum applied loads are compared to calculated critical loads in Table B-12. Modes of failure for Specimens B1 through B6 are summarized in Table B-13.

The ratio of test shear strength to predicted shear strength of the box webs was less than the similar ratio observed in the curved I-girder tests. Excessive out-of-plane web deformation was observed under large torque. The test reports suggest the out-of-plane bending of box webs due to torsion might cause them to be weaker than I-girder webs. The inclination of box webs also is thought to contribute to lower strengths (*62*). The



Specimen	b (in)	t_w (in.)	t_f (in)	b_1 (in.)	b_2 (in.)	d (in.)	R (in.)	α (deg.)	L (in.)	W (in.)
B1, B2	8.0	0.0995	0.25	12.75	17.0	9.0	134.4	36.59	85.8	16.5

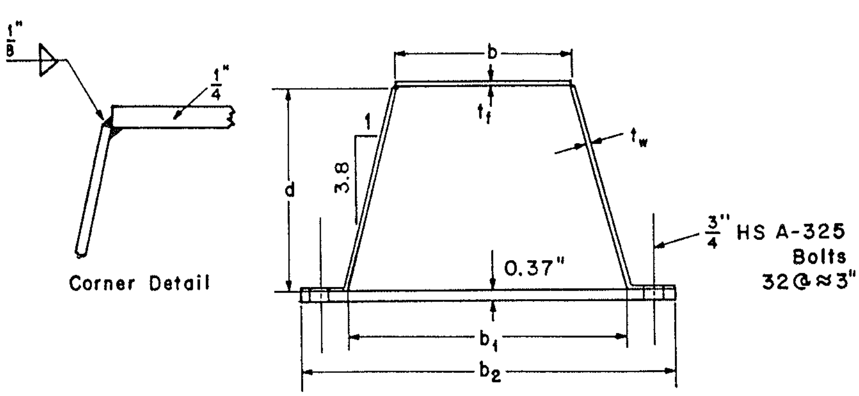
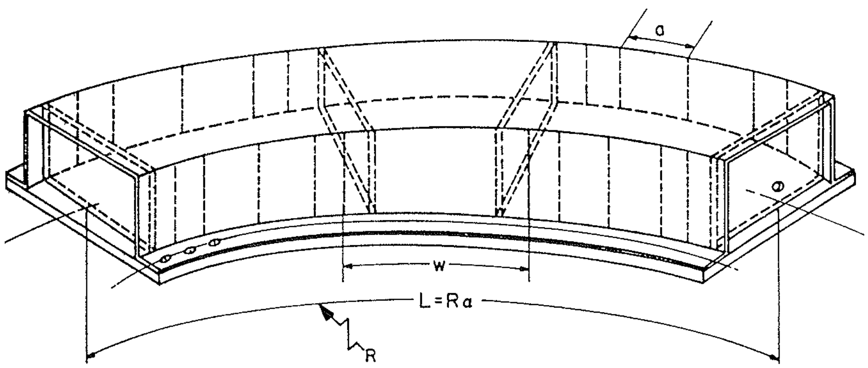


Figure B-7. Specimens B1 and B2 (63).



Specimen	b (in.)	t_w (in.)	t_f (in.)	b_1 (in.)	b_2 (in.)	d (in.)	R (in.)	α (deg.)	L (in.)	W (in.)	d_t (in.)
B3	12.75	0.0995	0.0995	12.75	17.0	9.0	646.5	7.92	89.3	25.8	2.08
B4											2.09
B5							134.4	36.59	85.8	26.8	2.04
B6											2.08

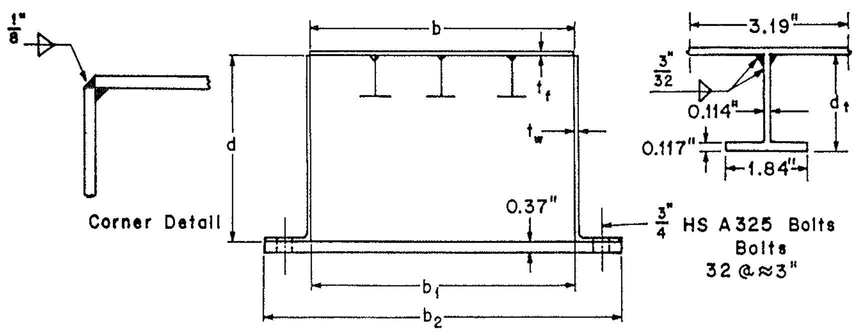


Figure B-8. Specimens B3-B6 (63).

compression flange strength was less than that in straight box flanges (*64*). Longitudinal stiffeners were observed to twist, permitting the flange to twist out of plane. Torsional shear stress was reported to contribute to this phenomenon, although curvature may have been a contributing factor.

Theoretical study indicated that curvature in the practical range was not expected to reduce the buckling strength (*62*). The ultimate strength of box-girder sections is dependent on three parameters: torsional shear stress, compression flange buckling resistance, and cross-section distortion (*63,65*). Research shows that the compression flange of curved boxes has little post-buckling strength (*62–64*). The torsional shear stress should be considered in conjunction with compression bending stress in determining buckling strength. Buckling stress is defined for three regions of slenderness: plastic, inelastic, and elastic.

Thirteen tests were performed in the elastic range on Specimens B7 and B8 (*64*). Tests to failure were subsequently performed (*65*). Three testing arrangements were used: concentrated loads at quarter points with simple-span supports (240 in.), a concentrated load at midspan with simple-span supports (240 in.), and two equal spans (120-120 in.) with equal concentrated loads at quarter points. The purpose of these tests was to evaluate deflection and behavior of box beams with practical web and flange slenderness. Measured displacements indicate that curvature of the web contributes to the overall flexural rigidity of the box, and internal cross frames are effective in reducing box deformation. Measured box rotations were greater than computed rotations, assuming no deformation of the box.

Retention of the box shape is accomplished by internal bracing members. Research has confirmed that closer-spaced bracing reduces distortion and distortion-related transverse bending stresses in the box elements. Culver suggested that additional research is needed to define the optimal bracing stiffness and spacing (*8*). Wright and Abdel-Samad presented a method to compute internal box cross frame forces and transverse bending stresses using a beam-on-elastic-foundation (BEF) analogy (*66*). Several tests were performed with different internal cross frame spacing (*64*). It was observed that cross frame spacing greatly affected both vertical deflection and rotation of the box.

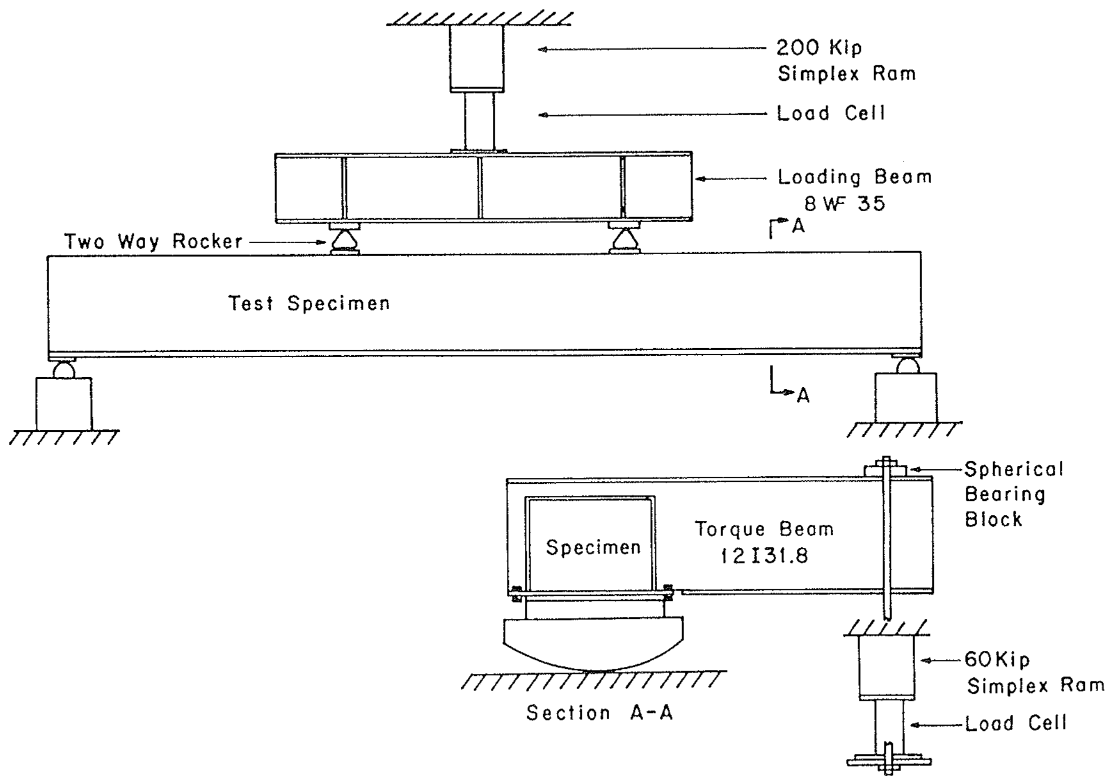


Figure B-9. Test arrangement for specimens B1-B6 (63).

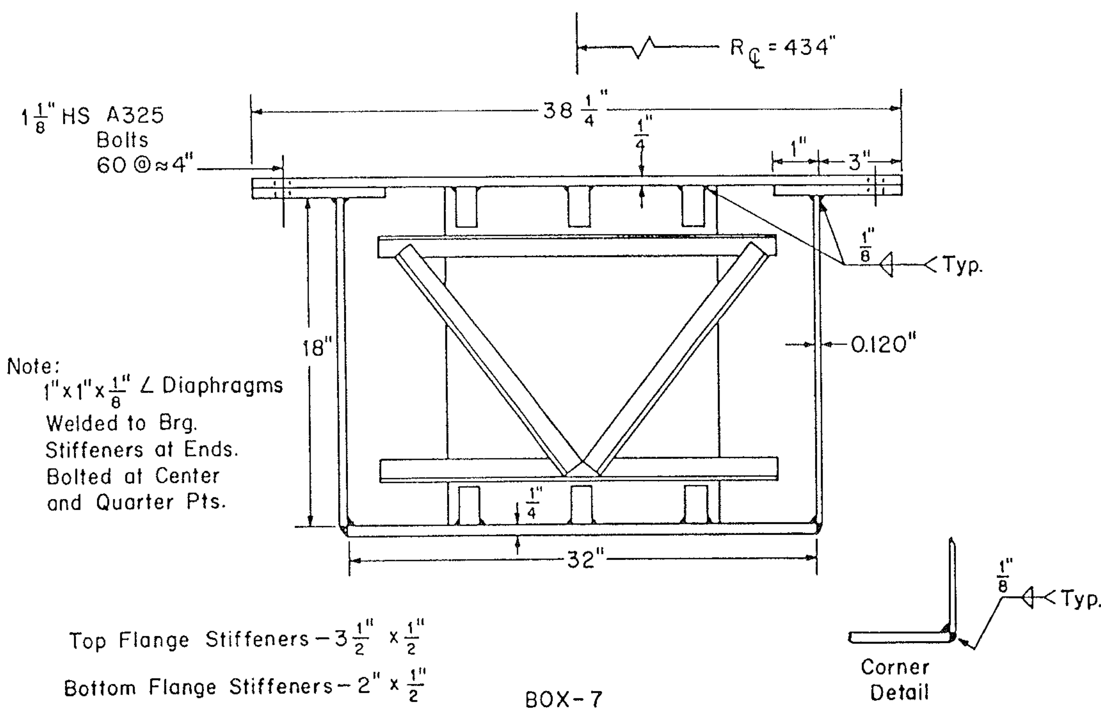


Figure B-10. Specimen B7 (64).

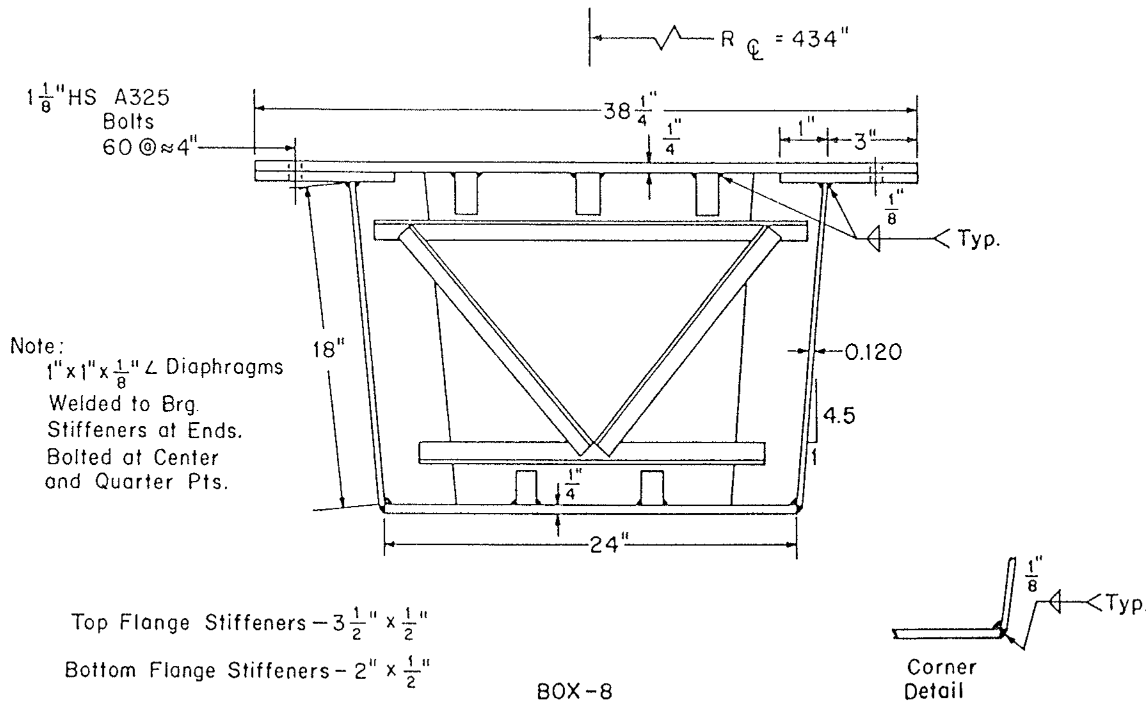


Figure B-11. Specimen B8 (64).

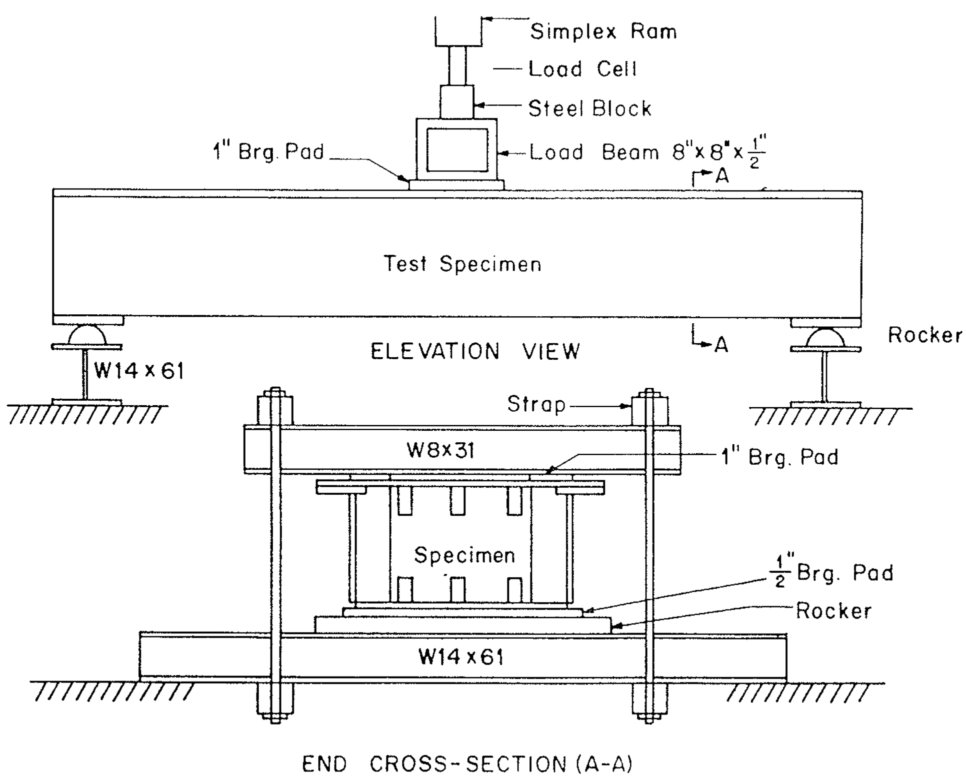


Figure B-12. Schematic loading arrangement for specimens B7 and B8 (64).

TABLE B-11 Dimensions of specimens B1-B8 (in.)(*63,64*)

Specimen	Width Top	Width Bot.	t_f	t_w	Depth	Radius	Span	Brace Space
B1*	8.00	12.75	0.25	0.10	9.0	134	86	16.5
B2*	8.00	12.75	0.25	0.10	9.0	134	86	16.5
B3*	12.75	12.72	0.10	0.10	9.0	646	89	25.8
B4*	12.75	12.75	0.10	0.10	9.0	646	89	25.8
B5*	12.75	12.75	0.10	0.10	9.0	134	86	26.8
B6*	12.75	12.75	0.10	0.10	9.0	134	86	26.8
B7**	32.25	32.00	0.25	0.12	18.0	434	120/ 240	Vary
B8**	32.52	24.00	0.25	0.12	18.0	434	120/ 240	Vary

** Bottom flange 17 in. wide by 0.37 in. thick.*
*** Bottom flange is 38.25 in. wide by 0.25 in. thick.*

TABLE B-12 Test results: specimens B1-B6 (*63*)

Test	V/F	Calculated loads								Maximum in Test	
		Shear		Shear		Bending		Bending			
		P	H	P	H	P	H	P	H	P	H
B1-T1	0.0	47	-	**	-	38	-	39	-	*20	0
B1-T2	0.1	33	120	37	133	38	143	38	147	36	130
B2-T2	0.2	26	185	**	**	37	275	37	283	29	205
B2-T3	0.0	47	-	**	-	39	-	39	-	42	0
B3-T1	0.0	70	-	**	-	71	-	71	-	59	0
B4-T1	0.3	45	194	**	**	65	232	65	293	52	234
B5-T1	0.1	45	65	**	**	71	86	71	102	56	80
B6-T1	0.3	36	155	**	**	67	232	67	290	47	202
B6-T2	0.0	52	-	**	-	73	-	73	-	58	0

UNITS: P: kips; H: kip-in (equal extra torque in addition to simple support reaction)
** This test was to examine test arrangement*
*** Load greater than full plastic moment capacity*
V/F = shear stress / flexure stress

The bracing connections in the boxes are important. Inspection of a bridge on one interchange revealed numerous cracks in the box webs (*67*). Analytical examination of the structure indicated large internal bracing forces and inadequate bracing connections, resulting in fatigue cracks in the web that required expensive retrofitting.

TABLE B-13 Failure modes of specimens B1-B6 (*63*)

Test	Failure Mode
B1-T2	Outer web buckle near end
B2-T2	Outer web buckle near end
B2-T1	Flange buckle in test section
B3-T1	Local buckle between stiff. in test section
B4-T1	Inner web buckle in test section
B5-T1	Inner web buckle in test section
B6-T1	Outer web buckle near end
B6-T2	Local buckle between stiff. in test section

The provisions for curved box flanges in the Guide Spec are based on strength provisions for straight box girders in AASHTO. These AASHTO equations in the Guide Spec, which give critical stresses for compression and tension flanges, have been modified to reflect the effect of torsional shear stress. Box webs are designed as if they were I-girder webs except that vertical shear must be adjusted for web inclination.

The only test of a trough-type box girder with top flange bracing was conducted by Daniels et al. (*10,13*). Comparison of measured deformation with predicted rotations and deflections was not performed.

C. Model Bridge Tests

Culver et al., Beal, and Nakai have tested small model curved I-girder bridges, not included here. The Beal and Nakai tests were terminated due to excessive deflection rather than failure. A small Plexiglas model was used by Christiano to study dynamic effects of moving loads on I-girders (*68*). This model was not tested to failure.

D. Field Measurements

Field tests of bridges during construction have been carried out by Buchanan et al. (*69*), Douglas et al. (*70*), Beal et al. (*71–74*), Gambrell et al. (*75*), and Goebel (*76*). During the fall of 1973, the I-695 and I-83 interchange, C Viaduct, located near Baltimore, Maryland, was tested and analyzed for both dead and live loads during and after construction (*21,69*). The superstructure consists of two trough-girders, 4.5-ft deep by 11-ft wide. The composite deck is 10-in. thick. The bridge has three continuous spans of 100-133-122 ft with a centerline radius of approximately 1,318 ft. Measured stresses and deflections correlate fairly well with analytical values from a finite difference method, CURSYS (*50*). Numerous live-load tests have been carried out on this bridge. To date, there are no reports comparing modern finite element analysis to the test results for the Baltimore bridge, or any other curved bridge tested. Analysis of these data should reveal a great deal about the behavior of curved steel bridges at service stress levels.

E. Structural Analysis

There are two critical elements regarding structural analysis of curved bridges: the analysis of the curved beam member and the analysis of the entire structure. Earliest attempts to analyze a curved I-girder were performed without the aid of computers (*77,78*) and were directed toward proper analysis of a single girder. Parallel analyses of structural frames were available by using various techniques such as moment distribution and slope deflection (*77*).

Early recognized limitations of linear solutions for the single curved-girder problem were attacked with approximate computer methods such as the approach by McManus (*47*). Grid analyses of bridges were developed in Britain and Germany in the 1950s. These early grid solutions were directed more toward straight-girder bridges and did not recognize nonuniform torsion.

Approximate methods were developed that recognized both nonuniform torsion and interaction between curved girders. The V-load method (*1*), developed for I-girders, is theoretically pure with regard to torsion due to only curvature and load distribution for static equilibrium. The method works best for framing without a deck. These limitations are rather severe, but the method provides very adequate solutions in many cases. The V-load method does not account for lateral bracing between girders in the plane of the flanges. Accuracy of the method with regard to live load is dependent on the ability of the user to assign load to the girders prior to commencing the V-load analysis.

Box-girder bridges have been analyzed with an approximate method, called the M/R method (*79*). With this method, the torsion due to curvature is assigned to each girder. The assumption is made that the girder is free to rotate without restraint of the deck or cross frames attached between boxes. It is also assumed that the box retains its shape, and that no other torsion is applied to the girder. These are rather strict limitations that many bridges do not meet.

These approximate methods of analysis have been found to be reasonably accurate for noncomposite dead loads and are capable of even accounting for a limited amount of skew. There are many bridge analyses, however, where approximate methods do not provide adequate results.

Early computer analyses used a two-dimensional grid model of the superstructure. Usually, these models were not capable of considering warping. Nonuniform torsion often produced large lateral bending stresses that needed to be considered in the stiffness of the member. St. Venant shear was not adequate to distribute loads properly. Special elements have been developed for considering warping (*22*) by using strength of materials principles that assume the cross-section shape is retained and the structure behaves linearly. The accuracy of this assumption varies with the girder section, but it is generally not very accurate for box-girder bridges.

The finite strip method also has been developed for analysis of curved beams. The bridge is divided into a series of curved strips that extend across the bridge width. The strips are restrained according to stiffness of cross frames and deck such that the effect of cross bracing can be recognized (*80*).

The finite element method permits the structure to be divided into a discrete number of elements connected at nodes with freedom to translate or rotate (*81*). The solution for compatible motions at nodes is performed by using matrix algebra. Different levels of modeling and analysis are available.

The simplest finite element models are linear-elastic grid models. One shortcoming of this type of analysis is that it does not permit correct modeling of the deck and bearings, because they are not usually located at the neutral axis of the girders as assumed in many grid analyses. This type of analysis generally cannot recognize warping. Curvature is modeled with a series of straight elements with small kinks at each node. However, there are programs that do consider warping of beam members (*22,82*). Some of these programs can recognize nonuniform torsion such that curved elements can be used. These methods do not recognize cross-section deformation or amplification.

Another level of linear elastic finite element modeling is the three-dimensional method. This method permits beams to be modeled to reflect warping as well as distortion. The beam cross section is modeled with a series of shell elements. Cross frames and deck can be modeled. The method permits consideration of horizontal shear in the deck and eccentricity of bearings (BSDI 3D System) (*83*). The method does not consider amplification effects or nonlinear material behavior.

Finite element methods must have loads applied and boundary conditions defined by experienced users. Application of these methods is a complex art. Several software packages are available for use by the bridge designer. These packages often help the user to determine boundary conditions and loading.

More advanced general purpose finite element programs are available that consider large deflections and/or nonlinear material behavior but are not used, at the present time, for design.

F. Vibration

Vibration investigations of curved girders have been divided into two components: the determination of the natural mode of free vibration of a single girder and the study of the movement of live load across the bridge that causes a forced dynamic behavior (*68*). Dynamic analyses demonstrate an interaction between three factors in the determination of the first mode of vibration (*68*). The three terms are

$$I_p/AR^2, I_o/I_p, \text{ and } y_o r_y/R^2$$

where

- I_p = polar moment of inertia about the center of gravity,
- I_o = polar moment of inertia about the shear center,
- R = radius of bridge,
- A = cross-section area of bridge,
- y_o = distance between shear center and center of gravity, and
- r_y = radius of gyration of bridge cross section.

There are no simple means available for determining the natural frequency of a curved bridge (*84*).

Bearing lateral restraints may be arranged such that a curved bridge can develop a free mode of vibration in the lateral direction, acting as an arch. This mode has been found to have a frequency half as large as the second mode, which is usually in the vertical plane (*85*).

G. Impact

Impact is a term used to describe the dynamic contribution to the deflections and stresses. It results from the bridge responding to moving loads. If the load traverses the bridge smoothly, the magnitude of impact is related to the ratio, *T,* where *T* equals time for a live load to traverse a span/natural frequency of span (*68*). The ratio, live-load mass/bridge mass, is not significant in normal bridges (*68*). Sprung loads cause greater impact than unsprung loads. Roadway roughness also is a contributing factor to impact.

Analytical studies of curved I-section bridges indicate slightly greater impact factors than for similar straight bridges for bending (*68*). However, the dynamic component of nonuniform torsion in bridges has been reported to be disproportionately larger than the dynamic component of flexural responses (*68*).

Similar analyses of curved box bridges indicate that dynamic effects are greater than those of straight box bridges (*86*). This may be due to the larger torsional stiffness of box girders compared to I-girders. The larger dynamic effects are reflected in the Guide Spec (*9*) provisions.

H. Fatigue

Concern has been expressed regarding the bending stresses due to web bowing at panel boundaries defined at flanges and stiffeners (*8,10,87*). Fatigue tests and measured deformations have been carried out on laboratory I- and box specimens (*11,87*). The I-girder tests involved a two-girder frame having a 40-ft span and 120-ft radius. The doubly symmetric girders were spaced at 5 ft. One frame had two girders with the same size flanges. In the other frame, the outside girder had larger flanges than the inside girder. For example, the outside girder of one frame had 10 × ¾-in. flanges, while the inside girder had 8 × ½-in. flanges. The girders were approximately 5-ft deep. Web slenderness varied from 139 to 186. Cross frames were spaced at 10 ft. Loads were applied between the two girders at approximately quarter-points of the span over the cross frames. There were five such frames built and fatigue tested.

Three single box-girder specimens were also tested (*10,87*). Each box was approximately 36-in. deep by 36-in. wide, with vertical webs. The bottom flange had longitudinal stiffeners. These specimens each had a 60-in.-wide stiffened top plate that provided closure of the sections and represented a concrete deck. Spans were 36 ft with a radius of 120 ft. Box specimens were loaded over the interior web at quarter points. Interior bracing was placed at 9-ft intervals.

Analysis of web panels using a double shell-technique by Brogan (*58*) and Culver et al. (*59*) indicated that a reduced slenderness would account for the effect of curvature on lateral web bending stresses such that the through-thickness bending stress in the curved web would roughly equal those in a straight-girder web with the maximum permitted AWS out-of-flatness. Daniels et al. presented a simpler formula based on similar work and the experimental results (*15*). Stiffener spacing was limited to the web depth in both the Guide Spec and Hanshin.

Results from these fatigue tests (*15*) indicated that the AASHTO fatigue provisions for straight girders could be applied to curved girders for the details examined.

There have been reported fatigue cracks of box girders (*67*). Retrofit procedures were developed and implemented.

III. DESIGN SPECIFICATIONS

A. Background of AASHTO Design Specifications

Design specifications are developed both from theoretical research and from experience. To better understand the genesis of the Guide Spec, it is necessary to examine the relevant history of the AASHTO *Standard Specifications for Highway Bridges* (*41*).

Until the early 1960s, AASHO dealt with only noncomposite stringer bridges. Steel stringer sections were nearly symmetrical with respect to the transverse axis through the web so the neutral axis and shear center were coincident and at middepth. All vertical loads were assumed to be resisted by only the steel section. Allowable stress design (ASD) was used. The permitted bending stress was 55 percent of the minimum specified yield stress, which was 33 ksi. Design live load was HS20-44. As it had been since the 1950s, the portion of a wheel line assigned to each stringer was determined by using S/5.5 as a wheel load distribution factor. Live-load deflection, based on the stiffness of the steel section only, was limited to span/800, assuming uniform participation of all stringers. Fatigue was treated with a provision that limited alternating stress derived from riveted joint fatigue tests.

The vast majority of bridges were simple spans. The stringer spacing of multistringer bridges was rarely over 9 ft and was more commonly between 7 and 8 ft. Rolled beams were used on spans up to 90 ft. Riveted girders with stringer-and-floor-beam arrangements were commonly used in longer spans. The minimum girder depth was specified as span/25 based on economics and, perhaps, to control deflections.

In the 1960s, composite design and welding were introduced into the AASHTO bridge design specifications. The effective width of deck was set at 12 times the deck thickness. This rule permitted inclusion of the entire deck of most bridges, because the deck thickness was usually 8 in. and the stringer spacing was 8 ft or less. The common steel grade, ASTM A36, had an enhanced minimum specified yield stress of 36 ksi. A more comprehensive fatigue specification based on American Welding Society (AWS) provisions was provided. The maximum web slenderness permitted without transverse stiffeners was increased from 50 to 150, with provisions limiting the shear stress. The span-to-depth limit was modified to permit span/30 for the steel section and span/25 for the composite section.

Continuous spans became prevalent but the stringer spacing generally did not exceed 9 ft. Methods of assignment of live load, limits on live-load deflection, and girder depth remained essentially unchanged, although a deflection limit of span/1,000 was introduced for bridges carrying pedestrians. Higher strength steels tend to lead toward greater deflections because less steel is required. Recognition of composite action also leads to a further reduction in steel section. All of these changes led inevitably toward increased dead- and live-load deflections.

There was no requirement to evaluate the noncomposite section for lateral torsional buckling or web bend-buckling prior to curing of the deck. The full post-buckling strength of the web was permitted in dead-load shear. Longitudinal stiffeners were permitted, but they were placed as though the section was doubly symmetric, and a check for the noncomposite condition was not required.

In the late 1960s, load factor design (LFD) was introduced. Girder stresses were factored and checked against an ultimate stress state. Girder strength was presented in terms of moment capacity. With LFD, loading assumptions (including wheel load distribution factors), live-load deflection limits, and span-to-depth limits remained the same. The common load condition for girder moments is: $1.3DL + 2.2(LL + I)$. A check for overload was also required where the stress due to the load condition, $DL + 1.67(LL + I)$, must be less than $0.95F_y$ (composite) or $0.80F_y$ (noncomposite). For spans greater than 40 ft, LFD provided economy by reducing girder weight compared to ASD.

The tension field concept was introduced into LFD, but not into ASD. With tension field, maximum transverse stiffener spacing was increased from D to $1.5D$. The maximum web slenderness permitted with transverse stiffeners was increased from 150 to 165.

Fatigue provisions became more stringent after the observation of fatigue cracks around welded details. Although quenched and tempered steels were permitted since the early 1960s, price limited their use to only a few applications.

Introduction of the curved-girder Guide Spec was the most important change in the late 1970s as far as design of steel bridges was concerned. The research leading to these guidelines was performed referencing the AASHTO straight-girder provisions. Because LFD was not widely accepted at the time, the Guide Spec was developed for both box and I-girders in ASD format.

The Guide Spec presented several major departures from the classical assumptions in AASHTO. For the first time in the United States, an analysis of the entire superstructure was required by specification. New types of analysis tools were required. Although this approach always had been used in Europe, it had never migrated to North America where wheel load distribution factors had reigned supreme. To circumvent the requirement of full superstructure analysis, approximate methods were developed that permitted use of a line-girder analysis with modifications.

Strength of curved I-sections was examined and presented in the Guide Spec as a modification of AASHTO's lateral buckling equation for partially braced sections. The modifications were in terms of ρ factors used to determine the moment at first yielding, considering vertical bending, warping, and lateral bending of the flanges. Only noncompact sections were considered.

The strength of the bottom flange of boxes was based on classical buckling equations in AASHTO for straight boxes. It was recognized that torsional shear would be important in the curved box elements; therefore, the bottom flange strength equations were modified to account for torsional shear in combination with tension and compression.

Web design of box and I-girders was based on elastic buckling of straight girders as done in ASD at the time. In fact, the Guide Spec referenced ASD provisions for web design.

Other requirements, such as safety factors, fatigue design, live-load deflection, and girder depth-to-span limits, were not specified, but referenced the AASHTO ASD provisions for straight girders. No investigation appears to have been made regarding the applicability of these requirements to curved girders.

Shortly after introduction of the Guide Spec ASD provisions, American Iron and Steel Institute (AISI) provided an LFD version that also was based on the CURT work (*16*). The major change between the LFD and ASD versions was that the plastic strength of I-girders was recognized when the flanges were compact. No redistribution of load was permitted with plastification. Again, the same load combinations and factors as used for straight girders were used. Unlike LFD for straight girders, webs were not designed considering tension field action. The maximum transverse stiffener spacing was equal to D as in AASHTO ASD at that time.

Later modifications to the ASD of straight girders included addition of tension field design for webs. The maximum web slenderness for transversely stiffened webs was increased to 192, and the maximum spacing of transverse stiffeners was increased to $3D$. Also, these slender webs could be unstiffened when the shear stress was below the elastic buckling stress with a factor of safety. Inadvertently, these changes were applied to curved-girder webs because of the Guide Spec referencing of AASHTO for web design. This oversight was later corrected in the Guide Spec by reinstating elastic design and limiting transverse stiffener spacing to D.

Other significant changes were made in AASHTO straight-girder design. Fatigue provisions again were made more critical with the addition of Category E and lower stress ranges in some categories. The case for "over 2 million cycles" (fatigue limit) was added for a single vehicle loading. The lateral buckling equation was modified to conform to American Institute of Steel Construction (AISC) provisions. The AISC equation is much more refined in that it recognizes behavior of the entire section rather than only the compression flange. The equation also includes an R_b factor that recognizes load shedding from the web to the compression flange and a C_b factor that recognizes moment gradient between brace points. The curved-girder provisions explicitly were based on assuming no load shedding, constant moment between brace points, and no load redistribution.

In the early 1980s, other changes, in addition to those in the specifications, occurred. Changes in framing had serious consequences. Typical girder spacing increased from 9 ft to as much as 15 ft, with the advent of new steel two-part deck forms that could span widths up to 15 ft with concomitant increases in deck thickness. These changes resulted in much larger loads being placed on the noncomposite girder and less than all of the deck cross section considered effective in composite action. More skewed bridges were being built to meet alignment requirements. Refined analysis techniques, emanating from curved-girder design and higher strength steels led to lighter steel girders with even higher stresses, particularly on the noncomposite section. This situation was recognized, and a constructibility check was added. The check included only lateral buckling of the top flange of straight I-girders for the noncomposite case. Web bend-buckling check of the noncomposite girder was not required, although the neutral axis of the noncomposite girder in positive moment regions could have been far below middepth. The shear capacity of the girder under noncomposite dead load is limited to the elastic buckling capacity in the AASHTO *Standard Specifications for Highway Bridges, 15th Edition* (*41*). Some constructibility requirements have been added to AASHTO in Article 10.50 but are not referenced in the Guide Spec. Some states have developed their own provisions for noncomposite curved girders prior to the deck curing (*88*).

Constructibility problems associated with curved girders are particularly onerous because no research has been performed on unsymmetrical (singly symmetric) I-girders. Although the Guide Spec permits consideration of the strength of compression and tension flanges separately, the research of curved sections considered only doubly symmetric girders with equal first-order compression and tension stresses.

The effective width limit of 12 times the deck thickness does not recognize a large portion of deck between widely spaced girders that refined analysis indicates is effective. While ignoring this portion of the deck is conservative for bending stresses, it is not conservative when computing the first moment of the deck with regard to the shear connectors. Because fatigue controls most shear connector designs, problems would not be expected for some time, if ever.

As the girders become more flexible, relative deflections between adjacent girders tend to increase for both dead and live load. Considering the girders are actually connected by the deck and cross frames, forces in components needed for restoring compatibility between the girders increase. This is particularly true in curved girders, where equilibrium requires interaction with adjacent girders through the cross frames and deck. Assumptions of uniform loading and equal live-load deflections certainly are not correct for curved girders. Uniform distribution of superimposed dead load also is not appropriate for curved girders and is questionable for straight girders. Girder depth limits, based on yield stress of

33 ksi noncomposite girders, may need to be examined before they are applied to composite curved girders of higher strength steels.

With this background, a review of the current Guide Spec provisions is presented.

B. Part I: AASHTO Guide Specifications: Allowable Stress Design Criteria

B.1 Curved Steel, Composite, and Hybrid I-Girder Bridges

1.1 General. The provisions cover design of superstructures with horizontally curved girders. They refer to the AASHTO *Standard Specifications for Highway Bridges,* Divisions I and II, which apply except where they are overridden by the provisions of the Guide Spec. (Guide Spec articles are in italic and AASHTO provisions are in bold type.)

1.3 Loads. Loads are essentially as prescribed in AASHTO. Specific reference is made to uplift, which is not permitted during placement of the concrete deck. Impact is as specified in **Article 3.8.2.** Centrifugal forces are specified in **Article 3.10.** The superstructure need not be designed for thermal forces; however, thermal movements as specified in *Articles 1.6 and 1.7* are permitted. This movement is presumed to occur without resistance.

1.4 Design Theory. This article covers analysis rather than design of the structure. A rational analysis of the entire superstructure is required. The analysis must consider the complete distribution of loads to the various members.

When the subtended angle is less than a specified value in *Table 1.4A,* the effects of curvature on the primary bending moments in the girders may be ignored. *Table 1.4A,* which is based on analysis of a number of bridges (*89*), differentiates between number of girders and number of spans as well as subtended angle. Other effects of curvature, including nonuniform torsion and cross frame forces, must be considered in all curved bridges, regardless of the subtended angle.

Consideration of torsion is required. Intermediate cross frames and diaphragms are assumed to distribute the internal torsion between individual girders. Nonuniform torsion, which causes lateral flange bending, must be considered by a rational method in all curved bridges.

The actual method of consideration of torsion in the analysis is not discussed. There are essentially three categories of analysis methods that are acceptable:

- The strength of materials method in which the girder cross sections are assumed not to deform and warping stresses may be considered (grid, two-dimensional finite element methods);
- Methods that consider the girder cross section to deform such that the flanges resist the total radial component of force (V-load method); and
- Methods that consider both warping and radial distortion (three-dimensional finite element methods).

The first method does not give a lateral flange moment directly from which a lateral flange bending stress can be computed. The second method recognizes the radial component of nonuniform torsion. If other torsional loads are applied to the girder, they may not be recognized by the second method.

Distribution factors based on the arrangement of bottom-flange bracing members is provided. These factors were developed empirically, assuming that bottom-flange bracing will be used and that a grid analysis will be performed. These provisions are stated as applicable to either right or skewed support bridges with spans between 80 and 300 ft, and with radius greater than 300 ft. Bridges with up to six girders in the cross section are included.

1.5 Fatigue. This article requires that lateral flange-bending stresses due to nonuniform torsion be considered, in addition to flexural stresses. Special mention is made of the need to accurately determine stress magnitudes at connections.

1.6 Expansion and Contraction. Movement due to temperature is to be considered radiating from a fixed support; therefore, movement permitted by bearings is along rays from a fixed point rather than along girder lines.

1.7 Bearings. Bearings must be designed to resist horizontal as well as vertical loads and permit lateral movement. Lateral loads are not specified, but it is assumed that the AASHTO provisions apply. Each bearing must also permit rotation about an axis longitudinal with the girder. If uplift occurs, the bearing must be designed to resist it.

1.8 Diaphragm, Cross Frames, and Lateral Bracing. The requirements in **Article 10.20** apply. Lines of cross frames must extend across the bridge in an uninterrupted line. They shall be as nearly full-depth of the girders as practical. Where feasible, gusset plates shall connect flanges and cross frames to transfer lateral load directly. In practice, most curved bridges have cross frames or diaphragms that are less than full depth of the girders, and force must travel from these members through the gussets and web to the girder flanges. Cross frames need not be used along skewed piers.

A method for computing cross frame forces is not specified. Dabrowski states that warping deformation of the girders, as well as deformation of the cross braces, should be considered in the analysis if cross frame forces are to be computed correctly (*6*).

Bottom flange lateral bracing need not be used. When it is used, an approximate formula to compute stresses in these members is provided.

B.2 *Curved Steel I-Girders*

1.9 General. The normal stress due to bending should be computed using the moment-of-inertia method. Nonuniform torsion is to be dealt with by using a rational method of analysis. Special mention is made of combined uniform bending and nonuniform bending stresses for fatigue.

1.10 Allowable Normal Flange Stress. Only noncompact sections are considered. Flange width-to-thickness is limited to $4,400/\sqrt{F_y}$. Although not stated, the flange tip stress is limited to first yield. It is limited by the average flange stress by using the old AASHTO equation for lateral torsional buckling multiplied by ρ_w and ρ_b The term, ρ_w reflects the effects of warping stress and curvature and ρ_b reflects the vertical bending stress and curvature. There are two equations for ρ_w: one applies when there is compression on the outer flange tip at brace points and the other applies when there is compression on the inner flange tip at brace points.

The ρ terms are a function of l/b_f, l/R, and f_w/f_b where l equals the arc length between flange brace points, R equals the girder radius, f_w equals the largest warping stress of the two brace points, and f_b equals the largest bending stress of the two brace points. The average stress is limited at the brace point having maximum moment.

The flange stress may be a maximum at a flange tip, either at the brace point or at midspan between brace points. The ρ factors deal with both points, although the warping stress need be computed only at brace points. To accomplish this, the unbraced section between brace points is assumed to be subjected to uniform moment and to be prismatic.

The research leading to the ρ factors was based on the study of sections that were doubly symmetric. Therefore, the bimoments and stresses are theoretically equal but opposite in the two flanges. The total computed flange tip stress at brace points, $f_b + f_w$, is limited to $0.55F_y$. This limit actually is not necessary for symmetrical sections with the ρ equations. The original work does not include this limit (*47*).

Although the equations may be applied to singly symmetric sections, there is no theoretical evidence that this is appropriate. AASHTO requires that the flange width, b_f, be reduced to $0.9b_f$ when the girder is singly symmetric. This reduction should be used in determining λ but not used in the computation of ρ_w, ρ_b, and f_w.

The unbraced length between torsional brace points may not be greater than 25 ft, 25 times the compression flange width, or 0.10 times the girder radius. The absolute value of the ratio, f_w/f_b, is limited to 0.5. Unfortunately, the provisions imply that this ratio remain inviolate at any stress level; whereas, it was intended only to apply to the fully stressed condition. The vertical bending stress may not be greater than that allowed for a straight girder with the same unbraced length and size of flange.

1.12 Thickness of Web Plate. (A) Girders Not Stiffened Longitudinally. Slenderness of webs without longitudinal stiffeners is limited by Equation (B-9) but may not be greater than 170. There is an additional limitation given in **Article 10.34.3.1.2,** based on the minimum specified yield stress in the compression flange. Equation (B-9) was developed by Daniels et al. (*15*) on the basis of fatigue tests and an extension of Culver's earlier analytical work (*60*):

$$\frac{D}{t_w} \leq \frac{23,000}{\sqrt{f_b}}\left[1.0 - 4\left(\frac{d_o}{R}\right)\right] \tag{B-9}$$

Culver originally had proposed the following equation (*8*):

$$\frac{D}{t_w} \leq \frac{23,000}{\sqrt{f_b}}\left[1.19 - 10\left(\frac{d_o}{R}\right) + 34\left(\frac{d_o}{R}\right)^2\right] \tag{B-10}$$

$$\frac{D}{t_w} \leq \frac{23,000}{\sqrt{f_b}} \quad \text{when} \quad \frac{d_o}{R} < 0.02 \tag{B-11}$$

Figure B-13 is a plot comparing the curvature factors of the two equations versus stiffener spacing, d_o. The factor, or the quantity inside the square bracket of Equation (B-10), is limited to 1.0 by Culver. The first part of the equations, $23,000/\sqrt{f_b}$, is the same as used for straight girders. The constant is based on web bend-buckling limits of a doubly symmetric girder.

Equation (B-10) was developed by Culver et al. using an arched-plate analytical model to study through-thickness stresses (*90*). He computed through-thickness stresses in a straight-girder web, assuming a maximum AWS (*91*) out-of-flatness. The curved webs have a slenderness that gives the same through-thickness bending stress. The aspect ratio was limited in the study to 1.0, which was the AASHTO aspect ratio limit at the time this work was performed. Only doubly symmetric girders with the web area equal to the total flange area were investigated. The through-thickness stress was found to be approximately $0.2F_{yw}$ for a straight-girder web. Web slenderness limits for straight girders were reported to be based, in part, on experimental results from earlier fatigue tests at Lehigh University (*15*).

The AASHTO provisions permit elimination of transverse stiffeners, if the shear stress is within allowable limits for straight girders. Culver et al. did not analyze unstiffened webs or perform any tests of slender unstiffened webs. The maximum aspect ratio Culver tested was 1.67 (*56–58*). Daniels tested full-size curved girders with aspect ratios up to 2.27 (*10*).

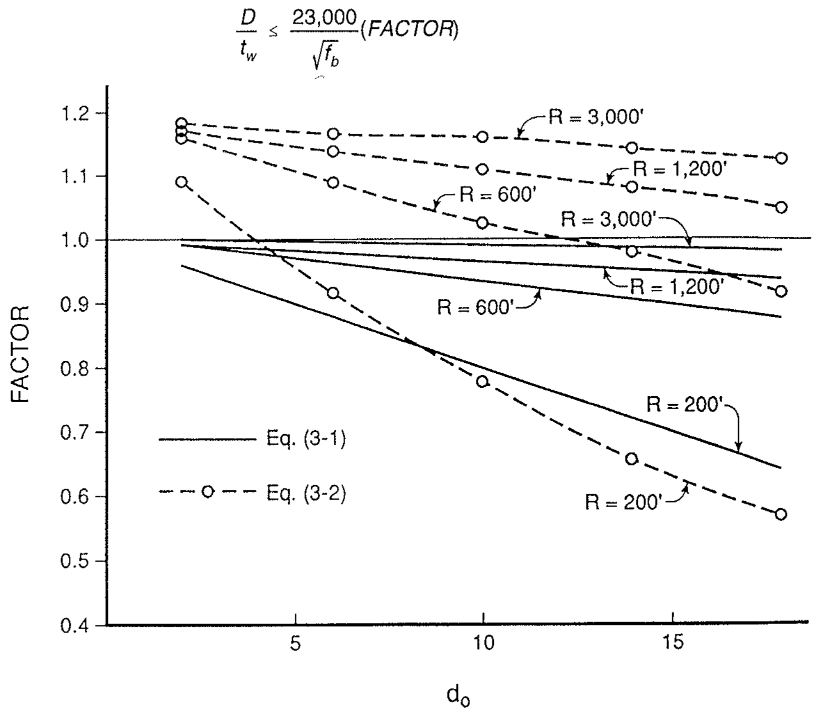


Figure B-13. Curvature factor versus d_o.

(B) Girders Stiffened Longitudinally. Webs with longitudinal stiffeners have slenderness limited by the following equation:

$$\frac{D}{t_w} \leq \frac{46{,}000}{\sqrt{f_b}}\left(1.0 - 2.9\sqrt{\frac{d_o}{R}} + 2.2\frac{d_o}{R}\right) \qquad \text{(B-12)}$$

Equation (B-12) was developed by Culver et al. (*60*) by using an analytical model similar to that used for webs with transverse stiffeners only. However, the model for longitudinally stiffened webs considered bending in both directions rather than only in a strip of web, as used for the transversely stiffened web. The longitudinally stiffened web investigation is presented in an addendum to Culver et al. (*60*). Again, the investigation was limited to doubly symmetric girders.

A single longitudinal stiffener should be placed at a distance $D/5$ from the compression flange. The web slenderness may not exceed $D/340$. Transverse stiffeners may not be spaced greater than D.

If two longitudinal stiffeners are used, the slenderness can be set at the maximum given in **Article 10.34.3.2.2,** with the design stress equal to the allowable normal flange stress. The function of the second longitudinal stiffener is to control through-thickness bending stresses in the tension side of the web; its use is unrelated to web bend-buckling. The second longitudinal stiffener should be located at $D/5$ from the tension flange.

In most cases, the girders are composite with loads placed both on the noncomposite section and on the composite section. This two-phase loading causes two potentially critical cases. On long spans where the dead load is large, the noncomposite girder may have more than half of the web in compression in positive moment regions. This situation may cause excessive bowing of the web, as has been reported in straight girders. There is no check required for the noncomposite loading case. When the girder becomes composite, the neutral axis shifts upward and another case must be checked.

1.13 Transverse Intermediate Stiffeners. Stiffener spacing is specified to permit the elastic-buckling capacity of the web to carry all shear. Transverse stiffeners are not required when the shear stress is less than that given by Equation (B-13) in **Article 10.34.4.1:**

$$F_v = \frac{7.33 \times 10^7}{(D/t_w)^2} \leq \frac{F_y}{3} \qquad \text{(B-13)}$$

Equation (B-13) gives the allowable shear stress with a factor of safety of 1.8 against elastic shear buckling for a flat plate loaded in pure shear, assuming simply supported edges at the flanges. When the curved-girder provisions were written, a similar equation gave 30 percent lower critical shear stress. The researchers believe that the older equation had been modified to account for the occurrence of coincident bending, because the provisions often are applied in negative bending regions where both bending and shear stresses are large. No evidence has been found that this situation was addressed in the development of the Guide Spec.

Culver's research of the bending stress in transverse stiffeners indicates that additional bending stresses in them because of the curvature of the web are small. However, rigidity of the stiffeners is increased compared to that specified for straight girders, when the transverse stiffener spacing is greater than 0.78 times the web depth (*92*). The required increase in rigidity of the stiffeners is implemented by including the curvature correction factor, X, to J in **Eq. (10-30)** in **Article 10.34.4.7.** The equation in the Guide Spec has a denominator equal to 10.92, which is effectively 10.0 in **Eq. (10-30).** Thus, the required moment of inertia for the transverse stiffeners in the Guide Spec is approximately 10 percent less than in AASHTO.

Hanshin (*38*) adopts a similar approach in stipulating the required increase in the rigidity of the stiffeners compared to those for straight girders. For double-sided stiffeners, the following equations are used:

$$\gamma_c \geq \gamma_s\{1.0 + (a - 0.69)Z[9.38a - 7.67 - (1.49a - 1.78)Z]\}, \quad (0.69 < a \leq 1.0) \qquad \text{(B-14)}$$

$$\gamma_c \geq \gamma_s, \quad (a \leq 0.69) \qquad \text{(B-15)}$$

For single-sided stiffeners, the following equations are used:

$$\gamma_c \geq \gamma_s\{1.0 + (a - 0.65)Z[12.67a - 10.42 - (1.99a - 2.49)Z]\}, \quad (0.65 < a \leq 1.0) \qquad \text{(B-16)}$$

$$\gamma_c \geq \gamma_s, \quad (a \leq 0.65) \qquad \text{(B-17)}$$

where

γ_c = required rigidity ratio of stiffeners in curved girders;
γ_s = required rigidity ratio of stiffeners in straight girders;
a = aspect ratio of web panels, d_o/D;
$Z = d_o^2/Rt_w\sqrt{1-\mu^2}$; and
μ = Poisson's ratio.

According to the Japan Road Association's *Specifications for Highway Bridges* (*39*), the web slenderness can be increased up to 20 percent in proportion to the square root of the ratio of the critical bending stress/computed bending stress for straight girders.

1.14 Longitudinal Stiffeners. Although Culver discusses longitudinal stiffeners on curved-girder webs (*8*), there was no consideration of these slender webs in conjunction with development of the ρ factors. The use of the ρ factors with longitudinally stiffened girders is perplexing.

The moment of inertia requirement for longitudinal stiffeners is the same as for straight girders. There is an additional requirement, however, placed on the radius of gyration of the longitudinal stiffener that is a function of the yield stress of the web and the transverse stiffener spacing. A portion of the web equal to 18 times its thickness should be used with the longitudinal stiffener in determining its radius of gyration, although a like amount of web is not used to determine the moment of inertia of the longitudinal stiffener according to AASHTO. The logic for the radius of gyration provisions, in addition to the moment of inertia, in the Guide Spec is not apparent.

B.3 Curved Composite I-Girders

1.15 General. This paragraph directs that the composite requirements of **Article 10.38.1** apply.

1.16 Effective Flange Width. The effective deck width used in straight girders applies to computations of bending and twisting moments and deformations. This effective width is generally 12 times the minimum slab thickness, according to **Article 10.38.3.**

1.17 Noncomposite Dead-Load Stresses. The vertical and torsional bending stresses are not to exceed the provisions of *Article 1.10,* which limit allowable normal flange stresses to first yield. The commentary recommends checking the girder for noncomposite loads including the deck casting sequence.

1.18 Composite Section Stresses. Lateral bending stress in the top flange, due to loads applied after the concrete has cured, may be ignored. Although somewhat ambiguous, the total stress due to vertical bending in the encased flange must be less than $0.55F_{yf}$.

Flanges not encased in concrete are designed partially braced with warping, due to all loads, considered. Culver discusses composite action of curved girders (*8*), but there is no indication that the application of the ρ values to these members was considered in the research.

Design for shear is the same as for noncomposite girders with all shear resisted by the web.

1.19 Shear Connectors. (A) Fatigue. **Article 10.38.5.1.1** should be used to design shear connectors for fatigue. There is no requirement to consider torsion. The commentary demonstrates that, for practical designs, only the vertical shear should be considered for fatigue.

(B) Ultimate Strength. Shear connectors should be designed according to **Article 10.38.2,** except the required capacity of the shear connectors shall be determined according to *Article 1.19.* The noncollinearity due to curvature must be considered when determining the ultimate strength requirements of the shear connectors. This is done by vectorially adding the bending shear and radial force due to the noncollinear component computed from the angle between points of maximum moment and zero moment—either the end of the bridge or point of dead-load contraflexure.

B.4 Curved Hybrid Girders

1.20 General. The hybrid girder concept was developed when there was a larger difference in price between grades of bridge steels than exists today. The price differential between ASTM A36 and A572 Gr50 is only about 3 cents per pound, which does not justify hybrid design.

Girder sections must have a vertical axis of symmetry through the web. Other requirements are given in **Article 10.40.1.** The provisions of *Article 1.10* limit the elastic stresses in the flange to $0.55F_{yf}$. This article permits yielding of the flanges and a portion of web when the girders are hybrid. An investigation of curved hybrid girders was performed by Culver (*8*). This work was based on the design of hybrid girders for first yield, which is similar to that for straight girders except that lateral flange bending is considered. If the lateral flange bending is large enough, such that the flange tip reaches yield prior to the lower strength web reaching yield, then hybrid has no effect.

B.5 Curved Composite Box Girders

1.24 General. Reference is made to the straight box-girder provisions in **Article 10.39.2** and *Articles 1.1* through *1.7* for curved I-girder bridges.

1.25 Loads. Loads as defined in **Section 3** are to be used except where modified.

(B) Impact. This section recently was modified based on work by Schelling et al. (*86*). Separate functions for impact are provided for the following actions: primary bending moment, torsion, shear, reactions, and deflections. Within each category, impact factors are further refined by span and radius. The article requires a dynamic analysis for structures outside the limits defined that includes spans greater than 200 ft, and radii less than 160 ft or greater than 800 ft. There must be from two to four boxes; the vehicle speed must be less than 70 mph; the number of continuous spans must be less than four; and the vehicle weight must be less than 0.30 times the bridge weight.

1.26 Design Theory. (A) General. This article is the same as *Article 1.1.* A rational analysis of the entire superstructure is required.

(B) Dead-Load Analysis and Design. The box must be checked for noncomposite loads prior to curing of the deck concrete.

(C) Torsion. Diaphragms are required between individual box girders, unless analysis indicates such members are not required.

(D) Warping Stresses. Warping normal stresses are of particular concern for fatigue but can be ignored in strength considerations if analysis shows them to be small. The commentary to this article also discusses transverse bending stresses in the webs with regard to fatigue. Several methods of analysis are mentioned.

1.27 Design of Web Plates. (A) Design Shear. The web must resist both vertical and torsional shear. Shear in inclined webs must be adjusted for the slope. The shear in the webs differs by the torsional component. However, transverse stiffeners are usually designed for only the critical web.

Unstiffened webs are permitted if the web shear stress is less than that determined in **Article 10.34.4.1.** However, no research on unstiffened curved box webs was found.

(B) Secondary Bending Stresses. Through-thickness bending stresses in the webs are to be limited to 20 ksi. If these stresses are due to live load, the web plates should be welded to the flanges with either full-penetration or double-sided fillet welds.

1.28 Design of Bottom Flange Plates. (A) Tension Flanges. The tensile stress is limited by the von Mises yield criterion, which reduces to the normal design stress of $0.55F_y$ when the shear stress is zero. The mathematical model used to develop the equations assumes a constant bending stress and a constant shear stress in the flange. The plate is assumed to be either completely elastic or completely plastic.

(B) Compression Flanges Unstiffened. There are three regions of plate slenderness. The plastic region includes flange plates with $b/t \leq 6{,}140/\sqrt{F_{yf}}\ X$, where $X = 1 + [\,4/3\ (\,f/\sqrt{F_{yf}} - 0.15)] \geq 1$. The allowable stress is based on the von Mises yield criterion. The term, X, is a factor that permits an increase in the slenderness prior to incipient buckling strength due to the relative shear stress and yield stress. The allowable stress is F_{yf} times Δ, where Δ reduces the allowable normal stress based on the von Mises yield criterion, considering the torsional shear stress.

The inelastic region includes plates with $b/t \leq 13{,}300/\sqrt{F_{yf}}$. In the inelastic region, the permissible bending stress is based on an assumed sinusoidal variation of stress between plastic and elastic values. Again, this variation differs from that for straight box-girder flanges because torsional shear stress is considered.

The elastic region for unstiffened flanges is defined as that with slenderness, b/t, greater than $13{,}300/\sqrt{F_{yf}}$. The allowable stress for these flanges is simply the elastic buckling stress with a buckling coefficient of 4.0 divided by the factor of safety. Again, the value is reduced according to the von Mises yield criterion. Flange slenderness greater than 60 should be avoided, and a longitudinal stiffener is recommended for $b/t > 45$.

There is no discussion concerning coincident torsional shear and bending stresses. It is implied that torsional stress is coincident with normal stress. The engineer can define the normal stress as either the average normal flange stress or the normal bending stress plus normal warping stress. Frequently, the maximum torsion is not produced by the same live-load condition as the one that produces the maximum bending stress.

(C) Compression Flanges Stiffened Longitudinally. Longitudinally stiffened flanges are treated similarly to unstiffened flanges, except that the buckling coefficient, k, may be between 2 and 4. A value of 4 for k assumes a buckling nodal line at each longitudinal stiffener and web. It yields the highest plate buckling stress for a given number of equally spaced longitudinal stiffeners and it requires the most rigid longitudinal stiffener.

(D) Compression Flange Stiffeners, General. Reference is made to the straight-girder provisions, **Article 10.39.4.5.**

1.29 Cross Frames, Diaphragms, and Lateral Bracing. (A) General. Cross frames and diaphragms help retain the shape of the box section when subjected to torsion. Connection of gusset plates to the flanges should be made to reduce through-thickness bending in the web. Gusset plates are to be coped where they intersect longitudinal welds.

(B) Top Lateral Bracing. Approximate equations are given for required bracing area and stiffness.

(C) Intermediate Cross Frames. Spacing of interior cross frames is based on radius and span but limited to 25 ft.

(D) Torsional Stress. An approximate equation is provided to determine when torsional stresses may be ignored. When the factor, $\Psi = L\ \sqrt{GK_T\ /\ EI_\omega}$, is greater than 0.4, stresses due to pure torsion may be ignored. When Ψ is greater than 10.0, warping torsion may be ignored. Warping stresses should be evaluated in essentially all multibox-girder bridges.

C. PART II: AASHTO Guide Specifications: Load Factor Design Criteria

C.1 Composite and Hybrid I-Girder Bridges

2.1 General. This section applies to rolled and welded I-girders. The LFD provisions of AASHTO, as well as Division II, apply. The term "moderate length" is used to limit applicability of the provisions. It is the researchers' opinion that "moderate length" does not apply to 500-ft spans as specified in AASHTO. However, there is no evidence that the term has been defined based on investigations of experience or analytical work.

The scope of the LFD provisions is the same as for the ASD provisions. Again, there are copious references to AASHTO.

2.2 Notation. A rather complete set of notation is presented, including some references to ice and earthquake, although these loads are not defined in the Guide Spec.

2.3 Loads. Loads are to be taken from **Article 10.43** and *Article 2.4* in the Guide Spec.

(A) Uplift. Uplift is to be checked for live load with no reduction for multiple presence. Uplift during construction also is to be avoided.

(B) Impact. Reference is made to **Article 3.8.**

(C) Superelevation and Centrifugal Forces. The distribution of live load to the wheels is to be determined according to **Article 3.10,** which considers curvature, superelevation, and vehicle speed.

(D) Thermal Forces. No allowance for thermal forces in the superstructure needs to be considered when thermal movements are allowed to occur. If movement is restrained, resulting forces need to be addressed.

2.4 Load Combinations and Load Factors. (A) Construction Loads. Stresses due to partial dead load and temporary construction vehicles should be factored by 1.3 and compared to the maximum design stresses permitted. Checks for stability or deflection are not required. Elastic analysis is required; however, there is no stipulation whether to use the first yield criterion or the plastic criterion.

(B) Service Loads. Unfactored dead and live load plus impact and centrifugal forces should be included when checking for fatigue and live-load deflection. No live-load deflection limits are provided.

(C) Overload. The overload condition differs from AASHTO in that centrifugal forces are included explicitly in *Group IA. Group II and Group III* load combinations are analogous to those in AASHTO.

(D) Maximum Design Loads. Elastic analysis is specified again in this section. *Group I* loading is: $DL + 5/3LL_T$, where LL_T is live load due to the heaviest expected legal vehicle, plus impact and centrifugal force. *Group I* loads are essentially the same as AASHTO LFD loads. A load condition for uplift, $0.90DL + 5/3LL_T$ live load positioned to cause maximum uplift, is required.

2.5 Design Theory. (A) General. A rational analysis that considers complete load distribution to all members is mandatory. This article is similar to *Article 1.4 Design Theory* for ASD, except that paragraph, *(F) Overload,* was added.

2.6 Fatigue. Similar to ASD.

2.7 Expansion and Contraction. Similar to ASD.

2.8 Bearings. Similar to ASD.

2.9 Diaphragms, Cross Frames, and Lateral Bracing. Similar to ASD.

C.2 Curved Steel I-Girders

2.10 General. Same as ASD.

2.11 Straight Elements, Splices, and Connections. This article requires that straight elements and other elements not affected by curvature, such as cross frames and bolted connections, be designed by using **Articles 10.48** through **10.56.** A check must be made for straight tension members, limiting the factored load to 0.85 times the ultimate tensile strength of the member.

2.12 Noncomposite Girder Design. (A) Limits of Applicability. These limits are similar to those for ASD. However, the first limit requires that the ratio, f_w/f_b, not exceed 0.5 at any point. The analogous requirement under ASD requires that this check be made only at brace points. The ASD requirement is consistent with the research, whereas this requirement seems to be inconsistent with the research.

By not limiting points to be investigated to points of maximum moment, locations with small bending stress must meet the required ratio. Near points of dead-load contraflexure, almost no warping stress is allowed if this provision is applied literally. This literal interpretation was not intended from the research.

(B) Maximum Normal Flange Stress. Equations defining strength of I-sections have separate factors for "Compact Sections" and "Noncompact Sections." The critical average compression stress for sections with compact flanges is defined for full plastification of the section, considering lateral and vertical bending. Different modifier factors are applied to Equation (B-7) by using $\overline{\rho}_b$ and $\overline{\rho}_w$, where λ is defined in Equation (B-7). These factors were developed assuming essentially the same parametric limitations described for the first yield criterion in ASD (*47*).

Flange slenderness for compact sections is limited to $3,200/\sqrt{F_{yf}}$ as in AASHTO (ASD) for straight girders. There is a footnote in the Guide Spec which, as recommended in the research (*8*), exempts the webs of these sections from compactness requirements This recommendation is based on tests of curved-girder webs having transverse stiffeners spaced at approximately the web depth and slenderness up to 150. Radii of the test girders were rather small, causing larger lateral-flange moments.

The original research was performed on doubly symmetric sections, so section properties for tension and compression flanges were the same. Results of the research were presented in terms of moment on the section, not critical stress (*47*). It is not apparent how a critical compression flange stress based on plastification of a singly symmetric section can be determined without consideration of the plastic properties of the section.

The Guide Spec limits the plastic critical stress of the girder to the yield moment by limiting the product, $\overline{\rho}_b \times \overline{\rho}_w$, to 1.0. The product can be as large as 1.13 when lateral bending stress is zero. The sum, $f_b + f_w$, is not limited to F_{yf} for compact sections.

The noncompact provisions are essentially the same as Part I, ASD, except critical stresses are used in place of the factor of safety. However, the sum, $f_w + f_b$, must be less than F_{yf}.

Compact section tension flange. The average normal tension stress is limited to F_{yf}. McManus (*47*) suggested the tension flange be limited to $\rho_w \rho_b$ times the value from the lateral buckling equation as for the compression flange, because only doubly symmetric sections were studied. The current specification is at odds with this requirement. There is no provision for computing different ρs for the tension flange when the section is singly symmetric. By limiting the average normal tension stress to F_{yf}, there may be no capacity remaining for the tension flange to resist lateral flange bending.

Noncompact section tension flange. This section is similar to the equivalent section in the ASD provisions, except that the sum of the factored bending and lateral stress is limited to F_{yf} instead of $0.55F_{yf}$.

(C) Web Design. These provisions are similar to ASD, except that web shear strength is used rather than shear stress. Unstiffened webs, transversely stiffened webs, and transversely and longitudinally stiffened webs are permitted. Interestingly, the allowable web slenderness for transversely stiffened webs is taken from Culver's work (*60*), while the equivalent requirement in ASD was taken from Daniels' work (*15*).

The moment of inertia of transverse stiffeners is approximately 9 percent greater than that required by the ASD provisions, because the constant, 10.92, is not included in the denominator as is the case in *Article 1.13*. A second longitudinal stiffener is permitted.

Design of longitudinal web stiffeners is based on the AASHTO provisions that are referenced. Girder web depth, *D*, is assumed to be $2D_c$ when more than half of the web is in compression.

C.3 Curved Composite I-Girders

2.13 General. ASD **Articles 10.38.1, 10.38.4,** and **10.38.5** are referenced for composite girders, not the LFD provisions.

2.14 Effective Flange Width. The effective width rules in **Article 10.38.3** are referenced. However, no instruction is provided regarding computation of warping and torsional constants for composite sections.

2.15 Noncomposite Dead-Load Stresses. The compression flange should be checked prior to the deck curing. When b_f/t_f is limited to $4,400/\sqrt{1.3 f_{dL1}}$, the flange tip stress, $f_{dL1} = f_b + f_w$, is limited to F_{yf}. For the compact flange condition, no check of tip stresses is required. Thus, the compact case permits flange yielding during construction.

2.16 Composite Section Stresses. Factored vertical bending stress in the top flange must be less than the flange yield stress. The bottom flange tip stress, $f_w + f_b$, is limited to the flange yield stress if the flange is noncompact.

2.17 Shear Connectors. The provisions of **Article 10.38.5** govern.

(A) Fatigue. The provisions of **Article 10.38.5.1.1** govern.

(B) Ultimate Strength. Similar to ASD.

C.4 Curved Hybrid Girders

2.18 General. Hybrid sections must be I-girders. The concept used in the development of these provisions is based on work for hybrid straight girders. The concept requires that a portion of the web may yield and be considered at yield when computing the resisting moment. Strain hardening is ignored.

The plastic moment in a curved hybrid I-girder is considered. A separate factor was developed for this case (*16*). Again, these provisions apply to singly symmetric curved girders, although the ρ factors were not developed for such girders.

A composite girder in the positive moment region prior to deck curing may not be considered hybrid. If it is hybrid, the web may not reach yield prior to curing of the deck. After the deck has cured, the bottom flange and web may be treated as hybrid, assuming full composite action. The addition of noncomposite and composite stresses is assumed to be appropriate. However, these stresses can be added only if superposition applies, which is the case when the noncomposite stresses do not produce yielding.

2.19 Maximum Normal Flange Stress. There are provisions for hybrid noncomposite and composite girders, which may be either compact or noncompact. There is no discussion regarding addition of noncomposite and composite stresses.

2.20 Plate Thickness Requirements. In determining required web thickness, f_b, of the compression flange used in *Article 2.12* is to be used. Actually, F_y, not f_b, is used in *Article 2.12* to determine web thickness.

2.21 Overload. The flange tip stress is not limited in hybrid girders, as it is in homogeneous girders in *Article 2.5.*

C.5 Curved Composite Box-Girder Bridges

These provisions are essentially the same as those for the ASD box girder, except that the allowable stress, $0.55F_y$, has been replaced with F_y so that critical stresses, rather than allowable stresses, can be used. If the top of a box is designed with individual flanges on each web, these flanges may be designed as either compact or noncompact.

D. Hanshin Guidelines

The *Hanshin Guidelines for the Design of Horizontally Curved Bridges* (draft) (*38*) refers to the Japanese Road Association's *Specifications for Highway Bridges* (*39*) for many basic requirements, but contains additional provisions reflecting curvature for many elements. The Hanshin provisions are based on working stress design as are the Japanese straight-girder design provisions. However, ultimate strength is frequently used to define member capacity rather than first yield or elastic buckling, as done normally in ASD.

The provisions do not distinguish between specification language and commentary. They instruct when either box or I-sections should be selected. Box girders are divided into three major cross-section types: single-cell mono-box with two webs, multicell mono-box with more than two webs, and multiple single-cell box. Aesthetics is discussed but not specified.

Framing is discussed with regard to economics. The use of straight interior girders with curved exterior girders is recommended. The use of top- and bottom-flange lateral bracing in certain bays is recommended for curved I-girder bridges with spans above approximately 80 ft. The need to keep the lateral bracing in or near the plane of the flanges is discussed. When this is not possible, the provisions require that transfer of the forces from the bracing to the flanges should be considered.

Effective deck width is discussed. Generally, more effective deck is used in Japanese composite bridges than is provided for in the AASHTO provisions (*9,41*).

Analysis is discussed in some length, but specific techniques are not designated. Four stress components are defined in a curved girder: vertical bending due to vertical loads applied to the girder, vertical bending due to curvature, warping torsion due to twisting, and vertical bending due to relative deflection of the girders. Extensive discussion is presented regarding analysis for each component. For example, a simple beam model is permitted for a mono-box. If overhangs are significant, a grillage method is suggested with "virtual" girders to receive the loads on the overhangs.

Strength provisions for the components are derived from inelastic finite element analyses and curve fitting. For example, this technique was used to develop the provisions for web longitudinal stiffener rigidity.

An extended discussion of thermal movement is presented. The provisions require bearings to be oriented considering restraint and friction forces. The provisions differentiate between sharp curvature and shallow curvature. Sharp curves may require that bearings be oriented to permit transverse thermal movement.

Expansion joints should be designed to be consistent with movement permitted by the bearings. If a finger joint is used, the fingers may be lined up with the girder rather than perpendicular to a joint. If the bearings permitted the bridge to expand transversely, then the fingers would be oriented perpendicular to the skewed joint.

Following the provisions, several design examples are presented.

IV. CURVED-GIRDER FABRICATION AND CONSTRUCTION

A. Introduction

The Guide Spec has been used to design hundreds of horizontally curved I- and box-girder bridges. Performance of the bridges, when completed, has been outstanding. However, there have been too many problems during their construction when compared to straight-girder bridges. The historical costs of curved-girder bridges appear to vary more than do the historical costs of their straight counterparts.

The cost to fabricate curved girders is expected to be somewhat greater than fabrication costs of straight girders of similar size. The increase is related to curving the girders and to handling them, both during fabrication and shipping. Erection of curved girders creates its own separate set of problems. Often additional falsework is needed when compared with what is required for straight girders of like span.

The use of curved girders, however, is generally economically advantageous for curved alignments and certainly their use contributes in an aesthetic way. The alternative to curved girders for curved alignments is the use of kinked straight girders or short simple spans with many joints. Curved girders are generally preferable to kinked girders because kinked girders generate rather large concentrated torque in the girders near the kinks and rather large cross frame forces in the same region. The axial force in the flanges changes direction at the point of noncollinearity, resulting in large lateral flange bending moments that are not dealt with in AASHTO. Some state agencies, such as PennDOT (*88*), require that the associated torsion and cross frame forces be treated in design. Kinked girders with curved roadway alignment cause a variable overhang distance that is expensive to construct and not attractive.

Similarly, the use of compound curved girders to accommodate a spiral roadway alignment is not economical. Compound curves cause a variable deck overhang distance that is expensive to form and compound curved girders are as difficult to build as spiral girders.

It is impractical to suggest the most efficient and economical steel sections that will be applicable to all states. Some states, including New York (*93*), Pennsylvania (*88*), and California (*94*), have developed their own specifications and manuals germane to local conditions. A survey of curved-girder bridges built in the United States in recent years is being made. This survey will be informative with regard to fabrication practice, as well as a source of knowledge regarding design parameters.

B. I-Girder Fabrication

There are three ways that horizontally curved I-girders may be produced: Girders may be fabricated as straight, and when fabrication is complete, curved to the desired radius; alternatively, flanges may be cut-curved to the required radius and the girder fabricated. Cold-curving is not a widely used method. It is rather a special method used for girders with small curvature by using proprietary roller-assembly devices.

Curving of the fabricated straight girder may be accomplished by either cold bending or by heat-curving. If the flanges are cut-curved, fabrication proceeds on the curved girder, which entails additional handling problems when the girder must be lifted. Final heat-straightening of cut-curved girders is usually required. However, the final heat-curving is much less time consuming than curving a straight girder. If the radius is large, fabrication of the straight girder and heat-curving is preferable. The break-even point on radius varies between fabricators.

Heat-curving may be performed while the girders are in either a vertical or horizontal position. Heat-curving is often preferred by fabricators because fabrication and handling of straight girders during fabrication is easier and straight flange plates can be cut from mill plate with less scrap than when flanges are cut-curved. There are disadvantages to heat-curving. The amount of curving possible is limited by the radius and flange width. As curvature increases, the time to perform the curving increases. To this day, heat-curving of girders remains an art despite a series of contributions by Brockenbrough (*95–99*).

Heat-curving may be performed by continuous heating of a flange tip, or by vee-heating toward the web at increments, along the girder. In either case, the heated side lengthens, causing the girder to become convex on the heated side. The yield stress of the heated portion is temporarily reduced. As the girder cools, its curvature reverses. The heated portion of the flange yields and the material is upset (i.e., the heated portion becomes thicker and the girder flange is left with a residual curve that is concave with respect to the heated side).

Heat-curving the top side of the flanges with the girder web in the horizontal position permits gravity to add to the bending due to heating. Gravity resists the reverse bending while cooling. Calculations are needed to ensure that girders do not yield during the heating procedure (*97*).

If the girder web is in the vertical position during heating, vertical bending effects due to the span between support points must be considered. Access to the top girder flange may be difficult if the girder is tall.

Heat-curving of heat-treated steels is not permitted because the material's mechanical properties may be altered.

B.1 Flange Plates

Most steel producing mills ship plates with a minimum width of 48 in. If the sum of the flange widths of a given thickness and grade is not at least 48 in., then the fabricator still must purchase a 48-in.-wide plate, thus generating unwanted scrap. If the flanges are to be cut-curved, different flange thicknesses or grades cause an even greater amount of scrap. Multiple flanges with equal thickness, grade, and length of interior plates permit the cut-curved flanges to be nested on a single plate with the crescent-shaped scrap losses occurring only once for each plate.

Many fabricators prefer a wide plate from which the flanges can be stripped after flange butt welding has been performed. This process is referred to as "slab welding." Economy is gained by requiring only one setup for the group of similar flange butt welds, rather than a setup for each flange. The design must use plates of the same width and length on each girder for the slab welding process to be applicable. Field sections at pier usually require two butt-welded splices in each flange; thus, the pier-flange plate is between two other flange plates. If the length of pier plate must vary between girders, slab welding cannot be used. It is common to locate splices of curved girders along a radial line, which causes the pier flange plates on the outside of the curve to be longer than the others. Alternatively, using the longest pier plate for all girders may permit efficient slab welding to be used.

Rolled shapes, according to AASHTO, may be cold bent if the curvature is not severe and the fabricator has proper equipment. However, some states may not permit cold-curving.

Cut-curving has become more popular with the advent of numerically controlled burning equipment. It is applicable to both box and I-girders. Estimation of fabrication time is improved with cut-curving, thus scheduling through the shop is more efficient. Analysis of curved-girder bridges often reveals significantly different moments for the girders within a cross section. Judicious selection of a flange thickness may permit a variation of girder capacities by changing flange width rather than thickness. Another option, although less economical, is to use the critical flange size in all girders. Choices in this regard are one reason for the highly variable cost of curved-girder bridges.

Usually curved girders can be welded with most girder machines where the machine is used to position the pieces and make flange-to-web welds. However, heat-curving is preferred in those shops where a straight girder is desirable with their machines.

B.2 Cross Frames

Cross frames on skewed and curved-girder bridges are main members and are designed for computed forces. They act to resist nonuniform torsion by transferring vertical and horizontal forces between girders and as lateral supports to the flanges. This action leads to lateral flange stress, which is a function of cross frame spacing, girder depth, and girder radius. As a result, curved girders often have cross frames more closely spaced and heavier than those used on right-straight-girder bridges. Their attachment to the girders of these bridges may require more than nominal size connections. Sometimes bolted connections are used for better fatigue performance. Cross frames of curved-girder bridges contribute a greater portion of the total fabrication cost than cross frame for right-straight-girder bridges.

C. Box-Girder Fabrication

Most box girders built in the United States and Canada are actually tub girders, having an open top that is braced with diagonal members. The tub girder is composed of two web plates, a single bottom flange plate, two top flange plates, internal cross bracing, and top flange bracing.

The top flange lateral bracing causes the girder to act as a closed section prior to the deck curing. It is attached directly to the top flanges or to the web stiffeners for gusset plates. In either case, the bracing with the top flanges acts as a truss. When the bracing is attached to a web, the stiffeners must be attached to the top flanges to transfer force from the bracing to the top flanges. Usually these connections are made by welding web stiffener ends to the inside of the flange. At times, a bolted connection is used.

Intermediate internal bracing along the box is often "K" shaped with the apex of the K at the top of the box to permit easier passage. These intermediate bracing members act to retain the shape of the box when it is subjected to torsion and to receive force from any external framing when it is used. The bracing members are either welded or bolted to transverse stiffeners that are, in turn, connected to the webs and flanges.

Internal diaphragms are used at bearings. The diaphragms are usually stiffened and have access holes for ease of passage during fabrication and inspection. The bearings are usually directly under the diaphragms and inboard of the webs. The details used on these diaphragms vary greatly, some being quite expensive. Standard design details do not seem to exist for these components. Burning out web plates to the correct camber that are both curved and inclined is mathematically challenging. Most competitive fabricators, however, have numerically controlled burning equipment that can produce box-web plates with little problem. The top flanges may be either cut-curved or heat-curved prior to attachment to the webs. Bottom flanges are always cut-curved.

It is possible to haunch inclined-web-box girders. In this case, the web should be from a developable surface, meaning that the final web shape should be able to be made by curving a flat plate about only one axis. This can be accomplished by holding the distance between the top flanges constant and varying the bottom flange width in the haunched regions. The Trenton Bridge, near Toronto, Ontario, is an excellent example of a steel haunched box-girder bridge.

Top flanges may be attached to the webs by using girder machines that position and orient the web and flange for welding. Alternatively, the fabrication can be done by positioning and clamping with less elaborate equipment. The curved top-flange-to-web fabrications are then welded to the bottom flange. If a girder machine is used for this process, the webs and bottom flanges are held in position with the machine. If a machine is not used, stiffeners are welded to the webs. The components are held in position with internal cross frames.

D. Fit-Up

Fabricators often are required to fit-up curved-girder bridges in the shop, prior to shipment, to ensure that girder splice bolt holes match and bolt holes in cross frames match holes in the gusset plates. Shop fit-up requires that much of the fabrication be completed prior to shipping.

Girder splices can be reamed without setting the girders in relative lateral position. The girder webs are often in the horizontal position during splice fit-up. Splice bolt holes can be drilled, or subpunched and reamed as with straight girders. In any case, fit-up is usually done in the near-zero-stress condition. A tacit assumption is that the girders and cross frames fit together under a near-zero-stress condition that corresponds to assumptions made for the design analysis.

Sometimes fabricators are not required to fit-up cross frames. Instead, oversized holes are permitted in one or both pieces of the cross frame connections. One advantage of not requiring shop fit-up of cross frames is that girders can be shipped as their fabrication is completed. Each girder splice can be fit-up with only the subsequent girder section available. The disadvantage of this method is that the oversized holes permit distortion of the bridge not considered in the analysis. Erection often becomes complicated as a result.

E. Erection

E.1 I-Girders

One of the advantages of a steel bridge is its ease of erection. For many girder spans, falsework is not required. However, curved girders often require falsework that would not be required for straight girders of the same span. Thus, curved-girder erection may be more expensive when compared with straight-girder erection and the need for falsework tends to reduce the advantage steel has with regard to erection.

To further complicate erection of curved-girder bridges, it is difficult to predict when falsework is required without rather sophisticated engineering. Many experienced erectors tend to err on the conservative side and use falsework.

Erectors use one of three means of stabilizing curved girders during erection. When the girder is rather straight, the ends of a simple-span girder are sometimes tied off, creating torsional restraint. If the girder has sufficient curvature to forbid stabilization of a single girder, it is necessary to use falsework or additional cranes. It is difficult to determine the need for temporary supports because there is little information on the behavior of slender unbraced curved I-girders. A third means of handling the girders is to connect two I-girders with cross bracing before lifting the assembly into place. This technique is limited by crane capacity.

Because curved girders interact through the cross frames, it is difficult to adjust their elevations. Lateral deflections are also a concern during the erection of curved girders. Although curved girders tend to rotate as well as deflect vertically, there usually is no reference to lateral deflections in the plans. When the girder is partially braced, it may rotate enough to make it very difficult to attach cross frames that were detailed for zero stress. Clearly, if the girders are adequately supported to ensure near-zero stress during erection, the problem of relative deflections is eliminated.

E.2 Box Girders

Curved box girders generally present fewer erection problems than curved I-girders. A single box girder is usually stable and may be erected with no special consideration. For a given bridge cross section, there are fewer box girders than I-girders, so there are fewer members to erect. Also, there are fewer lines of intermediate cross bracing between boxes.

One problem sometimes encountered with box girders is that torsion due to self-weight may induce enough cross-sectional distortion that fit-up of diaphragms between boxes is difficult, or impossible, without falsework. Torsion is most often due to skewed supports, but in extreme cases, may be due to curvature. This problem is less severe with I-girders because they are weak in torsion and can be more easily twisted to the proper attitude.

E.3 Oversized Bolt Holes

One problem encountered by erectors is the use of oversized bolt holes. Oversized bolt holes permit girders to deflect unpredictably. Although rarely used, oversized holes are permitted by AASHTO in girder splices, as well as in cross frame connections and bracing members. Oversized and slotted bolt holes are designed for lower slip values but for the same bearing strengths as standard-sized holes. The main advantage of oversized or slotted bolt holes is that the structure does not have to be subassembled by the fabricator.

Oversized holes permit unpredictable deflections and associated stresses that are different from those determined by analysis. It is often difficult for the erector to establish proper girder line and camber of curved girders when oversized holes are used for cross frame connections. The problem is exacerbated as curvature of the girder increases. For straight girders, oversized holes in the cross frame connections present less of a problem because little load is transferred between girders under the self-weight condition.

When standard-sized bolt holes are used, the erector can adjust the girder elevation and attitude until bolting of connections can be accomplished. If the holes are not stretched and filed reaming is not permitted, the deflected shape of the erected steel should closely resemble the computed shape. In fact, this condition is called for by **Division II-Article 11.2.2.** This article requires construction engineering to consider deflections and various loads during the erection and construction process to ensure that overstresses in the completed structure do not occur. There are a few instances where the contract calls for the contractor to acquire the service of an engineer. In most cases, curved-girder construction is carried out with little construction engineering. At present, there is no mechanism where the contractor can influence fabrication to match an erection plan.

F. Deck Placement

There are additional issues associated with placement of the concrete deck on curved-girder bridges. These issues relate to the fact that the girders in a bridge cross section deflect vertically different amounts as dead load is applied. The girder on the outside of the curve usually deflects more than the other girders, while the girder on the inside of the curve usually deflects less. Generally the concrete deck is screeded as it is placed. For example, when half of the deck in a span has been placed, the girders will have deflected approximately half of the total deflection for the deck load in that span. The girders have been cambered for the total deck deflection. If only half of the deck camber has been deflected out of the girders, only half of the differences between girder deflections will have occurred. This means that the relative position of the girders has not reached its final condition. If deck weight is added to the bridge such that the girder deflections are not in proportion to the load, there can be an error in the deck thickness.

Load applied to the deck overhang brackets can cause additional torque on the girders. Typically, exterior girders are designed for their share of the deck concrete applied to the girder. In fact, a significant portion of the deck weight applied to the outside girders is applied to the overhang brackets, creating torsion in these girders. The screed rails are also usually placed on the overhang brackets, creating additional torsion in the exterior girders. Exterior girders are usually not designed for this torsional load. Bracket deflection is usually not accounted for either.

V. FUTURE RESEARCH NEEDS

A. Introduction

Horizontally curved beams are one of the least understood structural elements in common use today. Designers, owners, and users will undoubtedly benefit from research leading to improved understanding of curved-girder bridge behavior. These research needs can be separated into two categories: improved assessment of the strength of their components and more efficient and aesthetic design forms. Although different, the two areas go hand-in-hand. Improved strength equations and details, alone, are likely to provide only marginal benefits. The confidence gained through improved understanding, however, has a much larger potential to bring about novel bridge forms.

New analysis tools, particularly finite element computer programs, are now available that permit detailed analyses of both structural elements and entire structures. These analysis tools have not been adequately applied to horizontally curved beam bridges. Field instrumentation of curved bridges carried out by several researchers over the past 20 years remains unexplored with the newest analysis tools. It is imperative that any new experimental research be paralleled with refined analytical investigations if the maximum benefits are to be gleaned from the experimental work.

There is a wide range of analysis techniques employed in the design of curved-girder bridges. The Guide Spec requires that the entire superstructure be considered in the analysis. However, approximate methods are permitted that have not well-defined limitations. Even the most refined methods have limitations and assumptions that remain unexplored. For these reasons, the research team believes that it is unwise to venture too far from past practice in design. There is a great need to quantify the reliability of various analysis methods and to explore how and when each method might be best applied. It is well known that all V-load analyses are not the same. It is less well known that all 3D finite element analyses do not provide the same results. Application of the methods needs to be studied and compared to field measurements.

Construction of curved bridges is an area where more research is needed. There have been numerous problems associated with construction and a wide range of opinion as to the best way to build curved-girder bridges. Although such bridges will remain too complex to pigeonhole the construction methods for them, there is a need for better fundamental understanding of the structural behavior of curved girders during fabrication and construction.

Much of the research has been directed toward I-girders. Recently, box girders have found favor in a number of states. The problems of the two types of curved girders are different. It does not seem practical to group the two girder types together. I-girders are torsionally weak and are difficult to ship and erect. Box girders are easier to ship and erect; however, erection problems are related with their torsional stiffness rather than weakness. The strength predictions of both types of girders are in need of improvement.

The behavior of curved bridges near ultimate load is unknown. Thus, the excess capacity of these bridges for load beyond design levels is not defined. If a stringer in a straight-girder bridge fails, the remaining girders in the bridge cross section are fully effective. This assumption cannot be safely made with all curved-girder bridges.

The research needs can be divided between box and I-girders—between construction and ultimate capacity and between element and overall behavior.

B. I-Beams

1. Unsymmetrical curved I-sections need to be studied because they are the most commonly used sections. Also multiple I-girders attached to a common composite deck should be investigated. These types of sections should be investigated both analytically and experimentally. Updated predictor equations for flexural and shear strength are needed to cover all types of I-sections.

2. Investigation of various types of web stiffening should be expanded for bending, shear, and combined bending and shear conditions. Tests of I-sections without stiffeners at web slenderness up to 200 should be made. Longitudinally stiffened webs need to be tested. Tests of various intermediate transverse stiffener configurations are required to develop an analytical model that represents the shear strength of curved-girder webs. These tests should be expanded to evaluate the effectiveness of various end details of stiffeners. The effect of stiffener spacing on the bend-buckling strength of curved girders with varying details is needed.

3. Better definition of local flange buckling is needed. There is a wide difference between the Hanshin provisions and the AASHTO provisions in this regard.

4. Tests of shear connectors that are subjected to large lateral forces near cross frames in fatigue are needed.

5. Evaluation of bottom flange bracing and related analysis techniques leading to the proper consideration of these members in design is needed. The effectiveness of these members during construction is not well understood. Because they are expensive to fabricate and erect, it would be useful to ascertain whether or not they should be used, and if so, when are they effective?

C. Box Beams

1. Although box distortion reduces its torsional stiffness and probably reduces its strength, the degree to which these changes occur is not defined. The effect of cross-section distortion on shear strength and bending strength needs to be investigated experimentally. Design equations must then be developed from the work.

The effectiveness of internal cross bracing in reducing cross-section distortion needs to be investigated. Design recommendations should evolve from such an investigation. How cross-section distortion affects box-girder analysis needs to be investigated.

2. Internal diaphragms are expensive and difficult to design. They serve a multitude of purposes including retention of box shape, bearing stiffener, and support for external diaphragms. Design rules for these members need to be advanced.

D. Constructibility

1. Most structures may be adequately analyzed by using small deflection theory. However, curved I-girders deflect so much laterally at times, that the adequacy of this assumption is brought into question. During lifting and when girders are temporarily unbraced over large distances, lateral deflections and twist are large. Research is needed to define which conditions limit the applicability of small deflection theory. Beyond these limits, large deflection theory and possibly inelastic theory as well would be required.

2. Research is required to determine when lateral deflection and twist limitations are needed in curved-bridge construction to ensure that stresses and deflections will not exceed those permitted.

E. Analysis

1. Techniques in use should be evaluated for a variety of curved bridges to determine the adequacy and limitations of each method. Approximate analysis methods (M/R or V-load methods) need to be compared to more refined methods to determine the best application techniques. The proper use of wheel load distribution factors is particularly laborious in this regard.

2. The failure mechanism of a curved bridge needs to be defined. Tests of full-scale bridges to failure are needed. Design specifications provide load factors based on tests that give an acceptable margin of safety for error between computed and actual strength. The load factor design provisions use an analysis factor of 1.3. The LRFD provisions use an analysis factor of 1.0, because the refined wheel load distribution factors and other methods of assigning load for straight girders are thought to be very accurate.

F. Design Forms

New design forms need to be studied to improve the efficiency of curved-girder bridges. Examples include precast concrete decks, large box girders, different shapes of haunched girders, tied-down spans, and post-tensioned decks. Different erection schemes and different construction methods, particularly for concrete deck construction will contribute to the efficiency of curved-girder bridge construction.

Many of these and other concepts can be accomplished with the proposed specifications. No fundamental research is needed, only design innovation.

ANNEX I

COMPARISON OF DEAD-LOAD ANALYSES

A three-girder, simple-span curved bridge, as shown in Figure I-1, is analyzed for noncomposite dead load by using the V-load method, finite difference method (*50*), and 3D finite element method (MSC/NASTRAN). The purpose of this analysis is to compare the resulting vertical bending stresses and lateral bending stresses of three different analysis techniques. The noncomposite dead loads (DL1) are 1.034 kips/ft for two outside girders and 1.247 kips/ft for the interior girder. All truss cross frame members have a cross-sectional area of 5 in^2.

The maximum vertical bending stresses computed from the three different methods are presented in Table I-1. Table I-2 gives the maximum lateral bending stresses.

Although a three-dimensional, finite element analysis computes element stresses directly from each element, additional computations are required to determine the flange lateral bending stresses in cases where the V-load method and finite difference method are used. The following computation examples illustrate the procedures used to generate values presented in Table I-2:

V-Load Method

At midspan, Girder No. 3, M_{max} = 1,973.5 k-ft, l = 12.5 ft, R = 300 ft, and D = 5.0 ft.

$$M_l = \frac{M_{max}\, l^2}{12DR} = \frac{1,973.5 \times 12 \times (12.5 \times 12)^2}{12 \times 60 \times 300 \times 12} = 205.6\ \text{k-in.}$$

$$f_w = \frac{M_l}{t_f b_f^{\,2}/6} = \frac{205.6}{1 \times 12^2/6} = 8.56\ \text{ksi}$$

Finite Difference Method

At midspan, Girder No. 3, Bimoment (BM) = 11,929 k-in^2, I_ω = 267,912 in^6, and ω_n = 183 in^2.

$$f_w = \frac{BM\omega_n}{I_\omega} = \frac{11,929 \times 183}{267,912} = 8.15\ \text{ksi}$$

Any additional normal stresses occurring due to cross-sectional distortion are included in a three-dimensional, first order finite element analysis. In the classical strength of materials analysis, cross sections are assumed to retain their original shape during deformation. Despite that, this assump-

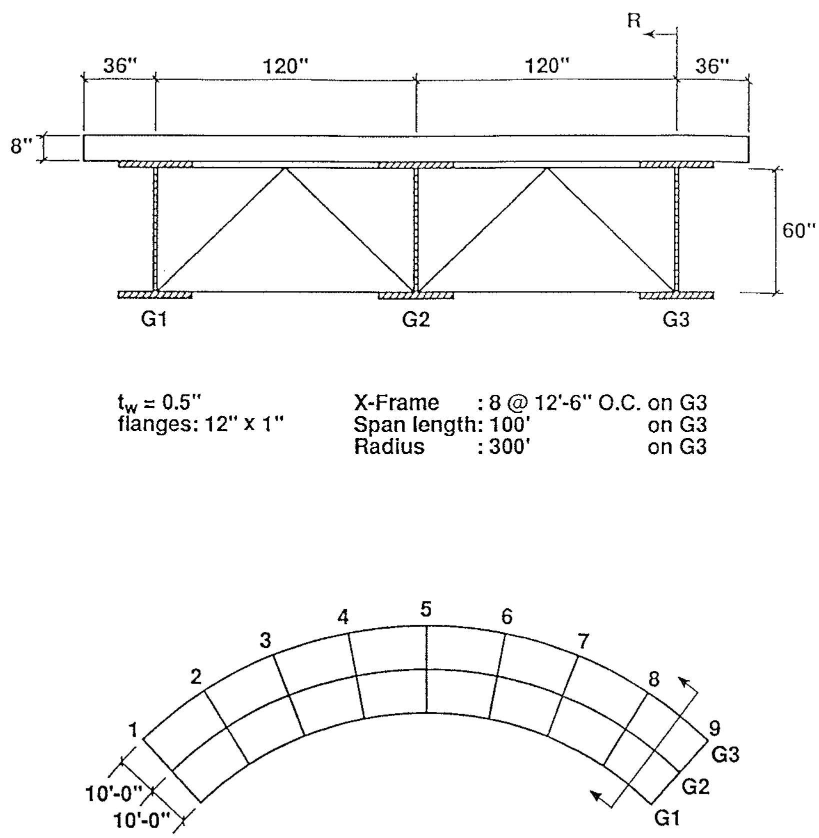


Figure I-1. Framing plan and schematic of cross section.

TABLE I-1 Maximum vertical bending stress (ksi)

Girder No.	V-load	Finite Difference	MSC/NASTRAN
1	5.77	6.35	7.02
2	17.10	15.66	15.90
3	23.20	23.88	23.80

tion may not be valid, particularly for thin-walled assemblages; it has been retained, nevertheless, mainly because in most well-engineered and braced structures, distortion effects are very small. Without the assumption, the governing equations become differential equations with variable coefficients, thereby rendering any closed-form solutions to the differential equations practically impossible.

According to Dabrowski (*6*) and Heins and Firmage (*100*), the distortional normal stress is defined by the induced stress occurring in thin-walled structural elements of the girder cross section. As a result of the variations in torsional shears, distortion of the cross section takes place. Although the rigorous analysis of distortional effects is quite complicated, Heins and Firmage (*100*) introduce the following simplified equation based on regression analyses:

$$\frac{f_d}{f_b + f_w} = \frac{AX^3 + BX^2 + CX + D}{100} \qquad \text{(I-1)}$$

where

f_d = induced normal distortional stress (ksi),
f_b = vertical bending stress (ksi),
f_w = warping stress (ksi),
X = cross frame spacing/span length, and
A,B,C,D = coefficients given.

For $R = 300$ ft and $L = 100$ ft, coefficients A, B, C, and D are determined by using linear interpolation. $A = -16.864$, $B = 8.8376$, $C = 10.2578$, $D = -0.5383$, $X = 12.5/100 = 0.125$.

$$\frac{f_d}{f_b + f_w} = \frac{-16.864 \times 0.125^3 + 8.8376 \times 0.125^2 + 10.2578 \times 0.125 - 0.5383}{100}$$

$$= 0.00849$$

$f_d = 0.00849 \times (23.20 + 8.56) = 0.27$ ksi V-load method

$f_d = 0.00849 \times (23.88 + 8.15) = 0.27$ ksi Finite difference method

The second order or amplification radial moment and, hence, stress due to the presence of axial force, in addition to the primary lateral-bending moment, may be evaluated by considering an amplification factor. There are four different equations presented to date for the in-plane buckling of a circular arch. A recent study by Kang and Yoo (*101*) gives the lowest buckling load, while an equation by Papangelis and Trahair (*102*) yields the highest buckling load. An equation giving a buckling load in the middle is to be used in this computation example. The following equation is supported by many researchers (*7,24,42,103*):

$$P_{cr} = \frac{EI}{R^2}\left(\frac{4\pi^2 R^2}{l_e^2} - 1\right) \qquad \text{(I-2)}$$

where l_e = equivalent simple-span length.

For the example, l_e may be taken as $0.6l$. $I = 1 \times 12^3/12 = 144$ in^4, $R = 300$ ft.

$$P_{cr} = \frac{29{,}000 \times 144}{(300 \times 12)^2}\left(\frac{4\pi^2 \times (300 \times 12)^2}{(12.5 \times 12 \times 0.6)^2} - 1\right) = 20{,}353 \text{ kips}$$

$$P = 23.20 \times 12 \times 1 = 278.40 \text{ kips}$$

Therefore, an amplification factor for the lateral flange bending may be computed as

$$AF = \left(\frac{1}{1 - \frac{P}{P_{cr}}}\right) = \left(\frac{1}{1 - \frac{278.40}{20{,}353}}\right) = 1.014$$

Finally, $AF(f_w + f_d) = 1.014\ (8.56 + 0.27) = 1.014\ (8.83) = 8.95$ ksi.

From Tables I-1 and I-2, stresses evaluated by the three different methods are fairly close. The three methods used are V-load, a modified one-dimensional, line-girder analysis; finite difference, a 2D grid analysis; and MSC/NASTRAN, a 3D finite element analysis. A 3D finite element analysis is capable of calculating the first order distortional stresses.

TABLE I-2 Maximum lateral bending stress (ksi)

Girder No	V-load	Finite Difference	MSC/NASTRAN
1	1.99	1.66	1.90
2	6.11	5.00	5.17
3	8.56	8.15	8.25

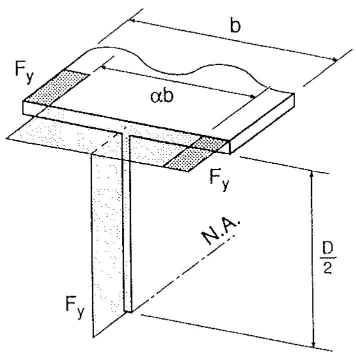


Figure I-2. Interactive yield stress distribution.

From these comparative analyses, it appears that the magnitude of the distortional stress is similar to that evaluated from the simplified procedure given by Heins and Firmage (*100*).

The yield stress distribution of an I-section, fully plastified under the combined action of vertical bending and lateral bending, is shown in Figure I-2.

The vertical bending moment is $F_yD(\alpha A_f + A_w/4)$. From this moment, the virtual load, q, is computed as $F_y(\alpha A_f + A_w/4)/R$. Although the second term is usually neglected in the V-load analysis, as shown in Equation (B-6), it is included in this example calculation to make the resulting lateral bending moment large. Assuming, conservatively, that the virtual load, q, is uniformly distributed between two adjacent brace points, the resulting lateral bending moment becomes $ql^2/12$. If there is a complete redistribution of moments to form a collapse mechanism, then the denominator should be 16 instead of 12. To be consistent with assumptions employed throughout the development of the Guide Spec and to be conservative, no redistribution of moment is assumed. With the dark-shaded stress blocks in Figure I-2, the resisting internal lateral bending moment is evaluated as $F_yA_fb(1 - \alpha^2)/4$. Equating these two expressions for the lateral bending moment and solving for α yields $\alpha = 0.861$. This implies that only 14 percent of the flange area is plastified due to lateral bending (or warping).

An amplification factor is recomputed for the section when it is fully plastified. The equivalent axial load, assuming $F_y = 50$ ksi, is $P = 50\ (0.861 \times 12 + 60 \times 0.5/4) = 891.6$ kips.

$$AF = \left(\frac{1}{1 - \frac{P}{P_{cr}}} \right) = \left(\frac{1}{1 - \frac{891.6}{20{,}353}} \right) = 1.046$$

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NOMENCLATURE

The following symbols are used in this document.

A_f	area of one flange of beam or girder (in^2)
A_w	area of web (in^2)
a	aspect ratio of web panel, d_o/D
BM	bimoment (k-in^2)
b	flange width (in.)
b_f	compression flange width (in.)
C_b	moment gradient correction factor
D	distance between flange components (in.)
D_c	depth of web in compression (in.)
DL	dead load (k/in.)
d_o	distance between transverse stiffeners (in.)
E	Young's modulus (psi)
e	eccentricity of applied load with respect to shear center (in.)
F	flexural stress in a box (psi)
F_c	allowable vertical bending stress in Hanshin (psi)
F_{cr}	critical lateral torsional buckling stress of straight girder (psi)
F_v	allowable shear stress (psi)
F_y	specified minimum yield stress (psi)
F_{yc}	specified yield stress of the compression flange (psi)
F_{yf}	yield stress of flange (psi)
F_{yw}	yield stress of web (psi)
f_b	calculated bending stress (psi)
f_{bc}	average normal stress in the compression flange of curved girder (psi)
f_{bs}	average normal stress in the compression flange of straight girder (psi)
f_d	distortion stress (psi)
f_{dL1}	noncomposite dead-load bending stress (psi)
f_c	computed vertical bending stress in Hanshin (psi)
f_l	lateral bending stress at brace point (psi)
f_v	calculated shear stress (psi)
f_w	calculated normal stress at the edge of the flange due to nonuniform torsion (lateral flange bending) for plate girders (psi)
G	shear modulus (psi)
H	torque (k-in.)
I	impact factor
I_o	polar moment of inertia about the shear center (in^4)
I_p	polar moment of inertia about the center of gravity (in^4)
I_ω	warping constant (in^6)
J	required rigidity ratio of one transverse stiffener to that of the web plate
K	plate buckling coefficient
K_T	St. Venant torsion (or pure torsion) constant (in^4)

l distance between lateral flange restraints (in.)
l_e equivalent simple span length for lateral bending moment, $0.6l$ (in.)
L span length (in.)
M bending moment (k-in.)
M_l flange lateral bending moment due to noncollinearity of flange forces due to flexure (k-in.)
M_p plastic moment (k-in.)
M_u predicted ultimate moment (k-in.)
M_{ut} test ultimate moment (k-in.)
M_y yield moment (k-in.)
M^* applied moment/buckling moment
P concentrated load (k)
P_{nl} concentrated load at first nonlinear deflection (k)
P_{ut} test ultimate concentrated load (k)
q equivalent uniform lateral load resulting from noncollinearity of flange resultant forces due to flexure (k/in.)
$q_{x_{SM}}$ equivalent uniform lateral load in McManus simplified model (k/in.)
R radius of girder (in.)
R_b flange stress reduction factor
r $\sqrt{t_f b_f^3/(12A_f)}$ (in.)
r_y radius of gyration, $\sqrt{I_y/A}$ (in.)
T (time for live load to traverse a span)/(natural frequency of the span)
t thickness (in.)
t_f thickness of flange (in.)
t_w thickness of web (in.)
u_2 lateral displacement of McManus model (in.)
V torsional shear stress in a box (psi)
V_p plastic shear capacity (k)
V_u ultimate shear force capacity (k)
w spacing between interior box diaphragms (in.)
X parameter defining transverse stiffener rigidity
y_o distance between shear center and center of gravity (in.)
Z curvature parameter
z distance from support to load (in.)
α effective flange width parameter
β position parameter of longitudinal stiffener
γ_c parameter defining stiffener rigidity of curved beams
γ_s parameter defining stiffener rigidity of straight beams
Δ reduction factor for the maximum compressive/tensile stress of box girder
λ slenderness parameter $[(l/r)F_y]/(4\pi^2E)$
μ Poisson's ratio
ρ_b curvature correction factor for flange parameters, noncompact
ρ_w curvature correction factor for warping stress, noncompact
ρ_{w1} curvature correction factors for warping stress, compact
ρ_{w2} curvature correction factors for warping stress, compact
$\bar{\rho}_w$ controlling ρ_{w1} or ρ_{w2}
$\bar{\rho}_b$ curvature correction factor for flange parameters, compact
σ_ω warping stress (psi)
ϕ angle of twist
ψ torsional rigidity ratio $\left(L\sqrt{GK_T/EI_\omega}\right)$
ω_n normalized unit warping (in^2)

APPENDIX C

A UNIFIED APPROACH FOR DESIGNING AND CONSTRUCTING HORIZONTALLY CURVED GIRDER BRIDGES

PROPOSED AASHTO SPECIFICATIONS (HIGHLIGHTS OF MAJOR CHANGES)

I. INTRODUCTION

Unlike manufacturing many similar objects, where it is possible to test a number of products, each bridge is unique. A design specification attempts to provide uniformity to each design with regard to load capacity and serviceability. Bridge design specifications are a means of presenting loads and design rules to ensure that each bridge is capable of carrying a known level of load with a consistent safety margin and for an expected life.

There are disadvantages to design specifications. They are based on the state of the art, thus they are directed to the form of structures being designed at the time. When rules of thumb are incorporated into the provisions, limitations on the form of the structure are inherent. Another disadvantage of rules of thumb is that the reasoning behind them is often lost to the next generation of designers. A commentary provides an excellent means of retaining this lore.

Horizontally curved girder bridges present a particularly challenging form of steel bridge with regard to both design and construction. The proposed curved-girder specifications have been written with the objective of providing maximum uniformity with regard to capacity while permitting the widest variety of framing practical. These conflicting objectives are addressed through adherence to sound structural engineering principles and minimal reliance on rules of thumb. The accompanying commentary explains the reasoning behind the provisions and provides some direction in their application.

II. SPECIFICATION FORMAT

Design specifications are contained within a format. Originally, AASHTO (*1*) was based on the allowable stress design (ASD) format. The load factor design (LFD) format was adopted as an alternative method in the 1960s. The load and resistance factor design (LRFD) format was recently adopted (*2*). The research team believes that the LRFD format will be the singular format within 3 years.

Each succeeding format is based on preceding formats. Thus, to fully understand any succeeding format, it is advisable to examine its predecessors. The proposed curved-girder specifications are presented in an LFD format; however, several of the provisions draw on other formats.

1. ASD Format

The computed member stresses are limited to the yield stress or elastic buckling stress of the member divided by the factor of safety. Stresses are computed from the nominal loads. Additional strength was available in the members, which is part of the logic of ASD involved in selecting factors of safety related to the serviceability of the bridge. For example, noncomposite design was used although it was known that when the deck rested on the stringers, the deck acted compositely and caused the bridge to be stiffer than when computed assuming noncomposite action. The general form of the ASD format is shown below:

$$\frac{\text{Yield or buckling stress}}{\text{Factor of safety}} \geq \text{Stress due to } \left[DL + (LL + I)\right]$$

Rewriting this equation in the AASHTO format gives

$$\frac{\text{Yield or buckling stress}}{1.82} \geq \text{Stress due to } \left[DL + (LL + I)\right]$$

AASHTO found that a factor of safety of 1.82 is appropriate.

In addition to the allowable stress requirements, fatigue limits are provided. Fatigue stresses have been found to be progressively more critical when welding is employed.

Other limitations include span-to-depth limitations and live-load deflection limits. There were three span-to-depth limits. Trusses were limited to approximately not less than $1/10$ of the span. Multistringer beams were limited to not less than $1/25$ of the span and girders in stringer-girder bridges were limited to not less than $1/15$ of the span. In 1935, a paragraph was added that permitted shallower members if the computed deflection was not greater than if the specified ratios had not been exceeded (*3*).

In 1936, the Bureau of Public Roads limited computed live-load deflection to $1/800$ of the span for bridges with light traffic and $1/1{,}100$ of the span for bridges in or adjacent to populous centers and bridges carrying heavy traffic (*3*). These requirements remain essentially unchanged to this day.

2. LFD Format

LFD was introduced into AASHTO in the late 1960s. With this format, new concepts were introduced. The strength of the member was defined as the ultimate strength rather than the yield or buckling stress. Some shapes were capable of carrying load in bending up to the plastic moment. The post buckling strength of webs in shear was recognized with the tension field concept.

The nominal loads from ASD were used. However, factors were applied to the loads rather than a safety factor to the strength. The general form is given below:

$$\Phi\text{Strength} \geq \gamma\left[\beta_{DL}DL + \beta_{LL}(LL + I)\right]$$

This equation can be rewritten as

$$\text{Strength} \geq \frac{\gamma}{\Phi}\left[\beta_{DL}DL + \beta_{LL}(LL + I)\right]$$

In the AASHTO format, the above equation becomes

$$\text{Strength} \geq 1.3\left[DL + 5/3(LL + I)\right]$$

According to Vincent, reliability of the strength predictions was implied by a Φ factor of 0.90 for most members (*4*). So the capacity side of the equation had an implied strength factor of 1.0, γ was divided by Φ. This resulted in an overall factor of 1.3, which was applied to dead and live loads (*4*). This overall factor accounted for variability of strength as well as variability of dead and live load. Vincent states that γ/Φ accounted for uncertainty of overall effects such as load application and analysis (*4*). Thus, γ/Φ was indeed a partial safety factor.

The β_{DL} is assumed to be 1.0. A β_{LL} of $5/3$ is applied to live load, which accounts for potential overloads. These factors were set (calibrated) to provide multistringer LFD formats having parity with ASD format at approximately 40-ft spans. At greater spans, LFD gives lighter designs because the ratio of dead-to-live load increases with span. Thus, the shortest bridges designed by using ASD have the smallest safety margin, while those bridges designed by using LFD have a more uniform margin of safety.

In addition to the strength requirement, serviceability requirements were also specified as in ASD. Serviceability requirements such as live-load deflection limits generally involve load factors of 1.0.

Overload criteria are introduced. This additional requirement was created to ensure that the strength limits do not permit occasional overloads to cause permanent deflection called set (*4*). Both ASD and LFD codes remain applicable.

3. LRFD Format

LRFD has been recently introduced into the AASHTO bridge specifications (*2*). Design provisions with similar formats have been adopted by AISC and AISI. In this format, the factors applied to the loads are called load factors. The Φ factor applied to the capacity predictor is called a resistance factor, which is usually less than 1.0 and is placed in the numerator (nominal capacity of the member). In Europe, a factor for the reliability of strength predictors is greater than 1.0. It is placed in the denominator like a safety factor and is referred to as a partial safety factor.

The general form for LRFD is given below:

$$\Phi\text{Strength} \geq \beta_{DL}DL + \beta_{LL}(LL + I)$$

Rewriting in the AASHTO format gives

$$\Phi\text{Strength} \geq 1.2DL + 1.6LL$$

A new live load was introduced to replace HS20. The new live load was made much heavier than HS20 to represent the current and future loads expected on the nation's highways. A new fatigue load was also specified. The fatigue loading is a single vehicle in only one lane. The vehicle produces less stress range than the HS20 vehicle.

The partial factor of safety, γ/Φ, is not present in the new format for strength. However, the load factor on dead load is increased from the 1.0 used in LFD to 1.2.

Wheel load distribution factors were changed significantly; generally they became smaller, leading to the assignment of less live load to a stringer.

The rules of thumb remained essentially unchanged regarding girder depth and live-load deflection limitations.

4. Limit States Design (LSD)

The LSD format is used in Europe and Canada. In this format, the factors applied to loads and capacity predictors are called partial safety factors, which implies a distribution of the margin of safety over several factors.

III. SPECIFICATION COMPONENTS

Each of the formats has similar components. The components are provided in a similar manner. The proposed curved girder provisions have the same components.

1. Loads

The most basic requirements of a design specification are the provisions regarding loads. The engineer determines the dead loads to be used in the design. The owner specifies the live loads the bridge is to carry. Many other loads such as wind, ice force, and thermal changes are much more arbitrary.

Historically, nominal loads have been specified in bridge design. Nominal loads may be defined as the expected level of load; they are not usually extreme event loads. Because nominal loads are used, an allowance should be made for extreme loads by load factors or factors of safety.

No new loads are defined in the proposed curved-girder provisions. However, the new LRFD fatigue live load with its special impact provision is specified. Wind loads are specified to be applied unidirectionally rather than perpendicularly to the bridge. If perpendicular loads are used on a curved structure, the wind must be assumed to change direction, which is unreasonable.

2. Load Application

Load application provides the means whereby loads are applied to the bridge for analysis. When bridges were designed as noncomposite simple spans with two main girders, loads were assumed transferred transversely to the two main girders according to statics; therefore, load application was rather straightforward. There was little variation in the resulting analyses between different designers.

In the 1940s, multistringer bridges became popular. It was not as evident how much live load should be assigned to a stringer with this type of construction. Variations in assumptions resulted in an increase in the variation of bridge strengths. Researchers at the University of Illinois developed a unified approach for assignment of live load to multistringer bridges (*5,6*). The results of this study became the wheel load distribution factors. Simple equations were developed that were based on a detailed study of the size and type of multistringer bridges being built at the time. The resulting stringer spacing over a constant (S/C) was a compromise between complexity and accuracy.

Even as the complexity and size of stringer bridges increased, the wheel load distribution factor remained unchanged until very recently (*7*). However, the Guide Spec (*8*) required that loads should be applied via analysis of the entire superstructure, because the strong interaction between girders and cross frames was significantly different from what was considered in the original study. The V-load method is often used to compute an additional moment due to curvature that is added to the moment computed based on a wheel load distribution factor.

Dead load is applied to the noncomposite structure based on statics. A similar approach is used in the proposed girder specifications. When load is applied off the shear center, the torque generated must be calculated.

All of the AASHTO formats permit superimposed dead load to be applied by assuming that it can be distributed uniformly to each girder. The proposed curved-girder provisions do not permit this assumption. Instead, loads must be applied on the deck as they occur. For example, these loads include parapets that often are on the extreme edge of the deck.

3. Structural Analysis

The bridge must be analyzed to obtain the load effects in the members. There are very few provisions regarding this type of analysis in AASHTO. Originally, analysis of simple noncomposite girders was straightforward. When composite design and continuous girders became prevalent, states began to develop new rules to be used within their jurisdiction. AASHTO rules regarding analysis remained nominal. The need for a unified approach for the calculation of these stresses is evident.

The use of the deck in tension in negative moment regions of continuous girders is recommended in LRFD and is required in the proposed curved-girder provisions. This behavior has been observed in most field tests. This requirement provides conservative predictions of moment in negative moment regions.

Because consideration of the entire superstructure is required in the proposed curved-girder provisions, more refined analysis techniques are implied. For example, consideration of lateral forces at bearings is required. A normal beam or grillage analysis assumes that the reactions are on the neutral axis and will not yield the needed horizontal forces.

Live-load analysis can be performed by the influence line or influence surface methods. In these cases, unit loads are applied to the bridge model. The actual loading to obtain load effects is applied to the influence lines or influence surfaces.

4. Load Effects

Load effects are the stresses, deflections, and other responses in the individual members used to check the adequacy of the design. The stresses are compared against those computed from strength predictors.

Normally, girder moments are computed from the analyses for various load conditions. Stresses are then computed from the moments. Sometimes the girder moment or shear can be compared directly. If the ultimate moment is used, noncomposite and composite moments are added and the total moment is compared to the capacity of the composite girder. When the critical moment is elastic, however, the sum of noncomposite and composite stresses must be compared with the critical stress.

The determination of stresses from girder moments requires that section properties including moment of inertia be computed. For composite sections, additional assumptions are involved. The amount of deck that is effective in resisting compression must be determined as well as its effective modulus. In AASHTO, the effective width of concrete deck for composite sections is specified as 12 times the structural thickness of the deck unless other limits control. In negative moment regions, only the longitudinal reinforcing is considered effective in resisting tension in computing stresses. In the proposed curved-girder provisions, the effective width of concrete is defined as center-to-center between stringers, which has the effect of increasing the effective concrete in most cases. The moment of inertia of the composite section is larger, but because the neutral axis is higher, the section modulus for the bottom of the section is changed very little. However, the first moment of the deck used to design shear connectors is greatly increased, requiring more shear connectors.

In negative moment regions, the full concrete area of the deck is used to determine the first moment for fatigue design of the shear connectors, but only the longitudinal reinforcing is used for strength determinations of the shear connectors.

Radial and tangential shear forces are required in shear connector design for fatigue to ensure that statics are met at the cross frames. The radial component for strength of shear connectors is computed in a slightly different manner than in the Guide Spec.

To compute bending stresses in the negative moment region for service loads including fatigue, the deck is considered uncracked. For strength, the deck is considered cracked with only the reinforcing effective.

Web bend-buckling is first computed assuming the noncomposite section. The web must be checked again for the composite dead load by adding the composite stress to the original noncomposite stress. The sum of these stresses determines the location of the neutral axis. The process is performed again for the live load on the composite section. This situation is somewhat complex for longitudinally stiffened webs, where the location of the longitudinal stiffener may be selected for one case and checked for the others. In all cases, the web is designed by assuming elastic buckling based on a flat web.

As computer analysis techniques have become prevalent, differences between various analyses have become more pronounced. For example, the proposed provisions require computation of cross frame forces, whereas their design must be simply standard details if no analysis of their forces is made. Different analyses may also give different responses for cross frame forces. The proposed curved-girder provisions give little instruction regarding analysis except that any section, including cross frames and deck, must be in equilibrium, which can be accomplished in any number of ways by the designer. When cross frames are attached to the web instead of the flanges, detail design of the force flow to the flange is required. Again, the designer can accomplish this in any manner that assures static equilibrium.

The proposed curved-girder provisions do not specifically consider reliability of analysis or the determination of load effects. Instead, the reliability is based on the unified approach in the determination of load effects. This approach is developed around an assumed analysis accuracy, which is the responsibility of the engineer. It is impossible to specify the level of refinement of analysis required for each bridge. Instead, practicality must be balanced against inaccuracy. If results from an analysis do not provide load effects that are in equilibrium, the engineer is given a clue that something is not right. The accuracy of any analysis depends on the particular structure. For example, the effect of skew on girder bridges is very significant. In many instances, a refined analysis is not required for a radially supported curved bridge but it might be needed for a highly skewed one.

5. Limit States

Limit states are the conditions that limit the design to ensure that the bridge will perform as planned. They involve the strength of the bridge and its serviceability. More recently, the constructibility of the bridge has been given more consideration. In the proposed curved-girder provisions, constructibility is considered a limit state.

Strength of a bridge is based on the accumulated strength of the component parts. If the parts meet the strength requirements of the design provisions, the bridge is assumed to have adequate strength. In the case of curved-girder bridges, the performance of a bridge at ultimate load is not known. It still seems conservative to make the same assumption. The proposed curved-girder provisions make no attempt to estimate the strength of curved-girder bridges other than on a member basis.

Serviceability tends to deal with overall behavior of the bridge. The deflections and overload are the main limit states under serviceability.

A. Strength and Stability

Methods for predicting the strength or stability of each structural member and connection are required. The predictors give the nominal strength of the component. Nominal strength is defined by assuming specified minimum material properties and specified geometrical properties. The nominal strength of each component must be greater than the factored nominal loads.

In the proposed curved-girder specifications, AASHTO is referred to for strength of components, such as shear connectors, bolts, and bracing members that are not affected by curvature. Strength of box flanges in compression is left unchanged from the Guide Spec.

Bending strength of flanges of I-girders presented special problems in development of the proposed curved-girder provisions. The original research had been performed on doubly symmetric noncomposite girders. Some test values were lower than those predicted by the equations.

There were two separate equations. The first equation predicted first yield of the section to be used on sections with noncompact flanges. The second equation predicted the strength based on the full plastic moment to be used on sections with compact flanges. Both equations gave only compressive stress values. Because the section was doubly symmetric, the tension flange was assumed to have approximately the same stress and did not need to be checked. The compact sections were defined as having web slenderness up to 150 and transverse stiffeners spaced at not more than the girder depth.

The proposed provisions need to give permitted stress levels for singly symmetric sections and composite sections. Additional assumptions are made to flesh out these needs. Web slenderness of transverse stiffened webs is limited to 150 and the stiffener spacing is limited to the girder depth. The slenderness of unstiffened webs is limited to 100. Compact flanges are not permitted with a longitudinally stiffened web where the web slenderness can be as high as 300. These requirements are more severe than those found in the current Guide Spec. Compact sections are defined as flanges having a limited width-to-thickness ratio and a single web in the center. These members are found on I-girders and the top flange of tub girders.

In the examination of the research, it was learned that the warping stress due to curvature was included in the development of the strength equations in both compact and noncompact flanges. One of the parameters that is required to evaluate flange strength is the ratio of the warping stress to the bending stress. If the warping stress due to curvature is included in this ratio, it is effectively double counted. It was learned that by ignoring the curvature warping stress in determining the ratio, much better correlation with test data can be obtained. The proposed curved-girder provisions reflect this finding.

The steel strength equations in AASHTO LRFD are similar to those in AISC LRFD. The lateral buckling equations in the 15th Edition of the AASHTO Standard Specifications resemble the equations in AISC. These equations are based on a strength of materials analysis that assumes that the cross section does not deform, regardless of its proportions. Earlier versions of AASHTO used a lateral torsional buckling equation based on the compression flange and part of the web resisted lateral torsional buckling, which assumes that the girder cross section may deform. It is this earlier equation that forms the basis of the curved I-girder strength predictors in the Guide Spec and has been maintained in the proposed provisions. Stresses in the tension and compression flanges are examined without regard to the entire section.

By ignoring the entire cross section, the proposed curved-girder specifications permit a unified approach to noncomposite and composite sections. Girders may be unsymmetrical and noncomposite for some dead load and composite for some dead and live load. If a doubly symmetrical, noncomposite girder bridge is designed, it is simply a special case and no separate provisions are needed. Stresses for noncomposite and composite cases are always additive. Thus, the section may be evaluated at the top and bottom at any time by using provisions that apply to the state of the flange at that time.

In the proposed curved-girder provisions, the rigidity of longitudinal stiffeners is increased due to the curvature effect. Such stiffeners are used to control bend-buckling and are subject to large bending stress. Their rigidity has been increased over that given in the Guide Spec or in AASHTO to a level similar to that in the Hanshin provisions (*9*).

The stability of the entire structure is not specified in AASHTO. In the proposed curved-girder provisions, stability is required during all stages of construction. The final structure is assumed stable. Unfortunately, the stability of curved girders cannot be related to an Eigenvalue problem because they start to bend laterally upon loading. Curved girders tend to translate prior to failure more often than do straight girders. There is no simple way to study this condition or to specify which section is adequate for given conditions. Large deflection theory could be useful in examining such behavior to ensure that internal lateral moments can be developed in the section to resist any lateral moment developed as the girder deflects. In this regard, box girders are much more stable than I-girders during erection.

B. Fatigue

Although fatigue is sometimes thought of as a serviceability requirement because the load factor is 1.0, it is actually a strength requirement. Fatigue design is addressed at the member and connection level. Nominal live loading is used. The ability to predict fatigue life for different details varies greatly because the scatter in test data varies with the detail. Thus, if mean values of life were used, the load factors would have to vary for each detail or the Φ factors would be greatly different. Instead, the Φ factor is built into the allowable stress range for each detail.

The fatigue design procedures of the AASHTO LRFD have been used in the proposed curved-girder provisions (*2*). This criterion uses a single vehicle. Application of the vehicle load is accomplished by considering the entire superstructure rather than a wheel load distribution factor.

As provided in the Guide Spec, the proposed curved-girder provisions limit slenderness of the stiffened web as a function of curvature to reduce through-thickness bending stresses at transverse stiffeners. This should reduce the likelihood of fatigue in the web at these welds.

The radial force component on shear connectors should be considered for fatigue in the proposed curved-girder provisions. The radial component is due to nonuniform torsion in the girder which may be due to curvature or other effects such as skew.

C. Serviceability

Deflection. The amount of energy contained in a girder is a function of deformation. As deformation of the girders within the structural system increases, its potential energy increases. If adjacent stringers deflect differing amounts, bracing between the stringers tends to minimize the difference in deflection, according to the law of potential energy. Minimization of relative girder deflection is desirable for the proper placement of the deck concrete and to ensure stability of the girders. It is accompanied by an increase in the bracing forces. Thus, as stringers become more flexible, bracing forces are very likely to increase. When the structure is skewed or curved, the bracing forces are further increased. Thus, controlling deflection tends to be more desirable for bridges with curved girders or skewed supports than for straight girders without skew.

AASHTO limits deflection through limiting girder depth as a function of span length. The limit was developed in the time of noncomposite girders and when the yield stress was usually 33 ksi. When composite construction was introduced, the minimum girder depth limit was *decreased* from $\frac{1}{25}$ of the span to $\frac{1}{30}$ of the span. By continuing to use the depth limits established for earlier designs, the deflection of newer bridges has been permitted to increase, and this increase often resulted in a concomitant increase in cross frame forces.

In the proposed curved-girder provisions, the preferred limit on the depth of the exterior steel girder has been increased to the original limit of $\frac{1}{25}$ of the span. Other girders are still limited to $\frac{1}{30}$ of the span. However, when the minimum specified yield stress is greater than 50 ksi, preferred girder depths are increased.

A deflection limit is an indirect manner of controlling cross frame or diaphragm actions and deck stresses. Since cross frame actions must be computed according to the proposed curved-girder provisions, it is possible that these actions can be controlled directly. In this way, shallower girders may be used on some bridges while even deeper girders may be required when these members are highly stressed such as with skewed supports. Such studied decisions by the designer are permitted in the proposed curved-girder specifications.

Live-load deflection is limited to ensure that the bridge is comfortable to users. The level of comfort is actually related to acceleration not the magnitude of deflection. The proposed curved-girder provisions limit acceleration through a deflection limit based on a computation of the frequency defined as the first flexural mode of vibration. This approach permits the desired behavior to be addressed directly by computation.

Lateral deflections occur in curved-girder bridges, causing problems during erection. Skewed supports are likely to exacerbate the problem. The provisions require that lateral deflections be considered for erection purposes.

Bearing rotation is associated with lateral movement. The Guide Spec requires that rotational bearings be provided in all cases. The proposed curved-girder provisions require that rotations at each bearing be determined and that a bearing designed for that rotation be provided. Through quantification of the rotations, less expensive bearings can often be used.

Overload. In certain instances, the maximum strength of the girder is defined in the inelastic range. The overload provision is provided to ensure that permanent set in the steel girders does not occur under expected live load. This loading condition is based on a dead-load factor of 1.0 and a live-load factor of $\frac{5}{3}$, when the design load is HS20 or greater. The elastic stress due to these loads is limited to $0.95F_y$ in composite girders and to $0.80F_y$ in noncomposite girders. These limits are based on tests of small bridges in the Ottawa AASHTO Bridge Test Program in the 1950s (*10*).

In the proposed curved-girder provisions, overload may control I-sections designed as compact. In this case, the girder is assumed to yield at its strength level. The overload limit must be applied based on the noncompact criteria, which may be rather severe because it limits the flange tip stress at brace points to the yield stress.

D. Constructibility

Each bridge must be built. Construction must be considered at the time of design if the most economical bridge is to result. It is common that composite girders are critical under noncomposite loading during construction. In the past decade, the AASHTO provisions have recognized the need to consider the construction condition during design. Requirements to investigate deck staging and noncomposite girder behavior have been added and other changes in this regard are being contemplated. It is now normal practice to specify the pouring sequence of the deck on the plans. The critical condition may be either the noncomposite or composite condition. To check the composite case, *stresses* from the two conditions must be added. Girder moments cannot be added because superposition does not apply when the bridge stiffness changes as the concrete cures.

The noncomposite section in the positive moment region has a compression flange that will be encased in concrete when the section is composite; but, for noncomposite dead load, the flange is usually braced only at cross frame locations. The neutral axis is much closer to the tension flange than it will be in the composite section; thus, more of the web is in compression. Often the noncomposite case controls design of the web and almost always controls the design of the top flange in compression.

The proposed curved-girder specifications recognize construction as a special limit state. However, critical stresses are determined for this state by using the same manner of equations as for other strength states. Dead-load stresses are limited to the noncompact criterion for I-girders to ensure that girders remain elastic under dead load.

If a longitudinal stiffener is used, web bend-buckling for both noncomposite and composite conditions must be checked. The provisions permit the location of the longitudinal stiffener with respect to the compression flange. Stresses for the sequential loading cases are accumulated.

During erection of the steel, the girders are subjected to self-weight with no, or minimal, torsional bracing. At this stage, the unbraced length often exceeds that permitted by the provisions. There is no method provided to check the stability of this condition. Instead, an analysis that considers large deflections may be required. This is beyond the specifics in the draft curved-girder provisions. Nevertheless, such analyses are recognized and permitted.

IV. DIVISION II-CONSTRUCTION

Construction provisions have been prepared to accompany the design provisions. The construction provisions are intended to augment those in the existing Division II, not supplant them. The reason for the additional provisions is to meet the needs that have become evident regarding the construction of curved-girder bridges over the past three decades.

Although horizontally curved girder bridges visually resemble straight-girder bridges, they are significantly different in structural behavior. Problems have been associated with erection of the steel and deck placement, in particular. These problems are manifest in their construction as well as in their design. To know the deformations and stresses in the completed curved-girder bridge, additional engineering attention is required compared to straight-girder bridges of similar span. The intended result of the new Division II is to further unify construction and design.

By providing explanation of the reasons for the requirements, the contractor is not limited in how the bridge will be built. The contractor is given the freedom to construct the bridge in any way that meets the assumptions made in the design. These requirements ensure that the bidding process will be as level as possible. A scheme of construction is provided on the plans, but the contractor is free to modify it in any way that is substantiated by engineering principles.

V. COMMENTARY

A commentary accompanies the provisions to provide the background and intent of the provisions. The research team hopes that the commentary will permit application of the provisions with confidence and lead to a willingness on the part of the designer to develop innovative designs that may go beyond the provisions but remain faithful to sound engineering principles.

VI. ECONOMICS

Another facet of structural design specifications is their reverse tendency with regard to economics. Normally, specifications are met to a minimum requirement because it is the least expensive way. In the case of design specifications, it is less expensive to exceed requirements in the design because it takes less engineering time and there is less risk of adverse consequences from an overdesigned structure. Because each bridge is unique, it is not possible to assign an expected cost of construction. So the owner finds it difficult to evaluate the efficiency of a design. One available means is to require that the plans meet a design specification that encompasses the desired attributes. Because engineering is an art requiring judgment, it is never possible to develop a perfect design specification.

There are several desirable attributes that a design specification should possess. The specification should provide common ground for both the owner and engineer. Clauses should be clearly written with a supporting commentary that provides the source of the provisions and their limitations and some guidance on their use. The design specification should provide for the application of engineering principles without excessive constraint. Rules of thumb should be carefully defined with regard to limitation of application and their source. There should always be latitude for the engineer to avoid the use of rules of thumb. The engineer may decide to design a structure to which the rules do not apply, or the engineer may find additional benefits from refined analysis techniques. Economic benefits, such as lighter girders, smaller connections, elimination of members, or evolution of a better framing or erection plan, may accrue through improved understanding of structural behavior.

A limitation may accompany design provisions so that design rules of thumb can be applied. For example, AASHTO requires that straight box-girder bridges have more than one box in the cross section. This limit is imposed because torsion is ignored in the straight box-girder bridge provisions and a single box is subjected to rather extreme torsion. The proposed curved-girder provisions permit a single straight box girder because torsion is considered. Thus, one restraint on economics imposed by a rule of thumb has been eliminated.

AASHTO also places limitations on the spacing of straight box girders to ensure that the wheel load distribution factors provided are appropriate. The curved-girder provisions require that the entire superstructure be analyzed. Live-load distribution need not be determined by using a rule of thumb but may be determined by analysis. Thus, any reasonable spacing is appropriate.

VII. SUMMARY

The proposed curved-girder specifications are based on sound engineering principles and the latest research. The proposed specifications are consistent with the AASHTO specifications in that material and loading provisions in AASHTO are referenced. Other provisions, such as splices and design of straight members, are simply ignored in the proposed specifications because those in the AASHTO specifications should be used. However, all provisions in the proposed specifications are self-contained in that no referencing to AASHTO is made within an article.

Construction is recognized as a limit state. Thus, designs are developed while recognizing the complications of construction. This step is logical when one recognizes that most of the problems associated with these bridges have been in their construction. Recognition of the importance of construction is further evidenced by the Division II provisions for curved girders that augment the construction provisions now in effect.

The research team has made every attempt to present the design and construction provisions in a logical and unified format. There is no reference to symmetrical or unsymmetrical sections. There is no reference to composite or noncomposite sections. Instead, flanges and webs are treated assuming the section is unsymmetrical. Sections may be either composite or noncomposite. If a flange is embedded in concrete, certain requirements are provided. Likewise, if the flange is not embedded in concrete, other requirements are provided. The requirements are the same regardless of the condition of the other flange. For example, the top flange of a tub girder is treated the same as if it were the top flange of an I-girder.

The webs of tub and I-girders are treated similarly. Webs are treated by assuming that they are in unsymmetrical sections, even if the steel girder happens to be symmetric. Even in these rare cases, the web will have to be treated as if it were part of an unsymmetrical section when the girder becomes composite. In the rare case when the girder does not become composite, it will simply be a special case of an unsymmetrical girder (i.e., symmetrical).

A unified methodology for computation of stresses is provided in the proposed specifications. The major deviation from existing AASHTO is the recognition of the difference of behavior of composite girders at service load from behavior at ultimate load. The composite girders are assumed to be uncracked for service load regardless of the sense of stress in the deck. At ultimate load, the deck is assumed cracked. The one deviation from this assumption is the design of shear connectors for strength when the deck is considered uncracked.

The proposed curved-girder provisions permit a wider range of structures to be designed, albeit with more effort on the part of the designer.

The proposed curved-girder provisions are an attempt to present a unified methodology for the analysis and design of horizontally curved girder highway bridges. The provisions are based on principles that are considered critical to well-designed bridges. The provisions attempt to present a unified means of loading, analyzing, and designing these complex structures. Because of their complexity, the engineer is called on to broadly apply the principles within the provisions when the rules have been exhausted.

VIII. REFERENCES

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APPENDIXES D THROUGH F

UNPUBLISHED MATERIAL

Appendixes D through F contained in the research agency's final report are not published herein. For a limited time, loan copies are available on request to NCHRP, Transportation Research Board, Box 289, Washington, D.C. 20055. The appendixes are titled as follows:

Appendix D: Recommended Specifications for Steel Curved-Girder Bridges and Commentary

Appendix E: Design Example, Horizontally Curved Steel I-Girder Bridge

Appendix F: Design Example, Horizontally Curved Steel Box-Girder Bridge