Chapter 8

STEEL STRUCTURE DESIGN REQUIREMENTS

8.1 GENERAL

8.1.1 Scope. The design, construction, and quality of steel components that resist seismic forces shall comply with the requirements of this chapter.

8.1.2 References. The following documents shall be used as specified in this chapter.


AISI—GP  Standard for Cold-Formed Steel Framing—General Provisions, American Iron and Steel Institute, 2001

AISI—PM  Standard for Cold-Formed Steel Framing—Prescriptive Method for One and Two-Family Dwellings, American Iron and Steel Institute, 2001

ASCE 8  Specification for the Design of Cold-formed Stainless Steel Structural Members, American Society of Civil Engineers, 2002.


ASCE 19  Structural Applications of Steel Cables for Buildings, American Society of Civil Engineers, 1996.


8.1.3 Definitions

Dead load: See Sec. 4.1.3.

Design strength: See Sec. 4.1.3.

Diaphragm: See Sec. 4.1.3.
Light-framed wall: See Sec. 12.1.3.
Light-framed shear wall: See Sec. 12.1.3.
Live load: See Sec. 4.1.3.
Nominal strength: See Sec. 4.1.3.
Registered design professional: See Sec. 2.1.3.
Required strength: See Sec. 4.1.3.
Seismic Design Category: See Sec. 1.1.4.
Seismic forces: See Sec. 1.1.4.
Shear panel: See Sec. 4.1.3.
Shear wall: See Sec. 4.1.3.
Story: See Sec. 4.1.3.
Structure: See Sec. 1.1.4.
Wall: See Sec. 4.1.3.

8.1.4 Notation

\( R \) See Sec. 4.1.4.

\( T_3 \) Net tension in steel cable due to dead load, prestress, and seismic load (Sec. 8.5).

\( T_4 \) Net tension in steel cable due to dead load, prestress, live load, and seismic load (Sec. 8.5).

\( \phi \) See Sec. 5.1.3.

\( \Omega_0 \) See Sec. 4.1.4.

8.2 GENERAL DESIGN REQUIREMENTS

8.2.1 Seismic Design Categories B and C. Steel structures assigned to Seismic Design Category B or C shall be of any construction permitted by the references in Sec. 8.1.2. An \( R \) factor as set forth in Table 4.3-1 for the appropriate steel system is permitted where the structure is designed and detailed in accordance with the requirements of AISC Seismic, as modified in Sec. 8.3, or in accordance with Sec. 8.4.1 and 8.4.2, for light-frame cold-formed steel wall systems. Systems not detailed in accordance with the above shall use the \( R \) factor in Table 4.3-1 designated for “Steel Systems Not Specifically Detailed for Seismic Resistance.”

8.2.2 Seismic Design Categories D, E, and F. Steel structures assigned to Seismic Design Category D, E, or F shall be designed and detailed in accordance with AISC Seismic as modified in Sec. 8.3. Light-frame cold-formed steel wall systems shall be designed and detailed in accordance with Sec. 8.4.2.

8.3 STRUCTURAL STEEL

8.3.1 Material properties for determination of required strength. Revise Table I-6-1 of AISC Seismic, as follows:

1. For the Application titled “Hot-rolled structural shapes and bars, All other grades,” change the \( R_y \) value from 1.1 to 1.2.

For the Application titled “All other products,” change the \( R_y \) value from 1.1 to 1.2.
8.4 COLD-FORMED STEEL

The design of cold-formed carbon or low-alloy steel members to resist seismic loads shall be in accordance with the requirements of AISI – NASPEC and AISI General, and the design of cold-formed stainless steel structural members to resist seismic loads shall be in accordance with the requirements of ASCE 8, except as modified by this section.

8.4.1 Modifications to references

Modify Sec. 1.5.2 of ASCE 8 by substituting a load factor of 1.0 in place of 1.5 for nominal earthquake load.

8.4.2 Light-frame walls. Where required in Sec. 8.2.1 or 8.2.2, cold-formed steel stud walls designed in accordance with AISI – NASPEC, AISI-GP and ASCE 8 shall also comply with the requirements of this section.

8.4.2.1 Boundary members. All boundary members, chords, and collectors shall be designed to transmit the specified induced axial forces.

8.4.2.2 Connections. Connections for diagonal bracing members, top chord splices, boundary members, and collectors shall have a design strength equal to or greater than the nominal tensile strength of the members being connected or \( \Omega_0 \) times the design seismic force. The pull-out resistance of screws shall not be used to resist seismic forces.

8.4.2.3 Braced bay members. In stud systems where the lateral forces are resisted by diagonal braces, the vertical and diagonal members in braced bays shall be anchored such that the bottom tracks are not required to resist uplift forces by bending of the track or track web. Both flanges of studs shall be braced to prevent lateral torsional buckling. In shear wall systems, the vertical boundary members shall be anchored so the bottom track is not required to resist uplift forces by bending of the track web.

8.4.2.4 Diagonal braces. Provision shall be made for pretensioning or other methods of installation of tension-only bracing to guard against loose diagonal straps.

8.4.2.5 Shear walls. Nominal shear strengths for shear walls framed with cold-formed steel studs are given in Table 8.4-1. Design shear strength shall be determined by multiplying the nominal shear strength by a \( \phi \) factor of 0.55. The height to length ratio of wall systems listed in Table 8.4-1 shall not exceed 2:1. In structures over one story in height, the assemblies in Table 8.4-1 shall not be used to resist horizontal loads contributed by forces imposed by masonry or concrete construction.

Panel thicknesses shown in Table 8.4-1 shall be considered to be minimums. No panels less than 24 in. wide shall be used. Plywood or oriented strand board structural panels shall be of a type that is manufactured using exterior glue. Framing members, blocking or strapping shall be provided at the edges of all sheets. Fasteners along the edges in shear panels shall be placed not less than 3/8 in. (9.5 mm) in from panel edges. Perimeter members at openings shall be provided and shall be detailed to distribute the shearing stresses. Wood sheathing shall not be used to splice such members.

Studs shall be a minimum 1-5/8 in. (41 mm) by 3-1/2 in. (89 mm) with a 3/8-in. (9.5 mm) return lip. Track shall be a minimum 1-1/4 in. (32 mm) by 3-1/2 in. (89 mm). Both studs and track shall have a minimum uncoated base metal thickness of 0.033 in. (0.84 mm), shall not have an uncoated base metal thickness greater than 0.048 in. (1.22 mm), and shall satisfy the requirements for ASTM A 653 SS, Grade 33, ASTM A 792 SS, Grade 33, or ASTM A 875 SS, Grade 33. Panel end studs and their uplift anchorage shall have the design strength to resist the forces determined by the seismic loads determined using Eq. 4.2-3 and Eq. 4.2-4.
Framing screws shall be No. 8 x 5/8 in. (16 mm) wafer head self-drilling. Plywood and OSB screws shall be a minimum No. 8 x 1 in. (25 mm) bugle head. Where horizontal straps are used to provide blocking they shall be a minimum 1-1/2 in. (38 mm) wide and of the same material as the stud and track. Such straps shall have a thickness at least as great as the thicker of that of the stud and the track.

<table>
<thead>
<tr>
<th>Assembly Description</th>
<th>Fastener Spacing at Panel Edges (in.)&lt;sup&gt;b&lt;/sup&gt;</th>
<th>Framing Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>15/32 rated Structural I sheathing (4-ply) plywood one side&lt;sup&gt;c&lt;/sup&gt;</td>
<td>780 990 1465 1625</td>
<td>24 in. o.c.</td>
</tr>
<tr>
<td>7/16 in. oriented strand board one side&lt;sup&gt;c&lt;/sup&gt;</td>
<td>700 915 1275 1700</td>
<td>24 in. o.c.</td>
</tr>
</tbody>
</table>

<sup>a</sup> For metric: 1 in. = 25.4 mm, 1 plf = 14.6 N/m.
<sup>b</sup> Screws in the field of the panel shall be installed 12 in. o.c. unless otherwise shown.
<sup>c</sup> Both flanges of the studs shall be braced in accordance with Sec. 8.4.2.3.

### 8.4.3 Prescriptive framing
One and two family dwellings are permitted to be designed and constructed in accordance to the provisions in the AISI—PM subject to the limitations therein.

### 8.4.4 Steel deck diaphragms
Steel deck diaphragms shall be made from materials which satisfy the requirements of AISI and ASCE 8. Nominal strengths shall be determined in accordance with approved analytical procedures or with test procedures prepared by a registered design professional experienced in testing of cold-formed steel assemblies and approved by the authority having jurisdiction. Design strengths shall be determined by multiplying the nominal strength by a resistance factor, \( \phi \), equal to 0.60 (for mechanically connected diaphragms) and equal to 0.50 (for welded diaphragms). The steel deck installation for the structure, including fasteners, shall comply with the test assembly arrangement. Quality standards established for the nominal strength test shall be the minimum standards required for the steel deck installation, including fasteners.

### 8.5 STEEL CABLES
The design strength of steel cables shall be determined in accordance with ASCE 19 except as modified by these Provisions. A load factor of 1.1 shall be applied to the prestress force included in \( T_3 \) and \( T_4 \) as defined in Sec. 3.1.2 of ASCE 19. In Sec. 3.2.1 of ASCE 19, item (c) shall be replaced with “1.5 \( T_3 \)” and item (d) shall be replaced with “1.5 \( T_4 \)”.

### 8.6 RECOMMENDED PROVISIONS FOR BUCKLING-RESTRAINED BRACED FRAMES
The following shall be used in conjunction with AISC Seismic.

#### 8.6.1 Symbols
- \( A_{sc} \): Area of the yielding segment of steel core, in.\(^2\) (BRBF)
- \( P_{yyc} \): Axial yield strength of steel core, kips (BRBF)
- \( Q_b \): Maximum unbalanced load effect applied to beam by braces, kips. (BRBF)
- \( \beta \): Compression strength adjustment factor (BRBF)
- \( w \): Tension strength adjustment factor. (BRBF)
8.6.2 Glossary

Buckling Restraint Braced Frame (BRBF): A diagonally braced frame meeting the requirements of Sec. 8.6.3 in which all members of the bracing system are subjected primarily to axial forces and in which the limit state of compression buckling of braces is precluded at forces and deformations corresponding to 1.5 times the Design Story Drift.

Buckling-Restraining System: A system of restraints that limits buckling of the steel core in BRBF. This system includes the casing on 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 1.5 times the Design Story Drift.

Casing: An element that resists forces transverse to the axis of the 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 in the axis of the brace.

Steel Core: The axial-force-resisting element of braces in BRBF. The steel core contains a yielding segment and connections to transfer its axial force to adjoining elements; it may also contain projections beyond the casing and transition segments between the projections and yielding segment.

8.6.3 BUCKLING-RESTRAINED BRACED FRAMES (BRBF)

8.6.3.1 Scope. Buckling-restrained braced frames (BRBF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the Design Earthquake. BRBF shall meet the requirements in this section.

8.6.3.2 Bracing Members

8.6.3.2.1 Composition: Bracing members shall be composed of a structural steel core and a system that restrains the steel core from buckling.

8.6.3.2.1.1 Steel core. The steel core shall be designed to resist the entire axial force in the brace.

8.6.3.2.1.1.1 Required strength of steel core. The required axial strength of the brace shall not exceed the design strength of the steel core, \( \phi P_{ysc} \), where \( \phi = 0.9 \)

\[ P_{ysc} = F_y A_{sc} \]

\( F_y \) = specified minimum yield strength of steel core

\( A_{sc} \) = net area of steel core

8.6.3.2.1.2 Detailing

8.6.3.2.1.2.1. Plates used in the steel core that are 2 in. thick or greater shall satisfy the minimum toughness requirements of Sec. 6.3 (AISC Seismic).

8.6.3.2.1.2.2. Splices in the steel core are not permitted.

8.6.3.2.1.2 Buckling-restraining system. The buckling-restraining system shall consist of the casing for the steel core. In stability calculations, beams, columns, and gussets connecting the core shall be considered parts of this system.

8.6.3.2.1.2.1 Restraint. The buckling-restraining system shall limit local and overall buckling of the
steel core for deformations corresponding to 1.5 times the Design Story Drift. The buckling-restraining system shall not be permitted to buckle within deformations corresponding to 1.5 times the Design Story Drift.

8.6.3.2.2 Testing. The design of braces shall be based upon results from qualifying cyclic tests in accordance with the procedures and acceptance criteria of Sec. 8.6.3.7. Qualifying test results shall consist of at least two successful cyclic tests: one is required to be a test of a brace subassemblage that includes brace connection rotational demands complying with Sec. 8.6.3.7.4 and the other shall be either a uniaxial or a subassemblage test complying with Sec. 8.6.3.7.5. Both test types are permitted to be based upon one of the following:

8.6.3.2.2.1 Types of qualifying tests

8.6.3.2.2.1.1. Tests reported in research or documented tests performed for other projects that are demonstrated to reasonably match project conditions.

8.6.3.2.2.1.2. Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, brace-end connection configurations, and matching assembly and quality control processes.

8.6.3.2.2.2 Applicability. Interpolation or extrapolation of test results for different member sizes shall be justified by rational analysis that demonstrates stress distributions and magnitudes of internal strains that are consistent with or less severe than the tested assemblies and that considers the adverse effects of larger material and variations in material properties. Extrapolation of test results shall be based upon similar combinations of steel core and buckling-restraining system sizes. Tests shall be permitted to qualify a design when the provisions of Sec. 8.6.3.7 are met.

8.6.3.2.2.3 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 Sec. 8.6.3.7.6.3 for the range of deformations corresponding to 1.5 times the Design Story Drift. 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.

8.6.3.2.2.4 Tension strength adjustment factor ($\omega$). Shall be calculated as the ratio of the maximum tension force measured from the qualification tests specified in Sec. 8.6.3.7.6.3 (for the range of deformations corresponding to 1.5 times the Design Story Drift) to the nominal yield strength of the Test Specimen. The larger value of $\omega$ from the two required qualification tests shall be used. Where the tested steel core material does not match that of the prototype, $\omega$ shall be based on coupon testing of the prototype material.

8.6.3.2.3 Quality assurance. The buckling restrained brace manufacturer shall establish a Quality Assurance Plan that complies with Sec. 16 (AISC Seismic) and the Code of Standard Practice for Steel Buildings and Bridges. The brace manufacturer shall submit the proposed Quality Assurance Plan to the Engineer of Record for review and approval. The fabrication of buckling restrained braces shall meet the requirements of the approved Quality Assurance Plan. Only buckling restrained braces meeting all applicable requirements of the approved Quality Assurance Plan will be used in construction.

8.6.3.3 Bracing connections

8.6.3.3.1 Required strength. The required strength of bracing connections in tension and compression (including beam-to-column connections if part of the bracing system) shall be $\beta\omega R_y P_{yse}$. 

Exception: The factor $R_y$ need not be applied if $P_{yse}$ is established using yield stress determined from a coupon test or mill certificate.
8.6.3.3.2 Gusset Plate. The design of connections shall include considerations of local and overall buckling.

8.6.3.4 Special requirements related to bracing configuration

8.6.3.4.1 V-type and inverted-V-type bracing. V-type and inverted-V-type braced frames shall meet the following requirements:

8.6.3.4.1.1 A beam that is intersected by braces shall be continuous between columns and shall be designed to resist the effects of load combinations stipulated by the Applicable Building Code, assuming the bracing is not present. For load combinations that include seismic, a load $Q_b$ shall be substituted for the term $E. Q_b$ is the maximum load effect applied to the beam by the braces. This vertical and horizontal load effect shall be calculated using $\beta \omega P_{yc}$ for the brace in compression and $\omega P_{yc}$ for the brace in tension. The required flexural strength for the load combinations that include seismic shall not exceed $M_y$ as defined in AISC LRFD Chapter F.

8.6.3.4.1.2 Beam stiffness. Beam deflections under the load combination $D+Q_b$ (as defined in 16.4a.1.) shall not exceed $L/240$, where $L$ is the beam span between column lines.

8.6.3.4.1.3 Deformation. For the purposes of brace design and testing, the calculated maximum deformation of braces shall be increased by including the effect of the vertical deflection of the beam under the loading defined in Sec. 8.6.3.4.1.1

8.6.3.4.1.4. Lateral support of the beam shall be provided when required for stability. The analysis shall include consideration of $Q_b$ and the axial force in the beam.

8.6.3.4.2 K-Type Bracing. K-type braced frames are not permitted for BRBF.

8.6.3.5 Columns. Columns in BRBF shall meet the following requirements:

8.6.3.5.1 Width-thickness Ratios. Compression elements of columns shall satisfy the width-thickness limitations in Table I-8-1 (AISC Seismic).

8.6.3.5.2 Splices. In addition to meeting the requirements in Sec. 8.3 (AISC Seismic), column splices in BRBF shall be designed to develop at least the nominal shear strength of the smaller connected member and 50 percent of the flexural strength of the smaller connected member. Splices shall be located in the middle one-third of the column clear height.

8.6.3.5.3 Required Strength. In addition to the requirements in Sec. 8.3 (AISC Seismic), the required strength of columns in BRBF shall be determined from load combinations as stipulated in the Applicable Building Code, except that the seismic axial forces shall be determined from the maximum brace forces that can be introduced at each level. The maximum brace tension force shall be taken as $\omega P_{yc}$. The maximum brace compression force shall be taken as $\beta \omega P_{yc}$. The required column strength need not exceed the maximum force that can be delivered by the system.

8.6.3.6 Beams. Beams in BRBF shall meet the following requirements:

8.6.3.6.1 Width-thickness ratios. Compression elements of beams shall satisfy the width-thickness limitations in Table I-8-1 (AISC Seismic).

8.6.3.6.2 Required Strength. The required strength of beams shall include the effects of dead and live loads in conjunction with axial forces corresponding to the maximum brace forces. The maximum brace tension force shall be taken as $\omega P_{yc}$. The maximum brace compression force shall be taken as $\beta \omega P_{yc}$.

8.6.3.7 Qualifying Cyclic Tests Of Buckling-Restrained Braces

8.6.3.7.1 Scope and purpose. This Appendix includes requirements for qualifying cyclic tests of individual buckling-restrained braces and buckling-restrained brace subassemblies, 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 in 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 can 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 regulatory agency.

This Appendix provides only minimum recommendations for simplified test conditions. If conditions in the actual building so warrant, additional testing shall be performed to demonstrate satisfactory and reliable performance of buckling-restrained braces during actual earthquake ground motions.

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

\( D_b \)  Deformation quantity used to control loading of test specimen (total brace end rotation for the subassemblage test specimen; total brace axial deformation for the brace test specimen) (Sec. 8.6.3.7.6).

\( D_{bm} \)  Value of deformation quantity, \( D_b \), corresponding to the design story drift (Sec. 8.6.3.7.6).

\( D_{by} \)  Value of deformation quantity, \( D_b \), at first significant yield of test specimen (Sec. 8.6.3.7.6).

8.6.3.7.3 Definitions

Brace Test Specimen: A single buckling-restrained brace element used for laboratory testing intended to model the brace in the Prototype.

Design Methodology: A set of step-by-step procedures, based on calculation or experiment, used to determine sizes, lengths, and details in the design of buckling-restrained braces and their connections.

Inelastic Deformation: The permanent or plastic portion of the axial displacement in a buckling-restrained brace, divided by the length of the yielding portion of the brace, expressed in percent.

Prototype: The brace, connections, members, steel properties, and other design, detailing, and construction features to be used in the actual building frame.

Subassemblage Test Specimen: The combination of the brace, the connections and testing apparatus that replicate as closely as practical the axial and flexural deformations of the brace in the Prototype.

Test Specimen: Brace Test Specimen or Subassemblage Test Specimen.

8.6.3.7.4 Subassemblage test specimen. The subassemblage test specimen shall satisfy the following requirements:

1. The mechanism for accommodating inelastic curvature 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.

2. The axial yield strength of the steel core of the brace in the subassemblage test specimen shall not be less than of that of the prototype as determined from mill certificate or coupon test.

3. 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.

4. The same documented design methodology shall be used for design of the subassemblage and brace and of the Prototype and for comparison of the rotational deformation demands on the subassemblage brace and on the prototype in the construction.
5. The calculated margins of safety for the prototype connection design, steel core projection stability, overall buckling and other relevant subassembly test specimen brace construction details, excluding the gusset plate, for the Prototype, shall equal or exceed those of the subassembly test specimen construction.

6. Lateral bracing of the subassembly test specimen shall replicate the lateral bracing in the prototype.

7. The brace test specimen and the prototype shall be manufactured in accordance with the same quality control and assurance processes and procedures.

Extrapolation beyond the limitations stated in this section shall be permitted subject to qualified peer review and building official approval.

8.6.3.7.5 Brace test specimen. The brace test specimen shall replicate as closely as is practical the pertinent design, detailing, construction features, and material properties of the prototype.

8.6.3.7.5.1 Design of brace test specimen. The same documented design methodology shall be used for the brace test specimen and the prototype. The design calculations shall demonstrate, at a minimum, the following requirements:

1. The calculated margin of safety for stability against overall buckling for the prototype shall equal or exceed that of the brace test specimen.
2. The calculated margins of safety for the brace test specimen and the prototype shall account for differences in material properties, including yield and ultimate stress, ultimate elongation, and toughness.

8.6.3.7.5.2 Manufacture of brace test specimen. The brace test specimen and the prototype shall be manufactured in accordance with the same quality control and assurance processes and procedures.

8.6.3.7.5.3 Similarity of brace test specimen and prototype. The brace test specimen shall meet the following requirements:

1. The cross-sectional shape and orientation of the steel core shall be the same as that of the prototype.
2. The axial yield strength of the steel core of the brace test specimen shall not vary by more than 50 percent from that of the prototype as determined from mill certificates or coupon tests.
3. The material for, and method of, separation between the steel core and the buckling restraining mechanism in the brace test specimen shall be the same as that in the prototype.

Extrapolation beyond the limitations stated in this section shall be permitted subject to qualified peer review and building official approval.

8.6.3.7.5.4 Connection details. The connection details used in the brace test specimen shall represent the Prototype connection details as closely as practical.

8.6.3.7.5.5 Materials

1. Steel core: The following requirements shall be satisfied for the steel core of the brace test specimen:
   a. The nominal yield stress of the prototype steel core shall be the same as that of the brace test specimen.
   b. The yield strength of the material of the steel core in the prototype shall not exceed 110 percent of that of the brace test specimen as determined from mill certificates or coupon tests.
   c. The specified minimum ultimate stress and strain of the prototype steel core shall meet
or exceed those of the brace test specimen.

2. Buckling-restraining mechanism: Materials used in the buckling-restraining mechanism of the brace test specimen shall be the same as those used in the prototype.

8.6.3.7.5.6 Welds. The welds on the test specimen shall replicate those on the prototype as close as practical. The following parameters shall be the same or more stringent in the prototype as in the test specimen: welding procedure specification, minimum filler metal toughness, welding positions, and inspection and nondestructive testing requirements and acceptance criteria.

8.6.3.7.5.7 Bolts. The bolted portions of the brace test specimen shall replicate the bolted portions of the prototype as closely as possible.

8.6.3.7.6 Loading history

8.6.3.7.6.1 General requirements. The test specimen shall be subjected to cyclic loads according to the requirements prescribed on Sec. 8.3.7.6.2 and 8.3.7.6.3. Additional increments of loading beyond those described in Sec. 8.3.7.6.3 are permitted. Each cycle shall include a full tension and full compression excursion to the prescribed deformation.

8.6.3.7.6.2 Test Control. The test shall be conducted by controlling the level of axial or rotational deformation, \(D_b\), imposed on the test specimen. As an alternate, the maximum rotational deformation may be applied and maintained as the protocol is followed for axial deformation.

8.6.3.7.6.3 Loading sequence. 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:

1. 6 cycles of loading at the deformation corresponding to \(D_b = D_{by}\)
2. 4 cycles of loading at the deformation corresponding to \(D_b = 0.50 D_{bm}\)
3. 4 cycles of loading at the deformation corresponding to \(D_b = 1 D_{bm}\)
4. 2 cycles of loading at the deformation corresponding to \(D_b = 1.5 D_{bm}\)
5. Additional complete cycles of loading at the deformation corresponding to \(D_b = 1 D_{bm}\) as required for the Brace Test Specimen to achieve a cumulative inelastic axial deformation of at least 140 times the yield deformation (not required for the subassemblage test specimen).

The design story drift shall not be taken as less than 0.01 times the story height for the purposes of calculating \(D_{bm}\). \(D_{bm}\) need not be taken as greater than \(5D_{by}\).

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.

8.6.3.7.7 Instrumentation. Sufficient instrumentation shall be provided on the test specimen to permit measurement or calculation of the quantities listed in Sec. 8.6.3.7.9.

8.6.3.7.8 Materials testing requirements

8.6.3.7.8.1 Tension testing requirements. Tension testing shall be conducted on samples of steel taken from the same material as that used to manufacture the steel core. Tension-test results from certified mill test reports shall be reported but are not permitted to be used in place of specimen testing for the purposes of this Section. Tension-test results shall be based upon testing that is conducted in accordance with Sec. 8.6.3.7.8.2.

8.6.3.7.8.2 Methods of tension testing. Tension testing shall be conducted in accordance with ASTM A6, ASTM A370, and ASTM E8, with the following exceptions:
1. The yield stress, \( F_y \), 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.

2. The loading rate for the tension test shall replicate, as closely as is practical, the loading rate used for the Test Specimen.

**8.6.3.7.9 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:

1. 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.

2. 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 holes, the size and grade of bolts, and all other pertinent details of the connections.

3. A listing of all other essential variables as listed in Sec. 8.6.3.7.4 or 8.6.3.7.5 as appropriate.

4. A listing or plot showing the applied load or displacement history.

5. A plot of the applied load versus the deformation \( (D_b) \). 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.

6. A chronological listing of significant test observations, including observations of yielding, slip, instability, transverse displacement along the Test Specimen and fracture of any portion of the Test Specimen and connections, as applicable.

7. The results of the material tests specified in Sec. 8.6.3.7.8.

8. 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.

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

**8.6.3.7.10 Acceptance criteria.** At least one subassemblage test shall be performed to satisfy the requirements of Sec. 8.6.3.7.4. At least one brace test shall be performed to satisfy the requirements of Sec. 8.6.3.7.5. Within the required protocol range all tests shall satisfy the following requirements:

1. The plot showing the applied load vs. displacement history shall exhibit stable, repeatable behavior with positive incremental stiffness.

2. There shall be no fracture, brace instability or brace end connection failure.

3. For brace tests, each cycle to a deformation greater than \( D_{by} \) the maximum tension and compression forces shall not be less than \( 1.0 \ P_{ysc} \).

4. For brace tests, each cycle to a deformation greater than \( D_{by} \) the ratio of the maximum compression force to the maximum tension force shall not exceed 1.3.

Other acceptance criteria may be adopted for the brace test specimen or subassemblage test specimen subject to qualified peer review and building official approval.

**8.7 RECOMMENDED PROVISIONS FOR SPECIAL STEEL PLATE WALLS**

The following shall be used in conjunction with AISC Seismic.
8.7.1 Symbols

- $t_w$: Thickness of the web
- $L_{cf}$: Clear distance between vertical boundary elements (VBEs) flanges.
- $h$: Distance between horizontal boundary elements (HBE) centerlines.
- $A_b$: The average of the cross-sectional area of a HBE bounding the panel.
- $A_c$: The average of the cross-sectional area of a VBE bounding the panel.
- $I_c$: Moment of inertia of a VBE.
- $L$: Distance between VBE centerlines.
- $\alpha$: Angle of web yielding.

8.7.2 Glossary

- **Webs**: The slender unstiffened steel plates connected to surrounding horizontal and vertical boundary elements to resist lateral loads.
- **Horizontal boundary elements (HBEs)**: Structural shapes oriented horizontally and framing the webs of special steel plate walls.
- **Vertical boundary elements (VBEs)**: Structural shapes oriented vertically and framing the webs of special steel plate walls.
- **Panel**: Each web and its surrounding elements constitute a panel.

8.7.3 Scope. Special steel plate walls (SSPWs) are expected to withstand significant inelastic deformations in the webs when subjected to the forces resulting from the motions of the design earthquake. The HBEs and VBEs adjacent to the webs shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded webs, except that plastic hinging at the ends of HBEs is permitted. SSPWs shall meet the requirements in this section.

8.7.4 Webs

8.7.4.1

The nominal strength of a panel is given by:

$$V_n = 0.42 F_y t_w L_{cf} \sin 2\alpha$$

where:

- $t_w$: the thickness of the web,
- $L_{cf}$: the clear distance between VBE flanges, and
- $\alpha$: is given by

$$\tan^4 \alpha = \frac{1 + \frac{t_w L}{2 A_c}}{1 + t_w h \left( \frac{1}{A_b} + \frac{h^3}{360 I_c L} \right)}$$

where:

- $h$: the distance between HBE centerlines,
- $A_b$: cross-sectional area of a HBE,
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\( A_c \) = cross-sectional area of a VBE,
\( I_c \) = moment of inertia of a VBE, and
\( L \) = the distance between VBE centerlines.

The panel design strength shall be \( \phi V_n \), where \( \phi \) is 0.9.

8.7.4.2 Panel aspect ratio. The ratio of panel length to height, \( L/h \), shall be greater than 0.8, but shall not exceed 2.5.

8.7.4.3 Openings in webs. Openings in webs shall be bounded on all sides by HBE’s and VBE’s extending the full width and height of the panel respectively, unless otherwise justified by testing and analysis.

8.7.4.4 Maximum slenderness ratio for plates. The maximum width-thickness ratio of plate elements shall be \( 25 \sqrt{E/F_y} \). The width shall be taken as the shortest distance between boundary elements.

8.7.5 Connections of webs to boundary elements. The required strength of Web connections to the surrounding HBE’s and VBE’s shall equal the expected yield strength, in tension, of the Web calculated at an angle \( \alpha \).

8.7.6 Horizontal and vertical boundary elements (HBEs and VBEs)

8.7.6.1 Strength of boundary elements. In addition to the requirements of Sect. 8.3 of AISC Seismic, the required strength of VBE’s shall be the based upon of the forces corresponding to the expected yield strength (in tension) of the Web calculated at an angle \( \alpha \).

The required strength of HBE’s shall be the greater of the forces corresponding to the expected yield strength (in tension) of the Web calculated at an angle \( \alpha \) or that determined from the load combinations in ASCE 7 assuming the web provides no support for gravity loads.

8.7.6.2 HBE to VBE connections. HBE to VBE connections shall be made with HBE flanges welded to VBE. HBE webs may be bolted or welded to VBE. Partial joint penetration welds are not permitted at the HBE flange weld. The connection shall have a required strength \( M_u \) of at least \( 1.1 R_y M_p \) of the HBE. The required shear strength \( V_u \) of a HBE-to-VBE connection shall be determined from the load combinations as stipulated in the ASCE 7 except that the required shear strength shall not be less than the shear corresponding to moments at each end equal to \( 1.1 R_y M_p \) together with the shear resulting from the expected tensile strength of the Webs yielding at an angle \( \alpha \).

8.7.6.3 Boundary elements compactness. The width-thickness ratios of HBEs and VBEs shall comply with the requirements in Table I-8-1(AISC Seismic)

Modify Footnotes b and c to Table I-8-1(AISC Seismic) by including SSPW to both footnotes.

8.7.6.4 Lateral Bracing. HBE’s shall be laterally braced at all intersections with VBE’s and at a spacing not to exceed \( 0.086 r_E E_s / F_y \). Both flanges of HBE’s shall be braced either directly or indirectly. The required strength of lateral bracing shall be at least 2 percent of the HBE’s flange nominal strength, \( F_y b_d f \). The required stiffness of all lateral bracing shall be determined in accordance with Equations C3-8 or C3-10 as applicable in the AISC LRFD. In these equations, \( M_o \) shall be computed as \( R_y Z F_y \).

8.7.6.5 VBE splices. VBE splices shall comply with the requirements of Sec. 8.4 (AISC Seismic).

8.7.6.6 Panel zones. The VBE panel zone next to the top and base horizontal boundary elements of the SSPW shall comply with the requirements in Sec. 9.3 (AISC Seismic).

8.7.6.7 Stiffness of vertical boundary elements. The VBE shall have moments of inertia about an axis perpendicular to the direction of the web plate, \( I_c \), not less than \( 0.00307 t_w h^4 / L \).
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