PRACTICAL IMPLEMENTATION OF STABILITY BRACING STRENGTH AND STIFFNESS GUIDELINES FOR STEEL I-GIRDER BRIDGES

DOMENIC COLETTI

MICHAEL GRUBB

BIOGRAPHY

Domenic Coletti is a Principal Professional Associate with the Raleigh, NC office of HDR. He has 28 years of structural design experience in Project Engineer, Lead Engineer, Senior Technical Advisor, and Project Manager roles, including the design of highway, railroad, and pedestrian bridges, as well as curved steel girder research. He has authored and presented numerous papers on bridge design and is a member or chair of technical committees of several national professional organizations.

Michael Grubb is a self-employed steel-bridge design consultant with M.A. Grubb & Associates, LLC in Wexford, PA. He has 36 years of experience in steel-bridge design, steel-bridge design specifications, straight and curved steel-bridge research, the development and delivery of training courses on steel-bridge design, and the development of comprehensive steel-bridge design examples.

SUMMARY

Traditionally, cross-frames for straight steel I-girder bridges have been designed with consideration of little more than wind loads and individual member slenderness criteria. While this practice has usually resulted in acceptable designs, the lack of quantification of design loads has been disconcerting to some engineers, and some have questioned if this practice is sufficient.

Recent research by Yura and Helwig has produced guidelines for assessing the minimum strength and stiffness requirements for bracing members such as the cross-frames of straight steel I-girder bridges with little or no skew, where simplified line-girder analysis methods, which do not produce any assessment of cross-frame member forces, are commonly used. However, specific guidance is lacking with regard to practical implementation of these guidelines within the context of composite steel I-girder bridge design performed under the provisions of the AASHTO LRFD Bridge Design Specifications. This paper recommends appropriate load factors and load combinations for use with these guidelines, and discusses recommendations for their implementation in positive- and negative-moment regions of multiple-span continuous steel I-girder bridges.

Practical Implementation of Stability Bracing Strength and Stiffness Guidelines for Steel I-Girder Bridges

Background

Various types of loads should be considered when designing cross-frames for straight steel I-girder bridges. Traditionally, many engineers would often design cross-frames for these types of bridges solely based on wind loads and individual member slenderness criteria. In some cases, standard cross-frame designs, based on generic calculations and/or successful past use, and requiring no bridge-specific analysis by the designer, have been utilized. In many cases, the resulting cross-frames have featured slender members and minimal connections. While these practices have usually resulted in acceptable designs, the lack of quantification of design loads has been disconcerting to some engineers, and some have questioned if this approach is sufficient. Simultaneously, bridges continue to be designed for increasingly longer span lengths with more slender girders, leading some designers to ask: "How strong does a cross-frame need to be to sufficiently function as bracing for a girder?"

Recent research has advanced the state-of-the-art in bridge engineering, particularly in the area of cross-frame design. For example, White, et al., Reference (*1*), provides key insights and practical guidance for analysis of straight steel I-girder bridges with moderately to severely skewed supports and specifically in the area of calculation of cross-frame forces; this work recommends use of refined structural analysis methods for these bridges. These methods feature direct calculation of the forces in individual cross-frame members due to a wide variety of loads including vertical loads (primarily gravity loads), along with horizontal loads induced by wind pressure, thermal expansion and contraction, etc.

Typically in straight steel I-girder bridges with moderately to severely skewed supports, the individual cross-frame member design forces resulting from gravity loading are quite significant. When the procedures outlined in the AASHTO LRFD Bridge Design Specifications, Reference *(2)*, are used, the Strength I limit state (comprised primarily of gravity loads, with no consideration of wind loads), generally controls the design of the cross-frame members, and the resulting cross-frame designs are typically much heavier than if only wind loading and member slenderness criteria are considered. Thus the refined analysis methods recommended by White, et al., Reference (*1*), inherently provide designers with reliable quantification of the controlling design loads for cross-frame members in straight steel I-girder bridges with moderately to severely skewed supports.

White, et al., Reference (*1*), also provide recommendations regarding the analysis of straight steel I-girder bridges in which the supports have little or no skew, specifically the use of traditional, simplified line-girder analysis methods. While line-girder analysis methods are efficient and effective for the design of the girders themselves in a steel I-girder bridge, they evaluate individual girders without consideration of system behavior and thus provide no assessment of cross-frame member forces. As a result, designers using line-girder analysis methods are still left with the question of how to quantify cross-frame design forces.

As previously mentioned, wind loading is one component of the loading in cross-frame members. Wind pressure on the fascia girders produces loads in the girders which are distributed in the structure through the cross-frames. Designers can calculate wind-induced cross-frame forces using any of a number of different approaches. Traditional hand-calculation approaches typically assume that some portion of this loading is carried to the girder top flange and from there directly into the concrete deck of the bridge, while the remaining portion of this loading is carried to the girder bottom flange. The bottom flange is then evaluated as a continuous beam subject to lateral loading and spanning between the cross-frames. The cross-frames receive the bottom flange loading and transmit this loading up to the concrete deck of the bridge, resulting in internal forces in the various cross-frame members (the bottom chord and the diagonals in the case of a truss-type cross frame). Alternately, if a 3D Finite Element Analysis (3D FEA) model of the bridge is available, wind pressures can be applied to the fascia girder and the model results used to predict cross-frame member forces; designers are encouraged to always evaluate the results of refined analysis models using simpler analysis methods.

These approaches to analyzing the effects of wind-induced loading are well established and reasonable in and of themselves. But these approaches do not consider other loads that may be occurring in the cross-frames and do not necessarily indicate the controlling loads in the cross-frame members. As a result, in the end, designers are still left with the question posed near the beginning of this paper: "How strong does a cross-frame need to be to sufficiently function as bracing for a girder?"

An answer to this question has been provided in Yura, Reference (*3*), Yura and Helwig, Reference (*4*), and the *AISC Specifications for Structural Steel Buildings (AISC Specifications)*, Reference (*5*). The fundamental research behind these documents established guidelines for assessing minimum strength and stiffness requirements for bracing members. The guidelines can be used to provide a means to calculate minimum strength and stiffness requirements for the cross-frames of straight steel I-girder bridges with little or no skew, where simplified line-girder analysis methods are commonly used.

However, specific guidance is lacking with regard to practical implementation of these guidelines within the context of composite steel I-girder bridge design performed under the provisions of the *AASHTO LRFD BDS*, Reference (*2*).

Faced with the challenges of applying Yura and Helwig's guidelines during a recent North Carolina Department of Transportation bridge design project and a recent project to update design recommendations published by the Pennsylvania Department of Transportation, the primary author of this paper reviewed Yura and Helwig's work, interpreted it in the context of the *AASHTO LRFD BDS*, and proposed appropriate load factors and load combinations for use with these guidelines. Recommendations for their implementation in positive- and negative-moment regions of multiple-span continuous steel I-girder bridges were also proposed. This paper provides a summary of those recommendations and reviews the implementation of those recommendations on the above-mentioned projects.

The recommendations presented below represent the interpretations and engineering judgment of the authors of this paper and should not be interpreted as being endorsed by Dr. Yura, Dr. Helwig or others.

Stability Bracing Strength and Stiffness Requirements

The design of a typical steel I-girder highway bridge superstructure generally consists of a structural steel framing system and a composite concrete deck. The structural steel framing typically includes a number of steel I-girders spanning between supports and connected to each other by a number of cross-members (typically called diaphragms or cross-frames). In general terms, particularly for bridges with little or no skew, the girders carry gravity loads via bending action, and the cross-frames function primarily as bracing members to enhance the stability of the girders by improving the girders' resistance to lateral torsional buckling.

Lateral Torsional Buckling: Most bridge engineers are familiar with the concept of lateral torsional buckling of steel I-girders. Lateral torsional buckling is one of several common failure modes for steel I-girders subject to major-axis bending, along with local flange buckling and tension flange yielding, as outlined by White, Reference (*6*), and implemented in the *AASHTO LRFD BDS*, Reference (*2*). When a steel I-girder undergoes major-axis bending, the compression flange behaves in a manner loosely analogous to an axially loaded column. Lateral buckling of the compression flange in an I-girder subject exclusively or primarily to major-axis bending is called lateral torsional buckling because the failure mode involves not only lateral buckling of the compression flange while the tension flange remains in its original position, but also a concomitant twisting (or torsional) deformation of the girder. Vertical buckling of the compression flange (buckling about the horizontal axis) is continuously restrained by connection to the girder web, but lateral buckling of the compression flange (buckling about the vertical axis) and twisting of the girder are restrained at discrete points; that is, at the locations of the bracing members.

In a typical steel I-girder bridge, the bracing is provided by cross-frames or diaphragms, which are spaced along the length of the girder. In the *AASHTO LRFD BDS*, Reference (*2*), a "cross-frame" is defined as a transverse truss framework connecting adjacent girders, while a "diaphragm" is defined as a vertically oriented solid-web transverse member connecting adjacent girders. In this paper, the term "cross-frame" is generally considered synonymous with the term "diaphragm", and the methods described herein may mostly be applied to both. The unbraced length of the compression flange equates to the cross-frame spacing, which is one of the primary variables affecting the flexural capacity of a steel I-girder. In situations where the flexural capacity of a steel I-girder is controlled by the lateral torsional buckling capacity, the designer can increase the capacity of the girder by either increasing the size of the girder compression flange (in particular the flange width) or by decreasing the cross-frame spacing.

Minimum Requirements for Stability Bracing: But what constitutes an adequate brace? In other words, how big do the members of a cross-frame need to be to function adequately as a brace point for the compression flange of a girder? Historically, there have been no widely accepted criteria for quantifying the required size of cross-frame members, other than perhaps providing sufficient capacity to carry design forces resulting from wind loads on the bridge, or meeting minimum slenderness requirements such as those presented in the *AASHTO LRFD BDS*, Reference (*2*), in Article 6.9.3:

$$\frac{Kl}{r} \leq 140 \tag{1}$$

Dr. Joseph Yura of the University of Texas at Austin proposed minimum strength and stiffness requirements for cross-frames functioning as braces for compression flanges of I-girders in major-axis bending. One of the earliest papers presenting these requirements on a wide basis was written by Yura, Reference (*3*). Yura's proposed requirements were eventually adopted by AISC in Appendix 6 of the AISC specifications [most recently published as the *Specifications for Structural Steel Buildings*, Reference (*5*)]. More recently, similar recommendations for stability bracing requirements were presented by Yura and Helwig in the Federal Highway Administration (FHWA) *Steel Bridge Design Handbook* (*FHWA SBDH*), Reference (*4*).

Yura's recommended requirements are fairly straightforward, and Reference (*3*) provides a clear, succinct and well-illustrated presentation of their derivation and their basic implementation, including a short design example. Readers are directed to that paper for a complete and fairly easy-to-follow discussion of Yura's concepts. The most important point is that stability bracing must possess both of the following characteristics:

- *Sufficient stiffness:* Stability bracing must have sufficient stiffness to control the lateral deflection of the girder compression flange under axial loading. If the stability bracing has insufficient stiffness, the lateral deflection of the compression flange will become large, and the magnitude of the lateral deflection will directly affect the magnitude of the lateral force applied to the brace.
- *Sufficient strength:* Stability bracing members must have sufficient strength to resist the lateral force applied to the brace by the compression flange as the flange undergoes lateral deflection during axial loading.

The measures of sufficient stiffness and strength are quantified by Yura in his work. Each of the three cited works that present these requirements, References (*3*), (*4*), and (*5*), uses slightly different variable names and/or formulations. For the purposes of this paper, the primary author generally used the variable names and formulations presented in the *FHWA SBDH*, Reference (*4*), which was written specifically for bridge design applications; however select variable names were adjusted when appropriate for consistency with the *AASHTO LRFD BDS*, Reference (*2*).

The equation for the calculation of the required bracing stiffness, $(\beta_T)_{req}$, is:

$$(\beta_T)_{req} = \frac{\beta_T}{\left(1 - \frac{\beta_T}{\beta_{sec}}\right)} \tag{2}$$

The equation for the calculation of the required bracing strength, $(M_{br})_{req}$, is:

$$(M_{br})_{req} = \frac{(0.005) L_b\, L\, M_f^2}{n\, E\, I_{eff}\, C_b^2\, h_o} \tag{3}$$

where:

β_T = overall required brace system stiffness (kip-in./rad)

$$= \frac{2.4\, L\, M_f^2}{\phi n\, E\, I_{eff}\, C_b^2} \tag{4}$$

β_{sec} = web distortional stiffness (kip-in./rad). For full-depth cross-frame connection plates, β_{sec} can be taken equal to infinity. For partial-depth diaphragm connection plates, the reader is directed to the *FHWA SBDH*, Reference (*4*). Further discussion of the β_{sec} term is provided later in this paper.

L = span length (in.)

M_f = maximum factored major-axis bending moment in the region (i.e. positive or negative moment region) and span under consideration for the Limit-State load combination under consideration (kip-in.)

ϕ = resistance factor for bracing = 0.80

n = number of cross-frames within the span

E = modulus of elasticity of steel, 29,000 ksi

I_{eff} = effective moment of inertia (in.^{4}) calculated as follows:

- For doubly symmetric girders:

$$I_{eff} = I_y \tag{5}$$

- For singly symmetric girders:

$$I_{eff} = I_{yc} + \left(\frac{t}{c}\right) I_{yt} \tag{6}$$

I_{yc}, I_{yt} = moments of inertia of the compression and tension flange, respectively, about the vertical centroidal axis of a single girder within the span under consideration (in.^{4})

I_y = noncomposite moment of inertia about the vertical centroidal axis of a single girder within the span under consideration (in.^{4})

c = distance from the centroid of the noncomposite steel section to the centroid of the compression flange (in.). The distance is taken as positive.

t = distance from the centroid of the noncomposite steel section to the centroid of the tension flange (in.). The distance is taken as positive.

C_b = moment gradient modifier

L_b = unbraced length (i.e., cross-frame spacing) (in.)

h_o = distance between the flange centroids (in.)

The required bracing stiffness, $(\beta_T)_{req}$, from Eq. (*2*) is checked against the actual overall brace system stiffness, $(\beta_T)_{act}$, given as:

$$(\beta_T)_{act} = \frac{1}{\left(\frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_g}\right)} \tag{7}$$

where:

β_b = cross-frame or diaphragm system stiffness (kip-in./rad)

β_{sec} = web distortional stiffness (kip-in./rad).

β_g = in-plane girder stiffness (kip-in./rad).

The formulations of β_b, β_{sec}, and β_g are dependent on the specific configuration of the brace system, the web (and its connections to the cross-frame or diaphragm), and the girder system, respectively. The reader is directed to the *FHWA SBDH*, Reference (*4*), for a full presentation of these formulations. Simplifications for some of these formulations are provided later in this paper.

The required bracing strength, $(M_{br})_{req}$, from $(M_{br})_{req} = \frac{(0.005) L_b\, L\, M_f^2}{n\, E\, I_{eff}\, C_b^2\, h_o}$ is converted to stability bracing forces in cross-frame members by dividing $(M_{br})_{req}$ by the distance between the centroids of the top and bottom chords to obtain the required stability chord forces, $(F_{br})_{req}$. The required stability forces in the diagonals may be obtained by multiplying $(F_{br})_{req}$ by L_d/s for an X-type cross-frame configuration (tension-compression), and by $2L_d/s$ for a K-type cross-frame configuration and for an X-type cross-frame configuration (tension only), where L_d is the length of the diagonal and s is the girder spacing.

Appendix 6 of the *AISC Specifications,* Reference (*5*) specifies a resistance factor, ϕ, of 0.75 in Eq. (*4*). However, for the *AASHTO LRFD BDS*, Reference (*2*), both the load model and the resistance model were included in the calibration process and in order to achieve the desired reliability index of 3.5 the resistance factors in the *AASHTO LRFD BDS* were typically set at a level that is 0.05 higher than those in the *AISC Specifications.* Therefore, the use of a resistance factor of 0.80 is recommended for bridge-design applications.

Overall, the equations for the minimum required bracing stiffness and strength are fairly simple, and the contributing terms should be easy to quantify for most routine bridge design situations.

Previous presentations of these requirements, however, lack instructions for the specific implementation of these provisions in practical bridge design situations. Some of the questions that might be asked by a bridge design engineer include:

- How should stability bracing forces be combined with other cross-frame member forces? Which limit states should be investigated, and which load combinations and load factors should be used?
- What, if any, considerations are there for application of these provisions in the positive-moment regions of routine composite steel I-girder bridges?
- How should these provisions be applied when investigating the negative-moment regions of multiple-span continuous composite steel I-girder bridges?
- What simplifications might be exercised when applying these provisions for the design of routine composite steel I-girder bridges?

The remainder of this paper will address these questions.

Limit States, Load Combinations and Load Factors

Section 2.0 of the *FHWA SBDH*, Reference (*4*), includes the following statement: "Using these equations the stability bracing forces are additive to the bracing forces resulting from a first-order type of analysis (dead load, live load, etc.)." However, the *FHWA SBDH* does not provide specific guidance on how to combine these forces, i.e., the *FHWA SBDH* does not indicate which limit states should be investigated or which load combinations and load factors should be used.

During the design of a recent project for the North Carolina Department of Transportation (NCDOT), the primary author of this paper faced this question. Following an examination of the stability bracing provisions and the *AASHTO LRFD BDS*, Reference (*2*), and some informal discussions with the authors of the stability bracing provisions, a project policy was developed for the design of three straight steel I-girder bridges, each consisting of two 2-span or 3-span continuous units. Later, similar questions were faced when preparing updates to the Pennsylvania Department of Transportation (PennDOT) *Design Manual 4* (DM4), Reference (*7*). The project policy used for the NCDOT design project was reviewed, and minor updates to the load factors for the construction loads were implemented. The resulting policy was then also published in the PennDOT BD-619M standard, Reference *(8)*. A summary of the subject limit states and load combinations is provided below.

Wind loads in the bracing members should be calculated using typical methods and then conservatively combined directly with the stability bracing forces in those members (noting that wind forces are fully reversible based on the reversible nature of wind direction), using load factors appropriate for the *AASHTO LRFD BDS* Limit State load combination under investigation. Appropriate load factors for wind loads (WS and WL) are provided below (note that these wind load factors will be changing in the next edition of the *AASHTO LRFD BDS*).

Stability bracing forces in the bracing members and bracing stiffness requirements should be determined using factored major-axis bending moments (M_f) based on dead load and live load effects, using load factors appropriate for the *AASHTO LRFD BDS* Limit State load combination under investigation as summarized below. The definitions of the abbreviations used for various loads (DC, DW, etc.) correspond to their definitions in the *AASHTO LRFD BDS*.

- Strength I, Final Condition, Composite, Negative Moment Regions: *1.25 DC + 1.5 DW + 1.75 LL*
- Strength I, Construction Condition, Noncomposite, Positive or Negative Moment Regions: *1.25 DC + 1.5 DW + 1.5 Construction Loads (including dynamic effects if applicable)*
- Strength III, Final Condition, Composite, Negative Moment Regions: *1.25 DC + 1.5 DW + 0 LL + 1.4 WS*
- Strength III, Construction Condition, Noncomposite, Positive or Negative Moment Regions: *1.25 DC + 1.25 DW + 1.25 WS + 1.25 Construction Loads (DC is dead load of structural steel only)*
- Strength V, Final Condition, Composite, Negative Moment Regions: *1.25 DC + 1.5 DW + 1.35 LL + 0.4 WS + 1.0 WL*
- Special Steel Construction Condition, Noncomposite, Positive or Negative Moment Regions: *1.4 DC + 1.5 Construction Loads (including dynamic effects if applicable)*

For the above-listed Limit State load combinations, the following notes apply:

- The Strength I, Construction Condition Limit State load combination should not include wind loading but should include full construction loads associated with deck placement, including consideration of construction live loads and dynamic effects as applicable.
- The Strength III, Construction Condition Limit State load combination should include wind loading and reduced construction loads (such as the weight of static construction equipment and stored materials with no construction live load). The Strength III, Construction Condition load combination need not be checked for deck placement conditions (the Strength I Limit State and Special Steel Construction Condition load combinations cover this condition).
- For the Construction Condition load combinations, DW should include only any applicable utility loads but not future wearing surface loading.
- Once the stability bracing forces have been calculated using these appropriately factored major-axis bending moment (M_f) values, they should be multiplied by a load factor of 1.0 for combination with other force effects in the appropriate load combinations when evaluating bracing strength and stiffness requirements.
- In other words, it is recommended to factor the M_f values used to calculate the stability bracing forces, but once those stability bracing forces are calculated (using factored M_f values) they do not need to be factored a second time for combination with other force effects in the bracing members (such as wind load force effects). It is further recommended that the calculated values of the stability bracing forces for each Limit State load combination be combined only with the other factored forces calculated for that same load combination.

Note that for the case of routine straight steel I-girder bridges in which the supports have little or no skew, the primary source of the M_f values is the line girder analysis used to design the girders, and the primary source of other cross-frame member force effects will likely be separate calculations of wind load force effects. The typical design process would be to first design the girders, choosing appropriate cross-frame spacings as part of the design process as has traditionally been done, and then determine the stability bracing forces (and other cross-frame force effects such as those due to wind loading) for use in designing the cross-frames themselves.

Stability Bracing Design in Positive-Moment Regions vs. Negative-Moment Regions

In Section 2.3.1 of the *FHWA SBDH*, Reference (*4*), M_f is defined as the "maximum moment within the span." For a simple-span bridge, the maximum moment within the span is clearly the maximum positive moment, typically at or near midspan. However, for a multiple-span continuous bridge, there will be both positive-moment regions and negative-moment regions, and the definition of "maximum moment within the span" becomes more complicated. Both the positive- and negative-moment regions should be investigated, but the specific application of Yura's recommendations, particularly in negative-moment regions, is not well defined in the published research.

Yura's research focused primarily on simple-span structures (entire span in positive moment) and primarily on the noncomposite condition. However, the fundamental mechanics underlying Yura's recommendations are somewhat generic and can be characterized in terms of considering compression flanges, regardless of whether that compression flange is the top flange or the bottom flange of the girder. Therefore, breaking down a multiple-span continuous bridge into separate positive-moment and negative-moment regions allows some opportunity to develop ways to interpret and apply Yura's basic recommendations.

Positive-Moment Regions: Positive-moment regions are relatively easy to address. The positive-moment regions in a multiple-span continuous bridge correspond approximately to the case of a simple-span bridge. The span length, L, can potentially be taken as the length between inflection points, but lacking research to back up this approach, it is recommended that the full span length be used. At that point, Yura's recommendations can be directly applied. The noncomposite condition is the only condition which needs investigation, since in positive-moment regions of composite bridges, once the deck is cast and hardened, the top flange (compression flange) is fully braced by the deck and further consideration of stability bracing requirements is not needed.

Negative-Moment Regions: The evaluation of stability bracing requirements in negative-moment regions is less transparent, and more extensive interpretation and engineering judgment is needed to develop policies for implementation of Yura's recommendations for practical bridge design situations. Currently, there is no research available to support recommendations for negative-moment regions, particularly in situations where a composite concrete deck may be bracing the top (tension) flange.

Depending on the stage of construction being investigated, the tension flange may be continuously braced by the hardened deck. It may eventually be determined that once placed and hardened, the deck will sufficiently brace the steel superstructure system in negative-moment regions such that further investigation of the stability bracing requirements for the cross-frames in negative-moment regions is not required. But until such research is conducted, it is prudent to investigate the stability bracing requirements in the negative-moment regions for both noncomposite and composite loading, assuming no beneficial contribution from the hardened deck bracing the tension flange of the girder. In the negative-moment region, avoid excessive conservatism by calculating the stability bracing requirements at the first cross-frame away from the pier, not at the pier. At the pier, it is reasonable to assume that anchor bolts and pier cross-frames will provide sufficient bracing by inspection.

Issues Common to both Positive- and Negative-Moment Regions: M_f should be the maximum factored moment corresponding to the region under investigation (i.e., the value of M_f used to evaluate stability bracing requirements in a positive-moment region should be the maximum positive moment in that region, and the value of M_f used to evaluate stability bracing requirements in a negative-moment region should be the maximum negative moment in that region).

Consider the appropriate cross-frame spacing in each region for the calculation of stability bracing forces. When the values of the variables in the two unbraced segments adjacent to a cross-frame are different, the cross-frame may be designed for the average of the values determined for both segments.

Consider appropriate section properties in each region for the calculation of stability bracing forces (i.e., use the noncomposite section properties of the girder at the point of maximum positive moment for the calculation of positive-moment region stability bracing forces. Use the noncomposite section properties for loads acting on the noncomposite section and use the section properties of the girder plus deck longitudinal reinforcing for loads acting on the composite section at the point of maximum negative moment for the calculation of negative-moment region stability bracing forces).

Practical Simplifications for Typical Routine Bridge Cross-Frames

Simplifications for Cross-Frame System Stiffness, β_b: Section 2.3.2 of the *FHWA SBDH*, Reference (*4*), discusses the stiffness of cross-frame and diaphragm systems, β_b. Figure 9 of the *FHWA SBDH*, titled "Stiffness Formulas for Twin Girder Cross Frames," provides three equations for the calculation of β_b based on the configuration of the cross-frame being investigated. These equations were derived for the case of a single cross-frame bracing two girders. However, in most practical bridge designs, there are more than just two girders. Based on discussions with the researchers, a number of options could be exercised to apply these equations for bridges with more than two girders.

One option would be to adapt the provisions described in Section 2.5 of the *FHWA SBDH*, Reference (*4*), titled "Lean-on or Staggered Bracing". If desired, the equations given in Figure 23 of the *FHWA SBDH* could be used, with the number of girders per cross frame (n_{gc}) calculated as:

$$n_{gc} = \frac{number\ of\ girders\ in\ cross\ section}{number\ of\ cross-frames}$$

However, a conservative, and much simpler approach is to use the equations from Figure 9 of the *FHWA SBDH* as they are presented for all cases. This conservatively treats the system as if it only had two girders even if the actual bridge cross-section has more than two girders. The conservatism introduced is not expected to be excessive.

Simplifications for Web Distortional Stiffness, β_{sec}: Section 2.3.3 of the *FHWA SBDH*, Reference (*4*), discusses the web distortional stiffness parameter, β_{sec}. The presentation is fairly comprehensive and shows several different possible configurations of the connection plates that stiffen the web and connect to the cross-frames.

In most typical bridges, the cross-frame is nearly full depth. In addition, Article 6.6.1.3.1 of the *AASHTO LRFD BDS*, Reference (*2*), requires that the connection plate/stiffener be full depth (i.e., the connection plate/stiffener must extend continuously from the top flange to the bottom flange and be attached to both flanges), except for diaphragm connections on rolled-beam bridges meeting certain conditions. The Commentary to Appendix 6.2 of the *AISC Specifications*, Reference (*5*), states, "When a cross-frame is attached near both flanges or is approximately the same depth as the girder, then web distortion will be insignificant so β_{sec} equals infinity." It goes on to say, "Cross-section distortion effects, β_{sec}, need not be considered when full-depth cross-frames are used for braces." Therefore, for the typical case of full-depth bridge cross-frames with full-depth connection plates/stiffeners, cross-section distortion effects can be neglected here and β_{sec} may simply be taken equal to infinity.

However, Article 6.7.4.2 of the *AASHTO LRFD BDS*, Reference (*2*) does allow cross-frames or diaphragms to be as shallow as 0.5 the depth of the beam for rolled beams, and 0.75 the depth of the girder for plate girders, and there are situations where the use of such details may be warranted. If partial-depth cross-frames or diaphragms are being used, especially if the connection plates/stiffeners are less than full depth (and are not connected to the top and bottom flanges), β_{sec} should be calculated using Eq. 17 of the *FHWA SBDH*, Reference (*4*). β_{sec} is calculated in this case by summing the inverses of the β_i terms, with the stiffness of the portion of the connection plate/stiffener within the height of the cross-frame taken as infinity (such that the inverse of the β_i term for that portion of the connection plate/stiffener is zero), and the stiffness of the portions of the connection plate/stiffener between the top of the cross-frame and the top flange and between the bottom of the cross-frame and the bottom flange calculated appropriately per Eq. 17 of the *FHWA SBDH*.

Simplifications for In-Plane Stiffness of Girders, β_g: Section 2.3.4 of the *FHWA SBDH*, Reference (*4*), states: "For a brace only at midspan in a multi-girder system, the contribution of the in-plane girder flexibility to the brace system stiffness is:" followed by Eq. 18 for β_g, the term representing the in-plane stiffness of the girders. However, in most bridge designs, there is more than just one brace at midspan. A discussion with the researchers clarified that the statement in the *FHWA SBDH* text reflects that it is possible to actually derive the in-plane girder stiffness effect for the case where additional braces are provided along the length of the span (the statement was based on study of a twin girder system). The assumption is that modeling the system assuming only one brace at midspan is the worst-case scenario, and that the calculations will be conservative when additional braces are provided.

For cases where there are more than two girders in the cross-section, it is reasonable to assume that calculating β_g based on the assumption of only one cross-frame at midspan is also conservative. However, for most routine bridge design situations, four, five, or more girders will be present and the effect of the β_g term will be much less significant. If the β_g term dominates the calculation of the overall torsional brace stiffness (calculated by Eq. 15 in the *FHWA SBDH*), that is an indication that a global, system mode of buckling may be possible and a more refined analysis may be warranted (refer also to Article 6.10.3.4.2 of the *AASHTO LRFD BDS*, Reference (*2*).

Practical Design Example

As mentioned earlier in this paper, the primary author was involved in the design of three similar straight steel plate girder bridges with slightly skewed supports. Each bridge had two 2-span or 3-span units. Span lengths ranged from 113′ to 164′. Girder spacing ranged from 9′-4″ to 10′-9″. Girder web depths ranged from 62″ to 74″. Cross-frame spacing ranged from approximately 21′ to 25′.

The girders were designed using a commercial line-girder analysis and design software package. The cross-frame designs considered wind loading (calculated using simple hand analysis methods) and stability bracing strength and stiffness requirements. Specific implementation of the stability bracing strength and stiffness requirements followed a project policy design memo consisting of recommendations essentially similar to those presented in this paper. The actual calculations were programmed into a MathCAD worksheet to facilitate repetitive design of the six similar steel plate girder units. Once the project design policy memo was developed and vetted, the actual calculations associated with implementing the stability bracing strength and stiffness requirements proved to be relatively simple. The resulting design loads were then evaluated using routine cross-frame member and connection design calculations.

The cross-frame designs were controlled by stability bracing strength and stiffness requirements but were not significantly different from previous designs, which evaluated only wind loads and basic slenderness criteria. While no single consistent trend was observed, the stability bracing member forces were generally the same magnitude as the wind forces in those same members. The factored stability bracing forces in top and bottom chord members in one case ranged up to 26 kips, and the factored stability bracing forces in the diagonal members in one case ranged up to 41 kips but were generally less than 20 kips and 25 kips respectively throughout the design of the three bridges. The final controlling cross-frame member design forces, considering both stability bracing forces and wind forces, were in one case as much as 110% greater than what the controlling design forces would have been if only wind loading were considered, but were generally less than approximately 50% greater. It also was found that stability bracing stiffness requirements were generally comparable to the strength requirements. It should be noted that these observations are not the result of any comprehensive study and are based only on the limited review of the design of three bridges of fairly routine, and similar, span length, girder spacing, cross-frame spacing, and girder and cross frame member size parameters.

Representative calculations are provided below to illustrate the application of the stability bracing provisions for a cross-frame in the negative moment region of a multiple-span continuous bridge (first cross-frame away from the interior support) for the final condition. The general arrangement of the subject cross-frame is presented in Figure 1. Recall that the equation for the calculation of the required bracing stiffness, $(\beta_T)_{req}$, is:

$$(\beta_T)_{req} = \frac{\beta_T}{\left(1-\frac{\beta_T}{\beta_{sec}}\right)} \tag{2}$$

The equation for the calculation of the required bracing strength, $(M_{br})_{req}$, is:

$$(M_{br})_{req} = \frac{(0.005)L_b\, L\, M_f^2}{n\, E\, I_{eff}\, C_b^2\, h_o} \tag{3}$$

where:

β_T = overall required brace system stiffness (kip-in./rad)

$$= \frac{2.4\, L\, M_f^2}{\phi n\, E\, I_{eff}\, C_b^2} \tag{4}$$

β_{sec} = web distortional stiffness (kip-in./rad). For full-depth cross-frame connection plates, β_{sec} can be taken equal to infinity. For this case, for illustration, the value of β_{sec} was calculated to be 12,910,512 kip-in./rad

L = span length (in.) = 147.5 ft = 1770 in.

M_f = maximum factored major-axis bending moment in the region (i.e. positive or negative moment region) and span under consideration for the Limit-State load combination under consideration (kip-in.)… see summary below

ϕ = resistance factor for bracing = 0.80

n = number of cross-frames within the span = 5

I_{eff} = effective moment of inertia (in.^{4}) = 981.5 in.^{4}

C_b = moment gradient modifier, conservatively taken as 1.0

L_b = unbraced length (i.e., cross-frame spacing) (in.) = 24.75 ft = 297 in.

h_o = distance between the flange centroids (in.) = 69.375 in.

The required bracing stiffness, $(\beta_T)_{req}$, from Eq. (*2*) is checked against the actual overall brace system stiffness, $(\beta_T)_{act}$, given as:

$$(\beta_T)_{act} = \frac{1}{\left(\frac{1}{\beta_b}+\frac{1}{\beta_{sec}}+\frac{1}{\beta_g}\right)} \tag{7}$$

where, by separate calculations for the subject cross-frame:

β_b = cross-frame system stiffness (kip-in./rad) = 1,540,514 kip-in./rad

β_{sec} = web distortional stiffness (kip-in./rad) = 12,910,512 kip-in./rad

β_g = in-plane girder stiffness (kip-in./rad) = 339,863 kip-in./rad

thus:

$(\beta_T)_{act}$ = 272,557 kip-in./rad

As a point of interest, it is informative to note the relative magnitude of each of the parameters β_b, β_{sec}, and β_g as presented here for this design example; specifically note that the value of β_{sec} is significantly larger than the values of β_b, and β_g.

As will be seen later in this paper, the subject cross-frame in this design example is a full-depth cross-frame. As was mentioned previously in this paper, for the case of full-depth bridge cross-frames with full-depth connection plates/stiffeners, cross-section distortion effects can be neglected here and β_{sec} may simply be taken equal to infinity. In this particular design example, substituting infinity as the value of β_{sec} would change the final calculated value of $(\beta_T)_{act}$ from 272,557 kip-in./rad to 278,432 kip-in./rad, only a 2% change.

For the subject bridge illustrated here, by separate calculations the maximum factored negative moments at the first cross-frame away from the support were 77,136 K-in. for the Strength I Limit State load combination, 36,696 K-in. for the Strength III Limit State load combination, and 67,860 K-in. for the Strength V Limit State load combination. Since it exhibits the largest value of the factored girder design moment, the Strength I Limit State load combination will control for evaluation of the bracing stiffness. Using 77,136 K-in as the value for M_f in Eqs. (*4*) and (*2*), the required cross-frame stiffness, $(\beta_T)_{req}$, is calculated to be 225,883 kip-in./rad. This is less than the actual overall brace system stiffness, $(\beta_T)_{act}$, which is 272,557 kip-in./rad. Thus the subject cross-frame has sufficient stiffness.

For the determination of the controlling bracing strength requirements, wind-induced loading effects must also be considered, and all three strength limit state load combinations (Strength I, III, and V) must be investigated. For the subject cross-frame, the unfactored (service level) wind-induced forces in the cross-frame chord and diagonal members were determined by simplified hand calculations to be 8.8 kips and 13.8 kips, respectively. The required bracing strength expressed as a moment applied to the cross-frame, $(M_{br})_{req}$, was calculated using Eq. (*3*), while the individual cross-frame chord and diagonal stability bracing forces were determined using the equations for K-type cross-frames, presented earlier in this paper. Select results are presented in Table 1; for this case of a cross-frame located in the negative moment region, the "composite, final condition" cases controlled over the "noncomposite, construction condition" cases mentioned earlier in this paper. In this instance, the Strength I Limit State load combination produced the controlling design forces, due primarily to the inclusion of live load force effects. For a similar cross-frame in a positive moment region the "composite, final condition" cases would not be investigated.

Limit State	Load Factors				Wind Loading		Stability Bracing Forces		Total Loading	
	γ_{DC}	γ_{DW}	γ_{LL}	γ_{WS}	W_{chord} (kips)	W_{diag} (kips)	S_{chord} (kips)	S_{diag} (kips)	F_{chord} (kips)	F_{diag} (kips)
Str I	1.25	1.5	1.75	0.0	0.0	0.0	25.8	40.5	25.8	40.5
Str III	1.25	1.5	0	1.4	12.3	19.3	5.8	9.2	18.1	28.5
Str V	1.25	1.5	1.35	0.4	3.5	5.5	19.9	31.4	23.5	36.9

Table 1: Summary of select cross-frame member design forces from example design.

The calculations presented above are only a representative example of the implementation of the stability bracing provisions for a single given cross-frame location, configuration, and loading; similar calculations were performed for other locations, configurations, and loadings for the subject bridges. The resulting cross-frames featured fairly reasonable member and connection sizes, as shown in the typical cross-frame detail presented in Figure 1:

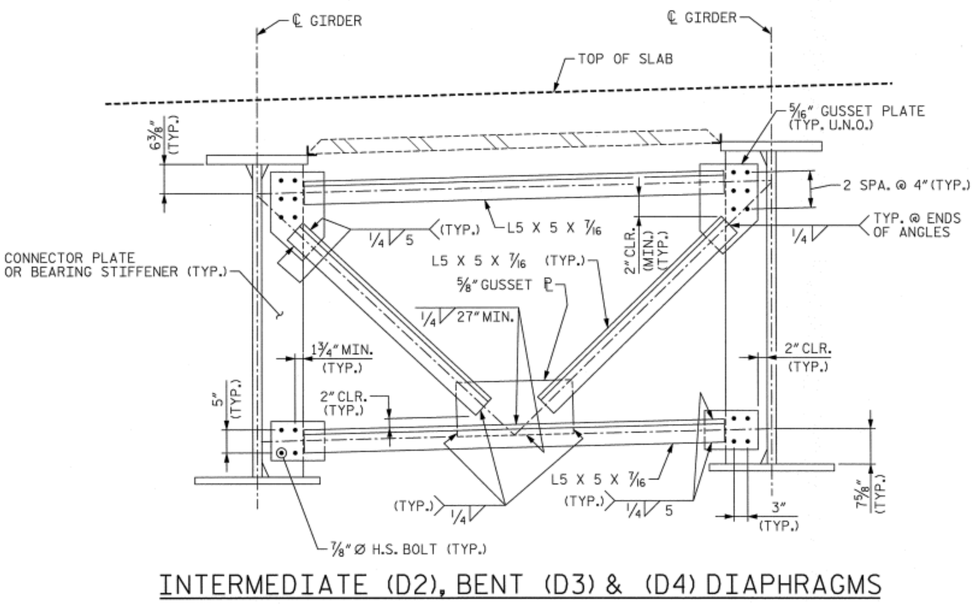

Figure 1: Cross-frame details from example design.

Miscellaneous Suggestions for Economical Cross-Frame Design

There are many industry reference documents that provide suggestions and recommendations for economical cross-frame design. A full listing of these references is beyond the scope of this paper, but two recommended sources are:

- The AASHTO/NSBA Steel Bridge Collaboration publishes a number of helpful guideline documents and guide specifications addressing a variety of topics related to economical steel bridge design and construction. In particular, AASHTO/NSBA Guideline *G12.1, Guidelines for Design for Constructibility*, Reference (*9*), addresses a wide range of issues related to economical steel bridge design, including economical cross-frame design. The 1st Edition of this guideline document was published in 2003; the 2nd Edition is currently being completed, with anticipated publication in 2016. All AASHTO/NSBA Steel Bridge Collaboration guideline documents and guide specifications are available for free download from the AASHTO Bookstore website and the NSBA website.
- PennDOT Standard BD-619M, Reference *(8)*, provides a fairly thorough set of recommendations for cross-frame design, including recommendations on analysis methods, framing plan geometry, cross-frame geometry and orientation, selection of cross-frame/diaphragm types, selection of cross-frame/diaphragm members, implementation of stability bracing strength and stiffness requirements, member and connection design procedures, and example cross-frame/diaphragm details. This PennDOT standard is available for free download from the PennDOT website.

Acknowledgements

The example design discussed in this paper was performed by HDR as part of a Task Order under a Limited Services Contract with the NCDOT Structures Management Unit for the design of project U-2519CB. The updating of PennDOT Standard BD-619M was performed by HDR and M.A. Grubb & Associates, LLC as part of a Task Order under a Structural and

Structure-Related Services Open End Agreement with PennDOT for updating various Standards and Design Manual sections to reflect advances in bridge engineering for steel girder bridges. The authors gratefully acknowledge the permission and support of both NCDOT and PennDOT in the development of this paper, particularly W. Kevin Fischer of the NCDOT Structures Management Unit, who reviewed a draft of this paper and provided valuable comments.

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