Improved Flexural Design Provisions for I-Shaped Members and Channels

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INTRODUCTION

The AISC LRFD Specification (AISC 1999) provides arguably the best combination of accuracy and practicality of all the standards for steel design throughout the world. Its provisions for flexural design of I-shaped members and channels are subdivided among three articles: Chapter F, which addresses the design of doubly-symmetric compact I-shapes and channels, Appendix F1, which addresses noncompact and slender-element I-shapes with the exception of sections with slender webs, and Appendix G, which addresses the design of members with slender webs. These provisions have many positive attributes; nevertheless, there are a number of areas in which improvements are possible, both in terms of design simplicity as well as accuracy and economy of the resulting products. In light of efforts to develop a unified AISC LRFD and ASD Specification for 2005, it is worthwhile to consider areas where improvements to prior Specification provisions are merited.

One area of potential improvement is the following. Different sets of equations are specified in Chapter F, Appendix F1 and Appendix G of (AISC 1999) to address the different types of members outlined above. Appendix G was developed largely in an independent fashion from Chapter F and Appendix F ((Cooper et al. 1978; Galambos 1978, Yura et al. 1978, Galambos and Ravindra 1976). The simplicity and accuracy of the Specification can be improved by unifying the concepts and approaches from these separate developments.

For instance, it is possible to express the elastic *lateral-torsional buckling* (LTB) resistance equations of Chapter F, Appendix F and Appendix G in a single unified fashion in terms of parameters that have a strong physical meaning and are commonly used in design calculations. Some of the parameters utilized in the (AISC 1999) equations are difficult to assign a physical meaning to. These parameters specifically are the terms X_1 and X_2 within Chapter F and the terms B_1 and B_2 for monosymmetric I-shapes in Appendix F1. White and Jung (2003a and b) discuss a number of considerations pertaining to the base elastic LTB equations.

In addition, improvements are possible in the handling of web slenderness effects on the calculated flexural resistance. Specifically, the provisions of Appendix F1 address the influence of web slenderness by requiring a separate *web local buckling* (WLB) limit state check. *Lateral-torsional buckling* (LTB) and *flange local buckling* (FLB) are investigated as separate and independent failure modes. As a result, the WLB resistance is handled in effect as a cap on the LTB and FLB strengths. Conversely, Appendix G handles the influence of web slenderness via a reduction factor R_{PG} within its LTB and FLB strength equations. Recent studies (Richter 1998; White et al. 2003a) show that the Appendix G type of approach is a truer representation of the physical limit states behavior. That is, the web slenderness affects the *shape* of the LTB and FLB strength curves, not just the *plateau* of the total strength as implied by Appendix F1. The

web slenderness effects are better represented as a strength reduction factor on the FLB and LTB resistances, not as a separate WLB limit state.

The above differences between Appendices F1 and G can produce a significant discontinuity in the calculated strength for a member with a web proportioned at the noncompact slenderness limit such that either of these provisions apply. Ideally, the predictions from these appendices should be the same for this hypothetical case. Possibly of greater importance though is the fact that the flexural design provisions as a whole can be written in a simpler and more easily understood fashion by handling the influence of the web slenderness in a manner similar to that in Appendix G throughout the Specification.

Another area where improvements to (AISC 1999) are possible is in the handling of monosymmetric I-shaped members in Appendix F1. The following shortcomings of these provisions need to be addressed:

1. Appendix F1 does not provide an equation for the noncompact unbraced length limit, L_r, for monosymmetric I-shapes. Many engineers assume that L_r must be calculated iteratively for these types of sections. White and Jung (2003b) suggest a simplified equation for L_r that is exact for doubly-symmetric I-shaped members and provides an accurate approximation for general I-shaped members and channels. They also derive a closed form equation for L_r from the rigorous beam theory solution for a general monosymmetric open-walled section member.
2. In the (AISC 1999) Appendix F1 LTB provisions, the compact unbraced length limit L_p for a monosymmetric I-shape is based on the radius of gyration of just the compression flange, r_{yc}. As a result, the predicted strength of a symmetric girder can be increased significantly by making it minutely monosymmetric. Use of r_{yc} within the L_p equation neglects the destabilizing effects of flexural compression within the web. An expression for the radius of gyration should be employed that reduces to a form applicable for doubly-symmetric I-shaped members in the limit that the cross-section is made doubly-symmetric.
3. A number of anomalies, inconsistencies and complexities exist in the application of the Appendix F1 WLB strength equations for monosymmetric I-shapes:
   a. Table B5.1 of (AISC 1999) requires that the web width-thickness ratio λ should be taken as h_p/t_w when comparing to the compact web slenderness limit λ_p. However, this table also states that λ should be taken as h/t_w when comparing to the noncompact-web slenderness limit λ_r. The Appendix F1 WLB strength equation for a noncompact web contains both λ_p and λ_r. Therefore, the proper definition of λ in this equation is ambiguous. This equation needs to be modified to make the terms λ, λ_p and λ_r consistent.
   b. For non-hybrid sections, the moment corresponding to λ_r for WLB is defined in (AISC 1999) as $M_r = F_{yf}S_x$. However, S_x is not defined. If S_x is always taken as the section modulus to the compression flange, $M_r = F_{yf}S_x$ can be greater than the plastic moment M_p for highly monosymmetric sections with a larger compression flange.
   c. It is possible that practical sections with a larger compression flange can satisfy the compact web limit $h_p/t_w \leq \lambda_p$ while also violating the noncompact limit $h/t_w \leq \lambda_r$. Therefore the web is classified as a compact element by one definition and as a slender element by another in (AISC 1999). This anomaly is an attribute of the inelastic

behavior of highly monosymmetric I-shapes with a larger compression flange. The neutral axis of these section types tends to move toward the compression flange as the member begins to yield. The proper classification of this type of cross-section needs to be addressed.

d. The (AISC 1999) web compactness limit λ_p for flexural members is independent of the depth of the web in compression. Conversely, the noncompact-web limit λ_r is expressed as a function of h_c/h in Appendix B5.1. The studies by White and Barth (1998) indicate that there are greater demands on the web for monosymmetric I-sections with a larger depth of the web in compression to be able to develop their plastic resistance M_p. That is, a more stringent web compactness limit is needed for these section types. The independently developed Q formula equations in (AASHTO 1998) also suggest that a stockier web is needed for an I-shaped member to be able to develop M_p when a larger depth of the web is in compression.

e. Finally, and possibly of greatest importance, Table B5.1 refers to Appendix B5.1 for specialized definitions of λ_r for monosymmetric I-sections. Appendix B5.1 then defines λ_r as a function of h/h_c. Conversely, Appendix G handles the web slenderness consistently for both doubly-symmetric and monosymmetric I-shaped members (within its R_{PG} equation) by (a) using h_c/t_w for the web width-thickness ratio and (b) comparing this web slenderness to a simple form of the λ_r equation from Table B5.1, i.e., $5.7\sqrt{E/F_{cr}}$ where F_{cr} can be taken conservatively as F_{yf}. This simple form for λ_r is specified only for doubly-symmetric sections in Table B5.1 and Appendix B5.1. The influence of the web slenderness can be handled more simply and consistently for all types of I-sections by using the Appendix G approach, i.e., defining the web slenderness in terms of h_c/t_w and comparing this width-thickness ratio to a slenderness limit that is independent of h_c, throughout the Specification.

A fourth area in which the (AISC 1999) provisions may be improved is in the general definition of the compact bracing limit L_p. Compiled data from uniform bending LTB experimental tests indicates that the (AISC 1999) L_p limits are somewhat too large (White et al. 2003a). For slender-web sections, where it may be considered appropriate to check the compression flange plus a portion of the web as an equivalent column, the base strength obtained by substituting an unbraced length of $L_b = L_p$ into the LRFD column equation is only $0.88M_y$. The original LTB equations proposed for slender-web members by Basler and Thurliman (1961) give a moment capacity for an I-girder with a web proportioned at the noncompact-web slenderness limit of only $0.92M_y$ at $L_b = L_p$. The comparable bracing limits in the British (BSI 1990), European (CEN 1993) and Australian (SAA 1998) standards are generally smaller than the L_p limits in (AISC 1999) for all types of I-sections, including compact I-shapes. The (AISC 1999) L_p limits were established in part by implicitly considering the influence of end restraint effects that often exist at unbraced lengths subjected to uniform bending (Yura et al. 1978). It is possible to allow the Engineer to account for these effects in a simple and practical, but explicit fashion. In as such, the benefits from end restraint are retained where justified without sacrificing accuracy in cases where the physical end restraint conditions are small.

Lastly, there are intricate interactions between Appendix F1, Chapter B and Appendix B for members with noncompact webs that are subjected to axial compression in addition to flexure. It is possible to dramatically simplify these portions of (AISC 1999) by taking an approach similar

to the one in the AISI LRFD (AISI 2001) and Australian (SAA 1998) steel design standards. Simply put, the strength of the member as a column (under zero bending) and the strength of the member as a beam (under zero axial compression) may be calculated as separate anchor points, and the beam-column strength may then be determined by a simple interaction equation between these strengths.

Many of the above suggested improvements pertain to sections *other* than compact rolled I-shapes. With the exception of the first and fourth areas discussed above (i.e., use of simpler unified LTB equations, and definition of L_p without implicitly assumed end restraint effects), the design of compact rolled I-sections is addressed very effectively within Chapter F of (AISC 1999). It is important that any changes to the Specification should improve upon, or at least maintain, the simplicity of the (AISC 1999) provisions for these member types.

This paper presents and discusses a potential set of unified provisions for the flexural resistance of I-shaped members and channels. The suggested provisions retain the many qualities of the prior AISC LRFD provisions, while also incorporating improvements in the above areas. The suggested provisions, which cover the same range of cross-sections as Chapter F, Appendix F and Appendix G of (AISC 1999), span only 6 pages in their entirety. Therefore, they are of a comparable length to the sum of the corresponding (AISC 1999) provisions while providing what the author considers to be an improved organization of the material. For ease of reference, the suggested provisions are attached as Appendix I. Related rules pertaining to flange and web compact and noncompact slenderness limits, which are the domain of Chapter B in (AISC 1999), are summarized in Appendix II. Appendix III provides flowcharts of the suggested provisions for I-shaped members in major-axis bending. These flowcharts are configured to illustrate the unified nature of the suggested procedures, with cross-references to the equation numbers from Appendix I. The suggested provisions and the flowcharts are configured to streamline their flow for routine design cases involving compact and/or braced members. Appendix IV gives updated definitions of symbols.

The following sections discuss the overall organization of the suggested provisions as well as key specifics associated with each of their articles. The paper closes with a summary of the unified nature of the provisions and their correlation with experimental test results.

It should be noted that a number of the suggested improvements also may be applied to other articles of Chapter F. For example, the suggested improved handling of web slenderness effects also can be applied for box-shaped members with noncompact webs.

OVERALL ORGANIZATION

It is proposed that all the provisions for I-shaped members and channels should be included together in a group of articles at the beginning of Chapter F, labeled F2 through F6. This allows for simplifications by taking advantage of the unified nature of the suggested approach.

Articles F2 through F6 are organized as follows. Article F2 addresses the flexural resistance of doubly-symmetric compact I-shaped members and channels bent about their major axis. Although it is possible to express the suggested provisions in a single article applicable for all types of I-shaped members and channels, the resulting complexity of addressing everything in one place would make the Specification more difficult to learn and to apply for routine design.

The Engineer should note that only the LTB limit state need be checked for the member types addressed by Article F2. The separate WLB limit state calculations are eliminated for all I-section types as discussed in the Introduction, and are not a consideration for compact I-shaped members and channels as in (AISC 1999). FLB also of course is not a consideration with respect to the flexural resistance of compact sections. Furthermore, the approach in (AISC 1999) of considering a separate "yielding" limit state is eliminated within the suggested provisions. The LTB resistance equations intrinsically include this limit state; therefore, the consideration of a separate yielding limit state is redundant.

Article F3 addresses the limited number of rolled I-shapes in which the flanges are noncompact. All of the ASTM A6 W, S, M, and HP shapes have compact webs at $F_y \leq 65$ ksi (450 MPa), which is currently the largest value of F_y utilized for these types of sections. This article simply introduces the FLB resistance equation for a section with a noncompact flange, and refers back to Article F2 for the other applicable resistance equations. The nominal resistance is taken as the smaller of the FLB and LTB resistances. Articles F2 and F3 together address all current rolled I-shapes. The Engineer should note that these articles span only 1.5 pages, including a number of user notes.

Article F4 addresses all other compact- and noncompact-web I-shaped members bent about their major axis. These provisions are somewhat more involved than the provisions of Articles F2 and F3 since they address the three separate limit states that can govern the flexural resistance of general I-shaped members: LTB, FLB, and for monosymmetric sections in which the compression flange is larger and $S_{xt} < S_{xc}$, a separate limit state referred to as tension flange yielding (TFY). The specific nature of the TFY limit state is discussed subsequently. The nominal resistance is taken as the smallest value obtained from the LTB, FLB and TFY equations. Although these provisions are of a more comprehensive nature, they require only two pages of the Specification.

Article F5 considers the strength of slender-web I-shaped members bent about their major axis. This article is closely related to Article F4 in its coverage, and is similar to Appendix G of (AISC 1999). Conversely, Article F4 is closest to the I-section provisions of (AISC 1999) Appendix F1 in its scope. Although Articles F4 and F5 could be combined into a single article (the flowcharts of Appendix III present the equations in this fashion), the Specification will be easier to learn and simpler to apply for both slender-web and noncompact-web members if each of these member types are addressed within separate articles. By virtue of references to the applicable equations of the previous articles, Article F5 requires only 1.5 pages.

The Engineer should note that the term "plate girder" is not used within the suggested provisions, as opposed to the use of this term to refer to the types of members for which Appendix G applies in (AISC 1999). In a general context, the term plate girder is often used to refer to a built-up section that is employed as a flexural member. The term "slender-web I-section" is more descriptive of the member type addressed by (AISC 1999) Appendix G and by the suggested Article F5.

Article F6 of the suggested provisions addresses minor-axis bending of I-shaped members and channels in one place. The handling of this limit state is effectively the same for all of these member types; therefore, there is no need to write separate provisions for the different types. These provisions require only one-half of a page.

It is suggested that the proportioning limits for slender-web I-shaped members specified in Appendix G of (AISC 1999) should be located in a separate article referred to in this paper as Article F13. Since these limits apply to I-sections in either major- or minor-axis bending, they are located more appropriately within this general article. Articles F5 and F6 reference Article F13 for these requirements.

The following sections of the paper address detailed considerations pertaining to each of the separate Articles F2 through F6 and F13 of the proposed provisions as well as the related Chapter B plate slenderness limits.

ARTICLE F2 – DOUBLY-SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

As discussed above, Article F2 addresses the flexural resistance of compact I-shaped members and channels bent about their major axis. Compact I-shapes are of course one of the predominant section types for which the AISC Specifications are employed. Therefore, a major goal is to streamline the provisions for these types of sections. As noted above, only the LTB limit state needs to be considered for members composed of these section types.

All the stability related flexural resistance equations within all the proposed provisions are based consistently on the logic of identifying the two anchor points shown in Fig. 1. Anchor point 1 is located at the length $L_b = L_p$ for LTB, or the flange slenderness $[\lambda = b_{fc}/2t_{fc}] = \lambda_p$ (for FLB of I-shapes) or $[\lambda = b_{fc}/t_{fc}] = \lambda_p$ (for FLB of channels), corresponding to development of the maximum potential flexural resistance. This resistance is labeled as M_{max} in the figure, where $M_{max} = M_p$ for compact-section members, but it is generally less than M_p for other cross-section types. Anchor point 2 is located at the smallest length $L_b = L_r$ or flange slenderness λ_r for which the LTB or FLB resistances are governed by elastic buckling. This resistance, i.e., the ordinate of anchor point 2, is taken as $F_{yr}S_x = 0.7F_yS_x$ (or $M_{yr} = 0.7M_y$) in Article F2. The inelastic buckling resistance is expressed simply as a line between these two anchor points. For $L_b > L_r$ or $\lambda > \lambda_r$, the nominal strength is defined explicitly as the theoretical elastic buckling moment. The basic format shown in Fig. 1, adopted from (AISC 1999), greatly facilitates the definition of simple yet comprehensive flexural resistance equations.

For compact I-shaped members and channels, the flexural resistance is simply equal to the plastic moment capacity M_p whenever the unbraced length is less than L_p (see Eq. F2-1). For unbraced lengths between this "compact unbraced length" limit and the "noncompact" limit L_r, the flexural resistance is expressed as a simple linear interpolation between M_p and the moment $F_{yr}S_x$ (see Eq. F2-2), where F_{yr} is given by Eq. F2-6.

The limit $F_{yr} = 0.7F_y$ of Eq. F2-6 is a simplification of the term $F_L = F_y - 10$ ksi ($F_y - 69$ MPa) for rolled shapes and $F_L = F_y - 16.5$ ksi ($F_y - 114$ MPa) for welded built-up shapes from (AISC 1999). (AISC 1999) does not provide an explicit value of F_L for non-welded built-up shapes. The notation F_{yr} is selected within the suggested provisions to emphasize that this term is a flexural stress corresponding to the nominal onset of yielding, with consideration of compression flange residual stress effects. Although it is rational to assume a smaller nominal residual stress effect for rolled shapes, such as define by the F_L equations in (AISC 1999), the statistical correlation with the supporting experimental test data is essentially the same using the suggested approach (White et al. 2003a).

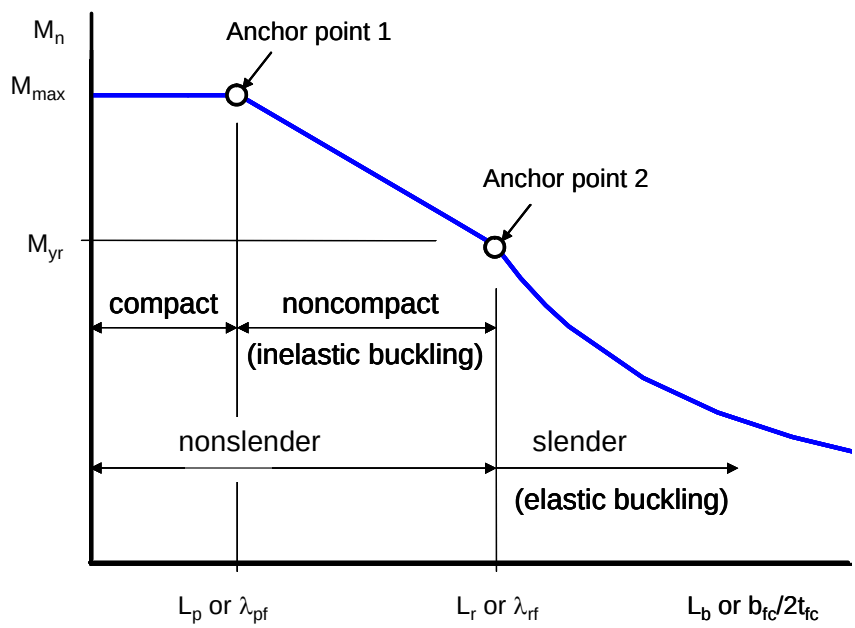


Fig. 1. Basic form of all stability related flexural resistance equations.

Equation F2-3 is a simple expression for the theoretical elastic LTB resistance, applicable for $L_b > L_r$ and $M_n < M_{yr}$. For the case of uniform bending ($C_b = 1.0$) and with r_{ts} specified by Eq. F2-7, this equation gives the exact beam theory based solution for the elastic LTB strength of doubly-symmetric I-shaped members from (Timoshenko and Gere 1961).

Equations F2-U1 and F2-U2 are simplifications of F2-7 for doubly-symmetric I-shapes. Equation F2-U1 is effectively an exact expression for r_{ts}^2, whereas Eq. F2-U2 is generally a conservative approximation obtained by assuming $h \cong d_f \cong d$. For most beam-type I-sections (e.g., $d/b_f \geq 1.7$), the conservatism associated with Eq. F2-U2 is typically less than five percent. However, for I-shapes with thick flanges and/or with shallow depths d (small d/b_f), the approximation given by Eq. F2-U2 can be significantly conservative. The usefulness of Eq. F2-U2 is that it is the traditional r_T for a doubly-symmetric I-shape employed within the LTB provisions of (AISC 1999) Appendix G.

If the suggested provisions were adopted, it is recommended that the section property tables of the AISC Manual should include the exact values for r_{ts}. The Engineer should note that the symbol r_{ts}, rather than r_t, is employed in Article F2 to denote that this radius of gyration is for *doubly-symmetric* I-shapes and channels only, and to guard against the erroneous use of Eqs. F2-7, F2-U1 or F2-U2 for r_t in the subsequent articles (the equations for r_t in the subsequent articles apply to all types of I-sections, whereas Eq. F2-7 applies solely to doubly-symmetric I-shapes and channels and Eqs. F2-U1 and F2-U2 apply solely to doubly-symmetric I-shapes).

The terms within Equation F2-3 are particularly simple to understand, and are easily calculated or are readily available from property tables. The fundamental elastic LTB terms within this equation are:

- the moment gradient factor C_b,

- the slenderness ratio $\frac{L_b}{r_{ts}}$, and
- the ratio of the torsional to the flexural efficiency of the cross-section, $\frac{Jc}{S_x d_f}$, which accounts for the influence of the St. Venant torsional stiffness on the elastic LTB resistance.

For I-shapes, c = 1 and the last term reduces to just $\frac{J}{S_x d_f}$. For I-shapes with rectangular flanges, d_f is the depth between the mid-thickness of the flanges (see Appendix IV). If $d_f \cong d$ is assumed, $\frac{J}{S_x d_f}$ can be written simply as $\frac{J}{2I_x}$.

The calculation of C_b is not addressed specifically within this paper. The Engineer is referred to (Helwig et al. 1997) for a basic modified form of the C_b equation from (AISC 1999) that addresses the LTB resistance of monosymmetric I-shapes more effectively, and for discussion of various important issues related to the proper calculation of C_b. Also, the Engineer is referred to (White and Chang 2003a) for application of the traditional AISC C_b equation to general cases including internal loadings within the unbraced length, moving loads and composite I-shaped members in negative bending. Both of these methods are in general accurate to conservative. The modified equation suggested by Helwig et al. (1997) gives more accurate results for members containing internal loads and with no bracing between the supports in the plane of major-axis bending. The procedure suggested by White and Chang (2003a) gives more accurate results for reversed-curvature bending of monosymmetric I-shapes when the variation of the moment is nearly linear within the unsupported length under consideration.

The source of the simplification within Eq. F2-3 relative to the corresponding form in (AISC 1999) involving the terms X_1 and X_2 is that the equation in (AISC 1999) is written in terms of the slenderness ratio $\frac{L_b}{r_y}$ whereas the suggested equation is written in terms of $\frac{L_b}{r_{ts}}$. Both Eq. F2-3 and the (AISC 1999) equation are exact expressions for the theoretical elastic LTB strength (given an exact value for C_b). The use of $\frac{L_b}{r_{ts}}$ leads to a simpler expression for this strength.

Equation F2-3 is also a far more effective expression for the elastic LTB resistance than the traditional equations in AISC Allowable Stress Design (ASD) (AISC 1989), where the larger of the values obtained from two separate simplified equations is used, one in which the influence the cross-section warping stiffness is neglected and the other in which the influence of the St. Venant torsional stiffness is not included. Clark and Hill (1960) discuss the background associated with the simplified ASD formulas, typically referred to as the double-formula approach, and summarize a number of the approximations considered by the Engineers associated with the early development of the modern AISC Specifications. A more complete summary of developments leading to the modern LTB provisions can be found in the paper by de Vries (1947), the discussions of this paper by Winter, Hall, Higgins, Van Eenam, Hill, Hussey, Brameld, Gaylord, and Julian, and in (SSRC 1976).

White and Jung (2003a) evaluate the contribution of the term $\frac{J}{S_x d_f}$ in Eq. F2-3 for members with F_y = 50 ksi (345 MPa) and a complete range of potential rolled I-section proportions. Figure 2 demonstrates the generally conservative nature of the traditional ASD double-formula approach compared to the exact Eq. F2-3 for a complete range of rolled wide-flange shapes with F_y = 50 ksi (345 MPa). This figure plots the ratio of the elastic critical moment to the yield moment, M_{cr}/M_y, obtained from the ASD double-formula procedure as a function of $\frac{S_x d_f}{J}$. The unbraced length is taken as $L_b = L_r$ from Eq. F2-5. All approximations other than the fundamental one of neglecting either the warping or St. Venant torsional stiffness are removed in the application of the double-formula approach within this plot. The exact Eq. F2-3 gives M_{cr}/M_y = 0.7 for all values of $\frac{S_x d_f}{J}$ in Fig. 2. One can observe that for sections with $\frac{S_x d_f}{J}$ approaching zero, generally heavy column-type wide-flange sections with $b_f \cong d$ and the stockiest flanges and webs, the double-formula approach gives highly accurate results. However, for the majority of rolled wide-flange shapes, the conservative error in the calculation of the critical moment is significant with this method. White and Jung (2003a) provide additional discussion of the fundamental improvement over the traditional ASD Specifications, which already exists in (AISC 1999). Equation F2-3 is just a re-statement of the exact elastic LTB equation from (AISC 1999), but in a simpler format. The Engineer can also observe the importance of the fundamental parameter $\frac{S_x d_f}{J}$ (or $\frac{J}{S_x d_f}$) from Fig. 2.

The term c in Eq. F2-3 is simply a "conversion factor" that allows the use of Eq. F2-3 for channels. Obviously, it is desirable for the Specification to be able to handle channel sections without the need for a separate set of provisions. The unique attribute of channels pertaining to their elastic LTB resistance, aside from their nonsymmetry, comes from the different C_w for these shapes. In line with the approach taken in the previous AISC Specifications, the suggested provisions do not give an explicit C_w equation for channels. This constant can be obtained from property tables or calculated from equations provided in references such as (SSRC 1998).

The physical significance of the term L_r as the abscissa of one of the anchor points for Eq. F2-2 has been explained in the above. Equation F2-5 is a general expression for L_r including the stiffnesses from both warping and St. Venant torsion. This equation gives the same result as the L_r equation from (AISC 1999) for compact doubly-symmetric I-shaped members and channels, but is based on the more fundamental parameters r_{ts}, $\frac{E}{F_{yr}}$ and $\frac{Jc}{S_x d_f}$ rather than r_y, X_1 and X_2.

Several issues associated with the selection of a more restrictive base L_p limit in the suggested provisions than specified in (AISC 1999) have already been discussed in the Introduction. Equation F2-4 gives the suggested L_p limit. White et al. (2003a) find that this equation gives an improved correlation with LTB test results of compact I-shapes in uniform bending relative to the equation $L_p = 1.76 r_y \sqrt{E/F_y}$ from (AISC 1999). For rolled I-shapes, r_{ts}/r_y varies from approximately 1.12 to 1.28. Therefore, Eq. F2-4 suggests that the L_p limit for compact I-shaped members and channels in (AISC 1999) should be between $1.2 r_y \sqrt{E/F_y}$ and

$1.4r_y\sqrt{E/F_y}$ if written in terms of r_y. Statistically, the differences between using Eq. F2-4 versus $L_p = 1.4r_y\sqrt{E/F_y}$ are negligible (White et al. 2003a). Equation F2-4 is selected such that a consistent definition of the radius of gyration is employed throughout all the suggested provisions. White and Jung (2003a) demonstrate that r_{ts} (or r_t in the subsequent provisions) is a more fundamental LTB parameter for I-shapes and channels than the radius of gyration r_y.

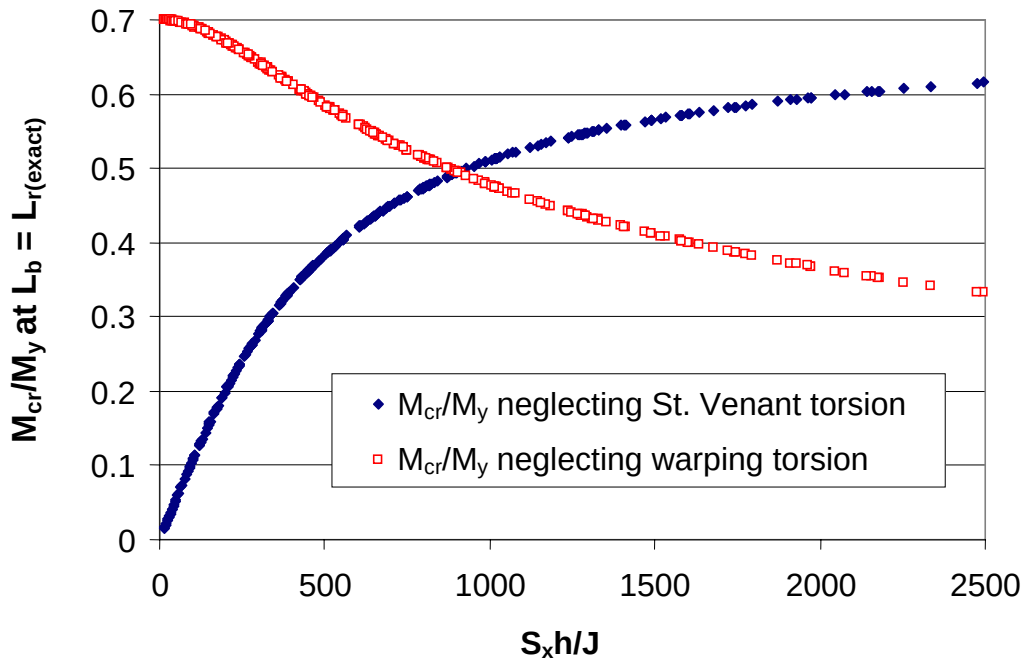


Fig. 2. Ratio of the critical moment obtained from the traditional double-formula approach of the AISC ASD Specifications to the yield moment M_y at $L_b = L_r$ from Eq. F2-5, plotted using $F_{yr} = 0.7F_y$ from Eq. F2-6 and using F_y = 50 ksi (345 MPa) for a comprehensive set of rolled wide-flange shapes (White and Jung 2003a).

Figures 3 and 4 show the implications of the suggested provisions for a representative beam-type wide-flange shape with a narrow flange (a W36x135 with $r_{ts}/r_y = 1.26$) and for a representative column-type wide-flange shape (a W14x132 with $r_{ts}/r_y = 1.13$) respectively. The suggested change in the L_p limit combined with the suggested change from $F_L = F_y - 10$ ksi (F_y – 69 MPa) to $F_{yr} = 0.7F_y$ results in strengths of 0.956 and $0.969M_p$ for unbraced lengths equal to the (AISC 1999) L_p limit. These changes result in strengths of 0.938 and 0.948 of the values predicted by the (AISC 1999) provisions for unbraced lengths equal to the (AISC 1999) L_r limit. Figures 3 and 4 illustrate the general result that the net influence of these changes on the base resistances for rolled wide-flange shapes is typically about four to six percent as a maximum. Also, they illustrate that the effect of the change from the equations for F_L in (AISC 1999) to the suggested F_{yr} equation is typically larger than the effect of the suggested change in the L_p equation. It should be noted that only the "anchor points" (see Fig. 1) and the corresponding inelastic buckling portion of the LTB strength curve are affected by these changes. The maximum strength plateau and the elastic buckling curve remain the same.

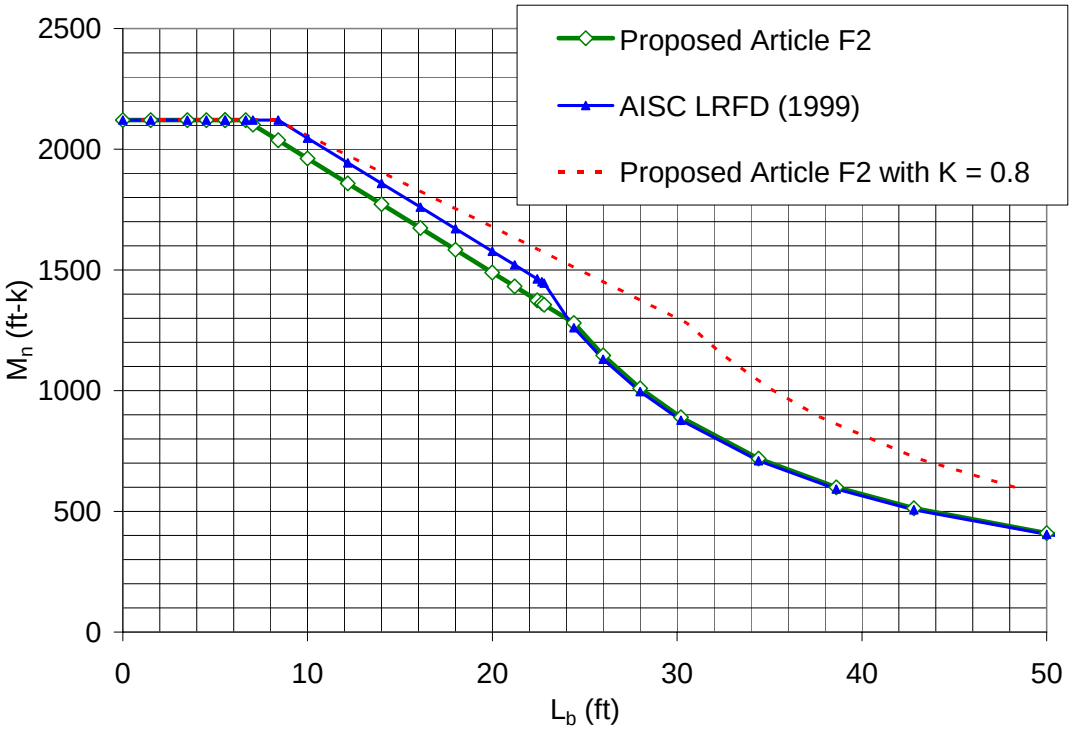


Fig. 3. Suggested and (AISC 1999) strengths for a W36x135 subjected to uniform bending (r_{ts}/r_y = 1.26).

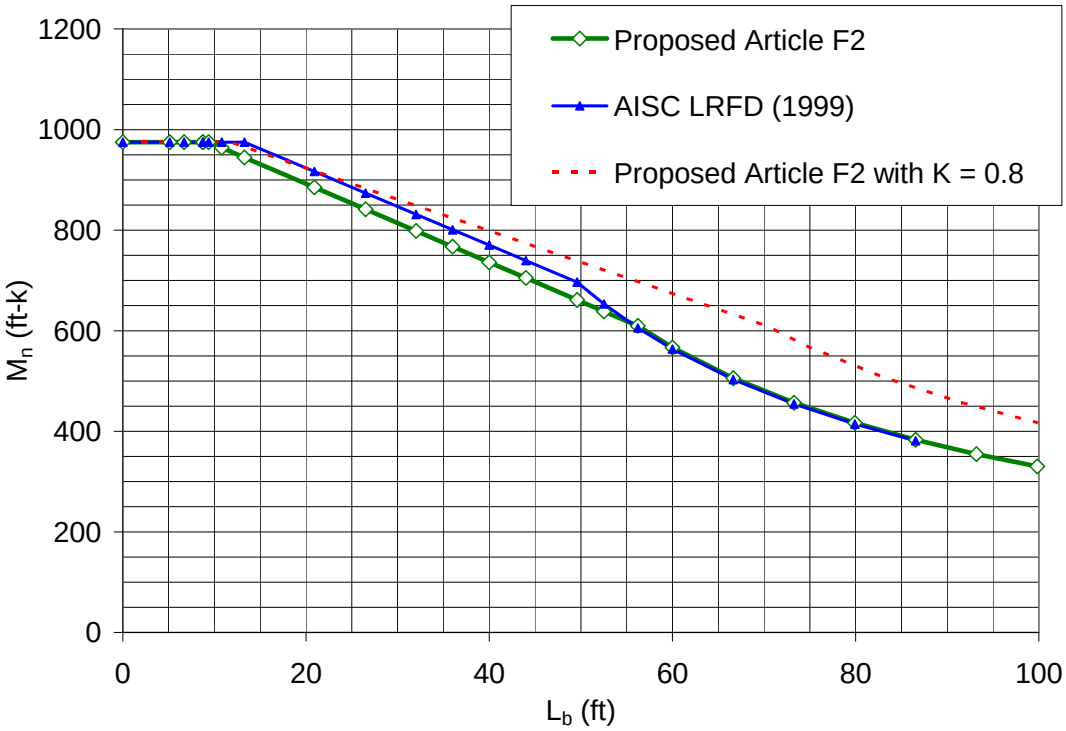


Fig. 4. Suggested and (AISC 1999) strengths for a W14x132 subjected to uniform bending (r_{ts}/r_y = 1.13).

Figures 3 and 4 also show the solution obtained for the strength, plotted in terms of the actual unbraced length L_b, if an effective length factor of K = 0.8 is assumed in the LTB calculations. In their assessment of inelastic beams under uniform bending moment, Lay and Galambos (1965) state that "K = 0.80 may be beyond anything likely to occur in normal practice." They base this assessment in part on the consideration of beams loaded typically at the one-third span locations, with the center unbraced segment subjected to uniform bending and the outside unbraced lengths subjected to a linear moment diagram. (SSRC 1998) recommends a simplified procedure, originally proposed by Nethercot and Trahair (1976), for calculation of elastic LTB effective lengths that gives K = 0.83 for this case. However, if the unbraced lengths adjacent to the segment under consideration are also subjected to uniform bending moment, the procedure suggested in (SSRC 1998) gives K = 1.0, and for all practical purposes, more rigorous buckling solutions will give values also essentially equal to 1.0. These cases do occur in some practical structures. Therefore, it is suggested that the commentary of the specification should provide specific guidance on when K can be taken conservatively as say 0.8 or calculated using the procedure suggested in (SSRC 1998). Implementation of this improvement would avoid the current minor compromise in the level of safety that occurs for situations where K is indeed close to 1.0 and L_b is near the L_p limit. Also, this improvement would result in a more rational consideration of the influence of end restraint in general. Furthermore, an added benefit is that it gives a larger LTB resistance than in (AISC 1999) for larger unbraced lengths when the influence of end restraint from adjacent framing is justified. White and Chang (2003b) present a generalization of the (SSRC 1998; Nethercot and Trahair 1976) LTB effective length procedure that addresses monosymmetric I-shapes and composite I-sections in negative bending.

It is recommended that the K factor should not be shown within the Specification equations. Rather, L_b should be defined as the effective laterally unbraced length, taken conservatively as the length between points braced against lateral displacement of the compression flange or between points braced to prevent twist of the cross-section.

Figures 5 and 6 illustrate the strengths predicted by the suggested Article F2 provisions versus the corresponding LTB strengths from the (AISC 1999), Canadian (CSA 2001), British (BSI 1990), Australian (SAA 1998) and Eurocode (CEN 1993) Standards. It can be observed that the (CSA 2001) inelastic LTB strength is somewhat larger than the other values. This more liberal prediction is based on a statistical analysis of only one set of experimental tests conducted by a single group of investigators (Baker and Kennedy 1984). Although these experimental results are considered to be reliable, other equally reliable experimental studies indicate smaller inelastic LTB strengths (White et al. 2003a). The L_p limit of the suggested provisions is close to the corresponding unbraced length limits in the British, Australian and European standards, and gives corresponding strengths close to the mean values from the inelastic LTB uniform bending tests considered in (White et al. 2003a) throughout the range of Eq. F2-2. Effective length factors $K \leq 1$ for LTB based on the procedure presented in (White and Chang 2003b) are used in the studies by White et al. (2003a). The results from (White et al. 2003a) for all types I-shapes in major-axis bending are summarized subsequently in this paper. The Engineer is referred to this reference for a more detailed assessment of the experimental data.

The British, Australian and European standards all suggest significantly smaller strengths within the elastic LTB range of the suggested and (AISC 1999) equations. This is a result of differences in the fundamental approach to the development of the nominal strength curves. The Australian strength curve is an approximate lower-bound to LTB test data assessed by Trahair

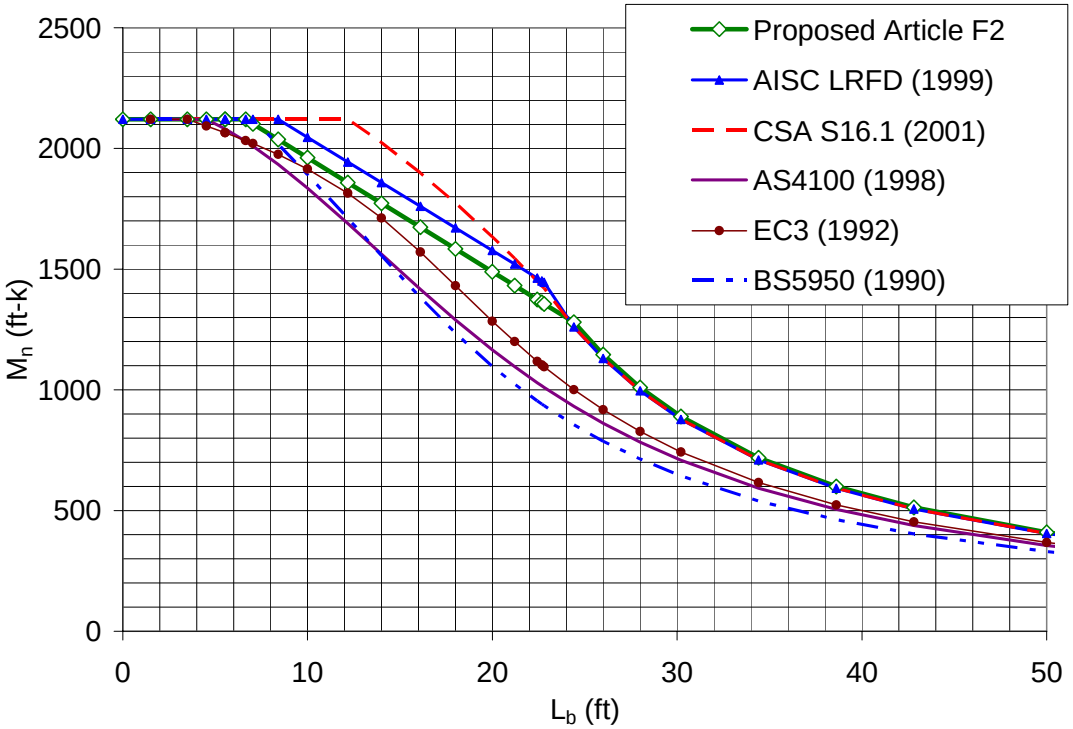


Fig. 5. Comparison of suggested and (AISC 1999) strengths to strength curves from other standards for a W36x135 subjected to uniform bending.

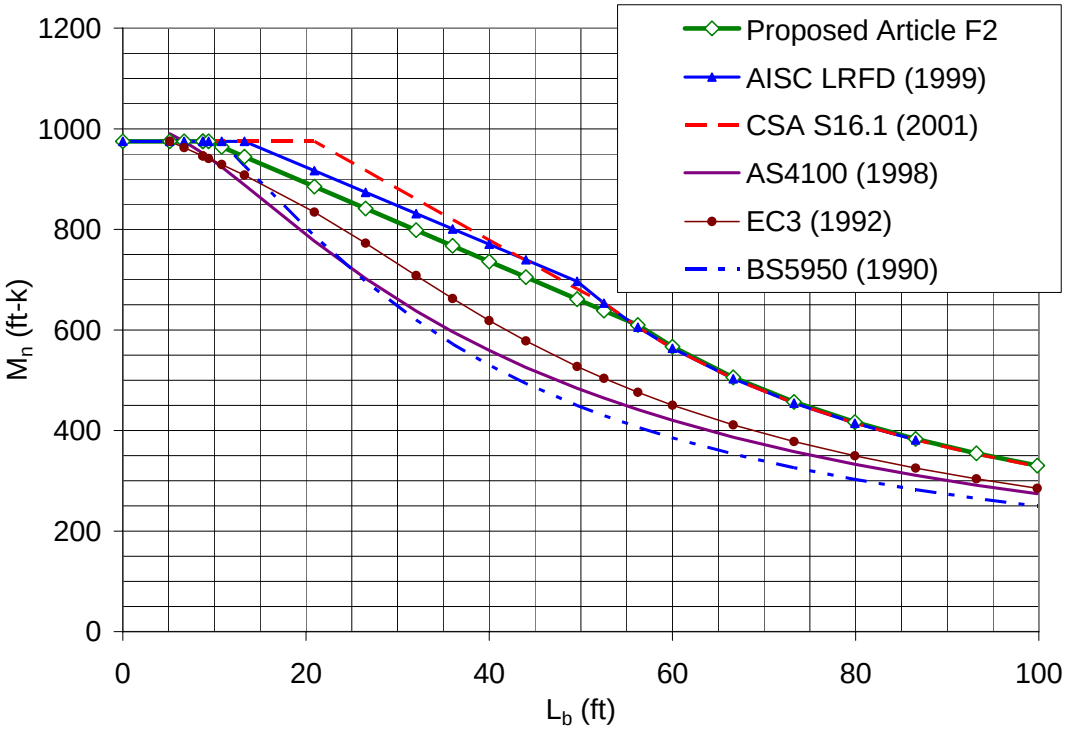


Fig. 6. Comparison of suggested and (AISC 1999) strengths to strength curves from other standards for a W14x132 subjected to uniform bending.

(1993). Conversely, the suggested and the (AISC 1999) elastic LTB equations give predictions close to the mean LTB strengths from uniform bending experimental tests considered originally by Galambos and Ravindra (1976) and more recently by White et al. (2003a). In the suggested and the (AISC 1999) approaches, the dispersion in the test to the predicted strengths is considered as a subsequent step, in conjunction with the statistics for the geometric and material properties at large, in the determination of the resistance factors ϕ.

ARTICLE F3 – DOUBLY-SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND NONCOMPACT FLANGES, BENT ABOUT THEIR MAJOR AXIS

Little needs to be said about the provisions in Article F3. Equation F3-1 is the same form as Equation F2-2. The inelastic FLB strength of a noncompact flange is represented by a linear interpolation between two anchor points, as illustrated in Fig. 1. The equations for the compact and noncompact flange limits λ_{pf} and λ_{rf} are effectively the same as specified in (AISC 1999) (see Appendix II Cases 1 and 2 for the suggested plate slenderness limits). The equations provided for Case 2 are general and can be applied for all types of I-sections. The λ_r equation for rolled I-shaped members and channels (Case 1) is obtained by substituting $k_c = 0.76$ and $F_{yr} = 0.7F_y$ into the corresponding Case 2 equation. The term k_c is the flange local buckling coefficient. A value of $k_c = 0.76$ is also used implicitly within the corresponding equation in (AISC 1999). The general noncompact flange limit, λ_r of Case 2, and the flange local buckling coefficient, k_c, are considered further in the discussion of Article F4.

The suggested inelastic FLB strength curve is changed somewhat relative to (AISC 1999) due to the use of $F_{yr} = 0.7F_{yc}$ in the suggested provisions versus $F_L = F_y - 10$ ksi ($F_y - 69$ MPa) for rolled I-shapes and $F_L = F_y - 16.5$ ksi ($F_y - 114$ MPa) for welded I-shapes in (AISC 1999). The corresponding inelastic FLB curves are compared for a section with $F_y = 50$ ksi (345 MPa) and $M_p/M_y = 1.14$ in Fig. 7. The largest $b_{fc}/2t_{fc}$ value of all the ASTM A6 rolled I-shapes is 11.5. Therefore the effect of the F_{yr} simplification on the strengths of rolled I-shapes is negligible. For a built-up welded I-shape with $F_y = 50$ ksi (345 MPa), the suggested FLB strength curve is slightly more liberal than the one specified in (AISC 1999).

Some mention of the philosophy adopted in writing the equations throughout the suggested provisions is in order at this point. The astute Engineer will recognize that the expression for F_{yr} (Eq. F2-6) as well as the expressions for λ_{pf} and λ_{rf} from Chapter B could be substituted into Eq. F3-1 to simplify this equation. Similar algebraic substitutions can be made at other locations within the suggested provisions. The Engineer might wish to make these substitutions in order to generate simplified forms of the equations for easier hand calculations. However, it is not recommended that the equations be written in this fashion within the Specification. Once these algebraic substitutions are made, the entire basis of the equations is hidden. In the opinion of the author, the Specification equations should be written to convey the concepts behind the associated provisions as clearly as possible. The λ_r equation of Case 1 in Appendix II deviates from this philosophy to streamline the procedures for noncompact rolled I-shapes, since the fundamental basis of the λ_p equations also is not conveyed within the suggested Table B4.1.

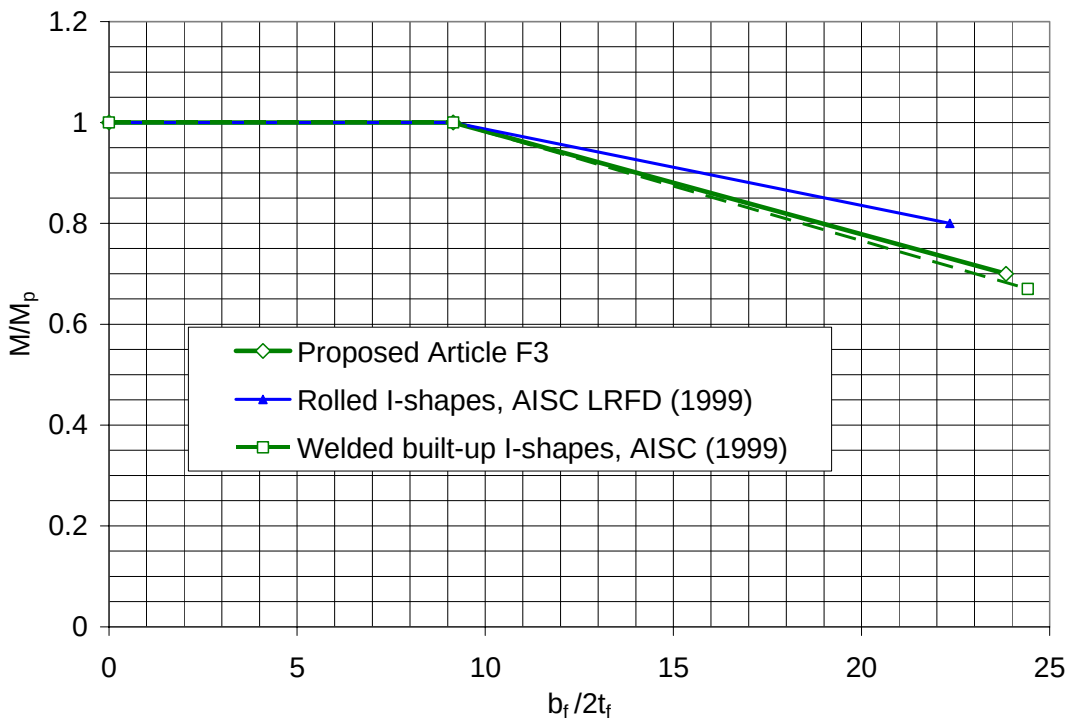


Fig. 7. Comparison of suggested and (AISC 1999) FLB strength corves for compact-web rolled and welded I-shapes with M_p/M_y = 1.14 and F_y = 50 ksi (345 MPa).

ARTICLE F4 – OTHER COMPACT AND NONCOMPACT WEB I-SHAPED MEMBERS BENT ABOUT THEIR MAJOR AXIS

Article F4 is where the improvements discussed in the introduction pertaining to elimination of separate WLB strength checks as well as the handling of monosymmetric I-shapes are implemented. For the cross-section types addressed by this article, both FLB and LTB must be checked in general. Also, as noted previously, for monosymmetric sections with a larger compression flange, a third limit state check associated with tension flange yielding (TFY) is implemented.

Article F4.1. Lateral-Torsional Buckling and Definition of Web Plastification Factors

The general LTB provisions for compact and noncompact-web I-shapes are implemented in Article F4.1. Equations F4-1 through F4-3 parallel Eqs. F2-1 through F2-3 of Article F2. However, the equations in Article F4 have a maximum potential capacity of $R_{pc}M_{yc}$ instead of the plastic moment resistance M_p. The term R_{pc} is referred to as the web plastification factor and is given by Eqs. F4-8. These equations are similar in form to the WLB equations of (AISC 1999). However, R_{pc} is applied to the LTB equations in Article F4.1, as well as to the FLB equations in Article F4.2, in a manner similar to (but not exactly the same as) the application of the R_{PG} parameter of Appendix G in (AISC 1999). Strictly speaking, R_{pc} is a modifier that is applied to the yield moment associated with the compression flange, M_{yc}, to obtain the maximum potential strength, shown as M_{max} in Fig. 1. The main aspect that is new relative to (AISC 1999) is that the unbraced length L_p required to reach $M_{max} = R_{pc}M_{yc}$ in Article F4-1, and the flange slenderness λ_p required to reach the same M_{max} in Article F4-2, are invariant with respect to the

web slenderness. Also, the web slenderness is expressed consistently throughout Article F4 as h_c/t_w. Since (AISC 1999) treats WLB as an independent strength check, the limits on L_b or $b_f/2t_f$ at which LTB or FLB would begin to govern the strength relative to the WLB equations increase as the WLB strength decreases. As noted in the introduction to this paper, the experimental tests conducted by Richter (1998) and the experimental data examined by White et al. (2003a) indicate that the suggested provisions are a truer representation of the physical strength than the equations in (AISC 1999). Also, by handling the effect of web slenderness in this way, significant discontinuities in the strength predicted by Appendices F1 and G of (AISC 1999) are avoided for members that have a web at the noncompact slenderness limit. The only discontinuity between the suggested noncompact-web and slender-web section provisions occurs due to the assumption of J = 0 in the LTB strength equations for a slender-web member. This aspect is addressed further in the discussion of Article F5.

Equation F4-8b for R_{pc} implements a simple linear interpolation between a flexural resistance equal to the yield moment capacity M_{yc} when the web slenderness is at its noncompact limit to the plastic moment capacity M_p in the limit that the web is compact. This is in essence what the WLB limit state equation in (AISC 1999) does as well. However, the governing factor on the member maximum strength for these types of members is not actually the buckling of the web. The maximum flexural resistance is reached generally due to buckling of the compression flange. This occurs after substantial yielding if the web and flange slenderness and lateral brace spacing are sufficiently compact. The flanges are the dominant components that contribute to the flexural resistance. Strictly speaking, the slenderness of the web influences the LTB and FLB capacities in a secondary fashion. The handling of the web slenderness effects through the parameter R_{pc} is a truer representation of the physical response.

Equation F4-3 is the same as Eq. F2-3 with the exception of the change in the maximum limit from M_p to $R_{pc}M_{yc}$, the implicit use of c = 1 (since this Article applies only to I-shapes), the explicit use of the section modulus to the compression flange S_{xc}, and the use of the radius of gyration term r_t. This equation is strictly derivable only for a doubly-symmetric I-shape. That is, when Eq. F4-9 is used to calculate r_t for a doubly-symmetric I-shape, Eq. F4-3 gives the exact solution from (Timoshenko and Gere 1961). However, White and Jung (2003b) show that this equation gives an accurate approximation of the exact beam theory based solution for elastic LTB of general monosymmetric I-shaped members, provided that J is multiplied by 0.8 for the unusual case of relatively shallow cross-sections ($D/b_{fc} < 2$ or $D/b_{ft} < 2$) or cross-sections with highly stocky tension flanges ($b_{ft}/t_{ft} < 10$) with $I_{yc}/I_{yt} > 1.5$. This requirement is stated after Eq. F4-10. Given this restriction, which is expected to seldom apply, the maximum unconservative error associated with Eq. F4-3 relative to the exact beam theory solution is nine percent for a comprehensive range of practical member geometries (White and Jung 2003b). A comparable I_{yc}-based equation specified in (AASHTO 1998) gives maximum unconservative errors up to 27 percent for the same set of parameters studied, or up to 21 percent if the cross-sections are restricted to $D/b_{fc} \geq 2$, $D/b_{ft} \geq 2$ and $b_{ft}/t_{ft} \geq 10$.

For highly monosymmetric sections with a smaller compression flange ($I_{yc}/I_{yt} < 0.5$), both Eq. F4-3 and the I_{yc}-based equation in (AASHTO 1998) tend to be conservative compared to rigorous open-walled section beam theory solutions. For $D/b_{fc} \geq 2$, $D/b_{ft} \geq 2$ and $b_{ft}/t_{ft} \geq 10$, the maximum conservatism associated with Eq. F4-3 is only 12 percent for $L_b = L_r$ and $I_{yc} = 0.1\ I_{yt}$ (White and Jung 2003b). However, for more general geometries, Eq. F4-3 can be as much as 13 percent conservative at $I_{yc} = 0.5\ I_{yt}$ and 35 percent conservative at $I_{yc} = 0.1\ I_{yt}$ relative to the exact

open-walled section beam theory solution. The maximum conservative errors associated with the (AASHTO 1998) equation are slightly larger. This is due to the fact that these equations do not account adequately for the restraint against lateral buckling of the compression flange provided by a large tension flange. Nevertheless, the distortional flexibility of the web can reduce the critical moment significantly for these types of members. White and Jung (2003b) show that web distortion can reduce the elastic buckling strength by more than 10 percent relative to Eq. 4-3 in a number of practical cases if the member is unstiffened and I_{yc}/I_{yt} is less than 0.5. The reduction is as much as 38 percent relative to the exact open-walled section beam-theory solution in extreme cases with $I_{yc}/I_{yt} = 0.1$. They also explain that one-sided transverse web stiffeners placed at a spacing ($a \leq 3h$) throughout the unbraced length are sufficient to develop 85 percent of the beam theory LTB strength in the most critical case studied, provided that: (1) $b_s \geq \max(b_{fc}, b_{ft})/4$, (2) $b_s \geq h/8$, (3) $b_s / t_s \leq 0.56\sqrt{E/F_y}$ and (4) $t_s \geq \max(t_{fc}, t_{ft}) / 4$. Two-sided stiffeners at ($a \leq 3h$) are sufficient provided that: (1) $b_s \geq \max(b_{fc}, b_{ft})/4$, (2) $b_s \geq h/14$, (3) $b_s / t_s \leq 0.56\sqrt{E/F_y}$ and (4) $t_s \geq \max(t_{fc}, t_{ft}) / 4$. Therefore, Article F4 specifies that transverse stiffeners, designed according to the shear strength provisions of Section G1.2, shall be provided at a spacing ($a \leq 3h$) throughout the unbraced length when $I_{yc}/I_{yt} < 0.5$. The above stiffener requirements are similar to the base transverse stiffener proportioning requirements in (AASHTO 2003). However, (AISC 1999) specifies only an area and a moment of inertia requirement for transverse stiffeners. It is suggested that the above rules (1) and (3) be added to the Specification provisions for proportioning of transverse stiffeners in Section G1.2. Rules (2) and (4) are implemented in Article F4. The Engineer should note that the conservative estimates of the exact beam-theory solution for members with a significantly smaller compression flange, tend to compensate for the maximum reduction in strength due to web distortion for the worst-case solutions with a stiffened web (White and Jung 2003b).

White and Jung (2003b) explain that I-shaped members with channel caps may be designed without the consideration of distortional buckling when the St. Venant torsion constant J is calculated by summing each of the individual component contributions from the I-section and from the channel. This calculation is conservative relative to the physical J for a built-up section.

Equation F4-U1 gives an approximate form for the radius of gyration r_t comparable to Eq. F2-U2. However, Eq. F4-U1 accounts for a depth of the web in compression other than h/2. This equation is obtained simply by setting $d_f = d = h$ in Eq. F4-9. The Engineer should note the importance of the web term $h_c t_w$ within the symbol a_w of this equation (see Eq. F4-10 for the definition of a_w applicable for an I-shaped member with a rectangular compression flange). Prior AASHTO Specifications for steel design have often used the radius of gyration of only the compression flange, $r_{yc} = b_{fc} / \sqrt{12}$, within design equations for elastic LTB. This approximation can lead to significantly unconservative predictions (White et al. 2001). The web term $h_c t_w$ accounts for the destabilizing effects of flexural compression within the web. Unfortunately, (AISC 1999) Appendix F1 makes a similar mistake by writing the L_p limit for monosymmetric I-shapes in terms of r_{yc}. As noted in the Introduction, this leads to a situation in which the LTB resistance for a doubly-symmetric member with L_b somewhat larger than the (AISC 1999) L_p limit can be increased significantly by making the cross-section slightly monosymmetric. Obviously, this discontinuity in the predicted strength between a doubly-symmetric and a similar monosymmetric member is not physical.

The term F_{yr} has a slightly different definition in Article F4 than in F2. The modified definition, given by Eqs. F4-5, accounts for the potential early nominal yielding at the tension flange in a highly monosymmetric section with a larger compression flange and a corresponding small depth of web in compression. The term F_yS_{xt} in this equation gives the nominal yield moment corresponding to the tension flange, M_{yt}. By dividing this term by S_{xc}, one obtains the nominal elastic compression flange stress when the moment reaches M_{yt}. Nominal yielding at the tension flange invalidates the elastic LTB strength equations. Therefore, F_{yr} should in general be taken as the smaller value of F_yS_{xt}/S_{xc} or $0.7F_y$ for a monosymmetric I-shape. Tension flange residual stress effects are neglected as having a minor effect on the LTB response. The Engineer should note that a value of $h_c/h = 0.35$ corresponds to $h_c/(h - h_c) = 0.54 \cong S_{xt}/S_{xc}$. Therefore, it can be concluded that the limit F_yS_{xt}/S_{xc} may govern the calculation of F_{yr} for practical monosymmetric I-shapes. Equation (F4-5b) also restricts F_{yr} to a minimum value of $0.5F_y$. This minimum value is sufficient to account for the effects of early tension flange yielding in all cases, and is necessary to avoid anomalous predictions in which the inelastic buckling curve between anchor points 1 and 2 in Fig. 1 produces larger strengths than the elastic buckling capacity associated with a given length (due to anchor point 2 being located at a small M_{yr} and at a corresponding large value of L_r). Appendix F1 of (AISC 1999) implements a limit of $F_L \leq F_{yt}\, S_{xt}$ that is similar to Eq. (F4-5), but potentially allows this anomaly to occur.

The L_p limit of Eq. F4-6 is the same form as that of Eq. F2-4. The use of the same form for a general monosymmetric shape is based on the experimental test data compiled by (White et al. 2003a). Also, this form gives the same result as Eq. F2-4 in the limit that a monosymmetric I-shape is made doubly-symmetric, thus avoiding discontinuities between the strengths calculated by Articles F2 and F4 for members that fall on the boundary between these articles.

As mentioned in the Introduction, the noncompact-web slenderness limit λ_{rw} within the suggested provisions is the same as the one used with h_c/t_w in the R_{PG} equation of (AISC 1999) Appendix G. For doubly-symmetric I-shapes the compact-web limit λ_{pw} is also the same as the corresponding limit in (AISC 1999). However, for monosymmetric I-sections, the shape factor M_p/M_y tends to be larger than for doubly-symmetric shapes and the demands on the web for the section to develop its plastic moment resistance M_p are generally larger (White and Barth 1998). The suggested λ_{pw} equation for monosymmetric I-sections, listed in Appendix II, reduces to the (AISC 1999) λ_{pw} equation when $M_p/M_y = 1.14$, which is a representative value for doubly-symmetric I-shapes. Accordingly, the maximum web compactness limit is restricted for such a section to the (AISC 1999) value for a doubly-symmetric I-shape. However, when M_p/M_y is significantly larger than 1.14, the suggested equation places significant additional restrictions on the web slenderness for the section to be defined as compact. The suggested λ_{pw} equation for monosymmetric I-sections is obtained as a simplification of the requirements to develop M_p implied by the equations in (White and Barth 1998). The web compactness requirement specified by this equation is also approximately the same as the requirement for development of M_p independently implemented by the Q formula in (AASHTO 1998). Both of these requirements are plotted as a function of M_p/M_y for $F_y = 50$ ksi (345 MPa) in Fig. 8.

Figure 9 shows the results for an example monosymmetric I-section in which there is a substantial change in the predicted capacity from (AISC 1999) using the suggested provisions. It is obvious that the (AISC 1999) inelastic buckling curve in this plot is too optimistic, since the inelastic buckling strength is reduced little relative to the elastic buckling solution. The differences in the strengths are caused by a combination of the various factors discussed above.

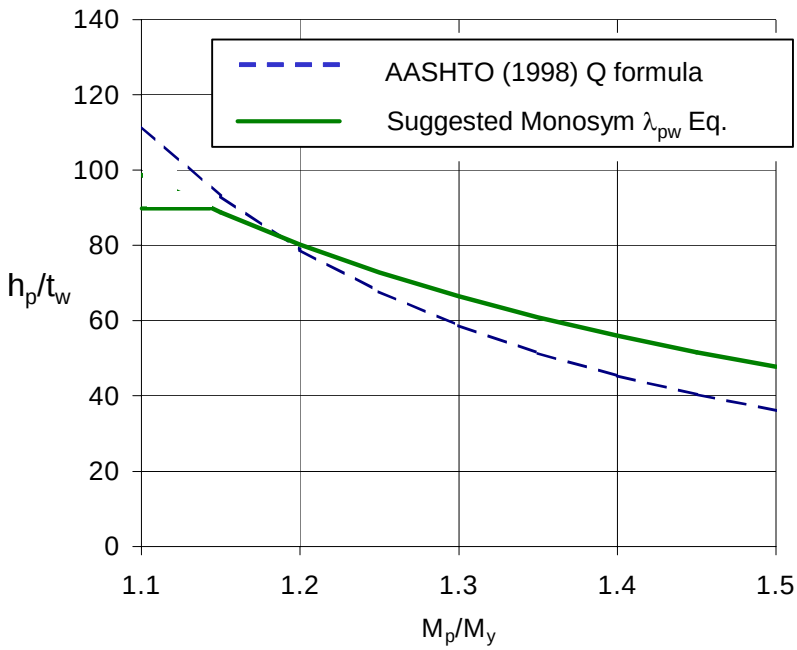


Fig. 8. Web compactness limits as a function of M_p/M_y from the AASHTO (1998) Q formula and from the suggested λ_{pw} equation for monosymmetric I-sections with F_y = 50 ksi (345 MPa).

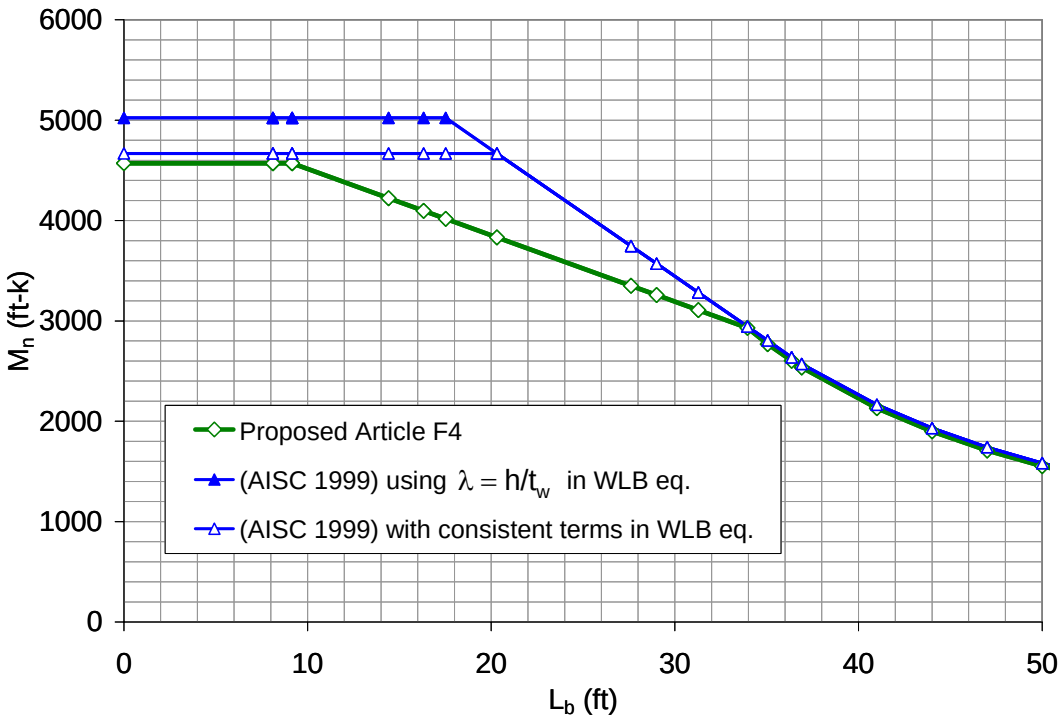


Fig. 9. Suggested and (AISC 1999) LTB strengths for a monosymmetric compact-flange noncompact-web girder subjected to uniform bending ($h \times t_w$ = 48 x 0.5 in, $b_{fc} \times t_{fc}$ =16 x 1 in, $b_{ft} \times t_{ft}$ = 16 x 1.5 in, F_y = 50 ksi (345 MPa), h_c/h = 1.13, h_p/h = 1.33, h_c/t_w = 109, h_p/t_w = 128, r_t = 4.15 in (105 mm), r_{yc} = 4.62 in (117 mm), M_p= 5175 ft-k (7017 kN-m), M_p/M_y =1.24).

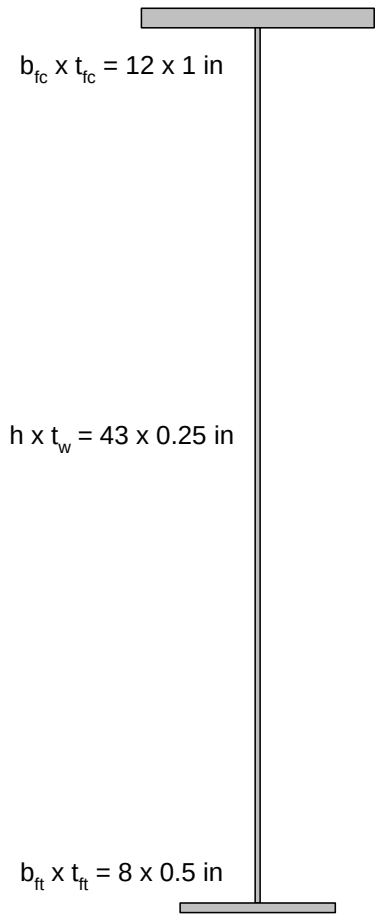
First, the radius of gyration r_t from Eq. F4-9 is 4.13 in (105 mm) for this section, versus r_{yc} = 4.62 in (117 mm). Secondly, the coefficient within the equation for L_p is 1.76 rather than 1.1 in (AISC 1999). Thirdly, since the WLB limit serves effectively as a cap on the (AISC 1999) LTB strength, the unbraced length at which the strength reaches the maximum flexural resistance based on the WLB equations is larger than the AISC L_p limit. Finally, because of the more stringent web compactness limit in the suggested provisions, R_{pc} is 1.092 whereas M_n/M_{yc} using the WLB equation of (AISC 1999) Appendix F1 with λ taken as h/t_w is 1.200. This interpretation of the WLB equation can be inferred from (Galambos and Ravindra 1976). Nevertheless, the definition $\lambda = h/t_w$ is not consistent with the definition of λ_p in (AISC 1999) for the web of a monosymmetric section. The (AISC 1999) WLB equation can be made consistent by taking λ as h_p/t_w and by multiplying λ_r by h_p/h prior to substituting these parameters into the WLB expression. When this is done, the (AISC 1999) curve shown in the figure with the smaller value for the maximum flexural capacity is obtained. The value from the (AISC 1999) WLB equation based on this solution is M_n/M_{yc} = 1.116. The inconsistency in the (AISC 1999) Appendix F1 WLB equation in the above example is addressed in the suggested provisions by defining both λ_{rw} and λ_{pw} in terms of $\lambda = h_c/t_w$.

Figure 10 illustrates one of the other shortcomings of the (AISC 1999) WLB provisions for monosymmetric I-shapes outlined in the introduction to this paper. This section is highly monosymmetric and has a larger compression flange. It satisfies the (AISC 1999) compact-web limit based on h_p/t_w, but also it is classified as a slender-web section since h/t_w is greater than the corresponding λ_r limit in the current Specification. This anomaly is addressed within the suggested provisions by defining this type of section to be compact (see the footnote to the table in Appendix II). The rationale for this interpretation is that prior to reaching the maximum potential strength, the yielding within the cross-section reduces the depth of the web in compression. Therefore, the web becomes inherently more stable as the maximum resistance of the section is approached. Furthermore, I-sections that produce the above anomaly tend to have values of h_c/t_w that are not significantly larger than λ_{rw}. Therefore, the effect of web bend buckling is minor.

The results for two experimental tests that have proportions satisfying the above special case are considered in (White et al. 2003b and c). The suggested flexural strength equations give predictions for these girders that are 5 percent conservative and 6 percent unconservative. These girders are subjected to large shear forces in addition to large bending moments, but the suggested flexural strength provisions govern the resistance relative to the (AISC 1999) shear strength equations. No moment-shear interaction is considered in calculating the strengths for these tests, although the shear forces are high enough to require this per (AISC 1999). These predictions are well within the general scatter band of the experimental tests considered within the above studies. Furthermore, the equations for R_{pc} are significantly simpler than the comparable WLB equations in (AISC 1999) for monosymmetric I-shapes. The WLB provisions in (AISC 1999) refer the Engineer from Appendix F1 to Chapter B and then to Appendix B for calculation of the compact and noncompact-web limits, and involve the complexities and inconsistencies discussed above.

The Engineer should note that the provisions of Article F4.1 reduce to the LTB provisions of Article F2 in the limit that the web slenderness approaches the compact limit λ_{pw}. In the limit that the web slenderness approaches the noncompact limit λ_{rw}, the provisions of Article F4.1 also

reduce to the LTB provisions of Article F5.1, except that in Article F5.1, the St. Venant torsional stiffness contribution is taken equal to zero, i.e., effectively J is taken equal to zero.



$F_{yf} = F_{yw} = 70$ ksi

$$\frac{M_p}{M_y} = 1.41, \quad \frac{h}{t_w} = 172, \quad \frac{h_c}{t_w} = 119, \quad \frac{h_p}{t_w} = 44, \quad \frac{h_c}{h_p} = 2.706, \quad \frac{h}{h_c} = 1.444$$

$$\lambda_{pw} = 3.76\sqrt{\frac{E}{F_y}} = 76.5 \quad \text{(AISC 1999)}$$

$$\lambda_{rw} = 1.49\sqrt{\frac{E}{F_y}}\left[1 + 2.83\frac{h}{h_c}\right] = 7.58\sqrt{\frac{E}{F_y}} = 154 \quad \text{(AISC 1999)}$$

$$\frac{h_p}{t_w} < \lambda_{pw}, \quad \therefore \text{ web is compact by (AISC 1999)}$$

$$\text{but also } \frac{h}{t_w} > \lambda_{rw}, \quad \therefore \text{ web is also slender by (AISC 1999)}$$

$$\lambda_{pw} = \frac{\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y} - 0.1\right)^2} = 2.30\sqrt{\frac{E}{F_y}} = 46.8 \quad \text{(suggested, in terms of } \frac{h_p}{t_w}\text{)}$$

$$\lambda_{pw} = \frac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y} - 0.1\right)^2} = 6.22\sqrt{\frac{E}{F_y}} = 127 \quad \text{(suggested, in terms of } \frac{h_c}{t_w}\text{)}$$

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_y}} = 116 \quad \text{(suggested)}$$

$$\frac{h_p}{t_w} < \lambda_{pw} \quad \therefore \text{web is compact, although } \frac{h_c}{t_w} > \lambda_{rw} \quad \text{(suggested)}$$

Fig. 10. Example monosymmetric I-shape illustrating the solution to ambiguities and anomalies in the handling of web slenderness effects in (AISC 1999).

Unbraced Length Limits for Development of the Maximum Flexural Resistance $R_{pc}M_{yc}$ in Segments Subjected to Moment Gradient

Equation F4-6 gives the maximum unbraced length for which the flexural resistance $R_{pc}M_{yc}$ can be developed. As in the comparable equation of (AISC 1999), this limit applies only to the uniform moment case. The maximum flexural resistance can be reached at larger unbraced

lengths when there is a moment gradient. For cases involving a moment gradient (i.e., $C_b > 1$), Eqs. F4-2 and F4-3 can be equated to $R_{pc}M_{yc}$ and solved for the corresponding unbraced length limit. The result is that the member strength can be expressed as $R_{pc}M_{yc}$ whenever

$$L_b \le L_p + \frac{\left(1-\frac{1}{C_b}\right)}{\left(1-\frac{F_{yr}S_{xc}}{R_{pc}M_{yc}}\right)}\left(L_r - L_p\right) \qquad \text{for } L_p < L_b \le L_r \tag{1}$$

and

$$L_b \le 1.95 r_t \frac{C_b S_{xc} E}{R_{pc} M_{yc}} \sqrt{\frac{J}{S_{xc} h}} \sqrt{1+\sqrt{1+6.76\left(\frac{R_{pc}M_{yc}}{C_b S_{xc} E}\frac{S_{xc}h}{J}\right)^2}} \qquad \text{for } L_b > L_r \tag{2}$$

These equations also can be applied with Article F2 by replacing $R_{pc}M_{yc}$ by M_p and by replacing r_t by r_{ts}. Furthermore, if the maximum resistance is limited by FLB, the corresponding FLB strength can be substituted for $R_{pc}M_{yc}$ in these equations to determine the maximum unbraced length at which the LTB resistance is greater than or equal to the FLB resistance.

Alternate Rigorous Beam-Theory Based Elastic LTB Equations for Monosymmetric I-Shaped Members

White and Jung (2003b) evaluate a number of forms for calculation of the elastic LTB resistance of monosymmetric I-shaped members as well as the corresponding abscissa for anchor point 2, L_r. The forms given by Eqs. F4-3 and F4-7 are recommended since these equations give accurate estimates of the corresponding exact values for a comprehensive range of practical monosymmetric shapes, and thus they are able to serve as a single base representation of the elastic LTB resistance for all I-shaped members and channels. However, White and Jung (2003b) also suggest a useful form of the rigorous open-walled section beam theory equations that can be substituted for Eqs. F4-3 and F4-7 if desired by the Engineer. These equations are:

$$M_n = C_b \frac{\pi^2 E I_y}{L_b^2}\left\{\frac{\beta_x}{2} + \sqrt{\left(\frac{\beta_x}{2}\right)^2 + \frac{C_w}{I_y}\left[1+0.0390\frac{J}{C_w}L_b^2\right]}\right\} \tag{3}$$

and

$$L_r = \frac{1.38}{S_{xc}}\frac{E}{F_{yr}}\sqrt{I_y J}\sqrt{\left[2.6\beta_x\frac{F_{yr}}{E}\frac{S_{xc}}{J}+1\right]+\sqrt{\left[2.6\beta_x\frac{F_{yr}}{E}\frac{S_{xc}}{J}+1\right]^2+27.0\frac{C_w}{I_y}\left(\frac{F_{yr}}{E}\frac{S_{xc}}{J}\right)^2}} \tag{4}$$

where β_x is defined as the coefficient of monosymmetry, which is represented with good accuracy by the expression

$$\beta_x = 0.9h\alpha\left(\frac{I_{yc}}{I_{yt}} - 1\right) \tag{5}$$

and where

$$C_w = h^2 I_{yc}\alpha \tag{6}$$

with

$$\alpha = \frac{1}{\frac{I_{yc}}{I_{yt}} + 1} \tag{7}$$

Equation (1) is still applicable if the above equations are employed. The equivalent of Eq. (2) based on the above equations is obtained by replacing F_{yr} in Eq. (4) by $R_{pc}M_{yc}/C_b$. Furthermore, the Engineer should note that distortional buckling is also a potential issue if these equations are employed when I_{yc}/I_{yt} is less than 0.5, unless the member is transversely stiffened as specified in Article F4. White and Jung (2003b) discuss the strength reductions due to web distortion for a wide range of practical doubly- and singly-symmetric I-sections.

Although not addressed explicitly within the suggested provisions, the recommended equations F4-1 through F4-3 can be applied to obtain a reasonable estimate of the capacity of typical composite I-section members in negative bending. The above rigorous equations are difficult to use for this purpose, since they depend on the ratio of I_{yc}/I_{yt}. Also, for composite I-shapes in negative bending, the rigorous beam theory solution is not valid since the buckling resistance is always influenced by the flexibility of the web and the corresponding resistance to web distortion. The suggested elastic LTB equation (Eq. F4-3) gives reasonable results that are accurate to somewhat conservative with respect to rigorous distortional buckling solutions for these types of members (White and Jung 2003b). The application of the suggested LTB Eqs. F4-1 through F4-3 to composite members in negative bending is addressed further in the discussion of Article F5.

Article F4.2. Compression Flange Local Buckling

Article F4.2 gives the general equations for the FLB strength of compact- and noncompact-web I-shaped members bent about their major axis. Equation F4-12 parallels Eq. F3-1, but is applicable to the general case of a member with a noncompact web, whereas Eq. F4-13 gives the corresponding theoretical elastic FLB strength. Equation F4-13 is an exact theoretical expression for the elastic FLB strength, given a theoretically exact value of k_c. The FLB coefficient k_c is defined in Table B4.1 (Appendix II) by the equation

$$k_c = 4\sqrt{h/t_w}\,,\ \ 0.35 \le k_c \le 0.76 \tag{8}$$

which gives a transition from a capped maximum value of $k_c = 0.76$ (corresponding to the assumed k_c value for rolled I-shapes) to a minimum limit of $k_c = 0.35$. The FLB coefficient for simply-supported edge conditions at the web-flange juncture is $k_c = 0.43$. Therefore, smaller values of k_c indicate that the web is tending to destabilize the flange. A value of h/t_w less than

28 is required to obtain $k_c = 0.76$ in the above equation whereas $k_c = 0.35$ for $h/t_w \geq 131$. This formula was developed originally by (Yura 1992) by equating the result from the LRFD strength equations to measured experimental capacities for a number of girders in which the flexural resistance was governed by FLB and back-solving for k_c. The majority of the data used in these developments was from (Johnson 1985). Figure 11 shows the suggested k_c relationship as well as the back-calculated k_c values from a number of FLB tests using the suggested provisions.

The Engineer should note that the majority of the data points in Fig. 11 actually correspond to slender-web members, i.e., to Article F5. The provisions of Article F4.2 reduce to the FLB provisions of Article F5.2 in the limit that the web slenderness approaches the noncompact limit λ_{rw}. Conversely, the provisions of Article F4.2 reduce to the FLB provisions of Article F3.2 in the limit that the web slenderness approaches the compact limit λ_{pw} and the cross-section is doubly-symmetric.

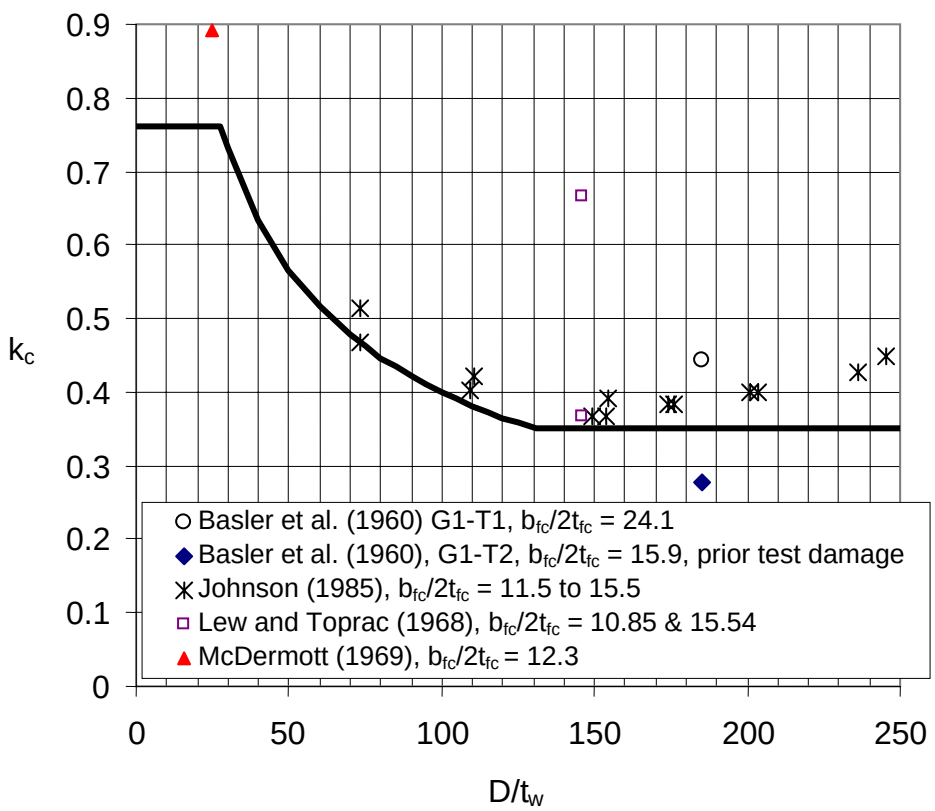


Fig. 11. Comparison of relationship for the flange local buckling coefficient k_c for welded built-up I-shapes to the k_c values back-calculated from experimental test strengths using the suggested flexural resistance equations, tests with $b_{fc}/2t_{fc} \geq 0.53\sqrt{E/F_{yc}}$.

The noncompact flange limits for I-sections in pure axial compression are also listed in Appendix II. These limits are the same as in (AISC 1999), but with modified descriptions to better indicate the intent of the current Specification. These limits are included here to highlight the attributes of the suggested and the (AISC 1999) rules for these limits. For I-sections in major-axis flexure, the general equation for λ_r specified in Appendix II is obtained by equating

the elastic FLB strength from Eq. (F4-13) to $M_{yr} = F_{yr} S_{xc}$ and solving for the corresponding value of λ. That is, a flange proportioned at $\lambda = \lambda_r$ is able to develop the moment M_{yr} prior to elastic FLB. The corresponding general equation for λ_r for a member subjected to pure axial compression is listed as Case 4 in Appendix II. This equation is obtained by setting the general equation for the flange elastic local buckling stress,

$$F_{cr} = \frac{0.9Ek_c}{\lambda^2} \tag{9}$$

to a value of $2.2F_y$ and solving for the corresponding value of λ. A value of $F_{cr} = 2.2F_y$ is sufficient for the flange to be able to develop an inelastic buckling strength in pure axial compression, including residual stress effects, approximately equal to the yield strength F_y. The same equation (Eq. (8)) is employed for k_c both for Case 2 (major-axis flexure) and for Case 4 (pure axial compression). Equation (8) is a simple approximate equation that provides reasonable general design values for the flange local buckling coefficient as a function of the web slenderness h/t_w. The Engineer should note that for most practical hot-rolled construction, flange local buckling either in major-axis flexure or in pure axial compression is always inelastic. In as such, the FLB strength tends to be insensitive to the calculated value for k_c. Also, because FLB is typically inelastic, rigorous theoretical derivations for k_c based on elastic local buckling strictly are not valid.

Article F4.3. Tension Flange Yielding

Article F4.3 addresses the tension flange yielding limit state. Equation F4-14 is simply a linear interpolation formula between the yield moment resistance M_{yt} for a section having a web at the noncompact limit λ_{rw} to the plastic moment capacity M_p when the web is compact. The term R_{pt} is the web plastification factor corresponding to this limit state, similar in concept to R_{pc}. As noted at the beginning of this article, this equation does not govern and need not be checked when $S_{xt} \geq S_{xc}$. However, the TFY limit state may govern the strength of members with a larger compression flange and $S_{xc} > S_{xt}$. The provisions of Article F4.3 reduce to the tension flange yielding provisions of Article F5.3 in the limit that the web slenderness approaches the noncompact limit λ_{rw}. In the limit that the web slenderness approaches the compact limit λ_{pw}, the tension flange yielding strength is equal to M_p.

ARTICLE F5 – SLENDER WEB I-SHAPED MEMBERS BENT ABOUT THEIR MAJOR AXIS

The provisions of the suggested Article F5 are similar in form to those of Article F4. However, they are somewhat simpler due to the assumption of J = 0 in the calculation of the LTB resistance, and due to the limit on the maximum strength of R_bM_{yc} or M_{yt}. The term R_b is the familiar strength reduction factor that accounts for post-buckling response of a slender web, that is, the shedding of web flexural stresses to the compression flange, and M_{yc} and M_{yt} are the yield moments corresponding to the compression and tension flanges respectively. The symbol R_b is suggested rather than the term R_{PG} from (AISC 1999). The term R_{PG} is no longer appropriate if it is decided not to use the term "plate girder" to refer to a slender-web member. The alternate term R_b is pneumonic since it accounts for the web "bend-buckling" behavior, and

furthermore, the symbol R_b has been used for this factor within the AASHTO Specifications for some time.

The term R_b is applied to the LTB and FLB strength expressions in Article F5 in a slightly different way than the application of the R_{pc} parameter in Article F4. Since R_b accounts for the elastic bend-buckling of a slender web and the subsequent web postbuckling strength, this term must be applied at the front of all the LTB and FLB strength equations. Conversely, R_{pc} is a modification factor that applies only to the calculation of the maximum strength plateau, M_{max}, shown in Fig. 1.

Equation F5-5 is equivalent to the corresponding R_{PG} equation in (AISC 1999) with the exception of one aspect that is missed in the current LRFD Specification. The term $M_{n(Rb = 1)}/S_{xc}$ in Eq. F5-5 is the same as the term F_{cr} in the (AISC 1999) R_{PG} equation when FLB or LTB govern the flexural resistance. By using F_{cr} or $M_{n(Rb = 1)}/S_{xc}$ in this way, the fact that the girder maximum strength might be reached before elastic web bend buckling occurs is accounted for. In these cases, there is no strength reduction due to the web post-buckling response, and thus $R_b = 1$. However, in members with a larger compression flange, the resistance in the suggested provisions and in (AISC 1999) may be governed by tension flange yielding. In this situation, the (AISC 1999) provisions do not account for the appropriate web bend buckling strength reduction at the governing strength limit. By use of $M_{n(Rb = 1)}/S_{xc}$, where $M_{n(Rb = 1)}$ is simply the governing flexural resistance with $R_b = 1$, this attribute of the potential behavior is addressed for girders with a larger compression flange.

As indicated by the first user note in Article F5, Eq. F5-3 is directly related to the general elastic LTB equation F4-3. The only difference between these equations is the assumption of $J = 0$ in Eq. F5-3, and the use of R_{pc} in Eq. F4-3 versus R_b in Eq. F5-3. As noted previously, the assumption of $J = 0$ in Eq. F5-3 results in a discontinuity in the LTB resistance for a cross-section that has a web proportioned at the noncompact-web limit λ_{rw} such that both Articles F4 and F5 could theoretically be applied. However, the effect of the St. Venant torsional stiffness on the elastic LTB resistance is often relatively minor for sections with these web proportions. This is because the values of h/b_f selected for these types of members often tend to be greater than or equal to 3.0. Therefore, the influence of the St. Venant torsional stiffness is similar to that shown previously for doubly-symmetric wide-flange sections with large $\frac{S_{xc}d_f}{J}$ in Fig. 2. White and Jung (2003a) discuss this attribute of the LTB strength equations in further detail. The St. Venant torsional stiffness can be more significant for monosymmetric shapes when the larger flange is very stocky, or when there are other attachments to one of the flanges such as cover plates or channel caps. However, web distortion effects can reduce the strength below that associated with the theoretical beam theory based LTB resistance when the web slenderness is close to or greater than its noncompact limit.

As the web depth-thickness ratio is increased beyond λ_{rw} and approaches the maximum values permitted by the (AISC 1999) Specifications (which are given by similar equations in Article F13 of the suggested provisions), the influence of the term $\frac{J}{S_{xc}d_f}$ tends to be even smaller and/or (particularly in the case of monosymmetric I-girders with only one flange providing a significant contribution to J and with highly slender webs) the influence of the St. Venant torsional stiffness is likely to be reduced significantly due to distortional buckling of the

web. Therefore, it is prudent both for the sake of simplicity as well as accuracy of the design provisions to assume J = 0 for highly slender webs. The noncompact-web limit λ_{rw} is a reasonable and convenient transition point to the use of J = 0. White and Jung (2003b) show evidence that the LTB solution with J = 0 is generally a lower-bound to the member distortional buckling strength.

There are two significant changes in the suggested Article F5 LTB and FLB equations relative to the Appendix G provisions of (AISC 1999). The (AISC 1999) Appendix G provisions implicitly assume an F_{yr} of $0.5F_y$ whereas the suggested provisions specify $F_{yr} = 0.7F_y$ with the exception of girders with $S_{xt}/S_{xc} < 0.7$. The usage of the larger F_{yr} limit in the suggested provisions is based on the experimental data compiled by White et al. (2003a, b, and c). Use of the suggested F_{yr} value removes another area where there can be a significant discontinuity between the flexural resistance predicted by Appendices F1 and G of (AISC 1999). Also, since the L_p equation for slender-web members in (AISC 1999) is based on the same definition of r_t as specified by Eq. F4-U1, but the coefficient in this equation is 1.76 rather than 1.1, the suggested Article F5 provisions reduce the L_p limit to 1.1/1.76 = 0.625 of its value in (AISC 1999). This change is substantiated by the experimental data for slender-web I-section members (White et al. 2003a). If the suggested L_p equation is substituted into the base original CRC based expression suggested by Basler and Thurliman (1961) and summarized by Cooper et al. (1978) for the LTB resistance, a strength of $0.97M_y$ is obtained for members with $R_b = 1$. If L_p/r_t is substituted as the equivalent slenderness ratio in the (AISC 1999) column strength formulas, a capacity of $0.95M_y$ is obtained. As noted previously, if the L_p equation from (AISC 1999) is substituted into these formulas, the corresponding strengths are significantly smaller at this limit. The Engineer should note that Cooper et al. (1978) originally suggested a value of $L_p = 146r_t\sqrt{F_y} = 0.86r_t\sqrt{E/F_y}$ for slender-web members. Also, it is emphasized that there is no discontinuity between the values of L_p for a member that is proportioned on the boundary between the suggested Articles F5 and F4. For a monosymmetric I-section, the discontinuity in the (AISC 1999) L_p values from $1.76r_t\sqrt{E/F_y}$ in Appendix G to $1.76r_{yc}\sqrt{E/F_y}$ in Appendix F1 can be significant.

The suggested L_p equation is more representative of the physical slender-web girder behavior, both from the perspective of the statistical evaluation of the supporting data as well as from a logical application of column strength formulas for these member types. Fortunately, the increase in the effective F_{yr} value from $0.5F_y$ to $0.7F_y$ compensates to some extent for this more restrictive L_p equation. Also, if a simple K value less than one can be justified based on the end conditions for the unsupported length, as noted previously in the discussion of Article F2, the strength predictions relative to (AISC 1999) will tend to be comparable or larger, particularly for larger unbraced lengths (see Fig. 3).

Expressions similar to Eqs. (1) and (2) can be specified to define the maximum unbraced length for which the maximum potential resistance R_bM_{yc} is achieved by slender-web I-shaped members subjected to moment gradient ($C_b > 1$). These equations are:

$$L_b \le L_p + \frac{\left(1 - \frac{1}{C_b}\right)}{\left(1 - \frac{F_{yr}}{F_y}\right)}\left(L_r - L_p\right) \qquad \text{for } L_p < L_b \le L_r \tag{9}$$

and

$$L_b \le \pi r_t \sqrt{\frac{C_b E}{F_y}} \qquad \text{for } L_b > L_r \tag{10}$$

Similar to the application of Eqs. (1) and (2), if the maximum strength is limited by FLB, the maximum unbraced length for which the LTB resistance is greater than or equal to the FLB strength is obtained by substituting M_n/R_bS_{xc} into Eqs. (9) and (10).

The tension flange yielding limit state of Article F5 is a particularly simple check. As noted by Basler and Thurlimann (1961), web bend-buckling does not have a significant influence on the tension flange stresses. Therefore, R_b is not applied in Eq. F5-9. This is consistent with the comparable equation in (AISC 1999).

An Engineer interested in designing a hybrid built-up member will notice that the suggested provisions no longer address these types of designs. The (AISC 1999) provisions have a number of inconsistencies or typographical errors with respect to the handling of hybrid built-up members. The suggested specifications do not address these member types in order to simplify the provisions. The rationale for this simplification is that, for the majority of constructed projects for which the AISC Specifications are employed, the use of hybrid built-up members does not provide significant benefits. (AASHTO 2003) gives provisions that address hybrid I-section members in a comprehensive fashion. These updated AASHTO provisions are similar to the suggested Chapter F provisions presented here.

Finally, one other important area is not addressed explicitly within the suggested provisions: lateral or local buckling of the compression flange within composite I-shaped members in negative bending. It should be noted that the provisions of Articles F4 and F5 can be applied to determine a reasonable conservative estimate of the flexural resistance for these member types (White and Jung 2003b). Since these types of unbraced lengths are often subjected to significant moment gradients, the C_b factor can be significantly larger than one. In many of these situations, inclusion of the C_b factor is sufficient to increase the capacity to the maximum potential value M_{max} shown in Fig. 1. Therefore, more rigorous solutions that account for torsional restraint from the slab and the distortional buckling of the steel I-section, such as discussed in (Bradford and Gao 1992) and (Oehlers and Bradford 1999), are not required. The C_b calculation procedure discussed by White and Chang (2003a) is well suited for these types of members. As discussed by White and Chang (2003a) and White and Jung (2003b), the rotational restraint from the slab about the member's longitudinal axis at the tension flange may be less than fully-fixed for large slender-web members with a relatively small slab. The equations from F5 provide a reasonable estimate of the compression flange lateral buckling capacity in these cases. Bradford and Gao (1992) assume fully-fixed torsional restraint at the tension flange. In most composite beams used in building construction, Article F4 would usually apply and would tend to provide a somewhat conservative estimate of the predictions by the

above more rigorous distortional buckling procedures. However, if C_b increases the strength to M_{max} in Fig. 1, the same design strength is obtained in spite of the conservative assumptions in the suggested LTB equations.

ARTICLE F6 – I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

Article F6 addresses the weak-axis flexural resistance of I-shaped members and channels in one location within the suggested provisions. LTB is never a consideration in this case, and therefore, the flexural resistance is governed entirely by flange local buckling. For purposes of simplicity, the suggested provisions adopt the approach taken in Appendix F1 of (AISC 1999) for this limit state. Appendix F1 of (AISC 1999) uses the same compact- and noncompact-flange limits λ_{pf} and λ_{rf} in minor-axis flexure as for major axis bending of rolled I-shapes. However, in recognition of the beneficial effects of the strain gradient within the flange plates for sections in minor-axis flexure, F_L is effectively set to F_y. Unfortunately, Appendix F1 defines the moment corresponding to first nominal yielding including residual stress effects – M_r in (AISC 1999) or M_{yr} in this paper – as F_yS_y. However, it defines λ_r as $0.83\sqrt{E/F_L}$, where $F_L = F_y - 10$ ksi (F_y – 69 MPa) for rolled I-shapes and $F_y - 16.5$ ksi (F_y – 114 MPa) for welded I-shapes. This apparent oversight leads to a discontinuity in the FLB strength curve at the noncompact-flange limit λ_{rf}. The suggested Article F6 provisions correct this discrepancy by taking $F_{yr} = F_y$ for FLB in minor-axis flexure, and by writing $\lambda_r = 0.83\sqrt{E/F_y}$ for weak-axis flexure of I-shapes and channels in Table B4.1 (see Case 3 of Appendix II). This equation is obtained by substituting a value of $k_c = 0.76$ along with $F_{yr} = F_y$ within the corresponding general form for λ_r listed under Case 2. The analytical solution for elastic local buckling of a flange plate simply-supported at the web flange juncture and subjected to a linear variation in the longitudinal normal stress between a value of zero at the web flange juncture and a maximum value at the flange tip is $k_c = 0.56$ (CEN 1993). The Engineer should note that Eq. F6-1 governs the resistance for the majority of rolled I-shapes and for all rolled channel shapes, since only a few I-shapes and no channel shapes have noncompact flanges.

It should be noted that for some section types considered in Chapter F, (AISC 1999) limits the maximum flexural resistance to $1.5M_y$. The largest shape factor M_p/M_y for weak-axis bending of rolled I-shapes is approximately 1.6 and practical values for built-up I-shapes will tend to be smaller. Therefore, the maximum potential flexural resistance for a rolled I-shape in Article F6 is effectively $M_n = M_p$. However, rolled channel sections generally have a large shape factor. Therefore, Eq. F6-1 effectively limits the resistance of these members to $1.6M_y = 1.6F_yS_y$. The Engineer should note that M_y and S_y are defined as the smaller of the values corresponding to the extreme tension and compression fibers in Appendix IV.

For the unusual case of a built-up channel section with noncompact or slender flanges bent such that the tips of the flanges are in tension, Eqs. F5-2 and F5-3 are particularly conservative. The theoretical value of the elastic FLB coefficient for such a plate subjected to a maximum compression at the web-flange juncture and an equal tension stress at the flange tip is $k_c = 23.8$ (CEN 1993). These nonroutine types of problems are addressed conservatively by Article F5 in order to maintain simplicity of the provisions.

A comprehensive approach to the design of I-shaped flexural members for combined major-axis bending, minor-axis bending, and torsion is provided in (AASHTO 2003). The (AASHTO 2003) provisions use resistance equations similar to those suggested in this paper for major-axis bending, but account for flange lateral bending due to combined weak-axis moment and warping torsion by effectively handling the flanges as equivalent beam-columns. The (AASHTO 2003) procedures are targeted at the design of both straight- and horizontally-curved bridge I-girders with flanges satisfying a fabrication limit of $b_f/2t_f \leq 12$ and elastically-computed flange lateral bending stresses $f_\ell \leq 0.6F_y$.

ARTICLE F13 – MISCELLANEOUS STRENGTH LIMIT STATES AND PROPORTIONING REQUIREMENTS FOR FLEXURAL MEMBERS

It is suggested that a separate article, labeled here as Article F13, should address various other limit states directly related to the flexural resistance as well as various section proportioning requirements. For instance, this section should address the limit state of tension rupture at sections containing holes within the tension flange. Also, it is suggested that the general proportioning requirements for slender-web and monosymmetric I-sections should also be placed within this article. Alternatively, these limits could be specified before the corresponding flexural resistance equations are presented. However, separate articles govern the major- and minor-axis flexural resistances of these types of members. Also, since slender-web and monosymmetric I-sections tend to be used less often for the types of structures typically designed by the AISC Specifications, these limits are relegated to a latter Article of Chapter F with references to this article from Articles F4, F5 and F6.

Equation F13-1 is a general limit intended to disallow the application of the I-shape flexural resistance provisions for sections that are approaching a tee shape.

Equations F13-2 and F13-3 are the same as the comparable equations in (AISC 1999) Appendix G, except that in Eq. F13-3, the residual stress effect is taken as $0.3F_y$ rather than 16.5 ksi (114 MPa). For F_y = 50 ksi (345 MPa), this equation gives a maximum limit on h/t_w of 244. The upper limit of $h/t_w = 260$ does not govern unless $F_y < 47$ ksi (320 MPa).

A maximum limit of 10 on the ratio of the web area to the compression flange area is retained from (AISC 1999). As noted in the commentary to the current Specification, this is a somewhat arbitrary limit to prevent the R_b expression from being applied to sections approaching a tee shape. The largest values of this parameter considered in the experimental tests documented by White et al. (2003a, b and c) are only in the vicinity of 4.0. Therefore, it is suggested that the use of prismatic members with values of this parameter significantly larger than 4.0 should be discouraged. (AASHTO 2003) effectively restricts the ratio of the web area to the compression flange area to a maximum value of 5.5 by requiring $b_f \geq h/6$ and $t_f \geq 1.1t_w$.

UNIFIED NATURE OF SUGGESTED PROVISIONS

The suggested provisions for I-shaped members in major-axis bending are flowcharted in Appendix III. These flowcharts are included to aid the Engineer in understanding the unified nature of the suggested provisions. The flexural resistances produced by Appendix III are identical to those calculated using the provisions summarized in Appendices I and II. However, in Appendix III, the flexural resistances are addressed for all the I-section types within one

unified set of equations. This is achieved by defining the web slenderness based parameters R_b, R_{pc} and R_{pt} for all the I-section types, and also by addressing the limit states of LTB, FLB and TFY for all types of I-sections. Figure III.1 illustrates the calculation of the web slenderness based parameters R_b, R_{pc} and R_{pt} as well as the general calculation of F_{yr}. R_b is taken equal to one for compact and noncompact webs and R_{pc} and R_{pt} are taken equal to one for slender webs. All of these terms are then used subsequently in the FLB procedures of Fig. III.2 and the LTB procedures of Fig. III.3. Figure III.2 presents the FLB equations in a generalized form. Figure III.3 then does the same for the LTB and TFY equations and illustrates the final calculation of the flexural resistance as the minimum of the applicable independent flexural resistance equations. As noted previously, although it is possible to write the suggested Specification provisions more concisely as a single article applicable to all types of I-shaped members and channels, this would make the Specification more difficult to learn and more difficult to apply for routine design. However, once the Engineer grasps the unified nature of the provisions, the more complex provisions of Article F4 and F5 are more easily learned and applied because of their similarity to Articles F2 and F3.

COMPARISON TO EXPERIMENTAL TEST RESULTS – DETERMINATION OF RESISTANCE AND SAFETY FACTORS

Figures 11 through 15 and Tables 1 through 6 summarize the ratio between strengths determined from experimental tests and the calculated nominal strengths per the suggested provisions, M_{test}/M_n, for a wide range of geometries, loading configurations, steel materials, and governing limit states on determinate beams and girders. A more detailed presentation and assessment of the data may be found in (White et al. 2003 a, b and c). Figures 11 and 12 plot M_{test}/M_n versus the normalized effective length $KL_b(F_{yc}/E)^{0.5}/r_t$ for 250 rolled and welded I-section uniform bending tests governed by LTB using the suggested provisions. These tests include members with large unbraced lengths that fall within the inelastic and elastic LTB regions of Fig. 1 as well as members with close brace spacing that fall within the compact region shown in this figure. The Engineer should note that the suggested L_p equation corresponds to a normalized value of 1.1 on the abscissa of these plots. The proposed L_r corresponds to a normalized value equal to 3.75 on the abscissa of these plots for slender-web members, and to values generally greater than 3.75 for noncompact-and compact-web members. The K values are calculated per the (SSRC 1998; Nethercot and Trahair 1976) procedure for doubly-symmetric members, and by White and Chang's (2003b) extension of this procedure for monosymmetric I-sections. As noted previously, this procedure is a simplified design-oriented approach that gives reasonably accurate to conservative elastic K values corresponding to LTB. Tables 1 and 2 give basic statistical data corresponding to Figs. 1 and 2, while Table 3 gives a statistical summary for these combined data sets. The data is subdivided into rolled and welded I-section groups to emphasize the well known fact that the dispersion in the test data for welded I-sections is somewhat larger than that for rolled I-sections (Fukumoto and Kubo 1977).

The summary statistics for the tests with $KL_b(F_{yc}/E)^{0.5}/r_t \leq 1$ and ≤ 2 in Table 3 are comparable to the corresponding statistics for braced determinate beams under uniform moment reported by Yura et al. (1978). Yura et al. (1978) report a mean and coefficient of variation on M_{test}/M_n of 1.02 and 6 % for 33 tests. Also, the summary statistics in Table 3 for $KL_b(F_{yc}/E)^{0.5}/r_t > 4$ are comparable to the statistics for 185 elastic LTB tests reported by Yura et al. (1978). The

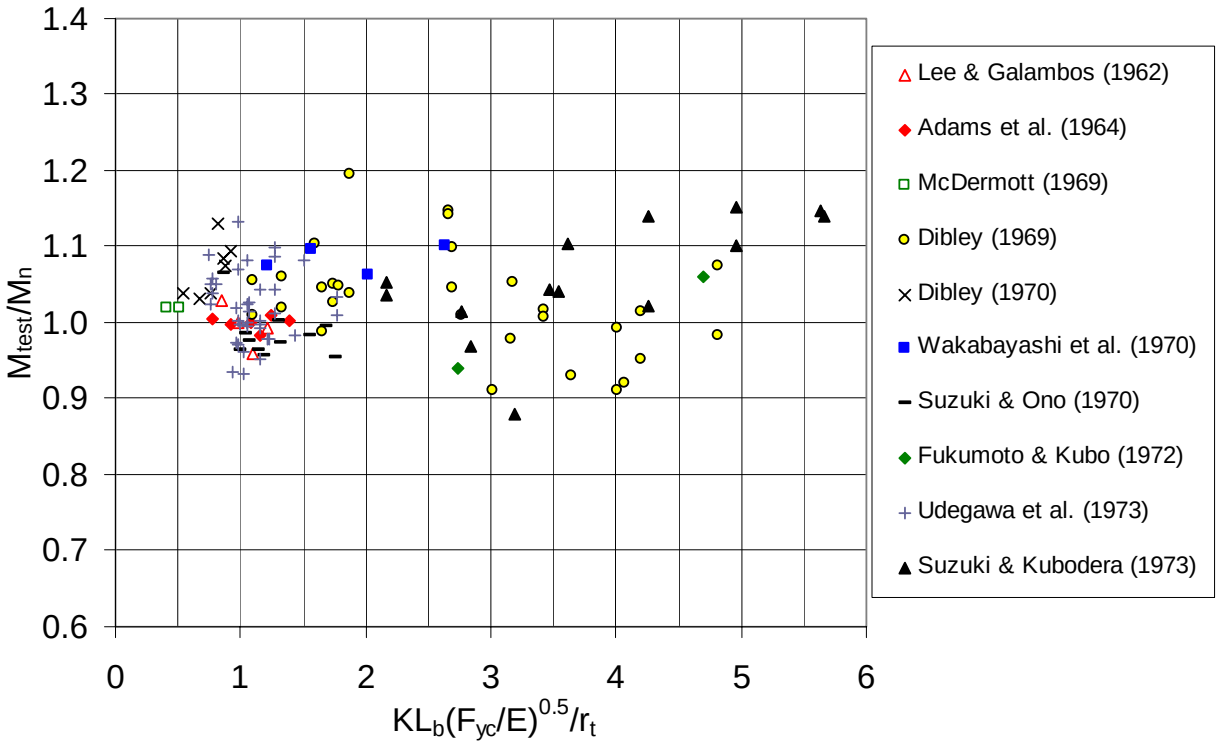


Fig. 11. M_{test}/M_n versus $KL_b(F_{yc}/E)^{0.5}/r_t$ for 122 rolled I-section uniform bending tests governed by LTB, K calculated per (SSRC 1998; Nethercot and Trahair 1976).

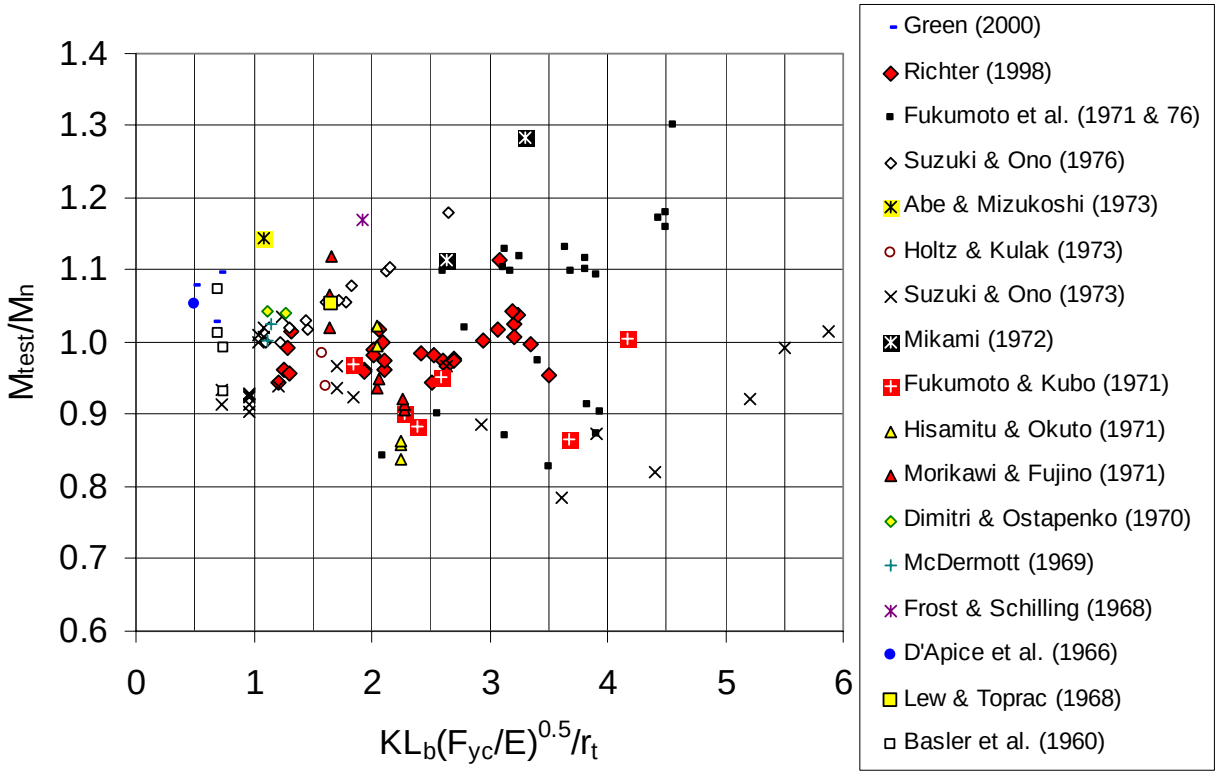


Fig. 12. M_{test}/M_n versus $KL_b(F_{yc}/E)^{0.5}/r_t$ for 128 welded I-section uniform bending tests governed by LTB, K calculated per (White and Chang 2003b).

Table 1. Statistical summary, M_{test}/M_n vs. $KL_b(F_{yc}/E)^{0.5}/r_t$ for 122 rolled I-section uniform bending tests governed by LTB, K calculated per (1998; Nethercot and Trahair 1976).

	$KL_b (F_{yc}/E)^{0.5} / r_t \leq 1$	$1 < KL_b (F_{yc}/E)^{0.5} / r_t \leq 2$	$2 < KL_b (F_{yc}/E)^{0.5} / r_t \leq 3$	$3 < KL_b (F_{yc}/E)^{0.5} / r_t \leq 4$	$KL_b (F_{yc}/E)^{0.5} / r_t > 4$	All Tests
Minimum	0.93	0.93	0.94	0.88	0.86	0.86
Median	1.03	1.01	1.05	1.01	1.02	1.02
Maximum	1.13	1.20	1.15	1.10	1.15	1.20
Mean	1.03	1.02	1.05	1.00	1.02	1.02
COV (%)	4.55	4.79	6.08	7.12	9.08	5.96
N	29	52	12	10	19	122

Table 2. Statistical summary, M_{test}/M_n vs. $KL_b(F_{yc}/E)^{0.5}/r_t$ for 128 welded I-section uniform bending tests governed by LTB, K calculated per (White and Chang 2003b).

	$KL_b (F_{yc}/E)^{0.5} / r_t \leq 1$	$1 < KL_b (F_{yc}/E)^{0.5} / r_t \leq 2$	$2 < KL_b (F_{yc}/E)^{0.5} / r_t \leq 3$	$3 < KL_b (F_{yc}/E)^{0.5} / r_t \leq 4$	$KL_b (F_{yc}/E)^{0.5} / r_t > 4$	All Tests
Minimum	0.90	0.92	0.84	0.78	0.82	0.78
Median	0.93	1.01	0.97	1.03	1.01	1.00
Maximum	1.10	1.17	1.18	1.28	1.30	1.30
Mean	0.98	1.01	0.97	1.01	1.06	1.00
COV (%)	7.14	5.49	7.98	11.70	14.16	9.06
N	15	39	38	27	9	128

Table 3. Statistical summary, M_{test}/M_n vs. $KL_b(F_{yc}/E)^{0.5}/r_t$ for 250 rolled and welded I-section uniform bending tests governed by LTB, K calculated per (White and Chang 2003b).

	$KL_b (F_{yc}/E)^{0.5} / r_t \leq 1$	$1 < KL_b (F_{yc}/E)^{0.5} / r_t \leq 2$	$2 < KL_b (F_{yc}/E)^{0.5} / r_t \leq 3$	$3 < KL_b (F_{yc}/E)^{0.5} / r_t \leq 4$	$KL_b (F_{yc}/E)^{0.5} / r_t > 4$	All Tests
Minimum	0.90	0.92	0.84	0.78	0.82	0.78
Median	1.02	1.01	0.98	1.02	1.02	1.01
Maximum	1.13	1.20	1.18	1.28	1.30	1.30
Mean	1.02	1.02	0.99	1.01	1.04	1.01
COV (%)	6.00	5.08	8.27	10.62	10.94	7.75
N	44	91	50	37	28	250

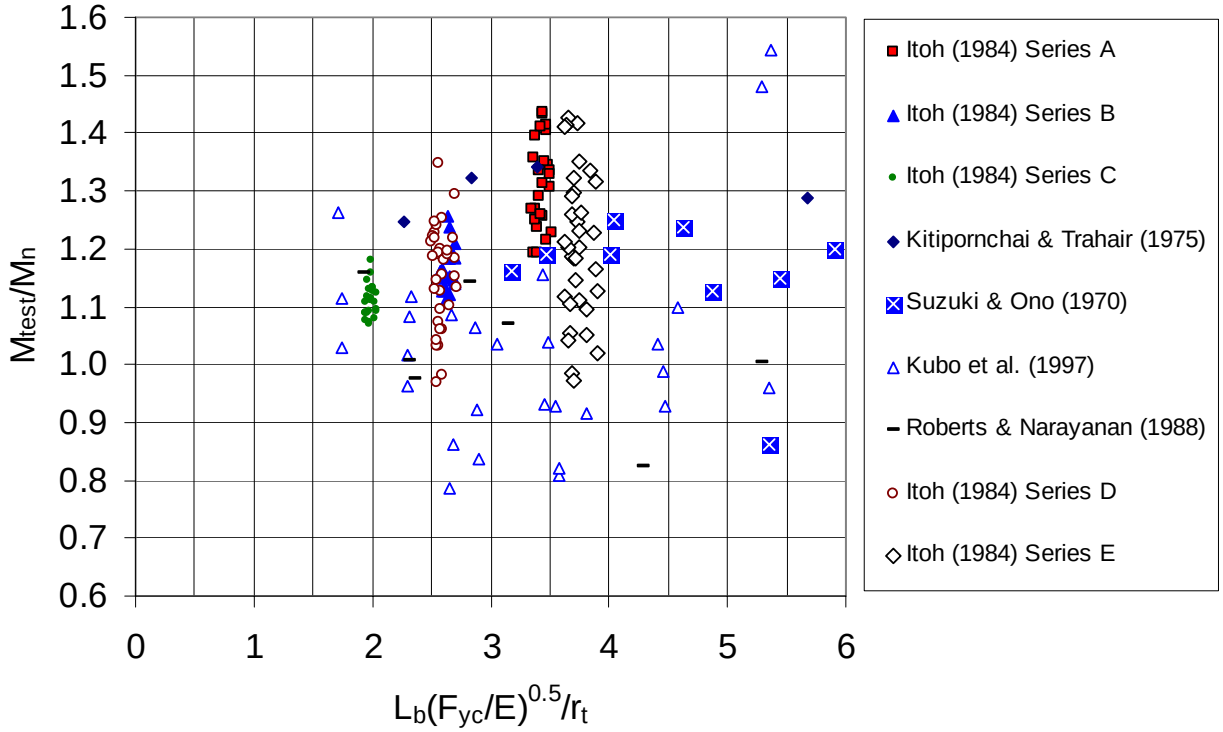


Fig. 13. M_{test}/M_n versus $L_b(F_{yc}/E)^{0.5}/r_t$ for 202 LTB tests of simply-supported I-section members braced at their ends and loaded by a concentrated load at their midspan, K=1.

Table 4. Statistical summary, M_{test}/M_n vs. $KL_b(F_{yc}/E)^{0.5}/r_t$ for 202 LTB tests of simply-supported I-section members braced at their ends and loaded by a concentrated load at their midspan, K=1.

	Rolled I-Sections	Doubly-Symmetric Welded I-Sections	Monosymmetric Welded I-Sections	All Tests
Minimum	0.86	0.79	0.81	0.79
Median	1.16	1.15	1.04	1.15
Maximum	1.44	1.48	1.54	1.54
Mean	1.19	1.15	1.04	1.16
COV (%)	8.70	11.57	14.05	11.44
N	91	83	28	202

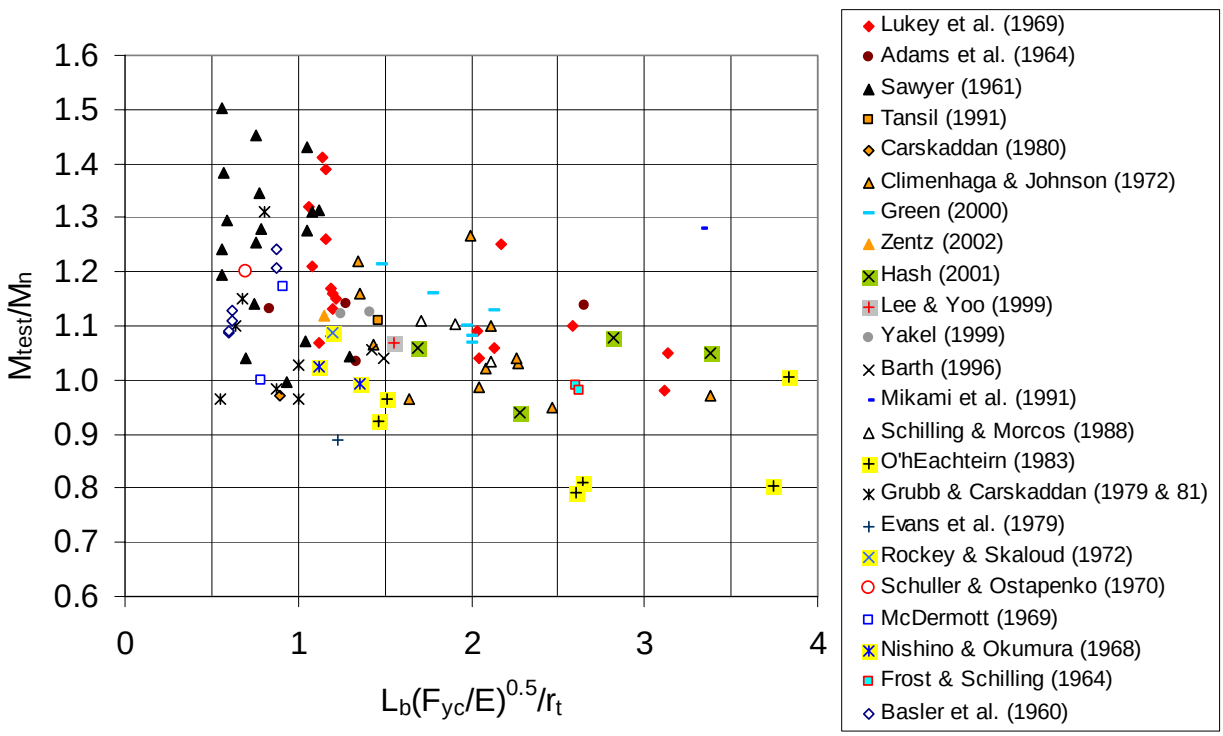


Fig. 14. M_{test}/M_n versus $L_b(F_{yc}/E)^{0.5}/r_t$ for 102 moment-gradient I-section member tests governed by LTB, K calculated per (White and Chang 2003b).

Table 5. Statistical summary, 102 moment-gradient I-section member tests governed by LTB, K calculated per (White and Chang 2003b).

	Minimum	Median	Minimum	Average	COV (%)	N
Rolled	0.98	1.17	1.50	1.20	11.54	40
Welded	0.79	1.07	1.31	1.06	10.24	62
All Tests	0.79	1.10	1.50	1.11	12.38	102

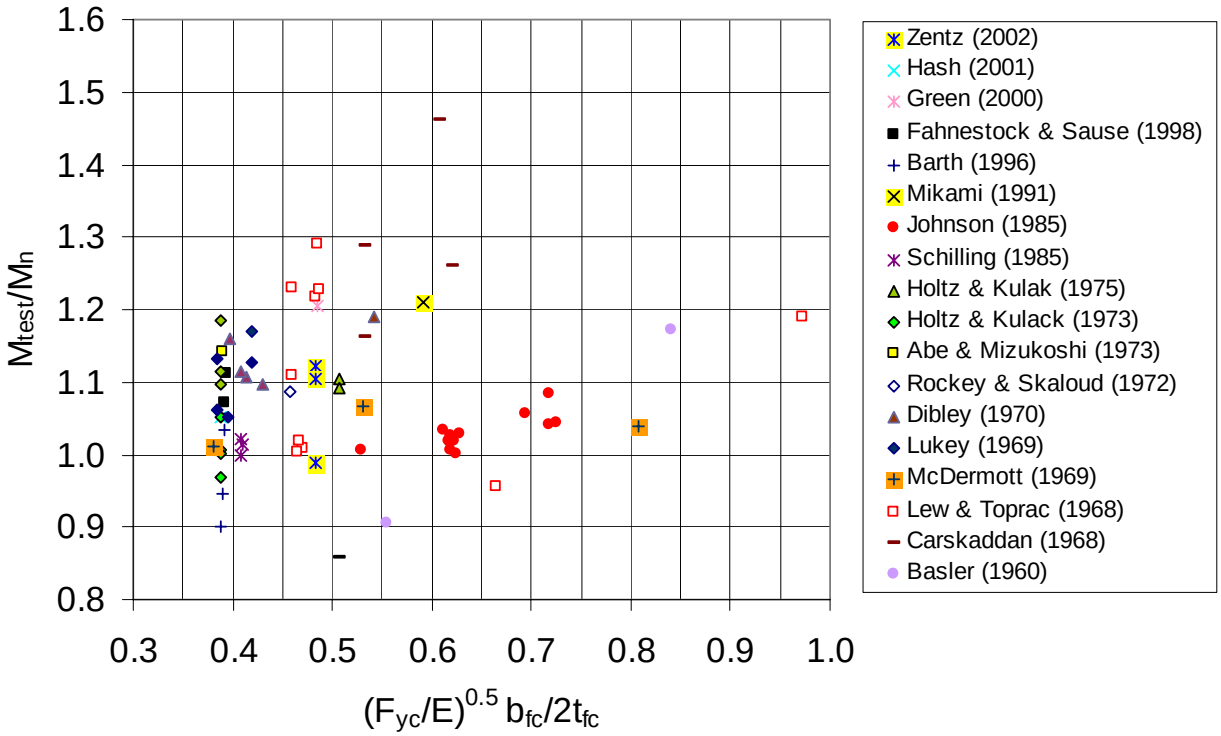


Fig. 15. M_{test}/M_n versus $(F_{yc}/E)^{0.5}\ b_{fc}/2t_{fc}$ for 70 I-section tests governed by FLB.

Table 6. Statistical summary, 70 I-section tests governed by FLB.

	Minimum	Median	Minimum	Average	COV (%)	N
Non-hybrid sections	0.90	1.05	1.21	1.06	6.75	50
Hybrid sections	0.86	1.11	1.46	1.13	12.98	20
All sections	0.86	1.06	1.46	1.08	9.40	70

compilation of test data in (White et al. 2003a) is focused on plastic and inelastic buckling of full-scale steel I-section members, whereas the above 185 tests considered by Yura et al. (1978) include various loading conditions and shapes that failed by elastic LTB. Yura et al. (1978) report a mean and coefficient of variation on M_{test}/M_n of 1.03 and 9.3 % for the above 185 tests. The Engineer should note that the mean values are close to 1.0 for all the groupings of the data in Table 3.

It should be noted that M_n is calculated in all cases within Figures 11 through 15 and Tables 1 through 6 using the nominal elastic modulus of E = 29 000 ksi (200 000 MPa). However, the measured values of the dimensions and of the material yield strengths are used in calculating the M_n values. The elastic modulus is not measured or reported in many of the experimental studies, and furthermore, it is well known that the accurate determination of the elastic modulus requires special procedures beyond those employed in ordinary tension coupon tests (Galambos and Ravindra 1978; ASTM1997).

Figure 13 shows the data for 202 LTB tests of simply-supported I-section members braced only at their ends and loaded by a concentrated load at their midspan. The effective length factor for LTB in all of these tests is K = 1. A majority of these tests were conducted with the midspan concentrated load applied at or slightly above the level of the top flange, but this load is applied at the mid-depth of the web and at the level of the bottom flange in a few tests (specifically in a number of the tests denoted by the solid square symbols with a white x inside of the solid square). The load-height effect in these tests is accounted for within the calculation of the C_b coefficient. Equation (5.6) of (SSRC 1998) is used. Table 4 presents the basic statistical data corresponding to these tests. Yura et al. (1978) do not report data for this specific category. The tests within this category are grouped into rolled I-sections, doubly-symmetric welded I-sections, and monosymmetric welded I-sections in this table. The rationale for this subdivision is that there is a significant difference in the statistics for each of these three groups. Only six of the tests in Figs. 11 and 12 were of monosymmetric I-section members, and therefore, separate reporting of the monosymmetric data for these tests is not considered to be statistically relevant. A large number of the data points that have M_{test}/M_n values less than one in Fig. 13 correspond to monosymmetric I-section tests with compact and noncompact webs ($2D_c/t_w$ = 68 to 93) considered in one study and by one group of investigators. These data points are denoted by the open triangles in the figure. It should also be noted that one-third of the tests considered in this study were doubly-symmetric I-sections, and that a significant number of these tests also have M_{test}/M_n values less than one.

Figure 14 presents the data for 102 I-section member tests governed by LTB in which the members are subjected to a moment gradient. Similar to the tests presented in Figs. 11 and 12, the K values are calculated per the (SSRC 1998; Nethercot and Trahair 1976) procedure for doubly-symmetric members, and by White and Chang's (2003b) extension of this procedure for monosymmetric I-sections. The moment gradient factor C_b is calculated for these tests using the equation

$$C_b = 1.75 + 1.05\frac{M_1}{M_2} + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.3 \tag{10}$$

where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment. Table 5 gives the basic statistical data for these tests, including a subdivision of the data into rolled and welded

I-section groups. All the moment diagrams in these tests are essentially linear. The rolled I-section data points are denoted by the solid symbols in Fig. 14. The nominal strength of all of the rolled I-section members considered in this plot is $M_n = M_p$; however, the actual strengths achieved in many of these tests are significantly larger than M_p, as high as $1.5M_p$ in one of the tests. Of particular note is that a significant fraction of these tests correspond to highly-compact plastic-design type cross-sections and relatively large moment gradients. The welded I-section tests tend to have thinner webs in aggregate, and therefore tend to have M_{test}/M_n values that are closer to 1.0. A significant number of these tests are also compact I-sections however. Furthermore, a number of tests included in this group are composite I-section members in negative bending, or all-steel I-section members with cover-plates on one side to simulate this type of design. These tests are denoted by the open symbols that are shaded orange inside of the symbol.

Although not a sufficient number to be statistically relevant as a separate group, one set of monosymmetric tests within Fig. 14 have particularly small M_{test}/M_n values. The corresponding data points are denoted by the black "+" signs with a yellow background within Fig. 14. These tests were conducted in a single study by one group of investigators. A distinguishing characteristic of these girders is that they have a smaller flange in compression and $2D_c/t_w \cong 100$, which is a larger web slenderness than other noncompact-web I-girder experimental tests compiled by White et al. (2003b). Also, the moment gradient in these girders is rather mild, producing C_b values of only 1.14 for the longer specimen lengths. The one data point at $[L_b(F_{yc}/E)^{0.5}/r_t, M_{test}/M_n] = [3.84, 1.01]$ corresponds to a test in which the member was nominally straight, that is, the specimen was fabricated such that the maximum offset of each flange was less than $L_b/1000$. The specimen for one of the data points corresponding to $[L_b(F_{yc}/E)^{0.5}/r_t, M_{test}/M_n] \cong [2.6, 0.8]$ was nominally straight, whereas the other data points for $L_b(F_{yc}/E)^{0.5}/r_t \cong 2.6$ and 3.8 are for nominally crooked specimens, i.e., specimens fabricated with a bow of approximately $L_b/250$. This group of tests gives some indication that monosymmetric I-sections with (1) a smaller flange in compression and (2) larger unsupported lengths within the inelastic buckling region may have difficulty in developing the calculated strengths. Nevertheless, other monosymmetric test results are predicted reasonably well. Further studies to corroborate the existing data for these types of members would be useful.

Finally, Fig. 15 plots the M_{test}/M_n values versus the normalized flange slenderness parameter $(F_{yc}/E)^{0.5}\, b_{fc}/2t_{fc}$ for 70 tests governed by the FLB strength equations in the suggested provisions. The compact flange limit corresponds to a value of 0.38 on the horizontal axis of this graph. It can be observed that a large fraction of the tests included within this group are specimens in which the flange slenderness only slightly violates the compact flange limit. Also, a significant number of moment-gradient as well as uniform bending tests are included in this group. However, no significant statistical differences are observed as a function of the shape of the moment diagram. The data is subdivided into 20 hybrid member tests versus 50 homogeneous or non-hybrid member tests. For the hybrid tests, the hybrid strength reduction factor detailed in (AASHTO 2003) is included within the calculations. The hybrid tests generally have a larger mean and a larger coefficient of variation than the non-hybrid tests. It should be noted that a number of the tests considered in Figs. 12, 13 and 14 are also of hybrid members. The hybrid strength reduction factor is calculated per (AASHTO 2003) for these members as well.

Preliminary calculations of resistance factors for LRFD similar to those described in (Yura et al. 1978) indicate similar values for ϕ. The specific values for the resistance factor vary somewhat depending on the groupings of the data. Therefore, a resistance factor of $\phi_b = 0.9$ for LRFD and a safety factor of $\Omega_b = 1.67$ for ASD appear to be appropriate. A further detailed assessment of the relevant experimental data should be made prior to establishing final values for these factors.

CONCLUDING REMARKS

The AISC LRFD (1999) Specification represents arguably the best combination of accuracy and practicality of all the standards for steel design throughout the world. In as such, this Specification is the most appropriate starting point for establishing a unified AISC LRFD and ASD Specification in 2005. However, there are areas where important improvements are possible. This paper presents and discusses detailed considerations in the development of suggested provisions for design of I-shaped members and channels subjected to major and minor-axis flexure. The suggested provisions span only six pages. Significant efforts have been made to simplify the provisions while also maintaining or improving their accuracy, consistency and generality. The author believes that the suggseted provisions represent a near optimum enhancement of the concepts and procedures established in the present AISC LRFD and ASD Specifications.

One important area of potential improvement is discussed within the introduction that is addressed only indirectly within the body of the paper. This improvement pertains to the intricate interactions between Appendix F1, Chapter B, and Appendix G in (AISC 1999) for beam-column members with noncompact webs. As noted in the introduction, it is possible to simplify these provisions greatly by determining the beam-column strength as follows: (1) calculating the strength of the member as a beam (under zero axial compression), (2) computing the strength of the member as a column (under zero bending), and (3) determining the resistance for the beam-column member using a simple interaction equation between these strengths. This is the type of approach taken in the AISI LRFD (AISI 2001) and Australian (SAA 1998) Standards. The nature and implications of this potential improvement are addressed in (White and Kim 2003).

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