# Comprehensive Update to AASHTO LRFD Provisions for Flexural Design of Bridge I-Girders

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## *Introduction*

The provisions of Article 6.10 of the 2004 AASHTO LRFD Specifications for Bridge Design, pertaining to the design of steel I-sections in flexure, have been revised in their entirety relative to the previous Specifications to simplify their logic, organization and application while also improving their accuracy and generality. This paper provides a brief overview of the new Article 6.10 provisions. Various developments pertaining to the technical background of the new provisions are summarized in (White 2004).

## *Handling of Flange Lateral Bending*

The provisions of Article 6.10 and their associated optional Appendices A and B provide a unified approach for consideration of combined major-axis bending and flange lateral bending from any source in both straight and horizontally curved I-girders. As such, general design equations are provided that include the consideration of both major-axis and flange lateral bending. For straight girders, flange lateral bending is caused by wind and by torsion from various origins. Sources of significant flange lateral bending due to torsion include eccentric slab overhang loads acting on cantilever forming brackets placed along exterior members, staggered cross-frames, and significant support skew. When the above effects are judged to be insignificant or incidental, the flange lateral bending stress, $f_\ell$, is simply set equal to zero in the appropriate equations. The format of the equations then reduces to the more conventional and familiar format for checking the nominal flexural resistance of I-sections in the absence of flange lateral bending.

The revised Article 6.10 provisions presently do not incorporate all the necessary requirements for horizontally curved bridge design. It is anticipated that these requirements will soon be incorporated under a separate NCHRP Project 12-52 effort, and that some additional restrictions will be placed on the application of the provisions in Article 6.10 to curved bridge design. The present revisions made in this Article lay the necessary groundwork for accomplishing that effort in an efficient manner.

***Composite Sections in Negative Flexure and Noncomposite Sections with Compact and Noncompact Webs – Appendix A of the New Specification Provisions***

For composite sections in negative flexure and noncomposite sections, the provisions within the body of Article 6.10 limit the nominal flexural resistance to a maximum of the moment at first yield. As a result, the nominal flexural resistance for these sections is conveniently expressed in terms of the elastically computed flange stress. When these sections satisfy specific steel grade requirements and have webs that are classified as either compact or noncompact, the optional provisions of Appendix A may be applied to determine the flexural resistance, which may exceed the moment at first yield. Therefore, the flexural resistance is expressed in terms of moment in Appendix A. The provisions of Appendix A are a direct extension of and are fully consistent with the main provisions of Article 6.10.

The previous Specifications defined *sections* as either compact or noncompact and did not explicitly distinguish between a *noncompact web* and a *slender web*. The proposed provisions make explicit use of these definitions because the noncompact web limit serves as a useful anchor point for a continuous representation of the maximum potential section capacities from values less than the nominal yield moment up to strengths equal to the plastic moment resistance. Because sections with compact or nearly compact webs are less commonly used, the provisions for sections with compact or noncompact webs have been placed in an appendix in order to simplify and streamline the main provisions. The main provisions within the body of Article 6.10 may be used for these types of sections to obtain an accurate to somewhat conservative determination of the flexural resistance calculated using Appendix A. For girders that are proportioned with webs near the noncompact web slenderness limit, the provisions of Article 6.10 and Appendix A produce the same strength for all practical purposes, with the exception of cases with large unsupported lengths sometimes encountered during construction. In these cases, Appendix A gives a larger more accurate flexural resistance calculation.

***Redistribution of Pier Moments in Continuous-Span Bridges – Appendix B of the New Specification Provisions***

Minor yielding at interior piers of continuous spans results in redistribution of the moments. For straight continuous-span flexural members that satisfy requirements intended to ensure adequate ductility and robustness of the pier sections, the procedures of Appendix B may be used to calculate the redistribution moments at the service and/or strength limit states. These provisions replace the former ten-percent redistribution allowance as well as the former inelastic analysis procedures. They provide a simple calculated percentage redistribution from interior-pier sections. This approach utilizes elastic moment envelopes and does not require the direct use of any inelastic analysis. In as such, the new procedures are substantially simpler and more streamlined than the inelastic analysis procedures of the previous Specifications. Where appropriate, these provisions make it possible to use prismatic sections along the entire length of the bridge or between field splices. This practice can improve overall fatigue resistance and provide significant fabrication economies.

### *Comprehensive Flowcharts and Summary of Fundamental Section Property Calcluations – Appendices C and D of the New Specification Provisions*

Flow charts for flexural design of I-sections according to the proposed provisions, along with a revised outline giving the basic steps for steel-bridge superstructure design, are provided in Appendix C of the new provisions. Fundamental section property calculations for flexural members previously found in Article 6.10.3 have been placed in a new Appendix D.

### *Overall Summary of Improvements*

A partial list of other important specific improvements to the Specifications, with some explanations of the changes, are provided below:

- In Article 6.10.1, numerous miscellaneous improvements have been made pertaining to the calculation of terms used throughout the subsequent provisions. These include simplified yet more complete and more general guidelines for calculation of stresses in composite sections, as well as guidelines for calculation of stresses and moments for checking of various stability limit states including the effects of combined major-axis and lateral bending. Also, the provisions for the handling of hybrid girders and variable web depth members are more complete and more general. The computation of the hybrid factor, $R_h$, has been generalized and reduced to a single equation. The web load shedding parameter associated with the web post-bend buckling response, $R_b$, has been simplified by removing its dependency on the applied load level. Instances in the prior Specifications where the flexural resistance depended on the applied loading have been eliminated or minimized throughout the proposed Specifications wherever possible. This mitigates potential difficulties associated with subsequent load rating. The calculation of $R_b$ has also been generalized to address in a simplified fashion the load shedding effects in composite girders in regions of positive flexure with thin webs where necessary. The calculation of $R_b$ for the majority of composite I-girders in positive bending has been eliminated. The provisions for calculation of the web bend buckling resistances, $F_{crw}$, are now located in one place within the Specifications, and one equation is now specified for these calculations. The checking of a potential fracture limit state in tension flanges containing holes has been dramatically simplified by writing the resistance in terms of a computed elastic flange stress acting on the tension flange gross area.

- The various general limits for proportioning of I-sections have been comprehensively reevaluated, updated to a simpler set of limits, and located in one brief article, Article 6.10.2. Web slenderness limits are expressed in terms of the web depth, D, rather than the depth of the web in compression, $D_c$, to allow for simpler proportioning of the web in preliminary design.

- The provisions for checking of constructibility have been updated and streamlined. For main flexural members, the primary constructibility checks amount to the prevention of nominal yielding or reliance on post-buckling resistance under the factored construction loads, in addition to ensuring that the members have adequate strength under these conditions. Both major-axis flexure and flange lateral bending are addressed. See Article 6.10.3.

- The service limit state checks of the new Article 6.10.4 pertaining to the control of permanent deformations have been updated to account for the effects of flange lateral bending. These updated checks can be particularly important for sections in positive flexure for which the flexural resistance in the absence of flange lateral bending is close to or equal to the plastic moment resistance $M_p$.

- The fatigue and fracture limit state checks of the new Article 6.10.5 have been simplified by recognizing that the web bend buckling check under the Service II Load Combination will generally control relative to the same check under the unfactored permanent load plus the specified factored fatigue load.

- The flexural resistance equations for composite sections in positive flexure have been rewritten to make them shorter and simpler, and to better characterize their original intent. The proposed equations in Article 6.10.7 give the nominal flexural resistance of compact sections as the plastic moment capacity $M_n = M_p$ when $D_p/D_t \leq 0.1$, where $D_p$ is the depth from the top of the slab to the plastic neutral axis of the section and $D_t$ is the total depth from the top of the slab to the bottom of the section. The resistance is reduced from $M_p$ as a linear function of $D_p/D_t$ for larger values of this parameter. For continuous-span members, the pier sections are required to have specific characteristics that ensure adequate ductility and robustness to allow them to handle potential inelastic rotations associated with the yielding in the positive moment regions required to develop resistances significantly larger than the yield moment resistance. These requirements are directly linked to the requirements for inelastic redistribution of pier section moments in the proposed Appendix B. If they are not satisfied, the positive moment flexural resistance is limited to $1.3R_hM_y$, as required in the previous Specifications when the pier sections were not compact.

- All of the flexural resistance provisions for flange local buckling (FLB) and lateral-torsional buckling (LTB) in Articles 6.10.8 and the related Article A6.3 are based consistently on the logic of identifying two anchor points, shown in Figure C6.10.8.2.1-1 of the proposed provisions. Anchor point 1 is located at the length $L_b = L_p$ (for LTB) or flange slenderness $b_{fc}/2t_{fc} = \lambda_{pf}$ (for FLB) corresponding to the development of the maximum potential flexural resistance (labeled as $F_{max}$ or $M_{max}$ in the figure). Anchor point 2 is located at the smallest length $L_b = L_r$ or flange slenderness $b_{fc}/2t_{fc} = \lambda_{rf}$ for which the FLB or LTB strengths are governed by elastic buckling.

- The FLB resistance equations within the proposed Article 6.10.8 and Appendix A6.3 are a more rational, liberal and accurate characterization of corresponding theoretical and experimental strengths than the FLB equations of the previous Specifications. In the previous Specifications, significantly larger flexural resistances are obtained relative to those specified by the FLB equations if the compression flange and a portion of the web are considered as an equivalent column. The FLB equations in the proposed provisions retain a similar degree of simplicity while achieving a more realistic strength characterization.

- The proposed provisions for the LTB resistance replace three different variations on writing the LTB resistance in the prior Specifications with one single consistent and more accurate set of equations that apply to all types of noncomposite members, as well as composite members in negative flexure. Several of the variations on the LTB equations in the previous Specifications produce slightly inconsistent or anomalous results. The proposed LTB resistance equations have been developed based on analysis of extensive experimental results from the past 50 years as well as consideration of various analytical predictions.

- The recommended LTB equations are shorter in form than other alternatives. The fundamental base elastic LTB equation in Article A6.3.3 is expressed in terms of the parameters $L_b/r_t$, and $J/S_{xc}h$, which are familiar to structural engineers and are commonly used in other design checks or are easily calculated. An equation is provided for calculation of $r_t$ in terms of basic I-section dimensional parameters. The commentary discusses typical values of key unbraced length limits in terms of the basic ratio to the compression flange width, i.e., $L_b/b_{fc}$. The commentary also provides specific guidance on checking LTB of unbraced lengths in which the member is nonprismatic, including variable web depth members.

- The appropriate definition and use of the equation for the moment gradient modifier, $C_b$, has been updated and clarified within the context of the moving load problem and the necessary use of moment envelope values in bridge design. Ambiguities in the definitions of $C_b$ have been removed such that these provisions are now "programmable." The simple traditional equation for $C_b$ is appropriately retained, but the definitions of the flange stress and moment ratios in this equation, $f_1/f_2$ and $M_1/M_2$, have been updated to eliminate extreme but practical cases where the previous Specification rules could easily be interpreted in a fashion resulting in significantly unconservative $C_b$ values.

- The format of the proposed flexural resistance equations eliminates a number of discontinuities in the calculated flexural resistances at certain unbraced lengths or flange and web slenderness values that existed in prior Specifications.

- The handling of wind moments using two different procedures in the previous Specifications is replaced by a simpler unified and consistent handling of flange lateral bending due to any source for all I-section types. The proposed handling of flange lateral bending effects, termed the one-third rule, is slightly conservative relative to the previous procedure for compact I-sections and is somewhat more accurate and more liberal than the prior procedure for other sections. The proposed rules for handling of flange lateral bending are based on extensive finite element parametric studies, as well as extensive comparisons to prior and to recent experimental test results.

- The shear resistance provisions of Article 6.10.9 have been updated to account for post-buckling strength in hybrid webs, and the moment-shear interaction equations in the previous Specifications have been eliminated based on the results of recent experimental and analytical research and extensive evaluation of prior experimental test data. The handling requirement given in previous Specifications, which required that additional transverse stiffeners be provided in regions of low shear in girders with thinner webs, has been eliminated because the maximum web slenderness for webs without longitudinal stiffeners is now limited to 150.

- The shear connector provisions of Article 6.10.10 are separated from the provisions for web shear design and updated to make them more compatible with comparable provisions given in the 2002 Guide Specifications for Horizontally Curved Steel Girder Highway Bridges, in anticipation of future unification of these provisions.

- The longitudinal stiffener design provisions of Article 6.10.11 have been updated to address the influence of having a web and/or a longitudinal stiffener with nominally smaller yield strength than that of the compression flange. These provisions have also been made consistent with the more general provisions within the 2002 curved-girder guide specifications in the limit that the horizontal radius of curvature approaches infinity.

- Appendix A defines new terms denoted by the symbols $R_{pc}$ and $R_{pt}$ and referred to as the web plastification factors for the handling of noncomposite sectons and composite sections in negative flexure. These factors account for a smooth continuous increase in the flexural resistance as the web slenderness is decreased from its noncompact slenderness limit, at which point the maximum potential resistance provided by Appendix A and by Article 6.10.8 are both equal to $R_hM_y$, to the compact slenderness limit, at which point the resistance is equal to the plastic moment capacity of the section, $M_p$. The terms $R_{pc}$ and $R_{pt}$ are applied in much the same way as the hybrid and bend buckling parameters $R_h$ and $R_b$ of the main provisions.

- The compact web slenderness limit of Appendix A has been modified to a form that is consistent with the compactness requirements implied by the Q formula of the prior Specifications, but is different from the basic limit provided in the main provisions of the prior Specifications. The modified web compactness limit accounts for the higher demands on the web for many sections that have a large shape factor $M_p/R_hM_y$, such as monosymmetric sections that have a large depth of the web in compression. Combined with the other provisions of Appendix A, the use of the modified web compactness limit eliminates the need for the interaction equation between the web and flange slenderness values that existed in previous Specifications for defining a compact section.

- The provisions of Appendix B account for the fact that the compression flange slenderness, $b_{fc}/2t_{fc}$, and the cross-section aspect ratio, $D/b_{fc}$, are the predominant factors that influence the ductility of the moment-rotation response at adequately braced interior-pier sections. These provisions apply to sections with compact, noncompact and slender webs.

- The flowcharts of the proposed flexural design provisions in Appendix C provide a complete picture of the overall process associated with their application. The Engineer will note that the design of noncomposite sections and composite sections in negative flexure is particularly simple in cases where the flange, web and/or unbraced length are compact. The flowcharts demonstrate that in these cases, the required calculations are particularly minimal.

- Provisions for checking web crippling and web local yielding have been added in Appendix D for webs without bearing stiffeners at locations subjected to concentrated loads not transmitted through a deck or deck system. These provisions are necessary for rational handling of these limit states in cases such as when bridge girders are incrementally launched during construction, as well as for a more rational determination of when bearing stiffeners are not required on rolled shapes.The LTB provisions of Articles 6.10.8.2.3 and Appendix A.3.3 are restated in Articles D6.4.1 and D6.4.2 in a fashion that allows the Engineer to directly calculate the unbraced length requirements necessary for the LTB strengths to achieve the "plateau" associated with the maximum potential flexural resistances, $F_{max}$ or $M_{max}$ shown in Figure C6.10.8.2.1-1. The commentary points out that $C_b$ values only slightly greater than one are sufficient to allow the use of $F_{max}$ or $M_{max}$ for the LTB resistances with significantly larger unbraced lengths than the corresponding unbraced length requirements for uniform major-axis bending. The commentary encourages the Engineer to apply the updated $C_b$ calculation procedures to take advantage of these benefits. Flowcharts of Articles D6.4.1 and D6.4.2 highlighting the target unbraced length requirements to reach $F_{max}$ or $M_{max}$ are provided at the end of Appendix C.

***General Philosophy and Approach***

The proposed provisions are organized to correspond more closely to the flow of the calculations necessary for the design of I-section flexural members. Each of the sub-articles are written such that they are largely self-contained, thus minimizing the need for reference to multiple articles to address any one of the essential design considerations. Many of the individual calculations and equations have been streamlined. Specific guidelines have been provided in the commentary in a number of areas where the previous Specifications have been largely silent. In as such, these provisions should result in greater ease of design. It is anticipated that after the initial stages of becoming accustomed to the proposed Specifications, Engineers will find them to be more logical and easier to understand.

***Anticipated Effect on Bridges***

Updates have been made to various resistance equations that improve the accuracy and generality of the provisions. In a number of cases, these updates will result in substantial improvements in the economy of steel I-beam and girder bridge construction. These improvements include: (1) larger flange local buckling resistances for girders with noncompact flanges, (2) larger lateral-torsional buckling resistances for unbraced lengths that marginally violated previous compactness rules, where Specifications defined a discontinuity in the flexural resistance at the compactness limit, (3) larger flexural resistances for noncompact web I-sections, (4) larger flexural resistances for compact or nearly compact web composite I-sections in negative flexure that employ noncompact cross-frame spacings, (5) fewer transverse stiffeners on hybrid members and near interior-pier regions of all types of members that require the use of such stiffeners, and (6) fewer cross-section transitions along the length of continuous-span members in straight bridges with regular framing when moment redistribution procedures are employed.

Large differences in the nominal flexural and shear resistance of members determined according to these proposed provisions and previous Specifications, where the proposed provisions provide a more conservative characterization of the resistance, have not been identified in verification studies and are not anticipated. A number of cases have been considered carefully where the proposed equations are somewhat more conservative than the previous equations. These cases generally tend to be on the esoteric fringes of practical designs, but nonetheless are important cases that need to be addressed in providing robust and safe Specifications. For instance, the provisions for the required radius of gyration of a longitudinal stiffener have been updated to properly account for the potential early yielding of this element if its nominal yield strength is smaller then that of the compression flange and/or web. For designs in which the yield strengths of these plate elements are nominally the same, the requirements are essentially the same as in the previous Specifications.

The most sweeping effect of these provisions is that they provide the groundwork for eventual development of fully unified Specifications for design of straight and curved I-girder bridges. Unified Specifications will further streamline and improve the overall efficiency of the design process for bridges that contain both curved and straight spans.

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