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Improved Design Specifications for Horizontally Curved Steel Girder Highway Bridges

FINAL REPORT

Prepared for
National Cooperative Highway Research Program
Transportation Research Board
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ABSTRACT

This report documents and presents the results of research on the development of improved design specifications for horizontally curved steel girder highway bridges.

The study commenced with a literature review of references that were collected and synthesized in a separate FHWA-sponsored Curved Steel Bridge Research Project. Twenty-two additional references, obtained during a fact-finding trip to Japan by a member of the Project 12-38 research team in December 1992, were also translated with the assistance of the FHWA and reviewed. Although this research was focused primarily on available information and technology, portions of the research were also devoted to improving on the current state-of-the-art where possible.

The Recommended Specifications adhere to sound structural engineering principles with minimal reliance on rules-of-thumb. The Commentary that has been prepared to accompany the new specifications explains the reasoning behind the provisions and provides some direction in their application.

The Recommended Specifications are divided into two divisions: Division I - Design includes a total of 13 sections plus references and Division II - Construction includes a total of 6 sections plus references. **Article 11.4.12** in Division II – Construction of the AASHTO Standard Specifications contains provisions for heat curving of steel girders. These provisions have not been repeated in the Recommended Specifications; however, their eventual inclusion in this document is suggested. In the development of the Recommended Specifications, every attempt was made not to repeat construction provisions given in the current AASHTO specifications. The specification provisions and

accompanying Commentary are presented in a two-column format in the Recommended Specifications for easy reference following a modern trend of specification presentation.

Two design examples have also been prepared to illustrate the application of the more significant design provisions.

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SUMMARY

The primary focus of National Cooperative Highway Research Program 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 consideration for adoption. The revised Guide Specifications were to be applicable to the design, fabrication and erection of horizontally curved steel I- and box-girder bridges.

Ten tasks were to be completed in this project. A detailed description of each task is given in Chapter 1. Six major deliverables, *a) I Girder Curvature Study, b) Curved Girder Design and Construction, Current Practice, c) A Unified Approach for Designing and Constructing Horizontally Curved Girder Bridges, Highlights of Major Changes, d) Recommended Specifications for Steel Curved-Girder Bridges and Commentary, e) Design Example, Horizontally Curved Steel I Girder Bridge, and f) Design Example, Horizontally Curved Steel Box Girder Bridge,* have been prepared and submitted. The deliverables are referred to in this report as APPENDICES A through F. However, as each deliverable is a standalone document, the documents are not duplicated in this report.

This report summarizes the development of the revised Guide Specifications and the accompanying deliverables.

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. Such an 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 utilize continuous horizontally curved steel girders.

Curved bridges using curved girders are usually characterized by simpler and more uniform construction details, since 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 ever-increasing right-of-way restrictions.

Curved girders are fabricated by cutting the flanges to fit the curve, by heat curving or by cold bending. The introduction of heat curving, along with associated fabrication aids, helped to increase the use of horizontally curved girders. Generally, two types of cross sections are used for horizontally curved steel girders: I girders (both non-composite 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 in addition to 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 directly 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 comparable 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 they may proceed as anticipated to ensure 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 problems that were not anticipated.

The *Guide Specifications for Horizontally Curved Highway Bridges* (*2*), hereafter referred to as the Guide Spec, were originally published in 1980 by the American

Association of State ·hway and Transportation Officials (AASHTO). Since that time, the Guide Spec has provided guidance to many engineers in the United States and Canada in designing horizontally curved girder bridges. However, there have also existed some significant deficiencies with the Guide Spec. The Research Problem Statement in the RFP for NCHRP Project 12-38 reads . . . "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 lead 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 preceding statement also mentions a lack of provisions in the Guide Spec related directly to the construction of curved bridges. Designers, builders and owners have expressed some disappointment with the Guide Spec because it does not specifically address a number of important construction and erection issues. Several states have modified the existing provisions based on 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 above-mentioned deficiencies in the Guide Spec; however, most of these attempts have been primarily editorial in nature. Without a major comprehensive overhaul, the primary deficiencies would have most likely remained intact.

B. Research Scope

The scope of 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 to AASHTO for consideration for adoption. 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 dated April 10, 1993, 10 specific tasks were defined by the Project Panel in order 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, 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 which 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 revised Guide 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 to be submitted not later than 8 months after initiation of the study. NCHRP approval of the interim report will be required before commencing Task 6. Two months will be required for review and approval of the 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, and reactions, and application of all provisions at critical sections throughout the bridge. Example problems shall cover aI- and box-girder multi-span continuous bridges with radial supports.*

Task 8. *Submit the draft specifications and example problems. Three months will be required for review by the NCHRP and AASHTO.*

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. The second draft and example problems will also be reviewed by the NCHRP and AASHTO and will also require 3 months to complete.*

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 dated September 4, 1992 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 U.S., an organized research effort on curved bridges had not been undertaken until the Pennsylvania Department of Transportation initiated an analytical study (Project 68-32) at Carnegie-Mellon University in February 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 U.S. without the aid of any official specifications related specifically to curved-bridge design. One of the earliest of these bridges was built in 1967 in Springfield, Massachusetts. This rather sharply curved bridge has two closed box girders in the cross section. The bridge was inspected recently and found to be in excellent condition after nearly 30 years of service. However, by present standards, the steel weight (approximately 90 pounds per square foot) is considered excessive.

Recognizing the complexity and the fact that there were no officially accepted design specifications for curved structures at the time, a comprehensive research project was initiated in October 1969. The project (FH-11-7389) was conducted under the auspices of the Federal Highway Administration with financial support from 25 participating state highway departments and was referred to as the CURT (Consortium of University Research Team) Project.

The consortium was composed of 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 co-workers at Carnegie-Mellon University studied the nominal bending strength, lateral stability, and local buckling of curved flanges and webs; Culver led 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 I and box 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 at about the same time. Many of their contributions were included in the Guide Spec and its Commentary.

B. Guide Spec

Based primarily on the work from CURT project, suggested tentative design criteria were assembled for both curved I girder 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.

About 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 at Washington University by Galambos and Stegmann (*13, 14*), based mainly on the work of Culver and McManus (*6*) and Mozer at Carnegie-Mellon University on the stability of horizontally curved members. The tentative Load Factor Design criteria for curved I girder 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 girder provisions and box girder provisions are separated into two parts and a Commentary is presented at the end of 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, Current Practice.

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-girder provisions and box-girder provisions are concatenated and one Commentary on both the I-girder 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, Current Practice.

C. Japanese Tests

In Japan, Nakai (*15-17*) performed tests on nine doubly-symmetric M-series I-shaped specimens. As in the case of 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 nearly pure negative bending with very rigid rotational restraints at the ends. Fukumoto (*18*) 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. Additional details on these Japanese tests are given in APPENDIX B, Current Practice.

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 titled *Guidelines for the Design of Horizontally Curved Girder Bridges (Draft)* (*19*). This specification was developed by the Hanshin Expressway Public Corporation in Japan and will hereafter be referred to as the Hanshin Guidelines. A copy of the Guidelines was obtained during a fact-finding trip to Japan by the principal investigator in December 1992 and a translation of the Guidelines was obtained in 1993 with the assistance of the FHWA.

The Hanshin Guidelines refer to the Japanese Road Association "Specifications for Highway Bridges" (*20*) for many of the more basic requirements and primarily contain only the provisions that are directly influenced by the effects of curvature. The provisions in the Guidelines are presented mainly in an allowable stress design format, as are the Japanese straight-girder design provisions. However, some of the provisions in the Guidelines 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 more detailed discussion of the background research that led to the formulation of the Hanshin Guidelines is given by Nakai and Yoo (*21*).

The Hanshin Guidelines do not distinguish between specification language and commentary. However, the Guidelines do suggest when either I girders or box girders should be selected. Box girders are divided into three major cross section types: single-cell mono-box with two webs, multi-cell 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.

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 feet. 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 (*2,22*).

Analysis is discussed at some length, but specific analysis 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 curvature and shallow curvature. Sharp curves may require that bearings be oriented to permit transverse thermal movement.

Expansion joints are to be 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 are to 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 the I Girder Curvature Study, APPENDIX A, conducted as part of this project.

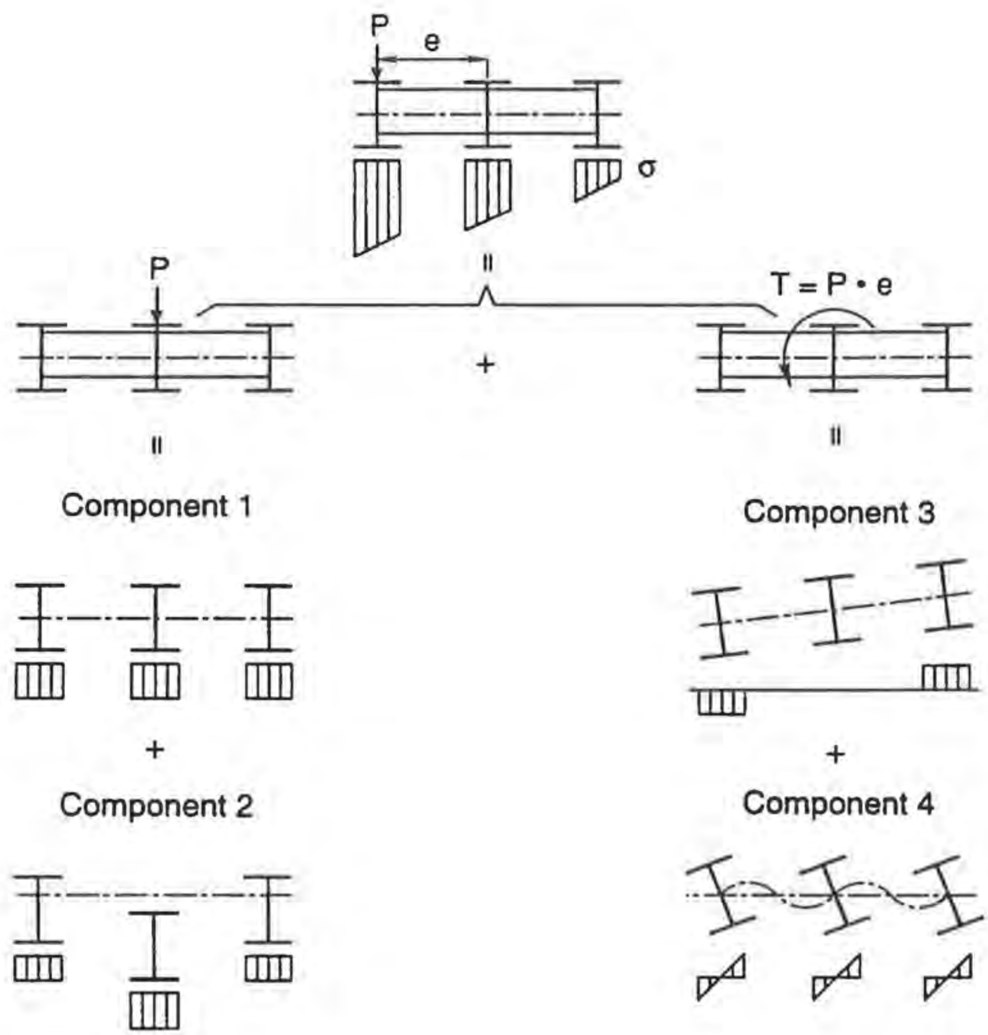

Figure 1 Four Normal Stress Components

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 (2-1).

$$\frac{f_b}{F_b} + \frac{f_\ell}{F_\ell} \leq 1.0 \qquad (2\text{-}1)$$

where f_b is the vertical bending stress, f_ℓ is the lateral flange bending stress, F_b is the allowable vertical bending stress considering any potential reduction due to lateral torsional buckling, and F_ℓ is the allowable lateral flange bending stress. It is noted that 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.

Hanshin uses a quadratic interaction relationship relating the normal stress and the shear stress for the box girder strength predictor, as shown in Equation (2-2).

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

where f_b is the total normal stress including stresses due to the vertical bending, lateral bending, and cross sectional distortion, f_v is the shear stress, and F_v is the 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 girder and box girder design examples presented in the Hanshin guidelines do not utilize 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 the NCHRP 12-38 Project, the Federal Highway Administration (FHWA) initiated a comprehensive multi-year research program on horizontally curved steel I girder highway bridges titled Curved Steel Bridge Research Project (DTFH61-92-C-00136). The planned 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 Interim Report I: Synthesis (*23*) that was prepared based on the work completed in this task, approximately

750 references were collected and stored in a Microsoft ACCESS database. Approximately 540 of these 750 references were deemed to be significant and were briefly reviewed by the investigator in order to document their most interesting highlights. Sorting out some of the more significant references on curved I girders into informative subcategories reveals the following distribution: Compression flange stability - 9; Web buckling - 12; Overall buckling - 16; Ultimate strength - 10; Cross bracing - 3. A significant number of these references have an academic rather than a practical slant. 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.

As can be seen from the above distribution, a majority of the references deal with the static analysis of curved bridges. There are 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, V-load Method) and refined methods (Finite Element Method, Finite Strip Method, Finite Difference Method, Analytical Solution to Differential Equations, Slope Deflection Method).

The Synthesis, which is probably the most comprehensive literature survey available on horizontally curved girders, was reviewed at length by the members of the Project 12-38 research team prior to initiating the development of the Recommended Specifications.

To properly examine experimentally the fundamental strength behavior of horizontally curved girders, it becomes highly 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, it was decided by the FHWA Project team 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 U.S. 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 later be reused in testing the bridge as a full-scale prototype.

In the first phase of the component test program, six non-composite 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, it is planned 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 are also members of 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, it is anticipated that portions of the Recommended Specifications developed in the NCHRP Project will later be updated based on data obtained from the various analytical and experimental studies being conducted in the FHWA Project.

CHAPTER 3 REVIEW OF PROJECT 12-38 WORK

A. Surveys of Owners and Fabricators

One of the ten tasks identified by the Project 12-38 panel was to conduct a survey of owners, designers, fabricators, and erectors of curved steel bridges in order to identify successful practices and problems that have occurred in design, fabrication, erection, and service.

In response to this request, a questionnaire was prepared by the project research team and was sent out to selected state bridge engineers. However, only a few responses were received and the information that was received was sketchy and not very informative. 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 that was received was sketchy and there was little or no information provided regarding erection experiences or problems. Thus, as directed by the Senior Program Officer for NCHRP, this task was abandoned.

B. I Girder Curvature Study

Although not included in the ten original tasks defined by the Project 12-38 panel, a need was identified to conduct a parametric study to determine appropriate rules regarding approximate analyses of curved I girder bridges. The Guide Spec currently 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. However, lateral flange bending effects must still be considered regardless of the curvature. The Hanshin Guidelines allow bridges with an subtended angle (calculated based on an effective span) of five degrees 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 (*24*) 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 (*25*) and in the new LRFD specifications (*26*).

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

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 degrees 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) 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 may be found in APPENDIX A, I Girder Curvature Study.

C. Curved Girder Design and Construction, Current Practice

One of the earliest curved I girder bridges was designed and constructed in Pittsburgh. The bridge was a three-span, two-girder structure with a radius of curvature less than 500 feet carrying California Avenue over Ohio River Boulevard. An early formulation of the "V-load" method (*27*) was used in the analysis. This 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-foot span and a radius of 477 feet. An I girder bridge carrying Route 65 over Pigeon Creek in Dalbarton, West Virginia was designed with spans of 65 and 72 feet and a radius of 127 feet. A simple-span I girder bridge in Minnesota was designed with a span of 104 feet and radius of 106 feet, with one abutment skewed approximately 45 degrees.

The economic benefits of horizontally curved girders become even more apparent when continuous multi-span curved structures became more common. Continuous curved girder construction has evolved to the point where continuous curved steel structures over 1,900 feet 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 feet and radii less than 1,000 feet. One unit of the Little Falls Bridge in New York has spans of 234-329-365-234 feet with a radius of 750 feet. This bridge has seven I girders in the cross section that vary in depth from 7 to 12 feet. One of the Shosone Dam Bridges in Glenwood Canyon, Colorado (completed in 1992) has spans of 219-274-216 feet and a radius of 955 feet. The bridge has four I girders varying in depth from 7 to 15 feet.

The cost to fabricate curved girders is typically somewhat 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 its own separate set of problems. Often additional falsework or temporary shoring is needed. In the case of cantilever construction on a multi-span alignment, insertion of the suspended span becomes problematic as the vertical camber and the rotation are always coupled.

The above is only a sample of the information on the current state-of-the-art in curved girder design and construction that has been assembled in APPENDIX B, Current Practice Report. that was prepared as part of this project. 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 contains a brief report titled *A Unified Approach for Designing and*

Constructing Horizontally Curved Girder Bridges Proposed AASHTO Specification (Highlights of Major Changes). This report was prepared to summarize the overall philosophy that was adopted in the development of the Recommended Specifications and to highlight some of the more significant changes that were introduced into the specifications. The report discusses the philosophy behind different specification formats and the development of the following basic specification components: 1. Loads, 2. Load Application, 3. Structural Analysis, 4. Load Effects, and 5. Limit States (strength and stability, fatigue, serviceability and constructibility). The development of each of these components in the Recommended Specifications is also summarized. The report 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. However, several of the provisions draw on other formats. For example, fatigue provisions are based on AASHTO LRFD *(26)*, 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 upon to broadly apply the principles when the provisions have been exhausted. It was felt by the research team to be the only means by 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 specification to explain the reasoning behind the provisions in order to help ensure their proper application. Both the specification provisions and the Commentary are presented in a two-column format similar to the format used in the AASHTO LRFD specifications.

E. Recommended Specifications

The following discussion highlights most of the significant sections of the Recommended Specifications. In particular, major deviations from current specifications, where they occur, are identified and the reasoning behind these deviations is discussed.

Division I - Design

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 feet and with radii greater than 100 feet. The provisions are limited to 300-foot spans because of the history of construction problems associated with curved bridges with spans exceeding 300 feet. Also, the approximations made in several of the formulas defining I girder strength and impact have not been studied for radii smaller than 100 feet. 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, in particular the effects of lateral flange bending and the effects 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 the AASHTO Standard Specifications for Highway Bridges, Divisions I and II, and the AASHTO LRFD Bridge Design Specifications. Where there is a conflict between the provisions in these specifications and the provisions in the Recommended Specifications, the provisions in the Recommended Specifications are to prevail.

A minimum of two I girders or one box-shaped girder is required in the bridge cross section in order to provide static equilibrium. Both closed box and tub girders are permitted. The earliest steel box girder bridges in the U.S. 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 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 explicitly cover the design of multi-cell box girders because there has been little published research in the U.S. regarding the design of these members. Multi-cell box girders have been used in the U.S. in the past, but are now uncommon. Although many different framing arrangements are permitted, girder-stringer-floor beam framing and a mix of I and box girders in the cross section are not treated, but these systems are not explicitly forbidden either.

Hybrid girders are not permitted in the Recommended Specifications because there has been limited research on curved hybrid girders. Girders having kinked alignments exhibit the same basic behavior as horizontally curved girders except that the non-collinearity 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 relatively 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, predominate over the provisions if a situation arises when 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, it is particularly important that basic engineering principles 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 AASHTO **Table 3.22.1A**. The strength limit state considers stability and/or yielding of each structural element. If the strength of any element, including splices anc onnections, 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 feet) be multiplied by a load factor of 0.75 when checking fatigue. An impact factor of 1.15 is applied to the live load.

NCHRP Preport 299 (*28)* suggests 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. NCHRP Report 299 does not attempt to quantify fatigue in long spans, continuous spans near inflection points and the importance of multiple trucks in adjacent lanes.

The FHWA report "Loading Spectrum Experienced by Bridge Structures in the United States" (*29*) addressed essentially truck weights measured by weigh-in-motion and not stress spectra causing fatigue damage. Thus, short spans were mainly 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.

Since neither Ref. 28 nor 29 address fatigue in long span bridges, the AASHTO

LRFD fatigue provisions do not appear to address them specifically either. 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 Spec 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.

Ref. 28 discusses 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." Ref. 28 computes 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 all 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 feet 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 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 0.80 F_y for non-composite girders and 0.95 F_y for composite girders.

At the request of the Project Panel, live-load deflections preferably are to be checked in the Recommended Specifications 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 one 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. NCHRP Report 380 *(30)* discusses 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 specifications do not recognize the state of stress in the concrete deck in determining the placement of the longitudinal reinforcement; the tensile strength of the concrete is ignored.

To address at least some of these issues, Article 2.4.3 of the Recommended Specifications requires that the one-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 inches, in order 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 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 explicitly 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.

Should curved girders not be fully shored to match the conditions assumed in the fabrication shop, the girders will begin to deflect and rotate immediately due to their own self-weight as they are erected until 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 accordingly 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. Should the accumulated deflections differ significantly from the dead-load deflections computed for the final condition, the Engineer may have to make a judgment on how to camber the girders to best 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 sequence of application of dead load be considered 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 are to be ignored for this relatively short-term condition.

The 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, 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 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 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 that are considered in the design 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 is to 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 is to 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 are to 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 one 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 to ensure control of permanent deflections (Articles 9.5 and 10.5).

The overload was developed from the AASHTO Road Tests (*31*) in Ottawa, Illinois in the late 1950s. It was 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 defined 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. It is of interest to note that using the standard wheel load distribution factors, two times a single lane loaded is about equal to 5/3 times multiple lanes loaded as follows:

Multiple lanes loaded with the normal live load

5/3 x S/5.5 x One Lane Moment = S/3.3 x Moment due to one lane

Single Lane loaded with two times the normal live load

2 x S/7 x One Lane Moment = S/3.5 x Moment due to one lane.

Thus, it is seen that the present overload is approximately equivalent to a single lane of two times the normal live load. At the time when the Load Factor Design provisions were written HS20 was the standard live load used in AASHTO. Thus, it is suggested that 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 for consideration 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 is to be taken from AASHTO **Article 3.14** unless overridden by the Owner. A footnote to **Table 3.22.1A** in the Standard Specifications permits a reduced load factor of 1.25 to 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 also be applied to the sidewalk load when the two loads are applied in combination; that is, when the vehicular live load is not permitted on the sidewalk.

Should the assumption be made that the vehicular live load is able to mount the sidewalk, the Recommended Specifications state that the sidewalk live loading 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** is silent on this issue.

3.5.6 Impact

Impact factors to be applied to vehicular live loads to account for dynamic amplification are specified in this article. The specified factors to be 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 Specifications. 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 seventy-five (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 are currently silent on the issue of 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 was deemed to be 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. However, in many instances, the specific number of cycles will not matter since fatigue details will be designed for an infinite life beyond the fatigue limit. However, should the design life of the detail be 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 are to 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 may 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. However, other load conditions may 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-degree Fahrenheit temperature difference between the deck and the girders is to 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 Concrete Segmental Bridges

(*32*). NCHRP Report No. 380 (*30*) discusses 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 current Guide Specifications, 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 a more rigorous approach is deemed to be necessary by the Engineer.

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 given 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, I Girder Curvature Study. The conditions given in Article 4.2.2 for closed box and tub girders are extracted from Ref. 33.

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. In essence, the Engineer must determine when approximate methods of analysis are appropriate for application to a given case. However, 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 *(34,35)* 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 Segmental Concrete Bridges (*32*) may be used to determine the effective width adjusted for shear lag.

4.5.2 Stresses in Composite Sections

This article deals in detail with the computation of stresses in composite sections; a subject which is given limited consideration in the current AASHTO 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 provisions state that positive live load moments are to be applied to the uncracked composite section at all limit states.

At the strength limit state only, negative live load moments are to 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 are to 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 is to be used for computing stresses due to both positive and negative superimposed dead load 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. It has been well established that concrete provides significant resistance to tensile stress at service load levels. Article 2.4.3 of the Recommended Specifications provides for 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 may 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. A recent paper by Brazegar and Maddipudi (*36*) addresses 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 (*34*) substantiates that stresses in the composite section are best predicted based on section properties computed assuming an uncracked section up to the overload level.

In addition to considering the concrete deck to be effective for computing flexural stresses at selected limit states, the provisions also consider the concrete deck to be 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 explicitly 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 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 feet or less, the effective width computed according to this requirement most always included all of the deck. With the ever-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 (*34*) reported that an entire 9.5-inch thick deck was fully effective in both positive and negative moment regions on a bridge with a 15-foot girder spacing. The AASHTO Guide Specifications for Segmental Concrete Bridges (32) 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 non-composite and composite sections be accumulated for checking all limit states. Thus, total stresses may not be determined by applying the sum of the non-composite and composite moments to the composite section. When computing the non-composite 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 nearly corresponding to this assumption.

Lateral flange bending stresses need not 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 non-composite 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 non-composite 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. Should precast deck panels be used, the Engineer may choose not to ignore the non-composite 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 specifications do not forbid these forms of construction, they are not dealt with directly either. The intent of this article is to enumerate several issues that should be considered when these forms of construction are used. Some of 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 mid-width to a vertical or inclined web. The flange can either be continuously or partially braced and can be either compact or non-compact. The strength equations in this article, as is the case in the current AASHTO specifications, were derived assuming a prismatic flange between brace points. Thus, changes in flange size should preferably 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 Footnote *c* to **Table 10.32.1A** of the AASHTO Standard Specifications should not be applied to the strength equations given in the Recommended Specifications.

The stress f_b in the strength equations is to 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 effects other than curvature is to 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. However, in the Recommended Specifications, the bending stress ratio need only be checked 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 0.5 F_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 non-compact 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 non-compact. The reason for this is that 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 earlier work done by Culver and McManus, 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 McManus and Culver. The maximum width-to-thickness ratio of 18 specified for compact flanges has been liberalized from the current value given 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 may not be considered compact when determining the critical flange stress.

Equation (5-1) for the critical flange stress is nearly identical to the equation from the Guide Spec. However, there have been some adjustments made to the equation based on further detailed study of the work done by Culver and McManus (6) under the CURT Project. In the equations for ρ_b and ρ_w, the term f_w has been replaced with the term f_m. f_m is defined as the factored lateral flange bending stress at the critical brace point *due to effects other than curvature.* Some of 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. However, this fact had not been made evident in the current Guide Spec. By setting f_m to zero, Equation (5-1) predicts lower strengths. This change causes Equation (5-1) to better predict the ultimate strength of curved test specimens. A more detailed discussion on this topic may be found in APPENDIX B, Current Practice Report. 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.

Equation (5-1) inherently recognizes the ability of sections with compact flanges to achieve full plastification under the combination of vertical and lateral bending. However, 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 Equation (5-2).

Equation (5-2) is retained from the current Guide Spec with one exception. For singly symmetric sections, the flange width, b_f, is now to be taken as $0.9\,b_f$ in the computation of the term λ. Equation (5-2) 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 (5-2) is only applicable when the critical stress computed from the equation is greater than $0.5\,F_y$, which is the typical case. For critical stresses less than $0.5\,F_y$, the Engineer may wish to instead consider using the classical Euler hyperbola to determine the critical stress because the parabolic shape of Equation (5-2) 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, Equation (5-3) was developed to address this shortcoming in a rational fashion. The limit on the combined vertical and lateral bending stress was derived based on the assumption that the flange is fully yielded under the combination of the vertical and lateral flange bending moments. The derivation of Equation (5-3) is discussed in greater detail on pp. 33-34 of APPENDIX A, I Girder Curvature Study. Based on this derivation, a small portion of the web is permitted to yield. Should the Engineer wish to prevent any yielding of the web, the factor of 1/3 in Equation (5-3) can be replaced by a factor of 1/2 (*37*). The critical average stress for a compact flange is taken as the smaller value from either Equation (5-1) or Equation (5-3).

5.2.2 Non-Compact Flanges

Non-compact compression flanges are partially braced flanges that are permitted to reach the yield stress at the flange tip without local buckling. Non-compact flanges must satisfy the width-to-thickness limitation given by Equation (5-4). 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 though when the flange slenderness exceeds 23 (*38,39*). During erection and deck placement when the compression flange is partially braced, the flange slenderness must satisfy Equation (5-4).

Equation (5-5) is essentially identical to the current equation given in the Guide Spec, except for an adjustment in the equations for the ρ factors to allow the length between brace points and the minimum radius in the panel to be input in units of feet rather than inches. Also, f_w has been replaced by f_m in the equations for ρ_w for reasons discussed previously. As reported in APPENDIX B, Current Practice Report, this change has the effect of increasing the predicted strength of curved girders with non-compact flanges and results in better correlation with the onset of initial non-linear behavior in curved test specimens. The flange tip stress is limited to the yield stress according to Equation (5-6). The critical average stress for a non-compact flange is taken as the smaller value from either Equation (5-5) or (5-6).

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 research 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 Equation (5-7) 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 Equation (5-8), which is the same equation that is applied to compact compression flanges.

Finally, it is specified that partially braced tension flanges meet the width-to-thickness requirement for non-compact compression flanges given by Equation (5-4) 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 Equation (5-4) 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 feet, 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. The permitted web slenderness increases linearly with the minimum radius in the panel according to Equation (6-1) up to a maximum slenderness of 150 at a minimum radius greater than or equal to 2,000 feet. 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 (6-2) 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 implicitly 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 is 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 given 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 Equation (6-3). Since 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 non-dimensional 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 Equation (6-7). 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 Equation (6-8). The calculated stiffener spacing d that is required to provide the necessary shear strength is to be used in computing k_w since the actual stiffener spacing, d_o, may be less than d for practical reasons. A similar requirement had existed at one time 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 (*19*). 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 U.S. 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 Equation (6-9). The bend-buckling coefficient is given by either Equation (6-10) or (6-11) 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 (6-10) and (6-11) 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 (6-10) and (6-11) were developed to allow the Engineer to compute the web bend-buckling capacity for any position of the longitudinal stiffener with respect to D_c. D_c is to 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.

Since 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 (6-10) or (6-11). Because simply supported boundary conditions were assumed in the development of these equations, it is conceivable that 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, or desirable, to use two longitudinal stiffeners on the web. Alternately, 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 is to be taken the same as the shear strength of a web panel without longitudinal stiffeners. As in 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 using the required stiffener spacing *d* in the panel, since *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* since 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 Equation (6-12) is essentially the same as the current requirement in the Guide Spec, only expressed in a non-dimensional 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 to be 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 Equation (6-9) is different for different conditions before and after the slab hardens. As a result, the determination of the optimum location is best left 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. Since the neutral-axis location does not vary greatly in negative bending regions before and after the slab hardens, it is suggested that an initial trial location of 0.4 D_c from the compression flange 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, it is also recommended that D_c for this case 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, I Girder Design Example.

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 in order to satisfy the analytical assumptions made in the theoretical development of the web bend-buckling strength given by Equation (6-9) and in the theoretical development of the rigidity requirement given by Equation (6-18), in which the longitudinal stiffeners were assumed to be restrained at their ends by the transverse stiffeners (*19,42*). 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. However, should the longitudinal stiffener be interrupted, 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 Equation (6-18) for the required moment of inertia of a longitudinal stiffener on a curved web panel was covered earlier as part of the discussion on the Hanshin Guidelines. The factor β in Equation (6-18) 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 Equation (6-12), which is the same requirement that is specified for transverse web stiffeners. The provisions also require the material grade of the longitudinal stiffener to be the same as for the web to which the stiffener is attached in order 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 commonly used bearing type. 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 are to 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 is to be used. For bearing stiffeners subject to significant eccentricities, it is specified that the stiffeners 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 is to be taken as 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 alluded 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. However, in this case, another load path should be provided through the steel since 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 1.35 F_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 Equation (6-12), 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 or lesser as 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. However, 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 are to 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.

As discussed previously, 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

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 here 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 is preferably to be uniform in this region. To satisfy equilibrium, the force P in the slab at the point of maximum live load moment is to be computed according to 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 Equations (7-3) and (7-4), respectively. The deck force given by Equation (7-4) is computed 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 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). However, in this case, the longitudinal slab force to be used in determining the resultant force P according to Equation (7-6) is to be taken as 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 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 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 times the full effective concrete width given in Article 4.5.2 times 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 Equation (7-10).

7.2.2 Fatigue

The required shear connector pitch for fatigue is to be computed according to the provisions of AASHTO LRFD **Article 6.10.7.4.1b**. According to the Recommended Specifications, the required pitch is to 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)** is to be computed from 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. A vector addition of the two shear range values is therefore conservative because the maximum longitudinal and radial shear ranges are not usually produced by concurrent loads.

Recall that 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 in order 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 above, 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 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 inches, except at simple supports where the value of *w* is halved. 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 Equation (7-13) is to 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 are to 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 and are to be distributed uniformly across the box flanges, as specified in Article 10.4.3.5. The shear connectors are also to 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 conservatively contain an equal number of shear connectors. Because of the conservatism inherent in the above requirements, the radial fatigue stress range given by either Equation (7-12) or Equation (7-13) need not 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 are to 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 are also to be considered in the design of the bearing device and superstructure. The provisions permit the superstructure to be supported either on bearing devices or though direct attachment to the substructure or pier cap beam as an integral connection.

8.2 Forces

Bearing devices are to 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, which 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 are to be designed to accommodate movements due to temperature changes in the superstructure and to accommodate rotations about 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 often do not allow the bearings to be oriented in such a manner, however.

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 since 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, 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 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 either be welded or rolled shapes. The webs must be attached at the mid-width of the flanges and the flanges can either be compact or non-compact.

This article also specifies recommended minimum flange proportions, which are not given in the current AASHTO specifications. Compression-flange widths are preferably not to be less than 0.2 times the girder depth, but in no case less than 0.15 times the girder depth. Flange thicknesses are preferably not to 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 their study was limited to doubly-symmetric girders, the recommended minimum flange proportions are deemed to be adequate for reasonably proportioned singly-symmetric I girders. The advent of composite design has 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 and 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 are to be considered primary members, are to contain both top and bottom chords, and are preferably to 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 and 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 is to be taken as 1.0 for single-angle members and is preferably to be taken as 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 having 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 have been reflected in the ASCE Manual No. 52 (*46*) covering 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 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 are to be designed according to the Load Factor Design provisions of **Article 10.54** of the Standard Specifications. Bracing members in tension are to be designed according to the provisions of **Article 10.18.4** of the Standard Specifications using the effective net area defined in **Article 10.9**.

Solid-plate diaphragms with span-to-depth ratios greater than 4 can be designed using 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 is not to exceed 30 feet. This limit is reduced to 25 feet 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 preceding requirement ensures that the maximum spacing does not exceed the maximum spacing limit of 25 feet 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 degrees 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 degrees 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 degrees. 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. However, there is 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 determined from the approximate Equation (4-1) given 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 are to be provided to support the deck and expansion devices. However, 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 a row of cross frames along an interior skewed support line to be eliminated is a reflection of modern practice and simplifies detailing since 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 are to be designed for the load effects due to the factored loads as determined from an appropriate analysis that comprehends 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 to be 48 inches, except at simple supports where w is to be taken as 24 inches. 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. However, when the skew is large, 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 in that 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 themselves 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, it may be desirable to 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 0.95 F_y and 0.80 F_y at composite and non-composite 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 (*31*), in which permanent sets in composite and non-composite 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 non-compact flanges given by Equation (5-5) 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.

Since these checks fall under the serviceability limit state, the Recommended Specifications state that the uncracked section is to be used to compute the bending stresses in composite sections for reasons discussed previously. 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** are to apply, except as modified in Articles 2.3 and 3.5.7 of the Recommended Specifications. The uncracked section is to 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 since the welds are near the mid-width 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 this high torsional resistance and because of 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-shaped 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, as discussed earlier.

For curved structures, warping or lateral flange bending stresses are less in box girders once the section is closed since 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 led to the development of 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. Since 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 are to be designed according to the provisions of Article 6 of the Recommended Specifications. At all limit states, the web shear is to be taken as the sum of the flexural and torsional shears. Since 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, it is specified that both webs 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 superelevated, it is preferable to rotate the entire cross section into the superelevated position to match the deck slope in order 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 acts as a uniformly distributed transverse load on the girder flanges and should be considered.

On tub girders, top flanges are to be braced with lateral bracing 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 are to 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 is preferably to 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 since 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 should uplift be 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 are to 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 is to 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 are not to exceed the critical stress given by Equation (10-2), which is based on the von Mises yield criterion for combined normal and shear stress and is discussed in more detail in a later section.

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 not believed that box girder bridges utilizing concrete diaphragms have yet been constructed in the U.S. 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 recognized in the design of 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 consisti g 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.

It is specified in the Recommended Specifications that the spacing of the internal bracing be 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 so determined is not to exceed 30 feet. By limiting the longitudinal warping stress to 10 percent of the longitudinal stress due to vertical bending, warping need not be considered in the design of the flanges at the strength limit state. Also, the full torsional stiffness of the box can be considered 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 has proven to be quite problematic for determining through-thickness transverse bending stresses. Transverse stiffeners that are attached to the web or box flange should be considered 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 may also need to 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 is not to 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 need not be continuous through the box webs. At internal supports and at intermediate locations between supports, external cross frames or diaphragms may also be used. At all external brace locations, there must be opposing bracing placed on the inside of the box to receive the forces.

Permanent external bracing at locations other than support points is usually not necessary. If an analysis indicates that the boxes will twist excessively when the deck is cast, temporary external bracing may be desirable. However, the effects of removal of the temporary bracing need to 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 are to be computed and are not be less than the critical stress given by Equation (10-2), which is discussed in more detail in a later section.

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 non-composite section at the constructibility limit state. Once the concrete deck hardens, the bracing is less effective since the deck usually has a much greater shear stiffness.

The bracing should preferably be connected to the top flanges, or else 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 non-composite 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 preferably should 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 unauthorized persons, birds and vermin.

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 is to 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 is to 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 is also to be considered. It is recommended that the inclination of the webs 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 in order to simplify the fabrication and the analysis.

10.4 Strength of Box Flanges

10.4.1 General

This article defines the critical stress for non-composite and composite box flanges. A box flange is defined as a flange that is connected to two webs. A box flange can either be 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 is not to exceed the critical stress defined in this article, unless shear lag is considered, in which case the longitudinal stress at the webs is not to exceed the critical stress.

10.4.2 Non-Composite Box Flanges

10.4.2.1 General

Non-composite top flanges of closed box sections are to 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 non-composite top or bottom box flanges under self-weight and applied permanent loads is not to exceed 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 non-composite box flanges not exceed 20 ksi, which is a limit retained from Article 1.27(B) of the Guide Spec. To satisfy the above 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 may be considered to be effective with the diaphragm. This requirement is similar to the provision for the width of web or diaphragm that may be considered to be effective with bearing stiffeners discussed previously. 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) (discussed below) 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, as discussed earlier. The torsional shear force computed from the shear flow is to be combined with the shear force due to bending. The critical shear stress, F_v, for a non-composite box flange is given by Equation (10-1). In the Guide Spec, a critical shear stress equal to the shear yield stress, or $F_y/\sqrt{3}$, is permitted. However, at this level of shear stress, there is also a significant reduction in the permitted average longitudinal flange stress. Therefore, to prevent this reduction and to 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 non-composite 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. Since instability or buckling is not a concern for tension flanges, the critical stress can be determined on the basis of yielding. Also, since 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 non-composite curved box flanges in the Recommended Specifications is identical to the critical compressive stress specified for non-composite 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 to be input into the equations 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 defined as 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 non-linear 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 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 gi\ by Equation (10-4). In-between these two regions is a transition region which reflects the fact that partial yielding due to residual stresses and initial imperfections does not permit the attainment of the elastic bucking 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 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 to be taken as 4.0 and the plate-buckling coefficient k_s for shear stress is to be taken as 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 Equation (10-4) or Equation (10-6) is to be used 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 Equation (10-6) or Equation (10-8) is to be used is based on the factor R_2 given by Equation (10-7). $R_2/\sqrt{F_y}$ is defined as 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 non-composite unstiffened box flange in compression becomes wider, the critical flange stress decreases to a level that soon becomes impractical. At some point, it becomes beneficial to add longitudinal flange stiffeners. The critical compressive stress for a non-composite 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 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 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 may indeed 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. It should be emphasized that 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 directly 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 taken equal to 1 and k is taken 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, 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 either be 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 either the Guide Spec or the Standard Specifications. Top composite box flanges are rare in the U.S. 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 up to now, it was felt that this concept could be economically viable in certain situations and that provisions should be 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 are to be computed using the uncracked section at all limit states. Flexural and shear forces are to be proportioned to the steel and concrete using the appropriate section properties. Creep is to be conservatively ignored in determining the dead and live load flexural and shear stresses in the concrete, but creep is to 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 is to be designed as a non-composite box flange according to the provisions of Article 10.4.2.

10.4.3.2 Tension Flanges

Longitudinal tensile stresses in the steel are to be computed using the uncracked section at the fatigue, constructibility and serviceability limit states. At the strength limit state, the tensile stresses in the steel are to be computed using the cracked section. The computed tensile stresses are not to 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 are not to 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 is not to exceed $0.85f_c'$.

10.4.3.4 Shear

The steel flange is to 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 are to 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 is also to be increased to satisfy the requirements of **Article 10.38.5.1.3** of the Standard Specifications. Finally, the shear connectors are to be checked for fatigue according to Equation (7-11), where F_{fat} is to be taken as 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 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 non-composite 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 is to 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 previously under Article 10.2.2.3.

Cross bracing and diaphragm members and their connections must also be checked for fatigue. The fatigue loading is to 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 since 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 are to essentially 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, are to 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** (and discussed below). 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 are to 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 strength of the member. 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 are to 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 are to 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 employed. Box flange splices and web splices are to be designed 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 are to 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 is to 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 else, 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 size 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 size bolt holes should preferably be used to ensure that the steel will fit together in the field. Oversize 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. However, it is strongly recommended that the use of oversize 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 is to 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 are to 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 non-composite 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 non-composite 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 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 non-composite bridges constructed using steels having a specified minimum yield stress of 50 ksi or less. These non-composite 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 weight and concrete weight are to 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 are to 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 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 is not to be included when making this deflection check. The deflection limit is to 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 Spec 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 No. 1 is a three-span continuous bridge with radial supports and average span lengths of 321 – 416 – 321 feet. Span 1 is curved with a radius of 500 feet measured at the longitudinal centerline. Spans 2 and 3 are tangent. The out-to-out width of the concrete deck is 48'-3". There are four girders in the cross section spaced at 13'-6". A 5'-0" wide sidewalk is on the outside of the curve. The girders are 14'-0" 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 No. 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.

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

In the tangent Span 3, which is furthest from the curved span, the deflections are more nearly 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, it is interesting to note that 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 feet and radial supports. The radius is 886 feet measured at the longitudinal centerline. The out-to-out deck width is 30'-6" with the girders spaced at 11'-6". There are no sidewalks and the design live load is HS25. The girders are 7'-6" 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 No. 2 for the case of the design live load are for two-lanes loaded.

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

Bridge 3 is a horizontally curved simple-span bridge with a span of 169 feet, a radius of 275 feet measured at the longitudinal centerline and skewed supports. The out-to-out width of the concrete deck is 36'-4". There are six girders in the cross section spaced at 6'-0". The girders are 5'-8" deep and the design live load is again HS25. Girder G6 is defined as the outermost convex girder. The governing deflections reported for Bridge No. 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 feet and a radius of 1,300 feet measured at the longitudinal centerline. The out-to-out deck width is approximately 83'-8" and there are seven girders in the cross

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

section spaced at approximately 12'-8". The girders are 6'-5" 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) are 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 feet and a radius of 642 feet measured at the longitudinal centerline. The out-to-out width of the concrete deck is 32 feet and there are four 7'-6" 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 No. 5 for the case of the design live load are for two-lanes loaded.

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. 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 for any steel element subjected to compression during construction, the critical buckling stress. As specified in Article 3.3, a load factor of 1.4 is to 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 non-compact 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 non-compact 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 by Articles 5.3 and 5.4, respectively. Critical stresses for non-composite box flanges during construction are given by Article 10.4.1. Finally, as mentioned previously, the slip resistance of bolted joints is to 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 $0.85f'_c$, with f'_c defined as 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 Recommended Specifications, if the tensile stress in the concrete at the constructibility limit state exceeds 0.7 times the modulus of rupture, a minimum of one-percent longitudinal reinforcement must be provided in the deck at that location for crack control. Since tensile stresses are limited and since construction loads are relatively short-term, stresses in composite sections at the constructibility limit state are to 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 are also to be determined for each critical stage of the construction.

13.5 Shipping

Field sections are to 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.

13.6 Steel Erection

The erection scheme provided on the Construction Plan is to denote a sequence of erection, which will insure 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 inform 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 upon removal of the supports must be furnished so that the proper stroke is provided to permit removal of the jacks at the supports. Provision 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 is to be sized and the design of the required connections is to be provided. Should temporary bracing be 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 are not to 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 may 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 are to be evaluated for the anticipated redecking sequence; that is, 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 three- to four-foot 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 Ref. 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. 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 equal panel lengths between brace points and that distributed lateral bracket loads on the flanges are uniform. Lateral flange bending stresses resulting from these moments are to 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 to then be used in Equations (5-5) and (5-7) 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 is to 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.

Division II - Construction

1. General

The Division II requirements for construction in the Recommended Specifications are to 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 are to be used by the Contractor to prepare a Construction Plan.

The Construction Plan shown in the Design Plans must indicate one possible means 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 plan shown in the Design Plans or may be developed independently by the Contractor. This Construction Plan is to 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. Should the Contractor's plan result in a change to the dead-load camber of the girders from that presented on the Design Plans, 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 directly 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 is to 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 is to 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 by either heat curving or by 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 is to be performed in accordance with the provisions of AASHTO **Articles 10.15.2** in Division I and **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 feet 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 is to 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 using either a press or a three-roll bender and is to 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 either be cut 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 since adjacent flanges would have nearly the same radius. Otherwise, it would be necessary to re-burn each flange with concomitant material loss. Vertical camber is to 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 to transfer lateral cross frame forces directly to the flanges, connection plates must be attached to both flanges by welding or bolting. A welded connection is not to 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 is to 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. Since 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 lying 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 is to 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 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 insure the proper location of anchor bolts. According to the Recommended Specifications, girder lengths are to be measured along the arc based on an ambient temperature of 68 degrees Fahrenheit. If girder lengths or anchor bolt locations are surveyed with a laser instrument, it is important that compensations be made 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 are to be preassembled. For multi-stringer bridges, such preassemblies are normally used to check the fit-up of field splices. For structures longer than 150 feet, the assemblies are not to be less than 150 feet 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 is also to be supported during shipment so that dynamic lateral flange bending stresses are controlled. To control vibrations during shipment, single unbraced I girders are preferably no to 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 5Hz. 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 is to 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 method of erection differing from that shown in the Construction Plan are to be accounted for by the Contractor according to the provisions of **Article 11.6.4.2** in Division II of the Standard Specifications to ensure that the integrity and load capacity of the completed bridge is maintained.

Bolted girder splices are to be field assembled according to the provisions of **Article 11.6.5** in Division II of the Standard Specifications, which ensure 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, is to be designed for the vertical and lateral loads specified in the Construction Plan. The elevation of the falsework is to 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 in order to minimize twisting of the steel and any deleterious stresses that might be induced.

4.3 Bearings

Bearings are to 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. It is recommended that unbraced lengths of curved I girders 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. This can be accomplished by the use of 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 either be 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 – are to 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 is responsible to provide an additional analysis for the new overhang loads that is 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 are to preferably be attached to the top flanges and bear near the bottom flange. Should it be necessary for the overhang brackets to bear directly on the web, the Contractor's Engineer must ensure that adequate precautions are taken 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 are to 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 are also to 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.

F. 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, I Girder Bridge Design

Example. The bridge has spans of 160-210-160 feet measured along the centerline of the bridge. The radius of the bridge is 700 feet at the center of the roadway. The supports are radial with respect to the roadway. The out-to-out deck width is 40.5 feet with the four girders in the cross section spaced at 11 feet. Live load is HS25 for the strength limit state and structural steel with a specified minimum yield stress of 50 ksi is 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.

G. 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, Box Girder Bridge Design Example. Like the preceding I girder example, the bridge has spans of 160-210-160 feet measured along the centerline of the bridge. The radius of the bridge is 700 feet at the center of the roadway. The supports are radial with respect to the roadway. The out-to-out deck width is 40.5 feet, with the webs of each trapezoidal tub girder spaced 10 feet apart and with a clear deck span of 12.5 feet 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 is 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 inches 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 the NCHRP to update and clarify the provisions of the existing Guide Specification, 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 the state of the art which included an holistic review of the literature, existing guidelines, laboratory testing, analytical studies and experience with the 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 which 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 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 on the Plans. In fact, the provisions of Division II - Construction require that the contractor must 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 directly in the AASHTO for composite bridges previously, it is believed important for curved girder and skewed bridges which are subjected to higher torsional shears than are tangent non-skewed 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 recognizes that construction sequencing and moving loads require a broader definition of negative moment regions.

The researchers have proposed provisions which define the overload live load specifically as two times an HS20 truck or lane loading placed in the critical location in a single lane. This approach is shown to be consistent with the original overload live load provided in the AASHTO Load Factor Design provisions. It is suggested that the effect of a single heavy truck(s) is more critical and should be considered in 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 is shown to have been 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 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 shown to be unconservative 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 were either 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. However, 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 which can be applied to a wider variety of bridge sites. For example, the provisions are limited to a maximum span of 300 feet. 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 an obvious 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. It is desirable that a more unified approach to I girder strength 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. It is likely that 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 U.S. 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 bridge at ultimate load has 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.

Clearly, 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

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APPENDIX B

Curved Girder Design and Construction, Current Practice

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APPENDIX C

A Unified Approach for Designing and Constructing Horizontally Curved Girder Bridges

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APPENDIX D

Recommended Specifications for Steel Curved-Girder Bridges and Commentary

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APPENDIX E

Design Example, Horizontally Curved Steel I Girder Bridge

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APPENDIX F

Design Example, Horizontally Curved Steel Box Girder Bridge

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