
Seismic Provisions for Structural Steel Buildings (1997) Supplement No. 2

November 10, 2000

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Structural Steel Buildings*, dated February 15, 1999

Prepared by the American Institute of Steel Construction, Inc.
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approved by the AISC Board of Directors



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1. Part I, Glossary

Delete the definition of *Inelastic Rotation of Beam-to-Column Connection*.

Add the following definitions:

“Interstory Drift Angle. Interstory displacement divided by story height, radians.

Zipper Column. A vertical (or nearly vertical) strut connecting the brace-to-beam intersection of an inverted-V-braced frame at one level to the brace-to-beam intersection at another level. See Figure C-13.4 (b).”

Change definition of *Reduced Beam Section* as follows:

“A ~~ductile~~ reduction in cross-section . . . <remainder unchanged>”

2. Part I, Section 2

Change this section as follows:

...

American Society of Civil Engineers
ASCE 7-9598

American Society for Testing and Materials

ASTM A6-9698

ASTM A500-9399

ASTM A673-95

ASTM A36-9697a

ASTM A501-9399

ASTM A913-95a97

ASTM A53-9699b

ASTM A572-94e99

ASTM A992-98

ASTM A283-93a

ASTM A588-9497a

American Welding Society

AWS D1.1-96D1.1:2000

....

3. Part I, Section 6.1

Add ASTM A992 to the list of material specifications that are approved for use in the Seismic Force Resisting System.

4. Part I, Section 6.2

Change this section as follows:

“When required in these Provisions, the required strength of a connection or related member shall be determined from the Expected Yield Strength F_{ye} of the connected member, where

$$F_{ye} = R_y F_y \quad (6-1)$$

F_y is the specified minimum yield strength of the grade of steel to be used. For rolled shapes and bars, R_y shall be as shown in Table I-6-1 ~~taken as 1.5 for ASTM A36 and 1.3 for A572 Grade 42. For rolled shapes and bars of other grades of steel and for plates, R_y shall be taken as 1.1.~~ Other values of R_y are permitted to be used if the value of F_{ye} is determined by testing that is conducted in accordance with the requirements for the specified grade of steel.

TABLE I-6-1
 R_y Values for Different Member Types

Application	R_y
Plates and all other products	1.1
Hot-rolled structural shapes and bars	
ASTM A36	1.5
A572 Grade 42	1.3
All other grades	1.1
Hollow Structural Sections	
ASTM A500, A501, A618 and A847	1.3
Steel Pipe	
ASTM A53	1.4

When both the required strength and the design strength calculations are made for the same member or connecting element, it is permitted to apply R_y to F_y in the determination of the design strength.”

5. Part I, Section 6.3

Change this section as follows:

“When they are used as members in the Seismic Force Resisting System, ASTM A6 Groups 3, 4, and 5 shapes with flanges 1½ in. thick and thicker, ~~ASTM A6 Groups 4 and 5 shapes~~, and plates that are ≥ 1½ in. thick or thicker in built-up cross-sections shall have a minimum Charpy V-Notch (CVN) toughness of 20 ft-lbs at 70 degrees F, determined as specified in LRFD Specification Section A3.1c.”

6. Part I, Section 7.3b

Change language as follows:

“All ~~complete joint penetration groove~~ welds used in primary members and connections in the Seismic Force Resisting System shall be made with a filler metal that has a minimum ~~CVN~~ Charpy V-notch toughness of 20 ft-lbs at minus 20 degrees F, as determined by AWS classification or manufacturer certification. This requirement . . . <remainder unchanged>”

7. Part I, Section 7.3c

Change this section as follows:

“For members and connections that are part of the Seismic Force Resisting System, discontinuities located within a plastic hinging zone as defined in Section 7.4a, created by errors or by fabrication or erection operations, such as tack welds, erection aids, air-arc gouging, and thermal ~~flame~~ cutting, shall be repaired as required by the Engineer of Record.”

8. Part I, Section 7.4

Add the following new section:

“7.4. Other Connections

7.4a. Welded shear studs shall not be placed on beam flanges within the zones of expected plastic hinging. The length of a plastic hinging zone shall be defined as one-half of the depth of the beam on either side of the theoretical hinge point. Decking arc-spot welds as required to secure decking shall be permitted. Decking attachments that penetrate the beam flanges shall not be used in the hinging zone.

7.4b. Welded, bolted, screwed, or shot-in attachments for perimeter edge angles, exterior facades, partitions, duct work, piping, or other construction shall not be placed within the expected zone of plastic hinging, as defined in Section 7.4a, or of other members of the *Seismic Force Resisting System* which are expected to undergo plastic hinging. Outside the defined hinge area, calculations, based on the expected moment at the plastic hinge, shall be made to demonstrate the adequacy of the member net section when connectors which penetrate the member are used.

Exception: Welded shear studs and other connections are permitted where they have been included in the connection tests used to qualify the connection.”

9. Part I, Section 8.3

Add the following new section:

“8.3c. Column splices in Special Moment Frames shall be located as described in Section 8.3a, and shall have a nominal flexural strength that is at least equal to the nominal flexural strength of the smaller column. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds. Weld backing need not be removed unless required by the Engineer of Record. The design shear strength of the connection shall be equal to or greater than the required shear strength calculated using the expected yield moments that can be developed at each end of the column.

Exception: The design strength of the column splice need not exceed the required strength as determined by inelastic analyses in which appropriate stress concentration factors have been considered for the type of connection being used.”

10. Part I, Section 9.2a

Change the first sentence as follows:

“The design of all beam-to-column joints and connections used in the Seismic Force Resisting System shall be based upon qualifying

cyclic test results in accordance with Appendix S that demonstrate an interstory drift angle of at least 0.04 radians ~~inelastic rotation of at least 0.03 radians.~~”

Add the following sentence at the end of the last paragraph:

“Columns and connection elements with a tested yield strength that is more than 15 percent above or below F_{ye} shall not be used in qualification testing.”

11. **Part I, Section 9.3a**

Change this section as follows:

“Shear Strength: The required ~~thickness~~ shear strength R_u of the panel zone shall be determined in accordance with the method used in proportioning the panel zone of the tested connection ~~by applying Load Combinations 4-1 and 4-2 to the connected beam or beams in the plane of the frame at the column. R_u need not exceed the shear force determined from 0.8 times $\Sigma R_y M_p$ of the beams framing to the column flanges at the connection.~~ As a minimum, the required shear strength R_u of the panel zone shall be determined from the summation of the moments at the column faces as determined by projecting the expected moments at the plastic hinge points to the column faces. The design shear strength $\phi_v R_v$ of the panel zone shall be determined using $\phi_v = 0.75$ ~~1.0~~. When $P_u \leq 0.75 P_y$, ...” [Remainder unchanged].

12. **Part I, Section 9.4b**

Change this section as follows:

“Width-Thickness Ratios: Beams shall comply with λ_p in Table I-9-1. When the ratio in Equation 9-3 is less than or equal to 2.0 ~~1.25~~, columns shall comply with λ_p in Table I-9-1. Otherwise, columns shall comply with λ_p in LRFD Specification Table B5.1.”

13. **Part I, Section 9.6**

Change definition of ΣM^*_{pb} as follows:

“The sum of the moment(s) in the beam(s) at the intersection of the beam and column centerlines. ΣM^*_{pb} is determined by summing the projections of the expected beam flexural strength(s) . . . ”

14. **Part I, Section 9.7a.1**

Change Section 9.7a.1. as follows:

1. Column flanges at beam-to-column connections require lateral support only at the level of the top flanges of the beams when a column is shown to remain elastic outside of the panel-zone. under either of the following conditions: It shall be permitted to assume that the column remains elastic when the ratio calculated using Equation 9-3 is greater than 2.
 - ~~a. The ratio calculated using Equation 9-3 is greater than 1.25.~~
 - ~~b. The column remains elastic under Load Combination 4-1.~~

15. Part I, Sections 10 & 11

Replace Sections 10 and 11 with the following:

10. INTERMEDIATE MOMENT FRAMES (IMF)

10.1. Scope

Intermediate Moment Frames (IMF) are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the Design Earthquake. IMF shall meet the requirements in this Section.

10.2. Beam-to-Column Joints and Connections

10.2a. The design of all beam-to-column joints and connections used in the Seismic Force Resisting System shall be based upon qualifying cyclic test results in accordance with Appendix S that demonstrate an interstory drift angle of at least 0.02 radians. Qualifying cyclic test results shall consist of at least two cyclic tests and shall meet the requirements in Section 9.2a.

10.2b. Beam-to-column connection testing shall demonstrate a flexural strength, determined at the column face, that is at least equal to the nominal plastic moment of the beam M_p at the required inelastic rotation (see Appendix S), except as follows:

1. When beam local buckling rather than beam yielding limits the flexural strength of the beam, or when connections incorporating a Reduced Beam Section are used, the flexural strength shall be taken as $0.8M_p$ of the tested beam.
2. Connections that accommodate the required rotations within the connection elements and

maintain the design strength as specified are permitted, provided it can be demonstrated by rational analysis that any additional drift due to connection deformation can be accommodated by the building. Such rational analysis shall include the effects of overall frame stability including second order effects.

3. The required shear strength V_u of a beam-to-column connection shall be determined using the load combination $1.2D + 0.5L + 0.2S$ plus the shear resulting from the application of a moment of magnitude equal to $1.1R_yF_yZ$ in the opposite sense on each end of the beam. Alternatively, a lesser value of V_u is permitted if justified by rational analysis. The required shear strength need not exceed the shear resulting from Load Combination 4-1.

10.3. Continuity Plates

Continuity plates shall be provided to be consistent with the tested connection.

11. ORDINARY MOMENT FRAMES (OMF)

11.1. Scope

Ordinary Moment Frames (OMF) are expected to withstand minimal inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the Design Earthquake. OMF shall meet the requirements in this Section.

11.2. Beam-to-Column Joints and Connections

11.2a. Beam-to-column connections shall be made with welds and/or high-strength bolts. Connections are permitted to be FR or PR moment connections as follows:

1. FR moment connections that are part of the Seismic Force Resisting System shall be designed for a required flexural strength M_u that is at least equal to $1.1R_yM_p$ of the beam or girder or the maximum moment that can be delivered by the system, whichever is less. For connections with welded flange joints, weld backing and weld tabs shall be removed except that the top-flange backing is permitted to

remain in place provided that it is attached to the column flange by a continuous fillet weld on the edge below the complete-joint-penetration groove weld. After removal of weld backing, the surface shall be repaired and a contouring fillet weld added. Weld tabs shall be removed and the surface finished flush and smooth. Single-sided partial-joint-penetration groove welds and single-sided fillet welds shall not be used to resist tensile forces in the connections. Double-sided partial-joint-penetration groove welds and double-sided fillet welds that resist tensile forces in the connections shall be designed to resist a required force of $1.1R_yF_yA_g$ of the connected element or part.

2. PR moment connections are permitted when the following requirements are met:
 - a. Such connections shall provide for the design strength as specified in Section 11.2a.1.
 - b. The nominal flexural strength of the connection shall be no less than 50 percent of M_p of the connected beam or column, whichever is less.
 - c. The stiffness and strength of the PR moment connections shall be considered in the design, including the effect on overall frame stability.

11.2b. For FR moment connections, the required shear strength V_u of a beam-to-column connection shall be determined using the load combination $1.2D + 0.5L + 0.2S$ plus the shear resulting from the application of $1.1R_yF_yZ$ in the opposite sense on each end of the beam. Alternatively, a lesser value of V_u is permitted if justified by rational analysis. For PR moment connections, V_u shall be determined from the load combination above plus the shear resulting from the maximum end moment that the PR moment connections are capable of resisting.

11.3. Continuity Plates

When FR moment connections are made by means of welds of beam flanges or beam-flange connection plates directly to column flanges, continuity plates shall be provided to transmit beam flange forces to the column web or webs. Such plates shall have a minimum thickness equal to that of the beam flange or beam-flange connection plate. The welded joints of the continuity plates to the column flanges shall be made with either complete-joint-penetration groove welds, two-sided partial-joint-penetration groove welds combined with reinforcing fillet welds, or two-sided fillet welds and shall provide a design strength that is at least equal to the design strength of the contact area of the plate with the column flange. The welded joints of the continuity plates to the column web shall have a design shear strength that is at least equal to the lesser of the following:

- a. The sum of the design strengths at the connections of the continuity plate to the column flanges.
- b. The design shear strength of the contact area of the plate with the column web.
- c. The weld design strength that develops the design shear strength of the column panel-zone.
- d. The actual force transmitted by the stiffener.

Continuity plates are not required if tested connections demonstrate that the intended inelastic rotation can be achieved without their use.

16. Part I, Section 13.2d

Change language as follows:

“Width-thickness Ratios: Width-thickness ratios of stiffened and unstiffened compression elements of braces shall meet the compactness requirements in LRFD Specification Table B5.1 (i.e., $\lambda < \lambda_p$) and the following requirements:

1. ~~Braced shall be compact (i.e., $\lambda < \lambda_p$).~~ The width-thickness ratio of angles shall not exceed $52 / \sqrt{F_y}$.
2. I-shaped members and channels used as braces shall comply with λ_p in Table I-9-1.
3. Round HSS shall have... <remainder unchanged>”

17. Part I, Sections 14.2 through 14.5

Delete Sections 14.2 through 14.5 and replace with the following:

“14.2 Strength

The required strength of the members and connections, other than brace connections, in OCBFs shall be based upon Load Combinations 4-1 and 4-2. The design strength of brace connections shall equal or exceed the expected tensile strength of the brace, determined as $R_y F_y A_g$. Braces with Kl/r greater than $720 / \sqrt{F_y}$ shall not be used in V or inverted-V configurations.”

18. Part I, Section 16

Add the following sentence at the end of the Section:

“When welds from web doubler plates or continuity plates occur in the ‘k-area’ of rolled steel columns, the ‘k-area’ adjacent to the welds shall be inspected after fabrication, as required by the Engineer of Record, using approved nondestructive methods conforming to AWS D1.1.”

19. Appendix S, Section S2

Replace this section with the following:

“S2. SYMBOLS

The number in parenthesis after the definition of a symbol refers to the Section number in which the symbol is first used.

θ	Interstory drift angle (S6)
γ	Link rotation angle (S6)”

20. Appendix S, Section S3

Add the following definitions:

“*Complete Loading Cycle*. A cycle of rotation taken from zero force to zero force, including one positive and one negative peak.”

“*Interstory Drift Angle*. Interstory displacement divided by story height, radians.”

Replace the definition of *Inelastic Rotation* with the following:

“*Inelastic Rotation*. The permanent or plastic portion of the rotation angle between a beam and the column or between a Link and the column of the Test Specimen, measured in radians. The