Proceedings of the
Annual Stability Conference
Structural Stability Research Council
Pittsburgh, Pennsylvania, May 10-14, 2011

Flange Bracing Requirements for Stability of Metal Building Systems

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Abstract
Stability bracing requirements for metal building frames generally fall outside the scope of AISC's Appendix 6 equations. This leads to various interpretations of how one should design bracing for these highly economized and complex framing systems. This paper offers an overview of current codified equations, discusses why several common building types do not adhere to the assumptions underlying these equations, and comments on potential design solutions for bracing design based on assessment of the brace strength requirements plus limiting the brace point movement under the expected strength loads. Results from virtual simulation of representative beam cases are discussed. Finally, a list of key observations is compiled offering insight into how increased economy and more uniform safety may be achieved.

1. Introduction

The most recent codified requirements for stability bracing of columns, beams, and beam-columns can be found in Appendix 6 of the 2010 AISC Specification (AISC 2010). These provisions provide simplified design equations for several important but basic bracing situations, namely "relative" and "nodal" lateral bracing of columns and beams, and "nodal" and "continuous" torsional bracing of beams. Unfortunately, the stability bracing systems in metal building construction as well as other general construction, often do not match well with these basic cases. Therefore, practical stability bracing design typically involves significant interpretation and extrapolation of the basic rules. These rules often result in conservative designs; however, the true conservatism or lack of conservatism of the various ad hoc extrapolations is largely unknown.

There are various attributes of metal building systems that place their stability bracing design outside the scope of AISC's Appendix 6. A few of these that are addressed in this paper are:

1. Metal building frames make extensive use of web tapered members. AISC's Appendix 6 only encompasses prismatic members.
2. The stiffness provided is assumed to be equal at each brace per Appendix 6. This is often not achieved due to variations in girt or purlin size, and in bracing diagonal lengths and angles of inclination. In addition, Appendix 6 assumes uniform spacing of braces.

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3. Knee joints may not provide rigid twisting and lateral restraint to rafter ends; the AISC equations are based on the assumption of rigid bracing at the member ends.
4. Warping restraint from joints and continuity with other more lightly-loaded member segments, and the combined action of diaphragms and discrete braces may contribute significantly to the stability of critical segments. These effects are not accounted for using Appendix 6.
5. AISC's Appendix 6 targets the design of the braces for a single upper-bound estimate of the stiffness and strength requirements. However, some economy may be gained by recognizing that the bracing stiffness and strength demands often reduce very sharply as one moves away from a critical bracing location.
6. The AISC equations do not count on any interactions between lateral, relative, and torsional bracing, yet metal building frames often are inherently designed with permutations of all bracing types.

This paper provides a broad overview of the requirements for strength and stiffness of flexural members. Beam bracing, in general, is more complicated than its column counterpart as bracing for beams must account for both flexural and torsional influences on the member (Yura et al., 1992; Yura and Helwig, 2009). The following specific AISC requirements and suggested simplified equations for bracing of beams via lateral and torsional bracing are discussed in this paper.

2. Current Specification Provisions for Stability Bracing

Nodal Lateral Bracing, Strength Requirement:
The AISC nodal lateral bracing strength requirement is

$$P_{br} = 0.01(M_r/h_o)C_{tN}C_d \qquad \text{(1, AISC C-A-6-4b)}$$

where M_r is the required flexural strength in the beam from LRFD or ASD load combinations; M_r/h_o is the required equivalent flange force from the LRFD or ASD load combinations, taken as the largest value within the member length; C_{tN} is the flange load height factor; C_d is the double curvature factor; h_o is the distance between flange centroids; and P_{br} is the required axial strength of the brace. The reader is referred to AISC (2010) for specific definitions of the terms.

Nodal Lateral Bracing, Stiffness Requirement:
A refined estimate of the lateral bracing stiffness from the AISC Commentary (2010) is

$$\beta_{br} = \psi\left[2(4-\frac{2}{n})\frac{(M_r/h_o)}{L_q}\right]C_{tN}C_d \qquad \text{(2, AISC C-A-6-3)}$$

where $\psi = 1/\varphi = 1/0.75 = 1.33$ for LRFD and $\psi = \Omega = 2.0$ for ASD; n is the number of intermediate brace points within the beam length between the "end" rigid bracing locations; and L_q is the unbraced length obtained by setting the resistance with $K = 1.0$ to the required moment.

Nodal Torsional Bracing, Stiffness Requirement:
The refined torsional bracing stiffness given by the AISC Commentary (2010) may be written as

$$\beta_T = 20\psi h_o^2 \left[\frac{M_r/C_b h_o}{P_{e,eff}}\right]\left[\frac{M_r/C_b h_o}{L_b}\right]\frac{(n_T+1)}{n_T}C_{tT} \qquad \text{(3, AISC A-6-11)}$$

where $\psi = 1/\varphi = 1/0.75 = 1.33$ for LRFD and $\psi = \Omega = 3.0$ for ASD (Ω is usually taken equal to $1.5/\varphi$, but it is taken as $1.5^2/0.75$ in this case since the moment term appears twice in the equation); L_b is the spacing between the torsional brace points, assumed constant in the development of the equation; M_r/C_b is the equivalent uniform moment for a given unbraced length within the member span; C_b is the equivalent uniform bending factor for a given unbraced length, based on flange stresses for non-prismatic members; C_{tT} is the torsional bracing factor accounting for effects of the height of the transverse load, and n_T is the number of intermediate nodal torsional brace points within the member length between the rigid "end" brace locations, where both twisting and lateral movement of the beam are prevented. Yura et al. (1992) recommend that for $n_T = 1$, the term $(n_T + 1)/n_T$ may be multiplied by 0.75; $P_{e.eff}$ is the effective flange buckling load, equal to $\pi^2 EI_{eff} / L_b^2$; E is the modulus of elasticity of steel = 29,000 ksi; $I_{eff} = I_y$ for doubly symmetric sections and $I_{yc} + \frac{t}{c} I_{yt}$ for singly symmetric sections; c is the distance between cross section centroid and the centroid of the compression flange; t is the distance between the cross-section centroid and the centroid of tension flange; I_{yc} is the lateral moment of inertia of the compression flange; and I_{yt} is lateral moment of inertia of the tension flange.

Nodal Torsional Bracing, Strength Requirement:
Given the stiffness from Eq. 3 above and assuming an initial layover of the web of $\theta = \theta_o = 0.002L_b/h_o$, the strength requirement may be estimated as:

$$M_{br} = \frac{\beta_T}{\psi}\theta_o = \frac{\beta_T}{\psi}\frac{L_b}{500h_o} = 0.002\frac{\beta_T}{\psi}\frac{L_b}{h_o} \qquad \text{(4, AISC C-A-6-8)}$$

Sharma (2010) studied the application of the above equations to metal building frame members and compared the results to full nonlinear shell FEA virtual simulation using Abaqus (Simulia 2010) for several large-scale metal building frames. One of his examples and its conclusions is presented below to provide a motivation for the topics discussed in the remainder of the paper. The reader is referred to Sharma (2010) for a detailed discussion of the results.

3. Motivating Example: 90 Foot Clear-Span Frame

Numerous insights can be gained from the study of a ninety foot clear span frame example from Kim (2010) and White and Kim (2006). The original design of the frame was performed by Mr. Duane Becker of Chief Industries. The design check calculations for this frame can be found in Kim (2010). An elevation view of one-half of the frame is shown below in Figure 1.

An ASD gravity load combination including a uniform snow load is considered to act on the frame, since this produces the largest moments. The following observations are noted:

1. The AISC equations give very conservative estimates of the stiffness demands; however, the brace strength equations tend to underestimate the maximum bracing strength demands at the limit load of the most critical brace.
2. If the frame is redesigned with wider flanges, the brace strength and stiffness demands decrease substantially.

3. The torsional brace stiffness provided by representative minimal purlin sizes lies on the knuckle of the knuckle curves for system strength versus brace stiffness for this frame. Thus, a small decrease in the brace stiffness produces a relatively large decrease in the system strength, while a significant increase in the brace stiffness has a relatively small effect on the response.

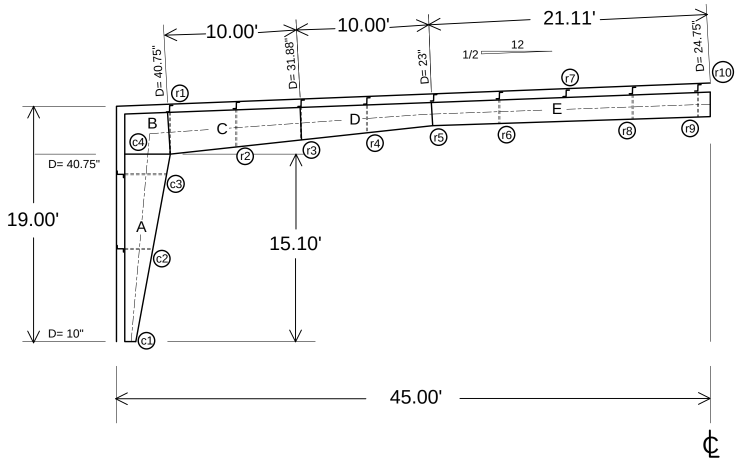

Figure 1: Elevation view of ninety foot clear-span frame, from Kim (2010)

The remainder of the paper is focused on a number of basic beam models aimed at shedding light on potential improvements for bracing of metal building structures, accounting for the above metal building system attributes, as well as other attributes mentioned in the introduction, and the corresponding behavior

4. Simplified Equations for Stability Bracing

The performance of torsional bracing is calculated via virtual test simulation in the following sections. However, prior to virtual simulations, it is useful to consider other potential simplified equations concerning the bracing demands. Based on research conducted by Tran (2009) and Sharma (2010), significant economy may be achieved in some cases through very basic estimates of the bracing strength and stiffness. Conceptually, by selecting an appropriate maximum brace force and an allowable rotation or deflection at this strength limit, a stiffness requirement can be extracted by dividing the brace force by a selected brace deflection limit. The following equations (from Sharma 2010) present one example of such a set of equations. These equations are referred to as the “simplified” equations in the subsequent discussions.

Nodal Lateral Bracing, Strength Requirement:

$$P_{brS} = 0.02P_r \tag{5}$$

The corresponding displacement limit is suggested as

$$\Delta_{max} = L_b/250 \tag{6}$$

Nodal Lateral Bracing, Stiffness Requirement:

If one divides the bracing strength requirement by the corresponding displacement limit, the following nodal bracing stiffness requirements are obtained:

$$\beta_{brS} = \frac{\alpha P_{brS}}{\Delta_{max}} = 5\frac{\alpha P_r}{L_b}$$

$$= 5\frac{P_r}{L_b}\ (LRFD)\ \ or\ \ \ 8\frac{P_r}{L_b}\ \ (ASD) \tag{7}$$

where α = 1.0 for LRFD and 1.6 for ASD.

Nodal Torsional Bracing, Strength Requirement:

$$M_{brS} = 0.02 M_r C_{tT} \tag{8}$$

The corresponding displacement limit is suggested as

$$\theta_{max} = \frac{L_b}{500 h_o} \tag{9}$$

Nodal Torsional Bracing, Stiffness Requirement:
Similar to the lateral requirement, if one simply divides the brace strength by the allowable rotation to calculate an effective stiffness, one obtains

$$\beta_{TS} = \frac{\alpha M_{brS}}{\theta_{max}}$$

$$= 10 C_{tT}\frac{M_r h_o}{L_b}\ (LRFD)\ \ or\ \ 16 C_{tT}\frac{M_r h_o}{L_b}\ (ASD) \tag{10}$$

5. Virtual Simulation Model Definition
General Layout:
To better understand the specific behavior, a series of individual beam cases were selected representing a variety of bracing, loading, and end condition scenarios. The base member selected was a W16x26. However, for most of the cases investigated, the distance between flange centroids (h_o) was doubled to 30.71 in. This modification was chosen as it created characteristic dimensions more representative of the proportions typically used in metal building frames. A summary of the beam cases is given below in Table 1.

Table 1: Beam Cases

Model	L (ft)	L_b (ft)	N	Loading[1]
L10-n1-U	20	10	1	Uniform
L10-n3-U	40	10	3	Uniform
L10-n5-FR	60	10	5	Full Reversal
L5-n10-FR	55	5	10	Full Reversal & Linear
L4-n10-FR	44	4	10	Full Reversal
L3-n10-FR	33	3	10	Full Reversal
Tapered	30	5	5	Linear

1. **Uniform** moment, full **reverse**-curvature bending, or **linear** variation in moment from a maximum to zero along the full length of the member.

Within each loading case, combinations of torsional, lateral, and relative bracing were applied with consideration given to flexible versus rigid end restraints. Table 2 provides an illustrative matrix of the scenarios investigated.

Table 2: Beam Case Scenarios

Model	Scenario[1]				
	T – RE	T – FE	L – RE	L – FE	R – RE
L10-n1-U	x	x	x	x	x
L10-n3-U	x	x	-	-	-
L10-n5-FR	x	x	x	x	x
L5-n10-FR	x	x	x	x	x
L4-n10-FR	x	x	-	-	-
L3-n10-FR	x	x	x	x	x
Tapered	x[2]	-	-	-	-

1. T is torsional bracing, L is lateral bracing, R is relative bracing, RE is rigid ends, and FE is flexible ends.
2. This torsional case includes an additional incidental lateral restraint, discussed subsequently.

In addition to the scenarios tabulated, L5-n10-FR was analyzed with non-compact flanges (the initial configurations' were compact), a compact and slender web (the original had a non-compact web) and was investigated with an incidental lateral stiffness applied in tandem with torsional stiffness.

Imperfections:
Initial residual stresses were included in the finite element models through the application of a residual stress pattern discussed in detail by Kim (2010). The residual stress pattern is fit to residual stress measurements provided by Prawel et al. (1974). A number of virtual test simulations were conducted by Kim (2010) comparing to experimental tests. The simulations show that this residual stress pattern provides a reasonable estimate of the experimental test results. This pattern is representative of welded I-section members commonly used in metal building construction.

Initial geometric imperfections were applied considering limits specified in the Metal Building Manufacturers Association's (MBMA) Metal Building Systems Manual (2006) and AISC's Code of Standard Practice (2010b). MBMA's standard allows a sweep of the member between brace points of *L/480* and an out-of-flatness of the web and flange of *D*/72, where *L* is the length of the member and *D* is the clear-depth between flanges. For the virtual simulations, the MBMA requirement was rounded to *L/500* for out-of-alignment and out-of-straightness of the unbraced segments. The web and flange imperfection was unaltered. The web and flange local buckling imperfections are obtained by summing various eigenvalue buckling modes, and the flange imperfections are obtained by explicit application of a flange sweep to maximize the critical brace force, using an influence line type of approach (Sharma 2010). These imperfections are obtained by various pre-analyses and are imposed as strain-free initial imperfections for the virtual test simulations. Figures 2 and 3 show a typical specified "control point" imperfection for the flange out-of-alignment and sweep and the corresponding deflected shape, respectively.

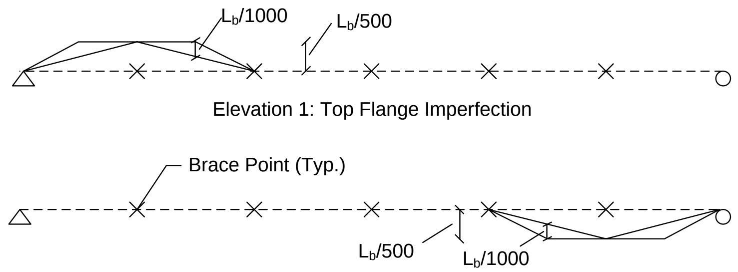

Figure 2: Applied imperfections to top and bottom flanges

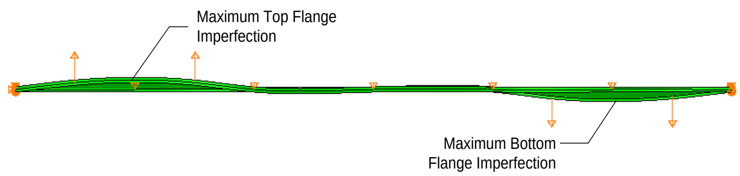

Figure 3: Exaggerated deflected shape from Abaqus
(view looking down on the top flange from above)

6. Virtual Simulation Results

In all of the scenarios, the simplified equations were used to calculate a target stiffness for the bracing scheme. Next, the beam was analyzed several times with fractions or multiples of the target stiffness. Finally, knuckle curves were created, plotting normalized member strength versus brace stiffness. These curves were then compared to the AISC and simplified requirements. In all cases, a stiffness and brace strength requirement was extracted from the knuckle curves at a value of stiffness required to reach 90% of the normalized capacity of the rigidly-braced member. The stiffness and strength required to get to 90% of the rigidly-braced beam capacity is discussed throughout this section. This limit has been suggested by a number of authors, e.g., Stanway et al. (1992a & b), as a reasonable criterion for brace design.

End Twisting and Lateral Restraint Effects:

The knee region is often the most critical region of a metal building frame due to the high moments at the rafter-column juncture. By considering the rafter as rigidly braced at its ends, one assumes that the column is providing full lateral and twisting restraint to the rafter ends. Obviously, this assumption is rarely met in practice. Thus, the effect of having a flexible end should be considered in the overall bracing design. This was simulated in all the flexible end cases by applying a torsional brace at each end with a stiffness equal to that of interior braces.

By comparing the torsional rigid end and flexible end cases (see Table 2), every flexible end case saw an increase in brace stiffness required to reach 90% of the system strength when compared with the rigid end cases. These increases ranged from 40% for cases involving uniform moment to 270% for cases with full reverse-curvature bending. Similarly, the brace strength requirements increased by 3% to 40%; however some brace force demands decreased.

In all cases, the AISC requirements were accurate to conservative for rigid end bracing, yet were often lacking capacity for flexible end braces. Contrary to AISC, the simplified equations ranged from unconservative for rigid ends and uniform bending to extremely conservative for rigid ends and reverse-curvature bending. However, the simplified equations provided reasonable conservative estimates of the required stiffness for full reverse-curvature bending when the ends were flexible.

For the lateral bracing cases, the simplified method was able to shave from 7 to 100% off of the stiffness requirements for rigid end bracing versus the AISC requirements while still providing an adequate design. Furthermore, the increase in brace stiffness demand in the cases with flexible ends was captured by the simplified equations in most cases.

Rapid Drop in Brace Demand Away from Critical Regions:
From initial analyses performed by Sharma (2010), sections of the rafter away from the critical knee or ridge regions often see significantly smaller stiffness and strength demands. Thus, rafters with a large number of brace locations have the potential to be designed with bracing stiffnesses much less than required at the critical regions. Of course, the critical loading for each brace would need to be considered via force envelopes from the global frame analysis.

In general, plots of normalized brace force for full reversal of moments showed a rapid attenuation of forces from the critical end segment. Figure 4 shows one such plot for the torsional braces in L5-n10-FR. One can see that the critical intermediate braces (1 and 10 in this case) and the rigid ends are the only braces with significant force in this beam. In addition, typically the interior brace locations often are subjected to smaller force from the moment envelopes relative to the local beam capacity, even in tapered members. Thus, one might suspect that trimming the interior brace stiffnesses might be a feasible (and safe) reduction in steel.

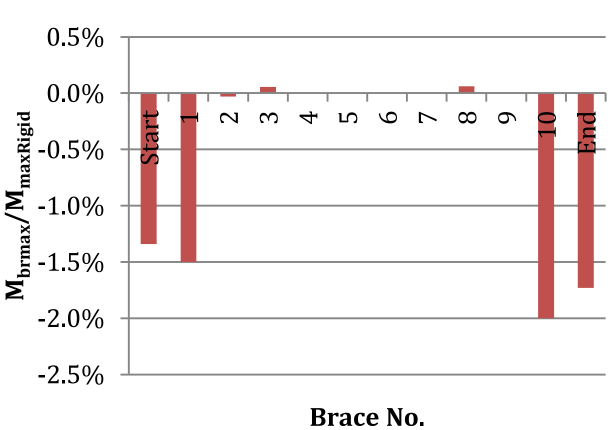

Figure 4: Normalized brace force at stiffness level to reach 90% of the system strength

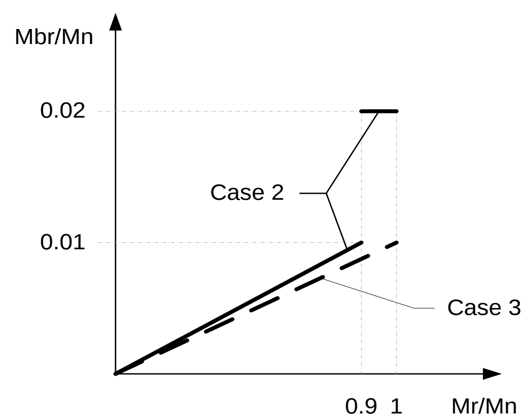

Figure 5: Normalized brace force versus normalized frame demand

Three additional cases were considered and run subsequent to the above findings:

Case 1: Each brace was allowed to have a different stiffness; calculated using Eq. 8 with M_r taken as the largest moment within each brace's adjacent unbraced lengths.

Case 2: Using the stiffness from Case 1 plus an additional reduction in brace stiffness based on adjusting the 0.02 factor used in Eq. 8 via the piecewise continuous "Case 2" curve from Fig. 5 above (dependent on the ratio M_r/M_n).

Case 3: Using the stiffness from Case 1 plus an additional reduction in brace stiffness based on adjusting the 0.02 factor used in Eq. 8 via the linear "Case 3" curve from Fig.5 above (dependent on the ratio M_r/M_n).

In these cases, the brace designs were based on the largest moment in the adjacent unbraced lengths for each brace while keeping the ends rigid. These cases produced maximum beam strengths of 99.9%, 99.6%, and 95.7%, respectively, of the capacity reached by assigning all braces the same stiffness (as shown in Fig. 4 above). Thus, a reduction of the brace stiffness away from the critical regions used in Cases 1, 2, and 3 appears to afford an economic advantage. It should be noted that all of these cases were analyzed under the specific loading diagrams mentioned in Table 1. The reductions may be more minor in practical frames where the design moment envelope must be considered and the member is tapered. Further analyses must be completed before specific recommendations can be offered. Larger attenuation of the brace forces along the member length is possible in situations where the LTB failure is within the elastic buckling range, and if the braces are designed considering significant partial bracing response.

Incidental Lateral Restraint Effects:
An argument can be made that frequently, multiple types of bracing act on a structure simultaneously. For instance, one might include the shear panel stiffness provided by a roof or wall diaphragm along with the flange bracing diagonals (torsional bracing) applied at the rafters. The current AISC Appendix 6 provisions do not count on any "coupling" of bracing systems.

For these scenarios, a nodal lateral stiffness equal to 10% of the AISC requirement (Eq. 2) was applied in addition the torsional stiffness (determined from previous analyses as the torsional stiffness required for the beam to reach 90% of its rigid-braced strength); giving each brace equal lateral and torsional stiffness. Then, the strength of the beam under this combined bracing scheme was compared to how much more than 90% of the rigid-strength it could now reach. For the case of L5-n10-FR, the system strength increased from 90% of the rigid-braced capacity to 94% of the rigid-braced capacity. The maximum normalized torsional brace force decreased from 2% with only torsional bracing to 0.5% with combined torsional and lateral bracing; a four-fold decrease! A minor decrease in the stiffness requirement was also observed when lateral stiffness was added to the torsional stiffness. Thus, the addition of lateral bracing as a supplement to torsional bracing not only increased the overall beam strength but also substantially decreased the strength design requirement for the torsional braces.

Small Brace Force up to the Limit Load:
From the virtual simulations, the brace force remains relatively low in the targeted case studies until just before the limit load is reached. Figure 6 below shows the normalized capacity versus normalized brace force for a range of stiffness for L5-n10-FR. One immediately notices the plateaus in strength corresponding to a sharp increase in brace force as the system limit load is approached. The stiffness associated with $0.5\beta_{TS}$, $1.0\beta_{TS}$, and $2.0\beta_{TS}$ all have a 90% up-crossing around 0.6% brace force, yet do not peak until around 2.5% for only a marginal strength gain. If it is considered sufficient for a brace to fail at a load level close to the otherwise maximum strength of the structure, maximum brace force requirements of approximately 2 % of the member moment appear to be sufficient from this study, and from a broader range of studies

considered by Sharma (2010). If this is not considered sufficient, the braces need to be designed in general for up to approximately 4 % of the local member internal force. It should be noted that both of these requirements are often larger than the requirements specified by AISC (2010) Appendix 6. These limits appear to be reasonable for both inelastic and elastic LTB cases.

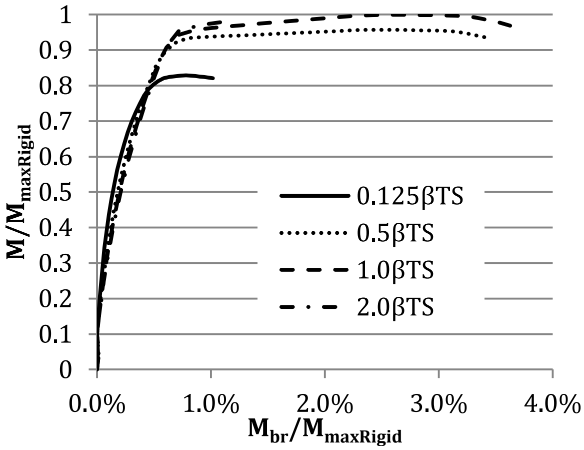

Figure 6: Beam strength v. brace force demand for L5-n10-FR

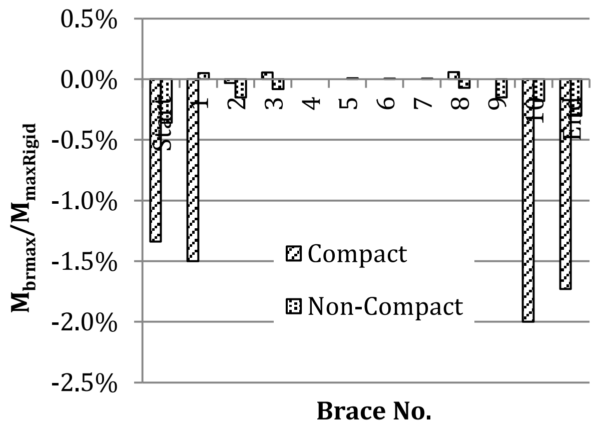

Figure 7: Flange slenderness comparison at stiffness to reach 90% of the system strength

Local-Buckling Protects Brace:

The original beam cases have a non-compact web and compact flanges. Permutations were created to look at the effects of the brace force and stiffness when the flanges are non-compact as well as where webs are slender versus compact. Figure 7 above shows the result of increasing the slenderness of the flanges from a classification of compact to non-compact for L5-n10-FR. It is apparent that there is a significant drop in the critical brace force in brace number 10. Also, the stiffness required to reach 90% of the capacity of the beam dropped over two-fold.

Unfortunately, a similar drop was not seen by increasing the web slenderness. Figure 8 below shows the effect of increasing the web slenderness. It should be noted that the critical imperfection was the same for all levels of slenderness. Thus, if one had selected an imperfection that emphasized the out-of-plane deformation of the web instead of the compression flange, a bigger drop may have been realized.

LTB K-Factor Consideration:

Typical design for bracing per AISC's Appendix 6 uses a K-factor of 1 for the critical unbraced lengths. The equations do not permit any benefit from warping restraint provided by unbraced segments adjacent to the critical segment. By completely restraining warping and lateral bending at the ends, the approximate K-factor is reduced to 0.5 and a substantial benefit is realized in bracing demand (see Figure 9 below) for L5-n10-FR (at the member design load level corresponding to K = 1, and at the limit load level in the virtual test simulation). For real systems, the actual K-factor will be bounded essentially by the 0.5 limit and would likely produce brace forces between the values in Fig. 9. Improved estimates of the torsional bracing stiffness requirements are obtained by Sharma (2010) for a number of example cases by using a $K < 1$ in the calculation of $P_{e.eff}$ of Eq. (3).

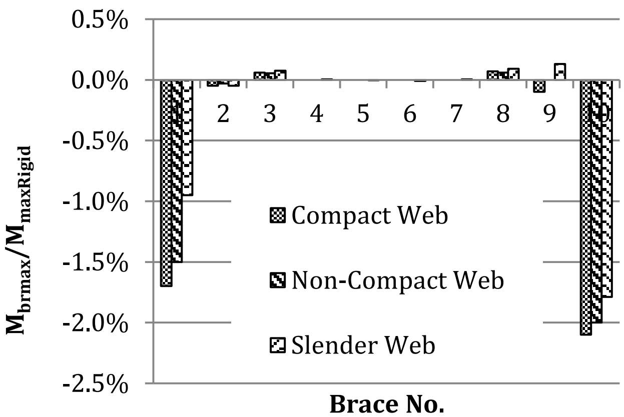

Figure 8: Web slenderness comparison at stiffness to reach 90% of system strength

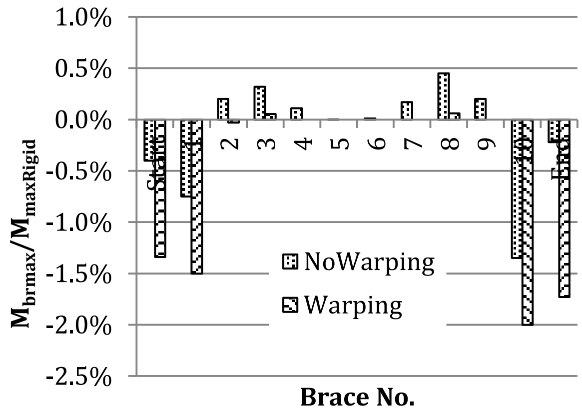

Figure 9: Warping restraint effects at stiffness to reach 90% of system strength

7. Summary of Key Observations

After compiling the results presented above with those by Sharma (2010), some key observations can be gleaned. Further analysis and simulation are warranted to corroborate these observations before any corresponding design recommendations can be proposed.

1. The AISC equations appear to work consistently well for rigid end, torsionally braced beams, despite their slight conservatism and are able to develop 90 % of the rigidly braced strength; a common criterion for brace design
2. Neither the AISC nor the example simplified equations seem to capture the effects of flexible end restraints for the torsionally braced beam all that well.
3. The lateral stiffness requirements are met most accurately by the simplified equations for both rigid and flexible ends in all cases except L10-n5-FR. A simplified form for the determination of lateral stiffness, i.e., Eq. 7, potentially can be used for more accurate estimates of the corresponding bracing demands.
4. Beams subjected to moment gradient can see substantially reduced stiffness demands in non-critical regions, but experience an increase in critical brace strength demands.
5. Very marginal or incidental lateral bracing restraint can be counted upon to supplement torsional bracing for a more economical bracing scheme. The addition of lateral bracing increases the beam strength slightly while significantly decreasing the strength requirements for the torsional bracing.
6. Brace forces typically remain relatively low until the limit load of the member is reached. Thus, frames that do not need full rigid-braced capacity or are not subject to reversals in inelastic deformation (such as may be present during seismic loading) may be able to be designed for less stringent brace force requirements.
7. Local buckling of the member appears to protect the braces (up to the system strength limit) by causing deformations inconsistent with the motion necessary to engage the brace. Local web or flange buckles do not contribute to the relative movement of the beam's brace points and thus, do not affect the strength or stiffness requirements. This seems especially true for sections controlled by flange local buckling. Upon reaching the system strength limit, the brace force demands generally increase rapidly in all cases.
8. The inclusion of inherent warping restraint by adjacent non-critical segments reduces the demands on the brace strength requirements as well as the required design stiffness.

9. The importance of the knuckle value as a *lower bound* stiffness requirement must be emphasized. Designers should choose bracing stiffness values that are sufficiently above the knuckle value so as to preclude slight variations in bracing stiffness causing drastic reductions in system capacity while maintaining an acceptable level of system economy. Since “rigid” bracing is unobtainable in practice, consideration must also be given to what percentage of rigid system strength is needed (90% was used extensively in this paper, see, e.g., Stanway et.al. 1992a & b).
10. When considering AISC’s Appendix 6 equations for rigid ends, the stiffness requirements consistently place the capacity of the section above the knuckle value. However, for flexible ends, the stiffness knuckle value is often larger than the Appendix 6 estimate. Thus, more work is needed to determine accurate (and safe) bracing requirement for beams with flexible ends.

8. Acknowledgements

The authors would like to thank MBMA for sponsoring this research and especially, the Steering Committee which has been instrumental in the development of this research project. However, the views expressed are solely those of the authors and may not represent the positions of the aforementioned organizations or individuals.

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