Beam-over-Column Bracing Requirements in Continuous Steel Frame Buildings

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ABSTRACT

Instability at beam-over-column connections in continuous steel frame building construction has resulted in numerous building failures. Despite these failures, and well-publicized stability research about this subject by others, many engineers still fail to appreciate the critical nature of these connections by omitting either beam web stiffeners, beam bottom flange lateral bracing, or both, at beam-over-column connections. One reason cited for this omission is that the beam was designed to preclude the use of either stiffeners or bracing at the beam-to-column intersection. It has been the authors' experience that justification for this omission typically results from either a failure to check, or a misapplication of, the web sidesway buckling provisions of AISC *Specification* Section J10.4.

This paper uses a finite element model to examine the web sidesway buckling provisions of AISC *Specification* Section J10.4 as they apply to beam-over-column connection design, and to examine the relative efficiencies of providing either beam web stiffeners or bracing at these connections. A parametric study was performed using a finite element frame model to compare the critical buckling loads of beams with beam web stiffeners, beam bottom flange bracing, or both, at beam-over-column intersections compared to unstiffened or laterally unbraced beams.

INTRODUCTION

Efficient design of continuous steel framing is typically characterized by balanced positive and negative moments, and adequate beam bracing to allow use of the full design flexural strength of the beam. In the design of continuous steel framing, it is necessary to consider both the stability of the beam in the clear span between columns, and the localized stability of the web of the beam under the concentrated forces that occur where the beams pass over the top of supporting columns. Due to the magnitude of these column forces, the provisions of AISC *Specification* Section J10.4 typically require the use of beam web stiffeners or beam flange bracing to preclude web sidesway buckling at these beam-to-column intersections.

A parametric study of a representative continuous steel frame demonstrates how the failure to check, or a misapplication of, the provisions of AISC *Specification* Section

J10.4 affects the performance of the continuous steel frame. The parametric study compares the critical buckling loads of beams with beam web stiffeners, beam bottom flange bracing, or both, at beam-over-column intersections compared to unstiffened or unbraced beams.

ANALYTICAL MODEL

The finite element frame model used for this study is a five-bay cantilever-suspended span frame, consisting of three bays where the beams cantilever beyond the columns at both ends, and two simple span beams suspended from the cantilevered ends between frames, see Figure 1. Framing consists of W10x49 columns spaced at 40 ft centers, and W21x44 beams. Cantilevered beams extend 6 ft beyond the columns at either end. Roof loads are delivered to the beam by open-web steel bar joists spaced at 6 ft 8 inch centers (that is, six equal spaces between columns).

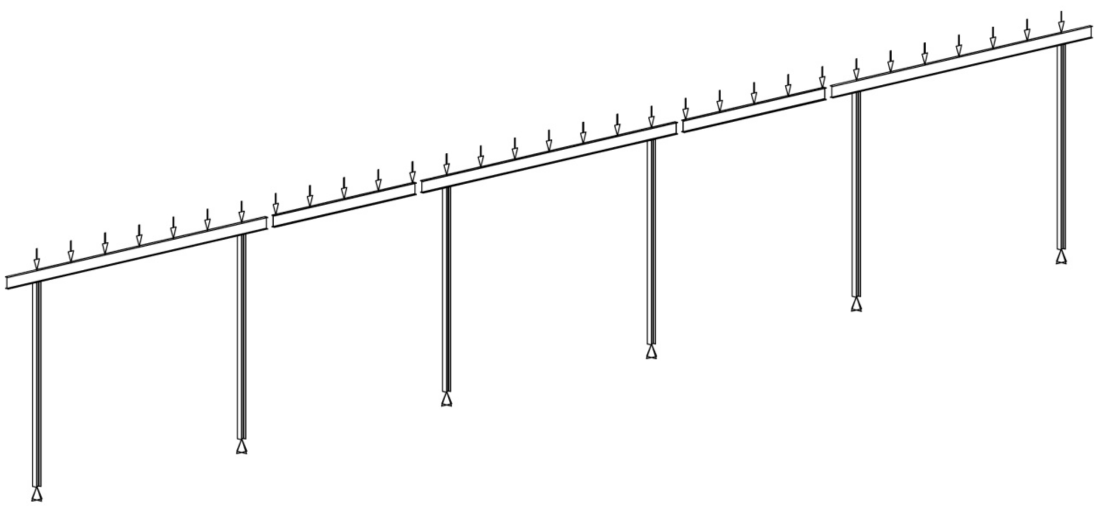


Figure 1. Frame model.

The analytical frame models were developed with the SAP2000 structural analysis program, authored by Computers and Structures, Inc., Berkeley, California, using Version 14.1.0. Flanges and webs of all members were input as finite element shells to allow direct calculation of beam buckling loads. Columns were considered to be pinned at the base and rigidly connected to the bottom flange of beams framing over the top of them. Suspended span beams were connected to the cantilevered beam ends using constraints at the web elements to simulate simple shear connection behavior (that is, no moment transfer). The top flange of the beam was braced against lateral translation at each joist location using infinitely rigid lateral braces.

Roof loads applied at the joist bearing locations included 20 psf for dead load and 16 psf for gravity live loads such as snow load. For a frame spacing of 20-feet and a joist spacing of 6.67-feet, the design joist end reaction is 4.8 kips using ASD load combinations. The resulting concentrated design reaction at the top of the column is 29 kips.

The required critical joist end reaction at each joist along the length of the beam using the safety factor for web sidesway buckling can be calculated as:

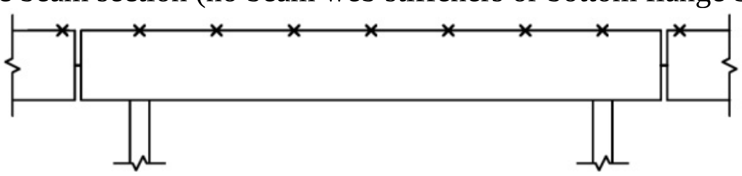
$$R_n = R_a\Omega = 4.8\ \text{kip} * 1.76 = 8.45\ \text{kips}$$

STUDY CASES

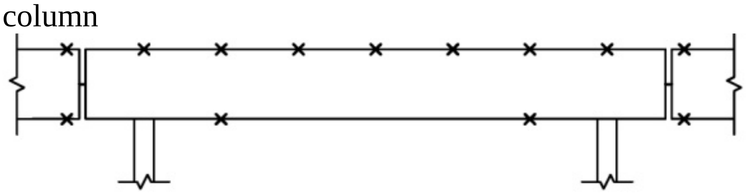
The analysis program determined the eigenvalue-eigenvector pair, or the buckling mode, for various framing cases by completing a linear buckling analysis to solve for the instability modes of the structure. Critical loads were then calculated from the buckling factor for admissible buckling modes. The critical loads were compared to reasonable design loads on the frame to evaluate safety, and to each other to evaluate the relative effectiveness of each case considered.

Studies examined 11 cases of beam web stiffeners at beam-over-column conditions and bracing. Beam web stiffeners used in the analysis were nominal 1/4 inch thick plates, and bracing was input as springs with a stiffness equal to the minimum required stiffness calculated according to the requirements of AISC *Specification* Appendix 6. Minimum required brace stiffness at the first joist along the length of the beam on either side of each column was calculated to be 24.0 kips/inch. Minimum required brace stiffness at other locations was calculated to be 12.0 kips/inch. Subsequent study conditions examined the effects of aligning beam web stiffeners with column flanges of varying thickness, and varying the bracing stiffness to something less than the minimum required stiffness. Individual framing conditions considered were as follows:

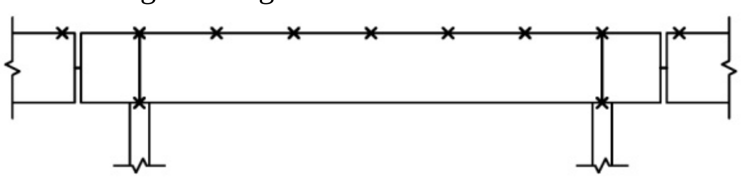
Case 1: Basic beam section (no beam web stiffeners or bottom flange bracing)



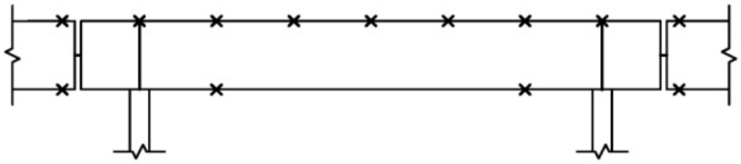
Case 2: Bottom flange bracing at first joist along the length of the beam on either side of each column



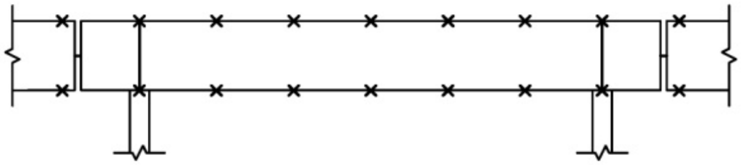
Case 3: Pair of full height 1/4 inch thick beam web stiffeners at column centerlines, bottom flange bracing at each column



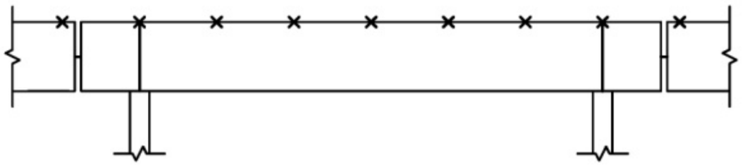
Case 4: Pair of full height 1/4 inch thick beam web stiffeners at column centerlines, bottom flange bracing at first joist along the length of the beam on either side of each column



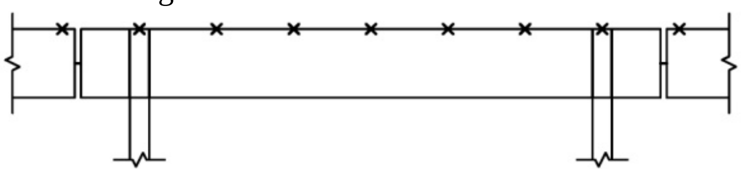
Case 5: Pair of full height 1/4 inch thick beam web stiffeners at column centerlines, bottom flange bracing at each joist



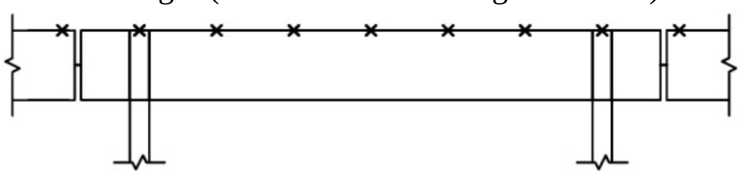
Case 6: Pair of full height 1/4 inch thick beam web stiffeners at column centerlines



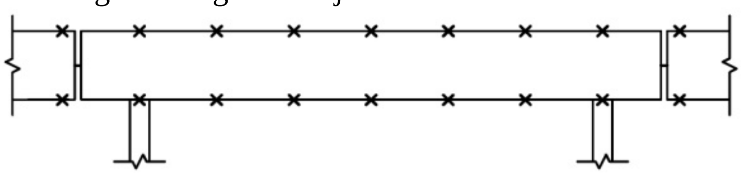
Case 7: Two pairs of full height 1/4 inch thick beam web stiffeners at each column aligned with column flanges



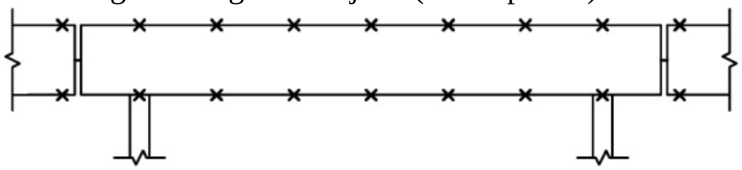
Case 8: Two pairs of full height 1/2 inch thick beam web stiffeners at each column aligned with column flanges (to match column flange thickness)



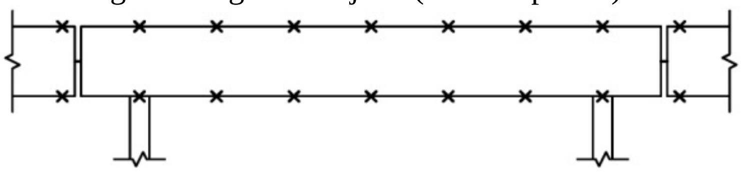
Case 9: Bottom flange bracing at each joist



Case 10: Bottom flange bracing at each joist (k=1 kip/inch)



Case 11: Bottom flange bracing at each joist (k=0.5 kip/inch)



ANALYSIS RESULTS

The linear buckling analysis results were reviewed for admissible buckling modes. Buckling modes due to frame sidesway and uplift loads were not considered for this paper. Analysis results are tabulated below:

Framing Conditions	Critical Load at Joist Reaction (kip)	Buckling Mode for Downward Roof Loads
Case 1	4.64	Web sidesway buckling
Case 2	14.02	Web sidesway buckling
Case 3	6.73	Beam lateral-torsional buckling
Case 4	25.45	Column buckling
Case 5	27.37	Column buckling
Case 6	6.73	Beam lateral-torsional buckling
Case 7	8.41	Beam lateral-torsional buckling
Case 8	9.81	Beam lateral-torsional buckling
Case 9	26.75	Column buckling
Case 10	11.08	Web sidesway buckling
Case 11	9.54	Web sidesway buckling

From the analysis results, the basic steel section without beam web stiffeners or bracing (Case 1) is inadequately designed. The calculated critical joist end reaction of 4.64 kips is approximately equal to the design service load of 4.80 kips, indicating that there is essentially no factor of safety, and is less than the required critical joist end reaction of 8.45 kips that is required to maintain adequate structural reliability. Similar results are shown for the case of providing beam web stiffeners alone (Case 6), and the combination of beam web stiffeners and bottom flange bracing at the column (Case 3). Critical loads for these two cases appear to be the same because buckling initiates due to inadequate lateral-torsional buckling strength in the unbraced portion of the beam just beyond the column. Providing two pairs of full height beam web stiffeners sized to match the column flange thickness aligned with the column flanges (Case 8) appears to increase the lateral-torsional buckling strength of the beam to a value above the required minimum nominal strength. For a similar case with nominal dimension 1/4-inch thick stiffeners (Case 7), the lateral-torsional buckling strength was approximately equal to the required minimum nominal strength. The critical load using the thicker stiffeners was approximately 15 percent greater than when using the thinner stiffeners.

Study results show that providing both beam web stiffeners and bottom flange bracing at each joist (Case 5), beam web stiffeners and bottom flange bracing at the first joist along the length of the beam on either side of each column (Case 4), or bottom flange bracing at each joist (Case 9) provide the greatest capacity. Column buckling occurs in these three cases at a critical load of at least 25 kips, which is at least five times the critical load of the basic beam section alone (Case 1). These results also indicate that for this design condition it is unnecessary to provide both bottom flange bracing at the column, and beam web stiffeners at the beam-over-column condition.

While we caution that applicable AISC *Specification* provisions require flange bracing to be a certain minimum stiffness, the study results for this design condition also demonstrate that providing bracing of any stiffness has a substantial impact on overall strength, and that adequate performance under design loads can be maintained even when bottom flange bracing stiffness is substantially lower than the required stiffness. The minimum required bracing stiffness in this case was 24.0 kips/inch. Decreasing the bracing stiffness to 1 kip/inch (Case 10) decreased the critical buckling load from 26.75 kips to 11.08 kips. Decreasing the bracing stiffness to 0.5 kips/inch (Case 11) decreased the critical buckling load from 26.75 kips to 9.54 kips. Both values are well above the required joist end reaction of 8.45 kips.

AISC *SPECIFICATION* SECTION J10.4

Special design considerations for webs and flanges under concentrated forces are specified in Section J10 of the AISC *Specification*. The applicable limits states include flange local bending, web local yielding, web crippling, web sidesway buckling, and web compression buckling. For the W21x44 beam analyzed, web sidesway buckling controls.

The web sideway buckling provisions are contained in Section J10.4. As explained in the commentary, the provisions of Section J10.4 apply only to bearing connections, and do not apply to moment connections. Two equations are provided to calculate the nominal strength R_n of the beam web: Equation J10-6 applies for conditions where the compression flange of the beam is restrained against rotation, and Equation J10-7 applies for conditions where the compression flange of the beam is not restrained against rotation. In the context of this section, the terms *restrained against rotation* and *not restrained against rotation* refer to ability of the beam flange to rotate in an absolute sense, not the ability of the beam flange to rotate relative to the opposite flange. For a beam-over-column condition, Equation J10-6 applies when the beam flange is essentially fixed to the top of the column, and Equation J10-7 applies when the beam flange is essentially pinned to the top of the column. Some engineering judgment is required to assess the degree of fixity that exists between the beam flange and the column.

The relevant provisions of Section J10.4 are reproduced below:

The nominal strength, R_n, for the limit state of web sidesway buckling shall be determined as follows:

1) *If the compression flange is restrained against rotation:*
   a) *For $(h/t_w)/(l/b_f) \leq 2.3$*

$$Rn = \frac{C_r t_w^3 t_f}{h^2}\left[1 + 0.4\left(\frac{h/t_w}{l/b_f}\right)^3\right]$$

   b) *For $(h/t_w)/(l/b_f) > 2.3$, the limit state of web sidesway buckling does not apply.*
2) *If the compression flange is not restrained against rotation:*
   a) *For $(h/t_w)/(l/b_f) \leq 1.7$*

$$Rn = \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{h/t_w}{l/b_f}\right)^3\right]$$

   b) *For $(h/t_w)/(l/b_f) > 1.7$, the limit state of web sidesway buckling does not apply.*

where, b_f = *flange width, in. (mm)*

C_r = *960,000 ksi (6.62×10^6 MPa) when $1.5M_a < M_y$ at the location of the force*
480,000 ksi (3.31×10^6 MPa) when $1.5M_a \geq M_y$ at the location of the force

h = *clear distance between flanges less the fillet or corner radius for rolled shapes, in. (mm)*

l = *largest laterally unbraced length along either flange at the point of load, in. (mm)*

t_f = *flange thickness, in. (mm)*

t_w = *web thickness, in. (mm)*

The choice of a proper laterally unbraced flange length l is critical to calculating the nominal strength R_n of the beam web. Assuming that points of moment inflection in the beam or laterally unbraced beam-over-column intersections are equivalent to physical lateral braces are frequent errors that yield incorrect results. A good rule of thumb is that if the compression flange of the beam is only held in place laterally by the stiffness of the beam web alone, the beam compression flange is not laterally braced.

The nominal strength R_n of the beam web was calculated using both Equations J10-6 and J10-7 for the design condition under consideration using various assumptions for the laterally unbraced flange length *l*. Laterally unbraced flange lengths were taken as the joist spacing (6.67 ft), the distance between points of inflection of the beam on either side of the column (13.34 ft), the distance between points of inflection in the mid-span portion of the beam (28 ft), the column spacing (40 ft), and the theoretical unbraced flange length if no bracing is provided (taken as *l*=100 ft in these equations). Results are summarized below:

***l* (ft)**	**R_n (kips) using J10-6**	**R_n (kips) using J10-7**
6.67	*Limit state does not apply.*	*Limit state does not apply.*
13.34	77	*Limit state does not apply.*
28	22	7
40	17	2
100	15	0

For comparison, the resulting concentrated design reaction at the top of the column is 29 kips. If the nominal strength of the beam web is less that the calculated concentrated design reaction, local lateral bracing at both flanges at the column or a pair of transverse stiffeners is required.

It can be seen from the results tabulated above that the only justification for omitting either beam web stiffeners or bottom flange bracing at the column for this design case is if the maximum laterally unbraced flange length is either the joist spacing (6.67 ft), or the distance between points of inflection of the beam on either side of the column (13.34 ft). As discussed by others (Yura, 2001) and as demonstrated in the analytical study above, however, the point of inflection does not constitute a physical brace point for the basic beam section and buckling initiates as sidesway buckling of the web. While this potential limit state is indicated by the sidesway buckling equations J10-6 and J10-7, use of an incorrect laterally unbraced flange length in these equations will lead the designer to conclude otherwise.

CONCLUSIONS

This paper used a finite element model to examine the web sidesway buckling provisions of AISC *Specification* Section J10.4 as they apply to beam-over-column connection design, and to examine the relative efficiencies of providing either beam web stiffeners or bracing at these connections. For the assumed design condition, analysis results demonstrated the following:

1. The critical load of the basic beam section (without bottom flange braces and without beam web stiffeners) was approximately one-fifth that of a beam with a combination of beam web stiffeners and bottom flange bracing at each joist intersection, a beam with a combination of beam web stiffeners and bottom flange bracing at the first joist along the length of the beam on either side of each

column, or a beam with bottom flange bracing at each joist location. There was no significant difference in capacity when both beam web stiffeners and bottom flange bracing were provided.

2. Calculation of the nominal strength R_n of the beam web according to Equations J10-6 and J10-7 demonstrates that the design condition considered by this paper requires beam web stiffeners or bottom flange bracing.
3. Incorrect assumptions concerning the laterally unbraced flange length l in Equations J10-6 and J10-7 can result in calculations indicating that beam web stiffeners or beam flange bracing are not required to prevent web sidesway buckling. These incorrect assumptions could include assuming that points of moment inflection in the beam, or laterally unbraced beam-over-column intersections, are equivalent to physical lateral braces. Analytical models presented in this paper demonstrate that web sidesway buckling can occur when these incorrect assumptions made.
4. Analytical models demonstrate that incorrect assumptions concerning the laterally unbraced flange length can also result in inadequate beam designs controlled by lateral-torsional buckling of the beam even when the web sidesway buckling provisions of Section J10.4 are met.

REFERENCES

American Institute of Steel Construction, Inc. (AISC) (2005). *Specification for Structural Steel Buildings*, ANSI/AISC 360-05 American Institute of Steel Construction, Chicago, IL.

Yura, J. A. (2001). "Fundamentals of Beam Bracing." *Engineering Journal*, AISC, Volume 38, Number 1, First Quarter, 11-26.