5. Steel and Cast Iron (Systematic Rehabilitation)

5.1 Scope

Rehabilitation measures for steel components and elements are described in this chapter. Information needed for systematic rehabilitation of steel buildings, as depicted in Step 4B of the Process Flow chart shown in Figure 1-1, is presented herein. A brief historical perspective is given in Section 5.2, with a more expanded version given in the *Commentary.*

Section 5.3 discusses material properties for new and existing construction, and describes material testing requirements for using the nonlinear procedures. A factor measuring the reliability of assumptions of in-place material properties is included in a kappa (κ) factor, used to account for accuracy of knowledge of the existing conditions. Evaluation methods for in-place materials are also described.

Sections 5.4 and 5.5 provide the attributes of steel moment frames and braced frames. The stiffness and strength properties of each steel component required for the linear and nonlinear procedures described in Chapter 3 are given. Stiffness and strength acceptance criteria are also given and are discussed within the context of Tables 2-1, 2-3, and 2-4, given in Chapter 2. These sections also provide guidance on choosing an appropriate rehabilitation strategy.

The appropriate procedures for evaluating systems with old and new components are discussed. Steel frames with concrete or masonry infills are briefly discussed, but the behavior of these systems and procedures for estimating the forces in the steel components are given in Chapters 6 (concrete) and 7 (masonry). Steel frames with attached masonry walls are discussed in this chapter and in Chapter 7.

Section 5.8 describes engineering properties for typical diaphragms found in steel buildings. These include bare metal deck, metal deck with composite concrete topping, noncomposite steel deck with concrete topping, horizontal steel bracing, and archaic diaphragms. The properties and behavior of wood diaphragms in steel buildings are presented in Chapter 8.

Engineering properties, and stiffness and strength acceptance criteria for steel piles are given in Section 5.9. Methods for calculating the forces in the piles are described in Chapter 4 and in the *Commentary* to Chapter 5.

5.2 Historical Perspective

The components of steel elements are columns, beams, braces, connections, link beams, and diaphragms. The 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 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 the *Commentary.*

As indicated in Chapter 1, 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

Quantification of in-place material properties and verification of the existing system configuration and condition are necessary to analyze or evaluate a building. This section identifies properties requiring consideration and provides guidelines for their acquisition. Condition assessment is an important aspect of planning and executing seismic rehabilitation of an existing building. One of the most important steps in condition assessment is a visit to the building for visual inspection.

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. The requirements given here are supplemental to those given in Section 2.7.

5.3.2 Properties of In-Place Materials and Components

5.3.2.1 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 yield (F_{ye}) and tensile (F_{te}) strengths of base and connection material, modulus of elasticity, ductility, toughness, elogational characteristics, and weldability. The term "expected strength" is used throughout this document in place of "nominal strength" since expected yield and tensile stresses are used in place of nominal values specified in AISC (1994a and b).

The effort required to determine these properties is related to the availability of original and updated construction documents, original quality of construction, accessibility, and condition of materials.

The determination of material properties is best accomplished through removal of samples and laboratory testing. Sampling may 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. Types and sizes of specimens should be in accordance with ASTM standards. Mechanical and metallurgical properties usually can be established from laboratory testing on the same sample. If a connector such as a bolt or rivet is removed for testing, a comparable bolt should be reinstalled at the time of sampling. Destructive removal of a welded connection sample must be accompanied by repair of the connection.

5.3.2.2 Component Properties

Behavior of components, including beams, columns, and braces, is dictated by such properties as area, width-to-thickness and slenderness ratios, lateral torsional buckling resistance, and connection details. Component properties of interest are:

- Original cross-sectional shape and physical dimensions
- Size and thickness of additional connected materials, including cover plates, bracing, and stiffeners
- Existing cross-sectional area, section moduli, moments of inertia, and torsional properties at critical sections
- As-built configuration of intermediate, splice, and end connections
- Current physical condition of base metal and connector materials, including presence of deformation.

Each of these properties is needed to characterize building performance in the seismic analysis. The starting point for establishing component properties should be construction documents. Preliminary review of these documents shall be performed to identify primary vertical- and lateral-load-carrying elements and systems, and their critical components and connections. In the absence of a complete set of building drawings, the design professional must direct a testing agency to perform a thorough inspection of the building to identify these elements and components as indicated in Section 5.3.3.

In the absence of degradation, statistical analysis has shown that mean component cross-sectional dimensions are comparable to the nominal published values by AISC, AISI, and other organizations. Variance in these dimensions is also small.

5.3.2.3 Test Methods to Quantify Properties

To obtain the desired in-place mechanical properties of materials and components, it is necessary to utilize proven destructive and nondestructive testing methods. To achieve the desired accuracy, mechanical properties should be determined in the laboratory. Particular laboratory test information that may be sought includes yield and tensile strength, elongation, and charpy notch toughness. For each test, industry standards published by the ASTM exist and shall be followed. The *Commentary* provides applicability information and references for these particular tests.

Of greatest interest to metal 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 this strength data. 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 is 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.

If any rehabilitative measures are needed and welded connection to existing components is required, the carbon equivalent of the existing component(s) shall be determined. Appropriate welding procedures are dependent upon the chemistry of base metal and filler material (for example, the elements in the IIW Carbon Equivalent formula). Consult Section 8 and its associated *Commentary* in the latest edition of ANSI/AWS D1.1 *Structural Welding Code*. Recommendations given in FEMA 267 (SAC, 1995) may also be followed.

5.3.2.4 Minimum Number of Tests

In order to quantify expected strength and other in-place properties accurately, it will sometimes be required that a minimum number of tests be conducted on representative components. As stated previously, the minimum number of tests is dictated by available data from original construction, the type of structural system employed, desired accuracy, and quality/condition of in-place materials. Access to the structural system will also be a factor in defining the testing program. As an alternative, the design professional may elect to utilize the default strength properties contained in Section 5.3.2.5 instead of the specified testing. However, in some cases these default values may only be used for a Linear Static Procedure (LSP).

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 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. It is recommended that the lower-bound default value for compressive strength of cast iron given in Table 5-1 be used.

The guidelines for determining the expected yield (F_{ye}) and tensile (F_{te}) strengths are given below.

- If original construction documents defining properties—including material test records or material test reports (MTR)—exist, material tests need not be carried out, at the discretion of the design professional. Default values from Table 5-2 may be used. Larger values may be used, at the discretion of the design professional, if available historical data substantiates them. Larger values should be used if the assumptions produce a larger demand on associated connections.

- If original construction documents defining properties are limited or do not exist, but the date of construction is known and the single material used is confirmed to be carbon steel, at least three strength coupons shall be randomly removed from each component type. Conservative material properties such as those given in Table 5-2 may be used in lieu of testing, at the discretion of the design professional.

- If no knowledge exists of the structural system and materials used, at least two strength tensile coupons should be removed from each component type for every four floors. If it is determined from testing that more than one material grade exists, additional testing should be performed until the extent of use for each grade in component fabrication has been established. If it is determined that all components are made from steel, the requirements immediately preceding this may be followed.

- In the absence of construction records defining welding filler metals and processes used, at least one weld metal sample for each construction type should be obtained for laboratory testing. The sample shall consist of both local base and weld metal, such that composite strength of the connection can be derived. Steel and weld filler material properties discussed in Section 5.3.2.3 should also be obtained. Because of the destructive nature and necessary repairs that follow, default strength properties may be substituted if original records on welding exist, unless the design professional requires more accurate data. 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. Special requirements for welded moment frames are given in FEMA 267 (SAC, 1995) and the latest edition of ANSI/AWS D1.1 *Structural Welding Code*.

- Testing requirements for bolts and rivets are the same as for other steel components as given above. In lieu of testing, default values from Table 5-2 may be used.

- For archaic materials, including wrought iron but excluding cast iron, at least three strength coupons shall be extracted for each component type for every four floors of construction. Should significant variability be observed, in the judgment of the design professional, additional tests shall be performed until an acceptable strength value is obtained. If initial tests provide material properties that are consistent with properties given in Table 5-1, tests are required only for every six floors of construction.

For all laboratory test results, the mean yield and tensile strengths may be interpreted as the expected strength for component strength calculations.

For other material properties, the design professional shall determine the particular need for this type of testing and establish an adequate protocol consistent with that given above. In general, it is recommended that a minimum of three tests be conducted.

If a higher degree of confidence in results is desired, the sample size shall be determined using ASTM Standard E22 guidelines. Alternatively, the prior knowledge of material grades from Section 5.3.2.5 may be used in conjunction with Bayesian statistics to gain greater confidence with the reduced sample sizes noted above. The design professional is encouraged to use the procedures contained in the *Commentary* in this regard.

5.3.2.5 Default Properties

The default expected strength values for key metallic material properties are contained in Tables 5-1 and 5-2. These values are conservative, representing mean values from previous research less two standard deviations. It is recommended that the results of any material testing performed be compared to values in these tables for the particular era of building construction. Additional testing is recommended if the expected yield and tensile strengths determined from testing are lower than the default values.

Default material strength properties may only be used in conjunction with Linear Static and Dynamic Procedures. For the nonlinear procedures, expected strengths determined from the test program given above shall be used. Nonlinear procedures may be used with the reduced testing requirements described in *Commentary* Section C5.3.2.5.

5.3.3 Condition Assessment

5.3.3.1 General

A condition assessment of the existing building and site conditions shall be performed as part of the seismic rehabilitation process. The goals of this assessment are:

- To examine the physical condition of primary and secondary components and the presence of any degradation

- To verify or determine the presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems

Table 5-1 Default Material Properties[1]

Early unit stresses used in tables of allowable loads as published in catalogs of the following mills

Year	**Rolling Mill**	**Expected Yield Strength, ksi**
	FOR CAST IRON[1]	
1873	Carnegie Kloman & Co. (“Factor of Safety 3”)	21
1874	New Jersey Steel & Iron Co.	18
1881–1884	Carnegie Brothers & Co., Ltd.	18 15
1884	The Passaic Rolling Mill Co.	18 15
1885	The Phoenix Iron Company	18
1885–1887	Pottsville Iron & Steel Co.	18
1889	Carnegie Phipps & Co., Ltd.	18 15
	FOR STEEL[1]	
1887	Pottsville Iron & Steel Co.	23
1889–1893	Carnegie Phipps & Co., Ltd.	24
1893–1908	Jones & Laughlins Ltd. Jones & Laughlins Steel Co.	24 18
1896	Carnegie Steel Co., Ltd.	24
1897–1903	The Passaic Rolling Mills Co.	24 18
1898–1919	Cambria Steel Co.	24 18
1900–1903	Carnegie Steel Company	24
1907–1911	Bethlehem Steel Co.	24
1915	Lackawanna Steel Co.	24 18

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

- To review other conditions—such as neighboring party walls and buildings, the presence of nonstructural components, and limitations for rehabilitation—that may influence building performance
- To formulate a basis for selecting a knowledge factor (see Section 5.3.4).

The physical condition of existing components and elements, and their connections, must be examined for presence of 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 shall 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 267

Table 5-2 Default Expected Material Strengths [1]

History of ASTM and AISC Structural Steel Specification Stresses

			ASTM Requirement	
Date	**Specification**	**Remarks**	**Expected Tensile Strength[2], F_{te}, ksi**	**Expected Yield Strength[2, 3] F_{ye}, ksi**
1900	ASTM, A9	Rivet Steel	50	30
	Buildings	Medium Steel	60	35
1901–1908	ASTM, A9	Rivet Steel	50	1/2 T.S.
	Buildings	Medium Steel	60	1/2 T.S.
1909–1923	ASTM, A9	Structural Steel	55	1/2 T.S.
	Buildings	Rivet Steel	48	1/2 T.S.
1924–1931	ASTM, A7	Structural Steel	55	1/2 T.S. or not less than 30
		Rivet Steel	46	1/2 T.S. or not less than 25
	ASTM, A9	Structural Steel	55	1/2 T.S. or not less than 30
		Rivet Steel	46	1/2 T.S. or not less than 25
1932	ASTM, A140-32T issued as a tentative revision to ASTM, A9 (Buildings)	Plates, Shapes, Bars	60	1/2 T.S. or not less than 33
		Eyebar flats unannealed	67	1/2 T.S. or not less than 36
1933	ASTM, A140-32T discontinued and ASTM, A9 (Buildings) revised Oct. 30, 1933	Structural Steel	55	1/2 T.S. or not less than 30
	ASTM, A9 tentatively revised to ASTM, A9-33T (Buildings)	Structural Steel	60	1/2 T.S. or not less than 33
	ASTM, A141-32T adopted as a standard	Rivet Steel	52	1/2 T.S. or not less than 28
1934 on	ASTM, A9	Structural Steel	60	1/2 T.S. or not less than 33
	ASTM, A141	Rivet Steel	52	1/2 T.S. or not less than 28

1. Duplicated from AISC “Iron and Steel Beams 1873 to 1952.”
2. Values shown in this table are based on mean minus two standard deviations and duplicated from “Statistical Analysis of Tensile Data for Wide-Flange Structural Shapes.” The values have been reduced by 10%, since originals are from mill tests.
3. T.S. = Tensile strength

Table 5-2 Default Expected Material Strengths [1] (continued)

Additional default assumptions

Date	Specification	Remarks	Expected Tensile Strength[2], F_{te}, ksi	Expected Yield Strength[2, 3] F_{ye}, ksi
1961 on	ASTM, A36	Structural Steel		
	Group 1		54	37
	Group 2		52	35
	Group 3		52	32
	Group 4		53	30
	Group 5		61	35
	ASTM, A572, Grade 50	Structural Steel		
	Group 1		56	41
	Group 2		57	42
	Group 3		60	44
	Group 4		62	43
	Group 5		71	44
	Dual Grade	Structural Steel		
	Group 1		59	43
	Group 2		60	43
	Group 3		64	46
	Group 4		64	44

1. Duplicated from AISC “Iron and Steel Beams 1873 to 1952.”
2. Values shown in this table are based on mean minus two standard deviations and duplicated from “Statistical Analysis of Tensile Data for Wide-Flange Structural Shapes.” The values have been reduced by 10%, since originals are from mill tests.
3. T.S. = Tensile strength

(SAC, 1995) 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 scope of a condition assessment shall include all primary structural elements and components involved in gravity and lateral load resistance. The degree of assessment performed also affects the κ factor that is used (see Section 5.3.4).

If coverings or other obstructions exist, indirect visual inspection through use of drilled holes and a fiberscope may be utilized. If this method is not appropriate, then local removal of covering materials will be necessary. The following guidelines shall be used.

- If detailed design drawings exist, exposure of at least one different primary connection shall occur for each connection type. If no deviations from the drawings exist, the sample may be considered representative. If deviations are noted, then removal of additional coverings from primary connections of that type must be done until the design professional has adequate knowledge to continue with the evaluation and rehabilitation.

- In the absence of construction drawings, the design professional shall establish inspection protocol that will provide adequate knowledge of the building needed for reliable evaluation and rehabilitation. For steel elements encased in concrete, it may be more cost effective to provide an entirely new lateral-load-resisting system.

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 shall 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 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 267 (SAC, 1995) for inspection of welded moment frames.

5.3.3.3 Quantifying Results

The results of the condition assessment shall be used in the preparation of building system models in the evaluation of seismic performance. To aid in this effort, the results shall be quantified and reduced, with the following specific topics addressed:

- Component section properties and dimensions
- Connection configuration and presence of any eccentricities
- Type and location of column splices
- Interaction of nonstructural components and their involvement in lateral load resistance

The acceptance criteria for existing components depends on the design professional’s knowledge of the condition of the structural system and material properties (as previously noted). All deviations noted between available construction records and as-built conditions shall be accounted for and considered in the structural analysis.

5.3.4 Knowledge (κ) Factor

As described in Section 2.7 and Tables 2-16 and 2-17, computation of component capacities and allowable deformations shall involve the use of a knowledge (κ) factor. For cases where a linear procedure will be used in the analysis, two categories of κ exist. This section further describes the requirements specific to metallic structural elements that must be accomplished in the selection of a κ factor.

A κ factor of 1.0 can be utilized when a thorough assessment is performed on the primary and secondary components and load path, and the requirements of Section 2.7 are met. The additional requirement for a κ factor of 1.0 is that the condition assessment be done in accordance with Section 5.3.3. In general, a κ factor of 1.0 may be used if the construction documents are available.

If the configuration and condition of an as-built component or connection are not adequately known (in the judgement of the design professional, because design documents are unavailable and it is deemed too costly to do a thorough condition assessment in accordance with Section 5.3.3), the κ factor used in the final component evaluation shall be reduced to 0.75. A κ factor of 0.75 shall be used for all cast and wrought iron components and their connectors. For encased components where construction documents are limited and knowledge of configuration and condition is incomplete, a factor of 0.75 shall be used. In addition, for steel moment and braced frames, the use of a κ factor of 0.75 shall occur when knowledge of connection details is incomplete. See also Section C2.7.2 in the *Commentary.*

5.4 Steel Moment Frames

5.4.1 General

Steel moment frames are those frames that develop their seismic resistance through bending of beams and columns and shearing of panel zones. Moment-resisting connections with calculable resistance are required between the members. The frames are categorized by the types of connection used and by the local and global stability of the members. Moment frames may act alone to resist seismic loads, or they may act in conjunction with concrete or masonry shear walls or braced steel frames to form a dual system. Special rules for design of new dual systems are included in AISC (1994a) and BSSC (1995).

Columns, beams, and connections are the components of moment frames. Beams and columns may be built-up members from plates, angles, and channels, cast or wrought iron segments, hot-rolled members, or cold-formed steel sections. Built-up members may be assembled by riveting, bolting, or welding. Connections between the members may be fully restrained (FR), partially restrained (PR), or nominally unrestrained (simple shear or pinned). The components may be bare steel, steel with a nonstructural coating for fire protection, or steel with either concrete or masonry encasement for fire protection.

Two types of frames are categorized in this document. Fully restrained (FR) moment frames are those frames for which no more than 5% of the lateral deflections arise from connection deformation. Partially restrained (PR) moment frames are those frames for which more than 5% of the lateral deflections result from connection deformation. In each case, the 5% value refers only to deflection due to beam-column deformation and not to frame deflections that result from column panel zone deformation.

5.4.2 Fully Restrained Moment Frames

5.4.2.1 General

Fully restrained (FR) moment frames are those moment frames with rigid connections. The connection shall be at least as strong as the weaker of the two members being joined. Connection deformation may contribute no more than 5% (not including panel zone deformation) to the total lateral deflection of the frame. If either of these conditions is not satisfied, the frame shall be characterized as partially restrained. 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 the *Commentary* and to FEMA 267 (SAC, 1995).

Fully restrained moment frames encompass both Special Moment Frames and Ordinary Moment Frames, defined in the Seismic Provisions for Structural Steel Buildings in Part 6 of AISC (1994a). These terms are not used in the *Guidelines*, but most of the requirements for these systems are reflected in AISC (1994a).

Requirements for general or seismic design of steel components given in AISC (1994a) or BSSC (1995) are to be followed unless superseded by provisions in these *Guidelines*. In all cases, the expected strength will be used in place of the nominal design strength by replacing F_y with F_{ye}.

5.4.2.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

Axial area. This is the complete area of rolled or built-up shapes. For built-up sections, the effective area should be reduced if adequate load transfer mechanisms are not available. For elements fully encased in concrete, the stiffness may be calculated assuming full composite action if most of the concrete may be expected to remain after the earthquake. Composite action may not be assumed for strength unless adequate load transfer and ductility of the concrete can be assured.

Shear area. This is based on standard engineering procedures. The above comments, related to built-up sections, concrete encased elements, and composite action of floor beam and slab, apply.

Moment of inertia. The calculation of rotational stiffness of steel beams and columns in bare steel frames shall follow standard engineering procedures. 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 adequate and identifiable shear transfer mechanism between the concrete and the steel.

Joint Modeling. Panel zone stiffness may be considered in a frame analysis by adding a panel zone element to the program. The beam flexural stiffness may also be adjusted to account for panel zone stiffness or flexibility and the stiffness of the concrete encasement. Use center line analysis for other cases. Strengthened members shall be modeled similarly to existing members. The approximate procedure suggested for calculation of stiffness of PR moment frames given below may be used to model panel zone effects, if available computer programs cannot explicitly model panel zones.

Connections. The modeling of stiffness for connections for FR moment frames is not required since, by definition, the frame displacements are not significantly

(<5%) affected by connection deformation. The strength of the connection must be great enough to carry the expected moment strength and resulting shear in the beam at a beam-to-column connection and shall be calculated using standard engineering procedures. Three types of connections are currently acknowledged as potentially fully restrained: (1) full penetration (full-pen) welds between the flanges of the beam and column flanges with bolted or welded shear connections between the column flange and beam web; (2) flange plate connections; and (3) end plate connections. If flange plate or end plate connections are too flexible or weak to be considered fully restrained, they must be considered to be partially restrained. Strength and stiffness properties for these two connections as PR connections are discussed in Section 5.4.3 and in the *Commentary.*

B. Nonlinear Static Procedure

- Use elastic component properties as outlined under Section 5.4.2.2A.

- Use appropriate nonlinear moment-curvature and interaction relationships for beams and beam-columns to represent plastification. These may be derived from experiment or analysis.

- Linear and nonlinear behavior of panel zones shall be included.

In lieu of a more rational analysis, the details of all segments of the load-deformation curve, as defined in Tables 5-4 and Figure 5-1 (an approximate, generalized, load-deformation curve for components of steel moment frames, braced frames, and plate walls), may be used. This curve may be modified by assuming a strain-hardening slope of 3% of the elastic slope. Larger strain-hardening slopes may be used if verified by experiment. If panel zone yielding occurs, a strain-hardening slope of 6% or larger should be used for the panel zone. It is recommended that strain hardening be considered for all components.

The parameters Q and Q_{CE} in Figure 5-1 are generalized component load and generalized component expected strength for the component. For beams and columns, θ is the plastic rotation of the beam or column, θ_y is the rotation at yield, Δ is displacement, and Δ_y is yield displacement. For panel zones, θ_y is the angular shear deformation in radians. Figure 5-2 defines

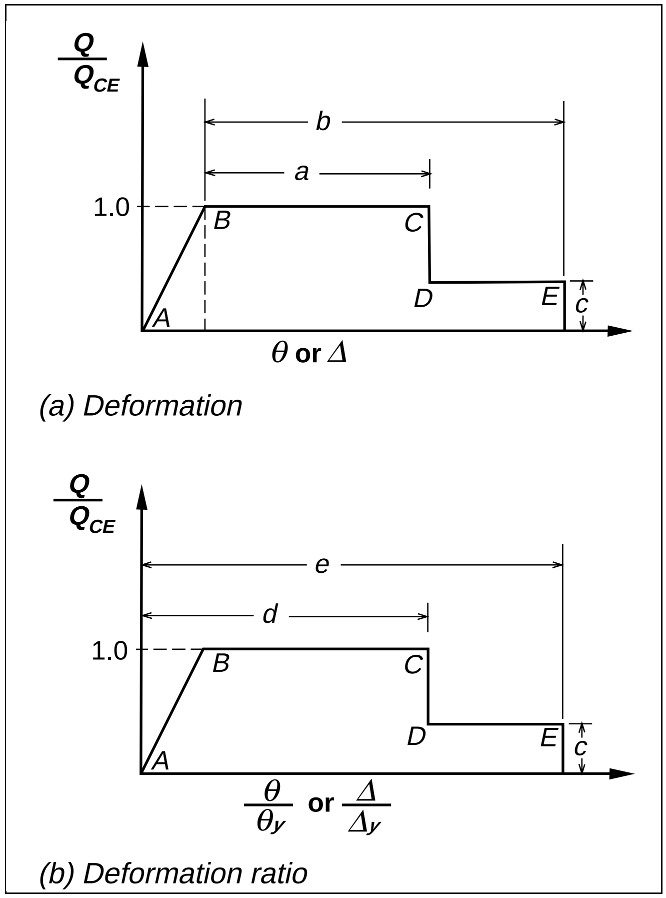

Figure 5-1 Definition of the a, b, c, d, and e Parameters in Tables 5-4, 5-6, and 5-8, and the Generalized Load-Deformation Behavior

chord rotation for beams. The chord rotation may be estimated by adding the yield rotation, θ_y, to the plastic rotation. Alternatively, the chord rotation may be estimated to be equal to the story drift. Test results for steel components are often given in terms of chord rotation. The equations for θ_y given in Equations 5-1 and 5-2 are approximate, and are based on the assumption of a point of contraflexure at mid-length of the beam or column.

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

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

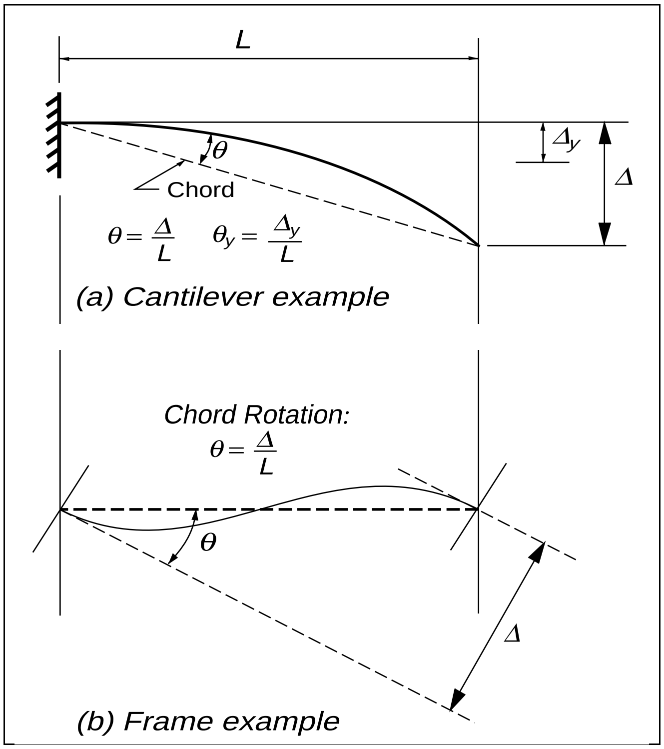

Figure 5-2 Definition of Chord Rotation

Q and Q_{CE} are the generalized component load and generalized component expected strength, respectively. For beams and columns, these refer to the plastic moment capacity, which is for:

Beams: $$Q_{CE} = M_{CE} = ZF_{ye} \quad (5\text{-}3)$$

Columns:

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

Panel Zones: $$Q_{CE} = V_{CE} = 0.55F_{ye}d_ct_p \quad (5\text{-}5)$$

where

d_c = Column depth, in.
E = Modulus of elasticity, ksi
F_{ye} = Expected yield strength of the material, ksi
I = Moment of inertia, in.^{4}
l_b = Beam length, in.
l_c = Column length, in.
M_{CE} = Expected moment strength
P = Axial force in the member, kips
P_{ye} = Expected axial yield force of the member = A_gF_{ye}, kips
Q = Generalized component load
Q_{CE} = Generalized component expected strength
t_p = Total panel zone thickness including doubler plates, in.
θ = Chord rotation
θ_y = Yield rotation
V_{CE} = Expected shear strength, kips
Z = Plastic section modulus, in.^{3}

C. Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be properly modeled. This behavior must be verified by experiment. This procedure is not recommended in most cases.

5.4.2.3 Strength and Deformation Acceptance Criteria

A. Linear Static and Dynamic Procedures

The strength and deformation acceptance criteria for these methods require that the load and resistance relationships given in Equations 3-18 and 3-19 in Chapter 3 be satisfied. The design strength of components in existing FR moment frames shall be determined using the appropriate equations for design strength given in Section 5.4.2.2 or in Part 6 of AISC (1994a), except that ϕ shall be taken as 1.0. Design restrictions given in AISC (1994a) shall be followed unless specifically superseded by provisions in these *Guidelines*.

Evaluation of component acceptability requires knowledge of the component expected strength, Q_{CE}, for Equation 3-18 and the component lower-bound strength, Q_{CL}, for Equation 3-19, and the component demand modifier, m, as given in Table 5-3 for Equation 3-18. Values for Q_{CE} and Q_{CL} for FR moment frame components are given in this section. Q_{CE} and Q_{CL} are used for deformation- and force-controlled components, respectively. Values for m are given in Table 5-3 for the Immediate Occupancy, Life Safety, and Collapse Prevention Performance Levels.

Beams. The design strength of beams and other flexural members is the lowest value obtained according to the limit state of yielding, lateral-torsional buckling, local flange buckling, or shear yielding of the web. For fully concrete-encased beams where the concrete is expected to remain in place, because of confining reinforcement, during the earthquake, assume $b_f = 0$ and $L_p = 0$ for the purpose of determining m. For bare beams bent about their major axes and symmetric about both axes, with $\frac{b_f}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$ (compact section) and $l_b < L_p$, the values for m are given in Table 5-3, and:

$$Q_{CE} = M_{CE} = M_{pCE} = ZF_{ye} \qquad (5\text{-}6)$$

where

b_f = Width of the compression flange, in.

t_f = Thickness of the compression flange, in.

l_b = Length of beam, in.

L_p = Limiting lateral unbraced length for full plastic bending capacity for uniform bending from AISC (1994a), in.

M_{CE} = Expected flexural strength, kip-in.

M_{pCE} = Expected plastic moment capacity, kip-in.

F_{ye} = Expected mean yield strength determined by the tests or given in Tables 5-1 or 5-2

If $\frac{b_f}{2t_f} > \frac{52}{\sqrt{F_y}}$ and $l_b > L_p$, values for m are given in Table 5-3. For cases where the moment diagram is nonuniform and $L_p < L_b < L_r$, but the nominal bending strength is still M_{pCE}, the value of m is obtained from Table 5-3. If $M_{CE} < M_{pCE}$ due to lateral torsional buckling, then the value of m shall be m_e, where

$$m_e = C_b\left[m - (m-1)\frac{(L_b - L_p)}{L_r - L_p}\right] \leq 8 \qquad (5\text{-}7)$$

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 (see AISC, 1994a)

L_p = Limiting unbraced length between points of lateral restraint for the full plastic moment capacity to be effective (see AISC, 1994a)

L_r = Limiting unbraced length between points of lateral support beyond which elastic lateral torsional buckling of the beam is the failure mode (see AISC, 1994a)

m = Value of m given in Table 5-3

m_e = Effective m from Equation 5-7

C_b = Coefficient to account for effect of nonuniform moment (see AISC, 1994a)

If the beam strength is governed by shear strength of the unstiffened web and $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_y}}$, then:

$$Q_{CE} = V_{CE} = 0.6F_{ye}A_w \qquad (5\text{-}8)$$

where

V_{CE} = Expected shear strength, kips

A_w = Nominal area of the web = $d_b t_w$, in.^{2}

t_w = Web thickness, in.

h = Distance from inside of compression flange to inside of tension flange, in.

For this case, use tabulated values for beams, row a, in Table 5-3. If $\frac{h}{t_w} > \frac{418}{\sqrt{F_y}}$, the value of V_{CE} should be calculated from provisions in Part 6 of AISC (1994a) and the value of m should be chosen using engineering judgment, but should be less than 8.

The limit state of local flange and lateral torsional buckling are not applicable to components either subjected to bending about their minor axes or fully encased in concrete, with confining reinforcement.

For built-up shapes, the strength may be governed by the strength of the lacing plates that carry component shear. For this case, the lacing plates are not as ductile as the component and should be designed for 0.5 times the *m* value in Table 5-3, unless larger values can be justified by tests or analysis. For built-up laced beams and columns fully encased in concrete, local buckling of the lacing is not a problem if most of the encasement can be expected to be in place after the earthquake.

Columns. The lower-bound strength, Q_{CL}, of steel columns under compression only is the lowest value obtained by the limit stress of buckling, local flange buckling, or local web buckling. The effective design strength should be calculated in accordance with provisions in Part 6 of AISC (1994a), but $\phi = 1.0$ and F_{ye} shall be used for existing components. Acceptance shall be governed by Equation 3-19 of these *Guidelines*, since this is a force-controlled member.

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

$$P_{CL} = A_g F_{cr} \tag{5-9}$$

where

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

$$F_{cr} = \frac{1.40 \times 10^5}{(l_c/r)^2} \text{ksi for } l_c/r > 108$$

Cast iron columns can only carry axial compression.

For steel columns under combined axial and bending stress, the column shall be considered to be deformation-controlled and the lower-bound strength shall be calculated by Equation 5-10 or 5-11.

For $\dfrac{P}{P_{CL}} \ge 0.2$

$$\frac{P}{P_{CL}} + \frac{8}{9}\left[\frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}}\right] \le 1.0 \tag{5-10}$$

For $\dfrac{P}{P_{CL}} < 0.2$

$$\frac{P}{2P_{CL}} + \frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}} \le 1.0 \tag{5-11}$$

where

P	=	Axial force in the column, kips
P_{CL}	=	Expected compression strength of the column, kips
M_x	=	Bending moment in the member for the x-axis, kip-in
M_{CEx}	=	Expected bending strength of the column for the x-axis, kip-in
M_{CEy}	=	Expected bending strength of the column for the y-axis
M_y	=	Bending moment in the member for the y-axis, kip-in
m_x	=	Value of *m* for the column bending about the x-axis
m_y	=	Value of *m* for the column bending about the y-axis

For columns under combined compression and bending, lateral bracing to prevent torsional buckling shall be provided as required by AISC (1994a).

Panel Zone. The strength of the panel zone shall be calculated as given in Equation 5-5.

Connections. By definition, the strength of FR connections shall be at least equal to, or preferably greater than, the strength of the members being joined. Some special considerations should be given to FR connections.

Full Penetration Welded Connections (Full-Pen). Full-pen connections (see Figure 5-3) have the beam flanges welded to the column flanges with complete penetration groove welds. A bolted or welded shear tab is also included to connect the beam web to the column. The strength and ductility of full-pen connections are not fully understood at this time. They are functions of the quality of construction, the l_b/d_b ratio of the beam (where l_b = beam length and d_b = beam depth), the weld material, the thickness of the beam and column flanges, the stiffness and strength of the panel zones, joint confinement, triaxial stresses, and other factors (see SAC, 1995). In lieu of further study, the value of *m* for Life Safety for beams with full-pen connections shall be not larger than

Table 5-3 Acceptance Criteria for Linear Procedures—Fully Restrained (FR) Moment Frames

	m Values for Linear Procedures[8]				
	Primary			Secondary	
Component/Action	**IO m**	**LS m**	**CP m**	**LS m**	**CP m**
Moment Frames					
Beams:					
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	2	6	8	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	1	2	3	3	4
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation					
Columns:					
For $P/P_{ye} < 0.20$					
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	2	6	8	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	1	1	2	2	3
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation					
For 0.2 ð P/P_{ye} ð 0.50[9]					
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	1	—[1]	—[2]	—[3]	—[4]
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	1	1	1.5	2	2
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation					
Panel Zones	1.5	8	11	NA	NA
Fully Restrained Moment Connections[7]					
For full penetration flange welds and bolted or welded web connection: beam deformation limits					
a. No panel zone yield	1	—[5]	—[6]	3	4
b. Panel zone yield	0.8	2	2.5	2	2.5

1. m = 9 (1 – 1.7 P/P_{ye})
2. m = 12 (1 – 1.7 P/P_{ye})
3. m = 15 (1 – 1.7 P/P_{ye})
4. m = 18 (1 – 1.7 P/P_{ye})
5. m = 6 – 0.125 d_b
6. m = 7 – 0.125 d_b
7. If construction documents verify that notch-tough rated weldment was used, these values may be multiplied by two.
8. For built-up numbers where strength is governed by the facing plates, use one-half these *m* values.
9. If P/P_{ye} > 0.5, assume column to be force-controlled.

$$m = 6.0 - 0.125\ d_b \tag{5-12}$$

In addition, if the strength of the panel zone is less than 0.9 times the maximum shear force that can be delivered by the beams, then the *m* for the beam shall be

$$m = 2 \tag{5-13}$$

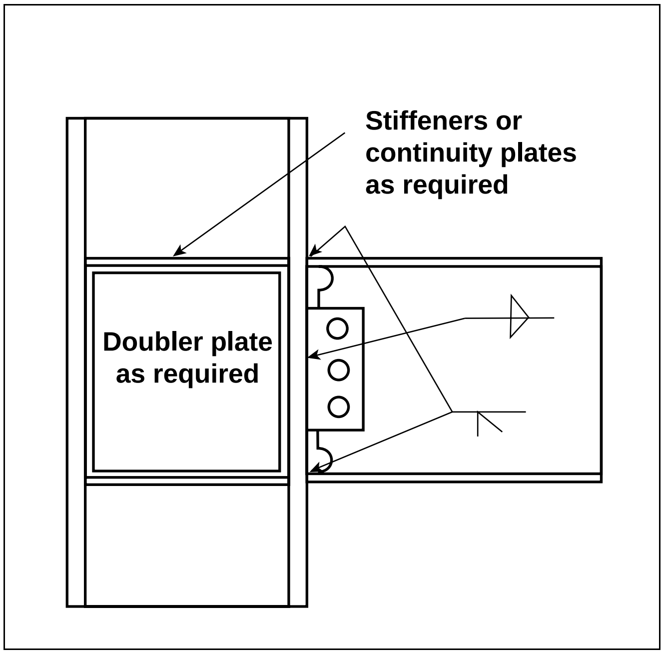

Figure 5-3 Full-Pen Connection in FR Connection with Variable Behavior

Flange Plate and End Plate Connections. The strength of these connections should be in accordance with standard practice as given in AISC (1994a and 1994b). Additional information for these connections is given below in Section 5.4.3.3.

Column Base Plates to Concrete Pile Caps or Footings. The strength of connections between column base plates and concrete pile caps or footings usually exceeds the strength of the columns. The strength of the base plate and its connection may be governed by the welds or bolts, the dimensions of the plate, or the expected yield strength, F_{ye}, of the base plate. The connection between the base plate and the concrete may be governed by shear or tension yield of the anchor bolts, loss of bond between the anchor bolts and the concrete, or failure of the concrete. Expected strengths for each failure type shall be calculated by rational analysis or the provisions in AISC (1994b). The values for *m* may be chosen from similar partially restrained end plate actions given in Table 5-5.

B. Nonlinear Static Procedure

The NSP requires modeling of the complete load-deformation relationship to failure for each component. This may be based on experiment, or on a rational analysis, preferably verified by experiment. In lieu of these, the conservative approximate behavior depicted by Figure 5-1 may be used. The values for Q_{CE} and θ_y shown in Figure 5-1 are the same as those used in the LSP and given in Section 5.4.2.2. Deformation control points and acceptance criteria for the Nonlinear Static and Dynamic Procedures are given in Table 5-4.

C. Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be modeled for this procedure. Guidelines for this are given in the *Commentary*. Deformation limits are given in Table 5-4.

5.4.2.4 Rehabilitation Measures for FR Moment Frames

Several options are available for rehabilitation of FR moment frames. In all cases, the compatibility of new and existing components and/or elements must be checked at displacements consistent with the Performance Level chosen. The rehabilitation measures are as follows:

- Add steel braces to one or more bays of each story to form concentric or eccentric braced frames. (Attributes and design criteria for braced frames are given in Section 5.5.) Braces significantly increase the stiffness of steel frames. Care should be taken when designing the connections between the new braces and the existing frame. The connection should be designed to carry the maximum probable brace force, which may be approximated as 1.2 times the expected strength of the brace.

- Add ductile concrete or masonry shear walls or infill walls to one or more bays of each story. Attributes and design requirements of concrete and masonry infills are given in Sections 6.7 and 7.5, respectively. This greatly increases the stiffness and strength of the structure. Do not introduce torsional stress into the system.

Table 5-4 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Fully Restrained (FR) Moment Frames

	$\frac{\Delta}{\Delta_y}$		Residual Strength Ratio	Plastic Rotation, Deformation Limits				
				Primary			Secondary	
Component/Action	***d***	***e***	***c***	**IO**	**LS**	**CP**	**LS**	**CP**
Beams[1]: a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	10	12	0.6	2	7	9	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	5	7	0.2	1	3	4	4	5
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation								
Columns[2]: For $P/P_{ye} < 0.20$ a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	10	12	0.6	2	7	9	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$			0.2	1	3	4	4	5
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation								

1. Add θ_y from Equations 5-1 or 5-2 to plastic end rotation to estimate chord rotation.
2. Columns in moment or braced frames need only be designed for the maximum force that can be delivered.
3. Deformation = $0.072 (1 - 1.7\, P/P_{ye})$
4. Deformation = $0.100 (1 - 1.7\, P/P_{ye})$
5. Deformation = $0.042 (1 - 1.7\, P/P_{ye})$
6. Deformation = $0.060 (1 - 1.7\, P/P_{ye})$
7. $0.043 - 0.0009\, d_b$
8. $0.035 - 0.0008\, d_b$
9. If $P/P_{ye} > 0.5$, assume column to be force-controlled.

Table 5-4 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Fully Restrained (FR) Moment Frames (continued)

Component/Action	$\frac{\Delta}{\Delta_y}$		Residual Strength Ratio	Plastic Rotation, Deformation Limits				
				Primary			Secondary	
	d	**e**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
For $0.2 \le P/P_{ye} \le 0.50$[9]								
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	—[3]	—[4]	0.2	0.04	—[5]	—[6]	0.019	0.031
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	2	2.5	0.2	1	1.5	1.8	1.8	2
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation								
	Plastic Rotation							
	a	**b**						
Panel Zones	0.052	0.081	0.800	0.004	0.025	0.043	0.055	0.067
Connections								
For full penetration flange weld, bolted or welded web: beam deformation limits								
a. No panel zone yield	—[7]	—[7]	0.200	0.008	—[8]	—[8]	0.017	0.025
b. Panel zone yield	0.009	0.017	0.400	0.003	0.005	0.007	0.010	0.013

1. Add θ_y from Equations 5-1 or 5-2 to plastic end rotation to estimate chord rotation.
2. Columns in moment or braced frames need only be designed for the maximum force that can be delivered.
3. Deformation = $0.072\ (1 - 1.7\ P/P_{ye})$
4. Deformation = $0.100\ (1 - 1.7\ P/P_{ye})$
5. Deformation = $0.042\ (1 - 1.7\ P/P_{ye})$
6. Deformation = $0.060\ (1 - 1.7\ P/P_{ye})$
7. $0.043 - 0.0009\ d_b$
8. $0.035 - 0.0008\ d_b$
9. If $P/P_{ye} > 0.5$, assume column to be force-controlled.

- Attach new steel frames to the exterior of the building. This scheme has been used in the past and has been shown to be very effective under certain conditions. Since this will change the distribution of stiffness in the building, the seismic load path must be carefully checked. The connections between the new and existing frames are particularly vulnerable. This 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.

- Reinforce the moment-resisting connections to force plastic hinge locations in the beam material away from the joint region. The idea behind this concept is that the stresses in the welded connection will be significantly reduced, thereby reducing the possibility of brittle fractures. This may not be effective if weld material with very low toughness was used in the full-pen connection. Strain hardening at the new hinge location may produce larger stresses at the weld than expected. Also, many fractures during past earthquakes are believed to have occurred at stresses lower than yield. Various methods, such as horizontal cover plates, vertical stiffeners, or haunches, can be employed. Other schemes that result in the removal of beam material may achieve the same purpose. Modification of all moment-resisting connections could significantly increase (or decrease, in the case of material removal) the structure's stiffness; therefore, recalculation of the seismic demands may be required. Modification of selected joints should be done in a rational manner that is justified by analysis. Guidance on the design of these modifications is discussed in SAC (1995).

- Adding damping devices may be a viable rehabilitation measure for FR frames. See Chapter 9 of these *Guidelines.*

5.4.3 Partially Restrained Moment Frames

5.4.3.1 General

Partially restrained (PR) moment frames are those frames for which deformation of the beam-to-column connections contributes greater than 5% of the story drift. A moment frame shall also be considered to be PR if the strength of the connections is less than the strength of the weaker of the two members being joined. A PR connection usually has two or more failure modes. The weakest failure mechanism shall be considered to govern the behavior of the joint. The beam and/or column need only resist the maximum force (or moment) that can be delivered by the connection. General design provisions for PR frames given in AISC (1994a) or BSSC (1995) shall apply unless superseded by these *Guidelines*. Equations for calculating nominal design strength shall be used for determining the expected strength, except $\phi = 1$, and F_{ye} shall be used in place of F_y.

5.4.3.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

Beams, columns, and panel zones. Axial area, shear area, moment of inertia, and panel zone stiffness shall be determined as given in Section 5.4.2.2 for FR frames.

Connections. The rotational stiffness K_θ of each PR connection shall be determined by experiment or by rational analysis based on experimental results. The deformation of the connection shall be included when calculating frame displacements. Further discussion of this is given in the *Commentary.* In the absence of more rational analysis, the stiffness may be estimated by the following approximate procedures:

The rotational spring stiffness, K_θ, may be estimated by

$$K_\theta = \frac{M_{CE}}{0.005} \tag{5-14}$$

where

M_{CE} = Expected moment strength, kip-in.

for:

- PR connections that are encased in concrete for fire protection, and where the nominal resistance, M_{CE}, determined for the connection includes the composite action provided by the concrete encasement
- PR connections that are encased in masonry, where composite action cannot be developed in the connection resistance
- Bare steel PR connections

For all other PR connections, the rotational spring stiffness may be estimated by

$$K_\theta = \frac{M_{CE}}{0.003} \tag{5-15}$$

The connection strength, M_{CE}, is discussed in Section 5.4.3.3.

As a simplified alternative analysis method to an exact PR frame analysis, where connection stiffness is modeled explicitly, the beam stiffness, EI_b, may be adjusted by

$$EI_b \; adjusted = \frac{1}{\frac{6h}{l_b^2 K_\theta} + \frac{1}{EI_b}} \tag{5-16}$$

where

K_θ = Equivalent rotational spring stiffness, kip-in./rad

M_{CE} = Expected moment strength, kip-in.

I_b = Moment of inertia of the beam, in.^{4}

h = Average story height of the columns, in.

l_b = Centerline span of the beam, in.

This adjusted beam stiffness may be used in standard rigid-connection frame finite element analysis. The joint rotation of the column shall be used as the joint rotation of the beam at the joint with this simplified analysis procedure.

B. Nonlinear Static Procedure

- Use elastic component properties as given in Section 5.4.3.2A.
- Use appropriate nonlinear moment-curvature or load-deformation behavior for beams, beam-columns, and panel zones as given in Section 5.4.2 for FR frames.

Use appropriate nonlinear moment-rotation behavior for PR connections as determined by experiment. In lieu of experiment, or more rational analytical procedure based on experiment, the moment-rotation relationship given in Figure 5-1 and Table 5-6 may be used. The parameters θ and θ_y are rotation and yield rotation. The value for θ_y may be assumed to be 0.003 or 0.005 in accordance with the provisions in Section 5.4.3.2A.

Q and Q_{CE} are the component moment and expected yield moment, respectively. Approximate values of M_{CE} for common types of PR connections are given in Section 5.4.3.3B.

C. Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be properly modeled based on experiment.

5.4.3.3 Strength and Deformation Acceptance Criteria

A. Linear Static and Dynamic Procedures

The strength and deformation acceptance criteria for these methods require that the load and resistance relationships given in Equations 3-18 and 3-19 in Chapter 3 be satisfied. The expected strength and other restrictions for a beam or column shall be determined in accordance with the provisions given above in Section 5.4.2.3 for FR frames.

Evaluation of component acceptability requires knowledge of the lower-bound component capacity, Q_{CL} for Equation 3-19 and Q_{CE} for Equation 3-18, and the ductility factor, m, as given in Table 5-5 for use in Equation 3-18. Values for Q_{CE} and Q_{CL} for beams and columns in PR frames are the same as those given in Section 5.4.2.3 and Table 5-3 for FR frames. Values for Q_{CE} for PR connections are given in this section. Control points and acceptance criteria for Figure 5-1 for PR frames are given in Table 5-6. Values for m are given in Table 5-5 for the Immediate Occupancy, Life Safety, and Collapse Prevention Performance Levels.

B. Nonlinear Static Procedure

The NSP requires modeling of the complete load-deformation relationship to failure for each component. This may be based on experiment, or a rational analysis, preferably verified by experiment. In lieu of these, the conservative and approximate behavior depicted by Figure 5-1 may be used. The values for Q_{CE} and θ_y are the same as those used in the LSP in Sections 5.4.2.2 and 5.4.3.2. The deformation limits and nonlinear control points, c, d, and e, shown in Figure 5-1 are given in Table 5-6.

The expected strength, Q_{CE}, for PR connections shall be based on experiment or accepted methods of analysis as given in AISC (1994a and b) or in the *Commentary.* In lieu of these, approximate conservative expressions for Q_{CE} for common types of PR connections are given below.

Riveted or Bolted Clip Angle Connection. This is a beam-to-column connection as defined in Figure 5-4. The expected moment strength of the connection, M_{CE}, may be conservatively determined by using the smallest value of M_{CE} computed using Equations 5-17 through 5-22.

Table 5-5 Acceptance Criteria for Linear Procedures—Partially Restrained (PR) Moment Frames

Component/Action	*m* Values for Linear Methods				
	Primary			Secondary	
	IO	LS	CP	LS	CP
Partially restrained moment connection					
For top and bottom clip angles[1]					
a. Rivet or bolt shear failure[2]	1.5	4	6	6	8
b. Angle flexure failure	2	5	7	7	14
c. Bolt tension failure[2]	1	1.5	2.5	4	4
For top and bottom T-stub[1]					
a. Bolt shear failure[2]	1.5	4	6	6	8
b. T-stub flexure failure	2	5	7	7	14
c. Bolt tension failure[2]	1	1.5	2.5	4	4
For composite top and clip angle bottom[1]					
a. Yield and fracture of deck reinforcement	1	2	3	4	6
b. Local yield and web crippling of column flange	1.5	4	6	5	7
c. Yield of bottom flange angle	1.5	4	6	6	7
d. Tensile yield of column connectors or OSL of angle	1	1.5	2.5	2.5	3.5
e. Shear yield of beam flange connections	1	2.5	3.5	3.5	4.5
For flange plates welded to column bolted or welded to beam[1]					
a. Failure in net section of flange plate or shear failure of bolts or rivets[2]	1.5	4	5	4	5
b. Weld failure or tension failure on gross section of plate	0.5	1.5	2	1.5	2
For end plate welded to beam bolted to column					
a. Yielding of end plate	2	5.5	7	7	7
b. Yield of bolts	1.5	2	3	4	4
c. Failure of weld	0.5	1.5	2	3	3

1. Assumed to have web plate or stiffened seat to carry shear. Without shear connection, this may not be downgraded to a secondary member. If d_b > 18 inches, multiply *m* values by $18/d_b$.
2. For high-strength bolts, divide these values by two.

If the shear connectors between the beam flange and the flange angle control the resistance of the connection:

$$Q_{CE} = M_{CE} = d_b(F_{ve}A_bN_b) \tag{5-17}$$

where

A_b = Gross area of rivet or bolt, in.^{2}
d_b = Overall beam depth, in.
F_{ve} = Unfactored nominal shear strength of the bolts or rivets given in AISC (1994a), ksi
N_b = Least number of bolts or rivets connecting the top or bottom flange to the angle

If the tensile capacity of the horizontal outstanding leg (OSL) of the connection controls the capacity, then P_{CE} is the smaller of

$$P_{CE} \le F_{ye}A_g \tag{5-18}$$

Table 5-6 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Partially Restrained (PR) Moment Frames

	Plastic Rotation[1]		Residual Force Ratio	Joint Rotation				
				Primary			Secondary	
	a	*b*	*c*	IO	LS	CP	LS	CP
Top and Bottom Clip Angles[1]								
a. Rivet or bolt shear[2]	0.036	0.048	0.200	0.008	0.020	0.030	0.030	0.040
b. Angle flexure	0.042	0.084	0.200	0.010	0.025	0.035	0.035	0.070
c. Bolt tension	0.016	0.025	1.000	0.005	0.008	0.013	0.020	0.020
Top and Bottom T-Stub[1]								
a. Rivet or bolt shear[2]	0.036	0.048	0.200	0.008	0.020	0.030	0.030	0.040
b. T-stub flexure	0.042	0.084	0.200	0.010	0.025	0.035	0.035	0.070
c. Rivet or bolt tension	0.016	0.024	0.800	0.005	0.008	0.013	0.020	0.020
Composite Top Angle Bottom[1]								
a. Deck reinforcement	0.018	0.035	0.800	0.005	0.010	0.015	0.020	0.030
b. Local yield column flange	0.036	0.042	0.400	0.008	0.020	0.030	0.025	0.035
c. Bottom angle yield	0.036	0.042	0.200	0.008	0.020	0.030	0.025	0.035
d. Connectors in tension	0.015	0.022	0.800	0.005	0.008	0.013	0.013	0.018
e. Connections in shear[2]	0.022	0.027	0.200	0.005	0.013	0.018	0.018	0.023
Flange Plates Welded to Column Bolted or Welded to Beam[2]								
a. Flange plate net section or shear in connectors	0.030	0.030	0.800	0.008	0.020	0.025	0.020	0.025
b. Weld or connector tension	0.012	0.018	0.800	0.003	0.008	0.010	0.010	0.015
End Plate Bolted to Column Welded to Beam								
a. End plate yield	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. Fracture of weld	0.012	0.018	0.800	0.003	0.008	0.010	0.015	0.015

1. If $d_b > 18$, multiply deformations by $18/d_b$. Assumed to have web plate to carry shear. Without shear connection, this may not be downgraded to a secondary member.
2. For high-strength bolts, divide rotations by 2.

$$P_{CE} \le F_{te} A_e \quad (5\text{-}19)$$

and

$$Q_{CE} = M_{CE} \le P_{CE}(d_b + t_a) \quad (5\text{-}20)$$

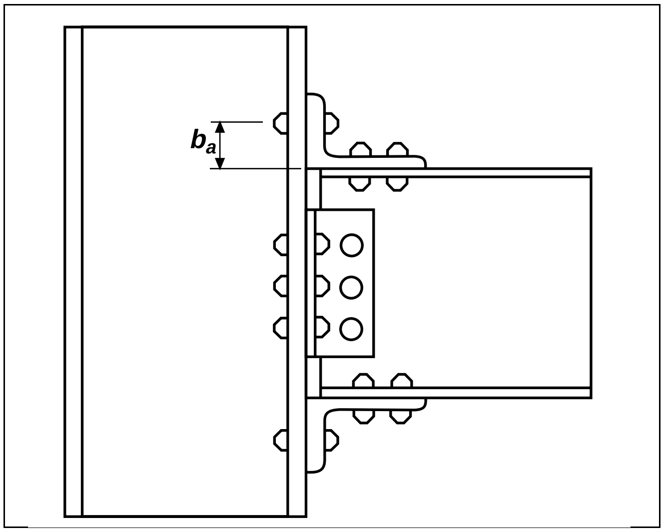

Figure 5-4 Clip Angle Connection

where

A_e = Effective net area of the OSL, in.^{2}
A_g = Gross area of the OSL, in.^{2}
P = Force in the OSL, kips
t_a = Thickness of angle, in.

If the tensile capacity of the rivets or bolts attaching the OSL to the column flange control the capacity of the connection:

$$Q_{CE} = M_{CE} = (d_b + b_a)(F_{te}A_cN_b) \qquad (5\text{-}21)$$

where

A_c = Rivet or bolt area, in.^{2}
b_a = Dimension in Figure 5-4, in.
F_{te} = Expected tensile strength of the bolts or rivets, ksi
N_b = Least number of bolts or rivets connecting top or bottom angle to column flange

Flexural yielding of the flange angles controls the expected strength if:

$$Q_{CE} = M_{CE} = \frac{wt_a^2F_{ye}}{4\left[b_a - \frac{t_a}{2}\right]}(d_b + b_a) \qquad (5\text{-}22)$$

where

b_a = Dimension shown in Figure 5-4, in.
w = Length of the flange angle, in.
F_{ye} = Expected yield strength

Riveted or Bolted T-Stub Connection. A riveted or bolted T-stub connection is a beam-to-column connection as depicted in Figure 5-5. The expected moment strength, M_{CE}, may be determined by using the smallest value of M_{CE} computed using Equations 5-23 through 5-25.

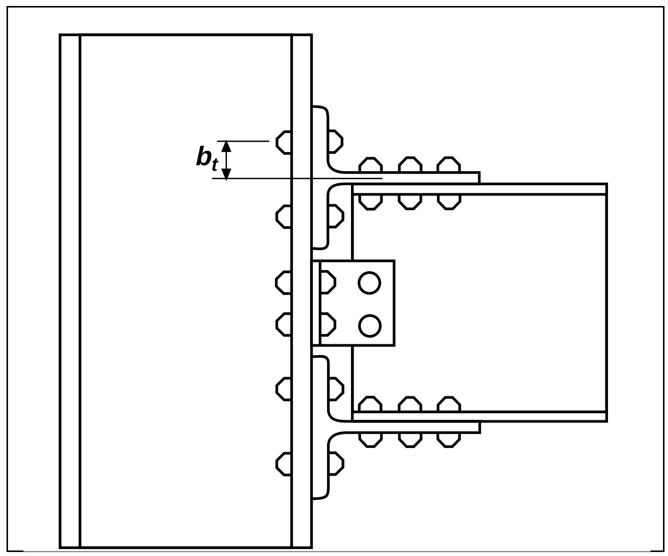

Figure 5-5 T-Stub Connection may be FR or PR Connection

If the shear connectors between the beam flange and the T-stub web control the resistance of the connection, use Equation 5-17.

If the tension capacity of the bolts or rivets connecting the T-stub flange to the column flange control the resistance of the connection:

$$Q_{CE} = M_{CE} = (d_b + 2b_t + t_s)(F_{te}A_bN_b) \qquad (5\text{-}23)$$

where

N_b = Number of fasteners in tension connecting the flanges of one T-stub to the column flange
t_s = Thickness of T-stub stem

If tension in the stem of the T-stub controls the resistance, use Equations 5-18 and 5-19 with A_g and A_e being the gross and net areas of the T-stub stem.

If flexural yielding of the flanges of the T-stub controls the resistance of the connection:

$$Q_{CE} = M_{CE} = \frac{(d_b + t_s)wt_f^2 F_{ye}}{2(b_t - k_1)} \qquad (5\text{-}24)$$

where

k_1 = Distance from the center of the T-stub stem to the edge of the T-stub flange fillet, in.

b_t = Distance between one row of fasteners in the T-stub flange and the centerline of the stem (Figure 5-5; different from b_a in Figure 5-4)

w = Length of T-stub, in.

t_f = Thickness of T-stub flange, in.

Flange Plate Connections. Flange plate connections are sometimes used as shown in Figure 5-6. This connection may be considered to be fully restrained if the strength is sufficient to develop the strength of the beam. The expected strength of the connection may be calculated as

$$Q_{CE} = M_{CE} = P_{CE}(d_b + t_p) \qquad (5\text{-}25)$$

where

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

t_p = Thickness of flange plate, in.

The strength of the welds must also be checked. The flange plates may also be bolted to the beam; in this case, the strength of the bolts and the net section of the flange plates must also be checked.

End Plate Connections. As shown in Figure 5-7, these may sometimes be considered to be FR if the strength is great enough to develop the expected strength of the beam. The strength may be governed by the bolts that are under combined shear and tension or bending in the end plate. The design strength $Q_{CE} = M_{CE}$ shall be computed in accordance with AISC (1994b) or by any other rational procedure supported by experimental results.

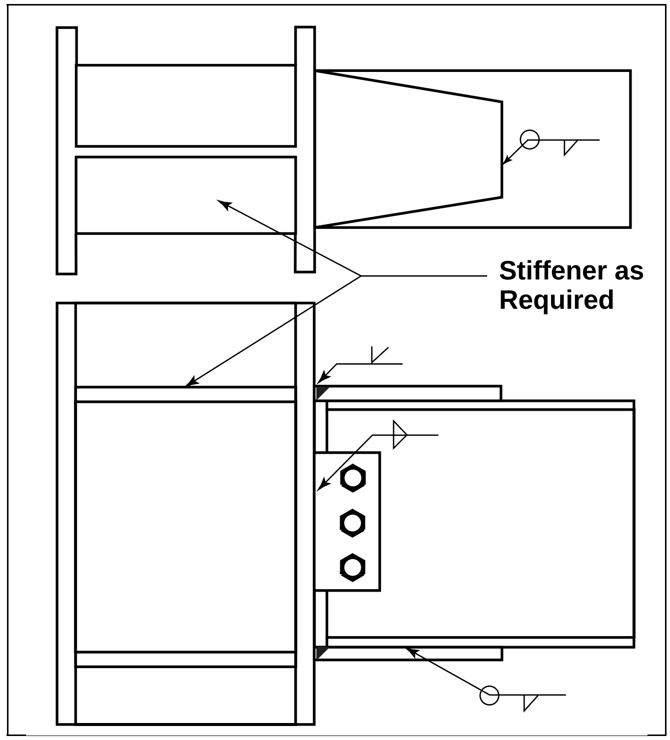

Figure 5-6 Flange Plate Connection may be FR or PR Connection

Composite Partially Restrained Connections. These may be used as shown in Figure 5-8. The equivalent rotational spring constant, K_θ, shall be that given by Equation 5-14. The behavior of these connections is complex, with several possible failure mechanisms. Strength calculations are discussed in the *Commentary.*

C. Nonlinear Dynamic Procedure

See Section 5.4.2.3.

5.4.3.4 Rehabilitation Measures for PR Moment Frames

The rehabilitation measures for FR moment frames will often work for PR moment frames as well (see Section 5.4.2.4). 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.

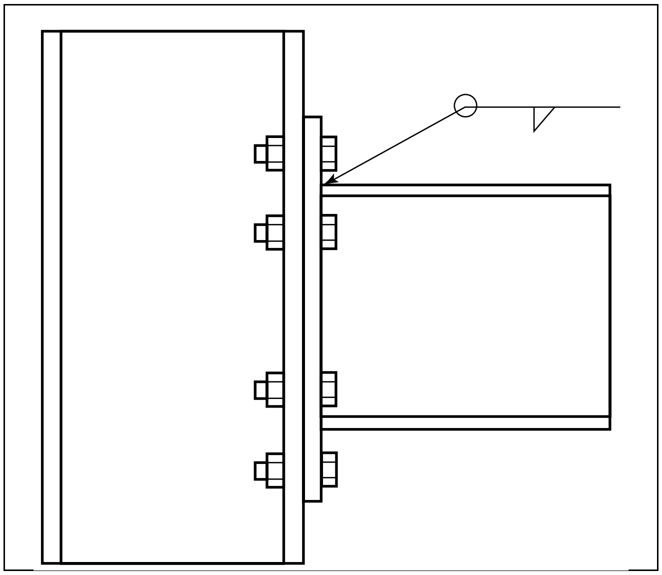

Figure 5-7 End Plate Connection may be FR or PR Connection

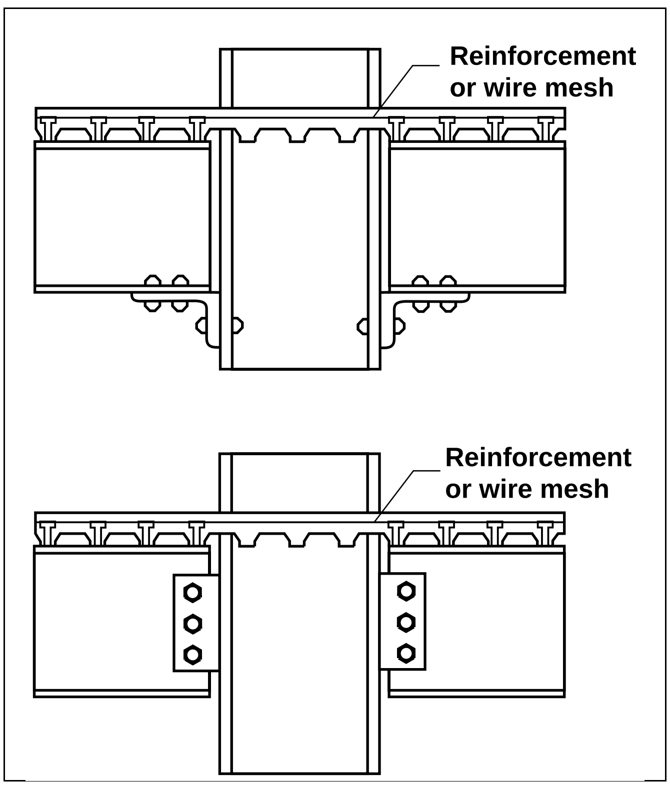

Figure 5-8 Two Configurations of PR Composite Connections

Connections in PR moment frames are usually the weak, flexible, or both, components. Connections may be rehabilitated by replacing rivets with high-strength bolts, adding weldment to supplement rivets or bolts, welding stiffeners to connection pieces or combinations of these measures.

5.5 Steel Braced Frames

5.5.1 General

The seismic resistance of steel braced frames is primarily derived from the axial force capacity of their components. Steel braced frames act as vertical trusses where the columns are the chords and the beams and braces are the web members. Braced frames may act alone or in conjunction with concrete or masonry walls, or steel moment frames, to form a dual system.

Steel braced frames may be divided into two types: concentric braced frames (CBF) and eccentric braced frames (EBF). Columns, beams, braces, and connections are the components of CBF and EBF. A link beam is also a component of an EBF. The components are usually hot-rolled shapes. The components may be bare steel, steel with a nonstructural coating for fire protection, steel with concrete encasement for fire protection, or steel with masonry encasement for fire protection.

5.5.2 Concentric Braced Frames (CBF)

5.5.2.1 General

Concentric braced frames are braced systems whose worklines essentially intersect at points. Minor eccentricities, where the worklines intersect within the width of the bracing member are acceptable if accounted for in the design.

5.5.2.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

Beams and Columns. Axial area, shear area, and moment of inertia shall be calculated as given in Section 5.4.2.2.

Connections. FR connections shall be modeled as given in Section 5.4.2.2. PR connections shall be modeled as given in Section 5.4.3.2.

Braces. Braces shall be modeled the same as columns for linear procedures.

B. Nonlinear Static Procedure

- Use elastic component properties as given in Section 5.4.2.2A.
- Use appropriate nonlinear moment curvature or load-deformation behavior for beams, columns, braces, and connections to represent yielding and buckling. Guidelines are given in Section 5.4.2.2 for beams and columns and Section 5.4.3.2 for PR connections.

Braces. Use nonlinear load-deformation behavior for braces as determined by experiment or analysis supported by experiment. In lieu of these, the load versus axial deformation relationship given in Figure 5-1 and Table 5-8 may be used. The parameters Δ and Δ_y are axial deformation and axial deformation at brace buckling. The reduction in strength of a brace after buckling must be included in the model. Elasto-plastic brace behavior may be assumed for the compression brace if the yield force is taken as the residual strength after buckling, as indicated by the parameter *c* in Figure 5-1 and Table 5-8. Implications of forces higher than this lower-bound force must be considered.

C. Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be properly based on experiment or generally accepted engineering practice. Guidelines for this are given in the *Commentary.*

5.5.2.3 Strength and Deformation Acceptance Criteria

A. Linear Static and Dynamic Procedures

The strength and deformation acceptance criteria for these methods require that the load and resistance relationships given in Equations 3-18 and 3-19 in Chapter 3 be satisfied. The design strength and other restrictions for a beam and column shall be determined in accordance with the provisions given in Section 5.4.2.3.

Evaluation of component acceptability requires knowledge of the component lower-bound capacity, Q_{CL}, for Equation 3-19 and Q_{CE} for Equation 3-18, and the ductility factor, *m*, as given in Table 5-7 for use in Equation 3-10. Columns shall be considered to be force-controlled members. Values for Q_{CE} and Q_{CL} for beams and columns are the same as those given in Section 5.4.2.3 for FR frames. Q_{CE} and Q_{CL} for PR connections are given in Section 5.4.3.3B. Braces are deformation-controlled components where the expected strength for the brace in compression is computed in the same manner as for columns given in Section 5.4.2.3.

Table 5-7 Acceptance Criteria for Linear Procedures—Braced Frames and Steel Shear Walls

	***m* Values for Linear Procedures**				
	Primary			**Secondary**	
Component/Action	**IO**	**LS**	**CP**	**LS**	**CP**
Concentric Braced Frames					
Columns:[1]					
a. Columns in compression[1]	Force-controlled member, use Equation 3-15 or 3-16.				
b. Columns in tension[1]	1	3	5	6	7
Braces in Compression[2]					
a. Double angles buckling in plane	0.8	6	8	7	9
b. Double angles buckling out of plane	0.8	5	7	6	8
c. W or I shape	0.8	6	8	6	8
d. Double channel buckling in plane	0.8	6	8	7	9
e. Double channel buckling out of plane	0.8	5	7	6	8
f. Rectangular concrete-filled cold-formed tubes	0.8	5	7	5	7

Table 5-7 Acceptance Criteria for Linear Procedures—Braced Frames and Steel Shear Walls (continued)

Component/Action	*m* Values for Linear Procedures				
	Primary			Secondary	
	IO	LS	CP	LS	CP
g. Rectangular cold-formed tubes					
1. $\frac{d}{t} \le \frac{90}{\sqrt{F_y}}$	0.8	5	7	5	7
2. $\frac{d}{t} \ge \frac{190}{\sqrt{F_y}}$	0.8	2	3	2	3
3. $\frac{90}{\sqrt{F_y}} \le \frac{d}{t} \le \frac{190}{\sqrt{F_y}}$	Use linear interpolation				
h. Circular hollow tubes					
1. $\frac{d}{t} \le \frac{1500}{F_y}$	0.8	5	7	5	7
2. $\frac{d}{t} \ge \frac{6000}{F_y}$	0.8	2	3	2	3
3. $\frac{1500}{F_y} \le \frac{d}{t} \le \frac{6000}{F_y}$	Use linear interpolation				
Braces in Tension[3]	1	6	8	8	10
Eccentric Braced Frames					
a. Beams	Governed by link				
b. Braces	Force-controlled, use Equation 3-19				
c. Columns in compression	Force-controlled, use Equation 3-19				
d. Columns in tension	1	3	5	6	7
Link beam[4]					
a.[5] $\frac{2M_{CE}}{eV_{CE}} < 1.6$ d: 16, e: 18, c: 1.00	1.5	9	13	13	15
b. $\frac{2M_{CE}}{eV_{CE}} < 2.6$	Same as for beam in FR moment frame; see Table 5-3				
c. $1.6 \le \frac{2M_{CE}}{eV_{CE}} < 2.6$	Use linear interpolation				

Table 5-7 Acceptance Criteria for Linear Procedures—Braced Frames and Steel Shear Walls (continued)

	m Values for Linear Procedures				
	Primary			Secondary	
Component/Action	**IO**	**LS**	**CP**	**LS**	**CP**
Steel Shear Walls[6]	1.5	8	12	12	14

1. Columns in moment or braced frames need only be designed for the maximum force that can be delivered.
2. Connections in braced frames should be able to carry 1.25 times the brace strength in compression, or the expected strength of the member in tension. Otherwise maximum value of m = 2.
3. For tension-only bracing systems, divide these m values by 2.
4. Assumes ductile detailing for flexural links.
5. Link beams with three or more web stiffeners. If no stiffeners, use half of these values. For one or two stiffeners, interpolate.
6. Applicable if stiffeners are provided to prevent shear buckling.

For common cross bracing configurations where both braces are attached to a common gusset plate where they cross at their midpoints, the effective length of each brace may 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), if the braces are back-to-back shapes attached to common gusset plates, the length shall be taken as the total length of the brace including gusset plates, and K, the effective length factor, (AISC, 1994a) may be assumed to be 0.8 for in-plane buckling and 1.0 for out-of-plane buckling.

Restrictions on bracing members, gusset plates, brace configuration, and lateral bracing of link beams are given in the seismic provisions of AISC (1994a). If the special requirements of Section 22.11.9.2 of AISI (1986) are met, then 1.0 may be added to the brace m values given in Table 5-7.

The strength of brace connections shall be the larger of the maximum force deliverable by the tension brace or 1.25 times the maximum force deliverable by the compression brace. If not, the connection shall be strengthened, or the m values and deformation limits shall be reduced to comparable values given for connectors with similar limit states (see Table 5-5).

Stitch plates for built-up members shall be spaced such that the largest slenderness ratio of the components of the brace is at most 0.4 times the governing slenderness ratio of the brace as a whole. The stitches for compressed members must be able to resist 0.5 times the maximum brace force where buckling of the brace will cause shear forces in the stitches. If not, stitch plates shall be added, or the m values in Table 5-7 and deformation limits in Table 5-8 shall be reduced by 50%. Values of m need not be less than 1.0.

B. Nonlinear Static Procedure

The NSP requires modeling of the complete nonlinear force-deformation relationship to failure for each component. This may be based on experiment, or analysis verified by experiment. Guidelines are given in the *Commentary*. In lieu of these, the conservative approximate behavior depicted in Figure 5-1 may be used. The values for Q_{CE} and θ_y are the same as those used for the LSP. Deformation parameters c, d, and e for Figure 5-1 and deformation limits are given in Table 5-8. The force-deformation relationship for the compression brace should be modeled as accurately as possible (see the *Commentary*). In lieu of this, the brace may be assumed to be elasto-plastic, with the yield force equal to the residual force that corresponds to the parameter c in Figure 5-1 and Table 5-8. This assumption is an estimate of the lower-bound brace force. Implications of forces higher than this must be considered.

C. Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be modeled for this procedure. Guidelines for this are given in the *Commentary*.

Table 5-8 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Braced Frames and Steel Shear Walls

	$\frac{\Delta}{\Delta_y}$		Residual Force Ratio	Deformation				
				Primary			Secondary	
Component/Action	***d***	***e***	***c***	**IO**	**LS**	**CP**	**LS**	**CP**
Concentric Braced Frames								
a. Columns in compression[1]	Force-controlled, use Equation 3-19							
b. Columns in tension[1]	6	8	1.000	1	4	6	7	8
Braces in Compression[2,3]								
a. Two angles buckle in plane	1	10	0.2	0.8	6	8	8	9
b. Two angles buckle out of plane	1	9	0.2	0.8	5	7	7	8
c. W or I shape	1	9	0.2	0.8	6	8	8	9
d. Two channels buckle in plane	1	10	0.2	0.8	6	8	8	9
e. Two channels buckle out of plane	1	9	0.2	0.8	5	7	7	8
f. Concrete-filled tubes	1	8	0.2	0.8	5	7	7	8
g. Rectangular cold-formed tubes 1. $\frac{d}{t} \le \frac{90}{\sqrt{F_y}}$	1	8	0.4	0.8	5	7	7	8
2. $\frac{d}{t} \ge \frac{190}{\sqrt{F_y}}$	1	4	0.2	0.8	2	3	3	4
3. $\frac{90}{\sqrt{F_y}} \le \frac{d}{t} \le \frac{190}{\sqrt{F_y}}$	Use linear interpolation							
h. Circular hollow tubes 1. $\frac{d}{t} \le \frac{1500}{F_y}$	1	10	0.4	0.8	5	7	6	9
2. $\frac{d}{t} \ge \frac{6000}{F_y}$	1	4	0.2	0.8	2	3	3	4
3. $\frac{1500}{F_y} \le \frac{d}{t} \le \frac{6000}{F_y}$	Use linear interpolation							
Braces in Tension	12	15	0.800	1	8	10	12	14
Eccentric Braced Frames								
a. Beams	Governed by link							
b. Braces	Force-controlled, use Equation 3-19							
c. Columns in compression	Force-controlled, use Equation 3-19							
d. Columns in tension	6	8	1.000	1	4	6	7	8

Table 5-8 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Braced Frames and Steel Shear Walls (continued)

Component/Action	$\frac{\Delta}{\Delta_y}$ d	$\frac{\Delta}{\Delta_y}$ e	Residual Force Ratio c	Deformation Primary IO	Primary LS	Primary CP	Secondary LS	Secondary CP
Link Beam[3]								
a.[4] $\frac{2M_{CE}}{eV_{CE}} \le 1.6$	16	18	0.80	1.5	12	15	15	17
b. $\frac{2M_{CE}}{eV_{CE}} \ge 2.6$	Same as for beam in FR moment frame (see Table 5-4)							
c. $1.6 \le \frac{2M_{CE}}{eV_{CE}} \le 2.6$	Use linear interpolation							
Steel Shear Walls[5]	15	17	.07	1.5	11	14	14	16

1. Columns in moment or braced frames need only be designed for the maximum force that can be delivered.
2. Δ_c is the axial deformation at expected buckling load.
3. Deformation is rotation angle between link and beam outside link or column. Assume Δ_y is 0.01 radians for short links.
4. Link beams with three or more web stiffeners. If no stiffeners, use half of these values. For one or two stiffeners, interpolate.
5. Applicable if stiffeners are provided to prevent shear buckling.

5.5.2.4 Rehabilitation Measures for Concentric Braced Frames

Provisions for moment frames may be applicable to braced frames. Braces that are insufficient in strength and/or ductility may be replaced or modified. Insufficient connections may also be modified. Columns may be encased in concrete to improve their performance. For further guidance, see Section 5.4.2.4 and the *Commentary*.

5.5.3 Eccentric Braced Frames (EBF)

5.5.3.1 General

For an EBF, the action lines of the braces do not intersect at the action line of the beam. The distance between the brace action lines where they intersect the beam action line is the eccentricity, *e*. The beam segment between these points is the link beam. The strength of the frame is governed by the strength of the link beam.

5.5.3.2 Stiffness for Analysis

A. Linear Static and Dynamic Procedures

The elastic stiffness of beams, columns, braces, and connections are the same as those used for FR and PR moment frames and CBF. The load-deformation model for a link beam must include shear deformation and flexural deformation.

The elastic stiffness of the link beam, K_e, is

$$K_e = \frac{K_s K_b}{K_s + K_b} \tag{5-26}$$

where

$$K_s = \frac{GA_w}{e} \tag{5-27}$$

and

$$K_b = \frac{12EI_b}{e^3} \tag{5-28}$$

where

A_w = $(d_b - 2t_f)\, t_w$, in.^{2}
e = Length of link beam, in.
G = Shear modulus, k/in.^{2}
K_e = Stiffness of the link beam, k/in.
K_b = Flexural stiffness, kip/in.
K_s = Shear stiffness, kip/in.

The strength of the link beam may be governed by shear, flexure, or the combination of these.

If $e \le \frac{1.6M_{CE}}{V_{CE}}$

$$Q_{CE} = V_{CE} = 0.6F_{ye}t_wA_w \tag{5-29}$$

where

M_{CE} = Expected moment, kip/in.

If $e > \frac{2.6M_{CE}}{V_{CE}}$

$$Q_{CE} = V_{CE} = 2\frac{M_{CE}}{e} \tag{5-30}$$

If $\frac{1.6M_{CE}}{V_{CE}} \le e \le \frac{2.6M_{CE}}{V_{CE}}$, use linear interpolation between Equations 5-29 and 5-30.

The yield deformation is the link rotation as given by

$$\theta_y = \frac{Q_{CE}}{K_e e} \tag{5-31}$$

B. Nonlinear Static Procedure

The NSP requires modeling of the complete nonlinear load-deformation relation to failure for each component. This may be based on experiment, or rational analysis verified by experiment. In lieu of these, the load versus deformation relationship given in Figure 5-1 and Table 5-8 may be used. Q_{CE} and θ_y are calculated in accordance with provisions given in AISC (1994a) or by rational analysis.

The nonlinear models used for beams, columns, and connections for FR and PR moment frames, and for the braces for a CBF, may be used.

C. Nonlinear Dynamic Procedure

The strength and deformation criteria require that the load and resistance relationships given in Equations 3-18 and 3-19 in Chapter 3 be satisfied. The complete hysteretic behavior of each component must be modeled for this procedure. Guidelines for this are given in the *Commentary.*

5.5.3.3 Strength and Deformation Acceptance Criteria

A. Linear Static and Dynamic Procedures

The modeling assumptions given for a CBF are the same as those for an EBF. Values for Q_{CE} and θ_y are given in Section 5.5.3.2 and *m* values are given in Table 5-7. The strength and deformation capacities of the link beam may be governed by shear strength, flexural strength, or their interaction. The values of Q_{CE} and θ_y are the same as those used in the LSP as given in Section 5.5.3.2A. Links and beams are deformation-controlled components and must satisfy Equation 3-18. Columns and braces are to be considered force-controlled members and must satisfy Equation 3-19.

The requirements for link stiffeners, link-to-column connections, lateral supports of the link, the diagonal brace and beam outside the link, and beam-to-column connections given in AISC (1994a) must be met. The brace should be able to carry 1.25 times the link strength to ensure link yielding without brace or column buckling. If this is not satisfied for existing buildings, the design professional shall make extra efforts to verify that the expected link strength will be reached before brace or column buckling. This may require additional inspection and material testing. Where the link beam is attached to the column flange with full-pen welds, the provisions for these connections is the same as for FR frame full-pen connections. The columns of an EBF are force-controlled members. The maximum force deliverable to a column should be calculated from the maximum brace forces equal to 1.25 times the calculated strength of the brace.

B. Nonlinear Static Procedure

The NSP requirements for an EBF are the same as those for a CBF. Modeling of the nonlinear load deformation of the link beam should be based on experiment, or rational analysis verified by experiment. In lieu of these, the conservative approximate behavior depicted in Figure 5-1 may be used. Values for Q_{CE} and θ_y are the same as those used for the LSP. Deformation limits are given in Table 5-8.

C. Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be properly modeled. This behavior must be verified by experiment. This procedure is not recommended in most cases.

5.5.3.4 Rehabilitation Measures for Eccentric Braced Frames

Many of the beams, columns, and braces may be rehabilitated using procedures given for moment frames and CBFs. Cover plates and/or stiffeners may be used for these components. The strength of the link beam 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.

5.6 Steel Plate Walls

5.6.1 General

A steel plate wall develops its seismic resistance through shear stress in the plate wall. In essence, it is a steel shear wall. A solid steel plate, with or preferably without perforations, fills an entire bay between columns and beams. The steel plate is welded to the columns on each side and to the beams above and below. Although these 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. These walls work in conjunction with other existing elements to resist seismic load. However, due to their stiffness, they attract much of the seismic shear. It is essential that the new load paths be carefully established.

5.6.2 Stiffness for Analysis

5.6.2.1 Linear Static and Dynamic Procedures

The most appropriate way to analyze a steel plate wall is to use a plane stress finite element model with the beams and columns as boundary elements. The global stiffness of the wall can be calculated. The modeling can be similar to that used for a reinforced concrete shear wall. A simple approximate stiffness K_w for the wall is

$$K_w = \frac{Ga\ t_w}{h} \tag{5-32}$$

where

G = Shear modulus of steel, ksi

a = Clear width of wall between columns, in.

h = Clear height of wall between beams, in.

t_w = Thickness of plate wall, in.

Other approximations of the wall stiffness based on principles of mechanics are acceptable.

5.6.2.2 Nonlinear Static Procedure

The elastic part of the load-deformation relationship for the wall is given in Section 5.6.2.1. The yield load, Q_{CE}, is given in the next section. The complete nonlinear load-deformation relationship should be based on experiment or rational analysis. In lieu of this, the approximate simplified behavior may be modeled using Figure 5-1 and Table 5-8.

5.6.2.3 Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be properly modeled. This behavior must be verified by experiment. This procedure is not recommended in most cases.

5.6.3 Strength and Deformation Acceptance Criteria

5.6.3.1 Linear Static and Dynamic Procedures

The strength and deformation acceptance criteria for these methods require that the load and resistance relationships given in Equations 3-18 and 3-19 in Chapter 3 be satisfied. The design strength of the steel wall shall be determined using the appropriate equations in Part 6 of AISC (1994a). The wall can be assumed to be like the web of a plate girder. Design restrictions for plate girder webs given in AISC (1994a), particularly those related to stiffener spacing, must be followed. Stiffeners should be spaced such that buckling of the wall does not occur. In this case

$$Q_{CE} = V_{CE} = 0.6 F_{ye} a t_w \qquad (5\text{-}33)$$

In lieu of stiffeners, the steel wall may be encased in concrete. If buckling is not prevented, equations for V_{CE} given in AISC (1994a) for plate girders may be used. The *m* values for steel walls are given in Table 5-7. A steel shear wall is a deformation-controlled component.

5.6.3.2 Nonlinear Static Procedure

The NSP requires modeling of the complete load-deformation behavior to failure. This may be based on experiment or rational analysis. In lieu of these, the conservative approximate behavior depicted in Figure 5-1 may be used, along with parameters given in Table 5-8. The equation for Q_{CE} is Equation 5-33. The yield deformation is

$$\theta_y = \frac{Q_{CE}}{K_w} \qquad (5\text{-}34)$$

5.6.3.3 Nonlinear Dynamic Procedure

The complete hysteretic behavior of each component must be properly modeled. This behavior must be verified by experiment. This procedure is not recommended in most cases.

5.6.4 Rehabilitation Measures

This is not an issue because steel walls in existing construction are rare.

5.7 Steel Frames with Infills

It is common for older existing steel frame buildings to have complete or partial infill walls of reinforced concrete or masonry. Due to the high wall stiffness relative to the frame stiffness, the infill walls will attract most of the seismic shear. In many cases, because these walls are unreinforced or lightly reinforced, their strength and ductility may be inadequate.

The engineering properties and acceptance criteria for the infill walls are presented in Chapter 6 for concrete and Chapter 7 for masonry. The walls may be considered to carry all of the seismic shear in these elements until complete failure of the walls has occurred. After that, the steel frames will resist the seismic forces. Before the loss of the wall, the steel frame adds confining pressure to the wall and enhances its resistance. However, the actual effective forces on the steel frame components are probably minimal. As the frame components begin to develop force they will deform; however, the concrete or masonry on the other side is stiffer so it picks up the load.

The analysis of the component should be done in stages and carried through each performance goal. At the point where the infill has been deemed to fail—as given in Chapter 6 or Chapter 7—the wall should be removed from the analytical model and the analysis resumed with only the bare steel frame in place. At this point, the engineering properties and acceptance criteria for the moment frame given above in Section 5.4 are applicable.

5.8 Diaphragms

5.8.1 Bare Metal Deck Diaphragms

5.8.1.1 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. The metal deck units are often composed of gage thickness steel sheets, from 22 gage down to 14 gage, two to three feet wide, and formed in a repeating pattern with ridges and valleys. Rib depths vary from 1-1/2 to 3 inches in most cases. Decking units are attached to each other and to the structural steel supports by welds or, in some more recent applications, by mechanical fasteners. In large roof structures, these roofs may have supplementary diagonal bracing. (See the description of horizontal steel bracing in Section 5.8.4.)

Chord and collector elements in these diaphragms are considered to be composed of the steel frame elements attached to the diaphragm. Load transfer to frame elements that act as chords or collectors in modern frames is through shear connectors, puddle welds, screws, or shot pins.

5.8.1.2 Stiffness for Analysis

A. Linear Static Procedure

The distribution of forces for existing diaphragms is based on flexible diaphragm assumption, with diaphragms acting as simply supported between the stiff vertical lateral-force-resisting elements. 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 can also be calculated using the Steel Deck Institute Diaphragm Design Manual (Section 3). The analysis should verify that the diaphragm strength is not exceeded for the elastic assumption to hold.

All criteria for existing diaphragms mentioned above apply to stiffened or strengthened diaphragms. Interaction of new and existing elements of strengthened diaphragms must be considered to ensure stiffness compatibility. Load transfer mechanisms between new and existing diaphragm elements must be considered.

Analyses should verify that diaphragm strength is not exceeded, so that elastic assumptions are still valid.

B. Nonlinear Static Procedure

Inelastic properties of diaphragms are usually not included in inelastic seismic analyses.

More flexible diaphragms, such as bare metal deck or deck-formed slabs with long spans between lateral-force-resisting elements, could be subject to inelastic action. Procedures for developing models for inelastic response of wood diaphragms in unreinforced masonry (URM) buildings could be used as the basis for an inelastic model of a bare metal deck diaphragm condition. A strain-hardening modulus of 3% could be used in the post-elastic region. If the weak link of the diaphragm is connection failure, then the element nonlinearity cannot be incorporated into the model.

5.8.1.3 Strength and Deformation Acceptance Criteria

Member 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). (See the references in Section 5.12 and *Commentary* Section C5.12.) Where allowable stresses are given, these may be multiplied by 2.0 in lieu of information provided by the manufacturer or other knowledgeable sources. If bare deck capacity is controlled by connections to frame members or panel buckling, then inelastic action and ductility are limited. Therefore, the deck should be considered to be a force-controlled member.

In many cases, diaphragm failure would not be a life safety consideration unless it led to a loss of bearing support or anchorage. Goals for higher performance would limit the amount of damage to the connections to insure that the load transfer mechanism was still intact. Deformations should be limited to below the threshold of deflections that cause damage to other elements (either structural or nonstructural) at specified Performance Levels.

The *m* value for shear yielding, or panel or plate buckling is 1, 2, or 3 for the IO, LS, or CP Performance Levels, respectively. Weld and connector failure is force-controlled.

The SDI calculations procedure should be used for strengths, or ICBO values with a multiplier may be used to bring allowable values to expected strength levels. Specific references are given in Section 5.12 and in the *Commentary,* Section C5.12.

Connections between metal decks and steel framing commonly use puddle welds. Connection capacity must be checked for the ability to transfer the total diaphragm reaction into the steel framing. Connection capacities are provided in ICBO reports, manufacturers' data, the SDI Manual, or the Welding Code for Sheet Steel, AWS D1.3. Other attachment systems, such as clips, are sometimes used.

5.8.1.4 Rehabilitation Measures

See the *Commentary.*

5.8.2 Metal Deck Diaphragms with Structural Concrete Topping

5.8.2.1 General

Metal deck diaphragms with structural concrete topping are frequently used on floors and roofs of buildings where there are typical floor gravity loads. The metal deck may be either a composite deck, which has indentations, or a noncomposite form deck. In both types of deck, the slab and deck act together to resist diaphragm loads. The concrete fill may be either normal or lightweight concrete, with reinforcing composed of wire mesh or small-diameter reinforcing steel. Additional slab reinforcing may be added at areas of high stress. The metal deck units are composed of gage thickness steel sheets, two to three feet wide, and are formed in a repeating pattern with ridges and valleys. Decking units are attached to each other and to structural steel supports by welds or, in some more recent applications, by mechanical fasteners. Concrete diaphragms in which the slab was formed and the beams are encased in concrete for fire protection may be considered to be similar to topped metal deck diaphragms.

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.

Chord and collector elements in these diaphragms are considered to be composed of the steel frame elements attached to the diaphragm. 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.

5.8.2.2 Stiffness for Analysis

A. Linear Static Procedure

For existing diaphragms, the distribution of forces may be based on a rigid diaphragm assumption if the diaphragm span-to-depth ratio is not greater than five to one. For greater ratios, justify with analysis. Diaphragm flexibility should be included in cases with larger spans and/or plan irregularities by three-dimensional analysis procedures and shell finite elements for the diaphragms. Diaphragm stiffness can be calculated using the SDI Design Manual, manufacturers’ catalogs, or with a representative concrete thickness.

All procedures for existing diaphragms noted above apply to strengthened diaphragms as well. Interaction of new and existing elements of strengthened diaphragms (stiffness compatibility) must be considered. Load transfer mechanisms between new and existing diaphragm components may need to be considered in determining the flexibility of the diaphragm.

All procedures for existing diaphragms noted above apply to new diaphragms. Interaction of new diaphragms with the existing frames must be considered. Load transfer mechanisms between new diaphragm components and existing frames may need to be considered in determining the flexibility of the diaphragm.

For all diaphragms, the analyses must verify that the diaphragm strength is not exceeded, so that elastic assumptions are still valid.

B. Nonlinear Static Procedure

Inelastic properties of diaphragms are usually not included in inelastic seismic analyses, but could be if the connections are adequate. More flexible diaphragms—such as bare metal deck or deck-formed slabs with long spans between lateral-force-resisting elements—could be subject to inelastic action. Procedures for developing models for inelastic response of wood diaphragms in URM buildings could be used as the basis for an inelastic model of a bare metal deck or long span composite diaphragm condition. If the weak link of the diaphragm is connection failure, the element nonlinearity cannot be incorporated into the model.

5.8.2.3 Strength and Deformation Acceptance Criteria

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 that this is the case.

In many cases, diaphragm failure would not be a life safety consideration unless it led to a loss of bearing support or anchorage. Goals for higher performance would limit the amount of damage to the connections or cracking in concrete-filled slabs in order to ensure that the load transfer mechanism was still intact. Deformations should be limited below the threshold of deflections that cause damage to other elements (either structural or nonstructural) at specified Performance Levels.

Connection failure is force-limited, so Equation 3-19 must be used. Shear failure of the deck requires cracking of the concrete and/or tearing of the metal deck, so the *m* values for IO, LS, and CP Performance Levels are 1, 2, and 3, respectively. See Section 5.8.6.3 for acceptance criteria for collectors.

SDI calculation procedures should be used for strengths, or ICBO values with a multiplier of 2.0 should be used to bring allowable values to a strength level. The deck will be considered elastic in most analyses.

Connector capacity must be checked for the ability to transfer the total diaphragm reaction into the supporting steel framing. This load transfer can be achieved by puddle welds and/or headed studs. For the connection of the metal deck to steel framing, puddle welds to beams are most common. Connector capacities are provided in ICBO reports, manufacturers' data, the SDI Manual, or the Welding Code for Sheet Steel, AWS D1.3. Shear studs replace puddle welds to beams where they are required for composite action with supporting steel beams.

Headed studs are most commonly used for connection of the concrete slab to steel framing. Connector capacities can be found using the AISC Manual of Steel Construction, UBC, or manufacturers' catalogs. When steel beams are designed to act compositely with the slab, shear connectors must have the capacity to transfer both diaphragm shears and composite beam shears. In older structures where the beams are encased in concrete, load transfer may be provided through bond between the steel and concrete.

5.8.2.4 Rehabilitation Measures

See the *Commentary.*

5.8.3 Metal Deck Diaphragms with Nonstructural Concrete Topping

5.8.3.1 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), does not have usable structural properties. If the concrete is reinforced, reinforcing consists of wire mesh or small-diameter reinforcing steel. Typically, the metal deck is a form deck or roof decks, so the only attachment between the concrete and metal deck is through bond and friction. The concrete fill is not designed to act compositely with the metal deck and has no positive structural attachment. The metal deck units are typically composed of gage thickness steel sheets, two to three feet wide, and formed in a repeating pattern with ridges and valleys. Decking units are attached to each other and structural steel supports by welds or, in some more recent applications, by mechanical fasteners.

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. Attachment to the steel frame is usually through puddle welds, typically spaced at one to two feet on center. Chord and collector elements in these diaphragms are composed of the steel frame elements attached to the diaphragm.

5.8.3.2 Stiffness for Analysis

A. Linear Static Procedure

The potential for composite action and modification of load distribution must be considered. 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. Interaction of new and existing elements of strengthened diaphragms (stiffness compatibility) must be considered, and the load transfer mechanisms between the new and existing diaphragm elements may need to be considered in determining the flexibility of the diaphragm. Similarly, the interaction of new diaphragms with existing frames must be carefully considered, as well as the load transfer mechanisms between them. Finally, the analyses must verify that diaphragm strength is not exceeded, so elastic assumptions are still valid.

B. Nonlinear Static Procedure

Inelastic properties of diaphragms are usually not included in inelastic seismic analyses. When nonstructural topping is present its capacity must be verified. More flexible diaphragms, such as bare metal deck or decks with inadequate nonstructural topping, could be subject to inelastic action. Procedures for developing models for inelastic response of wood diaphragms in URM buildings could be used as the basis for an inelastic model of a bare metal deck diaphragm condition. If a weak link of the diaphragm is connection failure, then the element nonlinearity cannot be incorporated into the model.

5.8.3.3 Strength and Deformation Acceptance Criteria

A. Linear Static Procedure

Capacities of steel deck diaphragms with nonstructural topping are provided by ICBO reports, by manufacturers, or in general by the SDI Manual. When the connection failure governs, or topping lacks adequate strength, inelastic action and ductility are limited. As a limiting case, the diaphragm shear may be computed using only the bare deck (see Section 5.8.1 for bare decks). Generally, there should be little or no inelastic action in the diaphragms, provided the connections to the framing members are adequate.

In many cases, diaphragm failure would not be a life safety consideration unless it led to a loss of bearing support or anchorage. Goals for higher Performance Levels would limit the amount of damage to the connections or cracking in concrete filled slabs, to ensure that the load transfer mechanism was still intact. Deformations should be limited below the threshold of deflections that cause damage to other elements (either structural or nonstructural) at specified performance levels.

Connection failure is force-limited, so Equation 3-19 must be used. Shear failure of the deck requires concrete cracking and/or tearing of the metal deck, so m values for IO, LS, and CP are 1, 2, and 3, respectively. Panel buckling or plate buckling have m values of 1, 2, and 3 for IO, LS, and CP. SDI calculation procedures should be used for strengths, or ICBO values with a multiplier to bring allowable values to strength levels.

5.8.3.4 Rehabilitation Measures

See the *Commentary.*

5.8.4 Horizontal Steel Bracing (Steel Truss Diaphragms)

5.8.4.1 General

Horizontal steel bracing (steel truss diaphragms) may be used in conjunction with bare metal deck roofs and in conditions where diaphragm stiffness and/or strength is inadequate to transfer shear forces. 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 ratio 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. Where concrete fill is provided over the metal decking, consideration of relative rigidities between the truss and concrete systems may be necessary.

5.8.4.2 Stiffness for Analysis

A. Linear Static Procedure

Existing truss diaphragm systems are modeled as horizontal truss elements (similar to braced steel frames) where axial stiffness controls the deflections. Joints are often taken as pinned. Where joints provide the ability for moment resistance or where eccentricities are introduced at the connections, joint rigidities should be considered. A combination of stiffness with that of concrete fill over metal decking may be necessary in some instances. Flexibility of truss diaphragms should be considered in distribution of lateral loads to vertical elements.

The procedures for existing diaphragms provided above apply to strengthened truss diaphragms. Interaction of new and existing elements of strengthened diaphragm systems (stiffness compatibility) must be considered in cases where steel trusses are added as part of a seismic upgrade. Load transfer mechanisms between new and existing diaphragm elements must be considered in determining the flexibility of the strengthened diaphragm.

The procedures for existing truss diaphragms mentioned above also apply to new diaphragms. Interaction of new truss diaphragms with existing frames must be considered. Load transfer mechanisms between new diaphragm elements and existing frames may need to be considered in determining the flexibility of the diaphragm/frame system.

For modeling assumptions and limitations, see the preceding comments related to truss joint modeling, force transfer, and interaction between diaphragm elements. Analyses are also needed to verify that elastic diaphragm response assumptions are still valid.

Acceptance criteria for the components of a truss diaphragm are the same as for a CBF.

B. Nonlinear Static Procedure

Inelastic properties of truss diaphragms are usually not included in inelastic seismic analyses. In the case of truss diaphragms, inelastic models similar to those of braced steel frames may be appropriate. Inelastic deformation limits of truss diaphragms may be different from those prescribed for braced steel frames (e.g., more consistent with that of a concrete-topped diaphragm).

5.8.4.3 Strength and Deformation Acceptance Criteria

Member capacities of truss diaphragm members may be calculated in a manner similar to those for braced steel frame members. It may be necessary to include gravity force effects in the calculations for some members of these trusses. Lateral support conditions provided by metal deck, with or without concrete fill, must be properly considered. Force transfer mechanisms between various members of the truss at the connections, and between trusses and frame elements, must be considered to verify the completion of the load path.

In many cases, diaphragm distress would not be a life safety consideration unless it led to a loss of bearing support or anchorage. Goals for higher Performance Levels would limit the amount of damage to the connections or bracing elements, to insure that the load transfer mechanism was still complete. Deformations should be limited below the threshold of deflections that cause damage to other elements (either structural or nonstructural) at specified Performance Levels. These values must be established in conjunction with those of braced steel frames.

The *m* values to be used are half of those for components of a CBF as given in Table 5-7.

A. Nonlinear Static Procedure

Procedures similar to those used for a CBF should be used, but deformation limits shall be half of those given for CBFs in Table 5-8.

5.8.4.4 Rehabilitation Measures

See the *Commentary.*

5.8.5 Archaic Diaphragms

5.8.5.1 General

Archaic diaphragms in steel buildings generally consist 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. Archaic steel diaphragm elements are almost always found in older steel buildings in conjunction with vertical systems that are 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.

5.8.5.2 Stiffness for Analysis

A. Linear Static Procedure

Existing archaic diaphragm systems are modeled as a horizontal diaphragm with equivalent thickness of arches and concrete fill. Development of truss elements between steel beams and compression elements of arches could also be considered. Flexibility of archaic diaphragms should be considered in the distribution of lateral loads to vertical elements, especially if spans are large.

All preceding comments for existing diaphragms apply for archaic diaphragms. Interaction of new and existing elements of strengthened elements (stiffness compatibility) must be considered in cases where steel trusses are added as part of a seismic upgrade. Load transfer mechanisms between new and existing diaphragm elements must be considered in determining the flexibility of the strengthened diaphragm.

For modeling assumptions and limitations, see the preceding comments related to force transfer, and interaction between diaphragm elements. Analyses are required to verify that elastic diaphragm response assumptions are valid.

B. Nonlinear Static Procedure

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.

5.8.5.3 Strength and Deformation Acceptance Criteria

Member capacities of archaic diaphragm components can be calculated assuming little or no tension capacity except for the steel beam members. Gravity force effects must be included in the calculations for all components of these diaphragms. Force transfer mechanisms between various members and between frame elements must be considered to verify the completion of the load path.

In many cases, diaphragm distress could result in life safety considerations, due to possible loss of bearing support for the elements of the arches. Goals for higher performance would limit the amount of diagonal tension stresses, to insure that the load transfer mechanism was still complete. Deformations should be limited below the threshold of deflections that cause damage to other elements (either structural or nonstructural) at specified Performance Levels. These values must be established in conjunction with those for steel frames. Archaic diaphragm components should be considered as force-limited, so Equation 3-19 must be used.

5.8.5.4 Rehabilitation Measures

See the *Commentary.*

5.8.6 Chord and Collector Elements

5.8.6.1 General

Chords and collectors for all the previously described diaphragms typically consist of the steel framing that supports the diaphragm. When structural concrete is present, additional slab reinforcing may 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 is typically attached to the deck with spot welds or by mechanical fasteners. When reinforcing acts as the chord or collector, load transfer occurs through bond between the reinforcing bars and the concrete.

5.8.6.2 Stiffness for Analysis

Modeling assumptions similar to those for equivalent frame members should be used.

5.8.6.3 Strength and Deformation Acceptance Criteria

Capacities of chords and collectors are provided by the AISC LRFD Specifications (1994a) and ACI-318 (ACI, 1995; see Chapter 6 for the citation) design guides. Inelastic action may occur, depending on the configuration of the diaphragm. It is desirable to design chord and collector components for a force that will develop yielding or ductile failure in either the diaphragm or vertical lateral-force-resisting system, so that the chords and collectors are not the weak link in the load path. In some cases, failure of chord and collector components may result in a life safety consideration when beams act as the chords or collectors and vertical support is compromised. Goals for higher performance would limit stresses and damage in chords and collectors, keeping the load path intact.

In buildings where the steel framing members that support the diaphragm act as collectors, the steel components may be alternately in tension and compression. If all connections to the diaphragm are sufficient, the diaphragm will prevent buckling of the chord member so values of m equal to 1, 6, and 8 may be used for IO, LS, and CP, respectively. If the diaphragm provides only limited support against buckling of the chord or collector, values of m equal to 1, 2, and 3 should be used. Where chords or collectors carry gravity loads along with seismic loads, they should be checked as members with combined loading using Equations 5-10 and 5-11. Welds and connectors joining the diaphragms to the collectors should be considered to be force-controlled.

5.8.6.4 Rehabilitation Measures

See the *Commentary.*

5.9 Steel Pile Foundations

5.9.1 General

Steel piles are one of the most common components for building foundations. Wide flange shapes (H piles) or structural tubes, with and without concrete infills, are the most commonly used shapes. Piles are usually driven in groups. A reinforced concrete pile cap is then cast over each group, and a steel column with a base plate is attached to the pile cap with anchor bolts.

The piles provide strength and stiffness to the foundation in one of two ways. Where very strong soil or rock lies at not too great a distance below the building site, the pile forces are transferred directly to the soil or rock at the bearing surface. Where this condition is not met, the piles are designed to transfer their load to the soil through friction. The design of the entire foundation is covered in Chapter 4 of these *Guidelines*. The design of the steel piles is covered in the following subsections.

5.9.2 Stiffness for Analysis

If the pile cap is below grade, the foundation attains much of its stiffness from the pile cap bearing against the soil. Equivalent soil springs may be derived as discussed in Chapter 4. The piles may also provide significant stiffness through bending and bearing against the soil. The effective pile contribution to stiffness is decreased if the piles are closely spaced; this group effect must be taken into account when calculating foundation and strength. For a more detailed description, see the *Commentary,* Section C5.9.2, and Chapter 4 of these *Guidelines.*

5.9.3 Strength and Deformation Acceptance Criteria

Buckling of steel piles is not a concern, since the soil provides lateral support. The moments in the piles may be calculated in one of two ways. The first is an elastic method that requires finding the effective point of fixity; the pile is then designed as a cantilever column. The second, a nonlinear method, requires a computer program that is available at no cost. Details are given in the *Commentary,* Section C5.9.2.

Once the axial force and maximum bending moments are known, the pile strength acceptance criteria are the same as for a steel column, as given in Equation 5-10. The expected axial and flexural strengths in Equation 5-10 are computed for an unbraced length equal to zero. Note that Equation 5-11 does not apply to steel piles. Exceptions to these criteria, where liquefaction is a concern, are discussed in the *Commentary,* Section C5.9.2.

5.9.4 Rehabilitation Measures for Steel Pile Foundations

Rehabilitation of the pile cap is covered in Chapter 6. Chapter 4 covers general criteria for the rehabilitation of the foundation element. In most cases, it is not possible to rehabilitate the existing piles. Increased stiffness and strength may be gained by driving additional piles near existing groups and then adding a new pile cap. Monolithic behavior can be gained by connecting the new and old pile caps with epoxied dowels, or other means.

5.10 Definitions

Beam: A structural member whose primary function is to carry loads transverse to its longitudinal axis; usually a horizontal member in a seismic frame system.

Braced frame: An essentially vertical truss system of concentric or eccentric type that resists lateral forces.

Concentric braced frame (CBF): A braced frame in which the members are subjected primarily to axial forces.

Connection: A link between components or elements that transmits actions from one component or element to another component or element. Categorized by type of action (moment, shear, or axial), connection links are frequently nonductile.

Continuity plates: Column stiffeners at the top and bottom of the panel zone.

Diagonal bracing: Inclined structural members carrying primarily axial load, employed to enable a structural frame to act as a truss to resist horizontal loads.

Dual system: A structural system included in buildings with the following features:

- An essentially complete space frame provides support for gravity loads.
- Resistance to lateral load is provided by concrete or steel shear walls, steel eccentrically braced frames (EBF), or concentrically braced frames (CBF) along with moment-resisting frames (Special Moment Frames, or Ordinary Moment Frames) that are capable of resisting at least 25% of the lateral loads.
- Each system is also designed to resist the total lateral load in proportion to its relative rigidity.

Eccentric braced frame (EBF): A diagonal braced frame in which at least one end of each diagonal bracing member connects to a beam a short distance from either a beam-to-column connection or another brace end.

Joint: An area where two or more ends, surfaces, or edges are attached. Categorized by the type of fastener or weld used and the method of force transfer.

Lateral support member: A member designed to inhibit lateral buckling or lateral-torsional buckling of a component.

Link: In an EBF, the segment of a beam that extends from column to brace, located between the end of a diagonal brace and a column, or between the ends of two diagonal braces of the EBF. The length of the link is defined as the clear distance between the diagonal brace and the column face, or between the ends of two diagonal braces.

Link intermediate web stiffeners: Vertical web stiffeners placed within the link.

Link rotation angle: The angle of plastic rotation between the link and the beam outside of the link derived using the specified base shear, *V*.

LRFD (Load and Resistance Factor Design): A method of proportioning structural components (members, connectors, connecting elements, and assemblages) using load and resistance factors such that no applicable limit state is exceeded when the structure is subjected to all design load combinations.

Moment frame: A building frame system in which seismic shear forces are resisted by shear and flexure in members and joints of the frame.

Nominal strength: The capacity of a structure or component to resist the effects of loads, as determined by (1) computations using specified material strengths and dimensions, and formulas derived from accepted principles of structural mechanics, or (2) field tests or laboratory tests of scaled models, allowing for modeling effects, and differences between laboratory and field conditions.

Ordinary Moment Frame (OMF): A moment frame system that meets the requirements for Ordinary Moment Frames as defined in seismic provisions for new construction in AISC (1994a), Chapter 5.

P-Δ effect: The secondary effect of column axial loads and lateral deflection on the shears and moments in various components of a structure.

Panel zone: The area of a column at the beam-to-column connection delineated by beam and column flanges.

Required strength: The load effect (force, moment, stress, as appropriate) acting on a component or connection, determined by structural analysis from the factored loads (using the most appropriate critical load combinations).

Resistance factor: A reduction factor applied to member resistance that accounts for unavoidable deviations of the actual strength from the nominal value, and the manner and consequences of failure.

Slip-critical joint: A bolted joint in which slip resistance of the connection is required.

Special Moment Frame (SMF): A moment frame system that meets the special requirements for frames as defined in seismic provisions for new construction.

Structural system: An assemblage of load-carrying components that are joined together to provide regular interaction or interdependence.

V-braced frame: A concentric braced frame (CBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span. Where the diagonal braces are below the beam, the system also is referred to as an "inverted V-brace frame," or "chevron bracing."

X-braced frame: A concentric braced frame (CBF) in which a pair of diagonal braces crosses near the mid-length of the braces.

Y-braced frame: An eccentric braced frame (EBF) in which the stem of the Y is the link of the EBF system.

5.11 Symbols

This list may not contain symbols defined at their first use if not used thereafter.

A_b Gross area of bolt or rivet, in.^{2}

A_c Rivet area, in.^{2}

A_e Effective net area, in.^{2}

A_f Flange area of member, in.^{2}

A_g Gross area, in.^{2}

A_{st} Area of link stiffener, in.^{2}

A_w Effective area of weld, in.^{2}

C_b Coefficient to account for effect of nonuniform moment; given in AISC (1994a)

E Young's modulus of elasticity, 29,000 ksi

F_{EXX} Classification strength of weld metal, ksi

F_{te} Expected tensile strength, ksi

F_v Design shear strength of bolts or rivets, ksi

F_y Specified minimum yield stress for the type of steel being used, ksi

F_{yb} F_y of a beam, ksi

F_{yc} F_y of a column, ksi

F_{ye} Expected yield strength, ksi

F_{yf} F_y of a flange, ksi

G Shear modulus of steel, 11,200 ksi

I_b Moment of inertia of a beam, in.^{4}

I_c Moment of inertia of a column

K Length factor for brace (see AISC, 1994a)

K_e Stiffness of a link beam, kip/in.

K_s Rotational stiffness of a connection, kip-in./rad

K_w Stiffness of wall, kip/in.

K_θ Rotational stiffness of a partially restrained connection, kip-in./rad

L Length of bracing member, in.

L_p The limiting unbraced length between points of lateral restraint for the full plastic moment capacity to be effective (see AISC, 1994a)

L_r The limiting unbraced length between points of lateral support beyond which elastic lateral torsional buckling of the beam is the failure mode (see AISC, 1994a)

M_{CE} Expected flexural strength of a member or joint, kip-in.

M_{CEx} Expected bending strength of a member about the x-axis, kip-in.

M_{CEy} Expected bending strength of a member about y-axis, kip-in.

M_p Plastic bending moment, kip-in.

M_x Bending moment in a member for the x-axis, kip-in.

M_y Bending moment in a member for the y-axis, kip-in.

N_b Number of bolts or rivets

P Axial force in a member, kips

PR Partially restrained

P_{cr} Critical compression strength of bracing, kips

P_{CL} Lower-bound axial strength of column, kips

P_u Required axial strength of a column or a link, kips

P_{ye} Expected yield axial strength of a member = $F_{ye}A_g$, kips

Q_{CE} Expected strength of a component or element at the deformation level under consideration in a deformation-controlled action

Q_{CL} Lower-bound estimate of the strength of a component or element at the deformation level under consideration for a force-controlled action

V_{CE} Expected shear strength of a member, kips

V_{CE} Shear strength of a link beam, kips

V_{ya} Nominal shear strength of a member modified by the axial load magnitude, kips

Z Plastic section modulus, in.^{3}

a Clear width of wall between columns

b Width of compression element, in.

b_a Connection dimension

b_{cf} Column flange width, in.

b_f Flange width, in.

b_t Connection dimension

d Overall depth of member, in.

d_b Overall beam depth, in.

d_c Overall column depth, in.

d_v Bolt or rivet diameter, in.

d_z Overall panel zone depth between continuity plates, in.

e EBF link length, in.

h Average story height above and below a beam-column joint

h Clear height of wall between beams

h Distance from inside of compression flange to inside of tension flange, in.

h_c Assumed web depth for stability, in.

h_v Height of story v

k_v Shear buckling coefficient

l_b Length of beam

l_c Length of column

m A modification factor used in the acceptance criteria of deformation-controlled components or elements, indicating the available ductility of a component action

m_e Effective m

m_x Value of m for bending about x-axis of a member

m_y Value of m for bending about y-axis of a member

r Governing radius of gyration, in.

r_y Radius of gyration about y axis, in.

t Thickness of link stiffener, in.

t_a Thickness of angle, in.

t_{bf} Thickness of beam flange, in.

t_{cf} Thickness of column flange, in.

t_f Thickness of flange, in.

t_p Thickness of panel zone including doubler plates, in.

t_p Thickness of flange plate, in.

t_w Thickness of web, in.

t_w Thickness of plate wall

t_z Thickness of panel zone (doubler plates not necessarily included), in.

w Length of flange angle

w_z Width of panel zone between column flanges, in.

Δ Generalized deformation, unitless

Δ_i Inter-story displacement (drift) of story i divided by the story height

Δ_y Generalized yield deformation, unitless

θ Generalized deformation, radians

θ_i Inter-story drift ratio, radians

θ_y Generalized yield deformation, radians

κ A reliability coefficient used to reduce component strength values for existing components based on the quality of knowledge about the components' properties (see Section 2.7.2)

λ Slenderness parameter

λ_p Limiting slenderness parameter for compact element

λ_r Limiting slenderness parameter for noncompact element

ρ Ratio of required axial force (P_u) to nominal shear strength (V_y) of a link

ρ_{lp} Yield deformation of a link beam

ϕ Resistance factor = 1.0

5.12 References

ACI, 1995, *Building Code Requirements for Reinforced Concrete: ACI 318-95*, American Concrete Institute, Detroit, Michigan.

AISC, 1994a, *Manual of Steel Construction, Load and Resistance Factor Design Specification for Structural Steel Buildings (LRFD), Volume I, Structural Members, Specifications and Codes*, American Institute of Steel Construction, Chicago, Illinois.

AISC, 1994b, *Manual of Steel Construction, Load and Resistance Factor Design, Volume II, Connections*, American Institute of Steel Construction, Chicago, Illinois.

AISI, 1973, *The Criteria for Structural Applications for Steel Cables for Building, 1973 Edition*, American Iron and Steel Institute, Washington, D.C.

AISI, 1986, *Specification for the Design of Cold-Formed Steel Structural Members*, August 10, 1986 edition with December 11, 1989 Addendum, American Iron and Steel Institute, Chicago, Illinois.

ASCE, 1990, *Specification for the Design of Cold-Formed Steel Stainless Steel Structural Members*, Report No. ASCE-8, American Society of Civil Engineers, New York, New York.

BSSC, 1992, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, developed by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report No. FEMA 178), Washington, D.C.

BSSC, 1995, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings, 1994 Edition, Part 1: Provisions and Part 2: Commentary,* prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency (Reports No. FEMA 222A and 223A), Washington, D.C.

SAC, 1995, *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, Report No. FEMA 267, developed by the SEAOC, ATC, and CUREE Joint Venture (Report No. SAC-95-02) for the Federal Emergency Management Agency, Washington, D.C.

SDI, latest edition, *SDI Design Manual for Composite Decks, Form Decks and Roof Decks*, Steel Diaphragm Institute.

SJI, 1990, *Standard Specification, Load Tables and Weight Tables for Steel Joists and Joist Girders*, Steel Joist Institute, 1990 Edition.