Historical Perspective on Horizontally Curved I Girder Bridge Design in the United States

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Abstract: This paper provides a historical overview of the development of horizontally curved steel I girder bridge design specifications in the United States. The background to the development of curved I girder design and analysis provisions in the 1993 and 2003 *AASHTO Guide Specifications for Horizontally Curved Highway Bridges* is discussed, and the status of recently completed and ongoing curved steel bridge research projects is summarized. In addition, the manuscript focuses on how the 2003 *Guide Specifications* can be used to analyze and design horizontally curved I girder bridges. Comparisons are made to prior provisions in the 1993 *Guide Specifications*.

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Introduction

Horizontally curved bridges continue to occupy a growing share of the United States bridge market. These structures, which constituted approximately a quarter of the steel bridges being constructed in the early 1990s [Structural Stability Research Council (SSRC) 1991], are often one of the few viable options at complicated interchanges or river crossings where limited site space or pier locations are available. Horizontally curved steel bridges also offer aesthetic and cost benefits over more traditional chorded structures that make their selection attractive even when site restrictions are not an issue. They are invariably shallower than curved concrete bridges, which can often result in shorter approaches or fewer retaining walls when compared to structures containing deeper girders. Horizontally curved steel bridges can also be skewed to meet site demands.

Given the benefits realized with the design and construction of horizontally curved steel bridges (e.g., reduced number of substructure units and length of deck overhangs; increased spans and traffic sight distances) and the continued decrease in available land space for new and replacement structures, the use of these bridge types is certain to increase. However, there is considerable additional complexity associated with their analysis, design and construction compared to that for typical straight bridges.

There are two general types of horizontally curved steel bridges. The box or tub girder is able to resist significant torsion if its shape is maintained with adequate internal bracing. This type of bridge was initially popular for that reason. The earliest box girder bridges had closed box sections (i.e., steel on four sides with a composite or noncomposite concrete deck). Occupational Safety and Health Administration work rules soon made it impractical to fabricate closed boxes in the United States and, therefore, tub girders with top lateral bracing replaced closed box girders.

Curved I girders are perhaps more commonly used for horizontally curved bridges. These members have very little torsional stiffness and are stable only when connected to other girders using cross frames or diaphragms. This manuscript addresses these types of bridges.

Curvature of the superstructure leads to combined bending and torsion in the girders, significant forces in diaphragm and bracing members, and considerably more interaction between components within the structural system than experienced in straight bridges with orthogonal (or nearly orthogonal) support lines. These effects must be accounted for in the analysis and design to ensure that the bridge components are proportioned properly. In addition to the extra complexity associated with the nature of curved bridges, the current American Association of State Highway and Transportation Officials (AASHTO) *Guide Specifications for Horizontally Curved Highway Bridges* (AASHTO 1993), hereafter referred to as the *1993 Guide Specs*, are generally perceived as being difficult to use. Also, these Specifications do not address a number of important design and construction issues, such as methods of preliminary analysis and steps for curved girder erection and fit-up. These omissions, coupled with expensive claims and lawsuits on some past projects, have led a number of states to either modify the Specifications or limit the use of curved steel bridges to specific spans or methods of erection (HDR 1995). Recently completed work (Hall et al. 1999) has produced an updated version of these Specifications (AASHTO 2003) that is expected to be published in 2003. These new Specifications are referred to in this manuscript as the *2003 Guide Specs*. The *2003 Guide Specs* have addressed some of the omissions within the *1993 Guide Specs*. Ongoing research into the behavior of horizontally curved I girder bridges is addressing further improvements, leading to the eventual development of curved steel bridge design provisions for Load and Resistance Factor Design (LRFD).

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This paper provides a historical overview of horizontally curved steel I girder bridge design specifications in the United States. It discusses: (1) background behind the development of the *1993* and *2003 Guide Specs*; (2) how the *2003 Guide Specs* can assist with the design of horizontally curved I girder bridges; and (3) the status of recently completed and ongoing research projects focusing on curved I girder bridges. It should be noted that at the date of final submission of this paper (January 2003), a final version of the *2003 Guide Specs* has not been published and there may be changes in the Specification that have not been discussed herein. Brief summaries of past and current research related directly to the development of AASHTO curved I girder specifications are provided. The reader is referred to other publications (Zureick et al. 1994; Hall et al. 1999; Linzell 1999; Zureick and Naqib 1999; White et al. 2001) for more detailed discussions of research in this area.

Background

Prior to the 1960s, minimal design and construction of horizontally curved steel bridges occurred. Curved steel girders were utilized only if using a chorded structure proved to be unfeasible, and these types of girders were designed without the aid of any guidelines or specifications. Despite the lack of specifications, engineers began to recognize the advantages associated with curved structures, and curved steel I and box girder bridges were being designed with increasing regularity in the 1960s.

The need to ensure uniform minimum standards of practice and safety for this structure type led directly to efforts to develop design guidelines that would put curved girder bridges on the same footing as more traditional highway bridges designed according to AASHTO Specifications. Specifically, this need led to creation of the Consortium of University Research Teams (CURT) project, a large-scale research project funded by 25 states and managed by the Federal Highway Administration (FHWA), in 1969. The consortium: (1) reviewed all existing publications on curved bridges; (2) conducted experimental and analytical research to augment existing information related to curved girders; (3) incorporated research results from ongoing state agency sponsored projects; (4) developed simplified analysis and design methods with accompanying aids and computer programs; and (5) correlated proposed analysis and design methods and procedures with existing analytical and experimental data.

Research performed by the CURT project centered on a series of scale model laboratory tests accompanied by theoretical work and analytical studies. I girder tests examined the behavior of single girders and girder pairs (Mozer and Culver 1970; Mozer et al. 1971, 1973) and studied their interaction with bracing members and adjacent girder lines in representative curved bridge systems (Brennan 1970, 1971, 1974). Small-scale system tests studied the level of interaction between adjacent curved girders and the role that bracing members played in that interaction and validated a proposed three-dimensional analysis method and corresponding computer code.

Theoretical and analytical work completed for the CURT project focused on the development of empirical models for predicting: (1) overall strength of doubly symmetric curved girder I sections in bending (McManus 1971); (2) local buckling behavior of curved girder flanges (Nasir 1970); and (3) behavior of web panels in flexure (Brogan 1974; Culver et al. 1972, 1973). From these experimental and mathematical studies, the CURT project team, in conjunction with the sponsoring states, developed a tentative set of Specifications for allowable stress design (ASD) of curved girder bridges (Culver 1972; CURT 1975).

The Task Committee on Curved Girders of the ASCE–AASHTO Committee on Flexural Members reviewed the proposed Specifications. They also considered additional curved steel bridge experimental and analytical studies that were completed concurrently with the CURT project (Heins 1972; Mondkar and Powell 1974). This work was combined with the CURT specifications and proposed by the Task Committee as guide specifications that were accepted by AASHTO in 1976 (Armstrong 1977).

Load factor design (LFD) criteria were added to the guide specifications through a research project sponsored by the American Iron and Steel Institute (AISI) in the mid-1970s (Stegmann and Galambos 1976). This project transformed the ASD criteria proposed by the CURT project team to an LFD format similar to that available in the 1973 *AASHTO Standard Specifications for Highway Bridges* (AASHTO 1973), hereafter referred to as the *1973 Standard Specs*. The LFD criteria were developed directly from the proposed CURT specifications.

The above LFD criteria were adopted by AASHTO and added to the ASD criteria to form the first edition of the *Guide Specs* (AASHTO 1980). These specifications were divided into two parts, Part I for ASD and Part II for LFD. Since their initial publication in 1980, eight interim revisions of the *Guide Specs* have been published and a second edition was published in 1993. While there have been a number of revisions since its initial publication, the document issued in 1980 is largely unchanged in its fundamental content.

Some U.S. research into the behavior of horizontally curved steel bridges continued after initial publication of the *Guide Specs* (e.g., Yoo and Carbine 1985). However, concerted efforts to improve upon these specifications were initiated in the early 1990s, after Task Group 14 of the Structural Stability Research Council on Horizontally Curved Girders published a report outlining problems associated with the Specifications in their current form and proposed areas for research (SSRC 1991). This report provided the impetus for a number of research projects aimed at developing updated and improved curved girder design specifications.

The first project, referred to here as the Curved Steel Bridge Research Project (CSBRP), was initiated by FHWA in 1992. Its goals were to: (1) collect and disseminate all curved bridge research completed in the U.S. and abroad; and (2) experimentally and analytically address the behavior of curved I girders in bending, shear, and combined bending and shear. This project also aimed to address curved bridge constructability issues. To date (January 2003), this project has tested a number of full-scale curved I girder specimens in single and realistic multi-girder configurations, and has performed an extensive number of computer simulations using sophisticated finite element models. An extensive number of publications have been produced from this research (Zurieck et al. 1994; Linzell 1999; Zureick et al. 2000; Jung and White 2001; Zureick et al. 2001).

Also, the National Cooperative Highway Research Program (NCHRP) initiated a project in 1993, NCHRP 12-38, to develop a set of improved specifications, hereafter referred to as the *Recommended Specs* (Hall and Yoo 1998). This project was aimed at improving the LFD and construction of curved steel bridges based upon current practice and technology. However, the project scope did not include the execution and/or incorporation of new research. The results of this project are published in NCHRP Report 424 (Hall et al. 1999). This report contains: (1) an overview of past and ongoing curved bridge research; (2) discussions of current U.S. curved bridge design and construction practices; (3) an overview of the *Recommended Specs* (Hall and Yoo 1998) for LFD and construction of curved I and box girders; and (4) recommendations for future research. Major changes between the proposed and the *1993 Guide Specs* are highlighted and discussed. The key products of NCHRP 12-38 are the *Recommended Specs*, an accompanying commentary, and design examples. The AASHTO Bridge Committee adopted these products, with minor modifications, in 1999 as the new *Guide Specifications for Horizontally Curved Steel Girder Highway Bridges*. These Specifications are referred to in this manuscript as the *2003 Guide Specs*.

A third project was initiated jointly by AISI and FHWA in 1999 to extend the results of experimental tests conducted under the CURT project and the CSBRP through refined nonlinear finite element analysis, and to develop unified maximum strength equations for load and resistance factor design of curved and straight steel bridge I girders. The results of this research are summarized in White et al. (2001). This report provides: (1) an extensive review and discussion of curved I girder maximum strength equations that have been proposed within the literature, including equations detailed within U.S. and Japanese design specifications; (2) a proposed set of modifications to the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2001), hereafter referred to as the *2001 LRFD Specs*, that extend the applicability of their straight I girder strength equations to address applied and/or induced combined vertical bending, lateral bending, torsion, and shear in both curved and straight I girders; and (3) correlation of the recommended design equations with prior experimental results and with results from a large finite element parametric study. The flexural strength design equations developed in this work are based on the concept of treating the girder flanges as equivalent beam–columns. This report addresses requirements for elastic analysis of the bridge superstructure to determine design stresses for use with the proposed equations. However, its primary focus is on maximum resistance equations that can be used with accurately computed elastic design stresses. These equations, as well as other recommendations for maximum strength equations, will be evaluated by the CSBRP prior to formulation of final recommendations for design implementation.

A fourth research project, NCHRP Project 12-52, is also underway. This project is revising and recalibrating the *2003 Guide Specs* so that LRFD can be applied to curved steel bridges. Data produced from the CSBRP and from White et al. (2001) is being utilized in conjunction with the work within NCHRP 12-38 to develop and calibrate a set of curved steel bridge design specifications that are compatible with the *2001 LRFD Specs*. At the present time (January 2003), a set of draft LRFD provisions have been developed based solely on the provisions of the *2003 Guide Specs*.

It should be noted that the *AASTHO Guide Specs* (AASHTO 1980, 1993, 2001) are one of only two specifications in the world dealing with the design of curved steel bridges. The other specifications are the *Guidelines for the Design of Horizontally Curved Girder Bridges* (Hanshin 1988), or the *Hanshin Guidelines*, which were published in draft form by the Hanshin Expressway Public Corporation. These guidelines were developed as an addendum to the Japan Road Association's *Specifications for Highway Bridges* [Japan Road Association (JRA) 1988]. They address differences in methodology relative to straight girder bridges that should be followed when a curved bridge is being designed. Research leading to development of the *Hanshin Guidelines* was completed during the 1970s and involved a number of single girder and girder component experimental studies coupled with analytical work. Summaries of this research can be found elsewhere (Nakai and Yoo 1988; Kitada et al. 1993; Zureick et al. 1994; Hall et al. 1999; White et al. 2001).

Curved I Girder Design

The *Guide Specs* first adopted by AASHTO in 1976 and used by designers during the 1970s, 1980s, and 1990s are generally perceived as being disjointed and difficult to interpret (SSRC 1991). Commentary accompanying the specifications lacks detail regarding the development and use of the design criteria, and a number of reference and supporting materials that augment the document are difficult to obtain or are unavailable. Therefore, the likelihood of misinterpretation or misuse of these specifications is quite high. Nevertheless, it is important to note that these problems have not been directly associated with any failure or poor performance of bridges designed using these documents. Problems with the construction of curved girders have occurred, but this aspect is not addressed in any of the previous *Guide Specs*.

Incorrect use or interpretation of the *1993 Guide Specs* could have been minimized if design procedures and/or examples had been included. The *2003 Guide Specs* include more detailed discussions related to the use of the specifications and design examples: one for a curved I girder bridge and one for a curved box girder bridge. These examples were developed as part of the NCHRP 12-38 project effort (Hall et al. 1999). They include in-depth discussions of procedures for preliminary and final analysis and design of curved steel bridges.

The sections that follow provide an overview of the curved bridge design process as proposed in the *2003 Guide Specs*. Generally, the design process can be divided into the following tasks:

1. Consideration of general parameters;
2. Preliminary design;
3. Preliminary analysis; and
4. Design refinement.

Item 4, which consists of additional analyses and detailed design steps, is repeated until a final design, one that meets all relevant criteria and satisfies the owner, is obtained. These tasks are used as a framework for discussion of the *2003 Guide Specs*.

It should be noted that equations addressed herein that are assigned a number in any of the AASHTO Specifications are initially referred to using the appropriate AASHTO number, with the corresponding equation number assigned in this manuscript abbreviated and shown in brackets (e.g., [Eq. (x)]). Thereafter, those equations are referred to using the manuscript equation number.

General Parameters

While the *1993 Guide Specs* include provisions for both ASD and LFD, the *2003 Guide Specs* adopt LFD provisions only. The *2003 Guide Specs* are separated into two divisions, similar to the *1996 Standard Specs* (AASHTO 1996a,b): Division I—*Design* and Division II—*Construction*. These Specifications are formatted utilizing a two-column approach similar to that employed by the *2001 LRFD Specs*.

Division I contains two sections not present in the *1993 Guide Specs* that present general information: Article 1, *General*, and Article 2, *Limit States*. Article 1 provides specific details on the limits of applicability of the *2003 Guide Specs* and discusses the overriding fundamental principles that govern these specifications. Complex bridges beyond the limits of the specifications can be designed by following the fundamental principles. Article 1.1 states that the specifications apply to horizontally curved I and single-cell box girder bridges with spans less than 91.5 m (300 ft) and radii greater than 30.5 m (100 ft). These limits are established from past experience, which has shown that constructing curved girders with spans greater than 91.5 m (300 ft) may require second-order analysis for certain erection conditions, and the current state of the research, which has not studied impact factors for girders with radii less than 30.5 m (100 ft). Curved bridges with the following framing arrangements are addressed by these specifications: (1) simple and continuous spans; (2) constant and variable girder spacings; (3) normal and skewed supports; (4) bifurcated alignments; (5) varying girder stiffnesses in a given bridge cross section; (6) discontinuous girders and girders with nonconcentric radii and kinked alignments; and (7) integral pier caps and abutments.

Article 2, *Limit States*, lists the limit states that must be considered during design. These limit states include: strength, fatigue, serviceability, and constructability. The constructability limit state is new in the *2003 Guide Specs* and it reflects a general theme throughout the document of emphasizing the consideration of construction effects on curved bridge performance. Load factors required for each limit state are discussed in the subsections of this article. The fatigue limit state, covered in Article 2.3, refers to provisions in the *2001 LRFD Specs* (Article 6.6.1). References to AASHTO LRFD criteria occur throughout the *2003 Guide Specs*.

Loads and load combinations are discussed in Article 3, *Loads*. Similar to the *1993 Guide Specs*, Article 3 specifies that the load combinations presented in Table 3.22.1A of the *1996 Standard Specs* shall be used except as modified. Article 3 specifies additional loading requirements that must be included in the design of curved bridges. Article 3.2 defines dead loads that should be included and emphasizes that sequencing effects need to be considered when applying dead loads to both composite and noncomposite superstructure designs. The effects of construction on curved bridge performance are further emphasized in Article 3.3, *Construction Loads*, where loads that must be considered during construction and their factors are discussed. This article requires that a load factor or 1.4 be applied to the dead load and construction equipment live loads when checking strength under construction. This is the average of values specified in *2001 LRFD Specs* Article 3.4.2. Article 3.3 requires a load factor of 1.0 for checking of deflections during construction. Uplift that may occur during construction must be examined to ensure that no instabilities or excessive deflections occur, with dead loads that resist uplift being factored by 0.9 and loads causing uplift being factored by 1.2.

Wind loads applied to projected surfaces are defined unidirectionally in the *2003 Guide Specs*. This deviates from the *1993 Guide Specs*, which specified that wind loads were to be applied either perpendicular or parallel to the superstructure. Article 3.4 states that the designer should apply wind loads in a manner that results in the most critical loading on the components being designed (i.e., girders, cross frames and bearings). By requiring unidirectional wind load application to the superstructure, the *2003 Guide Specs* eliminate past confusion associated with applying wind loads orthogonal to the curvilinear geometry.

Article 3.5 covers live loads due to vehicular traffic and includes definitions of design trucks that should be used along with subsections that detail how centrifugal forces, permit loads, overloads, sidewalk loads, and impact effects are to be addressed in the analysis and design. The likelihood of uplift under live load is specifically addressed using the following load combination:

$$D+2.0(\mathrm{LL}+\mathrm{I}) \tag{1}$$

Table 1. 2003 Guide Specs I Girder Impact Factors

Load effect	Impact factor	
	Vehicle	Lane
Girder bending moment, torsion and deflections	0.25	0.20
Reactions, shear, cross frame and diaphragms actions	0.30	0.25

and the specifications require that the effects from future absence of the wearing surface must be incorporated. Outside of the uplift criteria, the live load provisions are generally unchanged from the *1993 Guide Specs*, however, the commentary contains discussions and clarifications related to their use.

Effects of dynamic amplification are discussed in Article 3.5.6. Unlike the *1993 Guide Specs*, where impact loads are taken directly from *Standard Specs* Article 3.8.2, impact factors are defined directly in this article and are differentiated based on structure type. Impact factors are presented for vehicular and lane loads for girder flexural, torsional and deflection effects, girder reactions and shears, and cross frame and diaphragm forces. They are reproduced from Table 3.5.6.1 in Table 1 and are simplified versions of recommended factors in the *1993 Guide Specs*. A 15% fatigue impact factor is required following criteria from *2001 LRFD Specs* Article 3.6.2.1.

Fatigue load provisions are given in Article 3.5.7. The fatigue truck from *2001 LRFD Specs* Article 3.6.1.4, an HS20 truck with a rear axle spacing set at 9.1 m (30 ft), is adopted. Article 3.6, *Thermal Loads*, indicates that curved steel bridges shall be designed for uniform temperature changes following *1996 Standard Specs* Article 3.16, which is unchanged from criteria presented in the *1993 Guide Specs*. However, the likelihood of uplift in narrow bridges due to temperature changes now must be examined by considering temperature gradients between the deck and the girders greater than 4°C (25°F) when the deck width is less than one fifth the longest span length.

Preliminary Design

The *2003 Guide Specs* provides criteria for establishing preliminary framing parameters for I girders in Articles 9 and 12. Preliminary girder depths may be selected using information from Article 12.2, *Span-to-depth Ratio*. This ratio (L_{as}/D) is limited to 25 for girders with $F_y = 345$ MPa (50 ksi). The term L_{as} is the "arc girder length." A more restrictive requirement is specified for girders with higher yield strengths. The arc girder length, L_{as}, is to be taken as the actual arc span for simple spans, 0.9 times the arc span for continuous end spans, and 0.8 times the arc span for continuous interior spans, with the longest resulting span controlling the limit on the web depth D. 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 can make the bridge easier to erect.

Compression flange limiting dimensions are presented in Article 9.1, *General*. It is preferred that the flange width be greater than 20% of the web depth, with 15% being an absolute lower bound. It is recommended that flange thickness be greater than 1.5 times the web thickness. This value is selected to ensure some restraint from the flanges to enhance web bend-buckling capacity (consistent with assumptions of boundary conditions falling between simply supported and fully fixed at the web–flange juncture in web bend buckling-based design equations).

Additional flange and web thickness requirements are given in Articles 5 and 6, where strength criteria for compact and noncompact curved girders are presented. Noncompact flanges are limited to the slenderness ratio in Eq. (5-7) [Eq. (2)],

$$\frac{b_f}{t_f} \leqslant 1.02\sqrt{\frac{E}{(f_b+f_l)}} \leqslant 23 \tag{2}$$

where b_f=flange width (in.); t_f=flange thickness (in.); E=modulus of elasticity (ksi); f_b=the largest factored flange vertical bending stress (i.e., the average flange axial stress) at either brace point (ksi), positive when the stress is tensile, and f_l=corresponding total factored lateral bending stress in the flange due to curvature and any other effects which lead to lateral bending, positive when the flange tip furthest from the center of curvature is tensile and negative when the flange tip away from the center of curvature is in compression.

While the variables f_b and f_l are somewhat similar to f_b and f_w in the *1993 Guide Specs*, sign conventions switch from those used by McManus (1971) to match those from Dabrowski (1968), with positive vertical bending causing tension in the flange being examined and positive lateral bending causing tension at the flange tip farthest away from the center of curvature. Since Eq. (2) involves vertical and lateral bending stresses, it is difficult to use in preliminary design.

Based on data from experimental tests and finite element parametric studies, White et al. (2001) conclude that for b_f/t_f values up to 24, the fabrication and handling limit specified by the *2001 LRFD Specs* are acceptable for curved bridge I girders. That is, making the maximum limit on b_f/t_f a function of the calculated vertical and lateral bending stresses is not strictly necessary. However, they state that if b_f/t_f exceeds the straight-girder compactness limit of $0.76\sqrt{E/F_{yc}}$, where F_{yc} is the compression flange yield stress, the nominal flange strength needs to be reduced to account for the influence of flange local buckling.

Preliminary web thickness may be established using Eqs. (6-1) [Eq. (3)] and (6-2) [Eq. (4)] for unstiffened webs

$$\text{for } R \leqslant 213 \text{ m (700 ft)}$$

$$\frac{D}{t_w} \leqslant 100 \tag{3}$$

$$\text{for } R \geqslant 213 \text{ m (700 ft)}$$

$$\frac{D}{t_w} \leqslant 100 + 0.038(R-700) \leqslant 150 \tag{4}$$

where D=web distance between flanges (in.); t_w=web thickness (in.); and R=minimum radius of web panel (ft).

The limit of 100 for radii below 213 m (700 ft) was selected to approximately satisfy web compactness provisions in the *2001 LRFD Specs* for F_y=345 MPa (50 ksi), which is the maximum F_y allowed for the use of the compact flange flexural strength equations in the *2003 Guide Specs*. For webs with radii greater than 213 m (700 ft), the slenderness limit linearly increases to a maximum value of 150 at a radius of 610 m (2000 ft). The commentary states that, while unstiffened webs can be used, some research studies indicate that transverse stiffeners help retain the cross sectional shape and may enhance the flexural capacity and the fatigue resistance of the web. White et al. (2001) studied a range of curved I girders with D/t_w=160 and d_0/D of 1, 2, and 3, where d_0 is the transverse stiffener spacing. They found only a minor increase in flexural strength with more closely spaced stiffeners in their studies. Close stiffener spacing may be more beneficial for I girders with less slender webs, but should again provide little benefit for highly compact webs (such as those in typical rolled I sections).

Eqs. (3) and (4) are specified in the *2003 Guide Specs* largely due to a lack of test data on the shear capacity of unstiffened curved I girders for D/t_w greater than about 70. These slenderness limits were not given in the *1993 Guide Specs*. The limit of $D/t_w \leqslant 150$ is a handling requirement for straight I girders in the *2001 LRFD Specs*.

Article 6.3 addresses requirements for transversely stiffened webs and Article 6.4 addresses longitudinally and transversely stiffened webs. Web slenderness up to a D/t_w of 150 is allowed for transversely stiffened webs and D/t_w up to 300 is allowed for webs with longitudinal and transverse stiffeners. The longitudinally stiffened girder limit is based on tests conducted in Japan and corresponding limits in the *Hanshin Guidelines*. The *Hanshin Guidelines* permit D/t_w up to 250 for webs with a single longitudinal stiffener and D/t_w up to 300 for webs with two longitudinal stiffeners. The *2003 Guide Specs* require a transverse stiffener spacing less than or equal to the depth between the flanges (D) in all stiffened webs, as in the *Hanshin Guidelines*. Hall et al. (1999) state that relief from this requirement may be justified, for some curvatures, with additional testing.

Initial cross frame and diaphragm locations can be determined using criteria presented in Article 9.3.2, *Arrangement*. Hall et al. (1999) suggest that these components should be spaced as uniformly as possible, since research that led to the development of the flexural strength equations in Article 5 did not explicitly consider configurations with unequal cross frame or diaphragm spacing. The *2003 Guide Specs* limit the spacing of cross frames and diaphragms to 7.6 m (25 ft) when simplified analysis methods are employed. Regardless of analysis technique, spacings greater than 9.1 m (30 ft) are not allowed. This requirement is intended to ensure adequate lateral and torsional restraint of the curved I girders. Additional limits on cross frame locations are established as the design is refined. The following equation, developed from the V-load method, is provided in the Article 9.3.2 commentary as a guide for preliminary framing

$$l = \sqrt{\frac{5}{36} r_\sigma R b_f} \tag{5}$$

where l=cross frame spacing (ft); r_σ=desired bending stress ratio, $|f_l/f_b|$, with a recommended maximum value of 0.3; and R=girder radius of curvature (ft).

Transverse stiffener preliminary dimensions can be established using the width-to-thickness ratio criteria presented in Eq. (6-13) [Eq. (6)] in Article 6.5

$$\frac{b_s}{t_s} \leqslant 0.48\sqrt{\frac{E}{F_y}} \tag{6}$$

This equation is taken from the *2001 LRFD Specs* and is a nondimensionalized form of the slenderness limit provided in the *1993 Guide Specs*. Transverse stiffener width (b_s) must be greater than 51 mm (2 in.)+D/30 or one fourth of the narrower flange width. Eq. (6) also governs bearing and longitudinal stiffeners. The minimum width requirement has its origin from Ketchum (1920).

Preliminary Analysis

Unlike straight girder bridges, curved girders tend to transmit a significant fraction of their loads to the outer, convex side of the bridge. This leads to the requirement in the *1993* and *2003 Guide*

Specs that the analysis must treat the entire structure rather than examine a single girder using assumed load distributions. Therefore, a preliminary analysis of a curved girder bridge is more complex than that generally used for a straight bridge.

Article 4 of the *1993 Guide Specs* provides an approximate method for computing girder vertical and lateral flange bending moments for the outside girder. Other girders can often be preliminarily designed as straight members. *1993 Guide Specs* approximations attempt to take the presence of lateral bracing into account by applying factors to girder stresses found from a grid analysis. These approximate methods are not presented in the *2003 Guide Specs*.

The *2003 Guide Specs* contain extensive information pertinent to preliminary and detailed analyses of curved steel I girder bridges. While the *1993 Guide Specs* commentary also contained extensive analysis information, such as the approximate procedure detailed above, changes have been made in the *2003 Guide Specs*.

Various sections of the *2003 Guide Specs* discuss appropriate analysis procedures. Elastic analysis using small-deflection theory is recommended for most situations. However, the commentary to Article 1.2.5, *Large-Deflection Inelastic Behavior*, states that although inelastic analyses are not required, second-order elastic analyses are likely to give improved results for cases involving large lateral deflections. Second-order analyses are suggested for checking the stability of girders during construction, when bracing points are spaced at intervals greater than that permitted by the specifications. The commentary states that material inelastic behavior need not be considered, since redistribution of loads is not permitted by the specifications. This statement is based on the assumption that localized distributed yielding due to the combination of initial residual stresses with the stresses due to the loading conditions have a minor effect on strength. The designer is cautioned that initial geometric imperfections and residual stresses can have a significant effect on the maximum strength behavior, and in general should be included in any assessment of strength based on an analysis. If material nonlinearity, including residual stress effects, is not included in the analysis, then maximum stresses need to be held to some fraction of F_y or to values calculated using the design equations.

The commentary to Section 1.2 states that all components in a curved steel bridge (e.g., the deck, shear connectors, cross frames, and bearings) are load-carrying members and must be examined using the same level of refinement as applied to the girders and connections.

Recommended preliminary analysis procedures are largely covered in Articles 4.2 and 4.3.1. Article 4.2, *Neglect of Curvature Effects*, permits the engineer to ignore curvature effects when calculating vertical bending moments if: (1) the girders are concentric; (2) bearing lines are not skewed more than 10° from radial; and (3) L_{as}/R is less than 0.06 radians. If these criteria are met, then a line girder analysis, using a $S/5.5$ wheel load distribution factor, is permitted for calculation of vertical bending moments. However, torsion and flange lateral bending moments must be calculated and the member strength must be examined using design equations accounting for curvature effects.

Eq. (4-1) [Eq. (7)] is provided as an option for estimating flange lateral bending moments due to curvature in a girder that meets the aforementioned line girder analysis requirements and has nearly constant cross frame spacing. It appears as

$$M_{lat}=\frac{6Ml^2}{5RD} \tag{7}$$

where M_{lat}=lateral flange bending moment (k-ft); M=vertical bending moment (k-ft); and D=girder depth (in). No specific recommendations are provided for preliminary analysis for torques and shears. Fundamentally, the girder shears are directly tied to the girder vertical bending moments, and thus the specifications implicitly permit the calculation of these forces by line girder analysis under the above conditions. Also, the primary torsional stiffness and resistance in curved I girders is tied to restraint of flange warping, i.e., to flange lateral bending. Typical girder unsupported lengths, l, are such that the predicted contribution from St. Venant torsion is small based on thin-walled beam theory; if potential web distortion and raking of the flange under torsional moments is considered, this contribution is even smaller. The *1993* and *2003 Guide Specs* do not address girder shear stresses due to St. Venant torsion and focus only on lateral flange bending due to torsion in their design resistance checks.

Article 4.3.1, *Approximate Methods*, provides criteria for the use of the V-load method for the preliminary analysis of curved I girders. Unlike the *1993 Guide Specs*, where an additional grid analysis procedure for curved I girders is presented in Article 2.5, no other approximate methods are presented in the *2003 Guide Specs*.

More refined analysis methods are addressed in Article 4.3.2. These are most common computer methods based on finite element theory and they can be used for preliminary or detailed analyses. The *2003 Guide Specs* note that these types of analyses should incorporate support stiffness, including lateral restraint from integral abutments or integral piers, as well as bearing eccentricities. Article 4.4 states that the possibility of girder lift-off during deck placement should be considered by modifying analysis constraint conditions.

A number of issues related to the incorporation of cast-in-place and prestressed concrete decks into an analysis are discussed in Article 4.5. A number of provisions for addressing composite construction are also included. Recent field measurements indicate that uncracked sections best predict stresses for the fatigue, serviceability and constructability limit states (e.g., Yen et al. 1995) and this is reflected in the provisions with a cracked section analysis being recommended only for calculation of girder stresses for checking strength. The effective width is set equal to the full deck width over each girder, a change also supported by recent field research (Yen et al. 1995). The slab should be incorporated into the analysis in a manner that accounts for its ability to resist compressive, tensile and shear stresses. Consideration of shear lag effects is required.

Article 4.6 addresses the analysis of construction conditions. It requires that analyses be performed to account for construction sequencing effects.

Design Refinement

Analysis options recommended by the *2003 Guide Specs* are discussed in the above two sections. This section focuses on design criteria. The criteria are divided into six subtasks: consideration of strength, serviceability, fatigue and overload limit states; detailing; and evaluation of construction conditions.

Strength

Curved I girder flange and web strength criteria are addressed in the *2003 Guide Specs* in Article 5, *Flanges with One Web*, and Article 6, *Webs*. The flange strength criteria are taken from the *1993 Guide Specs*, but with some additional restrictions and requirements. The criteria apply to horizontally curved rectangular flange girders with a single vertical or inclined web attached at the mid-width of the flanges. The flanges may be compact or noncompact and either continuously or partially braced. Both composite and noncomposite girders are addressed.

The flange flexural resistance equations are based on the traditional parabolic CRC inelastic buckling equation, which was in use for straight I girder design at the time of their development (McManus 1971). The lateral torsional strength of an equivalent straight girder of length l (where l is the arc distance between the brace points) is computed and then multiplied by reduction factors (denoted by the symbol ρ), which account for the effects of horizontal curvature and flange lateral bending at the brace points on the vertical bending resistance. The vertical bending resistance is expressed in terms of the corresponding average flange axial stress.

For girders with compact flanges, the flange strength is taken as the smaller of

$$F_{\mathrm{cr1}}=F_{\mathrm{bs}}\overline{\rho_b}\,\overline{\rho_w} \tag{8}$$

and

$$F_{\mathrm{cr2}}=F_y-\tfrac{1}{3}|f_l| \tag{9}$$

where F_{bs}=lateral torsional buckling strength of the equivalent straight girder. The bending and warping ρ factors $(\overline{\rho_b},\overline{\rho_w})$ and F_{cr1} and F_{bs} are essentially the same as the ρ factors and F_{bu} and F_{bs} in the *1993 Guide Specs*. The term f_l is now used for the flange lateral bending stress in the equations for $\overline{\rho_b}$ and $\overline{\rho_w}$ instead of f_w, to reflect that these stresses can result from restraint of warping within the I girders as well as from all other potential sources of lateral bending in the girder flanges (e.g., wind loading). The $\overline{\rho_b}$ and $\overline{\rho_w}$ equations have also been modified so that flange widths can be given in inches and unbraced lengths in feet. The product of $\overline{\rho_b}\,\overline{\rho_w}$ is limited to 1.0.

For girders with noncompact flanges, the flexural strength (written in terms of the average flange axial stress) is taken as the smaller of

$$F_{\mathrm{cr1}}=F_{\mathrm{bs}}\rho_b\rho_w \tag{10}$$

and

$$F_{\mathrm{cr2}}=F_y-|f_l| \tag{11}$$

The terms ρ_b and ρ_w are again selected using similar equations to those presented in the *1993 Guide Specs*, with modifications allowing for flange widths in inches and unbraced lengths in feet and f_l being substituted for f_w.

McManus (1971) developed the ρ factors used in Eqs. (8) and (10) through a trial and error process, with the goal of developing simple design formulas that provided good estimates of computed results. They were formulated for doubly symmetric curved I beam segments with equal and opposite vertical and flange lateral bending end moments and were checked against experimental data and refined strength predictions. The flexural capacity for compact curved I girders was assumed to be reached at full plastification of the girder cross section in the development of Eq. (8). The flexural capacity of noncompact sections was assumed to be reached when the computed elastic stress reached the yield strength at one of the flange tips in the development of Eq. (10). McManus (1971) provides extensive discussion of the justification for these simple assumptions.

It should be noted that if the lateral to vertical bending stress ratio (f_l/f_b) is positive, the compact section Eq. (8) predicts a greater girder capacity with increasing f_l/f_b. However, the noncompact section Eq. (10) can predict either an increase or a decrease in girder capacity with increasing f_l/f_b. Conversely, if f_l/f_b is negative (i.e., compression due to f_l at the brace point compression flange tips away from the center of curvature), the compact section Eq. (8) predicts a decrease in capacity with increasingly negative f_l/f_b, and the noncompact section Eq. (10) predicts an increase in capacity with increasingly negative f_l/f_b. This trend for noncompact sections is apparently due to the fact that first yield, according to McManus's approximate second-order elastic calculations, is always further delayed by an increasingly negative f_l/f_b. The behavioral trend for compact sections is more intuitive in that, if f_l/f_b is positive, compression flange lateral bending at the brace points counters the tendency of the flange to bend outward from the center of curvature, whereas if f_l/f_b is negative, compression flange lateral bending at the ends of the unsupported segment is in the same direction as flange bending due to horizontal curvature.

The *Recommended Specifications* (Hall and Yoo 1998) and Hall et al. (1999) suggested that the flange lateral bending stresses should be separated into two parts: one part due to horizontal curvature and one part due to effects other than the horizontal curvature, to improve the accuracy Eqs. (8) and (10). More recent research (White et al. 2001) combined with data presented in Hall et al. (1999) indicates that using the definition of lateral bending stresses as originally provided by McManus (1971) provides more accurate predictions of flexural capacity on average, although they are still highly approximate and sometimes slightly unconservative. Therefore, the *2003 Guide Specs* do not distinguish between different contributions to the flange lateral bending stress. The stress f_l is simply the maximum flange lateral bending stress at the ends of the unsupported length calculated from a first-order analysis of the bridge superstructure.

The *2003 Guide Specs* recommend that flange sizes remain constant within the girder unsupported length, since the ρ factor equations were derived for a constant width flange. If the flange width is varied along the length of the unsupported segment, conservative assumptions should be made when evaluating the strength. The ratio f_l/f_b is limited to 0.5, as in the *1993 Guide Specs*. However, this requirement is relaxed for low stress levels where f_b is considerably lower than the flange strength for a similarly proportioned straight girder. Girder unbraced length limits have not changed from the *1993 Guide Specs*. These limits are $l\leqslant 25\,b_f$ and $l\leqslant R/10$. McManus's (1971) study adopted these parameters as practical maximum limits, based on an assessment of curved steel I girder bridge construction at the time of his research.

Compact flanges are limited to $F_y\leqslant 345$ MPa (50 ksi) and $b_f/t_f\leqslant 18$. The limit on b_f/t_f is essentially the flange compactness limit in the *2001 LRFD Specs* for $F_y=345$ MPa (50 ksi). White et al. (2001) find that this limit is adequate for basing the curved flange strength on general yielding, for $F_y=345$ MPa (50 ksi), without the need to consider any loss in strength due to flange local buckling.

Eq. (9) was not presented in the *1993 Guide Specs*. It was added because in some cases, the *1993 Guide Specs* equations could lead to a design in which the full plastic capacity of the flange was exceeded at a cross frame location. Similarly, for noncompact flange sections, Eq. (11) was added in the *2003 Guide Specs* to guard against some cases in which Eq. (10) gives a strength larger than first yield. The compact section criteria also now account for singly symmetric cross sections by using 90% of the flange width when determining λ for F_{bs}. This arbitrary reduction in the flange width was employed for singly symmetric sections in the AASHTO specifications at the time of development of the ρ factor equations, but it was not included as part of the *1993 Guide Specs*. Hall et al. (1999) reintroduced this reduction in the *2003 Guide Specs* to account for reductions in capacity due to lack of symmetry about the horizontal axis.

Noncompact flanges, which are defined in Article 5.2.2 as flanges with $b_f/t_f > 18$, are subjected to a restriction on the maximum flange slenderness presented in Eq. (2). As noted earlier, White et al. (2001) concluded that a simple limit of $b_f/t_f \leqslant 24$, the fabrication and handling limit for I girder flanges in the *2001 LRFD Spec*, is safe and sufficient if the flange strength is reduced to account for flange local buckling. White et al. (2001) propose a flange local buckling strength equation that is generally more liberal than the provisions in the *2003 Guide Specs*. The limit of 23 in Eq. (2) is based on experimental observations by Mozer and Culver (1970). Culver and Nasir (1971) observed that flange local buckling starts having a significant detrimental effect on girder vertical bending capacity in the vicinity of this limit. Also, Mozer and Culver (1970) tested two heat-curved and two cut-curved I girders with $b_f/t_f = 23$ and concluded that this limit was adequate if both vertical bending and lateral flange bending stresses are considered and the capacity is limited to initial yielding at the flange tips. Interestingly, the ratio $b_f/t_f = 23$ was the approximate maximum limit in the *1973 Standard Specs* for $F_y = 250$ MPa (36 ksi).

Tension flange stress levels are addressed in Article 5.3, *Partially Braced Tension Flanges*. Research behind the ρ factors did not consider tension flange behavior or singly symmetric girders. However, a tension flange check is necessary in general for singly symmetric girders. Therefore, the *2003 Guide Specs* apply compact section critical stress limits in Article 5.2.1 [Eqs. (8) and (9)] to the tension flange. These limits are considered conservative since the tension flange tends to straighten under load and stability should not be an issue.

Composite action is addressed in Article 5.4, *Continuously Braced Flanges*. Flange slenderness is limited to values calculated using Eq. (2) for noncompact flanges. If full lateral support of the compression flange due to the deck exists, then both compression and tension flange vertical bending stresses are limited to the yield stress. Lateral bending stresses can be ignored in the flange attached to the bridge deck once full composite action is developed.

Web strength design is covered in Article 6 of the *2003 Guide Specs*. Most of the *1993 Guide Specs* criteria for evaluating web capacity, which was based on shear and bend-buckling limits, has been retained in the *2003 Guide Specs*. Strength evaluation under compressive longitudinal and shearing stresses is divided into three groups in Articles 6.2–6.4: unstiffened webs; transversely stiffened webs; and transversely and longitudinally stiffened webs. Criteria are largely taken from 10.48.5 to 10.48.8 of the *1996 Standard Specs*, with some modifications being made to account for curvature. A web may be designed as unstiffened only if its slenderness ratio falls within limits presented in Eqs. (3) and (4).

Irrespective of the web design, the *2003 Guide Specs* divide their strength criteria into bending and shearing stress evaluations. Web flexural stresses are checked against a critical stress given by Eq. (6-3) [Eq. (12)]

$$F_{\mathrm{cr}} = \frac{0.9Ek}{\left(\dfrac{D}{t_w}\right)^2} \leqslant F_y \tag{12}$$

where k = bend buckling coefficient. This equation is included to limit stresses to the elastic bend-buckling stress or to first yield. It was not present in the *1993 Guide Specs* and it replaces equations presented in Article 2.12 of the *1993 Guide Specs* that limited the slenderness for stiffened and unstiffened webs. Stresses compared against this limit must be accumulated to account for the construction process and long and short-term composite loadings.

The web bend buckling coefficient, k, for unstiffened webs, is

$$k = 7.2\left(\frac{D}{D_c}\right)^2 \leqslant 7.2 \tag{13}$$

and for transversely stiffened webs

$$k = 9\left(\frac{D}{D_c}\right)^2 \leqslant 7.2 \tag{14}$$

For longitudinally and transversely stiffened webs, when $d_s/D_c \geqslant 0.4$

$$k = 5.17\left(\frac{D}{d_s}\right)^2 \tag{15}$$

or, when $d_s/D_c < 0.4$

$$k = 11.64\left(\frac{D}{D_c - d_s}\right)^2 \tag{16}$$

where D_c = depth of the web panel in compression; and d_s = distance between the longitudinal stiffener and the compression flange. Both composite and noncomposite sections are checked.

Eqs. (13) and (14) are similar to the bend-buckling coefficients used in *1996 Standard Specs* Eq. (10-173) for straight web panels during construction. However, for unstiffened webs, the original constant of 9 is reduced by 20% to 7.2 to provide an added level of safety against combined effects of flexure and shear on web buckling strength. The constant was kept at 9 for transversely stiffened webs since the stiffeners tend to enhance the curved web capacity (e.g., tension field action is neglected in the calculation of the web shear strength).

Note that Eqs. (15) and (16) allow for longitudinal stiffeners at any location on the web. This differs from *1993 Guide Specs* provisions that required longitudinal stiffeners at $D/5$ from the compression flange. This location was only appropriate for a section having it neutral axis at mid depth. The stiffener can now be placed in a location that enhances the performance for both noncomposite and composite loadings.

Factored web shears are checked using the critical shear strength, V_{cr}, which is determined using CV_p as in the *1993 Guide Specs*. V_p is the shear yielding strength from Eq. (10-115) in the *1996 Standard Specs*. The parameter C is the ratio of the web's elastic shear buckling and shear yielding strengths. It is evaluated for differing web slenderness ratios using nondimensionalized forms of the equations given in *1996 Standard Specs* Article 10.48.8.1.

Transversely stiffened web shear buckling coefficients are also determined using a formula from Article 10.48.8.1 of the *1996 Standard Specs*, with the symbol k_w used for the buckling coefficient k. Transversely stiffened webs are limited to a slenderness ratio of 150. *1993 Guide Specs* limits on the web panel width (equal to D for interior panels and $D/2$ for end panels) were retained since research conclusively indicating that larger widths could be appropriately used in curved I girders had not been performed. It can be argued that shorter panel widths at the girder ends are not required since tension field action is not counted upon in the shear resistance equations and there is no need for a short panel to anchor the tension field at the girder ends.

Transverse and longitudinal stiffeners are required when the web slenderness ratio exceeds 150. An upper bound slenderness ratio of 300 is applied, however, due to a lack of research into their behavior.

While the *1993 Guide Specs* included provisions for the design of hybrid girders in Articles 2.18–2.21, the *2003 Guide Specs* do not allow the design of curved hybrid girders. Hall et al. (1999) concluded that research on the response of hybrid curved girders is limited and insufficient to support design provisions for curved girders of this type.

Serviceability

The *1993 Guide Specs* contain no specific mention of serviceability limits for curved I girders. Designers are referred to the deflection limit criteria stated in Article 10.6 of the *Standard Specs*. Serviceability criteria are addressed in Article 12 of the *2003 Guide Specs*. Span-to-depth ratio requirements are covered in Article 12.2. Article 12.3 states that dead load deflections from steel, concrete, and other loads should be calculated and reported separately to help with camber calculations. Both vertical and lateral cambers may be required. Live load deflections are discussed in Article 12.4 and they match limits outlined in the *1996 Standard Specs* for service and impact loads. These limits are applied to each girder in the bridge cross section.

Fatigue

Fatigue provisions are outlined in Article 9.6 and this information is linked to fatigue criteria in Article 3.5. Although it is not stated explicitly, fatigue criteria outlined in *1996 Standard Specs* Article 10.3 also supplement provisions presented in the *2003 Guide Specs*. Fatigue checks must include the effects of both vertical and lateral bending on the details that are being evaluated. Specific details that must be evaluated are mentioned in the article and commentary.

Daniels and Herbein (1980) conducted the most recent experimental research regarding fatigue of curved steel I girder bridge elements. Based on this research, the following equation was proposed for load factor design (Daniels et al. 1980)

$$\frac{D}{t_w}=6.78\sqrt{\frac{F}{F_y}\left[1-4\left(\frac{d_0}{R}\right)\right]}\leqslant 192 \tag{17}$$

This equation guarded against potential fatigue problems due to web plate bending induced by horizontal curvature and it is more liberal than the corresponding equation in the LFD portion of the *1993 Guide Specs*. However, a modified version of this equation was adopted in the ASD portion of the *1993 Guide Specs*. For $d_0/D\leqslant 1$, (i.e., for stiffened webs), the *2003 Guide Specs* effectively restrict the web slenderness (D/t_w) more severely than the web slenderness limit in the LFD provisions of the *1993 Guide Specs* by limiting the web flexural stresses from Eq. (12) under all strength load combinations. However, for $d_0/D>1$, the additional web slenderness limit in the *1993 Guide Specs* can be more restrictive. By effectively restricting the web slenderness (D/t_w) based on Eq. (12), the *2003 Guide Specs* also consider directly the potential effect of girder monosymmetry, i.e., $D_c/D\neq 0.5$, in limiting the maximum web slenderness that can be used in design. Hall et al. (1999) state "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." The same statement can be applied to web slenderness restrictions. The studies by Zureick et al. (2001) and Jung and White (2001) address curved web requirements from the perspective of maximum strength. However, to the authors' knowledge, no studies are underway at the present time to further investigate the fatigue performance of curved I girder webs.

Overload

Provisions for proportioning members against overload are presented in Article 9.5, *Permanent Deflection*. The overload provisions are more detailed than those given in the *1993 Guide Specs*. Different stress limits are given for continuously braced and partially braced compression flanges and limits for the web and for other primary members are also provided. For continuously braced compression flanges, composite flange stresses should not be greater than $0.95F_y$ and noncomposite flange stresses no greater than $0.80F_y$. These are limits adopted from *Standard Specs* Article 10.57 and were used in the *1993 Guide Specs*. Partially braced compression flange stresses are limited to the noncompact section strength determined using Eq. (10), to ensure that secondary effects caused by curvature are accounted for. Lateral flange bending stresses at brace points are not checked at overload, since it is assumed that they act over a small area and offer little contribution to permanent set. Maximum web compressive stresses are conservatively calculated on the uncracked sections and are limited to bend-buckling stresses found using provisions in Article 6. Stresses in other primary members are limited to first yield.

Detailing

Transverse and longitudinal stiffener design is addressed in Articles 6.5 and 6.6 of the *2003 Guide Specs*, with bearing stiffeners discussed separately in Article 6.7. To ensure that the stiffeners perform adequately, it is stated that they should have the same yield stress as the girder. Transverse stiffener legs should have width-to-thickness ratios satisfying Eq. (6).

The moment of inertia for single or paired transverse stiffeners must be determined using the following formula:

$$I_{ts}=d_0t^3J \tag{18}$$

where d_0=panel width. Eq. (6-15) [Eq. (19)] is used to determine J, which is a nondimensionalized parameter accounting for web panel size, and it involves two parameters that account for curvature, X and Z. X modifies J to account for curvature effects to ensure that the stiffener is positioned and sized properly to establish a nodal line in the web. Z is a parameter adopted from the *Hanshin Guidelines* that incorporates the level of curvature. The equations used for calculating J, X, and Z appear as

$$J=\left[\left(\frac{1.58}{d/D}\right)^2-2\right]X\geqslant 0.5 \tag{19}$$

for $d_0/D=a\leqslant 0.78$

$$X=1.0 \tag{20}$$

for $0.78\leqslant a\leqslant 1.0$,

$$X=1+\left(\frac{a-0.78}{1775}\right)Z^4 \tag{21}$$

and

$$Z=\frac{0.079d_0^4}{Rt_w}\leqslant 10 \tag{22}$$

where d=required stiffener spacing; d_0=actual stiffener spacing which may be less than d; and R=girder radius.

Provisions for sizing longitudinal stiffeners are also derived from the *Hanshin Guidelines*. Stiffener width-to-thickness ratios are limited to values presented in Eq. (8) and their moments of inertia must satisfy the inequality in Eq. (6-19) [Eq. (23)]

$$I_{ls} \geqslant D t_w^3 (2.4 a^2 - 0.13) \beta \tag{23}$$

where

$$\beta = \frac{Z}{6} + 1 \tag{24}$$

when the stiffener is on the side of the web away from the center of curvature and:

$$\beta = \frac{Z}{12} + 1 \tag{25}$$

when the stiffener is on the side of the web towards the center of curvature. The term Z is from Eq. (22). By incorporating Z, the β term involves curvature effects and effectively increases the rigidity of the stiffeners to resist tendency of the curved web to bow outward under load. It is a simplified version of the longitudinal stiffener provision given in the *Hanshin Guidelines* for curved I girders, which was derived assuming that the stiffeners were uninterrupted on the web. Therefore, it is recommended that longitudinal stiffeners be placed on the web face opposite from any transverse stiffeners. At locations where longitudinal and transverse stiffeners must intersect, specific guidelines regarding their intersection with one another are given to ensure that the flexural and axial strength of the discontinued element is maintained.

Bearing stiffeners, which were not discussed separately in the *1993 Guide Specs*, are addressed in Article 6.7 of the *2003 Guide Specs*. These stiffeners should fit snugly against the flange that receives the concentrated load. Alternatively, they should be welded to that flange to transfer the reactions following criteria from *1996 Standard Specs* Article 10.34.6.1. If they are concentrically loaded, these stiffeners are designed following *1996 Standard Specs* criteria for a centrally loaded compression member in Article 10.54.1. Eccentrically loaded bearing stiffeners are to be designed as beam–columns following Article 10.54.2 of the *1996 Standard Specs*.

The design of cross frames, diaphragms, and bracing members is covered in Articles 9.3, *Cross Frames and Diaphragms*, and 9.4, *Flange Lateral Bracing*. Full depth cross frames or diaphragms are recommended, which is unchanged from the *1993 Guide Specs*; however, they may be staggered, which was not allowed in the *1993 Guide Specs*. Compression members should be proportioned following *1996 Standard Specs* Article 10.54 and effective length factors should be preferably set to 0.9 for all cross sections except single angles, which should have an effective length factor of 1.0 to more effectively represent their strength. Tension members should be proportioned following criteria in *1996 Standard Specs* Article 10.18.4 with net section criteria defined in Article 10.16.14.

Article 9.4 states that lateral flange bracing members shall be designed as primary bridge members. Therefore, they are designed for all possible loading effects. Lateral bracing of the top and/or bottom flanges can be used and the bracing members are attached to the flanges.

Article 11 addresses splices and connections. Splices should be designed for vertical and lateral bending and torsional loads following criteria presented in *1996 Standard Specs* Articles 10.18, 10.19, and 10.56. If composite construction is used, stresses applied to splices should be accumulated, accounting for the construction process and short and long term composite loadings. It is recommended that flange and web splices be designed separately, the flanges for compressive and/or tensile forces resulting from flexural effects and the web for both bending and shear effects. It is also recommended that the 75% and average rules required for splices by *1996 Standard Specs* Article 10.18.1.1, which suggest designing the web splice so that it is not a weak link in the girder, need not apply since shear stiffness offers minimal contribution to overall member stiffness.

Article 11.2, *Bolted Connections*, states that all curved bridge bolted connections should be evaluated as slip-critical at overload and construction limit states using *1996 Standard Specs* Article 10.57.3 to ensure that the geometry will be unaltered while the bridge is being constructed. Standard sized bolt holes must be used for all splices and oversized or slotted holes may be used in bracing connections if the design geometry can be maintained.

Shear connectors are designed and placed following Article 7.2. The article incorporates criteria from *1996 Standard Specs* Article 10.38.2 and modified provisions from *2001 LRFD Specs* Articles 6.10.7.4.1b and 6.10.7.4.2.

For girder end regions, the minimum number of shear connectors required for positive moment is adopted directly from *1996 Standard Specs* Eq. (10-61), which is a function of shear connector strength and force in the slab. However, the slab force now includes a radial component in Eq. (7-2) [Eq. (26)]:

$$P = \sqrt{\overline{P}_p^2 + \overline{F}_p^2} \tag{26}$$

where $\overline{P}_p$=longitudinal force in the slab determined as the smallest of the steel and concrete capacities in the composite section; and $\overline{F}_p$=approximate radial force in the slab determined in Eqs. (7-5) [Eq. (27)] as

$$\overline{F}_p = \overline{P}_p \frac{L_p}{R} \tag{27}$$

where L_p=arc length between the end of the girder and nearest point of maximum live positive moment; and R=smallest girder radius over L_p.

For interior girder regions, the minimum number of shear connectors required between adjacent maximum positive and negative moment points is defined using Eqs. (7-6)–(7-10), which are similar to the end region equations shown above except P subscripts are replaced with Ts. For both interior and end regions, radial force effects are directly accounted for through vector mathematics in the equation for the total longitudinal force, P. The derivation of these equations is based purely on static equilibrium principles. Although the *1993 Guide Specs* also attempted to account for the effects of curvature on slab forces for shear connector design, the approach approximated radial effects through estimation of additional longitudinal forces due to curvature instead of simply employing vector mechanics.

Shear connector checks for fatigue loads are similar to those in the *2001 LRFD Specs*. However, the torsional shear is added vectoraly to obtain the connector force using Eqs. (7-11) [Eq. (28)]–(7-13) [Eq. (29)]

$$V_{\mathrm{sr}} = \sqrt{(V_{\mathrm{fat}})^2 + (F_{\mathrm{fat}})^2} \tag{28}$$

$$F_{\mathrm{fat}} = \frac{A_{\mathrm{bot}} \sigma_{flg} l}{wR} \tag{29}$$

$$F_{\mathrm{fat}} = \frac{F_{\mathrm{CR}}}{w} \tag{30}$$

where V_{fat}=longitudinal fatigue shear range per unit length (k/in.); F_{fat}=radial fatigue shear range per unit length (k/in.); A_{bot}=area of the bottom flange (in.^{2}); σ_{flg}=bottom flange fatigue stress range (ksi); R=girder radius (ft); w=deck effective length (in.); and F_{CR}=cross frame force net range at top flange (kips). The term F_{fat} is calculated using the larger of Eqs. (29) and (30). Eq. (7-14) [Eq. (31)] specifies shear connector pitch required for fatigue design as

$$p=\frac{nZ_r}{V_{sr}} \tag{31}$$

where n=number of shear connectors in a cross section; Z_r=shear fatigue strength of an individual connector from *2001 LRFD Specs* Article 6.10.7.4.2; and V_{sr} is as defined in Eq. (28). If effects other than curvature, such as skew, predominantly cause torsion in the girder, Eq. (31) should be used.

Article 8, *Bearings*, covers the design and selection of bearings. General discussions of forces that shall be considered and deformations that shall be allowed are given. Although no direct references to AASHTO Specifications are made, it is stated that bearings should be designed following appropriate AASHTO provisions.

Construction

The *1993 Guide Specs* do not contain any sections that specifically address evaluating curved I girders during construction. In the *2003 Guide Specs*, construction issues for curved I girder bridges are addressed in Division I in Article 13, *Constructibility*, and in Division II, which focuses on construction and is similar in format to Division II in the *1996 Standard Specs*.

Article 13 states that a construction plan must be included as part of the final design that incorporates evaluations of stresses caused by the factored construction loads. These stresses are compared against strength limits in Article 5. Deflections shall be tracked throughout the proposed construction sequence so that the superstructure's final position closely matches the intended design geometry.

Additional information given in Article 13 relates to shipping and erection techniques that need to be specified in the construction plan. The erection sequence shall be clearly specified and, if temporary supports are employed, provisions shall be incorporated that account for temporary reactions that develop. Additional bracing members required for erection shall be sized assuming that they are primary members as specified in Article 9.4 and, if needed, their removal shall be incorporated into the erection sequence. The deck pour sequence shall be clearly specified and stresses caused by sequential placement of the deck shall be evaluated. Load effects caused by deck overhang brackets are checked. Two equations are provided: Eq. (C13-1) [Eq. (32)] and Eq. (C13-2) [Eq. (33)], to estimate additional lateral bending moments in the flanges that can result from torque generated by the deck overhangs. These equations are derived assuming that girder panel lengths are equal and that the brackets exert a uniform lateral load on the flange and they appear as

$$M_{lat}=0.08Fl^2 \tag{32}$$

$$M_{lat}=0.125Pl \tag{33}$$

where F=factored uniform lateral bracket force in flange (kips); P=concentrated lateral bracket force at mid panel (kips); and l=unbraced length (ft).

Division II provides guidelines for curved bridge fabricators and constructors to ensure that the structure is constructed as designed.

Conclusions

The *2003 Guide Specs* are largely the result of NCHRP Project 12-38. They represent a synthesis of prior guidelines, both in the U.S. and in Japan, laboratory testing, analytical studies, and experience with state-of-the-art design and construction of curved steel bridges within the United States as of December 1998. These specifications have corrected a number of deficiencies that existed within the *1993 Guide Specs*. However, execution of new research to further enhance the specifications was outside of the scope of NCHRP Project 12-38. In a number of areas, Hall et al. (1999) state that the provisions proposed by Project 12-38 are likely to be conservative, but that restrictive rules are implemented due to a lack of complete knowledge of the implications on curved bridge performance.

The *2003 Guide Specs* provide an important step toward the development of new state-of-the-art specifications for design and construction of horizontally-curved steel bridges. However, substantial new knowledge has been gained in FHWA and AISI-FHWA directed research during and since the completion of the effort leading to these specifications. Ongoing research within the CSBRP is providing additional advanced data pertaining to the behavior of curved steel I girder bridges and, in combination with the framework provided by NCHRP 12-38 and the recent research completed to date, is expected to lead to even further advances that may be implemented in curved steel bridge design specifications. It is hoped that all of these efforts can be brought to fruition within future LRFD provisions for design and construction of horizontally curved steel bridges. It is anticipated that the design of curved steel girder bridges will be incorporated into future *LRFD Specs*, thereby eliminating the *Guide Specs*. The development of curved girder provisions in an LRFD format will greatly facilitate the design of these types of structures.

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