Lateral Bracing of Seismic Beams

by
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In Special Moment Resisting Frames (SMRFs) that are designed with the Strong Column, Weak Beam (SCWB) rationale to prevent the overall collapse of the structure in a seismic event, plastic hinges form in the beam-to-column connections as shown in Figures 1 and 2 The plastic hinge locations in SMRFs with SlottedWeb™ (SW) connections form at the face of the column (Figure 1) whereas the plastic hinge locations in SMRFs with the Reduced Beam Section (RBS) or SidePlate™ (SP) connections form at approximately the depth of the beam from the face of the column (Figure 2)..

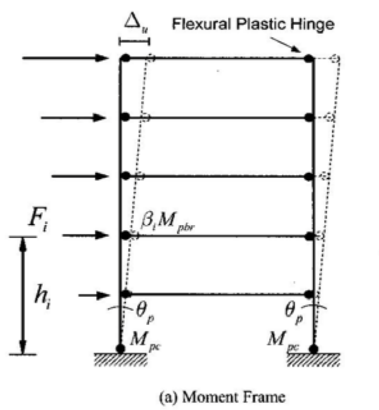


Figure 1. SMRF with SlottedWeb™ Connections

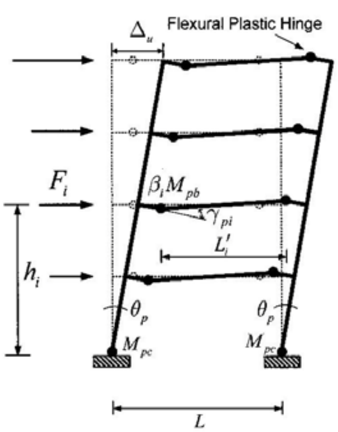
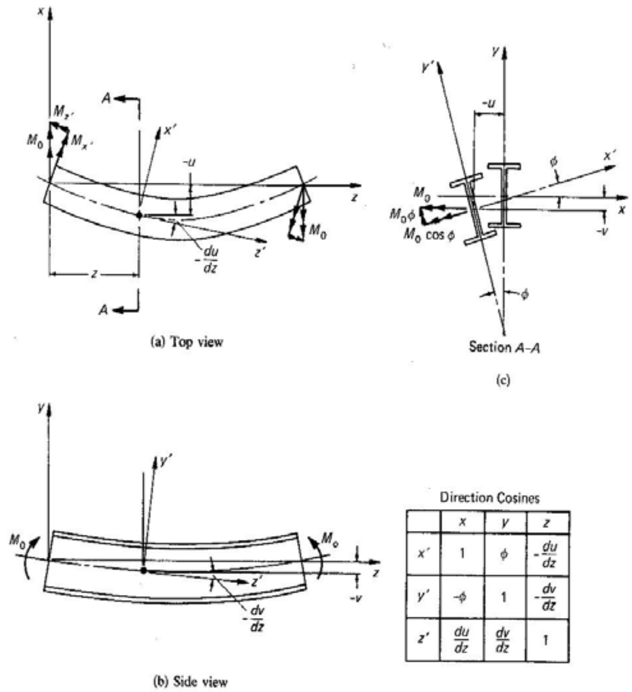


Figure 2. SMRF with RBS or SidePlate™ Connections

There are two modes of lateral torsional buckling that may occur in beams subject the seismic loadings. These modes are (1) inelastic global lateral torsional buckling of the beam as shown in Figure 3, and (2) inelastic local lateral torsional buckling in the beam-to-column connections of non-slotted beams as shown in the tested beams in Figure 4.



	x	y	z
x'	1	ϕ	$-\frac{du}{dz}$
y'	$-\phi$	1	$-\frac{dv}{dz}$
z'	$\frac{du}{dz}$	$\frac{dv}{dz}$	1

Figure 3. Global Lateral Torsional Buckling Mode of Beams
(From Salmon and Johnson Steel Structures: Design and Behavior)

Figure 4. RBS Lateral Torsional Buckling (University of Texas – Austin)

Maximum Unbraced Length for Beams vusing the SSDA SW Connection

The AISC *Specifications* global plastic design lateral bracing requirements apply to seismically loaded beams using the SSDA SW connection where local lateral torsional buckling does not occur at the connections. The maximum unbraced beam length required to prevent global lateral torsional buckling for beams in SMRFs using SSDA slotted web connections is given in the 13th Edition of the AISC *Specifications*, Appendix 1, page 16.1 – 154 in the section titled "Inelastic Analysis and Design." The plastic design maximum unbraced length equation is

$$L_{pd} = [0.12 + 0.076\,(M_2/M_1)]\,(E/F_y)\,r_y \qquad \text{Eq. (1)}$$

where:

M_2 = smaller moment at the end of unbraced length

M_1 = larger moment at the end of unbraced length

r_y = radius of gyration about the beam minor axis

and (M_2/M_1) is positive for moments which cause reverse curvature and negative for moments which cause single curvature.

In SMRFs under seismic loading with SW connections the plastic hinges and plastic moments in the beam occur at the face of the column with an inflection point at midspan as shown in the following moment diagram where the maximum unbraced length is shown equal to the span of the beam.

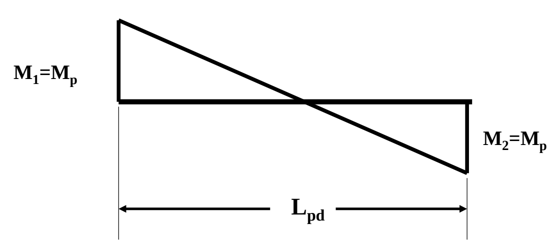


Figure 5. Moment Diagram for a Seismically Loaded Beam

For example, for a W36x150 beam with r_y = 2.47", F_y = 50 ksi, and E = 29,000 ksi the maximum unbraced clear span would be 280" or 23'- 4".

If the beam is laterally braced at midspan, one half the beam can be modeled as a cantilever beam laterally braced at the tip. In this case with $M_2 = 0$, Eq. (1) reduces to

$$L_{pd} = [0.12]\,(E/F_y)\,r_y \qquad \text{Eq. (2)}$$

where the maximum unbraced length is equal to one half the clear span of the beam.

For example, if a W36x150 beam with r_y = 2.47" ", F_y = 50 ksi, and E = 29,000 ksi is braced at midspan, L_{pd} in Eq. 2 is 14'-4" so that this beam can have a clear span of 28'-8" with only a single brace at midspan

Shown below are pp. 526 -527 "Design and Behavior of Steel Structures" by Salmon and Johnson (Harper and Row, 3rd Edition) which give the basis for Eq. (1) which is Eq. (9.6.2) in this text. As cited, this equation applies when "Plastic Moment (Mp) is Reached Along With Large Rotational Capacity" – i.e., for compact beam sections.
Tests performed at the University of Texas (J. Bansal, 1971) were the basis of Eq. 1.

9.6 LOAD AND RESISTANCE FACTOR DESIGN — I-SHAPED BEAMS SUBJECTED TO STRONG-AXIS BENDING

This section considers the full range of strength from laterally stable beams to situations where lateral-torsional buckling causes considerable strength reduction. Loading in the plane of the web is assumed.

The strength requirement according to LRFD-F2 may be stated

$$\phi_b M_n \geq M_u \qquad (9.6.1)$$

where ϕ_b = strength reduction factor for flexure = 0.90
M_n = nominal moment strength
M_u = factored service load moment (see Sec. 1.9)

Figure 9.6.1 shows the effect of laterally unbraced length L_b on the lateral-torsional buckling strength. Of course, local buckling may result in lower moment strength M_n if the plate element (flange or web) width/thickness ratios are too high.

Case 1: Plastic Moment is Reached ($M_n = M_p$) Along With Large Plastic Rotation Capacity ($R \geq 3$ in Fig. 9.3.2)

The section must be "compact" to prevent local buckling; that is, λ for the flange ($b_f/2t_f$) and for the web (h_c/t_w) must not exceed λ_p as discussed in Sec. 6.17 (values in Table 9.6.1) and lateral bracing must be provided such that

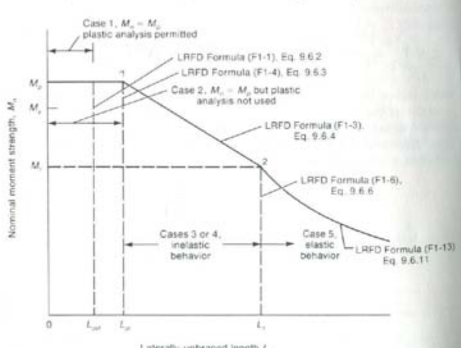


Figure 9.6.1 Nominal strength M_n of "compact" sections as affected by lateral-torsional buckling.

TABLE 9.6.1 SLENDERNESS RATIO LIMITS λ_p FOR "COMPACT" I-SHAPED BEAMS TO ACHIEVE PLASTIC MOMENT STRENGTH M_p (LRFD-B5.1)[a]

Yield Stress F_y (ksi)	Flange Local Buckling $\frac{b_f}{2t_f} = \frac{65}{\sqrt{F_y}}$	Web Local Buckling $\frac{h_c}{t_w} = \frac{640}{\sqrt{F_y}}$	Lateral-Torsional Buckling $\frac{L_p}{r_y} = \frac{300}{\sqrt{F_y}}$
36	10.8	107	50.0
42	10.0	98.8	46.3
45	9.7	95.4	44.7
50	9.2	90.5	42.4
55	8.8	86.3	40.5
60	8.4	82.6	38.7
65*	8.1	79.4	37.2

[a]Loading in the plane of the web; $M_p = F_y S_x$.
*The use of plastic moment strength M_p is restricted by LRFD-A5 and ASD-F1.1 to steels having $F_y \leq 65$ ksi.

the laterally unbraced length L_b does not exceed L_{pd}, where from Eq. 9.5.7,

$$L_{pd} = \frac{3600 + 2200 M_1/M_p}{F_y,\ \text{ksi}}\, r_y \qquad (9.6.2)$$

In this category, plastic analysis involving redistribution of moments as discussed in Chapter 10 may be used.

Case 2: Plastic Moment is Reached ($M_n = M_p$) But With Relatively Little Rotation Capacity ($R < 3$ in Fig. 9.3.2)

The section must be "compact" to prevent local buckling; that is, λ for the flange ($b_f/2t_f$) and for the web (h_c/t_w) must not exceed λ_p as discussed in Sec. 6.17 and lateral bracing must be provided such that λ (i.e., L_b/r_y) does not exceed λ_p. The three slenderness limits for various yield stresses are given in Table 9.6.1. This latter limit λ_p, from Eq. 9.5.5, means L_b may not exceed L_p when $C_b = 1.0$.

$$L_p = \frac{300}{\sqrt{F_y,\ \text{ksi}}}\, r_y \qquad (9.6.3)$$

Case 3: Lateral-Torsional Buckling of "Compact" Sections May Occur in the Inelastic Range ($M_p > M_n \geq M_r$)

This moment strength M_n is approximated as a linear relationship between points 1 (M_p at L_p) and 2 (M_r at L_r) on Fig. 9.6.1. Local buckling must be precluded. Since nearly all of the rolled sections are "compact", that is they

J. Bansal, "The Lateral Instability of Continuous Beams", AISI Report No. 3, American Iron and Steel Institute, New York, 1971.

Lateral Torsional Bracing Strength Requirements for Inelastic Steel Beams

A long used empirical procedure that has been accepted by the structural engineering profession to design the lateral bracing is given by Lay and Galambos, "Bracing Requirements for Inelastic Steel Beams", *Journal of the Structural Division*, ASCE, 92, ST2 (April 1966), pp. 207-228. This procedure recommends providing a brace that has strength equal to or greater than 2 percent of the compressive strength of the compression element being braced.

Commentary

In the 2005 AISC *Seismic Provisions*, Part 1 – Special Moment Frames, Section 9.8, Lateral Bracing of Beams, page 35, the maximum unbraced length is given as

$$L_b = 0.086\, r_y\, (E/F_y) \qquad \text{Eq. (3)}$$

The reason for the difference in the two maximum unbraced length criteria given in the AISC *Specifications* and the AISC *Seismic Provisions* is that under seismic loading, local lateral torsional buckling occurs near the ends in all beams that are not slotted. In beams using the SW connection the beam slots allow the beam flanges and the beam web to buckle independently which eliminates local lateral torsional buckling It is for this same reason that deep columns do not need to be laterally braced as stated explicitly in ANSI - ICC ER-5861 Section 2.3. None of the qualifying SSDA ATC-24 slotted web connection tests used lateral braces at the columns. Moreover, since the plastic hinges in beams that use the slotted web connection occur at the end of the shear plate, no lateral bracing of the beam is required at that location because it is essentially at the face of the column flange. In the following Attachment 1 (*Modern Steel Construction*, November 2000) the effects of lateral torsional buckling by RBS beams is discussed and a recommended bracing of the flange of a RBS connection is presented by AISC Regional Engineer Lanny J. Flynn.

Correspondence

Dear Editor:

I enjoyed the article, "Failure Analysis of a Column k-Area Fracture" written by John M. Barsom and J. V. Pellegrino, Jr., which appeared in the September 2000 issue of *Modern Steel Construction*. It was interesting to learn that the cause of the fracture in the test specimen was not influenced by a material defect or by the fracture toughness of the material in the k-area but rather that it was related to simply exceeding the tensile strength of the steel.

The article raises a good question. Why didn't this full-scale moment connection test provide the same robust performance exhibited in the numerous full-scale reduced beam section (RBS) tests performed by SAC and other independent researchers? To date well over 75 successful full-scale RBS tests have been performed by researchers throughout the world. Even though this RBS test provided a performance level that substantially exceeded the estimated moment connection rotational demands experienced during the Northridge earthquake, this particular test didn't perform as well as the numerous RBS connections previously tested.

In my opinion, the answer to this question can be found in the report "Cyclic Response of RBS Moment Connections: Weak-Axis Configuration and Deep Column Effects" written by Chia-Ming Uang, Chad Gilton and Brandon Chi. This report summarizes the research they conducted on deep column effects for the SAC Joint Venture Phase 2. The specimen discussed in the Barsom and Pellegrino article was tested as part of this research. This report states that torsion, primarily warping torsion, in the deep column was a cause of the high tensile stresses which helped initiate the column fracture.

It needs to be noted that there was no diaphragm or beam flange bracing at the hinge location of the test assembly. Previous full scale RBS connections tests using W14 columns exceeded SAC's performance criteria without aid of these additional brace elements. It is recognized that the RBS beam will tend to twist slightly due to lateral torsional buckling (LTB) as is shown in Figure 1. However when using heavy W14 or columns with similar torsional characteristics, LTB does not significantly reduce the overall performance of the connection.

Prior to LTB there is essentially no torsion applied to the column since the flange force is transmitted directly through the shear center as shown in Figure 2. When beam LTB occurs, a torque is produced about the column as is shown in Figure 3. Since E_{x-x} is much larger in deep column sections than in the shallow W14 sections the torque produced in the deep column sections by LTB will be larger. Also the torsional stiffness of the deeper column sections, which are typically selected in moment frames based on strong-axis stiffness, are less than an equivalent (and heavier) W14 section with respect to strong-axis stiffness. This obviously will result in a more critical torsional effect.

How do we, as structural engineers, typically handle torsion? First, we try to eliminate the torsion without compromising the economy and efficiency of the system. Or at least we should! One example of this as shown in Figure 4 is to provide a brace to eliminate or significantly reduce any torsion in the system. Even though additional cost is added to the structural frame by adding the brace, this cost is much less than the savings provided by using the deep column section. Typically, moment frame columns utilizing deep column sections are 40% to 50% lighter than frames using shallow column sections.

Another option is to design the structural sections so that they are strong and stiff enough to resist the torsion. In the aforementioned report, Uang et. al. suggest providing a column with a lower h/t_f^2 value for the deep column section than that provided by the column used in a successful full-scale test. In this formula, h is the depth of the column minus t_f and t_f is the column flange thickness. However, it should be noted that providing deep column sections with a lower h/t_f^2 value than that provided in a full scale test using W14 sections results in about the same column weight thus taking away one of the main benefits of using deep column sections.

A final alternative is to conduct the required test of the desired prototype design to demonstrate its ductility in accordance with AISC and SAC recommendations.

In summary, the key issue in this single isolated test among so many others was the torsional effect in deep columns, not any metallurgical or material concern. The use of deep columns is expected to be a focus of further discussions and/or studies by AISC and others.

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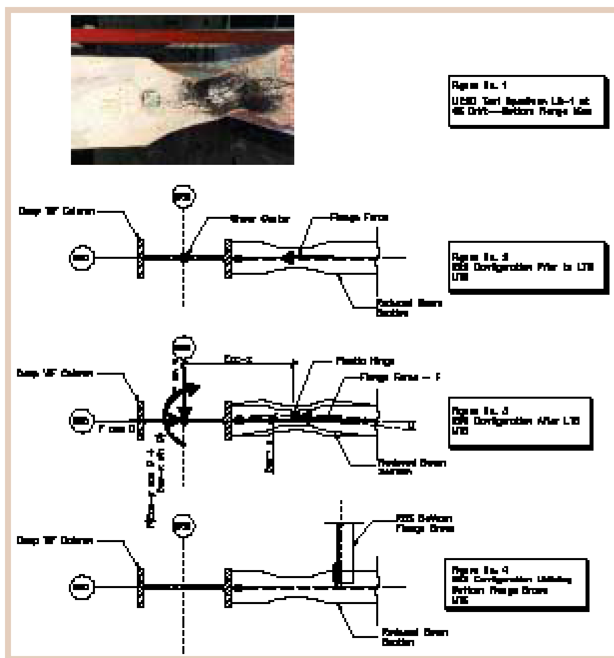

Attachment 1. RBS Lateral Torsional Buckling Effects
(*Modern Steel Construction*, November, 2000)

In the SSDA slotted web connection tests the column sizes ranged from W14 columns to W30 columns (as shown in Table 1) wherein the columns were laterally braced only at the top and bottom of the columns and not at the level of the beams. There were no column torsional or lateral instability problems in any of these tests that included the deep columns. Shown in Table 1, column 3, are the braced lengths of the cantilevered ATC-24 test beams measured from the face of the column to the center of the brace. Shown in Table 1, column 4, are the corresponding maximum unbraced length as given in the AISC *Specifications* for a cantilevered beam braced at the tip (Eq. 2) and the maximum unbraced length as given in the AISC *Seismic Provisions* (Eq. 3).

This table shows that the cantilever ATC-24 tests for the W33x141, W27x94, W24x94 and W36x170 beams all had unbraced lengths that exceeded the maximum unbraced lengths of the AISC *Seismic Provisions* (Eq. 3). None of these tests exhibited any lateral torsional buckling which demonstrates that the plastic design maximum unbraced length equation (Eq. (1) of the AISC *Specifications*) is appropriate for beams using the SSDA slotted web seismic connection.

AISC *Seismic Provisions*

These seismic lateral bracing requirements apply to beams where lateral torsional buckling would occur in the beams at the plastic hinge locations if the beam were not braced.

1. For local lateral torsional buckling at the connections both flanges must be laterally braced with the exception of where the beam supports a *concrete structural slab* that is connected between the protected zones with welded shear connectors.
2. For global lateral torsional buckling, the maximum brace spacing is

$$L_b = 0.0\ 86\ r_y\ E/F_y$$

- Refer to Section 9.8 of the AISC *Seismic Provisions.*
- Refer also to the 1993 LRFD *Specifications* Eq. C-F1-3, p. 6-199.
- This bracing length equation was developed by Lay and Galambos, “Inelastic Steel Beams Under Uniform Moment”, *Journal of the Structural Division,* ASCE, 91, ST6 (December, 1965), pp. 67-93.

3. The required strength of the lateral bracing provided adjacent to the plastic hinge given in Section 9.8 of the AISC *Seismic Provisions* shall be

$$P_u = 0.06\ M_u/h_o$$

- h_o is the distance between the flange centroids.
- The required stiffness is given by Eq. A-6-8 in Appendix 6 of the 13th Edition of the AISC *Specifications.*

- This strength requirement for local lateral buckling was generated from laboratory tests, e.g. as reported by Zekioglu, et al., "Designing After Northridge", *Modern Steel Construction*, March, 1997 where lateral brace forces at the RBS connections were found to be equal to 4% to 6% of the beam flange forces.

Table 1. Test Braced Length vs. Seismic Provisions Requirement

TEST Nos.	BEAM / COL.	Braced Length of ATC-24 Test Specimens	L_{pd} (AISC *Specifications*) / (AISC *Seismic Provisions*)
17 18	33x141 / 14x283 r_y = 2.43	156"= 13'	169"=14.1' / 121"=10.1'
19 20	27x94 / 14x176 r_y = 2.12	156"= 13'	148"=12.3' / 106"=8.84'
21 22	36x300 / 14x500 r_y = 3.83	156"=13'	266"=22' / 190"=15.9'
23 24	24x94 / 30x135 r_y = 1.98	148"= 12.33'	138"=11.5' / 98.9"=8.24'
25 26	36x170 / 30x235 r_y = 2.53	148"= 12.33'	176"=14.7' / 126"=10.5'
1	36x256 / 27x307 r_y = 2.65	66"= 5.5'	184"=15.4' / 132"=11.0'
2A 3A	36x393 / 14x550 (Grade 65) r_y = 3.90	72"= 6'	271"=22.6' / 194"=16.2'