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Seismic Provisions for Structural Steel Buildings

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Approved by the AISC Committee on Specifications



AMERICAN INSTITUTE OF STEEL CONSTRUCTION

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## **SYMBOLS**

The symbols listed below are to be used in addition to or replacements for those in the AISC Specification for Structural Steel Buildings. Where there is a duplication of the use of a symbol between the Provisions and the AISC Specification for Structural Steel Buildings, the symbol listed herein takes precedence. The section or table number in the right-hand column refers to where the symbol is first used.

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Symbol	<u>Definition</u>	Reference
$A_b$	Cross-sectional area of a horizontal	
	boundary element, in. <sup>2</sup> (mm <sup>2</sup> )	F5.5b
$A_c$	Cross-sectional area of a vertical boundary	
	element, in. <sup>2</sup> (mm <sup>2</sup> )	F5.5b
$A_{cw}$	Area of concrete between web plates, in. <sup>2</sup> (mm <sup>2</sup> )	H7.5b
$A_{ m f}$	Gross area of flange, in. <sup>2</sup> (mm <sup>2</sup> )	
$A_{g}$	Gross area, in. <sup>2</sup> (mm <sup>2</sup> )	E3.4a
$A_{\mathrm{lw}}$	Web area of link (excluding flanges), in. <sup>2</sup> (mm <sup>2</sup> )	
		F3.5b
$A_{s}$	Cross-sectional area of the structural steel core,	
	in. <sup>2</sup> (mm <sup>2</sup> )	
$A_{sc}$	Cross-sectional area of the yielding segment of	
	(mm <sup>2</sup> )	F4.5b
$A_{\rm sh}$	Minimum area of tie reinforcement, in. <sup>2</sup> (mm <sup>2</sup> ).	
$A_{sp}$	Horizontal area of stiffened steel plate in compos	
	wall, in. <sup>2</sup> (mm <sup>2</sup> )	
$A_{sr}$	Area of transverse reinforcement in coupling bea	
	in. <sup>2</sup> (mm <sup>2</sup> )	
$A_{sr}$	Area of longitudinal wall reinforcement provi	
	embedment length, L <sub>e</sub> , in. <sup>2</sup> (mm <sup>2</sup> )	
$A_{st}$	Horizontal cross-sectional area of the link stiffen	
	in. <sup>2</sup> (mm <sup>2</sup> )	F3.5b
$A_{sw}$	Area of steel web plates, in. <sup>2</sup> (mm <sup>2</sup> )	
$A_{tb}$	Area of transfer reinforcement required in each of	of the first and
	second regions attached to each of the top and bo	
	in. <sup>2</sup> (mm <sup>2</sup> )	H5.5c
$A_{\rm w}$	Area of steel beam web, in. <sup>2</sup> (mm <sup>2</sup> )	
$C_a$	Ratio of required strength to available strength	
$C_d$	Coefficient relating relative brace stiffness and c	
D	Dead load due to the weight of the structural eler	
	and permanent features on the building, kips (N)	
D	Outside diameter of round HSS, in. (mm)	Table D1.1
	2016 Seismic Provisions for Structural Steel Buildings	

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46	D	Diameter of the holes, in. (mm)F5.7a
47	E	Seismic load effect, kips (N)F1.4a
48	E	Modulus of elasticity of steel = 29,000 ksi
49		(200 000 MPa)Table D1.1
50	$E_{cl}$	Capacity-limited horizontal seismic load effect B2
51	$E_{mh}$	Horizontal seismic load effect, including the overstrength
52		factor, kips (N) or kip-in. (N-mm)B2
53	$F_{cr}$	Critical stress, ksi (MPa)
54	$F_{cre}$	Critical stress calculated from Specification Chapter E using
55		expected yield stress, ksi (MPa)F1.6a
56	$F_{v}$	Specified minimum yield stress, ksi (MPa). As used in the
57	,	Specification, "yield stress" denotes either the minimum
58		specified yield point (for those steels that have a yield point) or
59		the specified yield strength (for those steels that do not have a
60		yield point)
61	$F_{yb}$	Specified minimum yield stress of a beam, ksi (MPa) E3.4a
62	$F_{yc}$	Specified minimum yield stress of a column, ksi (MPa) .E3.4a
63	$F_{ysc}$	Specified minimum yield stress of the steel core, or actual yield
64	- ysc	stress of the steel core as determined from a coupon test, ksi
65		(MPa)F4.5b
66	$F_{ysr}$	Specified minimum yield stress of the ties, ksi (MPa) D1.4b
67	$F_{ysr}$	Specified minimum yield stress of transverse reinforcement, ksi
68	ı ysr	(MPa)
69	$F_{ysr}$	Specified minimum yield stress of transfer reinforcement, ksi
70	- ysr	(MPa)
71	$F_{yw}$	Specified minimum yield stress of web skin plates,
72	1 yw	ksi (MPa)H7.5b
73	$F_{\rm u}$	Specified minimum tensile strength, ksi (MPa)
74	H	Height of story, in. (mm)
75	$H_{\rm c}$	Clear height of the column between beam connections,
76	T <sub>C</sub>	including a structural slab, if present, in. (mm)F2.6d
77	I	Moment of inertia, in. 4 (mm <sup>4</sup> )
78	$I_b$	Moment of inertia of a horizontal boundary element taken
79	1ь	perpendicular to the direction of the web plate line, in. (mm <sup>4</sup> )
80		F5.4a
81	$I_c$	Moment of inertia of a vertical boundary element taken
82	1 <sub>C</sub>	perpendicular to the direction of the web plate line, in. (mm <sup>4</sup> )
83		F5.4a
84	Ţ	Moment of inertia about an axis in the plane of the EBF in. <sup>4</sup>
85	$I_y$	
0.6	T	(mm <sup>4</sup> )
86 87	l <sub>y</sub>	Effective length feeter E1.5h
	K	Effective length factor
88	L	Live load due to occupancy and moveable equipment, kips (N)
89	т	D1.4b
90	L	Length of column, in. (mm)
91	L	Length of truss span, in. (mm)E4.5c
92	L	Length of brace, in. (mm)F1.5b 2016 Seismic Provisions for Structural Steel Buildings
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93 94	L	Distance between vertical boundary element centerlines, in. (mm)
95	$L_b$	Length between points which are either braced against lateral
96	L <sub>0</sub>	displacement of compression flange or braced against twist of
97		the cross section, in. (mm)
98	$L_{c}$	Effective length = KL, in. (mm)
99	$L_{\rm cf}$	Clear length of beam, in. (mm)
100	$L_{cf}$	Clear distance between column flanges, in. (mm)F5.5b
101	L <sub>e</sub>	Embedment length of coupling beam, in. (mm)
102	L <sub>h</sub>	Distance between plastic hinge locations, as defined within the
103	Lh	test report or ANSI/AISC 358, in. (mm)
103	$L_{\rm s}$	Length of the special segment, in. (mm)
105	M <sub>a</sub>	Required flexural strength, using ASD load combinations, kip-
106	ı•ıa	in. (N-mm)
107	$M_{ m nc}$	Nominal flexural strength of a chord member of the special
107	<b>IVI</b> nc	segment, kip-in. (N-mm)
109	$M_{n,PR}$	Nominal flexural strength of PR connection at a rotation of 0.02
110	IVI <sub>n</sub> ,PR	rad, kip-in. (N-mm)
111	$M_p$	Plastic flexural strength, kip-in. (N-mm)
112	$\mathbf{M}_{p}$	Plastic flexural strength of a link, kip-in. (N-mm)F3.4a
113	$\mathbf{M}_{p}$	Plastic flexural strength of the steel, concrete-encased or
114	ı <b>vı</b> p	composite beam, kip-in. (N-mm)
115	$M_{p}$	Moment corresponding to plastic stress distribution over the
116	1 <b>v1</b> p	composite cross section, kip-in. (N-mm)
117	$M_{pc}$	Plastic flexural strength of the column, kip-in. (N-mm) . D2.5c
118	$M_{pcc}$	Plastic flexural strength of a composite column, kip-in. (N-mm)
119	1V1pcc	
120	$M_{p,exp}$	Expected flexural strength, kip-in. (N-mm)
121	$\mathbf{M}_{\mathrm{pr}}$	Probable maximum moment at the location of the plastic hinge,
122	1V1 <sub>pr</sub>	as determined in accordance with ANSI/AISC 358, or as
123		otherwise determined in a connection prequalification in
123		accordance with Section K1, or in a program of qualification
125		testing in accordance with Section K2, kip-in. (N-mm)E3.4a
126	$M_{\rm r}$	Required flexural strength, kip-in. (N-mm)
127	$M_{\rm u}$	Required flexural strength, using LRFD load combinations, kip-
128	1410	in. (N-mm)
129	$M_{ m v}$	Additional moment due to shear amplification from the location
130	$\mathbf{W}_{\mathbf{V}}$	of the plastic hinge to the column centerline, kip-in. (N-mm)
131		E3.4a
32	$ m M_{uv}$	Moment Additional moment due to shear amplification from the
$-33^{2}$	IVIUV	location of the plastic hinge to the column centerline, kip-in. (N-
134		mm)
135	$M^*_{pb}$	Projection of the expected flexural strength of the beam as
136	ıvı pb	defined in Section E3.4a, kip-in. (N-mm)
137	M* <sub>pc</sub>	Projection of the nominal flexural strength of the column as
138	IVI pc	defined in Section E3.4a, kip-in. (N-mm)
130		defined in Section L3.7a, kip-in. (11-11111)

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ment [LCA1]: Editorial modification (after : 4) to make this symbol consistent with Mv

## SYMBOLS-iv

	139	$M^*_{pcc}$	Moment in the column above or below the joint at the
ment [LCA2]: Editorial. Revised to make this of consistent with M*pc			intersection of the beam and column centerlines Projection of the
	41		nominal flexural strength of the composite or reinforced
	142		concrete column as defined in Section G3.4a, kip-in. (N-mm)
	143	3.6%	
	144	$M^*_{p,exp}$	Moment in the steel beam or concrete encased composite beam
ment [LCA3]: Editorial. Revised to make mbol consistent with M*pb	45		at the intersection of the beam and column centerlines Projection
· · · · · · · · · · · · · · · · · · ·	46		of the expected flexural strength of the steel or composite beam
	147		as defined in Section G3.4a, kip-in. (N-mm) G3.4a
	148	$N_{\rm r}$	Number of horizontal rows of perforationsF5.7a
]	149	$P_a$	Required axial strength using ASD load combinations, kips (N)
	150		Table D1.1
	151	$P_{ac}$	Required compressive strength using ASD load combinations,
	152		kips (N)
]	153	$P_b$	Axial design strength of wall at balanced condition, kips (N)
	154		H5.4
	155	$P_c$	Available axial strength, kips (N)E3.4a
	156	$P_n$	Nominal axial compressive strength, kips (N)
	157	$P_{nc}$	Nominal axial compressive strength of diagonal members of the
	158		special segment, kips (N)E4.5c
	159	$P_{nt}$	Nominal axial tensile strength of diagonal members of the
	160	- III	special segment, kips (N)
	161	$P_{\rm r}$	Required axial compressive strength, kips (N)E4.4d
	162	$P_{rc}$	Required axial strength, kips (N)
	163	$P_{\mathrm{u}}$	Required axial strength using LRFD load combinations, kips (N)
	164	ı u	Table D1.1
	165	$P_{uc}$	Required compressive strength using LRFD load combinations,
	166	1 uc	kips (N)
	167	D	
	168	$P_y$	Axial yield strength, kips (N)
		Pysc	Axial yield strength of steel core, kips (N)
	169	P <sub>ysc-max</sub>	Maximum specified axial yield strength of steel core, ksi
	170	, n	(MPa) F4.4d
	171	$P_{ysc-min}$	Minimum specified axial yield strength of steel core, ksi
	172		(MPa)F4.4d
	173	R	Seismic response modification coefficient
	174	R	Radius of the cut-out, in. (mm)F5.7b
	175	$R_c$	Factor to account for expected strength of concrete = 1.5 H5.5d
]	176	$R_n$	Nominal strength, kips (N)
	177	$\mathbf{R}_{t}$	Ratio of the expected tensile strength to the specified minimum
	178		tensile strength F <sub>u</sub>
	179	$R_y$	Ratio of the expected yield stress to the specified minimum yield
	180		stress, F <sub>y</sub>
	181	$R_{yr}$	Ratio of the expected yield stress of the transverse
	182	,	reinforcement material to the specified minimum yield
	183		stress
	184	$S_{diag}$	Shortest center-to-center distance between holes, in. (mm)
	185		F5.7a
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## SYMBOLS-v

186	$V_a$	Required shear strength using ASD load combinations, kips (N)
187		E1.6b
188	$V_{comp}$	Limiting expected shear strength of an encased composite
189		coupling beam, kips (N)
190	$V_n$	Nominal shear strength of link, kips (N)F3.3
191	$V_n$	Expected shear strength of a steel coupling beam, kips (N)
192		H4.5b
193	$V_{n,comp}$	Expected shear strength of an encased composite coupling beam,
194	, · · · · · · · · · · · · · · · · · · ·	kips (N)
195	$V_{ne}$	Expected vertical shear strength of the special segment, kips (N)
196	iic	E4.5c
197	$V_p$	Plastic shear strength of a link, kips (N)
198	$\overset{\cdot}{V_{r}}$	Required shear strength using LRFD or ASD load combinations,
199	*1	kips (N)
200	$V_{\rm u}$	Required shear strength using LRFD load combinations, kips (N)
201	<b>v</b> u	E1.6b
202	$V_{v}$	Nominal shear yield strength, kips (N)
202	•	Distance from the top of the steel beam to the top of concrete
	$Y_{con}$	
204	37	slab or encasement, in. (mm)
205	$Y_{PNA}$	Maximum distance from the maximum concrete compression
206	_	fiber to the plastic neutral axis, in. (mm)
207	Z	Plastic section modulus about the axis of bending, in. <sup>3</sup> (mm <sup>3</sup> )
208		D1.2a
209	$Z_{c}$	Plastic section modulus of the column about the axis of bending,
210		in. <sup>3</sup> (mm <sup>3</sup> )
211	$Z_{x}$	Plastic section modulus about x-axis, in. (mm <sup>3</sup> )E2.6g
212	a	Distance between connectors, in. (mm)F2.5b
213	b	Width of compression element as defined in Specification
214		Section B4.1, in. (mm)Table D1.1
215	b	Inside width of a box section, in. (mm)F3.5b
216	$b_{bf}$	Width of beam flange, in. (mm) E3.6f
217	$b_{\rm f}$	Width of flange, in. (mm)
218	$b_{\rm w}$	Thickness of wall pier, in. (mm)
219	$b_{\rm w}$	Width of wall, in. (mm)
220	$b_{wc}$	Width of concrete encasement, in. (mm)
221	d	Overall depth of beam, in. (mm)
222	d	Nominal bolt diameter, in. (mm)
223	d	Overall depth of link, in. (mm)
224	$d_c$	Effective depth of concrete encasement, in. (mm) H4.5b
225	$d_z$	$d$ –2 $t_f$ of the deeper beam at the connection, in. (mm)E3.6e
226	$d_z$ $d^*$	Distance between centroids of beam flanges or beam flange
227	u	connections to the face of the column, in. (mm) E3.6f
228	0	
	e e	Length of EBF link, in. (mm)
229	f'c	Specified compressive strength of concrete, ksi (MPa) D1.4b
230	g	Clear span of coupling beam, in. (mm)
231	h	Clear distance between flanges less the fillet or corner radius for

# SYMBOLS-vi

232		rolled shapes; and for built-up sections, the distance between
233		adjacent lines of fasteners or the clear distance between flanges
234		when welds are used; for tees, the overall depth; and for
235		rectangular HSS, the clear distance between the flanges less the
236		inside corner radius on each side, in. (mm) Table D1.1
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237	h	Distance between horizontal boundary element centerlines, in.
238		(mm) F5.4a
239	h	Overall depth of the boundary member in the plane of the wall,
240		in. (mm)
241	$h_{cc}$	Cross-sectional dimension of the confined core region in
242		composite columns measured center-to-center of the transverse
243		reinforcement, in. (mm)
244	$h_{o}$	Distance between flange centroids, in. (mm)
245	r	Governing radius of gyration, in. (mm)
246		Minimum radius of gyration, in: (mm)
247	$\mathbf{r}_{\mathbf{i}}$	F2.5b
248	$r_y$	Radius of gyration about y-axis, in. (mm) D1.2a
249	$\mathbf{r}_{\mathbf{y}}$	Radius of gyration of individual components about their weak
250		axis, in. (mm) E4.5e
251	S	Spacing of transverse reinforcement, in. (mm)
252	t	Thickness of element, in. (mm)
253	t	Thickness of column web or doubler plate, in. (mm)E3.6e
254	$t_{\rm bf}$	Thickness of beam flange, in. (mm)
255	$t_{ m eff}$	Effective web-plate thickness, in. (mm)
256	$t_{f}$	Thickness of flange, in. (mm)
257	$t_{lim}$	Limiting column flange thickness, in. (mm)E3.6f
258	t <sub>s</sub>	Thickness of steel web plate, in. (mm)
259	$t_{\rm w}$	Thickness of web, in. (mm)F3.5b
260	$t_{ m w}$	Web-plate thickness, in. (mm)
261	$t_{ m w}$	Thickness of wall, in. (mm) H7.4e
262	W <sub>min</sub>	Minimum of $w_1$ and $w_2$ , in. (mm)
263		Maximum spacing of tie bars in vertical and horizontal
264	$\mathbf{w}_1$	directions, in. (mm)
265	$\mathbf{w}_1$	Maximum spacing of tie bars or shear studs in vertical and
266		horizontal directions, in. (mm)
267	$\mathbf{w}_1, \mathbf{w}_2$	Vertical and horizontal spacing of tie bars, respectively,
268		in. (mm)
269	$W_{Z}$	Width of panel zone between column flanges, in. (mm)E3.6e
270	$\Delta$	Design story drift, in. (mm)F3.4a
271	$\Delta_{b}$	Deformation quantity used to control loading of test specimen
272	<b>⊸</b> 0	(total brace end rotation for the subassemblage test specimen;
273		total brace axial deformation for the brace test specimen), in.
274		(mm)
275	$\Delta_{ m bm}$	Value of deformation quantity, $\Delta_b$ , corresponding to the design
276		story drift, in. (mm)
277	$\Delta_{ m by}$	Value of deformation quantity, $\Delta_{b},$ at first yield of test specimen,

### SYMBOLS-vii

278		in. (mm)	K3.4c
279	$\Omega$	Safety factor	B3.2
280	$\Omega_{ m c}$	Safety factor for compression	Table D1.1
281	$\Omega_{ m o}$	System overstrength factor	B2
282	$\Omega_{ m v}$	Safety factor for shear strength of panel zone o	f beam-to-column
283		connections	
284	$\alpha_{\rm s}$	LRFD-ASD force level adjustment factor = 1	.0 for LRFD and
285		1.5 for ASD	D1.2a
286	α	Angle of diagonal members with the horizont	al, degreesE4.5c
287	α	Angle of web yielding, as measured relativ	e to the vertical,
288		degrees	F5.5b
289	α	Angle of the shortest center-to-center lines in	the opening array
290		to vertical, degrees	F5.7a
291	β	Compression strength adjustment factor	F4.2a
292	$\beta_1$	Factor relating depth of equivalent rectangula	r compressive
293		stress block to neutral axis depth, as defined i	
294	$\gamma_{total}$	Total link rotation angle	K2.4c
295	θ	Story drift angle, rad	K2.4b
296	$\lambda_{hd}, \lambda_{md}$	Limiting slenderness parameter for highly	and moderately
297		ductile compression elements, respectively	D1.1b
298	ф	Resistance factor	
299	фс	Resistance factor for compression	Table D1.1
300	$\phi_{ m v}$	Resistance factor for shear	E3.6e
301	$\frac{1}{\rho}$	Strength adjusted reinforcement ratio	H7.5b
302	ω	Strain hardening adjustment factor	F4.2a

350 Glossary

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The terms listed below are to be used in addition to those in the AISC Specification for Structural Steel Buildings. Some commonly used terms are repeated here for convenience.

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- Notes:
- (1) Terms designated with † are common AISI-AISC terms that are coordinated between the two standards developers.
- (2) Terms designated with \* are usually qualified by the type of load effect, for example, nominal tensile strength, available compressive strength, and design flexural strength.

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- Adjusted brace strength. Strength of a brace in a buckling-restrained braced frame at deformations corresponding to 2.0 times the design story drift.
- Adjusted link shear strength. Link shear strength including the material overstrength and strain hardening.
- Allowable strength\*†. Nominal strength divided by the safety factor,  $R_n/\Omega$ .
- Applicable building code†. Building code under which the structure is designed.
- ASD (allowable strength design)†. Method of proportioning structural components such that the allowable strength equals or exceeds the required strength of the component under the action of the ASD load combinations.
- ASD load combination†. Load combination in the applicable building code intended for allowable strength design (allowable stress design).
- Authority having jurisdiction (AHJ). Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of this Standard.
- Available strength\*†. Design strength or allowable strength, as applicable.
- Boundary member. Portion along wall or diaphragm edge strengthened with structural steel sections and/or longitudinal steel reinforcement and transverse reinforcement.
- Brace test specimen. A single buckling-restrained brace element used for laboratory testing intended to model the brace in the prototype.
- Braced frame†. An essentially vertical truss system that provides resistance to lateral forces and provides stability for the structural system.
- Buckling-restrained brace. A pre-fabricated, or manufactured, brace element consisting of a steel core and a buckling-restraining system as described in Section F4 and qualified by testing as required in Section K3.
- Buckling-restrained braced frame (BRBF). A diagonally braced frame employing buckling-restrained braces and meeting the requirements of Section F4.
- Buckling-restraining system. System of restraints that limits buckling of the steel core in BRBF. This system includes the casing surrounding the steel core and structural elements adjoining its connections. The

- buckling-restraining system is intended to permit the transverse expansion and longitudinal contraction of the steel core for deformations corresponding to 2.0 times the design story drift.
- Casing. Element that resists forces transverse to the axis of the diagonal brace thereby restraining buckling of the core. The casing requires a means of delivering this force to the remainder of the buckling-restraining system. The casing resists little or no force along the axis of the diagonal brace.

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- Capacity-limited seismic load. The capacity-limited horizontal seismic load effect,  $E_{cl}$ , determined in accordance with these Provisions, substituted for  $E_{mh}$ , and applied as prescribed by the load combinations in the applicable building code.
- Collector. Also known as drag strut; member that serves to transfer loads between diaphragms and the members of the vertical force-resisting elements of the seismic force-resisting system.
- Column base. Assemblage of structural shapes, plates, connectors, bolts and rods at the base of a column used to transmit forces between the steel superstructure and the foundation.
- Complete loading cycle. A cycle of rotation taken from zero force to zero force, including one positive and one negative peak.
- Composite beam. Structural steel beam in contact with and acting compositely with a reinforced concrete slab designed to act compositely for seismic forces.
- Composite brace. Concrete-encased structural steel section (rolled or built-up) or concrete-filled steel section used as a diagonal brace.
- Composite column. Concrete-encased structural steel section (rolled or built-up) or concrete-filled steel section used as a column.
- Composite eccentrically braced frame (C-EBF). Composite braced frame meeting the requirements of Section H3.
- Composite intermediate moment frame (C-IMF). Composite moment frame meeting the requirements of Section G2.
- Composite ordinary braced frame (C-OBF). Composite braced frame meeting the requirements of Section H1.
- Composite ordinary moment frame (C-OMF). Composite moment frame meeting the requirements of Section G1.
- Composite ordinary shear wall (C-OSW). Composite shear wall meeting the requirements of Section H4.
- Composite partially restrained moment frame (C-PRMF). Composite moment frame meeting the requirements of Section G4.

  Composite plate shear wall (C-PSW). Wall consisting of steel plate with
  - Composite plate shear wall (C-PSW). Wall consisting of steel plate with reinforced concrete encasement on one or both sides that provides out-of-plane stiffening to prevent buckling of the steel plate and meeting the requirements of Section H6.
- Composite shear wall. Steel plate wall panel composite with reinforced concrete wall panel or reinforced concrete wall that has steel or concrete-encased structural steel sections as boundary members.

- Composite slab. Reinforced concrete slab supported on and bonded to a formed steel deck that acts as a diaphragm to transfer load to and between elements of the seismic force resisting system.
- Composite special concentrically braced frame (C-SCBF). Composite braced frame meeting the requirements of Section H2.
- Composite special moment frame (C-SMF). Composite moment frame meeting the requirements of Section G3.
- Composite special shear wall (C-SSW). Composite shear wall meeting the requirements of Section H5.
- 451 Concrete-encased shapes. Structural steel sections encased in concrete.
  - Continuity plates. Column stiffeners at the top and bottom of the panel zone; also known as transverse stiffeners.
    - Coupling beam. Structural steel or composite beam connecting adjacent reinforced concrete wall elements so that they act together to resist lateral loads.
- 457 Demand critical weld. Weld so designated by these Provisions.

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- Design earthquake ground motion. The ground motion represented by the design response spectrum as specified in the applicable building code.
- Design story drift. Calculated story drift, including the effect of expected inelastic action, due to design level earthquake forces as determined by the applicable building code.
- Design strength\*†. Resistance factor multiplied by the nominal strength,  $\phi R_n$ .
  - Diagonal brace. Inclined structural member carrying primarily axial force in a braced frame.
  - Ductile limit state. Ductile limit states include member and connection yielding, bearing deformation at bolt holes, as well as buckling of members that conform to the seismic compactness limitations of Table D1.1. Rupture of a member or of a connection, or buckling of a connection element, is not a ductile limit state.
  - Eccentrically braced frame (EBF). Diagonally braced frame meeting the requirements of Section F3 that has at least one end of each diagonal brace connected to a beam with a defined eccentricity from another beam-to-brace connection or a beam-to-column connection.
  - Encased composite beam. Composite beam completely enclosed in reinforced concrete.
  - Encased composite column. Structural steel column completely encased in reinforced concrete.
- Engineer of record. Licensed professional responsible for sealing the contract documents.
- Exempted column. Column not meeting the requirements of Equation E3-1 for SMF.
- Expected tensile strength\*. Tensile strength of a member, equal to the specified minimum tensile strength, F<sub>u</sub>, multiplied by R<sub>t</sub>.
- Expected yield strength. Yield strength in tension of a member, equal to the expected yield stress multiplied by A<sub>g</sub>.

- Expected yield stress. Yield stress of the material, equal to the specified minimum yield stress, F<sub>v</sub>, multiplied by R<sub>v</sub>.
- Face bearing plates. Stiffeners attached to structural steel beams that are embedded in reinforced concrete walls or columns. The plates are located at the face of the reinforced concrete to provide confinement and to transfer loads to the concrete through direct bearing.
- 493 Filled composite column. HSS filled with structural concrete.
- Fully composite beam. Composite beam that has a sufficient number of steel headed stud anchors to develop the nominal plastic flexural strength of the composite section.
  - Highly ductile member. A member that meets the requirements for highly ductile members in Section D1.
  - Horizontal boundary element (HBE). A beam with a connection to one or more web plates in an SPSW.
  - Intermediate boundary element (IBE). A member, other than a beam or column, that provides resistance to web plate tension adjacent to an opening in an SPSW.
  - Intermediate moment frame (IMF). Moment frame system that meets the requirements of Section E2.
- 506 Inverted-V-braced frame. See V-braced frame.

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- k-area. The region of the web that extends from the tangent point of the web and the flange-web fillet (AISC "k" dimension) a distance of 1½ in. (38 mm) into the web beyond the k dimension.
- K-braced frame. A braced-frame configuration in which two or more braces connect to a column at a point other than a beam-to-column or strut-to-column connection.
- Link. In EBF, the segment of a beam that is located between the ends of the connections of two diagonal braces or between the end of a diagonal brace and a column. The length of the link is defined as the clear distance between the ends of two diagonal braces or between the diagonal brace and the column face.
- Link intermediate web stiffeners. Vertical web stiffeners placed within the link in EBF.
  - Link rotation angle. Inelastic angle between the link and the beam outside of the link when the total story drift is equal to the design story drift.
  - Link rotation angle, total. The relative displacement of one end of the link with respect to the other end (measured transverse to the longitudinal axis of the undeformed link), divided by the link length. The total link rotation angle includes both elastic and inelastic components of deformation of the link and the members attached to the link ends.
- Link design shear strength. Lesser of the available shear strength of the link based on the flexural or shear strength of the link member.
- Load-carrying reinforcement. Reinforcement in composite members designed and detailed to resist the required loads.
- Lowest anticipated service temperature (LAST). Lowest daily minimum temperature, or other suitable temperature, as established by the engineer of record.

- 534 LRFD (load and resistance factor design)†. Method of proportioning 535 structural components such that the design strength equals or exceeds 536 the required strength of the component under the action of the LRFD 537 load combinations.
- LRFD load combination†. Load combination in the applicable building code intended for strength design (load and resistance factor design).

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- Material test plate. A test specimen from which steel samples or weld metal samples are machined for subsequent testing to determine mechanical properties.
- Member brace. Member that provides stiffness and strength to control movement of another member out-of-the plane of the frame at the braced points.
- Moderately ductile member. A member that meets the requirements for moderately ductile members in Section D1.
- Multi-tiered braced frame (MTBF). A braced-frame configuration with two or more tiers of bracing between diaphragm levels or locations of out-of-plane bracing.
- Nominal strength\*†. Strength of a structure or component (without the resistance factor or safety factor applied) to resist load effects, as determined in accordance with the Specification.
- Ordinary cantilever column system (OCCS). A seismic force resisting-system in which the seismic forces are resisted by one or more columns that are cantilevered from the foundation or from the diaphragm level below and that meets the requirements of Section E5.
- Ordinary concentrically braced frame (OCBF). Diagonally braced frame meeting the requirements of Section F1 in which all members of the braced-frame system are subjected primarily to axial forces.
- Ordinary moment frame (OMF). Moment frame system that meets the requirements of Section E1.
- Overstrength factor,  $\Omega_0$ . Factor specified by the applicable building code in order to determine the overstrength seismic load, where required by these Provisions.
- Overstrength seismic load. The horizontal seismic load effect including overstrength determined using the overstrength factor,  $\Omega_0$ , and applied as prescribed by the load combinations in the applicable building code.
- Partially composite beam. Steel beam with a composite slab with a nominal flexural strength controlled by the strength of the steel headed stud anchors.
- Partially-restrained composite connection. Partially restrained (PR) connections as defined in the Specification that connect partially or fully composite beams to steel columns with flexural resistance provided by a force couple achieved with steel reinforcement in the slab and a steel seat angle or comparable connection at the bottom flange.
- Plastic hinge. Yielded zone that forms in a structural member when the plastic moment is attained. The member is assumed to rotate further as if hinged, except that such rotation is restrained by the plastic moment.

- Power-actuated fastener. Nail-like fastener driven by explosive powder, gas combustion, or compressed air or other gas to embed the fastener into structural steel.
- Prequalified connection. Connection that complies with the requirements of Section K1 or ANSI/AISC 358.
- Protected zone. Area of members or connections of members in which limitations apply to fabrication and attachments.

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- Prototype. The connection or diagonal brace that is to be used in the building (SMF, IMF, EBF, BRBF, C-IMF, C-SMF and C-PRMF).
- Provisions. Refers to this document, the AISC Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341).
- Quality assurance plan. Written description of qualifications, procedures, quality inspections, resources and records to be used to provide assurance that the structure complies with the engineer's quality requirements, specifications and contract documents.
- Reduced beam section. Reduction in cross section over a discrete length that promotes a zone of inelasticity in the member.
- Required strength\*. Forces, stresses and deformations acting on a structural component, determined by either structural analysis, for the LRFD or ASD load combinations, as appropriate, or as specified by the Specification and these Provisions.
- Resistance factor, φ†. Factor that accounts for unavoidable deviations of the nominal strength from the actual strength and for the manner and consequences of failure.
- Risk category. Classification assigned to a structure based on its use as specified by the applicable building code.
- Safety factor,  $\Omega^{\dagger}$ . Factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual load from the nominal load, uncertainties in the analysis that transforms the load into a load effect, and for the manner and consequences of failure.
- Seismic design category. A classification assigned to a structure based on its risk category and the severity of the design earthquake ground motion at the site.
- Seismic force-resisting system (SFRS). That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces prescribed in the applicable building code.
- Seismic response modification coefficient, R. Factor that reduces seismic load effects to strength level as specified by the applicable building code.
- Special cantilever column system (SCCS). A seismic force resisting-system in which the seismic forces are resisted by one or more columns that are cantilevered from the foundation or from the diaphragm level below and that meets the requirements of Section E6.
- Special concentrically braced frame (SCBF). Diagonally braced frame meeting the requirements of Section F2 in which all members of the braced-frame system are subjected primarily to axial forces.

- Special moment frame (SMF). Moment frame system that meets the requirements of Section E3.
- Special plate shear wall (SPSW). Plate shear wall system that meets the requirements of Section F5.
- Special truss moment frame (STMF). Truss moment frame system that meets the requirements of Section E4.
- Specification. Refers to the AISC Specification for Structural Steel Buildings (ANSI/AISC 360).
- Steel core. Axial-force-resisting element of a buckling-restrained brace. The steel core contains a yielding segment and connections to transfer its axial force to adjoining elements; it is permitted to also contain projections beyond the casing and transition segments between the projections and yielding segment.
  - Story drift angle. Interstory displacement divided by story height.

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- Strut. A horizontal member in a multi-tiered braced frame interconnecting brace connection points at columns.
- Subassemblage test specimen. The combination of members, connections and testing apparatus that replicate as closely as practical the boundary conditions, loading and deformations in the prototype.
- Test setup. The supporting fixtures, loading equipment and lateral bracing used to support and load the test specimen.
- Test specimen. A member, connection or subassemblage test specimen.
  - Test subassemblage. The combination of the test specimen and pertinent portions of the test setup.
  - V-braced frame. Concentrically braced frame (SCBF, OCBF, BRBF, C-OBF or C-SCBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span. Where the diagonal braces are below the beam, the system is also referred to as an inverted-V-braced frame.
- Vertical boundary element (VBE). A column with a connection to one or more web plates in an SPSW.
- X-braced frame. Concentrically braced frame (OCBF, SCBF, C-OBF or C-SCBF) in which a pair of diagonal braces crosses near the mid-length of the diagonal braces.
- Yield length ratio. In a buckling-restrained brace, the ratio of the length over which the core area is equal to A<sub>sc</sub>, to the length from intersection points of brace centerline and beam or column centerline at each end.

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701	ABBREVIATIONS
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703	The following abbreviations appear in the AISC Seismic Provisions for
704	Structural Steel Buildings. The abbreviations are written out where they first
705	appear within a Section.
706	••
707	ACI (American Concrete Institute)
708	AISC (American Institute of Steel Construction)
709	ANSI (American National Standards Institute)
710	ASCE (American Society of Civil Engineers)
711	ASD (allowable strength design)
712	AWS (American Welding Society)
713	BRBF (buckling-restrained braced frame)
714	C-EBF (composite eccentrically braced frame)
715	C-IMF (composite intermediate moment frame)
716	CJP (complete joint penetration)
717	C-OBF (composite ordinary braced frame)
718	C-OMF (composite ordinary moment frame)
719	C-OSW (composite ordinary shear wall)
720	C-PRMF (composite partially restrained moment frame)
721	CPRP (connection prequalification review panel)
722	C-PSW (composite plate shear wall)
723	C-SCBF (composite special concentrically braced frame)
724	C-SMF (composite special moment frame)
725	C-SSW (composite special shear wall)
726	CVN (Charpy V-notch)
727	EBF (eccentrically braced frame)
728	FCAW (flux cored arc welding)
729	FEMA (Federal Emergency Management Agency)
730	FR (fully restrained)
731	GMAW (gas metal arc welding)
732	HBE (horizontal boundary element)
733	HSS (hollow structural section)  IDE (intermediate boundary element)
734 735	IBE (intermediate boundary element) IMF (intermediate moment frame)
736	LAST (lowest anticipated service temperature)
737	LRFD (load and resistance factor design)
738	MT (magnetic particle testing)
739	MT-OCBF (multi-tiered ordinary concentrically braced frame)
740	MT-SCBF (multi-tiered special concentrically braced frame)
741	MT-BRBF (multi-tiered buckling-restrained braced frame)
742	NDT (nondestructive testing)
743	OCBF (ordinary concentrically braced frame)
744	OCCS (ordinary cantilever column system)
745	OMF (ordinary moment frame)
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#### **ABBREVIATIONS-2**

746	OVS	(oversized)
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- 747 PJP (partial joint penetration)
- 748 PR (partially restrained)
- 749 QA (quality assurance)
- 750 QC (quality control)
- 751 RBS (reduced beam section)
- 752 RCSC (Research Council on Structural Connections)
- 753 SCBF (special concentrically braced frame)
- 754 SCCS (special cantilever column system)
- 755 SDC (seismic design category)
- 756 SEI (Structural Engineering Institute)
- 757 SFRS (seismic force-resisting system)
- 758 SMAW (shielded metal arc welding)
- 759 SMF (special moment frame)
- 760 SPSPW (special perforated steel plate wall)
- 761 SPSW (special plate shear wall)
- 762 SRC (steel-reinforced concrete)
- 763 STMF (special truss moment frame)
- 764 UT (ultrasonic testing)
- 765 VBE (vertical boundary element)
- 766 WPQR (welder performance qualification records)
- 767 WPS (welding procedure specification)

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# CHAPTER A

#### GENERAL REQUIREMENTS

is chapter states the scope of the Provisions, summarizes referenced ecification, code and standard documents, and provides requirements for iterials and contract documents.

e chapter is organized as follows:

- A1. Scope
- A2. Referenced Specifications, Codes and Standards
- A3.
  - A4. Structural Design Drawings and Specifications

#### **SCOPE**

The Seismic Provisions for Structural Steel Buildings, hereafter referred to as these Provisions, shall govern the design, fabrication and erection of structural steel members and connections in the seismic force-resisting systems (SFRS), and splices and bases of columns in gravity framing systems of buildings and other structures with moment frames, braced frames and shear walls. Other structures are defined as those structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral force-resisting elements. These Provisions shall apply to the design of seismic force-resisting systems of structural steel or of structural steel acting compositely with reinforced concrete, unless specifically exempted by the applicable building code. Wherever these Provisions refer to the applicable building code and there is none, the loads, load combinations, system limitations, and general design requirements shall be those in ASCE/SEI 7.

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User Note: ASCE/SEI 7 (Table 12.2-1, Item H) specifically exempts structural steel systems in seismic design categories B and C from the requirements in these Provisions if they are designed in accordance with the AISC Specification for Structural Steel Buildings and the seismic loads are computed using a seismic response modification factor, R, of 3; composite systems are not covered by this exemption. These Provisions do not apply in seismic design category A.

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User Note: ASCE/SEI (Table 15.4-1) permits certain nonbuilding structures to be designed in accordance with the AISC Specification for

43	Structural Steel Buildings in lieu of the Provisions with an appropriately
44	reduced R factor.

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**User Note:** Composite seismic force-resisting systems include those systems with members of structural steel acting compositely with reinforced concrete, as well as systems in which structural steel members and reinforced concrete members act together to form a seismic force-resisting system.

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These Provisions shall be applied in conjunction with the AISC Specification for Structural Steel Buildings, hereafter referred to as the Specification. All requirements of the Specification are applicable unless otherwise stated in these Provisions. Members and connections of the SFRS shall satisfy the requirements of the applicable building code, the Specification, and these Provisions. The phrases "is permitted" and "are permitted" in these Provisions identify provisions that comply with the Specification, but are not mandatory.

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Building Code Requirements for Structural Concrete (ACI 318), as modified in these Provisions, shall be used for the design and construction of reinforced concrete components in composite construction. For the SFRS in composite construction incorporating reinforced concrete components designed in accordance with ACI 318, the requirements of Specification Section B3.1, Design for Strength Using Load and Resistance Factor Design, shall be used.

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# A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

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The documents referenced in these Provisions shall include those listed in Specification Section A2 with the following additions:

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# American Institute of Steel Construction (AISC)

- ANSI/AISC 360-16 Specification for Structural Steel Buildings
- 74 ANSI/AISC 358-16 Prequalified Connections for Special and
- 75 Intermediate Steel Moment Frames for Seismic Applications

# 76 American Welding Society (AWS)

- 77 AWS D1.8/D1.8M:2016 Structural Welding Code—Seismic Supplement
- 78 AWS B4.0:2007 Standard Methods for Mechanical Testing of Welds
- 79 (U.S. Customary Units)
- AWS B4.0M:2000 Standard Methods for Mechanical Testing of Welds
- 81 (Metric Customary Units)
- 82 AWS D1.4/D1.4M:2011 Structural Welding Code—Reinforcing Steel

#### A3. MATERIALS

#### A3.1. Material Specifications

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86 Structural steel used in the seismic force-resisting system (SFRS) shall satisfy the requirements of Specification Section A3.1, except as 87 modified in these Provisions. The specified minimum yield stress of 88 structural steel to be used for members in which inelastic behavior is 89 expected shall not exceed 50 ksi (345 MPa) for systems defined in 90 Chapters E, F, G and H, except that for systems defined in Sections E1, 91 92 F1, G1, H1 and H4 this limit shall not exceed 55 ksi (380 MPa). Either of 93 these specified minimum yield stress limits are permitted to be exceeded when the suitability of the material is determined by testing or other 94 95 rational criteria. 96 97 Exception: Specified minimum yield stress of structural steel shall not exceed 70 ksi (485 MPa) for columns in systems defined in Sections E3, 98 E4, G3, H1, H2 and H3, and for columns in all systems in Chapter F. 99 100 The structural steel used in the SFRS described in Chapters E, F, G and H shall meet one of the following ASTM Specifications: 101 102 103 Hot-rolled structural shapes 104 105 **ASTM A36/A36M** 106 ASTM A529/A529M 107 ASTM A572/A572M [Gr. 42 (290), 50 (345) or 55 (380)] 108 ASTM A588/A588M 109 ASTM A913/A913M [Gr. 50 (345), 60 (415), 65 (450) or 70 (485)] 110 ASTM A992/A992M 111 Hollow structural sections (HSS) 112 ASTM A500/A500M (Gr. B or C) 113 114 ASTM A501 ASTM A1085/A1085M 115 **ASTM A53/A53M** 116 117 118 Plates (c) 119 120 ASTM A36/A36M ASTM A529/A529M 121 122 ASTM A572/A572M [Gr. 42 (290), 50 (345) or 55 (380)] 123 ASTM A588/A588M 124 ASTM A1011/A1011M HSLAS Gr. 55 (380) 125 ASTM A1043/A1043M 126 127 (d) Bars 128 129 ASTM A36/A36M 130 ASTM A529/A529M 131 ASTM A572/A572M [Gr. 42 (290), 50 (345) or 55 (380)]

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ASTM A588/A588M

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The structural steel used for column base plates shall meet one of the preceding ASTM specifications or ASTM A283/A283M Grade D. Other steels and nonsteel materials in buckling-restrained braced frames are

ASTM A1011/A1011M HSLAS Gr. 55 (380)

permitted to be used subject to the requirements of Sections F4 and K3. **User Note**: This section only covers material properties for structural steel used in the SFRS and included in the definition of structural steel given in Section 2.1 of the AISC Code of Standard Practice. Other steel,

such as cables for permanent bracing, is not covered. Steel reinforcement used in components in composite SFRS is covered in Section A3.5.

# A3.2. Expected Material Strength

When required in these Provisions, the required strength of an element (a member or a connection of a member) shall be determined from the expected yield stress,  $R_yF_y$ , of the member or an adjoining member, as applicable, where  $F_y$  is the specified minimum yield stress of the steel to be used in the member and  $R_y$  is the ratio of the expected yield stress to the specified minimum yield stress,  $F_y$ , of that material.

 When required to determine the nominal strength,  $R_n$ , for limit states within the same member from which the required strength is determined, the expected yield stress,  $R_yF_y$ , and the expected tensile strength,  $R_tF_u$ , are permitted to be used in lieu of  $F_y$  and  $F_u$ , respectively, where  $F_u$  is the specified minimum tensile strength and  $R_t$  is the ratio of the expected tensile strength to the specified minimum tensile strength,  $F_u$ , of that material.

**User Note**: In several instances a member, or a connection limit state within that member, is required to be designed for forces corresponding to the expected strength of the member itself. Such cases include determination of the nominal strength,  $R_n$ , of the beam outside of the link in eccentrically braced frames, diagonal brace rupture limit states (block shear rupture and net section rupture in the diagonal brace in SCBF), etc. In such cases it is permitted to use the expected material strength in the determination of available member strength. For connecting elements and for other members, specified material strength should be used.

The values of  $R_y$  and  $R_t$  for various steel and steel reinforcement materials are given in Table A3.1. Other values of  $R_y$  and  $R_t$  are permitted if the values are determined by testing of specimens, similar in size and source to the materials to be used, conducted in accordance with the testing

requirements per the ASTM specifications for the specified grade of steel.



# TABLE A3.1 R<sub>y</sub> and R<sub>t</sub> Values for Steel and Steel Reinforcement Materials

Application	R <sub>y</sub>	Rt
Hot-rolled structural shapes and bars:		
ASTM A36/A36M	1.5	1.2
• ASTM A1043/A1043M Gr. 36 (250)	1.3	1.1
<ul> <li>ASTM A992/A992M</li> </ul>	1.1	1.1
• ASTM A572/A572M Gr. 50 (345) or 55 (380)	1.1	1.1
<ul> <li>ASTM A913/A913M Gr. 50 (345), 60 (415),</li> <li>65 (450), or 70 (485)</li> </ul>	1.1	1.1
• ASTM A588/A588M	1.1	1.1
• ASTM A1043/A1043M Gr. 50 (345)	1.2	1.1
• ASTM A529 Gr. 50 (345)	1.2	1.2
• ASTM A529 Gr. 55 (380)	1.1	1.2
Hollow structural sections (HSS):		
<ul> <li>ASTM A500/A500M Gr. B</li> </ul>	1.4	1.3
• ASTM A500/A500M Gr. C	1.3	1.2
• ASTM A501	1.4	1.3
• ASTM A53/A53M	1.6	1.2
• ASTM A1085/A1085M	1.25	1.15
Plates, Strips and Sheets:		
• ASTM A36/A36M	1.3	1.2
• ASTM A1043/A1043M Gr. 36 (250)	1.3	1.1
• ASTM A1011/A1011M HSLAS Gr. 55 (380)	1.1	1.1
• ASTM A572/A572M Gr. 42 (290)	1.3	1.0
• ASTM A572/A572M Gr. 50 (345), Gr. 55 (380)	1.1	1.2
• ASTM A588/A588M	1.1	1.2
• ASTM A1043/A1043M Gr. 50 (345)	1.2	1.1
Steel Reinforcement:		

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•	ASTM A615/A615M Gr. 60 (420)	1.2	1.2
•	ASTM A615/A615M Gr. 75 (520) and Gr. 80 (550)	1.1	1.2
•	ASTM A706/A706M Gr. 60 (420) and Gr. 80 (550)	1.2	1.2

**User Note:** The expected compressive strength of concrete may be estimated using values from Seismic Rehabilitation of Existing Buildings (ASCE/SEI 41-13).

# A3.3. Heavy Sections

For structural steel in the SFRS, in addition to the requirements of Specification Section A3.1c, hot rolled shapes with flange thickness equal to or greater than 1½ in. (38 mm) shall have a minimum Charpy V-notch toughness of 20 ft-lb (27 J) at 70°F (21°C), tested in the alternate core location as described in ASTM A6 Supplementary Requirement S30. Plates with thickness equal to or greater than 2 in. (50 mm) shall have a minimum Charpy V-notch toughness of 20 ft-lb (27 J) at 70°F (21°C), measured at any location permitted by ASTM A673, Frequency P, where the plate is used for the following:

- (a) Members built up from plate
  - (b) Connection plates where inelastic strain under seismic loading is expected
  - (c) The steel core of buckling-restrained braces

# A3.4. Consumables for Welding

# A3.4a. Seismic Force-Resisting System Welds

All welds used in members and connections in the SFRS shall be made with filler metals meeting the requirements specified in clause 6.3 of Structural Welding Code—Seismic Supplement (AWS D1.8/D1.8M), hereafter referred to as AWS D1.8/D1.8M.

**User Note:** AWS D1.8/D1.8M clauses 6.3.5, 6.3.6, 6.3.7 and 6.3.8 apply only to demand critical welds.

#### A3.4b. Demand Critical Welds

Welds designated as demand critical shall be made with filler metals meeting the requirements specified in AWS D1.8/D1.8M clause 6.3.

User Note: AWS D1.8/D1.8M requires that all seismic force-resisting system welds are to be made with filler metals classified using AWS A5 standards that achieve the following mechanical properties:

Filler Metal Classification Properties for Seismic Force-Resisting System Welds				
	Classification			
Property	70 ksi (480 MPa)	80 ksi (550 MPa)	90 ksi (620 MPa)	
Yield Strength, ksi (MPa)	58 (400) min.	68 (470) min.	78 (540) min.	
Tensile Strength, ksi (MPa)	70 (480) min.	80 (550) min.	90 (620) min.	
Elongation, %	22 min.	19 min.	17 min.	
CVN Toughness, ft-lb (J)	20 (27) min. @ 0 °F			
<sup>a</sup> Filler metals classified as meeting 20 ft-lbf (27 J) min. at a				

temperature lower than 0 °F (-18°C) also meet this requirement.

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In addition to the above requirements, AWS D1.8/D1.8M requires, unless otherwise exempted from testing, that all demand critical welds are to be made with filler metals receiving Heat Input Envelope Testing that achieve the following mechanical properties in the weld metal:

Mechanical Properties for Demand Critical Welds				
	Classification			
Property	70 ksi (480 MPa)	80 ksi (550 MPa)	90 ksi (620 MPa)	
Yield Strength, ksi (MPa)	58 (400) min.	68 (470) min.	78 (540) min.	
Tensile Strength, ksi (MPa)	70 (480) min.	80 (550) min.	90 (620) min.	
Elongation (%)	22 min.	19 min.	17 min.	
CVN Toughness, ft-lb (J)			40 (54) min. @ 50°F (10°C)	

<sup>&</sup>lt;sup>b</sup> For LAST of +50°F (+10°C). For LAST less than + 50°F (+10°C), see AWS D1.8/D1.8M clause 6.3.6.

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# A3.5. Concrete and Steel Reinforcement

<sup>&</sup>lt;sup>c</sup> Tests conducted in accordance with AWS D1.8/D1.8M Annex A meeting 40 ft-lb (54 J) min. at a temperature lower than +70°F (+20°C) also meet this requirement.

229 230 231 232 233 234		compo H5, H Concr compo	rete and steel reinforcement used in composite components in osite intermediate or special SFRS of Sections G2, G3, G4, H2, H3, I6 and H7 shall satisfy the requirements of ACI 318 Chapter 18. rete and steel reinforcement used in composite components in osite ordinary SFRS of Sections G1, H1 and H4 shall satisfy the rements of ACI 318 Section 18.2.1.4.
235	<b>A4.</b>	STRU	UCTURAL DESIGN DRAWINGS AND SPECIFICATIONS
236 237 238 239	A4.1.		ural design drawings and specifications shall indicate the work to
239 240 241		Code	formed, and include items required by the Specification, the AISC of Standard Practice for Steel Buildings and Bridges, the applicable ng code, and the following, as applicable:
242		(a)	Designation of the SFRS
243 244		(b)	Identification of the members and connections that are part of the SFRS
245		(c)	Locations and dimensions of protected zones
246 247		(d)	Connection details between concrete floor diaphragms and the structural steel elements of the SFRS
248 249		(e)	Shop drawing and erection drawing requirements not addressed in Section I1
250	A4.2.	Steel	Construction
<ul><li>251</li><li>252</li><li>253</li><li>254</li></ul>		drawi	dition to the requirements of Section A4.1, structural designings and specifications for steel construction shall indicate the ving items, as applicable:
255		(a)	Configuration of the connections
256		(b)	Connection material specifications and sizes
257		(c)	Locations of demand critical welds
258 259		(d)	Locations where gusset plates are to be detailed to accommodate inelastic rotation
260 261		(e)	Locations of connection plates requiring Charpy V-notch toughness in accordance with Section A3.3(b)
262 263 264		(f)	Lowest anticipated service temperature of the steel structure, if the structure is not enclosed and maintained at a temperature of 50°F (10°C) or higher

265		(g)	Locations where weld backing is required to be removed
266 267		(h)	Locations where fillet welds are required when weld backing is permitted to remain
268 269		(i)	Locations where fillet welds are required to reinforce groove welds or to improve connection geometry
270		(j)	Locations where weld tabs are required to be removed
271		(k)	Splice locations where tapered transitions are required
<ul><li>272</li><li>273</li></ul>		(1)	The shape of weld access holes, if a shape other than those provided for in the Specification is required
274 275 276 277		(m)	Joints or groups of joints in which a specific assembly order, welding sequence, welding technique or other special precautions where such items are designated to be submitted to the engineer of record
<ul><li>278</li><li>279</li></ul>	A4.3.	Comp	osite Construction
280 281 282 283 284		Section concre specifi	ition to the requirements of Section A4.1 and the requirements of n A4.2, as applicable, for the steel components of reinforced te or composite elements, structural design drawings and cations for composite construction shall indicate the following as applicable:
281 282 283		Section concre specifi	n A4.2, as applicable, for the steel components of reinforced ete or composite elements, structural design drawings and ecations for composite construction shall indicate the following
281 282 283 284 285 286		Section concress specifications,	n A4.2, as applicable, for the steel components of reinforced the or composite elements, structural design drawings and cations for composite construction shall indicate the following as applicable:  Bar placement, cutoffs, lap and mechanical splices, hooks and mechanical anchorage, placement of ties, and other transverse
281 282 283 284 285 286 287 288		Section concress specifications, (a)	n A4.2, as applicable, for the steel components of reinforced te or composite elements, structural design drawings and cations for composite construction shall indicate the following as applicable:  Bar placement, cutoffs, lap and mechanical splices, hooks and mechanical anchorage, placement of ties, and other transverse reinforcement  Requirements for dimensional changes resulting from

1200	CHAPTER B		
1201	GENERAL DESIGN REQUIREMENTS		
1202 1203	This chapter addresses the general requirements for the seismic design of steel structures that are applicable to all chapters of the Provisions.		
1204 1205 1206 1207 1208 1209 1210	This c	hapter is organized as follows:  B1. General Seismic Design Requirements B2. Loads and Load Combinations B3. Design Basis B4. System Type	
1211	<b>B1.</b>	GENERAL SEISMIC DESIGN REQUIREMENTS	
1212 1213 1214		The required strength and other seismic design requirements for seismic design categories, risk categories, and the limitations on height and irregularity shall be as specified in the applicable building code.	
1215 1216		The design story drift and the limitations on story drift shall be determined as required in the applicable building code.	
1217	<b>B2.</b>	LOADS AND LOAD COMBINATIONS	
1218 1219 1220 1221 1222		Where the required strength defined in these Provisions refers to the capacity-limited seismic load, the capacity-limited horizontal seismic load effect, $E_{cl}$ , shall be determined in accordance with these Provisions, substituted for $E_{mh}$ , and applied as prescribed by the load combinations in the applicable building code.	
1223 1224 1225 1226 1227 1228		Where the required strength defined in these Provisions refers to the overstrength seismic load, the horizontal seismic load effect including overstrength shall be determined using the overstrength factor, $\Omega_{\rm o}$ , and applied as prescribed by the load combinations in the applicable building code. Where the required strength refers to the overstrength seismic load, it is permitted to use the capacity-limited seismic load instead.	
1229 1230 1231 1232 1233 1234 1235		<b>User Note</b> : The seismic load effect including overstrength is defined in ASCE/SEI 7, Section 12.4.3. In ASCE/SEI 7 Section 12.4.3.1, the horizontal seismic load effect, $E_{mh}$ , is determined using Equation 12.4-7: $E_{mh} = \Omega_o Q_E$ . There is a cap on the value of $E_{mh}$ : it need not be taken larger than $E_{cl}$ . Thus, in effect, where these Provisions refer to overstrength seismic load, $E_{mh}$ is permitted to be based upon the overstrength factor, $Q_{ch}$ or $E_{ch}$ . However, where capacity-limited seismic load is required, it is	

1236 1237		intended that $E_{cl}$ replace $E_{mh}$ as specified in ASCE/SEI 7 Section 12.4.3.2 and use of ASCE/SEI 7 Equation 12.4-7 is not permitted.	
1238 1239 1240 1241 1242		In composite construction, incorporating reinforced concrete components designed in accordance with the requirements of ACI 318, the requirements of Specification Section B3.1, Design for Strength Using Load and Resistance Factor Design, shall be used for the seismic force-resisting system (SFRS).	
1243	В3.	DESIGN BASIS	
1244	B3.1.	Required Strength	
1245 1246		The required strength of structural members and connections shall be the greater of:	
1247 1248 1249		(a) The required strength as determined by structural analysis for the applicable load combinations, as stipulated in the applicable building code, and in Chapter C	
1250		(b) The required strength given in Chapters D, E, F, G and H	
1251	B3.2.	Available Strength	
1252 1253 1254 1255 1256 1257 1258		The available strength is stipulated as the design strength, $\phi R_n$ , for design in accordance with the provisions for load and resistance factor design (LRFD) and the allowable strength, $R_n/\Omega$ , for design in accordance with the provisions for allowable strength design (ASD). The available strength of systems, members and connections shall be determined in accordance with the Specification, except as modified throughout these Provisions.	
1259	<b>B4.</b>	SYSTEM TYPE	
1260 1261 1262 1263		The seismic force-resisting system (SFRS) shall contain one or more moment frame, braced frame or shear wall system conforming to the requirements of one of the seismic systems designated in Chapters E, F, G and H.	
1264	B5.	DIAPHRAGMS, CHORDS AND COLLECTORS	
1265	B5.1.	General	
1266 1267 1268 1269		Diaphragms and chords shall be designed for the loads and load combinations in the applicable building code. Collectors shall be designed for the load combinations in the applicable building code, including overstrength.	
1270	B5.2.	Truss Diaphragms	

When a truss is used as a diaphragm, all members of the truss and their connections shall be designed for forces calculated using the load combinations of the applicable building code, including overstrength.

## **Exceptions:**

 (a) The forces specified in this section need not be applied to the diagonal members of the truss diaphragms and their connections where these members and connections conform to the requirements of Sections F2.4a, F2.5a, F2.5b and F2.6c. Braces in K- or V- configurations and braces supporting gravity loads other than self-weight are not permitted under this exception.

**User Note:** Chords in truss diaphragms serve a function analogous to columns in vertical special concentrically braced frames, and should meet the requirements for highly ductile members as required for columns in Section F2.5a.

(b) The forces specified in this section need not be applied to truss diaphragms designed as a part of a three-dimensional system in which the seismic force-resisting system types consist of ordinary moment frames, ordinary concentrically braced frames, or combinations thereof, and truss diagonal members conform to Sections F1.4b and F1.5 and connections conform to Section F1.6.

1400		CHAPTER C		
1401		ANALYSIS		
1402 1403 1404	This chapter addresses design related analysis requirements. The chapter organized as follows:			
1405 1406 1407		C1. General Requirements C2. Additional Requirements C3. Nonlinear Analysis		
1408	C1.	GENERAL REQUIREMENTS		
1409 1410 1411		An analysis conforming to the requirements of the applicable building code and the Specification shall be performed for design of the system.		
1412 1413 1414 1415		When the design is based upon elastic analysis, the stiffness properties of component members of steel systems shall be based on elastic sections and those of composite systems shall include the effects of cracked sections.		
1416	C2.	ADDITIONAL REQUIREMENTS		
1417 1418		Additional analysis shall be performed as specified in Chapters E, F, G and H of these Provisions.		
1419	C3.	NONLINEAR ANALYSIS		
1420 1421 1422 1423		When nonlinear analysis is used to satisfy the requirements of these Provisions, it shall be performed in accordance with the applicable building code.		
1424 1425		<b>User Note:</b> ASCE/SEI 7 permits nonlinear analysis by a response history procedure. Material and geometric nonlinearities are to be included in the		
1426 1427 1428		analytical model. The main purpose is to determine expected member inelastic deformations and story drifts under representative ground motions. The analysis results also provide values of maximum expected		
1429 1430 1431		internal forces at locations such as column splices, which can be used as upper limits on required strength for design.		

1500		CHAPTER D
1501	GEN	ERAL MEMBER AND CONNECTION DESIGN REQUIREMENTS
1502		
1503 1504	This c	chapter addresses general requirements for the design of members and ections.
1505	The cl	napter is organized as follows:
1506 1507 1508 1509		<ul> <li>D1. Member Requirements</li> <li>D2. Connections</li> <li>D3. Deformation Compatibility of Non-SFRS Members and Connections</li> <li>D4. H-Piles</li> </ul>
1510	D1.	MEMBER REQUIREMENTS
1511 1512 1513		Members of moment frames, braced frames and shear walls in the seismic force-resisting system (SFRS) shall comply with the Specification and this section.
1514	D1.1.	Classification of Sections for Ductility
1515 1516 1517		When required for the systems defined in Chapters E, F, G, H and Section D4, members designated as moderately ductile members on highly ductile members shall comply with this section.
1518	D1.1a	. Section Requirements for Ductile Members
1519 1520 1521		Structural steel sections for both moderately ductile members and highly ductile members shall have flanges continuously connected to the web or webs.
1522 1523 1524		Encased composite columns shall comply with the requirements of Section D1.4b.1 for moderately ductile members and Section D1.4b.2 for highly ductile members.
1525 1526		Filled composite columns shall comply with the requirements of Section D1.4c for both moderately and highly ductile members.
1527 1528 1529		Concrete sections shall comply with the requirements of ACI 318 Section 18.4 for moderately ductile members and ACI 318 Section 18.6 and 18.7 for highly ductile members.
1530	D1.1b	. Width-to-Thickness Limitations of Steel and Composite Sections
1531 1532 1533		For members designated as moderately ductile members, the width-to-thickness ratios of compression elements shall not exceed the limiting width-to-thickness ratios, $\lambda_{md}$ , from Table D1.1.

For members designated as highly ductile members, the width-to-thickness ratios of compression elements shall not exceed the limiting width-to-thickness ratios,  $\lambda_{hd}$ , from Table D1.1.



TABLE D1.1
Limiting Width-to-Thickness Ratios for Compression Elements
For Moderately Ductile and Highly Ductile Members

Limiting Width-to-Thickness Ratio

	FOI IV	logerate	ly Ductile and F		wembers
	Description of Element	Width-to- Thickness Ratio	Limiting Width-to- λ <sub>hd</sub> Highly Ductile Members	λ <sub>md</sub> Moderately Ductile  Members	Example
Unstiffened Elements	Flanges of rolled or built-up I-shaped sections, channels and tees; legs of single angles or double angle members with separators; outstanding legs of pairs of angles in continuous contact	b/t	$0.32\sqrt{\frac{E}{R_yF_y}}$	$0.40\sqrt{\frac{E}{R_yF_y}}$	
	Flanges of H-pile sections per Section D4	b/t	not applicable	$0.48\sqrt{\frac{E}{R_yF_y}}$	p it
	Stems of tees	d/t	$0.32\sqrt{\frac{E}{R_yF_y}}$ [a]	$0.40\sqrt{\frac{E}{R_yF_y}}$	t_d
Stiffened Elements	Walls of rectangular HSS used as diagonal braces  Flanges of boxed I-shaped sections  Side plates of boxed I-shaped sections and walls of built-up box shapes used as diagonal braces  Flanges of built-up box shapes used as link beams	b/t b/t	$0.65 \sqrt{\frac{E}{R_y F_y}}$	$0.76\sqrt{\frac{E}{R_yF_y}}$	

	Webs of rolled or built-up I shaped sections and channels used as diagonal braces	h/t <sub>w</sub>	$1.57 \sqrt{\frac{E}{R_y F_y}}$	$1.57 \sqrt{\frac{E}{R_y F_y}}$	$-t_w h$ $-t_w h$
	Where used in beams or columns as flanges in uniform compression due to axial, flexure, or combined axial and flexure:  1) Walls of rectangular HSS  2) Flanges and side plates of boxed I-shaped sections, webs and flanges of built up have	b/t b/t, h/t	$0.65 \sqrt{\frac{E}{R_y F_y}}$	$1.18\sqrt{\frac{E}{R_yF_y}}$	
	of built-up box shapes				territorial i
			For C <sub>a</sub> ≤ 0.114	For C <sub>a</sub> ≤ 0114	
Stiffened Elements	Where used in beams, columns, or links, as webs in flexure, or		$2.57 \sqrt{\frac{E}{R_y F_y}} (1 - 1.04C_a)$ For C <sub>a</sub> > 0.114	For $C_2 > 0$ 114	$-t_{\pi}$ $h$ $-t_{\pi}$ $h$
ned E	combined axial and flexure:		$0.88\sqrt{\frac{E}{R_yF_y}}(2.68-C_a)$	$1.29\sqrt{\frac{E}{R_yF_y}}(2.12-C_a)$	h
Stiffe	1) Webs of rolled or built-up I-shaped	h/t <sub>w</sub>	$\geq 1.57 \sqrt{\frac{E}{R_y F_y}}$		11-
	sections or channels [b]		where	$\geq$ 1.57 $\sqrt{\frac{E}{R_y F_y}}$ where	
	2) Side plates of boxed I-shaped	h/t	1	$C_a = \frac{P_u}{\phi_c P_y} \text{ (LRFD)}$	
	3) Webs of built-up	h/t	$C_a = \frac{\Omega_c P_a}{P_y} \text{ (ASD)}$	$C_a = \frac{\Omega_c P_a}{P_y} \text{ (ASD)}$	
	box sections	.,,	$P_y = R_y F_y A_g$	$P_y = R_y F_y A_g$	
	Webs of built-up box sections used as EBF links	h/t	$0.67\sqrt{\frac{E}{R_yF_y}}$	$1.75\sqrt{\frac{E}{R_yF_y}}$	h h
	Webs of H-Pile sections	h/t <sub>w</sub>	not applicable	$1.57\sqrt{\frac{E}{R_yF_y}}$	

	Walls of round HSS	D/t	0.053 <u>E</u> R <sub>y</sub> F <sub>y</sub>	$0.062 \frac{E}{R_y F_y} [c]$	-Q-D
te Elements		b/t	$1.48\sqrt{\frac{E}{R_yF_y}}$	$2.37\sqrt{\frac{E}{R_yF_y}}$	
Composite	Walls of round filled composite members	D/t	0.085 <u>E</u> R <sub>y</sub> F <sub>y</sub>	$0.17 \frac{E}{R_y F_y}$	

For tee shaped compression members, the limiting width-to-thickness ratio for highly ductile members for the stem of the tee shall be  $_{0.40}\sqrt{\frac{E}{R_vF_v}}$  where either of the following conditions are satisfied:

- (1) Buckling of the compression member occurs about the plane of the stem.
- (2) The axial compression load is transferred at end connections to only the outside face of the flange of the tee resulting in an eccentric connection that reduces the compression stresses at the tip of the stem.
- For I-shaped beams in SMF systems, where  $C_a$  is less than or equal to 0.114, the limiting ratio h/t<sub>w</sub> shall not exceed 2.57  $\sqrt{\frac{E}{R_v F_v}}$ . For I-shaped beams in IMF systems, where  $C_a$  is less than or equal to 0.114, the

limiting width-to-thickness ratio shall not exceed 3.96  $\sqrt{\frac{E}{R_v F_v}}$ 

The limiting diameter-to-thickness ratio of round HSS members used as beams or columns shall not exceed  $0.077 \frac{E}{R_v F_v}$ 

#### **D1.2.** Stability Bracing of Beams

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When required in Chapters E, F, G and H, stability bracing shall be provided as required in this section to restrain lateral-torsional buckling of structural steel or concrete-encased beams subject to flexure and designated as moderately ductile members or highly ductile members.

**User Note:** In addition to the requirements in Chapters E, F, G and H to provide stability bracing for various beam members such as intermediate and special moment frame beams, stability bracing is also required for columns in the special cantilever column system (SCCS) in Section E6.

#### D1.2a. Moderately Ductile Members

#### 1. Steel Beams

The bracing of moderately ductile steel beams shall satisfy the following requirements:

1568 1569 1570	(a)	Both flanges of beams shall be laterally braced or the beam cross section shall be braced with point torsional bracing.
1571 1572 1573 1574	(b)	Beam bracing shall meet the requirements of Appendix 6 of the Specification for lateral or torsional bracing of beams, where the required flexural strength of the member shall be:
1575		$M_{r} = R_{y}F_{y}Z/\alpha_{s} $ (D1-1)
1576 1577 1578 1579 1580 1581 1582 1583		where $C_d = 1.0$ $R_y = \text{ratio of the expected yield stress to the specified minimum yield stress}$ $Z = \text{plastic section modulus about the axis of bending, in.}^3 \text{ (mm}^3\text{)}$ $\alpha_s = \text{LRFD-ASD force level adjustment factor}$ $= 1.0 \text{ for LRFD and } 1.5 \text{ for ASD}$
1584	(c)	Beam bracing shall have a maximum spacing of
1585		$L_b = 0.19 r_y E/(R_y F_y)$ (D1-2)
1586	2. Conc	rete-Encased Composite Beams
1587 1588		oracing of moderately ductile concrete-encased composite is shall satisfy the following requirements:
1589 1590 1591	(a)	Both flanges of members shall be laterally braced or the beam cross section shall be braced with point torsional bracing.
1592 1593 1594 1595	(b)	Lateral bracing shall meet the requirements of Appendix 6 of the Specification for lateral or torsional bracing of beams, where $M_r = M_{p,exp}$ of the beam as specified in Section G2.6d, and $C_d = 1.0$ .
1596	(c)	Member bracing shall have a maximum spacing of
1597		$L_b = 0.19r_y E/(R_y F_y)$ (D1-3)
1598 1599 1600		using the material properties of the steel section and $r_{\rm y}$ in the plane of buckling calculated based on the elastic transformed section.
1601	D1.2b. Highly Duct	ile Members
1602 1603		o the requirements of Sections D1.2a.1(a) and (b), and (d), the bracing of highly ductile beam members shall
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1604 1605 1606 1607	have a maximum spacing of $L_b$ =0.095 $r_y$ E/( $R_y$ F $_y$ ). For concrete-encased composite beams, the material properties of the steel section shall be used and the calculation for $r_y$ in the plane of buckling shall be based on the elastic transformed section.			
1608 <b>D1.2c.</b>	Special Bracing at Plastic Hinge Locations			
1609 1610	-		ng shall be located adjacent to expected plastic him re required by Chapters E, F, G or H.	ıge
1611	1.	Steel B	Beams	
1612 1613		For stru	uctural steel beams, such bracing shall satisfy the followisments:	ing
1614 1615 1616		(a)	Both flanges of beams shall be laterally braced or t member cross section shall be braced with point torsion bracing.	
1617 1618		(b)	The required strength of lateral bracing of each flan provided adjacent to plastic hinges shall be:	ige
1619			$P_{r} = 0.06R_{v}F_{v}Z/(\alpha_{s}h_{o}) $ (D1-	-4)
1620			where	
1621			h <sub>o</sub> = distance between flange centroids, in. (mm)	
1622 1623			The required strength of torsional bracing provide adjacent to plastic hinges shall be:	led
1624			$M_{r} = 0.06R_{y}F_{y}Z/\alpha_{s} $ (D1-	-5)
1625 1626 1627 1628 1629		(c)	The required bracing stiffness shall satisfy to requirements of Appendix 6 of the Specification stateral or torsional bracing of beams with $C_d = 1.0$ at where the required flexural strength of the beam shall taken as:	for and
1630			$M_{r} = R_{y}F_{y}Z/\alpha_{s} $ (D1-	-6)
1631	2.	Concr	ete-Encased Composite Beams	
1632 1633			ncrete-encased composite beams, such bracing shall satisflowing requirements:	sfy
1634 1635 1636		(a)	Both flanges of beams shall be laterally braced or the beam cross section shall be braced with point torsion bracing.	

1637 1638		(b)	The required strength of lateral bracing provided adjacent to plastic hinges shall be	
1639			$P_u = 0.06 M_{p,exp} / h_o$ (D1-7)	
1640			of the beam, where	
1641 1642 1643 1644			M <sub>p,exp</sub> = expected flexural strength of the steel, concrete-encased or composite beam, kipin. (N-mm); determined in accordance with Section G2.6d.	
1645 1646 1647			The required strength for torsional bracing provided adjacent to plastic hinges shall be $M_u = 0.06 M_{p,exp}$ of the beam.	
1648 1649 1650 1651 1652		(c)	The required bracing stiffness shall satisfy the requirements of Appendix 6 of the Specification for lateral or torsional bracing of beams where $M_r = M_u = M_{p,exp}$ of the beam is determined in accordance with Section G2.6d, and $C_d = 1.0$ .	
1653	D1.3.	Protected Zo	nes	
1654 1655 1656 1657		erection proce of a member of	es specified in Section I2.1 resulting from fabrication and edures and from other attachments are prohibited in the area or a connection element designated as a protected zone by ons or ANSI/AISC 358.	
1658 1659 1660 1661 1662		permitted in p otherwise dete with Section k	Velded steel headed stud anchors and other connections are rotected zones when designated in ANSI/AISC 358, or as ermined with a connection prequalification in accordance K1, or as determined in a program of qualification testing in ith Sections K2 and K3.	
1663	D1.4.	Columns		
1664 1665			noment frames, braced frames and shear walls shall satisfy nts of this section.	
1666	D1.4a	a. Required Strength		
1667 1668 1669		-	strength of columns in the SFRS shall be determined from eect of the following:	
1670 1671 1672		` '	and effect resulting from the analysis requirements for the able system per Sections E, F, G and H.	
1673 1674		using	ompressive axial strength and tensile strength as determined the overstrength seismic load. It is permitted to neglect	

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1675 1676 1677	applied moments in this determination unless the moment result from a load applied to the column between points of latera support.
1678 1679 1680 1681 1682 1683	For columns that are common to intersecting frames, determination of the required axial strength, including the overstrength seismic load or the capacity-limited seismic load, as applicable, shall consider the potentia for simultaneous inelasticity from all such frames. The direction of application of the load in each such frame shall be selected to produce the most severe load effect on the column.
1684	Exceptions:
1685 1686 1687 1688 1689 1690 1691	<ul> <li>(a) It is permitted to limit the required axial strength for such columns based on a three-dimensional nonlinear analysis in which ground motion is simultaneously applied in two orthogonal directions, in accordance with Section C3.</li> <li>(b) Columns common to intersecting frames that are part of Section E1, F1, G1, H1, H4 or combinations thereof need not be designed for these loads.</li> </ul>
1692	D1.4b. Encased Composite Columns
1693 1694 1695 1696 1697 1698	Encased composite columns shall satisfy the requirements of Specification Chapter I, in addition to the requirements of this section Additional requirements, as specified for moderately ductile members and highly ductile members in Sections D1.4b.1 and 2, shall apply a required in the descriptions of the composite seismic systems in Chapter G and H.
1699	1. Moderately Ductile Members
1700 1701	Encased composite columns used as moderately ductile members shall satisfy the following requirements:
1702 1703	(a) The maximum spacing of transverse reinforcement at the top and bottom shall be the least of the following:
1704	(1) one-half the least dimension of the section
1705	(2) 8 longitudinal bar diameters
1706	(3) 24 tie bar diameters
1707	(4) 12 in. (300 mm)
1708 1709 1710 1711	(b) This spacing shall be maintained over a vertical distance equal to the greatest of the following lengths, measured from each joint face and on both sides of any section where flexural yielding is expected to occur:

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			(1) one-sixth the vertical clear height of the column
1713			(2) the maximum cross-sectional dimension
1714			(3) 18 in. (450 mm)
1715 1716		(c)	Tie spacing over the remaining column length shall not exceed twice the spacing defined in Section D1.4b.1(1).
1717 1718 1719 1720 1721 1722 1723 1724 1725 1726 1727		(d)	Splices and end bearing details for encased composite columns in composite ordinary SFRS of Sections G1, H1 and H4 shall satisfy the requirements of the Specification and ACI 318 Section 10.7. The design shall comply with ACI 318 Sections 18.2.7 and 18.2.8. The design shall consider any adverse behavioral effects due to abrupt changes in either the member stiffness or the nominal tensile strength. Transitions to reinforced concrete sections without embedded structural steel members, transitions to bare structural steel sections, and column bases shall be considered abrupt changes.
1728 1729		(e)	Welded wire fabric shall be prohibited as transverse reinforcement.
1730	2.	Highly	y Ductile Members
1731 1732			ed composite columns used as highly ductile members shall Section D1.4b.1 in addition to the following requirements:
			· · · · · · · · · · · · · · · · · · ·
<ul><li>1732</li><li>1733</li></ul>		satisfy	Section D1.4b.1 in addition to the following requirements: Longitudinal load-carrying reinforcement shall satisfy the
1732 1733 1734 1735 1736		satisfy (a)	Section D1.4b.1 in addition to the following requirements:  Longitudinal load-carrying reinforcement shall satisfy the requirements of ACI 318 Section 18.7.4.  Transverse reinforcement shall be hoop reinforcement as defined in ACI 318 Chapter 18 and shall satisfy the
1732 1733 1734 1735 1736 1737		satisfy (a)	Section D1.4b.1 in addition to the following requirements:  Longitudinal load-carrying reinforcement shall satisfy the requirements of ACI 318 Section 18.7.4.  Transverse reinforcement shall be hoop reinforcement as defined in ACI 318 Chapter 18 and shall satisfy the following requirements:  (1) The minimum area of tie reinforcement, A <sub>sh</sub> , shall

	D-11
1748 1749 1750 1751	P <sub>n</sub> = nominal compressive strength of the composite column calculated in accordance with the Specification, kips (N)
1752 1753 1754 1755	h <sub>cc</sub> = cross-sectional dimension of the confined core measured center-to-center of the tie reinforcement, in. (mm)
1756 1757 1758	f' <sub>c</sub> = specified compressive strength of concrete, ksi (MPa) s = spacing of transverse
1758 1759 1760 1761	reinforcement measured along the longitudinal axis of the structural member, in. (mm)
1762 1763 1764 1765	Equation D1-8 need not be satisfied if the nominal strength of the concrete-encased structural steel section alone is greater than the load effect from a load combination of 1.0D+0.5L,
1766 1767 1768 1769	where D = dead load due to the weight of the structural elements and permanent
1770 1771 1772	features on the building, kips (N)  L = live load due to occupancy and moveable equipment, kips (N)
1773 1774 1775 1776	(2) The maximum spacing of transverse reinforcement along the length of the column shall be the lesser of six longitudinal load-carrying bar diameters or 6 in. (150 mm).
1776 1777 1778 1779 1780	Where transverse reinforcement is specified in Sections D1.4b.2(3), D1.4b.2(4), or D1.4b.2(5), the maximum spacing of transverse reinforcement along the member length shall be the lesser of
1781 1782 1783 1784	one-fourth the least member dimension or 4 in. (100 mm)." Confining reinforcement shall be spaced not more than 14 in. (350 mm) on center in the transverse direction.
1785 1786 (c) 1787 1788 1789 1790 1791 1792	required compressive strengths greater than 0.2P <sub>n</sub> , not including the overstrength seismic load, shall have transverse reinforcement as specified in Section D1.4b.2(2)(iii) over the total element length. This requirement need not be satisfied if the nominal strength
1/72	of the concrete-encased steel section alone is greater than 2016 Seismic Provisions for Structural Steel Buildings

1793		the load effect from a load combination of 1.0D+0.5L.
1794	(1)	
1795	(d)	Composite columns supporting reactions from
1796		discontinued stiff members, such as walls or braced
1797		frames, shall have transverse reinforcement as specified in
1798		Section D1.4b.2(2)(iii) over the full length beneath the
1799		level at which the discontinuity occurs if the required
1800		compressive strengthexceeds 0.1P <sub>n</sub> , not including the
1801 1802		overstrength seismic load. Transverse reinforcement shall extend into the discontinued member for at least the
1802		
		length required to develop full yielding in the concrete-
1804		encased steel section and longitudinal reinforcement. This
1805 1806		requirement need not be satisfied if the nominal strength
1807		of the concrete-encased steel section alone is greater than the load effect from a load combination of 1.0D+0.5L.
1808		the load effect from a load combination of 1.0D+0.3L.
1809	(a)	Energed composite columns used in a CSME shall
1810	(e)	Encased composite columns used in a C-SMF shall satisfy the following requirements:
1811		satisfy the following requirements.
1812		(1) Transverse reinforcement shall satisfy the
1813		requirements in Section D1.4b.2(2) at the top and
1814		bottom of the column over the region specified in
1815		
1816		Section D1.4b.1(2).
1817		(2) The strong-column/weak-beam design
1818		(2) The strong-column/weak-beam design requirements in Section G3.4a shall be satisfied.
1819		Column bases shall be detailed to sustain inelastic
1820		flexural hinging.
1821		nexurar minging.
1822		(3) The required shear strength of the column shall
1823		satisfy the requirements of ACI 318 Section
1824		18.7.6.1.
1825		10.7.0.1.
1826	(f)	When the column terminates on a footing or mat
1827	(1)	foundation, the transverse reinforcement as specified in
1828		this section shall extend into the footing or mat at least 12
1829		in. (300 mm). When the column terminates on a wall, the
1830		transverse reinforcement shall extend into the wall for at
1831		least the length required to develop full yielding in the
1832		concrete-encased shape and longitudinal reinforcement.
1833	D1.4c. Filled Compo	
	•	
1834	-	pplies to columns that meet the limitations of Specification
1835		Such columns shall be designed to satisfy the requirements
1836	-	on Chapter I, except that the nominal shear strength of the
1837	composite col	umn shall be the nominal shear strength of the structural

1838		steel section alone, based on its effective shear area.
1839 1840	D1.5.	Composite Slab Diaphragms
1841 1842		The design of composite floor and roof slab diaphragms for seismic effects shall meet the following requirements.
1843	D1.5a	. Load Transfer
1844 1845 1846		Details shall be provided to transfer loads between the diaphragm and boundary members, collector elements, and elements of the horizonta framing system.
1847	D1.5b	. Nominal Shear Strength
1848 1849 1850 1851 1852 1853		The nominal in-plane shear strength of composite diaphragms and concrete slab on steel deck diaphragms shall be taken as the nominal shear strength of the reinforced concrete above the top of the steel deck ribs in accordance with ACI 318 excluding Chapter 14. Alternatively, the composite diaphragm nominal shear strength shall be determined by integrated plane shear tests of concrete-filled diaphragms.
1854	D1.6.	BUILT-UP STRUCTURAL STEEL MEMBERS
1855 1856 1857		This section addresses connections between components of built-up members where specific requirements are not provided in the system chapters of these Provisions or in ANSI/AISC 358.
1858 1859 1860 1861		Connections between components of built-up members subject to inelastic behavior shall be designed for the expected forces arising from that inelastic behavior.
1862 1863 1864 1865		Connections between components of built-up members where inelastic behavior is not expected shall be designed for the load effect including the overstrength seismic forces.
1866 1867 1868 1869 1870		Where connections between elements of a built-up member are required in a protected zone, the connections shall have an available tensile strength equal to $R_y F_y t_p / \alpha_s$ of the weaker element for the length of the protected zone.
1871 1872 1873 1874		Built-up members may be used in connections requiring testing in accordance with the Provisions provided they are accepted by ANSI/AISC 358 for use in a prequalified joint or have been verified in a qualification test.

#### 1875 D2. CONNECTIONS

#### **D2.1. General**

1877 Connections, joints and fasteners that are part of the SFRS shall comply with Specification Chapter J, and with the additional requirements of this section.

Splices and bases of columns that are not designated as part of the SFRS shall satisfy the requirements of Sections D2.5a, D2.5c and D2.6.

Where protected zones are designated in connection elements by these Provisions or ANSI/AISC 358, they shall satisfy the requirements of Sections D1.3 and I2.1.

#### D2.2. Bolted Joints

Bolted joints shall satisfy the following requirements:

(a) The available shear strength of bolted joints using standard holes or short slotted holes perpendicular to the applied load shall be calculated as that for bearing-type joints in accordance with Specification Sections J3.6 and J3.10. The nominal bolt bearing and tearout equations per Section J3.10 of the Specification where deformation at the bolt hole at service load is a design consideration shall be used.

Exception: Where the required strength of a connection is based upon the expected strength of a member or element, it is permitted to use the bolt bearing and tearout equations in accordance with Specification Section J3.10 where deformation is not a design consideration.

(b) Bolts and welds shall not be designed to share force in a joint or the same force component in a connection.

**User Note**: A member force, such as a diagonal brace axial force, must be resisted at the connection entirely by one type of joint (in other words, either entirely by bolts or entirely by welds). A connection in which bolts resist a force that is normal to the force resisted by welds, such as a moment connection in which welded flanges transmit flexure and a bolted web transmits shear, is not considered to be sharing the force.

(c) Bolt holes shall be standard holes or short-slotted holes perpendicular to the applied load in bolted joints where the seismic load effects are transferred by shear in the bolts. Oversized holes or short-slotted holes are permitted in connections where the seismic load effects are transferred by tension in the bolts but not by shear in the bolts.

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1914	E	Exception:
1915 1916 1917	(	1) For diagonal braces, oversized holes are permitted in one connection ply only when the connection is designed as a slip-critical joint.
1918 1919 1920 1921 1922	(	2) Alternative hole types are permitted if designated in ANSI/AISC 358, or if otherwise determined in a connection prequalification in accordance with Section K1, or if determined in a program of qualification testing in accordance with Section K2 or Section K3.
1923 1924 1925 1926	n s	Jser Note: Diagonal brace connections with oversized holes must also satisfy other limit states including bolt bearing and bolt hear for the required strength of the connection as defined in sections F1, F2, F3 and F4.
1927 1928 1929 1930 1931	F	All bolts shall be installed as pretensioned high-strength bolts. Saying surfaces shall satisfy the requirements for slip-critical onnections in accordance with Specification Section J3.8 with a saying surface with a Class A slip coefficient or higher.
1932 1933 1934 1935	v	Exceptions: Connection surfaces are permitted to have coatings with a slip coefficient less than that of a Class A faying surface for the following:
1936 1937 1938	(	1) End plate moment connections conforming to the requirements of Section E1, or ANSI/AISC 358
1939 1940 1941	(	Bolted joints where the seismic load effects are transferred either by tension in bolts or by compression bearing but not by shear in bolts
1942 <b>D2.3.</b>	Welded Joints	
1943 1944	Welded Specifica	joints shall be designed in accordance with Chapter J of the ation.
1945 <b>D2.4.</b>	Continu	ity Plates and Stiffeners
1946 1947 1948	rolled sh	ign of continuity plates and stiffeners located in the webs of apes shall allow for the reduced contact lengths to the member and web based on the corner clip sizes in Section I2.4.
1949 <b>D2.5.</b>	Column	Splices
1950 <b>D2.5a.</b>	Location of Splices	
1951	For all b	building columns, including those not designated as part of the

1952 1953		SFRS, column splices shall be located 4 ft (1.2 m) or more away from the beam-to-column flange connections.		
1954	Excep	Exceptions:		
1955 1956 1957	(a)	When the column clear height between beam-to-column flange connections is less than 8 ft (2.4 m), splices shall be at half the clear height		
1958 1959 1960 1961	(b)	Column splices with webs and flanges joined by complete-joint-penetration groove welds are permitted to be located closer to the beam-to-column flange connections, but not less than the depth of the column		
1962	(c)	Splices in composite columns		
1963 1964 1965 1966	above safety	<b>Note:</b> Where possible, splices should be located at least 4 ft (1.2 m) the finished floor elevation to permit installation of perimeter cables prior to erection of the next tier and to improve sibility.		
1967	1967 <b>D2.5b. Required Strength</b>			
1968 1969	The re of:	equired strength of column splices in the SFRS shall be the greater		
1970 1971	(a)	The required strength of the columns, including that determined from Chapters E, F, G and H and Section D1.4a; or,		
1972 1973	(b)	The required strength determined using the overstrength seismic load.		
1974 1975 1976	subjec	ition, welded column splices in which any portion of the column is et to a calculated net tensile load effect determined using the rength seismic load shall satisfy all of the following requirements:		
1977 1978 1979 1980 1981	(a)	The available strength of partial-joint-penetration (PJP) groove welded joints, if used, shall be at least equal to 200% of the required strength. Exception: Partial-joint penetration (PJP) groove welds are excluded from this requirement according to the exceptions to Sections E2.6g, E3.6g and E4.6c.		
1982 1983 1984 1985	(b)	The available strength for each flange splice shall be at least equal to $0.5R_yF_yb_ft_f/\alpha_s$ , where $R_yF_y$ is the expected yield stress of the column material and $b_ft_f$ is the area of one flange of the smaller column connected.		
1986 1987	(c)	Where butt joints in column splices are made with complete- joint-penetration groove welds, when tension stress at any		

D-17 1988 location in the smaller flange exceeds  $0.30F_v/\alpha_s$  tapered transitions are required between flanges of unequal thickness or 1989 Such transitions shall be in accordance with AWS 1990 D1 8/D1 8M clause 4 2 1991 1992 D2.5c. Required Shear Strength 1993 For all building columns including those not designated as part of the SFRS, the required shear strength of column splices with respect to both 1994 orthogonal axes of the column shall be  $M_{pc}/(\alpha_s H)$ , where  $M_{pc}$  is the 1995 1996 lesser plastic flexural strength of the column sections for the direction in question, and H is the height of the story, which is permitted to be taken 1997 1998 as the distance between the centerline of floor framing at each of the 1999 levels above and below, or the distance between the top of floor slabs at each of the levels above and below. 2000 2001

The required shear strength of splices of columns in the SFRS shall be the greater of the above requirement or the required shear strength determined per Section D2.5b(a) and (b).

## D2.5d. Structural Steel Splice Configurations

Structural steel column splices are permitted to be either bolted or welded, or welded to one column and bolted to the other. Splice configurations shall meet all specific requirements in Chapters E, F, G or H

Splice plates or channels used for making web splices in SFRS columns shall be placed on both sides of the column web.

For welded butt joint splices made with groove welds, weld tabs shall be removed in accordance with AWS D1.8/D1.8M clause 6.11. Steel backing of groove welds need not be removed.

#### D2.5e. Splices in Encased Composite Columns

For encased composite columns, column splices shall conform to Section D1.4b and ACI 318 Section 18.7.4.3.

#### **2017 D2.6.** Column Bases

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The required strength of column bases, including those that are not designated as part of the SFRS, shall be calculated in accordance with this section.

The available strength of steel elements at the column base, including base plates, anchor rods, stiffening plates, and shear lug elements shall be in accordance with the Specification.

Where columns are welded to base plates with groove welds, weld tabs and weld backing shall be removed, except that weld backing located on the inside of flanges and weld backing on the web of I-shaped sections need not be removed if backing is attached to the column base plate with a continuous 5/16-in. fillet weld. Fillet welds of backing to the inside of column flanges are prohibited. Weld backing located on the inside of HSS and box columns need not be removed.

The available strength of concrete elements and reinforcing steel at the column base shall be in accordance with ACI 318. When the design of anchor rods assumes that the ductility demand is provided for by deformations in the anchor rods and anchorage into reinforced concrete, the design shall meet the requirements of ACI 318 Chapter 17. Alternatively, when the ductility demand is provided for elsewhere, the anchor rods and anchorage into reinforced concrete are permitted to be designed for the maximum loads resulting from the deformations occurring elsewhere including the effects of material overstrength and strain hardening.

**User Note:** When using concrete reinforcing steel as part of the anchorage embedment design, it is important to consider the anchor failure modes and provide reinforcement that is developed on both sides of the expected failure surface. See ACI 318 Chapter 17, including Commentary.

# D2.6a. Required Axial Strength

The required axial strength of column bases that are designated as part of the SFRS, including their attachment to the foundation, shall be the summation of the vertical components of the required connection strengths of the steel elements that are connected to the column base, but not less than the greater of:

- (a) The column axial load calculated using the overstrength seismic load
- (b) The required axial strength for column splices, as prescribed in Section D2.5

**User Note:** The vertical components can include both the axial load from columns and the vertical component of the axial load from diagonal members framing into the column base. Section D2.5 includes references to Section D1.4a and Chapters E, F, G and H. Where diagonal braces frame to both sides of a column, the effects of compression brace buckling should be considered in the summation of vertical components. See Section F2.3.

# D2.6b. Required Shear Strength

The required shear strength of column bases, including those not designated as part of the SFRS, and their attachments to the foundations, shall be the summation of the horizontal component of the required

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2067 2068			ction strengths of the steel elements that are connected to the n base as follows:
2069 2070 2071		(a)	For diagonal braces, the horizontal component shall be determined from the required strength of diagonal brace connections for the SFRS.
2072 2073		(b)	For columns, the horizontal component shall be equal to the lesser of the following:
2074			(1) $2R_yF_yZ/(\alpha_sH)$ of the column
2075			(2) The shear calculated using the overstrength seismic load.
2076 2077		(c)	The summation of the required strengths of the horizontal components shall not be less than $0.7F_yZ/(\alpha_sH)$ of the column.
2078		Excep	tions:
2079 2080		(a)	Single story columns with simple connections at both ends need not comply with Section D2.6b(b) or D2.6b(c).
2081 2082 2083		(b)	Columns that are part of the systems defined in Sections E1, F1, G1, H1, H4 or combinations thereof need not comply with D2.6b(c).
2084 2085 2086 2087 2088 2089		(c)	The minimum required shear strength per Section D2.6b(c) need not exceed the maximum load effect that can be transferred from the column to the foundation as determined by either a nonlinear analysis per Section C3, or an analysis that includes the effects of inelastic behavior resulting in 0.025H story drift at either the first or second story, but not both concurrently.
2090 2091 2092 2093 2094 2095		colum memb that ar	<b>Note:</b> The horizontal components can include the shear load from its and the horizontal component of the axial load from diagonal ers framing into the column base. Horizontal forces for columns is not part of the SFRS determined in accordance with this section lly will not govern over those determined according to Section (c)
2096	D2.6c	. Requi	red Flexural Strength
2097 2098 2099 2100 2101		founda design founda	e column bases are designed as moment connections to the ation, the required flexural strength of column bases that are ated as part of the SFRS, including their attachment to the ation, shall be the summation of the required connection strengths steel elements that are connected to the column base as follows:
2102 2103		(a)	For diagonal braces, the required flexural strength shall be at least equal to the required flexural strength of diagonal brace

connections.

2105 2106	(b)		olumns, the required flexural strength shall be at least equal elesser of the following:
2107		(1)	$1.1R_yF_yZ/\alpha_s$ of the column, or
2108 2109 2110		(2)	the moment calculated using the overstrength seismic load, provided that a ductile limit state in either the column base or the foundation controls the design.
2111 2112			Moments at column to column base connections designed as ections may be ignored.
2113 <b>D2.</b>	7. Com	posite C	Connections
2114 2115 2116 2117 2118 2119 2120	steel a struct calcul section testin	and concural step lating the on. Unle g, the r	applies to connections in buildings that utilize composite crete systems wherein seismic load is transferred between eel and reinforced concrete components. Methods for ne connection strength shall satisfy the requirements in this ess the connection strength is determined by analysis or models used for design of connections shall satisfy the quirements:
2121 2122	(a)		shall be transferred between structural steel and preed concrete through:
2123		(1)	direct bearing from internal bearing mechanisms;
2124		(2)	shear connection;
2125 2126 2127		(3)	shear friction with the necessary clamping force provided by reinforcement normal to the plane of shear transfer; or
2128		(4)	a combination of these means.
2129 2130 2131 2132 2133		combi mecha betwe	contribution of different mechanisms is permitted to be ined only if the stiffness and deformation capacity of the anisms are compatible. Any potential bond strength een structural steel and reinforced concrete shall be ignored e purpose of the connection force transfer mechanism.
2134 2135 2136 2137 2138	(b)	requir substa friction	nominal bearing and shear-friction strengths shall meet the rements of ACI 318 Chapter 16. Unless a higher strength is antiated by cyclic testing, the nominal bearing and shearon strengths shall be reduced by 25% for the composite sic systems described in Sections G3, H2, H3, H5 and H6.

2139 2140 2141	(c)	Face bearing plates consisting of stiffeners between the flanges of steel beams shall be provided when beams are embedded in reinforced concrete columns or walls.
2142 2143 2144 2145 2146	(d)	The nominal shear strength of concrete-encased steel panel zones in beam-to-column connections shall be calculated as the sum of the nominal strengths of the structural steel and confined reinforced concrete shear elements as determined in Section E3.6e and ACI 318 Section 18.8, respectively.
2147 2148 2149 2150 2151 2152 2153 2154 2155	(e)	Reinforcement shall be provided to resist all tensile forces in reinforced concrete components of the connections. Additionally, the concrete shall be confined with transverse reinforcement. All reinforcement shall be fully developed in tension or compression, as applicable, beyond the point at which it is no longer required to resist the forces. Development lengths shall be determined in accordance with ACI 318 Chapter 25. Additionally, development lengths for the systems described in Sections G3, H2, H3, H5 and H6 shall satisfy the requirements of ACI 318 Section 18.8.5
2156 2157	(f)	Composite connections shall satisfy the following additional requirements:
2158 2159 2160 2161 2162		(1) When the slab transfers horizontal diaphragm forces, the slab reinforcement shall be designed and anchored to carry the in-plane tensile forces at all critical sections in the slab, including connections to collector beams, columns, diagonal braces and walls.
2163 2164 2165 2166 2167 2168		(2) For connections between structural steel or composite beams and reinforced concrete or encased composite columns, transverse hoop reinforcement shall be provided in the connection region of the column to satisfy the requirements of ACI 318 Section 18.8, except for the following modifications:
2169 2170 2171 2172 2173		(i) Structural steel sections framing into the connections are considered to provide confinement over a width equal to that of face bearing plates welded to the beams between the flanges.
2174 2175 2176 2177 2178		(ii) Lap splices are permitted for perimeter ties when confinement of the splice is provided by face bearing plates or other means that prevents spalling of the concrete cover in the systems described in Sections G1, G2, H1 and H4.
2179		(iii) The longitudinal bar sizes and layout in reinforced 2016 Seismic Provisions for Structural Steel Buildings Draft dated December 18, 2015

2180 concrete and composite columns shall be detailed to minimize slippage of the bars through the 2181 beam-to-column connection due to high force 2182 transfer associated with the change in column 2183 2184 moments over the height of the connection. 2185 User Note: The commentary provides guidance for determining panel zone shear strength. 2186 **D2.8.** Steel Anchors 2187 2188 Where steel headed stud anchors or welded reinforcing bar anchors are 2189

2190 2191

part of the intermediate or special SFRS of Sections G2, G3, G4, H2, H3, H5 and H6, their shear and tensile strength shall be reduced by 25% from the specified strengths given in Specification Chapter I. The diameter of steel headed stud anchors shall be limited to 3/4 in. (19 mm).

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User Note: The 25% reduction is not necessary for gravity and collector components in structures with intermediate or special seismic forceresisting systems designed for the overstrength seismic load.

### **D3**. DEFORMATION COMPATIBILITY OF NON-SFRS MEMBERS AND CONNECTIONS

Where deformation compatibility of members and connections that are not part of the seismic force-resisting system (SFRS) is required by the applicable building code, these elements shall be designed to resist the combination of gravity load effects and the effects of deformations occurring at the design story drift calculated in accordance with the applicable building code.

User Note: ASCE/SEI 7 stipulates the above requirement for both structural steel and composite members and connections. Flexible shear connections that allow member end rotations in accordance with Specification Section J1.2 should be considered to satisfy these requirements. Inelastic deformations are permitted in connections or members provided they are self-limiting and do not create instability in the member. See the Commentary for further discussion.

#### **D4.** 2212 **H-PILES**

#### 2213 **D4.1.** Design Requirements

Design of H-piles shall comply with the requirements of the Specification regarding design of members subjected to combined loads. H-piles located in site classes E or F as defined by ASCE/SEI 7 shall satisfy the requirements for moderately ductile members of Section D1.1.

2218	D4.2.	Battered H-Piles
2219 2220 2221		If battered (sloped) and vertical piles are used in a pile group, the vertical piles shall be designed to support the combined effects of the dead and live loads without the participation of the battered piles.
2222	D4.3.	Tension
2223 2224 2225		Tension in each pile shall be transferred to the pile cap by mechanical means such as shear keys, reinforcing bars, or studs welded to the embedded portion of the pile.
2226 2227	D4.4.	Protected Zone
2228 2229 2230 2231		At each pile, the length equal to the depth of the pile cross section located directly below the bottom of the pile cap shall be designated as a protected zone meeting the requirements of Sections D1.3 and I2.1.

2300		CHAPTER E
2301		MOMENT-FRAME SYSTEMS
2302 2303 2304	require	chapter provides the basis of design, the requirements for analysis, and the ements for the system, members and connections for steel moment-frame systems. Lapter is organized as follows:
2305 2306 2307 2308 2309 2310		<ul> <li>E1. Ordinary Moment Frames (OMF)</li> <li>E2. Intermediate Moment Frames (IMF)</li> <li>E3. Special Moment Frames (SMF)</li> <li>E4. Special Truss Moment Frames (STMF)</li> <li>E5. Ordinary Cantilever Column Systems (OCCS)</li> <li>E6. Special Cantilever Column Systems (SCCS)</li> </ul>
2311 2312		<b>Note:</b> The requirements of this chapter are in addition to those required by the location and the applicable building code.
2313	E1.	ORDINARY MOMENT FRAMES (OMF)
2314	E1.1.	Scope
2315 2316		Ordinary moment frames (OMF) of structural steel shall be designed in conformance with this section.
2317	E1.2.	Basis of Design
2318 2319		OMF designed in accordance with these provisions are expected to provide minimal inelastic deformation capacity in their members and connections.
2320	E1.3.	Analysis
2321		There are no requirements specific to this system.
2322	E1.4.	System Requirements
2323		There are no requirements specific to this system.
2324	E1.5.	Members
2325	E1.5a.	Basic Requirements
2326 2327 2328 2329 2330		There are no limitations on width-to-thickness ratios of members for OMI beyond those in the Specification. There are no requirements for stability bracing of beams or joints in OMF, beyond those in the Specification. Structural stee beams in OMF are permitted to be composite with a reinforced concrete slab to resist gravity loads.
2331	E1.5b	. Protected Zones
2332		There are no designated protected zones for OMF members.  2016 Seismic Provisions for Structural Steel Buildings  Draft dated December 18, 2015  AMERICAN DESTRUCTION OF STRUCTURE OF STRUC
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2333	E1.6.	Connections
2334 2335		Beam-to-column connections are permitted to be fully restrained (FR) or partially restrained (PR) moment connections in accordance with this section.
2336	E1.6a.	Demand Critical Welds
2337 2338 2339		Complete-joint-penetration (CJP) groove welds of beam flanges to columns are demand critical welds, and shall satisfy the requirements of Sections A3.4b and I2.3.
2340	E1.6b	. FR Moment Connections
2341 2342		FR moment connections that are part of the seismic force-resisting system (SFRS) shall satisfy at least one of the following requirements:
2343 2344 2345 2346		(a) FR moment connections shall be designed for a required flexural strength that is equal to the expected beam flexural strength, $R_y M_p$ , multiplied by 1.1 and divided by $\alpha_s$ , where $\alpha_s$ = LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD.
2347 2348 2349		The required shear strength of the connection, $V_u$ or $V_a$ , as applicable, shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, $E_{\rm cl}$ , shall be taken as:
2350		$E_{cl} = 2(1.1R_y M_p)/L_{cf}$ (E1-1)
2351 2352 2353 2354 2355 2356		where $\begin{array}{rcl} & L_{cf} & = & clear \ length \ of \ beam, \ in. \ (mm) \\ & M_p & = & F_yZ, \ kip-in. \ (N-mm) \\ & R_y & = & ratio \ of \ expected \ yield \ stress \ to \ the \ specified \ minimum \ yield \ stress, \\ & F_y & \end{array}$
2357 2358 2359 2360 2361		(b) FR moment connections shall be designed for a required flexural strength and a required shear strength equal to the maximum moment and corresponding shear that can be transferred to the connection by the system, including the effects of material overstrength and strain hardening.
2362 2363 2364 2365 2366		User Note: Factors that may limit the maximum moment and corresponding shear that can be transferred to the connection include column yielding, panel zone yielding, the development of the flexural strength of the beam at some distance away from the connection when web tapered members are used, and others. Further discussion is provided in the commentary.
2367 2368 2369		For options (a) and (b) in Section E1.6b, continuity plates shall be provided as required by Sections J10.1, J10.2 and J10.3 of the Specification. The bending moment used to check for continuity plates shall be the same bending moment

2370 2371	used to design the beam-to-column commaximum moment that can be transferre
2372 2373 2374	(c) FR moment connections between we flange columns shall either satisfy the or shall satisfy the following requirer
2375 2376	(1) All welds at the beam-to-column of Chapter 3 of ANSI/AISC 358.
2377 2378	(2) Beam flanges shall be connected penetration groove welds.
2379 2380 2381	(3) The shape of weld access holes so of AWS D1.8/D1.8M. Weld accessordance with clause 6.10.2 of
2382	(4) Continuity plates shall satisfy the
2383 2384 2385 2386 2387 2388	Exception: The welded joints flanges are permitted to be comp sided partial-joint-penetration gr sided fillet welds, or combinat welds and fillet welds. The requiess than the available strength
2389 2390 2391 2392 2393	column flange.  (5) The beam web shall be connected groove weld extending between well as the connection designed Section E1.6b(a).
2394 2395	User Note: For FR moment connection
2396	checked in accordance with Specifica
2397	strength of the panel zone should be ba
2398	from the load combinations stipulated
2399	including the overstrength seismic load.

nection; in other words,  $1.1R_vM_n/\alpha_s$  or the d to the connection by the system.

- ide-flange beams and the flange of widehe requirements of Section E2.6 or E3.6, ments:
  - connection shall satisfy the requirements
  - to column flanges using complete-joint-
  - hall be in accordance with clause 6.10.1.2 ess hole quality requirements shall be in AWS D1.8/D1.8M.
  - requirements of Section E3.6f.
    - of the continuity plates to the column plete-joint-penetration groove welds, twooove welds with contouring fillets, twotions of partial-joint-penetration groove ired strength of these joints shall not be of the contact area of the plate with the
  - d to the column flange using either a CJP weld access holes, or using a bolted single for the required shear strength given in

ons, panel zone shear strength should be tion Section J10.6. The required shear sed on the beam end moments computed I by the applicable building code, not including the overstrength seismic load

## **E1.6c. PR Moment Connections**

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PR moment connections shall satisfy the following requirements:

- (a) Connections shall be designed for the maximum moment and shear from the applicable load combinations as described in Sections B2 and B3.
- (b) The stiffness, strength and deformation capacity of PR moment connections shall be considered in the design, including the effect on overall frame stability.
- (c) The nominal flexural strength of the connection,  $M_{n,PR}$ , shall be no less than 50% of M<sub>p</sub> of the connected beam.

2411 2412		Exception: For one-story structures, $M_{n,PR}$ shall be no less than 50% of $M_p$ of the connected column.
2413 2414		(d) $V_u$ or $V_a$ , as applicable, shall be determined per Section E1.6b(a) with $M_p$ in Equation E1-1 taken as $M_{n,PR}$ .
2415	E2.	INTERMEDIATE MOMENT FRAMES (IMF)
2416	E2.1.	Scope
2417 2418		Intermediate moment frames (IMF) of structural steel shall be designed in conformance with this section.
2419	E2.2.	Basis of Design
2420 2421 2422 2423 2424 2425		IMF designed in accordance with these provisions are expected to provide limited inelastic deformation capacity through flexural yielding of the IMF beams and columns, and shear yielding of the column panel zones. Design of connections of beams to columns, including panel zones and continuity plates, shall be based or connection tests that provide the performance required by Section E2.6b, and demonstrate this conformance as required by Section E2.6c.
2426	E2.3.	Analysis
2427		There are no requirements specific to this system.
2428	E2.4.	System Requirements
2429 2430	E2.4a.	. Stability Bracing of Beams
2431 2432		Beams shall be braced to satisfy the requirements for moderately ductile members in Section D1.2a.
2433 2434 2435 2436 2437 2438 2439		In addition, unless otherwise indicated by testing, beam braces shall be placed near concentrated forces, changes in cross section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the IMF. The placement of stability bracing shall be consistent with that documented for a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or in a program of qualification testing in accordance with Section K2.
2440 2441		The required strength of lateral bracing provided adjacent to plastic hinges shall be as required by Section D1.2c.

2442	E2.5.	Members
2443	E2.5a	Basic Requirements
2444 2445		Beam and column members shall satisfy the requirements of Section D1 for moderately ductile members, unless otherwise qualified by tests.
2446 2447		Structural steel beams in IMF are permitted to be composite with a reinforced concrete slab to resist gravity loads.
2448	E2.5b	. Beam Flanges
2449 2450 2451 2452 2453 2454 2455 2456		Changes in beam flange area in the protected zones, as defined in Section E2.5c shall be gradual. The drilling of flange holes or trimming of beam flange width is not permitted unless testing or qualification demonstrates that the resulting configuration is able to develop stable plastic hinges to accommodate the required story drift angle. The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or in a program of qualification testing in accordance with Section K2.
2457	E2.5c.	Protected Zones
2458 2459 2460 2461 2462 2463		The region at each end of the beam subject to inelastic straining shall be designated as a protected zone, and shall satisfy the requirements of Section D1.3 The extent of the protected zone shall be as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or as determined in a program of qualification testing in accordance with Section K2.
2464 2465 2466 2467 2468 2469		<b>User Note:</b> The plastic hinging zones at the ends of IMF beams should be treated as protected zones. The plastic hinging zones should be established as part of a prequalification or qualification program for the connection, in accordance with Section E2.6c. In general, for unreinforced connections, the protected zone will extend from the face of the column to one half of the beam depth beyond the plastic hinge point.
2470	E2.6.	Connections
2471	E2.6a	Demand Critical Welds
2472 2473		The following welds are demand critical welds, and shall satisfy the requirements of Section A3.4b and I2.3:
2474		(a) Groove welds at column splices
2475 2476 2477		(b) Welds at column-to-base plate connections

2478 2479	Exception: Welds need not be considered demand critical when both of the following conditions are satisfied:
2480 2481	(1) Column hinging at, or near, the base plate is precluded by conditions of restraint, and
2482 2483	(2) There is no net tension under load combinations including the overstrength seismic load.
2484 2485 2486 2487 2488	(c) Complete-joint-penetration groove welds of beam flanges and beam webs to columns, unless otherwise designated by ANSI/AISC 358, or otherwise determined in a connection prequalification in accordance with Section K1, or as determined in a program of qualification testing in accordance with Section K2.
2489 2490 2491 2492 2493 2494 2495	<b>User Note:</b> For the designation of demand critical welds, standards such as ANSI/AISC 358 and tests addressing specific connections and joints should be used in lieu of the more general terms of these Provisions. Where these Provisions indicate that a particular weld is designated demand critical, but the more specific standard or test does not make such a designation, the more specific standard or test should govern. Likewise, these standards and tests may designate welds as demand critical that are not identified as such by these Provisions.
2496	E2.6b. Beam-to-Column Connection Requirements
2497 2498	Beam-to-column connections used in the SFRS shall satisfy the following requirements:
2499 2500	(a) The connection shall be capable of accommodating a story drift angle of at least 0.02 rad.
2501 2502 2503	(b) The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80 M_p$ of the connected beam at a story drift angle of $0.02 \text{ rad}$ .
2504	E2.6c. Conformance Demonstration
2505 2506	Beam-to-column connections used in the SFRS shall satisfy the requirements of Section E2.6b by one of the following:
2507	(a) Use of IMF connections designed in accordance with ANSI/AISC 358.
2508	(b) Use of a connection prequalified for IMF in accordance with Section K1.
2509 2510 2511	(c) Provision of qualifying cyclic test results in accordance with Section K2. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:
2512 2513 2514	(1) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section K2

2515 2516 2517 2518	(2) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations and matching connection processes, within the limits specified in Section K2.
2519	E2.6d. Required Shear Strength
2520 2521 2522	The required shear strength of the connection shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, $E_{cl}$ , shall be taken as:
2523	$E_{cl} = 2\left[1.1R_{y}M_{p}\right]/L_{h} $ (E2-1)
2524 2525 2526 2527 2528 2529 2530 2531 2532 2533	where  L <sub>h</sub> = distance between beam plastic hinge locations as defined within the test report or ANSI/AISC 358, in. (mm)  M <sub>p</sub> = F <sub>y</sub> Z = plastic flexural strength, kip-in. (N-mm)  R <sub>y</sub> = ratio of the expected yield stress to the specified minimum yield stress, F <sub>y</sub> Exception: In lieu of Equation E2-1, the required shear strength of the connection shall be as specified in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or in a program of
<ul><li>2534</li><li>2535</li></ul>	qualification testing in accordance with Section K2.  E2.6e. Panel Zone
2536	There are no additional panel zone requirements.
2537 2538 2539 2540 2541	<b>User Note:</b> Panel zone shear strength should be checked in accordance with Section J10.6 of the Specification. The required shear strength of the panel zone should be based on the beam end moments computed from the load combination stipulated by the applicable building code, not including the overstrength seismic load.
2542	E2.6f. Continuity Plates
2543 2544	Continuity plates shall be provided in accordance with the provisions of Section E3.6f.
2545	E2.6g. Column Splices
2546	Column splices shall comply with the requirements of Section E3.6g.

2547	E3.	SPECIAL MOMENT FRAMES (SMF)
2548	E3.1.	Scope
2549 2550		Special moment frames (SMF) of structural steel shall be designed in conformance with this section.
2551	E3.2.	Basis of Design
2552 2553 2554 2555 2556 2557 2558 2559 2560 2561 2562		SMF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through flexural yielding of the SMF beams and limited yielding of column panel zones, or, where equivalent performance of the moment frame system is demonstrated by substantiating analysis and testing, through yielding of the connections of beams to columns Except where otherwise permitted in this section, columns shall be designed to be stronger than the fully yielded and strain-hardened beams or girders. Flexural yielding of columns at the base is permitted. Design of connections of beams to columns, including panel zones and continuity plates, shall be based or connection tests that provide the performance required by Section E3.6b, and demonstrate this conformance as required by Section E3.6c.
2563	E3.3.	Analysis
2564 2565		For special moment frame systems that consist of isolated planar frames, there are no additional analysis requirements.
2566 2567 2568 2569		For moment frame systems that include columns that form part of two intersecting special moment frames in orthogonal or multi-axial directions, the column analysis of Section E3.4a shall consider the potential for beam yielding in both orthogonal directions simultaneously.
2570 2571		<b>User Note:</b> For these columns, the required axial loads are defined in Section D1.4a(b).
2572	E3.4.	System Requirements
2573	E3.4a	. Moment Ratio
2574		The following relationship shall be satisfied at beam-to-column connections:
2575		$\frac{\Sigma M_{pc}^*}{\Sigma M_{pb}^*} > 1.0 \tag{E3-1}$
2576		where
2577 2578 2579 2580 2581		$\Sigma M_{pc}^*$ = sum of the projections of the nominal flexural strengths of the columns (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column, kip-in. (N-mm). It is permitted to determine $\Sigma M_{pc}^*$ as follows:
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2582	$= \Sigma Z_{c} \left( F_{yc} - \alpha_{s} P_{r} / A_{g} \right) $ (E3-2)
2583 2584	When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines shall be used.
2585 2586 2587	$\Sigma M_{pb}^*$ = sum of the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline, kip-in. (N-mm). It is permitted to determine $\Sigma M_{pb}^*$ as follows:
2588	$= \Sigma \left( \mathbf{M}_{pr} + \alpha_{s} \mathbf{M}_{v} \right) \tag{E3-3}$
2589 2590 2591 2592 2593 2594	<ul> <li>M<sub>pr</sub> = probable maximum moment at the location of the plastic hinge, as determined in accordance with ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or in a program of qualification testing in accordance with Section K2, kip-in. (N-mm)</li> <li>A<sub>g</sub> = gross area of column, in.<sup>2</sup> (mm<sup>2</sup>)</li> </ul>
2595	$F_{yb}$ = specified minimum yield stress of beam, ksi (MPa)
2596	F <sub>yc</sub> = specified minimum yield stress of column, ksi (MPa)
2597 2598 2599	<ul> <li>M<sub>v</sub> = additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on LRFD or ASD load combinations, kip-in. (N-mm)</li> </ul>
2600 2601	$Z_c$ = plastic section modulus of the column about the axis of bending, in. <sup>3</sup> (mm <sup>3</sup> )
2602	$P_r$ = required compressive strength according to Section D1.4a, kips (N)
2603 2604	Exception: The requirement of Equation E3-1 shall not apply if the following conditions in (a) or (b) are satisfied.
2605 2606 2607	(a) Columns with $P_{\rm rc} < 0.3 P_c$ for all load combinations other than those determined using the overstrength seismic load and that satisfy either of the following:
2608 2609	(1) Columns used in a one-story building or the top story of a multistory building.
2610 2611 2612 2613 2614 2615 2616 2617 2618	(2) Columns where: (1) the sum of the available shear strengths of all exempted columns in the story is less than 20% of the sum of the available shear strengths of all moment frame columns in the story acting in the same direction; and (2) the sum of the available shear strengths of all exempted columns on each moment frame column line within that story is less than 33% of the available shear strength of all moment frame columns on that column line. For the purpose of this exception, a column line is defined as a single line of columns or parallel lines of columns located within 10% of the plan dimension perpendicular to the line of columns.

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2620 2621 2622 2623 2624	<b>User Note:</b> For purposes of this exception, the available shear strengths of the columns should be calculated as the limit strengths considering the flexural strength at each end as limited by the flexural strength of the attached beams, or the flexural strength of the columns themselves, divided by H, where H is the story height.
2625	The nominal compressive strength, Pc, shall be
2626	$P_{c} = F_{yc} A_{g} / \alpha_{s} $ (E3-5)
2627	and $P_{rc} = P_{uc}$ (LRFD) or $P_{rc} = P_{ac}$ (ASD), as applicable .
2628 2629 2630	(b) Columns in any story that has a ratio of available shear strength to required shear strength that is 50% greater than the story above.
2631	E3.4b. Stability Bracing of Beams
2632 2633 2634	Beams shall be braced to satisfy the requirements for highly ductile members Section D1.2b.
2635 2636 2637	In addition, unless otherwise indicated by testing, beam braces shall be placed near concentrated forces, changes in cross section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of
2638 2639 2640 2641	the SMF. The placement of lateral bracing shall be consistent with that documented for a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or in a program of qualification testing in accordance with Section K2.
2642 2643	The required strength and stiffness of stability bracing provided adjacent to plastic hinges shall be as required by Section D1.2c.
2644	E3.4c. Stability Bracing at Beam-to-Column Connections
2645	1. Braced Connections
2646 2647 2648 2649 2650	When the webs of the beams and column are coplanar, and a column is shown to remain elastic outside of the panel zone, column flanges at beam-to-column connections shall require stability bracing only at the level of the top flanges of the beams. It is permitted to assume that the column remains elastic when the ratio calculated using Equation E3-1 is greater than 2.0.
2651 2652	When a column cannot be shown to remain elastic outside of the panel zone, the following requirements shall apply:
2653 2654 2655	(a) The column flanges shall be laterally braced at the levels of both the top and bottom beam flanges. Stability bracing is permitted to be either direct or indirect.  2016 Seismic Provisions for Structural Steel Buildings Draft dated December 18, 2015 American Institute of Steel Construction

2656 2657 2658 2659 2660 2661 2662	<b>User Note:</b> Direct stability bracing of the column flange is achieved through use of member braces or other members, deck and slab, attached to the column flange at or near the desired bracing point to resist lateral buckling. Indirect stability bracing refers to bracing that is achieved through the stiffness of members and connections that are not directly attached to the column flanges, but rather act through the column web or stiffener plates.
2663 2664 2665	(b)Each column-flange member brace shall be designed for a required strength that is equal to 2% of the available beam flange strength divided by $\alpha_s$ , $F_y b_f t_{bf}/\alpha_s$ .
2666	2. Unbraced Connections
2667 2668 2669 2670 2671	A column containing a beam-to-column connection with no member bracing transverse to the seismic frame at the connection shall be designed using the distance between adjacent member braces as the column height for buckling transverse to the seismic frame and shall conform to Specification Chapter H, except that:
2672 2673 2674	(a) The required column strength shall be determined from the load combinations in the applicable building code that include the overstrength seismic load.
2675 2676 2677	The overstrength seismic load, $E_{mh}$ , need not exceed 125% of the frame available strength based upon either the beam available flexural strength or panel zone available shear strength.
2678	(b) The slenderness L/r for the column shall not exceed 60
2679 2680 2681	where  L = length of column, in. (mm)  r = governing radius of gyration, in. (mm)
2682 2683 2684 2685	(c) The column required flexural strength transverse to the seismic frame shall include that moment caused by the application of the beam flange force specified in Section E3.4c(1)(b) in addition to the second-order moment due to the resulting column flange lateral displacement.
2686	E3.5. Members
2687 2688 2689 2690	E3.5a. Basic Requirements  Beam and column members shall satisfy the requirements of Section D1.1 for highly ductile members, unless otherwise qualified by tests.
2691 2692	Structural steel beams in SMF are permitted to be composite with a reinforced concrete slab to resist gravity loads.

2693	E3.5b	. Beam Flanges
2694 2695 2696 2697 2698 2699 2700 2701		Abrupt changes in beam flange area are prohibited in plastic hinge regions. The drilling of flange holes or trimming of beam flange width are not permitted unless testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges to accommodate the required story drift angle. The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or in a program of qualification testing in accordance with Section K2.
2703	E3.5c.	Protected Zones
2704 2705 2706 2707 2708 2709 2710		The region at each end of the beam subject to inelastic straining shall be designated as a protected zone, and shall satisfy the requirements of Section D1.3. The extent of the protected zone shall be as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or as determined in a program of qualification testing in accordance with Section K2.
2711 2712 2713 2714 2715 2716		<b>User Note:</b> The plastic hinging zones at the ends of SMF beams should be treated as protected zones. The plastic hinging zones should be established as part of a prequalification or qualification program for the connection, per Section E3.6c. In general, for unreinforced connections, the protected zone will extend from the face of the column to one half of the beam depth beyond the plastic hinge point.
2717	E3.6.	Connections
2718	E3.6a.	Demand Critical Welds
2719 2720		The following welds are demand critical welds, and shall satisfy the requirements of Section A3.4b and I2.3:
2721		(a) Groove welds at column splices
2722		(b) Welds at column-to-base plate connections
2723 2724		Exception: Welds need not be considered demand critical when both of the following conditions are satisfied:
2725 2726		(1) Column hinging at, or near, the base plate is precluded by conditions of restraint, and
2727 2728		(2) There is no net tension under load combinations including the overstrength seismic load.

(c) Complete-joint-penetration groove welds of beam flanges and beam webs to columns, unless otherwise designated by ANSI/AISC 358, or otherwise determined in a connection prequalification in accordance with Section K1, or as determined in a program of qualification testing in accordance with Section K2.
<b>User Note:</b> For the designation of demand critical welds, standards such as ANSI/AISC 358 and tests addressing specific connections and joints should be used in lieu of the more general terms of these Provisions. Where these Provisions indicate that a particular weld is designated demand critical, but the more specific standard or test does not make such a designation, the more specific standard or test consistent with the requirements in Chapter K should govern. Likewise, these standards and tests may designate welds as demand critical that are not identified as such by these Provisions.
E3.6b. Beam-to-Column Connections
Beam-to-column connections used in the seismic force-resisting system (SFRS) shall satisfy the following requirements:
(a) The connection shall be capable of accommodating a story drift angle of at least 0.04 rad.
(b) The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80 M_p$ of the connected beam at a story drift angle of 0.04 rad, unless equivalent performance of the moment frame system is demonstrated through substantiating analysis conforming to SEI/ASCE 7 Sections 12.2.1.1 or 12.2.1.2.
E3.6c. Conformance Demonstration
Beam-to-column connections used in the SFRS shall satisfy the requirements of Section E3.6b by one of the following:
(a) Use of SMF connections designed in accordance with ANSI/AISC 358.
(b) Use of a connection prequalified for SMF in accordance with Section K1.
(c) Provision of qualifying cyclic test results in accordance with Section K2. Results of at least two cyclic connection tests shall be provided and shall be based on one of the following:
(1) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section K2
(2) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Section K2

### 2768 E3.6d. Required Shear Strength 2769 The required shear strength of the connection shall be determined using the 2770 capacity-limited seismic load effect. The capacity-limited horizontal seismic load 2771 effect, E<sub>cl</sub>, shall be taken as: $E_{cl} = 2M_{pr}/L_h$ 2772 (E3-6)2773 where 2774 $L_h$ = distance between plastic hinge locations as defined within the test 2775 report or ANSI/AISC 358, in. (mm) 2776 = maximum probable moment at the plastic hinge location, as defined in $M_{pr}$ 2777 Section E3.4a, kip-in. (N-mm) 2778 2779 When E<sub>cl</sub> as defined in Equation E3-6 is used in ASD load combinations that are 2780 additive with other transient loads and that are based on ASCE/SEI 7, the 0.75 2781 combination factor for transient loads shall not be applied to Ecl. 2782 Where the exceptions to Equation E3-1 in Section E3.4a apply, the shear, Ecl, is 2783 permitted to be calculated based on the beam end moments corresponding to the expected flexural strength of the column multiplied by 1.1. 2784 2785 E3.6e. Panel Zone 2786 1. Required Shear Strength 2787 The required shear strength of the panel zone shall be determined from the 2788 summation of the moments at the column faces as determined by projecting 2789 the expected moments at the plastic hinge points to the column faces. The 2790 design shear strength shall be $\phi_v R_n$ and the allowable shear strength shall be 2791 $R_n/\Omega_v$ where $\phi_{\rm v} = 1.00 \, ({\rm LRFD}) \quad \Omega_{\rm v} = 1.50 \, ({\rm ASD})$ 2792 2793 and the nominal shear strength, R<sub>n</sub>, in accordance with the limit state of shear 2794 yielding, is determined as specified in Specification Section J10.6. 2795 Alternatively, the required thickness of the panel zone shall be determined in 2796 accordance with the method used in proportioning the panel zone of the tested 2797 or prequalified connection. 2798 Where the exceptions to Equation E3-1 in Section E3.4a apply, the beam 2799 moments used in calculating the required shear strength of the panel zone 2800 need not exceed those corresponding to the expected flexural strength of the 2801 column multiplied by 1.1.

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## 2. Panel Zone Thickness

The individual thicknesses, t, of column web and doubler plates, if used, shall conform to the following requirement:

$$t \ge \left(d_z + w_z\right) / 90 \tag{E3-7}$$

where

 $d_z = d - 2t_f$  of the deeper beam at the connection, in. (mm)

t = thickness of column web or <u>individual</u> doubler plate, in. (mm)

 $w_z$  = width of panel zone between column flanges, in. (mm)

When plug welds are used to join the doubler to the column web, it is permitted to use the total panel zone thickness to satisfy Equation E3-7. Additionally, the individual thicknesses of the column web and doubler plate shall satisfy Equation E3-7, where  $d_z$  and  $w_z$  are modified to be the distance between plug welds. When plug welds are required, a minimum of four plug welds shall be provided and spaced in accordance with Equation E3-7.

### 3. Panel Zone Doubler Plates

The thickness of doubler plates, if used, shall not be less than 0.25 in. (6 mm).

When used, doubler plates shall meet the following requirements.

Where the required strength of the panel zone exceeds the design strength, or where the panel zone does not comply with Equation E3-7, doubler plates shall be provided. Doubler plates shall be placed in contact with the web, or shall be spaced away from the web. Doubler plates with a gap of up to 1/16 in. (2 mm) between the doubler plate and the column web are permitted to be designed as being in contact with the web. When doubler plates are spaced away from the web, they shall be placed symmetrically in pairs on opposite sides of the column web.

Doubler plates in contact with the web shall be welded to the column flanges either using partial-joint-penetration groove welds in accordance with AWS D1.8/D1.8M clause 4 that extend from the surface of the doubler plate to the column flange, or by using fillet welds. Spaced doubler plates shall be welded to the column flanges using complete-joint-penetration groove welds, partial-joint-penetration groove welds, or fillet welds. The required strength of partial-joint-penetration groove welds or fillet welds shall equal the available shear yielding strength of the doubler plate thickness.

## (a) Doubler plates used without continuity plates

Doubler plates and the welds connecting the doubler plates to the column flanges shall extend at least 6 in. (150 mm) above and below the top and bottom of the deeper moment frame beam. For doubler plates in contact

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with the web, if the doubler plate thickness alone and the column web

thickness alone both satisfy Equation E3-7, then no weld is required along

2845 2846 2847 2848 2849 2850	the top and bottom edges of the doubler plate. If either the doubler plate thickness alone or the column web thickness alone does not satisfy Equation E3-7, then a minimum size fillet weld, as stipulated in Specification Table J2.4, shall be provided along the top and bottom edges of the doubler plate. These welds shall terminate 1.5 in. (75 mm) from the toe of the column fillet.
2851	(b) Doubler plates used with continuity plates
2852 2853	Doubler plates are permitted to be either extended above and below the continuity plates or placed between the continuity plates.
2854	(1) Extended doubler plates
2855 2856 2857 2858 2859 2860 2861	Extended doubler plates shall be in contact with the web. Extended doubler plates and the welds connecting the doubler plates to the column flanges shall extend at least 6 in. (150 mm) above and below the top and bottom of the deeper moment frame beam. Continuity plates shall be welded to the extended doubler plates in accordance with the requirements in Section E3.6f.2(c). No welds are required at the top and bottom edges of the doubler plate.
2862	(2) Doubler plates placed between continuity plates
2863 2864 2865 2866 2867 2868 2869 2870 2871 2872 2873	Doubler plates placed between continuity plates are permitted to be in contact with the web or away from the web. Welds between the doubler plate and the column flanges shall extend between continuity plates, but are permitted to stop no more than 1 in. (25 mm) from the continuity plate. The top and bottom of the doubler plate shall be welded to the continuity plates over the full length of the continuity plates in contact with the column web. The required strength of the doubler plate to continuity plate weld shall equal 75% of the available shear yield strength of the full doubler plate thickness over the contact length with the continuity plate.
2874 2875 2876 2877 2878 2879	<b>User Note:</b> When a beam perpendicular to the column web connects to a doubler plate, the doubler plate should be sized based on the shear from the beam end reaction in addition to the panel zone shear. When welding continuity plates to extended doubler plates, force transfer between the continuity plate and doubler plate must be considered. See commentary for further discussion.
2880	E3.6f. Continuity Plates
2881	Continuity plates shall be provided as required by this section.

Exception: This section shall not apply in the following cases:

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2883 2884		Where continuity plates are otherwise determined in a connection equalification in accordance with Section K1.
2885 2886 2887	co	There a connection is qualified in accordance with Section K2 fo anditions in which the test assembly omits continuity plates and matches the prototype beam and column sizes and beam span.
2888		
2889 2890	1. (	Conditions Requiring Continuity Plates
2891	(	Continuity plates shall be provided in the following cases:
2892 2893 2894 2895 2896 2897	(	Where the required strength at the column face exceeds the available column strength determined using the applicable loca limit states stipulated in Specification Section J10, where applicable. Where so required, continuity plates shall satisfy the requirements of Specification Section J10.8 and the requirement of Section E3.6f.2.
2898 2899 2900 2901		For connections in which the beam flange is welded to the column flange, the column shall have an available strength sufficient to resist an applied force consistent with the probable maximum moment at face of column, $M_{\rm f}$ .
2902		
2903		User Note: The beam flange force, Pf, corresponding to the
2904		probable maximum moment at the column face, M <sub>f</sub> may be
2905		determined as follows:
2906		
2907 2908		For connections with beam webs with a bolted connection to the column, P <sub>f</sub> is permitted to be determined assuming only the bean
2909		flanges participate in transferring the moment $M_f$ : $P_f = \frac{M_f}{\alpha_s d^*}$
2910		
2911		For connections with beam webs welded to the column, Pf is
2912		permitted to be determined assuming that the beam flanges and
2913		web participate proportionally in transferring the moment M <sub>t</sub>
2914		$P_{f} = \frac{0.85M_{f}}{\alpha_{s}d^{*}}$
2915		where
2916		M <sub>f</sub> = probable maximum moment at face of column a
2917		defined in ANSI/AISC 358 for a prequalified momen
2918		connection or as determined from qualification testing
2919		kip-in. (N-mm)
2920 2921		P <sub>f</sub> = required strength at the column face for local limi states in the column, kip (N)
2721		states in the column, kip (14)

2922 2923 2924	d* = distance between centroids of beam flanges or beam flange connections to the face of the column, in. (mm)
2925 2926	(b) Where the column flange thickness is less than the limiting thickness, t <sub>lim</sub> , determined in accordance with this provision.
2927 2928 2929	(1) Where the beam flange is welded to the flange of a wide- flange or built-up I-shaped column, the limiting column- flange thickness is
2930	$t_{lim} = \frac{b_{bf}}{6} \tag{E3-8}$
2931 2932 2933	(2) Where the beam flange is welded to the flange of the I-shape in a boxed wide-flange column, the limiting column-flange thickness is:
2934	$\mathbf{t}_{\text{lim}} = \frac{\mathbf{b}_{\text{bf}}}{12} \tag{E3-9}$
2935 2936 2937 2938 2939 2940 2941 2942	<b>User Note:</b> These continuity plate requirements apply only to wide-flange column sections. Detailed formulas for determining continuity plate requirements for box column shapes have not been developed. It is noted that the performance of moment connections is dependent on the column flange stiffness in distributing the strain across the beam-to-column flange weld. Designers should consider the relative stiffness of the box column flange compared to those of tested assemblies in resisting the beam flange force to determine the need for continuity plates.
2943 2944 2945 2946 2947	2. Continuity Plate Requirements Where continuity plates are required, they shall meet the requirements of this section.
2948	(a) Continuity Plate Width
2949	The width of the continuity plate shall be determined as follows:
2950 2951 2952	(1) For W-shape columns, continuity plates shall, at a minimum, extend from the column web to a point opposite the tips of the wider beam flanges.
2953 2954 2955	(2) For boxed wide flange columns, continuity plates shall extend the full width from column web to side plate of the column.

Continuity Plate Thickness

(b)

2957 2958	The minimum thickness of the plates shall be determined as follows:
2959 2960	(1) For one-sided connections, the continuity plate thickness shall be at least 50% of the thickness of the beam flange.
2961 2962 2963	(2) For two-sided connections, the continuity plate thickness shall be at least equal to 75% of the thickness of the thicker beam flange on either side of the column.
2964	(c) Continuity Plate Welding
2965 2966	Continuity plates shall be welded to column flanges using CJP groove welds.
2967 2968 2969 2970	Continuity plates shall be welded to column webs or extended doubler plates using groove welds or fillet welds. The required strength of the welded joints of continuity plates to the column web or extended doubler plate shall be the lesser of the following:
2971 2972 2973 2974 2975 2976 2977 2978 2979 2980 2981	(1) The sum of the available strengths in tension of the contact areas of the continuity plates to the column flanges that have attached beam flanges  (2) The available strength in shear of the contact area of the plate with the column web or extended doubler plate  (3) The available strength in shear of the column web, when the continuity plate is welded to the column web, or the available strength in shear of the doubler plate, when the continuity plate is welded to an extended doubler plate  E3.6g. Column Splices
2982	Column splices shall comply with the requirements of Section D2.5.
2983 2984 2985 2986	Exception: The required strength of the column splice including appropriate stress concentration factors or fracture mechanics stress intensity factors need not exceed that determined by a nonlinear analysis as specified in Chapter C.
2987 2988	1. Welded column flange splices using complete-joint-penetration groove welds
2989 2990 2991	Where welds are used to make the flange splices, they shall be complete-joint-penetration groove welds, unless otherwise permitted in Section E3.6g.2.
2992 2993 2994	2. Welded column flange splices using partial-joint-penetration groove welds

2995 2996 2997 2998 2999		Where the specified minimum yield stress of the column shafts does not exceed 60 ksi (415 MPa) and the thicker flange is at least 5% thicker than the thinner flange, partial-joint-penetration groove welds are permitted to make the flange splices, and shall comply with the following requirements:
3000 3001 3002		(a) The partial-joint-penetration flange weld or welds shall provide a minimum total effective throat of 85% of the thickness of the thinner column flange.
3003 3004 3005 3006 3007 3008 3009		(b) A smooth transition in the thickness of the weld is provided from the outside of the thinner flange to the outside of the thicker flange. The transition shall be at a slope not greater than 1 in 2.5, and may be accomplished by sloping the weld surface, by chamfering the thicker flange to a thickness no less than 5% greater than the thickness of the thinner flange, or by a combination of these two methods.
3010 3011		(c) Tapered transitions between column flanges of different width shall be provided in accordance with Section D2.5b(c).
3012 3013		(d) Where the flange weld is a double-bevel groove weld (i.e., on both sides of the flange):
3014 3015		(i) The unfused root face shall be centered within the middle half of the thinner flange, and
3016 3017 3018		(ii) Weld access holes that comply with the AISC Specification shall be provided in the column section containing the groove weld preparation.
3019 3020 3021		(e) Where the flange thickness of the thinner flange is not greater than 2.5 in. (64 mm), and the weld is a single-bevel groove weld, weld access holes shall not be required.
3022 3023	3.	Welded column web splices using complete-joint-penetration groove welds
3024 3025 3026 3027		The web weld or welds shall be made in a groove or grooves in the column web that extend to the access holes. The weld end(s) may be stepped back from the ends of the bevel(s) using a block sequence for approximately one weld size.
3028 3029	4.	Welded column web splices using partial-joint-penetration groove welds
3030 3031 3032 3033 3034		When partial-joint-penetration groove welds in column flanges that comply with Section E3.6g.2 are used, and the thicker web is at least 5% thicker than the thinner web, it shall be permitted to use partial-joint-penetration groove welds in column webs, and shall comply with the following requirements:

3035 3036 3037		(a)	The partial-joint-penetration web weld or welds shall provide a minimum total effective throat of 85% of the thickness of the thinner column web.
3038 3039 3040		(b)	A smooth transition in the thickness of the weld shall be provided from the outside of the thinner web to the outside of the thicker web.
3041 3042		(c)	Where the weld is a single-bevel groove, the thickness of the thinner web shall not be greater than $2.5$ in. $(64 \text{ mm})$ .
3043 3044 3045 3046 3047		(d)	Where no access hole is provided, the web weld or welds shall be made in a groove or grooves prepared in the column web extending the full length of the web between the k-areas. The weld end(s) are permitted to be stepped back from the ends of the bevel(s) using a block sequence for approximately one weld size.
3048 3049 3050 3051 3052		(e)	Where an access hole is provided, the web weld or welds shall be made in a groove or grooves in the column web that extend to the access holes. The weld end(s) are permitted to be stepped back from the ends of the bevel(s) using a block sequence for approximately one weld size.
3053		5. Bolted	column splices
3054 3055 3056 3057 3058	Bolted column splices shall have a required flexural strength that is at least equal to $R_y F_y Z_x / \alpha_s$ of the smaller column, where $Z_x$ is the plastic section modulus about the x-axis. The required shear strength of column web splices shall be at least equal to $\Sigma M_{pc}/(\alpha_s H_c)$ , where $\Sigma M_{pc}$ is the sum of the plastic flexural strengths at the top and bottom ends of the column.		
3059	E4.	SPECIAL TR	RUSS MOMENT FRAMES (STMF)
3060	E4.1.	Scope	
3061 3062	Special truss moment frames (STMF) of structural steel shall satisfy the requirements in this Section.		
3063	E4.2. Basis of Design		
3064 3065 3066 3067 3068 3069 3070		significant ine STMF shall b m) and overal outside of the	led in accordance with these provisions are expected to provide elastic deformation capacity within a special segment of the truss. The limited to span lengths between columns not to exceed 65 ft (20 l depth not to exceed 6 ft (1.8 m). The columns and truss segments a special segments shall be designed to remain essentially elastic trees that are generated by the fully yielded and strain-hardened int.
3071	E4.3.	Analysis	

Analysis of STMF shall satisfy the following requirements.

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E4.3. Analysis

### 3073 E4.3a. Special Segment 3074 The required vertical shear strength of the special segment shall be calculated for 3075 the applicable load combinations in the applicable building code. 3076 E4.3b. Nonspecial Segment 3077 The required strength of nonspecial segment members and connections, including 3078 column members, shall be determined using the capacity-limited horizontal seismic load effect. The capacity-limited horizontal seismic load effect, Ecl, shall 3079 3080 be taken as the lateral forces necessary to develop the expected vertical shear 3081 strength of the special segment acting at mid-length and defined in Section E4.5c. 3082 Second order effects at maximum design drift shall be included. 3083 **E4.4.** System Requirements 3084 E4.4a. Special Segment Each horizontal truss that is part of the SFRS shall have a special segment that is 3085 located between the quarter points of the span of the truss. The length of the 3086 special segment shall be between 0.1 and 0.5 times the truss span length. The 3087 3088 length-to-depth ratio of any panel in the special segment shall neither exceed 1.5 3089 nor be less than 0.67. 3090 Panels within a special segment shall either be all Vierendeel panels or all X-3091 braced panels; neither a combination thereof nor the use of other truss diagonal 3092 configurations is permitted. Where diagonal members are used in the special 3093 segment, they shall be arranged in an X pattern separated by vertical members. 3094 Diagonal members within the special segment shall be made of rolled flat bars of identical sections. Such diagonal members shall be interconnected at points 3095 3096 where they cross. The interconnection shall have a required strength equal to 0.25 3097 times the nominal tensile strength of the diagonal member. Bolted connections 3098 shall not be used for diagonal members within the special segment. 3099 Splicing of chord members is not permitted within the special segment, nor within 3100 one-half the panel length from the ends of the special segment. 3101 The required axial strength of the diagonal web members in the special segment 3102 due to dead and live loads within the special segment shall not exceed 3103 $0.03F_vA_o/\alpha_s$ E4.4b. Stability Bracing of Trusses 3104 3105 Each flange of the chord members shall be laterally braced at the ends of the 3106 special segment. The required strength of the lateral brace shall be $P_r = 0.06R_vF_vA_f/\alpha_s$ 3107 (E4-1)

3108	
3109 3110 3111	where $A_f = gross$ area of the flange of the special segment chord member, in. <sup>2</sup> $(mm^2)$
3112	E4.4c. Stability Bracing of Truss-to-Column Connections
3113 3114 3115	The columns shall be laterally braced at the levels of top and bottom chords of the trusses connected to the columns. The lateral braces shall have a required strength of
3116	$P_{\rm r} = 0.02 R_{\rm y} P_{\rm nc} / \alpha_{\rm s} \tag{E4-2}$
3117 3118 3119	where $P_{nc}$ = nominal compressive strength of the chord member at the ends, kips (N)
3120	E4.4d. Stiffness of Stability Bracing
3121 3122	The required brace stiffness shall meet the provisions of Specification Appendix 6, Section 6.2, where
3123	$P_{\rm r} = R_{\rm y} P_{\rm nc} / \alpha_{\rm s} \tag{E4-2}$
3124 3125	where $P_r$ = required axial compressive strength, kips (N)
3126 3127 3128	E4.5. Members
3129 3130	E4.5a. Basic Requirements
3131 3132 3133	Columns shall satisfy the requirements of Section D1.1 for highly ductile members.
3134 3135	E4.5b. Special Segment Members
3136 3137 3138 3139 3140 3141 3142	The available shear strength of the special segment shall be calculated as the sum of the available shear strength of the chord members through flexure, and of the shear strength corresponding to the available tensile strength and 0.3 times the available compressive strength of the diagonal members, when they are used. The top and bottom chord members in the special segment shall be made of identical sections and shall provide at least 25% of the required vertical shear strength.

3143 3144 3145	The available strength, $\phi P_n$ (LRFD) and $P_n/\Omega$ (ASD), determined in accordance with the limit state of tensile yielding, shall be equal to or greater than 2.2 times the required strength, where
3146	$\phi = 0.90 \text{ (LRFD)}$ $\Omega = 1.67 \text{ (ASD)}$
3147 3148 3149 3150	$P_{n} = F_{y} A_{g} \eqno(E4-4)$ E4.5c. Expected Vertical Shear Strength of Special Segment
3151 3152	The expected vertical shear strength of the special segment, $V_{\text{ne}}$ , at mid-length, shall be:
3153	$V_{ne} = \frac{3.60R_{y}M_{nc}}{L_{s}} + 0.036EI\frac{L}{L_{s}^{3}} + R_{y}(P_{nt} + 0.3P_{nc})\sin\alpha $ (E4-5)
3154 3155	where E = modulus of elasticity of a chord member of the special segment, ksi
3156 3157 3158 3159	(MPa)  I = moment of inertia of a chord member of the special segment, in. 4 (mm <sup>4</sup> )  L = span length of the truss, in. (mm)
3160 3161 3162 3163	<ul> <li>L<sub>s</sub> = length of the special segment, in. (mm)</li> <li>M<sub>nc</sub> = nominal flexural strength of a chord member of the special segment, kip-in. (N-mm)</li> <li>P<sub>nt</sub> = nominal tensile strength of a diagonal member of the special segment,</li> </ul>
3164 3165 3166 3167 3168 3169	<ul> <li>kips (N)</li> <li>P<sub>nc</sub> = nominal compressive strength of a diagonal member of the special segment, kips (N)</li> <li>R<sub>y</sub> = ratio of the expected yield stress to the specified minimum yield stress α = angle of diagonal members with the horizontal, degrees</li> </ul>
3170	E4.5d. Width-to-Thickness Limitations
3171 3172 3173 3174	Chord members and diagonal web members within the special segment shall satisfy the requirements of Section D1.1b for highly ductile members. The width-to-thickness ratio of flat bar diagonal members shall not exceed 2.5.
3174	E4.5e. Built-Up Chord Members
3176 3177 3178 3179	Spacing of stitching for built-up chord members in the special segment shall not exceed $0.04 {\rm Er_y/F_y}$ , where ${\rm r_y}$ is the radius of gyration of individual components about their weak axis.

3180	E4.5f.	Protected Zones			
3181 3182 3183 3184 3185 3186	The region at each end of a chord member within the special segment shall be designated as a protected zone meeting the requirements of Section D1.3. The protected zone shall extend over a length equal to two times the depth of the chord member from the connection with the web members. Vertical and diagona web members from end-to-end of the special segments shall be protected zones.				
3187	E4.6.	Connections			
3188	E4.6a.	4.6a. Demand Critical Welds			
3189 3190		The following welds are demand critical welds, and shall satisfy the requirements of Section A3.4b and I2.3:			
3191	(a) Groove welds at column splices				
3192	(b) Welds at column-to-base plate connections				
3193 3194	Exception: Welds need not be considered demand critical when both of the following conditions are satisfied:				
3195 3196	(1) Column hinging at, or near, the base plate is precluded by conditions of restraint, and				
3197 3198		(2) There is no net tension under load combinations including the overstrength seismic load.			
3199 3200	E4.6b.	Connections of Diagonal Web Members in the Special Segment			
3201 3202 3203 3204		The end connection of diagonal web members in the special segment shall have a required strength that is at least equal to the expected yield strength of the web member, determined as $R_yF_yA_g/\alpha_s$ .			
3205	E4.6c.	Column Splices			
3206 3207		Column splices shall comply with the requirements of Section E3.6g.			
3208	E5.	ORDINARY CANTILEVER COLUMN SYSTEMS (OCCS)			
3209	E5.1.	Scope			
3210 3211 3212		Ordinary cantilever column systems (OCCS) of structural steel shall be designed in conformance with this section.			

3213	E5.2.	Basis of Design		
3214 3215 3216		OCCS designed in accordance with these provisions are expected to provide minimal inelastic drift capacity through flexural yielding of the columns.		
3217	E5.3.	Analysis		
3218 3219		There are no requirements specific to this system.		
3220	E5.4.	System Requirements		
3221	E5.4a.	. Columns		
3222 3223 3224 3225		Columns shall be designed using the load combinations including the overstrength seismic load. The required axial strength, $P_{\rm rc}$ , shall not exceed 15% of the available axial strength, $P_{\rm c}$ , for these load combinations only.		
3226	E5.4b	Stability Bracing of Columns		
3227 3228		There are no additional stability bracing requirements for columns.		
3229	E5.5.	Members		
3230	E5.5a.	5.5a. Basic Requirements		
3231 3232		There are no additional requirements.		
3233	E5.5b.	. Column Flanges		
3234 3235		There are no additional column flange requirements.		
3236	E5.5c.	Protected Zones		
3237 3238		There are no designated protected zones.		
3239 3240	E5.6	Connections		
3241	E5.6a.	Demand Critical Welds		
3242 3243		No demand critical welds are required for this system.		
3244	E5.6b	Column Bases		
3245 3246		Column bases shall be designed in accordance with Section D2.6.		
3247 3248	E6.	SPECIAL CANTILEVER COLUMN SYSTEMS (SCCS)		

3249	E6.1.	Scope
3250 3251 3252		Special cantilever column systems (SCCS) of structural steel shall be designed in conformance with this section.
3253	E6.2.	Basis of Design
3254 3255 3256		SCCS designed in accordance with these provisions are expected to provide limited inelastic drift capacity through flexural yielding of the columns.
3257	E6.3.	Analysis
3258 3259		There are no requirements specific to this system.
3260	E6.4.	System Requirements
3261	E6.4a.	Columns
3262 3263 3264 3265		Columns shall be designed using the load combinations including the overstrength seismic load. The required strength, $P_{\rm rc}$ , shall not exceed 15% of the available axial strength, $P_c$ , for these load combinations only.
3266	E6.4b.	. Stability Bracing of Columns
3267 3268		Columns shall be braced to satisfy the requirements applicable to beams classified as moderately ductile members in Section D1.2a.
3269 3270	E6.5.	Members
3271	E6.5a.	Basic Requirements
3272 3273 3274		Column members shall satisfy the requirements of Section D1.1 for highly ductile members.
3275	E6.5b.	. Column Flanges
3276 3277 3278		Abrupt changes in column flange area are prohibited in the protected zone as designated in Section E6.5c.
3279	E6.5c.	Protected Zones
3280 3281 3282 3283		The region at the base of the column subject to inelastic straining shall be designated as a protected zone, and shall satisfy the requirements of Section D1.3. The length of the protected zone shall be two times the column depth.
3284	E6.6.	Connections
3285 3286	E6.6a.	Demand Critical Welds

3287	The following welds are demand critical welds, and shall satisfy the requirements
3288	of Section A3.4b and I2.3:
3289	
3290	(a) Groove welds at column splices
3291	(b) Welds at column-to-base plate connections

# E6.6b. Column Bases

3292 3293

3294

Column bases shall be designed in accordance with Section D2.6.



3400	CHAPTER F			
3401	BRACED FRAME AND SHEAR WALL SYSTEMS			
3402 3403	This chapter provides the basis of design, the requirements for analysis, and the requirements for the system, members and connections for steel braced-frame and shear-wall systems.			
3404 3405 3406 3407 3408 3409 3410	The ch	F1. Ordinary Concentrically Braced Frames (OCBF) F2. Special Concentrically Braced Frames (SCBF) F3. Eccentrically Braced Frames (EBF) F4. Buckling-Restrained Braced Frames (BRBF) F5. Special Plate Shear Walls (SPSW)		
3411 3412	<b>User Note:</b> The requirements of this chapter are in addition to those required by the Specification and the applicable building code.			
3413	F1.	ORDINARY CONCENTRICALLY BRACED FRAMES (OCBF)		
3414	F1.1.	Scope		
3415 3416		Ordinary concentrically braced frames (OCBF) of structural steel shall be designed in conformance with this section.		
3417	F1.2.	Basis of Design		
3418 3419 3420		This section is applicable to braced frames that consist of concentrically connected members. Eccentricities less than the beam depth are permitted if they are accounted for in the member design by determination of eccentric moments using the overstrength seismic load.		
3421 3422		OCBF designed in accordance with these provisions are expected to provide limited inelastic deformation capacity in their members and connections.		
3423	F1.3.	Analysis		
3424		There are no additional analysis requirements.		
3425	F1.4.	System Requirements		
3426	F1.4a.	V-Braced and Inverted V-Braced Frames		
3427 3428		Beams in V-type and inverted V-type OCBF shall be continuous at brace connections away from the beam-column connection and shall satisfy the following requirements:		
3429 3430 3431 3432		(a) The required strength of the beam shall be determined assuming that the braces provide no support of dead and live loads. For load combinations that include earthquake effects, the seismic load effect, E, on the beam shall be determined as follows:		

3433		(1)	The forces in braces in tension shall be assumed to be the least of the following:
3434			(i) The load effect based upon the overstrength seismic load
3435			(ii) The maximum force that can be developed by the system
3436		(2)	The forces in braces in compression shall be assumed to be equal to $0.3P_{n}$
3437 3438 3439	(	brac	a minimum, one set of lateral braces is required at the point of intersection of the res, unless the member has sufficient out-of-plane strength and stiffness to ensure ility between adjacent brace points.
3440	F1.4b.K-Braced Frames		
3441	K	L-type brac	eed frames shall not be used for OCBF.
3442	F1.4c. M	Aulti-tiere	d Braced Frames
3443 3444			y concentrically braced frame is permitted to be configured as a multi-tiered ne (MT-OCBF) when the following requirements are satisfied.
3445	(8	a) Brac	ces shall be used in opposing pairs at every tier level.
3446	(1	b) Brac	ced frames shall be configured with in-plane struts at each tier level.
3447	(0	e) Coli	umns shall be torsionally braced at every strut-to-column connection location.
3448 3449 3450 3451 3452 3453 3454	((	construction const	r Note: The requirements for torsional bracing are typically satisfied by necting the strut to the column to restrain torsional movement of the column. The trip must have adequate flexural strength and stiffness and an appropriate nection to the column to perform this function.  The required strength of brace connections shall be determined from the load abinations of the applicable building code, including the overstrength seismic law the horizontal seismic load effect, E, multiplied by a factor of 1.5.
3455 3456 3457 3458 3459	(6	com load tens	required axial strength of the struts shall be determined from the load abinations of the applicable building code, including the overstrength seismic l, with the horizontal seismic load effect, E, multiplied by a factor of 1.5. In ion-compression X-bracing, these forces shall be determined in the absence of appression braces.
3460 3461 3462	(1	com	required axial strengths of the columns shall be determined from the load binations of the applicable building code, including the overstrength seismic l, with the horizontal seismic load effect, E, multiplied by a factor of 1.5.
3463 3464 3465 3466 3467 3468	(§	to re As a plan	all load combinations, columns subjected to axial compression shall be designed exist bending moments due to second-order and geometric imperfection effects. A minimum, imperfection effects are permitted to be represented by an out-of-the horizontal notional load applied at every tier level and equal to 0.006 times the ical load contributed by the compression brace connecting the column at the tier of.

3469 3470		(h) When tension-only bracing is used, requirements (d), (e) and (f) need not be satisfied if:
3471 3472 3473 3474 3475 3476		<ul> <li>(1) All braces have a controlling slenderness ratio of 200 or more.</li> <li>(2) The braced frame columns are designed to resist additional in-plane bending moments due to the unbalanced lateral forces determined at every tier level using the capacity-limited seismic load based on expected brace strengths. The expected brace strength in tension is R<sub>y</sub>F<sub>y</sub>A<sub>g</sub>, where</li> </ul>
3477 3478 3479		$F_y$ = specified minimum yield stress, ksi (MPa) $R_y$ = ratio of the expected yield stress to the specified minimum yield stress, $F_y$
3480 3481 3482		The unbalanced lateral force at any tier level shall not be less than 5% of the larger horizontal brace component resisted by the braces below and above the tier level.
3483	F1.5.	Members
3484 3485	F1.5a.	<b>Basic Requirements</b> Braces shall satisfy the requirements of Section D1.1 for moderately ductile members.
3486 3487 3488		Exception: Braces in tension-only frames with slenderness ratios greater than 200 need not comply with this requirement.
3489	F1.5b.	Slenderness
3490		Braces in V or inverted-V configurations shall have $\frac{L_c}{r} \le 4\sqrt{E/F_y}$ .
3491		where
3492		E = modulus of elasticity of steel, ksi (MPa)
3493		$L_c$ = effective length of brace = KL, in. (mm)
3494		K = effective length factor
3495		r = governing radius of gyration, in. (mm)
3496	F1.5c.	Beams
3497 3498 3499		The required strength of beams and their connections shall be determined using the overstrength seismic load.
3500	F1.6.	Connections
3501	F1.6a.	<b>Brace Connections</b>
3502 3503		The required strength of diagonal brace connections shall be determined using the overstrength seismic load.

3504		Exception: The required strength of the brace connection need not exceed the following:
3505 3506 3507		(a) In tension, the expected yield strength divided by $\alpha_s$ , which shall be determined as $R_y F_y A_g / \alpha_s$ where $\alpha_s = LRFD$ -ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD.
3508 3509 3510 3511 3512		In compression, the expected brace strength in compression divided by $\alpha_s$ , which is permitted to be taken as the lesser of $R_y F_y A_g / \alpha_s$ and $1.1 F_{cre} A_g / \alpha_s$ , where $F_{cre}$ is determined from Specification Chapter E using the equations for $F_{cr}$ , except that the expected yield stress $R_y F_y$ is used in lieu of $F_y$ . The brace length used for the determination of $F_{cre}$ shall not exceed the distance from brace end to brace end.
3513 3514 3515		(c) When oversized holes are used, the required strength for the limit state of bolt slip need not exceed the seismic load effect based upon the load combinations without overstrength as stipulated by the applicable building code.
3516 3517 <b>F1.</b>	.7.	Ordinary Concentrically Braced Frames above Seismic Isolation Systems
3518 3519		OCBF above the isolation system shall satisfy the requirements of this section and of Section F1 except for Section F1.4a.
3520 <b>F1</b> .	.7a.	System Requirements
3521		Beams in V-type and inverted V-type braced frames shall be continuous between columns.
3522 <b>F1</b> .	.7b.	Members
3523		Braces shall have a slenderness ratio, $L_c / r \le 4\sqrt{E/F_y}$ .
3524 <b>F2.</b> 3525	•	SPECIAL CONCENTRICALLY BRACED FRAMES (SCBF)
		Special concentrically braced frames (SCBF) of structural steel shall be designed in conformance with this section. Collector beams that connect SCBF braces shall be considered to be part of the SCBF.
3532 <b>F2.</b> 3533	.2.	Basis of Design
3534 3535 3536 3537	-	This section is applicable to braced frames that consist of concentrically connected members. Eccentricities less than the beam depth are permitted if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.
3538 3539		SCBF designed in accordance with these provisions are expected to provide significant

3540 3541		inelastic deformation capacity primarily through brace buckling and yielding of the brace in tension.						
3542 3543	F2.3.	Analy	Analysis					
3544 3545 3546 3547		determ	equired strength of columns, beams, struts and connections in SCBF shall be sined using the capacity-limited seismic load effect. The capacity-limited horizontal c load effect, $E_{cl}$ , shall be taken as the larger force determined from the following es:					
3548 3549		(a)	An analysis in which all braces are assumed to resist forces corresponding to their expected strength in compression or in tension					
3550 3551 3552		(b)	An analysis in which all braces in tension are assumed to resist forces corresponding to their expected strength and all braces in compression are assumed to resist their expected post-buckling strength					
3553 3554 3555		(c)	For multi-tiered braced frames, analyses representing progressive yielding and buckling of the braces from weakest tier to strongest. Analyses shall consider both directions of frame loading.					
3556 3557			shall be determined to be in compression or tension neglecting the effects of gravity Analyses shall consider both directions of frame loading.					
3558		The ex	The expected brace strength in tension is $R_yF_yA_g$ , where $A_g$ is the gross area, in. <sup>2</sup> (mm <sup>2</sup> ).					
3559 3560 3561 3562 3563		and (1 equation length	The expected brace strength in compression is permitted to be taken as the lesser of $R_yF_yA_g$ and $(1/0.877)F_{cre}A_g$ where $F_{cre}$ is determined from Specification Chapter E using the equations for $F_{cr}$ , except that the expected yield stress $R_yF_y$ is used in lieu of $F_y$ . The brace length used for the determination of $F_{cre}$ shall not exceed the distance from brace end to brace end.					
3564 3565			The expected post-buckling brace strength shall be taken as a maximum of 0.3 times the expected brace strength in compression.					
3566 3567 3568 3569		F2.5b) ksi. Th	<b>User Note</b> : Braces with a slenderness ratio of 200 (the maximum permitted by Section F2.5b) buckle elastically for permissible materials; the value of $0.3F_{cr}$ for such braces is 2.1 ksi. This value may be used in Section F2.3(b) for braces of any slenderness and a liberal estimate of the required strength of framing members will be obtained.					
3570		Except	ions:					
3571 3572		(a)	It is permitted to neglect flexural forces resulting from seismic drift in this determination.					
3573		(b)	The required strength of columns need not exceed the least of the following:					
3574 3575			(1) The forces corresponding to the resistance of the foundation to overturning uplift					

3576			(2)	Forces as determined from nonlinear analysis as defined in Section C3.
3577		(c)	The re	quired strength of bracing connections shall be as specified in Section F2.6c.
3578			User N	Note: Exception (c) is only relevant for ASD.
3579	F2.4.	Systen	n Requ	irements
3580 3581 3582 3583 3584 3585 3586 3587	F2.4a.	Along direction horizon of each overstrisingle	any line on of for the or of the original or o	e <b>Distribution</b> e of braces, braces shall be deployed in alternate directions such that, for either brace parallel to the braces, at least 30% but no more than 70% of the total ce along that line is resisted by braces in tension, unless the available strength in compression is larger than the required strength resulting from the eismic load. For the purposes of this provision, a line of braces is defined as a barallel lines with a plan offset of 10% or less of the building dimension perthe line of braces.
3588 3589 3590 3591 3592 3593 3594 3595 3596		require system using t The red the load, a require	ed strent shall be analy quired somb pplied ted strent	ing diagonal braces along a frame line do not occur in the same bay, the gths of the diaphragm, collectors, and elements of the horizontal framing be determined such that the forces resulting from the post-buckling behavior axis requirements of Section F2.3 can be transferred between the braced bays. Strength of the collector need not exceed the required strength determined by sinations of the applicable building code, including the overstrength seismic of a building model in which all compression braces have been removed. The gths of the collectors shall not be based on a load less than that stipulated by building code.
3597 3598 3599	F2.4b.	Beams	that ar	ted V-Braced Frames e intersected by braces away from beam-to-column connections shall satisfy requirements:
3600		(a)	Beams	shall be continuous between columns.
3601 3602 3603		(b)		shall be braced to satisfy the requirements for moderately ductile members in D1.2a.
3604 3605 3606			V-type	inimum, one set of lateral braces is required at the point of intersection of the e (or inverted V-type) braced frames, unless the beam has sufficient out-of-strength and stiffness to ensure stability between adjacent brace points.
3607 3608 3609 3610			stiffne Appen	<b>Note</b> : One method of demonstrating sufficient out-of-plane strength and ss of the beam is to apply the bracing force defined in Equation A-6-7 of dix 6 of the Specification to each flange so as to form a torsional couple; this g should be in conjunction with the flexural forces determined from the

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Specification.

analysis required by Section F2.3. The stiffness of the beam (and its restraints) with

respect to this torsional loading should be sufficient to satisfy Equation A-6-8 of the

3614	F2.4c.	K-Br	aced F	Frames
3615		K-typ	e brac	ed frames shall not be used for SCBF.
3616	F2.4d.	Tensi	on-Or	nly Frames
3617		Tensi	on-onl	y frames shall not be used in SCBF.
3618				
3619				Γension-only braced frames are those in which the brace compression resistance
3620		is neg	lected	in the design and the braces are designed for tension forces only.
3621			_	
3622	F2.4e.	Multi	i-tiere	d Braced Frames
3623 3624				ncentrically braced frame is permitted to be configured as a multi-tiered braced SCBF) when the following requirements are satisfied.
3625		(a)	Brac	es shall be used in opposing pairs at every tier level.
3626		(b)	Strut	s shall satisfy the following requirements:
3627			(1)	Horizontal struts shall be provided at every tier level.
3628			(2)	Struts that are intersected by braces away from strut-to-column connections
3629			(-)	shall also meet the requirements of Section F2.4b. When brace buckling occurs
3630				out-of-plane, torsional moments arising from brace buckling shall be
3631				considered when verifying lateral bracing or minimum out-of-plane strength
3632				and stiffness requirements. The torsional moments shall correspond to
3633				$1.1R_y M_p/\alpha_s$ of the brace about the critical buckling axis, but need not exceed
3634				forces corresponding to the flexural resistance of the brace connection, where
3635				M <sub>p</sub> is the nominal plastic flexural strength, kip-in. (N-mm).
3636				э-ры шо шош риш и шогдан, ш-р ш (с т шиг).
3637		(c)	Colu	imns shall satisfy the following requirements:
3638		(-)		8 4
3639				(1) Columns shall be torsionally braced at every strut-to-column
3640				connection location. User Note: The requirements for torsional bracing are
3641				typically satisfied by connecting the strut to the column to restrain torsional
3642				movement of the column. The strut must have adequate flexural strength and
3643				stiffness and an appropriate connection to the column to perform this
3644				function.
3645				
3646			(2)	Columns shall have sufficient strength to resist forces arising from brace
3647			(-)	buckling. These forces shall correspond to $1.1R_yM_p/\alpha_s$ of the brace about
3648				the critical buckling axis, but need not exceed forces corresponding to the
3649				flexural resistance of the brace connections.
3650			(3)	For all load combinations, columns subjected to axial compression shall be
3651			( )	designed to resist bending moments due to second-order and geometric
3652				imperfection effects. As a minimum, imperfection effects are permitted to be
3653				represented by an out-of-plane horizontal notional load applied at every tier
3654				level and equal to 0.006 times the vertical load contributed by the

3655 3656 3657 3658			compression brace intersecting the column at the tier level. In all cases, the multiplier $B_1$ as defined in Appendix 8 of the Specification need not exceed 2.0.
3659 3660		(d)	Each tier in a multi-tiered braced frame shall be subject to the drift limitations of the applicable building code, but the drift shall not exceed 2% of the tier height.
3661	F2.5.	Meml	pers
3662	F2.5a.	Basic 1	Requirements
3663		Colum	ns, beams, and braces shall satisfy the requirements of Section D1.1 for highly ductile
3664			ers. Struts in SCBF-MTBF shall satisfy the requirements of Section D1.1 for
3665			ately ductile members.
3666	F2.5b.	Diago	nal Braces
3667		Braces	shall comply with the following requirements:
3668		(a)	Slenderness: Braces shall have a slenderness ratio, $L_c/r \le 200$ .
3669		(b)	Built-up Braces: The spacing of connectors shall be such that the slenderness ratio,
3670		(0)	$a/r_i$ , of individual elements between the connectors does not exceed 0.4 times the
3671			governing slenderness ratio of the built-up member.
3672			governing stenderness ratio of the bunt-up member.
3673			The sum of the available sheer strengths of the connectors shall equal or exceed the
3674			The sum of the available shear strengths of the connectors shall equal or exceed the available tensile strength of each element. The spacing of connectors shall be
3675			uniform. Not less than two connectors shall be used in a built-up member.
3676			Connectors shall not be located within the middle one-fourth of the clear brace
3677			length.
3678			
3679			Exception: Where the buckling of braces about their critical bucking axis does not
3680			cause shear in the connectors, the design of connectors need not comply with this
3681			provision.
3682			provident
3683		(c)	The brace effective net area shall not be less than the brace gross area. Where
3684		(0)	reinforcement on braces is used the following requirements shall apply:
3685			Termoreement on ordees is used the following requirements shall apply.
3686			(1) The specified minimum yield strength of the reinforcement shall be at least
3687			the specified minimum yield strength of the brace.
3688			the specified infilling yield strength of the brace.
			(2) The connections of the reinforcement to the brace shall have sufficient
3689			(2) The connections of the reinforcement to the brace shall have sufficient
3690 3691			strength to develop the expected reinforcement strength on each side of a reduced section.
3692	F2.5c.	Protec	ted Zones

The protected zone of SCBF shall satisfy Section D1.3 and include the following:

3694 3695		(a)	For braces, the center one-quarter of the brace length and a zone adjacent to each connection equal to the brace depth in the plane of buckling
3696 3697		(b)	Elements that connect braces to beams and columns
3698	F2.6.	Conne	ections
3699	F2.6a.	Dema	nd Critical Welds
3700 3701 3702			llowing welds are demand critical welds, and shall satisfy the requirements of Section and I2.3:  Groove welds at column splices
3703		(b)	Welds at column-to-base plate connections
3704 3705			Exception: Welds need not be considered demand critical when both of the following conditions are satisfied:
3706 3707			(1) Column hinging at, or near, the base plate is precluded by conditions of restraint, and
3708 3709 3710			(2) There is no net tension under load combinations including the overstrength seismic load.
3711		(c)	Welds at beam-to-column connections conforming to Section F2.6b(c)
3712	F2.6b.	Beam-	-to-Column Connections
3713 3714			a brace or gusset plate connects to both members at a beam-to-column connection, the etion shall conform to one of the following:
3715 3716		(a)	The connection assembly shall be a simple connection meeting the requirements of Specification Section B3.4a where the required rotation is taken to be 0.025 rad; or
3717 3718		(b)	The connection assembly shall be designed to resist a moment equal to the lesser of the following:
3719			(1) A moment corresponding to the expected beam flexural strength, $R_y M_p$ ,
3720			multiplied by 1.1 and divided by $\alpha_s$ .
3721 3722			(2) A moment corresponding to the sum of the expected column flexural strengths, $\Sigma(R_yF_yZ)$ , multiplied by 1.1 and divided by $\alpha_s$ .
3723 3724 3725			This moment shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the overstrength seismic load.
3726		(c)	The beam-to-column connection shall meet the requirements of Section E1.6b(c).

beam-to-column conr			equired to-colu ed belo	strength in tension, compression and flexure of brace connections (including mn connections if part of the braced-frame system) shall be determined as www. These required strengths are permitted to be considered independently
3732		1.	Requi	ired Tensile Strength
3733			The re	equired tensile strength is the lesser of the following:
3734			(a)	The expected yield strength in tension, of the brace, determined as $R_y F_y A_g$ ,
3735				divided by $\alpha_s$ .
3736				Exception:
3737 3738				Braces need not comply with the requirements of Equation J4-1 and J4-2 of the Specification for this loading.
3739 3740 3741 3742 3743				<b>User Note</b> : This exception applies to braces where the section is reduced or where the net section is effectively reduced due to shear lag. A typical case is a slotted HSS brace at the gusset plate connection. Section F2.5b requires braces with holes or slots to be reinforced such that the effective net area exceeds the gross area.
3744 3745 3746				The brace strength used to check connection limit states, such as brace block shear, may be determined using expected material properties as permitted by Section A3.2.
3747 3748			(b)	The maximum load effect, indicated by analysis, that can be transferred to the brace by the system.
3749 3750 3751				n oversized holes are used, the required strength for the limit state of bolt slip not exceed the seismic load effect determined using the overstrength seismic
3752			User	<b>Note:</b> For other limit states the loadings of (a) and (b) apply.
3753		2.	Requi	ired Compressive Strength
3754 3755 3756 3757			buckli divide	connections shall be designed for a required compressive strength, based on ing limit states, that is equal to the expected brace strength in compression ed by $\alpha_s$ , where the expected brace strength in compression is as defined in F2.3.
3758		3.	Accor	mmodation of Brace Buckling
3759 3760 3761			impos	connections shall be designed to withstand the flexural forces or rotations and by brace buckling. Connections satisfying either of the following provisions seemed to satisfy this requirement:

3762 3763 3764 3765 3766	(a) Required Flexural Strength: Brace connections designed to withstand the flexural forces imposed by brace buckling shall have a required flexural strength equal to the expected brace flexural strength multiplied by 1.1 and divided by α <sub>s</sub> . The expected brace flexural strength shall be determined as R <sub>y</sub> M <sub>p</sub> of the brace about the critical buckling axis.
3767 3768 3769 3770	(b) Rotation Capacity: Brace connections designed to withstand the rotations imposed by brace buckling shall have sufficient rotation capacity to accommodate the required rotation at the design story drift. Inelastic rotation of the connection is permitted.
3771 3772 3773 3774	<b>User Note:</b> Accommodation of inelastic rotation is typically accomplished by means of a single gusset plate with the brace terminating before the line of restraint. The detailing requirements for such a connection are described in the Commentary.
3775	4. Gusset Plates
3776 3777	For out-of-plane brace buckling, welds that attach a gusset plate directly to a beam flange or column flange shall have available shear strength equal to $0.6R_yF_yt_p/\alpha_s$
3778	times the joint length, where $F_y$ = specified minimum yield stress of the gusset plate, ksi (MPa)
3779	$R_y$ = ratio of the expected yield stress to the specified minimum yield stress of the gusset plate
	$t_p$ = thickness of the gusset plate, in. (mm)
3780 3781 3782 3783 3784	Exception: Alternatively, these welds may be designed to have available strength to resist gusset-plate edge forces corresponding to the brace force specified in Section F2.6c.2 combined with the gusset plate weak-axis flexural strength determined in the presence of those forces.
3785 3786 3787 3788	<b>User Note:</b> The expected shear strength of the gusset plate may be developed using double-sided fillet welds with leg size equal to $0.74t_p$ for ASTM A572 Grade 50 plate and $0.62t_p$ for ASTM A36 plate and E70 electrodes. Smaller welds may be justified using the exception.
3789	F2.6d. Column Splices
3790 3791 3792	Column splices shall comply with the requirements of Section D2.5. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds. Column splices shall be designed to develop at least 50% of the lesser plastic flexural strength, $\mathbf{M}_p$ ,
3793 3794 3795 3796	of the connected members, divided by $\alpha_s.$ The required shear strength shall be $\big(\Sigma M_p/\alpha_s\big)\!\big/H_c$ ,
ンノソロ	

3797 3798 3799 3800 3801 3802 3803	F3.	where $H_c = \text{clear height of the column between beam connections, including a structural slab, if present, in. (mm)}$ $\Sigma M_p = \text{sum of the plastic flexural strengths, } F_yZ, \text{ of the top and bottom ends of the column, kip-in. (N-mm)}$ $ECCENTRICALLY BRACED FRAMES (EBF)$
3804	F3.1.	Scope
3805 3806		Eccentrically braced frames (EBF) of structural steel shall be designed in conformance with this section.
3807	F3.2.	Basis of Design
3808 3809 3810 3811 3812 3813		This section is applicable to braced frames for which one end of each brace intersects a beam at an eccentricity from the intersection of the centerlines of the beam and an adjacent brace or column, forming a link that is subject to shear and flexure. Eccentricities less than the beam depth are permitted in the brace connection away from the link if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.
3814 3815		EBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through shear or flexural yielding in the links.
3816 3817 3818		Where links connect directly to columns, design of their connections to columns shall provide the performance required by Section F3.6e.1 and demonstrate this conformance as required by Section F3.6e.2.
3819	F3.3.	Analysis
3820 3821 3822 3823 3824 3825 3826 3827		The required strength of diagonal braces and their connections, beams outside links, and columns shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, $E_{cl}$ , shall be taken as the forces developed in the member assuming the forces at the ends of the links correspond to the adjusted link shear strength. The adjusted link shear strength shall be taken as $R_y$ times the link nominal shear strength, $V_n$ , given in Section F3.5b.2 multiplied by 1.25 for I-shaped links and 1.4 for box links.
3828		Exceptions:
3829 3830 3831 3832		(a) The effect of capacity-limited horizontal forces, E <sub>cl</sub> , is permitted to be taken as 0.88 times the forces determined in Section F3.3 for the design of the portions of beams outside links.

3833 3834 3835 3836		(b) It is permitted to neglect flexural forces resulting from seismic drift in this determination. Moment resulting from a load applied to the column between points of lateral support must be considered.
3837		(c) The required strength of columns need not exceed the lesser of the following:
3838		(1) Forces corresponding to the resistance of the foundation to overturning uplift
3839		(2) Forces as determined from nonlinear analysis as defined in Section C3.
3840 3841 3842		The inelastic link rotation angle shall be determined from the inelastic portion of the design story drift. Alternatively, the inelastic link rotation angle is permitted to be determined from nonlinear analysis as defined in Section C3.
3843 3844 3845		<b>User Note</b> : The seismic load effect, E, used in the design of EBF members, such as the required axial strength used in the equations in Section F3.5, should be calculated from the analysis above.
3846 3847	F3.4.	System Requirements
3848 3849 3850 3851	F3.4a.	Link Rotation Angle The link rotation angle is the inelastic angle between the link and the beam outside of the link when the total story drift is equal to the design story drift, $\Delta$ . The link rotation angle shall not exceed the following values:
3852		(a) For links of length $1.6M_p/V_p$ or less: 0.08 rad
3853 3854 3855 3856 3857		(b) For links of length $2.6M_p/V_p$ or greater: $0.02$ rad where $ \begin{array}{ccc} M_p &=& plastic \ flexural \ strength \ of \ a \ link, \ kip-in. \ (N-mm) \\ V_p &=& plastic \ shear \ strength \ of \ a \ link, \ kips \ (N) \end{array} $
3858 3859		Linear interpolation between the above values shall be used for links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$ .
3860 3861 3862 3863	F3.4b.	<b>Bracing of Link</b> Bracing shall be provided at both the top and bottom link flanges at the ends of the link for I-shaped sections. Bracing shall have an available strength and stiffness as required for expected plastic hinge locations by Section D1.2c.
3864	F3.5.	Members
3865 3866 3867	F3.5a.	<b>Basic Requirements</b> Brace members shall satisfy width-to-thickness limitations in Section D1.1 for moderately ductile members.
3868 3869		Column members shall satisfy width-to-thickness limitations in Section D1.1 for highly ductile members.

Where the beam outside of the link is a different section from the link, the beam shall satisfy the width-to-thickness limitations in Section D1.1 for moderately ductile members.

**User Note**: The diagonal brace and beam segment outside of the link are intended to remain essentially elastic under the forces generated by the fully yielded and strain hardened link. Both the diagonal brace and beam segment outside of the link are typically subject to a combination of large axial force and bending moment, and therefore should be treated as beam-columns in design, where the available strength is defined by Chapter H of the Specification.

Where the beam outside the link is the same member as the link, its strength may be determined using expected material properties as permitted by Section A3.2.

#### F3.5b. Links

Links subject to shear and flexure due to eccentricity between the intersections of brace centerlines and the beam centerline (or between the intersection of the brace and beam centerlines and the column centerline for links attached to columns) shall be provided. The link shall be considered to extend from brace connection to brace connection for center links and from brace connection to column face for link-to-column connections except as permitted by Section F3.6e.

## 1. Limitations

Links shall be I-shaped cross sections (rolled wide-flange sections or built-up sections), or built-up box sections. HSS sections shall not be used as links.

Links shall satisfy the requirements of Section D1.1 for highly ductile members.

Exceptions: Flanges of links with I-shaped sections with link lengths,  $e \le 1.6 \, M_p/V_p$ , are permitted to satisfy the requirements for moderately ductile members. Webs of links with box sections with link lengths,  $e \le 1.6 M_p/V_p$ , are permitted to satisfy the requirements for moderately ductile members.

The web or webs of a link shall be single thickness. Doubler-plate reinforcement and web penetrations are not permitted.

For links made of built-up cross sections, complete-joint-penetration groove welds shall be used to connect the web (or webs) to the flanges.

Links of built-up box sections shall have a moment of inertia,  $I_y$ , about an axis in the plane of the EBF limited to  $I_y > 0.67I_x$ , where  $I_x$  is the moment of inertia about an axis perpendicular to the plane of the EBF.

## 2. Shear Strength

3908 The link design shear strength,  $\phi_v V_n$ , and the allowable shear strength,  $V_n/\Omega_v$ , shall be 3909 the lower value obtained in accordance with the limit states of shear yielding in the 3910 web and flexural yielding in the gross section. For both limit states: 3911  $\phi_v = 0.90 \text{ (LRFD)} \qquad \Omega_v = 1.67 \text{ (ASD)}$ 3912 3913 3914 (a) For shear yielding: 3915  $V_n = V_p$ 3916 (F3-1)3917 3918  $V_p \ = 0.6 F_y A_{lw} \ for \ \alpha_s P_r \, / \, P_y \! \leq \! 0.15$ 3919 (F3-2) $V_p = 0.6F_y A_{tw} \sqrt{1 - (\alpha_s P_r / P_y)^2}$  for  $\alpha_s P_r / P_y > 0.15$ 3920 (F3-3) $A_{lw} = (d-2t_f)t_w$  for I-shaped link sections 3921 (F3-4)3922 =  $2(d-2t_f)t_w$  for box link sections (F3-5) $P_r = P_u$  (LRFD) or  $P_a$  (ASD), as applicable 3923 P<sub>u</sub> = required axial strength using LRFD load combinations, kips (N) 3924 P<sub>a</sub> = required axial strength using ASD load combinations, kips (N) 3925  $P_v$  = nominal axial yield strength =  $F_vA_g$ 3926 (F3-6)3927 3928 (b) For flexural yielding: 3929 3930  $V_n = 2M_p/e$ (F3-7)3931 3932 where  $M_p = F_y Z$  for  $\alpha_s P_r / P_y \le 0.15$ 3933 (F3-8) $M_p = F_y Z \left( \frac{1 - \alpha_s P_r / P_y}{0.85} \right) \text{ for } \alpha_s P_r / P_y > 0.15$ 3934 (F3-9)e = length of link, defined as the clear distance between the ends of two 3935 3936 diagonal braces or between the diagonal brace and the column face, 3937 in. (mm) 3938 **3. Link Length** 3939 3940 If  $P_r/P_c > 0.15$ , the length of the link shall be limited as follows: 3941 When  $\rho' \leq 0.5$  $e \le \frac{1.6M_p}{V_p}$ 3942 (F3-10)

When  $\rho' > 0.5$ 

$$e \leq \frac{1.6M_p}{V_p} (1.15-0.3\rho') \qquad (F3-11)$$

$$\frac{P_t}{V_p} \qquad (F3-12)$$

$$\frac{P_t}{V_y} \qquad (P_t}{V_y} \qquad (P_t}{V_y} \qquad (P_t}{V_y} \qquad (P_t}{V_y} \qquad (P_t}{V_y} \qquad ($$

Links shall be provided with intermediate web stiffeners as follows:

- (a) Links of lengths  $1.6M_p/V_p$  or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding  $(30t_w-d/5)$  for a link rotation angle of 0.08 rad or  $(52t_w-d/5)$  for link rotation angles of 0.02 rad or less. Linear interpolation shall be used for values between 0.08 and 0.02 rad.
- (b) Links of length greater than or equal to  $2.6M_p/V_p$  and less than  $5M_p/V_p$  shall be provided with intermediate web stiffeners placed at a distance of 1.5 times  $b_f$  from each end of the link.
- (c) Links of length between 1.6M<sub>p</sub>/V<sub>p</sub> and 2.6M<sub>p</sub>/V<sub>p</sub> shall be provided with intermediate web stiffeners meeting the requirements of (a) and (b) above.

Intermediate web stiffeners are not required in links of length greater than 5M<sub>p</sub>/V<sub>p</sub>.

Intermediate web stiffeners shall be full depth. For links that are less than 25 in. (635 mm) in depth, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners shall not be less than  $t_w$  or 3/8 in. (10 mm), whichever is larger, and the width shall be not less than  $(b_f/2) - t_w$ . For links that are 25 in. (635

3979 mm) in depth or greater, intermediate stiffeners with these dimensions are required on both sides of the web. 3980 3981 The required strength of fillet welds connecting a link stiffener to the link web is  $F_y A_{st}/\alpha_s$ , where  $A_{st}$  is the horizontal cross-sectional area of the link stiffener and  $F_y$ 3982 is the yield stress of the stiffener. The required strength of fillet welds connecting the 3983 stiffener to the link flanges is  $F_v A_{st}/(4\alpha_s)$ . 3984 5. 3985 **Link Stiffeners for Box Sections** 3986 Full-depth web stiffeners shall be provided on one side of each link web at the 3987 diagonal brace connection. These stiffeners are permitted to be welded to the outside 3988 or inside face of the link webs. These stiffeners shall each have a width not less than 3989 b/2, where b is the inside width of the box. These stiffeners shall each have a thickness not less than the larger of  $0.75t_w$  or  $\frac{1}{2}$  in. (13 mm). 3990 Box links shall be provided with intermediate web stiffeners as follows: 3991 For links of length 1.6M<sub>p</sub>/V<sub>p</sub> or less and with web depth-to-thickness ratio, 3992 (a)  $h/t_w$ , greater than or equal to  $0.67\sqrt{\frac{E}{R_vF_v}}$ , full-depth web stiffeners shall be 3993 provided on one side of each link web, spaced at intervals not exceeding 3994 3995  $20t_{\rm w}$ -(d- $2t_{\rm f}$ )/8. For links of length 1.6M<sub>p</sub>/V<sub>p</sub> or less and with web depth-to-thickness ratio, 3996 (b)  $h/t_w$ , less than  $0.67\sqrt{\frac{E}{R_vF_v}}$ , no intermediate web stiffeners are required. 3997 3998 For links of length greater than 1.6M<sub>p</sub>/V<sub>p</sub>, no intermediate web stiffeners are (c) 3999 required. Intermediate web stiffeners shall be full depth, and are permitted to be welded to the 4000 outside or inside face of the link webs. 4001 4002 4003 The required strength of fillet welds connecting a link stiffener to the link web is  $F_{_{v}}A_{_{\!st}}/\alpha_{_{s}}$  , where  $A_{\!st}$  is the horizontal cross-sectional area of the link stiffener. 4004 4005 4006 **User Note:** Stiffeners of box links need not be welded to link flanges. 4007 F3.5c. Protected Zones 4008 Links in EBFs are a protected zone, and shall satisfy the requirements of Section D1.3. 4009 F3.6. Connections

4010	F3.6a	. Dema	nd Cri	tical Welds
4011 4012			and I2.	
4013		(a)	Groov	ve welds at column splices
4014		(b)	Welds	s at column-to-base plate connections
4015 4016				tion: Welds need not be considered demand critical when both of the ving conditions are satisfied:
4017 4018			(1)	Column hinging at, or near, the base plate is precluded by conditions of restraint, and
4019 4020			(2)	There is no net tension under load combinations including the overstrength seismic load.
4021		(c)	Welds	s at beam-to-column connections conforming to Section F3.6b(c)
4022 4023		(d)	Where the co	e links connect to columns, welds attaching the link flanges and the link web to lumn
4024		(e)	In bui	lt-up beams, welds within the link connecting the webs to the flanges
4025	F3.6b	. Beam	-to-Col	umn Connections
4026 4027				e or gusset plate connects to both members at a beam-to-column connection, the hall conform to one of the following:
4028 4029		(a)		connection assembly is a simple connection meeting the requirements of fication Section B3.4a where the required rotation is taken to be 0.025 rad; or
4030 4031		(b)	The co	onnection assembly is designed to resist a moment equal to the lesser of the ving:
4032 4033			(1)	A moment corresponding to the expected beam flexural strength, $R_y M_{_p}$ , multiplied by 1.1 and divided by $\alpha_s.$
4034 4035			(2)	A moment corresponding to the sum of the expected column flexural strengths, $\Sigma(R_yF_yZ)$ , multiplied by 1.1and divided by $\alpha_s$ .
4036 4037 4038			brace	noment shall be considered in combination with the required strength of the connection and beam connection, including the diaphragm collector forces nined using the overstrength seismic load.
4039		(c)	The be	eam-to-column connection satisfies the requirements of Section E1.6b(c).
4040	F3.6c.	Brace	Conne	ections

4041 4042			zed holes are used, the required strength for the limit state of bolt slip need not eismic load effect determined using the overstrength seismic load
4043 4044		ections y restr	of braces designed to resist a portion of the link end moment shall be designed ained.
4045	F3.6d. Colur	nn Spl	ices
4046	$C_{-1}$	1.	1 11 1 14 14 1
4047 4048			ces shall comply with the requirements of Section D2.5. Where groove welds are the splice, they shall be complete-joint-penetration groove welds. Column
4049			be designed to develop at least 50% of the lesser plastic flexural strength, $M_{\rm p}$ ,
4050	of the	conne	cted members, divided by $\alpha_s$ .
4051 4052	The re	equired	I shear strength shall be $\Sigma M_p/(\alpha_s H_c)$ ,
4053			
4054	where	;	
4055	H	$I_c =$	clear height of the column between beam connections, including a structural
4056			slab, if present, in. (mm)
4057	Σ	$M_p =$	sum of the plastic flexural strengths, F <sub>y</sub> Z, at the top and bottom ends of the
4058		1	column, kip-in. (N-mm)
4059 4060	F3.6e. Link-	to-Col	umn Connections
4061	1.	Requ	nirements
4062 4063		I ink.	-to-column connections shall be fully restrained (FR) moment connections and
4064			satisfy the following requirements:
4065		Silaii	satisfy the following requirements.
4066		(a)	The connection shall be capable of sustaining the link rotation angle specified
4067		()	in Section F3.4a.
4068			
4069		(b)	The shear resistance of the connection, measured at the required link rotation
4070			angle, shall be at least equal to the expected shear strength of the link, R <sub>y</sub> V <sub>n</sub> ,
4071			as defined in Section F3.5b.2.
4072			
4073		(c)	The flexural resistance of the connection, measured at the required link
4074			rotation angle, shall be at least equal to the moment corresponding to the
4075			nominal shear strength of the link, V <sub>n</sub> , as defined in Section F3.5b.2.
4076			
4077	2.	Conf	Formance Demonstration
4078			
4079		Link	-to-column connections shall satisfy the above requirements by one of the
4080		follo	wing:
4081		(a)	Use a connection prequalified for EBF in accordance with Section K1.

4082			User Note: There are no prequalified link-to-column connections.
4083 4084 4085		(b)	Provide qualifying cyclic test results in accordance with Section K2. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:
4086 4087 4088 4089 4090 4091 4092 4093			<ol> <li>Tests reported in research literature or documented tests performed for other projects that are representative of project conditions, within the limits specified in Section K2.</li> <li>Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection material properties, within the limits specified in Section K2.</li> </ol>
4094 4095 4096			tion: Cyclic testing of the connection is not required if the following ions are met:
4090 4097 4098 4099 4100 4101 4102 4103 4104 4105		<ul><li>(a)</li><li>(b)</li><li>(c)</li><li>(d)</li></ul>	Reinforcement at the beam-to-column connection at the link end precludes yielding of the beam over the reinforced length. The available strength of the reinforced section and the connection equals or exceeds the required strength calculated based upon adjusted link shear strength as described in Section F3.3. The link length (taken as the beam segment from the end of the reinforcement to the brace connection) does not exceed $1.6M_p/V_p$ . Full depth stiffeners as required in Section F3.5b.4 are placed at the link-to-reinforcement interface.
4106 4107	F4.	BUCKLING	-RESTRAINED BRACED FRAMES (BRBF)
4107	F4.1.	Scope	
4109 4110 4111	74.0	conformance	rained braced frames (BRBF) of structural steel shall be designed in with this section.
4112 4113 4114 4115 4116	F4.2.	to beams and comember and co	applicable to frames with specially fabricated braces concentrically connected columns. Eccentricities less than the beam depth are permitted if the resulting connection forces are addressed in the design and do not change the expected astic deformation capacity.
4117 4118 4119 4120 4121 4122		inelastic defor Design of braand demonstratested and deta	ed in accordance with these provisions are expected to provide significant mation capacity primarily through brace yielding in tension and compression. ces shall provide the performance required by Sections F4.5b.1 and F4.5b.2, ate this conformance as required by Section F4.5b.3. Braces shall be designed, ailed to accommodate expected deformations. Expected deformations are those to a story drift of at least 2% of the story height or two times the design story

4123 4124	drift, whichever is larger, in addition to brace deformations resulting from deformation of the frame due to gravity loading.
4125 4126	BRBF shall be designed so that inelastic deformations under the design earthquake will occur primarily as brace yielding in tension and compression.
4127 <b>F4.2</b> 3 4128 4129	The adjusted brace strength shall be established on the basis of testing as described in this section.
4130 4131	Where required by these Provisions, brace connections and adjoining members shall be designed to resist forces calculated based on the adjusted brace strength.
4132	The adjusted brace strength in compression shall be $\beta \omega R_y P_{ysc}$ , where
4133	β= compression strength adjustment factor
4134	ω= strain hardening adjustment factor
4135	$P_{ysc}$ = axial yield strength of steel core, ksi (MPa)
4136	The adjusted brace strength in tension shall be $\omega R_y P_{ysc}$ .
4137 4138	Exception: The factor $R_y$ need not be applied if $P_{ysc}$ is established using yield stress determined from a coupon test.
4139	F4.2b. Adjustment Factors
4140	Adjustment factors shall be determined as follows:
4141 4142 4143 4144 4145	The compression strength adjustment factor, $\beta$ , shall be calculated as the ratio of the maximum compression force to the maximum tension force of the test specimen measured from the qualification tests specified in Section K3.4c at strains corresponding to the expected deformations. The larger value of $\beta$ from the two required brace qualification tests shall be used. In no case shall $\beta$ be taken as less than 1.0.
4146 4147 4148 4149 4150 4151 4152	The strain hardening adjustment factor, $\omega$ , shall be calculated as the ratio of the maximum tension force measured from the qualification tests specified in Section K3.4c at strains corresponding to the expected deformations to the measured yield force, $P_{ysc}$ , of the test specimen. The larger value of $\omega$ from the two required qualification tests shall be used. Where the tested steel core material of the subassemblage test specimen required in Section K3.2 does not match that of the prototype, $\omega$ shall be based on coupon testing of the prototype material.
4153	F4.2c. Brace Deformations
4154 4155 4156	The expected brace deformation shall be determined from the story drift specified in Section F4.2. Alternatively, the brace expected deformation is permitted to be determined from nonlinear analysis as defined in Section C3.
4157 <b>F4.3</b> .	Analysis
4158 4159	The required strength of columns, beams, struts and connections in BRBF shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal

4160 4161				effect, E <sub>cl</sub> , shall be taken as the forces developed in the member assuming the races correspond to their adjusted strength in compression or in tension.
4162 4163				e determined to be in compression or tension neglecting the effects of gravity es shall consider both directions of frame loading.
4164		The ac	djusted	brace strength in tension shall be as given in Section F4.2a.
4165		Excep	otions:	
4166 4167 4168 4169		(a)	determ lateral	ermitted to neglect flexural forces resulting from seismic drift in this ination. Moment resulting from a load applied to the column between points of support, including Section F4.4d loads, must be considered.
4170 4171 4172		(b)	(1)	uired strength of columns need not exceed the lesser of the following:  The forces corresponding to the resistance of the foundation to overturning uplift. Section F4.4d in-plane column load requirements shall be adhered to.
4173 4174			(2)	Forces as determined from nonlinear analysis as defined in Section C3.
4175	F4.4.	System	m Requ	irements
4176	F4.4a.	V- an	d Inver	ted V-Braced Frames
4177		V-typ	e and in	verted-V-type braced frames shall satisfy the following requirements:
4178 4179 4180 4181 4182 4183		(a)	and su applic live lo horizo	equired strength of beams and struts intersected by braces, their connections porting members shall be determined based on the load combinations of the able building code assuming that the braces provide no support for dead and eads. For load combinations that include earthquake effects, the vertical and ental earthquake effect, E, on the beam shall be determined from the adjusted strengths in tension and compression.
4184 4185 4186		(b)		s and struts shall be continuous between columns. Beams and struts shall be I to satisfy the requirements for moderately ductile members in Section .1.
4187 4188 4189			V-type	ninimum, one set of lateral braces is required at the point of intersection of the e (or inverted V-type) braces, unless the beam or strut has sufficient out-of-strength and stiffness to ensure stability between adjacent brace points.
4190 4191 4192 4193			bent in stiffne	<b>Note</b> : The beam has sufficient out-of-plane strength and stiffness if the beam in the horizontal plane meets the required brace strength and required brace is for column nodal bracing as prescribed in the Specification. Pu may be taken required compressive strength of the brace.
4194	F4.4b.	K-Bra	aced Fr	ames

K-type braced frames shall not be used for BRBF.

4196	F4.4c	Later	ral For	ce Dist	ribution				
4197 4198 4199					ssion strength adjustment factor, $\beta$ , as determined in Section F4.2b eral force distribution shall comply with the following:				
4200 4201 4202 4203 4204 4205 4206		direct horize of each load. lines	Along any line of braces, braces shall be deployed in alternate directions such that, for eit direction of force parallel to the braces, at least 30% but no more than 70% of the to horizontal force along that line is resisted by braces in tension, unless the available streng of each brace is larger than the required strength resulting from the overstrength seism load. For the purposes of this provision, a line of braces is defined as a single line or parallines with a plan offset of 10% or less of the building dimension perpendicular to the line braces.						
4207	F4.4d	Mult	i-tiered	l Brace	ed Frames				
4208 4209			_		ed braced frame is permitted to be configured as a multi-tiered braced when the following requirements are satisfied.				
4210 4211 4212		(a)	items	as req	of out-of-plane forces due to the mass of the structure and supported uired by the applicable building code shall be combined with the forces m the analyses required by Section F4.3.				
4213		(b)	Strut	s shall	be provided at every brace to column connection location.				
4214 4215		(c)	Colu	mns sh	all satisfy the following requirements:				
4216 4217 4218			(1)	for th	nns of multi-tiered braced frames shall be designed as simply supported e height of the frame between points of out-of-plane support and shall by the greater of the following in-plane load requirements at each tier:				
4219 4220 4221				(i)	Loads induced by the summation of frame shears from adjusted brace strengths between adjacent tiers from Section F4.3 analysis. Analysis shall consider variation in permitted core strength.				
4222 4223 4224 4225 4226					<b>User Note:</b> Specifying the BRB using the desired brace capacity, P <sub>ysc</sub> , rather than a desired core area is recommended for the multi-tiered buckling-restrained braced (BRB) frame to reduce the effect of material variability and allow for the design of equal or nearly equal tier capacities.				
4227 4228 4229				(ii)	A minimum notional load equal to 0.5% times the adjusted braced strength frame shear of the higher strength adjacent tier. The notional load shall be applied to create the greatest load effect on the column.				
4230 4231			(2)	Colur locati	nns shall be torsionally braced at every strut-to-column connection on.				
4232				User	<b>Note:</b> The requirements for torsional bracing are typically satisfied by				

4233 4234 4235			connecting the strut to the column to restrain torsional movement of column. The strut must have adequate flexural strength and stiffness and an appropriate connection to the column to perform this function.	
4236 4237 4238 4239			Each tier in a multi-tiered braced frame shall be subject to the drift limitations applicable building code, but the drift shall not exceed 2% of the tier height.	of the
4240	F4.5.	Membe	ers	
4241 4242 4243	F4.5a.		<b>Requirements</b> and columns shall satisfy the requirements of Section D1.1 for moderately drs.	uctile
4244	F4.5b.	Diagon	al Braces	
4245 4246	1.	Assemb	bly	
4247 4248		Braces s from bu	shall be composed of a structural steel core and a system that restrains the steel ackling.	l core
4249		(a)	Steel Core	
4250 4251			Plates used in the steel core that are 2 in. (50 mm) thick or greater shall satisf minimum notch toughness requirements of Section A3.3.	fy the
4252			Splices in the steel core are not permitted.	
4253		(b)	Buckling-Restraining System	
4254 4255 4256			The buckling-restraining system shall consist of the casing for the steel constability calculations, beams, columns and gussets connecting the core shall considered parts of this system.	
4257 4258			The buckling-restraining system shall limit local and overall buckling of the core for the expected deformations.	steel
4259 4260			<b>ote</b> : Conformance to this provision is demonstrated by means of testing as descon F4.5b.3.	ribed
4261 4262	2.	Availal	ble Strength	
4263 4264		The stee	el core shall be designed to resist the entire axial force in the brace.	
4265 4266 4267		(ASD),	ce design axial strength, $\phi P_{ysc}$ (LRFD), and the brace allowable axial strength, F in tension and compression, in accordance with the limit state of yielding, should be a follows:	-
4268 4269 4270			$P_{ysc} = F_{ysc} A_{sc} $ (2)	F4-1)
4271			$\phi = 0.90 \; (LRFD) \qquad \Omega = 1.67 \; (ASD)$ Seismic Provisions for Structural Steel Buildings Draft dated December 18, 2015 American Institute of Steel Construction	

4272 4273		where	
4274			areas sectional area of the violding segment of the steel care in 2 (mm²)
			cross-sectional area of the yielding segment of the steel core, in. <sup>2</sup> (mm <sup>2</sup> )
4275		J	specified minimum yield stress of the steel core, or actual yield stress of the
4276			steel core as determined from a coupon test, ksi (MPa)
4277			Load effects calculated based on adjusted brace strengths should not be based
4278		upon the ov	erstrength seismic load.
4279	3.		ce Demonstration
4280		_	of braces shall be based upon results from qualifying cyclic tests in accordance
4281		with the pro	ocedures and acceptance criteria of Section K3. Qualifying test results shall
4282		consist of a	t least two successful cyclic tests: one is required to be a test of a brace
4283			age that includes brace connection rotational demands complying with Section
4284			e other shall be either a uniaxial or a subassemblage test complying with Section
4285			test types shall be based upon one of the following:
4286		(a) Tests	reported in research or documented tests performed for other projects
4287		(b) Tests	that are conducted specifically for the project
4288			
4289		Interpolation	n or extrapolation of test results for different member sizes shall be justified by
4290			lysis that demonstrates stress distributions and magnitudes of internal strains
4291			with or less severe than the tested assemblies and that addresses the adverse
4292			
			riations in material properties. Extrapolation of test results shall be based upon
4293			binations of steel core and buckling-restraining system sizes. Tests are permitted
4294		to quality a	design when the provisions of Section K3 are met.
4295	F4.5c.	Protected Z	Zones
4296		The protecte	ed zone shall include the steel core of braces and elements that connect the steel
4297		-	ns and columns, and shall satisfy the requirements of Section D1.3.
4298	F4.6.	Connection	S
4299	F4.6a.	<b>Demand C</b> ı	ritical Welds
4300		The following	ng welds are demand critical welds, and shall satisfy the requirements of Section
4301		A3.4b and I	
4302		(a) Groo	ove welds at column splices
4303		(b) Weld	ds at the column-to-base plate connections
4304		Exce	eption: Welds need not be considered demand critical when both of the
4305			wing conditions are satisfied:
4306			Column hinging at, or near, the base plate is precluded by conditions of
4307			restraint, and

4308 4309		(a)	(2) There is no net tension under load combinations including the overstrength seismic load.  Worlds at beam to column connections conforming to Section E4.6b(c)
4310		(c)	Welds at beam-to-column connections conforming to Section F4.6b(c)
4311	F4.6b.	Beam	-to-Column Connections
4312 4313			a brace or gusset plate connects to both members at a beam-to-column connection, the ction shall conform to one of the following:
4314 4315		(a)	The connection assembly shall be a simple connection meeting the requirements of Specification Section B3.4a where the required rotation is taken to be 0.025 rad; or
4316 4317		(b)	The connection assembly shall be designed to resist a moment equal to the lesser of the following:
4318 4319			(1) A moment corresponding to the expected beam flexural strength, $R_y M_p$ multiplied by 1.1 and divided by $\alpha_s$ .
4320			(2) A moment corresponding to the sum of the expected column flexural
4321			strengths, $\Sigma(R_vF_vZ)$ , multiplied by 1.1 and divided by $\alpha_s$ ,
4322			where
4323			Z = plastic section modulus about the axis of bending, in.3 (mm3)
4324			This moment shall be considered in combination with the required strength of the
4325 4326			brace connection and beam connection, including the diaphragm collector forces determined using the overstrength seismic load.
4327		(c)	The beam-to-column connection shall meet the requirements of Section E1.6b(c).
4328	F4.6c.	Diago	nal Brace Connections
4329		1.	Required Strength
4330		1.	Required but engin
4331			The required strength of brace connections in tension and compression (including
4332			beam-to-column connections if part of the braced-frame system) shall be the adjusted
4333			brace strength divided by $\alpha_s$ , where the adjusted brace strength is as defined in
4334			Section F4.2a.
4335			When oversized holes are used, the required strength for the limit state of bolt slip
4336			need not exceed $P_{ysc}$ / $\alpha_s$ .
4337 4338		2.	Gusset Plate Requirements
4339 4340			Lateral bracing consistent with that used in the tests upon which the design is based is required.

4341 **User Note**: This provision may be met by designing the gusset plate for a transverse force consistent with transverse bracing forces determined from testing, by adding a 4342 4343 stiffener to it to resist this force, or by providing a brace to the gusset plate. Where the supporting tests did not include transverse bracing, no such bracing is required. 4344 Any attachment of bracing to the steel core must be included in the qualification 4345 4346 testing. 4347 F4.6d. Column Splices 4348 Column splices shall comply with the requirements of Section D2.5. Where groove welds 4349 are used to make the splice, they shall be complete-joint-penetration groove welds. Column 4350 splices shall be designed to develop at least 50% of the lesser plastic flexural strength, M<sub>n</sub>, 4351 4352 of the connected members, divided by  $\alpha_s$ . 4353 The required shear strength, V<sub>r</sub> shall be determined as follows: 4354 4355  $V_{r} = \frac{\sum M_{p}}{\alpha_{o} H_{o}}$ 4356 (F4-2)4357 where clear height of the column between beam connections, including a structural 4358  $H_{c}$ 4359 slab, if present, in. (mm) 4360 sum of the plastic flexural strengths, F<sub>v</sub>Z, top and bottom ends of the column,  $\Sigma M_p =$ 4361 kip-in. (N-mm) 4362 SPECIAL PLATE SHEAR WALLS (SPSW) 4363 F5. 4364 4365 F5.1. Scope Special plate shear walls (SPSW) of structural steel shall be designed in conformance with 4366 this section. This section is applicable to frames with steel web plates connected to beams 4367 4368 and columns. 4369 F5.2. Basis of Design 4370 SPSW designed in accordance with these provisions are expected to provide significant 4371 inelastic deformation capacity primarily through web plate yielding and as plastic-hinge formation in the ends of horizontal boundary elements (HBEs). Vertical boundary elements 4372 (VBEs) are not expected to yield in shear; VBEs are not expected to yield in flexure except 4373 4374 at the column base. F5.3. Analysis 4375

An analysis in conformance with the applicable building code shall be performed.

The required strength of web plates shall be 100% of the required shear strength of

The webs of SPSW shall not be considered as resisting gravity forces.

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(a)

4379		the frame from this analysis. The required strength of the frame consisting of VBEs
4380		and HBEs alone shall be not less than 25% of the frame shear force from this
4381		analysis.
4382	(b)	The required strength of HBEs, VBEs, and connections in SPSW shall be determined
4383		using the capacity-limited seismic load effect. The capacity-limited horizontal
4384		seismic load effect, Ecl, shall be determined from an analysis in which all webs are
4385		assumed to resist forces corresponding to their expected strength in tension at an
4386		angle, $\alpha$ , as determined in Section F5.5b and HBE are resisting flexural forces at
4387		each end equal to $1.1R_yM_p/\alpha_s$ . Webs shall be determined to be in tension
4388		neglecting the effects of gravity loads.
4389		The expected web yield stress shall be taken as R <sub>y</sub> F <sub>y</sub> . When perforated walls are
4390		used, the effective expected tension stress is as defined in Section F5.7a.4.
4391		Exception: The required strength of VBEs need not exceed the forces determined
4392		from nonlinear analysis as defined in Section C3.
4393		User Note: Shear forces per Equation £1-1 must be included in this analysis.
4394		Designers should be aware that in some cases forces from the analysis in the
4395		applicable building code will govern the design of HBEs.
4396		
4397		User Note: Shear forces in beams and columns are likely to be high and shear
4398		yielding must be evaluated.

# **F5.4.** System Requirements

# F5.4a. Stiffness of Boundary Elements

The stiffness of vertical boundary elements (VBEs) and horizontal boundary elements (HBEs) shall be such that the entire web plate is yielded at the design story drift. VBE and HBE conforming to the following requirements shall be deemed to comply with this requirement. The vertical boundary elements (VBEs) shall have moments of inertia about an axis taken perpendicular to the plane of the web,  $I_c$ , not less than  $0.0031t_wh^4/L$ . The horizontal boundary elements (HBEs) shall have moments of inertia about an axis taken perpendicular to the plane of the web,  $I_b$ , not less than  $0.0031L^4/h$  times the difference in web plate thicknesses above and below,

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4411	$I_b$	=	moment of inertia of a HBE taken perpendicular to the	direction of the web
4412			plate line, in. <sup>4</sup> (mm <sup>4</sup> )	
4413	$I_{c}$	=	moment of inertia of a VBE taken perpendicular to the	direction of the web
4414			plate line, in. <sup>4</sup> (mm <sup>4</sup> )	
4415	L	=	distance between VBE centerlines, in. (mm)	
4416	h	=	distance between HBE centerlines, in. (mm)	
4417	$t_{\rm w}$	=	thickness of the web, in. (mm)	

4418 4419 4420	F5.4b.	HBE-to-VBE Connection Moment Ratio The moment ratio provisions in Section E3.4a shall be met for all HBE/VBE intersections without including the effects of the webs.
4421 4422 4423	F5.4c.	<b>Bracing</b> HBE shall be braced to satisfy the requirements for moderately ductile members in Section D1.2a.
4424 4425 4426 4427	F5.4d.	<b>Openings in Webs</b> Openings in webs shall be bounded on all sides by intermediate boundary elements extending the full width and height of the panel respectively, unless otherwise justified by testing and analysis or permitted by Section F5.7.
4428	F5.5.	Members
4429 4430 4431	F5.5a.	<b>Basic Requirements</b> HBE, VBE and intermediate boundary elements shall satisfy the requirements of Section D1.1 for highly ductile members.
4432 4433 4434	F5.5b.	Webs The panel design shear strength, $\phi V_n$ (LRFD), and the allowable shear strength, $V_n/\Omega$ (ASD), in accordance with the limit state of shear yielding, shall be determined as follows:
4435		$V_n = 0.42F_y t_w L_{cf} \sin 2\alpha $ (F5-1)
4436 4437 4438 4439 4440 4441 4442		where $ \begin{array}{rcl} L_{cf} &=& \text{clear distance between column flanges, in. (mm)} \\ t_w &=& \text{thickness of the web, in. (mm)} \\ \alpha &=& \text{angle of web yielding in degrees, as measured relative to the vertical. The angle of inclination, } \alpha, \text{ is permitted to be taken as } 40^\circ, \text{ or is permitted to be calculated as follows:} \\ \\ \tan^4\alpha &=& \frac{1+\frac{t_wL}{2A_c}}{1+t_wh\left(\frac{1}{A_b}+\frac{h^3}{360I_cL}\right)} \end{aligned} $
4443 4444 4445 4446	TE 5 -	where $A_b = \text{cross-sectional area of an HBE, in.}^2 \text{ (mm}^2\text{)}$ $A_c = \text{cross-sectional area of a VBE, in.}^2 \text{ (mm}^2\text{)}$
4447	F5.5c.	<b>IDL</b>

4448 4449 HBE shall be designed to preclude flexural yielding at regions other than near the beam-to-column connection. Either of the following is deemed to comply with this requirement:

4450 4451		(a)	HBE with available strength to resist twice the simple-span beam moment based on gravity loading and web-plate yielding.
4452 4453 4454		(b)	HBE with available strength to resist the simple-span beam moment based on gravity loading and web-plate yielding and with reduced flanges meeting the requirements of ANSI/AISC 358 Section 5.8 Step 1 with $c=0.25b_{\rm f}$ .
4455 4456	F5.5d.		cted Zone of SPSW shall satisfy Section D1.3 and include the following:
4457		(a)	The webs of SPSW
4458 4459		(b)	Elements that connect webs to HBEs and VBEs
4460 4461 4462 4463		(c)	The plastic hinging zones at each end of HBEs, over a region ranging from the face of the column to one beam depth beyond the face of the column, or as otherwise specified in Section E3.5c
4464	F5.6.	Conne	ections
4465	F5.6a.	Dema	nd Critical Welds
4466 4467			llowing welds are demand critical welds, and shall satisfy the requirements of Section and I2.3:
4468 4469		(a) (b)	Groove welds at column splices Welds at column-to-base plate connections
4470 4471			Exception: Welds need not be considered demand critical when both of the following conditions are satisfied:
4472 4473 4474 4475			<ol> <li>Column hinging at, or near, the base plate is precluded by conditions of restraint, and</li> <li>There is no net tension under load combinations including the overstrength seismic load.</li> </ol>
4476		(c)	Welds at HBE-to-VBE connections
4477 4478	F5.6b.		to-VBE Connections to-VBE connections shall satisfy the requirements of Section E1.6b.
4479 4480 4481 4482 4483	1.	The re capaci shall b	red Strength equired shear strength of an HBE-to-VBE connection shall be determined using the ty-limited seismic load effect. The capacity-limited horizontal seismic load effect, $E_{\rm cl}$ , we taken as the shear calculated from Equation E1-1 together with the shear resulting the expected yield strength in tension of the webs yielding at an angle $\alpha$ .

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**Panel Zones** 

The VBE panel zone next to the top and base HBE of the SPSW shall comply with the requirements in Section E3.6e.

# F5.6c. Connections of Webs to Boundary Elements

The required strength of web connections to the surrounding HBE and VBE shall equal the expected yield strength, in tension, of the web calculated at an angle  $\alpha$ .

# F5.6d. Column Splices

Column splices shall comply with the requirements of Section D2.5. Where welds are used to make the splice, they shall be complete-joint-penetration groove welds. Column splices shall be designed to develop at least 50% of the lesser plastic flexural strength,  $M_{\rm p}$ , of the connected members, divided by  $\alpha_s$ . The required shear strength,  $V_r$ , shall be determined by Equation F4-2.

## F5.7. Perforated Webs

# F5.7a. Regular Layout of Circular Perforations

A perforated plate conforming to this section is permitted to be used as the web of an SPSW. Perforated webs shall have a regular pattern of holes of uniform diameter spaced evenly over the entire web-plate area in an array pattern so that holes align diagonally at a uniform angle to vertical. A minimum of four horizontal and four vertical lines of holes shall be used. Edges of openings shall have a surface roughness of  $500 \mu$ -in. (13 microns) or less.

## 1. Strength

The panel design shear strength,  $\phi V_n$  (LRFD), and the allowable shear strength,  $V_n/\Omega$  (ASD), in accordance with the limit state of shear yielding, shall be determined as follows for perforated webs with holes that align diagonally at 45° from the horizontal:

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$$V_{n} = 0.42F_{y}t_{w}L_{cf}\left(1 - \frac{0.7D}{S_{diag}}\right)$$
 (F5-3)

$$φ = 0.90 \text{ (LRFD)}$$
  $Ω = 1.67 \text{ (ASD)}$ 

where

D = diameter of the holes, in. (mm)

 $S_{diag}$  = shortest center-to-center distance between the holes measured on the 45° diagonal, in. (mm)

User Note: Perforating webs in accordance with Section F5.7a forces the development of web yielding in a direction parallel to that of the holes alignment. As such, for the case addressed by Section F5.7a,  $\alpha$  is equal to 45°.

# **2. Spacing**

The spacing,  $S_{diag}$ , shall be at least 1.67D.

The distance between the first holes and web connections to the HBEs and VBEs shall be at least D, but shall not exceed (D+0.7 $S_{diag}$ ).

#### 3. Stiffness

The stiffness of such regularly perforated infill plates shall be calculated using an effective web-plate thickness, t<sub>eff</sub>, given by:

$$t_{\text{eff}} = \frac{1 - \frac{\pi}{4} \left(\frac{D}{S_{\text{diag}}}\right)}{1 - \frac{\pi}{4} \left(\frac{D}{S_{\text{diag}}}\right) \left(1 - \frac{N_r D \sin \alpha}{H_c}\right)} t_w$$
 (F5-4)

4537 where

 $H_c$  = clear column (and web-plate) height between beam flanges, in. (mm)

 $N_r$  = number of horizontal rows of perforations

 $t_w$  = web-plate thickness, in. (mm)

 $\alpha$  = angle of the shortest center-to-center lines in the opening array to vertical,

degrees

# 4. Effective Expected Tension Stress

The effective expected tension stress to be used in place of the effective tension stress for analysis per Section F5.3 is  $R_vF_v(1-0.7 \text{ D/S}_{diag})$ .

## F5.7b. Reinforced Corner Cut-Out

Quarter-circular cut-outs are permitted at the corners of the webs provided that the webs are connected to a reinforcement arching plate following the edge of the cut-outs. The plates shall be designed to allow development of the full strength of the solid web and maintain its resistance when subjected to deformations corresponding to the design story drift. This is deemed to be achieved if the following conditions are met.

## 1. Design for Tension

The arching plate shall have the available strength to resist the axial tension force resulting from web-plate tension in the absence of other forces:

$$P_{r} = \frac{R_{y}F_{y}t_{w}R^{2}/\alpha_{s}}{4e}$$
 (F5-5)

4560 4561 4562 4563 where 4564 = radius of the cut-out, in. (mm) 4565  $R_v$  = ratio of the expected yield stress to the specified minimum yield stress  $= R(1 - \sqrt{2}/2)$ , in. (mm) 4566 (F5-6)4567 HBEs and VBEs shall be designed to resist the tension axial forces acting at the end of the 4568 4569 arching reinforcement. 4570 4571 2. **Design for Combined Axial and Flexural Forces** 4572 4573 The arching plate shall have the available strength to resist the combined effects of axial 4574 force and moment in the plane of the web resulting from connection deformation in the 4575 absence of other forces. These forces are:  $P_{\rm r} = \frac{15EI_{\rm y}}{\alpha_{\rm s} \left(16e^2\right)} \left(\frac{\Delta}{\rm H}\right)$ 4576 (F5-7)4577 The moments are:  $M_r = P_r e^{-\frac{1}{2}}$ 4578 (F5-8)4579 4580 where E = modulus of elasticity, ksi (MPa) 4581 4582 H = height of story, in. (mm) $I_v = moment of inertia of the plate about the y-axis, in.<sup>4</sup> (mm<sup>4</sup>)$ 4583  $\Delta$  = design story drift, in. (mm) 4584 4585 4586 HBEs and VBEs shall be designed to resist the combined axial and flexural forces acting at the end of the arching reinforcement. 4587

		CHAPTER G	G-1
4600		CHAPT	TER G
4601		COMPOSITE MOMENT	-FRAME SYSTEMS
4602 4603 4604 4605 4606	the req	napter provides the basis of design, uirements for the system, members at frame systems.	
4607	The ch	apter is organized as follows:	
4608 4609 4610 4611 4612		G1. Composite Ordinary Moment G2. Composite Intermediate Mom G3. Composite Special Moment F G4. Composite Partially Restraine	ent Frames (C-IMF) rames (C-SMF)
4613 4614		<b>Note:</b> The requirements of this chap Specification and the applicable bu	pter are in addition to those required ilding code.
4615	G1.	COMPOSITE ORDINARY MO	MENT FRAMES (C-OMF)
4616	G1.1.	Scope	
4617 4618 4619 4620 4621		conformance with this section. T frames with fully restrained (FR	mes (C-OMF) shall be designed in his section is applicable to moment ) connections that consist of either ete columns and structural steel, imposite beams.
4622	G1.2.	Basis of Design	
4623 4624 4625			vith these provisions are expected to ation capacity in their members and
4626 4627 4628		The requirements of Sections A1, D2.7, and Chapter C apply to C Chapters A, B, D, I, J and K do no	A2, A3.5, A4, B1, B2, B3, B4 and C-OMF. All other requirements in apply to C-OMF.

G1.3. Analysis

connections.

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User Note: Composite ordinary moment frames, comparable to

reinforced concrete ordinary moment frames, are only permitted in

seismic design categories B or below in ASCE/SEI 7. This is in contrast to steel ordinary moment frames, which are permitted in higher seismic design categories. The design requirements are

commensurate with providing minimal ductility in the members and

CHAPTER G	G-2

4037		There are no requirements specific to this system.
4638	G1.4.	System Requirements
4639		There are no requirements specific to this system.
4640	G1.5.	Members
4641 4642 4643		There are no additional requirements for steel or composite members beyond those in the Specification. Reinforced concrete columns shall satisfy the requirements of ACI 318, excluding Chapter 18.
4644	G1.5a	. Protected Zones
4645		There are no designated protected zones.
4646	G1.6.	Connections
4647 4648 4649		Connections shall be fully restrained (FR) and shall satisfy the requirements of Section D2.7.
4650	G1.6a	Demand Critical Welds
4651		There are no requirements specific to this system.
4652	<b>G2.</b>	COMPOSITE INTERMEDIATE MOMENT FRAMES (C-IMF)
4653	G2.1.	Scope
4654 4655 4656 4657 4658		Composite intermediate moment frames (C-IMF) shall be designed in conformance with this section. This section is applicable to moment frames with fully restrained (FR) connections that consist of composite or reinforced concrete columns and structural steel, concrete-encased composite or composite beams.
4659	G2.2.	Basis of Design
4660 4661 4662 4663 4664 4665 4666		C-IMF designed in accordance with these provisions are expected to provide limited inelastic deformation capacity through flexural yielding of the C-IMF beams and columns, and shear yielding of the column panel zones. Design of connections of beams to columns, including panel zones, continuity plates and diaphragms shall provide the performance required by Section G2.6b, and demonstrate this conformance as required by Section G2.6c.
4667 4668 4669 4670 4671		User Note: Composite intermediate moment frames, comparable to reinforced concrete intermediate moment frames, are only permitted in seismic design categories C or below in ASCE/SEI 7. This is in contrast to steel intermediate moment frames, which are permitted in higher seismic design categories. The design requirements are 2016 Seismic Provisions for Structural Steel Buildings  Draft dated December 18, 2015  American Institute of Steel Construction

CHAPTER G	G-3

4672		commensurate with providing limited ductility in the members and connections.
4674	G2.3.	Analysis
4675		There are no requirements specific to this system.
4676	G2.4.	System Requirements
4677	G2.4a	. Stability Bracing of Beams
4678 4679		Beams shall be braced to satisfy the requirements for moderately ductile members in Section D1.2a.
4680 4681 4682 4683		In addition, unless otherwise indicated by testing, beam braces shall be placed near concentrated forces, changes in cross section, and othe locations where analysis indicates that a plastic hinge will form during inelastic deformations of the C-IMF.
4684 4685		The required strength and stiffness of stability bracing provided adjacent to plastic hinges shall be in accordance with Section D1.2c.
4686	G2.5.	Members
4687	G2.5a	. Basic Requirements
4688 4689		Steel and composite members shall satisfy the requirements of Section D1.1 for moderately ductile members.
4690	G2.5b	. Beam Flanges
4691 4692 4693 4694 4695		Abrupt changes in the beam flange area are prohibited in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is not permitted unless testing or qualification demonstrates that the resulting configuration is able to develop stable plastic hinges to accommodate the required story drift angle.
4696	G2.5c.	. Protected Zones
4697 4698 4699		The region at each end of the beam subject to inelastic straining shall be designated as a protected zone, and shall satisfy the requirements of Section D1.3.
4700 4701 4702 4703		<b>User Note:</b> The plastic hinge zones at the ends of C-IMF beams should be treated as protected zones. In general, the protected zone will extend from the face of the composite column to one-half of the beam depth beyond the plastic hinge point.
4704 4705	G2.6.	Connections

CHAPTER G	G-4

4706 4707	Connections shall be fully-restrained (FR) and shall satisfy the requirements of Section D2 and this section.
4708	G2.6a. Demand Critical Welds
4709	There are no requirements specific to this system.
4710	G2.6b. Beam-to-Column Connections
4711 4712	Beam-to-composite column connections used in the SFRS shall satisfy the following requirements:
4713 4714	(a) The connection shall be capable of accommodating a story drift angle of at least 0.02 rad.
4715 4716 4717 4718 4719 4720	(b) The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at a story drift angle of $0.02$ rad, where $M_p$ is defined as the plastic flexural strength of the steel, concrete-encased or composite beams and shall satisfy the requirements of Specification Chapter I.
4721	G2.6c. Conformance Demonstration
4722 4723 4724 4725	Beam-to-column connections used in the SFRS shall satisfy the requirements of Section G2.6b by one of the following:
4726	(a) Use of C-IMF connections designed in accordance with
4727 4728	ANSI/AISC 358.  (b) Use of a connection prequalified for C-IMF in accordance with
4729 4730	Section K1.  (c) Results of at least two qualifying cyclic test results conducted
4731 4732	in accordance with Section K2. The tests are permitted to be based on one of the following:
4733 4734 4735	(1) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section K2.
4736 4737 4738	(2) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection
4739 4740 4741 4742	processes, within the limits specified in Section K2.  (d) Calculations that are substantiated by mechanistic models and component limit state design criteria consistent with these provisions.
4743	G2.6d. Required Shear Strength
4744 4745	The required shear strength of the connection shall be determined using the capacity-limited seismic load effect. The capacity-limited 2016 Seismic Provisions for Structural Steel Buildings  Draft dated December 18, 2015  American Institute of Steel Construction

4746	horizontal seismic load effect, Ecl, shall be taken as:
4747	$E_{cl} = 2(1.1M_{p,exp})/L_h$ (G2-1)
4748 4749 4750 4751 4752 4753 4754 4755	where $M_{p,exp}$ is the expected flexural strength of the steel, concrete-encased or composite beam, kip-in. (N-mm). For a concrete-encased or composite beam, $M_{p,exp}$ shall be calculated using the plastic stress distribution or the strain compatibility method. Applicable $R_y$ factors shall be used for different elements of the cross section while establishing section force equilibrium and calculating the flexural strength. $L_h$ shall be equal to the distance between beam plastic hinge locations, in. (mm).
4756 4757	<b>User Note:</b> For steel beams, $M_{p,exp}$ in Equation G2-1 may be taken as $R_y M_p$ of the beam.
4758	G2.6e. Connection Diaphragm Plates
4759 4760	Connection diaphragm plates are permitted for filled composite columns both external to the column and internal to the column.
4761 4762	Where diaphragm plates are used, the thickness of the plates shall be at least the thickness of the beam flange.
4763 4764 4765 4766 4767	The diaphragm plates shall be welded around the full perimeter of the column using either complete-joint-penetration groove welds or two sided fillet welds. The required strength of these joints shall not be less than the available strength of the contact area of the plate with the column sides.
4768 4769	Internal diaphragms shall have circular openings sufficient for placing the concrete.
4770	G2.6f. Column Splices
4771 4772 4773 4774	In addition to the requirements of Section D2.5, column splices shall comply with the requirements of this section. Where welds are used to make the splice, they shall be complete-joint-penetration groove welds. When column splices are not made with groove welds, they

In addition to the requirements of Section D2.5, column splices shall comply with the requirements of this section. Where welds are used to make the splice, they shall be complete-joint-penetration groove welds. When column splices are not made with groove welds, they shall have a required flexural strength that is at least equal to the nominal flexural strength,  $M_{\rm pcc}$ , of the smaller composite column. The required shear strength of column web splices shall be at least equal to  $\Sigma M_{\rm pcc}/H$ , where  $\Sigma M_{\rm pcc}$  is the sum of the plastic flexural strengths at the top and bottom ends of the composite column. For composite columns, the nominal flexural strength shall satisfy the requirements of Specification Chapter I including the required axial strength,  $P_{\rm rc}$ .

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4782	G3.	COMPOSITE SPECIAL MOMENT FRAMES (C-SMF)
4783	G3.1.	Scope
4784 4785 4786 4787 4788		Composite special moment frames (C-SMF) shall be designed in conformance with this section. This section is applicable to moment frames with fully restrained (FR) connections that consist of either composite or reinforced concrete columns and either structural steel or concrete-encased composite or composite beams.
4789	G3.2.	Basis of Design
4790 4791 4792 4793 4794 4795 4796 4797 4798 4799		C-SMF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through flexural yielding of the C-SMF beams and limited yielding of the column panel zones. Except where otherwise permitted in this section, columns shall be designed to be stronger than the fully yielded and strain-hardened beams or girders. Flexural yielding of columns at the base is permitted. Design of connections of beams to columns, including panel zones, continuity plates and diaphragms shall provide the performance required by Section G3.6b, and demonstrate this conformance as required by Section G3.6c.
4800	G3.3.	Analysis
4801 4802 4803 4804		For special moment frame systems that consist of isolated planar frames, there are no additional analysis requirements.
4805 4806 4807 4808 4809		For moment frame systems that include columns that form part of two intersecting special moment frames in orthogonal or multi-axial directions, the column analysis of Section G3.4a shall consider the potential for beam yielding in both orthogonal directions simultaneously.
4810	G3.4.	System Requirements
4811	G3.4a	. Moment Ratio
4812 4813		The following relationship shall be satisfied at beam-to-column connections:
4814		$\frac{\sum M_{pcc}^{*}}{\sum M_{p,exp}^{*}} > 1.0 $ (G3-1)
4815		where
16		$\Sigma M^*_{\text{ncc}} = \text{sum of the } \frac{\text{moments in the columns above and below}}{\text{moments in the columns above and below}}$

ment [LCA1]: **Editorial** change to be stent with E3.4a symbols.

4817

 $\Sigma M^*_{pcc}$  = sum of the moments in the columns above and below the joint at the intersection of the beam and column

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4857	G3.5. Members
4858	G3.5a. Basic Requirements
4859 4860 4861	Steel and composite members shall satisfy the requirements of Sections D1.1 for highly ductile members.
4862 4863 4864 4865 4866	Exception: Reinforced concrete-encased beams shall satisfy the requirements for Section D1.1 for moderately ductile members if the reinforced concrete cover is at least 2 in. (50 mm) and confinement is provided by hoop reinforcement in regions where plastic hinges are expected to occur under seismic deformations. Hoop reinforcement
4867 4868	shall satisfy the requirements of ACI 318 Section 18.6.4.
4869 4870 4871 4872	Concrete-encased composite beams that are part of C-SMF shall also satisfy the following requirement. The distance from the maximum concrete compression fiber to the plastic neutral axis shall not exceed:
4873	$Y_{PNA} = \frac{Y_{con} + d}{1 + \left(\frac{1,700  F_{y}}{E}\right)} $ (G3-2)
4874	where
4875 4876 4877 4878 4879 4880	<ul> <li>E = modulus of elasticity of the steel beam, ksi (MPa)</li> <li>F<sub>y</sub> = specified minimum yield stress of the steel beam, ksi (MPa)</li> <li>Y<sub>con</sub> = distance from the top of the steel beam to the top of the concrete, in. (mm)</li> <li>d = overall beam depth, in. (mm)</li> </ul>
4881	G3.5b. Beam Flanges
4882 4883 4884 4885 4886	Abrupt changes in beam flange area are prohibited in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is prohibited unless testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges to accommodate the required story drift angle.
4887	G3.5c. Protected Zones
4888 4889 4890	The region at each end of the beam subject to inelastic straining shall be designated as a protected zone, and shall satisfy the requirements of Section D1.3.
4891 4892	User Note: The plastic hinge zones at the ends of C-SMF beams should be treated as protected zones. In general, the protected zone

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4893 4894		will extend from the face of the composite column to one-half of the beam depth beyond the plastic hinge point.
4895 4896	G3.6.	Connections
4897 4898		Connections shall be fully restrained (FR) and shall satisfy the requirements of Section D2 and this section.
4899		<b>User Note:</b> All subsections of Section D2 are relevant for C-SMF.
4900	G3.6a	Demand Critical Welds
4901 4902		The following welds are demand critical welds, and shall satisfy the requirements of Section A3.4b and I2.3:
4903		(a) Groove welds at column splices
4904		(b) Welds at the column-to-base plate connections
4905 4906 4907		Exception: Welds need not be considered demand critical when both of the following conditions are satisfied:
4908 4909		(1) Column hinging at, or near, the base plate is precluded by conditions of restraint, and
4910 4911		(2) There is no net tension under load combinations including the overstrength seismic load.
4912 4913 4914 4915		(c) Complete-joint-penetration groove welds of beam flanges to columns, diaphragm plates that serve as a continuation of beam flanges, shear plates within the girder depth that transition from the girder to an encased steel shape, and beam webs to columns
4916	G3.6b	Beam-to-Column Connections
4917 4918		Beam-to-composite column connections used in the SFRS shall satisfy the following requirements:
4919 4920		(a) The connection shall be capable of accommodating a story drift angle of at least 0.04 rad.
4921 4922 4923 4924		(b) The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at a story drift angle of $0.04$ rad, where $M_p$ is calculated as in Section G2.6b.

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1925	G3.6c. Conformance Demonstration
1926 1927	Beam-to-composite column connections used in the SFRS shall satisfy the requirements of Section G3.6b by one of the following:
1928 1929	(a) Use of C-SMF connections designed in accordance with ANSI/AISC 358
1930 1931 1932	(b) Use of a connection prequalified for C-SMF in accordance with Section K1.
1933 1934 1935 1936 1937	(c) The connections shall be qualified using test results obtained in accordance with Section K2. Results of at least two cyclic connection tests shall be provided, and shall be based on one of the following:
1938 1939 1940 1941	(1) Tests reported in research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section K2.
1942 1943 1944 1945 1946	(2) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified by Section K2.
1947 1948 1949 1950 1951 1952	(d) When beams are uninterrupted or continuous through the composite or reinforced concrete column, beam flange welded joints are not used, and the connection is not otherwise susceptible to premature fracture, other substantiating data is permitted to demonstrate conformance.
1953 1954 1955 1956 1957 1958 1959	Connections that accommodate the required story drift angle within the connection elements and provide the measured flexural resistance and shear strengths specified in Section G3.6d are permitted. In addition to satisfying the preceding requirements, the design shall demonstrate that any additional drift due to connection deformation is accommodated by the structure. The design shall include analysis for stability effects of the overall frame, including second-order effects.
1960	G3.6d. Required Shear Strength
1961 1962 1963	The required shear strength of the connection, $V_u$ , shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, $E_{cl}$ , shall be taken as:
1964	$E_{cl} = 2[1.1M_{p,exp}]/L_h$ (G3-3)

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4965 4966 4967 4968 4969		where $M_{p,exp}$ is the expected flexural strength of the steel, concrete-encased, or composite beams. For concrete-encased or composite beams, $M_{p,exp}$ shall be calculated according to Section G2.6d, and $L_h$ shall be equal to the distance between beam plastic hinge locations, in. (mm).
4970	G3.6e.	Connection Diaphragm Plates
4971 4972		The continuity plates or diaphragms used for infilled column moment connections shall satisfy the requirements of Section G2.6e.
4973	G3.6f.	Column Splices
4974 4975		Composite column splices shall satisfy the requirements of Section G2.6f.
4976 4977	G4.	COMPOSITE PARTIALLY RESTRAINED MOMENT FRAMES (C-PRMF)
4978	G4.1.	Scope
4979 4980 4981 4982 4983 4984 4985	G4.2.	Composite partially restrained moment frames (C-PRMF) shall be designed in conformance with this section. This section is applicable to moment frames that consist of structural steel columns and composite beams that are connected with partially restrained (PR) moment connections that satisfy the requirements in Specification Section B3.4b(b).  Basis of Design
4986 4987 4988 4989 4990 4991 4992 4993		C-PRMF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through yielding in the ductile components of the composite PR beam-to-column moment connections. Flexural yielding of columns at the base is permitted. Design of connections of beams to columns shall be based on connection tests that provide the performance required by Section G4.6c, and demonstrate this conformance as required by Section G4.6d.
4994	G4.3.	Analysis
4995 4996 4997		Connection flexibility and composite beam action shall be accounted for in determining the dynamic characteristics, strength and drift of C-PRMF.

with an effective moment of inertia of the composite section.

For purposes of analysis, the stiffness of beams shall be determined

4998 4999

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5000	G4.4.	System Re	quirements
5001		There are n	o requirements specific to this system.
5002	G4.5.	Members	
5003	G4.5a	. Columns	
5004 5005			nns shall satisfy the requirements of Sections D1.1 for ductile members.
5006	G4.5b	. Beams	
5007 5008 5009 5010		the require solid slab s	beams shall be unencased, fully composite, and shall meet ments of Section D1.1 for moderately ductile members. A hall be provided for a distance of 12 in. (300 mm) from the column in the direction of moment transfer.
5011	G4.5c	. Protected	Zones
5012		There are n	o designated protected zones.
5013 5014 5015	G4.6.	Connection	ns shall be partially restrained (PR) and shall satisfy the
5016			ts of Section D2 and this section.
5017		<b>User Note:</b>	All subsections of Section D2 are relevant for C-PRMF.
5018	G4.6a	. Demand C	ritical Welds
5019 5020			ing welds are demand critical welds, and shall satisfy the ts of Section A3.4b and I2.3:
5021		(a) G	roove welds at column splices
5022		(b) W	elds at the column-to-base plate connections
5023 5024 5025		cr	sception: Welds need not be considered demand itical when both of the following conditions are tisfied:
5026 5027		(1	) Column hinging at, or near, the base plate is precluded by conditions of restraint, and
5028 5029		(2	) There is no net tension under load combinations including the overstrength seismic load.
5030 5031	G4.6b	. Required S	Strength
5032		The require	ed strength of the beam-to-column PR moment connections
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5033 5034	shall be determined including the effects of connection flexibility and second-order moments.
5035	G4.6c. Beam-to-Column Connections
5036 5037	Beam-to-composite column connections used in the SFRS shall satisfy the following requirements:
5038 5039	(a) The connection shall be capable of accommodating a connection rotation of at least 0.02 rad.
5040 5041 5042 5043 5044 5045	(b) The measured flexural resistance of the connection determined at the column face shall increase monotonically to a value of at least $0.5M_p$ of the connected beam at a connection rotation of $0.02$ rad, where $M_p$ is defined as the moment corresponding to plastic stress distribution over the composite cross section, and shall satisfy the requirements of Specification Chapter I.
5046	G4.6d. Conformance Demonstration
5047 5048 5049 5050 5051	Beam-to-column connections used in the SFRS shall satisfy the requirements of Section G4.6c by provision of qualifying cyclic test results in accordance with Section K2. Results of at least two cyclic connection tests shall be provided, and shall be based on one of the following:
5052 5053 5054 5055	(a) Tests reported in research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section K2.
5056 5057 5058 5059	(b) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified by Section K2.
5060	G4.6e. Column Splices

Column splices shall satisfy the requirements of Section G2.6f.

5100		CHAPTER H
5101	C	OMPOSITE BRACED-FRAME AND SHEAR-WALL SYSTEMS
5102 5103 5104 5105 5106	the req	napter provides the basis of design, the requirements for analysis, and quirements for the system, members and connections for composite frame and shear wall systems.
5107 5108 5109 5110 5111 5112 5113 5114 5115	The ch	H1. Composite Ordinary Braced Frames (C-OBF) H2. Composite Special Concentrically Braced Frames (C-SCBF) H3. Composite Eccentrically Braced Frames (C-EBF) H4. Composite Ordinary Shear Walls (C-OSW) H5. Composite Special Shear Walls (C-SSW) H6. Composite Plate Shear Walls—Concrete Encased (C-PSW/CE) H7. Composite Plate Shear Walls—Concrete Filled (C-PSW/CF)
5116 5117		<b>Note:</b> The requirements of this chapter are in addition to those required Specification and the applicable building code.
5118	H1.	COMPOSITE ORDINARY BRACED FRAMES (C-OBF)
5119	H1.1.	Scope
5120 5121 5122 5123 5124 5125 5126 5127		Composite ordinary braced frames (C-OBF) shall be designed in conformance with this section. Columns shall be structural steel, encased composite, filled composite or reinforced concrete members. Beams shall be either structural steel or composite beams. Braces shall be structural steel or filled composite members. This section is applicable to braced frames that consist of concentrically connected members where at least one of the elements (columns, beams or braces) is a composite or reinforced concrete member.
5128	H1.2.	Basis of Design
5129 5130 5131 5132		This section is applicable to braced frames that consist of concentrically connected members. Eccentricities less than the beam depth are permitted if they are accounted for in the member design by determination of eccentric moments.
5133 5134 5135		C-OBF designed in accordance with these provisions are expected to provide limited inelastic deformations in their members and connections.
5136 5137		The requirements of Sections A1, A2, A3.5, A4, B1, B2, B3, B4 and D2.7, and Chapter C apply to C-OBF. All other requirements in

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5138		Chapters A, B, D, I, J and K do not apply to C-OBF.
5139 5140 5141 5142 5143 5144 5145		<b>User Note:</b> Composite ordinary braced frames, comparable to other steel braced frames designed per the Specification using $R=3$ , are only permitted in seismic design categories A, B or C in ASCE/SEI 7. This is in contrast to steel ordinary braced frames, which are permitted in higher seismic design categories. The design requirements are commensurate with providing minimal ductility in the members and connections.
5146	H1.3.	Analysis
5147		There are no requirements specific to this system.
5148	H1.4.	System Requirements
5149		There are no requirements specific to this system.
5150	H1.5.	Members
5151	H1.5a	. Basic Requirements
5152		There are no requirements specific to this system.
5153	H1.5b	. Columns
5154 5155 5156		There are no requirements specific to this system. Reinforced concrete columns shall satisfy the requirements of ACI 318, excluding Chapter 18.
5157	Н1.5с.	. Braces
5158		There are no requirements specific to this system.
5159	H1.5d	. Protected Zones
5160		There are no designated protected zones.
5161	H1.6.	Connections
5162		Connections shall satisfy the requirements of Section D2.7.
5163	H1.6a	Demand Critical Welds
5164		There are no requirements specific to this system.
5165 5166	H2.	COMPOSITE SPECIAL CONCENTRICALLY BRACED FRAMES (C-SCBF)
5167	H2.1.	Scope

5168 5169 5170 5171 5172 5173		Composite special concentrically braced frames (C-SCBF) shall be designed in conformance with this section. Columns shall be encased or filled composite. Beams shall be either structural steel or composite beams. Braces shall be structural steel or filled composite members. Collector beams that connect C-SCBF braces shall be considered to be part of the C-SCBF.
5174	H2.2.	Basis of Design
5175 5176 5177 5178 5179		This section is applicable to braced frames that consist of concentrically connected members. Eccentricities less than the beam depth are permitted if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.
5180 5181 5182		C-SCBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through brace buckling and yielding of the brace in tension.
5183	H2.3.	Analysis
5184 5185 5186 5187		The analysis requirements for C-SCBF shall satisfy the analysis requirements of Section F2.3 modified to account for the entire composite section in determining the expected brace strengths in tension and compression.
5188	H2.4.	System Requirements
5189 5190 5191		The system requirements for C-SCBF shall satisfy the system requirements of Section F2.4. Composite braces are not permitted for use in multi-tiered braced frames.
5192	H2.5.	Members
5193	H2.5a.	Basic Requirements
5194 5195 5196 5197		Composite columns and steel or composite braces shall satisfy the requirements of Section D1.1 for highly ductile members. Steel or composite beams shall satisfy the requirements of Section D1.1 for moderately ductile members.
5198 5199 5200 5201		<b>User Note:</b> In order to satisfy this requirement, the actual width-to-thickness ratio of square and rectangular filled composite braces may be multiplied by a factor, [ $(0.264 + 0.0082L_c/r)$ ], for $L_c/r$ between 35 and 90; $L_c/r$ being the effective slenderness ratio of the brace.

5202

**H2.5b. Diagonal Braces** 

5203 5204 5205		require	aral steel and filled composite braces shall satisfy the ements for SCBF of Section F2.5b. The radius of gyration in F2.5b shall be taken as that of the steel section alone.
5206	H2.5c.	Protec	eted Zones
5207 5208			rotected zone of C-SCBF shall satisfy Section D1.3 and include lowing:
5209 5210 5211		ad	r braces, the center one-quarter of the brace length and a zone acent to each connection equal to the brace depth in the plane of ckling
5212		(b) Ele	ements that connect braces to beams and columns.
5213	H2.6.	Conne	ections
5214 5215		_	n of connections in C-SCBF shall be based on Section D2 and ovisions of this section.
5216	H2.6a	. Dema	nd Critical Welds
5217 5218			ollowing welds are demand critical welds, and shall satisfy the ements of Section A3.4b and I2.3:
5219		(a)	Groove welds at column splices
5220		(b)	Welds at the column-to-base plate connections
5221 5222 5223			Exception: Welds need not be considered demand critical when both of the following conditions are satisfied:
5224 5225			(1) Column hinging at, or near, the base plate is precluded by conditions of restraint, and
5226 5227			(2) There is no net tension under load combinations including the overstrength seismic load.
5228 5229		(c)	Welds at beam-to-column connections conforming to Section H2.6b(b)
5230	H2.6b	. Beam	-to-Column Connections
5231 5232 5233			a brace or gusset plate connects to both members at a beam-to- n connection, the connection shall conform to one of the ing:
5234 5235 5236		(a)	The connection shall be a simple connection meeting the requirements of Specification Section B3.4a where the required rotation is taken to be 0.025 rad; or  2016 Seismic Provisions for Structural Steel Buildings

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5237 (b) Beam-to-column connections shall satisfy the requirements for FR moment connections as specified in Sections D2, G2.6d and G2.6e.

The required flexural strength of the connection shall be determined from analysis and shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the overstrength seismic load.

**H2.6c. Brace Connections** 

Brace connections shall satisfy the requirement of Section F2.6c, except that the required strength shall be modified to account for the entire composite section in determining the expected brace strength in tension and compression. Applicable  $R_y$  factors shall be used for different elements of the cross section for calculating the expected brace strength. The expected brace flexural strength shall be determined as  $M_{p,exp}$ , where  $M_{p,exp}$  is calculated as specified in Section G2.6d.

**H2.6d.** Column Splices

In addition to the requirements of Section D2.5, column splices shall comply with the requirements of this section. Where welds are used to make the splice, they shall be complete-joint-penetration groove welds. When column splices are not made with groove welds, they shall have a required flexural strength that is at least equal to the nominal flexural strength,  $M_{pcc}$ , of the smaller composite column. The required shear strength of column web splices shall be at least equal to  $\Sigma M_{pcc}/H$ , where  $\Sigma M_{pcc}$  is the sum of the nominal flexural strengths at the top and bottom ends of the composite column. The nominal flexural strength shall satisfy the requirements of Specification Chapter I with consideration of the required axial strength,  $P_{rc}$ .

## H3. COMPOSITE ECCENTRICALLY BRACED FRAMES (C-EBF)

**H3.1. Scope** 

Composite eccentrically braced frames (C-EBF) shall be designed in conformance with this section. Columns shall be encased composite or filled composite. Beams shall be structural steel or composite beams. Links shall be structural steel. Braces shall be structural steel or filled composite members. This section is applicable to braced frames for which one end of each brace intersects a beam at an eccentricity from

5277 5278 5279		the intersection of the centerlines of the beam and an adjacent brace or column.
5280	H3.2.	Basis of Design
5281 5282		C-EBF shall satisfy the requirements of Section F3.2, except as modified in this section.
5283 5284 5285 5286 5287 5288 5289 5290		This section is applicable to braced frames for which one end of each brace intersects a beam at an eccentricity from the intersection of the centerlines of the beam and an adjacent brace or column, forming a link that is subject to shear and flexure. Eccentricities less than the beam depth are permitted in the brace connection away from the link if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.
5291 5292 5293		C-EBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through shear or flexural yielding in the links.
5294 5295		The available strength of members shall satisfy the requirements in the Specification, except as modified in this section.
5296	Н3.3.	Analysis
5297 5298		The analysis of C-EBF shall satisfy the analysis requirements of Section F3.3.
5299	H3.4.	System Requirements
5300 5301		The system requirements for C-EBF shall satisfy the system requirements of Section F3.4.
5302 5303	Н3.5.	Members
5304 5305 5306 5307		The member requirements of C-EBF shall satisfy the member requirements of Section F3.5.
5308 5309	Н3.6.	Connections
5310 5311		The connection requirements of C-EBF shall satisfy the connection requirements of Section F3.6 except as noted in the following.
5312	H3.6a.	Beam-to-Column Connections
5313 5314 5315		Where a brace or gusset plate connects to both members at a beam-to-column connection, the connection shall conform to one of the following:  2016 Seismic Provisions for Structural Steel Buildings

5316 5317 5318		(a)	The connection shall be a simple connection meeting the requirements of Specification Section B3.4a where the required rotation is taken to be 0.025 rad; or
5319 5320 5321 5322		(b)	Beam-to-column connections shall satisfy the requirements for fully restrained (FR) moment connections as specified in Sections D2, G2.6d and G2.6e.
5322 5323 5324 5325 5326 5327 5328			The required flexural strength of the connection shall be determined from analysis and shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the overstrength seismic load.
5329	H4. C	COMPO	OSITE ORDINARY SHEAR WALLS (C-OSW)
5330	H4.1.	Scope	
5331 5332 5333 5334 5335 5336		confor reinfor and compo	osite ordinary shear walls (C-OSW) shall be designed in mance with this section. This section is applicable to uncoupled reed concrete shear walls with composite boundary elements, coupled reinforced concrete shear walls, with or without site boundary elements, with structural steel or composite ng beams that connect two or more adjacent walls.
5337	H4.2.	Basis	of Design
5338 5339 5340		provid	W designed in accordance with these provisions are expected to the limited inelastic deformation capacity through yielding in the reced concrete walls and the steel or composite elements.
5341 5342			orced concrete walls shall satisfy the requirements of ACI 318 ling Chapter 18, except as modified in this section.
5343	H4.3.	Analy	sis
5344 5345		Analysthis se	sis shall satisfy the requirements of Chapter C as modified in ction.
5346 5347 5348		(a)	Uncracked effective stiffness values for elastic analysis shall be assigned in accordance with ACI 318 Chapter 6 for wall piers and composite coupling beams.
5349 5350 5351		(b)	When concrete-encased shapes function as boundary members, the analysis shall be based upon a transformed concrete section using elastic material properties.

**H4.4.** System Requirements

5353	In coupled walls, it is permitted to redistribute coupling beam forces
5354	vertically to adjacent floors. The shear in any individual coupling
5355	beam shall not be reduced by more than 20% of the elastically
5356	determined value. The sum of the coupling beam shear resistance over
5357	the height of the building shall be greater than or equal to the sum of
5358	the elastically determined values.

## H4.5. Members

## H4.5a. Boundary Members

Boundary members shall satisfy the following requirements:

- (a) The required axial strength of the boundary member shall be determined assuming that the shear forces are carried by the reinforced concrete wall and the entire gravity and overturning forces are carried by the boundary members in conjunction with the shear wall.
- (b) When the concrete-encased structural steel boundary member qualifies as a composite column as defined in Specification Chapter I, it shall be designed as a composite column to satisfy the requirements of Chapter I of the Specification.
- (c) Headed studs or welded reinforcement anchors shall be provided to transfer required shear strengths between the structural steel boundary members and reinforced concrete walls. Headed studs, if used, shall satisfy the requirements of Specification Chapter I. Welded reinforcement anchors, if used, shall satisfy the requirements of Structural Welding Code—Reinforcing Steel (AWS D1.4/D1.4M).

## H4.5b. Coupling Beams

## 1. Structural Steel Coupling Beams

Structural steel coupling beams that are used between adjacent reinforced concrete walls shall satisfy the requirements of the Specification and this section. The following requirements apply to wide flange steel coupling beams.

- (a) Steel coupling beams shall be designed in accordance with Chapters F and G of the Specification.
- (b) The available connection shear strength,  $\phi V_{n,connection}$ , shall be computed from Equations H4-1 and H4-1M, with  $\phi = 0.90$ .

		$V_{n,\text{connection}} = 1.54 \sqrt{f_{c}'} \left(\frac{b_{w}}{b_{f}}\right)^{0.66} \beta_{1} b_{f} L_{e} \left[\frac{0.58 - 0.22 \beta_{1}}{0.88 + \frac{g}{2L_{e}}}\right] $ (H4-1)
5390		
		$V_{n,\text{connection}} = 4.04 \sqrt{f_{c}'} \left(\frac{b_{w}}{b_{f}}\right)^{0.66} \beta_{1} b_{f} L_{e} \left[\frac{0.58 - 0.22 \beta_{1}}{0.88 + \frac{g}{2L_{e}}}\right] $ (S.I.) (H4-1M)
5391		
5392		where
5393		L <sub>e</sub> = embedment length of coupling beam
5394		measured from the face of the wall, in.
5395		(mm)
5396		$b_w = \text{thickness of wall pier, in. (mm)}$
5397		$b_f$ = beam flange width, in. (mm)
5398		f' <sub>c</sub> = concrete compressive strength, ksi (MPa)
5399		$\beta_1$ = factor relating depth of equivalent
5400		rectangular compressive stress block to
5401		neutral axis depth, as defined in ACI 318
5402		g = clear span of coupling beam, in. (mm)
5403		(c) Vertical wall reinforcement with nominal axial strength
5404		equal to the required shear strength, V <sub>n</sub> , of the coupling
5405		beam shall be placed over the embedment length of the
5406		beam with two-thirds of the steel located over the first
5407		half of the embedment length. This wall reinforcement
5408		shall extend a distance of at least one tension
5409		development length above and below the flanges of the
5410		coupling beam. It is permitted to use vertical
5411		reinforcement placed for other purposes, such as for
5412		vertical boundary members, as part of the required
5413		vertical reinforcement.
5414	2.	Composite Coupling Beams
5415		
5416		Encased composite sections serving as coupling beams shall
5417		satisfy the following requirements:
5418		(a) Coupling beams shall have an embedment length into
5419		the reinforced concrete wall that is sufficient to develop
5420		the required shear strength, where the connection
5421		strength is calculated with Equation H4-1 or H4-1M.
5422		The available shear strength of the composite beam,
5.422		11. 11. 11. 11. 11. 11. 11. 11. 11. 11.

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with  $\phi = 0.90$ .

 $\phi V_{n,\text{comp}},$  is computed from Equation H4-2 and H4-2M,

5422 5423

5424

H-10

5426 where  $A_{sr}$  = area of transverse reinforcement, in.<sup>2</sup> (mm<sup>2</sup>) 5427 5428  $F_{vsr}$  = specified minimum yield stress 5429 transverse reinforcement, ksi (MPa)  $V_p = 0.6F_vA_w$ , kips (N) 5430  $A_{\rm w}$  = area of steel beam web, in.<sup>2</sup> (mm<sup>2</sup>) 5431  $b_{wc}$  = width of concrete encasement, in. (mm) 5432 d<sub>c</sub> = effective depth of concrete encasement, in. 5433 5434 (mm) = spacing of transverse reinforcement, in. 5435

5437 **H4.5c. Protected Zones** 

5436

- 5438 There are no designated protected zones.
- 5439 **H4.6.** Connections
- There are no additional requirements beyond Section H4.5.

(mm)

- 5441 **H4.6a. Demand Critical Welds**
- There are no requirements specific to this system.
- 5443 H5. COMPOSITE SPECIAL SHEAR WALLS (C-SSW)
- 5444 **H5.1. Scope**

Composite special shear walls (C-SSW) shall be designed in conformance with this section. This section is applicable when reinforced concrete walls are composite with structural steel elements, including structural steel or composite sections acting as boundary members for the walls and structural steel or composite coupling beams that connect two or more adjacent reinforced concrete walls.

### 5451 **H5.2.** Basis of Design

C-SSW designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through yielding in the reinforced concrete walls and the steel or composite elements. Reinforced concrete wall elements shall be designed to provide inelastic deformations at the design story drift consistent with ACI 318 including Chapter 18. Structural steel and composite coupling beams

5458 5459 5460 5461 5462 5463 5464		drift the and the expect Structus provide	be designed to provide inelastic deformations at the design story arough yielding in flexure or shear. Coupling beam connections he design of the walls shall be designed to account for the ed strength including strain hardening in the coupling beams. The ural steel and composite boundary elements shall be designed to be inelastic deformations at the design story drift through any due to axial force.
5465 5466 5467		shear	V systems shall satisfy the requirements of Section H4 and the wall requirements of ACI 318 including Chapter 18, except as ted in this section.
5468 5469 5470 5471 5472 5473		critical can be Section consid	<b>Note</b> : Steel coupling beams can be proportioned to be shearl or flexural-critical. Coupling beams with lengths $g \le 1.6 M_p/V_p$ assumed to be shear-critical, where $g$ , $M_p$ , and $V_p$ are defined in H4.5b(1). Coupling beams with lengths $g \ge 2.6 M_p/V_p$ may be ered to be flexure-critical. Coupling beam lengths between these clues are considered to yield in flexure and shear simultaneously.
5474	Н5.3.	Analy	sis
5475 5476		Analys	sis requirements of Section H4.3 shall be met with the following ions:
5477 5478 5479		(a)	Cracked effective stiffness values for elastic analysis shall be assigned in accordance with ACI 318 Chapter 6 practice for wall piers and composite coupling beams.
5480 5481		(b)	Effects of shear distortion of the steel coupling beam shall be taken into account.
5482	H5.4.	System	n Requirements
<ul><li>5483</li><li>5484</li><li>5485</li></ul>		In add shall b	ition to the system requirements of Section H4.4, the following e satisfied:
5486 5487		(a)	In coupled walls, coupling beams shall yield over the height of the structure followed by yielding at the base of the wall piers.
5488 5489 5490 5491 5492 5493 5494 5495		(b)	In coupled walls, the axial design strength of the wall at the balanced condition, P <sub>b</sub> , shall equal or exceed the total required compressive axial strength in a wall pier, computed as the sum of the required strengths attributed to the walls from the gravity load components of the lateral load combination plus the sum of the expected beam shear strengths increased by a factor of 1.1 to reflect the effects of strain hardening of all the coupling beams framing into the walls.

5496	H5.5.	Members
5497	H5.5a	Ductile Elements
5498 5499		Welding on steel coupling beams is permitted for attachment of stiffeners, as required in Section F3.5b.4.
5500	H5.5b	. Boundary Members
5501 5502		Unencased structural steel columns shall satisfy the requirements of Section D1.1 for highly ductile members and Section H4.5a(a).
5503 5504 5505 5506 5507 5508 5509 5510 5511 5512 5513 5514		In addition to the requirements of Sections H4.3(b) and H4.5a(b), the requirements in this section shall apply to walls with concrete-encased structural steel boundary members. Concrete-encased structural steel boundary members that qualify as composite columns in Specification Chapter I shall meet the highly ductile member requirements of Section D1.4b(b). Otherwise, such members shall be designed as composite compression members to satisfy the requirements of ACI 318 including the special seismic requirements for boundary members in ACI 318 Section 18.10.6. Transverse reinforcement for confinement of the composite boundary member shall extend a distance of 2h into the wall, where h is the overall depth of the boundary member in the plane of the wall.
5515 5516		Headed studs or welded reinforcing anchors shall be provided as specified in Section H4.5a(c).
5517 5518 5519		Vertical wall reinforcement as specified in Section H4.5b.1(d) shall be confined by transverse reinforcement that meets the requirements for boundary members of ACI 318 Section 18.10.6.
5520	Н5.5с.	Steel Coupling Beams
5521 5522		The design and detailing of steel coupling beams shall satisfy the following:
5523 5524		(a) The embedment length, L <sub>e</sub> , of the coupling beam shall be computed from Equations H5-1 and H5-1M.

	г л
5525	$V_{n} = 1.54 \sqrt{f_{c}'} \left(\frac{b_{w}}{b_{f}}\right)^{0.66} \beta_{l} b_{f} L_{e} \left[\frac{0.58 - 0.22 \beta_{l}}{0.88 + \frac{g}{2L_{e}}}\right] $ (H5-1)
	$V_{n} = 4.04 \sqrt{f_{c}!} \left( \frac{b_{w}}{b_{f}} \right)^{0.66} \beta_{l} b_{f} L_{e} \left[ \frac{0.58 - 0.22 \beta_{l}}{0.88 + \frac{g}{2L_{e}}} \right] $ (H5-1M)
5526	
5527	where
5528 5529 5530 5531	L <sub>e</sub> = coupling beam embedment length considered to begin inside the first layer of confining reinforcement, nearest to the edge of the wall, in the wall boundary member, in, (mm)
5532 5533	g = clear span of the coupling beam plus the wall concrete cover at each end of the beam, in, (mm)
5534 5535	$V_n$ = expected beam shear strength computed from Equation H5-2, kips (N)
5536	$= \frac{2(1.1R_{y})M_{p}}{g} \le (1.1R_{y})V_{p} $ (H5-2)
5537	where
5538	A <sub>tw</sub> = area of steel beam web, in. <sup>2</sup> (mm <sup>2</sup> ).  M <sub>p</sub> = F <sub>y</sub> Z, kip-in. (N-mm)  V <sub>n</sub> = expected shear strength of a steel coupling beam, kips (N)
5539	$M_p = F_yZ_x$ , kip-in. (N-mm)
5540 5541	$V_n$ = expected shear strength of a steel coupling
5542	$V_p = 0.6F_yA_{tw}, kips(N)$
5543	v <sub>p</sub> o.or yr <sub>4w</sub> , kips (1v)
5544	(b) Structural steel coupling beams shall satisfy the requirements
5545	of Section F3.5b, except that for built-up cross sections, the
5546	flange-to-web welds are permitted to be made with two-sided
5547	fillet, partial-joint-penetration, or complete-joint-penetration
5548	groove welds that develop the expected strength of the beam.
5549	When required in Section F3.5b.4, the coupling beam rotation
5550	shall be assumed as a 0.08 rad link rotation unless a smaller

F3.5b.4.

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value is justified by rational analysis of the inelastic

deformations that are expected under the design story drift.

Face bearing plates shall be provided on both sides of the coupling beams at the face of the reinforced concrete wall.

These plates shall meet the detailing requirements of Section

5557	(c) Steel coupling beams shall comply with the requirements of
5558	Section D1.1 for highly ductile members. Flanges of coupling
5559	beams with I-shaped sections with $g \le 1.6 M_p/V_p$ are permitted
5560	to satisfy the requirements for moderately ductile members.
5561	(d) Embedded steel members shall be provided with two regions of
5562	vertical transfer reinforcement attached to both the top and
5563	bottom flanges of the embedded member. The first region shall
5564	be located to coincide with the location of longitudinal wall
5565	reinforcing bars closest to the face of the wall. The second shall
5566	be placed a distance no less than d/2 from the termination of
5567	the embedment length. All transfer reinforcement bars shall be
5568	fully developed where they engage the coupling beam flanges.
5569	It is permitted to use straight, hooked or mechanical anchorage
5570	to provide development. It is permitted to use mechanical
5571	couplers welded to the flanges to attach the vertical transfer
5572	bars. The area of vertical transfer reinforcement required is
5573	computed by Equation H5-1:
5574	computed by Equation 112 1.
5575	$A_{tb} \ge 0.03 f_c' L_e b_f / F_{vsr}$ (H5-1)
5576	$- tb = 3000 - c - e^{-1} / - ysr$
5577	where
5578	$A_{tb}$ = area of transfer reinforcement required in each of the first
5579	and second regions attached to each of the top and bottom
5580	flanges, in. <sup>2</sup> (mm <sup>2</sup> )
5581	$F_{ysr}$ = specified minimum yield stress of transfer reinforcement,
5582	ksi (MPa)
5583	$L_e = \text{embedment length, in. (mm)}$
5584	$b_f$ = beam flange width, in. (mm)
5585	
	f' <sub>c</sub> = concrete compressive strength, ksi (MPa)
5586 5587	The area of vertical transfer rainforcement shall not avered that
5587	The area of vertical transfer reinforcement shall not exceed that
5588	computed by Equation H5-2:
5589 5500	$\sum_{A} A = 0.091 \text{ b} A \qquad (H5.2)$
5590	$\sum A_{tb} < 0.08 L_{e} b_{w} - A_{sr} $ (H5-2)
5591	1
5592	where
5593	$\Sigma A_{tb}$ = total area of transfer reinforcement provided in both
5594	the first and second regions attached to either the top
5595	or bottom flange, in. <sup>2</sup> (mm <sup>2</sup> )
5596	$A_{sr}$ = area of longitudinal wall reinforcement provided over
5597	the embedment length, L <sub>e</sub> , in. <sup>2</sup> (mm <sup>2</sup> )
5598	$b_{\rm w} = {\rm width\ of\ wall,\ in.\ (mm)}$

## **H5.5d.** Composite Coupling Beams

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Encased composite sections serving as coupling beams shall satisfy the 2016 Seismic Provisions for Structural Steel Buildings
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5601 5602 5603 5604		F3.5b.4 Equation	ements of Section H5.5c except the requirements of Section 4 need not be met, and Equation H5-3 shall be used instead of on H4-2. For all encased composite coupling beams, the g expected shear strength, V <sub>comp</sub> , is:
5605			
5606			$=1.1R_{y}V_{p} + 0.08\sqrt{R_{c}f_{c}'}b_{wc}d_{c} + \frac{1.33R_{yr}A_{s}F_{ysr}d_{c}}{s}$ $=1.1R_{y}V_{p} + 0.21\sqrt{R_{c}f_{c}'}b_{wc}d_{c} + \frac{1.33R_{yr}A_{s}F_{ysr}d_{c}}{s}$ (H5-3) $=1.1R_{y}V_{p} + 0.21\sqrt{R_{c}f_{c}'}b_{wc}d_{c} + \frac{1.33R_{yr}A_{s}F_{ysr}d_{c}}{s}$ (S.I.)
5607			<u> </u>
5608			
5609		Where	
5610		Г	· 11 ( C) ( C) ( (A) (A) (A) (A) (A) (A) (A) (A) (A)
5611 5612			ysr = yield stress of transverse reinforcement, ksi (MPa) factor to account for expected strength of concrete = 1.5
5613		_	$_{\pi}$ = ratio of the expected yield stress of the transverse
5614		- Ly	reinforcement material to the specified minimum yield
5615			stress, F <sub>ysr</sub>
5616	H5.5e	. Protec	eted Zones
5617			
5618			ear span of the coupling beam between the faces of the shear
5619			shall be designated as a protected zone, and shall satisfy the
5620 5621		-	ements of Section D1.3. Attachment of stiffeners and face g plates as required by Section H5.5c(b) shall be permitted.
5622	Н5.6.	Conne	ections
5623	H5.6a	. Demar	nd Critical Welds
5624 5625			ollowing welds are demand critical welds, and shall satisfy the ements of Section A3.4b and I2.3:
5626		(a)	Groove welds at column splices
5627		(b)	Welds at the column-to-base plate connections
		( )	
5628			Exception: Welds need not be considered demand
5629			critical when both of the following conditions are
			1
5629			critical when both of the following conditions are
5629 5630 5631 5632			critical when both of the following conditions are satisfied:  (1) Column hinging at, or near, the base plate is precluded by conditions of restraint, and
<ul><li>5629</li><li>5630</li><li>5631</li></ul>			critical when both of the following conditions are satisfied:  (1) Column hinging at, or near, the base plate is

5635 5636	H5.6b.	. Column Splices
5637		Column splices shall be designed following the requirements of
5638		Section G2.6f.
5639	Н6.	COMPOSITE PLATE SHEAR WALLS - CONCRETE
5640	ENCA	SED (C-PSW/CE)
5641	Н6.1.	Scope
5642		Composite plate shear walls-concrete encased (C-PSW/CE) shall be
5643		designed in conformance with this section. C-PSW/CE consist of steel
5644 5645		plates with reinforced concrete encasement on one or both sides of the plate and structural steel or composite boundary members.
5646	Н6.2.	Basis of Design
5647		C-PSW/CE designed in accordance with these provisions are expected
5648		to provide significant inelastic deformation capacity through yielding
5649		in the plate webs. The horizontal boundary elements (HBEs) and
5650		vertical boundary elements (VBEs) adjacent to the composite webs
5651		shall be designed to remain essentially elastic under the maximum
5652		forces that can be generated by the fully yielded steel webs along with
5653		the reinforced concrete webs after the steel web has fully yielded,
5654		except that plastic hinging at the ends of HBEs is permitted.
5655	Н6.3.	Analysis
5656	H6.3a.	Webs
5657		The analysis shall account for openings in the web.
5658	H6.3b.	Other Members and Connections
5659		Columns, beams and connections in C-PSW/CE shall be designed to
5660		resist seismic forces determined from an analysis that includes the
5661		expected strength of the steel webs in shear, 0.6R <sub>y</sub> F <sub>y</sub> A <sub>sp</sub> , where A <sub>sp</sub> is
5662		the horizontal area of the stiffened steel plate, in. <sup>2</sup> (mm <sup>2</sup> ), and any
5663 5664		reinforced concrete portions of the wall active at the design story drift. The VBEs are permitted to yield at the base.
5665	H6.4.	System Requirements
5666	H6.4a.	Steel Plate Thickness
5667		Steel plates with thickness less than 3/8 in. (9.5 mm) are not permitted.
5668 5669	H6.4b	Stiffness of Vertical Boundary Elements
5670		The VBEs shall satisfy the requirements of Section F5.4a.
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5671	H6.4c. HBE-to-VBE Connection Moment Ratio
5672 5673	The beam-column moment ratio shall satisfy the requirements of Section F5.4b.
5674	H6.4d. Bracing
5675 5676	HBE shall be braced to satisfy the requirements for moderately ductile members.
5677	H6.4e. Openings in Webs
5678 5679	Boundary members shall be provided around openings in shear wall webs as required by analysis.
5680	H6.5. Members
5681	H6.5a. Basic Requirements
5682 5683	Steel and composite HBE and VBE shall satisfy the requirements of Section D1.1 for highly ductile members.
5684	H6.5b. Webs
5685 5686 5687	The design shear strength, $\phi V_n$ , or the allowable shear strength, $V_n/\Omega$ , for the limit state of shear yielding with a composite plate conforming to Section H6.5c shall be taken as:
5688	$V_n = 0.6A_{sp}F_y \tag{H6-1}$
5689	$\phi = 0.90 \text{ (LRFD)}$ $\Omega = 1.67 \text{ (ASD)}$
5690 5691 5692	where $F_y = \text{specified minimum yield stress of the plate, ksi (MPa)}$ $V_n = \text{nominal shear strength of the steel plate, kips (N)}$
5693 5694 5695 5696	The available shear strength of C-PSW/CE with a plate that does not meet the stiffening requirements in Section H6.5c shall be based upon the strength of the plate as given in Section F5.5 and shall satisfy the requirements of Specification Sections G2 and G3.
5697	H6.5c. Concrete Stiffening Elements
5698 5699 5700 5701 5702	The steel plate shall be stiffened by encasement or attachment to a reinforced concrete panel. Conformance to this requirement shall be demonstrated with an elastic plate buckling analysis showing that the composite wall is able to resist a nominal shear force equal to $V_n$ , as determined in Section H6.5b.
5703	The concrete thickness shall be a minimum of 4 in. (100 mm) on each

5704 5705 5706 5707 5708 5709 5710 5711 5712		(200 m Steel h provide reinforce provide require ratio in	nm) who neaded to possed conted in ments of both of the contents of the conten	crete is provided on both sides of the steel plate and 8 in. en concrete is provided on one side of the steel plate. stud anchors or other mechanical connectors shall be revent local buckling and separation of the plate and nerete. Horizontal and vertical reinforcement shall be the concrete encasement to meet or exceed the in ACI 318 Section 11.6 and 11.7. The reinforcement directions shall not be less than 0.0025. The maximum en bars shall not exceed 18 in. (450 mm).
5713	H6.5d	. Bound	ary Me	embers
5714 5715 5716 5717 5718 5719		resist t concret Compo satisfy	he exp te port site ar the rec	el and composite boundary members shall be designed to ected shear strength of steel plate and any reinforced ions of the wall active at the design story drift. In the reinforced concrete boundary members shall also puirements of Section H5.5b. Steel boundary members of the requirements of Section F5.
5720	Н6.5е.	Protec	ted Zoi	nes
5721		There a	ire no d	esignated protected zones.
5722	Н6.6.	Conne	ctions	
5723	H6.6a	. Deman	d Crit	ical Welds
5724 5725			_	g welds are demand critical welds, and shall satisfy the of Section A3.4b and I2.3:
5726		(a)	Groove	e welds at column splices
5727		(b)	Welds	at the column-to-base plate connections
5728 5729 5730 5731			-	tion: Welds need not be considered demand when both of the following conditions are ed:
5732 5733			(1)	Column hinging at, or near, the base plate is precluded by conditions of restraint, and
5734 5735			(2)	There is no net tension under load combinations including the overstrength seismic load.
5736		(c)	Welds	at HBE-to-VBE connections
5737	H6.6b	. HBE-t	o-VBE	Connections
5738 5739		HBE-to	o-VBE	connections shall satisfy the requirements of Section

## **H6.6c.** Connections of Steel Plate to Boundary Elements

The steel plate shall be continuously welded or bolted on all edges to the structural steel framing and/or steel boundary members, or the steel component of the composite boundary members. Welds and/or slipcritical high-strength bolts required to develop the nominal shear strength of the plate shall be provided.

### H6.6d. Connections of Steel Plate to Reinforced Concrete Panel

The steel anchors between the steel plate and the reinforced concrete panel shall be designed to prevent its overall buckling. Steel anchors shall be designed to satisfy the following conditions:

## 1. Tension in the Connector

The steel anchor shall be designed to resist the tension force resulting from inelastic local buckling of the steel plate.

## 2. Shear in the Connector

The steel anchors collectively shall be designed to transfer the expected strength in shear of the steel plate or reinforced concrete panel, whichever is smaller.

## **H6.6e. Column Splices**

In addition to the requirements of Section D2.5, column splices shall comply with the requirements of this section. Where welds are used to make the splice, they shall be complete-joint-penetration groove welds. When column splices are not made with groove welds, they shall have a required flexural strength that is at least equal to the nominal flexural strength,  $M_{pcc}$ , of the smaller composite column. The required shear strength of column web splices shall be at least equal to  $\Sigma M_{pcc}/H$ , where  $\Sigma M_{pcc}$  is the sum of the nominal flexural strengths at the top and bottom ends of the composite column. For composite columns, the nominal flexural strength shall satisfy the requirements of Specification Chapter I with consideration of the required axial strength,  $P_{rc}$ .

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# H7. COMPOSITE PLATE SHEAR WALLS—CONCRETE FILLED (C-PSW/CF)

## 5773 **H7.1. Scope**

Composite plate shear walls-concrete filled (C-PSW/CF) shall be designed in conformance with this section. This section is applicable to composite plate shear walls that consist of two planar steel web

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plates with concrete fill between the plates, with or without boundary elements. Composite action between the plates and concrete fill shall be achieved using either tie bars or a combination of tie bars and shear studs. The two steel web plates shall be of equal thickness and shall be placed at a constant distance from each other and connected using tie bars. When boundary members are included, they shall be either a half circular section of diameter equal to the distance between the two web plates or a circular concrete-filled steel tube.

# H7.2. Basis of Design

C-PSW/CF with boundary elements, designed in accordance with these provisions, are expected to provide significant inelastic deformation capacity through developing plastic moment strength of the composite C-PSW/CF cross section, by yielding of the entire skin plate and the concrete attaining its compressive strength. The cross section shall be detailed such that it is able to attain its plastic moment strength. Shear yielding of the steel web skin plates shall not be the governing mechanism.

C-PSW/CF without boundary elements designed in accordance to these provisions are expected to provide inelastic deformation capacity by developing yield moment strength of the composite C-PSW/CF cross section, by flexural tension yielding of the steel plates. The walls shall be detailed such that flexural compression yielding occurs before local buckling of the steel plates.

## H7.3. Analysis

Analysis shall satisfy the following:

- Effective flexural stiffness of the wall shall be calculated per (a) Specification Equation I2-12, with C<sub>3</sub> taken equal to 0.40.
- (b) The shear stiffness of the wall shall be calculated using the shear stiffness of the composite cross section.

## H7.4. System Requirements

## H7.4a. Steel Web Plate of C-PSW/CF with Boundary Elements

The maximum spacing of tie bars in vertical and horizontal directions,  $w_1$ 

$$w_{l} = 1.8t \sqrt{\frac{E}{F_{y}}} \tag{H7-1}$$

5820	where
5821	t = thickness of the steel web plate, in. (mm)
5822 5823 5824 5825	When tie bars are welded with the web plate, the thickness of the plate shall develop the tension strength of the tie bars.
5826 5827	H7.4b. Steel Plate of C-PSW/CF without Boundary Elements
5828 5829	The maximum spacing of tie bars in vertical and horizontal directions, $w_1$ ,:
5830	$w_1 = 1.0t \sqrt{\frac{E}{F_y}} $ (H7-2)
5831	where
5832	t = thickness of the steel web plate, in. (mm)
	The second of th
5833	
5834	H7.4c. Half Circular or Full Circular End of C-PSW/CF with Boundary
5835	Elements
5836	
5837	The D/t <sub>HSS</sub> ratio for the circular part of the C-PSW/CF cross section
5838	shall conform to:
5839	$\frac{D}{t_{HSS}} \le 0.044 \frac{E}{F_{y}} \tag{H7-3}$
5840	where
5841	D = outside diameter of round HSS, in. (mm)
5842	$t_{HSS} = thickness of HSS, in. (mm)$
5843	
5844	H7.4d. Spacing of Tie Bars in C-PSW/CF with or without Boundary
5845	Elements
5846	
5847	Tie bars shall be distributed in both vertical and horizontal directions,
5848	as specified in Equations H7-1 and H7-2.
5849	
5850	H7.4e. Tie Bar Diameter in C-PSW/CF with or without Boundary
5851	Elements
5852	
5853	Tie bars shall be designed to elastically resist the tension force, $T_{req}$ ,
5854	equal to:
5855	$T_{req} = T_1 + T_2$ (H7-4)
5856	T <sub>1</sub> is the tension force resulting from the locally buckled web plates
5857	developing plastic hinges on horizontal yield lines along the tie bars

and at mid-vertical distance between tie-bars, and is determined as follows:

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$$T_1 = 2\left(\frac{w_2}{w_1}\right) t_s^2 F_{y, plate}$$
 (H7-5)

5861 where

t<sub>s</sub> = the thickness of steel web plate provided, in. (mm)

 $w_1, w_2$  = vertical and horizontal spacing of tie bars, in. (mm), respectively

T<sub>2</sub> is the tension force that develops to prevent splitting of the concrete element on a plane parallel to the steel plate.

$$T_{2} = \left(\frac{t_{s} F_{y, plate} t_{w}}{4}\right) \left(\frac{w_{2}}{w_{l}}\right) \left[\frac{6}{18 \left(\frac{t_{w}}{w_{min}}\right)^{2} + 1}\right]$$
(H7-6)

5870 where

 $t_w = \text{total thickness of wall, in. (mm)}$ 

 $w_{min} = minimum of w_1 and w_2 in. (mm)$ 

## H7.4f. Connection between Tie Bars and Steel Plates

Connection of the tie bars to the steel plate shall be able to develop the full tension strength of the tie bar.

## H7.4g. Connection between C-PSW/CF Steel Components

Welds between the steel web plate and the half-circular or full-circular ends of the cross section shall be complete-joint-penetration groove welds.

## H7.4h. C-PSW/CF and Foundation Connection

The connection between C-PSW/CF and the foundation shall be detailed such that the connection is able to transfer the base shear force and the axial force acting together with the overturning moment, corresponding to 1.1 times the plastic composite flexural strength of the wall, where the plastic flexural composite strength is obtained by the plastic stress distribution method described in Specification Section I1.2a assuming that the steel components have reached a stress equal to the expected yield strength,  $R_yF_y$ , in either tension or compression and that concrete components in compression due to axial force and flexure have reached a stress of  $f_c'$ .

 **H7.5. Members** 

## H7.5a Flexural Strength

The nominal plastic moment strength of the C-PSW/CF with boundary elements shall be calculated considering that all the concrete in compression has reached its specified compressive strength,  $f_c$ , and that the steel in tension and compression has reached its specified minimum yield strength,  $F_y$ , as determined based on the location of the plastic neutral axis.

The nominal moment strength of the C-PSW/CF without boundary elements shall be calculated as the yield moment, M<sub>y</sub>, corresponding to yielding of the steel plate in flexural tension and first yield in flexural compression. The strength at first yield shall be calculated assuming a linear elastic stress distribution with maximum concrete compressive stress limited to 0.7 f<sub>c</sub> 'and maximum steel stress limited to F<sub>v</sub>.

**User Note:** The definition and calculation of the yield moment, M<sub>y</sub>, for C-PSW/CF without boundary elements is very similar to the definition and calculation of yield moment, M<sub>y</sub>, for noncompact filled composite members in Specification Section I3.4b(b).

## **H7.5b Shear Strength**

The available shear strength of C-PSW/CF shall be determined as follows:

(a) The design shear strength,  $\phi V_{ni}$ , or the allowable shear strength,  $V_{ni}/\Omega$ , of the C-PSW/CF with boundary elements shall be determined as follows:

 $V_{ni} = \kappa F_{y} A_{sw} \tag{H7-7}$ 

 $\phi_v = 0.90 \text{ (LRFD)}$   $\Omega_v = 1.67 \text{ (ASD)}$ 

5924 where

 $\kappa = 1.11 - 5.16\overline{\rho} \le 1.0$  (H7-8)

 $\overline{\rho}$  = strength adjusted reinforcement ratio

 $= \frac{A_{sw}F_{yw}}{A_{sw}\sqrt{1,000 f_c'}}$  (H7-9)

 $= \frac{1}{12} \frac{A_{sw} F_{yw}}{A_{ww} \sqrt{f'_{s}}}$  (H7-9M)

5928	F <sub>yw</sub> = specified minimum yield stress of web skin plates,
5929	ksi (MPa)
5930	$f'_c$ = specified compressive strength of concrete, ksi
5931	(MPa)
5932	$A_{sw}$ = area of steel web plates, in. $^2$ (mm $^2$ )
5933	$A_{cw}$ = area of concrete between web plates, in. <sup>2</sup> (mm <sup>2</sup> )
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## **User Note:** For most cases, $0.9 \le \kappa \le 1.0$ .

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(b) The nominal shear strength of the C-PSW/CF without boundary elements shall be calculated for the steel plates alone, in accordance with Section D1.4c.



6000 6001		CHAPTER I					
6002							
6003		FABRICATION AND ERECTION					
6004 6005 6006	This chapter addresses requirements for fabrication and erection.						
6007 6008		<b>Note:</b> All requirements of Specification Chapter M also apply, unless ically modified by these Provisions.					
6009 6010 6011	The cl	napter is organized as follows:					
6012 6013		<ul><li>I1. Shop and Erection Drawings</li><li>I2. Fabrication and Erection</li></ul>					
6014 6015 6016	I1.	SHOP AND ERECTION DRAWINGS					
6017	I1.1.	Shop Drawings for Steel Construction					
6018							
6019		Shop drawings shall indicate the work to be performed, and include					
6020		items required by the Specification, the AISC Code of Standard					
6021		Practice for Steel Buildings and Bridges, the applicable building code,					
6022		the requirements of Sections A4.1 and A4.2, and the following, as					
6023		applicable:					
6024							
6025		(a) Locations of pretensioned bolts					
6026		(b) Locations of Class A, or higher, faying surfaces					
6027		(c) Gusset plates drawn to scale when they are designed to					
6028		accommodate inelastic rotation					
6029		(d) Weld access hole dimensions, surface profile and finish					
6030		requirements					
6031		(e) Nondestructive testing (NDT) where performed by the					
6032		fabricator					
6033							
6034	I1.2.	<b>Erection Drawings for Steel Construction</b>					
6035	11.2.	Election Diawings for Steel Constitution					
6036		Erection drawings shall indicate the work to be performed, and include					
6037		items required by the Specification, the AISC Code of Standard					
6038		Practice for Steel Buildings and Bridges, the applicable building code,					
6039		the requirements of Sections A4.1 and A4.2, and the following, as					
6040		applicable:					
6041		аррисанс.					
6042		(a) Locations of pratensioned holts					
6042		(a) Locations of pretensioned bolts  (b) Those joints or groups of joints in which a specific assembly					
6044		(b) Those joints or groups of joints in which a specific assembly					
6044		order, welding sequence, welding technique or other special precautions are required					
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#### **Shop and Erection Drawings for Composite Construction** I1.3.

Shop drawings and erection drawings for the steel components of composite steel-concrete construction shall satisfy the requirements of Sections I1.1 and I1.2. The shop drawings and erection drawings shall also satisfy the requirements of Section A4.3.

User Note: For reinforced concrete and composite steel-concrete construction, the provisions of ACI 315 Details and Detailing of Concrete Reinforcement and ACI 315-R Manual of Engineering and Placing Drawings for Reinforced Concrete Structures apply.

#### **I2. FABRICATION AND ERECTION**

#### **I2.1. Protected Zone**

A protected zone designated by these Provisions or ANSI/AISC 358 shall comply with the following requirements:

- Within the protected zone, holes, tack welds, erection aids, air-(a) arc gouging, and unspecified thermal cutting from fabrication or erection operations shall be repaired as required by the engineer of record.
- (b) Steel headed stud anchors shall not be placed on beam flanges within the protected zone.
- Arc spot welds as required to attach decking are permitted. (c)
- Decking attachments that penetrate the beam flange shall not (d) be placed on beam flanges within the protected zone, except power-actuated fasteners up to 0.18 in. diameter are permitted.
- (e) Welded, bolted, or screwed attachments or power-actuated for perimeter edge angles, exterior facades, fasteners partitions, duct work, piping or other construction shall not be placed within the protected zone.

Exception: Other attachments are permitted where designated or approved by the engineer of record. See Section D1.3.

User Note: AWS D1.8/D1.8M clause 6.15 contains requirements for weld removal and the repair of gouges and notches in the protected zone.

**I2.2.** Bolted Joints

6092	<b>I2.2.</b>	<b>Bolted Joints</b>
6093		
6094		Bolted joints shall satisfy the requirements of Section D2.2.
6095		
6096	<b>I2.3.</b>	Welded Joints
6097		
6098		Welding and welded connections shall be in accordance with
6099		Structural Welding Code—Steel (AWS D1.1/D1.1M), hereafter
6100		referred to as AWS D1.1/D1.1M, and AWS D1.8/D1.8M.
6101		
6102		Welding procedure specifications (WPSs) shall be approved by the
6103		engineer of record.
6104		
6105		Weld tabs shall be in accordance with AWS D1.8/D1.8M clause 6.10,
6106		except at the outboard ends of continuity-plate-to-column welds, weld
6107		tabs and weld metal need not be removed closer than ¼ in. (6 mm)
6108		from the continuity plate edge.
6109		
6110		AWS D1.8/D1.8M clauses relating to fabrication shall apply equally to
6111		shop fabrication welding and to field erection welding.
6112		
6113		User Note: AWS D1.8/D1.8M was specifically written to provide
6114		additional requirements for the welding of seismic force resisting
6115		systems, and has been coordinated wherever possible with these
6116		Provisions. AWS D1.8/D1.8M requirements related to fabrication and
6117		erection are organized as follows, including normative (mandatory)
6118		annexes:
6119		
6120		(a) General Requirements
6121		(b) Reference Documents
6122		(c) Definitions
6123		(d) Welded Connection Details
6124		(e) Welder Qualification
6125		(f) Fabrication
6126		Annex A. WPS Heat Input Envelope Testing of Filler Metals for
6127		Demand Critical Welds
6128		Annex B. Intermix CVN Testing of Filler Metal Combinations
6129		(where one of the filler metals is FCAW-S)
6130		Annex C. Supplemental Welder Qualification for Restricted Access
6131		Welding
6132		Annex D. Supplemental Testing for Extended Exposure Limits for
6133		FCAW Filler Metals
6134		2 012 11 2 2002 21200000
6135		AWS D1.8/D1.8M requires the complete removal of all weld tab
6136		material, leaving only base metal and weld metal at the edge of the
6137		joint. This is to remove any weld discontinuities at the weld ends, as
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well as facilitate magnetic particle testing (MT) of this area. At
continuity plates, these Provisions permit a limited amount of weld tab
material to remain because of the reduced strains at continuity plates,
and any remaining weld discontinuities in this weld end region would
likely be of little significance. Also, weld tab removal sites at
continuity plates are not subjected to MT.

AWS D1.8/D1.8M clause 6 is entitled "Fabrication," but the intent of AWS is that all provisions of AWS D1.8/D1.8M apply equally to fabrication and erection activities as described in the Specification and in these Provisions.

## **I2.4.** Continuity Plates and Stiffeners

Corners of continuity plates and stiffeners placed in the webs of rolled shapes shall be detailed in accordance with AWS D1.8 clause 4.1.

6201	
6202	QUALITY CONTROL AND QUALITY ASSURANCE
6203	
6204	This chapter addresses requirements for quality control and quality assurance.
6205	
6206	User Note: All requirements of Specification Chapter N also apply, unless
6207	specifically modified by these Provisions.
6208	
6209	The chapter is organized as follows:
6210	
6211	J1. Scope
6212	J2. Fabricator and Erector Documents
6213	J3. Quality Assurance Agency Documents
6214	J4. Inspection and Nondestructive Testing Personnel
6215	J5. Inspection Tasks
6216	J6. Welding Inspection and Nondestructive Testing
6217	J7. Inspection of High-Strength Bolting
6218	J8. Other Steel Structure Inspections
6219	J9. Inspection of Composite Structures
6220	J10. Inspection of Piling
6221	
6222	J1. SCOPE
6223	
6224	Quality Control (QC) as specified in this chapter shall be provided by
6225	the fabricator, erector or other responsible contractor as applicable.
6226	Quality Assurance (QA) as specified in this chapter shall be provided
6227	by others when required by the authority having jurisdiction (AHJ),
6228	applicable building code (ABC), purchaser, owner or engineer of
6229	record (EOR). Nondestructive testing (NDT) shall be performed by the
6230	agency or firm responsible for Quality Assurance, except as permitted
6231	in accordance with Specification Section N7.
6232	
6233	User Note: The quality assurance plan of this section is considered
6234	adequate and effective for most seismic force resisting systems and
6235	should be used without modification. The quality assurance plan is
6236	intended to ensure that the seismic force resisting system is
6237	significantly free of defects that would greatly reduce the ductility of
6238	the system. There may be cases (for example, nonredundant major
6239	transfer members, or where work is performed in a location that is
6240	difficult to access) where supplemental testing might be advisable.
6241	Additionally, where the fabricator's or erector's quality control
6242	program has demonstrated the capability to perform some tasks this
6243	plan has assigned to quality assurance, modification of the plan could
6244	be considered.

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#### J2. FABRICATOR AND ERECTOR DOCUMENTS

#### J2.1. Documents to be Submitted for Steel Construction

In addition to the requirements of Specification Section N3.1, the following documents shall be submitted for review by the EOR or the EOR's designee, prior to fabrication or erection of the affected work, as applicable:

- (a) Welding procedure specifications (WPS)
- (b) Copies of the manufacturer's typical certificate of conformance for all electrodes, fluxes and shielding gasses to be used
- (c) For demand critical welds, applicable manufacturer's certifications that the filler metal meets the supplemental notch toughness requirements, as applicable. When the filler metal manufacturer does not supply such supplemental certifications, the fabricator or erector, as applicable, shall have the necessary testing performed and provide the applicable test reports
- (d) Manufacturer's product data sheets or catalog data for SMAW, FCAW and GMAW composite (cored) filler metals to be used
- (e) Bolt installation procedures
- (d) Specific assembly order, welding sequence, welding technique, or other special precautions for joints or groups of joints where such items are designated to be submitted to the engineer of record

#### J2.2. Documents to be Available for Review for Steel Construction

Additional documents as required by the EOR in the contract documents shall be available by the fabricator and erector for review by the EOR or the EOR's designee prior to fabrication or erection, as applicable.

The fabricator and erector shall retain their document(s) for at least one year after substantial completion of construction.

#### J2.3. Documents to be Submitted for Composite Construction

The following documents shall be submitted by the responsible

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6291 6292			actor for review by the EOR or the EOR's designee, prior to ete production or placement, as applicable:
6293 6294 6295		(a)	Concrete mix design and test reports for the mix design
6296		(b)	Reinforcing steel shop drawings
6297 6298		(c)	Concrete placement sequences, techniques and restriction
6299 6300	J2.4.	Docui	ments to be Available for Review for Composite Construction
6301			
6302			following documents shall be available from the responsible
6303			actor for review by the EOR or the EOR's designee prior to
6304		fabrica	ation or erection, as applicable, unless specified to be submitted:
6305			
6306		(a)	Material test reports for reinforcing steel
6307		<i>a</i> >	
6308		(b)	Inspection procedures
6309 6310		(a)	Nonconformance ancodim
6311		(c)	Nonconformance procedure
6312		(d)	Material control procedure
6313		(u)	Waterial control procedure
6314		(e)	Welder performance qualification records (WPQR) as required
6315		(0)	by AWS D1.4/D1.4M
6316			32
6317		(f)	QC Inspector qualifications
6318		` '	
6319		The re	esponsible contractor shall retain their document(s) for at least
6320		one ye	ear after substantial completion of construction.
6321			
6322	J3.	QUAI	LITY ASSURANCE AGENCY DOCUMENTS
6323			
6324			agency responsible for quality assurance shall submit the
6325		follow	ving documents to the authority having jurisdiction, the EOR,

ne following documents to the authority having jurisdiction, the EOR, and the owner or owner's designee:

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- QA agency's written practices for the monitoring and control of the agency's operations. The written practice shall include:
  - (1) The agency's procedures for the selection and administration of inspection personnel, describing the training, experience and examination requirements for qualification and certification of inspection personnel, and

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6337		(2) The agency's inspection procedures, including general
6338		inspection, material controls, and visual welding
6339		inspection
6340		
6341		(b) Qualifications of management and QA personnel designated
6342		for the project
6343		
6344		(c) Qualification records for inspectors and NDT technicians
6345		designated for the project
6346		
6347		(d) NDT procedures and equipment calibration records for NDT to
6348		be performed and equipment to be used for the project
6349		
6350		(e) For composite construction, concrete testing procedures and
6351		equipment
6352		
6353	J4.	INSPECTION AND NONDESTRUCTIVE TESTING
6354		PERSONNEL
6355		
6356		In addition to the requirements of Specification Sections N4.1 and
6357		N4.2, visual welding inspection and NDT shall be conducted by
6358		personnel qualified in accordance with AWS D1.8/D1.8M clause 7.2
6359		In addition to the requirements of Specification Section N4.3
6360		ultrasonic testing technicians shall be qualified in accordance with
6361		AWS D1.8/D1.8M clause 7.2.4.
6362		
6363		User Note: The recommendations of the International Code Council
6364		Model Program for Special Inspection should be considered a
6365		minimum requirement to establish the qualifications of a bolting
6366		inspector.
6367		
6368	J5.	INSPECTION TASKS
6369		
6370		Inspection tasks and documentation for QC and QA for the seismic
6371		force resisting system (SFRS) shall be as provided in the tables in
6372		Sections J6, J7, J8, J9 and J10. The following entries are used in the
6373		tables:
6374		
6375	J5.1.	Observe (O)
6376		
6377		The inspector shall observe these functions on a random, daily basis.
6378		Operations need not be delayed pending observations.
6379		
6380	J5.2.	Perform (P)

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These inspections shall be performed prior to the final acceptance of

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6383		the item.
6384		
6385	J5.3.	Document (D)
6386		
6387		The inspector shall prepare reports indicating that the work has been
6388		performed in accordance with the contract documents. The report
6389		need not provide detailed measurements for joint fit-up, WPS settings,
6390		completed welds, or other individual items listed in the tables. For
6391		shop fabrication, the report shall indicate the piece mark of the piece
6392		inspected. For field work, the report shall indicate the reference grid
6393		lines and floor or elevation inspected. Work not in compliance with the
6394		contract documents and whether the noncompliance has been
6395		satisfactorily repaired shall be noted in the inspection report.
6396		
6397	J5.4.	Coordinated Inspection
6398		
6399		Where a task is stipulated to be performed by both QC and QA,
6400		coordination of the inspection function between QC and QA is
6401		permitted in accordance with Specification Section N5.3.
6402		
6403	J6.	WELDING INSPECTION AND NONDESTRUCTIVE TESTING
6404		
6405		Welding inspection and nondestructive testing shall satisfy the
6406		requirements of the Specification, this section and AWS D1.8/D1.8M.
6407		
6408		User Note: AWS D1.8/D1.8M was specifically written to provide
6409		additional requirements for the welding of seismic force resisting
6410		systems, and has been coordinated when possible with these
6411		Provisions. AWS D1.8/D1.8M requirements related to inspection and
6412		nondestructive testing are organized as follows, including normative
6413		(mandatory) annexes:
6414		
6415		1. General Requirements
6416		7. Inspection
6417		Annex F. Supplemental Ultrasonic Technician Testing
6418		Annex G. Supplemental Magnetic Particle Testing Procedures
6419		Annex H. Flaw Sizing by Ultrasonic Testing
6420		
6421	J6.1.	Visual Welding Inspection
6422		
6423		All requirements of the Specification shall apply, except as specifically
6424		modified by AWS D1.8/D1.8M.

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Visual welding inspection shall be performed by both quality control

and quality assurance personnel. As a minimum, tasks shall be as listed in Tables J6-1, J6-2 and J6-3.

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TABLE J6-1 Visual Inspection Tasks Prior to Welding					
Vigual Increation Tacks Prior to Walding	Q	QC		QA	
Visual Inspection Tasks Prior to Welding	Task	Doc.	Task	Doc.	
Material identification (Type/Grade)	0	-	0	-	
Welder identification system	0	-	0	-	
Fit-up of Groove Welds (including joint geometry)  - Joint preparation  - Dimensions (alignment, root opening, root face, bevel)  - Cleanliness (condition of steel surfaces)  - Tacking (tack weld quality and location)  - Backing type and fit (if applicable)	P/O**	-	0	-	
Configuration and finish of access holes	0	-	0	-	
Fit-up of Fillet Welds - Dimensions (alignment, gaps at root) - Cleanliness (condition of steel surfaces) Tacking (tack wold quality and location)	P/O**	-	0	-	

<sup>-</sup> Tacking (tack weld quality and location)

\*\*\* Following performance of this inspection task for ten welds to be made by a given welder, with the welder demonstrating understanding of requirements and possession of skills and tools to verify these items, the Perform designation of this task shall be reduced to Observe, and the welder shall perform this task. Should the inspector determine that the welder has discontinued performance of this task, the task shall be returned to Perform until such time as the Inspector has re-established adequate assurance that the welder will perform the inspection tasks listed.

TABLE J6-2 Visual Inspection Tasks During Welding					
Visual Instruction Tester Desires Welding	Q	QC		QA	
Visual Inspection Tasks During Welding		Doc.	Task	Doc.	
WPS followed					
- Settings on welding equipment					
- Travel speed					
- Selected welding materials					
- Shielding gas type/flow rate	0	-	0	-	
- Preheat applied					
- Interpass temperature maintained (min/max.)					
- Proper position (F, V, H, OH)					
- Intermix of filler metals avoided unless approved					
Use of qualified welders	0	-	0	-	
Control and handling of welding consumables					
- Packaging	0	-	0	-	
- Exposure control					
Environmental conditions					
- Wind speed within limits	0	-	0	-	
- Precipitation and temperature					
Welding techniques					
- Interpass and final cleaning					
- Each pass within profile limitations	0	-	0	-	
- Each pass meets quality requirements					
No welding over cracked tacks	0	-	0	_	

TABLE J6-3 Visual Inspection Tasks After Wel	ding				
Visual Inspection Tools After Wolding		QC		QA	
Visual Inspection Tasks After Welding	Task	Doc.	Task	Doc.	
Welds cleaned	0	-	0	-	
Size, length, and location of welds	Р	-	Р	-	
Welds meet visual acceptance criteria					
- Crack prohibition					
- Weld/base-metal fusion					
- Crater cross section	Р	D	Р	D	
- Weld profiles and size					
- Undercut					
- Porosity					
k-area <sup>1</sup>	Р	D	Р	D	
Placement of reinforcing or contouring fillet welds (if required)	Р	D	Р	D	
Backing removed, weld tabs removed and finished, and fillet welds added (if required)	Р	D	Р	D	
Repair activities	Р	-	Р	D	

When welding of doubler plates, continuity plates or stiffeners has been performed in the k-area, visually inspect the web k-area for cracks within 3 in. (75 mm) of the weld. The visual inspection shall be performed no sooner than 48 hours following completion of the welding.

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#### J6.2. NDT of Welded Joints

In addition to the requirements of Specification Section N5.5, nondestructive testing of welded joints shall be as required in this section:

#### 

#### J6.2a. CJP Groove Weld NDT

 Ultrasonic testing (UT) shall be performed on 100% of CJP groove welds in materials 5/16 in. (8 mm) thick or greater. Ultrasonic testing in materials less than 5/16 in. (8 mm) thick is not required. Weld discontinuities shall be accepted or rejected on the basis of criteria of AWS D1.1/D1.1M Table 6.2. Magnetic particle testing shall be performed on 25% of all beam-to-column CJP groove welds. The rate of UT and MT is permitted to be reduced in accordance with Sections J6.2h and J6.2i, respectively.

Exception: For ordinary moment frames in structures in risk categories I or II, UT and MT of CJP groove welds are required only for demand critical welds.

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User Note: For structures in Risk Category III or IV, AISC 360 section N5.5b requires that the UT be performed by QA on all CJP groove welds subject to transversely applied tension loading in butt, Tand corner joints, in material 5/16 in. (8 mm) thick or greater.

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#### J6.2b. Column Splice and Column to Base Plate PJP Groove Weld NDT

UT shall be performed by QA on 100% of PJP groove welds in column splices and column to base plate welds. The rate of UT is permitted to be reduced in accordance with Section J6.2h.

UT shall be performed using written procedures and UT technicians qualified in accordance with AWS D1.8. The weld joint mock-ups used to qualify procedures and technicians shall include at least one single-bevel PJP groove welded joint and one double-bevel PJP groove welded joint, detailed to provide transducer access limitations similar to those to be encountered at the weld faces and by the column web. Rejection of discontinuities outside the groove weld throat shall be considered false indications in procedure and personnel qualification. Procedures qualified using mock-ups with artificial flaws 1/16 in. (1.5 mm) in their smallest dimension are acceptable permitted.

UT examination of welds using alternative techniques in compliance with AWS D1.1 Annex Q is acceptable permitted.

Weld discontinuities located within the groove weld throat shall be accepted or rejected on the basis of criteria of AWS D1.1/D1.1M Table 6.2, except when alternative techniques are used, the criteria shall be as provided in AWS D1.1 Annex Q.

#### J6.2c.Base Metal NDT for Lamellar Tearing and Laminations

After joint completion, base metal thicker than 1½ in. (38 mm) loaded in tension in the through-thickness direction in tee and corner joints, where the connected material is greater than 3/4 in. (19 mm) and contains CJP groove welds, shall be ultrasonically tested for discontinuities behind and adjacent to the fusion line of such welds. Any base metal discontinuities found within t/4 of the steel surface shall be accepted or rejected on the basis of criteria of AWS D1.1/D1.1M Table 6.2, where t is the thickness of the part subjected to the through-thickness strain.

#### J6.2d.Beam Cope and Access Hole NDT

At welded splices and connections, thermally cut surfaces of beam copes and access holes shall be tested using magnetic particle testing

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ment [LCA1]: Editorial change to use code

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6504 or penetrant testing, when the flange thickness exceeds 1½ in. (38 6505 mm) for rolled shapes, or when the web thickness exceeds 1½ in. (38 6506 mm) for built-up shapes. 6507

#### J6.2e. Reduced Beam Section Repair NDT

Magnetic particle testing shall be performed on any weld and adjacent area of the reduced beam section (RBS) cut surface that has been repaired by welding, or on the base metal of the RBS cut surface if a sharp notch has been removed by grinding.

#### J6.2f. Weld Tab Removal Sites

At the end of welds where weld tabs have been removed, magnetic particle testing shall be performed on the same beam-to-column joints receiving UT as required under Section J6.2b. The rate of MT is permitted to be reduced in accordance with Section J6.2i. MT of continuity plate weld tabs removal sites is not required.

#### J6.2g.Reduction of Percentage of Ultrasonic Testing

The reduction of percentage of UT is permitted to be reduced in accordance with Specification Section N5.5e, except no reduction is permitted for demand critical welds.

#### J6.2h. Reduction of Percentage of Magnetic Particle Testing

The amount of MT on CJP groove welds is permitted to be reduced if approved by the engineer of record and the authority having jurisdiction. The MT rate for an individual welder or welding operator is permitted to be reduced to 10%, provided the reject rate is demonstrated to be 5% or less of the welds tested for the welder or welding operator. A sampling of at least 20 completed welds for a job shall be made for such reduction evaluation. Reject rate is the number of welds containing rejectable defects divided by the number of welds completed. This reduction is prohibited on welds in the k-area, at repair sites, backing removal sites, and access holes.

#### J7. INSPECTION OF HIGH-STRENGTH BOLTING

Bolting inspection shall satisfy the requirements of Specification Section N5.6 and this section. Bolting inspection shall be performed by both quality control and quality assurance personnel. As a minimum, the tasks shall be as listed in Tables J7-1, J7-2 and J7-3.

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TABLE J7-1 Inspection Tasks Prior To Bolting							
QC QA							
Inspection Tasks Prior To Bolting		Doc.	Task	Doc.			
Proper fasteners selected for the joint detail	0	_	0	_			
Proper bolting procedure selected for joint detail	0	_	0	_			
Connecting elements, including the faying surface condition and hole preparation, if specified, meet applicable requirements	0	_	0	_			
Pre-installation verification testing by installation personnel observed for fastener assemblies and methods used	Р	D	0	D			
Proper storage provided for bolts, nuts, washers and other fastener components	0	_	0	_			

TABLE J7-2 Inspection Tasks During Bolting						
Increation Tasks During Balting	Q	C	Q	Α		
Inspection Tasks During Bolting	Task	Doc.	Task	Doc.		
Fastener assemblies placed in all holes and washers (if required) are positioned as required	0	_	0	_		
Joint brought to the snug tight condition prior to the pretensioning operation	0	_	0	=		
Fastener component not turned by the wrench prevented from rotating	0	_	0	_		
Bolts are pretensioned progressing systematically from the most rigid point toward the free edges	0	_	0	_		

TABLE J7-3 Inspection Tasks After Bolt	ing			
Inspection Tasks After Bolting	QC QA			
inspection rasks After Boiling	Task	Doc.	Task	Doc.
Document accepted and rejected connections	Р	D	Р	D

## J8. OTHER STEEL STRUCTURE INSPECTIONS

Other inspections of the steel structure shall satisfy the requirements of Specification Section N5.8 and this section. Such inspections shall be performed by both quality control and quality assurance personnel. Where applicable, the inspection tasks listed in Table J8-1 shall be performed.

TABLE J8-	1	
Other Inspection	Tasks	
Other Inspection Tasks	QC	QA

	Task	Doc	Task	Doc.
RBS requirements, if applicable - Contour and finish - Dimensional tolerances	Р	D	Р	D
Protected zone—no holes and unapproved attachments made by fabricator or erector, as applicable	Р	D	Р	D

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 **User Note:** The protected zone should be inspected by others following completion of the work of other trades, including those involving curtainwall, mechanical, electrical, plumbing and interior partitions. See Section A4.1(3).

#### J9. INSPECTION OF COMPOSITE STRUCTURES

Where applicable, inspection of composite structures shall satisfy the requirements of the Specification and this section. These inspections shall be performed by the responsible contractor's quality control personnel and by quality assurance personnel.

Where applicable, inspection of structural steel elements used in composite structures shall comply with the requirements of this Chapter. Where applicable, inspection of reinforced concrete shall comply with the requirements of ACI 318, and inspection of welded reinforcing steel shall comply with the applicable requirements of Section J6.1.

Where applicable to the type of composite construction, the minimum inspection tasks shall be as listed in Tables J9-1, J9-2 and J9-3.

# TABLE J9-1 Inspection of Composite Structures Prior to Concrete Placement

Inspection of Composite Structures Prior to Concrete	(	QC	QA	
Placement	Task	Doc	Task	Doc.
Material identification of reinforcing steel (Type/Grade)	0	_	0	_
Determination of carbon equivalent for reinforcing steel other than ASTM A706	0	_	0	1
Proper reinforcing steel size, spacing and orientation	0	_	0	1
Reinforcing steel has not been rebent in the field	0	-	0	
Reinforcing steel has been tied and supported as required	0	_	0	_
Required reinforcing steel clearances have been provided	0	_	0	_
Composite member has required size	0	_	0	_

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# TABLE J9-2 Inspection of Composite Structures during Concrete Placement

Inspection of Composite Structures during Concrete	(	QC .	QA	
Placement	Task Doc Task D			Doc.
Concrete: Material identification (mix design, compressive strength, maximum large aggregate size, maximum slump)	0	D	0	D
Limits on water added at the truck or pump	0	D	0	D
Proper placement techniques to limit segregation	0	-	0	-

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# TABLE J9-3 Inspection of Composite Structures after Concrete Placement

Instruction of Community Structures After Compute Pleasurent	(	2C	QA	
Inspection of Composite Structures After Concrete Placement		Doc	Task	Doc.
Achievement of minimum specified concrete compressive strength at specified age	_	D	_	D

CHAPTER J

# J-15

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#### J10. INSPECTION OF H-PILES

Where applicable, inspection of piling shall satisfy the requirements of this section. These inspections shall be performed by both the responsible contractor's quality control personnel and by quality assurance personnel. Where applicable, the inspection tasks listed in Table J10-1 shall be performed.

TABLE J10-1 Inspection of H-Piles						
Inspection of Piling		QC		QA		
		Doc.	Task	Doc.		
Protected zone—no holes and unapproved attachments made by the responsible contractor, as applicable	Р	D	Р	D		

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6600		CHAPTER K
6601 6602	PF	REQUALIFICATION AND CYCLIC QUALIFICATION TESTING PROVISIONS
6603 6604	This c	chapter addresses requirements for qualification and prequalification.
6605 6606 6607 6608 6609 6610	This cl	napter is organized as follows:  K1. Prequalification of Beam-to-Column and Link-to-Column Connections  K2. Cyclic Tests for Qualification of Beam-to-Column and Link-to Column Connections  K3. Cyclic Tests for Qualification of Buckling Restrained Braces
6611 6612	K1.	PREQUALIFICATION OF BEAM-TO-COLUMN AND LINK- TO-COLUMN CONNECTIONS
6613	K1.1.	Scope
6614 6615 6616 6617 6618 6619 6620 6621 6622		This section contains minimum requirements for prequalification of beam-to-column moment connections in SMF, IMF, C-SMF, and C-IMF, and link-to-column connections in EBF. Prequalified connections are permitted to be used, within the applicable limits of prequalification, without the need for further qualifying cyclic tests. When the limits of prequalification or design requirements for prequalified connections conflict with the requirements of these Provisions, the limits of prequalification and design requirements for prequalified connections shall govern.
6623	K1.2.	General Requirements
6624	K1.2a.	Basis for Prequalification
6625 6626 6627 6628 6629 6630 6631 6632 6633 6634 6635		Connections shall be prequalified based on test data satisfying Section K1.3, supported by analytical studies and design models. The combined body of evidence for prequalification must be sufficient to assure that the connection is able to supply the required story drift angle for SMF, IMF, C-SMF, and C-IMF systems, or the required link rotation angle for EBF, on a consistent and reliable basis within the specified limits of prequalification. All applicable limit states for the connection that affect the stiffness, strength and deformation capacity of the connection and the seismic force resisting system (SFRS) must be identified. The effect of design variables listed in Section K1.4 shall be addressed for connection prequalification.

K1.2b. Authority for Prequalification

Prequalification of a connection and the associated limits of prequalification shall be established by a connection prequalification review panel (CPRP) approved by the authority having jurisdiction.

#### **K1.3.** Testing Requirements

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Data used to support connection prequalification shall be based on tests conducted in accordance with Section K2. The CPRP shall determine the number of tests and the variables considered by the tests for connection prequalification. The CPRP shall also provide the same information when limits are to be changed for a previously prequalified connection. A sufficient number of tests shall be performed on a sufficient number of nonidentical specimens to demonstrate that the connection has the ability and reliability to undergo the required story drift angle for SMF, IMF, C-SMF, and C-IMF and the required link rotation angle for EBF, where the link is adjacent to columns. The limits on member sizes for prequalification shall not exceed the limits specified in Section K2.3b.

# **K1.4.** Prequalification Variables

In order to be prequalified, the effect of the following variables on connection performance shall be considered. Limits on the permissible values for each variable shall be established by the CPRP for the prequalified connection.

K1.4a. Beam and Column Parameters for SMF and IMF, Link and Column Parameters for EBF

- (a) Cross-section shape: wide flange, box or other
- 6663 (b) Cross-section fabrication method: rolled shape, welded shape or other
- 6665 (c) Depth
- 6666 (d) Weight per foot
- 6667 (e) Flange thickness
- 6668 (f) Material specification
- 6669 (g) Beam span-to-depth ratio (for SMF or IMF), or link length (for EBF)
- (h) Width-to-thickness ratio of cross-section elements
- 6672 (i) Lateral bracing

6673 6674 6675 6676	(j)	Column orientation with respect to beam or link: beam or link is connected to column flange, beam or link is connected to column web, beams or links are connected to both the column flange and web, or other
6677 6678	(k)	Other parameters pertinent to the specific connection under consideration
6679	K1.4b. Beam	and Column Parameters for C-SMF and C-IMF
6680 6681	(a)	For structural steel members that are part of a composite beam or column: specify parameters required in Section K1.4a.
6682	(b)	Overall depth of composite beam and column
6683	(c)	Composite beam span to depth ratio
6684	(d)	Reinforcing bar diameter
6685	(e)	Reinforcement material specification
6686	(f)	Reinforcement development and splice requirements
6687	(g)	Transverse reinforcement requirements
6688	(h)	Concrete compressive strength and density
6689	(i)	Steel anchor dimensions and material specification
6690 6691	(j)	Other parameters pertinent to the specific connection under consideration
6692	K1.4c. Beam	-to-Column or Link-to-Column Relations
6693	(a)	Panel zone strength for SMF, IMF, and EBF
6694	(b)	Joint shear strength for C-SMF and C-IMF
6695	(c)	Doubler plate attachment details for SMF, IMF, and EBF
6696	(d)	Joint reinforcement details for C-SMF and C-IMF
6697	(e)	Column-to-beam (or column-to-link) moment ratio
6698	K1.4d. Conti	nuity and Diaphragm Plates
6699 6700	(a)	Identification of conditions under which continuity plates or diaphragm plates are required
6701	(b)	Thickness, width and depth
6702	(c)	Attachment details

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6703	K1.4e.	. Welds	
6704 6705		(a)	Location, extent (including returns), type (CJP, PJP, fillet, etc.) and any reinforcement or contouring required
6706		(b)	Filler metal classification strength and notch toughness
6707		(c)	Details and treatment of weld backing and weld tabs
6708		(d)	Weld access holes: size, geometry and finish
6709 6710 6711		(e)	Welding quality control and quality assurance beyond that described in Chapter J, including NDT method, inspection frequency, acceptance criteria and documentation requirements
6712	K1.4f.	Bolts	
6713		(a)	Bolt diameter
6714		(b)	Bolt grade: ASTM A325, A325M, A490, A490M or other
6715		(c)	Installation requirements: pretensioned, snug-tight or other
6716		(d)	Hole type: standard, oversize, short-slot, long-slot or other
6717 6718		(e)	Hole fabrication method: drilling, punching, sub-punching and reaming, or other
6719 6720		(f)	Other parameters pertinent to the specific connection under consideration
6721	K1.4g	. Reinfo	orcement in C-SMF and C-IMF
6722		(a)	Location of longitudinal and transverse reinforcement
6723		(b)	Cover requirements
6724		(c)	Hook configurations and other pertinent reinforcement details
6725	K1.4h	. Qualit	y Control and Quality Assurance
6726 6727		-	rements that exceed or supplement requirements specified in ter J, if any.
6728	K1.4i.	Additio	onal Connection Details
6729 6730 6731		and A	ariables and workmanship parameters that exceed AISC, RCSC AWS requirements pertinent to the specific connection under deration, as established by the CPRP.
6732	K1.5.	Design	a Procedure

6733 6734 6735		prequ	omprehensive design procedure must be available for a alified connection. The design procedure must address all cable limit states within the limits of prequalification.
6736	K1.6.	Prequ	alification Record
6737 6738			equalified connection shall be provided with a written diffication record with the following information:
6739 6740 6741		(a)	General description of the prequalified connection and drawings that clearly identify key features and components of the connection
6742 6743 6744 6745		(b)	Description of the expected behavior of the connection in the elastic and inelastic ranges of behavior, intended location(s) of inelastic action, and a description of limit states controlling the strength and deformation capacity of the connection
6746 6747		(c)	Listing of systems for which connection is prequalified: SMF, IMF, EBF, C-SMF, or C-IMF.
6748 6749		(d)	Listing of limits for all applicable prequalification variables listed in Section K1.4
6750		(e)	Listing of demand critical welds
6751 6752		(f)	Definition of the region of the connection that comprises the protected zone
6753 6754		(g)	Detailed description of the design procedure for the connection, as required in Section K1.5
6755 6756		(h)	List of references of test reports, research reports and other publications that provided the basis for prequalification
6757		(i)	Summary of quality control and quality assurance procedures
6758			
6759 6760	K2.		IC TESTS FOR QUALIFICATION OF BEAM-TO- UMN AND LINK-TO-COLUMN CONNECTIONS
6761	K2.1.	Scope	
6762 6763 6764 6765 6766 6767		to-colu and li Provisi provide	ection provides requirements for qualifying cyclic tests of beamment connections in SMF, IMF, C-SMF, and C-IMF; nk-to-column connections in EBF, when required in these ions. The purpose of the testing described in this section is to e evidence that a beam-to-column connection or a link-to-n connection satisfies the requirements for strength and story

drift angle or link rotation angle in these Provisions. Alternative testing requirements are permitted when approved by the engineer of record and the authority having jurisdiction.

#### **K2.2.** Test Subassemblage Requirements

The test subassemblage shall replicate as closely as is practical the conditions that will occur in the prototype during earthquake loading. The test subassemblage shall include the following features:

- (a) The test specimen shall consist of at least a single column with beams or links attached to one or both sides of the column.
- (b) Points of inflection in the test assemblage shall coincide with the anticipated points of inflection in the prototype under earthquake loading.
- (c) Lateral bracing of the test subassemblage is permitted near load application or reaction points as needed to provide lateral stability of the test subassemblage. Additional lateral bracing of the test subassemblage is not permitted, unless it replicates lateral bracing to be used in the prototype.

#### **K2.3.** Essential Test Variables

The test specimen shall replicate as closely as is practical the pertinent design, detailing, construction features, and material properties of the prototype. The following variables shall be replicated in the test specimen.

# **K2.3a. Sources of Inelastic Rotation**

The inelastic rotation shall be computed based on an analysis of test specimen deformations. Sources of inelastic rotation include, but are not limited to, yielding of members, yielding of connection elements and connectors, yielding of reinforcing steel, inelastic deformation of concrete, and slip between members and connection elements. For beam-to-column moment connections in SMF, IMF, C-SMF, and C-IMF, inelastic rotation is computed based upon the assumption that inelastic action is concentrated at a single point located at the intersection of the centerline of the beam with the centerline of the column. For link-to-column connections in EBF, inelastic rotation shall be computed based upon the assumption that inelastic action is concentrated at a single point located at the intersection of the centerline of the link with the face of the column.

Inelastic rotation shall be developed in the test specimen by inelastic action in the same members and connection elements as anticipated in

the prototype (in other words, in the beam or link, in the column panel zone, in the column outside of the panel zone, or in connection elements) within the limits described below. The percentage of the total inelastic rotation in the test specimen that is developed in each member or connection element shall be within 25% of the anticipated percentage of the total inelastic rotation in the prototype that is developed in the corresponding member or connection element.

#### K2.3b. Members

 The size of the beam or link used in the test specimen shall be within the following limits:

- (a) The depth of the test beam or link shall be no less than 90% of the depth of the prototype beam or link.
- (b) For SMF, IMF and EBF, the weight per foot of the test beam or link shall be no less than 75% of the weight per foot of the prototype beam or link.
- (c) For C-SMF and C-IMF, the weight per foot of the structural steel member that forms part of the test beam shall be no less than 75% of the weight per foot of the structural steel member that forms part of the prototype beam.

The size of the column used in the test specimen shall correctly represent the inelastic action in the column, as per the requirements in Section K2.3a. In addition, in SMF, IMF, and EBF, the depth of the test column shall be no less than 90% of the depth of the prototype column. In C-SMF and C-IMF, the depth of the structural steel member that forms part of the test column shall be no less than 90% of the depth of the structural steel member that forms part of the prototype column.

The width-to-thickness ratios of compression elements of steel members of the test specimen shall meet the width-to-thickness limitations as specified in these Provisions for members in SMF, IMF, C-SMF, C-IMF, or EBF, as applicable.

Exception: The width-to-thickness ratios of compression elements of members in the test specimen are permitted to exceed the width-to-thickness limitations specified in these Provisions if both of the following conditions are met:

(a) The width-to-thickness ratios of compression elements of the members of the test specimen are no less than the width-to-thickness ratios of compression elements in the corresponding prototype members.

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6845	(b) Design features that are intended to restrain local buckling in	in
6846	the test specimen such as concrete encasement of stee	
6847	members, concrete filling of steel members and other simils	
6848	features are representative of the corresponding design	
6849	features in the prototype.	
6850	Extrapolation beyond the limitations stated in this section is permitted	h
6851	subject to qualified peer review and approval by the authority having	
6852	jurisdiction.	اج
6853	K2.3c. Reinforcing Steel Amount, Size and Detailing	
6854	The total area of the longitudinal reinforcing bars shall not be less that	an
6855	75% of the area in the prototype, and individual bars shall not have a	
6856	area less than 70% of the maximum bar size in the prototype.	<b>,</b> 111
6857	Design approaches and methods used for anchorage and development	nt
6858	of reinforcement, and for splicing reinforcement in the test specime	
6859	shall be representative of the prototype.	
6860	The amount, arrangement and hook configurations for transvers	se.
6861	reinforcement shall be representative of the bond, confinement and	
6862	anchorage conditions of the prototype.	10
6863	K2.3d. Connection Details	
		ıe.
6864	The connection details used in the test specimen shall represent the	
6864 6865	The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection	on
6864 6865 6866	The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection elements used in the test specimen shall be a full-scale representation	on on
6864 6865	The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection	on on
6864 6865 6866 6867	The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection elements used in the test specimen shall be a full-scale representation of the connection elements used in the prototype, for the member size	on on
6864 6865 6866 6867 6868	The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection elements used in the test specimen shall be a full-scale representation of the connection elements used in the prototype, for the member size being tested.  K2.3e.Continuity Plates	on on es
6864 6865 6866 6867 6868 6869	The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection elements used in the test specimen shall be a full-scale representation of the connection elements used in the prototype, for the member size being tested.  K2.3e.Continuity Plates  The size and connection details of continuity plates used in the test specimen shall represent the prototype connection.	on on es
6864 6865 6866 6867 6868 6869 6870 6871	The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection elements used in the test specimen shall be a full-scale representation of the connection elements used in the prototype, for the member size being tested.  K2.3e.Continuity Plates  The size and connection details of continuity plates used in the test specimen shall be proportioned to match the size and connection	on on es
6864 6865 6866 6867 6868 6869	The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection elements used in the test specimen shall be a full-scale representation of the connection elements used in the prototype, for the member size being tested.  K2.3e.Continuity Plates  The size and connection details of continuity plates used in the test specimen shall represent the prototype connection.	on on es
6864 6865 6866 6867 6868 6869 6870 6871 6872	The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection elements used in the test specimen shall be a full-scale representation of the connection elements used in the prototype, for the member size being tested.  K2.3e.Continuity Plates  The size and connection details of continuity plates used in the test specimen shall be proportioned to match the size and connection details of continuity plates used in the prototype connection as closely.	on on es
6864 6865 6866 6867 6868 6869 6870 6871 6872 6873	The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection elements used in the test specimen shall be a full-scale representation of the connection elements used in the prototype, for the member size being tested.  K2.3e.Continuity Plates  The size and connection details of continuity plates used in the tespecimen shall be proportioned to match the size and connection details of continuity plates used in the prototype connection as closely as possible.	on on es est on ly
6864 6865 6866 6867 6868 6869 6870 6871 6872 6873	The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection elements used in the test specimen shall be a full-scale representation of the connection elements used in the prototype, for the member size being tested.  K2.3e.Continuity Plates  The size and connection details of continuity plates used in the tespecimen shall be proportioned to match the size and connection details of continuity plates used in the prototype connection as closed as possible.  K2.3f.Steel Strength for Steel Members and Connection Elements	on on es est on lly
6864 6865 6866 6867 6868 6869 6870 6871 6872 6873 6874	The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection elements used in the test specimen shall be a full-scale representation of the connection elements used in the prototype, for the member size being tested.  K2.3e.Continuity Plates  The size and connection details of continuity plates used in the test specimen shall be proportioned to match the size and connection details of continuity plates used in the prototype connection as closed as possible.  K2.3f.Steel Strength for Steel Members and Connection Elements  The following additional requirements shall be satisfied for each steel	on on es est on lly
6864 6865 6866 6867 6868 6869 6870 6871 6872 6873 6874 6875 6876 6877	The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection elements used in the test specimen shall be a full-scale representation of the connection elements used in the prototype, for the member size being tested.  K2.3e.Continuity Plates  The size and connection details of continuity plates used in the test specimen shall be proportioned to match the size and connection details of continuity plates used in the prototype connection as closed as possible.  K2.3f.Steel Strength for Steel Members and Connection Elements  The following additional requirements shall be satisfied for each steemember or connection element of the test specimen that supplied in elastic rotation by yielding:	on on es est on ly
6864 6865 6866 6867 6868 6869 6870 6871 6872 6873 6874	The connection details used in the test specimen shall represent the prototype connection details as closely as possible. The connection elements used in the test specimen shall be a full-scale representation of the connection elements used in the prototype, for the member size being tested.  K2.3e.Continuity Plates  The size and connection details of continuity plates used in the test specimen shall be proportioned to match the size and connection details of continuity plates used in the prototype connection as closed as possible.  K2.3f.Steel Strength for Steel Members and Connection Elements  The following additional requirements shall be satisfied for each steemember or connection element of the test specimen that supplies	on on es est on ly

6881		prohibited for the purposes of this section.
6882 6883 6884 6885	(b)	The yield strength of the beam flange as tested in accordance with Section K2.6a shall not be more than 15% below $R_yF_y$ for the grade of steel to be used for the corresponding elements of the prototype.
6886 6887 6888 6889	(c)	The yield strength of the columns and connection elements shall not be more than 15% above or below $R_yF_y$ for the grade of steel to be used for the corresponding elements of the prototype. $R_yF_y$ shall be determined in accordance with Section A3.2.
6890 6891 6892 6893 6894 6895		<b>User Note:</b> Based upon the above criteria, steel of the specified grade with a specified minimum yield stress, $F_y$ , of up to and including 1.15 times the $R_yF_y$ for the steel tested should be permitted in the prototype. In production, this limit should be checked using the values stated on the steel manufacturer's material test reports.
6896	K2.3g.Steel St	trength and Grade for Reinforcing Steel
6897 6898 6899 6900 6901	designa specific shall r	rcing steel in the test specimen shall have the same ASTM ation as the corresponding reinforcing steel in the prototype. The ed minimum yield stress of reinforcing steel in the test specimen not be less than the specified minimum yield stress of the bonding reinforcing steel in the prototype.
6902	K2.3h.Concre	ete Strength and Density
6903 6904 6905 6906	connec more t	pecified compressive strength of concrete in members and tion elements of the test specimen shall be at least 75% and no han 125% of the specified compressive strength of concrete in responding members and connection elements of the prototype.
6907 6908		ompressive strength of concrete in the test specimen shall be ined in accordance with Section K2.6d.
6909 6910 6911 6912 6913 6914	connec density connec concre	ensity classification of the concrete in the members and ction elements of the test specimen shall be the same as the classification of concrete in the corresponding members and ction elements of the prototype. The density classification of the shall correspond to either normal weight, lightweight, alleight, or sand-lightweight as defined in ACI 318.
6915	K2.3i.Welded	Joints
6916	Welds	on the test specimen shall satisfy the following requirements:
6917	(a)	Welding shall be performed in conformance with Welding

Procedure Specifications (WPS) as required in AWS D1.1/D1.1M. The WPS essential variables shall satisfy the requirements in AWS D1.1/D1.1M and shall be within the parameters established by the filler-metal manufacturer. The tensile strength and Charpy V-notch (CVN) toughness of the welds used in the test specimen shall be determined by tests as specified in Section K2.6e, made using the same filler metal classification, manufacturer, brand or trade name, diameter, and average heat input for the WPS used on the test specimen. The use of tensile strength and CVN toughness values that are reported on the manufacturer's typical certificate of conformance in lieu of physical testing is prohibited for purposes of this section.

(b) The specified minimum tensile strength of the filler metal used for the test specimen shall be the same as that to be used for the welds on the corresponding prototype. The tensile strength of the deposited weld as tested in accordance with Section K2.6c shall not exceed the tensile strength classification of the filler metal specified for the prototype by more than 25 ksi (172 MPa).

User Note: Based upon the criteria in (2) above, should the tested tensile strength of the weld metal exceed 25 ksi (172 MPa) above the specified minimum tensile strength, the prototype weld should be made with a filler metal and WPS that will provide a tensile strength no less than 25 ksi (172 MPa) below the tensile strength measured in the material test plate. When this is the case, the tensile strength of welds resulting from use of the filler metal and the WPS to be used in the prototype should be determined by using an all-weld-metal tension specimen. The test plate is described in AWS D1.8/D1.8M clause A6 and shown in AWS D1.8/D1.8M Figure A 1

(c) The specified minimum CVN toughness of the filler metal used for the test specimen shall not exceed that to be used for the welds on the corresponding prototype. The tested CVN toughness of the weld as tested in accordance with Section K2.6c shall not exceed the minimum CVN toughness specified for the prototype by more than 50%, nor 25 ft-lb (34 kJ), whichever is greater.

**User Note:** Based upon the criteria in (3) above, should the tested CVN toughness of the weld metal in the material test specimen exceed the specified CVN toughness for the test specimen by 25 ft-lb (34 kJ) or 50%, whichever is greater, the

prototype weld should be made with a filler metal and WPS that will provide a CVN toughness that is no less than 25 ft-lb (34 kJ) or 33% lower, whichever is lower, below the CVN toughness measured in the weld metal material test plate. When this is the case, the weld properties resulting from the filler metal and WPS to be used in the prototype should be determined using five CVN test specimens. The test plate is described in AWS D1.8/D1.8M clause A6 and shown in AWS D1.8/D1.8M Figure A.1. 

- (d) The welding positions used to make the welds on the test specimen shall be the same as those to be used for the prototype welds.
- (e) Weld details such as backing, tabs and access holes used for the test specimen welds shall be the same as those to be used for the corresponding prototype welds. Weld backing and weld tabs shall not be removed from the test specimen welds unless the corresponding weld backing and weld tabs are removed from the prototype welds.
- (f) Methods of inspection and nondestructive testing and standards of acceptance used for test specimen welds shall be the same as those to be used for the prototype welds.

**User Note:** The filler metal used for production of the prototype is permitted to be of a different classification, manufacturer, brand or trade name, and diameter, provided that Sections K2.3f(b) and K2.3f(c) are satisfied. To qualify alternate filler metals, the tests as prescribed in Section K2.6c should be conducted.

#### **K2.3j.Bolted Joints**

The bolted portions of the test specimen shall replicate the bolted portions of the prototype connection as closely as possible. Additionally, bolted portions of the test specimen shall satisfy the following requirements:

- (a) The bolt grade (for example, ASTM A325, A325M, ASTM A490, A490M, ASTM F1852, ASTM F2280) used in the test specimen shall be the same as that to be used for the prototype, except that heavy hex bolts are permitted to be substituted for twist-off-type tension control bolts of equal minimum specified tensile strength, and vice versa.
- (b) The type and orientation of bolt holes (standard, oversize, short slot, long slot or other) used in the test specimen shall be the same as those to be used for the corresponding bolt holes in the prototype.

7001	(c) When inelastic rotation is to be developed either by yielding or
7002	by slip within a bolted portion of the connection, the method used to
7003	make the bolt holes (drilling, sub-punching and reaming, or other) in
7004	the test specimen shall be the same as that to be used in the
7005	corresponding bolt holes in the prototype.

(d) Bolts in the test specimen shall have the same installation (pretensioned or other) and faying surface preparation (no specified slip resistance, Class A or B slip resistance, or other) as that to be used for the corresponding bolts in the prototype.

#### K2.3k. Load Transfer Between Steel and Concrete

Methods used to provide load transfer between steel and concrete in the members and connection elements of the test specimen, including direct bearing, shear connection, friction and others, shall be representative of the prototype.

#### **K2.4.** Loading History

#### **K2.4a.** General Requirements

The test specimen shall be subjected to cyclic loads in accordance with the requirements prescribed in Section K2.4b for beam-to-column moment connections in SMF, IMF, C-SMF, and C-IMF, and in accordance with the requirements prescribed in Section K2.4c for link-to-column connections in EBF.

Loading sequences to qualify connections for use in SMF, IMF, C-SMF, or C-IMF with columns loaded orthogonally shall be applied about both axes using the loading sequence specified in Section K2.4b. Beams used about each axis shall represent the most demanding combination for which qualification or prequalification is sought. In lieu of concurrent application about each axis of the loading sequence specified in Section K2.4b, the loading sequence about one axis shall satisfy requirements of Section K2.4b while a concurrent load of constant magnitude, equal to the expected strength of the beam connected to the column about its orthogonal axis, shall be applied about the orthogonal axis.

Loading sequences other than those specified in Sections K2.4b and K2.4c are permitted to be used when they are demonstrated to be of equivalent or greater severity.

#### **K2.4b.** Loading Sequence for Beam-to-Column Moment Connections

Qualifying cyclic tests of beam-to-column moment connections in SMF, IMF, C-SMF and C-IMF shall be conducted by controlling the

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7039	story drift angle, $\theta$ , imposed on the test specimen, as specified below:
7040	(a) 6 cycles at $\theta = 0.00375$ rad
7041	(b) 6 cycles at $\theta = 0.005$ rad
7042	(c) 6 cycles at $\theta = 0.0075$ rad
7043	(d) 4 cycles at $\theta = 0.01$ rad
7044	(e) 2 cycles at $\theta = 0.015$ rad
7045	(f) 2 cycles at $\theta = 0.02$ rad
7046	(g) 2 cycles at $\theta = 0.03$ rad
7047	(h) 2 cycles at $\theta = 0.04$ rad
7048 7049	Continue loading at increments of $\theta = 0.01$ rad, with two cycles of loading at each step.
7050	K2.4c. Loading Sequence for Link-to-Column Connections
7051 7052 7053	Qualifying cyclic tests of link-to-column moment connections in EBF shall be conducted by controlling the total link rotation angle, $\gamma_{total}$ , imposed on the test specimen, as follows:
1033	imposed on the test specifien, as follows.
7054	(a) 6 cycles at $\gamma_{\text{total}} = 0.00375$ rad
7054	(a) 6 cycles at $\gamma_{\text{total}} = 0.00375 \text{ rad}$
7054 7055	(a) 6 cycles at $\gamma_{total} = 0.00375$ rad (b) 6 cycles at $\gamma_{total} = 0.005$ rad
7054 7055 7056	(a) 6 cycles at $\gamma_{total} = 0.00375$ rad (b) 6 cycles at $\gamma_{total} = 0.005$ rad (c) 6 cycles at $\gamma_{total} = 0.0075$ rad
7054 7055 7056 7057	(a) 6 cycles at $\gamma_{total} = 0.00375$ rad (b) 6 cycles at $\gamma_{total} = 0.005$ rad (c) 6 cycles at $\gamma_{total} = 0.0075$ rad (d) 6 cycles at $\gamma_{total} = 0.01$ rad
7054 7055 7056 7057 7058	(a) 6 cycles at $\gamma_{total} = 0.00375$ rad (b) 6 cycles at $\gamma_{total} = 0.005$ rad (c) 6 cycles at $\gamma_{total} = 0.0075$ rad (d) 6 cycles at $\gamma_{total} = 0.01$ rad (e) 4 cycles at $\gamma_{total} = 0.015$ rad
7054 7055 7056 7057 7058 7059	(a) 6 cycles at $\gamma_{total} = 0.00375$ rad (b) 6 cycles at $\gamma_{total} = 0.005$ rad (c) 6 cycles at $\gamma_{total} = 0.0075$ rad (d) 6 cycles at $\gamma_{total} = 0.01$ rad (e) 4 cycles at $\gamma_{total} = 0.015$ rad (f) 4 cycles at $\gamma_{total} = 0.02$ rad
7054 7055 7056 7057 7058 7059 7060	(a) 6 cycles at $\gamma_{total} = 0.00375$ rad (b) 6 cycles at $\gamma_{total} = 0.005$ rad (c) 6 cycles at $\gamma_{total} = 0.0075$ rad (d) 6 cycles at $\gamma_{total} = 0.01$ rad (e) 4 cycles at $\gamma_{total} = 0.015$ rad (f) 4 cycles at $\gamma_{total} = 0.02$ rad (g) 2 cycles at $\gamma_{total} = 0.03$ rad
7054 7055 7056 7057 7058 7059 7060 7061	(a) 6 cycles at $\gamma_{total} = 0.00375$ rad (b) 6 cycles at $\gamma_{total} = 0.005$ rad (c) 6 cycles at $\gamma_{total} = 0.0075$ rad (d) 6 cycles at $\gamma_{total} = 0.01$ rad (e) 4 cycles at $\gamma_{total} = 0.015$ rad (f) 4 cycles at $\gamma_{total} = 0.02$ rad (g) 2 cycles at $\gamma_{total} = 0.03$ rad (h) 1 cycle at $\gamma_{total} = 0.04$ rad
7054 7055 7056 7057 7058 7059 7060 7061 7062	(a) 6 cycles at $\gamma_{total} = 0.00375$ rad (b) 6 cycles at $\gamma_{total} = 0.005$ rad (c) 6 cycles at $\gamma_{total} = 0.0075$ rad (d) 6 cycles at $\gamma_{total} = 0.01$ rad (e) 4 cycles at $\gamma_{total} = 0.015$ rad (f) 4 cycles at $\gamma_{total} = 0.02$ rad (g) 2 cycles at $\gamma_{total} = 0.03$ rad (h) 1 cycle at $\gamma_{total} = 0.04$ rad (i) 1 cycle at $\gamma_{total} = 0.05$ rad

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7066		loading at each step.
7067	K2.5.	Instrumentation
7068 7069 7070		Sufficient instrumentation shall be provided on the test specimen to permit measurement or calculation of the quantities listed in Section K2.7.
7071	K2.6.	<b>Testing Requirements for Material Specimens</b>
7072 7073	K2.6a	Tension Testing Requirements for Structural Steel Material Specimens
7074 7075 7076 7077 7078 7079 7080		Tension testing shall be conducted on samples taken from material test plates in accordance with Section K2.6c. The material test plates shall be taken from the steel of the same heat as used in the test specimen. Tension-test results from certified material test reports shall be reported, but shall not be used in lieu of physical testing for the purposes of this section. Tension testing shall be conducted and reported for the following portions of the test specimen:
7081 7082		(a) Flange(s) and web(s) of beams and columns at standard locations
7083 7084		(b) Any element of the connection that supplies inelastic rotation by yielding
7085 7086	K2.6b	Tension Testing Requirements for Reinforcing Steel Material Specimens
7087 7088 7089 7090 7091 7092		Tension testing shall be conducted on samples of reinforcing steel in accordance with Section K2.6c. Samples of reinforcing steel used for material tests shall be taken from the same heat as used in the test specimen. Tension-test results from certified material test reports shall be reported, but shall not be used in lieu of physical testing for the purposes of this section.
7093 7094	K2.6c	. Methods of Tension Testing for Structural and Reinforcing Steel Material Specimens
7095 7096 7097		Tension testing shall be conducted in accordance with ASTM $A6/A6M$ , ASTM $A370$ , and ASTM $E8$ , as applicable, with the following exceptions:
7098 7099 7100		(a) The yield strength, $F_y$ , that is reported from the test shall be based upon the yield strength definition in ASTM A370, using the offset method at 0.002 in./in. strain.
7101		(b) The loading rate for the tension test shall replicate, as closely as

7102 practical, the loading rate to be used for the test specimen.

### **K2.6d.** Testing Requirements for Concrete

Test cylinders of concrete used for the test specimen shall be made and cured in accordance with ASTM C31. At least three cylinders of each batch of concrete used in a component of the test specimen shall be tested within five days before or after of the end of the cyclic qualifying test of the test specimen. Tests of concrete cylinders shall be in accordance with ASTM C39. The average compressive strength of the three cylinders shall be no less than 90% and no greater than 150% of the specified compressive strength of the concrete in the corresponding member or connection element of the test specimen. In addition, the average compressive strength of the three cylinders shall be no more than 3000 psi greater than the specified compressive strength of the concrete in the corresponding member or connection element of the test specimen.

Exception: If the average compressive strength of three cylinders is outside of these limits, the specimen is still acceptable if supporting calculations or other evidence is provided to demonstrate how the difference in concrete strength will affect the connection performance.

# **K2.6e.** Testing Requirements for Weld Metal Material Specimens

Weld metal testing shall be conducted on samples extracted from the material test plate, made using the same filler metal classification, manufacturer, brand or trade name and diameter, and using the same average heat input as used in the welding of the test specimen. The tensile strength and CVN toughness of weld material specimens shall be determined in accordance with Standard Methods for Mechanical Testing of Welds (AWS B4.0/B4.0M). The use of tensile strength and CVN toughness values that are reported on the manufacturer's typical certificate of conformance in lieu of physical testing is prohibited for use for purposes of this section.

The same WPS shall be used to make the test specimen and the material test plate. The material test plate shall use base metal of the same grade and type as was used for the test specimen, although the same heat need not be used. If the average heat input used for making the material test plate is not within  $\pm 20\%$  of that used for the test specimen, a new material test plate shall be made and tested.

#### **K2.7**. Test Reporting Requirements

For each test specimen, a written test report meeting the requirements of the authority having jurisdiction and the requirements of this section shall be prepared. The report shall thoroughly document all key

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7142 7143	feature inform	es and results of the test. The report shall include the following nation:
7144 7145 7146	(a)	A drawing or clear description of the test subassemblage, including key dimensions, boundary conditions at loading and reaction points, and location of lateral braces.
7147 7148 7149 7150 7151 7152 7153	(b)	A drawing of the connection detail showing member sizes, grades of steel, the sizes of all connection elements, welding details including filler metal, the size and location of bolt holes, the size and grade of bolts, specified compressive strength and density of concrete, reinforcing bar sizes and grades, reinforcing bar locations, reinforcing bar splice and anchorage details, and all other pertinent details of the connection.
7154 7155	(c)	A listing of all other essential variables for the test specimen, as listed in Section K2.3.
7156 7157	(d)	A listing or plot showing the applied load or displacement history of the test specimen.
7158	(e)	A listing of all welds to be designated demand critical.
7159 7160	(f)	Definition of the region of the member and connection to be designated a protected zone.
7161 7162 7163 7164 7165	(g)	A plot of the applied load versus the displacement of the test specimen. The displacement reported in this plot shall be measured at or near the point of load application. The locations on the test specimen where the loads and displacements were measured shall be clearly indicated.
7166 7167 7168 7169 7170	(h)	A plot of beam moment versus story drift angle for beam-to-column moment connections; or a plot of link shear force versus link rotation angle for link-to-column connections. For beam-to-column connections, the beam moment and the story drift angle shall be computed with respect to the centerline of the column.
7171 7172 7173 7174 7175 7176	(i)	The story drift angle and the total inelastic rotation developed by the test specimen. The components of the test specimen contributing to the total inelastic rotation shall be identified. The portion of the total inelastic rotation contributed by each component of the test specimen shall be reported. The method used to compute inelastic rotations shall be clearly shown.
7177 7178 7179 7180	(j)	A chronological listing of test observations, including observations of yielding, slip, instability, cracking and rupture of steel elements, cracking of concrete, and other damage of any portion of the test specimen as applicable.

7181	(k)	The controlling failure mode for the test specimen. If the test is
7182		terminated prior to failure, the reason for terminating the test
7183		shall be clearly indicated.

- (l) The results of the material specimen tests specified in Section K2.6.
  - (m) The welding procedure specifications (WPS) and welding inspection reports.

Additional drawings, data, and discussion of the test specimen or test results are permitted to be included in the report.

# **K2.8.** Acceptance Criteria

The test specimen must satisfy the strength and story drift angle or link rotation angle requirements of these Provisions for the SMF, IMF, C-SMF, C-IMF, or EBF connection, as applicable. The test specimen must sustain the required story drift angle or link rotation angle for at least one complete loading cycle.

# K3. CYCLIC TESTS FOR QUALIFICATION OF BUCKLING-RESTRAINED BRACES

# K3.1. Scope

This section includes requirements for qualifying cyclic tests of individual buckling-restrained braces and buckling-restrained brace subassemblages, when required in these provisions. The purpose of the testing of individual braces is to provide evidence that a buckling-restrained brace satisfies the requirements for strength and inelastic deformation by these provisions; it also permits the determination of maximum brace forces for design of adjoining elements. The purpose of testing of the brace subassemblage is to provide evidence that the brace-design is able to satisfactorily accommodate the deformation and rotational demands associated with the design. Further, the subassemblage test is intended to demonstrate that the hysteretic behavior of the brace in the subassemblage is consistent with that of the individual brace elements tested uniaxially.

Alternative testing requirements are permitted when approved by the engineer of record and the authority having jurisdiction. This section provides only minimum recommendations for simplified test conditions.

### **K3.2.** Subassemblage Test Specimen

7222 The subassemblage test specimen shall satisfy the following

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7224 7225 7226 7227 7228	(a)	The mechanism for accommodating inelastic rotation in the subassemblage test specimen brace shall be the same as that of the prototype. The rotational deformation demands on the subassemblage test specimen brace shall be equal to or greater than those of the prototype.
7229 7230 7231 7232 7233	(b)	The axial yield strength of the steel core, $P_{ysc}$ , of the brace in the subassemblage test specimen shall not be less than 90% of that of the prototype where both strengths are based on the core area, $A_{sc}$ , multiplied by the yield strength as determined from a coupon test.
7234 7235 7236	(c)	The cross-sectional shape and orientation of the steel core projection of the subassemblage test specimen brace shall be the same as that of the brace in the prototype.
7237 7238 7239 7240 7241 7242	(d)	The same documented design methodology shall be used for design of the subassemblage as used for the prototype, to allow comparison of the rotational deformation demands on the subassemblage brace to the prototype. In stability calculations, beams, columns and gussets connecting the core shall be considered parts of this system.
7243 7244 7245 7246 7247 7248 7249 7250 7251 7252	(e)	The calculated margins of safety for the prototype connection design, steel core projection stability, overall buckling and other relevant subassemblage test specimen brace construction details, excluding the gusset plate, for the prototype, shall equal or exceed those of the subassemblage test specimen construction. If the qualification brace test specimen required in Section K3.3 was also tested including the subassemblage requirements of this section, the lesser safety factor for overall buckling between that required in Section K3.3a(a) and that required in this section may be used.
7253 7254	(f)	Lateral bracing of the subassemblage test specimen shall replicate the lateral bracing in the prototype.
7255 7256 7257 7258	(g)	The brace test specimen and the prototype shall be manufactured in accordance with the same quality control and assurance processes and procedures.
7259 7260 7261	-	polation beyond the limitations stated in this section is permitted at to qualified peer review and approval by the authority having ction.

requirements:

**K3.3.** Brace Test Specimen

7263				
7264	The brace test specimen shall replicate as closely as is practical the			
7265	pertinent design, detailing, construction features and material			
7266	properties of the prototype.			
7267	K3.3a. Design of Brace Test Specimen			
7268	The same documented design methodology shall be used for the brace			
7269	test specimen and the prototype. The design calculations shall			
7270	demonstrate, at a minimum, the following requirements:			
7271	(a) The calculated margin of safety for stability against overall			
7272	buckling for the prototype shall equal or exceed that of the			
7273	brace test specimen.			
7274	(b) The calculated margins of safety for the brace test specimen and			
7275	the prototype shall account for differences in material			
7276	properties, including yield and ultimate stress, ultimate			
7277	elongation, and toughness.			
7278	K3.3b. Manufacture of Brace Test Specimen			
7279	The brace test specimen and the prototype shall be manufactured in			
7280	accordance with the same quality control and assurance processes and			
7281	procedures.			
7282	K3.3c. Similarity of Brace Test Specimen and Prototype			
7283	The brace test specimen shall meet the following requirements:			
7284	(a) The cross-sectional shape and orientation of the steel core shall			
7285	be the same as that of the prototype.			
7286	(b) The axial yield strength of the steel core, P <sub>ysc</sub> , of the brace test			
7287	specimen shall not be less than 30% nor more than 120% of the			
7288	prototype where both strengths are based on the core area, A <sub>sc</sub> ,			
7289	multiplied by the yield strength as determined from a coupon			
7290	test.			
7291	(c) The material for, and method of, separation between the steel			
7292	core and the buckling restraining mechanism in the brace test			
7293	specimen shall be the same as that in the prototype.			
7294	Extrapolation beyond the limitations stated in this section is permitted			
7295	subject to qualified peer review and approval by the authority having			
7296	jurisdiction.			
7297	K3.3d. Connection Details			

7298 7299	The connection details used in the brace test specimen shall represent the prototype connection details as closely as practical.			
7300	K3.3e. Materials			
7301		1.	Steel	Core
7302 7303				ollowing requirements shall be satisfied for the steel core brace test specimen:
7304 7305 7306			(a)	The specified minimum yield stress of the brace test specimen steel core shall be the same as that of the prototype.
7307 7308 7309			(b)	The measured yield stress of the material of the steel core in the brace test specimen shall be at least 90% of that of the prototype as determined from coupon tests.
7310 7311 7312			(c)	The specified minimum ultimate stress and strain of the brace test specimen steel core shall not exceed those of the prototype.
7313		2.	Buck	ling-Restraining Mechanism
7314 7315 7316				rials used in the buckling-restraining mechanism of the test specimen shall be the same as those used in the type.
7317	K3.3f.	Conne	ections	
7318 7319				bolted and pinned joints on the test specimen shall e on the prototype as close as practical.
7320	K3.4.	Loadi	ng His	tory
7321	K3.4a	. Gener	al Req	uirements
7322 7323 7324 7325 7326		the rec increm permit	quirements of ted. E	imen shall be subjected to cyclic loads in accordance with ents prescribed in Sections K3.4b and K3.4c. Additional f loading beyond those described in Section K3.4c are ach cycle shall include a full tension and full compression the prescribed deformation.
7327	K3.4b	. Test C	Contro	
7328 7329 7330 7331 7332		rotatio alterna applied	nal de ite, the	formation, $\Delta_b$ , imposed on the test specimen. As an emaximum rotational deformation is permitted to be maintained as the protocol is followed for axial

7333	K3.4c.	Loading Sequence
7334 7335 7336 7337		Loads shall be applied to the test specimen to produce the following deformations, where the deformation is the steel core axial deformation for the test specimen and the rotational deformation demand for the subassemblage test specimen brace:
7338		(a) 2 cycles of loading at the deformation corresponding to $\Delta_b = \Delta_{by}$
7339 7340		(b) 2 cycles of loading at the deformation corresponding to $\Delta_b = 0.50 \; \Delta_{bm}$
7341 7342		(c) 2 cycles of loading at the deformation corresponding to $\Delta_b=1$ $\Delta_{bm}$
7343 7344		(d) 2 cycles of loading at the deformation corresponding to $\Delta_b=1.5$ $\Delta_{bm}$
7345 7346		(e) 2 cycles of loading at the deformation corresponding to $\Delta_b=2.0$ $\Delta_{bm}$
7347 7348 7349 7350 7351		(f) Additional complete cycles of loading at the deformation corresponding to $\Delta_b = 1.5 \Delta_{bm}$ as required for the brace test specimen to achieve a cumulative inelastic axial deformation of at least 200 times the yield deformation (not required for the subassemblage test specimen)
7352 7353 7354 7355 7356		where $\Delta_{bm}\!\!=\!$
7357 7358 7359 7360 7361		The design story drift shall not be taken as less than 0.01 times the story height for the purposes of calculating $\Delta_{bm}$ . Other loading sequences are permitted to be used to qualify the test specimen when they are demonstrated to be of equal or greater severity in terms of maximum and cumulative inelastic deformation.
7362 7363 7364 7365 7366	K3.5.	Instrumentation Sufficient instrumentation shall be provided on the test specimen to permit measurement or calculation of the quantities listed in Section K3.7.
7367	K3.6.	<b>Materials Testing Requirements</b>
7368	K3.6a	Tension Testing Requirements

Tension testing shall be conducted on samples of steel taken from the same heat of steel as that used to manufacture the steel core. Tension test results from certified material test reports shall be reported but are prohibited in place of material specimen testing for the purposes of this Section. Tension test results shall be based upon testing that is conducted in accordance with Section K3.6b.

#### **K3.6b.** Methods of Tension Testing

 Tension testing shall be conducted in accordance with ASTM A6, ASTM A370 and ASTM E8, with the following exceptions:

- (a) The yield stress that is reported from the test shall be based upon the yield strength definition in ASTM A370, using the offset method of 0.002 strain.
- (b) The loading rate for the tension test shall replicate, as closely as is practical, the loading rate used for the test specimen.
- (c) The coupon shall be machined so that its longitudinal axis is parallel to the longitudinal axis of the steel core.

# **K3.7.** Test Reporting Requirements

For each test specimen, a written test report meeting the requirements of this Section shall be prepared. The report shall thoroughly document all key features and results of the test. The report shall include the following information:

- (a) A drawing or clear description of the test specimen, including key dimensions, boundary conditions at loading and reaction points, and location of lateral bracing, if any.
- (b) A drawing of the connection details showing member sizes, grades of steel, the sizes of all connection elements, welding details including filler metal, the size and location of bolt or pin holes, the size and grade of connectors, and all other pertinent details of the connections.
- (c) A listing of all other essential variables as listed in Sections K3.2 or K3.3.
- (d) A listing or plot showing the applied load or displacement history.
- (e) A plot of the applied load versus the deformation,  $\Delta_b$ . The method used to determine the deformations shall be clearly shown. The locations on the test specimen where the loads and deformations were measured shall be clearly identified.

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7407 7408 7409 7410	(f) A chronological listing of test observations, including observations of yielding, slip, instability, transverse displacement along the test specimen and rupture of any portion of the test specimen and connections, as applicable.
7411 7412	(g) The results of the material specimen tests specified in Section K3.6.
7413 7414 7415 7416 7417 7418	<ul> <li>(h) The manufacturing quality control and quality assurance plans used for the fabrication of the test specimen. These shall be included with the welding procedure specifications and welding inspection reports.</li> <li>Additional drawings, data and discussion of the test specimen or test</li> </ul>
7419	results are permitted to be included in the report.
7420 <b>K3.8.</b> 7421 7422	Acceptance Criteria  At least one subassemblege test that satisfies the requirements of
7422 7423 7424 7425 7426	At least one subassemblage test that satisfies the requirements of Section K3.2 shall be performed. At least one brace test that satisfies the requirements of Section K3.3 shall be performed. Within the required protocol range all tests shall satisfy the following requirements:
7427 7428 7429	(a) The plot showing the applied load vs. displacement history shall exhibit stable, repeatable behavior with positive incremental stiffness.
7430 7431	(b) There shall be no rupture, brace instability, or brace end connection failure.
7432 7433 7434	(c) For brace tests, each cycle to a deformation greater than $\Delta_{by}$ the maximum tension and compression forces shall not be less than the nominal strength of the core.
7435 7436 7437	(d) For brace tests, each cycle to a deformation greater than $\Delta_{by}$ the ratio of the maximum compression force to the maximum tension force shall not exceed 1.5.
7438	Other acceptance criteria are permitted to be adopted for the brace test

specimen or subassemblage test specimen subject to qualified peer

review and approval by the authority having jurisdiction.

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