5. Steel

5.1 Scope

This chapter sets forth requirements for the Systematic Rehabilitation of steel lateral-force-resisting elements within a building. The requirements of this chapter shall apply to existing steel components of a building system, rehabilitated steel components of a building system, and new steel components added to an existing building system.

Sections 5.2 and 5.3 specify data collection procedures for obtaining material properties and performing condition assessments. Section 5.4 specifies general requirements. Sections 5.5, 5.6, 5.7, and 5.8 provide modeling procedures, component strengths, acceptance criteria, and rehabilitation measures for steel moment-resisting frame, steel braced frame, steel plate shear wall, and steel frame with infill structures. Section 5.9 provides modeling procedures, strengths, acceptance criteria, and rehabilitation measures for diaphragms used in steel structures. Section 5.11 specifies requirements for steel piles. Section 5.11 specifies requirements for components of cast or wrought iron.

C5.1 Scope

Techniques for repair of earthquake-damaged steel components are not included in this standard. The design professional is referred to SAC publications *FEMA 351* and *FEMA 353* for information on design, evaluation, and repair of damaged steel moment resisting frame structures.

5.2 Material Properties Based on Historical Information

Available construction documents and as-built information shall be obtained as specified in Section 2.2. Use of material properties based on historical information as default values shall be permitted as specified in Section 5.3.2.5.

C5.2 Material Properties Based on Historical Information

Steel components of buildings include columns, beams, braces, connections, link beams, and diaphragms. Columns, beams, and braces may be built up with plates, angles, and/or channels connected together with rivets, bolts, or welds. The material used in older construction is likely to be mild steel with a specified yield strength between 30 ksi and 36 ksi. Cast iron was often used for columns in much older construction (before 1900). Cast iron was gradually replaced by wrought iron and then by steel. The connectors in older construction were usually mild steel rivets or bolts. These were later replaced by high-strength bolts and welds. The seismic performance of these components will depend heavily on the condition of the in-place material. A more detailed historical perspective is given in Section C5.2 of *FEMA 274*.

Great care should be exercised in selecting the appropriate rehabilitation approaches and techniques for application to historic buildings in order to preserve their unique characteristics.

5.3 Material Properties and Condition Assessment

5.3.1 General

Mechanical properties for steel materials and components shall be based on available construction documents and as-built conditions for the particular structure. Where such information fails to provide adequate information to quantify material properties or document the condition of the structure, such information shall be supplemented by material tests and assessments of existing conditions as required in Section 2.2.6.

Material properties of existing steel components shall be determined in accordance with Section 5.3.2. A condition assessment shall be conducted in accordance with Section 5.3.3. The extent of materials testing and condition assessment performed shall be used to determine the knowledge factor as specified in Section 5.3.4.

Use of default material properties shall be permitted in accordance with Section 5.3.2.5.

C5.3.1 General

The extent of in-place materials testing and condition assessment that must be accomplished is related to availability and accuracy of construction and as-built records, the quality of materials used and construction performed, and the physical condition of the structure. Data such as the properties and grades of material used in component and connection fabrication may be effectively used to reduce the amount of in-place testing required. The design professional is encouraged to research and acquire all available records from original construction.

5.3.2 Properties of In-Place Materials and Components

5.3.2.1 Material Properties

5.3.2.1.1 General

The following component and connection material properties shall be obtained for the as-built structure:

1. Yield and tensile strength of the base material.
2. Yield and tensile strength of the connection material.
3. Carbon equivalent of the base and connection material.

Structural steel components constructed after 1900 shall be classified based on ASTM specification and material grade and, if applicable, shape group in accordance with Table 5-2. Lower bound material properties shall be taken in accordance with Table 5-2 for material conforming to the specifications listed therein. For material grades not listed in Table 5-2, lower bound material properties shall be taken as nominal or specified properties, or shall be based on tests when the material grade or specified value is not known.

When materials testing is required by Section 2.2.6, test methods to determine ASTM designation and material grade or to quantify material properties shall be as specified in Section 5.3.2.3.

The minimum number of tests shall comply with the requirements of Section 5.3.2.4.

The carbon equivalent of the existing components shall be determined unless it is confirmed that the existing material provides suitable characteristics of weldability, by virtue of their conformance with a weldable material specification, or welding to existing components will not be performed as part of the rehabilitation. The welding procedures shall be determined based on the chemistry of the base material and filler material as specified in Section 8 of *AWS D1.1*. Material conforming to *ASTM A36/A36M-00*, *ASTM A242/A242M-00*, *ASTM A307-00*, *ASTM A572/572M-00*, *ASTM A913/A913M-00*, and *ASTM A972/A972M-00* shall be deemed to be weldable.

5.3.2.1.2 Nominal Properties

Nominal material properties, or properties specified in construction documents, shall be taken as lower bound material properties. Corresponding expected material properties shall be calculated by multiplying lower-bound values by an appropriate factor taken from Table 5-3 to translate from lower bound to expected values.

Where construction documents indicate the ultimate tensile strength of weld metal, the lower-bound strength of welds shall be taken as indicated in *AWS D1.1*. For construction pre-dating 1970, use of a nominal ultimate tensile strength of 60 ksi shall be permitted.

C5.3.2.1 Material Properties

Mechanical properties of component and connection material dictate the structural behavior of the component under load. Mechanical properties of greatest interest include the expected and lower bound estimates of yield (F_{ye}) and tensile (F_{te}) strengths of base and connection material, modulus of elasticity, ductility, toughness, elongational characteristics, and weldability.

Expected material properties should be used for deformation-controlled actions. Lower bound material properties should be used for force-controlled actions in lieu of nominal strengths specified in the AISC (1993) *LRFD Specifications*.

5.3.2.2 Component Properties

The following properties of components and their connections shall be obtained for the structure:

1. Size and thickness of connected materials, including cover plates, bracing, and stiffeners.

2. Cross-sectional area, section moduli, moments of inertia, and torsional properties of components at critical sections.

3. As-built configuration of intermediate, splice, and end connections.

4. Current physical condition of base metal and connector materials, including presence of deformation and extent of deterioration.

Review of available construction documents shall be performed to identify primary vertical- and lateral-load-carrying elements and systems, critical components and connections, and any modifications to components or overall configuration of the structure.

In the absence of deterioration, use of the nominal cross-sectional dimensions of components published by the American Institute of Steel Construction (AISC), the American Iron and Steel Institute (AISI), and other approved trade associations shall be permitted.

5.3.2.3 Test Methods to Quantify Properties

Laboratory testing of samples to determine in-place mechanical properties of materials and components shall be performed in compliance with consensus standards published by ASTM, the American National Standards Institute (ANSI), and other approved organizations.

The extent of in-place materials testing required to determine material properties shall be based on the availability of original and updated construction documents, original quality of construction, accessibility, and condition of materials.

The determination of material properties shall be accomplished through removal of samples and laboratory testing. Sampling shall take place in regions of reduced stress. If a connector such as a bolt or rivet is removed for testing, a comparable bolt shall be reinstalled at the time of sampling. Destructive removal of a welded connection sample shall be accompanied by repair of the connection.

Expected material properties shall be based on mean test values. Lower bound material properties shall be based on mean test values minus one standard deviation except that where the material is positively identified as conforming to a defined standard material specification, lower-bound properties need not be taken less than the nominal properties for that specification.

C5.3.2.3 Test Methods to Quantify Properties

FEMA 274 provides information and references for several test methods.

Sampling should take place in regions of reduced stress—such as flange tips at beam ends and external plate edges—to minimize the effects of reduced area.

Of greatest interest to steel building system performance are the expected yield and tensile strength of the installed materials. Notch toughness of structural steel and weld material is also important for connections that undergo cyclic loadings and deformations during earthquakes. Chemical and metallurgical properties can provide information on properties such as compatibility of welds with parent metal and potential lamellar tearing due to through-thickness stresses. Virtually all steel component elastic and inelastic limit states are related to yield and tensile strengths. Past research and accumulation of data by industry groups have resulted in published material mechanical properties for most primary metals and their date of fabrication. Section 5.3.2.5 provides default properties. This information may be used, together with tests from recovered samples, to rapidly establish expected strength properties for use in component strength and deformation analyses.

Review of other properties derived from laboratory tests—such as hardness, impact, fracture, and fatigue—is generally not needed for steel component capacity determination, but may be required for archaic materials and connection evaluation. These properties may not be needed in the analysis phase if significant rehabilitative measures are already known to be required.

To quantify material properties and analyze the performance of welded moment connections, more extensive sampling and testing may be necessary. This testing may include base and weld material chemical and metallurgical evaluation, expected strength determination, hardness, and charpy V-notch testing of the heat-affected zone and neighboring base metal, and other tests depending on connection configuration.

Recommendations given in *FEMA 351* may also be followed to select welding procedures for welding of rehabilitative measures to existing components.

5.3.2.4 Minimum Number of Tests

Materials testing is not required if material properties are available from original construction documents that include material test records or material test reports. If such properties differ from default material properties given in Tables 5-1 and 5-2, material properties for rehabilitation shall be selected such that the largest demands on components and connections are generated.

5.3.2.4.1 Usual Testing

The minimum number of tests to determine the yield and tensile strengths of steel materials for usual data collection shall be based on the following criteria:

1. If design drawings are incomplete or not available, at least one strength coupon from each steel component type shall be removed for testing, and one weld metal sample shall be obtained for testing.

2. If design drawings containing ASTM specification and material grade information are available, use of Table 5-2 to determine material properties shall be permitted without additional testing.

3. If design drawings containing material property information are available but the material properties are not listed in Table 5-2, use of nominal or specified material properties shall be permitted without additional testing.

5.3.2.4.2 Comprehensive Testing

The minimum number of tests to determine the yield and tensile strengths of steel materials for comprehensive data collection shall be based on the following criteria:

1. If original construction documents defining material properties are inconclusive or do not exist, but the date of construction is known and the material used is confirmed to be carbon steel, at least three strength coupons and three bolts and rivets shall be randomly removed from each component type.

2. If no knowledge of the structural system and materials used exists, at least two tensile strength coupons and two bolts and rivets shall be removed from each component type for every four floors. If it is determined from testing that more than one material grade exists, additional sampling and testing shall be performed until the extent of each grade in component fabrication has been established.

3. In the absence of construction records defining welding filler metals and processes used, at least one weld metal sample for each construction type shall be obtained for laboratory testing. The sample shall consist of both local base and weld metal to determine composite strength of the connection.

4. For archaic materials, at least three strength coupons shall be extracted for each component type for every four floors of construction. If initial tests provide material properties that are consistent with properties given in Table 5-1, tests shall be required for every six floors of construction only. If these tests provide material properties that are nonuniform, additional tests shall be performed until the extent of different materials is established.

For other material properties, a minimum of three tests shall be conducted.

The results of any material testing performed shall be compared to the default values in Tables 5-1 and 5-2 for the particular era of building construction. The amount of testing shall be doubled if the expected and lower-bound yield and tensile strengths determined from testing are lower than the default values.

C5.3.2.4 Minimum Number of Tests

In order to quantify expected strength and other properties accurately, a minimum number of tests may be required to be conducted on representative components.

Material properties of structural steel vary much less than those of other construction materials. In fact, the expected yield and tensile stresses are usually considerably higher than the nominal specified values. As a result, testing for material properties of structural steel may not be required. The properties of wrought iron are more variable than those of steel. The strength of cast iron components cannot be determined from small sample tests, since component behavior is usually governed by inclusions and other imperfections.

If ductility and toughness are required at or near the weld, the design professional may conservatively assume that no ductility is available, in lieu of testing. In this case the joint would have to be modified if inelastic demands are anticipated and the possibility of fractures cannot be tolerated. Special requirements for welded moment frames are given in *FEMA 351*.

If a higher degree of confidence in results is desired, either the sample size shall be determined using ASTM E22 criteria, or the prior knowledge of material grades from Section 5.3.2.5 should be used in conjunction with approved statistical procedures.

Design professionals may consider using Bayesian statistics and other statistical procedures contained in *FEMA 274* to gain greater confidence in the test results obtained from the sample sizes specified in this section.

5.3.2.5 Default Properties

The default lower bound material properties for steel components shall be as specified in Tables 5-1 and 5-2. Default expected strength material properties shall be determined by multiplying lower-bound values by an appropriate factor taken from Table 5-3.

Use of default material properties to determine component and connection strengths shall be permitted in conjunction with the linear analysis procedures of Chapter 3.

Table 5-1 Default Lower-Bound Material Strengths for Archaic Materials[1]

Properties based on tables of allowable loads as published in mill catalogs

Year	Material	Lower-Bound Yield Strength, ksi	Lower-Bound Tensile Strength, ksi
pre-1900	cast iron	18	—
pre-1900	steel	24	36

1. Modified from unit stress values in AISC “Iron and Steel Beams from 1873 to 1952.”

Table 5-2 Default Lower-Bound Material Strengths[1]

Properties based on ASTM and AISC Structural Steel Specification Stresses

Date	Specification	Remarks	Tensile Strength[2], ksi	Yield Strength[2], ksi
1900	ASTM, A9	Rivet Steel	50	30
	Buildings	Medium Steel	60	35
1901–1908	ASTM, A9	Rivet Steel	50	25
	Buildings	Medium Steel	60	30
1909–1923	ASTM, A9	Structural Steel	55	28
	Buildings	Rivet Steel	46	23
1924–1931	ASTM, A7	Structural Steel	55	30
		Rivet Steel	46	25
	ASTM, A9	Structural Steel	55	30
		Rivet Steel	46	25
1932	ASTM, A140-32T issued as a tentative revision to ASTM, A9 (Buildings)	Plates, Shapes, Bars	60	33
		Eyebar flats unannealed	67	36
1933	ASTM, A140-32T discontinued and ASTM, A9 (Buildings) revised Oct. 30, 1933	Structural Steel	55	30
	ASTM, A9 tentatively revised to ASTM, A9-33T (Buildings)	Structural Steel	60	33
	ASTM, A141-32T adopted as a standard	Rivet Steel	52	28
1934 on	ASTM, A9	Structural Steel	60	33
	ASTM, A141	Rivet Steel	52	28
1961 – 1990	ASTM, A36/A36M-00	Structural Steel		
	Group 1		62	44
	Group 2		59	41
	Group 3		60	39
	Group 4		62	37
	Group 5		70	41
1961 on	ASTM, A572, Grade 50	Structural Steel		
	Group 1		65	50
	Group 2		66	50
	Group 3		68	51
	Group 4		72	50
	Group 5		77	50
1990 on	A36/A36M-00 & Dual Grade	Structural Steel		
	Group 1		66	49
	Group 2		67	50
	Group 3		70	52
	Group 4		70	49

1. Lower-bound values for material prior to 1960 are based on minimum specified values. Lower-bound values for material after 1960 are mean minus one standard deviation values from statistical data.
2. The indicated values are representative of material extracted from the flanges of wide flange shapes.

Table 5-3 ***Factors to Translate Lower-Bound Steel Properties to Expected-Strength Steel Properties***

Property	Year	Specification	Factor
Tensile Strength	Prior to 1961		1.10
Yield Strength	Prior to 1961		1.10
Tensile Strength	1961-1990	ASTM A36/A36M-00	1.10
	1961-present	ASTM A572/A572M-89, Group 1	1.10
		ASTM A572/A572M-89, Group 2	1.10
		ASTM A572/A572M-89, Group 3	1.05
		ASTM A572/A572M-89, Group 4	1.05
		ASTM A572/A572M-89, Group 5	1.05
	1990-present	ASTM A36/A36M-00 & Dual Grade, Group 1	1.05
		ASTM A36/A36M-00 & Dual Grade, Group 2	1.05
		ASTM A36/A36M-00 & Dual Grade, Group 3	1.05
		ASTM A36/A36M-00 & Dual Grade, Group 4	1.05
Yield Strength	1961-1990	ASTM A36/A36M-00	1.10
	1961-present	ASTM A572/A572M-89, Group 1	1.10
		ASTM A572/A572M-89, Group 2	1.10
		ASTM A572/A572M-89, Group 3	1.05
		ASTM A572/A572M-89, Group 4	1.10
		ASTM A572/A572M-89, Group 5	1.05
	1990-present	ASTM A36/A36M-00, Rolled Shapes	1.50
		ASTM A36/A36M-00, Plates	1.10
		Dual Grade, Group 1	1.05
		Dual Grade, Group 2	1.10
		Dual Grade, Group 3	1.05
		Dual Grade, Group 4	1.05
Tensile Strength	All	Not Listed [1]	1.10
Yield Strength	All	Not Listed [1]	1.10

1. For materials not conforming to one of the listed specifications.

5.3.3 Condition Assessment

5.3.3.1 General

A condition assessment of the existing building and site shall be performed as specified in this section. A condition assessment shall include the following:

1. Examination of components that will receive earthquake-induced forces or deformations to identify the presence of degradation due to environmental or loading effects.
2. Verification of the presence and configuration of structural elements and components and their connections, and the continuity of load paths between components, elements, and systems.
3. Identification of other conditions including the presence of nonstructural components that influence building performance and impose limitations on rehabilitation.
4. Characterization of soil and foundation conditions as specified in Section 2.2.3.
5. Identification of adjacent buildings as specified in Section 2.2.4.

C5.3.3.1 General

The physical condition of existing components and elements and their connections must be examined for degradation. Degradation may include environmental effects (e.g., corrosion, fire damage, chemical attack) or past/current loading effects (e.g., overload, damage from past earthquakes, fatigue, fracture). The condition assessment should also examine for configurational problems observed in recent earthquakes, including effects of discontinuous components, improper welding, and poor fit-up.

Component orientation, plumbness, and physical dimensions should be confirmed during an assessment. Connections in steel components, elements, and systems require special consideration and evaluation. The load path for the system must be determined, and each connection in the load path(s) must be evaluated. This includes diaphragm-to-component and component-to-component connections. *FEMA 351* provides recommendations for inspection of welded steel moment frames.

The condition assessment also affords an opportunity to review other conditions that may influence steel elements and systems and overall building performance. Of particular importance is the identification of other elements and components that may contribute to or impair the performance of the steel system in question, including infills, neighboring buildings, and equipment attachments. Limitations posed by existing coverings, wall and ceiling space, infills, and other conditions shall also be defined such that prudent rehabilitation measures may be planned.

5.3.3.2 Scope and Procedures

The condition assessment shall include visual inspection of accessible structural elements and components involved in lateral-load resistance to verify information shown on available documents.

If coverings or other obstructions exist, either indirect visual inspection through use of drilled holes and a fiberscope shall be used, or direct visual inspection shall be performed by local removal of covering materials. When required by Section 2.2.6, visual and comprehensive condition assessments shall include the following:

1. If detailed design drawings exist, at least one connection of each connection type shall be exposed. If no deviations from the drawings exist, the sample shall be considered representative. If deviations from the existing drawings exist, then removal of additional coverings from connections of that type shall be done until the extent of deviations is determined.
2. In the absence of construction drawings, at least three primary connections of each connection type shall be exposed. If no deviations within a connection group are observed, the sample shall be considered representative. If deviations within a connection group are observed, then additional connections shall be exposed until the extent of deviations is determined.

C5.3.3.2 Scope and Procedures

For steel elements encased in concrete, it may be more cost effective to provide an entirely new lateral-load-resisting system than undertaking a visual inspection by removal of concrete encasement and repair.

Physical condition of components and connectors may also dictate the use of certain destructive and nondestructive test methods. If steel elements are covered by well-bonded fireproofing materials or encased in durable concrete, it is likely that their condition will be suitable. However, local removal of these materials at connections should be performed as part of the assessment. The scope of this removal effort is dictated by the component and element design. For example, in a braced frame, exposure of several key connections may suffice if the physical condition is acceptable and the configuration matches the design drawings. However, for moment frames, it may be necessary to expose more connection points because of varying designs and the critical nature of the connections. See *FEMA 351* for inspection of welded moment frames.

5.3.3.3 Basis for the Mathematical Building Model

The results of the condition assessment shall be used to create a mathematical building model.

If no damage, alteration, or degradation is observed in the condition assessment, component section properties shall be taken from design drawings. If some sectional material loss or deterioration has occurred, the loss shall be quantified by direct measurement and section properties shall be reduced accordingly using principles of structural mechanics.

5.3.4 Knowledge Factor

A knowledge factor (κ) for computation of steel component capacities and permissible deformations shall be selected in accordance with Section 2.2.6.4 with the following additional requirements specific to steel components.

A knowledge factor of 0.75 shall be used if the components and their connectors are composed of cast or wrought iron.

5.4 General Assumptions and Requirements

5.4.1 Stiffness

Component stiffnesses shall be calculated in accordance with Sections 5.5 through 5.11.

5.4.2 Design Strengths and Acceptance Criteria

5.4.2.1 General

Classification of steel component actions as deformation- or force-controlled, and calculation of design strengths, shall be as specified in Sections 5.5 through 5.10.

5.4.2.2 Deformation-Controlled Actions

Design strengths for deformation-controlled actions, Q_{CE}, shall be taken as expected strengths obtained experimentally or calculated using accepted principles of mechanics. Expected strength shall be defined as the mean maximum resistance expected over the range of deformations to which the component is likely to be subjected. When calculations are used to determine mean expected strength, expected material properties (including strain hardening) shall be used. Unless other procedures are specified in this standard, procedures contained in AISC (1993) *LRFD Specifications* to calculate design strength shall be permitted, except that the strength reduction factor, ϕ, shall be taken as unity. Deformation capacities for acceptance of deformation-controlled actions shall be as specified in Sections 5.5 through 5.11.

5.4.2.3 Force-Controlled Actions

Design strengths for force-controlled actions, Q_{CL}, shall be taken as lower-bound strengths obtained experimentally or calculated using established principles of mechanics. Lower-bound strength shall be defined as mean strength minus one standard deviation. When calculations are used to determine lower-bound strength, lower bound material properties shall be used. Unless other procedures are specified in this standard, procedures contained in AISC (1993) *LRFD Specifications* to calculate design strength shall be permitted, except that the strength reduction factor, ϕ, shall be taken as unity. Where alternative definitions of design strength are used, they shall be justified by experimental evidence.

5.4.3 Rehabilitation Measures

Upon determining that steel elements in an existing building are deficient for the selected Rehabilitation Objective, these elements shall be rehabilitated or replaced so they are no longer deficient. If replacement of the element is selected, the new element shall be designed in accordance with an approved building code.

5.5 Steel Moment Frames

5.5.1 General

The behavior of steel moment-resisting frames is generally dependent on the connection configuration and detailing. Table 5-4 identifies the various connection types for which acceptance criteria are provided. Modeling procedures, acceptance criteria, and rehabilitation measures for Fully Restrained (FR) Moment Frames and Partially Restrained (PR) Moment Frames shall be as defined in Sections 5.5.2 and 5.5.3, respectively.

C5.5.1 General

Steel moment frames are those frames that develop their seismic resistance through bending of steel beams and columns, and moment-resisting beam-column connections. A moment-resisting beam-column connection is one that is designed to develop moment resistance at the joint between the beam and the column and also designed to develop the shear resistance at the panel zone of the column. Beams and columns consist of either hot-rolled steel sections or cold-formed steel sections or built-up members from hot-rolled or cold-formed plates and sections. Built-up members are assembled by riveting, bolting, or welding. The components are either bare steel or steel with a nonstructural coating for protection from fire or corrosion or both, or steel with either concrete or masonry encasement.

Following the 1994 Northridge Earthquake, the SAC Joint Venture undertook a major program to address the issue of the seismic performance of moment-resisting steel frame structures. This program produced several documents which provide recommended criteria for the evaluation and upgrade of this building type. However, the design professional should be cautioned that there are some differences in the methodologies and specifics of this standard and the SAC procedures. While both methodologies utilize similar analysis procedures, there are some variations in the factors used to compute the pseudo lateral load in the LSP and NSP. When using the acceptance criteria of this section, the design professional should follow the procedures set forth in Chapter 3 of this standard without modification. The procedures in this standard and the SAC procedures are judged to result in comparable levels of drift demand.

Connections between the members shall be classified as fully restrained (FR) or partially restrained (PR), based on the strength and stiffness of the connection assembly. The connection types and definitions contained in Table 5-4, as well as the acceptance criteria for these connections, has been adopted from the referenced SAC documents, *FEMA 350, 351, 355D*, and *355F*. The number of connections identified is based on research that has shown behavior to be highly dependent on connection detailing. The design professional should refer to those guidelines for more detailed descriptions of these connections as well as a methodology for determining acceptance criteria for other connection types not included in this standard.

FEMA 351 provides an alternate methodology for determining column demands that has not been adopted into this standard.

Table 5-4 Steel Moment Frame Connection Types

Connection	Description[1, 2]	Type
Welded Unreinforced Flange (WUF)	Full-penetration welds between beam and columns, flanges, bolted or welded web, designed prior to code changes following the Northridge earthquake	FR
Bottom Haunch in WUF w/Slab	Welded bottom haunch added to existing WUF connection with composite slab[3]	FR
Bottom Haunch in WUF w/o Slab	Welded bottom haunch added to existing WUF connection without composite slab[3]	FR
Welded Cover Plate in WUF	Welded cover plates added to existing WUF connection[3]	FR
Improved WUF-Bolted Web	Full-penetration welds between beam and column flanges, bolted web[4]	FR
Improved WUF-Welded Web	Full-penetration welds between beam and column flanges, welded web[4]	FR
Free Flange	Web is coped at ends of beam to separate flanges, welded web tab resists shear and bending moment due to eccentricity due to coped web[4]	FR
Welded Flange Plates	Flange plate with full-penetration weld at column and fillet welded to beam flange[4]	FR
Reduced Beam Section	Connection in which net area of beam flange is reduced to force plastic hinging away from column face[4]	FR
Welded Bottom Haunch	Haunched connection at bottom flange only[4]	FR
Welded Top and Bottom Haunches	Haunched connection at top and bottom flanges[4]	FR
Welded Cover-Plated Flanges	Beam flange and cover-plate are welded to column flange[4]	FR
Top and Bottom Clip Angles	Clip angle bolted or riveted to beam flange and column flange	PR
Double Split Tee	Split tees bolted or riveted to beam flange and column flange	PR
Composite Top and Clip Angle Bottom	Clip angle bolted or riveted to column flange and beam bottom flange with composite slab	PR
Bolted Flange Plates	Flange plate with full-penetration weld at column and bolted to beam flange[4]	PR[5]
Bolted End Plate	Stiffened or unstiffened end plate welded to beam and bolted to column flange	PR[5]
Shear Connection w/ Slab	Simple connection with shear tab, composite slab	PR
Shear Connection w/o Slab	Simple connection with shear tab, no composite slab	PR

1. Where not indicated otherwise, definition applies to connections with bolted or welded web.
2. Where not indicated otherwise, definition applies to connections with or without composite slab.
3. Full-penetration welds between haunch or cover plate to column flange conform to the requirements of the AISC (1997) *Seismic Provisions.*
4. Full-penetration welds conform to the requirements of the AISC (1997) *Seismic Provisions*.
5. For purposes of modeling, the connection may be considered FR if it meets the strength and stiffness requirements of Section 5.5.2.1.

5.5.2 Fully Restrained Moment Frames

5.5.2.1 General

Fully restrained (FR) moment frames shall be those moment frames with connections identified as FR in Table 5-4.

Moment frames with connections not included in Table 5-4 shall be defined as FR if the joint deformations (not including panel zone deformation) do not contribute more than 10% to the total lateral deflection of the frame, and the connection is at least as strong as the weaker of the two members being joined. If either of these conditions is not satisfied, the frame shall be characterized as partially restrained.

Fully restrained moment frames encompass both Special Moment Frames and Ordinary Moment Frames, defined in AISC (1997) *Seismic Provisions*. These terms are not used in this standard, but the requirements for these systems and for general or seismic design of steel components specified in AISC (1993) *LRFD Specifications* or *ASCE 7* shall be followed for new elements designed as part of the seismic rehabilitation, unless superseded by provisions in this standard.

C5.5.2.1 General

FEMA 351 identifies two types of connections—Type 1 (ductile) and Type 2 (brittle). These definitions are not used in this standard since the distinction is reflected in the acceptance criteria for the connections.

The most common beam-to-column connection used in steel FR moment frames since the late 1950s required the beam flange to be welded to the column flange using complete joint penetration groove welds. Many of these connections have fractured during recent earthquakes. The design professional is referred to *FEMA 274* and to *FEMA 351*.

5.5.2.2 Stiffness

5.5.2.2.1 Linear Static and Dynamic Procedures

The stiffness of steel members (columns and beams) and connections (joints and panel zones used with the linear procedures of Chapter 3) shall be based on principles of structural mechanics and as specified in the AISC (1993) *LRFD Specifications* unless superseded by provisions of this section.

1. Axial Area and Shear Area: [For elements fully encased in concrete, calculation of the stiffness using full composite action shall be permitted if confining reinforcement is provided to allow the concrete to remain in place during an earthquake. Concrete enclosed on at least three sides, or over 75% of its perimeter, by elements of the structural steel member shall be permitted to be considered adequately confined to provide composite action.

2. Moment of Inertia: [For components encased in concrete, the stiffness shall include composite action, but the width of the composite section shall be taken as equal to the width of the flanges of the steel member and shall not include parts of the adjoining floor slab, unless there is an identifiable shear transfer mechanism between the concrete and the steel which is shown to meet the applicable acceptance criteria for the selected performance level.

3. Panel Zone Modeling: [Inclusion of panel zones flexibility shall be permitted in a frame analysis by adding a panel zone element to the mathematical model. Alternatively, adjustment of the beam flexural stiffness to account for panel zone flexibility shall be permitted. Where the expected shear strength of panel zones exceeds the flexural strength of the beams at a beam-column connection, and the stiffness of the panel zone is at least 10 times larger than the flexural stiffness of the beam, direct modeling of the panel zone shall not be required. In such cases, rigid offsets from the center of the column shall be permitted to represent the effective span of the beam. Use of center line analysis shall be permitted for other cases.

4. Joint Modeling: Modeling of connection stiffness for FR moment frames shall not be required except for joints that are intentionally reinforced to force formation of plastic hinges within the beam span, remote from the column face. For such joints, rigid elements shall be used between the column and the beam to represent the effective span of the beam.

5. Connections: Requirements of this section shall apply to connections identified as FR in Table 5-4 and those meeting the requirements of Section 5.5.2.1.

5.5.2.2.2 Nonlinear Static Procedure

If the Nonlinear Static Procedure of Chapter 3 is used, the following criteria shall apply:

1. Elastic component properties shall be modeled as specified in Section 5.5.2.2.1.
2. Plastification shall be represented by nonlinear moment-curvature and interaction relationships for beams and beam-columns derived from experiment or analysis.
3. Linear or nonlinear behavior of panel zones shall be included in the mathematical model except as indicated in Section 5.5.2.2.1, Item 3.

In lieu of relationships derived from experiment or analysis, the generalized load-deformation curve shown in Figure 5-1, with parameters a, b, c, as defined in Tables 5-6 and 5-7, shall be used for components of steel moment frames. Modification of this curve shall be permitted to account for strain-hardening of components as follows: (a) a strain-hardening slope of 3% of the elastic slope shall be permitted for beams and columns unless a greater strain-hardening slope is justified by test data; and (b) where panel zone yielding occurs, a strain-hardening slope of 6% shall be used for the panel zone unless a greater strain-hardening slope is justified by test data.

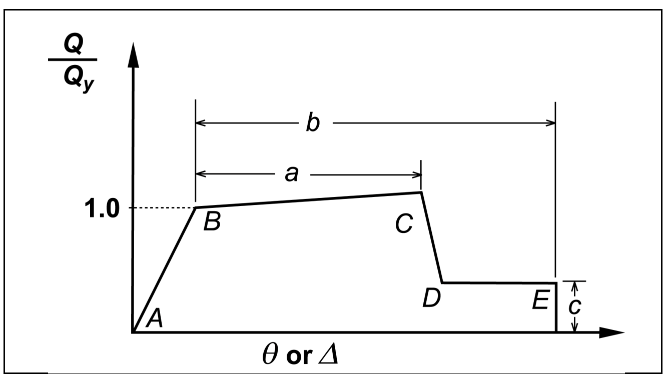

Figure 5-1 Generalized Force-Deformation Relation for Steel Elements or Components

The parameters Q and Q_{CE} in Figure 5-1 are generalized component load and generalized component expected strength, respectively. For beams and columns, θ is the total elastic and plastic rotation of the beam or column, θ_y is the rotation at yield, Δ is total elastic and plastic displacement, and Δ_y is yield displacement. For panel zones, θ_y is the angular shear deformation in radians. Figure 5-2 defines chord rotation for beams. The chord rotation shall be calculated either by adding the yield rotation, θ_y, to the plastic rotation or taken to be equal to the story drift. Use of Equations (5-1) and (5-2) to calculate the yield rotation, θ_y, where the point of contraflexure is anticipated to occur at the mid-length of the beam or column, respectively, shall be permitted.

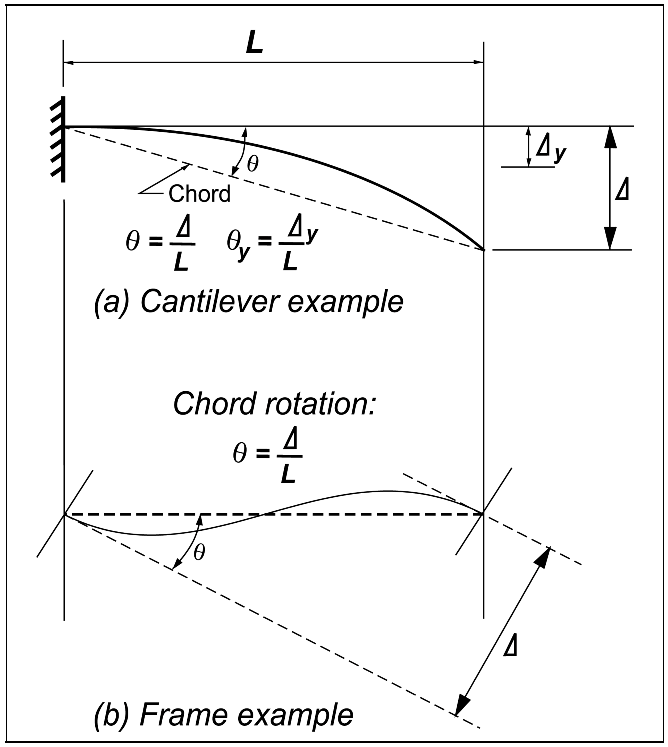

Figure 5-2 Definition of Chord Rotation

$$\text{Beams:} \quad \theta_y = \frac{ZF_{ye}l_b}{6EI_b} \tag{5-1}$$

$$\text{Columns:} \quad \theta_y = \frac{ZF_{ye}l_c}{6EI_c}\left(1 - \frac{P}{P_{ye}}\right) \tag{5-2}$$

Q and Q_{CE} are the generalized component load and generalized component expected strength, respectively. For flexural actions of beams and columns, Q_{CE} refers to the plastic moment capacity, which shall be calculated using Equations (5-3) and (5-4):

Beams:

$$Q_{CE} = M_{CE} = ZF_{ye} \tag{5-3}$$

Columns:

$$Q_{CE} = M_{CE} = 1.18ZF_{ye}\left(1 - \frac{P}{P_{ye}}\right) \leq ZF_{ye} \tag{5-4}$$

For panel zones, Q_{CE} refers to the plastic shear capacity of the panel zone, which shall be calculated using Equation (5-5):

Panel Zones:

$$Q_{CE} = V_{CE} = 0.55F_{ye}d_ct_p \tag{5-5}$$

where:

d_c = Column depth

E = Modulus of elasticity

F_{ye} = Expected yield strength of the material

I = Moment of inertia

l_b = Beam length

l_c = Column length

M_{CE} = Expected flexural strength

P = Axial force in the member at the target displacement for nonlinear static analyses, or at the instant of computation for nonlinear dynamic analyses. For linear analyses, P shall be taken as Q_{UF}, calculated in accordance with Section 3.4.2.1.2

P_{ye} = Expected axial yield force of the member = A_gF_{ye}

Q = Generalized component load

Q_{CE} = Generalized component expected strength

t_p = Total thickness of panel zone including doubler plates

θ = Chord rotation

θ_y = Yield rotation

V_{CE} = Expected shear strength

Z = Plastic section modulus

C5.5.2.2.2 Nonlinear Static Procedure

Strain hardening should be considered for all components. The design professional is directed to *FEMA 355D* for information concerning nonlinear behavior of various tested connection configurations.

5.5.2.2.3 Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component shall be determined experimentally.

C5.5.2.2.3 Nonlinear Dynamic Procedure

The design professional is directed to *FEMA 355D* for information concerning nonlinear behavior of various tested connection configurations.

5.5.2.3 Strength

5.5.2.3.1 General

Component strengths shall be computed in accordance with the general requirements of Section 5.4.2 and the specific requirements of this section.

5.5.2.3.2 Linear Static and Dynamic Procedures

1. Beams: The strength of elements of structural steel under flexural actions with negligible axial load present shall be calculated in accordance with this section. These actions shall be considered deformation-controlled.

The expected flexural strength, Q_{CE}, of beam components shall be determined using equations for design strength, M_p, given in AISC (1997) *Seismic Provisions*, except that ϕ shall be taken as 1.0 and F_{ye} shall be substituted for F_y. The component expected strength, Q_{CE}, of beams and other flexural deformation-controlled members shall be the lowest value obtained for the limit states of yielding, lateral-torsional buckling, local flange buckling, or shear yielding of the web.

For fully concrete-encased beams where confining reinforcement is provided to allow the concrete to remain in place during the earthquake, the values of $b_f = 0$ and $L_p = 0$ shall be permitted to be used. For bare beams bent about their major axes and symmetric about both axes, satisfying the requirements of compact sections, and $L_b < L_p$, Q_{CE} shall be computed in accordance with Equation (5-6):

$$Q_{CE} = M_{CE} = M_{pCE} = ZF_{ye} \tag{5-6}$$

where:

b_f = Width of the compression flange

L_b = Length of beam

L_p = Limiting lateral unbraced length for full plastic bending capacity for uniform bending from AISC (1993) *LRFD Specifications*

M_{pCE} = Expected plastic moment capacity

F_{ye} = Expected yield strength determined in accordance with Section 5.3.2.3

The limit states of local and lateral torsional buckling shall not be considered for components either subjected to bending about their minor axes or fully encased in concrete where confining reinforcement is provided to allow the concrete to remain in place during an earthquake.

If the beam strength is governed by the shear strength of the unstiffened web and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_y}}$, then V_{CE} shall be calculated in accordance with Equation (5-7):

$$Q_{CE} = V_{CE} = 0.6F_{ye}A_w \tag{5-7}$$

where:

V_{CE} = Expected shear strength

A_w = Nominal area of the web = $d_b t_w$

t_w = Web thickness

h = Distance from inside of compression flange to inside of tension flange

F_y = Yield strength; must be in ksi when used in Equation (5-7)

If $\frac{h}{t_w} > \frac{418}{\sqrt{F_y}}$, then the value of V_{CE} shall be calculated from AISC (1997) *Seismic Provisions*.

2. Columns: This section shall be used to evaluate flexural and axial strengths of structural steel elements with non-negligible axial load present. These actions shall be considered force-controlled.

The lower-bound strength, Q_{CL}, of steel columns under axial compression shall be the lowest value obtained for the limit states of column buckling, local flange buckling, or local web buckling. The effective design strength or the lower-bound axial compressive strength, P_{CL}, shall be calculated in accordance with AISC (1997) *Seismic Provisions*, taking ϕ=1.0 and using the lower-bound strength, F_{yLB}, for yield strength.

The expected axial strength of a column in tension, Q_{CE}, shall be computed in accordance with Equation (5-8):

$$Q_{CE} = T_{CE} = A_c F_{ye} \tag{5-8}$$

where:

A_c = Area of column

F_{ye} = Expected yield strength of column

T_{CE} = Expected tensile strength of column

3. Panel Zone: The strength of the panel zone shall be calculated using Equation (5-5).

4. FR Beam-Column Connections: The strength of connections shall be based on the controlling mechanism considering all potential modes of failure.

C5.5.2.3.2 Linear Static and Dynamic Procedures

4. FR Beam-Column Connections: The design professional is directed to *FEMA 351* for guidance in determining the strength of various FR connection configurations.

5. Column Base Plate to Concrete Pile Caps or Footings: The strength of connections between columns and concrete pile caps or footings shall be the lowest value obtained for the limit states of the strength of the columns, the strength of the base plates, and the strength of the anchor bolts. The strength of the base plate shall be the lowest value based on the following limit states: expected strength of the welds or bolts, expected bearing stress of the concrete, and expected yield strength, F_{ye}, of the base plate.

The strength of the anchor bolt connection between the base plate and the concrete shall be the lowest value based on the following limit states: shear or tension yield strength of the anchor bolts, loss of bond between the anchor bolts and the concrete, or failure of the concrete. Anchor bolt strengths for each failure type or limit state shall be calculated in accordance with *IBC*, Section 1913, using ϕ=1.0, or other approved procedure.

5.5.2.3.3 Nonlinear Static Procedure

The complete load-deformation relationship of each component as depicted in Figure 5-1 shall be determined in accordance with Section 5.5.2.2.2. The values for expected strength, Q_{CE}, shall be the same as those used for linear procedures as specified in Section 5.5.2.3.2.

5.5.2.3.4 Nonlinear Dynamic Procedures

The complete hysteretic behavior of each component shall be determined experimentally.

C5.5.2.3.4 Nonlinear Dynamic Procedures

The design professional is directed to *FEMA 355D* for information concerning nonlinear behavior of various tested connection configurations.

5.5.2.4 Acceptance Criteria

5.5.2.4.1 General

Component acceptance criteria shall be computed in accordance with the general requirements of Section 5.4.2 and the specific requirements of this section.

C5.5.2.4.1 General

The strength and behavior of steel moment-resisting frames is typically governed by the connections. The design professional is urged to determine the controlling limit state of the system when selecting the corresponding acceptance criterion.

5.5.2.4.2 Linear Static and Dynamic Procedures

1. Beams: The acceptance criteria of this section shall apply to flexural actions of elements of structural steel with negligible axial load. Beam flexure and shear shall be considered deformation-controlled.

For built-up shapes, the adequacy of lacing plates shall be evaluated using the provisions for tension braces in Section 5.6.2.4.

Values for the *m*-factor used in Equation (3-20) shall be as specified in Table 5-5. For fully concrete-encased beams where confining reinforcement is provided to allow the concrete to remain in place during an earthquake, the values of $b_f = 0$ and $L_p = 0$ shall be used for the purpose of determining m. *If* $Q_{CE} < M_{pCE}$ due to lateral torsional buckling, then m in Equation (3-20) shall be replaced by m_e, calculated in accordance with Equation (5-9):

$$m_e = C_b\left[m - (m-1)\frac{L_b - L_p}{L_r - L_p}\right] \tag{5-9}$$

where:

- L_b = Distance between points braced against lateral displacement of the compression flange, or between points braced to prevent twist of the cross-section, AISC (1993) *LRFD Specifications*
- L_p = Limiting unbraced length between points of lateral restraint for the full plastic moment capacity to be effective, AISC (1993) *LRFD Specifications*
- L_r = Limiting unbraced length between points of lateral support beyond which elastic lateral torsional buckling of the beam is the failure mode, AISC (1993) *LRFD Specifications*
- m = Value of m given in Table 5-5
- m_e = Effective m computed in accordance with Equation (5-9)
- C_b = Coefficient to account for effect of nonuniform moment, AISC (1993) *LRFD Specifications*

For built-up shapes, where the strength is governed by the strength of the lacing plates that carry component shear, the *m*-factor shall be taken as 0.5 times the applicable value in Table 5-5, unless larger values are justified by tests or analysis; however, *m* need not be taken less than 1.0. For built-up laced beams and columns fully encased in concrete, local buckling of the lacing need not be considered where confining reinforcement is provided to allow the encasement to remain in place during a design earthquake.

2. Columns: For steel columns under combined axial compression and bending stress, where the axial column load is less than 50% of the lower-bound axial column strength, P_{CL}, the column shall be considered deformation-controlled for flexural behavior and force-controlled for compressive behavior and the combined strength shall be evaluated by Equation (5-10) or (5-11).

For $0.2 \le \frac{P_{UF}}{P_{CL}} \le 0.5$

$$\frac{P_{UF}}{P_{CL}} + \frac{8}{9}\left[\frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}}\right] \le 1.0 \qquad (5\text{-}10)$$

For $\frac{P_{UF}}{P_{CL}} < 0.2$

$$\frac{P_{UF}}{2P_{CL}} + \frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}} \le 1.0 \qquad (5\text{-}11)$$

where:

P_{UF} = Axial force in the member computed in accordance with Section 3.4.2.1.2

P_{CL} = Lower-bound compression strength of the column

M_x = Bending moment in the member for the x-axis computed in accordance with Section 3.4.2.1.1

M_y = Bending moment in the member for the y-axis computed in accordance with Section 3.4.2.1.1

M_{CEx} = Expected bending strength of the column for the x-axis

M_{CEy} = Expected bending strength of the column for the y-axis

m_x = Value of *m* for the column bending about the x-axis in accordance with Table 5-5

m_y = Value of *m* for the column bending about the y-axis in accordance with Table 5-5

Steel columns with axial compressive forces exceeding 50% of the lower-bound axial compressive strength, P_{CL}, shall be considered force-controlled for both axial loads and flexure and shall be evaluated using Equation (5-12):

$$\frac{P_{UF}}{P_{CL}} + \frac{Mu_{Fx}}{M_{CLx}} + \frac{Mu_{Fy}}{M_{CLy}} \le 1 \qquad (5\text{-}12)$$

where:

P_{UF} = Axial load in the member, calculated in accordance with Section 3.4.2.1.2

M_{UFx} = Bending moment in the member about the x-axis, calculated in accordance with Section 3.4.2.1.2

M_{UFy} = Bending moment in the member about the y-axis, calculated in accordance with Section 3.4.2.1.2

M_{CLx} = Lower-bound flexural strength of the member about the x-axis

M_{CLy} = Lower-bound flexural strength of the member about the y-axis

Flexural strength shall be calculated in accordance with AISC (1997) *Seismic Provisions* taking ϕ=1.0 and using the lower-bound value for yield strength.

For columns under combined compression and bending, lateral bracing to prevent torsional buckling shall be provided as required by the AISC (1993) *LRFD Specifications*.

Steel columns under axial tension shall be considered deformation-controlled and shall be evaluated using Equation (3-20).

Steel columns under combined axial tension and bending stress shall be considered deformation-controlled and shall be evaluated using Equation (5-13):

$$\frac{T}{m_t T_{CE}} + \frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}} \le 1.0 \qquad (5\text{-}13)$$

where:

M_x = Bending moment in the member for the x-axis.

M_y = Bending moment in the member for the y-axis

M_{CEx} = Expected bending strength of the column for the x-axis

M_{CEy} = Expected bending strength of the column for the y-axis

m_t = Value of *m* for the column in tension based on Table 5-5

m_x = Value of *m* for the column bending about the x-axis based on Table 5-5

m_y = Value of *m* for the column bending about the y-axis based on Table 5-5

T = Tensile load in column

T_{CE} = Expected tensile strength of column computed in accordance with Equation (5-8)

3. Panel Zone: Shear behavior of panel zones shall be considered deformation-controlled and shall be evaluated using Equation (3-20), with the expected panel zone shear strength, Q_{CE}, calculated according to Equation (5-5) and *m*-factors taken from Table 5-5.

4. FR Beam-Column Connections: FR connections identified in Table 5-4 shall be considered deformation-controlled and evaluated in accordance with Equation (3-20), with Q_{UD} and Q_{CE} taken as the computed demand and capacity of the critical connection component respectively, and *m*-factors taken from Table 5-5 as modified below.

Connection acceptance criteria are dependent on the detailing of continuity plates (column stiffeners that align with the beam flanges), the strength of the panel zone, the beam span-to-depth ratio, and the slenderness of the beam web and flanges. Tabulated *m*-factors in Table 5-5 shall be modified as determined by the following four conditions. The modifications shall be cumulative, but *m*-factors need not be taken as less than 1.0.

1. If one of the following conditions is not met, the tabulated *m*-factors in Table 5-5 shall be multiplied by 0.8.

$$t_{cf} \geq \frac{b_{bf}}{5.2}$$

or

$$\frac{b_{bf}}{7} \leq t_{cf} < \frac{b_{bf}}{5.2} \text{ and continuity plates with } t \geq \frac{t_{bf}}{2}$$

or

$$t_{cf} < \frac{b_{bf}}{7} \qquad \text{and continuity plates with} \quad t \geq \frac{t_{bf}}{2}$$

2. If one of the following conditions is not met, the tabulated *m*-factors in Table 5-5 shall be multiplied by 0.8.

$$0.6 \leq \frac{V_{PZ}}{V_y} \leq 0.9$$

where $V_y = 0.55F_{ye(col)}d_ct_{cw}$ and V_{PZ} is the computed panel zone shear at the development of a hinge at the critical location of the connection. For M_{ye} at the face of the column,

$$V_{PZ} = \frac{\sum M_y beam}{d_b}\left(\frac{L}{L-d_c}\right)\left(\frac{h-d_b}{h}\right)$$

3. If the clear span-to-depth ratio, L/d, is greater than 10, the tabulated *m*-factors in Table 5-5 shall be multiplied by $1.4 - 0.04\frac{L}{d}$

4. If the beam flange and web meet the following conditions, the tabulated *m*-factors in Table 5-5 need not be modified for flange and web slenderness.

$$\frac{b_f}{2t_f} < \frac{52}{\sqrt{F_{ye}}} \quad \text{and} \quad \frac{h}{t_w} < \frac{418}{\sqrt{F_{ye}}}$$

If the beam flange or web slenderness values exceed either of the following limits, the tabulated *m*-factors in Table 5-5 shall be multiplied by 0.5.

$$\frac{b_f}{2t_f} > \frac{65}{\sqrt{F_{ye}}} \quad \text{or} \quad \frac{h}{t_w} > \frac{640}{\sqrt{F_{ye}}}$$

Straight line interpolation, based on the case that results in the lower modifier, shall be used for intermediate values of beam flange or web slenderness.

Type FR connections designed to promote yielding of the beam remote from the column face shall be considered force-controlled and shall be designed using Equation (5-14):

$$Q_{CLc} \geq Q_{CEb} \tag{5-14}$$

where:

Q_{CLc} = The lower-bound strength of the connection

Q_{CEb} = Expected bending strength of the beam

5. Column Base Plate to Concrete Pile Caps or Footings: For column base plate yielding, bolt yielding, and weld failure, the use of *m*-factors from Table 5-5, based on the respective limit states for partially restrained end plates, shall be permitted. Column-base connection limit states controlled by anchor bolt failure modes governed by the concrete shall be considered force-controlled.

C5.5.2.4.2 Linear Static and Dynamic Procedures

4. FR Beam-Column Connections. The continuity plate modifier is based on recommendations *FEMA 355F* for continuity plate detailing in relationship to column flange thickness.

The panel zone modifier is based on research in *FEMA 355F* indicating that connection performance is less ductile when the strength of the panel zone is either too great or too small compared to the flexural strength of the beam. The panel zone range between 60 percent to 90 percent of the beam strength is considered to provide balanced yielding between the beam and panel zone, which results in more desirable performance.

The clear span-to-depth ratio modifier for linear acceptance criteria reflects the decreased apparent ductility that arises due to increased elastic rotations for longer beams. The decreased plastic rotation capacity of beams with very small L/d ratios is not reflected directly. However, the modifier for linear criteria was developed so that it would be appropriate for the predominant case of L/d ratios greater than about 5.

The beam flange and web slenderness modifier is based on the same modifications to beam acceptance criteria contained in Table 5-5. While not an aspect of the connection itself, beam flange and web slenderness affect the behavior of the connection assembly.

Type FR connections designed to promote yielding of the beam in the span, remote from the column face, are discussed in *FEMA 350*.

5.5.2.4.3 Nonlinear Static and Dynamic Procedures

Calculated component actions shall satisfy the requirements of Section 3.4.3. Maximum permissible inelastic deformations shall be taken from Tables 5-6 and 5-7.

1. Beams: Flexural actions of beams shall be considered deformation-controlled. Permissible plastic rotation deformation shall be as indicated in Tables 5-6 and 5-7, where θ_y shall be calculated in accordance with Section 5.5.2.2.2.

2. Columns: Axial compressive loading of columns shall be considered force-controlled, with the lower-bound axial compression capacity, P_{CL}, computed in accordance with Section 5.5.2.4.2.

Flexural loading of columns, with axial loads at a target displacement less than 50% of P_{CL}, computed in accordance with Section 5.5.2.4.2, shall be considered deformation-controlled and maximum permissible plastic rotation demands on columns, in radians, shall be as indicated in Tables 5-6 and 5-7, dependent on the axial load present and the compactness of the section.

Flexural loading of columns, with axial loads at the target displacement greater than or equal to 50% of P_{CL}, computed in accordance with Section 5.5.2.4.2, shall be considered force-controlled and shall conform to Equation (5-11).

3. FR Connection Panel Zones: Plastic rotation demands on panel zones, shall be evaluated using the acceptance criteria provided in Tables 5-6 and 5-7.

4. FR Beam-Column Connections: FR connections identified in Table 5-4 shall be considered deformation-controlled and the plastic rotation predicted by analysis shall be compared with the acceptance criteria in Tables 5-6 and 5-7 as modified below. Connection acceptance criteria are dependent on the detailing of continuity plates, the strength of the panel zone, the beam span-to-depth ratio, and the slenderness of the beam web and flanges as determined by the following four conditions. The modifications shall be cumulative.

1. If one of the following conditions is not met, the tabulated plastic rotation in Tables 5-6 and 5-7 shall be multiplied by 0.8.

$$t_{cf} \geq \frac{b_{bf}}{5.2}$$

or

$$\frac{b_{bf}}{7} \leq t_{cf} < \frac{b_{bf}}{5.2} \text{ and continuity plates with } t \geq \frac{t_{bf}}{2}$$

or

$$t_{cf} < \frac{b_{bf}}{7} \quad \text{and continuity plates with} \quad t \geq t_{bf}$$

2. If the following condition is not met, the tabulated plastic rotations in Tables 5-6 and 5-7 shall be multiplied by 0.8.

$$0.6 \leq \frac{V_{PZ}}{V_y} \leq 0.9$$

where $V_y = 0.55F_{ye(col)}d_c t_{cw}$ and V_{PZ} is the computed panel zone shear at the development of a hinge at the critical location of the connection. For M_{ye} at the face of the column,

$$V_{PZ} = \frac{\sum M_y beam}{d_b}\left(\frac{L}{L-d_c}\right)\left(\frac{h-d_b}{h}\right)$$

3. If the clear span-to-depth ratio, L/d, is less than 8, the tabulated plastic rotations in Tables 5-6 and 5-7 shall be multiplied by $(0.5)\left[\frac{8-L/d}{3}\right]$

4. If the beam flange and web meet the following conditions, the tabulated plastic rotations in Tables 5-6 and 5-7 need not be modified for flange and web slenderness.

$$\frac{b_f}{2t_f} < \frac{52}{\sqrt{F_{ye}}} \quad \text{and} \quad \frac{h}{t_w} < \frac{418}{\sqrt{F_{ye}}}$$

If the beam flange or web slenderness values exceed either of the following limits, the tabulated plastic rotations Tables 5-6 and 5-7 shall be multiplied by 0.5.

$$\frac{b_f}{2t_f} > \frac{65}{\sqrt{F_{ye}}} \quad \text{or} \quad \frac{h}{t_w} > \frac{640}{\sqrt{F_{ye}}}$$

Straight line interpolation, based on the case that results in the lower modifier, shall be used for intermediate values of beam flange or web slenderness.

Type FR connections designed to promote yielding of the beam in the span remote from the column face, shall be considered force-controlled and shall be evaluated to assure that the lower-bound strength of the connection exceeds the expected flexural strength of the beam at the connection.

C5.5.2.4.3 Nonlinear Static and Dynamic Procedures

4. FR Beam-Column Connections: The continuity plate modifier is based on recommendations in *FEMA 355F* for continuity plate detailing in relationship to column flange thickness.

The panel zone modifier is based on research in *FEMA 355F* indicating that connection performance is less ductile when the strength of the panel zone is either too great or too small compared to the flexural strength of the beam. The panel zone range between 60 percent to 90 percent of the beam strength is considered to provide balanced yielding between the beam and panel zone, which results in more desirable performance.

The clear span-to-depth ratio modifier for nonlinear modeling and acceptance criteria reflects decreased plastic rotation capacity for beams with hinging occurring over a shorter length. This modifier is based on the plastic rotation capacities corresponding to the *FEMA 350* L/d limits of 5 and 8.

The beam flange and web slenderness modifier is based on the same modifications to beam acceptance criteria contained in Tables 5-6 and 5-7. While not an aspect of the connection itself, beam flange and web slenderness affects the behavior of the connection assembly.

Type FR connections designed to promote yielding of the beam in the span, remote from the column face, are discussed in *FEMA 350*.

5.5.2.5 Rehabilitation Measures

FR moment frame components that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 5.4.3 and other provisions of this standard.

C5.5.2.5 Rehabilitation Measures

The following measures, which are presented in greater detail in *FEMA 351*, may be effective in rehabilitating FR moment frames:

1. Add steel braces to one or more bays of each story to form concentric or eccentric braced frames to increase the stiffness of the frames. The attributes and design criteria for braced frames shall be as specified in Section 5.6. The location of added braces should be selected so as to not increase horizontal torsion in the system.

2. Add ductile concrete or masonry shear walls or infill walls to one or more bays of each story to increase the stiffness and strength of the structure. The attributes and design requirements of concrete and masonry shear walls shall be as specified in Sections 6.8 and 7.4, respectively. The attributes and design requirements of concrete and masonry infills shall be as specified in Sections 6.7 and 7.5, respectively. The location of added walls should be selected so as not to increase horizontal torsion in the system.

3. Attach new steel frames to the exterior of the building. The rehabilitated structure should be checked for the effects of the change in the distribution of stiffness, the seismic load path, and the connections between the new and existing frames. The rehabilitation scheme of attaching new steel frames to the exterior of the building has been used in the past and has been shown to be very effective under certain conditions. This rehabilitation approach may be structurally efficient, but it changes the architectural appearance of the building. The advantage is that the rehabilitation may take place without disrupting the use of the building.

4. Reinforce moment-resisting connections to force plastic hinge locations in the beam material away from the joint region to reduce the stresses in the welded connection, thereby reducing the possibility of brittle fractures. This scheme should not be used if the full-pen connection of the existing structure did not use weld material of sufficient toughness to avoid fracture at stresses lower than yield or when strain hardening at the new hinge location produces larger stresses than existing at the weld. The rehabilitation measures to reinforce selected moment-resisting connections shall consist of providing horizontal cover plates, vertical stiffeners, or haunches. Removal of beam material to force plastic hinge in the beam and away from the joint region shall also be permitted subject to the above restrictions. Guidance on the design of these modifications of FR moment connections is discussed in *FEMA 351*.

5. Add energy dissipation devices as specified in Chapter 9.

6. Increase the strength and stiffness of existing frames by welding steel plates or shapes to selected members.

5.5.3 Partially Restrained Moment Frames

5.5.3.1 General

Partially restrained (PR) moment frames shall be defined as those moment frames with connections identified as PR in Table 5-4. Moment frames with connections not included in Table 5-4 shall be defined as PR if the deformations of the beam-to-column joints contribute greater than 10% to the total lateral deflection of the frame or where the strength of the connections is less than the strength of the weaker of the two members being joined. For a PR connection with two or more failure modes, the weakest failure mechanism shall be considered to govern the behavior of the joint. Design provisions for PR frames specified in AISC (1997) *Seismic Provisions* or *ASCE 7* shall apply unless superseded by the provisions in this standard. Equations for calculating nominal design strength shall be used for determining the expected strength, except $\phi = 1$, and either the expected strength or lower-bound strength shall be used in place of F_y, as further indicated in this standard.

C5.5.3.1 General

Table 5-4 includes simple shear or pinned connections classified as PR connections. Although the gravity load carrying beams and columns are typically neglected in the lateral analysis of steel moment frame structures, SAC research contained in *FEMA 355D* indicates that these connections are capable of contributing non-negligible stiffness through very large drift demands. Including gravity load carrying elements (subject to the modeling procedures and acceptance criteria in this section) in the mathematical model could be used by the design engineer to reduce the demands on the moment frame elements.

5.5.3.2 Stiffness

5.5.3.2.1 Linear Static and Dynamic Procedures

1. Beams, columns, and panel zones: Axial area, shear area, moment of inertia, and panel zone stiffness shall be determined as specified in Section 5.5.2.2 for FR frames.

2. Connections: The rotational stiffness K_θ of each PR connection for use in PR frame analysis shall be determined by the procedure of this section, by experiment, or by an approved rational analysis. The deformation of the connection shall be included when calculating frame displacements.

The rotational spring stiffness, K_θ, shall be calculated in accordance with Equation (5-15):

$$K_\theta = \frac{M_{CE}}{0.005} \tag{5-15}$$

where:

M_{CE} = Expected moment strength of connection for the following PR connections:

1. PR connections encased in concrete, where the nominal resistance, M_{CE}, determined for the connection shall include the composite action provided by the concrete encasement.
2. PR connections encased in masonry, where composite action shall not be included in the determination of connection resistance, M_{CE}.
3. Bare steel PR connections.

For PR connections not listed above, the rotational spring stiffness shall be calculated in accordance with Equation (5-16):

$$K_\theta = \frac{M_{CE}}{0.003} \tag{5-16}$$

As a simplified alternative, modeling the frame as for FR joints but with the beam stiffness, EI_b, adjusted to account for the flexibility of the joints in accordance with Equation (5-17) shall be permitted:

$$EI_b\ adjusted = \frac{1}{\dfrac{6h}{L_b^2 K_\theta} + \dfrac{1}{EI_b}} \tag{5-17}$$

where:

K_θ = Equivalent rotational spring stiffness of connection per Equation (5-15) or (5-16)

M_{CE} = Expected moment strength

I_b = Moment of inertia of the beam

E = Modulus of elasticity

h = Average story height of the columns

L_b = Centerline span of the beam

When Equation (5-17) is used, the adjusted beam stiffness shall be used in standard rigid-connection frame analysis and the rotation of the connection shall be taken as the rotation of the beam at the joint.

C5.5.3.2.1 Linear Static and Dynamic Procedures

The design professional is directed to *FEMA 274* for information concerning stiffness properties and modeling guidelines for PR connections.

5.5.3.2.2 Nonlinear Static Procedure

If the Nonlinear Static Procedure of Chapter 3 is used, the following criteria shall apply:

1. The elastic component properties shall be modeled as specified in Section 5.5.3.2.1.

2. The nonlinear moment-curvature or load-deformation behavior for beams, beam-columns, and panel zones shall be modeled as specified in Section 5.5.2.2 for FR frames.

3. In lieu of relationships derived from experiment or analysis, the generalized load-deformation curve shown in Figure 5-1 with its parameters a, b, c as defined in Tables 5-6 and 5-7, shall be used to represent moment-rotation behavior for PR connections in accordance with Section 5.5.2.2.2. The value for θ_y shall be 0.005 for connections, for which Equation (5-15) in Section 5.5.3.2.1 applies, or 0.003 for all other connections.

C5.5.3.2.2 Nonlinear Static Procedure

The design professional is directed to *FEMA 355D* for information concerning nonlinear behavior of various tested connection configurations.

5.5.3.2.3 Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component shall be modeled as verified by experiment.

C5.5.3.2.3 Nonlinear Dynamic Procedure

The design professional is directed to *FEMA 355D* for information concerning nonlinear behavior of various tested connection configurations.

5.5.3.3 Strength

5.5.3.3.1 General

Component strengths shall be computed in accordance with the general requirements of Section 5.4.2 and the specific requirements of this section.

5.5.3.3.2 Linear Static and Dynamic Procedures

The strength of steel beams and columns in PR Frames being analyzed using linear procedures shall be computed in accordance with Section 5.5.2.3.2 for FR Frames.

The expected strength, Q_{CE}, for PR connections shall be based on procedures specified in AISC (1993) *LRFD Specifications*, based on experiment, or based on the procedures listed in the subsequent sections.

1. Top and Bottom Clip Angle Connection: The moment strength, M_{CE}, of the riveted or bolted clip angle connection, as shown in Figure 5-3, shall be the smallest value of M_{CE} computed for the following four limits states:

1.1 Limit State 1: If the shear connectors between the beam flange and the flange angle control the capacity of the connection, Q_{CE} shall be computed in accordance with Equation (5-18):

$$Q_{CE} = M_{CE} = d_b(F_{ve}A_bN_b) \qquad (5\text{-}18)$$

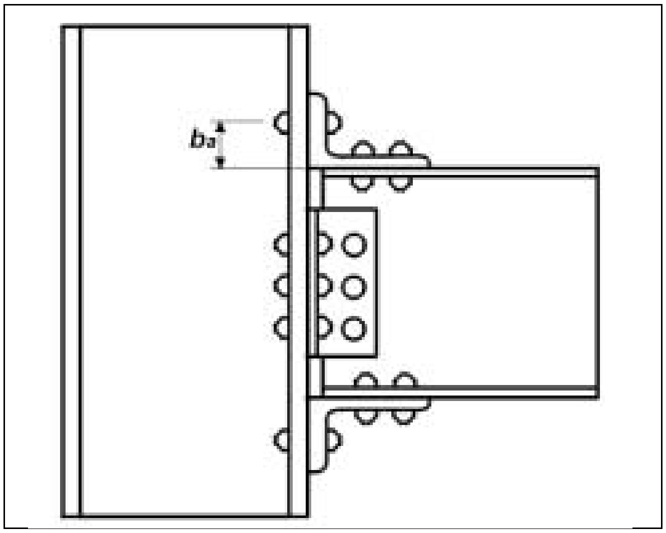

Figure 5-3 Top and Bottom Clip Angle Connection

where:

A_b = Gross area of rivet or bolt

d_b = Overall beam depth

F_{ve} = Unfactored nominal shear strength of the bolts or rivets given in AISC (1993) *LRFD Specifications*

N_b = Least number of bolts or rivets connecting the top or bottom angle to the beam flange

1.2 Limit State 2: If the tensile capacity of the horizontal leg of the connection controls the capacity, P_{CE} shall be taken as the smaller of that computed by Equation (5-19) or (5-20):

$$P_{CE} \leq F_{ye} A_g \tag{5-19}$$

$$P_{CE} \leq F_{te} A_e \tag{5-20}$$

and Q_{CE} shall be calculated in accordance with Equation (5-21):

$$Q_{CE} = M_{CE} \leq P_{CE}(d_b + t_a) \tag{5-21}$$

where:

F_{ye} = Expected yield strength of the angle

F_{te} = Expected tensile strength of the angle

A_e = Effective net area of the horizontal leg

A_g = Gross area of the horizontal leg

t_a = Thickness of angle

1.3 Limit State 3: If the tensile capacity of the rivets or bolts attaching the vertical outstanding leg to the column flange controls the capacity of the connection, Q_{CE} shall be computed in accordance with Equation (5-22):

$$Q_{CE} = M_{CE} = (d_b + b_a)(F_{te} A_b N_b) \tag{5-22}$$

where:

A_b = Gross area of rivet or bolt

b_a = Dimension in Figure 5-3

F_{te} = Expected tensile strength of the bolts or rivets

N_b = Least number of bolts or rivets connecting top or bottom angle to column flange

1.4 Limit State 4: If the flexural yielding of the flange angles controls the capacity of the connection, Q_{CE} shall be given by Equation (5-23):

$$Q_{CE} = M_{CE} = \frac{w t_a^2 F_{ye}}{4\left[b_a - \frac{t_a}{2}\right]}(d_b + b_a) \tag{5-23}$$

where:

b_a = Dimension shown in Figure 5-3

w = Length of the flange angle

2. Double Split Tee Connection: The moment strength, M_{CE}, of the double split tee (T-stub) connection, as shown in Figure 5-4, shall be the smallest value of M_{CE} computed for the following four limit states:

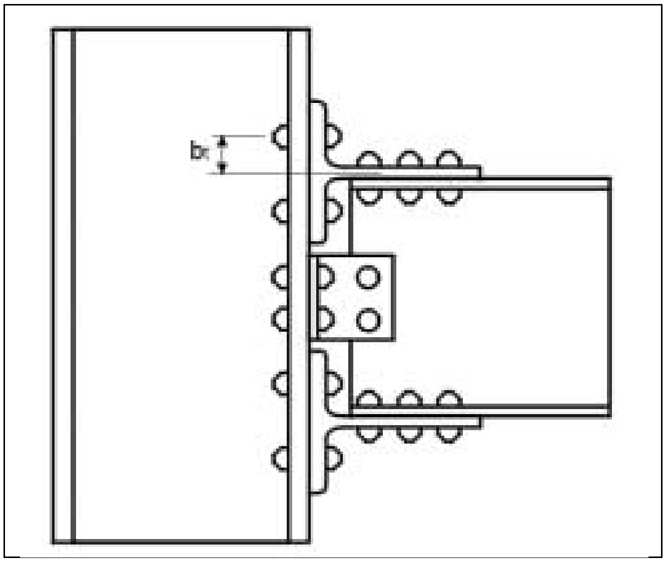

Figure 5-4 Double Split Tee Connection

2.1 Limit State 1: If the shear connectors between the beam flange and the web of the split tee control the capacity of the connection, Q_{CE} shall be calculated using Equation (5-18).

2.2 Limit State 2: If the tension capacity of the bolts or rivets connecting the flange of the split tee to the column flange control the capacity of the connection, Q_{CE} shall be calculated using Equation (5-24):

$$Q_{CE} = M_{CE} = (d_b + 2b_t + t_s)(F_{te}A_bN_b) \qquad (5\text{-}24)$$

where:

d_b = Overall beam depth

b_t = Distance between one row of fasteners in the split tee flange and the centerline of the stem as shown in Figure 5-4

t_s = Thickness of the split tee stem

F_{te} = Expected tensile strength of the bolts or rivets

A_b = Gross area of rivet or bolt

N_b = Number of fasteners in tension connecting the flanges of one split tee to the column flange

2.3 Limit State 3: If tension in the stem of the split tee controls the capacity of the connection, Equations (5-21) and (5-22) shall be used to determine Q_{CE}, with A_g and A_e being the gross and net areas of the split tee stem and replacing t_a with t_s.

2.4 Limit State 4: If flexural yielding of the flanges of the split tee controls the capacity of the connection, Q_{CE} shall be determined in accordance with Equation (5-25):

$$Q_{CE} = M_{CE} = \frac{(d_b + t_s)wt_f^2F_{ye}}{2(b_t - k_1)} \qquad (5\text{-}25)$$

where:

k_1 = Distance from the center of the split tee stem to the edge of the split tee flange fillet

b_t = Distance between one row of fasteners in the split tee flange and the centerline of the stem as shown in Figure 5-4

w = Length of split tee

t_f = Thickness of split tee flange

3. Bolted Flange Plate Connections: [For bolted flange plate connections, as shown in Figure 5-5, the flange plate shall be welded to the column and welded or bolted to the beam flange. This connection shall be considered fully restrained if its strength equals or exceeds the strength of the connected beam. The expected strength of the connection shall be calculated in accordance with Equation (5-26):

$$Q_{CE} = M_{CE} = P_{CE}(d_b + t_p) \qquad (5\text{-}26)$$

where:

P_{CE} = Expected strength of the flange plate connection as governed by the net section of the flange plate, the shear capacity of the bolts, or the strength of the welds to the column flange

t_p = Thickness of flange plate

d_b = Overall beam depth

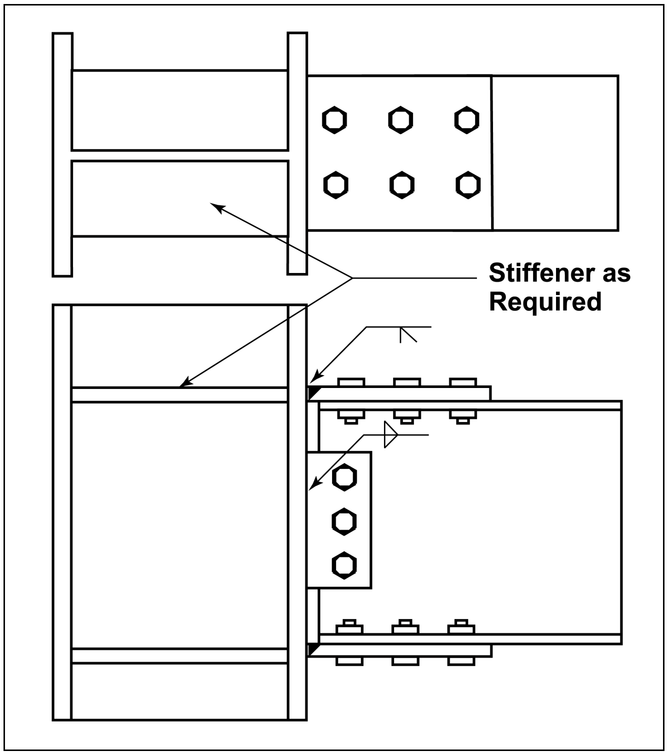

Figure 5-5 Bolted Flange Plate Connection

4. Bolted End Plate Connections: Bolted end plate connections, as shown in Figure 5-6, shall be considered FR if their expected and lower-bound strengths equal or exceed the expected strength of the connecting beam. The lower-bound strength, Q_{CL}=M_{CL}, shall be the value determined for the limit state of the bolts under combined shear and tension and the expected strength, Q_{CE} = M_{CE}, shall be determined for the limit state of bending in the end plate calculated in accordance with the procedures of the AISC (1993) *LRFD Specifications* or by another approved procedure

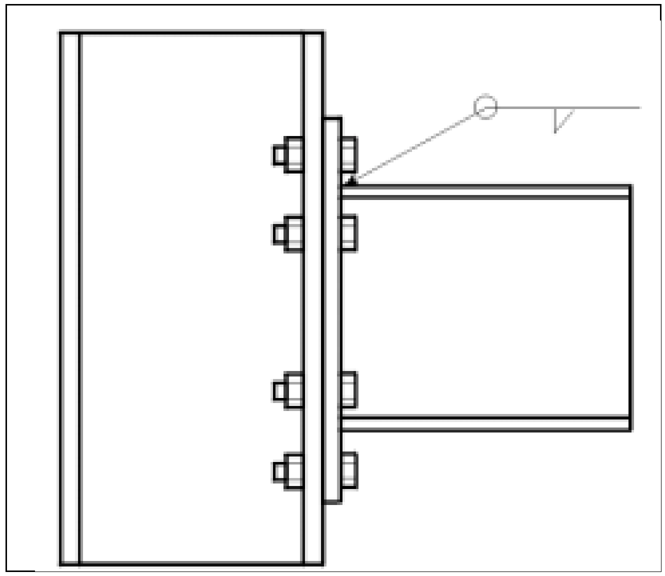

Figure 5-6 Bolted End Plate Connection

5. Composite Partially Restrained Connections: Strength and deformation acceptance criteria of composite partially restrained connections shall be based on approved rational analysis procedures and experimental evidence.

5.5.3.3.3 Nonlinear Static Procedure

The complete load-deformation relationship of each component as depicted by Figure 5-1 shall be determined in accordance with Section 5.5.2.2.2. The values for expected strength, Q_{CE}, of PR connections shall be the same as those used for linear procedures as specified in Section 5.5.3.3.2.

5.5.3.3.4 Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component shall be determined experimentally.

C5.5.3.3.4 Nonlinear Dynamic Procedure

The design professional is directed to *FEMA 355D* for information concerning nonlinear behavior of various tested connection configurations.

5.5.3.4 Acceptance Criteria

5.5.3.4.1 General

Component acceptance criteria shall be computed in accordance with the general requirements of Section 5.4.2 and the specific requirements of this section.

C5.5.3.4.1 General

The strength and behavior of partially restrained steel moment-resisting frames is typically governed by the connections. The design professional is urged to consider the acceptance criteria for the mechanism that controls the system.

5.5.3.4.2 Linear Static and Dynamic Procedures

Design actions shall be compared with design strengths in accordance with Section 3.4.2. *m*-factors for steel components and connections of PR frames shall be selected from Table 5-5. Limit states for which no *m*-factors are provided in Table 5-5 shall be considered force-controlled.

Acceptance criteria for steel beams and columns in PR frames shall be computed in accordance with Section 5.5.2.4.2.

5.5.3.4.3 Nonlinear Static and Dynamic Procedures

Calculated component actions shall satisfy the requirements of Section 3.4.3. Maximum permissible inelastic deformations shall be taken from Tables 5-6 and 5-7.

5.5.3.5 Rehabilitation Measures

PR moment frames that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 5.4.3 and other provisions of this standard.

C5.5.3.5 Rehabilitation Measures

The rehabilitation measures for FR moment frames described in C5.5.2.5 may be effective for PR moment frames as well. PR moment frames are often too flexible to provide adequate seismic performance. Adding concentric or eccentric bracing, or reinforced concrete or masonry infills, may be a cost-effective rehabilitation measure.

Connections in PR moment frames are usually components that are weak, flexible, or both. Connections may be rehabilitated by replacing rivets with high-strength bolts, adding weldment to supplement rivets or bolts, or welding stiffeners to connection pieces or combinations of these measures. Refer to *FEMA 351* for additional information concerning the rehabilitation of PR moment frames.

5.6 Steel Braced Frames

5.6.1 General

Steel braced frames shall be defined as those frames that develop seismic resistance primarily through axial forces in the components.

Modeling procedures and rehabilitation measures for concentric braced frames and eccentric braced frames shall be as specified in Sections 5.6.2 and 5.6.3, respectively. Components of concentric and eccentric braced frames shall include columns, beams, braces, and connections. Eccentric braced frames shall also include link beam components.

C5.6.1 General

Steel braced frames act as vertical trusses where the columns are the chords and the beams and braces are the web members.

Components can be either bare steel, steel with a nonstructural coating for fire protection, or steel with concrete or masonry encasement.

5.6.2 Concentric Braced Frames (CBF)

5.6.2.1 General

Concentric braced frames (CBF) shall be defined as braced frame systems where component worklines intersect at a single point in a joint, or at multiple points such that the distance between points of intersection, or eccentricity, e, is less than or equal to the width of the smallest member connected at the joint. Bending due to such eccentricities shall be considered in the design of the components.

5.6.2.2 Stiffness

5.6.2.2.1 Linear Static and Dynamic Procedures

Axial area, shear area, and moment of inertia shall be calculated as specified for FR frames in Section 5.5.2.2.1.

FR connections shall be modeled as specified in Section 5.5.2.2.1. PR connections shall be modeled as specified in Section 5.5.3.2.1.

Braces shall be modeled as columns as specified in Section 5.5.2.2.1.

5.6.2.2.2 Nonlinear Static Procedure

If the Nonlinear Static Procedure of Chapter 3 is used, the following criteria shall apply:

1. The elastic component properties shall be modeled as specified in Section 5.6.2.2.1.

2. The nonlinear moment-curvature or load-deformation behavior to represent yielding and buckling shall be as specified in Section 5.5.2.2.2 for beams and columns and Section 5.5.3.2.2 for PR connections.

In lieu of relationships derived from experiment or analysis, the nonlinear load-deformation behavior of braces shall be modeled as shown in Figure 5-1 with parameters as defined in Tables 5-6 and 5-7. For braces loaded in compression, the parameter Δ in Figure 5-1 shall represent plastic axial deformation. The parameter Δ_c shall represent the axial deformation at the expected buckling load. The reduction in strength of a brace after buckling shall be included in the model. Modeling of the compression brace behavior using elasto-plastic behavior shall be permitted if the yield force is assumed as the residual strength after buckling, as defined by parameter c in Figure 5-1 and Tables 5-6 and 5-7. Implications of forces higher than this lower-bound force shall be evaluated relative to other components to which the brace is connected. For braces in tension, the parameter Δ_T shall be the axial deformation at development of the expected tensile yield load in the brace.

C5.6.2.2.2 Nonlinear Static Procedure

The design professional is directed to *FEMA 274* for information regarding nonlinear load-deformation behavior of braces.

5.6.2.2.3 Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component shall be based on experiment or other approved method.

C5.6.2.2.3 Nonlinear Dynamic Procedure

The design professional is directed to *FEMA 274* for information concerning hysteretic behavior of braced-frame components.

5.6.2.3 Strength

5.6.2.3.1 General

Component strengths shall be computed in accordance with the general requirements of Section 5.4.2 and the specific requirements of this section.

5.6.2.3.2 Linear Static and Dynamic Procedures

The expected strength, Q_{CE}, of steel braces under axial compression shall be the lowest value obtained for the limit states of buckling or local buckling. The effective design strength, P_{CE}, shall be calculated in accordance with AISC (1993) *LRFD Specifications*, taking ϕ=1.0 and using the expected yield strength, F_{ye}, for yield strength.

For common cross bracing configurations where both braces cross at their midpoints and are attached to a common gusset plate, the effective length of each brace shall be taken as 0.5 times the total length of the brace including gusset plates for both axes of buckling. For other bracing configurations (chevron, V, single brace), the length shall be taken as the total length of the brace including gusset plates, and the effective length shall be taken as 0.8 times the total length for in-plane buckling and 1.0 times the total length for out-of-plane buckling.

The expected strength, Q_{CE}, of steel braces in tension shall be calculated as for columns, in accordance with Section 5.5.2.3.2.

Expected, Q_{CE}, and lower bound, Q_{CL}, strengths of beams and columns shall be calculated as for FR beams and columns in Section 5.5.2.3. Strength of beams with non-negligible axial load shall be as calculated for FR columns.

The lower-bound strength of brace connections shall be calculated in accordance with the AISC (1993) *LFRD Specifications*, taking ϕ=1.0 and using the lower-bound yield strength, F_{yLB}, for yield strength.

5.6.2.3.3 Nonlinear Static Procedure

In lieu of relationships derived by experiment or analysis, the complete load-deformation behavior of each component as depicted by Figure 5-1 shall be determined in accordance with Section 5.5.2.2.2. The values for expected strength, Q_{CE}, shall as specified in Section 5.6.2.3.2 for linear procedures.

5.6.2.3.4 Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component shall be determined experimentally.

C5.6.2.3.4 Nonlinear Dynamic Procedure

The design professional is directed to *FEMA 274* for information concerning hysteretic behavior of braced frame components.

5.6.2.4 Acceptance Criteria

5.6.2.4.1 General

Component acceptance criteria shall be computed in accordance with the general requirements of Section 5.4.2 and the specific requirements of this section.

Axial tension and compression in braces shall be considered deformation-controlled. Actions on beams and columns with non-negligible axial load shall be considered force- or deformation-controlled as determined for FR frame columns in Section 5.5.2.4. Compression, tension, shear, and bending actions on brace connections including gusset plates, bolts, welds, and other connectors shall be considered force-controlled.

5.6.2.4.2 Linear Static and Dynamic Procedures

Design actions shall be compared with design strengths in accordance with Section 3.4.2. *m*-factors for steel components shall be selected from Table 5-5.

Stitch plates for built-up members shall be spaced such that the largest slenderness ratio of the components of the brace does not exceed 0.4 times the governing slenderness ratio of the brace as a whole. The stitches for compression members shall be able to transfer the maximum force in one element to adjacent elements. If not, stitch plates shall be added, or the *m*-factors in Table 5-4 shall be reduced by 50%, but need not be taken less than 1.0.

5.6.2.4.3 Nonlinear Static and Dynamic Procedures

Calculated component actions shall satisfy the requirements of Section 3.4.3. Deformations limits shall be taken from Tables 5-6 and 5-7.

5.6.2.5 Rehabilitation Measures

Concentric braced frame components that do meet not the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 5.4.3 and other provisions of this standard.

C5.6.2.5 Rehabilitation Measures

The rehabilitation measures for FR moment frames described in Section C5.5.2.5 may be effective for braced frames. Other modifications which may be effective include replacement or modification of connections that are insufficient in strength and/or ductility, and encasement of columns in concrete to improve their performance.

5.6.3 Eccentric Braced Frames (EBF)

5.6.3.1 General

Eccentric braced frames (EBF) shall be defined as braced frames where component worklines do not intersect at a single point and the distance between points of intersection, or eccentricity, e, exceeds the width of the smallest member connected at the joint. The component segment between these points is defined as the link component with a span equal to the eccentricity.

5.6.3.2 Stiffness

5.6.3.2.1 Linear Static and Dynamic Procedures

The elastic stiffness of beams, columns, braces, and connections shall be the same as those specified for FR and PR moment frames and concentric braced frames. The load-deformation model for a link beam shall include shear deformation and flexural deformation.

The elastic stiffness of the link beam, K_e, shall be computed in accordance with Equation (5-27):

$$K_e = \frac{K_s K_b}{K_s + K_b} \tag{5-27}$$

where:

$$K_s = \frac{GA_w}{e} \tag{5-28}$$

$$K_b = \frac{12EI_b}{e^3} \tag{5-29}$$

A_w = $(d_b - 2t_f)\, t_w$

e = Length of link beam

G = Shear modulus

K_e = Stiffness of the link beam

K_b = Flexural stiffness

K_s = Shear stiffness

d_b = Beam depth

t_f = Thickness of flange

t_w = Thickness of web

5.6.3.2.2 Nonlinear Static Procedure

In lieu of relationships derived from experiment or analysis, the nonlinear load-deformation behavior of member of EBFs shall be modeled as shown in Figure 5-1 and in accordance with Section 5.5.2.2.2.

Nonlinear models for beams, columns, and connections for FR and PR moment frames, and for the braces for a CBF, shall be permitted.

The link rotation at yield shall be calculated in accordance with Equation (5-30):

$$\theta_y = \frac{Q_{CE}}{K_e e} \tag{5-30}$$

5.6.3.2.3 Nonlinear Dynamic Procedure

If the Nonlinear Dynamic Procedure is used, the complete hysteretic behavior of each component shall be modeled and shall be based on experiment or an approved rational analysis procedure.

C5.6.3.2.3 Nonlinear Dynamic Procedure

The design professional is directed to *FEMA 274* for guidelines on modeling the link beams and information regarding the hysteretic behavior of EBF components.

5.6.3.3 Strength

5.6.3.3.1 General

Component strengths shall be computed in accordance with the general requirements of Section 5.4.2 and the specific requirements of this section.

5.6.3.3.2 Linear Static and Dynamic Procedures

Lower-bound compressive strength, P_{CL}, of braces in eccentric braced frames shall be calculated as for columns in accordance with Section 5.5.2.3.2 except that lower-bound yield strength, F_{yLB}, shall be used for yield strength.

Expected, Q_{CE}, and lower bound, Q_{CL}, strengths of beams and columns shall be calculated as for FR beams and columns in Section 5.5.2.3. Strength of beams with non-negligible axial load shall be as calculated for FR columns.

The lower-bound strength of brace connections shall be calculated in accordance with AISC (1993) *LRFD Specifications*, taking ϕ=1.0 and using the lower-bound yield strength, F_{yLB}, for yield strength.

The strength of the link beam shall be governed by shear, flexure, or the combination of shear and flexure. M_{CE} shall be taken as the expected moment capacity and V_{CE} shall be taken as 0.6 $F_{ye}A_w$.

If $e \le \frac{1.6M_{CE}}{V_{CE}}$, Equation (5-31) shall be used to compute the expected strength of the link beam:

$$Q_{CE} = V_{CE} = 0.6F_{ye}A_w \quad (5\text{-}31)$$

If $e > \frac{2.6M_{CE}}{V_{CE}}$, Equation (5-32) shall be used to compute the expected strength of the link beam:

$$Q_{CE} = 2\frac{M_{CE}}{e} \quad (5\text{-}32)$$

Linear interpolation between Equations (5-31) and (5-32) shall be used for intermediate values of *e*.

5.6.3.3.3 Nonlinear Static Procedure

Strengths for EBFs shall be the same as those specified in Section 5.6.2.3.3 for CBFs. In lieu of relationships derived from experiment or analysis, the load-deformation behavior of each component, as depicted by Figure 5-1, shall be determined in accordance with Section 5.6.3.2.2.

5.6.3.3.4 Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component shall be determined experimentally.

5.6.3.4 Acceptance Criteria

5.6.3.4.1 General

Component acceptance criteria shall be computed in accordance with the general requirements of Section 5.4.2 and the specific requirements of this section.

Shear and flexure in link beams shall be considered deformation-controlled actions. All other actions, and actions on other EBF components, shall be considered force-controlled. Compression, tension, shear, and bending actions on brace connections including gusset plates, bolts, welds, and other connectors shall be considered force-controlled.

5.6.3.4.2 Linear Static and Dynamic Procedures

Design actions shall be compared with design strengths in accordance with Section 3.4.2. *m*-factors for steel components shall be selected from Table 5-5.

Link beams shall conform to the requirements of the AISC (1997) *Seismic Provisions* with regard to detailing. The brace connecting to a link beam shall be designed for 1.25 times the link strength to ensure link yielding without brace or column buckling. Where the link beam is attached to the column flange with full-pen welds, the provisions for these connections shall be the same as for FR frame full-pen connections. *m*-factors for flexure and shear in link beams shall be taken from Table 5-5.

C5.6.3.4.2 Linear Static and Dynamic Procedures

The acceptance criteria for full-penetration welded beam-to-column connections is based on testing of typical moment frame proportioning and span ratios.

5.6.3.4.3 Nonlinear Static and Dynamic Procedures

Calculated component actions shall satisfy the requirements of Section 3.4.3. Deformations limits shall be taken from Tables 5-6 and 5-7.

5.6.3.5 Rehabilitation Measures

Eccentric braced frame components that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 5.4.3 and other provisions of this standard.

C5.6.3.5 Rehabilitation Measures

The rehabilitation measures described in C5.4.2.4 for FR moment frames and in C5.5.2.4 for CBFs may be effective for many of the beams, columns, and braces. Cover plates and/or stiffeners may be effective in rehabilitating these components. The strength of the link may be increased by adding cover plates to the beam flange(s), adding doubler plates or stiffeners to the web, or changing the brace configuration.

5.7 Steel Plate Shear Walls

5.7.1 General

A steel plate shear wall, with or without perforations, shall be provided with boundary members on all four sides and shall be welded to these boundary elements. The steel plate walls shall be designed to resist seismic loads acting alone or in conjunction with other existing lateral load resisting elements. The boundary elements shall be evaluated as beams and/or columns.

C5.7.1 General

A steel plate wall develops its seismic resistance through shear stress in the plate wall.

Although steel plate walls are not common, they have been used to rehabilitate a few essential structures where Immediate Occupancy and operation of a facility is mandatory after a large earthquake. Due to their stiffness, the steel plate walls attract much of the seismic shear. It is essential that the new load paths be carefully established.

The provisions for steel plate walls in this standard assume that the plates are sufficiently stiffened to prevent buckling. The design professional is referred to Timler (2000) for additional information regarding the behavior and design of steel plate shear walls.

5.7.2 Stiffness

5.7.2.1 Linear Static and Dynamic Procedures

Use of a plane stress finite element with beams and columns as boundary elements to analyze a steel plate shear wall shall be permitted. The global stiffness of the wall, K_w, shall be calculated in accordance with Equation (5-33) unless another method based on principles of mechanics is used.

$$K_w = \frac{Ga\ t_w}{h} \tag{5-33}$$

where:

G = Shear modulus of steel
a = Clear width of wall between columns
h = Clear height of wall between beams
t_w = Thickness of plate wall

5.7.2.2 Nonlinear Static Procedure

The elastic stiffness of the load-deformation relationship for the wall shall be as specified in Section 5.7.2.1. The complete nonlinear load-deformation relationship shall be based on experiment or approved rational analysis. Alternatively, use of the generalized load-deformation relationship shown in Figure 5-1, as specified in Section 5.5.2.2.2, shall be permitted.

5.7.2.3 Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component shall be modeled by a rational procedure verified by experiment.

C5.7.2.3 Nonlinear Dynamic Procedure

This procedure is not recommended in most cases.

5.7.3 Strength

5.7.3.1 General

Component strengths shall be computed in accordance with the general requirements of Section 5.4.2 and the specific requirements of this section.

5.7.3.2 Linear Static and Dynamic Procedures

The expected strength of the steel wall, Q_{CE}, shall be determined using the applicable equations in Part 6 of AISC (1993) *LRFD Specifications*, with ϕ=1.0 and the expected yield strength, F_{ye}, substituted for F_y. The wall shall be permitted to be modeled as the web of a plate girder. If stiffeners are provided to prevent buckling, they shall be spaced such that buckling of the wall does not occur and the expected strength of the wall shall be determined by Equation (5-34):

$$Q_{CE} = V_{CE} = 0.6 F_{ye} a t_w \tag{5-34}$$

where:

a = clear width of the wall between columns.

5.7.3.3 Nonlinear Static and Dynamic Procedures

The generalized load-deformation curve shown in Figure 5-1, as specified in Section 5.5.2.2.2, shall be used to represent the complete load-deformation behavior of the steel shear wall to failure unless another load-deformation relationship based on experiment or approved rational analysis verified by experiment is used. The expected strength, Q_{CE}, shall be calculated in accordance with Equation (5-34).The yield deformation shall be calculated in accordance with Equation (5-35):

$$\Delta_y = \frac{Q_{CE}}{K_w} \tag{5-35}$$

5.7.4 Acceptance Criteria

5.7.4.1 Linear Static and Dynamic Procedures

Design actions shall be compared with design strengths in accordance with Section 3.4.2. *m*-factors for steel components shall be selected from Table 5-5.

Shear behavior in steel plate shear walls shall be considered a deformation-controlled action, with acceptance criteria as provided in Table 5-5. Design restrictions for plate girder webs given in AISC (1993) *LRFD Specifications*, including those related to stiffener spacing, shall be followed.

In lieu of providing stiffeners, the steel wall shall be permitted to be encased in concrete. If buckling is not prevented by the use of stiffeners, equations for V_{CE} given in AISC (1993) *LRFD Specifications* for plate girders shall be used to calculate the expected strength of the wall.

5.7.4.2 Nonlinear Static and Dynamic Procedures

Calculated component actions shall satisfy the requirements of Section 3.4.3. Deformation limits shall be taken from Tables 5-6 and 5-7.

5.7.5 Rehabilitation Measures

Steel plate walls that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 5.4.3 and other provisions of this standard.

C5.7.5 Rehabilitation Measures

Rehabilitation measures may include the addition of stiffeners, encasement in concrete, or the addition of concrete or steel plate shear walls.

5.8 Steel Frames with Infills

Steel frames with partial or complete infills of reinforced concrete or reinforced or unreinforced masonry shall be evaluated considering the combined stiffness of the steel frame and infill material.

The engineering properties and acceptance criteria for the infill walls shall comply with the requirements in Chapter 6 for concrete and Chapter 7 for masonry. Infill walls and frames shall be considered to carry the seismic force in composite action, considering the relative stiffness of each element, until complete failure of the walls has occurred. The interaction between the steel frame and infill shall be considered using procedures specified in Chapter 6 for concrete frames with infill. The analysis of each component shall be done in stages, considering the effects of interaction between the elements and carried through each performance level. At the point where the infill has been deemed to fail, as determined by the acceptance criteria specified in Chapter 6 or Chapter 7, the wall shall be removed from the analytical model. The analysis shall be resumed on the bare steel frame taking into consideration any vertical discontinuity created by the degraded wall. At this point, the engineering properties and acceptance criteria for the frame, as specified in Section 5.5, shall apply.

C5.8 Steel Frames with Infills

Seismic evaluation of infill walls is required because, in many cases, these walls are unreinforced or lightly reinforced, and their strength and ductility may be inadequate. Before the loss of the wall, the steel frame adds confining pressure to the wall and enhances its resistance. The actual effective forces on the steel frame components, however, are probably minimal. As the frame components attempt to develop force they deform and the stiffer concrete or masonry components on the far side of the member pick up load. However, beam end connections, column splices, and steel frame connections at the foundation should be investigated for forces due to interaction with the infill similar to procedures specified for concrete frames in Chapter 6.

The stiffness and resistance provided by concrete and/or masonry infills may be much larger than the stiffness of the steel frame acting alone with or without composite actions. Gaps or incomplete contact between the steel frame and the infill may negate some or all of this stiffness. These gaps may be between the wall and columns of the frame or between the wall and the top beam enclosing the frame. Different strength and stiffness conditions must be expected with different discontinuity types and locations. Therefore, the presence of any gaps or discontinuities between the infill walls and the frame must be determined and considered in the design and rehabilitation process. The resistance provided by infill walls may also be included if proper evaluation of the connection and interaction between the wall and the frame is made and if the strength, ductility, and properties of the wall are properly included.

The stiffness provided by infill masonry walls is excluded from the design and rehabilitation process unless integral action between the steel frame and the wall is verified. If complete or partial interaction between the wall and frame is verified, the stiffness is increased accordingly. The seismic performance of unconfined masonry walls is far inferior to that of confined masonry walls; therefore, the resistance of the attached wall can be used only if strong evidence as to its strength, ductility, and interaction with the steel frame is provided.

Table 5-5 Acceptance Criteria for Linear Procedures—Structural Steel Components

	m-factors for Linear Procedures[1]				
		Primary		Secondary	
Component/Action	**IO**	**LS**	**CP**	**LS**	**CP**
Beams – flexure					
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_{ye}}}$	2	6	8	10	12
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{640}{\sqrt{F_{ye}}}$	1.25	2	3	3	4
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used				
Columns – flexure [12, 13]					
For $P/P_{CL} < 0.20$					
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{300}{\sqrt{F_{ye}}}$	2	6	8	10	12
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{460}{\sqrt{F_{ye}}}$	1.25	1.25	2	2	3
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used				

Table 5-5 Acceptance Criteria for Linear Procedures—Structural Steel Components (continued)

Component/Action	*m*-factors for Linear Procedures[1]				
		Primary		Secondary	
	IO	LS	CP	LS	CP
For $0.2 < P/P_{CL} < 0.50$					
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{260}{\sqrt{F_{ye}}}$	1.25	—[2]	—[3]	—[4]	—[5]
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{400}{\sqrt{F_{ye}}}$	1.25	1.25	1.5	2	2
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used				
Column Panel Zones – Shear	1.5	8	11	12	12
Fully Restrained Moment Connections[15]					
WUF[14]	1.0	4.3-0.083*d*	3.9-0.043*d*	4.3-0.048*d*	5.5-0.064*d*
Bottom haunch in WUF with slab	1.6	2.7	3.4	3.8	4.7
Bottom haunch in WUF without slab	1.3	2.1	2.5	2.8	3.3
Welded cover plate in WUF[14]	2.4-0.030*d*	4.3-0.067*d*	5.4-0.090*d*	5.4-0.090*d*	6.9-0.118*d*
Improved WUF-bolted web[14]	1.4-0.008*d*	2.3-0.021*d*	3.1-0.032*d*	4.9-0.048*d*	6.2-0.065*d*
Improved WUF-welded web	2.0	4.2	5.3	5.3	6.7
Free flange[14]	2.7-0.032*d*	6.3-0.098*d*	8.1-0.129*d*	8.4-0.129*d*	11.0-0.172*d*
Reduced beam section[14]	2.2-0.008*d*	4.9-0.025*d*	6.2-0.032*d*	6.5-0.025*d*	8.4-0.032*d*
Welded flange plates					
a. Flange plate net section	1.7	3.3	4.1	5.7	7.3
b. Other limit states	force-controlled				
Welded bottom haunch	1.6	3.1	3.8	4.6	5.9
Welded top and bottom haunches	1.6	3.1	3.9	4.7	6.0
Welded cover-plated flanges	1.7	2.8	3.4	3.4	4.2

Table 5-5 Acceptance Criteria for Linear Procedures—Structural Steel Components (continued)

Component/Action	*m*-factors for Linear Procedures[1]				
		Primary		Secondary	
	IO	LS	CP	LS	CP
Partially Restrained Moment Connections					
Top and bottom clip angle[8]					
a. Shear failure of rivet or bolt (Limit State 1)[9]	1.5	4	6	6	8
b. Tension failure of horizontal leg of angle (Limit State 2)	1.25	1.5	2	1.5	2
c. Tension failure of rivet or bolt (Limit State 3)[9]	1.25	1.5	2.5	4	4
d. Flexural failure of angle (Limit State 4)	2	5	7	7	14
Double split tee[8]					
a. Shear failure of rivet or bolt (Limit State 1)[9]	1.5	4	6	6	8
b. Tension failure of rivet or bolt (Limit State 2)[9]	1.25	1.5	2.5	4	4
c. Tension failure of split tee stem (Limit State 3)	1.25	1.5	2	1.5	2
d. Flexural failure of split tee (Limit State 4)	2	5	7	7	14
Bolted flange plate[8]					
a. Failure in net section of flange plate or shear failure of bolts or rivets[9]	1.5	4	5	4	5
b. Weld failure or tension failure on gross section of plate	1.25	1.5	2	1.5	2
Bolted end plate					
a. Yield of end plate	2	5.5	7	7	7
b. Yield of bolts	1.5	2	3	4	4
c. Failure of weld	1.25	1.5	2	3	3
Composite top and clip angle bottom [8]					
a. Failure of deck reinforcement	1.25	2	3	4	6
b. Local flange yielding and web crippling of column	1.5	4	6	5	7
c. Yield of bottom flange angle	1.5	4	6	6	7
d. Tensile yield of rivets or bolts at column flange	1.25	1.5	2.5	2.5	3.5
e. Shear yield of beam flange connections	1.25	2.5	3.5	3.5	4.5
Shear connection with slab[14]	$1.6\text{-}0.005d_{bg}$	--	--	$13.0\text{-}0.290d_{bg}$	$17.0\text{-}0.387d_{bg}$
Shear connection without slab[14]	$4.9\text{-}0.097d_{bg}$	--	--	$13.0\text{-}0.290d_{bg}$	$17.0\text{-}0.387d_{bg}$

Table 5-5 Acceptance Criteria for Linear Procedures—Structural Steel Components (continued)

	m-factors for Linear Procedures[1]				
		Primary		Secondary	
Component/Action	**IO**	**LS**	**CP**	**LS**	**CP**
EBF Link Beam[7, 10]					
a. $e \leq \frac{1.6 M_{CE}}{V_{CE}}$	1.5	9	13	13	15
b. $e \geq \frac{2.6 M_{CE}}{V_{CE}}$	Same as for beams.				
c. $\frac{1.6 M_{CE}}{V_{CE}} < e < \frac{2.6 M_{CE}}{V_{CE}}$	Linear interpolation shall be used.				
Braces in Compression (except EBF braces)					
a. Double angles buckling in-plane	1.25	6	8	7	9
b. Double angles buckling out-of-plane	1.25	5	7	6	8
c. W or I shape	1.25	6	8	6	8
d. Double channels buckling in-plane	1.25	6	8	7	9
e. Double channels buckling out-of-plane	1.25	5	7	6	8
f. Concrete-filled tubes	1.25	5	7	5	7
g. Rectangular cold-formed tubes					
1. $\frac{d}{t} \leq \frac{90}{\sqrt{F_y}}$	1.25	5	7	5	7
2. $\frac{d}{t} \geq \frac{190}{\sqrt{F_y}}$	1.25	2	3	2	3
3. $\frac{90}{\sqrt{F_y}} \leq \frac{d}{t} \leq \frac{190}{\sqrt{F_y}}$	Linear interpolation shall be used.				
h. Circular hollow tubes					
1. $\frac{d}{t} \leq \frac{1500}{F_y}$	1.25	5	7	5	7

Table 5-5 Acceptance Criteria for Linear Procedures—Structural Steel Components (continued)

Component/Action	*m*-factors for Linear Procedures[1]				
		Primary		Secondary	
	IO	LS	CP	LS	CP
2. $\frac{d}{t} \geq \frac{6000}{F_y}$	1.25	2	3	2	3
3. $\frac{1500}{F_y} \leq \frac{d}{t} \leq \frac{6000}{F_y}$	Linear interpolation shall be used.				
Braces in Tension (except EBF braces)[16]	1.25	6	8	8	10
Beams, Columns in Tension (except EBF beams, columns)	1.25	3	5	6	7
Steel Plate Shear Walls[11]	1.5	8	12	12	14
Diaphragm Components					
Diaphragm shear yielding or panel or plate buckling	1.25	2	3	2	3
Diaphragm chords and collectors—full lateral support	1.25	6	8	6	8
Diaphragm chords and collectors—limited lateral support	1.25	2	3	2	3

1. For built-up members where the lacing plates do not meet the requirements of Section 5.6.2.4.2, divide *m*-factors by 2.0, but values need not be less than 1.0.
2. $m=9(1-1.7\ P/P_{CL})$.
3. $m=12(1-1.7\ P/P_{CL})$.
4. $m=15(1-1.7\ P/P_{CL})$.
5. $m=18(1-1.7\ P/P_{CL})$.
6. Not used.
7. Values are for link beams with three or more web stiffeners. If no stiffeners, divide values by 2.0, but values need not be less than 1.25. Linear interpolation shall be used for one or two stiffeners.
8. Web plate or stiffened seat shall be considered to carry shear. Without shear connection, action shall not be classified as secondary. If $d_b > 18$ inches, multiply *m*-factors by $18/d_b$, but values need not be less than 1.0.
9. For high-strength bolts, divide values by 2.0, but values need not be less than 1.25.
10. Assumes ductile detailing for flexural link, in accordance with AISC (1995) *LRFD Specifications*.
11. Applicable if stiffeners, or concrete backing, is provided to prevent buckling.
12. Columns in moment or braced frames shall be permitted to be designed for the maximum force delivered by connecting members. For rectangular or square columns, replace $b_t/2t_f$ with b/t, replace 52 with 110, and replace 65 with 190.
13. Columns with $P/P_{CL} > 0.5$ shall be considered force-controlled.
14. d is the beam depth; d_{bg} is the depth of the bolt group.
15. Tabulated values shall be modified as indicated in Section 5.5.2.4.2, item 4.
16. For tension-only bracing, m-factors shall be divided by 2.0.

Table 5-6 ***Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components***

	Modeling Parameters			Acceptance Criteria				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
Component/Action	**a**	**b**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
Beams—flexure								
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{640}{\sqrt{F_{ye}}}$	$4\theta_y$	$6\theta_y$	0.2	$0.25\theta_y$	$2\theta_y$	$3\theta_y$	$3\theta_y$	$4\theta_y$
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used							
Columns—flexure [2, 7]								
For $P/P_{CL} < 0.20$								
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{300}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$
b. $d\ \frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{460}{\sqrt{F_{ye}}}$	$4\theta_y$	$6\theta_y$	0.2	$0.25\theta_y$	$2\theta_y$	$3\theta_y$	$3\theta_y$	$4\theta_y$
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used							

Table 5-6 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components (continued)

Component/Action	Modeling Parameters			Acceptance Criteria				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
	a	b	c	IO	LS	CP	LS	CP
For $0.2 < P/P_{CL} < 0.50$								
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{260}{\sqrt{F_{ye}}}$	—[3]	—[4]	0.2	$0.25\theta_y$	—[5]	—[3]	—[6]	—[4]
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{400}{\sqrt{F_{ye}}}$	$1\theta_y$	$1.5\theta_y$	0.2	$0.25\theta_y$	$0.5\theta_y$	$0.8\theta_y$	$1.2\theta_y$	$1.2\theta_y$
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used							
Column Panel Zones	$12\theta_y$	$12\theta_y$	1.0	$1\theta_y$	$8\theta_y$	$11\theta_y$	$12\theta_y$	$12\theta_y$
Fully Restrained Moment Connections[13]								
WUF[12]	0.051-0.0013*d*	0.043-0.0006*d*	0.2	0.0128-0.0003*d*	0.0337-0.0009*d*	0.0284-0.0004*d*	0.0323-0.0005*d*	0.043-0.0006*d*
Bottom haunch in WUF with slab	0.026	0.036	0.2	0.0065	0.0172	0.0238	0.0270	0.036
Bottom haunch in WUF without slab	0.018	0.023	0.2	0.0045	0.0119	0.0152	0.0180	0.023
Welded cover plate in WUF[12]	0.056-0.0011*d*	0.056-0.0011*d*	0.2	0.0140-0.0003*d*	0.0319-0.0006*d*	0.0426-0.0008*d*	0.0420-0.0008*d*	0.056-0.0011*d*
Improved WUF-bolted web[12]	0.021-0.0003*d*	0.050-0.0006*d*	0.2	0.0053-0.0001*d*	0.0139-0.0002*d*	0.0210-0.0003*d*	0.0375-0.0005*d*	0.050-0.0006*d*
Improved WUF-welded web	0.041	0.054	0.2	0.0103	0.0312	0.0410	0.0410	0.054
Free flange[12]	0.067-0.0012*d*	0.094-0.0016*d*	0.2	0.0168-0.0003*d*	0.0509-0.0009*d*	0.0670-0.0012*d*	0.0705-0.0012*d*	0.094-0.0016*d*
Reduced beam section[12]	0.050-0.0003*d*	0.070-0.0003*d*	0.2	0.0125-0.0001*d*	0.0380-0.0002*d*	0.0500-0.0003*d*	0.0525-0.0002*d*	0.07-0.0003*d*
Welded flange plates								
a. Flange plate net section	0.03	0.06	0.2	0.0075	0.0228	0.0300	0.0450	0.06
b. Other limit states	force-controlled							
Welded bottom haunch	0.027	0.047	0.2	0.0068	0.0205	0.0270	0.0353	0.047
Welded top and bottom haunches	0.028	0.048	0.2	0.0070	0.0213	0.0280	0.0360	0.048
Welded cover-plated flanges	0.031	0.031	0.2	0.0078	0.0177	0.0236	0.0233	0.031

Table 5-6 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components (continued)

Component/Action	Modeling Parameters			Acceptance Criteria				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
	a	b	c	IO	LS	CP	LS	CP
Partially Restrained Moment Connections								
Top and bottom clip angle[9]								
a. Shear failure of rivet or bolt (Limit State 1)[8]	0.036	0.048	0.200	0.008	0.020	0.030	0.030	0.040
b. Tension failure of horizontal leg of angle (Limit State 2)	0.012	0.018	0.800	0.003	0.008	0.010	0.010	0.015
c. Tension failure of rivet or bolt (Limit State 3)[8]	0.016	0.025	1.000	0.005	0.008	0.013	0.020	0.020
d. Flexural failure of angle (Limit State 4)	0.042	0.084	0.200	0.010	0.025	0.035	0.035	0.070
Double split tee[9]								
a. Shear failure of rivet or bolt (Limit State 1)[8]	0.036	0.048	0.200	0.008	0.020	0.030	0.030	0.040
b. Tension failure of rivet or bolt (Limit State 2)[8]	0.016	0.024	0.800	0.005	0.008	0.013	0.020	0.020
c. Tension failure of split tee stem (Limit State 3)	0.012	0.018	0.800	0.003	0.008	0.010	0.010	0.015
d. Flexural failure of split tee (Limit State 4)	0.042	0.084	0.200	0.010	0.025	0.035	0.035	0.070
Bolted flange plate[9]								
a. Failure in net section of flange plate or shear failure of bolts or rivets[8]	0.030	0.030	0.800	0.008	0.020	0.025	0.020	0.025
b. Weld failure or tension failure on gross section of plate	0.012	0.018	0.800	0.003	0.008	0.010	0.010	0.015
Bolted end plate								
a. Yield of end plate	0.042	0.042	0.800	0.010	0.028	0.035	0.035	0.035
b. Yield of bolts	0.018	0.024	0.800	0.008	0.010	0.015	0.020	0.020
c. Failure of weld	0.012	0.018	0.800	0.003	0.008	0.010	0.015	0.015
Composite top clip angle bottom [9]								
a. Failure of deck reinforcement	0.018	0.035	0.800	0.005	0.010	0.015	0.020	0.030
b. Local flange yielding and web crippling of column	0.036	0.042	0.400	0.008	0.020	0.030	0.025	0.035

Table 5-6 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components (continued)

	Modeling Parameters			Acceptance Criteria				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
Component/Action	**a**	**b**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
c. Yield of bottom flange angle	0.036	0.042	0.200	0.008	0.020	0.030	0.025	0.035
d. Tensile yield of rivets or bolts at column flange	0.015	0.022	0.800	0.005	0.008	0.013	0.013	0.018
e. Shear yield of beam flange connection	0.022	0.027	0.200	0.005	0.013	0.018	0.018	0.023
Shear connection with slab[12]	0.029-0.0002d_{bg}	0.15-0.0036d_{bg}	0.400	0.0073-0.0001d_{bg}	--	--	0.1125-0.0027d_{bg}	0.15-0.0036d_{bg}
Shear connection without slab[12]	0.15-0.0036d_{bg}	0.15-0.0036d_{bg}	0.400	0.0375-0.0009d_{bg}	--	--	0.1125-0.0027d_{bg}	0.15-0.0036d_{bg}
EBF Link Beam[10, 11]								
a. $e \leq \frac{1.6\, M_{CE}}{V_{CE}}$	0.15	0.17	0.8	0.005	0.11	0.14	0.14	0.16
b. $e \geq \frac{2.6\, M_{CE}}{V_{CE}}$	Same as for beams.							
c. $\frac{.6\, M_{CE}}{V_{CE}} < e < \frac{2.6\, M_{CE}}{V_{CE}}$	Linear interpolation shall be used.							
Steel Plate Shear Walls [1]	$14\theta_y$	$16\theta_y$	0.7	$0.5\theta_y$	$10\theta_y$	$13\theta_y$	$13\theta_y$	$15\theta_y$

1. Values are for shear walls with stiffeners to prevent shear buckling.
2. Columns in moment or braced frames shall be permitted to be designed for the maximum force delivered by connecting members. For rectangular or square columns, replace $b_t/2t_f$ with b/t, replace 52 with 110, and replace 65 with 190.
3. Plastic rotation = 11 (1-1.7 P/P_{CL}) θ_y.
4. Plastic rotation = 17 (1-1.7 P/P_{CL}) θ_y.
5. Plastic rotation = 8 (1-1.7 P/P_{CL}) θ_y.
6. Plastic rotation = 14 (1-1.7 P/P_{CL}) θ_y.
7. Columns with $P/P_{CL} > 0.5$ shall be considered force-controlled.
8. For high-strength bolts, divide values by 2.0.
9. Web plate or stiffened seat shall be considered to carry shear. Without shear connection, action shall not be classified as secondary. If beam depth, $d_b > 18$ inches, multiply *m*-factors by $18/d_b$.
10. Deformation is the rotation angle between link and beam outside link or column.
11. Values are for link beams with three or more web stiffeners. If no stiffeners, divide values by 2.0. Linear interpolation shall be used for one or two stiffeners.
12. d is the beam depth; d_{bg} is the depth of the bolt group.
13. Tabulated values shall be modified as indicated in Section 5.5.2.4.2, item 4.

Table 5-7 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components

	Modeling Parameters			Acceptance Criteria				
				Plastic Deformation				
	Plastic Deformation		Residual Strength Ratio		Primary		Secondary	
Component/Action	**a**	**b**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
Braces in Compression (except EBF braces)[1]								
a. Double angles buckling in-plane	$0.5\Delta_c$	$9\Delta_c$	0.2	$0.25\Delta_c$	$5\Delta_c$	$7\Delta_c$	$7\Delta_c$	$8\Delta_c$
b. Double angles buckling out-of-plane	$0.5\Delta_c$	$8\Delta_c$	0.2	$0.25\Delta_c$	$4\Delta_c$	$6\Delta_c$	$6\Delta_c$	$7\Delta_c$
c. W or I shape	$0.5\Delta_c$	$8\Delta_c$	0.2	$0.25\Delta_c$	$5\Delta_c$	$7\Delta_c$	$7\Delta_c$	$8\Delta_c$
d. Double channels buckling in-plane	$0.5\Delta_c$	$9\Delta_c$	0.2	$0.25\Delta_c$	$5\Delta_c$	$7\Delta_c$	$7\Delta_c$	$8\Delta_c$
e. Double channels buckling out-of-plane	$0.5\Delta_c$	$8\Delta_c$	0.2	$0.25\Delta_c$	$4\Delta_c$	$6\Delta_c$	$6\Delta_c$	$7\Delta_c$
f. Concrete-filled tubes	$0.5\Delta_c$	$7\Delta_c$	0.2	$0.25\Delta_c$	$4\Delta_c$	$6\Delta_c$	$6\Delta_c$	$7\Delta_c$
g. Rectangular cold-formed tubes								
1. $\frac{d}{t} \le \frac{90}{\sqrt{F_y}}$	$0.5\Delta_c$	$7\Delta_c$	0.4	$0.25\Delta_c$	$4\Delta_c$	$6\Delta_c$	$6\Delta_c$	$7\Delta_c$
2. $\frac{d}{t} \ge \frac{190}{\sqrt{F_y}}$	$0.5\Delta_c$	$3\Delta_c$	0.2	$0.25\Delta_c$	$1\Delta_c$	$2\Delta_c$	$2\Delta_c$	$3\Delta_c$
3. $\frac{90}{\sqrt{F_y}} \le \frac{d}{t} \le \frac{190}{\sqrt{F_y}}$	Linear interpolation shall be used.							
h. Circular hollow tubes								
1. $\frac{d}{t} \le \frac{1500}{F_y}$	$0.5\Delta_c$	$9\Delta_c$	0.4	$0.25\Delta_c$	$4\Delta_c$	$6\Delta_c$	$5\Delta_c$	$8\Delta_c$
2. $\frac{d}{t} \ge \frac{6000}{F_y}$	$0.5\Delta_c$	$3\Delta_c$	0.2	$0.25\Delta_c$	$1\Delta_c$	$2\Delta_c$	$2\Delta_c$	$3\Delta_c$
3. $\frac{1500}{F_y} \le \frac{d}{t} \le \frac{6000}{F_y}$	Linear interpolation shall be used.							
Braces in Tension (except EBF braces)[2]	$11\Delta_T$	$14\Delta_T$	0.8	$0.25\Delta_T$	$7\Delta_T$	$9\Delta_T$	$11\Delta_T$	$13\Delta_T$
Beams, Columns in Tension (except EBF beams, columns)[2]	$5\Delta_T$	$7\Delta_T$	1.0	$0.25\Delta_T$	$3\Delta_T$	$5\Delta_T$	$6\Delta_T$	$7\Delta_T$

1. Δ_C is the axial deformation at expected buckling load.
2. Δ_T is the axial deformation at expected tensile yielding load.

5.9 Diaphragms

5.9.1 Bare Metal Deck Diaphragms

5.9.1.1 General

Metal deck diaphragms shall be composed of metal plate or gage thickness steel sheets formed in a repeating pattern with ridges and valleys. Decking units shall be attached to each other and to the structural steel supports by welds or by mechanical fasteners. Bare metal deck diaphragms shall be permitted to resist seismic loads acting alone or in conjunction with supplementary diagonal bracing complying with the requirements of Section 5.9.4. Steel frame elements, to which bare metal deck diaphragms are attached at their boundaries, shall be considered to be the chord and collector elements.

Criteria shall apply to existing diaphragms as well as to stiffened, strengthened, or otherwise rehabilitated diaphragms. Interaction of new and existing elements of rehabilitated diaphragms shall be evaluated to ensure stiffness compatibility. Load transfer mechanisms between new and existing diaphragm elements shall be evaluated.

C5.9.1.1 General

Bare metal deck diaphragms are usually used for roofs of buildings where there are very light gravity loads other than support of roofing materials. Load transfer to frame elements that act as chords or collectors in modern frames is through shear connectors, puddle welds, screws, or shot pins.

5.9.1.2 Stiffness

5.9.1.2.1 Linear Procedures

Metal deck diaphragms shall be classified as flexible, stiff, or rigid in accordance with Section 3.2.4. Flexibility factors for use in the analysis shall be calculated by an approved rational method.

C5.9.1.2.1 Linear Procedures

Flexibility factors for various types of metal decks are available from manufacturers' catalogs. In systems for which values are not available, values can be established by interpolating between the most representative systems for which values are available. Flexibility factors for use in the analysis can also be calculated using the SDI, *Steel Deck Institute Diaphragm Design Manual*.

5.9.1.2.2 Nonlinear Static Procedure

Inelastic properties of diaphragms shall not be included in inelastic seismic analyses if the weak link of the diaphragm is connection failure. Procedures for developing models for inelastic response of wood diaphragms in unreinforced masonry (URM) buildings shall be permitted for use as the basis of an inelastic model of a flexible metal diaphragm. A strain-hardening modulus of 3% shall be used in the post-elastic region.

5.9.1.3 Strength

The strength of bare metal deck diaphragms shall be determined in accordance with Section 5.4.2 and the specific requirements of the general requirements of this section.

Expected strength, Q_{CE}, for bare metal deck diaphragms shall be taken as two times allowable values specified in approved codes and standards, unless a larger value is justified by test data.

Lower-bound strengths, Q_{CL}, of welded connectors shall be as specified in the Welding Code for Sheet Steel, *AWS D1.3*, or other approved standard.

C5.9.1.3 Strength

Capacities of steel deck diaphragms are given in International Conference of Building Officials (ICBO) reports, in manufacturers' literature, or in the publications of the Steel Deck Institute (SDI). Where allowable stresses are given, these may be multiplied by 2.0 in lieu of information provided by the manufacturer or other knowledgeable sources.

Connections between metal decks and steel framing commonly use puddle welds. Connection capacities are provided in ICBO reports, manufacturers' data, the *SDI Manual*, or *AWS D1.3*. Other attachment systems, such as clips, are sometimes used.

5.9.1.4 Acceptance Criteria

Connections of bare metal deck diaphragms shall be considered force-controlled. Connection capacity shall be checked for the ability to transfer the total diaphragm reaction into the steel framing. Diaphragms that are governed by the capacity of the connections shall also be considered force-controlled. Bare metal deck diaphragms not governed by the capacity of the connections shall be considered deformation-controlled. *m*-factors for shear yielding or plate buckling shall taken from Table 5-5.

For the Life Safety Structural Performance Level, a loss of bearing support or anchorage of the deck shall not be permitted. For higher performance levels, the amount of damage to the connections shall not impair the load transfer between the diaphragm and the steel frame. Deformations shall not exceed the threshold of deflections that cause unacceptable damage to other elements (either structural or nonstructural) at specified performance levels.

C5.9.1.4 Acceptance Criteria

If bare deck capacity is controlled by connections to frame members or panel buckling, then inelastic action and ductility are limited and the deck should be considered to be a force-controlled member.

5.9.1.5 Rehabilitation Measures

Bare metal diaphragms that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 5.4.3 and other provisions of this standard.

C5.9.1.5 Rehabilitation Measures

The following measures may be effective in rehabilitating bare metal diaphragms:

1. Adding shear connectors for transfer of stress to chord or collector elements.
2. Strengthening existing chords or collectors by the addition of new steel plates to existing frame components.
3. Adding puddle welds or other shear connectors at panel perimeters.
4. Adding diagonal steel bracing to form a horizontal truss to supplement diaphragm strength.
5. Replacing nonstructural fill with structural concrete.
6. Adding connections between deck and supporting members.

5.9.2 Metal Deck Diaphragms with Structural Concrete Topping

5.9.2.1 General

Metal deck diaphragms with structural concrete topping, consisting of either a composite deck with indentations, or a noncomposite form deck and the concrete topping slab with reinforcement acting together, shall be permitted to resist diaphragm loads. The concrete fill shall be either normal or lightweight structural concrete, with reinforcing composed of wire mesh or reinforcing steel. Decking units shall be attached to each other and to structural steel supports by welds or by mechanical fasteners. The steel frame elements to which the topped metal deck diaphragm boundaries are attached shall be considered the chord and collector elements.

Criteria shall apply to existing diaphragms as well as new and rehabilitated diaphragms. Interaction of new and existing elements of rehabilitated diaphragms shall be evaluated for stiffness compatibility. Load transfer mechanisms between new and existing diaphragm components shall be considered in determining the flexibility of the diaphragm.

C5.9.2.1 General

Metal deck diaphragms with structural concrete topping are frequently used on floors and roofs of buildings where there are typical floor gravity loads. Concrete has structural properties that significantly add to diaphragm stiffness and strength. Concrete reinforcing ranges from light mesh reinforcement to a regular grid of small reinforcing bars (#3 or #4). Metal decking is typically composed of corrugated sheet steel from 22 ga. down to 14 ga. Rib depths vary from 1-1/2 to 3 inches in most cases. Attachment of the metal deck to the steel frame is usually accomplished using puddle welds at one to two feet on center. For composite behavior, shear studs are welded to the frame before the concrete is cast.

Load transfer to frame elements that act as chords or collectors in modern frames is usually through puddle welds or headed studs. In older construction where the frame is encased for fire protection, load transfer is made through bond.

5.9.2.2 Stiffness

5.9.2.2.1 Linear Procedures

For existing topped metal deck diaphragms, a rigid diaphragm assumption shall be permitted if the span-to-depth ratio is not greater than five-to-one. For greater span-to-depth ratios, and in cases with plan irregularities, diaphragm flexibility shall be explicitly included in the analysis in accordance with Section 3.2.4. Diaphragm stiffness shall be calculated using an approved method with a representative concrete thickness.

C5.9.2.2.1 Linear Procedures

Flexibility factors for topped metal decks are available from manufacturers' catalogs. For combinations in which values are not available, values can be established by interpolating between the most representative systems for which values are available. Flexibility factors for use in the analysis can also be calculated using the *SDI Manual*.

5.9.2.2.2 Nonlinear Procedures

Inelastic properties of diaphragms shall not be included in inelastic seismic analyses if the weak link in the diaphragm is connection failure. Procedures for developing models for inelastic response of wood diaphragms in unreinforced masonry (URM) buildings shall be permitted for use as the basis of an inelastic model of a flexible metal deck diaphragm with structural to concrete topping.

5.9.2.3 Strength

Capacities of metal deck diaphragms with structural concrete topping shall be established by an approved procedure

Alternatively, the expected strength, Q_{CE}, of topped metal deck diaphragms shall be taken as two times allowable values specified in approved codes and standards unless a larger value is justified by test data. Lower-bound strengths, Q_{CL}, of welded connectors shall be as specified in *AWS D1.3* or other approved standards. Lower-bound strengths, Q_{CL}, for headed stud connectors shall be as specified in AISC (1993) *LRFD Specifications*, with ϕ=1.0.

C5.9.2.3 Strength

Member capacities of steel deck diaphragms with structural concrete are given in manufacturers' catalogs, ICBO reports, or the *SDI Manual*. If composite deck capacity is controlled by shear connectors, inelastic action and ductility are limited. It would be expected that there would be little or no inelastic action in steel deck/concrete diaphragms, except in long span conditions; however, perimeter transfer mechanisms and collector forces must be considered to be sure this is the case. SDI calculation procedures or ICBO values with a multiplier of 2.0 should be used to bring allowable values to a strength level. Connector capacities may also be found in ICBO reports, manufacturers' data, or the *SDI Manual*.

5.9.2.4 Acceptance Criteria

Connections of metal deck diaphragms with structural concrete topping shall be considered force-controlled. Connection capacity shall be checked for the ability to transfer the total diaphragm reaction into the steel framing. Diaphragms that are governed by the capacity of the connections shall also be considered force-controlled. Topped metal deck diaphragms not governed by the capacity of the connections shall be considered deformation-controlled. *m*-factors for shear yielding shall be taken from Table 5-5.

For the Life Safety Structural Performance Level, a loss of bearing support or anchorage shall not be permitted. For higher performance levels, the amount of damage to the connections or cracking in concrete-filled slabs shall not impair the load transfer between the diaphragm and the steel frame. Deformations shall be limited to be below the threshold of deflections that cause damage to other elements (either structural or nonstructural) at specified performance levels. Acceptance criteria for collectors shall be as specified in Section 5.9.6.4.

Shear connectors for steel beams designed to act compositely with the slab shall have the capacity to transfer both diaphragm shears and composite beam shears. Where the beams are encased in concrete, use of bond between the steel and the concrete shall be permitted to transfer loads.

C5.9.2.4 Acceptance Criteria

Shear failure of topped metal deck diaphragms requires cracking of the concrete or tearing of the metal deck, so *m*-factors have been set at conservative levels.

5.9.2.5 Rehabilitation Measures

Metal deck diaphragms with structural concrete topping that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 5.4.3 and other provisions of this standard.

C5.9.2.5 Rehabilitation Measures

The following measures may be effective in rehabilitating metal deck diaphragms with structural concrete topping:

1. Adding shear connectors to transfer forces to chord or collector elements.
2. Strengthening existing chords or collectors by the addition of new steel plates to existing frame components, or attaching new plates directly to the slab by embedded bolts or epoxy.
3. Adding diagonal steel bracing to supplement diaphragm strength.

5.9.3 Metal Deck Diaphragms with Nonstructural Concrete Topping

5.9.3.1 General

Metal deck diaphragms with nonstructural concrete topping shall be evaluated as bare metal deck diaphragms, unless the strength and stiffness of the non-structural topping is substantiated through approved test data.

C5.9.3.1 General

Metal deck diaphragms with nonstructural concrete fill are typically used on roofs of buildings where there are very small gravity loads. The concrete fill, such as very lightweight insulating concrete (e.g., vermiculite), usually does not have usable structural properties and is most often unreinforced. Consideration of any composite action must be done with caution after extensive investigation of field conditions. Material properties, force transfer mechanisms, and other similar factors must be verified in order to include such composite action. Typically, the decks are composed of corrugated sheet steel from 22 gage down to 14 gage, and the rib depths vary from 9/16 to 3 inches in most cases.

5.9.3.2 Stiffness

5.9.3.2.1 Linear Procedures

The potential for composite action and modification of load distribution shall be considered. Interaction of new and existing elements of strengthened diaphragms shall be evaluated by maintaining stiffness compatibility between the two, and the load transfer mechanisms between the new and existing diaphragm elements shall be considered in determining the flexibility of the diaphragm. Similarly, the interaction of new diaphragms with existing frames shall be evaluated, as well as the load transfer mechanisms between them.

C5.9.3.2.1 Linear Procedures

Flexibility of the diaphragm will depend on the strength and thickness of the topping. It may be necessary to bound the solution in some cases using both rigid and flexible diaphragm assumptions.

5.9.3.2.2 Nonlinear Procedures

Inelastic properties of diaphragms shall not be included in inelastic seismic analyses if the weak link in the diaphragm is connection failure. Procedures for developing models for inelastic response of wood diaphragms in unreinforced masonry (URM) buildings shall be permitted as the basis of an inelastic model of a flexible bare metal deck diaphragm with nonstructural concrete topping.

5.9.3.3 Strength

Capacities of metal deck diaphragms with nonstructural topping shall be taken as specified for bare metal deck in Section 5.9.1. Capacities for welded and headed stud connectors shall be taken as specified in Section 5.9.2.3.

5.9.3.4 Acceptance Criteria

Connections of metal deck diaphragms with nonstructural concrete topping shall be considered force-controlled. Connection capacity shall be checked for the ability to transfer the total diaphragm reaction into the steel framing. Diaphragms that are governed by the capacity of the connections shall also be considered force-controlled. Topped metal deck diaphragms not governed by the capacity of the connections shall be considered deformation-controlled. *m*-factors for shear yielding or plate buckling shall taken from Table 5-5.

For the Life Safety Structural Performance Level, a loss of bearing support or anchorage shall not be permitted. For higher performance levels, the amount of damage to the connections or cracking in concrete filled slabs shall not impair the load transfer mechanism between the diaphragm and the steel frame. Deformations shall be limited to be below the threshold of deflections that cause damage to other elements (either structural or nonstructural) at specified performance levels.

C5.9.3.4 Acceptance Criteria

Generally, there should be little or no inelastic action in the diaphragms, provided the connections to the framing members are adequate. SDI calculation procedures should be used for strengths, or ICBO values with a multiplier of 2 should be used to bring allowable values to strength levels.

5.9.3.5 Rehabilitation Measures

Metal deck diaphragms with nonstructural concrete topping that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 5.4.3 and other provisions of this standard.

C5.9.3.5 Rehabilitation Measures

The following measures may be effective in rehabilitating metal deck diaphragms with nonstructural concrete topping:

1. Adding shear connectors to transfer forces to chord or collector elements.
2. Strengthening existing chords or collectors by the addition of new steel plates to existing frame components, or attaching new plates directly to the slab by embedded bolts or epoxy.
3. Adding puddle welds at panel perimeters of diaphragms.
4. Adding diagonal steel bracing to supplement diaphragm strength.
5. Replacing nonstructural fill with structural concrete.

5.9.4 Horizontal Steel Bracing (Steel Truss Diaphragms)

5.9.4.1 General

Horizontal steel bracing (steel truss diaphragms) shall be permitted to act as diaphragms independently or in conjunction with bare metal deck roofs. Where structural concrete fill is provided over the metal decking, relative rigidities between the steel truss and concrete systems shall be considered in the analysis.

Criteria shall apply to existing truss diaphragms, strengthened truss diaphragms, and new diaphragms.

Where steel truss diaphragms are added as part of a rehabilitation plan, interaction of new and existing elements of strengthened diaphragm systems (stiffness compatibility) shall be evaluated and the load transfer mechanisms between new and existing diaphragm elements shall be considered in determining the flexibility of the strengthened diaphragm.

Load transfer mechanisms between new diaphragm elements and existing frames shall be considered in determining the flexibility of the diaphragm/frame system.

C5.9.4.1 General

Steel truss diaphragm elements are typically found in conjunction with vertical framing systems that are of structural steel framing. Steel trusses are more common in long span situations, such as special roof structures for arenas, exposition halls, auditoriums, and industrial buildings. Diaphragms with a large span-to-depth ratios may often be stiffened by the addition of steel trusses. The addition of steel trusses for diaphragms identified to be deficient may provide a proper method of enhancement.

Horizontal steel bracing (steel truss diaphragms) may be made up of any of the various structural shapes. Often, the truss chord elements consist of wide flange shapes that also function as floor beams to support the gravity loads of the floor. For lightly loaded conditions, such as industrial metal deck roofs without concrete fill, the diagonal members may consist of threaded rod elements, which are assumed to act only in tension. For steel truss diaphragms with large loads, diagonal elements may consist of wide flange members, tubes, or other structural elements that will act in both tension and compression. Truss element connections are generally concentric, to provide the maximum lateral stiffness and ensure that the truss members act under pure axial load. These connections are generally similar to those of gravity-load-resisting trusses.

5.9.4.2 Stiffness

5.9.4.2.1 Linear Procedures

Truss diaphragm systems shall be modeled as horizontal truss elements (similar to braced steel frames) where axial stiffness controls deflections. Joints shall be permitted to be modeled as pinned except when joints provide moment resistance or where eccentricities exist at the connections. In such cases, joint rigidities shall be modeled. Flexibility of truss diaphragms shall be explicitly considered in distribution of lateral loads to vertical elements.

5.9.4.2.2 Nonlinear Procedures

Elastic properties of truss diaphragms shall be permitted in the model for inelastic seismic analyses. Inelastic models similar to those of braced steel frames shall be permitted.

5.9.4.3 Strength

Capacities of truss diaphragm members shall be calculated as specified for steel braced frame members in Section 5.6. Lateral support of truss diaphragm members provided by metal deck, with or without concrete fill, shall be considered in evaluation of truss diaphragm capacities. Gravity force effects shall be included in the calculations for those members that support gravity loads.

5.9.4.4 Acceptance Criteria

Force transfer mechanisms between various members of the truss at the connections, and between trusses and frame elements, shall be evaluated to verify the completion of the load path.

For the Life Safety Structural Performance Level, a loss of bearing support or anchorage shall not be permitted. For higher performance levels the amount of damage to the connections or bracing elements shall not result in the loss of the load transfer between the diaphragm and the steel frame. Deformations shall be limited to be below the threshold of deflections that cause damage to other elements (either structural or nonstructural) at specified performance levels.

5.9.4.4.1 Linear Procedures

Linear acceptance criteria for horizontal steel truss diaphragm components shall be as specified for concentric braced frames in Section 5.6.2.4 except that beam and column criteria need not be used. Use of *m*-factors specified for diagonal brace components, in lieu of those for beam and column components of braced frames, shall be permitted for strut and chord members in the truss.

5.9.4.4.2 Nonlinear Procedures

Nonlinear acceptance criteria for horizontal steel truss diaphragm components shall be as specified for concentric braced frames in Section 5.6.2.4 except that beam and column criteria need not be used. Use of plastic deformations specified for diagonal brace components, in lieu of those specified for beam and column components of braced frames, shall be permitted for strut and chord members in the truss.

5.9.4.5 Rehabilitation Measures

Steel truss diaphragms that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 5.4.3 and other provisions of this standard.

C5.9.4.5 Rehabilitation Measures

The following measures may be effective in rehabilitating steel truss diaphragms:

1. Diagonal components may be added to form additional horizontal trusses as a method of strengthening a weak existing diaphragm.

2. Existing chords components strengthened by the addition of shear connectors to enhance composite action.

3. Existing steel truss components strengthened by methods specified for braced steel frame members.

4. Truss connections strengthened by the addition of welds, new or enhanced plates, and bolts.

5. Structural concrete fill added to act in combination with steel truss diaphragms after verifying the effects of the added weight of concrete fill.

5.9.5 Archaic Diaphragms

5.9.5.1 General

Archaic diaphragms in steel buildings are those consisting of shallow brick arches that span between steel floor beams, with the arches packed tightly between the beams to provide the necessary resistance to thrust forces.

C5.9.5.1 General

Archaic steel diaphragm elements are almost always found in older steel buildings in conjunction with vertical systems of structural steel framing. The brick arches were typically covered with a very low-strength concrete fill, usually unreinforced. In many instances, various archaic diaphragm systems were patented by contractors.

5.9.5.2 Stiffness

5.9.5.2.1 Linear Procedures

Existing archaic diaphragm systems shall be modeled as a horizontal diaphragm with equivalent thickness of brick arches and concrete fill. Modeling of the archaic diaphragm as a truss with steel beams as tension elements and arches as compression elements shall be permitted. The flexibility of flexible archaic diaphragms shall be considered in calculating the distribution of lateral loads to vertical elements. Analysis results shall be evaluated to verify that diaphragm response remains elastic as assumed.

Interaction of new and existing elements of strengthened diaphragms shall be evaluated by checking the stiffness compatibility of the two in cases where steel trusses are added as part of a seismic upgrade. Load transfer mechanisms between new and existing diaphragm elements shall be considered in determining the flexibility of the strengthened diaphragm.

5.9.5.2.2 Nonlinear Procedures

Archaic diaphragms shall be required to remain in the elastic range unless otherwise approved.

C5.9.5.2.2 Nonlinear Procedures

Inelastic properties of archaic diaphragms should be chosen with caution for seismic analyses. For the case of archaic diaphragms, inelastic models similar to those of archaic timber diaphragms in unreinforced masonry buildings may be appropriate. Inelastic deformation limits of archaic diaphragms should be lower than those prescribed for a concrete-filled diaphragm.

5.9.5.3 Strength

Member capacities of archaic diaphragm components shall be permitted to be calculated, assuming no tension capacity exists for all components except steel beam members. Gravity force effects shall be included for components of these diaphragms. Force transfer mechanisms between various members and between frame elements shall be evaluated to verify the completion of the load path.

5.9.5.4 Acceptance Criteria

Archaic diaphragms shall be considered force-controlled. For the Life Safety Structural Performance Level, diaphragm deformations and displacements shall not lead to a loss of bearing support for the elements of the arches. For higher performance levels, the deformation due to diagonal tension shall not result in the loss of the load transfer mechanism. Deformations shall be limited below the threshold of deflections that cause damage to other elements (either structural or nonstructural) at specified performance levels. These values shall be established in conjunction with those for steel frames.

5.9.5.5 Rehabilitation Measures

Archaic diaphragms that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 5.4.3 and other provisions of this standard.

C5.9.5.5 Rehabilitation Measures

The following measures may be effective in rehabilitating archaic diaphragms:

1. Adding diagonal members to form a horizontal truss as a method of strengthening a weak archaic diaphragm.
2. Strengthening existing steel members by adding shear connectors to enhance composite action.
3. Removing weak concrete fill and replacing it with a structural concrete topping slab after verifying the effects of the added weight of concrete fill.

5.9.6 Chord and Collector Elements

5.9.6.1 General

Steel framing that supports the diaphragm shall be permitted as diaphragm chord and collector elements. When structural concrete is present, additional slab reinforcing shall be permitted to act as the chord or collector for tensile loads, while the slab carries chord or collector compression. When the steel framing acts as a chord or collector, it shall be attached to the deck with spot welds or by mechanical fasteners.

C5.9.6.1 General

When reinforcing acts as the chord or collector, load transfer occurs through bond between the reinforcing bars and the concrete.

5.9.6.2 Stiffness

Modeling assumptions specified for equivalent steel frame members in this chapter shall be used for chord and collector elements.

5.9.6.3 Strength

Capacities of structural steel chords and collectors shall be as specified for FR beams and columns in Section 5.5.2.3.2. Capacities for reinforcing steel embedded in concrete slabs and acting as chords or collectors shall be determined in accordance with the provisions of Chapter 6.

5.9.6.4 Acceptance Criteria

Inelastic action in chords and collectors shall be permitted if it is permitted in the diaphragm. Where such actions are permissible, chords and collectors shall be considered deformation-controlled. *m*-factors shall be taken from Table 5-5 and inelastic acceptance criteria shall be taken from FR beam and column components in Section 5.5. Where inelastic action is not permitted, chords and collectors shall be considered force-controlled components. Where chord and collector elements are force-controlled, Q_{UD} need not exceed the total force that can be delivered to the component by the expected strength of the diaphragm or the vertical elements of the lateral-force-resisting system. For the Life Safety Structural Performance Level, the deformations and displacements of chord and collector components shall not result in the loss of vertical support. For higher performance levels chords and collectors shall not impair the load path.

Welds and connectors joining the diaphragms to the chords and collectors shall be considered force-controlled. If all connections meet the acceptance criteria, the diaphragm shall be considered to prevent buckling of the chord member within the plane of the diaphragm. Where chords or collectors carry gravity loads in combination with seismic loads, they shall be checked as members with combined axial load and bending in accordance with Section 5.5.2.4.2.

5.9.6.5 Rehabilitation Measures

Chord and collector elements that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 5.4.3 and other provisions of this standard.

C5.9.6.5 Rehabilitation Measures

The following measures may be effective in rehabilitating chord and collector elements:

1. Strengthen the connection between diaphragms and chords or collectors.

2. Strengthen steel chords or collectors with steel plates attached directly to the slab with embedded bolts or epoxy, and strengthen slab chord or collectors with added reinforcing bars.

3. Add chord members.

5.10 Steel Pile Foundations

5.10.1 General

A pile shall provide strength and stiffness to the foundation either by bearing directly on soil or rock, by friction along the pile length in contact with the soil, or by a combination of these mechanisms. Foundations shall be evaluated as specified in Chapter 4. Concrete components of foundations shall conform with Chapter 6. The design of the steel piles shall comply with the requirements of this section.

C5.10.1 General

Steel piles of wide flange shape (H-piles) or structural tubes, with and without concrete infills, shall be permitted to be used to support foundation loads. Piles driven in groups should have a reinforced concrete pile cap to transfer loads from the superstructure to the piles.

5.10.2 Stiffness

If the pile cap is below grade, the foundation stiffness from the pile cap bearing against the soil shall be permitted to be represented by equivalent soil springs derived as specified in Chapter 4. Additional stiffness of the piles shall be permitted to be derived through bending and bearing against the soil. For piles in a group, the reduction in each pile's contribution to the total foundation stiffness and strength shall be made to account for group effects. Additional requirements for calculating the stiffness shall be as specified in Chapter 4.

5.10.3 Strength

Except in sites subject to liquefaction of soils, it shall be permitted to neglect buckling of portions of piles embedded in the ground. Flexural demands in piles shall be calculated either by nonlinear methods or by elastic methods for which the pile is treated as a cantilever column above a calculated point of fixity.

5.10.4 Acceptance Criteria

The acceptance criteria for the axial force and maximum bending moments for the pile strength shall be as specified for a steel column in Section 5.5.2.4.2 for linear methods and in Section 5.5.2.4.3 for nonlinear methods, where the lower-bound axial compression, expected axial tension and flexural strengths shall be computed for an unbraced length equal to zero for those portions of piles that are embedded in non-liquefiable soils.

Connections between steel piles and pile caps shall be considered force-controlled.

C5.10.4 Acceptance Criteria

Nonlinear methods require the use of a computer program. The design professional is referred to *FEMA 274* for additional information.

5.10.5 Rehabilitation Measures

Steel pile foundations that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section 5.4.3 and other provisions of this standard.

C5.10.5 Rehabilitation Measures

Rehabilitation of the concrete pile cap is specified in Chapter 6. Criteria for the rehabilitation of the foundation element are specified in Chapter 4. The following measure may be effective in rehabilitating steel pile foundations: driving additional piles near existing groups and then adding a new pile cap to increase stiffness and strength of the pile foundation. Monolithic behavior gained by connecting the new and old pile caps with epoxied dowels may also be effective. In most cases, it is not possible to rehabilitate the existing piles.

5.11 Cast and Wrought Iron

5.11.1 General

Existing components of cast and wrought iron shall be permitted to participate in resisting seismic forces in combination with concrete or masonry walls. Cast iron frames, in which beams and columns are integrally cast, shall not be permitted to resist seismic forces as primary elements of the lateral-force-resisting system. The ability of cast iron elements to resist the design displacements at the selected earthquake hazard level shall be evaluated.

5.11.2 Stiffness

The axial and flexural stiffness of cast iron shall be calculated using elastic section properties and a modulus of elasticity, E, of 25,000 kips per square inch.

5.11.3 Strength and Acceptance Criteria

Axial and flexural loads on cast iron components shall be considered to be force-controlled behaviors. Lower bound material properties for cast iron shall be based on Table 5-1.

The lower-bound strength of a cast iron column shall be calculated as:

$$Q_{CL} = P_{CL} = A_g F_{cr} \tag{5-36}$$

where:

A_g = Gross area of column

$F_{cr} = 12ksi$ for $l_c/r \le 108$

$F_{cr} = \dfrac{1.40\times10^5}{(l_c/r)^2} ksi$ for $l_c/r > 108$

Cast iron columns shall only be permitted to carry axial compression.