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American Iron and Steel Institute

AISI STANDARD

Commentary on North American Specification for the Design of Cold-Formed Steel Structural Members

2001 EDITION

The material contained herein has been developed by a joint effort of the American Iron and Steel Institute Committee on Specifications, the Canadian Standards Association Technical Committee on Cold Formed Steel Structural Members (S136), and Camara Nacional de la Industria del Hierro y del Acero (CANACERO) in Mexico. The organizations and the Committees have made a diligent effort to present accurate, reliable, and useful information on cold-formed steel design. The Committees acknowledge and are grateful for the contributions of the numerous researchers, engineers, and others who have contributed to the body of knowledge on the subject. Specific references are included in the *Commentary* on the *Specification*.

With anticipated improvements in understanding of the behavior of cold-formed steel and the continuing development of new technology, this material may eventually become dated. It is anticipated that future editions of this specification will update this material as new information becomes available, but this cannot be guaranteed.

The materials set forth herein are for general information only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a registered professional engineer. Indeed, in most jurisdictions, such review is required by law. Anyone making use of the information set forth herein does so at their own risk and assumes any and all resulting liability arising therefrom.

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PREFACE

This document provides a commentary on the 2007 edition of the *North American Specification for the Design of Cold-Formed Steel Structural Members.* This *Commentary* should be used in combination with the 2008 edition of the AISI *Cold-Formed Steel Design Manual*.

The purpose of the *Commentary* includes: (a) to provide a record of the reasoning behind, and justification for the various provisions of the *North American Specification* by cross-referencing the published supporting research data and to discuss the changes make in the current *Specification;* (b) to offer a brief but coherent presentation of the characteristics and performance of cold-formed steel structures to structural engineers and other interested individuals; (c) to furnish the background material for a study of cold-formed steel design methods to educators and students; and (d) to provide the needed information to those who will be responsible for future revisions of the *Specification.* The readers who wish to have more complete information, or who may have questions which are not answered by the abbreviated presentation of this *Commentary*, should refer to the original research publications.

Consistent with the *Specification*, the *Commentary* contains a main document, Chapters A through G, and Appendices 1 and 2, and Appendices A and B. A symbol ☞A,B is used in the main document to point out that additional discussions are provided in the corresponding country specific provisions in Appendices A and/or B.

The assistance and close cooperation of the North American Specification Committee under the Chairmanship of Professor Reinhold M. Schuster and the AISI Committee on Specifications under the Chairmanship of Mr. Roger L. Brockenbrough and the Vice Chairmanship of Mr. Jay W. Larson are gratefully acknowledged. Special thanks are extended to Professor Wei-Wen Yu for revising the draft of this *Commentary.* The Institute is very grateful to members of the Editorial Subcommittee and all members of the AISI Committee on Specifications for their careful review of the document and their valuable comments and suggestions. The background materials provided by various subcommittees are appreciated.

American Iron and Steel Institute
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COMMENTARY ON THE 2007 EDITION OF THE NORTH AMERICAN SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

INTRODUCTION

Cold-formed steel members have been used economically for building construction and other applications (Winter, 1959a, 1959b; Yu, 2000). These types of sections are cold-formed from steel sheet, strip, plate or flat bar in roll-forming machines or by press brake or bending operations. The thicknesses of steel sheets or strip generally used for cold-formed steel structural members range from 0.0147 in. (0.373 mm) to about 1/4 in. (6.35 mm). Steel plates and bars as thick as 1 in. (25.4 mm) can be cold-formed successfully into structural shapes.

In general, cold-formed steel structural members can offer several advantages for building construction (Winter, 1970; Yu, 2000): (1) light members can be manufactured for relatively light loads and/or short spans, (2) unusual sectional configurations can be produced economically by cold-forming operations and consequently favorable strength-to-weight ratios can be obtained, (3) load-carrying panels and decks can provide useful surfaces for floor, roof and wall construction, and in some cases they can also provide enclosed cells for electrical and other conduits, and (4) panels and decks not only withstand loads normal to their surfaces, but they can also act as shear diaphragms to resist forces in their own planes if they are adequately interconnected to each other and to supporting members.

The use of cold-formed steel members in building construction began in about the 1850s. However, in North America such steel members were not widely used in buildings until the publication of the first edition of the American Iron and Steel Institute (AISI) *Specification* in 1946 (AISI, 1946). This first design standard was primarily based on the research work sponsored by AISI at Cornell University since 1939. It was revised subsequently by the AISI Committee in 1956, 1960, 1962, 1968, 1980, and 1986 to reflect the technical developments and the results of continuing research. In 1991, AISI published the first edition of the *Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members* (AISI, 1991). Both *allowable stress design* (ASD) and *load and resistance factor design* (LRFD) *specifications* were combined into a single document in 1996. In Canada, the Canadian Standards Association (CSA) published its first edition of *Design of Light Gauge Steel Structural Members* in 1963 based on the 1962 edition of the AISI *Specification*. Subsequent editions were published in 1974, 1984, 1989 and 1994. The Canadian *Standard for Cold-Formed Steel Structural Members* (CSA, 1994) was based on the *Limit States Design* (LSD) method.

In Mexico, cold-formed steel structural members have also been designed on the basis of AISI *Specifications*. The 1962 edition of the AISI *Design Manual* (AISI, 1962) was translated to Spanish in 1965 (Camara, 1965).

The first edition of the unified *North American Specification* (AISI, 2001) was prepared and issued in 2001. It was applicable to the United States, Canada, and Mexico for the design of cold-formed steel structural members. The 2001 edition of the *Specification* was developed on the basis of the 1996 AISI *Specification* with the 1999 *Supplement* (AISI, 1996, 1999), the 1994 CSA *Standard* (CSA, 1994), and subsequent developments. In 2001, the term "Allowable Stress Design" was renamed to "Allowable Strength Design" to clarify the nature of this design method. In the *North American Specification*, the ASD and LRFD methods are used in the United States and Mexico, while the LSD method is used in Canada. The *Supplement* to the 2001 edition of the *North American Specification* was published in 2004 (AISI, 2004b), in which the new *Direct Strength Method* was added in the *Specification* as Appendix 1. Following the successful use of the first *North American Specification* for seven years, it was revised and expanded in 2007 on the basis of the results of continued research and new developments (AISI, 2007a). This updated edition of the *Specification* includes the new Appendix 2 for the Second-Order Analysis of structural systems. Additionally, Appendix A has been expanded to be applicable to Mexico and, consequently, Appendix C has been deleted.

In addition to the issuance of the design specification, AISI also published the first edition of the *Design Manual* in 1949 (AISI, 1949). This allowable stress design manual was revised later in 1956, 1961, 1962, 1968, 1977, 1983, and 1986. In 1991, the *LRFD Design Manual* was published for using the load and resistance factor design criteria. The AISI 1996 *Cold-Formed Design Manual* was prepared for the combined AISI ASD and LRFD *Specifications.* For using the 2001 edition of the *North American Specification*, AISI published the 2002 edition of the *Cold-Formed Steel Design Manual* (AISI, 2002). In 2008, the new *Design Manual* (AISI, 2008) will be published by AISI based on the 2007 edition of the *North American Specification*.

During the period from 1958 through 1983, AISI published *Commentaries* on several editions of the AISI design specification, which were prepared by Professor George Winter of Cornell University in 1958, 1961, 1962, and 1970. From 1983, the format used for the AISI *Commentary* has been changed in that the same section numbers are used in the *Commentary* as in the *Specification.* The *Commentary* on the 1996 AISI *Specification* was prepared by Professor Wei-Wen Yu of the University of Missouri-Rolla (Yu, 1996). The 2001 edition of the *Commentary* (AISI, 2001) was based on the *Commentary* on the 1996 AISI *Specification*. The current edition of the *Commentary* (AISI, 2007b) was updated for the 2007 edition of the *North American Specification* with extensive additions and revisions. It contains Chapters A through G, Appendices 1 and 2, and Appendices A and B, where commentary on provisions that are only applicable to a specific country is included in the corresponding Appendix.

As in previous editions of the *Commentary*, this document contains a brief presentation of the characteristics and the performance of cold-formed steel members, connections and assemblies. In addition, it provides a record of the reasoning behind, and the justification for, various provisions of the specification. A cross-reference is provided between various design provisions and the published research data.

In this *Commentary,* the individual sections, equations, figures, and tables are identified by the same notation as in the *Specification* and the material is presented in the same sequence. Bracketed terms used in the *Commentary* are equivalent terms that apply particularly to the LSD method in Canada.

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

A. GENERAL PROVISIONS

A1 Scope, Applicability, and Definitions

A1.1 Scope

The cross-sectional configurations, manufacturing processes and fabrication practices of cold-formed steel structural members differ in several respects from that of hot-rolled steel shapes. For cold-formed steel sections, the forming process is performed at, or near, room temperature by the use of bending brakes, press brakes, or roll-forming machines. Some of the significant differences between cold-formed sections and hot-rolled shapes are (1) absence of the residual stresses caused by uneven cooling due to hot-rolling, (2) lack of corner fillets, (3) presence of increased yield stress with decreased proportional limit and ductility resulting from cold-forming, (4) presence of cold-reducing stresses when cold-rolled steel stock has not been finally annealed, (5) prevalence of elements having large width-to-thickness ratios, (6) rounded corners, and (7) stress-strain curves can be either sharp-yielding type or gradual-yielding type.

The *Specification* is applicable only to cold-formed sections not more than 1 inch (25.4 mm) in thickness. Research conducted at the University of Missouri-Rolla (Yu, Liu, and McKinney, 1973b and 1974) has verified the applicability of the specification's provisions for such cases.

In view of the fact that most of the design provisions have been developed on the basis of the experimental work subject to static loading, the *Specification* is intended for the design of cold-formed steel structural members to be used for load-carrying purposes in buildings. For structures other than buildings, appropriate allowances should be made for dynamic effects.

☞A

A1.2 Applicability

The *Specification* (AISI, 2007a) is limited to the design of steel structural members cold-formed from carbon or low-alloy sheet, strip, plate or bar. The design can be made by using either the Allowable Strength Design (ASD) method or the Load and Resistance Factor Design (LRFD) method for the United States and Mexico. Only the Limit States Design (LSD) method is permitted in Canada.

In this Commentary, the bracketed terms are equivalent terms that apply particularly to LSD. A symbol ☞x is used to point out that additional provisions are provided in the country specific appendices as indicated by the letter, x.

Because of the diverse forms in which cold-formed steel structural members can be used, it is not possible to cover all design configurations by the design rules presented in the *Specification*. For those special cases where the available strength [factored resistance]* and/or stiffness cannot be so determined, it can be established either by (a) testing and evaluation in accord with the provisions of Chapter F, or (b) rational engineering analysis. Prior to 2001, the only option in such cases was testing. However, since 2001, in recognition of the fact that this was not always practical or necessary, the rational engineering analysis option was added. It is essential that such analysis be based on theory that is appropriate for the situation, any available test data that is relevant, and sound engineering judgment. Safety and resistance factors are provided for ease of use, but these factors should not be used if applicable safety factors or resistance factors in the main *Specification* are more conservative, where the main *Specification* refers to Chapters A through G, Appendices A and B, and Appendix 2. These provisions must not be used to circumvent the intent of the *Specification*. Where the provisions of Chapters B through G of the *Specification* and Appendices A and B apply, those provisions must be used and cannot be avoided by testing or rational analysis.

In 2004, Appendix 1, *Design of Cold-Formed Steel Structural Members Using the Direct Strength Method*, was introduced (AISI, 2004b). The Appendix provides an alternative design procedure for several Sections of Chapters C. The Direct Strength Method detailed in Appendix 1 requires (1) determination of the elastic buckling behavior of the member, and then provides (2) a series of nominal strength [resistance] curves for predicting the member strength based on the elastic buckling behavior. The procedure does not require effective width calculations, nor iteration, and instead uses gross properties and the elastic buckling behavior of the cross-section to predict the strength. The applicability of the provided provisions is detailed in the General Provisions of Appendix 1.

In 2007, Appendix 2, *Second-Order Analysis*, was added in the *Specification* (AISI, 2007a). The provisions of this Appendix are based on the studies conducted by Sarawit and Pekoz at Cornell University with due considerations given to flexural-torsional buckling, semi-rigid joints, and local instabilities. The second-order analysis was found to be more accurate than the effect length approach.

A1.3 Definitions

Many of the definitions in *Specification* Section A1.3 for ASD, LRFD and LSD are self-explanatory. Only those which are not self-explanatory are briefly discussed below.

General Terms

Effective Design Width

The effective design width is a concept which facilitates taking account of local buckling and post-buckling strength for compression elements. The effect of shear lag on short, wide flanges is also handled by using an effective design width. These matters are treated in Specification Chapter B, and the corresponding effective widths are discussed in the Commentary on that chapter.

Multiple-Stiffened Elements

Multiple-stiffened elements of two sections are shown in Figure C-A1.3-1. Each of the two outer sub-elements of section (1) are stiffened by a web and an intermediate stiffener while the middle sub-element is stiffened by two intermediate stiffeners. The two sub-elements of section (2) are stiffened by a web and the attached intermediate middle stiffener.

Stiffened or Partially Stiffened Compression Elements

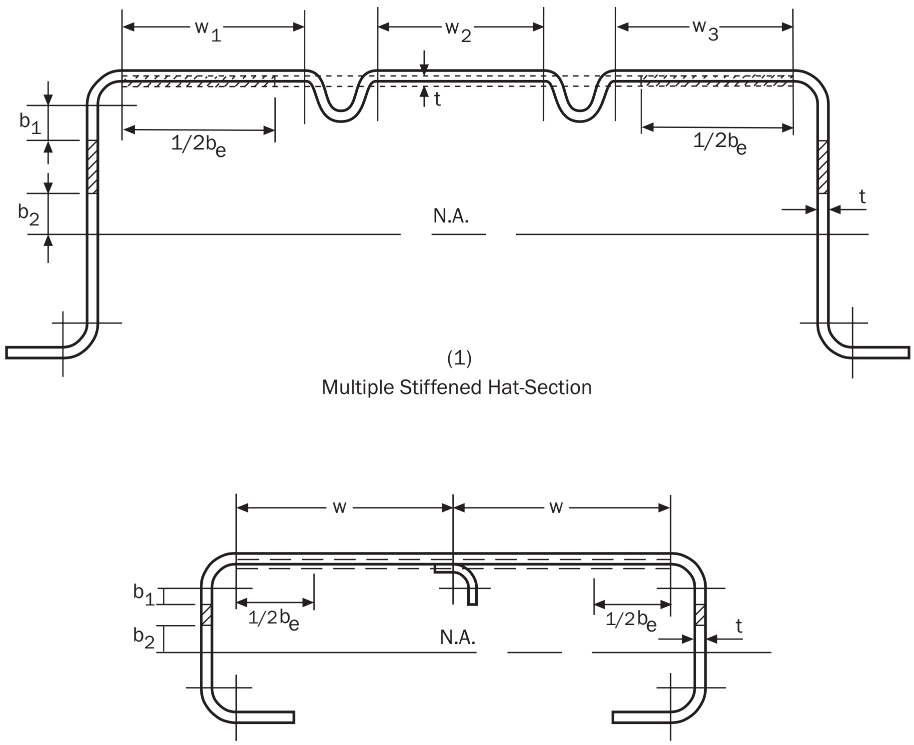
Stiffened compression elements of various sections are shown in Figure C-A1.3-2, in which sections (1) through (5) are for flexural members, and sections (6) through (9) are for compression members. Sections (1) and (2) each have a web and a lip to stiffen the compression element (i.e., the compression flange), the ineffective portion of which is shown shaded. For the explanation of these ineffective portions, see the discussion of Effective Design Width and Chapter B. Sections (3), (4), and (5) show compression elements stiffened by two webs. Sections (6) and (8) show edge stiffened flange elements that have a vertical element (web) and an edge stiffener (lip) to stiffen the elements while the web itself is stiffened by the flanges. Section (7) has four compression elements stiffening each other, and section (9) has each stiffened element stiffened by a lip and the other stiffened element.

Thickness

In calculating section properties, the reduction in thickness that occurs at corner bends is ignored, and the base metal thickness of the flat steel stock, exclusive of coatings, is used in all calculations for load-carrying purposes.

Flexural-Torsional Buckling

The 1968 edition of the Specification pioneered methods for computing column loads of cold-formed steel sections prone to buckle by simultaneous twisting and bending. This complex behavior may result in lower column loads than would result from primary buckling by flexure alone.



(1)
Multiple Stiffened Hat-Section

(2)
Multiple Stiffened Inverted "U"-Type Section

Flexural Members, such as Beams

Figure C-A1.3-1 Multiple-Stiffened Compression Elements

Unstiffened Compression Elements

Unstiffened elements of various sections are shown in Figure C-A1.3-3, in which sections (1) through (4) are for flexural members and sections (5) through (8) are for compression members. Sections (1), (2), and (3) have only a web to stiffen the compression flange element. The legs of section (4) provide mutual stiffening action to each other along their common edges. Sections (5), (6), and (7), acting as columns have vertical stiffened elements (webs) which provide support for one edge of the unstiffened flange elements. The legs of section (8) provide mutual stiffening action to each other.

ASD and LRFD Terms (USA and Mexico)

ASD (Allowable Strength Design, formerly *referred to as Allowable Stress Design)*

Allowable Strength Design (ASD) is a method of designing structural components such that the allowable strength (force or moment) permitted by various sections of the *Specification* is not exceeded when the structure is subjected to all appropriate combinations of nominal loads as given in Section A4.1.2 of Appendix A of the *Specification*.

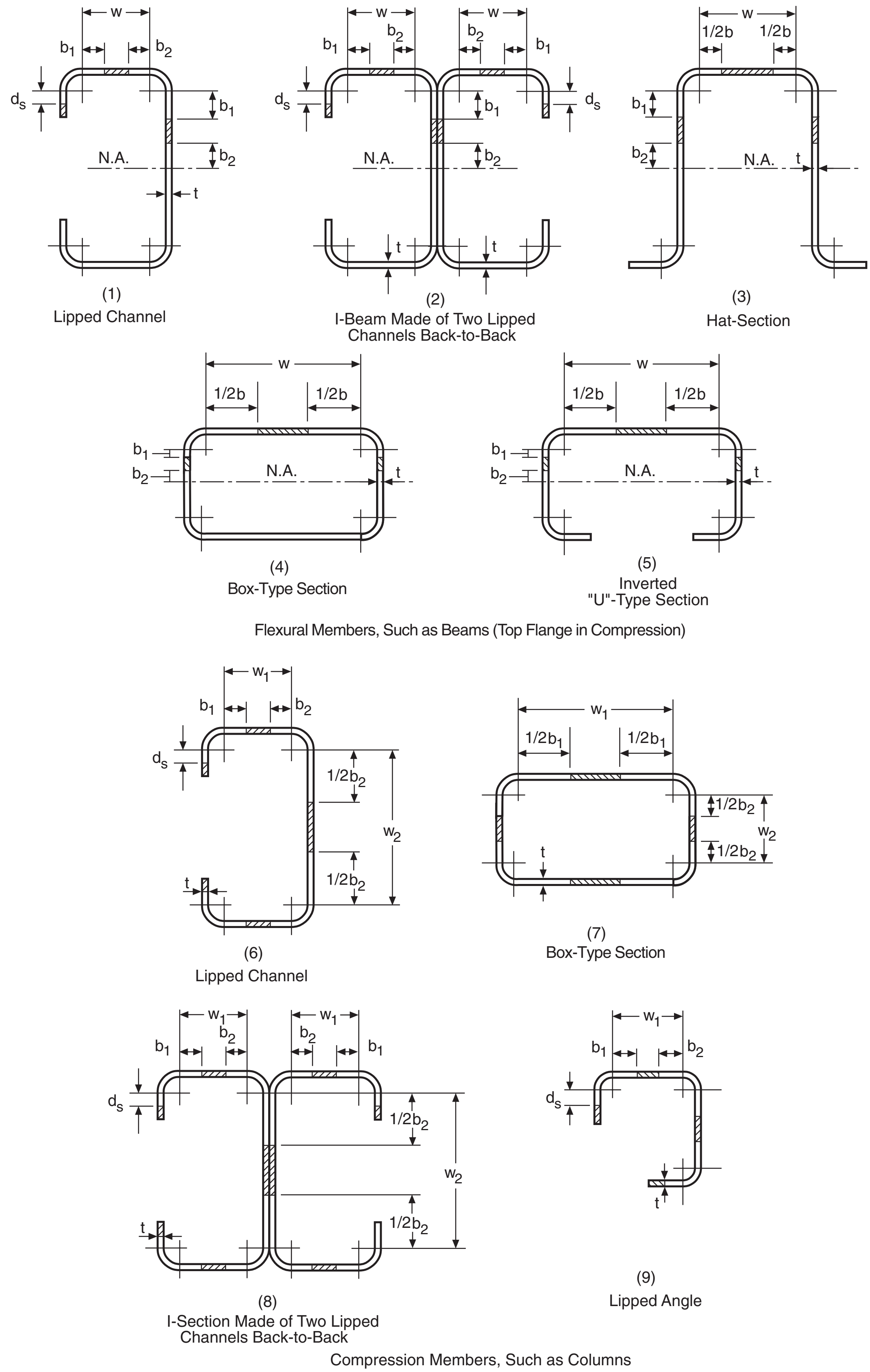


Figure C-A1.3-2 Stiffened Compression Elements

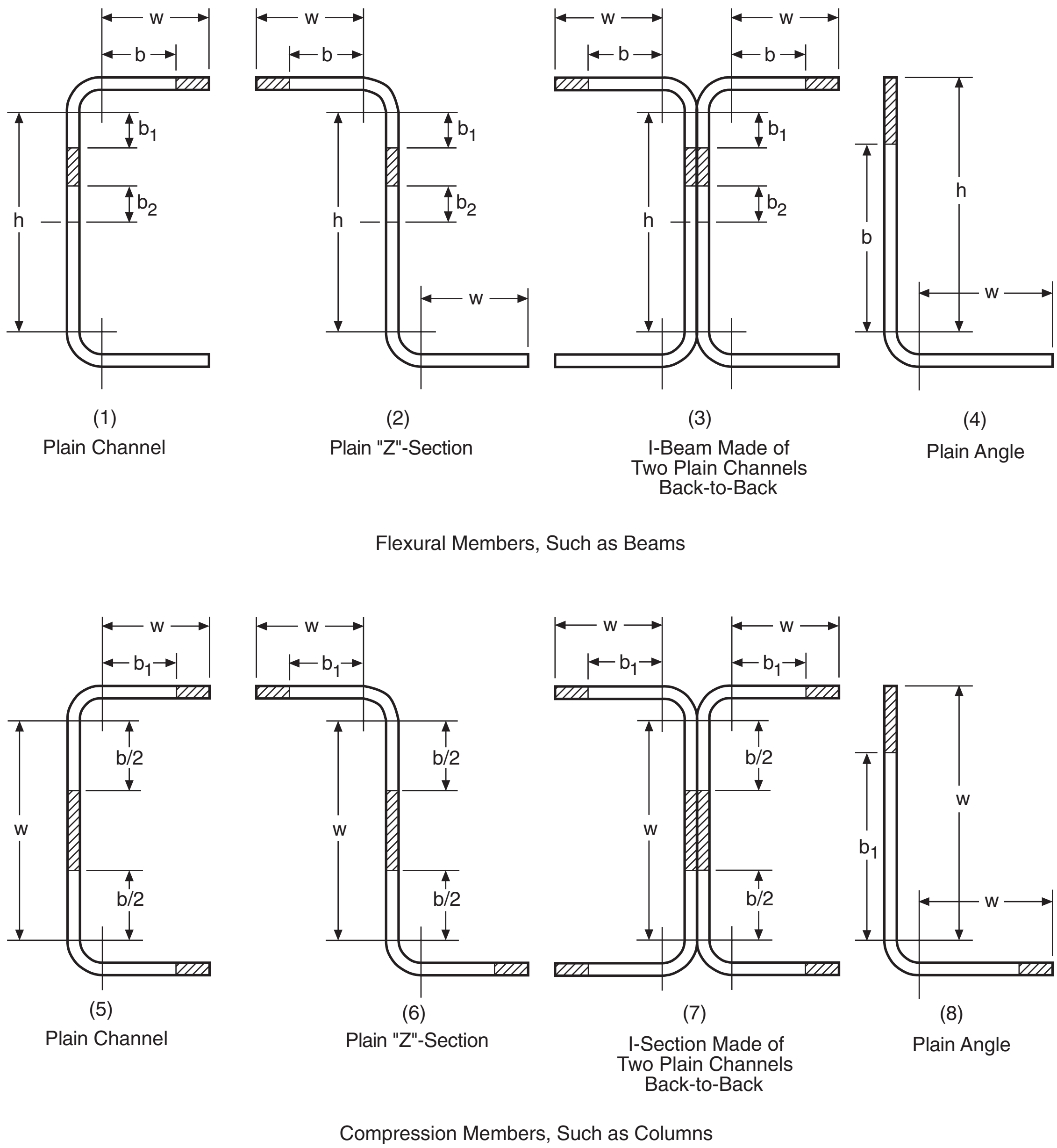


Figure C-A1.3-3 Unstiffened Compression Elements

LRFD (Load and Resistance Factor Design)

Load and Resistance Factor Design (LRFD) is a method of designing structural components such that the applicable limit state is not exceeded when the structure is subjected to all appropriate load combinations as given in Section A5.1.2 of Appendix A of the *Specification*. See also *Specification* Section A5.1.1 for LRFD strength requirements.

LSD Terms (Canada)

LSD (Limit States Design)

Limit States Design (LSD) is a method of designing structural components such that the applicable limit state is not exceeded when the structure is subjected to all appropriate load combinations as given in Section A6.1.2 of Appendix B of the *Specification*. See also *Specification* Section A6.1.1 for LSD requirements.

In the *North American Specification*, the terminologies for Limit States Design (LSD) are given in brackets parallel to those for load and resistance factor design (LRFD). The inclusion of LSD terminology is intended to help engineers who are familiar with LSD better understand the *Specification*.

It should be noted that the design concept used for the LRFD and the LSD methods is the same, except that the load factors, load combinations, assumed dead-to-live ratios, and target reliability indexes are slightly different. In most cases, same nominal strength [nominal resistance] equations are used for ASD, LRFD, and LSD approaches.

A1.4 Units of Symbols and Terms

The non-dimensional character of the majority of the *Specification* provisions is intended to facilitate design in any compatible systems of units (U.S. customary, SI or metric, and MKS systems).

The conversion of U.S. customary into SI metric units and MKS systems are given in parentheses through out the entire text of the Specification and Commentary. Table C-A1.4-1 is a conversion table for these three different units.

Table C-A1.4-1
Conversion Table

	To Convert	To	Multiply by
Length	in.	mm	25.4
	mm	in.	0.03937
	ft	m	0.30480
	m	ft	3.28084
Area	in^2	mm^2	645.160
	mm^2	in^2	0.00155
	ft^2	m^2	0.09290
	m^2	ft^2	10.7639
Force	kip	kN	4.448
	kip	kg	453.5
	lb	N	4.448
	lb	kg	0.4535
	kN	kip	0.2248
	kN	kg	101.96
	kg	kip	0.0022
	kg	N	9.808
Stress	ksi	MPa	6.895
	ksi	kg/cm^2	70.30
	MPa	ksi	0.145
	MPa	kg/cm^2	10.196
	kg/cm^2	ksi	0.0142
	kg/cm^2	MPa	0.0981

A2 Material

A2.1 Applicable Steels

The American Society for Testing and Materials (ASTM) is the basic source of steel designations for use with the *Specification.* Section A2.1 contains the complete list of ASTM Standards for steels that are accepted by the *Specification.* Dates of issue are included in Section A9. Other standards that are applicable to a specific country are listed in the corresponding Appendix.

In the AISI 1996 *Specification*, the ASTM A446 Standard was replaced by the ASTM A653/A653M Standard. At the same time, the ASTM A283/A283M Standard, High-Strength, Low-Alloy Steel (HSLAS) Grades 70 (480) and 80 (550) of ASTM A653/A653M and ASTM A715 were added.

In 2001, the ASTM A1008/A1008M and ASTM A1011/A1011M Standards replaced the ASTM A570/A570M, ASTM A607, ASTM A611, and ASTM A715 Standards. ASTM A1003/A1003M was added to the list of *Specification* Section A2.1.

In 2007, the ASTM A1039 Standard was added to the list of *Specification* Section A2.1. For all grades of steel, ASTM A1039 complies with the minimum required F_u/F_y ratio of 1.08. Thicknesses equal to or greater than 0.064 in. (1.6 mm) also meet the minimum elongation requirements of *Specification* Section A2.3.1 and no reduction in the specified minimum yield stress is required. However, steel thicknesses less than 0.064 in. (1.6 mm) with yield stresses greater than 55 ksi (380 MPa) do not meet the requirements of *Specification* Section A2.3.1 and are subject to the limitations of *Specification* Section A2.3.2.

The important material properties for the design of cold-formed steel members are: yield stress, tensile strength, and ductility. Ductility is the ability of a steel to undergo sizable plastic or permanent strains before fracturing and is important both for structural safety and for cold forming. It is usually measured by the elongation in a 2-inch (51 mm) gage length. The ratio of the tensile strength to the yield stress is also an important material property; this is an indication of strain hardening and the ability of the material to redistribute stress.

For the listed ASTM Standards, the yield stresses of steels range from 24 to 80 ksi (165 to 550 MPa or 1690 to 5620 kg/cm^2) and the tensile strengths vary from 42 to 100 ksi (290 to 690 MPa or 2950 to 7030 kg/cm^2). The tensile-to-yield ratios are no less than 1.13, and the elongations are no less than 10 percent. Exceptions are ASTM A653/A653M SS Grade 80 (550); specific thicknesses of ASTM A1039/A1039M 55 (380), 60 (410), 70 (480), and 80 (550), ASTM A1008/A1008M SS Grade 80 (550); and ASTM A792/A792M SS Grade 80 (550) steels with a specified minimum yield stress of 80 ksi (550 MPa or 5620 kg/cm^2), a specified minimum tensile strength of 82 ksi (565 MPa or 5770 kg/cm^2), and with no stipulated minimum elongation in 2 inches (51 mm). These low ductility steels permit only limited amounts of cold forming, require fairly large corner radii, and have other limits on their applicability for structural framing members. Nevertheless, they have been used successfully for specific applications, such as decks and panels with large corner radii and little, if any, stress concentrations. The conditions for use of these SS Grade 80 (550) steels are outlined in *Specification* Section A2.3.2.

For ASTM A1003/A1003M steel, even though the minimum tensile strength is not specified in the ASTM Standard for each of Types H and L Steels, the footnote of Table 2 of the Standard states that for Type H steels the ratio of tensile strength to yield stress shall not be less than 1.08. Thus, a conservative value of $F_u = 1.08\ F_y$ can be used for the design of cold-formed steel members using Type H steels. Based on the same Standard, a conservative value of $F_u = F_y$ can be used for the design of purlins and girts using Type L steels. In 2004, the *Specification* listing of ASTM A1003/A1003M steel was revised to list only the grades designated Type H, because this is the only grade that satisfies the criterion for unrestricted usage. Grades designated Type L can still be used but are subject to the restrictions of *Specification* Section A2.3.1.

A2.2 Other Steels

Comments on other steels are provided in the corresponding Appendices of this *Commentary*. ☜A,B

A2.3 Ductility

The nature and importance of ductility and the ways in which this property is measured were briefly discussed in *Commentary* Section A2.1.

Low-carbon sheet and strip steels with specified minimum yield stresses from 24 to 50 ksi (165 to 345 MPa or 1690 to 3520 kg/cm^2) need to meet ASTM specified minimum elongations in a 2-inch (51 mm) gage length of 11 to 30 percent. In order to meet the ductility requirements, steels with yield stresses higher than 50 ksi (345 MPa or 3520 kg/cm^2) are often low-alloy steels. However, SS Grade 80 (550) of ASTM A653/A653M, SS Grade 80 (550) of A1008/A1008M, SS Grade 80 (550) of A792/A792M, and SS Grade 80 (550) of A875/A875M steels are carbon steels, for which specified minimum yield stress is 80 ksi (550 MPa or 5620 kg/cm^2) and no elongation requirement is specified. These differ from the array of steels listed under *Specification* Section A2.1.

In 1968, because new steels of higher strengths were being developed, sometimes with lower elongations, the question of how much elongation is really needed in a structure was the focus of a study initiated at Cornell University. Steels were studied that had yield stresses ranging from 45 to 100 ksi (310 to 690 MPa or 3160 to 7030 kg/cm^2), elongations in 2 inches (51 mm) ranging from 50 to 1.3 percent, and tensile strength-to-yield stress ratios ranging from 1.51 to 1.00 (Dhalla, Errera and Winter, 1971; Dhalla and Winter, 1974a; Dhalla and Winter, 1974b). The investigators developed elongation requirements for ductile steels. These measurements are more accurate but cumbersome to make; therefore, the investigators recommended the following determination for adequately ductile steels: (1) The tensile strength-to-yield stress ratio shall not be less than 1.08 and (2) the total elongation in a 2-inch (51-mm) gage length shall not be less than 10 percent, or not less than 7 percent in an 8-inch (203-mm) gage length. Also, the *Specification* limits the use of Chapters B through E to adequately ductile steels. In lieu of the tensile-to-yield stress limit of 1.08, the *Specification* permits the use of elongation requirements using the measurement technique as given by Dhalla and Winter (1974a) (Yu, 2000). Further information on the test procedure should be obtained from "Standard Methods for Determination of Uniform and Local Ductility", *Cold-Formed Steel Design Manual*, Part VI (AISI, 2008). Because of limited experimental verification of the structural performance of members using materials having a tensile strength-to-yield stress ratio less than 1.08 (Macadam et al., 1988), the *Specification* limits the use of this material to purlins, girts, and curtain wall studs meeting the elastic design requirements of *Specification* Sections C3.1.1(a), C3.1.2, D6.1.1, D6.1.2, D6.2.1, and additional country specific requirements given in Appendices. Thus, the use of such steels in other applications (compression members, except closed box section compression members as stipulated in *Specification* Exception 2, tension members, other flexural members including those whose strength [resistance] is based on inelastic reserve capacity, etc.) is prohibited. However, in purlins, girts, and curtain wall studs, (with special country specific requirements given in Appendix A or B), concurrent axial loads of relatively small magnitude are acceptable providing the requirements of *Specification* Section C5.2 are met and $\Omega_c P/P_n$ does not exceed 0.15 for allowable strength design, $P_u/\phi_c P_n$ does not exceed 0.15 for the Load and Resistance Factor Design, and $P_f/\phi_c P_n$ does not exceed 0.15 for the Limit States Design. A,B

In 2007, curtain wall studs were added to the applications for materials having a tensile strength-to-yield stress ratio less than 1.08. Curtain wall studs are repetitive framing members that are typically spaced more closely than purlins and girts. Curtain wall studs are analogous to vertical girts; as such, they are not subjected to snow or other significant sustained gravity_loads. Pending future research regarding the cyclic performance of connections, an exception is noted on use of these lower ductility steels for curtain wall studs supporting heavy weight exterior walls in high seismic areas.

SS Grade 80 (550) of ASTM A653/A653M, SS Grade 80 (550) of ASTM A1008/A1008M, SS Grade 80 (550) of A792/A792M, and SS Grade 80 (550) of A875/A875M steels do not have adequate ductility as defined by *Specification* Section A2.3.1. Their use has been limited in *Specification* Section A2.3.2 to particular multiple-web configurations such as roofing, siding, and floor decking.

In the past, the yield stress used in design was limited to 75 percent of the specified minimum yield stress, or 60 ksi (414 MPa or 4220 kg/cm^2), and the tensile strength used in design was limited to 75 percent of the specified minimum tensile strength, or 62 ksi (427 MPa or 4360 kg/cm^2) whichever was lower. This introduced a higher safety factor, but still made low ductility steels, such as SS Grade 80 (550) and Grade E, useful for the named applications.

Based on the UMR research findings (Wu, Yu, and LaBoube, 1996), *Specification* Equation A2.3.2-1 was added in *Specification* Section A2.3.2 under Exception 1 to determine the reduced yield stress, $R_b F_{sy}$, for the calculation of the nominal flexural strength [moment resistance] of multiple-web sections such as roofing, siding and floor decking (AISI, 1999). For the unstiffened compression flange, *Specification* Equation A2.3.2-2 was added on the basis of a 1988 UMR study (Pan and Yu, 1988). This revision allows the use of a higher nominal bending strength [resistance] than previous editions of the AISI *Specification*. When the multiple-web section is composed of both stiffened and unstiffened compression flange elements, the smallest R_b should be used to determine the reduced yield stress for use on the entire section. Different values of the reduced yield stress could be used for positive and negative moments.

The equations provided in *Specification* Exception 1 can also be used for calculating the nominal flexural strength [resistance] when the design strengths [factored resistances] are determined on the basis of tests as permitted by the alternative method.

It should be noted that Exception 1 does not apply to the steel deck used for composite slabs when the deck is used as the tensile reinforcement. This limitation is to prevent the possible sudden failure of the composite slab due to lack of ductility of the steel deck.

For the calculation of web crippling strength [resistance] of deck panels, although the UMR study (Wu, Yu, and LaBoube, 1997) shows that the specified minimum yield stress can be used to calculate the web crippling strength [resistance] of deck panels, the *Specification* is adopting a conservative approach in *Specification* Section C3.4. The lesser of 0.75 F_{sy} and 60 ksi (414 MPa or 4220 kg/cm^2) is used to determine both the web crippling strength [resistance] and the shear strength [resistance] for the low ductility steels. This is consistent with the previous edition of the AISI *Specification*.

Another UMR study (Koka, Yu, and LaBoube, 1997) confirmed that for the connection design using SS Grade 80 (550) of A653/A653M steel, the tensile strength used in design should be taken as 75 percent of the specified minimum tensile strength or 62 ksi (427 MPa or 4360 kg/cm^2), whichever is less. It should be noted that the current design provisions are limited only to the design of members and connections subjected to static loading without the considerations of fatigue strength.

Load tests are permitted, but not for the purpose of using higher loads than can be calculated under *Specification* Chapters B through G.

For the calculation of the strength [resistance] of concentrically loaded compression members with a closed box section, *Specification* Exception 2 was added on the basis of a study at University of Sydney (Yang, Hancock, 2002). For short members where $F_n = F_y$ in *Specification* Section C4, the study shows that the limit of the yield stress used in the design can be 90 percent of the specified minimum yield stress F_{sy} for low ductility steels. Tests were performed on box-sections composed of G550 steel of AS1397 which is similar to ASTM A792 Grade 80. The box-section is formed by connecting the lips of two hat sections.

Further, for calculating the strength [resistance] of concentrically loaded long compression members, *Specification* Equations A2.3.2-3 and A2.3.2-4, based on the University of Sydney research findings (Yang, Hancock and Rasmussen, 2002), were added in the *Specification* Section A2.3.2 in Exception 2 when determining the nominal axial strength [nominal axial resistance] according to *Specification* Section C4.1.1. The reduction factor R_r specified in *Specification* Equation A2.3.2-4 is to be applied to the radius of gyration r and allows for the interaction of local and flexural (Euler) buckling of thin high strength low ductility steel sections. The reduction factor is a function of the length varying from 0.65 at KL = 0 to 1.0 at KL = $1.1L_0$, where L_0 is the length at which the local buckling stress equals the flexural buckling stress.

A2.4 Delivered Minimum Thickness

Sheet and strip steels, both coated and uncoated, may be ordered to nominal or minimum thickness. If the steel is ordered to minimum thickness, all thickness tolerances are over (+) and nothing under (-). If the steel is ordered to nominal thickness, the thickness tolerances are divided equally between over and under. Therefore, in order to provide the similar material thickness between the two methods of ordering sheet and strip steel, it was decided to require that the delivered thickness of a cold-formed product be at least 95 percent of the design thickness. Thus, it is apparent that a portion of the safety factor or resistance factor may be considered to cover minor negative thickness tolerances.

Generally, thickness measurements should be made in the center of flanges. For decking and siding, measurements should be made as close as practical to the center of the first full flat of the section. Thickness measurements should not be made closer to edges than the minimum distances specified in ASTM A568 Standard.

The responsibility of meeting this requirement for a cold-formed product is clearly that of the manufacturer of the product, not the steel producer.

In 2004, the country specific section, *Specification* Section A2.4a, was deleted from Appendix B.

A3 Loads

Comments on loads and load combinations for different countries are provided in the corresponding Appendices of this *Commentary*.

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A4 Allowable Strength Design

A4.1 Design Basis

The *Allowable Strength Design* method has been featured in AISI specifications beginning with the 1946 edition. It is included in the *Specification* along with the LRFD and the LSD methods for use in the United States, Mexico, and Canada since the 2001 edition.

A4.1.1 ASD Requirements

In the Allowable Strength Design approach, the required strengths (bending moments, axial forces, and shear forces) in structural members are computed by accepted methods of structural analysis for the specified nominal or working loads for all applicable load combinations determined according to *Specification* Section A4.1.2. These required strengths are not to exceed the allowable strengths permitted by the *Specification.* According to *Specification* Section A4.1.1, the allowable strength is determined by dividing the nominal strength by a safety factor as follows:

$$R \leq R_n/\Omega \qquad \text{(C-A4.1.1-1)}$$

where

R = required strength

R_n = nominal strength

Ω = safety factor

The fundamental nature of the safety factor is to compensate for uncertainties inherent in the design, fabrication, or erection of building components, as well as uncertainties in the estimation of applied loads. Appropriate safety factors are explicitly specified in various sections of the *Specification.* Through experience it has been established that the present safety factors provide satisfactory design. It should be noted that the ASD method employs only one safety factor for a given condition regardless of the type of load.

A4.1.2 Load Combinations for ASD

Comments for load combinations are provided in Appendix A of this *Commentary*.

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A5 Load and Resistance Factor Design

A5.1 Design Basis

A limit state is the condition at which the structural usefulness of a load-carrying element or member is impaired to such an extent that it becomes unsafe for the occupants of the structure, or the element no longer performs its intended function. Typical limit states for cold-formed steel members are excessive deflection, yielding, buckling and attainment of maximum strength after local buckling (i.e., postbuckling strength). These limit states have been established through experience in practice or in the laboratory, and they have been thoroughly investigated through analytical and experimental research. The background for the establishment of the limit states is extensively documented in (Winter, 1970; Pekoz, 1986b; and Yu, 2000), and a continuing research effort provides further improvement in understanding them.

Two types of limit states are considered in the load and resistance factor design method. They are: (1) the limit state of the strength required to resist the extreme loads during the intended life of the structure, and (2) the limit state of the ability of the structure to perform its intended function during its life. These two limit states are usually referred to as the limit state of strength and limit state of serviceability. Like the ASD method, the LRFD method focuses on the limit state of strength in *Specification* Section A5.1.1 and the limit state of serviceability in *Specification* Section A8.

A5.1.1 LRFD Requirements

For the limit state of strength, the general format of the LRFD method is expressed by the following equation:

$$\Sigma\gamma_i Q_i \leq \phi R_n \tag{C-A5.1.1-1}$$

or

$$R_u \leq \phi R_n$$

where

R_u = $\Sigma\gamma_i Q_i$ = required strength
R_n = nominal resistance
ϕ = resistance factor
γ_i = load factors
Q_i = load effects
ϕR_n = design strength

The nominal resistance is the strength of the element or member for a given limit state, computed for nominal section properties and for minimum specified material properties according to the appropriate analytical model which defines the strength. The resistance factor ϕ accounts for the uncertainties and variabilities inherent in the R_n, and it is usually less than unity. The load effects Q_i are the forces on the cross section (i.e, bending moment, axial force, or shear force) determined from the specified nominal loads by structural analysis and γ_i are the corresponding load factors, which account for the uncertainties and variabilities of the loads. The load factors for LRFD are discussed in the *Commentary* on Appendix A for the United States and Mexico.

The advantages of LRFD are: (1) the uncertainties and the variabilities of different types of loads and resistances are different (e.g., dead load is less variable than wind load), and so these differences can be accounted for by use of multiple factors, and (2) by using probability theory designs can ideally achieve a more consistent reliability. Thus LRFD provides the basis for a more rational and refined design method than is possible with the ASD method.

(a) Probabilistic Concepts

Safety Factors or load factors are provided against the uncertainties and variabilities which are inherent in the design process. Structural design consists of comparing nominal load effects Q to nominal resistances R, but both Q and R are random parameters (see Figure C-A5.1.1-1). A limit state is violated if R<Q. While the possibility of this event ever occurring is never zero, a successful design should, nevertheless, have only an acceptably small probability of exceeding the limit state. If the exact probability distributions of Q and R were known, then the probability of (R - Q) < 0 could be exactly determined for any design. In general the distributions of Q and R are not known, and only the means, Q_m and R_m, and the standard deviations, σ_Q and σ_R are available. Nevertheless it is possible to determine relative reliabilities of several designs by the scheme illustrated in Figure C-A5.1.1-2. The distribution curve shown is for ln(R/Q), and a limit state is exceeded when $ln(R/Q) \leq 0$. The area under $ln(R/Q) \leq 0$ is the probability of violating the limit state. The size of this area is dependent on the distance between the origin and the mean of ln(R/Q). For given statistical data R_m, Q_m, σ_R and σ_Q, the area under $ln(R/Q) \leq 0$ can be varied by

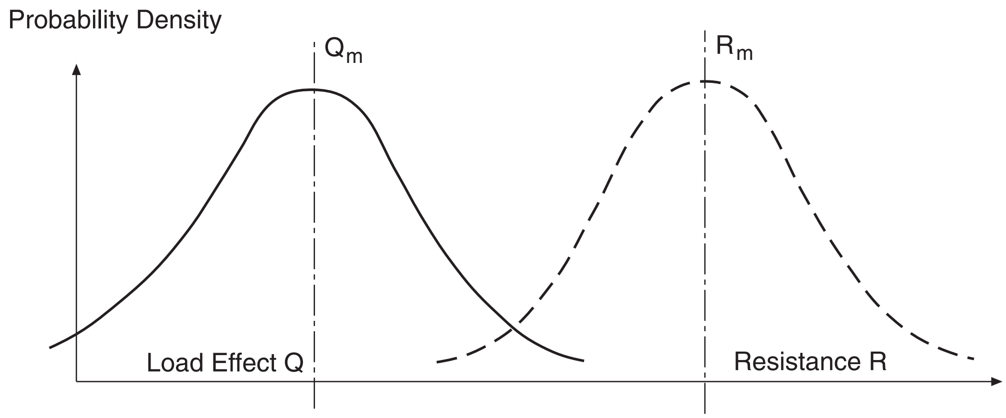

Figure C-A5.1.1-1 Definition of the Randomness Q and R

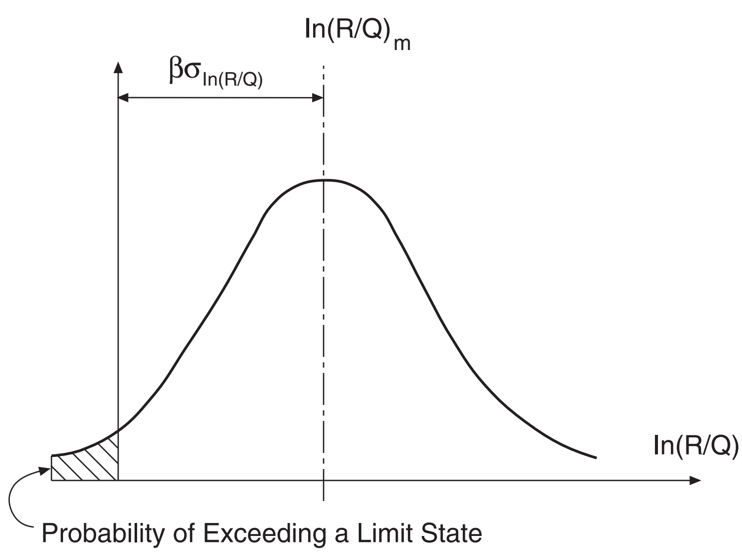

Figure C-A5.1.1-2 Definition of the Reliability Index β

changing the value of β (Figure C-A5.1.1-2), since $\beta\sigma_{ln(R/Q)} = ln(R/Q)_m$, from which approximately

$$\beta = \frac{\ln(R_m / Q_m)}{\sqrt{V_R^2 + V_Q^2}} \qquad \text{(C-A5.1.1-2)}$$

where $V_R = \sigma_R/R_m$ and $V_Q = \sigma_Q/Q_m$, the coefficients of variation of R and Q, respectively. The index β is called the "reliability index", and it is a relative measure of the safety of the design. When two designs are compared, the one with the larger β is more reliable.

The concept of the reliability index can be used for determining the relative reliability inherent in current design, and it can be used in testing out the reliability of new design formats, as illustrated by the following example of simply supported, braced beams subjected to dead and live loading.

The ASD design requirement of the *Specification* for such a beam is

$$S_eF_y/\Omega = (L_s^2 s/8)(D+L) \qquad \text{(C-A5.1.1-3)}$$

where

S_e = elastic section modulus based on the effective section

Ω = 5/3 = the safety factor for bending

F_y = specified yield stress

L_s = span length, and s = beam spacing

D and L are, respectively, the code specified dead and live load intensities.

The mean resistance is defined as (Ravindra and Galambos, 1978)

$$R_m = R_n(P_mM_mF_m) \qquad \text{(C-A5.1.1-4)}$$

In the above equation, R_n is the nominal resistance, which in this case is

$$R_n = S_eF_y \qquad \text{(C-A5.1.1-5)}$$

that is, the nominal moment predicted on the basis of the postbuckling strength of the compression flange and the web. The mean values P_m, M_m, and F_m, and the corresponding coefficients of variation V_P, V_M, and V_F, are the statistical parameters, which define the variability of the resistance:

P_m = mean ratio of the experimentally determined moment to the predicted moment for the actual material and cross-sectional properties of the test specimens

M_m = mean ratio of the actual yield stress to the minimum specified value

F_m = mean ratio of the actual section modulus to the specified (nominal) value

The coefficient of variation of R equals

$$V_R = \sqrt{V_P{}^2 + V_M{}^2 + V_F{}^2} \qquad \text{(C-A5.1.1-6)}$$

The values of these data were obtained from examining the available tests on beams having different compression flanges with partially and fully effective flanges and webs, and from analyzing data on yield stress values from tests and cross-sectional dimensions from many measurements. This information was developed from research (Hsiao, Yu, and Galambos, 1988a and 1990; Hsiao, 1989) and is given below:

$P_m = 1.11$, $V_P = 0.09$; $M_m = 1.10$, $V_M = 0.10$; $F_m = 1.0$, $V_F = 0.05$ and thus $R_m = 1.22R_n$ and $V_R = 0.14$.

The mean load effect is equal to

$$Q_m = (L_s^2 s/8)(D_m + L_m) \qquad \text{(C-A5.1.1-7)}$$

and

$$V_Q = \frac{\sqrt{(D_m V_D)^2 + (L_m V_L)^2}}{D_m + L_m} \qquad \text{(C-A5.1.1-8)}$$

where D_m and L_m are the mean dead and live load intensities, respectively, and V_D and V_L are the corresponding coefficients of variation.

Load statistics have been analyzed in a study of the National Bureau of Standards (NBS) (Ellingwood et al., 1980), where it was shown that D_m = 1.05D, V_D = 0.1; L_m = L, V_L = 0.25.

The mean live load intensity equals the code live load intensity if the tributary area is small enough so that no live load reduction is included. Substitution of the load statistics into Equations C-A5.1.1-7 and C-A5.1.1-8 gives

$$Q_m = \frac{L_s^2 s}{8}\left(\frac{1.05D}{L} + 1\right)L \qquad \text{(C-A5.1.1-9)}$$

$$V_Q = \frac{\sqrt{(1.05D/L)^2 V_D^2 + V_L^2}}{(1.05D/L + 1)} \qquad \text{(C-A5.1.1-10)}$$

Q_m and V_Q thus depend on the dead-to-live load ratio. Cold-formed steel beams typically have small D/L ratio, which may vary for different applications. Different D/L ratio may be assumed by different countries for developing design criteria. For the purposes of checking the reliability of these LRFD criteria it has been assumed that D/L = 1/5, and so $Q_m = 1.21L(L_s^2\, s/8)$ and V_Q = 0.21.

From Equations C-A5.1.1-3 and C-A5.1.1-5, the nominal resistance, R_n, can be obtained for D/L = 1/5 and Ω = 5/3 as follows:

$$R_n = 2L(L_s^2 s/8)$$

In order to determine the reliability index, β, from Equation C-A5.1.1-2, the R_m/Q_m ratio is required by considering $R_m = 1.22R_n$:

$$\frac{R_m}{Q_m} = \frac{1.22 \text{x} 2.0 \text{x} L(L_s^2 s/8)}{1.21L(L_s^2 s/8)} = 2.02$$

Therefore, from Equation C-A5.1.1-2,

$$\beta = \frac{\ln(2.02)}{\sqrt{0.14^2 + 0.21^2}} = 2.79$$

Of itself β= 2.79 for beams having different compression flanges with partially and fully effective flanges and webs designed by the *Specification* means nothing. However, when this is compared to β for other types of cold-formed steel members, and to β for designs of various types from hot-rolled steel shapes or even for other materials, then it is possible to say that this particular cold-formed steel beam has about an average reliability (Galambos et al., 1982).

(b) Basis for LRFD of Cold-Formed Steel Structures

A great deal of work has been performed for determining the values of the reliability index β inherent in traditional design as exemplified by the current structural design specifications such as the ANSI/AISC S360 for hot-rolled steel, the AISI *Specification* for cold-formed steel, the ACI Code for reinforced concrete members, etc. The studies for hot-rolled steel are summarized by Ravindra and Galambos (1978), where also many further papers are referenced which contain additional data. The determination of β for cold-formed steel elements or members is presented in several research reports of the University of Missouri-Rolla (Hsiao, Yu, and Galambos, 1988a; Rang, Galambos, and Yu, 1979a, 1979b, 1979c, and 1979d; Supornsilaphachai, Galambos, and Yu, 1979), where both the basic research data as well as the β's inherent in the AISI *Specification* are presented in great detail. The β's computed in the above referenced publications were developed with slightly different load statistics than those of this *Commentary,* but the essential conclusions remain the same.

The entire set of data for hot-rolled steel and cold-formed steel designs, as well as data for reinforced concrete, aluminum, laminated timber, and masonry walls was re-analyzed by Ellingwood, Galambos, MacGregor, and Cornell (Ellingwood et al., 1980; Galambos et al., 1982; Ellingwood et al., 1982) using (a) updated load statistics and (b) a more advanced level of probability analysis which was able to incorporate probability distributions and to describe the true distributions more realistically. The details of this extensive reanalysis are presented by the investigators. Only the final conclusions from the analysis are summarized below.

The values of the reliability index β vary considerably for the different kinds of loading, the different types of construction, and the different types of members within a given material design specification. In order to achieve more consistent reliability, it was suggested by Ellingwood et al. (1982) that the following values of β would provide this improved consistency while at the same time give, on the average, essentially the same design by the LRFD method as is obtained by current design for all materials of construction. These target reliabilities β_o for use in LRFD are:

Basic case: Gravity loading, $\beta_o = 3.0$

For connections: $\beta_o = 4.5$

For wind loading: $\beta_o = 2.5$

These target reliability indices are the ones inherent in the load factors recommended in the ASCE 7-98 Load Standard (ASCE, 1998).

For simply supported, braced cold-formed steel beams with stiffened flanges, which were designed according to the allowable strength design method in the current *Specification* or to any previous version of the AISI *Specification*, it was shown that for the representative dead-to-live load ratio of 1/5 the reliability index $\beta = 2.79$. Considering the fact that for other such load ratios, or for other types of members, the reliability index inherent in current cold-formed steel construction could be more or less than this value of 2.79, a somewhat lower target reliability index of $\beta_o = 2.5$ is recommended as a lower limit in the United States. The resistance factors ϕ were selected such that $\beta_o = 2.5$ is essentially the lower bound of the actual β's for members. In order to assure that failure of a structure is not initiated in the connections, a higher target reliability of $\beta_o = 3.5$ is recommended for joints and fasteners in the United States. These two targets of 2.5 and 3.5 for members and connections, respectively, are somewhat lower than those recommended by the ASCE 7-98 (i.e., 3.0 and 4.5, respectively), but they are essentially the same targets as are the basis for the AISC LRFD *Specification* (AISC, 1999). For wind loading, the same ASCE target value of β_o = 2.5 is used for connections in the US LRFD method. For flexural members such as individual purlins, girts, panels, and roof decks subjected to the combination of dead and wind loads, the target β_o value used in the United States is reduced to 1.5. With this reduced target reliability index, the design based on the US LRFD method is comparable to the US allowable strength design method.

(c) Resistance Factors

The following portions of this *Commentary* present the background for the resistance factors ϕ which are recommended for various members and connections in Chapters B through E (AISI, 1996). These ϕ factors are determined in conformance with the ASCE 7 load factors to provide approximately a target β_o of 2.5 for members and 3.5 for connections, respectively, for a typical load combination 1.2D+1.6L. For practical reasons, it is desirable to have relatively few different resistance factors, and so the actual values of β will differ from the derived targets. This means that

$$\phi R_n = c(1.2D+1.6L) = (1.2D/L+1.6)cL \qquad \text{(C-A5.1.1-11)}$$

where c is the deterministic influence coefficient translating load intensities to load effects.

By assuming D/L = 1/5, Equations C-A5.1.1-11 and C-A5.1.1-9 can be rewritten as follows:

$$R_n = 1.84(cL/\phi) \qquad \text{(C-A5.1.1-12)}$$

$$Q_m = (1.05D/L+1)cL = 1.21cL \qquad \text{(C-A5.1.1-13)}$$

Therefore,

$$R_m/Q_m = (1.521/\phi)(R_m/R_n) \qquad \text{(C-A5.1.1-14)}$$

The ϕ factor can be computed from Equation C-A5.1.1-15 on the basis of Equations C-A5.1.1-2, C-A5.1.1-4 and C-A5.1.1-14 (Hsiao, Yu and Galambos, 1988b, AISI 1996):

$$\phi = 1.521\,(P_m M_m F_m)\exp(-\beta_o\sqrt{V_R^{\,2} + V_Q^{\,2}}\,) \qquad \text{(C-A5.1.1-15)}$$

in which, β_o is the target reliability index. Other symbols were defined previously.

By knowing the ϕ factor, the corresponding safety factor, Ω, for allowable strength design can be computed for the load combination 1.2D+1.6L as follows:

$$\Omega = (1.2D/L + 1.6)/[\phi(D/L + 1)] \qquad \text{(C-A5.1.1-16)}$$

where D/L is the dead-to-live load ratio for the given condition.

A5.1.2 Load Factors and Load Combinations for LRFD

Comments for load factors and load combinations are provided in Appendix A of this *Commentary*.

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A6 Limit States Design

A6.1 Design Basis

Same as the LRFD method, a limit state is the condition at which the structural usefulness of a load-carrying element or member is impaired to such an extent that it becomes unsafe for the occupants of the structure, or the element no longer performs its intended function. Typical limit states for cold-formed steel members are excessive deflection, yielding, buckling and attainment of maximum strength after local buckling (i.e., postbuckling strength). These limit states have been established through experience in practice or in the laboratory, and they have been thoroughly investigated through analytical and experimental research.

Two types of limit states are considered in the *Limit States Design* method. They are: (1) the limit state of the strength required to resist the extreme loads during the intended life of the structure, and (2) the limit state of the ability of the structure to perform its intended function during its life. These two limit states are usually referred to as the limit state of strength and limit state of serviceability. The LSD method focuses on the limit state of strength in *Specification* Section A6.1.1 and the limit state of serviceability in *Specification* Section A8.

A6.1.1 LSD Requirements

For the limit state of strength, the general format of the LSD method is expressed by the following equation:

$$\phi R_n \geq \Sigma \gamma_i Q_i \quad \text{(C-A6.1.1-1)}$$

or

$$\phi R_n \geq R_f$$

where

R_f = $\Sigma\gamma_i Q_i$ = effect of factored loads
R_n = nominal resistance
ϕ = resistance factor
γ_i = load factors
Q_i = load effects
ϕR_n = factored resistance

The nominal resistance is the strength of the element or member for a given limit state, computed for nominal section properties and for minimum specified material properties according to the appropriate analytical model which defines the resistance. The resistance factor ϕ accounts for the uncertainties and variabilities inherent in the R_n, and it is usually less than unity. The load effects Q_i are the forces on the cross section (i.e, bending moment, axial force, or shear force) determined from the specified nominal loads by structural analysis and γ_i are the corresponding load factors, which account for the uncertainties and variabilities of the loads. The load factors for LSD are discussed in the *Commentary* on Appendix B.

Since the design basis for the LSD and the LRFD is the same, further discussions on how to obtain resistance factor using probability analysis can be obtained from Section A5.1.1 (c) of the *Commentary*. However, attention should be paid that target values for members and connections as well as the dead-to-live load ratio may vary from country to country. These variations lead to the differences in resistance factors. The dead-to-live ratio used in Canada is assumed to be 1/3, and the target of the reliability index for cold-formed steel structural members is 3.0 for members and 4.0 for connections. These target values are consistent with those used in other CSA design standards.

A6.1.2 Load Factors and Load Combinations for LSD

Comments for load factors and load combinations are provided in Appendix B of this

Commentary.

A7 Yield Stress and Strength Increase from Cold Work of Forming

A7.1 Yield Stress

The strength [resistance] of cold-formed steel structural members depends on the yield stress, except in those cases where elastic local buckling or overall buckling is critical. Because the stress-strain curve of steel sheet or strip can be either sharp-yielding type (Figure C-A7.1-1(a)) or gradual-yielding type (Figure C-A7.1-1(b)), the method for determining the yield point for sharp-yielding steel and the yield strength for gradual-yielding steel are based on the ASTM Standard A370 (ASTM, 1997). As shown in Figure C-A7.1-2(a), the yield point for sharp-yielding steel is defined by the stress level of the plateau. For gradual-yielding steel, the stress-strain curve is rounded out at the "knee" and the yield strength is determined by either the offset method (Figure C-A7.1-2(b)) or the extension under the load method (Figure C-A7.1-2(c)). The term *yield stress* used in the *Specification* applies to either yield point or yield strength. Section 1.2 of the AISI *Design Manual* (AISI, 2008) lists the minimum mechanical properties specified by the ASTM specifications for various steels.

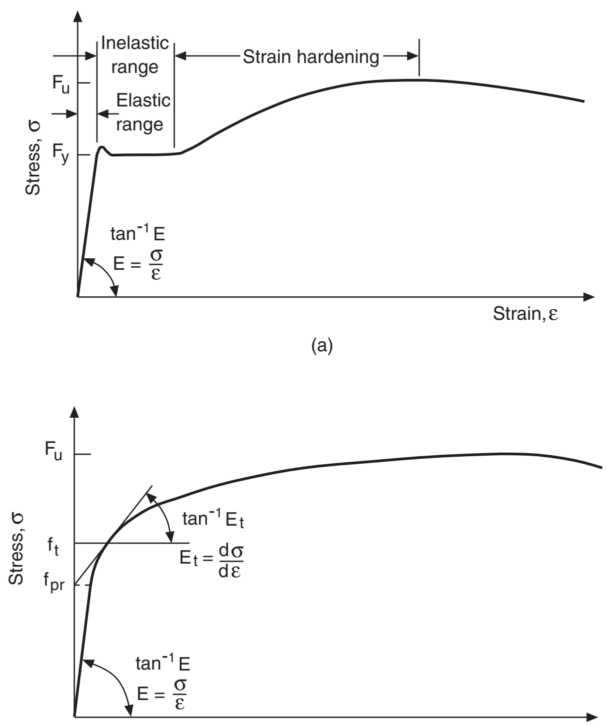


Figure C-A7.1-1 Stress-Strain Curves of Carbon Steel Sheet or Strip

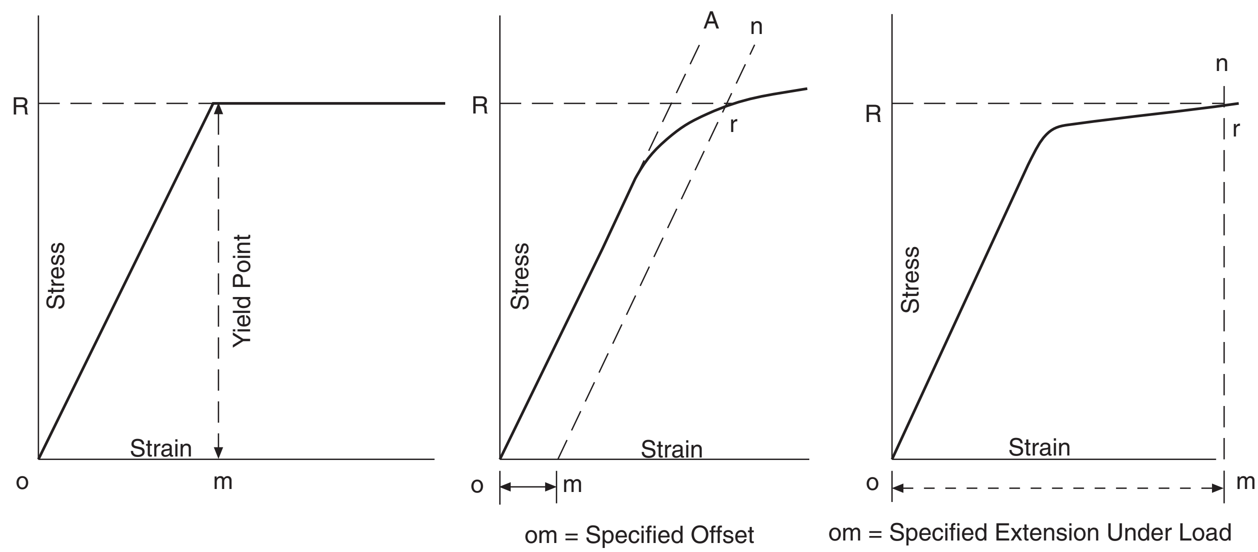
(a) Sharp Yielding, (b) Gradual Yielding

The strength [resistance] of members that are governed by buckling depends not only on the yield stress but also on the modulus of elasticity, E, and the tangent modulus, E_t. The modulus of elasticity is defined by the slope of the initial straight portion of the stress-strain curve (Figure C-A7.1-1). The measured values of E on the basis of the standard methods usually range from 29,000 to 30,000 ksi (200 to 207 GPa or 2.0x10^6 to 2.1x10^6 kg/cm^2). A value of 29,500 ksi (203 GPa or 2.07×10^6 kg/cm^2) is used in the *Specification* for design purposes. The tangent modulus is defined by the slope of the stress-strain curve at any stress level, as shown in Figure C-A7.1-1(b).

For sharp-yielding steels, E_t = E up to the yield point, but with gradual-yielding steels, E_t = E only up to the proportional limit, f_{pr}. Once the stress exceeds the proportional limit, the tangent modulus E_t becomes progressively smaller than the initial modulus of elasticity.

Various buckling provisions of the *Specification* have been written for gradual-yielding steels whose proportional limit is not lower than about 70 percent of the specified minimum yield stress.

Determination of proportional limits for information purposes can be done simply by using the offset method shown in Figure C-A7.1-2(b) with the distance "om" equal to 0.0001 length/length (0.01 percent offset) and calling the stress R where "mn" intersects the stress-strain curve at "r", the proportional limit.



(a) Showing Yield Point Corresponding with Top of Knee.

(b) Showing Yield Point or Yield Strength by the Offset Method. (Also Used for Proportional Limit)

(c) Determination of Yield Strength by Extension Under Load Method.

Figure C-A7.1-2 Stress-Strain Diagrams Showing Methods of Yield Point and Yield Strength Determination

A7.2 Strength Increase from Cold Work of Forming

The mechanical properties of the flat steel sheet, strip, plate or bar, such as yield stress, tensile strength, and elongation may be substantially different from the properties exhibited by the cold-formed steel sections. Figure C-A7.2-1 illustrates the increase of yield stress and tensile strength from those of the virgin material at the section locations in a cold-formed steel channel section and a joist chord (Karren and Winter, 1967). This difference can be attributed to cold working of the material during the cold-forming process.

The influence of cold work on mechanical properties was investigated by Chajes, Britvec, Winter, Karren, and Uribe at Cornell University in the 1960s (Chajes, Britvec, and Winter, 1963; Karren, 1967; Karren and Winter, 1967; Winter and Uribe, 1968). It was found that the changes of mechanical properties due to cold-stretching are caused mainly by strain-hardening and strain-aging, as illustrated in Figure C-A7.2-2 (Chajes, Britvec, and Winter, 1963). In this figure, curve A represents the stress-strain curve of the virgin material. Curve B is due to unloading in the strain-hardening range, curve C represents immediate reloading, and curve D is the stress-strain curve of reloading after strain-aging. It is interesting to note that the yield stresses of both curves C and D are higher than the yield point of the virgin material and that the ductilities decrease after strain hardening and strain aging.

Cornell research also revealed that the effects of cold work on the mechanical properties of corners usually depend on (1) the type of steel, (2) the type of stress (compression or tension), (3) the direction of stress with respect to the direction of cold work (transverse or longitudinal), (4) the F_u/F_y ratio, (5) the inside radius-to-thickness ratio (R/t), and (6) the amount of cold work. Among the above items, the F_u/F_y and R/t ratios are the most important factors to affect the change in mechanical properties of formed sections. Virgin material with a large F_u/F_y ratio possesses a large potential for strain hardening. Consequently as the F_u/F_y ratio increases, the effect of cold work on the increase in the yield stress of steel increases. Small inside radius-to-thickness ratios, R/t, correspond to a large degree of cold work in a corner, and therefore, for a given material, the smaller the R/t ratio, the larger the increase in yield stress.

Investigating the influence of cold work, Karren derived the following equations for the ratio of corner yield stress-to-virgin yield stress (Karren, 1967):

$$\frac{F_{y_c}}{F_{yv}} = \frac{B_c}{(R/t)^m} \qquad \text{(C-A7.2-1)}$$

where

$$B_c = 3.69\frac{F_{uv}}{F_{yv}} - 0.819\left(\frac{F_{uv}}{F_{yv}}\right)^2 - 1.79$$

and

$$m = 0.192\frac{F_{uv}}{F_{yv}} - 0.068$$

F_{yc} = corner yield stress
F_{yv} = virgin yield stress
F_{uv} = virgin ultimate tensile strength
R = inside bend radius
t = sheet thickness

With regard to the full-section properties, the tensile yield stress of the full section may be approximated by using a weighted average as follows:

$$F_{ya} = CF_{yc} + (1 - C)F_{yf} \qquad \text{(C-A7.2-2)}$$

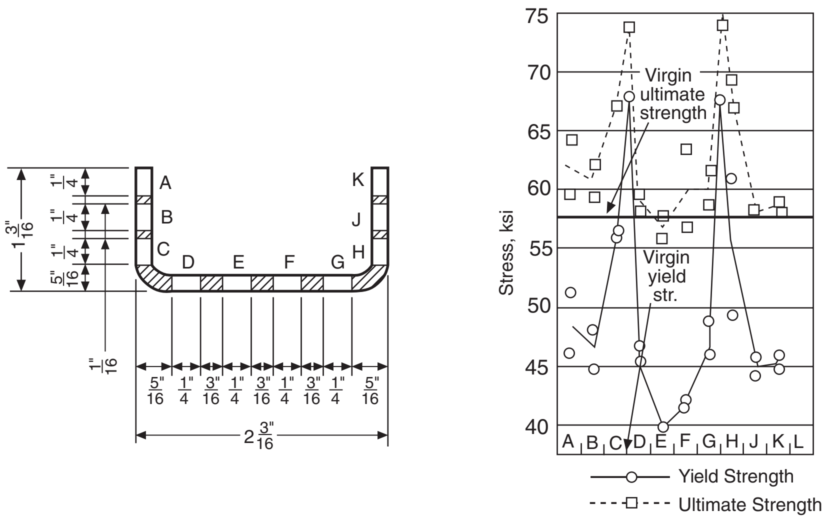
where
F_{ya} = full-section tensile yield stress
F_{yc} = average tensile yield stress of corners = $B_cF_{yv}/(R/t)^m$
F_{yf} = average tensile yield stress of flats
C = ratio of corner area to total cross-sectional area. For flexural members having

unequal flanges, the one giving a smaller C value is considered to be the controlling flange

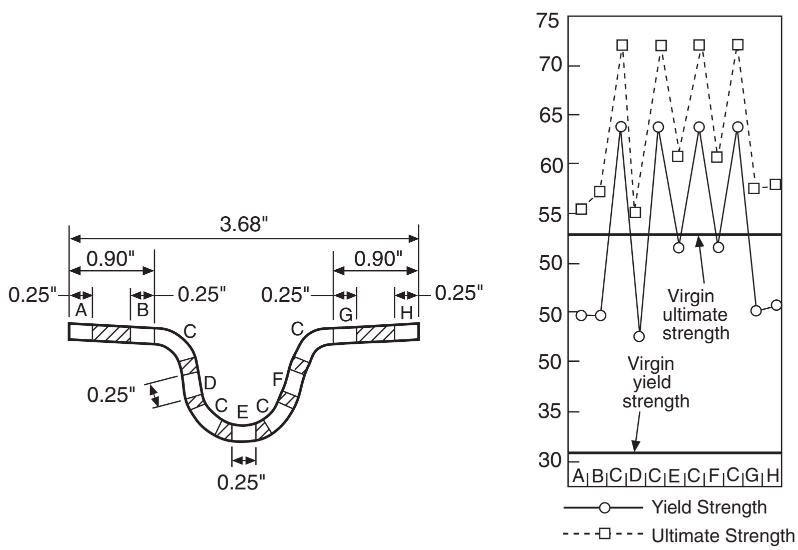
Good agreements between the computed and the tested stress-strain characteristics for a channel section and a joist chord section were demonstrated by Karren and Winter (Karren and Winter, 1967).

The limitation $F_{ya} \leq F_{uv}$ places an upper bound on the average yield stress. The intent of the upper bound is to limit stresses in flat elements that may not see significant increases in yield stress and tensile strength as compared to the virgin steel properties.

In the last three decades, additional studies have been made by numerous investigators. These investigations dealt with the cold-formed sections having large R/t ratios and with thick materials. They also considered residual stress distribution, simplification of design methods, and other related subjects. For details, see Yu (2000).



(a)



(b)

Figure C-A7.2-1 Effect of Cold-Work on Mechanical Properties in Cold-Formed Steel Sections. (a) Channel Section, (b) Joist Chord

In 1962, the AISI *Specification* permitted the utilization of cold work of forming on the basis of full section tests. Since 1968, the AISI *Specification* has allowed the use of the increased average yield stress of the section, F_{ya}, to be determined by (1) full section tensile tests, (2) stub column tests, or (3) computed in accordance with Equation C-A7.2-2. However, such a strength increase is limited only to relatively compact sections designed according to *Specification* Section C2 (tension members), Section C3.1 (bending strength excluding the use of inelastic reserve capacity), Section C4 (concentrically loaded compression members), Section C5 (combined axial load and bending), Section D4 (cold-formed steel light-frame construction), and Section D6.1 (purlins, girts and other members). Design Example of the 2008 *Cold-Formed Steel Design Manual* (AISI, 2008) demonstrates the use of strength increase from cold work of forming for a channel section to be used as a beam.

In some cases, when evaluating the effective width of the web, the reduction factor ρ according to *Specification* Section B2.3 may be less than unity but the sum of b_1 and b_2 of Figure B2.3-1 of the *Specification* may be such that the web is fully effective, and cold work of forming may be used. This situation only arises when the web width to flange width ratio, h_o/b_o, is less than or equal to 4.

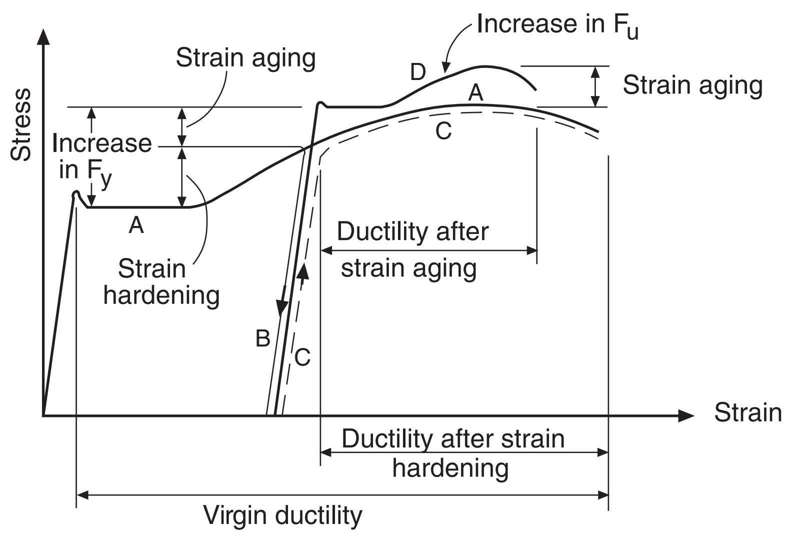


Figure C-A7.2-2 Effect of Strain Hardening and Strain Aging on Stress-Strain Characteristics

In the development of the AISI LRFD *Specification*, the following statistical data on material and cross-sectional properties were developed by Rang, Galambos and Yu (1979a and 1979b) for use in the derivation of resistance factors ϕ:

$(F_y)_m = 1.10F_y$; $M_m = 1.10$; $V_{fy} = V_M = 0.10$

$(F_{ya})_m = 1.10F_{ya}$; $M_m = 1.10$; $V_{Fya} = V_M = 0.11$

$(F_u)_m = 1.10F_u$; $M_m = 1.10$; $V_{Fu} = V_M = 0.08$

$F_m = 1.00$; $V_F = 0.05$

In the above expressions, m refers to mean value, V represents coefficient of variation, M and F are, respectively, the ratios of the actual-to-the nominal material property and cross-sectional property; and F_y, F_{ya}, and F_u are, respectively, the specified minimum yield stress, the average yield stress including the effect of cold forming, and the specified minimum tensile strength.

These statistical data are based on the analysis of many samples (Rang et al., 1978) and they are representative properties of materials and cross sections used in the industrial application of cold-formed steel structures.

A8 Serviceability

Serviceability limit states are conditions under which a structure can no longer perform its intended functions. Safety and strength [resistance] considerations are generally not affected by serviceability limit states. However, serviceability criteria are essential to ensure functional performance and economy of design.

Common conditions which may require serviceability limits are:

1. Excessive deflections or rotations which may affect the appearance or functional use of the structure. Deflections which may cause damage to non-structural elements should be considered.
2. Excessive vibrations which may cause occupant discomfort of equipment malfunctions.
3. Deterioration over time which may include corrosion or appearance considerations.

When checking serviceability, the designer should consider appropriate service loads, the response of the structure, and the reaction of building occupants.

Service loads that may require consideration include static loads, snow or rain loads, temperature fluctuations, and dynamic loads from human activities, wind-induced effects, or the operation of equipment. The service loads are actual loads that act on the structure at an arbitrary point in time. Appropriate service loads for checking serviceability limit states may only be a fraction of the nominal loads.

The response of the structure to service loads can normally be analyzed assuming linear elastic behavior. However, members that accumulate residual deformations under service loads may require consideration of this long-term behavior.

Serviceability limits depend on the function of the structure and on the perceptions of the observer. In contrast to the strength [resistance] limit states, it is not possible to specify general serviceability limits that are applicable to all structures. The *Specification* does not contain explicit requirements, however, guidance is generally provided by the applicable building code. In the absence of specific criteria, guidelines may be found in Fisher and West (1990), Ellingwood (1989), Murray (1991), AISC (2005) and ATC (1999).

A9 Referenced Documents

Other specifications and standards to which the *Specification* makes references to have been listed and updated in *Specification* Section A9 to provide the effective dates of these standards at the time of approval of this *Specification*.

Additional references which the designer may use for related information are listed at the end of the *Commentary.*

B. ELEMENTS

In cold-formed steel construction, individual elements of steel structural members are thin and the width-to-thickness ratios are large as compared with hot-rolled steel shapes. These thin elements may buckle locally at a stress level lower than the yield stress of steel when they are subjected to compression in flexural bending, axial compression, shear, or bearing. Figure C-B-1 illustrates some local buckling patterns of certain beams and columns (Yu, 2000).

Because local buckling of individual elements of cold-formed steel sections is a major design criterion, the design of such members should provide sufficient safety against the failure by local instability with due consideration given to the postbuckling strength of structural components. Chapter B of the *Specification* contains the design requirements for width-to-thickness ratios and the design equations for determining the effective widths of stiffened compression elements, unstiffened compression elements, elements with edge stiffeners or intermediate stiffeners, and beam webs. The design provisions are provided for the use of stiffeners in *Specification* Section C3.7 for flexural members.

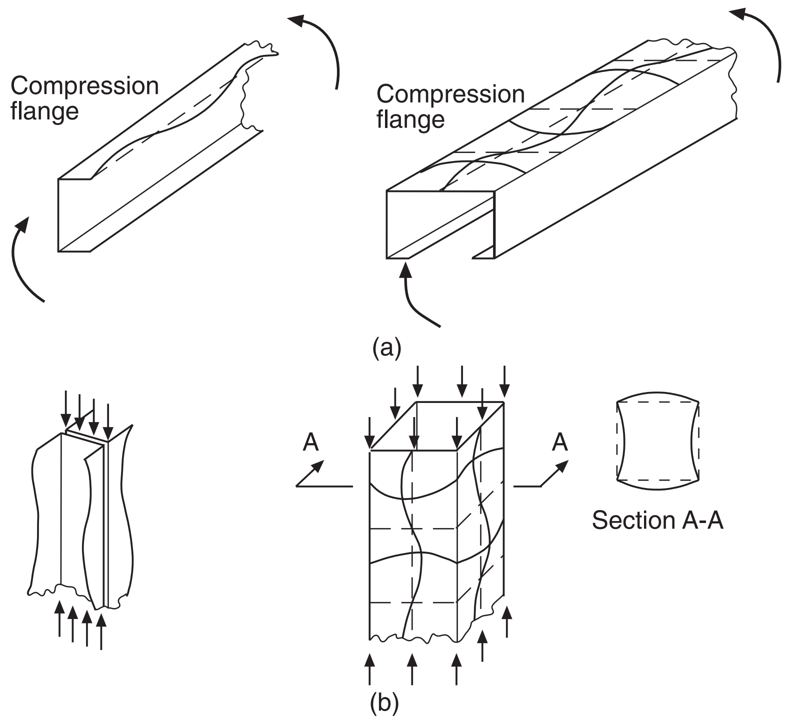


Figure C-B-1 Local Buckling of Compression Elements
(a) beams, (b) columns

B1 Dimensional Limits and Considerations

B1.1 Flange Flat-Width-to-Thickness Considerations

(a) Maximum Flat-Width-to-Thickness Ratios

Section B1.1 (a) of the *Specification* contains limitations on permissible flat-width-to-thickness ratios of compression elements. To some extent, these limitations are arbitrary. They do, however, reflect a long time experience and are intended to delimit practical ranges (Winter, 1970).

The limitation to a maximum w/t of 60 for the compression flanges having one longitudinal edge connected to a web and the other edge is stiffened by a simple lip is based on the fact that if the w/t ratio of such a flange exceeds 60, a simple lip with a relatively large depth would be required to stiffen the flange (Winter, 1970). The local instability of the lip would necessitate a reduction of the bending capacity to prevent premature buckling of the stiffening lip. This is the reason why the w/t ratio is limited to 60 for stiffened compression elements having one longitudinal edge connected to a web or flange element and the other is stiffened by a simple lip.

The limitation to w/t = 90 for compression flanges with any other kind of stiffeners indicates that thinner flanges with large w/t ratios are quite flexible and liable to be damaged in transport, handling and erection. The same is true for the limitation to w/t = 500 for stiffened compression elements with both longitudinal edges connected to other stiffened elements and for the limitation to w/t = 60 for unstiffened compression elements. The provision specifically states that wider flanges are not unsafe, but that when the w/t ratio of unstiffened flanges exceeds 30 and the w/t ratio of stiffened flanges exceeds 250, it is likely to develop noticeable deformation at the full design strength [resistance], without affecting the ability of the member to develop required strength [resistance]. In both cases the maximum w/t is set at twice that ratio at which first noticeable deformations are likely to appear, based on observations of such members under tests. These upper limits will generally keep such deformations to reasonable limits. In such cases where the limits are exceeded, tests in accordance with *Specification* Chapter F are required.

(b) Flange Curling

In beams which have unusually wide and thin, but stable flanges, (i.e., primarily tension flanges with large w/t ratios), there is a tendency for these flanges to curl under bending. That is, the portions of these flanges most remote from the web (edges of I-beams, center portions of flanges of box or hat beams) tend to deflect toward the neutral axis. An approximate, analytical treatment of this problem was given by Winter (1948b). Equation B1.1-1 of the *Specification* permits one to compute the maximum permissible flange width, w_f, for a given amount of flange curling, c_f.

It should be noted that Section B1.1(b) does not stipulate the amount of curling which can be regarded as tolerable, but an amount of curling in the order of 5 percent of the depth of the section is not excessive under usual conditions. In general, flange curling is not a critical factor to govern the flange width. However, when the appearance of the section is important, the out-of-plane distortion should be closely controlled in practice. Example of the AISI *Cold-Formed Steel Design Manual* (AISI, 2008) illustrates the design consideration for flange curling.

(c) Shear Lag Effects - Short Spans Supporting Concentrated Loads

For the beams of usual shapes, the normal stresses are induced in the flanges through shear stresses transferred from the web to the flange. These shear stresses produce shear strains in the flange which, for ordinary dimensions, have negligible effects. However, if flanges are unusually wide (relative to their length) these shear strains have the effect that the normal bending stresses in the flanges decrease with increasing distance from the web. This phenomenon is known as shear lag. It results in a non-uniform stress distribution across the width of the flange, similar to that in stiffened compression elements (see Section B2 of the *Commentary*), though for entirely different reasons. The simplest way of accounting for this stress variation in design is to replace the non-uniformly stressed flange of actual width w_f by one of reduced, effective width subject to uniform stress (Winter, 1970).

Theoretical analyses by various investigators have arrived at results which differ numerically (Roark, 1965). The provisions of Section B1.1(c) are based on the analysis and supporting experimental evidence obtained by detailed stress measurements on eleven beams (Winter, 1940). In fact, the values of effective widths in *Specification* Table B1.1(c) are taken directly from Curve A of Figure 4 of Winter (1940).

It will be noted that according to *Specification* Section B1.1(c), the use of a reduced width for stable, wide flanges is required only for concentrated load as shown in Figure C-B1.1-1. For uniform load it is seen from Curve B of the figure that the width reduction due to shear lag for any unrealistically large span-width ratios is so small as to be practically negligible.

The phenomenon of shear lag is of considerable consequence in naval architecture and aircraft design. However, in cold-formed steel construction it is infrequent that beams are so wide as to require significant reductions according to *Specification* Section B1.1(c). For design purpose, see Example of the AISI *Design Manual* (AISI, 2008).

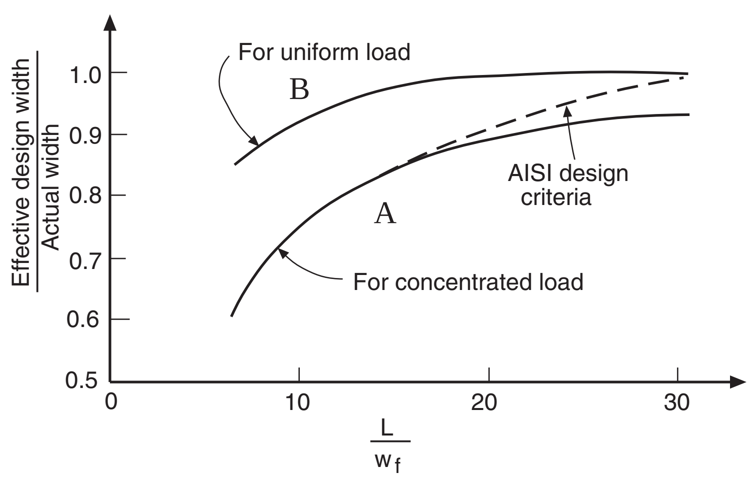


Figure C-B1.1-1 Analytical Curves for Determining Effective Width of Flange of Short Span Beams

B1.2 Maximum Web Depth-to-Thickness Ratios

Prior to 1980, the maximum web depth-to-thickness ratio, h/t, was limited to (a) 150 for cold-formed steel members with unreinforced webs and (b) 200 for members which are provided with adequate means of transmitting concentrated loads and/or reactions into the web. Based on the studies conducted at the University of Missouri-Rolla in the 1970s (LaBoube and Yu, 1978a, 1978b, and 1982b; Hetrakul and Yu, 1978 and 1980; Nguyen and Yu, 1978a and 1978b), the maximum h/t ratios were increased to (a) 200 for unreinforced webs, (b) 260 for using bearing stiffeners and (c) 300 for using bearing and intermediate stiffeners in the 1980 edition of the AISI *Specification*. These h/t limitations are the same as that used in the AISC Specification (AISC, 1989) for plate girders and are retained in the current edition of the

Specification. Because the definition for "h" was changed in the 1986 edition of the AISI *Specification* from the "clear distance between flanges" to the "depth of flat portion," measured along the plane of web, the prescribed maximum h/t ratio may appear to be more liberal. An unpublished study by LaBoube concluded that the present definition for h had negligible influence on the web strength [resistance].

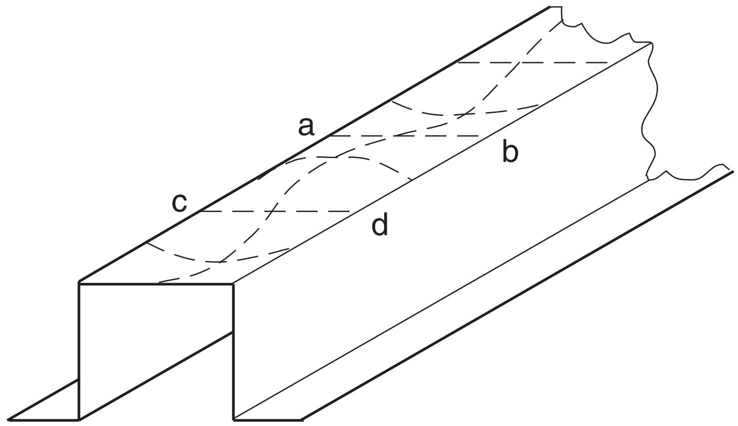


Figure C-B2-1 Local Buckling of Stiffened Compression Flange of Hat-Shaped Beam

B2 Effective Widths of Stiffened Elements

It is well known that the structural behavior and the load-carrying capacity of the stiffened compression element such as the compression flange of the hat section depend on the w/t ratio and the supporting condition along both longitudinal edges. If the w/t ratio is small, the stress in the compression flange can reach the yield stress of steel and the strength [resistance] of the compression element is governed by yielding. For the compression flange with large w/t ratios, local buckling (Figure C-B2-1) will occur at the following elastic critical buckling stress:

$$f_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)(w/t)^2} \tag{C-B2-1}$$

where

k = plate buckling coefficient (Table C-B2-1)
= 4 for stiffened compression elements supported by a web on each longitudinal edge
E = modulus of elasticity of steel
μ = Poisson's ratio = 0.3 for steel in the elastic range
w = flat width of the compression element
t = thickness of the compression element

When the elastic critical buckling stress computed according to Equation C-B2-1 exceeds the proportional limit of the steel, the compression element will buckle in the inelastic range (Yu, 2000).

Unlike one-dimensional structural members such as columns, stiffened compression elements will not collapse when the buckling stress is reached. An additional load can be carried by the element after buckling by means of a redistribution of stress. This phenomenon is known as post-buckling strength [resistance] of the compression elements and is most pronounced for stiffened compression elements with large w/t ratios. The mechanism of the

Table C-B2-1
Values of Plate Buckling Coefficients

Case	Boundary condition	Type of stress	Value of k for long plate
(a)	s.s. / s.s. s.s. / s.s.	Compression	4.0
(b)	Fixed / s.s. s.s. / Fixed	Compression	6.97
(c)	s.s. / s.s. s.s. / Free	Compression	0.425
(d)	Fixed / s.s. s.s. / Free	Compression	1.277
(e)	Fixed / s.s. s.s. / s.s.	Compression	5.42
(f)	s.s. / s.s. s.s. / s.s.	Shear	5.34
(g)	Fixed / Fixed Fixed / Fixed	Shear	8.98
(h)	s.s. / s.s. s.s. / s.s.	Bending	23.9
(i)	Fixed / Fixed Fixed / Fixed	Bending	41.8

post-buckling action of compression elements was discussed by Winter in previous editions of the AISI *Commentary* (Winter, 1970).

Imagine for simplicity a square plate uniformly compressed in one direction, with the unloaded edges simply supported. Since it is difficult to visualize the performance of such two-dimensional elements, the plate will be replaced by a model which is shown on Figure C-B2-2. It consists of a grid of longitudinal and transverse bars in which the material of the actual plate is thought to be concentrated. Since the plate is uniformly compressed, each of the longitudinal struts represents a column loaded by P/5, if P is the total load on the plate. As the load is gradually increased the compression stress in each of these struts will reach the critical column buckling value and all five struts will tend to buckle simultaneously. If these struts were simple columns, unsupported except at the ends, they would simultaneously collapse through unrestrained increasing lateral deflection. It is evident that this cannot occur in the grid model of the plate. Indeed, as soon as the longitudinal struts start deflecting at their buckling stress, the transverse bars, which are connected to them must stretch like ties in order to accommodate the imposed deflection. Like any structural material, they resist stretch and, thereby, have a restraining effect on the deflections of the longitudinal struts.

The tension forces in the horizontal bars of the grid model correspond to the so-called membrane stresses in a real plate. These stresses, just as in the grid model, come into play as soon as the compression stresses begin to cause buckling waves. They consist mostly of transverse tension, but also of some shear stresses, and they counteract increasing wave deflections, i.e. they tend to stabilize the plate against further buckling under the applied increasing longitudinal compression. Hence, the resulting behavior of the model is as follows: (a) there is no collapse by unrestrained deflections, as in unsupported columns, and (b) the various struts will deflect unequal amounts, those nearest the supported edges being held almost straight by the ties, those nearest the center being able to deflect most.

In consequence of (a), the model will not collapse and fail when its buckling stress (Equation C-B2-1) is reached; in contrast to columns it will merely develop slight deflections but will continue to carry increasing load. In consequence of (b), the struts (strips of the plate) closest to the center, which deflect most, "get away from the load," and hardly participate in carrying any further load increases. These center strips may in fact, even transfer part of their pre-buckling load to their neighbors. The struts (or strips) closest to the edges, held straight by the ties, continue to resist increasing load with hardly any increasing deflection. For the plate, this means that the hitherto uniformly distributed compression stress re-distributes itself in a manner shown on Figure C-B2-3, the stresses being largest at the edges and smallest in the center. With further increase in load this non-uniformity increases further, as also shown on Figure C-B2-3. The plate fails, i.e., refuses to carry any further load increases, only when the most highly stressed strips, near the supported edges, begin to yield, i.e., when the compression stress f_{max} reaches the yield stress F_y.

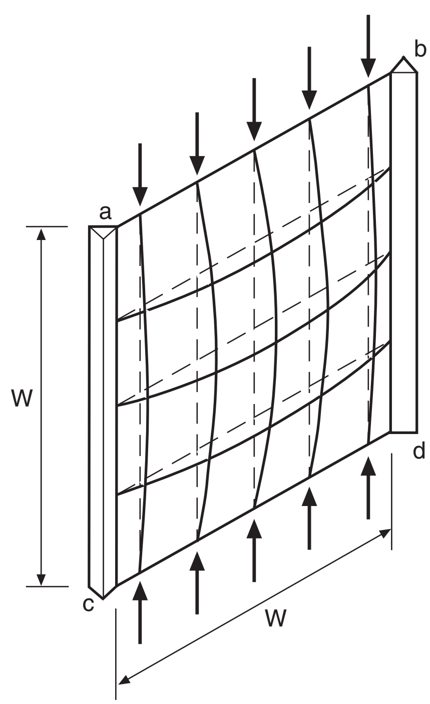


Figure C-B2-2 Postbuckling Strength [Resistance] Model

This postbuckling strength [resistance] of plates was discovered experimentally in 1928, and an approximate theory of it was first given by Th. v. Karman in 1932 (Bleich, 1952). It has been used in aircraft design ever since. A graphic illustration of the phenomenon of postbuckling strength [resistance] can be found in the series of photographs on Figure 7 of Winter (1959b).

The model of Figure C-B2-2 is representative of the behavior of a compression element supported along both longitudinal edges, as the flange in Figure C-B2-1. In fact, such elements buckle into approximately square waves.

In order to utilize the postbuckling strength [resistance] of the stiffened compression element for design purposes, the AISI *Specification* has used the *effective design width* approach to determine the sectional properties since 1946. In Section B2 of the present *Specification*, design equations for computing the effective widths are provided for the following four cases: (1) uniformly compressed stiffened elements, (2) uniformly compressed stiffened elements with circular or noncircular holes, (3) webs and other stiffened elements with stress gradient, (4) unstiffened elements with uniform or gradient stress, and (5) C-section webs with holes under stress gradient. The background information on various design requirements is discussed in subsequent sections.

B2.1 Uniformly Compressed Stiffened Elements

(a) *Effective Width for Strength [Resistance] Determination*

In the "effective design width" approach, instead of considering the nonuniform distribution of stress over the entire width of the plate *w*, it is assumed that the total load is carried by a fictitious effective width *b*, subject to a uniformly distributed stress equal to the edge stress f_{max}, as shown in Figure C-B2-3. The width b is selected so that the area under the curve of the actual nonuniform stress distribution is equal to the sum of the two parts of the equivalent rectangular shaded area with a total width b and an intensity of stress equal to the edge stress f_{max}.

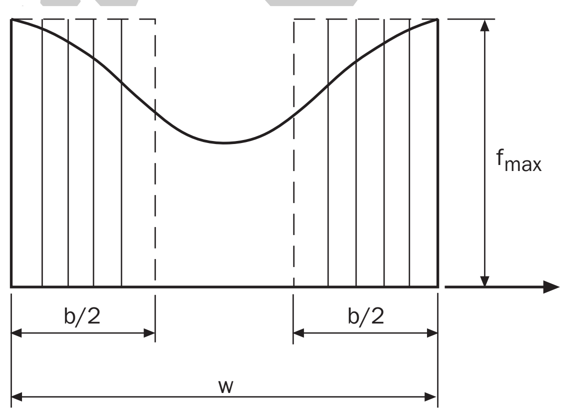


Figure C-B2-3 Stress Distribution in Stiffened Compression Elements

Based on the concept of "effective width" introduced by von Karman et al. (von Karman, Sechler and Donnell, 1932) and the extensive investigation on light-gage, cold-formed steel sections at Cornell University, the following equation was developed by Winter in 1946 for determining the effective width b for stiffened compression elements simply supported along both longitudinal edges:

$$b = 1.9t\sqrt{\frac{E}{f_{max}}}\left[1 - 0.475\left(\frac{t}{w}\right)\sqrt{\frac{E}{f_{max}}}\right] \quad \text{(C-B2.1-1)}$$

The above equation can be written in terms of the ratio of F_{cr}/f_{max} as follows:

$$\frac{b}{w} = \sqrt{\frac{F_{cr}}{f_{max}}}\left(1 - 0.25\sqrt{\frac{F_{cr}}{f_{max}}}\right) \quad \text{(C-B2.1-2)}$$

where F_{cr} is the critical elastic buckling stress of a plate, and is expressed in Equation C-B2-1.

Thus, the effective width expression (e.g., C-B2.1-1) provides a prediction of the nominal strength [resistance] based only on the critical elastic buckling stress and the applied stress of the plate. During the period from 1946 to 1968, the AISI design provision for the determination of the effective design width was based on Equation C-B2.1-1. A long-time accumulated experience has indicated that a more realistic equation, as shown below may be used for the determination of the effective width b (Winter, 1970):

$$b = 1.9t\sqrt{\frac{E}{f_{max}}}\left[1 - 0.415\left(\frac{t}{w}\right)\sqrt{\frac{E}{f_{max}}}\right] \quad \text{(C-B2.1-3)}$$

The correlation between the test data on stiffened compression elements and Equation C-B2.1-3 is illustrated by Yu (2000).

It should be noted that Equation C-B2.1-3 may also be rewritten in terms of the F_{cr}/f_{max} ratio as follows:

$$\frac{b}{w} = \sqrt{\frac{F_{cr}}{f_{max}}}\left(1 - 0.22\sqrt{\frac{F_{cr}}{f_{max}}}\right) \quad \text{(C-B2.1-4)}$$

Therefore, the effective width, b, can be determined as

$$b = \rho w \quad \text{(C-B2.1-5)}$$

where ρ = reduction factor

$$= (1 - 0.22/\sqrt{f_{max}/F_{cr}})/\sqrt{f_{max}/F_{cr}} = (1 - 0.22/\lambda)/\lambda \le 1 \quad \text{(C-B2.1-6)}$$

In Equation C-B2.1-6, λ is a slenderness factor determined below.

$$\lambda = \sqrt{f_{max}/F_{cr}} \quad \text{(C-B2.1-7)}$$

Figure C-B2.1-1 shows the relationship between ρ and λ. It can be seen that when $\lambda \le 0.673$, $\rho = 1.0$.

Based on Equations C-B2.1-5 through C-B2.1-7 and the unified approach proposed by Pekoz (1986b and 1986c), the 1986 edition of the AISI *Specification* adopted the nondimensional format in Section B2.1 for determining the effective design width, b, for uniformly compressed stiffened elements. The same design equations were used in the 1996 edition of the AISI *Specification* and was retained in this edition of the *North American Specification*. For design examples, see Part I of the AISI *Design Manual* (AISI, 2008).

(b) Effective Width for Serviceability Determination

The effective design width equations discussed above for strength [resistance] determination can also be used to obtain a *conservative* effective width, b_d, for serviceability determination. It is included in Section B2.1(b) of the *Specification* as Procedure I.

For stiffened compression elements supported by a web on each longitudinal edge, a study conducted by Weng and Pekoz (1986) indicated that Equations B2.1-8 through B2.1-10 of the *Specification* can yield a *more accurate estimate* of the effective width, b_d, for serviceability. These equations are given in Procedure II for additional design information. The design engineer has the option of using one of the two procedures for determining the effective width to be used for serviceability determination.

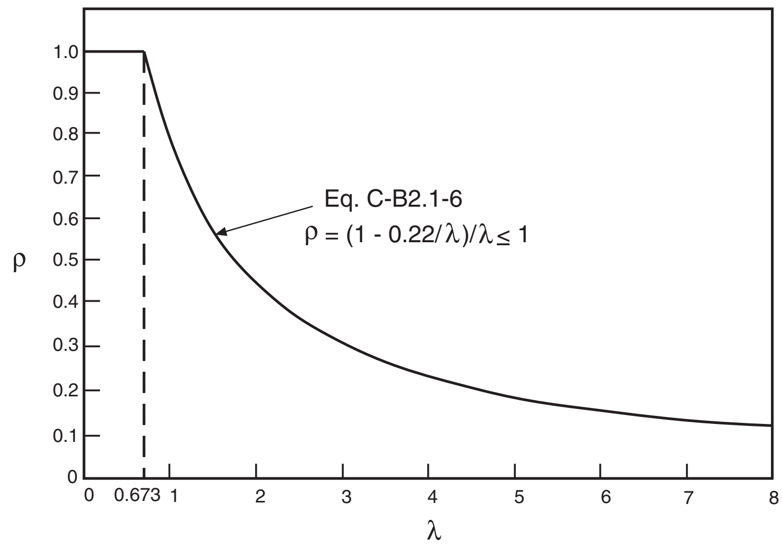


Figure C-B2.1-1 Reduction Factor, ρ, vs. Slenderness Factor, λ

B2.2 Uniformly Compressed Stiffened Elements with Circular or Non-Circular Holes

In cold-formed steel structural members, holes are sometimes provided in webs and/or flanges of beams and columns for duct work, piping, and other construction purposes. The presence of such holes may result in a reduction of the strength [resistance] of individual component elements and the overall strength [resistance] and stiffness of the members depending on the size, shape, and arrangement of holes, the geometric configuration of the cross section, and the mechanical properties of the material.

The exact analysis and the design of steel sections having perforations are complex, particularly when the shapes and the arrangement of holes are unusual. The limited design provisions included in Section B2.2 of the *Specification* for uniformly compressed stiffened elements with circular holes are based on a study conducted by Ortiz-Colberg and Pekoz at Cornell University (Ortiz-Colberg and Pekoz, 1981). For additional information on the structural behavior of perforated elements, see Yu and Davis (1973a) and Yu (2000).

In 2004, the *Specification* Equation B2.2-2 was revised to provide continuity at $\lambda = 0.673$.

In 2007, the provisions for non-circular holes were moved from *Specification* Section D4 to Section B2.2. Within the limitations stated for the size and spacing of perforations and section depth, the provisions were deemed appropriate for members with uniformly compressed stiffened elements, not just wall studs. The validity of this approach for C-sections wall studs was verified in a Cornell University project on wall studs reported by Miller and Pekoz (1989 and 1994). The limitations included in *Specification* Section B2.2 for the size and spacing of perforations and the depth of studs are based on the parameters used in the test program.

Although Figure B2.2-1 in the *Specification* shows a hole centered within the flat width, w, holes not centered within w are allowed. For such a case, the unstiffened strip, c, and resulting effective width, b, must be calculated separately for the strips on each side of the hole. For sections with perforations, which do not meet these limits, the effective area, A_e, can be determined by stub column tests.

The geometric limitations (w/t, etc.) and hole size for the circular and non-circular hole provisions in *Specification* Section B2.2 are not consistent with one another. This anomaly in the limitations is due to the differing scopes of the test programs that serve as the basis for these effective width equations. Ongoing research on perforations will provide a consistent, compatible design methodology in the future. The provisions for non-circular holes generally give a more conservative prediction of the effective width than the provisions for circular holes, as long as $d_h/w < 0.4$.

B2.3 Webs and other Stiffened Elements under Stress Gradient

When a beam is subjected to bending moment, the compression portion of the web may buckle due to the compressive stress caused by bending. The theoretical critical buckling stress for a flat rectangular plate under pure bending can be determined by Equation C-B2-1, except that the depth-to-thickness ratio, h/t, is substituted for the width-to-thickness ratio, w/t, and the plate buckling coefficient, k, is equal to 23.9 for simple supports as listed in Table C-B2-1.

Prior to 1986, the design of cold-formed steel beam webs was based on the full web depth with the allowable bending stress specified in the AISI *Specification*. In order to unify the design methods for web elements and compression flanges, the "effective design depth" approach was adopted in the 1986 edition of the AISI *Specification* on the basis of the studies made by Pekoz (1986b), Cohen and Pekoz (1987). This is a different approach as compared with the past practice of using a full area of the web element in conjunction with a reduced stress to account for local buckling and postbuckling strength (LaBoube and Yu, 1982b; Yu, 1985).

Prior to 2001, the b_1 and b_2 expressions used in the AISI *Specification* for the effective width of webs (Equations B2.3-3 through B2.3-5) implicitly assumed that the flange provided beneficial restraint to the web. Collected data (Cohen and Peköz (1987), Elhouar and Murray (1985), Ellifritt et al (1997), Hancock et al (1996), LaBoube and Yu (1978), Moreyra and Peköz (1993), Rogers and Schuster (1995), Schardt and Schrade (1982), Schuster (1992), Shan et al (1994), and Willis and Wallace (1990) as summarized in Schafer and Peköz (1999)) on flexural tests of C's and Z's indicate that *Specification* equations B2.3-3 through B2.3-5 can be unconservative if the overall web width (h_o) to overall flange width (b_o) ratio exceeds 4. Consequently, in 2001, in the absence of a comprehensive method for handling local web and flange interaction, the *North American Specification* adopted a two-part approach for the effective width of webs: an additional set of alternative expressions (Eqs B2.3-6 and B2.3-7), originally developed by Cohen and Pekoz (1987) were adopted for $h_o/b_o > 4$; while the expressions adopted in the 1986 edition of the AISI *Specification* (Equations B2.3-3 through B2.3-5) remain for $h_o/b_o \le 4$. For flexural members with local buckling in the web, the effect of these changes is that the strengths [resistances] will be somewhat lower when $h_o/b_o > 4$ compared with the 1996 AISI *Specification* (AISI, 1996). When compared with the CSA S136

(CSA, 1994) *Standard*, there are only minor changes for members with $h_o/b_o > 4$, but an increase in strength [resistance] will be experienced when $h_o/b_o \leq 4$.

It should be noted that in the *North American Specification*, the stress ratio ψ is defined as an absolute value. As a result, some signs for ψ have been changed in *Specification* Equations B2.3-2, B2.3-3, B2.3-6 and B2.3-7 as compared with the 1996 edition of the AISI *Specification* (AISI, 1996).

B2.4 C-Section Webs with Holes under Stress Gradient

Studies of the behavior of web elements with holes conducted at the University of Missouri-Rolla (UMR) serve as the basis for the design recommendations for bending alone, shear, web crippling, combinations of bending and shear, and bending and web crippling (Shan et al., 1994; Langan et al., 1994; Uphoff, 1996; Deshmukh, 1996). The *Specification* considers a hole to be any flat punched opening in the web without any edge stiffened openings.

The UMR design recommendations for a perforated web with stress gradient are based on the tests of full-scale C-section beams having h/t ratios as large as 200 and d_h/h ratios as large as 0.74. The test program considered only stud and joist industry standard web holes. These holes were rectangular with fillet corners, punched during the rolling process. For non-circular holes, the corner radii recommendation was adopted to avoid the potential of high stress concentration at the corners of a hole. Webs with circular holes and a stress gradient were not tested, however, the provisions are conservatively extended to cover this case. Other shaped holes must be evaluated by the virtual hole method described below, by test, or by other provisions of the *Specification*. The *Specification* is not intended to cover cross sections having repetitive 1/2 in. diameter holes.

Based on the study by Shan et al. (1994), it was determined that the nominal bending strength [resistance] of a C-section with a web hole is unaffected when $d_h/h < 0.38$. For situations where the $d_h/h \geq 0.38$, the effective depth of the web can be determined by treating the flat portion of the remaining web that is in compression as an unstiffened compression element.

Although these provisions are based on tests of singly-symmetric C-sections having the web hole centered at mid-depth of the section, the provisions may be conservatively applied to sections for which the full unreduced compression region of the web is less than the tension region. However, for cross sections having a compression region greater than the tension region, the web strength [resistance] must be determined by test in accordance with Section F1.

The provisions for circular and non-circular holes also apply to any hole pattern that fits within an equivalent virtual hole. For example, Figure C-B2.4-1 illustrates the L_h and d_h that may be used for a multiple hole pattern that fits within a non-circular virtual hole. Figure C-B2.4-2 illustrates the d_h that may be used for a rectangular hole that exceeds the 2.5 in. (64 mm) by 4.5 in. (114 mm) limit but still fits within an allowed circular virtual hole. For each case, the design provisions apply to the geometry of the virtual hole, not the actual hole or holes.

The effects of holes on shear strength [resistance] and web crippling strength [resistance] of C-section webs are discussed in Sections C3.2.2 and C3.4.2 of the *Commentary*, respectively.

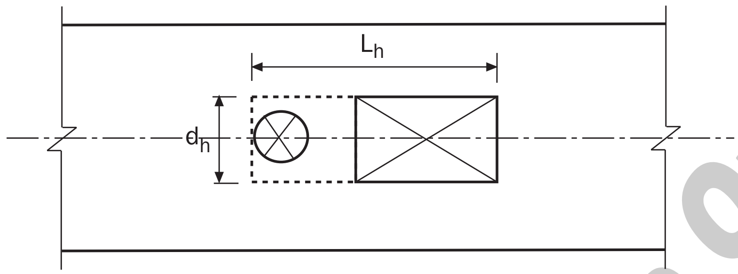


Figure C-B2.4-1 Virtual Hole Method for Multiple Openings

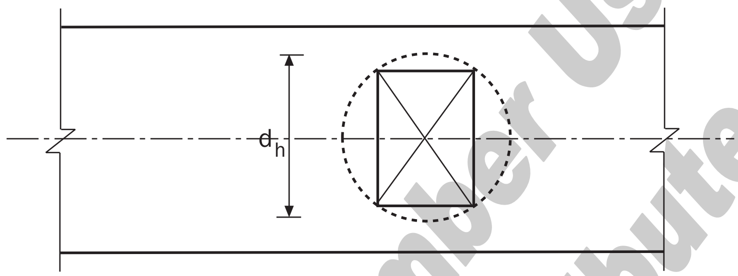


Figure C-B2.4-2 Virtual Hole Method for Opening Exceeding Limit

B3 Effective Widths of Unstiffened Elements

Similar to stiffened compression elements, the stress in the unstiffened compression elements can reach to the yield stress of steel if the w/t ratio is small. Because the unstiffened element has one longitudinal edge supported by the web and the other edge is free, the limiting width-to-thickness ratio of unstiffened elements is much less than that for stiffened elements.

When the w/t ratio of the unstiffened element is large, local buckling (Figure C-B3-1) will occur at the elastic critical stress determined by Equation C-B2-1 with a value of k=0.43. This buckling coefficient is listed in Table C-B2-1 for case (c). For the intermediate range of w/t ratios, the unstiffened element will buckle in the inelastic range. Figure C-B3-2 shows the relationship between the maximum stress for unstiffened compression elements and the w/t ratio, in which Line A is the yield stress of steel, Line B represents the inelastic buckling stress, Curves C and D illustrate the elastic buckling stress. The equations for Curves A, B, C, and D have been developed from previous experimental and analytical investigations and used for determining the allowable design stresses in the AISI *Specification* up to 1986 (Winter, 1970; Yu, 2000). Also shown in Figure C-B3-2 is Curve E, which represents the maximum stress on the basis of the postbuckling strength of the unstiffened element. The correlation between some test data on unstiffened elements and the predicted maximum stresses is shown in Figure C-B3-3 (Yu, 2000).

Prior to 1986, it had been a general practice to design cold-formed steel members with unstiffened flanges by using the allowable stress design approach. The effective width equation was not used in earlier editions of the AISI *Specification* due to lack of extensive experimental verification and the concern for excessive out-of-plane distortions under service loads.

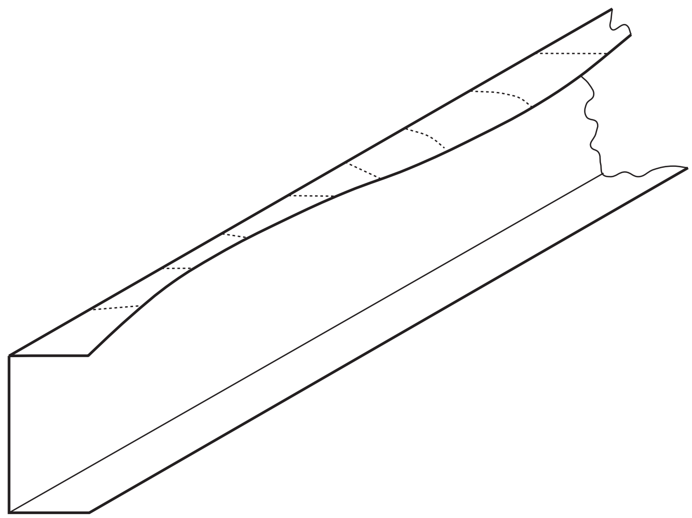


Figure C-B3-1 Local Buckling of Unstiffened Compression Flange

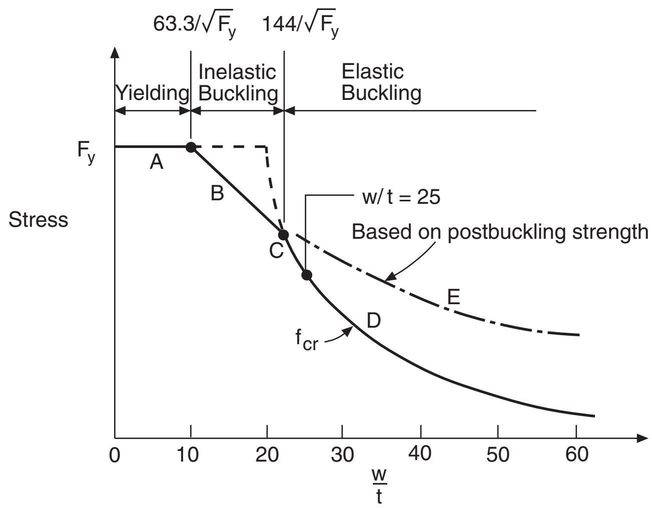


Figure C-B3-2 Maximum Stress for Unstiffened Compression Elements

In the 1970s, the applicability of the effective width concept to unstiffened elements under uniform compression was studied in detail by Kalyanaraman, Pekoz, and Winter at Cornell University (Kalyanaraman, Pekoz, and Winter, 1977; Kalyanaraman and Pekoz, 1978). The evaluation of the test data using k=0.43 was presented and summarized by Pekoz in the AISI report (Pekoz, 1986b), which indicates that Equation C-B2.1-6 developed for stiffened compression elements gives a conservative lower bound to the test results of unstiffened compression elements. In addition to the strength determination, the same study also investigated the out-of-plane deformations in unstiffened elements. The results of theoretical calculations and the test results on the sections having unstiffened elements with w/t=60 were presented by Pekoz in the same report. It was found that the maximum amplitude of the out-of-plane deformation at failure can be twice the thickness as the w/t ratio approaches 60. However, the deformations are significantly less under the service loads. Based on the above reasons and justifications, the effective design width approach was adopted for the first time in Section B3 of the 1986 AISI *Specification* for the design of cold-formed steel members having unstiffened compression elements.

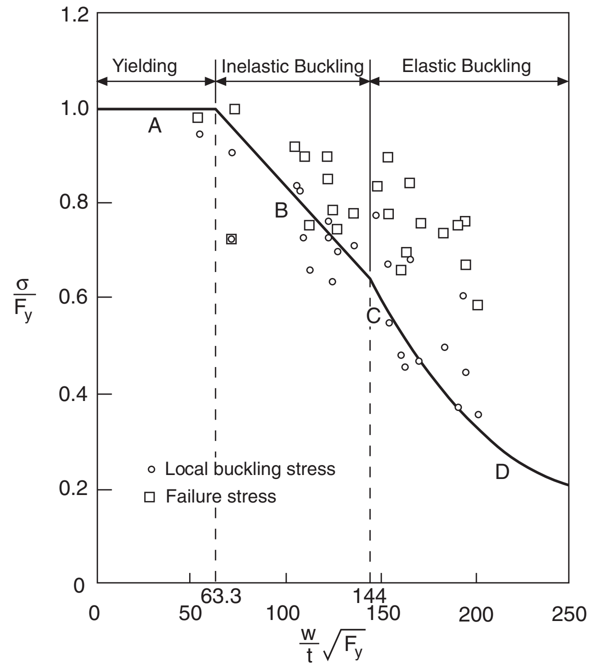


Figure C-B3-3 Correlation between Test Data and Predicted Maximum Stress

B3.1 Uniformly Compressed Unstiffened Elements

In the present *Specification*, it is specified that the effective widths, *b*, of uniformly compressed unstiffened elements can be determined in accordance with Section B2.1(a) of the *Specification* with the exception that the buckling coefficient k is taken as 0.43. This is a theoretical value for long plates. See case (c) in Table C-B2-1. For serviceability determination, the effective widths of uniformly compressed unstiffened elements can only be determined according to Procedure I of Section B2.1(b) of the *Specification*, because Procedure II was developed only for stiffened compression elements. See Part I of the AISI *Design Manual* for design examples (AISI, 2008).

B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

In concentrically loaded compression members and in flexural members where the unstiffened compression element is parallel to the neutral axis, the stress distribution is uniform prior to local buckling. However, when edge stiffeners of the compression element are present, the compressive stress in the edge stiffener is not uniform but varies in proportion to the distance from the neutral axis. The unstiffened element (the edge stiffener) in this case has compressive stress applied at both longitudinal edges. The unstiffened element of a section may also be subjected to stress gradients causing tension at one longitudinal edge and compression at the other longitudinal edge. This can occur in I-sections, plain channel sections and angle sections in minor axis bending.

Previous to the 2001 edition of the *Specification*, unstiffened elements with stress gradient were designed using the Winter effective width equation (Equation C-B2.1-4) and k=0.43. In 2004, Section B3.2 of the *Specification* adopted the effective width method for unstiffened

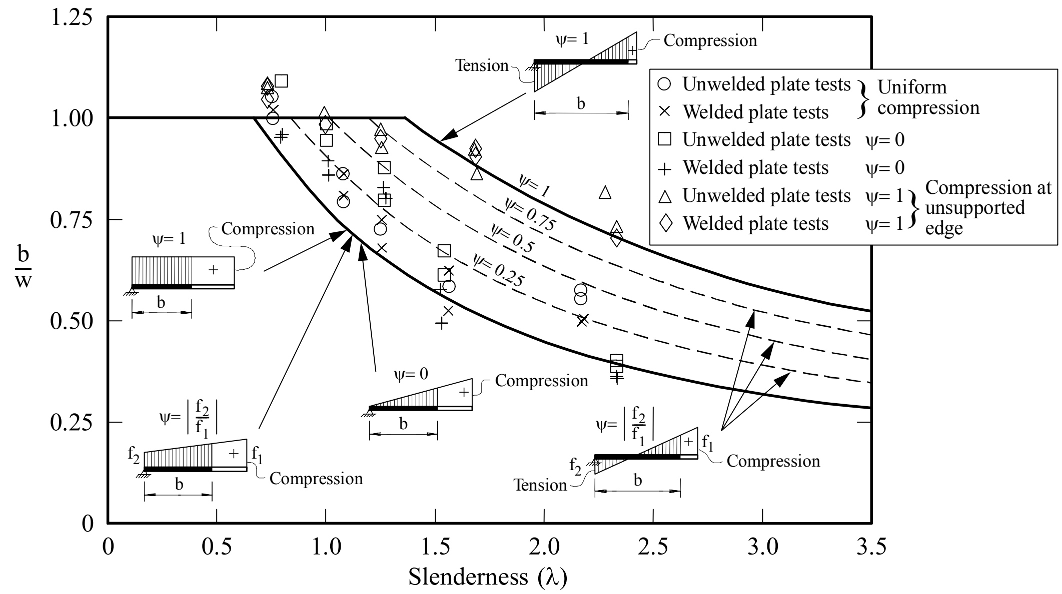


Figure C-B3.2-1 Effective Width vs. Plate Slenderness

elements with stress gradient proposed by Bambach and Rasmussen (2002a, 2002b and 2002c), based on an extensive experimental investigation of unstiffened plates tested as isolated elements in combined compression and bending. The effective width, b, (measured from the supported edge) of unstiffened elements with stress gradient causing compression at both longitudinal edges, is calculated using the Winter equation. For unstiffened elements with stress gradients causing tension at one longitudinal edge and compression at the other longitudinal edge, modified Winter equations are specified when tension exists at either the supported or the unsupported edges. The effective width equations apply to any unstiffened element under stress gradient, and are not restricted to particular cross-sections. Figure C-B3.2-1 demonstrates how the effective width of an unstiffened element increases as the stress at the supported edge changes from compression to tension. As shown in the figure, the effective width curve is independent of the stress ratio, ψ, when both edges are in compression. In this case, the effect of stress ratio is accounted for by the plate buckling coefficient, k, which varies with stress ratio and affects the slenderness, λ. When the supported edge is in tension and the unsupported edge is in compression, both the effective width curve and the plate buckling coefficient depend on the stress ratio, as per Equations B3.2-4 and B3.2-5 of the *Specification*.

Equations are provided for k, determined from the stress ratio, ψ, applied to the full element width such that iteration is not required, and k will usually be higher than 0.43. The equations for k are theoretical solutions for long plates assuming simple support along the longitudinal edge. A more accurate determination of k by accounting for interaction between adjoining elements is permitted for plain channels in minor axis bending (causing compression at the unsupported edge of the unstiffened element), based on research of plain channels in compression and bending by Yiu and Pekoz (2001).

The effective width is located adjacent to the supported edge for all stress ratios, including those producing tension at the unsupported edge. Research has found (Bambach and Rasmussen 2002a) that for the unsupported edge to be effective, tension must be applied over at least half of the width of the element starting at the unsupported edge. For less tension, the unsupported edge will buckle and the effective part of the element is located adjacent to the supported edge. Further, when tension is applied over half of the element or more starting at the unsupported edge, the compressed part of the element will remain effective for elements with w/t ratios less than the limits set out in Section B1.1 of the *Specification*.

The method for serviceability determination is based on the method used for stiffened elements with stress gradient in Section B2.3(b) of the *Specification*.

B4 Effective Width of Uniformly Compressed Elements with a Simple Lip Edge Stiffener

An edge stiffener is used to provide continuous support along a longitudinal edge of the compression flange to improve the buckling stress. In most cases, the edge stiffener takes the form of a simple lip. Other types of edge stiffeners can be beneficial and are also used for cold-formed steel members, but are not covered in *Specification* Section B4.

In order to provide necessary support for the compression element, the edge stiffener must possess sufficient rigidity. Otherwise it may buckle perpendicular to the plane of the element to be stiffened. As far as the design provisions are concerned, the 1980 and earlier editions of the AISI *Specification* included the requirements for the minimum moment of inertia of stiffeners to provide sufficient rigidity. When the size of the actual stiffener does not satisfy the required moment of inertia, the load-carrying capacity of the beam had to be determined either on the basis of a flat element disregarding the stiffener or through tests.

Both theoretical and experimental studies on the local stability of compression flanges stiffened by edge stiffeners have been carried out in the past. The design requirements included in Section B4 of the 1986 AISI *Specification* were based on the investigations on adequately stiffened and partially stiffened elements conducted by Desmond, Pekoz and Winter (1981a), with additional research work of Pekoz and Cohen (Pekoz, 1986b). These design provisions were developed on the basis of the critical buckling criterion and the postbuckling strength [resistance] criterion.

Specification Section B4 recognizes that the necessary stiffener rigidity depends upon the slenderness (w/t) of the plate element being stiffened. The interaction of the plate elements, as well as the degree of edge support, full or partial, is compensated for in the expressions for k, d_s, and A_s (Pekoz, 1986b).

In the 1996 edition of the AISI *Specification* (AISI, 1996), the design equations for buckling coefficient were changed for further clarity. The requirement of $140° \geq \theta \geq 40°$ for the applicability of these provisions was decided on an intuitive basis. For design examples, see Part I of the *Cold-Formed Steel Manual* (AISI, 2008).

Test data to verify the accuracy of the simple lip stiffener design was collected from a number of sources, both university and industry. These tests showed good correlation with the equations in *Specification* Section B4.

The 1996 *Commentary* provided a warning to the user that lip lengths with a d/t ratio greater than 14 may give unconservative results. Examination of available experimental data on both flexural members (Rogers and Schuster, 1996, Schafer and Pekoz, 1999) and compression members (Schafer, 2000) with edge stiffeners indicates that the *Specification* does not have an inherent problem for members with large d/t ratios. Existing experimental data covers d/t ratios as high as 35 for both flexural and compression members.

In 2001, Dinovitzer's expressions (Dinovitzer, et al., 1992) for n (Equation B4-11) were adopted, which eliminated a discontinuity that existed in the previous design expressions. The revised equation gives n =1/2 for w/t = 0.328S and n = 1/3 for w/t = S, in which S is also the maximum w/t ratio for a stiffened element to be fully effective.

In 2007, the expressions were limited to cover only simple lip edge stiffeners, as the previously employed expressions for complex lip stiffeners were found to be unconservative in comparison with rigorous nonlinear finite element analysis (Schafer, et al., 2006). Design of members with complex lips may be handled via the methods of *Specification* Appendix 1. In addition, the design provisions for the uniformly compressed elements with one intermediate stiffener were deleted in the 2007 edition of the *Specification* due to the fact that the effective width of such members can be determined in accordance with *Specification* Section B5.1.

B5 Effective Widths of Stiffened Elements with Single or Multiple Intermediate Stiffeners or Edge Stiffened Elements with Intermediate Stiffener(s)

B5.1 Effective Width of Uniformly Compressed Stiffened Elements with Single or Multiple Intermediate Stiffeners

The structural efficiency of a stiffened element always exceeds that of an unstiffened element with the same w/t ratio by a sizeable margin, except for low w/t ratios, for which the compression element is fully effective. When stiffened elements with large w/t ratios are used, the material is not employed economically inasmuch as an increasing proportion of the width of the compression element becomes ineffective. On the other hand, in many applications of cold-formed steel construction, such as panels and decks, maximum coverage is desired and, therefore, large w/t ratios are called for. In such cases, structural economy can be improved by providing intermediate stiffeners between webs.

The buckling behavior of rectangular plates with central stiffeners is discussed by Bulson (1969). For the design of cold-formed steel beams using intermediate stiffeners, the 1980 AISI *Specification* contained provisions for the minimum required moment of inertia, which was based on the assumption that an intermediate stiffener needed to be twice as rigid as an edge stiffener. In view of the fact that for some cases the design requirements for intermediate stiffeners included in the 1980 *Specification* could be unduly conservative (Pekoz, 1986b), the AISI design provisions were revised in 1986 according to Pekoz's research findings (Pekoz, 1986b and 1986c) and prior to 2007 could be found in Section B4.1 of the *Specification*. In 2007 the design of uniformly compressed elements with multiple or single intermediate stiffeners was merged. The multiple intermediate stiffener provisions were developed based on Pekoz's continuing research on intermediate stiffeners (Schafer and Pekoz 1998) and the finding that the method developed in B5.1 of the *Specification* for multiple intermediate stiffeners could provide the same reliability as the *Specification* Section B4.1 (AISI, 2001) method for single intermediate stiffeners (Yang and Schafer 2006).

Prior to 2001, the AISI *Specification* and the Canadian *Standard* provided different design provisions for determination of the effective widths of uniformly compressed stiffened elements with multiple intermediate stiffeners or edge stiffened elements with intermediate stiffeners. In the AISI *Specification*, the design requirements of Section B5 dealt with (1) the minimum moment of inertia of the intermediate stiffener, (2) the number of intermediate stiffeners considered to be effective, (3) the "equivalent element" of multiple-stiffened element having closely spaced intermediate stiffeners, (4) the effective width of sub-element with w/t > 60, and (5) the reduced area of stiffeners. In the Canadian *Standard*, a different design equation was used to determine the equivalent thickness.

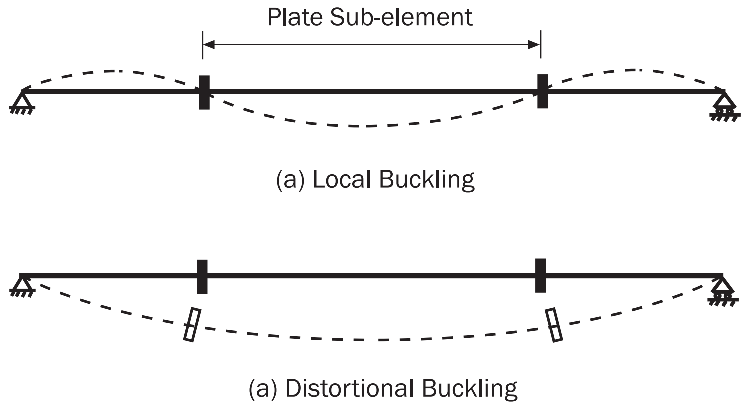


Figure C-B5.1-1 Local and Distortional Buckling of a Uniformly Compressed Element with Multiple Intermediate Stiffeners

In 2001, *Specification* Section B5.1 was revised to reflect recent research findings for flexural members with multiple intermediate stiffeners in the compression flange (Papazian et al. 1994, Schafer and Peköz 1998, Acharya and Schuster 1998). The method is based on determining the plate buckling coefficient for the two competing modes of buckling: local buckling, in which the stiffener does not move; and distortional buckling in which the stiffener buckles with the entire plate. See Figure C-B5.1-1. Experimental research shows that the distortional mode is prevalent for members with multiple intermediate stiffeners.

The reduction factor, ρ, is applied to the entire element (gross area of the element/thickness) instead of only the flat portions. Reducing the entire element to an effective width, which ignores the geometry of the stiffeners, for effective section property calculation allows distortional buckling to be treated consistently with the rest of the *Specification*, rather than as an "effective area" or other method. The resulting effective width must act at the centroid of the original element including the stiffeners. This insures that the neutral axis location for the member is unaffected by the use of the simple effective width, which replaces the more complicated geometry of the element with multiple intermediate stiffeners. One possible result of this approach is that the calculated effective width (b_e) may be greater than b_o. This may occur when ρ is near 1, and is due to the fact that b_e includes contributions from the stiffener area and b_o does not. As long as the calculated b_e is placed at the centroid of the entire element the use of $b_e > b_o$ is correct.

B5.2 Edge Stiffened Elements with Intermediate Stiffener(s)

The buckling modes for edge stiffened elements with one or more intermediate stiffeners include: local sub-element buckling, distortional buckling of the intermediate stiffener, and distortional buckling of edge stiffener, as shown in Figure C-B5.2-1. If the edge stiffened element is stocky ($b_o/t < 0.328S$) or the stiffener is large enough ($I_s > I_a$ and thus $k = 4$, per the rules of *Specification* Section B4) then the edge stiffened element performs as a stiffened element. In this case, effective width for local sub-element buckling and distortional buckling of the intermediate stiffener may be predicted by the rules of *Specification* Section B5.1.

However, an edge stiffened element does not have the same web rotational restraint as a stiffened element, therefore the constant R of *Specification* Section B5.1 is conservatively limited to be less than or equal to 1.0.

If the edge stiffened element is partially effective ($b_o/t > 0.328S$ and $I_s < I_a$ and thus $k < 4$, per the rules of *Specification* Section B4) then the intermediate stiffener(s) should be ignored and the provisions of *Specification* Section B4 followed. Elastic buckling analysis of the distortional mode for an edge stiffened element with intermediate stiffener(s) indicates that the effect of intermediate stiffener(s) on the distortional buckling stress is ±10 percent for practical intermediate and edge stiffener sizes.

When applying *Specification* Section B5.2 for effective width determination of edge stiffened elements with intermediate stiffeners, the effective width of the intermediately stiffened flange, b_e, is replaced by an equivalent flat section (as shown in *Specification* Figure B5.1-2). The edge stiffener should not be used in determining the centroid location of the equivalent flat effective width, b_e, for the intermediately stiffened flange.

Stub compression testing performed in 2003 demonstrates the adequacy of this approach (Yang and Hancock, 2003).

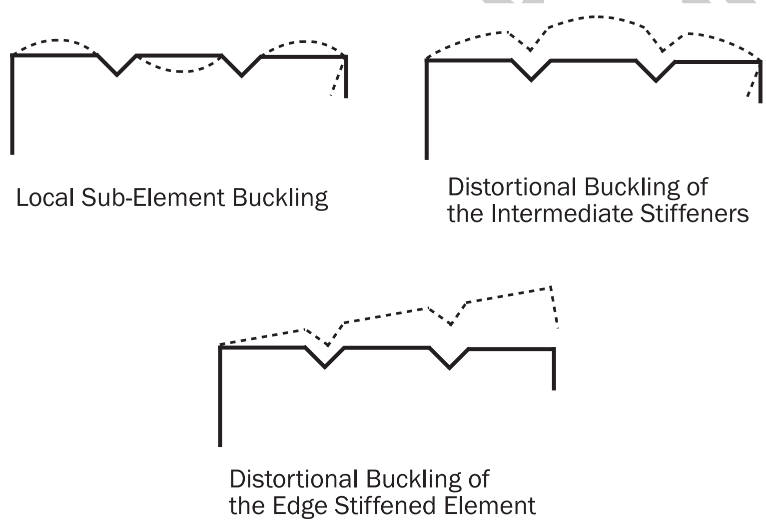


Figure C-B5.2-1 Buckling Modes in an Edge Stiffened Element with Intermediate Stiffeners

C. MEMBERS

This Chapter provides the design requirements for (a) tension members, (b) flexural members, (c) concentrically loaded compression members, and (d) members subjected to combined axial load and bending.

In 2007, the following design provisions were moved from Specification Chapter C, Members, to Section D6, Metal Roof and Wall Systems: (1) Flexural Members Having One Flange Through-Fastened to Deck or Sheathing, (2) Flexural Members Having One Flange Fastened to a Standing Seam Roof System, (3) Compression Members Having One Flange Through-Fastened to Deck or Sheathing, and (4) Strength [Resistance] of Standing Seam Panel System. For closed cylindrical tubular members the design provisions have been moved to new Section C3.1.3 for flexural members and new Section C4.1.5 for compression members.

In general, a common nominal strength [resistance] equation is provided in the *Specification* for a given limit state with a required safety factor (Ω) for Allowable Strength Design (ASD) and a resistance factor (ϕ) for Load and Resistance Factor design (LRFD) or Limit State Design (LSD). Design provisions that are applicable to a specific country are provided in the corresponding Appendix.

C1 Properties of Sections

The geometric properties of a member (i.e., area, moment of inertia, section modulus, radius of gyration, etc.) are evaluated using conventional methods of structural design. These properties are based upon either full cross-section dimensions, effective widths or net section, as applicable.

For the design of tension members, both gross and net sections are employed when computing the nominal tensile strength [resistance] of the axially loaded tension members.

For flexural members and axially loaded compression members, both full and effective dimensions are used to compute sectional properties. The full dimensions are used when calculating the critical load or moment, while the effective dimensions, evaluated at the stress corresponding to the critical load or moment, are used to calculate the nominal strength [resistance]. For serviceability consideration, the effective dimension should be determined for the compressive stress in the element corresponding to the service load. Pekoz (1986a and 1986b) discussed this concept in more detail.

Section 3 of Part I of the AISI *Design Manual* (AISI, 2008) deals with the calculation of sectional properties for C-sections, Z-sections, angles, hat sections, and decks.

C2 Tension Members

The design provisions of this section are given in Section C2 of the Appendices. The discussion for this section is provided in the *Commentary* on the corresponding Appendix.

☜A,B

C3 Flexural Members

For the design of cold-formed steel flexural members, consideration should be given to several design features: (a) bending strength [resistance] and serviceability, (b) shear strength [resistance] of webs and combined bending and shear, (c) web crippling strength [resistance] and combined bending and web crippling, and (d) bracing requirements. For some cases, special consideration should also be given to shear lag and flange curling due to the use of thin material. The design provisions for Items (a), (b) and (c) are provided in *Specification* Sections C3, and D6.1 and D6.2, while the requirements for lateral and stability bracing are given in *Specification* Sections D3 and D6.3. The treatments for flange curling and shear lag were discussed in Section B1.1(b) and (c) of the *Commentary*, respectively.

Example problems are given in Part II of the AISI *Cold-Formed Steel Design Manual* (AISI, 2008) for the design of flexural members.

C3.1 Bending

Bending strengths [resistances] of flexural members are differentiated according to whether or not the member is laterally braced. If such members are laterally supported, then they are proportioned according to the nominal section strength [resistance] *(Specification* Section C3.1.1). Since the distortional buckling has an intermediate buckling half wavelength, the distortional buckling still needs to be considered even for braced members. See the Direct Strength Method Design Guide (AISI, 2006) for detailed discussion and design examples. If they are laterally unbraced, then the limit state is lateral-torsional buckling (*Specification* Section C3.1.2). For C- or Z-sections with the tension flange attached to deck or sheathing and with compression flange laterally unbraced, the bending capacity is less than that of a fully braced member but greater than that of an unbraced member (*Specification* Section D6.1.1). For C- or Z-sections supporting a standing seam roof system under gravity or uplift loads, the bending capacity is greater than that of an unbraced member and may be equal to that of a fully braced member (*Specification* Section D6.1.2). Similarly, for standing seam roof systems, design provisions are provided in *Specification* Section D6.2.1 for evaluating the bending strength of the system based on tests. The governing nominal bending strength [resistance] is the smallest of the values determined from the applicable conditions.

C3.1.1 Nominal Section Strength [Resistance]

Specification Section C3.1.1 includes two design procedures for calculating the nominal section strength [resistance] of flexural members. Procedure I is based on Initiation of Yielding and Procedure II is based on Inelastic Reserve Capacity.

(a) *Procedure I - Based on Initiation of Yielding*

In Procedure I, the nominal moment, M_n, of the cross section is the effective yield moment, M_y, determined on the basis of the effective areas of flanges and the beam web. The effective width of the compression flange and the effective depth of the web can be computed from the design equations given in Chapter B of the *Specification*.

Similar to the design of hot-rolled steel shapes, the yield moment M_y of a cold-formed steel beam is defined as the moment at which an outer fiber (tension, compression, or both) first attains the yield stress of the steel. This is the maximum bending capacity to be used in elastic design. Figure C-C3.1.1-1 shows several types of stress distributions for yield moment based on different locations of the neutral axis. For balanced sections (Figure C-C3.1.1-1(a)) the outer fibers in the compression and tension flanges reach the yield stress at the same time. However, if the neutral axis is eccentrically located, as shown in Figures C-C3.1.1-1(b) and (c), the initial yielding takes place in the tension flange for case (b) and in the compression flange for case (c).

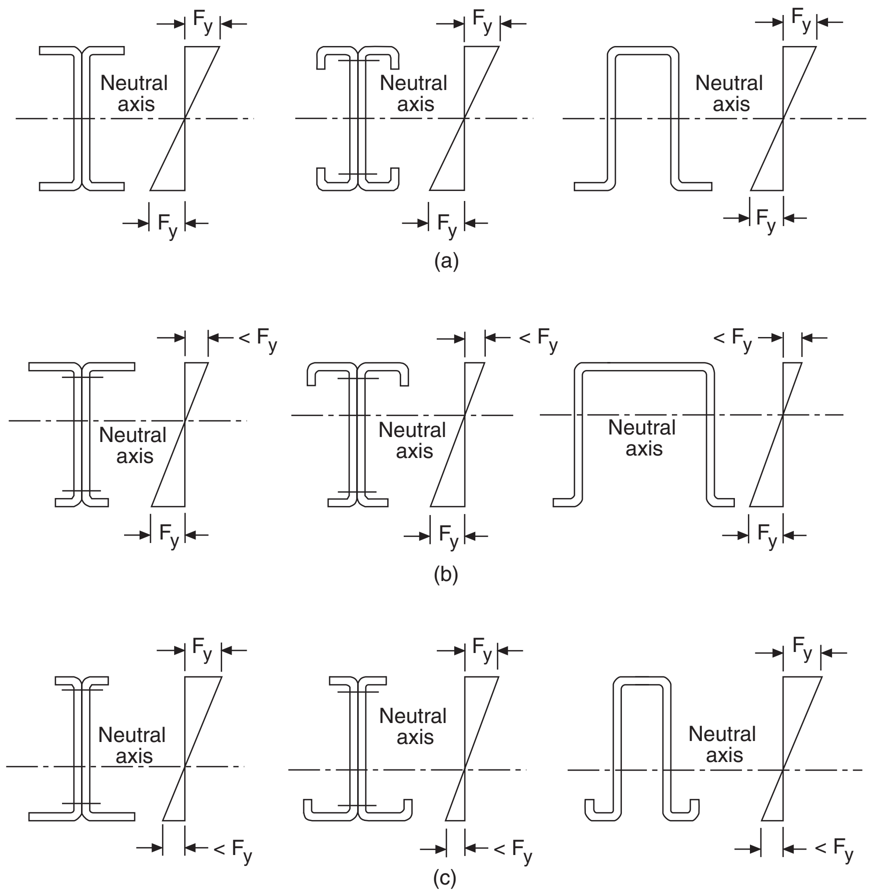


Figure C-C3.1.1-1 Stress Distribution for Yield Moment
(a) Balanced Sections, (b) Neutral Axis Close to Compression Flange,
(c) Neutral Axis Close to Tensions Flange

Accordingly, the nominal section strength [resistance] for *initiation of yielding* is calculated by using Equation C-C3.1.1-1:

$$M_n = S_e F_y \quad \text{(C-C3.1.1-1)}$$

where

F_y = design yield stress

S_e = elastic section modulus of the effective section calculated with the extreme compression or tension fiber at F_y.

For cold-formed steel design, S_e is usually computed by using one of the following two cases:

1. If the neutral axis is closer to the tension than to the compression flange, the maximum stress occurs in the compression flange, and therefore the plate slenderness ratio λ and the effective width of the compression flange are determined by the w/t ratio and f = F_y. Of course, this procedure is also applicable to those beams for which the neutral axis is located at the mid-depth of the section.
2. If the neutral axis is closer to the compression than to the tension flange, the maximum stress of F_y occurs in the tension flange. The stress in the compression flange depends on the location of the neutral axis, which is determined by the effective area of the section. The latter cannot be determined unless the compressive

stress is known. The closed-form solution of this type of design is possible but would be a very tedious and complex procedure. It is therefore customary to determine the sectional properties of the section by successive approximation.

For determining the design flexural strength [factored resistance], $\phi_b M_n$, by using the LRFD approach, slightly different resistance factors are used for the sections with stiffened or partially stiffened compression flanges and the sections with unstiffened compression flanges. These ϕ_b values were derived from the test results and a dead-to-live load ratio of 1/5. They provide the β values from 2.53 to 4.05 (AISI, 1991; Hsiao, Yu and Galambos, 1988a).

(b) Procedure II - Based on Inelastic Reserve Capacity

Prior to 1980, the inelastic reserve capacity of beams was not included in the AISI *Specification* because most cold-formed steel shapes have large width-to-thickness ratios that are considerably in excess of the limits required by plastic design.

In the 1970s and early 1980s, research work on the inelastic strength of cold-formed steel beams was carried out by Reck, Pekoz, Winter, and Yener at Cornell University (Reck, Pekoz and Winter, 1975; Yener and Pekoz, 1985a, 1985b). These studies showed that the inelastic reserve strength [resistance] of cold-formed steel beams due to partial plastification of the cross section and the moment redistribution of statically indeterminate beams can be significant for certain practical shapes. With proper care, this reserve strength [resistance] can be utilized to achieve more economical design of such members.

In order to utilize the available inelastic reserve strength [resistance] of certain cold-formed steel beams, design provisions based on the partial plastification of the cross section were added in the 1980 edition of the AISI *Specification*. The same provisions are retained in the 2001 and the 2007 editions of the *Specification*. According to Procedure II of Section C3.1.1(b) of the *Specification*, the nominal section strength [resistance], M_n, of those beams satisfying certain specific limitations can be determined on the basis of the inelastic reserve capacity with a limit of $1.25M_y$, where M_y is the effective yield moment. The ratio of M_n/M_y represents the inelastic reserve strength [resistance] of a beam cross section.

The nominal moment M_n is the maximum bending capacity of the beam by considering the inelastic reserve strength [resistance] through partial plastification of the cross section. The inelastic stress distribution in the cross section depends on the maximum strain in the compression flange, ε_{cu}. Based on the Cornell research work on hat sections having stiffened compression flanges (Reck, Pekoz and Winter, 1975), the AISI design provision limits the maximum compression strain to be $C_y\varepsilon_y$, where C_y is a compression strain factor determined by using the equations provided in *Specification* Section C3.1.1(b) (i) as shown in Figure C-C3.1.1-2.

On the basis of the maximum compression strain ε_{cu} allowed in the *Specification*, the neutral axis can be located by using Equation C-C3.1.1-2 and the nominal moment M_n can be determined by using Equation C-C3.1.1-3:

$$\int \sigma dA = 0 \tag{C-C3.1.1-2}$$

$$\int \sigma y dA = M_n \tag{C-C3.1.1-3}$$

where σ is the stress in the cross section.

The calculation of M_n based on inelastic reserve capacity is illustrated in Part I of the AISI *Cold-Formed Steel Design Manual* (AISI, 2008) and the textbook by Yu (2000).

In 2001, the shear force upper limit was clarified. The stress upper limit is $0.35F_y$ for ASD and $0.6F_y$ for LRFD and LSD in the *North American Specification*.

In 2004, additional *Specification* equations are provided in Section C3.1.1(b) for determining the nominal moment strength [resistance], M_n based on inelastic reserve capacity, for sections containing unstiffened compression elements under stress gradient. Based on research by Bambach and Rasmussen (2002b, 2002c) on I- and plain channel sections in minor axis bending, a compression strain factor C_y determines the maximum compressive strain on the unstiffened element of the section. The C_y values are dependent on the stress ratio ψ and slenderness ratio λ of the unstiffened element, determined in accordance with Section B3.2(a) of the *Specification*.

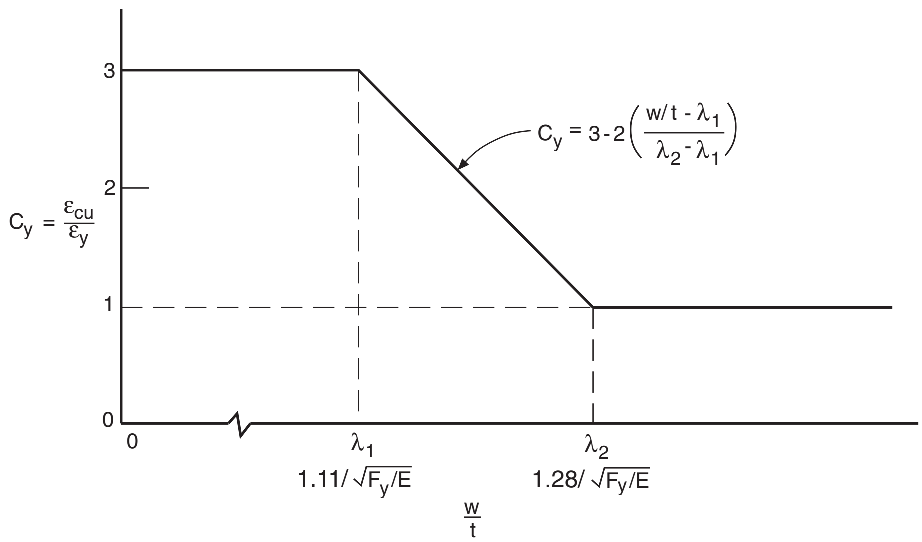


Figure C-C3.1.1-2 Factor C_y for Stiffened Compression Elements without Intermediate Stiffeners

C3.1.2 Lateral-Torsional Buckling Strength [Resistance]

The bending capacity of flexural members is not only governed by the strength [resistance] of the cross section, but can also be limited by the lateral-torsional buckling strength [resistance] of the member if braces are not adequately provided. The design provisions for determining the nominal lateral-torsional buckling strength [resistance] are given in *Specification* Section C3.1.2.1 for open cross section members and C3.1.2.2 for closed tubular members.

C3.1.2.1 Lateral-Torsional Buckling Strength [Resistance] for Open Cross Section Members

If a doubly-symmetric or singly-symmetric member in bending is laterally unbraced, it can fail in lateral-torsional buckling. For a beam having simply supported end conditions both laterally and torsionally, the elastic critical lateral-torsional buckling stress can be determined by Equation C-C3.1.2.1-1.

$$\sigma_{cr} = \frac{\pi}{LS_f}\sqrt{EI_yGJ\left(1+\frac{\pi^2EC_w}{GJL^2}\right)} \quad \text{(C-C3.1.2.1-1)}$$

For other than simply supported end conditions, Equation C-C3.1.2.1-1 can be generalized as given in Equation C-C3.1.2.1-1a (Galambos, 1998):

$$\sigma_{cr} = \frac{\pi}{(K_yL_y)S_f}\sqrt{EI_yGJ\left[1+\frac{\pi^2EC_w}{GJ(K_tL_t)^2}\right]} \quad \text{(C-C3.1.2.1-1a)}$$

In the above equation, K_y and K_t are effective length factors and L_y and L_t are unbraced lengths for bending about the y-axis and for twisting, respectively, E is the modulus of elasticity, G is the shear modulus, S_f is the elastic section modulus of the full unreduced section relative to the extreme compression fiber, I_y is the moment of inertia about the y-axis, C_w is the torsional warping constant, J is the Saint-Venant torsion constant, and L is the unbraced length.

For equal-flanged I-members with simply supported end conditions both laterally and torsionally, Equation C-C3.1.2.1-2 can be used to calculate the elastic critical buckling stress (Winter, 1947a; Yu, 2000):

$$\sigma_{cr} = \frac{\pi^2E}{2(L/d)^2}\sqrt{\left(\frac{I_y}{2I_x}\right)^2+\left(\frac{JI_y}{2(1+\mu)I_x{}^2}\right)\left(\frac{L}{\pi d}\right)^2} \quad \text{(C-C3.1.2.1-2)}$$

In Equation C-C3.1.2.1-2, the first term under the square root represents the lateral bending rigidity of the member, and the second term represents the Saint-Venant torsional rigidity. For thin-walled cold-formed steel sections, the first term usually exceeds the second term by a considerable margin.

For simply supported I-members with unequal flanges, the following equation has been derived by Winter for the lateral-torsional buckling stress (Winter, 1943):

$$\sigma_{cr} = \frac{\pi^2Ed}{2L^2S_f}\left(I_{yc} - I_{yt} + I_y\sqrt{1+\frac{4GJL^2}{\pi^2I_yEd^2}}\right) \quad \text{(C-C3.1.2.1-3)}$$

where I_{yc} and I_{yt} are the moments of inertia of the compression and tension portions of the full section, respectively, about the centroidal axis parallel to the web. Other symbols were defined previously. For equal-flange sections, $I_{yc} = I_{yt} = I_y/2$, Equations C-C3.1.2.1-2 and C-C3.1.2.1-3 are identical.

For other than simply supported end conditions, Equation C-C3.1.2.1-3 can be generalized as given in Equation C-C3.1.2.1-3a:

$$\sigma_{cr} = \frac{\pi^2Ed}{2(K_yL_y)^2S_f}\left(I_{yc} - I_{yt} + I_y\sqrt{1+\frac{4GJ(K_tL_t)^2}{\pi^2I_yEd^2}}\right) \quad \text{(C-C3.1.2.1-3a)}$$

In Equation C-C3.1.2.1-3a, the second term under the square root represents the Saint-Venant torsional rigidity, which can be neglected without any loss in economy. Therefore, Equation C-C3.1.2.1-3a can be simplified as shown in Equation C-C3.1.2.1-4 by considering $I_y = I_{yc} + I_{yt}$ and neglecting the term $4GJ(K_tL_t)^2/(\pi^2I_yEd^2)$:

$$\sigma_{cr} = \frac{\pi^2 E d I_{yc}}{(K_y L_y)^2 S_f} \qquad \text{(C-C3.1.2.1-4)}$$

Equation C-C3.1.2.1-4 was derived on the basis of a uniform bending moment and is conservative for other cases. For this reason σ_{cr} is modified by multiplying the right hand side by a bending coefficient C_b, to account for non-uniform bending and the symbol F_e is used for σ_{cr}, i.e.,

$$F_e = \frac{C_b \pi^2 E d I_{yc}}{(K_y L_y)^2 S_f} \qquad \text{(C-C3.1.2.1-5)}$$

where C_b is the bending coefficient, which can conservatively be taken as unity, or calculated from

$$C_b = 1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \le 2.3 \qquad \text{(C-C3.1.2.1-6)}$$

in which M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length.

The above Equation was used in the 1968, 1980, 1986, and 1991 editions of the AISI *Specification*. Because it is valid only for straight-line moment diagrams, Equation C-C3.1.2.1-6 was replaced by the following equation for C_b in the 1996 edition of the AISI *Specification* and is retained in this edition of the *Specification*:

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C} \qquad \text{(C-C3.1.2.1-7)}$$

where

M_{max} = absolute value of maximum moment in the unbraced segment

M_A = absolute value of moment at quarter point of unbraced segment

M_B = absolute value of moment at centerline of unbraced segment

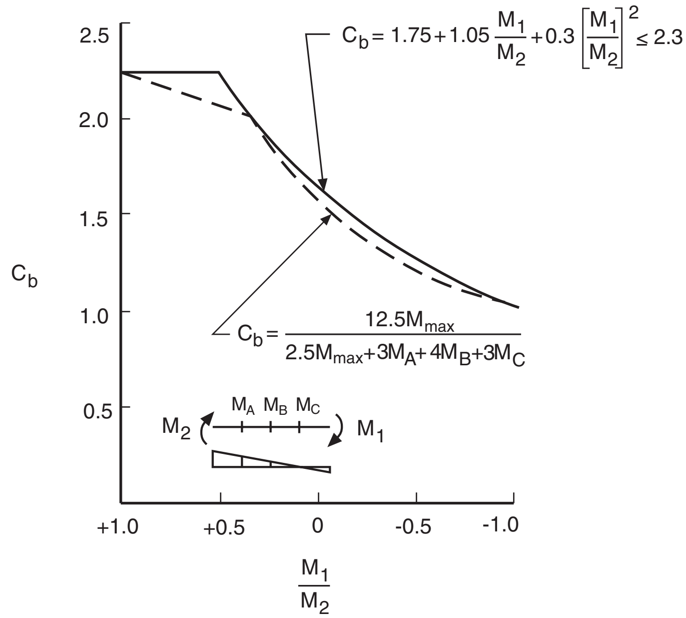


Figure C-C3.1.2.1-1 C_b for Straight Line Moment Diagram

M_C = absolute value of moment at three-quarter point of unbraced segment

Equation C-C3.1.2.1-7, derived from Kirby and Nethercot (1979), can be used for various shapes of moment diagrams within the unbraced segment. It gives more accurate solutions for fixed-end members in bending and moment diagrams which are not straight lines. This equation is the same as that being used in the ANSI/AISC S360 (AISC, 2005).

Figure C-C3.1.2.1-1 shows the differences between Equations C-C3.1.2.1-6 and C-C3.1.2.1-7 for a straight line moment diagram.

In 2001, effective length factor about the y-axis, K_y, was added to *Specification* Equations C3.1.2.1-14 and C3.1.2.1-15 on the basis of Equation C-C3.1.2.1-5. The K_y factor provides for other than simply supported end conditions. In addition, *Specification* Equation C3.1.2.1-14 have been permitted to be used for the design of singly-symmetric C-sections and I sections since the 1968 edition of the AISI *Specification*, and C3.1.2.1-15 has been permitted to be used for Z-sections since the 1996 edition of the AISI *Specification*.

Also in 2001, the requirement of taking C_b equal to unity when considering axial load and bending moment in *Specification* Section C5 was removed. This requirement was in place since both C_b and C_m in *Specification* Section C5 are adjustments for the moment gradient in the member and it was conservative to take C_b equal to unity. C_b is an adjustment to the critical moment for lateral-torsional buckling when the bending moment is not constant and C_m adjusts the magnitude of the second order p-delta moment in the member. Since these are two separate quantities, it is appropriate to use both C_b and C_m in evaluating the member under combined loads. However, it is still conservative to take C_b equal to unity.

It should be noted that Equations C-C3.1.2.1-1a and C-C3.1.2.1-5 apply only to elastic buckling of cold-formed steel members in bending when the computed theoretical buckling stress is less than or equal to the proportional limit. When the computed stress exceeds the proportional limit, the beam behavior will be governed by inelastic buckling. The inelastic buckling stress, F_c, can be computed from Equation C-C3.1.2.1-8 (Yu, 2000):

$$F_c = \frac{10}{9} F_y \left(1 - \frac{10F_y}{36F_e}\right) \qquad \text{(C-C3.1.2.1-8)}$$

where F_e is the elastic critical lateral-torsional buckling stress.

Equations C-C3.1.2.1-5 and C-C3.1.2.1-8 with K_y = 1.0 and L_y = L were used in the 1968, 1980 and 1986 editions of the AISI *Specification* to develop the allowable stress design equations for lateral-torsional buckling of I-members. In the 1986 edition of the AISI *Specification*, in addition to the use of Equations C-C3.1.2.1-5 and C-C3.1.2.1-8 for determining the critical stresses, more design equations (*Specification* Equations C3.1.2.1-4, C3.1.2.1-5, and C3.1.2.1-10) for elastic critical stress were added as alternative methods. These additional equations were developed from the previous studies conducted by Pekoz, Winter and Celebi on flexural-torsional buckling of thin-walled sections under eccentric loads (Pekoz and Winter, 1969a; Pekoz and Celebi, 1969b) and are retained in this edition of the *Specification*. These general design equations can be used for singly-, doubly- and point-symmetric sections. Consequently, the elastic critical lateral-torsional buckling stress can be determined by the following equation:

$$F_e = \frac{C_b A r_o}{S_f}\sqrt{\sigma_{ey}\sigma_t} \qquad \text{(C-C3.1.2.1-9)}$$

where σ_{ey} and σ_t are the elastic buckling stresses as defined in *Specification* Equations C3.1.2.1-8 and C3.1.2.1-9, respectively.

It should be noted that point-symmetric sections such as Z-sections with equal flanges will buckle laterally at lower strengths than doubly- and singly-symmetric sections. A conservative design approach has been and is being used in the *Specification*, in which the elastic critical buckling stress is taken to be one-half of that for I-members.

Regarding the inelastic critical buckling stress, the following equation was used for calculating the critical moment in Section C3.1.2(a) of the 1986 edition of the AISI *Specification* instead of using Equation C-C3.1.2.1-8 for inelastic critical buckling stress:

$$(M_{cr})_I = M_y\left[1 - \frac{M_y}{4(M_{cr})_e}\right] \qquad \text{(C-C3.1.2.1-10)}$$

in which $(M_{cr})_e$ is the elastic critical buckling moment. In 1996, the basic inelastic lateral-torsional buckling curve for singly-, doubly-, and point-symmetric sections in AISI *Specification* Section C3.1.2.1(a) was redefined to be consistent with the inelastic lateral-torsional buckling curve for I- or Z-sections in *Specification* Section C3.1.2.1(b). The general shape of the curve as represented by Equation C-C3.1.2.1-8 is also consistent with the preceding edition of the *Specification* (AISI, 1986).

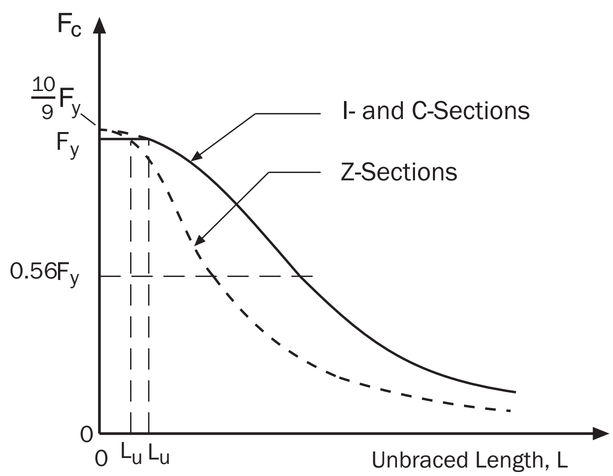


Figure C-C3.1.2.1-2 Lateral-Torsional Buckling Stress

As specified in *Specification* Section C3.1.2.1, lateral-torsional buckling is considered to be elastic up to a stress equal to $0.56F_y$. The inelastic region is defined by a Johnson parabola from $0.56F_y$ to $(10/9)F_y$ at an unsupported length of zero. The (10/9) factor is based on the partial plastification of the section in bending (Galambos, 1963). A flat plateau is created by limiting the maximum stress to F_y, which enables the calculation of the maximum unsupported length for which there is no stress reduction due to lateral-torsional instability. This maximum unsupported length can be calculated by setting F_y equal to F_c in Equation C-C3.1.2.1-8.

This liberalization of the inelastic lateral-torsional buckling curve for singly-, doubly-, and point-symmetric sections has been confirmed by research in beam-columns (Pekoz and Sumer, 1992) and wall studs (Niu and Pekoz, 1994).

The elastic and inelastic critical stresses for the lateral-torsional buckling strength are shown in Figure C-C3.1.2.1-2. For any unbraced length, L, less than L_u, lateral-torsional buckling does not need to be considered, where L_u is determined by setting $F_e = 2.78F_y$ and $L_u = L_y = L_t$. L_u may be calculated using the expression given below (AISI, 1996):

(a) for Singly-, doubly- and point-symmetric sections:

$$L_u = \left\{ \frac{GJ}{2C_1} + \left[\frac{C_2}{C_1} + \left(\frac{GJ}{2C_1} \right)^2 \right]^{0.5} \right\}^{0.5} \qquad \text{(C-C3.1.2.1-11)}$$

where

$$C_1 = \frac{7.72}{AE}\left[\frac{K_yF_yS_f}{C_b\pi r_y}\right]^2 \text{ for singly- and doubly-symmetric sections} \qquad \text{(C-C3.1.2.1-12)}$$

$$C_1 = \frac{30.9}{AE}\left[\frac{K_yF_yS_f}{C_b\pi r_y}\right]^2 \text{ for point-symmetric sections} \qquad \text{(C-C3.1.2.1-13)}$$

$$C_2 = \frac{\pi^2 EC_w}{(K_t)^2} \qquad \text{(C-C3.1.2.1-14)}$$

(b) for I-, C- or Z-sections bent about the centroidal axis perpendicular to the web, the following equations may be used in lieu of (a) (AISI, 1996):

For doubly-symmetric I-sections and singly-symmetric C-sections:

$$L_u = \frac{1}{K_y}\left[\frac{0.36C_b\pi^2 EdI_{yc}}{F_yS_f}\right]^{0.5} \qquad \text{(C-C3.1.2.1-15)}$$

For point-symmetric Z-sections:

$$L_u = \frac{1}{K_y}\left[\frac{0.18C_b\pi^2 EdI_{yc}}{F_yS_f}\right]^{0.5} \qquad \text{(C-C3.1.2.1-16)}$$

For members with unbraced length, $L \le L_u$, or elastic lateral-torsional buckling stress, $F_e \ge 2.78F_y$, the flexural strength [moment resistance] is determined in accordance with C3.1.1(a).

The above discussion dealt only with the lateral-torsional buckling strength [resistance] of locally stable beams. For locally unstable beams, the interaction of the local buckling of the compression elements and overall lateral-torsional buckling of members may result in a reduction of the lateral-torsional buckling strength [resistance] of the member. The effect of local buckling on the critical moment is considered in Section C3.1.2.1 of the *Specification* by using the elastic section modulus S_c based on an effective section. i.e.,

$$M_n = F_cS_c \qquad \text{(C-C3.1.2.1-17)}$$

where

F_c = Elastic or inelastic critical lateral-torsional buckling stress

S_c = Elastic section modulus of effective section calculated at a stress F_c relative to the extreme compression fiber

Using the above nominal lateral-torsional buckling strength [resistance] with a resistance factor of ϕ_b = 0.90, the values of β vary from 2.4 to 3.8 for the LRFD method.

The research conducted by Ellifritt, Sputo and Haynes (1992) has indicated that when the unbraced length is defined as the spacing between intermediate braces, the equations used in *Specification* Section C3.1.2.1 may be conservative for cases where one mid-span brace is used, but may be unconservative where more than one intermediate brace is used.

The above mentioned research (Ellifritt, Sputo, and Haynes, 1992) and the study of Kavanagh and Ellifritt (1993 and 1994) have shown that a discretely braced beam, not attached to deck and sheathing, may fail either by lateral-torsional buckling between braces, or by distortional buckling at or near the braced point. See Section C3.1.4 for commentary on distortional buckling strength.

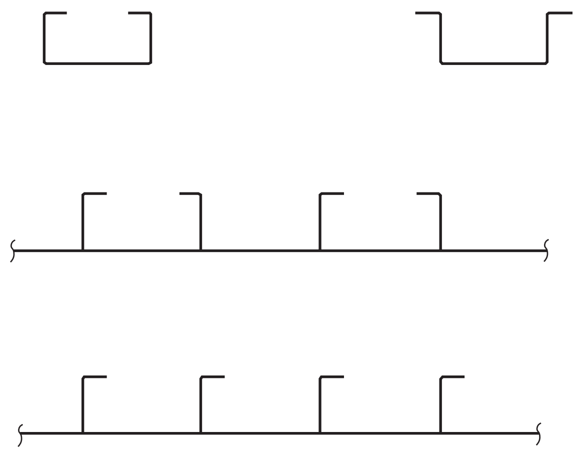


Figure C-C3.1.2.1-3 Combined Sheet-Stiffener Sections

Figure C-C3.1.2.1-4 Lateral Buckling of U-Shaped Beam

The problems discussed above dealt with the type of lateral-torsional buckling of I-members, C-sections, and Z-shaped sections for which the entire cross section rotates and deflects in the lateral direction as a unit. But this is not the case for U-shaped beams and the combined sheet-stiffener sections as shown in Figure C-C3.1.2.1-3. For this case, when the section is loaded in such a manner that the brims and the flanges of stiffeners are in compression, the tension flange of the beam remains straight and does not displace laterally; only the compression flange tends to buckle separately in the lateral direction, accompanied by out-of-plane bending of the web, as shown in Figure C-

C3.1.2.1-4, unless adequate bracing is provided.

The precise analysis of the lateral buckling of U-shaped beams is rather complex. The compression flange and the compression portion of the web act not only like a column on an elastic foundation, but the problem is also complicated by the weakening influence of the torsional action of the flange. For this reason, the design procedure outlined in Section 2 of Part V (Supplementary Information) of the AISI *Cold-Formed Steel Design Manual* (AISI, 2008) for determining the nominal stress for laterally unbraced compression flanges is based on the considerable simplification of an analysis presented by Douty (1962).

In 1964, Haussler presented rigorous methods for determining the strength [resistance] of elastically stabilized beams (Haussler, 1964). In his methods, Haussler also treated the unbraced compression flange as a column on an elastic foundation and maintained more rigor in his development.

A comparison of Haussler's method with Douty's simplified method indicates that the latter may provide a lower value of critical stress.

An additional study of laterally unbraced compression flanges has been made at Cornell University (Serrette and Pekoz, 1992, 1994 and 1995). An analytical procedure has been developed for determining the distortional buckling strength [resistance] of the standing seam roof panel. The predicted maximum capacities have been compared with experimental results.

C3.1.2.2 Lateral-Torsional Buckling Strength [Resistance] for Closed Box Members

Due to the high torsional stiffness of closed box sections, lateral-torsional buckling is not critical in typical design considerations, even for bending about the major axis. Deflection limits will control most designs due to the large values of L_u. However, lateral-torsional buckling can control the design when the unbraced length is larger than L_u, which is determined by setting the inelastic buckling stress of *Specification* Equation C3.1.2.1-2 equal to F_y, with F_e set equal to *Specification* Equation C3.1.2.2-2.

In computing the lateral-torsional buckling stress of closed box sections, the warping constant, C_w, may be neglected since the effect of non-uniform warping of box sections is small. The development of *Specification* Equation C3.1.2.2-2 can be found in the SSRC Guide (Galambos, 1998). As a result of adding Section C3.1.2.2 to the *Specification*, *Specification* Section D3.3 has been deleted.

The Saint-Venant torsional constant, J, of a box section, neglecting the corner radii, may be conservatively determined as follows:

$$J = \frac{2(ab)^2}{(a/t_1) + (b/t_2)} \tag{C-C3.1.2.2-1}$$

where

a = distance between web centerlines

b = distance between flange centerlines

t_1 = thickness of flanges

t_2 = thickness of webs

In 2001, the unbraced length, L, in *Specification* Equation C3.1.2.2-2 was replaced with K_yL_y, where K_y is the effective length factor for bending about the y-axis. The K_y factor provides for other than simply supported end conditions. Detailed discussions are provided in Section C3.1.2.1 of the *Commentary*.

C3.1.3 Flexural Strength [Resistance] of Closed Cylindrical Tubular Members

The discussion on cylindrical tubular member behavior and buckling modes are provided in Commentary Section C4.1.5. It should be noted that the design provisions of *Specification* Sections C3.1.3 and C4.1.5 are applicable only for members having a ratio of outside diameter-to-wall thickness, D/t, not greater than $0.441E/F_y$ because the design of extremely thin tubes will be governed by elastic local buckling resulting in an uneconomical design. In addition, cylindrical tubular members with unusually large D/t ratios are very sensitive to geometric imperfections.

For thick cylinders in bending, the initiation of yielding does not represent a failure condition as is generally assumed for axial loading. Failure is at the plastic moment capacity, which is at least 1.29 times the moment at first yielding. In addition, the conditions for inelastic local buckling are not as severe as in axial compression due to the stress gradient.

Specification Equations C3.1.3-2, C3.1.3-3 and C3.1.3-4 are based upon the work reported by Sherman (1985) and an assumed minimum shape factor of 1.25. This slight reduction in the inelastic range has been made to limit the maximum bending stress to $0.75F_y$, a value typically used for solid sections in bending for the ASD method. The reduction also brings the criteria closer to a lower bound for inelastic local buckling. A small range of elastic local buckling has been included so that the upper D/t limit of $0.441E/F_y$ is the same as for axial compression.

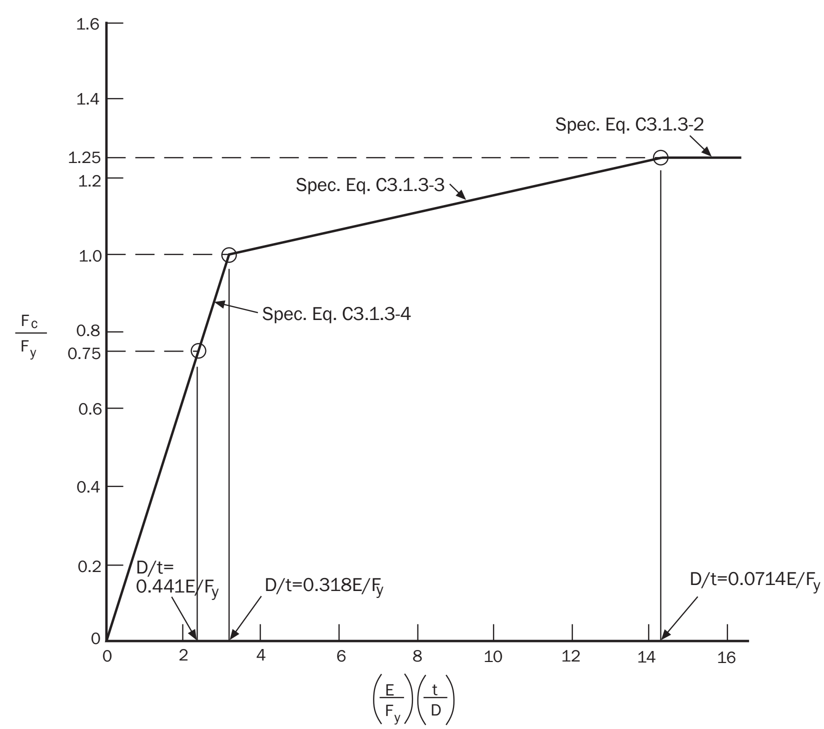


Figure C-C3.1.3-1 Nominal Flexural Strength of Cylindrical Tubular Members

All three equations for determining the nominal flexural strength [moment resistance] of closed cylindrical tubular members are shown in Figure C-C3.1.3-1. These equations have been used in the AISI *Specification* since 1986 and are retained in this *Specification*. In 1999, the limiting D/t ratios for *Specification* Equations C3.1.3-2 and C3.1.3-3 have been revised to provide an appropriate continuity. The safety factor Ω_b and the resistance factor ϕ_b are the same as that used in *Specification* Section C3.1.1 for sectional bending strength.

C3.1.4 Distortional Buckling Strength [Resistance]

Distortional buckling is an instability that may occur in members with edge stiffened flanges, such as lipped C- and Z-sections. As shown in Figure C-C3.1.4-1, this buckling mode is characterized by instability of the entire flange, as the flange along with the edge stiffener rotates about the junction of the compression flange and the web. The length of the buckling wave in distortional buckling is considerably longer than local buckling, and noticeably shorter than lateral-torsional buckling. The *Specification* provisions of Section B4 partially account for distortional buckling, but research has shown that a separate limit state check is required (Ellifritt, Sputo, and Haynes 1992, Hancock, Rogers, and Schuster 1996, Kavanagh and Ellifritt 1994, Schafer and Peköz 1999, Hancock 1997, Yu and Schafer 2003, 2006). Thus, in 2007, *Specification* Section C3.1.4 was added to address distortional buckling as a separate limit state.

Determination of the nominal strength in distortional buckling (*Specification* Equation C3.1.4-2) was validated by testing. Results of one such study (Yu and Schafer 2006) are shown in Figure C-C3.1.4-2. The Direct Strength Method of Appendix 1 of the *Specification* also uses Equation C3.1.4-2. In addition, the Australian/New Zealand Specification (AS/NZS 4600) has used Equation C3.1.4-2 since 1996. Calibration of the safety and resistance factors for Equation C3.1.4-2 is provided in the commentary to Appendix 1.

Distortional buckling is unlikely to control the strength if (a) edge stiffeners are sufficiently stiff and thus stabilize the flange (as is often the case for C-sections, but typically not for Z-sections due to the use of sloping lips), (b) unbraced lengths are long and lateral-torsional buckling strength limits the capacity, or (c) adequate rotational restraint is provided to the compression flange from attachments (panels, sheathing, etc.).

The primary difficulty in calculating the strength in distortional buckling is to efficiently estimate the elastic distortional buckling stress, F_d. Recognizing the complexity of this calculation, this *Specification* section provides three alternatives: C3.1.4(a) provides a conservative prediction for unrestrained C- and Z-sections, C3.1.4(b) provides a more comprehensive method for C- and Z-Section members and any open section with a single web and single edge stiffened compression flange, and C3.1.4(c) offers the option to use rational elastic buckling analysis, e.g., see the Appendix 1 commentary. The equations of C3.1.4(a) assume the compression flange is unrestrained; however, the methods of C3.1.4(b) and (c) allow for a rotational restraint, k_ϕ, to be included to account for attachments which restrict flange rotation.

While it is always conservative to ignore the rotational restraint, k_ϕ, in many cases it may be beneficial to include this effect. Due to the large variety of possible conditions, no specific method is provided for determining the rotational restraint. Instead, per Section A1.2 of the *Specification,* k_ϕ may be estimated by testing or rational engineering analysis. Test determination of k_ϕ may use AISI S901 (AISI 2002). K from this method is a lower bound estimate of k_ϕ. The member lateral deformation may be removed from the measured lateral deformation to provide a more accurate estimate of k_ϕ.

Testing on 8 in. and 9.5 in. (203 and 241 mm) deep Z-sections with a thickness between 0.069 in. (1.75 mm) and 0.118 in. (3.00 mm), through-fastened 12 in. (205 mm) o.c., to a 36 in. (914 mm) wide, 1 in. (25.4 mm) and 1.5 in. (38.1 mm) high steel panels, with up to 6 in. (152 mm) of blanket insulation between the panel and the Z-section, results in a k_ϕ between 0.15 to 0.44 kip-in./rad./in. (0.667 to 1.96 kN-mm/rad./mm) (MRI 1981).

Additional testing on C- and Z-sections with pairs of through-fasteners provides considerably higher rotational stiffness: for 6 and 8 in. (152 and 203 mm) deep C-sections with a thickness between 0.054 and 0.097 in. (1.27 and 2.46 mm), fastened with pairs of fasteners on each side of a 1.25 in. (31.8 mm) high steel panel flute at 12 in. (305 mm) o.c., k_ϕ is 0.4 kip-in./rad./in. (1.78 kN-mm/rad./mm); and for 8.5 in. (216 mm) deep Z-sections with a thickness between 0.070 in. and 0.120 in. (1.78 mm to 3.05 mm), fastened with pairs of fasteners on each side of 1.25 in. (31.8 mm) high steel panel flute at 12 in. (305 mm) o.c., k_ϕ is 0.8 kip-in./rad./in. (3.56 kN-mm/rad./mm) (Yu and Schafer 2003, Yu 2005).

Examples of rational engineering analysis to estimate the rotational stiffness are provided in the Direct Strength Method Design Guide (AISI 2006). For a flexural member, k_ϕ can be approximated as:

$$k_\phi \approx EI/(W/2) \qquad \text{(C-C3.1.4-1)}$$

where E is the modulus of the attached material, I is the moment of inertia of the engaged attachment, and W is the member spacing. The primary complication in such a method is determining how much of the attachment (decking, sheathing, etc.) is engaged when the flange attempts to deform. For the Z-sections tested in Yu (2005) experimental k_ϕ is 0.8 kip-in./rad./in. (3.56 kN-mm/rad./mm). Using an estimate of EI/(W/2) the rational engineering values are k_ϕ of 9 kip-in./rad/in. (40.0 kN-mm/rad./mm) if the entire panel, flutes and all, are engaged; k_ϕ of 1.2 kip-in./rad/in. (5.34 kN-mm/rad./mm) if only the corrugated bottom panel, but not the flutes, is engaged; and k_ϕ of 0.003 kip-in./rad./in. (0.0133 kN-mm/rad./mm) if plate bending of the t = 0.019 in. (0.483 mm) panel occurs. The observed panel engagement is between the last two estimates, and assuming the corrugated bottom pan, but not the 1.25 in. (31.8 mm) high flutes is engaged is reasonable.

For members with wood sheathing attached, little experimental information is available. The problem has been studied numerically using the same paired fastener detail as in Yu's (2005) and Yu and Schafer (2003) tests but replacing the steel panel with a simulated wood member, thickness = 0.5 in. (12.7 mm), E = 1000 ksi (6900 MPa), and μ = 0.3. The calculated k_ϕ is 5.1 kip-in./rad./in. (22.7 kN-mm/rad./mm) for 6 and 8 in. (152 to 203 mm) deep C-sections with a thickness between 0.054 and 0.097 in. (1.37 and 2.46 mm); and k_ϕ is 4.1 kip-in./rad./in. (18.2 kN-mm/rad./mm) for 8.5 in. (216 mm) deep Z-sections with thickness between 0.070 and 0.120 in. (1.78 mm and 3.05 mm). From calculations assuming a fully engaged 1/2 in. (12.7 mm) thick wood sheet on top of C- or Z-section members spaced 12 in. (305 mm) apart, k_ϕ is predicted to be 1.7 kip-in./rad./in. (7.56 kN-mm/rad./mm). Thus, use of EI/(W/2) provides a reasonably conservative approximation, with I calculated assuming the full engagement of wood sheet.

The presence of moment gradient can also increase the distortional buckling moment (or equivalently stress, F_d). However, this increase is lessened if the moment gradient occurs over a longer length. Thus, in determining the influence of moment gradient (β) the ratio of the end moments, M_1/M_2, and the ratio of the critical distortional buckling length to the unbraced length, L/L_m, should both be accounted for. Yu (2005) performed elastic buckling analysis with shell finite element models of C- and Z-sections under different moment gradients to examine this problem. Significant scatter exists in the results, therefore a lower bound prediction (*Specification* Equation C3.1.4-11) for the increase was selected.

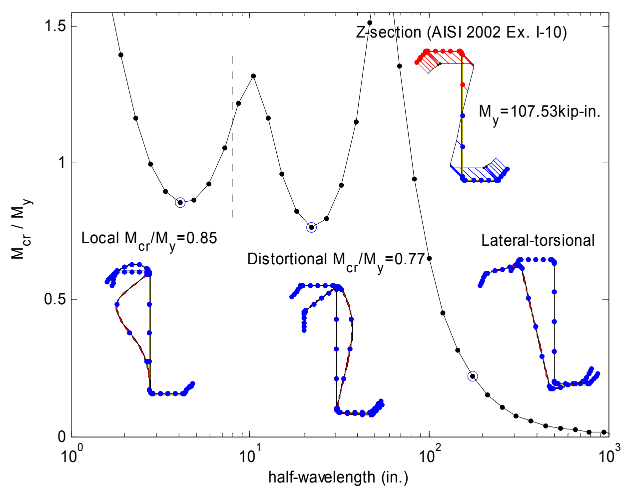


Figure C-C3.1.4-1 Rational Elastic Buckling Analysis of a Z-Section under Restrained Bending Showing Local, Distortional, and Lateral-Torsional Buckling Modes

(a) Simplified Provision for Unrestrained C- and Z-Sections with Simple Lip Stiffeners

The provision of *Specification* Section C3.1.4(a) provides a conservative approximation to the distortional buckling length, L_{cr}, and stress, F_d, for C- and Z-sections with simple lip stiffeners bent about an axis perpendicular to the web. The provision ignores any rotational restraint, which would restrain distortional buckling. The expressions were specifically derived as a conservative simplification to those provided in *Specification* Sections C3.1.4(b) and (c).

(b) For C- and Z-Sections or any Open Section with a Stiffened Compression Flange Extending to One Side of the Web where the Stiffener is either a Simple Lip or a Complex Edge Stiffener

The provisions of *Specification* Section C3.1.4(b) provide a general method for calculation of the distortional buckling stress, F_d, for any open section with an edge stiffened compression flange, including complex edge stiffeners. The provisions of *Specification* Section C3.1.4(b) also provide a more refined answer for any C- and Z-section including those meeting the criteria of C3.1.4(a). The expressions employed here are derived in Schafer and Peköz (1999) and verified for complex stiffeners in Schafer et al. (2006). The equations used for the distortional buckling stress, F_d, in AS/NZS 4600 are also similar to those in *Specification* Section C3.1.4 (b), except that when the web is very slender and is restrained by the flange, AS/NZS 4600 uses a simpler, conservative treatment. Since the provided expressions can be complicated, solutions for the geometric properties of C- and Z-sections based on centerline dimensions are provided in Table C-C3.1.4(b)-1.

(c) Rational Elastic Buckling Analysis

Rational elastic buckling analysis consists of any method following the principles of mechanics to arrive at an accurate prediction of the elastic distortional buckling stress (moment). It is important to note that this is a rational elastic buckling analysis and not simply an arbitrary rational method to determine ultimate strength. A variety of rational computational and analytical methods can provide the elastic buckling moment with a high degree of accuracy. Complete details are provided in Section 1.1.2 of the commentary to Appendix 1 of the *Specification*. The safety and resistance factors of this section have been shown to apply to a wide variety of cross-sections undergoing distortional buckling (via the methods of Appendix 1). As long as the member falls within the geometric limits of main *Specification* Section B1.1, the same safety and resistance factors have been assumed to apply. Application of the β expression, to account for moment gradient, as provided in *Specification* Section C3.1.4(b) is a rational extension to solutions which do not typically account for moment gradient such as the finite strip method.

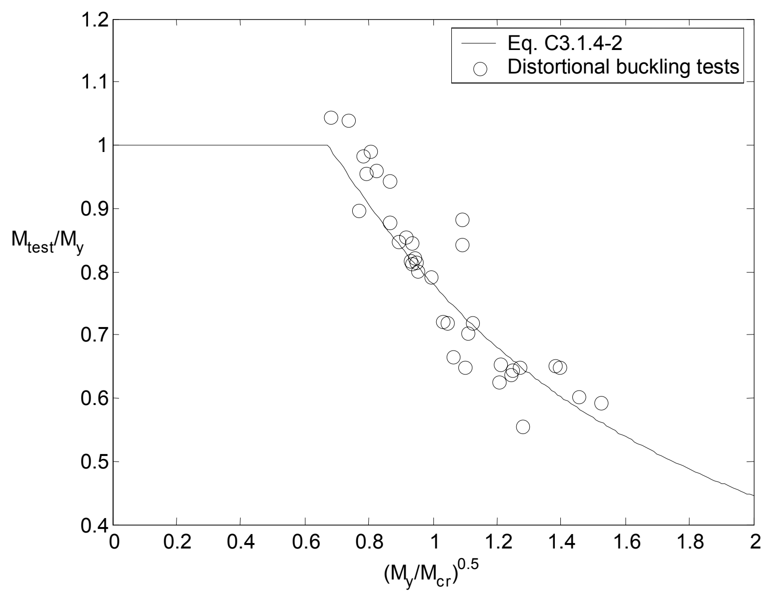
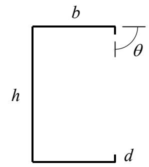
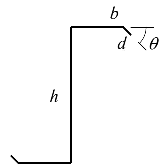


Figure C-C3.1.4-2 Performance of Distortional Buckling Prediction with Test Data on Common C- and Z-sections in Bending (Yu and Schafer 2006)

Table C-C3.1.4(b)-1
Geometric flange properties for C- and Z-sections





$A_f = (b+d)t$	$A_f = (b+d)t$
$J_f = \frac{1}{3}bt^3 + \frac{1}{3}dt^3$	$J_f = \frac{1}{3}bt^3 + \frac{1}{3}dt^3$
$I_{xf} = \dfrac{t\left(t^2b^2 + 4bd^3 + t^2bd + d^4\right)}{12(b+d)}$	$I_{xf} = \dfrac{t\left(t^2b^2 + 4bd^3 - 4bd^3\cos^2(\theta) + t^2bd + d^4 - d^4\cos^2(\theta)\right)}{12(b+d)}$
$I_{yf} = \dfrac{t\left(b^4 + 4db^3\right)}{12(b+d)}$	$I_{yf} = \dfrac{t\left(b^4 + 4db^3 + 6d^2b^2\cos(\theta) + 4d^3b\cos^2(\theta) + d^4\cos^2(\theta)\right)}{12(b+d)}$
$I_{xyf} = \dfrac{tb^2d^2}{4(b+d)}$	$I_{xyf} = \dfrac{tbd^2\sin(\theta)(b + d\cos(\theta))}{4(b+d)}$
$C_{wf} = 0$	$C_{wf} = 0$
$x_o = \dfrac{b^2}{2(b+d)}$	$x_o = \dfrac{b^2 - d^2\cos(\theta)}{2(b+d)}$
$h_x = \dfrac{-\left(b^2 + 2db\right)}{2(b+d)}$	$h_x = \dfrac{-\left(b^2 + 2db + d^2\cos(\theta)\right)}{2(b+d)}$
$h_y = y_o = \dfrac{-d^2}{2(b+d)}$	$h_y = y_o = \dfrac{-d^2\sin(\theta)}{2(b+d)}$

C3.2 Shear

C3.2.1 Shear Strength [Resistance] of Webs without Holes

The shear strength [resistance] of beam webs is governed by either yielding or buckling, depending on the h/t ratio and the mechanical properties of steel. For beam webs having small h/t ratios, the nominal shear strength [resistance] is governed by shear yielding, i.e.,

$$V_n = A_w\tau_y = A_wF_y/\sqrt{3} \approx 0.60F_yht \qquad \text{(C-C3.2.1-1)}$$

in which A_w is the area of the beam web computed by (ht), and τ_y is the yield stress of steel in shear, which can be computed by $F_y/\sqrt{3}$.

For beam webs having large h/t ratios, the nominal shear strength [resistance] is governed by elastic shear buckling (Yu, 2000), i.e.,

$$V_n = A_w\tau_{cr} = \frac{k_v\pi^2EA_w}{12(1-\mu^2)(h/t)^2} \qquad \text{(C-C3.2.1-2)}$$

in which τ_{cr} is the critical shear buckling stress in the elastic range, k_v is the shear buckling coefficient, E is the modulus of elasticity, μ is the Poisson's ratio, h is the web depth, and t is the web thickness. By using μ = 0.3, the shear strength [resistance], V_n, can be determined as follows:

$$V_n = 0.904Ek_v t^3 / h \qquad \text{(C-C3.2.1-3)}$$

For beam webs having moderate h/t ratios, the nominal shear strength [resistance] is based on inelastic shear buckling (Yu, 2000), i.e.,

$$V_n = 0.64t^2 \sqrt{k_v F_y E} \qquad \text{(C-C3.2.1-4)}$$

The *Specification* provisions are applicable for the design of webs of beams and decks either with or without transverse web stiffeners.

The nominal strength [resistance] equations of Section C3.2.1 of the *Specification* are similar to the nominal shear strength [resistance] equations given in the AISI LRFD *Specification* (AISI, 1991). The acceptance of these nominal strength [resistance] equations for cold-formed steel sections has been considered in the study summarized by LaBoube and Yu (1978a).

Previous editions of the AISI ASD *Specification* (AISI, 1986) used three different safety factors when evaluating the allowable shear strength [resistance] of an unreinforced web because it was intended to use the same nominal strength [resistance] equations for the AISI and AISC Specifications. To simplify the design of shear using only one safety factor for ASD and one resistance factor for LRFD, Craig (Craig, 1999) carried out a calibration using the data by LaBoube and Yu (LaBoube, 1978a). Based on this work, the constant used in *Specification* Equation C3.2.1-3 was reduced from 0.64 to 0.60. In addition, the ASD safety factor for yielding, elastic and inelastic buckling is now taken as 1.60, with a corresponding resistance factor of 0.95 for LRFD and 0.80 for LSD.

C3.2.2 Shear Strength [Resistance] of C-Section Webs With Holes

For C-section webs with holes, Schuster et al. (1995) and Shan et al. (1994) investigated the degradation in web shear strength [resistance] due to the presence of a web perforation. The test program considered a constant shear distribution across the perforation, and included d_0/h ratios ranging from 0.20 to 0.78, and h/t ratios of 91 to 168. Schuster's q_s equation was developed with due consideration for the potential range of both punched and field cut holes. Three hole geometries, rectangular with corner fillets, circular, and diamond, were considered in the test program. Eiler (1997) extended the work of Schuster and Shan for the case of constant shear along the longitudinal axis of the perforation. He also studied linearly varying shear but this case is not included in the *Specification*. The development of Eiler's reduction factor, q_s, utilized the test data of both Schuster et al. (1995) and Shan et al. (1994). The focus of the test programs was on the behavior of slender webs with holes. Thus for stocky web elements with $h/t \le 0.96\sqrt{Ek_v/F_y}$, an anomaly exists; the calculated available shear strength [resistance] is independent of t when h is constant. In this region, the calculated available shear strength [resistance] is valid but may be somewhat conservative.

The provisions for circular and non-circular holes also apply to any hole pattern that fits within an equivalent virtual hole. Figure C-B2.4-1 illustrates the L_h and d_h that may be used for a multiple hole pattern that fits within a non-circular virtual hole. Figure C-B2.4-2 illustrates the d_h that may be used for a rectangular hole that fits within a circular virtual hole. For each case, the design provisions apply to the geometry of the virtual hole geometry, not the actual hole or holes.

C3.3 Combined Bending and Shear

For cantilever beams and continuous beams, high bending stresses often combine with high shear stresses at the supports. Such beam webs must be safeguarded against buckling due to the combination of bending and shear stresses.

For disjointed flat rectangular plates, the critical combination of bending and shear stresses can be approximated by the following interaction equation (Bleich, 1952), which is part of a unit circle:

$$\left(\frac{f_b}{f_{cr}}\right)^2+\left(\frac{\tau}{\tau_{cr}}\right)^2=1.0 \qquad \text{(C-C3.3-1)}$$

or

$$\sqrt{\left(\frac{f_b}{f_{cr}}\right)^2+\left(\frac{\tau}{\tau_{cr}}\right)^2}=1.0 \qquad \text{(C-C3.3-2)}$$

where f_b is the actual compressive bending stress, f_{cr} is the theoretical buckling stress in pure bending, τ is the actual shear stress and τ_{cr} is the theoretical buckling stress in pure shear. The above equation was found to be conservative for beam webs with adequate transverse stiffeners, for which a diagonal tension field action can be developed. Based on the studies made by LaBoube and Yu (1978b), Equation C-C3.3-3 was developed for beam webs with transverse stiffeners satisfying the requirements of *Specification* Section C3.7.

$$0.6\frac{f_b}{f_{b_{max}}}+\frac{\tau}{\tau_{max}}=1.3 \qquad \text{(C-C3.3-3)}$$

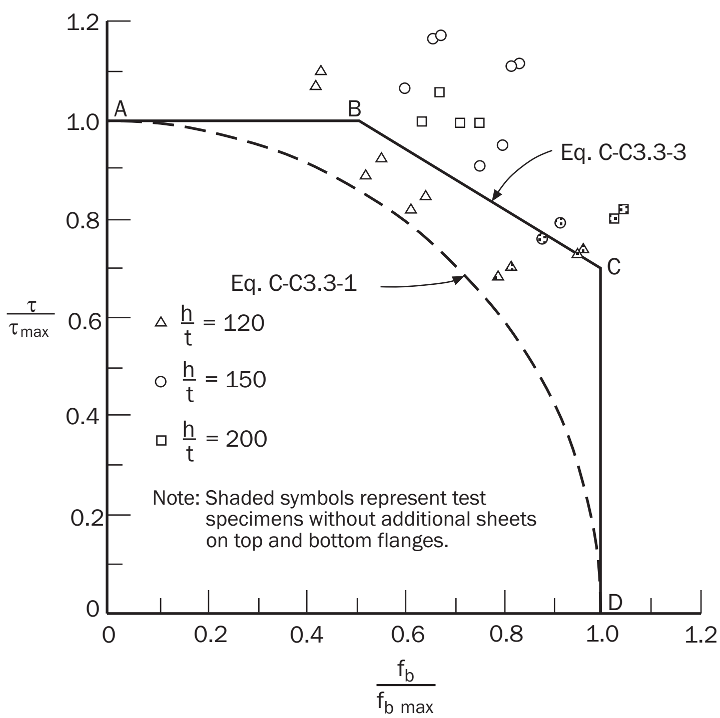


Figure C-C3.3-1 Interaction Diagram for τ/τ_{max} and f_b/f_{bmax}

The above equation was added to the AISI *Specification* in 1980. The correlations between Equation C-C3.3-3 and the test results of beam webs having a diagonal tension field action are shown in Figure C-C3.3-1.

C3.3.1 ASD Method

Since 1986, the AISI ASD *Specification* uses strength ratios (i.e., moment ratio for bending and force ratio for shear) instead of stress ratios for the interaction equations. *Specification* Equations C3.3.1-1 and C3.3.1-2 are based on Equations C-C3.3-2 and C-C3.3-3, respectively, by using the allowable moment, M_{nxo}/Ω_b, and the allowable shear force, V_n/Ω_v.

C3.3.2 LRFD and LSD Methods

For the load and resistance factor design and the limit states design, the interaction equations for combined bending and shear are also based on Equations C-C3.3-2 and C-C3.3-3 as given in *Specification* Equations C3.3.2-1 and C3.3.2-2 by using the required and design strengths. In both equations, different symbols are used for the required flexural strength [factored moment] and the required shear strength [factored shear] according to the LRFD and the LSD methods.

C3.4 Web Crippling

C3.4.1 Web Crippling Strength [Resistance] of Webs without Holes

Since cold-formed steel flexural members generally have large web slenderness ratios, the webs of such members may cripple due to the high local intensity of the load or reaction. Figure C-C3.4.1-1 shows typical web crippling failure modes of unreinforced single hat sections (Figure C-C3.4.1-1(a)) and of I-sections (Figure C-C3.4.1-1(b)) unfastened to the support.

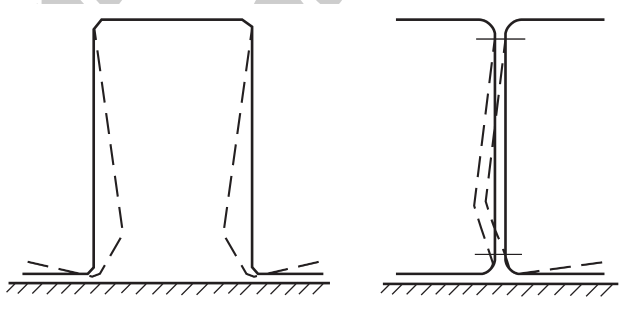


Figure C-C3.4.1-1 Web Crippling of Cold-Formed Steel Sections

In the past, the buckling problem of plates and the web crippling behavior of cold-formed steel members under locally distributed edge loading have been studied by numerous investigators (Yu, 2000). It has been found that the theoretical analysis of web crippling for cold-formed steel flexural members is rather complicated because it involves the following factors: (1) nonuniform stress distribution under the applied load and adjacent portions of the web, (2) elastic and inelastic stability of the web element, (3) local yielding in the immediate region of load application, (4) bending produced by eccentric load (or reaction) when it is applied on the bearing flange at a distance beyond the curved transition of the web, (5) initial out-of-plane imperfection of plate elements, (6) various edge restraints provided by beam flanges and interaction between flange and web elements, and (7) inclined webs for decks and panels.

For these reasons, the present AISI design provision for web crippling is based on the extensive experimental investigations conducted at Cornell University by Winter and Pian (1946) and Zetlin (1955a); at the University of Missouri-Rolla by Hetrakul and Yu (1978 and 1979), Yu (1981), Santaputra (1986), Santaputra, Parks and Yu (1989), Bhakta, LaBoube and Yu (1992), Langan, Yu and LaBoube (1994), Cain, LaBoube and Yu (1995) and Wu, Yu and LaBoube (1997); at the University of Waterloo by Wing (1981), Wing and Schuster (1982), Prabakaran (1993), Gerges (1997), Gerges and Schuster (1998), Prabakaran and Schuster (1998), Beshara (1999), and Beshara and Schuster (2000 and 2000a); and at the University of Sydney by Young and Hancock (1998). In these experimental investigations, the web crippling tests were carried out under the following four loading conditions for beams having single unreinforced webs and I-beams, single hat sections and multi-web deck sections:

1. End one-flange (EOF) loading
2. Interior one-flange (IOF) loading
3. End two-flange (ETF) loading
4. Interior two-flange (ITF) loading

All loading conditions are illustrated in Figure C-C3.4.1-2. In Figures (a) and (b), the distances between bearing plates were kept to no less than 1.5 times the web depth in order to avoid the two-flange loading action. Application of the various load cases is shown in

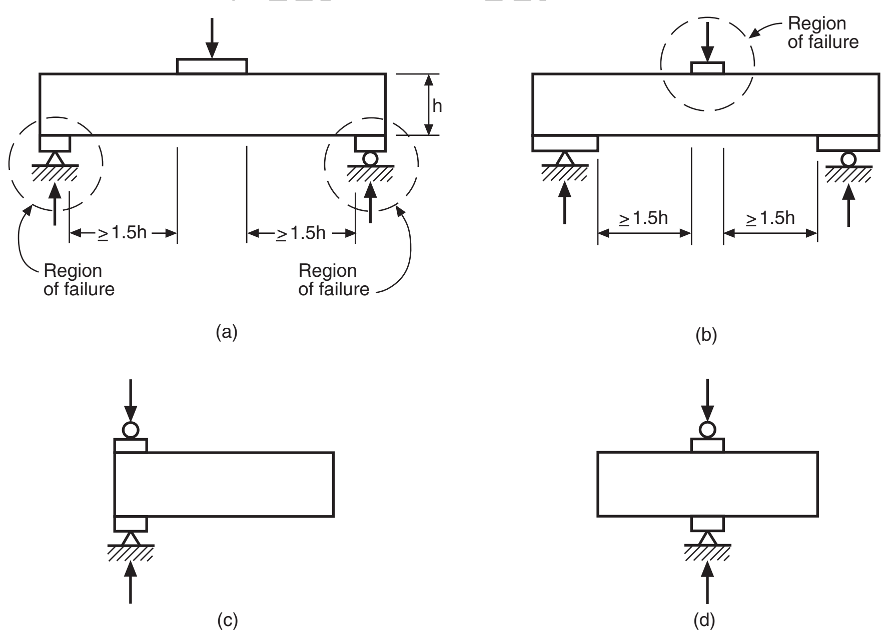


Figure C-C3.4.1-2 Loading Conditions for Web Crippling Tests
(a) EOF Loading, (b) IOF Loading, (c) ETF Loading, (d) ITF Loading

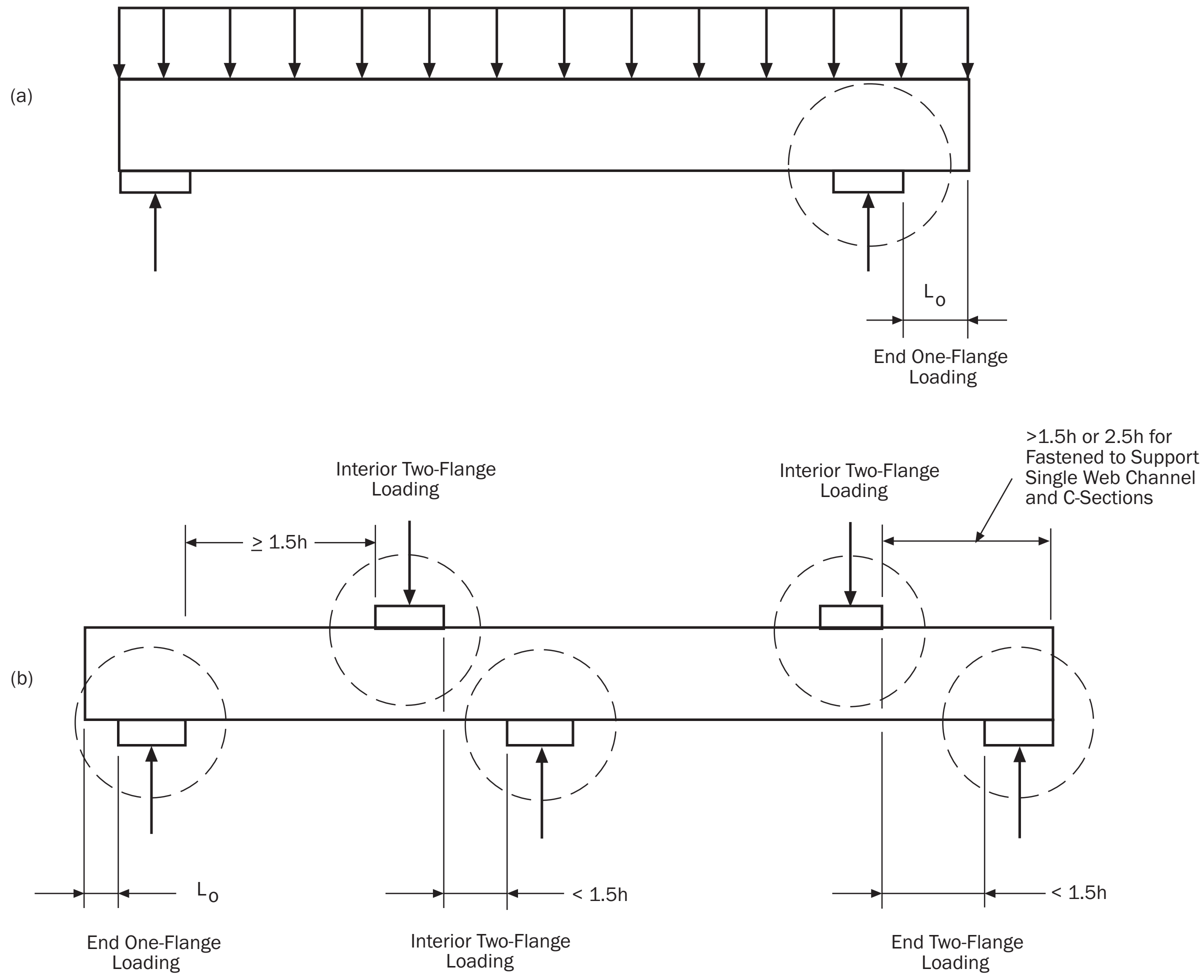


Figure C-C3.4.1-3 Application of Loading Cases

Figure C-C3.4.1-3 and the assumed reaction or load distributions are illustrated in Figure C-C3.4.1-4.

In the 1996 edition of the AISI *Specification*, and in previous editions, different web crippling equations were used for the various loading conditions stated above. These equations were based on experimental evidence (Winter, 1970; Hetrakul and Yu, 1978) and the assumed distributions of loads or reactions acting on the web as shown in Figure C-C3.4.1-4. The equations were also based on the type of section geometry, i.e., shapes having single webs and I-sections (made of two channels connected back to back, by welding two angles to a channel, or by connecting three channels). C-and Z-sections, single hat sections and multi-web deck sections were considered in the single web member category. I-sections made of two channels connected back to back by a line of connectors near each flange or similar sections that provide a high degree of restraint against rotation of the web were treated separately. In addition, different equations were used for sections with stiffened or partially stiffened flanges and sections with unstiffened flanges.

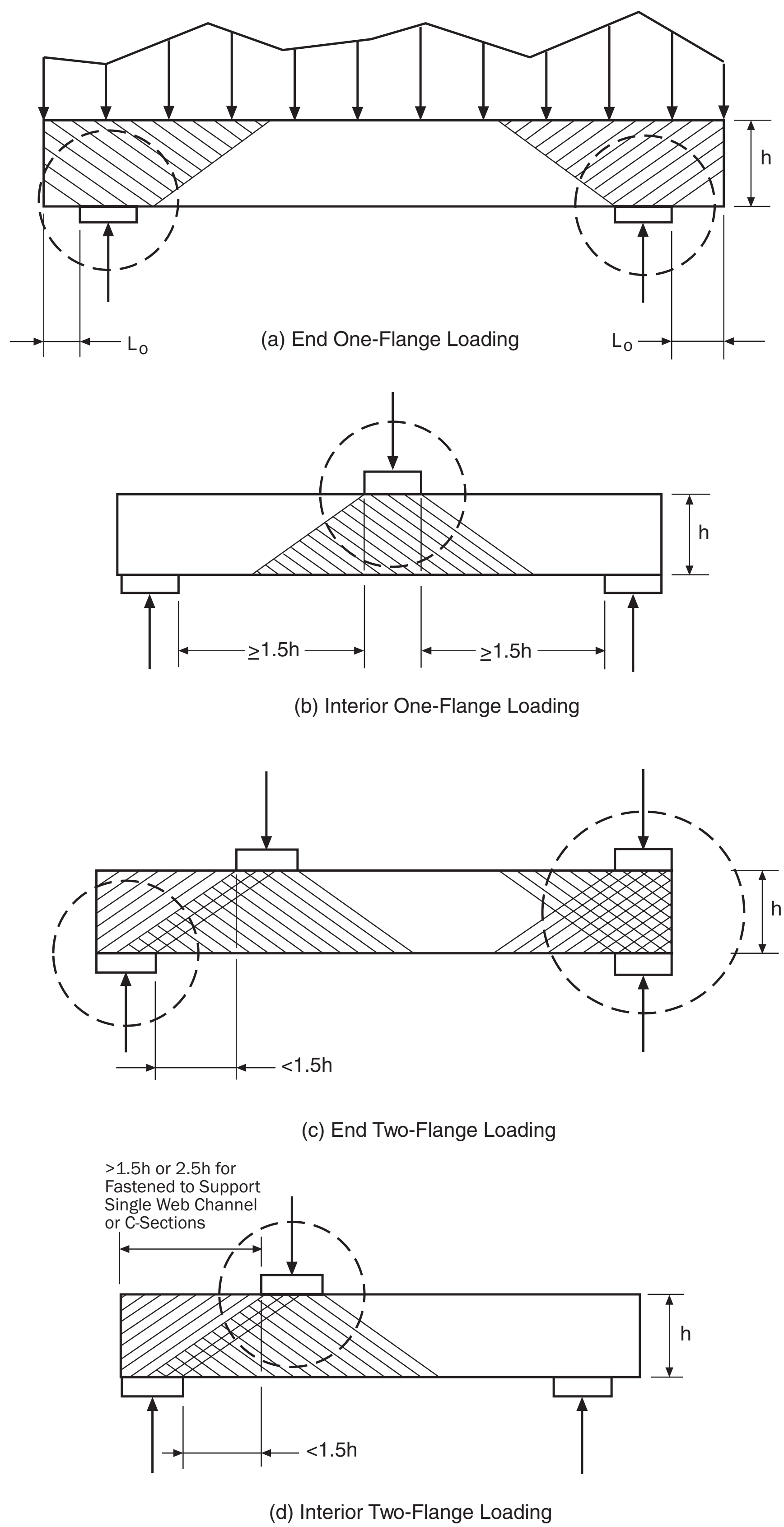


Figure C-C3.4.1-4 Assumed Distribution of Reaction or Load

Prabakaran (1993) and Prabakaran and Schuster (1998) developed one consistent unified web crippling equation with variable coefficients (*Specification* Equation C3.4.1-1). These coefficients accommodate one or two flange loading for both end and interior loading conditions of various section geometries. Beshara (1999) extended the work of Prabakaran and Schuster (1998) by developing new web crippling coefficients using the available data as summarized by Beshara and Schuster (2000). The web crippling coefficients are summarized in Tables C3.4.1-1 to C3.4.1-5 of the *Specification* and the parametric limitations given are based on the experimental data that was used in the development of the web crippling coefficients. From *Specification* Equation C3.4.1-1, it can be seen that the nominal web crippling strength of cold-formed steel members depends on an overall web crippling coefficient, C, the web thickness, t, the yield stress, F_y, the web inclination angle, θ, the inside bend radius coefficient, C_R, the inside bend radius ratio, R/t, the bearing length coefficient, C_N, the bearing length ratio, N/t, the web slenderness coefficient, C_h, and the web slenderness ratio, h/t.

This new equation is presented in a normalized format and is non-dimensional, allowing for any consistent system of measurement to be used. Consideration was given to whether or not the test specimens were fastened to the bearing plate/support during testing. It was discovered that some of the test specimens in the literature were not fastened to the bearing plate/support during testing, which can make a considerable difference in the web crippling capacity of certain sections and loading conditions. Therefore, it was decided to separate the data on the basis of members being fastened to the bearing plate/support and those not being fastened to the bearing plate/support. The fastened to the bearing plate/support data in the literature were primarily based on specimens being bolted to the bearing plate/support, hence, a few control tests were carried out by Schuster, the results of which are contained in (Beshara 1999), using self-drilling screws to establish the web crippling integrity in comparison to the bolted data. Based on these tests, the specimens with self-drilling screws performed equally well in comparison to the specimens with bolts. Fastened to the bearing plate/support in practice can be achieved by either using bolts, self-drilling/self-tapping screws or by welding. What is important is that the flange elements are restrained from rotating at the location of load application. In fact, in most cases, the flanges are frequently completely restrained against rotation by some type of sheathing material that is attached to the flanges.

The data was further separated in the *Specification* based on section type, as follows.

1) Built-up sections (Table C3.4.1-1);

2) Single web channel and C-sections (Table C3.4.1-2);

3) Single web Z-sections (Table C3.4.1-3);

4) Single hat sections (Table C3.4.1-4); and

5) Multi-web deck sections (Table C3.4.1-5).

In the case of unfastened built-up members such as I-sections (not fastened to the bearing plate/support), the available data was for specimens that were fastened together with a row of fasteners near each flange line of the member (Winter and Pian 1946) and Hetrakul and Yu (1978) as shown in Figure C-C3.4.1-5(a). For the fastened built-up member data of I-sections (fastened to the bearing plate/support), the specimens were fastened together with two rows of fasteners located symmetrically near the centerline length of the member, as shown in Figure C-C3.4.1-5(b) (Bhakta, LaBoube and Yu, 1992).

Calibrations were carried out by Beshara and Schuster (2000) in accordance with Supornsilaphachai, Galambos and Yu (1979) to establish the safety factors, Ω, and the resistance factors, ϕ, for each web crippling case. Based on these calibrations, different safety factors and corresponding resistance factors are presented in the web crippling coefficient tables for the particular load case and section type. In 2005, the safety and the resistance factors for built-up and single hat sections with interior one-flange loading case have been revised based on a more consistent calibration. For the fastened built-up sections, the factors were revised from 1.65 to 1.75 (for ASD), 0.90 to 0.85 (for LRFD) and 0.80 to 0.75 (for LSD). For the fastened single hat section, the factors were revised from 1.90 to 1.80 (for ASD) and 0.80 to 0.85 (for LRFD). For the unfastened single hat sections, the factors were revised from 1.70 to 1.80 (for ASD), 0.90 to 0.80 (for LRFD) and 0.75 to 0.70 (for LSD). Also in 2005 the coefficients for built-up sections were revised to remove inconsistencies between unfastened and fastened conditions and to better reflect the calibration for the safety factor and the resistance factors. Also, a minimum bearing length of 3/4 in. (19 mm) was introduced based on the data used in the development of the web crippling coefficients. For fastened to support single web C- and Z-section members under interior two-flange loading or reaction, the distance from the edge of bearing to the end of the member (Figure C-C3.4.1-2(d)) must be extended at least 2.5h. This requirement is necessary because a total of 5h specimen length was used for the test setup shown in Figure C-C3.4.1-2(d) (Beshara, 1999). The 2.5h length is conservatively taken from the edge of bearing rather than the centerline of bearing.

The assumed distributions of loads or reactions acting on the web of a member, as shown in Figure C-C3.4.1-4, are independent of the flexural response of the member. Due to the flexural action, the point of bearing will vary relative to the plane of bearing, resulting in a non-uniform bearing load distribution on the web. The value of P_n will vary because of a transition from the interior one-flange loading (Figure C3.4.1-4(b)) to the end one-flange loading (Figure C3.4.1-4(a)) condition. These discrete conditions represent the experimental basis on which the design provisions were founded (Winter, 1970; Hetrakul and Yu, 1978). Based on additional updated calibrations, the resistance factor for Canada LSD for the unfastened interior one-flange loading (IOF) case in Table C3.4.1-4 was changed from 0.75 to 0.70 in 2004.

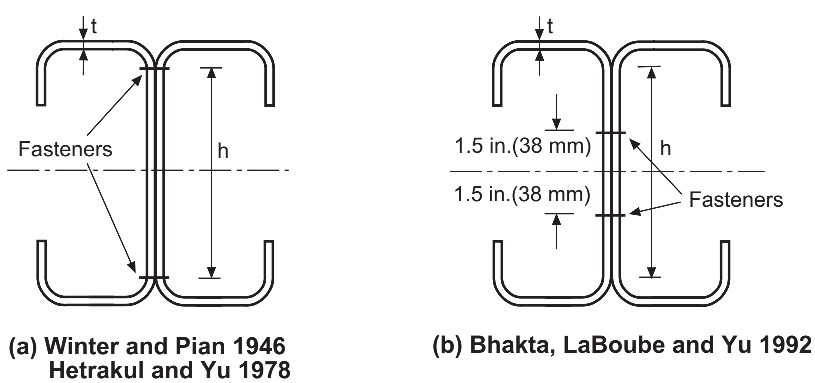


Figure C-C3.4.1-5 Typical Bolt Pattern for I-Section Test Specimens

The research indicates that a Z-section having its end support flange bolted to the section's supporting member through two 1/2-in. (12.7 mm) diameter bolts will experience an increase in end-one-flange web crippling capacity (Bhakta, LaBoube and Yu, 1992; Cain, LaBoube and Yu, 1995). The increase in load-carrying capacity was shown to range from 27 to 55 percent for the sections under the limitations prescribed in the *Specification*. A lower bound value of 30 percent increase was permitted in *Specification* Section C3.4 of the 1996 *Specification*. This is now incorporated under "Fastened to Support" condition.

In 2005, the R/t limit in Table C3.4.1-3 regarding Interior-one-flange loading for fastened Z-sections was changed from 5 to 5.5 to achieve consistency with *Specification* Equations C3.5.1-3 and C3.5.2-3 which stipulate a limit of R/t = 5.5.

For two nested Z-sections, the 1996 AISI *Specification* permitted the use of a slightly different safety factor and resistance factor for the interior one flange loading condition. This is no longer required since the new web crippling approach now takes this into account in Table C3.4.1-3 of the *Specification* under the category of "Fastened to Support" for the interior one flange loading case.

The previous web crippling coefficients in Table C3.4.1-5 for end one flange loading (EOF) of multi-web deck sections in the design provisions (AISI 2001) were based on limited data. This data was based on specimens that were not fastened to the support during testing, hence, the previous coefficients for this case were also being used conservatively for the case of fastened to the support. Based on extensive testing, web crippling coefficients were developed by James A. Wallace (2003) for both the unfastened and fastened case of EOF loading. Calibrations were also carried out to establish the respective safety factors and resistance factors.

In 2004, the definitions of "one-flange loading" and "two-flange loading" were revised according to the test setup, specimen lengths, development of web crippling coefficients, and calibration of safety factors and resistance factors. In Figures C-C3.4.1-3 and C-C3.4.1-4 of the *Commentary*, the distances from the edge of bearing to the end of the member were revised to be consistent with the *Specification*.

Specification Equation C3.4.1-2 for single web C- and Z-sections with an overhang or overhangs is based on a study of the behavior and resultant failure loads from an end-one-flange loading investigation performed at the University of Missouri-Rolla (Holesapple and LaBoube, 2002). This Equation is applicable within the limits of the investigation. The UMR test results indicated that in some situations with overhangs, the interior-one-flange load capacity may not be achieved and the interior-one-flange loading condition was therefore removed from Figures C-C3.4.1-3 and C-C3.4.1-4.

C3.4.2 Web Crippling Strength [Resistance] of C-Section Webs with Holes

Studies by Langan et al. (1994), Uphoff (1996) and Deshmukh (1996) quantified the reduction in web crippling capacity when a hole is present in a web element. These studies investigated both the end-one-flange and interior-one-flange loading conditions for h/t and d_h/h ratios as large as 200 and 0.81, respectively. The studies revealed that the reduction in web crippling strength is influenced primarily by the size of the hole as reflected in the d_h/h ratio and the location of the hole, x/h ratio.

The provisions for circular and non-circular holes also apply to any hole pattern that fits within an equivalent virtual hole. Figure C-B2.4-1 illustrates the L_h and d_h that may be used for a multiple hole pattern that fits within a non-circular virtual hole. Figure C-B2.4-2 illustrates the d_h that may be used for a rectangular hole that fits within a circular virtual hole. For each case, the design provisions apply to the geometry of the virtual hole geometry, not the actual hole or holes.

C3.5 Combined Bending and Web Crippling

C3.5.1 ASD Method

This *Specification* contains interaction equations for the combination of bending and web crippling. *Specification* Equations C3.5.1-1 and C3.5.1-2 are based on an evaluation of available experimental data using the web crippling equation included in the 2001 edition of the *Specification* (LaBoube, Schuster, and Wallace, 2002). The experimental data is based on research studies conducted at the University of Missouri-Rolla (Hetrakul and Yu, 1978 and 1980; Yu, 1981 and 2000), Cornell University (Winter and Pian, 1946), and the University of Sydney (Young and Hancock, 2000). For embossed webs, crippling strength [resistance] should be determined by tests according to *Specification* Chapter F.

The exception clause included in *Specification* Section C3.5.1 for single unreinforced webs applies to the interior supports of continuous spans using decks and beams, as shown in Figure C-C3.5-1. Results of continuous beam tests of steel decks (Yu, 1981) and several independent studies by manufacturers indicate that, for these types of members, the postbuckling behavior of webs at interior supports differs from the type of failure mode occurring under concentrated loads on single span beams. This postbuckling strength [resistance] enables the member to redistribute the moments in continuous spans. For this reason, *Specification* Equation C3.5.1-1 is not applicable to the interaction between bending and the reaction at interior supports of continuous spans. This exception clause applies only to the members shown in Figure C-C3.5-1 and similar situations explicitly described in *Specification* Section C3.5.1.

The exception clause should be interpreted to mean that the effects of combined bending and web crippling need not be checked for determining load-carrying capacity. Furthermore the positive bending resistance of the beam should be at least 90 percent of the negative bending resistance in order to insure the safety implied by the *Specification*.

Using this procedure the service loads may (1) produce slight deformations in the member over the support, (2) increase the actual compressive bending stresses over the support to as high as 0.8 F_y, and (3) result in additional bending deflection of up to 22 percent due to elastic moment redistribution.

If load-carrying capacity is not the primary design concern because of the above behavior, the designer is urged to use *Specification* Equation C3.5.1-1.

In 1996, additional design information was added to *Specification* Section C3.5.1(c) for two nested Z-shapes. These design provisions are based on the research conducted at the University of Wisconsin-Milwaukee, University of Missouri-Rolla, and a metal building manufacturer (LaBoube, Nunnery and Hodges, 1994). The web crippling and bending behavior of unreinforced nested web elements is enhanced because of the interaction of the nested webs. The design equation is based on the experimental results obtained from 14 nested web configurations. These configurations are typically used by the metal building industry.

Based on the test data of LaBoube, Nunnery, and Hodges (1994), in 2003, the interaction equation for the combined effects of bending and web crippling was re-evaluated because new web crippling equation was adopted for Section C3.4.1 of the *Specification*.

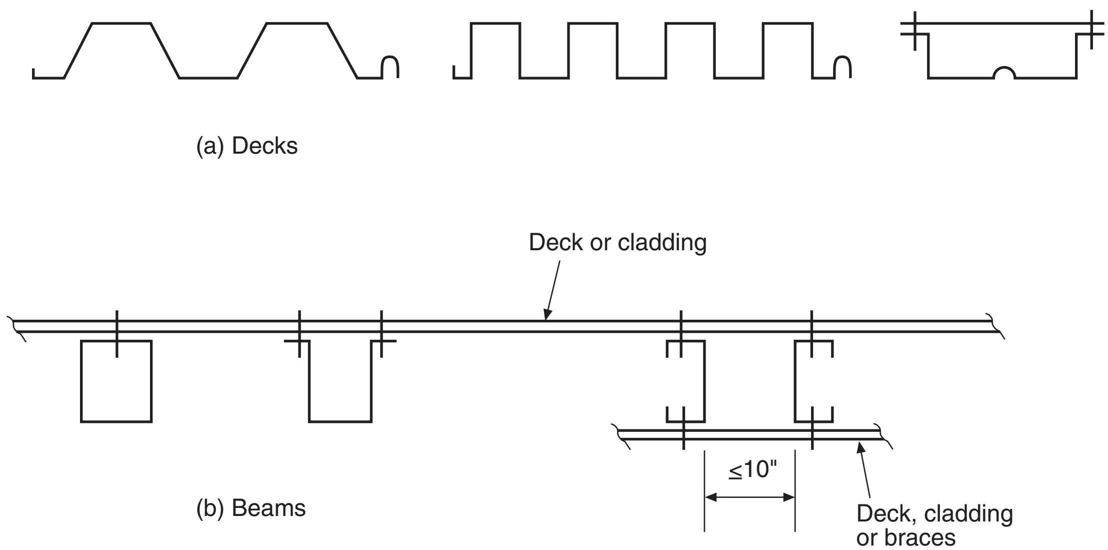


Figure C-C3.5-1 Sections Used for Exception Clause of Specification Section C3.5

C3.5.2 LRFD and LSD Methods

For the load and resistance factor design and the limit states design methods, *Specification* Equations C3.5.2-1 and C3.5.2-2 are based on the same equations as used for ASD using the required and design strengths. In both equations, different symbols are used for the required strength [resistance] for the concentrated load or reaction due to factored loads, and the required flexural strength [factored moment] according to the LRFD and the LSD methods.

In the development of the original LRFD equations, a total of 551 tests were calibrated for combined bending and web crippling strength [resistance]. Based on ϕ_w = 0.75 for single unreinforced webs and ϕ_w = 0.80 for I-sections, the values of reliability index vary from 2.5 to 3.3 as summarized in the AISI *Commentary* (AISI, 1991).

For two nested Z-shapes, *Specification* Equation C3.5.2-3 was derived from the same research work discussed in Section C3.5.1 for *Specification* Equation C3.5.1-3.

C3.6 Combined Bending and Torsional Loading

When the transverse loads applied to a bending member do not pass through the shear center of the cross-section of the member, twisting and torsional stresses can develop. The torsional stresses consist of pure torsional shear stresses, shear stresses due to warping and normal stresses due to warping. References such as the AISC Steel Design Guide (AISC, 1997a) "Torsional Analysis of Structural Steel Members" describe the effect of torsion and how these stresses may be calculated.

Open cold-formed steel sections have little resistance to torsion, thus severe twisting and large stresses can develop. In many situations, however, the connection between a beam and the member delivering the load to the beam is such that it constrains twisting and in effect causes the resultant load to act as though it is delivered through the shear center. In such cases the torsional effects do not occur. Positive connections between the load bearing flange and supported elements, in general, prevent torsional effects. An example of this is a purlin supporting a through fastened roof panel that will prevent movement in the plane of the roof panel. It is important that the designer ensure that torsion is adequately constrained when evaluating a specific situation.

In situations where torsional loading cannot be avoided, discrete bracing will reduce the torsional effects. Torsional bracing at the third points of the span would be adequate for most light construction applications. The bracing should be designed to prevent torsional twisting at the braced points.

Specification Section C3.6 provides design criteria for a member that is subjected to torsional loading. The provision uses a reduction factor to reduce the nominal moment strength as determined by *Specification* Section C3.1.1(a) This reduction factor requires calculation of both the usual bending stresses and the torsional warping stresses at critical points on the cross-section. The largest combination of these is the denominator of the reduction factor while the bending stress alone at this same point is the numerator. The member is then selected based on bending alone with the effect of torsion accounted for by the reduction in the nominal moment capacity.

The largest combination of compression stresses on the cross section may occur at the junction of the web and flange or at the junction of the edge of flange and flange stiffener. The second condition has the more severe effect on reducing the moment capacity of the member. This can occur when the applied load is positioned off the member away from both the web and the shear center. This is shown from the test results reported in the referenced paper by Bogdan M. Put and others (Put et al., 1999). This is not a practical situation for structural assemblies, however this location of the critical compression stresses would occur at the position of a torsional brace located at mid-span of a member. To allow for the more favorable situation, the provisions of *Specification* Section C3.6 permit the moment capacity to be increased by 15% when the critical combination of compressive stresses occurs at the junction of the flange and web. This is also supported by tests on channels conducted by Winter in 1950 (Winter et al., 1950), which indicated that an overstress of 15% did not significantly affect the carrying capacity.

The provisions of this Section need not be used in combination with the bending provisions in *Specification* Sections D6.1.1 and D6.1.2 since these provisions are based on tests in which torsional effects were present.

C3.7 Stiffeners

C3.7.1 Bearing Stiffeners

Design requirements for attached bearing stiffeners (previously called transverse stiffeners) and for shear stiffeners were added in the 1980 AISI *Specification* and were unchanged in the 1986 *Specification*. The same design equations are retained in Section C3.7 of the current *Specification*. The term "transverse stiffener" was renamed to "bearing stiffeners" in 2004. The nominal strength [resistance] equation given in Item (a) of *Specification* Section C3.7.1 serves to prevent end crushing of the bearing stiffeners, while the nominal strength [resistance] equation given in Item (b) is to prevent column-type buckling of the web-stiffeners. The equations for computing the effective areas (A_b and A_c) and the effective widths (b_1 and b_2) were adopted from Nguyen and Yu (1978a) with minor modifications.

The available experimental data on cold-formed steel bearing stiffeners were evaluated by Hsiao, Yu and Galambos (1988a). A total of 61 tests were examined. The resistance factor of 0.85 used for the LRFD method was selected on the basis of the statistical data. The corresponding reliability indices vary from 3.32 to 3.41.

In 1999, the upper limit of w/t_s ratio for the unstiffened elements of cold-formed steel bearing stiffeners was revised from $0.37\sqrt{E/F_{ys}}$ to $0.42\sqrt{E/F_{ys}}$ for the reason that the former was calculated based on the allowable strength design approach, while the latter is based on the effective area approach. The revision provided the same basis for the stiffened and unstiffened elements of cold-formed steel bearing stiffeners.

C3.7.2 Bearing Stiffeners in C-Section Flexural Members

The provisions of this section are based on the research by Fox and Schuster (2002), which investigated the behavior of stud and track type bearing stiffeners in cold-formed steel C-section flexural members. These stiffeners fall outside of the scope of *Specification* Section C3.7.1. The research program investigated bearing stiffeners subjected to two-flange loading at both interior and end locations, and with the stiffener positioned between the member flanges and on the back of the member. A total of 263 tests were carried out on different stiffened C-section assemblies. The design expression in *Specification* Section C3.7.2 is a simplified method applicable with the limits of the test program. A more detailed beam-column design method is described in Fox (2002).

C3.7.3 Shear Stiffeners

The requirements for shear stiffeners included in *Specification* Section C3.7.3 were primarily adopted from the AISC Specification (1978). The equations for determining the minimum required moment of inertia (*Specification* Equation C3.7.3-1) and the minimum required gross area (*Specification* Equation C3.7.3-2) of attached shear stiffeners are based on the studies summarized by Nguyen and Yu (1978a). In *Specification* Equation C3.7.3-1, the minimum value of $(h/50)^4$ was selected from the AISC Specification (AISC, 1978).

For the LRFD method, the available experimental data on the shear strength [resistance] of beam webs with shear stiffeners were calibrated by Hsiao, Yu and Galambos (1988a). The statistical data used for determining the resistance factor were summarized in the AISI *Design Manual* (AISI, 1991). Based on these data, the reliability index was found to be 4.10 for $\phi = 0.90$.

C3.7.4 Non-Conforming Stiffeners

Tests on rolled-in stiffeners covered in *Specification* Section C3.7.4 were not conducted in the experimental program reported by Nguyen and Yu (1978). Lacking reliable information, the available strength [resistance] of stiffeners should be determined by special tests.

C4 Concentrically Loaded Compression Members

Axially loaded compression members should be designed for the following limit states depending on the configuration of the cross-section, thickness of material, unbraced length, and end restraint: (1) yielding, (2) overall column buckling (flexural buckling, torsional buckling, or flexural-torsional buckling), (3) local buckling of individual elements, and (4) distortional buckling. The first three limit states are discussed in Section C4.1 and ditortional buckling limit state is discussed in Section C4.2. For the design tables and example problems on columns, see Parts I and III of the AISI *Cold-Formed Steel Design Manual* (AISI, 2008).

C4.1 Nominal Strength for Yielding, Flexural, Flexural-Torsional and Torsional Buckling

In this section, the limit states of yielding and overall column buckling are discussed.

A. *Yielding*

It is well known that a very short, compact column under an axial load may fail by yielding. The yield load is determined by Equation C-C4.1-1:

$$P_y = A_g F_y \tag{C-C4.1-1}$$

where A_g is the gross area of the column and F_y is the yield stress of steel.

B. *Flexural Buckling of Columns*

(a) *Elastic Buckling Stress*

A slender, axially loaded column may fail by overall flexural buckling if the cross-section of the column is a doubly-symmetric shape, closed shape (square or rectangular tube), cylindrical shape, or point-symmetric shape. For singly-symmetric shapes, flexural buckling is one of the possible failure modes. Wall studs connected with sheathing material can also fail by flexural buckling.

The elastic critical buckling load for a long column can be determined by the following Euler equation:

$$(P_{cr})_e = \frac{\pi^2 EI}{(KL)^2} \tag{C-C4.1-2}$$

where $(P_{cr})_e$ is the column buckling load in the elastic range, E is the modulus of elasticity, I is the moment of inertia, K is the effective length factor, and L is the unbraced length. Accordingly, the elastic column buckling stress is

$$(F_{cr})_e = \frac{(P_{cr})_e}{A_g} = \frac{\pi^2 E}{(KL/r)^2} \tag{C-C4.1-3}$$

in which r is the radius of gyration of the full cross section, and KL/r is the effective slenderness ratio.

(b) *Inelastic Buckling Stress*

When the elastic column buckling stress computed by Equation C-C4.1-3 exceeds the proportional limit, F_{pr}, the column will buckle in the inelastic range. Prior to 1996, the following equation was used in the AISI *Specification* for computing the inelastic column buckling stress:

$$(F_{cr})_I = F_y\left(1 - \frac{F_y}{4(F_{cr})_e}\right) \tag{C-C4.1-4}$$

It should be noted that because the above equation is based on the assumption that $F_{pr} = F_y/2$, it is applicable only for $(F_{cr})_e \geq F_y/2$.

By using λ_c as the column slenderness parameter instead of slenderness ratio, KL/r, Equation C-C4.1-4 can be rewritten as follows:

$$(F_{cr})_I = \left(1 - \frac{\lambda_c{}^2}{4}\right)F_y \qquad \text{(C-C4.1-5)}$$

where

$$\lambda_c = \sqrt{\frac{F_y}{(F_{cr})_e}} = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} \qquad \text{(C-C4.1-6)}$$

Accordingly, Equation C-C4.1-5 is applicable only for $\lambda_c \le \sqrt{2}$.

(c) *Nominal Axial Strength [Compressive Resistance] for Locally Stable Columns*

If the individual components of compression members have small w/t ratios, local buckling will not occur before the compressive stress reaches the column buckling stress or the yield stress of steel. Therefore, the nominal axial strength [compressive resistance] can be determined by the following equation:

$$P_n = A_g F_{cr} \qquad \text{(C-C4.1-7)}$$

where
P_n = nominal axial strength
A_g = gross area of the column
F_{cr} = column buckling stress

(d) *Nominal Axial Strength [Compressive Resistance] for Locally Unstable Columns*

For cold-formed steel compression members with large w/t ratios, local buckling of individual component plates may occur before the applied load reaches the nominal axial strength [compressive resistance] determined by Equation C-C4.1-7. The interaction effect of the local and overall column buckling may result in a reduction of the overall column strength [resistance]. From 1946 through 1986, the effect of local buckling on column strength was considered in the AISI *Specification* by using a form factor Q in the determination of allowable stress for the design of axially loaded compression members (Winter, 1970; Yu, 2000). Even though the Q-factor method was used successfully for the design of cold-formed steel compression members, research work conducted at Cornell University and other institutions have shown that this method is capable of improvement. On the basis of the test results and analytical studies of DeWolf, Pekoz, Winter, and Mulligan (DeWolf, Pekoz and Winter, 1974; Mulligan and Pekoz, 1984) and Pekoz's development of a unified approach for the design of cold-formed steel members (Pekoz, 1986b), the Q-factor method was eliminated in the 1986 edition of the AISI *Specification*. In order to reflect the effect of local buckling on the reduction of column strength, the nominal axial strength [compressive resistance] is determined by the critical column buckling stress and the *effective area*, A_e, instead of the full sectional area. When A_e cannot be calculated, such as when the compression member has dimensions or geometry beyond the range of applicability of the AISI *Specification*, the effective area A_e can be determined experimentally by stub column tests using the procedure given in Part VI of the AISI *Design Manual* (AISI, 2008). For a more in-depth discussion of the background for these provisions, see Pekoz (1986b). Therefore, the nominal axial strength [compressive resistance] of cold-formed steel compression members can be determined by the following equation:

$$P_n = A_eF_{cr} \tag{C-C4.1-8}$$

where F_{cr} is either elastic buckling stress or inelastic buckling stress whichever is applicable, and A_e is the effective area at F_{cr}.

In the 1986 edition of the AISI *Specification*, the nominal axial strength [resistance] for C-

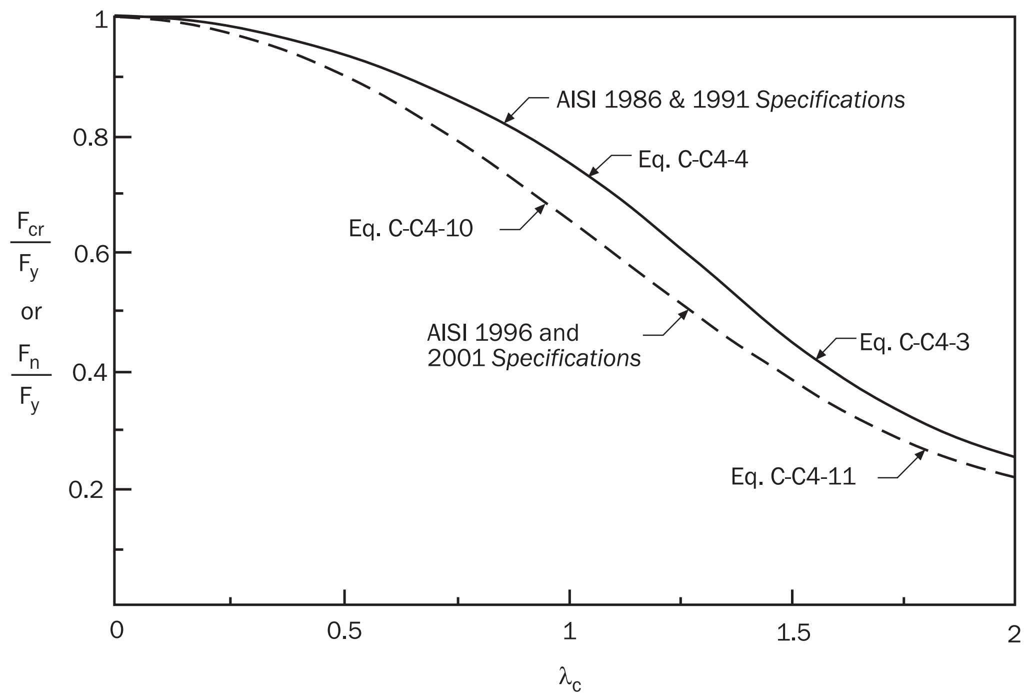


Figure C-C4.1-1 Comparison between the Critical Buckling Stress Equations

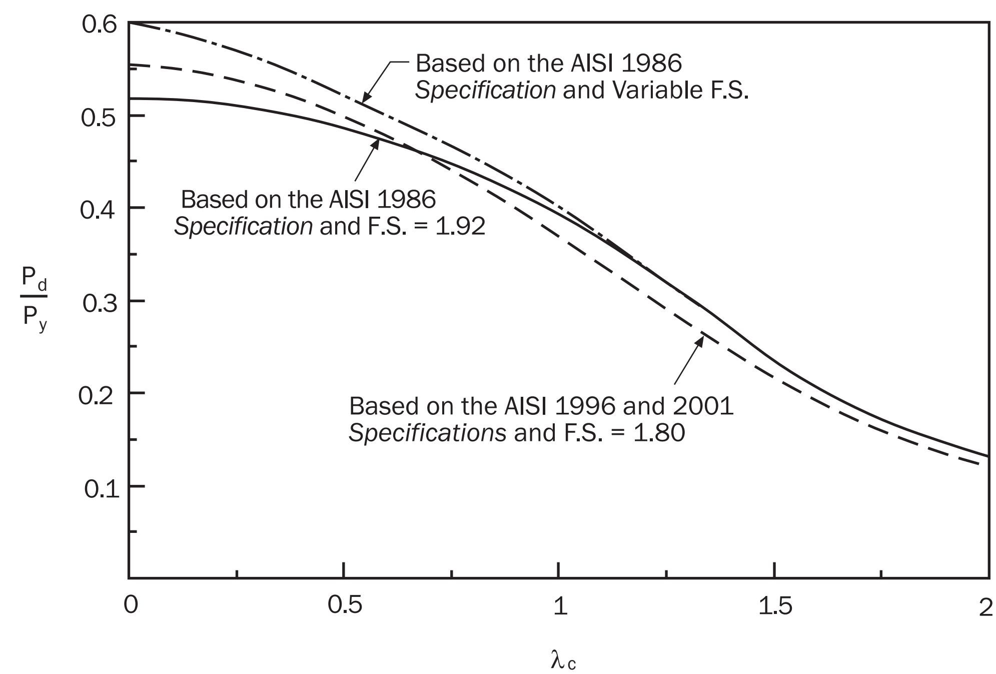


Figure C-C4.1-2 Comparison between the Design Axial Strengths [Resistances], P_d

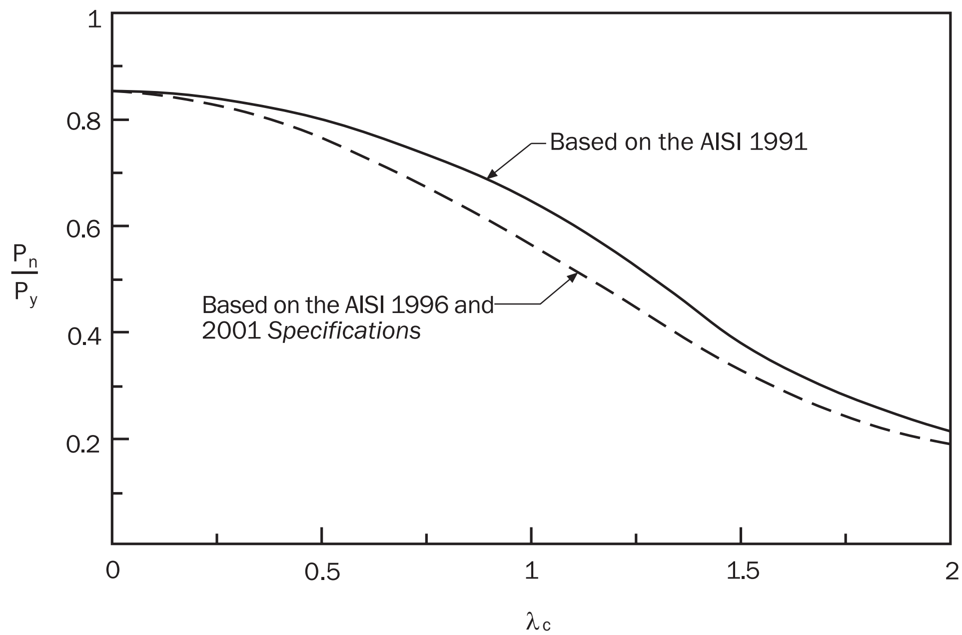


Figure C-C4.1-3 Comparison between the Nominal Axial Strengths [Resistances], P_n

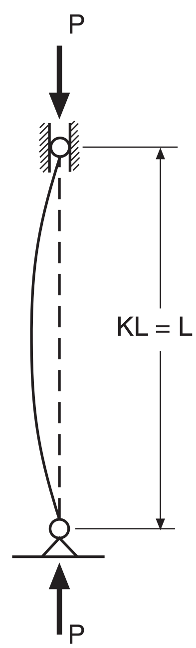


Figure C-C4.1-4 Overall Column Buckling

and Z-sections and single angle sections was limited by Equation C-C4.1-9, which is determined by the local buckling stress of the unstiffened element and the area of the full cross-section:

$$P_n = \frac{A\pi^2 E}{25.7(w/t)^2} \qquad \text{(C-C4.1-9)}$$

This equation was deleted since the 1996 edition of the *Specification* based on a study conducted by Rasmussen at the University of Sydney (Rasmussen, 1994) and validated by Rasmussen and Hancock (1992).

In the 1996 AISI *Specification*, the design equations for calculating the inelastic and elastic flexural buckling stresses have been changed to those used in the AISC LRFD *Specification* (AISC, 1993). As given in the *Specification* Section C4.1(a), these design equations are as follows:

For $\lambda_c \leq 1.5$: $F_n = (0.658^{\lambda_c^2})F_y$ (C-C4.1-10)

For $\lambda_c > 1.5$: $F_n = \left[\frac{0.877}{\lambda_c^2}\right]F_y$ (C-C4.1-11)

where F_n is the nominal flexural buckling stress which can be either in the elastic range or in the inelastic range depending on the value of $\lambda_c = \sqrt{F_y / F_e}$, and F_e is the elastic flexural buckling stress calculated by using Equation C-C4.1-3. Consequently, the equation for determining the nominal axial strength [compressive resistance] can be written as

$P_n = A_e F_n$ (C-C4.1-12)

which is Equation C4.1-1 of the *Specification*.

The reasons for changing the design equations from Equation C-C4.1-4 to Equation C-C4.1-10 for inelastic buckling stress and from Equation C-C4.1-3 to Equation C-C4.1-11 for elastic buckling stress are:

1. The revised column design equations (Equations C-C4.1-10 and C-C4.1-11) are based on a different basic strength [resistance] model and were shown to be more accurate by Pekoz and Sumer (1992). In this study, 299 test results on columns and beam-columns were evaluated. The test specimens included members with component elements in the post-local buckling range as well as those that were locally stable. The test specimens included members subject to flexural buckling as well as flexural-torsional buckling.
2. Because the revised column design equations represent the maximum strength [resistance] with due consideration given to initial crookedness and can provide the better fit to test results, the required safety factor can be reduced. In addition, the revised equations enable the use of a single safety factor for all λ_c values even though the nominal axial strength [compressive resistance] of columns decreases as the slenderness increases because of initial out-of-straightness. By using the selected safety factor and resistance factor, the results obtained from the ASD and LRFD approaches would be approximately the same for a live-to-dead load ratio of 5.0.

The design provisions included in the AISI ASD *Specification* (AISI, 1986), the LRFD *Specification* (AISI, 1991), the 1996 *Specification* and the current *Specification* (AISI, 2001, 2007) are compared in Figures C-C4.1-1, C-C4.1-2, and C-C4.1-3.

Figure C-C4.1-1 shows a comparison of the critical flexural buckling stresses used in the 1986, 1991, 1996, 2001 and 2007 *Specifications*. The equations used to plot these two curves are indicated in the figure. Because of the use of a relatively smaller safety factor in the 1996, 2001 and 2007 *Specifications*, it can be seen from Figure C-C4.1-2 that the design capacity is increased for thin columns with low slenderness parameters and decreased for high slenderness parameters. However, the differences would be less than 10%. For the LRFD method, the differences between the nominal axial strengths [compressive resistances] used for the 1991, 1996, 2001 and the 2007 LRFD design provisions are shown in Figure C-C4.1-3. The curve for the LSD provisions would be the same as the curve for LRFD.

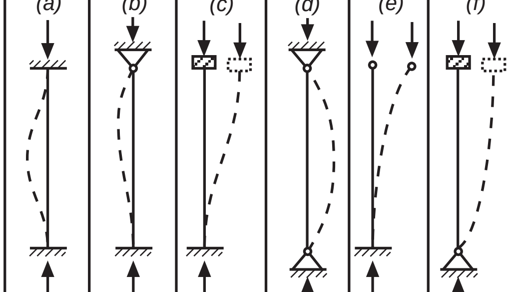
(e) Effective Length Factor, K

The effective length factor K accounts for the influence of restraint against rotation and translation at the ends of a column on its load-carrying capacity. For the simplest case, a column with both ends hinged and braced against lateral translation, buckling occurs in a single half-wave and the effective length KL, being the length of this half-wave, is equal to the actual physical length of the column (Figure C-C4.1-4); correspondingly, for this case, K = 1. This situation is approached if a given compression member is part of a structure which is braced in such a manner that no lateral translation (sidesway) of one end of the column relative to the other can occur. This is so for columns or studs in a structure with diagonal bracing, diaphragm bracing, shear-wall construction or any other provision which prevents horizontal displacement of the upper relative to the lower column ends. In these situations it is safe and only slightly, if at all, conservative to take K = 1.

If translation is prevented and abutting members (including foundations) at one or both ends of the member are rigidly connected to the column in a manner which provides substantial restraint against rotation, K-values smaller than 1 (one) are sometimes justified. Table C-C4.1-1 provides the theoretical K values for six idealized conditions in which joint rotation and translation are either fully realized or nonexistent. The same table also includes the K values recommended by the Structural Stability Research Council for design use (Galambos, 1998).

In trusses, the intersection of members provides rotational restraint to the compression members at service loads. As the collapse load is approached, the member stresses

Table C-C4.1-1
Effective Length Factors K for Concentrically Loaded Compression Members

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

approach the yield stress which greatly reduces the restraint they can provide. For this reason K value is usually taken as unity regardless of whether they are welded, bolted, or connected by screws. However, when sheathing is attached directly to the top flange of a continuous compression chord, research (Harper, LaBoube and Yu, 1995) has shown that the K values may be taken as 0.75 (AISI, 1995).

On the other hand, when no lateral bracing against sidesway is present, such as in the portal frame of Figure C-C4.1-5, the structure depends on its own bending stiffness for lateral stability. In this case, when failure occurs by buckling of the columns, it invariably takes place by the sidesway motion shown. This occurs at a lower load than the columns would be able to carry if they where braced against sidesway and the figure shows that

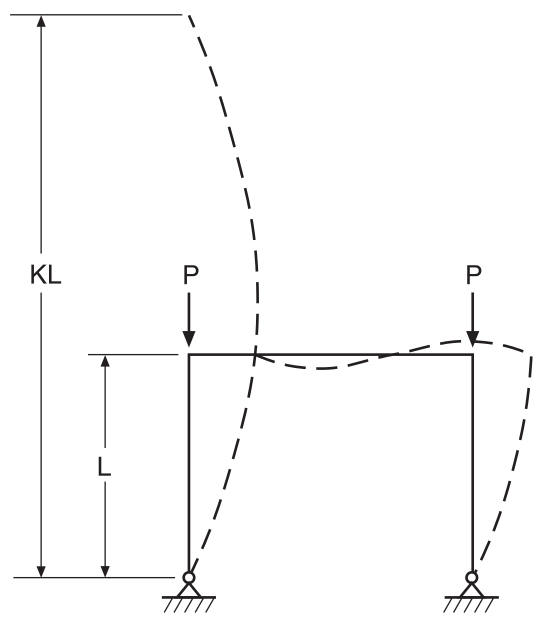


Figure C-C4.1-5 laterally Unbraced Portal Frame

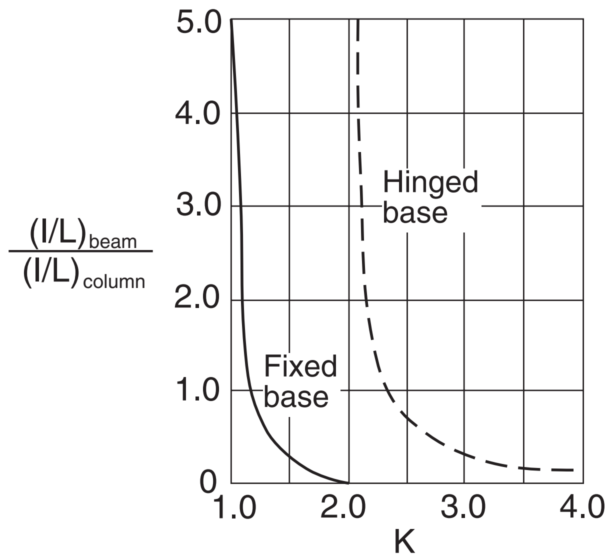


Figure C-C4.1-6 Effective Length Factor K in Laterally Unbraced Portal Frames

the half-wave length into which the columns buckle is longer than the actual column length. Hence, in this case K is larger than 1 (one) and its value can be read from the graph of Figure C-C4.1-6 (Winter et al., 1948a and Winter, 1970). Since column bases are rarely either actually hinged or completely fixed, K-values between the two curves should be estimated depending on actual base fixity.

Figure C-C4.1-6 can also serve as a guide for estimating K for other simple situations. For multi-bay and/or multi-story frames, simple alignment charts for determining K are given in the AISC Commentaries (AISC, 1989; 1999; 2005). For additional information on frame stability and second order effects, see SSRC *Guide to Stability Design Criteria for Metal Structures* (Galambos, 1998) and the AISC Specifications and Commentaries.

If roof or floor slabs, anchored to shear walls or vertical plane bracing systems, are counted upon to provide lateral support for individual columns in a building system, their stiffness must be considered when functioning as horizontal diaphragms (Winter, 1958a).

C. *Torsional Buckling of Columns*

It was pointed out at the beginning of this section that purely torsional buckling, i.e., failure by sudden twist without concurrent bending, is also possible for certain cold-formed open shapes. These are all point-symmetric shapes (in which shear center and centroid coincide), such as doubly-symmetric I-shapes, anti-symmetric Z-shapes, and such unusual sections as cruciforms, swastikas, and the like. Under concentric load, torsional buckling of such shapes very rarely governs design. This is so because such members of realistic slenderness will buckle flexurally or by a combination of flexural and local buckling at loads smaller than those which would produce torsional buckling. However, for relatively short members of this type, carefully dimensioned to minimize local buckling, such torsional buckling cannot be completely ruled out. If such buckling is elastic, it occurs at the critical stress σ_t calculated as follows (Winter, 1970):

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \qquad \text{(C-C4.1-13)}$$

The above equation is the same as *Specification* Equation C3.1.2.1-9, in which A is the full cross-sectional area, r_o is the polar radius of gyration of the cross section about the shear center, G is the shear modulus, J is Saint-Venant torsion constant of the cross section, E is the modulus of elasticity, C_w is the torsional warping constant of the cross section, and K_t L_t is the effective length for twisting.

For inelastic buckling, the critical torsional buckling stress can also be calculated according to Equation C-C4.1-10 by using σ_t as F_e in the calculation of λ_c.

D. *Flexural-Torsional Buckling of Columns*

As discussed previously, concentrically loaded columns can buckle in the flexural buckling mode by bending about one of the principal axes; or in the torsional buckling mode by twisting about the shear center; or in the flexural-torsional buckling mode by simultaneous bending and twisting. For singly-symmetric shapes such as channels, hat sections, angles, T-sections, and I-sections with unequal flanges, for which the shear center and centroid do not coincide, flexural-torsional buckling is one of the possible buckling modes as shown in Figure C-C4.1-7. Unsymmetric sections will always buckle in the flexural-torsional mode.

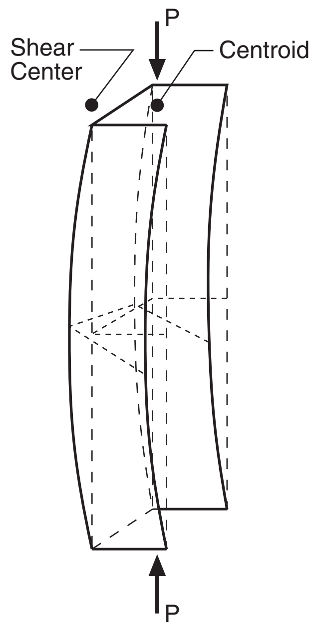


Figure C-C4.1-7 Flexural-Torsional Buckling of a Channel in Axial Compression

It should be emphasized that one needs to design for flexural-torsional buckling only when it is physically possible for such buckling to occur. This means that if a member is so connected to other parts of the structure such as wall sheathing that it can only bend but cannot twist, it needs to be designed for flexural buckling only. This may hold for the entire member or for individual parts. For instance, a channel member in a wall or the chord of a roof truss is easily connected to girts or purlins in a manner which prevents twisting at these connection points. In this case flexural-torsional buckling needs to be checked only for the unbraced lengths between such connections. Likewise, a doubly-symmetric compression member can be made up by connecting two spaced channels at intervals by batten plates. In this case each channel constitutes an "intermittently fastened component of a built-up shape." Here the entire member, being doubly-symmetric, is not subject to flexural-torsional buckling so that this mode needs to be checked only for the individual component channels between batten connections (Winter, 1970).

The governing elastic flexural-torsional buckling load of a column can be found from the following equation, (Chajes and Winter, 1965; Chajes, Fang and Winter, 1966; Yu, 2000):

$$P_n = \frac{1}{2\beta}\left[(P_x + P_z) - \sqrt{(P_x + P_z)^2 - 4\beta P_x P_z}\right] \qquad \text{(C-C4.1-14)}$$

If both sides of this equation are divided by the cross-sectional area A, one obtains the equation for the elastic, flexural-torsional buckling stress F_e as follows:

$$F_e = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right] \qquad \text{(C-C4.1-15)}$$

For this equation, as in all provisions which deal with flexural-torsional buckling, the x-axis is the axis of symmetry; $\sigma_{ex} = \pi^2 E/(K_x L_x/r_x)^2$ is the flexural Euler buckling stress about the x-axis, σ_t is the torsional buckling stress (Equation C-C4.1-13) and $\beta = 1-(x_o/r_o)^2$. It is worth noting that the flexural-torsional buckling stress is always lower than the Euler stress σ_{ex} for flexural buckling about the symmetry axis. Hence, for these singly-symmetric sections, flexural buckling can only occur, if at all, about the y-axis which is the principal axis perpendicular to the axis of symmetry.

For inelastic buckling, the critical flexural-torsional buckling stress can also be calculated by using Equation C-C4.1-10.

An inspection of Equation C-C4.1-15 will show that in order to calculate β and σ_t, it is necessary to determine x_o = distance between shear center and centroid, J = Saint-Venant torsion constant, and C_w = warping constant, in addition to several other, more familiar cross-sectional properties. Because of these complexities, the calculation of the flexural-torsional buckling stress cannot be made as simple as that for flexural buckling. Formulas for typical C-, Z-sections, angle and hat sections are provided in Part I of the *Design Manual* (AISI, 2008).

For one thing, any singly-symmetric shape can buckle either flexurally about the y-axis or flexural-torsionally, depending on its detailed dimensions. For instance, a channel stud with narrow flanges and wide web will generally buckle flexurally about the y-axis (axis parallel to web); in contrast a channel stud with wide flanges and a narrow web will generally fail in flexural-torsional buckling. If flexural-torsional buckling is indicated, the information and design aids in Parts I and VII of the AISI *Design Manual* (AISI, 2008) facilitate and expedite the necessary calculations.

The above discussion refers to members subject to flexural-torsional buckling, but made up of elements whose w/t ratios are small enough so that no local buckling will occur. For shapes which are sufficiently thin, i.e., with w/t ratios sufficiently large, local buckling can combine with flexural-torsional buckling similar to the combination of local with flexural buckling. For this case, the effect of local buckling on the flexural-torsional buckling strength can also be handled by using the effective area, A_e, determined at the stress F_n for flexural-torsional buckling.

E. Additional Design Consideration for Angles

During the development of a unified approach to the design of cold-formed steel members, Pekoz realized the possibility of a reduction in column strength due to initial sweep (out-of-straightness) of angle sections. Based on an evaluation of the available test results, an initial out-of-straightness of L/1000 was recommended by Pekoz for the design of concentrically loaded compression angle members and beam-columns in the 1986 edition of the AISI *Specification*. Those requirements were retained in Sections C4.1, C5.2.1, and C5.2.2 of the 1996 edition of the *Specification*. A study conducted at the University of Sydney (Popovic, Hancock, and Rasmussen, 1999) indicated that for the design of singly-symmetric unstiffened angles sections under the axial compression load, the required additional moment about the minor principal axis due to initial sweep should only be applied to those angle sections, for which the effective area at stress F_y is less than the full, unreduced cross-sectional area. Consequently, clarifications have been made in Sections C5.2.1 and C5.2.2 of the 2001 edition of the AISI *Specification* to reflect the research findings.

F. Slenderness Ratios

The slenderness ratio, KL/r, of all compression members preferably should not exceed 200, except that during construction only, KL/r should not exceed 300. In 1999, the above recommendations were moved from the *Specification* to the *Commentary*.

The maximum slenderness ratios on compression and tension members have been stipulated in steel design standards for many years but are not mandatory in the AISI *Specification*.

The KL/r limit of 300 is still recommended for most tension members in order to control serviceability issues such as handling, sag and vibration. The limit is not mandatory, however, because there are a number of applications where it can be shown that such factors are not detrimental to the performance of the structure or assembly of which the member is a part. Flat strap tension bracing is a common example of an acceptable type of tension member where the KL/r limit of 300 is routinely exceeded.

The compression member KL/r limits are recommended not only to control handling, sag and vibration serviceability issues but also to flag possible strength [resistance] concerns. The AISI *Specification* provisions adequately predict the capacities of slender columns and beam-columns but the resulting strengths [resistances] are quite small and the members relatively inefficient. Slender members are also very sensitive to eccentrically applied axial load because the moment magnification factors given by $1/\alpha$ will be large.

C4.1.1 Sections Not Subject to Torsional or Flexural-Torsional Buckling

If concentrically loaded compression members can buckle in the flexural buckling mode by bending about one of the principal axes, the nominal flexural buckling strength [resistance] of the column should be determined by using Equation C4.1-1 of the *Specification*. The elastic flexural buckling stress is given in Equation C4.1.1-1 of the *Specification*, which is the same as Equation C-C4.1-3 of the *Commentary*. This provision is applicable to doubly-symmetric sections, closed cross sections and any other sections not subject to torsional or flexural-torsional buckling.

C4.1.2 Doubly- or Singly-Symmetric Sections Subject to Torsional or Flexural-Torsional Buckling

As discussed previously in Section C4.1, torsional buckling is one of the possible buckling modes for doubly- and point-symmetric sections. For singly-symmetric sections, flexural-torsional buckling is one of the possible buckling modes. The other possible buckling mode is flexural buckling by bending about the y-axis (i.e., assuming x-axis is the axis of symmetry).

For torsional buckling, the elastic buckling stress can be calculated by using Equation C-C4.1-13. For flexural-torsional buckling, Equation C-C4.1-15 can be used to compute the elastic buckling stress. The following simplified equation for elastic flexural-torsional buckling stress is an alternative permitted by the AISI *Specification*:

$$F_e = \frac{\sigma_t \sigma_{ex}}{\sigma_t + \sigma_{ex}} \tag{C-C4.1-16}$$

The above equation is based on the following interaction relationship given by Pekoz and Winter (1969a):

$$\frac{1}{P_n} = \frac{1}{P_x} + \frac{1}{P_z} \tag{C-C4.1-17}$$

or

$$\frac{1}{F_e} = \frac{1}{\sigma_{ex}} + \frac{1}{\sigma_t} \tag{C-C4.1-18}$$

Research at the University of Sydney (Popovic, Hancock, and Rasmussen, 1999) has shown that singly-symmetric unstiffened cold-formed steel angles, which have a fully effective cross-section under yield stress, do not fail in a flexural-torsional mode and can be designed based on flexural buckling alone as specified in *Specification* Section C4.1.1. There is also no need to include a load eccentricity for these sections when using *Specification* Section C5.2.1 or Section C5.2.2 as explained in Item E of Section C4.1.

C4.1.3 Point-Symmetric Sections

This section of the *Specification* is for the design of discretely braced point-symmetric section subjected to axial compression. An example of a point-symmetric section is a lipped or unlipped Z-section with equal flanges. The critical elastic buckling stress of point-symmetric sections is the lesser of the two possible buckling modes, the elastic torsional buckling stress, σ_t, as defined in *Specification* Equation C3.1.2.1-9 or the elastic flexural buckling stress about its minor principal axis, as defined in *Specification* Equation C4.1.1-1. Figure C-D3.2.1-5 shows the relationship of the principal axes to the x and y axes of a lipped Z-section. The elastic flexural buckling stress should be calculated for axis 2.

C4.1.4 Nonsymmetric Sections

For nonsymmetric open shapes the analysis for flexural-torsional buckling becomes extremely tedious unless its need is sufficiently frequent to warrant computerization. For one thing, instead of the quadratic equations, cubic equations have to be solved. For another, the calculation of the required section properties, particularly C_w, becomes quite complex. The method of calculation is given in Parts I and V of the AISI *Design Manual* (AISI, 2008) and the book by Yu (2000). Section C4.1.4 of the *Specification* states that calculation according to this section shall be used or tests according to Chapter F shall be made when dealing with nonsymmetric open shapes.

C4.1.5 Closed Cylindrical Tubular Sections

Closed thin-walled cylindrical tubular members are economic sections for compression and torsional members because of their large ratio of radius of gyration to area, the same radius of gyration in all directions, and the large torsional rigidity. Like other cold-formed steel compression members, cylindrical tubes must be designed to provide adequate safety not only against overall column buckling but also against local buckling. It is well known that the classical theory of local buckling of longitudinally compressed cylinders overestimates the actual buckling strength [resistance] and that inevitable imperfections and residual stresses reduce the actual strength [resistance] of compressed tubes radically below the theoretical value. For this reason, the design provisions for local buckling have been based largely on test results.

Local Buckling Stress

Considering the post-buckling behavior of the axially compressed cylinder and the important effect of the initial imperfection, the design provisions included in the AISI *Specification* were originally based on Plantema's graphic representation and the additional results of cylindrical shell tests made by Wilson and Newmark at the University of Illinois (Winter, 1970).

From the tests of compressed tubes, Plantema found that the ratio F_{ult}/F_y depends on the parameter $(E/F_y)(t/D)$, in which t is the wall thickness, D is the mean diameter of the tube, and F_{ult} is the ultimate stress or collapse stress. As shown in Figure C-C4.1-8, line 1 corresponds to the collapse stress below the proportional limit, line 2 corresponds to the collapse stress between the proportional limit and the yield stress, and line 3 represents the collapse stress occurring at yield stress. In the range of line 3, local buckling will not occur before yielding. In ranges 1 and 2, local buckling occurs before the yield stress is reached. The cylindrical tubes should be designed to safeguard against local buckling.

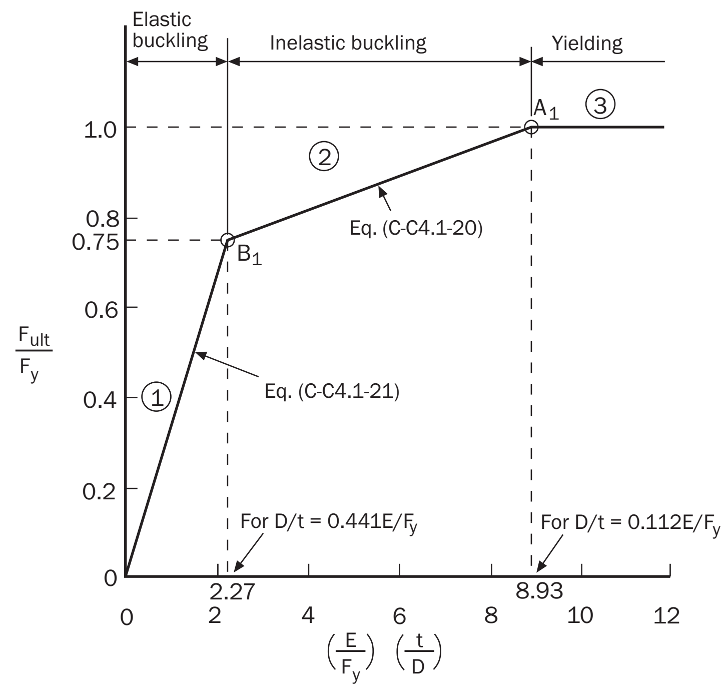


Figure C-C4.1-8 Critical Stress of Cylindrical Tubes for Local Buckling

Based on a conservative approach, the *Specification* specifies that when the D/t ratio is smaller than or equal to $0.112E/F_y$, the tubular member shall be designed for yielding. This provision is based on point A_1, for which $(E/F_y)(t/D) = 8.93$.

When $0.112E/F_y < D/t < 0.441E/F_y$, the design of tubular members is based on the inelastic local buckling criteria. For the purpose of developing a design equation for inelastic buckling, point B_1 was selected to represent the proportional limit. For point B_1,

$$\left(\frac{E}{F_y}\right)\left(\frac{t}{D}\right) = 2.27, \qquad \frac{F_{ult}}{F_y} = 0.75 \tag{C-C4.1-19}$$

Using line A_1B_1, the maximum stress of cylindrical tubes can be represented by

$$\frac{F_{ult}}{F_y} = 0.037\left(\frac{E}{F_y}\right)\left(\frac{t}{D}\right) + 0.667 \qquad \text{(C-C4.1-20)}$$

When D/t ≥ 0.441E/F_y, the following equation represents Line 1 for elastic local buckling stress:

$$\frac{F_{ult}}{F_y} = 0.328\left(\frac{E}{F_y}\right)\left(\frac{t}{D}\right) \qquad \text{(C-C4.1-21)}$$

The correlations between the available test data and Equations C-C4.1-20 and C-C4.121 are shown in Figure C-C4.1-9. The definition of symbol "D" was changed from "mean diameter" to "outside diameter" in the 1986 AISI *Specification* in order to be consistent with the general practice.

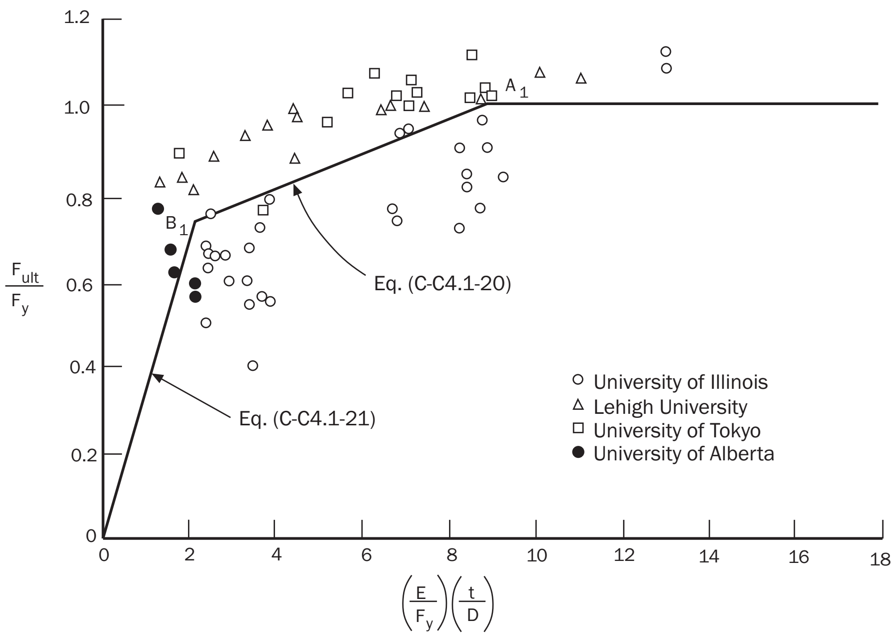


Figure C-C4.1-9 Correlation between Test Data and AISI Criteria for Local Buckling of Cylindrical Tubes under Axial Compression

As indicated in *Commentary* Section C3.1.3, *Specification* Section C4.1.5 is only applicable to members having a ratio of outside diameter-to-wall thickness, D/t, not greater than 0.441E/F_y because the design of extremely thin tubes will be governed by elastic local buckling resulting in an uneconomical design. In addition, cylindrical tubular members with unusually large D/t ratios are very sensitive to geometric imperfections.

When closed cylindrical tubes are used as concentrically loaded compression members the nominal axial strength [compressive resistance] is determined by the same equation as given in *Specification* Section C4.1, except that (1) the nominal buckling stress, F_e, is determined only for flexural buckling and (2) the effective area, A_e, is calculated by Equation C-C4.1-22:

$$A_e = [1-(1-R^2)(1-A_o/A)]A \qquad \text{(C-C4.1-22)}$$

where

$$R = \sqrt{F_y / 2F_e} \quad \text{(C-C4.1-23)}$$

$$A_o = \left[\frac{0.037}{DF_y / tE} + 0.667 \right] A \le A \quad \text{(C-C4.1-24)}$$

A = area of the unreduced cross section.

Equation C-C4.1-24 is used for computing the reduced area due to local buckling. It is derived from Equation C-C4.1-20 for inelastic local buckling stress (Yu, 2000).

In 1999, the coefficient, R, was limited to one (1.0) so that the effective area, A_e, will always be less than or equal to the unreduced cross sectional area, A. To simplify the equations, $R = F_y/(2F_e)$ is used rather than $R = \sqrt{F_y / (2F_e)}$ as in the previous edition of the AISI *Specification*. The equation for the effective area is simplified to $A_e = A_o + R(A - A_o)$ as given in Equation C4.1.5-1 of the *North American Specification*.

C4.2 Distortional Buckling Strength [Resistance]

Distortional buckling is an instability that may occur in members with edge stiffened flanges, such as lipped C- and Z-sections. As shown in Figure C-C4.2-1, this buckling mode is characterized by instability of the entire flange, as the flange along with the edge stiffener rotates about the junction of the flange and the web. The length of the buckling wave in distortional buckling is considerably longer than local buckling, and noticeably shorter than flexural or flexural-torsional buckling. The *Specification* provisions of Section B4 partially account for distortional buckling, but research has shown that a separate limit state check is required (Schafer 2002). Thus, in 2007, treating distortional buckling as a separate limit state, *Specification* Section C3.1.4 was added to address distortional buckling in beams and *Specification* Section C4.2 was added to address distortional buckling in columns. Note, as stated in the *Specification*, when a member is designed in accordance with Section D6.1.3, Compression Members Having One Flange Through-Fastened to Deck or Sheathing, the

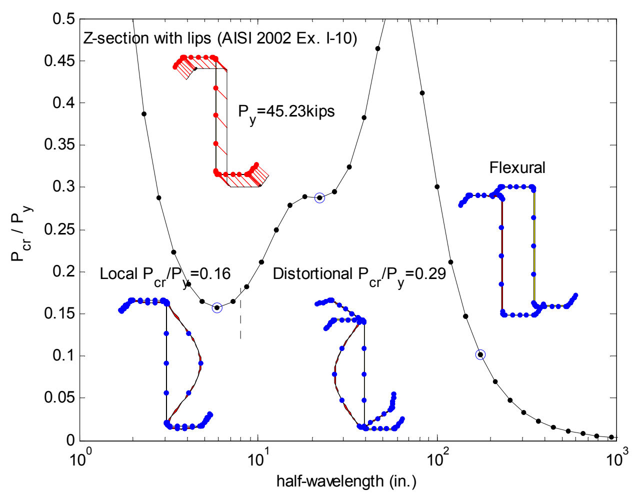


Figure C-C4.2-1 Rational Elastic Buckling Analysis of a Z-Section under Compression Showing Local, Distortional, and Flexural Buckling Modes

Section C4.2 Distortional Buckling Strength provisions need not be applied since distortional buckling is inherently included as a limit state in the Section D6.1.3 strength prediction equations.

Determination of the nominal strength in distortional buckling (*Specification* Equation C4.2-2) was validated by testing. Equation C4.2-2 was originally developed for the Direct Strength Method of Appendix 1 of the *Specification*. Calibration of the safety and resistance factors for *Specification* Equation C4.2-2 is provided in the commentary to Appendix 1. In addition, the Australian/New Zealand Specification (AS/NZS 4600) has used an expression of similar form to *Specification* Equation C4.2-2, but yielding slightly less conservative strength predictions than Equation C4.2-2, since 1996.

Distortional buckling is unlikely to control the strength of a column if (a) the web is slender and triggers local buckling far in advance of distortional buckling, as is the case for many common C-sections, (b) edge stiffeners are sufficiently stiff and thus stabilize the flange (as is often the case for C-sections, but typically not for Z-sections due to the use of sloping lip stiffeners), (c) unbraced lengths are long and flexural or flexural-torsional buckling strength limits the capacity, or (d) adequate rotational restraint is provided to the flanges from attachments (panels, sheathing, etc.).

The primary difficulty in calculating the strength in distortional buckling is to efficiently estimate the elastic distortional buckling stress, F_d. Recognizing the complexity of this calculation this section of the *Specification* provides three alternatives: *Specification* Section C4.2(a) provides a conservative prediction for unrestrained C- and Z-sections, Section C4.2(b) provides a more comprehensive method for C- and Z-Section members and any open section with a single web and flanges of the same dimension, and Section C4.2(c) offers the option to use rational elastic buckling analysis. See the Appendix 1 commentary for further discussion. The equations of Section C4.2(a) assume the compression flange is unrestrained; however, the methods of Sections C4.2(b) and (c) allow for a rotational restraint, k_ϕ, to be included to account for attachments which restrict flange rotation. Additional guidance on k_ϕ is provided in the *Commentary* Section C3.1.4.

(a) Simplified Provision for Unrestrained C- and Z-sections with Simple Lip Stiffeners

The provision of *Specification* Section C4.2(a) provides a conservative approximation to the distortional buckling stress, F_d, for C- and Z-sections with simple lip stiffeners. The expressions were specifically derived as a conservative simplification to those provided in Sections C4.2(b) and (c). For many common sections the provisions of Section C4.2(a) may be used to show that distortional buckling of the column will not control the capacity.

(b) For C- and Z-Sections or Hat Sections or Any Open Section with Stiffened Flanges of Equal Dimension where the Stiffener is either a Simple Lip or a Complex Edge Stiffener

The provisions of *Specification* Section C4.2(b) provide a general method for calculation of the distortional buckling stress, F_d, for any open section with equal edge stiffened compression flanges, including those with complex edge stiffeners. The provisions of *Specification* Section C4.2(b) also provide a more refined answer for any C- and Z-section including those meeting the criteria of Section C4.2(a). The expressions employed here are derived in Schafer (2002) and verified for complex stiffeners in Schafer et al. (2006). The equations used for the distortional buckling stress, F_d, in AS/NZS 4600 are also similar to those in *Specification* Section C4.2(b), except that when the web is very slender and is restrained by the flange, AS/NZS 4600 uses a simpler, conservative treatment. Since the provided expressions can be complicated, solutions for the geometric properties of C- and Z-sections based on centerline dimensions are provided in Table C-C3.1.4(b)-1.

(c) *Rational Elastic Buckling Analysis*

Rational elastic buckling analysis consists of any method following the principles of mechanics to arrive at an accurate prediction of the elastic distortional buckling stress. It is important to note that this is a rational elastic buckling analysis and not simply an arbitrary rational method to determine strength. A variety of rational computational and analytical methods can provide the elastic buckling moment with a high degree of accuracy. Complete details are provided in Section 1.1.2 of the Commentary to Appendix 1 of the *Specification*. The safety and resistance factors of this section have been shown to apply to a wide variety of cross-sections undergoing distortional buckling (via the methods of Appendix 1). As long as the member falls within the geometric limits of main *Specification* Section B1.1 the same safety and resistance factors have been assumed to apply.

C5 Combined Axial Load and Bending

In the 1996 edition of the AISI *Specification*, the design provisions for combined axial load and bending were expanded to include expressions for the design of members subject to combined tensile axial load and bending. In the 2001 and this edition, combined axial and bending for the limit states design (LSD) method has been added. The design approach of the LSD method is the same as the LRFD method.

C5.1 Combined Tensile Axial Load and Bending

These provisions apply to concurrent bending and tensile axial load. If bending can occur without the presence of tensile axial load, the member must also conform to the provisions of *Specification* Sections C3, D4 and D6.1. Care must be taken not to overestimate the tensile load as this could be unconservative.

C5.1.1 ASD Method

Specification Equation C5.1.1-1 provides a design criterion to prevent yielding of the tension flange of a member under combined tensile axial load and bending. *Specification* Equation C5.1.1-2 provides a design criterion to prevent failure of the compression flange.

C5.1.2 LRFD and LSD Methods

Similar to the ASD method, two interaction equations are included in *Specification* Section C5.1.2 for the LRFD and the LSD methods. *Specification* Equations C5.1.2-1 and C5.1.2-2 are used to prevent the failure of the tension flange and compression flange, respectively. In both equations, different symbols are used for the required tensile axial strength [factored tension] and the required flexural strength [factored moment] according to the LRFD and the LSD methods.

C5.2 Combined Compressive Axial Load and Bending

Cold-formed steel members under a combination of compressive axial load and bending are usually referred to as beam-columns. The bending may result from eccentric loading, transverse loads, or applied moments. Such members are often found in framed structures, trusses, and exterior wall studs. For the design of such members, interaction equations have been developed for locally stable and unstable beam-columns on the basis of thorough comparison with rigorous theory and verified by the available test results (Pekoz, 1986a; Pekoz and Sumer, 1992).

The structural behavior of beam-columns depends on the shape and dimensions of the cross section, the location of the applied eccentric load, the column length, the end restraint, and the condition of bracing. In this edition of the *Specification*, the ASD method is included in Section C5.2.1. *Specification* Section C5.2.2 is for the LRFD and the LSD methods.

In 2007, the *Specification* introduced the second order analysis approach as an optional method of stability analysis. This new method is provided in Appendix 2 and specifies the use of a geometrically non-linear second order analysis for determining the required moments and axial loads [factored moments and axial loads] for use in *Specification* Sections C5.2.1 and C5.2.2. The moments and axial loads are the maximums in a member. Appendix 2 also specifies the values for K_x, K_y, α_x, α_y, C_{mx} and C_{my} to be used. Detailed discussion is provided in the commentary on Appendix 2.

The previous effective length approach is still permitted. In this case, the required moments and axial forces for the ASD method and the required strengths [factored moments and axial forces] for the LRFD and LSD methods are derived from a first order elastic analysis and stability effects are accounted for by choosing appropriate K-factors in combination with α_x, α_y, C_{mx} and C_{my} calculated in accordance with *Specification* Sections C5.2.1 and C5.2.2.

To avoid situations of the load $\Omega_c P$ (or $\overline{P}$) exceeding the Euler buckling load P_E, the amplification factor α is limited to a positive value in the 2007 *Specification*.

C5.2.1 ASD Method

When a beam-column is subject to an axial load P and end moments M as shown in Figure C-C5.2-1(a), the combined axial and bending stress in compression is given in Equation C-C5.2.1-1 as long as the member remains straight:

$$f = \frac{P}{A} + \frac{M}{S} \quad \text{(C-C5.2.1-1)}$$

$$= f_a + f_b$$

where

f = combined stress in compression
f_a = axial compressive stress
f_b = bending stress in compression
P = applied axial load
A = cross-sectional area
M = bending moment
S = section modulus

It should be noted that in the design of such a beam-column by using the ASD method, the combined stress should be limited by certain allowable stress F, that is,

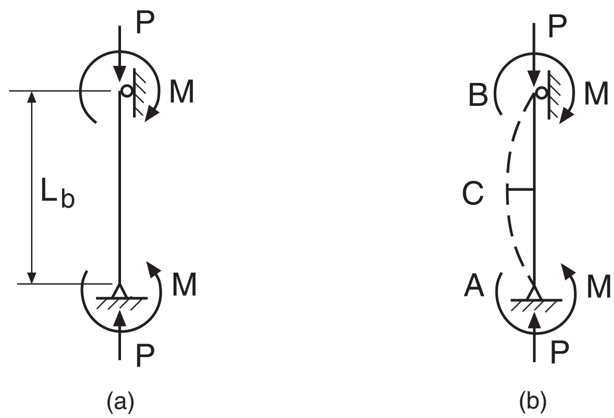


Figure C-C5.2-1 Beam-Column Subjected to Axial Loads and End Moments

$$f_a + f_b \leq F$$

or

$$\frac{f_a}{F} + \frac{f_b}{F} \leq 1.0 \qquad \text{(C-C5.2.1-2)}$$

As specified in Sections C3.1, D6.1 and C4 of the *Specification*, the safety factor Ω_c for the design of compression members is different from the safety factor Ω_b for beam design. Therefore Equation C-C5.2.1-2 may be modified as follows:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \qquad \text{(C-C5.2.1-3)}$$

where

F_a = allowable stress for the design of compression members

F_b = allowable stress for the design of beams

If the strength ratio is used instead of the stress ratio, Equation C-C5.2.1-3 can be rewritten as follows:

$$\frac{P}{P_a} + \frac{M}{M_a} \leq 1.0 \qquad \text{(C-C5.2.1-4)}$$

where

P = applied axial load = Af_a

P_a = allowable axial load = AF_a

M = applied moment = Sf_b

M_a = allowable moment = SF_b

According to Equation C-A4.1.1-1,

$$P_a = \frac{P_n}{\Omega_c}$$

$$M_a = \frac{M_n}{\Omega_b}$$

In the above equations, P_n and Ω_c are given in *Specification* Sections C4 and D6.1, while M_n and Ω_b are specified in *Specification* Sections C3.1 and D6.1. Substituting the above expressions into Equation C-C5.2.1-4, the following interaction equation *(Specification* Equation C5.2.1-3), can be obtained:

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b M}{M_n} \le 1.0 \qquad \text{(C-C5.2.1-5)}$$

Equation C-C5.2.1-4 is a well-known interaction equation, which has been adopted in several specifications for the design of beam-columns. It can be used with reasonable accuracy for short members and members subjected to a relatively small axial load. It should be realized that in practical applications, when end moments are applied to the member, it will be bent as shown in Figure C-C5.2-1(b) due to the applied moment M and the secondary moment resulting from the applied axial load P and the deflection of the member. The maximum bending moment at mid-length (point C) can be represented by

$$M_{max} = \Phi M \qquad \text{(C-C5.2.1-6)}$$

where

M_{max} = maximum bending moment at mid-length

M = applied end moments

Φ = amplification factor

It can be shown that the amplification factor Φ may be computed by

$$\Phi = \frac{1}{1 - P/P_E} \qquad \text{(C-C5.2.1-7)}$$

where P_E = elastic column buckling load (Euler load) = $\pi^2 EI/(KL_b)^2$. Applying a safety factor Ω_c to P_E, Equation C-C5.2.1-7 may be rewritten as

$$\Phi = \frac{1}{1 - \Omega_c P/P_E} \qquad \text{(C-C5.2.1-8)}$$

If the maximum bending moment M_{max} is used to replace M, the following interaction equation can be obtained from Equations C-C5.2.1-5 and C-C5.2.1-8:

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b M}{(1 - \Omega_c P/P_E) M_n} \le 1.0 \qquad \text{(C-C5.2.1-9)}$$

It has been found that Equation C-C5.2.1-9, developed for a member subjected to an axial compressive load and equal end moments, can be used with reasonable accuracy for braced members with unrestrained ends subjected to an axial load and a uniformly distributed transverse load. However, it could be conservative for compression members in unbraced frames (with sidesway), and for members bent in reverse curvature. For this reason, the interaction equation given in Equation C-C5.2.1-9 should be further modified by a coefficient C_m, as shown in Equation C-C5.2.1-10, to account for the effect of end moments:

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_m M}{\alpha M_n} \le 1.0 \qquad \text{(C-C5.2.1-10)}$$

The above equation is *Specification* Equation C5.2.1-1, in which $\alpha = 1 - \Omega_c P/P_E$.

In Equation C-C5.2.1-10, C_m can be determined for one of the three cases defined in *Specification* Section C5.2.1. For Case 1, C_m is given as 0.85. In Case 2, it can be computed by Equation C-C5.2.1-11 for restrained compression members braced against joint translation and not subject to transverse loading:

$$C_m = 0.6 - 0.4\frac{M_1}{M_2} \qquad \text{(C-C5.2.1-11)}$$

where M_1/M_2 is the ratio of smaller to the larger end moments. For Case 3, C_m may be approximated by using the value given in the AISC Commentaries for the applicable condition of transverse loading and end restraint (AISC, 1989, 1999, and 2005).

Figure C-C5.2-2 illustrates the interaction relation. In order to simplify the illustration, bending about only one axis is considered in Figure C-C5.2-2 and the safety factors, Ω_c and Ω_b, are taken as unity. The ordinate is the compressive axial load on the member and the abscissa is the bending moment. When the moment is zero, the limiting axial load is P_n determined in accordance with *Specification* Section C4, which is based on column buckling and local buckling. When the axial load is zero, the limiting moment, M_n, is determined in accordance with *Specification* Sections C3 and D6.1 and is the lowest of the effective yield moment, the moment based on inelastic reserve capacity (if applicable) or the moment based on lateral-torsional buckling. The interaction relation cannot exceed either of these limits.

When *Specification* Equation C5.2.1-1 is plotted in Figure C-C5.2-2, the axial load limit is P_n and the moment limit is M_n/C_m, which will exceed M_n when C_m < 1. Therefore, *Specification* Equation C5.2.1-2 is used as a mathematical stratagem to limit the moment to M_n and match the rigorous solution at low axial loads. The interaction limit is the lower of the two equations as shown by hash marks. *Specification* Equation C5.2.1-2 is a linear relation between the nominal axial yield strength $P_{no} = F_yA_e$ and M_n, and does not represent a failure state over its whole range. If *Specification* Equation C5.2.1-2 uses the moment capacity based only on yield or local buckling, $M_{no} = F_yS_{eff}$, it would be represented by the dashed line, which could exceed an M_n limit based on lateral-torsional buckling. Clearly, load combinations in the shaded region would be unconservative. If M_n is determined by M_{no}, the relation in Figure C-C5.2-2 still apply. If $C_m/\alpha \geq 1$, *Specification* Equation C5.2.1-1 controls.

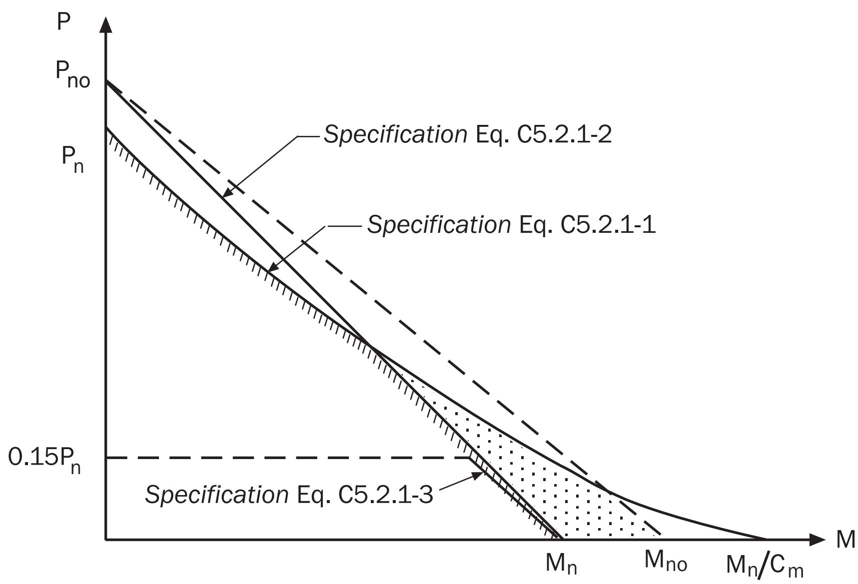


Figure C-C5.2-2 Interaction Relations

For low axial loads, *Specification* Equation C5.2.1-3 may be used. This is a conservative simplification of the interaction relation defined by *Specification* Equations C5.2.1-1 and C5.2.1-2.

In 2001, a requirement of each individual ratio in *Specification* Equations C5.2.1-1 to C5.2.1-3 not exceeding unity was added to avoid situations of the load $\Omega_c P$ exceeding the Euler buckling load P_E, which leads to amplification factor Φ (given in Equation C-C5.2.1-8) negative.

For the design of angle sections using the ASD method, the required additional bending moment of PL/1000 about the minor principal axis is discussed in Item E of Section C4 of the *Commentary*.

C5.2.2 LRFD and LSD Methods

The LRFD and the LSD methods use the same interaction equations as the ASD method, except that $\phi_c P_n$ and $\phi_b M_n$ are used for design strengths [factored resistances]. In addition, the required axial strength [factored compressive force], P_u or P_f, and the required flexural strength [factored moment], M_u or M_f, are to be determined from factored loads according to the requirements of Section A5.1.2 of the *Specification* Appendix A for USA and Mexico, and Appendix B for Canada. In *Specification* Equations C5.2.2-1 through C5.2.2-3, symbols $\overline{P}$ and $\overline{M}$ are used for the required compressive axial strength [factored compressive force] and the required flexural strength [factored moment] for both the LRFD and the LSD methods.

It should be noted that, as compared with the 1991 edition of the AISI LRFD *Specification*, the definition of factor α was changed in the AISI 1996 and this edition of the *Specification* by eliminating the ϕ_c term because the term P_E is a deterministic value and hence does not require a resistance factor.

The interaction equations used in *Specification* Section C5.2.2 are the same as that used in the AISI LRFD Specification (AISI, 1991) but they are different as compared with the AISC Specifications (AISC, 1999 and 2005) due to the lack of sufficient evidence for cold-formed steel columns to adopt the AISC criteria.

Similar to *Specification* Section C5.2.1, ASD Method, the requirement of each individual ratio in *Specification* Equations C5.2.2-1 to C5.2.2-3 not exceeding unity was added in 2001.

For the design of angle sections using the LRFD and the LSD methods, the required additional bending moment of PL/1000 about the minor principal axis was discussed in Item E of Section C4 of the *Commentary*.

D. STRUCTURAL ASSEMBLIES AND SYSTEMS

D1 Built-Up Sections

I-Sections made by connecting two C-sections back to back are one type of built-up section that is often used as either flexural or compression members. Cases (2) and (8) of Figure C-A1.2-2 and Cases (3) and (7) of Figure C-A1.2-3 show several built-up I-sections. For built-up flexural members, the *Specification* is limited to two back-to-back C-sections. For built-up compression members, other sections can be used.

D1.1 Flexural Members Composed of Two Back-to-Back C-Sections

For the I-sections to be used as flexural members, the longitudinal spacing of connectors is limited by Equation D1.1-1 of the *Specification*. The first requirement is an arbitrarily selected limit to prevent any possible excessive distortion of the top flange between connectors. The second is based on the strength [resistance] and arrangement of connectors and the intensity of the load acting on the beam (Yu, 2000).

The second requirement for maximum spacing of connectors required by *Specification* Equation D1.1-1 is based on the fact that the shear center of the C-section is neither coincident with nor located in the plane of the web; and that when a load Q is applied in the plane of the web, it produces a twisting moment Qm about its shear center, as shown in Figure C-D1.1-1. The tensile force of the top connector T_s can then be computed from the equality of the twisting moment Qm and the resisting moment $T_s g$, that is

$$Qm = T_s g \tag{C-D1.1-1}$$

$$T_s = \frac{Qm}{g} \tag{C-D1.1-2}$$

Considering that q is the intensity of the load and that s is the spacing of connectors as shown in Figure C-D1.1-2, the applied load is Q=qs/2. The maximum spacing s_{max} used in the *Specification* can easily be obtained by substituting the above value of Q into Equation C-D1.1-2 of this *Commentary*. The determination of the load intensity q is based upon the type of loading applied to the beam. The requirement of three times the uniformly distributed load is applied to reflect that the assumed uniform load will not really be uniform. The *Specification* prescribes a conservative estimate of the applied loading to account for the likely concentration of loads near the welds or other connectors that join the two C-sections.

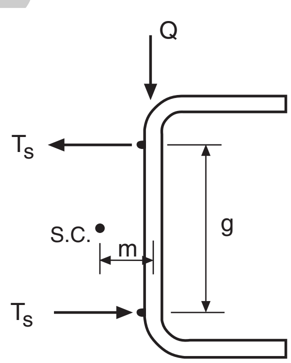


Figure C-D1.1-1 Tensile Force Developed in the Connector for C-Section

For simple C-sections without stiffening lips at the outer edges,

$$m = \frac{w_f^2}{2w_f + d/3} \tag{C-D1.1-3}$$

For C-sections with stiffening lips at the outer edges,

$$m = \frac{w_f dt}{4I_x}\left[w_f d + 2D\left(d - \frac{4D^2}{3d}\right)\right] \tag{C-D1.1-4}$$

where

w_f = Projection of flanges from the inside face of the web (For C-sections with flanges of unequal width, w_f shall be taken as the width of the wider flange)

d = Depth of C-section or beam

D = Overall depth of lip

I_x = Moment of inertia of one C-section about its centroidal axis normal to the web

In addition to the above considerations on the required strength [effect of factored loads] of connections, the spacing of connectors should not be so great as to cause excessive distortion between connectors by separation along the top flange. In view of the fact that C-sections are connected back to back and are continuously in contact along the bottom flange, a maximum spacing of L/3 may be used. Considering the possibility that one connection may be defective, a maximum spacing of s_{max} = L/6 is the first requirement in *Specification* Equation D1.1-1.

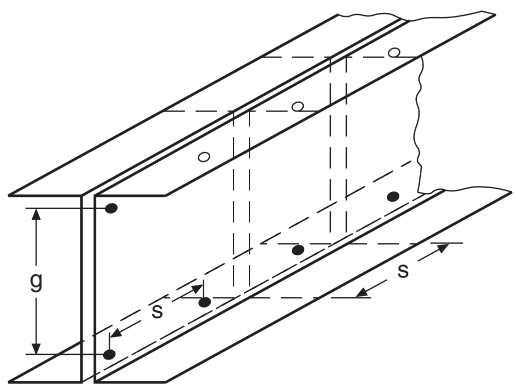


Figure C-D1.1-2 Spacing of Connectors

D1.2 Compression Members Composed of Two Sections in Contact

Compression members composed of two shapes joined together at discrete points have a reduced shear rigidity. The influence of this reduced shear rigidity on the buckling stress is taken into account by modifying the slenderness ratio used to calculate the elastic critical buckling stress (Bleich, 1952). The overall slenderness and the local slenderness between connected points both influence the compressive resistance. The combined action is expressed by the modified slenderness ratio given by the following:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \tag{C-D1.2-1}$$

Note that in this expression, the overall slenderness ratio, $(KL/r)_o$, is computed about the same axis as the modified slenderness ratio, $(KL/r)_m$. Further, the modified slenderness ratio, $(KL/r)_m$, replaces KL/r in the *Specification* Section C4 for both flexural and flexural-torsional buckling.

This modified slenderness approach is used in other steel standards, including the AISC (AISC, 1999 and 2005), CSA S136 (CSA S136, 1994), and CAN/CSA S16.1 (CAN/CSA S16.1-94, 1994).

To prevent the flexural buckling of the individual shapes between intermediate connectors, the intermediate fastener spacing, a, is limited such that a/r_i does not exceed one half the governing slenderness ratio of the built-up member (i.e. $a/r_i \leq 0.5(KL/r)_o$). This intermediate fastener spacing requirement is consistent with the previous edition of the AISI *Specification* with the one half factor included to account for any one of the connectors becoming loose or ineffective. Note that the previous edition of S136 (S136, 1994) had no limit on fastener spacing.

The importance of preventing shear slip in the end connection is addressed by the prescriptive requirements in *Specification* Section D1.2(2) adopted from the AISC (AISC, 1999) and CAN/CSA S16.1 (CAN/CSA S16.1-94, 1994). These provisions were added to the *North American Specification* since 2001.

The intermediate fastener(s) or weld(s) at any longitudinal member tie location is required, as a group, to transmit a force equal to 2.5 percent of the nominal axial strength [resistance] of the built-up member. A longitudinal member tie is defined as a location of interconnection of the two members in contact. In the 2001 edition of the *Specification*, a 2.5 percent total force determined in accordance with appropriate load combinations was used for design of the intermediate fastener(s) or weld(s). This requirement was adopted from CSA S136-94. In 2004, the requirement has been changed to be a function of the nominal axial strength. This change is to ensure that the nominal axial strength [resistance] of the built-up member is valid and is not compromised by the strength [resistance] of the member interconnections.

Note that the provision in *Specification* Section D1.2 has been substantially taken from research in hot-rolled built-up members connected with bolts or welds. These hot-rolled provisions have been extended to include other fastener types common in cold-formed steel construction (such as screws) provided they meet the 2.5 percent requirement for shear strength [resistance] and the conservative spacing requirement $a/r_i \leq 0.5(KL/r)_o$.

D1.3 Spacing of Connections in Cover Plated Sections

When compression elements are joined to other parts of built-up members by intermittent connections, these connectors must be closely spaced to develop the required strength [effect of forces] of the connected element. Figure C-D1.3-1 shows a box-shaped beam made by connecting a flat sheet to an inverted hat section. If the connectors are appropriately placed, this flat sheet will act as a stiffened compression element with a width, w, equal to the distance between rows of connectors, and the sectional properties can be calculated accordingly. This is the intent of the provisions in Section D1.3 of the *Specification*.

Section D1.3(a) of the *Specification* requires that the necessary shear strength [resistance] be provided by the same standard structural design procedure that is used in calculating flange connections in bolted or welded plate girders or similar structures.

Section D1.3(b) of the *Specification* ensures that the part of the flat sheet between two adjacent connectors will not buckle as a column (see Figure C-D1.3-1) at a stress less than $1.67f_c$, where f_c is the stress at service load in the connected compression element (Winter, 1970; Yu, 2000). The AISI requirement is based on the following Euler equation for column buckling:

$$\sigma_{cr} = \frac{\pi^2 E}{(KL/r)^2}$$

by substituting $\sigma_{cr} = 1.67f_c$, $K = 0.6$, $L = s$, and $r = t/\sqrt{12}$. This provision is conservative because the length is taken as the center distance instead of the clear distance between connectors, and the coefficient K is taken as 0.6 instead of 0.5, which is theoretical value for a column with fixed end supports.

Section D1.3(c) ensures satisfactory spacing to make a row of connectors act as a continuous line of stiffening for the flat sheet under most conditions (Winter, 1970; Yu, 2000).

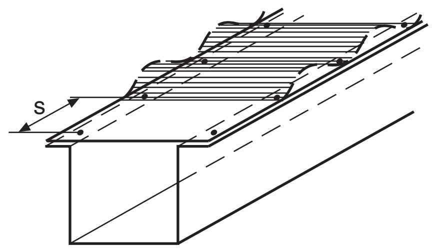


Figure C-D1.3-1 Spacing of Connectors in Composite Section

D2 Mixed Systems

When cold-formed steel members are used in conjunction with other construction materials, the design requirements of the other material specifications also must be satisfied.

D3 Lateral and Stability Bracing

Bracing design requirements were expanded in the 1986 AISI *Specification* to include a general statement regarding bracing for symmetrical beams and columns and specific requirements for the design of roof systems subjected to gravity load. These requirements are retained in this *Specification.* B

Lateral restraints are applied to the top flange of C- and Z-sections to resist the tendency of Z-sections to translate laterally, and the tendency of both Z- and C-sections to twist due to eccentrically applied loads. By restraining lateral displacement and torsional rotation, second order effects are minimized. Anchorage is most commonly applied along the frame lines due to the effectiveness and ease in which the forces are transferred out of the system. In the absence of substantial diaphragm stiffness, anchorage may be required along the interior of the span to prevent large lateral displacements. Torsional braces applied along the span of a Z- or C-section provide an alternative to interior anchorage.

D3.1 Symmetrical Beams and Columns

There are no simple, generally accepted techniques for determining the required strength [resistance] and stiffness for discrete braces in steel construction. Winter (1960) offered a partial solution and others have extended this knowledge (Haussler, 1964; Haussler and Pahers, 1973; Lutz and Fisher, 1985; Salmon and Johnson, 1990; Yura, 1993; SSRC, 1993). The design engineer is encouraged to seek out the stated references to obtain guidance for design of a brace or brace system.

☜B

D3.2 C-Section and Z-Section Beams

C-sections and Z-sections used as beams to support transverse loads applied in the plane of the web may twist and deflect laterally unless adequate lateral supports are provided. Section D3.2 of the *Specification* includes the requirements for spacing and design of braces, when neither flange of the beam is braced by deck or sheathing material. The bracing requirements for members having one flange connected to deck or sheathing materials are provided in D6.3.1.

☜B

D3.2.1 Neither Flange Connected to Sheathing that Contributes to the Strength and Stability of the C- or Z- section

(a) *Bracing of C-Section Beams*

If C-sections are used singly as beams, rather than being paired to form I-sections, they should be braced at intervals so as to prevent them from rotating in the manner indicated in Figure C-D3.2.1-1. Figure C-D3.2.1-2, for simplicity, shows two C-sections braced at intervals against each other. The situation is evidently much the same as in the composite I-section of Figure C-D1.1-2, except that the role of the connectors is now played by the braces. The difference is that the two C-sections are not in contact, and that the spacing of braces is generally considerably larger than the connector spacing. In consequence, each C-section may actually rotate very slightly between braces, and this will cause some additional stresses, which superimpose on the usual, simple bending stresses. Bracing should be so arranged that: (1) these additional stresses are small enough not to reduce the load-carrying capacity of the C-section (as compared to what it would be in the continuously braced condition); and (2) rotations should be kept small enough to be unobjectionable on the order of 1 to 2 degrees.

In order to obtain the information for developing bracing provisions, different C-section shapes have been tested at Cornell University (Winter, 1970). Each of these was tested with full, continuous bracing; without any bracing; and with intermediate bracing at two different spacings. In addition to this experimental work, an approximate method of analysis was developed and checked against the test results. A condensed account of this was given by Winter, Lansing and McCalley (1949b). It is indicated in that reference that the above requirements are satisfied for most distributions of beam load if between supports not less than three equidistant braces are placed (i.e., at quarter-points of the span, or closer). The exception is the case where a large part of the total load of the beam is concentrated over a short portion of the span; in this case an additional brace should be placed at such a load. Correspondingly, previous editions of the AISI *Specification* (AISI, 1986; AISI, 1991) provided that the distance between braces shall not be greater than one-quarter of the span; it also defined the conditions under which an additional brace should be placed at a load concentration.

For such braces to be effective it is not only necessary that their spacing be appropriately limited; in addition, their strength [resistance] should suffice to provide the force required to prevent the C-section from rotating. It is, therefore, necessary also to determine the forces that will act in braces, such as those forces shown in Figure C-D3.2.1-3. These forces are found if one considers that the action of a load applied in the plane of the web (which causes a torque Qm) is equivalent to that same load when applied at the shear center (where it causes no torque) plus two forces P = Qm/d which, together, produce the same torque Qm. As is sketched in Figure C-D3.2.1-4, and shown in some detail by Winter, Lansing and McCalley (1949b), each half of the channel can then be regarded as a continuous beam loaded by the horizontal forces and supported at the brace points. The horizontal brace force is then, simply, the appropriate reaction of this continuous beam. The provisions of *Specification* Section D3.2.1 provide expressions for determining bracing forces P_{L1} and P_{L2}, which the braces are required to resist at each flange.

(b) Bracing of Z-Section Beams

Most Z-sections are anti-symmetrical about the vertical and horizontal centroidal

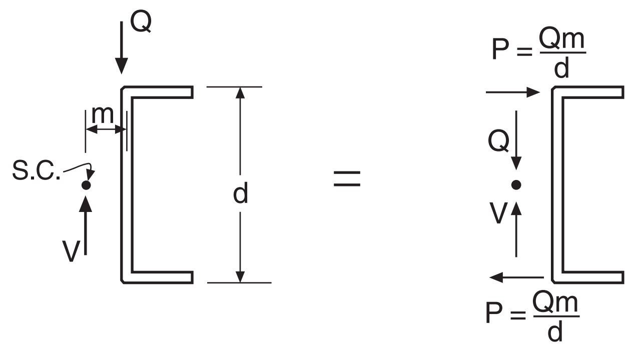


Figure C-D3.2.1-3 Lateral Forces Applied to C-Section

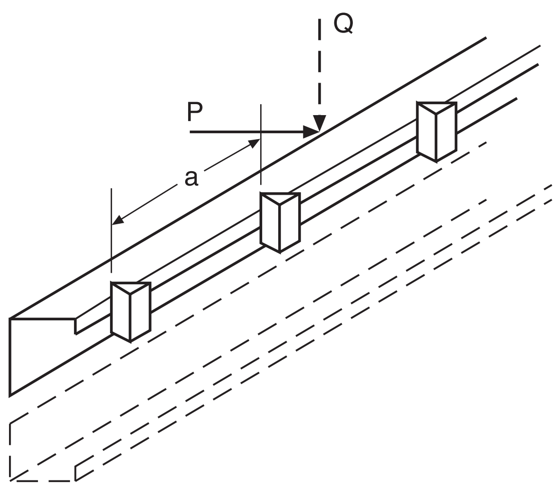


Figure C-D3.2.1-4 Half of C-Section Treated as a Continuous Beam Loaded by Horizontal Forces

axes, i.e. they are point-symmetrical. In view of this, the centroid and the shear center coincide and are located at the midpoint of the web. A load applied in the plane of the web has, then, no lever arm about the shear center (m = 0) and does not tend to produce the kind of rotation a similar load would produce on a C-section. However, in Z-sections the principal axes are oblique to the web (Figure C-D3.2.1-5). A load applied in the plane of the web, resolved in the direction of the two axes, produces deflections along each of them. By projecting these deflections onto the horizontal and vertical planes it is found that a Z-beam loaded vertically in the plane of the web deflects not only vertically but also horizontally. If such deflection is permitted to occur then the loads, moving sideways with the beam, are no longer in the same plane with the reactions at the ends. In consequence, the loads produce a twisting moment about the line connecting the reactions. In this manner it is seen that a Z-beam, unbraced between ends and loaded in the plane of the web, deflects laterally and also twists. Not only are these deformations likely to interfere with a proper functioning of the beam, but the additional stresses caused by them produce failure at a load considerably lower than when the same beam is used fully braced.

In order to obtain information for developing appropriate bracing provisions, tests have been carried out on three different Z-sections at Cornell University, unbraced as well as with variously spaced intermediate braces. In addition, an approximate method of analysis has been developed and checked against the test results. An account of this was given by Zetlin and Winter (1955b). Briefly, it is shown that intermittently braced Z-beams can be analyzed in much the same way as intermittently braced C-beams. It is merely necessary, at the point of each actual vertical load Q, to apply a fictitious horizontal load $Q(I_{xy}/I_x)$ or $Q[I_{xy}/(2I_x)]$ to each flange. One can then compute the vertical and horizontal deflections, and the corresponding stresses, in conventional ways by utilizing the convenient axes x and y (rather than 1 and 2, Figure C-D3.2.1-5), except that certain modified section properties have to be used. To control the lateral deflection, brace forces, P, must statically balance the fictitious force.

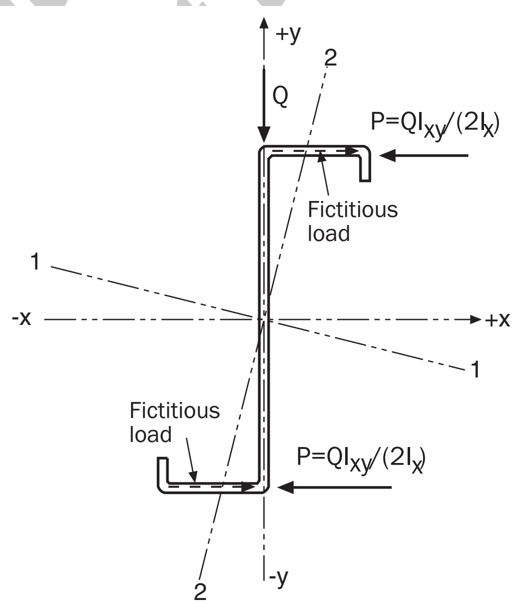


Figure C-D3.2.1-5 Principal Axis of Z-Section

In this manner it has been shown that as to location of braces the same provisions that apply to C-sections are also adequate for Z-sections. Likewise, the forces in the braces are again obtained as the reactions of continuous beams horizontally loaded by fictitious loads P. It should, however, be noted that the direction of the bracing forces in Z-beams is different from the direction in C-beams. In the Z-beam, the bracing forces are acting in the same direction, as shown in Fig. C-D3.2.1-5 in order to constrain bending of the section about the axis x-x in Figure C-D3.2.1-5. The directions of the bracing forces in the C-beam flanges are in the opposite direction as shown in Figure C-D3.2.1-3 in order to resist the torsion caused by the applied load. In the previous edition of the *Specification*, the magnitude of the Z-beam bracing force was shown as $P = Q(I_{xy}/I_x)$ on each flange. In 2001, this force was corrected to $P = Q[I_{xy}/(2I_x)]$.

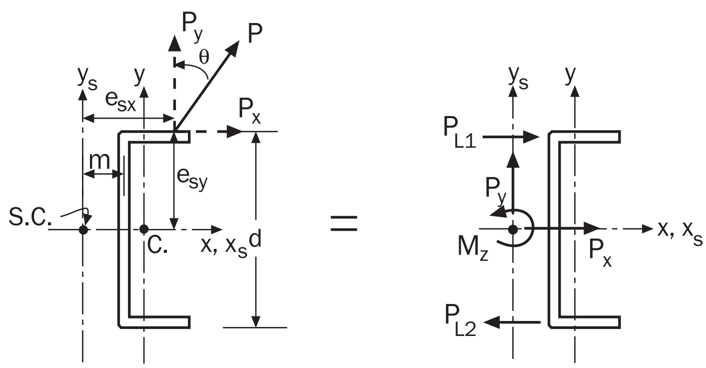


Figure C-D3.2.1-6 C-Section Member Subjected to a Concentrated Load

(c) Slope Effect and Eccentricity

For a C- or Z-section member subjected to an arbitrary load, bracing forces, P_{L1} and P_{L2}, on flanges need to resist (1) force component P_x that is perpendicular to the web, (2) the torsion caused by eccentricity about the shear center, and (3) for the Z-section member, the lateral movement caused by component P_y, that is parallel to the web.

To develop a set of equations applicable to any loading conditions, the x and y axes are oriented such that one of the flanges is located in the quadrant with both x and y axes positive. Since the torsion should be calculated about the shear center, coordinates x_s and y_s that go through the shear center and parallel to x and y axes are established. Load eccentricities e_x and e_y should be measured based on x_s and y_s coordinate system.

For the C-section member as shown in Figure C-D3.2.1-6, the bracing forces on both flanges are given in Equations C-D3.2.1-1 and C-D3.2.1-2.

$$P_{L1} = -\frac{P_x}{2} + \frac{M_z}{d} \quad \text{(C-D3.2.1-1)}$$

$$P_{L2} = -\frac{P_x}{2} - \frac{M_z}{d} \quad \text{(C-D3.2.1-2)}$$

$$M_z = -P_x e_{sy} + P_y e_{sx} \quad \text{(C-D3.2.1-3)}$$

where d = overall depth of the web; e_{sx}, e_{sy} = eccentricities of design load about the shear center in x_s- and y_s-direction, respectively; P_x, P_y = components of design load in x- and y-direction, respectively; M_z = torsional moment about the shear center; and P_{L1} = bracing force applied to the flange located in the quadrant with both positive x and y axes, and P_{L2} = bracing force applied on the other flange. Positive P_{L1} and P_{L2} indicate that a restraint is required to prevent the movement of the corresponding flange in the negative x-direction.

For a special case where design load, Q, is through the web, as shown in Figure C-D3.2.1-3, P_y = -Q, P_x = 0; e_{sx} = m, e_{sy} = d/2, and from Equation C-D3.2.1-3, M_z = -Qm. Therefore

$$P_{L1} = -Qm/d \qquad \text{(C-D3.2.1-4)}$$

$$P_{L2} = Qm/d \qquad \text{(C-D3.2.1-5)}$$

In which, m = distance from centerline of web to the shear center.

For the Z-section member as shown in Figure C-D3.2.1-7, bracing forces, P_{L1} and P_{L2}, are given in Equations C-D3.2.1-6 and C-D3.2.1-7.

$$P_{L1} = P_y\left(\frac{I_{xy}}{2I_x}\right) - \frac{P_x}{2} + \frac{M_z}{d} \qquad \text{(C-D3.2.1-6)}$$

$$P_{L2} = P_y\left(\frac{I_{xy}}{2I_x}\right) - \frac{P_x}{2} - \frac{M_z}{d} \qquad \text{(C-D3.2.1-7)}$$

where I_x, I_{xy} = unreduced moment of inertia and product of inertia; respectively. Other variables are defined under the discussion for C-section members.

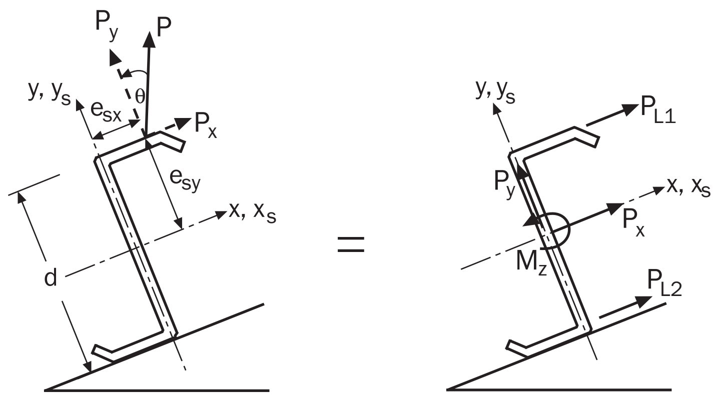


Figure C-D3.2.1-7 A Z-Section Member Subjected to an Arbitrary Load

Assuming that a gravity load, P, acts at 1/3 of the top flange width, b_f, and the Z-Section member rests on a sloped roof with an angle of θ, $P_x = -P\sin\theta$; $P_y = -P\cos\theta$; $e_{sx} = b_f/3$; e_{sy} = d/2 and $M_z = P\sin\theta(d/2) - P\cos\theta(b_f/3)$. Substituting the above expressions into equations C-D3.2.1-6 and C-D3.2.1-7 results in

$$P_{L1} = -P\cos\theta\left(\frac{I_{xy}}{2I_x}\right) + P\sin\theta - \frac{Pb_f\cos\theta}{3d}$$

$$P_{L2} = -P\cos\theta\left(\frac{I_{xy}}{2I_x}\right) + \frac{Pb_f\cos\theta}{3d}$$

In considering the distribution of loads and the braces along the member length, it is required that the resistance at each brace location along the member length be greater than or equal to the design load within a distance of 0.5a on each side of the brace for distributed loads. For concentrated loads, the resistance at each brace location should be greater than or equal to the concentrated design load within a distance 0.3a each side of the brace, plus 1.4(1-l/a) times each design load located farther than 0.3a but not farther than 1.0a from the brace. In the above, a is the distance between centerline of braces along the member length and l is the distance from concentrated load to the brace to be considered.

(d) *Spacing of Braces*

During the period from 1956 through 1996, the AISI *Specification* required that braces be attached both to the top and bottom flanges of the beam, at the ends and at intervals not greater than one-quarter of the span length, in such a manner as to prevent tipping at the ends and lateral deflection of either flange in either direction at intermediate braces. The lateral-torsional buckling equations provided in *Specification* Section C3.1.2.1 can be used to predict the moment capacity of the member. Beam tests conducted by Ellifritt, Sputo and Haynes (1992) have shown that for typical sections, a mid-span brace may reduce service load horizontal deflections and rotations by as much as 80 percent when compared to a completely unbraced beam. However, the restraining effect of braces may change the failure mode from lateral-torsional buckling to distortional buckling of the flange and lip at a brace point. The natural tendency of the member under vertical load is to twist and translate in such a manner as to relieve the compression on the lip. When such movement is restrained by intermediate braces, the compression on the stiffening lip is not relieved, and may increase. In this case, local distortional buckling may occur at loads lower than that predicted by the lateral-torsional buckling equations of *Specification* Section C3.1.2.1.

Research (Ellifritt, Sputo and Haynes, 1992) has also shown that the lateral-torsional buckling equations of *Specification* Section C3.1.2.1 predict loads, which are conservative for cases where one mid-span brace is used but may be unconservative where more than one intermediate brace is used. Based on such research findings, Section D3.2.1 of the *Specification* was revised in 1996 to eliminate the requirement of quarter-point bracing. It is suggested that, minimally, a mid-span brace be used for C-section and Z-section beams to control lateral deflection and rotation at service loads. The lateral-torsional buckling strength [resistance] of an open cross section member should be determined by *Specification* Section C3.1.2.1 using the distance between centerlines of braces "a" as the unbraced length of the member "L" in all design equations. In any case, the user is permitted to perform tests, in accordance with *Specification* Section F1, as an alternative, or use a rigorous analysis, which accounts for biaxial bending and torsion.

Section D3.2.1 of the *Specification* provides the lateral forces for which these discrete braces must be designed.

The *Specification* permits omission of discrete braces when all loads and reactions on a beam are transmitted through members that frame into the section in such a manner as to effectively restrain the member against torsional rotation and lateral displacement. Frequently, this occurs in the end walls of metal buildings.

In 2007, the title of this section was changed to clarify that it is and was formerly anticipated that the C- and Z-sections covered by these provisions would be supporting sheathing and be loaded as a result of providing this support function. The revised title reflects that the supported sheathing is not contributing to the strength and stiffness of these members by virtue of the nature of its connection to the C- and Z-sections.

D3.3 Bracing of Axially Loaded Compression Members

The requirements for bracing a single compression member were developed from a study by Green et. al (2004). With the exception of the compression member force used for design, the requirements for brace strength for a single compression member are similar to those in the AISC *Specification for Structural Steel Buildings* for compression member nodal bracing (AISC, 2005). The requirements for brace stiffness for a single compression member are similar to the AISC provisions, with the exception that 2(4-(2/n)) instead of 8 is used as the multiplier for the bracing stiffness. AISC assumes n = infinity. It is considered that this simplification is too conservative for cold-formed steel structures. Analytical modeling by Sputo and Beery (2006) has shown that these provisions may be applied to members of varied cross sections.

For the calculation of bracing strength [resistance] and stiffness, the nominal strength [resistance] of the member, P_n, is used rather than the required strength [factored resistance]. It is considered that using the full braced strength is the proper formulation, since the equations for the member strength [resistance] (axial, flexural, and combined axial and flexural) consider that the member be able to develop the full braced strength [resistance].

The brace provisions for lateral translation assume that the braces are perpendicular to the compression member being braced and located in the plane of buckling. The stiffness requirements include the contributions of the bracing members, connections, and anchorage details.

In addition to the requirement to brace against lateral translation, there is a torsional demand for members subject to torsional or flexural-torsional buckling which is not accounted for by this section, and which may be determined through rational analysis or other methods. In any case, torsional effects should be considered in the design of bracing.

D4 Cold-Formed Steel Light-Frame Construction

In 2007, the scope of Section D4 on Wall Studs and Wall Stud Assemblies of the 2001 edition of the *Specification* with 20004 Supplement was broadened to Light-Frame Construction. This was done in order to recognize the growing use of cold-formed steel framing in a broader range of residential and light commercial framing applications and to provide a means for either requiring or accepting use of the various ANSI-approved standards that have been developed by the AISI Committee on Framing Standards.

The *North American Standard for Cold-Formed Steel Framing - General Provisions* addresses requirements for construction with cold-formed steel framing that are common to prescriptive and engineered design. Use of the *General Provisions* is mandatory for the design and installation of structural members and non-structural members utilized in cold-formed steel repetitive framing applications where the specified minimum base steel thickness is between 18 mils (0.0179 inches) (0.455mm) and 118 mils (0.1180 inches) (2.997 mm) because certain requirements, such as corrosion protection, product designators, manufacturing and installation tolerances are not addressed adequately by the *Specification*.

The other referenced standards include the following:

(a) The *North American Standard for Cold-Formed Steel Framing – Header Design* [*Header Standard*] provides technical information and specifications for designing headers made from cold-formed steel. Use of the *Header Standard* is optional for the design and installation of cold-formed steel box and back-to-back headers, and double and single L-headers for load carrying purposes in buildings because individual structural members of a header assembly can be designed fully, albeit often less efficiently, using the *Specification* alone.

(b) The *North American Standard for Cold-Formed Steel Framing – Truss Design* [*Truss Standard*] provides technical information and specifications on cold-formed steel truss construction. Use of the *Truss Standard* is mandatory for the design of cold-formed steel trusses for load carrying purposes in buildings because certain requirements, such as design responsibilities, design requirements specific to truss assemblies using C-shape, hat-shape and z-shape sections and gusset plates, as well as manufacturing, quality criteria, installation and testing as they relate to the design of cold-formed steel trusses are not addressed adequately by the *Specification*.

(c) The *North American Standard for Cold-Formed Steel Framing – Wall Stud Design* (*Wall Stud Standard*) provides technical information and specifications for designing wall studs made from cold-formed steel. Use of the *Wall Stud Standard* is optional for the design and installation of cold-formed steel studs for both structural and non-structural walls in buildings because individual structural members of a wall stud assembly can be designed fully, albeit often less efficiently, using the *Specification* alone. For more comments on the design and use of wall studs, see Section D4.1 of this *Commentary*.

(d) The *North American Standard for Cold-Formed Steel Framing—Floor and Roof System Design* (*FRSD Standard*) provides technical information and specifications for designing floor and roof systems made from cold-formed steel. Use of the *FRSD Standard* is optional for the design and installation of cold-formed steel framing for floor and roof systems in buildings because individual structural members of a floor and roof system assembly can be designed fully, albeit often less efficiently, using the *Specification* alone.

See Appendix A for commentary on the country specific standards. ☞A

These framing standards are available for adoption and use in the United States, Canada and Mexico, and provide an integrated treatment of Allowable Strength Design (ASD), Load and Resistance Factor Design (LRFD), and Limit States Design (LSD). These framing standards do not preclude the use of other materials, assemblies, structures or designs not meeting the criteria herein, when the other materials, assemblies, structures or designs demonstrate equivalent performance for the intended use to those specified in the standards.

Other framing standards have been developed by the AISI Committee on Framing Standards, but are not yet North American in scope. These framing standards are currently available for adoption and use in the United States, and are referenced directly in the U.S. building codes.

D4.1 All Steel Design of Wall Stud Assemblies

It is well known that column strength [resistance] can be increased considerably by using adequate bracing, even though the bracing is relatively flexible. This is particularly true for those sections generally used as load-bearing wall studs which have large I_x/I_y ratios.

Cold-formed I-, C-, Z-, or box-type studs are generally used in walls with their webs placed perpendicular to the wall surface. The walls may be made of different materials, such as fiberboard, pulp board, plywood, or gypsum board. If the wall material is strong enough and there is adequate attachment provided between wall material and studs for lateral support of the studs, then the wall material can contribute to the structural economy by increasing the usable strength [resistance] of the studs substantially.

In order to determine the necessary requirements for adequate lateral support of the wall studs, theoretical and experimental investigations were conducted in the 1940s by Green, Winter, and Cuykendall (1947). The study included 102 tests on studs and 24 tests on a variety of wall material. Based on the findings of this earlier investigation, specific AISI provisions were developed for the design of wall studs.

In the 1970s, the structural behavior of columns braced by steel diaphragms was a special subject investigated at Cornell University and other institutions. The renewed investigation of wall-braced studs has indicated that the bracing provided for studs by steel panels is of the shear diaphragm type rather than the linear type, which was considered in the 1947 study. Simaan (1973) and Simaan and Pekoz (1976), which are summarized by Yu (2000), contain procedures for computing the strength [resistance] of C- and Z-section wall studs that are braced by sheathing materials. The bracing action is due to both the shear rigidity and the rotational restraint supplied by the sheathing material. The treatment by Simaan (1973) and Simaan and Pekoz (1976) is quite general and includes the case of studs braced on one as well as on both flanges. However, the provisions of Section D4 of the 1980 AISI *Specification* dealt only with the simplest case of identical sheathing material on both sides of the stud. For simplicity, only the restraint due to the shear rigidity of the sheathing material was considered.

The 1989 Addendum to the AISI 1986 *Specification* included the design limitations from the *Commentary* and introduced stub column tests and/or rational analysis for the design of studs with perforations (Davis and Yu, 1972; Rack Manufacturers Institute, 1990).

In 1996, the design provisions were revised to permit (a) all steel design and (b) sheathing braced design of wall studs with either solid or perforated webs. For sheathing braced design, in order to be effective, sheathing must retain its design strength [resistance] and integrity for the expected service life of the wall. Of particular concern is the use of gypsum sheathing in a moist environment.

In 2004 the sheathing braced design provisions were removed from the *Specification* and a requirement added that sheathing braced design be based on appropriate theory, tests, or rational engineering analysis that can be found in AISI (2004); Green, Winter, and Cuykendall (1947); Simaan (1973); and Simaan and Pekoz (1976).

In 2007, in addition to the revisions of the Sepecification Section D4 as discussed in Section D4 of this *Commentary*, the provisions for non-circular holes were moved from *Specification* Section D4.1 to Section B2.2 on Uniformly Compressed Stiffened Elements with Circular or Non-Circular Holes. Within the limitations stated for the size and spacing of perforations and section depth, the provisions were deemed appropriate for members with uniformly compressed stiffened elements, not just wall studs.

D5 Floor, Roof or Wall Steel Diaphragm Construction

In building construction, it has been a common practice to provide a separate bracing system to resist horizontal loads due to wind load, blast force, or earthquake. However, steel floor and roof panels, with or without concrete fill, are capable of resisting horizontal loads in addition to the beam strength [resistance] for gravity loads if they are adequately interconnected to each other and to the supporting frame. The effective use of steel floor and roof decks can therefore eliminate separate bracing systems and result in a reduction of building costs. For the same reason, wall panels can provide not only enclosure surface and support normal loads, but they can also provide diaphragm action in their own planes.

The structural performance of a diaphragm construction can be evaluated by either calculations or tests. Several analytical procedures exist, and are summarized in the literature (Steel Deck Institute, 2004; Metal Construction Association, 2004; Department of Army, 1992; and ECCS, 1977). Analytical methods depend on the capacity of the connections between the panels and structural supports. The support thickness and mechanical properties must be considered. As an example, the tilting potential of screws is discussed in Section E4.3 and is distinct from the bearing capacity controlled by panels. When using analytical methods, refer to the applicability limits. Tested performance is measured using the procedures of the Standard Method for Static Load Testing of Framed Floor, Roof and Wall Diaphragm Construction for Buildings, ASTM E455. Part VI of the AISI *Design Manual* (AISI, 2008) contains the Test Procedure with Commentary on Cantilever Test Method for Cold-Formed Diaphragms. Yu (2000) provides a general discussion of structural diaphragm behavior.

The safety factors and resistance factors listed in the *Specification* are based on a recalibration of the full-scale test data summarized in the Steel Deck Institute Diaphragm Design Manual, First Edition. The recalibration used the method of *Specification* Section A5.1.1 and F1.1 and the load factors in ASCE 7-98. The most probable diaphragm D/L load ratio is zero and this was used in the recalibration. The dominant diaphragm limit state is connection related. Consistent with *Commentary* Section A 5.1.1(b), the calibration used $\beta_o = 3.5$ for all load effects except wind load. The US LRFD method allows $\beta_o = 2.5$ for connections subjected to wind loads. For both welds and screws calibration using $\beta_o = 2.5$ suggests factors less severe than $\phi = 0.8$ and $\Omega = 2.0$. Because of concerns over weld quality control and to avoid significant departures from the SDI historically accepted values and the previous edition's Table D5, $\phi = 0.70$ and $\Omega = 2.35$ were conservatively selected for wind loads. These values more closely equate to a calibration using $\beta_o \geq 3.0$. Since diaphragm stiffness is typically determined from the test data at 0.4 times the nominal load, this selection also avoids inconsistencies between strength and stiffness service determinations.

Consistent with confidence in construction quality control and the test data, the recalibration provides a distinction between screw fasteners and welded connections for load combinations not involving wind loading. The calibration of resistance to seismic loads is based on a load factor of 1.6 and is consistent with AISC. The safety factor for welded diaphragms subjected to earthquake loading is slightly larger than those for other loading types. That factor is also slightly larger than the recalibration suggested. The increase is due to the greater toughness demands required by seismic loading, uncertainty over load magnitudes, and concern over weld quality control. When the load factor for earthquake loading is one, the 0.7 multiplier of ASCE 7 - 98 is allowed in ASD and the safety factors of Table D5 apply. If a local code requires a seismic load factor of 1.6, the factors of Table D5 still apply.

The Steel Deck Institute (1987) and the Department of Army (1992) have consistently recommended a safety factor of two to limit "out of plane buckling" of diaphragms. Out of plane buckling is related to panel profile, while the other diaphragm limit state is connection related. The remainder of the *Specification* requires different safety and resistance factors for the two limit states and larger safety factors for connection controlled states. The safety and resistance factor for panel buckling were changed and the limit state being considered was clarified relative to the previous edition. The prescribed factors for out of plane panel buckling are constants for all loading types.

The *Specification* allows mechanical fasteners other than screws. The diaphragm shear value using any fastener must not be based on a safety factor less than the individual fastener shear strength safety factor unless: 1) sufficient data exists to establish a system effect, 2) an analytical method is established from the tests, and 3) test limits are stated.

D6 Metal Roof and Wall Systems

For members with one flange connected to deck or metal sheathing, the member flexural and compression strengths as well as bracing requirements are provided in *Specification* Section D6.

D6.1 Purlins and Girts and Other Members

D6.1.1 Flexural Members Having One Flange Through-Fastened to Deck or Sheathing

For beams having the tension flange attached to deck or sheathing and the compression flange unbraced, e.g., a roof purlin or wall girt subjected to wind suction, the bending capacity is less than a fully braced member, but greater than an unbraced member. This partial restraint is a function of the rotational stiffness provided by the panel-to-purlin connection. The *Specification* contains factors that represent the reduction in capacity from a fully braced condition. These factors are based on experimental results obtained for both simple and continuous span purlins (Pekoz and Soroushian, 1981 and 1982; LaBoube, 1986; Haussler and Pahers, 1973; LaBoube, et al., 1988; Haussler, 1988; Fisher, 1996).

The R factors for simple span C-sections and Z-sections up to 8.5 inches (216 mm) in depth have been increased from the 1986 *Specification,* and a member design yield stress limit is added based on the work by Fisher (1996).

As indicated by LaBoube (1986), the rotational stiffness of the panel-to-purlin connection is primarily a function of the member thickness, sheet thickness, fastener type and fastener location. To ensure adequate rotational stiffness of the roof and wall systems designed using the AISI provisions, *Specification* Section D6.1.1 explicitly states the acceptable panel and fastener types.

Continuous beam tests were made on three equal spans and the R values were calculated from the failure loads using a maximum positive moment, $M = 0.08\ wL^2$.

The provisions of *Specification* Section D6.1.1 apply to beams for which the tension flange is attached to deck or sheathing and the compression flange is completely unbraced. Beams with discrete point braces on the compression flange may have a bending capacity greater than those completely unbraced. Available data from simple span tests (Pekoz and Soroushian, 1981 and 1982; LaBoube and Thompson, 1982a; LaBoube, et al., 1988; LaBoube and Golovin, 1990) indicate that for members having a lip edge stiffener at an angle of 75 degrees or greater with the plane of the compression flange and braces to the compression flange located at third points or more frequently, member capacities may be increased over those without discrete braces.

For the LRFD method, the use of the reduced nominal flexural strength [resistance] (*Specification* Equation D6.1.1-1) with a resistance factor of $\phi_b = 0.90$ provides the β values varying from 1.5 to 1.60 which are satisfactory for the target value of 1.5. This analysis was based on the load combination of 1.17 W - 0.9D using a reduction factor of 0.9 applied to the load factor for the nominal wind load, where W and D are nominal wind and dead loads, respectively (Hsiao, Yu and Galambos, 1988a; AISI, 1991).

In 2007 the panel depth was reduced from 1-1/4 inch (32 mm) to 1-1/8 inch (29 mm). This reduction in depth was justified because the behavior during full-scale tests indicated that the panel deformation was restricted to a relatively small area around the screw attachment of the panel to the purlin. Also, tests by LaBoube (1986) demonstrated that the panel depth did not influence the rotational stiffness of the panel to purlin attachment.

Prior to the 2001 edition, the *Specification* specifically limited the applicability of these provisions to continuous purlin systems in which any given span length did not vary from any other span length by more than 20 percent. This limitation was included in recognition of the fact that the research was based on systems with equal bay spacing. In 2007, the *Specification* was revised to permit purlin systems with adjacent span lengths varying more than 20 percent to use the reduction factor, R, for the simply supported condition. The revision allows a row of continuous purlins to be treated with a continuous beam condition R-factor in some bays and a simple span beam condition R-factor in others. The 20 percent span variation rule is a local effect and as such, only variation in adjacent spans is relevant.

D6.1.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

The design provision of this section is only applicable to the United States and Mexico. The discussion for this section is provided in the *Commentary* on Appendix A. ☜A

D6.1.3 Compression Members Having One Flange Through-Fastened to Deck or Sheathing

For axially loaded C- or Z- sections having one flange attached to deck or sheathing and the other flange unbraced, e.g., a roof purlin or wall girt subjected to wind or seismic generated compression forces, the axial load capacity is less than a fully braced member, but greater than an unbraced member. The partial restraint relative to weak axis buckling is a function of the rotational stiffness provided by the panel-to-purlin connection. *Specification* Equation D6.1.3-1 is used to calculate the weak axis capacity. This equation is not valid for sections attached to standing seam roofs. The equation was developed by Glaser, Kaehler and Fisher (1994) and is also based on the work contained in the reports of Hatch, Easterling and Murray (1990) and Simaan (1973).

A limitation on the maximum yield stress of the C- or Z- section is not given in the *Specification* since *Specification* Equation D6.1.3-1 is based on elastic buckling criteria. A limitation on minimum length is not contained in the *Specification* because Equation D6.1.3-1 is conservative for spans less than 15 feet. The gross area, A, has been used rather than the effective area, A_e, because the ultimate axial stress is generally not large enough to result in a significant reduction in the effective area for common cross section geometries.

As indicated in the *Specification*, the strong axis axial load capacity is determined assuming that the weak axis of the strut is braced.

The controlling axial capacity (weak or strong axis) is suitable for usage in the combined axial load and bending equations in Section C5 of the *Specification* (Hatch, Easterling, and Murray, 1990).

D6.1.4 Compression of Z-Section Members Having One Flange Fastened to a Standing Seam Roof

The design provision of this section is only applicable to the United States and Mexico. The discussion for this section is provided in the *Commentary* on Appendix A. ☜A

D6.2 Standing Seam Roof Panel Systems

D6.2.1 Strength [Resistance] of Standing Seam Roof Panel Systems

Under gravity loading, the nominal strength [nominal resistance] of many panels can be calculated accurately. Under uplift loading, nominal strength [nominal resistance] of standing seam roof panels and their attachments or anchors cannot be calculated with accuracy. Therefore, it is necessary to determine the nominal strength [nominal resistance] by testing. Three test protocols have been used in this effort: FM 4471 developed by Factory Mutual, CEGS 07416 by the Corps of Engineers and E1592 by ASTM. In Supplement No. 1 to the 1996 Edition of the *Specification*, (AISI, 1999), only the ASTM E1592-95 procedure was approved. In 2004, the Factory Mutual and Corps of Engineers protocols were also approved, provided that testing was in accordance with the AISI test procedure defined in S906. While these test procedures have a common base, none defines a design strength [factored resistance]. *Specification* Section D6.2.1 and AISI S906, "Standard Procedures for Panel and Anchor Structural Tests", adopted in 1999, added closure to the question by defining appropriate resistance and safety factors. The safety factors determined in Section D6.2.1 will vary depending on the characteristics of the test data. In 2006 limits were placed on the safety factor and resistance factor determined in this section, to require a minimum safety factor of 1.67 and a maximum resistance factor of 0.9.

The *Specification* permits end conditions other than those prescribed by ASTM E1592-01. Areas of the roof plane that are sufficiently far enough away from crosswise restraint can be simulated by testing the open/open condition that was permitted in the 1995 edition of ASTM E1592. In addition, eave and ridge configurations that do not provide crosswise restraint can be evaluated.

The relationship of strength [resistance] to serviceability limits may be taken as strength limit/serviceability limit = 1.25, or

$$\Omega_{serviceability} = \Omega_{strength}/1.25 \qquad \text{(C-D6.2.1-1)}$$

It should be noted that the purpose of the test procedure specified in *Specification* Section D6.2.1 is not to set up guidelines to establish the serviceability limit. The purpose is to define the method of determining the available strength [factored resistance] whether based on the serviceability limit or on the nominal strength [resistance]. The Corps of Engineers Procedure CEGS 07416 (1991) requires a safety factor of 1.65 on strength [resistance] and 1.3 on serviceability. A buckling or crease does not have the same consequences as a failure of a clip. In the latter case, the roof panel itself may become detached and expose the contents of a building to the elements of the environment. Further, Galambos (1988a) recommended a value of 2.0 for the target reliability index, β_o, when slight damage is expected and a value of 2.5 when moderate damage is expected. The resulting ratio is 1.25.

In *Specification* Section D6.2.1, a target reliability index of 2.5 is used for connection limits. It is used because the consequences of a panel fastener failure (β_o = 2.5) are not nearly so severe as the consequences of a primary frame connection failure (β_o = 3.5). The intermittent nature of wind load as compared to the relatively long duration of snow load further justifies the use of β_o = 2.5 for panel anchors. In *Specification* Section D6.2.1, the coefficient of variation of the material factor, V_M, is recommended to be 0.08 for failure limited by anchor or connection failure, and 0.10 for limits caused by flexural or other modes of failure. *Specification* Section D6.2.1 also eliminates the limit on coefficient of variation of the test results, V_p, because consistent test results often lead to V_p values lower than the 6.5 percent value set in *Specification* Section F1. The elimination of the limit will be beneficial when test results are consistent.

The value for the number of tests for fasteners is set as the number of anchors tested with the same tributary area as the anchor that failed. This is consistent with design practice where anchors are checked using a load calculated based on tributary area. Actual anchor loads are not calculated from a stiffness analysis of the panel in ordinary design practice.

D6.3 Roof System Bracing and Anchorage

D6.3.1 Anchorage of Bracing for Purlin Roof Systems Under Gravity Load with Top Flange Connected to Metal Sheathing

In metal roof systems utilizing C- or Z-purlins, the application of gravity loads will cause torsion in the purlin and lateral displacements of the roof system. These effects are due to the slope of the roof, the loading of the member eccentric to its shear center, and for Z-purlins, the inclination of the principal axes. The torsional effects are not accounted for in the design provisions of Sections C3.1 and D6.1, and lateral displacements may create instability in the system. Lateral restraint is typically provided by the roof sheathing and lateral anchorage devices to minimize the lateral movement and the torsional effects. The anchorage devices are designed to resist the lateral anchorage force and provide the appropriate level of stiffness to ensure the overall stability of the purlins.

The calculation procedure in *Specification* Equations D6.3.1-1 through D6.3.1-6 determines the anchorage force by first calculating an upper bound force for each purlin, P_i, at the line of anchorage. This upper bound force is then distributed to anchorage devices and reduced due to the system stiffness based on the relative effective stiffness of each component. For the calculation procedure, the anchorage devices are modeled as linear springs located at the top of the purlin web. The stiffness of anchorage devices that do not attach at this location must be adjusted, through analysis or testing, to an equivalent lateral stiffness at the top of the web. This adjustment must include the influence of the attached purlin but not include any reduction due to the flexibility of the sheathing to purlin connection. *Specification* Equation D6.3.1-4 establishes an effective lateral stiffness for each anchorage device, relative to each purlin, that has been adjusted for the flexibility of the roof system between the purlin location and the anchorage location. It is important to note that the units of A_p are area per unit width. Therefore the bay length, L, in this equation must have units consistent with the unit width used for establishing A_p. The resulting product, LA_p, has units of area. The total effective stiffness for a given purlin is then calculated with *Specification* Equation D6.3.1-5 by summing the effective stiffness relative to each anchorage device and the system stiffness from *Specification* Equation D6.3.1-6. The force generated by an individual purlin is calculated by Equation D6.3.1-2, and then distributed to an anchorage device based on the relative stiffness ratio in *Specification* Equation D6.3.1-1.

Lateral bracing forces will accumulate within the roof sheathing, and must be transferred into the anchorage devices. The strength of the elements in this load path must be verified. AISI S912, Test Procedures for Determining a Strength Value for a Roof Panel-to-Purlin-to-Anchorage Device Connection, provides a means to determine a lower bound strength [resistance] for the complete load path. For through-fastened roof systems, this strength [resistance] value can be reasonably estimated by rational analysis by assuming that the roof fasteners within twelve inches of the anchorage device participate in the force transfer.

The 1986 through 2001 *Specifications* included brace force equations that were based on the work by Murray and Elhouar (1985) with various extensions from subsequent work. The original work assumed the applied loading was parallel to the purlin webs. The later addition of the "cosθ" and "sinθ" terms attempted to account for the roof slope but it failed to correctly model the system effect for higher sloped roofs. Tests by Lee and Murray (2001) and Seek and Murray (2004) showed generally that the brace force equations conservatively predicted the lateral anchorage forces at slopes less than 1:12 but predicted unconservative lateral anchorage forces at steeper slopes. The new procedure outlined in *Specification* Section D6.3.1 was formulated to correlate better with test results. Also, the original work was based on the application of one anchorage device to a group of purlins. Until the work of Sears and Murray (2007) a generally accepted manual technique to extend this procedure to roofs with multiple anchors was not available.

Prior to the work by Seek and Murray (2006, 2007) and Sears and Murray (2007), the anchorage devices were assumed to have a constant and relatively high lateral stiffness. The current provisions recognize the finite stiffness of the anchorage device, and the corresponding decrease in anchorage forces for more flexible anchorage devices. *Specification* Equation D6.3.1-7 establishes a minimum effective stiffness that must be provided to limit the lateral displacement at the anchorage device to d/20. This required stiffness does not represent the required stiffness of each anchorage device, but instead the total stiffness provided by the stiffness of the purlin system (K_{sys}) and the anchorage devices relative to the most remote purlin.

Several alternative rational analysis methods have been developed to predict lateral anchorage forces for Z-section roof systems. A method for calculating lateral anchorage forces is presented by Seek and Murray (2006, 2007). The method is similar to the procedure outlined in *Specification* Section D6.3.1 but uses a more complex method derived from mechanics to determine the lateral force introduced into the system at each Z-section, P_i, and distributes the force to the components of the system according to the relative lateral stiffness of each of the components. The method is more computationally intensive but allows for analysis of more complex bracing configurations such as supports plus third points lateral anchorage and supports plus third points torsional braces.

A method to predict lateral anchorage forces using the finite element method is presented in Seek and Murray (2004). The model uses shell finite elements to model the Z-sections and sheathing in the roof system. The model accurately represents Z-section behavior and is capable of handling configurations other than lateral anchorage applied at the top flange. However, the computational complexity limits the size of the roof system that can be modeled by this method.

Rational analysis may also be performed using the elastic stiffness model developed by Sears and Murray (2007) upon which the provisions of *Specification* Section D6.3.1 are based. The model uses frame finite elements to represent the Z-sections and a truss system to represent the diaphragm. The model is computationally efficient allowing for analysis of large systems.

D6.3.2 Alternate Lateral and Stability Bracing for Purlin Roof Systems

Tests (Shadravan and Ramseyer, 2007) have shown that C- and Z-sections can reach the capacity determined by *Specification* Section C3.1 through the application of torsional braces along the span of the member. Torsional braces applied between pairs of purlins prevent twist of the section at a discrete location. The moments developed due to the torsional brace can be resolved by forces in the plane of the web of each section and do not require external anchorage at the location of the brace. The vertical forces should, however, be accounted for when determining the applied load on the section.

Torsional braces should be applied at or near each flange of the Z- or C-section to prevent deformation of the web of the section and insure the effectiveness of the brace. When twist of the section is thus prevented, a section may deflect laterally and retain its strength [resistance]. Second order moments can be resisted by the rotational restraints. Therefore, a more liberal lateral deflection of L/180 between the supports is permitted for a C- or Z- section with torsional braces. Anchorage is required at the frame line to prevent excessive deformation at the support location that undermines the strength [resistance] of the section. A lateral displacement limit therefore is imposed along the frame lines to insure that adequate restraint along the frame lines is provided.

E. CONNECTIONS AND JOINTS

E1 General Provisions

Welds, bolts, screws, rivets, and other special devices such as metal stitching and adhesives are generally used for cold-formed steel connections (Brockenbrough, 1995). The 2007 edition of the *Specification* contains provisions in Chapter E for welded connections, bolted connections, and screw connections. Among the above three commonly used types of connections, the design provisions for using screws were developed in 1993 and were included in the 1996 AISI *Specification* for the first time. The following brief discussions deal with the applications of rivets and other special devices:

(a) *Rivets*

While hot rivets have little application in cold-formed steel construction, cold rivets find considerable use, particularly in special forms, such as blind rivets (for application from one side only), tubular rivets (to increase bearing area), high shear rivets, and explosive rivets. For the design of connections using cold rivets, the provisions for bolted connections may be used as a general guide, except that the shear strength [resistance] of rivets may be quite different from that of bolts. Additional design information on the strength [resistance] of rivets should be obtained from manufacturers or from tests.

(b) *Special devices*

Special devices include: (1) metal stitching, achieved by tools that are special developments of the common office stapler, and (2) connecting by means of special clinching tools that draw the sheets into interlocking projections.

Most of these connections are proprietary devices for which information on strength [resistance] of connections must be obtained from manufacturers or from tests carried out by or for the user. Guidelines provided in *Specification* Chapter F are to be used in these tests.

The plans and/or specifications are to contain adequate information and design requirement data for the adequate detailing of each connection if the connection is not detailed on the engineering design drawings.

In this edition of the *Specification*, the ASD, LRFD and LSD design provisions for welded and bolted connections were based on the 1996 edition of the AISI *Specification* with some revisions and additions which will be discussed in subsequent sections.

E2 Welded Connections

Welds used for cold-formed steel construction may be classified as fusion welds (or arc welds) and resistance welds. Fusion welding is used for connecting cold-formed steel members to each other as well as connecting such members to heavy, hot-rolled steel framing (such as floor panels to beams of the steel frame). It is used in groove welds, arc spot welds, arc seam welds, fillet welds, and flare groove welds.

The design provisions contained in this *Specification* section for fusion welds have been based primarily on experimental evidence obtained from an extensive test program conducted at Cornell University. The results of this program are reported by Pekoz and McGuire (1979) and summarized by Yu (2000). All possible failure modes are covered in the *Specification* since 1996, whereas the earlier *Specification* mainly dealt with shear failure.

For most of the connection tests reported by Pekoz and McGuire (1979), the onset of yielding was either poorly defined or followed closely by failure. Therefore, in the provisions of this section, rupture rather than yielding is used as a more reliable criterion of failure.

The welded connection tests, which served as the basis of the provisions given in *Specification* Sections E2.1 through E2.5, were conducted on sections with single and double sheets. See *Specification* Figures E2.2-1 and E2.2-2. The largest total sheet thickness of the cover plates was approximately 0.15 inch (3.81 mm). However, within this *Specification*, the validity of the equations was extended to welded connections in which the thickness of the thinnest connected part is 0.18 inch (4.57 mm) or less. For arc spot welds, the maximum thickness of a single sheet (*Specification* Figure E2.2.1.2-1) and the combined thickness of double sheets (*Specification* Figure E2.2.1.2-2) are set at 0.15 inch (3.81 mm).

In 2001, the safety factors and resistance factors in this section were modified for consistency based on the research work by Tangorra, Schuster, and LaBoube (2001).

For design tables and example problems on welded connections, see Part IV of the *Design Manual* (AISI, 2008).

See Appendix A or B for additional commentary. ☜ **A,B**

E2.1 Groove Welds in Butt Joints

The design equations for determining nominal strength [resistance] for groove welds in butt joints have been taken from the AISC LRFD *Specification* (AISC, 1993). Therefore, the AISC definition for the effective throat thickness, t_e, is equally applicable to this section of the *Specification*. Prequalified joint details are given in AWS D1.3-98 (AWS, 1998) or other equivalent weld standards.

E2.2 Arc Spot Welds

Arc spot welds (puddle welds) used for connecting thin sheets are similar to plug welds used for relatively thicker plates. The difference between plug welds and arc spot welds is that the former are made with prepunched holes, but for the latter no prepunched holes are required. Instead, a hole is burned in the top sheet by the arc and then filled with weld metal to fuse it to the bottom sheet or a framing member. The provisions of Section E2.2 apply to plug welds as well as spot welds.

E2.2.1 Shear

E2.2.1.1 Minimum Edge Distance

The edge distance requirements provided in the *Specification* Section E2.2.1.1 are to ensure the connection provides the sufficient strength for preventing shear failure of connected part in the direction of stress. Compared with previous editions of the AISI *Specification*, the limiting F_u/F_{sy} ratio was revised to be consistent with *Specification* Section A2.3.1.

E2.2.1.2 Shear Strength [Resistance] for Sheet(s) Welded to a Thicker Supporting Member

The Cornell tests (Pekoz and McGuire, 1979) identified four modes of failure for arc spot welds, which are addressed in this *Specification* section. They are: (1) shear failure of welds in the fused area, (2) tearing of the sheet along the contour of the weld with the tearing spreading the sheet at the leading edge of the weld, (3) sheet tearing combined with buckling near the trailing edge of the weld, and (4) shearing of the sheet behind weld. It should be noted that many failures, particularly those of the plate tearing type, may be preceded or accompanied by considerable inelastic out-of-plane deformation of the type indicated in Figure C-E2.2-1. This form of behavior is similar to that observed in wide, pin-connected plates. Such behavior should be avoided by closer spacing of welds. When arc spot welds are used to connect two sheets to a framing member as shown in *Specification* Figure E2.2.1.1-2, consideration should also be given to the possible shear failure between thin sheets.

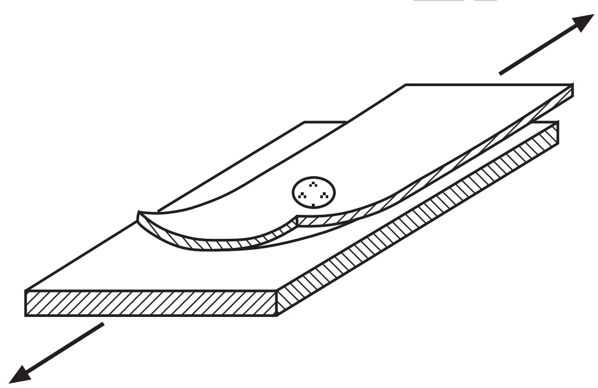


Figure C-E2.2-1 Out of Plane Distortion of Welded Connection

The thickness limitation of 0.15 inch (3.81 mm) is due to the range of the test program that served as the basis of these provisions. On sheets below 0.028 inch (0.711 mm) thick, weld washers are required to avoid excessive burning of the sheets and, therefore, inferior quality welds.

In the AISI 1996 *Specification*, Equation E2.2-1 was revised to be consistent with the research report (Pekoz and McGuire, 1979).

In 2001, the equation used for determining d_a for multiple sheets was revised to be (d-t).

E2.2.1.3 Shear Strength [Resistance] for Sheet-to-Sheet Connections

The Steel Deck Institute Design Manual (SDI, 1987) stipulates that the shear strength for a sheet-to-sheet arc spot weld connection be taken as 75% of the strength of a sheet-to-structural connection. SDI further stipulates that the sheet-to-structural connection strength [resistance] be defined by *Specification* Equation E2.2.1.2-2. This design provision was adopted by the *Specification* in 2004. Prior to accepting the SDI design recommendation, a review of the pertinent research by Luttrell (SDI, 1987) was performed by LaBoube (LaBoube, 2001). The test data thickness range that is reflected in the *Specification* documents the scope of Luttrell's test program. SDI suggests that sheet-to-sheet welds are problematic for thickness less than 0.0295 in. (0.75 mm). Such welds result in "blow holes" but the perimeter must be fused to be effective.

Quality control for sheet–to-sheet connections is not within the purview of AWS D1.3. However, using AWS D1.3 as a guide, the following quality control/assurance guidelines are suggested:

(1) Measure the visible diameter of the weld face,
(2) Ensure no cracks in the welds,
(3) Maximum undercut = 1/8 of the weld circumference, and
(4) Sheets are to be in contact with each other.

E2.2.2 Tension

For tensile capacity of arc spot welds, the design provisions in the 1989 Addendum were based on the tests reported by Fung (1978) and the study made by Albrecht (1988). Those provisions were limited to sheet failure with restrictive limitations on material properties and sheet thickness. These design criteria were revised in 1996 because the tests conducted at the University of Missouri-Rolla (LaBoube and Yu, 1991 and 1993) have shown that two potential limit states may occur. The most common failure mode is that of sheet tearing around the perimeter of the weld. This failure condition was found to be influenced by the sheet thickness, the average weld diameter, and the material tensile strength. In some cases, it was found that tensile failure of the weld can occur. The strength [resistance] of the weld was determined to be a function of the cross-section of the fused area and tensile strength of the weld material. Based on analysis by LaBoube (LaBoube, 2001), the nominal strength [resistance] equation was changed in 2001 to reflect the ductility of the sheet, F_u/F_y, and the sheet thickness, the average weld diameter, and the material tensile strength.

The multiple safety factors and resistance factors recognize the behavior of a panel system with many connections versus the behavior of a member connection and the potential for a catastrophic failure in each application. In *Specification* Section E2.2.2 a target reliability index of 3.0 for the United States and Mexico and 3.5 for Canada is used for the panel connection limit, whereas a target reliability index of 3.5 for the United States and Mexico and 4 for Canada is used for the other connection limit. Precedence for the use of a smaller target reliability index for systems was established in Section D6.2.1 of the *Specification*.

Tests (LaBoube and Yu, 1991 and 1993) have also shown that when reinforced by a weld washer, thin sheet weld connections can achieve the design strength [resistance] given by *Specification* Equation E2.2.2-2 using the thickness of the thinner sheet.

The equations given in the *Specification* were derived from the tests for which the applied tension load imposed a concentric load on the weld, as would be the case, for example, for the interior welds on a roof system subjected to wind uplift. Welds on the perimeter of a roof or floor system would experience an eccentric tensile loading due to wind uplift. Tests have shown that as much as a 50 percent reduction in nominal connection strength [resistance] could occur because of the eccentric load application (LaBoube and Yu, 1991 and 1993). Eccentric conditions may also occur at connection laps depicted by Figure C-E2.2-2.

At a lap connection between two deck sections as shown in Figure C-E2.2-2, the length of the unstiffened flange and the extent of the encroachment of the weld into the unstiffened flange have a measurable influence on the strength [resistance] of the welded connection (LaBoube and Yu, 1991). The *Specification* recognizes the reduced capacity of this connection detail by imposing a 30 percent reduction on the calculated nominal strength [resistance].

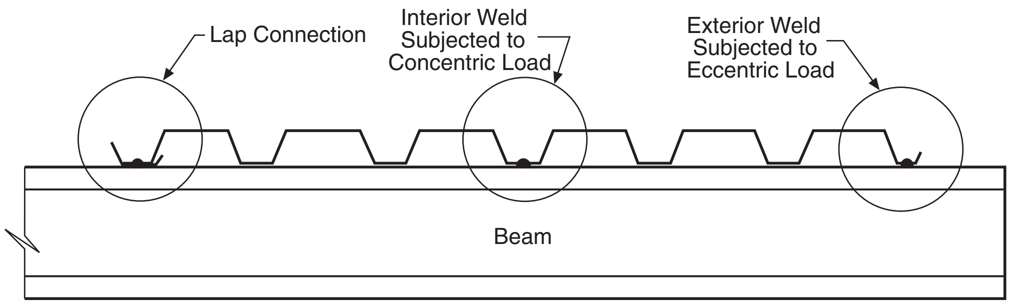


Figure C-E2.2-2 Interior Weld, Exterior Weld and Lap Connection

E2.3 Arc Seam Welds

The general behavior of arc seam welds is similar to that of arc spot welds. No simple shear failures of arc seam welds were observed in the Cornell tests (Pekoz and McGuire, 1979). Therefore, *Specification* Equation E2.3-1, which accounts for shear failure of welds, is adopted from the AWS welding provisions for sheet steel (AWS, 1998).

Specification Equation E2.3-2 is intended to prevent failure by a combination of tensile tearing plus shearing of the cover plates.

E2.4 Fillet Welds

For fillet welds on the lap joint specimens tested in the Cornell research (Pekoz and McGuire, 1979), the dimension, w_1, of the leg on the sheet edge generally was equal to the sheet thickness; the other leg, w_2, often was two or three times longer than w_1 (See *Specification* Figure E2.4-1). In connections of this type, the fillet weld throat commonly is larger than the throat of a conventional fillet welds of the same size. Usually ultimate failure of fillet welded joints has been found to occur by the tearing of the plate adjacent to the weld, See Figure C-E2.4-1.

In most cases, the higher strength of the weld material prevents weld shear failure, therefore, the provisions of this *Specification* section are based on sheet tearing. Because specimens up to 0.15 inch (3.81 mm) thickness were tested in the Cornell research (Pekoz and McGuire, 1979), the last provision in this section is to cover the possibility that for sections thicker than 0.15 inch (3.81 mm), the throat dimension may be less than the thickness of the cover plate and the tear may occur in the weld rather than in the plate material. Additional research at the University of Sydney (Zhao and Hancock, 1995) has further indicated that weld throat failure may even occur between the thickness of 0.10 in. (2.54 mm) to 0.15 in. (3.81 mm). Accordingly, the *Specification* was revised, in 2001, to require weld strength [resistance] check when the plate thickness is greater than 0.10 in. (2.54 mm). For high strength materials with yield stress of 65 ksi (448 MPa) or higher, research at the University of Sydney (Teh and Hancock, 2000) has shown that weld throat failure does not occur in materials less than 0.10 in. (2.54 mm) thick and that the AISI *Specification* provisions based on sheet strength are satisfactory for high strength material less than 0.10 in. (2.54 mm) thick.

Prequalified fillet welds are given in AWS D1.3-98 (AWS, 1998) or other equivalent weld standards.

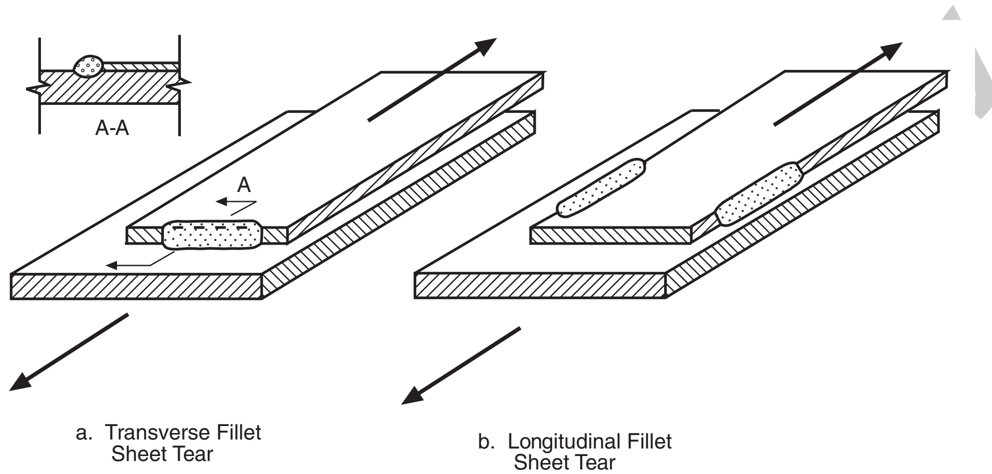


Figure C-E2.4-1 Fillet Weld Failure Modes

E2.5 Flare Groove Welds

The primary mode of failure in cold-formed steel sections welded by flare groove welds, loaded transversely or longitudinally, also was found to be sheet tearing along the contour of the weld. See Figure C-E2.5-1.

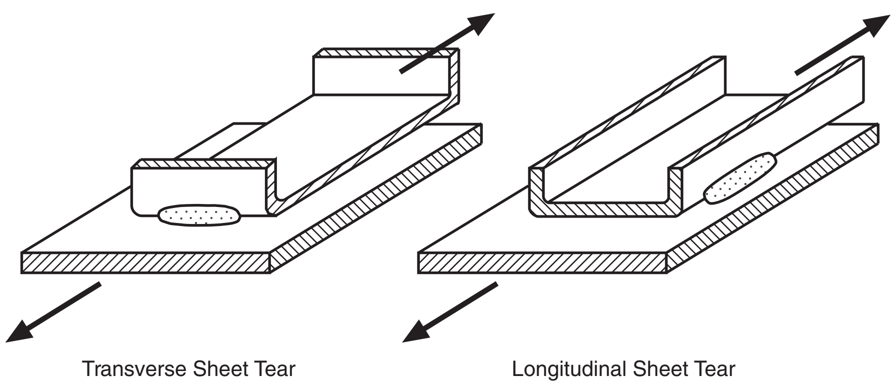


Figure C-E2.5-1 Flare Groove Weld Failure Modes

Except for *Specification* Equation E2.5-4, the provisions of this *Specification* section are intended to prevent shear tear failure. *Specification* Equation E2.5-4 covers the possibility that thicker sections may have effective throats less than the thickness of the channel and weld failure may become critical.

In the 1996 edition of the AISI *Specification*, the former *Specification* Figure E2.5-4 was replaced by four new drawings to describe in more detail the different possible flare bevel groove weld uses. *Specification* Figures E2.5-4 and E2.5-5 show the condition where the weld is filled flush to the surface. This weld is a prequalified weld in AWS D1.3-98 (AWS, 1998) which provides the definition of the effective throat for this type of weld. The distinction of double and single shear requirements in the *Specification* for flare groove welds is indicated on these figures. *Specification* Figures E2.5-6 and E2.5-7 show flare bevel groove welds which are frequently used in cold-formed steel construction in which the weld is not filled flush to the surface. The vertical leg of the weld can either be greater, Figure E2.5-6, or less, Figure E2.5-7, than the radius of outside bend surface. The definition of the horizontal leg of the weld in each case is slightly different as indicated. No change was needed in the *Specification* requirements from previous editions except in the definitions of the effective throat for use in *Specification* Equation E2.5-4.

In 2001, the *Specification* was revised to require that weld strength be checked when the plate thickness is greater than 0.10 in. (2.54 mm) based on the research by Zhao and Hancock (1995).

E2.6 Resistance Welds

The shear values for outside sheets of 0.125 inch (3.18 mm) or less in thickness are based on "Recommended Practice for Resistance Welding Coated Low-Carbon Steels," AWS C1.3-70, (Table 2.1 - Spot Welding Galvanized Low-Carbon Steel). Shear values for outside sheets thicker than 0.125 inch (3.18 mm) are based upon "Recommended Practices for Resistance Welding," AWS C1.1-66, (Table 1.3 - Pulsation Welding Low-Carbon Steel) and apply to pulsation welding as well as spot welding. They are applicable for all structural grades of low-carbon steel, uncoated or galvanized with 0.90 oz/ft^2 (275 g/m^2) of sheet, or less, and are based on values selected from AWS C1.3-70, Table 2.1; and AWS C1.1-66, Table 1.3. The above values may also be applied to medium carbon and low-alloy steels. Spot welds in such steels give somewhat higher shear strengths than those upon which the above values are based; however, they may require special welding conditions. In view of the fact that AWS C1.1-66 and AWS C1.3-70 Standards were incorporated in AWS C1.1-2000, resistance welds should be performed in accordance with AWS C1.1-2000 (AWS, 2000).

In the 2001 edition and this edition of the *Specification*, a design equation is used to determine the nominal shear strength [resistance] that replaces the tabulated values given in the previous specifications. The upper limit of *Specification* Equations E2.6-1, E2.6-3 and E2.6-5 is selected to best fit the data provided in AWS C1.3-70, Table 2.1 and AWS C1.1-66, Table 1.3. Shear strength [resistance] values for welds with the thickness of the thinnest outside sheet greater than 0.180 in. (4.57 mm) have been excluded in *Specification* Equations E2.6-2, E2.6-4 and E2.6-6 due to the thickness limit set forth in *Specification* Section E2.

E2.7 Rupture in Net Section of Members other than Flat Sheets (Shear Lag)

Shear lag has a debilitating effect on the nominal tensile strength [resistance] of a cross section. The AISI *Specification* addresses the shear lag effect on tension members other than flat sheets in welded connections. The AISC Specification's design approach has been adopted.

When computing U for combinations of longitudinal and transverse welds, L is taken as the length of the longitudinal weld because the transverse weld does little to minimize shear lag. For angle or channel sections, the distance, $\bar{x}$, from shear plane to centroid of the cross section is defined in Figure C-E2.7.

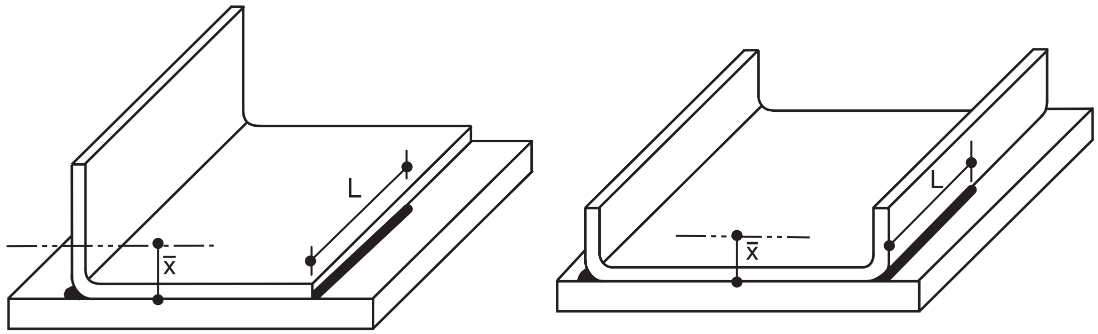


Figure C-E2.7 $\bar{x}$ Definition for Sections with Fillet Welding

E3 Bolted Connections

The structural behavior of bolted connections in cold-formed steel construction is somewhat different from that in hot-rolled heavy construction, mainly because of the thinness of the connected parts. Prior to 1980, the provisions included in the AISI *Specification* for the design of bolted connections were developed on the basis of the Cornell tests (Winter, 1956a, 1956b). These provisions were updated in 1980 to reflect the results of additional research performed in the United States (Yu, 1982) and to provide a better coordination with the specifications of the Research Council on Structural Connections (RCSC, 1980) and AISC (1978). In 1986, design provisions for maximum size of bolt holes and the allowable tension stress for bolts were added in the AISI *Specification* (AISI, 1986). In the 1996 edition of the AISI *Specification*, minor changes of the safety factors were made for computing the allowable and design tensile and shear strengths [resistances] of bolts. The allowable tension stress for the bolts subject to the combination of shear and tension was determined by the equations provided in *Specification* Table E3.4-2 with the applicable safety factor.

(a) Scope

Previous studies and practical experiences have indicated that the structural behavior of bolted connections used for joining *relatively thick* cold-formed steel members is similar to that for connecting hot-rolled shapes and built-up members. The AISI *Specification* criteria are applicable only to cold-formed steel members or elements less than 3/16 inch (4.76 mm) in thickness. For materials not less than 3/16 inch (4.76 mm), reference is made to the specifications or standards stipulated in Section E3a of Appendix A or B.

Because of lack of appropriate test data and the use of numerous surface conditions, this *Specification* does not provide design criteria for slip-critical (also called friction-type) connections. When such connections are used with cold-formed members where the thickness of the thinnest connected part is less than 3/16 inch (4.76 mm), it is recommended that tests be conducted to confirm their design capacity. The test data should verify that the specified design capacity for the connection provides a sufficient safety against initial slip at least equal to that implied by the provisions of the specifications or standards listed in Section E3a of Appendix A or B. In addition, the safety against ultimate capacity should be at least equal to that implied by this *Specification* for bearing-type connections.

The *Specification* provisions apply only when there are no gaps between plies. The designer should recognize that the connection of a rectangular tubular member by means of bolt(s) through such members may have less strength [resistance] than if no gap existed. Structural performance of connections containing unavoidable gaps between plies would require tests in accordance with *Specification* Section F1.

(b) Materials

This section lists five different types of fasteners which are normally used for cold-formed steel construction. In view of the fact that A325 and A490 bolts are available only for diameters of 1/2 inch (12.7 mm) and larger, A449 and A354 Grade BD bolts should be used as an equivalent of A325 and A490 bolts, respectively, whenever smaller bolts (less than 1/2 inch (12.7 mm) in diameter) are required.

During recent years, other types of fasteners, with or without special washers, have been widely used in steel structures using cold-formed steel members. The design of these fasteners should be determined by tests in accordance with Chapter F of this *Specification*.

(c) Bolt Installation

Bolted connections in cold-formed steel structures use either mild or high-strength steel bolts and are designed as a bearing-type connection. Bolt pretensioning is not required because the ultimate strength of a bolted connection is independent of the level of bolt preload. Installation must ensure that the bolted assembly will not come apart during service. Experience has shown that bolts installed to a snug tight condition do not loosen or "back-off" under normal building conditions and are not subject to vibration or fatigue.

Bolts in slip-critical connections, however, must be tightened in a manner which assures the development of the fastener tension forces required by the Research Council on Structural Connections (1985 and 2000) for the particular size and type of bolts. Turn-of-nut rotations specified by the Research Council on Structural Connections may not be applicable because such rotations are based on larger grip lengths than are encountered in usual cold-formed construction. Reduced turn-of-the-nut values would have to be established for the actual combination of grip and bolt. A similar test program (RCSC, 1985 and 1988) could establish a cut-off value for calibrated wrenches. Direct tension indicators (ASTM F959), whose published clamping forces are independent of grip, can be used for tightening slip-critical connections.

(d) Hole Sizes

Design information for oversized and slotted holes is included in the Appendices because such holes are often used in practice to meet dimensional tolerances during erection.

☜A,B

E3.1 Shear, Spacing and Edge Distance

The design provisions of this section are given in Section E3.1 of Appendix A. The discussion for this section is provided in the *Commentary* on the corresponding Appendix.

☜A

E3.2 Rupture in Net Section (Shear Lag)

The design provisions of this section are given in Section E3.2 of Appendix A. The discussion for this section is provided in the *Commentary* on the corresponding Appendix.

☜A

E3.3 Bearing

Previous bolted connection tests have shown that the bearing strength [resistance] of bolted connections depends on (1) the tensile strength F_u of the connected parts, (2) the thickness of connected parts, (3) the diameter of bolt, (4) joints with single shear and double shear conditions, (5) the F_u/F_y ratio, and (6) the use of washers (Winter, 1956a and 1956b; Chong and Matlock, 1974; Yu, 1982 and 2000). These design parameters were used in the 1996 and earlier editions of the AISI *Specification* for determining the bearing strength [resistance] between bolt and connected parts (AISI, 1996).

In the Canadian Standard (CSA, 1994), the d/t ratio was also used in the design equation for determining the bearing strength [resistance] of bolted connections.

In this edition of the *Specification*, the design format and tables for determining the bearing strength [resistance] without consideration of bolt hole deformation were revised in 2001 on the basis of the research work conducted at the University of Sydney (Rogers and Hancock, 1998) and at the University of Waterloo (Wallace, Schuster, and LaBoube, 2001a and 2001b).

E3.3.1 Strength [Resistance] Without Consideration of Bolt Hole Deformation

Rogers and Hancock (Rogers and Hancock, 1998) developed the design equation for bearing of bolted connections with washers (*Specification* Table E3.3.1-1). Based on research at the University of Waterloo (Wallace, Schuster, and LaBoube, 2001a), the Rogers and Hancock equation was extended to bolted connections without washers and to the inside sheet of double shear connections with or without washers (*Specification* Table E3.3.1-2). In *Specification* Table E3.3.1-1, the bearing factor C depends on the ratio of bolt diameter to member thickness, d/t. The design equations in the *Specification* Section E3.3.1 are based on available test data. Thus, for sheets thinner than 0.024 in. (0.61 mm), tests must be performed to determine the structural performance.

The safety factor and resistance factor are based on calibration of available test data (Wallace, Schuter, and LaBoube, 2001b).

E3.3.2 Strength [Resistance] With Consideration of Bolt Hole Deformation

Based on research at the University of Missouri-Rolla (LaBoube and Yu, 1995), design equations have been developed that recognize the presence of hole elongation prior to reaching the limited bearing strength [resistance] of a bolted connection. The researchers adopted an elongation of 0.25 in. (6.4 mm) as the acceptable deformation limit. This limit is consistent with the permitted elongation prescribed for hot-rolled steel.

Since the nominal strength value with consideration of bolt hole deformation should not exceed the nominal strength without consideration of the hole deformation, this limit was added in 2004.

E3.4 Shear and Tension in Bolts

The design provisions of this section are given in Section E3.4 of Appendix A or B. In Appendix A, the commentary is provided for Section E3.4. ☜A

E4 Screw Connections

Results of over 3500 tests worldwide were analyzed to formulate screw connection provisions (Pekoz, 1990). European Recommendations (1987) and British Standards (1992) were considered and modified as appropriate. Since the provisions apply to many different screw connections and fastener details, a greater degree of conservatism is implied than is otherwise typical within this *Specification*. These provisions are intended for use when a sufficient number of test results is not available for the particular application. A higher degree of accuracy can be obtained by testing any particular connection geometry (AISI, 1992).

Over 450 elemental connection tests and eight diaphragm tests were conducted in which compressible fiberglass insulation, typical of that used in metal building roof systems (MBMA, 2002), was placed between the two pieces of steel (between steel sheet samples in the elemental connection tests and between the deck and purlin in the diaphragm tests) (Lease and Easterling, 2006a, 2006b). The results indicate that the equations in Section E4 of the *Specification* are valid for applications that incorporate 6-3/8 in. (162 mm) or less of compressible fiberglass insulation.

Screw connection tests used to formulate the provisions included single fastener specimens as well as multiple fastener specimens. However, it is recommended that at least two screws should be used to connect individual elements. This provides redundancy against under-torquing, over-torquing, etc., and limits lap shear connection distortion of flat unformed members such as straps.

Proper installation of screws is important to achieve satisfactory performance. Power tools with adjustable torque controls and driving depth limitations are usually used.

For the convenience of designers, Table C-E4-1 gives the correlation between the common number designation and the nominal diameter for screws. See Figure C-E4-1 for the

Table C-E4-1 Nominal Diameter for Screws

Number Designation	Nominal Diameter, d in.	mm
0	0.060	1.52
1	0.073	1.85
2	0.086	2.18
3	0.099	2.51
4	0.112	2.84
5	0.125	3.18
6	0.138	3.51
7	0.151	3.84
8	0.164	4.17
10	0.190	4.83
12	0.216	5.49
1/4	0.250	6.35

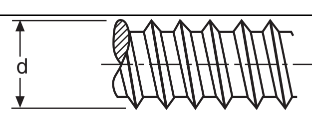


Figure C-E4-1 Nominal Diameter for Screws

measurement of nominal diameters.

E4.1 Minimum Spacing

Minimum Spacing is the same as specified for bolts.

E4.2 Minimum Edge and End Distances

In 2001, the minimum edge distance was decreased from 3d to 1.5d with a provision added for nominal shear strength based on end distance.

E4.3 Shear

E4.3.1 Connection Shear Limited by Tilting and Bearing

Screw connections loaded in shear can fail in one mode or in combination of several modes. These modes are screw shear, edge tearing, tilting and subsequent pull-out of the screw, and bearing of the joined materials.

Tilting of the screw followed by threads tearing out of the lower sheet reduces the connection shear capacity from that of the typical connection bearing strength (Figure C-E4.3-1).

These provisions are focused on the tilting and bearing failure modes. Two cases are given depending on the ratio of thicknesses of the connected members. Normally, the head

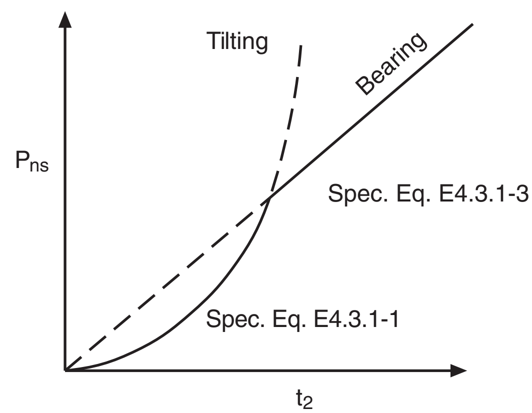
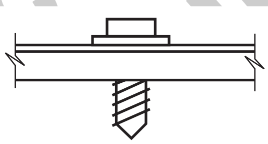
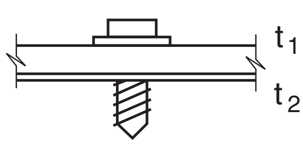


Figure C-E4.3-1 Comparison of Tilting and Bearing



tilting	N/A
bearing	$P_{ns} = 2.7\, t_1 d F_{u1}$ or
bearing	$P_{ns} = 2.7\, t_2 d F_{u2}$

Figure C-E4.3-2 Design Equations for $t_2/t_1 \geq 2.5$



tilting	$P_{ns} = 4.2\, (t_2^3 d)^{1/2} F_{u2}$ or
bearing	$P_{ns} = 2.7\, t_1 d F_{u1}$ or
bearing	$P_{ns} = 2.7\, t_2 d F_{u2}$

Figure C-E4.3-3 Design Equations for $t_2/t_1 \leq 1.0$

of the screw will be in contact with the thinner material as shown in Figure C-E4.3-2. However, when both members are the same thickness, or when the thicker member is in contact with the screw head, tilting must also be considered as shown in Figure C-E4.3-3.

It is necessary to determine the lower bearing capacity of the two members based on the product of their respective thicknesses and tensile strengths.

E4.3.2 Connection Shear Limited by End Distance

The provisions of this section are given in Section E4.3.2 of the Appendices. The discussion of this section is provided in the *Commentary* on the corresponding Appendix.

A,B

E4.3.3 Shear in Screws

Shear strength [resistance] of the screw fastener itself should be known and documented from testing. Screw strength should be established and published by the manufacturer. In order to prevent the brittle and sudden shear fracture of the screw, the *Specification* applies a 25 percent adjustment to the safety factor or the resistance factor where determined in accordance with *Specification* Section F1.

E4.4 Tension

Screw connections loaded in tension can fail either by pulling out of the screw from the plate (pull-out) or pulling of material over the screw head and the washer, if a washer is present, (pull-over) or by tensile fracture of the screw. The serviceability concerns of gross distortion are not covered by the equations given in *Specification* Section E4.4.

Diameter and rigidity of the fastener head assembly as well as sheet thickness and tensile strength have a significant effect on the pull-over failure load of a connection.

There are a variety of washers and head styles in use. Washers must be at least 0.050 inch (1.27 mm) thick to withstand bending forces with little or no deformation.

E4.4.1 Pull-Out

For the limit state of pull-out, *Specification* Equation E4.4.1-1 was derived on the basis of the modified European Recommendations and the results of a large number of tests. The statistic data on pull-out design considerations were presented by Pekoz (1990).

E4.4.2 Pull-Over

For the limit state of pull-over, *Specification* Equation E4.4.2-1 was derived on the basis of the modified British Standard and the results of a series of tests as reported by Pekoz (1990). In 2007, a rational allowance was included to cover the contribution of steel washers beneath screw heads. For the special case of screws with domed washers, that is washers that are not solid or do not seat flatly against the sheet metal in contact with the washer, the calculated nominal pull-over strength [resistance] should not exceed $1.5t_1d'_wF_{u1}$ with d'_w = 5/8 in. (16 mm). The 5/8 in. (16 mm) limit does not apply to solid steel washers in full contact with the sheet metal. In accordance with *Specification* Section E4, testing is allowed as an alternative method to determine fastener capacity. To use test data in design, the tested material should be consistent with the design. When a polygon shaped washer is used and capacity is determined using *Specification* Equation E4.4.2-1, the washer should have rounded corners to prevent premature tearing.

E4.4.3 Tension in Screws

Tensile strength [resistance] of the screw fastener itself should be known and documented from testing. Screw strength [resistance] should be established and published by the manufacturer. In order to prevent the brittle and sudden tensile fracture of the screw, the *Specification* applies a 25 percent adjustment to the safety factor or the resistance factor where determined in accordance with Section F1.

E4.5 Combined Shear and Pull-Over

Research pertaining to the behavior of a screw connection has been conducted at West Virginia University (Luttrell, 1999). Based on a review and analysis of West Virginia University's data for the behavior of a screw connection subject to combined shear and tension (Zwick and LaBoube, 2002), equations were derived that enable the evaluation of the strength of a screw connection when subjected to combined shear and tension. The tests indicated that at failure the sheet beneath the screw head pulled over the head of the screw or the washer. Therefore, the nominal tensile strength is based solely on P_{nov}. Although both non-linear and linear equations were developed, for ease of computation and because the linear equation provides regions of Q/P_{ns} and T/P_{nov} equal to unity, the linear equation was adopted for the *Specification*. The proposed equation is based on the following test program limits:

0.0285 in. (0.724 mm) $\le t_1 \le$ 0.0445 in. (1.13 mm)

No. 12 and No. 14 self-drilling screws with or without washers

$d_w \le$ 0.75 in. (19.1 mm)

62 ksi (427 MPa or 4360 kg/cm^2)$\le F_{u1} \le$ 70.7 ksi (487 MPa or 4970 kg/cm^2)

$t_2 / t_1 \ge 2.5$

The limit $t_2 / t_1 \ge 2.5$ reflects the fact that the test program (Luttrell, 1999) focused on connections having sheet thicknesses that precluded the tilting limit state from occurring. Thus, this limit ensures that the design equations will only be used when tilting limit state is not the control limit state.

The linear form of the equation as adopted by the *Specification* is similar to the following more conservative linear design equation that has been used by engineers:

$$Q/P_{ns} + T/P_{nov} \le 1.0$$

An eccentric load on a clip connection may create a non-uniform stress distribution around the fastener. For example, tension tests on roof panel welded connections have shown that under an eccentrically applied tension force the resulting connection capacity is 50% of the tension capacity under a uniformly applied tension force. Thus, the *Specification* stipulates that the pull-over strength shall be taken as 50% of P_{nov}. If the eccentric load is applied by a rigid member such as a clip, the resulting tension force on the screw may be uniform, thus the force in the screw can be determined by mechanics and the capacity of the fastener should be reliably estimated by P_{nov}. Based on the field performance of screw attached panels, the 30 percent reduction associated with welds at sidelaps need not be applied when evaluating the strength of sidelap screw connections at supports or sheet to sheet. The reduction is due to transverse prying or peeling. It is acceptable to apply the 50 percent reduction at panel ends due to longitudinal prying.

E5 Rupture

The design provisions of this section are given in Section E5 of the Appendices. The discussion of this section is provided in the *Commentary* on the corresponding Appendix. ☜A,B

E6 Connections to other Materials

E6.1 Bearing

The design provisions for the nominal bearing strength [resistance] on the other materials should be derived from appropriate material specifications.

E6.2 Tension

This Section is included in the *Specification* to raise the awareness of the design engineer regarding tension on fasteners and the connected parts.

E6.3 Shear

This Section is included in the *Specification* to raise the awareness of the design engineer regarding the transfer of shear forces from steel components to adjacent components of other materials.

F. TESTS FOR SPECIAL CASES

All tests for (1) the determination and confirmation of structural performance and (2) the determination of mechanical properties must be made by an independent testing laboratory or by a manufacturer's testing laboratory. Information on tests for cold-formed steel diaphragms can be found in *Design of Light Gage Steel Diaphragms* (AISI, 1967). A general discussion of structural diaphragms is given in *Cold-Formed Steel Design* (Yu, 2000).

F1 Tests for Determining Structural Performance

This *Specification* section contains provisions for proof of structural adequacy by load tests. This section is restricted to those cases permitted under Section A1.2 of the *Specification* or specifically permitted elsewhere in the *Specification*.

F1.1 Load and Resistance Factor Design and Limit States Design

The determination of load-carrying capacity of the tested elements, assemblies, connections, or members is based on the same procedures used to calibrate the LRFD design criteria, for which the ϕ factor can be computed from Equation C-A5.1.1-15. The correction factor C_P is used in *Specification* Equation F1.1-2 for determining the ϕ factor to account for the influence due to a small number of tests (Pekoz and Hall, 1988b and Tsai, 1992). It should be noted that when the number of tests is large enough, the effect of the correction factor is negligible. In the 1996 edition of the AISI *Specification*, Equation F1.1-3 was revised because the old formula for C_P could be unconservative for combinations of a high V_P and a small sample size (Tsai, 1992). This revision enables the reduction of the minimum number of tests from four to three identical specimens. Consequently, the ± 10 percent deviation limit was relaxed to ± 15 percent. The use of C_P with a minimum V_P reduces the need for this restriction. In *Specification* Equation F1.1-3, a numerical value of $C_P = 5.7$ was found for $n = 3$ by comparison with a two-parameter method developed by Tsai (1992). It is based on the given value of V_Q and other statistics listed in *Specification* Table F1, assuming that V_P will be no larger than about 0.20. The requirements of *Specification* Section F1.1(a) for $n = 3$ help to ensure this.

The 6.5 percent minimum value of V_P, when used in *Specification* Equation F1.1-2 for the case of three tests, produces safety factors similar to those of the 1986 edition of the AISI ASD *Specification*, i.e. approximately 2.0 for members and 2.5 for connections. The LRFD calibration reported by Hsiao, Yu and Galambos (1988a) indicates that V_P is almost always greater than 0.065 for common cold-formed steel components, and can sometimes reach values of 0.20 or more. The minimum value for V_P helps to prevent potential unconservatism compared to values of V_P implied in LRFD design criteria.

In evaluating the coefficient of variation V_P from test data, care must be taken to use the coefficient of variation for a sample. This can be calculated as follows:

$$V_P = \frac{\sqrt{s^2}}{R_m}$$

where

s^2 = sample variance of all test results

$$= \frac{1}{n-1}\sum_{i=1}^{n}\left(R_i - R_m\right)^2$$

R_m = mean of all test results

R_i = test result i of n total results

Alternatively, V_P can be calculated as the sample standard deviation of n ratios R_i/R_m.

For beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced (subject to wind uplift), the calibration is based on a load combination of 1.17W-0.9D with D/W = 0.1 (see Section D6.1.1 of this *Commentary* for detailed discussion).

The statistical data needed for the determination of the resistance factor are listed in *Specification* Table F1. The data listed for screw connections were added in 1996 on the basis of the study of bolted connections reported by Rang, Galambos, and Yu (1979b). The same statistical data of M_m, V_M, F_m, and V_F have been used by Pekoz in the development of the design criteria for screw connections (Pekoz, 1990).

In 1999, two entries were added to Table F1, one for "Structural Members Not Listed Above" and the other for "Connections Not Listed Above". It was considered necessary to include these values for members and connections not covered by one of the existing classifications. The statistical values were taken as the most conservative values in the existing table.

In 2004, the statistic data V_M for screw bearing strength was revised from 0.10 to 0.08. This revision is based on the tensile strength statistic data provided in the UMR research report (Rang, Galambos, and Yu, 1979b). In addition, V_f was revised from 0.10 to 0.05 to reflect the tolerance of the cross-sectional area of the screw.

In 2007, additional entries were made to Table F1 to provide statistical data for all limit states included within the *Specification* for the standard connection types. The entry "Connections Not Listed Above" is intended to provide statistical data for connections other than welded, bolted, or screwed.

Also in 2007 the specification more clearly defined the appropriate material properties that are to be used when evaluating test results by specifying that supplier provided properties are not to be used.

F1.2 Allowable Strength Design

The equation for the safety factor Ω (*Specification* Equation F1.2-2) converts the resistance factor ϕ from LRFD test procedures in *Specification* Section F1.1 to an equivalent safety factor for the allowable strength design. The average of the test results, R_n, is then divided by the safety factor to determine an allowable strength [resistance]. It should be noted that *Specification* Equation F1.2-2 is identical with Equation C-A5.1.1-16 for D/L=0.

F2 Tests for Confirming Structural Performance

Members, connections and assemblies that can be designed according to the provisions of Chapters A through E of the *Specification* need no confirmation of calculated results by test. However, special situations may arise where it is desirable to confirm by test the results of calculations. Tests may be called for by the manufacturer, the engineer, or a third party.

Since design is in accordance with the *Specification*, all that is needed is that the tested specimen or assembly demonstrates a strength [resistance] not less than the applicable nominal resistance, R_n.

F3 Tests for Determining Mechanical Properties

F3.1 Full Section

Explicit methods for utilizing the effects of cold work are incorporated in Section A7.2 of the *Specification*. In that section, it is specified that as-formed mechanical properties, in particular the yield stress, can be determined either by full-section tests or by calculating the strength of the corners and computing the weighted average for the strength of corners and flats. The strength of flats can be taken as the virgin strength of the steel before forming, or can be determined by special tension tests on specimens cut from flat portions of the formed section. This *Specification* section spells out in considerable detail the types and methods of these tests, and their number as required for use in connection with *Specification* Section A7.2. For details of testing procedures which have been used for such purposes, but which in no way should be regarded as mandatory, see AISI *Specification* (1968), Chajes, Britvec and Winter (1963), and Karren (1967). A *Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns* is included in Part VI of the AISI *Design Manual* (AISI, 2008).

F3.2 Flat Elements of Formed Sections

Specification Section F3.2 provides the basic requirements for determining the mechanical properties of flat elements of formed sections. These tested properties are to be used in *Specification* Section A7.2 for calculating the average yield stress of the formed section by considering the strength increase from cold work of forming.

F3.3 Virgin Steel

For steels other than the ASTM Specifications listed in *Specification* Section A2.1, the tensile properties of the virgin steel used for calculating the increased yield stress of the formed section should also be determined in accordance with the Standard Methods of ASTM A370 (1997).

G. DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS AND CONNECTIONS FOR CYCLIC LOADING (FATIGUE)

Fatigue in a cold-formed steel member or connection is the process of initiation and subsequent growth of a crack under the action of a cyclic or repetitive load. The fatigue process commonly occurs at a stress level less than the static failure condition.

When fatigue is a design consideration, its severity is determined primarily by three factors: (1) the number of cycles of loading, (2) the type of member and connection detail, and (3) the stress range at the detail under consideration (Fisher et al. 1998).

Fluctuation in stress, which does not involve tensile stress, does not cause crack propagation and is not considered to be a fatigue situation.

When fabrication details involving more than one category occur at the same location in a member, the design stress range at the location must be limited to that of the most restrictive category. By locating notch-producing fabrication details in regions subject to a small range of stress, the need for a member larger than required by static loading will often be eliminated.

For axially stressed angle members the *Specification* allows the effects of eccentricity on the weld group to be ignored provided the weld lengths L_1 and L_2 are proportional such that the centroid of the weld group falls between "$\bar{x}$" and "b/2" in Figure C-G1(a). When the weld lengths L_1 and L_2 are so proportioned, the effects of eccentric loads causing moment about x-x in Figure C-G1(b) also need not be considered.

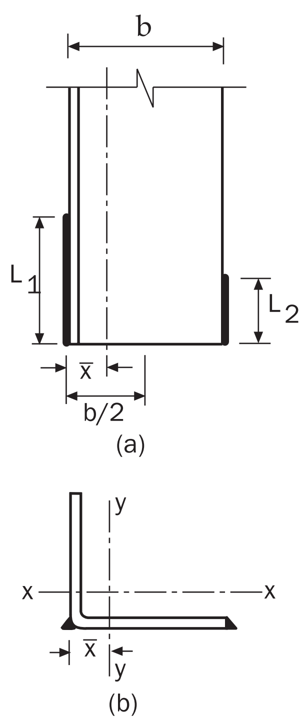


Figure C-G1, Welded Angle Members

Research by Barsom et al. (1980) and Klippstein (1988, 1985, 1981, 1980) developed fatigue information on the behavior of sheet and plate steel weldments and mechanical connections. Although research indicates that the values of F_y and F_u do not influence fatigue behavior, the *Specification* provisions are based on tests using ASTM A715 (Grade 80), ASTM A607 Grade 60, and SAE 1008 (F_y = 30 ksi). Using regression analysis, mean fatigue life curves (S-N curves) with the corresponding standard deviation were developed. The fatigue resistance S-N curve has been expressed as an exponential relationship between stress range and life cycle (Fisher et al, 1970). The general relationship is often plotted as a linear log-log function, Eq. C-G1.

$$\log N = C_f - m \log F_{SR} \quad \text{(C-G1)}$$

$$C_f = b - (n\,s) \quad \text{(C-G2)}$$

where

N = number of full stress cycles

m = slope of the mean fatigue analysis curve

F_{SR} = effective stress range

B = intercept of the mean fatigue analysis curve from Table C-G1

n = number of standard deviations to obtain a desired confidence level
= 2 for C_f given in the Table G1 of the *Specification*

s = approximate standard deviation of the fatigue data
= 0.25 (Klippstein, 1988)

The database for these design provisions are based upon cyclic testing of real joints; therefore, stress concentrations have been accounted for by the categories in Table G1 of the *Specification*. It is not intended that the allowable stress ranges should be compared to "hot-spot" stresses determined by finite element analysis. Also, calculated stresses computed by ordinary analysis need not be amplified by stress concentration factors at geometrical discontinuities and changes of cross section. All categories were found to have a common slope with m = -3. Equation G3-1 of the *Specification* is to be used to calculate the design stress range for the chosen design life, N. Table G1 of the *Specification* provides a classification system for the various stress categories. This also provides the constant C_f that is applicable to the stress category that is required for calculating the design stress range F_{SR}.

Table C-G1 Intercept for Mean Fatigue Curves

Stress Category	b
I	11.0
II	10.5
III	10.0
IV	9.5

The provisions for bolts and threaded parts were taken from the AISC Specification (AISC, 1999).

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Appendix 1:

Commentary on Appendix 1

Design of Cold-Formed Steel

Structural Members

Using the Direct Strength Method

2007 EDITION

APPENDIX 1: COMMENTARY ON APPENDIX 1 DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS USING THE DIRECT STRENGTH METHOD

1.1 GENERAL PROVISIONS

1.1.1 Applicability

The Direct Strength Method of Appendix 1 is an alternative procedure for determining the strength [resistance] and stiffness of cold-formed steel members (beams and columns). The reliability of Appendix 1 is insured by using calibrated safety factor, Ω, and resistance factor, ϕ, within set geometric limits, and conservative Ω and ϕ for other configurations. The applicability of Appendix 1 to all beams and columns implies that in some situations competing methods may exist for strength determination of a member: the main *Specification** and Appendix 1. In this situation there is no preferred method. Either method may be used to determine the strength [resistance]. The fact that one method may give a greater or lower strength [resistance] prediction in a given situation does not imply an increased accuracy for either method. The Ω and ϕ factors are designed to insure that both methods reach their target reliability.

The method of Appendix 1 provides solutions for beams and columns only, but these solutions must be combined with the regular provisions of the main *Specification* to cover other cases: shear, beam-columns, etc. For example, an application to purlin design was completed using the provisions of this Appendix for the bending strength, and then those calculations were augmented by shear, and shear and bending interaction calculations, in line with the main *Specification* (Quispe and Hancock, 2002). Further, beam-columns may be conservatively examined using the provisions of the main *Specification*, by replacing the beam and column design strength [factored resistance] with the provisions of this Appendix, or beam-columns may be analyzed using the actual stress state (Schafer, 2002b).

The pre-qualified columns and beams only include members without perforations (punchouts). Members with perforations generally may be designed by the main *Specification*. For perforated members not covered by the *Specification* one may want to consider a rational analysis method, which partially employs the methods of this Appendix. The key issue in such a rational analysis is the accurate determination of the elastic local, distortional, and global buckling loads (or moments) for the member with perforations. Numerical (e.g., finite element) analysis where the holes are explicitly considered is one option in this case.

Note:

* The *North American Specification for the Design of Cold-Formed Steel Structural Members*, Chapters A through G and Appendices A and B and Appendix 2, is herein referred to as the main *Specification*.

1.1.1.1 Pre-qualified Columns

An extensive amount of testing has been performed on concentrically loaded, pin-ended, cold-formed steel columns (Kwon and Hancock, 1992; Lau and Hancock, 1987; Loughlan, 1979; Miller and Peköz, 1994; Mulligan, 1983; Polyzois et al., 1993; Thomasson, 1978). Data from these researchers were compiled and used for calibration of the Direct Strength Method. The geometric limitations listed in Appendix 1 are based on these experiments. In 2006 the pre-qualified category of Lipped C-Section and Rack Upright were merged, as a rack upright is a C-section with a complex stiffener. In addition, the complex stiffener limits from the original Rack Upright category were relaxed to match those found for C-section beams with complex stiffeners (Schafer, et al., 2006).

It is intended that as more cross-sections are verified for use in the Direct Strength Method, these tables and sections will be augmented. Companies with proprietary sections may wish to perform their own testing and follow Chapter F of the main *Specification* to justify the use of lower Ω and higher ϕ factors for a particular cross-section. Alternatively, member geometries that are not pre-qualified may still use the method of Appendix 1, but with the increased Ω and reduced ϕ factors consistent with any rational analysis method as prescribed in A1.2 of the main *Specification*.

1.1.1.2 Pre-qualified Beams

An extensive amount of testing has been performed on laterally braced beams (Cohen, 1987; Ellifritt et al., 1997; LaBoube and Yu, 1978; Moreyara, 1993; Phung and Yu, 1978; Rogers, 1995; Schardt and Schrade, 1982; Schuster, 1992; Shan et al., 1994; Willis and Wallace, 1990) and on hats and decks (Acharya and Schuster, 1998; Bernard, 1993; Desmond, 1977; Höglund, 1980; König, 1978; Papazian et al., 1994). Data from these researchers were compiled and used for calibration of the Direct Strength Method. The geometric limitations listed in the Appendix are based on the experiments performed by these researchers. The original geometric limits were extended to cover C- and Z-section beams with complex lip stiffeners based on the work of Schafer et al. (2006). For rounded edge stiffeners or other edge stiffeners that do not meet the geometric criteria either for pre-qualified simple, or complex, stiffeners one may still use the method of Appendix 1, but instead with the rational analysis Ω and ϕ factors prescribed in A1.2 of the main *Specification*. See the note on pre-qualified columns for further commentary on members which do not meet the pre-qualified geometric limits.

Users of this Appendix should be aware that pre-qualified beams with large flat width-to-thickness ratios in the compression flange will be conservatively predicted by the method of this Appendix when compared to the main *Specification* (Schafer and Peköz, 1998). However, the same beam with small longitudinal stiffeners in the compression flange will be well predicted using this Appendix.

1.1.2 Elastic Buckling

The elastic buckling load is the load in which the equilibrium of the member is neutral between two alternative states: buckled and straight. Thin-walled cold-formed steel members have at least 3 relevant elastic buckling modes: local, distortional, and global (Figure C-1.1.2-1). The global buckling mode includes flexural, torsional, or flexural-torsional buckling for columns, and lateral-torsional buckling for beams. Traditionally, the main *Specification* has only addressed local and global buckling. Further, the main *Specification*'s approach to local buckling is to conceptualize the member as a collection of "elements" and investigate local buckling of each element separately.

The method of this Appendix provides a means to incorporate all three relevant buckling modes into the design process. Further, all buckling modes are determined for the member as a whole rather than element by element. This insures that compatibility and equilibrium are maintained at element junctures. Consider, as an example, the lipped C-Section shown in pure compression in Figure C-1.1.2-1(a). The member's local elastic buckling load from the analysis is:

$P_{cr\ell}$ = 0.12 x 48.42 kips = 5.81 kips (25.84 kN).

The column has a gross area (A_g) of 0.881 in^2 (568.4 mm^2), therefore,

$f_{cr\ell} = P_{cr\ell}/A_g$ = 6.59 ksi (45.44 MPa)

The main *Specification* determines a plate buckling coefficient, k, for each element, then f_{cr}, and finally the effective width. The centerline dimensions (ignoring corner radii) are h = 8.94 in. (227.1 mm), b = 2.44 in. (62.00 mm), d = 0.744 in. (18.88 mm), and t = 0.059 in. (1.499 mm), the critical buckling stress, f_{cr} of each element as determined from the main *Specification*:

lip: k = 0.43, $f_{cr\ell\text{-lip}} = 0.43[\pi^2E/(12(1-\mu^2))](t/d)^2$ = 72.1 ksi (497 MPa)

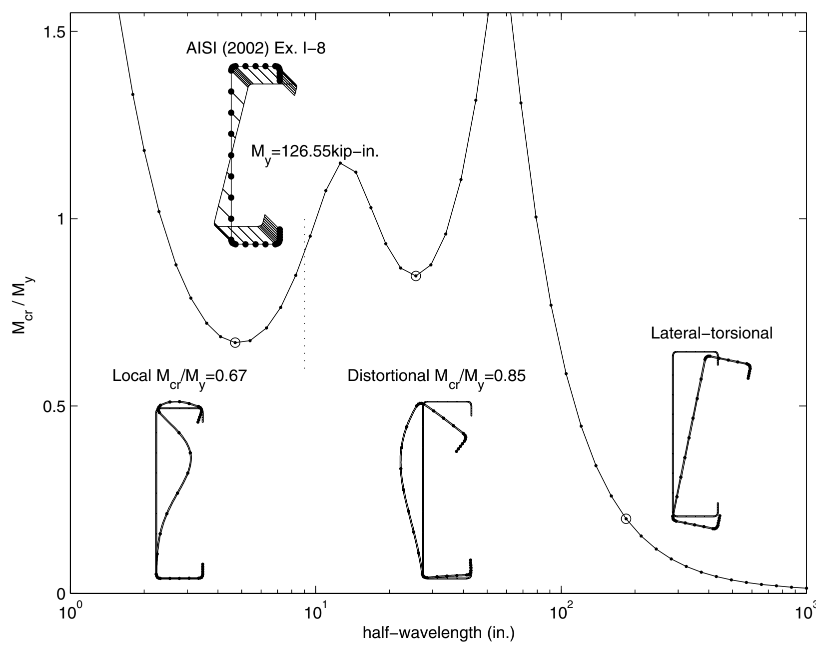
flange: k = 4, $f_{cr\ell\text{-flange}} = 4.0[\pi^2E/(12(1-\mu^2))](t/b)^2$ = 62.4 ksi (430 MPa)

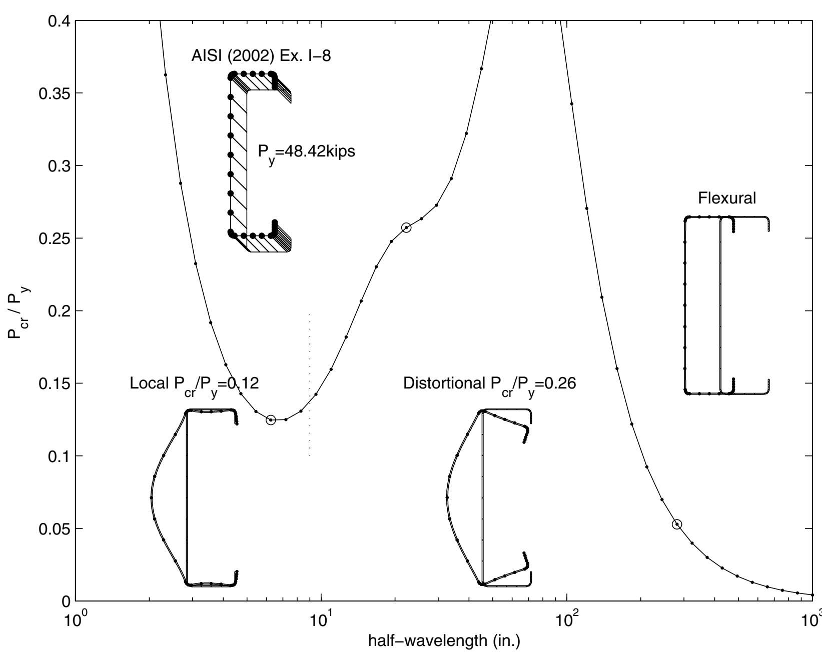
web: k = 4, $f_{cr\ell\text{-web}} = 4.0[\pi^2E/(12(1-\mu^2))](t/h)^2$ = 4.6 ksi (32.0 MPa)

Each element predicts a different buckling stress, even though the member is a connected group. These differences in the buckling stress are ignored in the main *Specification*. The high flange and lip buckling stresses have little relevance given the low web buckling stress. The finite strip analysis, which includes the interaction amongst the elements, shows that the flange aids the web significantly in local buckling, increasing the web buckling stress from 4.6 ksi (32.0 MPa) to 6.59 ksi (45.4 MPa), but the buckling stress in the flange and lip are much reduced due to the same interaction. Comparisons to the distortional buckling stress (f_{crd}) using k from B4.2 of the main *Specification* do no better (Schafer and Peköz, 1999; Schafer, 2002).

The method of this Appendix allows rational analysis to be used for determining the local, distortional and global buckling load or moment. Specific guidance on elastic buckling determination follows. Users are reminded that the strength of a member is not equivalent to the elastic buckling load (or moment) of the member. In fact the elastic buckling load can be lower than the actual strength, for slender members with considerable post-buckling reserve; or the elastic buckling load can be fictitiously high due to ignoring inelastic effects. Nonetheless, the elastic buckling load is a useful reference load for determining a member's slenderness and ultimately its strength.

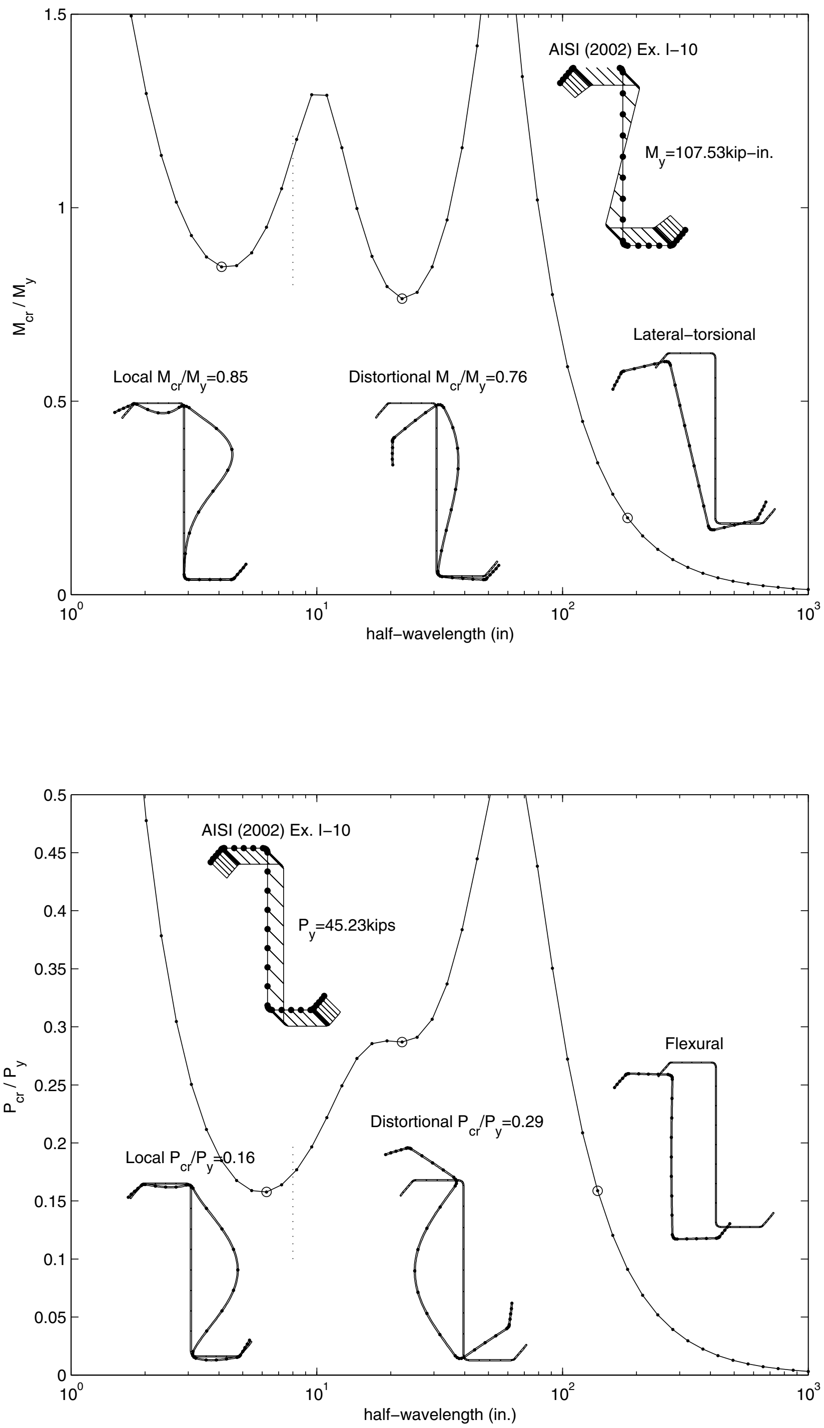
Manual and numerical solutions for elastic buckling prediction are covered in the following sections. It is permissible to mix the manual and numerical methods; in some cases it is even advantageous. For example, numerical solutions for member local and distortional buckling are particularly convenient; however, unusual long column bracing conditions $(KL)_x \neq (KL)_y \neq (KL)_t$ may often be handled with less confusion using the traditional manual formulas. Use of the numerical solutions is generally encouraged, but verification with the manual solutions can aid in building confidence in the numerical solution.





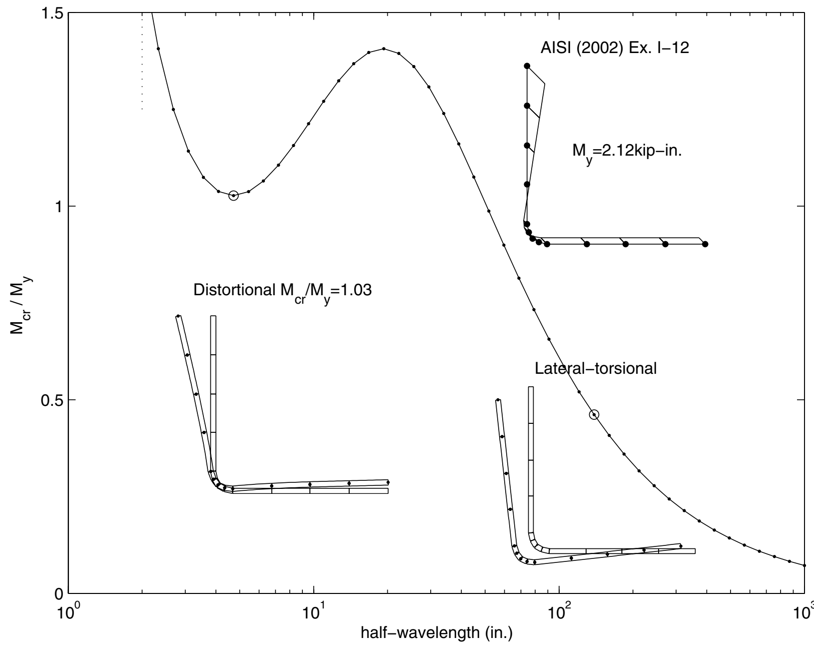
(a) 9CS2.5x059 of AISI 2002 Cold-Formed Steel Design Manual Example I-8

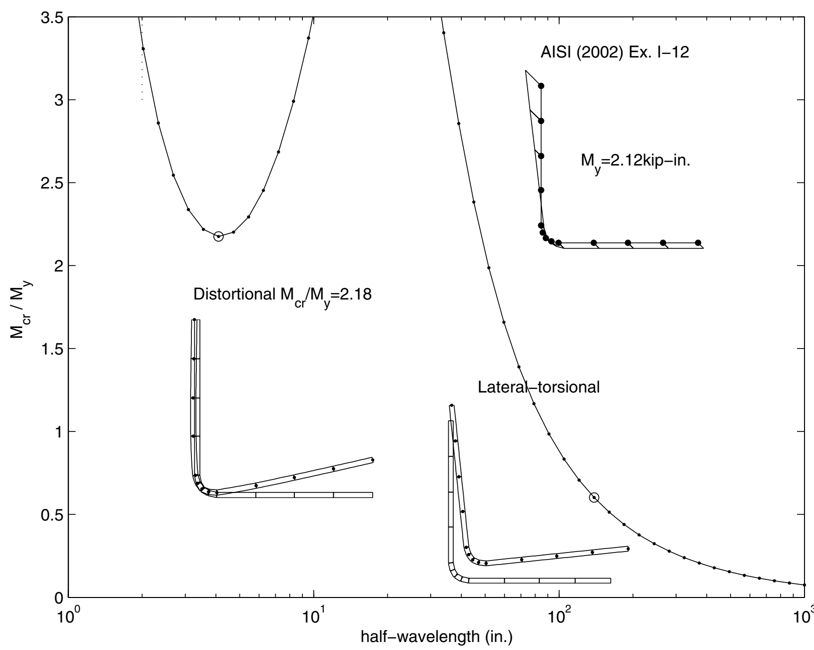
Figure C-1.1.2-1 Examples of Bending and Compression Elastic Buckling Analysis with Finite Strip Method



(b) 8ZS2.25x059 of AISI 2002 Cold-Formed Steel Design Manual Example I-10

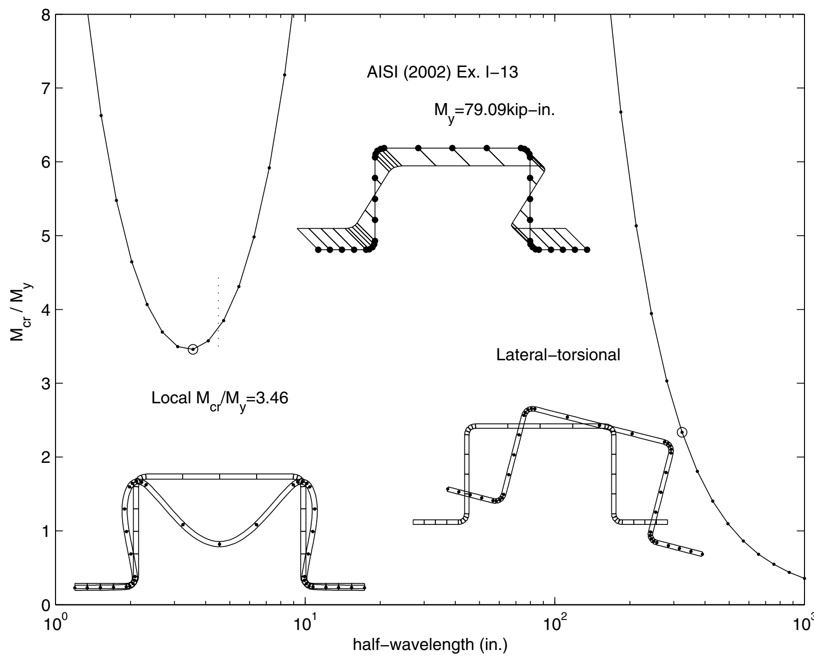
Figure C-1.1.2-1 Examples of Bending and Compression Elastic Buckling Analysis with Finite Strip Method (cont.)

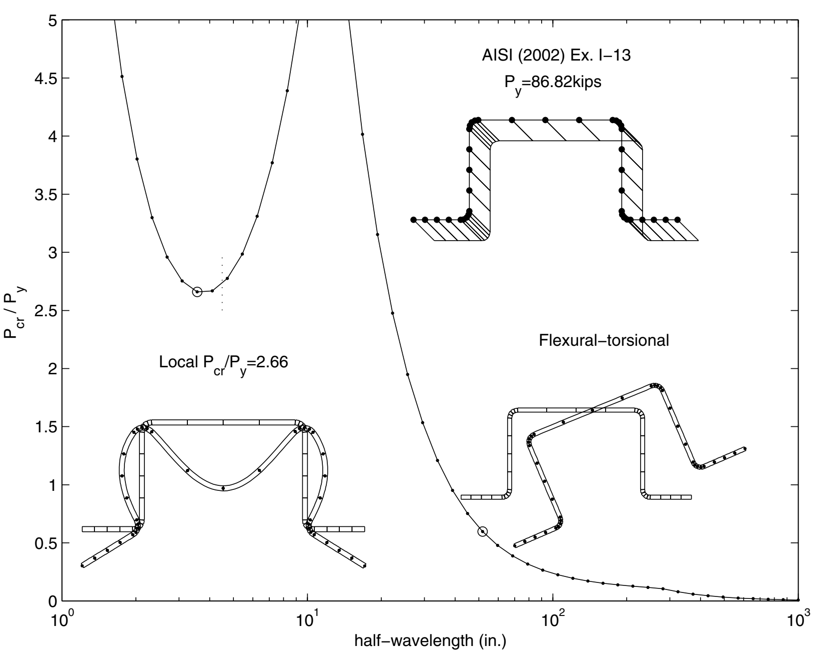




(c) 2LU2x060 of AISI 2002 Cold-Formed Steel Design Manual Example I-12

Figure C-1.1.2-1 Examples of Bending and Compression Elastic Buckling Analysis with Finite Strip Method (cont.)





(d) 3HU4.5x135 of AISI 2002 Cold-Formed Steel Design Manual Example I-13

Figure C-1.1.2-1 Examples of Bending and Compression Elastic Buckling Analysis with Finite Strip Method (cont.)

1.1.2.1 Elastic Buckling - Numerical Solutions

A variety of numerical methods: finite element, finite differences, boundary element, generalized beam theory, finite strip analysis, and others, may provide accurate elastic buckling solutions for cold-formed steel beams and columns.

Traditional finite element analysis using thin plate or shell elements may be used for elastic buckling prediction. Due to the common practice of using polynomial shape functions, the number of elements required for reasonable accuracy can be significant. Finite element analysis books such as Cook et al. (1989) and Zienkiewicz and Taylor (1989, 1991) explain the basic theory; while a number of commercial implementations can provide accurate elastic buckling answers if implemented with care. Finite difference solutions for plate stability are implemented by Harik et al. (1991) and others. The boundary element method may also be used for elastic stability (Elzein, 1991).

Generalized beam theory, developed by Schardt (1989), extended by Davies et al. (1994) and implemented by Davies and Jiang (1996, 1998), and Silvestre and Camotim (2002a, 2002b) has been shown to be a useful tool for elastic stability analysis of cold-formed steel members. The ability to separate the different buckling modes makes the method especially amenable to design methods.

Finite strip analysis is a specialized variant of the finite element method. For elastic stability of cold-formed steel structures, it is one of the most efficient and popular methods. Cheung and Tham (1998) explains the basic theory while Hancock et al. (2001) and Schafer (1997) provide specific details for stability analysis with this method. Hancock and his researchers (see Hancock et al., 2001 for full references and descriptions) pioneered the use of finite strip analysis for stability of cold-formed steel members and convincingly demonstrated the important potential of finite strip analysis in both cold-formed steel design and behavior.

The Direct Strength Method of this Appendix emphasizes the use of finite strip analysis for elastic buckling determination. Finite strip analysis is a general tool that provides accurate elastic buckling solutions with a minimum of effort and time. Finite strip analysis, as implemented in conventional programs, does have limitations, the two most important ones are

- the model assumes the ends of the member are simply supported, and
- the cross-section may not vary along its length.

These limitations preclude some analysis from readily being used with the finite strip method, but despite these limitations the tool is useful, and a major advance over plate buckling solutions and plate buckling coefficients (k's) that only partially account for the important stability behavior of cold-formed steel members.

The American Iron and Steel Institute has sponsored research that, in part, has led to the development of the freely available program, CUFSM, which employs the finite strip method for elastic buckling determination of any cold-formed steel cross-section. The program is available at www.ce.jhu.edu/bschafer/cufsm and runs on any PC with Windows 9x, NT, 2000, XP. Tutorials and examples are available online at the same address.

1.1.2.1.1 Local Buckling via Finite Strip ($P_{cr\ell}$, $M_{cr\ell}$)

In the finite strip method, members are loaded with a reference stress distribution: pure compression for finding P_{cr}, and pure bending for finding M_{cr} (see Figure C-1.1.2-1).

Determination of the buckling mode requires consideration of the half-wavelength and mode shape of the member. Special attention is given to the half-wavelength and mode shape for local, distortional, and global buckling via finite strip analysis in the following sections.

Half-wavelength

Local buckling minima occur at half-wavelengths that are less than the largest characteristic dimension of the member under compressive stresses. For the examples of Figure C-1.1.2-1, this length has been demarcated with a short vertical dashed line. For instance, the largest out-to-out dimension for the lipped channel of Figure C-1.1.2-1 (a) is 9 in. (229 mm), therefore the cutoff for local buckling is at 9 in. (229 mm). Minima in the buckling curves that fall at half-wavelengths less than this length are considered as local buckling modes. Buckling modes occurring at longer lengths are either distortional or global in nature.

The criteria of limiting the half-wavelength for local buckling to less than the largest outside dimension under compressive stresses is based on the following. Local buckling of a simply supported plate in pure compression occurs in square waves, i.e., it has a half-wavelength that is equal to the plate width (the largest outside dimension). If any stress gradient exists on the plate, or any beneficial restraint is provided to the edges of the plate by other elements, the critical half-wavelength will be less than the width of the plate. Therefore, local buckling, with the potential for stable post-buckling response, is assumed to occur only when the critical half-wavelength is less than the largest potential “plate” (i.e., outside dimension with compressive stresses applied) in a member.

Mode shape

Local buckling involves significant distortion of the cross-section, but this distortion involves only rotation, not translation, at the fold lines of the member. The mode shapes for members with edge stiffened flanges such as those of the lipped cee or zee provide a direct comparison between the difference between local buckling and distortional buckling. Note the behavior at the flange/lip junction – for local buckling only rotation occurs, for distortional buckling translation occurs.

Discussion

Local buckling may be indistinct from distortional buckling in some members. For example, buckling of the unlipped angle may be considered as local buckling by the main *Specification*, but is considered as distortional buckling as shown in Figure C-1.1.2-1(c), because of the half-wavelength of the mode, and the characteristics of the mode shape. By the definitions of this Appendix, no local buckling mode exists for this member. Local buckling may be at half-wavelengths much less than the characteristic dimension if intermediate stiffeners are in place, or if the element undergoes large tension and small compressive stress.

Users may encounter situations where they would like to consider the potential for bracing to retard local buckling. Springs may be added to a numerical model to include the effect of external bracing. Care should be used if the bracing only provides support in one direction (such as a deck on a compression flange) as the increase of the local buckling strength is limited in such a case. In general, since local buckling occurs at short wavelengths, it is difficult to effectively retard this mode by external bracing. Changes to the geometry of the member (stiffeners, change of thickness, etc.) should be pursued instead.

1.1.2.1.2 Distortional Buckling via Finite Strip (P_{crd}, M_{crd})

Half-wavelength

Distortional buckling occurs at a half-wavelength intermediate to local and global buckling modes, as shown in the figures given in C-1.1.2-1. The half-wavelength is typically several times larger than the largest characteristic dimension of the member. The half-wavelength is highly dependent on both the loading and the geometry.

Mode shape

Distortional buckling involves both translation and rotation at the fold line of a member. Distortional buckling involves distortion of one portion of the cross-section and predominately rigid response of a second portion. For instance, the edge stiffened flanges of the lipped cee and zee are primarily responding as one rigid piece while the web is distorting.

Discussion

Distortional buckling may be indistinct (without a minimum) even when local buckling and long half-wavelength (global) buckling are clear. The lipped cee and zee in bending show this basic behavior. For some members distortional buckling may not occur.

Bracing can be effective in retarding distortional buckling and boosting the strength [resistance] of a member. Continuous bracing may be modeled by adding a continuous spring in a finite strip model. For discrete bracing of distortional buckling, when the unbraced length is less than the critical distortional half-wavelength, best current practice is to use the buckling load (or moment) at the unbraced length. The key consideration for distortional bracing is limiting the rotation at the compression flange/web juncture.

1.1.2.1.3 Global (Euler) Buckling via Finite Strip (P_{cre}, M_{cre})

Global bucking modes for columns include: flexural, torsional and flexural-torsional buckling. For beams bent about their strong-axis, lateral-torsional buckling is the global buckling mode of interest.

Half-wavelength

Global (or "Euler") buckling modes: flexural, torsional, or flexural-torsional for columns, lateral-torsional for beams, occur as the minimum mode at long half-wavelengths.

Mode Shape

Global buckling modes involve translation (flexure) and/or rotation (torsion) of the entire cross-section. No distortion exists in any of the elements in the long half-wavelength buckling modes.

Discussion

Flexural and distortional buckling may interact at relatively long half-wavelengths making it difficult to determine long column modes at certain intermediate to long lengths. When long column end conditions are not simply supported, or when they are dissimilar for flexure and torsion, higher modes are needed for determining the appropriate buckling load. By examining higher modes in a finite strip analysis, distinct flexural and flexural-torsional modes may be identified. Based on the boundary conditions, the effective length, KL, for a given mode can be determined. With KL known, then P_{cre} (or M_{cre}) for that mode may be read directly from the finite strip at a half-wavelength of KL by using the curve corresponding to the appropriate mode. For beams, C_b of the main *Specification* may be employed to account for the moment gradient. Mixed flexural and torsional boundary conditions may not be directly treated. Alternatively, traditional manual solutions may be used for global buckling modes with different bracing conditions.

Elastic Buckling – Manual Solutions

Local buckling

Manual solutions for member local buckling rely on the use of element plate buckling coefficients, as given below.

For columns,

$$P_{cr\ell} = A_g f_{cr\ell} \tag{C-1.1.2-1}$$

A_g = gross area

$f_{cr\ell}$ = local buckling stress

For beams,

$$M_{cr\ell} = S_g f_{cr\ell} \tag{C-1.1.2-2}$$

S_g = gross section modulus to the extreme compression fiber

$f_{cr\ell}$ = local buckling stress at the extreme compression fiber

and

$$f_{cr\ell} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \tag{C-1.1.2-3}$$

where

E = Young's Modulus

μ = Poisson's ratio

t = element thickness

w = element flat width

k = element (plate) buckling coefficient. Local plate buckling coefficients for an isolated element may be predicted through use of commentary Table C-B2-1. Schafer and Peköz (1999) present additional expressions for stiffened and unstiffened elements under a stress gradient. Elastic local buckling of a member may be conservatively approximated by using the minimum of the local buckling stress of the elements, which make up the member. However, using the minimum element solution and ignoring interaction may be excessively conservative for predicting member local buckling. To alleviate this, hand methods that account for the interaction of two elements are available. Solutions include two stiffened or edge stiffened elements (a flange and a web) under a variety of loading cases Schafer (2001, 2002); and local buckling of an edge stiffened element, including lip/flange interaction (Schafer and Peköz, 1999).

Distortional Buckling

Distortional buckling of members with edge stiffened flanges may also be predicted by manual solutions. Unfortunately, the complicated interaction that occurs between the edge stiffened flange and the web leads to cumbersome and lengthy formulas.

For columns,

$$P_{crd} = A_g f_{crd} \qquad \text{(C-1.1.2-3)}$$

A_g = gross area of the member

f_{crd} = distortional buckling stress (see below)

For beams,

$$M_{crd} = S_f f_{crd} \qquad \text{(C-1.1.2-4)}$$

S_f = gross section modulus to the extreme compression fiber

f_{crd} = distortional buckling stress at the extreme compression fiber. Solutions and design aids for f_{crd} are available for beams (Hancock et al., 1996; Hancock, 1997; Schafer and Peköz, 1999) and for columns (Lau and Hancock, 1987; Schafer 2002). Design aids for flanges with unusual edge stiffeners (e.g., Bambach et al., 1998) or flexural members with a longitudinal stiffener in the web (Schafer, 1997) are also available. See the *Commentary* on the Main *Specification* Sections C3.1.4 and C4.2 for additional information.

Global Buckling

Global buckling of members is calculated in the main *Specification*. Therefore, for both beams and columns, extensive closed-form expressions are already available and may be used for manual calculation. See the *Commentary* to main *Specification* Sections C4 and C3 for additional details.

For columns,

$$P_{cre} = A_g f_{cre} \qquad \text{(C-1.1.2-5)}$$

A_g = gross area of the member

f_{cre} = minimum of the elastic critical flexural, torsional, or flexural-torsional buckling stress. f_{cre} is equal to F_e of Section C4 of the main *Specification*. The hand methods presented in *Specification* Sections C4.1.1 through C4.1.4 provide all necessary formula. Note, Section C4.1.4 specifically addresses the long-standing practice that F_e (or f_{cre}) may be calculated by rational analysis. Rational analysis hand solutions to long column buckling are available - see the *Commentary* for main *Specification* Section C4.1.4 as well as Yu (2000) or Hancock et al. (2001). The hand calculations may be quite lengthy, particular if member properties x_o and C_w are unknown.

For beams,

$$M_{cre} = S_f f_{cre} \qquad \text{(C-1.1.2-6)}$$

S_f = gross section modulus to the extreme compression fiber

f_{cre} = elastic critical lateral-torsional buckling stress. f_{cre} is equal to F_e of main *Specification* Section C3.1.2.1 for open cross-section members and C3.1.2.2 for closed cross-section members. Hand solutions are well established for doubly- and singly-symmetric sections, but not so for point symmetric sections (zees). F_e

of point-symmetric sections is taken as half of the value for doubly-symmetric sections. Rational numerical analysis may be desirable in cases where a close to exact solution is required.

1.1.3 Serviceability Determination

The provisions of this Appendix use a simplified approach to deflection calculations that assume the moment of inertia of the section for deflection calculations is linearly proportional to the strength of the section, determined at the allowable stress of interest. This approximation avoids lengthy effective section calculations for deflection determination.

1.2 MEMBERS

1.2.1 Column Design

Commentary Section C4 provides a complete discussion on the behavior of cold-formed steel columns as it relates to the main *Specification*. This commentary addresses the specific issues raised by the use of the Direct Strength Method of Appendix 1 for the design of cold-formed steel columns. The thin-walled nature of cold-formed columns complicates behavior and design. Elastic buckling analysis reveals at least three buckling modes: local, distortional, and Euler (flexural, torsional, or flexural-torsional) that must be considered in design. Therefore, in addition to usual considerations for steel columns: material non-linearity (e.g., yielding), imperfections, and residual stresses, the individual role and potential for interaction of buckling modes must also be considered. The Direct Strength Method of this Appendix emerged through the combination of more refined methods for local and distortional buckling prediction, improved understanding of the post-buckling strength and imperfection sensitivity in distortional failures, and the relatively large amount of available experimental data.

Fully effective or compact columns are generally well predicted by conventional column curves (AISC, 2001; Galambos, 1998, etc.). Therefore, the long column strength, P_{ne}, follows the same practice as the main *Specification* and uses the AISC (2001) curves for strength prediction. The main *Specification* provides the long column strength in terms of a stress, F_n (Equations C4.1-2 and C4.1-3). In the Direct Strength Method this is converted from a stress to a strength by multiplying the gross area, A_g, resulting in the formulas for P_{ne} given in Appendix 1.

In the main *Specification*, column strength is calculated by multiplying the nominal column buckling stress, F_n, by the effective area, A_e, calculated at F_n. This accounts for local buckling reductions in the actual column strength (i.e., local-global interaction). In the Direct Strength Method, this calculation is broken into two parts: the long column strength without any reduction for local buckling (P_{ne}) and the long column strength considering local-global interaction ($P_{n\ell}$).

The strength curves for local and distortional buckling of a fully braced column are presented in Figure C-1.2.1-1. The curves are presented as a function of slenderness, which in this case refers to slenderness in the local or distortional mode, as opposed to traditional long column slenderness. Inelastic and post-buckling regimes are observed for both local and distortional buckling modes. The magnitude of the post-buckling reserve for the distortional buckling mode is less than the local buckling mode, as may be observed by the location of the strength curves in relation to the critical elastic buckling curve.

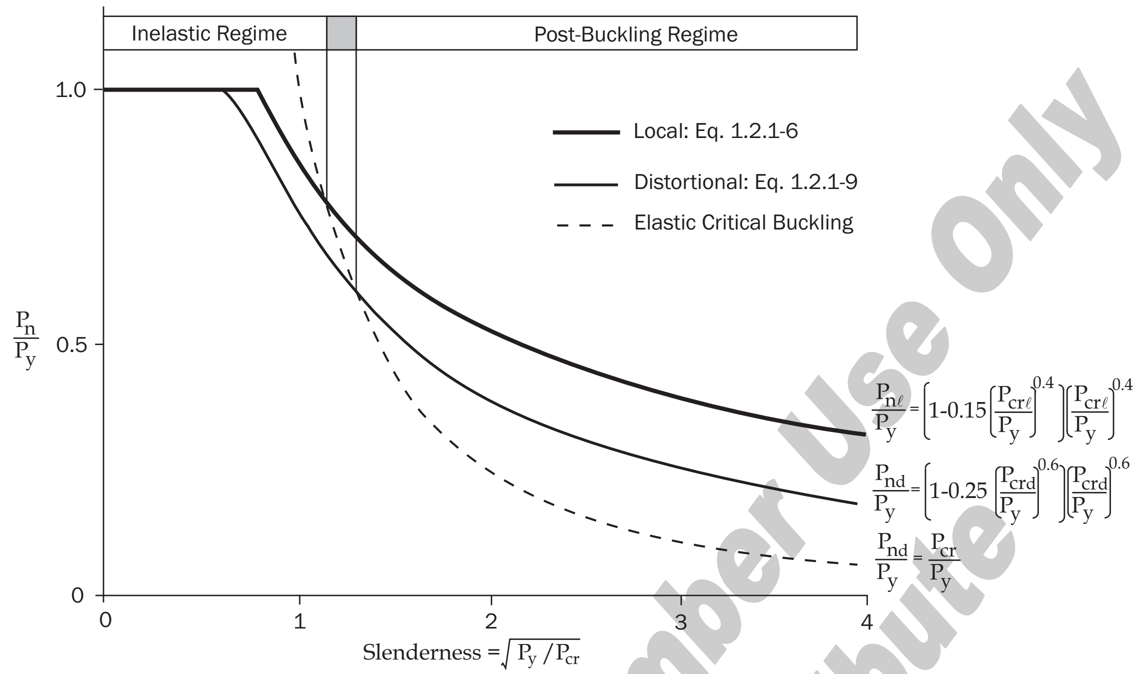


Figure C-1.2.1-1 Local and Distortional Direct Strength Curves for a Braced Column (Pne = Py)

The development and calibration of the Direct Strength provisions for columns are reported in Schafer (2000, 2002). The reliability of the column provisions was determined using the test data of Appendix Section 1.1.1.1 and the provisions of Chapter F of the main *Specification*. Based on a target reliability, β, of 2.5, a resistance factor, ϕ, of 0.84 was calculated for all the investigated columns. Based on this information the safety and resistance factors of Appendix Section 1.2.1 were determined for the pre-qualified members. For the United States and Mexico ϕ = 0.85 was selected; while for Canada ϕ = 0.80 since a slightly higher reliability, β, of 3.0 is employed. The safety factor, Ω, was back calculated from ϕ at an assumed dead to live load ratio of 1 to 5. Since the range of pre-qualified members is relatively large, extensions of the Direct Strength Method to geometries outside the pre-qualified set is allowed. Given the uncertain nature of this extension, increased safety factors and reduced resistance factors are applied in that case, per the rational analysis provisions of Section A1.2(b) of the main *Specification*.

The provisions of Appendix 1, applied to the columns of Section 1.1.1.1, are summarized in Figure C-1.2.1-2 below. The controlling strength is either by Appendix 1 Section 1.2.1.2, which considers local buckling interaction with long column buckling, or by Section 1.2.1.3, which considers the distortional mode alone. The controlling strength (minimum predicted of the two modes) is highlighted for the examined members by the choice of marker. Overall performance of the method can be judged by examination of Figure C-1.2.1-2. Scatter exists throughout the data set, but the trends in strength are clearly shown, and further, the scatter (variance) is similar to that of the main *Specification*.

1.2.1.1 Flexural, Torsional, or Flexural-Torsional Buckling

As discussed in detail above, the strength expressions for long wavelength buckling of columns follow directly from Section C4 of the main *Specification*. These provisions are identical to those used for compact section hot-rolled columns in the AISC Specification (2001) and are fully discussed in the *Commentary* to Section C4. The axial elastic strength, P_{ne}, calculated in this section represents the upper bound capacity for a given column. Actual column strength is determined by considering reductions that may occur due to local buckling, and performing a separate check on the distortional mode. See Section 1.1.2 for information on rational analysis methods for calculation of P_{cre}.

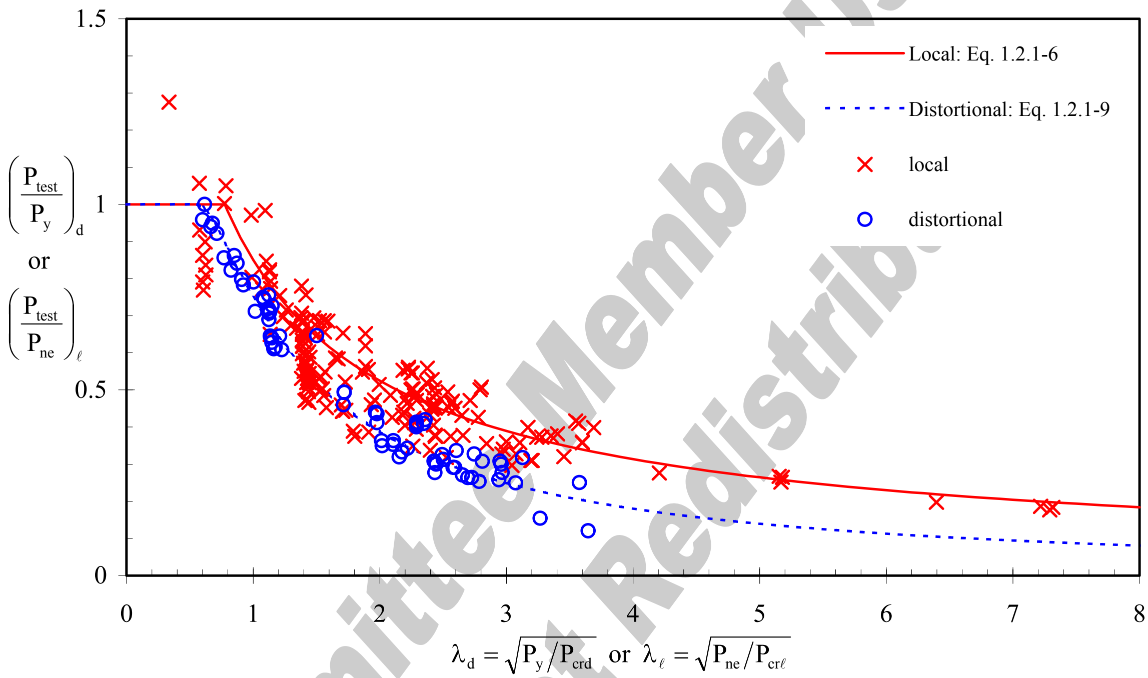


Figure C-1.2.1-2 Direct Strength Method for Concentrically Loaded Pin-Ended Columns

1.2.1.2 Local Buckling

The expression selected for local buckling of columns is shown in Figure C-1.2.1-1 and Figure C-1.2.1-2 and is discussed in Section 1.2.1. The potential for local-global interaction is presumed, thus the column strength in local buckling is limited to a maximum of the long column strength, P_{ne}. See Section 1.1.2 for information on rational analysis methods for calculation of $P_{cr\ell}$.

1.2.1.3 Distortional Buckling

The expression selected for distortional buckling of columns is shown in Figure C-1.2.1-1 and Figure C-1.2.1-2 and is discussed in Section 1.2.1. Based on experimental test data and on the success of the Australian/New Zealand code (see Hancock et al., 2001 for discussion and Hancock et al. 1994 for further details) the distortional buckling strength is limited to P_y instead of P_{ne}. This presumes that distortional buckling failures are independent of long-column behavior, i.e., little if any distortional-global interaction exists. See Section 1.1.2 for information on rational analysis methods for calculation of P_{crd}.

1.2.2 Beam Design

Commentary Section C3 provides a complete discussion on the behavior of cold-formed steel beams as it relates to the main *Specification*. This commentary addresses the specific issues raised by the use of the Direct Strength Method of Appendix 1 for the design of cold-formed steel beams.

The thin-walled nature of cold-formed beams complicates behavior and design. Elastic buckling analysis reveals at least three buckling modes: local, distortional, and lateral-torsional buckling (for members in strong-axis bending) that must be considered in design. The Direct Strength Method of this Appendix emerged through the combination of more refined methods for local and distortional buckling prediction, improved understanding of the post-buckling strength and imperfection sensitivity in distortional failures, and the relatively large amount of available experimental data.

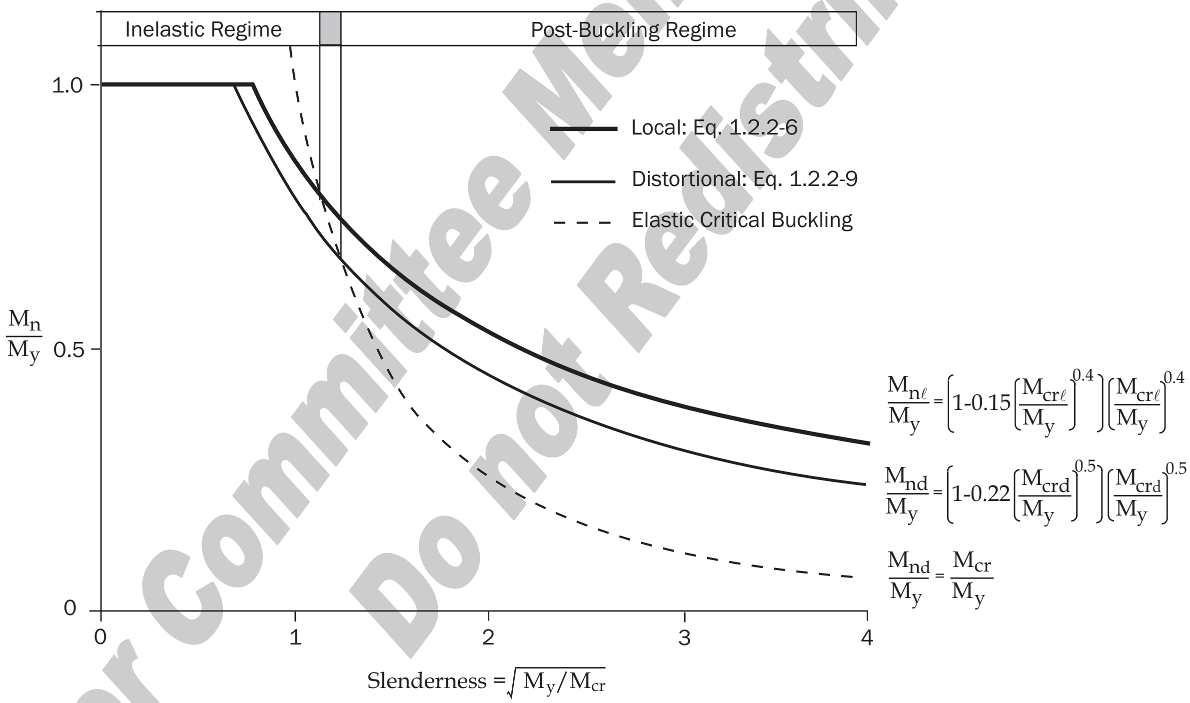


Figure C-1.2.2-1 Local and Distortional Direct Strength Curves for a Braced Beam (Mne = My)

The lateral-torsional buckling strength, M_{ne}, follows the same practice as the main *Specification*. The main *Specification* provides the lateral-torsional buckling strength in terms of a stress, F_c (Equations C3.1.2.1-2, -3, -4 and -5). In the Direct Strength Method, this is converted from a stress to a moment by multiplying by the gross section modulus, S_f, resulting in the formulas for M_{ne} given in Appendix 1.

In the main *Specification*, for beams that are not fully braced and locally unstable, beam strength is calculated by multiplying the predicted stress for failure in lateral-buckling, F_c, by the effective section modulus, S_c, determined at stress F_c. This accounts for local buckling reductions in the lateral-torsional buckling strength (i.e., local-global interaction). In the Direct Strength Method, this calculation is broken into two parts: the lateral-torsional buckling strength without any reduction for local buckling (M_{ne}) and the strength considering local-global interaction ($M_{n\ell}$).

The strength curves for local and distortional buckling of a fully braced beam are presented in Figure C-1.2.2-1 and compared to the critical elastic buckling curve. While the strength in both the local and distortional modes exhibit both an inelastic regime and a post-buckling regime, the post-buckling reserve for the local mode is predicted to be greater than that of the distortional mode.

The reliability of the beam provisions was determined using the test data of Section 1.1.1.2 and the provisions of Chapter F of the main *Specification*. Based on a target reliability, β, of 2.5, a resistance factor, ϕ, of 0.90 was calculated for all the investigated beams. Based on this information the safety and resistance factors of Appendix Section 1.2.2 were determined for the pre-qualified members. For the United States and Mexico ϕ = 0.90; while for Canada ϕ = 0.85 because Canada employs a slightly higher reliability, β, of 3.0. The safety factor, Ω, is back calculated from ϕ at an assumed dead to live load ratio of 1 to 5. Since the range of pre-qualified members is relatively large, extensions of the Direct Strength Method to geometries outside the pre-qualified set is allowed. However, given the uncertain nature of this extension, increased safety factors and reduced resistance factors are applied in that case, per the rational analysis provisions of Section A1.2(b) of the main *Specification*.

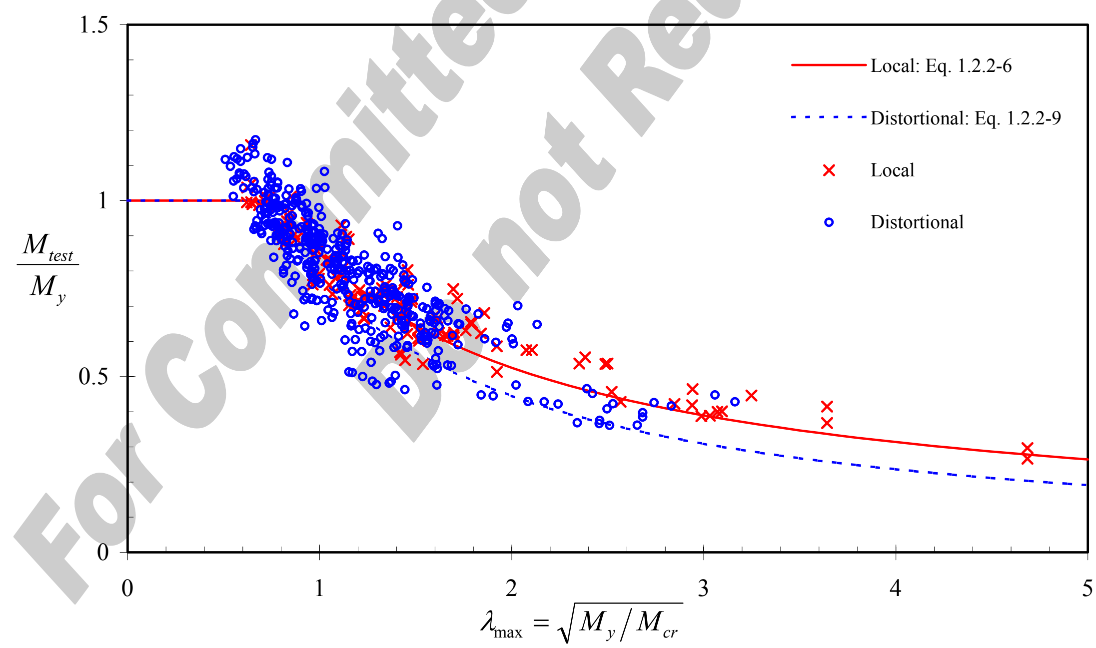


Figure C-1.2.2-2 Direct Strength Method for laterally braced beams

The provisions of Appendix 1, applied to the beams of Section 1.1.1.2, are summarized in

Figure C-1.2.2-2. The controlling strength is determined either by Section 1.2.2.2, which considers local buckling interaction with lateral-torsional buckling, or by Section 1.2.2.3, which considers the distortional mode alone. The controlling strength (minimum predicted of the two modes) is highlighted for the examined members by the choice of marker. Overall performance of the method can be judged by examination of Figure C-1.2.2-2. The scatter shown in the data is similar to that of the main *Specification.*

1.2.2.1 Lateral-Torsional Buckling

As discussed in detail above, the strength expressions for lateral-torsional buckling of beams follow directly from Section C3 of the main *Specification* and are fully discussed in Section C3 of the *Commentary*. The lateral-torsional buckling strength, M_{ne}, calculated in this section represents the upperbound capacity for a given beam. Actual beam strength is determined by considering reductions that may occur due to local buckling and performing a separate check on the distortional mode. See Section 1.1.2 for information on rational analysis methods for calculation of M_{cre}.

1.2.2.2 Local Buckling

The expression selected for local buckling of beams is shown in Figures C-1.2.2-1 and C-1.2.2-2 and is discussed in Section 1.2.2. The use of the Direct Strength Method for local buckling and the development of the empirical strength expression is given in Schafer and Peköz (1998). The potential for local-global interaction is presumed; thus, the beam strength in local buckling is limited to a maximum of the lateral-torsional buckling strength, M_{ne}. For fully braced beams, the maximum M_{ne} value is the yield moment, M_y. See Section 1.1.2 for information on rational analysis methods for calculation of $M_{cr\ell}$.

1.2.2.3 Distortional Buckling

The expression selected for distortional buckling of beams is shown in Figures C-1.2.2-1 and C-1.2.2-2 and is discussed in Section 1.2.2. Based on experimental test data and on the success of the Australian/New Zealand code (see Hancock, 2001 for discussion) the distortional buckling strength is limited to M_y instead of M_{ne}. This presumes that distortional buckling failures are independent of lateral-torsional buckling behavior, i.e., little if any distortional-global interaction exists. See Section 1.1.2 for information on rational analysis methods for calculation of M_{crd}.

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Appendix 2:

Commentary on Appendix 2

Second-Order Analysis

2007 EDITION

APPENDIX 2: COMMENTARY ON APPENDIX 2 SECOND-ORDER ANALYSIS

The provisions of this Appendix are based on Sarawit (2003), Sarawit and Pekoz (2006) and AISC (2005). The provisions here are supported by an extensive study on Industrial Steel Storage Racks (2006) sponsored at Cornell University by the Rack Manufacturers Institute and the American Iron and Steel Institute. The subject of Notional Loads is discussed fully in the Commentary to Appendix 7 of AISC (2005). The application to cold-formed steel structures has to include the frequently encountered flexural-torsional buckling, semi-rigid joints and local instabilities. In Sarawit (2003) and Sarawit and Pekoz (2006) it is shown that the second order analysis gives more accurate results than the effective length approach.

2.1 General Requirements

Required strengths [factored forces and moments] are determined by analysis according to *Specification* Appendix 2 and the members have to satisfy the provisions of Section C5 of the main body of the *Specification*. In checking the strength [resistance] by *Specification* Section C5 magnification of the moments does not need to be included since the second order analysis gives the magnified moments.

Since the frame stability is considered by the second order analysis, nominal axial strength [resistance] in *Specification* Section C5.2 should be determined with an effective length coefficient equal to 1.0.

2.2 Design and Analysis Constraints

Second order frame analysis is permitted either on the out-of-plumb geometry without notional loads or on the plumb geometry by applying notional loads or minimum lateral loads as defined in *Specification* Appendix 2. If second order elastic analysis is used, namely inelasticity effects are not modeled explicitly; axial and flexural stiffnesses are to be reduced as specified in *Specification* Appendix 2.

It is required to carry out a second-order analysis that considers both the effect of loads acting on the deflected shape of a member between joints or nodes (P-δ effects) and the effect of loads acting on the displaced location of joints or nodes in a structure (P-Δ effects). On a member level P-δ effects need to be modeled explicitly. Adding a node or nodes along the length of the member will suffice. These intermediate nodes do not need to account for the initial out-of-straightness for the member. This is because for members, the design equations used include the presence of δ imperfection and thus member strength is already calibrated to include the effect of P-δ.

The 20 percent reduction in member stiffness EI, namely multiplying EI by 0.8, that is used in the AISC Specification (2005) is applied only to E for convenience in analysis. The reasoning for the 20 percent reduction in EI as well as the inelastic buckling factor τ_b is provided in the commentary to the AISC Specification. Part of the justification for 20 percent reduction in member stiffness is based on a resistance factor of 0.9 used in the AISC for columns. However in the AISI Specification the resistance factor is less than 0.9. For this reduced resistance factor, the adequacy of 20 percent reduction in member stiffness for cold-formed steel frames can be deduced from the studies described in Sarawit and Pekoz (2006), which is based on Sarawit (2003). Sarawit and Pekoz (2006) shows that for typical industrial storage rack frames with a wide variety of section properties, configurations, and behavior modes, a reduction of 10percent in member stiffnesses results in an increased conservatism of 10 percent in the calculated load carrying capacity. A 20 percent reduction in member stiffnesses would lead to an increased conservatism of 20 percent in the calculated load carrying capacity. A parametric study of individual columns in Sarawit and Pekoz (2006) shows that some unconservative results can be obtained in a few instances if the stiffness of members is not reduced in the analysis. Reducing the stiffness by 20 percent gives satisfactory results for these cases.

It should be noted that the nominal axial and flexural strengths [resistances] used in the interaction equations of Section C5.2 do not need to be calculated based on reduced value of E.

APPENDIX 2 REFERENCES

American Institute of Steel Construction (2005), Specification for Structural Steel Buildings, March 9, 2005.

Sarawit, A. (2003), Cold-Formed Steel Frame and Beam-Column Design, PhD Thesis, and Research Report 03-03, Department of Civil and Environmental Engineering, Cornell University, Ithaca, New York, March 2003.

Sarawit, A. And T. Pekoz (2006), "Notional Load Method for Industrial Steel Storage Racks," *Thin-Walled Structures*, Elserier, Vol. 44, No. 12, December 2006.

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Appendix A:

Commentary on Provisions Applicable to the United States and Mexico

2007 EDITION

APPENDIX A: COMMENTARY ON PROVISIONS APPLICABLE TO THE UNITED STATES AND MEXICO

The *Commentary* on Appendix A provides a record of reasoning behind, and justification for, provisions that are applicable to the United States and Mexico. The format used herein is consistent with that used in Appendix A of the *Specification*.

A1.1a Scope

In the 2007 edition of the *Specification*, both the Allowable Strength Design and the Load and Resistance Factor Design are permitted to be used in a design.

A2.2 Other Steels

Although the use of ASTM-designated steels listed in *Specification* Section A2.1 is encouraged, other steels may also be used in cold-formed steel structures, provided they satisfy the requirements stipulated in this provision.

In 2004, these requirements were clarified and revised. The *Specification* has long required that such "other steels" conform to the chemical and mechanical requirements of one of the listed specifications or "other published specification." Specific requirements for a published specification have been detailed in the definitions under General Terms, A1.3. It is important to note that, by this definition, published requirements must be established before the steel is ordered, not by a post-order screening process. The requirements must include minimum tensile properties, chemical composition limits, and for coated sheets, coating properties. Testing procedures must be in accord with the referenced ASTM specifications. A proprietary specification of a manufacturer, purchaser, or producer could qualify as a published specification if it meets the definition requirements.

As an example of these *Specification* provisions, it would not be permissible to establish a minimum yield stress or minimum tensile strength, greater than that ordered to a standard ASTM grade, by reviewing mill test reports or conducting additional tests. However, it would be permissible to publish a manufacturer's or producer's specification before the steel is ordered requiring that such enhanced properties be furnished as a minimum. Testing to verify that the minimum properties are achieved could be done by the manufacturer or the producer. The intent of these provisions is to ensure that the material factor M_m (see Chapter F) will be maintained at about 1.10, corresponding to an assumed typical 10 percent overrun in tensile properties for ASTM grades.

Special additional requirements have been added to qualify unidentified material. In such a case, the manufacturer must run tensile tests sufficient to establish that the yield stress and tensile strength of each master coil are at least 10 percent greater than the applicable published specification. As used here, master coil refers to the coil being processed by the manufacturer. Of course, the testing must always be adequate to ensure that specified minimum properties are met, as well as the ductility requirements of *Specification* Section A2.3.

Where the material is used for fabrication by welding, care must be exercised in selection of chemical composition or mechanical properties to ensure compatibility with the welding process and its potential effect on altering the tensile properties.

A2.3a Ductility

The low ductility steel application is limited for curtain wall stud application in heavy weight exterior walls in seismic areas with Design Categories D, E and F.

A3 Loads

A3.1 Nominal Loads

The *Specification* does not establish the dead, live, snow, wind, earthquake or other loading requirements for which a structure should be designed. These loads are typically covered by the applicable building code. Otherwise, the American Society of Civil Engineers Standard, ASCE/SEI 7 (ASCE, 2005) should be used as the basis for design.

Recognized engineering procedures should be employed to reflect the effect of impact loads on a structure. For building design, reference may be made to AISC publications (AISC, 1989; AISC 1999, AISC 2005).

When gravity and lateral loads produce forces of opposite sign in members, consideration should be given to the minimum gravity loads acting in combination with wind or earthquake loads.

A4.1.2 Load Combinations for ASD

In 2001, the *Specification* was revised to specify that all loads and load combinations were required to follow the applicable building code. In the absence of an applicable building code, loads and load combinations should be determined according to the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7 (ASCE, 2005).

When steel decks are used for roof and floor composite construction, steel decks should be designed to carry the concrete dead load, the steel dead load, and the construction live load. The construction load is based on the sequential loading of concrete as specified in the ANSI/ASCE Standard 3-91 (ASCE, 1991) and in the Design Manual of Steel Deck Institute (SDI, 2006).

A5.1.2 Load Factors and Load Combinations for LRFD

In 2001, the *Specification* was revised to specify that all loads and load combinations were required to follow the applicable building code. In the absence of an applicable building code, loads and load combinations should be determined according to the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7 (ASCE, 2005).

In view of the fact that building codes and ASCE/SEI 7 do not provide load factors and load combinations for roof and floor composite construction using cold-formed steel deck, the following load combination may be used for this type of composite construction:

$$1.2D_S + 1.6C_W + 1.4C$$

where

D_s = weight of steel deck

C_w = weight of wet concrete during construction

C = construction load, including equipment, workmen and formwork, but excluding the weight of the wet concrete.

The above load combination provides safety construction practices for cold-formed steel decks and panels which otherwise may be damaged during construction. The load factor used for the weight of wet concrete is 1.6 because of delivering methods and an individual sheet can be subjected to this load. The use of a load factor of 1.4 for the construction load is comparable to the allowable strength design method.

C2 Tension Members

As described in *Specification* Section C2, the nominal tensile strength [resistance] of axially loaded cold-formed steel tension members is determined either by yielding of the gross area of the cross-section or by rupture of the net area of the cross section. At locations of connections, the nominal tensile strength [resistance] is also limited by the capacities specified in *Specification* Sections E2.7, E3, and E5 for tension in connected parts.

Yielding in the gross section indirectly provides a limit on the deformation that a tension member can achieve. The definition of yielding in the gross section to determine the tensile strength [resistance] is well established in hot-rolled steel construction.

For the LRFD Method, the resistance factor of $\phi_t = 0.75$ used for rupture of the net section is consistent with the ϕ factor used in the AISC Specification (AISC, 2005). The resistance factor $\phi_t = 0.90$ used for yielding in the gross section was also selected to be consistent with the AISC Specification (AISC, 2005).

D4 Cold-Formed Steel Light-Frame Construction

In addition to the standards listed in *Specification* Section D4, the following standard should be applicable to the United States:

(e) The *North American Standard for Cold-Formed Steel Framing – Lateral Design* (*Lateral Standard*) addresses the design of lateral force resisting systems to resist wind and seismic forces in a wide range of buildings constructed with cold-formed steel framing. Use of the *Lateral Standard* is mandatory for the design and installation of cold-formed steel light-framed shear walls, diagonal strap bracing (that is part of a structural wall) and diaphragms to resist wind, seismic and other in-plane lateral loads because certain requirements, such as design requirements specific to shear walls and diaphragms sheathed with wood structural panels, gypsum board, fiberboard and steel sheet, as well as special seismic requirements for these and systems using diagonal strap bracing are not adequately addressed by the *Specification*.

D6.1.2 Beams Having One Flange Fastened to a Standing Seam Roof System

For beams supporting a standing seam roof system, e.g. a roof purlin subjected to dead plus live load, or uplift from wind load, the bending capacity is greater than the bending strength of an unbraced member and may be equal to the bending strength of a fully braced member. The bending capacity is governed by the nature of the loading, gravity or uplift, and the nature of the particular standing seam roof system. Due to the availability of many different types of standing seam roof systems, an analytical method for determining positive and negative bending capacities has not been developed at the present time. However, in order to resolve this issue relative to the gravity loading condition, Section D6.1.2 was added in the 1996 edition of the AISI *Specification* for determining the nominal flexural strength of beams having one flange fastened to a standing seam roof system. In *Specification* Equation D6.1.2-1, the reduction factor, R, can be determined by AISI S908 published by AISI (AISI, 2004). Application of the base test method for uplift loading was subsequently validated after further analysis of the research results.

D6.1.4 Compression of Z-Section Members Having One Flange Fastened to a Standing Seam Roof

The strength of axially loaded Z-sections having one flange attached to standing seam roof may be limited by either a combination of torsional buckling and lateral buckling in the plane of the roof, or by flexural buckling in a plane perpendicular to the roof. As in the case of Z-sections carrying gravity or wind loads as beams, the roof diaphragm and purlin clips provide a degree of torsional and lateral bracing restraint that is significant, but not necessarily sufficient, to develop the full strength of the cross section.

Specification Equation D6.1.4-1 predicts the lateral buckling strength using an ultimate axial buckling stress ($k_{af}RF_y$) that is a percentage of the ultimate flexural stress (RF_y) determined from uplift tests performed using AISI S908, Base Test Method for Purlins Supporting a Standing Seam Roof System, as published by AISI. This equation, developed by Stolarczyk, el al. (2002), was derived empirically from elastic finite element buckling studies and calibrated to the results of a series of tests comparing flexural and axial strengths using the uplift "Base Test" setup. The gross area, A, has been used rather than the effective area, A_e, because the ultimate axial stress is generally not large enough to result in a significant reduction in the effective area for common cross section geometries.

Specification Equation D6.1.4-1 may be used with the results of uplift "Base Tests" conducted with and without discrete point bracing. There is no limitation on the minimum length because Equation D6.1.4-1 is conservative for spans that are smaller than that tested under the "Base Test" provisions.

The strength of longer members may be governed by axial buckling perpendicular to the roof; consequently, the provisions of Specification Sections C4.1 and C4.1.1 should also be checked for buckling about the strong axis.

D6.2.1a Strength [Resistance] of Standing Seam Roof Panel Systems

The introduction of the wind uplift loading required strength factor of 0.67 was a result of research conducted to correlate the static uplift capacity represented by tests performed in accordance with S906 (AISI, 2008) and the dynamic behavior of real wind, Surry et. al. (2007). This research utilized two separate methods of comparison. The first method utilized full-scale tests conducted at Mississippi State University (MSU) using simulated wind loads on a portion of a standing seam metal roof and the second method utilized model-scale wind tunnel tests carried out at the University of Western Ontario of an aeroelastic "failure" model of the same roof system. In spite of these significantly different approaches, the results obtained were very consistent. It was found that the E1592 uniform pressure test contains conservatism of about 50 percent for the roof system tested by both approaches, and up to about 80 percent for the other roof systems tested only at MSU. This conservatism arises if the roof system is required to withstand the code-recommended pressure applied as uniform pressure in the E1592 test, without accounting for the reality of the dynamic spatially-varying properties of the wind-induced pressures. The limits of applicability of this factor (panel thickness and width) are conservatively listed based on the scope of the research. The failure mode is restricted to those failures associated with the load in the clip because this was how the research measured and compared the static and dynamic capacities. The required strength factor of 0.67 is not permitted to be used with other observed failures. In addition, the research does not support or confirm whether interpolation would be appropriate between E1592 tests of the same roof system with different spans, where one test meets the requirements, such as a clip failure, and another test does not, such as a panel failure.

E2a Welded Connections

The upper limit of the *Specification* applicability was revised in 2004 from 0.18 in. (4.57 mm) to 3/16 in. (4.76 mm). This change was made to be consistent with the limit given in the AWS D1.3 (1998).

The design provisions for welded connections were developed based primarily on experimental evidence obtained from an extensive test program conducted at Cornell University. In addition, the Cornell research provided the experimental basis for the AWS Structural Welding Code for Sheet Steel (AWS, 1998). In most cases, the provisions of the AWS code are in agreement with this *Specification* section.

The terms used in this *Specification* section agree with the standard nomenclature given in the AWS Welding Structural Code for Sheet Steel (AWS, 1998).

For welded material thicknesses greater than 3/16 in. (4.76 mm), AISC Specification (2005) should be followed.

E3a Bolted Connections

In Table E3a of Appendix A, the maximum size of holes for bolts having diameters not less than 1/2 inch (12.7 mm) is based on the specifications of the Research Council on Structural Connections and the American Institute of Steel Construction (RCSC, 2000 and 2004; AISC, 1989, 1999, and 2005), except that for the oversized hole diameter, a slightly larger hole diameter is permitted.

For bolts having diameters less than 1/2 inch (12.7 mm), the diameter of a standard hole is the diameter of bolt plus 1/32 inch (0.794 mm). This maximum size of bolt holes is based on previous editions of the AISI *Specification*.

When using oversized holes care must be exercised by the designer to ensure that excessive deformation due to slip will not occur at working loads. Excessive deformations which can occur in the direction of the slots may be prevented by requiring bolt pretensioning.

Short-slotted holes are usually treated in the same manner as oversized holes. Washers or backup plates should be used over oversized or short-slotted holes in an outer ply unless suitable performance is demonstrated by tests. For connections using long-slotted holes, *Specification* Section E3.4 requires the use of washers or back-up plates and that the shear capacity of bolts be determined by tests because a reduction in strength may be encountered. An exception to the provisions for slotted holes is made in the case of slotted holes in lapped and nested zees. Resistance is provided in this situation partially by the nested components, rather than direct bolt shear and bearing. An oversize or slotted hole is required for proper fit-up due to offsets inherent in nested parts. Recent research (Bryant and Murray, 2001) has shown that lapped and nested zee members with 1/2 in. (12.7 mm) diameter bolts without washers and 9/16 in. x 7/8 in. (14.3 mm x 22.2 mm) slotted holes in the direction of stress can develop the full moment in the lap.

E3.1 Shear, Spacing and Edge Distance

The provisions for minimum spacing and edge distance were revised in 1980 to include additional design requirements for bolted connections with standard, oversized, and slotted holes. The minimum edge distance of each individual connected part, e_{min}, is determined by using the tensile strength of steel (F_u) and the thickness of connected part. According to the different ranges of the F_u/F_{sy} ratio, two different safety and resistance factors are used for determining the required minimum edge distance. These design provisions are based on the following basic equation established from the test results:

$$e = \frac{P}{F_u t} \qquad \text{(C-E3.1-1)}$$

in which e is the required minimum edge distance to prevent shear failure of the connected part for a force, P, transmitted by one bolt, and t is the thickness of the thinnest connected part. For design purpose, a safety factor of 2.0 and a resistance factor of 0.70 are used for $F_u/F_{sy} \geq 1.08$. For $F_u/F_{sy} < 1.08$, a safety factor of 2.22 and a resistance factor of 0.60 are used according to the degree of correlation between the above equation and the test data. In addition, several requirements were added to the AISI *Specification* in 1980 concerning (1) the minimum distance between centers of holes, as required for installation of bolts, (2) the required clear distance between edges of two adjacent holes, and (3) the minimum distance between the edge of the hole and the end of the member. The same design provisions were retained in the 1986 AISI *Specification* and were also used in the 1996 AISI *Specification*, except that the limiting F_u/F_{sy} ratio has been reduced from 1.15 to 1.08 for the consistency with *Specification* Section A2.3.1. The test data used for the development of Equation C-E3.1-1 are documented by Winter (1956a and 1956b) and Yu (1982, 1985, and 2000).

E3.2 Rupture in Net Section (Shear Lag)

In the AISI *Specification*, the nominal tensile strength [resistance], P_n, of the net section of bolt connected parts is based on the loads determined by *Specification* Sections C2 and E3.2, whichever is smaller. In the use of the equations provided in *Specification* Section E3.2, the following design features should be noted:

1. The provisions are applicable only to the thinnest connected part less than 3/16 inch (4.76 mm) in thickness. For materials thicker than 3/16 inch (4.76 mm), the design should follow the specifications or standards stipulated in Section E3a of Appendix A or B.
2. The nominal tensile strength, P_n, on the net section of a bolt connected member is determined by the tensile strength of the connected part (F_u), and the ratio "d/s" for connections with a single bolt or a single row of bolts perpendicular to the force.
3. Different equations are given for bolted connections with and without washers (Chong and Matlock, 1975).

4. The nominal tensile strength on the net section of a connected member is based on the type of joint, either a single shear lap joint or a double shear butt joint.

The presence of staggered or diagonal hole patterns in a bolted connection has long been recognized as increasing the net section area for the limit state of rupture in the net section. LaBoube and Yu (1995) summarized the findings of a limited study of the behavior of bolted connections having staggered hole patterns. The research showed that when a staggered hole pattern is present, the width of a rupture plane can be adjusted by use of $s'^2/4g$.

Because of the lack of test data necessary for a more accurate design formulation, a discontinuity between this *Specification* and the specifications or standards, stipulated in Appendix A, may occur. The presence of a discontinuity should not be a significant design issue because the use of the staggered hole patterns is not common in cold-formed steel applications.

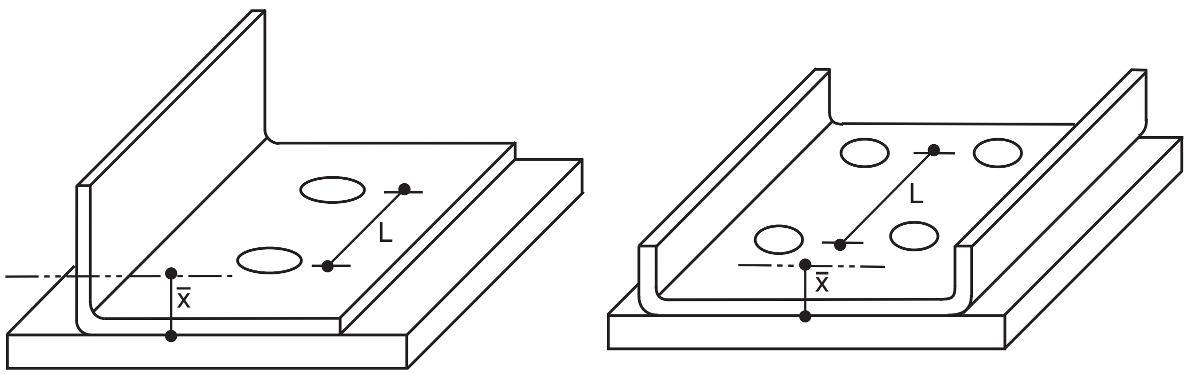


Figure C-E3.2-1 $\bar{x}$ Definition for Sections with Bolted Connections

Shear lag has a debilitating effect on the tensile capacity of a cross section. Based on UMR research (LaBoube and Yu, 1995) design equations have been developed that can be used to estimate the influence of the shear lag. The research demonstrated that the shear lag effect differs for an angle and a channel. For both cross sections, however, the key parameters that influence shear lag are the distance from the shear plane to the center of gravity of the cross section and the length of the bolted connection (Fig. C-E3.2-1). The research showed that for cold-formed sections using single bolt connections, bearing usually controlled the nominal strength, not rupture in the net section.

Previous tests showed that for flat sheet connections using a single bolt or a single row having multiple bolts perpendicular to the force (Chong and Matlock, 1975; Carill, LaBoube and Yu, 1994), the joint rotation and out-of plane deformation of flat sheets are excessive. The strength reduction due to tearing of steel sheets in the net section is considered by *Specification* Equations. E3.2-2 and E3.2-4 according to the d/s ratio and the use of washers (AISI, 1996). For flat sheet connections using multiple bolts in the line of force and having less out-of-plane deformations, the strength reduction is not required in this edition of the *Specification* (Rogers and Hancock, 1998).

For flat sheet connections having staggered hole patterns as shown in Figure C-E3.2-2, the nominal tensile strength of path ABDE can be determined by *Specification* Section E3.2(a). In addition, the nominal tensile strength of the staggered path ABCDE can be determined by *Specification* Section E3.2(b). For this case, *Specification* Equation E3.2-2 can be used to compute F_t as long as each line of bolts parallel to the force has only one bolt.

The value for ϕ used with *Specification* Equation E3.2-8 is based on statistical analysis of the test data with a corresponding value of β = 3.5 for LRFD. The Ω values are unchanged from previous editions of the AISI ASD *Specification*.

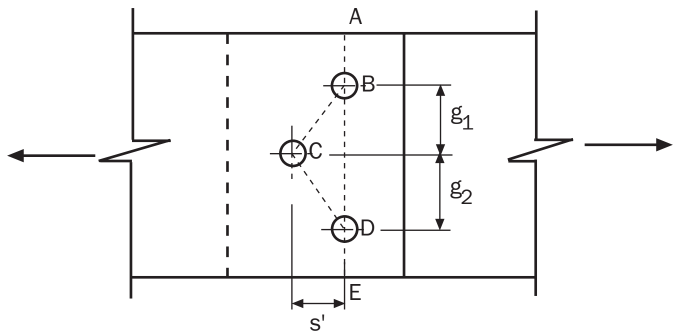


Figure C-E3.2-2 Flat Sheet Connections Having Staggered Holes

E3.4 Shear and Tension in Bolts

For the design of bolted connections, the allowable shear stresses for bolts have been provided in the AISI *Specification* for cold-formed steel design since 1956. However, the allowable tension stresses were not provided in *Specification* Section E3.4 for bolts subject to tension until 1986. In *Specification* Table E3.4-1, the allowable stresses specified for A307 (d ≥ 1/2 inch (12.7 mm)), A325, and A490 bolts were based on Section 1.5.2.1 of the AISC *Specification* (1978). It should be noted that the same values were also used in Table J3.2 of the AISC ASD *Specification* (1989). For A307, A449, and A354 bolts with diameters less than 1/2-inch (12.7 mm), the allowable tension stresses were reduced by 10 percent, as compared with these bolts having diameters not less than 1/2 inch (12.7 mm), because the average ratio of (tensile-stress area)/(gross-area) for 1/4-inch (6.35 mm) and 3/8-inch (9.53 mm) diameter bolts is 0.68, which is about 10 percent less than the average area ratio of 0.75 for 1/2-inch (12.7 mm) and 1-inch (25.4 mm) diameter bolts. In the AISI ASD/LRFD *Specification* (1996), Table E3.4-1 provided nominal tensile strengths for various types of bolts with applicable safety factors. The allowable tension stresses computed from F_{nt}/Ω were approximately the same as that permitted by the AISI 1986 ASD *Specification*. The same table also gave the resistance factor to be used for the LRFD method.

The design provisions for bolts subjected to a combination of shear and tension were added in AISI *Specification* Section E3.4 in 1986. Those design equations were based on Section 1.6.3 of the AISC *Specification* (AISC, 1978) for the design of bolts used for bearing-type connections.

In 1996, *Specification* Tables E3.4-2 to E3.4-5, which listed the equations for determining the reduced nominal tension stress, F'_{nt}, for bolts subjected to the combination of shear and tension were included in the *Specification* and were retained in the 2001 edition. In 2007, *Specification* Tables E3.4-2 to E3.4-5 were replaced by *Specification* Equations E3.4-2 and E3.4-3 to determine the reduced tension stress of bolts subjected to the combined tension and shear. *Specification* Equations E3.4-2 and E3.4-3 were adopted to be consistent with the AISC Specification (AISC, 2005).

Note that when the required stress, f, in either shear or tension, is less than or equal to 20 per cent of the corresponding available stress, the effects of combined stress need not be investigated.

For bolted connection design, the possibility of pullover of the connected sheet at the bolt head, nut, or washer should also be considered when bolt tension is involved, especially for thin sheathing material. For unsymmetrical sections, such as C- and Z-sections used as purlins or girts, the problem is more severe because of the prying action resulting from rotation of the member which occurs as a consequence of loading normal to the sheathing. The designer should refer to applicable product code approvals, product specifications, other literature, or tests.

For design tables and example problems on bolted connections, see Part IV of the *Design Manual* (AISI, 2008).

E4.3.2 Connection Shear Limited by End Distance

The nominal shear per fastener as limited by edge distance is the same as that specified for bolts.

E5 Rupture

Connection tests conducted by Birkemoe and Gilmor (1978) have shown that on coped beams a tearing failure mode as shown in Figure C-E5-1(a) can occur along the perimeter of the holes. Hardash and Bjorhovde (1985) have demonstrated these effects for tension members as illustrated in Figure C-E5-1(b) and Figure C-E5-2. The provisions provided in *Specification* Section E5 for shear rupture have been adopted from the AISC Specification (AISC, 1978). For additional design information on tension rupture strength [resistance] and block shear rupture strength [resistance] of connections (Figures C-E5-1 and C-E5-2), refer to the AISC Specifications (AISC, 1989, 1999, and 2005).

Block shear is a limit state in which the resistance is determined by the sum of the shear strength [resistance] on a failure path(s) parallel to the force and the tensile strength [resistance] on the segment(s) perpendicular to the force, as shown in Figure C-E5-2. A comprehensive test program does not exist regarding block shear for cold-formed steel members. However, a limited study conducted at the University of Missouri-Rolla indicates that the AISC LRFD equations may be applied to cold-formed steel members. The ϕ (LRFD) and Ω (ASD) values for block shear were taken from the AISI 1996 edition of the *Specification*, and are based on the performance of fillet welds. In calculating the net web area A_{wn}, for coped beams, the web depth is taken as the flat portion of the web as illustrated in Fig. C-E5-3.

The summary paper "AISC LRFD Rules for Block Shear in Bolted Connections – A Review" (Kulak and Grondin, 2001) provides a summary of test data for block shear rupture strength. In 2004, Equations E5.3-1 and E5.3-2 were adopted for the limit state of block shear rupture for bolted cold-formed steel connections because eccentricity in cold-formed steel sections is usually small. In theory, provisions for block shear could also be applied to screw connections. However, because the final placement location of self-drilling screws cannot be assured, a block shear check is of little significance. Also, tests performed at the University of Missouri-Rolla have indicated that the current design equations for shear and tilting provide a reasonably good estimate of the connection performance for multiple screws in a pattern (LaBoube and Sokol, 2002).

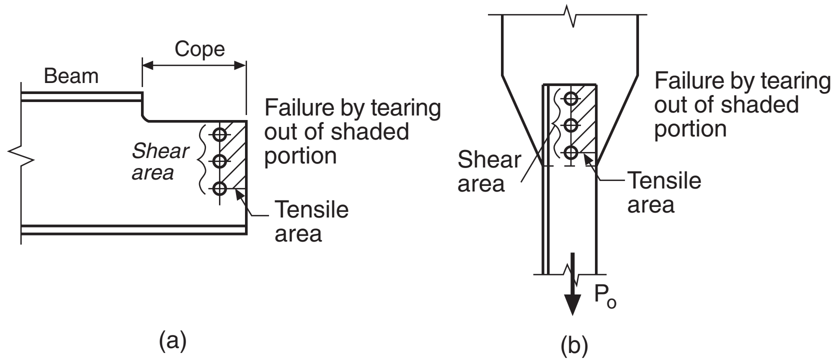


Figure C-E5-1 Failure Modes for Block Shear Rupture

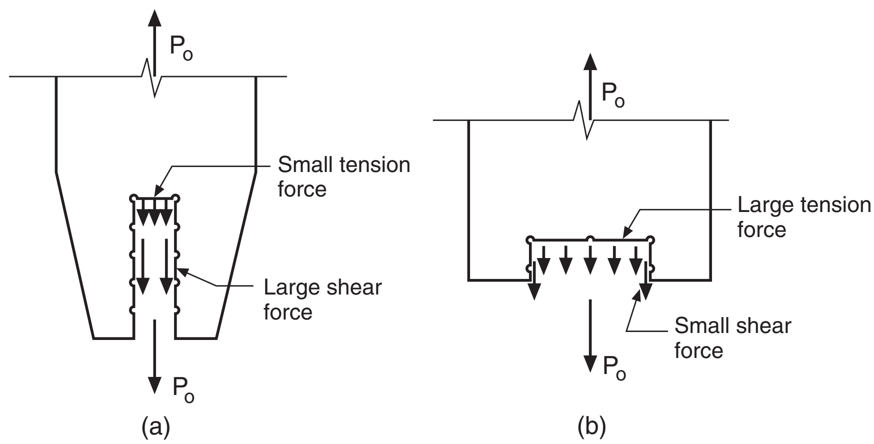


Figure C-E5-2 Block Shear Rupture in Tension

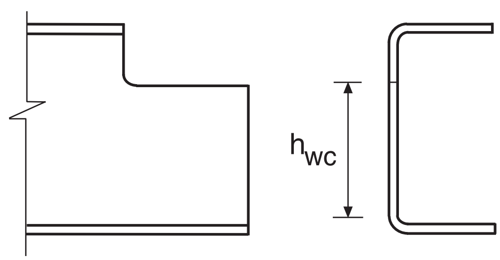


Figure C-E5-3 Definition of h_{wc}

CANADIAN STANDARDS ASSOCIATION

Appendix B:

Commentary on Provisions

Applicable to Canada

2007 EDITION

APPENDIX B: COMMENTARY ON PROVISIONS APPLICABLE TO CANADA

This commentary on Appendix B of the *Specification* provides a record of reasoning behind, and justification for, provisions that are applicable only to Canada. Only those sections of Appendix B of the *Specification* are addressed herein or where additional commentary is required beyond what is already contained in the *Commentary on the 2007 Edition of the North American Specification for the Design of Cold-Formed Steel Structural Members* (hereinafter referred to as the *Commentary*). The format used herein is consistent with that used in Appendix B of the *Specification*.

In comparison to Appendix B of the 2001 edition of CSA Standard S136, a few changes have been incorporated into this *Specification*. The most significant ones are as follows:

a) The entire Section A2.4a, including Table B-A2.4-1 [Hot-Dipped Metallic Coating Thickness Allowances], has been deleted from Appendix B. Information on metallic coating thicknesses can now be found in Sheet Steel Facts #10, published by the Canadian Sheet Steel Building Institute (CSSBI) and is available at www.cssbi.ca.
b) The entire Section A3 on loads has been revised in accordance with the 2005 Edition of the *National Building Code of Canada.*
c) Some changes have been made in Section C2.2 on rupture of net section tension members, and a new section has been added for coped beams.

A2.1a Applicable Steels

CSA Standard G40.20/G40.21 is referenced because it is widely used in Canada for structural quality bars and plate.

A2.2.2 Other Steels

Provisions are included for determining the mechanical properties of unidentified structural steels.

A2.3.1a Ductility

The use of low ductility steel has been limited to *curtain wall stud* applications in specific low seismic areas.

A3 Loads

The load provisions contained in Appendix B of CSA S136-01 were changed to be compatible with the changes that are incorporated in Part 4 of the *National Building Code of Canada* (NBC) 2005. This entails the following:

(1) The version of Limit States Design in NBC 2005 is based on the companion action format, which is being adopted world-wide and is a more rational method of combining loads than the previous version.
(2) NBC 1995 distinguished wind load for different categories of buildings using a return period approach, an increase in design loads for earthquake based on building use by means of an importance factor, and made no allowance for different snow loads based on

the occupancy of the structure. In NBC 2005, it was decided to harmonize the approach used, and so the importance factor methodology was chosen for snow, wind and earthquake loads.

A6 Limit States Design

In limit states design, the resistance of a structural component is checked against the various limit states. For the ultimate limit states resistance, the structural member must retain its load-carrying capacity up to the factored load levels. For serviceability limit states, the performance of the structure must be satisfactory at specified load levels. Specified loads are those prescribed by the *National Building Code of Canada*. Examples of serviceability requirements include deflections and the possibility of vibrations.

Section A6 of the *Specification* sets forth the fundamental safety criterion that must be met, namely:

Factored resistance ≥ effect of factored loads

The factored resistance is given by the product ϕR_n, where ϕ is the resistance factor which is applied to the nominal member resistance, R_n. The resistance factor is intended to take into account the fact that the resistance of the member may be less than anticipated, due to variability of the material properties, dimensions, and workmanship, and also to take into account the type of failure and uncertainty in the prediction of the resistance.

The resistance factor does not, however, cover gross human errors. Human errors cause most structural failures and typically these human errors are "gross" errors. Gross errors are completely unpredictable and are not covered by the overall safety factor inherent in buildings.

In limit states design, structural reliability is specified in terms of a safety index, β, determined through a statistical analysis of the loads and resistances. The safety index is directly related to the structural reliability of the design; hence, increasing β increases the reliability, and decreasing β decreases the reliability. The safety index, β, is also directly related to the load and resistance factors used in the design.

The *National Building Code of Canada* defines a set of load factors, load combination factors, and specified minimum loads to be used in the design, hence fixing the position of the nominal load distribution and the factored load distribution. The design Standard is then obligated to specify the appropriate resistance function.

Those responsible for writing a design Standard are given the load distribution and load factors, and must calibrate the resistance factors, ϕ, such that the safety index, β, reaches a certain target value. The technical committee responsible for CSA Standard S136 elected to use a target safety index of 3.0 for members and 4.0 for connections.

In order to determine the loading for calibration, it was assumed that 80% of cold-formed steel is used in panel form (e.g., roof or floor deck, wall panels, etc.) and the remaining 20% for structural sections (purlins, girts, studs, etc.). An effective load factor was arrived at by assuming live-to-dead load ratios and their relative frequencies of occurrence.

Probabilistic studies show that consistent probabilities of failure are determined for all live-to-dead load ratios when a live load factor of 1.50 and a dead load factor of 1.25 are used.

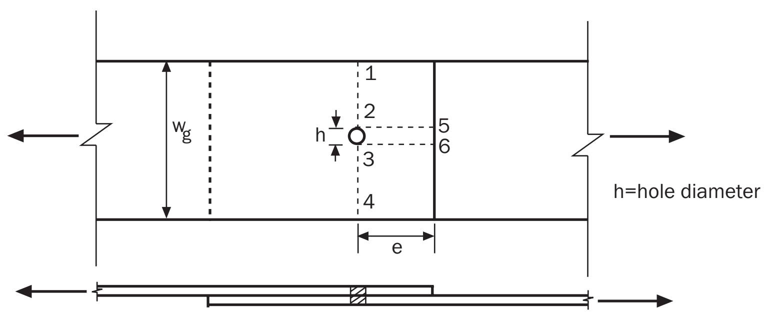
C2 Tension Members

The general provisions for the design of tension members have not changed with respect to the CSA Standard S136-01. The only change that was made involves staggered connections.

C2.2 Rupture of Net Section

Based on the research carried out by LaBoube and Yu (1995), a correction was made by only applying the 0.9 factor to the direct tension and stagger failure paths. In CSA S136-01, the 0.9 factor was also applied to the shear failure path. See *Commentary* for detailed explanation.

Examples of tension members are shown in Figures B-C2.2-1 and B-C2.2-2. Block tear-out can also occur at the end of a coped beam, where the applied force is a shear at the end of a beam. This force causes tension on horizontal planes and shear on vertical planes. An example is shown in Figure B-C2.2-3. Other possible failure paths should also be checked.



Failure Path 1, 2, 3, 4

$L_c = L_t$

$L_t = (w_g - h)$

$L_c = (w_g - h)$

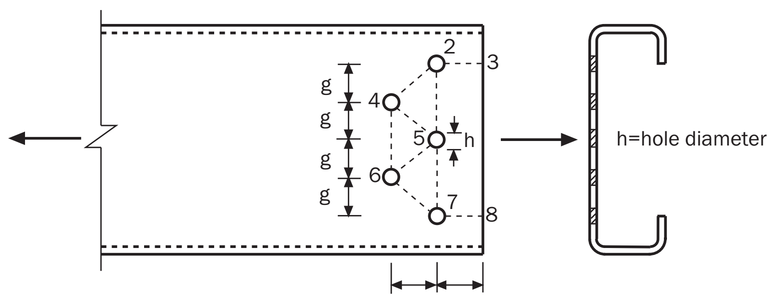
Failure Path 5, 2, 3, 6

$L_c = 0.6L_v$

$L_v = 2(e - h/2)$

$L_c = 0.6[2(e - h/2)] = 1.2e - 0.6h$

Figure B-C2.2-1 Potential Failure Paths of Single Lap Joint



Failure Path 3, 2, 4, 6, 7, 8

$L_c = 0.9[L_t + L_s] + 0.6L_v$

$L_t = (2g - h)$

$L_s = 2(g + s^2/4g - h)$

$L_v = (2e - h)$

$L_c = 0.9[(2g - h) + 2(g + s^2/4g - h)]$
$\quad + 0.6(2e - h)$

Failure Path 3, 2, 4, 5, 6, 7, 8

$L_c = 0.9[L_t + L_s] + 0.6L_v$

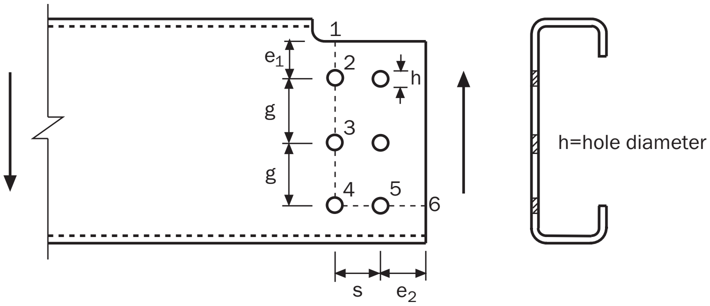
$L_t = 0$

$L_s = 4(g + s^2/4g - h)$

$L_v = (2e - h)$

$L_c = 0.9[4(g + s^2/4g - h)] + 0.6(2e - h)$

Figure B-C2.2-2 Potential Failure Paths of Stiffened Channel



<u>Failure Path 1, 2, 3, 4, 5, 6</u>

$L_c = L_t + 0.6L_v$

$L_t = (s + e_2 - 1.5h)$

$L_v = (e_1 + 2g - 2.5h)$

$L_c = (s + e_2 - 1.5h) + 0.6(e_1 + 2g - 2.5h)$

Figure B-C2.2-3 Potential Failure Path of Coped Stiffened Channel

The provision regarding block tear-out of Section C2.2 was rewritten in accordance with the latest research by Kulak and Grondin (2001). A new section on coped beams was also added as per the recommendations by these authors.

D3a Lateral and Stability Bracing

The provisions of this section cover members loaded in the plane of the web. Conditions may occur that cause a lateral component of the load to be transferred through the bracing member to supporting structural members. In such a case, these lateral forces shall be additive to the requirements of this section. The provisions in the *Specification* recognize the distinctly different behavior of the members to be braced, as defined in Section D3.1 and D3.2 of this Appendix. The term "discrete braces" is used to identify those braces that are only connected to the member to be braced for this express purpose.

D3.1a Symmetrical Beams and Columns

D3.1.1 Discrete Bracing for Beams

This section was revised to retain the 2% requirement for the compressive force in the compressive flange of a flexural member at the braced location only. The discrete bracing provisions for columns are provided in Section D3.3.

D3.2a C-Section and Z-Section Beams

This section covers bracing requirements of channel and Z-sections and any other section in which the applied load in the plane of the web induces twist.

D3.2.2 Discrete Bracing

This section provides for brace intervals to prevent the member from rotating about the shear centre for channels or from rotating about the point of symmetry for Z-sections. The spacing must be such that any stresses due to the rotation tendency are small enough so that they will not significantly reduce the load-carrying capacity of the member. The rotation must also be small enough (in the order of 2°) to be not objectionable as a service requirement.

Based on tests and the study by Winter et al. (1949b), it was found that these requirements are satisfied for any type of load if braces are provided at intervals of one-quarter of the span, with the exception of concentrated loads requiring braces near the point of application.

Fewer brace points may be used if it can be shown to be acceptable by rational analysis or testing in accordance with Chapter F of the *Specification*, recognizing the variety of conditions, including the case where loads are applied out of the plane of the web.

For sections used as purlins with a standing seam roof, the number of braces per bay is often determined by rational analysis and/or testing. The requirement for a minimum number of braces per bay is to recognize that predictability of the lateral support and rotational restraint is limited on account of the many variables such as fasteners, insulation, friction coefficients, and distortion of roof panels under load.

D3.2.3 One Flange Braced by Deck, Slab, or Sheathing

Forces generated by the tendency for lateral movement and/or twist of the beams, whether cumulative or not, must be transferred to a sufficiently stiff part of the framing system. There are several ways in which this transfer may be accomplished:

(a) by the deck, slab, or sheathing providing a rigid diaphragm capable of transferring the forces to the supporting structure;

(b) by arranging equally loaded pairs of members facing each other;

(c) by direct axial force in the covering material that can be transferred to the supporting structure or balanced by opposing forces;

(d) by a system of sag members such as rods, angles, or channels that transfer the forces to the supporting structure; or

(e) by any other method that designers may select to transfer forces to the supporting structure.

For all types of single web beams, the flange that is not attached to the deck or sheathing material may be subject to compressive stresses under certain loading arrangements, such as beams continuous over supports or under wind load. The elastic lateral support to this flange provided through the web may allow an increase in limit stress over that calculated by assuming that the compressive flange is a column, with pinned ends at points of lateral bracing. Research indicates that the compressive limit stress is also sensitive to the rotational flexibility of the joint between the beam and the deck or sheathing material.

This section is intended to apply even when the flange that is not attached to the sheathing material is in tension.

E2a Welded Connections

See *Commentary* for detailed information. Both fabricators and erectors must be certified under CSA Standard W47.1 for arc welding and CSA Standard W55.3 for resistance welding.

This provision extends the certification requirements to the welding of cold-formed members or components to other construction, e.g., welding steel deck to structural steel framing.