COMMENTARY
on the Specification for Structural Steel Buildings

March 9, 2005

(The Commentary is not a part of ANSI/AISC 360-05, *Specification for Structural Steel Buildings*, but is included for informational purposes only.)

INTRODUCTION

The Specification is intended to be complete for normal design usage.

The Commentary furnishes background information and references for the benefit of the design professional seeking further understanding of the basis, derivations and limits of the specification.

The Specification and Commentary are intended for use by design professionals with demonstrated engineering competence.

Commentary Glossary

The Commentary uses the following terms in addition to the terms defined in the Glossary of the Specification. Only the terms listed below are *italicized* where they first appear in the Commentary text.

Alignment chart. Nomograph for determining the effective length factor *K* for some types of columns.

Biaxial bending. Simultaneous bending of a member about two perpendicular axes.

Brittle fracture. Abrupt cleavage with little or no prior ductile deformation.

Column curve. Curve expressing the relationship between axial column strength and slenderness ratio.

Critical load. Load at which a perfectly straight member under compression may either assume a deflected position or may remain undeflected, or a beam under flexure may either deflect and twist out of plane or remain in its in-plane deflected position, as determined by a theoretical stability analysis.

Cyclic load. Repeatedly applied external load that may subject the structure to fatigue.

Drift damage index. Parameter used to measure the potential damage caused by interstory drift.

Effective moment of inertia. Moment of inertia of the cross section of a member that remains elastic when partial plastification of the cross section takes place, usually under the combination of residual stress and applied stress. Also, the moment of inertia based on effective widths of elements that buckle locally. Also, the moment of inertia used in the design of partially composite members.

Effective stiffness. Stiffness of a member computed using the effective moment of inertia of its cross section.

Fatigue threshold. Stress range at which fatigue cracking will not initiate regardless of the number of cycles of loading.

First order plastic analysis. *Structural analysis* based on the assumption of rigid-plastic behavior—in other words, that equilibrium is satisfied throughout the structure and the stress is at or below the yield stress—and in which equilibrium conditions are formulated on the undeformed structure.

Flexible connection. Connection permitting a portion, but not all, of the simple beam rotation of a member end.

Flexural-torsional buckling. Buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

Inelastic action. Material deformation that does not disappear on removal of the force that produced it.

Inelastic strength. Strength of a structure or component after material has achieved the *yield stress* at sufficient locations that a strength *limit state* is reached.

Interstory drift. Lateral deflection of a floor relative to the lateral deflection of the floor immediately below, divided by the distance between floors, $(\delta_n - \delta_{n-1})/h$.

Permanent load. *Load* in which variations over time are rare or of small magnitude. All other *loads* are *variable loads*.

Primary member. For ponding analysis, beam or girder that supports the concentrated reactions from the secondary members framing into it.

Residual stress. Stress that remains in an unloaded member after it has been formed into a finished product. (Examples of such stresses include, but are not limited to, those induced by cold bending, cooling after rolling or welding).

Rigid frame. Structure in which connections maintain the angular relationship between beam and column members under load.

Secondary member. For ponding analysis, beam or joist that directly supports the distributed ponding loads on the roof of the structure.

Sidesway. Lateral movement of a structure under the action of lateral loads, unsymmetrical vertical loads or unsymmetrical properties of the structure.

Sidesway buckling. Buckling mode of a multistory frame precipitated by the relative lateral displacements of joints, leading to failure by sidesway of the frame.

Squash load. Column area multiplied by the yield stress.

St. Venant torsion. Portion of the torsion in a member that induces only shear stresses in the member.

Strain hardening. Phenomenon wherein ductile steel, after undergoing considerable deformation at or just above yield point, exhibits the capacity to resist substantially higher loading than that which caused initial yielding.

Subassemblage. Truncated portion of a structural frame.

Tangent modulus. At any given stress level, the slope of the stress-strain curve of a material in the inelastic range as determined by the compression test of a small specimen under controlled conditions.

Total building drift. Lateral frame deflection at the top of the most occupied floor divided by the height of the building to that level, Δ/H.

Undercut. Notch resulting from the melting and removal of base metal at the edge of a weld.

Variable load. Load with substantial variation over time.

Warping torsion. Portion of the total resistance to torsion that is provided by resistance to warping of the cross section.

Yield plateau. Portion of the stress-strain curve for uniaxial tension or compression in which the stress remains essentially constant during a period of substantially increased strain.

CHAPTER A

GENERAL PROVISIONS

A1.   SCOPE

The scope of this Specification is broader than that of the two AISC Specifications that it replaces: the 1999 *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC, 2000b) and the 1989 *ASD Specification* (AISC, 1989). This Specification combines these two previous Specifications and incorporates the provisions of the *Load and Resistance Factor Design Specification for Steel Hollow Structural Sections* (AISC, 2000), the *Specification for Allowable Stress Design of Single-Angle Members* (AISC, 1989) and the *Load and Resistance Factor Design Specification for Single-Angle Members* (AISC, 2000a). The basic purpose of the provisions in this Specification is the determination of the available and nominal strength of the members, connections and other components of steel building structures. The nominal strength is usually defined in terms of resistance to a load effect, such as axial force, bending moment, shear or torque, but in some instances it is expressed in terms of a stress.

This Specification provides two methods of design:

(1) **Load and Resistance Factor Design (LRFD):** The nominal strength is multiplied by a resistance factor ϕ, and the resulting design strength is then required to equal or exceed the required strength determined by structural analysis for the appropriate LRFD load combination specified by the applicable building code.

(2) **Allowable Strength Design (ASD):** The nominal strength is divided by a safety factor Ω, and the resulting allowable strength is then required to equal or exceed the required strength determined by structural analysis for the appropriate ASD load combination specified by the applicable building code.

This Specification gives provisions for determining the values of the nominal strengths according to the applicable limit states and lists the corresponding values of the resistance factor ϕ and the safety factor Ω. The ASD safety factors are calibrated to give the same structural reliability and the same component size as the LRFD method at a live-to-dead load ratio of 3.

This Specification is applicable to both buildings and other structures. Many structures found in petrochemical plants, power plants, and other industrial applications are designed, fabricated and erected in a manner similar to buildings. It is not intended that this Specification address steel structures with vertical and lateral load-resisting systems that are not similar to buildings, such as those constructed of shells or catenary cables.

For the purposes of this Specification, HSS are defined as hollow structural sections with constant wall thickness and a round, square or rectangular cross section that is constant along the length of the member. HSS are manufactured by forming skelp (strip or plate) to the desired shape and joining the edges with a continuously welded seam. Published information is available describing the details of the various methods used to manufacture HSS (Graham, 1965; STI, 1996).

The *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005) defines the practices that are the commonly accepted standards of custom and usage for structural steel fabrication and erection. As such, the *Code of Standard Practice* is primarily intended to serve as a contractual document to be incorporated into the contract between the buyer and seller of fabricated structural steel. Some parts of the *Code of Standard Practice*, however, form the basis for some of the provisions in this Specification. Therefore, the *Code of Standard Practice* is referenced in selected locations in this Specification to maintain the ties between these documents, where appropriate.

A2.   REFERENCED SPECIFICATIONS, CODES AND STANDARDS

Section A2 provides references to documents cited in this Specification. Note that not all grades of a particular material specification are necessarily approved for use according to this Specification. For a list of approved materials and grades, see Section A3.

A3.   MATERIAL

1.   Structural Steel Materials

1a.   ASTM Designations

There are hundreds of steel materials and products. This Specification lists those products/materials that are commonly useful to structural engineers and those that have a history of satisfactory performance. Other materials may be suitable for specific applications, but the evaluation of those materials is the responsibility of the engineer specifying them. In addition to typical strength properties, considerations for materials may include but are not limited to strength properties in transverse directions, ductility, formability, soundness, weldability including sensitivity to thermal cycles, notch toughness and other forms of crack sensitivity, coatings and corrosivity. Consideration for product form may include material considerations in addition to effects of production, tolerances, testing, reporting and surface profiles.

Hot-Rolled Structural Shapes. The grades of steel approved for use under this Specification, covered by ASTM specifications, extend to a yield stress of 100 ksi (690 MPa). Some of the ASTM specifications specify a minimum yield point, while others specify a minimum yield strength. The term "yield stress" is used in this Specification as a generic term to denote either the yield point or the yield strength.

It is important to be aware of limitations of availability that may exist for some combinations of strength and size. Not all structural section sizes are included in the various material specifications. For example, the 60 ksi (415 MPa) yield stress steel in the A572/A572M specification includes plate only up to $1^{1}/_{4}$ in. (32 mm) in thickness. Another limitation on availability is that even when a product is included in this Specification, it may be infrequently produced by the mills. Specifying these products may result in procurement delays or require ordering large quantities directly from the producing mills. Consequently, it is prudent to check availability before completing the details of a design. The AISC web site provides this information (www.aisc.org) and AISC's *Modern Steel Construction* publishes tables on availability twice per year.

Properties in the direction of rolling are of principal interest in the design of steel structures. Hence, yield stress as determined by the standard tensile test is the principal mechanical property recognized in the selection of the steels approved for use under this Specification. It must be recognized that other mechanical and physical properties of rolled steel, such as anisotropy, ductility, notch toughness, formability, corrosion resistance, etc., may also be important to the satisfactory performance of a structure.

It is not possible to incorporate in the Commentary adequate information to impart full understanding of all factors that might merit consideration in the selection and specification of materials for unique or especially demanding applications. In such a situation the user of the Specification is advised to make use of reference material contained in the literature on the specific properties of concern and to specify supplementary material production or quality requirements as provided for in ASTM material specifications. One such case is the design of highly restrained welded connections (AISC, 1973). Rolled steel is anisotropic, especially insofar as ductility is concerned; therefore, weld contraction strains in the region of highly restrained welded connections may exceed the strength of the material if special attention is not given to material selection, details, workmanship and inspection.

Another special situation is that of fracture control design for certain types of service conditions (AASHTO, 1998). For especially demanding service conditions such as structures exposed to low temperatures, particularly those with impact loading, the specification of steels with superior notch toughness may be warranted. However, for most buildings, the steel is relatively warm, strain rates are essentially static, and the stress intensity and number of cycles of full design stress are low. Accordingly, the probability of fracture in most building structures is low. Good workmanship and good design details incorporating joint geometry that avoids severe stress concentrations are generally the most effective means of providing fracture-resistant construction.

Hollow Structural Sections (HSS). Specified minimum tensile properties are summarized in Table C-A3.1 for various HSS and pipe material specifications and grades. ASTM A53 Grade B is included as an approved pipe material

TABLE C-A3.1
Minimum Tensile Properties of HSS
and Pipe Steels

Specification	Grade	F_y, ksi (MPa)	F_u, ksi (MPa)
ASTM A53	**B**	35 (240)	60 (415)
ASTM A500 (round)	**A** **B** **C**	33 (228) 42 (290) 46 (317)	45 (311) 58 (400) 62 (428)
ASTM A500 (rectangular)	**A** **B** **C**	39 (269) 46 (317) 50 (345)	45 (311) 58 (400) 62 (428)
ASTM A501	**–**	36 (248)	58 (400)
ASTM A618 (round)	**I and II** **III**	50 (345) 50 (345)	70 (483) 65 (450)
ASTM A847	**–**	50 (345)	70 (483)
CAN/CSA-G40.20/G40.21	**350W**	51 (350)	65 (450)

specification because it is the most readily available round product in the United States. Other North American HSS products that have properties and characteristics that are similar to the approved ASTM products are produced in Canada under the *General Requirements for Rolled or Welded Structural Quality Steel* (CSA, 2003). In addition, pipe is produced to other specifications that meet the strength, ductility and weldability requirements of the materials in Section A3, but may have additional requirements for notch toughness or pressure testing.

Pipe can be readily obtained in ASTM A53 material and round HSS in ASTM A500 Grade B is also common. For rectangular HSS, ASTM A500 Grade B is the most commonly available material and a special order would be required for any other material. Depending upon size, either welded or seamless round HSS can be obtained. In North America, however, all ASTM A500 rectangular HSS for structural purposes are welded. Rectangular HSS differ from box sections in that they have uniform thickness except for some thickening in the rounded corners.

ASTM A500 Grade A material does not meet the ductility "limit of applicability" for direct connections in Section K2.3a(12). This limit requires that $F_y/F_u \leq$ 0.8. In determining that other materials meet the ductility limit, it is important to note that ASTM A500 permits the yield strength to be determined by either the 0.2 percent offset method or at 0.5 percent elongation under load (EUL). Since ASTM A500 materials are cold-formed and have rounded stress-strain curves with no *yield plateau*, the latter method indicates yield strengths greater than the 0.2 percent offset. The ductility limit is intended to apply to yield strengths determined by the 0.2 percent offset. However, mill reports may indicate the EUL yield, raising concerns that the material does not have adequate ductility. Supplemental tension tests may be required to determine the 0.2 percent offset yield strength.

Even though ASTM A501 includes rectangular HSS, hot-formed rectangular HSS are not currently produced in the United States. The *General Requirements for Rolled or Welded Structural Quality Steel* (CSA, 2003) includes Class C (cold-formed) and Class H (cold-formed and stress relieved) HSS. Class H HSS have relatively low levels of *residual stress*, which enhances their performance in compression and may provide better ductility in the corners of rectangular HSS.

1c.   Rolled Heavy Shapes

The web-to-flange intersection and the web center of heavy hot-rolled shapes, as well as the interior portions of heavy plates, may contain a more coarse grain structure and/or lower notch toughness material than other areas of these products. This is probably caused by ingot segregation, the somewhat lesser deformation during hot rolling, higher finishing temperature, and the slower cooling rate after rolling for these heavy sections. This characteristic is not detrimental to suitability for compression members or for nonwelded members. However, when heavy cross sections are joined by splices or connections using complete-joint-penetration welds that extend through the coarser and/or lower notch-tough interior portions, tensile strains induced by weld shrinkage may result in cracking. An example is a complete-joint-penetration groove welded connection of a heavy cross section beam to any column section. When members of lesser thickness are joined by complete-joint-penetration groove welds, which induce smaller weld shrinkage strains, to the finer grained and/or more notch-tough surface material of ASTM A6/A6M shapes and heavy built-up cross sections, the potential for cracking is significantly lower. An example is a complete-joint-penetration groove welded connection of a nonheavy cross-section beam to a heavy cross-section column.

For critical applications such as primary tension members, material should be specified to provide adequate notch toughness at service temperatures. Because of differences in the strain rate between the Charpy V-Notch (CVN) impact test and the strain rate experienced in actual structures, the CVN test is conducted at a temperature higher than the anticipated service temperature for the structure. The location of the CVN test specimens ("alternate core location") is specified in ASTM A6/A6M, Supplemental Requirement S30.

The notch toughness requirements of Section A3.1c are intended only to provide material of reasonable notch toughness for ordinary service applications. For unusual applications and/or low temperature service, more restrictive requirements and/or notch toughness requirements for other section sizes and thicknesses may be appropriate. To minimize the potential for fracture, the notch toughness requirements of Section A3.1c must be used in conjunction with good design and fabrication procedures. Specific requirements are given in Sections J1.5, J1.6, J2.6 and J2.7.

For rotary-straightened W-shapes, an area of reduced notch toughness has been documented in a limited region of the web immediately adjacent to the flange. This

region may exist in W-shapes of all weights, not just heavy shapes. Considerations in design and detailing that recognize this situation are presented in Chapter J.

2. Steel Castings and Forgings

There are a number of ASTM specifications for steel castings. The SFSA *Steel Castings Handbook* (SFSA, 1995) recommends ASTM A216 as a product useful for steel structures. In addition to the requirements of this Specification, SFSA recommends that various other requirements be considered for cast steel products. It may be appropriate to inspect the first piece cast using magnetic particle inspection in accordance with ASTM E125, degree 1a, b, or c. Radiographic inspection level III may be desirable for critical sections of the first piece cast. Ultrasonic testing (UT) in compliance with ASTM E609 may be appropriate for first cast piece over 6 in. thick. Design approval, sample approval, periodic nondestructive testing of the mechanical properties, chemical testing, and selection of the correct welding specification should be among the issues defined in the selection and procurement of cast steel products. Refer to SFSA (1995) for design information about cast steel products.

3. Bolts, Washers and Nuts

The ASTM standard specification for A307 bolts covers two grades of fasteners. Either grade may be used under this Specification; however, it should be noted that Grade B is intended for pipe flange bolting and Grade A is the grade long in use for structural applications.

4. Anchor Rods and Threaded Rods

ASTM F1554 is the primary specification for anchor rods. Since there is a limit on the maximum available length of ASTM A325/A325M and ASTM A490/A490M bolts, the attempt to use these bolts for anchor rods with design lengths longer than the maximum available lengths has presented problems in the past. The inclusion of ASTM A449 and A354 materials in this Specification allows the use of higher strength material for bolts longer than ASTM A325/A325M and ASTM A490/A490M bolts.

The engineer of record should specify the required strength for threaded rods used as load-carrying members.

5. Filler Metal and Flux for Welding

The AWS Filler Metal Specifications listed in Section A3.5 are general specifications that include filler metal classifications suitable for building construction, as well as classifications that may not be suitable for building construction. The AWS D1.1, *Structural Welding Code Steel* (AWS, 2004) lists in Table 3.1 various electrodes that may be used for prequalified welding procedure specifications, for the various steels that are to be joined. This list specifically does not include various classifications of filler metals that are not suitable for structural steel applications. Filler metals listed under the various AWS A5 specifications may or

may not have specified notch toughness properties, depending on the specific electrode classification. Section J2.6 identifies certain welded joints where notch toughness of filler metal is needed in building construction. There may be other situations where the engineer of record may elect to specify the use of filler metals with specified notch toughness properties, such as for structures subject to high loading rate, cyclic loading or seismic loading. Since AWS D1.1 does not automatically require that the filler metal used have specified notch toughness properties, it is important that filler metals used for such applications be of an AWS classification where such properties are required. This information can be found in the AWS Filler Metal Specifications and is often contained on the filler metal manufacturer's certificate of conformance or product specification sheets.

When specifying filler metal and/or flux by AWS designation, the applicable standard specifications should be carefully reviewed to assure a complete understanding of the designation reference. This is necessary because the AWS designation systems are not consistent. For example, in the case of electrodes for shielded metal arc welding (AWS A5.1), the first two or three digits indicate the nominal tensile strength classification, in ksi, of the filler metal and the final two digits indicate the type of coating. For metric designations, the first two digits times 10 indicate the nominal tensile strength classification in MPa. In the case of mild steel electrodes for submerged arc welding (AWS A5.17), the first one or two digits times 10 indicate the nominal tensile strength classification for both U.S. customary and metric units, while the final digit or digits times 10 indicate the testing temperature in degrees F, for filler metal impact tests. In the case of low-alloy steel covered arc welding electrodes (AWS A5.5), certain portions of the designation indicate a requirement for stress relief, while others indicate no stress relief requirement.

Engineers do not, in general, specify the exact filler metal to be employed on a particular structure. Rather, the decision as to which welding process and which filler metal are to be utilized is usually left with the fabricator or erector. Codes restrict the usage of certain filler materials, or impose qualification testing to prove the suitability of the specific electrode, so as to make certain that the proper filler metals are used.

A4.   STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS

The abbreviated list of requirements in this Specification is intended to be compatible with and a summary of the more extensive requirements in Section 3 of the *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005). The user should refer to Section 3 of the *Code of Standard Practice for Steel Buildings and Bridges* for further information.

CHAPTER B

DESIGN REQUIREMENTS

B1. GENERAL PROVISIONS

Previous editions of the Specification contained a section entitled "Types of Construction," for example, Section A2 in the 1999 *LRFD Specification* (AISC, 2000b). In this Specification there is no such section and the requirements related to "types of construction" have been divided between Section B1, Section B3.6, and Section J1.

Historically, "Types of Construction" was the section that established what type of structures the Specification covers. The preface to the 1999 *LRFD Specification* (AISC, 2000b) suggests that the purpose of the Specification is "to provide design criteria for routine use and not to provide specific criteria for infrequently encountered problems." The preface to the 1978 *Specification* (AISC, 1978) contains similar language. While "routine use" may be difficult to describe, the contents of "Types of Construction" have been clearly directed at ordinary building frames with beams, columns and their connections.

The 1969 *Specification* (AISC, 1969) classified "types of construction" as Type 1, 2 or 3. The primary distinction among these three types of construction was the nature of the connections of the beams to the columns. Type 1 construction comprised "*rigid frames*," now called moment-resisting frames that had connections capable of transmitting moment. Type 2 construction comprised "simple frames" with no moment transfer between beams and columns. Type 3 construction comprised "semi-rigid frames." Type 3 construction used partially restrained connections and was allowed if a predictable and reliable amount of connection flexibility and moment transfer was demonstrable.

The 1986 *LRFD Specification* (AISC, 1986) changed the designation from Type 1, 2 or 3 to the designations FR (Fully Restrained) and PR (Partially Restrained). In these designations the term "restraint" refers to the degree of moment transfer and the associated deformation in the connections. The 1986 *LRFD Specification* also used the term "simple framing" to refer to structures with "simple connections," that is, connections with negligible moment transfer. In essence, FR was equivalent to Type 1, "simple framing" was equivalent to Type 2, and PR was equivalent to Type 3 construction.

Type 2 construction of earlier specifications and "simple framing" of the 1986 *LRFD Specification* had additional provisions that allowed the wind loads to be carried by moment resistance of selected joints of the frame provided that:

(1) The connections and connected members have capacity to resist the wind moments;

(2) The girders are adequate to carry the full gravity load as "simple beams"; and

(3) The connections have adequate inelastic rotation capacity to avoid overstress of the fasteners or welds under combined gravity and wind loading.

The justification of considering the so-called "wind connections" as both simple (for gravity loads) and moment resisting (for wind loads) was provided in Sourochnikoff (1950) and Disque (1964). The basic argument asserts that the connections actually have some moment resistance but that the strength is low enough that under wind loads the connections would sustain inelastic deformations. Under repeated wind loads, then, the connection response would "shake down" to a condition wherein the moments in the connections under gravity loads would be very small but the elastic resistance of the connections to wind moments would remain the same as the initial resistance. These additional provisions for Type 2 construction have been used successfully for many years. More recent recommendations for this type of system are provided in Geschwindner and Disque (2005).

Section B1 widens the purview of this Specification to a broader class of construction types. It recognizes that a structural system is a combination of members connected in such a way that the structure can respond in different ways to meet different design objectives under different loads. Even within the purview of ordinary buildings, there can be an enormous variety in the design details.

This Specification is still meant to be primarily applicable to the common types of building frames with gravity loads carried by beams and girders and lateral loads carried by moment frames, braced frames or shear walls. However, there are many unusual buildings for which this Specification is also applicable. Rather than to attempt to establish the purview of the Specification with an exhaustive classification of construction types, Section B1 requires that the design of members and their connections be consistent with the intended use of the structure and the assumptions made in the analysis of the structure.

B2.   LOADS AND LOAD COMBINATIONS

The loads and load combinations for this Specification are given in the applicable building code. In the absence of a specific local, regional or national building code, the load combinations and the nominal loads (for example, D, L, L_r, S, R, W and E) are the loads specified in Sections 3 through 9 of SEI/ASCE 7, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2002). The latest 2002 edition of SEI/ASCE 7 has adopted, in most aspects, the seismic design provisions from the NEHRP *Recommended Provisions* (NEHRP, 1997), as have the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2002). The reader is referred to the commentaries of these documents for an expanded discussion on loads, load factors and seismic design.

This Specification permits design for strength by either LRFD or ASD.

LRFD Load Combinations. If LRFD is selected, the load combination requirements are defined in Section 2.3 of SEI/ASCE 7, while if ASD is selected, the load combination requirements are defined in Section 2.4 of that standard. In either case, it is assumed that the nominal loads—D, L, L_r, S, R, W and E—are as specified in Sections 3 through 9 of SEI/ASCE 7, or their equivalent, as stipulated by the authority having jurisdiction. The engineer should understand that the bases for the load combinations in Sections 2.3 and 2.4 of SEI/ASCE 7 are different.

The load combinations in Section 2.3 of SEI/ASCE 7 are based on modern probabilistic load modeling and a comprehensive survey of reliabilities inherent in traditional design practice (Galambos, Ellingwood, MacGregor, and Cornell, 1982; Ellingwood, MacGregor, Galambos, and Cornell, 1982). These load combinations utilize a "principal action-companion action format," which is based on the notion that the maximum combined load effect occurs when one of the time-varying loads takes on its maximum lifetime value (principal action) while the other *variable loads* are at "arbitrary point-in-time" values (companion actions), the latter being loads that would be measured in a load survey at any arbitrary time. The dead load, which is considered to be permanent, is the same for all combinations in which the load effects are additive. Research has shown that this approach to load combination analysis is consistent with the manner in which loads actually combine on structural elements and systems in situations in which strength limit states may be approached. The load factors reflect uncertainty in individual load magnitudes and in the analysis that transforms load to load effect. The nominal loads in Sections 3 through 9 of SEI/ASCE 7 are substantially in excess of the arbitrary point-in-time values. The nominal live, wind and snow loads historically have been associated with mean return periods of approximately 50 years, while the nominal earthquake effect in NEHRP (1997) is associated with a mean return period of approximately 2,500 years. To avoid having to specify both a maximum and an arbitrary point-in-time value for each load type, some of the specified load factors are less than unity in SEI/ASCE 7 combinations (2) through (5).

Load combinations (6) and (7) of SEI/ASCE 7, Section 2.3, apply specifically to cases in which the structural actions due to lateral forces and gravity loads counteract one another. In that case, where the dead load stabilizes the structure, the load factor on dead load is 0.9.

ASD Load Combinations. The load combinations in Section 2.4 of SEI/ASCE 7 for ASD are similar to those that have been used in allowable stress design for the past four decades. In ASD, safety is provided by the safety factor, Ω, and the nominal loads in the basic combinations (1) through (3) are not factored. The reduction in the combined time-varying load effect in combinations (4) and (6) is achieved by the load combination factor 0.75. This load combination factor dates back to the 1972 edition of ANSI Standard A58.1, the predecessor of SEI/ASCE 7. It should be noted that in SEI/ASCE 7, the 0.75 factor applies *only* to combinations of

variable loads; it is irrational to reduce the dead load because it is *always* present and does not fluctuate in time. The load factor $0.6D$ in load combinations (7) and (8) in Section 2.4 of SEI/ASCE 7 addresses the situation in which the effects of lateral or uplift forces counteract the effect of gravity loads. This eliminates a deficiency in the traditional treatment of counteracting loads in allowable stress design and emphasizes the importance of checking stability. The earthquake load effect is multiplied by 0.7 in combinations (5) and (8) to align allowable strength design for earthquake effects with the definition of E in Section 9 of SEI/ASCE 7 which is based on strength principles.

The load combinations in Sections 2.3 and 2.4 of SEI/ASCE 7 apply only to design for strength limit states. Neither of these account for gross error or negligence.

Serviceability Load Combinations. Serviceability limit states and associated load factors are covered in Appendix B of SEI/ASCE 7. That Appendix contains a number of suggested load combinations for checking serviceability. While the nominal loads appearing in those equations are defined in Sections 3 through 7 of SEI/ASCE 7, the performance objectives for serviceability checking are different from those for checking strength, and thus the combinations and load factors are different.

B3.   DESIGN BASIS

Load and Resistance Factor Design (LRFD) and Allowable Strength Design (ASD) are distinct methods. They are equally acceptable by this Specification, but their provisions are not identical and not interchangeable. Indiscriminate use of combinations of the two methods could result in design error. For these reasons they are specified as alternatives. There are, however, circumstances in which the two methods could be used in the design, modification or renovation of a structural system without conflicting, such as providing modifications to a structural floor system of an older building after assessing the as-built conditions.

1.   Required Strength

This Specification permits the use of elastic, inelastic or plastic structural analysis. Generally, design is performed by elastic analysis. Provisions for inelastic and plastic analysis are given in Appendix 1. The required strength is determined by the appropriate methods of structural analysis.

In some circumstances, as in the proportioning of stability bracing members that carry no calculated forces (see, for example, Appendix 6), the required strength is explicitly stated in this Specification.

2.   Limit States

A limit state is a condition in which a structural system or component becomes unfit for its intended purpose, when it is exceeded. Limit states may be dictated by functional requirements, such as maximum deflections or drift; they may be related to structural behavior, such as the formation of a plastic hinge or mechanism;

or they may represent the collapse of the whole or part of the structure, such as by instability or fracture. The design provisions provided make certain that the probability of reaching a limit state is acceptably small by stipulating the combination of load factors, resistance or safety factors, nominal loads and nominal strengths consistent with the design assumptions.

Two kinds of limit states apply to structures: (1) strength limit states define safety against local or overall failure conditions during the intended life of the structure; and (2) serviceability limit states define functional requirements. This Specification, like other structural design codes, primarily focuses on strength limit states because of overriding considerations of public safety. This does not mean that limit states of serviceability are not important to the designer, who must provide for functional performance and economy of design. However, serviceability considerations permit more exercise of judgment on the part of the designer.

Strength limit states vary from element to element, and several limit states may apply to a given element. The following strength limit states are the most common: yielding, formation of a plastic hinge, member or overall frame instability, lateral-torsional buckling, local buckling, rupture and fatigue. The most common serviceability limit states include unacceptable elastic deflections and drift, unacceptable vibrations, and permanent deformations.

3.   Design for Strength Using Load and Resistance Factor Design (LRFD)

Design for strength by LRFD is performed in accordance with Equation B3-1. The left side of Equation B3-1, R_u, represents the required strength computed by structural analysis based on loads stipulated in SEI/ASCE 7 (ASCE, 2002), Section 2.3 (or their equivalent), while the right side, ϕR_n, represents the limiting structural resistance, or *design strength*, provided by the member.

The resistance factor ϕ in this Specification is equal to or less than 1.0. When compared to the nominal strength, R_n, computed according to the methods given in Chapters D through K, a ϕ-value of less than 1.0 accounts for inaccuracies of the theory and variations in mechanical properties and dimensions of members and frames. For limit states where $\phi = 1.0$, the nominal strength is judged to be sufficiently conservative when compared to the actual strength that no further reduction is needed.

The LRFD provisions are based on: (1) probabilistic models of loads and resistance; (2) a calibration of the LRFD provisions to the 1978 edition of the ASD Specification for selected members; and (3) the evaluation of the resulting provisions by judgment and past experience aided by comparative design office studies of representative structures.

In the probabilistic basis for LRFD (Ravindra and Galambos, 1978; Ellingwood and others, 1982), the load effects Q and the resistances R are modeled as statistically independent random variables. In Figure C-B3.1, relative frequency

distributions for Q and R are portrayed as separate curves on a common plot for a hypothetical case. As long as the resistance R is greater than (to the right of) the effects of the loads Q, a margin of safety for the particular limit state exists. However, because Q and R are random variables, there is a small probability that R may be less than Q, in other words, $R < Q$. The probability of this limit state is related to the degree of overlap of the frequency distributions in Figure C-B3.1, which depends on their relative positioning (R_m *versus* Q_m) and their dispersions.

The probability that R is less than Q depends on the distribution shapes of each of the many variables (material, loads, etc.) that determine resistance and total load effect. Often, only the means and the standard deviations or coefficients of variation of the many variables involved in the makeup of R and Q can be estimated. However, this information is sufficient to build an approximate design provision that is independent of the knowledge of these distributions, by stipulating the following design condition:

$$\beta\sqrt{V_R^2 + V_Q^2} \le \ln\left(R_m/Q_m\right) \qquad \text{(C-B3-1)}$$

In this equation, R_m and Q_m are the mean values and V_R and V_Q are the coefficients of variation, respectively, of the resistance R and the load effect Q. For structural elements and the usual loading, R_m, Q_m, and the coefficients of variation, V_R and V_Q, can be estimated, so a calculation of

$$\beta = \frac{\ln\left(R_m/Q_m\right)}{\sqrt{V_R^2 + V_Q^2}} \qquad \text{(C-B3-2)}$$

will give a comparative value of the measure of reliability of a structure or component. The parameter, β, is denoted the "safety" or "reliability" index.

Extensions to the determination of β in Equation C-B3-2 to accommodate additional probabilistic information and more complex design situations are described in Ellingwood and others (1982) and have been used in the development of the recommended load combinations in SEI/ASCE 7.
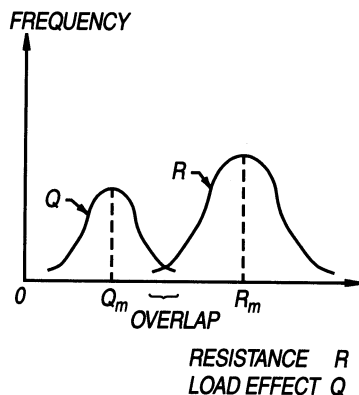

Fig. C-B3.1. Frequency distribution of load effect Q and resistance R.

The original studies for the statistical properties (mean values and coefficients of variation) used to develop the LRFD provisions for the basic material properties and for steel beams, columns, composite beams, plate girders, beam-columns and connection elements are presented in a series of eight articles in the September 1978 issue of the *Journal of the Structural Division*, ASCE (Vol. 104, ST9). The corresponding load statistics are given in Galambos and others (1982). Based on these statistics, the values of β inherent in the 1978 *Specification* (AISC, 1978) were evaluated under different load combinations (live/dead, wind/dead, etc.) and for various tributary areas for typical members (beams, columns, beam-columns, structural components, etc.). As might be expected, there was a considerable variation in the range of β-values. For example, compact rolled beams (flexure) and tension members (yielding) had β-values that decreased from about 3.1 at $L/D = 0.50$ to 2.4 at $L/D = 4$. This decrease is a result of ASD applying the same factor to dead load, which is relatively predictable, and live load, which is more variable. For bolted or welded connections, β was on the order of 4 to 5.

The variation of β that was inherent to ASD is reduced substantially in LRFD by specifying several target β-values and selecting load and resistance factors to meet these targets. The Committee on Specifications set the point at which LRFD is calibrated to ASD at $L/D = 3.0$ for braced compact beams in flexure and tension members at yield. The resistance factor, φ, for these limit states is 0.90, and the implied β is approximately 2.6 for members and 4.0 for connections. The larger β-value for connections reflects the fact that connections are expected to be stronger than the members they connect. Limit states for other members are handled similarly.

The databases on steel strength used in previous editions of the *LRFD Specification* were based mainly on research conducted prior to 1970. An important recent study of the material properties of structural shapes (Bartlett, Dexter, Graeser, Jelinek, Schmidt, and Galambos, 2003) addressed changes in steel production methods and steel materials that have occurred over the past 15 years. It was concluded that the new steel material characteristics did not warrant changes in the φ-values.

4.   Design for Strength Using Allowable Strength Design (ASD)

The ASD method is provided in this Specification as an equal alternative to LRFD for use by engineers who prefer to deal with ASD load combinations and allowable stresses in the traditional ASD format. The term "allowable strength" has been introduced to emphasize that the basic equations of structural mechanics that underlie the provisions are the same for LRFD and ASD. This represents a departure from the past when LRFD and ASD were governed by separate specifications.

Traditional ASD is based on the concept that the maximum stress in a component shall not exceed a certain allowable stress under normal service conditions. The load effects are determined on the basis of an elastic analysis of the structure, while the allowable stress is the limiting stress (at yielding, instability, fracture, etc.) divided by a safety factor. The magnitude of the safety factor and the resulting

allowable stress depend on the particular governing limit state against which the design must produce a certain margin of safety. For any single element, there may be a number of different allowable stresses that must be checked.

The safety factor in traditional ASD provisions was a function of both the material and the component being considered. It may have been influenced by factors such as member length, member behavior, load source and anticipated quality of workmanship. The traditional safety factors were based solely on experience and have remained unchanged for over 50 years. Although ASD-designed structures have performed adequately over the years, the actual level of safety provided was never known. This was the prime drawback of the traditional ASD approach. An illustration of typical performance data is provided in Bjorhovde (1978), where theoretical and actual safety factors for columns are examined.

Design for strength by ASD is performed in accordance with Equation B3-2. The ASD method provided in the Specification recognizes that the controlling modes of failure are the same for structures designed by ASD and LRFD. Thus, the nominal strength that forms the foundation of LRFD is the same nominal strength that provides the foundation for ASD. When considering available strength, the only difference between the two methods is the resistance factor in LRFD, ϕ, and the safety factor in ASD, Ω.

In developing appropriate values of Ω for use in this Specification, the aim was to assure similar levels of safety and reliability for the two methods. A straight forward approach for relating the resistance factor and the safety factor was developed. As already mentioned, the original LRFD Specification was calibrated to the 1978 *ASD Specification* at a live load to dead load ratio of 3. Thus, by equating the designs for the two methods at a ratio of live-to-dead load of 3, the relationship between ϕ and Ω can be determined. Using the live plus dead load combinations, with $L = 3D$, yields

$$\text{For LRFD}: \quad \phi R_n = 1.2D + 1.6L = 1.2D + 1.6 \times 3D = 6D \qquad \text{(C-B3-3)}$$

$$R_n = \frac{6D}{\phi}$$

$$\text{For ASD}: \qquad \frac{R_n}{\Omega} = D + L = D + 3D = 4D \qquad \text{(C-B3-4)}$$

$$R_n = \frac{4D}{\Omega}$$

Equating R_n from the LRFD and ASD formulations and solving for Ω yields

$$\Omega = \frac{6D}{\phi} \times \frac{1}{4D} = \frac{1.5}{\phi} \qquad \text{(C-B3-5)}$$

A similar approach was used to obtain the majority of values of Ω throughout the Specification.

5.   Design for Stability

Section B3.5 provides the charging language for Chapter C on design for stability.

6.   Design of Connections

Section B3.6 provides the charging language for Chapter J on the design of connections. Chapter J covers the proportioning of the individual elements of a connection (angles, welds, bolts, etc.) once the load effects on the connection are known. Section B3.6 establishes that the modeling assumptions associated with the structural analysis must be consistent with the conditions used in Chapter J to proportion the connecting elements.

In many situations, it is not necessary to include the connection elements as part of the analysis of the structural system. For example, simple and FR connections may often be idealized as pinned or fixed, respectively, for the purposes of structural analysis. Once the analysis has been completed, the deformations or forces computed at the joints may be used to proportion the connection elements. The classification of FR (fully restrained) and simple connections is meant to justify these idealizations for analysis with the provision that if, for example, one assumes a connection to be FR for the purposes of analysis, then the actual connection must meet the FR conditions. In other words, it must have adequate strength and stiffness, as described in the provisions and discussed below.

In certain cases, the deformation of the connection elements affects the way the structure resists load and hence the connections must be included in the analysis of the structural system. These connections are referred to as partially restrained (PR) moment connections. For structures with PR connections, the connection flexibility must be estimated and included in the structural analysis, as described in the following sections. Once the analysis is complete, the load effects and deformations computed for the connection can be used to check the adequacy of the connecting elements.

For simple and FR connections, the connection proportions are established after the final analysis of the structural design is completed, thereby greatly simplifying the design cycle. In contrast, the design of PR connections (like member selection) is inherently iterative because one must assume values of the connection proportions in order to establish the force-deformation characteristics of the connection needed to perform the structural analysis. The life-cycle performance characteristics (shakedown) must also be considered. The adequacy of the assumed proportions of the connection elements can be verified once the outcome of the structural analysis is known. If the connection elements are inadequate, then the values must be revised and the structural analysis repeated. The potential benefits of using PR connections for various types of framing systems are discussed extensively in the literature [for example, Lorenz, Kato, and Chen (1993); Leon (1994)].

Connection Classification. The basic assumption made in classifying connections is that the most important behavioral characteristics of the connection can be modeled by a moment-rotation (M-θ) curve. Figure C-B3.2 shows a typical M-θ

curve. Implicit in the moment-rotation curve is the definition of the connection as being a region of the column and beam along with the connecting elements. The connection response is defined this way because the rotation of the member in a physical test is generally measured over a gage length that incorporates the contributions of not only the connecting elements, but also the ends of the members being connected and the column panel zone.

Examples of connection classification schemes include those in Bjorhovde, Colson, and Brozzetti (1990) and Eurocode 3 (1992). These classifications account directly for the stiffness, strength and ductility of the connections.

Connection Stiffness. Because the nonlinear behavior of the connection manifests itself even at low moment-rotation levels, the initial stiffness of the connection K_i (shown in Figure C-B3.2) does not adequately characterize connection response at service levels. Furthermore, many connection types do not exhibit a reliable initial stiffness, or it exists only for a very small moment-rotation range. The secant stiffness K_S at service loads is taken as an index property of connection stiffness. Specifically, $K_S = M_S/\theta_S$ where M_S and θ_S are the moment and rotation, respectively, at service loads. In the discussion below, L and EI are the length and bending rigidity, respectively, of the beam.

If $K_S L/EI \geq 20$, then it is acceptable to consider the connection to be fully restrained (in other words, able to maintain the angles between members). If $K_S L/EI \leq 2$, then it is acceptable to consider the connection to be simple (in other words, rotates without developing moment). Connections with stiffnesses between these two limits are partially restrained and the stiffness, strength and ductility of the connection must be considered in the design (Leon, 1994). Examples of FR, PR and simple connection response curves are shown in Figure C-B3.3. The solid dot θ_S reflects the service load level and thereby defines the secant stiffness.
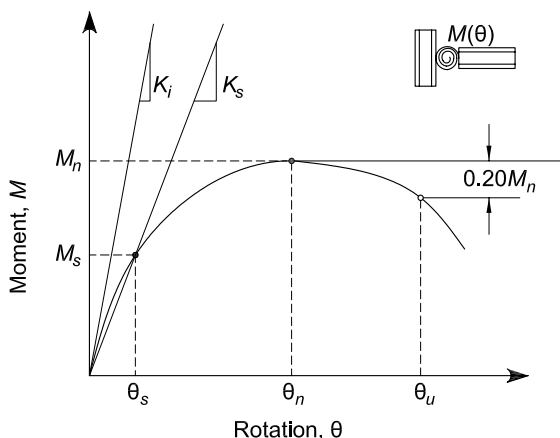

Fig. C-B3.2. Definition of stiffness, strength and ductility characteristics of the moment-rotation response of a partially restrained connection.

Connection Strength. The strength of a connection is the maximum moment that it is capable of carrying M_n, as shown in Figure C-B3.2. The strength of a connection can be determined on the basis of an ultimate limit-state model of the connection, or from a physical test. If the moment-rotation response does not exhibit a peak load then the strength can be taken as the moment at a rotation of 0.02 radian (Hsieh and Deierlein, 1991; Leon, Hoffman, and Staeger, 1996).

It is also useful to define a lower limit on strength below which the connection may be treated as a simple connection. Connections that transmit less than 20 percent of the fully plastic moment of the beam at a rotation of 0.02 radian may be considered to have no flexural strength for design. However, it should be recognized that the aggregate strength of many weak connections can be important when compared to that of a few strong connections (FEMA, 1997).

In Figure C-B3.3, the grey dot M_n indicates the maximum strength and the associated rotation θ_n. The open dot θ_u is the maximum rotation capacity. Note that it is possible for an FR connection to have a strength less than the strength of the beam. It is also possible for a PR connection to have a strength greater than the strength of the beam.

The strength of the connection must be adequate to resist the moment demands implied by the design loads.

Connection Ductility. If the connection strength substantially exceeds the fully plastic moment of the beam, then the ductility of the structural system is controlled by the beam and the connection can be considered elastic. If the connection strength only marginally exceeds the fully plastic moment of the beam, then the connection may experience substantial inelastic deformation before the beam reaches its full strength. If the beam strength exceeds the connection strength,
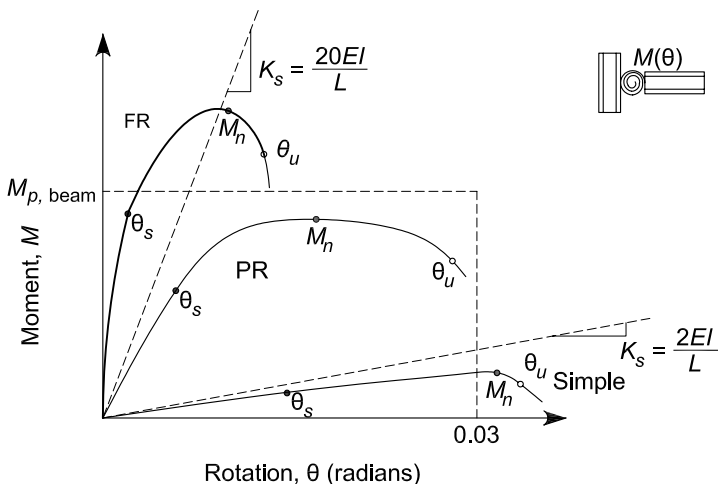

Fig. C-B3.3. Classification of moment-rotation response of fully restrained (FR), partially restrained (PR) and simple connections.

then deformations can concentrate in the connection. The ductility required of a connection will depend upon the particular application. For example, the ductility requirement for a braced frame in a nonseismic area will generally be less than the ductility required in a high seismic area. The rotation ductility requirements for seismic design depend upon the structural system (AISC, 2002).

In Figure C-B3.2, the rotation capacity, q_u, can be defined as the value of the connection rotation at the point where either (a) the resisting strength of the connection has dropped to $0.8M_n$ or (b) the connection has deformed beyond 0.03 radian. This second criterion is intended to apply to connections where there is no loss in strength until very large rotations occur. It is not prudent to rely on these large rotations in design.

The available rotation capacity, θ_u, should be compared with the rotation required at the strength limit state, as determined by an analysis that takes into account the nonlinear behavior of the connection. (Note that for design by ASD, the rotation required at the strength limit state should be assessed using analyses conducted at 1.6 times the ASD load combinations.) In the absence of an accurate analysis, a rotation capacity of 0.03 radian is considered adequate. This rotation is equal to the minimum beam-to-column connection capacity as specified in the seismic provisions for special moment frames (AISC, 2002). Many types of PR connections, such as top and seat-angle details, meet this criterion.

Structural Analysis and Design. When a connection is classified as PR the relevant response characteristics of the connection must be included in the analysis of the structure to determine the member and connection forces, displacements and the frame stability. Therefore, PR construction requires, first, that the moment-rotation characteristics of the connection be known and, second, that these characteristics be incorporated in the analysis and member design.

Typical moment-rotation curves for many PR connections are available from one of several databases [for example, Goverdhan (1983); Ang and Morris (1984); Nethercot (1985); and Kishi and Chen (1986)]. Care should be exercised when utilizing tabulated moment-rotation curves not to extrapolate to sizes or conditions beyond those used to develop the database since other failure modes may control (ASCE Task Committee on Effective Length, 1997). When the connections to be modeled do not fall within the range of the databases, it may be possible to determine the response characteristics from tests, simple component modeling, or finite element studies (FEMA, 1995). Examples of procedures to model connection behavior are given in the literature (Bjorhovde, Brozzetti, and Colson, 1988; Chen and Lui, 1991; Bjorhovde, Colson, Haaijer, and Stark, 1992; Lorenz and others, 1993; Chen and Toma, 1994; Chen, Goto, and Liew, 1995; Bjorhovde, Colson, and Zandonini, 1996; Leon, Hoffman, and Staeger, 1996; Leon and Easterling, 2002).

The degree of sophistication of the analysis depends on the problem at hand. Usually, design for PR construction requires separate analyses for the serviceability

and strength limit states. For serviceability, an analysis using linear springs with a stiffness given by K_S (see Figure C-B3.2) is sufficient if the resistance demanded of the connection is well below the strength. When subjected to strength load combinations, a more careful procedure is needed so that the characteristics assumed in the analysis are consistent with those of the connection response. The response is especially nonlinear as the applied moment approaches the connection strength. In particular, the effect of the connection nonlinearity on second-order moments and other stability checks need to be considered (ASCE Task Committee on Effective Length, 1997).

7.   Design for Serviceability

Section B3.7 provides the charging language for Chapter L on design for serviceability.

8.   Design for Ponding

As used in this Specification, ponding refers to the retention of water due solely to the deflection of flat roof framing. The amount of this water is dependent on the flexibility of the framing. Lacking sufficient framing stiffness, the accumulated weight of the water can result in the collapse of the roof. The problem becomes catastrophic when more water causes more deflection, resulting in more room for more water until the roof collapses. Detailed provisions for determining ponding stability and strength are given in Appendix 2.

9.   Design for Fatigue

Section B3.9 provides the charging language for Appendix 3 on design for fatigue.

10.   Design for Fire Conditions

Section B3.10 provides the charging language for Appendix 4 on structural design for fire resistance. Qualification testing is an acceptable alternative to design by analysis for providing fire resistance. It is anticipated that the basis will be ASCE/SFPE Standard 28 (ASCE, 1999), ASTM Standard E119 (ASTM, 2000), and similar documents.

11.   Design for Corrosion Effects

Steel members may deteriorate in particular service environments. This deterioration may appear either in external corrosion, which would be visible upon inspection, or in undetected changes that would reduce its strength. The designer should recognize these problems by either factoring a specific amount of tolerance for damage into the design or providing adequate protection systems (for example, coatings, cathodic protection) and/or planned maintenance programs so that such problems do not occur.

Because the interior of an HSS is difficult to inspect, some concern has been expressed regarding internal corrosion. However, good design practice can eliminate the concern and the need for expensive protection. Corrosion occurs in the presence

of oxygen and water. In an enclosed building, it is improbable that there would be sufficient reintroduction of moisture to cause severe corrosion. Therefore, internal corrosion protection is a consideration only in HSS exposed to weather.

In a sealed HSS, internal corrosion cannot progress beyond the point where the oxygen or moisture necessary for chemical oxidation is consumed (AISI, 1970). The oxidation depth is insignificant when the corrosion process must stop, even when a corrosive atmosphere exists at the time of sealing. If fine openings exist at connections, moisture and air can enter the HSS through capillary action or by aspiration due to the partial vacuum that is created if the HSS is cooled rapidly (Blodgett, 1967). This can be prevented by providing pressure-equalizing holes in locations that make it impossible for water to flow into the HSS by gravity.

Situations where conservative practice would recommend an internal protective coating include: (1) open HSS where changes in the air volume by ventilation or direct flow of water is possible; and (2) open HSS subject to a temperature gradient that would cause condensation.

HSS that are filled or partially filled with concrete should not be sealed. In the event of fire, water in the concrete will vaporize and may create pressure sufficient to burst a sealed HSS. Care should be taken to keep water from remaining in the HSS during or after construction, since the expansion caused by freezing can create pressure that is sufficient to burst an HSS.

Galvanized HSS assemblies should not be completely sealed because rapid pressure changes during the galvanizing process tend to burst sealed assemblies.

12.   Design Wall Thickness for HSS

ASTM A500 tolerances allow for a wall thickness that is not greater than ± 10 percent of the nominal value. Because the plate and strip from which electric-resistance-welded (ERW) HSS are made are produced to a much smaller thickness tolerance, manufacturers in the United States consistently produce ERW HSS with a wall thickness that is near the lower-bound wall thickness limit. Consequently, AISC and the Steel Tube Institute of North America (STI) recommend that 0.93 times the nominal wall thickness be used for calculations involving engineering design properties of ERW HSS. This results in a weight (mass) variation that is similar to that found in other structural shapes. Submerged-arc-welded (SAW) HSS are produced with a wall thickness that is near the nominal thickness and require no such reduction. The design wall thickness and section properties based upon this thickness have been tabulated in AISC and STI publications since 1997.

B4.   CLASSIFICATION OF SECTIONS FOR LOCAL BUCKLING

For the purposes of this Specification, steel sections are divided into compact sections, noncompact sections and slender-element sections. Compact sections are capable of developing a fully plastic stress distribution and they possess a rotation capacity of approximately 3 before the onset of local buckling (Yura,

Galambos, and Ravindra, 1978). Noncompact sections can develop partial yielding in compression elements before local buckling occurs, but will not resist inelastic local buckling at the strain levels required for a fully plastic stress distribution. Slender-element sections have one or more compression elements that will buckle elastically before the yield stress is achieved.

Limiting Width-Thickness Ratios. The dividing line between compact and non-compact sections is the limiting width-thickness ratio λ_p. For a section to be compact, all of its compression elements must have width-thickness ratios equal to or smaller than the limiting λ_p.

A second limiting width-thickness ratio is λ_r, representing the dividing line between noncompact sections and slender-element sections. As long as the width-thickness ratio of a compression element does not exceed the limiting value λ_r, elastic local buckling will not govern its strength. However, for those cases where the width-thickness ratios exceed λ_r, elastic buckling strength must be considered. Design procedures for such slender-element compression sections are given in Section E7 for members under pure axial compression, and in Sections F3.2, F5.3, F6.2, F7.2, F8.2, F9.3 and F10.3 for beams with a cross section that contains slender plate elements.

The values of the limiting ratios λ_p and λ_r specified in Table B4.1 are similar to those in the 1989 *Specification* (AISC, 1989) and Table 2.3.3.3 of Galambos (1978), except that $\lambda_p = 0.38\sqrt{E/F_y}$, limited in Galambos (1978) to indeterminate beams when moments are determined by elastic analysis and to determinate beams, was adopted for all conditions on the basis of Yura and others (1978).

For greater inelastic rotation capacities than provided by the limiting values λ_p given in Table B4.1, for structures in areas of high seismicity, see Section 8 and Table I-8-1 of the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2005).

Flanges of Built-Up I-Shaped Sections. For built-up I-shaped sections under axial compression (Case 4 in Table B4.1), modifications have been made to the flange local buckling criterion to include web-flange interaction. The k_c in the λ_r *limit* and in Equations E7-7 through E7-9 is the same that is used for flexural members in Equations F3-2 and F5-9. Theory indicates that the web-flange interaction in axial compression is at least as severe as in flexure. Rolled shapes are excluded from this provision because there are no standard sections with proportions where the interaction would occur. In built-up sections where the interaction causes a reduction in the flange local buckling strength, it is likely that the web is also a thin stiffened element.

The k_c factor accounts for the interaction of flange and web local buckling demonstrated in experiments reported in Johnson (1985). The maximum limit of 0.76 corresponds to $F_{cr} = 0.69E/\lambda^2$ which was used as the local buckling strength in editions of both the ASD and LRFD Specifications. An $h/t_w = 27.5$ is

required to reach $k_c = 0.76$. Fully fixed restraint for an unstiffened compression element corresponds to $k_c = 1.3$ while zero restraint gives $k_c = 0.42$. Because of web-flange interactions it is possible to get $k_c < 0.42$ from the new k_c formula. If $h/t_w > 5.70\sqrt{E/F_y}$ use $h/t_w = 5.70\sqrt{E/F_y}$ in the k_c equation, which corresponds to the 0.35 limit.

Webs in Flexure. New formulas for λ_p are presented in Case 11 in Table B4.1 for I-shaped beams with unequal flanges. These provisions are based on research reported in White (2003).

Rectangular HSS in Compression. The limits for rectangular HSS walls in uniform compression (Case 12 in Table B4.1) have been used in AISC Specifications since 1969. They are based on Winter (1968), where adjacent stiffened compression elements in box sections of uniform thickness were observed to provide negligible torsional restraint for one another along their corner edges. The λ_p limit for plastic analysis is adopted from *Limit States Design of Steel Structures* (CSA, 1994). The web slenderness limits are the same as those used for webs in wide-flange shapes.

Lower values of λ_p are specified for high-seismic design in the *Seismic Provisions for Structural Steel Buildings* based upon tests (Lui and Goel, 1987) that have shown that rectangular HSS braces subjected to reversed axial load fracture catastrophically under relatively few cycles if a local buckle forms. This was confirmed in tests (Sherman, 1995) where rectangular HSS braces sustained over 500 cycles when a local buckle did not form, even though general column buckling had occurred, but failed in less than 40 cycles when a local buckle developed. The seismic λ_p is based upon tests (Lui and Goel, 1987) of HSS that had a small enough *b/t* ratio so that braces performed satisfactorily for members with reasonable column slenderness. Filling the rectangular HSS with lean concrete (concrete mixed with a low proportion of cement) has been shown to effectively stiffen the HSS walls and improve cyclic performance.

Rectangular HSS in Flexure. A significant change from previous editions of the Specification is the compactness limit for webs in rectangular HSS flexural members (Case 13 in Table B4.1). The previously used value of $\lambda_p = 3.76\sqrt{E/F_y}$ was reduced to $\lambda_p = 2.42\sqrt{E/F_y}$. This change was introduced because tests reported in Wilkinson and Hancock (1998 and 2002) showed that HSS beams with geometries at the previous limiting compactness had hardly any rotation capacity available and were thus unable to deliver a target rotation capacity of 3.

Round HSS in Compression. The λ_r limit for round HSS in compression (Case 15 in Table B4.1) was first used in the 1978 *ASD Specification*. It was recommended in Schilling (1965) based upon research reported in Winter (1968). The same limit was also used to define a compact shape in bending in the 1978 *ASD Specification*. However, the limits for λ_p and λ_r were changed in the 1986 *LRFD Specification* based upon experimental research on round HSS in bending (Sherman, 1985;

Galambos, 1998). Excluding the use of round HSS with $D/t > 0.45E/F_y$ was also recommended in Schilling (1965).

Following the SSRC recommendations (Galambos, 1998) and the approach used for other shapes with slender compression elements, a Q factor is used for round sections to account for interaction between local and column buckling in Section E7.2(c). The Q factor is the ratio between the local buckling stress and the yield stress. The local buckling stress for the round section is taken from the AISI provisions based on *inelastic action* (Winter, 1970) and is based on tests conducted on fabricated and manufactured cylinders. Subsequent tests on fabricated cylinders (Galambos, 1998) confirm that this equation is conservative.

Round HSS in Flexure. The high shape factor for round hollow sections (Case 15 in Table B4.1) makes it impractical to use the same slenderness limits to define the regions of behavior for different types of loading. In Table B4.1, the values of λ_p for a compact shape that can achieve the plastic moment, and λ_r for bending, are based on an analysis of test data from several projects involving the bending of round HSS in a region of constant moment (Sherman and Tanavde, 1984; Galambos, 1998). The same analysis produced the equation for the inelastic moment capacity in Section F7. However, a more restrictive value of λ_p is required to prevent inelastic local buckling from limiting the plastic hinge rotation capacity needed to develop a mechanism in a round HSS (Sherman, 1976).

The values of λ_r for axial compression and for bending are both based on test data. The former value has been used in building specifications since 1968 (Winter, 1970). Section F8 also limits the D/t ratio for any round section to $0.45E/F_y$. Beyond this, the local buckling strength decreases rapidly, making it impractical to use these sections in building construction.

B5.   FABRICATION, ERECTION AND QUALITY CONTROL

Section B5 provides the charging language for Chapter M on fabrication, erection and quality control.

B6.   EVALUATION OF EXISTING STRUCTURES

Section B6 provides the charging language for Appendix 5 on the evaluation of existing structures.

CHAPTER C

STABILITY ANALYSIS AND DESIGN

Chapter C addresses the stability analysis and design requirements for steel buildings and related structures. The chapter has been reorganized from the previous Specifications into two parts: Section C1 outlines general requirements for stability and specific stability requirements for individual members (for example, beams, columns, braces) and for systems, including moment frames, braced frame and shear walls, gravity frame systems, and combined systems. Section C2 addresses the calculation of required strengths including the definition of acceptable analysis methods and specific constraints to be placed on the analysis and design procedures. A discussion of the effective length factor, K, the column buckling stress, F_e, and associated buckling analysis methods is provided at the end of the commentary chapter.

C1. STABILITY DESIGN REQUIREMENTS

1. General Requirements

The stability of structures must be considered from the standpoint of the structure as a whole, including not only the compression members, but also the beams, bracing systems and connections. Stability of individual components must also be provided. Considerable attention has been given in the technical literature to this subject, and various methods are available to provide stability (Galambos, 1998). In all approaches, the method of analysis and the equations for component strengths are inextricably interlinked. Traditionally, the effects of unavoidable geometric imperfections (within fabrication and erection tolerances) and distributed yielding at strength limit states (including residual stress effects) are addressed solely within member strength equations. Correspondingly, structural analysis is conducted using the nominal or undeformed structure geometry and elastic stiffness. This Specification addresses this traditional approach, termed the Effective Length Method in this commentary, as well as a new approach which is termed the Direct Analysis Method, addressed in Appendix 7. The Direct Analysis Method includes nominal geometric imperfection and stiffness reduction effects directly within the structural analysis. In either the Effective Length or the Direct Analysis Method, structural analysis by itself is not sufficient to provide for the stability of the structure as a whole. The overall stability of the structure as well as the stability of individual elements is provided for by the combined calculation of the required strengths by structural analysis and the satisfaction of the member and connection design provisions of this Specification.

In general, it is essential that an accurate second-order analysis of the structure be performed. The analysis should consider the influence of second-order effects (including P-Δ and P-δ effects as shown in Figure C-C1.1) and of flexural, shear and axial deformations. More rigorous analysis methods allow formulations of simpler limit state models. One such example can be found in Appendix 7, where the new Direct Analysis Method is presented as an alternative method to improve and simplify design for stability. In this case, the inclusion of nominal geometric imperfection and member stiffness reduction effects directly in the analysis allows the use of $K = 1.0$ in calculating the in-plane column strength, P_n, within the beam-column interaction equations of Chapter H. This simplification comes about because the Direct Analysis Method provides a better estimate of the true load effects within the structure. The Effective Length Method, in contrast, includes the above effects indirectly within the member strength equations.

2.   Member Stability Design Requirements

Chapters E through I contain the necessary provisions for satisfying member stability (in other words, the available strengths) given the load effects obtained from structural analysis and given specific bracing conditions assumed in the calculation of the member strengths. Where beam and column members rely upon braces that are not part of the lateral load resisting system to define their unbraced length, the braces themselves must have sufficient strength and stiffness to control member movement at the brace points. Appendix 6 contains all the requirements for braces that were previously contained within Chapter C of the 1999 *LRFD Specification* (AISC, 2000b). Design requirements for braces that are part of the lateral load resisting system (that is, braces that are included within the analysis of the structure) are addressed within Chapter C.

P-Δ = Effect of loads acting on the displaced location of joints or nodes in a structure.

P-δ = Effect of loads acting on the deflected shape of a member between joints or nodes.
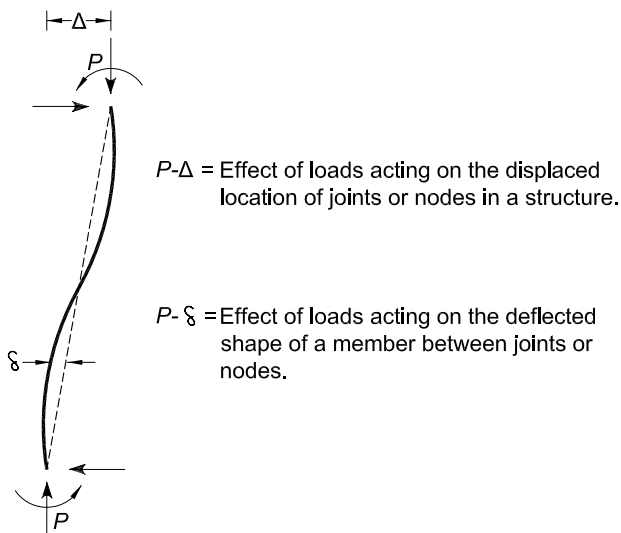
Fig. C-C1.1. P-Δ and P-δ effects in beam-columns.

3.   System Stability Design Requirements

Lateral stability can be provided by braced frames, shear-wall systems, moment frames or any other comparable lateral load resisting systems. Where combined systems are used, it is important that consideration be given to the transfer of forces and load sharing between systems, and to the destabilizing effect of vertical load carrying elements not participating as part of the lateral load resisting system (for example, leaning columns).

3a.   Braced-Frame and Shear-Wall Systems

Braced-frame systems are commonly analyzed and designed as vertically cantilevered pin-connected truss systems, ignoring any secondary moments within the system. The effective length factor, K, of components of the braced frame is normally taken to be 1.0, unless a smaller value is justified by structural analysis and the member and connection design is consistent with this assumption. Use of a K-factor less than 1.0 is discussed further at the end of this commentary chapter.

3b.   Moment-Frame Systems

Moment-frame systems rely primarily upon the flexural stiffness of the connected beams and columns although the reduction in the stiffness due to shear deformations can be important and should be considered where column bays are short and/or members are deep. Except as noted in Section C2.2a(4), Section C2.2b and Appendix 7, the design of all columns and beam-columns must be based on an effective length, KL, greater than the actual length determined as specified in Section C2. The Direct Analysis Method in Appendix 7, as well as the provisions of Sections C2.2a(4) and C2.2b, provide the means for proportioning columns with $K = 1.0$.

3c.   Gravity Framing Systems

Columns in gravity framing systems can be designed as pin-ended columns with $K = 1.0$. However, the destabilizing effect (P-Δ effect) of the gravity load on all such columns and the load transfer from these columns to the lateral load resisting system must be accounted for in the design of the lateral load resisting system. Methods for including this leaning column effect in the design of the lateral system are discussed in Commentary Section C2.

3d.   Combined Systems

When combined systems are used, structural analysis must proportion the lateral loads to the various systems with due regard to the relative stiffness of each system and the load transfer path between them. Consideration must be given to the variation in stiffness inherent in concrete or masonry shear walls due to various degrees of cracking possible. This applies both to serviceability load combinations and strength load combinations. It is prudent for the designer to consider a range of possible stiffnesses, with due regard to shrinkage, creep and load history, in order to envelope the likely behavior and provide sufficient strength in all interconnecting

elements between systems. Once the loads are determined on each system, the
design must conform to all requirements for the respective systems.

C2.   CALCULATION OF REQUIRED STRENGTHS

This Specification recognizes a variety of analysis and design procedures for
assessing the response of lateral load resisting systems. These include the use
of second-order inelastic and plastic methods with specially developed computer
software, effective length factors in conjunction with second-order elastic analysis,
the Direct Analysis Method, and simplified first-order elastic methods suitable for
manual calculation. Accordingly, Section C2 addresses several general analysis
approaches commonly used and defines certain constraints that must be placed on
the analysis and design with each method so as to provide a safe design.

1.   Methods of Second-Order Analysis

Some of the key differences between the 1999 *LRFD Specification* (AISC, 2000b)
and this Specification involve requirements for minimum stiffness and strength of
steel frames. The provisions in AISC (2000b) imposed the following two require-
ments on braced frames only:

(1) A minimum brace strength of

$$P_{br} = 0.004 \Sigma P_u$$

(2) A minimum brace stiffness of

$$\beta_{br} = 2 \Sigma P_u / (\phi L) \text{ where } \phi = 0.75$$

By substituting the minimum required brace stiffness, β_{br}, into the B_2 equation
below [Equation C1-4 in AISC (2000b) where $\beta_{br} = \Sigma H / \Delta_{oh}$], it can be observed
that the above minimum brace stiffness is equivalent to providing $B_2 \leq 1.6$. The
minimum brace force, $P_{br} = 0.004 \Sigma P_u$, is the force one would obtain in the brace
by doing a first-order elastic analysis at the strength load level, including an initial
out-of-plumbness of 0.002 times the story height, L, and assuming an amplification
from second-order effects of 2.0. The amplification of 2.0 is determined using
$\beta_{br} = 2 \Sigma P_u / (\phi L)$ in the B_2 equation below, but without including the ϕ factor on
stiffness.

$$B_2 = \frac{1}{1 - \dfrac{\Sigma P_u \Delta_{oh}}{\Sigma HL}} = \frac{1}{1 - \dfrac{\Sigma P_u}{\beta_{br} L}} \qquad \text{(C-C2-1)}$$

In contrast, this Specification imposes a minimum stiffness on all frames by ap-
plication of a B_2 limit of 1.5 unless the more accurate Direct Analysis Method
of Appendix 7 is used. The Direct Analysis Method addresses the influence of
nominal geometric imperfections (for example, out-of-plumbness) and stiffness
reductions due to distributed yielding directly within the analysis, in which case
the above stiffness and strength requirements are accounted for in a direct manner.
Setting the B_2 equation above to 1.5 is equivalent to imposing a minimum frame
stiffness of $\beta_{br} = 3 \Sigma P_u / L$ which is 12 percent larger than in AISC (2000b) for

braced systems. The 12 percent difference is a consequence of setting the B_2 limit at 1.5 for all frames designed without the use of the more accurate Direct Analysis Method. Additional discussion about upper limits on B_2 can be found in Appendix 7, Section 7.3.

In the development of this Specification, it was considered to require an additive notional load of $0.002\Sigma Y_i$ with all load combinations for all B_2 levels. However, $(\Sigma H + 0.002\Sigma Y_i)/\Sigma H$ is close to 1.0 for all of the lateral load combinations in SEI/ASCE 7 (ASCE, 2002), and for $B_2 \leq 1.5$, the additional internal forces caused by applying $0.002\Sigma Y_i$ in combination with the required lateral loadings are small and may be neglected. Therefore, $0.002\Sigma Y_i$ is required only as a minimum lateral load in the gravity load-only combinations within Section C2.2a. Conversely, for frames with $B_2 > 1.5$, the P-Δ effects associated with the amplified lateral deflections due to initial out-of-plumbness plus the additional amplified defections due to distributed yielding or other incidental causes can be significant at strength load levels. Therefore, for these stability-sensitive structures the Direct Analysis Method of Appendix 7 is required with the use of an additive notional lateral load of $N_i = 0.002\Sigma Y_i$.

1a.   General Second-Order Elastic Analysis

Section C2.1a states that any second-order elastic analysis method that captures both the P-Δ and P-δ effects, when one or both are significant to the accurate determination of internal forces, may be used. The amplification of first-order analysis forces by the traditional B_1 and B_2 factors as defined in Section C2.1b is one method of conducting an approximate second-order elastic analysis. In addition, the section states that all flexural, shear and axial deformations that significantly affect the stability of the structure and its elements in general must be considered. Also, in the Direct Analysis Method, nominal geometric imperfections and member stiffness reduction due to residual stresses must be directly included in the analysis.

The Direct Analysis Method is more sensitive to the accuracy of the second-order elastic analysis than the Effective Length Method. The Direct Analysis method may be used in the analysis and design of all lateral load resisting systems. The Commentary to Appendix 7, Sections 7.1 and 7.3, contains specific guidelines on the requirements for rigorous second-order elastic analysis, and provides benchmark problems that may be used to determine the adequacy of a particular analysis method. Software programs being used in the analysis should be tested with these benchmark problems to check their accuracy and to understand their limitations. Also, it is essential for the designer to apply the specific constraints applicable to the analysis-design method being used.

It is important to recognize that traditional elastic analysis methods, even those that properly consider second-order effects, are based on the undeformed geometry and nominal member properties and stiffnesses. Initial imperfections in the structure, such as out-of-plumbness, fabrication tolerances, incidental patterned

gravity loading, temperature gradients across the structure, foundation settlements, etc., as well as residual member stresses and general softening of the structure at the strength limit state, combine with the destabilizing effects of the vertical loads to increase the magnitude of load effects in the structure above those predicted by traditional analysis methods. This is particularly true for stability-sensitive structures containing large vertical loads with small lateral load requirements, leading to relatively low lateral load resistance. Limits on B_2 are placed on some of the analysis-design methods to limit the potential underestimation of load effects in stability-sensitive structures. Note that B_2 may be determined directly as the ratio of the second-order to the first-order lateral displacements at each story in the structure, $\Delta_{2nd\ order}/\Delta_{1st\ order}$, (the appropriate definition when a second-order analysis is performed), or as defined by Equation C2-3 (the appropriate definition when an amplified first-order analysis is performed). This underestimation of load effects is particularly important in the design of restraining girders of moment frames and braces in braced frames. Within the Effective Length Method, the in-plane column strength, P_n, accounts for the above effects by inclusion of the effective length factor and the use of the column strength curve of Section E3. However, the increases in the magnitude of the internal forces due to these effects are not accounted for within other member and connection design equations. The Direct Analysis Method in Appendix 7 overcomes these shortcomings in the traditional Effective Length Method. Therefore, it is recommended for use, particularly in stability-sensitive structures.

1b.   Second-Order Analysis by Amplified First-Order Elastic Analysis

Section C2.1b addresses the traditional amplified first-order analysis method that has long been part of this Specification. It has been expanded for use in systems where axial load is predominant, such as braced frames and truss systems, as well as moment frames. Where properly applied, this method constitutes an acceptable elastic second-order analysis method.

This first-order analysis method defines amplification factors B_1 and B_2 that are applied to the first-order forces so as to obtain an estimate of the second-order forces. In the general case, a member may have first-order load effects not associated with sidesway that are multiplied by B_1 and first-order load effects produced by sidesway that are multiplied by B_2. The factor B_1 is required to estimate the P-δ effects on the nonsway moments, M_{nt}, in axially loaded members, while the factor B_2 is required to estimate the P-Δ effect in frame components of braced, moment and/or combined framing systems. The P-Δ and P-δ effects are shown graphically in Figure C-C1.1 for a beam column. The effect of B_1 and B_2 amplification of moments is shown in Figure C-C2.1.

The factor B_2 applies only to internal forces associated with sidesway and is calculated for an entire story. In building frames designed to limit Δ_H/L to a predetermined value, the factor B_2 may be found in advance of designing individual members by using the target maximum limit on Δ_H/L within Equation C2-6b.

In determining B_2 and the second-order effects on the lateral load resisting system, it is important that Δ_H include not only the interstory displacement in the plane of the lateral load resisting system, but also any additional displacement in the floor or roof diaphragm or horizontal framing system that may increase the overturning effect of columns attached to and "leaning" against the horizontal system. Either the maximum displacement or a weighted average displacement, weighted in proportion to column load, should be considered.

Drift limits may also be set for design of various categories of buildings so that the effect of secondary bending is reduced (ATC, 1978; Kanchanalai and Lu, 1979). However, drift limits alone are not sufficient to allow stability effects to be neglected (LeMessurier, 1977).

Both types of first-order moments, M_{nt} and M_{lt}, may be induced by gravity loads. M_{nt} is defined as a moment developed in a member with frame sidesway prevented. M_{lt} is the moment developed within a member due to frame sidesway. If a significant restraining force is necessary to prevent sidesway of an unsymmetrical structure or an unsymmetrically loaded symmetrical structure, the moments induced by releasing the restraining force contribute to the M_{lt} moments. In most reasonably symmetric frames, this effect will be small. If the moment $B_2 M_{lt}$ is added algebraically to the $B_1 M_{nt}$ moment developed with sidesway prevented, as defined by Equation C2-1a, a reasonably accurate value of M_r results in most cases. A rigorous second-order elastic analysis is recommended for accurate determination of the frame internal forces when B_1 is larger than about 1.2. End moments produced in sidesway frames by lateral loads from wind or earthquake are always M_{lt} moments. Note that, in general, axial forces must also be amplified
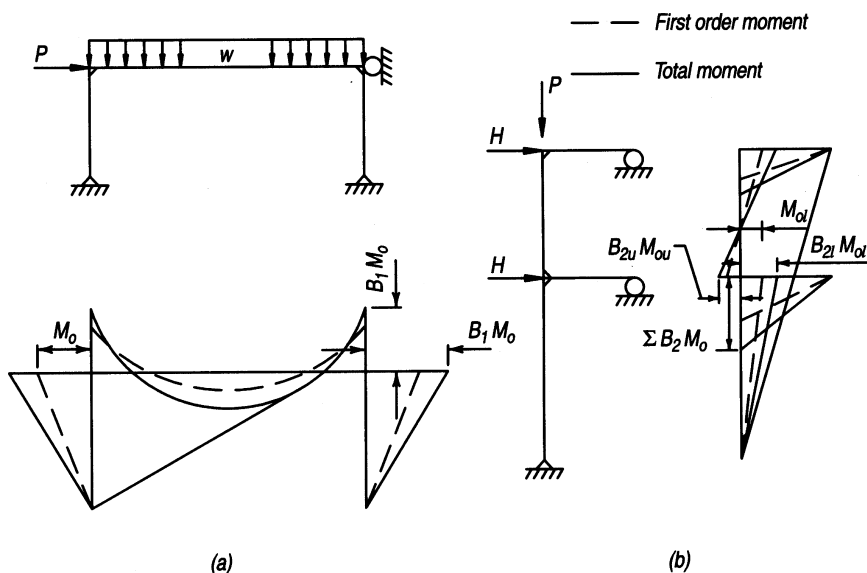
Fig. C-C2.1. Moment amplification.

according to Equation C2-1b for braced and moment frames, although the effect may be small in many low rise moment frames.

When first-order end moments in members subject to axial compression are magnified by B_1 and B_2 factors, equilibrium requires that they be balanced by moments in connected members (for example, see Figure C-C2.1). The associated second-order internal moments in the connected members can be calculated satisfactorily in most cases by amplifying the moments in all the members of the lateral load resisting system, in other words, the columns and the beams, by their corresponding B_1 and B_2 values. For beam members, the larger of the B_2 values from the story above or below is used. Connections shall also be designed to resist the magnified end moments. Alternatively, the difference between the magnified moment and the first-order moment in the column(s) at a given joint may be distributed to any other moment-resisting members attached to the compressed member (or members) in proportion to the relative stiffness of the uncompressed members. Minor imbalances may be neglected in the judgment of the engineer. This latter method is considerably more tedious than the above recommended method. Complex conditions, such as occur when there is significant magnification in several members meeting at a joint, may require an actual second-order elastic analysis rather than an amplified first-order analysis.

In braced and moment frames, P_n is governed by the maximum slenderness ratio regardless of the plane of bending, if the member is subject to significant biaxial bending, or if Section H1.3 is not utilized. Section H1.3 is an alternative approach for checking beam-column strength that provides for the separate checking of beam-column in-plane and out-of-plane stability in members predominantly subject to bending within the plane of the frame. However, P_{e1} and P_{e2} expressed by Equations C2-5 and C2-6a are always calculated using the slenderness ratio in the plane of bending. Thus, when flexure in a beam-column is about the strong axis only, two different values of slenderness ratio may be involved in the amplified first-order elastic and design calculations.

The value of $R_M = 0.85$ within Equation C2-6b is based on an approximate upper-bound influence of P-δ effects on the amplification of the sidesway displacements in practical moment frames (LeMessurier, 1977).

The second-order internal forces from separate structural analyses cannot normally be combined by superposition since second-order amplification depends, in a nonlinear fashion, on the total axial forces within the structure. Therefore, a separate second-order analysis must be conducted for each load combination considered in the design. The first-order internal forces, calculated prior to amplification within the amplified first-order elastic analysis procedure of Section C2.1b, may be superimposed to determine the total first-order internal forces.

When bending occurs about both the x- and the y-axes, the required flexural strength, calculated about each axis, is amplified by B_1 based on the value of C_m and P_{e1} in Equation C2-2 corresponding to the moment gradient in the

beam-column and its slenderness ratio in the plane of bending. A similar amplification by B_2 in the required flexural strength must occur for ΣP_{e2} in Equation C2-3 corresponding to the in-plane response.

Equations C2-2 and C2-4 are used to approximate the maximum second-order moments in compression members with no relative joint translation and no transverse loads between the ends of the member. Figure C-C2.2a compares the approximation for C_m in Equation C2-4 to the exact theoretical solution for beam-columns subjected to applied end moments (Chen and Lui, 1987). This figure plots the approximate and analytical values of C_m versus the end-moment ratio M_1/M_2 for several levels of $P/P_e(P_e = P_{e1}$ with $K = 1)$. Figure C-C2.2b shows the corresponding approximate and analytical solutions for the maximum second-order elastic moment within the member, M_r, versus the axial load level, P/P_e, for several values of the end moment ratio M_1/M_2.

For beam-columns with transverse loadings, the second-order moment can be approximated by

$$C_m = 1 + \psi \left(\frac{\alpha P_r}{P_{e1}} \right) \tag{C-C2-2}$$

for simply supported members

where

$$\psi = \frac{\pi^2 \delta_o EI}{M_o L^2} - 1$$

δ_o = maximum deflection due to transverse loading, in. (mm)

M_o = maximum first-order moment within the member due to the transverse
        loading, kip-in. (N-mm)

α = 1.0 (LRFD) or 1.6 (ASD)
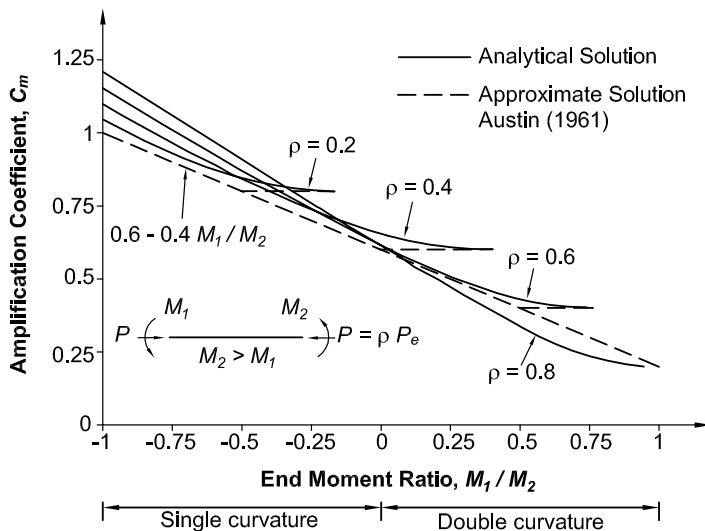
Fig. C-C2.2a. Equivalent moment factor C_m for beam-columns subjected to applied
end moments.

For restrained ends, some limiting cases are given in Table C-C2.1 together with two cases of simply supported beam-columns (Iwankiw, 1984). These values of C_m are always used with the maximum moment in the member. For the restrained-end cases, the values of B_1 are most accurate if values of $K < 1.0$, corresponding to the member end conditions, are used in calculating P_{e1}. In lieu of using the equations above, $C_m = 1.0$ is used conservatively for all transversely loaded members. It can be shown that the use of $C_m = 0.85$ for members with restrained ends, specified in previous Specifications, can sometimes result in a significant under-estimation of the internal moments. Therefore, the use of $C_m = 1.0$ is recommended as a simple conservative approximation for all cases involving transversely loaded members.

2. Design Requirements

Section C2.2 contains requirements for two of the three methods of elastic analysis and design of lateral load resisting frames allowed by this Specification: (a) design by elastic second-order analysis; and (b) design by elastic first-order analysis. Conformance to all the constraints of these methods as specified in this section satisfies the requirements of Section C1.1. Appendix 7 addresses the third method of analysis and design called the Direct Analysis Method. Both methods listed in this section specify that the structure should be analyzed using the nominal geometry and the nominal elastic stiffnesses (*EI, EA*) for all members, which is the traditional approach. In order to limit potential errors in the load effects in the structure from these simplified analyses, it is necessary to limit the sidesway amplification, as represented by $\Delta_{2nd\ order}/\Delta_{1st\ order}$ (or equivalently, the
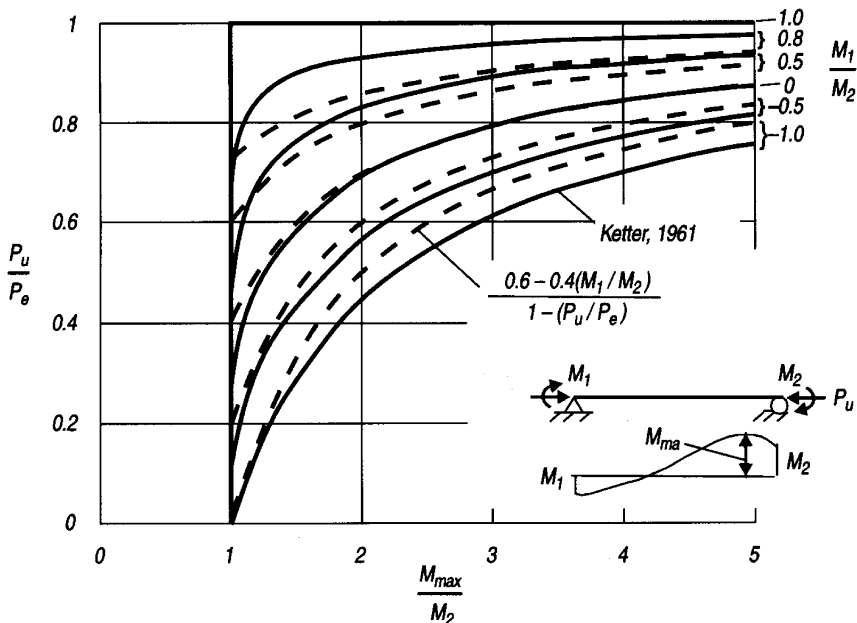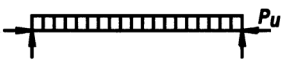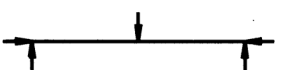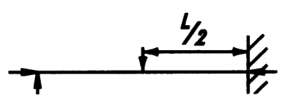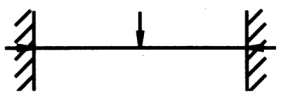

Fig. C-C2.2b. Second-order moments for beam-columns subjected to applied end moments.

TABLE C-C2.1 Amplification Factors ψ and C_m		
Case	Ψ	C_m
	0	1.0
	−0.4	$1 - 0.4\dfrac{P_u}{P_{e1}}$
	−0.4	$1 - 0.4\dfrac{P_u}{P_{e1}}$
	−0.2	$1 - 0.2\dfrac{P_u}{P_{e1}}$
	−0.3	$1 - 0.3\dfrac{P_u}{P_{e1}}$
	−0.2	$1 - 0.2\dfrac{P_u}{P_{e1}}$

B_2 amplifier), in each story of the frame for all load combinations. A limit of 1.5 on $\Delta_{2nd\,order}/\Delta_{1st\,order}$ is specified for each of the methods addressed in Section C2.2a and C2.2b. Otherwise, the Direct Analysis Method in Appendix 7 is required. The Direct Analysis Method is applicable for any building frame, regardless of the sidesway amplification or B_2 value, and its use is encouraged.

It is important to note that the sidesway amplification or B_2 limits specified in Chapter C and Appendix 7 are based on Equation C2-3 which specifies a first-order elastic analysis using the nominal geometry and properties of the structure.

2a.   Design by Second-Order Analysis

It is essential that the analysis of the frame be carried out at the strength limit state because of the nonlinearity associated with second-order effects. For design by the ASD method, this load level is estimated to be 1.6 times the ASD load combinations. This requirement is specified in clause (2).

Clause (3) in this section requires that, for all gravity load only combinations, a minimum lateral load of $0.002Y_i$ shall be applied at each level of the structure, where Y_i is the design gravity load acting on level i. Note that the load is to be applied independently in two orthogonal directions on the structure. Note also

that the column strengths, P_n, in moment frames must be based on the effective buckling length, KL, or the column buckling stress, F_e, where either KL or F_e is determined from a sidesway buckling analysis of the structure. A detailed discussion of the K-factor, the column buckling stress, F_e, and associated sidesway buckling analysis methods is provided at the end of this commentary chapter.

In the special case where the sidesway amplification $\Delta_{2nd\ order}/\Delta_{1st\ order}$ (or B_2) ≤ 1.1, the frame design may be based on the use of $K = 1.0$ for the columns, as specified in clause (4). By limiting the sidesway amplification (or B_2 level) to a maximum value of 1.1, the resulting unconservative error is limited to a maximum of approximately 6 percent within the in-plane beam-column strength checks of Chapter H (White and Hajjar, 1997).

For all cases, braced frames may be designed on the basis of $K = 1.0$.

2b. Design by First-Order Analysis

This section provides a method for designing frames using a first-order elastic analysis with $K = 1.0$, provided the sidesway amplification $\Delta_{2nd\ order}/\Delta_{1st\ order} \leq$ 1.5 (or $B_2 \leq 1.5$, where B_2 is determined as specified within the amplified first-order elastic analysis procedure of Section C2.1) and the required compressive strength of all members that are part of the lateral load resisting frame (other than truss members whose flexural stiffness is neglected in the analysis) have $\alpha P_r < 0.5 P_y$. All load combinations must include an additional lateral load, N_i, applied in combination with other loads at each level of the structure specified by Equation C2-8. Note that the load is to be applied independently in two orthogonal directions on the structure. If drift occurs under gravity load, then the minimum load should be applied in the direction of the drift. This equation is derived from the Direct Analysis Method as shown in the commentary to Appendix 7. It is based on an assumed $\Delta_{2nd\ order}/\Delta_{1st\ order}$ (or B_2) value of 1.5. Initial out-of-plumbness does not need to be considered in the calculation of Δ. Equation C2-8 is based on the clause within Appendix 7 that permits the notional load to be applied as a minimum lateral load in the *gravity load only* combinations and not in combination with other lateral loads when $\Delta_{2nd\ order}/\Delta_{1st\ order}$ (or B_2) ≤ 1.5. The minimum value of N_i of $0.0042Y_i$ is based on the assumption of a minimum first-order drift ratio due to any effects of $\Delta/L = 0.002$. Note that a target maximum drift ratio, corresponding to drifts under either the LRFD strength load combinations or 1.6 times the ASD strength load combinations, can be assumed at the start of design to determine the additional lateral load N_i. As long as that drift ratio is not exceeded at any strength load level, the design will be conservative.

The nonsway amplification of beam-column moments is addressed within the procedure specified in this section by applying the B_1 amplifier of Section C2.1 conservatively to the total member moments. In many cases involving beam-columns not subject to transverse loading between supports in the plane of bending, $B_1 = 1.0$.

Further explanation of this first-order design procedure is provided at the end of Appendix 7.

Determination of Effective Length Factor, *K*, or the Column Buckling Stress, *F*ₑ

There are two uses for the effective length factor, *K*, within the Specification:

(1) *Amplified first-order analysis. K* is used in the determination of the elastic buckling load, P_{e1}, for a member, or ΣP_{e2} for a building story, for calculation of the corresponding amplification factors B_1 and B_2 within the amplified first-order elastic analysis procedure of Section C2.1b; and

(2) *Column flexural buckling strength, P_n. K* is used in the determination of the column flexural buckling strength, P_n, from Chapter E, which may be based either on elastic or inelastic buckling analysis.

Each of these uses is discussed in detail below. The section begins, however, with a discussion of some background on the effective length factor, *K*, and some traditional approaches to determine *K*, most notably from the alignment charts.

Traditional Approaches to Calculating K—The Alignment Charts. A wide range of methods have been suggested in the engineering literature for the calculation of column effective length factors, *K* (Kavanagh, 1962; Johnston, 1976; LeMessurier, 1977; ASCE Task Committee on Effective Length, 1997; White and Hajjar, 1997a). These range from simple idealizations of single columns such as shown in Table C-C2.2 to complex buckling solutions for specific frames and loading conditions. In some types of frames, *K*-factors are easily estimated or calculated and they serve as a convenient tool for stability design. In other types of structures, the determination of accurate *K*-factors is tedious by hand procedures, and system stability may be assessed more effectively without the consideration of member *K* values at all. This latter approach is addressed in more detail later in this section.

The most common method for determining *K* is with the use of the alignment charts, also commonly referred to as the nomographs, shown in Figure C-C2.3 for frames with sidesway inhibited and Figure C-C2.4 for frames with sidesway uninhibited. (Kavanagh, 1962) The appropriate subassemblages upon which the charts are based are shown in the figure, along with the alignment chart. The alignment charts are based on assumptions of idealized conditions which seldom exist in real structures. These assumptions are as follows:

1. Behavior is purely elastic
2. All members have constant cross section.
3. All joints are rigid.
4. For columns in frames with sidesway inhibited, rotations at opposite ends of the restraining beams are equal in magnitude and opposite in direction, producing single curvature bending.
5. For columns in frames with sidesway uninhibited, rotations at opposite ends of the restraining beams are equal in magnitude and direction, producing reverse curvature bending.

TABLE C-C2.2
Approximate Values of Effective Length Factor, K

Buckled shape of column is shown by dashed line.	(a)	(b)	(c)	(d)	(e)	(f)
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code	Rotation fixed and translation fixed / Rotation free and translation fixed / Rotation fixed and translation free / Rotation free and translation free					

6. The stiffness parameter $L\sqrt{P/EI}$ of all columns is equal.
7. Joint restraint is distributed to the column above and below the joint in proportion to EI/L for the two columns.
8. All columns buckle simultaneously.
9. No significant axial compression force exists in the girders.

The alignment chart for sidesway inhibited frames shown in Figure C-C2.3 is based on the following equation:

$$\frac{G_A G_B}{4}(\pi/K)^2 + \left(\frac{G_A + G_B}{2}\right)\left(1 - \frac{\pi/K}{\tan(\pi/K)}\right) + \frac{2\tan(\pi/2K)}{(\pi/K)} - 1 = 0$$

The alignment chart for sidesway uninhibited frames shown in Figure C-C2.4 is based on the following equation:

$$\frac{G_A G_B (\pi/K)^2 - 36}{6(G_A + G_B)} - \frac{(\pi/K)}{\tan(\pi/K)} = 0$$

where

$$G = \frac{\Sigma(E_c I_c/L_c)}{\Sigma(E_g I_g/L_g)} = \frac{\Sigma(EI/L)_c}{\Sigma(EI/L)_g}$$

The subscripts A and B refer to the joints at the ends of the column being considered. The symbol Σ indicates a summation of all members rigidly connected to

that joint and lying in the plane in which buckling of the column is being considered. E_c is the modulus of the column, I_c is the moment of inertia of the column, and L_c is the unsupported length of the column. E_g is the modulus of the girder, I_g is the moment of inertia of the girder, and L_g is the unsupported length of the girder or other restraining member. I_c and I_g are taken about axes perpendicular to the plane of buckling being considered. The alignment chart is valid for different materials if an appropriate effective rigidity, EI, is used in the calculation of G.

For column ends supported by, but not rigidly connected to, a footing or foundation, G is theoretically infinity but unless designed as a true friction-free pin, may be taken as 10 for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1.0. Smaller values may be used if justified by analysis.

Theoretical K values obtained from the alignment charts for various idealized end conditions, rotation fixed or free and translation fixed or free, are shown in Table C-C2.2 along with practical recommendations for use in actual design.

It is important to remember that the alignment charts are based on the assumptions of idealized conditions previously discussed and that these conditions seldom exist
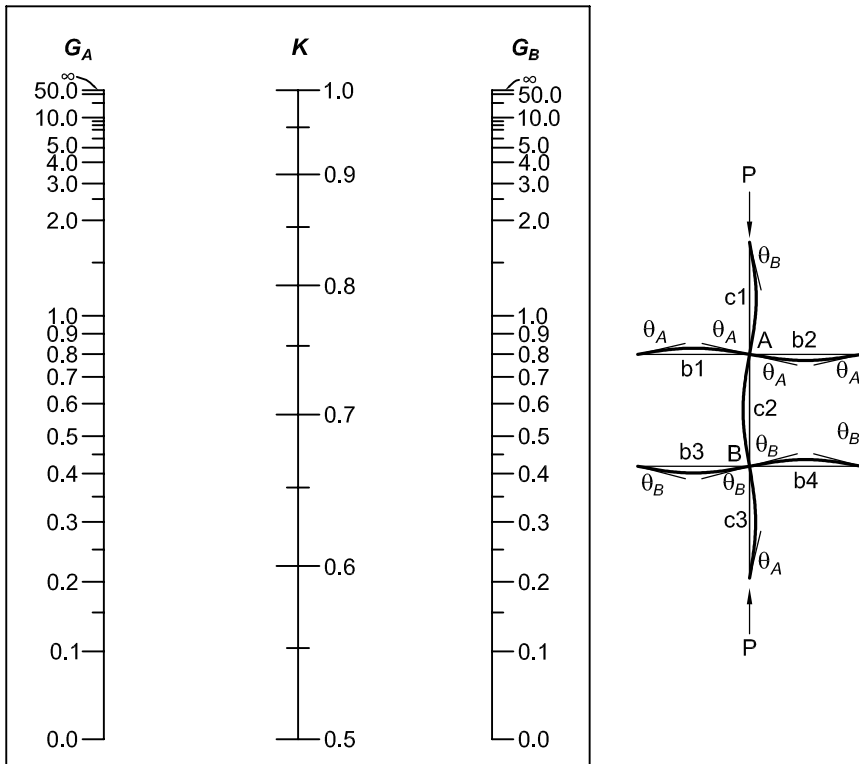

Fig. C-C2.3. Alignment chart—sideway inhibited (braced frame).

in real structures. Therefore, adjustments are required when these assumptions are violated and the alignment charts are still to be used. Adjustments for common design conditions that apply to both sidesway conditions are:

1. To account for inelasticity in columns, replace $(E_c I_c)$ with $\tau_a(E_c I_c)$ for all columns in the expression for G_A and G_B. The stiffness reduction factor, τ_a, is discussed later in this section.
2. For girders containing significant axial load, multiply the $(EI/L)_g$ by the factor $(1-Q/Q_{cr})$ where Q is the axial load in the girder and Q_{cr} is the in-plane buckling load of the girder based on $K = 1.0$.

For sidesway inhibited frames, these adjustments for different beam end conditions may be made:

1. If the far end of a girder is fixed, multiply the $(EI/L)_g$ of the member by 2.0.
2. If the far end of the girder is pinned, multiply the $(EI/L)_g$ of the member by 1.5.

For sidesway uninhibited frames and girders with different boundary conditions, the modified girder length, L'_g, should be used in place of the actual girder length,
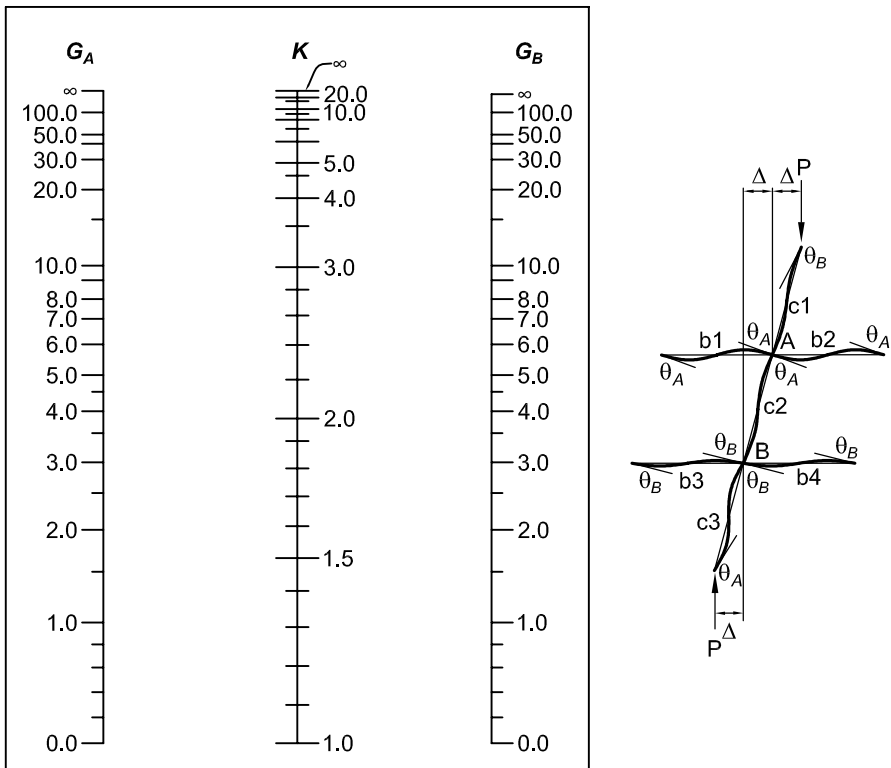

Fig. C-C2.4. Alignment chart—sidesway uninhibited (moment frame).

where

$$L'_g = L_g \left(2 - M_F/M_N\right)$$

M_F is the far end girder moment and M_N is the near end girder moment from a first-order lateral analysis of the frame. The ratio of the two moments is positive if the girder is in reverse curvature. If M_F/M_N is more than 2.0, then L'_g becomes negative, in which case G is negative and the alignment chart equation must be used.

1. If the far end of a girder is fixed, multiply the $(EI/L)_g$ of the member by $2/3$.
2. If the far end of the girder is pinned, multiply the $(EI/L)_g$ of the member by 0.5.

One important assumption in the development of the alignment charts is that all beam-column connections are fully restrained (FR connections). As seen above, when the far end of a beam does not have an FR connection that behaves as assumed, an adjustment must be made. When a beam connection at the column under consideration is a shear only connection—that is, there is no moment—then that beam can not participate in the restraint of the column and it cannot be considered in the $\Sigma(EI/L)_g$ term of the equation for G. Only FR connections can be used directly in the determination of G. PR connections with a documented moment-rotation response can be utilized, but the $(EI/L)_g$ of each beam must be adjusted to account for the connection flexibility. The ASCE Task Committee on Effective Length (ASCE, 1997) provides a detailed discussion of frame stability with PR connections.

Amplified First-Order Elastic Analysis (Section C2.1b). In this application of the effective length factor, K is used in the determination of the elastic critical buckling load, P_{e1}, for a member, or ΣP_{e2}, for a building story. These elastic critical buckling loads are then used for calculation of the corresponding amplification factors B_1 and B_2.

B_1 is used to estimate the P-δ effects on the nonsway moments, M_{nt}, in axially loaded members. K_1 is calculated in the plane of bending on the basis of no translation of the ends of the member and is normally set to 1.0, unless a smaller value is justified on the basis of analysis. There are also P-δ effects on the sway moments, M_{lt}, as explained previously in the discussion of Equation C2-6b.

B_2 is used to determine the P-Δ effect on the various components of moment, braced and/or combined framing systems. K_2 is calculated in the plane of bending through a sidesway buckling analysis. K_2 may be determined from the sidesway uninhibited alignment chart, Figure C-C2.4, without any correction for story buckling discussed later. ΣP_{e2} from the lateral load resisting columns with K_2 calculated in this way is an accurate estimate of the story elastic sidesway buckling strength. The contribution to the story sidesway buckling strength from leaning columns is zero, and therefore, these columns are not included in the summation in Equation C2-6a. However, the total story vertical load, including all columns in the story, is used for $\alpha \Sigma P_{nt}$ in Equation C2-3.

Since the amplified first-order elastic analysis involves the calculation of elastic buckling loads as a measure of frame and column stiffness, only elastic K factors are appropriate for this use.

Column Flexural Buckling Strength, P_n (Chapter E). In this application of effective length factors, K is used in the determination of the column flexural buckling strength, P_n, which may be based either in an elastic or inelastic buckling analysis.

The column elastic buckling stress, F_e, or the corresponding column axial force at incipient story elastic sidesway buckling, P_e, may be used directly in the calculation of the column flexural buckling strength, P_n. This is because the column strength equations of Chapter E (Equations E3-2 and E3-3) are a function of the ratio F_e/F_y. In fact, if the column axial stress at incipient buckling, F_e, is determined from any appropriate system buckling model, this value of F_e is all that is needed for the calculation of P_n.

The elastic column buckling stress, F_e, is given by Equation E3-4 as shown below:

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2} \tag{C-C2-3}$$

This equation uses the effective length factor, K, determined by a buckling analysis of a braced frame or a moment frame. F_e can also be obtained directly from a buckling analysis in which the column buckling load is P_e and

$$F_e = \frac{P_e}{A_g} \tag{C-C2-4}$$

Other approaches for the determination of the effective length factor and the critical buckling load using simplified relationships have been presented in the literature. Several of these will be discussed later in this section.

Braced Frames: If $K < 1$ is used for the calculation of P_n in braced frames, the additional demands on stability bracing and the influence on the second-order moments in beams providing restraint to the columns must be considered. This Specification does not address the additional demands on bracing members from the use of $K < 1$. Generally, a rigorous second-order elastic analysis is necessary for calculation of the second-order moments in beams providing restraint to column members designed based on $K < 1$. Therefore, design using $K = 1$ is recommended, except in those special situations where the additional calculations are deemed justified.

Moment Frames: It is important to recognize that sidesway instability of a moment frame is a story phenomenon involving the sum of the sway resistances of each column in the story and the sum of the factored gravity loads in the columns in that story. No individual column in a story can buckle in a sidesway mode without all the columns in that story also buckling. If each column in a story of an moment frame is designed to support its own P and P-Δ moment such that the contribution of each column to the lateral stiffness, or to the story buckling load, is

proportional to the axial load supported by the column, then all the columns will buckle simultaneously. Under this condition, there is no interaction among the columns in the story; column sway instability and frame instability occur at the same time. However, many common framing systems can be used that redistribute the story $P\text{-}\Delta$ effects to the columns in that story in proportion to their individual stiffnesses. This redistribution can be accomplished using such elements as floor diaphragms or horizontal trusses. In a moment frame that contains columns that contribute little or nothing to the sway stiffness of the story, such columns are referred to as leaning columns and they can be designed using $K = 1.0$. The other columns in the story must be designed to support the destabilizing $P\text{-}\Delta$ moments developed from the loads on these leaning columns. Similarly, the more highly loaded columns in a story will redistribute some of their $P\text{-}\Delta$ moments to the more lightly loaded columns. This phenomenon must be considered in the determination of K and F_e for all the columns in the story for the design of moment frames. The proper K-factor for calculation of P_n in moment frames, accounting for these effects, is denoted in the following by the symbol K_2.

Two methods for evaluating story frame stability, as measured by ΣP_{e2} for a story, are recognized: the story stiffness method (LeMessurier, 1976; LeMessurier, 1977) and the story buckling method (Yura, 1971). These are reflected in Chapter C with Equations C2-6b and C2-6a, respectively.

For the story stiffness method, K_2 is defined by

$$K_2 = \sqrt{\frac{\Sigma P_r}{(0.85 + 0.15 R_L)\, P_r} \left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{\Sigma HL}\right)} \geq \sqrt{\frac{\pi^2 EI}{L^2}\left(\frac{\Delta_H}{1.7 HL}\right)} \quad \text{(C-C2-5)}$$

This value of K_2 may be used in Equation C-C2-3 or directly in the equations of Chapter E. It is possible that certain columns, having only a small contribution to the lateral load resistance in the overall frame, will have a K_2 value less than 1.0 based on the term to the left of the inequality. The limit on the right-hand side is a minimum value for K_2 that accounts for the interaction between sidesway and nonsidesway buckling (ASCE Task Committee on Effective Length, 1997; White and Hajjar, 1997a). The term H is the shear in the column under consideration, produced by the lateral forces used to compute Δ_H.

It is important to note that this equation for K_2 is not appropriate for use in Equation C2-6a for determining ΣP_{e2} and B_2 in Section C2.1b. It has been derived only for the determination of P_n defined in Chapter E.

Alternatively, Equation C-C2-5 can be reformulated to obtain the column buckling load for use in Equation C-C2-4 as

$$P_{e2} = \left(\frac{\Sigma HL}{\Delta_H}\right)\frac{P_r}{\Sigma P_r}(0.85 + 0.15 R_L) \leq 1.7 HL/\Delta_H \qquad \text{(C-C2-6)}$$

$$R_L = \frac{\Sigma P_r \; leaning \; columns}{\Sigma P_r \; all \; columns} \qquad \text{(C-C2-7)}$$

ΣP_r in Equations C-C2-5 and C-C2-6 includes all columns in the story, including any leaning columns, and P_r is for the column under consideration. The column load, P_{e2}, calculated from Equation C-C2-6 may be larger than $\pi^2 EI/L^2$ but may not be larger than the limit on the right hand side of this equation. R_L is the ratio of the vertical column load for all leaning columns in the story to the vertical load of all the columns in the story. This factor approaches 1.0 for systems with a large percentage of leaning columns. The purpose of R_L is to account for the debilitating influence of the P-δ effect on the sidesway stiffness of the columns of a story.

Note that ΣP_{e2} given by Equation C2-6b in the story stiffness method is expressed in terms of a building's story drift ratio Δ_H/L from a first-order lateral load analysis at a given applied lateral load level. In preliminary design, this may be taken in terms of a target maximum value for this drift ratio. This approach focuses the engineer's attention on the most fundamental stability requirement in building frames, providing adequate overall story stiffness in relation to the total vertical load, $\alpha \Sigma P_r$, supported by the story. The elastic story stiffness expressed in terms of the drift ratio and the total horizontal load acting on the story is $\Sigma H/(\Delta_H/L)$.

Story Buckling Method. For the story buckling method, K_2 is defined by

$$K_2 = \sqrt{\frac{\pi^2 EI/L^2}{P_r}\left(\frac{\Sigma P_r}{\Sigma \frac{\pi^2 EI}{(K_{n2}L)^2}}\right)} \geq \sqrt{\frac{5}{8}} K_{n2} \qquad \text{(C-C2-8)}$$

where K_{n2} is defined as the K value determined directly from the alignment chart in Figure C-C2.4. Again, the value for K_2 calculated from the above equation may be less than 1.0. The limit on the right hand side of this equation is a minimum value for K_2 that accounts for the interaction between sidesway and nonsidesway buckling (ASCE Task Committee on Effective Length, 1997; White and Hajjar, 1997a; Geschwindner, 2002; AISC-SSRC, 2003). It is again important to note that this equation for K_2 is *not* appropriate for use in Equation C2-6a for determining ΣP_{e2} and B_2 in Section C2.1b. It has been derived only for the determination of P_n defined in Chapter E.

Alternatively, Equation C-C2-8 can be reformulated to obtain the column buckling load for use in Equation C-C2-4 as

$$P_{e2} = \left(\frac{P_r}{\Sigma P_r}\right) \Sigma \frac{\pi^2 EI}{(K_{n2}L)^2} \leq 1.6 \frac{\pi^2 EI}{(K_{n2}L)^2} \qquad \text{(C-C2-9)}$$

The column load, P_{e2}, calculated from Equation C-C2-9, may be greater than $\pi^2 EI/L^2$ but may not be larger than the limit on the right-hand side of this equation. ΣP_r in Equations C-C2-8 and C-C2-9 includes all columns in the story, including any leaning columns, and P_r is for the column under consideration. K_{n2} in Equations C-C2-8 and C-C2-9 above is determined from the alignment chart in

Figure C-C2.4. Note also that the value of P_n, calculated using K_2 by either method cannot be taken greater than P_n, based on sidesway inhibited buckling. Other methods to calculate K_2 are given in previous editions of this commentary and are discussed elsewhere (ASCE Task Committee on Effective Length, 1997; White and Hajjar, 1997a; Geschwindner, 2002; AISC-SSRC, 2003).

Another simple formula for K_2 (LeMessurier, 1995), based only on the column end moments, is shown below:

$$K_2 = \left[1 + (1 - M_1/M_2)^4\right]\sqrt{1 + \frac{5}{6}\frac{\Sigma P_r \ leaning \ columns}{\Sigma P_r \ nonleaning \ columns}} \quad \text{(C-C2-10)}$$

M_1 is the smaller and M_2 the larger end moment in the column. These moments are determined from a first-order analysis of the frame under wind load. Column inelasticity is considered in the derivation of this equation. The unconservative error in P_n using the above equation is less than 3 percent, as long as the following inequality is satisfied:

$$\left(\frac{\Sigma P_y \ nonleaning \ columns}{\Sigma HL/\Delta_H}\right)\left(\frac{\Sigma P_r \ all \ columns}{\Sigma P_r \ nonleaning \ columns}\right) \le 0.45 \quad \text{(C-C2-11)}$$

As with the other approaches for determining K_2 in this section, this equation for K_2 is not appropriate for use in Equation C2-6a for determining ΣP_{e2} and B_2 in Section C2.1b.

Adjustments in K_2 for Column Inelasticity and Determination of P_n. Adjustments in the effective length factor, K_2, or the column buckling stress, F_e, in the calculation of the column strengths, P_n, can be made based on an inelastic buckling analysis of the frame and the inelasticity inherent in the column under the governing load combination (Yura, 1971; ASCE Task Committee on Effective Length, 1997). Columns loaded into the inelastic range of behavior can be viewed as having a tangent modulus, E_T, that is smaller than E. For such columns, $E_c = E_T$ in the equation for G, which usually gives smaller G values, and therefore, smaller K-factors than those based on elastic behavior. Note that it is usually conservative to base the calculation of P_n on elastic K-factors. For more accurate solutions, inelastic K-factors can be determined from the alignment chart method by using τ_a times E_c for E_c in the equation for G where $\tau_a = E_T/E$ is the column inelastic stiffness reduction factor. Depending on how it is calculated, τ_a may account for both a reduction in the stiffness of columns due to geometric imperfections and spread of plasticity from residual stresses under high compression loading:

(a) For $P_n/P_y \le 0.39$ (elastic):

$$\tau_a = 1.0$$

(b) For $P_n/P_y > 0.39$ (inelastic):

$$\tau_a = -2.724(P_n/P_y)\ln(P_n/P_y) \quad \text{(C-C2-12)}$$

where P_y is the column squash load, $F_y A_g$, and P_n is the nominal column strength. It should be noted the determination of τ_a is in general an iterative process because

P_n (a function of F_e) is dependent upon τ_a. A conservative simplification that eliminates this iterative process is to use $\alpha P_r / \phi_c$ in place of P_n.

Column inelasticity can be considered in determining K_2 (Equations C-C2-5 and C-C2-8) or P_{e2} (Equations C-C2-6 and C-C2-9) for the story stiffness method and the story buckling method. In the story stiffness method, $\tau_a I_c$ can be substituted for I_c for all columns in the frame analysis used to determine Δ_H. In addition, $\tau_a I_c$ can be used in place of I in Equation C-C2-5. In the story buckling method, τ_a is used in the determination of K_{n2} from the alignment chart in Equations C-C2-8 and C-C2-9 and also in those same equations by replacing I_c with $\tau_a I_c$.

If the column inelastic buckling load (P_{e2} from Equations C-C2-6 and C-C2-9 above, modified for inelasticity as described in the above paragraph) is used to determine F_e from Equation C-C2-4 for use in Chapter E (Equations E3-2 and E3-3), then its value must be divided by τ_a as shown below:

$$F_e = \frac{P_{e2} \ (inelastic)}{\tau_a A_g} \qquad \text{(C-C2-13)}$$

The term in the numerator of the above equation denotes the load in the column at incipient inelastic buckling (ASCE Task Committee on Effective Length, 1997). Alternatively, if an inelastic K_2 is determined using τ_a as described in the previous paragraph, this K factor may be substituted directly into Equation C-C2-3 for calculation of F_e.

Some Conclusions Regarding K. It is important to note that column design using K-factors can be tedious and confusing for complex building structures containing leaning columns and/or combined framing systems, particularly where column inelasticity is considered. This confusion can be avoided if the Direct Analysis Method of Appendix 7 is used, where P_n is always based on $K = 1.0$. Also, the first-order elastic design-analysis method of Section C2.2b is based on the Direct Analysis Method, and hence also uses $K = 1.0$ in the determination of P_n. Furthermore, under certain circumstances where B_2 is sufficiently low, a K-factor of 1.0 may be assumed in design by second-order analysis as specified in Section C2.2a (4). For frames that satisfy this clause, it is not appropriate to use $K = 1.0$ in the calculation of B_2 using Equations C2-6a and C2-3. The use of Equation C2-6b is recommended for the calculation of B_2 within this context.

CHAPTER D

DESIGN OF MEMBERS FOR TENSION

The provisions of Chapter D do not account for eccentricities between the lines of action of connected assemblies.

D1. SLENDERNESS LIMITATIONS

The advisory upper limit on slenderness in the User Note is based on professional judgment and practical considerations of economics, ease of handling and care required so as to minimize inadvertent damage during fabrication, transport, and erection. This slenderness limit is not essential to the structural integrity of tension members; it merely assures a degree of stiffness such that undesirable lateral movement ("slapping" or vibration) will be unlikely. Out-of-straightness within reasonable tolerances does not affect the strength of tension members. Applied tension tends to reduce, whereas compression tends to amplify, out-of-straightness.

For single angles, the radius of gyration about the z-axis produces the maximum l/r and, except for very unusual support conditions, the maximum Kl/r.

D2. TENSILE STRENGTH

Because of *strain hardening*, a ductile steel bar loaded in axial tension can resist without rupture a force greater than the product of its gross area and its specified minimum yield stress. However, excessive elongation of a tension member due to uncontrolled yielding of its gross area not only marks the limit of its usefulness but can precipitate failure of the structural system of which it is a part. On the other hand, depending upon the reduction of area and other mechanical properties of the steel, the member can fail by rupture of the net area at a load smaller than required to yield the gross area. Hence, general yielding of the gross area and rupture of the net area both constitute limit states.

The length of the member in the net area is generally negligible relative to the total length of the member. *Strain hardening* is easily reached in the vicinity of holes and yielding of the net area at fastener holes does not constitute a limit state of practical significance.

Except for HSS that are subjected to *cyclic load* reversals, there is no information that the factors governing the strength of HSS in tension differ from those for other structural shapes, and the provisions in Section D2 apply. Because the number of different end connection types that are practical for HSS is limited, the

determination of the net effective area A_e can be simplified using the provisions in Chapter K.

D3.   AREA DETERMINATION

1.   Gross Area

For HSS, ASTM A500 tolerances allow for a wall thickness that is not greater than $\pm$ 10 percent under thickness; consequently the gross area for ASTM A500 HSS is to be computed using 93 percent of the nominal wall thickness. This reduction is included in the tabulated properties for these sections that are included in the AISC *Manual of Steel Construction* (AISC, 2005a).

2.   Net Area

The critical net area is based on net width and load transfer at a particular chain. Because of possible damage around a hole during drilling or punching operations, $1/16$ in. (1.5 mm) is added to the nominal hole diameter when computing the critical net area.

3.   Effective Net Area

Section D3.3 deals with the effect of shear lag, applicable to both welded and bolted tension members. The reduction coefficient U is applied to the net area A_n of bolted members and to the gross area A_g of welded members. As the length of the connection l is increased, the shear lag effect diminishes. This concept is expressed empirically by the equation for U. Using this expression to compute the effective area, the estimated strength of some 1,000 bolted and riveted connection test specimens, with few exceptions, correlated with observed test results within a scatterband of ± 10 percent (Munse and Chesson, 1963). Newer research provides further justification for the current provisions (Easterling and Gonzales, 1993).

For any given profile and configuration of connected elements, $\bar{x}$ is the perpendicular distance from the connection plane, or face of the member, to the centroid of the member section resisting the connection force, as shown in Figure C-D3.1. The length l is a function of the number of rows of fasteners or the length of weld. The length l is illustrated as the distance, parallel to the line of force, between the first and last row of fasteners in a line for bolted connections. The number of bolts in a line, for the purpose of the determination of l, is determined by the line with the maximum number of bolts in the connection. For staggered bolts, the out-to-out dimension is used for l, as shown in Figure C-D3.2.

There is insufficient data for establishing a value of U if all lines have only one bolt, but it is probably conservative to use A_e equal to the net area of the connected element. The limit states of block shear (Section J4.3) and bearing (Section J3.10), which must be checked, will probably control the design.
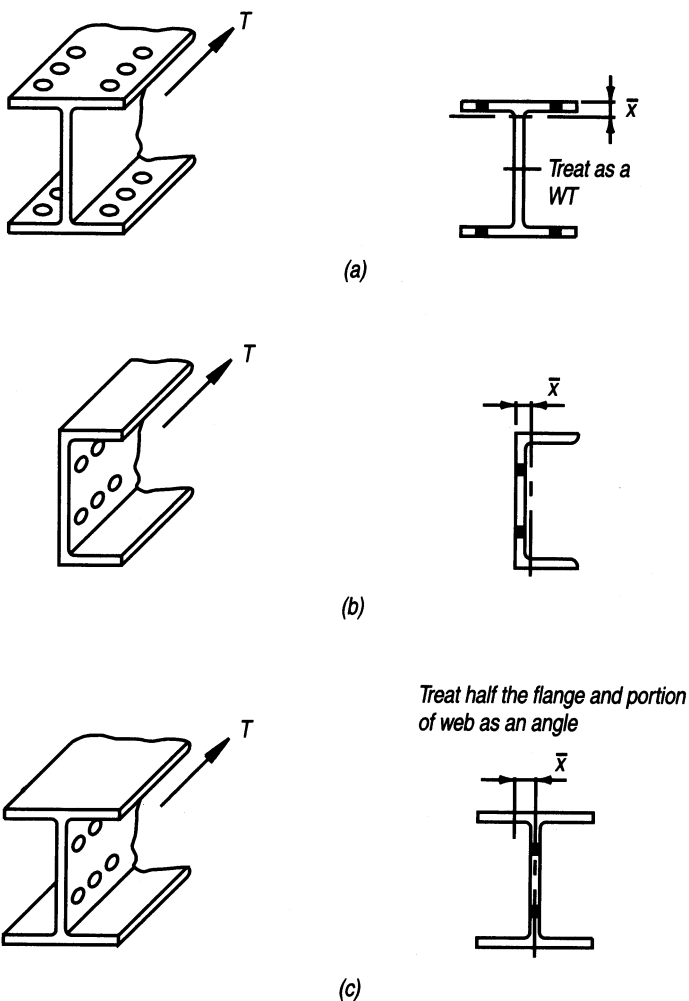
(a)

(b)

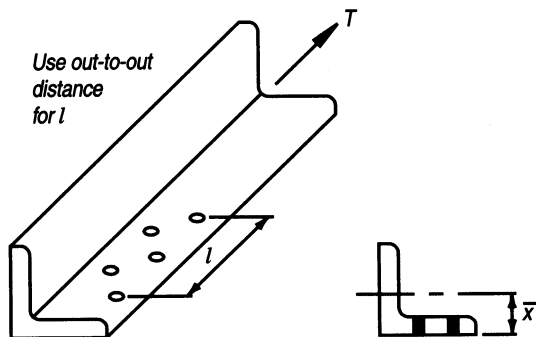*(c)*

Fig. C-D3.1. Determination of $\bar{x}$ for U.

Fig. C-D3.2. Determination of l for U for bolted connections with staggered holes.

Significant eccentricity may exist within the connection if U is less than 0.6. For values of U less than 0.6 the connection may be used only if the provisions for members subject to combined bending and axial force are satisfied in the design of the member.

For welded connections, l is the length of the weld parallel to the line of force as shown in Figure C-D3.3 for longitudinal and longitudinal plus transverse welds.

End connections for HSS in tension are commonly made by welding around the perimeter of the HSS; in this case, there is no shear lag or reduction in the gross area. Alternatively, an end connection with gusset plates can be used. Single gusset plates may be welded in longitudinal slots that are located at the centerline of the cross section. Welding around the end of the gusset plate may be omitted for statically loaded connections to prevent possible *undercutting* of the gusset and having to bridge the gap at the end of the slot. In such cases, the net area at the end of the slot is the critical area as illustrated in Figure C-D3.4. Alternatively, a
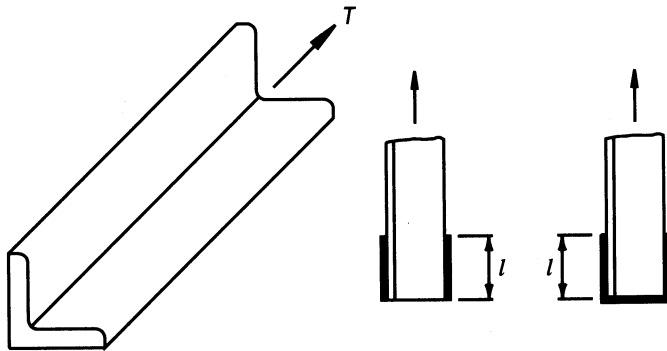
Fig. C-D3.3. Determination of l for U for connections with longitudinal and transverse welds.
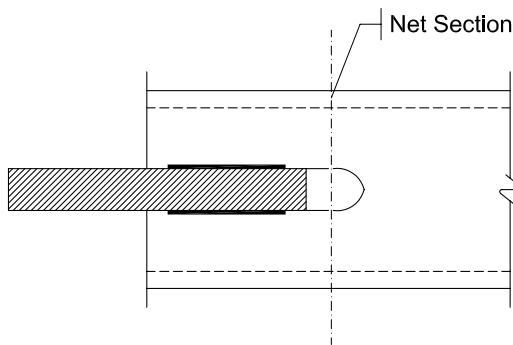
Fig. C-D3.4. Net area through slot for single gusset plate.

pair of gusset plates can be welded to opposite sides of a rectangular HSS with flare bevel groove welds with no reduction in the gross area.

For end connections with gusset plates, the general provisions for shear lag in Case 2 of Table D3.1 can be simplified and the connection eccentricity $\bar{x}$ can be explicitly defined as in Cases 5 and 6. In Cases 5 and 6 it is implied that the weld length, l, should not be less than the depth of the HSS. This is consistent with the weld length requirements in Case 4. In Case 5, the use of $U = 1$ when $l \geq 1.3D$ is based on research (Cheng and Kulak, 2000) that shows that fracture occurs only in short connections and that, in long connections, the round HSS tension member necks within its length and failure is by member yielding and eventual fracture.

The shear lag factors given in Cases 7 and 8 of Table D3.1 were located in the commentary of the 1999 *LRFD Specification* (AISC, 2000b) and are now given as alternate U values to the value determined from $1 − \bar{x}/l$ given for Case 2 in Table D3.1. It is permissible to use the larger of the two values.

D4.   BUILT-UP MEMBERS

Although not commonly used, built-up member configurations using lacing, tie plates and perforated cover plates are permitted by this Specification. The length and thickness of tie plates are limited by the distance between the lines of fasteners, h, which may be either bolts or welds.
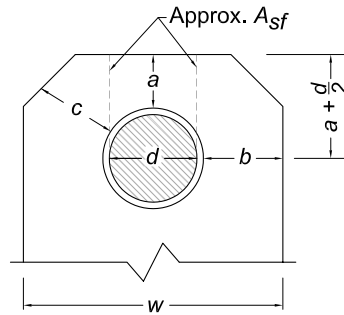
D5.   PIN-CONNECTED MEMBERS

Pin-connected members are occasionally used as tension members with very large dead loads. Pin-connected members are not recommended when there is sufficient variation in live loading to cause wearing of the pins in the holes. The dimensional requirements presented in Specification Section D5.2 must be met to provide for the proper functioning of the pin.

1.   Tensile Strength

The tensile strength requirements for pin-connected members use the same ϕ and Ω values as elsewhere in this Specification for similar limit states. However, the definitions of effective net area for tension and shear are different, as shown in Figure C-D5.1.

2.   Dimensional Requirements

Dimensional requirements for pin-connected members are illustrated in Figure C-D5.1.

Dimensional Requirements

　　　　1. $a \geq 4/3\ b_{eff}$
　　　　2. $w \geq 2b_{eff} + d$
　　　　3. $c \geq a$

where

　　　　$b_{eff} = 2t + 0.625$ in. (16 mm) $\leq b$

Fig. C-D5.1. Dimensional requirements for pin-connected members.

D6.　EYEBARS

Forged eyebars have generally been replaced by pin-connected plates or eyebars thermally cut from plates. Provisions for the proportioning of eyebars contained in this Specification are based upon standards evolved from long experience with forged eyebars. Through extensive destructive testing, eyebars have been found to provide balanced designs when they are thermally cut instead of forged. The more conservative rules for pin-connected members of nonuniform cross section and for members not having enlarged "circular" heads are likewise based on the results of experimental research (Johnston, 1939).

Stockier proportions are required for eyebars fabricated from steel having a yield stress greater than 70 ksi (485 MPa) to eliminate any possibility of their "dishing" under the higher design stress.
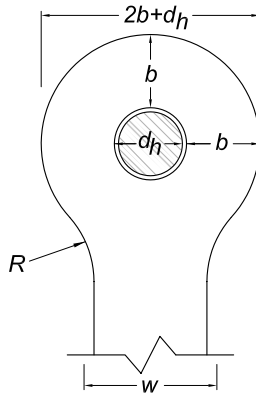
1.　Tensile Strength

The tensile strength of eyebars is determined as for general tension members, except that, for calculation purposes, the width of the body of the eyebar is limited to eight times its thickness.

2.　Dimensional Requirements

Dimensional limitations for eyebars are illustrated in Figure C-D6.1.

Dimensional Requirements

$t \geq \frac{1}{2}$ in. (13mm) (Exception is provided in D6.2)

$w \leq 8t$

$d \geq \frac{7}{8}w$

$d_h \leq d + \frac{1}{32}$ in. (1mm)

$R \geq d_h + 2b$

$\frac{2}{3}w \leq b \leq \frac{3}{4}w$ (Upper limit is for calculation purposes only)

Fig. C-D6.1. Dimensional limitations for eyebars.

CHAPTER E

DESIGN OF MEMBERS FOR COMPRESSION

E1. GENERAL PROVISIONS

The basic column equations in Section E3 are based on a reasonable conversion of research data into strength equations (Tide, 1985; Tide, 2001). These equations are essentially the same as those in the three previous editions of the *LRFD Specification* (see the discussion in Commentary Section E3 for further discussion). The one significant difference between the previous *LRFD Specifications* and this Specification is that the resistance factor φ has been increased from 0.85 to 0.90. The reasons for this increase are the changes in industry practice since the original calibrations were performed in the 1970s.

In the original research on the probability-based strength of steel columns (Bjorhovde, 1972; Bjorhovde, 1978) three *column curves* were recommended. These three *column curve*s were the mean equations of data bands of columns of similar manufacture. For example, hot-formed and cold-formed heat treated HSS columns fell into the data band of highest strength [SSRC Column Category P1 in Galambos (1998), Chapter 3], while welded built-up wide-flange columns made from universal mill plates were included in the data band of lowest strength (SSRC Column Category P3). The largest group of data clustered around SSRC Column Category P2. Had the original *LRFD Specification* opted for using all three *column curves* for the respective column categories, probabilistic analysis would have resulted in a resistance factor equal to φ = 0.90 (Galambos, 1983; Galambos, 1998). It was decided, however, to employ only one *column curve*, SSRC Column Category P2, for all column types. This resulted in a larger data spread and thus in a larger coefficient of variation, and so a resistance factor φ = 0.85 was adopted for the column equations to achieve a reliability comparable to that of beams.

The single *column curve* and the resistance factor of 0.85 were selected by the AISC Committee on Specifications in 1981 when the first draft of the *LRFD Specification* was developed (AISC, 1986). Since then there have been a number of changes in industry practice: (1) welded built-up shapes are no longer manufactured from universal mill plates; and (2) the yield strength of steel has increased with the standard constructional steel (ASTM 992) having a nominal yield stress of 50 ksi (345 MPa). The spread of the yield stress, in other words, its coefficient of variation, has been reduced (Bartlett and others, 2003).

An examination of the SSRC *Column Curve* Selection Table [Figure 3.27 in Galambos (1998)] reveals that there is no longer any SSRC P3 *Column Curve* Category. It is now possible to conservatively use only the statistical data for SSRC Column

Category P2 for the probabilistic determination of the reliability of columns. The curves in Figures C-E1.1 and C-E1.2 show the variation of the reliability index β with the live-to-dead load ratio L/D in the range of 1 to 5 for LRFD with $\phi = 0.90$ and ASD with $\Omega = 1.67$, respectively, for $F_y = 50$ ksi (345 MPa). The reliability index does not fall below $\beta = 2.6$. This is comparable to the reliability of beams. The ASD method gives higher reliability in the lower L/D range than the LRFD method.
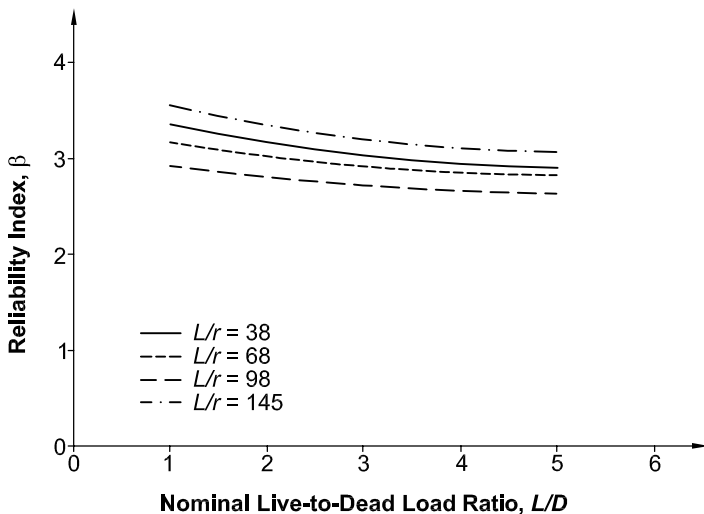
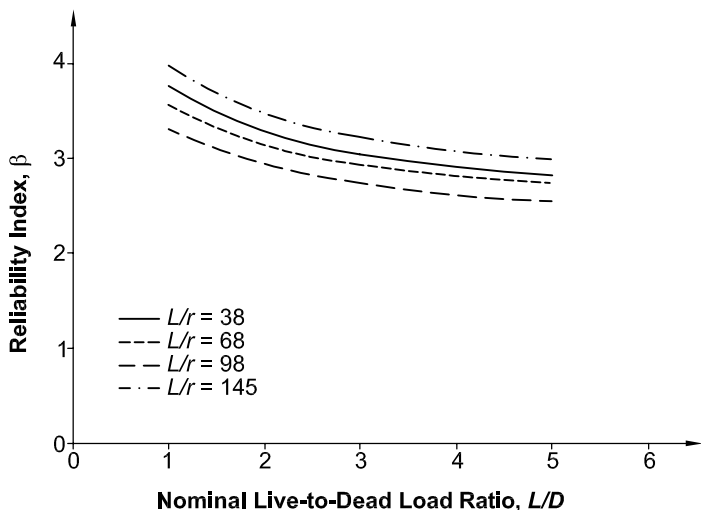Fig. C-E1.1. Reliability of columns (LRFD).

Fig. C-E1.2. Reliability of columns (ASD).

E2.   SLENDERNESS LIMITATIONS AND EFFECTIVE LENGTH

The concept of a maximum limiting slenderness ratio has experienced an evolutionary change from a mandatory ". . . The slenderness ratio, *KL/r*, of compression members shall not exceed 200 . . . " in the 1978 Specification to no restriction at all in this Specification. The 1978 *ASD* and the 1999 *LRFD Specifications* (AISC, 1978; AISC, 2000b) provide a transition from the rigid mandatory limit to no limit by the flexible provision that ". . . the slenderness ratio, KL/r, preferably should not exceed 200. . . ." This latter restriction is actually no limit at all, so the present Specification has disposed with the provision altogether. However, the designer should keep in mind that columns with a slenderness ratio of more than 200 will have a critical stress (Equation E3-4) less than 6.3 ksi (43.5 MPa). The traditional upper limit of 200 was based on professional judgment and practical construction economics, ease of handling, and care required to minimize inadvertent damage during fabrication, transport and erection. It is not recommended to exceed this limit for compression members except for cases where special care is exercised by the fabricator and erector.

E3.   COMPRESSIVE STRENGTH FOR FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

Section E3 applies to compression members with compact and noncompact sections, as defined in Section B4.

The column strength equations in Section E3 are the same as those in the previous editions of the LRFD Specification, with the exception of the cosmetic replacement of the nondimensional slenderness ratio $\lambda_c = \dfrac{KL}{\pi r}\sqrt{\dfrac{F_y}{E}}$ by the more familiar $\dfrac{KL}{r}$. For the convenience of those calculating the elastic buckling stress directly, without first calculating *K*, the limits on use of Equations E3-2 and E3-3 are also provided in terms of F_e.

Comparisons between the previous column design curves and the new one are shown in Figures C-E3.1 and C-E3.2 for the case of $F_y = 50$ ksi (345 MPa). The curves show the variation of the available column strength with the slenderness ratio for LRFD and ASD, respectively. The LRFD curves reflect the change of the resistance factor ϕ from 0.85 to 0.90, as was explained in Commentary Section E1 above. For both LRFD and ASD, the new column equations give somewhat more economy than the previous editions of the Specification.

The limit between elastic and inelastic buckling is defined to be $\dfrac{KL}{r} = 4.71\sqrt{\dfrac{E}{F_y}}$ or $F_e = 0.44 F_y$. For convenience, these limits are defined in Table C-E3.1 for the common values of F_y.

One of the key parameters in the column strength equations is the elastic critical stress, F_e. Equation E3-4 presents the familiar Euler form for F_e. However, F_e can

be determined by other means also, including a direct frame buckling analysis, as permitted in Chapter C, or from a torsional or *flexural-torsional* buckling analysis addressed in Section E4.

The column strength equations of Section E3 are generic equations that can be used for frame buckling and for torsional or flexural-torsional buckling (Section E4); they can also be entered with a modified slenderness ratio for single-angle members (Section E5); and they can be modified by the *Q*-factor for columns with slender elements (Section E7).
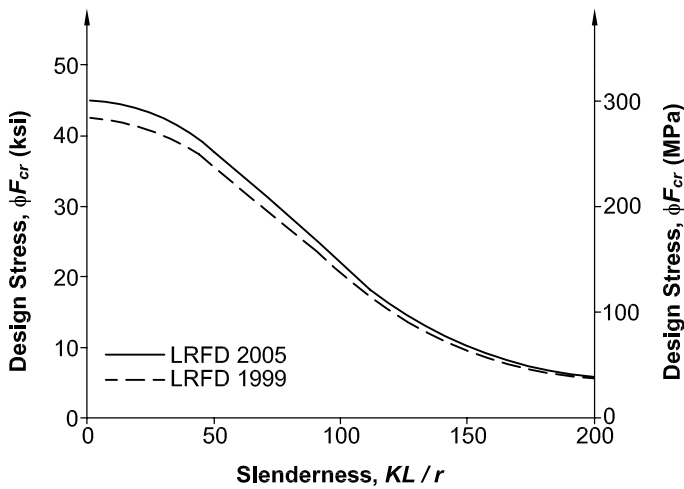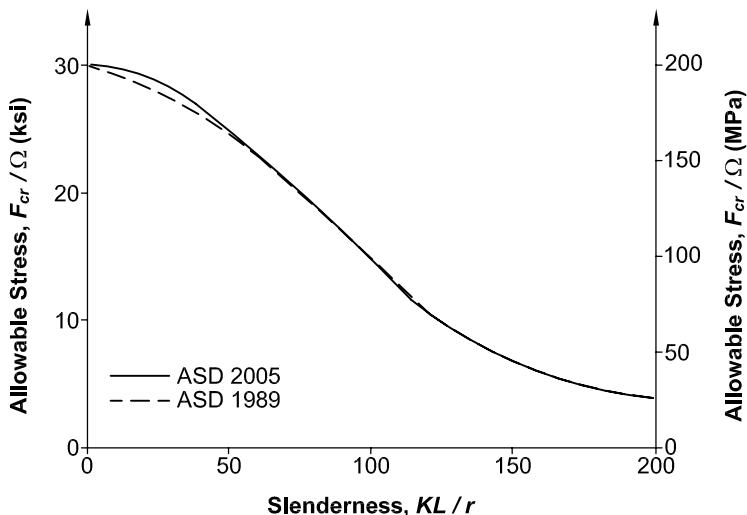

Fig. C-E3.1. LRFD column curves compared.

Fig. C-E3.2. ASD column curves compared.

TABLE C-E3.1
Limiting Values of *KL / r* and *F$_e$*

F_y ksi (MPa)	Limiting $\frac{KL}{r}$	F_e ksi (MPa)
36 (248)	134	15.8 (109)
50 (345)	113	22.0 (152)
60 (414)	104	26.4 (182)
70 (483)	96	30.8 (212)

E4.   COMPRESSIVE STRENGTH FOR TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

Section E4 applies to singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up columns, with compact and noncompact sections, as defined in Section B4 for uniformly compressed elements.
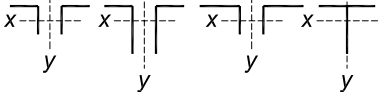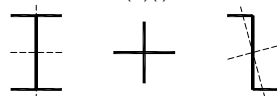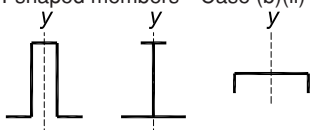
The equations in Section E4 for determining the torsional and *flexural-torsional* elastic buckling loads of columns are derived in texts on structural stability [for example, Timoshenko and Gere (1961); Bleich (1952); Galambos (1968); Chen and Atsuta (1977)]. Since these equations apply only to elastic buckling, they must be modified for inelastic buckling by using the torsional and flexural-torsional critical stress, F_{cr}, in the column equations of Section E3.

Torsional buckling of symmetric shapes and flexural-torsional buckling of unsymmetrical shapes are failure modes usually not considered in the design of hot-rolled columns. They generally do not govern, or the *critical load* differs very little from the weak-axis planar buckling load. Torsional and flexural-torsional buckling modes may, however, control the strength of symmetric columns manufactured from relatively thin plate elements and unsymmetric columns and symmetric columns having torsional unbraced lengths significantly larger than the weak-axis flexural unbraced lengths. Equations for determining the critical stress for such columns are given in Section E4. Table C-E4.1 serves as a guide for selecting the appropriate equations.

The simpler method of calculating the buckling strength of double-angle and T-shaped members (Equation E4-2) uses directly the y-axis flexural strength from the column equations of Section E3 (Galambos, 1991). Tees that conform to the limits of Table C-E4.2 need not be checked for flexural-torsional buckling.

Equations E4-4 and E4-11 contain a torsional buckling effective length factor K_z. This factor may be conservatively taken as $K_z = 1.0$. For greater accuracy, $K_z = 0.5$ if both ends of the column have a connection that restrains warping, say by boxing the end over a length at least equal to the depth of the member. If one end

TABLE C-E4.1 Selection of Equations for Torsional and Flexural-Torsional Buckling	
Type of Cross Section	**Applicable Equations in Section E4**
Double angle and T-shaped members— Case (a) in Section E4. 	E4-2 and E4-3
All doubly symmetric shapes and Z-shapes— Case (b)(i) 	E4-4
Singly symmetric members except double angles and T-shaped members—Case (b)(ii) 	E4-5
Unsymmetrical shapes—Case (b)(iii) 	E4-6

TABLE C-E4.2 Limiting Proportions for Tees		
Shape	**Ratio of Full Flange Width to Profile Depth**	**Ratio of Flange Thickness to Stem Thickness**
Built-up tees	≥ 0.50	≥ 1.25
Rolled tees	≥ 0.50	≥ 1.10

of the member is restrained from warping and the other end is free to warp, then $K_z = 0.7$.

At points of bracing both lateral and/or torsional bracing shall be provided, as required in Appendix 6. Seaburg and Carter (1997) provides an overview of the fundamentals of torsional loading for structural steel members. Design examples are also included.

E5.   SINGLE-ANGLE COMPRESSION MEMBERS

Section E5 addresses the design of single angles subjected to an axial compressive load effect introduced through one connected leg. The attached leg is to be

fixed to a gusset plate or the projecting leg of another member by welding or by a bolted connection with at least two bolts. The equivalent slenderness expressions in this section presume significant restraint about the y-axis, which is perpendicular to the connected leg. This leads to the angle member tending to bend and buckle primarily about the x-axis. For this reason L/r_x is the slenderness parameter used. The modified slenderness ratios indirectly account for bending in the angles due to the eccentricity of loading and for the effects of end restraint from the truss chords. The values for box trusses reflect greater rotational end restraint as compared to that provided by planar trusses.

The equivalent slenderness expressions also presume a degree of rotational restraint. Equations E5-3 and E5-4 [Case (b)] assume a higher degree of x-axis rotational restraint than do Equations E5-1 and E5-2 [Case (a)]. Equations E5-3 and E5-4 are essentially equivalent to those employed for equal-leg angles as web members in latticed transmission towers in ASCE 10-97 (ASCE, 2000).

In space trusses, the web members framing in from one face typically restrain the twist of the chord at the panel points and thus provide significant x-axis restraint of the angles under consideration. It is possible that the chords of a planar truss well restrained against twist justify use of Case (b), in other words, Equations E5-3 and E5-4. Similarly, simple single-angle diagonal braces in braced frames could be considered to have enough end restraint such that Case (a), in other words, Equations E5-1 and E5-2 could be employed for their design. This procedure, however, is not intended for the evaluation of the compressive strength of x-braced single angles.

The procedure in Section E5 permits use of unequal-leg angles attached by the smaller leg provided that the equivalent slenderness is increased by an amount that is a function of the ratio of the longer to the shorter leg lengths, and has an upper limit on L/r_z.

If the single-angle compression members cannot be evaluated using the procedures in this section, use the provisions of Section H2. In evaluating P_n, the effective length due to end restraint should be considered. With values of effective length factors about the geometric axes, one can use the procedure in Lutz (1992) to compute an effective radius of gyration for the column. To obtain results that are not too conservative, one must also consider that end restraint reduces the eccentricity of the axial load of single-angle struts and thus the value of f_b used in the flexural term(s) in Equation H2-1.

E6.   BUILT-UP MEMBERS

Section E6 addresses the strength and dimensional requirements of built-up members composed of two or more shapes interconnected by stitch bolts or welds.

1. Compressive Strength

The longitudinal spacing of connectors connecting components of built-up compression members must be such that the slenderness ratio L/r of individual shapes does not exceed three-fourths of the slenderness ratio of the entire member. However, this requirement does not necessarily ensure that the effective slenderness ratio of the built-up member is equal to that of a built-up member acting as a single unit. Section E6.1 gives equations for modified slenderness ratios that are based on research and take into account the effect of shear deformation in the connectors (Zandonini, 1985). Equation E6-1 for snug-tight intermediate connectors is empirically based on test results. Equation E6-2 is derived from theory and verified by test data. In both cases the end connection must be welded or fully tensioned bolted (Aslani and Goel, 1991). The connectors must be designed to resist the shear forces that develop in the buckled member. The shear stresses are highest where the slope of the buckled member is the steepest (Bleich, 1952). Fastener spacing less than the maximum required for strength may be needed to ensure a close fit over the entire faying surface of components in continuous contact. Special requirements for weathering steel members exposed to atmospheric corrosion are given in Brockenbrough (1983).

2. Dimensional Requirements

Section E6.2 provides requirements for dimensioning built-up members that cannot be stated in terms of calculated stress but are based upon judgment and experience.

E7. MEMBERS WITH SLENDER ELEMENTS

The structural engineer designing with hot-rolled plates and shapes will seldom find an occasion to turn to Section E7 of the Specification. Among rolled shapes the most frequently encountered cases requiring the application of this section are columns containing angles with thin legs and tee-shaped columns having slender stems. Special attention to the determination of Q must be given when columns are made by welding or bolting thin plates together.

The provisions of Section E7 address the modifications to be made when one or more plate elements in the column cross sections are slender. A plate element is considered to be slender if its width-thickness ratio exceeds the limiting value λ_r defined in Table B4.1. As long as the plate element is not slender, it can support the full yield stress without local buckling. When the cross section contains slender elements, the slenderness reduction factor Q defines the ratio of the stress at local buckling to the yield stress, F_y. The yield stress, F_y, is replaced by the value QF_y in the column equations of Section E3. These equations are repeated as Equations E7-2 and E7-3. This approach to dealing with columns with slender elements has been used since the 1969 Specification (AISC, 1969), emulating the 1969 AISI Specification (AISI, 1969). Prior to 1969, the AISC practice was to remove the width of the plate that exceeded the limit λ_r and check the remaining cross section for conformance with the allowable stress, which proved inefficient and

uneconomical. The equations in Section E7 are almost identical to the original equations, with one notable exception that will be discussed subsequently.

This Specification makes a distinction between columns having unstiffened and stiffened elements. Two separate philosophies are used: Unstiffened elements are considered to have attained their limit state when they reach the theoretical local buckling stress. Stiffened elements, on the other hand, make use of the post-buckling strength inherent in a plate that is supported on both of its longitudinal edges, such as in HSS columns. The effective width concept is used to obtain the added post-buckling strength. This dual philosophy reflects 1969 practice in the design of cold-formed columns. Subsequent editions of the AISI Specifications, in particular, the *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2001), hereafter referred to as the AISI *North American Specification* adopted the effective width concept for both stiffened and unstiffened columns. Following editions of the AISC Specification (including this Specification) did not follow the example set by AISI for unstiffened plates because the advantages of the post-buckling strength do not become available unless the plate elements are very slender. Such dimensions are common for cold-formed columns, but are rarely encountered in structures made from hot-rolled plates.

1.   **Slender Unstiffened Elements, Q_s**

Equations for the slender element reduction factor, Q_s, are given in Section E7.1 for outstanding elements in rolled shapes (Case a), built-up shapes (Case b), single angles (Case c), and stems of tees (Case d). The underlying scheme for these provisions is illustrated in Figure C-E7.1. The curves show the relationship between the Q-factor and a non-dimensional slenderness ratio $\dfrac{b}{t}\sqrt{\dfrac{F_y}{E}\dfrac{12(1-v^2)}{\pi^2 k}}$.
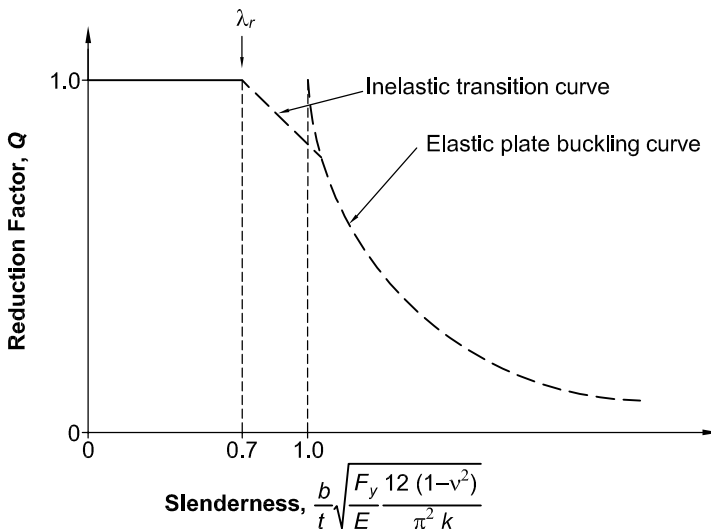

Fig. C-E7.1. Definition of Q_s for unstiffened slender elements.

The width b and thickness t are defined for the applicable cross sections in Section B4; $\nu = 0.3$ (Poisson's ratio), and k is the plate buckling coefficient characteristic of the type of plate edge-restraint. For single angles, $k = 0.425$ (no restraint is assumed from the other leg), and for outstanding flange elements and stems of tees, k equals approximately 0.7, reflecting an estimated restraint from the part of the cross section to which the plate is attached on one of its edges, the other edge being free.

The curve relating Q to the plate slenderness ratio has three components: (i) a part where $Q = 1$ when the slenderness factor is less than or equal to 0.7 (the plate can be stressed up to its yield stress), (ii) the elastic plate buckling portion when buckling is governed by $F_{cr} = \dfrac{\pi^2 E k}{12(1 - \nu^2)\left(\frac{b}{t}\right)^2}$, and (iii) a transition range that empirically accounts for the effect of early yielding due to *residual stresses* in the shape. Generally this transition range is taken as a straight line. The development of the provisions for unstiffened elements is due to the research of Winter and his co-workers, and a full listing of references is provided in the Commentary to the AISI *North American Specification* (AISI, 2001). The slenderness provisions are illustrated for the example of slender flanges of rolled shapes in Figure C-E7.2.

The equations for the unstiffened projecting flanges, angles and plates in built-up cross sections (Equations E7-7 through E7-9) have a history that starts with the research reported in Johnson (1985). It was noted in tests of beams with slender flanges and slender webs that there was an interaction between the buckling of the flanges and the distortions in the web causing an unconservative prediction of strength. A modification based on the equations recommended in Johnson (1985) appeared first in the 1989 *ASD Specification* (AISC, 1989).
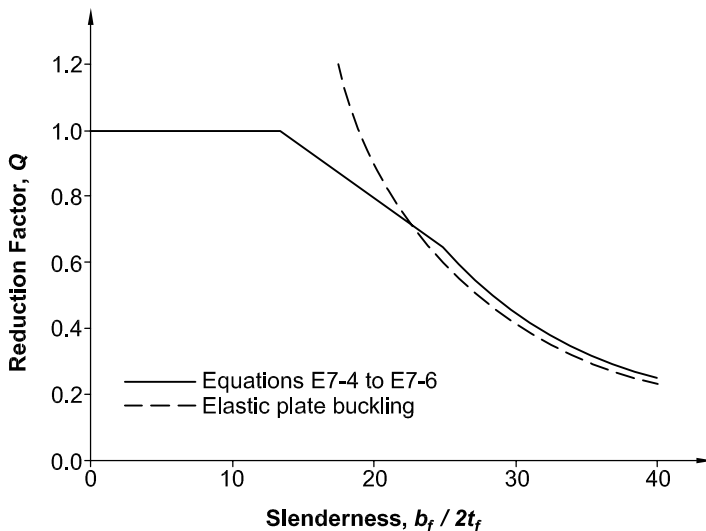

Fig. C-E7.2. Q *for rolled wide-flange columns of* $F_y = 50$ *ksi (345 MPa).*

Modifications to simplify the original equations were introduced in the 1993 *LRFD Specification* (AISC, 1993), and these equations have remained unchanged in the present Specification. The influence of web slenderness is accounted for by the introduction of the factor

$$k_c = \frac{4}{\sqrt{\dfrac{h}{t_w}}} \qquad \text{(C-E7-1)}$$

into the equations for λ_r and Q, where k_c shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

2. Slender Stiffened Elements, Q_a

While for slender unstiffened elements the Specification for local buckling is based on the limit state of the onset of plate buckling, an improved approach based on the effective width concept is used for the compressive strength of stiffened elements in columns. This method was first proposed in von Kármán, Sechler, and Donnell (1932). This was later modified in Winter (1947) to provide a transition between very slender elements and stockier elements shown by tests to be fully effective. As modified in Winter (1947) for the AISI *North American Specification* (AISI, 2001), the ratio of effective width to actual width increases as the level of compressive stress applied to a stiffened element in a member is decreased, and takes the form

$$\frac{b_e}{t} = 1.9\sqrt{\frac{E}{f}}\left[1 - \frac{C}{(b/t)}\sqrt{\frac{E}{f}}\right] \qquad \text{(C-E7-2)}$$

where f is taken as F_{cr} of the column based on $Q = 1.0$, and C is a constant based on test results (Winter, 1947).

The basis for cold-formed steel columns in the AISI *North American Specification* editions since the 1970s is $C = 0.415$. The original AISI coefficient 1.9 in Equation C-E7-2 is changed to 1.92 in the Specification to reflect the fact that the modulus of elasticity E is taken as 29,500 ksi (203 400 MPa) for cold-formed steel, and 29,000 ksi (200 000 MPa) for hot-rolled steel.

For the case of square and rectangular box-sections of uniform thickness, where the sides provide negligible rotational restraint to one another, the value of $C = 0.38$ in Equation E7-18 is higher than the value of $C = 0.34$ in Equation E7-17. Equation E7-17 applies to the general case of stiffened plates in uniform compression where there is substantial restraint from the adjacent flange or web elements. The coefficients $C = 0.38$ and $C = 0.34$ are smaller than the corresponding value of $C = 0.415$ in the AISI *North American Specification* (AISI, 2001), reflecting the fact that hot-rolled steel sections have stiffer connections between plates due to welding or fillets in rolled shapes than do cold-formed shapes.

The classical theory of longitudinally compressed cylinders overestimates the actual buckling strength, often by 200 percent or more. Inevitable imperfections of shape and the eccentricity of the load are responsible for the reduction in actual

strength below the theoretical strength. The limits in Section E7.2(c) are based upon test evidence (Sherman, 1976), rather than theoretical calculations, that local buckling will not occur if

$$\frac{D}{t} \leq \frac{0.11E}{F_y}$$

When D/t exceeds this value but is less than

$$\frac{D}{t} \leq \frac{0.45E}{F_y}$$

Equation E7-19 provides a reduction in the local buckling reduction factor Q. This Specification does not recommend the use of round HSS or pipe columns with

$$\frac{D}{t} > \frac{0.45E}{F_y}$$

CHAPTER F

DESIGN OF MEMBERS FOR FLEXURE

F1.   GENERAL PROVISIONS

Chapter F applies to members subject to simple bending about one principal axis of the cross section. Section F2 gives the provisions for the flexural strength of doubly symmetric compact I-shaped and channel members subject to bending about their major axis. For most designers, the provisions in this section will be sufficient to perform their everyday designs. The remaining sections of Chapter F address less frequently occurring cases encountered by structural engineers. Since there are many such cases, many equations and many pages in the Specification, the table in User Note F1.1 is provided as a map for navigating through the cases considered in Chapter F. The coverage of the chapter is extensive and there are many equations that appear formidable; however, it is stressed again that for most designs, the engineer need seldom go beyond Section F2.

For all sections covered in Chapter F, the highest possible nominal flexural strength is the plastic moment, $M_n = M_p$. Being able to use this value in design represents the optimum use of the steel. In order to attain M_p the beam cross section must be compact and the member must be laterally braced. Compactness depends on the flange and web plate width-to-thickness ratios, as defined in Section B4. When these conditions are not met, the available nominal flexural strength diminishes. All sections in Chapter F treat this reduction in the same way. For laterally braced beams, the plastic moment region extends over the range of plate width-thickness ratios λ terminating at λ_p. This is the compact condition. Beyond these limits the nominal moment reduces linearly until λ reaches λ_r. This is the range where the section is noncompact. Beyond λ_r the section is a slender-element section.

These three ranges are illustrated in Figure C-F1.1 for the case of rolled wide-flange members for the limit state of flange local buckling. The curve in Figure C-F1.1 shows the relationship between the flange width-thickness ratio $b_f/2t_f$ and the nominal flexural strength, M_n.

The basic relationship between the nominal flexural strength, M_n, and the unbraced length, L_b, for the limit state of lateral-torsional buckling is shown in Figure C-F1.2 for a compact section [W27×84 (W690×125), $F_y = 50$ ksi (345 MPa)] subjected to uniform bending, $C_b = 1.0$.

There are four principal zones defined on the basic curve by the lengths L_{pd}, L_p, and L_r. Equation F2-5 defines the maximum unbraced length L_p to reach M_p

with uniform moment. Elastic lateral-torsional buckling will occur when the un-braced length is greater than L_r given by Equation F2-6. Equation F2-2 defines the range of inelastic lateral-torsional buckling as a straight line between the defined limits M_p at L_p and $0.7F_yS_x$ at L_r. Buckling strength in the elastic region is given by Equations F2-3 and F2-4 for I-shaped members. The length
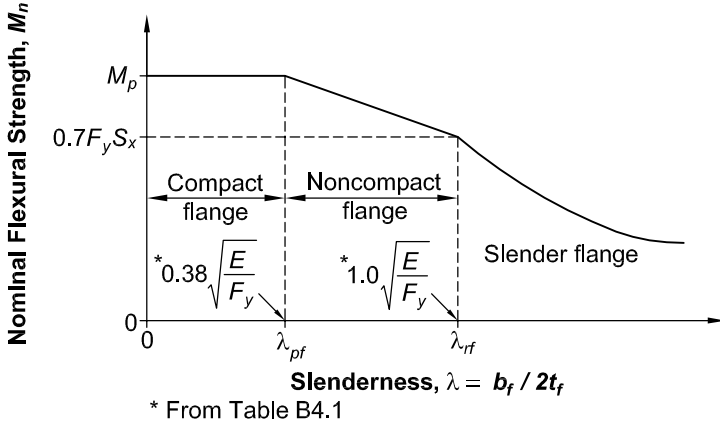

Fig. C-F1.1. Nominal flexural strength as a function of the flange width-thickness ratio of rolled I-shapes.
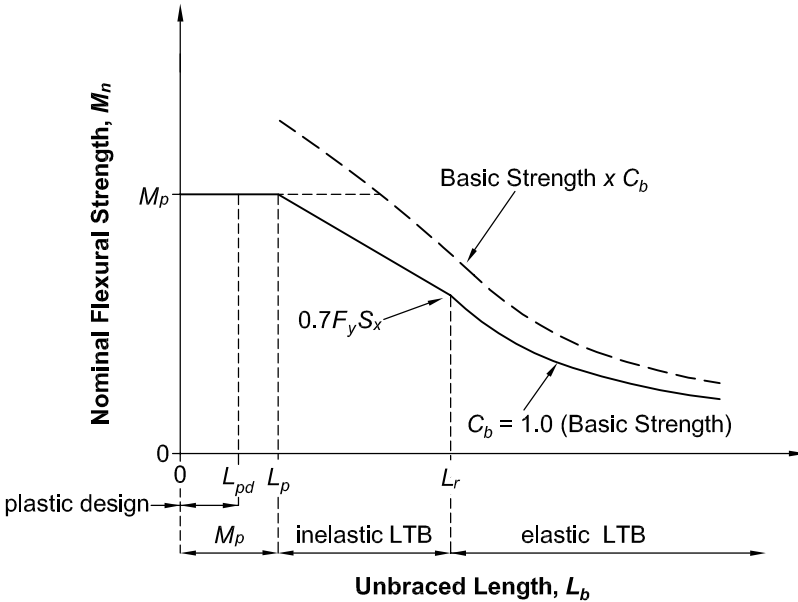

Fig. C-F1.2. Nominal flexural strength as a function of unbraced length and moment gradient.

L_{pd} is defined in Appendix 1 as the limiting unbraced length needed for plastic design.

For moment diagrams along the member other than uniform moment, the lateral buckling strength is obtained by multiplying the basic strength in the elastic and inelastic region by C_b as shown in Figure C-F1.2. However, in no case can the maximum moment capacity exceed the plastic moment M_p. Note that L_p given by Equation F2-5 is merely a definition that has physical meaning only when $C_b = 1.0$. For C_b greater than 1.0, members with larger unbraced lengths can reach M_p, as shown by the curve for $C_b > 1.0$ in Figure C-F1.2. This length is calculated by setting Equation F2-2 equal to M_p and solving for L_b using the actual value of C_b.

The equation

$$C_b = 1.75 + 1.05 \left(\frac{M_1}{M_2} \right) + 0.3 \left(\frac{M_1}{M_2} \right)^2 \qquad \text{(C-F1-1)}$$

has been used since 1961 in AISC Specifications to adjust the lateral-torsional buckling equations for variations in the moment diagram within the unbraced length. However, this equation is only applicable to moment diagrams that consist of straight lines between braced points—a condition that is rare in beam design. The equation provides a lower bound to the solutions developed in Salvadori (1956). Equation C-F1-1 can be easily misinterpreted and misapplied to moment diagrams that are not linear within the unbraced segment. Kirby and Nethercot (1979) present an equation that applies to various shapes of moment diagrams within the unbraced segment. Their original equation has been slightly adjusted to give Equation C-F1-2 (Equation F1-1 in the body of the Specification):

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C} \qquad \text{(C-F1-2)}$$

This equation gives a more accurate solution for a fixed-end beam, and gives approximately the same answers as Equation C-F1-1 for moment diagrams with straight lines between points of bracing. C_b computed by Equation C-F1-2 for moment diagrams with other shapes show good comparison with the more precise but also more complex equations (Galambos, 1998). The absolute values of the three quarter-point moments and the maximum moment regardless of its location are used in Equation C-F1-2. The maximum moment in the unbraced segment is always used for comparison with the nominal moment M_n. The length between braces, not the distance to inflection points is used. It is still satisfactory to use C_b from Equation C-F1-1 for straight-line moment diagrams within the unbraced length.

The equations for the limit state of lateral-torsional buckling in Chapter F assume that the loads are applied along the beam centroidal axis. C_b may be conservatively taken equal to 1.0, with the exception of some cases involving unbraced cantilevers or members with no bracing within the span and with significant loading applied to the top flange. If the load is placed on the top flange and the flange is not braced, there is a tipping effect that reduces the critical moment; conversely, if the load is suspended from an unbraced bottom flange, there is a stabilizing effect that increases the critical moment (Galambos, 1998). For unbraced top flange loading on compact I-shaped members, the reduced critical moment may be conservatively approximated by setting the square root expression in Equation F2-4 equal to unity.

An effective length factor of unity is implied in the critical moment equations to represent the worst-case simply supported unbraced segment. Consideration of any end restraint due to adjacent nonbuckled segments on the critical segment can increase its strength. The effects of beam continuity on lateral-torsional buckling have been studied, and a simple conservative design method, based on the analogy to end-restrained nonsway columns with an effective length less than unity, has been proposed (Galambos, 1998).

F2. DOUBLY SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

Section F2 applies to members with compact I-shaped or channel cross sections subject to bending about their major axis; hence the only limit state to consider is lateral-torsional buckling. Almost all rolled wide-flange shapes listed in the AISC *Manual of Steel Construction* are eligible to be designed by the provisions of this section, as indicated in the User Note in the Specification.

The equations in Section F2 are identical to the corresponding equations in Section F1 of the 1999 *LRFD Specification* (AISC, 2000b), although they are presented in different form. The following table gives the list of equivalent equations:

TABLE C-F2.1
Comparison of Equations for Nominal Flexural Strength

1999 AISC/LRFD Specification Equations	Current Specification Equations
F1-1	F2-1
F1-2	F2-2
F1-13	F2-3 and F2-4

The only difference between the two specifications is that the stress at the interface between inelastic and elastic buckling has been changed from $F_y - F_r$ in the 1999

edition to $0.7F_y$ herein. In the previous Specification the residual stress, F_r, for rolled and welded shapes was different, namely 10 ksi (69 MPa) and 16.5 ksi (114 MPa), respectively, while in this Specification the *residual stress* was taken as $0.3F_y$ so that the value of $F_y - F_r = 0.7F_y$ was adopted. This change was made in the interest of simplicity with negligible effect on economy.

The elastic lateral-torsional buckling stress, F_{cr}, of Equation F2-4:

$$F_{cr} = \frac{C_b\pi^2 E}{\left(\dfrac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \qquad \text{(C-F2-1)}$$

is identical to Equation F1-13 in the 1999 *LRFD Specification* (AISC, 2000b):

$$F_{cr} = \frac{M_{cr}}{S_x} = \frac{C_b\pi}{L_b S_x}\sqrt{EI_y GJ + \left(\frac{\pi E}{L_b}\right)^2 I_y C_w} \qquad \text{(C-F2-2)}$$

if $c = 1$ (see Section F2 for definition) and

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x}; \quad h_o = d - t_f; \quad \text{and} \quad \frac{2G}{\pi^2 E} = 0.0779$$

Equation F2-5 is the same as F1-4 in the 1999 *LRFD Specification* (AISC, 2000b), and F2-6 corresponds to F1-6. It is obtained by setting $F_{cr} = 0.7F_y$ in Equation F2-4 and solving for L_b. The term r_{ts} can conservatively be calculated as the radius of gyration of the compression flange plus one-sixth of the web.

These provisions have been simplified when compared to the previous ASD provisions based on a more informed understanding of beam limit states behavior. The maximum allowable stress obtained in these provisions may be slightly higher than the previous limit of $0.66F_y$, since the true plastic strength of the member is reflected by use of the plastic section modulus in Equation F2-1. The Section F2 provisions for unbraced length are satisfied through the use of two equations, one for inelastic lateral-torsional buckling (Equation F2-2), and one for elastic lateral-torsional buckling (Equation F2-3). Previous ASD provisions placed an arbitrary stress limit of $0.6F_y$ when a beam was not fully braced and required that three equations be checked with the selection of the largest stress to determine the strength of a laterally unbraced beam. With the current provisions, once the unbraced length is determined, the member strength can be obtained directly from these equations.

F3. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES BENT ABOUT THEIR MAJOR AXIS

Section F3 is a supplement to Section F2 for the case where the flange of the section is noncompact or slender (see Figure C-F1.1, linear variation of M_n

between λ_{pf} and λ_{rf}). As pointed out in the user note of Section F2, very few rolled wide-flange shapes are subject to this criterion.

F4. OTHER I-SHAPED MEMBERS WITH COMPACT OR NONCOMPACT WEBS BENT ABOUT THEIR MAJOR AXIS

Section F4 has no direct counterpart in previous AISC Specifications except for the lateral buckling provisions for singly symmetric sections in Table A-F1.1 in the 1999 *LRFD Specification* (AISC, 2000b). These provisions are not carried over to the present Specification. The provisions of Section F4 are applicable to doubly symmetric wide-flange beams with slender flanges and to singly symmetric wide-flange members with compact, noncompact, and slender flanges, and noncompact webs (see the Table in User Note F1.1). This part of Chapter F essentially deals with welded I-shaped beams where the webs are not slender. The following section, F5, considers welded I-shapes with slender webs. The contents of Section F4 are based on White (2004).

Three limit states are considered: (a) lateral-torsional buckling (LTB); (b) flange local buckling (FLB); and (c) tension flange yielding (TFY). The effect of inelastic buckling of the web is taken care of indirectly by multiplying the moment causing yielding in the compression flange by a factor R_{pc} and the moment causing yielding in the tension flange by a factor R_{pt}. These two factors can vary from unity to as high as 1.6. Conservatively, they can be assumed to equal 1.0. The following steps are provided as a guide to the determination of R_{pc} and R_{pt}.

Step 1. Calculate h_p and h_c: See Figure C-F4.1.

Step 2. Determine web slenderness and yield moments in compression and tension:

$$
\left. \begin{array}{l}
\lambda = \dfrac{h_c}{t_w} \\[2ex]
S_{xc} = \dfrac{I_x}{y}; \quad S_{xt} = \dfrac{I_x}{d-y} \\[2ex]
M_{yc} = F_y S_{xc}; \quad M_{yt} = F_y S_{xt}
\end{array} \right\} \qquad \text{(C-F4-1)}
$$

Step 3. Determine λ_{pw} and λ_{rw}

$$
\left. \begin{array}{l}
\lambda_{pw} = \dfrac{\dfrac{h_c}{h_p}\sqrt{\dfrac{E}{F_y}}}{\left[\dfrac{0.54 M_p}{M_y} - 0.09\right]^2} \leq 3.76\sqrt{\dfrac{E}{F_y}} \\[5ex]
\lambda_{rw} = 5.70\sqrt{\dfrac{E}{F_y}}
\end{array} \right\} \qquad \text{(C-F4-2)}
$$

If $\lambda > \lambda_{rw}$ then the web is slender and the design is governed by Section F5.

Step 4. Calculate R_{pc} and R_{pt} by Equations F4-9a or F4-9b and F4-15a or F4-15b, respectively.

The basic maximum nominal moment is $R_{pc}M_{yc} = R_{pc}F_y S_{xc}$ if the flange is in compression, and $R_{pt}M_{yt} = R_{pt}F_y S_{xt}$ if it is in tension. Thereafter, the provisions are the same as for doubly symmetric members in Sections F2 and F3. For the limit state of lateral-torsional buckling, I-shaped members with cross sections that have unequal flanges are treated as if they were doubly symmetric I-shapes. That is, Equations F2-4 and F2-6 are the same as Equations F4-5 and F4-8, except the former use S_x and the latter use S_{xc}, the elastic section moduli of the entire section and of the compression side, respectively. This is a simplification that tends to be somewhat conservative if the compression flange is smaller than the tension flange, and it is somewhat unconservative when the reverse is true. It is also required to check for tension flange yielding if the tension flange is smaller than the compression flange (Section F4.3).

For a more accurate solution, especially when the loads are not applied at the centroid of the member, the designer is directed to Chapter 5 of the SSRC Guide (Galambos, 1998; Galambos, 2001; White and Jung, 2003). White gives the following alternative equations in lieu of Equations F4-5 and F4-8:

$$M_n = C_b \frac{\pi^2 EI_y}{L_b^2} \left\{ \frac{\beta_x}{2} + \sqrt{\left(\frac{\beta_x}{2}\right)^2 + \frac{C_w}{I_y}\left[1 + 0.0390\frac{J}{C_w}L_b^2\right]} \right\} \quad \text{(C-F4-3)}$$

$$L_r = \frac{1.38E\sqrt{I_y J}}{S_{xc}F_{yr}}\sqrt{\frac{2.6\beta_x F_{yr}S_{xc}}{EJ} + 1 + \sqrt{\left[\frac{2.6\beta_x F_{yr}S_{xc}}{EJ} + 1\right]^2 + \frac{27.0C_w}{I_y}\left(\frac{F_{yr}S_{xc}}{EJ}\right)^2}}$$
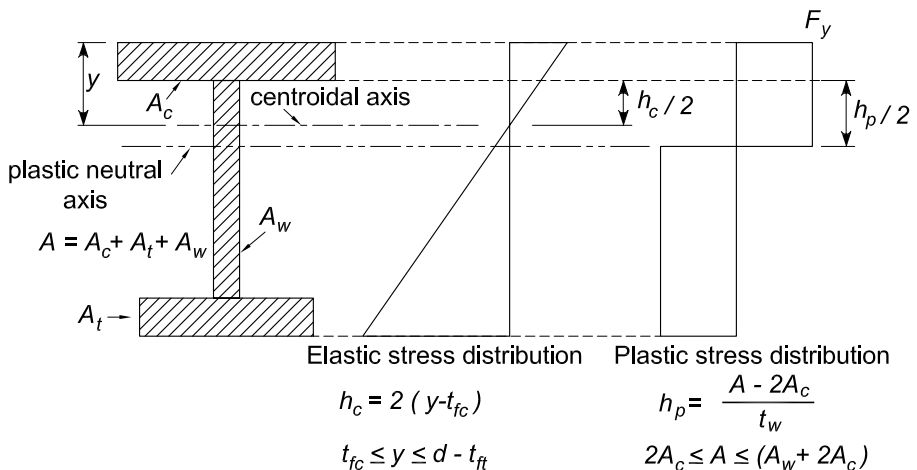$$\text{(C-F4-4)}$$

Fig. C-F4.1. Elastic and plastic stress distributions.

where the coefficient of monosymmetry, $\beta_x = 0.9h\alpha\left(\dfrac{I_{yc}}{I_{yt}} - 1\right)$,

the warping constant, $C_w = h^2 I_{yc}\alpha$, and $\alpha = \dfrac{1}{\dfrac{I_{yc}}{I_{yt}} + 1}$.

F5. DOUBLY SYMMETRIC AND SINGLY SYMMETRIC I-SHAPED MEMBERS WITH SLENDER WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies for doubly and singly symmetric I-shaped welded plate girders with a slender web, that is, $\dfrac{h_c}{t_w} > \lambda_r = 5.70\sqrt{\dfrac{E}{F_y}}$. The applicable limit states are lateral-torsional buckling, compression flange local buckling and tension flange local yielding. The provisions in this section have changed little since 1963. They are similar to the provisions in Section A-G2 in the 1999 *LRFD Specification* (AISC, 2000b), and similar to the provisions in Section G2 in the 1989 *ASD Specification* (AISC, 1989). The provisions for plate girders are based on research reported in Basler and Thurlimann (1963).

There is no seamless transition between the equations in Section F4 and F5. Thus the bending strength of a girder with $F_y = 50$ ksi (345 MPa) and a web slenderness $h/t_w = 137$ is not close to that of a girder with $h/t_w = 138$. These two slenderness ratios are on either side of the limiting ratio. This gap is caused by the discontinuity between the lateral-torsional buckling resistances predicted by Section F4 and those predicted by Section F5 due to the implicit use of $J = 0$ in Section F5. However, for typical noncompact web section members close to the noncompact web limit, the influence of J on the lateral-torsional buckling resistance is relatively small (for example, the calculated L_r values including J versus using $J = 0$ typically differ by less than 10 percent). The implicit use of $J = 0$ in Section F5 is intended to account for the influence of web distortional flexibility on the lateral-torsional buckling resistance for slender-web I-section members.

F6. I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

I-shaped members and channels bent about their minor axis do not experience lateral-torsional buckling or web buckling. The only limit states to consider are yielding and flange local buckling. The user note informs the designer of the few rolled shapes that need to be checked for flange local buckling.

F7. SQUARE AND RECTANGULAR HSS AND BOX-SHAPED MEMBERS

The provisions for the nominal flexural strength of HSS include the limit states of yielding and local buckling. Square and rectangular HSS bent about the minor axis are not subject to lateral-torsional buckling.

Because of the high torsional resistance of the closed cross-section, the critical unbraced lengths L_p and L_r that correspond to the development of the plastic moment and the yield moment, respectively, are very large. For example, as shown in Figure C-F7.1, an HSS $20 \times 4 \times {}^5/_{16}$ (HSS $508 \times 101.6 \times 7.9$), which has one of the largest depth-width ratios among standard HSS, has L_p of 6.7 ft (2.0 m) and L_r of 137 ft (42 m) as determined in accordance with the 1993 *LRFD Specification* (AISC, 1993). An extreme deflection limit might correspond to a length-to-depth ratio of 24 or a length of 40 ft (12 m) for this member. Using the specified linear reduction between the plastic moment and the yield moment for lateral-torsional buckling, the plastic moment is reduced by only 7 percent for the 40-ft (12 m) length. In most practical designs where the moment gradient C_b is larger than unity, the reduction will be nonexistent or insignificant.

The provisions for local buckling of noncompact rectangular HSS are also the same as those in the previous sections of this chapter: $M_n = M_p$ for $b/t \leq \lambda_p$, and a linear transition from M_p to $F_y S_x$ when $\lambda_p < b/t \leq \lambda_r$. The equation for the effective width of the compression flange when b/t exceeds λ_r is the same as that used for rectangular HSS in axial compression except that the stress is taken as the yield stress. This implies that the stress in the corners of the compression flange is at yield when the ultimate post-buckling strength of the flange is reached. When using the effective width, the nominal flexural strength is determined from the effective section modulus to the compression flange using the distance from the shifted neutral axis. A slightly conservative estimate of the nominal flexural strength can be obtained by using the effective width for both the compression and tension flange, thereby maintaining the symmetry of the cross section and simplifying the calculations.
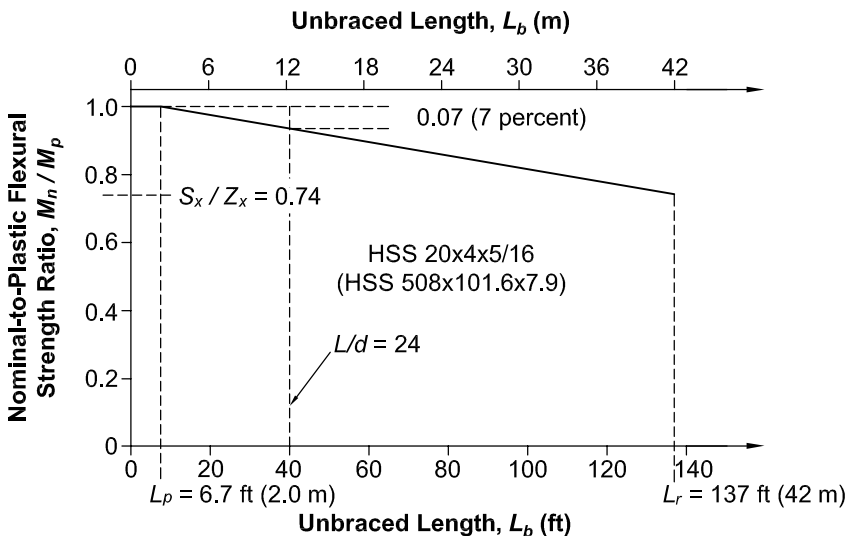

Fig. C-F7.1. Lateral-torsional buckling of rectangular HSS.

F8.   ROUND HSS

Round HSS are not subject to lateral-torsional buckling. The failure modes and post-buckling behavior of round HSS can be grouped into three categories (Sherman, 1992; Galambos, 1998):

(a) For low values of D/t, a long plastic plateau occurs in the moment-rotation curve. The cross section gradually ovalizes, local wave buckles eventually form, and the moment resistance subsequently decays slowly. Flexural strength may exceed the theoretical plastic moment due to *strain hardening*.

(b) For intermediate values of D/t, the plastic moment is nearly achieved but a single local buckle develops and the flexural strength decays slowly with little or no plastic plateau region.

(c) For high values of D/t HSS, multiple buckles form suddenly with very little ovalization and the flexural strength drops quickly.

The flexural strength provisions for round HSS reflect these three regions of behavior and are based upon five experimental programs involving hot-formed seamless pipe, electric-resistance-welded pipe and fabricated tubing (Galambos, 1998).

F9.   TEES AND DOUBLE ANGLES LOADED IN THE PLANE OF SYMMETRY

The lateral-torsional buckling (LTB) strength of singly symmetric tee beams is given by a fairly complex formula (Galambos, 1998). Equation F9-4 is a simplified formulation based on Kitipornchai and Trahair (1980). See also Ellifritt, Wine, Sputo, and Samuel (1992).

The C_b factor used for I-shaped beams is unconservative for tee beams with the stem in compression. For such cases $C_b = 1.0$ is appropriate. When beams are bent in reverse curvature, the portion with the stem in compression may control the LTB resistance even though the moments may be small relative to other portions of the unbraced length with $C_b \approx 1.0$. This is because the LTB strength of a tee with the stem in compression may be only about one-fourth of the strength for the stem in tension. Since the buckling strength is sensitive to the moment diagram, C_b has been conservatively taken as 1.0. In cases where the stem is in tension, connection details should be designed to minimize any end restraining moments that might cause the stem to be in compression.

No limiting stem width-thickness ratio, λ_r, is provided in this section to account for the local buckling of the stem when it is in compression. The reason for this omission is that the lateral-torsional buckling equations (Equations F9-4 and F9-5) also give the local buckling strength as L_b approaches zero. This is not immediately evident, because when $L_b = 0$ is substituted into these equations one obtains, after some algebraic manipulations, $M_{cr} = 0/0$, which is a mathematically indeterminate expression. From elementary calculus such a problem is solved by differentiating the numerator and the denominator as often as needed

to arrive at an explicit expression using L'Hospital's rule. If this operation is performed twice, one can obtain the following equation for the critical moment of combined lateral-torsional and local buckling:

$$M_{cr,L_b=0} = \frac{\pi\, EJ\sqrt{\dfrac{G}{E}}}{4.6d} = 0.424\frac{EJ}{d} \qquad\qquad \text{(C-F9-1)}$$

The relationship between the unbraced length and the critical moment for a WT18×67.5 (WT460×100.5) [$F_y = 50$ ksi (345 MPa)] tee beam, with the stem in flexural compression, is shown in Figure C-F9.1.

Although flexure about the y-axis of tees and double angles does not occur frequently, guidance is given here to address this condition. The yield limit state and the local buckling limit state of the flange can be checked by using Equations F6-1 through F6-4. Lateral-torsional buckling can conservatively be calculated by assuming the flange acts alone as a rectangular beam, using Equations F11-2 through F11-4. Alternately an elastic critical moment given as

$$M_e = \frac{\pi}{L_b}\sqrt{EI_x GJ} \qquad\qquad \text{(C-F9-2)}$$
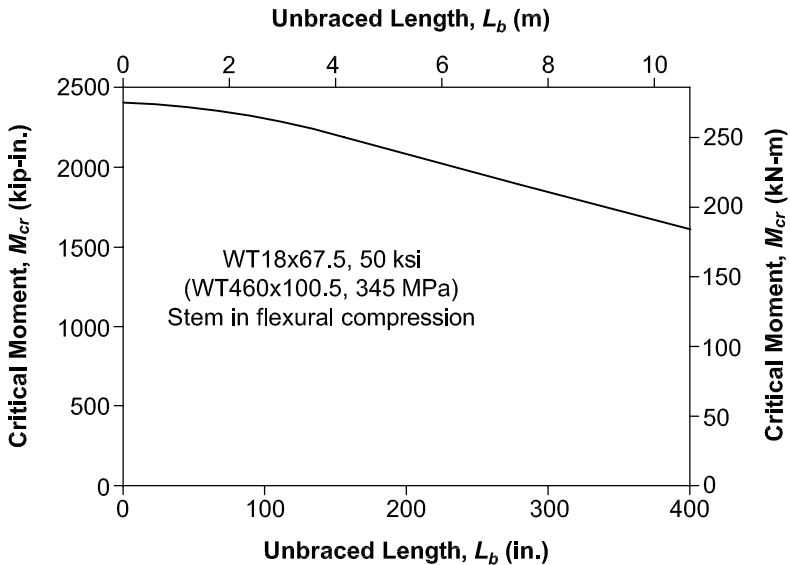
Critical moment when $L_b = 0$: M_{LB} = 2409 kip-in. (274.25 kN-m)
This is also local buckling of the stem under flexural compression.
Yield moment: M_y = 2485 kip-in. (282.90 kN-m)

Fig. C-F9.1. Critical moment for a tee beam
[WT18×67.5 (WT460×100.5), $F_y = 50$ ksi (345 MPa)].

may be used in Equations F10-2 or F10-3 to obtain the nominal flexural strength.

F10.   SINGLE ANGLES

Flexural strength limits are established for the limit states of yielding, local buckling and lateral-torsional buckling of single-angle beams. In addition to addressing the general case of unequal-leg single angles, the equal-leg angle is treated as a special case. Furthermore, bending of equal-leg angles about a geometric axis, an axis parallel to one of the legs, is addressed separately as it is a common case of angle bending.

The tips of an angle refer to the free edges of the two legs. In most cases of unrestrained bending, the flexural stresses at the two tips will have the same sign (tension or compression). For constrained bending about a geometric axis, the tip stresses will differ in sign. Provisions for both tension and compression at the tip should be checked as appropriate, but in most cases it will be evident which controls.

Appropriate serviceability limits for single-angle beams need also to be considered. In particular, for longer members subjected to unrestrained bending, deflections are likely to control rather than lateral-torsional or local buckling strength.

The provisions in this section follow the general format for nominal flexural resistance (see Figure C-F1.2). There is a region of full yielding, a linear transition to the yield moment, and a region of local buckling.

1.   Yielding

The strength at full yielding is limited to a shape factor of 1.50 applied to the yield moment. This leads to a lower bound plastic moment for an angle that could be bent about any axis, inasmuch as these provisions are applicable to all flexural conditions. The 1.25 factor originally used was known to be a conservative value. Recent research work (Earls and Galambos, 1997) has indicated that the 1.50 factor represents a better lower bound value. Since the shape factor for angles is in excess of 1.50, the nominal design strength $M_n = 1.5M_y$ for compact members is justified provided that instability does not control.

2.   Lateral-Torsional Buckling

Lateral-torsional buckling may limit the flexural strength of an unbraced single-angle beam. As illustrated in Figure C-F10.1, Equation F10-2 represents the elastic buckling portion with the maximum nominal flexural strength, M_n, equal to 75 percent of the theoretical buckling moment, M_e. Equation F10-3 represents the inelastic buckling transition expression between $0.75M_y$ and $1.5M_y$. The maximum beam flexural strength $M_n = 1.5M_y$ will occur when the theoretical buckling moment, M_e, reaches or exceeds $7.7M_y$. These equations are

modifications of those developed from the results of Australian research on single angles in flexure and on an analytical model consisting of two rectangular elements of length equal to the actual angle leg width minus one-half the thickness (AISC, 1975; Leigh and Lay, 1978; Leigh and Lay, 1984; Madugula and Kennedy, 1985).

When bending is applied about one leg of a laterally unrestrained single angle, the angle will deflect laterally as well as in the bending direction. Its behavior can be evaluated by resolving the load and/or moments into principal axis components and determining the sum of these principal axis flexural effects. Section F10.2(i) is provided to simplify and expedite the calculations for this common situation with equal-leg angles.

For such unrestrained bending of an equal-leg angle, the resulting maximum normal stress at the angle tip (in the direction of bending) will be approximately 25 percent greater than the calculated stress using the geometric axis section modulus. The value of M_e in Equation F10-5 and the evaluation of M_y using 0.80 of the geometric axis section modulus reflect bending about the inclined axis shown in Figure C-F10.2.

The deflection calculated using the geometric axis moment of inertia has to be increased 82 percent to approximate the total deflection. Deflection has two components, a vertical component (in the direction of applied load) 1.56 times the calculated value and a horizontal component of 0.94 times the calculated value. The resultant total deflection is in the general direction of the weak principal axis bending of the angle (see Figure C-F10.2). These unrestrained bending deflections should be considered in evaluating serviceability and will often control the design over lateral-torsional buckling.
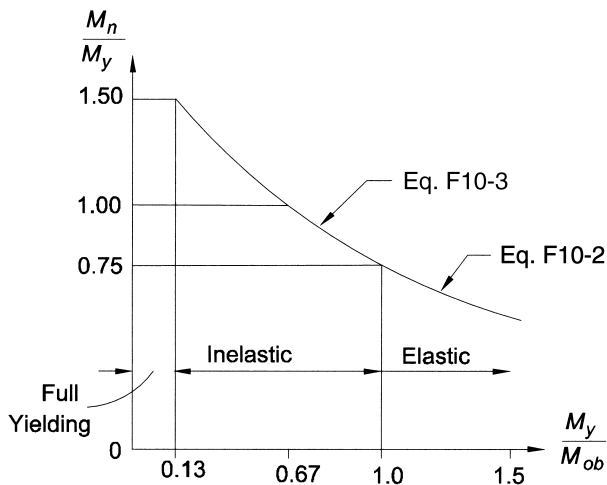

Fig. C-F10.1. Lateral-torsional buckling limits of a single-angle beam.

The horizontal component of deflection being approximately 60 percent of the vertical deflection means that the lateral restraining force required to achieve purely vertical deflection must be 60 percent of the applied load value (or produce a moment 60 percent of the applied value) which is very significant.

Lateral-torsional buckling is limited by M_e (Leigh and Lay, 1978; Leigh and Lay, 1984) in Equation F10-4a, which is based on

$$M_{cr} = \frac{2.33Eb^4t}{(1 + 3\cos^2\theta)(Kl)^2} \times$$

$$\left[\sqrt{\sin^2\theta + \frac{0.156(1 + 3\cos^2\theta)(Kl)^2t^2}{b^4}} + \sin\theta \right] \qquad \text{(C-F10-1)}$$

(the general expression for the critical moment of an equal-leg angle) with $\theta = 45°$ or the condition where the angle tip stress is compressive (see Figure C-F10.3). Lateral-torsional buckling can also limit the flexural strength of the cross section when the maximum angle tip stress is tensile from geometric axis flexure, especially with use of the flexural strength limits in Section F10.2. Using $\theta = 45°$ in Equation C-F10-1, the resulting expression is Equation F10-4b with a +1 instead of −1 as the last term.

Stress at the tip of the angle leg parallel to the applied bending axis is of the same sign as the maximum stress at the tip of the other leg when the single angle is unrestrained. For an equal-leg angle this stress is about one-third of the maximum stress. It is only necessary to check the nominal bending strength based on the tip of the angle leg with the maximum stress when evaluating such an angle. Since this maximum moment per Section F10.2(ii) represents combined principal axis moments and Equation F10-5 represents the design limit for these
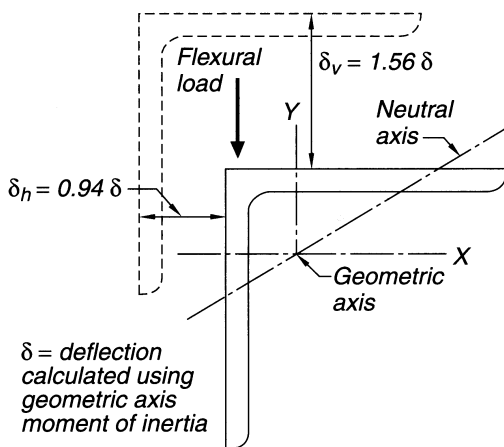

Fig. C-F10.2. Geometric axis bending of laterally unrestrained equal-leg angles.

combined flexural moments, only a single flexural term needs to be considered when evaluating combined flexural and axial effects.

For unequal-leg angles without lateral-torsional restraint, the applied load or moment must be resolved into components along the two principal axes in all cases and design must be for *biaxial bending* using the interaction equations in Chapter H.

Under major axis bending of equal-leg angles, Equation F10-5 in combination with Equations F10-2 and F10-3 controls the available moment against overall lateral-torsional buckling of the angle. This is based on M_e, given earlier with $\theta = 0$.

Lateral-torsional buckling for this case will reduce the stress below $1.5M_y$ only for $l/t \geq 3675C_b/F_y (M_e = 7.7M_y)$. If the lt/b^2 parameter is small (less than approximately $0.87C_b$ for this case), local buckling will control the available moment and M_n based on lateral-torsional buckling need not be evaluated. Local buckling must be checked using Section F10.3.

Lateral-torsional buckling about the major principal w-axis of an unequal-leg angle is controlled by M_e in Equation F10-6. The section property β_w reflects the location of the shear center relative to the principal axis of the section and the bending direction under uniform bending. Positive β_w and maximum M_e occurs when the shear center is in flexural compression while negative β_w and minimum M_e occur when the shear center is in flexural tension (see Figure C-F10.4). This β_w effect is consistent with behavior of singly symmetric I-shaped beams, which are more stable when the compression flange is larger than the tension flange. For principal w-axis bending of equal-leg angles, β_w is equal to zero due to symmetry and Equation F10-6 reduces to Equation F10-5 for this special case.
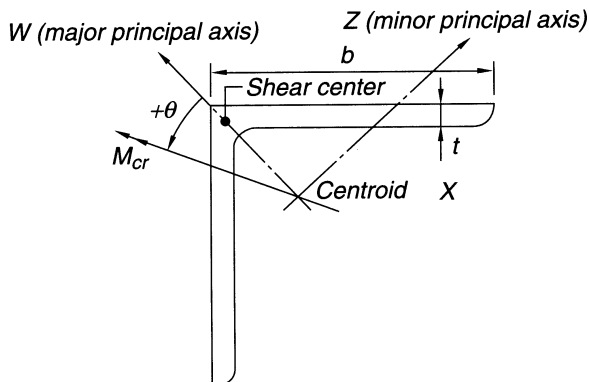

Fig. C-F10.3. Equal-leg angle with general moment loading.

For reverse curvature bending, part of the unbraced length has positive β_w, while the remainder has negative β_w; conservatively, the negative value is assigned for that entire unbraced segment.

The factor β_w is essentially independent of angle thickness (less than one percent variation from mean value) and is primarily a function of the leg widths. The average values shown in Table C-F10.1 may be used for design.

3. Leg Local Buckling

The b/t limits have been modified to be more representative of flexural limits rather than using those for single angles under uniform compression. Typically the flexural stresses will vary along the leg length permitting the use of the stress limits given. Even for the geometric axis flexure case, which produces uniform compression along one leg, use of these limits will provide a conservative value when compared to the results reported in Earls and Galambos (1997).

F11. RECTANGULAR BARS AND ROUNDS

The provisions in Section F11 apply to solid bars with round and rectangular cross section. The prevalent limit state for such members is the attainment of the full plastic moment, M_p. The exception is the lateral-torsional buckling of rectangular bars where the depth is larger than the width. The requirements for design are identical to those given previously in Table A-F1.1 in the 1999 *LRFD Specification* (AISC, 2000b). Since the shape factor for a rectangular cross section is 1.5 and for a round section is 1.7, consideration must be given to serviceability issues such as excessive deflection or permanent deformation under service-load conditions.

F12. UNSYMMETRICAL SHAPES

When the design engineer encounters beams that do not contain an axis of symmetry, or any other shape for which there are no provisions in the other sections of Chapter F, the stresses are to be limited by the yield stress or the elastic buckling
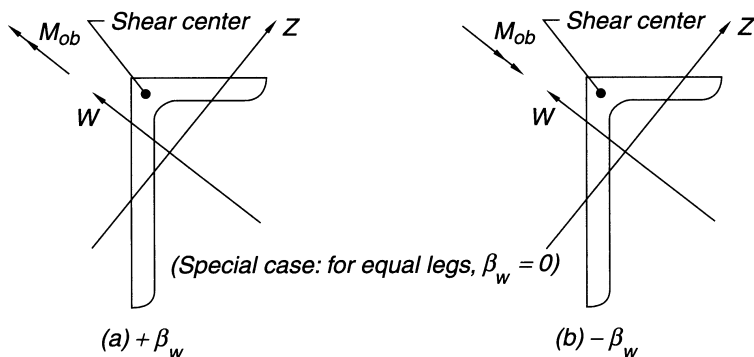

Fig. C-F10.4. *Unequal-leg angle in bending.*

TABLE C-F10.1 β_w Values for Angles	
Angle Size [in. (mm)]	**β_w [in. (mm)]***
9 × 4 (229 × 102)	6.54 (116)
8 × 6 (203 × 152)	3.31 (84.1)
8 × 4 (203 × 102)	5.48 (139)
7 × 4 (178 × 102)	4.37 (111)
6 × 4 (152 × 102)	3.14 (79.8)
6 × 3.5 (152 × 89)	3.69 (93.7)
5 × 3.5 (127 × 89)	2.40 (61.0)
5 × 3 (127 × 76)	2.99 (75.9)
4 × 3.5 (102 × 89)	0.87 (22.1)
4 × 3 (102 × 76)	1.65 (41.9)
3.5 × 3 (89 × 76)	0.87 (22.1)
3.5 × 2.5 (89 × 64)	1.62 (41.1)
3 × 2.5 (76 × 64)	0.86 (21.8)
3 × 2 (76 × 51)	1.56 (39.6)
2.5 × 2 (64 × 51)	0.85 (21.6)
Equal legs	0.00

*$\beta_w = \dfrac{1}{I_w} \displaystyle\int_A z\left(w^2 + z^2\right) dA - 2z_o$ where z_o is the coordinate along the z-axis of the shear center with respect to the centroid, and I_w is the moment of inertia for the major principal axis; β_w has positive or negative value depending on direction of bending (see Figure C-F10.4)

stress. The stress distribution and/or the elastic buckling stress must be determined from principles of structural mechanics, text books or handbooks, such as the SSRC Guide (Galambos, 1998), papers in journals, or finite element analyses. Alternatively, the designer can avoid the problem by selecting cross sections from among the many choices given in the previous sections of Chapter F.

F13.   PROPORTIONS OF BEAMS AND GIRDERS

1.   Hole Reductions

Historically, provisions for proportions of rolled beams and girders with holes in the tension flange were based upon either a percentage reduction independent of material strength or a calculated relationship between the tension rupture and tension yield strengths of the flange, with resistance factors or safety factors included in the calculation. In both cases, the provisions were developed based upon tests of steel with a specified minimum yield stress of 36 ksi (248 MPa) or less.

More recent tests (Dexter and Altstadt, 2004; Yuan, Swanson, and Rassati, 2004) indicate that the flexural strength on the net section is better predicted by comparison of the quantities $F_y A_{fg}$ and $F_u A_{fn}$, with slight adjustment when the ratio of F_y to F_u exceeds 0.8. If the holes remove enough material to affect the member strength, the critical stress is adjusted from F_y to $(F_u A_{fn}/A_{fg})$ and this value is conservatively applied to the elastic section modulus S_x.

2.   **Proportioning Limits for I-Shaped Members**

The provisions of this section are taken directly from Appendix G, Section G1 of the 1999 *LRFD Specification* (AISC, 2000b). They have been part of the plate-girder design requirements since 1963; they are derived from Basler and Thurlimann (1963). The web depth-thickness limitations are provided so as to prevent the flange from buckling into the web. Equation F13-4 is slightly modified from the corresponding Equation A-G1-2 in the 1999 Specification to recognize the change in this Specification in the definition of *residual stress* from a flat 16.5 ksi (114 MPa) used previously to 30 percent of the yield stress, as shown by the following derivation,

$$\frac{0.48E}{\sqrt{F_y\left(F_y + 16.5\right)}} \approx \frac{0.48E}{\sqrt{F_y\left(F_y + 0.3F_y\right)}} = \frac{0.42E}{F_y} \qquad \text{(C-F13-1)}$$

CHAPTER G

DESIGN OF MEMBERS FOR SHEAR

G1. GENERAL PROVISIONS

Chapter G applies to webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles and HSS, and shear in the weak direction of singly or doubly symmetric shapes.

Two methods for determining the shear strength of singly or doubly symmetric I-shaped beams and built-up sections are presented. The method of Section G2 does not utilize the post-buckling strength of the web, while the method of Section G3 utilizes the post-buckling strength.

G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

Section G2 deals with the shear strength of webs of wide-flange or I-shaped members, as well as webs of tee-shapes, that are subject to shear and bending in the plane of the web. The provisions in Section G2 apply to the general case when an increase of strength due to tension field action is not permitted. Conservatively, these provisions may be applied also when it is not desired to use the tension field action enhancement for convenience in design. Consideration of the effect of bending on the shear strength is not required because the effect is deemed negligible.

1. Nominal Shear Strength

The nominal shear strength of a web is defined by Equation G2-1, a product of the shear yield force $0.6F_y A_w$ and the shear-buckling reduction factor C_v.

The provisions of case (a) in Section G2.1 for rolled I-shaped members with $h/t_w \leq 2.24\sqrt{E/F_y}$ are similar to previous LRFD provisions, with the exception that ϕ has been increased from 0.90 to 1.00 (with a corresponding decrease of the safety factor from 1.67 to 1.5), thus making these provisions consistent with previous provisions for allowable stress design. The value of ϕ of 0.90 is justified by comparison with experimental test data and recognizes the minor consequences of shear yielding, as compared to those associated with tension and compression yielding, on the overall performance of rolled I-shaped members. This increase is applicable only to the shear yielding limit state of I-shaped members.

Case (b) in Section G2.1 uses the shear buckling reduction factor, C_v, shown in Figure C-G2.1. The curve for C_v has three segments.

For webs with $h/t_w \leq 1.10\sqrt{Ek_v/F_y}$, the nominal shear strength V_n is based on shear yielding of the web, with C_v given by Equation G2-3. This h/t_w limit was determined by setting the critical stress causing shear buckling, F_{cr}, equal to the yield stress of the web, $F_{yw} = F_y$, in Equation 35 of Cooper, Galambos, and Ravindra (1978).

When $h/t_w > 1.10\sqrt{Ek_v/F_y}$, the web shear strength is based on buckling. It has been suggested to take the proportional limit as 80 percent of the yield stress of the web (Basler, 1961). This corresponds to $h/t_w = (1.10/0.8)\left(\sqrt{Ek_v/F_y}\right)$.

When $h/t_w > 1.37\sqrt{Ek_v/F_y}$, the web strength is determined from the elastic buckling stress given by Equation 6 of Cooper and others (1978) and Equation 9-7 in Timoshenko and Gere (1961):

$$F_{cr} = \frac{\pi^2 Ek_v}{12\left(1 - v^2\right)(h/t_w)^2} \qquad \text{(C-G2-1)}$$

C_v in Equation G2-5 was obtained by dividing F_{cr} from Equation C-G2-1 by $0.6F_y A_w$ and using $E = 29{,}000$ ksi (200 000 MPa) and $v = 0.3$.

A straight-line transition for C_v (Equation G2-4) is used between the limits given by $1.10\sqrt{k_v E/F_y} < h/t_w \leq 1.37\sqrt{k_v E/F_y}$.

The plate buckling coefficient, k_v, for panels subject to pure shear having simple supports on all four sides is given by Equation 4.24 in Galambos (1998).

$$k_v = \begin{cases} 4.00 + \dfrac{5.34}{(a/h)^2} & \text{for } a/h \leq 1 \\[2mm] 5.34 + \dfrac{4.00}{(a/h)^2} & \text{for } a/h > 1 \end{cases} \qquad \text{(C-G2-2)}$$
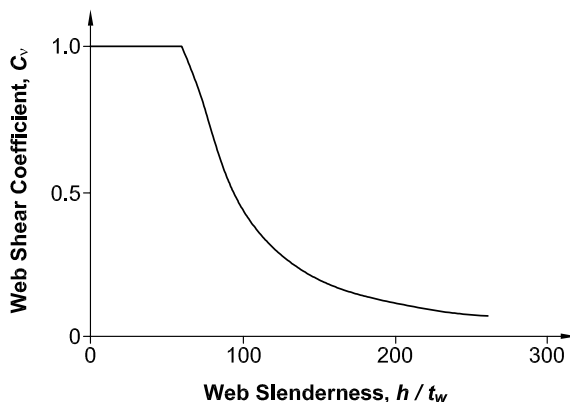

Fig. C-G2.1. Shear buckling coefficient C_v for $F_y = 50$ ksi (345 MPa) and $k_v = 5.0$.

For practical purposes and without loss of accuracy, these equations have been simplified herein and in AASHTO (1998) to

$$k_v = 5 + \frac{5}{(a/h)^2} \qquad \text{(C-G2-3)}$$

When the panel ratio a/h becomes large, as in the case of webs without transverse stiffeners, then $k_v = 5$. Equation C-G2-3 applies as long as there are flanges on both edges of the web. For tee-shaped beams the free edge is unrestrained and for this situation $k_v = 1.2$ (JCRC, 1971).

The provisions of Section G2.1 assume monotonically increasing loads. If a flexural member is subjected to load reversals causing cyclic yielding over large portions of a web, such as may occur during a major earthquake, special design considerations may apply (Popov, 1980).

2. Transverse Stiffeners

When transverse stiffeners are needed, they must be rigid enough to cause a buckling node to form along the line of the stiffener. This requirement applies whether or not tension field action is counted upon. The required moment of inertia of the stiffener is the same as in AASHTO (1996), but it is different from the formula $I_{st} \geq (h/50)^4$ in the 1989 *ASD Specification* (AISC, 1989). Equation G2-5 is derived in Chapter 11 of Salmon and Johnson (1996). The origin of the formula can be traced to Bleich (1952).

G3. TENSION FIELD ACTION

The provisions of Section G3 apply when it is intended to account for the enhanced strength of webs of built-up members due to tension field action.

1. Limits on the Use of Tension Field Action

The panels of the web of a built-up member, bounded on top and bottom by the flanges and on each side by the transverse stiffeners, are capable of carrying loads far in excess of their "web buckling" load. Upon reaching the theoretical web buckling limit, very slight lateral web displacements will have developed. These deformations are of no structural significance, because other means are still present to provide further strength.

When transverse stiffeners are properly spaced and are strong enough to act as compression struts, membrane stresses due to shear forces greater than those associated with the theoretical web buckling load form diagonal tension fields in the web panels. The resulting combination in effect provides a Pratt truss that furnishes the strength to resist applied shear forces unaccounted for by the linear buckling theory.

The key point in the development of tension field action in the web of plate girders is the ability of the stiffeners to support the compression from the two panels on either side of the stiffener. In the case of end panels there is a panel

only on one side. The support of the tension field forces is also reduced when the panel aspect ratio becomes too large. For this reason the inclusion of the tension field enhancement is not permitted for end panels and when *a/h* exceeds 3.0 or $\left[\dfrac{260}{(h/t_w)}\right]^2$.

2. Nominal Shear Strength with Tension Field Action

Analytical methods based on tension field action have been developed (Basler and Thurlimann, 1963; Basler, 1961) and corroborated in an extensive program of tests (Basler, Yen, Mueller, and Thurlimann, 1960). Equation G3-2 is based on this research. The second term in the bracket represents the relative increase of the panel shear strength due to tension field action.

3. Transverse Stiffeners

The vertical component of the tension field force that is developed in the web panel must be resisted by the transverse stiffener. In addition to the rigidity required to keep the line of the stiffener as a nonmoving point for the buckled panel, as provided for in Section G2.2, the stiffener must also have a large enough area to resist the tension field reaction. Equation G3-3 often controls the design of the stiffeners.

G4. SINGLE ANGLES

Shear stresses in single-angle members are the result of the gradient of the bending moment along the length (flexural shear) and the torsional moment.

The maximum elastic stress due to flexural shear is

$$f_v = \frac{1.5V_b}{bt} \tag{C-G4-1}$$

where V_b is the component of the shear force parallel to the angle leg with width b and thickness t. The stress is constant throughout the thickness, and it should be calculated for both legs to determine the maximum. The coefficient 1.5 is the calculated value for equal leg angles loaded along one of the principal axes. For equal leg angles loaded along one of the geometric axes, this factor is 1.35. Factors between these limits may be calculated conservatively from $V_b Q/It$ to determine the maximum stress at the neutral axis. Alternatively, if only flexural shear is considered, a uniform flexural shear stress in the leg of V_b/bt may be used due to inelastic material behavior and stress redistribution.

If the angle is not laterally braced against twist, a torsional moment is produced equal to the applied transverse load times the perpendicular distance e to the shear center, which is at the point of intersection of the centerlines of the two legs. Torsional moments are resisted by two types of shear behavior: pure torsion (*St. Venant torsion*) and *warping torsion* [see Seaburg and Carter (1997)]. The shear stresses due to restrained warping are small compared to the *St. Venant torsion* (typically less than 20 percent) and they can be neglected for practical purposes. The applied torsional moment is then resisted by pure shear stresses that are

constant along the width of the leg (except for localized regions at the toe of the leg), and the maximum value can be approximated by

$$f_v = \frac{M_T t}{J} = \frac{3M_T}{At} \qquad \text{(C-G4-2)}$$

where

J = torsional constant (approximated by $\Sigma(bt^3/3)$ when precomputed value is unavailable)

A = angle cross-sectional area

For a study of the effects of warping, see Gjelsvik (1981). Torsional moments from laterally unrestrained transverse loads also produce warping normal stresses that are superimposed on the bending stresses. However, since the warping strength of single angles is relatively small, this additional bending effect, just like the warping shear effect, can be neglected for practical purposes.

G5.   RECTANGULAR HSS AND BOX MEMBERS

The two webs of a closed-section rectangular cross section resist shear the same way as the single web of an I-shaped plate girder or wide-flange beam, and therefore, the provisions of Section G2 apply.

G6.   ROUND HSS

Little information is available on round HSS subjected to transverse shear and the recommendations are based on provisions for local buckling of cylinders due to torsion. However, since torsion is generally constant along the member length and transverse shear usually has a gradient; it is recommended to take the critical stress for transverse shear as 1.3 times the critical stress for torsion (Brockenbrough and Johnston, 1981; Galambos, 1998). The torsion equations apply over the full length of the member, but for transverse shear it is reasonable to use the length between the points of maximum and zero shear force. Only thin HSS may require a reduction in the shear strength based upon first shear yield. Even in this case, shear will only govern the design of round HSS for the case of thin sections with short spans.

In the equation for the nominal shear strength, V_n, of round HSS, it is assumed that the shear stress at the neutral axis, calculated as VQ/Ib, is at F_{cr}. For a thin round section with radius R and thickness t, $I = \pi R^3 t$, $Q = 2R^2 t$ and $b = 2t$. This gives the stress at the centroid as $V/\pi Rt$, in which the denominator is recognized as half the area of the round HSS.

G7.   WEAK AXIS SHEAR IN SINGLY AND DOUBLY SYMMETRIC SHAPES

The nominal shear strength of singly and doubly symmetric I-shapes is governed by the equations of Section G2 with the plate buckling coefficient equal to $k_v = 1.2$, the same as the web of a tee-shape. The maximum plate slenderness of all rolled shapes is $(b_f/2t_f) = 13.8$, and for $F_y = 100$ ksi (690 MPa) the value

of $1.10\sqrt{\dfrac{k_v E}{F_y}} = 1.10\sqrt{\dfrac{1.2 \times 29000}{100}} = 20.5$. Thus $C_v = 1.0$ except for built-up shapes with very slender flanges.

G8. BEAMS AND GIRDERS WITH WEB OPENINGS

Web openings in structural floor members may be used to accommodate various mechanical, electrical and other systems. Strength limit states, including local buckling of the compression flange or of the web, local buckling or yielding of the tee-shaped compression zone above or below the opening, lateral buckling and moment-shear interaction, or serviceability may control the design of a flexural member with web openings. The location, size and number of openings are important and empirical limits for them have been identified. One general procedure for assessing these effects and the design of any needed reinforcement for both steel and composite beams is given in the ASCE *Specification for Structural Steel Beams with Web Openings* (ASCE, 1999), with background information provided in Darwin (1990) and in ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete (1992) and ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete (1992a).

CHAPTER H

DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION

Chapters D, E, F and G of this Specification address members subject to only one type of force: axial tension, axial compression, flexure and shear, respectively. Chapter H addresses members subject to a combination of two or more of the individual forces defined above, as well as possibly by additional forces due to torsion. The provisions fall into two categories: (a) the majority of the cases that can be handled by an interaction equation involving sums of ratios of required strengths to the available strengths; and (b) cases where the stresses due to the applied forces are added and compared to limiting buckling or yield stresses. Designers will have to consult the provisions of Sections H2 and H3 only in rarely occurring cases.

H1. DOUBLY AND SINGLY SYMMETRIC MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

1. Doubly and Singly Symmetric Members in Flexure and Compression

Section H1 contains design provisions for prismatic members under combined flexure and compression and under combined flexure and tension for doubly and singly symmetric sections. The provisions of Section H1 apply typically to rolled wide-flange shapes, channels, tee-shapes, round, square and rectangular HSS, solid rounds, squares, rectangles or diamonds, and any of the many possible combinations of doubly or singly symmetric shapes fabricated from plates and/or shapes by welding or bolting. The interaction equations accommodate flexure about one or both principal axes as well as axial compression or tension.

In 1923, the first AISC Specification required that the stresses due to flexure and compression be added and that the sum not exceed the allowable value. An interaction equation appeared first in the 1936 Specification, stating "Members subject to both axial and bending stresses shall be so proportioned that the quantity $\frac{f_a}{F_a} + \frac{f_b}{F_b}$ shall not exceed unity," in which F_a and F_b are, respectively, the axial and flexural allowable stresses permitted by this Specification, and f_a and f_b are the corresponding stresses due to the axial force and the bending moment, respectively. This linear interaction equation was in force until the 1961 Specification, when it was modified to account for frame stability and for the $P\text{-}\delta$ effect, that is, the secondary bending between the ends of the members (Equation C-H1-1). The $P\text{-}\Delta$ effect, that is, the second-order bending moment due to story sway, was not accommodated.

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \dfrac{f_a}{F_e'}\right) F_b} \leq 1.0 \qquad \text{(C-H1-1)}$$

The allowable axial stress F_a is determined for an effective length that is larger than unity for moment frames. The term $\dfrac{1}{1 - \dfrac{f_a}{F_e'}}$ is the amplification of the interspan moment due to member deflection multiplied by the axial force (the P-δ effect). C_m accounts for the effect of the moment gradient. This interaction equation has been part of all the subsequent editions of the AISC ASD Specifications since 1961.

A new approach to the interaction of flexural and axial forces was introduced in the 1986 AISC *LRFD Specification* (AISC, 1986). The following is an explanation of the thinking behind the interaction curves used. The equations

$$\frac{P}{P_y} + \frac{8}{9}\frac{M_{pc}}{M_p} = 1 \quad \text{for } \frac{P_u}{P_y} \geq 0.2$$

$$\frac{P}{2P_y} + \frac{M_{pc}}{M_p} = 1 \quad \text{for } \frac{P_u}{P_y} < 0.2 \qquad \text{(C-H1-2)}$$

define the lower-bound curve for the interaction of the nondimensional axial strength P/P_y and flexural strength M/M_p for compact wide-flange stub-columns bent about their x-axis. The cross section is assumed to be fully yielded in tension and compression. The symbol M_{pc} is the plastic moment strength of the cross section in the presence of an axial force P. The curve representing Equation C-H1-2 almost overlaps the analytically exact curve for the major-axis bending of a W8×31 (W200×46.1) cross section (see Figure C-H1.1). The equations for the exact yield capacity of a wide-flange shape are (ASCE, 1971):

$$\text{for } 0 \leq \frac{P}{P_y} \leq \frac{t_w\left(d - 2t_f\right)}{A}$$

$$\frac{M_{pc}}{M_p} = 1 - \frac{A^2\left(\dfrac{P}{P_y}\right)^2}{4t_w Z_x}$$

$$\text{for } \frac{t_w\left(d - 2t_f\right)}{A} \leq \frac{P}{P_y} \leq 1 \qquad \text{(C-H1-3)}$$

$$\frac{M_{pc}}{M_p} = \frac{A\left(1 - \dfrac{P}{P_y}\right)}{2Z_x}\left[d - \frac{A\left(1 - \dfrac{P}{P_y}\right)}{2b_f}\right]$$

The equation approximating the average yield strength of wide-flange shapes is

$$\frac{M_{pc}}{M_p} = 1.18\left(1 - \frac{P}{P_y}\right) \leq 1 \qquad \text{(C-H1-4)}$$

The curves in Figure C-H1.2 show the exact and approximate yield interaction curves for wide-flange shapes bent about the y-axis, and the exact curves for

the solid rectangular and round shapes. It is evident that the lower-bound AISC interaction curves are very conservative for these shapes.

The idea of portraying the strength of stub beam-columns was extended to actual beam-columns with actual lengths by normalizing the required flexural strength, M_u, of the beam by the nominal strength of a beam without axial force, M_n, and the required axial strength, P_u, by the nominal strength of a column without bending moment, P_n. This rearrangement results in a translation and rotation of the original stub-column interaction curve, as seen in Figure C-H1.3.
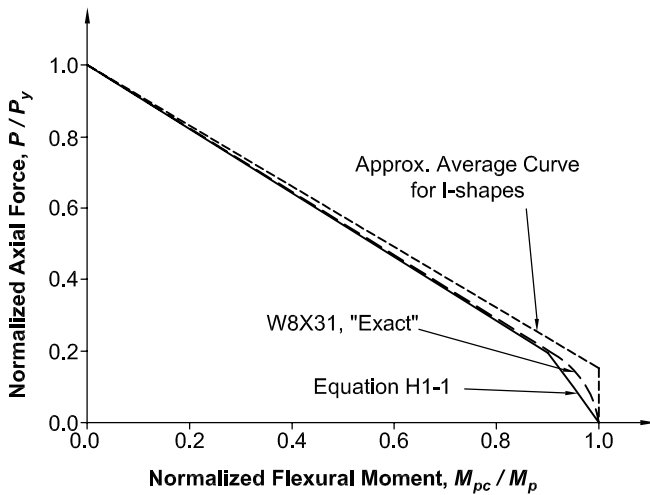

Fig. C-H1.1. Stub-column interaction curves: plastic moment versus axial force for wide-flange shapes, major-axis flexure [$W8\times31$ ($W200\times46.1$), $F_y = 50$ ksi (345 MPa)].
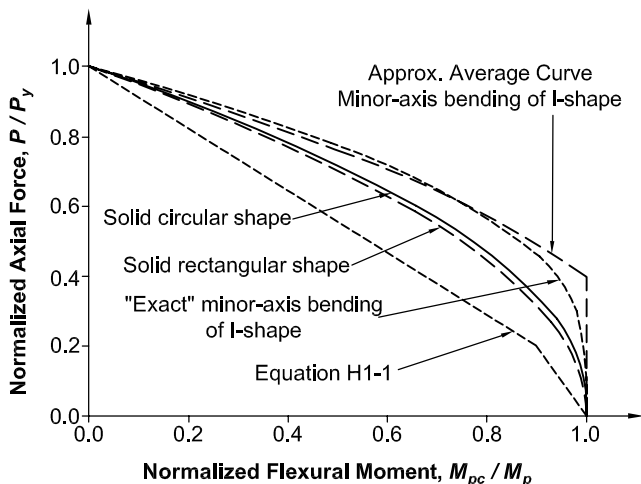

Fig. C-H1.2. Stub-column interaction curves: plastic moment versus axial force for solid round and rectangular sections and for wide-flange shapes, minor-axis flexure.

The normalized equations corresponding to the beam-column with length effects included are shown as Equation C-H1-5:

$$\frac{P_u}{P_n} + \frac{8}{9}\frac{M_u}{M_n} = 1 \text{ for } \frac{P_u}{P_n} \geq 0.2$$

$$\frac{P_u}{2P_n} + \frac{M_u}{M_n} = 1 \text{ for } \frac{P_u}{P_n} < 0.2 \qquad \text{(C-H1-5)}$$

The interaction equations are designed to be very versatile. The terms in the denominator fix the endpoints of the interaction curve. The nominal flexural strength, M_n, is determined by the appropriate provisions from Chapter F. It encompasses the limit states of yielding, lateral-torsional buckling, flange local buckling and web local buckling.

The axial term, P_n, is governed by the provisions of Chapter E, and it can accommodate compact or slender columns, as well as the limit states of major and minor axis buckling, and torsional and flexural-torsional buckling. Furthermore, P_n is calculated for the applicable effective length of the column to take care of frame stability effects, if the procedures of Section C.2-1a and Section C.2-1b are used to determine the required moments and axial forces. These moments and axial forces include the amplification due to second-order effects.

The utility of the interaction equations is further enhanced by the fact that they also permit the consideration of *biaxial bending*.

2. Doubly and Singly Symmetric Members in Flexure and Tension

Section H1.1 considers the most frequently occurring cases in design: members under flexure and axial compression. Section H1.2 addresses the less frequent cases of flexure and axial tension. Since axial tension increases the bending
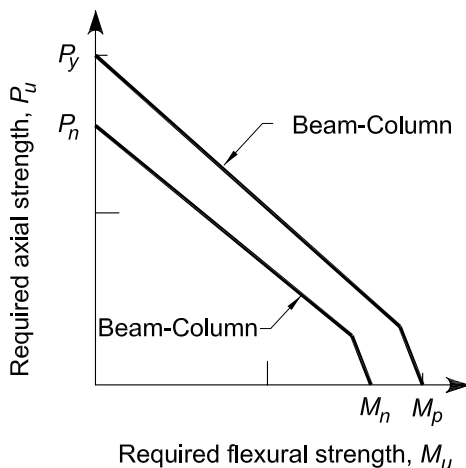

Fig. C-H1.3. Interaction curve for stub beam-column and beam-column.

stiffness of the member to some extent, Section H1.2 permits the increase of the bending terms in the interaction equations in proportion to $\sqrt{1 + \dfrac{P_u}{P_{ey}}}$.

3.   Doubly Symmetric Members in Single Axis Flexure and Compression

The linear interaction Equation C-H1-5 is conservative for cases where the axial limit state is out-of-plane buckling and the flexural limit state is lateral-torsional buckling for doubly symmetric wide-flange sections with moment applied about the x-axis (Galambos, 1998). Section H1.3 gives an optional equation for such beam-columns.

The two curves in Figure C-H1.4 illustrate the difference between the bi-linear and the parabolic interaction equations for the case of a W27×84 (W690×125) beam-column.

The relationship between Equations H1-1 and H1-2 is further illustrated in Figures C-H1.5 (for LRFD) and C-H1.6 (for ASD). The curves relate the required axial force, P (ordinate), and the required bending moment, M (abscissa), when the interaction Equations H1-1 and H1-2 are equal to unity. The positive values of P are compression and the negative values are tension. The curves are for a 10 ft (3 m) long W16×26 [$F_y = 50$ ksi (345 MPa)] member. The solid curve is for in-plane behavior, that is, lateral bracing prevents lateral-torsional buckling. The dotted curve represents Equation H1-1 for the case when there are no lateral braces between the ends of the beam-column. In the region of the tensile axial force, the curve is modified by the term $\sqrt{1 + \dfrac{P}{P_y}}$, as permitted in Section H1.2. The dashed
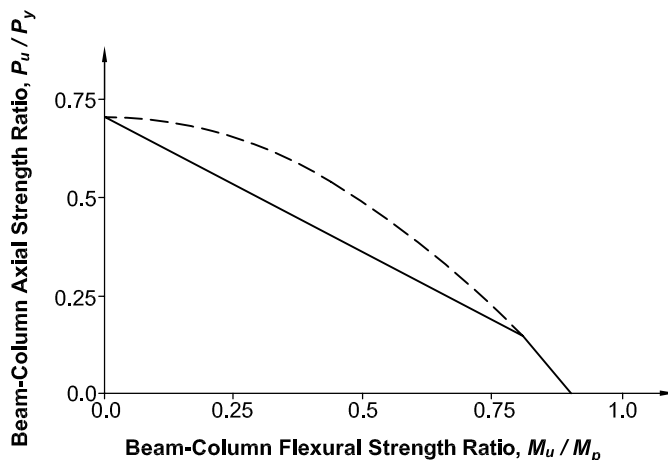

Fig. C-H1.4. Comparison between bi-linear (Equation H1-1) and parabolic (Equation H1-2) interaction equations [W27×84 (W690×125), F_y = 50 ksi (345 MPa), L_b = 10 ft (3.05 m), C_b = 1.75].

curve is Equation H1-2. For a given compressive or tensile axial force, the latter equation allows a larger bending moment over most of its domain of applicability.

H2. UNSYMMETRIC AND OTHER MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

The provisions of Section H1 apply to beam-columns with cross sections that are either doubly or singly symmetric. However, there are many cross sections that are unsymmetrical, such as unequal leg angles and any number of possible fabricated sections. For these situations the interaction equation of Section H1 may not be appropriate. The linear interaction $\dfrac{f_a}{F_a} + \dfrac{f_{bw}}{F_{bw}} + \dfrac{f_{bz}}{F_{bz}} \leq 1.0$ provides a conservative and simple way to deal with such problems. The lower case stresses f are the required axial and flexural stresses computed by elastic analysis for the applicable loads, including second-order effects where appropriate, and the upper case stresses F are the available stresses corresponding to the limit state of yielding or buckling. The subscripts w and z refer to the principal axes of the unsymmetric cross section. This Specification leaves the option to the designer to use the Section H2 interaction equation for cross sections that would qualify for the more liberal interaction equation of Section H1.
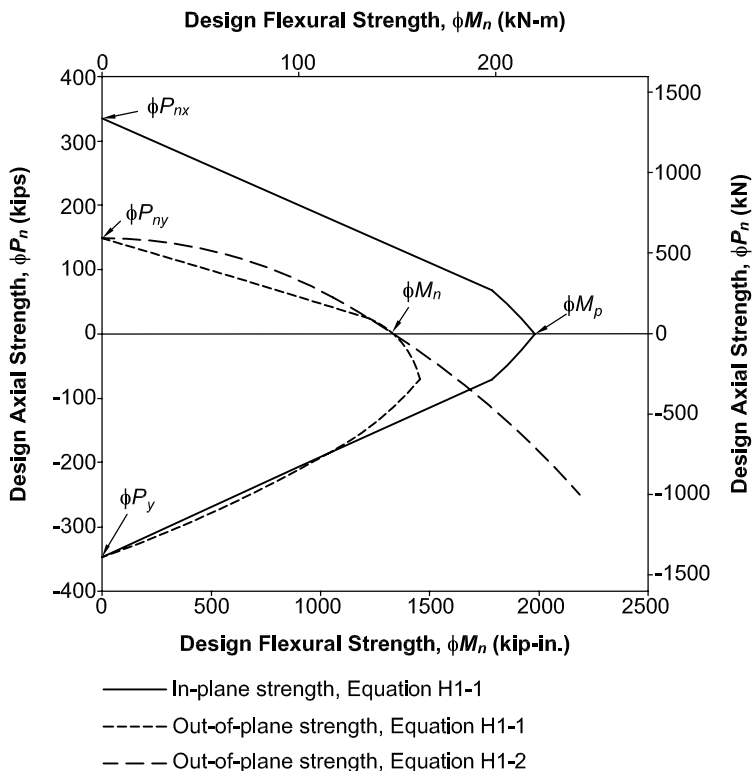

Fig. C-H1.5. *Beam-columns under compressive and tensile axial force (tension is shown as negative) (LRFD) [W16×26 (W410×38.8), $F_y = 50$ ksi (345 MPa), $L_b = 10$ ft (3.05 m)].*

The interaction equation, Equation H2-1, applies equally to the case where the axial force is in tension.

H3.   MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR AND/OR AXIAL FORCE

Section H3 provides provisions for cases not covered in the previous two sections. The first two parts of this section address the design of HSS members, and the third part is a general provision directed to cases where the designer encounters torsion in addition to normal stresses and shear stresses.

1.   Torsional Strength of Round and Rectangular HSS

Hollow structural sections (HSS) are frequently used in space-frame construction and in other situations wherein significant torsional moments must be resisted by the members. Because of its closed cross section, an HSS is far more efficient in resisting torsion than an open cross section such as a W-shape or a channel. While normal and shear stresses due to restrained warping are usually significant in shapes of open cross section, they are insignificant in closed cross sections. The total torsional moment can be assumed to be resisted by pure torsional shear stresses. These are often referred in the literature as *St. Venant torsional* stresses.
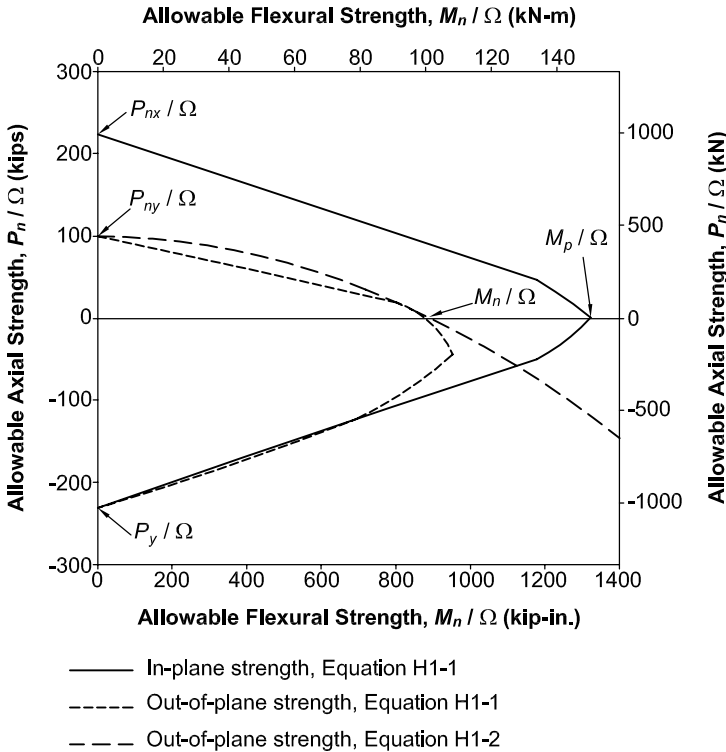

Fig. C-H1.6. Beam-columns under compressive and tensile axial force (tension is shown as negative) (ASD) [W16×26 (W410×38.8), $F_y = 50$ ksi (345 MPa), $L_b = 10$ ft (3.05 m)].

The pure torsional shear stress in HSS sections is assumed to be uniformly distributed along the wall of the cross section, and it is equal to the torsional moment, T_u, divided by a torsional shear constant for the cross section, C. In a limit state format, the nominal torsional resisting moment is the shear constant times the critical shear stress, F_{cr}.

For round HSS, the torsional shear constant is equal to the polar moment of inertia divided by the radius, which leads to

$$C = \frac{\pi t (D - t)^2}{2} \qquad \text{(C-H3-1)}$$

For rectangular HSS, the torsional shear constant is obtained as $2t A_o$ using the membrane analogy (Timoshenko, 1956), where A_o is the area bounded by the midline of the section. Conservatively assuming an outside corner radius of $2t$, the midline radius is $1.5t$ and

$$A_o = t^2 (B - t)(H - t) \frac{9(4 - \pi)}{4} \qquad \text{(C-H3-2)}$$

resulting in

$$C = 2t (B - t)(H - t) - 4.5t^3 (4 - \pi) \qquad \text{(C-H3-3)}$$

The resistance factor ϕ and the safety factor Ω are the same as for flexural shear in Chapter G.

When considering local buckling in round HSS subjected to torsion, most structural members will either be long or of moderate length and the provisions for short cylinders will not apply. The elastic local buckling strength of long cylinders is unaffected by end conditions and the critical stress is given in Galambos (1998) as

$$F_{cr} = \frac{K_t E}{\left(\dfrac{D}{t} \right)^{\frac{3}{2}}} \qquad \text{(C-H3-4)}$$

The theoretical value of K_t is 0.73 but a value of 0.6 is recommended to account for initial imperfections. An equation for the elastic local buckling stress for round HSS of moderate length ($L > 5.1D^2/t$) where the edges are not fixed at the ends against rotation is given in Schilling (1965) and Galambos (1998) as

$$F_{cr} = \frac{1.23E}{\left(\dfrac{D}{t} \right)^{\frac{5}{4}} \sqrt{\dfrac{L}{D}}} \qquad \text{(C-H3-5)}$$

This equation includes a 15 percent reduction to account for initial imperfections. The length effect is included in this equation for simple end conditions, and the approximately 10 percent increase in buckling strength is neglected for edges fixed at the end. A limitation is provided so that the shear yield strength $0.6F_y$ is not exceeded.

The critical stress provisions for rectangular HSS are identical to the flexural shear provisions of Section G2 with the shear buckling coefficient equal to $k_v = 5.0$. The shear distribution due to torsion is uniform in the longest sides of a rectangular HSS, and this is the same distribution that is assumed to exist in the web of a W-shape beam. Therefore, it is reasonable that the provisions for buckling are the same in both cases.

2. HSS Subject to Combined Torsion, Shear, Flexure and Axial Force

Several interaction equation forms have been proposed in the literature for load combinations that produce both normal and shear stresses. In one common form, the normal and shear stresses are combined elliptically with the sum of the squares (Felton and Dobbs, 1967):

$$\left(\frac{f}{F_{cr}}\right)^2 + \left(\frac{f_v}{F_{vcr}}\right)^2 \leq 1 \qquad \text{(C-H3-6)}$$

In a second form, the first power of the ratio of the normal stresses is used:

$$\left(\frac{f}{F_{cr}}\right) + \left(\frac{f_v}{F_{vcr}}\right)^2 \leq 1 \qquad \text{(C-H3-7)}$$

The latter form is somewhat more conservative, but not overly so (Schilling, 1965), and this is the form used in this Specification:

$$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \leq 1.0 \qquad \text{(C-H3-8)}$$

where the terms with the subscript r represent the required strengths, and the ones with the subscript c are the corresponding available strengths. Normal effects due to flexural and axial load effects are combined linearly and then combined with the square of the linear combination of flexural and torsional shear effects. When an axial compressive load effect is present, the required flexural strength, M_c, is to be determined by second-order analysis.

3. Strength of Non-HSS Members under Torsion and Combined Stress

This section covers all the cases not previously covered. Examples are built-up unsymmetric crane-girders and many other types of odd-shaped built-up cross sections. The required stresses are determined by elastic stress analysis based on established theories of structural mechanics. The three limit states to consider and the corresponding available stresses are:

(1) Yielding under normal stress—F_y
(2) Yielding under shear stress—$0.6F_y$
(3) Buckling—F_{cr}

In most cases it is sufficient to consider normal stresses and shear stresses separately because maximum values rarely occur in the same place in the cross section or at the same place in the span. Seaburg and Carter (1997) provides a complete discussion on torsional analysis of open shapes.

CHAPTER I

DESIGN OF COMPOSITE MEMBERS

Chapter I includes extensive technical and format changes as well as significant new material when compared to previous editions of the Specification. The major technical changes consist of new design provisions for composite columns (Section I2), which now include new cross-sectional strength models, provisions for tension and shear design, and a liberalization of the slenderness limits for HSS. Other significant technical changes have been made in the shear stud strength provisions (Section I3.2d): the use of an ultimate strength model for ASD design of composite beams (Section I3.2) and new material limitations (Section I1.2).

The main format changes in Chapter I include the elimination of the former Section I1, Design Assumptions and Definitions. The contents of that Section have been moved to the Glossary, the notation section, or other locations in the Specification and the section has been replaced by a section on General Provisions. Other format changes are as follows: the separation of composite column design into distinct provisions for concrete-encased sections and concrete-filled sections; and the incorporation of the former Section I5, Shear Connectors, into the current Section I3. In addition, the extensive historical notes on the development of composite design provisions present in the Commentary of the previous editions of the Specification have been eliminated as that material is now considered to be widely known.

I1.   GENERAL PROVISIONS

Design of composite sections requires consideration of both steel and concrete behavior. These provisions were developed with the intent both to minimize conflicts between current steel and concrete design provisions and to give proper recognition to the advantages of composite design. As a result of the attempt to minimize conflicts, this Specification now uses a cross-sectional strength approach for column design consistent with that used in reinforced concrete design (ACI, 2002). This approach, in addition, results in a consistent treatment of cross-sectional strengths for both composite columns and beams.

This Specification assumes that the user is familiar with reinforced concrete design specifications such as ACI (2002) and does not repeat many of the provisions needed for the concrete portion of the design, such as material specifications, anchorage and splice lengths, and shear and torsion provisions.

The provisions in Chapter I address strength design of the composite sections only. The designer needs to consider the loads resisted by the steel section alone when determining load effects during the construction phase. The designer also needs to consider deformations throughout the life of the structure and the appropriate

cross section for those deformations. When considering these latter limit states, due allowance should be made for the additional long-term changes in stresses and deformations due to creep and shrinkage of the concrete.

1.   Nominal Strength of Composite Sections

The strength of composite sections shall be computed based on either of the two approaches presented in this Specification. The first is the strain compatibility approach, which provides a general calculation method. The second is the plastic stress distribution approach, which is a subset of the strain compatibility approach. The plastic stress distribution method provides a simple and convenient calculation method for the most common design situations, and is thus treated first.

1a.   Plastic Stress Distribution Method

The plastic stress distribution method is based on the assumption of linear strain across the cross section and elasto-plastic behavior. It assumes that the concrete has reached its crushing strength in compression at a strain of 0.003 and a corresponding stress (typically $0.85 f_c'$) on a rectangular stress block, and that the steel has exceeded its yield strain, typically taken as F_y/E_s.

Based on these simple assumptions, the cross-sectional strength for different combinations of axial force and bending moment may be approximated, for typical composite column cross-sections. The actual interaction diagram for moment and axial force for a composite section based on a plastic stress distribution is similar to that of a reinforced concrete section as shown in Figure C-I1.1. As a simplification, for concrete-encased sections, a conservative linear interaction between four or five anchor points, depending on axis of bending, can be used (Roik and Bergmann, 1992; Galambos, 1998). These points are identified as A, B, C, D and E in Figure C-I1.1.

The plastic stress approach for columns assumes that no slip has occurred between the steel and concrete portions and that the required width-to-thickness ratios prevent local buckling from occurring until extensive yielding has taken place. Tests and analyses have shown that these are reasonable assumptions at the ultimate limit states for both concrete-encased steel sections with shear connectors and for
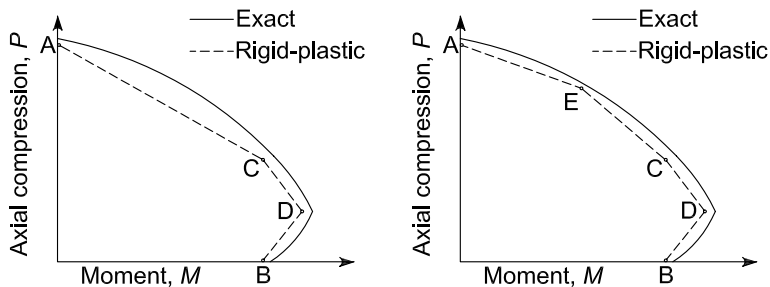

Fig. C-I1.1. Comparison between exact and simplified moment-axial compressive force envelopes.

HSS sections that comply with these provisions (Galambos, 1998; Hajjar, 2000; Shanmugam and Lakshmi, 2001). For circular HSS, these provisions allow for the increase of the usable concrete stress to $0.95 f'_c$ to account for the beneficial effects of the restraining hoop action arising from transverse confinement (Leon and Aho, 2002).

Based on similar assumptions, but allowing for slip between the steel beam and the composite slab, simplified expressions can also be derived for typical composite beam sections. Strictly speaking, these distributions are not based on slip, but on the strength of the shear connection. Full interaction is assumed if the shear connection strength exceeds that of either (a) the tensile yield strength of the steel section or the compressive strength of the concrete slab when the composite beam is loaded in positive moment, or (b) the tensile yield strength of the longitudinal reinforcing bars in the slab or the compressive strength of the steel section when loaded in negative moment. When shear connectors are provided in sufficient numbers to fully develop this flexural strength, any slip that occurs prior to yielding has a negligible affect on behavior. When full interaction is not present, the beam is said to be partially composite. The effects of slip on the elastic properties of a partially composite beam can be significant and should be accounted for, if significant, in calculations of deflections and stresses at service loads. Approximate elastic properties of partially composite beams are given in Commentary Section I3.

1b.   Strain-Compatibility Approach

The principles used to calculate cross-sectional strength in Section I1.1a may not be applicable to all design situations or possible cross-sections. As an alternative, Section I1.1b permits the use of a generalized strain-compatibility approach that allows the use of any reasonable strain-stress model for the steel and concrete.

2.   Material Limitations

The material limitations given in Section I1.2 reflect the range of material properties available from experimental testing (Galambos, 1998; Hajjar, 2000; Shanmugam and Lakshmi, 2001; Leon and Aho, 2002). As for reinforced concrete design, a limit of 10 ksi (70 MPa) is imposed for strength calculations, both to reflect the scant data available above this strength and the changes in behavior observed, particularly for brittle failure modes such as shear. A lower limit of 3 ksi (21 MPa) is specified for both normal and lightweight concrete and an upper limit of 6 ksi (42 MPa) is specified for lightweight concrete to encourage the use of good quality, yet readily available, grades of structural concrete. The use of higher strengths in computing the modulus of elasticity is permitted, and the limits given can be extended for strength calculations if appropriate testing and analyses are carried out.

3.   Shear Connectors

This section provides basic shear connector details and material specifications. Nominal yield and tensile strengths of typical ASTM A108 Type B studs are 51 ksi (350 MPa) and 65 ksi (450 MPa), respectively (AWS 2004).

I2. AXIAL MEMBERS

In Section I2, the design of concrete-encased and concrete-filled composite columns is treated separately, although they have much in common. The intent is to facilitate design by keeping the general principles and detailing requirements for each type of column separate.

An ultimate strength cross-section model is used to determine the section strength (Leon and Aho, 2002). This model is similar to that used in previous LRFD Specifications. The major difference is that the full strength of the reinforcing steel and concrete are accounted for rather than the 70 percent that was used in those previous specifications. In addition, these provisions give the strength of the composite section as a force, while the previous approach had converted that force to an equivalent stress. Since the reinforcing steel and concrete had been arbitrarily discounted, the previous provisions did not accurately predict strength for columns with a low percentage of steel.

The design for length effects is consistent with that for steel columns. The equations used are the same as those in Chapter E, albeit in a slightly different format, and as the percent of concrete in the section decreases, the design defaults to that of a steel section. Comparisons between the provisions in the Specification and experimental data show that the method is generally conservative but that the coefficient of variation obtained is large (Leon and Aho, 2002).

1. Encased Composite Columns

1a. Limitations

(1) In this Specification, the use of composite columns is extended from the previous minimum steel ratio of 4 percent (area of steel shape divided by the gross area of the member) down to columns with a minimum of 1 percent. This is a direct result of using an ultimate strength cross-sectional approach, and removes the previous discontinuities in design that occurred as the steel ratio decreased below 4 percent.

(2) The specified minimum quantity for transverse reinforcement is intended to provide good confinement to the concrete.

(3) A minimum amount of longitudinal reinforcing steel is prescribed so that at least four continuous corner bars are used (see Section I2.1f). Other longitudinal bars may be needed to provide the required restraint to the cross-ties, but that longitudinal steel cannot be counted towards the cross-sectional strength unless it is continuous and properly anchored. It is expected that the limit will seldom be reached in practice, except for the case of a very large cross section.

1b. Compressive Strength

The compressive strength of the cross section is given as the sum of the ultimate strengths of the components. The strength is not capped as in reinforced concrete column design for a combination of the following reasons: (1) the resistance factor has been lowered from 0.85 in previous editions to 0.75 in this Specification; (2) the

required transverse steel provides better performance than a typical reinforced concrete column; (3) the presence of a steel section near the center of the section reduces the possibility of a sudden failure due to buckling of the longitudinal reinforcing steel; and (4) in most cases there will be significant load eccentricities (in other words, moments) present due to the size of the member and the typical force introduction mechanisms.

1c.   Tensile Strength

The new Section I2.1c has been added to clarify the tensile strength to be used in situations where uplift is a concern and for computations related to beam-column interaction. The provision focuses on the limit state of yield on gross area. Where appropriate for the structural configuration, consideration should also be given to other tensile strength and connection strength limit states as specified in Chapters D and J.

1d.   Shear Strength

This new material has been added to provide guidance for the shear strength of composite columns. The provisions require either the use of the steel section alone plus the contribution from any transverse shear reinforcement present in the form of ties or the shear strength calculated based on the reinforced concrete portion of the cross-section alone (in other words, longitudinal and transverse reinforcing bars plus concrete). This implies the following shear strengths:

$$V_n = 0.6 F_y A_w + A_{st} F_{yr} \frac{d}{s}$$
$$\phi = 0.9 \, \text{(LRFD)} \qquad \Omega = 1.67 \, \text{(ASD)}$$

or

$$V_n = 2\sqrt{f'_c}\, bd + A_{st} F_{yr} \frac{d}{s}$$
$$\phi = 0.75 \, \text{(LRFD)} \qquad \Omega = 2.00 \, \text{(ASD)}$$

It would be logical to suggest provisions where both the contributions of the steel section and the reinforced concrete are superimposed; however, there is little research available on this topic.

1e.   Load Transfer

To avoid overstressing either the structural steel section or the concrete at connections in encased composite columns, a transfer of load by direct bearing, shear connection, or a combination of both is required. Although it is recognized that force transfer also occurs by direct bond interaction between the steel and concrete, this is typically ignored for encased composite columns (Griffis, 1992).

When shear connectors are used in encased composite columns, a uniform spacing is appropriate in most situations, but when large forces are applied, other connector arrangements may be needed to avoid overloading the component (steel section or concrete encasement) to which the load is applied directly.

When a supporting concrete area is wider on all sides than the loaded area, the nominal bearing strength for concrete may be taken as

$$N_b = 0.85 f'_c \sqrt{A_2/A_1} \qquad \text{(C-I2-1)}$$

where A_1 is the loaded area and A_2 is the maximum area of the supporting surface that is geometrically similar and concentric with the loaded area. The value of $\sqrt{A_2/A_1}$ must be less than or equal to 2. This Specification uses the maximum nominal bearing strength of $1.7 f'_c A_B$. The resistance factor for bearing, ϕ_B, is 0.65 (and the associated safety factor Ω_B is 2.31) in accordance with ACI (2002).

2.   Filled Composite Columns

2a.   Limitations

(1) As discussed for encased columns, it is now permissible to design composite columns with a steel ratio as low as 1 percent.

(2) The specified minimum wall slenderness has been liberalized from previous editions of the *LRFD Specification*. Those editions did not differentiate between buckling of an unfilled and a filled HSS. The new provisions take into account the restraining effect of the concrete on the local buckling of the section wall.

2b.   Compressive Strength

The compressive strength of the cross section is given as the sum of the ultimate strengths of the components. The beneficial confining effect of a circular HSS can be taken into account by increasing the crushing strength of the concrete to $0.95 f'_c$.

2c.   Tensile Strength

As for encased columns, this new Section I2.2c has been added to clarify tensile strength.

2d.   Shear Strength

See commentary to Section I2.1d.

2e.   Load Transfer

To avoid overstressing either the structural steel section or the concrete at connections in filled composite columns, a transfer of load by direct bearing, shear connection, or direct bond interaction is permitted, with the mechanism providing the largest resistance being permissible for use. However, superposition of these force transfer mechanisms is not permitted for filled composite columns, as the experimental data indicate that direct bearing or shear connection often do not initiate until after direct bond interaction has been breached, and little experimental data is available about the interaction of direct bearing and shear interaction in filled composite columns.

Force transfer by direct bond is commonly used in filled composite columns as long as the connections are detailed to limit local deformations (API, 1993; Roeder

and others, 1999). However, there is large scatter in the experimental data on the bond strength and associated force transfer length of filled composite columns, particularly when comparing tests in which the concrete core is pushed through the steel tube (push-out tests) to tests in which a beam is connected just to the steel tube and beam shear is transferred to the filled composite column. The added eccentricities of the connection tests typically raise the bond strength of the filled composite columns.

A reasonable lower bound value of bond strength of filled composite columns that meet the provisions of Section I2 is 60 psi (0.4 MPa). While push-out tests often show bond strengths below this value, eccentricity introduced into the connection is likely to increase the bond strength to this value or higher. Experiments also indicate that a reasonable assumption for the distance along the length of the filled composite column required to transfer the force from the steel HSS to the concrete core is approximately equal to the width of a rectangular HSS or the diameter of a round HSS, both above and below the point of load transfer.

One approach to estimating the direct bond interaction for filled HSS is presented below with recommendations for ϕ and Ω. These equations assume that one face of a rectangular filled composite column, or one-half of the perimeter of a circular filled composite column, is engaged in the transfer of stress by direct bond interaction. Higher values of nominal bond strength may be warranted for specific conditions. The scatter in the data leads to the recommended low value of the resistance factor, ϕ, and the corresponding high value of the safety factor, Ω.

(a) For rectangular HSS filled with concrete:

$$V_{in} = b^2 C_{in} F_{in} \qquad\qquad \text{(C-I2-2)}$$

$$\phi = 0.45 \text{ (LRFD)} \qquad \Omega = 3.33 \text{ (ASD)}$$

where

V_{in} = nominal bond strength, kips (N)
F_{in} = nominal bond stress = 60 psi (0.40 MPa)
b = width of HSS along face transferring load, in. (mm)
C_{in} = 1 if the filled composite column extends only above or below the point of load transfer
     = 2 if the filled composite column extends both above and below the point of load transfer

(b) For round HSS filled with concrete:

$$V_{in} = 0.5\pi D^2 C_{in} F_{in} \qquad\qquad \text{(C-I2-3)}$$

$$\phi = 0.45 \text{ (LRFD)} \qquad \Omega = 3.33 \text{ (ASD)}$$

where

V_{in} = nominal bond strength, kips (N)
F_{in} = nominal bond stress = 60 psi (0.40 MPa)
D = diameter of HSS, in. (mm)

C_{in} = 1 if the filled composite column extends only above or below the point
         of load transfer

   = 2 if the filled composite column extends both above and below the point
         of load transfer

As with encased columns, this specification assumes that the most advantageous combination of loaded area and concrete area are used to determine bearing strength. Thus, the nominal bearing strength is taken as $1.7 f'_c A_B$.

2f.   Detailing Requirements

When shear connectors are used in filled composite columns, the provisions require that they be placed a distance of 2.5 times the width of a rectangular HSS or 2.5 times the diameter of a round HSS, both above and below the load transfer region. In most such situations, a uniform spacing is appropriate. However, when large forces are applied, other connector arrangements may be needed to avoid overloading the steel section or concrete core to which the load is applied directly.

I3.   FLEXURAL MEMBERS

1.   General

Three types of composite beams are addressed in this section: fully encased steel beams, concrete-filled HSS, and steel beams with mechanical anchorage to the slab.

When a composite beam is controlled by deflection, the design should limit the behavior of the beam to the elastic range under serviceability load combinations. Alternatively, the amplification effects of inelastic behavior should be considered when deflection is checked.

It is often not practical to make accurate stiffness calculations of composite flexural members. Comparisons to short-term deflection tests indicate that the effective moment of inertia, I_{eff}, is 15 to 30 percent lower than that calculated based on linear elastic theory (I_{equiv}). Therefore, for realistic deflection calculations, I_{eff} should be taken as $0.75\, I_{equiv}$.

As an alternative, one may use a lower bound moment of inertia, I_{lb}, as defined below:

$$I_{lb} = I_s + A_s(Y_{ENA} - d_3)^2 + (\Sigma Q_n/F_y)(2d_3 + d_1 - Y_{ENA})^2 \qquad \text{(C-I3-1)}$$

where

A_s = area of steel cross section, in.^{2} (mm^2)

d_1 = distance from the compression force in the concrete to the top of the steel
        section, in. (mm)

d_3 = distance from the resultant steel tension force for full section tension yield
        to the top of the steel, in. (mm)

I_{lb} = lower bound moment of inertia, in.^{4} (mm^4)

I_s = moment of inertia for the structural steel section, in.^{4} (mm^4)

ΣQ_n = sum of the nominal strengths of shear connectors between the point of maximum positive moment and the point of zero moment to either side, kips (kN)

$Y_{ENA} = [(A_s d_3 + (\Sigma Q_n/F_y)(2d_3 + d_1))/(A_s + (\Sigma Q_n/F_y))]$

The use of constant stiffness in elastic analyses of continuous beams is analogous to the practice in reinforced concrete design. The stiffness calculated using a weighted average of moments of inertia in the positive moment region and negative moment regions may take the following form:

$$I_t = a I_{pos} + b I_{neg} \qquad\qquad \text{(C-I3-2)}$$

where

I_{pos} = effective moment of inertia for positive moment, in.^{4} (mm^4)

I_{neg} = effective moment of inertia for negative moment, in.^{4} (mm^4)

The effective moment of inertia is based on the cracked transformed section considering the degree of composite action. For continuous beams subjected to gravity loads only, the value of a may be taken as 0.6 and the value of b may be taken as 0.4. For composite beams used as part of a lateral force resisting system in moment frames, the value of a and b may be taken as 0.5 for calculations related to drift.

In cases where elastic behavior is desired, the cross-sectional strength of composite members is based on the superposition of elastic stresses including consideration of the effective section modulus at the time each increment of load is applied. For cases where elastic properties of partially composite beams are needed, the elastic moment of intertia may be approximated by

$$I_{eff} = I_s + \sqrt{\left(\Sigma Q_n/C_f\right)}\,(I_{tr} - I_s) \qquad\qquad \text{(C-I3-3)}$$

where

I_s = moment of inertia for the structural steel section, in.^{4} (mm^4)

I_{tr} = moment of inertia for the fully composite uncracked transformed section, in.^{4} (mm^4)

ΣQ_n = strength of shear connectors between the point of maximum positive moment and the point of zero moment to either side, kips (N)

C_f = compression force in concrete slab for fully composite beam; smaller of $A_s F_y$ and $0.85 f'_c A_c$, kips (N)

A_c = area of concrete slab within the effective width, in.^{2} (mm^2)

The effective section modulus S_{eff}, referred to the tension flange of the steel section for a partially composite beam, may be approximated by

$$S_{eff} = S_s + \sqrt{\left(\Sigma Q_n/C_f\right)}\,(S_{tr} - S_s) \qquad\qquad \text{(C-I3-4)}$$

where

S_s = section modulus for the structural steel section, referred to the tension flange, in.^{3} (mm^3)

S_{tr} = section modulus for the fully composite uncracked transformed section, referred to the tension flange of the steel section, in.^{3} (mm^3)

Equations C-I3-3 and C-I3-4 should not be used for ratios, $\Sigma Q_n / C_f$, less than 0.25. This restriction is to prevent excessive slip, as well as substantial loss in beam stiffness. Studies indicate that Equations C-I3-3 and C-I3-4 adequately reflect the reduction in beam stiffness and strength, respectively, when fewer connectors are used than required for full composite action (Grant and others, 1977).

U.S. practice does not generally require the following items be considered. They are highlighted here for a designer who chooses to construct something for which these items might apply.

1. Horizontal shear strength of the slab: For the case of girders with decks with narrow troughs or thin slabs, shear strength of the slab may govern the design (for example, see Figure C-I3.1). Although the configuration of decks built in the U.S. tends to preclude this mode of failure, it is important that it be checked if the force in the slab is large or an unconventional assembly is chosen. The shear strength of the slab may be calculated as the superposition of the shear strength of the concrete plus the contribution of any slab steel crossing the shear plane. The required shear strength, as shown in the figure, is given by the difference in the force between the regions inside and outside the potential failure surface. Where experience has shown that longitudinal cracking detrimental to serviceability is likely to occur, the slab should be reinforced in the direction transverse to the supporting steel section. It is recommended that the area of such reinforcement be at least 0.002 times the concrete area in the longitudinal direction of the beam and that it be uniformly distributed.

2. Rotational capacity of hinging zones: There is no required rotational capacity for hinging zones. Where plastic redistribution to collapse is allowed, the moments
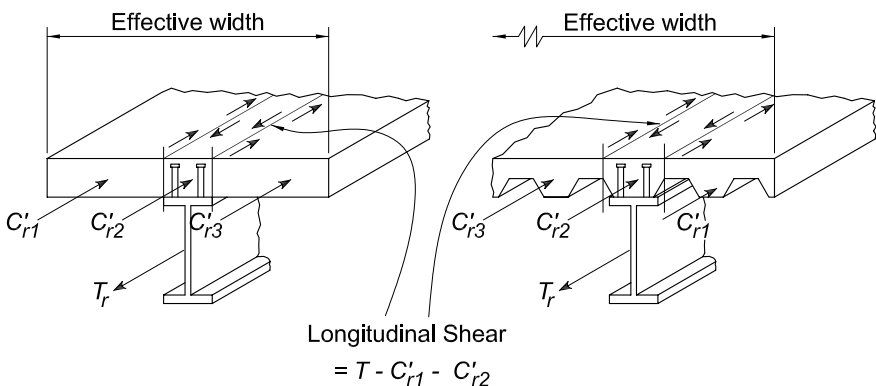

Fig. C-I3.1. Longitudinal shear in the slab (after Chien and Ritchie, 1984).

at a cross section may be as much as 30 percent lower than those given by a corresponding elastic analysis. This reduction in load effects is predicated, however, on the ability of the system to deform through very large rotations. To achieve these rotations, very strict local buckling and lateral-torsional buckling requirements must be fulfilled (Dekker and others, 1995). For cases in which a 10 percent redistribution is utilized (see Appendix 1), the required rotation capacity is within the limits provided by the local and lateral-torsional buckling provisions of Chapter F. Therefore, a rotational capacity check is not normally required for designs using this provision.

3. Minimum amount of shear connection: There is no minimum requirement for the amount of shear connection. Design aids in the U.S. often limit partial composite action to a minimum of 25 percent for practical reasons, but two issues arise with the use of low degrees of partial composite action. First, less than 50 percent composite action requires large rotations to reach the available flexural strength of the member and can result in very limited ductility after the nominal strength is reached. Second, low composite action results in an early departure from elastic behavior in both the beam and the studs. The current provisions, which are based on ultimate strength concepts, have eliminated checks for ensuring elastic behavior under service load combinations, and this can be an issue if low degrees of partial composite action are used.

4. Long-term deformations due to shrinkage and creep: There is no direct guidance in the computation of the long-term deformations of composite beams due to creep and shrinkage. The long-term deformation due to shrinkage can be calculated with the simplified model shown in Figure C-I3.2, in which the effect of shrinkage is taken as an equivalent set of end moments given by the shrinkage force (long-term restrained shrinkage strain times modulus of concrete times effective area of concrete) times the eccentricity between the center of the slab and the elastic neutral axis. If the restrained shrinkage coefficient for the aggregates is not known, the shrinkage strain for these calculations may be taken as 0.02 percent. The long-term deformations due to creep, which can be quantified using a model similar to that shown in the figure, are small unless the spans are long and the permanent live loads large. For shrinkage and creep effects, special attention should be given to lightweight aggregates, which tend to have higher creep coefficients and moisture absorption and lower modulus of elasticity than conventional aggregates, exacerbating any potential deflection problems. Engineering judgment is required, as calculations for long-term deformations require consideration of the many variables involved and because linear superposition of these effects is not strictly correct (ACI, 1997; Viest and others, 1997).

1a.   Effective Width

The same effective width rules apply to composite beams with a slab on either one side or both sides of the beam. In cases where the effective stiffness of a beam with a one-sided slab is important, special care should be exercised since this model

can substantially overestimate stiffness (Brosnan and Uang, 1995). To simplify design, effective width is based on the full span, center-to-center of supports, for both simple and continuous beams.

1b.   Shear Strength

A conservative approach to shear provisions for composite beams is adopted by assigning all shear to the steel section web. This neglects any concrete slab contribution and serves to simplify design.

1c.   Strength during Construction

Composite beam design requires care in considering the loading history. Loads applied to an unshored beam before the concrete has cured are resisted by the steel section alone, and only loads applied after the concrete has cured are considered to be resisted by the composite section. It is usually assumed for design purposes that concrete has hardened when it attains 75 percent of its design strength. Unshored beam deflection caused by fresh concrete tends to increase slab thickness and dead load. For longer spans this may lead to instability analogous to roof ponding. Excessive increase of slab thickness may be avoided by beam camber. Pouring the slab to a constant thickness will also help eliminate the possibility of ponding instability (Ruddy, 1986). When forms are not attached to the top flange, lateral bracing of the steel beam during construction may not be continuous and the unbraced length may control flexural strength, as defined in Chapter F.

This Specification does not include special requirements for strength during construction. For these noncomposite beams, the provisions of Chapter F apply.

Load combinations for construction loads should be determined for individual projects according to local conditions, using ASCE (2002) as a guide.
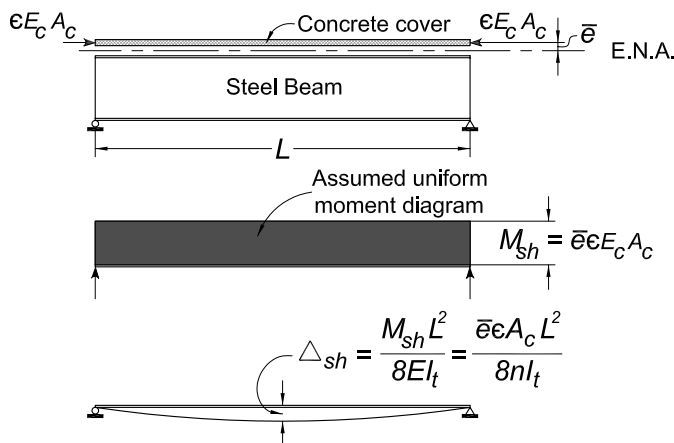

Fig. C-I3.2. Calculation of shrinkage effects [from Chien and Ritchie (1984)].

2.   Strength of Composite Beams with Shear Connectors

Section I3.2 applies to simple and continuous composite beams with shear connectors, constructed with or without temporary shores.

2a.   Positive Flexural Strength

The flexural strength of a composite beam in the positive moment region may be controlled by the strength of the steel section, the concrete slab or the shear connectors. In addition, web buckling may limit flexural strength if the web is slender and a large portion of the web is in compression.

According to Table B5.1, local web buckling does not reduce the plastic strength of a bare steel beam if the beam depth-to-web thickness ratio is not larger than $3.76\sqrt{E/F_y}$. In the absence of web buckling research on composite beams, the same ratio is conservatively applied to composite beams.

For beams with more slender webs, this Specification conservatively adopts first yield as the flexural strength limit. In this case, stresses on the steel section from permanent loads applied to unshored beams before the concrete has cured must be superimposed on stresses on the composite section from loads applied to the beams after hardening of concrete. For shored beams, all loads may be assumed to be resisted by the composite section.

When first yield is the flexural strength limit, the elastic transformed section is used to calculate stresses on the composite section. The modular ratio, $n = E/E_c$, used to determine the transformed section, depends on the specified unit weight and strength of concrete.

2b.   Negative Flexural Strength

Loads applied to a continuous composite beam with shear connectors throughout its length, after the slab is cracked in the negative moment region, are resisted in that region by the steel section and by properly anchored longitudinal slab reinforcement. When an adequately braced compact steel section and adequately developed longitudinal reinforcing bars act compositely in the negative moment region, the nominal flexural strength is determined from plastic stress distributions.

2c.   Strength of Composite Beams with Formed Steel Deck

Figure C-I3.3 is a graphic presentation of the terminology used in Section I3.2c.

The design rules for composite construction with formed steel deck are based upon a study (Grant and others, 1977) of the then-available test results. The limiting parameters listed in Section I3.2c were established to keep composite construction with formed steel deck within the available research data.

The minimum spacing of 18 in. for connecting composite decking to the support is intended to address a minimum uplift requirement during the construction phase prior to placing concrete.

2d.   Shear Connectors

(1) Load Transfer for Positive Moment

When studs are used on beams with formed steel deck, they may be welded directly through the deck or through prepunched or cut-in-place holes in the deck. The usual procedure is to install studs by welding directly through the deck; however, when the deck thickness is greater than 16 gage (1.5 mm) for single thickness, or 18 gage (1.2 mm) for each sheet of double thickness, or when the total thickness of galvanized coating is greater than 1.25 ounces/sq. ft (0.38 kg/m^2), special precautions and procedures recommended by the stud manufacturer should be followed.

Composite beam tests in which the longitudinal spacing of shear connectors was varied according to the intensity of the static shear, and duplicate beams
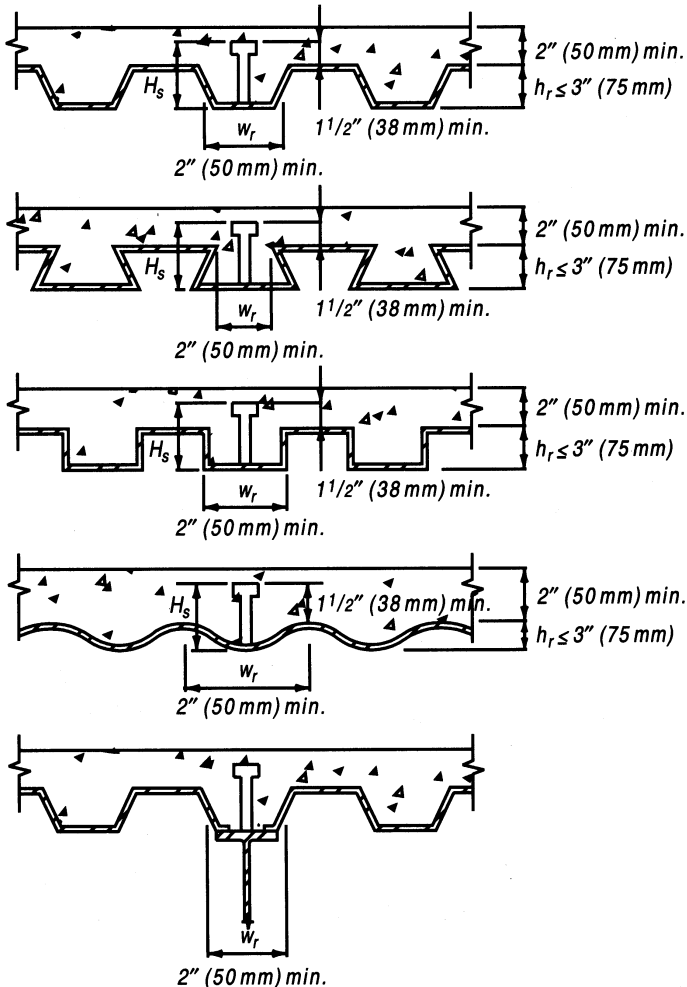

Fig. C-I3.3. Steel deck limits.

in which the connectors were uniformly spaced, exhibited approximately the same ultimate strength and approximately the same amount of deflection at nominal loads. Under distributed load conditions, only a slight deformation in the concrete near the more heavily stressed connectors is needed to re-distribute the horizontal shear to other less heavily stressed connectors. The important consideration is that the total number of connectors be sufficient to develop the shear on either side of the point of maximum moment. The provisions of this Specification are based upon this concept of composite action.

In computing the available flexural strength at points of maximum negative bending, reinforcement parallel to the steel beam within the effective width of the slab may be included, provided such reinforcement is properly anchored beyond the region of negative moment. However, shear connectors are required to transfer the ultimate tensile force in the reinforcement from the slab to the steel beam.

When steel deck includes units for carrying electrical wiring, crossover headers are commonly installed over the cellular deck perpendicular to the ribs. These create trenches that completely or partially replace sections of the concrete slab above the deck. These trenches, running parallel to or transverse to a composite beam, may reduce the effectiveness of the concrete flange. Without special provisions to replace the concrete displaced by the trench, the trench should be considered as a complete structural discontinuity in the concrete flange.

When trenches are parallel to the composite beam, the effective flange width should be determined from the known position of the trench.

Trenches oriented transverse to composite beams should, if possible, be located in areas of low bending moment and the full required number of studs should be placed between the trench and the point of maximum positive moment. Where the trench cannot be located in an area of low moment, the beam should be designed as noncomposite.

(3) Strength of Stud Shear Connectors

Considerable research has been published in recent years indicating that shear stud strength equations in previous AISC Specifications are unconservative. Specifically, it has been understood for some time that the stud strength values given by Equation I5-1 in previous LRFD Specifications, in combination with the old Equations I3-2 and I3-3, which modified the strength based on whether the deck was perpendicular or parallel to the beams, are higher than those derived from either pushout or beam tests for studs embedded in modern steel decks (Jayas and Hosain, 1988; 1988a; Mottram and Johnson, 1990; Easterling, Gibbings, and Murray, 1993; Roddenberry and others, 2002) Equation I5-1 in the previous specifications is similar to the new Equation I3-5 but without the R_g and R_p factors.

Other codes use a stud strength expression similar to the previous AISC *LRFD Specification*; the stud strength is reduced by a ϕ factor of 0.8 in the Canadian code (CSA, 1994) and by an even lower partial safety factor ($\phi = 0.60$) for the corresponding stud strength equations in Eurocode 4 (2003).

The origin of this discrepancy can be traced to the way the old equations for stud strength were developed. The old approach was developed based on tests on solid slabs, and, as noted by the current R_p and R_g factors in the new Equation I3-4, the current approach remains valid for this case. Following studies reported in Robinson (1967) and Fisher (1970), Grant and others developed expressions for stud strength that accounted for the presence of the steel deck by including additional variables related to the deck and stud geometries (Grant and others, 1977). However, most of those tests were conducted with decks that were formed specifically for the tests from flat steel sheets.

The majority of composite steel floor decks used today have a stiffening rib in the middle of each deck flute. Because of the stiffener, studs must be welded off-center in the deck rib. Recent studies have shown that shear studs behave differently depending upon their location within the deck rib (Lawson, 1992; Easterling and others, 1993; Van der Sanden, 1995; Yuan, 1996; Johnson and Yuan, 1998; Roddenberry and others, 2002; Roddenberry and others, 2002a). The so-called "weak" (unfavorable) and "strong" (favorable) positions are illustrated in Figure C-I3.4. Furthermore, the maximum value shown in these studies for studs welded through steel deck is on the order of 0.7 to 0.75 $F_u A_{sc}$. Studs placed in the weak position have strengths as low as $0.5 F_u A_{sc}$.

The strength of stud connectors installed in the ribs of concrete slabs on formed steel deck with the ribs oriented perpendicular to the steel beam is reasonably estimated by the strength of stud connectors computed from Equation I3-3, which sets the default value for shear stud strength equal to that for the weak stud position. Both AISC (1997) and the Steel Deck Institute (SDI, 1999) recommend that studs be detailed in the strong position, but ensuring that studs are placed in the strong position is not necessarily an easy task because it is not always easy for the installer to determine where along the beam the particular rib is located, relative to the end, midspan or point of zero shear. Therefore, the installer may not be clear on which is the strong and which is the weak position.
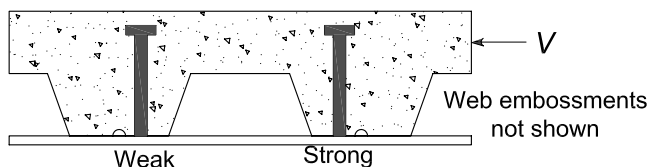

Fig. C-I3.4. Weak and strong stud positions [from Roddenberry and others (2002a)].

In most composite floors designed today, the ultimate strength of the composite section is governed by the stud strength, as full composite action is typically not the most economical solution to resist the required strength. The degree of composite action, as represented by the ratio $\Sigma Q_n/F_y A_s$ (the total shear connection strength divided by the yield strength of the steel cross section), influences the flexural strength as shown in Figure C-I3.5.

It can be seen from Figure C-I3.5 that a relatively large change in shear connection strength results in a much smaller change in flexural strength. Thus, formulating the influence of steel deck on shear connector strength by conducting beam tests and back-calculating through the flexural model, as was done in the past, lead to an inaccurate assessment of stud strength when installed in metal deck.

The changes in the 2005 Specification are not a result of either structural failures or performance problems. Designers concerned about the strength of existing structures need to note that the slope of the curve shown in Figure C-I3.5 is rather flat as the degree of composite action approaches one. Thus, even a large change in shear stud strength does not result in a proportional decrease of the flexural strength. In addition, as noted above, the current expression does not account for all the possible shear force transfer mechanisms, primarily because many of them are difficult or impossible to quantify. However, as noted in the Commentary to Section I3.1, as the degree of composite action decreases, the deformation demands on shear studs increase. This effect is reflected by the increasing slope of the relationship shown in Figure C-I3.5 as the degree of composite action decreases. Thus designers should be careful
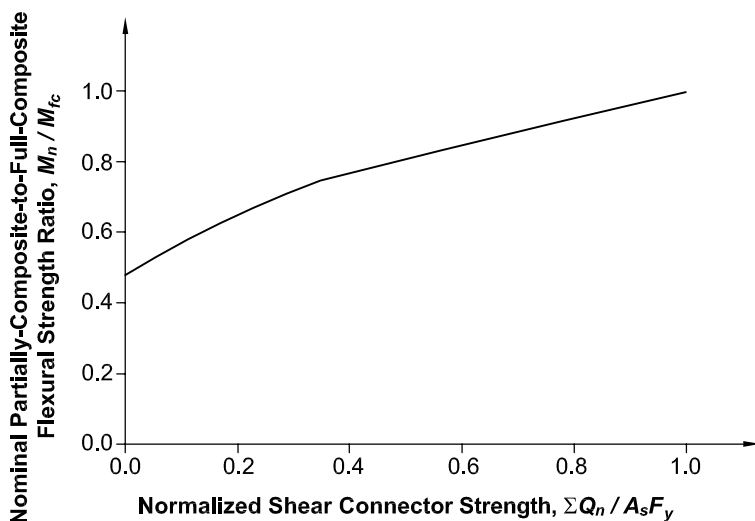

Fig. C-I3.5. Normalized flexural strength versus shear connection strength ratio [W16×31 (W410×46.1), $F_y = 50$ ksi (345 MPa), Y2 = 4.5 in. (114 mm)] (after Easterling and others, 1993).

when evaluating the strength of existing composite beams with 50 percent composite action or less.

(4) Strength of Channel Shear Connectors

Equation I3-4 is a modified form of the formula for the strength of channel connectors presented in Slutter and Driscoll (1965), which was based on the results of pushout tests and a few simply supported beam tests with solid slabs by Viest and others (1952). The modification has extended its use to lightweight concrete.

Eccentricities need not be considered in the weld design for cases where the welds at the toe and heel of the channel are greater than $3/16$ in. and the connector meets the following requirements:

$$1.0 \leq \frac{t_f}{t_w} \leq 5.5$$

$$\frac{H}{t_w} \geq 8.0$$

$$\frac{L_c}{t_f} \geq 6.0$$

$$0.5 \leq \frac{R}{t_w} \leq 1.6$$

where t_f is the connector flange thickness, t_w is the connector web thickness, H is the height of the connector, L_c is the length of the connector, and R is the radius of the fillet between the flange and web of the connector.

(6) Shear Connector Placement and Spacing

Uniform spacing of shear connectors is permitted, except in the presence of heavy concentrated loads.

Studs not located directly over the web of a beam tend to tear out of a thin flange before attaining full shear-resisting strength. To guard against this contingency, the size of a stud not located over the beam web is limited to $2^1/2$ times the flange thickness (Goble, 1968). The practical application of this limitation is to select only beams with flanges thicker than the stud diameter divided by 2.5.

The minimum spacing of connectors along the length of the beam, in both flat soffit concrete slabs and in formed steel deck with ribs parallel to the beam, is six diameters; this spacing reflects development of shear planes in the concrete slab (Ollgaard and others, 1971). Because most test data are based on the minimum transverse spacing of four diameters, this transverse spacing was set as the minimum permitted. If the steel beam flange is narrow, this spacing requirement may be achieved by staggering the studs with a minimum transverse spacing of three diameters between the staggered row of studs. When deck ribs are parallel to the beam and the design requires more studs than can

be placed in the rib, the deck may be split so that adequate spacing is available for stud installation. Figure C-I3.6 shows possible connector arrangements.

3. Flexural Strength of Concrete-Encased and Filled Members

Tests of concrete-encased beams demonstrated that: (1) the encasement drastically reduces the possibility of lateral-torsional instability and prevents local buckling of the encased steel; (2) the restrictions imposed on the encasement practically prevent bond failure prior to first yielding of the steel section; and (3) bond failure does not necessarily limit the moment strength of an encased steel beam (ASCE, 1979). Accordingly, this Specification permits three alternative design methods for determination of the nominal flexural strength: (a) based on the first yield in the tension flange of the composite section; (b) based on the plastic flexural strength of the steel section alone; and (c) based on the plastic flexural strength of the composite section or the strain-compatibility method. Method (c) is applicable only when shear connectors are provided along the steel section and reinforcement of the concrete encasement meets the specified detailing requirements. No limitations are placed on the slenderness of either the composite beam or the elements of the steel section, since the encasement effectively inhibits both local and lateral buckling.

In method (a), stresses on the steel section from permanent loads applied to unshored beams before the concrete has hardened must be superimposed on stresses on the composite section from loads applied to the beams after hardening of the concrete. In this superposition, all permanent loads should be multiplied by the dead load factor and all live loads should be multiplied by the live load factor. For shored beams, all loads may be assumed as resisted by the composite section. Complete interaction (no slip) between the concrete and steel is assumed.

I4. COMBINED AXIAL FORCE AND FLEXURE

As with all frame analyses in this Specification, required strengths for composite beam-columns should be obtained from second-order analysis or amplified first-order analysis. With respect to the assessment of the available strength, the Specification provisions for interaction between axial force and flexure in composite
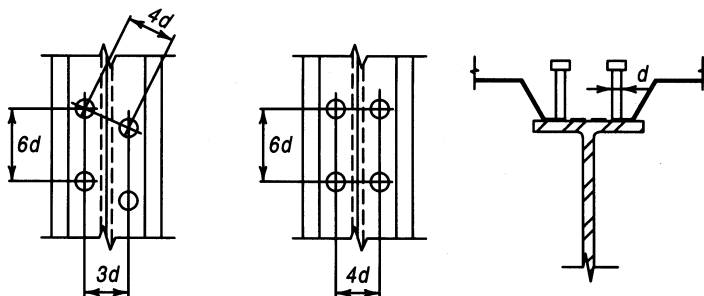

Fig. C-I3.6. Shear connector arrangements.

members do not provide explicit equations. However, the provisions provide guidance in Section I4 on the principles that can serve to establish an interaction diagram similar to those used in reinforced concrete design.

Composite Beam-Columns. The available axial strength, including the effects of buckling, and the available flexural strength can be calculated using either the plastic stress distribution method or the strain-compatibility method. There are three simplified approaches for determining the strength of composite beam-columns discussed below that take advantage of strength determination for a limited number of cases and interpolation for all other cases.

The first approach applies to doubly symmetric composite beam-columns, the most common geometry found in building construction. For this case, the interaction equations of Section H1 provide a conservative assessment of the available strength of the member for combined axial compression and flexure. These provisions may also be used for combined axial tension and flexure. The degree of conservatism generally depends on the extent of concrete contribution to the overall strength, relative to the steel contribution. Thus, for example, the equations are generally more conservative for members with high concrete compressive strength as compared to members with low concrete compressive strength.

The second approach is based on developing interaction surfaces for combined axial compression and flexure at the nominal strength level, using the plastic stress distribution method. This results in interaction surfaces similar to those shown in Figures C-I1.1 and C-I4.1. The five points identified in Figure C-I4.1 are defined by the plastic stress distribution used in their determination. Point *A* is the pure axial strength determined according to Equations I2-2 or I2-13. Point *B* is determined as the flexural strength of the section determined according to the provisions of Section I3. Point *C* corresponds to a plastic neutral axis location that results in the same flexural capacity as Point *B* but with axial load. Point *D* corresponds to an axial strength of one half of that determined for Point *C*. Point *E* is an arbitrary point necessary to better reflect bending strength for *y*-axis bending of encased shapes and bending of filled HSS. Linear interpolation between these anchor points may be used. However, with this approach, care should be taken in reducing Point *D* by a resistance factor or to account for member slenderness, as that may lead to an unsafe situation whereby additional flexural strength is permitted at a lower axial compressive strength than predicted by the cross section strength of the member. Once the nominal strength interaction surface is determined, length effects according to Equations I2-6 and I2-7 must be applied. The available strength is then determined by applying the compression and bending resistance factors or safety factors.

The third approach is a simplified bilinear approach as shown in Figure C-I4.1. After the column axial strength (Point *A* in Figure C-I4.1) is computed using Equation I2-2 for concrete-encased sections or Equation I2-13 for concrete-filled

sections, this strength is reduced by the length effects using Equations E2-6 or E2-7 to obtain P_n, or Point A_λ. The resistance factor, ϕ_c, or safety factor, Ω_c, is then applied to this value to become the anchor point for design on the vertical axis, A_d. The anchor point on the horizontal axis, Point B_d, is given by the flexural strength of the section, Point B, modified by the appropriate bending resistance factor or safety factor.

Point C is then adjusted downward by the same length effect reduction as applied to Point A, to obtain Point C_λ. Point C_λ is then adjusted down by ϕ_c or Ω_c and to the left by ϕ_b or Ω_b to obtain Point C_d. A straight line approximation may then be used between Points A_d, C_d and B_d, as shown in the figure. Using linear interpolation between Points A_d, C_d and B_d in Figure C-I4.1, the following interaction equations may be derived for composite beam-columns subjected to combined axial compression plus biaxial flexure:

If $P_r < P_C$

$$\frac{M_{rx}}{M_{Cx}} + \frac{M_{ry}}{M_{Cy}} \leq 1 \qquad \text{(C-I4-1a)}$$

If $P_r \geq P_C$

$$\frac{P_r - P_C}{P_A - P_C} + \frac{M_{rx}}{M_{Cx}} + \frac{M_{ry}}{M_{Cy}} \leq 1 \qquad \text{(C-I4-1b)}$$

where

P_r = required compressive strength, kips (N)
P_A = available axial compressive strength at Point A, kips (N)
P_C = available axial compressive strength at Point C, kips (N)
M_r = required flexural strength, kip-in. (N-mm)
M_C = available flexural strength at Point C, kip-in. (N-mm)
x = subscript relating symbol to strong axis bending
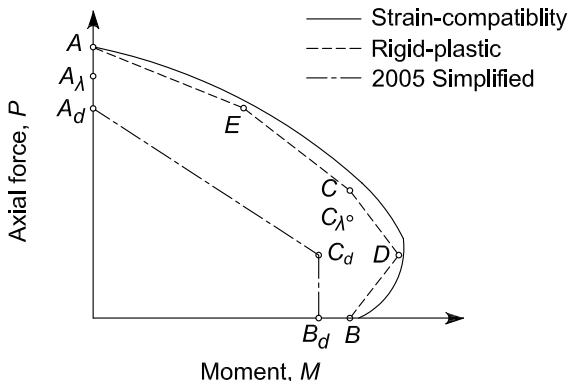y = subscript relating symbol to weak axis bending

Fig. C-I4.1. Interaction diagram for composite beam-column design.

For design according to Section B3.3, (LRFD)

$P_r = P_u$ = required compressive strength using LRFD load combinations, kips (N)

$P_A = P_{Ad}$ = design axial compressive strength, determined in accordance with Section I2, kips (N)

$P_C = P_{Cd}$ = design axial compressive strength at point C_d in Figure C-I4.1, kips (N)

M_r = required flexural strength using LRFD load combinations, kip-in. (N-mm)

$M_C = M_{Cd} = M_{Bd}$ design flexural strength, determined in accordance with Section I3, kip-in. (N-mm)

$\phi_c = 0.75$

$\phi_b = 0.90$

For design according to Section B3.4, (ASD)

$P_r = P_a$ = required compressive strength using ASD load combinations, kips (N)

$P_A = P_{Ad}$ = allowable compressive strength, determined in accordance with Section I2, kips (N)

$P_C = P_{Cd}$ = allowable axial compressive strength at Point C_d in Figure C-I4.1, kips (N)

M_r = required flexural strength using ASD load combinations, kip-in. (N-mm)

$M_C = M_{Cd} = M_{Bd}$ allowable flexural strength, determined in accordance with Section I3, kip-in. (N-mm).

$\Omega_c = 2.00$

$\Omega_b = 1.67$

For biaxial bending, the value of P_C may be different when computed for the major and minor axis. The smaller of the two values should be used in Equation C-I4-1b and for the limits of Equations C-I4-1a and b.

Composite Beams Subjected to Combined Axial Force and Flexure. Combined axial force and flexure on composite floor beams has not been addressed directly in this chapter. Composite floor beam members (steel beams composite with floor slabs at their top flange) with axial loading may include collector elements (drag struts) and stabilizing elements for sloping column members. Few detailed design guidelines exist for such members; preliminary guidance for seismic design is given in AISC (2002).

Load combinations as set forth in ASCE (2002) should be used to determine the applicable loading at critical sections, including secondary bending effects of eccentrically applied loading. Adequate means to transmit axial loading to and from the steel section should be provided. Where shear connectors are used, the top flanges may be considered braced for compressive loading at the shear connector locations. Additional shear connectors may be required for axial load transfer and added flexure as determined from the load combinations. For shear studs added to

transfer axial loads between beams and slabs, Q_n may be determined in accordance with Section I3. For load combinations resulting in compressive loading of the lower flange, length effects between brace points should be considered. Inflection points should not be considered brace points for torsional buckling of the unbraced flange. For discussion and design methodology, the reader is referred to Galambos (1998).

I5.   SPECIAL CASES

Tests are required for construction that falls outside the limits given in this Specification. Different types of shear connectors may require different spacing and other detailing than stud and channel connectors.

CHAPTER J

DESIGN OF CONNECTIONS

The provisions of Chapter J cover the design of connections not subject to *cyclic loads*. Wind and other environmental loads are generally not considered to be *cyclic loads*. The provisions generally apply to connections other than HSS and box members. See Chapter K for HSS and box member connections and Appendix 3 for fatigue provisions.

J1. GENERAL PROVISIONS

Selection of weld type (CJP versus fillet versus PJP) depends on base connection geometry (butt versus T or corner) and required strength, among other issues discussed in this Section. Consideration of notch effects and the ability to evaluate with NDE may be appropriate for cyclically loaded joints or joints expected to deform plastically.

1. Design Basis

In the absence of defined design loads, a minimum design load should be considered. Historically, a value of 10 kips (44 kN) for LRFD and 6 kips (27 kN) for ASD have been used as reasonable values. For elements such as lacing, sag rods, girts or small simple members, a load more appropriate to the size and use of the part should be used. Design requirements for the installed elements as well as construction loads need to be considered when specifying minimum loads for connections.

2. Simple Connections

Simple connections are considered in Sections B3.6a. and J1.2. In Section B3.6a "simple" connections are defined (with further elaboration in the Commentary for Section B3.6) as a guide to idealization of the structure for the purpose of analysis. The assumptions made in the analysis determine the outcome of the analysis that serves as the basis for design (for connections that means the force and deformation demands that the connection must resist). Section J1.2 focuses on the actual proportioning of the connection elements to achieve the required resistance. In short, Section B3.6a establishes the modeling assumptions that determine the design forces and deformations for use in Section J1.2.

Sections B3.6a and J1.2 are not mutually exclusive. If a "simple" connection is assumed for analysis, the actual connection, as finally designed, must deliver performance consistent with that assumption. For a simple connection it is important to verify that the performance is consistent with the design assumptions; in other

words, the connection must be able to meet the required rotation and must not introduce strength and stiffness that significantly alter the mode of response.

3.   Moment Connections

Two types of moment connections are defined in Section B3.6b: fully restrained (FR) and partially restrained (PR). FR moment connections must have sufficient strength and stiffness to transfer moment and maintain the angle between connected members. PR moment connections are designed to transfer moments but also allow rotation between connected members as the loads are resisted. The response characteristics of a PR connection must be documented in the technical literature or established by analytical or experimental means. The component elements of a PR connection must have sufficient strength, stiffness and deformation capacity to satisfy the design assumptions.

4.   Compression Members with Bearing Joints

The provisions for "compression members other than columns finished to bear" are intended to account for member out-of-straightness and also to provide a degree of robustness in the structure so as to resist unintended or accidental lateral loadings that may not have been considered explicitly in the design.

A provision analogous to that in Section J1.4(b)(i), requiring that splice materials and connectors have an available strength of at least 50 percent of the required compressive strength, has been in the AISC Specifications for more than 40 years. The current Specification clarifies this requirement by stating that the force for proportioning the splice materials and connectors is a tensile force. This avoids uncertainty as to how to handle situations where compression on the connection imposes no force on the connectors.

Proportioning the splice materials and connectors for 50 percent of the required member strength is simple, but can be very conservative. In Section J1.4(b)(ii), the Specification offers an alternative that addresses directly the design intent of these provisions. The lateral load of 2 percent of the required compressive strength of the member simulates the effect of a kink at the splice, caused by an end finished slightly out-of-square or other construction condition. Proportioning the connection for the resulting moment and shear also provides a degree of robustness in the structure.

5.   Splices in Heavy Sections

Solidified but still hot filler metal contracts significantly as it cools to ambient temperature. Shrinkage of large groove welds between elements that are not free to move so as to accommodate the shrinkage causes strains in the material adjacent to the weld that can exceed the yield point strain. In thick material the weld shrinkage is restrained in the thickness direction, as well as in the width and length directions, causing triaxial stresses to develop that may inhibit the ability

to deform in a ductile manner. Under these conditions, the possibility of *brittle fracture* increases.

When splicing hot-rolled shapes with flange thicknesses exceeding 2 in. (50 mm) or heavy welded built-up members, these potentially harmful weld shrinkage strains can be avoided by using bolted splices, fillet-welded lap splices, or splices that combine a welded and bolted detail (see Figure C-J1.1). Details and techniques that perform well for materials of modest thickness usually must be changed or supplemented by more demanding requirements when welding thick material. The provisions of AWS D1.1 (AWS, 2004) are minimum requirements that apply to most structural welding situations; however, when designing and fabricating welded splices of hot-rolled shapes with flange thicknesses exceeding 2 in. (50 mm) and similar built-up cross sections, special consideration must be given to all aspects of the welded splice detail:

(1) Notch-toughness requirements should be specified for tension members; see Commentary Section A3.
(2) Generously sized weld access holes (see Section J1.6) are required to provide increased relief from concentrated weld shrinkage strains, to avoid close juncture of welds in orthogonal directions, and to provide adequate clearance for the exercise of high quality workmanship in hole preparation, welding and for ease of inspection.
(3) Preheating for thermal cutting is required to minimize the formation of a hard surface layer.
(4) Grinding of copes and access holes to bright metal to remove the hard surface layer is required, along with inspection using magnetic particle or dye-penetrant methods, to verify that transitions are free of notches or cracks.

In addition to tension splices of truss chord members and tension flanges of flexural members, other joints fabricated from heavy sections subject to tension should be given special consideration during design and fabrication.
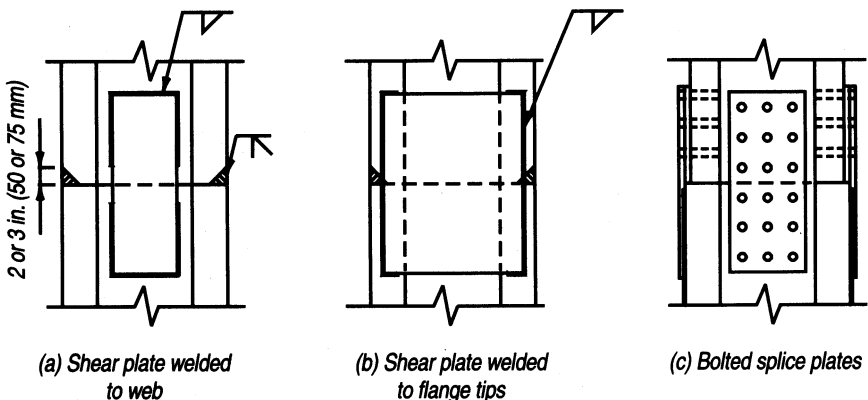

Fig. C-J1.1. Alternative splices that minimize weld restraint tensile stresses.

Previous editions of this Specification mandated that backing bars and weld tabs be removed from all splices of heavy sections. These requirements were deliberately removed from this edition, being judged unnecessary and, in some situations, potentially resulting in more harm than good. The Specification still permits the engineer of record to specify their removal when this is judged appropriate.

The previous requirement for the removal of backing bars necessitated, in some situations, that such operations be performed out-of-position; that is, the welding required to restore the backgouged area had to be applied in the overhead position. This may necessitate alternate equipment for gaining access, different welding equipment, processes and/or procedures, and other practical constraints. When box sections made of plate are spliced, access to the interior side (necessary for backing removal) is typically impossible.

Weld tabs that are left in place on splices act as "short attachments" and attract little stress. Even though it is acknowledged that weld tabs might contain regions of inferior quality weld metal, the stress concentration effect is minimized since little stress is conducted through the attachment.

6.   Beam Copes and Weld Access Holes

Beam copes and weld access holes are frequently required in the fabrication of structural components. The geometry of these structural details can affect the components' performance. The size and shape of beam copes and weld access holes can have a significant effect on the ease of depositing sound weld metal, the ability to conduct nondestructive examinations, and the magnitude of the stresses at the geometric discontinuities produced by these details.

Weld access holes used to facilitate welding operations are required to have a minimum length from the toe of the weld preparation (see Figure C-J1.2) equal to 1.5 times the thickness of the material in which the hole is made. This minimum length is expected to accommodate and relieve a significant amount of the weld shrinkage strains at the web-to-flange intersection.

The height of the weld access hole must provide sufficient clearance for ease of welding and inspection and must be large enough to allow the welder to deposit sound weld metal through and beyond the web. A weld access hole height equal to 1.5 times the thickness of the material with the access hole but not less than 1 in. (25 mm) has been judged to satisfy these welding and inspection requirements. The height of the weld access hole need not exceed 2 in. (50 mm).

The geometry of the reentrant corner between the web and the flange determines the level of stress concentration at that location. A 90° reentrant corner having a very small radius produces a very high stress concentration that may lead to rupture of the flange. Consequently, to minimize the stress concentration at this location, the edge of the web shall be sloped or curved from the surface of the flange to the reentrant surface of the access hole.

Stress concentrations along the perimeter of beam copes and weld access holes also can affect the performance of the joint. Consequently, all beam copes and weld access holes are required to be free of stress raisers such as notches and gouges.

Stress concentrations at web-to-flange intersections of built-up shapes can be decreased by terminating the weld away from the access hole. Thus, for built-up shapes with fillet welds or partial-joint-penetration groove welds that join the web to the flange, the weld access hole may terminate perpendicular to the flange, provided that the weld is terminated at least one thickness of the web away from the access hole.

7.   Placement of Welds and Bolts

Slight eccentricities between the gravity axis of single and double angle members and the center of gravity of connecting bolts or rivets have long been ignored as having negligible effect on the static strength of such members. Tests have shown that similar practice is warranted in the case of welded members in statically loaded structures (Gibson and Wake, 1942).

Alternate 1   Alternate 2   Alternate 3

Rolled Shapes and Built-Up Shapes   Built-Up Shapes Assembled
Assembled Prior to Cutting the Access Hole   After Cutting the Access Hole
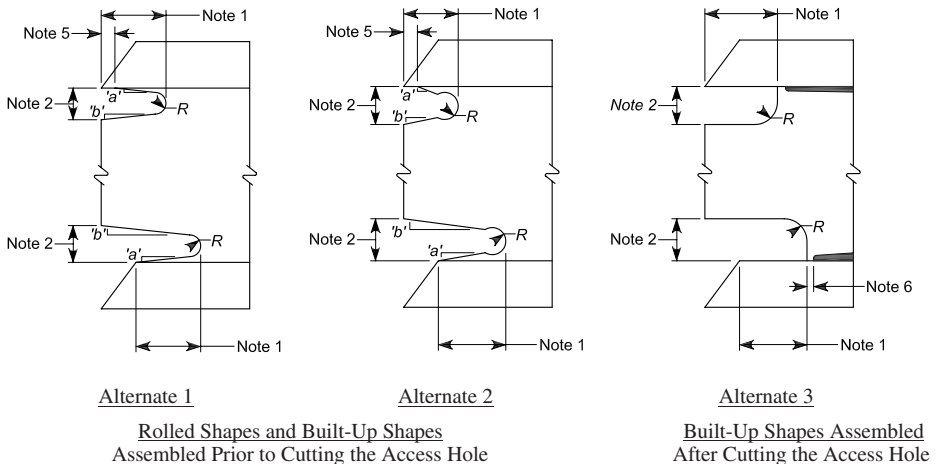
Notes: There are typical details for joints welded from one side against steel backing. Alternative details
   are discussed in the commentary text.
   (1) Width: Greater of $1.5 \times t_w$ or $1^1/2$ in. (38 mm). Tolerance is $\pm^1/4$ in. (6 mm).
   (2) Height: Greater of $1.5 \, t_w$ or 1 in. (25 mm) but need not exceed 2 in. (50 mm).
   (3) R: $^3/8$ in. min. (8 mm). Grind the thermally cut surfaces of access holes in heavy shapes as
       defined in Section A3.1c and A3.1d.
   (4) Slope 'a' forms a transition from the web to the flange. Slope 'b' may be horizontal.
   (5) The bottom of the top flange is to be contoured to permit the tight fit of backing bars where they
       are to be used.
   (6) The web-to-flange weld of built-up members is to be held back a distance of at least the weld
       size from the edge of the access hole.

Fig. C-J1.2. Weld access hole geometry.

However, the fatigue life of eccentrically loaded welded angles has been shown to be very short (Kloppel and Seeger, 1964). Notches at the roots of fillet welds are harmful when alternating tensile stresses are normal to the axis of the weld, as could occur due to bending when axial cyclic loading is applied to angles with end welds not balanced about the neutral axis. Accordingly, balanced welds are required when such members are subjected to cyclic loading (see Figure C-J1.3).

8.   Bolts in Combination with Welds

As in previous editions, this Specification does not permit bolts to share the load with welds except for bolts in shear connections. The conditions for load sharing have, however, changed substantially based on recent research (Kulak and Grondin, 2001). For shear-resisting connections with longitudinally loaded fillet welds, load sharing between the longitudinal welds and bolts in standard holes or short-slotted holes transverse to the direction of the load is permitted, but the contribution of the bolts is limited to 50 percent of the available strength of the equivalent bearing-type connection. Both A307 and high-strength bolts are permitted. The heat of welding near bolts will not alter the mechanical properties of the bolts.

In making alterations to existing structures, the use of welding to resist loads other than those produced by existing dead load present at the time of making the alteration is permitted for riveted connections and high-strength bolted connections if the bolts are pretensioned to the levels in Table J3.1 or J3.1M prior to welding.

The restrictions on bolts in combination with welds do not apply to typical bolted/welded beam-to-girder and beam-to-column connections and other comparable connections (Kulak, Fisher, and Struik, 1987).
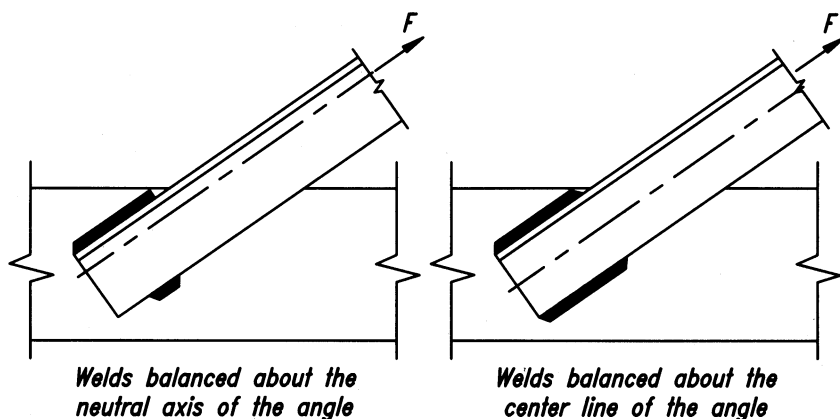

Welds balanced about the   Welds balanced about the
neutral axis of the angle   center line of the angle

Fig. C-J1.3. Balanced welds.

9.   High-Strength Bolts in Combination with Rivets

When high-strength bolts are used in combination with rivets, the ductility of the rivets permits the direct addition of the strengths of the two fastener types.

10.   Limitations on Bolted and Welded Connections

Pretensioned bolts, slip-critical bolted connections, or welds are required whenever connection slip can be detrimental to the performance of the structure or there is a possibility that nuts will back off. Snug-tightened high-strength bolts are recommended for all other connections.

J2.   WELDS

Selection of weld type [complete-joint-penetration (CJP) groove weld versus fillet versus partial-joint-penetration (PJP) groove weld] depends on base connection geometry (butt versus T or corner), in addition to required strength, and other issues discussed below. Consideration of notch effects and the ability to evaluate with nondestructive testing may be appropriate for cyclically loaded joints or joints expected to deform plastically.

1.   Groove Welds

1a.   Effective Area

Effective throats larger than those in Table J2.1 can be qualified by tests. The weld reinforcement is not used in determining the effective throat of a groove weld.

1b.   Limitations

Table J2.3 provides a minimum size of PJP groove weld for a given thickness of the thinner part joined. Structural steel with a specified minimum yield stress of 50 ksi (350 MPa) is the prevalent material. The use of prequalified weld procedures is prevalent in structural welding. The minimum weld sizes required in this Specification are appropriate for filler metal prequalified with 50-ksi (350 MPa) base metal. Also, see the commentary to Section J2.2b for fillet weld limitations.

2.   Fillet Welds

2a.   Effective Area

The effective throat of a fillet weld is based on the root of the joint and the face of the diagrammatic weld; hence this definition gives no credit for weld penetration or reinforcement at the weld face. Some welding procedures produce a consistent penetration beyond the root of the weld. This penetration contributes to the strength of the weld. However, it is necessary to demonstrate that the weld procedure to be used produces this increased penetration. In practice, this can be

done initially by cross-sectioning the runoff plates of the joint. Once this is done, no further testing is required, as long as the welding procedure is not changed.

2b. Limitations

Table J2.4 provides the minimum size of a fillet weld for a given thickness of the thinner part joined. The requirements are not based on strength considerations, but on the quench effect of thick material on small welds. Very rapid cooling of weld metal may result in a loss of ductility. Furthermore, the restraint to weld metal shrinkage provided by thick material may result in weld cracking. The use of the thinner part to determine the minimum size weld is based on the prevalence of the use of filler metal considered to be "low hydrogen." Because a $5/16$-in. (8 mm) fillet weld is the largest that can be deposited in a single pass by the SMAW process and still be considered prequalified under AWS D1.1, $5/16$ in. (8 mm) applies to all material $3/4$ in. (19 mm) and greater in thickness, but minimum preheat and interpass temperatures are required by AWS D1.1. Both the engineer of record and the shop welder must be governed by the requirements.

Table J2.3 gives the minimum effective throat thickness of a PJP groove weld. Notice that for PJP groove welds Table J2.3 goes up to a plate thickness of over 6 in. (150 mm) and a minimum weld throat of $5/8$ in. (16 mm), whereas for fillet welds Table J2.4 goes up to a plate thickness of over $3/4$ in. (19 mm) and a minimum leg size of fillet weld of only $5/16$ in. (8 mm). The additional thickness for PJP groove welds is intended to provide for reasonable proportionality between weld and material thickness.

For thicker members in lap joints, it is possible for the welder to melt away the upper corner, resulting in a weld that appears to be full size but actually lacks the required weld throat dimension. See Figure C-J2.1(a). On thinner members, the full weld throat is likely to be achieved, even if the edge is melted away.
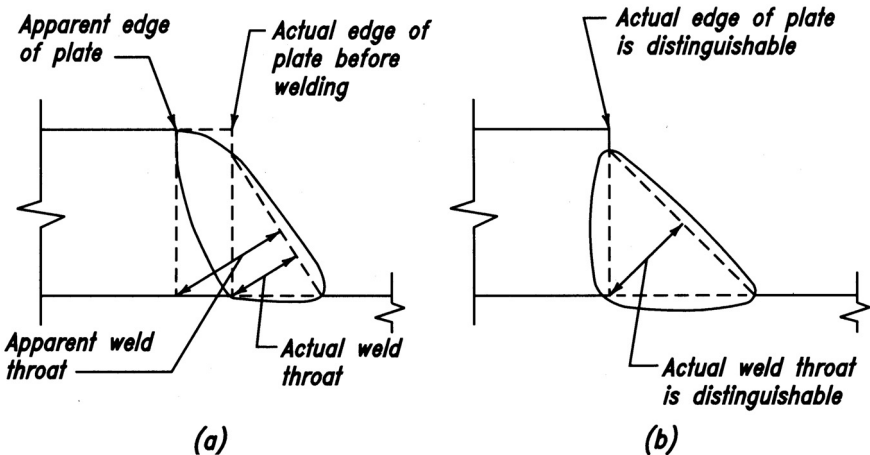

Fig. C-J2.1. Identification of plate edge.

Accordingly, when the plate is $1/4$ in. (6 mm) or thicker, the maximum fillet weld size is $1/16$ in. (2 mm) less than the plate thickness, ensuring that the edge remains behind [see Figure C-J2.1(b)].

Where longitudinal fillet welds are used alone in a connection (see Figure C-J2.2), Section J2.2b requires that the length of each weld be at least equal to the width of the connecting material because of shear lag (Freeman, 1930).

By providing a minimum lap of five times the thickness of the thinner part of a lap joint, the resulting rotation of the joint when pulled will not be excessive, as shown in Figure C-J2.3. Fillet welded lap joints under tension tend to open and apply a tearing action at the root of the weld as shown in Figure C-J2.4(b), unless restrained by a force F as shown in Figure C-J2.4(a).

End returns are not essential for developing the capacity of fillet welded connections and have a negligible effect on their strength. Their use has been encouraged to ensure that the weld size is maintained over the length of the weld, to enhance the fatigue resistance of cyclically loaded flexible end connections, and to increase the plastic deformation capability of such connections.

The weld capacity database on which the specifications were developed had no end returns. This includes the study reported in Higgins and Preece (1968), the
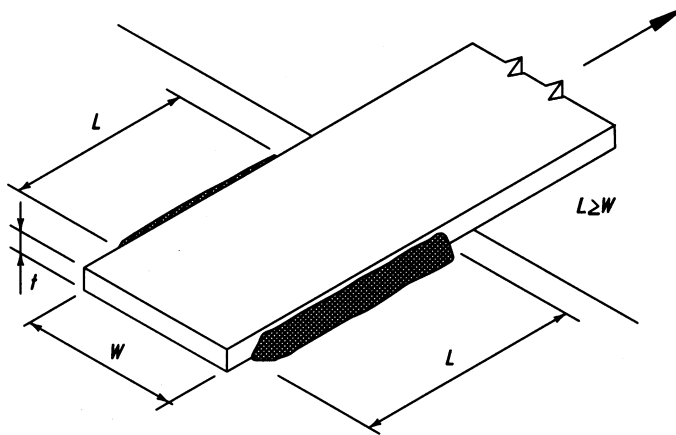

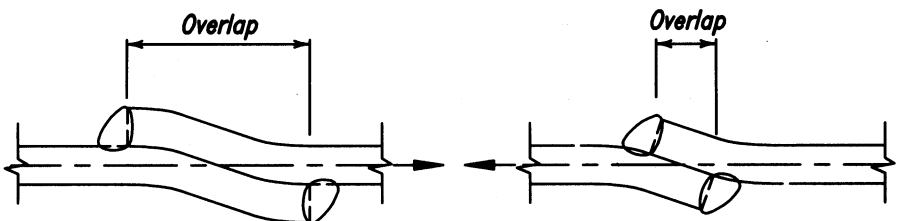*Fig. C-J2.2. Longitudinal fillet welds.*

Fig. C-J2.3. Minimum lap.

seat angle tests in Lyse and Schreiner (1935), the seat and top angle tests in Lyse and Gibson (1937), the tests on beam webs welded directly to a column or girder by fillet welds in Johnston and Deits (1942), and the tests on eccentrically loaded welded connections reported in Butler, Pal, and Kulak (1972). Hence, the current strength values and joint-capacity models do not require end returns when the required weld size is provided. Johnston and Green (1940) noted that movement consistent with the design assumption of no end restraint (in other words, joint flexibility) was enhanced without end returns. They also verified that greater plastic deformation of the connection was achieved when end returns existed, although the strength was not significantly different.

When longitudinal fillet welds parallel to the stress are used to transmit the load to the end of an axially loaded member, the welds are termed "end loaded." Typical examples of such welds include, but are not limited to: (a) longitudinally welded lap joints at the end of axially loaded members; (b) welds attaching bearing stiffeners; and (c) similar cases. Typical examples of longitudinally loaded fillet welds are not considered end loaded include, but are not limited to: (a) welds that connect plates or shapes to form built-up cross sections in which the shear force is applied to each increment of length of weld depending upon the distribution of the shear along the length of the member; and (b) welds attaching beam web connection angles and shear plates because the flow of shear force from the beam or girder web to the weld is essentially uniform throughout the weld length; that is, the weld is not end-loaded despite the fact that it is loaded parallel to the weld axis. Neither does the reduction coefficient, β, apply to welds attaching stiffeners to webs because the stiffeners and welds are not subject to calculated axial stress but merely serve to keep the web flat.

The distribution of stress along the length of end-loaded fillet welds is not uniform and is dependent upon complex relationships between the stiffness of the longitudinal fillet weld relative to the stiffness of the connected materials. Experience has shown that when the length of the weld is equal to approximately 100 times the weld size or less, it is reasonable to assume the effective length is equal to or less than the actual length. For weld lengths greater than 100 times the weld size, the effective length should be taken less than the actual length. The reduction coefficient, β, provided in Section J2.2b is the equivalent to that given in Eurocode 3 (1992), which is a simplified approximation of exponential
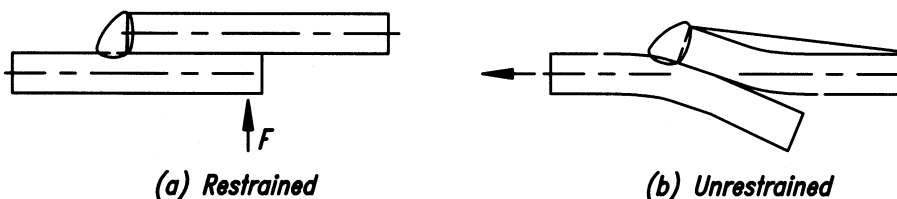

Fig. C-J2.4. Restraint of lap joints.

formulas developed by finite element studies and tests preformed in Europe over many years. The provision is based on the combined consideration of the nominal strength for fillet welds with leg size less than $^1/_4$ in. (6 mm) and of a judgment-based serviceability limit of slightly less than $^1/_{32}$ in. (1 mm) displacement at the end of the weld for welds with leg size $^1/_4$ in. (6 mm) and larger. The mathematical form of the β factor implies that the minimum strength of an end-loaded weld is achieved when the length is approximately 300 times the leg size. Because it is illogical to conclude that the total strength of a weld longer than 300 times the weld size is more than that of a shorter weld, the length reduction coefficient is taken as 0.6 when the weld length is greater than 300 times the leg size.

In most cases, fillet weld terminations do not affect the strength or serviceability of connections. However, in certain cases the disposition of welds affect the planned function of the connection, and notches may affect the static strength and/or the resistance to crack initiation if *cyclic loads* of sufficient magnitude and frequency occur. For these cases, terminations before the end of the joint are specified to provide the desired profile and performance. In cases where profile and notches are less critical, terminations are permitted to be run to the end. In most cases, stopping the weld short of the end of the joint will not reduce the strength of the weld. The small loss of weld area due to stopping the weld short of the end of the joint by one to two weld sizes is not typically considered in the calculation of weld strength. Only short weld lengths will be significantly affected by this.

The following situations require special attention:

(1) For lapped joints where one part extends beyond the end or edge of the part to which it is welded and if the parts are subject to calculated tensile stress at the start of the overlap, it is important that the weld terminate a short distance from the stressed edge. For one typical example, the lap joint between the tee chord and the web members of a truss, the weld should not extend to the edge of the tee stem (see Figure C-J2.5). The best technique to avoid inadvertent notches at this critical location is to strike the welding arc at a point slightly back from the edge and proceed with welding in the direction away from the edge (see Figure C-J2.6). Where framing angles extend beyond the end of the beam web to which they are welded, the free end of the beam web is subject to zero stress; thus, it is permissible for the fillet weld to extend continuously across the top end, along the side and along the bottom end of the angle to the extreme end of the beam (see Figure C-J2.7).

(2) For connections such as framing angles and framing tees, which are assumed in the design of the structure to be *flexible connections*, the top and bottom edges of the outstanding legs or flanges must be left unwelded over a substantial portion of their length to assure flexibility of the connection. Tests have shown that the static strength of the connection is the same with or without end returns; therefore the use of returns is optional, but if used, their length must be restricted to not more than four times the weld size (Johnston and Green, 1940) (see Figure C-J2.8).
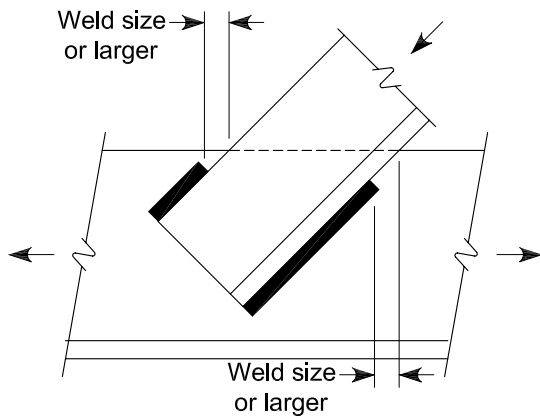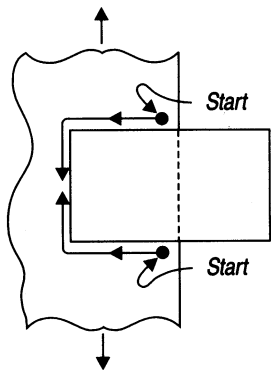
Fig. C-J2.5. Fillet welds near tension edges.

Fig. C-J2.6. Suggested direction of welding travel to avoid notches.
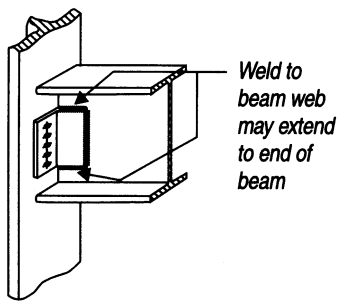
Fig. C-J2.7. Fillet weld details on framing angles.

(3) Experience has shown that when ends of intermediate transverse stiffeners on the webs of plate girders are not welded to the flanges (the usual practice), small torsional distortions of the flange occur near shipping bearing points in the normal course of shipping by rail or truck and may cause high out-of-plane bending stresses (up to the yield point) and fatigue cracking at the toe of the web-to-flange welds. This has been observed even with closely fitted stiffeners. The intensity of these out-of-plane stresses may be effectively limited and cracking prevented if "breathing room" is provided by terminating the stiffener weld away from the web-to-flange welds. The unwelded distance should not exceed six times the web thickness so that column buckling of the web within the unwelded length does not occur.

(4) For fillet welds that occur on opposite sides of a common plane, it is difficult to deposit a weld continuously around the corner from one side to the other without causing a gouge in the corner of the parts joined; therefore the welds must be interrupted at the corner (see Figure C-J2.9).

Fig. C-J2.8. Flexible connection returns optimal unless subject to fatigue.

Fig. C-J2.9. Details for fillet welds that occur on opposite sides of a common plane.

3.   **Plug and Slot Welds**
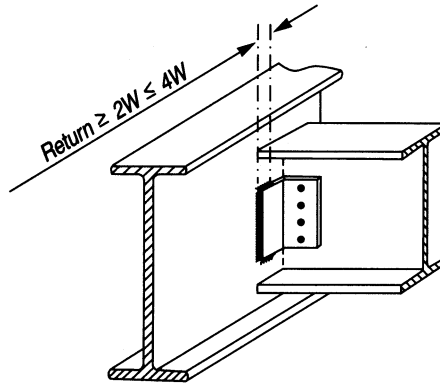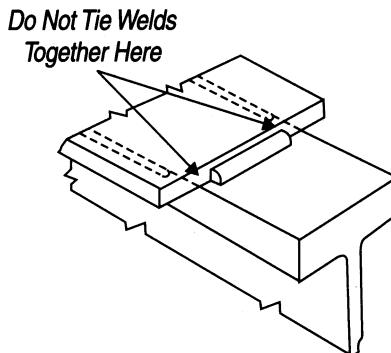
A plug weld is a weld made in a circular hole in one member of a joint fusing that member to another member. Both plug and slot welds are only applied to lap joints. Care should be taken when plug or slot welds are applied to structures subject to cyclic loading as the fatigue performance of these welds is limited. A slot weld is a weld made in an elongated hole in one member of a joint fusing that member to another member. A fillet weld inside a hole or slot is not a plug weld. A "puddle weld", typically used for joining decking to the supporting steel, is not the same as a plug weld.

3a.   **Effective Area**

When plug and slot welds are detailed in accordance with Section J2.3b, the strength of the weld is controlled by the size of the fused area between the weld and the base metal. The total area of the hole or slot is used to determine the effective area.

3b.   **Limitations**

Plug and slot welds are limited to situations where they are loaded in shear, or where they are used to prevent elements of a cross section from buckling, such as for web doubler plates on deeper rolled sections. Plug and slot welds are only allowed where the applied loads result in shear between the joined materials—they are not to be used to resist direct tensile loads.

The geometric limitations on hole and slot sizes are prescribed in order to provide a geometry that is conducive to good fusion. Deep, narrow slots and holes make it difficult for the welder to gain access and see the bottom of the cavity into which weld metal must be placed. Where access is difficult, fusion may be limited, and the strength of the connection reduced.

4.   **Strength**

The strength of welds is governed by the strength of either the base material or the deposited weld metal. Table J2.5 presents the nominal weld strengths and the ϕ and Ω factors, as well as the limitations on filler metal strength levels.

The strength of a joint that contains a complete-joint-penetration (CJP) groove weld, whether loaded in tension or compression, is dependent upon the strength of the base metal, and no computation of the strength of the CJP groove weld is required. For tension applications, matching strength filler metal is required, as defined in AWS D1.1 Table 3.1. For compression applications, up to a 10 ksi (70 MPa) decrease in filler metal strength is permitted, which is equivalent to one strength level.

CJP groove welds loaded in tension or compression parallel to the weld axis, such as for the groove welded corners of box columns, do not transfer primary loads across the joint. In cases such as this, no computation of the strength of the CJP groove weld strength is required.

CJP groove welded tension joints are intended to provide strength equivalent to the base metal, therefore matching filler metal is required. CJP groove welds have been shown not to exhibit compression failure even when they are undermatched. The amount of undermatching before unacceptable deformation occurs has not been established, but one standard strength level is conservative and therefore permitted. Joints in which the weld strength is calculated based on filler metal classification strength can be designed using any filler metal strength equal to or less than matching. Filler metal selection is still subject to compliance with AWS D1.1.

The nominal strength of partial-joint-penetration (PJP) groove welded joints in compression is higher than for other joints because compression limit states are not observed on weld metal until significantly above the yield strength.

Connections that contain PJP groove welds designed to bear in accordance with Section J1.4(b), and where the connection is loaded in compression, are not limited in capacity by the weld since the surrounding base metal can transfer compression loads. When not designed in accordance with Section J1.4(b), an otherwise similar connection must be designed considering the possibility that either the weld or the base metal may be the critical component in the connection.

The factor of 0.6 on F_{EXX} for the tensile strength of PJP groove welds is an arbitrary reduction that has been in effect since the early 1960s to compensate for the notch effect of the unfused area of the joint, uncertain quality in the root of the weld due to the inability to perform nondestructive evaluation, and the lack of a specific notch-toughness requirement for filler metal. It does not imply that the tensile failure mode is by shear stress on the effective throat, as in fillet welds.

Column splices have historically been connected with relatively small PJP groove welds. Frequently, erection aids are available to resist construction loads. Columns are intended to be in contact bearing in splices and on base plates. Section M4.4 recognizes that, in the as-fitted product, the contact may not be consistent across the joint and therefore provides rules assuring some contact that limits the potential deformation of weld metal and the material surrounding it. These welds are intended to hold the columns in place, not to transfer the compressive loads. Additionally, the effects of very small deformation in column splices are accommodated by normal construction practices. Similarly, the requirements for base plates and normal construction practice assure some bearing at bases. Therefore the compressive stress in the weld metal does not need to be considered as the weld metal will deform and subsequently stop when the columns bear. Other PJP groove welded joints connect members that may be subject to unanticipated loads and may fit with a gap. Where these connections are finished to bear, fit-up may not be as good as that specified in Section M4.4 but some bearing is anticipated so the weld is to be designed to resist loads defined in Section J1.4(b) using the factors, strengths and effective areas in Table J2.5. Where the joints

connect members that are not finished to bear, the welds are designed for the total required load using the available strengths, and areas in Table J2.5.

In Table J2.5 the nominal strength of fillet welds is determined from the effective throat area, whereas the strength of the connected parts is governed by their respective thicknesses. Figure C-J2.10 illustrates the shear planes for fillet welds and base material:

(1) Plane 1-1, in which the strength is governed by the shear strength of the material A.
(2) Plane 2-2, in which the strength is governed by the shear strength of the weld metal.
(3) Plane 3-3, in which the strength is governed by the shear strength of the material B.

The strength of the welded joint is the lowest of the strengths calculated in each plane of shear transfer. Note that planes 1-1 and 3-3 are positioned away from the fusion areas between the weld and the base material. Tests have demonstrated that the stress on this fusion area is not critical in determining the shear strength of fillet welds (Preece, 1968).

The shear planes for plug and PJP groove welds are shown in Figure C-J2.11 for the weld and base metal. Generally the base metal will govern the shear strength.

When weld groups are loaded in shear by an external load that does not act through the center of gravity of the group, the load is eccentric and will tend to cause a relative rotation and translation between the parts connected by the weld. The point about which rotation tends to take place is called the instantaneous center of rotation. Its location is dependent upon the load eccentricity, geometry of the weld group, and deformation of the weld at different angles of the resultant elemental force relative to the weld axis.

The individual strength of each unit weld element can be assumed to act on a line perpendicular to a ray passing through the instantaneous center and that element's location (see Figure C-J2.12).

The ultimate shear strength of weld groups can be obtained from the load defor-mation relationship of a single-unit weld element. This relationship was originally
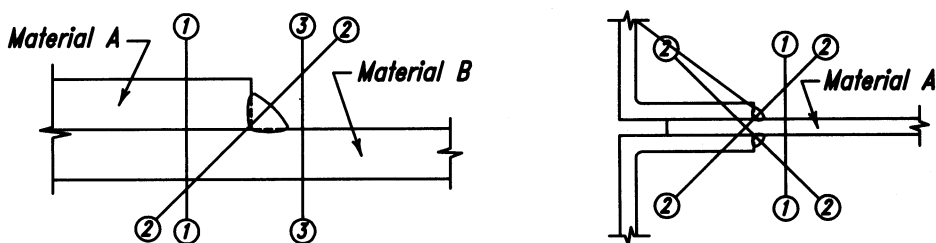
Fig. C-J2.10. Shear planes for fillet welds loaded in longitudinal shear.

given in Butler and others (1972) for E60 (E43) electrodes. Curves for E70 (E48) electrodes were reported in Lesik and Kennedy (1990).

Unlike the load-deformation relationship for bolts, strength and deformation performance in welds are dependent on the angle that the resultant elemental force makes with the axis of the weld element as shown in Figure C-J2.12. The actual load deformation relationship for welds is given in Figure C-J2.13, taken from Lesik and Kennedy (1990). Conversion of the SI equation to U.S. customary units results in the following weld strength equation for R_n:

$$R_n = 0.852(1.0 + 0.50 \sin^{1.5}\theta) F_{EXX} A_w \tag{C-J2-1}$$
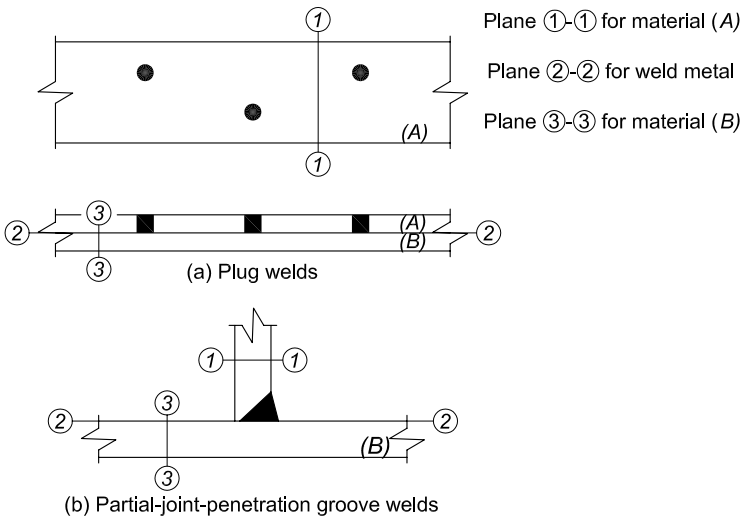
Fig. C-J2.11. Shear planes for plug and PJP groove welds.
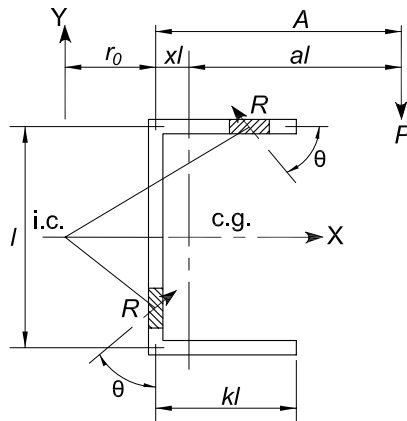
Fig. C-J2.12. Weld element nomenclature.

Because the maximum strength is limited to $0.60F_{EXX}$ for longitudinally loaded welds ($\theta = 0°$), the Specification provision provides, in the reduced equation coefficient, a reasonable margin for any variation in welding techniques and procedures. To eliminate possible computational difficulties, the maximum deformation in the weld elements is limited to $0.17w$. For design convenience, a simple elliptical formula is used for $f(p)$ to closely approximate the empirically derived polynomial in Lesik and Kennedy (1990).

The total strength of all the weld elements combine to resist the eccentric load and, when the correct location of the instantaneous center has been selected, the three in-plane equations of statics ($\Sigma F_x = 0$, $\Sigma F_y = 0$, $\Sigma M = 0$) will be satisfied. Numerical techniques, such as those given in Brandt (1982), have been developed to locate the instantaneous center of rotation subject to convergent tolerances.

5.   Combination of Welds

When determining the capacity of a combination PJP groove weld and fillet weld contained within the same joint, the total throat dimension is not the simple addition of the fillet weld throat and the groove weld throat. In such cases, the resultant throat of the combined weld (dimension from root perpendicular to face of fillet weld) must be determined and the design based upon this dimension.

Fig. C-J2.13. Load deformation relationship.

6.   Filler Metal Requirements

Applied and *residual stresses* and geometrical discontinuities from backup bars
with associated notch effects contribute to sensitivity to fracture. Additionally,
some weld metals in combination with certain procedures result in welds with
low notch toughness. Accordingly, this Specification requires a minimum spec-
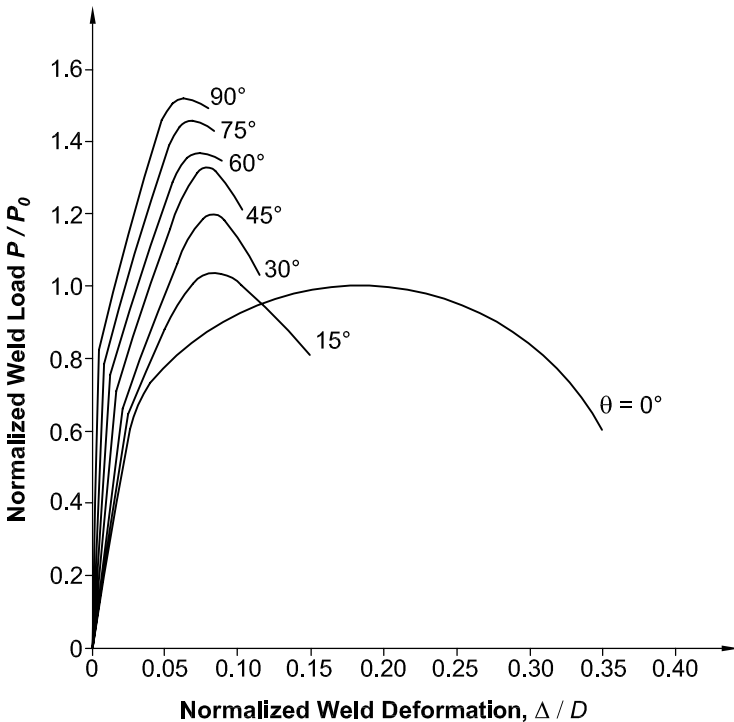ified toughness for weld metals in those joints that are subject to more signif-
icant applied stresses and toughness demands. The level of toughness required
is selected as one level more conservative than the base metal requirement for
hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm).

7.   Mixed Weld Metal

Problems can occur when incompatible weld metals are used in combination
and notch-tough composite weld metal is required. For instance, tack welds de-
posited using a self-shielded process with aluminum deoxidizers in the electrodes
and subsequently covered by SAW weld passes can result in a composite weld
metal with low notch-toughness, despite the fact that each process by itself could
provide notch-tough weld metal.

Potential concern about intermixing weld metal types is limited to situations
where one of the two weld metals is deposited by the self-shielded flux-cored
arc welding (FCAW-s) process. Changes in tensile and elongation properties
have been demonstrated to be of insignificant consequence. Notch toughness is
the property that can be affected the most. Many compatible combinations of
FCAW-s and other processes are commercially available.

J3.   BOLTS AND THREADED PARTS

1.   High-Strength Bolts

In general, the use of high-strength bolts is required to conform to the provisions
of the *Specification for Structural Joints Using ASTM A325 or A490 Bolts* (RCSC,
2004) as approved by the Research Council on Structural Connections. Kulak
(2002) provides an overview of the properties and use of high-strength bolts.

Occasionally the need arises for the use of high-strength bolts of diameters and
lengths in excess of those available for ASTM A325 or A325M and ASTM A490
or A490M bolts. For joints requiring diameters in excess of $1^1/_2$ in. (38 mm) or
lengths in excess of about 8 in. (200 mm), Section J3.1 permits the use of ASTM
A449 bolts and ASTM A354 Grade BC and BD threaded rods. Note that anchor
rods are more preferably specified as ASTM F1554 material.

Snug-tight installation is permitted for static applications involving only ASTM
A325 or A325M bolts in tension or combined shear and tension. Two studies have
been conducted to investigate possible reductions in strength because of varying
levels of pretension in bolts within the same connection. The first investigation
focused on nine, two-bolt tee stubs connected in a back-to-back configuration
using $3/_4$-in. diameter A325 bolts (Johnson, 1996). The bolt pretensions were

varied from pretensioned to snug tight to finger tight. No significant loss of strength was noted as compared to the case with both fasteners pretensioned—even with one fastener pretensioned and the other finger tight. The second study tested 32 additional two-bolt tee stubs but considered both ASTM A325 and A490 fasteners and two, four-bolt tee stubs (Amrine and Swanson, 2004). The study found that no significant loss of strength resulted from having different pretensions in bolts within the same connection, even with ASTM A490 fasteners.

There are practical cases in the design of structures where slip of the connection is desirable to allow for expansion and contraction of a joint in a controlled manner. Regardless of whether force transfer is required in the direction normal to the slip direction, the nuts should be hand-tightened with a spud wrench and then backed off one-quarter turn. Furthermore, it is advisable to deform the bolt threads or use a locking nut or jamb nut to ensure that the nut does not back off further under service conditions. Thread deformation is commonly accomplished with a cold chisel and hammer applied at one location. Note that tack-welding of the nut to the bolt threads is not recommended.

2.   Size and Use of Holes

To provide some latitude for adjustment in plumbing a frame during erection, three types of enlarged holes are permitted, subject to the approval of the designer. The nominal maximum sizes of these holes are given in Table J3.3 or J3.3M. The use of these enlarged holes is restricted to connections assembled with bolts and is subject to the provisions of Sections J3.3 and J3.4.

3.   Minimum Spacing

The minimum spacing dimensions of $2^2/_3$ times and 3 times the nominal diameter are to facilitate construction and do not necessarily satisfy the bearing and tearout strength requirements in Section J3.10.

4.   Minimum Edge Distance

The minimum edge distances given in Table J3.4 and Table J3.4M are to facilitate construction and do not necessarily satisfy the bearing and tearout strength requirements in Section J3.10. Lesser values are permitted if the requirements of Section J3.10 are satisfied.

5.   Maximum Spacing and Edge Distance

Limiting the edge distance to not more than 12 times the thickness of an outside connected part, but not more than 6 in. (150 mm), is intended to provide for the exclusion of moisture in the event of paint failure, thus preventing corrosion between the parts that might accumulate and force these parts to separate. More restrictive limitations are required for connected parts of unpainted weathering steel exposed to atmospheric corrosion.

6.   **Tension and Shear Strength of Bolts and Threaded Parts**

Tension loading of fasteners is usually accompanied by some bending due to the deformation of the connected parts. Hence, the resistance factor, ϕ, and the safety factor, Ω, are relatively conservative. The nominal tensile stress values in Table J3.2 were obtained from the equation

$$F_{nt} = 0.75 F_u \tag{C-J3-2}$$

The factor of 0.75 included in this equation accounts for the approximate ratio of the effective area of the threaded portion of the bolt to the area of the shank of the bolt for common sizes. Thus A_b is defined as the area of the unthreaded body of the bolt and the value reported for F_{nt} in Table J3.2 is calculated as $0.75 F_u$.

The tensile strength given by Equation C-J3-2 is independent of whether the bolt was initially installed pretensioned or snug-tightened. Recent tests confirm that the performance of ASTM A325 and A325M bolts in tension not subjected to fatigue are unaffected by the original installation condition (Amrine and Swanson, 2004; Johnson, 1996; Murray, Kline, and Rojani, 1992). While the equation was developed for bolted connections, it was also conservatively applied to threaded parts (Kulak and others, 1987).

For ASTM A325 or A325M bolts, no distinction is made between small and large diameters, even though the minimum tensile strength, F_u, is lower for bolts with diameters in excess of 1 in. (24 mm). It was felt that such a refinement was not justified, particularly in view of the conservative resistance factor, ϕ, and safety factor, Ω, the increasing ratio of tensile area to gross area, and other compensating factors.

The values of nominal shear stress in Table J3.2 were obtained from the following equations:

$F_{nv} = 0.50 F_u$, when threads are excluded from the shear planes   (C-J3-3)

$F_{nv} = 0.40 F_u$, when threads are not excluded from the shear plane   (C-J3-4)

The factors 0.50 and 0.40 account for the effect of shear and for the reduced area of the threaded portion of the fastener when the threads are not excluded from the shear plane. When determining the shear strength of a fastener, the area, A_b, is multiplied by the number of shear planes. While developed for bolted connections, the equations were also conservatively applied to threaded parts. The value given for ASTM A307 bolts was obtained from Equation C-J3-4 but is specified for all cases regardless of the position of threads.

In connections consisting of only a few fasteners, the effect of differential strain on the shear in bearing fasteners is negligible (Kulak and others, 1987; Fisher, Galambos, Kulak, and Ravindra, 1978). In longer joints, the differential strain produces an uneven distribution of load between fasteners, those near the end taking a disproportionate part of the total load, so that the maximum strength per

fastener is reduced. This Specification does not limit the length but requires a 20 percent reduction in shear strength for connections longer than 50 in. (1.2 m). The resistance factor, ϕ, and the safety factor, Ω, for shear in bearing-type connections already accommodate the effects of differential strain in connections less than 50 in. (1.2 m) in length. The above discussion is primarily applicable to end-loaded connections, but is applied to all connections to maintain simplicity.

Additional information regarding the development of the provisions in this section can be found in the Commentary to the RCSC *Specification* (RCSC, 2004).

7.   Combined Tension and Shear in Bearing-Type Connections

Tests have shown that the strength of bearing fasteners subject to combined shear and tension resulting from externally applied forces can be closely defined by an ellipse (Kulak and others, 1987). The relationship is expressed as

$$\left(\frac{f_t}{\phi F_{nt}}\right)^2 + \left(\frac{f_v}{\phi F_{nv}}\right)^2 = 1 \qquad \text{(LRFD)} \qquad\qquad \text{(C-J3-5a)}$$

$$\left(\frac{\Omega f_t}{F_{nt}}\right)^2 + \left(\frac{\Omega f_v}{F_{nv}}\right)^2 = 1 \qquad \text{(ASD)} \qquad\qquad \text{(C-J3-5b)}$$

In these equations, f_v and f_t are the required shear stress and tensile stress, respectively, and F_{nv} and F_{nt} are the nominal shear and tensile stresses, respectively. The elliptical relationship can be replaced, with only minor deviations, by three straight lines as shown in Figure C-J3.1. The sloped portion of the straight-line representation is

$$\left(\frac{f_t}{\phi F_{nt}}\right) + \left(\frac{f_v}{\phi F_{nv}}\right) = 1.3 \qquad \text{(LRFD)} \qquad\qquad \text{(C-J3-6a)}$$

$$\left(\frac{\Omega f_t}{F_{nt}}\right) + \left(\frac{\Omega f_v}{F_{nv}}\right) = 1.3 \qquad \text{(ASD)} \qquad\qquad \text{(C-J3-6b)}$$

which results in Equations J3-3a and J3-3b.

This latter representation offers the advantage that no modification of either type of stress is required in the presence of fairly large magnitudes of the other type. Note that Equations J3-3a and J3-3b can be rewritten so as to find the nominal shear strength per unit area, F'_{nv}, as a function of the required tensile stress, f_t. These formulations are

$$F'_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \le F_{nv} \qquad \text{(LRFD)} \qquad\qquad \text{(C-J3-7a)}$$

$$F'_{nt} = 1.3 F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \le F_{nt} \qquad \text{(ASD)} \qquad\qquad \text{(C-J3-7b)}$$

The linear relationship was adopted for use in Section J3.7; generally, use of the elliptical relationship is acceptable (see Figure C-J3.1). A similar formulation

using the elliptical solution is

$$F'_{nt} = F_{nt}\sqrt{1 - \left(\frac{f_v}{\phi F_{nv}}\right)^2} \le F_{nt} \qquad \text{(LRFD)} \qquad \text{(C-J3-8a)}$$

$$F'_{nt} = F_{nt}\sqrt{1 - \left(\frac{\Omega f_{nv}}{F_{nv}}\right)^2} \le F_{nt} \qquad \text{(ASD)} \qquad \text{(C-J3-8b)}$$

8.   High-Strength Bolts in Slip-Critical Connections

Connections should be classified as slip-critical only when the slip is deemed by the engineer of record to affect the serviceability of the structure by excessive distortion or cause a reduction in strength or stability even though the available strength of the connection is adequate. For example, connections subject to fatigue and connections with oversized holes or slots parallel to the direction of load should be designed as slip-critical. Most connections with standard holes can be designed as bearing-type connections without concern for serviceability. For connections with three or more bolts in standard holes or slots perpendicular to the direction of force, the freedom to slip generally does not exist because one or more of the bolts are in bearing before the load is applied.

Slip resistance in bolted connections has traditionally been viewed as a serviceability limit state and these connections have been designed to resist slip due to load effects from serviceability combinations and checked as bearing connections due to load effects from strength load combinations. There are conditions,
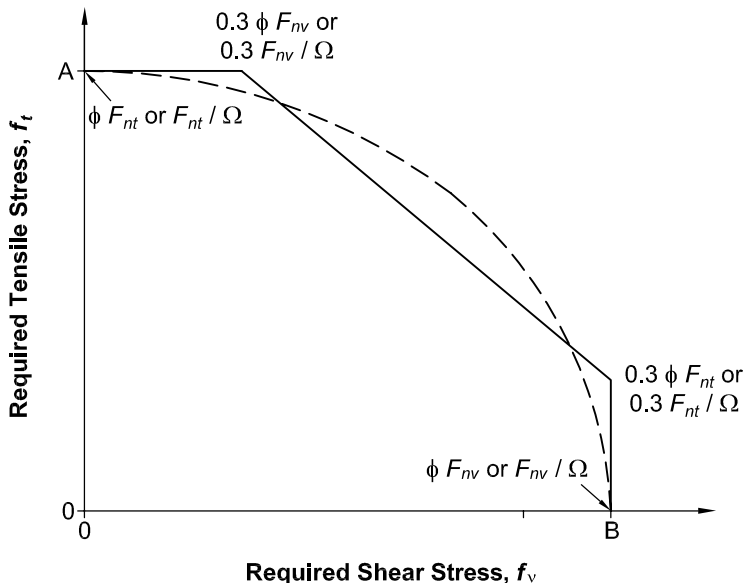

Fig. C-J3.1. Straight-line representation of elliptical solution.

however, where the deformations due to slip in connections with oversized holes and slotted holes parallel to the load could result in an increased load larger than the strength limit state. Examples where the usual assumption of serviceability-governed slip resistance may not apply are:

- High aspect ratio braced frames where the slip permitted by slots or oversized holes is relatively large and could potentially result in large P-Δ effects;
- Long-span, flat roof trusses with oversized holes, where slip could result in excessively large loads due to ponding;
- Built-up compression members where slip between the individual element ends could increase the member effective length and thus significantly reduce buckling strength;
- Any condition where the normal analysis assumption of an undeformed structure (small deflections) could be violated by connection slip resulting in increased load.

The Commentary to the 1999 *LRFD Specification* (AISC, 2000b) cautioned engineers about such conditions when utilizing long-slotted holes parallel to the direction of the load:

> If the connection is designed so that it will not slip under the effects of service loads, then the effect of the factored loads on the deformed structure (deformed by the maximum amount of slip in the long slots at all locations) must be included in the structural analysis. Alternatively, the connection can be designed so that it will not slip at loads up to the factored load level.

However, neither the 1999 *LRFD Specification* (AISC, 2000b) nor its Commentary provided the engineer with any guidance for designing to prevent slip at the factored load level. Since the AISC *LRFD Manual of Steel Construction*, 3$^{\text{rd}}$ edition (AISC, 2001) also provided two separate design aids, Tables 7-15 and 7-16, one that indicated the use of service load combinations and one that indicated the use of strength load combinations, it was sometimes believed that the use of Table 7-15 would guard against slip due to load effects from service load combinations and the use of Table 7-16 would guard against slip due to load effects from strength load combinations. These are incorrect interpretations as both tables lead to the same final result, that is, to prevent slip due to load effects from service load combinations.

The Commentary to the 1999 *LRFD Specification* (AISC, 2000b) states, "Slip of slip-critical connections is likely to occur at approximately 1.4 to 1.5 times the service load." This is based on the use of a resistance factor $\phi = 1.00$, standard holes, and calibrated wrench installation. The use of $\phi = 0.85$ for oversized and short-slotted holes and $\phi = 0.70$ for long-slotted holes perpendicular and $\phi = 0.60$ for long-slotted holes parallel to the load, increases this resistance to approximately 1.7 times the service load for oversized and short-slotted holes

and even greater slip resistance for long-slotted holes. The use of the turn-of-the-nut installation method also increases slip resistance by approximately 10 to 15 percent. Hence connections with oversized and slotted holes, even when designed for the serviceability limit state provisions of the 1999 *LRFD Specification* (AISC, 2000b), will resist slip due to load effects from the strength load combinations.

Determining Required Resistance to Slip. This Specification permits all slip-critical connections with bolts in standard holes or in slotted holes perpendicular to the direction of the force to be designed as being governed by serviceability. The slight variations in geometry, which can occur due to connection slip when using this type of hole, will not change the normal analysis assumptions or result in an increase in load.

The fundamental design requirement for all connections with bolts in oversized holes and slotted holes parallel to the load is to prevent slip at the strength limit state, which conservatively assumes that the corresponding potential for change in geometry will not be negligible and that connection slip will result in significant load increase.

The engineer of record is permitted to make the determination that the effect of slip will not result in increased loads and, therefore, to design any slip-critical connection for the serviceability limit state. In either case, the design slip resistance is calculated using the load effects from either the LRFD load combinations or the ASD load combinations and the appropriate resistance factor, ϕ, or safety factor, Ω. All slip-critical connections, whether designed for the serviceability or strength limit state, must be checked for shear and bearing using the appropriate design loads.

The reliability required when designing to resist slip due to load effects from strength load combinations is subject to some interpretation. Traditionally, connection limit states require a β for bolts and fillet welds of 4.0. This is because many limit states associated with connection failure are associated with a sudden, nonductile joint separation. Since connection slip will not result in a sudden separation of the joint as long as the connection is checked as a bearing-type connection due to load effects from strength load combinations, knowing the exact level of reliability for slip resistance due to strength load combinations is not critical to connection performance. Resistance and safety factors along with the hole factors proposed for oversized holes and slotted holes approach those necessary to achieve a reliability index of 4.0. However, because of the complex factors involved in calculating the reliability of slip-critical connections and the lack of extensive statistical data on slip resistance of oversized and slotted holes, the checks for bearing and shear due to strength load combinations are required for both design methods.

Factors that Affect Slip Resistance of Joints. The following paragraphs outline the key factors affecting slip resistance in bolted steel connections:

Slip Coefficient of the Faying Surface. This Specification has combined the previous Class A and Class C surfaces into a single Class A surface category that includes unpainted clean mill scale surfaces or surfaces with Class A coatings on a blasted-cleaned surface, and hot-dip galvanized and roughened surfaces with a coefficient of friction $\mu = 0.35$. This is a slight increase in value from the previous Class A coefficient. Class B surfaces, unpainted blast-cleaned surfaces, or surfaces with Class B coatings on blast-cleaned steel remain the same at $\mu = 0.50$.

Pretensioning Method and D_u. Four bolt pretensioning methods are recognized by AISC: turn-of-the-nut, calibrated wrench, twist-off type tension-control bolt assemblies, and direct tension indicating assemblies. The mean pretension force in the bolts varies according to the method of installation. The lowest mean value is when the calibrated wrench method is used: 1.13 times the minimum specified. The turn-of-the-nut method results in a mean pretension of 1.22 to 1.35 times the minimum specified, depending on the amount the bolt is turned and the bolt grade. While the statistical information on the mean pretension level of bolts installed in the field using direct tension indicators and twist-off type tension-control bolt assemblies is limited, tests indicate they will fall somewhere between the calibrated wrench and the turn-of-the-nut method. Thus, this specification uses the minimum of these values, 1.13, for all methods of installation. This results in varying reliabilities for differing conditions. Regardless of the method used to pretension the bolts, it is important that the installation of slip-critical connections meet all of the requirements listed in the RCSC *Specification* (RCSC, 2004).

Hole Size. High-strength bolts properly installed in oversized and short-slotted holes using washers as specified in the RCSC *Specification* (RCSC, 2004) have the same resistance to slip as similar bolts in standard holes. The hole factor, $h_{sc} = 0.85$, is used to increase resistance to slip for this type of connection because of the possible consequences of increased movement with these connections. The hole factor for long-slotted holes, $h_{sc} = 0.70$, serves both to increase slip resistance for this type of connection similar to the oversized holes and to compensate for a slight loss in pretension and slip resistance due to the length of a long slot. Previous editions of the Specification had a further reduction in the hole factor, $h_{sc} = 0.60$, for slots parallel to the direction of the load. This was, in effect, a design for a strength limit state for this type of hole and the same result is achieved using the ϕ or Ω factors given in this Specification.

9. **Combined Tension and Shear in Slip-Critical Connections**

The slip resistance of a slip-critical connection is reduced if there is applied tension. The factor, k_s, is a multiplier that reduces the nominal slip resistance given by Equation J3-4 as a function of the applied tension load.

10. **Bearing Strength at Bolt Holes**

Provisions for bearing strength of pins differ from those for bearing strength of bolts; refer to Section J7.

Bearing strength values are provided as a measure of the strength of the material upon which a bolt bears, not as a protection to the fastener, which needs no such protection. Accordingly, the same bearing value applies to all joints assembled by bolts, regardless of fastener shear strength or the presence or absence of threads in the bearing area.

Material bearing strength may be limited either by bearing deformation of the hole or by tearout (a bolt-by-bolt block shear rupture) of the material upon which the bolt bears. Kim and Yura (1996) and Lewis and Zwerneman (1996) confirmed the bearing strength provisions for the bearing case wherein the nominal bearing strength R_n is equal to $CdtF_u$ and C is equal to 2.4, 3.0 or 2.0 depending upon hole type and/or acceptability of hole ovalization at ultimate load, as indicated in Section J3.10. However, this same research indicated the need for different bearing strength provisions when tearout failure would control. Appropriate equations for bearing strength as a function of clear distance L_c are therefore provided and this formulation is consistent with that in the RCSC *Specification* (RCSC, 2004).

Frank and Yura (1981) demonstrated that hole elongation greater than $1/4$ in. (6 mm) will generally begin to develop as the bearing force is increased beyond $2.4dtF_u$, especially if it is combined with high tensile stress on the net section, even though rupture does not occur. For a long-slotted hole with the slot perpendicular to the direction of force, the same is true for a bearing force greater than $2.0dtF_u$. An upper bound of $3.0dtF_u$ anticipates hole ovalization [deformation greater than $1/4$ in. (6 mm)] at maximum strength.

Additionally, to simplify and generalize such bearing strength calculations, the current provisions have been based upon a clear-distance formulation. Previous provisions utilized edge distances and bolt spacings measured to hole centerlines with adjustment factors to account for varying hole type and orientation, as well as minimum edge distance requirements.

11.   Tension Fasteners

With any connection configuration where the fasteners transmit a tensile force to the HSS wall, a rational analysis must be used to determine the appropriate limit states. These may include a yield-line mechanism in the HSS wall and/or pullout through the HSS wall, in addition to applicable limit states for the fasteners subject to tension.

**J4.   AFFECTED ELEMENTS OF MEMBERS
        AND CONNECTING ELEMENTS**

Sections J4 and J5 of previous editions of the Specification have been combined in Section J4.

1.   Strength of Elements in Tension

Tests have shown that yielding will occur on the gross section before the tensile capacity of the net section is reached if the ratio A_n/A_g is greater than or equal to

0.85 (Kulak and others, 1987). Since the length of connecting elements is small compared to the member length, inelastic deformation of the gross section is limited. Hence, the effective net area A_n of the connecting element is limited to $0.85A_g$ in recognition of the limited capacity for inelastic deformation, and to provide a reserve capacity.

2.   Strength of Elements in Shear

In previous editions of the LRFD Specifications, the resistance factor for shear yielding had been 0.90, equivalent to a safety factor of 1.67. In ASD, the allowable shear yielding stress was $0.4F_y$, equivalent to a safety factor of 1.5. To make the LRFD approach in this Specification consistent with prior editions of the *ASD Specification*, the resistance and safety factors for shear yielding in this Specification are 1.0 and 1.5, respectively. The resulting increase in LRFD design strength of approximately 10 percent is justified by the long history of satisfactory performance of ASD use.

3.   Block Shear Strength

Tests on coped beams indicated that a tearing failure mode (rupture) can occur along the perimeter of the bolt holes as shown in Figure C-J4.1 (Birkemoe and Gilmor, 1978). This block shear mode combines tensile failure on one plane and shear failure on a perpendicular plane. The failure path is defined by the centerlines of the bolt holes.

The block shear failure mode is not limited to coped ends of beams; other examples are shown in Figures C-J4.1 and C-J4.2. The block shear failure mode must also be checked around the periphery of welded connections.

This Specification has adopted a conservative model to predict block shear strength. The mode of failure in coped beam webs and angles is different than that of gusset plates because the shear resistance is present on only one plane, in which case there must be some rotation of the block of material that is providing the total resistance. Although tensile failure is observed through the net section
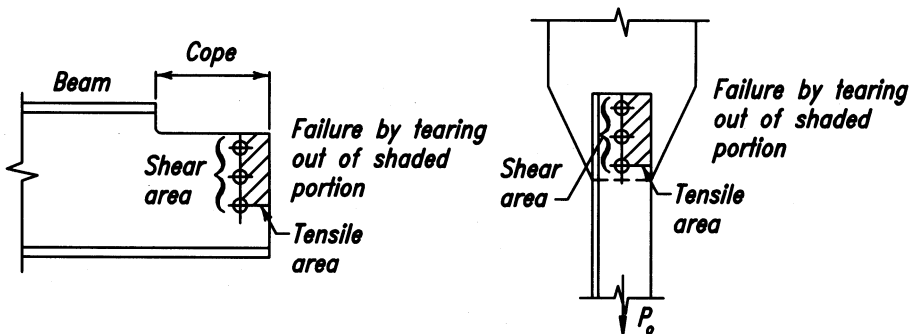
Fig. C-J4.1. Failure surface for block shear rupture limit state.

on the end plane, the distribution of tensile stresses is not always uniform (Ricles and Yura, 1983; Kulak and Grondin, 2001a). A reduction factor, U_{bs}, has been included in Equation J4-5 to approximate the non-uniform stress distribution on the tensile plane. The tensile stress distribution is non-uniform in the two row connection in Figure C-J4.2(b) because the rows of bolts nearest the beam end pick up most of the shear load.

Block shear is a rupture or tearing phenomenon, not a yielding limit state. However, gross yielding on the shear plane can occur when tearing on the tensile plane commences if $0.6F_u A_{nv}$ exceeds $0.6F_y A_{gv}$. Hence, Equation J4-5 limits the term $0.6F_y A_{gv}$ to not greater than $0.6F_u A_{nv}$. Equation J4-5 is consistent with the philosophy in Chapter D for tension members where the gross area is used for the limit state of yielding and the net area is used for the limit state of rupture.
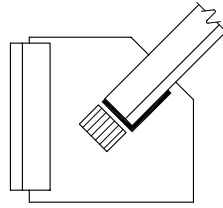
Welded Angle
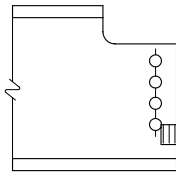
Single-row beam
end connections
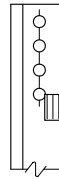
Angle Ends
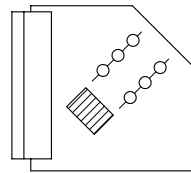
Gusset Plates

(a) Cases for which $U_{bs} = 1.0$
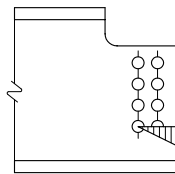
Multiple-row beam
end connections

(b) Case for which $U_{bs} = 0.5$

Fig. C-J4.2. Block shear tensile stress distributions.

4. Strength of Elements in Compression

To simplify connection calculations, the nominal strength of elements in compression when the element slenderness ratio is not greater than 25 is $F_y A_g$, which is a very slight increase over that obtained if the provisions of Chapter F are used. For more slender elements, the provisions of Chapter F apply.

J5. FILLERS

The practice of securing fillers by means of additional fasteners, so that they are, in effect, an integral part of a shear-connected component, is not required where a connection is designed for slip at member required strength levels. In such connections, the resistance to slip between the filler and either connected part is comparable to that which would exist between the connected parts if no filler were present.

Filler plates may be used in lap joints of welded connections that splice parts of different thickness, or where there may be an offset in the joint.

J6. SPLICES

The nominal strength of the smaller plate must be developed when groove-welded splices are used in plate girders and beams. For other connections it is sufficient to provide a connection to resist the required force at the joint.

J7. BEARING STRENGTH

In general, the bearing strength design of milled surfaces is governed by the limit state of bearing (local compressive yielding) at nominal loads, resulting in a stress of $0.9F_y$. Adequate safety is provided by post-yield strength as deformation increases. Tests on pin connections (Johnston, 1939) and on rockers (Wilson, 1934) have confirmed this behavior.

As used throughout the Specification, the terms "milled surface," "milled" and "milling" are intended to include surfaces that have been accurately sawed or finished to a true plane by any suitable means.

J8. COLUMN BASES AND BEARING ON CONCRETE

The provisions of this section are identical to equivalent provisions in ACI 318 (ACI, 2002).

J9. ANCHOR RODS AND EMBEDMENTS

The term "anchor rod" is used for threaded rods embedded in concrete to anchor structural steel. The term "rod" is intended to clearly indicate that these are threaded rods, not structural bolts, and should be designed as threaded parts per Table J3.2 using the material specified in Section A3.4.

Generally, the largest tensile force for which anchor rods need to be designed is that produced by bending moment at the column base and augmented by any uplift caused by the overturning tendency of a building under lateral load.

Shear at the base of a column is seldom resisted by bearing of the column base plate against the anchor rods. Even considering the lowest conceivable slip coefficient, the friction due to the vertical load on a column is generally more than sufficient to result in the transfer by frictional resistance of any likely amount of shear from the column base to the foundation. The possible exception is at the base of braced frames and moment frames where larger shear forces may require that shear transfer be accomplished by embedding the column base or providing a shear key at the top of the foundation.

The anchor rod hole sizes listed in Tables C-J9.1 and C-J9.1M are recommended to accommodate the tolerance required for setting anchor rods cast in concrete. These larger hole sizes are not detrimental to the integrity of the supported structure when used with proper washers. The slightly conical hole that results from punching operations or thermal cutting is acceptable.

If plate washers are utilized to resolve horizontal shear, bending in the anchor rod must be considered in the design and the layout of anchor rods must accommodate plate washer clearances. In this case special attention must be given to weld clearances, accessibility, edge distances on plate washers, and the effect of the tolerances between the anchor rod and the edge of the hole.

It is important that the placement of anchor rods be coordinated with the placement and design of reinforcing steel in the foundations as well as the design and overall size of base plates. It is recommended that the anchorage device at the anchor rod bottom be as small as possible to avoid interference with the reinforcing steel in the foundation. A heavy-hex nut or forged head is adequate to develop the concrete shear cone. See DeWolf and Ricker (1990) for design of base plates and anchor rods along with ACI 318 (ACI, 2002) and ACI 349 (ACI, 2001) for embedment design. Also see OSHA *Safety and Health Regulations for Construction*, Standards—29 CFR 1926 Subpart R—Steel Erection (OSHA, 2001) for anchor rod design and construction requirements for erection safety.

J10.   FLANGES AND WEBS WITH CONCENTRATED FORCES

This Specification separates flange and web strength requirements into distinct categories representing different limit states, namely, flange local bending (Section J10.1), web local yielding (Section J10.2), web crippling (Section J10.3), web *sidesway buckling* (Section J10.4), web compression buckling (Section J10.5), and web panel-zone shear (Section J10.6).

These limit state provisions are applied to two distinct types of concentrated forces normal to member flanges:

TABLE C-J9.1
Anchor Rod Hole Diameters, in.

Anchor Rod Diameter	Anchor Rod Hole Diameter
$1/2$	$1^1/_{16}$
$5/8$	$1^3/_{16}$
$3/4$	$1^5/_{16}$
$7/8$	$1^9/_{16}$
1	$1^{13}/_{16}$
$1^1/_4$	$2^1/_{16}$
$1^1/_2$	$2^5/_{16}$
$1^3/_4$	$2^3/_4$
≥ 2	$d_b + 1^1/_4$

TABLE C-J9.1M
Anchor Rod Hole Diameters, mm.

Anchor Rod Diameter	Anchor Rod Hole Diameter
18	32
22	36
24	42
27	48
30	51
33	54
36	60
39	63
42	74

Single concentrated forces may be tensile (such as those delivered by tension hangers) or compressive (such as those delivered by bearing plates at beam interior positions, reactions at beam ends, and other bearing connections). Flange local bending applies only for tensile forces, web local yielding applies to both tensile and compressive forces, and the remainder of these limit states apply only to compressive forces. Double concentrated forces, one tensile and one compressive, form a couple on the same side of the loaded member, such as that delivered to column flanges through welded and bolted moment connections.

Transverse stiffeners, also called continuity plates, and web doubler plates are only required when the demand (the transverse concentrated force) exceeds the available strength. It is often more economical to choose a heavier member than to provide such reinforcement (Carter, 1999; Troup, 1999). The demand may be determined as the largest flange force from the various load cases, although the demand may also be taken as the gross area of the attachment delivering the force multiplied by the specified minimum yield strength, F_y. Stiffeners and/or doublers and their attaching welds are sized for the difference between the demand and the applicable limit state strength. Requirements for stiffeners are provided

in Sections J10.7 and J10.8 and requirements for doublers are provided in Section J10.9.

1.   Flange Local Bending

Where a tensile force is applied through a plate welded across a flange, that flange must be sufficiently rigid to prevent deformation of the flange and the corresponding high stress concentration in the weld in line with the web.

The effective column flange length for local flange bending is $12t_f$ (Graham, Sherbourne, Khabbaz, and Jensen, 1960). Thus, it is assumed that yield lines form in the flange at $6t_f$ in each direction from the point of the applied concentrated force. To develop the fixed edge consistent with the assumptions of this model, an additional $4t_f$, and therefore a total of $10t_f$, is required for the full flange-bending strength given by Equation J10-1. In the absence of applicable research, a 50 percent reduction has been introduced for cases wherein the applied concentrated force is less than $10t_f$ from the member end.

The strength given by Equation J10-1 was originally developed for moment connections but also applies to single concentrated forces such as tension hangers consisting of a plate welded to the bottom flange of a beam and transverse to the beam web. In the original tests, the strength given by Equation J10-1 was intended to provide a lower bound to the force required for weld fracture, which was aggravated by the uneven stress and strain demand on the weld caused by the flange deformation (Graham, Sherbourne, and Khabbaz, 1959).

Recent tests on welds with minimum Charpy V-Notch (CVN) toughness requirements show that weld fracture is no longer the failure mode when the strength given by Equation J10-1 is exceeded. Rather, it was found that the strength given by Equation J10-1 is consistently less than the force required to separate the flanges in typical column sections by $1/4$ in. (6 mm) (Hajjar, Dexter, Ojard, Ye, and Cotton, 2003; Prochnow, Ye, Dexter, Hajjar, and Cotton, 2000). This amount of flange deformation is on the order of the tolerances in ASTM A6, and it is believed that if the flange deformation exceeded this level it could be detrimental to other aspects of the performance of the member, such as flange local buckling. Although this deformation could also occur under compressive normal forces, it is customary that flange local bending is checked only for tensile forces (because the original concern was weld fracture). Therefore it is not required to check flange local bending for compressive forces.

The provision in Section J10.1 is not applicable to moment end-plate and tee-stub type connections. For these connections, see Carter (1999) or the AISC *Manual of Steel Construction* (AISC, 2005a).

2.   Web Local Yielding

The web local yielding provisions (Equations J10-2 and J10-3) apply to both compressive and tensile forces of bearing and moment connections. These

provisions are intended to limit the extent of yielding in the web of a member into which a force is being transmitted. The provisions are based on tests on two-sided directly welded girder-to-column connections (cruciform tests) (Sherbourne and Jensen, 1957) and were derived by considering a stress zone that spreads out with a slope of 2:1. Graham and others (1960) report pull-plate tests and suggest that a 2.5:1 stress gradient would be more appropriate.

Recent tests confirm that the provisions given by Equations J10-2 and J10-3 are slightly conservative and that the yielding is confined to a length consistent with the slope of 2.5:1 (Hajjar and others, 2003; Prochnow and others, 2000).

3.   Web Crippling

The web crippling provisions (Equations J10-4 and J10-5) apply only to compressive forces. Originally, the term "web crippling" was used to characterize phenomena now called local web yielding, which was then thought to also adequately predict web crippling. The first edition of the AISC *LRFD Specification* (AISC, 1986) was the first AISC Specification to distinguish between local web yielding and local web crippling. Web crippling was defined as crumpling of the web into buckled waves directly beneath the load, occurring in more slender webs, whereas web local yielding is yielding of that same area, occurring in stockier webs.

Equations J10-4 and J10-5 are based on research reported in Roberts (1981). The increase in Equation J10-5b for $N/d > 0.2$ was developed after additional testing to better represent the effect of longer bearing lengths at ends of members (Elgaaly and Salkar, 1991). All tests were conducted on bare steel beams without the expected beneficial contributions of any connection or floor attachments. Thus, the resulting provisions are considered conservative for such applications. Kaczinski, Schneider, Dexter, and Lu (1994) reported tests on cellular box beams with slender webs and confirmed that these provisions are appropriate in this type of structure as well.

The equations were developed for bearing connections but are also generally applicable to moment connections.

The web crippling phenomenon has been observed to occur in the web adjacent to the loaded flange. For this reason, a half-depth stiffener (or stiffeners) or a half-depth doubler plate is needed to eliminate this limit state.

4.   Web Sidesway Buckling

The web *sidesway buckling* provisions (Equations J10-6 and J10-7) apply only to compressive forces in bearing connections and do not apply to moment connections. The web *sidesway buckling* provisions were developed after observing several unexpected failures in tested beams (Summers and Yura, 1982; Elgaaly, 1983). In those tests the compression flanges were braced at the concentrated

load, the web was subjected to compression from a concentrated load applied to the flange and the tension flange buckled (see Figure C-J10.1).

Web *sidesway buckling* will not occur in the following cases:

(a) For flanges restrained against rotation (such as when connected to a slab), when

$$\frac{h/t_w}{l/b_f} > 2.3 \qquad \text{(C-J10-1)}$$

(b) For flanges *not* restrained against rotation, when

$$\frac{h/t_w}{l/b_f} > 1.7 \qquad \text{(C-J10-2)}$$

where l is as shown in Figure C-J10.2.

Web *sidesway buckling* can be prevented by the proper design of lateral bracing or stiffeners at the load point. It is suggested that local bracing at both flanges be designed for 1 percent of the concentrated force applied at that point. If stiffeners are used, they must extend from the load point through at least one-half the beam or girder depth. In addition, the pair of stiffeners must be designed to carry the full load. If flange rotation is permitted at the loaded flange, neither stiffeners nor doubler plates are effective.

5.   **Web Compression Buckling**

The web compression buckling provision (Equation J10-8) applies only when there are compressive forces on both flanges of a member at the same cross section, such as might occur at the bottom flange of two back-to-back moment connections under gravity loads. Under these conditions, the member web must have its slenderness ratio limited to avoid the possibility of buckling. Equation J10-8 is applicable to a pair of moment connections, and to other pairs of compressive forces applied at both flanges of a member, for which N/d is approximately less than 1. When N/d is not small, the member web should be designed as a compression member in accordance with Chapter E.

Equation J10-8 is predicated on an interior member loading condition. In the absence of applicable research, a 50 percent reduction has been introduced for cases wherein the compressive forces are close to the member end.
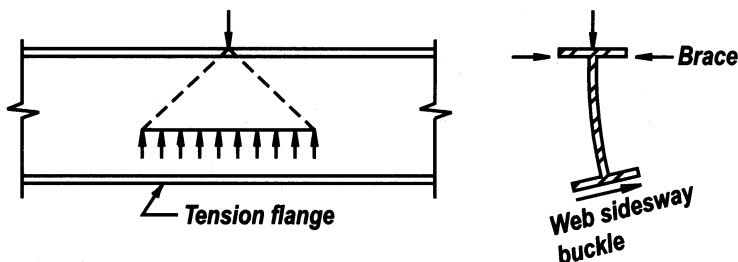
Fig. C-J10.1. Web sidesway buckling.

6.   Web Panel-Zone Shear

Column web shear stresses may be significant within the boundaries of the rigid connection of two or more members with their webs in a common plane. Such webs must be reinforced when the required force ΣF_u for LRFD or ΣF for ASD along plane A-A in Figure C-J10.3 exceeds the column web available strength ϕR_v or R_v/Ω, respectively, where

for LRFD

$$\Sigma F_u = \frac{M_{u1}}{d_{m1}} + \frac{M_{u2}}{d_{m2}} - V_u \qquad \text{(C-J10-3a)}$$

and

$M_{u1} = M_{u1L} + M_{u1G} =$ sum of the moments due to the factored lateral loads, M_{u1L}, and the moments due to factored gravity loads, M_{u1G}, on the windward side of the connection, kip-in. (N-mm)

$M_{u2} = M_{u2L} - M_{u2G} =$ difference between the moments due to the factored lateral loads M_{u2L} and the moments due to factored gravity loads, M_{u2G}, on the windward side of the connection, kip-in. (N-mm)

for ASD

$$\Sigma F = \frac{M_{a1}}{d_{m1}} + \frac{M_{a2}}{d_{m2}} - V \qquad \text{(C-J10-3b)}$$
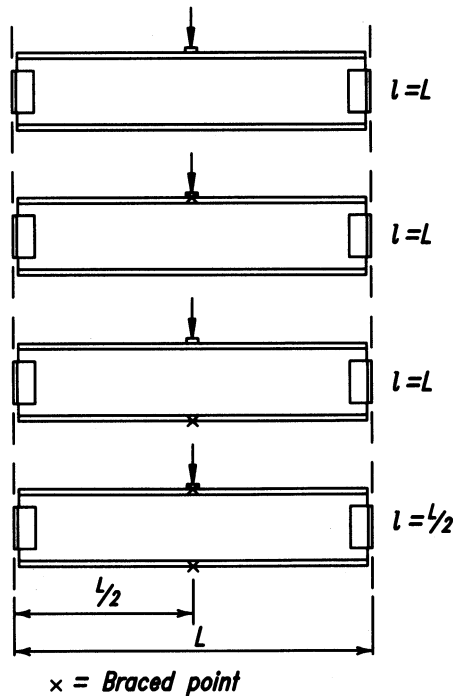
Fig. C-J10.2. Unbraced flange length for web sidesway buckling.

and

$M_{a1} = M_{a1L} + M_{a1G}$ = sum of the moments due to the nominal lateral loads, M_{a1L}, and the moments due to nominal gravity loads, M_{aG}, on the windward side of the connection, kip-in. (N-mm)

$M_{a2} = M_{a2L} + M_{a2G}$ = difference between the moments due to the nominal lateral loads, M_{a2L}, and the moments due to nominal gravity loads, M_{a2G}, on the windward side of the connection, kip-in. (N-mm)

d_{m1}, d_{m2} = distance between flange forces in the moment connection, in. (mm)

Historically (and conservatively), 0.95 times the beam depth has been used for d_m.

If, for LRFD $\Sigma F_u \leq \phi R_v$, or for ASD $\Sigma F \leq R_v / \Omega$, no reinforcement is necessary, in other words, $t_{req} \leq t_w$, where t_w is the column web thickness.

Equations J10-9 and J10-10 limit panel-zone behavior to the elastic range. While such connection panels possess large reserve capacity beyond initial general shear yielding, the corresponding inelastic joint deformations may adversely affect the strength and stability of the frame or story (Fielding and Huang, 1971; Fielding and Chen, 1973). Panel-zone shear yielding affects the overall frame stiffness and, therefore, the resulting second-order effects may be significant. The shear/axial interaction expression of Equation J10-10, as shown in Figure C-J10.4, provides elastic panel behavior.

If adequate connection ductility is provided and the frame analysis considers the inelastic panel-zone deformations, then the additional inelastic shear strength is recognized in Equations J10-11 and J10-12 by the factor

$$\left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w} \right)$$
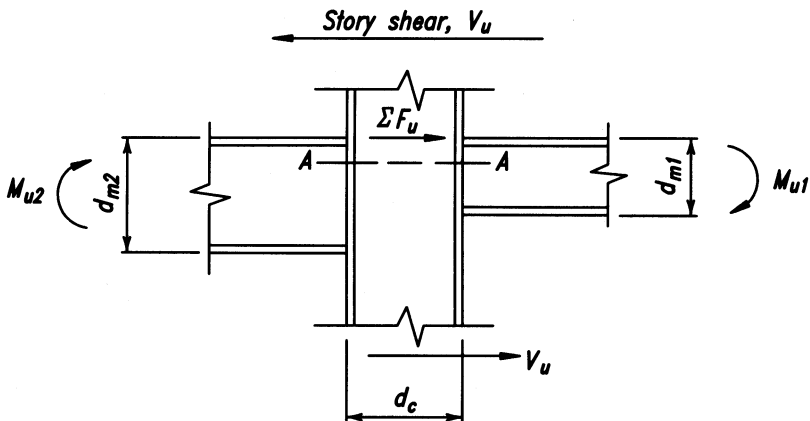
Fig. C-J10.3. LRFD forces in panel zone (ASD forces are similar).

This inelastic shear strength has been most often utilized for the design of frames in high seismic design and should be used when the panel zone is designed to develop the strength of the members from which it is formed.

The shear/axial interaction expression incorporated in Equation J10-12 (see Figure C-J10.5) recognizes that when the panel-zone web has completely yielded in shear, the axial column load is carried in the flanges.

7. Unframed Ends of Beams and Girders

Full-depth stiffeners are required at unframed ends of beams and girders not otherwise restrained to avoid twisting about their longitudinal axes.

8. Additional Stiffener Requirements for Concentrated Forces

See Carter (1999), Troup (1999), and Murray and Sumner (2004) for guidelines on column stiffener design.
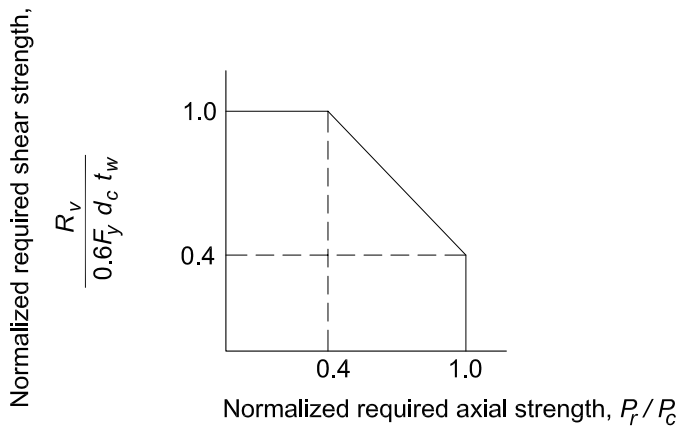
Fig. C-J10.4. Interaction of shear and axial force—elastic.
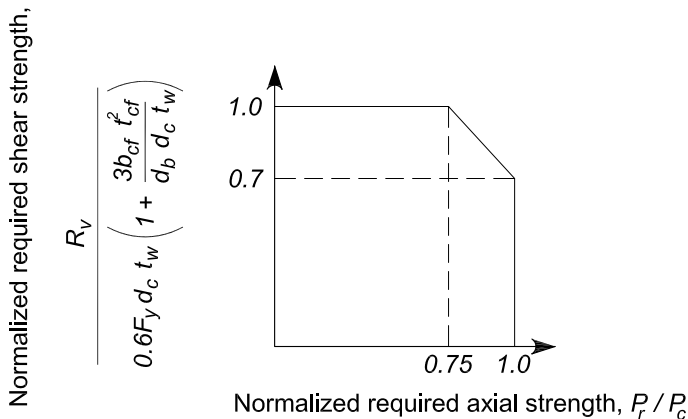
Fig. C-J10.5. Interaction of shear and axial force—inelastic.

For rotary straightened W-shapes, an area of reduced notch toughness is sometimes found in a limited region of the web immediately adjacent to the flange, referred to as the "k-area," as illustrated in Figure C-J10.6 (Kaufmann, Metrovich, Pense, and Fisher, 2001). Following the 1994 Northridge Earthquake, there was a tendency to specify thicker continuity plates that were groove welded to the web and flange and thicker doubler plates that were often groove welded in the gap between the doubler plate and the flanges. These welds were highly restrained and may have caused cracking during fabrication in some cases (Tide, 1999).

AISC (1997a) recommended that the welds for continuity plates should terminate away from the k-area, which is defined as the "region extending from approximately the midpoint of the radius of the fillet into the web approximately 1 to 1^1/$_2$ in. (25 to 38 mm) beyond the point of tangency between the fillet and web."
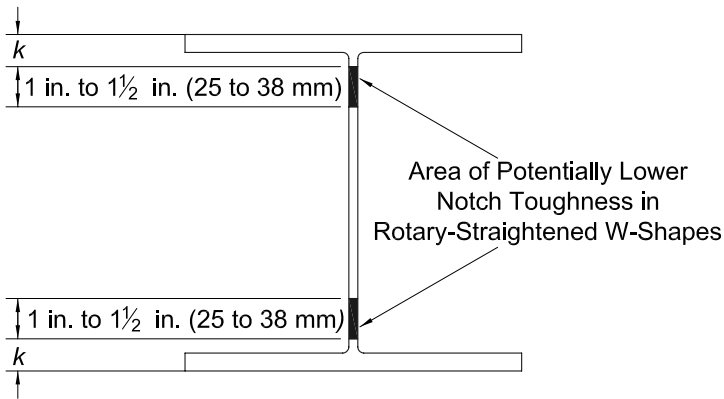
Fig. C-J10.6. Representative "k-area" of a wide-flange shape.
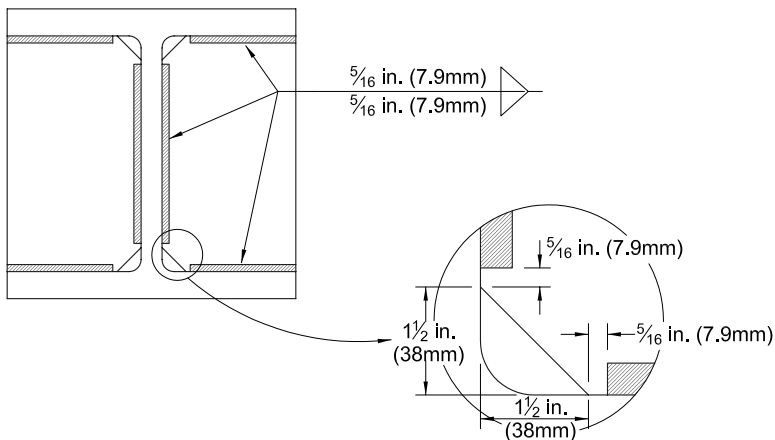
Fig. C-J10.7. Recommended placement of stiffener fillet welds to avoid contact with "k-area."

Recent pull-plate testing (Dexter and Melendrez, 2000; Prochnow and others, 2000; Hajjar and others, 2003) and full-scale beam-column joint testing (Bjorhovde, Goland, and Benac, 1999; Dexter, Hajjar, Prochnow, Graeser, Galambos, and Cotton, 2001; Lee, Cotton, Dexter, Hajjar, Ye, and Ojard, 2002) has shown that this problem can be avoided if the column stiffeners are fillet welded to both the web and the flange, the corner is clipped at least $1^1/_2$ in. (38 mm), and the fillet welds are stopped short by a weld leg length from the edges of the cutout, as shown in Figure C-J10.7. These tests also show that groove welding the stiffeners to the flanges or the web is unnecessary, and that the fillet welds performed well with no problems. If there is concern regarding the development of the stiffeners using fillet welds, the corner clip can be made so that the dimension along the flange is $3/_4$ in. (20 mm) and the dimension along the web is $1^1/_2$ in. (38 mm).

Recent tests have also shown the viability of fillet welding doubler plates to the flanges, as shown in Figure C-J10.8 (Prochnow and others, 2000; Dexter and others, 2001; Lee and others, 2002; Hajjar and others, 2003). It was found that it
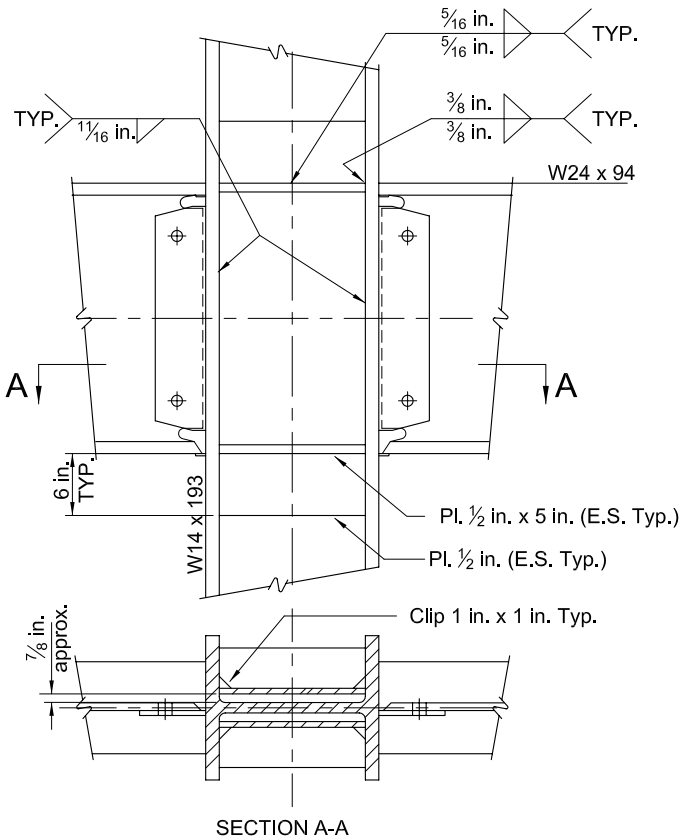
Fig. C-J10.8. Example of fillet welded doubler plate and stiffener details.

is not necessary to groove weld the doubler plates and that they do not need to be in contact with the column web to be fully effective.

9.   Additional Doubler Plate Requirements for Concentrated Forces

When required, doubler plates are to be designed using the appropriate limit state requirements for the type of loading. The sum of the strengths of the member element and the double plate(s) must exceed the required strength and the doubler plate must be welded to the member element.

CHAPTER K

DESIGN OF HSS AND BOX MEMBER CONNECTIONS

Chapter K addresses the strength of HSS and box member welded connections. The provisions are based upon failure modes that have been reported in international research on HSS, much of which has been sponsored and synthesized by CIDECT (International Committee for the Development and Study of Tubular Construction) since the 1960s. This work has also received critical review by the International Institute of Welding (IIW) Subcommission XV-E on "Welded Joints in Tubular Structures." The HSS connection design recommendations are generally in accord with the last edition of the design recommendations by this Subcommission (IIW, 1989). Some minor modifications to the IIW recommended provisions for some limit states have been made by the adoption of the formulations for the same limit states elsewhere in this Specification. The IIW connection design recommendations referred to above have also been implemented and supplemented in later design guides by CIDECT (Wardenier, Kurobane, Packer, Dutta, and Yeomans, 1991; Packer, Wardenier, Kurobane, Dutta, and Yeomans, 1992), in the design guide by the Canadian Institute of Steel Construction (Packer and Henderson, 1997) and in Eurocode 3 (2002). Parts of these IIW design recommendations are also incorporated in AWS (2004). A large amount of research data generated by CIDECT research programs up to the mid-1980s is summarized in CIDECT Monograph No. 6 (Giddings and Wardenier, 1986). Further information on CIDECT publications and reports can be obtained from their website: *www.cidect.com.*

The scopes of Sections K2 and K3 note that the centerlines of the branch member(s) and the chord members must lie in a single plane. For other configurations, such as multiplanar connections, connections with partially or fully flattened branch member ends, double chord connections, connections with a branch member that is offset so that its centerline does not intersect with the centerline of the chord or connections with round branch members joined to a square or rectangular chord member, the provisions of IIW (1989), CIDECT, Wardenier and others (1991), Packer and others (1992), CISC, Packer and Henderson (1997), Marshall (1992), AWS (2004), or other verified design guidance or tests can be used.

K1. CONCENTRATED FORCES ON HSS

1. Definitions of Parameters

Some of the notation used in Chapter K is illustrated in Figure C-K1.1.

2. Limits of Applicability

The limits of applicability in Section K1.2 stem primarily from limitations on tests conducted to date.

3.   Concentrated Force Distributed Transversely

Sections K1.3 and K1.4, although pertaining to all concentrated forces on HSS, are particularly oriented towards plate-to-HSS welded connections and this application is displayed in tabular form in Table C-K1.1 (a) and (b). In addition to the design provisions in the Specification, Table C-K1.1(b) also gives flexural strengths for some plate-to-round HSS connections. Most of the equations (after application of appropriate resistance factors for LRFD) conform to CIDECT Design Guides 1 and 3 (Wardenier and others, 1991; Packer and others, 1992) with updates in accordance with CIDECT Design Guide 9 (Kurobane, Packer, Wardenier, and Yeomans, 2004). The latter includes revisions for longitudinal plate-to-rectangular HSS connections (Equation K1-9) based on extensive experimental and numerical studies reported in Kosteski and Packer (2003). The provisions for the limit state of sidewall crippling of rectangular HSS, Equations K1-5 and K1-6, conform to web crippling expressions elsewhere in this Specification, and not to CIDECT or IIW recommendations. If a longitudinal plate-to-rectangular HSS connection is made by passing the plate through a slot in the HSS and then welding the plate to both the front and back HSS faces to form a "through-plate connection," the nominal strength can be taken as twice that given by Equation K1-9 (Kosteski and Packer, 2003).

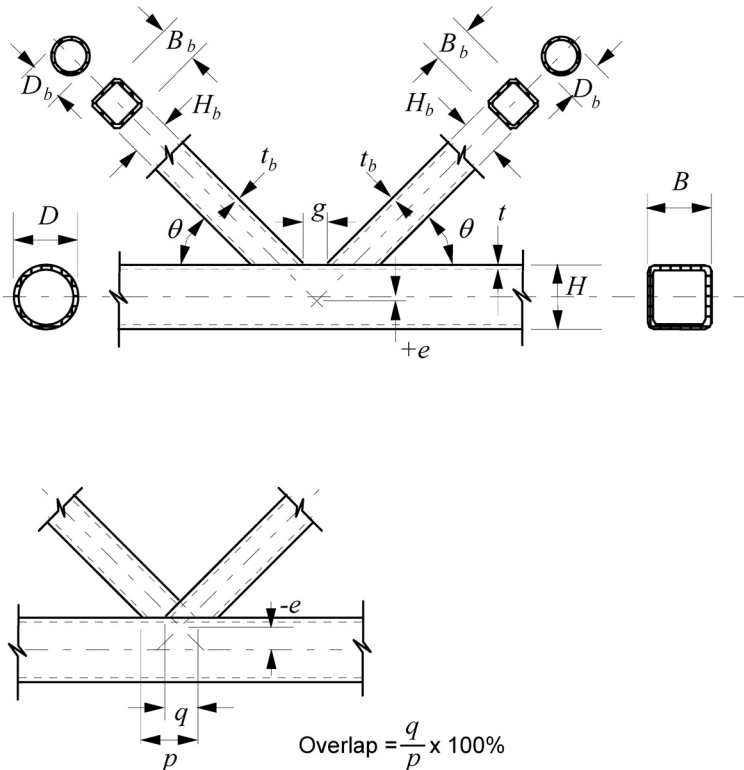$$\text{Overlap} = \frac{q}{p} \times 100\%$$

Fig. C-K1.1. Common notation for HSS connections.

The equations given for transverse plate-to-HSS connections can also be adapted for wide-flange beam-to-HSS PR moment connections, by treating the beam flanges as a pair of transverse plates and ignoring the beam web. For such wide-flange beam connections, the beam moment is thus produced by a force couple in the beam flanges. The connection flexural strength is then given by the plate-to-HSS connection strength multiplied by the distance between the beam flange centers. In Table C-K1.1(a) there is no check for the limit state of chord wall plastification for transverse plate-to-rectangular HSS connections, because this will not govern the design in practical cases. However, if there is a major compression load in the HSS, such as when it is used as a column, one should be aware that this compression load in the main member has a negative influence on the yield line plastification failure mode of the connecting chord wall (via a Q_f factor). In such a case, the designer can utilize guidance in CIDECT Design Guide No. 9 (Kurobane and others, 2004).

4. Concentrated Force Distributed Longitudinally at the Center of the HSS Diameter or Width, and Acting Perpendicular to the HSS Axis

See commentary for Section K1.3.
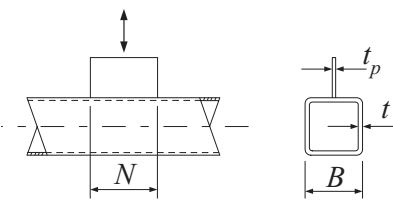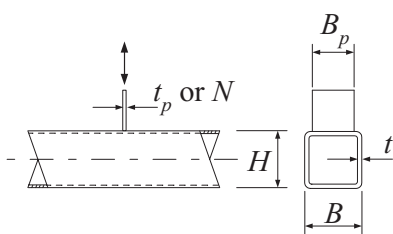
5. Concentrated Force Distributed Longitudinally at the Center of the HSS Width, and Acting Parallel to the HSS Axis

Section K1.5 applies to longitudinal plate connections loaded in shear. These recommendations are based on Sherman and Ales (1991), Sherman (1995a) and Sherman (1996) that investigated a large number of simple framing connections between wide-flange beams and rectangular HSS columns, in which the load transferred was predominantly shear. A review of costs also showed that single-plate and single-angle connections were the most economical, with double-angle and fillet-welded tee connections being more expensive. Through-plate and flare-bevel welded tee connections were among the most expensive (Sherman, 1995a). Over a wide range of connections tested, only one limit state was identified for the rectangular HSS column: punching shear failure related to end rotation of the beam, when a thick shear plate was joined to a relatively thin-walled HSS. Compliance with the inequality given by K1-10 precludes this HSS failure mode. This design rule is valid providing the HSS wall is not classified as a *slender element*. An extrapolation of inequality K1-10 has also been made for round HSS columns, subject to the round HSS cross-section not being classified as a *slender element*.

6. Concentrated Axial Force on the End of a Rectangular HSS with a Cap Plate
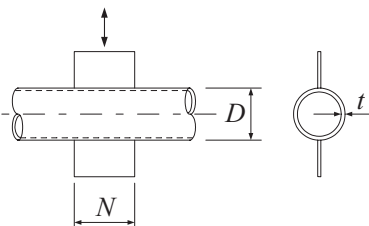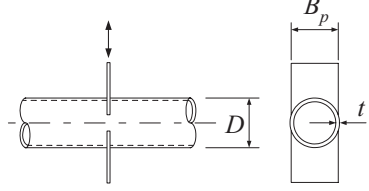
In Section K1.6, two limit states are given for the strength of a square or rectangular HSS wall with load transferred through a cap plate (or the flange of a T-stub), as shown in Figure C-K1.2. In general, the rectangular HSS could have dimensions of $B \times H$, but the illustration shows the bearing length (or width), N, oriented for lateral load dispersion into the wall of dimension B. A conservative distribution

TABLE C-K1.1 (a)
Nominal Strengths of Plate-to-Rectangular HSS Connections

Connection Type	Connection Nominal Strength
Longitudinal plate	$\beta \leq 0.85$ Basis: chord wall plastification $$R_n = \frac{F_y t^2}{1 - \frac{t_p}{B}}\left(\frac{2N}{B} + 4\sqrt{1 - \frac{t_p}{B}}\,Q_f\right)$$
Transverse plate where $\beta = \dfrac{B_p}{B}$	$\beta \approx 1.0$ Basis: HSS side wall strength
	Tension and compression: $R_n = 2F_y t[5k + N]$ Compression in T-connections: $$R_n = 1.6t^2\left[1 + \frac{3N}{H - 3t}\right]\sqrt{EF_y}\,Q_f$$ Compression in cross-connections: $$R_n = \frac{48t^3}{H - 3t}\sqrt{EF_y}\,Q_f$$
	$0.85 \leq \beta \leq 1 - 2t/B$ Basis: punching shear failure
	$$R_n = 0.6F_y t[2t_p + 2B_{ep}]$$
	All β Basis: uneven load distribution
	$$R_n = \frac{10}{B/t}F_y t B_p \leq F_{yp} t_p B_p$$

Functions and Range of Validity

$\dfrac{B}{t} \leq 35$ for the loaded HSS wall in transverse connections and
     ≤ 40 for longitudinal connections

$0.25 < \dfrac{B_p}{B} \leq 1.0$ for transverse connections

$B_{ep} = \dfrac{10B_p}{B/t}$ but $\leq B_p$

$k = $ outside corner radius of HSS $\geq 1.5t$

$Q_f = 1.0$ (chord in tension, for transverse connections)

$Q_f = 1.3 - 0.4\dfrac{U}{\beta}$ but ≤ 1.0 (chord in compression, for transverse connections)

$Q_f = \sqrt{1 - U^2}$ (for longitudinal connections)

slope can be assumed as 2.5:1 from each face of the tee web (Wardenier and others, 1991; Kitipornchai and Traves, 1989), which produces a dispersed load width of $(5t_p + N)$. If this is less than B, only the two side walls of dimension B are effective in resisting the load, and even they will both be only partially effective. If $(5t_p + N) \geq B$, all four walls of the rectangular HSS will be engaged, and all

TABLE C-K1.1 (b)
Nominal Strengths of Plate-to-Round HSS Connections

Connection Type	Connection Nominal Strength		
	Axial Force	Bending in Plane	Bending out of Plane
Longitudinal plate	Chord plastification: $R_n = 5.5 F_y t^2 \left(1 + 0.25 \dfrac{N}{D}\right) Q_f$	$M_n = N R_n$	—
Transverse plate	$R_n = F_y t^2 \left[\dfrac{5.5}{1 - 0.81 \dfrac{B_p}{D}} \right] Q_f$	—	$M_n = 0.5 B_p R_n$

Functions and Range of Validity

$\dfrac{D}{t} \leq 50$ for T-connections and ≤ 40 for cross-connections

$0.2 < \dfrac{B_p}{D} \leq 1.0$ for transverse connections

$Q_f = 1.0$ (chord in tension)

$Q_f = 1.0 - 0.3U(1 + U)$ but ≤ 1.0 (chord in compression)

will be fully effective; however, the cap plate (or T-stub flange) must be sufficiently thick for this to happen. In Equations K1-11 and K1-12 the size of any weld legs has been conservatively ignored. If the weld leg size is known, it is acceptable to assume load dispersion from the toes of the welds. The same load dispersion model as shown in Figure C-K1.2 can also be applied to round HSS-to-cap plate connections.

K2. HSS-TO-HSS TRUSS CONNECTIONS

The classification of HSS truss-type connections as K- (which includes N-), Y- (which includes T-), or cross- (also known as X-) connections is based on the method of force transfer in the connection, not on the physical appearance of the connection. Examples of such classification are shown in Figure C-K2.1.

As noted in Section K2, when branch members transmit part of their load as K-connections and part of their load as T-, Y-, or cross-connections, the adequacy of each branch is determined by linear interaction of the proportion of the branch load involved in each type of load transfer. One K-connection, shown in Figure C-K2.1(b), illustrates that the branch force components normal to the chord member may differ by as much as 20 percent and still be deemed to exhibit K-connection behavior. This is to accommodate slight variations in branch member forces along a typical truss, caused by a series of panel point loads. The N-connection in Figure C-K2.1(c), however, has a ratio of branch force components normal to the chord member of 2:1. In this case, the connection is analyzed as both a "pure" K-connection (with balanced branch forces) and a cross- (or X-) connection (because the remainder of the diagonal branch load is being transferred through the connection), as shown in Figure C-K2.2. For the diagonal tension branch in that connection, the following check is also made:

$$(0.5P\sin\theta/K\text{-connection available strength})$$
$$+ (0.5P\sin\theta/\text{cross-connection available strength}) \leq 1.0$$

If the gap size in a gapped K- (or N-) connection [for example, Figure C-K2.1(a)] becomes large and exceeds the value permitted by the eccentricity limit, the "K-connection" should be treated as two independent Y-connections. In cross-connections, such as Figure C-K2.1(e), where the branches are close together or overlapping, the combined "footprint" of the two branches can be taken as the loaded area on the chord member. In K-connections such as Figure C-K2.1(d), where a branch has very little or no loading, the connection can be treated as a Y-connection, as shown.

The design of welded HSS connections is based on potential limit states that may arise for a particular connection geometry and loading, which in turn represent possible failure modes that may occur within prescribed limits of applicability.
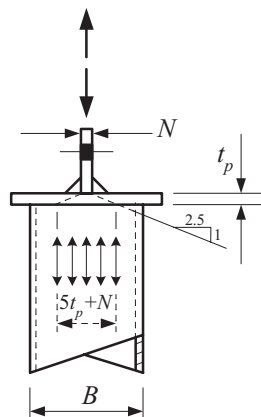
Fig. C-K1.2. Load dispersion from a concentrated force through a cap plate.
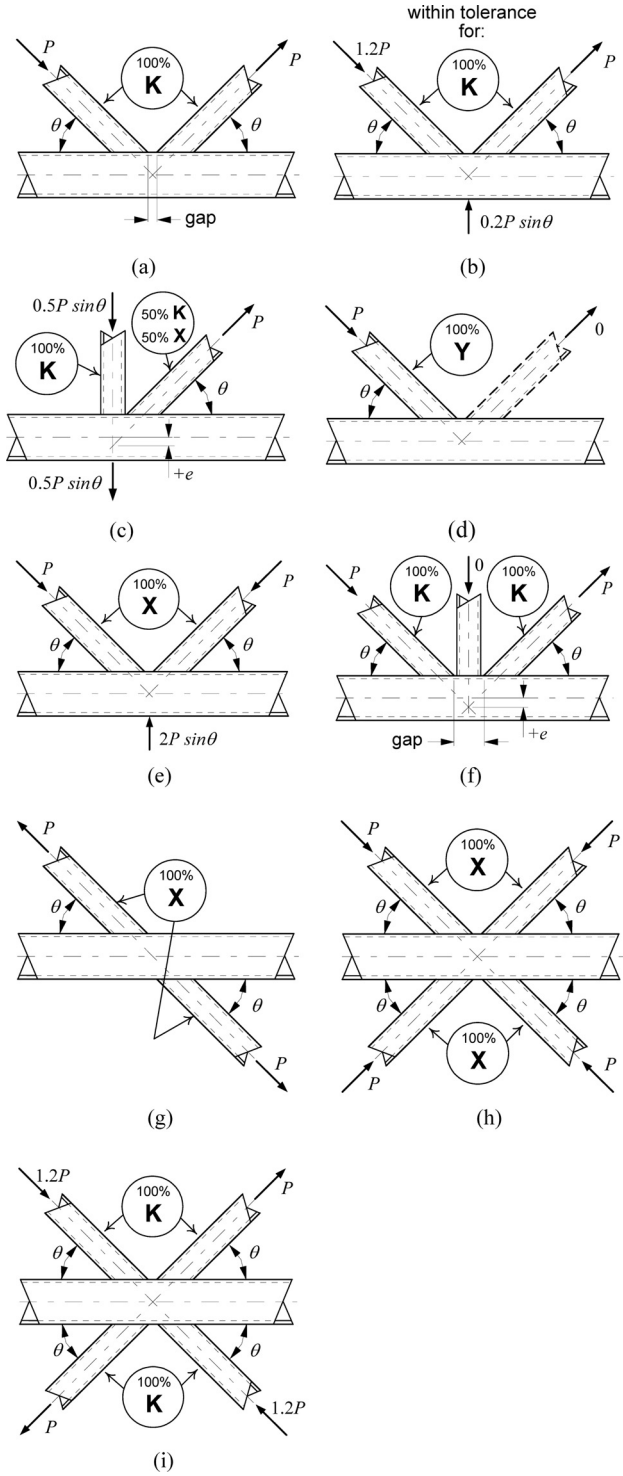
Fig. C-K2.1. Examples of HSS connection classification.

Some typical failure modes for truss-type connections, shown for rectangular HSS, are given in Figure C-K2.3.

1.   Definitions of Parameters

Some parameters are defined in Figure C-K1.1.

2.   Criteria for Round HSS

The limits of validity in Section K2.2a generally represent the parameter range over which the equations have been verified in experiments. The following limitations bear explanation:

(2) The minimum branch angle is a practical limit for good fabrication. Smaller branch angles are possible, but prior agreement with the fabricator should be made.

(5) The wall slenderness limit for the compression branch is a restriction so that connection strength is not reduced by branch local buckling.

(6) The minimum width ratio limit for gapped K-connections has been added in this Specification as a precaution, because Packer (2004) showed that for width ratios less than 0.4, Equation K2-6 may be potentially unconservative when evaluated against proposed equations for the design of such connections by the American Petroleum Institute (API, 1993).

(7) The restriction on the minimum gap size is only stated so that adequate space is available to enable welding at the toes of the branches to be satisfactorily performed.

(8) The restriction on the minimum overlap is applied so that there is an adequate interconnection of the branches, to enable effective shear transfer from one branch to the other.

The provisions given in Sections K2.2b and K2.2c are generally based, with the exception of the punching shear provision, on semi-empirical "characteristic strength" expressions, which have a confidence of 95 percent, taking into account the variation in experimental test results as well as typical variations in mechanical and geometric properties. These "characteristic strength" expressions are then multiplied by resistance factors for LRFD or divided by safety factors for ASD to further allow for the relevant failure mode. In the case of the chord plastification failure mode a ϕ factor of 0.9 or Ω factor of 1.67 is applied, whereas in the case
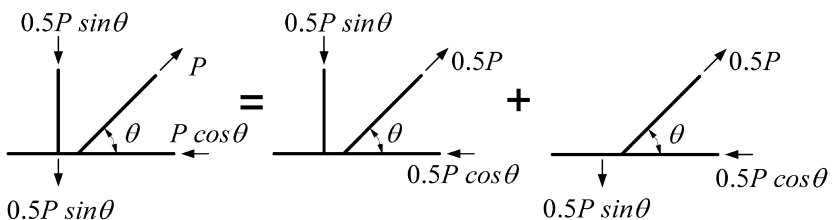
Fig. C-K2.2. Checking of K-connection with imbalanced branch member loads.

of punching shear a φ factor of 0.95 or a Ω factor of 1.58 is applied. The latter φ factor is 1.0 (equivalent to Ω of 1.50) in many recommendations or specifications [for example, IIW (1989), Packer and Henderson (1997), and Wardenier and others (1991)] to reflect the large degree of reserve strength beyond the analytical nominal strength expression, which is itself based on the shear yield (rather than ultimate) strength of the material. In this Specification, however, a φ factor of 0.95 or Ω factor of 1.58 is applied to maintain consistency with the factors for similar failure modes in Section K2.3. The shear failure resistance has also been taken as $0.95(0.6F_y) = 0.57F_y$, and elsewhere in Sections K2 and K3 as well, whereas

(a) Chord plastification   *(b) Punching shear failure of the chord*

(c) Uneven load distribution, in the tension branch   *(d) Uneven load distribution, in the compression branch*

(e) Shear yielding of the chord, in the gap   *(f) Chord sidewall failure*
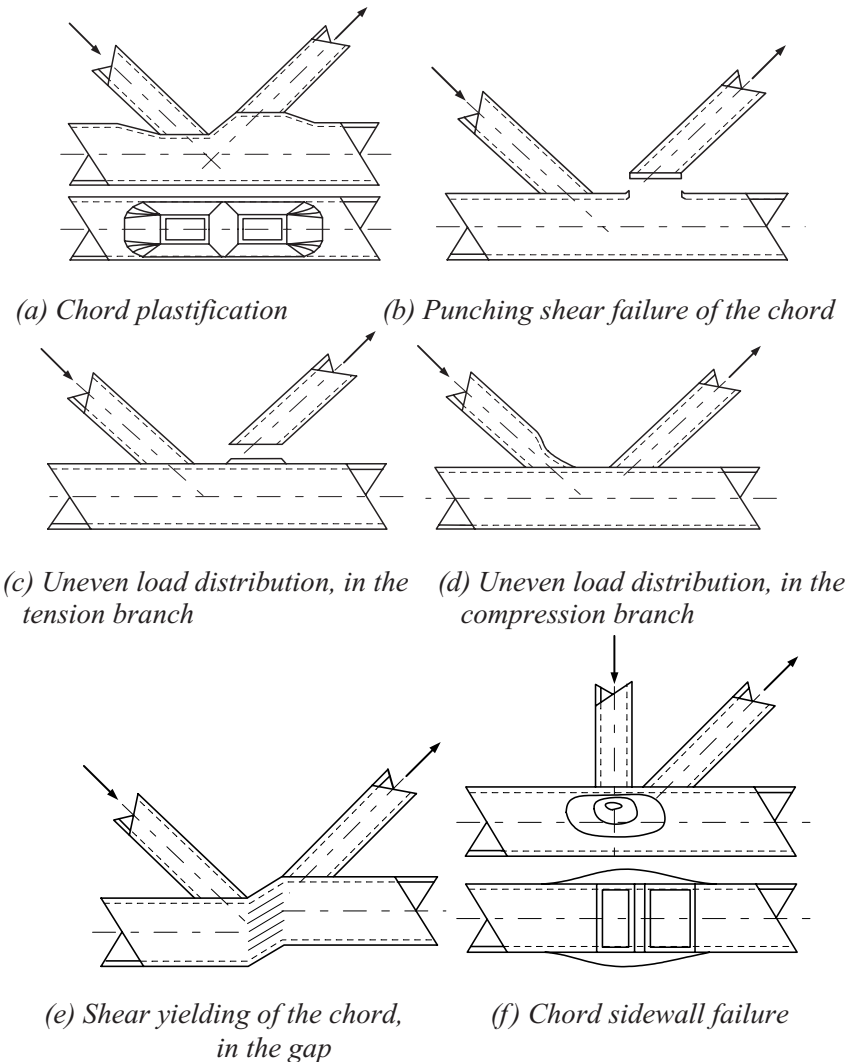
Fig. C-K2.3. Typical limit states for HSS-to-HSS truss connections.

IIW (1989) uses a von Mises shear yield resistance formulation of $1.0(F_y/\sqrt{3}) = 0.58F_y$. One should note that if the ultimate stress, F_u, were adopted as a basis for a punching shear rupture criterion, the accompanying ϕ would be 0.75 and Ω would be 2.0, as elsewhere in this Specification. Then, $0.75(0.6\ F_u) = 0.45\ F_u$ would yield a very similar value to $0.95(0.6\ F_y) = 0.57\ F_y$, and in fact the latter is even more conservative for HSS with specified nominal F_y/F_u ratios less than 0.79. Equation K2-4 need not be checked when $\beta > (1 - 1/\gamma)$ because this is the physical limit at which the branch can punch into (or out of) the main tubular member.

With round HSS in axially loaded K-connections, the size of the compression branch dominates the determination of the connection strength. Hence, the term D_b in Equation K2-6 pertains only to the compression branch and is not an average of the two branches. Thus, if one requires the connection strength expressed as a force in the tension branch, one can resolve the answer from Equation K2-6 into the direction of the tension branch, using Equation K2-8. That is, it is not necessary to repeat a calculation similar to Equation K2-6 with D_b as the tension branch. Note that Section K2.2c deals with branches subject to axial loading only. This is because there should only be axial forces in the branches of a typical planar K-connection if the truss structural analysis is performed according to one of the recommended methods, which are:

 (i) pin-jointed analysis; or
(ii) analysis using web members pin-connected to continuous chord members, as shown in Figure C-K2.4.

3.   Criteria for Rectangular HSS

The limits of validity in Section K2.3a generally represent the parameter range over which the design provisions have been verified in experiments. They are also
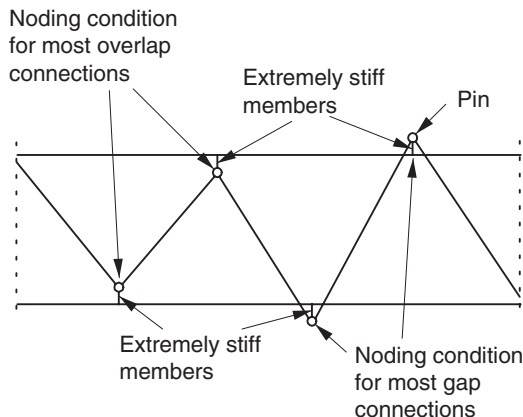
Fig. C-K2.4. Modeling assumption using web members pin-connected to continuous chord members.

set to eliminate the occurrence of certain failure modes for particular connection types, thereby making connection design easier. The following limitations from Section K2.3a bear explanation:

(2) The minimum branch angle is another practical limit for fabrication. Smaller branch angles are possible, but prior agreement with the fabricator should be made.

(8) The restriction on the minimum overlap is applied to ensure that there is an adequate interconnection of the branches to provide effective shear transfer from one branch to the other.

The restriction on the minimum gap ratio in Section K2.3c is modified from IIW (1989), according to Packer and Henderson (1997), to be more practical. The minimum gap size, g, is only specified so that adequate space is available to enable welding at the toes of the branches to be satisfactorily performed.

Equation K2-13 represents an analytical yield line solution for flexure of the connecting chord face. This nominal strength equation serves to limit connection deformations and is known to be well below the ultimate connection strength. A ϕ factor of 1.0 or Ω factor of 1.5 is thus appropriate. When the branch width exceeds 0.85 of the chord width this yield line failure mechanism will result in a noncritical design load.

The limit state of punching shear, evident in Equations K2-14 and K2-21, is based on the effective punching shear perimeter around the branch, with the total branch perimeter being an upper limit on this length. The term β_{eop} represents the chord face effective punching shear width ratio, adjacent to one (Equation K2-21) or two (Equation K2-14) branch walls transverse to the chord axis. This β_{eop} term incorporates a ϕ factor of 0.8 or Ω factor of 1.88. Applying to generally one dimension of the rectangular branch footprint, this was deemed by AWS to be similar to a global ϕ factor of 0.95 or Ω factor of 1.58 for the whole expression, so this expression for punching shear was implemented into AWS (2004) with an overall ϕ of 0.95. This ϕ factor of 0.95 or Ω factor of 1.58 has been carried over to this Specification and this topic is discussed further in Section K2.2. Notes below Equations K2-14 and K2-21 indicate when this failure mode is either physically impossible or noncritical. In particular, note that Equation K2-21 is noncritical for square HSS branches.

Equation K2-15 is generally in accord with a limit state given in IIW (1989), but with the k term [simply t in IIW (1989)] modified to be compatible with Equation K1-4, which in turn is derived from loads on I-shaped members. Equations K2-16 and K2-17 are in a format different than used internationally [for example, IIW (1989)] for this limit state and are unique to this Specification, having been replicated from Equations K1-5 and K1-6, along with their associated ϕ and Ω factors. These latter equations in turn are HSS versions (for two webs) of equations for I-shaped members with a single web.

The limit state of "uneven load distribution", which is manifested by local buckling of a compression branch or premature yield failure of a tension branch, represented by Equations K2-18 and K2-22, is checked by summing the effective areas of the four sides of the branch member. For T-, Y- and cross-connections the two walls of the branch transverse to the chord are likely to be only partially effective (Equation K2-18), whereas for gapped K-connections one wall of the branch transverse to the chord is likely to be only partially effective (Equation K-22). This reduced effectiveness is primarily a result of the flexibility of the connecting face of the chord, as incorporated in Equations K2-19 and K2-23. The effective width term b_{eoi} has been derived from research on transverse plate-to-HSS connections (as cited below for overlapped K-connections) and incorporates a ϕ factor of 0.8 or Ω factor of 1.88. Applying the same logic described above for the limit state of punching shear, a global ϕ factor of 0.95 or Ω factor of 1.58 was adopted in AWS D1.1 (AWS, 2004), and this has been carried over to this Specification [although, as noted previously, a ϕ factor of 1.0 is used in IIW (1989)].

For T-, Y- and cross-connections with $\beta \leq 0.85$, the connection strength is determined by Equation K2-13 only.

For axially loaded, gapped K-connections, plastification of the chord connecting face under the "push-pull" action of the branches is by far the most prevalent and critical failure mode. Indeed, if all the HSS members are square, this failure mode is critical and Equation K2-20 is the only one to be checked. This formula for chord face plastification is a semi-empirical "characteristic strength" expression, which has a confidence of 95 percent, taking into account the variation in experimental test results as well as typical variations in mechanical and geometric properties. Equation K2-20 is then multiplied by a ϕ factor for LRFD or divided by an Ω factor for ASD to further allow for the failure mode and provide an appropriate safety margin. A reliability calibration (Packer, Birkemoe, and Tucker, 1984) for this equation, using a database of 263 gapped K-connections and the exponential expression for the resistance factor (with a safety index of 3.0 and a coefficient of separation of 0.55) derived a ϕ factor of 0.89 (Ω factor of 1.69), while also imposing the parameter limits of validity. Since this failure mode dominates the test database, there is insufficient supporting test data to calibrate Equations K2-21 and K2-22.

For the limit state of shear yielding of the chord in the gap of gapped K-connections, Section K2.3c(c) differs from international practice [for example, IIW (1989)] but recommends application of another section of this Specification, Section G5. This limit state need only be checked if the chord member is rectangular (in other words, not square) and is also oriented such that the shorter wall of the chord section lies in the plane of the truss, hence providing a more critical chord shear condition due to the short "webs." The axial force present in the gap region of the chord member may also have an influence on the shear capacity of the chord webs in the gap region.

For K-connections, the scope covers both gapped and overlapped connections, although the latter are generally more difficult and more expensive to fabricate than

K-connections with a gap. However, an overlapped connection will, in general, produce a connection with a higher static strength, a stiffer truss, and a connection with a higher fatigue resistance, than its gapped connection counterpart. Note that Sections K2.3c and K2.3d deal with branches subject to axial loading only. This is because there should only be axial forces in the branches of a typical planar K-connection if the truss structural analysis is performed according to one of the recommended methods, which are:

(i) pin-jointed analysis, or
(ii) analysis using web members pin-connected to continuous chord members, as shown in Figure C-K2.4.

For rectangular HSS, the sole failure mode to be considered for design of overlapped connections is the limit state of "uneven load distribution" in the branches, manifested by either local buckling of the compression branch or premature yield failure of the tension branch. The design procedure presumes that one branch is welded solely to the chord and hence only has a single cut at its end. This can be considered "good practice" and the "thru member" is termed the overlapped member. For partial overlaps of less than 100 percent, the other branch is then double-cut at its end and welded to both the thru branch as well as the chord. The branch to be selected as the "thru" or overlapped member should be the one with the larger overall width. If both branches have the same width, the thicker branch should be the overlapped branch. For a single failure mode to be controlling (and not have failure by one branch punching into or pulling out of the other branch, for example), limits are placed on various connection parameters, including the relative width and relative thickness of the two branches. The foregoing fabrication advice for rectangular HSS also pertains to round HSS overlapped K-connections, but the latter involves more complicated profiling of the branch ends to provide good saddle fits.

Overlapped rectangular HSS K-connection strength calculations (Equations K2-24, K2-25 and K2-26) are performed initially just for the overlapping branch, regardless of whether it is in tension or compression, and then the resistance of the overlapped branch is determined from that. The equations for connection strength, expressed as a force in a branch, are based on the load-carrying contributions of the four side walls of the overlapping branch and follow the design recommendations of the International Institute of Welding (IIW, 1989; Packer and Henderson, 1997; AWS, 2004). The effective widths of overlapping branch member walls transverse to the chord (b_{eoi} and b_{eov}) depend on the flexibility of the surface on which they land, and are derived from plate-to-HSS effective width measurements (Rolloos, 1969; Wardenier, Davies, and Stolle, 1981; Davies and Packer, 1982). The constant of 10 in the b_{eoi} and b_{eov} terms has already been reduced from values determined in tests and incorporates a ϕ factor of 0.80 or Ω factor of 1.88 in those terms. Applying the same logic described above for the limit state of punching shear in T-, Y- and cross-connections, a global ϕ factor of 0.95 or Ω factor of 1.58 was adopted by AWS D1.1 and this has been carried over to this Specification [although as noted previously a ϕ factor of 1.0 is used by IIW (1989)].

The applicability of Equations K2-24, K2-25 and K2-26 depends on the amount of overlap, O_v, where $O_v = (q/p) \times 100\%$. It is important to note that p is the projected length (or imaginary footprint) of the overlapping branch on the connecting face of the chord, even though it does not physically contact the chord. Also, q is the overlap length measured along the connecting face of the chord beneath the region of overlap of the branches. This is illustrated in Figure C-K1.1.

A maximum overlap of 100 percent occurs when one branch sits completely on the other branch. In such cases, the overlapping branch is sometimes moved slightly up the overlapped branch so that the heel of the overlapping branch can be fillet welded to the face of the overlapped branch. If the connection is fabricated in this manner, an overlap slightly greater than 100 percent is created. In such cases, the connection strength for a rectangular HSS connection can be calculated by Equation K2-26 but with the B_{bi} term replaced by another b_{eov} term. Also, with regard to welding details, it has been found experimentally that it is permissible to just tack weld the "hidden toe" of the overlapped branch, providing that the components of the two branch member forces normal to the chord substantially balance each other. The "hidden toe" should be fully welded to the chord if the normal components of the two branch forces differ by more than 20 percent. If the components of the two branch forces normal to the chord do in fact differ significantly, the connection should also be checked for behavior as a T-, Y- or cross-connection, using the combined footprint and the net force normal to the chord (see Figure C-K2.1).

The design of "Welds to Branches" may be performed in either of two ways:

(a) The welds may be proportioned to develop the capacity of the connected branch wall, at all points along the weld length. This may be appropriate if the branch loading is complex or the loading is not known by the weld designer. Welds sized in this manner represent an upper limit on the required weld size and may be excessively conservative in some situations.

(b) The welds may be designed as "fit for purpose," to resist branch forces that are typically known in HSS truss-type connections. Many HSS truss web members have low axial loads, for a variety of possible reasons, and in such situations this weld design philosophy is ideal. However, the nonuniform loading of the weld perimeter due to the flexibility of the connecting HSS face must be taken into account by using weld effective lengths. Suitable effective lengths for various rectangular HSS connections subject to branch axial loading are given in Section K2.3e. These provisions are similar to those given in AWS (2004) and are based on full-scale HSS connection and truss tests that studied weld failures (Frater and Packer, 1992; 1992a; Packer and Cassidy, 1995). Adequate reliability is still obtained with the effective length expressions given if the directional strength increase allowed with fillet welds is used (Packer, 1995). Examples of weld joints in which weld effective lengths are less than 100 percent of the total weld length are shown in Figure C-K2.5. Most HSS trusses have the web members inclined to the chord at angles less than

50 degrees, in which cases the weld length around each branch perimeter in a K-connection will be 100 percent effective, as can be seen from Equation K2-31. Similar advice to that given in Section K2.3e is replicated in Section K1.3b for welds to transverse plates joined to rectangular HSS.

K3. HSS-TO-HSS MOMENT CONNECTIONS

Section K3 on HSS-to-HSS connections under moment loading is applicable to frames with PR or FR moment connections, such as Vierendeel girders. The provisions of Section K3 are not generally applicable to typical planar triangulated trusses (which are covered by Section K2), since the latter should be analyzed in a manner which results in no bending moments in the web members (see Commentary on Section K2). Thus, K-connections with moment loading on the branches are not covered by this Specification.

Available testing for HSS-to-HSS moment connections is much less extensive than that for axially-loaded T-, Y-, cross- and K-connections. Hence, the governing limit states to be checked for axially-loaded connections have been used as a basis for the possible limit states in moment-loaded connections. Thus, the design criteria for round HSS moment connections are based on the limit states of chord plastification and punching shear failure, with ϕ and Ω factors consistent with Section K2, while the design criteria for rectangular HSS moment connections are based on the limit states of plastification of the chord connecting face, chord side wall crushing, uneven load distribution and chord distortional failure, with ϕ and Ω factors consistent with Section K2. The "chord distortional failure" mode is applicable only to rectangular HSS T-connections with an out-of-plane bending moment on the branch. Rhomboidal distortion of the branch can be prevented by the use of
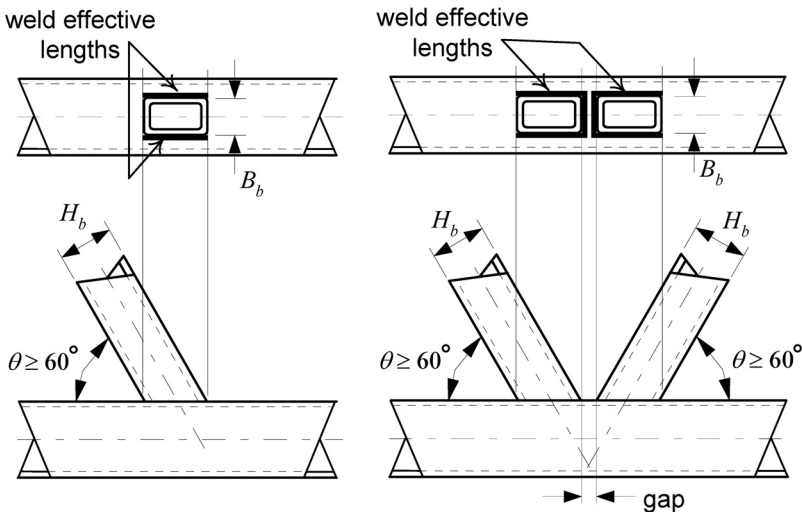
Fig. C-K2.5. Weld effective lengths for particular rectangular HSS connections.

stiffeners or diaphragms to maintain the rectangular cross-sectional shape of the chord. The limits of applicability of the equations in Section K3 are predominantly reproduced from Section K2. The basis for the equations in Section K3 is Eurocode 3 (2002), which represents one of the most up-to-date consensus specifications or recommendations on welded HSS-to-HSS connections. The equations in Section K3 have also been adopted in CIDECT Design Guide No. 9 (Kurobane and others, 2004).

CHAPTER L

DESIGN FOR SERVICEABILITY

L1. GENERAL PROVISIONS

Serviceability limit states are conditions in which the functions of a building are impaired because of local damage, deterioration or deformation of building components, or occupant discomfort. While serviceability limit states generally do not involve collapse of a building, loss of life or injury, they can seriously impair the building's usefulness and lead to costly repairs and other economic consequences. Serviceability provisions are essential to provide satisfactory performance of building structural systems. Neglect of serviceability may result in structures that are excessively flexible or otherwise perform unacceptably in service.

The three general types of structural behavior that are indicative of impaired serviceability in steel structures are:

(1) Excessive deflections or rotations that may affect the appearance, function or drainage of the building or may cause damaging transfer of load to nonstructural components and attachments;

(2) Excessive vibrations produced by the activities of the building occupants, mechanical equipment, or wind effects, which may cause occupant discomfort or malfunction of building service equipment; and

(3) Excessive local damage (local yielding, buckling, slip or cracking) or deterioration (weathering, corrosion and discoloration) during the service life of the structure.

Serviceability limit states depend on the occupancy or function of the building, the perceptions of its occupants, and the type of structural system. Limiting values of structural behavior intended to provide adequate levels of serviceability should be determined by a team consisting of the building owner/developer, the architect and the structural engineer after a careful analysis of all functional and economic requirements and constraints. In arriving at serviceability limits, the team should recognize that building occupants are able to perceive structural deformations, motions, cracking or other signs of distress at levels that are much lower than those that would indicate impending structural damage or failure. Such signs of distress may be viewed as an indication that the building is unsafe and diminish its economic value, and therefore must be considered at the time of design.

Service loads that may require consideration in checking serviceability include: (1) static loads from the occupants, snow or rain on the roof, or temperature fluctuations; and (2) dynamic loads from human activities, wind effects, the operation of mechanical or building service equipment, or traffic near the building. Service loads are loads that act on the structure at an arbitrary point in time, and may be only a fraction of the corresponding nominal load. The response of the structure to

service loads generally can be analyzed assuming elastic behavior. Members that accumulate residual deformations under service loads also may require examination with respect to this long-term behavior.

Serviceability limit states and appropriate load combinations for checking conformance to serviceability requirements can be found in ASCE 7, *Minimum Design Loads for Buildings and Other Structures*, Appendix B, and the commentary to Appendix B (ASCE, 2002).

L2.   CAMBER

Camber is frequently specified in order to provide a level surface under *permanent loads*, for reasons of appearance or for alignment with other work. In normal circumstances camber does nothing to prevent excessive deflection or vibration. Camber in trusses is normally created by adjustment of member lengths prior to making shop connections. It is normally introduced in beams by controlled heating of selected portions of the beam or by cold bending, or both. Designers should be aware of practical limits presented by normal fabricating and erection practices. The *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005) provides tolerances on actual camber and recommends that all cambers be measured in the fabricating shop on unstressed members, along general guidelines. Further information on camber may be found in Ricker (1989).

L3.   DEFLECTIONS

Excessive vertical deflections and misalignment arise primarily from three sources: (1) gravity loads, such as dead, live and snow loads; (2) effects of temperature, creep and differential settlement; and (3) construction tolerances and errors. Such deformations may be visually objectionable; cause separation, cracking or leakage of exterior cladding, doors, windows and seals; and cause damage to interior components and finishes. Appropriate limiting values of deformations depend on the type of structure, detailing and intended use (Galambos and Ellingwood, 1986). Historically, common deflection limits for horizontal members have been 1/360 of the span for floors subjected to reduced live load and 1/240 of the span for roof members. Deflections of about 1/300 of the span (for cantilevers, 1/150 of the length) are visible and may lead to general architectural damage or cladding leakage. Deflections greater than 1/200 of the span may impair operation of moveable components such as doors, windows and sliding partitions.

Deflection limits depend very much on the function of the structure and the nature of the supported construction. Traditional limits expressed as a fraction of the span length should not be extrapolated beyond experience. For example, the traditional limit of 1/360 of the span worked well for controlling cracks in plaster ceilings with spans common in the first half of the twentieth century. Many structures with more flexibility have performed satisfactorily with the now common, and more forgiving, ceiling systems. On the other hand, with the advent of longer structural

spans, serviceability problems have been observed with flexible grid ceilings where actual deflections were far less than 1/360 of the span, because the distance between partitions or other elements that may interfere with ceiling deflection are far less than the span of the structural member. Proper control of deflections is a complex subject requiring careful application of professional judgment. West, Fisher, and Griffis (2003) provide an extensive discussion of the issues.

Deflection computations for composite beams should include an allowance for slip, creep and shrinkage (see Commentary Section I3.1).

In certain long-span floor systems, it may be necessary to place a limit (independent of span) on the maximum deflection to minimize the possibility of damage of adjacent nonstructural elements (ISO, 1977). For example, damage to nonload-bearing partitions may occur if vertical deflections exceed more than about $3/8$ in. (10 mm) unless special provision is made for differential movement (Cooney and King, 1988); however, many components can and do accept larger deformations.

Load combinations for checking static deflections can be developed using first-order reliability analysis (Galambos and Ellingwood, 1986). Current static deflection guidelines for floor and roof systems are adequate for limiting superficial damage in most buildings. A combined load with an annual probability of being exceeded of 5 percent is appropriate in most instances. For serviceability limit states involving visually objectionable deformations, repairable cracking or other damage to interior finishes, and other short-term effects, the suggested load combinations are:

$$D + L$$
$$D + 0.5S$$

For serviceability limit states involving creep, settlement or similar long-term or permanent effects, the suggested load combination is

$$D + 0.5L$$

The dead load effect, D, may be that portion of dead load that occurs following attachment of nonstructural elements. For example, in composite construction, the dead load effects frequently are taken as those imposed after the concrete has cured. For ceiling related calculations, the dead load effects may include only those loads placed after the ceiling structure is in place.

L4. DRIFT

Drift (lateral deflection) in a steel building is a serviceability issue primarily from the effects of wind. Drift limits are imposed on buildings to minimize damage to cladding and to nonstructural walls and partitions. Lateral frame deflection is evaluated for the building as a whole, where the applicable parameter is the *total building drift* (defined as the lateral frame deflection at the top of the most occupied floor divided by the height of the building to that level, Δ / H). For each floor, the applicable parameter is *interstory drift* [defined as the lateral deflection of a floor

relative to the lateral deflection of the floor immediately below, divided by the distance between floors, $(\delta_n - \delta_{n-1})/h]$.

Typical drift limits in common usage vary from $H/100$ to $H/600$ for *total building drift* and $h/200$ to $h/600$ for *interstory drift*, depending on building type and the type of cladding or partition materials used. The most widely used values are H (or h)/400 to H (or h)/500 (ASCE Task Committee on Drift Control of Steel Building Structures, 1988). An absolute limit on *interstory drift* is sometimes imposed by designers in light of evidence that damage to nonstructural partitions, cladding and glazing may occur if the *interstory drift* exceeds about $3/8$ in. (10 mm), unless special detailing practices are employed to accommodate larger movements (Cooney and King, 1988; Freeman, 1977). Many components can accept deformations that are significantly larger. More specific information on the damage threshold for building materials is available in the literature (Griffis, 1993).

It is important to recognize that frame racking or shear distortion (in other words, strain) is the real cause of damage to building elements such as cladding and partitions. Lateral drift only captures the horizontal component of the racking and does not include potential vertical racking (as from differential column shortening in tall buildings), which also contributes to damage. Moreover, some lateral drift may be caused by rigid body rotation of the cladding or partition which by itself does not cause strain and therefore damage. A more precise parameter, the *drift damage index*, used to measure the potential damage, has been proposed (Griffis, 1993).

It must be emphasized that a reasonably accurate estimate of building drift is essential to controlling damage. The structural analysis must capture all significant components of potential frame deflection including flexural deformation of beams and columns, axial deformation of columns and braces, shear deformation of beams and columns, beam-column joint rotation (panel-zone deformation), the effect of member joint size, and the P-Δ effect (Charney, 1990). For many low rise steel frames with normal bay widths of 30 to 40 ft (9 to 12 m), use of center-to-center dimensions between columns without consideration of actual beam column joint size and panel zone effects will usually suffice for checking drift limits. The stiffening effect of nonstructural cladding, walls and partitions may be taken into account if substantiating information (stress versus strain behavior) regarding their effect is available.

The level of wind load used in drift limit checks varies among designers depending upon the frequency with which the potential damage can be tolerated. Some designers use the same nominal wind load (wind load specified by the building code without a load factor) as used for the strength design of the members (typically a 50 or 100 year mean recurrence interval wind load). Other designers use a 10 year or 20 year mean recurrence interval wind load (Griffis, 1993; ASCE, 2002). Use of factored wind loads (nominal wind load multiplied by the wind load factor) is generally considered to be very conservative when checking serviceability.

It is important to recognize that drift control limits by themselves in wind-sensitive buildings do not provide comfort of the occupants under wind load. See Section L6 for additional information regarding perception to motion in wind sensitive buildings.

L5. VIBRATION

The increasing use of high-strength materials with efficient structural systems and open plan architectural layouts leads to longer spans and more flexible floor systems having less damping. Therefore, floor vibrations have become an important design consideration. Acceleration is the recommended standard for evaluation.

An extensive treatment of vibration in steel-framed floor systems and pedestrian bridges is found in Murray and others (1997). This guide provides basic principles and simple analytical tools to evaluate steel-framed floor systems and footbridges for vibration serviceability due to human activities, including walking and rhythmic activities. Both human comfort and the need to control movement for sensitive equipment are considered.

L6. WIND-INDUCED MOTION

Designers of wind-sensitive buildings have long recognized the need for controlling annoying vibrations under the action of wind to protect the psychological well-being of the occupants (Chen and Robertson, 1972). The perception of building motion under the action of wind may be described by various physical quantities including maximum displacement, velocity, acceleration, and rate of change of acceleration (sometimes called "jerk"). Acceleration has become the standard for evaluation because it is readily measured in the field and can be easily calculated analytically. Human response to building motion is a complex phenomenon involving many psychological and physiological factors. Perception and tolerance thresholds of acceleration as a measure of building motion are known to depend on factors such as frequency of the building, occupant gender, age, body posture (sitting, standing or reclining), body orientation, expectation of motion, body movement, visual cues, acoustic clues, and the type of motion (translational or torsional) (ASCE, 1981). Different thresholds and tolerance levels exist for different people and responses can be very subjective. It is known that some people can become accustomed to building motion and tolerate higher levels than others. Limited research exists on this subject but certain standards have been applied for design as discussed below.

Acceleration in wind-sensitive buildings may be expressed as either root mean square (RMS) or peak acceleration. Both measures are used in practice and there is no clear agreement as to which is the more appropriate measure of motion perception. Some researchers believe that peak acceleration during wind storms is a better measure of actual perception but that RMS acceleration during the entire course of a wind storm is a better measure of actual discomfort. Target peak

accelerations of 21 milli-g (0.021 times the acceleration of gravity) for commercial buildings (occupied mostly during daylight hours) and 15 milli-g for residential buildings (occupied during the entire day) under a 10-year mean recurrence interval wind storm have been successfully used in practice for many tall building designs (Griffis, 1993). The target is generally more strict for residential buildings because of the continuous occupancy, the perception that people are less sensitive and more tolerant at work than at home, the fact that there is more turnover in commercial buildings, and the fact that commercial buildings are more easily evacuated for peak wind events. Peak acceleration and RMS acceleration in wind sensitive buildings are related by the "peak factor" best determined in a wind tunnel study and generally in the range of 3.5 for tall buildings (in other words, peak acceleration = peak factor × RMS acceleration). Guidance for design acceleration levels used in building design may be found in the literature (Chen and Robertson, 1972; Griffis, 1993; Hansen and Reed, 1973; Irwin, 1986; NRCC, 1990).

It is important to recognize that perception to building motion is strongly influenced by building mass and available damping as well as stiffness (Vickery, Isyumov, and Davenport, 1983). For this reason, building drift limits by themselves should not be used as the sole measure of controlling building motion (Islam, Ellingwood, and Corotis, 1990). Damping levels for use in evaluating building motion under wind events are generally taken as approximately 1 percent of critical damping for steel buildings.

L7.   EXPANSION AND CONTRACTION

The satisfactory accommodation of expansion and contraction cannot be reduced to a few simple rules, but must depend largely upon the judgment of a qualified engineer.

The problem is likely to be more serious in buildings with masonry walls than with prefabricated units. Complete separation of the framing at widely spaced expansion joints is generally more satisfactory than more frequently located devices that depend upon the sliding of parts in bearing, and usually less expensive than rocker or roller expansion bearings.

Creep and shrinkage of concrete and yielding of steel are among the causes, other than temperature, for dimensional changes. Conditions during construction, such as temperature effects before enclosure of the structure, should also be considered.

Guidelines for the recommended size and spacing of expansion joints in buildings may be found in NRC (1974).

L8.   CONNECTION SLIP

In bolted connections with bolts in holes having only small clearances, such as standard holes and slotted holes loaded transversely to the axis of the slot, the amount of possible slip is small. Slip at these connections is not likely to have

serviceability implications. Possible exceptions include certain unusual situations where the effect of slip is magnified by the configuration of the structure, such as a connection at the base of a shallow cantilever beam or post where a small amount of bolt slip may produce unacceptable rotation and deflection.

This Specification requires that connections with oversized holes or slotted holes loaded parallel to the axis of the slot be designed as slip-critical connections. For a discussion of slip at these connections see the Commentary to Section J3.8. Where slip at service loads is a realistic possibility in these connections, the effect of connection slip on the serviceability of the structure must be considered.

CHAPTER M

FABRICATION, ERECTION AND QUALITY CONTROL

M1. SHOP AND ERECTION DRAWINGS

Supplementary information relevant to shop drawing documentation and associated fabrication, erection and inspection practices may be found in the *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005) and in Schuster (1997).

M2. FABRICATION

1. Cambering, Curving and Straightening

The use of heat for straightening or cambering members is permitted for A514/A514M and A852/A852M steel, as it is for other steels. However, the maximum temperature permitted is 1,100 °F (590 °C) compared to 1,200 °F (650 °C) for other steels.

Cambering of flexural members, when required by the contract documents, may be accomplished in various ways. In the case of trusses and girders, the desired curvature can be built in during assembly of the component parts. Within limits, rolled beams can be cold-cambered.

Local application of heat has long been used as a means of straightening or cambering beams and girders. The method depends upon an ultimate shortening of the heat-affected zones. A number of such zones, on the side of the member that would be subject to compression during cold-cambering or "gagging," are heated enough to be "upset" by the restraint provided by surrounding unheated areas. Shortening takes place upon cooling.

While the final curvature or camber can be controlled by these methods, it must be realized that some deviation due to workmanship considerations and permanent change due to handling is inevitable. Camber is usually defined by one mid-ordinate, as control of more than one point is difficult and not normally required. Reverse cambers are difficult to achieve and are discouraged. Long cantilevers are sensitive to camber and may deserve closer control.

2. Thermal Cutting

Thermal cutting is preferably done by machine. The requirement for a positive preheat of 150 °F (66 °C) minimum when beam copes and weld access holes are thermally cut in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) and in built-up shapes made of material more than 2 in.

(50 mm) thick tends to minimize the hard surface layer and the initiation of cracks. This requirement for preheat for thermal cutting does not apply when the radius portion of the access hole or cope is drilled and the thermally cut portion is essentially linear. Such thermally cut surfaces are required to be ground and inspected in accordance with Section J1.6.

4.   Welded Construction

To avoid weld contamination, the light oil coating that is generally present after manufacturing an HSS should be removed with a suitable solvent in locations where welding will be performed. In cases where an external coating has been applied at the mill, the coating should be removed at the location of welding or the manufacturer should be consulted regarding the suitability of welding in the presence of the coating.

5.   Bolted Construction

In most connections made with high-strength bolts, it is only required to install the bolts to the snug-tight condition. This includes bearing-type connections where slip is permitted and, for ASTM A325 or A325M bolts only, tension (or combined shear and tension) applications where loosening or fatigue due to vibration or load fluctuations are not design considerations.

It is suggested that snug-tight bearing-type connections with ASTM A325 or A490 bolts be used in applications where A307 bolts are permitted.

This section provides rules for the use of oversized and slotted holes paralleling the provisions that have been in the RCSC Specification since 1972 (RCSC, 2004), extended to include A307 bolts, which are outside the scope of the RCSC Specification.

The Specification previously limited the methods used to form holes, based on common practice and equipment capabilities. Fabrication methods have changed and will continue to do so. To reflect these changes, this Specification has been revised to define acceptable quality instead of specifying the method used to form the holes, and specifically to permit thermally cut holes. AWS C4.7, Sample 3, is useful as an indication of the thermally cut profile that is acceptable (AWS, 1977). The use of numerically controlled or mechanically guided equipment is anticipated for the forming of thermally cut holes. To the extent that the previous limits may have related to safe operation in the fabrication shop, fabricators are referred to equipment manufacturers for equipment and tool operating limits.

10.   Drain Holes

Because the interior of an HSS is difficult to inspect, concern is sometimes expressed regarding internal corrosion. However, good design practice can eliminate the concern and the need for expensive protection.

Corrosion occurs in the presence of oxygen and water. In an enclosed building, it is improbable that there would be sufficient reintroduction of moisture to cause

severe corrosion. Therefore, internal corrosion protection is a consideration only in HSS that are exposed to weather.

In a sealed HSS, internal corrosion cannot progress beyond the point where the oxygen or moisture necessary for chemical oxidation is consumed (AISI, 1970). The oxidation depth is insignificant when the corrosion process must stop, even when a corrosive atmosphere exists at the time of sealing. If fine openings exist at connections, moisture and air can enter the HSS through capillary action or by aspiration due to the partial vacuum that is created if the HSS is cooled rapidly (Blodgett, 1967). This can be prevented by providing pressure-equalizing holes in locations that make it impossible for water to flow into the HSS by gravity.

Situations where an internal protective coating may be required include: (1) open HSS where changes in the air volume by ventilation or direct flow of water is possible; and (2) open HSS subject to a temperature gradient that causes condensation. In such instances it may also be prudent to use a minimum $5/16$ in. (8 mm) wall thickness.

HSS that are filled or partially filled with concrete should not be sealed. In the event of fire, water in the concrete will vaporize and may create pressure sufficient to burst a sealed HSS. Care should be taken to ensure that water does not remain in the HSS during or after construction, since the expansion caused by freezing can create pressure that is sufficient to burst an HSS.

Galvanized HSS assemblies should not be completely sealed because rapid pressure changes during the galvanizing process tend to burst sealed assemblies.

11. Requirements for Galvanized Members

Cracking has been observed in steel members during hot-dip galvanizing. The occurrence of these cracks has been correlated to several characteristics including, but not limited to, highly restrained details, base material chemistry, galvanizing practices, and fabrication workmanship. The requirement to grind beam copes before galvanizing will not prevent all cope cracks from occurring during galvanizing. However, it has been shown to be an effective means to reduce the occurrence of this phenomenon.

Galvanizing of structural steel and hardware such as fasteners is a process that depends on special design detailing and fabrication to achieve the desired level of corrosion protection. ASTM publishes a number of standards relating to galvanized structural steel:

ASTM A123 (ASTM, 2002) provides a standard for the galvanized coating and its measurement and includes provisions for the materials and fabrication of the products to be galvanized.

ASTM A153 (ASTM, 2001) is a standard for galvanized hardware such as fasteners that are to be centrifuged.

ASTM A384 (ASTM, 2002a) is the Standard Practice for Safeguarding Against Warpage and Distortion During Hot-Dip Galvanizing. It includes information on factors that contribute to warpage and distortion as well as suggestions for correction for fabricated assemblies.

ASTM A385 (ASTM, 2001a) is the Standard Practice for Providing High Quality Zinc coatings. It includes information on base materials, venting, treatment of contacting surfaces, and cleaning. Many of these provisions should be indicated on design and detail drawings.

ASTM A780 (ASTM, 2001b) provides for repair of damaged and uncoated areas of hot-dip galvanized coatings.

M3. SHOP PAINTING

1. General Requirements

The surface condition of unpainted steel framing of long-standing buildings that have been demolished has been found to be unchanged from the time of its erection, except at isolated spots where leakage may have occurred. Even in the presence of leakage, the shop coat is of minor influence (Bigos, Smith, Ball, and Foehl, 1954).

This Specification does not define the type of paint to be used when a shop coat is required. Final exposure and individual preference with regard to finish paint are factors that determine the selection of a proper primer. A comprehensive treatment of the subject is found in SSPC (2000).

3. Contact Surfaces

Special concerns regarding contact surfaces of HSS should be considered. As a result of manufacturing, a light oil coating is generally present on the outer surface of the HSS. If paint is specified, HSS must be cleaned of this oil coating with a suitable solvent; see SSPC (2000).

5. Surfaces Adjacent to Field Welds

This Specification allows for welding through surface materials, including appropriate shop coatings that do not adversely affect weld quality nor create objectionable fumes.

M4. ERECTION

2. Bracing

For information on the design of temporary lateral support systems and components for low-rise buildings see Fisher and West (1997).

4. Fit of Column Compression Joints and Base Plates

Tests on spliced full-size columns with joints that had been intentionally milled out-of-square, relative to either strong or weak axis, demonstrated that the load-carrying capacity was the same as that for similar columns without splices (Popov

and Stephen, 1977). In the tests, gaps of $^1/_{16}$ in. (2 mm) were not shimmed; gaps of $^1/_4$ in. (6 mm) were shimmed with nontapered mild steel shims. Minimum size partial-joint-penetration groove welds were used in all tests. No tests were performed on specimens with gaps greater than $^1/_4$ in. (6 mm).

5.   Field Welding

The purpose of wire brushing shop paint on surfaces adjacent to joints to be field welded is to reduce the possibility of porosity and cracking and also to reduce any environmental hazard. Although there are limited tests that indicate that painted surfaces result in sound welds without wire brushing, other tests have resulted in excessive porosity and/or cracking when welding coated surfaces. Wire brushing to reduce the paint film thickness minimizes weld rejection. Grinding or other treatment beyond wire brushing is not necessary.

M5.   QUALITY CONTROL

To facilitate quality control, inspection, and identification, reference should be made to the *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005).

5.   Identification of Steel

Material identification procedures should be sufficient to show the material specification designations and to tie the material to any special material requirements, such as notch toughness when specified.

APPENDIX 1

INELASTIC ANALYSIS AND DESIGN

1.1. GENERAL PROVISIONS

The design of statically indeterminate steel structures according to Appendix 1 is based on their *inelastic strength*. Although design could be performed according to Section B3.4 (ASD) if the appropriate load factor were included in the analysis, this process is more complicated than simply performing design according to Section B3.3 (LRFD). For this reason, only LRFD provisions are provided. An exception is permitted in Section 1.3, as discussed below.

1.2. MATERIALS

Extensive past research on the plastic and inelastic behavior of continuous beams, *rigid frames* and connections has amply demonstrated the suitability of steel with yield stress levels up to 65 ksi (450 MPa) (ASCE, 1971).

1.3. MOMENT REDISTRIBUTION

The provision of Section 1.3 has been a part of the Specification since the 1949 edition. The permission of applying a redistribution of 10 percent of the elastically calculated bending moment at points of interior support due to gravity loading on continuous compact beams gives partial recognition to the philosophy of plastic design. Figure C-A-1.1 illustrates the application of this provision by comparing calculated moment diagrams with the diagrams altered by this provision.

1.4. LOCAL BUCKLING

Inelastic design requires that, up to the formation of the plastic mechanism or up to the peak of the inelastic load-deflection curve, the moments at the plastic hinge locations remain at the level of the plastic moment. This implies that the member must have sufficient inelastic rotation capacity to permit the redistribution of the moments. Sections that are designated as compact in Section B4 have a rotation capacity of approximately 3 and are suitable for plastic design. The limiting width/thickness ratio designated as λ_r in Table B4.1 is the maximum slenderness ratio for this rotation capacity to be achieved. Further discussion of the antecedents of these provisions is given in Commentary Section B4.

The additional slenderness limits in Equations A-1-1 through A-1-4 apply to cases not covered in Table B4.1. The equations for height-to-thickness ratio limits of webs of wide-flange members and rectangular HSS under combined flexure and compression have been taken from Table B5.1 of the 1999 *LRFD Specification* (AISC, 2000b). These provisions have been part of the plastic design requirements

since the 1969 Specification, and they are based on research documented in *Plastic Design in Steel, A Guide and a Commentary* (ASCE, 1971). The equations for the flanges of HSS and other boxed sections (Equation A-1-3) and for round HSS (Equation A-1-4) are from the *Specification for the Design of Steel Hollow Structural Sections* (AISC, 2000).

The use of single-angle, tee and double-angle sections in statically indeterminate beams in plastic design is not recommended since the inelastic rotation capacity in the regions where the moment produces compression in an outstanding leg will typically not be sufficient.

1.5. STABILITY AND SECOND-ORDER EFFECTS

Section 1.5 requires that the equilibrium equations be formulated on the geometry of the deformed structure for frames designed by plastic or inelastic analysis.

Continuous, braced beams not subject to axial loads can be designed by *first-order plastic analysis*. Braced frames and moment frames having small axial loads in the members that are braced to prevent lateral-torsional buckling and loaded so as to produce bending about the major axis only may also be designed by first-order plastic analysis, provided that the requirements of Chapters C (the B_1 and B_2 amplification factors), E (column equations) and H (interaction equations) are accounted for. First-order plastic analysis is treated in ASCE (1971), in steel design textbooks [for example, Salmon and Johnson (1996) and Galambos, Lin, and Johnston (1996)], in textbooks dedicated entirely to plastic design [for example: Horne and Morris, (1982); Chen and Sohal (1995); and Bruneau, Uang, and
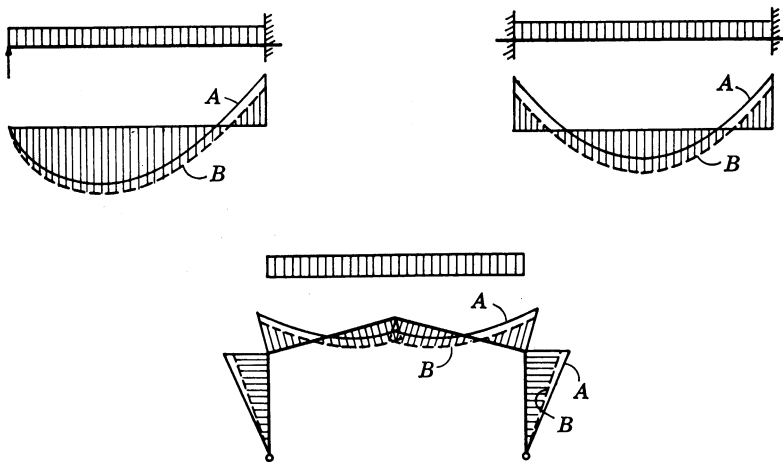
A = Actual moment diagram
B = Modified diagram corresponding to 10 percent moment reduction allowance at interior supports

Fig. C-A-1.1. Examples of effects of 10 percent moment redistribution.

Whittaker (1998)] and in structural engineering handbooks (Gaylord, Gaylord, and Stallmeyer, 1997).

First-order plastic analysis is applicable to continuous beams and low-rise frames with small axial loads. For these simple structures the tools of plastic analysis are readily available to the designer from books giving simple ways of calculating the plastic mechanism loads. This is not so for the case of general moment frames, where a full second-order inelastic analysis must be performed for the determination of the load effects on the members and the connections. The state-of-the-art of inelastic frame analysis is discussed in Chapter 16 of Galambos (1998). Textbooks [for example, Chen and Sohal (1995) and McGuire, Gallagher, and Ziemian (2000)] present the basic approaches to inelastic analysis, as well as worked examples and computer programs for use by students studying the subject.

1. Braced Frames

In Section 1.5.1 two constraints are given for the plastic design of braced frames: (1) the bracing system shall remain elastic; and (2) the axial force in any column must not exceed 85 percent of the *squash load*, $F_y A_g$.

2. Moment Frames

The provision in Section 1.5.2 restricts the axial force in any column to 75 percent of the *squash load*. This provision, as well as the corresponding one in Section 1.5.1, is a cautionary limitation because at high levels of axial force insufficient research has been conducted to ensure that sufficient inelastic rotation capacity remains in the member.

1.6. COLUMNS AND OTHER COMPRESSION MEMBERS

Columns in braced frames and moment frames that are designed on the basis of first-order inelastic analysis or a plastic mechanism analysis are proportioned according to the requirements of Section E3, with an effective length determined by methods of stability analysis. For moment frames, the effective length may exceed unity.

1.7. BEAMS AND OTHER FLEXURAL MEMBERS

The plastic moment, M_p, is the maximum moment that acts at the plastic hinge. When a wide-flange member is subject to flexure about its major axis, the ratio of the plastic moment to the yield moment is approximately 1.1 to 1.2. However, if flexure is about the minor axis, this ratio can exceed 1.6. A limit of $1.6M_y$ is imposed in order to prevent excessive yielding under service loads.

Portions of members that would be required to rotate inelastically as a plastic hinge, while the moments are redistributed to eventually form a plastic mechanism, need more closely spaced bracing than similar parts of a continuous frame designed in accordance with elastic theory. Equations A-1-7 and A-1-8 define the maximum permitted unbraced length in the vicinity of plastic hinges for wide-flange shapes

bent about their major axis, and for rectangular shapes and symmetric box beams, respectively. These equations are identical to those in the 1999 *LRFD Specification* (AISC, 1999). They are different from the corresponding equations in Chapter N of the 1989 *ASD Specification* (AISC, 1989). The new equations are based on research reported in Yura and others (1978).

Some requirements that were in the plastic design chapter of the 1989 *ASD Specification* (AISC, 1989) are no longer explicitly enumerated in Appendix 1. One of these is the provision that web stiffeners are required at a point of load application where a plastic hinge would form. However, the provisions of Section J10 apply for plastic as well as elastic design. No mention is made of shear requirements, but the requirements of Chapter G apply. The plastic shear strength is $V_p = V_n = 0.6F_y A_w$ (Equation G2-1, with C_v equal to 1.0). The maximum permitted plastic web slenderness limit for plastic design is thus equal to

$$(h/t_w)_p = 1.1\sqrt{k_v E/F_y} = 1.1\sqrt{5E/F_y} = 2.5\sqrt{E/F_y} \qquad \text{(C-A-1-1)}$$

with a shear buckling coefficient $k_v = 5$. The plastic shear strength of $0.6F_y A_w$ is a liberalization of the previously used $0.55F_y A_w$ that was recommended in ASCE (1971) based on extensive research.

1.8.   MEMBERS UNDER COMBINED FORCES

Members subject to bending moment and axial force are subject to the provisions of the interaction equations in Section H1. If the member contains a plastic hinge within its span or at its end, and bending is about the major axis of a doubly symmetric section, then the member must be laterally braced near the hinge location (Equation A-1-7 or A-1-8). When the unbraced length of the member exceeds these limits, the inelastic rotation capacity may be impaired, due to the combined influence of lateral and torsional deformation, to such an extent that plastic action is not achievable. However, if the required moment is small enough so the limitations of the interaction equations in Section H1 are fulfilled, the member will be strong enough to function at a joint where required hinge action is provided in another member entering the joint. If the forces on the beam-column include torsion, plastic design is not permitted by this Specification.

1.9.   CONNECTIONS

The connections adjacent to plastic hinges must be designed with sufficient strength and ductility to sustain the forces and deformations imposed under the required loads. The practical implementation of this rule is that the applicable requirements of Chapter J must be strictly adhered to. The provisions for connection design in Chapter J have been developed from plasticity theory and verified by extensive testing, as discussed in ASCE (1971) and in many books and papers. Thus the connections that meet these provisions are inherently qualified for use in plastically designed structures.

APPENDIX 2

DESIGN FOR PONDING

Ponding stability is determined by ascertaining that the conditions of Equations A-2-1 and A-2-2 of Appendix 2 are fulfilled. These equations provide a conservative evaluation of the stiffness required to avoid runaway deflection, giving a factor of safety of four against ponding instability.

Since Equations A-2-1 and A-2-2 yield conservative results, it may be advantageous to perform a more detailed stress analysis to check whether a roof system that does not meet the above equations is still safe against ponding failure.

For the purposes of Appendix 2, *secondary members* are the beams or joists that directly support the distributed ponding loads on the roof of the structure, and *primary members* are the beams or girders that support the concentrated reactions from the *secondary members* framing into them. Representing the deflected shape of the primary and critical *secondary member* as a half-sine wave, the weight and distribution of the ponded water can be estimated, and, from this, the contribution that the deflection each of these members makes to the total ponding deflection can be expressed as follows (Marino, 1966):

For the *primary member*

$$\Delta_w = \frac{\alpha_p \Delta_o \left[1 + 0.25\pi\alpha_s + 0.25\pi\rho(1 + \alpha_s)\right]}{1 - 0.25\pi\alpha_p\alpha_s} \tag{C-A-2-1}$$

For the *secondary member*

$$\delta_w = \frac{\alpha_s \delta_o \left[1 + \frac{\pi^2}{32}\alpha_p + \frac{\pi^2}{8\rho}(1 + \alpha_p) + 0.185\alpha_s\alpha_p\right]}{1 - 0.25\pi\alpha_p\alpha_s} \tag{C-A-2-2}$$

In these expressions Δ_o and δ_o are, respectively, the primary and secondary beam deflections due to loading present at the initiation of ponding, and

$$\alpha_p = C_p / \left(1 - C_p\right)$$
$$\alpha_s = C_s / \left(1 - C_s\right)$$
$$\rho = \delta_o / \Delta_o = C_s / C_p$$

Using the above expressions for Δ_w and δ_w, the ratios Δ_w / Δ_o and δ_w / δ_o can be computed for any given combination of primary and secondary beam framing using the computed values of parameters C_p and C_s, respectively, defined in the Specification.

Even on the basis of unlimited elastic behavior, it is seen that the ponding deflections would become infinitely large unless

$$\left(\frac{C_p}{1 - C_p}\right)\left(\frac{C_s}{1 - C_s}\right) < \frac{4}{\pi} \tag{C-A-2-3}$$

Since elastic behavior is not unlimited, the effective bending strength available in each member to resist the stress caused by ponding action is restricted to the difference between the yield stress of the member and the stress f_o produced by the total load supported by it before consideration of ponding is included.

Note that elastic deflection is directly proportional to stress. The admissible amount of ponding in either the primary or critical (midspan) *secondary member*, in terms of the applicable ratio Δ_w/Δ_o and δ_w/δ_o, can be represented as $(0.8F_y - f_o)/f_o$, assuming a factor of safety of 1.25 against yielding under the ponding load. Substituting this expression for Δ_w/Δ_o and δ_w/δ_o, and combining with the foregoing expressions for Δ_w and δ_w, the relationship between the critical values for C_p and C_s and the available elastic bending strength to resist ponding is obtained. The curves presented in Figures A-2.1 and A-2.2 are based upon this relationship. They constitute a design aid for use when a more exact determination of required flat roof framing stiffness is needed than given by the Specification provision that $C_p + 0.9C_s \leq 0.25$.

Given any combination of primary and secondary framing, the stress index is computed as follows:

For the *primary member*

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \tag{C-A-2-4}$$

For the *secondary member*

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_s \tag{C-A-2-5}$$

where

  f_o = the stress due to $D + R$ (D = nominal dead load, R = nominal load due to rain-water or ice exclusive of the ponding contribution), ksi (MPa)

Depending upon geographic location, this loading should include such amount of snow as might also be present, although ponding failures have occurred more frequently during torrential summer rains when the rate of precipitation exceeded the rate of drainage runoff and the resulting hydraulic gradient over large roof areas caused substantial accumulation of water some distance from the eaves.

Given the size, spacing, and span of a tentatively selected combination of primary and secondary beams, for example, one may enter Figure A-2.1 at the level of the computed stress index U_p, determined for the primary beam; move horizontally to the computed C_s value of the secondary beams; then move downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is larger than the value of C_p computed for

the given *primary member*; if not, a stiffer primary or secondary beam, or combination of both, is required.

If the roof framing consists of a series of equally spaced wall-bearing beams, the beams would be considered as *secondary members*, supported on an infinitely stiff *primary member*. For this case, one would use Figure A-2.2. The limiting value of C_s would be determined by the intercept of a horizontal line representing the U_s value and the curve for $C_p = 0$.

The ponding deflection contributed by a metal deck is usually such a small part of the total ponding deflection of a roof panel that it is sufficient merely to limit its moment of inertia [in.^{4} per foot (mm^4 per meter) of width normal to its span] to 0.000025 (3940) times the fourth power of its span length, as provided in Equation A-2-2. However, the stability against ponding of a roof consisting of a metal roof deck of relatively slender depth-span ratio, spanning between beams supported directly on columns, may need to be checked. This can be done using Figures A-2.1 or A-2.2 with the following computed values:

U_p = stress index for the supporting beam
U_s = stress index for the roof deck
C_p = flexibility constant for the supporting beams
C_s = flexibility constant for one foot width of the roof deck ($S = 1.0$)

Since the shear rigidity of the web system is less than that of a solid plate, the moment of inertia of steel joists and trusses should be taken as somewhat less than that of their chords (Heinzerling, 1987).

APPENDIX 3

DESIGN FOR FATIGUE

When the limit state of fatigue is a design consideration, its severity is most significantly affected by the number of load applications, the magnitude of the stress range, and the severity of the stress concentrations associated with particular details. Issues of fatigue are not normally encountered in building design; however, when encountered and if the severity is great enough, fatigue is of concern and all provisions of Appendix 3 must be satisfied.

3.1. GENERAL

In general, members or connections subject to less than a few thousand cycles of loading will not constitute a fatigue condition except possibly for cases involving full reversal of loading and particularly sensitive categories of details. This is because the applicable cyclic design stress range will be limited by the static design stress. At low levels of cyclic tensile stress, a point is reached where the stress range is so low that fatigue cracking will not initiate regardless of the number of cycles of loading. This level of stress is defined as the *fatigue threshold*, F_{TH}.

Extensive test programs using full-size specimens, substantiated by theoretical stress analysis, have confirmed the following general conclusions (Fisher, Frank, Hirt, and McNamee, 1970; Fisher, Albrecht, Yen, Klingerman, and McNamee, 1974):

(1) Stress range and notch severity are the dominant stress variables for welded details and beams;
(2) Other variables such as minimum stress, mean stress, and maximum stress are not significant for design purposes; and
(3) Structural steels with yield points of 36 to 100 ksi (250 to 690 MPa) do not exhibit significantly different fatigue strengths for given welded details fabricated in the same manner.

3.2. CALCULATION OF MAXIMUM STRESSES AND STRESS RANGES

Fluctuation in stress that does not involve tensile stress does not cause crack propagation and is not considered to be a fatigue situation. On the other hand, in elements of members subject solely to calculated compressive stress, fatigue cracks may initiate in regions of high tensile *residual stress*. In such situations, the cracks generally do not propagate beyond the region of the residual tensile stress, because the *residual stress* is relieved by the crack. For this reason, stress ranges that are completely in compression need not be investigated for fatigue. For cases involving cyclic reversal of stress, the calculated stress range must be taken as the

sum of the compressive stress and the tensile stress caused by different directions or patterns of the applied live load.

3.3. DESIGN STRESS RANGE

Fatigue resistance has been derived from an exponential relationship between the number of cycles to failure N and the stress range, S_r, called an $S - N$ relationship, of the form

$$N = \frac{C_f}{S_r^n} \qquad \text{(C-A-3-1)}$$

The general relationship is often plotted as a linear log-log function ($Log\ N = A - n\ Log\ S_r$). Figure C-A-3.1 shows the family of fatigue resistance curves identified as Categories A, B, B′, C, C′, D, E and E′. These relationships were established based on an extensive database developed in the United States and abroad (Keating and Fisher, 1986). The design stress range has been developed by adjusting the coefficient C_f so that a design curve is provided that lies two standard deviations of the standard error of estimate of the fatigue cycle life below the mean $S - N$ relationship of the actual test data. These values of C_f correspond to a probability of failure of 2.5 percent of the design life.

Prior to the 1999 AISC *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC, 2000b), stepwise tables meeting the above criteria of cycles of loading, stress categories and design stress ranges were provided in the specifications. A single table format (Table A-3.1) was introduced in the 1999 AISC *LRFD Specification* that provides the stress categories, ingredients
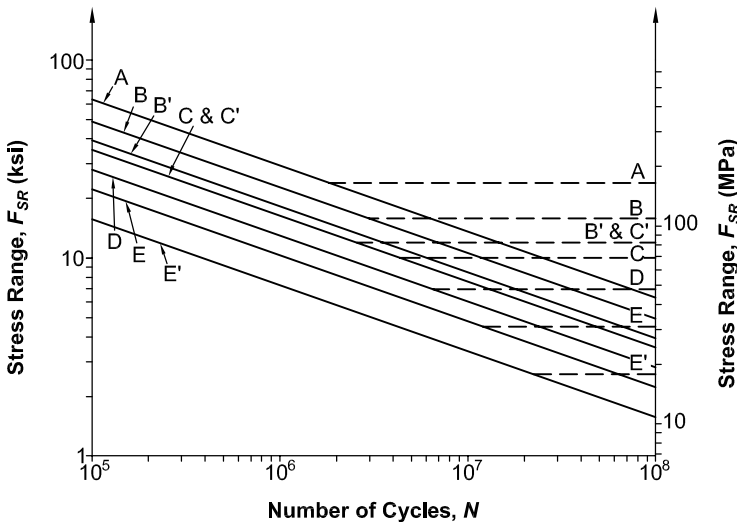

Fig. C-A-3.1. Fatigue resistance curves.

for the applicable equation, and information and examples including the sites of concern for potential crack initiation (AISC, 2000b).

Table A-3.1 is organized into 8 sections of general conditions for fatigue design, as follows:

– Section 1 provides information and examples for the steel material at copes, holes, cutouts or as produced.
– Section 2 provides information and examples for various types of mechanically fastened joints including eyebars and pin plates.
– Section 3 provides information related to welded connections used to join built-up members, such as longitudinal welds, access holes and reinforcements.
– Section 4 deals only with longitudinal load carrying fillet welds at shear splices.
– Section 5 provides information for various types of groove and fillet welded joints that are transverse to the applied cyclic stress.
– Section 6 provides information on a variety of groove welded attachments to flange tips and web plates as well as similar attachments connected with either fillet or partial-joint-penetration groove welds.
– Section 7 provides information on several short attachments to structural members.
– Section 8 collects several miscellaneous details such as shear connectors, shear on the throat of fillet, plug and slot welds, and their impact on base metal. It also provides for tension on the stress area of various bolts, threaded anchor rods and hangers.

A similar format and consistent criteria are used by other specifications.

When fabrication details involving more than one stress category occur at the same location in a member, the stress range at that location must be limited to that of the most restrictive category. The need for a member larger than required by static loading will often be eliminated by locating notch-producing fabrication details in regions subject to smaller ranges of stress.

A detail not explicitly covered before 1989 was added in 1999 to cover tension-loaded plate elements connected at their end by transverse partial-joint-penetration groove or fillet welds in which there is more than a single site for the initiation of fatigue cracking, one of which will be more critical than the others depending upon welded joint type and size and material thickness (Frank and Fisher, 1979). Regardless of the site within the joint at which potential crack initiation is considered, the design stress range provided is applicable to connected material at the toe of the weld.

3.4.   BOLTS AND THREADED PARTS

The fatigue resistance of bolts subject to tension is predictable in the absence of pretension and prying action; provisions are given for such nonpretensioned

details as hanger rods and anchor rods. In the case of pretensioned bolts, deformation of the connected parts through which pretension is applied introduces prying action, the magnitude of which is not completely predictable (Kulak and others, 1987). The effect of prying is not limited to a change in the average axial tension on the bolt but includes bending in the threaded area under the nut. Because of the uncertainties in calculating prying effects, definitive provisions for the design stress range for bolts subject to applied axial tension are not included in this Specification. To limit the uncertainties regarding prying action on the fatigue of pretensioned bolts in details which introduce prying, the design stress range provided in Table A-3.1 is appropriate for extended cyclic loading only if the prying induced by the applied load is small.

Nonpretensioned fasteners are not permitted under this Specification for joints subject to cyclic shear forces. Bolts installed in joints meeting all the requirements for slip-critical connections survive unharmed when subject to cyclic shear stresses sufficient to fracture the connected parts; provisions for such bolts are given in Section 2 of Table A-3.1.

3.5.   SPECIAL FABRICATION AND ERECTION REQUIREMENTS

It is essential that when longitudinal backing bars are to be left in place, they be continuous or spliced using flush-ground complete-joint-penetration groove welds before attachment to the parts being joined. Otherwise, the transverse non-fused section constitutes a crack-like defect that can lead to premature fatigue failure or even *brittle fracture* of the built-up member.

In transverse joints subjected to tension a lack-of-fusion plane in T-joints acts as an initial crack-like condition. In groove welds, the root at the backing bar often has discontinuities that can reduce the fatigue resistance of the connection. Removing the backing, back gouging the joint, and rewelding eliminates the undesirable discontinuities.

The addition of contoured fillet welds at transverse complete-joint-penetration groove welds in T- and corner joints and at reentrant corners reduces the stress concentration and improves fatigue resistance.

Experimental studies on welded built-up beams demonstrated that if the surface roughness of flame-cut edges was less than 1000 μin. (25 μm), fatigue cracks would not develop from the flame-cut edge but from the longitudinal fillet welds connecting the beam flanges to the web (Fisher and others, 1970; Fisher and others, 1974). This provides Category B fatigue resistance without the necessity for grinding flame-cut edges.

Reentrant corners at cuts, copes and weld access holes provide a stress concentration point that can reduce fatigue resistance if discontinuities are introduced by punching or thermal cutting. Reaming sub-punched holes and grinding the

thermally cut surface to bright metal prevents any significant reduction in fatigue resistance.

The use of run-off tabs at transverse butt-joint groove welds enhances weld soundness at the ends of the joint. Subsequent removal of the tabs and grinding of the ends flush with the edge of the member removes discontinuities that are detrimental to fatigue resistance.

APPENDIX 4

STRUCTURAL DESIGN FOR FIRE CONDITIONS

4.1.   GENERAL PROVISIONS

Appendix 4 provides structural engineers with guidance in designing steel-framed building systems and components, including columns, and floor and truss assemblies, for fire conditions. Compliance with the performance objective in Section 4.1.1 can be demonstrated by either structural analysis or component qualification testing.

Thermal expansion and progressive decrease in strength and stiffness are the primary structural responses to elevated temperatures that may occur during fires. An assessment of a design of building components and systems based on structural mechanics that allows designers to address the fire-induced restrained thermal expansions, deformations and material degradation at elevated temperatures can lead to a more robust structural design for fire conditions.

Glossary
Terms pertinent to the design of structural components and systems for fire conditions are presented in the glossary. Terms in common with those in other fire-resistant design documents developed by the SFPE, ICC, NFPA, ASTM and similar organizations are defined in a manner consistent with those documents.

4.1.1.   Performance Objective

The performance objective underlying the provisions and guidelines in this Specification is that of life safety. Fire safety levels should depend on the building occupancy, height of building, the presence of active fire mitigation measures, and the effectiveness of fire-fighting. Three limit states exist for elements serving as fire barriers (compartment walls and floors): (1) heat transmission leading to unacceptable rise of temperature on the unexposed surface; (2) breach of barrier due to cracking or loss of integrity; and (3) loss of load-bearing capacity. In general, all three must be considered by the engineer to achieve the desired performance. These three limit states are interrelated in fire-resistant design. For structural elements that are not part of a separating element, the governing limit state is loss of load-bearing capacity.

Specific performance objectives for a facility are determined by the stakeholders in the building process, within the context of the above general performance objective and limit states. In some instances, applicable building codes may stipulate that steel in buildings of certain occupancies and heights be protected by fire-resistant materials or assemblies to achieve specified performance goals.

4.1.4.   Load Combinations and Required Strength

Fire safety measures are aimed at three levels: (1) to prevent the outbreak of fires through elimination of ignition sources or hazardous practices; (2) to prevent uncontrolled fire development and flashover through early detection and suppression; and (3) to prevent loss of life or structural collapse through fire protection systems, compartmentation, exit ways, and provision of general structural integrity and other passive measures. Specific structural design provisions to check structural integrity and risk of progressive failure due to severe fires can be developed from principles of structural reliability theory (Ellingwood and Leyendecker, 1978; Ellingwood and Corotis, 1991).

The limit state probability of failure due to fire can be written as

$$P(F) = P(F|D,I)P(D|I)P(I) \qquad \text{(C-A-4-1-1)}$$

where $P[I]$ = probability of ignition, $P[D|I]$ = probability of development of a structurally significant fire, and $P[F|D,I]$ = probability of failure, given the occurrence of the two preceding events. Measures taken to reduce $P(I)$ and $P(D|I)$ are mainly nonstructural in nature. Measures taken by the structural engineer to design fire resistance into the structure impact $P(F|D,I)$.

The development of structural design requirements requires a target reliability level, reliability being measured by $P(F)$ in Equation C-A-4-1-1. Analysis of reliability of structural systems for gravity dead and live load (Galambos, Ellingwood, MacGregor, and Cornell, 1982) suggests that the limit state probability of individual steel members and connections is on the order of 10^{-5} to 10^{-4}/year. For redundant steel frame systems, $P(F)$ is on the order of 10^{-6} to 10^{-5}. The *de minimis* risk, that is, the level below which the risk is of regulatory or legal concern and the economic or social benefits of risk reduction are small, is on the order of 10^{-7} to 10^{-6}/year (Pate-Cornell, 1994). If $P(I)$ is on the order of 10^{-4}/year for typical buildings and $P(D|I)$ is on the order of 10^{-2} for office or commercial buildings in urban areas with suppression systems or other protective measures, then $P(F|D,I)$ should be approximately 0.1 to ascertain that the risk due to structural failure caused by fire is socially acceptable.

The use of first-order structural reliability analysis based on this target (conditional) limit state probability leads to the gravity load combination presented as Equation A-4-1. Load combination Equation A-4-1 is the same as Equation C2-3 that appears in Commentary C2.5 of SEI/ASCE 7 (ASCE, 2002), where the probabilistic bases for load combinations for extraordinary events is explained in detail. The factor 0.9 is applied to the dead load when the effect of the dead load is to stabilize the structure; otherwise, the factor 1.2 is applied. The companion action load factors on L and S in that equation reflect the fact that the probability of a coincidence of the peak time-varying load with the occurrence of a fire is negligible (Ellingwood and Corotis, 1991).

Commentary C2.5 of ASCE (2002) contains a second equation that includes 0.2W. That equation is provided so that the stability of the system is checked. The same purpose is accomplished by requiring that the frame be checked under the effect of a small notional lateral load equal to 0.2 percent of story gravity force, acting in combination with the gravity loads. The required strength of the structural component or system designed using these load combinations is on the order of 60 percent to 70 percent of the required strength under full gravity or wind load at normal temperature.

4.2.   STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

4.2.1.   Design-Basis Fire

Once a fuel load has been agreed upon for the occupancy, the designer should demonstrate the effect of various fires on the structure by assessing the temperature-time relationships for various ventilation factors. These relations may result in different structural responses, and it is useful to demonstrate the capability of the structure to withstand such exposures. The effects of a localized fire should also be assessed to ascertain that local damage is not excessive. Based on these results, connections and edge details can be specified to provide a structure that is sufficiently robust.

4.2.1.1.   Localized Fire

Localized fires may occur in large open spaces, such as the pedestrian area of covered malls, concourses of airport terminals, warehouses, and factories, where fuel packages are separated by large aisles or open spaces. In such cases, the radiant heat flux can be estimated by a point source approximation, requiring the heat release rate of the fire and separation distance between the center of the fuel package and the closest surface of the steelwork. The heat release rate can be determined from experimental results or may be estimated if the mass loss rate per unit floor area occupied by the fuel is known. Otherwise, a steady-state fire may be assumed.

4.2.1.2.   Post-Flashover Compartment Fires

Caution should be exercised when determining temperature-time profiles for spaces with high aspect ratios, for example, 5:1 or greater, or for large spaces, for example, those with an open (or exposed) floor area in excess of 5,000 ft^2 (465 m^2). In such cases, it is unlikely that all combustibles will burn in the space simultaneously. Instead, burning will be most intense in, or perhaps limited to, the combustibles nearest to a ventilation source. For modest-sized compartments with low aspect ratios, the temperature history of the design fire can be determined by algebraic equations or computer models, such as those described in the SFPE *Handbook of Fire Protection Engineering* (SFPE, 2002).

4.2.1.3. Exterior Fires

A design guide is available for determining the exposure resulting from an exterior fire (AISI, 1979).

4.2.1.4. Fire Duration

Caution should be exercised when determining the fire duration for spaces with high aspect ratios, for example, 5:1 or greater, or for large spaces, for example, those with a floor area in excess of 5,000 ft^2 (465 m^2). The principal difficulty lies in obtaining a realistic estimate for the mass loss rate, given that all combustibles within the space may not be burning simultaneously. Failure to recognize uneven burning will result in an overestimation of the mass burning rate and an underestimation of the fire duration by a significant margin. Note: some computation methods may implicitly determine the duration of the fire, in which case the calculation of mass loss rate is unnecessary.

Where a parametric curve is used to define a post-flashover fire, the duration is determined by means of the fuel versus ventilation provisions, not explicitly by loss of mass. This clause should not limit the use of temperature-time relationships to those where duration is calculated, as stated above, as these tend to be localized fires and external fire.

4.2.1.5. Active Fire Protection Systems

Due consideration should be given to the reliability and effectiveness of active fire protection systems when describing the design-basis fire. When an automatic sprinkler system is installed, the total fuel load may be reduced by up to 60 percent (Eurocode 1, 1991). The maximum reduction in the fuel load should be considered only when the automatic sprinkler system is considered to be of the highest reliability, for example, reliable and adequate water supply, supervision of control valves, regular schedule for maintenance of the automatic sprinkler system developed in accordance with NFPA (2002), or alterations of the automatic sprinkler system are considered any time alterations for the space are considered.

For spaces with automatic smoke and heat vents, computer models are available to determine the smoke temperature (SFPE, 2002). Reduction in the temperature profile as a result of smoke and heat vents should only be considered for reliable installations of smoke and heat vents. As such, a regular maintenance schedule for the vents needs to be established in accordance with NFPA (2002a).

4.2.2. Temperatures in Structural Systems under Fire Conditions

The heat transfer analysis may range from one-dimensional analyses where the steel is assumed to be at uniform temperature to three-dimensional analyses. The uniform temperature assumption is appropriate in a "lumped heat capacity analysis" where a steel column, beam or truss element is uniformly heated along the entire length and around the entire perimeter of the exposed section

and the protection system is uniform along the entire length and around the entire perimeter of the section. In cases with nonuniform heating or where different protection methods are used on different sides of the column, a one-dimensional analysis should be conducted for steel column assemblies. Two-dimensional analyses are appropriate for beams, bar joists or truss elements supporting floor or roof slabs.

Heat transfer analyses should consider changes in material properties with increasing temperature for all materials included in the assembly. This may be done in the lumped heat capacity analysis using an effective property value, determined at a temperature near the estimated mid-point of the temperature range expected to be experienced by that component over the duration of the exposure. In the one- and two-dimensional analyses, the variation in properties with temperature should be explicitly included.

The boundary conditions for the heat transfer analysis shall consider radiation heat transfer in all cases and convection heat transfer if the exposed element is submerged in the smoke or is being subjected to flame impingement. The presence of fire resistive materials in the form of insulation, heat screens or other protective measures shall be taken into account, if appropriate.

Lumped Heat Capacity Analysis. This first-order analysis to predict the temperature rise of steel structural members can be conducted using algebraic equations iteratively. This approach assumes that the steel member has a uniform temperature, applicable to cases where the steel member is unprotected or uniformly protected (on all sides), and is exposed to fire around the entire perimeter of the assembly containing the steel member. Caution should be used when applying this method to steel beams supporting floor and roof slabs, as the approach will overestimate the temperature rise in the beam. In addition, where this analysis is used as input for the structural analysis of a fire-exposed, steel beam supporting a floor and roof slab, the thermally induced moments will not be simulated as a result of the uniform temperature assumption.

Unprotected steel members. The temperature rise in an unprotected steel section in a short time period shall be determined by

$$\Delta T_s = \frac{a}{c_s \left(\dfrac{W}{D} \right)} (T_F - T_s)\, \Delta t \qquad \text{(C-A-4-2-1)}$$

The heat transfer coefficient, a, is determined from

$$a = a_c + a_r \qquad \text{(C-A-4-2-2)}$$

where
a_c = convective heat transfer coefficient
a_r = radiative heat transfer coefficient, given as

$$a_r = \frac{5.67 \times 10^{-8} \varepsilon_F}{T_F - T_S} \left(T_F^4 - T_s^4 \right)$$

For the standard exposure, the convective heat transfer coefficient, a_c, can be approximated as 25 W/m²-°C. The parameter, ε_F, accounts for the emissivity

TABLE C-A-4-2.1
Guidelines for Estimating ε_F

Type of Assembly	ε_F
Column, exposed on all sides	0.7
Floor beam: Imbedded in concrete floor slab, with only bottom flange of beam exposed to fire	0.5
Floor beam, with concrete slab resting on top flange of beam	
Flange width : beam depth ratio ≥ 0.5	0.5
Flange width : beam depth ratio < 0.5	0.7
Box girder and lattice girder	0.7

of the fire and the view factor. Estimates for ε_F, are suggested in Table C-A-4-2.1.

For accuracy reasons, a maximum limit for the time step, Δt, is suggested as 5 sec.

The fire temperature needs to be determined based on the results of the design fire analysis. As alternatives, the standard time-temperature curves indicated in ASTM E119 (ASTM, 2000) for building fires or ASTM E1529 (ASTM, 2000a) for petrochemical fires may be selected.

Protected Steel Members. This method is most applicable for steel members with contour protection schemes, in other words, where the insulating or (protection) material follows the shape of the section. Application of this method for box protection methods will generally result in the temperature rise being overestimated. The approach assumes that the outside insulation temperature is approximately equal to the fire temperature. Alternatively, a more complex analysis may be conducted which determines the exterior insulation temperature from a heat transfer analysis between the assembly and the exposing fire environment.

If the thermal capacity of the insulation is much less than that for the steel, such that the following inequality is satisfied:

$$c_s W/D > 2d_p \rho_p c_p \qquad \text{(C-A-4-2-3)}$$

Then, Equation C-A-4-2-4 can be applied to determine the temperature rise in the steel:

$$\Delta T_s = \frac{k_p}{c_s d_p \dfrac{W}{D}}(T_F - T_s)\,\Delta t \qquad \text{(C-A-4-2-4)}$$

If the thermal capacity of the insulation needs to be considered (such that the inequality in Equation C-A-4-2-3 is not satisfied), then Equation C-A-4-2-5 should be applied:

$$\Delta T_s = \frac{k_p}{d_p}\left[\frac{T_F - T_s}{c_s \dfrac{W}{D} + \dfrac{c_p \rho_p d_p}{2}}\right]\Delta t \qquad \text{(C-A-4-2-5)}$$

The maximum limit for the time step, Δt, should be 5 sec.

Ideally, material properties should be considered as a function of temperature. Alternatively, material properties may be evaluated at a mid-range temperature expected for that component. For protected steel members, the material properties may be evaluated at 300 °C, and for protection materials, a temperature of 500 °C may be considered.

External Steelwork. Temperature rise can be determined by applying the following equation:

$$\Delta T_s = \frac{q''}{c_s \left(\dfrac{W}{D}\right)} \Delta t \qquad \text{(C-A-4-2-6)}$$

where q'' is the net heat flux incident on the steel member

Advanced Calculation Methods. The thermal response of steel members may be assessed by application of a computer model. A computer model for analyzing the thermal response of the steel members should consider the following:

- Exposure conditions established based on the definition of a design fire. The exposure conditions need to be stipulated either in terms of a time-temperature history, along with radiation and convection heat transfer parameters associated with the exposure, or as an incident heat flux. The incident heat flux is dependent on the design fire scenario and the location of the structural assembly. The heat flux emitted by the fire or smoke can be determined from a fire hazard analysis. Exposure conditions are established based on the definition of a design fire. The exposure conditions are stipulated either in terms of a time-temperature history, along with radiation and convection heat transfer parameters associated with the exposure, or as an incident heat flux.
- Temperature-dependent material properties.
- Temperature variation within the steel member and any protection components, especially where the exposure varies from side to side.

Nomenclature:
    A_m surface area of a member per unit length, ft (m)
    A_p area of the inner surface of the fire protection material per unit length of the member, ft (m)
    A_c cross-sectional area, in.^{2} (m^2)
    D heat perimeter, in. (m)
    T temperature, °F (°C)
    V volume of a member per unit length, in.^{2} (m^2)
    W weight (mass) per unit length, lb/ft (kg/m)
    a heat transfer coefficient, Btu/ft^2·sec·°F (W/m^2·°C)
    c specific heat, Btu/lb·°F (J/kg·°C)
    d thickness, in. (m)

$h_{net,d}$	design value of the net heat flux per unit area, Btu/sec·ft^2 (W/m^2)	
k	thermal conductivity, Btu/ft·sec·°F (W/m·°C)	
l	length, ft (m)	
t	time in fire exposure, seconds	
Δt	time interval, seconds	
ρ	density, lb/ft^3 (kg/m^3)	

Subscripts:

a	steel
c	convection
m	member
p	fire protection material
r	radiation
s	steel
t	dependent on time
T	dependent on temperature

4.2.3.   Material Strengths at Elevated Temperatures

The properties for steel and concrete at elevated temperatures are adopted from the ECCS *Model Code on Fire Engineering* (ECCS, 2001), Section III.2, "Material Properties." These generic properties are consistent with those in Eurocodes 3 (Eurocode 3, 2002) and 4 (Eurocode 4, 2003), and reflect the consensus of the international fire engineering and research community. The background information for the mechanical properties of structural steel at elevated temperatures can be found in Cooke (1988) and Kirby and Preston (1988).

4.2.4.   Structural Design Requirements

The resistance of the structural system in the design basis fire may be determined by:

(a) Structural analysis of individual elements where the effects of restraint to thermal expansion and bowing may be ignored but the reduction in strength and stiffness with increasing temperature is incorporated.

(b) Structural analysis of assemblies/subframes where the effects of restrained thermal expansion and thermal bowing are considered by incorporating geometric and material nonlinearities.

(c) Global structural analysis where restrained thermal expansion, thermal bowing, material degradation and geometric nonlinearity are considered.

4.2.4.1.   General Structural Integrity

The requirement for general structural integrity is consistent with that appearing in Section 1.4 of ASCE (2002). Structural integrity is the ability of the structural system to absorb and contain local damage or failure without developing into a progressive collapse that involves the entire structure or a disproportionately large part of it.

The Commentary C1.4 to Section 1.4 of ASCE (2002) contains guidelines for the provision of general structural integrity. Compartmentation (subdivision of buildings/stories in a building) is an effective means of achieving resistance to progressive collapse as well as preventing fire spread, as a cellular arrangement of structural components that are well tied together provides stability and integrity to the structural system as well as insulation.

4.2.4.2. Strength Requirements and Deformation Limits

As structural elements are heated, their expansion is restrained by adjacent element and connections. Material properties degrade with increasing temperature. Load transfer can occur from hotter elements to adjacent cooler elements. Excessive deformation may be of benefit in a fire as it allows release of thermally induced stresses. Deformation is acceptable once horizontal and vertical separation as well as the overall load bearing capacity of the structural system is maintained.

4.2.4.3. Methods of Analysis

4.2.4.3a. Advanced Methods of Analysis

Advanced methods are required when the overall structural system response to fire, the interaction between structural members and separating elements in fire, or the residual strength of the structural system following a fire must be considered.

4.2.4.3b. Simple Methods of Analysis

Simple methods may suffice when a structural member or component can be assumed to be subjected to uniform heat flux on all sides and the assumption of a uniform temperature is reasonable as, for example, in a free-standing column.

4.2.4.4. Design Strength

The design strength for structural steel members and connections is calculated as ϕR_n, in which R_n = nominal strength, in which the deterioration in strength at elevated temperature is taken into account, and ϕ is the resistance factor. The nominal strength is computed as in Chapters C, D, E, F, G, H, I, J and K of the Specification, using material strength and stiffnesses at elevated temperatures defined in Tables A-4.2.1 and A-4.2.2. While ECCS (2001) and Eurocode 1 (1991) specify partial material factors as equal to 1.0 for "accidental" limit states, the uncertainties in strength at elevated temperatures are substantial and in some cases are unknown. Accordingly, the resistance factors herein are the same as those at ordinary conditions.

4.3. DESIGN BY QUALIFICATION TESTING

Qualification testing is an acceptable alternative to design by analysis for providing fire resistance. It is anticipated that the basis will be ASCE (1998), ASTM (2000) and similar documents.

An unrestrained condition is one in which expansion at the support of a load carrying element is not resisted by forces external to the element and the supported ends are free to expand and rotate. A steel member bearing on a wall in a single span or at the end span of multiple spans should be considered unrestrained when the wall has not been designed and detailed to resist thermal thrust.

REFERENCES

References that are cited both here and elsewhere in the Commentary are listed in the main list of references of the Commentary.

AISI (1979), *Fire-Safe Structural Design—A Design Guide*, American Iron and Steel Institute, Washington, DC.

ASCE (1998), *Standard Calculation Methods for Structural Fire Protection*, ASCE Standard 29-99, American Society of Civil Engineers, Reston, VA.

ASTM (2000a), "Standard Test Methods to Determine Effects of Large Hydrocarbon Pool Fires on Structural Members and Assemblies (Standard E1529-00)," American Society for Testing and Materials, Philadelphia, PA.

Cooke, G.M.E. (1988), "An Introduction to the Mechanical Properties of Structural Steel at Elevated Temperatures," *Fire Safety Journal*, Vol. 13, pp. 45–54.

ECCS (2001), *Model Code on Fire Engineering*, 1st Edition, European Convention for Constructional Steelwork Technical Committee 3, Brussels, Belgium.

Ellingwood, B., and Leyendecker, E.V. (1978), "Approaches for Design Against Progressive Collapse," *Journal of the Structural Division*, ASCE, Vol. 104, No. 3, pp. 413–423.

Ellingwood, B., and Corotis, R.B. (1991), "Load Combinations for Building Exposed to Fires," *Engineering Journal*, AISC, Vol. 28, No. 1, pp. 37–44.

Kirby, B.R. and Preston, R.R. (1988), "High Temperature Properties of Hot-Rolled Structural Steels for Use in Fire Engineering Design Studies," *Fire Safety Journal*, Vol. 13, pp. 27–37.

NFPA (2002), *Standard for the Inspection, Testing, and Maintenance of Water-Based Fire Protection Systems*, NFPA 25, National Fire Protection Association, Quincy, MA.

NFPA (2002a), *Standard on Smoke and Heat Venting*, NFPA 204, National Fire Protection Association, Quincy, MA.

Pate-Cornell, E. (1994), "Quantitative Safety Goals for Risk Management of Industrial Facilities," *Structural Safety*, Vol. 13, No. 3, pp. 145–157.

SFPE (2002), *Handbook of Fire Protection Engineering*, 3rd Edition, DiNenno, P.J. (ed.), National Fire Protection Association, Quincy, MA.

BIBLIOGRAPHY

The following references provide further information on key issues related to fire-resistant design of steel building systems and components, and are representative of the extensive literature on the topic. The references were selected because they are archival in nature or otherwise easily accessible by engineers seeking to design fire-resistance into building structures.

AISI (1980), *Designing Fire Protection for Steel Columns*, American Iron and Steel Institute, Washington, DC.

Bailey, C.G. (2000), "The Influence of the Thermal Expansion of Beams on the Structural Behavior of Columns in Steel-Framed Structures During a Fire," *Engineering Structures*, Vol. 22, No. 7, pp. 755–768.

Bennetts, I.D., and Thomas, I.R. (2002), "Design of Steel Structures under Fire Conditions," *Progress in Structural Engineering and Materials*, Vol. 4, No. 1, pp. 6–17.

Brozzetti, J. and others (1983), "Safety Concepts and Design for Fire Resistance of Steel Structures," IABSE Surveys S-22/83, IABSE Periodica 1/1983, ETH-Honggerberg, Zurich, Switzerland.

Chalk, P.L., and Corotis, R.B. (1980), "Probability Model for Design Live Loads," *Journal of the Structures Division*, ASCE, Vol. 106, No. ST10, pp. 2,017–2,033.

Chan, S.L., and Chan, B.H.M. (2001), "Refined Plastic Hinge Analysis of Steel Frames under Fire," *Steel and Composite Structures*, Vol. 1, No. 1, pp. 111–130.

CIB W14 (1983), "A Conceptual Approach Towards a Probability Based Design Guide on Structural Fire Safety," *Fire Safety Journal*, Vol. 6, No. 1, pp. 1–79.

CIB W14 (2001), "Rational Safety Engineering Approach to Fire Resistance of Buildings," CIB Report No. 269, International Council for Research and Innovation in Building and Construction, Rotterdam, the Netherlands.

Culver, C.G. (1978), "Characteristics of Fire Loads in Office Buildings," *Fire Technology*, Vol. 1491, pp. 51–60.

Gewain, R.G., and Troup, E.W.J. (2001), "Restrained Fire Resistance Ratings in Structural Steel Buildings," *Engineering Journal*, AISC Vol. 38, No. 2, pp. 78–89.

Huang, Z., Burgess, I.W., and Plank, R.J. (2000), "Three-Dimensional Analysis of Composite Steel-Framed Buildings in Fire," *Journal of Structural Engineering*, ASCE, Vol. 126, No. 3, pp. 389–397.

Jeanes, D.C. (1985), "Application of the Computer in Modeling Fire Endurance of Structural Steel Floor Systems," *Fire Safety Journal*, Vol. 9, pp. 119–135.

Kruppa, J. (2000), "Recent Developments in Fire Design," *Progress in Structures Engineering and Materials*, Vol. 2, No. 1, pp. 6–15.

Lane, B. (2000), "Performance-Based Design for Fire Resistance," *Modern Steel Construction*, AISC, December, pp. 54–61.

Lawson, R.M. (2001), "Fire Engineering Design of Steel and Composite Buildings," *Journal of Constructional Steel Research*, Vol. 57, pp. 1,233–1,247.

Lie, T.T. (1978), "Fire Resistance of Structural Steel," *Engineering Journal*, AISC, Vol. 15, No. 4, pp. 116–125.

Lie, T.T., and Almand, K.H. (1990), "A Method to Predict the Fire Resistance of Steel Building Columns," *Engineering Journal*, AISC, Vol. 27, pp. 158–167.

Magnusson, S.E. and Thelandersson, S. (1974), "A Discussion of Compartment Fires," *Fire Technology*, Vol. 10, No. 4, pp. 228–246.

Milke, J.A. (1985), "Overview of Existing Analytical Methods for the Determination of Fire Resistance," *Fire Technology*, Vol. 21, No. 1, pp. 59–65.

Milke, J.A. (1992), "Software Review: Temperature Analysis of Structures Exposed to Fire," *Fire Technology*, Vol. 28, No. 2, pp. 184–189.

Newman, G. (1999), "The Cardington Fire Tests," Proceedings of the North American Steel Construction Conference, Toronto, Canada, AISC, Chicago, Illinois, pp. 28.1–28.22.

Nwosu, D.I., and Kodur, V.K.R. (1999), "Behavior of Steel Frames Under Fire Conditions," *Canadian Journal of Civil Engineering*, Vol. 26, pp. 156–167.

Ruddy, J.L., Marlo, J.P., Ioannides, S.A., and Alfawakiri, F. (2003), *Fire Resistance of Structural Steel Framing*, Steel Design Guide No. 19, American Institute of Steel Construction, Inc., Chicago, IL.

Sakumoto, Y. (1992), "High-Temperature Properties of Fire-Resistant Steel for Buildings," *Journal of Structural Engineering*, ASCE, Vol. 18, No. 2, pp. 392–407.

Sakumoto, Y. (1999), "Research on New Fire-Protection Materials and Fire-Safe Design," *Journal of Structural Engineering*, ASCE, Vol. 125, No. 12, pp. 1,415–1,422.

Toh, W.S., Tan, K.H., and Fung, T.C. (2001), "Strength and Stability of Steel Frames in Fire: Rankine Approach," *Journal of Structural Engineering*, ASCE, Vol. 127, No. 4, pp. 461–468.

Usmani, A.S., Rotter, J.M., Lamont, S., Sanad, A.M., and Gillie, M. (2001), "Fundamental Principles of Structural Behaviour Under Thermal Effects," *Fire Safety Journal*, Vol. 36, No. 8.

Wang, Y.C., and Moore, D.B. (1995), "Steel Frames in Fire: Analysis," *Engineering Structures*, Vol. 17, No. 6, pp. 462–472.

Wang, Y.C., and Kodur, V.K.R. (2000), "Research Toward Use of Unprotected Steel Structures," *Journal of Structural Engineering*, ASCE, Vol. 120, No. 12, pp. 1,442–1,450.

Wang, Y.C. (2000), "An Analysis of the Global Structural Behavior of the Cardington Steel-Framed Building During the Two BRE Fire Tests," *Engineering Structures*, Vol. 22, pp. 401–412.

APPENDIX 5

EVALUATION OF EXISTING STRUCTURES

5.1. GENERAL PROVISIONS

The load combinations referred to in this chapter pertain to gravity loading because it is the most prevalent condition encountered. If other loading conditions are a consideration, such as lateral loads, the appropriate load combination from ASCE (2002) or from the applicable building code should be used. The engineer of record for a project is generally established by the owner.

5.2. MATERIAL PROPERTIES

1. Determination of Required Tests

The extent of tests required depends on the nature of the project, the criticality of the structural system or member evaluated, and the availability of records pertinent to the project. Thus, the engineer of record has the responsibility to determine the specific tests required and the locations from which specimens are to be obtained.

2. Tensile Properties

Samples required for tensile tests should be removed from regions of reduced stress, such as at flange tips at beam ends and external plate edges, to minimize the effects of the reduced area. The number of tests required will depend on whether they are conducted to merely confirm the strength of a known material or to establish the strength of some other steel.

It should be recognized that the yield stress determined by standard ASTM methods and reported by mills and testing laboratories is somewhat greater than the static yield stress because of dynamic effects of testing. Also, the test specimen location may have an effect. These effects have already been accounted for in the nominal strength equations in the Specification. However, when strength evaluation is done by load testing, this effect should be accounted for in test planning because yielding will tend to occur earlier than otherwise anticipated. The static yield stress, F_{ys}, can be estimated from that determined by routine application of ASTM methods, F_y, by the following equation (Galambos, 1978; Galambos, 1998):

$$F_{ys} = R(F_y - 4) \qquad \text{(C-A-5-2-1)}$$
$$[\text{S.I.} : F_{ys} = R(F_y - 27)] \qquad \text{(C-A-5-2-1M)}$$

where

F_{ys} = static yield stress, ksi (MPa)
F_y = reported yield stress, ksi (MPa)
R = 0.95 for tests taken from web specimens
      = 1.00 for tests taken from flange specimens

The R factor in Equation C-A-5-2-1 accounts for the effect of the coupon location on the reported yield stress. Prior to 1997, certified mill test reports for structural shapes were based on specimens removed from the web, in accordance with ASTM A6/A6M (ASTM, 2003). Subsequently the specified coupon location was changed to the flange. During 1997–1998, there was a transition from web specimens to flange specimens as the new provisions of ASTM A6/A6M (ASTM, 2003) were adopted.

4. Base Metal Notch Toughness

The engineer of record shall specify the location of samples. Samples shall be cored, flame cut or saw cut. The engineer of record will determine if remedial actions are required, such as the possible use of bolted splice plates.

5. Weld Metal

Because connections typically are more reliable than structural members, strength testing of weld metal is not usually necessary. However, field investigations have sometimes indicated that complete-joint-penetration groove welds, such as at beam-to-column connections, were not made in accordance with AWS D1.1 (AWS, 2004). The specified provisions in AWS D1.1, Section 5.2.4 provide a means for judging the quality of such a weld. Where feasible, any samples removed should be obtained from compression splices rather than tension splices, because the effects of repairs to restore the sampled area are less critical.

6. Bolts and Rivets

Because connections typically are more reliable than structural members, removal and strength testing of fasteners is not usually necessary. However, strength testing of bolts is required where they can not be properly identified otherwise. Because removal and testing of rivets is difficult, assuming the lowest rivet strength grade simplifies the investigation.

5.3. EVALUATION BY STRUCTURAL ANALYSIS

2. Strength Evaluation

Resistance and safety factors reflect variations in determining strength of members and connections, such as uncertainty in theory and variations in material properties and dimensions. If an investigation of an existing structure indicates that there are variations in material properties or dimensions significantly greater than those anticipated in new construction, the engineer of record should consider the use of more conservative values.

5.4. EVALUATION BY LOAD TESTS

1. Determination of Live Load Rating by Testing

Generally, structures that can be designed according to the provisions of this Specification need no confirmation of calculated results by test. However, special situations may arise when it is desirable to confirm by tests the results of calculations. Minimal test procedures are provided to determine the live load rating of a structure. However, in no case is the live load rating determined by test to exceed that which can be calculated using the provisions of this Specification. This is not intended to preclude testing to evaluate special conditions or configurations that are not adequately covered by this Specification.

It is essential that the engineer of record take all necessary precautions to ascertain that the structure does not fail catastrophically during testing. A careful assessment of structural conditions before testing is a fundamental requirement. This includes accurate measurement and characterization of the size and strength of members, connections and details. All safety regulations of OSHA and other pertinent bodies must be strictly adhered to. Shoring and scaffolding should be used as required in the proximity of the test area to mitigate against unexpected circumstances. Deformations must be carefully monitored and structural conditions must be continually evaluated. In some cases it may be desirable to monitor strains as well.

The engineer of record must use judgment to determine when deflections are becoming excessive and terminate the tests at a safe level even if the desired loading has not been achieved. Incremental loading is specified so that deformations can be accurately monitored and the performance of the structure carefully observed. Load increments should be small enough initially so that the onset of significant yielding can be determined. The increment can be reduced as the level of inelastic behavior increases, and the behavior at this level carefully evaluated to determine when to safely terminate the test. Periodic unloading after the onset of inelastic behavior will help the engineer of record determine when to terminate the test to avoid excessive permanent deformation or catastrophic failure.

It must be recognized that the margin of safety at the maximum load level used in the test may be very small, depending on such factors as the original design, the purpose of the tests, and the condition of the structure. Thus, it is imperative that all appropriate safety measures be adopted. It is recommended that the maximum live load used for load tests be selected conservatively. It should be noted that experience in testing more than one bay of a structure is limited.

The provision limiting increases in deformations for a period of one hour is given so as to have positive means that the structure is stable at the loads evaluated.

2. Serviceability Evaluation

In certain cases serviceability performance must be determined by load testing. It should be recognized that complete recovery (in other words, return to initial

deflected shape) after removal of maximum load is unlikely because of phenomena such as local yielding, slip at the slab interface in composite construction, creep in concrete slabs, localized crushing or deformation at shear connections in slabs, slip in bolted connections, and effects of continuity. Because most structures exhibit some slack when load is first applied, it is appropriate to project the load-deformation curve back to zero load to determine the slack and exclude it from the recorded deformations. Where desirable, the applied load sequence can be repeated to demonstrate that the structure is essentially elastic under service loads and that the permanent set is not detrimental.

5.5.   EVALUATION REPORT

Extensive evaluation and load testing of existing structures is often performed when appropriate documentation no longer exists or when there is considerable disagreement about the condition of a structure. The resulting evaluation is only effective if well documented, particularly when load testing is involved. Furthermore, as time passes, various interpretations of the results can arise unless all parameters of the structural performance, including material properties, strength, and stiffness, are well documented.

APPENDIX 6

STABILITY BRACING FOR COLUMNS AND BEAMS

6.1. GENERAL PROVISIONS

The design requirements of Appendix 6 consider two general types of bracing systems, relative and nodal, as shown in Figure C-A-6.1.

A relative column brace system (such as diagonal bracing or shear walls) is attached to two locations along the length of the column that defines the unbraced length. The relative brace system shown consists of the diagonal and the strut that controls the movement at one end of the unbraced length, *A*, with respect to the other end of the unbraced length, *B*. The diagonal and the strut both contribute to the strength and stiffness of the relative brace system. However, when the strut is a floor beam, its stiffness is large compared to the diagonal so the diagonal controls the strength and stiffness of the relative brace.

A nodal brace controls the movement only at the particular brace point, without direct interaction with adjacent braced points. Therefore to define an unbraced length, there must be additional adjacent brace points as shown in Figure C-A-6.1. The two nodal column braces at *C* and *D* that are attached to the rigid abutment define the unbraced length for which $K = 1.0$ can be used. For beams a cross frame between two adjacent beams at midspan is a nodal brace because it prevents twist of the beams only at the particular cross frame location. The unbraced length is half the span length. The twist at the ends of the two beams is prevented by the beam-to-column connections at the end supports. Similarly, a nodal lateral brace attached at midspan to the top flange of the beams and a rigid support assumes that there is no lateral movement at the column locations.

The brace requirements are intended to enable a member to potentially reach a maximum load based on the unbraced length between the brace points and $K = 1.0$. This is not the same as the no-sway buckling load as illustrated in Figure C-A-6.2 for a braced cantilever. The critical stiffness is 1.0 P_e/L, corresponding to $K = 1.0$. A brace with five times this stiffness is necessary to reach 95 percent of the $K = 0.7$ limit. Theoretically, an infinitely stiff brace is required to reach the no-sway limit. Bracing required to reach specified rotation capacities or ductility limits is beyond the scope of these recommendations. Member inelasticity has no significant effect on the brace requirements (Yura, 1995).

Winter developed the concept of a dual requirement for bracing design: strength and stiffness (Winter, 1958; Winter, 1960). The brace force is a function of the initial column out-of-straightness, Δ_o, and the brace stiffness, β. For a relative

brace system, the relationship between column load, brace stiffness and sway displacement is shown in Figure C-A-6.3. If $\beta = \beta_i$, the critical brace stiffness for a perfectly plumb member, then $P = P_e$ only if the sway deflection gets very large. Unfortunately, such large displacements produce large brace forces. For practical design, Δ must be kept small at the factored load level.
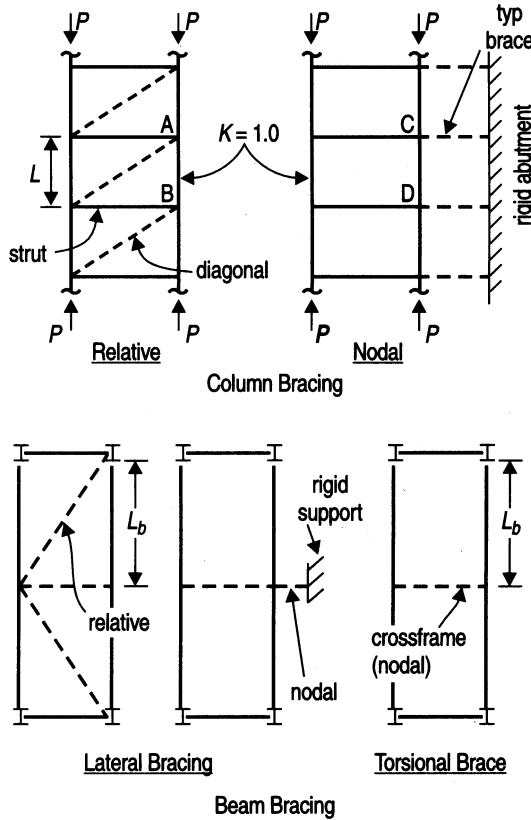
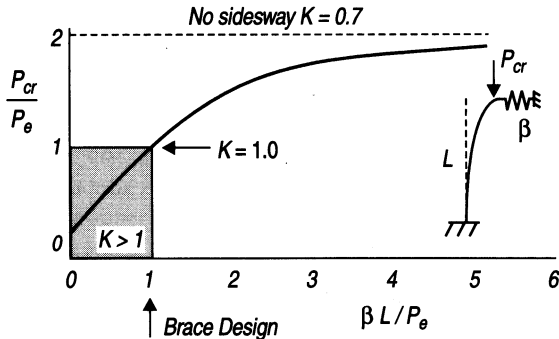*Fig. C-A-6.1. Types of bracing.*

Fig. C-A-6.2. Braced cantilever.

The brace stiffness requirements, β_{br}, for frames, columns, and beams were chosen as twice the critical stiffness. All brace stiffness requirements use a $\phi = 0.75$. For the relative brace system shown in Figure C-A-6.3, $\beta_{br} = 2\beta_i$ gives $P_{br} = 0.4\% P_e$ for $\Delta_o = 0.002L$. If the brace stiffness provided, β_{act}, is different from the requirement, then the brace force or brace moment can be multiplied by the following factor:

$$\frac{1}{2 - \dfrac{\beta_{br}}{\beta_{act}}} \qquad \text{(C-A-6-1)}$$

No ϕ is specified in the brace strength requirements since ϕ is included in the component design strength provisions in other chapters of this Specification.

The initial displacement, Δ_o, for relative and nodal braces is defined with respect to the distance between adjacent braces, as shown in Figure C-A-6.4. The initial Δ_o is a displacement from the straight position at the brace points caused by sources other than brace elongations from gravity loads or compressive forces, such as displacements caused by wind or other lateral forces, erection tolerances, column
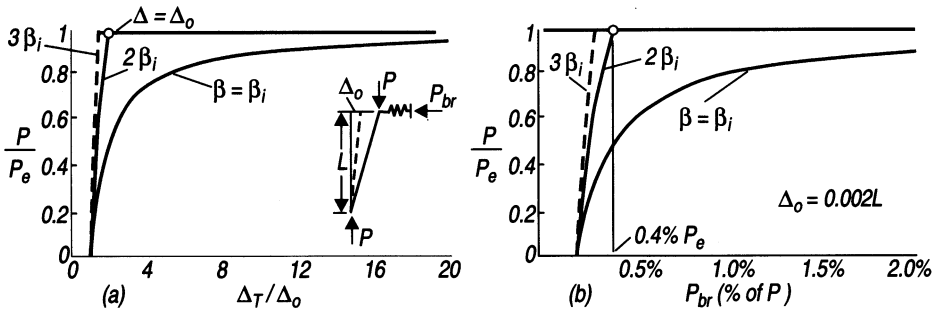

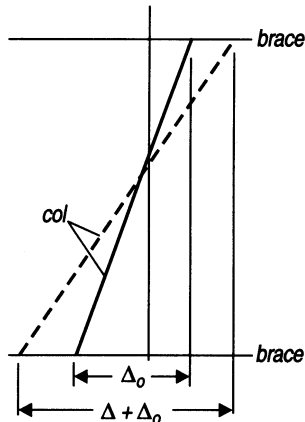*Fig. C-A-6.3. Effect of initial out-of-plumbness.*

Fig. C-A-6.4. Definitions of initial displacements for relative and nodal braces.

shortening, etc. The brace force recommendations for frames, columns and beam lateral bracing are based on an assumed $\Delta_o = 0.002L$, where L is the distance between adjacent brace points. For torsional bracing of beams, an initial twist angle, θ_o, is assumed where $\theta_o = 0.002L/h_o$, and h_o is the distance between flange centroids. For other Δ_o and θ_o values, use direct proportion to modify the brace strength requirements, P_{br} and M_{br}. For cases where it is unlikely that all columns in a story are out-of-plumb in the same direction, Chen and Tong recommend an average $\Delta_o = 0.002L/\sqrt{n_o}$ where n_o columns, each with a random Δ_o, are to be stabilized by the brace system (Chen and Tong, 1994). This reduced Δ_o would be appropriate when combining the stability brace forces with wind and seismic forces.

Brace connections, if they are flexible or can slip, should be considered in the evaluation of the bracing stiffness as follows:

$$\frac{1}{\beta_{act}} = \frac{1}{\beta_{conn}} + \frac{1}{\beta_{brace}} \qquad\qquad \text{(C-A-6-2)}$$

The brace system stiffness, β_{act}, is less than the smaller of the connection stiffness, β_{conn}, or the stiffness of the brace, β_{brace}. Slip in connections with standard holes need not be considered except when only a few bolts are used. When evaluating the bracing of rows of columns or beams, consideration must be given to the accumulation of the brace forces along the length of the brace that results in a different displacement at each beam or column location. In general, brace forces can be minimized by increasing the number of braced bays and using stiff braces.

6.2. COLUMNS

For nodal column bracing, the critical stiffness is a function of the number of intermediate braces (Winter, 1958; Winter, 1960). For one intermediate brace, $\beta_i = 2P/L_b$, and for many braces $\beta_i = 4P/L_b$. The relationship between the critical stiffness and the number of braces, n, can be approximated (Yura, 1995) as $\beta_i = N_i P/L_b$, where $N_i = 4 - 2/n$. The most severe case (many braces) was adopted for the brace stiffness requirement, $\beta_{br} = 2 \times 4P/L_b$. The brace stiffness, Equation A-6-4, can be reduced by the ratio, $N_i/4$, to account for the actual number of braces.

The unbraced length, L_b, in Equation A-6-4 is assumed to be equal to the length L_q that enables the column to reach P_u. When the actual bracing spacing is less than L_q, the calculated required stiffness may become quite conservative since the stiffness equations are inversely proportional to L_b. In such cases, L_q can be substituted for L_b. (This substitution is also applicable for the beam nodal bracing formulations given in Equations A-6-8 and A-6-9.) For example, a W12×53 (W310×79) with $P_u = 400$ kips (1 780 kN) can have a maximum unbraced length of 14 ft (4.3 m) for A36 (A36M) steel. If the actual bracing spacing is 8 ft (2.4 m), then 14 ft (4.3 m) may be used in Equation A-6-4 to determine the required stiffness. The use of L_q in Equation A-6-4 provides reasonable estimates of the brace stiffness requirements; however, the solution can still result in conservative

estimates of the stiffness requirements. Improved accuracy can be obtained by treating the system as a continuous bracing system as discussed in Galambos (1998) and Lutz and Fisher (1985).

With regards to the brace strength requirements, Winter's rigid model only accounts for force effects from lateral displacements and would derive a brace force of 0.8 percent P_u, which accounts only for lateral displacement force effects. To account for the additional force due to member curvature, this theoretical force has been increased to $1.0\% P_u$.

6.3.   BEAMS

Beam bracing must prevent twist of the section, not lateral displacement. Both lateral bracing (for example, joists attached to the compression flange of a simply supported beam) and torsional bracing (for example, a cross frame or diaphragm between adjacent girders) can effectively control twist. Lateral bracing systems that are attached near the beam centroid are ineffective. For beams with double curvature, the inflection point can not be considered a brace point because twist occurs at that point (Galambos, 1998). A lateral brace on one flange near the inflection point also is ineffective. In double curvature cases the lateral brace near the inflection point must be attached to both flanges to prevent twist, or torsional bracing must be used. The beam brace requirements are based on the recommendations in Yura (1993).

1.   Lateral Bracing

For lateral bracing, the following stiffness requirement was derived following Winter's approach:

$$\beta_{br} = 2N_i(C_b P_f)\, C_t C_d/\phi L_b \tag{C-A-6-3}$$

where

N_i = 1.0 for relative bracing
      = $(4-2/n)$ for discrete bracing
n = number of intermediate braces
P_f = beam compressive flange force
      = $\pi^2 EI_{yc}/L_b^2$
I_{yc} = out-of-plane moment of inertia of the compression flange
C_b = moment modifier from Chapter F
C_t = accounts for top flange loading (use $C_t = 1.0$ for centroidal loading)
      = $1 + (1.2/n)$
C_d = double curvature factor (compression in both flanges)
      = $1 + (M_S/M_L)^2$
M_S = smallest moment causing compression in each flange
M_L = largest moment causing compression in each flange

The C_d factor varies between 1.0 and 2.0 and is applied only to the brace closest to the inflection point. The term $(2N_i C_t)$ can be conservatively approximated as

10 for any number of nodal braces and 4 for relative bracing and $(C_b P_f)$ can be approximated by M_u/h which simplifies Equation C-A-6-3 to the stiffness requirements given by Equations A-6-6 and A-6-8. Equation C-A-6-3 can be used in lieu of Equations A-6-6 and A-6-8.

The brace strength requirement for relative bracing is

$$P_{br} = 0.004 M_u C_t C_d / h_o \qquad \text{(C-A-6-4a)}$$

and for nodal bracing

$$P_{br} = 0.01 M_u C_t C_d / h_o \qquad \text{(C-A-6-4b)}$$

They are based on an assumed initial lateral displacement of the compression flange of $0.002 L_b$. The brace strength requirements of Equations A-6-5 and A-6-7 are derived from Equations C-A-6-4a and C-A-6-4b assuming top flange loading $(C_t = 2)$. Equations C-A-6-4a and C-A-6-4b can be used in lieu of Equations A-6-5 and A-6-7, respectively.

2. **Torsional Bracing**

    Torsional bracing can either be attached continuously along the length of the beam (for example, metal deck or slabs) or be located at discrete points along the length of the member (for example, cross frames). With respect to the girder response, torsional bracing attached to the tension flange is just as effective as a brace attached at mid-depth or to the compression flange. Although the girder response is generally not sensitive to the brace location, the position of the brace on the cross section does have an effect on the stiffness of the brace itself. For example, a torsional brace attached on the bottom flange will often bend in single curvature (for example, with a flexural stiffness of $2EI/L$ based on the brace properties), while a brace attached on the top flange will often bend in reverse curvature (for example, with a flexural stiffness of $6EI/L$ based on the brace properties). Partially restrained connections can be used if their stiffness is considered in evaluating the torsional brace stiffness.

    The torsional brace requirements are based on the buckling strength of a beam with a continuous torsional brace along its length presented in Taylor and Ojalvo (1966) and modified for cross-section distortion in Yura (1993).

$$M_u \leq M_{cr} = \sqrt{(C_{bu}M_o)^2 + \frac{C_b^2 EI_y \overline{\beta}_T}{2C_{tt}}} \qquad \text{(C-A-6-5)}$$

    The term $(C_{bu}M_o)$ is the buckling strength of the beam without torsional bracing. $C_{tt} = 1.2$ when there is top flange loading and $C_{tt} = 1.0$ for centroidal loading. $\overline{\beta}_T = n\beta_T/L$ is the continuous torsional brace stiffness per unit length or its equivalent when n nodal braces, each with a stiffness β_T, are used along the span L and the 2 accounts for initial out-of-straightness. Neglecting the unbraced beam buckling term gives a conservative estimate of the torsional brace stiffness requirement (Equation A-6-11).

The strength requirements for beam torsional bracing were developed based upon an assumed initial twist imperfection of $\theta_o = 0.002L_b/h_o$, where h_o is equal to the depth of the beam. Providing at least twice the ideal stiffness results in a brace force, $M_{br} = \beta_T\theta_o$. Using the LRFD formulation of Equation A-6-11 (without ϕ), the strength requirement for the torsional bracing is

$$M_{br} = \beta_T\theta_o = \frac{2.4LM_u^2}{nEI_yC_b^2}\frac{L_b}{500h_o} \qquad \text{(C-A-6-6)}$$

To obtain Equation A-6-9, the equation was simplified as follows:

$$M_{br} = \frac{2.4LM_u^2}{nEI_yC_b^2}\frac{L_b}{500h_o}\frac{\pi^2L_b^2}{\pi^2L_b^2} = \frac{2.4\pi^2M_uL}{500nL_bC_b^2}\frac{M_u}{h_o}\frac{L_b^2}{C_b\pi^2EI_y} \qquad \text{(C-A-6-7)}$$

The term M_u/h_o can be approximated as the flange force, P_f, and the term $L_b^2/C_b\pi^2EI_y$ can be represented as the reciprocal of twice the buckling strength of the flange ($^1/_2P_f$). Substituting for these terms and evaluating the constants results in

$$M_{br} = \frac{0.024M_uL}{nC_bL_b} \qquad \text{(C-A-6-8)}$$

which is the expression given in Equation A-6-9.

Equations A-6-9 and A-6-12 give the strength and stiffness requirements for doubly symmetric beams. For singly symmetric sections these equations will generally be conservative. Better estimates of the strength requirements for torsional bracing of singly symmetric sections can be obtained with Equation C-A-6-6 by replacing I_y with I_{eff} as given in the following expression:

$$I_{eff} = I_{yc} + \frac{t}{c}I_{yt} \qquad \text{(C-A-6-9)}$$

where t is the distance from the neutral axis to the extreme tensile fibers, c is the distance from the neutral axis to the extreme compressive fibers, and I_{yc} and I_{yt} are the respective moments of inertia of compression and tension flanges about an axis through the web. Good estimates of the stiffness requirements of torsional braces for singly symmetric I-shaped beams may be obtained using Equation A-6-11 and replacing I_y with I_{eff} given in Equation C-A-6-9.

The β_{sec} term in Equations A-6-10, A-6-12 and A-6-13 accounts for cross-section distortion. A web stiffener at the brace point reduces cross-sectional distortion and improves the effectiveness of a torsional brace. When a cross frame is attached near both flanges or a diaphragm is approximately the same depth as the girder, then web distortion will be insignificant so β_{sec} equals infinity. The required bracing stiffness, β_{Tb}, given by Equation A-6-10 was obtained by solving the following expression that represents the brace system stiffness including distortion effects:

$$\frac{1}{\beta_T} = \frac{1}{\beta_{Tb}} + \frac{1}{\beta_{sec}} \qquad \text{(C-A-6-10)}$$

Parallel chord trusses with both chords extended to the end of the span and attached to supports can be treated like beams. In Equations A-6-5 through A-6-9, M_u may be taken as the maximum compressive chord force times the depth of the truss to determine the brace strength and stiffness requirements. Cross-section distortion effects, β_{sec}, need not be considered when full-depth cross frames are used for bracing. When either chord does not extend to the end of the span, consideration should be given to control twist near the ends of the span by the use of cross frames or ties.

APPENDIX 7

DIRECT ANALYSIS METHOD

Appendix 7, the direct analysis method, addresses a new method for the stability analysis and design of structural steel systems comprised of moment frames, braced frames, shear walls or combinations thereof (AISC-SSRC, 2003a). While the precise formulation of the method is unique to the AISC Specification, some of its features have similarities to other major design specifications around the world including the Eurocodes, the Australian Standard, the Canadian Standard and ACI 318.

The direct analysis method has been developed with the goal of more accurately determining the load effects in the structure in the analysis stage and eliminating the need for calculating the effective buckling length (K factor) for columns in the first term of the beam-column interaction equations. This method is, therefore, a major step forward in the design of steel moment frames from past editions of the Specification. In addition, the method can be used for the design of braced frames and combined frame systems. Thus, this one method can be used for the design of all types of steel framed structures used in practice. The method can be expanded in the future beyond its use as a second-order elastic analysis tool as presented here. For example, it can be applied with inelastic or plastic analysis. Also, it can be used in the analysis of composite structures, although this application is not explicitly addressed in this Specification.

Chapter C requires that the direct analysis method, as described herein, be used wherever the value of the *sidesway* amplification ratio $\Delta_{2nd\,order}/\Delta_{1st\,order}$ (or B_2 from Equation C2-3), determined from a first-order analysis of the structure, exceeds 1.5. The method may also be used in lieu of the methods described in Chapter C for the analysis and design of any lateral load resisting frame in a steel building.

7.1. GENERAL REQUIREMENTS

There are potentially many parameters and behavioral effects that influence the stability of steel-framed structures (Birnstiel and Iffland, 1980; McGuire, 1992; White and Chen, 1993; ASCE Task Committee on Effective Length, 1997; Deierlein and White, 1998). Three of the most important aspects of stability behavior include geometric nonlinearities, spread-of-plasticity, and member limit states. These aspects ultimately govern frame deformations under applied loads and the resulting load effects in the structure.

Geometric Nonlinearities and Imperfections. Modern stability design provisions are based on the premise that the member forces are calculated by second-order elastic analysis, where equilibrium is satisfied on the deformed geometry of the

structure. The amplification of first-order analysis forces by the traditional B_1 and B_2 factors in Chapter C is one method of conducting an approximate second-order elastic analysis. Where stability effects are significant, consideration must be given to initial geometric imperfections in the structure due to fabrication and erection tolerances. In the development and calibration of the direct analysis method, initial geometric imperfections are conservatively assumed to be equal to the maximum fabrication and erection tolerances permitted by the AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005). For columns and frames, this implies a member out-of-straightness equal to $L/1000$, where L is the member length between brace or framing points, and a frame out-of-plumbness equal to $H/500$, where H is the story height. The out-of-plumbness also may be limited by the absolute bounds specified in the *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005).

Spread of Plasticity. The direct analysis method is also calibrated against inelastic distributed-plasticity analyses that account for the spread of plasticity through the member cross-section and along the member length. The nominal thermal *residual stresses* in W-shape members are assumed to have a maximum value of $0.3F_y$ in compression at the flange tips and to be distributed according to the so-called Lehigh pattern—a linear variation across the flanges and uniform tension in the web (Deierlein and White, 1998).

Member Limit States. Member strength may be controlled by one or more of the following limit states: cross-section yielding, local buckling, flexural buckling, and lateral-torsional or *flexural-torsional buckling*. For beam-columns in single axis flexure and compression, the analysis results from the direct analysis method may be used with the new interaction equations in Chapter H, which address in-plane flexural buckling and out-of-plane lateral torsional instability separately. The separate interaction equations reduce the conservatism in the 1999 *LRFD Specification* (AISC, 2000b) provisions, which combine the two limit state checks into one equation, by using the most severe combination of in-plane or out-of-plane limits for $P_u/\phi P_n$ and $M_u/\phi M_n$. A significant advantage of the direct analysis method is that the in-plane check with P_n in the interaction equation is determined using $K = 1.0$ (in other words, $KL = L$).

Second-Order Analysis. The stability design provisions of Chapter C are developed for use with second-order elastic analysis. It is important that all component and connection deformations that contribute to the lateral displacement of the structure be considered in the analysis. In practice, there are alternative approaches one can employ for conducting second-order analyses, some of which are more rigorous than others.

Rigorous second-order analyses are those that accurately model all significant second-order effects. Rigorous analyses include solution of the governing differential equation, either through stability functions or computer frame analysis programs that model these effects (McGuire, 1992; Deierlein and White, 1998).

Many (but not all) modern commercial computer programs are capable of rigorous analyses, although this should be verified by the user for each particular program. Methods that modify first-order analysis results through second-order amplifiers (for example, B_1 and B_2 factors) are in some cases accurate enough to constitute a rigorous analysis. The use of the B_1 and B_2 amplifiers is permitted, even when $B_2 > 1.5$, provided they are determined using the reduced stiffnesses defined in Equations A-7-2 and A-7-3.

Approximate second-order analyses are those that do not meet the requirements of rigorous analysis. A common type of approximate analysis is one that captures only P-Δ effects due to member end translations (for example, *interstory drift*) but fails to capture P-δ effects due to curvature of the member relative to its chord. Where P-δ effects are significant, errors arise in approximate methods that do not accurately account for the effect of P-δ moments on amplification of both local member moments and the global (Δ) displacements. These errors can occur both with second-order computer analysis programs and with the B_1 and B_2 amplifiers. (Maleck and White, 2003) suggest an equation equivalent to Equation A-7-1 to distinguish cases where P-δ effects can be safely ignored. Alternatively, the engineer should verify the accuracy of the second-order analysis by comparisons to known solutions for conditions similar to those in the structure. Examples of the errors one may encounter are discussed in LeMessurier (1977) and Deierlein and White (1998).

It is suggested that in most building structures, the second-order *sidesway* amplification (or the equivalent B_2), calculated with the reduced stiffness, should be kept no greater than $\Delta_{2nd\ order}/\Delta_{1st\ order} = 2.5$. At larger amplification levels, small changes in gravity loads or stiffnesses result in relatively large changes in *sidesway* deflections and internal second-order forces, due to large geometric nonlinearities. Also note that stiffness requirements for control of seismic drift are included in many building codes that prohibit amplification or B_2 levels from exceeding approximately 1.5 to 1.6 (typically calculated, for steel structures, without use of a reduced stiffness) (ICC, 2003).

Effective Length Method versus the Direct Analysis Method. The effective length method for assessing member axial compressive strength, as discussed in Chapter C of this Commentary, has been used in various forms in the AISC Specification since 1961. The provisions of the current Chapter C are essentially the same as those in the 1999 *LRFD Specification* (AISC, 2000b), with the exception that: (1) limits are placed on the magnitude of second-order effects (as quantified by the ($\Delta_{2nd\ order}/\Delta_{1st\ order}$) or B_2 limit of 1.5); and (2) a minimum lateral load of $0.002Y_i$ (where Y_i is the design gravity load acting on level i) is required to be placed at each level of the structure for all gravity load-only combinations. These limits and requirements are specified for the effective length method (which uses the nominal geometry and elastic stiffness) to limit errors caused by not explicitly accounting in the analysis for initial out-of-plumbness and member stiffness

reduction due to spread of plasticity. The method is based on calculating effective column buckling lengths, *KL,* which have their basis in elastic (or inelastic) stability theory. In the effective length method, the effective buckling length *KL,* or alternatively the equivalent elastic column buckling load, $P_e = \pi^2 EI/(KL)^2$, is used to calculate an axial compressive strength, P_n, through an empirical *column curve* that accounts for geometric imperfections and distributed yielding (including *residual stress* effects). This column strength is then combined with the flexural strength, M_n, and second-order member forces, P_u and M_u, in the beam-column interaction equations.

Differences between the effective length method and the direct analysis method lie predominantly in the in-plane strength check. Figure C-A-7.1(a) shows a plot of the in-plane interaction equation for the effective length method, where the anchor point on the vertical axis, P_{nKL}, is determined using an effective buckling length. Also shown in this plot is the same interaction equation with the first term based on the yield load, P_y. For W-shape members, this in-plane beam-column interaction equation is a reasonable estimate of the internal force state associated with full cross-section plastification. The *P* versus *M* response of a typical member, obtained from second-order spread-of-plasticity analysis and labeled "actual response," indicates the maximum axial force, P_u, that the member can sustain prior to the onset of instability. The load-deflection response from a second-order elastic analysis using the nominal geometry and elastic stiffness, as conducted with the effective length method, is also shown. The "actual response" curve has larger moments than the above second-order elastic curve due to the combined effects of distributed yielding and geometric imperfections, which are not included in the second-order elastic analysis. In the effective length method, the intersection of the second-order elastic analysis curve with the P_{nKL} interaction curve determines the member strength. The plot in Figure C-A-7.1(a) shows that the effective length method is calibrated to give a resultant axial strength, P_u, consistent with the actual response. For slender columns, the calculation of the effective length *KL* (and P_{nKL}) is critical to achieving an accurate solution when using the effective length method.

While the effective length method is calibrated to accurately assess the resultant in-plane member strength, one consequence of the procedure is that it underestimates the actual internal moments under the factored loads (see Figure C-A-7.1(a)). This is inconsequential for the beam-column in-plane strength check (since P_{nKL} reduces the effective strength in the correct proportion); however, the reduced moment can affect the design of the beams and connections, which provide rotational restraint to the column. This is of greatest concern when the calculated moments are small and axial loads are large, such that P-Δ moments induced by column out-of-plumbness can be significant.

A major advantage of the direct analysis method is that it more accurately captures the internal forces in the structure, particularly for the cases where there are high gravity loads and low lateral loads. This advantage comes at the expense of

applying notional lateral loads to the structure and reducing the frame stiffness as part of the analysis input.

7.2. NOTIONAL LOADS

Notional loads are lateral loads that are applied at each framing level and are specified in terms of the gravity loads applied at that level. The gravity loads used to determine the notional load must be equal to or greater than the gravity loads associated with the load combination being evaluated. Notional loads must be applied in the direction that adds to the destabilizing effects under the specified load combination.

The purpose of notional loads is to account for the destabilizing effects of geometric imperfections, nonideal conditions (such as incidental patterned gravity load effects, temperature gradients across the structure, foundation settlement, uneven column shortening, or any other effects that could induce sway that is not explicitly considered in the analysis), inelasticity in structural members, or
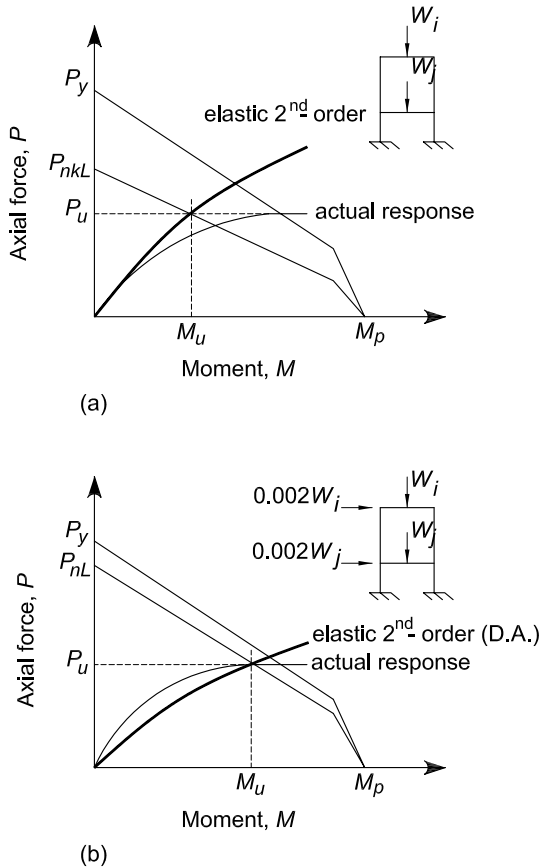

Fig. C-A-7.1. Comparison of in-plane beam-column interaction checks for (a) the effective length method and (b) the direct analysis method.

combinations thereof. While it accounts for any or all of these potential effects, the magnitude of the notional load $0.002Y_i$ can be thought of as representing an initial out-of-plumbness in each story of the structure of 1/500 times the story height. If a smaller value can be justified by the designer, it is permitted to adjust the magnitude of the notional load proportionately. Note that it is also permissible to model the structure in an assumed out-of-plumb state in lieu of applying the notional load.

7.3.   DESIGN-ANALYSIS CONSTRAINTS

The direct analysis method begins with the basic requirement to calculate accurately the internal load effects using a rigorous second-order analysis. This stipulation is placed on the method to afford the luxury of using $K = 1.0$ in the first term of the beam-column interaction equation. In order to obtain accuracy in the calculation of second-order effects, certain constraints must be placed on the method as discussed below.

The first constraint (clause 1) requires that a rigorous second-order analysis be conducted that accounts for both P-Δ and P-δ effects. P-Δ effects are the effects of loads acting on the displaced location of joints or nodes in a structure. P-δ effects are the effect of loads acting on the deflected shape of a member between joints or nodes. Two benchmark problems have been established to determine whether an analysis method meets the requirements of a rigorous second-order analysis adequate for use in the direct analysis method. The problem descriptions and their rigorous differential equation solutions are shown in Figure C-A-7.2. Case 1 is a simply supported beam column subject to a uniform transverse load between supports. This problem contains only a P-δ effect since there is no translation of one end of the member relative to the other. The second problem is a flagpole column with a lateral load at its top. This problem contains both P-Δ and P-δ effects. Figure C-A-7.3 plots the results for the maximum moment and deflection as a function of the applied load P/P_{eL} using the rigorous solution. Note also that if the magnitude of the axial load on the member is less than or equal to $0.15P_{eL}$ (where $P_{eL} = \pi^2 EI/L^2$), then it is permitted to ignore the P-δ effect on the lateral displacement Δ of the structure as the error in doing so is relatively small (Maleck and White, 2003). However, the P-δ effect on the internal moment in the member must be considered (see Figures C-A-7.2 and C-A-7.3). When using the benchmark problems to assess the correctness of a second-order analysis method or computer program, the computer model should utilize joints only at the ends of the member (unless joints are planned on being used along the member length in the actual structure to be modeled). Both moments and deflections should be checked at the location shown for various levels of axial load on the member (including loads that result in moment and deflection amplification, M_{max}/M_o and y_{max}/y_o, of more than 2.5) the results should agree within 3 percent. Other possible benchmark problems can be found in Chen and Lui (1987), which contains the

rigorous solution for a simply-supported beam-column subject to compression and applied end moments and also a solution for a fixed-ended beam-column subject to compression and uniformly distributed loads. Typically, the calculation of accurate internal M_r values is more difficult in problems where member load and/or displacement boundary conditions are not symmetrical.

The second constraint (clause 2) requires the application of a notional load $N_i = 0.002Y_i$, where Y_i is the gravity load from the appropriate load combination acting on level i. The notional loads are required to account for the destabilizing effects of initial imperfections and other conditions that may induce sway not explicitly modeled in the structure. Note that the notional load coefficient 0.002 is based on an initial out-of-plumbness ratio from all effects equal to 1/500. Where a different value can be justified, the coefficient may be adjusted proportionally. When second-order effects are kept to a level so that the *sidesway* amplification $\Delta_{2nd\ order}/\Delta_{1st\ order}$ or $B_2 \leq 1.5$ (1.71 using the reduced elastic stiffness), then it is permitted to apply the notional loads only in the gravity load-only combinations and not in combination with other lateral loads. At this low range of *sidesway*
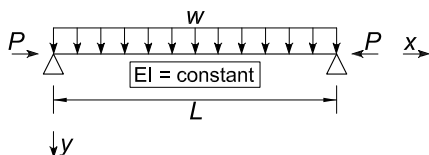
$$M_{MAX}\left(@x = \frac{L}{2}\right) = \frac{wL^2}{8}\left[\frac{2(\sec u - 1)}{u^2}\right] \quad \text{where } u = \sqrt{\frac{PL^2}{4EI}}, \; M_o = \frac{wL^2}{8}$$

$$y_{MAX}\left(@x = \frac{L}{2}\right) = \frac{5wL^4}{384EI}\left[\frac{12(2\sec u - u^2 - 2)}{5u^4}\right] \quad \text{where } y_o = \frac{5wL^4}{384EI}$$

Case 1
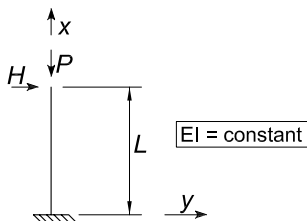
$$M_{MAX}(@x = 0) = HL\left(\frac{\tan \alpha}{\alpha}\right) \quad \text{where } \alpha = \sqrt{\frac{PL^2}{EI}}, \; M_o = HL$$

$$y_{MAX}(@x = L) = \frac{HL^3}{3EI}\left(\frac{3(\tan \alpha - \alpha)}{\alpha^3}\right) \quad \text{where } y_o = \frac{HL^3}{3EI}$$

Case 2

Fig. C-A-7.2. Benchmark problems.

amplification or B_2, the resulting errors in the internal forces are relatively small. If the notional loads are applied in combination with other lateral loads, there is no need for checking a B_2 limit. In all cases it is permitted to use the assumed out-of-plumbness geometry in the analysis of the structure in lieu of applying notional loads as an acceptable way to account for the geometric imperfection effects.

The third constraint (clauses 3 and 4) requires that the analysis be based on a reduced stiffness ($EI^* = 0.8\tau_b EI$ and $EA^* = 0.8EA$) in the structure. There are two reasons for imposing the reduced stiffness for analysis. First, for frames with slender members, where the limit state is governed by elastic stability, the
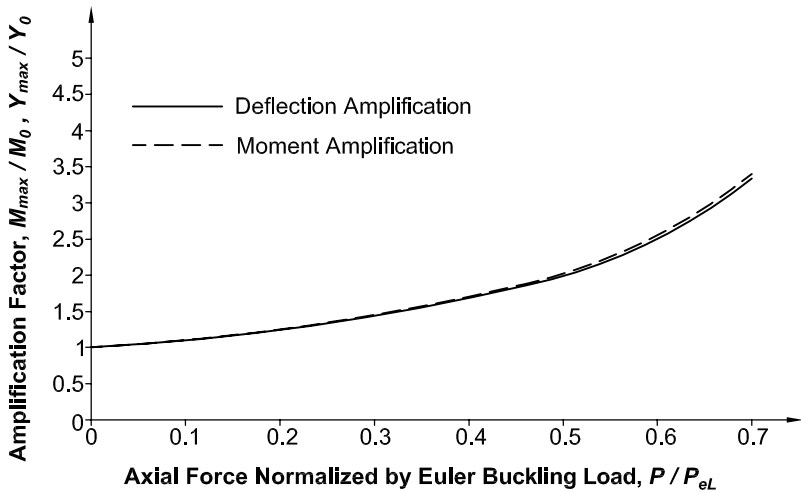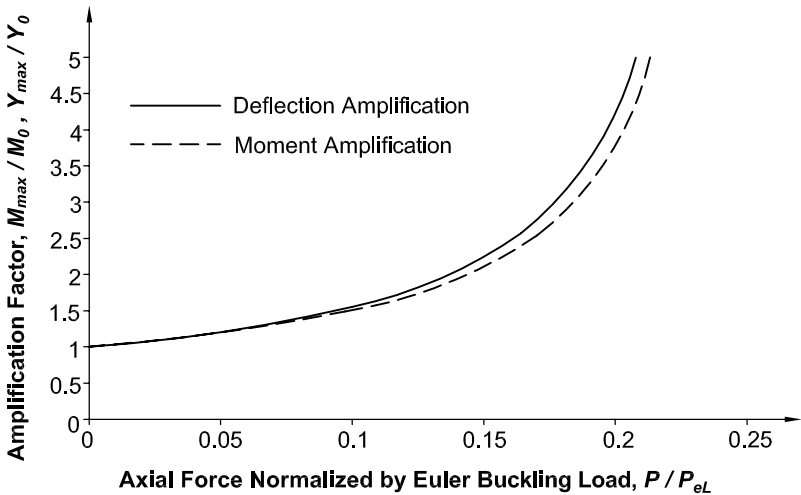
Fig. C-A-7.3. Maximum moment and deflection values as a function of axial force for benchmark problems.

0.8 factor on stiffness results in a system available strength equal to 0.8 times the elastic stability limit. This is roughly equivalent to the margin of safety implied by design of slender columns by the effective length procedure where the design strength $\phi P_n = 0.9(0.877)P_e = 0.79 P_e$ where P_e is the elastic *critical load*, 0.90 is the specified resistance factor, and 0.877 is a reduction factor in the *column curve* equation (Equation E3-3). Second, for frames with intermediate or stocky columns, the $0.8\tau_b$ factor reduces the stiffness to account for inelastic softening prior to the members reaching their design strength. The τ_b factor is similar to the inelastic stiffness reduction factor implied in the *column curve* to account for loss of stiffness under high compression loads ($P_u > 0.5 P_y$), and the 0.8 factor accounts for additional softening under combined axial compression and bending. It is a fortuitous coincidence that the reduction coefficients for both slender and stocky columns are close enough, such that the single reduction factor of $0.8\tau_b$ works over the full range of slenderness. The reduced stiffness and notional load requirements only pertain to analyses for strength limit states. They do not apply to analyses of serviceability conditions of excessive deflections, vibration, etc. For ease of application in design practice, where $\tau_b = 1$, the reduction on EI and EA can be applied by modifying E in the analysis. However, for computer programs that do semi-automated design, one should ascertain that the reduced E is applied only for the second-order analysis. The elastic modulus should not be reduced in nominal strength equations that include E (for example, M_n for laterally unbraced beams). As shown in Figure C-A-7.1(b), the net effect of modifying the analysis in the manner just described is to amplify the second-order forces such that they are closer to the actual internal forces in the structure. It is for this reason that the beam-column interaction for in-plane flexural buckling is checked using an axial strength P_{nL} calculated from the *column curve* using the actual unbraced member length L, in other words, with $K = 1.0$.

In cases where the flexibility of other structural components (for example, connections, flexible column base details, or horizontal trusses acting as diaphragms) is modeled explicitly in the analysis, the stiffness of the other structural components should be reduced as well. Conservatively, the stiffness reduction may be taken as $EA^* = 0.8EA$ and/or $EI^* = 0.8EI$ for all cases. Surovek-Maleck, White, and Leon (2004) discuss the appropriate reduction of connection stiffnesses in the analysis of PR frames.

Simplified First-Order Analysis Based on the Direct Analysis Method (K = 1.0). The direct analysis method provides the technical basis for the provisions of Section C2.2b for design by first-order elastic analysis with $K = 1.0$ (Kuchenbecker, White, and Surovek-Maleck, 2004). The method is based on an assumed out-of-plumbness in the structure $\Delta_o / L = 0.002$, a target maximum drift ratio Δ / L, and reduced stiffnesses in the frame members ($0.8\tau_b EI$ and $0.8EA$). The first-order analysis is carried out using the nominal (unreduced) stiffness, and the above stiffness reduction is accounted for solely within the calculation of amplification factors. The method is applicable to braced, moment and combined frames. This

method has a number of distinct advantages compared to the amplified first-order elastic approach specified in Chapter C:

(1) The second-order internal forces and moments are determined directly as part of the first-order analysis.
(2) There is no need to subdivide the analysis into artificial NT and LT parts.

Kuchenbecker and others (2004) present a general form of the suggested method. If the above approach is employed, it can be shown that for $B_2 \leq 1.5$ and $\tau_b = 1.0$ the required additional lateral load to be applied with other lateral loads in a first-order analysis of the structure, using the nominal (unreduced) stiffness, can be determined as:

$$N_i = \left(\frac{B_2}{1 - 0.2B_2} \right) \frac{\Delta}{L} Y_i \geq \left(\frac{B_2}{1 - 0.2B_2} \right) 0.002Y_i \qquad \text{(C-A-7-3-1)}$$

where B_2 and Y_i are as defined in Chapter C, and Δ_H/L is the target maximum first-order drift ratio due to either the LRFD strength load combinations or 1.6 times the ASD strength load combinations. Note that if B_2 (based on the unreduced stiffness) is set to the 1.5 limit prescribed in Chapter C, then,

$$N_i = 2.1 \, (\Delta/L)Y_i \geq 0.0042 \, Y_i \qquad \text{(C-A-7-3-2)}$$

This is the additional lateral load required in Section C2.2b(2) of Chapter C.

REFERENCES

AASHTO (1998), *Load and Resistance Factor Design Specification for Highway Bridges*, 2nd edition, American Association of State Highway and Transportation Officials, Washington, DC.

ACI (1997), *Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures*, ACI 209R-92, American Concrete Institute, Farmington Hills, MI.

ACI (2001), *Code Requirements for Nuclear Safety Related Concrete Structures*, ACI 349-01, American Concrete Institute, Farmington Hills, MI.

ACI (2002), *Building Code Requirements for Structural Concrete*, ACI 318-02 and ACI 318M-02, American Concrete Institute, Farmington Hills, MI.

AISC (1969), *Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings*, American Institute of Steel Construction, Inc., Chicago, IL.

AISC (1973), "Commentary on Highly Restrained Welded Connections," *Engineering Journal*, AISC, Vol. 10, No. 3, 3rd Quarter, pp. 61–73.

AISC (1975), *Australian Standard AS1250*, Australian Institute of Steel Construction, Sydney, Australia.

AISC (1978), *Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings*, American Institute of Steel Construction, Inc., Chicago, IL.

AISC (1986), *Load and Resistance Factor Design Specification for Structural Steel Buildings*, American Institute of Steel Construction, Inc., Chicago, IL.

AISC (1989), *Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design*, American Institute of Steel Construction, Inc., Chicago, IL.

AISC (1989), *Allowable Stress Design Manual of Steel Construction*, American Institute of Steel Construction, Inc., Chicago, IL.

AISC (1989), *Specification for Allowable Stress Design of Single Angle Members*, American Institute of Steel Construction, Inc., Chicago, IL.

AISC (1993), *Load and Resistance Factor Design Specification for Structural Steel Buildings*, American Institute of Steel Construction, Inc., Chicago, IL.

AISC (1997), *A Guide to Engineering and Quality Criteria for Steel Structures*, American Institute of Steel Construction, Inc, Chicago, IL.

AISC (1997a), "AISC Advisory Statement on Mechanical Properties Near the Fillet of Wide Flange Shapes and Interim Recommendations, January 10, 1997," *Modern*

Steel Construction, American Institute of Steel Construction, Chicago, IL, February, p. 18.

AISC (2000), *Specification for the Design of Steel Hollow Structural Sections*, American Institute of Steel Construction, Inc., Chicago, IL.

AISC (2000a), *LRFD Specification for Single Angle Members*, American Institute of Steel Construction, Inc., Chicago, IL.

AISC (2000b), *Specification for Structural Steel Buildings—Load and Resistance Factor Design*, December 27, 1999, American Institute of Steel Construction, Inc., Chicago, IL.

AISC (2001), *LRFD Manual of Steel Construction,* 3rd ed., American Institute of Steel Construction, Inc., Chicago, IL.

AISC (2005), *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341–05, American Institute of Steel Construction, Inc., Chicago, IL.

AISC (2005), *Code of Standard Practice for Steel Buildings and Bridges,* American Institute of Steel Construction, Inc., Chicago, IL.

AISC (2005a), *Manual of Steel Construction,* American Institute of Steel Construction, Inc., Chicago, IL.

AISC-SSRC (2003), "Basic Design for Stability: Lecture 3—Frame Stability—Alignment Charts and Modifications," American Institute of Steel Construction and Structural Stability Research Council, Chicago, IL.

AISC-SSRC (2003a), "Background and Illustrative Examples on Proposed Direct Analysis Method for Stability Design of Moment Frames," Technical White Paper, AISC Technical Committee 10, AISC-SSRC Ad Hoc Committee on Frame Stability, American Institute of Steel Construction, Inc., Chicago, IL.

AISI (1970), "Interior Corrosion of Structural Steel Closed Sections," Bulletin 18, February, American Iron and Steel Institute, Washington, DC.

AISI (1969), *Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, DC.

AISI (2001), *North American Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, DC.

Ang, K.M., and Morris, G.A. (1984), "Analysis of Three-Dimensional Frames with Flexible Beam-Column Connections," *Canadian Journal of Civil Engineering*, Vol. 11, No. 2, pp. 245–254.

API (1993), *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms—Load and Resistance Factor Design*, 1st edition, American Petroleum Institute, Washington, DC, July.

Amrine, J.J., and Swanson, J.A. (2004), "Effects of Variable Pretension on Bolted Connection Behavior," *Engineering Journal*, AISC, Vol. 41, No. 3, 3rd Quarter, pp. 107–116.

ASCE (1971), *Plastic Design in Steel, A Guide and a Commentary*, ASCE Manuals and Reports on Engineering Practice No. 41, American Society of Civil Engineers, New York, NY.

ASCE (1979), *Structural Design of Tall Steel Buildings*, American Society of Civil Engineers, New York, NY.

ASCE (1981), "Planning and Environmental Criteria for Tall Buildings, A Monograph on Planning and Design of Tall Buildings," Vol. PC, Chapter PC-13, American Society of Civil Engineers, New York, NY.

ASCE (1999), *Specification for Structural Steel Beams with Web Openings*, SEI/ASCE 23–97, American Society of Civil Engineers, Reston, VA.

ASCE (2000), *Design of Latticed Steel Transmission Structures*, ASCE 10-97, American Society of Civil Engineers, Reston, VA.

ASCE (2002), *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-02, American Society of Civil Engineers, Reston, VA.

ASCE (2003), *Standard Calculation Methods for Structural Fire Protection*, SEI/ASCE/SFPE 29-99, American Society of Civil Engineers, Reston, VA.

ASCE Task Committee on Drift Control of Steel Building Structures (1988), "Wind Drift Design of Steel-Framed Buildings: State of the Art," *Journal of the Structural Division*, ASCE, Vol. 114, No. 9, pp. 2,085–2,108.

ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete (1992), "Proposed Specification for Structural Steel Beams with Web Openings," *Journal of Structural Engineering*, ASCE, Vol. 118, No. ST12, December, pp. 3,315–3,324.

ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete (1992a), "Commentary on Proposed Specification for Structural Steel Beams with Web Openings," *Journal of Structural Engineering*, ASCE, Vol. 118, No. ST12, December, pp. 3,325–3,349.

ASCE Task Committee on Effective Length (1997), *Effective Length and Notional Load Approaches for Assessing Frame Stability: Implications for American Steel Design*, American Society of Civil Engineers, New York, NY.

Aslani, F. and Goel, S.C. (1991), "An Analytical Criteria for Buckling Strength of Built-Up Compression Members," *Engineering Journal*, AISC, Vol. 28, No. 4, 4th Quarter, pp. 159–168.

ASTM (2000), *Standard Test Methods for Fire Tests of Buildings and Construction Materials*, ASTM E119-00a, American Society for Testing and Materials, West Conshohocken, PA.

ASTM (2001), *Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware,* ASTM A153/A153M-01a, American Society for Testing and Materials, West Conshohocken, PA.

ASTM (2001a), *Standard Practice for Providing High-Quality Zinc Coatings (Hot-Dip)*, ASTM A385-01, American Society for Testing and Materials, West Conshohocken, PA.

ASTM (2001b), *Standard Practice for Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings*, ASTM A780-01, American Society for Testing and Materials, West Conshohocken, PA.

ASTM (2002), *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products*, ASTM A123/A123M-02, American Society for Testing and Materials, West Conshohocken, PA.

ASTM (2002a), *Standard Practice for Safeguarding Against Warpage and Distortion During Hot-Dip Galvanizing of Steel Assemblies*, ASTM A384/A384M-02, American Society for Testing and Materials, West Conshohocken, PA.

ASTM (2003), *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*, ASTM A6/A6M-04b, American Society for Testing and Materials, West Conshohocken, PA.

ATC (1978), "Tentative Provisions for the Development of Seismic Regulations for Buildings," Publication 3-06, Applied Technology Council, Redwood City, CA, June.

Austin, W.J. (1961), "Strength and Design of Metal Beam-Columns," *Journal of the Structural Division*, ASCE, Vol. 87, No. ST4, April, pp. 1–32.

AWS (1977), *Criteria for Describing Oxygen-Cut Surfaces*, AWS C4.1-77, American Welding Society, Miami, FL.

AWS (2004), *Structural Welding Code—Steel*, AWS D1.1/D1.1M:2004, 19th edition, American Welding Society, Miami, FL.

Bartlett, R.M., Dexter, R.J., Graeser, M.D., Jelinek, J.J., Schmidt, B.J., and Galambos, T.V. (2003), "Updating Standard Shape Material Properties Database for Design and Reliability," *Engineering Journal,* AISC, Vol. 40, No. 1, pp. 2–14.

Basler, K., Yen, B.T., Mueller, J.A., and Thurlimann, B. (1960) "Web Buckling Tests on Welded Plate Girders," Welding Research Council Bulletin No. 64, September.

Basler, K. (1961), "Strength of Plate Girders in Shear," *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, October, pp. 151–180.

Basler, K., and Thurlimann, B. (1963), "Strength of Plate Girders in Bending," *Transactions of the American Society of Civil Engineers*, Vol. 128, Part II, p. 655–682.

Bigos, J., Smith, G.W., Ball, E.F., and Foehl, P.J. (1954), "Shop Paint and Painting Practice," *Proceedings of AISC National Engineering Conference*, Milwaukee, WI, American Institute of Steel Construction, Chicago, IL.

Birkemoe, P.C., and Gilmor, M.I. (1978), "Behavior of Bearing-Critical Double-Angle Beam Connections," *Engineering Journal*, AISC, Vol. 15, No. 4, 4th Quarter, pp. 109–115.

Birnsteil, C., and Iffland, J.S.B. (1980), "Factors Influencing Frame Stability," *Journal of the Structures Division*, ASCE, Vol. 106, No. 2, pp. 491–504.

Bjorhovde, R. (1972), "Deterministic and Probabilistic Approaches to the Strength of Steel Columns," Ph.D. Dissertation, Lehigh University, Bethlehem, PA, May.

Bjorhovde, R. (1978), "The Safety of Steel Columns," *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, September, pp. 1371–1387.

Bjorhovde, R. (1988), "Columns: From Theory to Practice," *Engineering Journal*, AISC, Vol. 25, No. 1, 1st Quarter, pp. 21–34.

Bjorhovde, R., Brozzetti, J., and Colson, A. (eds.) (1988), *Connections in Steel Structures: Behaviour, Strength and Design*, Elsevier Applied Science, London, England.

Bjorhovde, R., Colson, A., and Brozzetti, J. (1990), "Classification System for Beam-to-Column Connections," *Journal of Structural Engineering*, ASCE, Vol. 116, No. 11, pp. 3,059–3,076.

Bjorhovde, R., Colson, A., Haaijer, G., and Stark, J.W.B. (eds.) (1992), *Connections in Steel Structures II: Behavior, Strength and Design*, American Institute of Steel Construction, Inc., Chicago, IL.

Bjorhovde, R., Colson, A., and Zandonini, R. (eds.) (1996), *Connections in Steel Structures III: Behaviour, Strength and Design*, Pergamon Press, London, England.

Bjorhovde, R., Goland, L.J., and Benac, D.J. (1999), "Tests of Full-Scale Beam-to-Column Connections," Southwest Research Institute, San Antonio, TX.

Bleich, F. (1952), *Buckling Strength of Metal Structures*, McGraw-Hill, New York, NY.

Blodgett, O.W. (1967), "The Question of Corrosion in Hollow Steel Sections," *Welding Design Studies in Steel Structures*, Lincoln Electric Company, D610.163, August, Cleveland, OH.

Brandt, G.D. (1982), "A General Solution for Eccentric Loads on Weld Groups," *Engineering Journal*, AISC, Vol. 19, No. 3, 3rd Quarter, pp. 150–159.

Brockenbrough, R.B., and Johnston, B.G. (1981), *USS Steel Design Manual*, United States Steel Corporation, Pittsburgh, PA.

Brockenbrough, R.L. (1983), "Considerations in the Design of Bolted Joints for Weathering Steel," *Engineering Journal*, AISC, Vol. 20, No. 1, 1st Quarter, pp. 40–45.

Brosnan, D.P., and Uang, C.M. (1995), "Effective Width of Composite L-Beams in Buildings," *Engineering Journal*, AISC, Vol. 30, No. 2, pp. 73–81.

Bruneau, M., Uang, C.-M., and Whittaker, A. (1998), *Ductile Design of Steel Structures*, McGraw Hill, New York, NY.

Butler, L.J., Pal, S., and Kulak, G.L. (1972), "Eccentrically Loaded Welded Connections," *Journal of the Structural Division*, ASCE, Vol. 98, No. ST5, May, pp. 989–1,005.

Carter, C.J. (1999), *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications*, Steel Design Guide Series No. 13, American Institute of Steel Construction, Inc., Chicago, IL.

Charney, F.A. (1990), "Wind Drift Serviceability Limit State Design of Multi-story Buildings" *Journal of Wind Engineering and Industrial Aerodynamics*, Vol. 36.

Chen, P.W., and Robertson, L.E. (1972) "Human Perception Thresholds of Horizontal Motion," *Journal of Structural Engineering*, ASCE, Vol. 98, No. ST8, August.

Chen, S., and Tong, G. (1994), "Design for Stability: Correct Use of Braces," *Steel Structures*, Journal of the Singapore Structural Steel Society, Vol. 5, No. 1, December, pp. 15–23.

Chen, W.F., and Atsuta, T. (1977), *Theory of Beam Columns, Volume II: Space Behavior and Design*, McGraw-Hill, New York, NY.

Chen, W.F., and Lui, E.M. (1987), *Structural Stability: Theory and Implementation*, Elsevier, New York, NY.

Chen, W.F., and Lui, E.M. (1991), *Stability Design of Steel Frames*, CRC Press, Boca Raton, FL.

Chen, W.F., and Toma, S. (1994), *Advanced Analysis of Steel Frames*, CRC Press, Boca Raton, FL.

Chen, W.F., and Sohal, I. (1995), *Plastic Design and Second-Order Analysis of Steel Frames*, Springer Verlag, New York, NY.

Chen, W.F., Goto, Y., and Liew, J.Y.R. (1995), *Stability Design of Semi-Rigid Frames*, John Wiley and Sons, New York, NY.

Cheng, J.J.R., and Kulak, G.L. (2000), "Gusset Plate Connection to Round HSS Tension Members," *Engineering Journal*, AISC, Vol. 37, No. 4, 4th Quarter, pp. 133–139.

Chien, E.Y.L., and Ritchie, J.K. (1984), *Composite Floor Systems*, Canadian Institute of Steel Construction, Willowdale, Ontario, Canada.

Cooney, R.C., and King, A.B. (1988), "Serviceability Criteria for Buildings," BRANZ Report SR14, Building Research Association of New Zealand, Porirua, New Zealand.

Cooper, P.B., Galambos, T.V., and Ravindra, M.K. (1978), "LRFD Criteria for Plate Girders," *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, September, pp. 1389–1407.

CSA (1994), *Limit States Design of Steel Structures*, CSA S16.1, Canadian Standards Association, Rexdale, Ontario, Canada.

CSA (2000), *Limit States Design of Steel Structures*, CSA S16.1, Canadian Standards Association, Rexdale, Ontario, Canada.

CSA (2003). *General Requirements for Rolled or Welded Structural Quality Steel/ Structural Quality Steel*, CAN/CSA-G40.20/G40.21–98(R2003), Canadian Standards Association, Mississauga, Ontario, Canada.

Darwin, D. (1990), *Steel and Composite Beams with Web Openings*, AISC Steel Design Guide Series No. 2, American Institute of Steel Construction, Chicago, IL.

Davies, G., and Packer, J.A. (1982), "Predicting the Strength of Branch Plate—RHS Connections for Punching Shear," *Canadian Journal of Civil Engineering*, Vol. 9, pp. 458–467.

Deierlein, G.G., and White, D.W. (1998), "Frame Stability," Chapter 16, *Guide to Stability Design Criteria for Metal Structures*, 5th edition, John Wiley and Sons, New York, NY.

Dekker, N.W., Kemp, A.R., and Trinchero, P. (1995), "Factors Influencing the Strength of Continuous Composite Beams in Negative Bending," *Journal of Constructional Steel Research*, Vol. 34, Nos. 2–3, pp. 161–185.

Dexter, R.J., and Melendrez, M.I. (2000), "Through-Thickness Properties of Column Flanges in Welded Moment Connections," *Journal of Structural Engineering*, ASCE, Vol. 126, No. 1, pp. 24–31.

Dexter, R.J., Hajjar, J.F., Prochnow, S.D., Graeser, M.D., Galambos, T.V., and Cotton, S.C. (2001), "Evaluation of the Design Requirements for Column Stiffeners and Doublers and the Variation in Properties of A992 Shapes," *Proceedings of the North American Steel Construction Conference*, Fort Lauderdale, FL, May 9–12, 2001, American Institute of Steel Construction, Inc., Chicago, IL, pp. 14.1–14.21.

Dexter, R.J., and Altstadt, S.A. (2004), "Strength and Ductility of Tension Flanges in Girders," *Recent Developments in Bridge Engineering*, *Proceedings of the Second New York City Bridge Conference,* October 20–21, 2003, New York, NY, Mahmoud, K.M. (ed.), A.A. Balkema/Swets & Zeitlinger, Lisse, the Netherlands, 2003, pp. 67–81.

DeWolf, J.T., and Ricker, D.T. (1990), *Column Base Plates*, Steel Design Guide Series No. 1, American Institute of Steel Construction, Inc., Chicago, IL.

Disque, R.O. (1964), "Wind Connections with Simple Framing," *Engineering Journal*, AISC, Vol. 1, No. 3, July, pp. 101–103.

Earls, C.J., and Galambos, T.V. (1997), "Design Recommendations for Equal Leg Single Angle Flexural Members," *Journal of Constructional Steel Research*, Vol. 43, Nos. 1–3, pp. 65–85.

Easterling, W.S., Gibbings, D.R., and Murray, T.M. (1993), "Strength of Shear Studs in Steel Deck on Composite Beams and Joists," *Engineering Journal*, AISC, Vol. 30, No. 2, 2nd Quarter, pp. 44–55.

Easterling, W.S., and Gonzales, L. (1993), "Shear Lag Effects in Steel Tension Members," *Engineering Journal*, AISC, Vol. 30, No. 3, 3rd Quarter, pp. 77–89.

Elgaaly, M. (1983), "Web Design under Compressive Edge Loads," *Engineering Journal*, AISC, Vol. 20, No. 4, 4th Quarter, pp. 153–171.

Elgaaly, M. and Salkar, R. (1991), "Web Crippling Under Edge Loading," *Proceedings of AISC National Steel Construction Conference*, Washington, DC.

Ellifritt, D.S., Wine, G., Sputo, T., and Samuel, S. (1992), "Flexural Strength of WT Sections," *Engineering Journal*, AISC, Vol. 29, No. 2, 2nd Quarter, pp. 67–74.

Ellingwood, B.E., MacGregor, J.G., Galambos, T.V., and Cornell, C.A. (1982), "Probability-Based Load Criteria: Load Factors and Load Combinations," *Journal of the Structural Division*, ASCE, Vol. 108, No. 5, pp. 978–997.

Eurocode 1 (1991)*, Basis of Design and Actions on Structures*, ENV 1991-2-2, Comite Europeen de Normalisation (CEN), Brussels, Belgium.

Eurocode 3 (1992), *Design of Steel Structures, Part 1: General Rules and Rules for Buildings*, ENV 1993-1-1:1992, Comite Europeen de Normalisation (CEN), Brussels, Belgium.

Eurocode 3 (2002), *Design of Steel Structures—General Rules—Part 1–8: Design of Joints: Section 7: Hollow Section Joints*, prEN1993-1-8, Final Draft, Comite Europeen de Normalisation (CEN), Brussels, Belgium.

Eurocode 4 (2003), *Design of Composite Steel and Concrete Structures,* prEN 1994-1-1, Comite Europeen de Normalisation (CEN), Brussels, Belgium.

FEMA (1995), *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, Bulletin No. 267, Federal Emergency Management Agency, Washington, DC.

FEMA (1997), "Seismic Performance of Bolted and Riveted Connections" *Background Reports; Metallurgy, Fracture Mechanics, Welding, Moment Connections and Frame Systems Behavior*, Bulletin No. 288, Federal Emergency Management Agency, Washington, DC.

Felton, L.P., and Dobbs, M.W. (1967), "Optimum Design of Tubes for Bending and Torsion," *Journal of the Structural Division*, ASCE, Vol. 93, No. ST4, pp. 185–200.

Fielding, D.J. and Huang, J.S. (1971), "Shear in Steel Beam-to-Column Connections," *The Welding Journal*, AWS, Vol. 50, No. 7, Research Supplement, pp. 313–326.

Fielding, D.J., and Chen, W.F. (1973), "Steel Frame Analysis and Connection Shear Deformation," *Journal of the Structural Division*, ASCE, Vol. 99, No. ST1, January, pp. 1–18.

Fisher, J.M. and West, M.A. (1997), *Erection Bracing of Low-Rise Structural Steel Buildings*, Steel Design Guide Series No. 10, American Institute of Steel Construction, Inc., Chicago, IL.

Fisher, J.W. (1970), "Design of Composite Beams with Formed Metal Deck," *Engineering Journal*, AISC, Vol. 3, No. 7, pp. 88–96.

Fisher, J.W., Albrecht, P.A., Yen, B.T., Klingerman, D.J., and McNamee, B.M. (1974), "Fatigue Strength of Steel Beams with Welded Stiffeners and Attachments," National Cooperative Highway Research Program, Report 147, Washington, DC.

Fisher, J.W., Frank, K.H., Hirt, M.A., and McNamee, B.M. (1970), "Effect of Weldments on the Fatigue Strength of Beams," National Cooperative Highway Research Program, Report 102, Washington, DC.

Fisher, J.W., Galambos, T.V., Kulak, G.L., and Ravindra, M.K. (1978), "Load and Resistance Factor Design Criteria for Connectors," *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, September, pp. 1,427–1,441.

Frank, K.H., and Fisher, J.W. (1979), "Fatigue Strength of Fillet Welded Cruciform Joints," *Journal of the Structural Division*, ASCE, Vol. 105, No. ST9, September.

Frank, K.H., and Yura, J.A. (1981), "An Experimental Study of Bolted Shear Connections," FHWA/RD-81/148, Federal Highway Administration, Washington, DC, December.

Frater, G.S., and Packer, J.A. (1992), "Weldment Design for RHS Truss Connections. I: Applications," *Journal of Structural Engineering*, ASCE, Vol. 118, No. 10, pp. 2784–2803.

Frater, G.S., and Packer, J.A. (1992a), "Weldment Design for RHS Truss Connections. II: Experimentation," *Journal of Structural Engineering*, ASCE, Vol. 118, No. 10, pp. 2804–2820.

Freeman, F.R. (1930), "The Strength of Arc-Welded Joints," *Proceedings of the Institute of Civil Engineers*, Vol. 231, London, England.

Freeman, S. (1977), "Racking Tests of High Rise Building Partitions," *Journal of the Structural Division*, ASCE, Vol. 103, No. 8, pp. 1673–1685.

Galambos, T.V. (1968), *Structural Members and Frames*, Prentice-Hall, Englewood Cliffs, NJ.

Galambos. T.V. (1978), "Proposed Criteria for Load and Resistance Factor Design of Steel Building Structures," AISI Bulletin No. 27, American Iron and Steel Institute, Washington, DC, January.

Galambos, T.V. (1983), "Reliability of Axially Loaded Columns," *Engineering Structures*, Vol. 5, No. 1, pp. 73–78.

Galambos, T.V., Ellingwood, B., MacGregor, J.G., and Cornell, C.A. (1982), "Probability-Based Load Criteria: Assessment of Current Design Practice," *Journal of the Structural Division*, ASCE, Vol. 108, No. ST5, May, pp. 959–977.

Galambos, T.V., and Ellingwood, B. (1986), "Serviceability Limit States: Deflections," *Journal of the Structural Division*, ASCE, Vol. 112, No. 1, pp. 67–84.

Galambos, T.V. (1991), "Design of Axially Loaded Compressed Angles," *Proceedings of the Annual Technical Session and Meeting*, Chicago, IL, April 15–17, 1991, Structural Stability Research Council, Bethlehem, PA, pp. 353–367.

Galambos, T.V., Lin, F.J., and Johnston, B.G. (1996), *Basic Steel Design with LRFD*, Prentice Hall, Englewood Cliffs, NJ.

Galambos, T.V. (ed.) (1998), *Guide to Stability Design Criteria for Metal Structures*, Structural Stability Research Council, 5th edition, John Wiley and Sons, New York, NY.

Galambos, T.V. (2001), "Strength of Singly Symmetric I-Shaped Beam-Columns," *Engineering Journal*, AISC, Vol. 38, No. 2, 2nd Quarter, pp. 65–77.

Gaylord, E.H. Jr., Gaylord, C.N., and Stallmeyer, J.E. (1997), *Structural Engineering Handbook*, 4th ed., McGraw Hill, New York, NY.

Geschwindner, L. (2002), "A Practical Approach to Frame Analysis, Stability and Leaning Columns," *Engineering Journal,* AISC, Vol. 39, No. 4, 4th Quarter, pp. 167–181.

Geschwindner, L.F., and Disque, R.O. (2005), "Flexible Moment Connections for Unbraced Frames Subject to Lateral Forces—A Return to Simplicity," *Engineering Journal,* AISC, Vol. 42, No. 2, 2nd Quarter.

Gibson, G.T., and Wake, B.T. (1942), "An Investigation of Welded Connections for Angle Tension Members," *The Welding Journal*, AWS, January, p. 44.

Giddings, T.W., and Wardenier, J. (1986), "The Strength and Behaviour of Statically Loaded Welded Connections in Structural Hollow Sections," CIDECT Monograph No. 6, Sections 1–10, British Steel Corporation Tubes Division, Corby, United Kingdom.

Gjelsvik, A. (1981), *The Theory of Thin-Walled Bars*, John Wiley and Sons, New York, NY.

Goble, G.G. (1968), "Shear Strength of Thin Flange Composite Specimens," *Engineering Journal*, AISC, Vol. 5, No. 2, 2nd Quarter, pp. 62–65.

Goverdhan, A.V. (1983), "A Collection of Experimental Moment Rotation Curves: Evaluation of Predicting Equations for Semi-Rigid Connections," M.S. Thesis, Vanderbilt University, Nashville, TN.

Graham, J.D., Sherbourne, A.N., and Khabbaz, R.N. (1959), "Welded Interior Beam-to-Column Connections," American Institute of Steel Construction, Inc., Chicago, IL.

Graham, J.D., Sherbourne, A.N., Khabbaz, R.N., and Jensen, C.D. (1960), "Welded Interior Beam-to-Column Connections," Welding Research Council, Bulletin No. 63, pp. 1–28.

Graham, R.R. (1965), "Manufacture and Use of Structural Tubing," *Journal of Metals*, TMS, Warrendale, PA, September.

Grant, J.A., Fisher, J.W., and Slutter, R.G. (1977), "Composite Beams with Formed Steel Deck," *Engineering Journal*, AISC, Vol. 14, No. 1, 1st Quarter, pp. 24–43.

Griffis, L.G. (1992), *Load and Resistance Factor Design of W-Shapes Encased in Concrete*, Steel Design Guide Series No. 6, American Institute of Steel Construction, Inc., Chicago, IL.

Griffis, L.G. (1993), "Serviceability Limit States Under Wind Load," *Engineering Journal*, AISC, Vol. 30, No. 1, 1st Quarter, pp. 1–16.

Hajjar, J.F. (2000), "Concrete-Filled Steel Tube Columns under Earthquake Loads," *Progress in Structural Engineering and Materials*, Vol. 2, No. 1, pp. 72–82.

Hajjar, J.F., Dexter, R.J., Ojard, S.D., Ye, Y., and Cotton, S.C. (2003), "Continuity Plate Detailing for Steel Moment-Resisting Connections," *Engineering Journal*, AISC, 4th Quarter, pp. 81–97.

Hansell, W.C., Galambos, T.V., Ravindra, M.K., and Viest, I.M. (1978), "Composite Beam Criteria in LRFD," *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, September, pp. 1,409–1,426.

Hansen, R.J., Reed, J.W., and Vanmarcke, E.H. (1973), "Human Response to Wind-Induced Motion of Buildings," *Journal of the Structural Division*, ASCE, Vol. 99, No. ST7, pp. 1589–1606.

Heinzerling, J.E. (1987), "Structural Design of Steel Joist Roofs to Resist Ponding Loads," Technical Digest No. 3, Steel Joist Institute, Myrtle Beach, SC.

Higgins, T.R., and Preece, F.R. (1968), "AWS-AISC Fillet Weld Study, Longitudinal and Transverse Shear Tests," Internal Report, Testing Engineers, Inc., Oakland, CA, May 31.

Horne, M.R., and Morris, L.J. (1982), *Plastic Design of Low-Rise Frames*, MIT Press, Cambridge, MA.

Hsieh, S.H., and Deierlein, G.G. (1991), "Nonlinear Analysis of Three-Dimensional Steel Frames with Semi-Rigid Connections," *Computers and Structures*, Vol. 41, No. 5, pp. 995–1009.

ICC (2003), *International Building Code*, International Code Council, Falls Church, VA.

IIW (1989), "Design Recommendations for Hollow Section Joints—Predominantly Statically Loaded," 2nd edition, IIW Document XV-701-89, IIW Annual Assembly, Subcommission XV-E, International Institute of Welding, Helsinki, Finland.

Irwin, A.W. (1986), "Motion in Tall Buildings," *Second Century of the Skyscraper*, Beedle, L. S. (ed.), Van Nostrand Reinhold Co., New York, NY.

Islam, M.S., Ellingwood, B., and Corotis, R.B. (1990), "Dynamic Response of Tall Buildings to Stochastic Wind Load," *Journal of Structural Engineering*, ASCE, Vol. 116, No. 11, November, pp. 2,982–3,002.

ISO (1977), "Bases for the Design of Structures—Deformations of Buildings at the Serviceability Limit States," ISO 4356, International Standards Organization, Geneva, Switzerland.

Iwankiw, N. (1984), "Note on Beam-Column Moment Amplification Factor," *Engineering Journal*, AISC, Vol. 21, No. 1, 1st Quarter, pp. 21–23.

JCRC (1971), *Handbook of Structural Stability*, Japanese Column Research Council, English translation, pp. 3–22.

Jayas, B.S., and Hosain, M.U. (1988), "Composite Beams with Perpendicular Ribbed Metal Deck," *Composite Construction in Steel and Concrete II*, Buckner, C. D., and Viest, I.M. (eds.), American Society of Civil Engineers, New York, NY, pp. 511–526.

Jayas, B.S., and Hosain, M.U. (1988a), "Behaviour of Headed Studs in Composite Beams: Push-Out Tests," *Canadian Journal of Civil Engineering*, Vol. 15, pp. 240–253.

Johnson, D.L. (1985), "An Investigation into the Interaction of Flanges and Webs in Wide-Flange Shapes," *Proceedings of the Annual Technical Session and Meeting*, Cleveland, OH, April 16–17, 1985, Structural Stability Research Council, Bethlehem, PA, pp. 397–405.

Johnson, D.L. (1996), "Final Report on Tee Stub Tests," Butler Research Report, Grandview, MO, May.

Johnson, R.P., and Yuan, H. (1998), "Existing Rules and New Tests for Stud Shear Connectors in Troughs of Profiled Sheeting," *Proceedings of the Institution of Civil Engineers: Structures and Buildings*, Vol. 128, No. 3, pp. 244–251.

Johnston, B.G. (1939), "Pin-Connected Plate Links," *Transactions of the ASCE*, Vol. 104, pp. 314–339.

Johnston, B.G., and Deits, G.R. (1942) "Tests of Miscellaneous Welded Building Connections," *The Welding Journal*, AWS, November, p. 5.

Johnston, B.G., and Green, L.F. (1940), "Flexible Welded Angle Connections," *The Welding Journal*, AWS, October.

Johnston, B.G. (ed.) (1976), *Guide to Stability Design for Metal Structures*, 3rd edition, Structural Stability Research Council, John Wiley and Sons, New York, NY.

Kaczinski, M.R., Schneider, C.R., Dexter, R.J., and Lu, L.-W. (1994), "Local Web Crippling of Unstiffened Multi-Cell Box Sections," *Proceedings of the ASCE Structures Congress '94*, Atlanta, GA, Vol. 1, American Society of Civil Engineers, New York, NY, pp. 343–348.

Kaufmann, E.J., Metrovich, B., Pense, A.W., and Fisher, J.W. (2001), "Effect of Manufacturing Process on k-Area Properties and Service Performance," *Proceedings of the North American Steel Construction Conference*, Fort Lauderdale, FL, May

9–12, 2001, American Institute of Steel Construction, Inc., Chicago, IL, pp. 17.1–17.24.

Kanchanalai, T., and Lu, L.W. (1979), "Analysis and Design of Framed Columns under Minor Axis Bending," *Engineering Journal*, AISC, Vol. 16, No. 2, 2nd Quarter, pp. 29–41.

Kavanagh, T.C. (1962), "Effective Length of Framed Columns," *Transactions of the American Society of Civil Engineers*, Vol. 127, pp. 81–101.

Keating, P.B., and Fisher, J.W. (1986), "Evaluation of Fatigue Tests and Design Criteria on Welded Details," NCHRP Report No. 286, Transportation Research Board, Washington DC, September.

Kim, H.J., and Yura, J.A. (1996), "The Effect of End Distance on the Bearing Strength of Bolted Connections," PMFSEL Report No. 96–1, University of Texas, Austin, TX.

Kirby, P.A., and Nethercot, D.A. (1979), *Design for Structural Stability*, John Wiley and Sons, Inc., New York, NY.

Kishi, N., and Chen, W.F. (1986), Data Base of Steel Beam-to-Column Connections, Vol. 1 and 2, Structural Engineering Report No. CE-STR-86-26, School of Civil Engineering, Purdue University, West Lafayette, IN.

Kitipornchai, S., and Trahair, N.S. (1980), "Buckling Properties of Monosymmetric I-Beams," *Journal of the Structural Division*, ASCE, Vol. 109, No. ST5, May, pp. 941–957.

Kitipornchai, S., and Traves, W.H. (1989), "Welded-Tee End Connections for Circular Hollow Tubes," *Journal of Structural Engineering*, ASCE, Vol. 115, No.12, pp. 3,155–3,170.

Kloppel, K., and Seeger, T. (1964), "Dauerversuche Mit Einsohnittigen Hv-Verbindurgen Aus ST37," *Der Stahlbau*, Vol. 33, No. 8, August, pp. 225–245 and Vol. 33, No. 11, November, pp. 335–346.

Kosteski, N., and Packer, J.A. (2003), "Longitudinal Plate and Through Plate-to-HSS Welded Connections," *Journal of Structural Engineering*, ASCE, Vol. 129, No. 4, pp. 478–486.

Kuchenbecker, G.H., White, D.W., and Surovek-Maleck, A.E. (2004), "Simplified Design of Building Frames Using First-Order Analysis and $K = 1$," *Proceedings of the Annual Technical Session and Meeting*, Long Beach, CA, March 24–27, 2004, Structural Stability Research Council, Rolla, MO, pp. 119–138.

Kulak, G.L., Fisher, J.W., and Struik, J.H.A. (1987), *Guide to Design Criteria for Bolted and Riveted Joints*, 2nd edition, John Wiley and Sons, New York, NY.

Kulak, G.L. and Grondin, G.Y. (2001), "Strength of Joints that Combine Bolts and Welds," *Engineering Journal*, AISC, Vol. 38, No. 2, 2nd Quarter, pp. 89–98.

Kulak, G.L., and Grondin, G.Y. (2001a), "AISC LRFD Rules for Block Shear—A Review," *Engineering Journal*, AISC, Vol. 38, No. 4, 4th Quarter, pp. 199–203.

Kulak, G. (2002), *High Strength Bolts: A Primer for Structural Engineers*, Steel Design Guide 17, American Institute of Steel Construction, Inc., Chicago, IL.

Kurobane, Y., Packer, J.A., Wardenier, J., and Yeomans, N.F. (2004), "Design Guide for Structural Hollow Section Column Connections," CIDECT Design Guide No. 9, CIDECT (ed.) and Verlag TÜV Rheinland, Köln, Germany.

Lawson, R.M. (1992), "Shear Connection in Composite Beams," *Composite Construction in Steel and Concrete II*, Easterling, W. S., and Roddis, W. M. K. (eds.), American Society of Civil Engineers, New York, NY.

Lee, D., Cotton, S., Dexter, R.J., Hajjar, J.F., Ye, Y., and Ojard, S.D. (2002), "Column Stiffener Detailing and Panel Zone Behavior of Steel Moment Frame Connections," Report No. ST-01-3.2, Department of Civil Engineering, University of Minnesota, Minneapolis, MN.

Leigh, J.M. and Lay, M.G. (1978), "Laterally Unsupported Angles with Equal and Unequal Legs," Report MRL 22/2, July, Melbourne Research Laboratories, Clayton, Victoria, Australia.

Leigh, J.M., and Lay, M.G. (1984), "The Design of Laterally Unsupported Angles," *Steel Design Current Practice*, Section 2, Bending Members, American Institute of Steel Construction, Inc., Chicago, IL, January.

LeMessurier, W.J. (1976), "A Practical Method of Second Order Analysis, Part 1—Pin-Jointed Frames," *Engineering Journal*, AISC, Vol. 13, No. 4, 4th Quarter, pp. 89–96.

LeMessurier, W.J. (1977), "A Practical Method of Second Order Analysis, Part 2—Rigid Frames," *Engineering Journal*, AISC, Vol. 14, No. 2, 2nd Quarter, pp. 49–67.

LeMessurier, W.J. (1995), "Simplified K Factors for Stiffness Controlled Designs," *Restructuring: America and Beyond*, *Proceedings of ASCE Structures Congress XIII*, Boston, MA, April 2–5, 1995, American Society of Civil Engineers, New York, NY, pp. 1797–1812.

Leon, R.T. (1994), "Composite Semi-Rigid Construction," *Engineering Journal*, AISC, Vol. 31. No. 2, 2nd Quarter, pp. 57–67.

Leon, R.T., Hoffman, J., and Staeger, T. (1996), *Design of Partially-Restrained Composite Connections*, Steel Design Guide Series No. 8, American Institute of Steel Construction, Inc., Chicago, IL.

Leon, R.T. (2001), "A Critical Review of Current LRFD Provisions for Composite Columns," *Proceedings of the Annual Technical Session and Meeting*, Structural Stability Research Council, Fort Lauderdale, FL, May 9–12, 2001, Structural Stability Research Council, Gainesville, FL, pp. 189–208.

Leon, R.T. and Aho, M.F. (2002), "Towards New Design Provisions for Composite Columns," *Composite Construction in Steel and Concrete IV*, Hajjar, J.F., Hosain, M., Easterling, W.S., and Shahrooz, B.M. (eds.), American Society of Civil Engineers, Reston, VA.

Leon, R.T. and Easterling, W.S. (eds.) (2002), *Connections in Steel Structures IV—Behavior, Strength and Design*, American Institute of Steel Construction, Inc., Chicago, IL.

Lewis, B.E., and Zwerneman, F.J. (1996), "Edge Distance, Spacing, and Bearing in Bolted Connections," Research Report, Department of Civil and Environmental Engineering, Oklahoma State University, Stillwater, OK, July.

Lesik, D.F., and Kennedy, D.J.L. (1990), "Ultimate Strength of Fillet Welded Connections Loaded in Plane," *Canadian Journal of Civil Engineering*, Vol. 17, No. 1, pp. 55–67.

Lorenz, R.F., Kato, B., and Chen, W.F. (eds.) (1993), *Semi-Rigid Connections in Steel Frames*, Council for Tall Buildings and Urban Habitat, Bethlehem, PA.

Lui, Z., and Goel, S.C. (1987), "Investigation of Concrete-Filled Steel Tubes Under Cyclic Bending and Buckling," UMCE Report 87-3, Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, MI.

Lutz, L.A., and Fisher, J.M. (1985), "A Unified Approach for Stability Bracing Requirements, *Engineering Journal*, AISC, Vol. 22, No. 4, 4th Quarter, pp. 163–167.

Lutz, L.A. (1992), "Critical Slenderness of Compression Members with Effective Lengths about Non-Principal Axes," *Proceedings of the Annual Technical Session and Meeting*, April 6–7, 1992, Pittsburgh, PA, Structural Stability Research Council, Bethlehem, PA.

Lyse, I., and Schreiner (1935), "An Investigation of Welded Seat Angle Connections," *The Welding Journal*, AWS, February, p. 1.

Lyse, I., and Gibson, G.J. (1937), "Effect of Welded Top Angles on Beam-Column Connections," *The Welding Journal*, AWS, October.

Madugula, M.K.S., and Kennedy, J.B. (1985), *Single and Compound Angle Members*, Elsevier Applied Science, New York, NY.

Maleck, A.E., and White, D.W. (2003), "Direct Analysis Approach for the Assessment of Frame Stability: Verification Studies," *Proceedings of the Annual Technical Session and Meeting*, Baltimore, MD, April 2–5, 2003, Structural Stability Research Council, Bethlehem, PA, pp. 423–442.

Marino, F.J. (1966), "Ponding of Two-Way Roof Systems," *Engineering Journal*, AISC, Vol. 3, No. 3, 3rd Quarter, pp. 93–100.

Marshall, P.W. (1992), *Design of Welded Tubular Connections: Basis and Use of AWS Code Provisions*, Elsevier, Amsterdam, The Netherlands.

McGuire, W. (1992), "Computer-Aided Analysis," *Constructional Steel Design: An International Guide*, Dowling, P.J., Harding, J.E., and Bjorhovde, R. (eds.), Elsevier Applied Science, New York, NY, pp. 915–932.

McGuire, W., Gallagher, R.H., and Ziemian, R.D. (2000), *Matrix Structural Analysis*, 2nd edition, John Wiley and Sons, New York, NY.

Mottram, J.T., and Johnson, R.P. (1990), "Push Tests on Studs Welded Through Profiled Steel Sheeting," *The Structural Engineer*, Vol. 68, No. 10, pp. 187–193.

Munse, W.H., and Chesson, Jr., E., (1963), "Riveted and Bolted Joints: Net Section Design," *Journal of the Structural Division*, ASCE, Vol. 89, No. ST1, February, pp. 49–106.

Murray, T.M., Allen, D.E., and Ungar, E.E. (1997), *Floor Vibrations Due to Human Activity*, Steel Design Guide Series No. 11, American Institute for Steel Construction, Chicago, IL.

Murray, T.M., Kline, D.P., and Rojani, K.B. (1992), "Use of Snug-Tightened Bolts in End-Plate Connections," in *Connections in Steel Structures II*, R. Bjorhovde, A. Colson, G. Haaijer, and J.W.B. Stark, (eds.), American Institute of Steel Construction, Inc., Chicago, IL.

Murray, T.M., and Sumner, E.A. (2004), *End-Plate Moment Connections—Wind and Seismic Applications*, Steel Design Guide 4, 2nd edition, American Institute of Steel Construction, Chicago, IL.

NRC (1974), "Expansion Joints in Buildings," Technical Report No. 65, Standing Committee on Structural Engineering of the Federal Construction Council, Building Research Advisory Board, Division of Engineering, National Research Council, National Academy of Sciences, Washington, DC.

NRCC (1990), *National Building Code of Canada*, National Research Council of Canada, Ottawa, Ontario, Canada.

NEHRP (1997), NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, Federal Emergency Management Agency Report, FEMA 302, Washington, DC.

Nethercot, D.A. (1985), "Steel Beam to Column Connections—A Review of Test Data and Their Applicability to the Evaluation of the Joint Behaviour of the Performance of Steel Frames," CIRIA, London, England.

Ollgaard, J.G., Slutter, R.G., and Fisher, J.W. (1971), "Shear Strength of Stud Shear Connections in Lightweight and Normal Weight Concrete," *Engineering Journal*, AISC, Vol. 8, No. 2, 2nd Quarter, pp. 55–64.

OSHA (2001), *Safety and Health Regulations for Construction*, Standards—29 CFR 1926 Subpart R—Steel Erection, Occupational Safety and Health Administration, Washington, DC.

Packer, J.A., Birkemoe, P.C., and Tucker, W.J. (1984), "Canadian Implementation of CIDECT Monograph No. 6," CIDECT Report No. 5AJ-84/9-E, University of Toronto, Toronto, Canada.

Packer, J.A., Wardenier, J., Kurobane, Y., Dutta, D., and Yeomans, N. (1992), "Design Guide for Rectangular Hollow Section (RHS) Joints under Predominantly Static Loading," CIDECT Design Guide No. 3, CIDECT (ed.) and Verlag TÜV Rheinland, Köln, Germany.

Packer, J.A. (1995), "Design of Fillet Welds in Rectangular Hollow Section T, Y and X Connections using New North American Code Provisions," *Connections in Steel Structures III: Behaviour, Strength and Design*, Bjorhovde, R., Colson, A., and Zandonini, R. (eds.), Pergamon Press, Amsterdam, The Netherlands, pp. 463–472.

Packer, J.A., and Cassidy, C.E. (1995), "Effective Weld Length for HSS T, Y and X Connections," *Journal of Structural Engineering*, ASCE, Vol. 121, No. 10, pp. 1,402–1,408.

Packer, J.A., and Henderson, J.E. (1997), *Hollow Structural Section Connections and Trusses—A Design Guide*, 2nd edition, Canadian Institute of Steel Construction, Toronto, Canada.

Packer, J.A. (2004), "Reliability of Welded Tubular K-Connection Resistance Expressions," International Institute of Welding (IIW) Document XV-E-04-291, University of Toronto, Toronto, Canada.

Popov, E.P. (1980), "An Update on Eccentric Seismic Bracing," *Engineering Journal*, AISC, Vol. 17, No. 3, 3rd Quarter, pp. 70–71.

Popov, E.P., and Stephen, R.M. (1977), "Capacity of Columns with Splice Imperfections," *Engineering Journal*, AISC, Vol. 14, No. 1, 1st Quarter, pp. 16–23.

Preece, F.R. (1968), "AWS-AISC Fillet Weld Study—Longitudinal and Transverse Shear Tests," Testing Engineers, Inc., Los Angeles, CA, May.

Prochnow, S.D., Ye, Y., Dexter, R.J., Hajjar, J.F., and Cotton, S.C. (2000), "Local Flange Bending and Local Web Yielding Limit States in Steel Moment Resisting Connections," *Connections in Steel Structures IV—Behavior, Strength and Design*, Leon, R. T. (ed.), American Institute of Steel Construction, Inc., Chicago, IL, pp. 318–328.

Ravindra, M.K., and Galambos, T.V. (1978), "Load and Resistance Factor Design for Steel," *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, September, pp. 1337–1353.

RCSC (2004), *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Research Council on Structural Connections, American Institute of Steel Construction, Inc., Chicago, IL.

Ricker, D.T. (1989), "Cambering Steel Beams," *Engineering Journal*, AISC, Vol. 26, No. 4, 4th Quarter, 136–142.

Ricles, J.M., and Yura, J.A. (1983), "Strength of Double-Row Bolted Web Connections," *Journal of the Structural Division*, ASCE, Vol. 109, No. ST1, January, pp. 126–142.

Roberts, T.M. (1981), "Slender Plate Girders Subjected to Edge Loading," *Proceedings of the Institution of Civil Engineers*, Part 2, No. 71, September.

Robinson, H. (1967), "Tests of Composite Beams with Cellular Deck," *Journal of the Structural Division*, ASCE, Vol. 93, No. ST4, pp. 139–163.

Roddenberry, M.R., Easterling, W.S., and Murray, T.M. (2002) "Behavior and Strength of Welded Stud Shear Connectors," Report No. CE/VPI–02/04, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Roddenberry, M.R., Lyons, J.C., Easterling, W.S., and Murray, T.M. (2002a), "Performance and Strength of Welded Shear Studs," *Composite Construction in Steel and Concrete IV*, Hajjar, J.F., Hosain, M, Easterling, W.S., and Shahrooz, B.M. (eds.), American Society of Civil Engineers, Reston, VA, pp. 458–469.

Roeder, C.W., Cameron, B., and Brown, C.B. (1999), "Composite Action in Concrete Filled Tubes," *Journal of Structural Engineering*, ASCE, Vol. 125, No. 5, May, pp. 477–484.

Roik, K., and Bergmann, R. (1992), "Composite Column," *Constructional Steel Design: An International Guide*, Dowling, P.J., Harding, J.E., and Bjorhovde, R. (eds.), Elsevier Applied Science, London, United Kingdom.

Rolloos, A. (1969), "The Effective Weld Length of Beam to Column Connections without Stiffening Plates," Stevin Report 6-69-7-HL, Delft University of Technology, Delft, The Netherlands.

Ruddy, J. (1986), "Ponding of Concrete Deck Floors," *Engineering Journal*, AISC, Vol. 23, No. 3, 3rd Quarter, pp. 107–115.

Salmon, C.G., and Johnson, J.E. (1996), *Steel Structures, Design and Behavior*, 4th edition, HarperCollins College Publishers, New York, NY.

Salvadori, M. (1956), "Lateral Buckling of Eccentrically Loaded I-Columns," *Transactions of the ASCE*, Vol. 122, No. 1.

Schilling, C.G. (1965), Buckling Strength of Circular Tubes, *Journal of the Structural Division*, ASCE, Vol. 91, No. ST5, paper 4520.

Schuster, J.W. (1997), *Structural Steel Fabrication Practices*, McGraw-Hill, New York, NY.

SDI (1999), *Standard Practice Details*, Steel Deck Institute, Fox River Grove, IL.

Seaburg, P.A., and Carter, C.J. (1997), *Torsional Analysis of Structural Steel Members*, Steel Design Guide Series No. 9, American Institute of Steel Construction, Inc., Chicago, IL.

SFSA (1995), *Steel Castings Handbook*, Steel Founders Society of America, Crystal Lake, IL.

Shanmugam, N.E., and Lakshmi, B. (2001), "State of the Art Report on Steel-Concrete Composite Columns," *Journal of Constructional Steel Research*, Vol. 57, No. 10, October, pp. 1,041–1,080.

Sherbourne, A.N., and Jensen, C.D. (1957), "Direct Welded Beam Column Connections," Report. No. 233.12, Fritz Laboratory, Lehigh University, Bethlehem, PA.

Sherman, D.R. (1976), "Tentative Criteria for Structural Applications of Steel Tubing and Pipe," American Iron and Steel Institute, Washington, DC, August.

Sherman, D.R., and Tanavde, A.S. (1984), "Comparative Study of Flexural Capacity of Pipes," Internal Report, Department of Civil Engineering, University of Wisconsin–Milwaukee, WI, March.

Sherman, D.R. (1985), "Bending Equations for Circular Tubes," *Proceedings of the Annual Technical Session and Meeting*, Cleveland, OH, April 16–17, 1985, Structural Stability Research Council, Bethlehem, PA, pp. 251–262.

Sherman, D.R., and Ales, J.M. (1991), "The Design of Shear Tabs with Tubular Columns," *Proceedings of the National Steel Construction Conference*, Washington, DC, American Institute of Steel Construction, Chicago, IL, pp. 1.2–1.22.

Sherman, D.R. (1992), "Tubular Members," *Constructional Steel Design—An International Guide,* Dowling, P.J., Harding, J.H., and Bjorhovde, R. (eds.), Elsevier Applied Science, London, United Kingdom, pp. 91–104.

Sherman, D.R. (1995), "Stability Related Deterioration of Structures," *Proceedings of the Annual Technical Session and Meeting*, Kansas City, MO, March 27–28, 1995, Structural Stability Research Council, Bethlehem, PA.

Sherman, D.R. (1995a), "Simple Framing Connections to HSS Columns," *Proceedings of the National Steel Construction Conference*, San Antonio, Texas, American Institute of Steel Construction, Chicago, IL, pp. 30.1–30.16.

Sherman, D.R. (1996), "Designing with Structural Tubing," *Engineering Journal*, AISC, Vol. 33, No. 3, 3rd Quarter, pp. 101–109.

Slutter, R.G., and Driscoll, G.C. (1965), "Flexural Strength of Steel-Concrete Composite Beams," *Journal of the Structural Division*, ASCE, Vol. 91, No. ST2, April, pp. 71–99.

Sourouchnikoff, B. (1950), "Wind Stresses in Semi–Rigid Connections of Steel Framework," *Transactions of the ASCE*, Vol. 115, pp. 382–402.

SSPC (2000), *Systems and Specifications SSPC Painting Manual*, Vol. 2, 8th edition, Systems and Specifications, The Society of Protective Coatings, Pittsburgh, PA.

SSRC Task Group 20 (1979), "A Specification for the Design of Steel-Concrete Composite Columns," *Engineering Journal*, AISC, Vol. 16, No. 4, 4th Quarter, pp. 101–115.

STI (1996), *Principal Producers and Capabilities*, Steel Tube Institute, Mentor, OH.

Summers, P.A., and Yura, J.A. (1982), "The Behavior of Beams Subjected to Concentrated Loads," Phil M. Ferguson Structural Engineering Laboratory Report No. 82–5, University of Texas, Austin, TX, August.

Surovek-Maleck, A., White, D.W., and Leon, R.T. (2004), "Direct Analysis and Design of Partially-Restrained Steel Framing Systems," *Journal of Structural Engineering*, ASCE.

Taylor, A.C., and Ojalvo, M. (1966), "Torsional Restraint of Lateral Buckling," *Journal of the Structural Division*, ASCE, Vol. 92, No. ST2, pp. 115–129.

Tide, R.H.R. (1985), "Reasonable Column Design Equations," *Proceedings of the Annual Technical Session and Meeting*, Cleveland, OH, April 16–17, 1985, Structural Stability Research Council, Bethlehem, PA.

Tide, R.H.R. (1999), "Evaluation of Steel Properties and Cracking in the 'k'-area of W Shapes," *Engineering Structures*, Vol. 22, pp. 128–124.

Tide, R.H.R. (2001), "A Technical Note: Derivation of the LRFD Column Design Equations," *Engineering Journal*, AISC, Vol. 38, No. 3, 3rd Quarter, pp. 137–139.

Timoshenko, S.P. (1956), *Strength of Materials*, Vol. II, 3rd edition, D. Van Nostrand, New York, NY.

Timoshenko, S.P., and Gere, J.M. (1961), *Theory of Elastic Stability*, McGraw-Hill Book Company, New York, NY.

Troup, E.W. (1999), "Effective Contract and Shop Drawings for Structural Steel," *Proceedings of the AISC National Steel Construction Conference*, Toronto, Ontario, May 19–21, 1999, American Institute of Steel Construction, Inc., Chicago, IL pp. 37-1–37-15.

Van der Sanden, P.G.F.J. (1995), "The Behaviour of a Headed Stud Connection in a 'New' Push Test including a Ribbed Slab. Tests: Main Report," BKO Report No. 95-16, Eindhoven University of Technology, Eindhoven, The Netherlands, March.

Vickery, B.J., Isyumov, N., Davenport, A.G. (1983), "The Role of Damping, Mass and Stiffness in the Reduction of Wind Effects on Structures," *Journal of Wind Engineering and Industrial Aerodynamics*, Vol. 11, Nos. 1–3, pp. 285–294.

Viest, I.M., Siess, C.P., Appleton, J.H., and Newmark, N. (1952), "Full-Scale Tests of Channel Shear Connectors and Composite T-Beams," Bulletin Series No. 405, Vol. 50, No. 29, University of Illinois Engineering Experiment Station, University of Illinois, Urbana, IL.

Viest, I.M. (1956), "Investigation of Stud Shear Connectors for Composite Concrete and Steel T-Beams," *Journal of the ACI*, Vol. 27, American Concrete Institute, Detroit, MI, April.

Viest, I.M., Colaco, J.P., Furlong, R.W., Griffis, L.G., Leon, R.T., and Wyllie, L.A., Jr. (1997), *Composite Construction: Design for Buildings*, McGraw-Hill, New York, NY.

von Kármán, T., Sechler, E.E., and Donnell, L.H. (1932), "The Strength of Thin Plates in Compression," *Transactions of the ASME*, Vol. 54.

Wardenier, J., Davies, G., and Stolle, P. (1981), "The Effective Width of Branch Plate to RHS Chord Connections in Cross Joints," Stevin Report 6-81-6, Delft University of Technology, Delft, The Netherlands.

Wardenier, J., Kurobane, Y., Packer, J.A., Dutta, D., and Yeomans, N. (1991), *Design Guide for Circular Hollow Section (CHS) Joints under Predominantly Static Loading*, CIDECT Design Guide No. 1, CIDECT (ed.) and Verlag TÜV Rheinland, Köln, Germany.

West, M.A., Fisher, J.M., and Griffis, L.A. (2003), *Serviceability Design Considerations for Steel Buildings*, Steel Design Guide No. 3, 2nd edition, American Institute of Steel Construction, Inc., Chicago, IL.

White, D.W., and Chen, W.F. (ed.) (1993), *Plastic Hinge Based Methods for Advanced Analysis and Design of Steel Frames: An Assessment of State-of-the-Art*, Structural Stability Research Council, Bethlehem, PA.

White, D.W., and Hajjar, J.F. (1997), "Design of Steel Frames without Consideration of Effective Length," *Engineering Structures*, Vol. 19, No. 10, pp. 797–810.

White, D.W., and Hajjar, J.F. (1997a), "Buckling Models and Stability Design of Steel Frames: a Unified Approach," *Journal of Constructional Steel Research*, Vol. 42, No. 3, pp. 171–207.

White, D.W. (2003), "Improved Flexural Design Provisions for I-Shaped Members and Channels," Structural Engineering, Mechanics and Materials Report No. 23, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, GA.

White, D.W. and Jung, S.K (2003). "Simplified Lateral-Torsional Buckling Equations for Singly-Symmetric I-Section Members," Structural Engineering, Mechanics and Materials Report No. 24b, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, GA.

White, D.W. (2004), "Unified Flexural Resistance Equations for Stability Design of Steel I-Section Members Overview," Structural Engineering, Mechanics and Materials Report No. 24a, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, GA.

Wilkinson, T., and Hancock, G.J. (1998), "Tests to Examine Compact Web Slenderness of Cold-Formed RHS," *Journal of Structural Engineering*, ASCE, Vol. 124, No. 10, October, pp. 1166–1174.

Wilkinson, T., and Hancock, G.J. (2002), "Predicting the Rotation Capacity of Cold-Formed RHS Beams Using Finite Element Analysis," *Journal of Constructional Steel Research*, Vol. 58, No. 11, November, pp. 1,455–1,471.

Wilson, W.M. (1934), "The Bearing Value of Rollers," Bulletin No. 263, University of Illinois Engineering Experiment Station, Urbana, IL.

Winter, G. (1947), "Strength of Thin Steel Compression Flanges," *Transactions of the ASCE*, Vol. 112, 1947, p. 547.

Winter, G. (1958), "Lateral Bracing of Columns and Beams," *Journal of the Structural Division*, ASCE, Vol. 84, No. ST2, March, pp. 1,561-1–1,561-22.

Winter, G. (1960), "Lateral Bracing of Columns and Beams," *Transactions of the ASCE*, Vol. 125, Part 1, pp. 809–825.

Winter, G. (1968), *Commentary on the Specification for the Design of Cold-Formed Steel Members*, American Iron and Steel Institute, Washington, DC.

Winter, G. (1970), *Light Gage Cold-Formed Steel Design Manual: Commentary of the 1968 edition*, American Iron and Steel Institute, Washington, DC.

Yuan, H. (1996), "The Resistances of Stud Shear Connectors with Profiled Sheeting," Ph.D. Dissertation, Department of Engineering, The University of Warwick, Coventry, United Kingdom.

Yuan, Q., Swanson, J., and Rassati, G.A. (2004), "An Investigation of Hole Making Practices in the Fabrication of Structural Steel," Internal Report, Department of Civil and Environmental Engineering, University of Cincinnati, Cincinnati, OH.

Yura, J.A. (1971), "The Effective Length of Columns in Unbraced Frames," *Engineering Journal*, AISC, Vol. 8, No. 2, 2nd Quarter, April, pp. 37–42.

Yura, J.A., Galambos, T.V., and Ravindra, K. (1978), "The Bending Resistance of Steel Beams," *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, September, pp. 1355–1370.

Yura, J.A. (1993), "Fundamentals of Beam Bracing," *Is Your Structure Suitably Braced?*, *Proceedings of the Annual Technical Session and Meeting*, Milwaukee, Wisconsin, April 6–7, 1993, Structural Stability Research Council, Bethlehem, PA.

Yura, J.A. (1995), "Bracing for Stability-State-of-the-Art," *Proceedings of the ASCE Structures Congress XIII*, Boston, MA, April 2–5, 1995, American Society of Civil Engineers, New York, NY, pp. 88–103.

Zandonini, R. (1985), "Stability of Compact Built-Up Struts: Experimental Investigation and Numerical Simulation," *Costruzioni Metalliche*, No. 4.