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Cold-Formed Steel Design - Vol. 1

American Iron and Steel Institute (AISI)

AISI MANUAL

Cold-Formed Steel Design - Vol. 1

2013 Edition

American Iron and Steel Institute

SECTION GUIDE

VOLUME 1

Part I

Part II

Part III

Part IV

Part V

Part VI

VOLUME 2

Part VII

Part VIII

AISI MANUAL

Cold-Formed Steel Design – Vol. 1

2013 Edition

American Iron and Steel Institute

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The material contained herein has been developed by the American Iron and Steel Institute (AISI) Committee on Specifications for the Design of Cold-Formed Steel Structural Members. The Committee has made a diligent effort to present accurate, reliable, and useful information on cold-formed steel design. The Committee acknowledges and is grateful for the contributions of the numerous researchers, engineers, and others who have contributed to the body of knowledge on the subject.

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 AISI will publish updates of 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.

The AISI *Cold-Formed Steel Design Manual* has been produced by the Steel Market Development Institute, a business unit of AISI.

First Printing – June 2014

Produced by Computerized Structural Design, S.C.
Milwaukee, Wisconsin

This edition of the AISI *Cold-Formed Steel Design Manual* is dedicated to Richard (Dick) Kaehler, P.E. In 1996, Dick developed the first AISI combined ASD and LRFD *Cold-Formed Steel Design Manual*, and he has developed every edition of the *Manual* since then. As a highly respected professional in structural analysis, design and testing, Dick is noted for his expertise in developing design manuals, design guides and computer programs. He has the unique ability to illustrate the applications of design standards through numerical examples by weaving his knowledge and experience within his products. Engineers, students and general users have greatly benefited from his many contributions.

For several years, Dick has contributed his expertise and insight to the industry as a member of the AISI Committee on Specifications for the Design of Cold-Formed Steel Structural Members and its various subcommittees. In recognition of Dick's many contributions he has been awarded Emeritus status on the AISI Committee on Specifications. In addition to his contributions to furthering the application of cold-formed steel, Dick has been actively involved in many other voluntary steel industry committees.

The cold-formed steel industry gratefully acknowledges his many enduring contributions.

PREFACE

The 2013 Edition of the *Cold-Formed Steel Design Manual* consists of a total of eight parts in two volumes. Parts I through VI are included in Volume 1. Parts VII and VIII are included in Volume 2. The information in Volume 1 (Parts I-VI) is supplemental to and should be used in conjunction with Volume 2 (Parts VII and VIII), the 2012 Edition of the *North American Specification for the Design of Cold-Formed Steel Structural Member*s and *Commentary*.

Volume 1:

Part I, Dimensions and Properties contains: (a) information regarding the availability and properties of steels referenced in the *Specification*, (b) tables of cross-section properties, and (c) formulas and examples of calculations of cross-section properties.

Part II, Beam Design contains: (a) tables and charts to aid in beam design, and (b) beam design example problems.

Part III, Column Design contains: (a) tables to aid in column design, and (b) column design example problems.

Part IV, Connection Design contains: (a) tables to aid in connection design, and (b) connection example problems.

Part V, Supplementary Information contains: (a) a table of *Specification* cross-references to the examples provided in this manual, (b) design procedures of specification nature which are not included in the *Specification* itself, either because they are infrequently used or are regarded as too complex for routine design, and (c) other information intended to assist users of cold-formed steel.

Part VI, Test Procedures contains: (a) a bibliography of other pertinent test methods, and (b) test calibration example problems.

Volume 2:

Part VII, North American Specification contains the 2012 Edition of the *North American Specification for the Design of Cold-Formed Steel Structural Members*.

Part VIII, Commentary on the North American Specification contains the 2012 Edition of the *Commentary on the North American Specification for the Design of Cold-Formed Steel Structural Members.*

In addition to updating the *Design Manual* for conformance with the 2012 Edition of the *North American Specification*, the following changes and improvements or additions have been made:

- Design tables and charts have been revised to remove cross-sections not commonly available.

- The following design examples have been added to illustrate new design provisions in the *Specification*:
    (a) Effective section properties of panel section with large radii
    (b) Effective section properties of cellular deck with intermittent fasteners between deck and cover plate

(c) Four span continuous standing seam roof system
(d) C-Section members with openings using the Direct Strength Method in Parts II and III
(e) Shear strength by Direct Strength Method in Part II
(f) Inelastic reserve strength by Direct Strength Method in Part II
(g) Braced frame design with consideration of second-order analysis in Part III
(h) Flare bevel groove weld with $t > 0.10$ in.
(i) Flare V groove weld
(j) Top arc seam sidelap weld
(k) Power-actuated fasteners in shear and tension
(l) Test method to support rational-analysis in Part VI

- AISI test standards have been removed from the *Design Manual* and are available free to download from the AISI website (www.steel.org).

- The 2012 Edition of the *North American Specification for the Design of Cold-Formed Steel Structural Members* has been added as Part VII.

- The 2012 Edition of the *Commentary on North American Specification for the Design of Cold-Formed Steel Structural Members* has been added as Part VIII.

AISI acknowledges the technical information provided by the Steel Deck Institute in the Steel Deck section in Part I, and the exemplary efforts of Richard C. Kaehler with Computerized Structural Design, S.C., in developing this *Design Manual*. Special thanks also are also extended to the members of the AISI Education Subcommittee:

Roger A. LaBoube, *Chairman*	W. Don Allen	James K. Crews
Perry S. Green	Jeffery M. Klaiman	Jay W. Larson
Yanqi Li	John A. Mattingly	Cristopher D. Moen
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American Iron and Steel Institute
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TABLE OF CONTENTS

PART I
DIMENSIONS AND PROPERTIES

For Use With the
2012 Edition of the
North American Specification for the Design
of Cold-Formed Steel Structural Members

PART I - DIMENSIONS AND PROPERTIES

SECTION 1 - STEELS - AVAILABILITY AND PROPERTIES

1.1 Introduction to Table of Referenced Steels

The table of referenced steels is provided as a guide for material selection. It summarizes the scope of the corresponding ASTM Standards, identifies product classifications and lists important material properties.

Referenced Steels

There are 17 referenced ASTM Standards for steels that are accepted for use with the *North American Specification for the Design of Cold-Formed Steel Structural Members (Specification).* Use of these referenced steels is encouraged; however, other steels may also be used in cold-formed steel structures provided they satisfy the provisions of *Specification* Section A2.2.

Product Classifications

Of the referenced steels, 5 are for plate and bar, 1 is for plate, 1 is for sheet and strip, 8 are for sheet, and 2 are for tubing products.

ASTM classifies hot-rolled steel products as being either sheet, strip, plate or bar, based on size, as follows:

Table 1.1

Product Classification - Hot-Rolled Steel

Width, w in.	Thickness, t in.			
	$0.2300 \le t$	$0.203 \le t \le 0.230$	$0.1800 \le t \le 0.2030$	$0.044 \le t \le 0.180$
$w \le 3½$	bar	bar	strip	strip
$3½ < w \le 6$	bar	bar	strip	strip
$6 < w \le 8$	bar	strip	strip	strip
$8 < w \le 12$	plate (1)	strip	strip	strip
$12 < w \le 48$	plate (2)	sheet	sheet	sheet
$48 < w$	plate (2)	plate (2)	plate (2)	sheet

(1) Strip, only when ordered in coils.
(2) Sheet, only when ordered in coils.

ASTM classifies cold-rolled carbon and high-strength low-alloy (HSLA) sheet steel products, including hot-dip coated, based on size, as follows:

Table 1.2

Product Classification - Cold-Rolled Sheet Steel

Width, w in.	Thickness, t in.	
	Carbon Steel	HSLA Steel
$w \le 12$	$t \le 0.082$	$0.019 \le t \le 0.082$
$12 < w$	$t \le 0.142$	$0.020 \le t$

Structural Properties

The structural properties significant to cold-formed steel structures are listed in Table 1.3 of referenced steels; and include yield stress, tensile strength, elongation in 2 inches, and the ratio of tensile strength to yield stress. Total elongation in 2 inches is a measure of ductility, the ability of a steel to undergo sizable plastic or permanent strains before fracturing. The ratio of tensile strength to yield stress is an indication of the ability of the material to redistribute stress. As a convenience, the *Specification* lists the referenced steels in categories according to percent elongation.

1.2 Summary of Scope and Principal Tensile Properties, ASTM Specifications for Referenced Steels

Table 1.3

Summary of Scope and Principal Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A36/A36M-12 This specification covers carbon steel shapes, plates and bars for use in riveted, bolted, or welded construction of bridges and buildings, and for general structural purposes. Supplemental requirements are provided for use where additional testing or additional restrictions are required by the purchaser. Such requirements apply only when specified in the purchase order. When the steel is to be welded, a welding procedure suitable for the grade of steel and intended use or service is to be utilized.	Plates and Bars	--	36	58 / 80	23	1.61
A242/A242M-04(2009) This specification covers high-strength low-alloy structural steel shapes, plates and bars for welded, riveted, or bolted construction intended primarily for use as structural members where savings in weight or added durability are important. The atmospheric corrosion resistance of the steel in most environments is substantially better than that of carbon structural steels with or without copper addition. When properly exposed to the atmosphere, this steel can be used bare (unpainted) for many applications. This specification is limited to material up to 4 in. [100 mm], inclusive, in thickness. When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service is to be utilized.	Plates and Bars $t \le 3/4$ in.	--	50	70	21	1.40

Table 1.3

Summary of Scope and Principal Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A283/A283M-12 This specification covers two grades of carbon steel plates for general applications. When the steel is to be welded, a welding procedure suitable for the grade of steel and intended use or service is to be utilized.	Plate	C D	30 33	55 / 75 60 / 80	25 23	1.83 1.82
A500/A500M-10a This specification covers cold-formed welded and seamless carbon steel round, square, rectangular, or special shape structural tubing for welded, riveted, or bolted construction of bridges and buildings, and for general structural purposes. This tubing is produced in both welded and seamless sizes with a periphery of 88 in. [2235 mm] or less and a wall of 0.875 in. [22 mm] or less. Grade D requires heat treatments. Products manufactured to this specification may not be suitable for those applications such as dynamically loaded elements in welded structures, etc., where low-temperature notch-toughness properties may be important.	Round Tubing Shaped Tubing	A B C D A B C D	33 42 46 36 39 46 50 36	45 58 62 58 45 58 62 58	25 23 21 23 25 23 21 23	1.36 1.38 1.35 1.61 1.15 1.26 1.24 1.61
A529/A529M-05(2009) This specification covers carbon-manganese steel shapes, plates and bars for use in riveted, bolted, or welded construction of buildings and for general structural purposes. Material under this specification is available in two grades: Grade 50 for plates to 1 in. [25.4 mm] thick and to 15 in. [380 mm] wide, bars to 3 ½ in. [90 mm] thick, and shapes with flange or leg thickness to 1 ½ in. [40 mm]; Grade 55 for plates to 1 in. [25.4 mm] thick and to 15 in. [380 mm] wide, bars to 3 in. [75 mm] thick, and shapes with flange or leg thickness to 1 ½ in. [40 mm]. When the steel is to be welded, a welding procedure suitable for the grade of steel and intended use or service is to be utilized.	Plates and Bars	50 55	50 55	70 / 100 70 / 100	21 20	1.40 1.27

Table 1.3

Summary of Scope and Principal Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A572/A572M-12 This specification covers five grades of high-strength low-alloy structural steel shapes, plates, sheet piling, and bars. Grades 42 [290], 50 [345], and 55 [380] are intended for riveted, bolted, or welded construction. Grades 60 [415] and 65 [450] are intended for riveted or bolted construction of bridges or for riveted, bolted, or welded construction in other applications. For applications such as welded bridge construction, where notch toughness is important, notch toughness requirements are to be negotiated between the purchaser and the producer. When the steel is to be welded, a welding procedure suitable for the grade of steel and intended use or service is to be utilized.	Plates and Bars	42 50 55 60 65	42 50 55 60 65	60 65 70 75 80	24 21 20 18 17	1.43 1.30 1.27 1.25 1.23
A588/A588M-10 This specification covers high-strength low-alloy structural steel shapes, plates and bars for welded, riveted, or bolted construction but intended primarily for use in welded bridges and buildings where savings in weight or added durability are important. The atmospheric corrosion resistance of this steel in most environments is substantially better than that of carbon structural steels with or without copper addition. When properly exposed to the atmosphere this steel can be used bare (unpainted) for many applications. This specification is limited to material up to 8 in. [200 mm] inclusive in thickness. When the steel is to be welded, a welding procedure suitable for the grade of steel and intended use or service is to be utilized.	Plates and Bars t ≤ 4 in.	--	50	70	21	1.40

Table 1.3

Summary of Scope and Principal Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A606/A606M-09a This specification covers high-strength, low-alloy, hot- and cold-rolled sheet and strip in cut lengths or coils, intended for use in structural and miscellaneous purposes, where savings in weight or added durability are important. These steels have enhanced atmospheric corrosion resistance and are supplied in two types: Type 2 contains 0.20% minimum copper based on cast or heat analysis (0.18% minimum Cu for product check). Type 4 contains additional alloying elements and provides a level of corrosion resistance substantially better than that of carbon steels with or without copper addition. When properly exposed to the atmosphere, Type 4 can be used bare (unpainted) for many applications.	Sheet and Strip	Hot Rolled -As Rolled Cold Rolled	50 45	70 65	22 22	1.40 1.44

Table 1.3

Summary of Scope and Principal Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A653/A653M-11 This specification covers steel sheet, zinc coated (galvanized) or zinc-iron alloy coated (galvannealed) by the hot dip process in coils and cut lengths. Included are several grades based on yield stress in structural steel (SS grades 33 to 80), high-strength low-alloy (HSLAS grades 33 to 80), and high-strength low-alloy with improved formability (HSLA-F grades 40 to 80). Products furnished under A653/A653M must conform to the latest revision of A924/A924M except as otherwise indicated in the specification.	Sheet	SS				
		33	33	45	20	1.36
		37	37	52	18	1.41
		40	40	55	16	1.38
		50 Class 1	50	65	12	1.30
		50 Class 3	50	70	12	1.40
		50 Class 4	50	60	12	1.20
		55	55	70	11	1.27
		60	60	70	10	1.17
		70	70	80	9	1.14
		80 Class 1	80	82	--	1.03
		80 Class 2	80	82	--	1.03
		80 Class 3	80	82	3	1.03
		HSLAS				
		40	40	50	22	1.25
		50	50	60	20	1.20
		55 Class 1	55	70	16	1.27
		55 Class 2	55	65	18	1.18
		60	60	70	16	1.17
		70	70	80	12	1.14
		80	80	90	10	1.13
		HSLAS-F				
		40	40	50	24	1.25
		50	50	60	22	1.20
		55 Class 1	55	70	18	1.27
		55 Class 2	55	65	20	1.18
		60	60	70	18	1.17
		70	70	80	14	1.14
		80	80	90	12	1.13
A792/A792M-10 This specification covers 55% aluminum-zinc alloy-coated steel sheet in coils and cut lengths The product is intended for applications requiring corrosion resistance or heat resistance or both. The product is available as Commercial Steel (CS), Forming Steel (FS), Drawing Steel (DS), High Temperature Steel (HTS), and Structural Steel (SS).	Sheet	SS				
		33	33	45	20	1.36
		37	37	52	18	1.41
		40	40	55	16	1.38
		50 Class 1	50	65	12	1.30
		50 Class 4	50	60	12	1.20
		60	60	70	10	1.17
		70	70	80	9	1.14
		80 Class 1	80	82	--	1.03
		80 Class 2	80	82	--	1.03
		80 Class 3	80	82	3	1.03

Table 1.3

Summary of Scope and Principal Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A847/A847M-12 This specification covers cold-formed welded and seamless high-strength low-alloy round, square, rectangular, or special shaped structural tubing for welded, riveted, or bolted construction of bridges and buildings and for general structural purposes where high strength and enhanced atmospheric corrosion resistance are required. The atmospheric corrosion resistance of this steel in most environments is substantially better than carbon steel with or without copper addition. When properly exposed to the atmosphere, this steel can be used bare (unpainted) for many applications. When this steel is used in welded construction, the welding procedure shall be suitable for the steel and the intended service. This tubing is produced in welded sizes with a maximum periphery of 64 in. [1626 mm] and a maximum wall of 0.625 in. [15.9 mm], and in seamless with a maximum periphery of 32 in. [813 mm] and a maximum wall of 0.500 in. [12.7 mm]. Products manufactured under this specification may not be suitable for those applications where low-temperature notch-toughness properties may be important, such as dynamically loaded elements in welded structures.	Round and Shaped Tubing	--	50	70	19	1.40
A875/A875M-13 This specification covers steel sheet, in coils and cut lengths, metallic-coated by the hot-dip process, with zinc-5% aluminum alloy coating. The Zn-5Al alloy coating also contains small amounts of elements other than zinc and aluminum that are intended to improve processing and the characteristics of the coated product. The coating is produced as two types: zinc-5% aluminum-mischmetal alloy (Type I) and zinc-5% aluminum-0.1% magnesium alloy (Type II), and in two coating structures (classes). The coated sheet is produced in several coating designations (coating weight [mass]). The material is intended for applications requiring corrosion resistance, formability, and paintability. The steel sheet is produced in a number of designations, types, grades, and classes designed to be compatible with differing application requirements.	Sheet	SS				
		33	33	45	20	1.36
		37	37	52	18	1.41
		40	40	55	16	1.38
		50 Class 1	50	65	12	1.30
		50 Class 3	50	70	12	1.40
		80	80	82	--	1.03
		HSLAS				
		50	50	60	20	1.20
		60	60	70	16	1.17
		70	70	80	12	1.14
		80	80	90	10	1.13
		HSLAS-F				
		50	50	60	22	1.20
		60	60	70	18	1.17
		70	70	80	14	1.14
		80	80	90	12	1.13

Table 1.3

Summary of Scope and Principal Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A1003/A1003M-13a This specification covers coated steel sheet used in the manufacture of cold-formed framing members, such as, but not limited to, studs, joists, purlins, girts, and track. The sheet steel used for cold-formed framing members includes metallic coated, painted metallic coated, or painted nonmetallic coated. The grade designations use the following suffix indicators: H - high ductility, L - low ductility, and NS - nonstructural. H and L are associated with structural or load-bearing applications and NS with nonstructural or nonload-bearing applications. *Additionally, test values must show a minimum value of 1.08 for F_u/F_y.	Sheet	ST33H	33	45	10	1.36*
		ST37H	37	52	10	1.41*
		ST40H	40	55	10	1.38*
		ST50H	50	65	10	1.30*
		ST55H	55	70	10	1.27*
		ST60H	60	70	10	1.17*
		ST70H	70	80	10	1.14*
		ST80H	80	90	10	1.13*
		ST33L	33	--	3	--
		ST37L	37	--	3	--
		ST40L	40	--	3	--
		ST50L	50	--	3	--
		ST55L	55	--	3	--
		ST60L	60	--	3	--
		ST70L	70	--	3	--
		ST80L	80	--	3	--

Table 1.3

Summary of Scope and Principal Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A1008/A1008M-12a This specification covers cold-rolled carbon structural, high-strength low-alloy, and high-strength low-alloy with improved formability, solution hardened, and bake hardenable steel sheet, in coils and cut lengths. The product is produced in a number of designations including structural steel (SS grades 25 to 80), high-strength low-alloy steel (HSLAS grades 45 to 70), and high-strength low-alloy steel with improved formability (HSLAS-F grades 50 and 70). HSLAS-F steel has improved formability compared to HSLAS. The steel is fully deoxidized, made to fine grain practice, and includes micro-alloying elements such as columbium, vanadium, and zirconium. The steel shall be treated to achieve inclusion control. Cold-rolled steel sheet is supplied for either exposed or unexposed applications. Within the latter category, cold-rolled sheet is specified either "temper rolled" or "annealed last."	Sheet	SS				
		25	25	42	26	1.68
		30	30	45	24	1.50
		33 Type 1	33	48	22	1.45
		33 Type 2	33	48	22	1.45
		40 Type 1	40	52	20	1.30
		40 Type 2	40	52	20	1.30
		45	45	60	20	1.33
		50	50	65	18	1.30
		60	60	75	12	1.25
		70	70	85	6	1.21
		80	80	82	--	1.03
		HSLAS Class 1				
		45	45	60	22	1.33
		50	50	65	20	1.30
		55	55	70	18	1.27
		60	60	75	16	1.25
		65	65	80	15	1.23
		70	70	85	14	1.21
		HSLAS Class 2				
		45	45	55	22	1.22
		50	50	60	20	1.20
		55	55	65	18	1.18
		60	60	70	16	1.17
		65	65	75	15	1.15
		70	70	80	14	1.14
		HSLAS-F				
		50	50	60	22	1.20
		60	60	70	18	1.17
		70	70	80	16	1.14
		80	80	90	14	1.13

Table 1.3

Summary of Scope and Principal Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A1011/A1011M-12b						
This specification covers hot-rolled carbon structural, high-strength low-alloy, and high-strength low-alloy with improved formability, and ultra-high-strength steel sheet and strip, in coils and cut lengths. The product is produced in a number of designations including structural steel (SS), high-strength low-alloy steel (HSLAS), high-strength low-alloy steel with improved formability (HSLAS-F), and ultra-high-strength (UHSS).	Sheet	SS				
		30	30	49	25-21	1.63
		33	33	52	23-18	1.58
		36 Type 1	36	53	22-17	1.47
		36 Type 2	36	58/80	21-16	1.61
		40	40	55	21-15	1.38
		45 Type 1	45	60	19-14	1.33
		45 Type 2	45-60	60	20-14	1.33
		50	50	65	17-11	1.30
HSLAS-F steel has improved formability compared to HSLAS. The steel is fully deoxidized, made to fine grain practice, and includes microalloying elements such as columbium, vanadium, and zirconium. The steel is treated to achieve inclusion control.		55	55	70	15-9	1.27
		60	60	75	14-8	1.25
		70	70	85	13-7	1.21
		80	80	95	12-6	1.19
		HSLAS Class 1				
UHSS steel has increased strength compared with HSLA-F. The steel is killed and made to a fine ferritic grain practice and includes microalloying elements such as columbium, titanium, vanadium, and molybdenum to achieve inclusion control.		45	45	60	25-23	1.33
		50	50	65	22-20	1.30
		55	55	70	20-18	1.27
		60	60	75	18-16	1.25
		65	65	80	16-14	1.23
		70	70	85	14-12	1.21
		HSLAS Class 2				
		45	45	55	25-23	1.22
		50	50	60	22-20	1.20
		55	55	65	20-18	1.18
		60	60	70	18-16	1.17
		65	65	75	16-14	1.15
		70	70	80	14-12	1.14
		HSLAS-F				
		50	50	60	24-22	1.20
		60	60	70	22-20	1.17
		70	70	80	20-18	1.14
		80	80	90	18-16	1.13

Table 1.3

Summary of Scope and Principal Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A1039/A1039M-13						
This specification covers commercial and structural steel sheet in coils and cut lengths produced by the twin-roll casting process. Designations include: commercial steel (CS Types A, B and D, structural steel (SS grades 30 to 80) and high-strength low-alloy steel (HSLA grades 45 to 65). Mechanical properties are specified for SS and HSLA grades but are non-mandatory for CS grades.	Sheet	SS				
		30	30	49	24-21	1.63
		33	33	52	22-18	1.58
		36 Type 1	36	53	21-17	1.47
		36 Type 2	36	58/60	20-16	1.61
		40	40	55	20-15	1.38
		45	45	60	18-13	1.33
		50	50	65	16-11	1.30
		55	55	70	14-9	1.27
		60	60	70	13-8	1.17
		70	70	80	12-7	1.14
		80	80	90	11-6	1.13
		HSLAS Class 1				
		45	45	60	18	1.33
		50	50	65	15	1.30
		55	55	70	13	1.27
		60	60	75	11	1.25
		65	65	80	11	1.23
		70	70	85	8	1.21
		80	80	90	7	1.13
		HSLAS Class 2				
		45	45	55	18	1.22
		50	50	60	15	1.20
		55	55	65	13	1.18
		60	60	70	11	1.17
		65	65	75	11	1.15
		70	70	80	8	1.14
		80	80	90	7	1.13

Table 1.3
Summary of Scope and Principal Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A1063/A1063M-11a This specification covers steel sheet in coils and cut lengths produced by the twin-roll casting process. Designations are as follows: Commercial steel (CS Types A, B, and C), structural steel (SS grades 33 to 80), and high-strength low-alloy steel (HSLAS grades 45 to 80). Mechanical properties are specified for SS and HSLA grades but are non-mandatory for CS grades.	Sheet	SS				
		33	33	45	20	1.36
		37	37	52	18	1.41
		40	40	55	15	1.38
		45	45	60	13	1.33
		50	50	65	11	1.30
		55	55	70	9	1.27
		60	60	70	8	1.17
		70	70	80	7	1.14
		80 Class 1	80	90	6	1.13
		80 Class 2	80	83	--	1.04
		HSLAS Class 1				
		45	45	60	18	1.33
		50	50	65	15	1.30
		55	55	70	13	1.27
		60	60	75	11	1.25
		65	65	80	11	1.23
		70	70	85	8	1.21
		80	80	95	7	1.19
		HSLAS Class 2				
		45	45	55	18	1.22
		50	50	60	15	1.20
		55	55	65	13	1.18
		60	60	70	11	1.17
		65	65	75	11	1.15
		70	70	80	8	1.14
		80	80	90	7	1.13

1.3 Material Thickness

Historically, sheet and strip steels have been ordered from the steel producer using one of the following systems to specify thickness:

Minimum Thickness: When ordered to a minimum thickness, all thickness tolerances are over (+) and nothing under (-). Steel is generally ordered to a minimum thickness when the design is based on minimum strength requirements that depend on having a guaranteed minimum thickness for the sheet product.

Nominal Thickness: When ordered to a nominal thickness, thickness tolerances are equally divided between over (+) and under (-). Steel is generally ordered to a nominal thickness when the equipment to be used to process the material is designed for a certain thickness.

Gauge (Gage) Thickness: Gauge thickness is an obsolete method of specifying sheet and strip steel thickness. Gauge numbers are only a very rough approximation of steel thickness and should not be used to order, design or specify any sheet or strip steel product.

Hot-dip coated sheet products are typically specified by total product thickness, including the coating. The relevant ASTM specifications for the various coated sheet products include values for the thickness of the coating itself.

Design Thickness

The steel thickness used in design should be the thickness of the uncoated base steel sheet or strip. Coatings such as paint or zinc add little or no structural strength and should not be included in the design thickness.

Delivered Minimum Thickness

Since there are tolerances in either of the two acceptable methods of ordering sheet and strip steel thickness, it would be unreasonable to expect the delivered minimum thickness of a cold-formed steel product to exactly match the design thickness. *Specification* provisions cover minor negative thickness tolerances. Thus, 95 percent of the design thickness has been set as the minimum delivered thickness of a cold-formed steel product.

If the delivered minimum thickness is less than 95 percent of the design thickness, an analysis should be performed to determine if the delivered product is adequate to meet its intended purpose. Generally, thickness measurements may be made anywhere across the width of the sheet, but not closer to the edges than the minimum distances specified in the relevant ASTM specifications. Thickness at bends, such as corners, may be less than 95 percent of design thickness, due to cold-forming effects, and still be acceptable.

SECTION 2 - REPRESENTATIVE COLD-FORMED STEEL SECTIONS

2.1 Representative Versus Actual Sections

The cross-sections defined in Tables I-1 to I-8 are intended to be representative of some of the sections in use by, or available from, manufacturers and fabricators. The specific sections listed in these tables are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members. Although these sections are useful for preliminary design, designers should consult the literature of cold-formed product providers for actual section property information when specifying cold-formed products.

Standard joists/studs and track sections are identified using the product designator given in *AISI S201-12 North American Standard for Cold-Formed Steel Framing – Product Data*. Sections other than joists/studs and tracks are identified by a convention given in Section 2.1.2 below, developed by AISI for use in this *Manual*.

2.1.1 Joist/Stud and Track Section Nomenclature

Joists/studs and tracks are identified in this *Manual* by the codes described below. The standard nomenclature is formed by concatenating the following information:

1. Depth in 1/100th inch. For joists/studs, the depth is the outside depth. For tracks, the depth is the inside depth (the depth of the stud the track fits over).
2. Style: S = Joist/Stud (C-Section With Lips), T = Track (C-Section Without Lips)
3. Flange Width in 1/100th inch
4. "-"
5. Minimum base material thickness (95% of design thickness) in 1/1000th inch

For example, a section with the designation 600S162-54 is a joist/stud (C-Section With Lips), with a depth of 6 inches, a flange width of 1 ⅝ inches and a minimum thickness of 0.054 inch. Other details, such as bend radii and lip lengths are found in Tables I-2 and I-3.

This naming convention is an industry standard of the cold-formed steel framing industry.

2.1.2 Other Section Nomenclature

The naming convention used for the other representative sections was developed only to simplify the charts, tables and example problems throughout the *Manual*. The section names are formed by concatenating the following information:

1. Depth in inches
2. Section Profile: C = C-Section, Z = Z-Section, L = Equal Leg Angle, H = Hat Section
3. Code for Stiffened or Unstiffened Flanges: S = Stiffened, U = Unstiffened
4. Flange Width in inches
5. "x"
6. Thickness in 1/1000th inch

For example, a section with the designation 9CS3x075 is a C-Section With Lips, with a depth of 9 inches, a flange width of 3 inches and a thickness of 0.075 inch. Other details, such as bend radii and lip lengths are found in Tables I-1 and I-4 through I-8.

This naming convention is not an industry standard. Individual manufacturers and industry groups have adopted their own systems, and these systems should be used when specifying actual products.

2.2 Notes on Tables

(a) Tabulated section properties are shown to three significant figures, while dimensions are given to three decimal places. However, in some cases space limitations made it impractical to adhere strictly to this guideline.

(b) The weight of these sections is calculated based on a steel weight of 40.8 pounds per square foot per inch thickness.

(c) Where they apply, the algebraic formulae presented in Section 3 of Part I formed the basis of the calculations for these tables.

(d) Tables I-1 to I-8 inclusive are Gross Section Property Tables. Effective section properties can be found in Parts II and III for beams and columns, respectively.

(e) In Table I-8, the orientation of the x-axis is vertical to be consistent with the provisions of *Specification* Section C3.1.2.1 which defines the x-axis as the axis of symmetry for singly-symmetric sections.

(f) Section dimensions are defined in the figures provided in each table. Section properties are defined in the *Specification*, Symbols and Definitions.

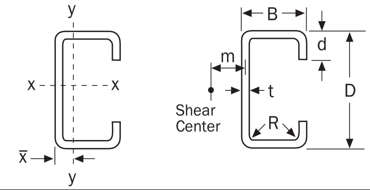
(g) Sections denoted with * have webs with h/t > 200 and may require stiffeners depending on application.

2.3 Gross Section Property Tables

Table I - 1

Gross Section Properties

C-Sections With Lips

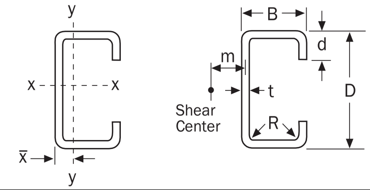


ID	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
12CS4x105	12.000	4.000	0.105	0.885	0.1875	2.20	7.48	47.5	7.91	4.65	4.27	1.45	1.39	1.05	1.64	0.00808	122	6.74	5.52	-2.63
12CS4x085	12.000	4.000	0.085	0.836	0.1875	1.78	6.05	38.6	6.43	4.66	3.45	1.17	1.39	1.03	1.63	0.00429	98.5	6.76	5.52	-2.62
12CS4x070	12.000	4.000	0.070	0.800	0.1875	1.47	4.98	31.9	5.31	4.66	2.84	0.955	1.39	1.02	1.63	0.00239	80.8	6.77	5.52	-2.62
12CS3.5x105	12.000	3.500	0.105	0.885	0.1875	2.09	7.12	43.8	7.29	4.57	3.07	1.17	1.21	0.879	1.41	0.00769	89.1	6.81	5.23	-2.23
12CS3.5x085	12.000	3.500	0.085	0.836	0.1875	1.69	5.76	35.6	5.93	4.58	2.48	0.942	1.21	0.865	1.40	0.00408	71.7	6.83	5.23	-2.22
12CS3.5x070	12.000	3.500	0.070	0.800	0.1875	1.40	4.75	29.4	4.90	4.59	2.04	0.773	1.21	0.855	1.40	0.00228	58.8	6.85	5.24	-2.22
12CS2.5x105	12.000	2.500	0.105	0.885	0.1875	1.88	6.40	36.3	6.05	4.39	1.34	0.692	0.843	0.567	0.951	0.00692	40.4	7.56	4.71	-1.47
12CS2.5x085	12.000	2.500	0.085	0.836	0.1875	1.52	5.18	29.5	4.92	4.40	1.08	0.557	0.843	0.555	0.947	0.00367	32.6	7.59	4.71	-1.46
12CS2.5x070	12.000	2.500	0.070	0.800	0.1875	1.26	4.27	24.4	4.06	4.41	0.893	0.457	0.844	0.546	0.943	0.00205	26.8	7.61	4.72	-1.45
10CS4x105	10.000	4.000	0.105	0.885	0.1875	1.99	6.76	31.0	6.20	3.95	4.04	1.42	1.43	1.15	1.73	0.00731	81.7	5.70	5.06	-2.83
10CS4x085	10.000	4.000	0.085	0.836	0.1875	1.61	5.47	25.2	5.05	3.96	3.27	1.14	1.43	1.14	1.72	0.00388	65.7	5.72	5.07	-2.82
10CS4x070	10.000	4.000	0.070	0.800	0.1875	1.33	4.51	20.9	4.17	3.97	2.69	0.937	1.43	1.13	1.72	0.00217	53.8	5.73	5.07	-2.81
10CS4x065	10.000	4.000	0.065	0.788	0.1875	1.23	4.19	19.4	3.88	3.97	2.50	0.869	1.43	1.13	1.71	0.00173	49.9	5.73	5.07	-2.81
10CS3.5x105	10.000	3.500	0.105	0.885	0.1875	1.88	6.40	28.5	5.69	3.89	2.91	1.15	1.24	0.971	1.49	0.00692	59.5	5.59	4.74	-2.41
10CS3.5x085	10.000	3.500	0.085	0.836	0.1875	1.52	5.18	23.1	4.63	3.90	2.35	0.926	1.24	0.957	1.48	0.00367	47.8	5.61	4.74	-2.40
10CS3.5x070	10.000	3.500	0.070	0.800	0.1875	1.26	4.27	19.1	3.83	3.90	1.94	0.759	1.24	0.947	1.48	0.00205	39.2	5.63	4.74	-2.39
10CS3.5x065	10.000	3.500	0.065	0.788	0.1875	1.17	3.96	17.8	3.56	3.91	1.80	0.704	1.24	0.943	1.48	0.00164	36.3	5.63	4.74	-2.39
10CS2.5x105	10.000	2.500	0.105	0.885	0.1875	1.67	5.69	23.3	4.66	3.73	1.28	0.683	0.873	0.632	1.02	0.00615	27.0	5.78	4.15	-1.60
10CS2.5x085	10.000	2.500	0.085	0.836	0.1875	1.35	4.61	19.0	3.79	3.74	1.03	0.550	0.874	0.619	1.02	0.00326	21.8	5.81	4.16	-1.59
10CS2.5x070	10.000	2.500	0.070	0.800	0.1875	1.12	3.79	15.7	3.14	3.75	0.852	0.451	0.874	0.610	1.01	0.00182	17.9	5.83	4.16	-1.59
10CS2.5x065	10.000	2.500	0.065	0.788	0.1875	1.04	3.52	14.6	2.92	3.75	0.792	0.418	0.874	0.607	1.01	0.00146	16.6	5.84	4.17	-1.59
10CS2x105	10.000	2.000	0.105	0.885	0.1875	1.57	5.33	20.7	4.15	3.64	0.739	0.486	0.687	0.478	0.791	0.00576	16.1	6.28	3.90	-1.22
10CS2x085	10.000	2.000	0.085	0.836	0.1875	1.27	4.32	16.9	3.38	3.65	0.601	0.392	0.688	0.466	0.788	0.00306	13.0	6.32	3.90	-1.21
10CS2x070	10.000	2.000	0.070	0.800	0.1875	1.05	3.56	14.0	2.79	3.65	0.496	0.322	0.689	0.457	0.785	0.00171	10.7	6.34	3.91	-1.21
10CS2x065	10.000	2.000	0.065	0.788	0.1875	0.971	3.30	13.0	2.60	3.66	0.461	0.298	0.689	0.454	0.784	0.00137	9.91	6.35	3.91	-1.21

Table I - 1

Gross Section Properties

C-Sections With Lips

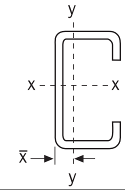


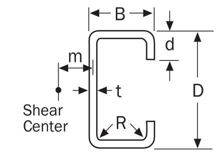
ID	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
9CS2.5x105	9.000	2.500	0.105	0.885	0.1875	1.57	5.33	18.1	4.02	3.40	1.24	0.676	0.888	0.670	1.06	0.00576	21.5	5.03	3.89	-1.68
9CS2.5x085	9.000	2.500	0.085	0.836	0.1875	1.27	4.32	14.7	3.27	3.41	1.00	0.545	0.889	0.658	1.06	0.00306	17.3	5.06	3.90	-1.67
9CS2.5x070	9.000	2.500	0.070	0.800	0.1875	1.05	3.56	12.2	2.71	3.41	0.828	0.447	0.890	0.648	1.05	0.00171	14.2	5.09	3.90	-1.66
9CS2.5x065	9.000	2.500	0.065	0.788	0.1875	0.971	3.30	11.3	2.52	3.42	0.769	0.415	0.890	0.645	1.05	0.00137	13.1	5.09	3.90	-1.66
9CS2.5x059	9.000	2.500	0.059	0.773	0.1875	0.881	3.00	10.3	2.29	3.42	0.698	0.376	0.890	0.641	1.05	0.00102	11.9	5.10	3.90	-1.66
8CS4x105	8.000	4.000	0.105	0.885	0.1875	1.78	6.05	18.6	4.64	3.23	3.76	1.38	1.45	1.28	1.83	0.00654	50.4	4.92	4.68	-3.06
8CS4x085	8.000	4.000	0.085	0.836	0.1875	1.44	4.90	15.1	3.78	3.24	3.04	1.11	1.45	1.27	1.83	0.00347	40.4	4.93	4.68	-3.05
8CS4x070	8.000	4.000	0.070	0.800	0.1875	1.19	4.03	12.5	3.13	3.25	2.50	0.913	1.45	1.26	1.82	0.00194	33.1	4.94	4.68	-3.04
8CS4x065	8.000	4.000	0.065	0.788	0.1875	1.10	3.74	11.6	2.91	3.25	2.33	0.847	1.45	1.25	1.82	0.00155	30.6	4.94	4.68	-3.04
8CS4x059	8.000	4.000	0.059	0.773	0.1875	0.999	3.40	10.6	2.65	3.25	2.11	0.767	1.45	1.25	1.81	0.00116	27.7	4.95	4.68	-3.03
8CS3.5x105	8.000	3.500	0.105	0.885	0.1875	1.67	5.69	16.9	4.23	3.18	2.71	1.12	1.27	1.09	1.59	0.00615	36.7	4.66	4.31	-2.62
8CS3.5x085	8.000	3.500	0.085	0.836	0.1875	1.35	4.61	13.8	3.45	3.19	2.19	0.904	1.27	1.07	1.58	0.00326	29.5	4.68	4.31	-2.61
8CS3.5x070	8.000	3.500	0.070	0.800	0.1875	1.12	3.79	11.4	2.85	3.20	1.81	0.741	1.27	1.06	1.58	0.00182	24.1	4.69	4.32	-2.60
8CS3.5x065	8.000	3.500	0.065	0.788	0.1875	1.04	3.52	10.6	2.65	3.20	1.68	0.687	1.27	1.06	1.57	0.00146	22.3	4.69	4.32	-2.60
8CS3.5x059	8.000	3.500	0.059	0.773	0.1875	0.940	3.20	9.65	2.41	3.20	1.52	0.623	1.27	1.05	1.57	0.00109	20.2	4.70	4.32	-2.60
8CS2.5x105	8.000	2.500	0.105	0.885	0.1875	1.46	4.98	13.6	3.41	3.05	1.19	0.669	0.903	0.715	1.10	0.00538	16.7	4.39	3.64	-1.77
8CS2.5x085	8.000	2.500	0.085	0.836	0.1875	1.18	4.03	11.1	2.78	3.06	0.969	0.539	0.904	0.702	1.10	0.00285	13.4	4.42	3.65	-1.76
8CS2.5x070	8.000	2.500	0.070	0.800	0.1875	0.976	3.32	9.21	2.30	3.07	0.800	0.442	0.905	0.692	1.09	0.00159	11.0	4.44	3.65	-1.75
8CS2.5x065	8.000	2.500	0.065	0.788	0.1875	0.906	3.08	8.57	2.14	3.08	0.743	0.410	0.905	0.689	1.09	0.00128	10.2	4.44	3.65	-1.75
8CS2.5x059	8.000	2.500	0.059	0.773	0.1875	0.822	2.80	7.79	1.95	3.08	0.674	0.372	0.906	0.685	1.09	0.000954	9.22	4.45	3.65	-1.75
8CS2x105	8.000	2.000	0.105	0.885	0.1875	1.36	4.62	12.0	3.00	2.97	0.696	0.478	0.716	0.544	0.863	0.00499	9.95	4.50	3.35	-1.36
8CS2x085	8.000	2.000	0.085	0.836	0.1875	1.10	3.74	9.79	2.45	2.98	0.566	0.385	0.717	0.532	0.859	0.00265	8.01	4.53	3.35	-1.35
8CS2x070	8.000	2.000	0.070	0.800	0.1875	0.906	3.08	8.11	2.03	2.99	0.467	0.316	0.718	0.523	0.856	0.00148	6.58	4.56	3.36	-1.34
8CS2x065	8.000	2.000	0.065	0.788	0.1875	0.841	2.86	7.54	1.89	3.00	0.435	0.294	0.719	0.520	0.855	0.00118	6.10	4.57	3.36	-1.34
8CS2x059	8.000	2.000	0.059	0.773	0.1875	0.763	2.60	6.86	1.72	3.00	0.395	0.266	0.719	0.516	0.854	0.000886	5.53	4.58	3.36	-1.34
7CS4x105	7.000	4.000	0.105	0.885	0.1875	1.67	5.69	13.7	3.91	2.86	3.59	1.36	1.46	1.36	1.89	0.00615	38.1	4.63	4.53	-3.20
7CS4x085	7.000	4.000	0.085	0.836	0.1875	1.35	4.61	11.2	3.19	2.87	2.91	1.09	1.47	1.35	1.88	0.00326	30.5	4.64	4.53	-3.19
7CS4x070	7.000	4.000	0.070	0.800	0.1875	1.12	3.79	9.24	2.64	2.88	2.39	0.898	1.47	1.33	1.88	0.00182	24.9	4.64	4.53	-3.18
7CS4x065	7.000	4.000	0.065	0.788	0.1875	1.04	3.52	8.60	2.46	2.88	2.22	0.833	1.47	1.33	1.87	0.00146	23.1	4.65	4.53	-3.17
7CS4x059	7.000	4.000	0.059	0.773	0.1875	0.940	3.20	7.82	2.23	2.88	2.02	0.754	1.46	1.33	1.87	0.00109	20.9	4.65	4.53	-3.17

Table I - 1

Gross Section Properties

C-Sections With Lips





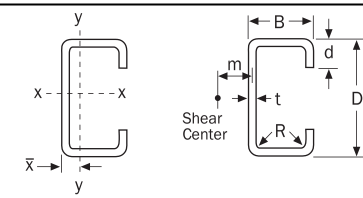
ID	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
7CS2.5x105	7.000	2.500	0.105	0.885	0.1875	1.36	4.62	9.94	2.84	2.71	1.15	0.660	0.918	0.766	1.15	0.00499	12.6	3.85	3.41	-1.86
7CS2.5x085	7.000	2.500	0.085	0.836	0.1875	1.10	3.74	8.11	2.32	2.72	0.929	0.532	0.919	0.753	1.15	0.00265	10.1	3.87	3.42	-1.86
7CS2.5x070	7.000	2.500	0.070	0.800	0.1875	0.906	3.08	6.72	1.92	2.72	0.767	0.437	0.920	0.743	1.14	0.00148	8.27	3.89	3.42	-1.85
7CS2.5x065	7.000	2.500	0.065	0.788	0.1875	0.841	2.86	6.25	1.79	2.73	0.713	0.405	0.921	0.740	1.14	0.00118	7.66	3.90	3.42	-1.85
7CS2.5x059	7.000	2.500	0.059	0.773	0.1875	0.763	2.60	5.69	1.63	2.73	0.647	0.367	0.921	0.735	1.14	0.000886	6.93	3.90	3.42	-1.84
6CS4x105	6.000	4.000	0.105	0.885	0.1875	1.57	5.33	9.64	3.21	2.48	3.40	1.33	1.47	1.45	1.95	0.00576	27.8	4.40	4.42	-3.35
6CS4x085	6.000	4.000	0.085	0.836	0.1875	1.27	4.32	7.88	2.63	2.49	2.75	1.07	1.47	1.43	1.94	0.00306	22.2	4.41	4.42	-3.34
6CS4x070	6.000	4.000	0.070	0.800	0.1875	1.05	3.56	6.53	2.18	2.50	2.27	0.879	1.47	1.42	1.94	0.00171	18.1	4.41	4.41	-3.32
6CS4x065	6.000	4.000	0.065	0.788	0.1875	0.971	3.30	6.08	2.03	2.50	2.11	0.815	1.47	1.42	1.94	0.00137	16.7	4.41	4.41	-3.32
6CS4x059	6.000	4.000	0.059	0.773	0.1875	0.881	3.00	5.53	1.84	2.51	1.91	0.739	1.47	1.41	1.93	0.00102	15.1	4.42	4.41	-3.32
6CS2.5x105	6.000	2.500	0.105	0.885	0.1875	1.25	4.26	6.91	2.30	2.35	1.09	0.649	0.931	0.826	1.20	0.00461	9.20	3.40	3.21	-1.98
6CS2.5x085	6.000	2.500	0.085	0.836	0.1875	1.01	3.45	5.65	1.88	2.36	0.883	0.523	0.933	0.812	1.20	0.00244	7.36	3.43	3.21	-1.97
6CS2.5x070	6.000	2.500	0.070	0.800	0.1875	0.836	2.84	4.69	1.56	2.37	0.729	0.429	0.934	0.802	1.19	0.00136	6.01	3.44	3.21	-1.96
6CS2.5x065	6.000	2.500	0.065	0.788	0.1875	0.776	2.64	4.36	1.45	2.37	0.677	0.398	0.934	0.799	1.19	0.00109	5.56	3.45	3.21	-1.96
6CS2.5x059	6.000	2.500	0.059	0.773	0.1875	0.704	2.40	3.97	1.32	2.37	0.615	0.361	0.935	0.795	1.19	0.000817	5.03	3.45	3.21	-1.95
4CS4x105	4.000	4.000	0.105	0.885	0.1875	1.36	4.62	3.87	1.93	1.69	2.92	1.25	1.47	1.66	2.10	0.00499	12.9	4.15	4.33	-3.71
4CS4x085	4.000	4.000	0.085	0.836	0.1875	1.10	3.74	3.18	1.59	1.70	2.37	1.01	1.47	1.65	2.09	0.00265	10.2	4.16	4.32	-3.69
4CS4x070	4.000	4.000	0.070	0.800	0.1875	0.906	3.08	2.64	1.32	1.71	1.96	0.828	1.47	1.64	2.08	0.00148	8.25	4.16	4.31	-3.68
4CS4x065	4.000	4.000	0.065	0.788	0.1875	0.841	2.86	2.46	1.23	1.71	1.82	0.767	1.47	1.63	2.08	0.00118	7.62	4.16	4.31	-3.67
4CS4x059	4.000	4.000	0.059	0.773	0.1875	0.763	2.60	2.25	1.12	1.72	1.65	0.695	1.47	1.63	2.07	0.000886	6.87	4.16	4.31	-3.67
4CS2.5x105	4.000	2.500	0.105	0.885	0.1875	1.04	3.55	2.67	1.34	1.60	0.936	0.617	0.947	0.981	1.33	0.00383	4.30	2.83	2.92	-2.26
4CS2.5x085	4.000	2.500	0.085	0.836	0.1875	0.845	2.87	2.20	1.10	1.61	0.762	0.497	0.950	0.967	1.32	0.00203	3.40	2.84	2.92	-2.24
4CS2.5x070	4.000	2.500	0.070	0.800	0.1875	0.696	2.37	1.83	0.917	1.62	0.630	0.408	0.952	0.957	1.31	0.00114	2.75	2.85	2.92	-2.24
4CS2.5x065	4.000	2.500	0.065	0.788	0.1875	0.646	2.20	1.71	0.855	1.63	0.586	0.379	0.952	0.953	1.31	0.000910	2.54	2.86	2.92	-2.23
4CS2.5x059	4.000	2.500	0.059	0.773	0.1875	0.586	1.99	1.56	0.780	1.63	0.532	0.343	0.953	0.949	1.31	0.000680	2.29	2.86	2.92	-2.23
4CS2x105	4.000	2.000	0.105	0.979	0.1875	0.958	3.26	2.30	1.15	1.55	0.576	0.475	0.775	0.788	1.10	0.00352	2.91	2.42	2.53	-1.84
4CS2x085	4.000	2.000	0.085	0.930	0.1875	0.776	2.64	1.89	0.947	1.56	0.471	0.384	0.779	0.775	1.10	0.00187	2.31	2.44	2.53	-1.83
4CS2x070	4.000	2.000	0.070	0.894	0.1875	0.639	2.17	1.58	0.791	1.57	0.390	0.316	0.782	0.766	1.09	0.00104	1.87	2.46	2.53	-1.82
4CS2x065	4.000	2.000	0.065	0.881	0.1875	0.593	2.02	1.47	0.737	1.58	0.363	0.294	0.782	0.763	1.09	0.000835	1.73	2.46	2.53	-1.82
4CS2x059	4.000	2.000	0.059	0.867	0.1875	0.538	1.83	1.35	0.673	1.58	0.331	0.266	0.784	0.759	1.09	0.000625	1.56	2.47	2.54	-1.82

Table I - 2

Gross Section Properties

Joists/Studs

C-Sections With Lips



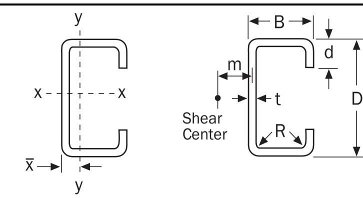
ID	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
1200S250-97	12.000	2.500	0.1017	0.625	0.1526	1.78	6.05	34.0	5.67	4.37	1.12	0.565	0.794	0.513	0.867	0.00613	32.7	7.93	4.64	-1.33
1200S250-68	12.000	2.500	0.0713	0.625	0.1070	1.26	4.30	24.5	4.08	4.40	0.836	0.421	0.813	0.513	0.884	0.00214	24.0	7.69	4.68	-1.36
1200S250-54*	12.000	2.500	0.0566	0.625	0.0849	1.01	3.43	19.7	3.28	4.42	0.683	0.344	0.823	0.513	0.892	0.00108	19.5	7.58	4.70	-1.38
1200S200-97	12.000	2.000	0.1017	0.625	0.1526	1.68	5.70	30.4	5.07	4.26	0.635	0.392	0.615	0.381	0.656	0.00578	19.1	9.11	4.41	-0.987
1200S200-68	12.000	2.000	0.0713	0.625	0.1070	1.19	4.05	21.9	3.66	4.29	0.479	0.296	0.634	0.380	0.673	0.00202	14.2	8.74	4.46	-1.02
1200S200-54*	12.000	2.000	0.0566	0.625	0.0849	0.953	3.24	17.7	2.94	4.31	0.394	0.243	0.643	0.379	0.681	0.00102	11.6	8.58	4.47	-1.03
1200S162-97	12.000	1.625	0.1017	0.500	0.1526	1.58	5.36	27.0	4.49	4.14	0.332	0.245	0.459	0.272	0.470	0.00543	10.3	11.3	4.22	-0.691
1200S162-68	12.000	1.625	0.0713	0.500	0.1070	1.12	3.81	19.5	3.25	4.17	0.255	0.188	0.477	0.269	0.485	0.00190	7.74	10.6	4.26	-0.719
1200S162-54*	12.000	1.625	0.0566	0.500	0.0849	0.896	3.05	15.7	2.62	4.19	0.212	0.156	0.486	0.268	0.493	0.000957	6.34	10.3	4.28	-0.732
1000S250-97	10.000	2.500	0.1017	0.625	0.1526	1.58	5.36	21.8	4.37	3.72	1.07	0.557	0.825	0.573	0.932	0.00543	21.6	6.03	4.08	-1.45
1000S250-68	10.000	2.500	0.0713	0.625	0.1070	1.12	3.81	15.8	3.15	3.75	0.799	0.415	0.844	0.574	0.950	0.00190	15.9	5.88	4.12	-1.49
1000S250-54	10.000	2.500	0.0566	0.625	0.0849	0.896	3.05	12.7	2.54	3.76	0.653	0.339	0.854	0.575	0.958	0.000957	12.9	5.81	4.14	-1.51
1000S250-43*	10.000	2.500	0.0451	0.625	0.0712	0.718	2.44	10.2	2.04	3.77	0.531	0.276	0.861	0.575	0.965	0.000486	10.5	5.76	4.16	-1.52
1000S200-97	10.000	2.000	0.1017	0.625	0.1526	1.47	5.01	19.3	3.87	3.62	0.610	0.388	0.643	0.427	0.711	0.00508	12.7	6.68	3.84	-1.09
1000S200-68	10.000	2.000	0.0713	0.625	0.1070	1.05	3.57	14.0	2.80	3.65	0.460	0.292	0.662	0.427	0.729	0.00178	9.40	6.44	3.88	-1.12
1000S200-54	10.000	2.000	0.0566	0.625	0.0849	0.839	2.85	11.3	2.26	3.67	0.378	0.240	0.671	0.427	0.737	0.000896	7.67	6.33	3.90	-1.14
1000S200-43*	10.000	2.000	0.0451	0.625	0.0712	0.672	2.29	9.09	1.82	3.68	0.309	0.196	0.677	0.426	0.743	0.000456	6.24	6.26	3.91	-1.15
1000S162-97	10.000	1.625	0.1017	0.500	0.1526	1.37	4.67	17.0	3.39	3.52	0.320	0.243	0.483	0.305	0.514	0.00473	6.83	8.05	3.63	-0.768
1000S162-68	10.000	1.625	0.0713	0.500	0.1070	0.978	3.33	12.3	2.47	3.55	0.247	0.187	0.502	0.303	0.531	0.00166	5.12	7.62	3.67	-0.798
1000S162-54	10.000	1.625	0.0566	0.500	0.0849	0.783	2.66	9.95	1.99	3.57	0.204	0.155	0.511	0.302	0.538	0.000836	4.20	7.43	3.69	-0.812
1000S162-43*	10.000	1.625	0.0451	0.500	0.0712	0.627	2.13	8.03	1.61	3.58	0.168	0.127	0.518	0.301	0.545	0.000425	3.43	7.31	3.71	-0.823
800S250-97	8.000	2.500	0.1017	0.625	0.1526	1.37	4.67	12.8	3.20	3.05	1.01	0.546	0.858	0.650	1.01	0.00473	13.1	4.54	3.56	-1.61
800S250-68	8.000	2.500	0.0713	0.625	0.1070	0.978	3.33	9.26	2.32	3.08	0.752	0.407	0.877	0.653	1.03	0.00166	9.65	4.45	3.60	-1.64
800S250-54	8.000	2.500	0.0566	0.625	0.0849	0.783	2.66	7.47	1.87	3.09	0.614	0.333	0.886	0.654	1.04	0.000836	7.85	4.42	3.62	-1.66
800S250-43	8.000	2.500	0.0451	0.625	0.0712	0.627	2.13	6.02	1.50	3.10	0.500	0.271	0.893	0.654	1.04	0.000425	6.37	4.39	3.63	-1.68

Table I - 2

Gross Section Properties

Joists/Studs

C-Sections With Lips



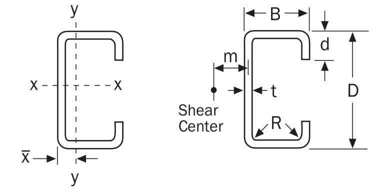
ID	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
800S200-97	8.000	2.000	0.1017	0.625	0.1526	1.27	4.32	11.2	2.80	2.97	0.577	0.381	0.674	0.487	0.777	0.00438	7.68	4.76	3.28	-1.21
800S200-68	8.000	2.000	0.0713	0.625	0.1070	0.907	3.08	8.14	2.04	3.00	0.435	0.288	0.692	0.488	0.796	0.00154	5.71	4.62	3.32	-1.25
800S200-54	8.000	2.000	0.0566	0.625	0.0849	0.726	2.47	6.57	1.64	3.01	0.357	0.236	0.701	0.489	0.804	0.000775	4.66	4.56	3.34	-1.27
800S200-43	8.000	2.000	0.0451	0.625	0.0712	0.582	1.98	5.30	1.33	3.02	0.292	0.193	0.708	0.489	0.811	0.000395	3.80	4.52	3.35	-1.28
800S200-33*	8.000	2.000	0.0346	0.625	0.0765	0.448	1.52	4.10	1.02	3.02	0.227	0.150	0.712	0.488	0.817	0.000179	2.97	4.52	3.36	-1.29
800S162-97	8.000	1.625	0.1017	0.500	0.1526	1.17	3.97	9.71	2.43	2.88	0.305	0.239	0.511	0.349	0.568	0.00403	4.11	5.48	3.05	-0.866
800S162-68	8.000	1.625	0.0713	0.500	0.1070	0.836	2.84	7.09	1.77	2.91	0.235	0.184	0.530	0.349	0.586	0.00142	3.09	5.22	3.09	-0.899
800S162-54	8.000	1.625	0.0566	0.500	0.0849	0.670	2.28	5.74	1.43	2.93	0.195	0.152	0.539	0.348	0.594	0.000715	2.54	5.11	3.11	-0.914
800S162-43	8.000	1.625	0.0451	0.500	0.0712	0.537	1.83	4.63	1.16	2.94	0.160	0.125	0.546	0.348	0.601	0.000364	2.08	5.04	3.13	-0.926
800S162-33*	8.000	1.625	0.0346	0.500	0.0765	0.414	1.41	3.58	0.896	2.94	0.125	0.0980	0.550	0.347	0.607	0.000165	1.63	5.03	3.14	-0.936
800S137-97	8.000	1.375	0.1017	0.375	0.1526	1.09	3.72	8.60	2.15	2.81	0.170	0.152	0.394	0.258	0.423	0.00377	2.35	6.57	2.90	-0.630
800S137-68	8.000	1.375	0.0713	0.375	0.1070	0.782	2.66	6.30	1.58	2.84	0.134	0.120	0.414	0.257	0.440	0.00133	1.79	6.12	2.94	-0.661
800S137-54	8.000	1.375	0.0566	0.375	0.0849	0.627	2.13	5.11	1.28	2.86	0.112	0.100	0.423	0.256	0.448	0.000670	1.48	5.93	2.96	-0.676
800S137-43	8.000	1.375	0.0451	0.375	0.0712	0.503	1.71	4.13	1.03	2.87	0.0931	0.0831	0.430	0.255	0.454	0.000341	1.21	5.81	2.98	-0.687
800S137-33*	8.000	1.375	0.0346	0.375	0.0765	0.388	1.32	3.20	0.800	2.87	0.0732	0.0653	0.435	0.254	0.460	0.000155	0.957	5.78	2.99	-0.696
600S250-97	6.000	2.500	0.1017	0.625	0.1526	1.17	3.97	6.50	2.17	2.36	0.923	0.529	0.889	0.754	1.10	0.00403	6.95	3.46	3.10	-1.80
600S250-68	6.000	2.500	0.0713	0.625	0.1070	0.836	2.84	4.73	1.58	2.38	0.688	0.395	0.908	0.758	1.12	0.00142	5.15	3.43	3.14	-1.84
600S250-54	6.000	2.500	0.0566	0.625	0.0849	0.670	2.28	3.82	1.27	2.39	0.563	0.323	0.917	0.759	1.13	0.000715	4.19	3.41	3.16	-1.86
600S250-43	6.000	2.500	0.0451	0.625	0.0712	0.537	1.83	3.08	1.03	2.40	0.458	0.263	0.923	0.760	1.14	0.000364	3.41	3.41	3.18	-1.87
600S200-97	6.000	2.000	0.1017	0.625	0.1526	1.07	3.63	5.61	1.87	2.29	0.530	0.371	0.705	0.570	0.859	0.00368	4.08	3.35	2.77	-1.38
600S200-68	6.000	2.000	0.0713	0.625	0.1070	0.764	2.60	4.10	1.37	2.32	0.400	0.280	0.723	0.573	0.878	0.00130	3.05	3.29	2.81	-1.42
600S200-54	6.000	2.000	0.0566	0.625	0.0849	0.613	2.08	3.32	1.11	2.33	0.329	0.230	0.732	0.574	0.887	0.000655	2.49	3.26	2.83	-1.43
600S200-43	6.000	2.000	0.0451	0.625	0.0712	0.492	1.67	2.68	0.894	2.34	0.268	0.188	0.739	0.574	0.894	0.000334	2.03	3.25	2.84	-1.45
600S200-33	6.000	2.000	0.0346	0.625	0.0765	0.379	1.29	2.08	0.692	2.34	0.209	0.147	0.743	0.574	0.901	0.000151	1.59	3.25	2.86	-1.46
600S162-97	6.000	1.625	0.1017	0.500	0.1526	0.966	3.28	4.80	1.60	2.23	0.283	0.233	0.542	0.411	0.636	0.00333	2.15	3.56	2.50	-0.997
600S162-68	6.000	1.625	0.0713	0.500	0.1070	0.693	2.36	3.52	1.17	2.26	0.218	0.180	0.561	0.413	0.655	0.00117	1.63	3.43	2.54	-1.03
600S162-54	6.000	1.625	0.0566	0.500	0.0849	0.556	1.89	2.86	0.954	2.27	0.181	0.149	0.570	0.414	0.663	0.000594	1.34	3.38	2.56	-1.05
600S162-43	6.000	1.625	0.0451	0.500	0.0712	0.447	1.52	2.32	0.772	2.28	0.148	0.123	0.576	0.414	0.670	0.000303	1.10	3.35	2.58	-1.06
600S162-33	6.000	1.625	0.0346	0.500	0.0765	0.344	1.17	1.79	0.598	2.28	0.116	0.0959	0.581	0.413	0.677	0.000137	0.861	3.35	2.59	-1.07

Table I - 2

Gross Section Properties

Joists/Studs

C-Sections With Lips



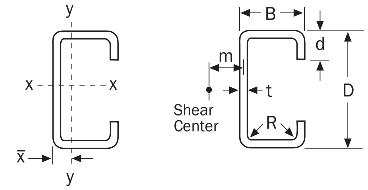
ID	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
600S137-97	6.000	1.375	0.1017	0.375	0.1526	0.889	3.02	4.19	1.40	2.17	0.159	0.149	0.423	0.305	0.480	0.00307	1.22	4.05	2.33	-0.734
600S137-68	6.000	1.375	0.0713	0.375	0.1070	0.640	2.17	3.09	1.03	2.20	0.126	0.117	0.443	0.306	0.497	0.00108	0.930	3.82	2.37	-0.768
600S137-54	6.000	1.375	0.0566	0.375	0.0849	0.514	1.75	2.52	0.839	2.21	0.105	0.0984	0.452	0.306	0.506	0.000549	0.769	3.72	2.39	-0.784
600S137-43	6.000	1.375	0.0451	0.375	0.0712	0.413	1.40	2.04	0.681	2.22	0.0871	0.0815	0.459	0.306	0.513	0.000280	0.633	3.66	2.41	-0.796
600S137-33	6.000	1.375	0.0346	0.375	0.0765	0.318	1.08	1.58	0.527	2.23	0.0685	0.0641	0.464	0.305	0.519	0.000127	0.500	3.65	2.42	-0.807
550S162-68	5.500	1.625	0.0713	0.500	0.1070	0.657	2.24	2.86	1.04	2.09	0.213	0.178	0.568	0.433	0.675	0.00111	1.34	3.08	2.41	-1.07
550S162-54	5.500	1.625	0.0566	0.500	0.0849	0.528	1.80	2.32	0.845	2.10	0.176	0.148	0.577	0.434	0.684	0.000564	1.10	3.04	2.43	-1.09
550S162-43	5.500	1.625	0.0451	0.500	0.0712	0.424	1.44	1.88	0.685	2.11	0.145	0.122	0.584	0.435	0.691	0.000288	0.905	3.02	2.45	-1.10
550S162-33	5.500	1.625	0.0346	0.500	0.0765	0.327	1.11	1.46	0.530	2.11	0.113	0.0952	0.589	0.434	0.697	0.000130	0.713	3.02	2.46	-1.11
400S200-68	4.000	2.000	0.0713	0.625	0.1070	0.622	2.11	1.59	0.795	1.60	0.349	0.268	0.750	0.696	0.983	0.00105	1.32	2.45	2.41	-1.64
400S200-54	4.000	2.000	0.0566	0.625	0.0849	0.500	1.70	1.29	0.646	1.61	0.287	0.221	0.758	0.697	0.993	0.000534	1.08	2.45	2.43	-1.66
400S200-43	4.000	2.000	0.0451	0.625	0.0712	0.402	1.37	1.05	0.524	1.62	0.235	0.180	0.764	0.698	1.000	0.000272	0.886	2.45	2.45	-1.68
400S200-33	4.000	2.000	0.0346	0.625	0.0765	0.310	1.05	0.812	0.406	1.62	0.183	0.141	0.769	0.698	1.01	0.000124	0.697	2.46	2.46	-1.69
400S162-68	4.000	1.625	0.0713	0.500	0.1070	0.550	1.87	1.35	0.673	1.56	0.192	0.173	0.591	0.511	0.745	0.000933	0.677	2.27	2.07	-1.22
400S162-54	4.000	1.625	0.0566	0.500	0.0849	0.443	1.51	1.10	0.549	1.57	0.159	0.143	0.600	0.512	0.754	0.000473	0.560	2.25	2.09	-1.24
400S162-43	4.000	1.625	0.0451	0.500	0.0712	0.357	1.21	0.892	0.446	1.58	0.131	0.118	0.606	0.513	0.761	0.000242	0.460	2.25	2.11	-1.25
400S162-33	4.000	1.625	0.0346	0.500	0.0765	0.275	0.935	0.692	0.346	1.59	0.103	0.0923	0.611	0.512	0.768	0.000110	0.363	2.25	2.12	-1.26
400S137-68	4.000	1.375	0.0713	0.375	0.1070	0.497	1.69	1.17	0.583	1.53	0.112	0.113	0.475	0.384	0.574	0.000842	0.375	2.27	1.85	-0.922
400S137-54	4.000	1.375	0.0566	0.375	0.0849	0.401	1.36	0.953	0.477	1.54	0.0939	0.0949	0.484	0.385	0.583	0.000428	0.311	2.24	1.87	-0.940
400S137-43	4.000	1.375	0.0451	0.375	0.0712	0.323	1.10	0.776	0.388	1.55	0.0778	0.0787	0.491	0.386	0.591	0.000219	0.257	2.22	1.89	-0.954
400S137-33	4.000	1.375	0.0346	0.375	0.0765	0.249	0.847	0.603	0.302	1.56	0.0612	0.0618	0.496	0.385	0.597	0.0000994	0.204	2.22	1.90	-0.965
362S200-68	3.625	2.000	0.0713	0.625	0.1070	0.595	2.02	1.27	0.698	1.46	0.337	0.265	0.753	0.726	1.01	0.00101	1.09	2.35	2.36	-1.70
362S200-54	3.625	2.000	0.0566	0.625	0.0849	0.479	1.63	1.03	0.568	1.47	0.277	0.218	0.761	0.727	1.02	0.000511	0.896	2.35	2.38	-1.72
362S200-43	3.625	2.000	0.0451	0.625	0.0712	0.385	1.31	0.836	0.461	1.47	0.227	0.178	0.767	0.728	1.02	0.000261	0.734	2.35	2.40	-1.73
362S200-33	3.625	2.000	0.0346	0.625	0.0765	0.297	1.01	0.648	0.358	1.48	0.177	0.139	0.772	0.728	1.03	0.000118	0.577	2.36	2.41	-1.74
362S162-68	3.625	1.625	0.0713	0.500	0.1070	0.524	1.78	1.07	0.590	1.43	0.186	0.171	0.596	0.535	0.765	0.000887	0.552	2.12	2.00	-1.26
362S162-54	3.625	1.625	0.0566	0.500	0.0849	0.422	1.43	0.873	0.482	1.44	0.154	0.142	0.605	0.536	0.774	0.000451	0.457	2.11	2.02	-1.28
362S162-43	3.625	1.625	0.0451	0.500	0.0712	0.340	1.16	0.710	0.392	1.45	0.127	0.117	0.611	0.537	0.782	0.000230	0.376	2.11	2.04	-1.30
362S162-33	3.625	1.625	0.0346	0.500	0.0765	0.262	0.891	0.551	0.304	1.45	0.0993	0.0913	0.616	0.537	0.789	0.000105	0.297	2.12	2.05	-1.31

Table I - 2

Gross Section Properties

Joists/Studs

C-Sections With Lips



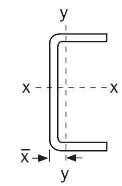
ID	Dimensions							Properties of Full Section													
								Axis x-x			Axis y-y										
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
362S137-68	3.625	1.375	0.0713	0.375	0.1070	0.470	1.60	0.923	0.509	1.40	0.109	0.112	0.481	0.403	0.592	0.000797	0.302	2.06	1.76	-0.959
362S137-54	3.625	1.375	0.0566	0.375	0.0849	0.380	1.29	0.756	0.417	1.41	0.0911	0.0939	0.490	0.405	0.601	0.000405	0.251	2.04	1.79	-0.978
362S137-43	3.625	1.375	0.0451	0.375	0.0712	0.306	1.04	0.616	0.340	1.42	0.0755	0.0779	0.497	0.406	0.608	0.000207	0.208	2.03	1.80	-0.991
362S137-33	3.625	1.375	0.0346	0.375	0.0765	0.236	0.803	0.479	0.264	1.42	0.0594	0.0612	0.501	0.405	0.615	0.0000942	0.165	2.03	1.81	-1.00
350S162-68	3.500	1.625	0.0713	0.500	0.1070	0.515	1.75	0.985	0.563	1.38	0.184	0.170	0.597	0.544	0.772	0.000872	0.514	2.07	1.98	-1.28
350S162-54	3.500	1.625	0.0566	0.500	0.0849	0.415	1.41	0.804	0.460	1.39	0.152	0.141	0.606	0.545	0.782	0.000443	0.426	2.07	2.00	-1.30
350S162-43	3.500	1.625	0.0451	0.500	0.0712	0.334	1.14	0.655	0.374	1.40	0.125	0.116	0.612	0.546	0.789	0.000227	0.350	2.07	2.01	-1.31
350S162-33	3.500	1.625	0.0346	0.500	0.0765	0.258	0.876	0.508	0.291	1.40	0.0981	0.0909	0.617	0.546	0.796	0.000103	0.277	2.07	2.03	-1.32
250S162-68	2.500	1.625	0.0713	0.500	0.1070	0.444	1.51	0.450	0.360	1.01	0.162	0.162	0.605	0.625	0.835	0.000752	0.268	1.81	1.85	-1.42
250S162-54	2.500	1.625	0.0566	0.500	0.0849	0.358	1.22	0.370	0.296	1.02	0.135	0.135	0.613	0.627	0.845	0.000383	0.223	1.81	1.87	-1.44
250S162-43	2.500	1.625	0.0451	0.500	0.0712	0.289	0.983	0.302	0.242	1.02	0.111	0.111	0.620	0.628	0.852	0.000196	0.184	1.82	1.89	-1.46
250S162-33	2.500	1.625	0.0346	0.500	0.0765	0.223	0.759	0.235	0.188	1.03	0.0870	0.0872	0.624	0.628	0.859	0.0000891	0.146	1.83	1.90	-1.47
250S137-68	2.500	1.375	0.0713	0.375	0.1070	0.390	1.33	0.386	0.309	0.995	0.0956	0.107	0.495	0.479	0.653	0.000661	0.138	1.61	1.56	-1.10
250S137-54	2.500	1.375	0.0566	0.375	0.0849	0.316	1.07	0.318	0.255	1.00	0.0802	0.0897	0.504	0.481	0.663	0.000337	0.115	1.61	1.58	-1.12
250S137-43	2.500	1.375	0.0451	0.375	0.0712	0.255	0.868	0.261	0.209	1.01	0.0665	0.0745	0.511	0.482	0.670	0.000173	0.0959	1.62	1.60	-1.13
250S137-33	2.500	1.375	0.0346	0.375	0.0765	0.197	0.671	0.203	0.163	1.02	0.0524	0.0586	0.515	0.482	0.677	0.0000787	0.0764	1.63	1.61	-1.14

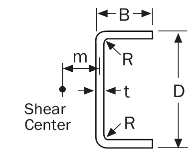
Table I - 3

Gross Section Properties

Tracks

C-Sections Without Lips





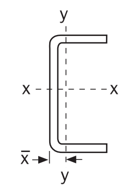
ID	Dimensions				Area	wt/ft	Properties of Full Section												
							Axis x-x			Axis y-y									
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
1200T200-97	12.356	2.000	0.1017	0.1525	1.63	5.53	29.8	4.82	4.28	0.410	0.240	0.502	0.289	0.476	0.00560	11.9	10.1	4.37	-0.714
1200T200-68	12.250	2.000	0.0713	0.1069	1.14	3.88	20.8	3.39	4.27	0.294	0.171	0.508	0.277	0.483	0.00193	8.43	9.92	4.36	-0.725
1200T200-54*	12.198	2.000	0.0566	0.0849	0.905	3.08	16.5	2.70	4.27	0.236	0.136	0.510	0.272	0.487	0.000966	6.71	9.85	4.36	-0.730
1200T150-97	12.356	1.500	0.1017	0.1525	1.52	5.18	26.0	4.21	4.13	0.176	0.135	0.340	0.191	0.301	0.00525	5.33	13.0	4.17	-0.441
1200T150-68	12.250	1.500	0.0713	0.1069	1.07	3.63	18.1	2.96	4.12	0.127	0.0964	0.345	0.179	0.307	0.00181	3.79	12.7	4.16	-0.450
1200T150-54*	12.198	1.500	0.0566	0.0849	0.848	2.88	14.4	2.36	4.12	0.103	0.0773	0.348	0.173	0.310	0.000906	3.03	12.6	4.16	-0.454
1200T125-97	12.356	1.250	0.1017	0.1525	1.47	5.01	24.1	3.90	4.04	0.102	0.0931	0.264	0.151	0.222	0.00508	3.17	15.5	4.07	-0.322
1200T125-68	12.250	1.250	0.0713	0.1069	1.03	3.51	16.8	2.75	4.04	0.0744	0.0669	0.268	0.138	0.227	0.00175	2.27	15.1	4.06	-0.329
1200T125-54*	12.198	1.250	0.0566	0.0849	0.820	2.79	13.3	2.19	4.03	0.0601	0.0537	0.271	0.131	0.230	0.000876	1.82	15.0	4.06	-0.333
1000T200-97	10.356	2.000	0.1017	0.1525	1.42	4.83	19.1	3.69	3.66	0.397	0.237	0.528	0.323	0.519	0.00490	7.92	7.36	3.79	-0.791
1000T200-68	10.250	2.000	0.0713	0.1069	0.997	3.39	13.3	2.59	3.65	0.284	0.168	0.534	0.312	0.527	0.00169	5.58	7.25	3.78	-0.803
1000T200-54	10.198	2.000	0.0566	0.0849	0.792	2.69	10.5	2.06	3.65	0.228	0.135	0.537	0.306	0.531	0.000845	4.43	7.20	3.77	-0.809
1000T200-43*	10.161	2.000	0.0451	0.0712	0.631	2.15	8.36	1.65	3.64	0.183	0.108	0.539	0.302	0.534	0.000428	3.54	7.16	3.77	-0.813
1000T150-97	10.356	1.500	0.1017	0.1525	1.32	4.49	16.4	3.17	3.53	0.172	0.133	0.361	0.213	0.332	0.00455	3.56	9.26	3.58	-0.495
1000T150-68	10.250	1.500	0.0713	0.1069	0.926	3.15	11.4	2.23	3.52	0.124	0.0954	0.366	0.201	0.339	0.00157	2.52	9.09	3.57	-0.505
1000T150-54	10.198	1.500	0.0566	0.0849	0.735	2.50	9.06	1.78	3.51	0.0998	0.0765	0.368	0.195	0.342	0.000785	2.01	9.00	3.57	-0.509
1000T150-43*	10.161	1.500	0.0451	0.0712	0.586	1.99	7.21	1.42	3.51	0.0804	0.0614	0.370	0.191	0.345	0.000397	1.61	8.94	3.56	-0.513
1000T125-97	10.356	1.250	0.1017	0.1525	1.27	4.32	15.1	2.91	3.45	0.100	0.0923	0.281	0.167	0.247	0.00438	2.12	10.9	3.48	-0.363
1000T125-68	10.250	1.250	0.0713	0.1069	0.890	3.03	10.5	2.05	3.44	0.0727	0.0663	0.286	0.154	0.253	0.00151	1.51	10.7	3.47	-0.372
1000T125-54	10.198	1.250	0.0566	0.0849	0.707	2.40	8.33	1.63	3.43	0.0587	0.0533	0.288	0.148	0.256	0.000755	1.21	10.6	3.47	-0.376
1000T125-43*	10.161	1.250	0.0451	0.0712	0.563	1.92	6.63	1.31	3.43	0.0474	0.0428	0.290	0.143	0.259	0.000382	0.973	10.5	3.46	-0.379
800T200-97	8.356	2.000	0.1017	0.1525	1.22	4.14	11.2	2.68	3.03	0.379	0.232	0.558	0.369	0.571	0.00420	4.79	5.18	3.21	-0.889
800T200-68	8.250	2.000	0.0713	0.1069	0.855	2.91	7.79	1.89	3.02	0.272	0.165	0.564	0.358	0.580	0.00145	3.36	5.11	3.20	-0.902
800T200-54	8.198	2.000	0.0566	0.0849	0.679	2.31	6.15	1.50	3.01	0.218	0.132	0.567	0.353	0.584	0.000725	2.66	5.07	3.20	-0.908
800T200-43	8.161	2.000	0.0451	0.0712	0.541	1.84	4.89	1.20	3.01	0.175	0.106	0.569	0.349	0.587	0.000367	2.12	5.04	3.19	-0.913
800T200-33*	8.146	2.000	0.0346	0.0764	0.415	1.41	3.75	0.921	3.01	0.135	0.0817	0.571	0.345	0.589	0.000166	1.64	5.03	3.19	-0.917

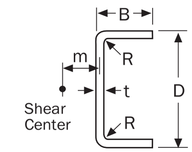
Table I - 3

Gross Section Properties

Tracks

C-Sections Without Lips





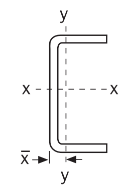
ID	Dimensions						Properties of Full Section												
							Axis x-x			Axis y-y									
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
800T150-97	8.356	1.500	0.1017	0.1525	1.12	3.80	9.48	2.27	2.91	0.165	0.132	0.385	0.243	0.372	0.00385	2.16	6.26	2.99	-0.564
800T150-68	8.250	1.500	0.0713	0.1069	0.783	2.66	6.59	1.60	2.90	0.119	0.0941	0.390	0.231	0.379	0.00133	1.53	6.13	2.98	-0.575
800T150-54	8.198	1.500	0.0566	0.0849	0.622	2.11	5.21	1.27	2.90	0.0961	0.0754	0.393	0.226	0.383	0.000664	1.22	6.07	2.98	-0.580
800T150-43	8.161	1.500	0.0451	0.0712	0.496	1.69	4.14	1.02	2.89	0.0774	0.0605	0.395	0.221	0.386	0.000336	0.972	6.03	2.98	-0.584
800T150-33*	8.146	1.500	0.0346	0.0764	0.381	1.29	3.18	0.781	2.89	0.0600	0.0467	0.397	0.217	0.388	0.000152	0.751	6.01	2.98	-0.588
800T125-97	8.356	1.250	0.1017	0.1525	1.07	3.62	8.61	2.06	2.84	0.0967	0.0911	0.301	0.189	0.279	0.00367	1.30	7.26	2.89	-0.417
800T125-68	8.250	1.250	0.0713	0.1069	0.748	2.54	6.00	1.45	2.83	0.0703	0.0655	0.307	0.177	0.286	0.00127	0.920	7.10	2.88	-0.427
800T125-54	8.198	1.250	0.0566	0.0849	0.594	2.02	4.75	1.16	2.83	0.0568	0.0526	0.309	0.171	0.289	0.000634	0.735	7.02	2.88	-0.432
800T125-43	8.161	1.250	0.0451	0.0712	0.473	1.61	3.77	0.925	2.82	0.0458	0.0423	0.311	0.166	0.292	0.000321	0.589	6.96	2.87	-0.436
800T125-33*	8.146	1.250	0.0346	0.0764	0.363	1.24	2.90	0.711	2.82	0.0356	0.0327	0.313	0.162	0.294	0.000145	0.456	6.93	2.88	-0.439
600T200-97	6.356	2.000	0.1017	0.1525	1.01	3.45	5.77	1.82	2.39	0.355	0.226	0.591	0.432	0.635	0.00350	2.51	3.53	2.66	-1.02
600T200-68	6.250	2.000	0.0713	0.1069	0.712	2.42	3.99	1.28	2.37	0.254	0.161	0.597	0.422	0.644	0.00121	1.75	3.48	2.65	-1.03
600T200-54	6.198	2.000	0.0566	0.0849	0.565	1.92	3.14	1.01	2.36	0.204	0.129	0.600	0.418	0.649	0.000604	1.38	3.46	2.65	-1.04
600T200-43	6.161	2.000	0.0451	0.0712	0.451	1.53	2.49	0.810	2.35	0.163	0.103	0.602	0.414	0.652	0.000306	1.10	3.45	2.64	-1.04
600T200-33	6.146	2.000	0.0346	0.0764	0.346	1.18	1.91	0.623	2.35	0.126	0.0794	0.604	0.411	0.655	0.000138	0.847	3.44	2.65	-1.05
600T150-97	6.356	1.500	0.1017	0.1525	0.913	3.10	4.78	1.50	2.29	0.156	0.129	0.414	0.285	0.421	0.00315	1.14	3.95	2.42	-0.656
600T150-68	6.250	1.500	0.0713	0.1069	0.641	2.18	3.31	1.06	2.27	0.113	0.0920	0.419	0.275	0.430	0.00109	0.797	3.87	2.41	-0.669
600T150-54	6.198	1.500	0.0566	0.0849	0.509	1.73	2.61	0.843	2.27	0.0907	0.0737	0.422	0.269	0.434	0.000543	0.632	3.84	2.40	-0.675
600T150-43	6.161	1.500	0.0451	0.0712	0.406	1.38	2.07	0.673	2.26	0.0730	0.0592	0.424	0.265	0.437	0.000275	0.504	3.81	2.40	-0.680
600T150-33	6.146	1.500	0.0346	0.0764	0.311	1.06	1.59	0.517	2.26	0.0566	0.0457	0.426	0.262	0.439	0.000124	0.390	3.80	2.40	-0.684
600T150-30	6.141	1.500	0.0312	0.0781	0.281	0.954	1.43	0.467	2.26	0.0512	0.0413	0.427	0.261	0.440	0.0000911	0.352	3.80	2.40	-0.685
600T150-27*	6.136	1.500	0.0283	0.0796	0.255	0.866	1.30	0.424	2.26	0.0465	0.0375	0.428	0.260	0.441	0.0000680	0.320	3.79	2.40	-0.686
600T125-97	6.356	1.250	0.1017	0.1525	0.862	2.93	4.28	1.35	2.23	0.0919	0.0894	0.327	0.221	0.321	0.00297	0.685	4.43	2.31	-0.491
600T125-68	6.250	1.250	0.0713	0.1069	0.605	2.06	2.97	0.950	2.22	0.0668	0.0642	0.332	0.210	0.329	0.00103	0.483	4.33	2.30	-0.503
600T125-54	6.198	1.250	0.0566	0.0849	0.480	1.63	2.34	0.756	2.21	0.0539	0.0516	0.335	0.204	0.332	0.000513	0.384	4.28	2.29	-0.508
600T125-43	6.161	1.250	0.0451	0.0712	0.383	1.30	1.86	0.604	2.21	0.0435	0.0415	0.337	0.200	0.335	0.000260	0.307	4.24	2.29	-0.513
600T125-33	6.146	1.250	0.0346	0.0764	0.294	0.999	1.43	0.465	2.20	0.0338	0.0321	0.339	0.196	0.337	0.000117	0.238	4.22	2.29	-0.516
600T125-30	6.141	1.250	0.0312	0.0781	0.265	0.901	1.29	0.420	2.20	0.0306	0.0290	0.340	0.195	0.338	0.0000860	0.215	4.22	2.29	-0.518
600T125-27*	6.136	1.250	0.0283	0.0796	0.241	0.818	1.17	0.381	2.20	0.0278	0.0264	0.340	0.194	0.339	0.0000642	0.196	4.21	2.29	-0.519

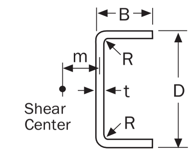
Table I - 3

Gross Section Properties

Tracks

C-Sections Without Lips





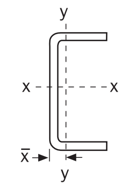
ID	Dimensions						Properties of Full Section													
							Axis x-x			Axis y-y										
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
550T200-68	5.750	2.000	0.0713	0.1069	0.676	2.30	3.27	1.14	2.20	0.248	0.159	0.606	0.443	0.663	0.00115	1.43	3.16	2.52	-1.07
550T200-54	5.698	2.000	0.0566	0.0849	0.537	1.83	2.58	0.905	2.19	0.199	0.127	0.609	0.438	0.668	0.000573	1.13	3.14	2.52	-1.08
550T200-43	5.661	2.000	0.0451	0.0712	0.428	1.46	2.04	0.722	2.19	0.160	0.102	0.611	0.435	0.671	0.000290	0.900	3.13	2.51	-1.08
550T200-33	5.646	2.000	0.0346	0.0764	0.329	1.12	1.57	0.555	2.18	0.123	0.0787	0.613	0.431	0.674	0.000131	0.694	3.12	2.52	-1.09
550T150-68	5.750	1.500	0.0713	0.1069	0.605	2.06	2.70	0.939	2.11	0.111	0.0912	0.427	0.289	0.445	0.00103	0.655	3.42	2.27	-0.698
550T150-54	5.698	1.500	0.0566	0.0849	0.480	1.63	2.13	0.747	2.11	0.0889	0.0731	0.430	0.284	0.449	0.000513	0.519	3.39	2.26	-0.704
550T150-43	5.661	1.500	0.0451	0.0712	0.383	1.30	1.69	0.596	2.10	0.0716	0.0587	0.433	0.280	0.452	0.000260	0.414	3.36	2.26	-0.709
550T150-33	5.646	1.500	0.0346	0.0764	0.294	0.999	1.29	0.459	2.10	0.0555	0.0453	0.434	0.276	0.455	0.000117	0.320	3.35	2.26	-0.714
550T150-30	5.641	1.500	0.0312	0.0781	0.265	0.901	1.17	0.414	2.10	0.0502	0.0410	0.435	0.275	0.455	0.0000860	0.289	3.35	2.26	-0.715
550T150-27	5.636	1.500	0.0283	0.0796	0.241	0.818	1.06	0.376	2.10	0.0456	0.0372	0.436	0.274	0.456	0.0000642	0.263	3.35	2.26	-0.716
550T125-68	5.750	1.250	0.0713	0.1069	0.569	1.94	2.41	0.839	2.06	0.0656	0.0638	0.340	0.221	0.341	0.000965	0.397	3.77	2.15	-0.526
550T125-54	5.698	1.250	0.0566	0.0849	0.452	1.54	1.90	0.668	2.05	0.0530	0.0512	0.342	0.215	0.345	0.000483	0.315	3.72	2.15	-0.532
550T125-43	5.661	1.250	0.0451	0.0712	0.360	1.23	1.51	0.533	2.05	0.0428	0.0412	0.345	0.211	0.348	0.000244	0.252	3.69	2.14	-0.537
550T125-33	5.646	1.250	0.0346	0.0764	0.277	0.941	1.16	0.410	2.05	0.0332	0.0318	0.346	0.207	0.350	0.000110	0.195	3.68	2.15	-0.541
550T125-30	5.641	1.250	0.0312	0.0781	0.250	0.848	1.04	0.371	2.05	0.0301	0.0288	0.347	0.206	0.351	0.0000810	0.176	3.67	2.15	-0.542
550T125-27	5.636	1.250	0.0283	0.0796	0.226	0.770	0.948	0.336	2.05	0.0273	0.0262	0.348	0.205	0.352	0.0000604	0.160	3.67	2.15	-0.543
400T200-68	4.250	2.000	0.0713	0.1069	0.569	1.94	1.62	0.761	1.69	0.227	0.153	0.632	0.519	0.725	0.000965	0.702	2.39	2.17	-1.21
400T200-54	4.198	2.000	0.0566	0.0849	0.452	1.54	1.27	0.604	1.68	0.182	0.123	0.635	0.515	0.730	0.000483	0.551	2.38	2.17	-1.22
400T200-43	4.161	2.000	0.0451	0.0712	0.360	1.23	1.00	0.482	1.67	0.146	0.0982	0.637	0.512	0.734	0.000244	0.436	2.37	2.16	-1.22
400T200-33	4.146	2.000	0.0346	0.0764	0.277	0.941	0.768	0.371	1.67	0.113	0.0757	0.639	0.509	0.737	0.000110	0.336	2.37	2.17	-1.23
400T150-68	4.250	1.500	0.0713	0.1069	0.498	1.69	1.31	0.615	1.62	0.102	0.0883	0.453	0.343	0.496	0.000844	0.320	2.31	1.86	-0.804
400T150-54	4.198	1.500	0.0566	0.0849	0.396	1.35	1.03	0.489	1.61	0.0822	0.0708	0.456	0.339	0.501	0.000422	0.252	2.30	1.86	-0.811
400T150-43	4.161	1.500	0.0451	0.0712	0.315	1.07	0.811	0.390	1.60	0.0662	0.0568	0.458	0.335	0.504	0.000214	0.200	2.29	1.86	-0.817
400T150-33	4.146	1.500	0.0346	0.0764	0.242	0.823	0.622	0.300	1.60	0.0513	0.0439	0.460	0.332	0.507	0.0000966	0.155	2.28	1.86	-0.821
400T150-30	4.141	1.500	0.0312	0.0781	0.218	0.742	0.561	0.271	1.60	0.0464	0.0396	0.461	0.331	0.508	0.0000708	0.140	2.28	1.86	-0.823
400T150-27	4.136	1.500	0.0283	0.0796	0.198	0.673	0.509	0.246	1.60	0.0422	0.0360	0.461	0.330	0.509	0.0000529	0.127	2.28	1.86	-0.824

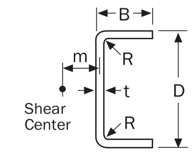
Table I - 3

Gross Section Properties

Tracks

C-Sections Without Lips





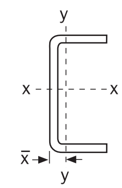
ID	Dimensions						Properties of Full Section												
							Axis x-x			Axis y-y									
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
400T125-68	4.250	1.250	0.0713	0.1069	0.462	1.57	1.15	0.541	1.58	0.0611	0.0620	0.364	0.264	0.386	0.000783	0.194	2.40	1.73	-0.614
400T125-54	4.198	1.250	0.0566	0.0849	0.367	1.25	0.904	0.431	1.57	0.0493	0.0498	0.367	0.259	0.390	0.000392	0.154	2.37	1.73	-0.621
400T125-43	4.161	1.250	0.0451	0.0712	0.293	0.995	0.716	0.344	1.56	0.0398	0.0400	0.369	0.255	0.394	0.000198	0.122	2.36	1.72	-0.626
400T125-33	4.146	1.250	0.0346	0.0764	0.225	0.764	0.549	0.265	1.56	0.0309	0.0309	0.371	0.251	0.396	0.0000897	0.0946	2.35	1.73	-0.630
400T125-30	4.141	1.250	0.0312	0.0781	0.203	0.689	0.495	0.239	1.56	0.0280	0.0280	0.371	0.250	0.397	0.0000658	0.0855	2.35	1.73	-0.632
400T125-27	4.136	1.250	0.0283	0.0796	0.184	0.625	0.449	0.217	1.56	0.0254	0.0254	0.372	0.249	0.398	0.0000491	0.0777	2.35	1.73	-0.633
400T125-18*	4.122	1.250	0.0188	0.0843	0.122	0.416	0.298	0.145	1.56	0.0171	0.0170	0.374	0.246	0.400	0.0000144	0.0520	2.34	1.73	-0.637
362T200-68	3.875	2.000	0.0713	0.1069	0.543	1.85	1.31	0.675	1.55	0.221	0.151	0.638	0.543	0.743	0.000919	0.564	2.24	2.09	-1.25
362T200-54	3.823	2.000	0.0566	0.0849	0.431	1.47	1.02	0.536	1.54	0.177	0.121	0.641	0.539	0.748	0.000460	0.442	2.24	2.09	-1.26
362T200-43	3.786	2.000	0.0451	0.0712	0.344	1.17	0.808	0.427	1.53	0.142	0.0969	0.643	0.536	0.752	0.000233	0.350	2.23	2.09	-1.27
362T200-33	3.771	2.000	0.0346	0.0764	0.264	0.897	0.619	0.329	1.53	0.110	0.0747	0.645	0.533	0.754	0.000105	0.269	2.23	2.09	-1.27
362T150-68	3.875	1.500	0.0713	0.1069	0.471	1.60	1.05	0.542	1.49	0.0995	0.0873	0.460	0.361	0.511	0.000799	0.257	2.10	1.77	-0.836
362T150-54	3.823	1.500	0.0566	0.0849	0.374	1.27	0.823	0.431	1.48	0.0801	0.0700	0.463	0.356	0.516	0.000400	0.202	2.09	1.77	-0.844
362T150-43	3.786	1.500	0.0451	0.0712	0.298	1.01	0.650	0.344	1.48	0.0644	0.0562	0.465	0.353	0.519	0.000202	0.160	2.08	1.77	-0.850
362T150-33	3.771	1.500	0.0346	0.0764	0.229	0.779	0.499	0.264	1.48	0.0499	0.0434	0.467	0.350	0.522	0.0000914	0.124	2.08	1.77	-0.854
362T150-30	3.766	1.500	0.0312	0.0781	0.207	0.703	0.449	0.239	1.48	0.0451	0.0392	0.467	0.348	0.523	0.0000671	0.112	2.07	1.77	-0.856
362T150-27	3.761	1.500	0.0283	0.0796	0.188	0.637	0.408	0.217	1.48	0.0410	0.0356	0.468	0.348	0.524	0.0000501	0.102	2.07	1.77	-0.857
362T125-68	3.875	1.250	0.0713	0.1069	0.436	1.48	0.921	0.475	1.45	0.0597	0.0613	0.370	0.277	0.399	0.000738	0.156	2.13	1.63	-0.641
362T125-54	3.823	1.250	0.0566	0.0849	0.346	1.18	0.723	0.378	1.45	0.0481	0.0493	0.373	0.273	0.404	0.000369	0.123	2.11	1.63	-0.648
362T125-43	3.786	1.250	0.0451	0.0712	0.276	0.938	0.571	0.302	1.44	0.0388	0.0396	0.375	0.269	0.407	0.000187	0.0978	2.10	1.63	-0.654
362T125-33	3.771	1.250	0.0346	0.0764	0.212	0.720	0.438	0.232	1.44	0.0301	0.0306	0.377	0.266	0.409	0.0000845	0.0756	2.09	1.63	-0.658
362T125-30	3.766	1.250	0.0312	0.0781	0.191	0.649	0.395	0.210	1.44	0.0273	0.0277	0.378	0.265	0.410	0.0000620	0.0684	2.09	1.63	-0.659
362T125-27	3.761	1.250	0.0283	0.0796	0.173	0.589	0.358	0.191	1.44	0.0248	0.0252	0.378	0.264	0.411	0.0000463	0.0622	2.09	1.63	-0.661
362T125-18	3.747	1.250	0.0188	0.0843	0.115	0.392	0.238	0.127	1.44	0.0167	0.0168	0.380	0.261	0.413	0.0000136	0.0416	2.08	1.63	-0.665
350T200-68	3.750	2.000	0.0713	0.1069	0.534	1.81	1.21	0.647	1.51	0.218	0.151	0.640	0.552	0.749	0.000904	0.522	2.20	2.07	-1.26
350T200-54	3.698	2.000	0.0566	0.0849	0.424	1.44	0.949	0.513	1.50	0.175	0.120	0.642	0.548	0.754	0.000453	0.409	2.19	2.07	-1.27
350T200-43	3.661	2.000	0.0451	0.0712	0.338	1.15	0.749	0.409	1.49	0.140	0.0965	0.645	0.545	0.758	0.000229	0.323	2.19	2.07	-1.28
350T200-33	3.646	2.000	0.0346	0.0764	0.259	0.882	0.574	0.315	1.49	0.108	0.0744	0.647	0.542	0.761	0.000104	0.249	2.19	2.07	-1.29

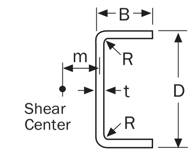
Table I - 3

Gross Section Properties

Tracks

C-Sections Without Lips





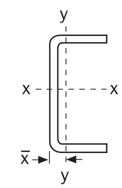
ID	Dimensions						Properties of Full Section												
							Axis x-x			Axis y-y									
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
350T150-68	3.750	1.500	0.0713	0.1069	0.462	1.57	0.972	0.518	1.45	0.0986	0.0870	0.462	0.367	0.516	0.000783	0.238	2.04	1.74	-0.847
350T150-54	3.698	1.500	0.0566	0.0849	0.367	1.25	0.761	0.412	1.44	0.0793	0.0697	0.465	0.362	0.521	0.000392	0.187	2.02	1.74	-0.855
350T150-43	3.661	1.500	0.0451	0.0712	0.293	0.995	0.601	0.329	1.43	0.0638	0.0559	0.467	0.359	0.525	0.000198	0.148	2.01	1.74	-0.861
350T150-33	3.646	1.500	0.0346	0.0764	0.225	0.764	0.461	0.253	1.43	0.0494	0.0432	0.469	0.356	0.527	0.0000897	0.114	2.01	1.74	-0.866
350T150-30	3.641	1.500	0.0312	0.0781	0.203	0.689	0.416	0.228	1.43	0.0447	0.0390	0.470	0.355	0.528	0.0000658	0.103	2.01	1.74	-0.867
350T150-27	3.636	1.500	0.0283	0.0796	0.184	0.625	0.377	0.207	1.43	0.0406	0.0355	0.470	0.354	0.529	0.0000491	0.0939	2.01	1.74	-0.869
350T125-68	3.750	1.250	0.0713	0.1069	0.427	1.45	0.851	0.454	1.41	0.0591	0.0611	0.372	0.283	0.403	0.000723	0.144	2.05	1.60	-0.650
350T125-54	3.698	1.250	0.0566	0.0849	0.339	1.15	0.668	0.361	1.40	0.0477	0.0491	0.375	0.278	0.408	0.000362	0.114	2.03	1.60	-0.658
350T125-43	3.661	1.250	0.0451	0.0712	0.270	0.919	0.528	0.288	1.40	0.0385	0.0394	0.377	0.274	0.412	0.000183	0.0904	2.02	1.59	-0.663
350T125-33	3.646	1.250	0.0346	0.0764	0.208	0.705	0.405	0.222	1.40	0.0299	0.0305	0.379	0.271	0.414	0.0000828	0.0699	2.01	1.59	-0.668
350T125-30	3.641	1.250	0.0312	0.0781	0.187	0.636	0.365	0.200	1.40	0.0270	0.0276	0.380	0.270	0.415	0.0000607	0.0632	2.01	1.59	-0.669
350T125-27	3.636	1.250	0.0283	0.0796	0.170	0.577	0.331	0.182	1.40	0.0246	0.0251	0.381	0.269	0.416	0.0000453	0.0574	2.01	1.60	-0.670
350T125-18	3.622	1.250	0.0188	0.0843	0.113	0.384	0.220	0.121	1.40	0.0165	0.0168	0.382	0.266	0.418	0.0000133	0.0384	2.00	1.60	-0.675
250T200-68	2.750	2.000	0.0713	0.1069	0.462	1.57	0.600	0.437	1.14	0.196	0.143	0.652	0.631	0.800	0.000783	0.251	1.92	1.92	-1.40
250T200-54	2.698	2.000	0.0566	0.0849	0.367	1.25	0.466	0.346	1.13	0.157	0.115	0.654	0.628	0.806	0.000392	0.195	1.92	1.92	-1.41
250T200-43	2.661	2.000	0.0451	0.0712	0.293	0.995	0.366	0.275	1.12	0.126	0.0918	0.657	0.625	0.810	0.000198	0.153	1.92	1.92	-1.41
250T200-33	2.646	2.000	0.0346	0.0764	0.225	0.764	0.280	0.212	1.12	0.0974	0.0707	0.658	0.623	0.813	0.0000897	0.118	1.93	1.92	-1.42
250T150-68	2.750	1.500	0.0713	0.1069	0.391	1.33	0.472	0.344	1.10	0.0893	0.0833	0.478	0.427	0.561	0.000663	0.114	1.61	1.53	-0.953
250T150-54	2.698	1.500	0.0566	0.0849	0.311	1.06	0.368	0.273	1.09	0.0718	0.0667	0.481	0.423	0.566	0.000332	0.0887	1.61	1.53	-0.961
250T150-43	2.661	1.500	0.0451	0.0712	0.248	0.842	0.289	0.217	1.08	0.0578	0.0535	0.483	0.420	0.570	0.000168	0.0698	1.61	1.53	-0.968
250T150-33	2.646	1.500	0.0346	0.0764	0.190	0.647	0.221	0.167	1.08	0.0447	0.0413	0.485	0.418	0.573	0.0000759	0.0539	1.61	1.53	-0.973
250T150-30	2.641	1.500	0.0312	0.0781	0.172	0.583	0.200	0.151	1.08	0.0404	0.0373	0.486	0.417	0.574	0.0000557	0.0486	1.61	1.53	-0.975
250T150-27	2.636	1.500	0.0283	0.0796	0.156	0.529	0.181	0.137	1.08	0.0368	0.0339	0.486	0.416	0.575	0.0000416	0.0442	1.61	1.53	-0.976
250T125-68	2.750	1.250	0.0713	0.1069	0.355	1.21	0.409	0.297	1.07	0.0539	0.0587	0.389	0.332	0.444	0.000602	0.0689	1.51	1.36	-0.740
250T125-54	2.698	1.250	0.0566	0.0849	0.282	0.960	0.318	0.236	1.06	0.0435	0.0471	0.392	0.328	0.449	0.000301	0.0539	1.50	1.36	-0.749
250T125-43	2.661	1.250	0.0451	0.0712	0.225	0.765	0.250	0.188	1.06	0.0351	0.0379	0.395	0.325	0.453	0.000153	0.0425	1.49	1.36	-0.755
250T125-33	2.646	1.250	0.0346	0.0764	0.173	0.588	0.192	0.145	1.05	0.0272	0.0293	0.397	0.322	0.456	0.0000690	0.0328	1.49	1.36	-0.760
250T125-30	2.641	1.250	0.0312	0.0781	0.156	0.530	0.173	0.131	1.05	0.0246	0.0265	0.397	0.321	0.456	0.0000506	0.0297	1.49	1.36	-0.762
250T125-27	2.636	1.250	0.0283	0.0796	0.142	0.481	0.157	0.119	1.05	0.0224	0.0241	0.398	0.320	0.457	0.0000378	0.0270	1.49	1.36	-0.763
250T125-18	2.622	1.250	0.0188	0.0843	0.0941	0.320	0.104	0.0794	1.05	0.0150	0.0161	0.400	0.317	0.460	0.0000111	0.0180	1.49	1.36	-0.767

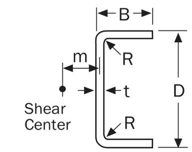
Table I - 3

Gross Section Properties

Tracks

C-Sections Without Lips



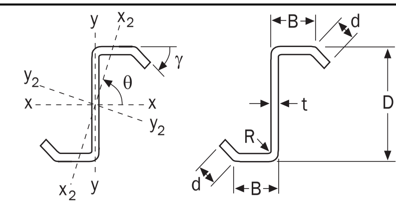


ID	Dimensions						Properties of Full Section												
							Axis x-x			Axis y-y									
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\overline{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
162T125-33	1.771	1.250	0.0346	0.0764	0.143	0.485	0.0772	0.0872	0.736	0.0238	0.0275	0.408	0.386	0.499	0.0000569	0.0128	1.22	1.21	-0.868
162T125-30	1.766	1.250	0.0312	0.0781	0.129	0.437	0.0695	0.0788	0.735	0.0215	0.0249	0.409	0.386	0.500	0.0000417	0.0115	1.22	1.21	-0.870
162T125-27	1.761	1.250	0.0283	0.0796	0.117	0.397	0.0630	0.0716	0.735	0.0196	0.0226	0.410	0.385	0.501	0.0000312	0.0105	1.22	1.21	-0.872
162T125-18	1.747	1.250	0.0188	0.0843	0.0776	0.264	0.0417	0.0478	0.733	0.0131	0.0151	0.411	0.382	0.503	0.00000915	0.00699	1.22	1.22	-0.876

Table I - 4

Gross Section Properties

Z-Sections With Lips

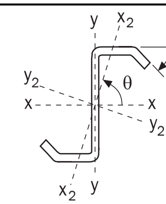


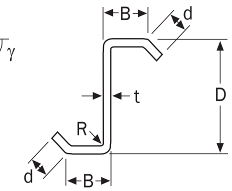
ID	Dimensions								Properties of Full Section												
									Axis x-x			Axis y-y									
	D	B	t	d	γ	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	I_{xy}	I_{x2}	I_{y2}	r_{min}	θ	J	C_w
	in.	in.	in.	in.	deg	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{4}	in.^{4}	in.^{4}	in.	deg	in.^{4}	in.^{6}
12ZS3.25x105	12.000	3.250	0.105	0.990	50	0.1875	2.09	7.12	43.7	7.29	4.57	4.67	1.22	1.49	10.2	2.18	46.2	1.02	76.3	0.00769	123
12ZS3.25x085	12.000	3.250	0.085	0.960	50	0.1875	1.70	5.76	35.5	5.92	4.58	3.75	0.982	1.49	8.21	1.76	37.5	1.02	76.3	0.00408	98.6
12ZS3.25x070	12.000	3.250	0.070	0.930	50	0.1875	1.40	4.75	29.3	4.89	4.58	3.07	0.806	1.48	6.74	1.44	31.0	1.02	76.4	0.00228	80.8
12ZS2.75x105	12.000	2.750	0.105	0.990	50	0.1875	1.99	6.76	40.0	6.67	4.49	3.13	0.938	1.25	7.78	1.55	41.6	0.883	78.6	0.00731	85.1
12ZS2.75x085	12.000	2.750	0.085	0.960	50	0.1875	1.61	5.47	32.5	5.42	4.49	2.51	0.755	1.25	6.28	1.25	33.8	0.880	78.6	0.00388	68.4
12ZS2.75x070	12.000	2.750	0.070	0.930	50	0.1875	1.33	4.51	26.8	4.47	4.50	2.05	0.620	1.24	5.16	1.02	27.9	0.878	78.7	0.00217	56.0
12ZS2.25x105	12.000	2.250	0.105	0.990	50	0.1875	1.88	6.40	36.3	6.05	4.39	1.96	0.692	1.02	5.71	1.03	37.2	0.741	80.8	0.00692	55.5
12ZS2.25x085	12.000	2.250	0.085	0.960	50	0.1875	1.53	5.18	29.5	4.91	4.40	1.57	0.557	1.02	4.61	0.831	30.2	0.738	80.9	0.00367	44.6
12ZS2.25x070	12.000	2.250	0.070	0.930	50	0.1875	1.26	4.27	24.3	4.06	4.40	1.28	0.457	1.01	3.78	0.681	24.9	0.736	80.9	0.00205	36.5
10ZS3.25x105	10.000	3.250	0.105	0.990	50	0.1875	1.88	6.40	28.4	5.69	3.89	4.67	1.22	1.58	8.41	2.00	31.1	1.03	72.4	0.00692	81.8
10ZS3.25x085	10.000	3.250	0.085	0.960	50	0.1875	1.53	5.18	23.1	4.62	3.89	3.75	0.982	1.57	6.79	1.61	25.3	1.03	72.5	0.00367	65.9
10ZS3.25x070	10.000	3.250	0.070	0.930	50	0.1875	1.26	4.27	19.1	3.82	3.90	3.07	0.806	1.56	5.59	1.32	20.9	1.02	72.6	0.00205	54.0
10ZS3.25x065	10.000	3.250	0.065	0.920	50	0.1875	1.17	3.96	17.8	3.55	3.90	2.85	0.747	1.56	5.18	1.22	19.4	1.02	72.6	0.00164	50.1
10ZS3.25x059	10.000	3.250	0.059	0.910	50	0.1875	1.06	3.60	16.1	3.23	3.91	2.58	0.677	1.56	4.70	1.11	17.6	1.02	72.6	0.00123	45.3
10ZS2.75x105	10.000	2.750	0.105	0.990	50	0.1875	1.78	6.05	25.9	5.17	3.81	3.13	0.938	1.33	6.44	1.43	27.6	0.897	75.2	0.00654	57.0
10ZS2.75x085	10.000	2.750	0.085	0.960	50	0.1875	1.44	4.90	21.0	4.21	3.82	2.51	0.755	1.32	5.20	1.15	22.4	0.894	75.4	0.00347	45.8
10ZS2.75x070	10.000	2.750	0.070	0.930	50	0.1875	1.19	4.03	17.4	3.47	3.83	2.05	0.620	1.32	4.27	0.943	18.5	0.892	75.4	0.00194	37.5
10ZS2.75x065	10.000	2.750	0.065	0.920	50	0.1875	1.10	3.74	16.2	3.23	3.83	1.90	0.575	1.31	3.96	0.874	17.2	0.891	75.5	0.00155	34.8
10ZS2.75x059	10.000	2.750	0.059	0.910	50	0.1875	0.999	3.40	14.7	2.94	3.83	1.72	0.521	1.31	3.59	0.792	15.6	0.890	75.5	0.00116	31.5
10ZS2.25x105	10.000	2.250	0.105	0.990	50	0.1875	1.67	5.69	23.3	4.66	3.73	1.96	0.692	1.08	4.72	0.962	24.3	0.758	78.1	0.00615	37.3
10ZS2.25x085	10.000	2.250	0.085	0.960	50	0.1875	1.36	4.61	18.9	3.79	3.74	1.57	0.557	1.08	3.81	0.773	19.7	0.756	78.2	0.00326	30.0
10ZS2.25x070	10.000	2.250	0.070	0.930	50	0.1875	1.12	3.79	15.6	3.13	3.75	1.28	0.457	1.07	3.13	0.633	16.3	0.753	78.2	0.00182	24.5
10ZS2.25x065	10.000	2.250	0.065	0.920	50	0.1875	1.04	3.52	14.5	2.91	3.75	1.19	0.424	1.07	2.90	0.587	15.2	0.753	78.3	0.00146	22.8
10ZS2.25x059	10.000	2.250	0.059	0.910	50	0.1875	0.940	3.20	13.2	2.64	3.75	1.08	0.384	1.07	2.63	0.531	13.8	0.752	78.3	0.00109	20.6

Table I - 4

Gross Section Properties

Z-Sections With Lips



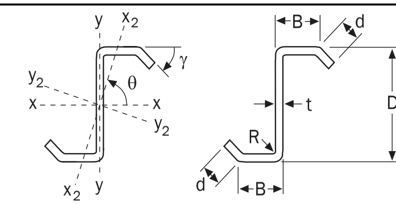


ID	Dimensions								Properties of Full Section												
									Axis x-x			Axis y-y									
	D	B	t	d	γ	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	I_{xy}	I_{x2}	I_{y2}	r_{min}	θ	J	C_w
	in.	in.	in.	in.	deg	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{4}	in.^{4}	in.^{4}	in.	deg	in.^{4}	in.^{6}
9ZS2.25x105	9.000	2.250	0.105	0.990	50	0.1875	1.57	5.33	18.1	4.02	3.39	1.96	0.692	1.12	4.22	0.920	19.1	0.766	76.2	0.00576	29.7
9ZS2.25x085	9.000	2.250	0.085	0.960	50	0.1875	1.27	4.32	14.7	3.27	3.40	1.57	0.557	1.11	3.41	0.739	15.5	0.763	76.3	0.00306	23.8
9ZS2.25x070	9.000	2.250	0.070	0.930	50	0.1875	1.05	3.56	12.2	2.70	3.41	1.28	0.457	1.11	2.80	0.605	12.8	0.761	76.4	0.00171	19.5
9ZS2.25x065	9.000	2.250	0.065	0.920	50	0.1875	0.971	3.30	11.3	2.51	3.41	1.19	0.424	1.11	2.60	0.561	11.9	0.760	76.4	0.00137	18.1
9ZS2.25x059	9.000	2.250	0.059	0.910	50	0.1875	0.881	3.00	10.3	2.28	3.41	1.08	0.384	1.11	2.36	0.508	10.8	0.759	76.4	0.00102	16.4
8ZS3.25x105	8.000	3.250	0.105	0.990	50	0.1875	1.67	5.69	16.9	4.23	3.18	4.67	1.22	1.67	6.66	1.75	19.8	1.02	66.3	0.00615	49.9
8ZS3.25x085	8.000	3.250	0.085	0.960	50	0.1875	1.36	4.61	13.8	3.44	3.19	3.75	0.982	1.66	5.38	1.41	16.1	1.02	66.5	0.00326	40.1
8ZS3.25x070	8.000	3.250	0.070	0.930	50	0.1875	1.12	3.79	11.4	2.85	3.19	3.07	0.806	1.66	4.43	1.15	13.3	1.02	66.6	0.00182	32.9
8ZS3.25x065	8.000	3.250	0.065	0.920	50	0.1875	1.04	3.52	10.6	2.65	3.20	2.85	0.747	1.66	4.11	1.07	12.4	1.02	66.6	0.00146	30.5
8ZS3.25x059	8.000	3.250	0.059	0.910	50	0.1875	0.940	3.20	9.62	2.41	3.20	2.58	0.677	1.66	3.73	0.970	11.2	1.02	66.7	0.00109	27.6
8ZS2.75x105	8.000	2.750	0.105	0.990	50	0.1875	1.57	5.33	15.3	3.82	3.12	3.13	0.938	1.41	5.09	1.28	17.1	0.902	70.0	0.00576	34.9
8ZS2.75x085	8.000	2.750	0.085	0.960	50	0.1875	1.27	4.32	12.4	3.11	3.13	2.51	0.755	1.41	4.11	1.03	13.9	0.899	70.2	0.00306	28.0
8ZS2.75x070	8.000	2.750	0.070	0.930	50	0.1875	1.05	3.56	10.3	2.57	3.14	2.05	0.620	1.40	3.38	0.841	11.5	0.897	70.3	0.00171	23.0
8ZS2.75x065	8.000	2.750	0.065	0.920	50	0.1875	0.971	3.30	9.56	2.39	3.14	1.90	0.575	1.40	3.14	0.780	10.7	0.896	70.3	0.00137	21.3
8ZS2.75x059	8.000	2.750	0.059	0.910	50	0.1875	0.881	3.00	8.69	2.17	3.14	1.72	0.521	1.40	2.85	0.706	9.71	0.895	70.4	0.00102	19.3
8ZS2.25x105	8.000	2.250	0.105	0.990	50	0.1875	1.46	4.98	13.6	3.41	3.05	1.96	0.692	1.16	3.73	0.872	14.7	0.772	73.7	0.00538	22.9
8ZS2.25x085	8.000	2.250	0.085	0.960	50	0.1875	1.19	4.03	11.1	2.77	3.06	1.57	0.557	1.15	3.01	0.700	12.0	0.769	73.9	0.00285	18.4
8ZS2.25x070	8.000	2.250	0.070	0.930	50	0.1875	0.976	3.32	9.18	2.30	3.07	1.28	0.457	1.15	2.47	0.573	9.89	0.767	74.0	0.00159	15.1
8ZS2.25x065	8.000	2.250	0.065	0.920	50	0.1875	0.906	3.08	8.54	2.13	3.07	1.19	0.424	1.15	2.30	0.532	9.20	0.766	74.0	0.00128	14.0
8ZS2.25x059	8.000	2.250	0.059	0.910	50	0.1875	0.822	2.80	7.76	1.94	3.07	1.08	0.384	1.14	2.08	0.481	8.36	0.765	74.0	0.000954	12.7
7ZS2.25x105	7.000	2.250	0.105	0.990	50	0.1875	1.36	4.62	9.92	2.83	2.70	1.96	0.692	1.20	3.23	0.815	11.1	0.774	70.5	0.00499	17.1
7ZS2.25x085	7.000	2.250	0.085	0.960	50	0.1875	1.10	3.74	8.09	2.31	2.71	1.57	0.557	1.20	2.61	0.655	9.01	0.772	70.6	0.00265	13.8
7ZS2.25x070	7.000	2.250	0.070	0.930	50	0.1875	0.906	3.08	6.70	1.91	2.72	1.28	0.457	1.19	2.15	0.536	7.45	0.769	70.8	0.00148	11.3
7ZS2.25x065	7.000	2.250	0.065	0.920	50	0.1875	0.841	2.86	6.23	1.78	2.72	1.19	0.424	1.19	1.99	0.497	6.92	0.769	70.8	0.00118	10.5
7ZS2.25x059	7.000	2.250	0.059	0.910	50	0.1875	0.763	2.60	5.67	1.62	2.72	1.08	0.384	1.19	1.81	0.450	6.29	0.768	70.9	0.000886	9.46

Table I - 4

Gross Section Properties

Z-Sections With Lips

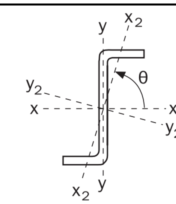


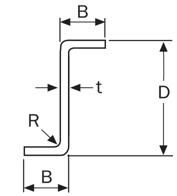
ID	Dimensions								Properties of Full Section												
									Axis x-x			Axis y-y									
	D	B	t	d	γ	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	I_{xy}	I_{x2}	I_{y2}	r_{min}	θ	J	C_w
	in.	in.	in.	in.	deg	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{4}	in.^{4}	in.^{4}	in.	deg	in.^{4}	in.^{6}
6ZS2.25x105	6.000	2.250	0.105	0.990	50	0.1875	1.25	4.26	6.89	2.30	2.35	1.96	0.692	1.25	2.73	0.745	8.11	0.771	66.0	0.00461	12.3
6ZS2.25x085	6.000	2.250	0.085	0.960	50	0.1875	1.02	3.45	5.63	1.88	2.36	1.57	0.557	1.25	2.21	0.599	6.60	0.768	66.3	0.00244	9.85
6ZS2.25x070	6.000	2.250	0.070	0.930	50	0.1875	0.836	2.84	4.66	1.55	2.36	1.28	0.457	1.24	1.82	0.490	5.46	0.766	66.4	0.00136	8.07
6ZS2.25x065	6.000	2.250	0.065	0.920	50	0.1875	0.776	2.64	4.34	1.45	2.37	1.19	0.424	1.24	1.69	0.455	5.08	0.765	66.5	0.00109	7.48
6ZS2.25x059	6.000	2.250	0.059	0.910	50	0.1875	0.704	2.39	3.95	1.32	2.37	1.08	0.384	1.24	1.53	0.412	4.62	0.764	66.6	0.000817	6.77
4ZS2.25x070	4.000	2.250	0.070	0.930	50	0.1875	0.696	2.37	1.82	0.910	1.62	1.28	0.457	1.36	1.17	0.355	2.75	0.714	51.5	0.00114	3.38
4ZS2.25x065	4.000	2.250	0.065	0.920	50	0.1875	0.646	2.20	1.70	0.848	1.62	1.19	0.424	1.36	1.09	0.329	2.56	0.714	51.6	0.000910	3.14
4ZS2.25x059	4.000	2.250	0.059	0.910	50	0.1875	0.586	1.99	1.55	0.773	1.62	1.08	0.384	1.36	0.986	0.298	2.32	0.713	51.7	0.000680	2.84
3.5ZS1.5x070	3.500	1.500	0.070	0.680	50	0.1875	0.521	1.77	0.985	0.563	1.38	0.396	0.208	0.872	0.472	0.134	1.25	0.508	61.0	0.000850	0.830
3.5ZS1.5x065	3.500	1.500	0.065	0.670	50	0.1875	0.483	1.64	0.919	0.525	1.38	0.367	0.193	0.871	0.438	0.125	1.16	0.508	61.1	0.000681	0.769
3.5ZS1.5x059	3.500	1.500	0.059	0.660	50	0.1875	0.439	1.49	0.838	0.479	1.38	0.331	0.175	0.868	0.398	0.113	1.06	0.507	61.2	0.000509	0.695

Table I - 5

Gross Section Properties

Z-Sections Without Lips



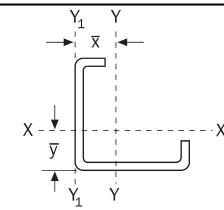


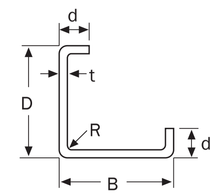
ID	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	I_{xy}	I_{x2}	I_{y2}	r_{min}	θ	J	C_w
	Dimensions						Properties of Full Section												
							Axis x-x			Axis y-y									
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{4}	in.^{4}	in.^{4}	in.	deg	in.^{4}	in.^{6}
8ZU1.25x105	8.000	1.250	0.105	0.1875	1.06	3.60	7.88	1.97	2.73	0.120	0.100	0.337	0.598	0.0745	7.93	0.265	85.6	0.00389	1.55
8ZU1.25x090	8.000	1.250	0.090	0.1875	0.911	3.10	6.82	1.70	2.74	0.105	0.0872	0.340	0.519	0.0651	6.86	0.267	85.6	0.00246	1.35
8ZU1.25x075	8.000	1.250	0.075	0.1875	0.762	2.59	5.73	1.43	2.74	0.0892	0.0735	0.342	0.439	0.0553	5.77	0.269	85.6	0.00143	1.15
8ZU1.25x060	8.000	1.250	0.060	0.1875	0.612	2.08	4.63	1.16	2.75	0.0727	0.0596	0.345	0.356	0.0450	4.66	0.271	85.6	0.000734	0.943
8ZU1.25x048	8.000	1.250	0.048	0.1875	0.491	1.67	3.73	0.933	2.76	0.0590	0.0481	0.347	0.288	0.0365	3.75	0.273	85.5	0.000377	0.767
6ZU1.25x105	6.000	1.250	0.105	0.1875	0.849	2.89	3.78	1.26	2.11	0.120	0.100	0.376	0.446	0.0667	3.84	0.280	83.2	0.00312	0.818
6ZU1.25x090	6.000	1.250	0.090	0.1875	0.731	2.49	3.28	1.09	2.12	0.105	0.0872	0.379	0.388	0.0583	3.33	0.282	83.1	0.00197	0.718
6ZU1.25x075	6.000	1.250	0.075	0.1875	0.612	2.08	2.77	0.922	2.13	0.0892	0.0735	0.382	0.328	0.0495	2.81	0.285	83.1	0.00115	0.612
6ZU1.25x060	6.000	1.250	0.060	0.1875	0.492	1.67	2.24	0.746	2.13	0.0727	0.0596	0.384	0.266	0.0404	2.27	0.287	83.1	0.000590	0.501
6ZU1.25x048	6.000	1.250	0.048	0.1875	0.395	1.34	1.81	0.602	2.14	0.0590	0.0481	0.387	0.216	0.0328	1.83	0.288	83.1	0.000303	0.408
4ZU1.25x090	4.000	1.250	0.090	0.1875	0.551	1.87	1.21	0.603	1.48	0.105	0.0872	0.437	0.257	0.0481	1.26	0.296	77.5	0.00149	0.287
4ZU1.25x075	4.000	1.250	0.075	0.1875	0.462	1.57	1.02	0.510	1.49	0.0892	0.0735	0.439	0.217	0.0410	1.07	0.298	77.5	0.000866	0.245
4ZU1.25x060	4.000	1.250	0.060	0.1875	0.372	1.26	0.829	0.415	1.49	0.0727	0.0596	0.442	0.177	0.0334	0.868	0.300	77.5	0.000446	0.201
4ZU1.25x048	4.000	1.250	0.048	0.1875	0.299	1.02	0.671	0.336	1.50	0.0590	0.0481	0.444	0.143	0.0272	0.703	0.302	77.5	0.000229	0.164
4ZU1.25x036	4.000	1.250	0.036	0.1875	0.225	0.765	0.510	0.255	1.51	0.0449	0.0364	0.447	0.109	0.0207	0.534	0.303	77.5	0.0000972	0.126
3.625ZU1.25x090	3.625	1.250	0.090	0.1875	0.517	1.76	0.950	0.524	1.36	0.105	0.0872	0.451	0.232	0.0455	1.01	0.297	75.6	0.00140	0.228
3.625ZU1.25x075	3.625	1.250	0.075	0.1875	0.434	1.47	0.805	0.444	1.36	0.0892	0.0735	0.453	0.197	0.0388	0.856	0.299	75.6	0.000813	0.195
3.625ZU1.25x060	3.625	1.250	0.060	0.1875	0.349	1.19	0.655	0.361	1.37	0.0727	0.0596	0.456	0.160	0.0317	0.696	0.301	75.6	0.000419	0.161
3.625ZU1.25x048	3.625	1.250	0.048	0.1875	0.281	0.954	0.531	0.293	1.38	0.0590	0.0481	0.458	0.130	0.0258	0.564	0.303	75.6	0.000216	0.131
3.625ZU1.25x036	3.625	1.250	0.036	0.1875	0.212	0.719	0.403	0.222	1.38	0.0449	0.0364	0.461	0.0984	0.0197	0.428	0.305	75.6	0.0000914	0.100
2.5ZU1.25x090	2.500	1.250	0.090	0.1875	0.416	1.41	0.392	0.314	0.971	0.105	0.0872	0.503	0.158	0.0350	0.462	0.290	66.1	0.00112	0.0953
2.5ZU1.25x075	2.500	1.250	0.075	0.1875	0.349	1.19	0.334	0.267	0.978	0.0892	0.0735	0.505	0.134	0.0299	0.393	0.293	66.2	0.000655	0.0819
2.5ZU1.25x060	2.500	1.250	0.060	0.1875	0.282	0.957	0.273	0.218	0.984	0.0727	0.0596	0.508	0.109	0.0245	0.321	0.295	66.2	0.000338	0.0676
2.5ZU1.25x048	2.500	1.250	0.048	0.1875	0.227	0.771	0.222	0.178	0.990	0.0590	0.0481	0.510	0.0888	0.0200	0.261	0.297	66.3	0.000174	0.0554
2.5ZU1.25x036	2.500	1.250	0.036	0.1875	0.171	0.582	0.169	0.136	0.995	0.0449	0.0364	0.512	0.0675	0.0153	0.199	0.299	66.3	0.0000739	0.0426
1.5ZU1.25x090	1.500	1.250	0.090	0.1875	0.326	1.11	0.119	0.159	0.604	0.105	0.0872	0.568	0.0925	0.0193	0.205	0.243	47.2	0.000880	0.0275
1.5ZU1.25x075	1.500	1.250	0.075	0.1875	0.274	0.932	0.102	0.136	0.611	0.0892	0.0735	0.570	0.0788	0.0167	0.175	0.246	47.4	0.000514	0.0239
1.5ZU1.25x060	1.500	1.250	0.060	0.1875	0.222	0.753	0.0845	0.113	0.617	0.0727	0.0596	0.573	0.0645	0.0138	0.143	0.249	47.6	0.000266	0.0199
1.5ZU1.25x048	1.500	1.250	0.048	0.1875	0.179	0.608	0.0693	0.0924	0.623	0.0590	0.0481	0.575	0.0525	0.0113	0.117	0.252	47.8	0.000137	0.0164
1.5ZU1.25x036	1.500	1.250	0.036	0.1875	0.135	0.459	0.0532	0.0710	0.628	0.0449	0.0364	0.577	0.0401	0.00874	0.0894	0.254	48.0	0.0000583	0.0127

Table I - 6

Gross Section Properties

Equal Leg Angles With Lips





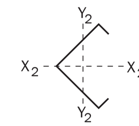
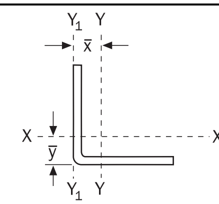
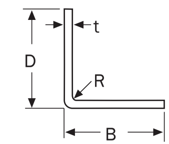
ID	Size		t	d	R	Area	wt/ft	Properties of Full Section												
								Axis x-x and y-y				r_{y1}	J	C_w	r_o				
	D	B						I_x	S_x	r	$\bar{x} = \bar{y}$					I_{y2}	r_{y2}	j	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.^{4}	in.	in.	in.
4LS4x135	4.000	4.000	0.135	0.500	0.1875	1.12	3.79	1.98	0.691	1.33	1.13	1.75	0.00678	0.0907	2.46	0.812	0.853	3.08	-1.58
4LS4x105	4.000	4.000	0.105	0.500	0.1875	0.879	2.99	1.59	0.556	1.35	1.13	1.76	0.00323	0.0791	2.49	0.657	0.864	3.10	-1.60
4LS4x090	4.000	4.000	0.090	0.500	0.1875	0.759	2.58	1.39	0.484	1.35	1.13	1.76	0.00205	0.0717	2.50	0.574	0.870	3.11	-1.61
4LS4x075	4.000	4.000	0.075	0.500	0.1875	0.636	2.16	1.18	0.410	1.36	1.13	1.77	0.00119	0.0631	2.52	0.488	0.875	3.12	-1.62
4LS4x060	4.000	4.000	0.060	0.500	0.1875	0.512	1.74	0.959	0.334	1.37	1.13	1.77	0.000615	0.0533	2.53	0.397	0.881	3.13	-1.63
3LS3x135	3.000	3.000	0.135	0.500	0.1875	0.846	2.88	0.850	0.402	1.00	0.886	1.34	0.00514	0.0479	1.89	0.359	0.652	2.34	-1.25
3LS3x105	3.000	3.000	0.105	0.500	0.1875	0.669	2.28	0.692	0.327	1.02	0.882	1.35	0.00246	0.0420	1.92	0.294	0.663	2.36	-1.27
3LS3x090	3.000	3.000	0.090	0.500	0.1875	0.579	1.97	0.607	0.286	1.02	0.881	1.35	0.00156	0.0382	1.93	0.258	0.668	2.37	-1.28
3LS3x075	3.000	3.000	0.075	0.500	0.1875	0.486	1.65	0.517	0.244	1.03	0.879	1.36	0.000912	0.0337	1.95	0.221	0.674	2.38	-1.29
3LS3x060	3.000	3.000	0.060	0.500	0.1875	0.392	1.33	0.423	0.199	1.04	0.878	1.36	0.000471	0.0286	1.96	0.181	0.679	2.39	-1.30
2.5LS2.5x135	2.500	2.500	0.135	0.500	0.1875	0.711	2.42	0.497	0.286	0.836	0.762	1.13	0.00432	0.0318	1.60	0.216	0.552	1.97	-1.08
2.5LS2.5x105	2.500	2.500	0.105	0.500	0.1875	0.564	1.92	0.408	0.234	0.850	0.759	1.14	0.00207	0.0281	1.63	0.179	0.562	1.98	-1.11
2.5LS2.5x090	2.500	2.500	0.090	0.500	0.1875	0.489	1.66	0.359	0.206	0.857	0.757	1.14	0.00132	0.0256	1.65	0.158	0.568	1.99	-1.12
2.5LS2.5x075	2.500	2.500	0.075	0.500	0.1875	0.411	1.40	0.307	0.176	0.864	0.755	1.15	0.000771	0.0226	1.66	0.135	0.573	2.00	-1.13
2.5LS2.5x060	2.500	2.500	0.060	0.500	0.1875	0.332	1.13	0.252	0.144	0.871	0.754	1.15	0.000399	0.0192	1.68	0.111	0.579	2.01	-1.14
2LS2x135	2.000	2.000	0.135	0.500	0.1875	0.576	1.96	0.257	0.189	0.668	0.639	0.924	0.00350	0.0192	1.32	0.118	0.452	1.58	-0.925
2LS2x105	2.000	2.000	0.105	0.500	0.1875	0.459	1.56	0.213	0.156	0.681	0.636	0.932	0.00169	0.0171	1.35	0.0984	0.463	1.60	-0.946
2LS2x090	2.000	2.000	0.090	0.500	0.1875	0.399	1.36	0.189	0.138	0.688	0.634	0.936	0.00108	0.0156	1.37	0.0874	0.468	1.61	-0.957
2LS2x075	2.000	2.000	0.075	0.500	0.1875	0.336	1.14	0.163	0.119	0.695	0.632	0.940	0.000631	0.0139	1.38	0.0754	0.473	1.62	-0.968
2LS2x060	2.000	2.000	0.060	0.500	0.1875	0.272	0.926	0.134	0.0980	0.702	0.631	0.943	0.000327	0.0118	1.39	0.0624	0.479	1.63	-0.979

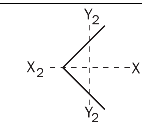
Table I - 7

Gross Section Properties

Equal Leg Angles Without Lips





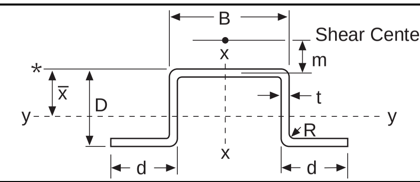


ID	Size		t	R	Area	wt/ft	Properties of Full Section: Axis x-x and Axis y-y				r_{y1}	J	C_w	r_o				
	D	B					I_x	S_x	r	$\bar{x} = \bar{y}$					I_{y2}	r_{y2}	j	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.^{4}	in.	in.	in.
4LU4x135	4.000	4.000	0.135	0.1875	1.047	3.56	1.69	0.577	1.27	1.07	1.66	0.00636	0.000	2.29	0.653	0.790	2.87	-1.41
4LU4x105	4.000	4.000	0.105	0.1875	0.818	2.78	1.33	0.453	1.28	1.05	1.66	0.00301	0.000	2.29	0.515	0.794	2.87	-1.42
4LU4x090	4.000	4.000	0.090	0.1875	0.703	2.39	1.15	0.390	1.28	1.05	1.65	0.00190	0.000	2.30	0.445	0.796	2.88	-1.42
4LU4x075	4.000	4.000	0.075	0.1875	0.587	2.00	0.964	0.326	1.28	1.04	1.65	0.00110	0.000	2.30	0.374	0.798	2.88	-1.42
4LU4x060	4.000	4.000	0.060	0.1875	0.471	1.60	0.776	0.262	1.28	1.04	1.65	0.000565	0.000	2.31	0.301	0.800	2.88	-1.42
3LU3x135	3.000	3.000	0.135	0.1875	0.777	2.64	0.700	0.321	0.949	0.815	1.25	0.00472	0.000	1.71	0.266	0.585	2.16	-1.06
3LU3x105	3.000	3.000	0.105	0.1875	0.608	2.07	0.554	0.252	0.954	0.803	1.25	0.00224	0.000	1.72	0.211	0.589	2.17	-1.06
3LU3x090	3.000	3.000	0.090	0.1875	0.523	1.78	0.478	0.217	0.956	0.797	1.25	0.00141	0.000	1.72	0.183	0.591	2.17	-1.06
3LU3x075	3.000	3.000	0.075	0.1875	0.437	1.49	0.402	0.182	0.959	0.791	1.24	0.000820	0.000	1.73	0.154	0.593	2.17	-1.07
3LU3x060	3.000	3.000	0.060	0.1875	0.351	1.19	0.324	0.146	0.961	0.785	1.24	0.000421	0.000	1.73	0.124	0.595	2.18	-1.07
2.5LU2.5x135	2.500	2.500	0.135	0.1875	0.642	2.18	0.399	0.220	0.788	0.691	1.05	0.00390	0.000	1.42	0.149	0.482	1.81	-0.881
2.5LU2.5x105	2.500	2.500	0.105	0.1875	0.503	1.71	0.316	0.174	0.793	0.678	1.04	0.00185	0.000	1.43	0.119	0.486	1.82	-0.885
2.5LU2.5x090	2.500	2.500	0.090	0.1875	0.433	1.47	0.274	0.150	0.795	0.672	1.04	0.00117	0.000	1.43	0.103	0.488	1.82	-0.887
2.5LU2.5x075	2.500	2.500	0.075	0.1875	0.362	1.23	0.230	0.126	0.797	0.666	1.04	0.000679	0.000	1.44	0.0871	0.491	1.82	-0.889
2.5LU2.5x060	2.500	2.500	0.060	0.1875	0.291	0.989	0.186	0.101	0.800	0.660	1.04	0.000349	0.000	1.44	0.0705	0.493	1.82	-0.891
2LU2x135	2.000	2.000	0.135	0.1875	0.507	1.72	0.199	0.139	0.626	0.566	0.844	0.00308	0.000	1.13	0.0731	0.380	1.46	-0.705
2LU2x105	2.000	2.000	0.105	0.1875	0.398	1.35	0.159	0.110	0.631	0.554	0.840	0.00146	0.000	1.14	0.0586	0.384	1.47	-0.709
2LU2x090	2.000	2.000	0.090	0.1875	0.343	1.17	0.138	0.0947	0.633	0.548	0.837	0.000926	0.000	1.14	0.0511	0.386	1.47	-0.711
2LU2x075	2.000	2.000	0.075	0.1875	0.287	0.976	0.116	0.0796	0.636	0.541	0.835	0.000538	0.000	1.15	0.0432	0.388	1.47	-0.713
2LU2x060	2.000	2.000	0.060	0.1875	0.231	0.785	0.0940	0.0642	0.638	0.535	0.833	0.000277	0.000	1.15	0.0351	0.390	1.47	-0.715

Table I - 8

Gross Section Properties

Hat Sections Without Lips



* Note vertical orientation of x-axis.

ID	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\overline{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
10HU5x075	10.000	5.000	0.075	1.050	0.1875	1.98	6.74	11.0	3.15	2.35	23.7	4.28	3.46	4.46	4.80	0.00371	98.4	9.70	10.1	-9.22
8HU12x135	8.000	12.000	0.135	1.670	0.1875	4.10	13.9	111	14.8	5.21	36.3	7.09	2.98	2.88	3.63	0.0249	762	9.06	8.80	-6.44
8HU12x105	8.000	12.000	0.105	1.340	0.1875	3.13	10.7	83.6	11.6	5.17	26.8	5.13	2.93	2.77	3.59	0.0115	597	8.93	8.66	-6.31
8HU8x105	8.000	8.000	0.105	1.340	0.1875	2.71	9.23	35.2	6.73	3.60	23.3	4.83	2.93	3.19	3.74	0.00997	222	8.17	8.29	-6.87
8HU8x075	8.000	8.000	0.075	0.980	0.1875	1.90	6.44	24.1	4.92	3.57	15.6	3.16	2.87	3.05	3.71	0.00355	161	8.05	8.14	-6.73
8HU4x075	8.000	4.000	0.075	0.980	0.1875	1.60	5.42	5.69	1.96	1.89	12.4	2.82	2.78	3.62	3.81	0.00299	31.9	7.77	8.12	-7.39
8HU4x060	8.000	4.000	0.060	0.840	0.1875	1.26	4.30	4.47	1.61	1.88	9.66	2.18	2.77	3.56	3.84	0.00152	25.8	7.76	8.10	-7.37
6HU9x135	6.000	9.000	0.135	1.670	0.1875	3.15	10.7	49.7	8.24	3.97	16.4	4.43	2.28	2.29	2.73	0.0192	177	6.97	6.75	-4.95
6HU9x105	6.000	9.000	0.105	1.340	0.1875	2.40	8.16	36.8	6.42	3.92	12.1	3.16	2.24	2.19	2.73	0.00882	140	6.85	6.63	-4.86
6HU6x105	6.000	6.000	0.105	1.340	0.1875	2.08	7.09	15.7	3.71	2.74	10.4	2.98	2.24	2.51	2.78	0.00766	51.6	6.21	6.32	-5.23
6HU6x075	6.000	6.000	0.075	0.915	0.1875	1.44	4.88	10.4	2.70	2.69	6.80	1.87	2.18	2.36	2.80	0.00269	37.7	6.11	6.18	-5.12
6HU3x075	6.000	3.000	0.075	0.915	0.1875	1.21	4.12	2.48	1.06	1.43	5.36	1.67	2.11	2.79	2.79	0.00227	7.55	5.81	6.10	-5.54
6HU3x060	6.000	3.000	0.060	0.760	0.1875	0.954	3.24	1.92	0.872	1.42	4.15	1.27	2.09	2.72	2.85	0.00115	6.04	5.83	6.09	-5.55
6HU3x048	6.000	3.000	0.048	0.660	0.1875	0.757	2.57	1.51	0.714	1.41	3.25	0.979	2.07	2.68	2.88	0.000581	4.88	5.83	6.08	-5.54
4HU6x135	4.000	6.000	0.135	1.670	0.1875	2.21	7.51	16.9	3.72	2.76	5.42	2.34	1.57	1.69	1.74	0.0134	22.6	4.81	4.63	-3.36
4HU6x105	4.000	6.000	0.105	1.340	0.1875	1.66	5.66	12.0	2.85	2.69	3.96	1.64	1.54	1.59	1.80	0.00612	17.9	4.74	4.56	-3.34
4HU4x105	4.000	4.000	0.105	1.340	0.1875	1.45	4.94	5.31	1.64	1.91	3.39	1.55	1.53	1.81	1.73	0.00534	6.89	4.17	4.26	-3.49
4HU4x075	4.000	4.000	0.075	0.915	0.1875	0.986	3.35	3.30	1.16	1.83	2.18	0.937	1.49	1.68	1.85	0.00185	4.83	4.15	4.21	-3.49
4HU2x075	4.000	2.000	0.075	0.915	0.1875	0.836	2.84	0.830	0.451	0.997	1.70	0.839	1.43	1.97	1.65	0.00157	1.15	3.74	3.99	-3.59
4HU2x060	4.000	2.000	0.060	0.750	0.1875	0.653	2.22	0.617	0.365	0.972	1.30	0.623	1.41	1.91	1.78	0.000784	0.829	3.83	4.05	-3.66
4HU2x048	4.000	2.000	0.048	0.618	0.1875	0.513	1.74	0.469	0.299	0.956	1.00	0.468	1.40	1.86	1.86	0.000394	0.640	3.88	4.06	-3.69
3HU4.5x135	3.000	4.500	0.135	1.670	0.1875	1.74	5.90	8.28	2.19	2.18	2.47	1.52	1.19	1.37	1.18	0.0105	5.66	3.69	3.52	-2.49
3HU4.5x105	3.000	4.500	0.105	1.340	0.1875	1.30	4.41	5.69	1.63	2.10	1.80	1.05	1.18	1.29	1.29	0.00477	4.20	3.64	3.48	-2.52
3HU3x105	3.000	3.000	0.105	1.340	0.1875	1.14	3.87	2.62	0.956	1.52	1.53	0.992	1.16	1.46	1.13	0.00419	1.90	3.08	3.17	-2.53
3HU3x075	3.000	3.000	0.075	0.915	0.1875	0.761	2.59	1.52	0.648	1.41	0.977	0.585	1.13	1.33	1.33	0.00143	1.14	3.13	3.19	-2.62

2.4 Steel Deck

Steel decks are structural products that are designed to resist roof or floor loads. Usual deck products are roof deck, non-composite floor deck (form deck) and composite floor deck. The structural properties (strength and stiffness) are obtained from the applicable Steel Deck Institute (SDI) Standard for that product:

ANSI/SDI RD-2010 *Standard for Steel Roof Deck*
ANSI/SDI NC-2010 *Standard for Non-Composite Steel Floor Deck*
ANSI/SDI C-2011 *Standard for Composite Steel Floor Deck-Slabs*

The SDI Standards reference the use of the *Specification* for calculation of strength and stiffness, and for steel material specifications. Design is permitted by either the Allowable Strength Design (ASD) or Load and Resistance Factor Design (LRFD) methods. In addition to strength and stiffness, the SDI Standards provide additional structural design and construction requirements, including construction loads, that are necessary for proper deck design and installation. The SDI Standards are adopted by reference into the International Building Code.

In addition to supporting gravity and wind uplift loads, steel deck can also serve as a structural diaphragm to resist in-plane loads due to wind or seismic loading.

One important item to consider for all deck products is the required service life and the environment in which the deck product must exist. For instance, if a roof deck is to have insulation board applied with fasteners that penetrate the deck, then consideration should be given to corrosion of the fasteners and the deck. The same reasoning applies to form deck and floor deck in humid areas or areas subject to water or salt application. Insurance requirements and fire ratings can also affect the finish selection.

2.4.1 Deck Profiles

Figure 2.4-1 shows cross-sections of industry standard steel deck profiles. These profiles are merely representative of what is available. Consult the literature of manufacturers to obtain further information about these and other available deck profiles. The different profiles lend themselves to different uses:

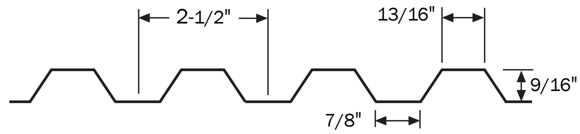
(a) Form Deck: Form deck is commonly used to span between floor joists to serve as form work for cast-in-place concrete floor systems. The 9⁄16 inch deep profile shown is the shallowest available. Deeper profiles are available for longer spans and heavier loads. Flutes may be trapezoidal as shown or sinusoidal.

(b) Narrow Rib Deck (NR): Narrow rib deck is used as roof decking in climatic areas where minimal roof insulation is required. The narrow rib provides a small rib opening that thin rigid insulation can bridge. Narrow rib deck is the least efficient structurally but has the advantage of easy attachment of roofing materials since a high percentage of the material is in the roof plane.

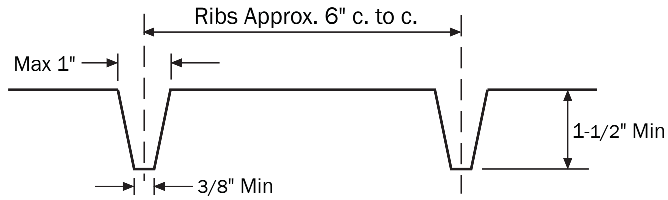
(c) Intermediate Rib Deck (IR): Intermediate rib deck is used as roof decking. It is somewhat more efficient structurally than narrow rib deck but requires thicker insulation to span the wider rib opening.

(d) Wide Rib Deck (WR): Wide rib deck is used as roof deck where insulation can span the wide rib openings. Because of energy demands, this is the most common profile. It is the most structurally efficient cross section of the 1½ inch roof deck profiles.

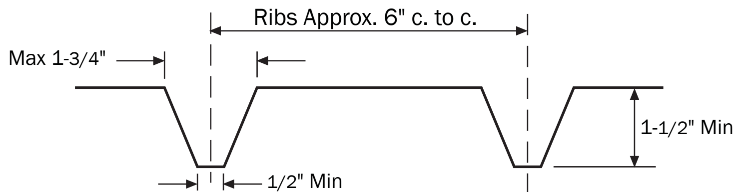
(e) Deep Rib Deck (3DR): Deep rib deck is used where long spans between joists or purlins occur and/or the deck spans are subject to larger loads.



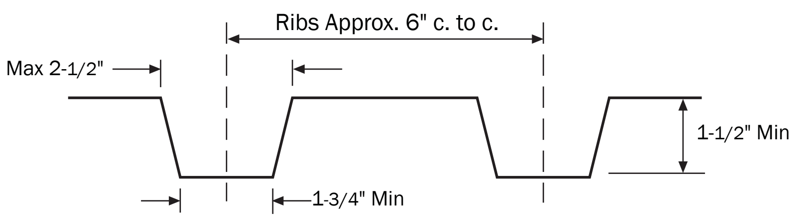
(a) 9/16" Form Deck (Representative)



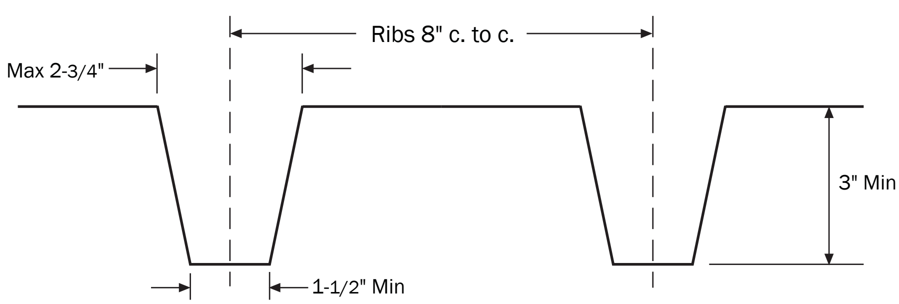
(b) Narrow Rib Deck Type NR



(c) Intermediate Rib Deck Type IR



(d) Wide Rib Deck Type WR



(e) Deep Rib Deck Type 3DR

Figure 2.4-1 Deck Profiles

In addition to these industry standard profiles, manufacturers produce proprietary steel deck profiles that provide special functional enhancements, such as acoustical absorption capability and cellular raceway features.

The thickness of steel deck is commonly designated by gage. The equivalent design thicknesses given in Section 2.4.3 are the minimum allowed at each designated gage by the Steel Deck Institute (SDI). Actual material thicknesses provided by manufacturers vary. Steel must conform to *Specification* Section A2.

2.4.2 Maximum Spans

Recommended ASD Maximum Spans for Construction and Maintenance Loads

F_y = 33 ksi

Type*		Span Condition	ASD Maximum Recommended Spans	
			Span ft-in.	Roof Deck Cantilever ft-in.
Narrow Rib Deck	NR22 NR22	1 2 or more	2′-11″ 3′-7″	0′-10″
	NR20 NR20	1 2 or more	3′-11″ 4′-10″	1′-0″
	NR18 NR18	1 2 or more	5′-0″ 6′-2″	1′-3″
	NR16 NR16	1 2 or more	6′-5″ 7′-11″	1′-7″
Intermediate Rib Deck	IR22 IR22	1 2 or more	3′-5″ 4′-3″	0′-11″
	IR20 IR20	1 2 or more	4′-3″ 5′-3″	1′-1″
	IR18 IR18	1 2 or more	5′-10″ 7′-2″	1′-6″
	IR16 IR16	1 2 or more	7′-6″ 9′-3″	1′-10″
Wide Rib Deck	WR22 WR22	1 2 or more	5′-8″ 6′-11″	1′-6″
	WR20 WR20	1 2 or more	7′-0″ 8′-7″	1′-10″
	WR18 WR18	1 2 or more	9′-6″ 11′-8″	2′-5″
	WR16 WR16	1 2 or more	12′-2″ 15′-0″	3′-0″
Deep Rib Deck	3DR22 3DR22	1 2 or more	11′-11″ 14′-7″	3′-4″
	3DR20 3DR20	1 2 or more	15′-4″ 18′-11″	4′-2″
	3DR18 3DR18	1 2 or more	21′-1″ 26′-0″	5′-7″
	3DR16 3DR16	1 2 or more	27′-5″ 33′-9″	7′-1″

* Deck section properties are provided in Section 2.4.3

2.4.3 Section Properties

The Steel Deck Institute (SDI) used the most conservative combinations of the dimensions for each roof deck profile shown in Figure 2.4-1 (with the exception of the form deck) to calculate the section properties listed in the table below. The values are therefore not representative of any one manufacturer but represent the lowest value that might occur. As a result, load tables based on these properties are conservative. Form deck profiles vary greatly and their profiles are not established by SDI. The form deck profile shown is an actual manufacturer's product. These properties can be considered as representative, but actual properties may be higher or lower.

In the table below, S_t is the effective section modulus for compression at the top; S_b is the effective section modulus for compression at the bottom; and I is the moment of inertia at $0.6F_y$. The weight provided is the dead weight of the deck for use in the load combinations and should not be used as a basis for ordering. The values given are based on steel with yield stress of 33 ksi, with the exception of the form deck values, which are based on 80 ksi.

Steel Deck - Section Properties

Type	Common Designation	Design Thickness in.	Weight lb/ft² Painted	Weight lb/ft² Galvanized	I in.⁴/ft	S_t in.³/ft	S_b in.³/ft
9/16″ Form Deck	28 gage	0.0149	0.8	0.9	0.012	0.036	0.037
	26 gage	0.0179	0.9	1.0	0.015	0.046	0.047
	24 gage	0.0239	1.2	1.3	0.020	0.065	0.064
	22 gage	0.0295	1.5	1.6	0.025	0.080	0.079
1½″ Narrow Rib Deck	NR22	0.0295	1.6	1.7	0.099	0.089	0.098
	NR20	0.0358	2.0	2.1	0.138	0.118	0.118
	NR18	0.0474	2.6	2.7	0.181	0.152	0.157
	NR16	0.0598	3.3	3.4	0.233	0.196	0.198
1½″ Intermediate Rib Deck	IR22	0.0295	1.6	1.7	0.110	0.105	0.113
	IR20	0.0358	2.0	2.1	0.141	0.130	0.137
	IR18	0.0474	2.6	2.7	0.200	0.177	0.181
	IR16	0.0598	3.3	3.4	0.254	0.227	0.228
1½″ Wide Rib Deck	WR22	0.0295	1.7	1.8	0.147	0.171	0.180
	WR20	0.0358	2.1	2.2	0.191	0.212	0.225
	WR18	0.0474	2.8	2.9	0.274	0.288	0.296
	WR16	0.0598	3.5	3.6	0.353	0.370	0.372
3″ Deep Rib Deck	3DR22	0.0295	2.1	2.2	0.602	0.361	0.410
	3DR20	0.0358	2.5	2.6	0.781	0.466	0.515
	3DR18	0.0474	3.3	3.4	1.142	0.641	0.692
	3DR16	0.0598	4.5	4.6	1.555	0.833	0.882

SECTION 3 - CALCULATION OF SECTION PROPERTIES

3.1 Linear Method for Computing Properties of Formed Sections

Computation of properties of formed sections may be simplified by using a so-called linear method, in which the material of the section is considered concentrated along the centerline of the steel sheet and the area elements replaced by straight or curved "line elements." The thickness dimension, t, is introduced after the linear computations have been completed.

The total area of the section is found from the relation: $\text{Area} = \text{Lt}$, where L is the total length of all line elements.

The moment of inertia of the section, I, is found from the relation: $I = I't$, where I' is the moment of inertia of the centerline of the steel sheet. The section modulus is computed as usual by dividing I or $I't$ by the distance from the neutral axis to *the extreme fiber*, not to the centerline of the extreme element.

First power dimensions, such as x, y, and r (radius of gyration) are obtained directly by the linear method and do not involve the thickness dimension.

When the flat width, w, of a stiffened compression element is reduced for design purposes, the effective design width, b, is used directly to compute the total effective length L_{eff} of the line elements.

The elements into which most sections may be divided for application of the linear method consist of straight lines and circular arcs. For convenient reference, the moments of inertia and location of centroid of such elements are identified in the sketches and equations in Section 3.2.

The equations for line elements are exact, since the line as such has no thickness dimension; but in computing the properties of an actual section, where the line element represents an actual element with a thickness dimension, the results will be approximate for the following reasons:

(1) The moment of inertia of a straight actual element about its longitudinal axis is considered negligible.

(2) The moment of inertia of a straight actual element inclined to the axes of reference is slightly larger than that of the corresponding line element, but for elements of like length the error involved is even less than the error involved in neglecting the moment of inertia of the element about its longitudinal axis. Obviously, the error disappears when the element is normal to the axis.

(3) Small errors are involved in using the properties of a linear arc to find those of an actual corner, but with the usual small corner radii the error in the location of the centroid of the corner is of little importance, and the moment of inertia generally negligible. When the mean radius of a circular element is over four times its thickness, as for tubular sections and for sheets with circular corrugations, the errors in using linear arc properties practically disappear.

Using the computed values of I_x, I_y, and I_{xy} the moment of inertia about principal axes of the section can be calculated by the following equation:

$$I_{\substack{Max,\\Min}} = \frac{I_x + I_y}{2} \pm \sqrt{\left(\frac{I_x - I_y}{2}\right)^2 + I_{xy}{}^2}$$

where I_x and I_y are the moment of inertia of the section about x- and y-axis, respectively, and I_{xy} is the product of inertia.

The angle between the x-axis and the minor principal axis is

$$\theta = \frac{\pi}{2} + \frac{1}{2}\tan^{-1}\left[\frac{2I_{xy}}{I_y - I_x}\right] \quad \text{(in radians)}$$

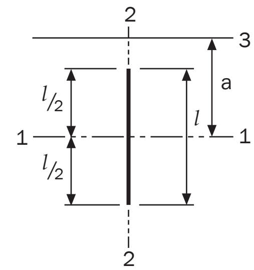
3.2 Properties of Line Elements

3.2.1 Straight Line Elements

Moments of inertia of straight line elements can be calculated using the equations given below:

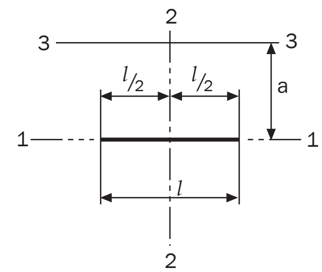
$$I_1 = \frac{l^3}{12} \qquad I_2 = 0$$

$$I_3 = la^2 + \frac{l^3}{12} = l\left(a^2 + \frac{l^2}{12}\right)$$



$$I_1 = 0 \qquad I_2 = \frac{l^3}{12}$$

$$I_3 = la^2$$

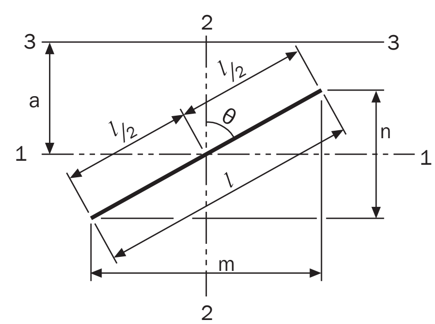


$$I_1 = \left[\frac{\cos^2\theta}{12}\right]l^3 = \frac{ln^2}{12}$$

$$I_2 = \left[\frac{\sin^2\theta}{12}\right]l^3 = \frac{lm^2}{12}$$

$$I_{12} = \left[\frac{\sin\theta\cos\theta}{12}\right]l^3 = \frac{lmn}{12}$$

$$I_3 = la^2 + \frac{ln^2}{12} = l\left(a^2 + \frac{n^2}{12}\right)$$



3.2.2 Circular Line Elements

Moments of inertia of circular line elements can be calculated using the equations given below:

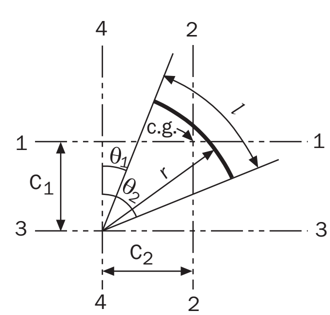
R = inside radius
r = median radius

$$\theta \text{ (expressed in radians)} = \frac{\pi\theta}{180} \text{ (expressed in degrees)}$$

$$l = (\theta_2 - \theta_1)r$$

$$c_1 = \frac{\sin\theta_2 - \sin\theta_1}{\theta_2 - \theta_1}r \qquad c_2 = \frac{\cos\theta_1 - \cos\theta_2}{\theta_2 - \theta_1}r$$

$$I_1 = \left[\frac{\theta_2 - \theta_1 + \sin\theta_2\cos\theta_2 - \sin\theta_1\cos\theta_1}{2} - \frac{(\sin\theta_2 - \sin\theta_1)^2}{\theta_2 - \theta_1}\right]r^3$$



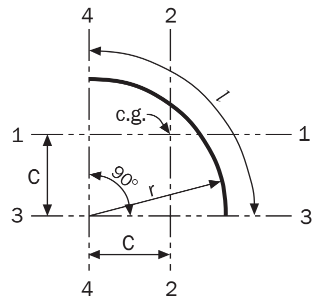
$$I_2 = \left[\frac{\theta_2-\theta_1-\sin\theta_2\cos\theta_2+\sin\theta_1\cos\theta_1}{2}-\frac{(\cos\theta_1-\cos\theta_2)^2}{\theta_2-\theta_1}\right]r^3$$

$$I_{12} = \left[\frac{\sin^2\theta_2-\sin^2\theta_1}{2}+\frac{(\sin\theta_2-\sin\theta_1)(\cos\theta_2-\cos\theta_1)}{\theta_2-\theta_1}\right]r^3$$

$$I_3 = \left[\frac{\theta_2-\theta_1+\sin\theta_2\cos\theta_2-\sin\theta_1\cos\theta_1}{2}\right]r^3$$

$$I_4 = \left[\frac{\theta_2-\theta_1-\sin\theta_2\cos\theta_2+\sin\theta_1\cos\theta_1}{2}\right]r^3 \qquad I_{34} = \left[\frac{\sin^2\theta_2-\sin^2\theta_1}{2}\right]r^3$$

Case I: $\theta_1 = 0; \quad \theta_2 = 90°$



$l = \pi r/2 = 1.57r$

$c = 0.637r$

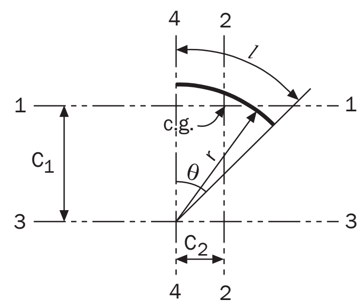
$I_1 = I_2 = 0.149r^3$

$I_{12} = -0.137r^3$

$I_3 = I_4 = 0.785r^3$

$I_{34} = 0.5r^3$

Case II: $\theta_1 = 0; \quad \theta_2 = \theta$



$l = \theta r$

$$c_1 = \frac{r\sin\theta}{\theta}$$

$$c_2 = \frac{r(1\text{-}\cos\theta)}{\theta}$$

$$I_1 = \left[\frac{\theta+\sin\theta\cos\theta}{2}-\frac{\sin^2\theta}{\theta}\right]r^3 \qquad I_2 = \left[\frac{\theta-\sin\theta\cos\theta}{2}-\frac{(1-\cos\theta)^2}{\theta}\right]r^3$$

$$I_{12} = \left[\frac{\sin^2\theta}{2}+\frac{\sin\theta(\cos\theta-1)}{\theta}\right]r^3$$

$$I_3 = \left[\frac{\theta+\sin\theta\cos\theta}{2}\right]r^3 \qquad I_4 = \left[\frac{\theta-\sin\theta\cos\theta}{2}\right]r^3$$

$$I_{34} = \left[\frac{\sin^2\theta}{2}\right]r^3$$

3.3 Properties of Sections

Section properties of some sections can be calculated using the equations given below. The following are to be noted:

(1) Three different types of dimensions are used: capital letters (A) for outside dimensions, lower case barred letters ($\bar{a}$) for centerline dimensions, lower case letters (a) for flat dimensions. The flat dimensions are required to obtain properties such as moment of inertia, I, where corners are assumed to be round. The centerline dimensions are needed for torsional properties, such as C_w, where corners are assumed to be square. The outside dimensions are shown because they are the dimensions usually given in tables.

(2) All expressions consider the sections to contain round corners with the exception of those for some torsional properties (m, j and C_w). These expressions are based on a

square corner approximation with the exception that round corner values are used for quantities such as area and moment of inertia which appear in the torsional property expressions. However, nominal stresses calculated by this procedure are sufficiently accurate for routine engineering design of sections with small ratios of corner radius to thickness.

(3) In the moment of inertia calculations, all quantities are accounted for except the moment of inertia of a flat element about its own axis when this is the weak axis. Moments of inertia of corners about their own axis are included to provide for the case of sections with large corner radii.

(4) All expressions are given for the full, unreduced sections.

3.3.1 Equal Leg Angles (Singly-Symmetric) With and Without Lips

1. Basic parameters

$$a = A' - \left[r + t/2 + \alpha\left(r + t/2\right)\right]^*$$

$$\bar{a} = A' - \left[t/2 + \alpha t/2\right]$$

$$c = \alpha\left[C' - \left(r + t/2\right)\right]$$

$$\bar{c} = \alpha\left[C' - t/2\right]$$

$$u = \pi r/2 = 1.57r$$

2. Cross-sectional area

$$A = t\left[2a + u + \alpha\left(2c + 2u\right)\right]$$

3. Distance between centroid and centerlines of webs

$$\bar{x}_c = \bar{y}_c = \frac{t}{A}\left\{a\left(\frac{a}{2} + r\right) + u\left(0.363r\right) + \alpha\left[c\left(a + \frac{c}{2} + 3r\right) + u\left(a + 2r\right)\right]\right\}$$

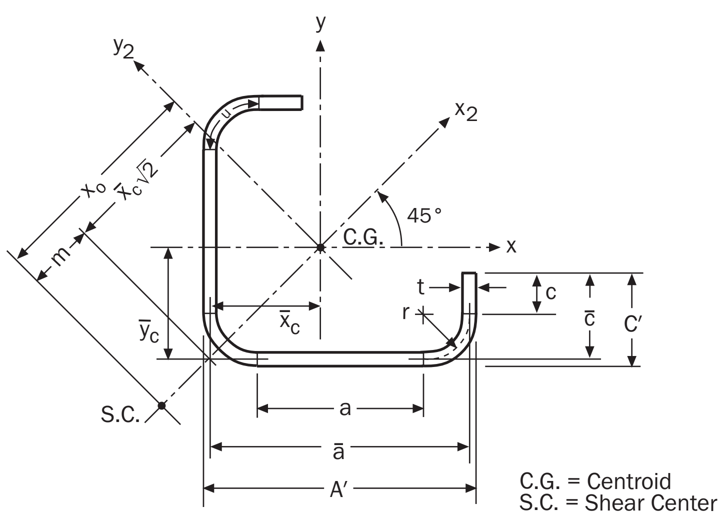


Figure 3.3.1-1
Equal Leg Angle (Singly Symmetric) With Lips

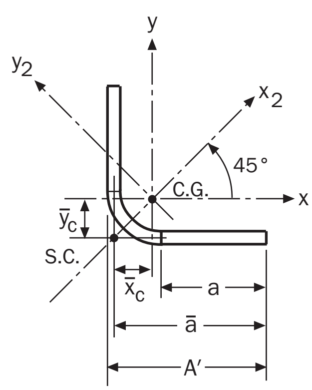


Figure 3.3.1-2
Equal Leg Angle (Singly Symmetric) Without Lips

* For sections with lips, $\alpha = 1.0$; for sections without lips, $\alpha = 0$.

4. Distance between centroid and outside of webs

$$\bar{x} = \bar{y} = \bar{x}_c + \frac{t}{2}$$

5. Moment of inertia about x and y axes

$$I_x = I_y = t\left\{\begin{array}{l} a\left(\frac{a}{2}+r\right)^2 + \frac{a^3}{12} + u(0.363r)^2 + 0.149r^3 \\ +\alpha\left[\begin{array}{l} c(a+2r)^2 + \frac{c^3}{12} + c\left(\frac{c}{2}+r\right)^2 + u(a+1.637r)^2 \\ +u(0.363r)^2 + (2)(0.149)r^3 \end{array}\right]\end{array}\right\} - A\bar{x}_c^2$$

6. Product of inertia about x and y axes

$$I_{xy} = t\left\{\begin{array}{l} -0.137r^3 + u(0.363r)^2 \\ +2\alpha\left[c(a+2r)\left(\frac{c}{2}+r\right) + 0.137r^3 + u(a+1.637r)(0.363r)\right]\end{array}\right\} - A\bar{x}_c\bar{y}_c$$

7. Moment of inertia about y_2-axis

$$I_{y2} = I_x + I_{xy}$$

8. Distance between shear center and centerline of square corner

$$m = \frac{\bar{a}\bar{c}^2\sqrt{2}}{2}\frac{(3\bar{a}-2\bar{c})}{\left[2\bar{a}^3 - (\bar{a}-\bar{c})^3\right]}$$

9. St. Venant torsion constant

$$J = \frac{t^3}{3}\left[2a + u + \alpha(2c+2u)\right]$$

10. Warping constant

$$C_w = \frac{\bar{a}^4\bar{c}^3 t}{6}\frac{(4\bar{a}+3\bar{c})}{\left[2\bar{a}^3 - (\bar{a}-\bar{c})^3\right]}$$

11. Distance from centroid to shear center*

$$x_o = -\left(\bar{x}_c\sqrt{2} + m\right)$$

12. Parameter used to determine elastic critical moment

$$j = \frac{\sqrt{2}t}{48I_{y2}}\left(\bar{a}^4 + 4\bar{a}^3\bar{c} - 6\bar{a}^2\bar{c}^2 + \bar{c}^4\right) - x_o$$

* Negative sign indicates x_o is measured in negative x_2 direction.

3.3.2 C-Sections (Singly-Symmetric) With and Without Lips and Hat Sections (Singly-Symmetric)

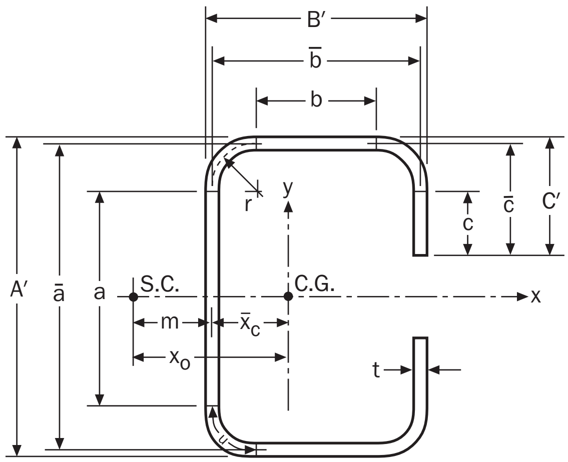


Figure 3.3.2-1
C-Section (Singly Symmetric) With Lips

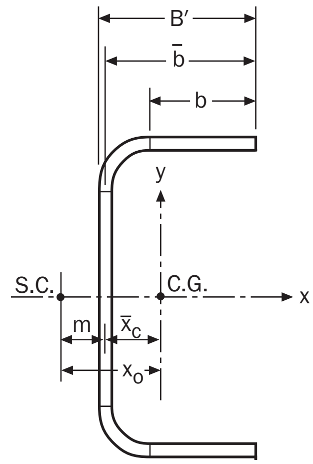
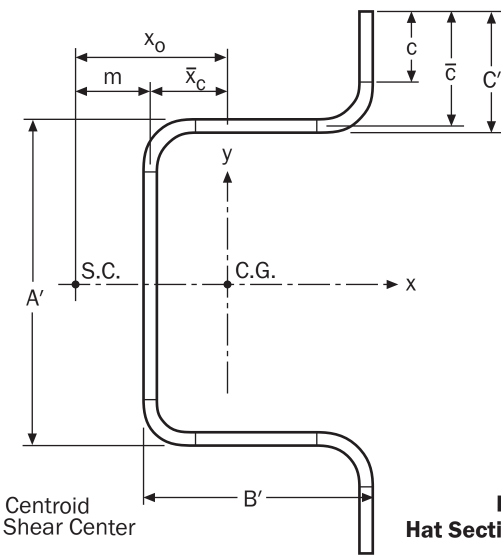


Figure 3.3.2-2
C-Section (Singly Symmetric) Without Lips



C.G. = Centroid
S.C. = Shear Center

Figure 3.3.2-3
Hat Section (Singly Symmetric)

1. Basic parameters

$a = A' - (2r + t)$

$\bar{a} = A' - t$

$b = B' - [r + t/2 + \alpha(r + t/2)]$*

$\bar{b} = B' - (t/2 + \alpha t/2)$

$c = \alpha[C' - (r + t/2)]$

$\bar{c} = \alpha(C' - t/2)$

$u = \pi r/2 = 1.57r$

2. Cross-sectional area

$A = t[a + 2b + 2u + \alpha(2c + 2u)]$

* For C-Sections with lips and Hat Sections, α = 1.0; for C-Sections without lips, α = 0.

3. Moment of inertia about x-axis

a) C-Section:

$$I_x = 2t\left\{\begin{array}{l} 0.0417a^3 + b\left(\frac{a}{2}+r\right)^2 + u\left(\frac{a}{2}+0.637r\right)^2 + 0.149r^3 \\ +\alpha\left[0.0833c^3 + \frac{c}{4}(a-c)^2 + u\left(\frac{a}{2}+0.637r\right)^2 + 0.149r^3\right] \end{array}\right\}$$

b) Hat Section:

$$I_x = 2t\left\{\begin{array}{l} 0.0417a^3 + b\left(\frac{a}{2}+r\right)^2 + u\left(\frac{a}{2}+0.637r\right)^2 + 0.149r^3 \\ +\alpha\left[0.0833c^3 + \frac{c}{4}(a+c+4r)^2 + u\left(\frac{a}{2}+1.363r\right)^2 + 0.149r^3\right] \end{array}\right\}$$

4. Distance between centroid and web centerline

$$\bar{x}_c = \frac{2t}{A}\left\{b\left(\frac{b}{2}+r\right) + u(0.363r) + \alpha\left[u(b+1.637r) + c(b+2r)\right]\right\}$$

5. Distance between centroid and outside of web

$$\bar{x} = \bar{x}_c + \frac{t}{2}$$

6. Moment of inertia about y-axis

$$I_y = 2t\left\{\begin{array}{l} b\left(\frac{b}{2}+r\right)^2 + \frac{b^3}{12} + 0.356r^3 \\ +\alpha\left[c(b+2r)^2 + u(b+1.637r)^2 + 0.149r^3\right] \end{array}\right\} - A\bar{x}_c^{\,2}$$

7. Distance between shear center and web centerline

a) C-Section:

$$m = \bar{b}\left[\frac{3\bar{a}^2\bar{b} + \alpha\bar{c}\left(6\bar{a}^2 - 8\bar{c}^2\right)}{\bar{a}^3 + 6\bar{a}^2\bar{b} + \alpha\bar{c}\left(8\bar{c}^2 - 12\overline{ac} + 6\bar{a}^2\right)}\right]$$

b) Hat Section:

$$m = \bar{b}\left[\frac{3\bar{a}^2\bar{b} + \alpha\bar{c}\left(6\bar{a}^2 - 8\bar{c}^2\right)}{\bar{a}^3 + 6\bar{a}^2\bar{b} + \alpha\bar{c}\left(8\bar{c}^2 + 12\overline{ac} + 6\bar{a}^2\right)}\right]$$

8. Distance between centroid and shear center

$x_o = -(\bar{x}_c + m)$*

9. St. Venant torsion constant

$$J = \frac{t^3}{3}\left[a + 2b + 2u + \alpha(2c + 2u)\right]$$

* Negative sign indicates x_o is measured in the negative x direction.

10. Warping constant

a) C-Section:

$$C_w = \frac{\bar{a}^2\bar{b}^2 t}{12}\left\{\frac{2\bar{a}^3\bar{b} + 3\bar{a}^2\bar{b}^2 + \alpha\begin{bmatrix}48\bar{c}^4 + 112\bar{b}\bar{c}^3 + 8\bar{a}\bar{c}^3 + 48\bar{a}\bar{b}\bar{c}^2 \\ +12\bar{a}^2\bar{c}^2 + 12\bar{a}^2\bar{b}\bar{c} + 6\bar{a}^3\bar{c}\end{bmatrix}}{6\bar{a}^2\bar{b} + \left(\bar{a} + \alpha 2\bar{c}\right)^3 - \alpha 24\bar{a}\bar{c}^2}\right\}$$

b) Hat Section:

$$C_w = \frac{\bar{a}^2\bar{b}^2 t}{12}\left\{\frac{2\bar{a}^3\bar{b} + 3\bar{a}^2\bar{b}^2 + \alpha\begin{bmatrix}48\bar{c}^4 + 112\bar{b}\bar{c}^3 + 8\bar{a}\bar{c}^3 - 48\bar{a}\bar{b}\bar{c}^2 \\ -12\bar{a}^2\bar{c}^2 + 12\bar{a}^2\bar{b}\bar{c} + 6\bar{a}^3\bar{c}\end{bmatrix}}{6\bar{a}^2\bar{b} + \left(\bar{a} + \alpha 2\bar{c}\right)^3}\right\}$$

11. Parameter β_w

$$\beta_w = -\left[\frac{t\bar{x}_c\bar{a}^3}{12} + t\bar{x}_c^{\,3}\bar{a}\right]$$

12. Parameter β_f

$$\beta_f = \frac{t}{2}\left[\left(\bar{b} - \bar{x}_c\right)^4 - \bar{x}_c^{\,4}\right] + \frac{t\bar{a}^2}{4}\left[\left(\bar{b} - \bar{x}_c\right)^2 - \bar{x}_c^{\,2}\right]$$

13. Parameter β_l

a) C-Section:

$$\beta_l = \alpha\left\{2\bar{c}t\left(\bar{b} - \bar{x}_c\right)^3 + \frac{2}{3}t\left(\bar{b} - \bar{x}_c\right)\left[\left(\frac{\bar{a}}{2}\right)^3 - \left(\frac{\bar{a}}{2} - \bar{c}\right)^3\right]\right\}$$

b) Hat Section:

$$\beta_l = 2\bar{c}t\left(\bar{b} - \bar{x}_c\right)^3 + \frac{2}{3}t\left(\bar{b} - \bar{x}_c\right)\left[\left(\frac{\bar{a}}{2} + \bar{c}\right)^3 - \left(\frac{\bar{a}}{2}\right)^3\right]$$

14. Parameter used in determination of elastic critical moment

$$j = \frac{1}{2I_y}\left(\beta_w + \beta_f + \beta_l\right) - x_o$$

3.3.3 I-Sections With Unequal Flanges (Singly-Symmetric) and T-Sections (Singly-Symmetric)

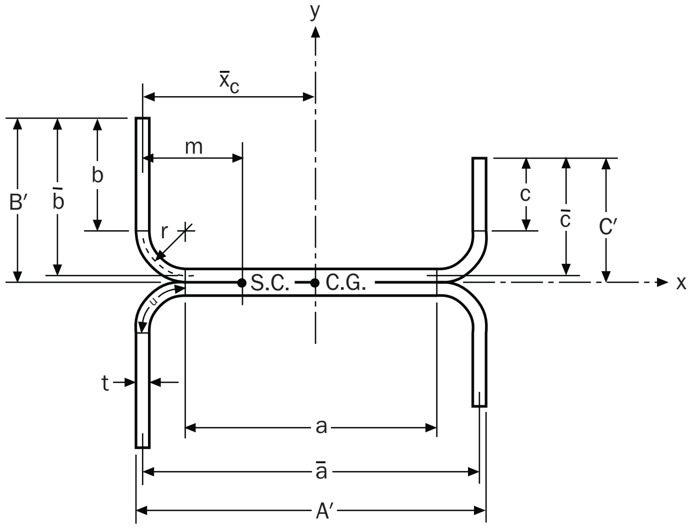


Figure 3.3.3-1
I-Section With Unequal Flanges (Singly Symmetric)

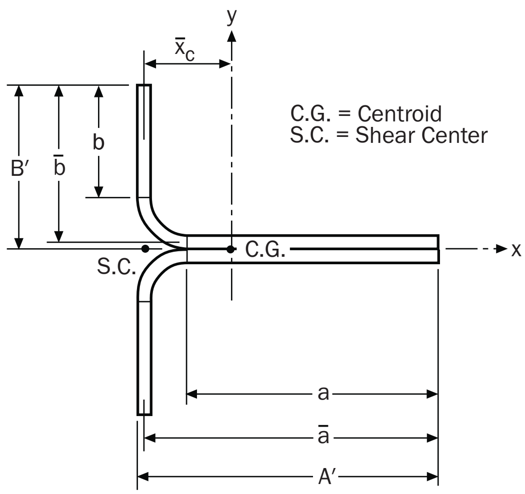


Figure 3.3.3-2
T-Section (Singly Symmetric)

1. Basic parameters

$$a = A' - \left[r + t/2 + \alpha\left(r + t/2\right)\right]^*$$

$$\bar{a} = A' - \left(t/2 + \alpha t/2\right)$$

$$b = B' - \left(r + t/2\right)$$

$$\bar{b} = B' - t/2$$

$$c = \alpha\left[C' - \left(r + t/2\right)\right]$$

$$\bar{c} = \alpha\left(C' - t/2\right)$$

$$u = \pi r/2 = 1.57r$$

2. Cross-sectional area

$$A = t\left[2a + 2b + 2u + \alpha\left(2c + 2u\right)\right]$$

* For I-Sections, α = 1.0; for T-Sections, α = 0.

3. Moment of inertia about x-axis

$$I_x = 2t\left\{\begin{array}{l} b\left(\dfrac{b}{2}+r+\dfrac{t}{2}\right)^2+\dfrac{b^3}{12}+u\left(0.363r+\dfrac{t}{2}\right)^2+0.149r^3 \\ +\alpha\left[c\left(\dfrac{c}{2}+r+\dfrac{t}{2}\right)^2+\dfrac{b^3}{12}+u\left(0.363r+\dfrac{t}{2}\right)^2+0.149r^3\right]\end{array}\right\}$$

4. Distance between centroid and longer flange centerline

$$\bar{x}_c = \frac{2t}{A}\left\{u0.363r + a\left(a/2+r\right)+\alpha\left[u\left(a+1.637r\right)+c\left(a+2r\right)\right]\right\}$$

5. Distance between centroid and outside of longer flange

$$\bar{x} = \bar{x}_c + \frac{t}{2}$$

6. Moment of inertia about y-axis

$$I_y = 2t\left\{\begin{array}{l} 0.358^3 + a\left(\dfrac{a}{2}+r\right)^2+\dfrac{a^3}{12} \\ +\alpha\left[u\left(a+1.637\,r\right)^2+0.149r^3+c\left(a+2r\right)^2\right]\end{array}\right\} - A\bar{x}_c^{\,2}$$

7. Distance between shear center and longer flange centerline

$$m = \bar{a}\left(1-\frac{\bar{b}^3}{\bar{b}^3+\bar{c}^3}\right)$$

8. Distance between shear center and centroid

$$x_o = -\left(\bar{x}_c - m\right)*$$

9. St. Venant torsion constant

$$J = \frac{2t^3}{3}\left[a+b+u+\alpha\left(u+c\right)\right]$$

10. Warping constant

For I-Sections the value of C_w is twice the value of each channel if fastened at the middle of the webs; however, if the two channels are continuously welded at both edges of the web to form the I-Section, the warping constants are as follows:

a) Unlipped I-Sections and T-Sections:

$$C_w = \frac{t\bar{a}^2}{12}\left(\frac{8\bar{b}^3\bar{c}^3}{\bar{b}^3+\bar{c}^3}\right)$$

b) Doubly symmetric, lipped I-Sections:

$\bar{c}$ = length of lip, see Figure 3.3.2-1

$$C_w = \frac{t\bar{b}^2}{3}\left(\bar{a}^2\bar{b}+3\bar{a}^2\bar{c}+6\bar{a}\,\bar{c}^2+4\bar{c}^3\right)$$

* Negative sign indicates x_o is measured in the negative x direction.

11. Parameter used in determination of elastic critical moment

$$j = \frac{t}{2I_y}\left[\begin{array}{l} -2\bar{x}_c\bar{b}\left(\bar{x}_c^{\,2}+\bar{b}^2/3\right)+2\bar{c}\left(\bar{a}-\bar{x}_c\right)\left[\left(\bar{a}-\bar{x}_c\right)^2+\bar{c}^2/3\right] \\ +\dfrac{1}{2}\left[\left(\bar{a}-\bar{x}_c\right)^4-\bar{x}_c^{\,4}\right] \end{array}\right]-x_o$$

3.3.4 Z-Sections (Point-Symmetric) With and Without Lips

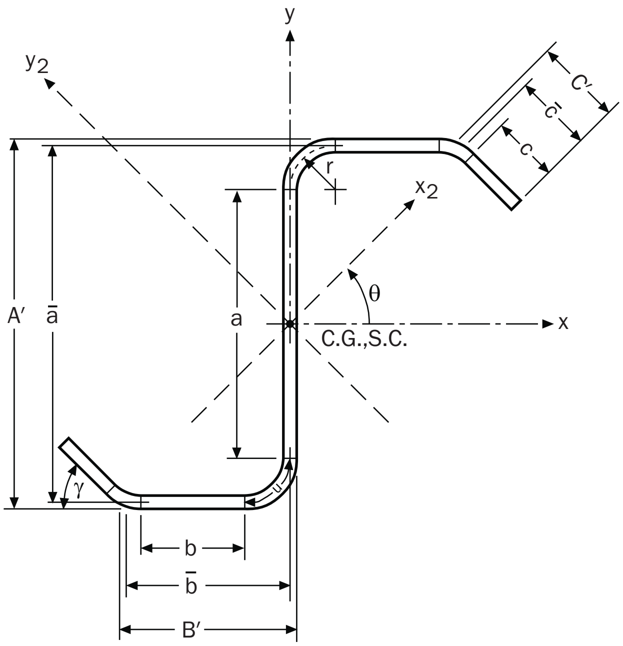


Figure 3.3.4-1
Z-Section (Point Symmetric) With Lips

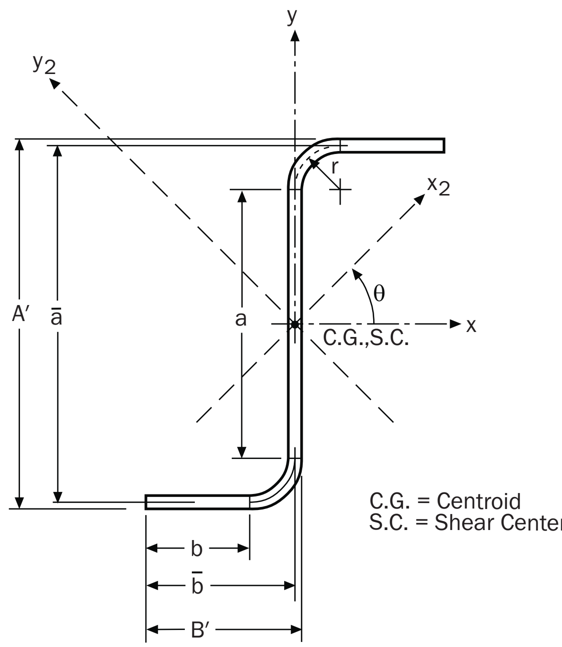


Figure 3.3.4-2
Z-Section (Point Symmetric) Without Lips

1. Basic parameters

$a = A' - (2r + t)$

$\bar{a} = A' - t$

$b = B' - \left[r + t/2 + \alpha(r + t/2)\tan(\gamma/2)\right]$*

$\bar{b} = B' - \left[t/2 + (\alpha t/2)\tan(\gamma/2)\right]$

$c = \alpha\left[C' - (r + t/2)\tan(\gamma/2)\right]$

$\bar{c} = \alpha\left[C' - (t/2)\tan(\gamma/2)\right]$

$u_1 = \pi r/2 = 1.57r$

$u_2 = \gamma r$, where γ is in radians

2. Cross-sectional area

$A = t\left[a + 2b + 2u_1 + \alpha(2c + 2u_2)\right]$

* For sections with lips, $\alpha = 1.0$; for sections without lips, $\alpha = 0$.

3. Moment of inertia about x-axis

$$I_x = 2t\left\{\begin{array}{l} 0.0417a^3 + b\left(a/2+r\right)^2 + u_1\left(a/2+0.637r\right)^2 + 0.149r^3 \\ +\alpha\left[\begin{array}{l}\left(\dfrac{\gamma+\sin\gamma\cos\gamma}{2} - \dfrac{\sin^2\gamma}{\gamma}\right)r^3 + u_2\left(a/2+\dfrac{r\sin\gamma}{\gamma}\right)^2 \\ +\dfrac{c^3\sin^2\gamma}{12} + c\left(a/2 + r\cos\gamma - \dfrac{c}{2}\sin\gamma\right)^2\end{array}\right]\end{array}\right\}$$

4. Moment of inertia about y-axis

$$I_y = 2t\left\{b\left(b/2+r\right)^2 + \frac{b^3}{12} + 0.356r^3 + \alpha\left[\begin{array}{l} c\left(b+r\left(1+\sin\gamma\right)+\dfrac{c}{2}\cos\gamma\right)^2 + \dfrac{c^3\cos^2\gamma}{12} \\ +u_2\left(b+r+\dfrac{r\left(1-\cos\gamma\right)}{\gamma}\right)^2 \\ +\left[\dfrac{\gamma-\sin\gamma\cos\gamma}{2} - \dfrac{\left(1-\cos\gamma\right)^2}{\gamma}\right]r^3\end{array}\right]\right\}$$

5. Product of inertia (See note below)

$$I_{xy} = 2t\left\{\begin{array}{l} b\left(a/2+r\right)\left(b/2+r\right) + 0.5r^3 + 0.285ar^2 \\ +\alpha\left[\begin{array}{l} c\left(b+r\left(1+\sin\gamma\right)+\dfrac{c}{2}\cos\gamma\right)\left(\dfrac{a}{2}+r\cos\gamma-\dfrac{c}{2}\sin\gamma\right) \\ +\left(\dfrac{\sin^2\gamma}{2}+\dfrac{\sin\gamma\left(\cos\gamma-1\right)}{\gamma}\right)r^3 - \dfrac{c^3\sin\gamma\cos\gamma}{12} \\ +u_2\left(b+r+\dfrac{r\left(1-\cos\gamma\right)}{\gamma}\right)\left(\dfrac{a}{2}+\dfrac{r\sin\gamma}{\gamma}\right)\end{array}\right]\end{array}\right\}$$

6. Angle between x-axis and minor principal axis, in radians (See note below)

$$\theta = \frac{\pi}{2} + 0.5\arctan\left(\frac{2I_{xy}}{I_y - I_x}\right)$$

7. Moment of inertia about x_2 axis (See note below)

$$I_{x2} = I_x\cos^2\theta + I_y\sin^2\theta - 2I_{xy}\sin\theta\cos\theta$$

8. Moment of inertia about y_2 axis (See note below)

$$I_{y2} = I_x\sin^2\theta + I_y\cos^2\theta + 2I_{xy}\sin\theta\cos\theta$$

Note: The algebraic signs in Equations 5, 6, 7, and 8 are correct for the cross-section oriented with respect to the coordinate axes as shown in Figure 3.3.4-1 and Figure 3.3.4-2.

9. Radius of gyration about any axis

$$r = \sqrt{I/A}$$

10. Minimum radius of gyration, about x_2 axis

$$r_{min} = \sqrt{I_{x2}/A}$$

11. St. Venant torsion constant

$$J = \frac{t^3}{3}\left[a + 2b + 2u_1 + \alpha\left(2c + 2u_2\right)\right]$$

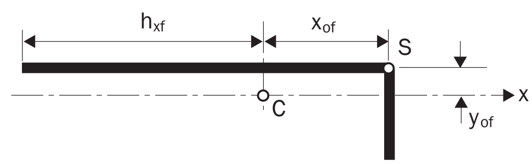
12. Warping constant

$$C_w = \frac{t}{12}\left\{\frac{\bar{a}^2\bar{b}^3\left(2\bar{a}+\bar{b}\right)+\alpha\left[\begin{array}{l}\bar{b}^2\left(4\bar{c}^4+16\bar{b}\bar{c}^3+6\bar{a}^3\bar{c}+4\bar{a}^2\bar{b}\bar{c}+8\bar{a}\bar{c}^3\right)\\+6\bar{a}\bar{b}\bar{c}^2\left(\bar{a}+\bar{b}\right)\left(2\bar{b}\sin\gamma+\bar{a}\cos\gamma\right)\\+4\bar{a}\bar{b}\bar{c}^3\left(2\bar{a}+4\bar{b}+\bar{c}\right)\sin\gamma\cos\gamma\\+\bar{c}^3\left(2\bar{a}^3+4\bar{a}^2\bar{b}-8\bar{a}\bar{b}^2+\bar{a}^2\bar{c}-16\bar{b}^3-4\bar{b}^2\bar{c}\right)\cos^2\gamma\end{array}\right]}{\bar{a}+2\left(\bar{b}+\alpha\bar{c}\right)}\right\}$$

3.4 Distortional Buckling Properties

The equations below provide the flange section properties used in distortional buckling calculations in *Specification* Sections C3.1.4(a) and C4.2(a). The equations are approximations based on centerline dimensions and sharp corners. The properties calculated by this procedure are sufficiently accurate for routine engineering design of sections with small ratios of corner radius to thickness.

3.4.1 Flanges With 90 Degree Lips



1. Basic parameters

$b = B' - t$

$d = C' - \frac{t}{2}$

2. Flange section properties

$A_f = \left(b+d\right)t$

$J_f = \left(1/3\right)bt^3 + \left(1/3\right)dt^3$

$$I_{xf} = \frac{t\left(t^2b^2 + 4bd^3 + t^2bd + d^4\right)}{12\left(b+d\right)}$$

$$I_{yf} = \frac{t\left(b^4 + 4db^3\right)}{12\left(b+d\right)}$$

$$I_{xyf} = \frac{tb^2d^2}{4\left(b+d\right)}$$

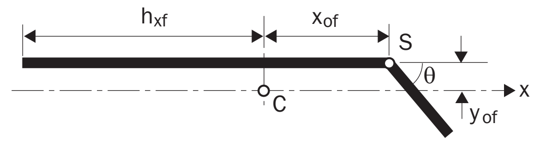
$C_{wf} = 0$

$$x_{of} = \frac{b^2}{2\left(b+d\right)}$$

$$h_{xf} = \frac{-\left(b^2 + 2db\right)}{2\left(b+d\right)}$$

$$h_{yf} = y_{of} = \frac{-d^2}{2\left(b+d\right)}$$

3.4.2 Flanges With Sloped Lips



1. Basic parameters

$$b = B' - t$$

$$d = C' - \frac{t}{2}$$

2. Flange section properties

$$A_f = (b+d)t$$

$$J_f = (1/3)bt^3 + (1/3)dt^3$$

$$I_{xf} = \frac{t\left(t^2b^2 + 4bd^3 - 4bd^3\cos^2\theta + t^2bd + d^4 - d^4\cos^2\theta\right)}{12(b+d)}$$

$$I_{yf} = \frac{t\left(b^4 + 4db^3 + 6d^2b^2\cos\theta + 4d^3b\cos^2\theta + d^4\cos^2\theta\right)}{12(b+d)}$$

$$I_{xyf} = \frac{tbd^2\sin\theta(b + d\cos\theta)}{4(b+d)}$$

$$C_{wf} = 0$$

$$x_{of} = \frac{b^2 - d^2\cos\theta}{2(b+d)}$$

$$h_{xf} = \frac{-\left(b^2 + 2db + d^2\cos\theta\right)}{2(b+d)}$$

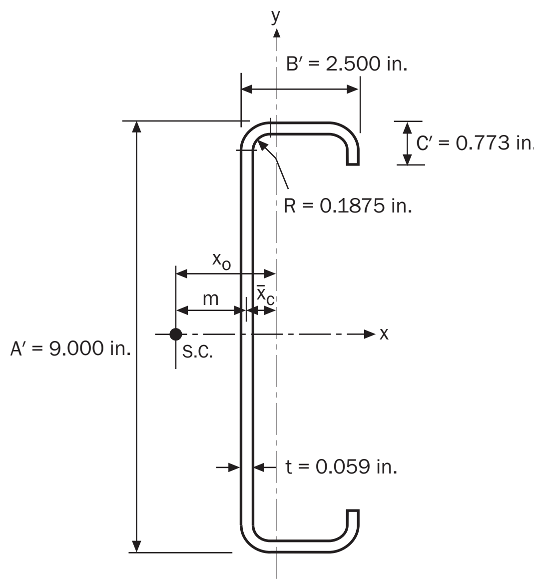
$$h_{yf} = y_{of} = \frac{-d^2\sin\theta}{2(b+d)}$$

3.5 Gross Section Properties - Example Problems

The following example problems are intended to illustrate the use of the gross section property equations presented in this Part of the *Manual*. These should be used in conjunction with the other parts of the *Manual*.

As a general rule, section properties are computed to three significant figures. In some cases, where the properties are used in subsequent calculations, the properties are calculated to four significant figures to preserve precision. Dimensions are generally given to the nearest one thousandth of an inch. In some cases it was impractical to adhere strictly to these guidelines. Slight discrepancies should be expected between the calculated section properties computed in the examples and those given in the tables in Parts I, II and III of this *Manual* which were calculated by computer.

Example I-1: C-Section With Lips - Gross Section Properties



Given:

1. Section: 9CS2.5x059 as shown above

Required:

1. Axial and flexural properties
2. Torsional properties

Solution:

1. Axial and Flexural Properties

a. Basic parameters

$A' = 9.000$ in.

$B' = 2.500$ in.

$C' = 0.773$ in.

$t = 0.059$ in.

$R = 0.1875$ in.

$\alpha = 1.0$ (section has stiffener lips)

$$r = R + t/2$$
$$= 0.1875 + 0.059/2 = 0.217 \text{ in.}$$

$$a = A' - (2r + t)$$
$$= 9.000 - [(2)(0.217) + 0.059] = 8.507 \text{ in.}$$

$$\bar{a} = A' - t$$
$$= 9.000 - 0.059 = 8.941 \text{ in.}$$

$$b = B' - [r + t/2 + \alpha(r + t/2)]$$
$$= 2.500 - [0.217 + 0.059/2 + 1.0(0.217 + 0.059/2)] = 2.007 \text{ in.}$$

$$\bar{b} = B' - [t/2 + \alpha\, t/2]$$
$$= 2.500 - [0.059/2 + (1.0)(0.059/2)] = 2.441 \text{ in.}$$

$$c = \alpha[C' - (r + t/2)]$$
$$= 1.0[0.773 - (0.217 + 0.059/2)] = 0.527 \text{ in.}$$

$$\bar{c} = \alpha\left[C' - (t/2)\right]$$
$$= 1.0\left[0.773 - (0.059/2)\right] = 0.744 \text{ in.}$$
$$u = \pi r/2$$
$$= \pi(0.217)/2 = 0.341 \text{ in.}$$

b. Cross-section area

$$A = t\left[a + 2b + 2u + \alpha(2c + 2u)\right]$$
$$= 0.059\left[8.507 + (2)(2.007) + (2)(0.341) + 1.0\left((2)(0.527) + (2)(0.341)\right)\right]$$
$$= 0.881 \text{ in.}^2$$

c. Moment of inertia about the x-axis

$$I_x = 2t\left\{\begin{array}{l} 0.0417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149r^3 \\ +\alpha\left[0.0833c^3 + \dfrac{c}{4}(a - c)^2 + u\left(\dfrac{a}{2} + 0.637r\right)^2 + 0.149r^3\right] \end{array}\right\}$$

$$= (2)(0.059)\left\{\begin{array}{l} 0.0417(8.507)^3 + 2.007(8.507/2 + 0.217)^2 \\ +0.341\left[8.507/2 + 0.637(0.217)\right]^2 + 0.149(0.217)^3 \\ +1.0\left[\begin{array}{l} (0.0833)(0.527)^3 + \dfrac{0.527}{4}(8.507 - 0.527)^2 \\ +0.341\left(8.507/2 + (0.637)(0.217)\right)^2 + (0.149)(0.217)^3 \end{array}\right] \end{array}\right\}$$

$$= 0.118\left[25.67 + 40.11 + 6.577 + 0.0015 + 1.0(0.012 + 8.390 + 6.577 + 0.0015)\right]$$
$$= 10.3 \text{ in.}^4$$

d. Distance between centroid and web centerline

$$\bar{x}_c = \frac{2t}{A}\left\{b(b/2 + r) + u(0.363r) + \alpha\left[u(b + 1.637r) + c(b + 2r)\right]\right\}$$

$$= \frac{(2)(0.059)}{0.881}\left\{\begin{array}{l} 2.007(2.007/2 + 0.217) + (0.341)(0.363)(0.217) \\ +1.0\left[0.341\left(2.007 + (1.637)(0.217)\right) + 0.527\left(2.007 + (2)(0.217)\right)\right] \end{array}\right\}$$

$$= 0.1339\left\{2.450 + 0.0269 + 1.0\left[0.806 + 1.286\right]\right\}$$
$$= 0.612 \text{ in.}$$

e. Moment of inertia about the y-axis

$$I_y = 2t\left\{\begin{array}{l} b(b/2 + r)^2 + b^3/12 + 0.356r^3 \\ +\alpha\left[c(b + 2r)^2 + u(b + 1.637r)^2 + 0.149r^3\right] \end{array}\right\} - A\bar{x}_c^{\,2}$$

$$= (2)(0.059)\left\{\begin{array}{l} 2.007\left(\dfrac{2.007}{2} + 0.217\right)^2 + \dfrac{(2.007)^3}{12} + 0.356(0.217)^3 \\ +1.0\left[\begin{array}{l} 0.527\left(2.007 + (2)(0.217)\right)^2 + \\ 0.341\left(2.007 + (1.637)(0.217)\right)^2 + 0.149(0.217)^3 \end{array}\right] \end{array}\right\} - (0.881)(0.612)^2$$

$$= 0.118\left\{2.990 + 0.674 + 0.0036 + 1.0\left[3.140 + 1.903 + 0.0015\right]\right\} - 0.330$$
$$= 0.698 \text{ in.}^4$$

f. Distance between shear center and web centerline

$$m = \bar{b}\left[\frac{3\bar{a}^2\bar{b} + \alpha\bar{c}\left(6\bar{a}^2 - 8\bar{c}^2\right)}{\bar{a}^3 + 6\bar{a}^2\bar{b} + \alpha\bar{c}\left(8\bar{c}^2 - 12\bar{a}\bar{c} + 6\bar{a}^2\right)}\right]$$

$$= 2.441\left[\frac{(3)(8.941)^2(2.441) + (1.0)(0.744)\left[(6)(8.941)^2 - (8)(0.744)^2\right]}{\begin{array}{c}(8.941)^3 + (6)(8.941)^2(2.441) \\ +(1.0)(0.744)\left[(8)(0.744)^2 - (12)(8.941)(0.744) + (6)(8.941)^2\right]\end{array}}\right]$$

$$= 2.441\left[\frac{585.4 + 353.6}{714.8 + 1171 + 300.8}\right]$$

$$= 1.048 \text{ in.}$$

g. Distance between centroid and shear center

$$x_o = -\left(\bar{x}_c + m\right)$$

$$= -\left(0.612 + 1.048\right)$$

$$= -1.660 \text{ in.}$$

2. Torsional Properties

a. St. Venant torsional constant

$$J = \frac{t^3}{3}\left[a + 2b + 2u + \alpha\left(2c + 2u\right)\right]$$

$$= \frac{(0.059)^3}{3}\left[8.507 + (2)(2.007) + (2)(0.341) + 1.0\left((2)(0.527) + (2)(0.341)\right)\right]$$

$$= 0.00102 \text{ in.}^4$$

b. Warping constant

$$C_w = \frac{\bar{a}^2\bar{b}^2 t}{12}\left\{\frac{2\bar{a}^3\bar{b} + 3\bar{a}^2\bar{b}^2 + \alpha\left[\begin{array}{l}48\bar{c}^4 + 112\bar{b}\bar{c}^3 + 8\bar{a}\bar{c}^3 + 48\bar{a}\bar{b}\bar{c}^2 \\ +12\bar{a}^2\bar{c}^2 + 12\bar{a}^2\bar{b}\bar{c} + 6\bar{a}^3\bar{c}\end{array}\right]}{6\bar{a}^2\bar{b} + \left(\bar{a} + \alpha 2\bar{c}\right)^3 - \alpha 24\bar{a}\bar{c}^2}\right\}$$

$$= \frac{(8.941)^2(2.441)^2(0.059)}{12}\left\{\frac{\begin{array}{l}2(8.941)^3(2.441) + 3(8.941)^2(2.441)^2 \\ +1.0\left[\begin{array}{l}48(0.744)^4 + 112(2.441)(0.744)^3 \\ +8(8.941)(0.744)^3 \\ +48(8.941)(2.441)(0.744)^2 + 12(8.941)^2(0.744)^2 \\ +12(8.941)^2(2.441)(0.744) + 6(8.941)^3(0.744)\end{array}\right]\end{array}}{\begin{array}{l}6(8.941)^2(2.441) + \left(8.941 + 1.0(2)(0.744)\right)^3 \\ -1.0(24)(8.941)(0.744)^2\end{array}}\right\}$$

$$= 2.342\left\{\frac{3489 + 1429 + 1.0\left[\begin{array}{l}14.71 + 112.6 + 29.46 + 579.9 \\ +531.0 + 1742 + 3191\end{array}\right]}{1171 + 1134 - 118.8}\right\}$$

$$= 11.9 \text{ in.}^6$$

c. Parameter used in determination of elastic critical moment

$$\beta_w = -\left[\frac{t\bar{x}_c\bar{a}^3}{12} + t\bar{x}_c{}^3\bar{a}\right] = -\left[\frac{(0.059)(0.612)(8.941)^3}{12} + (0.059)(0.612)^3(8.941)\right]$$

$$= -2.272 \text{ in.}^5$$

$$\beta_f = \frac{t}{2}\left[\left(\bar{b}-\bar{x}_c\right)^4 - \bar{x}_c^4\right] + \frac{t\bar{a}^2}{4}\left[\left(\bar{b}-\bar{x}_c\right)^2 - \bar{x}_c^2\right]$$

$$= \frac{0.059}{2}\left[(2.441-0.612)^4 - (0.612)^4\right] + \frac{(0.059)(8.941)^2}{4}\left[(2.441-0.612)^2 - 0.612^2\right]$$

$$= 3.829 \text{ in.}^5$$

$$\beta_l = \alpha\left\{2\bar{c}t\left(\bar{b}-\bar{x}_c\right)^3 + \frac{2}{3}t\left(\bar{b}-\bar{x}_c\right)\left[\left(\bar{a}/2\right)^3 - \left(\bar{a}/2-\bar{c}\right)^3\right]\right\}$$

$$= 1.0\left\{\begin{array}{l}(2)(0.744)(0.059)(2.441-0.612)^3 \\ +\frac{2}{3}(0.059)(2.441-0.612)\left[\left(\frac{8.941}{2}\right)^3 - \left(\frac{8.941}{2}-0.744\right)^3\right]\end{array}\right\}$$

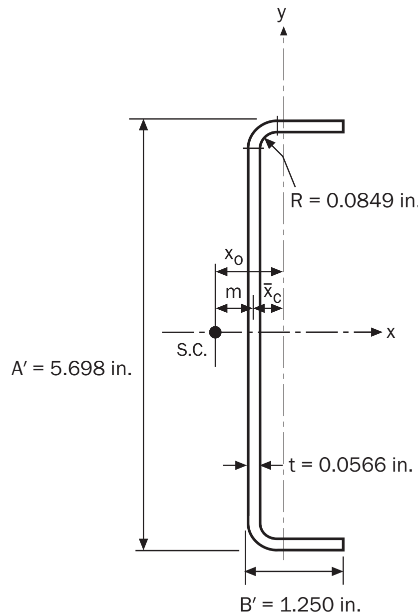
$$= 3.242 \text{ in.}^5$$

$$j = \frac{1}{2I_y}\left(\beta_w + \beta_f + \beta_l\right) - x_o$$

$$= \frac{1}{(2)(0.698)}(-2.272 + 3.829 + 3.242) - (-1.660)$$

$$= 5.10 \text{ in.}$$

Example I-2: C-Section Without Lips - Gross Section Properties



Given:

1. Section: Track 550T125-54 as shown above

Required:

1. Axial and flexural properties
2. Torsional properties

Solution:

1. Axial and Flexural Properties

a. Basic parameters

$A' = 5.698$ in.

$B' = 1.250$ in.

$C' = 0.000$ in.

$t = 0.0566$ in.

$R = 0.0849$ in.

$\alpha = 0.0$ (section does not have stiffener lips)

$r = R + t/2 = 0.0849 + 0.0566/2 = 0.113$ in.

$$a = A' - (2r + t)$$
$$= 5.698 - [(2)(0.113) + 0.0566] = 5.415 \text{ in.}$$

$$\bar{a} = A' - t$$
$$= 5.698 - 0.0566 = 5.641 \text{ in.}$$

$$b = B' - [r + t/2 + \alpha(r + t/2)]$$
$$= 1.250 - [0.113 + 0.0566/2 + 0.0] = 1.109 \text{ in.}$$

$$\bar{b} = B' - [t/2 + \alpha t/2]$$
$$= 1.250 - [0.0566/2 + 0.0] = 1.222 \text{ in.}$$

$$c = \alpha[C' - (r + t/2)] = 0.0 \text{ in.}$$

$$\bar{c} = \alpha[C' - (t/2)] = 0.0 \text{ in.}$$

$$u = \pi r/2$$
$$= \pi(0.113)/2 = 0.177 \text{ in.}$$

b. Cross-section area

$$A = t\left[a + 2b + 2u + \alpha(2c + 2u)\right]$$
$$= 0.0566\left[5.415 + (2)(1.109) + (2)(0.177) + 0.0\right]$$
$$= 0.452 \text{ in.}^2$$

c. Moment of inertia about the x-axis

$$I_x = 2t\left\{\begin{matrix} 0.0417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149r^3 \\ +\alpha\left[0.0833c^3 + \frac{c}{4}(a - c)^2 + u\left(\frac{a}{2} + 0.637r\right)^2 + 0.149r^3\right] \end{matrix}\right\}$$
$$= (2)(0.0566)\left\{\begin{matrix} (0.0417)(5.415)^3 + 1.109(5.415/2 + 0.113)^2 \\ +0.177\left[5.415/2 + 0.637(0.113)\right]^2 + 0.149(0.113)^3 + 0.0 \end{matrix}\right\}$$
$$= 1.90 \text{ in.}^4$$

d. Distance between centroid and web centerline

$$\bar{x}_c = \frac{2t}{A}\left\{b(b/2 + r) + u(0.363r) + \alpha\left[u(b + 1.637r) + c(b + 2r)\right]\right\}$$
$$= \frac{(2)(0.0566)}{0.452}\left\{1.109(1.109/2 + 0.113) + (0.177)(0.363)(0.113) + 0.0\right\}$$
$$= 0.187 \text{ in.}$$

e. Moment of inertia about the y-axis

$$I_y = 2t\left\{\begin{matrix} b(b/2 + r)^2 + b^3/12 + 0.356r^3 \\ +\alpha\left[c(b + 2r)^2 + u(b + 1.637r)^2 + 0.149r^3\right] \end{matrix}\right\} - A\bar{x}_c^2$$
$$= (2)(0.0566)\left\{\begin{matrix} 1.109(1.109/2 + 0.113)^2 + (1.109)^3/12 + 0.356(0.113)^3 \\ +0.0 \end{matrix}\right\} - (0.452)(0.187)^2$$
$$= 0.0531 \text{ in.}^4$$

f. Distance between shear center and web centerline

$$m = \bar{b}\left[\frac{3\bar{a}^2\bar{b} + \alpha\bar{c}\left(6\bar{a}^2 - 8\bar{c}^2\right)}{\bar{a}^3 + 6\bar{a}^2\bar{b} + \alpha\bar{c}\left(8\bar{c}^2 - 12\overline{ac} + 6\bar{a}^2\right)}\right]$$
$$= 1.222\left[\frac{(3)(5.641)^2(1.222) + 0.0}{(5.641)^3 + (6)(5.641)^2(1.222) + 0.0}\right]$$
$$= 0.345 \text{ in.}$$

g. Distance between centroid and shear center

$$x_o = -(\bar{x}_c + m)$$
$$= -(0.187 + 0.345) = -0.532 \text{ in.}$$

2. Torsional Properties

a. St. Venant torsional constant

$$J = \frac{t^3}{3}\left[a + 2b + 2u + \alpha(2c + 2u)\right]$$

$$= \frac{(0.0566)^3}{3}\left[5.415 + (2)(1.109) + (2)(0.177) + 0.0\right]$$

$$= 0.000483 \text{ in.}^4$$

b. Warping constant

$$C_w = \frac{\bar{a}^2\bar{b}^2 t}{12}\left\{\frac{2\bar{a}^3\bar{b} + 3\bar{a}^2\bar{b}^2 + \alpha\begin{bmatrix}48\bar{c}^4 + 112\bar{b}\bar{c}^3 + 8\bar{a}\bar{c}^3 + 48\bar{a}\bar{b}\bar{c}^2 \\ +12\bar{a}^2\bar{c}^2 + 12\bar{a}^2\bar{b}\bar{c} + 6\bar{a}^3\bar{c}\end{bmatrix}}{6\bar{a}^2\bar{b} + (\bar{a} + \alpha 2\bar{c})^3 - \alpha 24\bar{a}\bar{c}^2}\right\}$$

$$= \frac{(5.641)^2(1.222)^2(0.0566)}{12}\left\{\frac{2(5.641)^3(1.222) + (3)(5.641)^2(1.222)^2 + 0.0}{(6)(5.641)^2(1.222) + (5.641 + 0.0)^3 - 0.0}\right\}$$

$$= 0.316 \text{ in.}^6$$

c. Parameter used in determination of elastic critical moment

$$\beta_w = -\left[\frac{t\bar{x}_c\bar{a}^3}{12} + t\bar{x}_c^{\,3}\bar{a}\right]$$

$$= -\left[\frac{(0.0566)(0.187)(5.641)^3}{12} + (0.0566)(0.187)^3(5.641)\right]$$

$$= -0.1604 \text{ in.}^5$$

$$\beta_f = \frac{t}{2}\left[(\bar{b} - \bar{x}_c)^4 - \bar{x}_c^{\,4}\right] + \frac{t\bar{a}^2}{4}\left[(\bar{b} - \bar{x}_c)^2 - \bar{x}_c^{\,2}\right]$$

$$= \frac{0.0566}{2}\left[(1.222 - 0.187)^4 - (0.187)^4\right] + \frac{(0.0566)(5.641)^2}{4}\left[(1.222 - 0.187)^2 - 0.187^2\right]$$

$$= 0.4990 \text{ in.}^5$$

$$\beta_l = \alpha\left\{2\bar{c}t(\bar{b} - \bar{x}_c)^3 + \frac{2}{3}t(\bar{b} - \bar{x}_c)\left[(\bar{a}/2)^3 - (\bar{a}/2 - \bar{c})^3\right]\right\}$$

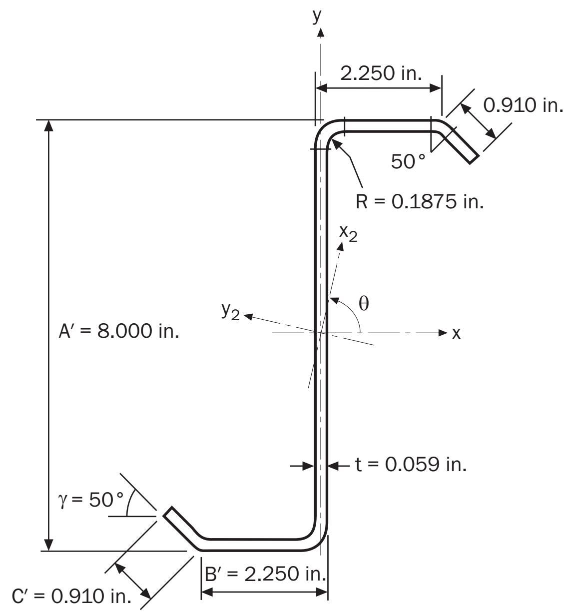
$$= 0.0 \text{ in.}^5$$

$$j = \frac{1}{2I_y}(\beta_w + \beta_f + \beta_l) - x_o$$

$$= \frac{1}{(2)(0.0531)}(-0.1604 + 0.4990 + 0.0) - (-0.532)$$

$$= 3.72 \text{ in.}$$

Example I-3: Z-Section With Lips - Gross Section Properties



Given:

1. Section: 8ZS2.25x059 as shown above

Required:

1. Axial and flexural properties
2. Torsional properties

Solution:

1. Axial and Flexural Properties

a. Basic parameters

$A' = 8.000$ in.

$B' = 2.250$ in.

$C' = 0.910$ in.

$t = 0.059$ in.

$R = 0.1875$ in.

$\alpha = 1.0$ (section has stiffener lips)

$r = R + t/2 = 0.1875 + 0.059/2 = 0.217$ in.

$\gamma = 50\pi/180 = 0.8727$ radians

$$a = A' - (2r + t)$$
$$= 8.000 - \left[(2)(0.217) + 0.059\right] = 7.507 \text{ in.}$$

$$\bar{a} = A' - t$$
$$= 8.000 - 0.059 = 7.941 \text{ in.}$$

$$b = B' - \left[r + t/2 + \alpha(r + t/2)\tan(\gamma/2)\right]$$
$$= 2.250 - \left[0.217 + 0.059/2 + 1.0(0.217 + 0.059/2)\tan(0.8727/2)\right] = 1.889 \text{ in.}$$

$$\bar{b} = B' - \left[t/2 + (\alpha t/2)\tan(\gamma/2)\right]$$
$$= 2.250 - \left[0.059/2 + (1.0)(0.059/2)\tan(0.8727/2)\right] = 2.207 \text{ in.}$$

$$c = \alpha\left[C' - (r + t/2)\tan(\gamma/2)\right]$$
$$= 1.0\left[0.910 - (0.217 + 0.059/2)\tan(0.8727/2)\right] = 0.795 \text{ in.}$$

$$\bar{c} = \alpha\left[C' - (t/2)\tan(\gamma/2)\right]$$
$$= 1.0\left[0.910 - (0.059/2)\tan(0.8727/2)\right] = 0.896 \text{ in.}$$

$$u_1 = \pi r/2$$
$$= \pi(0.217)/2 = 0.341 \text{ in.}$$

$$u_2 = \gamma r$$
$$= (0.8727)(0.217) = 0.189 \text{ in.}$$

b. Cross-section area

$$A = t\left[a + 2b + 2u_1 + \alpha(2c + 2u_2)\right]$$
$$= 0.059\left[7.507 + (2)(1.889) + (2)(0.341) + 1.0\{(2)(0.795) + (2)(0.189)\}\right]$$
$$= 0.822 \text{ in.}^2$$

c. Moment of inertia about the x-axis

$$I_x = 2t\left\{\begin{array}{l} 0.0417a^3 + b(a/2+r)^2 + u_1(a/2+0.637r)^2 + 0.149r^3 \\ +\alpha\left[\begin{array}{l}\left(\dfrac{\gamma+\sin\gamma\cos\gamma}{2} - \dfrac{\sin^2\gamma}{\gamma}\right)r^3 + u_2\left(a/2 + \dfrac{r\sin\gamma}{\gamma}\right)^2 \\ +\dfrac{c^3\sin^2\gamma}{12} + c\left(a/2 + r\cos\gamma - \dfrac{c}{2}\sin\gamma\right)^2\end{array}\right]\end{array}\right\}$$

$$= 2(0.059)\left\{\begin{array}{l} 0.0417(7.507)^3 + 1.889(7.507/2+0.217)^2 \\ +0.341\left(7.507/2+0.637(0.217)\right)^2 + 0.149(0.217)^3 \\ +(1.0)\left[\begin{array}{l}\left(\dfrac{0.8727+\sin(0.8727)\cos(0.8727)}{2} - \dfrac{\sin^2(0.8727)}{0.8727}\right)(0.217)^3 \\ +0.189\left(7.507/2 + \dfrac{(0.217)\sin(0.8727)}{0.8727}\right)^2 + \dfrac{(0.795)^3\sin^2(0.8727)}{12} \\ +(0.795)\left(\dfrac{7.507}{2} + 0.217\cos(0.8727) - \dfrac{0.795}{2}\sin(0.8727)\right)^2\end{array}\right]\end{array}\right\}$$

$$= 0.118\{17.64 + 29.78 + 5.165 + 0.0015 + 0.0001 + 2.940 + 0.0246 + 10.24\}$$
$$= 7.763 \text{ in.}^4$$

d. Moment of inertia about the y-axis

$$I_y = 2t\left\{b(b/2+r)^2 + \frac{b^3}{12} + 0.356r^3 + \alpha\left[\begin{array}{l} c\left(b + r(1+\sin\gamma) + \dfrac{c}{2}\cos\gamma\right)^2 + \dfrac{c^3\cos^2\gamma}{12} \\ +u_2\left(b + r + \dfrac{r(1-\cos\gamma)}{\gamma}\right)^2 \\ +\left[\dfrac{\gamma - \sin\gamma\cos\gamma}{2} - \dfrac{(1-\cos\gamma)^2}{\gamma}\right]r^3\end{array}\right]\right\}$$

$$I_y = (2)(0.059)\left\{\begin{array}{l} 1.889(1.889/2+0.217)^2 + (1.889)^3/12 + 0.356(0.217)^3 \\ +(1.0)\left[\begin{array}{l} 0.795\left(1.889+0.217(1+\sin(0.8727))+\dfrac{0.795}{2}\cos(0.8727)\right)^2 \\ +\dfrac{(0.795)^3\cos^2(0.8727)}{12} \\ +0.189\left(1.889+0.217+\dfrac{0.217(1-\cos(0.8727))}{0.8727}\right)^2 \\ +\left(\dfrac{0.8727-\sin(0.8727)\cos(0.8727)}{2}-\dfrac{(1-\cos(0.8727))^2}{0.8727}\right)(0.217)^3 \end{array}\right] \end{array}\right\}$$

$$= 0.118\{2.548+0.5617+0.0036+5.080+0.0173+0.910+0.0004\}$$

$$= 1.076 \text{ in.}^4$$

e. Product of inertia

$$I_{xy} = 2t\left\{\begin{array}{l} b(a/2+r)(b/2+r)+0.5r^3+0.285ar^2 \\ +\alpha\left[\begin{array}{l} c\left(b+r(1+\sin\gamma)+\dfrac{c}{2}\cos\gamma\right)\left(\dfrac{a}{2}+r\cos\gamma-\dfrac{c}{2}\sin\gamma\right) \\ +\left(\dfrac{\sin^2\gamma}{2}+\dfrac{\sin\gamma(\cos\gamma-1)}{\gamma}\right)r^3-\dfrac{c^3\sin\gamma\cos\gamma}{12} \\ +u_2\left(b+r+\dfrac{r(1-\cos\gamma)}{\gamma}\right)\left(a/2+\dfrac{r\sin\gamma}{\gamma}\right) \end{array}\right] \end{array}\right\}$$

$$= (2)(0.059)\left\{\begin{array}{l} 1.889(7.507/2+0.217)(1.889/2+0.217)+(0.5)(0.217)^3 \\ +0.285(7.507)(0.217)^2 \\ +1.0\left[\begin{array}{l} 0.795\left(1.889+0.217(1+\sin(0.8727))+\dfrac{0.795}{2}\cos(0.8727)\right) \\ \text{x}\left(7.507/2+0.217\cos(0.8727)-\dfrac{0.795}{2}\sin(0.8727)\right) \\ +\left(\dfrac{\sin^2(0.8727)}{2}+\dfrac{\sin(0.8727)(\cos(0.8727)-1)}{0.8727}\right)0.217^3 \\ -\dfrac{(0.795)^3\sin(0.8727)\cos(0.8727)}{12} \\ +0.189\left(1.889+0.217+\dfrac{0.217(1-\cos(0.8727))}{0.8727}\right) \\ \text{x}\left(7.507/2+\dfrac{0.217\sin(0.8727)}{0.8727}\right) \end{array}\right] \end{array}\right\}$$

$$= 0.118\{8.712+0.0051+0.1007+7.211-0.0206+1.636\}$$

$$= 2.082 \text{ in.}^4$$

f. Angle between x-axis and minor principal axis, in radians

$$\theta = \frac{\pi}{2}+\frac{1}{2}\tan^{-1}\left(\frac{2I_{xy}}{I_y - I_x}\right)$$

$$= \frac{\pi}{2}+\frac{1}{2}\tan^{-1}\left(\frac{2(2.082)}{1.076-7.763}\right)$$

$$= 1.292 \text{ radians} = 74.0 \text{ degrees}$$

g. Moment of inertia about x_2-axis, computed using angles in radians

$$I_{x2} = I_x\cos^2\theta + I_y\sin^2\theta - 2I_{xy}\sin\theta\cos\theta$$

$$= 7.763\cos^2(1.292)+1.076\sin^2(1.292)-(2)(2.082)\sin(1.292)\cos(1.292)$$

$$= 0.481 \text{ in.}^4$$

h. Moment of inertia about y_2-axis, computed using angles in radians

$$I_{y2} = I_x\sin^2\theta + I_y\cos^2\theta + 2I_{xy}\sin\theta\cos\theta$$

$$= 7.763\sin^2(1.292)+1.076\cos^2(1.292)+(2)(2.082)\sin(1.292)\cos(1.292)$$

$$= 8.36 \text{ in.}^4$$

i. Minimum radius of gyration, about x_2-axis

$$r_{min} = \sqrt{I_{x2}/A}$$

$$= \sqrt{0.481/0.822}$$

$$= 0.765 \text{ in.}$$

2. Torsional Properties

a. St. Venant torsional constant

$$J = \frac{t^3}{3}\left[a+2b+2u_1+\alpha(2c+2u_2)\right]$$

$$= \frac{0.059^3}{3}\left[7.507+(2)(1.889)+(2)(0.341)+1.0\left((2)(0.795)+(2)(0.189)\right)\right]$$

$$= 0.000954 \text{ in.}^4$$

b. Warping constant

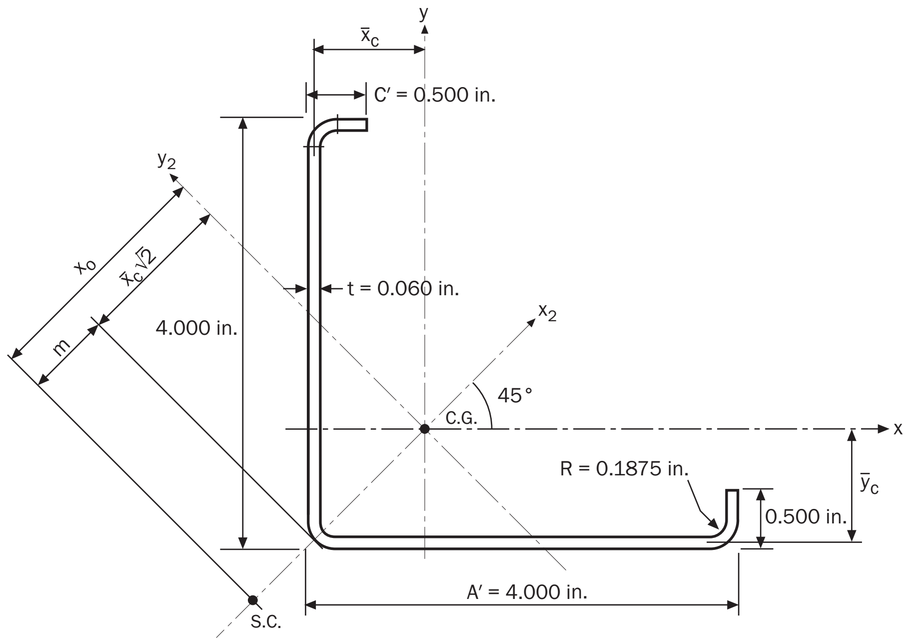
$$C_w = \frac{t}{12}\left\{\frac{\bar{a}^2\bar{b}^3(2\bar{a}+\bar{b})+\alpha\left[\begin{array}{l}\bar{b}^2\left[4\bar{c}^4+16\bar{b}\bar{c}^3+6\bar{a}^3\bar{c}+4\bar{a}^2\bar{b}\bar{c}+8\bar{a}\bar{c}^3\right]\\ +6\bar{a}\bar{b}\bar{c}^2(\bar{a}+\bar{b})(2\bar{b}\sin\gamma+\bar{a}\cos\gamma)\\ +4\bar{a}\bar{b}\bar{c}^3(2\bar{a}+4\bar{b}+\bar{c})\sin\gamma\cos\gamma\\ +\bar{c}^3\left[2\bar{a}^3+4\bar{a}^2\bar{b}-8\bar{a}\bar{b}^2+\bar{a}^2\bar{c}-16\bar{b}^3-4\bar{b}^2\bar{c}\right]\cos^2\gamma\end{array}\right]}{\bar{a}+2(\bar{b}+\alpha\bar{c})}\right\}$$

$$C_w = \frac{0.059}{12}\left\{\frac{\left((7.941)^2(2.207)^3\left[(2)(7.941)+2.207\right]\right)+(1.0)\left[\begin{array}{l}(2.207)^2\left[\begin{array}{l}(4)(0.896)^4+(16)(2.207)(0.896)^3\\+(6)(7.941)^3(0.896)\\+(4)(7.941)^2(2.207)(0.896)\\+(8)(7.941)(0.896)^3\end{array}\right]\\+(6)(7.941)(2.207)(0.896)^2(7.941+2.207)\left[\begin{array}{l}(2)(2.207)\sin(0.8727)\\+(7.941)\cos(0.8727)\end{array}\right]\\+(4)(7.941)(2.207)(0.896)^3\left[\begin{array}{l}(2)(7.941)\\+(4)(2.207)\\+0.896\end{array}\right]\sin(0.8727)\cos(0.8727)\\+(0.896)^3\left[\begin{array}{l}(2)(7.941)^3+(4)(7.941)^2(2.207)\\-(8)(7.941)(2.207)^2\\+(7.941)^2(0.896)\\-16(2.207)^3-4(2.207)^2(0.896)\end{array}\right]\cos^2(0.8727)\end{array}\right]}{7.941+2\left(2.207+1.0(0.896)\right)}\right\}$$

$$= \frac{0.059}{12}\left(\frac{12262+1.0(15901+7270+635.8+331.6)}{14.15}\right)$$

$$= 12.6 \text{ in.}^6$$

Example I-4: Equal Leg Angle With Lips - Gross Section Properties



Given:

1. Section: 4LS4x060 as shown above

Required:

1. Axial and flexural properties
2. Torsional properties

Solution:

1. Axial and Flexural Properties

a. Basic parameters

$A' = 4.000$ in.

$C' = 0.500$ in.

$t = 0.060$ in.

$R = 0.1875$ in.

$\alpha = 1.0$ (section has stiffener lips)

$r = R + t/2 = 0.1875 + 0.060/2 = 0.218$ in.

$$a = A' - \left[r + t/2 + \alpha\left(r + t/2\right)\right]$$
$$= 4.000 - \left[0.218 + 0.060/2 + 1.0\left(0.218 + 0.060/2\right)\right] = 3.504 \text{ in.}$$

$$\bar{a} = A' - \left[t/2 + \alpha t/2\right]$$
$$= 4.000 - \left[0.060/2 + (1.0)(0.060)/2\right] = 3.940 \text{ in.}$$

$$c = \alpha\left[C' - \left(r + t/2\right)\right]$$
$$= (1.0)\left[0.500 - \left(0.218 + 0.060/2\right)\right] = 0.252 \text{ in.}$$

$$\bar{c} = \alpha\left[C' - t/2\right]$$
$$= (1.0)\left[0.500 - 0.060/2\right] = 0.470 \text{ in.}$$

$$u = \pi r/2 = \pi(0.218)/2 = 0.342 \text{ in.}$$

b. Cross-section area

$$A = t\left[2a + u + \alpha(2c + 2u)\right]$$

$$= 0.060\left[(2)(3.504) + 0.342 + (1.0)\left((2)(0.252) + (2)(0.342)\right)\right]$$

$$= 0.512 \text{ in.}^2$$

c. Distance between centroid and centerlines of webs

$$\bar{x}_c = \bar{y}_c = \frac{t}{A}\left\{a\left(\frac{a}{2} + r\right) + u(0.363r) + \alpha\left[c\left(a + \frac{c}{2} + 3r\right) + u(a + 2r)\right]\right\}$$

$$= \frac{0.060}{0.512}\left\{\begin{array}{l}(3.504)\left(\dfrac{3.504}{2} + 0.218\right) + (0.342)(0.363)(0.218) \\ +(1.0)\left[0.252\left(3.504 + \dfrac{0.252}{2} + (3)(0.218)\right) + 0.342\left(3.504 + (2)(0.218)\right)\right]\end{array}\right\}$$

$$= 1.097 \text{ in.}$$

d. Moment of inertia about x- and y-axes

$$I_x = I_y = t\left\{\begin{array}{l}a\left(\dfrac{a}{2} + r\right)^2 + \dfrac{a^3}{12} + u(0.363r)^2 + 0.149r^3 \\ +\alpha\left[\begin{array}{l}c(a + 2r)^2 + \dfrac{c^3}{12} + c\left(\dfrac{c}{2} + r\right)^2 + u(a + 1.637r)^2 \\ +u(0.363r)^2 + (2)(0.149)r^3\end{array}\right]\end{array}\right\} - A\bar{x}_c^2$$

$$= 0.060\left\{\begin{array}{l}3.504\left(\dfrac{3.504}{2} + 0.218\right)^2 + \dfrac{(3.504)^3}{12} \\ +(0.342)\left[(0.363)(0.218)\right]^2 + (0.149)(0.218)^3 \\ +1.0\left[\begin{array}{l}0.252\left[3.504 + (2)(0.218)\right]^2 + \dfrac{(0.252)^3}{12} \\ +0.252\left(\dfrac{0.252}{2} + 0.218\right)^2 \\ +0.342\left[3.504 + (1.637)(0.218)\right]^2 \\ +0.342\left[0.363(0.218)\right]^2 + (2)(0.149)(0.218)^3\end{array}\right]\end{array}\right\} - (0.512)(1.097)^2$$

$$= 0.958 \text{ in.}^4$$

e. Product of inertia about x- and y- axes

$$I_{xy} = t\left\{\begin{array}{l}-0.137r^3 + u(0.363r)^2 \\ +2\alpha\left[c(a + 2r)\left(\dfrac{c}{2} + r\right) + 0.137r^3 + u(a + 1.637r)(0.363r)\right]\end{array}\right\} - A\bar{x}_c\bar{y}_c$$

$$I_{xy} = 0.060\left\{\begin{array}{l}-0.137(0.218)^3 + 0.342\left[0.363(0.218)\right]^2 \\ +(2)(1.0)\left[\begin{array}{l}0.252\left[3.504 + 2(0.218)\right]\left(\dfrac{0.252}{2} + 0.218\right) \\ +0.137(0.218)^3 \\ +0.342\left[3.504 + 1.637(0.218)\right](0.363)(0.218)\end{array}\right]\end{array}\right\} - (0.512)(1.097)(1.097)$$

$= 0.060\{-0.00142+0.00214+(2.0)[0.3416+0.00142+0.1045]\}-0.6161$

$= -0.562 \text{ in.}^4$

f. Moment of inertia about y_2-axis

$I_{y2} = I_x + I_{xy}$

$= 0.958+(-0.562)=0.396 \text{ in.}^4$

2. Torsional Properties

a. Distance between shear center and centerline of square corner

$$m = \frac{\bar{a}\bar{c}^2\sqrt{2}}{2}\frac{(3\bar{a}-2\bar{c})}{\left[2\bar{a}^3-(\bar{a}-\bar{c})^3\right]}$$

$$= \frac{(3.940)(0.470)^2\sqrt{2}}{2}\frac{[(3)(3.940)-(2)(0.470)]}{\left[(2)(3.940)^3-(3.940-0.470)^3\right]} = 0.083 \text{ in.}$$

b. St. Venant torsion constant

$$J = \frac{t^3}{3}\left[2a+u+\alpha(2c+2u)\right]$$

$$= \frac{0.060^3}{3}\left[(2)(3.504)+0.342+1.0\left(2(0.252)+2(0.342)\right)\right] = 0.000615 \text{ in.}^4$$

c. Warping constant

$$C_w = \frac{\bar{a}^4\bar{c}^3t}{6}\frac{(4\bar{a}+3\bar{c})}{\left[2\bar{a}^3-(\bar{a}-\bar{c})^3\right]}$$

$$= \frac{(3.940)^4(0.470)^3(0.060)}{6}\frac{(4)(3.940)+(3)(0.470)}{\left[(2)(3.940)^3-(3.940-0.470)^3\right]} = 0.0533 \text{ in.}^6$$

d. Distance from centroid to shear center

$$x_o = -\left(\bar{x}_c\sqrt{2}+m\right)$$

$$= -\left(1.097\sqrt{2}+0.083\right) = -1.634 \text{ in.}$$

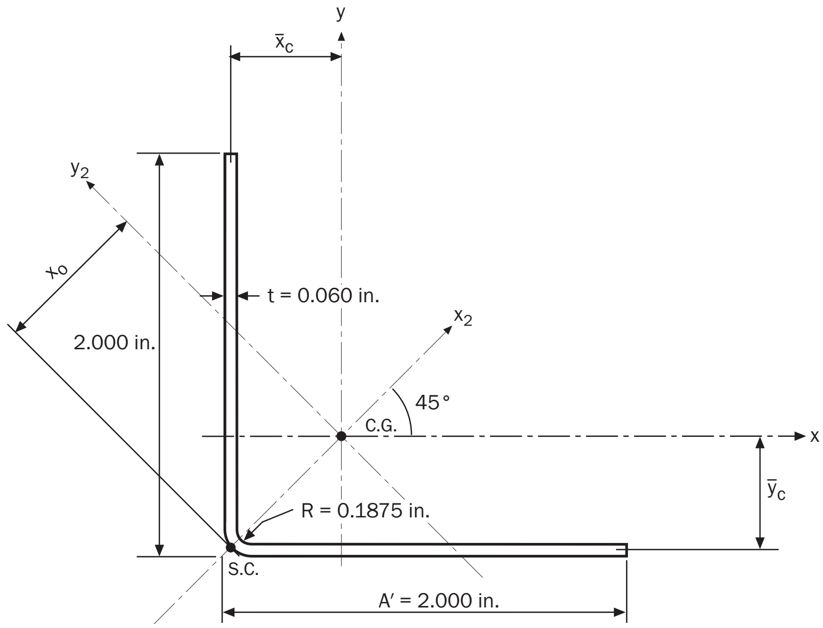
e. Parameter used to determine elastic critical moment

$$j = \frac{\sqrt{2}t}{48I_{y2}}\left[\bar{a}^4+4\bar{a}^3\bar{c}-6\bar{a}^2\bar{c}^2+\bar{c}^4\right]-x_o$$

$$= \frac{\sqrt{2}(0.060)}{(48)(0.396)}\begin{bmatrix}(3.940)^4+(4)(3.940)^3(0.470) \\ -(6)(3.940)^2(0.470)^2+(0.470)^4\end{bmatrix}-(-1.634)$$

$= 3.13$ in.

Example I-5: Equal Leg Angle Without Lips - Gross Section Properties



Given:

1. Section 2LU2x060 as shown above

Required:

1. Axial and flexural properties
2. Torsional properties

Solution:

1. Axial and Flexural Properties

a. Basic parameters:

$A' = 2.000$ in.

$C' = 0.000$ in.

$t = 0.060$ in.

$R = 0.1875$ in.

$\alpha = 0.0$ (section does not have stiffener lips)

$r = R + t/2 = 0.1875 + 0.060/2 = 0.218$ in.

$a = A' - \left[r + t/2 + \alpha\left(r + t/2\right)\right]$

$= 2.000 - \left[0.218 + 0.060/2 + 0.0\right] = 1.752$ in.

$\bar{a} = A' - \left[t/2 + \alpha t/2\right]$

$= 2.000 - \left[0.060/2 + 0.0\right] = 1.970$ in.

$c = \alpha\left[C' - \left(r + t/2\right)\right]$

$= 0.0$ in.

$\bar{c} = \alpha\left[C' - \left(t/2\right)\right]$

$= 0.0$ in.

$u = \pi r/2$

$= \pi\left(0.218\right)/2 = 0.342$ in.

b. Cross-section area

$$A = t\left[2a + u + \alpha\left(2c + 2u\right)\right]$$
$$= 0.060\left[\left(2\right)\left(1.752\right) + 0.342 + 0.0\right]$$
$$= 0.231 \text{ in.}^2$$

c. Distance between centroid and centerlines of webs

$$\bar{x}_c = \bar{y}_c = \frac{t}{A}\left\{a\left(\frac{a}{2} + r\right) + u\left(0.363r\right) + \alpha\left[c\left(a + \frac{c}{2} + 3r\right) + u\left(a + 2r\right)\right]\right\}$$
$$= \frac{0.060}{0.231}\left\{\left(1.752\right)\left(\frac{1.752}{2} + 0.218\right) + \left(0.342\right)\left(0.363\right)\left(0.218\right) + 0.0\right\}$$
$$= 0.505 \text{ in.}$$

d. Moment of inertia about x- and y-axes

$$I_x = I_y = t\left\{\begin{array}{l} a\left(\frac{a}{2} + r\right)^2 + \frac{a^3}{12} + u\left(0.363r\right)^2 + 0.149r^3 \\ +\alpha\left[\begin{array}{l} c\left(a + 2r\right)^2 + \frac{c^3}{12} + c\left(\frac{c}{2} + r\right)^2 + u\left(a + 1.637r\right)^2 \\ +u\left(0.363r\right)^2 + \left(2\right)\left(0.149\right)r^3 \end{array}\right] \end{array}\right\} - A\bar{x}_c^2$$
$$= 0.060\left\{\begin{array}{l} 1.752\left(\frac{1.752}{2} + 0.218\right)^2 + \frac{\left(1.752\right)^3}{12} \\ +\left(0.342\right)\left[\left(0.363\right)\left(0.218\right)\right]^2 + \left(0.149\right)\left(0.218\right)^3 + 0.0 \end{array}\right\} - \left(0.231\right)\left(0.505\right)^2$$
$$= 0.0940 \text{ in.}^4$$

e. Product of inertia about x- and y- axes

$$I_{xy} = t\left\{\begin{array}{l} -0.137r^3 + u\left(0.363r\right)^2 \\ +2\alpha\left[c\left(a + 2r\right)\left(\frac{c}{2} + r\right) + 0.137r^3 + u\left(a + 1.637r\right)\left(0.363r\right)\right] \end{array}\right\} - A\bar{x}_c\bar{y}_c$$
$$= 0.060\left\{-0.137\left(0.218\right)^3 + 0.342\left[0.363\left(0.218\right)\right]^2 + 0.0\right\} - \left(0.231\right)\left(0.505\right)\left(0.505\right)$$
$$= -0.0589 \text{ in.}^4$$

f. Moment of inertia about y_2-axis

$$I_{y2} = I_x + I_{xy}$$
$$= 0.0940 + \left(-0.0589\right)$$
$$= 0.0351 \text{ in.}^4$$

2. Torsional Properties

a. Distance between shear center and centerline of square corner

$$m = \frac{\bar{a}\bar{c}^2\sqrt{2}}{2}\frac{\left(3\bar{a} - 2\bar{c}\right)}{\left[2\bar{a}^3 - \left(\bar{a} - \bar{c}\right)^3\right]}$$
$$= 0.000 \text{ in.}$$

b. St. Venant torsional constant

$$J = \frac{t^3}{3}\left[2a + u + \alpha\left(2c + 2u\right)\right]$$

$$= \frac{(0.060)^3}{3}\left[(2)(1.752)+0.342+0.0\right]$$

$= 0.000277$ in.^{4}

c. Warping constant

$$C_w = \frac{\bar{a}^4\bar{c}^3 t}{6}\frac{(4\bar{a}+3\bar{c})}{\left[2\bar{a}^3-(\bar{a}-\bar{c})^3\right]}$$

$= 0.000$ in.^{6}

d. Distance from centroid to shear center

$$x_o = -\left(\bar{x}_c\sqrt{2}+m\right)$$

$$= -\left(0.505\sqrt{2}+0.000\right)$$

$= -0.714$ in.

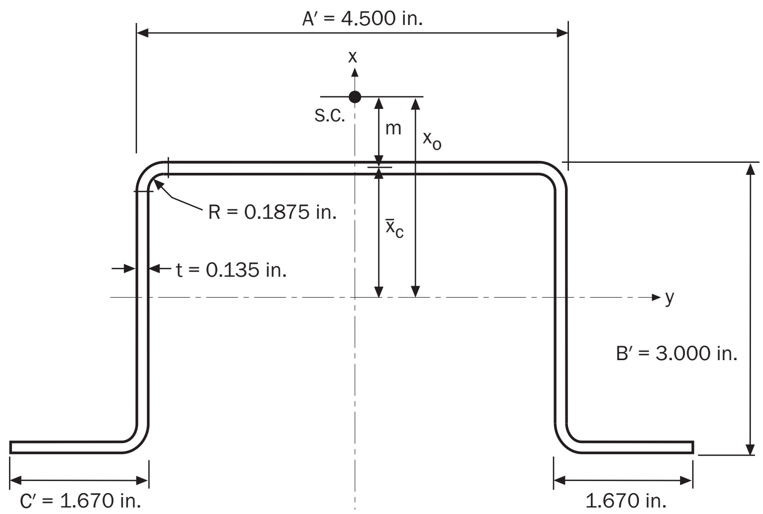
e. Parameter used to determine elastic critical moment

$$j = \frac{\sqrt{2}t}{48I_{y2}}\left[\bar{a}^4+4\bar{a}^3\bar{c}-6\bar{a}^2\bar{c}^2+\bar{c}^4\right]-x_o$$

$$= \frac{\sqrt{2}(0.060)}{(48)(0.0351)}\left[(1.970)^4+0.0-0.0+0.0\right]-(-0.714)$$

$= 1.47$ in.

Example I-6: Hat Section Without Lips - Gross Section Properties



Given:

1. Section: 3HU4.5x135 as shown above

Required:

1. Axial and flexural properties
2. Torsional properties

Solution:

1. Axial and Flexural Properties

a. Basic parameters

$A' = 4.50$ in.

$B' = 3.00$ in.

$C' = 1.670$ in.

$t = 0.135$ in.

$R = 0.1875$ in.

$\alpha = 1.0$

$r = R + t/2$

$= 0.1875 + 0.135/2 = 0.255$ in.

$a = A' - (2r + t)$

$= 4.50 - [(2)(0.255) + 0.135] = 3.855$ in.

$\bar{a} = A' - t$

$= 4.50 - 0.135 = 4.365$ in.

$$b = B' - \left[r + \frac{t}{2} + \alpha\left(r + \frac{t}{2}\right)\right]$$

$= 3.00 - [0.255 + 0.135/2 + 1.0(0.255 + 0.135/2)] = 2.355$ in.

$\bar{b} = B' - (t/2 + \alpha t/2)$

$= 3.00 - [0.135/2 + (1.0)(0.135/2)] = 2.865$ in.

$c = \alpha[C' - (r + t/2)]$

$= 1.0[1.670 - (0.255 + 0.135/2)] = 1.348$ in.

$$\bar{c} = \alpha(C' - t/2)$$
$$= 1.0\left[1.670 - (0.135/2)\right] = 1.603 \text{ in.}$$
$$u = \pi r/2$$
$$= \pi(0.255)/2 = 0.401 \text{ in.}$$

b. Cross-section area

$$A = t\left[a + 2b + 2u + \alpha(2c + 2u)\right]$$
$$= 0.135\left[3.855 + (2)(2.355) + (2)(0.401) + 1.0\{(2)(1.348) + (2)(0.401)\}\right]$$
$$= 1.737 \text{ in.}^2$$

c. Moment of inertia about the x-axis

$$I_x = 2t\left\{\begin{aligned}&0.0417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149r^3\\ &+\alpha\left[0.0833c^3 + \frac{c}{4}(a + c + 4r)^2 + u\left(\frac{a}{2} + 1.363r\right)^2 + 0.149r^3\right]\end{aligned}\right\}$$

$$= (2)(0.135)\left\{\begin{aligned}&0.0417(3.855)^3 + 2.355(3.855/2 + 0.255)^2\\ &+0.401\left[3.855/2 + 0.637(0.255)\right]^2 + 0.149(0.255)^3\\ &+1.0\left[\begin{aligned}&(0.0833)(1.348)^3 + \frac{1.348}{4}(3.855 + 1.348 + (4)(0.255))^2\\ &+0.401(3.855/2 + (1.363)(0.255))^2 + (0.149)(0.255)^3\end{aligned}\right]\end{aligned}\right\}$$

$$= 0.270\{2.389 + 11.22 + 1.752 + 0.0025 + 1.0[0.2040 + 13.05 + 2.076 + 0.0025]\}$$
$$= 8.29 \text{ in.}^4$$

d. Distance between centroid and web centerline

$$\bar{x}_c = \frac{2t}{A}\{b(b/2 + r) + u(0.363r) + \alpha\left[u(b + 1.637r) + c(b + 2r)\right]\}$$

$$= \frac{(2)(0.135)}{1.737}\left\{\begin{aligned}&2.355(2.355/2 + 0.255) + (0.401)(0.363)(0.255)\\ &+1.0\left[0.401(2.355 + (1.637)(0.255)) + 1.348(2.355 + (2)(0.255))\right]\end{aligned}\right\}$$

$$= 0.1554\{3.374 + 0.0371 + 1.0\ [1.112 + 3.862]\}$$
$$= 1.303 \text{ in.}$$

e. Moment of inertia about the y-axis

$$I_y = 2t\left\{\begin{aligned}&b(b/2 + r)^2 + b^3/12 + 0.356r^3\\ &+\alpha\left[c(b + 2r)^2 + u(b + 1.637r)^2 + 0.149r^3\right]\end{aligned}\right\} - A\bar{x}_c^{\,2}$$

$$= (2)(0.135)\left\{\begin{aligned}&2.355(2.355/2 + 0.255)^2 + \frac{(2.355)^3}{12} + 0.356(0.255)^3\\ &+1.0\left[\begin{aligned}&1.348(2.355 + (2)(0.255))^2\\ &+0.401(2.355 + 1.637(0.255))^2\\ &+0.149(0.255)^3\end{aligned}\right]\end{aligned}\right\} - (1.737)(1.303)^2$$

$$= 0.270\{4.833 + 1.088 + 0.0059 + 1.0[11.06 + 3.082 + 0.0025]\} - 2.949$$
$$= 2.470 \text{ in.}^4$$

f. Distance between shear center and web centerline

$$m = \bar{b}\left[\frac{3\bar{a}^2\bar{b}+\alpha\bar{c}\left(6\bar{a}^2-8\bar{c}^2\right)}{\bar{a}^3+6\bar{a}^2\bar{b}+\alpha\bar{c}\left[8\bar{c}^2+12\bar{a}\bar{c}+6\bar{a}^2\right]}\right]$$

$$= 2.865\left[\frac{(3)(4.365)^2(2.865)+(1.0)(1.603)\left((6)(4.365)^2-(8)(1.603)^2\right)}{\begin{array}{c}(4.365)^3+(6)(4.365)^2(2.865)\\+(1.0)(1.603)\left[(8)(1.603)^2+(12)(4.365)(1.603)+(6)(4.365)^2\right]\end{array}}\right]$$

$$= 2.865\left[\frac{163.8+150.3}{83.17+327.5+350.8}\right]$$

$$= 1.182 \text{ in.}$$

g. Distance between centroid and shear center

$$x_o = -\left(\bar{x}_c+m\right)$$

$$= -(1.303+1.182)$$

$$= -2.485 \text{ in.}$$

2. Torsional Properties

a. St. Venant torsional constant

$$J = \frac{t^3}{3}\left[a+2b+2u+\alpha(2c+2u)\right]$$

$$= \frac{0.135^3}{3}\left[3.855+(2)(2.355)+(2)(0.401)+1.0\{(2)(1.348)+(2)(0.401)\}\right]$$

$$= 0.0106 \text{ in.}^4$$

b. Warping constant

$$C_w = \frac{\bar{a}^2\bar{b}^2t}{12}\left\{\frac{2\bar{a}^3\bar{b}+3\bar{a}^2\bar{b}^2+\alpha\left[\begin{array}{l}48\bar{c}^4+112\bar{b}\bar{c}^3+8\bar{a}\bar{c}^3-48\bar{a}\bar{b}\bar{c}^2\\-12\bar{a}^2\bar{c}^2+12\bar{a}^2\bar{b}\bar{c}+6\bar{a}^3\bar{c}\end{array}\right]}{6\bar{a}^2\bar{b}+\left(\bar{a}+\alpha 2\bar{c}\right)^3}\right\}$$

$$= \frac{(4.365)^2(2.865)^2(0.135)}{12}\left\{\frac{\begin{array}{l}(2)(4.365)^3(2.865)+(3)(4.365)^2(2.865)^2\\+1.0\left[\begin{array}{l}(48)(1.603)^4+(112)(2.865)(1.603)^3\\+(8)(4.365)(1.603)^3\\-(48)(4.365)(2.865)(1.603)^2-(12)(4.365)^2(1.603)^2\\+(12)(4.365)^2(2.865)(1.603)+(6)(4.365)^3(1.603)\end{array}\right]\end{array}}{(6)(4.365)^2(2.865)+\left(4.365+(1.0)(2)(1.603)\right)^3}\right\}$$

$$= 1.759\left\{\frac{476.6+469.2+1.0\left[\begin{array}{l}316.9+1322+143.8-1543\\-587.5+1050+799.9\end{array}\right]}{327.5+434.0}\right\}$$

$$= 5.65 \text{ in.}^6$$

c. Parameter used in determination of elastic critical moment

$$\beta_w = -\left[\frac{t\bar{x}_c\bar{a}^3}{12} + t\bar{x}_c^{\,3}\bar{a}\right]$$

$$= -\left[\frac{(0.135)(1.303)(4.365)^3}{12} + (0.135)(1.303)^3(4.365)\right]$$

$$= -2.523 \text{ in.}^5$$

$$\beta_f = \frac{t}{2}\left[\left(\bar{b}-\bar{x}_c\right)^4 - \bar{x}_c^{\,4}\right] + \frac{t\bar{a}^2}{4}\left[\left(\bar{b}-\bar{x}_c\right)^2 - \bar{x}_c^{\,2}\right]$$

$$= \frac{0.135}{2}\left[(2.865-1.303)^4 - (1.303)^4\right] + \frac{(0.135)(4.365)^2}{4}\left[(2.865-1.303)^2 - 1.303^2\right]$$

$$= 0.6844 \text{ in.}^5$$

$$\beta_l = 2\bar{c}t\left(\bar{b}-\bar{x}_c\right)^3 + \frac{2}{3}t\left(\bar{b}-\bar{x}_c\right)\left[\left(\frac{\bar{a}}{2}+\bar{c}\right)^3 - \left(\frac{\bar{a}}{2}\right)^3\right]$$

$$= \left\{\begin{aligned} &(2)(1.603)(0.135)(2.865-1.303)^3 \\ &+\frac{2}{3}(0.135)(2.865-1.303)\left[\left(\frac{4.365}{2}+1.603\right)^3 - \left(\frac{4.365}{2}\right)^3\right]\end{aligned}\right\}$$

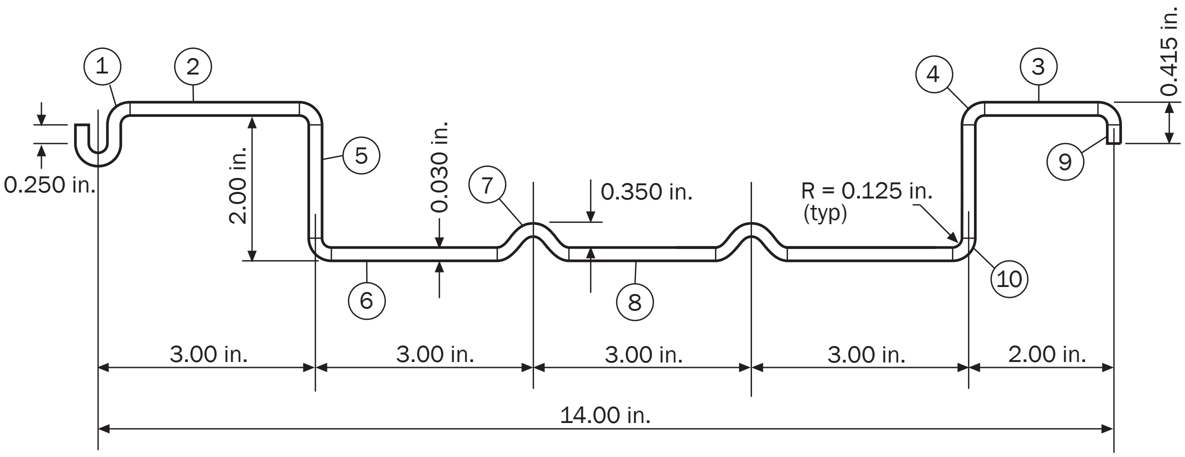
$$= 7.814 \text{ in.}^5$$

$$j = \frac{1}{2I_y}\left(\beta_w + \beta_f + \beta_l\right) - x_o$$

$$= \frac{1}{(2)(2.470)}(-2.523 + 0.6844 + 7.814) - (-2.485)$$

$$= 3.69 \text{ in.}$$

Example I-7: Wall Panel Section With Intermediate Stiffeners - Gross Section Properties



Given:

1. Section: Shown in sketch above

Required:

1. Gross section properties

Solution:

Since no closed formed solution is available, the properties must be determined by parts.

1. Elements 4 and 10

90° corners:

$r = R + t/2 = 0.125 + 0.030/2 = 0.140$ in.

Length of arc:

$u = 1.57r = (1.57)(0.140) = 0.220$ in.

Distance of c.g. from center of radius:

$c = 0.637r = (0.637)(0.140) = 0.089$ in.

Distance of c.g. from top of panel:

$y = 0.125 + 0.030 - 0.089 = 0.066$ in. (element 4)

$y = 2.00 + (0.089 - 0.125) = 1.964$ in. (element 10)

I'_x (each arc):

$I'_x = 0.149r^3 = (0.149)(0.140)^3 = 0.0004$ in.^{3}

2. Element 7

$r = 0.140$ in., $\theta = 45° = 0.785$ radians

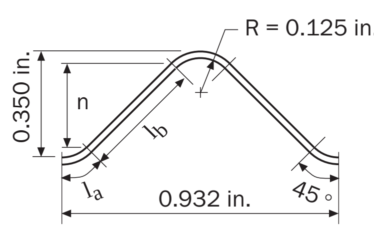
$c_1 = \dfrac{r \sin\theta}{\theta} = (0.140)(0.707)/0.785 = 0.126$ in.

$n = 0.350 - (2)(0.140)(1 - \cos(0.785))$

$= 0.350 - 0.082 = 0.268$ in.

$l_b = 0.268/\sin(0.785) = 0.379$ in.

$l_a = \theta r = (0.785)(0.140) = 0.110$ in.



I'_1 (straight portions):

$I'_1 = (2)(1/12)(l_b)n^2$

$= (2)(1/12)(0.379)(0.268)^2 = 0.0045 \text{ in.}^3$

I'_1 (each arc):

$$I'_1 = \left[\frac{\theta + \sin\theta\cos\theta}{2} - \frac{\sin^2\theta}{\theta}\right]r^3$$

$$= \left[\frac{0.785 + \sin(0.785)\cos(0.785)}{2} - \frac{\sin^2(0.785)}{0.785}\right](0.140)^3$$

$= 0.000017 \text{ in.}^3 \approx 0$

By inspection, take advantage of symmetry and locate reference axis at 1/2 element.

depth $= (0.350 + 0.030)/2 = 0.190$ in.

Segment	L (in.)	y (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 about own axis (in.^{3})
Upper Radii	2(0.110)=0.220	0.161	0.035	0.0057	----
Straight Segments	2(0.379) = 0.758	0.000	0.000	-----	0.0045
Lower Radii	2(0.110)=0.220	-0.161	- 0.035	0.0057	----
Sum $\sum$	1.198		0.000	0.0114	0.0045

$y_{cg} = \sum Ly/\sum L$

$= 0.000/1.198 = 0.000$ in. (at centerline as expected)

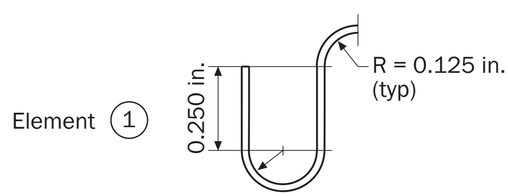
$\sum I'_x = \sum Ly^2 + \sum I'_1 - y_{cg}{}^2 \sum L$

$= 0.0114 + 0.0045 - (0.000)^2(1.198) = 0.0159 \text{ in.}^3$

Distance of c.g. from top of panel

$\bar{y} = 2.030 - (0.350 + 0.030)/2 = 1.840$ in.

3. Element 1



Segment	L (in.)	y from top flange (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 about own axis (in.^{3})
90° Corner	0.220	0.155 - 0.089 = 0.066	0.015	0.0010	----
Straight Segments	(2)(0.250) = 0.500	0.280	0.140	0.0392	0.0026
Semi-Circle	(2)(0.220) = 0.440	0.405 + 0.089 = 0.494	0.217	0.1074	0.0008
Sum $\sum$	1.160		0.372	0.1476	0.0034

$y_{cg} = \sum Ly/\sum L$

$= 0.372/1.160 = 0.321$ in.

$$\sum I'_x = \sum Ly^2 + \sum I'_1 - y_{cg}{}^2 \sum L$$

$$= 0.1476 + 0.0034 - (0.321)^2 (1.160) = 0.0314 \text{ in.}^3$$

4. Total Section

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
1	1.160	0.321	0.372	0.120	0.031
2	2.580	0.015	0.039	0.001	---
3	1.720	0.015	0.026	---	---
4	(3)(0.220) = 0.660	0.066	0.044	0.003	0.001
5	(2)(1.720) = 3.440	1.015	3.492	3.544	0.848
6	(2)(2.394) = 4.788	2.015	9.648	19.440	---
7	(2)(1.198) = 2.396	1.840	4.409	8.112	0.032
8	2.068	2.015	4.167	8.397	---
9	0.260	0.285	0.074	0.021	0.001
10	(2)(0.220) = 0.440	1.964	0.864	1.697	0.001
Sum $\sum$	19.512		23.135	41.335	0.914

$$\bar{y} = \sum Ly / \sum L$$

$$= 23.135/19.512 = 1.186 \text{ in. from top fiber}$$

$$I_x = \left[\sum Ly^2 + \sum I'_1 - \bar{y}^2 \sum L\right] t$$

$$= \left[41.335 + 0.914 - (1.186)^2 (19.512)\right] 0.030 = 0.444 \text{ in.}^4$$

$$S_{ft} = I_x/\bar{y}$$

$$= 0.444/1.186 = 0.374 \text{ in.}^3$$

$$S_{fb} = I_x/(d - \bar{y})$$

$$= 0.444/(2.030 - 1.186) = 0.526 \text{ in.}^3$$

$$A = t\sum L = (0.030)(19.512) = 0.585 \text{ in.}^2$$

3.6 Effective Section Properties

Specification provisions dealing with the strength and serviceability of flexural and compression members require the calculation of effective section properties, I_e, S_e and A_e, to account for strength and stiffness reductions due to local buckling of slender compression elements. Effective section properties are calculated by substituting a fictitious reduced width in place of the true flat width of each flat element subject to local buckling. The theoretical basis of this technique is presented in Section B2 of the *Commentary* and in texts such as Yu(2010)[1] and Hancock et. al. (2001).[2]

The general procedure is as follows:

1. Establish the distribution of stresses in the cross-section. The cross section stress distribution and maximum stress are given in the applicable section of Chapter C or D. Effective areas, A_e, are calculated using a uniform axial stress. Effective flexural properties, I_e and S_e, are calculated using a flexural stress distribution. Effective section properties for strength are calculated either at a maximum stress level of F_y or, in the case of members subject to flexural, flexural-torsional, torsional or lateral-torsional buckling, at the nominal buckling stress, F_n. Effective section properties for serviceability are calculated at the serviceability stress levels.
2. Under the assumed stress distribution, identify each flat element subject to either uniform compression or a stress gradient with compression on at least one edge. For each such element, determine the corresponding section from *Specification* Sections B2 through B5, which are categorized by boundary conditions of the element and stress distribution in the element. Stiffened elements are elements attached to other elements on all edges. Unstiffened elements have one edge free. There are several *Specification* sections that address complex conditions, such as flanges with stiffener lips and flat elements with multiple stiffeners. When such specialized sections apply, they should be used in lieu of the more general provisions.
3. Using the stresses and appropriate *Specification* section identified above, determine the flat width, w, stress level, f, and plate buckling coefficient, k, of each flat compression element. These parameters are then used in *Specification* Section B2 to calculate the effective width of each element, b.
4. Using the effective widths of flat compression elements calculated above with the full properties of other elements, recalculate the effective section properties of the cross-section.

In many situations the calculation of effective section properties is iterative in nature. For example, in calculating the strength of a beam, the stresses in the cross-section are usually calculated initially using the centroid of the full cross-section. Using these stresses, the effective widths of the compression elements are determined. Using these effective widths, a new cross-section centroid is determined. Using the newly determined centroid, stresses are recalculated and the procedure is repeated until convergence, which is indicated when the centroidal axis stops shifting. Upon convergence, the effective moment of inertia, I_e, and effective section modulus, S_e are then calculated using the final effective widths.

In the examples presented herein, for beam section properties, the effective section properties are computed using one of the following two procedures:

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[1] Yu, W.W. and Roger A. LaBoube, *Cold-Formed Steel Design – Fourth Edition*, John Wiley & Sons, New York, NY, 2010.

[2] Hancock, G.J., Murray, T.M., Ellifritt, D.S., *Cold-Formed Steel Structures to the AISI Specification*, Marcel Dekker, Inc., New York, NY, 2001.

1. If the neutral axis of the effective section is at mid-depth of the section or closer to the tension flange than to the compression flange, the maximum stress occurs in the compression flange, thus the effective width of the compression flange and the effective width of the web elements can be calculated assuming an extreme compression fiber stress equal to the yield stress or other specified maximum stress. This case is not iterative in nature unless the web is not fully effective.

2. If the neutral axis of the effective section is closer to the compression flange than to the tension flange, the compressive stress must be known in order to calculate the effective widths of the compression elements. The compressive stresses depend upon the location of the neutral axis which in turn depends on the effective widths, thus the solution is iterative in nature. Some of the example problems demonstrate this iterative procedure.

For uniformly stressed sections, i.e. column sections, the compression stress does not vary with the distance from the centroid as in flexural members, thus iteration is not required.

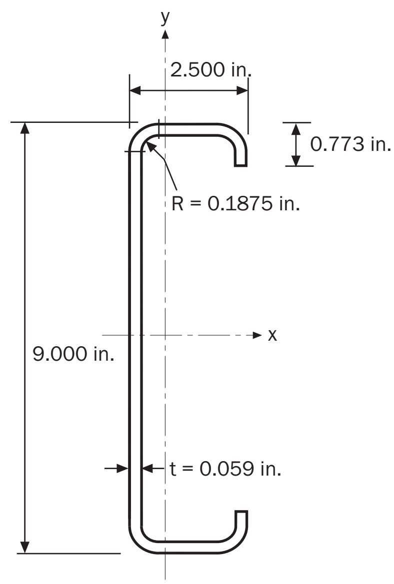
Although curved elements with large ratios of radius to thickness are also subject to local buckling, the *Specification* does not include provisions for the calculation of the effective properties of these elements. For members including such elements, the use of the Direct Strength Method in Appendix 1 is recommended. In addition, when the corner radius to thickness ratio exceeds 10, the reduced restraint to the connected element should be considered through application of a rational analysis method such as the one provided in *Commentary* Section B1.3, or alternatively, the Direct Strength Method provided in Appendix 1.

3.7 Effective Section Properties - Example Problems

The following example problems are intended to illustrate various provisions of the *Specification*, especially those involving the calculation of effective section properties. These should be used in conjunction with the other parts of the *Design Manual*. Many of the calculations are referenced in Parts II and III.

The calculations were done using the same guidelines on precision presented in Section 3.5 of Part I of the *Design Manual*.

Example I-8: C-Section With Lips - Effective Section Properties



Given:

1. Steel: F_y = 55 ksi
2. Section: 9CS2.5x059 as shown above

Required:

1. Effective section modulus, S_e, based on initiation of yielding
2. Effective area, A_e, at a uniform compressive stress of 37.25 ksi (as used in Example III-1)

Solution:

See Example I-1 for basic geometric parameters.

1. Effective Section Modulus, S_e, at Initiation of Yielding

An iterative approach is generally required.

For the first iteration, assume a compression stress of f = F_y = 55 ksi in the top fiber of the section and that the neutral axis is 4.500 in. below the top fiber.

a. Compression flange: from Section B4

$w = b = 2.007$ in.

$w/t = 2.007/0.059 = 34.02 \leq 60$ OK (Section B1.1(a)(1))

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-7)

$= 1.28\sqrt{29500/55} = 29.64 \therefore w/t > 0.328S \Rightarrow$ calculate effective width of flange

Compute k of the flange based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \leq t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4\text{-}8)$$

$$= 399(0.059)^4\left[\frac{34.02}{29.64} - 0.328\right]^3 \leq (0.059)^4\left[115\frac{34.02}{29.64} + 5\right]$$

$= 0.00266$ in.^{4} > 0.00166 in.^{4} $\therefore I_a = 0.00166$ in.^{4}

$d = c = 0.527$ in.

$\theta = 90$ degrees

$I_s = \left(d^3 t \sin^2\theta\right)/12$ (*Eq*. B4-10)

$= (0.527)^3(0.059)\sin^2(90°)/12 = 0.000720\text{ in.}^4$

$R_I = I_s/I_a \le 1$ (*Eq*. B4-9)

$= 0.000720/0.00166 = 0.434 < 1$ OK

$n = \left[0.582 - \dfrac{w/t}{4S}\right] \ge \dfrac{1}{3}$ (*Eq*. B4-11)

$= \left[0.582 - \dfrac{34.02}{(4)(29.64)}\right] \ge \dfrac{1}{3}$

$= 0.295 < 1/3 \quad \therefore\ n = 1/3$

$D = 0.773$ in.

$D/w = 0.773/2.007 = 0.385 < 0.8$ OK (From Table B4-1)

$k = \left(4.82 - \dfrac{5D}{w}\right)(R_I)^n + 0.43 \le 4$ (From Table B4-1)

$= \left(4.82 - \dfrac{(5)(0.773)}{2.007}\right)(0.434)^{1/3} + 0.43 = 2.62 < 4$ OK

$F_{cr} = k\dfrac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\dfrac{t}{w}\right)^2$ (*Eq*. B2.1-5)

$= 2.62\dfrac{\pi^2(29500)}{12\left(1-0.3^2\right)}\left(\dfrac{1}{34.02}\right)^2 = 60.36\text{ ksi}$

$\lambda = \sqrt{\dfrac{f}{F_{cr}}}$ (*Eq*. B2.1-4)

$= \sqrt{\dfrac{55}{60.36}} = 0.955 > 0.673 \therefore$ flange is subject to local buckling

$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq*. B2.1-3)

$= (1 - 0.22/0.955)/0.955 = 0.806$

$b = \rho w$ (*Eq*. B2.1-2)

$= (0.806)(2.007) = 1.618\text{ in.}$

b. Stiffener lip: from Section B3.2-a

$w/t = d/t = 0.527/0.059 = 8.93$

Maximum stress in lip (by similar triangles)

$f = f_1 = 55[4.500 - 0.059/2 - 0.217]/4.500 = 51.99\text{ ksi}$

$f_2 = 55[4.500 - 0.773]/4.500 = 45.55\text{ ksi}$

$\psi = |f_2/f_1|$ (*Eq*. B3.2-1)

$= |45.55/51.99| = 0.876$

$k = \dfrac{0.578}{\psi + 0.34}$ (*Eq*. B3.2-2)

$= \dfrac{0.578}{0.876 + 0.34} = 0.475$

$F_{cr} = k \dfrac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\dfrac{t}{w}\right)^2$ (*Eq.* B2.1-5)

$= 0.475 \dfrac{\pi^2 (29500)}{12\left(1-0.3^2\right)}\left(\dfrac{1}{8.93}\right)^2 = 158.8$ ksi

$\lambda = \sqrt{\dfrac{f}{F_{cr}}}$ (*Eq.* B2.1-4)

$= \sqrt{\dfrac{51.99}{158.8}} = 0.572 < 0.673 \therefore$ lip is not subject to local buckling

$d'_s = d = 0.527$ in.

$d_s = d'_s(R_I)$ (*Eq.* B4-6)

$= (0.527)(0.434) = 0.229$ in.

c. Web: from Section B2.3

$w/t = 8.507/0.059 = 144.2$

$\psi = |f_2/f_1|$ (*Eq.* B2.3-1)

Assuming the neutral axis is at the section centerline, determine the maximum flexural stress in the web by similar triangles.

$f_1 = (55)(4.500 - 0.059/2 - 0.217)/4.500 = 51.99$ ksi

By symmetry

$f_2 = -f_1 = -51.99$ ksi

$\psi = |f_2/f_1| = |-51.99/51.99| = 1.0$ (*Eq.* B2.3-1)

$k = 4 + 2(1+\psi)^3 + 2(1+\psi)$ (*Eq.* B2.3-2)

$= 4 + 2(1+1)^3 + 2(1+1) = 24.0$

$F_{cr} = 24.0 \dfrac{\pi^2 (29500)}{12\left(1-0.3^2\right)}\left(\dfrac{1}{144.2}\right)^2 = 30.77$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{51.99}{30.77}} = 1.300 > 0.673 \therefore$ web may be subject to local buckling (*Eq.* B2.1-4)

$\rho = (1 - 0.22/1.300)/1.300 = 0.639$ (*Eq.* B2.1-3)

$b_e = \rho w = (0.639)(8.507) = 5.436$ in. (*Eq.* B2.1-2)

$h_o/b_o = 9.000/2.500 = 3.60 < 4.0$

$\therefore b_1 = b_e/(3+\psi)$ (*Eq.* B2.3-3)

$= 5.436/(3+1.0) = 1.359$ in.

For $\psi > 0.236$

$b_2 = b_e/2 = 5.436/2 = 2.718$ in. (*Eq.* B2.3-4)

$b_1 + b_2 \le w/2$

$1.359 + 2.718 = 4.077 < 8.507/2 = 4.254 \quad \therefore$ web is not fully effective for this iteration

d. Recompute properties by parts

Represent the ineffective portion of the web as an element with a negative length

$b_{neg} = -(4.254 - 4.077) = -0.177$ in.

Its centroidal location below the top fiber:

$$y = t/2 + r + b_1 - b_{neg}/2$$

$$= (0.059/2) + 0.217 + 1.359 - (-0.177/2) = 1.694 \text{ in.}$$

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
Top Flange	1.618	0.030	0.049	0.001	---
Bottom Flange	2.007	8.971	18.005	161.521	---
Web	8.507	4.500	38.282	172.267	51.304
Negative Web Element	-0.177	1.694	-0.300	-0.508	0.000
Top Inside Corner	0.341	0.108	0.037	0.004	0.002
Bottom Inside Corner	0.341	8.892	3.032	26.962	0.002
Top Outside Corner	0.341	0.108	0.037	0.004	0.002
Bottom Outside Corner	0.341	8.892	3.032	26.962	0.002
Top Lip	0.229	0.360	0.082	0.030	0.001
Bottom Lip	0.527	8.490	4.474	37.986	0.012
Sum $\sum$	14.075		66.730	425.229	51.325

$$\bar{y} = \sum Ly / \sum L$$

$$= 66.730/14.075 = 4.741 \text{ in. below top fiber}$$

$$I_{xe} = \left[\sum I'_x + \sum Ly^2 - \bar{y}^2 \sum L\right] t$$

$$= \left[51.325 + 425.229 - (4.741)^2 (14.075)\right](0.059) = 9.451 \text{ in.}^4$$

2. Second Iteration with New Neutral Axis Location

The calculated neutral axis location (4.741 in.) does not equal the assumed neutral axis location (4.500 in.); therefore, another iteration is required.

a. Compression flange

Since the neutral axis is below the centerline, the maximum flexural stress, F_y, will occur at the top flange. The previous solution using F_y will still be valid.

b. Stiffener lip

The change in neutral axis location will change the maximum stress and stress gradient.

$$f = f_1 = (55)(4.741 - 0.059/2 - 0.217)/4.741 = 52.14 \text{ ksi}$$

$$f_2 = (55)(4.741 - 0.773)/4.741 = 46.03 \text{ ksi}$$

$$\psi = |f_2/f_1| = |46.03/52.14| = 0.883 \quad (Eq.\ B3.2\text{-}1)$$

$$k = \frac{0.578}{0.883 + 0.34} = 0.473 \quad (Eq.\ B3.2\text{-}2)$$

$$F_{cr} = 0.473 \frac{\pi^2 (29500)}{12(1 - 0.3^2)} \left(\frac{1}{8.93}\right)^2 = 158.1 \text{ ksi} \quad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{52.14}{158.1}} = 0.574 < 0.673 \therefore \text{ lip is not subject to local buckling} \quad (Eq.\ B2.1\text{-}4)$$

From above: $d'_s = 0.527$ in. $d_s = 0.229$ in.

c. Web

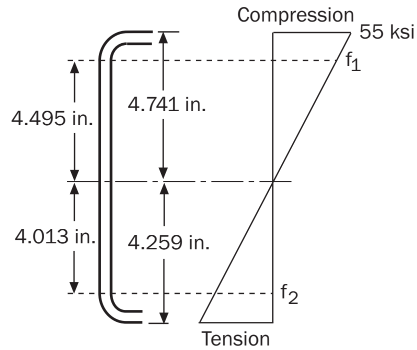
Compute new stresses at edges of web, correcting for the shift in the neutral axis.

$f_1 = 55(4.741 - 0.059/2 - 0.217)/4.741$

$= 52.14$ ksi

$f_2 = -55(9.000 - 4.741 - 0.059/2 - 0.217)/4.741$

$= -46.55$ ksi



$\psi = |f_2/f_1| = |-46.55/52.14| = 0.893$ (*Eq.* B2.3-1)

$k = 4 + 2(1+0.893)^3 + 2(1+0.893) = 21.35$ (*Eq.* B2.3-2)

$F_{cr} = 21.35\dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{1}{144.2}\right)^2 = 27.38$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{52.14}{27.38}} = 1.380 > 0.673 \therefore$ web may be subject to local buckling (*Eq.* B2.1-4)

$\rho = (1 - 0.22/1.380)/1.380 = 0.609$ (*Eq.* B2.1-3)

$b_e = \rho w = (0.609)(8.507) = 5.181$ in. (*Eq.* B2.1-2)

$b_1 = b_e/(3+\psi)$ (*Eq.* B2.3-3)

$= 5.181/(3+0.893) = 1.331$ in.

For $\psi > 0.236$

$b_2 = b_e/2 = 5.181/2 = 2.591$ in. (*Eq.* B2.3-4)

$b_1 + b_2 = 1.331 + 2.591 = 3.922$ in.

Depth of compression block = 4.495 in. > 3.922 in. ; therefore, the web is not fully effective.

d. Recompute properties by parts

Represent the ineffective portion of the web as an element with a negative area.

$b_{neg} = -(4.495 - 3.922) = -0.573$ in.

Its centroidal location below the top fiber

$y = t/2 + r + b_1 - b_{neg}/2$

$= (0.059/2) + 0.217 + 1.331 - (-0.573/2) = 1.864$ in.

Element	L (in.)	y from top fiber(in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
Top Flange	1.618	0.030	0.049	0.001	---
Bottom Flange	2.007	8.971	18.005	161.521	---
Web	8.507	4.500	38.282	172.267	51.304
Negative Web Element	-0.573	1.864	-1.068	-1.991	-0.016
Top Inside Corner	0.341	0.108	0.037	0.004	0.002
Bottom Inside Corner	0.341	8.892	3.032	26.962	0.002
Top Outside Corner	0.341	0.108	0.037	0.004	0.002
Bottom Outside Corner	0.341	8.892	3.032	26.962	0.002
Top Lip	0.229	0.360	0.082	0.030	0.001
Bottom Lip	0.527	8.490	4.474	37.986	0.012
Sum $\sum$	13.679		65.962	423.746	51.309

$\bar{y} = \sum Ly / \sum L$

$= 65.962/13.679 = 4.822$ in. below top fiber

$I_{xe} = \left[\sum I'_x + \sum Ly^2 - \bar{y}^2 \sum L\right]$

$= \left[51.309 + 423.746 - (4.822)^2 (13.679)\right](0.059) = 9.263 \text{ in.}^4$

$S_e = I_{xe}/y$

$= 9.263/4.822 = 1.92 \text{ in.}^3$

3. Further Iterations

The calculated neutral axis location (4.822 in.) does not exactly match the assumed neutral axis location (4.741 in.) but the calculated I_{xe} and S_e are within two percent of the fully converged solution. After further iterations (not shown) the solution converges to:

$\bar{y} = 4.859$ in.

$I_{xe} = 9.18 \text{ in.}^4$

$S_e = 1.89 \text{ in.}^3$

4. Effective Area, A_e, at a Uniform Compressive Stress of 37.25 ksi

a. Compression flange: taking parameters from 1 (a) above

$f = 37.25$ ksi

$S = 1.28\sqrt{29500/37.25} = 36.02$ (*Eq*. B4-7)

$w/t = 34.02 > 0.328S$ => check effective width of flange

$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right]$ (*Eq*. B4-8)

$= 399(0.059)^4\left[\frac{34.02}{36.02} - 0.328\right]^3 \le (0.059)^4\left[115\frac{34.02}{36.02} + 5\right]$

$= 0.00113 \text{ in.}^4 < 0.00138 \text{ in.}^4 \quad \therefore I_a = 0.00113 \text{ in.}^4$

$R_I = I_s/I_a = 0.000720/0.00113 = 0.637 < 1.0$ OK (*Eq*. B4-9)

$n = \left[0.582 - \frac{w/t}{4S}\right] \ge \frac{1}{3}$ (*Eq*. B4-11)

$= \left[0.582 - \frac{34.02}{(4)(36.02)}\right] \ge \frac{1}{3} = 0.346 > 1/3$ OK

$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \le 4$ (From Table B4-1)

$= \left(4.82 - \frac{(5)(0.773)}{2.007}\right)(0.637)^{0.346} + 0.43 = 2.906 < 4$ OK

$F_{cr} = 2.906\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{34.02}\right)^2 = 66.95$ ksi (*Eq*. B2.1-5)

$\lambda = \sqrt{\frac{37.25}{66.95}} = 0.746 > 0.673 \therefore$ flange is subject to local buckling (*Eq*. B2.1-4)

$\rho = (1 - 0.22/0.746)/0.746 = 0.945$ (*Eq*. B2.1-3)

$b = \rho w = (0.945)(2.007) = 1.897$ in. (*Eq*. B2.1-2)

b. Stiffener lip: taking parameters from 1 (b) above

$f = 37.25$ ksi

$k = 0.43$

$F_{cr} = 0.43\dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{1}{8.93}\right)^2 = 143.8$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{37.25}{143.8}} = 0.509 < 0.673 \therefore$ lip is not subject to local buckling (*Eq.* B2.1-4)

$d'_s = w = 0.527$ in.

$d_s = d'_s(R_I) = (0.527)(0.637) = 0.336$ in. (*Eq.* B4-6)

c. Web: from Section B2.1

$f = 37.25$ ksi

$k = 4.0$

$F_{cr} = 4.0\dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{1}{144.2}\right)^2 = 5.13$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{37.25}{5.13}} = 2.695 > 0.673 \therefore$ web is subject to local buckling (*Eq.* B2.1-4)

$\rho = (1-0.22/2.695)/2.695 = 0.341$ (*Eq.* B2.1-3)

$b = \rho w = (0.341)(8.507) = 2.901$ in. (*Eq.* B2.1-2)

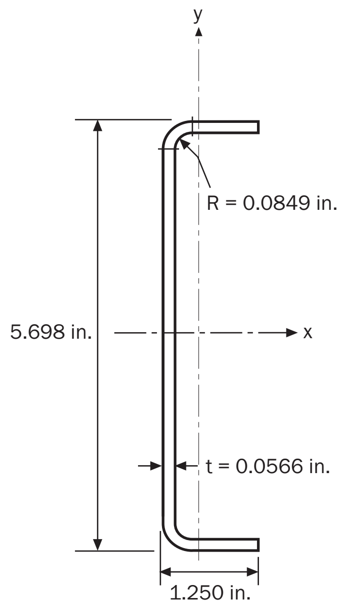
Sum of the effective widths of the elements

Element	L (in.)
Top Flange	1.897
Bottom Flange	1.897
Web	2.901
Top Inside Corner	0.341
Bottom Inside Corner	0.341
Top Outside Corner	0.341
Bottom Outside Corner	0.341
Top Lip	0.336
Bottom Lip	0.336
Sum $\sum$	8.731

$A_e = t\sum L$

$= (0.059)(8.731) = 0.515$ in.^{2}

Example I-9: C-Section Without Lips - Effective Section Properties



Given:

1. Steel: F_y = 33 ksi
2. Section: Track 550T125-54 as shown above

Required:

1. Effective section modulus, S_e, at a maximum bending stress, f, of 30.93 ksi (as used in Example II-4)

Solution:

See Example I-2 for basic geometric parameters.

1. Effective Section Modulus, S_e, at f = 30.93 ksi

An iterative approach is generally required.

For the first iteration, assume a compression stress of f = 30.93 ksi in the top fiber of the section and a neutral axis location at the mid-height of the web, 2.849 in. below the top fiber.

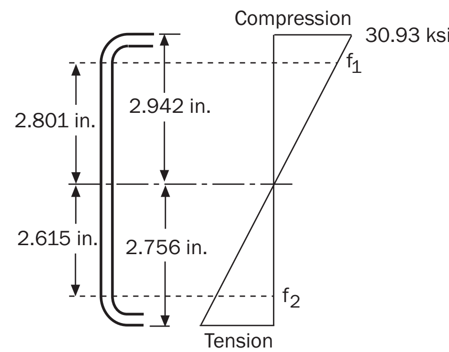
a. Compression flange is a uniformly compressed unstiffened element (Section B3.1)

$$w = B - r - t/2$$
$$= 1.250 - 0.113 - 0.0566/2$$
$$= 1.109 \text{ in.}$$
$$w/t = 1.109/0.0566 = 19.59 < 60 \quad \text{OK} \qquad \text{(Section B1.1(a)(3))}$$
$$k = 0.43$$
$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \qquad (\textit{Eq. } \text{B2.1-5})$$
$$= 0.43\frac{\pi^2 (29500)}{12\left(1-0.3^2\right)}\left(\frac{1}{19.59}\right)^2 = 29.87 \text{ ksi}$$
$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (\textit{Eq. } \text{B2.1-4})$$
$$= \sqrt{\frac{30.93}{29.87}} = 1.018 > 0.673 \therefore \text{ flange is subject to local buckling}$$

$$\rho = (1-0.22/\lambda)/\lambda \quad (Eq.\ B2.1\text{-}3)$$
$$= (1-0.22/1.018)/1.018 = 0.770$$
$$b = \rho w \quad (Eq.\ B2.1\text{-}2)$$
$$= (0.770)(1.109)$$
$$= 0.854 \text{ in.}$$

b. Compute new neutral axis location and check web as a stiffened element under a stress gradient.

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
Top Flange	0.854	0.0283	0.024	0.001	---
Top Radius	0.177	0.0693	0.012	0.001	0.000
Web	5.415	2.8490	15.427	43.952	13.232
Bottom Radius	0.177	5.6287	0.996	5.608	0.000
Bottom Flange	1.109	5.6697	6.288	35.649	----
Sum $\sum$	7.732		22.747	85.211	13.232



$$\bar{y} = \sum Ly/\sum L$$
$$= 22.747/7.732 = 2.942 \text{ in. below top fiber}$$
$$I_{xe} = \left[\sum I'_x + \sum Ly^2 - \bar{y}^2 \sum L\right]t$$
$$= \left[13.232 + 85.211 - (2.942)^2(7.732)\right](0.0566)$$
$$= 1.784 \text{ in.}^4$$
$$S_e = I_{xe}/\bar{y} = 1.784/2.942 = 0.606 \text{ in.}^3$$

Check Web:

By similar triangles

$$f_1 = \left(\frac{2.942 - 0.113 - 0.0566/2}{2.942}\right)(30.93) = 29.44 \text{ ksi}$$
$$f_2 = -\left(\frac{5.698 - 2.942 - 0.113 - 0.0566/2}{2.942}\right)(30.93) = -27.49 \text{ ksi}$$
$$\psi = |f_2/f_1| = |-27.49/29.44| = 0.934 \quad (Eq.\ B2.3\text{-}1)$$
$$k = 4 + 2(1+\psi)^3 + 2(1+\psi) \quad (Eq.\ B2.3\text{-}2)$$
$$= 4 + 2(1+0.934)^3 + 2(1+0.934) = 22.34$$
$$w = 5.698 - 2(0.113) - 0.0566 = 5.415 \text{ in.}$$
$$w/t = 5.415/0.0566 = 95.67$$
$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \quad (Eq.\ B2.1\text{-}5)$$
$$= 22.34\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{95.67}\right)^2 = 65.08 \text{ ksi}$$
$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B2.1\text{-}4)$$
$$= \sqrt{\frac{29.44}{65.08}} = 0.673 \therefore b_e = w = 5.415 \text{ in.}$$

$h_o/b_o = 5.698/1.25 = 4.6 > 4.0$

$\therefore\ b_1 = b_e/(3+\psi)$ (*Eq*. B2.3-6)

$= 5.415/(3+0.934) = 1.376$ in.

$b_2 = b_e/(1+\psi) - b_1$ (*Eq*. B2.3-7)

$= 5.415/(1+0.934) - 1.376 = 1.424$ in.

$b_1 + b_2 = 1.376 + 1.424 = 2.800$ in.

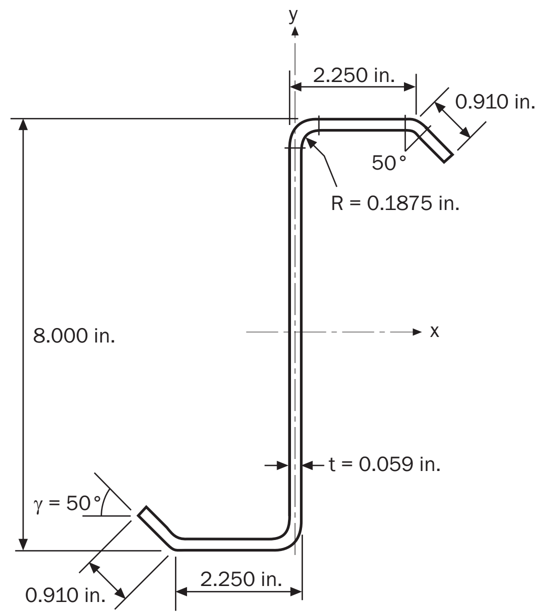
Width of compression block

$2.942 - 0.0566/2 - 0.113 = 2.801$ in. $\cong (b_1 + b_2) = 2.800$ in.

Therefore the web is fully effective and no further iteration is required.

$S_e = 0.606$ in.^{3}

Example I-10: Z-Section With Lips - Effective Section Properties



Given:

1. Steel: F_y = 55 ksi
2. Section: 8ZS2.25x059 as shown above

Required:

1. Effective section modulus, S_e, based on initiation of yielding, as required in Example II-2
2. Effective moment of inertia based on Procedure I of Section C3.1.1 for deflection determination at a service moment equal to 60% of the fully braced nominal moment, M_n
3. Effective area, A_e, at a uniform compressive stress of 25.9 ksi, as required in Example III-6

Solution:

See Example I-3 for basic geometric parameters.

1. Effective Section Modulus, S_e, at Initiation of Yielding

An iterative approach is generally required since the location of the neutral axis is dependant on the effective section properties.

For the first iteration, assume a compression stress of f = F_y = 55 ksi in the top fiber of the section and that the neutral axis is 4.000 in. below the top fiber.

a. Compression flange: from Section B4

$w = b = 1.889$ in.

$w/t = 1.889/0.059 = 32.0 < 60$ OK (Section B1.1(a)(1))

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-7)

$= 1.28\sqrt{29500/55} = 29.64 \therefore w/t \geq 0.328\,S \Rightarrow$ check effective width of flange

Compute k of flange based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \leq t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4\text{-}8)$$

$$= 399(0.059)^4\left[\frac{32.0}{29.64} - 0.328\right]^3 \leq (0.059)^4\left[115\frac{32.0}{29.64} + 5\right]$$

$= 0.00205$ in.$^4 > 0.00156$ in.^{4} $\therefore I_a = 0.00156$ in.^{4}

$d = c = 0.795$ in.

$I_s = \left(d^3 t \sin^2\theta\right)/12$ (*Eq.* B4-10)

$= \left((0.795)^3(0.059)\sin^2(50°)\right)/12 = 0.00145 \text{ in.}^4$

$R_I = I_s/I_a \le 1$ (*Eq.* B4-9)

$= 0.00145/0.00156 = 0.929$

$n = \left[0.582 - \dfrac{w/t}{4S}\right] \ge \dfrac{1}{3}$ (*Eq.* B4-11)

$= \left[0.582 - \dfrac{32.0}{(4)(29.64)}\right] \ge \dfrac{1}{3}$

$= 0.312 < 1/3 \quad \therefore n = 1/3$

$D = 0.910$ in.

$D/w = 0.910/1.889 = 0.48 < 0.8$ OK

$k = \left(4.82 - \dfrac{5D}{w}\right)(R_I)^n + 0.43 \le 4$ (From Table B4-1)

$= \left(4.82 - \dfrac{(5)(0.910)}{1.889}\right)(0.929)^{1/3} + 0.43 = 2.78 < 4$ OK

$F_{cr} = k\dfrac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\dfrac{t}{w}\right)^2$ (*Eq.* B2.1-5)

$= 2.78\dfrac{\pi^2(29500)}{12\left(1-0.3^2\right)}\left(\dfrac{1}{32.0}\right)^2 = 72.38$ ksi

$\lambda = \sqrt{\dfrac{f}{F_{cr}}}$ (*Eq.* B2.1-4)

$= \sqrt{\dfrac{55}{72.38}} = 0.872 > 0.673$ $\therefore$ flange is subject to local buckling

$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)

$= (1 - 0.22/0.872)/0.872 = 0.857$

$b = \rho w$ (*Eq.* B2.1-2)

$= (0.857)(1.889) = 1.619$ in.

b. Stiffener lip: from Section B3.2(a)

$w/t = d/t = 0.795/0.059 = 13.5$

Maximum stress in lip, f_1 (by similar triangles)

$f = f_1 = 55\left[4.000 - 0.059/2 - 0.217\left(1-\cos(50°)\right)\right]/4.000 = 53.5$ ksi

$f_2 = 55\left[4.000 - 0.059/2 - 0.217\left(1-\cos(50°)\right) - (0.795)\sin(50°)\right]/4.000 = 45.2$ ksi

$\psi = \left|f_2/f_1\right| = \left|45.2/53.5\right| = 0.845$ (*Eq.* B3.2-1)

$k = \dfrac{0.578}{\psi + 0.34}$ (*Eq.* B3.2-2)

$= \dfrac{0.578}{0.845 + 0.34} = 0.488$

$F_{cr} = k\dfrac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\dfrac{t}{w}\right)^2$ (*Eq.* B2.1-5)

$= 0.488\dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{1}{13.5}\right)^2 = 71.4$ ksi

$\lambda = \sqrt{\dfrac{f}{F_{cr}}}$ (*Eq.* B2.1-4)

$= \sqrt{\dfrac{53.5}{71.4}} = 0.866 > 0.673$ $\therefore$ lip is subject to local buckling

$\rho = (1-0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)

$= (1-0.22/0.866)/0.866 = 0.861$

$d'_s = b = \rho w$ (*Eq.* B2.1-2)

$= (0.861)(0.795) = 0.684$ in.

$d_s = d'_s(R_I)$ (*Eq.* B4-6)

$= (0.684)(0.929) = 0.635$ in.

c. Web: from Section B2.3

$w/t = 7.507 / 0.059 = 127.2$

$\psi = |f_2/f_1|$ (*Eq.* B2.3-1)

Assuming the neutral axis is at the mid-height of the section, find the maximum flexural stress in the web by similar triangles.

$f_1 = (55)(4.000-0.059-0.1875)/4.000 = 51.61$ ksi

By symmetry

$f_2 = -f_1 = -51.61$ ksi

$\psi = |f_2/f_1| = |-51.61/51.61| = 1.0$ (*Eq.* B2.3-1)

$k = 4+2(1+\psi)^3+2(1+\psi)$ (*Eq.* B2.3-2)

$= 4+2(1+1)^3+2(1+1) = 24.0$

$F_{cr} = 24.0\dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{1}{127.2}\right)^2 = 39.55$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{51.61}{39.55}} = 1.142 > 0.673$ $\therefore$ web may be subject to local buckling (*Eq.* B2.1-4)

$\rho = (1-0.22/1.142)/1.142 = 0.707$ (*Eq.* B2.1-3)

$b_e = \rho w = (0.707)(7.507) = 5.307$ in. (*Eq.* B2.1-2)

$h_o/b_o = 8.000/2.250 = 3.6 < 4.0$

$\therefore b_1 = b_e/(3+\psi)$ (*Eq.* B2.3-3)

$= 5.307/(3+1) = 1.327$ in.

For $\psi > 0.236$

$b_2 = b_e/2 = 5.307/2 = 2.654$ in. (*Eq.* B2.3-4)

$b_1 + b_2 \le w/2$

$1.327 + 2.654 = 3.981$ in. $> 7.507/2 = 3.754$ in. $\therefore$ web is fully effective for this iteration

d. Recompute properties by parts

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
Top Flange	1.619	0.030	0.049	0.001	---
Bottom Flange	1.889	7.971	15.057	120.021	---
Web	7.507	4.000	30.028	120.112	35.255
Top Inside Corner	0.341	0.108	0.037	0.004	0.002
Bottom Inside Corner	0.341	7.892	2.691	21.239	0.002
Top Outside Corner	0.189	0.056	0.011	0.001	0.000
Bottom Outside Corner	0.189	7.944	1.501	11.927	0.000
Top Lip	0.635	0.350	0.222	0.078	0.013
Bottom Lip	0.795	7.589	6.033	45.786	0.025
Sum $\sum$	13.505		55.629	319.169	35.297

$$\bar{y} = \sum Ly / \sum L$$

$$= 55.629/13.505 = 4.119 \text{ in. below top fiber}$$

$$I_{xe} = \left[\sum I'_x + \sum Ly^2 - \bar{y}^2 \sum L\right] t$$

$$= \left[35.297 + 319.169 - (4.119)^2 (13.505)\right](0.059) = 7.40 \text{ in.}^4$$

Second iteration with new neutral axis location

The calculated neutral axis location (4.119 in.) does not equal the assumed neutral axis location (4.000 in.); therefore, another iteration is required.

a. Compression flange

Since the neutral axis is below the centerline, the maximum flexural stress, F_y, will occur at the top flange. The previous solution using F_y will still be valid.

b. Stiffener lip

The change in neutral axis location will change the stress gradient and consequently the maximum stress in the stiffener slightly. This may cause a minor change in the effective width of the stiffener. Neglect in this case.

c. Web

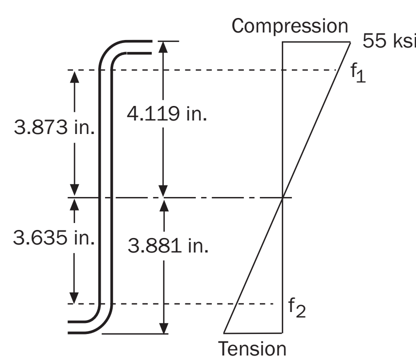
Compute new stresses at edges of web, correcting for the shift in the neutral axis.

$$f_1 = 55(4.119 - 0.059 - 0.1875)/4.119$$

$$= 51.71 \text{ ksi}$$

$$f_2 = -55(8.000 - 4.119 - 0.059 - 0.1875)/4.119$$

$$= -48.53 \text{ ksi}$$



$$\psi = |f_2/f_1| = |-48.53/51.71| = 0.939 \quad (Eq.\ B2.3\text{-}1)$$

$$k = 4 + 2(1 + 0.939)^3 + 2(1 + 0.939) = 22.46 \quad (Eq.\ B2.3\text{-}2)$$

$$F_{cr} = 22.46 \frac{\pi^2 (29500)}{12(1 - 0.3^2)} \left(\frac{1}{127.2}\right)^2 = 37.01 \text{ ksi} \quad (Eq.\ B2.1\text{-}5)$$

$\lambda = \sqrt{\dfrac{51.71}{37.01}} = 1.182 > 0.673 \therefore$ web may be subject to local buckling (*Eq.* B2.1-4)

$\rho = (1-0.22/1.182)/1.182 = 0.689$ (*Eq.* B2.1-3)

$b_e = \rho w = (0.689)(7.507) = 5.172$ in. (*Eq.* B2.1-2)

$b_1 = b_e/(3+\psi)$ (*Eq.* B2.3-3)

$= 5.172/(3+0.939) = 1.313$ in.

For $\psi > 0.236$

$b_2 = b_e/2 = 5.172/2 = 2.586$ in. (*Eq.* B2.3-4)

$b_1 + b_2 = 1.313 + 2.586 = 3.899$ in.

Width of compression block

$4.119 - 0.059 - 0.1875 = 3.873 \text{ in.} < 3.899 \text{ in.} \therefore$ web is not subject to local buckling

d. Recompute properties

There was no further reduction in the effective widths of the elements, therefore use previous solution:

$\bar{y} = 4.119$ in. below top fiber

$I_{xe} = 7.394 \text{ in.}^4$

$S_e = I_{xe}/\bar{y}$

$S_e = 7.394/4.119 = 1.80 \text{ in.}^3$

$M_n = S_e F_y$ (*Eq.* C3.1.1-1)

$= (1.80)(55) = 99.0$ kip-in.

2. Effective Moment of Inertia, I_{xe}, at a Service Load of 60% of M_n; M = (0.60)(99.0) = 59.4 kip-in.

A conservative approximation of flexural deflections can be obtained by performing an elastic beam analysis using the effective moment of inertia of the cross-section calculated with the maximum extreme fiber stress set to the maximum flexural stress occurring under serviceability loading. In the case of continuous beams, the average of the moments of inertia in maximum positive and negative bending can be used.

Assume the maximum compressive stress is approximately $(0.60)F_y = (0.60)(55) = 33$ ksi. The calculations are otherwise the same as above.

a. Compression flange: from Section B4

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-7)

$= 1.28\sqrt{29500/33} = 38.3$

$w/t = 32.0 > 0.328\,S \Rightarrow$ check effective width of flange

Compute flange k based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4\text{-}8)$$

$$= 399(0.059)^4\left[\frac{32.0}{38.3} - 0.328\right]^3 \le 0.059^4\left[115\frac{32.0}{38.3} + 5\right]$$

$= 0.000632 \text{ in.}^4 < 0.00122 \text{ in.}^4 \therefore I_a = 0.000632 \text{ in.}^4$

$R_I = I_s/I_a \le 1$ (*Eq.* B4-9)

$= 0.00145/0.000632 = 2.29 \therefore R_I = 1.0$

$$n = \left[0.582 - \frac{w/t}{4S}\right] \geq \frac{1}{3} \quad (Eq.\ B4\text{-}11)$$

$$= \left[0.582 - \frac{32.0}{(4)(38.3)}\right] \geq \frac{1}{3}$$

$$= 0.373 > 1/3 \quad \therefore\ n = 0.373$$

$$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \leq 4 \quad \text{(From Table B4-1)}$$

$$= \left(4.82 - \frac{(5)(0.910)}{1.889}\right)(1.0)^{0.373} + 0.43 = 2.84 < 4\ \text{OK}$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \quad (Eq.\ B2.1\text{-}5)$$

$$= 2.84\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{32.0}\right)^2 = 73.95\ \text{ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{33}{73.95}} = 0.668 < 0.673\ \therefore\ \text{flange is not subject to local buckling}$$

b. Stiffener lip: from Section B3.2(a)

$w/t = d/t = 0.795/0.059 = 13.5$

Maximum stress in lip (by similar triangles)

$$f = f_1 = 33\left[4.000 - 0.059/2 - 0.217\left(1-\cos(50°)\right)\right]/4.000 = 32.12\ \text{ksi}$$

$$f_2 = 33\left[4.000 - 0.059/2 - 0.217\left(1-\cos(50°)\right) - 0.795\sin(50°)\right]/4.000 = 27.09\ \text{ksi}$$

$$\psi = |f_2/f_1| \quad (Eq.\ B3.2\text{-}1)$$

$$= |27.09/32.12| = 0.843$$

$$k = \frac{0.578}{\psi + 0.34} \quad (Eq.\ B3.2\text{-}2)$$

$$= \frac{0.578}{0.843 + 0.34} = 0.489$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \quad (Eq.\ B2.1\text{-}5)$$

$$= 0.489\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{13.5}\right)^2 = 71.54\ \text{ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{32.12}{71.54}} = 0.670 < 0.673\ \therefore\ \text{lip is not subject to local buckling}$$

$$d_s = d'_s(R_I) \quad (Eq.\ B4\text{-}6)$$

$$= (0.795)(1.0) = 0.795\ \text{in.}$$

c. Web: from Section B2.3

By inspection, the web is fully effective at a maximum flange stress of 33 ksi, since it was shown in Parts 1 and 2 above to be fully effective at a maximum flange stress of 55 ksi.

d. Since all elements are fully effective at the assumed stress level, use the gross moment of inertia from Table I-4.

$I_x = 7.76 \text{ in.}^4$

3. Effective Area, A_e, at a Uniform Compressive Stress of 25.9 ksi

From Section 2 above, it can be concluded that the flange will be fully effective at a stress of 25.9 ksi, since it is fully effective at higher stress levels.

a. Check lips

$k = 0.43$

$$F_{cr} = 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{13.5}\right)^2 = 62.91 \text{ ksi} \qquad (Eq.\ \text{B2.1-5})$$

$$\lambda = \sqrt{\frac{25.9}{62.91}} = 0.642 < 0.673 \therefore \text{ lips are not subject to local buckling} \qquad (Eq.\ \text{B2.1-4})$$

b. Check web

$k = 4.0$

$$F_{cr} = 4.0\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{127.2}\right)^2 = 6.59 \text{ ksi} \qquad (Eq.\ \text{B2.1-5})$$

$$\lambda = \sqrt{\frac{25.9}{6.59}} = 1.982 > 0.673 \therefore \text{ web is subject to local buckling} \qquad (Eq.\ \text{B2.1-4})$$

$$\rho = (1-0.22/1.982)/1.982 = 0.449 \qquad (Eq.\ \text{B2.1-3})$$

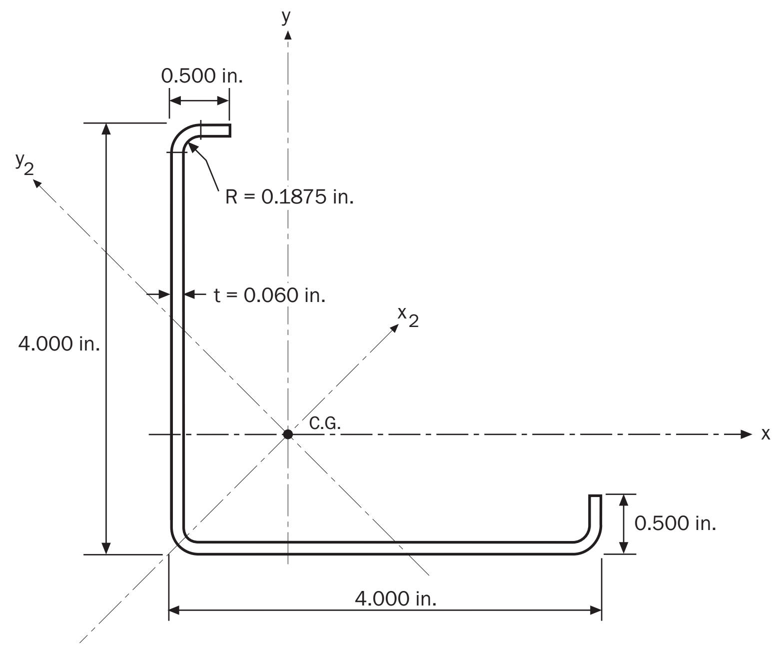
$$b_e = \rho w = (0.449)(7.507) = 3.371 \text{ in.} \qquad (Eq.\ \text{B2.1-2})$$

To find A_e, subtract the ineffective area of the web from the gross area.

From Table I-4 or Example I-3, $A_{gross} = 0.822 \text{ in.}^2$

$A_e = 0.822 - (7.507 - 3.371)(0.059) = 0.578 \text{ in.}^2$

Example I-11: Equal Leg Angle With Lips - Effective Section Properties



Given:

1. Steel: F_y = 50 ksi
2. Section: 4LS4x060 as shown above

Required:

Effective area, A_e, at a uniform compression stress of 14.7 ksi, as required in Example III-5

Solution:

See Example I-4 for basic parameters. Treat each leg as a uniformly compressed element with an edge stiffener (Section B4).

a. Legs

w = 3.504 in.

t = 0.060 in.

$w/t = 3.504 / 0.060 = 58.4$

$$S = 1.28\sqrt{E/f} \qquad (Eq.\ B4\text{-}7)$$

$$= 1.28\sqrt{29500/14.7}$$

$$= 57.3$$

$w/t > 0.328S$, therefore check effective width of leg

Compute k of flange based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \qquad (Eq.\ B4\text{-}8)$$

$$= 399(0.060)^4\left[\frac{58.4}{57.3} - 0.328\right]^3 \le 0.060^4\left[115\frac{58.4}{57.3} + 5\right]$$

$= 0.00171\ in.^4 > 0.00158\ in.^4\ \therefore\ I_a = 0.00158\ in.^4$

$d = c = 0.252$ in.

$I_s = \left(d^3 t \sin^2\theta\right)/12$ (*Eq.* B4-10)

$= \dfrac{(0.252)^3(0.060)\sin^2(90°)}{12} = 0.0000800 \text{ in.}^4$

$R_I = I_s/I_a \le 1$ (*Eq.* B4-9)

$= 0.0000800/0.00158 = 0.0506$

$n = \left[0.582 - \dfrac{w/t}{4S}\right] \ge \dfrac{1}{3}$ (*Eq.* B4-11)

$= \left[0.582 - \dfrac{58.4}{(4)(57.3)}\right] \ge \dfrac{1}{3}$

$= 0.327 < 1/3 \;\therefore\; n = 1/3$

$D = 0.500$ in.

$D/w = 0.500/3.504 = 0.143 < 0.25$

$k = 3.57(R_I)^n + 0.43 \le 4$ (From Table B4-1)

$= 3.57(0.0506)^{1/3} + 0.43 = 1.75$

$F_{cr} = k\dfrac{\pi^2 E}{12(1-\mu^2)}\left(\dfrac{t}{w}\right)^2$ (*Eq.* B2.1-5)

$= 1.75\dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{1}{58.4}\right)^2 = 13.7$ ksi

$\lambda = \sqrt{\dfrac{f}{F_{cr}}}$ (*Eq.* B2.1-4)

$= \sqrt{\dfrac{14.7}{13.7}} = 1.036 > 0.673 \;\therefore$ leg is subject to local buckling

$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)

$= (1 - 0.22/1.036)/1.036 = 0.760$

$b = \rho w$ (*Eq.* B2.1-2)

$= (0.760)(3.504) = 2.663$ in.

b. Stiffener Lips

Check stiffener effective width

$w = d = 0.252$ in.

$w/t = 0.252/0.060 = 4.20$

$k = 0.43$

$F_{cr} = 0.43\dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{1}{4.20}\right)^2 = 650$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{14.7}{650}} = 0.150 < 0.673 \;\therefore$ stiffener is not subject to local buckling (*Eq.* B2.1-4)

$d'_s = b = w = 0.252$ in. (*Eq.* B2.1-1)

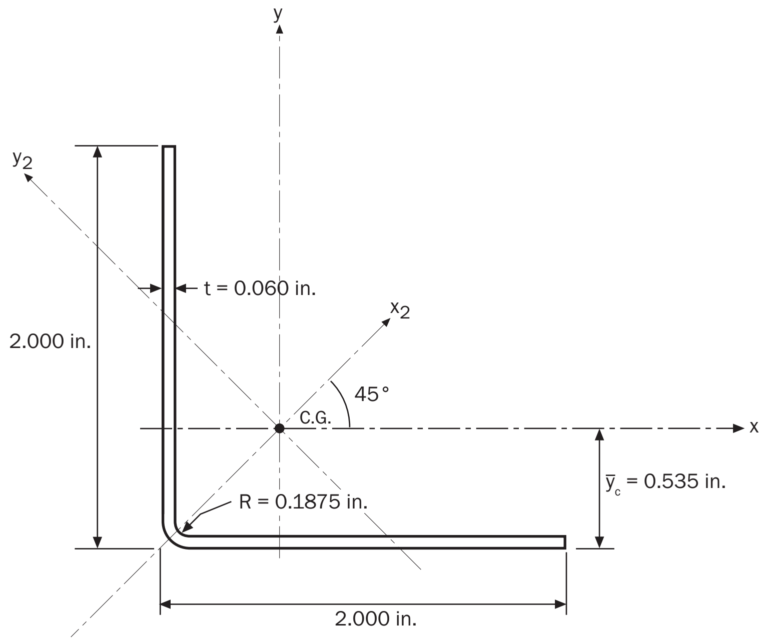
$d_s = d'_s(R_I) = (0.252)(0.0506) = 0.013$ in. (*Eq.* B4-6)

Summing the effective widths,

$A_e = t\left[2(b + d_s) + 3u\right]$

$= 0.060\left[2(2.663 + 0.013) + (3)(0.342)\right] = 0.383 \text{ in.}^2$

Example I-12: Equal Leg Angle Without Lips - Effective Section Properties



Given:

1. Steel: F_y = 33 ksi
2. Section: 2LU2x060 as shown above

Required:

1. Effective section modulus, S_e, at f = F_y at the extreme fibers, for flexure about the x-axis with compression on the top
2. Effective area, A_e, at f = 12.0 ksi

Solution:

Refer to Example I-5 for basic parameters.

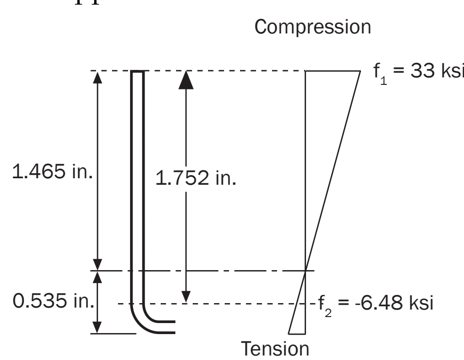
1. Effective Section Modulus, S_e, With Compression on Top (Bottom Flange in Tension)

Treat the upstanding leg as an unstiffened element with a stress gradient (Section B3.2). Stress at tip of web will be at the yield stress in compression. At the opposite end of the web adjacent to the flange, the leg will be in tension.

$$w = 2.000 - 0.060 - 0.1875 = 1.752 \text{ in.}$$

$$f_1 = 33 \text{ ksi}$$

$$f_2 = 33.0\frac{-(0.535 - 0.060 - 0.1875)}{2.000 - 0.535} = -6.48 \text{ ksi}$$



$$\psi = |f_2/f_1| \qquad (Eq.\ B3.2\text{-}1)$$

$$= |-6.48/33| = 0.196$$

$$k = 0.57 + 0.21\psi + 0.07\psi^2 \qquad (Eq.\ B3.2\text{-}5)$$

$$k = 0.57 + 0.21(0.196) + 0.07(0.196)^2 = 0.614$$

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 0.614\frac{\pi^2\left(29500\right)}{12\left(1-0.3^2\right)}\left(\frac{0.060}{1.752}\right)^2 = 19.2\text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{33}{19.2}} = 1.311 > 0.673\left(1+\psi\right) = 0.805 \therefore \text{ leg is subject to local buckling}$$

$$\rho = \left(1+\psi\right)\frac{\left(1-\frac{0.22\left(1+\psi\right)}{\lambda}\right)}{\lambda} \qquad (Eq.\ B3.2\text{-}4)$$

$$= \left(1+0.196\right)\frac{\left(1-\frac{0.22\left(1+0.196\right)}{1.311}\right)}{1.311} = 0.729$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$$= \left(0.729\right)\left(1.752\right) = 1.277\text{ in.}$$

Effective section properties

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
Web	1.277	1.114	1.423	1.585	0.174
Corner	0.342	1.891	0.647	1.223	0.002
Flange	1.752	1.970	3.451	6.799	----
Sum $\sum$	3.371		5.521	9.607	0.176

$$\bar{y} = \sum Ly/\sum L = 5.521/3.371 = 1.638\text{ in. below top fiber}$$

The neutral axis has shifted, which will result in a change in the stress gradient in the upstanding leg of the angle. Recompute the effective properties with the new neutral axis position.

$$f_1 = 33\text{ ksi}$$

$$f_2 = 33.0\frac{-\left(2.00-1.638-0.060-0.1875\right)}{1.638} = -2.31\text{ ksi}$$

$$\psi = \left|-2.31/33\right| = 0.0700 \qquad (Eq.\ B3.2\text{-}1)$$

$$k = 0.57+0.21\left(0.0700\right)+0.07\left(0.0700\right)^2 = 0.585 \qquad (Eq.\ B3.2\text{-}5)$$

$$F_{cr} = 0.585\frac{\pi^2\left(29500\right)}{12\left(1-0.3^2\right)}\left(\frac{0.060}{1.752}\right)^2 = 18.3\text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{33}{18.3}} = 1.343 \qquad (Eq.\ B2.1\text{-}4)$$

$1.343 > 0.673\left(1+\psi\right) = 0.720 \therefore$ leg is subject to local buckling

$$\rho = (1+0.0700)\frac{\left(1-\frac{0.22(1+0.0700)}{1.343}\right)}{1.343} = 0.657 \qquad (Eq.\ B3.2\text{-}4)$$

$$b = \rho w = (0.657)(1.752) = 1.151\text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

Effective section properties

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I′$_x$ about own axis (in.^{3})
Web	1.151	1.177	1.355	1.595	0.127
Corner	0.342	1.891	0.647	1.223	0.002
Flange	1.752	1.970	3.451	6.799	----
Sum $\sum$	3.245		5.453	9.617	0.129

$$\bar{y} = \sum Ly/\sum L = 5.453/3.245 = 1.680\text{ in. below top fiber}$$

The neutral axis has shifted again, which will result in a change in the stress gradient in the upstanding leg of the angle. Recompute the effective properties with the new neutral axis position.

$$f_1 = 33\text{ ksi}$$

$$f_2 = 33.0\frac{-(2.00-1.680-0.060-0.1875)}{1.680} = -1.42\text{ ksi}$$

$$\psi = |-1.42/33| = 0.0430 \qquad (Eq.\ B3.2\text{-}1)$$

$$k = 0.57+0.21(0.0430)+0.07(0.0430)^2 = 0.579 \qquad (Eq.\ B3.2\text{-}5)$$

$$F_{cr} = 0.579\frac{\pi^2(29500)}{12\left(1-0.3^2\right)}\left(\frac{0.060}{1.752}\right)^2 = 18.1\text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{33}{18.1}} = 1.350 \qquad (Eq.\ B2.1\text{-}4)$$

$1.350 > 0.673(1+\psi)$ ∴ leg is subject to local buckling

$$\rho = (1+0.0430)\frac{\left(1-\frac{0.22(1+0.0430)}{1.350}\right)}{1.350} = 0.641 \qquad (Eq.\ B3.2\text{-}4)$$

$$b = \rho w = (0.641)(1.752) = 1.123\text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

Effective section properties

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I′$_x$ about own axis (in.^{3})
Web	1.123	1.191	1.337	1.593	0.118
Corner	0.342	1.891	0.647	1.223	0.002
Flange	1.752	1.970	3.451	6.799	----
Sum $\sum$	3.217		5.435	9.615	0.120

$\bar{y} = \sum Ly/\sum L = 5.435/3.217 = 1.689$ in. below top fiber

The neutral axis has shifted again, but the change is less than 0.01 in. Compute the effective properties with the new neutral axis position.

$$I_{xe} = \left[\sum I'_x + \sum Ly^2 - \bar{y}^2 \sum L\right]t$$

$$= \left[0.120 + 9.615 - (1.689)^2 (3.217)\right]0.060 = 0.0335 \text{ in.}^4$$

$$S_{et} = \frac{I_{xe}}{y_t} = \frac{0.0335}{1.689} = 0.0198 \text{ in.}^3$$

$$S_{eb} = \frac{I_{xe}}{y_b} = \frac{0.0335}{2.000 - 1.689} = 0.108 \text{ in.}^3$$

2. Effective Area, A_e, at f = 12.0 ksi

Treat flanges as uniformly compressed unstiffened element (Section B3.1)

$f = 12.0$ ksi

$k = 0.43$

$w = 1.752$ in.

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 0.43\frac{\pi^2 (29500)}{12(1-0.3^2)}\left(\frac{0.060}{1.752}\right)^2 = 13.45 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{12.0}{13.45}} = 0.945 > 0.673 \therefore \text{ leg is subject to local buckling}$$

$$\rho = (1 - 0.22/\lambda)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$

$$= (1 - 0.22/0.945)/0.945 = 0.812$$

$$b = \rho w = (0.812)(1.752) = 1.423 \text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

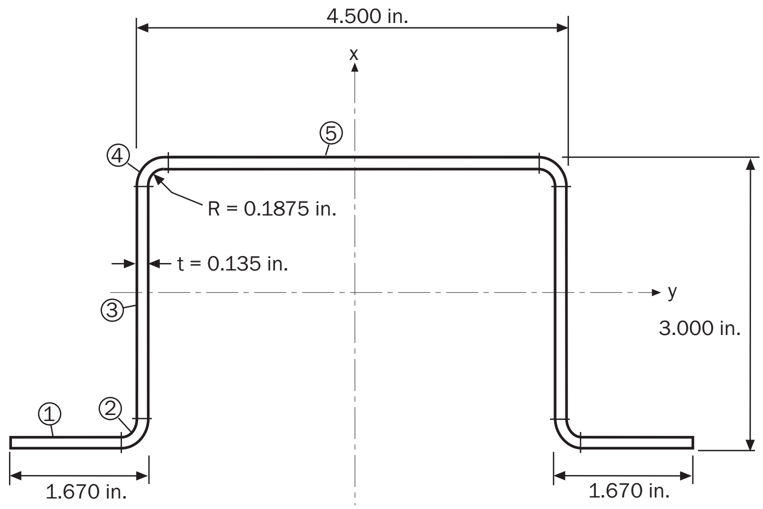
Effective area

$$A_e = t\sum L$$

$$= (0.060)\left[(2)(1.423) + 0.342\right]$$

$$= 0.191 \text{ in.}^2$$

Example I-13: Hat Section - Effective Section Properties Using Inelastic Reserve Capacity



Given:

1. Steel: F_y = 50 ksi
2. Section: 3HU4.5x135 as shown in sketch above
3. Top flange continuously braced

Required:

1. Determine the nominal flexural strength, M_{ny}, with the top flange in compression, based on initiation of yielding.
2. Determine the nominal flexural strength, M_{ny}, with the top flange in compression, based on inelastic reserve capacity.
3. Determine the effective area, A_e, at a uniform compressive stress of 50 ksi.

Solution:

Refer to Example I-6 for derivation of basic parameters.

1. Nominal Flexural Strength Based on Initiation of Yielding (Section C3.1.1(a))

Computation of I_y, first approximation:

Assume a compressive stress of f = F_y = 50 ksi in the top fiber of the section.

Assume the web is fully effective.

Element 3:

$h/t = 2.355/0.135 = 17.44 < 200$ OK. Assumed fully effective (Section B1.2-(a))

Element 5:

$w/t = 3.855/0.135 = 28.56 < 500$ OK (Section B1.1-(a)-(2))

k = 4.0 (fully stiffened element)

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \quad (Eq.\ B2.1\text{-}5)$$

$$= 4\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{28.56}\right)^2 = 130.8 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{50}{130.8}} = 0.618 < 0.673 \ \therefore \text{ flange is not subject to local buckling}$$

$$b = w \qquad (Eq.\ B2.1\text{-}1)$$

$$= 3.855 \text{ in.}$$

Effective section properties about y-axis:

Element	L (in.)	x from top fiber (in.)	Lx (in.^{2})	Lx^2 (in.^{3})	I'_y about own axis (in.^{3})
1	(2)(1.348) = 2.696	2.932	7.905	23.176	---
2	(2)(0.401) = 0.802	2.840	2.278	6.469	0.004
3	(2)(2.355) = 4.710	1.500	7.065	10.598	2.177
4	(2)(0.401) = 0.802	0.161	0.129	0.021	0.004
5	3.855	0.068	0.262	0.018	---
Sum $\sum$	12.865		17.639	40.282	2.185

Distance of neutral axis from top fiber,

$$\bar{x} = \sum Lx / \sum L = 17.639/12.865 = 1.371 \text{ in.}$$

$$I_y = \left[\sum Lx^2 + \sum I'_y - \bar{x}^2 \sum L\right] t$$

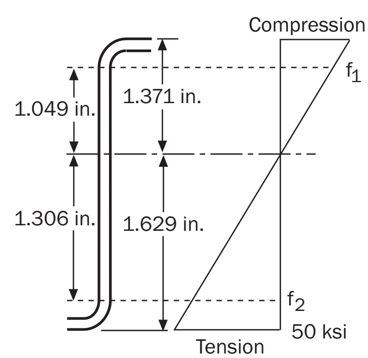
$$= \left[40.282 + 2.185 - (1.371)^2 (12.865)\right](0.135)$$

$$= 2.469 \text{ in.}^4$$

Since the distance of the top compression fiber from the neutral axis is less than one half of the beam depth, a compressive stress of f equal to F_y at the top fiber will not govern as assumed at the beginning of item 1. The actual compressive stress will be less than F_y, so the flange will still be fully effective. The tension flange will yield first.

Therefore,

Check web (element 3) under new assumed stress distribution



$$f_1 = (1.049/1.629)(50) = 32.20 \text{ ksi (compression)}$$

$$f_2 = -(1.306/1.629)(50) = -40.09 \text{ ksi (tension)}$$

$$\psi = |f_2/f_1| = |-40.09/32.20| = 1.245$$

$$k = 4 + 2(1+\psi)^3 + 2(1+\psi)$$

$$= 4 + 2(1+1.245)^3 + 2(1+1.245)$$

$$= 31.12$$

$$F_{cr} = 31.12 \frac{\pi^2 (29500)}{12(1-0.3^2)} \left(\frac{1}{17.44}\right)^2 = 2728 \text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{32.20}{2728}} = 0.109 < 0.673 \qquad (Eq.\ B2.1\text{-}4)$$

$b = w$ (*Eq.* B2.1-1)

$b_e = 2.355$ in.

$h_o/b_o = 3.0/4.5 = 0.67 < 4.0$

$b_1 = b_e/(3+\psi)$ (*Eq.* B2.3-3)

$= 2.355/(3+1.245) = 0.555$ in.

For $\psi > 0.236$

$b_2 = b_e/2$ (*Eq.* B2.3-4)

$= 2.355/2 = 1.178$ in.

$b_1+b_2 = 0.555 + 1.178 = 1.733$ in. > 1.049 in. (compression portion of web)

Therefore, web is fully effective.

$S_e = I_y/(d-\bar{x}) = 2.469/(3.000-1.371) = 1.516\text{ in.}^3$

$M_n = S_eF_y$ (*Eq.* C3.1.1-1)

$= (1.516)(50)$

$= 75.8$ kip-in.

2. Nominal Flexural Strength Based on Inelastic Reserve Capacity (Section C3.1.1(b))

Compute the maximum compression strain.

$$\lambda_1 = \frac{1.11}{\sqrt{F_y / E}} \quad (Eq.\ C3.1.1\text{-}3)$$

$$= \frac{1.11}{\sqrt{50 / 29500}} = 26.96$$

$$\lambda_2 = \frac{1.28}{\sqrt{F_y/E}} \quad (Eq.\ C3.1.1\text{-}4)$$

$$= \frac{1.28}{\sqrt{50/29500}} = 31.09$$

w/t of compression flange = 28.56

For $26.96 = \lambda_1 < w/t < \lambda_2 = 31.09$

$$C_y = 3-2\left(\frac{w/t-\lambda_1}{\lambda_2-\lambda_1}\right) \quad (Eq.\ C3.1.1\text{-}2)$$

$$= 3-2\left(\frac{28.56-26.96}{31.09-26.96}\right) = 2.23$$

Therefore, the maximum compression strain is 2.23 times the yield strain, e_y. The tension strain is not limited.

Compute location of e_y on a strain diagram such that the maximum compression strain does not exceed 2.23 e_y and the summation of longitudinal forces is zero[3].

Approximate distance from neutral axis to the outer compression fiber, y_c (not considering the effect of radii):

$t = 0.135$ in.

$b_t = 2(1.670) = 3.340$ in.

$b_c = 4.500$ in.

$d = 3.000$ in.

---

[3]Reck, Pekoz and Winter, "Inelastic Strength of Cold-Formed Steel Beams," *Journal of the Structural Division*, November 1975, ASCE.

$x_c \;=\; (1/4)(b_t - b_c + 2d)$

$\;\;\;\;\; = (1/4)[3.340 - 4.500 + 2(3.000)] = 1.210 \text{ in.}$

$x_p \;=\; x_c/C_y$

$\;\;\;\;\; = 1.210/2.23 = 0.543$ in.

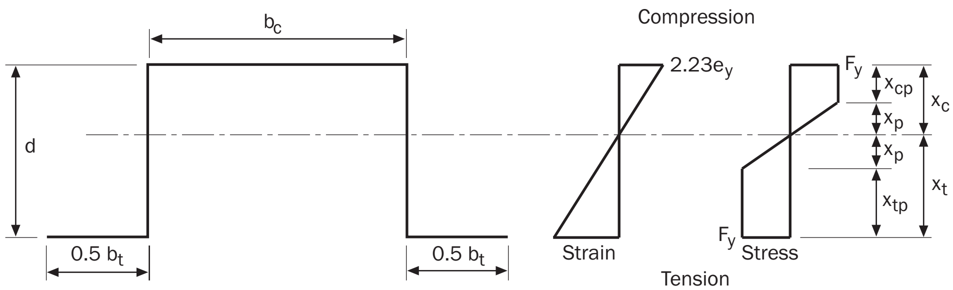
$x_t \;=\; d - x_c$

$\;\;\;\;\; = 3.000 - 1.210 = 1.790$ in.

$x_{cp} \;=\; x_c - x_p$

$\;\;\;\;\; = 1.210 - 0.543 = 0.667$ in.

$x_{tp} \;=\; x_t - x_p = 1.790 - 0.543 = 1.247$ in.



Summing moments of stresses in component plates:

$$M_n = F_y t\left\{b_c x_c + 2x_{cp}\left[x_p + \frac{x_{cp}}{2}\right] + \frac{4}{3}x_p{}^2 + 2x_{tp}\left[x_p + \frac{x_{tp}}{2}\right] + b_t x_t\right\}$$

$$M_n = (50.0)(0.135)\left\{\begin{array}{c}(4.500)(1.210) + (2)(0.667)\left[0.543 + \dfrac{0.667}{2}\right] + \dfrac{4}{3}(0.543)^2 \\ +2(1.247)\left[0.543 + \dfrac{1.247}{2}\right] + (3.340)(1.790)\end{array}\right\}$$

$M_n \;=\; 107.3$ kip-in.

M_n shall not exceed $1.25S_eF_y = 1.25(75.8) = 94.8$ kip-in. CONTROLS

Therefore

$M_n \;=\; 1.25S_eF_y = 94.8$ kip-in.

The inelastic reserve capacity can be used assuming the following conditions are met:

(1) The member is not subject to twisting, lateral, torsional, or flexural-torsional buckling.

(2) The effect of cold-forming is not included in determining the yield stress, F_y.

(3) The ratio of depth of the compressed portion of the web to its thickness does not exceed λ_1:
$(x_c - r - t/2)/t = (1.210 - 0.255 - 0.135/2)/0.135 = 6.6 < \lambda_1 = 26.96$ OK

(4) The shear force does not exceed $0.35F_y$ times the web area, ht, for ASD, and $0.6F_y$ht for LRFD.

(5) The angle between any web and the vertical does not exceed 30°.

3. Effective Area, A_e, at a Uniform Compressive Stress of f=50 ksi (Section C4)

Element 5: Uniformly Compressed Stiffened Element (Section B2.1)

$w/t = 3.855/0.135$
$= 28.56$

$k = 4.0$

$$F_{cr} = 4.0\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{28.56}\right)^2 = 130.8 \text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{50}{130.8}} = 0.618 < 0.673 \text{ (flange is fully effective)} \qquad (Eq.\ B2.1\text{-}4)$$

$b = w$ (*Eq.* B2.1-1)
$= 3.855$ in.

Element 3: Uniformly Compressed Element with a Simple Lip Edge Stiffener (Section B4)

$w = 2.355$ in.

$w/t = 2.355/0.135 = 17.4 < 60$ OK (Section B1.1(a)(1))

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-7)

$= 1.28\sqrt{29500/50} = 31.1 \therefore w/t \geq 0.328S \Rightarrow$ check effective width of element

Compute k of element based on stiffener lip (element 1) properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \leq t^4\left[115\frac{w/t}{S} + 5\right] \qquad (Eq.\ B4\text{-}8)$$

$$= 399(0.135)^4\left[\frac{17.4}{31.1} - 0.328\right]^3 \leq (0.135)^4\left[115\frac{17.4}{31.1} + 5\right]$$

$= 0.00164 \text{ in.}^4 < 0.0230 \text{ in.}^4 \therefore I_a = 0.00164 \text{ in.}^4$

$d = 1.348$ in.

$\theta = 90$ degrees

$I_s = (d^3 t \sin^2\theta)/12$ (*Eq.* B4-10)

$= (1.348)^3(0.135)\sin^2(90°)/12 = 0.0276 \text{ in.}^4$

$R_I = I_s/I_a \leq 1$ (*Eq.* B4-9)

$= 0.0276/0.00164 = 16.8 > 1 \quad \therefore R_I = 1$

$D = 1.67$ in.

$D/w = 1.67/2.355 = 0.71 < 0.8$ OK (From Table B4-1)

$$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \leq 4 \qquad \text{(From Table B4-1)}$$

$$= \left(4.82 - \frac{(5)(1.67)}{2.355}\right)(1)^n + 0.43 = 1.70 < 4 \text{ OK}$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 1.70\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{17.4}\right)^2 = 150 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{50}{150}} = 0.577 < 0.673 \therefore \text{ element is not subject to local buckling}$$

$b = w = 2.355$ in. (*Eq.* B2.1-1)

Element 1: Uniformly Compressed Unstiffened Element (Section B3.1 and B4)

$w/t = 1.348/0.135 = 9.99$

$k = 0.43$

$$F_{cr} = 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{9.99}\right)^2 = 115 \text{ ksi} \quad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{50}{115}} = 0.659 < 0.673 \ \therefore \text{ element is not subject to local buckling}$$

$$d'_s = b = w \quad (Eq.\ B2.1\text{-}1)$$

$$= 1.348 \text{ in.}$$

$$d_s = d'_s(R_I) \quad (Eq.\ B4\text{-}6)$$

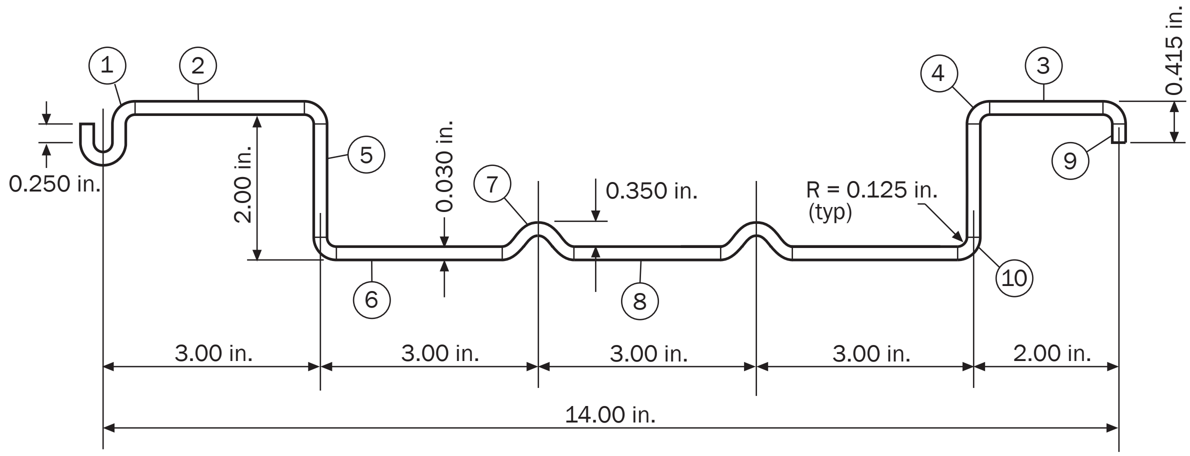
$$= (1.348)(1.0) = 1.348 \text{ in.}$$

Effective Area = Gross Area:

Element	L (in.)
1	(2)(1.348) = 2.696
2	(2)(0.401) = 0.802
3	(2)(2.355) = 4.710
4	(2)(0.401) = 0.802
5	3.855
Sum $\sum$	12.865

$$A_e = t\sum L = (0.135)(12.865) = 1.74 \text{ in.}^2$$

Example I-14: Wall Panel Section With Intermediate Stiffeners - Effective Section Properties



Given:

1. Steel: F_y = 50 ksi
2. Section: Shown in sketch above. Refer to Example I-7 for gross properties of elements. Section is assumed to be fully braced against member buckling.

Required:

1. Nominal flexural strength per panel, M_n, for positive and negative bending.
2. Effective moment of inertia, I_{eff}, at a moment of $0.6M_n$ with compression on the top. Use Procedure II from Part (b) of Section B2.1 to compute effective widths of stiffened elements at service load.

Solution:

1. Section Modulus, S_e, for Nominal Flexural Strength - Compression on Top

From Example I-7, the neutral axis is 1.186 in. below the top fibers of the gross cross-section (below the mid depth); therefore, the compression stress at the top fiber will govern and will equal F_y.

Unstiffened Lip of Element 1 from Section B3.2(a)

Although a more precise check of this unstiffened element with a stress gradient could be conducted using Section B3.2, since this is a very short element perform a simpler, conservative, preliminary check using Section B3.1 and $f=F_y$.

w = 0.250 in.

k = 0.43

f = F_y (Conservative assumption for preliminary check)

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \quad (Eq.\ B2.1\text{-}5)$$

$$= 0.43\frac{\pi^2 (29500)}{12\left(1-0.3^2\right)}\left(\frac{0.030}{0.250}\right)^2 = 165.1 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{50}{165.1}} = 0.550 < 0.673 \ \therefore \text{ lip is not subject to local buckling}$$

No need to calculate more precisely, since the lip is fully effective using conservative assumptions

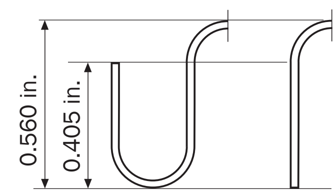
$b = w = 0.250$ in.

Stiffened Flat Element of Element 1 with Stress Gradient from Section B2.3

By inspection, the stress levels will be lower than those used to check the unstiffened element and k will be at least 4.0. Consequently, the element will be fully effective, since the unstiffened element is fully effective and the length is the same.

Element 2 from Section B4(a)

Evaluate element 2 as a uniformly compressed element with an edge stiffener. Prior editions of the *Specification* included provisions for evaluating the effective width of a uniformly compressed element with stiffeners other than simple lips; however, these provisions are no longer included. For purposes of evaluating the plate buckling coefficient, k, of the flange, the complex stiffener (element 1) will be simplified in a conservative way. A simple edge stiffener extending to the bottom of the complex stiffener with a flat width of 0.405 in. will be substituted, neglecting the beneficial contribution of the rest of the stiffener. For a more exact solution, the use of the Direct Strength provisions in *Specification* Appendix 1 is recommended.



$$w = 3.000 - 3(0.125 + 0.030/2) = 2.580 \text{ in.}$$

$$f = F_y = 50 \text{ ksi}$$

$$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/50} = 31.09 \qquad (Eq.\ B4\text{-}7)$$

$$w/t = 2.580/0.030 = 86.0 > 0.328S \quad \therefore \text{ check effective width}$$

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \qquad (Eq.\ B4\text{-}8)$$

$$= 399(0.030)^4\left[\frac{86.0}{31.09} - 0.328\right]^3 \le (0.030)^4\left[115\frac{86.0}{31.09} + 5\right]$$

$$= 0.00468 \text{ in.}^4 > 0.000262 \text{ in.}^4 \quad \therefore I_a = 0.000262 \text{ in.}^4$$

Using the simplified stiffener lip

$$I_s = \left(d^3 t \sin^2\theta\right)/12 \qquad (Eq.\ B4\text{-}10)$$

$$= (0.405)^3(0.030)\sin^2(90°)/12 = 0.000166 \text{ in.}^4$$

$$R_I = I_s/I_a \le 1 \qquad (Eq.\ B4\text{-}9)$$

$$= 0.000166/0.000262 = 0.634 < 1.0 \quad \therefore R_I = 0.634$$

$$n = \left(0.582 - \frac{w/t}{4S}\right) \ge \frac{1}{3} \qquad (Eq.\ B4\text{-}11)$$

$$= \left(0.582 - \frac{86.0}{4(31.09)}\right) \ge \frac{1}{3}$$

$$= -0.110 < 1/3 \quad \therefore n = 1/3$$

Using the simplified stiffener lip flat length of 0.405 in.,

$D/w = 0.560/2.580 = 0.217 < 0.25$; therefore,

$$k = 3.57\left(R_I\right)^n + 0.43 \le 4 \qquad \text{(From Table B4-1)}$$

$$= 3.57\left(0.634\right)^{1/3} + 0.43 = 3.50$$

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \qquad (\textit{Eq.}\ \text{B2.1-5})$$

$$= 3.50\frac{\pi^2\left(29500\right)}{12\left(1-0.3^2\right)}\left(\frac{0.030}{2.580}\right)^2 = 12.62\,\text{ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (\textit{Eq.}\ \text{B2.1-4})$$

$$= \sqrt{\frac{50.0}{12.62}} = 1.99 > 0.673 \ \therefore\ \text{element is subject to local buckling}$$

$$\rho = \left(1-0.22/\lambda\right)/\lambda \qquad (\textit{Eq.}\ \text{B2.1-3})$$

$$= \left(1-0.22/1.99\right)/1.99 = 0.447$$

$$b = \rho w \qquad (\textit{Eq.}\ \text{B2.1-2})$$

$$= \left(0.447\right)\left(2.580\right) = 1.153\,\text{in.}$$

Element 9 from Section B3.2(a)

Although a more precise check of this unstiffened element with a stress gradient could be conducted using Section B3.2, since this is a very short element, perform a simpler conservative preliminary check using Section B3.1 and $f=F_y$.

$$w = 0.415 - 0.030 - 0.125 = 0.260\ \text{in.}$$

$$k = 0.43$$

$f < F_y$, but use F_y as a conservative value

$$F_{cr} = 0.43\frac{\pi^2\left(29500\right)}{12\left(1-0.3^2\right)}\left(\frac{0.030}{0.260}\right)^2 = 152.6\ \text{ksi} \qquad (\textit{Eq.}\ \text{B2.1-5})$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (\textit{Eq.}\ \text{B2.1-4})$$

$$= \sqrt{\frac{50.0}{152.6}} = 0.572 < 0.673 \ \therefore\ \text{element is not subject to local buckling}$$

$$d'_s = b = w = 0.260\ \text{in.} \qquad (\textit{Eq.}\ \text{B2.1-1})$$

Element 3 from Section B4(a)

$$w = 2.000 - 2\left(0.125 + 0.030/2\right) = 1.720\,\text{in.}$$

$$f = F_y = 50\ \text{ksi}$$

$$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/50} = 31.09 \qquad (\textit{Eq.}\ \text{B4-7})$$

$$w/t = 1.720/0.030 = 57.33 > 0.328S \ \therefore\ \text{check effective width}$$

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \qquad (\textit{Eq.}\ \text{B4-8})$$

$$= 399\left(0.030\right)^4\left[\frac{57.33}{31.09} - 0.328\right]^3 \le \left(0.030\right)^4\left[115\frac{57.33}{31.09} + 5\right]$$

$= 0.00113 \text{ in.}^4 > 0.000176 \text{ in.}^4 \;\therefore\; I_a = 0.000176 \text{ in.}^4$

$I_s = (d^3 t \sin^2\theta)/12$ (*Eq.* B4-10)

$= (0.260)^3(0.030)\sin^2(90°)/12 = 0.0000439 \text{ in.}^4$

$R_I = I_s/I_a \le 1$ (*Eq.* B4-9)

$= 0.0000439/0.000176 = 0.249$

$n = \left(0.582 - \dfrac{w/t}{4S}\right) \ge \dfrac{1}{3}$ (*Eq.* B4-11)

$= \left(0.582 - \dfrac{57.33}{4(31.09)}\right) \ge \dfrac{1}{3}$

$= 0.121 < 1/3 \;\therefore\; n = 1/3$

$D/w = 0.415/1.720 = 0.241 < 0.25$

$k = 3.57(0.249)^{1/3} + 0.43 = 2.68 < 4$ (From Table B4.1)

$F_{cr} = 2.68\dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{0.030}{1.720}\right)^2 = 21.74 \text{ ksi}$ (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{f}{F_{cr}}} = \sqrt{\dfrac{50.0}{21.74}} = 1.517 > 0.673 \;\therefore$ element is subject to local buckling (*Eq.* B2.1-4)

$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)

$= (1 - 0.22/1.517)/1.517 = 0.564$

$b = \rho w$ (*Eq.* B2.1-2)

$= (0.564)(1.720) = 0.970 \text{ in.}$

Effective width of element 9

$d_s = d'_s(R_I) = (0.260)(0.249) = 0.065 \text{ in.}$ (*Eq.* B4-6)

Effective section properties about x-axis, assuming element 5 is fully effective:

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
1	1.160	0.321	0.372	0.120	0.031
2	1.153	0.015	0.017	---	---
3	0.970	0.015	0.015	---	---
4	0.660	0.066	0.044	0.003	0.001
5	3.440	1.015	3.492	3.544	0.848
6	4.788	2.015	9.648	19.440	---
7	2.396	1.840	4.409	8.112	0.032
8	2.068	2.015	4.167	8.397	---
9	0.065	0.188	0.012	0.002	---
10	0.440	1.964	0.864	1.697	0.001
Sum $\sum$	17.140		23.040	41.315	0.913

$\bar{y} = \sum Ly / \sum L = 23.040/17.140 = 1.344$ in.; below mid-depth as assumed

$I_x = \left[\sum Ly^2 + \sum I'_x - \bar{y}^2 \sum L\right] t$

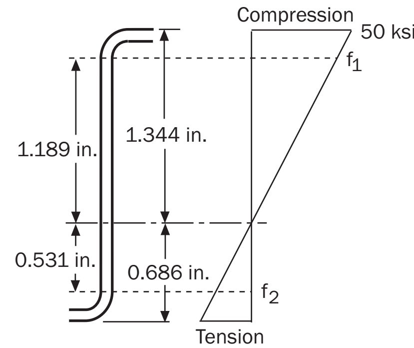
$= \left[41.315 + 0.913 - (1.344)^2 (17.140)\right](0.030) = 0.338 \text{ in.}^4$

$S_{xt} = I_x / y_{cg} = 0.338/1.344 = 0.251 \text{ in.}^3$

$M_n = S_e F_y = 0.251(50) = 12.6$ kip-in (*Eq.* C3.1.1-1)

Element 5 from Section B2.3(a): check assumption that element is fully effective

$y_{cg} = 1.344$ in.



$f_1 = \left[(1.344 - 0.125 - 0.030)/1.344\right](50)$

$= 44.23$ ksi

$f_2 = -\left[(2.030 - 0.125 - 0.030 - 1.344)/1.344\right](50)$

$= -19.75$ ksi

$\psi = |f_2/f_1| = |-19.75/44.23| = 0.447$ (*Eq.* B2.3-1)

$k = 4 + 2(1+\psi)^3 + 2(1+\psi)$ (*Eq.* B2.3-2)

$= 4 + 2(1+0.447)^3 + 2(1+0.447) = 12.95$

$f = f_1$

$w = 2.030 - (2)(0.125 + 0.030) = 1.720$ in.

$F_{cr} = 12.95 \dfrac{\pi^2 (29500)}{12(1-0.3^2)} \left(\dfrac{0.030}{1.720}\right)^2 = 105.0$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{f}{F_{cr}}}$ (*Eq.* B2.1-4)

$= \sqrt{\dfrac{44.23}{105.0}} = 0.649 < 0.673$; therefore, $b_e = w$

$b_e = w = 1.720$ in. (*Eq.* B2.1-1)

$h_o/b_o = 2.030/2.890 = 0.702 < 4.0$

$b_1 = b_e/(3+\psi)$ (*Eq.* B2.3-3)

$= 1.720/(3+0.447) = 0.499$ in.

For $\psi > 0.236$

$b_2 = b_e/2$ (*Eq.* B2.3-4)

$= 1.720/2 = 0.860$ in.

$w_c = 1.344 - 0.030 - 0.125 = 1.189$ in. (compression portion of web)

$b_1 + b_2 = 0.499 + 0.860 = 1.359$ in. > 1.189 in.

Thus element 5 is fully effective so properties above are correct.

2. Moment of Inertia for Deflection Determination - Compression on Top, $M_s = 0.6M_n = 7.56$ kip-in.

A conservative approximation of flexural deflections can be obtained by performing an elastic beam analysis using the effective moment of inertia of the cross-section calculated with the maximum extreme fiber stress set to the maximum flexural stress occurring under serviceability loading. In the case of continuous beams, the average of the moments of inertia in maximum positive and negative bending can be used.

For computation of a first approximation of I_{eff}, assume a compressive stress of $f = 0.6F_y = 30$ ksi in the top fibers of the section. Since all elements except 2, 3 and 9 were fully effective at 50 ksi, they will still be fully effective at this lower stress level. Check elements 2, 3 and 9.

Element 2 from Section B4(a)

Use the same simplifying assumption for the complex stiffener lip that was used above.

$$w = 3.000 - 3(0.125 + 0.030/2) = 2.580 \text{ in.}$$

$$f = 30 \text{ ksi}$$

$$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/30} = 40.14 \qquad (Eq.\ B4\text{-}7)$$

$$w/t = 2.580/0.030 = 86.0 > 0.328S \quad \therefore \text{ check effective width}$$

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \qquad (Eq.\ B4\text{-}8)$$

$$= 399(0.030)^4\left[\frac{86.0}{40.14} - 0.328\right]^3 \le (0.030)^4\left[115\frac{86.0}{40.14} + 5\right]$$

$$I_a = 0.00193 \text{ in.}^4 > 0.000204 \text{ in.}^4 \quad \therefore I_a = 0.000204 \text{ in.}^4$$

$$I_s = 0.000166 \text{ in.}^4 \text{ from above}$$

$$R_I = I_s/I_a \le 1 \qquad (Eq.\ B4\text{-}9)$$

$$= 0.000166/0.000204 = 0.814$$

$$n = \left(0.582 - \frac{w/t}{4S}\right) \ge \frac{1}{3} \qquad (Eq.\ B4\text{-}11)$$

$$= \left(0.582 - \frac{86.0}{4(40.14)}\right) \ge \frac{1}{3}$$

$$= 0.046 < 1/3 \quad \therefore n = 1/3$$

$$k = 3.57(0.814)^{1/3} + 0.43 = 3.76 < 4 \qquad (\text{From Table B4-1})$$

$$F_{cr} = 3.76\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{2.580}\right)^2 = 13.56 \text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{30.0}{13.56}} = 1.487 > 0.673 \quad \therefore \text{ element is subject to local buckling} \qquad (Eq.\ B2.1\text{-}4)$$

$$\lambda_c = 0.256 + 0.328(w/t)\sqrt{F_y/E} \qquad (Eq.\ B2.1\text{-}10)$$

$$= 0.256 + 0.328(2.580/0.030)\sqrt{50/29500} = 1.417$$

for $\lambda \ge \lambda_c$,

$$\rho = \left(0.41 + 0.59\sqrt{F_y/f_d} - 0.22/\lambda\right)/\lambda \qquad (Eq.\ B2.1\text{-}9)$$

$$= \left(0.41 + 0.59\sqrt{50/30} - 0.22/1.487\right)/1.487 = 0.688$$

$b = \rho w$ (*Eq.* B2.1-2)

$= (0.688)(2.580) = 1.775$ in.

Elements 3 and 9 from Section B4(a)

$w = 2.000 - 2(0.125 + 0.030/2) = 1.720$ in.

$f = 30$ ksi

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/30} = 40.14$ (*Eq.* B4-7)

$w/t = 1.720/0.030 = 57.33 > 0.328S \therefore$ check effective width

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right]$$ (*Eq.* B4-8)

$$= 399(0.030)^4\left[\frac{57.33}{40.14} - 0.328\right]^3 \le (0.030)^4\left[115\frac{57.33}{40.14} + 5\right]$$

$= 0.000430$ in.$^4 > 0.000137$ in.$^4 \therefore I_a = 0.000137$ in.^{4}

$I_s = 0.0000439$ in.^{4} from above

$R_I = I_s/I_a = 0.0000439/0.000137 = 0.320$ (*Eq.* B4-9)

$$n = \left(0.582 - \frac{57.33}{4(40.14)}\right) \ge \frac{1}{3}$$ (*Eq.* B4-11)

$= 0.225 < 1/3 \quad \therefore n = 1/3$

$D/w = 0.415/1.72 = 0.241 < 0.25$; therefore,

$k = 3.57(0.320)^{1/3} + 0.43 = 2.87$ (From Table B4-1)

$$F_{cr} = 2.87\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{1.720}\right)^2 = 23.28 \text{ ksi}$$ (*Eq.* B2.1-5)

$$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{30.0}{23.28}} = 1.135 > 0.673$$ $\therefore$ element is subject to local buckling (*Eq.* B2.1-4)

$\lambda_c = 0.256 + 0.328(1.720/0.030)\sqrt{50/29500} = 1.030$ (*Eq.* B2.1-10)

for $\lambda \ge \lambda_c$

$\rho = \left(0.41 + 0.59\sqrt{50/30} - 0.22/1.135\right)/1.135 = 0.862$ (*Eq.* B2.1-9)

$b = \rho w = (0.862)(1.720) = 1.483$ in. (*Eq.* B2.1-2)

Effective width of element 9

$d_s = d'_s(R_I) = (0.260)(0.320) = 0.083$ in. (*Eq.* B4-6)

Effective section properties about x-axis

Element	L (in.)	y from top fiber (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I'_x about own axis ($in.^3$)
1	1.160	0.321	0.372	0.120	0.031
2	1.775	0.015	0.027	---	---
3	1.483	0.015	0.022	---	---
4	0.660	0.066	0.044	0.003	0.001
5	3.440	1.015	3.492	3.544	0.848
6	4.788	2.015	9.648	19.440	---
7	2.396	1.840	4.409	8.112	0.032
8	2.068	2.015	4.167	8.397	---
9	0.083	0.197	0.016	0.003	---
10	0.440	1.964	0.864	1.697	0.001
Sum $\sum$	18.293		23.061	41.316	0.913

$$\bar{y} = \sum Ly / \sum L = 23.061/18.293 = 1.261 \text{ in.}$$

$$I_e = \left[\sum Ly^2 + \sum I'_x - \bar{y}^2 \sum L\right] t$$

$$= \left[41.316 + 0.913 - (1.261)^2 (18.293)\right](0.030) = 0.394 \text{ in.}^4$$

$$S_e = I_e/\bar{y} = 0.394/1.261 = 0.312 \text{ in.}^3$$

$$M_n = S_e f = (0.312)(30) = 9.36 \text{ kip-in} > M_s = 7.56 \text{ kip-in.}$$

For the second approximation, estimate the compression stress, f, in the top fibers of the section at M = 7.56 kip-in. by extrapolation:

M = 12.6 kip-in. at f = 50 ksi

M = 9.36 kip-in. at f = 30 ksi

for M =7.56 kip-in.:

$$(12.6 - 9.36)/(50 - 30) = (9.36 - 7.56)/(30 - f)$$

f = 18.9 ksi

For the second approximation, assume a compression stress of f = 18.9 ksi in the top fiber of the section.

Element 2 from Section B4(a)

w = 2.580 in.

f = 18.9 ksi

$$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/18.9} = 50.57 \quad (Eq.\ B4\text{-}7)$$

$$w/t = 2.580/0.030 = 86.0 > 0.328S \ \therefore \text{ check local buckling}$$

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4\text{-}8)$$

$$= 399(0.030)^4\left[\frac{86.0}{50.57} - 0.328\right]^3 \le (0.030)^4\left[115\frac{86.0}{50.57} + 5\right]$$

$= 0.000836$ in.^{4} > 0.000162 in.^{4} ; therefore, $I_a = 0.000162$ in.^{4}

I_s = 0.000166 in.^{4} from above

R_I = $I_s/I_a \le 1$, by inspection R_I = 1.0 (*Eq.* B4-9)

$k = 3.57(1.0)^{1/3} + 0.43 = 4.0$ (From Table B4-1)

$$F_{cr} = 4.0\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{2.580}\right)^2 = 14.42 \text{ ksi} \quad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{18.9}{14.42}} = 1.145 > 0.673 \quad \therefore \text{ element is subject to local buckling} \quad (Eq.\ B2.1\text{-}4)$$

λ_c = 1.417 (from above)

for $0.673 < \lambda < \lambda_c$

$\rho = (1.358 - 0.461/\lambda)/\lambda$ (*Eq.* B2.1-8)

$= (1.358 - 0.461/1.145)/1.145 = 0.834$

$b = \rho w = (0.834)(2.580) = 2.152$ in. (*Eq.* B2.1-2)

Elements 3 and 9 from Section B4(a)

w = 1.720 in.

f = 18.9 ksi

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/18.9} = 50.57$ (*Eq.* B4-7)

$w/t = 1.720/0.030 = 57.33 > 0.328S \quad \therefore$ check local buckling

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4\text{-}8)$$

$$= 399(0.030)^4\left[\frac{57.33}{50.57} - 0.328\right]^3 \le (0.030)^4\left[115\frac{57.33}{50.57} + 5\right]$$

$= 0.000169$ in.^{4} > 0.000110 in.^{4} $\therefore$ I_a = 0.000110 in.^{4}

$R_I = I_s/I_a = 0.0000439/0.000110 = 0.399$ (*Eq.* B4-9)

$$n = \left(0.582 - \frac{57.33}{4(50.57)}\right) \ge \frac{1}{3} \quad (Eq.\ B4\text{-}11)$$

$= 0.299 < 1/3 \quad \therefore n = 1/3$

$k = 3.57(0.399)^{1/3} + 0.43 = 3.06$ (From Table B4-1)

$$F_{cr} = 3.06\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{1.720}\right)^2 = 24.82 \text{ ksi} \quad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{18.9}{24.82}} = 0.873 > 0.673 \quad \therefore \text{ element is subject to local buckling} \quad (Eq.\ B2.1\text{-}4)$$

λ_c = 1.030 from above (*Eq.* B2.1-10)

for $0.673 < \lambda < \lambda_c$

$\rho = (1.358 - 0.461/\lambda)/\lambda$ (*Eq.* B2.1-8)

$= (1.358 - 0.461/0.873)/0.873 = 0.951$

$b = \rho w = (0.951)(1.720) = 1.636$ in. (*Eq.* B2.1-2)

Effective width of element 9

d_s = $d'_s(R_I) = (0.260)(0.399) = 0.104$ in. (*Eq.* B4-6)

Effective section properties about x-axis

Element	L (in.)	y from top fiber (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I'_x about own axis ($in.^3$)
1	1.160	0.321	0.372	0.120	0.031
2	2.152	0.015	0.032	---	---
3	1.636	0.015	0.025	---	---
4	0.660	0.066	0.044	0.003	0.001
5	3.440	1.015	3.492	3.544	0.848
6	4.788	2.015	9.648	19.440	---
7	2.396	1.840	4.409	8.112	0.032
8	2.068	2.015	4.167	8.397	---
9	0.104	0.207	0.022	0.004	---
10	0.440	1.964	0.864	1.697	0.001
Sum $\sum$	18.844		23.075	41.317	0.913

$\bar{y}$ = $\sum Ly / \sum L = 23.075/18.844 = 1.225$ in.

I_e = $\left[\sum Ly^2 + \sum I'_x - \bar{y}^2 \sum L\right] t$

= $\left[41.317 + 0.913 - (1.225)^2 (18.844)\right](0.030) = 0.419$ in.^{4}

S_e = $I_e/\bar{y} = 0.419/1.225 = 0.342$ in.^{3}

M_n = $S_e f = (0.342)(18.9) = 6.46$ kip-in < M_s = 7.56 kip-in.

For the third approximation, estimate the compression stress, f, in the top fibers of the section at M = 7.56 kip-in. by interpolation:

M = 9.36 kip-in. at f = 30 ksi

M = 6.46 kip-in. at f = 18.9 ksi

for M =7.56 kip-in.:

$(9.36 - 6.46)/(30 - 18.9) = (9.36 - 7.56)/(30 - f)$

f = 23.1 ksi

Repeating the previous calculations (not shown) with f =23.1 ksi gives:

I_e = 0.419 in.^{4}

S_e = 0.342 in.^{3}

M = $S_e f$ = (0.342)(23.1) = 7.90 kip-in. ≅ 7.56 kip-in.

Therefore, the effective moment of inertia is:

I_e = 0.419 in.^{4}

3. Section Modulus, S_e, for Nominal Flexural Strength - Compression on Bottom

If the neutral axis is closer to the compression flange than to the tension flange, the compression stress is less than F_y and is unknown, and therefore, the effective width of the compression flange and the effective section properties must be determined by an iterative method.

By inspection, elements 1, 2, 3, 4, and 9 are in tension and are therefore fully effective. Assume compression stress will govern, i.e., f = F_y = 50 ksi in the bottom compression fibers of the section.

Elements 6, 7 and 8 from Section B5.1

Check the effective width of the intermediately stiffened elements at the bottom of the panel per Section B5.1.1 for the case of two identical stiffeners, equally spaced.

$n = 2$

$A_g = (4.788 + 2.396 + 2.068)(0.030) = 0.278 \text{ in.}^2$ (from Example I-7)

$A_s = (1.198)(0.030) = 0.0359 \text{ in.}^2$

$b_o = 3.000 + 3.000 + 3.000 - (2)(0.125 + 0.030/2) = 8.720 \text{ in.}$

$h = 2.030 - 2(0.125 + 0.030) = 1.720 \text{ in.}$

$I_{sp} = I'_x t + A_s y^2$

$= (0.0159)(0.030) + (0.0359)(0.380/2 - 0.030/2)^2 = 0.00158 \text{ in.}^4$

$$\gamma = \frac{10.92 I_{sp}}{b_o t^3} \quad (Eq.\ \text{B5.1.1-4})$$

$$= \frac{(10.92)(0.00158)}{(8.720)(0.030)^3} = 73.3$$

$$\delta = \frac{A_s}{b_o t} \quad (Eq.\ \text{B5.1.1-5})$$

$$= \frac{0.0359}{8.72(0.030)} = 0.137$$

$$\beta = \left(1 + \gamma(n+1)\right)^{1/4} \quad (Eq.\ \text{B5.1.1-3})$$

$$= (1 + 73.3(2+1))^{1/4} = 3.86$$

$$k_d = \frac{\left(1+\beta^2\right)^2 + \gamma(1+n)}{\beta^2\left(1+\delta(n+1)\right)} \quad (Eq.\ \text{B5.1.1-2})$$

$$= \frac{\left(1+3.86^2\right)^2 + 73.3(1+2)}{3.86^2\left(1+0.137(2+1)\right)} = 22.5$$

$b_o/h = 8.720/1.72 = 5.07 > 1.0$; therefore,

$$R = \frac{11 - b_o/h}{5} \ge \frac{1}{2} \quad (Eq.\ \text{B5.1-6})$$

$$= \frac{11 - 8.720/1.720}{5} = 1.186$$

$Rk_d = (1.186)(22.5) = 26.7$

$$k_{loc} = 4\left(b_o/b_p\right)^2 \quad (Eq.\ \text{B5.1.1-1})$$

$$= 4(8.720/2.396)^2 = 53.0$$

$k = \min(Rk_d, k_{loc}) = \min(26.7, 53.0) = 26.7$ $\therefore$ distortional buckling controls

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{b_o}\right)^2 \quad (Eq.\ \text{B5.1-4})$$

$$= 26.7\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{8.720}\right)^2 = 8.43 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{50}{8.43}} = 2.435 > 0.673 \therefore \text{ element is subject to buckling} \qquad (\textit{Eq. } \text{B5.1-3})$$

$$\rho = (1-0.22/\lambda)/\lambda \qquad (\textit{Eq. } \text{B5.1-2})$$

$$= (1-0.22/2.435)/2.435 = 0.374$$

$$b_e = \rho\left(\frac{A_g}{t}\right) \qquad (\textit{Eq. } \text{B5.1-1})$$

$$= 0.374\left(\frac{0.278}{0.030}\right) = 3.466 \text{ in.}$$

location of centroid of stiffened element

$$\bar{y} = \frac{(4.788)(2.015)+(2.396)(1.840)+(2.068)(2.015)}{4.788+2.396+2.068} = 1.970 \text{ in.}$$

Effective section properties about x-axis, assuming element 5 is fully effective:

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
1	1.160	0.321	0.372	0.120	0.031
2	2.580	0.015	0.039	0.001	---
3	1.720	0.015	0.026	---	---
4	0.660	0.066	0.044	0.003	0.001
5	3.440	1.015	3.492	3.544	0.848
6, 7, 8	3.466	1.970	6.828	13.451	---
9	0.260	0.285	0.074	0.021	0.001
10	0.440	1.964	0.864	1.697	0.001
Sum $\sum$	13.726		11.739	18.837	0.882

$$\bar{y} = \sum Ly/\sum L = 11.739/13.726 = 0.855 \text{ in.} < 1.015 \text{ in. (above centerline)}$$

$$y_c = 2.030 - 0.855 = 1.175 \text{ in.}$$

Therefore, compression stress controls as assumed.

$$I_e = \left[\sum Ly^2 + \sum I'_x - \bar{y}^2 \sum L\right]t$$

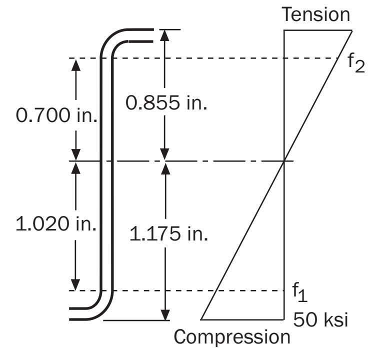
$$= \left[18.837 + 0.882 - (0.855)^2(13.726)\right](0.030) = 0.291 \text{ in.}^4$$

$$S_{et} = I_e/\bar{y} = 0.291/0.855 = 0.340 \text{ in.}^3$$

$$S_{eb} = 0.291/1.175 = 0.248 \text{ in.}^3$$

$$M_n = \min(S_{et}, S_{eb})F_y = 0.248(50) = 12.4 \text{ kip-in.} \qquad (\textit{Eq. } \text{C3.1.1-1})$$

Element 5 from Section B2.3(a): check assumption that element is fully effective



$f_1 = \left[\left(1.175 - 0.125 - 0.030\right)/1.175\right]\left(50\right)$

$= 43.40$ ksi

$f_2 = -\left[\left(0.855 - 0.125 - 0.030\right)/1.175\right]\left(50\right)$

$= -29.79$ ksi

$\psi = \left|f_2 / f_1\right| = \left|-29.79/43.40\right| = 0.686$ (*Eq.* B2.3-1)

$k = 4 + 2\left(1+\psi\right)^3 + 2\left(1+\psi\right)$ (*Eq.* B2.3-2)

$= 4 + 2\left(1+0.686\right)^3 + 2\left(1+0.686\right) = 16.96$

$f = f_1$

$w = 1.720$ in.

$F_{cr} = 16.96 \dfrac{\pi^2\left(29500\right)}{12\left(1-0.3^2\right)}\left(\dfrac{0.030}{1.720}\right)^2 = 137.6$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{f}{F_{cr}}}$ (*Eq.* B2.1-4)

$= \sqrt{\dfrac{43.40}{137.6}} = 0.562 < 0.673$

$b_e = w = 1.720$ in. (*Eq.* B2.1-1)

$h_o/b_o = 2.03/8.72 = 0.23 < 4$

$b_1 = b_e/(3+\psi)$ (*Eq.* B2.3-3)

$= 1.720/(3+0.686) = 0.467$ in.

For $\psi > 0.236$

$b_2 = b_e/2$ (*Eq.* B2.3-4)

$= 1.720/2 = 0.860$ in.

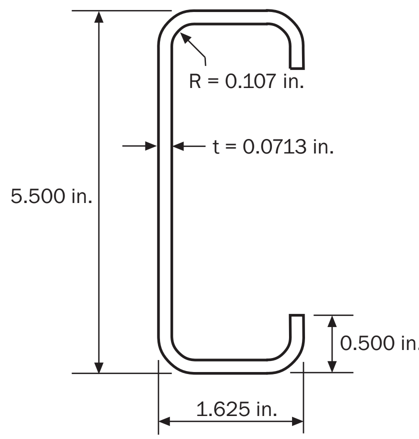
$w_c = 1.175 - 0.030 - 0.125 = 1.020$ in. (compression portion of web)

$b_1+b_2 = 0.467 + 0.860 = 1.327$ in. > 1.020 in.

Thus element 5 is fully effective so properties above are correct.

3.8 Special Topics

Example I-15: Strength Increase From Cold Work of Forming



Given:

1. $F_{yv} = F_y = 33$ ksi
2. $F_{uv} = F_u = 45$ ksi
3. Section: Stud 550S162-68 as shown above
4. Section to be used as a beam and is fully braced against lateral-torsional buckling

Required:

1. Determine the nominal flexural strength, M_n, considering the increase in strength resulting from the cold work of forming, using the provisions of *Specification* Section A7.2.

Solution:

1. Check the Limitations

In order to use *Eq.* A7.2-1 for computing the average tensile yield stress for the beam flange, the geometry of the section and yield stress must be such that the quantity ρ is unity as determined according to Section B2 for each of the flat elements of the section. In the case of webs under a stress gradient, if the sum of b_1 and b_2 from Section B2.3 at least equals the depth of the compression portion of the web, the web is considered to meet this criteria, even if ρ is less than 1.0.

Assume $\rho = 1.0$ for each flat element and check the elements at a maximum flexural stress, f, of F_{ya}.

Eq. A7.2-2 is applicable only when $F_{uv}/F_{yv} \geq 1.2$, $R/t \leq 7$ and the minimum included angle $\leq 120°$.

$F_{uv}/F_{yv} = 45/33 = 1.36 > 1.2$ OK

$R/t = 0.107/0.0713 = 1.50 \leq 7$ OK

$\theta = 90° < 120°$ OK

Therefore, *Eq.* A7.2-2 can be used to determine F_{yc}

2. Calculation of F_{yc}

$$B_c = 3.69\left(F_{uv}/F_{yv}\right) - 0.819\left(F_{uv}/F_{yv}\right)^2 - 1.79 \qquad (Eq.\ A7.2\text{-}3)$$

$$= 3.69(1.36) - 0.819(1.36)^2 - 1.79 = 1.714$$

$$m = 0.192\left(F_{uv}/F_{yv}\right) - 0.068 \qquad (Eq.\ A7.2\text{-}4)$$

$$= 0.192(1.36) - 0.068 = 0.193$$

$F_{yc} = B_c F_{yv} / (R/t)^m$ (*Eq.* A7.2-2)

$= 1.714(33)/(1.50)^{0.193} = 52.30\,\text{ksi}$

3. Calculation of F_{ya}

$r = R + t/2 = 0.107 + 0.0713/2 = 0.143$ in.

Cross-sectional area of corner = $(\pi/2)(0.143)(0.0713) = 0.0160\,\text{in.}^2$

Total corner cross-sectional area of the controlling flange

$= (0.0160)(2) = 0.0320\,\text{in.}^2$

Flat width of the compression flange

$w = b - 2(t + R)$

$= 1.625 - 2(0.0713 + 0.107) = 1.268$ in.

Full cross-sectional area of the controlling flange

$A_{flange} = 0.0320 + (1.268)(0.0713) = 0.122\,\text{in.}^2$

$C = 0.0320/0.122 = 0.262$

$F_{yf} = F_{yv} = 33$ ksi

$F_{ya} = CF_{yc} + (1 - C)F_{yf}$ (*Eq.* A7.2-1)

$= (0.262)(52.30) + (1 - 0.262)(33) = 38.06\,\text{ksi}$

4. Check Effective Width Assumptions

Recheck each flat compression element based on an extreme fiber compressive bending stress of 38.06 ksi. It can be demonstrated, by calculations not shown, that each flat element is fully effective, therefore, the increase from cold work of forming may be used.

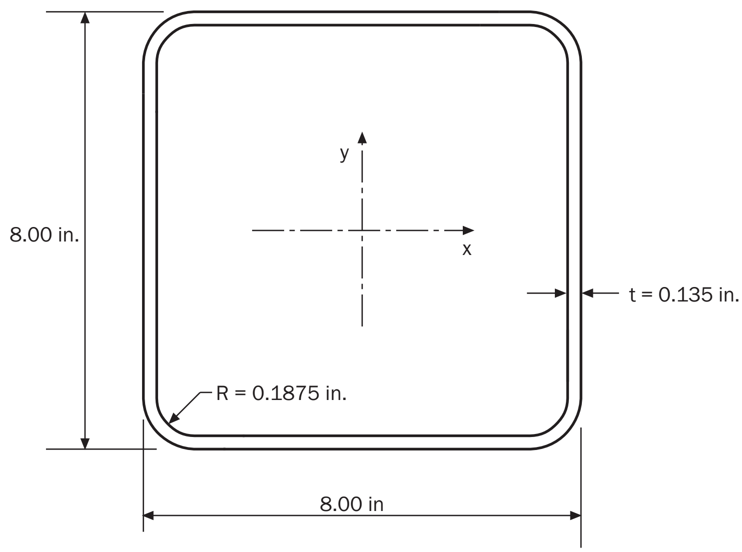
5. Calculation of M_n

$M_n = S_e F_y$ (*Eq.* C3.1.1-1)

where F_y is taken as F_{ya} per Section A7.2

$M_n = (1.041)(38.06) = 39.6$ kip-in.

Example I-16: Shear Lag



Given:

1. Steel: F_y = 50 ksi
2. Section: 8 x 8 x 0.135 square tube
3. Span: L = 3 ft., with simple supports
4. Loading: Concentrated load at mid-span
5. A_{gross} = 4.19 in.^{2}
6. I_{gross} = 42.8 in.^{4}

Required:

1. Determine the ASD flexural allowable strength, M_n/Ω_b
2. Determine the LRFD flexural design strength, $\phi_b M_n$

Solution:

Compute the nominal flexural strength, M_n, as the lesser of the values determined according to Sections C3.1 and B1.1(c).

1. Nominal Moment Strength, M_n, Based on Initiation of Yielding (Section C3.1.1)

Since the member is not subject to lateral-torsional buckling, compute the nominal strength using Section C3.1.1.

Check compression flange in accordance with Section B2.1 with f = 50 ksi and k = 4.00. The compression flange is found to have an effective width of 5.071 in., by calculations not shown.

Check webs in accordance with Section B2.3. The reduced effective width of the compression flange will cause the neutral axis to shift towards the tension flange. Using f_1 and f_2 for the new position of the neutral axis, the webs are found to be fully effective, by calculation not shown.

The net section properties can then be calculated as:

I_x = 37.7 in.^{4}

S_e = 8.74 in.^{3}

M_n= 437 kip-in.

2. Nominal Moment Strength, M_n, Considering Shear Lag (Section B1.1(c))

$$w_f = \left[8.0-(2)(0.135)\right]/2 = 3.865 \text{ in.}$$

L/w_f = (3)(12)/3.865 = 9.31 < 30

Because the L/w_f ratio is less than 30, and the member carries a concentrated load, consideration of shear lag is required.

Interpolating from Table B1.1(c):

for $L/w_f = 10$, effective design width/actual width = 0.73

for $L/w_f = 8$, effective design width/actual width = 0.67

for $L/w_f = 9.31$,

effective design width/actual width = $0.67 + \left(\frac{0.73-0.67}{10-8}\right)(9.31-8) = 0.71$

Therefore, the maximum effective design widths of the compression and tension flanges between the inside of the webs are $0.71\left[8.0-(2)(0.135)\right] = 5.488\,\text{in.}$

The effective flat width of the flanges is:

$b_{max} = 5.488 - 2R = 5.488 - 2(0.1875) = 5.113\,\text{in.}$

Recalculate properties using effective compression and tension flange widths of 5.113 in. (governed by Section B1.1).

$I_{xe} = 33.43\ \text{in.}^4$

$S_e = 8.36\ \text{in.}^3$

$M_n = (8.36)(50) = 418\,\text{kip-in.}$ CONTROLS

3. Determination of the ASD Flexural Allowable Design Strength:

$M \leq M_n/\Omega_b$ (*Eq.* A4.1.1-1)

$\Omega_b = 1.67$

$M \leq 418/1.67 = 250\,\text{kip-in.}$

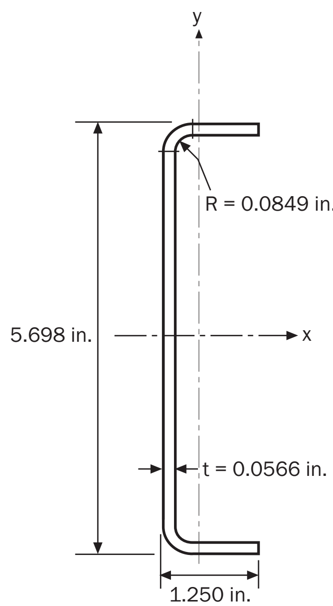
4. Determination of the LRFD Flexural Design Strength:

$M_u \leq \phi_b M_n$ (*Eq.* A5.1.1-1)

$\phi_b = 0.90$

$M_u \leq (0.90)(418) = 376\,\text{kip-in.}$

Example I-17: Flange Curling



Given:

1. Steel: F_y = 33 ksi
2. Section: Track 550T125-54 as shown above
3. Compression flange braced against lateral buckling

Required:

1. Determine the amount of curling of the compression flange at a maximum flexural compressive stress of 30.93 ksi, as used in Example II-4.

Solution:

1. Average Stress in Compression Flange, f_{av}

From Example I-9

w = 1.109 in.

b = 0.854 in.

$f_{av} = f(b/w) = 30.93(0.854/1.109) = 23.82$ ksi

2. Curling of the Compression Flange, c_f

$w_f = 1.250 - 0.0566 = 1.193$ in.

$$w_f = \sqrt{0.061tdE/f_{av}}\sqrt[4]{(100c_f/d)} \quad (Eq.\ B1.1\text{-}1)$$

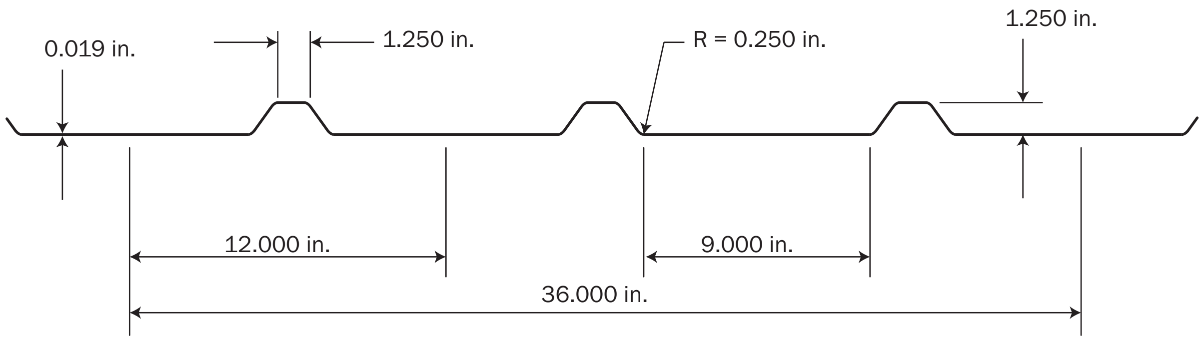
Solving for c_f:

$$c_f = \frac{w_f^{\,4}}{100d\left[0.061tE/f_{av}\right]^2}$$

$$= \frac{1.193^4}{(100)(5.698)\left[(0.061)(0.0566)(29500)/23.82\right]^2}$$

$c_f = 0.000194$ in.

Example I-18: Section Properties of Cross-Sections With Large Corner Radii



Given:

1. Steel: F_y = 80 ksi
2. Section: Panel section as shown above
3. Compression flange braced against lateral buckling

Required:

1. Determine the ASD and LRFD available strengths, with compression on the top at a maximum stress of F_y.

Solution:

Check R/t ratio:

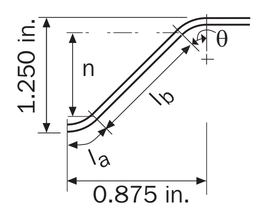
$$R / t = 0.25 / 0.019 = 13.16 > 10$$

Therefore, use the Direct Strength Method or rational analysis for the determination of effective section properties per Section B1.3. Use the rational analysis method outlined in the *Commentary* Section B1.3.

1. Gross Section Properties

For a one foot wide tributary width, neglecting laps,

Top and bottom corners



$\theta = 1.161\,\text{rads}$ (calculations not shown)

$$l_a = \theta\left(R + \frac{t}{2}\right) = 1.161\left(0.25 + \frac{0.0190}{2}\right) = 0.3013\,\text{in.}$$

$$I'_x = \left(\frac{\theta + \sin\theta\cos\theta}{2} - \frac{\sin^2\theta}{\theta}\right)\left(R + \frac{t}{2}\right)^3$$

$$= \left(\frac{1.161 + \sin 1.161 \cos 1.161}{2} - \frac{\sin^2 1.161}{1.161}\right)\left(0.25 + \frac{0.019}{2}\right)^3 = 0.000675\,\text{in.}^3$$

Top corners:

$$y_1 = \frac{t}{2} + \left(R + \frac{t}{2}\right) - \left(R + \frac{t}{2}\right)\left(\frac{\sin\theta}{\theta}\right) = \frac{t}{2} + \left(R + \frac{t}{2}\right)\left(1 - \frac{\sin\theta}{\theta}\right)$$

$$= \frac{0.019}{2} + \left(0.25 + \frac{0.019}{2}\right)\left(1 - \frac{\sin 1.161}{1.161}\right) = 0.0640\,\text{in.}$$

Bottom corners:

$$y_2 = d - \frac{t}{2} - \left(R + \frac{t}{2}\right)\left(1 - \frac{\sin\theta}{\theta}\right)$$

$$= 1.25 - \frac{0.019}{2} - \left(0.25 + \frac{0.019}{2}\right)\left(1 - \frac{\sin 1.161}{1.161}\right) = 1.1860\,\text{in.}$$

Webs

$$n = d - 2\left(R + \frac{t}{2}\right)(1 - \cos\theta) - t$$

$$= 1.25 - 2\left(0.25 + \frac{0.0190}{2}\right)(1 - \cos(1.161)) - 0.0190 = 0.9188\,\text{in.}$$

$$l_b = n/\sin\theta = 0.9188/\sin(1.161) = 1.0017\,\text{in.}$$

$$I'_x = \frac{l_b n^2}{12} = \frac{1.0017(0.9188)^2}{12} = 0.0705\ \text{in.}^3$$

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
Top Flat	1.250	0.0095	0.0119	0.000113	-----
2 Top Corners	2(0.3013)	0.0640	2(0.0193)	2(0.00123)	2(0.000675)
2 Webs	2(1.0017)	0.625	2(0.6261)	2(0.3913)	2(0.0705)
2 Bottom Corners	2(0.3013)	1.1860	2(0.3573)	2(0.4238)	2(0.000675)
Bottom Flat	9.00	1.2405	11.1645	13.8496	-----
Sum	13.459		13.182		

$$\bar{y} = \sum Ly/\sum L = 13.182/13.459 = 0.979\,\text{in. below top fiber}$$

First iteration

For first iteration, use $\bar{y}$ of gross properties.

Top flat

By inspection, f = F_y = 80 ksi

w = 1.250 in.

$k = 4.0$

From *Commentary* Section B1.3:

$$R_{R1} = R_{R2} = 1.08 - (R/t)/50 \le 1.0 \qquad (\textit{Eq. C-B1.3-2})$$

$$= 1.08 - \left(\frac{0.25}{0.019}\right)\Big/50 = 0.817$$

$$k_R = kR_{R1}R_{R2} = 4.0(0.817)(0.817) = 2.67 \qquad (\textit{Eq. C-B1.3-1})$$

$$F_{cr} = k_R \frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (\text{from } \textit{Eq. B2.1-5})$$

$$= 2.67 \frac{\pi^2 29500}{12(1-0.3^2)}\left(\frac{0.019}{1.25}\right)^2 = 16.5\,\text{ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{80}{16.5}} = 2.20 > 0.673 \qquad (Eq.\ B2.1\text{-}4)$$

$$\rho = (1 - 0.22 / \lambda)/\lambda = (1 - 0.22 / 2.20)/2.20 = 0.409 \qquad (Eq.\ B2.1\text{-}3)$$

$$b = \rho w = 0.409(1.25) = 0.511\,\text{in.} \qquad (Eq.\ B2.1\text{-}2)$$

Webs

Distance to flat from outside

$$y_f = \frac{d - n}{2} = \frac{1.25 - 0.9188}{2} = 0.166\,\text{in.}$$

$$f_1 = \left(\frac{\bar{y} - y_f}{\bar{y}}\right)F_y = \left(\frac{0.979 - 0.166}{0.979}\right)80 = 66.4\,\text{ksi}$$

$$f_2 = \left[\frac{\bar{y} - (d - y_f)}{\bar{y}}\right]F_y = \left[\frac{0.979 - (1.25 - 0.166)}{0.979}\right]80 = -8.58\,\text{ksi} \ \ \text{(tension)}$$

$$\psi = |f_2/f_1| = |-8.58/66.4| = 0.129 \qquad (Eq.\ B2.3\text{-}1)$$

$$k = 4 + 2(1+\psi)^3 + 2(1+\psi) \qquad (Eq.\ B2.3\text{-}2)$$

$$= 4 + 2(1+0.129)^3 + 2(1+0.129) = 9.14$$

$$f = f_1 = 66.4\,\text{ksi}$$

$$F_{cr} = 9.14\frac{\pi^2 29500}{12(1-0.3^2)}\left(\frac{0.019}{1.0017}\right)^2 = 87.7\,\text{ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{66.4}{87.7}} = 0.870 > 0.673 \qquad (Eq.\ B2.1\text{-}4)$$

$$\rho = (1 - 0.22/\lambda)/\lambda = (1 - 0.22/0.870)/0.870 = 0.859 \qquad (Eq.\ B2.1\text{-}3)$$

$$w = l_b = 1.0017\,\text{in.}$$

$$b_e = \rho w = 0.859(1.0017) = 0.860\,\text{in.} \qquad (Eq.\ B2.1\text{-}2)$$

Since $h_o/b_o < 4$:

$$b_1 = b_e / (3+\psi) = 0.860/(3+0.129) = 0.275\,\text{in.} \qquad (Eq.\ B2.3\text{-}3)$$

$$b_2 = b_e - b_1 = 0.860 - 0.275 = 0.585\,\text{in.} \qquad (Eq.\ B2.3\text{-}5)$$

Web compression length

$$l_c = w\frac{f_1}{f_1 - f_2} = 1.0017\frac{66.4}{66.4 + 8.58} = 0.887\,\text{in.}$$

Ineffective compression length

$$l_i = l_c - l_e = 0.887 - 0.860 = 0.027\,\text{in.}$$

For purposes of this example, ignore the small ineffective compression length.

Element	L (in.)	y from top fiber (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I'_x about own axis ($in.^3$)
Top Flat	0.511	0.0095	0.00485	0.0000461	-----
2 Top Corners	2(0.3013)	0.0640	2(0.0193)	2(0.00123)	2(0.000675)
2 Webs	2(1.0017)	0.625	2(0.6261)	2(0.3913)	2(0.0705)
2 Bottom Corners	2(0.3013)	1.1860	2(0.3573)	2(0.4238)	2(0.000675)
Bottom Flat	9.00	1.2405	11.1645	13.8496	-----
Sum	12.720		13.175	15.482	0.1437

$$\bar{y} = \sum Ly / \sum L = 13.175/12.720 = 1.04 \text{ in. below top fiber}$$

Assuming the slight shift in $\bar{y}$ will not cause a significant change in section properties, the nominal flexural strength, M_n, is computed as:

$$I_{xe} = \left[\sum Ly^2 + \sum I'_x - \bar{y}^2 \sum L\right] t$$

$$= [15.482 + 0.1437 - 1.04^2(12.720)]0.019 = 0.0355 \text{ in.}^4$$

$$S_e = \frac{I_{xe}}{\bar{y}} = \frac{0.0355}{1.04} = 0.0341 \text{ in.}^3$$

$$M_n = S_e f = 0.0341(80) = 2.73 \text{ kip-in./ft}$$

Available strengths

Because a rational engineering analysis was used to compute the effective section properties, use the safety and resistance factors given in Section A1.2(c)

ASD

$M \le M_n/\Omega_b$ (*Eq.* A4.1.1-1)

$\Omega_b = 2.00$

$M \le 2.73/2.00 = 1.37 \text{ kip-in./ft}$

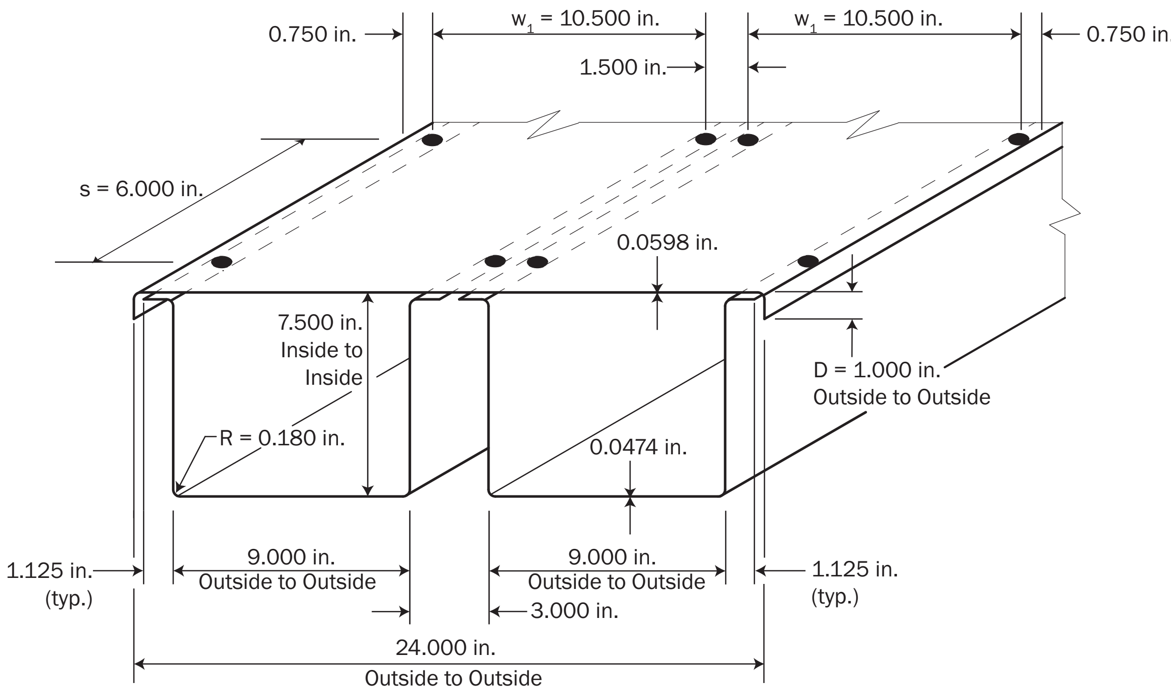
LRFD

$M_u \le \phi_b M_n$ (*Eq.* A5.1.1-1)

$\phi_b = 0.80$

$M_u \le (0.80)(2.73) = 2.18 \text{ kip-in./ft}$

Example I-19: Cellular Deck With Intermittent Fasteners



Given:

1. Geometry as shown in the figure. Deck is shown upside down for convenience.
2. F_y = 33 ksi
3. Weld spacing, s = 6.00 in.
4. Cover plate thickness = 0.0598 in.
5. Deck thickness = 0.0474 in.
6. Inside radii, R = 0.180 in.
7. Total depth, D = 7.6072 in.

Required:

1. Determine if longitudinal buckling needs to be considered in determining the effective section properties based on the given weld spacing, s, in accordance with Section D1.3.
2. Determine the ASD available flexural strength of the cellular deck with compression in the cover plate.

Solution:

1. Determine if Longitudinal Buckling of the Cover Plate Needs to be Considered (Section D1.3)

a) Calculate the compression stress in the cover plate, f_c, and determine s_{max} (Section D1.3)

Gross Section Properties

Item	L	n	y†	t	I′₁	nA	nAy	nAy²	nI′₁
Units	in.		in.	in.	in.^{4}	in.^{2}	in.^{3}	in.^{4}	in.^{4}
Plate flat	23.5204	1	0.0000	0.0598	0.0000	1.4065	0.0000	0.0000	0.0000
Plate arc	0.3297	2	0.0763	0.0598	0.0001	0.0394	0.0030	0.0002	0.0002
Plate stiff	0.7602	2	0.5900	0.0598	0.0022	0.0909	0.0536	0.0316	0.0044
Deck stiffener	0.9450	4	0.0536	0.0474	0.0000	0.1792	0.0096	0.0005	0.0000
Deck top arc	0.3200	4	0.1275	0.0474	0.0001	0.0607	0.0077	0.0010	0.0004
Deck web	7.0926	4	3.8036	0.0474	1.4093	1.3448	5.1149	19.4551	5.6373
Deck bot arc	0.3200	4	7.4797	0.0474	0.0001	0.0607	0.4538	3.3944	0.0004
Deck bot flat	8.5452	2	7.5536	0.0474	0.0000	0.8101	6.1192	46.2220	0.0000
					Σ	3.9923	11.7618	69.1048	5.6427
							Σ	74.7475	

†Distance from centerline of top cover plate to center of gravity of element under consideration.

$$y_{cg} = \sum Ay / \sum A$$
$$= 11.7618/3.9923 = 2.95\,\text{in.}$$
$$y_t = y_{cg} + 0.5t$$
$$= 2.95 + 0.5(0.0598) = 2.98\ \text{in.}$$
$$y_b = D - y_t$$
$$= 7.6072 - 2.98 = 4.63\ \text{in.}$$
$$I_x = \sum Ay^2 + \sum I_1' - \bar{y}^2 \sum A$$
$$= 69.1048 + 5.6427 - (2.95)^2\, 3.9923 = 40.0\,\text{in.}^4 = 20.0\,\text{in.}^4 / \text{ft}$$
$$S_{xtop} = I_x / y_t$$
$$= 40.0/2.98 = 13.4\,\text{in.}^3 = 6.70\,\text{in.}^3 / \text{ft}$$
$$S_{xbot} = I_x / y_b$$
$$= 40.0/4.63 = 8.64\,\text{in.}^3 = 4.32\,\text{in.}^3 / \text{ft}$$

Assuming f_{max} tension in the bottom = 33 ksi, and stress distribution based on gross properties,

$$f_c = f_{max}\left(\frac{y_t}{y_b}\right)$$
$$= 33\left(\frac{2.98}{4.63}\right) = 21.2\,\text{ksi}$$

b) Check Section D1.3 for connection spacing

$s_{max} = 1.16t\sqrt{E/f_c}$ (Section D1.3(b))

$= 1.16(0.0598)\sqrt{29500/21.2} = 2.59\,\text{in.} < 6.00\text{ in. NG}$

$s_{max} = 3w = 3(10.5) = 31.5\,\text{in.}$ (Section D1.3(c))

$w/t = 10.5/0.0598 = 176$

$0.50\sqrt{E/F_y} = 0.50\sqrt{29500/33} = 15.0 < w/t$; therefore,

$s_{max} = 1.33t\sqrt{E/F_y} = 1.33\left(0.0598\sqrt{29500/33}\right) = 2.38\,\text{in.}$

Section D1.3(b) controls.

s_{max} = 2.61 in. < 6.00 in.; therefore, longitudinal buckling needs to be considered in determining the effective section properties per Section B2.5.

2. Determine ASD Flexural Strength of the Cellular Deck With Compression in Cover Plate

a) Check Section B2.5 for strength based on the given connection spacing

Calculate critical column buckling stress of cover plate:

$F_c = 3.29E/(s/t)^2$ (*Eq*. B2.5-4)

$= 3.29(29500)/(6.00/0.0598)^2 = 9.64\,\text{ksi}$

b) Determine the effective width of the 10.5 in. cover plate segments between welds

Calculate critical buckling stress per *Eq*. B2.1-5 using k = 4.0:

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \quad (Eq.\ B2.1\text{-}5)$$

$$= 4.0\frac{\pi^2 29500}{12(1-0.3^2)}\left(\frac{0.0598}{10.5}\right)^2 = 3.46\,\text{ksi}$$

Assume the neutral axis shifts such that the extreme fiber stress in the cover plate = F_y and by calculations not shown:

$y_t = 4.30\,\text{in.}$

$y_b = 7.6072 - 4.30 = 3.31\,\text{in.}$

$f = 33.0\text{ ksi}$

Because $f > F_c$

$$\lambda_t = \sqrt{\frac{F_c}{F_{cr}}} = \sqrt{\frac{9.64}{3.46}} = 1.67 > 0.673 \quad (Eq.\ B2.5\text{-}3)$$

$$\rho_t = (1.0 - 0.22/\lambda_t)/\lambda_t \quad (Eq.\ B2.5\text{-}2)$$

$$= (1.0 - 0.22/1.67)/1.67 = 0.520 \quad \text{OK}$$

$$\rho_m = 8\left(\frac{F_y}{f}\right)\sqrt{\frac{tF_c}{df}} \le 1.0 \quad (Eq.\ B2.5\text{-}5)$$

$$= 8\left(\frac{33.0}{33.0}\right)\sqrt{\frac{0.0598(9.64)}{7.6072(33.0)}} \le 1.0$$

$$= 0.383 \le 1.0 \quad \text{OK}$$

$$\rho = \rho_t \rho_m \qquad (Eq.\ B2.5\text{-}1)$$
$$= 0.520(0.383)$$
$$= 0.199$$

The reduction factor per Section B2.1,

$$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{33.0}{3.46}} = 3.09 > 0.673 \qquad (Eq.\ B2.1\text{-}4)$$

$$\rho = (1 - 0.22/\lambda)/\lambda = (1 - 0.22/3.09)/3.09 = 0.301 \qquad (Eq.\ B2.1\text{-}3)$$

The reduction factor determined per Section B2.5 controls.

The effective width of each 10.5 in. segment between cover plate welds is:

$$b = \rho w = 0.199(10.5) = 2.09 \text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

c) Determine the effective width of the 1.5 in. cover plate segment between welds

Calculate critical buckling stress per *Eq.* B2.1-5 using k = 4.0:

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 4.0\frac{\pi^2 29500}{12(1-0.3^2)}\left(\frac{0.0598}{1.5}\right)^2 = 170\,\text{ksi}$$

Assume the neutral axis shifts such that the stress in the extreme fiber of the cover plate = F_y.

Therefore, because $f > F_c$:

$$\lambda_t = \sqrt{\frac{F_c}{F_{cr}}} = \sqrt{\frac{9.64}{170}} = 0.238 < 0.673\text{; therefore,} \qquad (Eq.\ B2.5\text{-}3)$$

$$\rho_t = 1.0$$

$$\rho_m = 8\left(\frac{F_y}{f}\right)\sqrt{\frac{tF_c}{df}} \le 1.0 \qquad (Eq.\ B2.5\text{-}5)$$

$$= 8\left(\frac{33.0}{33.0}\right)\sqrt{\frac{0.0598(9.64)}{7.6072(33.0)}} \le 1.0$$

$$= 0.383 \le 1.0 \quad \text{OK}$$

$$\rho = \rho_t \rho_m \qquad (Eq.\ B2.5\text{-}1)$$
$$= 1.00(0.383)$$
$$= 0.383$$

The reduction factor per Section B2.1,

$$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{33.0}{170}} = 0.441 < 0.673 \qquad (Eq.\ B2.1\text{-}4)$$

Since $\lambda < 0.673$, no reduction per Section B2.1 is required and the reduction factor determined per Section B2.5 controls.

The effective area of the 1.5 in. segment of cover plate between welds is:

$$b = \rho w = 0.383(1.50) = 0.575 \text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

d) Determine the effective width of the 0.75 in. cover plate segments between outside weld and stiffener

The effective width of the edge stiffener and flat portion between the outside welds and the stiffener is calculated in accordance with Section B4(a) with modifications as follows:

$w = e = 0.750 - 0.180 - 0.0598 = 0.510$ in.

$f = 33.0$ ksi (see previous calculations)

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/33.0} = 38.3$ (*Eq.* B4-7)

$w/t = 0.510/0.0598 = 8.53$

Since w/t = 8.53 < 0.328S = 12.6, the 0.510 in. segment of cover plate between outside welds and stiffener is fully effective and $\rho_t = 1.00$.

$$\rho_m = 8\left(\frac{F_y}{f}\right)\sqrt{\frac{tF_c}{df}} \le 1.0 \qquad (Eq.\ B2.5\text{-}5)$$

$$= 8\left(\frac{33.0}{33.0}\right)\sqrt{\frac{0.0598(9.64)}{7.6072(33.0)}} \le 1.0$$

$$= 0.383 \le 1.0 \quad \text{OK}$$

$$\rho = \rho_t\rho_m = 1.00(0.383) = 0.383 \qquad (Eq.\ B2.5\text{-}1)$$

The effective width of each 0.510 in. segment of cover plate between outside welds and stiffener is:

$$b = \rho w = 0.383(0.510) = 0.195 \text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

e) Determine the effective width of the cover plate stiffener

$d = 1.00 - 0.180 - 0.0598 = 0.760$ in.

$w_1 = 10.5$ in. (distance between welds)

For the edge stiffener, d_s and I_a are determined using w' and f' in lieu of w and f as follows:

$f = 33.0$ ksi (stress in cover plate)

$\rho_m = 0.383$ (from cover plate calculations)

$F_c = 9.64$ ksi (buckling stress of cover plate determined from calculations above)

$e = 0.750 - 0.180 - 0.0598 = 0.510$ in.

$w' = 2e + \min(0.75s, w_1) = 2(0.510) + \min[(0.75)(6), 10.5] = 5.52$ in. (*Eq.* B2.5-7)

$f' = \max(\rho_m f, F_c) = \max[0.383(33.0), 9.64] = 12.6$ ksi

The distance from the neutral axis to the top of the stiffener flat is calculated by:

$y_t = 4.30 - 0.0598 - 0.180 = 4.06$ in.

The distance from the neutral axis to the bottom of the stiffener is calculated by:

$y_b = 4.06 - 0.760 = 3.30$ in.

The stresses based on, f', at each end of the stiffener are:

$$f_1 = -12.6\left(\frac{4.06}{4.30}\right) = -11.9 \text{ ksi}$$

$$f_2 = -12.6\left(\frac{3.30}{4.30}\right) = -9.67 \text{ ksi}$$

Determine the plate buckling coefficient and buckling stress in accordance with Section B3.2(a)(1):

$$\psi = |f_2 / f_1| = |(-9.67)/(-11.9)| = 0.813 \qquad (Eq.\ \text{B3.2-1})$$

$$k = \frac{0.578}{\psi + 0.34} = \frac{0.578}{0.813 + 0.34} = 0.501 \qquad (Eq.\ \text{B3.2-2})$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ \text{B2.1-5})$$

$$= 0.501\frac{\pi^2 29500}{12(1-0.3^2)}\left(\frac{0.0598}{0.760}\right)^2 = 82.7\,\text{ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{11.9}{82.7}} = 0.379 \le 0.673 \qquad (Eq.\ \text{B2.1-4})$$

Because $\lambda \le 0.673$, $d'_s = d = 0.760$ in.

$$S = 1.28\sqrt{E/f'} = 1.28\sqrt{29500/12.6} = 61.9 \qquad (\text{From } Eq.\ \text{B4-7})$$

$$\frac{w'}{t} = \frac{5.52}{0.0598} = 92.3$$

Since $w'/t > 0.328S$, calculate the adequate moment of inertia so that each component will behave as a stiffened element:

$$I_a = 399t^4\left[\frac{w'/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w'/t}{S} + 5\right] \qquad (\text{From } Eq.\ \text{B4-8})$$

$$= 399(0.0598)^4\left[\frac{92.3}{61.9} - 0.328\right]^3 \le (0.0598)^4\left[115\left(\frac{92.3}{61.9}\right) + 5\right]$$

$$= 0.00803 > 0.00226$$

$$= 0.00226 \text{ in.}^4$$

$$I_s = \frac{d^3 t(\sin^2\theta)}{12} = \frac{(0.760)^3(0.0598)(\sin^2 90^\circ)}{12} = 0.00219 \text{ in.}^4 \qquad (Eq.\ \text{B4-10})$$

$$R_I = I_s / I_a \le 1 \qquad (Eq.\ \text{B4-9})$$

$$= 0.00219 / 0.00226 = 0.969 \le 1 \quad \text{OK}$$

The effective width of each cover plate stiffener is:

$$d_s = d'_s(R_I) = (0.760)(0.969) = 0.736 \text{ in.} \qquad (Eq.\ \text{B4-6})$$

The ineffective width of each cover plate stiffener is:

$$b_{neg} = 0.760 - 0.736 = 0.0240 \text{ in.}$$

$$y_{neg} = 1.000 - 0.0598/2 - 0.0240/2 = 0.958 \text{ in. from the centerline of the upper plate}$$

f) Determine the effective width of the deck stiffener segments

The beneficial effect of stiffening from the intermittent welds is conservatively not considered for these stiffener segments.

$$w = 1.125 - 0.180 = 0.945 \text{ in.}$$

$$f = 33.0\frac{4.30 - 0.0598}{4.30} = 32.5\,\text{ksi}$$

Calculate critical buckling stress per *Eq.* B2.1-5 and k = 0.43:

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 0.43\frac{\pi^2 29500}{12\left(1-0.3^2\right)}\left(\frac{0.0474}{0.945}\right)^2 = 28.8\,\text{ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{32.5}{28.8}} = 1.06 \qquad (Eq.\ B2.1\text{-}4)$$

$$\rho = \left(1-0.22/\lambda\right)/\lambda \le 1.0 \qquad (Eq.\ B2.1\text{-}3)$$

$$= \left(1.0-0.22/1.06\right)/1.06 \le 1.0$$

$$= 0.748 \le 1.0 \quad \text{OK}$$

The effective width of the 0.945 in. segments of deck plate is:

$$b = \rho w = 0.748\left(0.945\right) = 0.707 \text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

The ineffective width of the 0.945 in. segments of deck plate is:

$$b_{neg} = 0.945 - 0.707 = 0.238 \text{ in.}$$

g) Check if webs are fully effective (Section B2.3)

Top of web

$$y_{top} = 4.30 - 0.0598 - 0.0474 - 0.18 = 4.01\,\text{in.}$$

$$f_1 = -33.0\frac{4.01}{4.30} = -30.8\,\text{ksi}$$

Bottom of web

$$y_{bottom} = 3.31 - 0.0474 - 0.18 = 3.08\,\text{in.}$$

$$f_2 = 33.0\frac{3.08}{4.30} = 23.6\,\text{ksi}$$

$$\psi = \left|f_2/f_1\right| = \left|23.6/\left(-30.8\right)\right| = 0.766 \qquad (Eq.\ B2.3\text{-}1)$$

$$k = 4 + 2\left(1+\psi\right)^3 + 2\left(1+\psi\right) \qquad (Eq.\ B2.3\text{-}2)$$

$$= 4 + 2\left(1+0.766\right)^3 + 2\left(1+0.766\right) = 18.5$$

$$w = 7.6072 - 0.0598 - 2\left(0.0474 + 0.180\right) = 7.09\,\text{in.}$$

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 18.5\frac{\pi^2 29500}{12\left(1-0.3^2\right)}\left(\frac{0.0474}{7.09}\right)^2 = 22.0\,\text{ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{30.8}{22.0}} = 1.18 > 0.673 \qquad (Eq.\ B2.1\text{-}4)$$

$$\rho = \left(1-0.22/\lambda\right)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$

$$= \left(1-0.22/1.18\right)/1.18 = 0.689$$

$$b_e = b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$= 0.689(7.09) = 4.89\,\text{in.}$

For the webs, $h_o/b_o = 7.55/1.17 = 6.45 > 4$; therefore,

$b_1 \quad = b_e/(3+\psi)$ (*Eq.* B2.3-6)

$= 4.89/(3+0.766) = 1.30\,\text{in.}$

$b_2 \quad = b_e/(1+\psi) - b_1$ (*Eq.* B2.3-7)

$= 4.89/(1+0.766) - 1.30 = 1.47\,\text{in.}$

$b_1 + b_2 = 1.30 + 1.47 = 2.77\,\text{in.}$

Compression width of web = 4.01 in. > 2.77 in.; therefore, web is not fully effective

Negative portion of web

$b_{neg} = 4.01 - 2.77 = 1.24\,\text{in.}$

$y_{neg} = \frac{0.0598}{2} + 0.0474 + 0.18 + b_1 + 0.5b_{neg} = \frac{0.0598}{2} + 0.0474 + 0.18 + 1.30 + 0.5(1.24) = 2.18\,\text{in.}$ from the centerline of the upper plate.

h) Calculate effective section properties

The total effective width of the top flat cover plate segments is calculated by:

$\sum b = 2.09(2) + 0.575 + 0.195(2) = 5.15\,\text{in.}$

The total ineffective width of the top flat cover plate segments is calculated by:

$b_{neg} = 23.5 - 5.15 = 18.4$ in.

The net section properties are calculated by subtracting ineffective areas from the gross section calculation.

Effective Section Properties

Item Units	L in.	n	y† in.	t in.	I′₁ in.⁴	nA in.²	nAy in.³	nAy² in.⁴	nI′₁ in.⁴
Gross Properties						3.9923	11.7618	69.1048	5.6427
Ineffective Top Cover Plate Area	-18.4	1	0.0000	0.0598	0.0000	-1.10	0.000	0.000	0.0000
Ineffective Top Cover Plate Stiffener Area	-0.0240	2	0.958	0.0598	0.0000	-0.00287	-0.00275	-0.00263	0.0000
Ineffective Deck Top Stiffeners	-0.238	4	0.0536	0.0474	0.0000	-0.0451	-0.00242	-0.0001	0.0000
Ineffective Web Area	-1.24	4	2.18	0.0474	-0.0075	-0.235	-0.512	-1.12	-0.0300
					$\sum$	2.61	11.2	68.0	5.61
							$\sum$	73.6	

†Distance from centerline of top cover plate to center of gravity of element under consideration.

$y_{cg} = \sum Ay / \sum A = 11.2/2.61 = 4.29\,\text{in.}$

$y_t = y_{cg} + 0.5t = 4.29 + 0.5(0.0598) = 4.32\ \text{in.}$

Note that the trial value of $y_t = 4.30$ in. is approximately equal to the calculated value above. Further iteration is judged to be unnecessary; however, additional iteration should be conducted if a more refined solution is warranted for the application under consideration.

$y_b = D - y_t = 7.6072 - 4.32 = 3.29\,\text{in.}$

$I_{xe} = \sum Ay^2 + \sum I_1' - \bar{y}^2 \sum A = 68.0 + 5.61 - (4.29)^2\, 2.61 = 25.6\,\text{in.}^4 = 12.8\,\text{in.}^4\ /\ \text{ft}$

$S_{xe,top} = I_{xe}/y_t = 25.6/4.32 = 5.93\,\text{in.}^3 = 2.97\,\text{in.}^3\ /\ \text{ft}$

$S_{xe,bot} = I_{xe}/y_b = 25.6/3.29 = 7.78\,\text{in.}^3 = 3.89\ \text{in.}^3\ /\ \text{ft}$

$M_n = F_y S_e = 33(2.97) = 98.0\ \text{kip-in./ft}$ (*Eq.* C3.1.1-1)

$\Omega = 1.67$

$\dfrac{M_n}{\Omega} = 98.0\ /\ 1.67 = 58.7\ \text{kip-in./ft}$

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TABLE OF CONTENTS

PART II
BEAM DESIGN

For Use With the
2012 Edition of the
North American Specification for the Design
of Cold-Formed Steel Structural Members

PART II - BEAM DESIGN

Strength design of cold-formed steel beams requires the consideration of the limit states of:

1. Flexural yielding and local buckling
2. Lateral-torsional buckling
3. Distortional buckling
4. Web shear yielding or buckling, including interaction with flexure
5. Web crippling, including interaction with flexure
6. Combined flexure and torsion

Specification Section C3 includes provisions for the evaluation of these limit states. For beams that are parts of certain structural systems, Section D6 provides specific provisions for those members. In addition to these limit states, consideration of bracing and anchorage forces is necessary in some cases. For shear lag effects, see *Specification* Section B1.1(c).

Yielding and Local Buckling: The strength of all flexural members is limited by the combined limit state of yielding and local buckling, which is evaluated using Section C3.1.1(a). Yielding strength is calculated using the effective section modulus, S_e. For members in which all elements are fully effective, additional strength can be calculated considering the effects of the cold work of forming using Section A7.2. For flexural members not subject to twisting or lateral, torsional or flexural-torsional buckling, the strength may alternatively be calculated based on the inelastic reserve capacity using Section C3.1.1(b). See *Manual* Section 3.6 and Examples I-8 through I-10 and I-12 through I-15 in Part I for further information on the calculation of effective section properties and the yielding strength of flexural members.

Lateral-Torsional Buckling: General provisions for calculating the lateral-torsional buckling strength of unbraced or discretely braced members are found in *Specification* Section C3.1.2. Although the specifics vary somewhat for different cross-section shapes, the general procedure involves 1) determination of the elastic lateral-torsional buckling stress, 2) transformation of the elastic buckling stress to a critical buckling stress, taking into account the effects of inelasticity and 3) determination of the effective section modulus with the extreme compression fiber at the critical buckling stress. Section D6 of the *Specification* provides specialized provisions for the lateral-torsional buckling of flexural members that are elements of metal roof and wall systems, including through-fastened purlins and girts, and standing seam roofs.

Distortional Buckling: The distortional buckling limit state involves the cross-sectional deformations of two or more elements acting as a group, e.g., the rotation of the flange and lip of a C-Section about the web-to-flange junction. The *Specification* and *Commentary* provide three levels of provisions for this limit state in Section C3.1.4. *Commentary* Section C3.1.4, *Simplified Method for Unrestrained C- and Z- Sections With Simple Lips*, requires a simple calculation using basic cross-section dimensions and produces a conservative, and sometimes very conservative, result. This approach can sometimes be used to quickly determine that distortional buckling is not a controlling limit state. For those cases where the extra work is justified, *Specification* Section C3.1.4(a) can be used, which requires considerably more complex calculations, but produces accurate results. *Specification* Section C3.1.4(b) provides a framework for the use of computerized numerical methods to evaluate distortional buckling. This approach requires fewer calculations than Section C3.1.4(a) and is especially useful for cross-sections that do not meet the limits of applicability of the other two approaches. For all three approaches, the general procedure involves 1) determination of the elastic distortional buckling stress, 2) determination of the corresponding elastic buckling moment using the gross section modulus of the cross-section and 3) transformation of the elastic buckling moment to a nominal flexural strength, taking into account the effects of inelasticity and post-buckling strength.

Shear Strength: The shear limit state checks are similar to those for hot-rolled shapes. Depending upon the slenderness of the web, the shear strength may be limited by yielding, inelastic buckling or

elastic buckling. Section C3.2 contains provisions for the shear strength of webs with and without holes. The combined limit state of flexural yielding and shear is evaluated according to the provisions of Section C3.3.

Web Crippling: Web crippling of cold-formed members is a localized buckling and yielding of the web in the vicinity of a concentrated load. The web crippling strength is a function of the load bearing length, web thickness and height, bend radius, yield strength and member geometry. Strengths are determined by Section C3.4.1 with the use of empirically derived coefficients provided in a series of tables. For C-Sections with web holes in the vicinity of the concentrated load, a strength reduction factor must be applied using Section C3.4.2. The combined limit state of flexure and web crippling is evaluated using Section C3.5.

Torsion: Flexural members not loaded through the shear center generate torsional stresses in the cross-section unless they are braced against torsion. Section C3.6 contains provisions for reducing the flexural strength due to torsional warping. Although C-Sections loaded in their strong axis through the flange would be expected to exhibit torsion, it is not considered necessary to apply these provisions to C-Sections used in metal roof and wall systems when determining system strength using the provisions of Section D6.

SECTION 1 - BENDING

1.1 Notes on the Tables

(a) With the exception of the joists/studs and tracks, the specific sections listed in these tables are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members.

(b) The section designations listed in these tables correspond to those for which dimensions and properties are given in Tables I-1 to I-5 and I-8.

(c) The effective section modulus values are calculated as the effective moment of inertia at the indicated stress level divided by the distance to the controlling extreme fiber. In calculating the nominal strength of these sections, additional checks, such as the provisions of Chapter C of the *Specification,* should also be taken into account where applicable.

(d) Tabulated section properties are shown to three significant figures. Dimensions are given to three decimal places.

(e) Where they apply, the algebraic formulae presented in Section 3 of Part I formed the basis of the calculations for these tables.

(f) The effective section properties listed in Tables II-1 through II-6 were computed using the yield stresses listed in the tables, except where a value of F_{ya} is given in Table II-2, the provisions of Section A7 of the *Specification* for strength increase from cold work of forming have been used. Sections were considered eligible for the cold work of forming increase in yield stress if ρ=1.0 for each flat element, except that webs may have ρ<1.0 if the sum of b_1 plus b_2 from Section B2.3 equals or exceeds the width of the compression portion of the web.

(g) The values labeled I_e in Tables II-1 through II-6 are effective moments of inertia calculated at nominal moments that are 60 percent of M_{nxo}. They represent lower bound values of I_x for use in estimating deflections at ordinary service loads.

(h) The effects of standard factory punchouts in joists/studs have been included in Table II-2. These punchouts are considered in joists and studs with flange widths of 1.625 in. or less. Standard punchout sizes are 1.5 in. by 4.5 in. for sections with depths of 3.5 inches or more and 0.75 in. by 4.5 in. for sections with shallower depths. Punchouts are not included in the

calculations for sections with web h/t ratios in excess of 200 due to the limits in Section B2.4. Those sections are marked with a "*".

(i) Dashes in the place of data values in the F_{ya} columns of Table II-2 indicate that the section is not eligible for strength increase due to the effects of cold work of forming for the listed yield stress

(j) In Tables II-2 and II-3, strengths of standard joists/studs and tracks are given for the steel grades in which they are readily available. Grade 50 joists/studs and tracks are shown in bold with a shaded background.

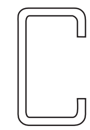
1.2 Beam Property Tables

Table II – 1

F_y = 55 ksi

Beam Properties[2]
C-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)
ϕ_v = 0.80 (LSD)



Section	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}
12CS4x105	14.4	368	6.70	46.3
12CS4x085	7.63	264	4.79	36.5
12CS4x070	4.25	189	3.44	29.6
12CS3.5x105	14.4	358	6.51	43.4
12CS3.5x085	7.63	261	4.74	34.9
12CS3.5x070	4.25	188	3.42	28.2
12CS2.5x105	14.4	309	5.62	36.0
12CS2.5x085	7.63	232	4.22	28.8
12CS2.5x070	4.25	169	3.07	23.4
10CS4x105	17.5	286	5.19	30.3
10CS4x085	9.25	220	4.01	23.5
10CS4x070	5.15	158	2.87	19.1
10CS4x065	4.12	140	2.54	17.7
10CS3.5x105	17.5	277	5.04	28.2
10CS3.5x085	9.25	214	3.89	22.6
10CS3.5x070	5.15	158	2.86	18.1
10CS3.5x065	4.12	139	2.53	16.7
10CS2.5x105	17.5	256	4.66	23.3
10CS2.5x085	9.25	204	3.70	19.0
10CS2.5x070	5.15	153	2.78	15.7
10CS2.5x065	4.12	134	2.43	14.6
10CS2x105	17.5	221	4.02	20.7
10CS2x085	9.25	172	3.13	16.7
10CS2x070	5.15	136	2.47	13.6
10CS2x065	4.12	124	2.26	12.5
9CS2.5x105	19.5	221	4.02	18.1
9CS2.5x085	10.3	176	3.19	14.7
9CS2.5x070	5.76	136	2.47	12.2
9CS2.5x065	4.60	121	2.21	11.3
9CS2.5x059	3.44	104	1.89	10.3
8CS4x105	19.5	211	3.83	18.1
8CS4x085	11.7	162	2.95	14.1
8CS4x070	6.52	127	2.31	11.3
8CS4x065	5.22	112	2.03	10.4
8CS4x059	3.90	95.1	1.73	9.41
8CS3.5x105	19.5	205	3.72	16.8
8CS3.5x085	11.7	157	2.86	13.5
8CS3.5x070	6.52	124	2.26	10.7
8CS3.5x065	5.22	112	2.04	9.84
8CS3.5x059	3.90	95.4	1.74	8.90
8CS2.5x105	19.5	188	3.41	13.6
8CS2.5x085	11.7	149	2.71	11.1
8CS2.5x070	6.52	115	2.10	9.21
8CS2.5x065	5.22	105	1.91	8.57
8CS2.5x059	3.90	93.0	1.69	7.79

Section	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}
8CS2x105	19.5	165	3.00	12.0
8CS2x085	11.7	135	2.45	9.79
8CS2x070	6.52	111	2.03	8.11
8CS2x065	5.22	104	1.89	7.54
8CS2x059	3.90	93.3	1.70	6.86
7CS4x105	19.5	176	3.21	13.4
7CS4x085	12.8	135	2.46	10.4
7CS4x070	7.53	106	1.93	8.32
7CS4x065	6.02	96.7	1.76	7.64
7CS4x059	4.49	83.1	1.51	6.88
7CS2.5x105	19.5	156	2.84	9.94
7CS2.5x085	12.8	124	2.26	8.11
7CS2.5x070	7.53	95.8	1.74	6.72
7CS2.5x065	6.02	87.3	1.59	6.25
7CS2.5x059	4.49	78.4	1.42	5.69
6CS4x105	18.8	144	2.62	9.44
6CS4x085	12.8	110	2.00	7.34
6CS4x070	8.65	86.1	1.57	5.87
6CS4x065	7.12	78.5	1.43	5.40
6CS4x059	5.31	69.5	1.26	4.83
6CS2.5x105	18.8	127	2.30	6.91
6CS2.5x085	12.8	101	1.83	5.65
6CS2.5x070	8.65	77.7	1.41	4.69
6CS2.5x065	7.12	70.7	1.29	4.36
6CS2.5x059	5.31	63.4	1.15	3.97
4CS4x105	11.8	85.7	1.56	3.81
4CS4x085	9.69	65.2	1.19	2.98
4CS4x070	8.05	50.6	0.921	2.38
4CS4x065	7.46	46.0	0.837	2.19
4CS4x059	6.15	40.6	0.739	1.95
4CS2.5x105	11.8	73.5	1.34	2.67
4CS2.5x085	9.69	59.1	1.08	2.20
4CS2.5x070	8.05	45.4	0.825	1.83
4CS2.5x065	7.46	41.2	0.750	1.71
4CS2.5x059	6.15	36.9	0.671	1.56
4CS2x105	11.8	63.2	1.15	2.30
4CS2x085	9.69	52.1	0.947	1.89
4CS2x070	8.05	43.5	0.791	1.58
4CS2x065	7.46	40.4	0.734	1.47
4CS2x059	6.15	36.1	0.656	1.35

Notes:

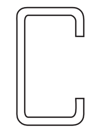
1. Shear and moment strengths given are nominal strengths [resistances]. To obtain available strengths [factored resistances], these values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD).
2. The distortional buckling limit state is not considered in this table. Consideration of distortional buckling may result in lower strengths when restraint against distortional buckling is not provided. See Table II-7 for distortional buckling strengths.

Table II – 2

F_y = 33 ksi, F_u = 45 ksi
F_y = 50 ksi, F_u = 65 ksi

Beam Properties[3]
Joists/Studs
C-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)
ϕ_v = 0.80 (LSD)



Section	V_n [1] kips	F_{ya} [2] ksi	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}
1200S250-97	13.0	-	252	5.04	33.8
1200S250-68	4.43	-	150	3.01	23.6
1200S250-54*	2.20	-	107	2.15	18.4
1200S200-97	13.0	-	233	4.66	30.2
1200S200-68	4.43	-	148	2.96	20.9
1200S200-54*	2.20	-	104	2.07	16.3
1200S162-97	11.9	-	205	4.09	26.7
1200S162-68	4.43	-	132	2.64	18.4
1200S162-54*	2.20	-	95.7	1.91	14.3
1000S250-97	15.8	56.2	235	4.18	21.8
1000S250-68	5.35	-	138	2.77	15.7
1000S250-54	2.66	-	94.0	1.88	12.7
1000S250-43*	1.34	-	53.4	1.62	10.2
1000S200-97	15.8	-	187	3.74	19.3
1000S200-68	5.35	-	121	2.42	13.7
1000S200-54	2.66	-	85.3	1.70	10.8
1000S200-43*	1.34	-	48.5	1.47	8.60
1000S162-97	11.5	-	163	3.27	17.0
1000S162-68	5.35	-	108	2.15	12.0
1000S162-54	2.66	-	78.6	1.57	9.39
1000S162-43*	1.34	-	43.0	1.30	7.52
800S250-97	17.4	56.2	172	3.05	12.8
800S250-68	6.75	-	103	2.06	9.24
800S250-54	3.34	-	76.3	1.52	7.38
800S250-43	1.68	-	43.3	1.31	6.02
800S200-97	17.4	57.6	161	2.80	11.2
800S200-68	6.75	55.4	109	1.96	8.14
800S200-54	3.34	-	74.9	1.50	6.57
800S200-43	1.68	-	42.7	1.29	5.30
800S200-33*	0.758	-	26.9	0.816	4.10
800S162-97	9.50	-	121	2.43	9.71
800S162-68	5.39	-	83.2	1.66	7.07
800S162-54	3.34	-	61.4	1.23	5.60
800S162-43	1.68	-	33.6	1.02	4.50
800S162-33*	0.758	-	23.4	0.710	3.38
800S137-97	9.50	-	107	2.15	8.60
800S137-68	5.39	-	73.4	1.47	6.28
800S137-54	3.34	-	54.2	1.08	4.97
800S137-43	1.68	-	29.6	0.896	4.00
800S137-33*	0.758	-	20.5	0.622	3.00
600S250-97	16.8	56.2	116	2.06	6.50
600S250-68	8.56	-	69.3	1.39	4.72
600S250-54	4.52	-	53.4	1.07	3.76
600S250-43	2.26	-	30.3	0.918	3.08

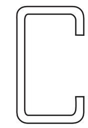
Section	V_n [1] kips	F_{ya} [2] ksi	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}
600S200-97	16.8	57.6	108	1.87	5.61
600S200-68	8.56	55.4	73.0	1.32	4.10
600S200-54	4.52	-	50.8	1.01	3.32
600S200-43	2.26	-	28.8	0.873	2.68
600S200-33	1.02	-	20.5	0.621	2.06
600S162-97	6.09	59.2	94.7	1.60	4.80
600S162-68	4.61	56.6	65.9	1.16	3.52
600S162-54	3.12	55.3	50.6	0.916	2.86
600S162-43	1.98	36.3	27.9	0.767	2.32
600S162-33	1.02	-	19.1	0.577	1.79
600S137-97	6.09	60.8	84.8	1.40	4.19
600S137-68	4.61	-	51.5	1.03	3.09
600S137-54	3.12	-	38.8	0.777	2.52
600S137-43	1.98	-	21.3	0.645	2.04
600S137-33	1.02	-	15.0	0.455	1.55
550S162-68	4.05	56.6	58.4	1.03	2.86
550S162-54	3.01	55.3	44.9	0.811	2.32
550S162-43	1.92	36.3	24.7	0.681	1.88
550S162-33	1.12	-	16.9	0.512	1.46
400S200-68	7.79	55.4	42.4	0.766	1.59
400S200-54	5.39	-	29.4	0.589	1.29
400S200-43	2.78	-	16.8	0.509	1.05
400S200-33	1.56	-	12.0	0.365	0.804
400S162-68	2.17	-	32.4	0.648	1.32
400S162-54	1.96	-	24.9	0.498	1.07
400S162-43	1.30	-	13.7	0.417	0.870
400S162-33	0.952	-	9.86	0.299	0.669
400S137-68	2.17	-	27.9	0.558	1.14
400S137-54	1.96	-	21.4	0.428	0.929
400S137-43	1.30	-	11.8	0.359	0.754
400S137-33	0.952	-	8.54	0.259	0.580
362S200-68	6.99	55.4	37.3	0.673	1.26
362S200-54	5.39	-	25.9	0.517	1.03
362S200-43	2.78	-	14.8	0.448	0.836
362S200-33	1.64	-	10.6	0.321	0.642
362S162-68	1.61	-	28.7	0.574	1.05
362S162-54	1.63	-	22.2	0.444	0.854
362S162-43	1.08	-	12.3	0.372	0.694
362S162-33	0.834	-	8.84	0.268	0.535
362S137-68	1.61	-	24.7	0.493	0.902
362S137-54	1.63	-	19.1	0.381	0.737
362S137-43	1.08	-	10.6	0.320	0.600
362S137-33	0.834	-	7.66	0.232	0.462

Table II – 2

F_y = 33 ksi, F_u = 45 ksi
F_y = 50 ksi, F_u = 65 ksi

Beam Properties³
Joists/Studs
C-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)
ϕ_v = 0.80 (LSD)



Section	V_n ¹ kips	F_{ya} ² ksi	M_{nxo} ¹ kip-in.	S_e in.³	I_e in.⁴	Section	V_n ¹ kips	F_{ya} ² ksi	M_{nxo} ¹ kip-in.	S_e in.³	I_e in.⁴
350S162-68	1.44	-	27.5	0.549	0.965	250S137-68	0.830	57.8	17.8	0.308	0.386
350S162-54	1.52	-	21.3	0.426	0.787	250S137-54	0.903	56.2	13.7	0.244	0.318
350S162-43	1.01	-	11.8	0.357	0.640	250S137-43	0.631	36.9	7.56	0.205	0.261
350S162-33	0.779	-	8.49	0.257	0.494	250S137-33	0.638	-	5.20	0.158	0.203
250S162-68	0.830	56.6	20.2	0.357	0.450						
250S162-54	0.903	55.3	15.7	0.284	0.370						
250S162-43	0.631	36.3	8.72	0.240	0.302						
250S162-33	0.638	-	5.94	0.180	0.235						

Notes:

* Web h/t > 200, therefore bearing stiffeners are required.

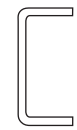
1. Shear and moment strengths given are nominal strengths [resistances]. To obtain available strengths [factored resistances], these values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD).
2. Where values are given for F_{ya}, flexural strength is calculated using the strength increase from cold working, based on an average yield stress of F_{ya}.
3. The distortional buckling limit state is not considered in this table. Consideration of distortional buckling may result in lower strengths when restraint against distortional buckling is not provided. See Table II-8 for distortional buckling strengths.

Table II – 3

Beam Properties[2]
Tracks
C-Sections Without Lips

F_y = 33 ksi, F_u = 45 ksi
F_y = 50 ksi, F_u = 65 ksi

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)
ϕ_v = 0.80 (LSD)



Section	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}
1200T200-97	12.6	191	3.82	29.0
1200T200-68	4.34	103	2.06	18.0
1200T200-54*	2.17	67.5	1.35	13.0
1200T150-97	12.6	181	3.62	25.7
1200T150-68	4.34	99.3	1.99	16.6
1200T150-54*	2.17	65.7	1.31	12.0
1200T125-97	12.6	172	3.44	23.7
1200T125-68	4.34	96.7	1.93	15.7
1200T125-54*	2.17	64.3	1.29	11.5
1000T200-97	15.2	154	3.08	18.6
1000T200-68	5.22	84.2	1.68	11.8
1000T200-54	2.60	55.6	1.11	8.56
1000T200-43*	1.32	28.4	0.861	6.72
1000T150-97	15.2	145	2.90	16.4
1000T150-68	5.22	81.0	1.62	10.8
1000T150-54	2.60	53.9	1.08	7.88
1000T150-43*	1.32	27.6	0.837	6.19
1000T125-97	15.2	138	2.75	15.1
1000T125-68	5.22	78.7	1.58	10.2
1000T125-54	2.60	52.8	1.06	7.48
1000T125-43*	1.32	27.0	0.819	5.88
800T200-97	17.4	117	2.35	10.8
800T200-68	6.54	65.5	1.31	7.05
800T200-54	3.26	43.6	0.872	5.15
800T200-43	1.65	22.3	0.676	4.04
800T200-33*	0.744	14.0	0.424	2.79
800T150-97	17.4	110	2.19	9.48
800T150-68	6.54	62.8	1.25	6.36
800T150-54	3.26	42.2	0.844	4.69
800T150-43	1.65	21.6	0.655	3.69
800T150-33*	0.744	13.7	0.414	2.57
800T125-97	17.4	103	2.06	8.61
800T125-68	6.54	60.8	1.22	5.96
800T125-54	3.26	41.2	0.824	4.42
800T125-43	1.65	21.1	0.640	3.48
800T125-33*	0.744	13.4	0.407	2.44
600T200-97	17.4	78.4	1.57	5.56
600T200-68	8.56	48.6	0.973	3.54
600T200-54	4.36	35.9	0.717	2.64
600T200-43	2.20	18.6	0.565	2.08
600T200-33	0.996	11.0	0.333	1.54

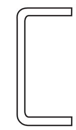
Section	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}
600T150-97	17.4	72.2	1.44	4.78
600T150-68	8.56	44.6	0.891	3.16
600T150-54	4.36	30.5	0.609	2.40
600T150-43	2.20	15.6	0.474	1.89
600T150-33	0.996	10.0	0.303	1.33
600T150-30	0.730	8.36	0.253	1.16
600T150-27*	0.545	7.06	0.214	1.01
600T125-97	17.4	67.4	1.35	4.28
600T125-68	8.56	42.9	0.858	2.97
600T125-54	4.36	29.6	0.592	2.24
600T125-43	2.20	15.2	0.461	1.77
600T125-33	0.996	9.80	0.297	1.26
600T125-30	0.730	8.22	0.249	1.09
600T125-27*	0.545	6.95	0.210	0.958
550T200-68	8.56	42.9	0.857	2.89
550T200-54	4.77	31.5	0.630	2.15
550T200-43	2.41	16.3	0.495	1.69
550T200-33	1.09	10.1	0.307	1.25
550T150-68	8.56	40.2	0.804	2.57
550T150-54	4.77	29.7	0.595	1.93
550T150-43	2.41	15.4	0.468	1.52
550T150-33	1.09	10.2	0.310	1.11
550T150-30	0.798	8.29	0.251	0.994
550T150-27	0.595	6.85	0.208	0.892
550T125-68	8.56	38.4	0.769	2.38
550T125-54	4.77	26.7	0.535	1.81
550T125-43	2.41	13.7	0.416	1.43
550T125-33	1.09	8.90	0.270	1.03
550T125-30	0.798	7.47	0.226	0.897
550T125-27	0.595	6.32	0.192	0.786
400T200-68	8.33	27.4	0.549	1.41
400T200-54	5.39	19.8	0.397	1.04
400T200-43	2.78	10.3	0.311	0.810
400T200-33	1.50	7.25	0.220	0.581
400T150-68	8.33	25.6	0.513	1.24
400T150-54	5.39	18.7	0.374	0.918
400T150-43	2.78	9.68	0.293	0.719
400T150-33	1.50	6.88	0.208	0.519
400T150-30	1.10	6.03	0.183	0.458
400T150-27	0.823	5.08	0.154	0.409

Table II – 3

F_y = 33 ksi, F_u = 45 ksi
F_y = 50 ksi, F_u = 65 ksi

Beam Properties[2]
Tracks
C-Sections Without Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)
ϕ_v = 0.80 (LSD)



Section	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.[3]	I_e in.[4]	Section	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.[3]	I_e in.[4]
400T125-68	8.33	24.4	0.488	1.13	350T125-68	7.26	20.4	0.407	0.839
400T125-54	5.39	17.9	0.359	0.849	350T125-54	5.39	14.9	0.297	0.626
400T125-43	2.78	9.30	0.282	0.666	350T125-43	2.78	7.69	0.233	0.490
400T125-33	1.50	6.63	0.201	0.484	350T125-33	1.64	5.46	0.165	0.354
400T125-30	1.10	5.82	0.176	0.427	350T125-30	1.26	4.78	0.145	0.312
400T125-27	0.823	5.14	0.156	0.380	350T125-27	0.943	4.22	0.128	0.277
400T125-18*	0.242	2.32	0.0701	0.241	350T125-18	0.277	2.03	0.0615	0.174
362T200-68	7.52	24.0	0.480	1.14	250T200-68	5.12	14.8	0.296	0.517
362T200-54	5.39	17.3	0.345	0.832	250T200-54	4.10	10.4	0.209	0.371
362T200-43	2.78	8.92	0.270	0.649	250T200-43	2.17	5.37	0.163	0.288
362T200-33	1.64	6.28	0.190	0.464	250T200-33	1.64	3.71	0.112	0.203
362T150-68	7.52	22.4	0.449	0.993	250T150-68	5.12	13.8	0.276	0.445
362T150-54	5.39	16.3	0.325	0.734	250T150-54	4.10	9.84	0.197	0.324
362T150-43	2.78	8.42	0.255	0.574	250T150-43	2.17	5.07	0.154	0.252
362T150-33	1.64	5.95	0.180	0.414	250T150-33	1.64	3.52	0.107	0.180
362T150-30	1.22	5.21	0.158	0.364	250T150-30	1.33	3.06	0.0929	0.157
362T150-27	0.910	4.60	0.140	0.323	250T150-27	1.10	2.69	0.0815	0.139
362T125-68	7.52	21.3	0.427	0.907	250T125-68	5.12	13.1	0.262	0.402
362T125-54	5.39	15.6	0.312	0.678	250T125-54	4.10	9.42	0.188	0.297
362T125-43	2.78	8.08	0.245	0.531	250T125-43	2.17	4.86	0.147	0.231
362T125-33	1.64	5.74	0.174	0.384	250T125-33	1.64	3.40	0.103	0.166
362T125-30	1.22	5.03	0.152	0.339	250T125-30	1.33	2.96	0.0896	0.145
362T125-27	0.910	4.45	0.135	0.301	250T125-27	1.10	2.60	0.0788	0.129
362T125-18	0.267	2.10	0.0637	0.189	250T125-18	0.392	1.46	0.0443	0.0778
350T200-68	7.26	22.9	0.458	1.05	162T125-33	1.06	1.92	0.0583	0.0656
350T200-54	5.39	16.4	0.329	0.770	162T125-30	0.956	1.66	0.0504	0.0574
350T200-43	2.78	8.49	0.257	0.600	162T125-27	0.866	1.45	0.0440	0.0505
350T200-33	1.64	5.97	0.181	0.428	162T125-18	0.484	0.831	0.0252	0.0297
350T150-68	7.26	21.4	0.428	0.919					
350T150-54	5.39	15.5	0.310	0.679					
350T150-43	2.78	8.01	0.243	0.530					
350T150-33	1.64	5.66	0.172	0.382					
350T150-30	1.26	4.95	0.150	0.336					
350T150-27	0.943	4.37	0.132	0.298					

Notes:

* Web h/t > 200, therefore bearing stiffeners are required.

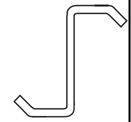
1. Shear and moment strengths given are nominal strengths [resistances]. To obtain available strengths [factored resistances], these values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD).
2. C-Sections Without Lips are not subject to distortional buckling.

Table II – 4

F_y = 55 ksi

Beam Properties[2]
Z-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)
ϕ_v = 0.80 (LSD)



Section	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	Section	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}
12ZS3.25x105	14.4	364	6.61	43.7	8ZS3.25x105	19.5	208	3.79	16.9
12ZS3.25x085	7.63	266	4.84	35.5	8ZS3.25x085	11.7	160	2.91	13.8
12ZS3.25x070	4.25	195	3.54	29.2	8ZS3.25x070	6.52	128	2.33	10.9
12ZS2.75x105	14.4	339	6.16	39.8	8ZS3.25x065	5.22	117	2.13	10.1
12ZS2.75x085	7.63	241	4.38	31.8	8ZS3.25x059	3.90	101	1.83	9.25
12ZS2.75x070	4.25	180	3.28	25.9	8ZS2.75x105	19.5	208	3.78	15.3
12ZS2.25x105	14.4	309	5.62	36.0	8ZS2.75x085	11.7	156	2.84	12.4
12ZS2.25x085	7.63	239	4.35	28.7	8ZS2.75x070	6.52	124	2.25	10.3
12ZS2.25x070	4.25	184	3.34	23.2	8ZS2.75x065	5.22	114	2.07	9.56
10ZS3.25x105	17.5	282	5.13	28.4	8ZS2.75x059	3.90	100	1.82	8.62
10ZS3.25x085	9.25	217	3.95	23.1	8ZS2.25x105	19.5	187	3.41	13.6
10ZS3.25x070	5.15	163	2.97	18.7	8ZS2.25x085	11.7	153	2.77	11.1
10ZS3.25x065	4.12	145	2.64	17.2	8ZS2.25x070	6.52	124	2.25	9.18
10ZS3.25x059	3.08	125	2.28	15.7	8ZS2.25x065	5.22	110	2.01	8.54
10ZS2.75x105	17.5	282	5.12	25.9	8ZS2.25x059	3.90	98.8	1.80	7.76
10ZS2.75x085	9.25	213	3.87	21.0	7ZS2.25x105	19.5	156	2.83	9.92
10ZS2.75x070	5.15	161	2.93	17.4	7ZS2.25x085	12.8	127	2.31	8.09
10ZS2.75x065	4.12	144	2.61	16.2	7ZS2.25x070	7.53	103	1.87	6.70
10ZS2.75x059	3.08	124	2.26	14.6	7ZS2.25x065	6.02	91.8	1.67	6.23
10ZS2.25x105	17.5	249	4.53	23.3	7ZS2.25x059	4.49	82.2	1.49	5.67
10ZS2.25x085	9.25	194	3.53	18.8	6ZS2.25x105	18.8	126	2.30	6.89
10ZS2.25x070	5.15	150	2.73	15.3	6ZS2.25x085	12.8	103	1.88	5.63
10ZS2.25x065	4.12	131	2.38	14.1	6ZS2.25x070	8.65	83.7	1.52	4.66
10ZS2.25x059	3.08	114	2.08	12.8	6ZS2.25x065	7.12	74.5	1.36	4.34
9ZS2.25x105	19.5	221	4.01	18.1	6ZS2.25x059	5.31	66.6	1.21	3.95
9ZS2.25x085	10.3	180	3.27	14.7	4ZS2.25x070	8.05	49.0	0.892	1.82
9ZS2.25x070	5.76	145	2.64	12.2	4ZS2.25x065	7.46	43.6	0.794	1.70
9ZS2.25x065	4.60	130	2.36	11.3	4ZS2.25x059	6.15	39.0	0.708	1.55
9ZS2.25x059	3.44	111	2.01	10.3	3.5ZS1.5x070	6.90	31.0	0.563	0.985
					3.5ZS1.5x065	6.42	28.9	0.525	0.918
					3.5ZS1.5x059	5.86	26.3	0.479	0.838

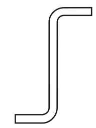
Notes:

1. Shear and moment strengths given are nominal strengths [resistances]. To obtain available strengths [factored resistances], these values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD).
2. The distortional buckling limit state is not considered in this table. Consideration of distortional buckling may result in lower strengths when restraint against distortional buckling is not provided. See Table II-9 for distortional buckling strengths.

Table II - 5

Beam Properties[3]
Z-Sections Without Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)
ϕ_v = 0.80 (LSD)



Section	F_y = 33 ksi				F_y = 50 ksi				Maximum Fully-Effective Moment [2]	
	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.
8ZU1.25x105	15.1	65.0	1.97	7.88	18.6	98.5	1.97	7.88	124	123
8ZU1.25x090	11.1	56.2	1.70	6.82	13.6	83.6	1.67	6.82	77.4	75.9
8ZU1.25x075	7.70	46.7	1.41	5.73	8.04	64.5	1.29	5.73	44.6	43.0
8ZU1.25x060	4.10	33.9	1.03	4.63	4.10	46.0	0.919	4.49	22.7	21.5
8ZU1.25x048	2.09	24.1	0.730	3.59	2.09	32.2	0.644	3.40	11.6	10.9
6ZU1.25x105	11.3	41.6	1.26	3.78	17.1	63.1	1.26	3.78	137	78.8
6ZU1.25x090	9.70	36.1	1.09	3.28	13.6	54.1	1.08	3.28	85.3	48.7
6ZU1.25x075	7.70	30.1	0.914	2.77	9.47	43.8	0.876	2.77	48.9	27.6
6ZU1.25x060	4.92	23.2	0.704	2.24	5.59	32.3	0.646	2.18	24.9	13.9
6ZU1.25x048	2.85	17.0	0.514	1.76	2.85	23.0	0.460	1.70	12.6	7.00
4ZU1.25x090	6.14	19.9	0.603	1.21	9.30	29.8	0.595	1.21	212	26.8
4ZU1.25x075	5.16	16.6	0.505	1.02	7.82	23.9	0.479	1.02	121	15.3
4ZU1.25x060	4.16	12.7	0.386	0.829	6.06	18.2	0.364	0.803	60.9	7.71
4ZU1.25x048	3.15	9.65	0.292	0.647	3.88	13.8	0.276	0.624	30.8	3.90
4ZU1.25x036	1.77	6.75	0.204	0.465	1.87	9.71	0.194	0.447	12.8	1.63
3.625ZU1.25x090	5.47	17.3	0.524	0.950	8.29	25.8	0.517	0.950	212	23.3
3.625ZU1.25x075	4.60	14.5	0.439	0.805	6.98	20.8	0.416	0.805	121	13.3
3.625ZU1.25x060	3.72	11.0	0.335	0.655	5.63	15.8	0.315	0.634	60.8	6.72
3.625ZU1.25x048	3.00	8.36	0.253	0.511	3.88	11.9	0.239	0.492	30.7	3.40
3.625ZU1.25x036	1.77	5.83	0.176	0.367	2.09	8.35	0.167	0.352	12.8	1.42
2.5ZU1.25x090	3.47	10.4	0.314	0.392	5.25	15.4	0.309	0.392	213	14.0
2.5ZU1.25x075	2.93	8.70	0.264	0.334	4.44	12.4	0.247	0.334	120	8.01
2.5ZU1.25x060	2.38	6.59	0.200	0.273	3.61	9.31	0.186	0.264	60.2	4.06
2.5ZU1.25x048	1.93	4.95	0.150	0.213	2.92	6.98	0.140	0.204	30.3	2.07
2.5ZU1.25x036	1.46	3.40	0.103	0.152	2.18	4.81	0.0963	0.145	12.5	0.866
1.5ZU1.25x090	1.68	5.24	0.159	0.119	2.55	7.78	0.156	0.119	217	7.06
1.5ZU1.25x075	1.45	4.43	0.134	0.102	2.19	6.22	0.124	0.102	121	4.09
1.5ZU1.25x060	1.19	3.35	0.101	0.0845	1.81	4.66	0.0931	0.0814	59.9	2.10
1.5ZU1.25x048	0.978	2.49	0.0754	0.0664	1.48	3.45	0.0690	0.0631	29.9	1.07
1.5ZU1.25x036	0.751	1.68	0.0510	0.0473	1.14	2.34	0.0468	0.0447	12.3	0.454

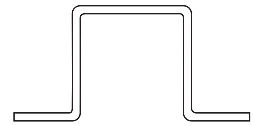
Notes:

1. Shear and moment strengths given are nominal strengths [resistances]. To obtain available strengths [factored resistances], these values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD).
2. M_{web} and M_{flange} are the highest nominal moments at which the web and flange, respectively, are fully effective.
3. Z-Sections Without Lips are not subject to distortional buckling.

Table II - 6

Beam Properties[3]
Hat Sections Without Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)
ϕ_v = 0.80 (LSD)



Section	F_y = 33 ksi						F_y = 50 ksi					
		Compression on Top			Compression on Bottom			Compression on Top			Compression on Bottom	
	V_n [1] kips	M_{nyo}[1,2] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{nyo}[1,2] kip-in.	S_e in.^{3}	V_n [1] kips	M_{nyo}[1,2] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{nyo}[1,2] kip-in.	S_e in.^{3}
10HU5x075	12.7	137	4.15	23.5	124	3.77	12.7	203	4.06	22.6	173	3.46
8HU12x135	39.3	222	6.72	34.4	234	7.09	59.6	326	6.52	32.4	355	7.09
8HU12x105	30.2	155	4.70	23.8	169	5.13	37.1	228	4.55	22.3	243	4.87
8HU8x105	30.2	153	4.63	22.6	160	4.83	37.1	225	4.51	21.5	236	4.72
8HU8x075	15.4	95.5	2.89	14.0	96	2.90	16.1	141	2.81	13.2	134	2.68
8HU4x075	15.4	92.5	2.80	12.4	90.0	2.73	16.1	137	2.75	12.3	128	2.55
8HU4x060	8.20	69.8	2.12	9.59	63.7	1.93	8.20	104	2.07	9.25	89	1.79
6HU9x135	28.6	143	4.33	16.4	146	4.43	43.4	212	4.23	15.8	222	4.43
6HU9x105	22.5	99.4	3.01	11.55	104	3.16	34.1	146	2.93	10.89	158	3.16
6HU6x105	22.5	97.3	2.95	10.42	98.4	2.98	34.1	144	2.88	10.27	149	2.98
6HU6x075	15.4	58.8	1.78	6.58	61.5	1.86	18.9	86.7	1.73	6.25	89.9	1.80
6HU3x075	15.4	55.1	1.67	5.36	55.1	1.67	18.9	83.4	1.67	5.36	83.4	1.67
6HU3x060	9.85	41.8	1.27	4.15	41.4	1.25	11.2	62.3	1.25	4.15	59.2	1.18
6HU3x048	5.70	31.7	0.962	3.25	29.7	0.901	5.70	47.1	0.943	3.17	41.9	0.838
4HU6x135	17.9	77.3	2.34	5.42	77.3	2.34	27.2	116	2.33	5.42	117	2.34
4HU6x105	14.2	53.9	1.63	3.96	54.3	1.64	21.5	80.0	1.60	3.93	82.2	1.64
4HU4x105	14.2	51.2	1.55	3.39	51.2	1.55	21.5	77.6	1.55	3.39	77.6	1.55
4HU4x075	10.3	30.8	0.935	2.18	30.9	0.937	15.6	45.8	0.917	2.18	46.8	0.937
4HU2x075	10.3	27.7	0.839	1.70	27.7	0.839	15.6	41.9	0.839	1.70	41.9	0.839
4HU2x060	8.33	20.6	0.623	1.30	20.6	0.623	12.1	31.2	0.623	1.30	31.2	0.623
4HU2x048	6.30	15.4	0.468	1.002	15.4	0.468	7.76	23.4	0.468	1.002	23.4	0.468
3HU4.5x135	12.6	50.0	1.52	2.47	50.0	1.52	19.1	75.8	1.52	2.47	75.8	1.52
3HU4.5x105	10.0	34.7	1.05	1.80	34.7	1.05	15.2	52.5	1.05	1.80	52.6	1.05
3HU3x105	10.0	32.7	0.992	1.53	32.7	0.992	15.2	49.6	0.992	1.53	49.6	0.992
3HU3x075	7.35	19.3	0.585	0.977	19.3	0.585	11.1	29.3	0.585	0.977	29.3	0.585

Notes:

1. Shear and moment strengths given are nominal strengths [resistances]. To obtain available strengths [factored resistances], these values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD).
2. Y-axis is horizontal.
3. Distortional buckling has not been considered in this table. The Direct Strength Method in *Specification* Appendix 1 can be used to determine the distortional buckling strength.

1.3 Distortional Buckling Flexural Strength Tables

Tables II-7, II-8 and II-9 provide computed distortional buckling properties for the representative C-Sections, joists/studs and Z-Sections with lips, respectively. The values in these tables have been calculated for use with Section C3.1.4(a).

(a) The values in the column under the heading L_{cr} can be used to calculate β with *Eq.* C3.1.4-7. β accounts for the moment gradient over the distortional unbraced length, L_m. It is always conservative to take β as 1.0. In cases where the bending moment is almost constant over the length L_m, β will be calculated to be approximately 1.0.

(b) The values in the column under the heading F_d/β may be multiplied by β to give F_d directly, in lieu of using *Eq.* C3.1.4-6. This will result in the neglect of any rotational stiffness provided by bracing or sheathing, which is conservative where such rotational stiffness exists. These values have been calculated assuming that the unbraced length for distortional buckling, L_m, is greater than or equal to L_{cr}. In cases where the unbraced length is shorter than L_{cr}, these values will produce conservative strengths.

(c) Where a known rotational stiffness, k_ϕ, from bracing or sheathing is available, the values in the columns under the headings $k_{\phi fe}$, $\tilde{k}_{\phi fg}$, $k_{\phi we}$ and $\tilde{k}_{\phi wg}$ may be used in *Eq.* C3.1.4-6 to calculate a more exact value of F_d. These values have been calculated assuming that the unbraced length for distortional buckling, L_m, is greater than or equal to L_{cr}. In cases where the unbraced length is shorter than L_{cr}, these values will produce conservative strengths.

(d) The values in the columns under the headings $M_{n(\beta=1)}$ are nominal distortional buckling strengths calculated assuming β=1.0, i.e. no moment gradient. These values will be conservative if β>1.0.

Table II – 7

F_y = 55 ksi

Distortional Buckling Properties[1]
Flexural Strength
C-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD)
ϕ_b = 0.85 (LSD)

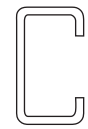
Section	Per Section 3.1.4(a)						
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d/β ksi	M_n[1] $_{(\beta=1)}$ (F_y=55 ksi) kip-in.
12CS4x105	29.5	1.01	0.0393	0.919	0.00576	42.8	309
12CS4x085	31.8	0.520	0.0272	0.477	0.00405	31.9	224
12CS4x070	34.2	0.282	0.0193	0.262	0.00290	24.5	166
12CS3.5x105	27.3	1.02	0.0326	0.943	0.00669	50.0	302
12CS3.5x085	29.4	0.524	0.0225	0.488	0.00470	37.2	220
12CS3.5x070	31.6	0.284	0.0159	0.267	0.00337	28.6	163
12CS2.5x105	22.3	1.05	0.0213	1.03	0.00974	67.0	278
12CS2.5x085	24.1	0.538	0.0146	0.526	0.00684	49.6	203
12CS2.5x070	25.9	0.291	0.0102	0.285	0.00490	38.0	152
10CS4x105	28.2	1.19	0.0431	1.06	0.00370	48.1	254
10CS4x085	30.4	0.613	0.0299	0.555	0.00260	36.0	185
10CS4x070	32.7	0.334	0.0211	0.305	0.00186	27.8	138
10CS4x065	33.6	0.265	0.0185	0.243	0.00163	25.3	123
10CS3.5x105	26.1	1.20	0.0358	1.08	0.00431	57.1	247
10CS3.5x085	28.1	0.618	0.0247	0.565	0.00302	42.7	181
10CS3.5x070	30.2	0.336	0.0174	0.310	0.00216	33.0	135
10CS3.5x065	31.1	0.266	0.0153	0.247	0.00190	29.9	121
10CS2.5x105	21.3	1.24	0.0234	1.16	0.00631	80.8	228
10CS2.5x085	23.0	0.634	0.0160	0.598	0.00442	60.4	168
10CS2.5x070	24.8	0.343	0.0112	0.326	0.00316	46.6	127
10CS2.5x065	25.5	0.272	0.00979	0.259	0.00278	42.3	113
10CS2x105	18.6	1.27	0.0185	1.23	0.00812	93.9	212
10CS2x085	20.1	0.647	0.0125	0.630	0.00569	70.1	158
10CS2x070	21.7	0.349	0.00873	0.341	0.00407	54.0	119
10CS2x065	22.3	0.276	0.00760	0.270	0.00357	48.9	107
9CS2.5x105	20.7	1.36	0.0247	1.25	0.00490	88.4	202
9CS2.5x085	22.4	0.697	0.0169	0.648	0.00343	66.3	150
9CS2.5x070	24.1	0.378	0.0118	0.354	0.00245	51.3	113
9CS2.5x065	24.8	0.299	0.0103	0.282	0.00215	46.6	102
9CS2.5x059	25.8	0.221	0.00863	0.209	0.00181	41.2	88.2
8CS4x105	26.7	1.46	0.0483	1.28	0.00215	54.4	198
8CS4x085	28.7	0.752	0.0334	0.673	0.00151	40.8	145
8CS4x070	30.9	0.410	0.0236	0.372	0.00108	31.7	109
8CS4x065	31.8	0.325	0.0207	0.297	0.000944	28.8	97.3
8CS4x059	33.0	0.241	0.0173	0.221	0.000796	25.4	84.1
8CS3.5x105	24.6	1.47	0.0400	1.30	0.00250	65.2	193
8CS3.5x085	26.6	0.758	0.0276	0.682	0.00175	49.0	142
8CS3.5x070	28.6	0.413	0.0195	0.376	0.00125	38.0	107
8CS3.5x065	29.4	0.327	0.0171	0.300	0.00110	34.5	95.5
8CS3.5x059	30.6	0.242	0.0143	0.223	0.000926	30.5	82.7
8CS2.5x105	20.1	1.51	0.0262	1.37	0.00369	96.5	176
8CS2.5x085	21.7	0.776	0.0179	0.712	0.00258	72.6	131
8CS2.5x070	23.4	0.421	0.0126	0.390	0.00184	56.4	99.7
8CS2.5x065	24.1	0.334	0.0109	0.311	0.00161	51.3	89.6
8CS2.5x059	25.1	0.246	0.00916	0.230	0.00136	45.4	77.8

Table II – 7

F_y = 55 ksi

Distortional Buckling Properties[1]
Flexural Strength
C-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD)
ϕ_b = 0.85 (LSD)



Section	Per Section 3.1.4(a)						
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d/β ksi	M_n[1] $_{(\beta=1)}$ (F_y=55 ksi) kip-in.
8CS2x105	17.6	1.55	0.0207	1.43	0.00477	117	164
8CS2x085	19.0	0.791	0.0140	0.740	0.00333	88.2	123
8CS2x070	20.5	0.428	0.00977	0.404	0.00238	68.5	93.9
8CS2x065	21.1	0.339	0.00851	0.321	0.00208	62.3	84.5
8CS2x059	21.9	0.250	0.00709	0.237	0.00176	55.1	73.6
7CS4x105	25.8	1.65	0.0516	1.44	0.00155	58.1	171
7CS4x085	27.8	0.850	0.0357	0.758	0.00108	43.7	126
7CS4x070	29.9	0.464	0.0253	0.420	0.000774	34.0	94.4
7CS4x065	30.8	0.368	0.0221	0.335	0.000679	30.9	84.6
7CS4x059	31.9	0.273	0.0185	0.250	0.000572	27.3	73.2
7CS2.5x105	19.5	1.70	0.0280	1.52	0.00267	105	150
7CS2.5x085	21.0	0.876	0.0191	0.794	0.00186	79.5	113
7CS2.5x070	22.6	0.476	0.0134	0.437	0.00133	61.9	85.9
7CS2.5x065	23.3	0.377	0.0117	0.348	0.00116	56.4	77.3
7CS2.5x059	24.2	0.279	0.00979	0.259	0.000980	49.9	67.3
6CS4x105	24.8	1.89	0.0557	1.66	0.00106	62.6	144
6CS4x085	26.7	0.980	0.0386	0.873	0.000741	47.1	106
6CS4x070	28.7	0.536	0.0273	0.484	0.000528	36.7	80.2
6CS4x065	29.6	0.426	0.0239	0.387	0.000464	33.4	72.0
6CS4x059	30.7	0.315	0.0200	0.288	0.000391	29.6	62.4
6CS2.5x105	18.7	1.96	0.0303	1.73	0.00183	115	125
6CS2.5x085	20.2	1.01	0.0207	0.907	0.00128	87.2	94.3
6CS2.5x070	21.8	0.549	0.0145	0.500	0.000909	68.1	72.2
6CS2.5x065	22.4	0.435	0.0126	0.399	0.000797	62.0	65.1
6CS2.5x059	23.3	0.322	0.0106	0.297	0.000671	55.0	56.8
4CS4x105	22.4	2.75	0.0683	2.42	0.000388	75.3	92.5
4CS4x085	24.1	1.43	0.0472	1.28	0.000271	57.0	69.0
4CS4x070	26.0	0.784	0.0334	0.712	0.000193	44.5	52.5
4CS4x065	26.7	0.623	0.0292	0.569	0.000169	40.6	47.2
4CS4x059	27.8	0.462	0.0245	0.425	0.000143	36.0	41.1
4CS2.5x105	16.9	2.83	0.0371	2.48	0.000676	141	73.5
4CS2.5x085	18.3	1.46	0.0253	1.31	0.000471	107	58.6
4CS2.5x070	19.7	0.800	0.0178	0.725	0.000334	84.3	45.4
4CS2.5x065	20.3	0.636	0.0155	0.579	0.000293	76.9	41.1
4CS2.5x059	21.1	0.471	0.0130	0.432	0.000246	68.4	36.1
4CS2x105	15.8	2.83	0.0284	2.50	0.000770	183	63.2
4CS2x085	17.2	1.46	0.0191	1.32	0.000531	142	52.1
4CS2x070	18.6	0.798	0.0132	0.729	0.000374	113	42.6
4CS2x065	19.2	0.634	0.0114	0.582	0.000328	103	38.8
4CS2x059	20.0	0.469	0.00951	0.434	0.000274	92.3	34.3

Notes:

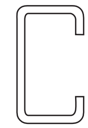
1. Flexural strengths given are nominal strengths [resistances]. To obtain available strengths [factored resistances], these values must be modified by the safety factor (ASD) or resistance factor (LRFD, LSD).

Table II – 8

F_y = 33 ksi
F_y = 50 ksi

Distortional Buckling Properties[1]
Joists/Studs – Flexural Strength
C-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD)
ϕ_b = 0.85 (LSD)



Section	Per Section 3.1.4(a)						
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d/β ksi	M_n[1] $_{(\beta=1)}$ kip-in.
1200S250-97	17.9	1.05	0.0256	1.06	0.0142	53.2	226
1200S250-68	21.7	0.328	0.0128	0.326	0.00698	33.1	136
1200S250-54*	24.5	0.156	0.00811	0.154	0.00441	24.7	97.5
1200S200-97	15.7	1.07	0.0189	1.18	0.0180	60.9	212
1200S200-68	19.0	0.334	0.00953	0.352	0.00889	37.2	128
1200S200-54*	21.5	0.158	0.00606	0.164	0.00562	27.5	91.4
1200S162-97	12.1	1.15	0.0162	1.54	0.0292	59.1	186
1200S162-68	14.5	0.359	0.00845	0.435	0.0146	34.5	110
1200S162-54*	16.4	0.169	0.00543	0.196	0.00925	24.9	78.1
1000S250-97	17.0	1.24	0.0283	1.17	0.00924	64.3	186
1000S250-68	20.7	0.387	0.0141	0.368	0.00453	40.6	114
1000S250-54	23.4	0.184	0.00889	0.176	0.00285	30.6	82.1
1000S250-43*	26.4	0.0891	0.00566	0.0857	0.00180	23.4	46.2
1000S200-97	14.9	1.26	0.0209	1.28	0.0118	77.7	175
1000S200-68	18.1	0.394	0.0105	0.391	0.00579	48.2	108
1000S200-54	20.5	0.187	0.00665	0.184	0.00364	36.0	77.8
1000S200-43*	23.2	0.0902	0.00424	0.0891	0.00231	27.4	43.7
1000S162-97	11.4	1.38	0.0183	1.61	0.0193	79.5	155
1000S162-68	13.7	0.427	0.00938	0.467	0.00957	47.1	94.1
1000S162-54	15.6	0.200	0.00600	0.214	0.00606	34.3	67.4
1000S162-43*	17.6	0.0960	0.00384	0.100	0.00386	25.5	37.6
800S250-97	16.1	1.51	0.0317	1.35	0.00544	76.9	144
800S250-68	19.5	0.472	0.0158	0.433	0.00265	49.2	89.8
800S250-54	22.1	0.225	0.00996	0.209	0.00166	37.4	65.3
800S250-43	25.0	0.110	0.00633	0.103	0.00105	28.8	36.8
800S200-97	14.0	1.54	0.0236	1.44	0.00699	97.6	136
800S200-68	17.1	0.481	0.0118	0.453	0.00340	61.6	85.4
800S200-54	19.4	0.229	0.00745	0.217	0.00213	46.5	62.4
800S200-43	21.9	0.111	0.00475	0.106	0.00135	35.6	35.0
800S200-33*	25.2	0.0481	0.00280	0.0464	0.000790	26.3	24.3
800S162-97	10.6	1.70	0.0209	1.74	0.0116	106	120
800S162-68	12.9	0.522	0.0106	0.521	0.00569	64.1	75.3
800S162-54	14.7	0.245	0.00674	0.243	0.00359	47.3	54.8
800S162-43	16.6	0.118	0.00431	0.116	0.00227	35.5	30.6
800S162-33*	19.1	0.0505	0.00255	0.0498	0.00134	25.8	21.1
800S137-97	8.04	1.89	0.0196	2.31	0.0195	107	107
800S137-68	9.66	0.581	0.0103	0.653	0.00970	61.6	66.1
800S137-54	10.9	0.271	0.00667	0.293	0.00615	44.0	47.5
800S137-43	12.3	0.128	0.00430	0.136	0.00393	32.1	26.3
800S137-33*	14.2	0.0543	0.00255	0.0564	0.00232	22.7	17.9
600S250-97	14.9	1.94	0.0367	1.66	0.00273	91.3	103
600S250-68	18.2	0.612	0.0182	0.546	0.00132	59.3	65.2
600S250-54	20.6	0.293	0.0115	0.267	0.000821	45.4	48.0
600S250-43	23.2	0.143	0.00731	0.133	0.000517	35.2	27.1
600S200-97	13.0	1.98	0.0273	1.74	0.00353	121	93.5
600S200-68	15.9	0.623	0.0136	0.564	0.00170	77.5	61.8
600S200-54	18.0	0.297	0.00862	0.274	0.00106	59.0	45.7
600S200-43	20.4	0.145	0.00549	0.135	0.000668	45.5	25.7
600S200-33	23.4	0.0631	0.00324	0.0598	0.000390	33.9	18.0

Table II – 8

F_y = 33 ksi
F_y = 50 ksi

Distortional Buckling Properties[1]
Joists/Studs – Flexural Strength
C-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD)
ϕ_b = 0.85 (LSD)

Section	Per Section 3.1.4(a)						
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d/β ksi	$M_n{}^1{}_{(\beta=1)}$ kip-in.
600S162-97	9.84	2.19	0.0244	2.00	0.00594	138	80.0
600S162-68	12.0	0.674	0.0123	0.621	0.00288	85.3	54.7
600S162-54	13.6	0.318	0.00781	0.296	0.00180	63.8	40.5
600S162-43	15.4	0.153	0.00498	0.144	0.00114	48.5	22.6
600S162-33	17.8	0.0659	0.00294	0.0627	0.000665	35.6	15.8
600S137-97	7.34	2.50	0.0236	2.50	0.0101	148	69.8
600S137-68	8.91	0.755	0.0122	0.735	0.00500	86.8	48.3
600S137-54	10.1	0.351	0.00779	0.339	0.00315	63.0	35.5
600S137-43	11.4	0.166	0.00499	0.161	0.00199	46.8	19.7
600S137-33	13.2	0.0706	0.00296	0.0683	0.00117	33.6	13.7
550S162-68	11.7	0.728	0.0129	0.660	0.00234	91.3	49.4
550S162-54	13.3	0.343	0.00816	0.316	0.00146	68.5	36.7
550S162-43	15.1	0.166	0.00521	0.154	0.000921	52.2	20.6
550S162-33	17.4	0.0715	0.00308	0.0675	0.000538	38.5	14.4
400S200-68	14.4	0.898	0.0167	0.794	0.000632	97.8	38.5
400S200-54	16.3	0.432	0.0106	0.391	0.000392	75.1	28.9
400S200-43	18.4	0.212	0.00672	0.195	0.000246	58.3	16.3
400S200-33	21.2	0.0927	0.00397	0.0871	0.000143	43.8	11.5
400S162-68	10.8	0.964	0.0151	0.841	0.00109	111	33.7
400S162-54	12.3	0.458	0.00958	0.408	0.000675	84.4	25.5
400S162-43	13.9	0.222	0.00611	0.202	0.000423	64.9	14.3
400S162-33	16.1	0.0962	0.00361	0.0894	0.000246	48.2	10.1
400S137-68	8.01	1.07	0.0151	0.932	0.00192	118	29.1
400S137-54	9.11	0.501	0.00960	0.443	0.00120	87.4	22.3
400S137-43	10.3	0.239	0.00614	0.215	0.000754	66.0	12.5
400S137-33	11.9	0.102	0.00363	0.0939	0.000439	48.2	8.83
362S200-68	14.0	0.983	0.0175	0.867	0.000496	103	34.3
362S200-54	15.9	0.473	0.0111	0.428	0.000307	79.0	25.8
362S200-43	18.0	0.232	0.00706	0.214	0.000193	61.5	14.5
362S200-33	20.7	0.102	0.00417	0.0956	0.000112	46.2	10.3
362S162-68	10.6	1.05	0.0159	0.911	0.000855	117	29.5
362S162-54	12.0	0.500	0.0101	0.444	0.000530	89.1	22.7
362S162-43	13.6	0.243	0.00642	0.220	0.000332	68.6	12.7
362S162-33	15.7	0.106	0.00379	0.0978	0.000193	51.1	9.07
362S137-68	7.81	1.17	0.0158	0.997	0.00152	125	25.5
362S137-54	8.88	0.546	0.0101	0.477	0.000946	92.7	19.9
362S137-43	10.1	0.261	0.00645	0.233	0.000593	70.2	11.1
362S137-33	11.6	0.112	0.00381	0.102	0.000345	51.4	7.90
350S162-68	10.5	1.09	0.0162	0.938	0.000785	120	28.1
350S162-54	11.9	0.516	0.0102	0.458	0.000487	90.8	21.8
350S162-43	13.5	0.251	0.00653	0.227	0.000305	70.0	12.2
350S162-33	15.5	0.109	0.00386	0.101	0.000177	52.1	8.71
250S162-68	9.63	1.47	0.0191	1.26	0.000344	140	18.0
250S162-54	10.9	0.702	0.0121	0.620	0.000213	107	14.7
250S162-43	12.4	0.343	0.00773	0.310	0.000133	83.0	7.97
250S162-33	14.3	0.150	0.00456	0.139	0.000076	62.2	5.95
250S137-68	7.11	1.61	0.0191	1.33	0.000619	149	15.4
250S137-54	8.09	0.759	0.0122	0.647	0.000383	112	12.7
250S137-43	9.17	0.365	0.00777	0.321	0.000239	85.6	6.88
250S137-33	10.6	0.158	0.00459	0.142	0.000138	63.3	5.17

Notes:

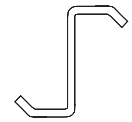
1. Flexural strengths given are nominal strengths [resistances]. To obtain available strengths [factored resistances], these values must be modified by the safety factor (ASD) or resistance factor (LRFD, LSD).

Table II – 9

Distortional Buckling Properties[1]
Flexural Strength
Z-Sections With Lips

Fy=55 ksi

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD)
ϕ_b = 0.85 (LSD)



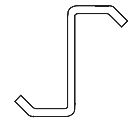
Section	Per Section 3.1.4(a)						
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d/β ksi	M_n[1] $_{(\beta=1)}$ (F_y=55 ksi) kip-in.
12ZS3.25x105	24.3	1.07	0.0391	0.987	0.00832	43.5	287
12ZS3.25x085	26.6	0.544	0.0263	0.505	0.00566	32.8	209
12ZS3.25x070	28.9	0.293	0.0183	0.274	0.00400	25.4	155
12ZS2.75x105	21.9	1.09	0.0326	1.04	0.0101	49.9	276
12ZS2.75x085	24.0	0.553	0.0219	0.526	0.00687	37.5	201
12ZS2.75x070	26.1	0.297	0.0152	0.284	0.00485	29.0	150
12ZS2.25x105	19.3	1.12	0.0268	1.11	0.0127	56.5	262
12ZS2.25x085	21.2	0.566	0.0180	0.559	0.00866	42.2	191
12ZS2.25x070	23.0	0.303	0.0124	0.299	0.00612	32.5	142
10ZS3.25x105	23.2	1.26	0.0429	1.12	0.00537	49.4	235
10ZS3.25x085	25.4	0.640	0.0288	0.579	0.00365	37.5	172
10ZS3.25x070	27.6	0.345	0.0200	0.317	0.00257	29.3	129
10ZS3.25x065	28.5	0.273	0.0174	0.252	0.00224	26.7	115
10ZS3.25x059	29.8	0.201	0.0145	0.186	0.00187	23.7	99.7
10ZS2.75x105	20.9	1.28	0.0358	1.17	0.00654	58.0	226
10ZS2.75x085	22.9	0.651	0.0240	0.599	0.00444	43.9	166
10ZS2.75x070	24.9	0.350	0.0166	0.326	0.00313	34.2	125
10ZS2.75x065	25.7	0.277	0.0145	0.258	0.00273	31.1	112
10ZS2.75x059	26.9	0.204	0.0120	0.191	0.00228	27.6	96.6
10ZS2.25x105	18.4	1.32	0.0295	1.24	0.00827	67.6	215
10ZS2.25x085	20.2	0.666	0.0198	0.628	0.00562	51.0	158
10ZS2.25x070	22.0	0.357	0.0137	0.339	0.00396	39.6	119
10ZS2.25x065	22.7	0.282	0.0119	0.269	0.00346	36.0	106
10ZS2.25x059	23.7	0.207	0.0098	0.198	0.00289	31.9	92.1
9ZS2.25x105	17.9	1.45	0.0312	1.33	0.00644	73.7	191
9ZS2.25x085	19.7	0.731	0.0209	0.677	0.00437	55.8	141
9ZS2.25x070	21.4	0.393	0.0144	0.367	0.00307	43.5	106
9ZS2.25x065	22.1	0.311	0.0125	0.291	0.00268	39.6	95.3
9ZS2.25x059	23.1	0.228	0.0104	0.215	0.00224	35.1	82.7
8ZS3.25x105	21.9	1.54	0.0480	1.34	0.00313	56.2	183
8ZS3.25x085	24.1	0.782	0.0323	0.695	0.00212	43.0	135
8ZS3.25x070	26.1	0.423	0.0224	0.382	0.00149	33.7	101
8ZS3.25x065	27.0	0.335	0.0195	0.304	0.00130	30.7	90.9
8ZS3.25x059	28.2	0.247	0.0162	0.226	0.00108	27.4	78.8
8ZS2.75x105	19.7	1.56	0.0401	1.38	0.00382	67.0	175
8ZS2.75x085	21.7	0.795	0.0269	0.712	0.00259	51.1	130
8ZS2.75x070	23.6	0.429	0.0186	0.390	0.00182	40.0	98.0
8ZS2.75x065	24.3	0.339	0.0162	0.310	0.00159	36.5	87.9
8ZS2.75x059	25.4	0.250	0.0134	0.230	0.00132	32.5	76.3
8ZS2.25x105	17.4	1.60	0.0331	1.44	0.00486	80.1	166
8ZS2.25x085	19.1	0.812	0.0221	0.738	0.00329	61.0	123
8ZS2.25x070	20.8	0.437	0.0153	0.402	0.00231	47.7	93.5
8ZS2.25x065	21.5	0.346	0.0133	0.319	0.00202	43.5	84.0
8ZS2.25x059	22.4	0.254	0.0110	0.236	0.00168	38.6	73.0
7ZS2.25x105	16.8	1.80	0.0354	1.59	0.00352	87.1	142
7ZS2.25x085	18.5	0.915	0.0237	0.819	0.00238	66.5	106
7ZS2.25x070	20.1	0.493	0.0164	0.448	0.00167	52.2	80.6
7ZS2.25x065	20.8	0.390	0.0142	0.356	0.00146	47.7	72.5
7ZS2.25x059	21.7	0.287	0.0118	0.264	0.00121	42.4	63.1

Table II – 9

Fy=55 ksi

Distortional Buckling Properties[1]
Flexural Strength
Z-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD)
ϕ_b = 0.85 (LSD)



Section	Per Section 3.1.4(a)						
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d/β ksi	M_n[1] $(\beta=1)$ (F_y=55 ksi) kip-in.
6ZS2.25x105	16.2	2.07	0.0383	1.79	0.00243	94.8	118
6ZS2.25x085	17.8	1.05	0.0256	0.929	0.00164	72.7	88.6
6ZS2.25x070	19.3	0.568	0.0177	0.510	0.00115	57.3	67.7
6ZS2.25x065	20.0	0.449	0.0153	0.406	0.00100	52.4	61.0
6ZS2.25x059	20.9	0.331	0.0127	0.302	0.00083	46.7	53.2
4ZS2.25x070	17.5	0.823	0.0217	0.733	0.00042	70.5	42.6
4ZS2.25x065	18.0	0.653	0.0188	0.585	0.00036	64.6	38.5
4ZS2.25x059	18.9	0.482	0.0156	0.436	0.00030	57.8	33.7
3.5ZS1.5x070	10.6	1.03	0.0180	0.885	0.00075	102	29.5
3.5ZS1.5x065	11.0	0.811	0.0157	0.704	0.00065	92.8	26.8
3.5ZS1.5x059	11.4	0.596	0.0130	0.522	0.00054	82.3	23.5

Notes:

1. Flexural strengths given are nominal strengths [resistances]. To obtain available strengths [factored resistances], these values must be modified by the safety factor (ASD) or resistance factor (LRFD, LSD).

1.4 Calculation of L_u

For members bent about the centroidal axis perpendicular to the web, calculation of lateral-torsional buckling strength is unnecessary when the unbraced length is less than a length, L_u, which results in a critical elastic flexural stress, F_e, that is $2.78F_y$. L_u may be calculated using the following formulas. All terms are as defined in Section C3.1.2 of the *Specification.*

(a) For singly-, doubly-, and point symmetric sections:

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

where

(1) For singly- and doubly-symmetric sections:

$$C_1 = \frac{7.72}{AE} \left[\frac{K_y F_y S_f}{C_b \pi r_y} \right]^2 \qquad (\textit{Eq. C-C3.1.2.1-12})$$

$$C_2 = \frac{\pi^2 E C_w}{(K_t)^2} \qquad (\textit{Eq. C-C3.1.2.1-14})$$

(2) For point-symmetric sections:

$$C_1 = \frac{30.9}{AE} \left[\frac{K_y F_y S_f}{C_b \pi r_y} \right]^2 \qquad (\textit{Eq. C-C3.1.2.1-13})$$

$$C_2 = \frac{\pi^2 E C_w}{(K_t)^2} \qquad (\textit{Eq. C-C3.1.2.1-14})$$

(b) For I-, C- or Z-Sections bent about the centroidal axis perpendicular to the web (x-axis), in lieu of (a), the following equations may be used:

(1) For doubly-symmetric I-Sections and singly-symmetric C-Sections:

$$L_u = \sqrt{\frac{0.36 C_b \pi^2 E d I_{yc}}{F_y S_f (K_y)^2}} \qquad (\textit{Eq. C-C3.1.2.1-15})$$

(2) For point-symmetric Z-Sections:

$$L_u = \sqrt{\frac{0.18 C_b \pi^2 E d I_{yc}}{F_y S_f (K_y)^2}} \qquad (\textit{Eq. C-C3.1.2.1-16})$$

(c) For closed box members:

$$L_u = \frac{0.36 C_b \pi}{F_y S_f} \sqrt{EGJI_y} \qquad (\textit{Eq. C3.1.2.2-1})$$

1.5 Notes on the Charts

(a) With the exception of joists/studs, the specific sections listed in these charts are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members.

(b) The section designations listed in these charts are a subset of those for which dimensions and properties are given in Tables I-1, I-2 and I-4. This subset is intended to represent those sections most commonly used in routine design.

(c) The nominal flexural strength, M_n, is given as a function of unbraced length. In these charts the torsional unbraced length is assumed to equal the y-axis unbraced length and $K_y = K_t = 1.0$.

(d) The effects of standard factory punchouts in joists/studs have been included in Charts II-2a and II-2b. These punchouts are considered in joists/studs with flange widths of 1.625 in. or less. Standard punchout sizes are 1.5 in by 4.5 in. for sections with depths of 3.5 inches or more and 0.75 in. by 4.5 in. for sections with shallower depths. Punchouts are not included in the calculations for sections with web h/t ratios in excess of 200 due to the limits in Section B2.4. Those sections are marked with a "*".

(e) The flexural strengths were computed using the nominal yield stress listed in the charts, except for the sections in Chart II-2, where the provisions of Section A7 of the *Specification* for strength increase from cold work of forming have been used where the section was eligible. Sections were considered eligible for the cold work of forming increase in yield stress if ρ=1.0 for each flat element, except that webs may have ρ<1.0 if the sum of b_1 plus b_2 from Section B2.3 equals or exceeds the width of the compression portion of the web.

(f) To obtain ASD design values, the nominal strengths in these charts must be divided by Ω_b=1.67 (ASD).

(g) To obtain LRFD or LSD design values, the nominal strengths in these charts must be multiplied by ϕ_b=0.90 (ASD, LSD).

(h) A solid line indicates that the section is the lightest available in the chart for a given nominal strength and unbraced length. A dashed line indicates that there is a lighter section available in the chart for that combination of nominal strength and unbraced length.

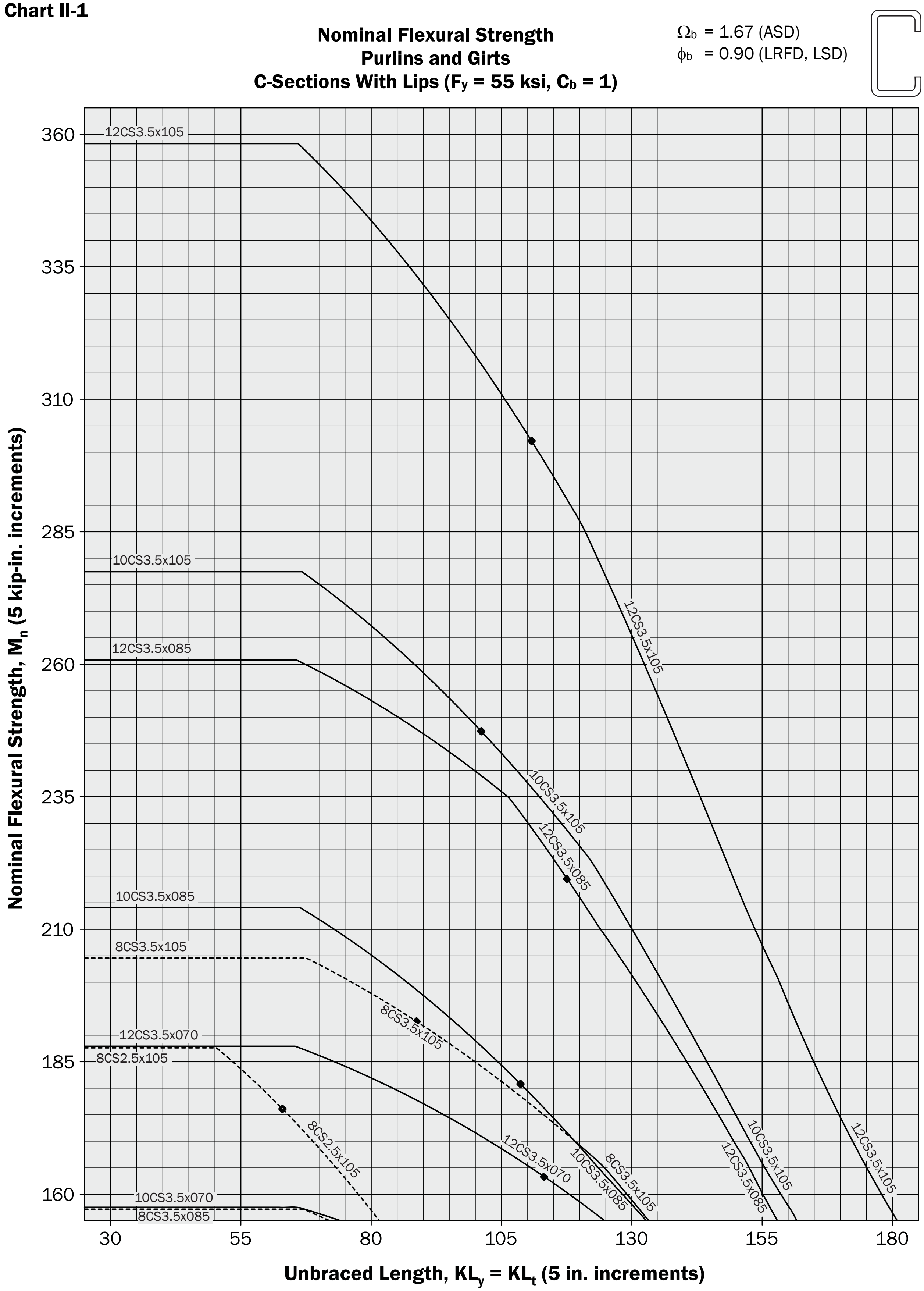
(i) A diamond on a curve designates the smallest possible nominal distortional buckling strength and the corresponding lateral-torsional unbraced length for that section calculated using Section C3.1.4(a) without distortional bracing. A more refined analysis using Sections C3.1.4(a) or C3.1.4(b) may give a larger available distortional buckling strength in cases where significant distortional bracing is present, or a significant moment gradient exists, or the spacing of discrete distortional braces is smaller than L_{cr}.

(j) Unbraced lengths in these charts are arbitrarily limited to a maximum of 40 times the depth of the section.

(k) Shear, web crippling, combined bending and shear, combined bending and web crippling and deflection must also be checked and they are not considered in these charts.

1.6 Beam Charts

Chart II-1



Nominal Flexural Strength
Purlins and Girts
C-Sections With Lips (F_y = 55 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Nominal Flexural Strength, M_n (5 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (5 in. increments)
360
335
310
285
260
235
210
185
160
30
55
80
105
130
155
180
12CS3.5x105
10CS3.5x105
12CS3.5x085
10CS3.5x085
8CS3.5x105
12CS3.5x070
8CS2.5x105
10CS3.5x070
8CS3.5x085

Chart II-1

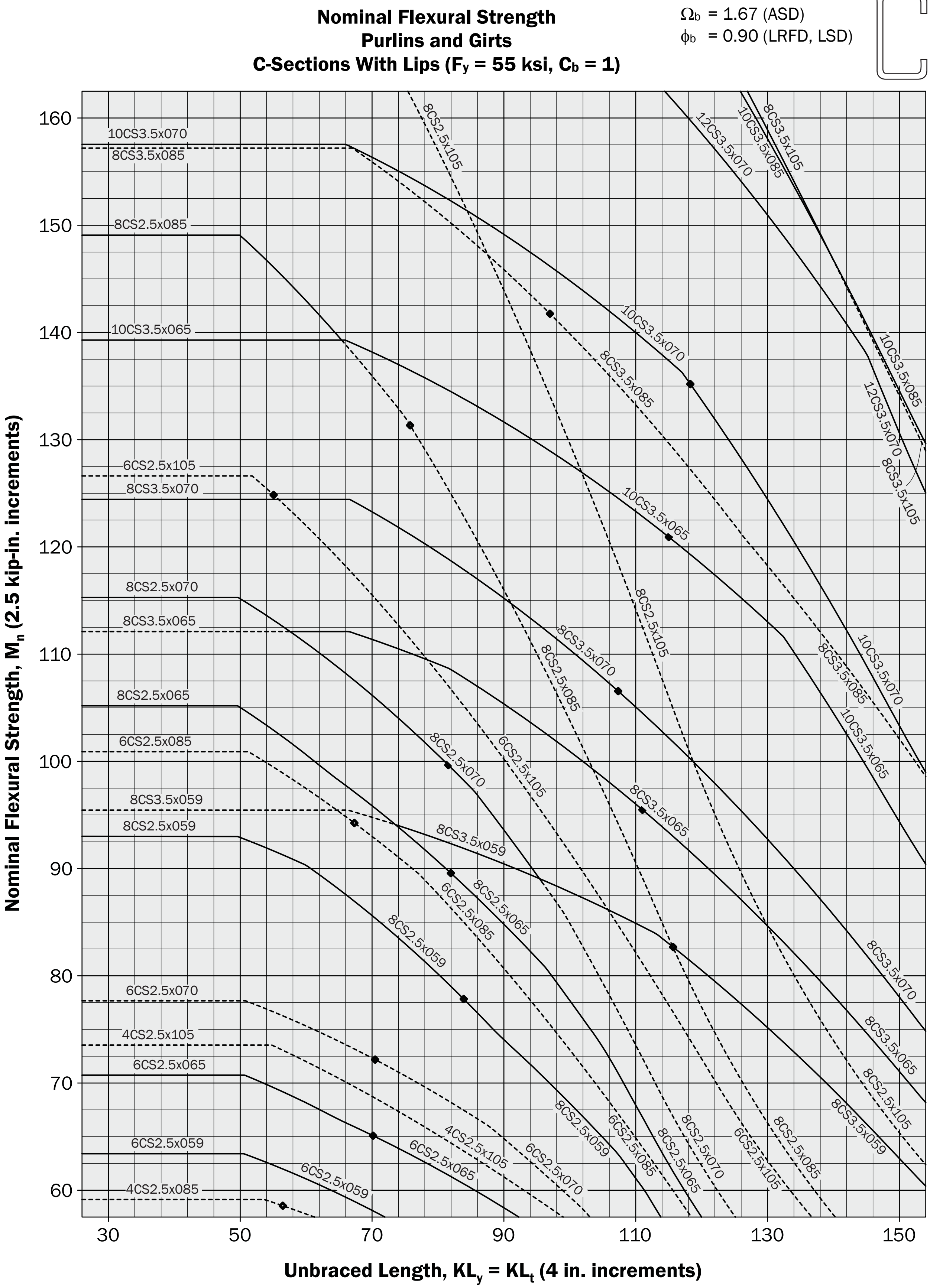

Nominal Flexural Strength
Purlins and Girts
C-Sections With Lips (F_y = 55 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Nominal Flexural Strength, M_n (2.5 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (4 in. increments)
160
150
140
130
120
110
100
90
80
70
60
30
50
70
90
110
130
150
10CS3.5x070
8CS3.5x085
8CS2.5x085
10CS3.5x065
6CS2.5x105
8CS3.5x070
8CS2.5x070
8CS3.5x065
8CS2.5x065
6CS2.5x085
8CS3.5x059
8CS2.5x059
6CS2.5x070
4CS2.5x105
6CS2.5x065
6CS2.5x059
4CS2.5x085
8CS2.5x105
12CS3.5x070
10CS3.5x085
8CS3.5x105
8CS2.5x085
6CS2.5x105
4CS2.5x105
6CS2.5x070
8CS2.5x070
6CS2.5x085
6CS2.5x065
6CS2.5x059

Chart II-1

Nominal Flexural Strength
Purlins and Girts
C-Sections With Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

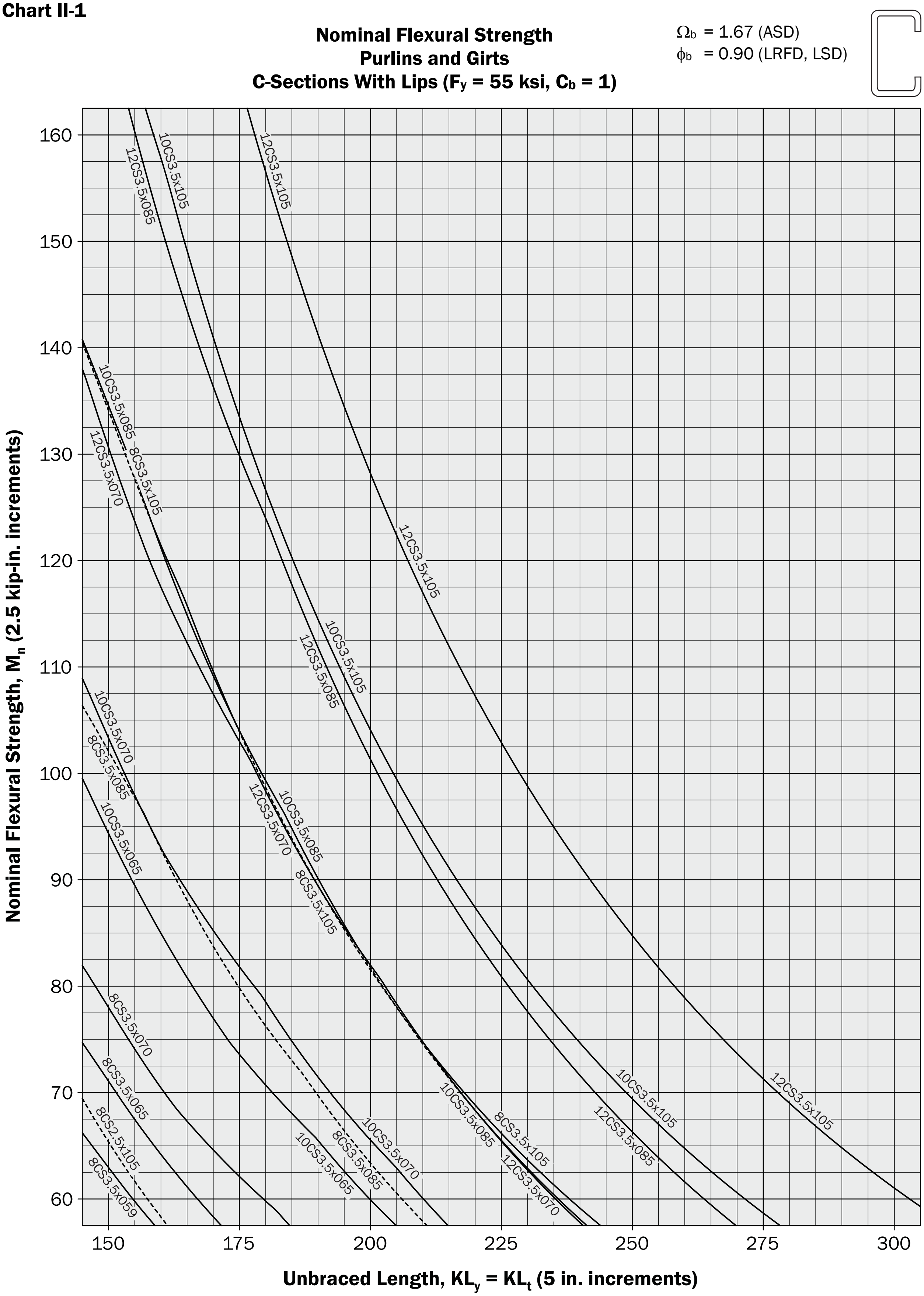


Chart II-1

Nominal Flexural Strength
Purlins and Girts
C-Sections With Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

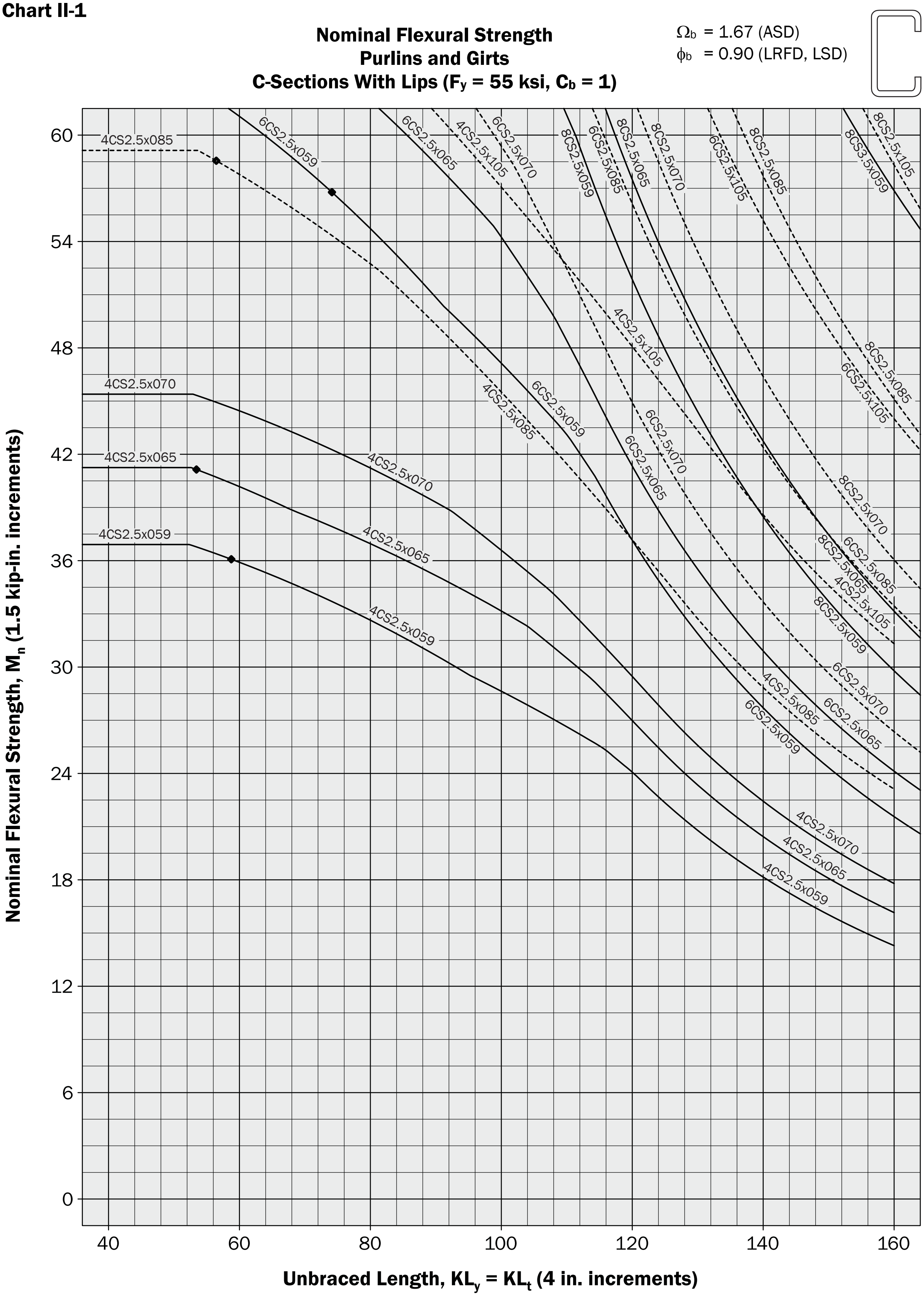


Chart II-1

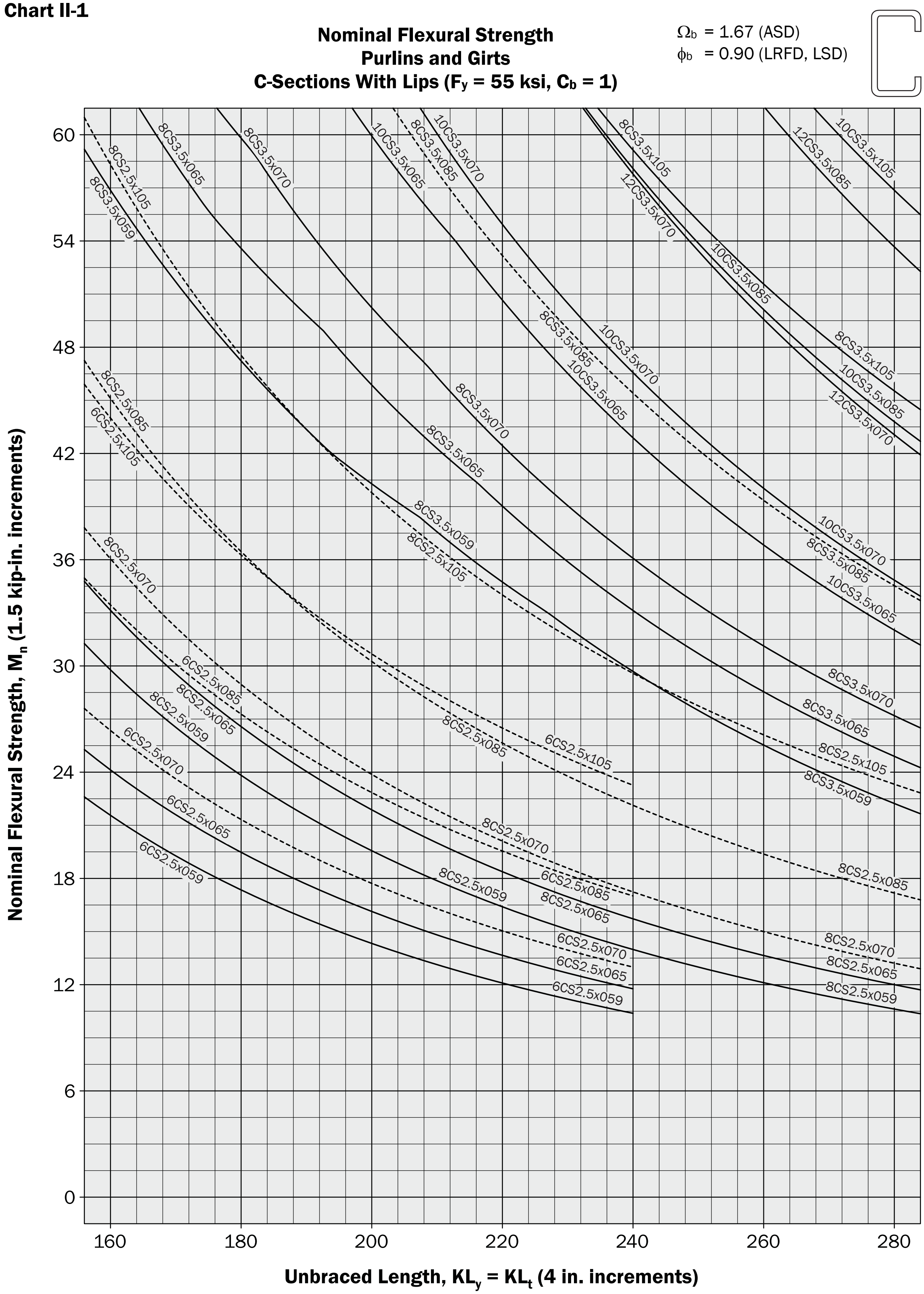

Nominal Flexural Strength
Purlins and Girts
C-Sections With Lips (F_y = 55 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Nominal Flexural Strength, M_n (1.5 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (4 in. increments)
0
6
12
18
24
30
36
42
48
54
60
160
180
200
220
240
260
280
8CS2.5x105
8CS3.5x059
8CS3.5x065
8CS3.5x070
10CS3.5x065
8CS3.5x085
10CS3.5x070
8CS3.5x105
12CS3.5x070
12CS3.5x085
10CS3.5x105
10CS3.5x085
8CS2.5x085
6CS2.5x105
8CS2.5x070
6CS2.5x085
6CS2.5x070
6CS2.5x065
6CS2.5x059
8CS2.5x065
8CS2.5x059
8CS2.5x085
6CS2.5x105
8CS2.5x070
6CS2.5x085
6CS2.5x070
6CS2.5x065
6CS2.5x059

Chart II-1

Nominal Flexural Strength
Purlins and Girts
C-Sections With Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

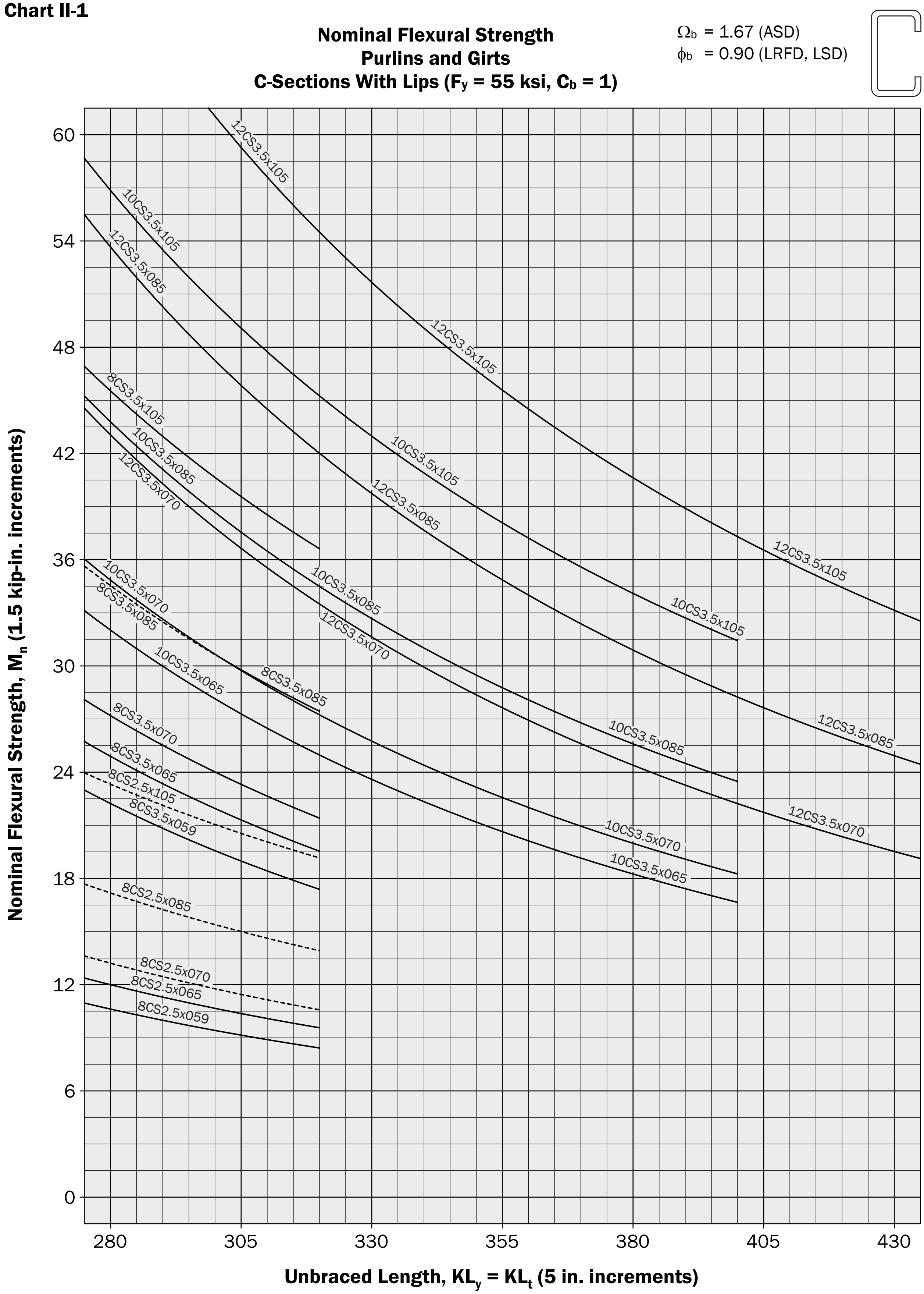


Chart II-2a

Nominal Flexural Strength
Joists/Studs
C-Sections With Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

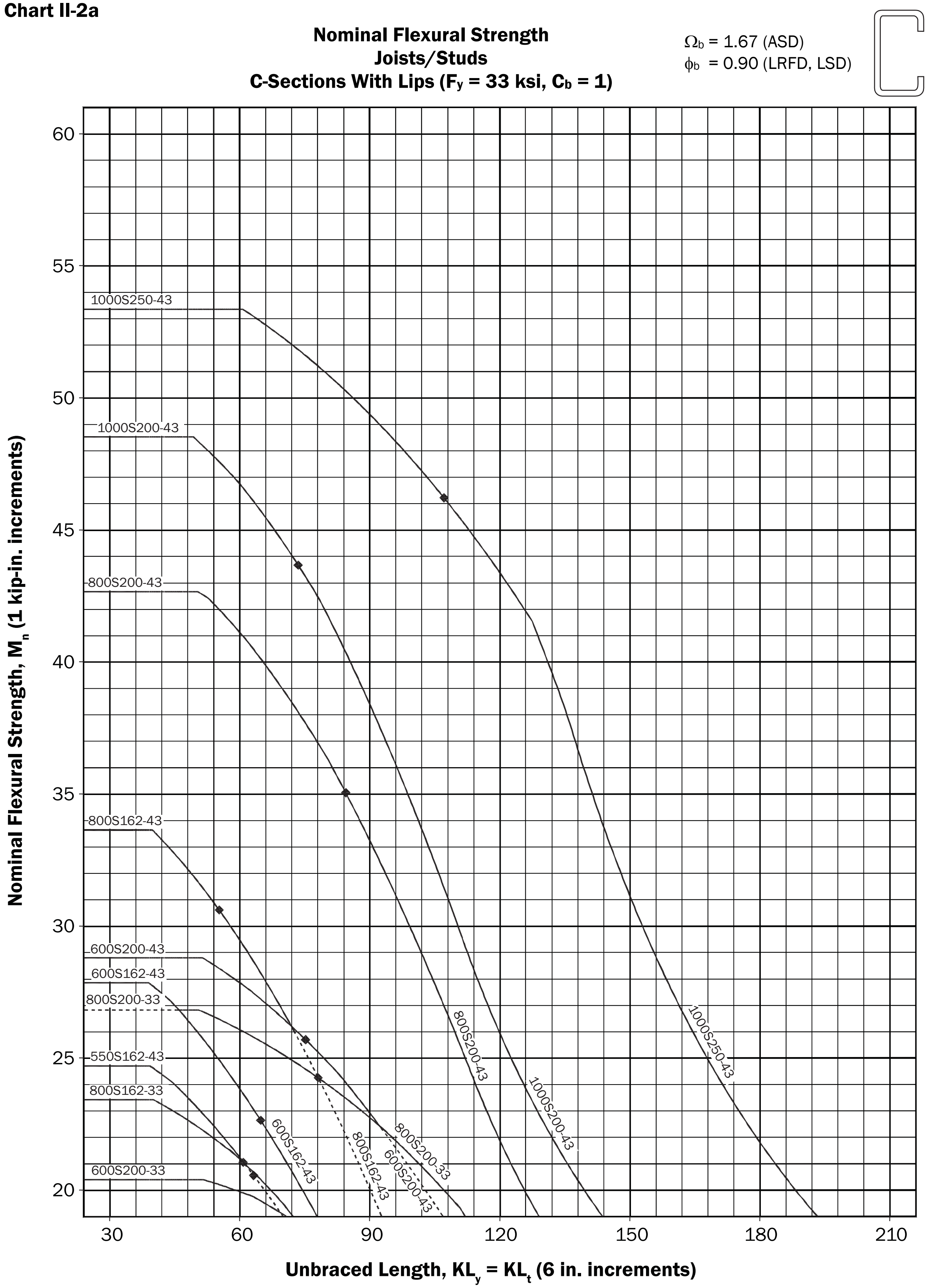
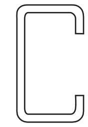


Chart II-2a

Nominal Flexural Strength
Joists/Studs
C-Sections With Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)



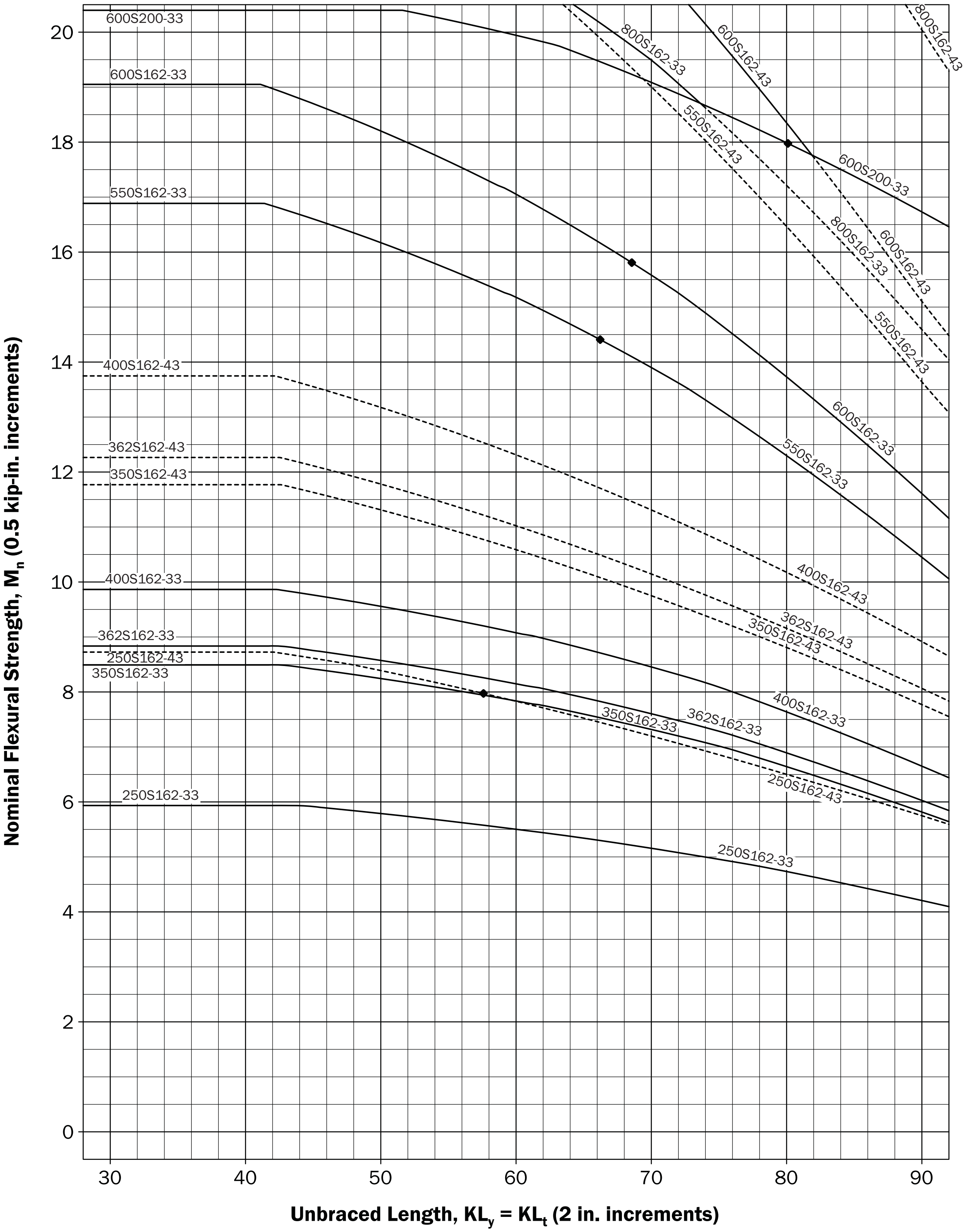
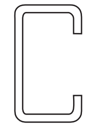


Chart II-2a

Nominal Flexural Strength
Joists/Studs
C-Sections With Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)



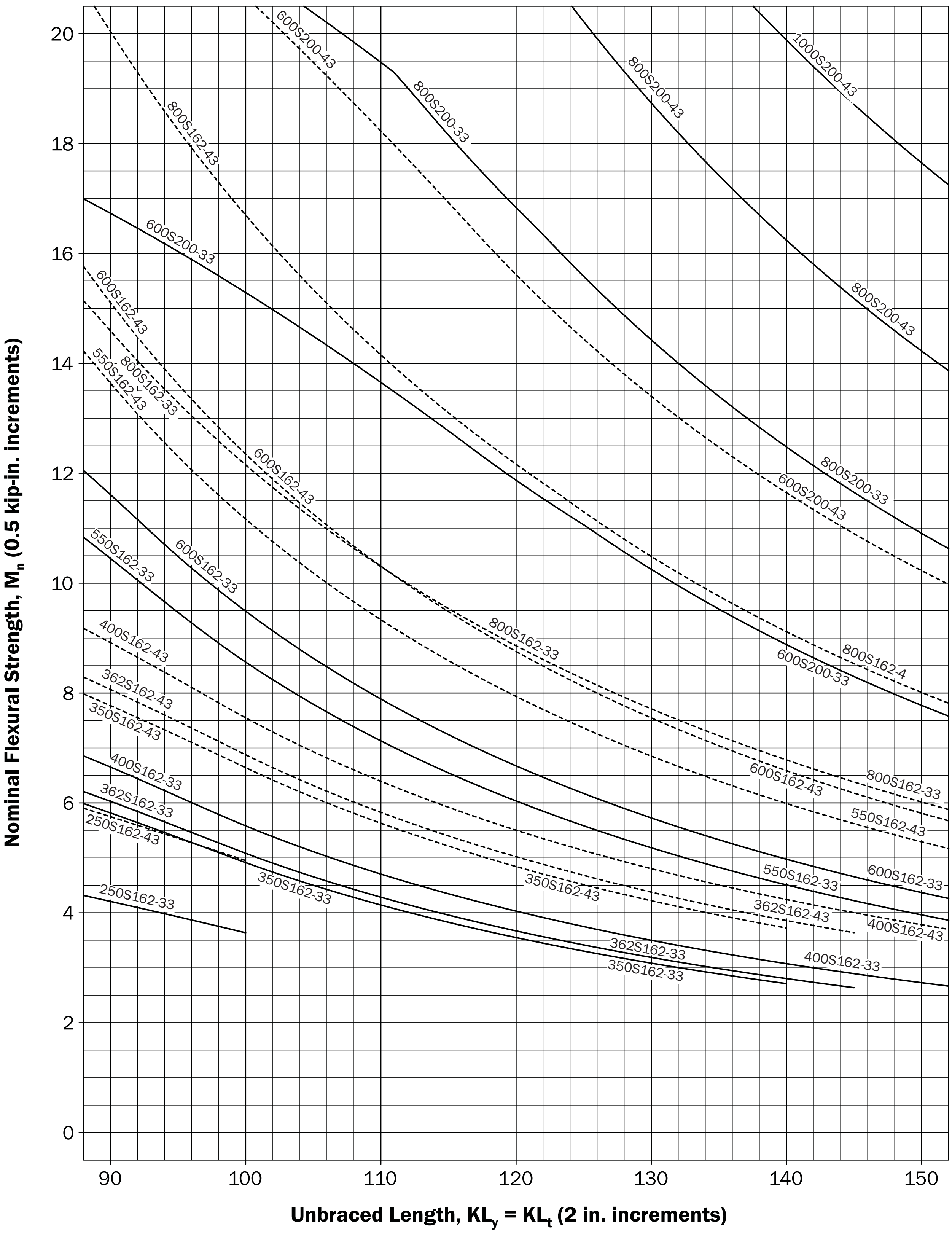


Chart II-2a

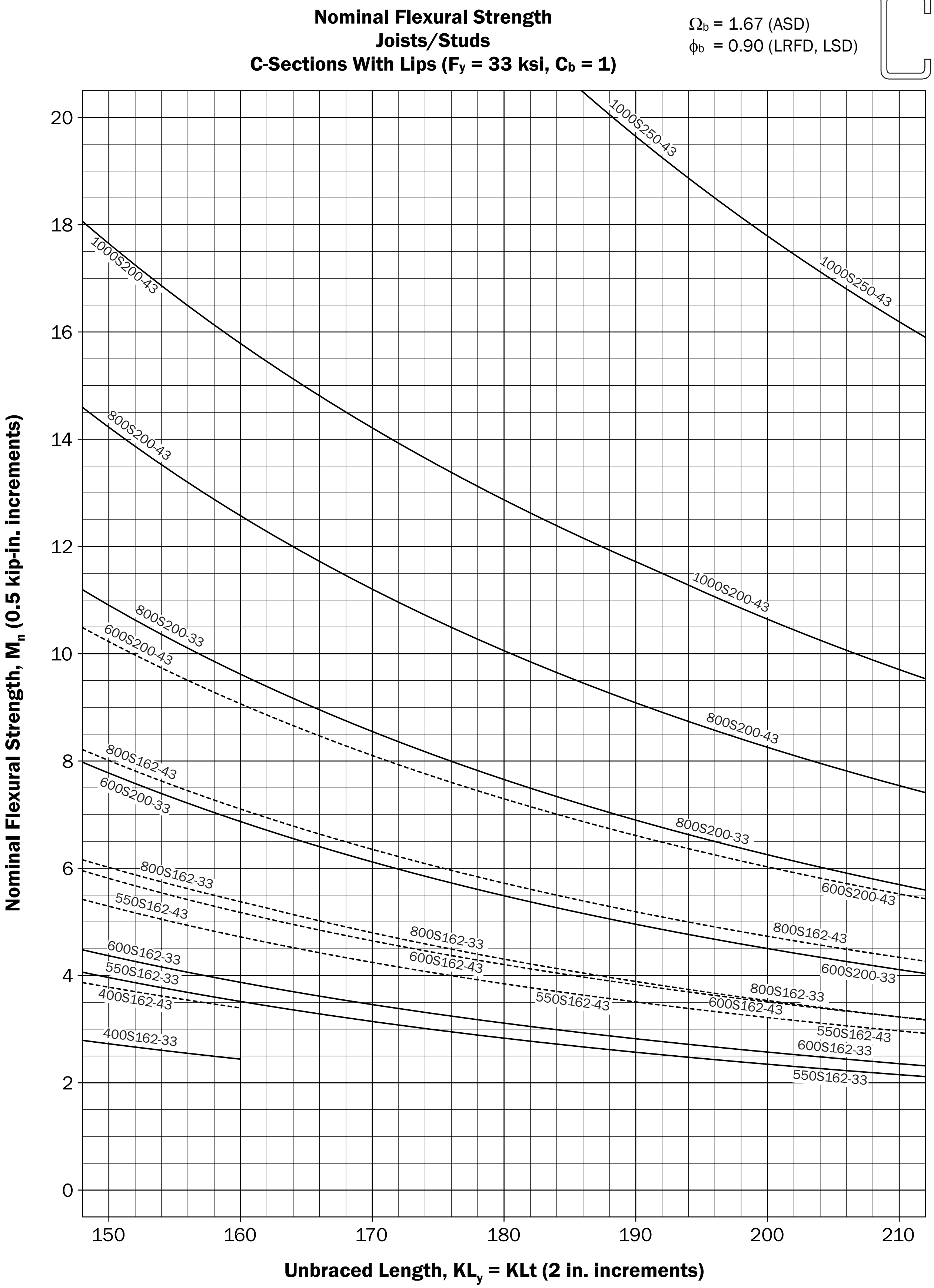

Nominal Flexural Strength
Joists/Studs
C-Sections With Lips (F_y = 33 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Nominal Flexural Strength, M_n (0.5 kip-in. increments)
Unbraced Length, KL_y = KLt (2 in. increments)
0
2
4
6
8
10
12
14
16
18
20
150
160
170
180
190
200
210
1000S250-43
1000S200-43
800S200-43
800S200-33
600S200-43
800S162-43
600S200-33
800S162-33
550S162-43
600S162-33
550S162-33
400S162-43
400S162-33
600S162-43
550S162-43

Chart II-2a

Nominal Flexural Strength
Joists/Studs
C-Sections With Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

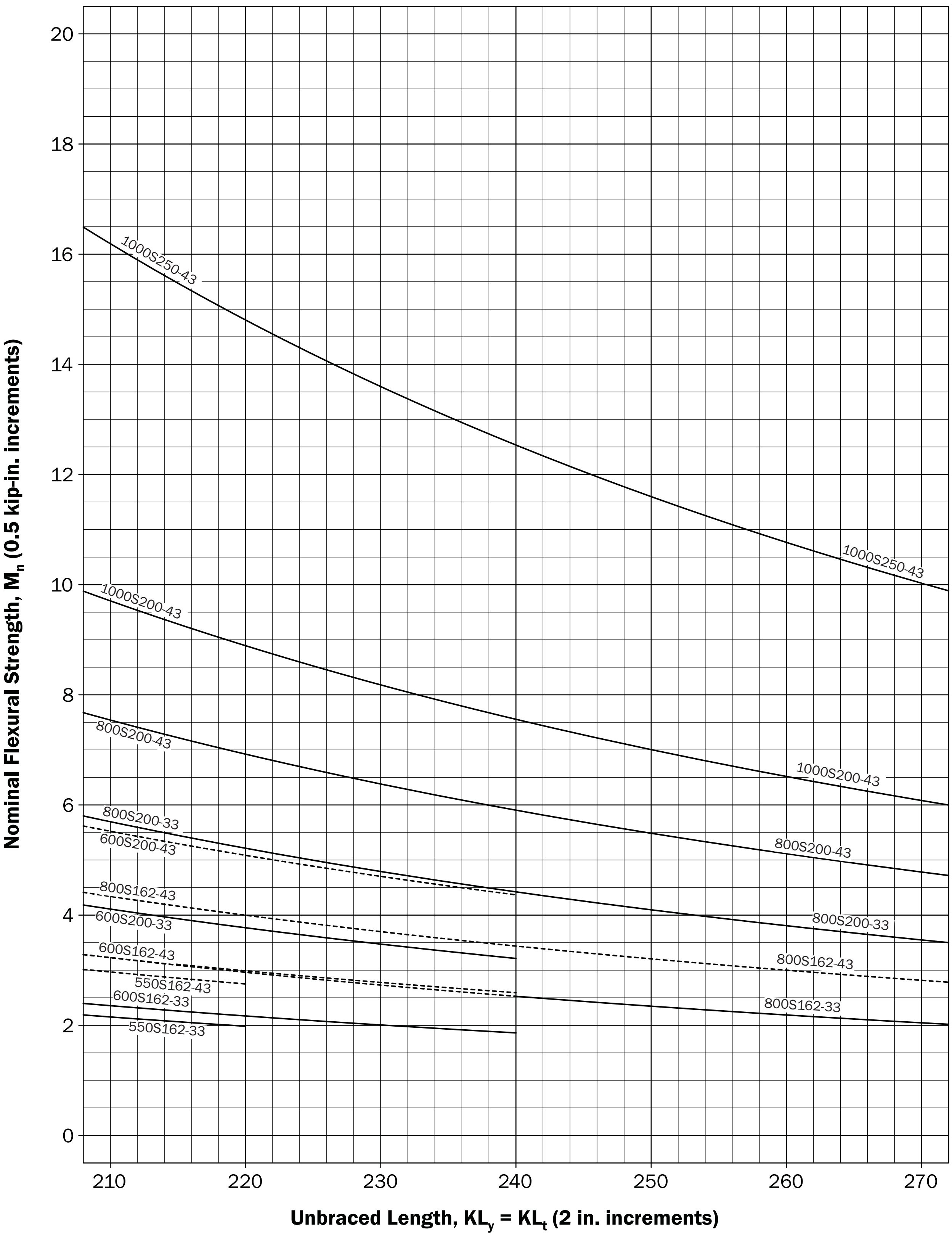


Chart II-2a

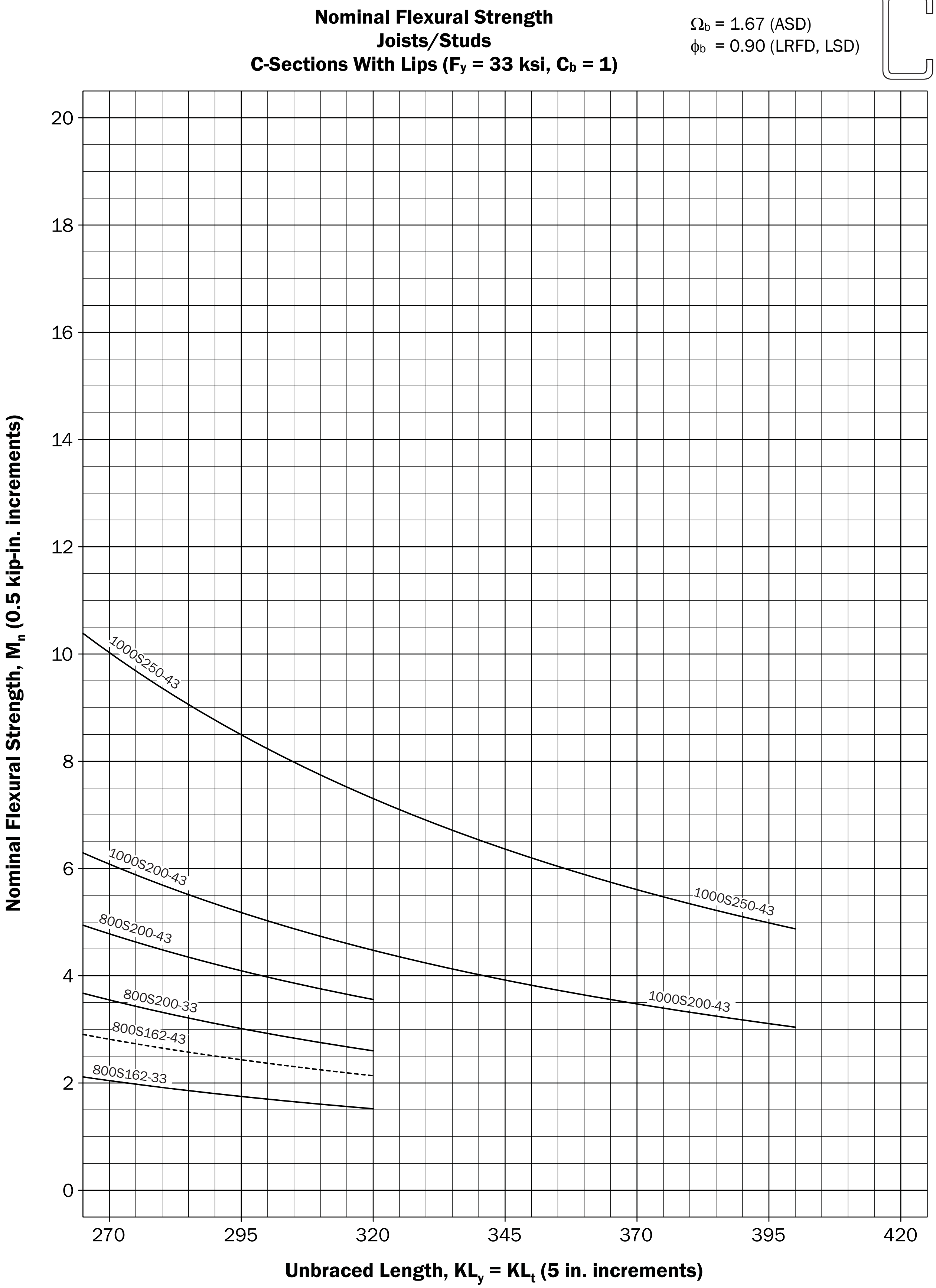
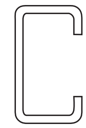

Nominal Flexural Strength
Joists/Studs
C-Sections With Lips (F_y = 33 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
1000S250-43
1000S200-43
800S200-43
800S200-33
800S162-43
800S162-33
1000S250-43
1000S200-43
0
2
4
6
8
10
12
14
16
18
20
270
295
320
345
370
395
420
Nominal Flexural Strength, M_n (0.5 kip-in. increments)
Unbraced Length, KL_y = KL_t (5 in. increments)

Chart II-2b

Nominal Flexural Strength
Joists/Studs
C-Sections With Lips (F_y = 50 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)



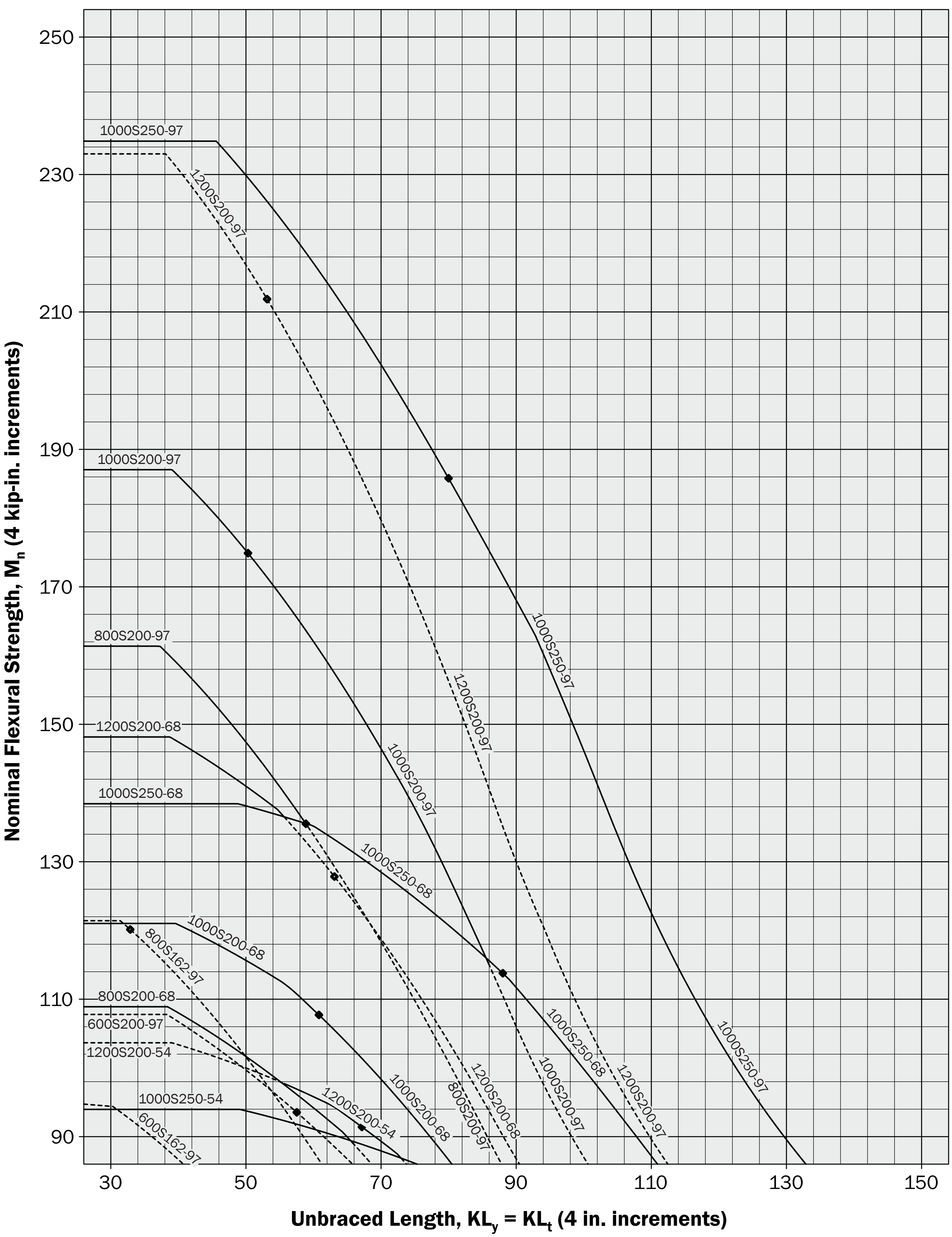


Chart II-2b

Nominal Flexural Strength
Joists/Studs
C-Sections With Lips (F_y = 50 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

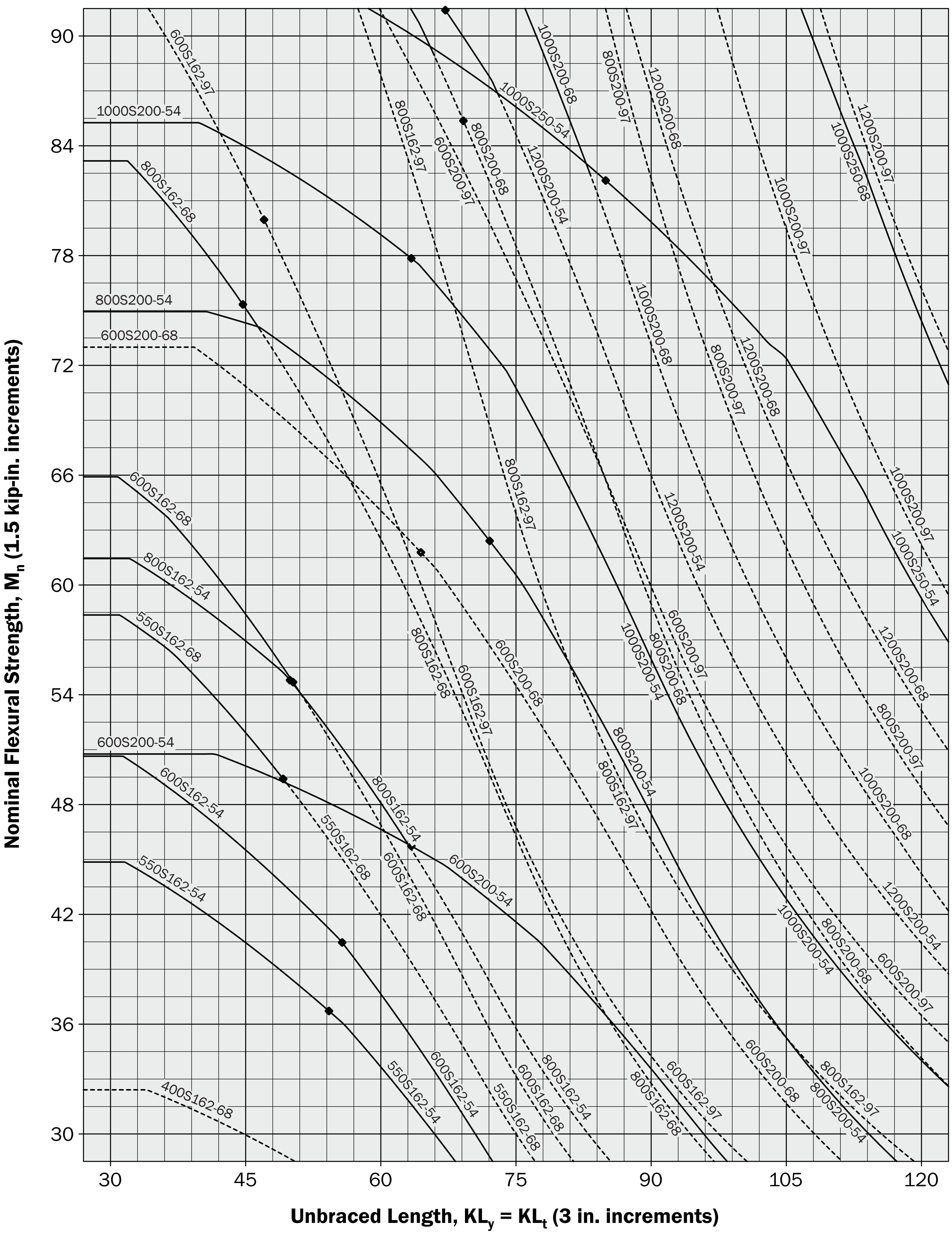


Chart II-2b

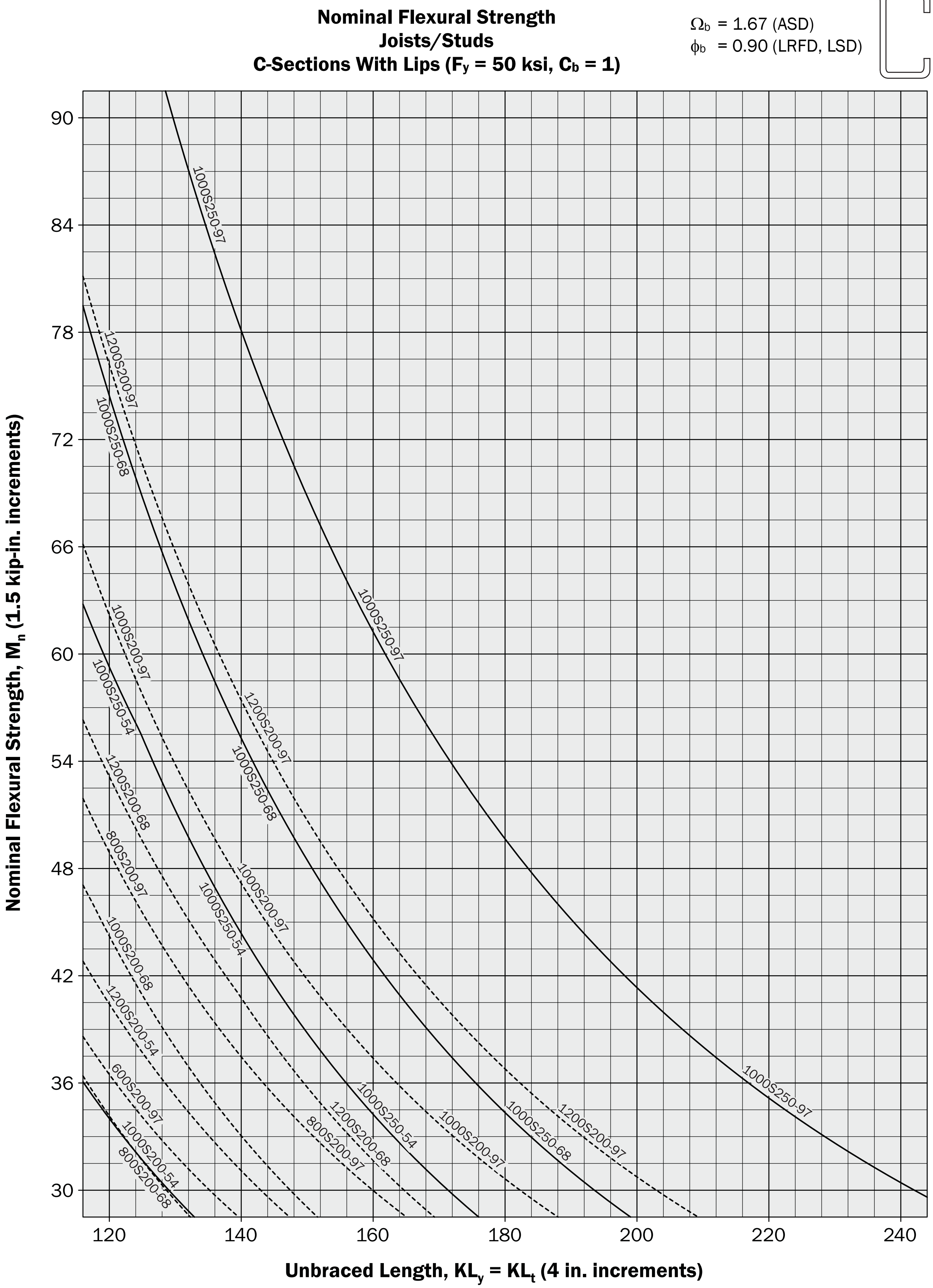

Nominal Flexural Strength
Joists/Studs
C-Sections With Lips (F_y = 50 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Nominal Flexural Strength, M_n (1.5 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (4 in. increments)
1000S250-97
1200S200-97
1000S250-68
1000S200-97
1000S250-54
1200S200-68
800S200-97
1000S200-68
1200S200-54
600S200-97
1000S200-54
800S200-68
90
84
78
72
66
60
54
48
42
36
30
120
140
160
180
200
220
240

Chart II-2b

Nominal Flexural Strength
Joists/Studs
C-Sections With Lips (F_y = 50 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

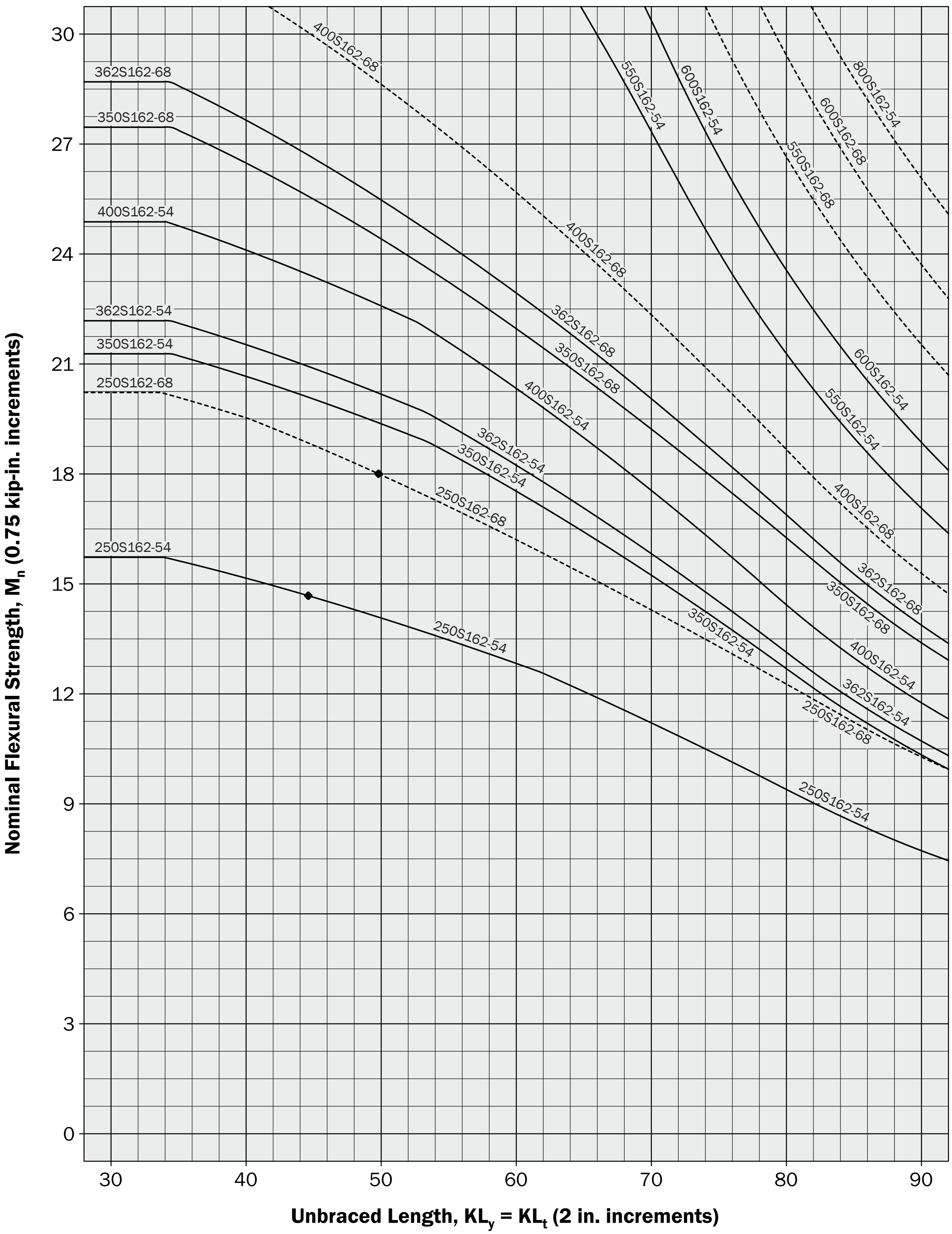


Chart II-2b

Nominal Flexural Strength
Joists/Studs
C-Sections With Lips (F_y = 50 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

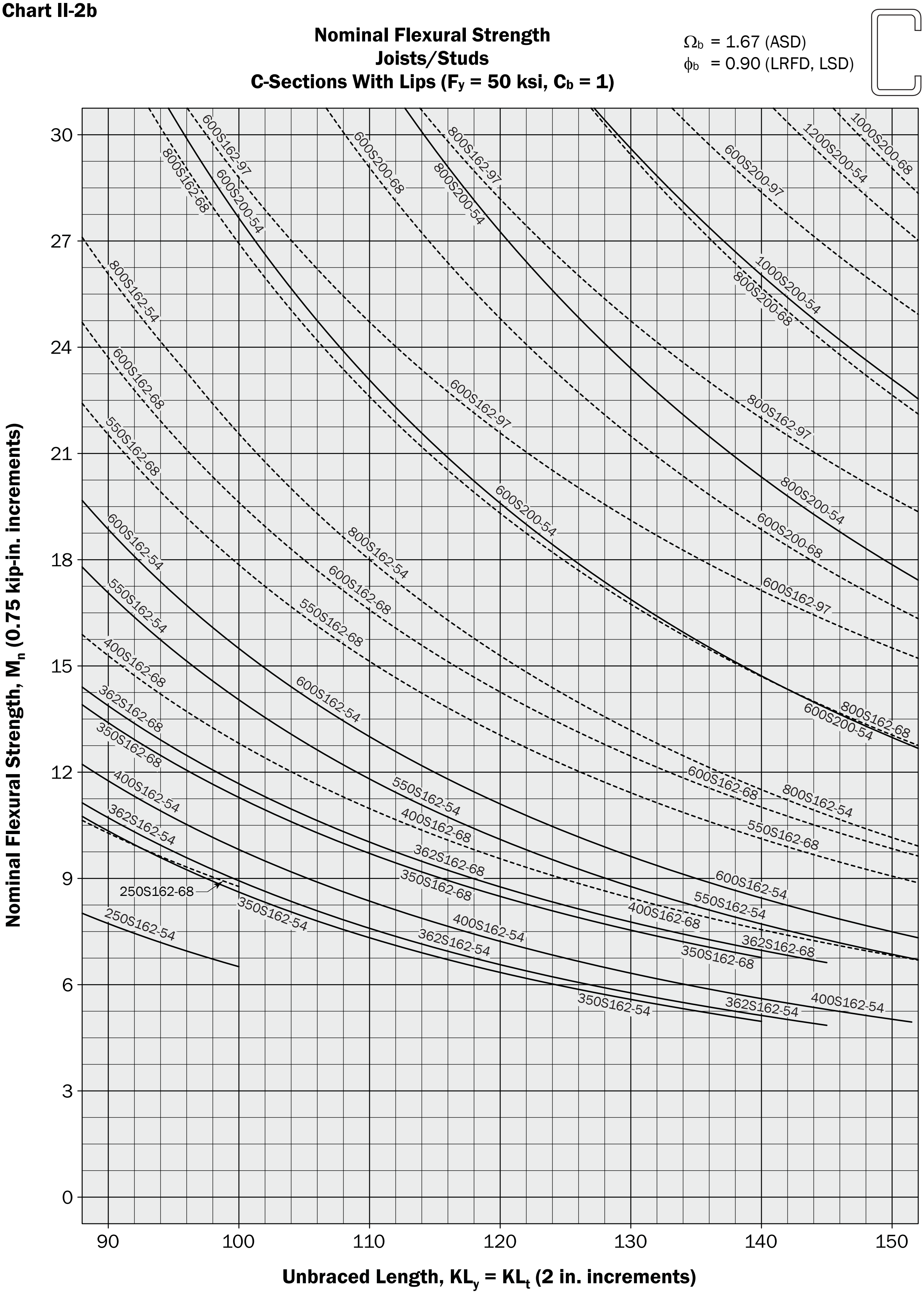


Chart II-2b

Nominal Flexural Strength
Joists/Studs
C-Sections With Lips (F_y = 50 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

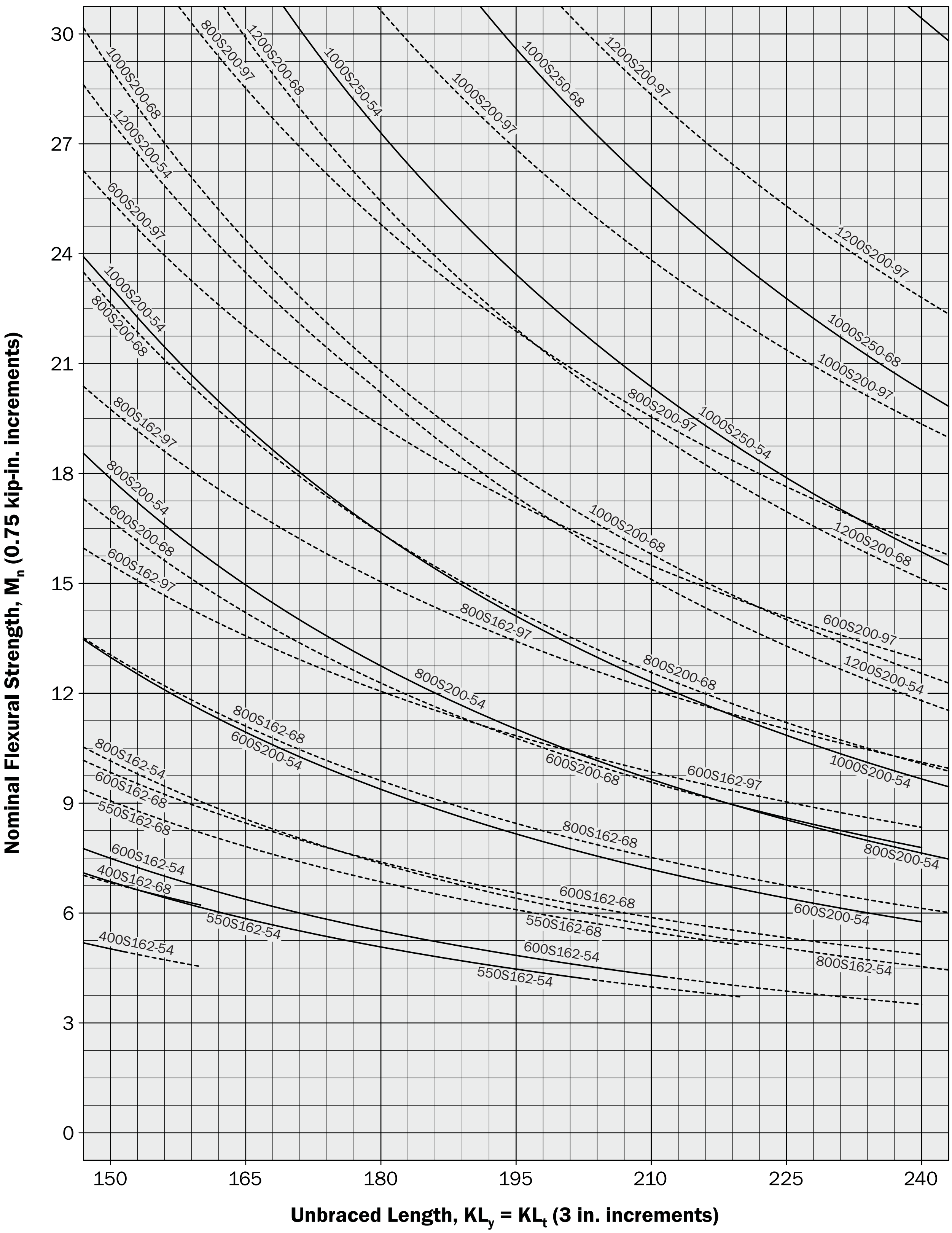


Chart II-2b

Nominal Flexural Strength
Joists/Studs
C-Sections With Lips (F_y = 50 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

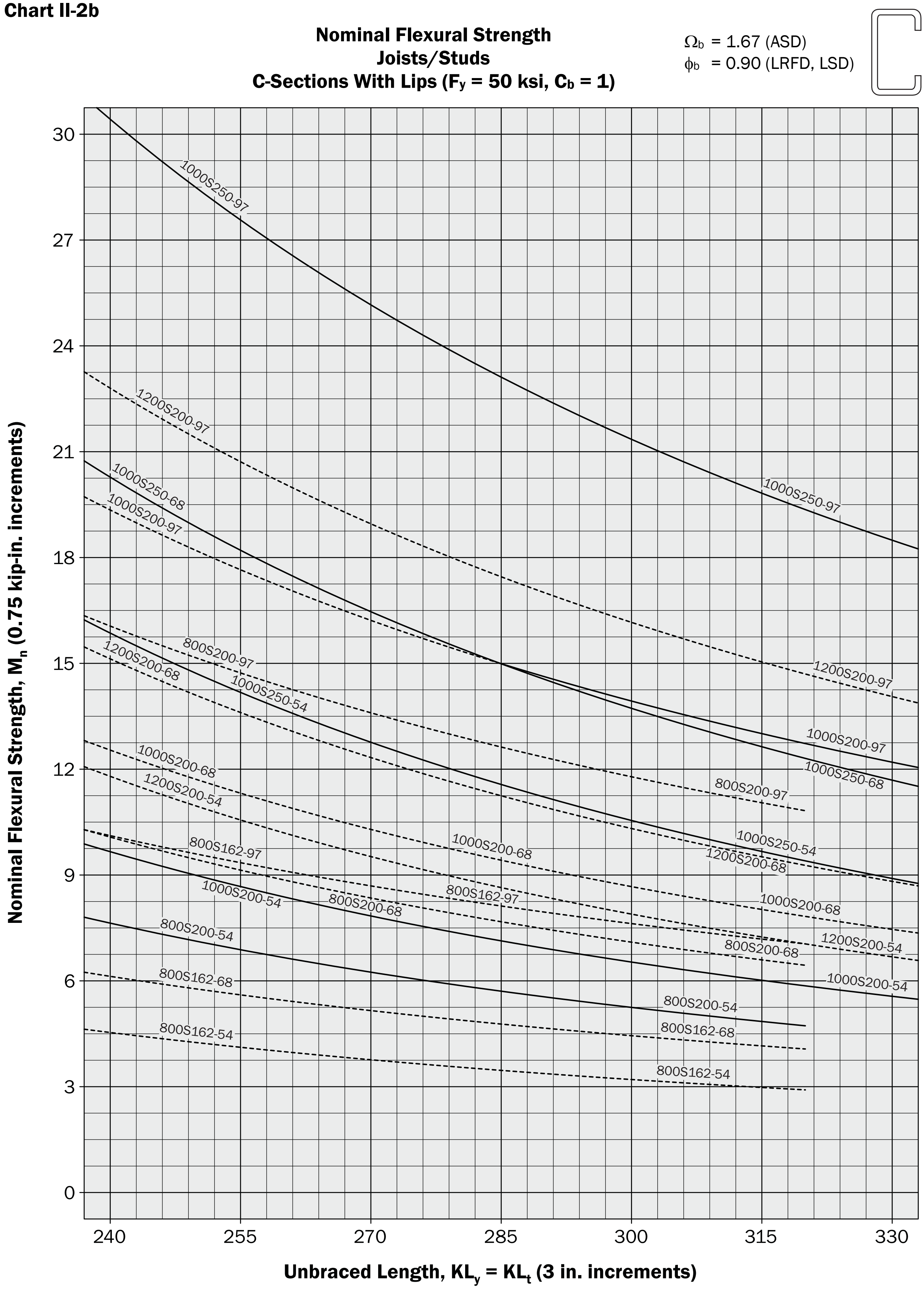


Chart II-2b

Nominal Flexural Strength
Joists/Studs
C-Sections With Lips (F_y = 50 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

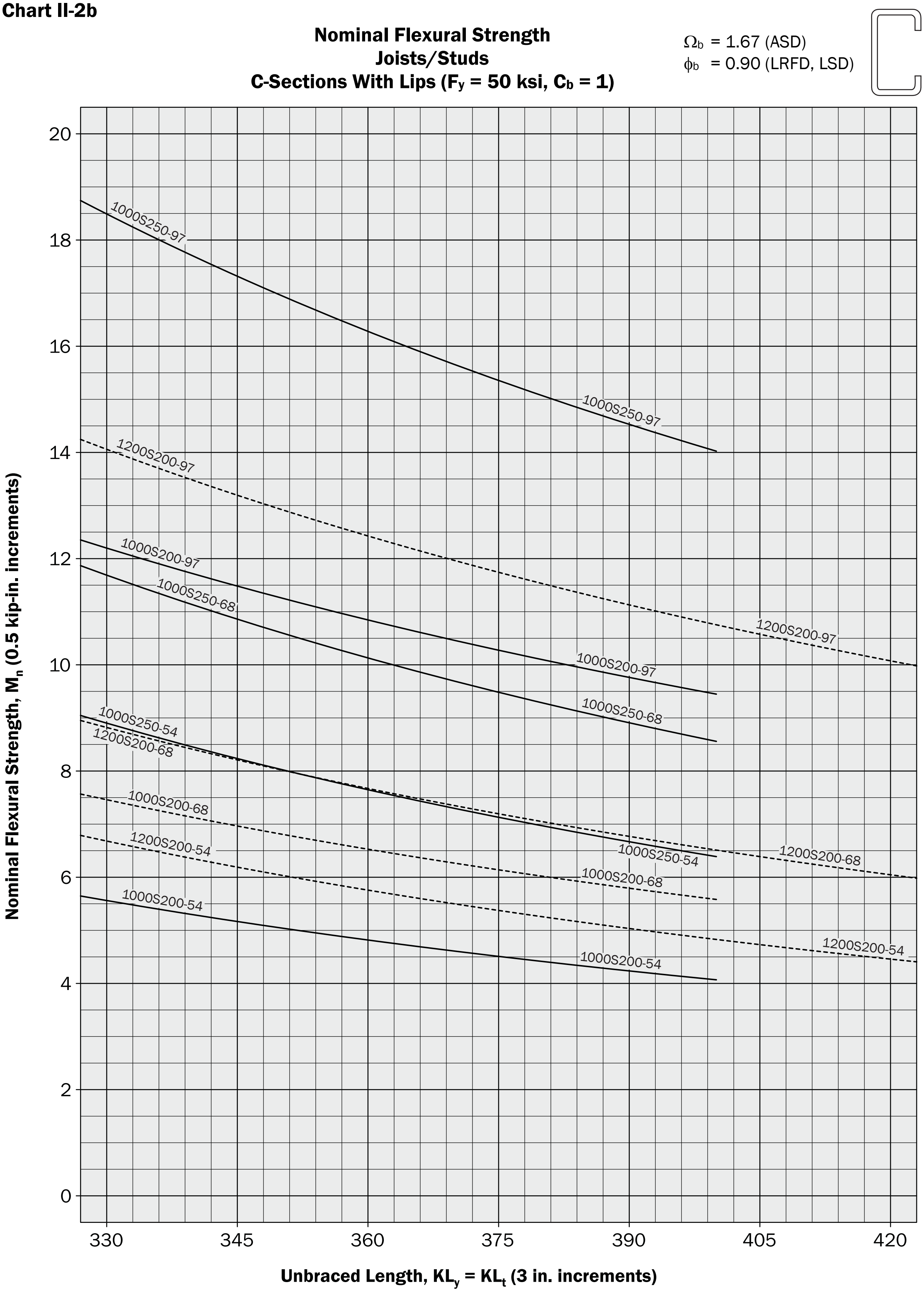


Chart II-3

Nominal Flexural Strength
Purlins and Girts
Z-Sections With Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

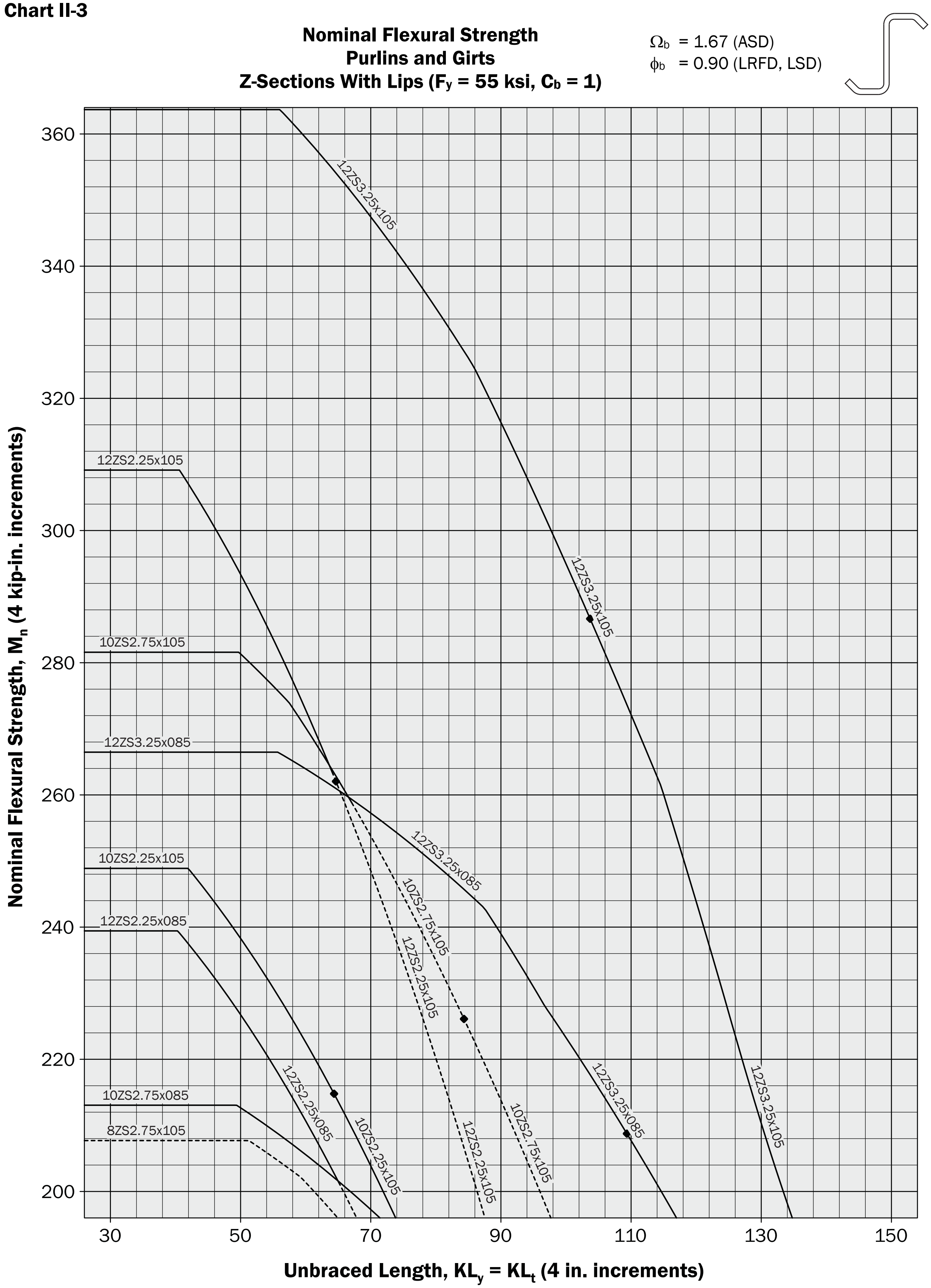


Chart II-3

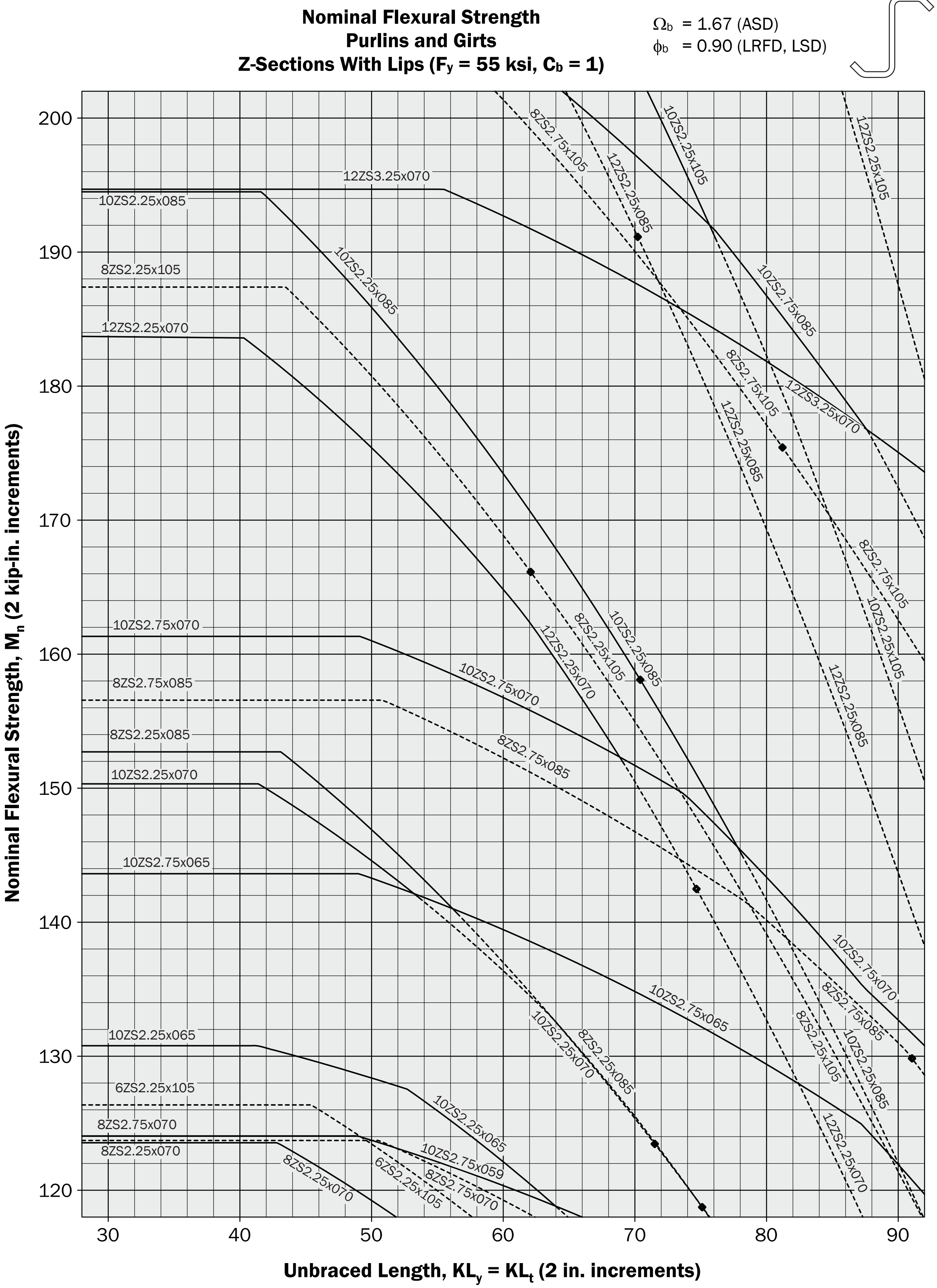

Nominal Flexural Strength
Purlins and Girts
Z-Sections With Lips (F_y = 55 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Nominal Flexural Strength, M_n (2 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (2 in. increments)
12ZS3.25x070
10ZS2.25x085
8ZS2.25x105
12ZS2.25x070
10ZS2.75x070
8ZS2.75x085
8ZS2.25x085
10ZS2.25x070
10ZS2.75x065
10ZS2.25x065
6ZS2.25x105
8ZS2.75x070
8ZS2.25x070
8ZS2.75x105
12ZS2.25x085
10ZS2.25x105
12ZS2.25x105
10ZS2.75x085
10ZS2.75x059

Chart II-3

Nominal Flexural Strength
Purlins and Girts
Z-Sections With Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

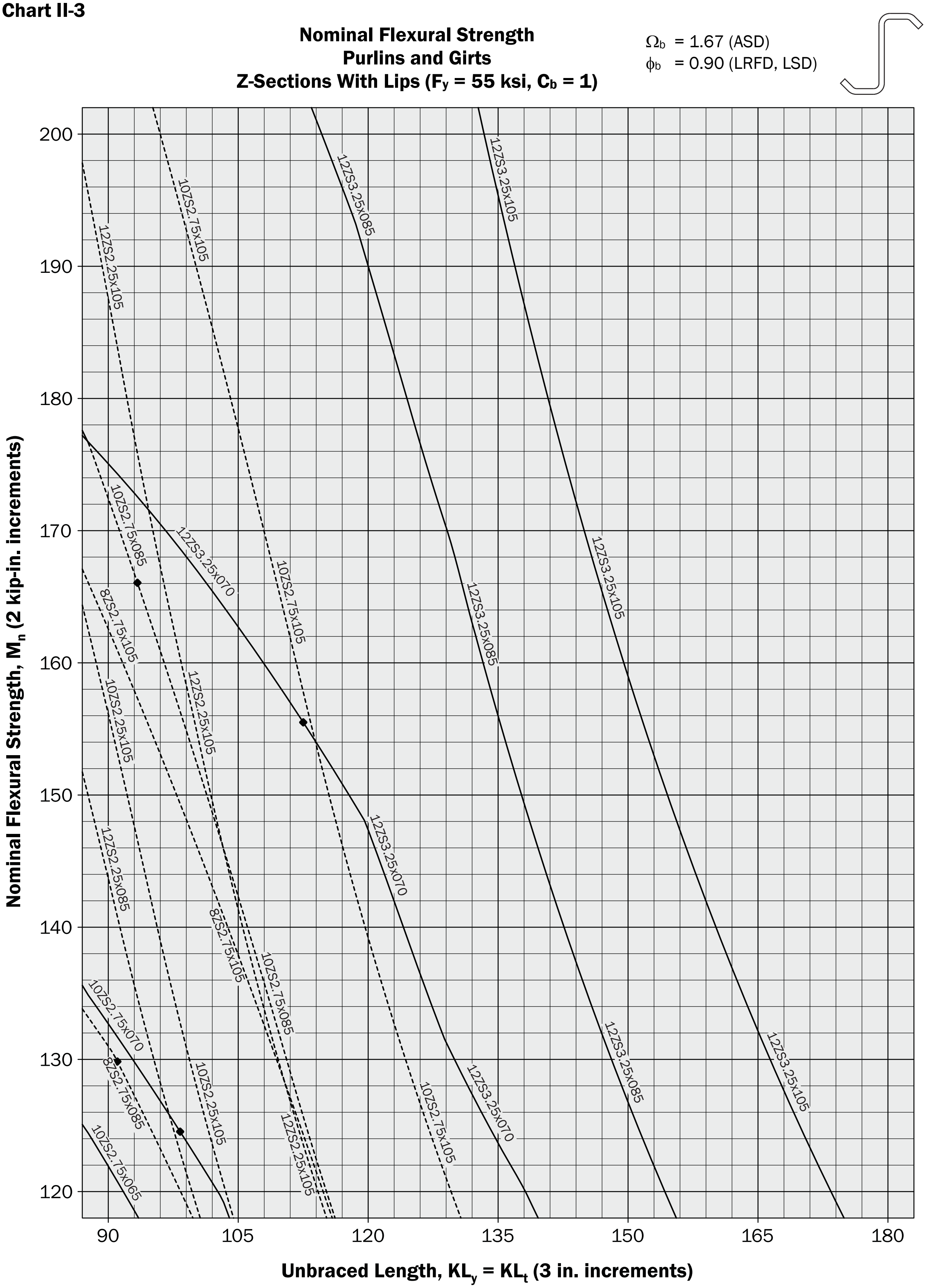
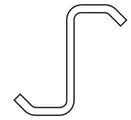


Chart II-3

Nominal Flexural Strength
Purlins and Girts
Z-Sections With Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)



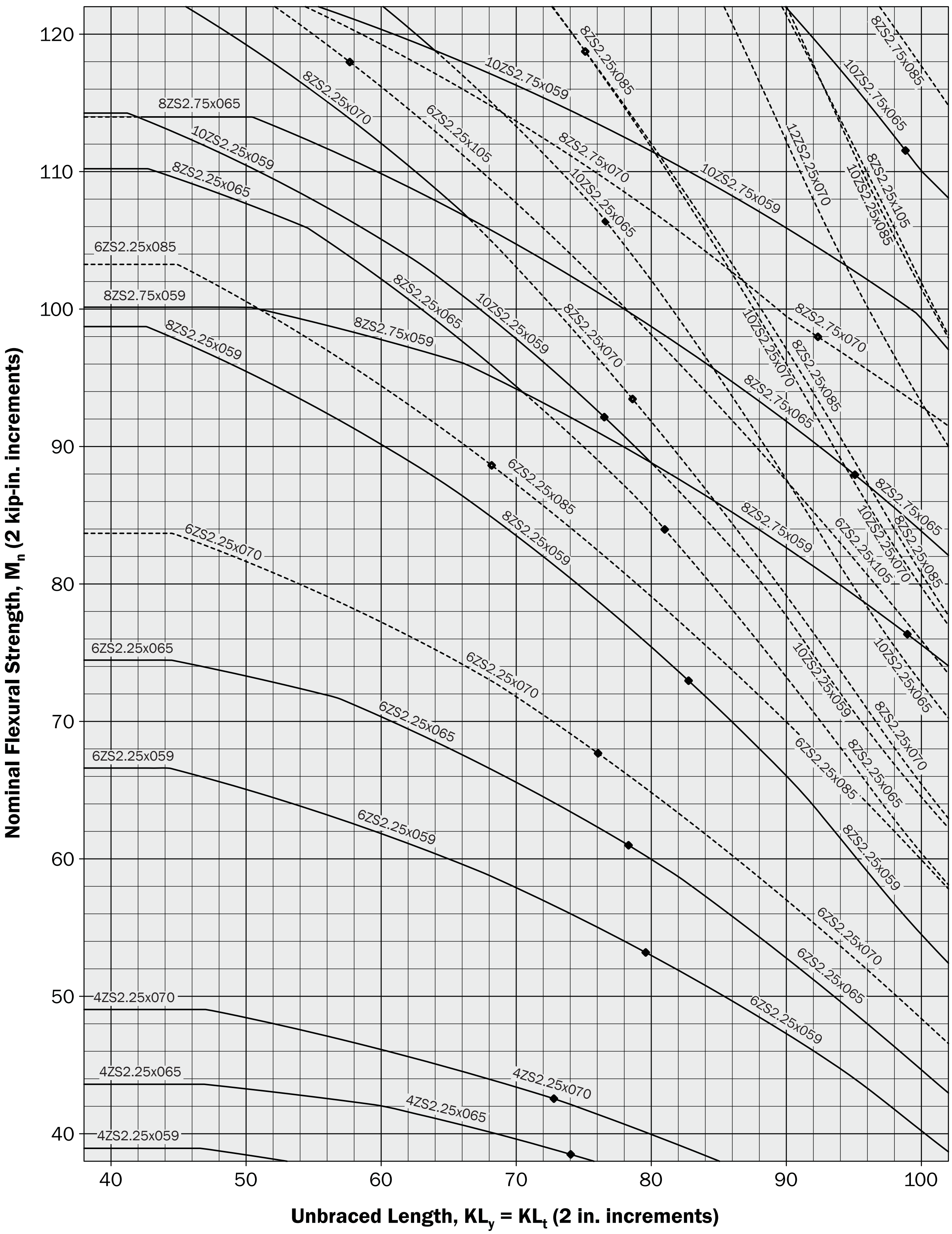
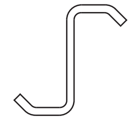


Chart II-3

Nominal Flexural Strength
Purlins and Girts
Z-Sections With Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)



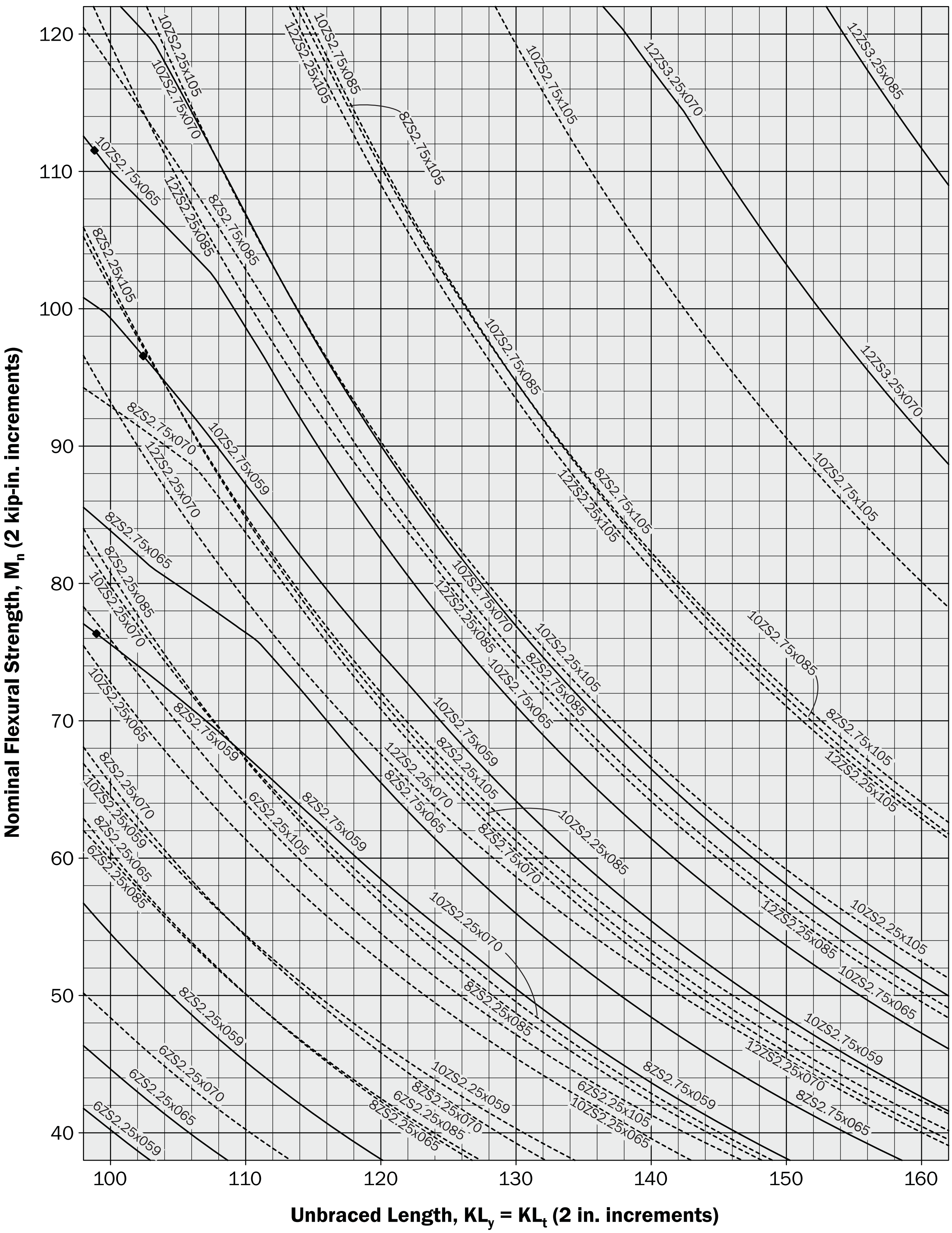


Chart II-3

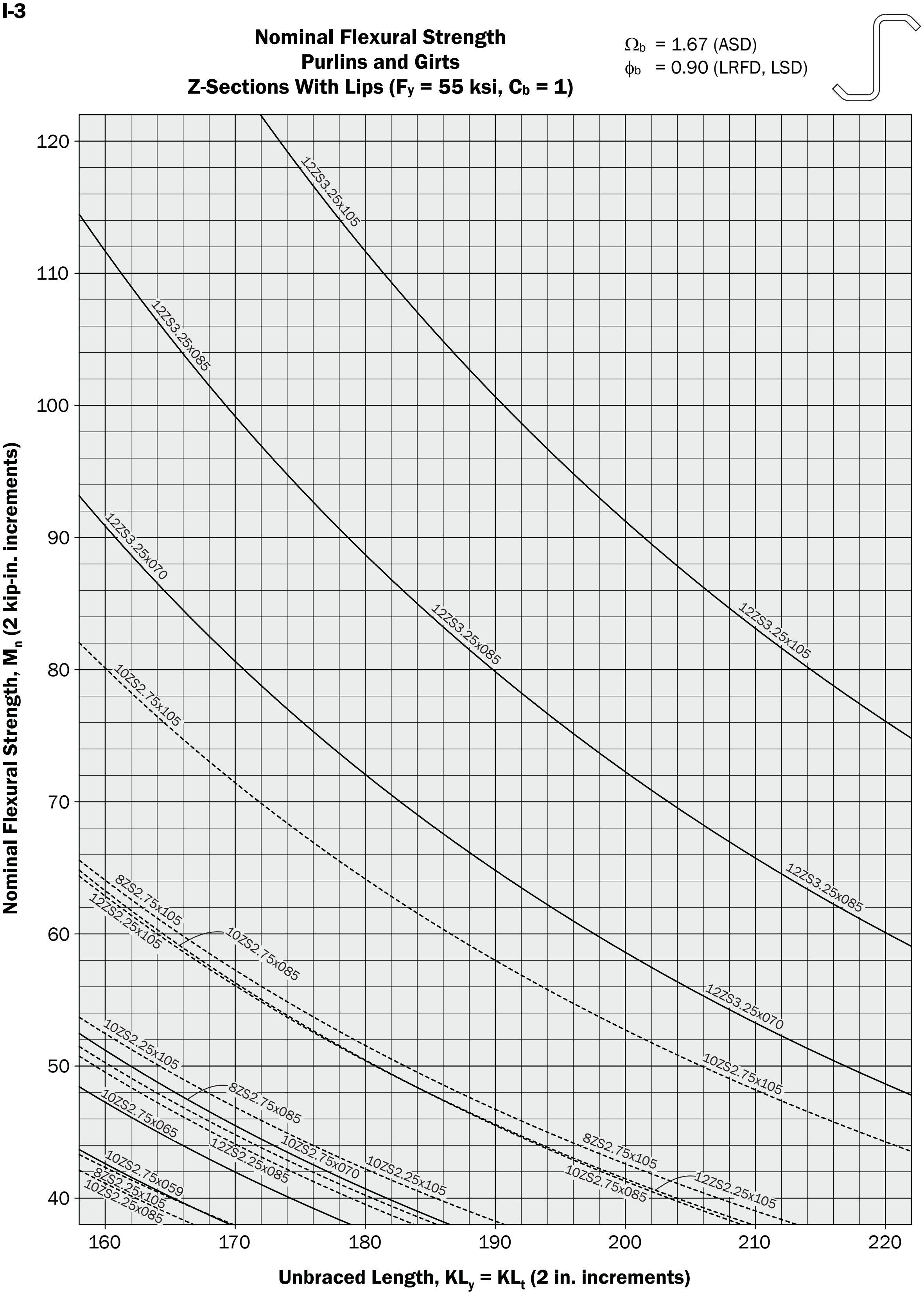

Nominal Flexural Strength
Purlins and Girts
Z-Sections With Lips (F_y = 55 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
12ZS3.25x105
12ZS3.25x085
12ZS3.25x070
10ZS2.75x105
8ZS2.75x105
12ZS2.25x105
10ZS2.75x085
10ZS2.25x105
8ZS2.75x085
10ZS2.75x065
12ZS2.25x085
10ZS2.75x070
10ZS2.25x105
10ZS2.75x059
8ZS2.25x105
10ZS2.25x085
8ZS2.75x105
10ZS2.75x085
12ZS2.25x105
Nominal Flexural Strength, M_n (2 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (2 in. increments)
40
50
60
70
80
90
100
110
120
160
170
180
190
200
210
220

Chart II-3

Nominal Flexural Strength
Purlins and Girts
Z-Sections With Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

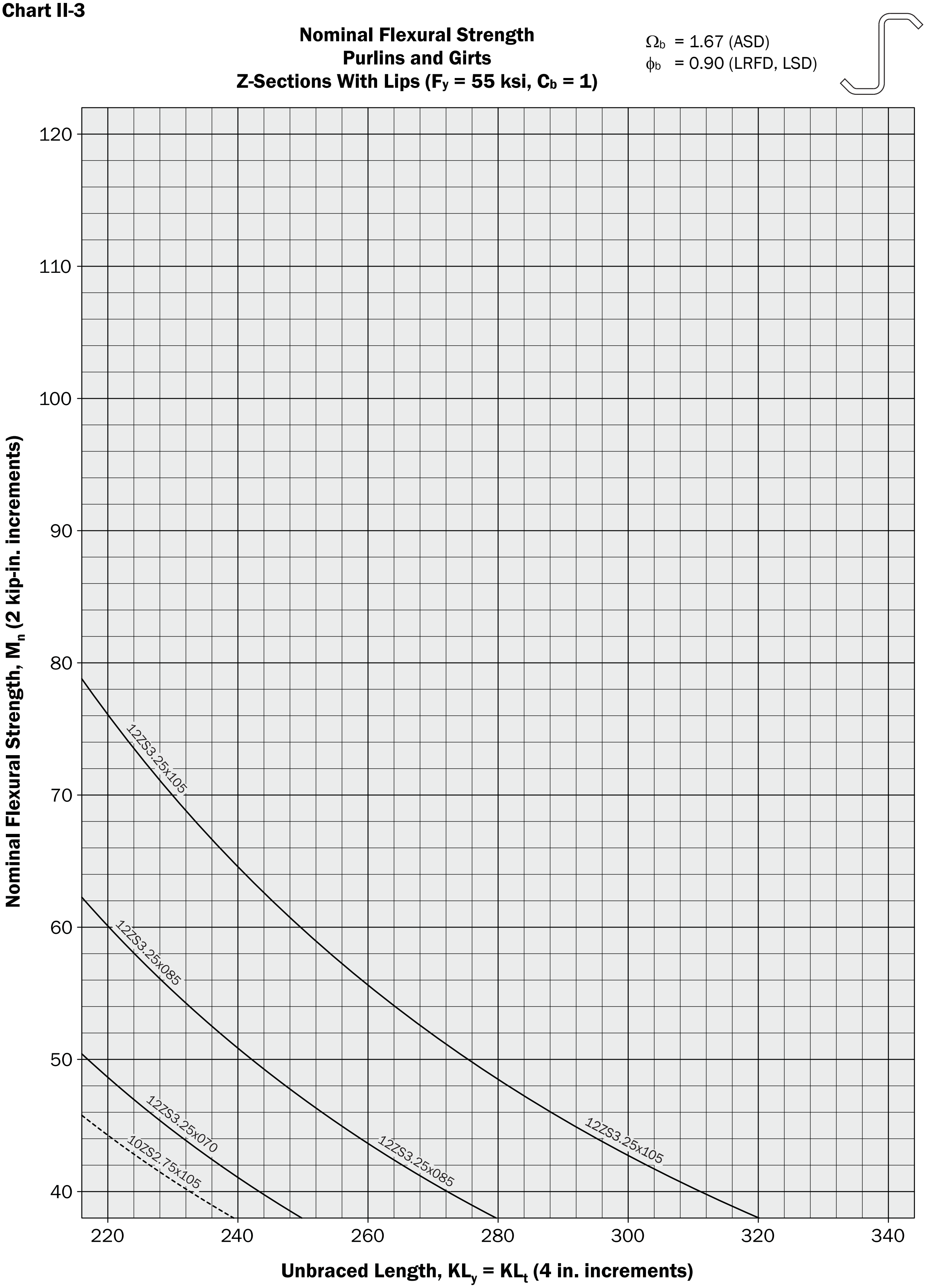


Chart II-3

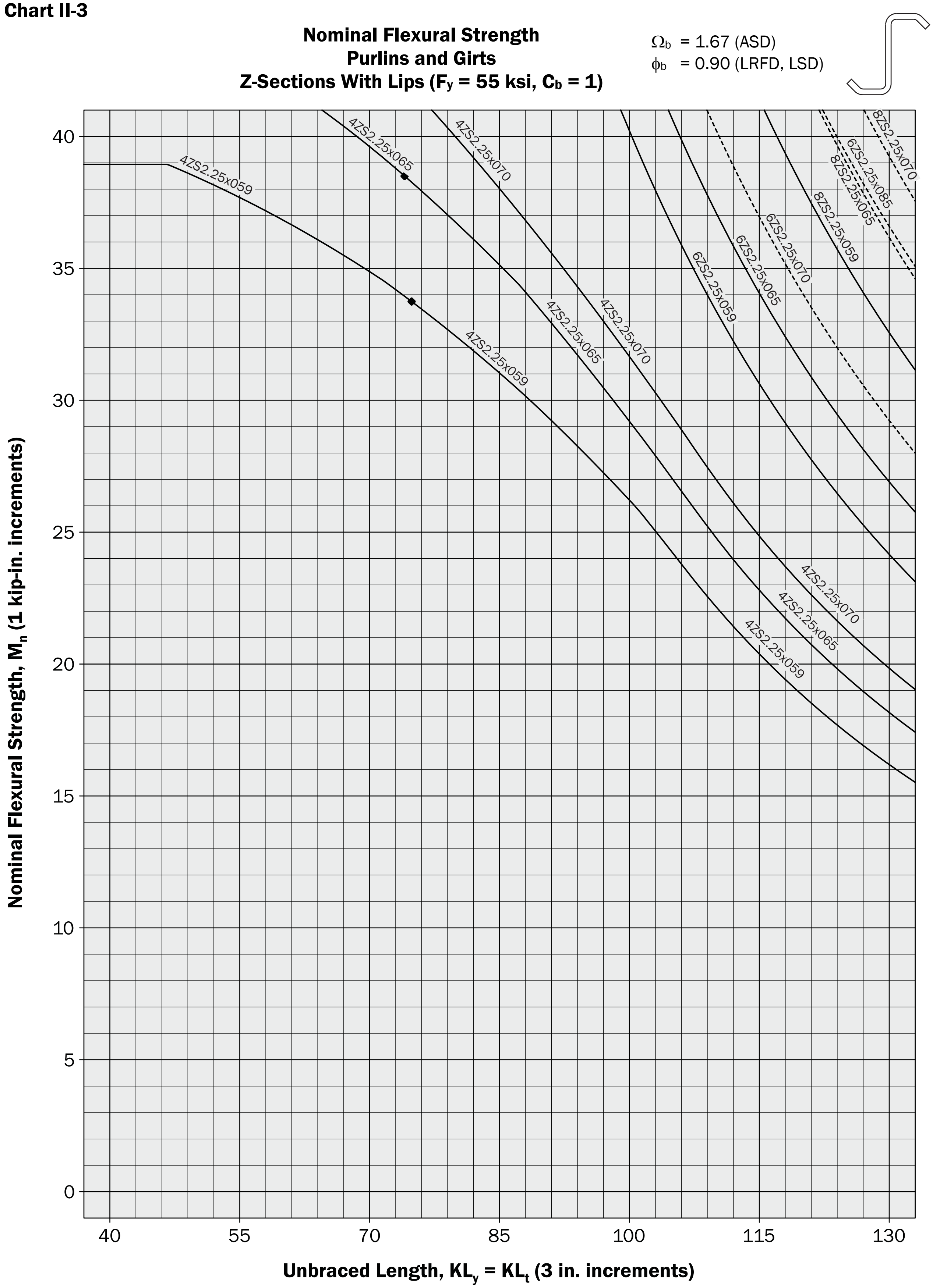
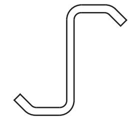

Nominal Flexural Strength
Purlins and Girts
Z-Sections With Lips (F_y = 55 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
4ZS2.25x059
4ZS2.25x065
4ZS2.25x070
6ZS2.25x059
6ZS2.25x065
6ZS2.25x070
6ZS2.25x085
8ZS2.25x059
8ZS2.25x065
8ZS2.25x070
Nominal Flexural Strength, M_n (1 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (3 in. increments)
0
5
10
15
20
25
30
35
40
40
55
70
85
100
115
130

Chart II-3

Nominal Flexural Strength
Purlins and Girts
Z-Sections With Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)



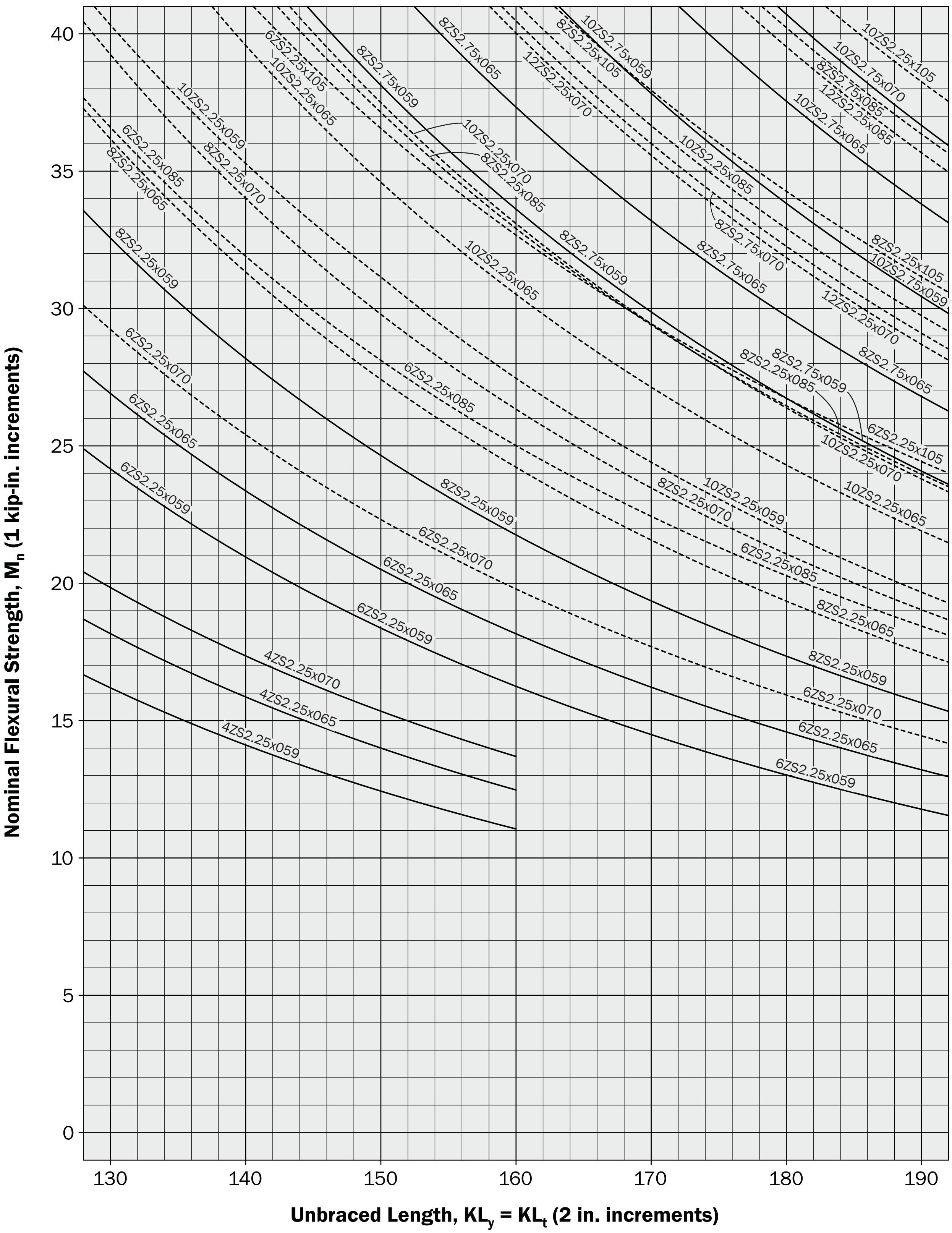


Chart II-3

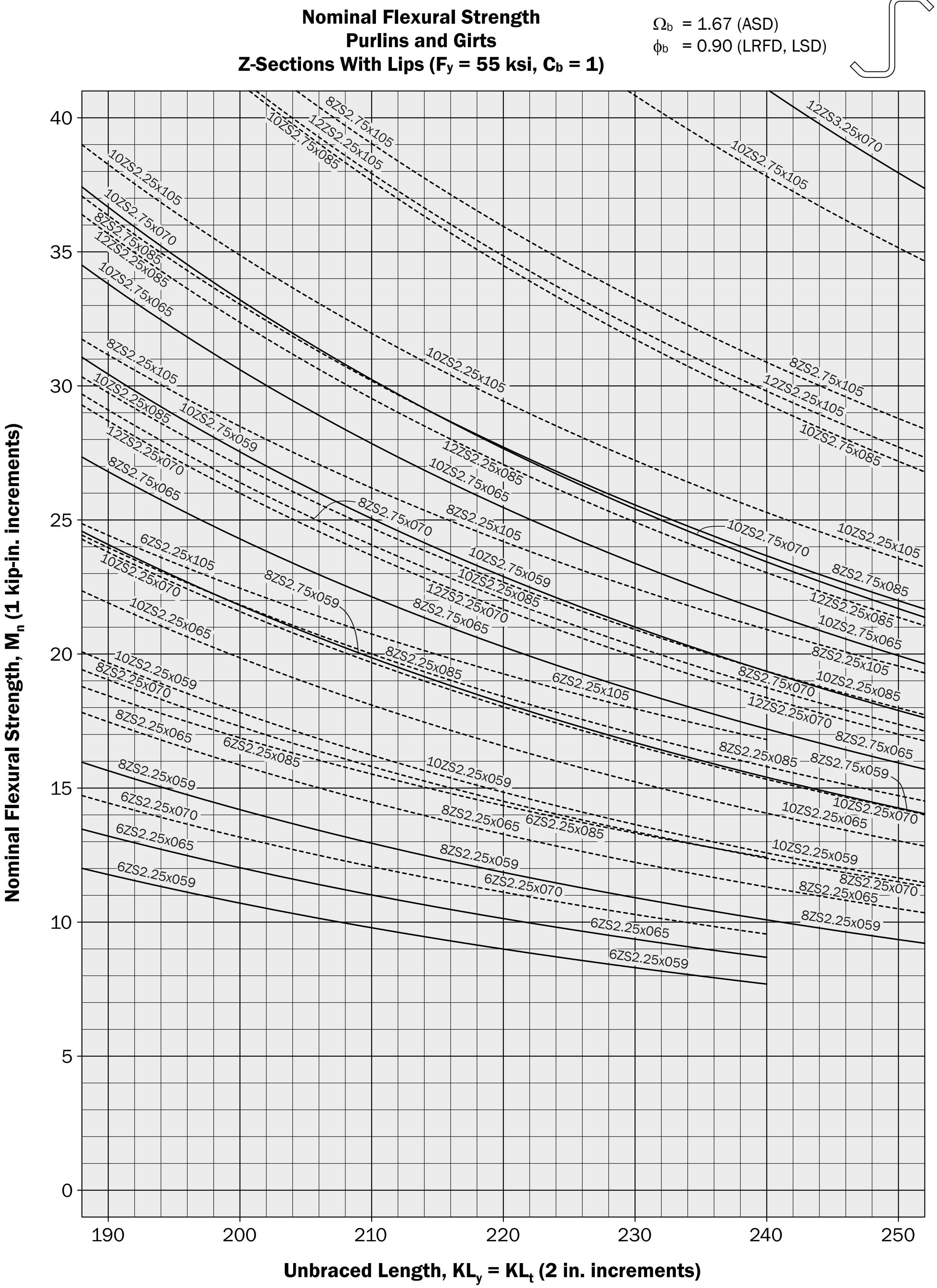

Nominal Flexural Strength
Purlins and Girts
Z-Sections With Lips (F_y = 55 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)
Nominal Flexural Strength, M_n (1 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (2 in. increments)
0
5
10
15
20
25
30
35
40
190
200
210
220
230
240
250
12ZS3.25x070
10ZS2.75x105
8ZS2.75x105
12ZS2.25x105
10ZS2.75x085
10ZS2.25x105
10ZS2.75x070
8ZS2.75x085
12ZS2.25x085
10ZS2.75x065
8ZS2.25x105
10ZS2.25x085
10ZS2.75x059
12ZS2.25x070
8ZS2.75x065
8ZS2.75x070
6ZS2.25x105
10ZS2.25x070
8ZS2.75x059
10ZS2.25x065
8ZS2.25x085
10ZS2.25x059
8ZS2.25x070
8ZS2.25x065
6ZS2.25x085
8ZS2.25x059
6ZS2.25x070
6ZS2.25x065
6ZS2.25x059

Chart II-3

Nominal Flexural Strength
Purlins and Girts
Z-Sections With Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)

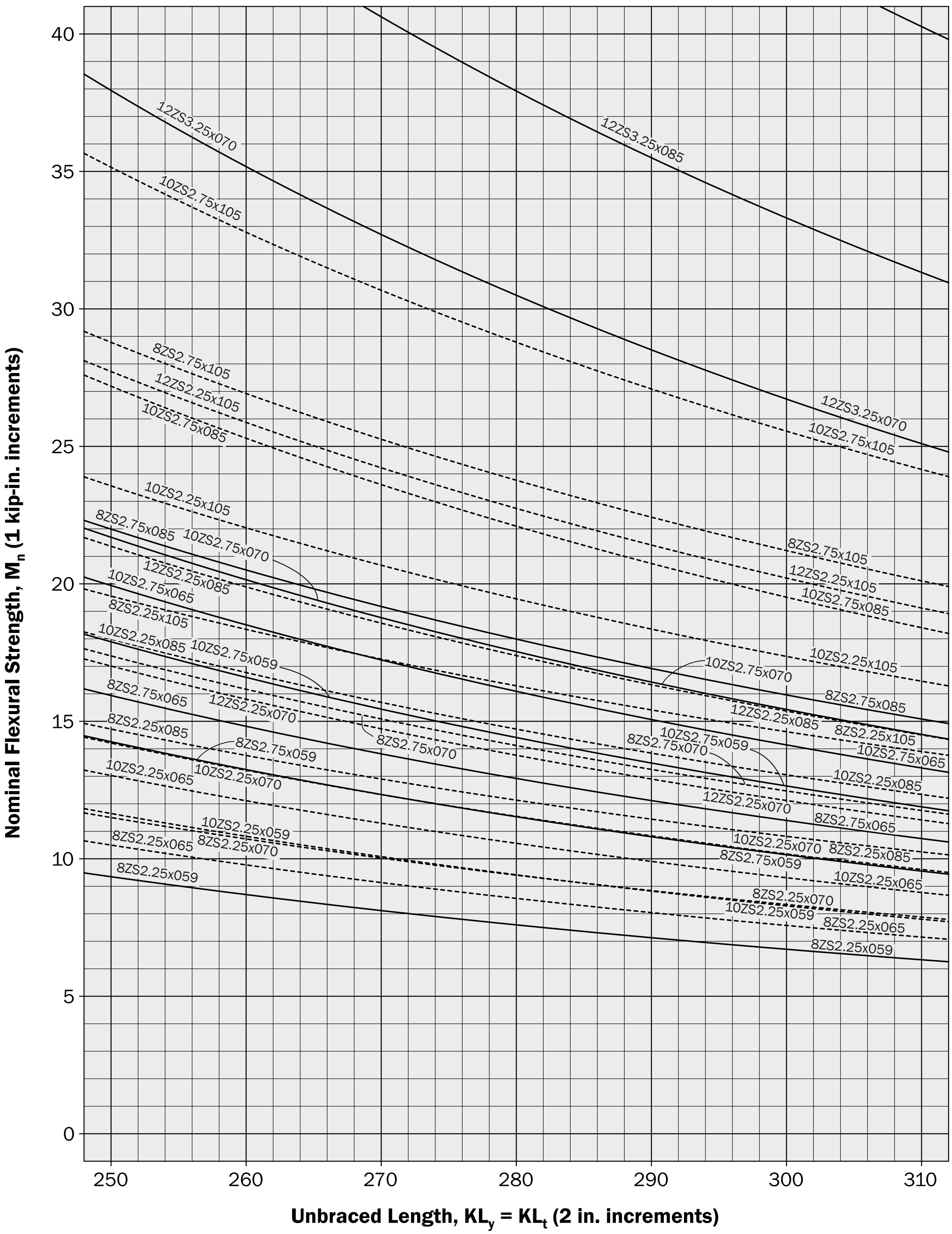
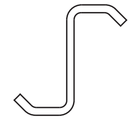
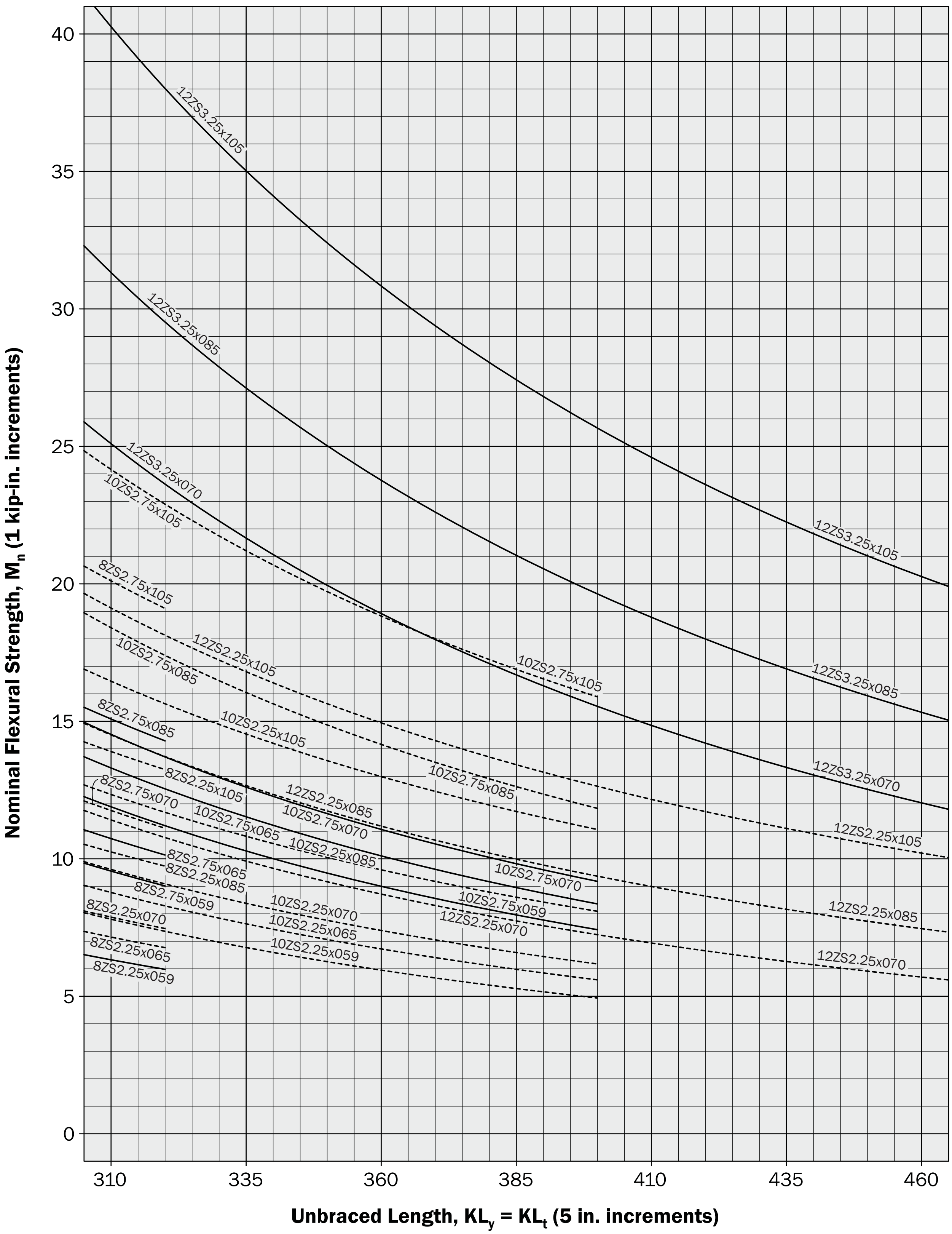


Chart II-3

Nominal Flexural Strength
Purlins and Girts
Z-Sections With Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD, LSD)





SECTION 2 – COMBINED BENDING AND SHEAR

2.1 Notes on the Tables

(a) With the exception of the joists/studs, the specific sections listed in these tables are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members.

(b) The section designations listed in these tables correspond to those for which dimensions and properties are given in Tables I-1, I-2 and I-4.

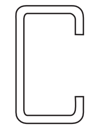
(c) Tables II-10a, II-11a and II-12a incorporate safety factors and are valid for ASD use only. Tables II-10b, II-11b and II-12b incorporate resistance factors and are valid for LRFD and LSD use only.

(d) The effects of standard factory punchouts in joists/studs have been included in Tables II-11a, and II-11b. These punchouts are considered in joists/studs with flange widths of 1.625 in. or less. Standard punchout sizes are 1.5 in. by 4.5 in. for sections with depths of 3.5 inches or more and 0.75 in. by 4.5 in. for sections with shallower depths. Punchouts are not included in the calculations for sections with web h/t ratios in excess of 200 due to the limits in Section B2.4. Those sections are marked with a "*".

2.2 Combined Shear and Bending Tables

Table II - 10a

ASD-Combined Shear and Bending[2,3]
Purlins/Girts
C-Sections With Lips



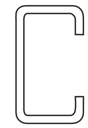
F_y = 55 ksi

Section	V [1] kips	M [1] kip-in.	Section	V [1] kips	M [1] kip-in.	Section	V [1] kips	M [1] kip-in.
12CS3.5x105	9.02	0.00	8CS3.5x105	12.2	0.00	8CS2.5x070	4.08	0.00
	8.72	55.5		11.8	31.7		3.94	17.9
	7.82	107		10.5	61.3		3.53	34.5
	6.38	152		8.61	86.6		2.88	48.8
	4.51	186		6.08	106		2.04	59.8
	2.34	207		3.15	118		1.06	66.7
	0.00	215		0.00	123		0.00	69.0
12CS3.5x085	4.77	0.00	8CS3.5x085	7.33	0.00	8CS2.5x065	3.26	0.00
	4.61	40.4		7.08	24.4		3.15	16.3
	4.13	78.1		6.35	47.1		2.82	31.5
	3.37	110		5.18	66.6		2.31	44.5
	2.39	135		3.67	81.5		1.63	54.5
	1.23	151		1.90	90.9		0.844	60.8
	0.00	156		0.00	94.1		0.00	63.0
12CS3.5x070	2.66	0.00	8CS3.5x070	4.08	0.00	8CS2.5x059	2.43	0.00
	2.57	29.1		3.94	19.3		2.35	14.4
	2.30	56.3		3.53	37.3		2.11	27.8
	1.88	79.6		2.88	52.7		1.72	39.4
	1.33	97.5		2.04	64.5		1.22	48.2
	0.688	109		1.06	72.0		0.630	53.8
	0.00	113		0.00	74.5		0.00	55.7
10CS3.5x105	10.9	0.00	8CS3.5x065	3.26	0.00	6CS2.5x105	11.7	0.00
	10.6	43.0		3.15	17.4		11.3	19.6
	9.48	83.1		2.82	33.6		10.2	37.9
	7.74	117		2.31	47.5		8.29	53.6
	5.47	144		1.63	58.1		5.86	65.7
	2.83	160		0.844	64.8		3.04	73.2
	0.00	166		0.00	67.1		0.00	75.8
10CS3.5x085	5.78	0.00	8CS3.5x059	2.43	0.00	6CS2.5x085	7.97	0.00
	5.58	33.2		2.35	14.8		7.70	15.6
	5.01	64.1		2.11	28.6		6.91	30.2
	4.09	90.6		1.72	40.4		5.64	42.7
	2.89	111		1.22	49.5		3.99	52.3
	1.50	124		0.630	55.2		2.06	58.4
	0.00	128		0.00	57.2		0.00	60.4
10CS3.5x070	3.22	0.00	8CS2.5x105	12.2	0.00	6CS2.5x070	5.41	0.00
	3.11	24.4		11.8	29.1		5.22	12.0
	2.79	47.2		10.5	56.2		4.68	23.3
	2.28	66.7		8.61	79.4		3.82	32.9
	1.61	81.7		6.08	97.3		2.70	40.3
	0.833	91.1		3.15	109		1.40	44.9
	0.00	94.3		0.00	112		0.00	46.5
10CS3.5x065	2.57	0.00	8CS2.5x085	7.33	0.00	6CS2.5x065	4.45	0.00
	2.49	21.6		7.08	23.1		4.30	11.0
	2.23	41.7		6.35	44.6		3.85	21.2
	1.82	59.0		5.18	63.1		3.14	29.9
	1.29	72.2		3.67	77.3		2.22	36.7
	0.666	80.6		1.90	86.2		1.15	40.9
	0.00	83.4		0.00	89.3		0.00	42.4

Table II - 10a

ASD-Combined Shear and Bending[2,3]
Purlins/Girts
C-Sections With Lips

F_y = 55 ksi



Section	V [1] kips	M [1] kip-in.	Section	V [1] kips	M [1] kip-in.	Section	V [1] kips	M [1] kip-in.
6CS2.5x059	3.32	0.00	4CS2.5x085	6.06	0.00	4CS2.5x065	4.66	0.00
	3.21	9.83		5.85	9.17		4.50	6.39
	2.87	19.0		5.25	17.7		4.04	12.4
	2.35	26.8		4.28	25.0		3.30	17.5
	1.66	32.9		3.03	30.7		2.33	21.4
	0.859	36.7		1.57	34.2		1.21	23.9
	0.00	38.0		0.00	35.4		0.00	24.7
4CS2.5x105	7.40	0.00	4CS2.5x070	5.03	0.00	4CS2.5x059	3.84	0.00
	7.14	11.4		4.86	7.03		3.71	5.72
	6.40	22.0		4.36	13.6		3.33	11.1
	5.23	31.1		3.56	19.2		2.72	15.6
	3.70	38.1		2.52	23.5		1.92	19.1
	1.91	42.5		1.30	26.3		0.995	21.3
	0.00	44.0		0.00	27.2		0.00	22.1

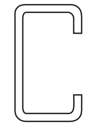
Notes:

1. Shear and moment strengths have been divided by the appropriate safety factors. This table is for ASD use only.
2. Shear strengths have been calculated assuming no transverse stiffeners.
3. Linear interpolation between values is permitted.

Table II – 10b

LRFD/LSD-Combined Shear and Bending[2,3]
Purlins/Girts
C-Sections With Lips

F_y = 55 ksi



Section	$\overline{V}$[1] kips		$\overline{M}$[1] kip-in.	Section	$\overline{V}$[1] kips		$\overline{M}$[1] kip-in.
	LRFD	LSD			LRFD	LSD	
12CS3.5x105	13.7	11.6	0.00	8CS3.5x105	18.5	15.6	0.00
	13.2	11.2	83.4		17.9	15.0	47.7
	11.9	10.0	161		16.0	13.5	92.1
	9.70	8.17	228		13.1	11.0	130
	6.86	5.78	279		9.25	7.79	159
	3.55	2.99	311		4.79	4.03	178
	0.00	0.00	322		0.00	0.00	184
12CS3.5x085	7.25	6.11	0.00	8CS3.5x085	11.1	9.38	0.00
	7.00	5.90	60.8		10.8	9.06	36.6
	6.28	5.29	117		9.65	8.13	70.7
	5.13	4.32	166		7.88	6.63	100
	3.63	3.05	203		5.57	4.69	123
	1.88	1.58	227		2.88	2.43	137
	0.00	0.00	235		0.00	0.00	141
12CS3.5x070	4.04	3.40	0.00	8CS3.5x070	6.20	5.22	0.00
	3.90	3.29	43.8		5.99	5.04	29.0
	3.50	2.95	84.6		5.37	4.52	56.0
	2.86	2.41	120		4.38	3.69	79.2
	2.02	1.70	146		3.10	2.61	97.0
	1.05	0.880	163		1.60	1.35	108
	0.00	0.00	169		0.00	0.00	112
10CS3.5x105	16.6	14.0	0.00	8CS3.5x065	4.96	4.17	0.00
	16.1	13.5	64.6		4.79	4.03	26.1
	14.4	12.1	125		4.29	3.61	50.4
	11.8	9.90	177		3.50	2.95	71.3
	8.32	7.00	216		2.48	2.09	87.4
	4.30	3.62	241		1.28	1.08	97.5
	0.00	0.00	250		0.00	0.00	101
10CS3.5x085	8.79	7.40	0.00	8CS3.5x059	3.70	3.12	0.00
	8.49	7.15	49.9		3.57	3.01	22.2
	7.61	6.41	96.3		3.20	2.70	43.0
	6.21	5.23	136		2.62	2.20	60.7
	4.39	3.70	167		1.85	1.56	74.4
	2.27	1.91	186		0.958	0.806	83.0
	0.00	0.00	193		0.00	0.00	85.9
10CS3.5x070	4.89	4.12	0.00	8CS2.5x105	18.5	15.6	0.00
	4.72	3.98	36.7		17.9	15.0	43.7
	4.24	3.57	70.9		16.0	13.5	84.4
	3.46	2.91	100		13.1	11.0	119
	2.45	2.06	123		9.25	7.79	146
	1.27	1.07	137		4.79	4.03	163
	0.00	0.00	142		0.00	0.00	169
10CS3.5x065	3.91	3.29	0.00	8CS2.5x085	11.1	9.38	0.00
	3.78	3.18	32.4		10.8	9.06	34.7
	3.39	2.85	62.7		9.65	8.13	67.1
	2.77	2.33	88.6		7.88	6.63	94.9
	1.96	1.65	109		5.57	4.69	116
	1.01	0.853	121		2.88	2.43	130
	0.00	0.00	125		0.00	0.00	134

Table II – 10b

LRFD/LSD-Combined Shear and Bending[2,3]
Purlins/Girts
C-Sections With Lips

F_y = 55 ksi

Section	$\overline{V}$[1] kips LRFD	$\overline{V}$[1] kips LSD	$\overline{M}$[1] kip-in.	Section	$\overline{V}$[1] kips LRFD	$\overline{V}$[1] kips LSD	$\overline{M}$[1] kip-in.
8CS2.5x070	6.20	5.22	0.00	6CS2.5x059	5.04	4.25	0.00
	5.99	5.04	26.9		4.87	4.10	14.8
	5.37	4.52	51.9		4.37	3.68	28.5
	4.38	3.69	73.4		3.57	3.00	40.4
	3.10	2.61	89.9		2.52	2.12	49.4
	1.60	1.35	100		1.31	1.10	55.1
	0.00	0.00	104		0.00	0.00	57.1
8CS2.5x065	4.96	4.17	0.00	4CS2.5x105	11.2	9.47	0.00
	4.79	4.03	24.5		10.9	9.14	17.1
	4.29	3.61	47.3		9.74	8.20	33.1
	3.50	2.95	66.9		7.95	6.69	46.8
	2.48	2.09	82.0		5.62	4.73	57.3
	1.28	1.08	91.4		2.91	2.45	63.9
	0.00	0.00	94.7		0.00	0.00	66.2
8CS2.5x059	3.70	3.12	0.00	4CS2.5x085	9.21	7.75	0.00
	3.57	3.01	21.7		8.89	7.49	13.8
	3.20	2.70	41.9		7.97	6.71	26.6
	2.62	2.20	59.2		6.51	5.48	37.6
	1.85	1.56	72.5		4.60	3.88	46.1
	0.958	0.806	80.9		2.38	2.01	51.4
	0.00	0.00	83.7		0.00	0.00	53.2
6CS2.5x105	17.8	15.0	0.00	4CS2.5x070	7.65	6.44	0.00
	17.2	14.5	29.5		7.39	6.22	10.6
	15.4	13.0	57.0		6.62	5.58	20.4
	12.6	10.6	80.6		5.41	4.55	28.9
	8.91	7.51	98.7		3.82	3.22	35.4
	4.61	3.88	110		1.98	1.67	39.5
	0.00	0.00	114		0.00	0.00	40.9
6CS2.5x085	12.1	10.2	0.00	4CS2.5x065	7.09	5.97	0.00
	11.7	9.86	23.5		6.85	5.77	9.61
	10.5	8.84	45.4		6.14	5.17	18.6
	8.57	7.22	64.2		5.01	4.22	26.3
	6.06	5.10	78.6		3.54	2.98	32.2
	3.14	2.64	87.7		1.83	1.55	35.9
	0.00	0.00	90.8		0.00	0.00	37.1
6CS2.5x070	8.22	6.92	0.00	4CS2.5x059	5.84	4.92	0.00
	7.94	6.69	18.1		5.64	4.75	8.60
	7.12	6.00	35.0		5.06	4.26	16.6
	5.81	4.90	49.4		4.13	3.48	23.5
	4.11	3.46	60.5		2.92	2.46	28.8
	2.13	1.79	67.5		1.51	1.27	32.1
	0.00	0.00	69.9		0.00	0.00	33.2
6CS2.5x065	6.76	5.69	0.00				
	6.53	5.50	16.5				
	5.85	4.93	31.8				
	4.78	4.03	45.0				
	3.38	2.85	55.1				
	1.75	1.47	61.5				
	0.00	0.00	63.7				

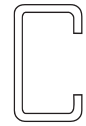
Notes:

1. Shear and moment strengths [resistances] have been multiplied by the appropriate resistance factors. This table is for LRFD and LSD use only.
2. Shear strengths [resistances] have been calculated assuming no transverse stiffeners.
3. Linear interpolation between values is permitted.

Table II - 11a

F_y = 33 ksi
F_y = 50 ksi

ASD-Combined Shear and Bending[2,3]
Joists/Studs
C-Sections With Lips

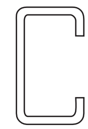


Section	V [1] kips	M [1] kip-in.	Section	V [1] kips	M [1] kip-in.	Section	V [1] kips	M [1] kip-in.
1200S200-97	8.15	0.00	1000S200-97	9.86	0.00	800S200-43	1.05	0.00
	7.87	36.1		9.53	29.0		1.01	6.61
	7.05	69.8		8.54	56.0		0.910	12.8
	5.76	98.7		6.97	79.2		0.743	18.1
	4.07	121		4.93	97.0		0.525	22.1
	2.11	135		2.55	108		0.272	24.7
	0.00	140		0.00	112		0.00	25.5
1200S200-68	2.77	0.00	1000S200-68	3.35	0.00	800S200-33*	0.474	0.00
	2.68	23.0		3.23	18.8		0.458	4.17
	2.40	44.4		2.90	36.2		0.410	8.06
	1.96	62.7		2.37	51.2		0.335	11.4
	1.38	76.8		1.67	62.8		0.237	14.0
	0.717	85.7		0.866	70.0		0.123	15.6
	0.00	88.7		0.00	72.5		0.00	16.1
1200S200-54*	1.38	0.00	1000S200-54	1.66	0.00	800S162-97	5.94	0.00
	1.33	16.1		1.60	13.2		5.74	18.8
	1.19	31.0		1.44	25.5		5.14	36.4
	0.974	43.9		1.17	36.1		4.20	51.4
	0.688	53.8		0.830	44.2		2.97	63.0
	0.356	60.0		0.430	49.3		1.54	70.2
	0.00	62.1		0.00	51.1		0.00	72.7
1000S250-97	9.86	0.00	1000S200-43*	0.836	0.00	800S162-68	3.37	0.00
	9.53	36.4		0.807	7.52		3.25	12.9
	8.54	70.3		0.724	14.5		2.92	24.9
	6.97	99.4		0.591	20.5		2.38	35.2
	4.93	122		0.418	25.2		1.68	43.1
	2.55	136		0.216	28.1		0.871	48.1
	0.00	141		0.00	29.1		0.00	49.8
1000S250-68	3.35	0.00	800S200-97	10.9	0.00	800S162-54	2.09	0.00
	3.23	21.5		10.5	25.0		2.02	9.52
	2.90	41.4		9.43	48.3		1.81	18.4
	2.37	58.6		7.70	68.3		1.48	26.0
	1.67	71.8		5.44	83.7		1.05	31.9
	0.866	80.1		2.82	93.3		0.541	35.5
	0.00	82.9		0.00	96.6		0.00	36.8
1000S250-54	1.66	0.00	800S200-68	4.22	0.00	800S162-43	1.05	0.00
	1.60	14.6		4.08	16.9		1.01	5.21
	1.44	28.1		3.65	32.6		0.910	10.1
	1.17	39.8		2.98	46.1		0.743	14.2
	0.830	48.7		2.11	56.5		0.525	17.4
	0.430	54.4		1.09	63.0		0.272	19.5
	0.00	56.3		0.00	65.2		0.00	20.1
1000S250-43*	0.836	0.00	800S200-54	2.09	0.00	800S162-33*	0.474	0.00
	0.807	8.27		2.02	11.6		0.458	3.63
	0.724	16.0		1.81	22.4		0.410	7.01
	0.591	22.6		1.48	31.7		0.335	9.92
	0.418	27.7		1.05	38.9		0.237	12.1
	0.216	30.9		0.541	43.3		0.123	13.5
	0.00	32.0		0.00	44.9		0.00	14.0

Table II - 11a

F_y = 33 ksi
F_y = 50 ksi

ASD-Combined Shear and Bending[2,3]
Joists/Studs
C-Sections With Lips



Section	V [1] kips	M [1] kip-in.	Section	V [1] kips	M [1] kip-in.	Section	V [1] kips	M [1] kip-in.
600S200-97	10.5	0.00	600S162-54	1.95	0.00	400S162-68	1.36	0.00
	10.1	16.7		1.88	7.85		1.31	5.02
	9.07	32.3		1.69	15.2		1.17	9.70
	7.40	45.6		1.38	21.4		0.959	13.7
	5.24	55.9		0.973	26.3		0.678	16.8
	2.71	62.3		0.504	29.3		0.351	18.7
	0.00	64.5		0.00	30.3		0.00	19.4
600S200-68	5.35	0.00	600S162-43	1.24	0.00	400S162-54	1.22	0.00
	5.17	11.3		1.20	4.32		1.18	3.86
	4.63	21.9		1.07	8.34		1.06	7.45
	3.78	30.9		0.877	11.8		0.864	10.5
	2.67	37.9		0.620	14.4		0.611	12.9
	1.38	42.2		0.321	16.1		0.316	14.4
	0.00	43.7		0.00	16.7		0.00	14.9
600S200-54	2.82	0.00	600S162-33	0.638	0.00	400S162-43	0.809	0.00
	2.73	7.87		0.616	2.95		0.782	2.13
	2.44	15.2		0.553	5.70		0.701	4.12
	2.00	21.5		0.451	8.07		0.572	5.82
	1.41	26.3		0.319	9.88		0.405	7.13
	0.731	29.4		0.165	11.0		0.209	7.95
	0.00	30.4		0.00	11.4		0.00	8.23
600S200-43	1.42	0.00	550S162-68	2.53	0.00	400S162-33	0.595	0.00
	1.37	4.46		2.44	9.04		0.575	1.53
	1.23	8.62		2.19	17.5		0.515	2.95
	1.00	12.2		1.79	24.7		0.421	4.18
	0.708	14.9		1.27	30.3		0.298	5.12
	0.366	16.7		0.655	33.8		0.154	5.71
	0.00	17.2		0.00	34.9		0.00	5.91
600S200-33	0.638	0.00	550S162-54	1.88	0.00	362S162-68	1.00	0.00
	0.616	3.18		1.82	6.95		0.970	4.45
	0.553	6.14		1.63	13.4		0.869	8.59
	0.451	8.68		1.33	19.0		0.710	12.2
	0.319	10.6		0.940	23.3		0.502	14.9
	0.165	11.9		0.487	25.9		0.260	16.6
	0.00	12.3		0.00	26.9		0.00	17.2
600S162-97	3.80	0.00	550S162-43	1.20	0.00	362S162-54	1.02	0.00
	3.68	14.7		1.16	3.83		0.982	3.44
	3.30	28.4		1.04	7.40		0.880	6.64
	2.69	40.1		0.848	10.5		0.719	9.39
	1.90	49.1		0.599	12.8		0.508	11.5
	0.985	54.8		0.310	14.3		0.263	12.8
	0.00	56.7		0.00	14.8		0.00	13.3
600S162-68	2.88	0.00	550S162-33	0.698	0.00	362S162-43	0.676	0.00
	2.78	10.2		0.674	2.62		0.653	1.90
	2.49	19.7		0.605	5.06		0.585	3.67
	2.04	27.9		0.494	7.15		0.478	5.19
	1.44	34.2		0.349	8.76		0.338	6.36
	0.745	38.1		0.181	9.77		0.175	7.09
	0.00	39.5		0.00	10.1		0.00	7.34

Table II - 11a

F_y = 33 ksi

F_y = 50 ksi

ASD-Combined Shear and Bending[2,3]
Joists/Studs
C-Sections With Lips

Section	V [1] kips	M [1] kip-in.	Section	V [1] kips	M [1] kip-in.	Section	V [1] kips	M [1] kip-in.
362S162-33	0.521	0.00	350S162-43	0.631	0.00	**250S162-54**	**0.564**	**0.00**
	0.503	1.37		0.610	1.82		**0.545**	**2.44**
	0.451	2.65		0.547	3.52		**0.489**	**4.71**
	0.369	3.74		0.446	4.98		**0.399**	**6.66**
	0.261	4.58		0.316	6.10		**0.282**	**8.16**
	0.135	5.11		0.163	6.81		**0.146**	**9.10**
	0.00	5.29		0.00	7.05		**0.00**	**9.42**
350S162-68	**0.897**	**0.00**	350S162-33	0.487	0.00	250S162-43	0.394	0.00
	0.866	**4.26**		0.470	1.32		0.381	1.35
	0.777	**8.22**		0.422	2.54		0.342	2.61
	0.634	**11.6**		0.344	3.60		0.279	3.69
	0.448	**14.2**		0.243	4.40		0.197	4.52
	0.232	**15.9**		0.126	4.91		0.102	5.05
	0.00	**16.4**		0.00	5.08		0.00	5.22
350S162-54	**0.947**	**0.00**	**250S162-68**	**0.519**	**0.00**	250S162-33	0.399	0.00
	0.915	**3.30**		**0.501**	**3.13**		0.385	0.920
	0.820	**6.37**		**0.449**	**6.05**		0.345	1.78
	0.670	**9.01**		**0.367**	**8.56**		0.282	2.51
	0.473	**11.0**		**0.259**	**10.5**		0.199	3.08
	0.245	**12.3**		**0.134**	**11.7**		0.103	3.43
	0.00	**12.7**		**0.00**	**12.1**		0.00	3.55

Notes:

1. Shear and moment strengths have been divided by the appropriate safety factors. This table is for ASD use only.
2. Shear strengths have been calculated assuming no transverse stiffeners.
3. Linear interpolation between values is permitted.

* Web h/t > 200, therefore bearing stiffeners are required.

Table II – 11b

F_y = 33 ksi
F_y = 50 ksi

LRFD/LSD-Combined Shear and Bending[2,3]
Joists/Studs
C-Sections With Lips

Section	$\overline{V}$[1] kips LRFD	$\overline{V}$[1] kips LSD	$\overline{M}$[1] kip-in.	Section	$\overline{V}$[1] kips LRFD	$\overline{V}$[1] kips LSD	$\overline{M}$[1] kip-in.
1200S200-97	12.4	10.4	0.00	1000S200-97	15.0	12.6	0.00
	12.0	10.1	54.3		14.5	12.2	43.6
	10.7	9.03	105		13.0	10.9	84.2
	8.75	7.37	148		10.6	8.93	119
	6.19	5.21	182		7.50	6.31	146
	3.20	2.70	203		3.88	3.27	163
	0.00	0.00	210		0.00	0.00	168
1200S200-68	4.21	3.55	0.00	1000S200-68	5.08	4.28	0.00
	4.07	3.42	34.5		4.91	4.14	28.2
	3.65	3.07	66.7		4.40	3.71	54.5
	2.98	2.51	94.3		3.60	3.03	77.0
	2.11	1.77	115		2.54	2.14	94.3
	1.09	0.918	129		1.32	1.11	105
	0.00	0.00	133		0.00	0.00	109
1200S200-54*	2.09	1.76	0.00	1000S200-54	2.52	2.13	0.00
	2.02	1.70	24.1		2.44	2.05	19.9
	1.81	1.53	46.6		2.19	1.84	38.4
	1.48	1.25	66.0		1.78	1.50	54.3
	1.05	0.881	80.8		1.26	1.06	66.5
	0.542	0.456	90.1		0.653	0.550	74.1
	0.00	0.00	93.3		0.00	0.00	76.7
1000S250-97	15.0	12.6	0.00	1000S200-43*	1.27	1.07	0.00
	14.5	12.2	54.7		1.23	1.03	11.3
	13.0	10.9	106		1.10	0.926	21.8
	10.6	8.93	149		0.898	0.756	30.9
	7.50	6.31	183		0.635	0.535	37.8
	3.88	3.27	204		0.329	0.277	42.2
	0.00	0.00	211		0.00	0.00	43.7
1000S250-68	5.08	4.28	0.00	800S200-97	16.5	13.9	0.00
	4.91	4.14	32.2		16.0	13.5	37.6
	4.40	3.71	62.3		14.3	12.1	72.6
	3.60	3.03	88.1		11.7	9.85	103
	2.54	2.14	108		8.27	6.97	126
	1.32	1.11	120		4.28	3.61	140
	0.00	0.00	125		0.00	0.00	145
1000S250-54	2.52	2.13	0.00	800S200-68	6.41	5.40	0.00
	2.44	2.05	21.9		6.20	5.22	25.4
	2.19	1.84	42.3		5.56	4.68	49.0
	1.78	1.50	59.8		4.54	3.82	69.3
	1.26	1.06	73.2		3.21	2.70	84.9
	0.653	0.550	81.7		1.66	1.40	94.7
	0.00	0.00	84.6		0.00	0.00	98.0
1000S250-43*	1.27	1.07	0.00	800S200-54	3.18	2.68	0.00
	1.23	1.03	12.4		3.07	2.58	17.5
	1.10	0.926	24.0		2.75	2.32	33.7
	0.898	0.756	34.0		2.25	1.89	47.7
	0.635	0.535	41.6		1.59	1.34	58.4
	0.329	0.277	46.4		0.822	0.693	65.1
	0.00	0.00	48.0		0.00	0.00	67.4

Table II – 11b

F_y = 33 ksi
F_y = 50 ksi

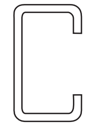
LRFD/LSD-Combined Shear and Bending[2,3]
Joists/Studs
C-Sections With Lips

Section	$\overline{V}$ [1] kips LRFD	$\overline{V}$ [1] kips LSD	$\overline{M}$ [1] kip-in.	Section	$\overline{V}$ [1] kips LRFD	$\overline{V}$ [1] kips LSD	$\overline{M}$ [1] kip-in.
800S200-43	1.60	1.34	0.00	600S200-97	15.9	13.4	0.00
	1.54	1.30	9.94		15.4	12.9	25.1
	1.38	1.16	19.2		13.8	11.6	48.5
	1.13	0.951	27.1		11.3	9.48	68.6
	0.798	0.672	33.3		7.96	6.70	84.0
	0.413	0.348	37.1		4.12	3.47	93.7
	0.00	0.00	38.4		0.00	0.00	97.0
800S200-33*	0.720	0.606	0.00	600S200-68	8.13	6.85	0.00
	0.696	0.586	6.27		7.85	6.61	17.0
	0.624	0.525	12.1		7.04	5.93	32.8
	0.509	0.429	17.1		5.75	4.84	46.5
	0.360	0.303	21.0		4.07	3.42	56.9
	0.186	0.157	23.4		2.10	1.77	63.5
	0.00	0.00	24.2		0.00	0.00	65.7
800S162-97	9.02	7.60	0.00	600S200-54	4.29	3.61	0.00
	8.72	7.34	28.3		4.14	3.49	11.8
	7.82	6.58	54.6		3.72	3.13	22.8
	6.38	5.37	77.3		3.03	2.55	32.3
	4.51	3.80	94.6		2.15	1.81	39.6
	2.34	1.97	106		1.11	0.935	44.1
	0.00	0.00	109		0.00	0.00	45.7
800S162-68	5.12	4.31	0.00	600S200-43	2.15	1.81	0.00
	4.94	4.16	19.4		2.08	1.75	6.71
	4.43	3.73	37.4		1.86	1.57	13.0
	3.62	3.05	52.9		1.52	1.28	18.3
	2.56	2.15	64.8		1.08	0.906	22.4
	1.32	1.12	72.3		0.557	0.469	25.0
	0.00	0.00	74.9		0.00	0.00	25.9
800S162-54	3.18	2.68	0.00	600S200-33	0.970	0.817	0.00
	3.07	2.58	14.3		0.937	0.789	4.78
	2.75	2.32	27.6		0.840	0.707	9.23
	2.25	1.89	39.1		0.686	0.578	13.0
	1.59	1.34	47.9		0.485	0.408	16.0
	0.822	0.693	53.4		0.251	0.211	17.8
	0.00	0.00	55.3		0.00	0.00	18.5
800S162-43	1.60	1.34	0.00	600S162-97	5.78	4.87	0.00
	1.54	1.30	7.84		5.59	4.70	22.1
	1.38	1.16	15.1		5.01	4.22	42.6
	1.13	0.951	21.4		4.09	3.44	60.3
	0.798	0.672	26.2		2.89	2.44	73.8
	0.413	0.348	29.2		1.50	1.26	82.4
	0.00	0.00	30.3		0.00	0.00	85.3
800S162-33*	0.720	0.606	0.00	600S162-68	4.38	3.68	0.00
	0.696	0.586	5.46		4.23	3.56	15.4
	0.624	0.525	10.5		3.79	3.19	29.7
	0.509	0.429	14.9		3.09	2.61	41.9
	0.360	0.303	18.3		2.19	1.84	51.4
	0.186	0.157	20.4		1.13	0.954	57.3
	0.00	0.00	21.1		0.00	0.00	59.3

Table II – 11b

F_y = 33 ksi
F_y = 50 ksi

LRFD/LSD-Combined Shear and Bending[2,3]
Joists/Studs
C-Sections With Lips



Section	$\overline{V}$[1] kips LRFD	$\overline{V}$[1] kips LSD	$\overline{M}$[1] kip-in.	Section	$\overline{V}$[1] kips LRFD	$\overline{V}$[1] kips LSD	$\overline{M}$[1] kip-in.
600S162-54	2.96	2.49	0.00	400S162-68	2.06	1.74	0.00
	2.86	2.41	11.8		1.99	1.68	7.55
	2.56	2.16	22.8		1.78	1.50	14.6
	2.09	1.76	32.2		1.46	1.23	20.6
	1.48	1.25	39.5		1.03	0.868	25.3
	0.766	0.645	44.0		0.533	0.449	28.2
	0.00	0.00	45.6		0.00	0.00	29.2
600S162-43	1.88	1.59	0.00	400S162-54	1.86	1.56	0.00
	1.82	1.53	6.49		1.79	1.51	5.80
	1.63	1.37	12.5		1.61	1.36	11.2
	1.33	1.12	17.7		1.31	1.11	15.8
	0.942	0.794	21.7		0.929	0.782	19.4
	0.488	0.411	24.2		0.481	0.405	21.6
	0.00	0.00	25.1		0.00	0.00	22.4
600S162-33	0.970	0.817	0.00	400S162-43	1.23	1.04	0.00
	0.937	0.789	4.44		1.19	1.00	3.20
	0.840	0.707	8.57		1.07	0.897	6.19
	0.686	0.578	12.1		0.870	0.733	8.75
	0.485	0.408	14.8		0.615	0.518	10.7
	0.251	0.211	16.6		0.318	0.268	12.0
	0.00	0.00	17.1		0.00	0.00	12.4
550S162-68	3.85	3.24	0.00	400S162-33	0.904	0.762	0.00
	3.72	3.13	13.6		0.874	0.736	2.30
	3.33	2.81	26.3		0.783	0.660	4.44
	2.72	2.29	37.1		0.640	0.539	6.28
	1.92	1.62	45.5		0.452	0.381	7.69
	0.996	0.839	50.7		0.234	0.197	8.58
	0.00	0.00	52.5		0.00	0.00	8.88
550S162-54	2.86	2.41	0.00	362S162-68	1.53	1.28	0.00
	2.76	2.33	10.4		1.47	1.24	6.69
	2.48	2.08	20.2		1.32	1.11	12.9
	2.02	1.70	28.5		1.08	0.908	18.3
	1.43	1.20	35.0		0.763	0.642	22.4
	0.740	0.623	39.0		0.395	0.333	24.9
	0.00	0.00	40.4		0.00	0.00	25.8
550S162-43	1.82	1.53	0.00	362S162-54	1.54	1.30	0.00
	1.76	1.48	5.76		1.49	1.26	5.17
	1.58	1.33	11.1		1.34	1.13	9.98
	1.29	1.08	15.7		1.09	0.920	14.1
	0.911	0.767	19.3		0.772	0.650	17.3
	0.472	0.397	21.5		0.400	0.337	19.3
	0.00	0.00	22.2		0.00	0.00	20.0
550S162-33	1.06	0.894	0.00	362S162-43	1.03	0.865	0.00
	1.02	0.863	3.93		0.992	0.835	2.86
	0.919	0.774	7.60		0.889	0.749	5.52
	0.750	0.632	10.7		0.726	0.612	7.81
	0.531	0.447	13.2		0.513	0.432	9.56
	0.275	0.231	14.7		0.266	0.224	10.7
	0.00	0.00	15.2		0.00	0.00	11.0

Table II – 11b

F_y = 33 ksi
F_y = 50 ksi

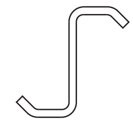
LRFD/LSD-Combined Shear and Bending[2,3]
Joists/Studs
C-Sections With Lips

Section	$\bar{V}$ [1] kips		$\bar{M}$ [1] kip-in.	Section	$\bar{V}$ [1] kips		$\bar{M}$ [1] kip-in.
	LRFD	LSD			LRFD	LSD	
362S162-33	0.792	0.667	0.00	250S162-68	0.789	0.664	0.00
	0.765	0.644	2.06		0.762	0.641	4.71
	0.686	0.578	3.98		0.683	0.575	9.10
	0.560	0.472	5.62		0.558	0.470	12.9
	0.396	0.334	6.89		0.394	0.332	15.8
	0.205	0.173	7.68		0.204	0.172	17.6
	0.00	0.00	7.95		0.00	0.00	18.2
350S162-68	1.36	1.15	0.00	250S162-54	0.858	0.722	0.00
	1.32	1.11	6.40		0.829	0.698	3.66
	1.18	0.994	12.4		0.743	0.626	7.08
	0.964	0.812	17.5		0.607	0.511	10.0
	0.682	0.574	21.4		0.429	0.361	12.3
	0.353	0.297	23.9		0.222	0.187	13.7
	0.00	0.00	24.7		0.00	0.00	14.2
350S162-54	1.44	1.21	0.00	250S162-43	0.599	0.505	0.00
	1.39	1.17	4.96		0.579	0.488	2.03
	1.25	1.05	9.58		0.519	0.437	3.93
	1.02	0.857	13.5		0.424	0.357	5.55
	0.720	0.606	16.6		0.300	0.252	6.80
	0.373	0.314	18.5		0.155	0.131	7.58
	0.00	0.00	19.2		0.00	0.00	7.85
350S162-43	0.960	0.808	0.00	250S162-33	0.606	0.510	0.00
	0.927	0.780	2.74		0.585	0.493	1.38
	0.831	0.700	5.30		0.525	0.442	2.67
	0.678	0.571	7.49		0.429	0.361	3.78
	0.480	0.404	9.17		0.303	0.255	4.63
	0.248	0.209	10.2		0.157	0.132	5.16
	0.00	0.00	10.6		0.00	0.00	5.34
350S162-33	0.740	0.623	0.00				
	0.715	0.602	1.98				
	0.641	0.540	3.82				
	0.523	0.441	5.40				
	0.370	0.312	6.62				
	0.192	0.161	7.38				
	0.00	0.00	7.64				

Notes:
1. Shear and moment strengths [resistances] have been multiplied by the appropriate resistance factors. This table is for LRFD and LSD use only.
2. Shear strengths [resistances] have been calculated assuming no transverse stiffeners.
3. Linear interpolation between values is permitted.
* Web h/t > 200, therefore bearing stiffeners are required.

Table II - 12a

ASD-Combined Shear and Bending[2,3]
Purlins/Girts
Z-Sections With Lips



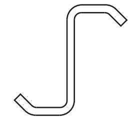
F_y = 55 ksi

Section	V [1] kips	M [1] kip-in.	Section	V [1] kips	M [1] kip-in.	Section	V [1] kips	M [1] kip-in.
12ZS3.25x105	9.02	0.00	10ZS2.75x085	5.78	0.00	10ZS2.25x065	2.57	0.00
	8.72	56.4		5.58	33.0		2.49	20.3
	7.82	109		5.01	63.7		2.23	39.2
	6.38	154		4.09	90.1		1.82	55.4
	4.51	189		2.89	110		1.29	67.9
	2.34	210		1.50	123		0.666	75.7
	0.00	218		0.00	127		0.00	78.4
12ZS3.25x085	4.77	0.00	10ZS2.75x070	3.22	0.00	10ZS2.25x059	1.92	0.00
	4.61	41.2		3.11	25.0		1.86	17.7
	4.13	79.7		2.79	48.3		1.66	34.2
	3.37	113		2.28	68.3		1.36	48.4
	2.39	138		1.61	83.7		0.961	59.3
	1.23	154		0.833	93.3		0.498	66.1
	0.00	159		0.00	96.6		0.00	68.5
12ZS3.25x070	2.66	0.00	10ZS2.75x065	2.57	0.00	8ZS2.75x105	12.2	0.00
	2.57	30.2		2.49	22.3		11.8	32.2
	2.30	58.3		2.23	43.0		10.5	62.2
	1.88	82.4		1.82	60.9		8.61	87.9
	1.33	101		1.29	74.6		6.08	108
	0.688	113		0.666	83.2		3.15	120
	0.00	117		0.00	86.1		0.00	124
12ZS2.25x105	9.02	0.00	10ZS2.75x059	1.92	0.00	8ZS2.75x085	7.33	0.00
	8.72	47.9		1.86	19.2		7.08	24.2
	7.82	92.6		1.66	37.2		6.35	46.8
	6.38	131		1.36	52.6		5.18	66.2
	4.51	160		0.961	64.4		3.67	81.1
	2.34	179		0.498	71.8		1.90	90.4
	0.00	185		0.00	74.3		0.00	93.6
12ZS2.25x085	4.77	0.00	10ZS2.25x105	10.9	0.00	8ZS2.75x070	4.08	0.00
	4.61	37.1		10.6	38.6		3.94	19.2
	4.13	71.6		9.48	74.5		3.53	37.0
	3.37	101		7.74	105		2.88	52.4
	2.39	124		5.47	129		2.04	64.2
	1.23	138		2.83	144		1.06	71.6
	0.00	143		0.00	149		0.00	74.1
12ZS2.25x070	2.66	0.00	10ZS2.25x085	5.78	0.00	8ZS2.75x065	3.26	0.00
	2.57	28.5		5.58	30.1		3.15	17.7
	2.30	55.0		5.01	58.2		2.82	34.2
	1.88	77.8		4.09	82.3		2.31	48.3
	1.33	95.3		2.89	101		1.63	59.2
	0.688	106		1.50	112		0.844	66.0
	0.00	110		0.00	116		0.00	68.3
10ZS2.75x105	10.9	0.00	10ZS2.25x070	3.22	0.00	8ZS2.75x059	2.43	0.00
	10.6	43.6		3.11	23.3		2.35	15.5
	9.48	84.3		2.79	45.0		2.11	30.0
	7.74	119		2.28	63.6		1.72	42.4
	5.47	146		1.61	78.0		1.22	52.0
	2.83	163		0.833	86.9		0.630	57.9
	0.00	169		0.00	90.0		0.00	60.0

Table II - 12a

ASD-Combined Shear and Bending[2,3]
Purlins/Girts
Z-Sections With Lips

F_y = 55 ksi



Section	V [1] kips	M [1] kip-in.	Section	V [1] kips	M [1] kip-in.	Section	V [1] kips	M [1] kip-in.
8ZS2.25x105	12.2	0.00	6ZS2.25x105	11.7	0.00	6ZS2.25x059	3.32	0.00
	11.8	29.0		11.3	19.6		3.21	10.3
	10.5	56.1		10.2	37.8		2.87	20.0
	8.61	79.3		8.29	53.5		2.35	28.2
	6.08	97.2		5.86	65.5		1.66	34.6
	3.15	108		3.04	73.1		0.859	38.5
	0.00	112		0.00	75.7		0.00	39.9
8ZS2.25x085	7.33	0.00	6ZS2.25x085	7.97	0.00	4ZS2.25x070	5.03	0.00
	7.08	23.7		7.70	16.0		4.86	7.60
	6.35	45.7		6.91	30.9		4.36	14.7
	5.18	64.6		5.64	43.7		3.56	20.8
	3.67	79.1		3.99	53.5		2.52	25.4
	1.90	88.3		2.06	59.7		1.30	28.4
	0.00	91.4		0.00	61.8		0.00	29.4
8ZS2.25x070	4.08	0.00	6ZS2.25x070	5.41	0.00	4ZS2.25x065	4.66	0.00
	3.94	19.1		5.22	13.0		4.50	6.76
	3.53	37.0		4.68	25.1		4.04	13.1
	2.88	52.3		3.82	35.4		3.30	18.5
	2.04	64.1		2.70	43.4		2.33	22.6
	1.06	71.5		1.40	48.4		1.21	25.2
	0.00	74.0		0.00	50.1		0.00	26.1
8ZS2.25x065	3.26	0.00	6ZS2.25x065	4.45	0.00	4ZS2.25x059	3.84	0.00
	3.15	17.1		4.30	11.6		3.71	6.04
	2.82	33.0		3.85	22.3		3.33	11.7
	2.31	46.7		3.14	31.6		2.72	16.5
	1.63	57.2		2.22	38.6		1.92	20.2
	0.844	63.8		1.15	43.1		0.995	22.5
	0.00	66.1		0.00	44.6		0.00	23.3
8ZS2.25x059	2.43	0.00						
	2.35	15.3						
	2.11	29.6						
	1.72	41.8						
	1.22	51.2						
	0.630	57.1						
	0.00	59.1						

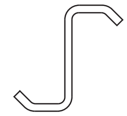
Notes:

1. Shear and moment strengths [resistances] have been divided by the appropriate safety factors. This table is for ASD use only.
2. Shear strengths [resistances] have been calculated assuming no transverse stiffeners.
3. Linear interpolation between values is permitted.

Table II – 12b

LRFD/LSD-Combined Shear and Bending[2,3]
Purlins/Girts
Z-Sections With Lips

F_y = 55 ksi

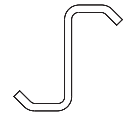


Section	$\bar{V}$[1] kips LRFD	$\bar{V}$[1] kips LSD	$\bar{M}$[1] kip-in.	Section	$\bar{V}$[1] kips LRFD	$\bar{V}$[1] kips LSD	$\bar{M}$[1] kip-in.
12ZS3.25x105	13.7	11.6	0.00	10ZS2.75x085	8.79	7.40	0.00
	13.2	11.2	84.7		8.49	7.15	49.6
	11.9	10.0	164		7.61	6.41	95.7
	9.70	8.17	231		6.21	5.23	135
	6.86	5.78	283		4.39	3.70	166
	3.55	2.99	316		2.27	1.91	185
	0.00	0.00	327		0.00	0.00	191
12ZS3.25x085	7.25	6.11	0.00	10ZS2.75x070	4.89	4.12	0.00
	7.00	5.90	62.0		4.72	3.98	37.6
	6.28	5.29	120		4.24	3.57	72.6
	5.13	4.32	169		3.46	2.91	103
	3.63	3.05	207		2.45	2.06	126
	1.88	1.58	231		1.27	1.07	140
	0.00	0.00	239		0.00	0.00	145
12ZS3.25x070	4.04	3.40	0.00	10ZS2.75x065	3.91	3.29	0.00
	3.90	3.29	45.4		3.78	3.18	33.5
	3.50	2.95	87.6		3.39	2.85	64.7
	2.86	2.41	124		2.77	2.33	91.5
	2.02	1.70	152		1.96	1.65	112
	1.05	0.880	169		1.01	0.853	125
	0.00	0.00	175		0.00	0.00	129
12ZS2.25x105	13.7	11.6	0.00	10ZS2.75x059	2.92	2.46	0.00
	13.2	11.2	72.0		2.82	2.38	28.9
	11.9	10.0	139		2.53	2.13	55.9
	9.70	8.17	197		2.07	1.74	79.0
	6.86	5.78	241		1.46	1.23	96.7
	3.55	2.99	269		0.756	0.637	108
	0.00	0.00	278		0.00	0.00	112
12ZS2.25x085	7.25	6.11	0.00	10ZS2.25x105	16.6	14.0	0.00
	7.00	5.90	55.7		16.1	13.5	58.0
	6.28	5.29	108		14.4	12.1	112
	5.13	4.32	152		11.8	9.90	158
	3.63	3.05	186		8.32	7.00	194
	1.88	1.58	208		4.30	3.62	216
	0.00	0.00	215		0.00	0.00	224
12ZS2.25x070	4.04	3.40	0.00	10ZS2.25x085	8.79	7.40	0.00
	3.90	3.29	42.8		8.49	7.15	45.3
	3.50	2.95	82.7		7.61	6.41	87.5
	2.86	2.41	117		6.21	5.23	124
	2.02	1.70	143		4.39	3.70	151
	1.05	0.880	160		2.27	1.91	169
	0.00	0.00	165		0.00	0.00	175
10ZS2.75x105	16.6	14.0	0.00	10ZS2.25x070	4.89	4.12	0.00
	16.1	13.5	65.6		4.72	3.98	35.0
	14.4	12.1	127		4.24	3.57	67.6
	11.8	9.90	179		3.46	2.91	95.7
	8.32	7.00	219		2.45	2.06	117
	4.30	3.62	245		1.27	1.07	131
	0.00	0.00	253		0.00	0.00	135

Table II – 12b

LRFD/LSD-Combined Shear and Bending[2,3]
Purlins/Girts
Z-Sections With Lips

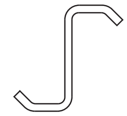
F_y = 55 ksi



Section	$\overline{V}$[1] kips		$\overline{M}$[1] kip-in.	Section	$\overline{V}$[1] kips		$\overline{M}$[1] kip-in.
	LRFD	LSD			LRFD	LSD	
10ZS2.25x065	3.91	3.29	0.00	8ZS2.25x105	18.5	15.6	0.00
	3.78	3.18	30.5		17.9	15.0	43.7
	3.39	2.85	58.9		16.0	13.5	84.3
	2.77	2.33	83.3		13.1	11.0	119
	1.96	1.65	102		9.25	7.79	146
	1.01	0.853	114		4.79	4.03	163
	0.00	0.00	118		0.00	0.00	169
10ZS2.25x059	2.92	2.46	0.00	8ZS2.25x085	11.1	9.38	0.00
	2.82	2.38	26.6		10.8	9.06	35.5
	2.53	2.13	51.4		9.65	8.13	68.7
	2.07	1.74	72.8		7.88	6.63	97.1
	1.46	1.23	89.1		5.57	4.69	119
	0.756	0.637	99.4		2.88	2.43	133
	0.00	0.00	103		0.00	0.00	137
8ZS2.75x105	18.5	15.6	0.00	8ZS2.25x070	6.20	5.22	0.00
	17.9	15.0	48.4		5.99	5.04	28.8
	16.0	13.5	93.5		5.37	4.52	55.6
	13.1	11.0	132		4.38	3.69	78.6
	9.25	7.79	162		3.10	2.61	96.3
	4.79	4.03	181		1.60	1.35	107
	0.00	0.00	187		0.00	0.00	111
8ZS2.75x085	11.1	9.38	0.00	8ZS2.25x065	4.96	4.17	0.00
	10.8	9.06	36.4		4.79	4.03	25.7
	9.65	8.13	70.3		4.29	3.61	49.6
	7.88	6.63	99.5		3.50	2.95	70.2
	5.57	4.69	122		2.48	2.09	86.0
	2.88	2.43	136		1.28	1.08	95.9
	0.00	0.00	141		0.00	0.00	99.3
8ZS2.75x070	6.20	5.22	0.00	8ZS2.25x059	3.70	3.12	0.00
	5.99	5.04	28.8		3.57	3.01	23.0
	5.37	4.52	55.7		3.20	2.70	44.4
	4.38	3.69	78.7		2.62	2.20	62.9
	3.10	2.61	96.4		1.85	1.56	77.0
	1.60	1.35	108		0.958	0.806	85.9
	0.00	0.00	111		0.00	0.00	88.9
8ZS2.75x065	4.96	4.17	0.00	6ZS2.25x105	17.8	15.0	0.00
	4.79	4.03	26.6		17.2	14.5	29.4
	4.29	3.61	51.3		15.4	13.0	56.9
	3.50	2.95	72.6		12.6	10.6	80.4
	2.48	2.09	88.9		8.91	7.51	98.5
	1.28	1.08	99.2		4.61	3.88	110
	0.00	0.00	103		0.00	0.00	114
8ZS2.75x059	3.70	3.12	0.00	6ZS2.25x085	12.1	10.2	0.00
	3.57	3.01	23.3		11.7	9.86	24.0
	3.20	2.70	45.1		10.5	8.84	46.4
	2.62	2.20	63.8		8.57	7.22	65.7
	1.85	1.56	78.1		6.06	5.10	80.4
	0.958	0.806	87.1		3.14	2.64	89.7
	0.00	0.00	90.2		0.00	0.00	92.9

Table II – 12b

LRFD/LSD-Combined Shear and Bending[2,3]
Purlins/Girts
Z-Sections With Lips



F_y = 55 ksi

Section	$\overline{V}$[1] kips		$\overline{M}$[1] kip-in.	Section	$\overline{V}$[1] kips		$\overline{M}$[1] kip-in.
	LRFD	LSD			LRFD	LSD	
6ZS2.25x070	8.22	6.92	0.00	4ZS2.25x070	7.65	6.44	0.00
	7.94	6.69	19.5		7.39	6.22	11.4
	7.12	6.00	37.7		6.62	5.58	22.1
	5.81	4.90	53.3		5.41	4.55	31.2
	4.11	3.46	65.2		3.82	3.22	38.2
	2.13	1.79	72.7		1.98	1.67	42.6
	0.00	0.00	75.3		0.00	0.00	44.1
6ZS2.25x065	6.76	5.69	0.00	4ZS2.25x065	7.09	5.97	0.00
	6.53	5.50	17.4		6.85	5.77	10.2
	5.85	4.93	33.5		6.14	5.17	19.6
	4.78	4.03	47.4		5.01	4.22	27.8
	3.38	2.85	58.1		3.54	2.98	34.0
	1.75	1.47	64.8		1.83	1.55	37.9
	0.00	0.00	67.1		0.00	0.00	39.3
6ZS2.25x059	5.04	4.25	0.00	4ZS2.25x059	5.84	4.92	0.00
	4.87	4.10	15.5		5.64	4.75	9.08
	4.37	3.68	30.0		5.06	4.26	17.5
	3.57	3.00	42.4		4.13	3.48	24.8
	2.52	2.12	51.9		2.92	2.46	30.4
	1.31	1.10	57.9		1.51	1.27	33.9
	0.00	0.00	60.0		0.00	0.00	35.1

Notes:

1. Shear and moment strengths [resistances] have been multiplied by the appropriate resistance factors. This table is for LRFD and LSD use only.
2. Shear strengths [resistances] have been calculated assuming no transverse stiffeners.
3. Linear interpolation between values is permitted

SECTION 3 – WEB CRIPPLING

3.1 Notes on the Tables

(a) With the exception of the joists/studs, the specific sections listed in these tables are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members.

(b) The section designations listed in these tables correspond to those for which dimensions and properties are given in Tables I-1, I-2 and I-4.

(c) The values provided in Tables II-13, II-14 and II-15 are nominal strengths [resistances] which do not incorporate safety or resistance factors. These nominal strengths [resistances] must be modified by safety factors (ASD) or resistance factors (LRFD, LSD) for use in design. See the appropriate *Specification* sections for more information.

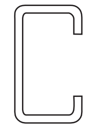
(d) The nominal crippling strengths [resistances] of joists/studs in Table II-14 do not include any allowance for the effects of standard punchouts. These punchouts are present in joists/studs with flange widths of 1.625 in. or less. Standard punchout sizes are 1.5 in. by 4.5 in. for sections with depths of 3.5 inches or more and 0.75 in. by 4.5 in. for sections with shallower depths. Tables II-16a and II-16b provide modification factors that can be used to calculate the reduced crippling strength taking into account the standard punchouts in cases where the punchouts do not overlap the bearing length, N.

3.2 Web Crippling Tables

Table II - 13

F_y = 55 ksi

Web Crippling, P_n, kips [1,2,4]
Purlins/Girts
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	Bearing Length[4], N, in.				ASD	LRFD	LSD
			1	2	4	6	Ω_w	ϕ_w	ϕ_w
12CS3.5x105	Fastened	A	3.25	3.94	4.93	5.69	1.75	0.85	0.75
		B	7.00	7.88	9.12	10.1	1.65	0.90	0.80
		C	2.78	3.09	3.53	3.87	1.75	0.85	0.75
		D	8.87	9.59	10.6	11.4	1.75	0.85	0.75
	Unfastened	A	3.25	3.94	4.93	5.69	1.85	0.80	0.70
		B	7.00	7.88	9.12	10.1	1.65	0.90	0.80
		C	3.04	3.20	3.44	3.62	1.65	0.90	0.80
		D	6.43	7.27	8.46	9.38	1.90	0.80	0.65
12CS3.5x085	Fastened	A	2.13	2.61	3.29	3.81	1.75	0.85	0.75
		B	4.45	5.05	5.89	6.54	1.65	0.90	0.80
		C	1.64	1.84	2.12	2.34	1.75	0.85	0.75
		D	5.52	6.01	6.71	7.24	1.75	0.85	0.75
	Unfastened	A	2.13	2.61	3.29	3.81	1.85	0.80	0.70
		B	4.45	5.05	5.89	6.54	1.65	0.90	0.80
		C	1.70	1.80	1.95	2.06	1.65	0.90	0.80
		D	3.25	3.71	4.35	4.85	1.90	0.80	0.65
12CS3.5x070	Fastened	A	1.44	1.77	2.25	2.62	1.75	0.85	0.75
		B	2.91	3.33	3.92	4.37	1.65	0.90	0.80
		C	0.983	1.11	1.29	1.43	1.75	0.85	0.75
		D	3.54	3.88	4.36	4.73	1.75	0.85	0.75
	Unfastened	A	1.44	1.77	2.25	2.62	1.85	0.80	0.70
		B	2.91	3.33	3.92	4.37	1.65	0.90	0.80
		C	0.967	1.03	1.12	1.19	1.65	0.90	0.80
		D	1.49	1.71	2.03	2.27	1.90	0.80	0.65
10CS3.5x105	Fastened	A	3.32	4.04	5.05	5.83	1.75	0.85	0.75
		B	7.08	7.96	9.21	10.2	1.65	0.90	0.80
		C	3.04	3.38	3.86	4.23	1.75	0.85	0.75
		D	9.26	10.0	11.1	11.9	1.75	0.85	0.75
	Unfastened	A	3.32	4.04	5.05	5.83	1.85	0.80	0.70
		B	7.08	7.96	9.21	10.2	1.65	0.90	0.80
		C	3.24	3.41	3.67	3.86	1.65	0.90	0.80
		D	6.43	7.28	8.47	9.39	1.90	0.80	0.65
10CS3.5x085	Fastened	A	2.19	2.68	3.38	3.91	1.75	0.85	0.75
		B	4.50	5.11	5.96	6.62	1.65	0.90	0.80
		C	1.83	2.05	2.36	2.60	1.75	0.85	0.75
		D	5.80	6.32	7.05	7.62	1.75	0.85	0.75
	Unfastened	A	2.19	2.68	3.38	3.91	1.85	0.80	0.70
		B	4.50	5.11	5.96	6.62	1.65	0.90	0.80
		C	1.84	1.95	2.10	2.23	1.65	0.90	0.80
		D	3.25	3.71	4.36	4.86	1.90	0.80	0.65
10CS3.5x070	Fastened	A	1.48	1.83	2.32	2.70	1.75	0.85	0.75
		B	2.95	3.37	3.97	4.43	1.65	0.90	0.80
		C	1.13	1.27	1.48	1.64	1.75	0.85	0.75
		D	3.75	4.11	4.62	5.02	1.75	0.85	0.75
	Unfastened	A	1.48	1.83	2.32	2.70	1.85	0.80	0.70
		B	2.95	3.37	3.97	4.43	1.65	0.90	0.80
		C	1.06	1.13	1.23	1.30	1.65	0.90	0.80
		D	1.49	1.72	2.03	2.27	1.90	0.80	0.65

Table II - 13

F_y = 55 ksi

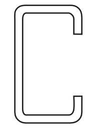
Web Crippling, P_n, kips [1,2,4]
Purlins/Girts
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	Bearing Length[4], N, in.				ASD	LRFD	LSD
			1	2	4	6	Ω_w	ϕ_w	ϕ_w
10CS3.5x065	Fastened	A	1.27	1.58	2.01	2.34	1.75	0.85	0.75
		B	2.51	2.88	3.40	3.80	1.65	0.90	0.80
		C	0.930	1.05	1.23	1.36	1.75	0.85	0.75
		D	3.17	3.48	3.93	4.27	1.75	0.85	0.75
	Unfastened	A	1.27	1.58	2.01	2.34	1.85	0.80	0.70
		B	2.51	2.88	3.40	3.80	1.65	0.90	0.80
		C	0.852	0.910	0.992	1.05	1.65	0.90	0.80
		D	1.02	1.18	1.40	1.57	1.90	0.80	0.65
8CS3.5x105	Fastened	A	3.41	4.15	5.18	5.98	1.75	0.85	0.75
		B	7.16	8.06	9.32	10.3	1.65	0.90	0.80
		C	3.32	3.69	4.22	4.62	1.75	0.85	0.75
		D	9.69	10.5	11.6	12.5	1.75	0.85	0.75
	Unfastened	A	3.41	4.15	5.18	5.98	1.85	0.80	0.70
		B	7.16	8.06	9.32	10.3	1.65	0.90	0.80
		C	3.46	3.65	3.92	4.13	1.65	0.90	0.80
		D	6.44	7.29	8.48	9.40	1.90	0.80	0.65
8CS3.5x085	Fastened	A	2.25	2.76	3.48	4.03	1.75	0.85	0.75
		B	4.56	5.18	6.04	6.71	1.65	0.90	0.80
		C	2.04	2.29	2.64	2.90	1.75	0.85	0.75
		D	6.12	6.67	7.44	8.03	1.75	0.85	0.75
	Unfastened	A	2.25	2.76	3.48	4.03	1.85	0.80	0.70
		B	4.56	5.18	6.04	6.71	1.65	0.90	0.80
		C	1.99	2.11	2.28	2.41	1.65	0.90	0.80
		D	3.26	3.72	4.36	4.86	1.90	0.80	0.65
8CS3.5x070	Fastened	A	1.53	1.89	2.40	2.79	1.75	0.85	0.75
		B	3.00	3.42	4.03	4.50	1.65	0.90	0.80
		C	1.29	1.45	1.69	1.87	1.75	0.85	0.75
		D	3.99	4.37	4.92	5.33	1.75	0.85	0.75
	Unfastened	A	1.53	1.89	2.40	2.79	1.85	0.80	0.70
		B	3.00	3.42	4.03	4.50	1.65	0.90	0.80
		C	1.16	1.24	1.35	1.43	1.65	0.90	0.80
		D	1.49	1.72	2.03	2.28	1.90	0.80	0.65
8CS3.5x065	Fastened	A	1.32	1.64	2.08	2.43	1.75	0.85	0.75
		B	2.55	2.92	3.45	3.85	1.65	0.90	0.80
		C	1.07	1.22	1.42	1.57	1.75	0.85	0.75
		D	3.38	3.72	4.19	4.55	1.75	0.85	0.75
	Unfastened	A	1.32	1.64	2.08	2.43	1.85	0.80	0.70
		B	2.55	2.92	3.45	3.85	1.65	0.90	0.80
		C	0.941	1.00	1.10	1.16	1.65	0.90	0.80
		D	1.02	1.18	1.40	1.57	1.90	0.80	0.65
8CS3.5x059	Fastened	A	1.09	1.35	1.73	2.02	1.75	0.85	0.75
		B	2.05	2.36	2.80	3.14	1.65	0.90	0.80
		C	0.843	0.959	1.12	1.25	1.75	0.85	0.75
		D	2.72	3.00	3.39	3.70	1.75	0.85	0.75
	Unfastened	A	1.09	1.35	1.73	2.02	1.85	0.80	0.70
		B	2.05	2.36	2.80	3.14	1.65	0.90	0.80
		C	0.707	0.758	0.828	0.883	1.65	0.90	0.80
		D	0.536	0.621	0.741	0.833	1.90	0.80	0.65

Table II - 13

F_y = 55 ksi

Web Crippling, P_n, kips [1,2,4]
Purlins/Girts
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	Bearing Length[4], N, in.				ASD	LRFD	LSD
			1	2	4	6	Ω_w	ϕ_w	ϕ_w
8CS2.5x105	Fastened	A	3.41	4.15	5.18	5.98	1.75	0.85	0.75
		B	7.16	8.06	9.32	10.3	1.65	0.90	0.80
		C	3.32	3.69	4.22	4.62	1.75	0.85	0.75
		D	9.69	10.5	11.6	12.5	1.75	0.85	0.75
	Unfastened	A	3.41	4.15	5.18	5.98	1.85	0.80	0.70
		B	7.16	8.06	9.32	10.3	1.65	0.90	0.80
		C	3.46	3.65	3.92	4.13	1.65	0.90	0.80
		D	6.44	7.29	8.48	9.40	1.90	0.80	0.65
8CS2.5x085	Fastened	A	2.25	2.76	3.48	4.03	1.75	0.85	0.75
		B	4.56	5.18	6.04	6.71	1.65	0.90	0.80
		C	2.04	2.29	2.64	2.90	1.75	0.85	0.75
		D	6.12	6.67	7.44	8.03	1.75	0.85	0.75
	Unfastened	A	2.25	2.76	3.48	4.03	1.85	0.80	0.70
		B	4.56	5.18	6.04	6.71	1.65	0.90	0.80
		C	1.99	2.11	2.28	2.41	1.65	0.90	0.80
		D	3.26	3.72	4.36	4.86	1.90	0.80	0.65
8CS2.5x070	Fastened	A	1.53	1.89	2.40	2.79	1.75	0.85	0.75
		B	3.00	3.42	4.03	4.50	1.65	0.90	0.80
		C	1.29	1.45	1.69	1.87	1.75	0.85	0.75
		D	3.99	4.37	4.92	5.33	1.75	0.85	0.75
	Unfastened	A	1.53	1.89	2.40	2.79	1.85	0.80	0.70
		B	3.00	3.42	4.03	4.50	1.65	0.90	0.80
		C	1.16	1.24	1.35	1.43	1.65	0.90	0.80
		D	1.49	1.72	2.03	2.28	1.90	0.80	0.65
8CS2.5x065	Fastened	A	1.32	1.64	2.08	2.43	1.75	0.85	0.75
		B	2.55	2.92	3.45	3.85	1.65	0.90	0.80
		C	1.07	1.22	1.42	1.57	1.75	0.85	0.75
		D	3.38	3.72	4.19	4.55	1.75	0.85	0.75
	Unfastened	A	1.32	1.64	2.08	2.43	1.85	0.80	0.70
		B	2.55	2.92	3.45	3.85	1.65	0.90	0.80
		C	0.941	1.00	1.10	1.16	1.65	0.90	0.80
		D	1.02	1.18	1.40	1.57	1.90	0.80	0.65
8CS2.5x059	Fastened	A	1.09	1.35	1.73	2.02	1.75	0.85	0.75
		B	2.05	2.36	2.80	3.14	1.65	0.90	0.80
		C	0.843	0.959	1.12	1.25	1.75	0.85	0.75
		D	2.72	3.00	3.39	3.70	1.75	0.85	0.75
	Unfastened	A	1.09	1.35	1.73	2.02	1.85	0.80	0.70
		B	2.05	2.36	2.80	3.14	1.65	0.90	0.80
		C	0.707	0.758	0.828	0.883	1.65	0.90	0.80
		D	0.536	0.621	0.741	0.833	1.90	0.80	0.65
6CS2.5x105	Fastened	A	3.51	4.27	5.34	6.16	1.75	0.85	0.75
		B	7.26	8.16	9.45	10.4	1.65	0.90	0.80
		C	3.65	4.06	4.63	5.08	1.75	0.85	0.75
		D	10.2	11.0	12.2	13.1	1.75	0.85	0.75
	Unfastened	A	3.51	4.27	5.34	6.16	1.85	0.80	0.70
		B	7.26	8.16	9.45	10.4	1.65	0.90	0.80
		C	3.71	3.92	4.21	4.43	1.65	0.90	0.80
		D	6.45	7.29	8.49	9.41	1.90	0.80	0.65

Table II - 13

F_y = 55 ksi

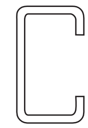
Web Crippling, P_n, kips [1,2,4]
Purlins/Girts
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	Bearing Length[4], N, in.				ASD Ω_w	LRFD ϕ_w	LSD ϕ_w
			1	2	4	6			
6CS2.5x085	Fastened	A	2.33	2.85	3.60	4.17	1.75	0.85	0.75
		B	4.63	5.25	6.13	6.81	1.65	0.90	0.80
		C	2.28	2.56	2.95	3.25	1.75	0.85	0.75
		D	6.48	7.06	7.88	8.51	1.75	0.85	0.75
	Unfastened	A	2.33	2.85	3.60	4.17	1.85	0.80	0.70
		B	4.63	5.25	6.13	6.81	1.65	0.90	0.80
		C	2.16	2.29	2.47	2.62	1.65	0.90	0.80
		D	3.26	3.72	4.37	4.87	1.90	0.80	0.65
6CS2.5x070	Fastened	A	1.59	1.96	2.49	2.90	1.75	0.85	0.75
		B	3.05	3.48	4.10	4.57	1.65	0.90	0.80
		C	1.47	1.66	1.93	2.13	1.75	0.85	0.75
		D	4.26	4.67	5.25	5.69	1.75	0.85	0.75
	Unfastened	A	1.59	1.96	2.49	2.90	1.85	0.80	0.70
		B	3.05	3.48	4.10	4.57	1.65	0.90	0.80
		C	1.28	1.37	1.49	1.58	1.65	0.90	0.80
		D	1.50	1.72	2.04	2.28	1.90	0.80	0.65
6CS2.5x065	Fastened	A	1.37	1.70	2.17	2.52	1.75	0.85	0.75
		B	2.59	2.97	3.51	3.92	1.65	0.90	0.80
		C	1.24	1.40	1.63	1.81	1.75	0.85	0.75
		D	3.62	3.98	4.49	4.88	1.75	0.85	0.75
	Unfastened	A	1.37	1.70	2.17	2.52	1.85	0.80	0.70
		B	2.59	2.97	3.51	3.92	1.65	0.90	0.80
		C	1.04	1.11	1.21	1.29	1.65	0.90	0.80
		D	1.03	1.18	1.41	1.58	1.90	0.80	0.65
6CS2.5x059	Fastened	A	1.13	1.41	1.80	2.10	1.75	0.85	0.75
		B	2.09	2.41	2.86	3.20	1.65	0.90	0.80
		C	0.986	1.12	1.31	1.46	1.75	0.85	0.75
		D	2.93	3.23	3.66	3.98	1.75	0.85	0.75
	Unfastened	A	1.13	1.41	1.80	2.10	1.85	0.80	0.70
		B	2.09	2.41	2.86	3.20	1.65	0.90	0.80
		C	0.791	0.847	0.926	0.987	1.65	0.90	0.80
		D	0.537	0.622	0.743	0.835	1.90	0.80	0.65
4CS2.5x105	Fastened	A	3.63	4.42	5.52	6.37	1.75	0.85	0.75
		B	7.37	8.29	9.60	10.6	1.65	0.90	0.80
		C	4.04	4.49	5.13	5.63	1.75	0.85	0.75
		D	10.8	11.7	12.9	13.9	1.75	0.85	0.75
	Unfastened	A	3.63	4.42	5.52	6.37	1.85	0.80	0.70
		B	7.37	8.29	9.60	10.6	1.65	0.90	0.80
		C	4.02	4.24	4.56	4.80	1.65	0.90	0.80
		D	6.46	7.31	8.50	9.42	1.90	0.80	0.65
4CS2.5x085	Fastened	A	2.42	2.96	3.74	4.33	1.75	0.85	0.75
		B	4.71	5.35	6.24	6.93	1.65	0.90	0.80
		C	2.57	2.88	3.32	3.66	1.75	0.85	0.75
		D	6.92	7.54	8.41	9.08	1.75	0.85	0.75
	Unfastened	A	2.42	2.96	3.74	4.33	1.85	0.80	0.70
		B	4.71	5.35	6.24	6.93	1.65	0.90	0.80
		C	2.37	2.51	2.71	2.87	1.65	0.90	0.80
		D	3.27	3.73	4.38	4.88	1.90	0.80	0.65

Table II - 13

F_y = 55 ksi

Web Crippling, P_n, kips [1,2,4]
Purlins/Girts
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	Bearing Length[4], N, in. 1	2	4	6	ASD Ω_w	LRFD ϕ_w	LSD ϕ_w
4CS2.5x070	Fastened	A	1.66	2.05	2.60	3.03	1.75	0.85	0.75
		B	3.11	3.55	4.18	4.66	1.65	0.90	0.80
		C	1.69	1.91	2.22	2.45	1.75	0.85	0.75
		D	4.59	5.03	5.65	6.13	1.75	0.85	0.75
	Unfastened	A	1.66	2.05	2.60	3.03	1.85	0.80	0.70
		B	3.11	3.55	4.18	4.66	1.65	0.90	0.80
		C	1.42	1.52	1.65	1.75	1.65	0.90	0.80
		D	1.50	1.72	2.04	2.29	1.90	0.80	0.65
4CS2.5x065	Fastened	A	1.43	1.78	2.26	2.64	1.75	0.85	0.75
		B	2.64	3.03	3.58	4.00	1.65	0.90	0.80
		C	1.44	1.63	1.89	2.10	1.75	0.85	0.75
		D	3.92	4.30	4.85	5.27	1.75	0.85	0.75
	Unfastened	A	1.43	1.78	2.26	2.64	1.85	0.80	0.70
		B	2.64	3.03	3.58	4.00	1.65	0.90	0.80
		C	1.17	1.24	1.36	1.44	1.65	0.90	0.80
		D	1.03	1.18	1.41	1.58	1.90	0.80	0.65
4CS2.5x059	Fastened	A	1.19	1.48	1.89	2.20	1.75	0.85	0.75
		B	2.14	2.46	2.92	3.27	1.65	0.90	0.80
		C	1.16	1.32	1.54	1.71	1.75	0.85	0.75
		D	3.18	3.51	3.97	4.33	1.75	0.85	0.75
	Unfastened	A	1.19	1.48	1.89	2.20	1.85	0.80	0.70
		B	2.14	2.46	2.92	3.27	1.65	0.90	0.80
		C	0.892	0.955	1.04	1.11	1.65	0.90	0.80
		D	0.538	0.624	0.744	0.836	1.90	0.80	0.65

Notes:

1. Web crippling strengths are nominal strengths [resistances] calculated without consideration of holes or other openings. To obtain available strengths [factored resistances], the values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD), which are provided in the table.
2. Strength reductions factors for openings must be calculated in accordance with the provisions of *Specification* Section C3.4.2.
3. Case A
   End Reaction, Opposing Loads Spaced > 1.5h

   Case B
   Interior Reactions, Opposing Loads Spaced > 1.5h

   Case C
   End Reaction, Opposing Loads Spaced ≤ 1.5h

   Case D
   Interior Reactions, Opposing Loads Spaced ≤ 1.5h
4. Linear interpolation is permitted between bearing lengths.

Table II – 14

F_y = 33 ksi
F_y = 50 ksi

Web Crippling, P_n, kips [1,2,4]
Joists/Studs
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	Bearing Length[4], N, in.				ASD	LRFD	LSD
			1	2	4	6	Ω_w	ϕ_w	ϕ_w
1200S200-97	Fastened	A	2.83	3.44	4.31	4.98	1.75	0.85	0.75
		B	6.21	6.99	8.10	8.96	1.65	0.90	0.80
		C	2.36	2.63	3.00	3.29	1.75	0.85	0.75
		D	7.61	8.24	9.14	9.82	1.75	0.85	0.75
	Unfastened	A	2.83	3.44	4.31	4.98	1.85	0.80	0.70
		B	6.21	6.99	8.10	8.96	1.65	0.90	0.80
		C	2.72	2.87	3.09	3.25	1.65	0.90	0.80
		D	6.55	7.42	8.65	9.59	1.90	0.80	0.65
1200S200-68	Fastened	A	1.45	1.79	2.27	2.64	1.75	0.85	0.75
		B	3.16	3.60	4.24	4.73	1.65	0.90	0.80
		C	0.964	1.09	1.26	1.40	1.75	0.85	0.75
		D	3.50	3.84	4.31	4.67	1.75	0.85	0.75
	Unfastened	A	1.45	1.79	2.27	2.64	1.85	0.80	0.70
		B	3.16	3.60	4.24	4.73	1.65	0.90	0.80
		C	1.17	1.24	1.35	1.43	1.65	0.90	0.80
		D	3.41	3.92	4.64	5.19	1.90	0.80	0.65
1000S250-97	Fastened	A	2.90	3.53	4.42	5.10	1.75	0.85	0.75
		B	6.28	7.07	8.19	9.05	1.65	0.90	0.80
		C	2.58	2.87	3.29	3.61	1.75	0.85	0.75
		D	7.95	8.61	9.55	10.3	1.75	0.85	0.75
	Unfastened	A	2.90	3.53	4.42	5.10	1.85	0.80	0.70
		B	6.28	7.07	8.19	9.05	1.65	0.90	0.80
		C	2.90	3.06	3.29	3.47	1.65	0.90	0.80
		D	6.56	7.43	8.66	9.60	1.90	0.80	0.65
1000S250-68	Fastened	A	1.49	1.84	2.34	2.72	1.75	0.85	0.75
		B	3.20	3.65	4.30	4.79	1.65	0.90	0.80
		C	1.10	1.24	1.44	1.60	1.75	0.85	0.75
		D	3.71	4.06	4.56	4.95	1.75	0.85	0.75
	Unfastened	A	1.49	1.84	2.34	2.72	1.85	0.80	0.70
		B	3.20	3.65	4.30	4.79	1.65	0.90	0.80
		C	1.28	1.36	1.48	1.57	1.65	0.90	0.80
		D	3.42	3.93	4.65	5.20	1.90	0.80	0.65
1000S250-54	Fastened	A	0.968	1.21	1.54	1.80	1.75	0.85	0.75
		B	2.06	2.38	2.83	3.17	1.65	0.90	0.80
		C	0.605	0.689	0.808	0.899	1.75	0.85	0.75
		D	2.23	2.46	2.79	3.04	1.75	0.85	0.75
	Unfastened	A	0.968	1.21	1.54	1.80	1.85	0.80	0.70
		B	2.06	2.38	2.83	3.17	1.65	0.90	0.80
		C	0.729	0.782	0.856	0.913	1.65	0.90	0.80
		D	2.25	2.61	3.11	3.51	1.90	0.80	0.65
1000S200-97	Fastened	A	2.90	3.53	4.42	5.10	1.75	0.85	0.75
		B	6.28	7.07	8.19	9.05	1.65	0.90	0.80
		C	2.58	2.87	3.29	3.61	1.75	0.85	0.75
		D	7.95	8.61	9.55	10.3	1.75	0.85	0.75
	Unfastened	A	2.90	3.53	4.42	5.10	1.85	0.80	0.70
		B	6.28	7.07	8.19	9.05	1.65	0.90	0.80
		C	2.90	3.06	3.29	3.47	1.65	0.90	0.80
		D	6.56	7.43	8.66	9.60	1.90	0.80	0.65

Table II – 14

F_y = 33 ksi
F_y = 50 ksi

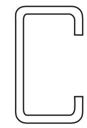
Web Crippling, P_n, kips [1,2,4]
Joists/Studs
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	Bearing Length[4], N, in.				ASD	LRFD	LSD
			1	2	4	6	Ω_w	ϕ_w	ϕ_w
1000S200-68	Fastened	A	1.49	1.84	2.34	2.72	1.75	0.85	0.75
		B	3.20	3.65	4.30	4.79	1.65	0.90	0.80
		C	1.10	1.24	1.44	1.60	1.75	0.85	0.75
		D	3.71	4.06	4.56	4.95	1.75	0.85	0.75
	Unfastened	A	1.49	1.84	2.34	2.72	1.85	0.80	0.70
		B	3.20	3.65	4.30	4.79	1.65	0.90	0.80
		C	1.28	1.36	1.48	1.57	1.65	0.90	0.80
		D	3.42	3.93	4.65	5.20	1.90	0.80	0.65
1000S200-54	Fastened	A	0.968	1.21	1.54	1.80	1.75	0.85	0.75
		B	2.06	2.38	2.83	3.17	1.65	0.90	0.80
		C	0.605	0.689	0.808	0.899	1.75	0.85	0.75
		D	2.23	2.46	2.79	3.04	1.75	0.85	0.75
	Unfastened	A	0.968	1.21	1.54	1.80	1.85	0.80	0.70
		B	2.06	2.38	2.83	3.17	1.65	0.90	0.80
		C	0.729	0.782	0.856	0.913	1.65	0.90	0.80
		D	2.25	2.61	3.11	3.51	1.90	0.80	0.65
800S200-97	Fastened	A	2.98	3.62	4.54	5.24	1.75	0.85	0.75
		B	6.35	7.15	8.29	9.16	1.65	0.90	0.80
		C	2.83	3.15	3.61	3.95	1.75	0.85	0.75
		D	8.33	9.02	10.0	10.8	1.75	0.85	0.75
	Unfastened	A	2.98	3.62	4.54	5.24	1.85	0.80	0.70
		B	6.35	7.15	8.29	9.16	1.65	0.90	0.80
		C	3.11	3.28	3.53	3.72	1.65	0.90	0.80
		D	6.57	7.44	8.67	9.61	1.90	0.80	0.65
800S200-68	Fastened	A	1.54	1.91	2.42	2.81	1.75	0.85	0.75
		B	3.24	3.71	4.36	4.86	1.65	0.90	0.80
		C	1.25	1.41	1.64	1.82	1.75	0.85	0.75
		D	3.94	4.31	4.84	5.25	1.75	0.85	0.75
	Unfastened	A	1.54	1.91	2.42	2.81	1.85	0.80	0.70
		B	3.24	3.71	4.36	4.86	1.65	0.90	0.80
		C	1.40	1.49	1.62	1.72	1.65	0.90	0.80
		D	3.42	3.93	4.65	5.21	1.90	0.80	0.65
800S200-54	Fastened	A	1.01	1.25	1.60	1.87	1.75	0.85	0.75
		B	2.10	2.42	2.88	3.23	1.65	0.90	0.80
		C	0.717	0.816	0.957	1.06	1.75	0.85	0.75
		D	2.40	2.65	3.00	3.27	1.75	0.85	0.75
	Unfastened	A	1.01	1.25	1.60	1.87	1.85	0.80	0.70
		B	2.10	2.42	2.88	3.23	1.65	0.90	0.80
		C	0.817	0.875	0.958	1.02	1.65	0.90	0.80
		D	2.25	2.61	3.12	3.51	1.90	0.80	0.65
800S200-43	Fastened	A	0.432	0.544	0.701	0.822	1.75	0.85	0.75
		B	0.894	1.04	1.25	1.41	1.65	0.90	0.80
		C	0.262	0.301	0.357	0.400	1.75	0.85	0.75
		D	0.959	1.07	1.22	1.34	1.75	0.85	0.75
	Unfastened	A	0.432	0.544	0.701	0.822	1.85	0.80	0.70
		B	0.894	1.04	1.25	1.41	1.65	0.90	0.80
		C	0.306	0.330	0.365	0.391	1.65	0.90	0.80
		D	0.940	1.10	1.33	1.50	1.90	0.80	0.65

Table II – 14

F_y = 33 ksi
F_y = 50 ksi

Web Crippling, P_n, kips [1,2,4]
Joists/Studs
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	Bearing Length[4], N, in.				ASD Ω_w	LRFD ϕ_w	LSD ϕ_w
			1	2	4	6			
800S162-97	Fastened	A	1.97	2.39	2.99	3.46	1.75	0.85	0.75
		B	4.19	4.72	5.47	6.05	1.65	0.90	0.80
		C	1.87	2.08	2.38	2.61	1.75	0.85	0.75
		D	5.50	5.96	6.60	7.10	1.75	0.85	0.75
	Unfastened	A	1.97	2.39	2.99	3.46	1.85	0.80	0.70
		B	4.19	4.72	5.47	6.05	1.65	0.90	0.80
		C	2.05	2.16	2.33	2.45	1.65	0.90	0.80
		D	4.33	4.91	5.72	6.35	1.90	0.80	0.65
800S162-68	Fastened	A	1.02	1.26	1.60	1.86	1.75	0.85	0.75
		B	2.14	2.45	2.88	3.21	1.65	0.90	0.80
		C	0.827	0.934	1.08	1.20	1.75	0.85	0.75
		D	2.60	2.85	3.20	3.47	1.75	0.85	0.75
	Unfastened	A	1.02	1.26	1.60	1.86	1.85	0.80	0.70
		B	2.14	2.45	2.88	3.21	1.65	0.90	0.80
		C	0.922	0.983	1.07	1.13	1.65	0.90	0.80
		D	2.26	2.60	3.07	3.44	1.90	0.80	0.65
800S162-54	Fastened	A	0.664	0.827	1.06	1.24	1.75	0.85	0.75
		B	1.39	1.60	1.90	2.13	1.65	0.90	0.80
		C	0.473	0.539	0.631	0.703	1.75	0.85	0.75
		D	1.58	1.75	1.98	2.16	1.75	0.85	0.75
	Unfastened	A	0.664	0.827	1.06	1.24	1.85	0.80	0.70
		B	1.39	1.60	1.90	2.13	1.65	0.90	0.80
		C	0.539	0.578	0.633	0.675	1.65	0.90	0.80
		D	1.48	1.72	2.06	2.32	1.90	0.80	0.65
800S162-43	Fastened	A	0.432	0.544	0.701	0.822	1.75	0.85	0.75
		B	0.894	1.04	1.25	1.41	1.65	0.90	0.80
		C	0.262	0.301	0.357	0.400	1.75	0.85	0.75
		D	0.959	1.07	1.22	1.34	1.75	0.85	0.75
	Unfastened	A	0.432	0.544	0.701	0.822	1.85	0.80	0.70
		B	0.894	1.04	1.25	1.41	1.65	0.90	0.80
		C	0.306	0.330	0.365	0.391	1.65	0.90	0.80
		D	0.940	1.10	1.33	1.50	1.90	0.80	0.65
600S200-97	Fastened	A	3.07	3.73	4.67	5.39	1.75	0.85	0.75
		B	6.44	7.25	8.40	9.28	1.65	0.90	0.80
		C	3.12	3.47	3.97	4.35	1.75	0.85	0.75
		D	8.77	9.49	10.5	11.3	1.75	0.85	0.75
	Unfastened	A	3.07	3.73	4.67	5.39	1.85	0.80	0.70
		B	6.44	7.25	8.40	9.28	1.65	0.90	0.80
		C	3.34	3.53	3.79	3.99	1.65	0.90	0.80
		D	6.58	7.45	8.68	9.63	1.90	0.80	0.65
600S200-68	Fastened	A	1.60	1.98	2.51	2.92	1.75	0.85	0.75
		B	3.30	3.77	4.43	4.94	1.65	0.90	0.80
		C	1.43	1.61	1.87	2.07	1.75	0.85	0.75
		D	4.20	4.60	5.17	5.60	1.75	0.85	0.75
	Unfastened	A	1.60	1.98	2.51	2.92	1.85	0.80	0.70
		B	3.30	3.77	4.43	4.94	1.65	0.90	0.80
		C	1.54	1.64	1.78	1.89	1.65	0.90	0.80
		D	3.43	3.94	4.66	5.21	1.90	0.80	0.65

Table II – 14

F_y = 33 ksi
F_y = 50 ksi

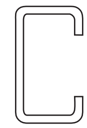
Web Crippling, P_n, kips [1,2,4]
Joists/Studs
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	Bearing Length[4], N, in. 1	2	4	6	ASD Ω_w	LRFD ϕ_w	LSD ϕ_w
600S200-54	Fastened	A	1.05	1.31	1.67	1.95	1.75	0.85	0.75
		B	2.14	2.47	2.93	3.28	1.65	0.90	0.80
		C	0.844	0.961	1.13	1.25	1.75	0.85	0.75
		D	2.59	2.86	3.24	3.53	1.75	0.85	0.75
	Unfastened	A	1.05	1.31	1.67	1.95	1.85	0.80	0.70
		B	2.14	2.47	2.93	3.28	1.65	0.90	0.80
		C	0.916	0.982	1.08	1.15	1.65	0.90	0.80
		D	2.25	2.61	3.12	3.52	1.90	0.80	0.65
600S200-43	Fastened	A	0.453	0.570	0.736	0.862	1.75	0.85	0.75
		B	0.913	1.06	1.28	1.44	1.65	0.90	0.80
		C	0.324	0.372	0.441	0.494	1.75	0.85	0.75
		D	1.05	1.17	1.34	1.47	1.75	0.85	0.75
	Unfastened	A	0.453	0.570	0.736	0.862	1.85	0.80	0.70
		B	0.913	1.06	1.28	1.44	1.65	0.90	0.80
		C	0.353	0.381	0.420	0.451	1.65	0.90	0.80
		D	0.942	1.10	1.33	1.51	1.90	0.80	0.65
600S200-33	Fastened	A	0.267	0.340	0.442	0.520	1.75	0.85	0.75
		B	0.516	0.607	0.737	0.837	1.65	0.90	0.80
		C	0.163	0.190	0.227	0.256	1.75	0.85	0.75
		D	0.577	0.648	0.750	0.828	1.75	0.85	0.75
	Unfastened	A	0.267	0.340	0.442	0.520	1.85	0.80	0.70
		B	0.516	0.607	0.737	0.837	1.65	0.90	0.80
		C	0.165	0.179	0.200	0.216	1.65	0.90	0.80
		D	0.383	0.454	0.555	0.632	1.90	0.80	0.65
600S162-97	Fastened	A	3.07	3.73	4.67	5.39	1.75	0.85	0.75
		B	4.25	4.79	5.55	6.13	1.65	0.90	0.80
		C	2.06	2.29	2.62	2.87	1.75	0.85	0.75
		D	5.79	6.27	6.95	7.47	1.75	0.85	0.75
	Unfastened	A	2.02	2.46	3.08	3.56	1.85	0.80	0.70
		B	4.25	4.79	5.55	6.13	1.65	0.90	0.80
		C	2.20	2.33	2.50	2.64	1.65	0.90	0.80
		D	4.34	4.92	5.73	6.35	1.90	0.80	0.65
600S162-68	Fastened	A	1.06	1.30	1.66	1.92	1.75	0.85	0.75
		B	2.18	2.49	2.92	3.26	1.65	0.90	0.80
		C	0.942	1.06	1.23	1.37	1.75	0.85	0.75
		D	2.77	3.04	3.41	3.70	1.75	0.85	0.75
	Unfastened	A	1.06	1.30	1.66	1.92	1.85	0.80	0.70
		B	2.18	2.49	2.92	3.26	1.65	0.90	0.80
		C	1.01	1.08	1.17	1.25	1.65	0.90	0.80
		D	2.26	2.60	3.08	3.44	1.90	0.80	0.65
600S162-54	Fastened	A	0.692	0.862	1.10	1.29	1.75	0.85	0.75
		B	1.41	1.63	1.93	2.17	1.65	0.90	0.80
		C	0.557	0.634	0.744	0.828	1.75	0.85	0.75
		D	1.71	1.88	2.14	2.33	1.75	0.85	0.75
	Unfastened	A	0.692	0.862	1.10	1.29	1.85	0.80	0.70
		B	1.41	1.63	1.93	2.17	1.65	0.90	0.80
		C	0.605	0.648	0.710	0.757	1.65	0.90	0.80
		D	1.49	1.73	2.06	2.32	1.90	0.80	0.65

Table II – 14

F_y = 33 ksi
F_y = 50 ksi

Web Crippling, P_n, kips [1,2,4]
Joists/Studs
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	Bearing Length[4], N, in. 1	2	4	6	ASD Ω_w	LRFD ϕ_w	LSD ϕ_w
600S162-43	Fastened	A	0.453	0.570	0.736	0.862	1.75	0.85	0.75
		B	0.913	1.06	1.28	1.44	1.65	0.90	0.80
		C	0.324	0.372	0.441	0.494	1.75	0.85	0.75
		D	1.05	1.17	1.34	1.47	1.75	0.85	0.75
	Unfastened	A	0.453	0.570	0.736	0.862	1.85	0.80	0.70
		B	0.913	1.06	1.28	1.44	1.65	0.90	0.80
		C	0.353	0.381	0.420	0.451	1.65	0.90	0.80
		D	0.942	1.10	1.33	1.51	1.90	0.80	0.65
600S162-33	Fastened	A	0.267	0.340	0.442	0.520	1.75	0.85	0.75
		B	0.516	0.607	0.737	0.837	1.65	0.90	0.80
		C	0.163	0.190	0.227	0.256	1.75	0.85	0.75
		D	0.577	0.648	0.750	0.828	1.75	0.85	0.75
	Unfastened	A	0.267	0.340	0.442	0.520	1.85	0.80	0.70
		B	0.516	0.607	0.737	0.837	1.65	0.90	0.80
		C	0.165	0.179	0.200	0.216	1.65	0.90	0.80
		D	0.383	0.454	0.555	0.632	1.90	0.80	0.65
550S162-68	Fastened	A	1.62	2.00	2.53	2.94	1.75	0.85	0.75
		B	3.31	3.78	4.45	4.96	1.65	0.90	0.80
		C	1.48	1.67	1.93	2.14	1.75	0.85	0.75
		D	4.27	4.68	5.26	5.70	1.75	0.85	0.75
	Unfastened	A	1.62	2.00	2.53	2.94	1.85	0.80	0.70
		B	3.31	3.78	4.45	4.96	1.65	0.90	0.80
		C	1.57	1.68	1.82	1.93	1.65	0.90	0.80
		D	3.43	3.94	4.66	5.22	1.90	0.80	0.65
550S162-54	Fastened	A	1.06	1.32	1.69	1.97	1.75	0.85	0.75
		B	2.15	2.48	2.94	3.30	1.65	0.90	0.80
		C	0.879	1.00	1.17	1.31	1.75	0.85	0.75
		D	2.64	2.91	3.30	3.60	1.75	0.85	0.75
	Unfastened	A	1.06	1.32	1.69	1.97	1.85	0.80	0.70
		B	2.15	2.48	2.94	3.30	1.65	0.90	0.80
		C	0.944	1.01	1.11	1.18	1.65	0.90	0.80
		D	2.25	2.62	3.13	3.52	1.90	0.80	0.65
550S162-43	Fastened	A	0.459	0.578	0.745	0.874	1.75	0.85	0.75
		B	0.918	1.07	1.28	1.45	1.65	0.90	0.80
		C	0.341	0.392	0.464	0.520	1.75	0.85	0.75
		D	1.07	1.20	1.37	1.50	1.75	0.85	0.75
	Unfastened	A	0.459	0.578	0.745	0.874	1.85	0.80	0.70
		B	0.918	1.07	1.28	1.45	1.65	0.90	0.80
		C	0.366	0.395	0.436	0.467	1.65	0.90	0.80
		D	0.943	1.10	1.33	1.51	1.90	0.80	0.65
550S162-33	Fastened	A	0.272	0.345	0.449	0.528	1.75	0.85	0.75
		B	0.519	0.611	0.742	0.842	1.65	0.90	0.80
		C	0.175	0.203	0.243	0.274	1.75	0.85	0.75
		D	0.594	0.668	0.772	0.852	1.75	0.85	0.75
	Unfastened	A	0.272	0.345	0.449	0.528	1.85	0.80	0.70
		B	0.519	0.611	0.742	0.842	1.65	0.90	0.80
		C	0.173	0.188	0.209	0.226	1.65	0.90	0.80
		D	0.384	0.455	0.555	0.632	1.90	0.80	0.65

Table II – 14

F_y = 33 ksi
F_y = 50 ksi

Web Crippling, P_n, kips[1,2,4]
Joists/Studs
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	Bearing Length[4], N, in.				ASD Ω_w	LRFD ϕ_w	LSD ϕ_w
			1	2	4	6			
400S162-68	Fastened	A	1.67	2.06	2.61	3.04	1.75	0.85	0.75
		B	3.36	3.84	4.51	5.03	1.65	0.90	0.80
		C	1.64	1.85	2.14	2.37	1.75	0.85	0.75
		D	4.51	4.94	5.55	6.02	1.75	0.85	0.75
	Unfastened	A	1.67	2.06	2.61	3.04	1.85	0.80	0.70
		B	3.36	3.84	4.51	5.03	1.65	0.90	0.80
		C	1.70	1.81	1.97	2.09	1.65	0.90	0.80
		D	3.43	3.95	4.67	5.22	1.90	0.80	0.65
400S162-54	Fastened	A	1.10	1.37	1.75	2.05	1.75	0.85	0.75
		B	2.18	2.52	2.99	3.36	1.65	0.90	0.80
		C	0.996	1.13	1.33	1.48	1.75	0.85	0.75
		D	2.81	3.11	3.52	3.84	1.75	0.85	0.75
	Unfastened	A	1.10	1.37	1.75	2.05	1.85	0.80	0.70
		B	2.18	2.52	2.99	3.36	1.65	0.90	0.80
		C	1.04	1.11	1.22	1.30	1.65	0.90	0.80
		D	2.26	2.62	3.13	3.52	1.90	0.80	0.65
400S162-43	Fastened	A	0.479	0.602	0.777	0.911	1.75	0.85	0.75
		B	0.935	1.09	1.31	1.47	1.65	0.90	0.80
		C	0.398	0.457	0.541	0.606	1.75	0.85	0.75
		D	1.16	1.29	1.47	1.62	1.75	0.85	0.75
	Unfastened	A	0.479	0.602	0.777	0.911	1.85	0.80	0.70
		B	0.935	1.09	1.31	1.47	1.65	0.90	0.80
		C	0.409	0.441	0.487	0.522	1.65	0.90	0.80
		D	0.944	1.11	1.33	1.51	1.90	0.80	0.65
400S162-33	Fastened	A	0.285	0.362	0.471	0.555	1.75	0.85	0.75
		B	0.530	0.625	0.758	0.861	1.65	0.90	0.80
		C	0.214	0.249	0.298	0.336	1.75	0.85	0.75
		D	0.650	0.731	0.846	0.934	1.75	0.85	0.75
	Unfastened	A	0.285	0.362	0.471	0.555	1.85	0.80	0.70
		B	0.530	0.625	0.758	0.861	1.65	0.90	0.80
		C	0.199	0.216	0.241	0.260	1.65	0.90	0.80
		D	0.384	0.455	0.556	0.633	1.90	0.80	0.65
362S162-68	Fastened	A	1.68	2.08	2.64	3.07	1.75	0.85	0.75
		B	3.37	3.85	4.53	5.05	1.65	0.90	0.80
		C	1.68	1.90	2.20	2.44	1.75	0.85	0.75
		D	4.58	5.02	5.64	6.11	1.75	0.85	0.75
	Unfastened	A	1.68	2.08	2.64	3.07	1.85	0.80	0.70
		B	3.37	3.85	4.53	5.05	1.65	0.90	0.80
		C	1.74	1.85	2.01	2.14	1.65	0.90	0.80
		D	3.43	3.95	4.67	5.23	1.90	0.80	0.65
362S162-54	Fastened	A	1.11	1.38	1.77	2.07	1.75	0.85	0.75
		B	2.19	2.53	3.01	3.37	1.65	0.90	0.80
		C	1.03	1.17	1.37	1.53	1.75	0.85	0.75
		D	2.86	3.16	3.58	3.91	1.75	0.85	0.75
	Unfastened	A	1.11	1.38	1.77	2.07	1.85	0.80	0.70
		B	2.19	2.53	3.01	3.37	1.65	0.90	0.80
		C	1.06	1.14	1.25	1.33	1.65	0.90	0.80
		D	2.26	2.62	3.13	3.52	1.90	0.80	0.65

Table II – 14

F_y = 33 ksi
F_y = 50 ksi

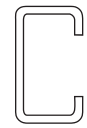
Web Crippling, P_n, kips [1,2,4]
Joists/Studs
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	Bearing Length[4], N, in.				ASD	LRFD	LSD
			1	2	4	6	Ω_w	ϕ_w	ϕ_w
362S162-43	Fastened	A	0.484	0.609	0.786	0.921	1.75	0.85	0.75
		B	0.940	1.09	1.31	1.48	1.65	0.90	0.80
		C	0.414	0.476	0.563	0.630	1.75	0.85	0.75
		D	1.18	1.32	1.50	1.65	1.75	0.85	0.75
	Unfastened	A	0.484	0.609	0.786	0.921	1.85	0.80	0.70
		B	0.940	1.09	1.31	1.48	1.65	0.90	0.80
		C	0.421	0.454	0.501	0.537	1.65	0.90	0.80
		D	0.945	1.11	1.34	1.51	1.90	0.80	0.65
362S162-33	Fastened	A	0.289	0.367	0.478	0.563	1.75	0.85	0.75
		B	0.534	0.628	0.763	0.866	1.65	0.90	0.80
		C	0.225	0.262	0.313	0.353	1.75	0.85	0.75
		D	0.666	0.749	0.867	0.957	1.75	0.85	0.75
	Unfastened	A	0.289	0.367	0.478	0.563	1.85	0.80	0.70
		B	0.534	0.628	0.763	0.866	1.65	0.90	0.80
		C	0.206	0.224	0.250	0.269	1.65	0.90	0.80
		D	0.385	0.456	0.556	0.633	1.90	0.80	0.65
350S162-68	Fastened	A	1.69	2.08	2.65	3.08	1.75	0.85	0.75
		B	3.38	3.86	4.54	5.06	1.65	0.90	0.80
		C	1.70	1.92	2.22	2.46	1.75	0.85	0.75
		D	4.60	5.04	5.67	6.14	1.75	0.85	0.75
	Unfastened	A	1.69	2.08	2.65	3.08	1.85	0.80	0.70
		B	3.38	3.86	4.54	5.06	1.65	0.90	0.80
		C	1.75	1.87	2.03	2.15	1.65	0.90	0.80
		D	3.44	3.95	4.67	5.23	1.90	0.80	0.65
350S162-54	Fastened	A	1.11	1.39	1.78	2.08	1.75	0.85	0.75
		B	2.20	2.53	3.01	3.38	1.65	0.90	0.80
		C	1.04	1.18	1.39	1.55	1.75	0.85	0.75
		D	2.88	3.18	3.60	3.93	1.75	0.85	0.75
	Unfastened	A	1.11	1.39	1.78	2.08	1.85	0.80	0.70
		B	2.20	2.53	3.01	3.38	1.65	0.90	0.80
		C	1.07	1.15	1.26	1.34	1.65	0.90	0.80
		D	2.26	2.62	3.13	3.53	1.90	0.80	0.65
350S162-43	Fastened	A	0.486	0.612	0.789	0.925	1.75	0.85	0.75
		B	0.942	1.10	1.32	1.48	1.65	0.90	0.80
		C	0.419	0.482	0.570	0.638	1.75	0.85	0.75
		D	1.19	1.32	1.52	1.66	1.75	0.85	0.75
	Unfastened	A	0.486	0.612	0.789	0.925	1.85	0.80	0.70
		B	0.942	1.10	1.32	1.48	1.65	0.90	0.80
		C	0.425	0.459	0.506	0.543	1.65	0.90	0.80
		D	0.945	1.11	1.34	1.51	1.90	0.80	0.65
350S162-33	Fastened	A	0.290	0.369	0.480	0.565	1.75	0.85	0.75
		B	0.535	0.630	0.764	0.867	1.65	0.90	0.80
		C	0.229	0.266	0.319	0.359	1.75	0.85	0.75
		D	0.672	0.755	0.874	0.964	1.75	0.85	0.75
	Unfastened	A	0.290	0.369	0.480	0.565	1.85	0.80	0.70
		B	0.535	0.630	0.764	0.867	1.65	0.90	0.80
		C	0.209	0.227	0.253	0.273	1.65	0.90	0.80
		D	0.385	0.456	0.556	0.634	1.90	0.80	0.65

Table II – 14

F_y = 33 ksi
F_y = 50 ksi

Web Crippling, P_n, kips [1,2,4]
Joists/Studs
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	Bearing Length[4], N, in.				ASD Ω_w	LRFD ϕ_w	LSD ϕ_w
			1	2	4	6			
250S162-68	Fastened	A	1.73	2.14	2.72	3.16	1.75	0.85	0.75
		B	3.42	3.91	4.60	5.12	1.65	0.90	0.80
		C	1.84	2.07	2.41	2.66	1.75	0.85	0.75
		D	4.81	5.27	5.92	6.42	1.75	0.85	0.75
	Unfastened	A	1.73	2.14	2.72	3.16	1.85	0.80	0.70
		B	3.42	3.91	4.60	5.12	1.65	0.90	0.80
		C	1.86	1.98	2.16	2.29	1.65	0.90	0.80
		D	3.44	3.95	4.68	5.23	1.90	0.80	0.65
250S162-54	Fastened	A	1.15	1.43	1.83	2.14	1.75	0.85	0.75
		B	2.23	2.57	3.05	3.42	1.65	0.90	0.80
		C	1.14	1.30	1.52	1.69	1.75	0.85	0.75
		D	3.03	3.34	3.79	4.13	1.75	0.85	0.75
	Unfastened	A	1.15	1.43	1.83	2.14	1.85	0.80	0.70
		B	2.23	2.57	3.05	3.42	1.65	0.90	0.80
		C	1.15	1.23	1.35	1.44	1.65	0.90	0.80
		D	2.26	2.62	3.14	3.53	1.90	0.80	0.65
250S162-43	Fastened	A	0.503	0.632	0.816	0.956	1.75	0.85	0.75
		B	0.956	1.11	1.34	1.51	1.65	0.90	0.80
		C	0.467	0.537	0.636	0.712	1.75	0.85	0.75
		D	1.26	1.40	1.61	1.76	1.75	0.85	0.75
	Unfastened	A	0.503	0.632	0.816	0.956	1.85	0.80	0.70
		B	0.956	1.11	1.34	1.51	1.65	0.90	0.80
		C	0.462	0.498	0.550	0.589	1.65	0.90	0.80
		D	0.946	1.11	1.34	1.51	1.90	0.80	0.65
250S162-33	Fastened	A	0.302	0.384	0.499	0.588	1.75	0.85	0.75
		B	0.544	0.641	0.778	0.883	1.65	0.90	0.80
		C	0.262	0.305	0.365	0.411	1.75	0.85	0.75
		D	0.720	0.810	0.937	1.03	1.75	0.85	0.75
	Unfastened	A	0.302	0.384	0.499	0.588	1.85	0.80	0.70
		B	0.544	0.641	0.778	0.883	1.65	0.90	0.80
		C	0.231	0.251	0.280	0.302	1.65	0.90	0.80
		D	0.385	0.457	0.557	0.635	1.90	0.80	0.65

Notes:

1. Web crippling strengths are nominal strengths [resistances] calculated without consideration of holes or other openings. To obtain available strengths [factored resistances], the values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD), which are provided in the table.
2. Strength reduction factors for openings must be calculated in accordance with the provisions of *Specification* Section C3.4.2. See Tables II-16a and II-16b for tabulated reduction factors.
3. Case A
   End Reaction, Opposing Loads Spaced > 1.5h

   Case B
   Interior Reactions, Opposing Loads Spaced > 1.5h

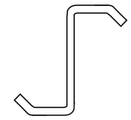
   Case C
   End Reaction, Opposing Loads Spaced ≤ 1.5h

   Case D
   Interior Reactions, Opposing Loads Spaced ≤ 1.5h
4. Linear interpolation is permitted between bearing lengths.

Table II - 15

F_y = 55 ksi

Web Crippling, P_n, kips [1,3]
Purlins/Girts
Z-Sections With Lips

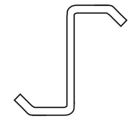


Section	Fastened or Unfastened	Case[2]	Bearing Length[3], N, in.				ASD Ω_w	LRFD ϕ_w	LSD ϕ_w
			1	2	4	6			
12ZS3.25x105	Fastened	A	3.25	3.94	4.93	5.69	1.75	0.85	0.75
		B	7.00	7.88	9.12	10.1	1.65	0.90	0.80
		C	3.48	3.96	4.63	5.15	1.75	0.85	0.75
		D	9.35	10.0	11.0	11.8	1.85	0.80	0.70
	Unfastened	A	2.80	2.87	2.97	3.04	1.80	0.85	0.75
		B	7.00	7.88	9.12	10.1	1.65	0.90	0.80
		C	3.04	3.20	3.44	3.62	1.65	0.90	0.80
		D	6.43	7.27	8.46	9.38	1.90	0.80	0.65
12ZS3.25x085	Fastened	A	2.13	2.61	3.29	3.81	1.75	0.85	0.75
		B	4.45	5.05	5.89	6.54	1.65	0.90	0.80
		C	2.03	2.33	2.75	3.08	1.75	0.85	0.75
		D	5.68	6.13	6.78	7.27	1.85	0.80	0.70
	Unfastened	A	1.82	1.87	1.93	1.99	1.80	0.85	0.75
		B	4.45	5.05	5.89	6.54	1.65	0.90	0.80
		C	1.70	1.80	1.95	2.06	1.65	0.90	0.80
		D	3.25	3.71	4.35	4.85	1.90	0.80	0.65
12ZS3.25x070	Fastened	A	1.44	1.77	2.25	2.62	1.75	0.85	0.75
		B	2.91	3.33	3.92	4.37	1.65	0.90	0.80
		C	1.19	1.38	1.64	1.85	1.75	0.85	0.75
		D	3.53	3.84	4.27	4.60	1.85	0.80	0.70
	Unfastened	A	1.22	1.26	1.31	1.34	1.80	0.85	0.75
		B	2.91	3.33	3.92	4.37	1.65	0.90	0.80
		C	0.967	1.03	1.12	1.19	1.65	0.90	0.80
		D	1.49	1.71	2.03	2.27	1.90	0.80	0.65
12ZS2.25x105	Fastened	A	3.25	3.94	4.93	5.69	1.75	0.85	0.75
		B	7.00	7.88	9.12	10.1	1.65	0.90	0.80
		C	3.48	3.96	4.63	5.15	1.75	0.85	0.75
		D	9.35	10.0	11.0	11.8	1.85	0.80	0.70
	Unfastened	A	2.80	2.87	2.97	3.04	1.80	0.85	0.75
		B	7.00	7.88	9.12	10.1	1.65	0.90	0.80
		C	3.04	3.20	3.44	3.62	1.65	0.90	0.80
		D	6.43	7.27	8.46	9.38	1.90	0.80	0.65
12ZS2.25x085	Fastened	A	2.13	2.61	3.29	3.81	1.75	0.85	0.75
		B	4.45	5.05	5.89	6.54	1.65	0.90	0.80
		C	2.03	2.33	2.75	3.08	1.75	0.85	0.75
		D	5.68	6.13	6.78	7.27	1.85	0.80	0.70
	Unfastened	A	1.82	1.87	1.93	1.99	1.80	0.85	0.75
		B	4.45	5.05	5.89	6.54	1.65	0.90	0.80
		C	1.70	1.80	1.95	2.06	1.65	0.90	0.80
		D	3.25	3.71	4.35	4.85	1.90	0.80	0.65
12ZS2.25x070	Fastened	A	1.44	1.77	2.25	2.62	1.75	0.85	0.75
		B	2.91	3.33	3.92	4.37	1.65	0.90	0.80
		C	1.19	1.38	1.64	1.85	1.75	0.85	0.75
		D	3.53	3.84	4.27	4.60	1.85	0.80	0.70
	Unfastened	A	1.22	1.26	1.31	1.34	1.80	0.85	0.75
		B	2.91	3.33	3.92	4.37	1.65	0.90	0.80
		C	0.967	1.03	1.12	1.19	1.65	0.90	0.80
		D	1.49	1.71	2.03	2.27	1.90	0.80	0.65

Table II - 15

F_y = 55 ksi

Web Crippling, P_n, kips [1,3]
Purlins/Girts
Z-Sections With Lips

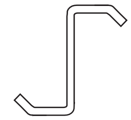


Section	Fastened or Unfastened	Case[2]	Bearing Length[3], N, in.				ASD Ω_w	LRFD ϕ_w	LSD ϕ_w
			1	2	4	6			
10ZS2.75x105	Fastened	A	3.32	4.04	5.05	5.83	1.75	0.85	0.75
		B	7.08	7.96	9.21	10.2	1.65	0.90	0.80
		C	3.86	4.39	5.14	5.71	1.75	0.85	0.75
		D	9.97	10.7	11.7	12.5	1.85	0.80	0.70
	Unfastened	A	2.81	2.87	2.97	3.04	1.80	0.85	0.75
		B	7.08	7.96	9.21	10.2	1.65	0.90	0.80
		C	3.24	3.41	3.67	3.86	1.65	0.90	0.80
		D	6.43	7.28	8.47	9.39	1.90	0.80	0.65
10ZS2.75x085	Fastened	A	2.19	2.68	3.38	3.91	1.75	0.85	0.75
		B	4.50	5.11	5.96	6.62	1.65	0.90	0.80
		C	2.32	2.66	3.14	3.50	1.75	0.85	0.75
		D	6.13	6.62	7.31	7.85	1.85	0.80	0.70
	Unfastened	A	1.82	1.87	1.94	1.99	1.80	0.85	0.75
		B	4.50	5.11	5.96	6.62	1.65	0.90	0.80
		C	1.84	1.95	2.10	2.23	1.65	0.90	0.80
		D	3.25	3.71	4.36	4.86	1.90	0.80	0.65
10ZS2.75x070	Fastened	A	1.48	1.83	2.32	2.70	1.75	0.85	0.75
		B	2.95	3.37	3.97	4.43	1.65	0.90	0.80
		C	1.41	1.63	1.94	2.18	1.75	0.85	0.75
		D	3.87	4.21	4.68	5.04	1.85	0.80	0.70
	Unfastened	A	1.22	1.26	1.31	1.35	1.80	0.85	0.75
		B	2.95	3.37	3.97	4.43	1.65	0.90	0.80
		C	1.06	1.13	1.23	1.30	1.65	0.90	0.80
		D	1.49	1.72	2.03	2.27	1.90	0.80	0.65
10ZS2.75x065	Fastened	A	1.27	1.58	2.01	2.34	1.75	0.85	0.75
		B	2.51	2.88	3.40	3.80	1.65	0.90	0.80
		C	1.16	1.34	1.60	1.80	1.75	0.85	0.75
		D	3.24	3.52	3.93	4.25	1.85	0.80	0.70
	Unfastened	A	1.05	1.08	1.12	1.16	1.80	0.85	0.75
		B	2.51	2.88	3.40	3.80	1.65	0.90	0.80
		C	0.852	0.910	0.992	1.05	1.65	0.90	0.80
		D	1.02	1.18	1.40	1.57	1.90	0.80	0.65
10ZS2.75x059	Fastened	A	1.05	1.30	1.66	1.94	1.75	0.85	0.75
		B	2.02	2.33	2.76	3.09	1.65	0.90	0.80
		C	0.885	1.03	1.24	1.39	1.75	0.85	0.75
		D	2.55	2.79	3.12	3.38	1.85	0.80	0.70
	Unfastened	A	0.859	0.886	0.924	0.954	1.80	0.85	0.75
		B	2.02	2.33	2.76	3.09	1.65	0.90	0.80
		C	0.635	0.679	0.743	0.792	1.65	0.90	0.80
		D	0.536	0.620	0.740	0.832	1.90	0.80	0.65
10ZS2.25x105	Fastened	A	3.32	4.04	5.05	5.83	1.75	0.85	0.75
		B	7.08	7.96	9.21	10.2	1.65	0.90	0.80
		C	3.86	4.39	5.14	5.71	1.75	0.85	0.75
		D	9.97	10.7	11.7	12.5	1.85	0.80	0.70
	Unfastened	A	2.81	2.87	2.97	3.04	1.80	0.85	0.75
		B	7.08	7.96	9.21	10.2	1.65	0.90	0.80
		C	3.24	3.41	3.67	3.86	1.65	0.90	0.80
		D	6.43	7.28	8.47	9.39	1.90	0.80	0.65

Table II - 15

F_y = 55 ksi

Web Crippling, P_n, kips [1,3]
Purlins/Girts
Z-Sections With Lips

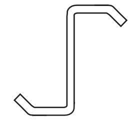


Section	Fastened or Unfastened	Case[2]	Bearing Length[3], N, in.				ASD Ω_w	LRFD ϕ_w	LSD ϕ_w
			1	2	4	6			
10ZS2.25x085	Fastened	A	2.19	2.68	3.38	3.91	1.75	0.85	0.75
		B	4.50	5.11	5.96	6.62	1.65	0.90	0.80
		C	2.32	2.66	3.14	3.50	1.75	0.85	0.75
		D	6.13	6.62	7.31	7.85	1.85	0.80	0.70
	Unfastened	A	1.82	1.87	1.94	1.99	1.80	0.85	0.75
		B	4.50	5.11	5.96	6.62	1.65	0.90	0.80
		C	1.84	1.95	2.10	2.23	1.65	0.90	0.80
		D	3.25	3.71	4.36	4.86	1.90	0.80	0.65
10ZS2.25x070	Fastened	A	1.48	1.83	2.32	2.70	1.75	0.85	0.75
		B	2.95	3.37	3.97	4.43	1.65	0.90	0.80
		C	1.41	1.63	1.94	2.18	1.75	0.85	0.75
		D	3.87	4.21	4.68	5.04	1.85	0.80	0.70
	Unfastened	A	1.22	1.26	1.31	1.35	1.80	0.85	0.75
		B	2.95	3.37	3.97	4.43	1.65	0.90	0.80
		C	1.06	1.13	1.23	1.30	1.65	0.90	0.80
		D	1.49	1.72	2.03	2.27	1.90	0.80	0.65
10ZS2.25x065	Fastened	A	1.27	1.58	2.01	2.34	1.75	0.85	0.75
		B	2.51	2.88	3.40	3.80	1.65	0.90	0.80
		C	1.16	1.34	1.60	1.80	1.75	0.85	0.75
		D	3.24	3.52	3.93	4.25	1.85	0.80	0.70
	Unfastened	A	1.05	1.08	1.12	1.16	1.80	0.85	0.75
		B	2.51	2.88	3.40	3.80	1.65	0.90	0.80
		C	0.852	0.910	0.992	1.05	1.65	0.90	0.80
		D	1.02	1.18	1.40	1.57	1.90	0.80	0.65
10ZS2.25x059	Fastened	A	1.05	1.30	1.66	1.94	1.75	0.85	0.75
		B	2.02	2.33	2.76	3.09	1.65	0.90	0.80
		C	0.885	1.03	1.24	1.39	1.75	0.85	0.75
		D	2.55	2.79	3.12	3.38	1.85	0.80	0.70
	Unfastened	A	0.859	0.886	0.924	0.954	1.80	0.85	0.75
		B	2.02	2.33	2.76	3.09	1.65	0.90	0.80
		C	0.635	0.679	0.743	0.792	1.65	0.90	0.80
		D	0.536	0.620	0.740	0.832	1.90	0.80	0.65
8ZS2.75x105	Fastened	A	3.41	4.15	5.18	5.98	1.75	0.85	0.75
		B	7.16	8.06	9.32	10.3	1.65	0.90	0.80
		C	4.28	4.87	5.70	6.34	1.75	0.85	0.75
		D	10.6	11.4	12.5	13.4	1.85	0.80	0.70
	Unfastened	A	2.81	2.88	2.97	3.04	1.80	0.85	0.75
		B	7.16	8.06	9.32	10.3	1.65	0.90	0.80
		C	3.46	3.65	3.92	4.13	1.65	0.90	0.80
		D	6.44	7.29	8.48	9.40	1.90	0.80	0.65
8ZS2.75x085	Fastened	A	2.25	2.76	3.48	4.03	1.75	0.85	0.75
		B	4.56	5.18	6.04	6.71	1.65	0.90	0.80
		C	2.63	3.02	3.56	3.98	1.75	0.85	0.75
		D	6.63	7.16	7.91	8.49	1.85	0.80	0.70
	Unfastened	A	1.82	1.87	1.94	1.99	1.80	0.85	0.75
		B	4.56	5.18	6.04	6.71	1.65	0.90	0.80
		C	1.99	2.11	2.28	2.41	1.65	0.90	0.80
		D	3.26	3.72	4.36	4.86	1.90	0.80	0.65

Table II - 15

F_y = 55 ksi

Web Crippling, P_n, kips [1,3]
Purlins/Girts
Z-Sections With Lips

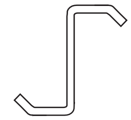


Section	Fastened or Unfastened	Case[2]	Bearing Length[3], N, in.				ASD Ω_w	LRFD ϕ_w	LSD ϕ_w
			1	2	4	6			
8ZS2.75x070	Fastened	A	1.53	1.89	2.40	2.79	1.75	0.85	0.75
		B	3.00	3.42	4.03	4.50	1.65	0.90	0.80
		C	1.65	1.91	2.27	2.55	1.75	0.85	0.75
		D	4.25	4.61	5.14	5.53	1.85	0.80	0.70
	Unfastened	A	1.22	1.26	1.31	1.35	1.80	0.85	0.75
		B	3.00	3.42	4.03	4.50	1.65	0.90	0.80
		C	1.16	1.24	1.35	1.43	1.65	0.90	0.80
		D	1.49	1.72	2.03	2.28	1.90	0.80	0.65
8ZS2.75x065	Fastened	A	1.32	1.64	2.08	2.43	1.75	0.85	0.75
		B	2.55	2.92	3.45	3.85	1.65	0.90	0.80
		C	1.38	1.60	1.91	2.14	1.75	0.85	0.75
		D	3.57	3.89	4.34	4.69	1.85	0.80	0.70
	Unfastened	A	1.05	1.08	1.13	1.16	1.80	0.85	0.75
		B	2.55	2.92	3.45	3.85	1.65	0.90	0.80
		C	0.941	1.00	1.10	1.16	1.65	0.90	0.80
		D	1.02	1.18	1.40	1.57	1.90	0.80	0.65
8ZS2.75x059	Fastened	A	1.09	1.35	1.73	2.02	1.75	0.85	0.75
		B	2.05	2.36	2.80	3.14	1.65	0.90	0.80
		C	1.08	1.25	1.50	1.70	1.75	0.85	0.75
		D	2.84	3.11	3.48	3.76	1.85	0.80	0.70
	Unfastened	A	0.860	0.887	0.925	0.955	1.80	0.85	0.75
		B	2.05	2.36	2.80	3.14	1.65	0.90	0.80
		C	0.707	0.758	0.828	0.883	1.65	0.90	0.80
		D	0.536	0.621	0.741	0.833	1.90	0.80	0.65
8ZS2.25x105	Fastened	A	3.41	4.15	5.18	5.98	1.75	0.85	0.75
		B	7.16	8.06	9.32	10.3	1.65	0.90	0.80
		C	4.28	4.87	5.70	6.34	1.75	0.85	0.75
		D	10.6	11.4	12.5	13.4	1.85	0.80	0.70
	Unfastened	A	2.81	2.88	2.97	3.04	1.80	0.85	0.75
		B	7.16	8.06	9.32	10.3	1.65	0.90	0.80
		C	3.46	3.65	3.92	4.13	1.65	0.90	0.80
		D	6.44	7.29	8.48	9.40	1.90	0.80	0.65
8ZS2.25x085	Fastened	A	2.25	2.76	3.48	4.03	1.75	0.85	0.75
		B	4.56	5.18	6.04	6.71	1.65	0.90	0.80
		C	2.63	3.02	3.56	3.98	1.75	0.85	0.75
		D	6.63	7.16	7.91	8.49	1.85	0.80	0.70
	Unfastened	A	1.82	1.87	1.94	1.99	1.80	0.85	0.75
		B	4.56	5.18	6.04	6.71	1.65	0.90	0.80
		C	1.99	2.11	2.28	2.41	1.65	0.90	0.80
		D	3.26	3.72	4.36	4.86	1.90	0.80	0.65
8ZS2.25x070	Fastened	A	1.53	1.89	2.40	2.79	1.75	0.85	0.75
		B	3.00	3.42	4.03	4.50	1.65	0.90	0.80
		C	1.65	1.91	2.27	2.55	1.75	0.85	0.75
		D	4.25	4.61	5.14	5.53	1.85	0.80	0.70
	Unfastened	A	1.22	1.26	1.31	1.35	1.80	0.85	0.75
		B	3.00	3.42	4.03	4.50	1.65	0.90	0.80
		C	1.16	1.24	1.35	1.43	1.65	0.90	0.80
		D	1.49	1.72	2.03	2.28	1.90	0.80	0.65

Table II - 15

F_y = 55 ksi

Web Crippling, P_n, kips [1,3]
Purlins/Girts
Z-Sections With Lips

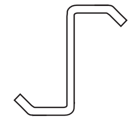


Section	Fastened or Unfastened	Case[2]	Bearing Length[3], N, in.				ASD Ω_w	LRFD ϕ_w	LSD ϕ_w
			1	2	4	6			
8ZS2.25x065	Fastened	A	1.32	1.64	2.08	2.43	1.75	0.85	0.75
		B	2.55	2.92	3.45	3.85	1.65	0.90	0.80
		C	1.38	1.60	1.91	2.14	1.75	0.85	0.75
		D	3.57	3.89	4.34	4.69	1.85	0.80	0.70
	Unfastened	A	1.05	1.08	1.13	1.16	1.80	0.85	0.75
		B	2.55	2.92	3.45	3.85	1.65	0.90	0.80
		C	0.941	1.00	1.10	1.16	1.65	0.90	0.80
		D	1.02	1.18	1.40	1.57	1.90	0.80	0.65
8ZS2.25x059	Fastened	A	1.09	1.35	1.73	2.02	1.75	0.85	0.75
		B	2.05	2.36	2.80	3.14	1.65	0.90	0.80
		C	1.08	1.25	1.50	1.70	1.75	0.85	0.75
		D	2.84	3.11	3.48	3.76	1.85	0.80	0.70
	Unfastened	A	0.860	0.887	0.925	0.955	1.80	0.85	0.75
		B	2.05	2.36	2.80	3.14	1.65	0.90	0.80
		C	0.707	0.758	0.828	0.883	1.65	0.90	0.80
		D	0.536	0.621	0.741	0.833	1.90	0.80	0.65
6ZS2.25x105	Fastened	A	3.51	4.27	5.34	6.16	1.75	0.85	0.75
		B	7.26	8.16	9.45	10.4	1.65	0.90	0.80
		C	4.77	5.42	6.34	7.05	1.75	0.85	0.75
		D	11.4	12.3	13.5	14.4	1.85	0.80	0.70
	Unfastened	A	2.81	2.88	2.97	3.05	1.80	0.85	0.75
		B	7.26	8.16	9.45	10.4	1.65	0.90	0.80
		C	3.71	3.92	4.21	4.43	1.65	0.90	0.80
		D	6.45	7.29	8.49	9.41	1.90	0.80	0.65
6ZS2.25x085	Fastened	A	2.33	2.85	3.60	4.17	1.75	0.85	0.75
		B	4.63	5.25	6.13	6.81	1.65	0.90	0.80
		C	2.99	3.43	4.05	4.53	1.75	0.85	0.75
		D	7.20	7.78	8.60	9.22	1.85	0.80	0.70
	Unfastened	A	1.82	1.87	1.94	1.99	1.80	0.85	0.75
		B	4.63	5.25	6.13	6.81	1.65	0.90	0.80
		C	2.16	2.29	2.47	2.62	1.65	0.90	0.80
		D	3.26	3.72	4.37	4.87	1.90	0.80	0.65
6ZS2.25x070	Fastened	A	1.59	1.96	2.49	2.90	1.75	0.85	0.75
		B	3.05	3.48	4.10	4.57	1.65	0.90	0.80
		C	1.93	2.23	2.66	2.98	1.75	0.85	0.75
		D	4.68	5.08	5.66	6.10	1.85	0.80	0.70
	Unfastened	A	1.22	1.26	1.31	1.35	1.80	0.85	0.75
		B	3.05	3.48	4.10	4.57	1.65	0.90	0.80
		C	1.28	1.37	1.49	1.58	1.65	0.90	0.80
		D	1.50	1.72	2.04	2.28	1.90	0.80	0.65
6ZS2.25x065	Fastened	A	1.37	1.70	2.17	2.52	1.75	0.85	0.75
		B	2.59	2.97	3.51	3.92	1.65	0.90	0.80
		C	1.63	1.89	2.25	2.53	1.75	0.85	0.75
		D	3.96	4.31	4.81	5.20	1.85	0.80	0.70
	Unfastened	A	1.05	1.08	1.13	1.16	1.80	0.85	0.75
		B	2.59	2.97	3.51	3.92	1.65	0.90	0.80
		C	1.04	1.11	1.21	1.29	1.65	0.90	0.80
		D	1.03	1.18	1.41	1.58	1.90	0.80	0.65

Table II - 15

F_y = 55 ksi

Web Crippling, P_n, kips [1,3]
Purlins/Girts
Z-Sections With Lips



Section	Fastened or Unfastened	Case[2]	Bearing Length[3], N, in.				ASD Ω_w	LRFD ϕ_w	LSD ϕ_w
			1	2	4	6			
6ZS2.25x059	Fastened	A	1.13	1.41	1.80	2.10	1.75	0.85	0.75
		B	2.09	2.41	2.86	3.20	1.65	0.90	0.80
		C	1.30	1.51	1.81	2.04	1.75	0.85	0.75
		D	3.18	3.47	3.89	4.21	1.85	0.80	0.70
	Unfastened	A	0.861	0.889	0.927	0.956	1.80	0.85	0.75
		B	2.09	2.41	2.86	3.20	1.65	0.90	0.80
		C	0.791	0.847	0.926	0.987	1.65	0.90	0.80
		D	0.537	0.622	0.743	0.835	1.90	0.80	0.65
4ZS2.25x070	Fastened	A	1.66	2.05	2.60	3.03	1.75	0.85	0.75
		B	3.11	3.55	4.18	4.66	1.65	0.90	0.80
		C	2.26	2.62	3.12	3.50	1.75	0.85	0.75
		D	5.20	5.65	6.29	6.77	1.85	0.80	0.70
	Unfastened	A	1.23	1.26	1.31	1.35	1.80	0.85	0.75
		B	3.11	3.55	4.18	4.66	1.65	0.90	0.80
		C	1.42	1.52	1.65	1.75	1.65	0.90	0.80
		D	1.50	1.72	2.04	2.29	1.90	0.80	0.65
4ZS2.25x065	Fastened	A	1.43	1.78	2.26	2.64	1.75	0.85	0.75
		B	2.64	3.03	3.58	4.00	1.65	0.90	0.80
		C	1.93	2.23	2.67	3.00	1.75	0.85	0.75
		D	4.43	4.82	5.38	5.81	1.85	0.80	0.70
	Unfastened	A	1.05	1.09	1.13	1.16	1.80	0.85	0.75
		B	2.64	3.03	3.58	4.00	1.65	0.90	0.80
		C	1.17	1.24	1.36	1.44	1.65	0.90	0.80
		D	1.03	1.18	1.41	1.58	1.90	0.80	0.65
4ZS2.25x059	Fastened	A	1.19	1.48	1.89	2.20	1.75	0.85	0.75
		B	2.14	2.46	2.92	3.27	1.65	0.90	0.80
		C	1.56	1.82	2.18	2.46	1.75	0.85	0.75
		D	3.58	3.91	4.38	4.74	1.85	0.80	0.70
	Unfastened	A	0.863	0.890	0.929	0.958	1.80	0.85	0.75
		B	2.14	2.46	2.92	3.27	1.65	0.90	0.80
		C	0.892	0.955	1.04	1.11	1.65	0.90	0.80
		D	0.538	0.624	0.744	0.836	1.90	0.80	0.65

Notes:

1. Web crippling strengths are nominal strengths [resistances] calculated without consideration of holes or other openings. To obtain available strengths [factored resistances], the values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD), which are provided in the table.
2. Case A
   End Reaction, Opposing Loads Spaced > 1.5h

   Case B
   Interior Reactions, Opposing Loads Spaced > 1.5h

   Case C
   End Reaction, Opposing Loads Spaced ≤ 1.5h

   Case D
   Interior Reactions, Opposing Loads Spaced ≤ 1.5h
3. Linear interpolation is permitted between bearing lengths.

Table II - 16a

Web Crippling Reduction Factor, R_c, for Interior Loading[1,2]
Joists/Studs
C-Sections With Lips

Loading Condition	Joist/ Stud Depth (in.)	Hole Depth (in.)	Distance between edge of Hole and Edge of Bearing (in.)						
			0.5	1	2	4	8	12	18
Interior One-Flange N ≥ 3 in.	12	1.5	0.90	0.90	0.90	0.91	0.93	0.95	0.98
	10	1.5	0.90	0.90	0.90	0.91	0.94	0.96	0.99
	8	1.5	0.89	0.90	0.90	0.92	0.95	0.97	1.00
	6	1.5	0.89	0.90	0.91	0.92	0.96	1.00	1.00
	5.5	1.5	0.89	0.90	0.91	0.93	0.97	1.00	1.00
	4	1.5	0.89	0.90	0.91	0.94	0.99	1.00	1.00
	3.625	1.5	0.89	0.89	0.91	0.94	1.00	1.00	1.00
	3.5	1.5	0.89	0.89	0.91	0.94	1.00	1.00	1.00
	2.5	0.75	0.90	0.91	0.93	0.98	1.00	1.00	1.00

Notes:
1. These factors only apply to openings with the listed dimensions.
2. Linear interpolation of R_c is permitted.

Table II - 16b

Web Crippling Reduction Factor, R_c, for End Loading[1,2]
Joists/Studs
C-Sections With Lips

Loading Condition	Joist/ Stud Depth (in.)	Hole Depth (in.)	Distance between edge of Hole and Edge of Bearing (in.)						
			0.5	1	1.5	2	3	4	5
End One-Flange N ≥ 1 in.	12	1.5	0.97	0.98	0.98	0.98	0.99	1.00	1.00
	10	1.5	0.96	0.97	0.97	0.98	0.99	0.99	1.00
	8	1.5	0.95	0.96	0.96	0.97	0.98	0.99	1.00
	6	1.5	0.93	0.94	0.95	0.95	0.97	0.98	1.00
	5.5	1.5	0.93	0.93	0.94	0.95	0.96	0.98	1.00
	4	1.5	0.89	0.90	0.91	0.92	0.95	0.97	0.99
	3.625	1.5	0.88	0.89	0.90	0.92	0.94	0.96	0.99
	3.5	1.5	0.87	0.89	0.90	0.91	0.94	0.96	0.99
	2.5	0.75	0.92	0.94	0.96	0.98	1.00	1.00	1.00

Notes:
1. These factors only apply to openings with the listed dimensions.
2. Linear interpolation of R_c is permitted.

SECTION 4 – EXAMPLE PROBLEMS

Example II-1: Four Span Continuous C-Purlins Attached to Through-Fastened Roof* - LRFD

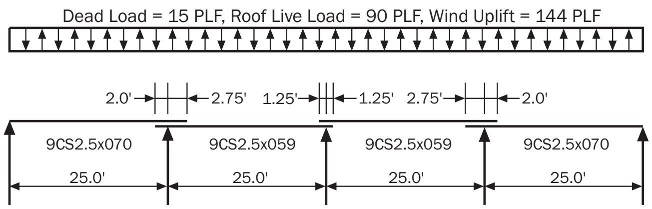


Figure 1 - Spans and Loading
Note: Lap dimensions are to connection points of purlins

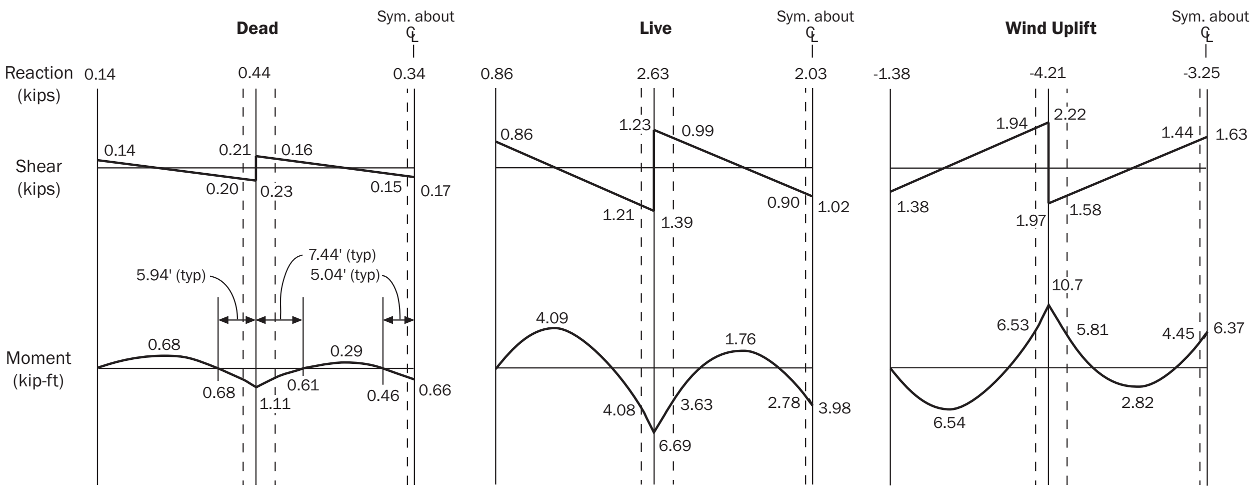


Figure 2 - Reactions, Shears and Moments
Note: Moments and forces are from unfactored nominal loads

Given:

1. Four span C-purlin system using laps at interior support points to create continuity (see Figures 1 and 2).
2. Roof covering is attached with through-fasteners along entire length of purlins.
3. F_y = 55 ksi
4. Purlins are lapped back to back over supports but all face in the same direction in a given bay.
5. Bottom flanges are bolted to supporting members with a bearing length of 5 in.
6. The roof covering provides a rotational stiffness to the top flange of the purlins of 0.300 kip-in./rad./in.

Required:

1. Check the design using LRFD with ASCE/SEI 7-10 load combinations for:
   a. Gravity Loads
   b. Wind Uplift Loads

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* For design of purlins supporting standing seam roof, see AISI publication D111-09: *Design Guide for Cold-Formed Steel Purlin Roof Framing Systems.*

Solution:

1. Assumptions for Analysis and Application of Specification Provisions

The *Specification* does not define the methods of analysis to be used; these judgments are the responsibility of the designer. The following assumptions are considered good practice but are not intended to prohibit other approaches:

a. The purlins are connected within the lapped portions in a manner that achieves full continuity between the individual purlin members.
b. In the continuous beam analysis, the shear and moment diagrams are based on continuous non-prismatic members between supports in which I_x within the lapped portions is the sum of the I_x of the individual members. Gross I_x values are used for the beam analysis.
c. The strength within the lapped portions is assumed to be the sum of the strengths of the individual members.
d. The attachment of the roof covering to the purlin provides continuous lateral-torsional support to the top flange.
e. The roof covering provides continuous distortional bracing of 0.300 kip-in./rad./in.
f. For gravity loads, the region at and near the interior supports is assumed to be not subject to lateral-torsional or distortional buckling between the support and the ends of the laps.
g. Under uniform gravity loading, the negative moment region between the end of the lap and the inflection point is assumed to have lateral-torsional and distortional buckling unbraced lengths equal to the distance from the end of the lap to the inflection point.
h. Since the loading, geometry and materials are symmetrical, only the first two spans are checked.

2. Calculation of Section Properties

Based on the design procedures illustrated in Examples I-1 and I-8 and Tables I-1 and II-1 of the AISI *Cold-Formed Steel Design Manual*, the following section properties have been obtained for the two C-Sections:

Section	9CS2.5x059	9CS2.5x070
D (in.)	9.00	9.00
t (in.)	0.059	0.070
R (in.)	0.1875	0.1875
A (in.^{2})	0.881	1.05
I_x (in.^{4})	10.3	12.2
S_f (in.^{3})	2.29	2.71
S_e (in.^{3})	1.89	2.47
I_y (in.^{4})	0.698	0.828
r_y (in.)	0.890	0.890
r_o (in.)	3.90	3.90
J (in.^{4})	0.00102	0.00171
C_w (in.^{6})	11.9	14.2

3. Check Gravity Loads

a. Strength for bending only (Section C3.1)

Required strength

Load combinations considered:

1.4D

1.2D + 1.6L_r

By inspection, 1.2D + 1.6L_r controls:

M_u = 1.2M_D + 1.6M_{Lr}

End span, from left to right:

Maximum positive moment: $M_u = (1.2)(0.68) + (1.6)(4.09) = 7.36$ kip-ft

Negative moment at end of right lap: $M_u = (1.2)(0.68) + (1.6)(4.08) = 7.34$ kip-ft

Negative moment at support: $M_u = (1.2)(1.11) + (1.6)(6.69) = 12.0$ kip-ft

Interior span, from left to right:

Negative moment at end of left lap: $M_u = (1.2)(0.61) + (1.6)(3.63) = 6.54$ kip-ft

Maximum positive moment: $M_u = (1.2)(0.29) + (1.6)(1.76) = 3.16$ kip-ft

Negative moment at end of right lap: $M_u = (1.2)(0.46) + (1.6)(2.78) = 5.00$ kip-ft

Negative moment at center support: $M_u = (1.2)(0.66) + (1.6)(3.98) = 7.16$ kip-ft

Design flexural strength

Compute the lowest of the applicable flexural strengths from Sections C3.1.1 (initiation of yielding), C3.1.2.1 (lateral-torsional buckling), C3.1.4 (distortional buckling) and D6.1.1 (flexural strength of members having tension flange through-fastened to deck or sheathing).

End span:

At location of maximum positive moment

The section is assumed to be fully braced against lateral-torsional buckling, but distortional buckling and yielding strengths must be calculated.

Calculate the design distortional buckling strength per Section C3.1.4.

A conservative distortional buckling strength can be calculated using *Commentary* Section C3.1.4 for members meeting the limitations of that section. In this case, use the more accurate provisions of *Specification* Section C3.1.4(a) to take advantage of the stiffness provided by the roof covering.

The cross-section has a single web and a single edge-stiffened flange as required by Section C3.1.4(a). Consider the contribution of the attached roof panel, which has a rotational stiffness, k_ϕ = 0.300 kip-in./rad./in. From Table II-7, for the 9CS2.5x070,

$k_{\phi fe} = 0.378$ kip

$\tilde{k}_{\phi fg} = 0.0118$ in.^{2}

$k_{\phi we} = 0.354$ kip

$\tilde{k}_{\phi wg} = 0.00245$ in.^{2}

Since there is no significant moment gradient in the vicinity of the maximum positive moment, use β = 1.0.

Calculate the elastic distortional buckling stress, F_d.

$$F_d = \beta \frac{k_{\phi fe} + k_{\phi we} + k_{\phi}}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \qquad (Eq.\ C3.1.4\text{-}6)$$

$$= (1.0)\frac{0.378 + 0.354 + 0.300}{0.0118 + 0.00245} = 72.4 \text{ ksi}$$

Calculate the design distortional buckling strength per Section C3.1.4

$$M_y = S_{fy}F_y \qquad (Eq.\ C3.1.4\text{-}4)$$

$$= (2.71)(55) = 149 \text{ kip-in.}$$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (2.71)(72.4) = 196 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y/M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{149/196} = 0.872 > 0.673 \text{ therefore,}$$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$$= (1 - 0.22(1/0.872))(1/0.872)(149) = 128 \text{ kip-in.} = 10.6 \text{ kip-ft}$$

$$\phi_b M_n = (0.90)(10.6) = 9.54 \text{ kip-ft} > 7.36 \text{ kip-ft OK} \qquad (Eq.\ A5.1.1\text{-}1)$$

Calculate the design strength based on initiation of yielding per Section C3.1.1(a)

For the exterior purlin, t = 0.070 in.

$$M_n = S_e F_y = (2.47)(55) = 136 \text{ kip-in.} = 11.3 \text{ kip-ft} \qquad (Eq.\ C3.1.1\text{-}1)$$

$$\phi_b M_n = (0.90)(11.3) = 10.2 \text{ kip-ft} > 7.36 \text{ kip-ft OK} \qquad (Eq.\ A5.1.1\text{-}1)$$

In the region of negative moment between the end of the right lap and the inflection point:

Calculate the design lateral-torsional buckling strength per Section C3.1.2.1(a).

Determine the nominal strength using the distance from the inflection point to the end of the right lap as the unbraced length.

$$L_y = L_t = 5.94 - 2.00 = 3.94 \text{ ft} = 47.3 \text{ in.}$$

C_b = 1.67 (Conservatively assumes linear moment diagram in this region.)

$$\sigma_{ey} = \frac{\pi^2 E}{\left(K_y L_y / r_y\right)^2} \qquad (Eq.\ C3.1.2.1\text{-}8)$$

$$= \frac{\pi^2 29500}{\left[(1.0)(47.3)/0.890\right]^2} = 103 \text{ ksi}$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$= \frac{1}{1.05(3.90)^2}\left[(11300)(0.00171) + \frac{\pi^2(29500)(14.2)}{\left[(1.0)(47.3)\right]^2}\right] = 117 \text{ ksi}$$

$$F_e = \frac{C_b r_o A}{S_f}\sqrt{\sigma_{ey}\sigma_t} \qquad (Eq.\ C3.1.2.1\text{-}4)$$

$$= \frac{(1.67)(3.90)(1.05)}{2.71}\sqrt{(103)(117)} = 277 \text{ ksi}$$

$$2.78F_y = (2.78)(55) = 153\,\text{ksi}$$

Since $F_e > 2.78\ F_y$, the section is not subject to lateral-torsional buckling.

Calculate the design distortional buckling strength per Section C3.1.4.

Calculate the elastic distortional buckling stress, F_d, for the negative moment region. Since the compression flange has no sheeting, there is no distortional restraint of the bottom flange, $k_\phi = 0$. Use *Specification* Section C3.1.4(a), since it provides a less conservative result than *Commentary* Section C3.1.4.

$$F_d = \beta \frac{k_{\phi fe} + k_{\phi we} + k_\phi}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \qquad (Eq.\ C3.1.4\text{-}6)$$

$$F_d / \beta = \frac{0.378 + 0.354 + 0.000}{0.0118 + 0.00245} = 51.4 \text{ ksi}$$

Alternatively, F_d/β, for the case where $k_\phi = 0$ may be taken from Table II-7,

$L_{cr} = 24.1$ in.

$F_d/\beta = 51.3 \text{ ksi} \approx 51.4 \text{ ksi}$

The bottom flange is not restrained from rotation by panel or other discrete bracing. The unbraced length for distortional buckling, L_m, is taken as the distance between the end of the lap and the inflection point.

$L_m = 47.3$ in. (from above).

$$L = \min(L_{cr},\ L_m)$$

$$= \min(24.1,\ 47.3) = 24.1 \text{ in.}$$

The moments at the ends of the segment are:

$M_1 = 0.0$ kip-ft at the inflection point

$M_2 = 7.34$ kip-ft at the lap

$$\beta = 1.0 \le 1 + 0.4(L/L_m)^{0.7}(1 - M_1/M_2)^{0.7} \le 1.3 \qquad (Eq.\ C3.1.4\text{-}7)$$

$$= 1.0 \le 1 + 0.4(24.1/47.3)^{0.7}(1 - 0.0/7.34)^{0.7} \le 1.3$$

$$= 1.0 \le 1.25 \le 1.3$$ therefore, use $\beta = 1.25$

$F_d = \beta(F_d/\beta)$ (using F_d/β from Eq. C3.1.4-6)

$$= 1.25(51.4) = 64.3 \text{ ksi}$$

Calculate the design distortional buckling strength per Section C3.1.4

$$M_y = S_{fy}F_y \qquad (Eq.\ C3.1.4\text{-}4)$$

$$= (2.71)(55) = 149\,\text{kip-in.}$$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (2.71)(64.3) = 174\,\text{kip-in.}$$

$$\lambda_d = \sqrt{M_y/M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{149/174} = 0.925 > 0.673$$ therefore,

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$$= (1 - 0.22(1/0.925))(1/0.925)(149) = 123 \text{ kip-in.} = 10.2 \text{ kip-ft}$$

$$\phi_b M_n = (0.90)(10.2) = 9.18 \text{ kip-ft} > 7.34 \text{ kip-ft OK} \quad (Eq.\ A5.1.1\text{-}1)$$

Calculate the design strength based on the initiation of yielding using Section C3.1.1(a).

$$M_n = S_e F_y = (2.47)(55) = 136 \text{ kip-in.} = 11.3 \text{ kip-ft} \quad (Eq.\ C3.1.1\text{-}1)$$

$$\phi_b M_n = (0.90)(11.3) = 10.2 \text{ kip-ft} > 7.34 \text{ kip-ft OK} \quad (Eq.\ A5.1.1\text{-}1)$$

In the lapped region over the support:

The lapped section is assumed to be sufficiently restrained against lateral-torsional buckling and distortional buckling.

Calculate the design flexural strength based on initiation of yielding per Section C3.1.1(a), summing the strength of the two overlapped purlins:

For the exterior purlin, t = 0.070 in.

$$M_n = S_e F_y = (2.47)(55) = 136 \text{ kip-in.} = 11.3 \text{ kip-ft} \quad (Eq.\ C3.1.1\text{-}1)$$

For the interior purlin, t = 0.059 in.

$$M_n = S_e F_y = (1.89)(55) = 104 \text{ kip-in.} = 8.66 \text{ kip-ft} \quad (Eq.\ C3.1.1\text{-}1)$$

For the combined strength of purlins,

$$\phi_b M_n = (0.90)(11.3 + 8.66) = 18.0 \text{ kip-ft} > 12.0 \text{ kip-ft OK} \quad (Eq.\ A5.1.1\text{-}1)$$

Interior span:

In the region of negative moment between the end of the left lap and the inflection point:

Calculate the design lateral-torsional buckling strength per Section C3.1.2.1(a).

Determine the nominal strength using the distance from the inflection point to the end of the left lap as the unbraced length.

$L_y = L_t = 7.44 - 2.75 = 4.69$ ft = 56.3 in.

$C_b = 1.67$ (Conservatively assumes linear moment diagram in this region.)

$$\sigma_{ey} = \frac{\pi^2 29500}{\left[(1.0)(56.3)/0.890\right]^2} = 72.8 \text{ ksi} \quad (Eq.\ C3.1.2.1\text{-}8)$$

$$\sigma_t = \frac{1}{0.881(3.90)^2}\left[(11300)(0.00102) + \frac{\pi^2(29500)(11.9)}{\left[(1.0)(56.3)\right]^2}\right] = 82.4 \text{ ksi} \quad (Eq.\ C3.1.2.1\text{-}9)$$

$$F_e = \frac{(1.67)(3.90)(0.881)}{2.29}\sqrt{(72.8)(82.4)} = 194 \text{ ksi} \quad (Eq.\ C3.1.2.1\text{-}4)$$

$2.78F_y = (2.78)(55) = 153$ ksi

Since $F_e > 2.78\ F_y$, the section is not subject to lateral-torsional buckling.

Calculate the design distortional buckling strength per Section C3.1.4.

Since there is no distortional restraint of the bottom flange, take F_d/β from Table II-7

From Table II-7, for the 9CS2.5x059,

$F_d/\beta = 41.2$ ksi

$L_{cr} = 25.8$ in.

The unbraced length for distortional buckling, L_m, is taken as the distance between the end of the lap and the inflection point.

$L_m = 56.3$ in. (from above)

$L \;=\; \min(L_{cr},\, L_m)$

$\;\;\;= \min(25.8,\; 56.3) = 25.8 \text{ in.}$

The moments at the ends of the segment are:

M_1 = 0.0 kip-ft at the inflection point

M_2 = 6.54 kip-ft at the lap

$\beta \;=\; 1.0 \le 1 + 0.4(L/L_m)^{0.7}(1 - M_1/M_2)^{0.7} \le 1.3$ (*Eq.* C3.1.4-7)

$\;\;\;= 1.0 \le 1 + 0.4(25.8/56.3)^{0.7}(1 - 0.0/6.54)^{0.7} \le 1.3$

$\;\;\;= 1.0 \le 1.23 \le 1.3$ therefore, use β = 1.23

$F_d \;=\; \beta(F_d/\beta)$

$\;\;\;= 1.23(41.2)$ = 50.7 ksi

Calculate the design distortional buckling strength per Section C3.1.4.

$M_y \;=\; S_{fy}F_y$ (*Eq.* C3.1.4-4)

$\;\;\;= (2.29)(55) = 126 \text{ kip-in.}$

$M_{crd} \;= S_f F_d$ (*Eq.* C3.1.4-5)

$\;\;\;= 2.29(50.7) = 116 \text{ kip-in.}$

$\lambda_d \;= \sqrt{M_y/M_{crd}}$ (*Eq.* C3.1.4-3)

$\;\;\;= \sqrt{126/116} = 1.04 > 0.673$ therefore,

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (\textit{Eq. } \text{C3.1.4-2})$$

$\;\;\;= (1 - 0.22(1/1.04))(1/1.04)(126)$ = 95.5 kip-in. = 7.96 kip-ft

$\phi_b M_n = (0.90)(7.96)$ = 7.16 kip-ft > 6.54 kip-ft OK (*Eq.* A5.1.1-1)

Calculate the design strength based on the initiation of yielding using Section C3.1.1(a).

$M_n = S_e F_y = (1.89)(55)$ = 104 kip-in. = 8.66 kip-ft (*Eq.* C3.1.1.-1)

$\phi_b M_n = (0.90)(8.66)$ = 7.79 kip-ft > 6.54 kip-ft OK (*Eq.* A5.1.1-1)

At the location of maximum positive moment

The section is assumed to be fully braced against lateral-torsional buckling.

Calculate the design distortional buckling strength per Section C3.1.4.

The cross-section has a single web and a single edge-stiffened flange as required by Section C3.1.4(a). Consider the contribution of the attached roof panel, which has a rotational stiffness, k_ϕ = 0.300 kip-in./rad./in. From Table II-7, for the 9CS2.5x059,

$k_{\phi fe} = 0.221 \text{ kip}$

$\tilde{k}_{\phi fg} = 0.00863 \text{ in.}^2$

$k_{\phi we} = 0.209 \text{ kip}$

$\tilde{k}_{\phi wg} = 0.00181 \text{ in.}^2$

Since there is no significant moment gradient in the vicinity of the maximum positive moment, use β = 1.0.

Calculate the elastic distortional buckling stress, F_d.

$$F_d = \beta \frac{k_{\phi fe} + k_{\phi we} + k_{\phi}}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \qquad (Eq.\ C3.1.4\text{-}6)$$

$$= (1.0)\frac{0.221 + 0.209 + 0.300}{0.00863 + 0.00181} = 69.9 \text{ ksi}$$

Calculate the design distortional buckling strength per Section C3.1.4

$$M_y = S_{fy}F_y \qquad (Eq.\ C3.1.4\text{-}4)$$

$$= (2.29)(55) = 126 \text{ kip-in.}$$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (2.29)(69.9) = 160 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y / M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{126/160} = 0.887 > 0.673 \text{ therefore,}$$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$$= (1 - 0.22(1/0.887))(1/0.887)(126) = 107 \text{ kip-in.} = 8.90 \text{ kip-ft}$$

$$\phi_b M_n = (0.90)(8.90) = 8.01 \text{ kip-ft} > 3.16 \text{ kip-ft OK} \qquad (Eq.\ A5.1.1\text{-}1)$$

Calculate the design strength based on the initiation of yielding using Section C3.1.1(a).

$$\phi_b M_n = (0.90)(8.66) = 7.79 \text{ kip-ft} > 3.16 \text{ kip-ft OK} \qquad (Eq.\ A5.1.1\text{-}1)$$

In the region of negative moment between the end of the right lap and the inflection point:

Determine the nominal strength using the distance from the inflection point to the end of the right lap as the unbraced length.

$$L = 5.04 - 1.25 = 3.79 \text{ ft} = 45.5 \text{ in.}$$

By inspection, this condition is less severe than the left lap, since the unbraced length is shorter and the required strength is less, therefore the section is OK.

In the lapped region over the center support:

At the negative moment at the center support, the section is assumed to be fully braced.

Calculate the nominal strength based on initiation of yielding per Section C3.1.1(a), summing the strength of the two overlapped purlins:

For the combined strength of purlins,

$$\phi_b M_n = (0.90)(8.66 + 8.66) = 15.6 \text{ kip-ft} > 7.16 \text{ kip-ft OK} \qquad (Eq.\ A5.1.1\text{-}1)$$

b. Strength for shear only (Section C3.2)

Required strength

By inspection, the load combination $1.2D + 1.6L_r$ controls:

$$V_u = 1.2V_D + 1.6V_{Lr}$$

End span, from left to right:

At left support: $V_u = (1.2)(0.14) + (1.6)(0.86) = 1.54$ kips

At end of right lap: $V_u = (1.2)(0.20) + (1.6)(1.21) = 2.18$ kips

At left side of first interior support: $V_u = (1.2)(0.23) + (1.6)(1.39) = 2.50$ kips

Interior span, from left to right:

At right side of first interior support: $V_u = (1.2)(0.21) + (1.6)(1.23) = 2.22$ kips

At end of left lap: $V_u = (1.2)(0.16) + (1.6)(0.99) = 1.78$ kips

At end of right lap: $V_u = (1.2)(0.15) + (1.6)(0.90) = 1.62$ kips

At center support: $V_u = (1.2)(0.17) + (1.6)(1.02) = 1.84$ kips

Design strength

End span:

At the left support and right lap, t=0.070 in. By inspection the right lap controls.

For t = 0.070 in.,

$h = D - 2t - 2R = 9.00 - 2(0.070) - 2(0.1875) = 8.485$ in.

$h/t = 8.485 / 0.070 = 121$

Since the web is unreinforced, $k_v = 5.34$

$1.51\sqrt{Ek_v/F_y} = 1.51\sqrt{(29500)(5.34)/55} = 80.8$

For $\frac{h}{t} > 1.51\sqrt{Ek_v/F_y}$,

$$F_v = \frac{\pi^2 E k_v}{12(1-\mu^2)(h/t)^2} \quad (Eq.\ C3.2.1\text{-}4a)$$

$$= \frac{\pi^2(29500)(5.34)}{12(1-0.3^2)(8.485/0.070)^2} = 9.69 \text{ ksi}$$

$$V_n = A_w F_v \quad (Eq.\ C3.2.1\text{-}1)$$

$$= (8.485)(0.070)(9.69) = 5.76 \text{ kips}$$

Alternately, V_n can be taken from Table II-1, Beam Properties, C-Sections With Lips, F_y = 55 ksi. For a 9CS2.5x070, V_n is 5.76 kips.

$\phi_v V_n = (0.95)(5.76) = 5.47$ kips > 2.18 kips OK (*Eq.* A5.1.1-1)

At the left side of the first interior support, sum the strength of the two overlapped purlins:

For t = 0.059 in.,

$h = 9.00 - 2(0.059) - 2(0.1875) = 8.507$ in.

$h/t = 8.507 / 0.059 = 144 > 80.8$

$$F_v = \frac{\pi^2(29500)(5.34)}{12(1-0.3^2)(8.507/0.059)^2} = 6.85 \text{ ksi} \quad (Eq.\ C3.2.1\text{-}4a)$$

$$V_n = A_w F_v \quad (Eq.\ C3.2.1\text{-}1)$$

$$= (8.507)(0.059)(6.85) = 3.44 \text{ kips}$$

Alternately, V_n can be taken from Table II-1, Beam Properties, C-Sections With Lips, F_y = 55 ksi. For a 9CS2.5x059, V_n is 3.44 kips.

For the combined strength of the purlins,

$\phi_v V_n = (0.95)(5.76 + 3.44) = 8.74$ kips > 2.50 kips OK (*Eq.* A5.1.1-1)

Interior span:

At the right side of the first interior support, use the strength computed above:

$\phi_v V_n$ = 8.74 kip > 2.22 kips OK (*Eq.* A5.1.1-1)

By inspection of the left and right laps, the left lap controls

$\phi_v V_n = (0.95)(3.44) = 3.27$ kips > 1.78 kips OK (*Eq.* A5.1.1-1)

At the center support, sum the strength of the two overlapped purlins:

For the combined strength of the purlins,

$\phi_v V_n = (0.95)(3.44+3.44) = 6.54$ kips > 1.84 kips OK (*Eq.* A5.1.1-1)

c. Strength for combined bending and shear (Section C3.3.2)

End span:

$$\sqrt{\left(\frac{\overline{M}}{\phi_b M_{nxo}}\right)^2 + \left(\frac{\overline{V}}{\phi_v V_n}\right)^2} \le 1.0 \qquad (Eq.\ C3.3.2\text{-}1)$$

where

M_{nxo} = M_n calculated based on the initiation of yielding per Section C3.1.1

$\overline{M}$ = M_u

$\overline{V}$ = V_u

ϕ_b = 0.90

ϕ_v = 0.95

At the left side of the right lap, t = 0.070 in.

$$\sqrt{\left(\frac{7.34}{(0.90)(11.3)}\right)^2 + \left(\frac{2.18}{(0.95)(5.76)}\right)^2} = 0.824 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.2\text{-}1)$$

At first interior support,

$$\sqrt{\left(\frac{12.0}{(0.90)(11.3+8.66)}\right)^2 + \left(\frac{2.50}{(0.95)(5.76+3.44)}\right)^2} = 0.727 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.2\text{-}1)$$

Interior span:

At end of laps, t = 0.059 in. Left lap controls by inspection.

$$\sqrt{\left(\frac{6.54}{(0.90)(8.66)}\right)^2 + \left(\frac{1.78}{(0.95)(3.44)}\right)^2} = 1.0 = 1.0 \text{ OK} \qquad (Eq.\ C3.3.2\text{-}1)$$

At center support,

$$\sqrt{\left(\frac{7.16}{(0.90)(8.66+8.66)}\right)^2 + \left(\frac{1.84}{(0.95)(3.44+3.44)}\right)^2} = 0.539 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.2\text{-}1)$$

d. Web crippling strength (Section C3.4)

Required strength

By inspection, the load combination 1.2D + 1.6L_r controls:

$P_u = 1.2P_D + 1.6P_{Lr}$

Supports, from left to right:

At left support: $P_u = (1.2)(0.14) + (1.6)(0.86) = 1.54$ kips

At first interior support: $P_u = (1.2)(0.44) + (1.6)(2.63) = 4.74$ kips

At center support: $P_u = (1.2)(0.34) + (1.6)(2.03) = 3.66$ kips

Design strength

$$P_n = Ct^2F_y\sin\theta\left(1-C_R\sqrt{\frac{R}{t}}\right)\left(1+C_N\sqrt{\frac{N}{t}}\right)\left(1-C_h\sqrt{\frac{h}{t}}\right) \quad (Eq.\ C3.4.1\text{-}1)$$

where

F_y= 55 ksi
θ = 90 degrees
R = 0.1875 in.
N= bearing length = 5.0 in.

At end supports:

h = 8.485 in.
t = 0.070 in.

From Table C3.4.1-2, using the coefficients for the case of:

Fastened to Support/One-Flange Loading or Reaction/End

C = 4
C_R = 0.14
C_N = 0.35
C_h = 0.02
ϕ_w = 0.85

Check Limit: R/t = 0.1875/0.070 = 2.7 < 9 OK

$$P_n = (4)(0.070)^2(55)\sin(90)\left(1-0.14\sqrt{\frac{0.1875}{0.070}}\right)\left(1+0.35\sqrt{\frac{5.0}{0.070}}\right)\left(1-0.02\sqrt{\frac{8.485}{0.070}}\right)$$

$$= 2.56 \text{ kips} \quad (Eq.\ C3.4.1\text{-}1)$$

$$\phi_w P_n = (0.85)(2.56) = 2.18 \text{ kips} > 1.54 \text{ kips OK} \quad (Eq.\ A5.1.1\text{-}1)$$

At interior supports:

From Table C3.4.1-2, using the coefficients for the case of:

Fastened to Support/One-Flange Loading or Reaction/Interior

C = 13
C_R = 0.23
C_N = 0.14
C_h = 0.01
ϕ_w = 0.90

for t = 0.070 in.,

Check Limit: R/t = 0.1875/0.070 = 2.7 < 5 OK

$$P_n = (13)(0.070)^2(55)\sin(90)\left(1-0.23\sqrt{\frac{0.1875}{0.070}}\right)\left(1+0.14\sqrt{\frac{5.0}{0.070}}\right)\left(1-0.01\sqrt{\frac{8.485}{0.070}}\right)$$

$$= 4.24 \text{ kips} \quad (Eq.\ C3.4.1\text{-}1)$$

for t = 0.059 in.,

Check Limit: R/t = 0.1875/0.059 = 3.2 < 5 OK

$$P_n = (13)(0.059)^2(55)\sin(90)\left(1-0.23\sqrt{\frac{0.1875}{0.059}}\right)\left(1+0.14\sqrt{\frac{5.0}{0.059}}\right)\left(1-0.01\sqrt{\frac{8.507}{0.059}}\right)$$

$$= 2.96 \text{ kips} \quad (Eq.\ C3.4.1\text{-}1)$$

Using the values of P_n calculated from *Eq.* C3.4.1-1 above,

At first interior support,

$\phi_w P_n = (0.90)(4.24+2.96) = 6.48$ kips > 4.74 kips OK (*Eq.* A5.1.1-1)

At center support,

$\phi_w P_n = (0.90)(2.96+2.96) = 5.33$ kips > 3.66 kips OK (*Eq.* A5.1.1-1)

e. Combined bending and web crippling (Section C3.5.2(a))

$$0.91\left(\frac{\overline{P}}{P_n}\right)+\left(\frac{\overline{M}}{M_{nxo}}\right) \le 1.33\phi \quad (Eq.\ C3.5.2\text{-}1)$$

where

$\overline{P} = P_u$

$\overline{M} = M_u$

P_n = the sum of P_n of each purlin at the support

M_{nxo} = the sum of M_n of each purlin at the support calculated based on the initiation of yielding per Section C3.1.1

$\phi = 0.90$

$1.33\phi = (1.33)(0.90) = 1.20$

At the first interior support,

$$0.91\left(\frac{4.74}{4.24+2.96}\right)+\left(\frac{12.0}{11.3+8.66}\right) = 1.20 \le 1.20 \quad \text{OK} \quad (Eq.\ C3.5.2\text{-}1)$$

At the center support,

$$0.91\left(\frac{3.66}{2.96+2.96}\right)+\left(\frac{7.16}{8.66+8.66}\right) = 0.976 < 1.20 \quad \text{OK} \quad (Eq.\ C3.5.2\text{-}1)$$

4. Check Uplift Loads

a. Strength for bending only (Section D6.1.1)

Required strength

By inspection, load combination 0.9D + 1.0W controls.

$M_u = 0.9M_D + 1.0M_W$

End span:

Maximum negative moment: $M_u = (0.9)(0.68)+(1.0)(-6.54) = -5.93$ kip-ft

Interior span:

Maximum negative moment: $M_u = (0.9)(0.29)+(1.0)(-2.82) = -2.56$ kip-ft

Design strength

$M_n = R\,S_e\,F_y$ (*Eq.* D6.1.1-1)

R = 0.60, assuming all 15 conditions of Section D6.1.1 are satisfied

ϕ_b = 0.90

End span:

For t = 0.070 in.,

$M_n = (0.60)(2.47)(55) = 81.5$ kip-in. = 6.79 kip-ft (*Eq.* D6.1.1-1)

$\phi_b M_n = (0.90)(6.79) = 6.11$ kip-ft > 5.93 kip-ft OK (*Eq.* A5.1.1-1)

Interior span:

For t = 0.059 in.,

$M_n = (0.60)(1.89)(55) = 62.4$ kip-in. = 5.20 kip-ft (*Eq.* D6.1.1-1)

$\phi_b M_n = (0.90)(5.20) = 4.68$ kip-ft > 2.56 kip-ft OK (*Eq.* A5.1.1-1)

b. Other comments

All other regions of the system have their compression flange braced by the roof panel. Since the magnitude of the shears, moments and reactions are less than those of the gravity case, it can be concluded by inspection that the design satisfies the *Specification* criteria for uplift.

5. Anchorage Forces

A check of anchorage forces is required to complete the design. Refer to the *Design Guide for Cold-Formed Steel Purlin Roof Framing Systems** for further information and examples of this check.

* AISI publication D111-09: *Design Guide for Cold-Formed Steel Purlin Roof Framing Systems.*

Example II-2: Four Span Continuous Z-Purlins Attached to Through-Fastened Roof* - ASD

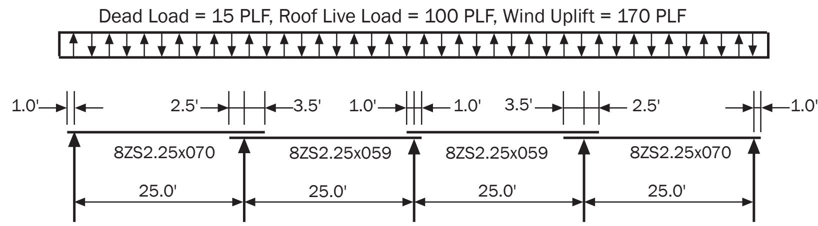

Figure 1 - Spans and Loading

Note: Lap dimensions are to connection points of purlins

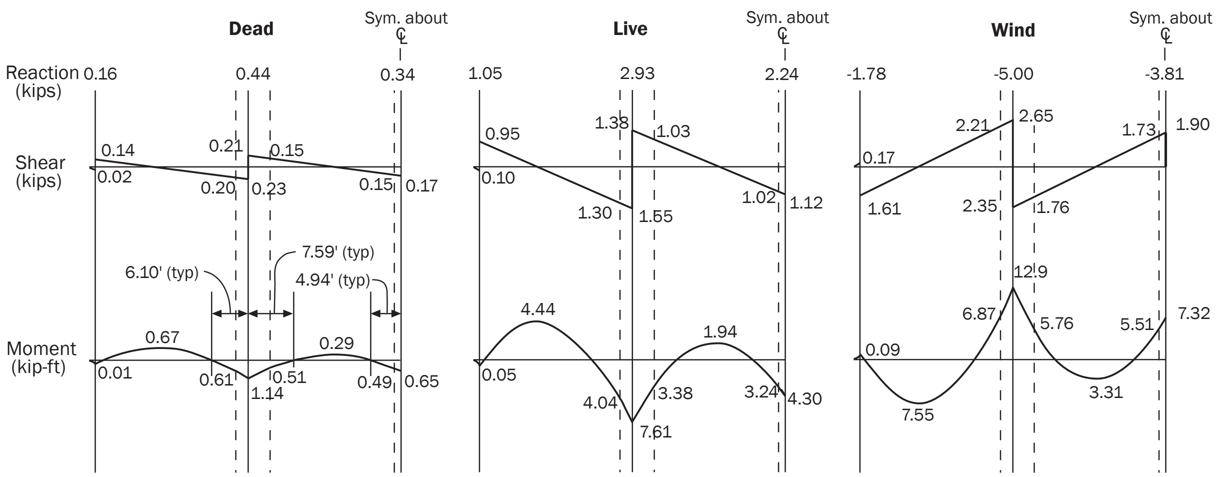

Figure 2 - Reactions, Shears and Moments

Note: Moments and forces are from unfactored nominal loads

Given:

1. Four span Z-purlin system using laps at interior support points to create continuity (see Figures 1 and 2).
2. Roof covering is attached with through-fasteners along entire length of purlins.
3. F_y = 55 ksi
4. No discrete bracing lines: anti-roll clips at each support at every fourth purlin line
5. Bottom flanges are bolted to a 0.25 in. thick supporting member with a bearing length of 5 in.
6. The roof covering provides a rotational stiffness to the top flange of the purlins of 0.300 kip-in./rad./in.

Required:

1. Check the design using ASD with ASCE/SEI 7-10 load combinations for:

   (a) Gravity loads

   (b) Wind uplift loads

2. Compute the anchorage forces at the supports under gravity loads.

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* For design of purlins supporting standing seam roof, see AISI publication D111-09: *Design Guide for Cold-Formed Steel Purlin Roof Framing Systems.*

Solution:

1. Assumptions for Analysis and Application of Specification Provisions

The *Specification* does not define the methods of analysis to be used; these judgments are the responsibility of the designer. The following assumptions are considered good practice but are not intended to prohibit other approaches:

a. The purlins are connected within the lapped portions in a manner that achieves full continuity between the individual purlin members.
b. In the continuous beam analysis, the shear and moment diagrams are based on continuous non-prismatic members between supports in which I_x within the lapped portions is the sum of the I_x of the individual members. Gross values of I_x are used for the beam analysis.
c. The strength within the lapped portions is assumed to be the sum of the strengths of the individual members.
d. The attachment of the roof covering to the purlin provides continuous lateral-torsional support to the top flange.
e. The roof covering provides continuous distortional bracing of 0.300 kip-in./rad./in.
f. For gravity loads, the region at and near the interior supports is assumed to be not subject to lateral-torsional or distortional buckling between the support and the ends of the laps.
g. Under uniform gravity loading, the negative moment region between the end of the lap and the inflection point is assumed to have lateral-torsional and distortional buckling unbraced lengths equal to the distance from the end of the lap to the inflection point.
h. Since the loading, geometry and materials are symmetrical, only the first two spans are checked.

2. Calculation of Section Properties

Based on the design procedures illustrated in Examples I-3 and I-10 and Table I-4 and II-4 of the *AISI Cold-Formed Steel Design Manual*, the following section properties have been obtained for the two Z-sections:

Section	8ZS2.25x059	8ZS2.25x070
D (in.)	8.000	8.000
t (in.)	0.059	0.070
R (in.)	0.1875	0.1875
I_x (in.^{4})	7.76	9.18
S_f (in.^{3})	1.94	2.30
S_e (in.^{3})	1.80	2.25
I_y (in.^{4})	1.08	1.28

3. Check Gravity Loads

a. Strength for bending only (Section C3.1)

Required allowable strength
By inspection, the load combination $D + L_r$ controls:

$M = M_D + M_{Lr}$

Overhang:

Maximum negative moment:	M = 0.01 + 0.05 = 0.06 kip-ft

End span, from left to right:

Maximum positive moment:	M = 0.67 + 4.44 = 5.11 kip-ft
Negative moment at end of right lap:	M = 0.61 + 4.04 = 4.65 kip-ft
Negative moment at support:	M = 1.14 + 7.61 = 8.75 kip-ft

Interior span, from left to right:

Negative moment at end of left lap:	M = 0.51 + 3.38 = 3.89 kip-ft
Maximum positive moment:	M = 0.29 + 1.94 = 2.23 kip-ft
Negative moment at end of right lap:	M = 0.49 + 3.24 = 3.73 kip-ft
Negative moment at center support:	M = 0.65 + 4.30 = 4.95 kip-ft

Allowable flexural strength

Compute the lowest of the applicable flexural strengths from Sections C3.1.1 (initiation of yielding), C3.1.2 (lateral-torsional buckling), C3.1.4 (distortional buckling), and D6.1.1 (flexural strength of members having tension flange through-fastened to deck or sheathing).

Overhang:

By inspection, due to the short unbraced length and insignificant bending moment, the overhang is acceptable. Calculations for the negative moment region of the end span below demonstrate that the allowable bending strength of the section greatly exceeds the required strength at the overhang.

End span:

At location of maximum positive moment

The section is assumed to be fully braced against lateral-torsional buckling, but distortional buckling and yielding strengths must be calculated.

Calculate the allowable distortional buckling strength per Section C3.1.4.

A conservative distortional buckling strength can be calculated using *Commentary* Section C3.1.4 for members meeting the limitations of that section. In this case, use the more accurate provisions of *Specification* Section C3.1.4(a) to take advantage of the stiffness provided by the roof covering.

Calculate the elastic distortional buckling stress, F_d.

The cross-section has a single web and a single edge-stiffened flange as required by Section C3.1.4(a). Consider the contribution of the attached roof panel, which has a rotational stiffness, k_ϕ = 0.300 kip-in./rad./in. From Table II-9 for the 8ZS2.25x0.070,

$$k_{\phi fe} = 0.437 \text{ kip}$$

$$\tilde{k}_{\phi fg} = 0.0153 \text{ in.}^2$$

$$k_{\phi we} = 0.402 \text{ kip}$$

$$\tilde{k}_{\phi wg} = 0.00231 \text{ in.}^2$$

Since there is no significant moment gradient in the vicinity of the maximum positive moment, use β = 1.0.

$$F_d = \beta \frac{k_{\phi fe} + k_{\phi we} + k_\phi}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \qquad (Eq.\ C3.1.4\text{-}6)$$

$$=(1.0)\frac{0.437+0.402+0.300}{0.0153+0.00231} = 64.7 \text{ ksi}$$

Calculate the allowable distortional buckling strength per Section C3.1.4

$$M_y = S_{fy}F_y \qquad (Eq.\ C3.1.4\text{-}4)$$

$$= (2.30)(55) = 127 \text{ kip-in.}$$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (2.30)(64.7) = 149 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y / M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{127/149} = 0.923 > 0.673 \text{ therefore,}$$

$$M_n = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$$= \left(1-0.22\left(1/0.923\right)\right)\left(1/0.923\right)(127) = 105 \text{ kip-in.} = 8.73 \text{ kip-ft}$$

$$\frac{M_n}{\Omega_b} = \frac{8.73}{1.67} = 5.23 \text{ kip-ft} > 5.11 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

Calculate the allowable strength based on initiation of yielding per Section C3.1.1(a).

For the exterior purlin, t = 0.070 in.

$$M_n = S_e F_y = (2.25)(55) = 124 \text{ kip-in.} = 10.3 \text{ kip-ft} \qquad (Eq.\ C3.1.1\text{-}1)$$

$$\frac{M_n}{\Omega_b} = \frac{10.3}{1.67} = 6.17 \text{ kip-ft} > 5.11 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

In the region of negative moment between the end of the right lap and the inflection point:

Calculate the allowable lateral-torsional buckling strength per Section C3.1.2.1(b).

Determine the nominal strength using the distance from the inflection point to the end of the right lap as the unbraced length.

$$L_y = 6.10 - 2.50 = 3.60 \text{ ft} = 43.2 \text{ in.}$$

$$K_y = 1.0$$

$$I_{yc} = \frac{I_y}{2} = \frac{1.28}{2} = 0.640 \text{ in.}^4$$

$C_b = 1.67$ (Conservatively assumes linear moment diagram in this region)

$$F_e = \frac{C_b \pi^2 E d I_{yc}}{2S_f\left(K_y L_y\right)^2} \qquad (Eq.\ C3.1.2.1\text{-}15)$$

$$= \frac{1.67\pi^2(29500)(8.0)(0.640)}{(2)(2.30)\left(1.0(43.2)\right)^2} = 290 \text{ ksi}$$

$2.78F_y = (2.78)(55) = 153$ ksi

Since $F_e > 2.78F_y$, the section is not subject to lateral-torsional buckling.

Calculate the allowable distortional buckling strength per Section C3.1.4.

Calculate the elastic distorsional buckling stress, F_d, for the negative moment region. Since the compression flange has no sheeting, there is no distortional restraint of the bottom flange, $k_\phi = 0$. Use *Specification* Section C3.1.4(a) since it provides a less conservative result than *Commentary* Section C3.1.4.

$$F_d = \beta \frac{k_{\phi fe} + k_{\phi we} + k_{\phi}}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \qquad (Eq.\ C3.1.4\text{-}6)$$

$$F_d/\beta = \frac{0.437 + 0.402 + 0.0}{0.0153 + 0.00231} = 47.6 \text{ ksi}$$

Alternately, F_d/β, for the case where $k_\phi = 0$, may be taken from Table II-9:

$$F_d/\beta = 47.7 \text{ ksi} \approx 47.6 \text{ ksi}$$

From Table II-9,

L_{cr} = 20.8 in.

The bottom flange is not restrained from rotation by panel or other discrete bracing. Therefore, the unbraced length for distortional buckling, L_m, is taken as the distance between the end of the lap and the inflection point.

L_m = 43.2 in. (from above)

$L = \min(L_{cr}, L_m)$

$= \min(20.8, 43.2) = 20.8$ in.

The moments at the ends of the segment are:

M_1 = 0.0 kip-ft at the inflection point

M_2 = 4.65 kip-ft at the lap

$$\beta = 1.0 \le 1 + 0.4(L/L_m)^{0.7}(1 - M_1/M_2)^{0.7} \le 1.3 \qquad (Eq.\ C3.1.4\text{-}7)$$

$$= 1.0 \le 1 + 0.4(20.8/43.2)^{0.7}(1 - 0.0/4.65)^{0.7} \le 1.3$$

$= 1.0 \le 1.24 \le 1.3$ therefore, use $\beta = 1.24$

$F_d = \beta(F_d/\beta)$

$= 1.24(47.6) = 59.0$ ksi

Calculate the allowable distortional buckling strength per Section C3.1.4

$$M_y = S_{fy}F_y \qquad (Eq.\ C3.1.4\text{-}4)$$

$$= (2.30)(55) = 127 \text{ kip-in.}$$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (2.30)(59.0) = 136 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y/M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$= \sqrt{127/136} = 0.966 > 0.673$ therefore,

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$= (1 - 0.22(1/0.966))(1/0.966)(127) = 102$ kip-in. = 8.46 kip-ft

$$\frac{M_n}{\Omega_b} = \frac{8.46}{1.67} = 5.07 \text{ kip-ft} > 4.65 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

Calculate the allowable strength based on the initiation of yielding using Section C3.1.1(a).

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

$= (2.25)(55.0) = 124$ kip-in. = 10.3 kip-ft

$$\frac{M_n}{\Omega_b} = \frac{10.3}{1.67} = 6.17 \text{ kip-ft} > 4.65 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

In the lapped region over the support:

The lapped section is assumed to be sufficiently restrained against lateral-torsional buckling and distortional buckling.

Calculate the allowable flexural strength based on initiation of yielding per Section C3.1.1(a), summing the strength of the two overlapped purlins:

For the exterior purlin, t = 0.070 in.

$$M_n = S_e F_y = (2.25)(55) = 124 \text{ kip-in.} = 10.3 \text{ kip-ft} \qquad (Eq.\ C3.1.1\text{-}1)$$

For the interior purlin, t = 0.059 in.

$$M_n = S_e F_y = (1.80)(55) = 99.0 \text{ kip-in.} = 8.25 \text{ kip-ft} \qquad (Eq.\ C3.1.1\text{-}1)$$

For the combined strength of purlins,

$$\frac{M_n}{\Omega_b} = \frac{10.3 + 8.25}{1.67} = 11.1 \text{ kip-ft} > 8.75 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

Interior span:

In the region of negative moment between the end of the left lap and the inflection point:

Calculate the allowable lateral-torsional buckling strength per Section C3.1.2.1(b).

Determine the nominal strength using the distance from the inflection point to the end of the left lap as the unbraced length.

$L_y = 7.59 - 3.50 = 4.09$ ft = 49.1 in.

$K_y = 1.0$

$$I_{yc} = \frac{I_y}{2} = \frac{1.08}{2} = 0.540 \text{ in.}^4$$

C_b = 1.67 (Conservatively assumes linear moment diagram in this region)

$$F_e = \frac{1.67\pi^2(29500)(8.0)(0.540)}{(2)1.94(1.0(49.1))^2} = 225 \text{ ksi} \qquad (Eq.\ C3.1.2.1\text{-}15)$$

$2.78F_y = (2.78)(55) = 153$ ksi

Since $F_e > 2.78F_y$, the section is not subject to lateral-torsional buckling.

Calculate the allowable distortional buckling strength per Section C3.1.4.

Since there is no distortional restraint of the bottom flange, take F_d/β from Table II-9

From Table II-9, for the 8ZS2.25x059,

$F_d/\beta = 38.6$ ksi

L_{cr} = 22.4 in.

The unbraced length for distortional buckling, L_m, is taken as the distance between the end of the lap and the inflection point.

L_m = 49.1 in. (from above).

$L = \min(L_{cr}, L_m)$

$= \min(22.4, 49.1) = 22.4$ in.

The moments at the ends of the segment are:

M_1 = 0.0 kip-ft at the inflection point

M_2 = 3.89 kip-ft at the lap

$$\beta = 1.0 \le 1 + 0.4(L/L_m)^{0.7}(1 - M_1/M_2)^{0.7} \le 1.3 \qquad (Eq.\ C3.1.4\text{-}7)$$

$= 1.0 \le 1 + 0.4\left(22.4/49.1\right)^{0.7}\left(1 - 0.0/3.89\right)^{0.7} \le 1.3$

$= 1.0 \le 1.23 \le 1.3$ therefore, use $\beta = 1.23$

$F_d = \beta\left(F_d/\beta\right)$

$= 1.23\left(38.6\right) = 47.5$ ksi

Calculate the allowable distortional buckling strength per Section C3.1.4

$M_y = S_{fy}F_y$ (*Eq.* C3.1.4-4)

$= (1.94)(55) = 107$ kip-in.

$M_{crd} = S_f F_d$ (*Eq.* C3.1.4-5)

$= (1.94)(47.5) = 92.2$ kip-in.

$\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* C3.1.4-3)

$= \sqrt{107/92.2} = 1.08 > 0.673$ therefore,

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (\textit{Eq. } \text{C3.1.4-2})$$

$= \left(1 - 0.22\left(1/1.08\right)\right)\left(1/1.08\right)\left(107\right) = 78.9$ kip-in. $= 6.57$ kip-ft

$\frac{M_n}{\Omega_b} = \frac{6.57}{1.67} = 3.93$ kip-ft > 3.89 kip-ft OK (*Eq.* A4.1.1-1)

Calculate the allowable strength based on the initiation of yielding using Section C3.1.1(a).

$M_n = S_e F_y = (1.80)(55.0) = 99.0$ kip-in. $= 8.25$ kip-ft (*Eq.* C3.1.1-1)

$\frac{M_n}{\Omega_b} = \frac{8.25}{1.67} = 4.94$ kip-ft > 3.89 kip-ft OK (*Eq.* A4.1.1-1)

At the location of maximum positive moment:

The section is assumed to be fully braced against lateral-torsional buckling.

Calculate the allowable distortional buckling strength per Section C3.1.4.

Calculate the elastic distortional buckling stress, F_d.

The cross section has a single web and a single edge-stiffened flange as required by Section C3.1.4(a). Consider the contribution of the attached roof panel, which has a rotational stiffness, $k_\phi = 0.300$ kip-in./rad./in. From Table II-9 for the 8ZS2.25x0.059,

$k_{\phi fe} = 0.254$ kip

$\tilde{k}_{\phi fg} = 0.0110$ in.^{2}

$k_{\phi we} = 0.236$ kip

$\tilde{k}_{\phi wg} = 0.00168$ in.^{2}

Since there is no significant moment gradient in the vicinity of the maximum positive moment, use $\beta = 1.0$.

$$F_d = \beta\frac{k_{\phi fe} + k_{\phi we} + k_\phi}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \qquad (\textit{Eq. } \text{C3.1.4-6})$$

$$= (1.0)\frac{0.254 + 0.236 + 0.300}{0.0110 + 0.00168} = 62.3 \text{ ksi}$$

Calculate the allowable distortional buckling strength per Section C3.1.4

$$M_y = S_{fy}F_y \quad (Eq.\ C3.1.4\text{-}4)$$
$$= (1.94)(55) = 107 \text{ kip-in.}$$

$$M_{crd} = S_f F_d \quad (Eq.\ C3.1.4\text{-}5)$$
$$= (1.94)(62.3) = 121 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y / M_{crd}} \quad (Eq.\ C3.1.4\text{-}3)$$
$$= \sqrt{107/121} = 0.940 > 0.673 \text{ therefore,}$$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ C3.1.4\text{-}2)$$
$$= (1 - 0.22(1/0.940))(1/0.940)(107) = 87.2 \text{ kip-in.} = 7.27 \text{ kip-ft}$$

$$\frac{M_n}{\Omega_b} = \frac{7.27}{1.67} = 4.35 \text{ kip-ft} > 2.23 \text{ kip-ft OK} \quad (Eq.\ A4.1.1\text{-}1)$$

Calculate the allowable strength based on initiation of yielding per Section C3.1.1(a).

$$\frac{M_n}{\Omega_b} = \frac{8.25}{1.67} = 4.94 \text{ kip-ft} > 2.23 \text{ kip-ft OK} \quad (Eq.\ A4.1.1\text{-}1)$$

In the region of negative moment between the end of the right lap and the inflection point:

Determine the nominal strength using the distance from the inflection point to the end of the right lap as the unbraced length.

$$L = 4.94 - 1.00 = 3.94 \text{ ft} = 47.3 \text{ in.}$$

By inspection, this condition is less severe than the left lap, since the unbraced length is shorter and the required strength is less; therefore, the section is OK.

In the lapped region over the center support:

The section is assumed to be fully braced at the center support for negative moment.

Calculate the nominal strength based on initiation of yielding per Section C3.1.1(a), summing the strength of the two overlapped purlins:

For the combined strength of purlins,

$$\frac{M_n}{\Omega_b} = \frac{8.25 + 8.25}{1.67} = 9.88 \text{ kip-ft} > 4.95 \text{ kip-ft OK} \quad (Eq.\ A4.1.1\text{-}1)$$

b. Strength for shear only (Section C3.2)

Required strength
By inspection, the load combination D + L_r controls:

$V = V_D + V_{Lr}$

Overhang: By inspection, the overhang does not control.

End span, from left to right:

At left support:	V = 0.14 + 0.95 = 1.09 kips
At end of right lap:	V = 0.20 + 1.30 = 1.50 kips
At left side of first interior support:	V = 0.23 + 1.55 = 1.78 kips

Interior span, from left to right:

At right side of first interior support:	V = 0.21 + 1.38 = 1.59 kips

At end of left lap: $V = 0.15 + 1.03 = 1.18$ kips
At end of right lap: $V = 0.15 + 1.02 = 1.17$ kips
At center support: $V = 0.17 + 1.12 = 1.29$ kips

Allowable strength

End span:

At the left support and right lap, t = 0.070 in. By inspection the right lap controls.

For t = 0.070 in.,

$$h = D - 2t - 2R = 8.00 - 2(0.070) - 2(0.1875) = 7.485 \text{ in.}$$

$$\frac{h}{t} = \frac{7.485}{0.070} = 107$$

Since the web is unreinforced, $k_v = 5.34$.

$$1.51\sqrt{Ek_v/F_y} = 1.51\sqrt{(29500)(5.34)/55} = 80.8$$

For $h/t > 1.51\sqrt{Ek_v/F_y}$,

$$F_v = \frac{\pi^2 Ek_v}{12(1-\mu^2)(h/t)^2} \qquad (Eq.\ C3.2.1\text{-}4a)$$

$$= \frac{\pi^2(29500)(5.34)}{12(1-0.3^2)(7.485/0.070)^2} = 12.5 \text{ ksi}$$

$$V_n = A_w F_v \qquad (Eq.\ C3.2.1\text{-}1)$$

$$= (7.485)(0.070)(12.5) = 6.55 \text{ kips}$$

Alternately, V_n can be taken from Table II-4, Beam Properties, Z-Sections With Lips, F_y = 55 ksi. For a 8ZS2.25x070, V_n is 6.52 kips.

$$\frac{V_n}{\Omega_v} = \frac{6.55}{1.60} = 4.09 \text{ kips} > 1.50 \text{ kips OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

At the left side of first interior support, sum the strength of the two overlapped purlins:

For t = 0.059 in.,

$$h = D - 2t - 2R = 8.00 - 2(0.059) - 2(0.1875) = 7.507 \text{ in.}$$

$$\frac{h}{t} = \frac{7.507}{0.059} = 127 > 80.8$$

$$F_v = \frac{\pi^2(29500)(5.34)}{12(1-0.3^2)(7.507/0.059)^2} = 8.79 \text{ ksi} \qquad (Eq.\ C3.2.1\text{-}4a)$$

$$V_n = A_w F_v \qquad (Eq.\ C3.2.1\text{-}1)$$

$$= (7.507)(0.059)(8.79) = 3.89 \text{ kips}$$

Alternately, V_n can be taken from Table II-4, Beam Properties, Z-Sections With Lips, F_y = 55 ksi. For a 8ZS2.25x059, V_n is 3.90 kips.

For the combined strength of purlins,

$$\frac{V_n}{\Omega_v} = \frac{3.89 + 6.55}{1.60} = 6.53 \text{ kips} > 1.78 \text{ kips OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

Interior span:

At the right side of the first interior support, use the strength computed above.

$$\frac{V_n}{\Omega_v} = 6.53\text{kips} > 1.59\text{ kips OK} \qquad (\textit{Eq. } A4.1.1\text{-}1)$$

By inspection of the left and right laps, the left lap controls.

$$\frac{V_n}{\Omega_v} = \frac{3.89}{1.60} = 2.43\text{ kips} > 1.18\text{ kips OK} \qquad (\textit{Eq. } A4.1.1\text{-}1)$$

At the center support, sum the strength of the two overlapped purlins:

For the combined strength of purlins:

$$\frac{V_n}{\Omega_v} = \frac{3.89+3.89}{1.60} = 4.86\text{ kips} > 1.29\text{ kips OK} \qquad (\textit{Eq. } A4.1.1\text{-}1)$$

c. Strength for combined bending and shear (Section C3.3.1)

End span:

$$\sqrt{\left(\frac{\Omega_b M}{M_{nxo}}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2} \le 1.0 \qquad (\textit{Eq. } C3.3.1\text{-}1)$$

where

M_{nxo} = M_n, calculated based on the initiation of yielding per Section C3.1.1

Ω_b = 1.67

Ω_v = 1.60

To the left of the right lap, t = 0.070 in.

$$\sqrt{\left(\frac{(1.67)(4.65)}{10.3}\right)^2 + \left(\frac{(1.60)(1.50)}{6.55}\right)^2} = 0.838 < 1.0\text{ OK} \qquad (\textit{Eq. } C3.3.1\text{-}1)$$

At first interior support,

$$\sqrt{\left(\frac{(1.67)(8.75)}{10.3+8.25}\right)^2 + \left(\frac{(1.60)(1.78)}{6.55+3.89}\right)^2} = 0.834 < 1.0\text{ OK} \qquad (\textit{Eq. } C3.3.1\text{-}1)$$

Interior span:

At ends of laps, t = 0.059 in. Left lap controls by inspection.

$$\sqrt{\left(\frac{(1.67)(3.89)}{8.25}\right)^2 + \left(\frac{(1.60)(1.18)}{3.89}\right)^2} = 0.925 < 1.0\text{ OK} \qquad (\textit{Eq. } C3.3.1\text{-}1)$$

At center support,

$$\sqrt{\left(\frac{(1.67)(4.95)}{8.25+8.25}\right)^2 + \left(\frac{(1.60)(1.29)}{3.89+3.89}\right)^2} = 0.567 < 1.0\text{ OK} \qquad (\textit{Eq. } C3.3.1\text{-}1)$$

d. Web crippling strength (Section C3.4)

Required strength

By inspection, the load combination D + L_r controls:

$P = P_D + P_{Lr}$

Supports, from left to right:

At left support:	P = 0.16 + 1.05 = 1.21 kips
At first interior support:	P = 0.44 + 2.93 = 3.37 kips
At center support:	P = 0.34 + 2.24 = 2.58 kips

Allowable strength

$$P_n = Ct^2F_y\sin\theta\left(1-C_R\sqrt{\frac{R}{t}}\right)\left(1+C_N\sqrt{\frac{N}{t}}\right)\left(1-C_h\sqrt{\frac{h}{t}}\right) \quad (Eq.\ C3.4.1\text{-}1)$$

where

F_y = 55 ksi
θ = 90 degrees
R = 0.1875 in.
N = bearing length = 5.0 in.

At end supports:

h = 7.485 in.
t = 0.070 in.

From Table C3.4.1-3, using the coefficients for the case of:

Fastened to Support/One-Flange Loading or Reaction/End

C = 4
C_R = 0.14
C_N = 0.35
C_h = 0.02
Ω_w = 1.75

Check Limit: R/t = 0.1875/0.070 = 2.7 < 9 OK

$$P_n = (4)(0.070)^2(55)\sin(90)\left(1-0.14\sqrt{\frac{0.1875}{0.070}}\right)\left(1+0.35\sqrt{\frac{5.0}{0.070}}\right)\left(1-0.02\sqrt{\frac{7.485}{0.070}}\right)$$

$$= 2.61 \text{ kips} \quad (Eq.\ C3.4.1\text{-}1)$$

Alternately, P_n can be taken from Table II-15, Web Crippling, Z-Sections With Lips. Using values for Fastened, Case A, for a 8ZS2.25x070 with a yield stress of 55 ksi, P_n can be interpolated as:

$$P_n = 0.5(2.40+2.79) = 2.60 \text{ kips.}$$

Calculate strength increase from overhang

$$P_{nc} = \alpha P_n \quad (Eq.\ C3.4.1\text{-}2)$$

where

L_o = 12.0 in.

$$\alpha = \frac{1.34(L_o/h)^{0.26}}{0.009(h/t)+0.3} \geq 1.0 \quad (Eq.\ C3.4.1\text{-}3)$$

$$= \frac{1.34(12.0/7.485)^{0.26}}{0.009(7.485/0.070)+0.3} = 1.20$$

Using the value of P_n calculated from *Eq.* C3.4.1-1 above,

$$P_{nc} = (1.20)(2.61) = 3.13 \text{ kips} \quad (Eq.\ C3.4.1\text{-}2)$$

Check upper limit of interior one-flange loading. From Table C3.4.1-3, using the coefficients for the case of:

Fastened to Support/One-Flange Loading or Reaction/Interior

C = 13
C_R = 0.23
C_N = 0.14
C_h = 0.01

$\Omega_w = 1.65$

for t = 0.070 in.,

Check Limit: $R/t = 0.1875/0.070 = 2.7 < 5.5$ OK

$$P_n = (13)(0.070)^2(55)\sin(90)\left(1-0.23\sqrt{\frac{0.1875}{0.070}}\right)\left(1+0.14\sqrt{\frac{5.0}{0.070}}\right)\left(1-0.01\sqrt{\frac{7.485}{0.070}}\right)$$

$= 4.28$ kips (*Eq.* C3.4.1-1)

Alternately, P_n can be taken from Table II-15, Web Crippling, Z-Sections With Lips. Using values for Fastened, Case B, for a 8ZS2.25x070 with a yield stress of 55 ksi, P_n can be interpolated as:

$$P_n = 0.5(4.03+4.50) = 4.27 \text{ kips} > 3.13 \text{ kips OK}$$

$$\frac{P_{nc}}{\Omega_w} = \frac{3.13}{1.75} = 1.79 \text{ kips} > 1.21 \text{ kips OK}$$ (*Eq.* A4.1.1-1)

At interior supports:

From Table C3.4.1-3, using the coefficients for the case of:

Fastened to Support/One-Flange Loading or Reaction/Interior

$C = 13$
$C_R = 0.23$
$C_N = 0.14$
$C_h = 0.01$
$\Omega_w = 1.65$

for t = 0.070 in.,

$P_n = 4.28$ kips, calculated above

for t = 0.059 in.,

Check Limit: $R/t = 0.1875/0.059 = 3.2 < 5.5$ OK

$$P_n = (13)(0.059)^2(55)\sin(90)\left(1-0.23\sqrt{\frac{0.1875}{0.059}}\right)\left(1+0.14\sqrt{\frac{5.0}{0.059}}\right)\left(1-0.01\sqrt{\frac{7.507}{0.059}}\right)$$

$= 2.98$ kips (*Eq.* C3.4.1-1)

Alternately, P_n can be taken from Table II-15, Web Crippling, Z-Sections With Lips. Using values for Fastened, Case B, for a 8ZS2.25x059 with a yield stress of 55 ksi P_n can be interpolated as:

$$P_n = 0.5(2.80+3.14) = 2.97 \text{ kips.}$$

Using the values of P_n calculated from *Eq.* C3.4.1-1 above,

At first interior support,

$$\frac{P_n}{\Omega_w} = \frac{4.28+2.98}{1.65} = 4.40 \text{ kips} > 3.37 \text{ kips OK}$$ (*Eq.* A4.1.1-1)

At center support,

$$\frac{P_n}{\Omega_w} = \frac{2.98+2.98}{1.65} = 3.61 \text{ kips} > 2.58 \text{ kips OK}$$ (*Eq.* A4.1.1-1)

e. Combined bending and web crippling (Section C3.5.1(c))

$$0.86\left(\frac{P}{P_n}\right)+\left(\frac{M}{M_{nxo}}\right)\le\frac{1.65}{\Omega} \quad (Eq.\ C3.5.1\text{-}3)$$

where

M_{nxo} = the sum of M_n of each purlin at the support calculated based on the initiation of yielding per Section C3.1.1

P_n = the sum of P_n of each purlin at the support

Ω = 1.70

Check the limits for the controlling (thinner) section:

h/t = 7.507/0.059 = 127 < 150 OK

N/t = 5.0/0.059 = 85 < 140 OK

F_y = 55 ksi < 70 ksi OK

R/t = 0.1875/0.059 = 3.2 < 5.5 OK

t_{max}/t_{min} = 0.070/0.059 = 1.19 < 1.3 OK

All other limits are assumed to be satisfied as well.

At the first interior support,

$$0.86\left(\frac{3.37}{4.28+2.98}\right)+\left(\frac{8.75}{10.3+8.25}\right)=0.871<\frac{1.65}{1.70}=0.971\text{ OK} \quad (Eq.\ C3.5.1\text{-}3)$$

At the center support,

$$0.86\left(\frac{2.58}{2.98+2.98}\right)+\left(\frac{4.95}{8.25+8.25}\right)=0.672<0.971\quad\text{OK} \quad (Eq.\ C3.5.1\text{-}3)$$

4. Check Uplift Loads

a. Strength for bending only (Section D6.1.1)

Required strength

By inspection, the load combination 0.6D + 0.6W controls.

$M = 0.6M_D + 0.6M_w$

End span:

Maximum negative moment: $M=0.6(0.67)-0.6(7.55)=-4.13$ kip-ft

Interior span:

Maximum negative moment: $M=0.6(0.29)-0.6(3.31)=-1.81$ kip-ft

Allowable strength

$$M_n = RS_eF_y \quad (Eq.\ D6.1.1\text{-}1)$$

R $=0.70$, assuming all 15 conditions of Section D6.1.1 are satisfied

End span:

For t = 0.070 in.,

$$M_n = (0.70)(2.25)(55) = 86.6\text{ kip-in.} = 7.22\text{ kip-ft} \quad (Eq.\ D6.1.1\text{-}1)$$

$$\frac{M_n}{\Omega_b}=\frac{7.22}{1.67}=4.32\text{ kip-ft} > 4.13\text{ kip-ft OK} \quad (Eq.\ A4.1.1\text{-}1)$$

Interior span:

For t = 0.059 in.,

$$M_n = (0.70)(1.80)(55) = 69.3 \text{ kip-in.} = 5.78 \text{ kip-ft} \qquad (\textit{Eq. } \text{D6.1.1-1})$$

$$\frac{M_n}{\Omega_b} = \frac{5.78}{1.67} = 3.46 \text{ kip-ft} > 1.81 \text{ kip-ft OK} \qquad (\textit{Eq. } \text{A4.1.1-1})$$

b. Other comments

All other regions of the system have their compression flange braced by the roof panel. Since the magnitude of the shears, moments and reactions are less than those of the gravity case, it can be concluded by inspection that the design satisfies the *Specification* criteria for uplift.

5. Anchorage Forces Under Gravity Loads

A check of anchorage forces is required to complete the design. Refer to the *Design Guide for Cold-Formed Steel Purlin Roof Framing Systems** for further information and examples of this check.

* AISI publication D111-09: *Design Guide for Cold-Formed Steel Purlin Roof Framing Systems.*

Example II-3: Four Span Continuous Standing-Seam Roof System* - ASD

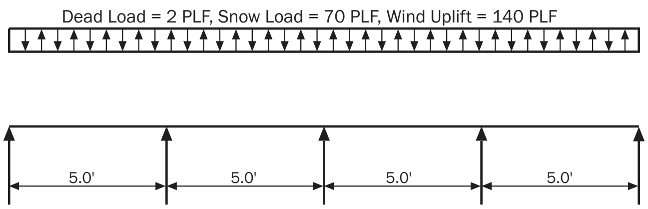

Figure 1 - Spans and Loading

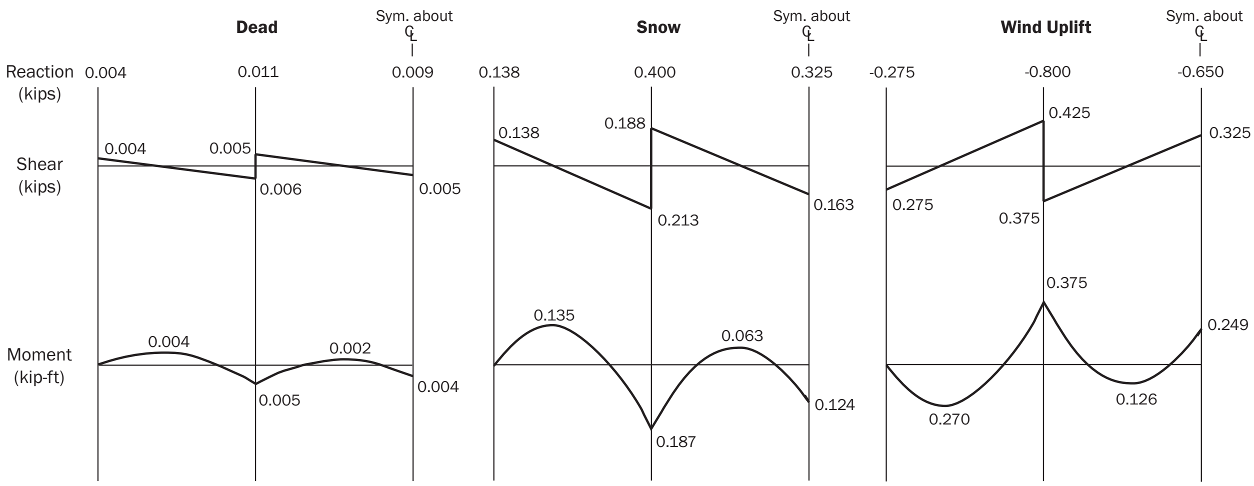

Figure 2 - Reactions, Shears and Moments

Note: Moments and forces are from unfactored nominal loads

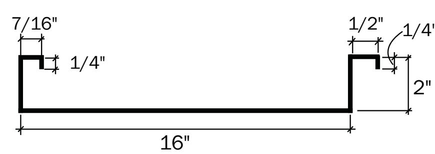

Figure 3 - Panel Cross Section

Given:

1. Four span standing seam roof panel.
2. Panel: F_y = 50 ksi; t = 0.024 in.; R = 0.048 in.
3. Purlins: F_y = 55 ksi; t_{min} = 0.060 in.; t_{max} = 0.135 in.
4. Use ASD approach.

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* For design of purlins supporting standing seam roof, see AISI publication D111-09: *Design Guide for Cold-Formed Steel Purlin Roof Framing Systems.*

Required:
1. Allowable strength of panel under gravity loading.
2. Allowable strength of panel under uplift loading.
3. Allowable flexural strength of purlins under gravity loading.

Solution:

From the design procedures of the *Specification*, the following panel section properties can be obtained

I_x = 0.191 in.^{4}
S_f = 0.113 in.^{3} (top), 0.612 in.^{3} (bottom)

Effective section modulus at yield stress level:

S_e = 0.101 in.^{3} (top), 0.618 in.^{3} (bottom) - Positive Bending
= 0.086 in.^{3} (top), 0.085 in.^{3} (bottom) - Negative Bending

Note: When computing effective section properties, the *Specification* permits w/t ratios larger than 500 per Section B1.1. This is the situation for the computations of S_e (bottom panel in compression).
Positive moment is defined as a moment producing compression stresses on the top of the panel.

1. Allowable Strength of Panel Under Gravity Loads

a. Strength for bending only (Section C3.1.1)

Required strength:

M = $M_D + M_S$
Maximum positive moment: M = 0.004 + 0.135 = 0.139 kip-ft
Maximum negative moment: M = 0.005 + 0.187 = 0.192 kip-ft

Allowable strength:

Positive Moment:

$$M_n = S_eF_y = \frac{0.101(50)}{12} = 0.421 \text{ kip-ft} \qquad (\textit{Eq. } \text{C3.1.1-1})$$

$$\frac{M_n}{\Omega} = \frac{0.421}{1.67} = 0.252 \text{ kip-ft} > 0.139 \text{ kip-ft OK} \qquad (\textit{Eq. } \text{A4.1.1-1})$$

Negative Moment:

$$M_n = S_eF_y = \frac{0.085(50)}{12} = 0.354 \text{ kip-ft} \qquad (\textit{Eq. } \text{C3.1.1-1})$$

M_n/Ω = 0.354/1.67 = 0.212 kip-ft > 0.192 kip-ft OK (*Eq.* A4.1.1-1)

b. Strength for shear only (Section C3.2)

Required strength:

$V = V_D + V_S = 0.006 + 0.212 = 0.218$ kip

Allowable strength:

For t = 0.024 in., h = 2-2(0.024+0.048)=1.856 in., h/t = 77.3 < $1.51\sqrt{Ek_v/F_y}$ = 84.7

$$F_v = \frac{0.60\sqrt{Ek_vF_y}}{h/t} \qquad (Eq.\ C3.2.1\text{-}3)$$

$$= \frac{0.60\sqrt{29500(5.34)(50)}}{1.856/0.024} = 21.75 \text{ ksi}$$

$$V_n = A_wF_v = (1.856)(0.024)(21.75) = 0.971 \text{ kip} \qquad (Eq.\ C3.2.1\text{-}1)$$

$$\frac{V_n}{\Omega_v} = \frac{0.971}{1.60} = 0.607 \text{ kip/web} \qquad (Eq.\ A4.1.1\text{-}1)$$

$$= (2)0.607 = 1.21 \text{ kips} > 0.218 \text{ kips} \quad \text{OK}$$

c. Strength for combined bending and shear (Section C3.3)

Required strength:

For the first interior support

$M = M_D + M_S = 0.005 + 0.187 = 0.192$ kip-ft

$V = V_D + V_S = 0.006 + 0.212 = 0.218$ kip

$$\sqrt{\left(\frac{\Omega_b M}{M_{nxo}}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2} \le 1.0 \qquad (Eq.\ C3.3.1\text{-}1)$$

$$\sqrt{\left(\frac{0.192}{0.212}\right)^2 + \left(\frac{0.218}{1.21}\right)^2} = 0.923 < 1.0 \qquad \text{OK}$$

d. Web crippling strength (Section C3.4), not required if standoff clips are used.

Required strength:

$P = P_D + P_S$

End support: P = 0.004 + 0.138 = 0.142 kip/ft

First interior support: P = 0.011 + 0.400 = 0.411 kip/ft

Allowable strength:

The following assumes a bearing length of 2½ in.

At end supports use Eq. C3.4.1-1

$$P_n = Ct^2F_y \sin\theta\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \qquad (Eq.\ C3.4.1\text{-}1)$$

where:

F_y = 50 ksi

θ = 90 degrees

R = 0.048 in.

N = bearing length = 2.5 in.

h = 1.856 in.

t = 0.024 in.

From Table C3.4.1-5, using the coefficients for the case of:
Unfastened to Support/One-Flange Loading/End:

C = 3
C_R = 0.04
C_N = 0.29
C_h = 0.028
Ω_w = 2.45

Check Limits: h/t ≤ 200, N/t ≤ 210, N/h ≤ 3.0, R/t ≤ 20

h/t = 1.856/0.024 = 77.3, N/t = 2.5/0.024 = 104, N/h = 2.5/1.856 = 1.35
R/t = 0.048/0.024 = 2. All OK

$$P_n = 3(0.024^2)(50)\sin(90)\left(1-0.04\sqrt{\frac{0.048}{0.024}}\right)\left(1+0.29\sqrt{\frac{2.5}{0.024}}\right)\left(1-0.028\sqrt{\frac{1.856}{0.024}}\right)$$
$$= 0.243 \text{ kip per web}$$

$$\frac{P_n}{\Omega_w} = \frac{0.243(2)}{2.45}\left(\frac{12}{16}\right) = 0.149 \text{ kip/ft} > 0.142 \text{ kip} \quad \text{OK} \qquad (\textit{Eq. } A4.1.1\text{-}1)$$

At interior supports use Eq. C3.4.1-1 of the *Specification*

$$P_n = Ct^2F_y\sin\theta\left(1-C_R\sqrt{\frac{R}{t}}\right)\left(1+C_N\sqrt{\frac{N}{t}}\right)\left(1-C_h\sqrt{\frac{h}{t}}\right) \qquad (\textit{Eq. } C3.4.1\text{-}1)$$

where:

From Table C3.4.1-5, using the coefficients for the case of:
Unfastened to Support/One-Flange Loading/Interior:

C = 8
C_R = 0.10
C_N = 0.17
C_h = 0.004
Ω_w = 1.75

$$P_n = 8(0.024^2)(50)\sin(90)\left(1-0.10\sqrt{\frac{0.048}{0.024}}\right)\left(1+0.17\sqrt{\frac{2.5}{0.024}}\right)\left(1-0.004\sqrt{\frac{1.856}{0.024}}\right)$$
$$= 0.522 \text{ kip per web}$$

$$\frac{P_n}{\Omega_w} = \frac{0.522(2)}{1.75}\left(\frac{12}{16}\right) = 0.447 \text{ kip/ft} > 0.411 \text{ kip/ft} \quad \text{OK} \qquad (\textit{Eq. } A4.1.1\text{-}1)$$

e. Combined bending and web crippling (Section C3.5)

The following equation must be used to check combined bending and web crippling for the panel sections because the web spacing exceeds 10 in.

$$0.91\left(\frac{P}{P_n}\right)+\left(\frac{M}{M_{nxo}}\right)\le\frac{1.33}{\Omega} \qquad (Eq.\ C3.5.1\text{-}1)$$

At the interior supports

$$0.91\left(\frac{0.411}{2(0.522)}\left(\frac{16}{12}\right)\right)+\left(\frac{0.192}{0.354}\right)\le\frac{1.33}{1.70} \qquad (Eq.\ C3.5.1\text{-}1)$$

1.02 > 0.782 NG Thicker panel or closer support spacing is required.

2. Allowable Strength of Panel Under Uplift Loading (Section D6.1.2)

The *Specification* requires that the uplift strength of standing seam roofs be determined by testing in accordance with AISI S906-13, *Standard Procedures for Panel and Anchor Structural Tests*. The results of these tests are not shown here.

3. Allowable Flexural Strength of Purlins Under Gravity Loading (Section D6.1.2)

The available strength of Z-purlin with the top flange supporting a standing seam roof panel system can be determined using the provisions of Section C3.1.2.1 or determined by the following equation:

$$M_n = RS_eF_y \qquad (Eq.\ D6.1.2\text{-}1)$$

Where R is determined based on AISI S908-13, *Base Test Method for Purlins Supporting a Standing Seam Roof System*. Base tests were conducted for the purlin cross-section shown in Figure 4 below. Apply the evaluation procedure as prescribed in AISI S908 and determine the appropriate reduction factors.

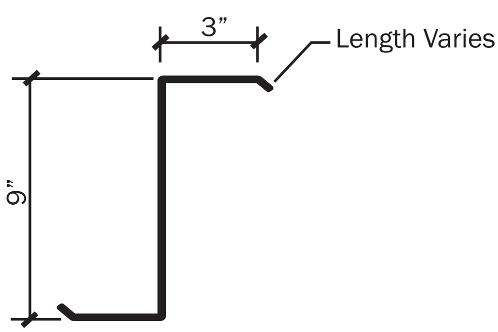


Figure 4 - Tested Purlin

In accordance with Section 8.2 of AISI S908, three tests were conducted for the thickest profile and three tests were conducted for the thinnest profile with purlins facing in the same direction. The span length, L, for all tests was 25 ft. The following summarizes the data:

Thickest profile, t = 0.135 in. (measured)

Test No.	Failure Load, p_{ts} (lb/ft^2)	Specimen wt., p_d (lb/ft^2)
1	55.6	2.0
2	56.8	2.0
3	56.4	2.0

Thinnest profile, t = 0.06 in. (measured)

Test No.	Failure Load, p_{ts} (lb/ft^2)	Specimen wt., p_d (lb/ft^2)
1	18.4	1.0
2	18.0	1.0
3	17.6	1.0

The maximum anticipated purlin spacing, B, and the tributary width of the purlins tested, s, are both 5.0 ft.

The tests were performed under the following conditions:
1. The tests were for Z-sections under gravity loading,
2. All purlins were facing the same directions,
3. No discrete bracing was included, and
4. The first purlin to fail in each test was the downhill purlin.

Per Section 9.1 of AISI 908, the computed test failure load is determined by Equation 1.

$$w_{ts} = \left(p_{ts} + p_d\right)s + 2P_L\left(\frac{d}{B}\right) \quad \text{(AISI S908 } \textit{Eq.}\text{ (1))}$$

$$= \left(p_{ts} + p_d\right)5.00 + 2P_L\left(\frac{9.00}{60.0}\right)$$

Determine the lateral anchorage force, P_L.

In accordance with Section 9.1 of AISI S908, the anchorage force is computed as:

$$P_L = 0.5\left[\frac{C_2}{1000}\frac{I_{xy}L}{I_x d} + C_3\frac{0.25bt}{d^2}\right](p_{ts} + p_d)s \quad \text{(AISI S908 } \textit{Eq.}\text{ (2))}$$

For t = 0.135 in.,

C_2 = 8.3

C_3 = 28

I_{xy} = 8.49 in.^{4}

I_x = 27.0 in.^{4}

$$P_L = 0.5\left[\frac{8.3}{1000}\frac{(8.49)(25)(12)}{(27.0)(9.00)} + 28\frac{0.25(3.00)(0.135)}{(9.00)^2}\right](p_{ts} + p_d)5.00$$

$$= 0.305(p_{ts} + p_d)$$

For t = 0.060 in.,

I_{xy} = 3.78 in.^{4}

I_x = 12.3 in.^{4}

$$P_L = 0.5\left[\frac{8.3}{1000}\frac{(3.78)(300)}{(12.3)(9.00)} + 28\frac{0.25(3.00)(0.060)}{(9.00)^2}\right](p_{ts} + p_d)5.00$$

$$= 0.251\left(p_{ts} + p_d\right)$$

Compute the failure load, w_{ts}

For t = 0.135 in.,

Test No.	$p_{ts} + p_d$ (lbs/ft^2)	P_L (lbs/ft)	w_{ts} (lbs/ft)
1	57.6	17.6	293
2	58.8	17.9	299
3	58.4	17.8	297

For t = 0.060 in.,

Test No.	$p_{ts} + p_d$ (lbs/ft^2)	P_L (lbs/ft)	w_{ts} (lbs/ft)
1	19.4	4.87	98.5
2	19.0	4.77	96.4
3	18.6	4.67	94.4

Compute the modification factor, R_t, where the subscript, t, denotes a tested value.

The purlin strength modification factor, R_t, is determined by Sections 9.2 through 9.5 of AISI S908.

M_{ts} = $w_{ts}L^2/8$, single span failure moment (AISI S908 *Eq.* (5))

L = 25 ft., the test purlin span length

M_{nt} = $S_{et}F_{yt}$, computed nominal strength of the fully braced section (AISI S908 *Eq.* (6))

When evaluating M_{nt}, the measured yield stress, F_{yt}, and the measured geometry for each Test No. must be used. Because the intent of this example is to demonstrate the application of AISI S908, F_{yt} and S_{et} were taken as constants for each of the three tests.

The computed flexural strength is determined using Section C3.1.1(a) of the *Specification*. S_{et} is the computed effective section modulus using the measured section properties and the measured yield strength, F_{yt}. The measured yield strength is determined using the procedures of ASTM A370.

For t = 0.135 in.

Test No.	M_{ts} (kip-in.)	F_{yt} (ksi)	S_{et} (in.^{3})	M_{nt} (kip-in.)	$R_t = M_{ts}/M_{nt}$
1	275	56.10	5.97	335	0.821
2	280	56.10	5.97	335	0.836
3	278	56.10	5.97	335	0.830
		Mean		335	0.829
		Standard Deviation			0.008

For t = 0.060 in.

Test No.	M_{ts} (kip-in)	F_{yt} (ksi)	S_{et} (in.^{3})	M_{nt} (kip-in)	$R_t = M_{ts}/M_{nt}$
1	92.3	57.26	2.01	115	0.803
2	90.4	57.26	2.01	115	0.786
3	88.5	57.26	2.01	115	0.770
		Mean		115	0.786
		Standard Deviation			0.017

Compute the reduction factor, R

The purlin strength reduction factor, R, is computed by using Section 9.6 of AISI S908.

$$R = \left[\frac{R_{tmax} - R_{tmin}}{\bar{M}_{ntmax} - \bar{M}_{ntmin}}\right]\left(M_n - \bar{M}_{ntmin}\right) + R_{tmin} \leq 1.0 \qquad \text{(AISI S908 } Eq. \text{ (8))}$$

where the symbols are defined in Section 9.6 of AISI S908.

R_{tmin} = 0.786 - 0.017 = 0.769
R_{tmax} = 0.829 - 0.008 = 0.821
$\bar{M}_{ntmin}$ = 115 kip-in.
$\bar{M}_{ntmax}$ = 335 kip-in.

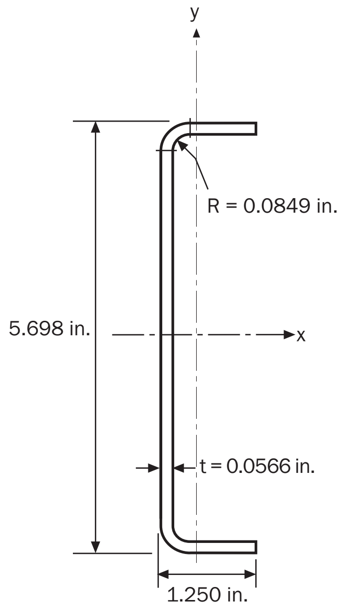
Using the above numerical values, the reduction factor for any purlin thickness can be determined from Equation (8) of S908. For example, in the table below R values for 0.135, 0.090, and 0.060 thicknesses are shown. The nominal moments are based on F_y = 55 ksi

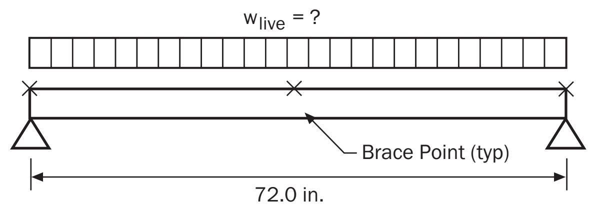
Purlin Thickness (in.)	Fully Braced M_n (kip-in.)	R	RM_n (kip-in.)
0.135	330	0.820	271
0.090	195	0.788	154
0.060	113	0.769	86.9

The nominal flexural strength for each purlin is the product of RM_n.

Note that the R values for this example decrease with decreasing thicknesses. The reader should not assume that this is typical for all standing seam systems.

Example II-4: C-Section Without Lips Braced at Mid-Span





Given:

1. Steel: F_y = 33 ksi
2. Section: Track 550T125-54 as shown in sketch above.
3. Simple span of 72.0 in.
4. Member is braced against twisting and lateral deflection at mid-span and ends. Ends are reinforced against crippling
5. Member is loaded through the shear center and parallel to the web.
6. Gross section properties from Table I-3:

   t = 0.0566 in.

   R = 0.0849 in.

   A = 0.452 in.^{2}

   S_x = 0.668 in.^{3}

   r_y = 0.342 in.

   r_o = 2.15 in.

   J = 0.000483 in.^{4}

   C_w = 0.315 in.^{6}

Required:

Calculate the largest permitted uniformly distributed service load, w_{live}, assuming a negligible dead load, using ASD and LRFD. Compute w_{live} based on flexural strength and check shear.

Solution:

The beam is subject to lateral-torsional buckling, but not subject to distortional buckling, because all of the buckling modes involving change in the cross-sectional shape are local buckling modes (flange local buckling and web local buckling).

1. Lateral-Torsional Buckling Strength (Section C3.1.2.1)

For singly symmetric sections bent about the axis of symmetry,

$$F_e = \frac{C_b r_o A}{S_f}\sqrt{\sigma_{ey}\sigma_t} \qquad (Eq.\ C3.1.2.1\text{-}4)$$

$$\sigma_{ey} = \frac{\pi^2 E}{\left(K_y L_y / r_y\right)^2} \qquad (Eq.\ C3.1.2.1\text{-}8)$$

$$= \frac{\pi^2 (29500)}{\left((1.0)(36.0)/0.342\right)^2}$$

$$= 26.28 \text{ ksi}$$

$$\sigma_t = \frac{1}{A r_o^2}\left[GJ + \frac{\pi^2 E C_w}{(K_t L_t)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$\sigma_t = \frac{1}{(0.452)(2.15)^2}\left[(11300)(0.000483) + \frac{\pi^2 (29500)(0.315)}{\left[(1.0)(36.0)\right]^2}\right]$$

$$= 36.48 \text{ ksi}$$

Calculate C_b assuming a unit uniform loading

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C} \qquad (Eq.\ C3.1.2.1\text{-}6)$$

$$M_{max} = \frac{wL^2}{8} = \frac{(1.0)(72.0)^2}{8} = 648.0 \text{ kip-in. at brace point}$$

$$M_A = \frac{7wL^2}{128} = \frac{(7)(1.0)(72.0)^2}{128} = 283.5 \text{ kip-in. at 1/4 point of unbraced segment}$$

$$M_B = \frac{3wL^2}{32} = \frac{(3)(1.0)(72.0)^2}{32} = 486.0 \text{ kip-in. at center point of unbraced segment}$$

$$M_C = \frac{15wL^2}{128} = \frac{(15)(1.0)(72.0)^2}{128} = 607.5 \text{ kip-in. at 3/4 point of unbraced segment}$$

$$C_b = \frac{(12.5)(648.0)}{(2.5)(648.0) + (3)(283.5) + (4)(486.0) + (3)(607.5)} \qquad (Eq.\ C3.1.2.1\text{-}6)$$

$$= 1.30$$

$$F_e = \frac{(1.30)(2.15)(0.452)}{0.668}\sqrt{(26.28)(36.48)} = 58.6 \text{ ksi} \qquad (Eq.\ C3.1.2.1\text{-}4)$$

$0.56F_y = (0.56)(33.0) = 18.5$ ksi

$2.78F_y = (2.78)(33.0) = 91.7$ ksi

For $2.78F_y > F_e > 0.56F_y$:

$$F_c = \frac{10}{9}F_y\left(1 - \frac{10F_y}{36F_e}\right) \qquad (Eq.\ C3.1.2.1\text{-}2)$$

$$= \frac{10}{9}(33.0)\left(1 - \frac{(10)(33.0)}{(36)(58.6)}\right) = 30.93 \text{ ksi}$$

From Example I-9 with f = 30.93 ksi,

S_c = 0.606 in.^{3}

$M_n = S_cF_c$ (*Eq.* C3.1.2.1-1)

$= (0.606)(30.93)$

= 18.7 kip-in.

2. Permitted Uniform Live Load, w_{live}

ASD

Allowable Strength (ASD)

$$M \le \frac{M_n}{\Omega_b} = \frac{18.7}{1.67} = 11.2 \text{ kip-in.}$$ (*Eq.* A4.1.1-1)

$$w_{live} \le \frac{8M}{L^2} = \frac{(8)(11.2)}{(72.0)^2} = 0.0173 \text{ kips/in.} = 207 \text{ plf}$$

LRFD

Design Strength (LRFD)

$$M_u \le \phi_b M_n = (0.90)(18.7) = 16.8 \text{ kip-in.}$$ (*Eq.* A5.1.1-1)

Live load factor = 1.6

$$1.6w_{live} = \frac{(8)(16.8)}{(72.0)^2}$$

w_{live} = 0.0162 kips/in. = 194 plf

3. Check Shear (Section C3.2.1)

$$h = D - 2(t + R) = 5.698 - 2(0.0566 + 0.0849) = 5.415 \text{ in.}$$

$$h/t = 5.415/0.0566 = 95.7$$

$$\sqrt{Ek_v/F_y} = \sqrt{(29500)(5.34)/33.0} = 69.1$$

$$1.51\sqrt{Ek_v/F_y} = (1.51)(69.1) = 104.3$$

For $\sqrt{Ek_v/F_y} < h/t \le 1.51\sqrt{Ek_v/F_y}$

$$F_v = \frac{0.6\sqrt{Ek_vF_y}}{(h/t)}$$ (*Eq.* C3.2.1-3)

$$= \frac{0.6\sqrt{(29500)(5.34)(33)}}{(95.7)}$$

= 14.3 ksi

$V_n = A_wF_v$ (*Eq.* C3.2.1-1)

$= (5.415)(0.0566)(14.3) = 4.38$ kips

ASD

Allowable strength

$$V = \frac{w_{live}L}{2} \le \frac{V_n}{\Omega_v}$$ (*Eq.* A4.1.1-1)

Ω_v = 1.60

$$V = \frac{(0.0173)(72.0)}{2} = 0.623 \text{ kip} < \frac{4.38}{1.60} = 2.74 \text{ kips OK}$$

LRFD

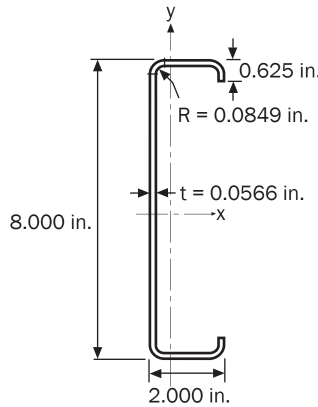
Design strength

$$V_u = \frac{1.6 w_{live} L}{2} < \phi_v V_n \qquad (Eq.\ A5.1.1\text{-}1)$$

$$\phi_v = 0.95$$

$$V_u = \frac{(1.6)(0.0162)(72.0)}{2} = 0.933 \text{ kip} < (0.95)(4.38) = 4.16 \text{ kips OK}$$

Example II-5: Distortional Buckling of C-Section



1. Steel: F_y = 50 ksi, F_u = 65 ksi
2. Section: 800S200-54 as shown above, spaced at 24 in. on center
3. The section is simply supported, uniformly loaded in flexure and fully braced against lateral-torsional buckling.
4. The section is sheeted on the compression flange with 7/16 in. OSB attached with #8 fasteners 12 in. on center. The rotational distortional buckling restraint provided by the sheathing is 0.0957 kip-in./rad./in.

Required:

Calculate the ASD flexural strength of the section, including consideration of distortional buckling.

Solution:

Gross section properties from Table I-2:

h_o = D = 8.000 in.

b_o = B = 2.000 in.

t = 0.0566 in.

d_o = d = 0.625 in.

R = 0.0849 in.

S_f = S_x = 1.64 in.^{3}

x_o = –1.27 in.

θ = 90°

Effective section properties at initiation of yielding from Table II-2:

S_e = 1.50 in.^{3}

The allowable strength is the lowest calculated in accordance with sections C3.1.1 (nominal section strength), C3.1.2 (lateral-torsional buckling strength) and C3.1.4 (distortional buckling strength).

1. Nominal Section Strength – Section C3.1.1

Compute the strength at initiation of yielding, including the effects of local buckling, per Section C3.1.1(a).

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

$$= (1.50)(50) = 75.0 \text{ kip-in. or } 6.25 \text{ kip-ft}$$

Calculate the allowable strength.

$$\frac{M_n}{\Omega_b} = \frac{6.25}{1.67} = 3.74 \text{ kip-ft} \qquad (Eq.\ A4.1.1\text{-}1)$$

2. Lateral-Torsional Buckling Strength – Section C3.1.2

Lateral-torsional buckling resistance is provided by the OSB. Requirements for the assumption of full lateral-torsional bracing of floor and roof systems are provided in Sections B1, B1.2.1 and B4 of the Floor and Roof System Design Standard (AISI-S210)*. Assume the OSB meets these requirements and provides full lateral-torsional bracing; therefore, the section is not subject to lateral-torsional buckling.

3. Distortional Buckling Strength – Section C3.1.4

Distortional buckling strength is determined using an elastic distortional buckling stress, F_d, which may be calculated by any of the three methods provided in *Commentary* Section C3.1.4 and *Specification* Section C3.1.4. Each method will be considered separately below.

Calculate distortional buckling strength using the simplified approach in Commentary Section C3.1.4

Commentary Section C3.1.4 provides a simplified and often very conservative estimate of the elastic distortional buckling stress, F_d. For sections meeting the geometric limitations, it can be used to produce a quick calculation to dismiss distortional buckling as a controlling limit state in some situations, but more accurate and liberal results can usually be obtained using *Specification* Sections C3.1.4(a) and C3.1.4(b). This method does not account for the favorable restraining influence of the sheathing. The section 800S200-54 meets the geometric limitations of *Commentary* Section C3.1.4.

$$F_d = \beta k_d \frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{b_o}\right)^2 \qquad (Eq.\ C\text{-}C3.1.4\text{-}1)$$

where

$$\beta = 1.0 \le 1 + 0.4\left(L/L_m\right)^{0.7}\left(1 + M_1/M_2\right)^{0.7} \le 1.3 \qquad (Eq.\ C\text{-}C3.1.4\text{-}2)$$

β accounts for the favorable effect of a moment gradient. For the simply supported, uniformly loaded beam, the moments M_1 and M_2 on either side of the point of maximum moment are equal; therefore, β = 1.0 and there is no benefit.

$$k_d = 0.5 \le 0.6\left(\frac{b_o D \sin\theta}{h_o t}\right)^{0.7} \le 8.0 \qquad (Eq.\ C\text{-}C3.1.4\text{-}4)$$

$$= 0.5 \le 0.6\left(\frac{2.000(0.625)(1.0)}{8.000(0.0566)}\right)^{0.7} \le 8.0$$

$$= 0.5 \le 1.22 \le 8.0 \quad \text{Use } 1.22$$

$$F_d = 1.0(1.22)\frac{\pi^2\, 29500}{12\left(1-0.3^2\right)}\left(\frac{0.0566}{2.000}\right)^2 = 26.1 \text{ ksi} \qquad (Eq.\ C\text{-}C3.1.4\text{-}1)$$

Calculate the nominal distortional buckling moment per Section C3.1.4

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* AISI S210, *North American Cold-Formed Steel Framing Standard, Floor and Roof System Design*, American Iron and Steel Institute, Washington, D.C.

$$M_y = S_{fy}F_y \qquad (Eq.\ C3.1.4\text{-}4)$$

$$= (1.64)(50) = 82.0 \text{ kip-in.}$$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (1.64)(26.1) = 42.8 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y / M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{82.0/42.8} = 1.38 > 0.673 \text{ therefore,}$$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$$= \left(1 - 0.22\left(\frac{42.8}{82.0}\right)^{0.5}\right)\left(\frac{42.8}{82.0}\right)^{0.5}(82.0) = 49.8 \text{ kip-in. or } 4.15 \text{ kip-ft}$$

Calculate the allowable strength.

$$\frac{M_n}{\Omega_b} = \frac{4.15}{1.67} = 2.49 \text{ kip-ft} \qquad (Eq.\ A4.1.1\text{-}1)$$

Calculate distortional buckling strength using Section C3.1.4(a)

Section C3.1.4(a) provides a complex but more precise calculation method for the elastic distortional buckling stress, F_d. The formulas are based on analytical model of flexural-torsional buckling of the flange as a column restrained at the web/flange juncture by the available rotational stiffness from bending/buckling of the web plate. The *Specification* commentary provides formulas for the required cross-section properties in Table C-C3.1.4(a)-1. These formulas can also be found in *Manual* Part I, Section 3.4. Calculated properties for the tabulated standard cross-sections are given in Tables II-7 through II-9.

Calculate flange properties for use in C3.1.4(a):

$$h = h_o - t$$

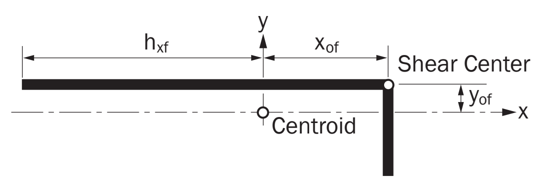
$$= 8.000 - 0.0566 = 7.943 \text{ in.}$$

$$b = b_o - t$$

$$= 2.000 - 0.0566 = 1.943 \text{ in.}$$

$$d = d_o - 0.5t$$

$$= 0.625 - 0.5(0.0566) = 0.597 \text{ in.}$$



$$A_f = (b+d)t$$

$$= (1.943 + 0.597)0.0566 = 0.144 \text{ in.}^2$$

$$I_{xf} = \frac{t\left(t^2b^2 + 4bd^3 + t^2bd + d^4\right)}{12(b+d)}$$

$$= \frac{0.0566\left(\begin{matrix}(0.0566)^2(1.943)^2 + 4(1.943)(0.597)^3 \\ +(0.0566)^2(1.943)(0.597) + (0.597)^4\end{matrix}\right)}{12(1.943+0.597)} = 0.00334 \text{ in.}^4$$

$$I_{yf} = \frac{t\left(b^4 + 4db^3\right)}{12(b+d)}$$

$$= \frac{0.0566\left((1.943)^4 + 4(0.597)(1.943)^3\right)}{12(1.943 + 0.597)} = 0.0590 \text{ in.}^4$$

$$I_{xyf} = \frac{tb^2d^2}{4(b+d)}$$

$$= \frac{0.0566(1.943)^2(0.597)^2}{4(1.943+0.597)} = 0.00750 \text{ in.}^4$$

$$x_{of} = \frac{b^2}{2(b+d)}$$

$$= \frac{(1.943)^2}{2(1.943+0.597)} = 0.743 \text{ in.}$$

$$y_{of} = \frac{-d^2}{2(b+d)}$$

$$= \frac{-(0.597)^2}{2(1.943+0.597)} = -0.0702 \text{ in.}$$

$$h_{xf} = \frac{-(b^2 + 2db)}{2(b+d)}$$

$$= \frac{-\left((1.943)^2 + 2(0.597)(1.943)\right)}{2(1.943+0.597)} = -1.20 \text{ in.}$$

$$J_f = \frac{bt^3 + dt^3}{3} = \frac{(1.943)(0.0566)^3 + (0.597)(0.0566)^3}{3} = 0.000154 \text{ in.}^4$$

$$C_{wf} = 0.0 \text{ in.}^6$$

Calculate the length, L_{cr}, over which the elastic distortional buckling half-wave forms if there is no distortional bucking restraint from the sheathing.

$$L_{cr} = \left(\frac{4\pi^4 h_o(1-\mu^2)}{t^3}\left(I_{xf}(x_{of}-h_{xf})^2 + C_{wf} - \frac{I_{xyf}^2}{I_{yf}}(x_{of}-h_{xf})^2\right) + \frac{\pi^4 h_o^4}{720}\right)^{1/4} \quad (Eq.\ C3.1.4\text{-}8)$$

$$= \left(\frac{4\pi^4(8.000)(1-0.3^2)}{(0.0566)^3}\left(\begin{array}{l} 0.00334(0.743-(-1.20))^2 + 0.0 \\ -\dfrac{(0.00750)^2}{0.0590}(0.743-(-1.20))^2 \end{array}\right) + \frac{\pi^4(8.000)^4}{720}\right)^{1/4} = 19.4 \text{ in.}$$

Based on the assumption of continuous partially effective bracing for distortional buckling, take the assumed distortional buckling length, L, as equal to L_{cr}.

$L = L_{cr} = 19.4$ in.

Calculate the elastic rotational stiffness provided by the flange to the flange/web junction, $k_{\phi fe}$.

$$k_{\phi fe} = \left(\frac{\pi}{L}\right)^4\left(EI_{xf}(x_{of}-h_{xf})^2 + EC_{wf} - E\frac{I_{xyf}^2}{I_{yf}}(x_{of}-h_{xf})^2\right) + \left(\frac{\pi}{L}\right)^2 GJ_f \quad (Eq.\ C3.1.4\text{-}9)$$

$$k_{\phi fe} = \left(\frac{\pi}{19.4}\right)^4 \begin{pmatrix} (29500)(0.00334)(0.743-(-1.20))^2 \\ +(29500)(0.0) \\ -29500\dfrac{(0.00750)^2}{0.0590}(0.743-(-1.20))^2 \end{pmatrix} + \left(\frac{\pi}{19.4}\right)^2 (11300)(0.000154)$$

$$= 0.228 \text{ kip}$$

Calculate the elastic rotational stiffness provided by the web to the flange/web junction, $k_{\phi we}$.

$$k_{\phi we} = \frac{Et^3}{12(1-\mu^2)}\left(\frac{3}{h_o} + \left(\frac{\pi}{L}\right)^2 \frac{19h_o}{60} + \left(\frac{\pi}{L}\right)^4 \frac{h_o^3}{240}\right) \quad (Eq.\ C3.1.4\text{-}10)$$

$$= \frac{29500(0.0566)^3}{12(1-(0.3)^2)}\left(\frac{3}{8.000} + \left(\frac{\pi}{19.4}\right)^2 \frac{19(8.000)}{60} + \left(\frac{\pi}{19.4}\right)^4 \frac{(8.000)^3}{240}\right)$$

$$= 0.217 \text{ kip}$$

Calculate the geometric rotational stiffness demanded by the flange from the flange/web junction, $\tilde{k}_{\phi fg}$.

$$\tilde{k}_{\phi fg} = \left(\frac{\pi}{L}\right)^2 \left[A_f\left((x_{of}-h_{xf})^2\left(\frac{I_{xyf}}{I_{yf}}\right)^2 - 2y_{of}(x_{of}-h_{xf})\left(\frac{I_{xyf}}{I_{yf}}\right) + h_{xf}^2 + y_{of}^2\right) + I_{xf} + I_{yf}\right]$$

$(Eq.$ C3.1.4-11)

$$= \left(\frac{\pi}{19.4}\right)^2 \left[0.144\begin{pmatrix} (0.743-(-1.20))^2\left(\dfrac{0.00750}{0.0590}\right)^2 \\ -2(-0.0702)(0.743-(-1.20))\left(\dfrac{0.00750}{0.0590}\right) \\ +(-1.20)^2 + (-0.0702)^2 \end{pmatrix} + 0.00334 + 0.0590\right]$$

$$= 0.00745 \text{ in.}^2$$

Calculate the geometric rotational stiffness demanded by the web from the flange/web junction, $\tilde{k}_{\phi wg}$.

$$\tilde{k}_{\phi wg} = \frac{h_o t\pi^2}{13440}\left(\frac{\left[45360(1-\xi_{web})+62160\right]\left(\dfrac{L}{h_o}\right)^2 + 448\pi^2 + \left(\dfrac{h_o}{L}\right)^2\left[53+3(1-\xi_{web})\right]\pi^4}{\pi^4 + 28\pi^2\left(\dfrac{L}{h_o}\right)^2 + 420\left(\dfrac{L}{h_o}\right)^4}\right)$$

$(Eq.$ C3.1.4-12)

$$= \frac{8.000(0.0566)\pi^2}{13440}\left(\frac{\begin{array}{c}\left[45360(1-2)+62160\right]\left(\frac{19.4}{8.000}\right)^2 \\ +448\pi^2+\left(\frac{8.000}{19.4}\right)^2\left[53+3(1-2)\right]\pi^4\end{array}}{\pi^4+28\pi^2\left(\frac{19.4}{8.000}\right)^2+420\left(\frac{19.4}{8.000}\right)^4}\right)$$

$= 0.00213$ in.^{2}

In *Eq.* C3.1.4-12 above, ξ_{web} is taken as 2, since the section is in pure symmetrical bending.

Alternatively, for this standard section, the values of $k_{\phi fe}, k_{\phi we}, \tilde{k}_{\phi fg}$ and $\tilde{k}_{\phi wg}$ could be taken from Table II-8 as 0.229 kip, 0.217 kip, 0.00745 in.^{2} and 0.00213 in.^{2}, respectively.

Calculate the elastic distortional buckling stress, F_d.

$$F_d = \beta\frac{k_{\phi fe}+k_{\phi we}+k_{\phi}}{\tilde{k}_{\phi fg}+\tilde{k}_{\phi wg}} \qquad (Eq.\ C3.1.4\text{-}6)$$

Since there is no beneficial moment gradient at the mid-span, $\beta = 1.0$.

If the beneficial restraining effect of the floor sheathing is ignored for simplicity, $k_\phi = 0.0$.

$$F_d = (1.0)\frac{0.228+0.217+0.0}{0.00745+0.00213} = 46.5 \text{ ksi} \qquad (Eq.\ C3.1.4\text{-}6)$$

Alternatively, for this standard section, the value of F_d/β could be taken from Table II-8 as 46.5 ksi.

Note that this elastic distortional buckling stress is approximately 80% higher than that calculated using the simplified provisions of *Commentary* Section C3.1.4.

If the restraining effect of the floor sheathing is included, $k_\phi = 0.0957$ kip

$$F_d = (1.0)\frac{0.228+0.217+0.0957}{0.00745+0.00213} = 56.4 \text{ ksi} \qquad (Eq.\ C3.1.4\text{-}6)$$

Note that this elastic distortional buckling stress is approximately 120% higher than that calculated using the simplified provisions of *Commentary* Section C3.1.4.

Calculate the nominal distortional buckling moment per Section C3.1.4, considering the sheathing.

$M_y = 82.0 \text{ kip-in. (from above)}$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (1.64)(56.4) = 92.5 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y/M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{82.0/92.5} = 0.942 > 0.673 \text{ therefore,}$$

$$M_n = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$$= (1-0.22(1/0.942))(1/0.942)(82.0) = 66.7 \text{ kip-in. or } 5.56 \text{ kip-ft}$$

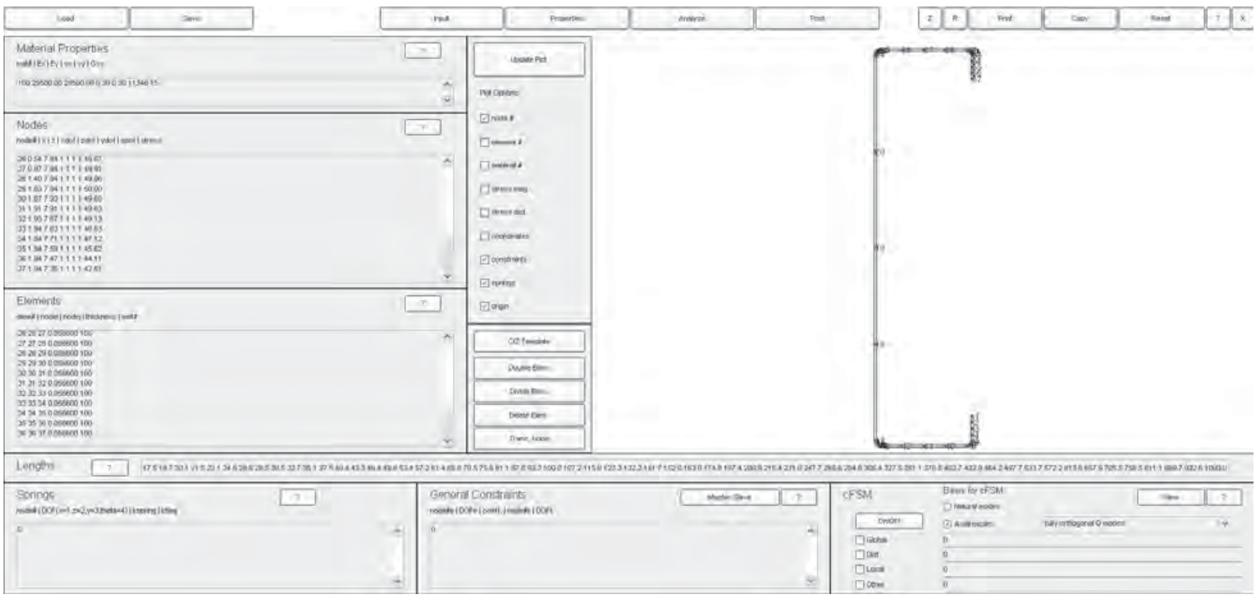
Calculate the allowable distortional buckling strength.

$$\frac{M_n}{\Omega_b} = \frac{5.56}{1.67} = 3.33 \text{ kip-ft} \qquad (Eq.\ A4.1.1\text{-}1)$$

Calculate distortional buckling strength using Section C3.1.4(b)

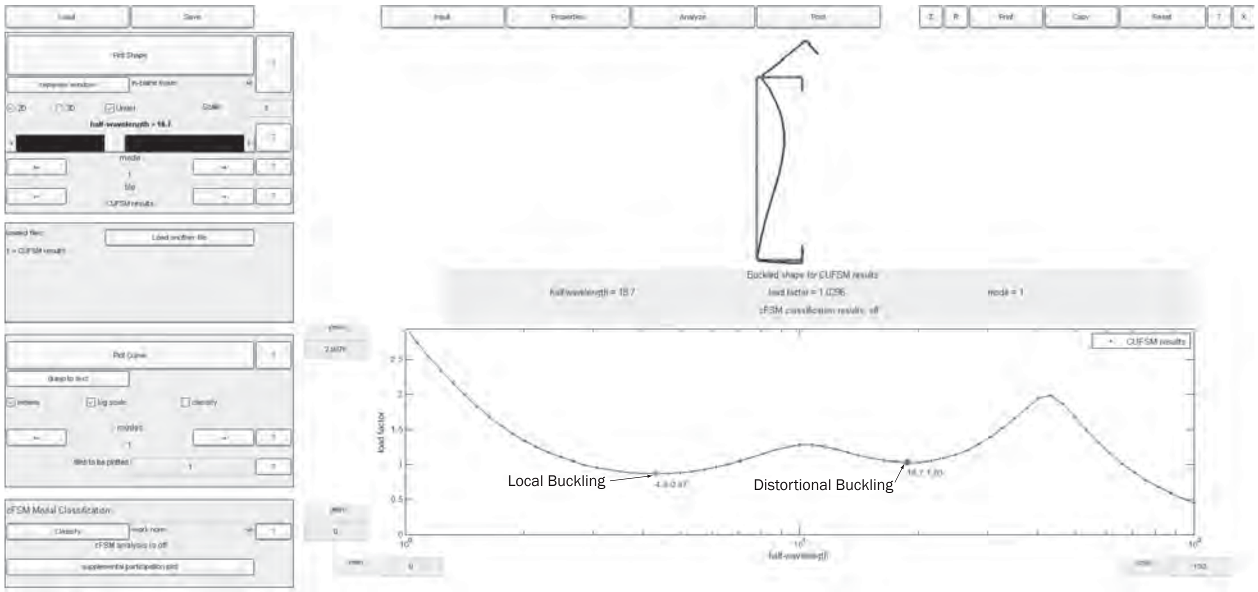
Section C3.1.4(b) permits the use of a "rational buckling analysis" to determine the elastic distortional buckling strength stress, F_d. The approach followed below determines the elastic distortional buckling stress, F_d, from an elastic buckling computer model. The stress, F_d, is then used in the calculation of the distortional buckling strength in a manner similar to the calculations above.

The cross-section is modeled using the finite strip software program CUFSM.* The input screen is shown below.



The model is built using the program's C/Z template. No rotational restraint of the compression flange, k_ϕ, is included in this version of the model. A linear stress gradient from F_y in compression at the top to F_y in tension at the bottom is applied as a reference loading. The results of the analysis are shown on the screen below.

* Schafer, B.W., Ádány, S. "Buckling analysis of cold-formed steel members using CUFSM: conventional and constrained finite strip methods." *Eighteenth International Specialty Conference on Cold-Formed Steel Structures*, Orlando, FL. October 2006.

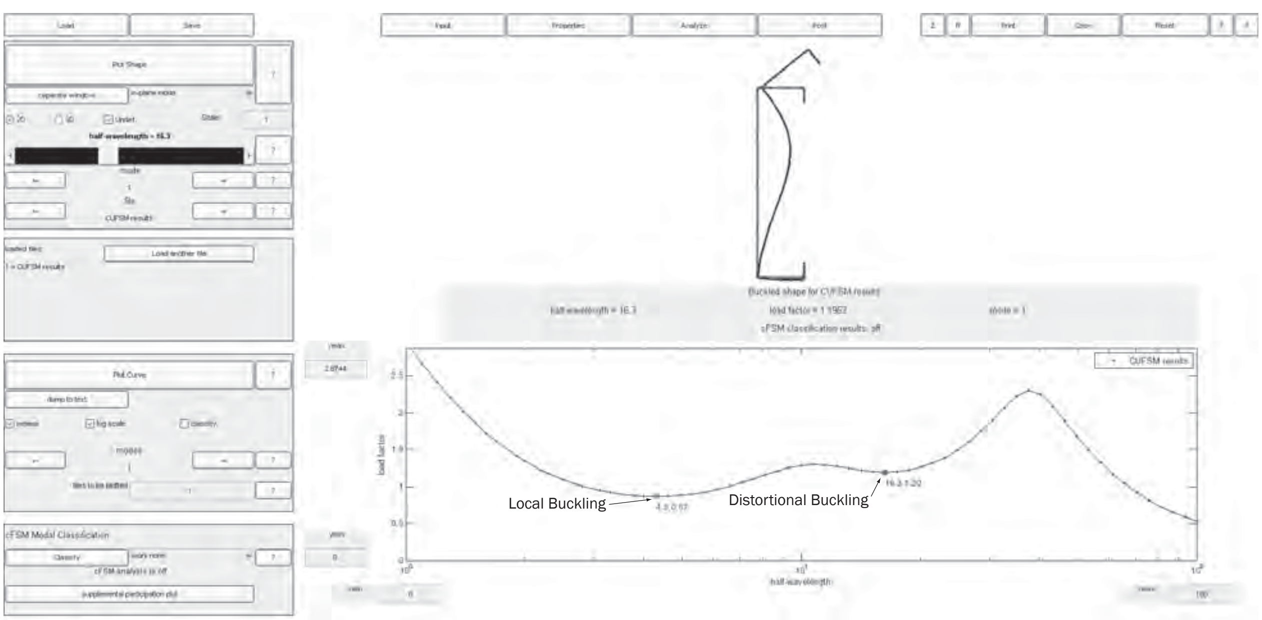


The result plots show that, without rotational restraint of the compression flange, distortional buckling occurs at a half-wavelength of 18.7 in. at a load factor of 1.03 times the applied loading. Thus, the elastic distortional buckling stress, F_d, is computed as,

$$F_d = 1.03F_y = (1.03)(50.0) = 51.5 \text{ ksi}$$

This can be compared with the elastic distortional buckling stress of 26.1 ksi calculated using *Commentary* Section C3.1.4 and 46.5 ksi calculated using *Specification* Section C3.1.4(a).

When a rotational stiffness of 0.0957 kip-in./rad./in. from the sheathing is added to the joint at the center of the compression flange, the results of the analysis are as shown in the following figure.



With rotational restraint of the compression flange, distortional buckling occurs at a loading of 1.20 times the applied load. Thus, the elastic distortional buckling stress, F_d, is computed as,

$$F_d = 1.20F_y = (1.20)(50.0) = 60.0 \text{ ksi}$$

This can be compared to an elastic distortional buckling stress of 56.4 ksi calculated from Section C3.1.4(a).

Calculate the nominal distortional buckling moment per Section C3.1.4.

$M_y = 82.0$ kip-in. (from above)

$M_{crd} = S_f F_d$ (*Eq.* C3.1.4-5)

$= (1.64)(60.0) = 98.4$ kip-in.

$\lambda_d = \sqrt{M_y / M_{crd}}$ (*Eq.* C3.1.4-3)

$= \sqrt{82.0/98.4} = 0.913 > 0.673$ therefore,

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (\textit{Eq. } C3.1.4\text{-}2)$$

$= (1 - 0.22(1/0.913))(1/0.913)(82.0) = 68.2$ kip-in. or 5.68 kip-ft

Calculate the allowable distortional buckling strength.

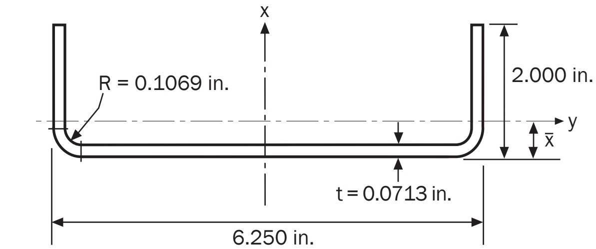
$$\frac{M_n}{\Omega_b} = \frac{5.68}{1.67} = 3.40 \text{ kip-ft} \quad (\textit{Eq. } A4.1.1\text{-}1)$$

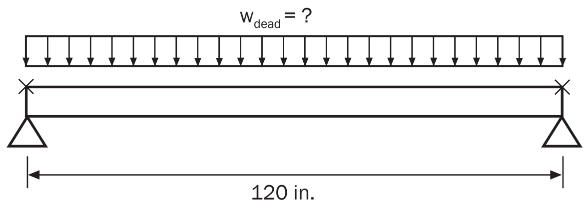
The table below compares the results of the three alternative approaches to calculating the distortional buckling strength with the strength calculated per Section C3.1.1 at the initiation of yielding, including the effects of local buckling.

Limit State	Specification Section	Allowable Flexural Strength M_n/Ω_b (kip-ft): Not Considering Sheathing Rotational Restraint	Allowable Flexural Strength M_n/Ω_b (kip-ft): Considering Sheathing Rotational Restraint
Distortional Buckling	*Commentary* C3.1.4	2.49	-
	C3.1.4(a)	-	3.33
	C3.1.4(b)	-	3.40
Yielding, including local buckling	C3.1.1(a)	3.74	

Note that Sections C3.1.4(a) and C3.1.4(b) produce similar results, both of which are significantly higher than those from *Commentary* Section C3.1.4. Since the distortional buckling strength is lower than the strength based on initiation of yielding, distortional buckling governs, regardless of the method selected to evaluate distortional buckling.

Example II-6: C-Section Without Lips in Weak Axis Bending





t = 0.0713 in.
R = 0.1069 in.
A = 0.712 in.^{2}
I_x = 3.99 in.^{4}
r_x = 2.37 in.
$\bar{x}$ = 0.422 in.
S_y = 0.161 in.^{3}
J = 0.00121 in.^{4}
C_w = 1.75 in.^{6}
j = 3.48 in.
r_o = 2.65 in.

Given:

1. Steel: F_y = 33 ksi
2. Section: Track 600T200-68 oriented with the lips up as shown in sketch above, used as a cable tray supporting dead load only.
3. Simple span of 120 in.
4. Braced against twisting and lateral deflection at ends. Ends reinforced against crippling

Required:

1. Largest permitted uniformly distributed service dead load, w_{dead}, using ASD and LRFD. Compute w_{dead} based on flexural strength, ignoring shear and serviceability.

Solution:

The beam is subject to lateral-torsional buckling, but not subject to distortional buckling.

1. Lateral-Torsional Buckling Strength (Section C3.1.2.1)

For singly-symmetric sections bent about the centroidal axis perpendicular to the axis of symmetry, use Section C3.1.2.1(a)(2)

$$F_e = \frac{C_s A\sigma_{ex}}{C_{TF}S_f}\left[j + C_s\sqrt{j^2 + r_o^2\left(\sigma_t/\sigma_{ex}\right)}\right] \qquad (Eq.\ C3.1.2.1\text{-}10)$$

$$\sigma_{ex} = \frac{\pi^2 E}{\left(K_x L_x/r_x\right)^2} \qquad (Eq.\ C3.1.2.1\text{-}11)$$

$$= \frac{\pi^2(29500)}{\left((1.0)(120)/2.37\right)^2}$$

$$= 113.6 \text{ ksi}$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{\left(K_t L_t\right)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$\sigma_t = \frac{1}{(0.712)(2.65)^2}\left[(11300)(0.00121)+\frac{\pi^2(29500)(1.75)}{\left[(1.0)(120)\right]^2}\right]$$

$= 9.81$ ksi

C_s = – 1, since there is tension on the shear center side of the centroid

C_{TF} = 1, since the mid-span moment is larger than the end moments

$$F_e = \frac{(-1)(0.712)(113.6)}{(1)(0.161)}\left[3.48+(-1)\sqrt{(3.48)^2+(2.65)^2\left((9.81)/(113.6)\right)}\right] \quad (Eq.\ C3.1.2.1\text{-}10)$$

= 43.24 ksi

$0.56F_y = (0.56)(33.0) = 18.5$ ksi

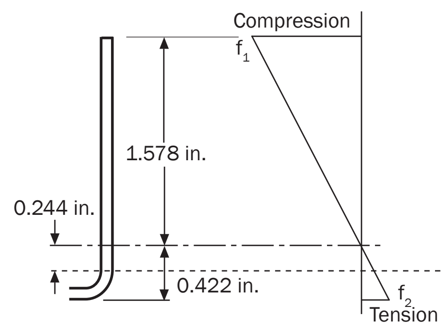
$2.78F_y = (2.78)(33.0) = 91.7$ ksi

For $2.78F_y > F_e > 0.56F_y$:

$$F_c = \frac{10}{9}F_y\left(1-\frac{10F_y}{36F_e}\right) \quad (Eq.\ C3.1.2.1\text{-}2)$$

$$= \frac{10}{9}(33.0)\left(1-\frac{(10)(33.0)}{(36)(43.24)}\right) = 28.89 \text{ ksi}$$

2. Calculate the Effective Section Modulus, S_e, at F_c = 28.89 ksi



The horizontal channel web is in tension; therefore it is fully effective. Evaluate the vertical flanges under the stress gradient.

f_1 = 28.89 ksi, maximum at tips of flanges

Assuming the section is fully effective for the first approximation,

$$f_2 = -28.89\frac{0.422-0.0713-0.1069}{2.000-0.422} = -4.46 \text{ ksi}$$

Using Section B3.2(a)(2)

$$\psi = |f_2/f_1| = |-4.46/28.89| = 0.154 \quad (Eq.\ B3.2\text{-}1)$$

$$k = 0.57+0.21\psi+0.07\psi^2 \quad (Eq.\ B3.2\text{-}5)$$

$$= 0.57+0.21(0.154)+0.07(0.154)^2 = 0.604$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \quad (Eq.\ B2.1\text{-}5)$$

$$= 0.604\frac{\pi^2 29500}{12(1-0.3^2)}\left(\frac{0.0713}{1.822}\right)^2 = 24.66 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{28.89}{24.66}} = 1.082$$

$0.673(1+\psi) = 0.673(1+0.154) = 0.777 < \lambda$; therefore,

$$\rho = (1+\psi)\frac{\left(1-\frac{0.22(1+\psi)}{\lambda}\right)}{\lambda} \qquad (Eq.\ B3.2\text{-}4)$$

$$= (1+0.154)\frac{\left(1-\frac{0.22(1+0.154)}{1.082}\right)}{1.082} = 0.816$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$$= 0.816(1.822) = 1.487 \text{ in.}$$

Using Alternative 1 method for unstiffened C-sections

Check applicability

$b_o/h_o = 2.000/6.250 = 0.320$

$0.1 \le 0.320 \le 1.0$ OK

$$k = 0.145(b_o/h_o) + 1.256 \qquad (Eq.\ B3.2\text{-}11)$$

$$= 0.145(2.000/6.250) + 1.256 = 1.302$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 1.302\frac{\pi^2 29500}{12(1-0.3^2)}\left(\frac{0.0713}{1.822}\right)^2 = 53.16 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{28.89}{53.16}} = 0.737 < 0.856\text{; therefore the flange is fully effective}$$

$$b = w = 1.822 \text{ in.} \qquad (Eq.\ B2.1\text{-}1)$$

3. Calculate the Nominal Strength, M_n

Use the results of the less conservative Alternative 1 method. All elements are fully effective; therefore, use the full section modulus.

$S_c = S_y = 0.161 \text{ in.}^3$

$$M_n = S_c F_c \qquad (Eq.\ C3.1.2.1\text{-}1)$$

$$= (0.161)(28.89)$$

$$= 4.65 \text{ kip-in.}$$

4. Permitted Uniform Service Dead Load, w_{dead}

ASD

Allowable strength

$$M \le \frac{M_n}{\Omega_b} = \frac{4.65}{1.67} = 2.78 \text{ kip-in.} \qquad (Eq.\ A4.1.1\text{-}1)$$

$$w_{dead} \le \frac{8M}{L^2} = \frac{(8)(2.78)}{(120)^2} = 0.00154 \text{ kips/in.} = 18.5 \text{ plf}$$

LRFD

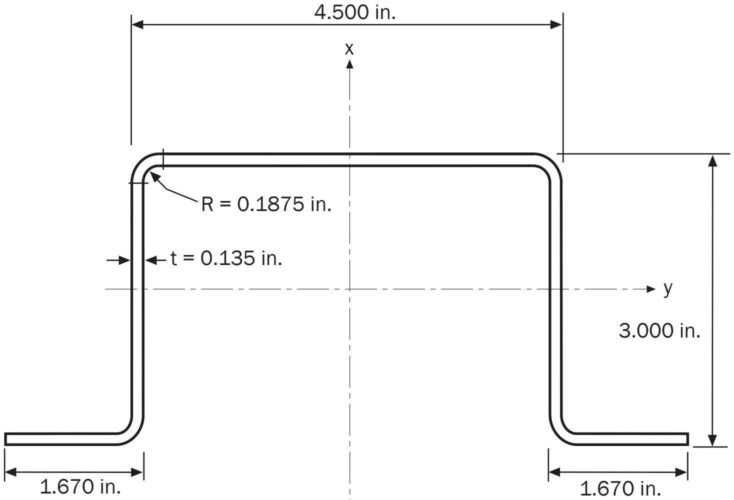
Design strength

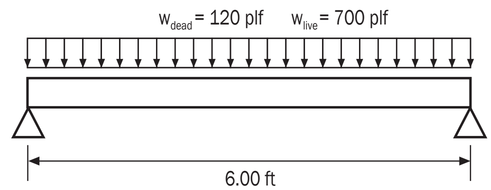
$M_u \leq \phi M_n = (0.90)(4.65) = 4.19$ kip-in. (*Eq.* A5.1.1-1)

Dead load factor = 1.4

$$w_{dead} \leq \frac{(8)(4.19)}{(120)^2(1.4)} = 0.00166 \text{ kips/in.} = 20.0 \text{ plf}$$

Example II-7: Fully Braced Hat Section





Given:

1. Steel: F_y = 50 ksi
2. Section: 3HU4.5x135 shown in sketch above
3. Top flange is in compression and is fully braced.

Required:

1. Check the flexural adequacy of a 6 ft long simple span beam with:
   Dead load, w_{dead} = 120 plf
   Live load, w_{live} = 700 plf
2. Do not consider inelastic reserve.
3. Check using both ASD and LRFD.

Solution:

1. Calculate Nominal Strength, M_n (Section C3.1.1)

Since the member is fully braced and not subject to lateral-torsional or distortional buckling, calculate the nominal strength using Section C3.1.1.

From Example I-13 or Table II-6, S_e = 1.52 in.^{3}

$$M_n = S_eF_y \qquad (Eq.\ C3.1.1\text{-}1)$$

$$= (1.52)(50) = 76.0 \text{ kip-in.}$$

2. Calculate Required and Available Strength

Bending moments

$$M_D = \frac{w_{dead}L^2}{8} = \frac{(0.120)(6)^2}{8} = 0.540 \text{ kip-ft} = 6.48 \text{ kip-in.}$$

$$M_L = \frac{w_{live}L^2}{8} = \frac{(0.700)(6)^2}{8} = 3.15 \text{ kip-ft} = 37.8 \text{ kip-in.}$$

ASD

Required allowable strength

$$M = M_D + M_L = 6.48 + 37.8 = 44.3 \text{ kip-in.}$$

Allowable strength

$M < M_n/\Omega_b$ (*Eq.* A4.1.1-1)

$\Omega_b = 1.67$

$$\frac{M_n}{\Omega_b} = \frac{76.0}{1.67} = 45.5 \text{ kip-in.} > 44.3 \text{ kip-in. OK}$$

LRFD

Required strength

$$\begin{aligned} M_u &= 1.2M_D + 1.6M_L \\ &= (1.2)(6.48) + (1.6)(37.8) \\ &= 68.3 \text{ kip-in.} \end{aligned}$$

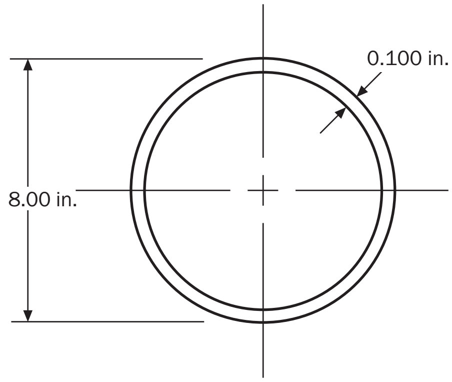
Design strength

$M_u < \phi_b M_n$ (*Eq.* A5.1.1-1)

$\phi_b = 0.90$

$\phi_b M_n = (0.90)(76.0) = 68.4$ kip-in. > 68.3 kip-in. OK

Example II-8: Tubular Section – Round



Given:

1. Steel: F_y = 42 ksi
2. Section: Shown in sketch above

Required:

1. Determine the ASD allowable flexural strength, M_n/Ω_b .
2. Determine the LRFD design flexural strength, $\phi_b M_n$.

Solution:

1. Nominal Flexural Strength (Section C3.1.3):

Ratio of outside diameter to wall thickness,

$D/t \;=\; 8.00/0.100 = 80.0$

Check limit

$D/t \;<\; 0.441E/F_y = 0.441(29500/42) = 310 \quad \text{OK}$

Full section properties

$$S_f = \pi\frac{(\text{Outside Diameter})^4 - (\text{Inside Diameter})^4}{32(\text{Outside Diameter})}$$

$$= \pi\frac{(8.00)^4 - (7.80)^4}{(32)(8.00)}$$

$$= 4.84 \text{ in.}^3$$

Determine the governing equation

$0.0714E/F_y = 0.0714(29500/42) = 50.2$

$0.318E/F_y = 0.318(29500/42) = 223$

Since $0.0714E/F_y < D/t < 0.318E/F_y$

$$F_c = \left[0.970 + 0.020\left(\frac{E/F_y}{D/t}\right)\right]F_y \qquad (Eq.\ C3.1.3\text{-}3)$$

$$= \left[0.970 + 0.020\left(\frac{29500/42}{80.0}\right)\right]42 = 48.1 \text{ ksi}$$

$$M_n = F_c S_f \qquad (Eq.\ C3.1.3\text{-}1)$$

$$M_n = (48.1)(4.84) = 233 \text{ kip-in.}$$

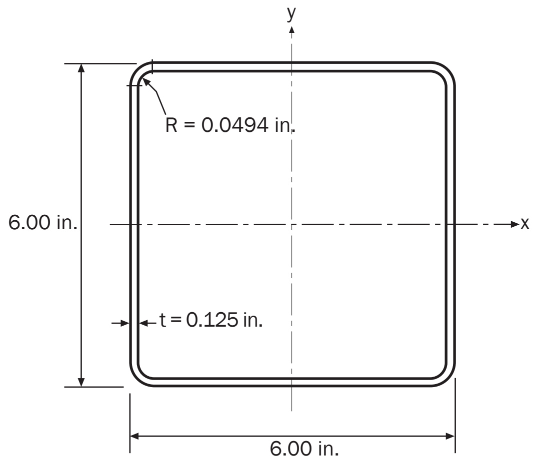
2. ASD Allowable Flexural Strength

$$\frac{M_n}{\Omega_b} = \frac{233}{1.67} = 140 \text{ kip-in.}$$

3. LRFD Design Flexural Strength

$$\phi_b M_n = (0.95)(233) = 221 \text{ kip-in.}$$

Example II-9: Tubular Section – Rectangular



Given:

1. Steel: F_y = 46 ksi
2. Section: HSS 6x6x⅛ (from Table 1-12, AISC *Steel Construction Manual*, 2010)
3. Calculated gross properties using nominal dimensions above

   A = 2.91 in.^{2}

   I = 16.7 in.^{4}

   S = 5.56 in.^{3}

3. Simple span length = 10.0 ft
4. Laterally braced at both ends

Required:

1. Determine the ASD flexural allowable strength, M_n/Ω_b .
2. Determine the LRFD design flexural strength, $\phi_b M_n$.
3. Compare the calculated available flexural strengths to those calculated using the AISC procedures. The inside radius given above is selected to give the same flat width, w, for the flanges and webs used in the AISC calculations.

Solution:

Compute the nominal flexural strength, M_n, according to *Specification* Sections C3.1.1(a) and C3.1.2.2 for closed box members.

1. Nominal Flexural Strength, M_n, Initiation of Yielding (Section C3.1.1)

Check the compression flange as a uniformly compressed compression element in accordance with Section B2.1. By calculations not shown, the effective width of the compression flange width is found to be 4.608 in.

Check webs in accordance with Section B2.3. The webs are found to be fully effective, by calculations not shown.

The effective section properties can then be calculated as:

I_x = 15.5 in.^{4}

S_e = 4.95 in.^{3}

The nominal flexural strength is calculated as:

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

$$= (4.95)(46) = 228 \text{ kip-in. or } 19.0 \text{ kip-ft}$$

2. Nominal Flexural Strength, M_n, Lateral-Torsional Buckling (Section C3.1.2.2)

According to *Specification Eq.* C3.1.2.2-1, the minimum unbraced length of a closed box member subject to lateral-torsional buckling, L_u, is calculated as:

$$L_u = \frac{0.36 C_b \pi}{F_y S_f}\sqrt{EGJI_y} \quad (Eq.\ C3.1.2.2\text{-}1)$$

where

C_b = 1.0 (assumed)

I_y (full section) = 16.7 in.^{4}

S_f (full section) = 5.56 in.^{3}

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

where

$a = b = D - t = 6.00 - 0.125 = 5.875$ in.

$t_1 = t_2 = 0.125$ in.

$$J = \frac{2(5.875)^4}{2(5.875/0.125)} = 25.3 \text{ in.}^4 \quad (Eq.\ C\text{-}C3.1.2.2\text{-}1)$$

therefore,

$$L_u = \frac{0.36(1.0)\pi}{(46)(5.56)}\sqrt{(29500)(11300)(25.3)(16.7)} = 1660 \text{ in.} = 138 \text{ ft} \quad (Eq.\ C3.1.2.2\text{-}1)$$

Since L_u > 10.0 ft, the member is not subject to lateral-torsional buckling and the flexural strength is based on Section C3.1.1(a). That is,

M_n = 19.0 kip-ft

3. ASD Allowable Flexural Strength

$$\frac{M_n}{\Omega_b} = \frac{19.0}{1.67} = 11.4 \text{ kip-ft}$$

4. LRFD Design Flexural Strength

$$\phi_b M_n = (0.90)(19.0) = 17.1 \text{ kip-ft.}$$

5. Comparison of AISI and AISC Available Flexural Strengths

The available flexural strengths calculated above are compared with those calculated in accordance with the 2010 AISC Specification in the table below. AISC strengths are taken from Table 3-13 of the 14th Edition of the AISC Steel Construction Manual.

Available Strength	AISI kip-ft	AISC kip-ft	$\frac{\text{AISI}}{\text{AISC}}$
ASD: M_n/Ω_b	11.4	10.4	1.10
LRFD: $\phi_b M_n$	17.1	15.6	1.10

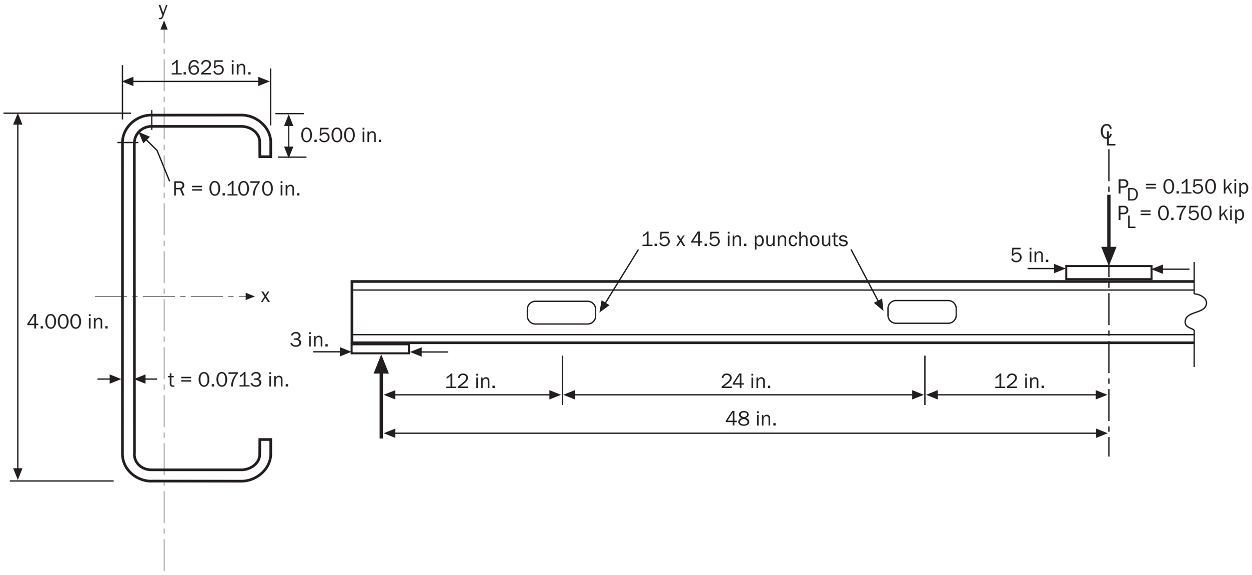
The differences between the AISI and AISC strengths shown in the table above are due to the following factors:

a. The AISI calculations were performed using the nominal wall thickness of the HSS, while the AISC calculations were performed using 93% of the nominal wall thickness as required by AISC.

b. The calculations of the effective width of the compression flange are slightly different in the two specifications, including slightly different values of E.

c. The differences shown in the table above are representative of HSS sections having slender elements, but actual ratios vary somewhat based on section dimensions and yield stress.

Example II-10: C-Section With Openings



Given:

1. Steel: F_y = 50 ksi, F_u = 65 ksi
2. Section: 400S162-68 as shown above
3. Section is simply supported, fully braced against lateral-torsional and distortional buckling, and is fastened to support.
4. 1.5 in. by 4.5 in. web punchouts with 0.25 inch corner radii located as shown above. Note that the location of punchouts is often not known with this precision.

Required:

Check the adequacy of the section considering:

a. Flexure
b. Shear
c. Web Crippling
d. Combined Bending and Shear
e. Combined Bending and Web Crippling

Use:

1. ASD – ASCE/SEI 7-10 ASD load combination D + L
2. LRFD - ASCE/SEI 7-10 LRFD load combination 1.2D + 1.6L

Neglect self-weight of beam

Solution:

1. Flexural Strength

a) Required strength

ASD required strength

$$P = P_D + P_L = 0.150 + 0.750 = 0.900 \text{ kip}$$
$$V = P/2 = 0.900/2 = 0.450 \text{ kip}$$

At center, away from holes

$$M = \frac{PL}{4} = \frac{(0.900)(8.0)}{4} = 1.80 \text{ kip-ft} = 21.6 \text{ kip-in.}$$

At edge of hole closest to center

$$M = V\left[L/2-(12.0-2.25)\right]=(0.450)\left[96.0/2-9.75\right]=17.2\text{ kip-in.}$$

LRFD required strength

$$P_u = 1.2P_D + 1.6P_L = (1.2)(0.150)+(1.6)(0.750)=1.38\text{ kips}$$

$$V_u = P_u/2 = 1.38/2 = 0.690\text{ kip}$$

At center, away from holes

$$M_u = \frac{P_uL}{4} = \frac{(1.38)(8.0)}{4} = 2.76 \text{ kip-ft} = 33.1 \text{ kip-in.}$$

At edge of hole closest to center

$$M_u = V_u\left[L/2-(12.0-2.25)\right]=(0.690)\left[(96.0/2)-9.75\right]=26.4\text{ kip-in.}$$

b) Flexural strength without holes

The member is not subject to lateral-torsional buckling, so compute strength using Section C3.1.1 with effective section modulus, S_e, at $f = F_y$.

It can be shown that, in the area without holes, the section is eligible for strength increase using the cold work of forming provisions of Section A7.2.

$F_y = F_{ya} = 56.6$ ksi (calculations not shown)

$S_e = 0.670\text{ in.}^3$ (calculations not shown)

$M_n = S_eF_y$ (*Eq.* C3.1.1-1)

$= (0.670)(56.6) = 37.9$ kip-in.

c) Nominal flexural strength with holes

The member is not subject to lateral-torsional buckling, so compute strength using Section C3.1.1 with effective section modulus, S_e, at $f = F_y$.

Check web using Section B2.4 - "C-Section Webs with Holes under Stress Gradient".

$d_h = 1.5$ in.

$L_h = 4.5$ in.

$h = 4.00 - 2(0.1070 + 0.0713) = 3.643$ in.

Check limits

$d_h/h = 1.5/3.643 = 0.412 < 0.7$ OK

$h/t = 3.643/0.0713 = 51.1 < 200$ OK

Holes are centered at mid-depth of web OK

Clear distance between holes $= 24.0 - 4.5 = 19.5$ in. > 18.0 in. OK

Corner radii = 0.25 in. $> (2)(0.0713) = 0.143$ in. OK

$d_h < 2.5$ in. OK

$L_h = 4.5$ in. OK

$d_h > 9/16$ in. OK

Since $d_h/h > 0.38$, treat compression portion of web as a uniformly compressed unstiffened element as follows:

$w = (h - d_h)/2 = (3.643 - 1.50)/2 = 1.072$ in.

$k = 0.43$

Calculate first estimate of f_1 at the top of the flat width using similar triangles with gross properties.

$$f = f_1 = 50\left(\frac{4.00/2 - 0.0713 - 0.1070}{4.00/2}\right) = 45.5 \text{ ksi}$$

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \qquad (\textit{Eq. } \text{B2.1-5})$$

$$= 0.43\frac{\pi^2\left(29500\right)}{12\left(1-0.3^2\right)}\left(\frac{0.0713}{1.072}\right)^2 = 50.7 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (\textit{Eq. } \text{B2.1-4})$$

$$= \sqrt{\frac{45.5}{50.7}} = 0.947 > 0.673 \;\; \therefore \text{ web is subject to local buckling}$$

$$\rho = \left(1-0.22/\lambda\right)/\lambda \qquad (\textit{Eq. } \text{B2.1-3})$$

$$= \left(1-0.22/0.947\right)/0.947 = 0.811$$

$$b = \rho w \qquad (\textit{Eq. } \text{B2.1-2})$$

$$= \left(0.811\right)\left(1.072\right) = 0.869 \text{ in.}$$

Since the web is not fully effective, the cross-section is not eligible for design using the cold work of forming provision in this area.

Check flange and lip

It can be shown that the flange and lip are fully effective at this stress level (calculations not shown).

Recompute section properties

Calculate the effective section modulus, S_e, deducting both the 1.50 inch hole and the ineffective portion of the compression area of the web. Using the methods illustrated in the examples in Part I, the effective flexural properties can be computed as:

y_c = 2.03 in. (from top fiber)
I_{xe} = 1.32 in.^{4}
S_{xe} = 0.648 in.^{3}

Further iterations

The shift in the centroid causes a very slight change to the stress distribution and consequently a very small change in the value of f_1 at the top of the flat width of the web, but not enough to change the values calculated above.

Nominal flexural strength

$$M_n = S_e F_y \qquad (\textit{Eq. } \text{C3.1.1-1})$$

$$= \left(0.648\right)\left(50\right) = 32.4 \text{ kip-in.}$$

Alternatively, M_n can be taken from Table II-2. For a 400S162-68 with F_y = 50 ksi,

M_n = 32.4 kip-in.

d) Available strength

ASD allowable strength

$\Omega_b = 1.67$

At center, away from holes

$$\frac{M_n}{\Omega_b} = \frac{37.9}{1.67} = 22.7 \text{ kip-in.} > 21.6 \text{ kip-in. OK}$$

At holes nearest center

$$\frac{M_n}{\Omega_b} = \frac{32.4}{1.67} = 19.4 \text{ kip-in.} > 17.2 \text{ kip-in. OK}$$

LRFD design strength

$$\phi_b = 0.90$$

At center, away from holes

$$\phi_b M_n = (0.90)(37.9) = 34.1 \text{ kip-in.} > 33.1 \text{ kip-in. OK}$$

At holes nearest center

$$\phi_b M_n = (0.90)(32.4) = 29.2 \text{ kip-in.} > 26.4 \text{ kip-in. OK}$$

2. Shear Strength

a) Required strength

ASD required strength

$V = 0.450$ kip (from above)

LRFD required strength

$V_u = 0.690$ kip (from above)

b) Shear strength without holes - Section C3.2.1

$h/t = 51.1$ (computed above)

$$\sqrt{Ek_v/F_y} = \sqrt{(29500)(5.34)/50} = 56.1$$

Since $h/t < \sqrt{Ek_v/F_y}$,

$$F_v = 0.60F_y \qquad (Eq.\ C3.2.1\text{-}2)$$
$$= (0.60)(50) = 30 \text{ ksi}$$

$$V_n = A_w F_v \qquad (Eq.\ C3.2.1\text{-}1)$$
$$= (3.643)(0.0713)(30) = 7.79 \text{ kips}$$

c) Shear strength with holes - Section C3.2.2

Limits same as those checked above OK

$$c = h/2 - d_h/2 \qquad (Eq.\ C3.2.2\text{-}3)$$
$$= 3.643/2 - 1.50/2 = 1.07 \text{ in.}$$
$$c/t = 1.07/0.0713 = 15.0$$

Since $5 < c/t < 54$,

$$q_s = c/(54t) \qquad (Eq.\ C3.2.2\text{-}1)$$
$$= 1.07/[(54)(0.0713)] = 0.278$$
$$V_n = q_s V_n = (0.278)(7.79) = 2.17 \text{ kips}$$

Alternatively, V_n can be taken from Table II-2. For a 400S162-68 with F_y = 50 ksi,

V_n = 2.17 kips

d) Available strength

ASD allowable strength

$$\Omega_v = 1.60$$

$$\frac{V_n}{\Omega_v} = \frac{2.17}{1.60} = 1.36 \text{ kips} > 0.450 \text{ kip OK}$$

LRFD design strength

$$\phi_v = 0.95$$

$$\phi_v V_n = (0.95)(2.17) = 2.06 \text{ kips} > 0.690 \text{ kip OK}$$

3. Combined Bending and Shear Strength

ASD

Near the center of the beam (no holes)

$$\sqrt{\left(\frac{\Omega_b M}{M_{nxo}}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2} \le 1.0 \qquad (\textit{Eq. } \text{C3.3.1-1})$$

$$\sqrt{\left(\frac{(1.67)(21.6)}{37.9}\right)^2 + \left(\frac{(1.60)(0.450)}{7.79}\right)^2} = 0.956 < 1.0 \text{ OK}$$

At edge of the hole closest to the center

$$\sqrt{\left(\frac{(1.67)(17.2)}{32.4}\right)^2 + \left(\frac{(1.60)(0.450)}{2.17}\right)^2} = 0.947 < 1.0 \text{ OK} \qquad (\textit{Eq. } \text{C3.3.1-1})$$

Alternatively, this case can be checked with Table II-11a. For a 400S162-68 with F_y = 50 ksi, using a required allowable moment, M, of 17.2 kip-in., conservatively interpolate the maximum permitted shear, V.

for M = 16.8 kip-in., $V \le 0.678$ kip
for M = 18.7 kip-in., $V \le 0.351$ kip
for M = 17.2 kip-in., interpolating,

$$V \le 0.351 + \left(\frac{18.7 - 17.2}{18.7 - 16.8}\right)(0.678 - 0.351) = 0.609 \text{ kip} > 0.450 \text{ kip OK}$$

LRFD

Near the center of the beam (no holes)

$$\sqrt{\left(\frac{\overline{M}}{\phi_b M_{nxo}}\right)^2 + \left(\frac{\overline{V}}{\phi_v V_n}\right)^2} \le 1.0 \qquad (\textit{Eq. } \text{C3.3.2-1})$$

$$\overline{M} = M_u$$

$$\overline{V} = V_u$$

$$\sqrt{\left(\frac{33.1}{(0.90)(37.9)}\right)^2 + \left(\frac{0.690}{(0.95)(7.79)}\right)^2} = 0.975 < 1.0 \text{ OK} \qquad (\textit{Eq. } \text{C3.3.2-1})$$

At edge of hole closest to the center

$$\sqrt{\left(\frac{26.4}{(0.90)(32.4)}\right)^2 + \left(\frac{0.690}{(0.95)(2.17)}\right)^2} = 0.965 < 1.0 \text{ OK} \qquad (\textit{Eq. } \text{C3.3.2-1})$$

Alternatively, this case can be checked with Table II-11b. For a 400S162-68 with F_y = 50 ksi, using a required moment, M_u, of 26.4 kip-in., conservatively interpolate the maximum permitted factored shear, V_u.

for M_u = 28.2 kip-in., $V_u \leq 0.533$ kip
for M_u = 25.3 kip-in., $V_u \leq 1.03$ kips
for M_u = 26.4 kip-in., interpolating,

$$V_u \leq 0.533 + \left(\frac{28.2-26.4}{28.2-25.3}\right)(1.03-0.533) = 0.841 \text{ kip} > 0.690 \text{ kip OK}$$

4. Web Crippling Strength

a) Required strength

ASD required strength

End condition

$P = V = 0.450$ kip

Interior condition

$P = P_D + P_L = 0.150 + 0.750 = 0.900$ kip

LRFD required strength

End condition

$P_u = V_u = 0.690$ kip

Interior condition

$P_u = 1.2P_D + 1.6P_L = (1.2)(0.150) + (1.6)(0.750) = 1.38$ kips

b) Web crippling strength without holes - Section C3.4.1

θ = 90 degrees
R = 0.1070 in.
t = 0.0713 in.
h = 3.643 in.

End Condition

N = 3.0 in.

From Table C3.4.1-2

Check limits

$h/t = 51.1 < 200$ OK (computed above)
$N/t = 3.0/0.0713 = 42.1 < 210$ OK
$N/h = 3.0/3.643 = 0.823 < 2.0$ OK

For conditions of Fastened to Support/Stiffened or Partially Stiffened Flanges/One Flange Loading/End Condition, from Table C3.4.1-2:

$C = 4$
$C_R = 0.14$
$C_N = 0.35$
$C_h = 0.02$
$\Omega_w = 1.75$
$\phi_w = 0.85$
$R/t = 0.1070/0.0713 = 1.50 < 9$ OK

$$P_n = Ct^2F_y\sin\theta\left(1-C_R\sqrt{\frac{R}{t}}\right)\left(1+C_N\sqrt{\frac{N}{t}}\right)\left(1-C_h\sqrt{\frac{h}{t}}\right) \qquad (Eq.\ C3.4.1\text{-}1)$$

$$= (4)(0.0713)^2(50)\sin(90)\left(1-0.14\sqrt{\frac{0.1070}{0.0713}}\right)\left(1+0.35\sqrt{\frac{3.0}{0.0713}}\right)\left(1-0.02\sqrt{\frac{3.643}{0.0713}}\right)$$

$$= 2.36 \text{ kips}$$

Alternatively, P_n can be conservatively interpolated from Table II-14. For a 400S162-68 with F_y = 50 ksi, fastened to support, case A:

for N = 2 in., P_n = 2.06 kips

for N = 4 in., P_n = 2.61 kips

for N = 3 in., interpolating, P_n = 0.5(2.06 + 2.61) = 2.34 kips

Interior Condition

N = 5.0 in.

From Table C3.4.1-2

Check limits (other limits checked above)

$N/t = 5.0/0.0713 = 70.1 < 210$ OK

$N/h = 5.0/3.643 = 1.37 < 2.0$ OK

For conditions of Fastened to Support/Stiffened or Partially Stiffened Flanges/One Flange Loading/Interior Condition

$C = 13$

$C_R = 0.23$

$C_N = 0.14$

$C_h = 0.01$

$\Omega_w = 1.65$

$\phi_w = 0.90$

$R/t = 1.50 < 5.0$ OK

$$P_n = Ct^2F_y\sin\theta\left(1-C_R\sqrt{\frac{R}{t}}\right)\left(1+C_N\sqrt{\frac{N}{t}}\right)\left(1-C_h\sqrt{\frac{h}{t}}\right) \qquad (Eq.\ C3.4.1\text{-}1)$$

$$= (13)(0.0713)^2(50)\sin(90)\left(1-0.23\sqrt{\frac{0.1070}{0.0713}}\right)\left(1+0.14\sqrt{\frac{5.0}{0.0713}}\right)\left(1-0.01\sqrt{\frac{3.643}{0.0713}}\right)$$

$$= 4.79 \text{ kips}$$

Alternatively, P_n can be conservatively interpolated from Table II-14. For a 400S162-68 with F_y = 50 ksi, fastened to support, case B:

for N = 4 in., P_n = 4.51 kips

for N = 6 in., P_n = 5.03 kips

for N = 5 in., interpolating, $P_n = 0.5(4.51+5.03) = 4.77$ kips

c) Web crippling strength with holes - Section C3.4.2

Limits same as those checked above OK

End condition

$x = 12.0 - 4.50/2 - 3.0/2 = 8.25$ in. (distance between web hole and edge of bearing)

$R_c = 1.01 - 0.325d_h/h + 0.083x/h \le 1.0$ (*Eq.* C3.4.2-1)

$= 1.01 - (0.325)(1.50)/3.643 + (0.083)(8.25)/3.643 = 1.06 > 1$ Use 1.0

$P_n = R_c P_n = (1.0)(2.36) = 2.36$ kips

Alternatively, R_c can be extrapolated from Table II-16b. For stud depth = 4 in., x >> 5 in.,

$R_c = 1.00$

Interior condition

$x = 12.0 - 4.50/2 - 5.0/2 = 7.25$ in. (distance between web hole and edge of bearing)

$R_c = 0.90 - 0.047 d_h/h + 0.053 x/h \le 1.0$ (*Eq.* C3.4.2-2)

$= 0.90 - (0.047)(1.50)/3.643 + (0.053)(7.25)/3.643 = 0.986 < 1.0$ OK

$P_n = R_c P_n = (0.986)(4.79) = 4.72$ kips

Alternatively, R_c can be conservatively interpolated from Table II-16a. For depth = 4 in.,

for x = 4 in., $R_c = 0.94$
for x = 8 in., $R_c = 0.99$
for x = 7.25 in., interpolating, $R_c = 0.94 + \left(\frac{7.25-4}{8-4}\right)(0.99-0.94) = 0.98$

d) Available strength

ASD allowable strength

End condition

$\Omega_w = 1.75$

$\frac{P_n}{\Omega_w} = \frac{2.36}{1.75} = 1.35$ kips > 0.450 kip OK

Interior condition

$\Omega_w = 1.65$

$\frac{P_n}{\Omega_w} = \frac{4.72}{1.65} = 2.86$ kips > 0.900 kip OK

LRFD design strength

End condition

$\phi_w = 0.85$

$\phi_w P_n = (0.85)(2.36) = 2.01$ kips > 0.690 kip OK

Interior condition

$\phi_w = 0.90$

$\phi_w P_n = (0.90)(4.72) = 4.25$ kips > 1.38 kip OK

5. Combined Bending and Web Crippling

Concentrated load at center of beam controls

ASD

$$0.91\left(\frac{P}{P_n}\right) + \left(\frac{M}{M_{nxo}}\right) \le \frac{1.33}{\Omega} \qquad (Eq.\ C3.5.1\text{-}1)$$

$$0.91\left(\frac{0.900}{4.72}\right) + \left(\frac{21.6}{37.9}\right) \le \frac{1.33}{1.70}$$

$0.743 < 0.782$ OK

LRFD

$$0.91\left(\frac{\bar{P}}{P_n}\right) + \left(\frac{\bar{M}}{M_{nxo}}\right) \le 1.33\phi \qquad (Eq.\ C3.5.2\text{-}1)$$

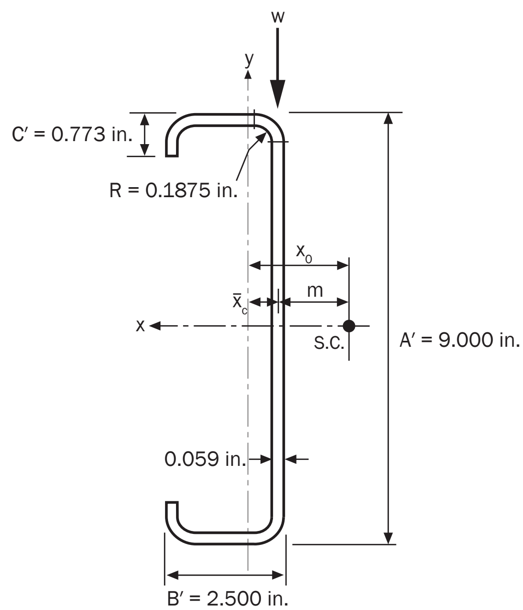
$\overline{P} = P_u$

$\overline{M} = M_u$

$$0.91\left(\frac{1.38}{4.72}\right)+\left(\frac{33.1}{37.9}\right)\leq 1.33(0.90)$$

$1.14 < 1.20$ OK

Example II-11: C-Section With Combined Bending and Torsional Loading



Given:

1. Steel: F_y = 55 ksi
2. Section: 9CS2.5x059
3. Gross Section Properties (from Example I-1 or Table I-1)

   $I_x = 10.3\ \text{in.}^4$ $S_x = 2.29\ \text{in.}^3$ $J = 0.00102\ \text{in.}^4$ $C_w = 11.9\ \text{in.}^6$

   $x_0 = -1.66\ \text{in.}$ $m = 1.05\ \text{in.}$ $\bar{x} = 0.641\ \text{in.}$

4. Effective Section Properties (from Example I-8 or Table II-1)

   $I_{xe} = 9.18\ \text{in.}^4$ $S_{xe} = 1.89\ \text{in.}^3$ $\bar{y} = 4.859\ \text{in.}$

5. The member is a simply supported beam spanning 25 feet supporting a uniformly distributed load.
6. The load is applied vertically in the plane of the web.
7. The beam has torsional braces at both ends of the member and at the brace points specified below.

Required:

Determine the nominal flexural strength, M_n, and allowable uniform loading based on initiation of yielding of the effective section considering the effects of torsion. Consider alternate conditions of:

1. A single brace at mid-span
2. Two braces, each at the one-third points of the span

Compute the shear stresses, including the effects of torsion for the condition of a single brace at mid-span.

Assumptions:

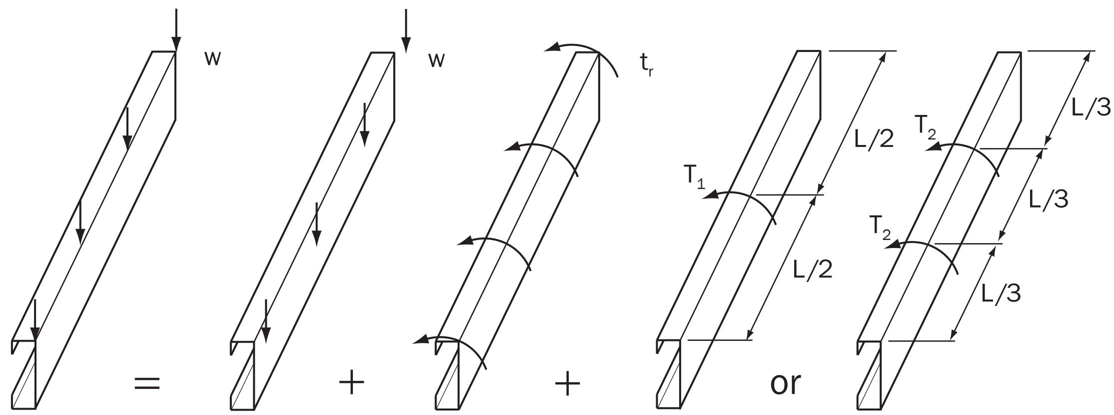
1. Rotation is completely restrained at the member ends and at the braces.

2. The member is free to warp at both ends.

Solution:

A torsional reduction factor, R, is calculated using Section C3.6 and applied to the nominal strength calculated using Section C3.1.1(a). Note that this reduction factor is not applied to other limit states, such as lateral-torsional buckling or distortional buckling.

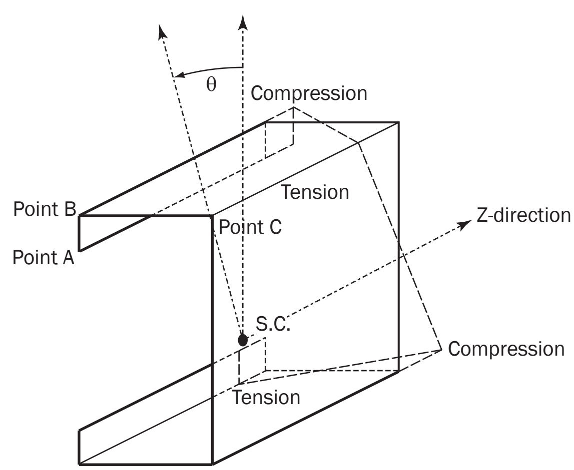
This solution is based on the method described in *AISC Design Guide 9: Torsional Analysis of Structural Steel Members*[*] (DG 9). The actual loading is modeled by superimposing the three conditions as shown in the figure below.



Load along web	Load through shear center Simple bending	Distributed torque DG 9 Case 4 $t_r = w\,x_o$	Brace at mid-span DG 9 Case 3 $\alpha = 0.5$ T_1	Brace at 1/3 points DG 9 Case 3 $\alpha = 0.33$ or 0.67 $T_2 = T_2$
	Loading = #1	#2	#3	#3A

Torsional warping normal stresses are calculated using the second derivative of the angle of rotation, θ, with respect to the position, z, along the length of the member.

The sign convention for use with all torsion expressions are shown in the figure to the right. Note that calculated values for θ and θ″ may be either positive or negative. The proper sign for these calculated values must be used for torsional stress calculations. Calculated positive values are in the directions shown.



Positive rotation and warping stresses

[*] Seaburg, P.A and Carter, C.J, *Design Guide 9: Torsional Analysis of Structural Steel Members*, American Institute of Steel Construction, Chicago, IL, 1997.

For the singly-symmetric channel, only the compression side need be checked for combined bending and warping.

Normal stresses due to warping

$$f_{ws} = EW_{ns}\theta'' \qquad \text{(AISC Design Guide 9, } \textit{Eq}\text{. 4.3a)}$$

where W_{ns} are normalized warping functions (section properties) of the cross-section at each point of consideration given by:

Point A, at tip of flange stiffener

$$W_A = \frac{\bar{a}\left(m - \bar{b}\right)}{2} - \bar{c}\left(m + \bar{b}\right)$$

Point B, at junction of the flange and stiffener

$$W_B = \frac{\bar{a}\left(m - \bar{b}\right)}{2}$$

Point C, at junction of the flange and web

$$W_C = \frac{\bar{a}m}{2}$$

where,

$\bar{a}$ = centerline web height = 8.941 in.
$\bar{b}$ = centerline flange width = 2.441 in.
$\bar{c}$ = centerline lip length = 0.744 in.
m = distance from shear center to web centerline = 1.05 in.

The torsional warping properties for this section are:

$$W_A = \frac{(8.941)(1.05 - 2.441)}{2} - (0.744)(1.05 + 2.441) = -8.82 \text{ in.}^2$$

$$W_B = \frac{(8.941)(1.05 - 2.441)}{2} = -6.22 \text{ in.}^2$$

$$W_C = \frac{(8.941)(1.05)}{2} = 4.69 \text{ in.}^2$$

Formulas for rotation due to a number of torsional loadings are given in Appendix C.4 of DG 9. Summarized below are those used in subsequent calculations.

For Loading #2 above, use DG 9 Case 4 – Uniformly distributed torque on member with pinned ends.

$$\theta_t = \frac{t_r a^2}{GJ}\left[\frac{L^2}{2a^2}\left(\frac{z}{L} - \frac{z^2}{L^2}\right) + \cosh\left(\frac{z}{a}\right) - \tanh\left(\frac{L}{2a}\right)\sinh\left(\frac{z}{a}\right) - 1.0\right]$$

where

$$a = \sqrt{\frac{EC_w}{GJ}} \qquad \text{(AISC Design Guide 9, } \textit{Eq}\text{. 3.12)}$$

Differentiating twice with respect to z yields

$$\theta_t'' = \frac{t_r}{GJ}\left[-1.0 + \cosh\left(\frac{z}{a}\right) - \tanh\left(\frac{L}{2a}\right)\sinh\left(\frac{z}{a}\right)\right]$$

For Loadings #3 and #3A above, use DG 9 Case 3 – Concentrated torque at αL.

for $0 \le z \le \alpha L$

$$\theta_T = \frac{TL}{GJ}\left[(1.0-\alpha)\frac{z}{L} + \frac{a}{L}\left(\frac{\sinh\left(\frac{\alpha L}{a}\right)}{\tanh\left(\frac{L}{a}\right)} - \cosh\left(\frac{\alpha L}{a}\right)\right)\sinh\left(\frac{z}{a}\right)\right]$$

and

$$\theta_T'' = \frac{T}{aGJ}\left[\left(\frac{\sinh\left(\frac{\alpha L}{a}\right)}{\tanh\left(\frac{L}{a}\right)} - \cosh\left(\frac{\alpha L}{a}\right)\right)\sinh\left(\frac{z}{a}\right)\right]$$

Note that the reduction factor, R, defined in *Eq*. C3.6-1, is a ratio of calculated stresses. These calculated stresses are directly proportional to the value of the applied uniform load. Thus a load of any magnitude can be used to calculate R. In this example, a load of $w = 10$ pounds/foot is used.

1. Mid-Span Bracing

For mid-span bracing, the stresses are maximum at mid-span. Stresses are calculated using full unreduced section properties in accordance with *Specification* Section C3.6. Combine Loadings #1, #2 and #3.

Loading #1 - Simple bending through the shear center

$$f_b = \frac{Mc}{I_x}$$

$$M = \frac{wL^2}{8} = \frac{10(25)^2(12)}{8(1000)} = 9.38 \text{ kip-in.}$$

Stresses at top flange points A, B and C are all compression stresses.

$$f_{bA} = -\frac{9.38(4.500 - 0.773)}{10.3} = -3.39 \text{ ksi}$$

$$f_{bB} = f_{bC} = -\frac{9.38(4.500)}{10.3} = -4.10 \text{ ksi}$$

Loading #2 - Uniformly distributed torque – use DG 9 Case 4.

$$\theta_t = \frac{t_r a^2}{GJ}\left[\frac{L^2}{2a^2}\left(\frac{z}{L} - \frac{z^2}{L^2}\right) + \cosh\left(\frac{z}{a}\right) - \tanh\left(\frac{L}{2a}\right)\sinh\left(\frac{z}{a}\right) - 1.0\right]$$

where

$$t_r = \frac{10(1.05)}{12(1000)} = 0.000875 \text{ kip-in./in.}$$

$$a = \sqrt{\frac{29500(11.9)}{11300(0.00102)}} = 175 \text{ in.}$$

$L = 25(12) = 300$ in. $\quad$ $L/a = 300/175 = 1.71$

$z = 150$ in. $\quad$ $z/L = 150/300 = 0.500$ $\quad$ $z/a = 150/175 = 0.857$

$$\theta_t = \frac{0.000875(175)^2}{11300(0.00102)}\left[\begin{array}{l}\frac{(1.71)^2}{2}\left(0.500-(0.500)^2\right)+\cosh(0.857)\\ -\tanh\left(\frac{1.71}{2}\right)\sinh(0.857)-1.0\end{array}\right]$$

= 0.199 radians

$$\theta_t'' = \frac{0.000875}{11300(0.00102)}\left[-1.0+\cosh(0.857)-\tanh\left(\frac{1.71}{2}\right)\sinh(0.857)\right]$$

= -21.2 x 10^{-6}

Loading #3 - Brace at Mid-Span - use DG 9 Case 3 with α = 0.5.

for $0 \le z \le \alpha L$

$$\theta_T = \frac{TL}{GJ}\left[(1.0-\alpha)\frac{z}{L}+\frac{a}{L}\left(\frac{\sinh\frac{\alpha L}{a}}{\tanh\frac{L}{a}}-\cosh\frac{\alpha L}{a}\right)\sinh\frac{z}{a}\right]$$

and

$$\theta_T'' = \frac{T}{aGJ}\left[\left(\frac{\sinh\frac{\alpha L}{a}}{\tanh\frac{L}{a}}-\cosh\frac{\alpha L}{a}\right)\sinh\frac{z}{a}\right]$$

Set T = 1.0 kip-in. to find the rotation per kip-in. at mid-span.

$$\theta_T = \frac{1.0(300)}{11300(0.00102)}\left[(1-0.5)0.5+\frac{1}{1.71}\left(\frac{\sinh(0.5(1.71))}{\tanh(1.71)}-\cosh(0.5(1.71))\right)\sinh(0.857)\right]$$

= 1.21 radians

$$\theta_T'' = \frac{1.0}{175(11300)(0.00102)}\left[\left(\frac{\sinh(0.5(1.71))}{\tanh(1.71)}-\cosh(0.5(1.71))\right)\sinh(0.857)\right]$$

= -172 x 10^{-6}

Calculate the required value of torque provided by mid-span brace to prevent rotation at mid-span.

$$\theta = \theta_t + T_1\theta_T = 0.199 + T_1(1.21) = 0$$

T_1 = -0.164 kip-in.

Using this brace force, combine the calculated values for θ'' from each loading to obtain θ'' for the mid-span braced condition.

$$\theta'' = \theta_t'' + T_1\theta_T'' = -21.2\times10^{-6} - 0.164\left(-172\times10^{-6}\right) = 7.01\times10^{-6}$$

The torsional warping normal stresses are:

$$f_w = EW_n\theta'' = 29500W_n\left(7.01\times10^{-6}\right) = 0.207W_n$$

$$f_{wA} = 0.207(-8.82) = -1.83\text{ ksi}$$

$$f_{wB} = 0.207(-6.22) = -1.29\text{ ksi}$$

$$f_{wC} = 0.207(4.69) = 0.971\text{ ksi}$$

Determine the location of the maximum combined flexural and warping normal stress.

$$f_A = f_{bA} + f_{wA} = -3.39 - 1.83 = -5.22\text{ ksi}$$

$f_B = f_{bB} + f_{wB} = -4.10 - 1.29 = -5.39$ ksi CONTROLS

$$f_C = f_{bC} + f_{wC} = -4.10 + 0.971 = -3.13 \text{ ksi}$$

Calculate the reduction factor.

$$R = \frac{f_{bending\_max}}{f_{bending} + f_{torsion}} \qquad (Eq.\ C3.6\text{-}1)$$

$$= \frac{-4.10}{-4.10 - 1.29} = 0.761$$

Note that this value occurs at the intersection of the flange and stiffener; therefore, no increase is permitted.

Calculate the nominal yielding strength.

$$M_n = RS_eF_y$$

$$= (0.761)(1.89)(55) = 79.1 \text{ kip-in.}$$

Allowable uniform load using safety factor of $\Omega_b = 1.67$

$M_n / \Omega_b = 79.1 / 1.67 = 47.4$ kip-in.

$M = w L^2 / 8 = w\, 25^2 / 8 = 78.1w$ kip-ft or $937\, w$ kip-in.

Equating $47.4 = 937w$ provides $w = 0.0506$ kip/ft

Other applicable limit states should also be evaluated (not shown).

2. Third-Point Bracing

For this condition, stresses are calculated at both the third-points and at mid-span, since it is not obvious by inspection which location will govern. Superimpose the stresses from Loadings #1, #2 and #3A. Use DG 9 Case 3 to calculate θ and θ'' at these points due to the torsional restraint provided by the braces. The value of the torque at the brace points is calculated by requiring that the value of θ be zero at these two points. Note by symmetry, the torques at the braces are equal.

Loading #1 - Simple bending through the shear center

Flexural stresses mid-span are the same as previously calculated. Those at the third-points are:

$$M = \frac{wL^2}{9} = \frac{10(25)^2\,12}{9(1000)} = 8.33 \text{ kip-in.}$$

$$f_{bA} = -\frac{8.33}{10.3}(4.500 - 0.773) = -3.01 \text{ ksi}$$

$$f_{bB} = f_{bC} = -\frac{8.33}{10.3}(4.500) = -3.64 \text{ ksi}$$

Loading #2 - Uniformly Distributed Torque – Use DG 9 Case 4

Values at mid-span are as previously calculated. Those at third-points are:

$z = L/3 = 100$ in. $z/L = 0.333$ $\qquad z/a = 0.571$

$$\theta_{t1/3} = \frac{0.000875(175)^2}{11300(0.00102)}\left[\begin{array}{l}\dfrac{(1.71)^2}{2}\left(0.333 - (0.333)^2\right) + \cosh(0.571) - \\ \tanh\left(\dfrac{1.71}{2}\right)\sinh(0.571) - 1.0\end{array}\right]$$

$= 0.173$ radians

By symmetry, rotation at the $2/3$ point is equal to the rotation at the $1/3$ point: $\theta_{t1/3} = \theta_{t2/3}$.

$$\theta''_{t1/3} = \frac{0.000875}{11300(0.00102)}\left[-1.0 + \cosh(0.571) - \tanh\left(\frac{1.71}{2}\right)\sinh(0.571)\right]$$

$= -19.0 \times 10^{-6}$

By symmetry, θ'' at the 2/3 point is equal to θ'' at the 1/3 point: $\theta''_{t2/3} = \theta''_{t1/3}$

Loading #3A - Braces at third-points – Use DG 9 Case 3 with $\alpha = 0.667$

Apply the brace torque at 2/3 point and calculate θ_T and θ''_T at z = L/3, z = L/2 and z = 2L/3.

For z = L/3 = 100 in. and α = 0.667

$$\theta_{T1/3} = \frac{1.0(300)}{11300(0.00102)}\left[(1-0.667)0.333+\frac{1}{1.71}\left(\frac{\frac{\sinh(0.667(1.71))}{\tanh(1.71)}-}{\cosh(0.667(1.71))}\right)\sinh(0.571)\right]$$

= 0.826 radian

$$\theta''_{T1/3} = \frac{1.0}{175(11300)(0.00102)}\left[\left(\frac{\sinh(0.667(1.71))}{\tanh(1.71)}-\cosh(0.667(1.71))\right)\sinh(0.571)\right]$$

$= -67.1 \times 10^{-6}$

For z = L/2 = 150 in.

$$\theta_{T1/2} = \frac{1.0(300)}{11300(0.00102)}\left[(1-0.667)0.500+\frac{1}{1.71}\left(\frac{\frac{\sinh(0.667(1.71))}{\tanh(1.71)}-}{\cosh(0.667(1.71))}\right)\sinh(0.857)\right]$$

= 1.03 radian

$$\theta''_{T1/2} = \frac{1.0}{175(11300)(0.00102)}\left[\left(\frac{\sinh(0.667(1.71))}{\tanh(1.71)}-\cosh(0.667(1.71))\right)\sinh(0.857)\right]$$

$= -108 \times 10^{-6}$

For z = 2L/3 = 200 in.

$$\theta_{T2/3} = \frac{1.0(300)}{11300(0.00102)}\left[(1-0.667)0.667+\frac{1}{1.71}\left(\frac{\frac{\sinh(0.667(1.71))}{\tanh(1.71)}-}{\cosh(0.667(1.71))}\right)\sinh(1.14)\right]$$

= 0.982 radian

$$\theta''_{T2/3} = \frac{1.0}{175(11300)(0.00102)}\left[\left(\frac{\sinh(0.667(1.71))}{\tanh(1.71)}-\cosh(0.667(1.71))\right)\sinh(1.14)\right]$$

$= -156 \times 10^{-6}$

Calculate the value of the torques at third-points required to prevent rotation at those brace points.

$\theta_{1/3} = \theta_{t1/3} + T_2\theta_{T1/3} + T_2\theta_{T2/3} = 0.173 + 0.826T_2 + 0.982T_2 = 0$

$T_2 = -0.0957$ kip-in.

Calculate torsional warping normal stresses at the 1/3 and 2/3 points.

At z = L/3

$$\theta''_{1/3} = \theta''_{t1/3} + T_2\theta''_{T1/3} + T_2\theta''_{T2/3}$$
$$= -19.0 \times 10^{-6} + (-0.0957)(-67.1 \times 10^{-6}) + (-0.0957)(-156 \times 10^{-6})$$
$$= 2.35 \times 10^{-6}$$

$$f_w = 29500W_n(2.35 \times 10^{-6}) = 0.0693W_n$$

$$f_{wA} = 0.0693(-8.82) = -0.611 \text{ ksi}$$
$$f_{wB} = 0.0693(-6.22) = -0.431 \text{ ksi}$$
$$f_{wC} = 0.0693(4.69) = 0.325 \text{ ksi}$$

Determine the location of the maximum combined flexural and warping normal stress.

$$f_A = f_{bA} + f_{wA} = -3.01 - 0.611 = -3.62 \text{ ksi}$$
$$f_B = f_{bB} + f_{wB} = -3.64 - 0.431 = -4.07 \text{ ksi} \quad \text{CONTROLS}$$
$$f_C = f_{bC} + f_{wC} = -3.64 + 0.325 = -3.32 \text{ ksi}$$

Calculate the reduction factor at the 1/3 and 2/3 points.

$$R = \frac{-3.64}{-3.64 - 0.431} = 0.894 \qquad (Eq.\ C3.6\text{-}1)$$

Calculate the nominal yielding strength at the 1/3 and 2/3 points.

$$M_n = RS_eF_y$$
$$= (0.894)(1.89)(55) = 92.9 \text{ kip-in.}$$

Calculate torsional warping normal stresses at mid-span.

At z = L/2

$$\theta''_{1/2} = \theta''_{t1/2} + 2T_2\theta''_{T1/2} = -21.2 \times 10^{-6} + 2(-0.0957)(-108 \times 10^{-6})$$
$$= -0.529 \times 10^{-6}$$

$$f_w = 29500W_n(-0.529 \times 10^{-6}) = -0.0156W_n$$

$$f_{wA} = -0.0156(-8.82) = 0.138 \text{ ksi}$$
$$f_{wB} = -0.0156(-6.22) = 0.0970 \text{ ksi}$$
$$f_{wC} = -0.0156(4.69) = -0.0732 \text{ ksi}$$

Determine the location of the maximum combined flexural and warping normal stress.

$$f_A = f_{bA} + f_{wA} = -3.39 + 0.138 = -3.25 \text{ ksi}$$
$$f_B = f_{bB} + f_{wB} = -4.10 + 0.0970 = -4.00 \text{ ksi}$$
$$f_C = f_{bC} + f_{wC} = -4.10 - 0.0732 = -4.17 \text{ ksi} \quad \text{CONTROLS}$$

Calculate the reduction factor at mid-span.

$$R = (1.15)\frac{-4.10}{-4.10 - 0.0732} = 1.13 > 1.0 \qquad (Eq.\ C3.6\text{-}1)$$

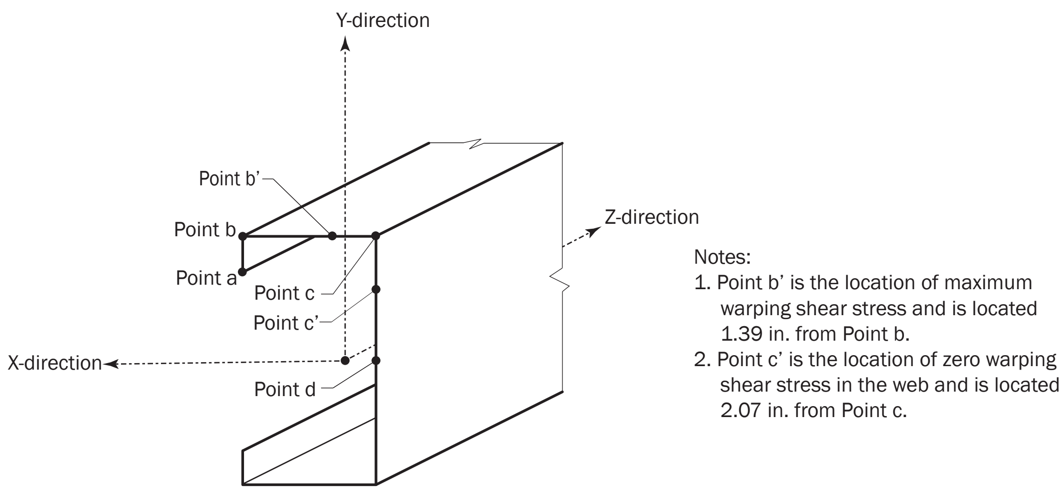
Since R exceeds 1.0, take R as 1.0 at mid-span. The 15% increase is permitted since the maximum combined stress occurs at the junction of the flange and web.

Calculate the nominal yielding strength at mid-span.

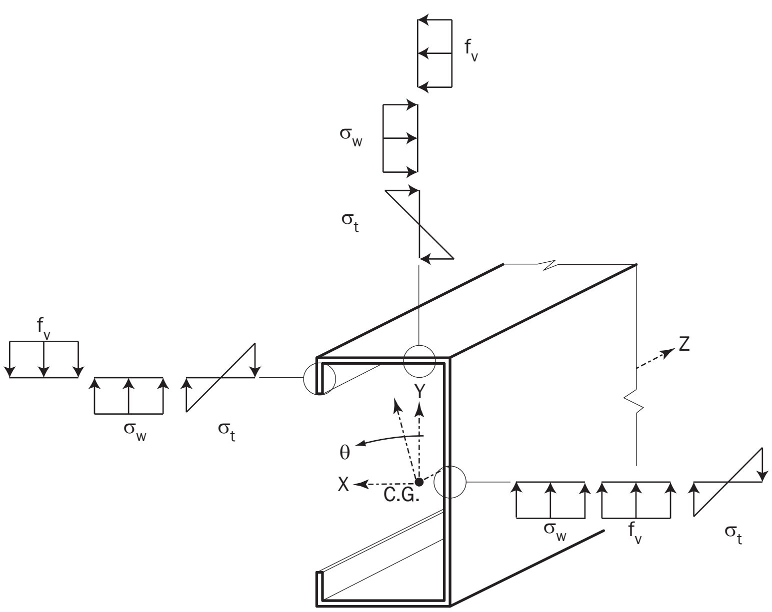
$$M_n = RS_eF_y$$
$$= (1.0)(1.89)(55) = 104 \text{ kip-in.}$$

3. Shear Stresses, Including the Effects of Torsion for the Condition of Mid-Span Bracing

The key positions for shear stresses caused by both the torsion and bending are shown in the figure below. The directions and distributions are shown in the next figure. Directions are all positive. The figure represents the face of cut at distance "z" from the left support looking in the positive "z" direction. From the calculations above of 1. Mid-Span Bracing, the allowable distributed load w = 0.0506 kip/ft and corresponding mid-span brace force = -0.164 kip-in.



Stress Location Points



Positive Rotation and Shear Stresses

Flexural shear stresses, f_v

These are calculated in the same manner as a beam loaded in bending alone.

$$f_v = \frac{VQ}{I_x t}$$

where:

f_v = flexural shear stress, ksi
V = the total shear for applied loading at point "z" along the beam, kips
Q = static moment of area outside the point at which the shear is calculated, $in.^3$
I = overall moment of inertia of the cross-section, $in.^4$
t = thickness of the cross section where stress is calculated, in.

These stresses vary over the cross-section depending on the cross sectional property, "Q". At any point, the stresses are constant across the thickness of the member and parallel to the sides of the element.

Shear, V, along span:

z = 0 $\quad V = -0.633$ kip
z= 8.33 ft $\quad V = -0.211$ kip
z = 12.5 ft $\quad V = 0$

$$\frac{Q}{I_x t} = \sum \frac{L(\bar{y})}{I_x}:$$

Pt. a: $\frac{Q}{I_x t} = 0.0$

Pt. b: $\frac{Q}{I_x t} = \left(\frac{Q}{I_x t}\right)_a + \frac{\bar{c}(0.5\bar{a} - 0.5\bar{c})}{I_x} = 0.0 + \frac{0.744[0.5(8.941) - 0.5(0.744)]}{10.3} = 0.296$

Pt. c: $\frac{Q}{I_x t} = \left(\frac{Q}{I_x t}\right)_b + \frac{\bar{b}(0.5)\bar{a}}{I_x} = 0.296 + \frac{2.441(0.5)(8.941)}{10.3} = 1.36$

Pt. d: $\frac{Q}{I_x t} = \left(\frac{Q}{I_x t}\right)_c + \frac{(0.5\bar{a})(0.25\bar{a})}{I_x} = 1.36 + \frac{(0.5)(8.941)(0.25)(8.941)}{10.3} = 2.33$

Using these values in above formula yields

Flexural Shear Stresses (ksi)			
Point	Location z (in.)		
	0.0	100	150
a	0.00	0.00	0.00
b	-0.19	-0.06	0.00
c	-0.86	-0.29	0.00
d	-1.47	-0.49	0.00

Torsional warping shear stresses, σ_w.

These are calculated from the formula:

$$\sigma_w = \frac{-E\, S_w \theta'''}{t} \quad \text{(AISC Design Guide 9, \textit{Eq}. 4-2a)}$$

where:

σ_w = the torsional warping shear stress, ksi

E = modulus of elasticity of material, ksi

S_w = warping statical moment at the point on the member where shear stress is calculated, in.^{4}

θ''' = third derivative of the angle of rotation at the point z along the beam,

t = thickness of the member where stress is calculated, in

These stresses are constant across the thickness of the member.

Calculation of warping shear stresses requires the section properties, S_w, warping statical moment at the point of member where stress is calculated. These values can be calculated by summation formulas*. The calculations are not shown here. For this section:

Point	S_w (in.^{4})
a	0.000
b	-0.330
b′	-0.586
c	-0.440
c′	0.000
d	0.179

The torsional functions required in the stress equations can be obtained from the first part of this example. Load #2 provides rotational functions for an applied torque of

t_r = 0.000875 kip-in/in. θ_t = 0.199 radians

For this case, the applied torsional moment is

t_r = 0.0506 x 1.05/12 = 0.00443 kip-in./in.

Hence the rotation at mid-span for the eccentric distributed load is based on the allowable distributed load, w, per Part 1:

θ_t = (0.00443/0.000875) (0.199) = 1.01 radians

Likewise the value of the rotation at mid-span for concentrated torque of T = 1.0, using Load #3, is 1.21 radians.

Hence the value of T to prevent any rotation at mid-span is:

$\theta = 1.01 + 1.21\ T = 0$

solving for T: T = -0.835 kip-in.

* Galambos, T.V., *Structural Members and Frames*, Prentice-Hall, Englewood Cliffs, NJ, 1968

Combining the two loadings provides a formula for θ,

$$\theta = \begin{Bmatrix} 0.00443\left(\frac{a^2}{GJ}\right)\left[\left(\frac{L^2}{2a^2}\right)\left(\frac{z}{L}-\frac{z^2}{L^2}\right)+\cosh\left(\frac{z}{a}\right)-\tanh\left(\frac{L}{2a}\right)\sinh\left(\frac{z}{a}\right)-1.0\right]+ \\ -0.835\left(\frac{L}{GJ}\right)\left[(1-\alpha)\frac{z}{L}+\frac{a}{L}\left(\frac{\sinh\left(\frac{\alpha L}{a}\right)}{\tanh\left(\frac{L}{a}\right)}-\cosh\left(\frac{\alpha L}{a}\right)\right)\sinh\left(\frac{z}{a}\right)\right] \end{Bmatrix}$$

This expression is differentiated three times with respect to z to obtain the functions θ′ and θ‴.

$$\theta' = \begin{Bmatrix} 0.00443\left(\frac{a}{GJ}\right)\left[\left(\frac{L^2}{2a}\right)\left(\frac{1}{L}-\frac{2z}{L^2}\right)+\sinh\left(\frac{z}{a}\right)-\tanh\left(\frac{L}{2a}\right)\cosh\left(\frac{z}{a}\right)\right]+ \\ \left(\frac{-0.835}{GJ}\right)\left[(1-\alpha)+\left(\frac{\sinh\left(\frac{\alpha L}{a}\right)}{\tanh\left(\frac{L}{a}\right)}-\cosh\left(\frac{\alpha L}{a}\right)\right)\cosh\left(\frac{z}{a}\right)\right] \end{Bmatrix}$$

$$\theta'' = \begin{Bmatrix} \left(\frac{0.00443}{GJ}\right)\left[-1.0+\cosh\left(\frac{z}{a}\right)-\tanh\left(\frac{L}{2a}\right)\sinh\left(\frac{z}{a}\right)\right]+ \\ \left(\frac{-0.835}{aGJ}\right)\left[\left(\frac{\sinh\left(\frac{\alpha L}{a}\right)}{\tanh\left(\frac{L}{a}\right)}-\cosh\left(\frac{\alpha L}{a}\right)\right)\sinh\left(\frac{z}{a}\right)\right] \end{Bmatrix}$$

$$\theta''' = \begin{Bmatrix} \left(\frac{0.00443}{aGJ}\right)\left[\sinh\left(\frac{z}{a}\right)-\tanh\left(\frac{L}{2a}\right)\cosh\left(\frac{z}{a}\right)\right]+ \\ \left(\frac{-0.835}{a^2GJ}\right)\left[\left(\frac{\sinh\left(\frac{\alpha L}{a}\right)}{\tanh\left(\frac{L}{a}\right)}-\cosh\left(\frac{\alpha L}{a}\right)\right)\cosh\left(\frac{z}{a}\right)\right] \end{Bmatrix}$$

Values of θ‴:

At support, z = 0

$$\theta''' = \begin{Bmatrix} \left(\frac{0.00443}{(175)(11300)(0.00102)}\right)\left[\sinh\left(\frac{0}{175}\right)-\tanh\left(\frac{300}{2(175)}\right)\cosh\left(\frac{0}{175}\right)\right]+ \\ \left(\frac{-0.835}{(175)^2(11300)(0.00102)}\right)\left[\left(\frac{\sinh\left(\frac{(0.5)(300)}{175}\right)}{\tanh\left(\frac{300}{175}\right)}-\cosh\left(\frac{(0.5)(300)}{175}\right)\right)\cosh\left(\frac{0}{175}\right)\right] \end{Bmatrix}$$

$\theta''' = -6.75 \times 10^{-7}$

At 1/3 span (z = 100 in.), $\theta''' = 5.36 \times 10^{-7}$ (Calculations not shown)
At mid-span (z = 150 in.), $\theta''' = 1.18 \times 10^{-6}$ (Calculations not shown)

Warping shear stresses, $\sigma_w = \dfrac{-E\,S_w\theta'''}{t} = \dfrac{-29500\,S_w\theta'''}{0.059}$

Warping Shear Stresses, σ_w (ksi)				
Point	**S_w (in.^{4})**	**Location z (in.)**		
		0.0	**100**	**150**
a	0.000	0.00	0.00	0.00
b	-0.330	-0.11	+0.09	+0.19
b′	-0.586	-0.20	+0.16	+0.35
c	-0.440	-0.15	+0.12	+0.26
c'	0.000	0.00	0.00	0.00
d	0.179	0.06	-0.05	-0.11

Pure torsional shear stress, σ_t.

These are calculated from the formula:

$\sigma_t = G\,t\,\theta'$

where:
σ_t is the pure torsional shear stress, ksi
G = shear modulus of elasticity of the material, ksi
t = thickness of the member where stress is calculated, in.
θ' = first derivative of the angle of rotation at the point z along the beam,

Values of θ':
At support:

$$\theta' = \left\{ \begin{array}{l} 0.00443\left(\dfrac{175}{11300(0.00102)}\right)\left[\left(\dfrac{300^2}{2(175)}\right)\left(\dfrac{1}{300} - \dfrac{2(0)}{300^2}\right) + \sinh\left(\dfrac{0}{175}\right) - \tanh\left(\dfrac{300}{2(175)}\right)\cosh\left(\dfrac{0}{175}\right)\right] + \\ \left(\dfrac{-0.835}{11300(0.00102)}\right)\left[(1-0.5) + \left(\dfrac{\sinh\left(\dfrac{0.5(300)}{175}\right)}{\tanh\left(\dfrac{300}{175}\right)} - \cosh\left(\dfrac{0.5(300)}{175}\right)\right)\cosh\left(\dfrac{0}{175}\right)\right] \end{array} \right\}$$

$\theta' = 7.50 \times 10^{-4}$

At 1/3 span $\theta' = -5.93 \times 10^{-4}$ (Calculations not shown)
At mid-span $\theta' = 0$ (Calculations not shown)

Pure torsional shear stress, $\sigma_t = Gt\theta' = (11300)(0.059)\theta'$

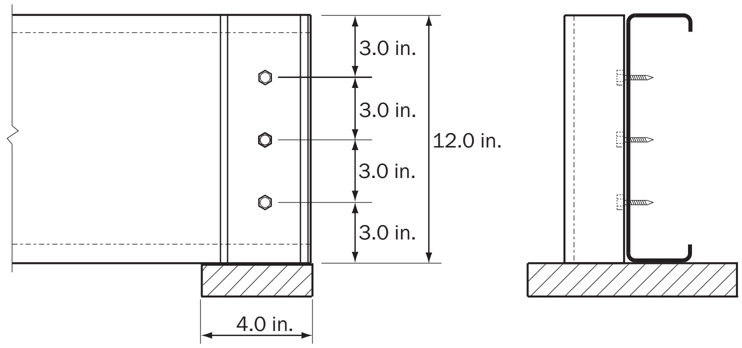
Pure Torsional Shear Stresses, σ_t (ksi)			
Point	Location z (in.)		
	0.0	100	150
a	0.500	-0.395	0.00
b	0.500	-0.395	0.00
b′	0.500	-0.395	0.00
c	0.500	-0.395	0.00
c′	0.500	-0.395	0.00
d	0.500	-0.395	0.00

These stresses are equal but opposite and maximum at the edges of the element; they are distributed linearly across the thickness of the member.

Observations

1. The largest calculated shear stress due to pure torsional shear stress is:
   σ_t = 0.500 ksi calculated at the support location.
2. The largest calculated shear stress due to member bending is:
   f_v = 1.47 ksi calculated at mid-web depth at the support location.
3. The largest calculated torsional warping shear stress:
   σ_w = 0.35 ksi calculated between the web and flange stiffener at mid-span.
4. The shear stresses are well below the bending stresses in this member.

Example II-12: Web Crippling



Given:

1. Flexural member: Stud 1200S200-68 (50 ksi)
2. Bearing stiffener: Stud 362S162-33 (33 ksi)

Required:

Calculate the available bearing strength of the joist section with the C-section bearing stiffener using both ASD and LRFD

Solution:

Calculate the available ASD and LRFD strength using Section C3.7.

Use Section C3.7.1 if the w/t_s limits for the stiffener are not exceeded.

1. Check Applicability Limits for Section C3.7.1

Check web of stiffener:

$$w/t_s = \frac{D-2(R+t_s)}{t_s}$$

$$= \frac{3.625-2(0.0765+0.0346)}{0.0346} = 98.3$$

$$\text{Limit} = 1.28\sqrt{E/F_y}$$

$$= 1.28\sqrt{29500/33} = 38.3 < 98.3 \quad \text{NG; therefore, try Section C3.7.2}$$

2. Check Applicability Limits for Section C3.7.2

(a) The stiffener has full bearing; therefore, use 100% of the calculated capacity. OK

(b) The stiffener is a C-section with a web depth of 3.625 in. > 3.5 in. minimum. The stiffener has a thickness of 0.0346 in. > 0.0329 in. minimum. OK

(c) The stiffener is attached to the flexural member with three screws. OK

(d) The distance from the flexural member flanges to the first fastener is d/4 > d/8 minimum. OK

(e) The length of the stiffener is equal to the depth of the flexural member. OK

(f) The bearing width is greater than 1 ½ in. OK

3. Calculate Nominal Strength, P_n, using Section C3.7.2

Calculate the nominal bearing strength.

$$P_n = 0.7\left(P_{wc} + A_e F_y\right) \ge P_{wc} \qquad (Eq.\ C3.7.2\text{-}1)$$

From Table II-14 for a 1200S200-68 (50 ksi), Fastened to the support, Case C, N = 4 in.

P_{wc} = 1.26 kips (flexural member)

From Table III-2 for a 362S162-33 (33 ksi)

P_n = 5.72 kips (stiffener) = $A_e F_y$

Nominal strength

$$P_n = 0.7\left(1.26 + 5.72\right) \ge 1.26 \text{ kips} \qquad (Eq.\ C3.7.2\text{-}1)$$

$= 4.89$ kips > 1.26 kips; therefore, use 4.89 kips

4. Available Strength

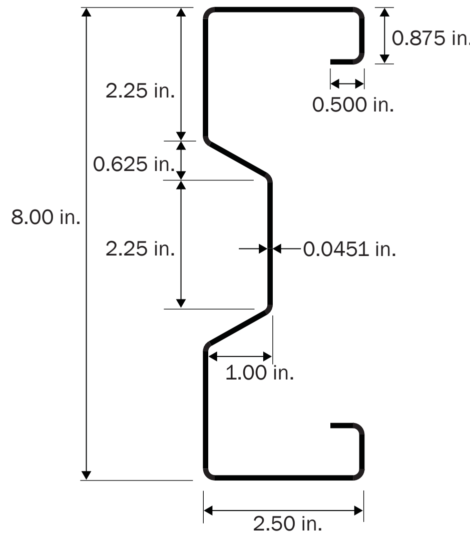
ASD - Allowable strength

$$\frac{P_n}{\Omega} = \frac{4.89}{1.70} = 2.88 \text{ kips} \qquad (Eq.\ A4.1.1\text{-}1)$$

LRFD - Design strength

$$\phi P_n = 0.90\left(4.89\right) = 4.40 \text{ kips} \qquad (Eq.\ A5.1.1\text{-}1)$$

Example II-13: Web-Stiffened C-Section by the Direct Strength Method – Flexure



Given:

1. Steel: F_y = 50 ksi, F_u = 65 ksi
2. Sigma section (C-Section with web stiffener) as shown above
3. The member is a simply supported flexural member fully braced against lateral-torsional buckling.

Required:

Calculate the ASD and LRFD available flexural strengths using the Direct Strength procedure from *Specification* Appendix 1

Solution:

Although the Direct Strength Method may be used for any cross-section, it is particularly well suited to this example, since the cross-section is somewhat complex and the *Specification* has no provisions for the complex edge stiffeners on the flanges.

1. Perform a Finite Strip Analysis

A finite strip analysis of the cross-section is performed using a program such as CUFSM*. A pure flexural stress distribution is assumed with the extreme fibers at F_y. Results from the analysis include the bending moment under the assumed stress distribution, M_y, and a graph of the section buckling strength versus unbraced length, shown below.

From the analysis:

Yield moment

M_y = 86.4 kip-in.

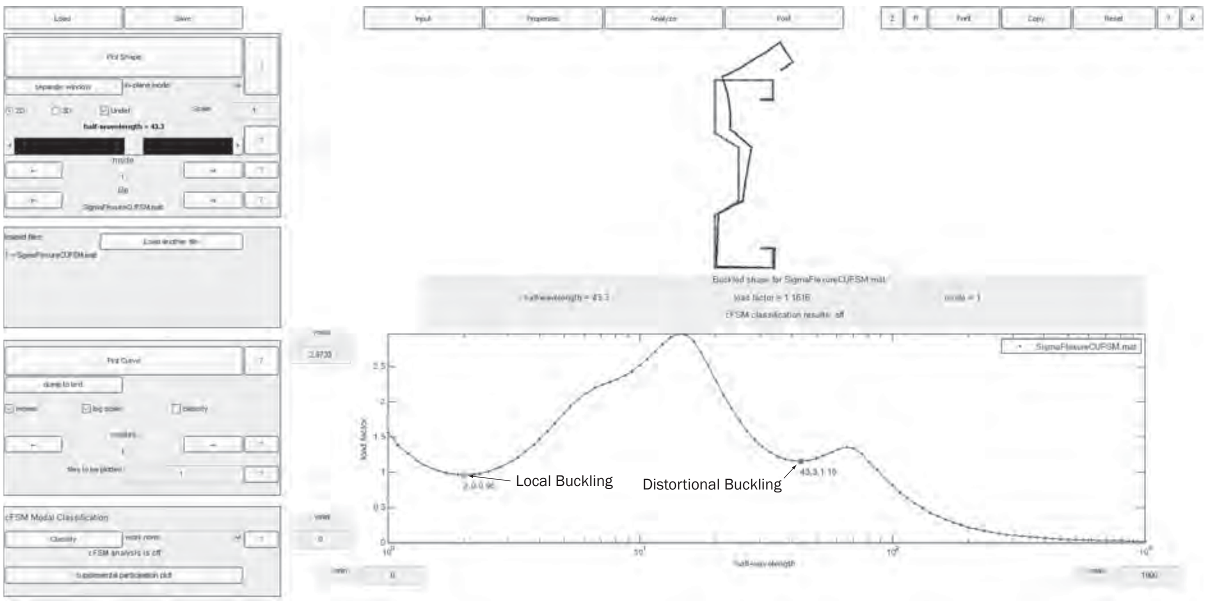
* Schafer, B.W., Ádány, S. "Buckling analysis of cold-formed steel members using CUFSM: conventional and constrained finite strip methods." Eighteenth International Specialty Conference on Cold-Formed Steel Structures, Orlando, FL. October 2006. Available at www.ce.jhu.edu/bschafer/cufsm

Critical elastic local buckling moment

$$M_{cr\ell} = 0.96M_y = (0.96)(86.4) = 82.9 \text{ kip-in.}$$

Critical elastic distortional buckling moment

$$M_{crd} = 1.16M_y = (1.16)(86.4) = 100 \text{ kip-in.}$$



2. Calculate the Nominal Flexural Strength

Per Section 1.2.2 of Appendix 1, take M_n as the lowest of the nominal strengths for lateral-torsional buckling, M_{ne}, local buckling, $M_{n\ell}$ and distortional buckling, M_{nd}.

1) Lateral-torsional buckling: In this case, since the member is fully braced against lateral-torsional buckling,

$$M_{ne} = M_y = 86.4 \text{ kip-in.} \qquad (Eq.\ 1.2.2\text{-}3)$$

2) Local buckling:

$$\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}} \qquad (Eq.\ 1.2.2\text{-}9)$$

$$= \sqrt{86.4/82.9} = 1.02$$

Since $\lambda_\ell > 0.776$,

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}8)$$

$$= \left(1 - 0.15\left(\frac{82.9}{86.4}\right)^{0.4}\right)\left(\frac{82.9}{86.4}\right)^{0.4} 86.4 = 72.4 \text{ kip-in.}$$

3) Distortional buckling:

$$\lambda_d = \sqrt{M_y/M_{crd}} \qquad (Eq.\ 1.2.2\text{-}19)$$

$$= \sqrt{86.4/100} = 0.93$$

Since $\lambda_d > 0.673$,

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}18)$$

$$= \left(1 - 0.22\left(\frac{100}{86.4}\right)^{0.5}\right)\left(\frac{100}{86.4}\right)^{0.5} 86.4 = 71.0\ \text{kip-in.}$$

4) The nominal flexural strength is therefore 71.0 kip-in., governed by distortional buckling.

3. Calculate the Available Strengths

Check the limitations for prequalified beams in Table 1.1.1-2 to determine the appropriate strength reduction factors. Since there is no prequalified category for C-sections with web stiffeners and complex lips, use the strength reduction factors from Section A1.2(c)

ASD - Allowable strength

$$\frac{M_n}{\Omega} = \frac{71.0}{2.00} = 35.5\ \text{kip-in.} \qquad (Eq.\ A4.1.1\text{-}1)$$

LRFD - Design strength

$$\phi M_n = 0.80(71.0) = 56.8\ \text{kip-in.} \qquad (Eq.\ A5.1.1\text{-}1)$$

Example II-14: Lipped C-Section With Web Perforations by the Direct Strength Method – Flexure

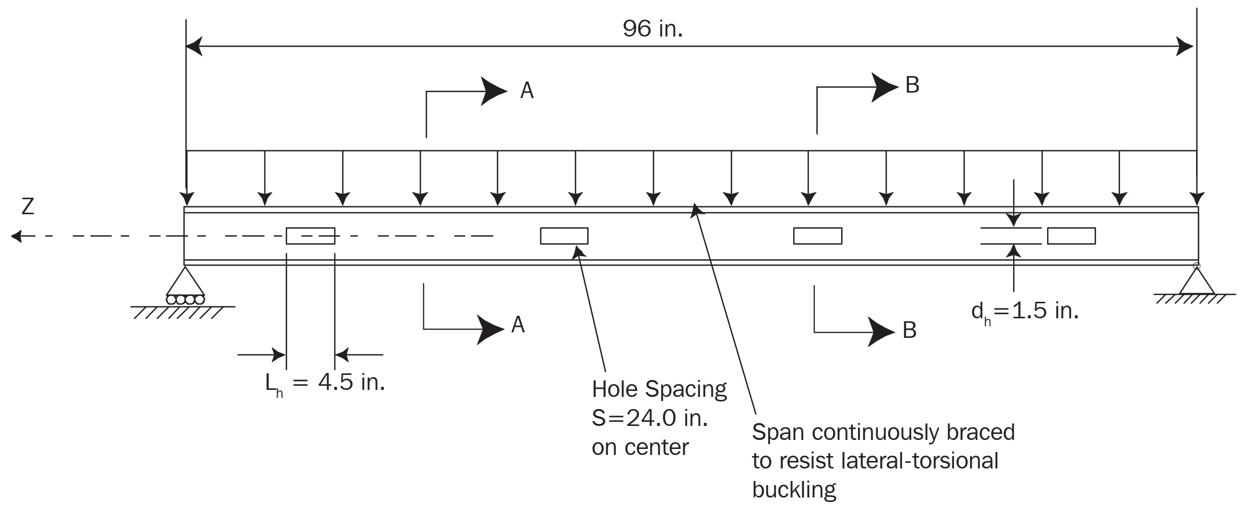


Figure 1 - Beam Dimensions and Boundary Conditions

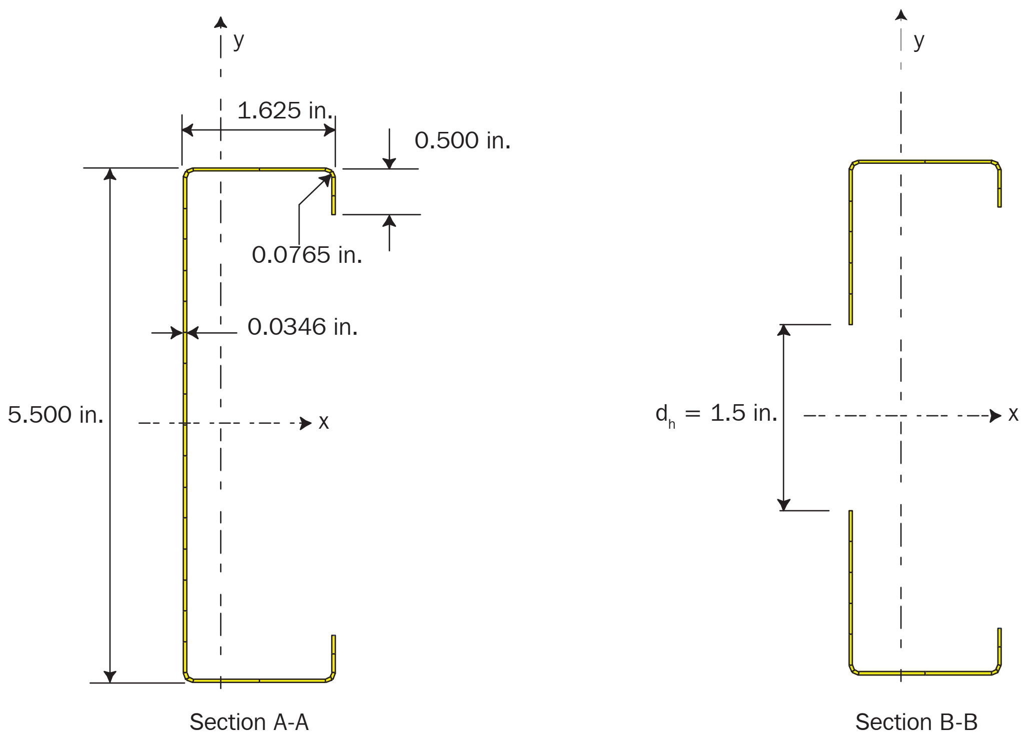


Figure 2 – Gross and Net Cross-Sections

Given:

1. Joist geometry and loading as shown in Figure 1. The cross-section is a 550S162-33 C-section with lips with F_y=33 ksi.
2. The joist carries a uniform vertical load and is assumed to be fully braced against lateral-torsional buckling. Distortional buckling and local buckling are viable strength limit states.
3. The joist has web perforations as shown in Figures 1 and 2.

Required
Determine the LRFD and ASD available buckling strengths.
Solution
The gross section and net yield strengths are calculated with the section property calculator in CUFSM. To determine the net section properties in CUFSM, assign a thickness of zero to the elements at the location of the perforations, but do not delete them. Assuming 33 ksi steel, M_y=18.0 kip-in. and M_{ynet}=17.8 kip-in.

1. Calculate the Elastic Local Buckling Strength

Local buckling in a cold-formed steel beam with holes is assumed to occur as either buckling in the gross cross-section between holes, $M_{cr\ell nh}$, or buckling of the compressed strip adjacent to a hole, $M_{cr\ell h}$.The buckled mode shape with the lowest critical buckling load defines the critical elastic local buckling strength, $M_{cr\ell}$.

The gross-section elastic buckling curve generated with CUFSM, (see Figure 3) is used to obtain $M_{cr\ell nh}$. Taking the first minimum on the elastic buckling curve, $M_{cr\ell nh}$ = 17.6 kip-in. at a half-wavelength $L_{cr\ell nh}$ = 3.0 in.

The net-section elastic buckling curve is generated in CUFSM by modifying the gross section node and element geometry such that one finite strip element with t = 0 spans across the hole (Figure 4). A reference moment of 1 kip-in. is applied to the cross-section and the CUFSM Properties screen is used to calculate the corresponding stress distribution. The zero thickness element is then deleted. The resulting mode shape and elastic buckling curve is provided in Figure 4. The lowest buckling load of the unstiffened strip occurs at a half-wavelength less than the length of the perforation ($L_{cr\ell h}$ = 3.80 in. versus L_{hole} = 4.5 in.) indicating that the buckled half-wave can form within the length of the hole, and therefore $M_{cr\ell h}$ = 9.60 kip-in.

Local buckling is predicted to occur in the net cross-section since $M_{cr\ell h} < M_{cr\ell nh}$ and therefore $M_{cr\ell}$ = 9.60 kip-in. The local elastic buckling moment is 45% lower at a hole, indicating that buckling will tend to occur as unstiffened strip buckling rather than in the web of the gross cross-section between holes.

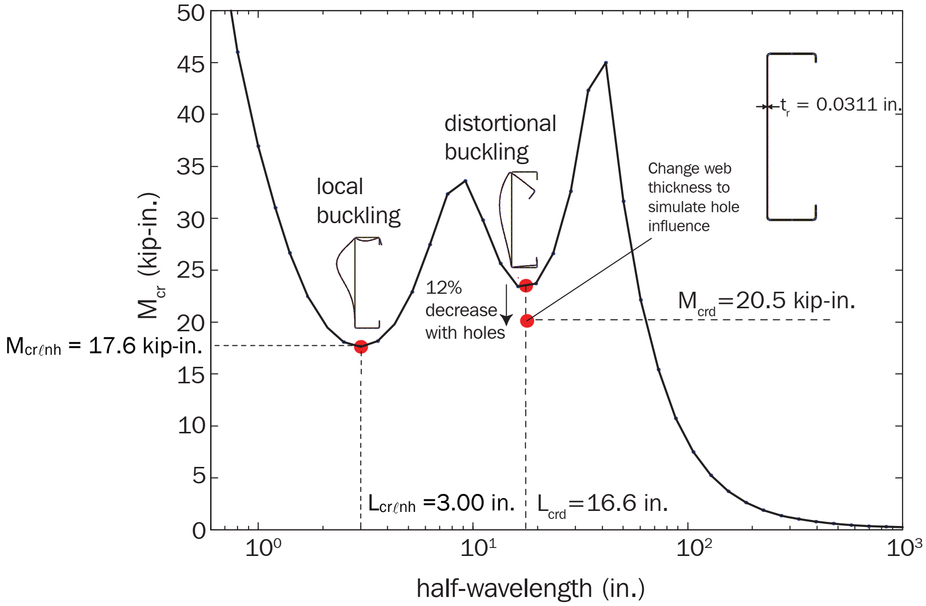


Figure 3 - Elastic Buckling Curve of Gross Cross-Section

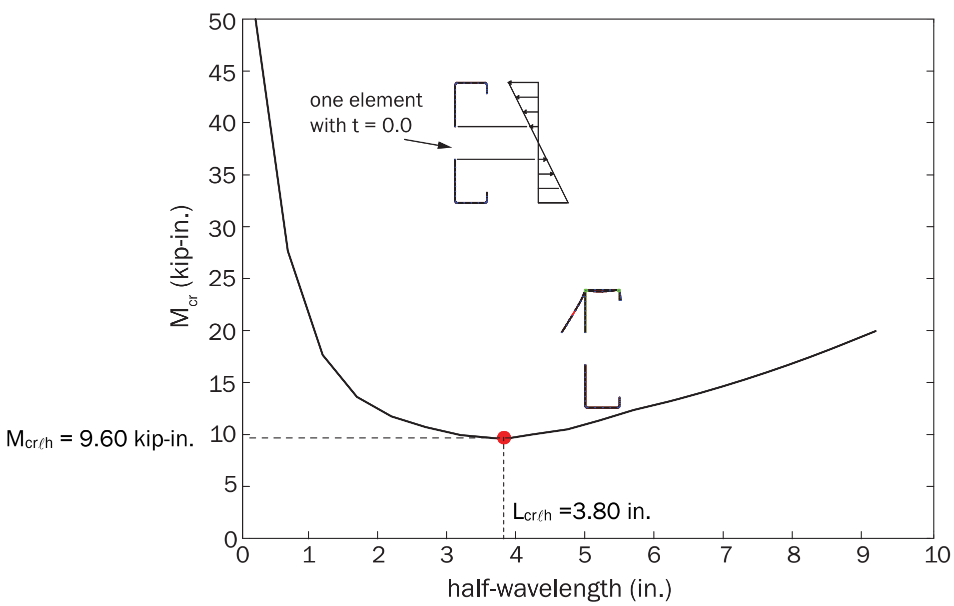


Figure 4 - Local Buckling Curve of Net Cross-Section

2. Calculate the Elastic Distortional Buckling Strength

The critical elastic buckling moment for distortional buckling, including the influence of web holes, is calculated by first obtaining the distortional buckling half-wavelength from a finite strip analysis of the gross cross-section (L_{crd}=16.6 in., see Figure 3). The reduced web bending stiffness caused by a hole over one distortional half-wavelength is simulated by reducing the cross-section thickness of the web to t_r.

$$t_r = \left(1 - \frac{L_h}{L_{crd}}\right)^{1/3} t \qquad (Eq.\ C\text{-}1.1.2\text{-}2)$$

$$= \left(1 - \frac{4.5}{16.6}\right)^{1/3} 0.0346 = 0.0311\,\text{in.}$$

For L_h=4.5 in. and t=0.0346 in., t_r=0.0311 in., which is then used in a second finite strip analysis performed just at L_{crd}= 16.6 in., resulting in M_{crd}=20.5 kip-in. The presence of perforations reduces M_{crd} by 12% when compared to the distortional buckling moment of 23.4 kip-in. for a beam without holes. The beneficial influence of the moment gradient on M_{crd} (Yu 2005)* is negligible and not considered because the beam's span length is much longer than L_{crd}.

3. Calculate the Available Strength

Inputs from the elastic buckling analysis include:

$M_y = 18.0$ kip-in.
$M_{ynet} = 17.8$ kip-in.

$M_{cr\ell} = 9.60$ kip-in.
$M_{crd} = 20.5 \cdot$ kip-in.

DSM nominal global buckling strength

$M_{ne} = M_y$ because beam is fully braced against lateral-torsional buckling (*Eq.* 1.2.2-3)

$M_{ne} = 18.0$ kip-in.

DSM nominal local buckling strength (local slenderness including influence of holes)

$$\lambda_\ell = \sqrt{\frac{M_{ne}}{M_{cr\ell}}} \qquad (Eq.\ 1.2.2\text{-}9)$$

$$= \sqrt{\frac{18.0}{9.60}} = 1.37$$

Because $\lambda_\ell > 0.776$:

* Yu, C. (2005). "Distortional buckling of cold-formed steel members in bending," Ph.D. Thesis, Johns Hopkins University, Baltimore.

$$M_{n\ell} = \left[1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right]\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}8)$$

$$= \left[1 - 0.15\left(\frac{9.60}{18.0}\right)^{0.4}\right]\left(\frac{9.60}{18.0}\right)^{0.4} 18.0 = 12.4\,\text{kip-in.}$$

DSM nominal distortional buckling strength (local slenderness including influence of holes)

$$\lambda_d = \sqrt{\frac{M_y}{M_{crd}}} \qquad (Eq.\ 1.2.2\text{-}22)$$

$$= \sqrt{\frac{18.0}{20.5}} = 0.937$$

$$\lambda_{d1} = 0.673\left(\frac{M_{ynet}}{M_y}\right)^3 \qquad (Eq.\ 1.2.2\text{-}27)$$

$$= 0.673\left(\frac{17.8}{18.0}\right)^3 = 0.651$$

$$\lambda_{d2} = 0.673\left[1.7\left(\frac{M_y}{M_{ynet}}\right)^{2.7} - 0.7\right] \qquad (Eq.\ 1.2.2\text{-}28)$$

$$= 0.673\left[1.7\left(\frac{18.0}{17.8}\right)^{2.7} - 0.7\right] = 0.708$$

Because $\lambda_d > \lambda_{d2}$

$$M_{nd} = \left[1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right]\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}18)$$

$$= \left[1 - 0.22\left(\frac{20.5}{18.0}\right)^{0.5}\right]\left(\frac{20.5}{18.0}\right)^{0.5} 18.0 = 14.7\,\text{kip-in.}$$

The nominal flexural strength (including holes) is taken as the least of the global, local and distortional buckling strengths.

$$M_n = \min\left(M_{ne}, M_{n\ell}, M_{nd}\right)$$
$$= \min\left(18.0, 12.4, 14.7\right) = 12.4\,\text{kip-in.}$$

Because the cross-section is pre-qualified per Section 1.1.1.2, use of resistance and safety factors from Section 1.2.2.1 is permitted.

LRFD – Design strength

$$\phi_b = 0.90$$
$$\phi_b M_n = 0.90(12.4) = 11.2\,\text{kip-in.}$$

ASD – Allowable strength

$$\Omega_b = 1.67$$

$$\frac{M_n}{\Omega_b} = \frac{12.4}{1.67} = 7.43 \text{ kip-in.}$$

Local buckling at a hole is predicted as the governing limit state, with a decrease in flexural strength of approximately 15% when compared to the same beam without holes.

Example II-15: C-Section With Lips: Shear Strength by the Direct Strength Method

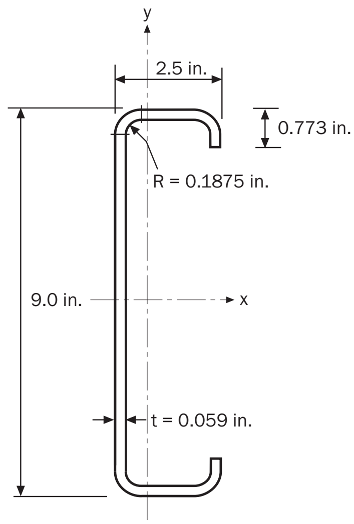


Figure 1 – Member Cross-Section

Given:

1. Geometry and loading as shown in Figure 1. The cross-section is a 9CS2.5x059 C-Section with lips with F_y=55 ksi.

Required

1. Determine the LRFD and ASD available shear strengths using the Direct Strength Method.
2. Check combined flexure and bending under LRFD conditions using M_u = 6.54 kip-ft and V_u = 1.78 kips from Example II-1.

Solution

1. Available Shear Strength

Performing a finite strip shear analysis on the section gives a critical elastic shear buckling load of 4.06 kips at a half-wavelength of approximately 9 in.

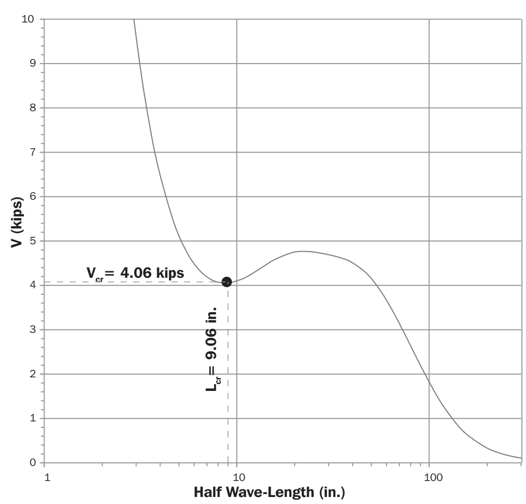


Figure 2 – Results of Finite Strip Shear Buckling Analysis

Compute the available shear strength using the provisions of Appendix 1, Section 1.2.2.2.1.

$$A_w = ht = 8.507(0.059) = 0.502\,\text{in.}^2 \qquad (Eq.\ 1.2.2\text{-}35)$$

$$V_y = 0.6A_wF_y = 0.6(0.502)(55) = 16.6\,\text{kips} \qquad (Eq.\ 1.2.2\text{-}34)$$

$$\lambda_v = \sqrt{\frac{V_y}{V_{cr}}} = \sqrt{\frac{16.6}{4.06}} = 2.02 \qquad (Eq.\ 1.2.2\text{-}33)$$

For $\lambda_v > 1.227$

$$V_n = V_{cr} = 4.06\,\text{kips} \qquad (Eq.\ 1.2.2\text{-}32)$$

LRFD – Design strength

$$\phi_v = 0.95$$

$$\phi_v V_n = 0.95(4.06) = 3.86\,\text{kips}$$

ASD – Allowable strength

$$\Omega_v = 1.60$$

$$\frac{V_n}{\Omega_v} = \frac{4.06}{1.60} = 2.54\,\text{kips}$$

Compared with the design strength for this cross-section calculated per Section C3.2.1 of the *Specification*, as illustrated in Example II-1, the shear strength calculated by the Direct Strength Method is 18% higher.

2. Combined Bending and Shear Strength

Check the combined bending and shear strength using the provisions of Appendix 1, Section 1.2.2.3.

Required strengths from Example II-1 at the end of the left lap of the interior span:

M_u = 6.54 kip-ft = 78.5 kip-in.
V_u = 1.78 kips

Nominal strengths:

Bending:

$$M_{ne} = M_y = F_yS_f$$
$$= 55(2.29) = 126\,\text{kip-in.}$$

From a finite strip analysis, $M_{cr\ell}$ = 85.5 kip-in.

$$\lambda_\ell = \sqrt{\frac{M_{ne}}{M_{cr\ell}}} = \sqrt{\frac{126}{85.5}} = 1.214 > 0.776 \qquad (Eq.\ 1.2.2\text{-}9)$$

$$M_{n\ell o} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}8)$$

$$= \left(1 - 0.15\left(\frac{85.5}{126}\right)^{0.4}\right)\left(\frac{85.5}{126}\right)^{0.4} 126 = 94.0\,\text{kip-in.}$$

$$\phi_b = 0.90$$

Shear

V_n = 4.06 kips from above

$\phi_v = 0.95$

Combined strength:

$$\sqrt{\left(\frac{\overline{M}}{\phi_b M_{n\ell o}}\right)^2 + \left(\frac{\overline{V}}{\phi_v V_n}\right)^2} \leq 1.0 \qquad \text{(from } \textit{Eq}\text{. C3.3.2-1)}$$

$$\sqrt{\left(\frac{78.5}{0.90(94.0)}\right)^2 + \left(\frac{1.78}{0.95(4.06)}\right)^2} = 1.04 > 1.0 \quad \text{NG}$$

The Direct Strength Method provisions are more conservative than the effective width provisions in this example.

Example II-16: C-Section With Lips: Inelastic Reserve Strength by the Direct Strength Method

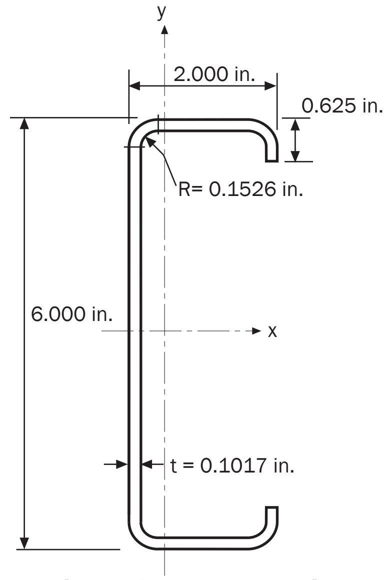


Figure 1 – Cross-Section

Given:

1. Geometry and loading as shown in Figure 1. The cross-section is a 600S200-97 C-Section with lips with F_y=50 ksi.
2. The member is fully braced against lateral torsional buckling and unbraced against distortional buckling
3. Properties and strengths from finite strip analysis:
   $M_{cr\ell} = 448$ kip-in.
   M_{crd} = 259 kip-in.
   S_x = 1.87 in.^{3}

Required

1. Determine the LRFD and ASD available flexural strengths using the Direct Strength Method.

Solution

Yield strength:

$$M_y = S_x F_y = 1.87(50) = 93.5\,\text{kip-in.}$$

Plastic yield strength:

$$Z_x = 2\left[\begin{array}{l} \left(\dfrac{h_o}{2} - R_o\right) t \dfrac{\left(\dfrac{h_o}{2}\right) - R_o}{2} + \\ (b_o - 2R_o) t \left(\dfrac{h_o - t}{2}\right) + \\ (D - R_o) t \left(\dfrac{h_o}{2} - R_o - \dfrac{D - R_o}{2}\right) \\ +\pi\left(R_o - \dfrac{t}{2}\right) t \left(\dfrac{h_o}{2} - R_o + 0.637\left(R_o - \dfrac{t}{2}\right)\right) \end{array}\right]$$

h_o = 6.0 in.
b_o = 2.0 in.

D = 0.625 in.
R = 0.1526 in.
t = 0.1017 in.
$R_o = R + t$ = 0.1526 + 0.1017 = 0.2543 in.

$$Z_x = 2\left[\begin{array}{l} \left(\dfrac{6.0}{2} - 0.2543\right)(0.1017)\dfrac{\left(\dfrac{6.0}{2}\right) - 0.2543}{2} + \\ \left[2.0 - 2(0.2543)\right](0.1017)\left(\dfrac{6.0 - 0.1017}{2}\right) + \\ (0.625 - 0.2543)(0.1017)\left(\dfrac{6.0}{2} - 0.2543 - \dfrac{0.625 - 0.2543}{2}\right) \\ +\pi\left(0.2543 - \dfrac{0.1017}{2}\right)(0.1017)\left(\dfrac{6.0}{2} - 0.2543 + 0.637\left(0.2543 - \dfrac{0.1017}{2}\right)\right) \end{array}\right]$$
$$= 2(0.3834 + 0.4473 + 0.0965 + 0.1806) = 2.22\,\text{in.}^3$$

$$M_p = Z_x F_y = 2.22(50) = 111\,\text{kip-in.}$$

Lateral-torsional buckling:
Assuming continuous lateral bracing, per *Eq*. 1.2.2-5, the upper limit of M_{ne} is
$M_{ne} = M_p$ = 111 kip-in.

Local buckling:
$\lambda_\ell = \sqrt{M_y/M_{cr\ell}} = \sqrt{93.5/448} = 0.457$ (*Eq*. 1.2.2-12)
Since $\lambda_\ell \le 0.776$ and $M_{ne} \ge M_y$, Section 1.2.2.1.2.1.2 is applicable
$C_{y\ell} = \sqrt{0.776/\lambda_\ell} = \sqrt{0.776/0.457} = 1.30 \le 3$ OK (*Eq*. 1.2.2-13)
For symmetric section:
$M_{n\ell} = M_y + \left(1 - 1/C_{y\ell}^2\right)\left(M_p - M_y\right)$ (*Eq*. 1.2.2-10)
$= 93.5 + \left(1 - 1/1.30^2\right)(111 - 93.5) = 101\,\text{kip-in.}$

Distortional buckling:
$\lambda_d = \sqrt{M_y/M_{crd}} = \sqrt{93.5/259} = 0.601$ (*Eq*. 1.2.2-22)
Since $\lambda_d \le 0.673$
$C_{yd} = \sqrt{0.673/\lambda_d} = \sqrt{0.673/0.601} = 1.06 \le 3$ OK (*Eq*. 1.2.2-23)
$M_{nd} = M_y + \left(1 - 1/C_{yd}^2\right)\left(M_p - M_y\right)$ (*Eq*. 1.2.2-20)
$= 93.5 + \left(1 - 1/1.06^2\right)(111 - 93.5) = 95.4\,\text{kip-in.}$

Nominal flexural strength
$$M_n = \min\left(M_{ne}, M_{n\ell}, M_{nd}\right) = \min(111, 101, 95.4)$$
$$= 95.4\,\text{kip-in.}$$

Available strengths

The cross-section is prequalified per Section 1.1.1.2; therefore,

LRFD – Design strength

$$\phi_b = 0.90$$
$$\phi_b M_n = 0.90(95.4) = 85.9 \text{ kip-in.}$$

ASD – Allowable strength

$$\Omega_b = 1.67$$
$$\frac{M_n}{\Omega_b} = \frac{95.4}{1.67} = 57.1 \text{ kip-in.}$$

TABLE OF CONTENTS

PART III
COLUMN DESIGN

For Use With the
2012 Edition of the
North American Specification for the Design
of Cold-Formed Steel Structural Members

PART III - COLUMN DESIGN

The design of cold-formed steel columns requires the consideration of the limit states of:

1. Combined overall member buckling and local buckling, and
2. Distortional buckling

Specification Section C4 includes provisions for the evaluation of these limit states. For columns that are parts of certain structural systems, Section D6 includes provisions that supersede some of the general provisions of Section C4.

Overall and Local Buckling: The strength of all columns is limited by the combined limit state of global and local buckling, which is evaluated using Section C4.1. Although the specifics vary somewhat for different cross-section shapes, the general procedure involves 1) determination of the elastic column buckling stress, 2) transformation of the elastic buckling stress to a critical buckling stress, taking into account the effects of inelasticity and 3) determination of the effective area with the section at the critical buckling stress. See *Manual* Part I, Section 3.6 and Examples I-8 and I-10 through I-13 for further information on the calculation of effective area of compression members.

The elastic buckling stress is taken as the lowest of the applicable buckling stresses for flexural (Euler) buckling, torsional buckling and flexural-torsional buckling. All cross-sections are subject to flexural buckling about their principal axes, per Section C4.1.1.

All doubly-symmetric sections are subject to torsional buckling and most singly-symmetric sections, such as C-Sections, are subject to flexural-torsional buckling per Section C4.1.2. Unlipped singly-symmetric angles having fully effective areas, A_e, at a stress of F_y are exempt from the flexural-torsional provisions and designed based on flexural buckling about the principal axis. Point-symmetric sections, such as Z-Sections, are subject torsional and flexural buckling per Section C4.1.3.

Section D6 of the *Specification* provides specialized provisions for the flexural-torsional buckling of compression members that are elements of metal roof and wall systems, including through-fastened purlins and girts and standing seam roofs.

Distortional Buckling: The distortional buckling limit state involves the cross-sectional deformations of two or more elements acting as a group, e.g., the rotation of the flange and lip of a C-Section about the web-to-flange junction. The *Specification* and *Commentary* provide three levels of provisions for this limit state. *Commentary* Section C4.2 requires a simple calculation using basic cross-section dimensions and produces a conservative, and sometimes very conservative, result. This approach can sometimes be used to quickly establish that distortional buckling is not a controlling limit state. For those cases where the extra work is justified, *Specification* Section C4.2(a) can be used, which requires considerably more complex calculations, but produces accurate results. *Specification* Section C4.2(b) provides a framework for the use of computerized numerical methods to evaluate distortional buckling. This approach requires fewer calculations than Section C4.2(a) and is especially useful for cross-sections that do not meet the limits of applicability of the other two approaches. For all three approaches, the general procedure involves 1) determination of the elastic distortional buckling stress, 2) determination of the corresponding elastic buckling force using the gross area of the cross-section and 3) transformation of the elastic buckling force to a nominal axial strength, taking into account the effects of inelasticity and post-buckling strength. As an alternative method to *Specification* Section C4, the Direct Strength Method provided in Appendix 1 can also be used to determine member strengths.

For members whose required strengths are determined by first-order analysis, combined flexure and axial force must be checked using Section C5. Alternatively, Appendix 2 permits the use of second-order analysis for the determination of required strengths. In this case, Section C5 is still

used to evaluate members subject to combined flexure and axial force, but the moment modifiers and effective length factors used in Section C5 are set to unity.

SECTION 1 - CONCENTRICALLY LOADED COLUMNS

1.1 Notes on the Tables

(a) With the exception of the studs/joists and tracks, the sections listed in these tables are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members.

(b) The section designations listed in these tables correspond to those for which dimensions and properties are given in Tables I-1, I-2 and I-3.

(c) Tabulated properties and capacities are shown to three significant figures.

(d) Where they apply, the algebraic formulae presented in Section 3 of Part I formed the basis of the calculations for these tables.

(e) The strengths listed in Tables III-1 to III-9 inclusive were computed using the yield stress listed in the tables. Cold work of forming increases were not included.

(f) Tables III-1, III-2 and III-3 give the nominal axial strength, P_n, for fully braced C-Sections at the yield stress listed in the respective tables. Distortional buckling is not considered.

(g) The values labeled P_{web}, P_{flange} and P_{lip} in Tables III-1, III-2 and III-3 are the highest nominal forces at which the web, flange and lip (if applicable) respectively are fully effective. These values are only meaningful where they do not exceed P_{no} for the section and yield stress in question. A value of 0.00 for P_{web} in Table III-2 indicates that a reduction in web area is required at any stress level when standard punchouts are used.

(h) Tables III-4, III-5 and III-6 give tabulated critical buckling lengths, stiffness coefficients, elastic buckling stresses and nominal axial strengths for the limit state of distortional buckling for use with Section C4.2(a). Rotational restraint from sheathing or discrete bracing is not considered in the values given for the stiffness coefficients, elastic buckling stresses and nominal flexural strengths. To incorporate the strength increases resulting from significant continuous rotational bracing or discrete distortional bracing spaced at less than L_{cr}, use the provisions of *Specification* Sections C4.2(a) or C4.2(b).

(i) Tables III-7, III-8 and III-9 give the nominal axial strength, P_n, for C-Sections with varying x- and y-axis unbraced lengths. In all cases, the torsional unbraced length is assumed to equal the y-axis unbraced length and $K_y = K_t = 1.0$. Lengths are arbitrarily cut off at a KL/r_x ratio of approximately 100.

(j) The calculated values in Tables III-1 through III-9 are nominal strengths. These values must be modified by a safety factor, Ω_c, for ASD or a resistance factor, ϕ_c, for LRFD or LSD. See the appropriate *Specification* section for more information.

(k) The effects of standard factory punchouts in studs/joists have been included in Tables III-2 and III-8. These punchouts are considered in studs with flange widths of 1.625 in. or less. Standard punchout sizes are 1.5 in. by 4.5 in. for sections with depths of 3.5 inches or more and 0.75 in. by 4.5 in. for sections with shallower depths.

(l) Dashes in the place of data values in the P_n columns of Tables III-2, III-3 and III-5 indicate that the section is not available in the listed grade of steel. Blank data values in Tables III-7, III-8 and III-9 indicate that the section is not available in the listed grade of steel or that KL/r_y exceeds 200.

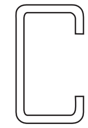
1.2 Nominal Axial Strength Tables - Braced Columns

Table III – 1

F_y = 55 ksi

Braced Column Properties [3]
Purlins/Girts
C-Sections With Lips

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)



Section	P_n at $f=F_y$ kips [1]	Maximum Effective Force, kips [2]		
		P_{web} kips	P_{flange} kips	P_{lip} kips
12CS4x105	64.4	8.99	45.8	33.8
12CS4x085	45.6	4.74	26.0	20.2
12CS4x070	33.3	2.63	15.6	12.9
12CS3.5x105	63.5	8.56	54.0	41.2
12CS3.5x085	44.9	4.51	30.8	24.5
12CS3.5x070	33.0	2.51	18.5	15.6
12CS2.5x105	62.0	7.70	82.2	70.1
12CS2.5x085	44.8	4.06	47.0	41.2
12CS2.5x070	31.8	2.25	28.3	25.9
10CS4x105	63.9	12.0	45.3	33.3
10CS4x085	45.3	6.29	25.8	20.0
10CS4x070	33.1	3.49	15.4	12.7
10CS4x065	29.4	2.79	12.7	10.8
10CS3.5x105	63.0	11.3	53.5	40.7
10CS3.5x085	44.7	5.96	30.5	24.2
10CS3.5x070	32.9	3.31	18.4	15.4
10CS3.5x065	29.3	2.64	15.2	13.0
10CS2.5x105	61.5	10.1	81.7	69.6
10CS2.5x085	44.6	5.29	46.7	40.9
10CS2.5x070	31.7	2.94	28.2	25.8
10CS2.5x065	28.0	2.35	23.3	21.7
10CS2x105	55.7	9.43	109	104
10CS2x085	41.6	4.96	62.5	60.3
10CS2x070	32.0	2.75	38.0	37.7
10CS2x065	29.0	2.20	31.1	31.6
9CS2.5x105	61.1	11.8	81.4	69.3
9CS2.5x085	44.4	6.20	46.5	40.8
9CS2.5x070	31.6	3.44	28.1	25.7
9CS2.5x065	28.0	2.75	23.2	21.7
9CS2.5x059	24.3	2.05	18.1	17.4
8CS4x105	63.1	17.2	44.6	32.5
8CS4x085	44.9	9.05	25.4	19.5
8CS4x070	32.9	5.01	15.2	12.5
8CS4x065	29.2	4.00	12.5	10.6
8CS4x059	25.0	2.98	9.73	8.56
8CS3.5x105	62.2	16.2	52.7	27.3
8CS3.5x085	44.3	8.51	30.1	16.7
8CS3.5x070	32.7	4.72	18.2	11.0
8CS3.5x065	29.1	3.77	15.0	9.38
8CS3.5x059	25.0	2.81	11.7	7.68
8CS2.5x105	60.7	14.2	80.9	64.5
8CS2.5x085	44.2	7.44	46.3	38.3
8CS2.5x070	31.4	4.13	28.0	24.3
8CS2.5x065	27.9	3.29	23.1	20.6
8CS2.5x059	24.2	2.46	18.1	16.6

Section	P_n at $f=F_y$ kips [1]	Maximum Effective Force, kips [2]		
		P_{web} kips	P_{flange} kips	P_{lip} kips
8CS2x105	54.9	13.2	108	103
8CS2x085	41.2	6.91	62.1	59.9
8CS2x070	31.8	3.83	37.7	37.4
8CS2x065	28.8	3.06	30.9	31.4
8CS2x059	25.1	2.28	23.3	25.0
7CS4x105	62.5	21.7	44.0	32.0
7CS4x085	44.6	11.4	25.1	19.2
7CS4x070	32.7	6.28	15.0	12.3
7CS4x065	29.1	5.02	12.4	10.5
7CS4x059	24.9	3.74	9.63	8.46
7CS2.5x105	60.1	17.6	80.4	68.3
7CS2.5x085	43.9	9.22	46.0	40.2
7CS2.5x070	31.3	5.10	27.8	25.4
7CS2.5x065	27.7	4.07	23.0	21.4
7CS2.5x059	24.1	3.03	18.0	17.2
6CS4x105	61.7	28.5	43.2	31.2
6CS4x085	44.2	14.9	24.7	18.8
6CS4x070	32.5	8.23	14.8	12.1
6CS4x065	28.9	6.57	12.2	10.3
6CS4x059	24.8	4.89	9.50	8.33
6CS2.5x105	59.4	22.8	79.6	67.5
6CS2.5x085	43.5	11.9	45.6	39.8
6CS2.5x070	31.1	6.58	27.6	25.2
6CS2.5x065	27.6	5.25	22.8	21.2
6CS2.5x059	24.0	3.91	17.8	17.1
4CS4x105	58.8	51.6	39.3	27.3
4CS4x085	42.7	27.8	22.7	16.7
4CS4x070	31.6	15.8	13.8	11.0
4CS4x065	28.2	12.8	11.4	9.38
4CS4x059	24.3	9.70	8.92	7.68
4CS2.5x105	56.4	47.7	76.7	64.5
4CS2.5x085	41.9	24.7	44.0	38.3
4CS2.5x070	30.2	13.6	26.7	24.3
4CS2.5x065	26.9	10.8	22.1	20.6
4CS2.5x059	23.5	8.02	17.3	16.6
4CS2x105	51.7	43.8	103	104
4CS2x085	39.8	22.7	58.4	59.5
4CS2x070	31.3	12.5	33.8	35.0
4CS2x065	28.4	9.92	27.4	28.6
4CS2x059	24.5	7.37	20.7	21.8

Notes:

1. Axial strengths given are nominal strengths [resistances]. To obtain the available strengths [factored resistances], these values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD).
2. P_{web}, P_{flange} and P_{lip} are the highest nominal axial compression forces at which the web, flange and lip, respectively, are fully effective.
3. The distortional buckling limit state is not considered in this table. Distortional buckling strengths are provided in Table III-4.

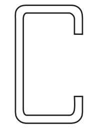
Table III – 2

F_y = 33 ksi

F_y = 50 ksi

Braced Column Properties [3]
Joists/Studs
C-Sections With Lips

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)



Section	P_n at $f=F_y$ kips [1]	Maximum Effective Force, kips [2]		
		P_{web}	P_{flange}	P_{lip}
1200S250-97	**49.9**	6.74	60.2	41.3
1200S250-68	**28.2**	2.29	24.2	19.1
1200S250-54*	**19.7**	1.14	13.6	12.1
1200S200-97	**47.2**	6.35	80.4	59.3
1200S200-68	**28.2**	2.16	31.5	26.5
1200S200-54*	**19.2**	1.07	17.7	16.6
1200S162-97	**35.0**	0.00	80.7	56.1
1200S162-68	**21.9**	0.00	30.8	23.4
1200S162-54*	**15.6**	0.00	17.0	14.2
1000S250-97	**49.4**	8.74	59.8	40.8
1000S250-68	**28.0**	2.96	24.0	19.0
1000S250-54	**19.6**	1.47	13.5	12.1
1000S250-43*	11.2	0.739	7.76	7.92
1000S200-97	**46.7**	8.18	79.9	58.8
1000S200-68	**28.1**	2.77	31.4	26.3
1000S200-54	**19.2**	1.38	17.6	16.5
1000S200-43*	10.9	0.693	9.61	10.6
1000S162-97	**34.7**	0.00	80.5	55.9
1000S162-68	**21.8**	0.00	30.7	23.3
1000S162-54	**15.5**	0.00	16.9	14.2
1000S162-43*	8.55	0.00	9.72	9.04
800S250-97	**48.7**	12.2	59.1	40.1
800S250-68	**27.8**	4.11	23.8	18.7
800S250-54	**19.5**	2.04	13.4	11.9
800S250-43	11.1	1.02	7.71	7.86
800S200-97	**46.0**	11.3	79.3	58.1
800S200-68	**27.9**	3.82	31.2	26.1
800S200-54	**19.1**	1.89	17.5	16.4
800S200-43	10.9	0.949	9.55	10.5
800S200-33*	7.10	0.429	4.30	5.68
800S162-97	**34.2**	0.00	80.0	55.4
800S162-68	**21.6**	0.00	30.5	23.2
800S162-54	**15.5**	0.00	16.9	14.1
800S162-43	8.52	0.00	9.68	9.00
800S162-33*	5.87	0.00	4.91	5.42

Section	P_n at $f=F_y$ kips [1]	Maximum Effective Force, kips [2]		
		P_{web}	P_{flange}	P_{lip}
800S137-97	**30.4**	0.00	84.3	56.0
800S137-68	**19.0**	0.00	31.7	21.2
800S137-54	**13.5**	0.00	17.2	12.3
800S137-43	7.38	0.00	9.64	7.53
800S137-33*	5.04	0.00	5.07	4.40
600S250-97	**47.5**	19.4	57.9	38.9
600S250-68	**27.4**	6.45	23.4	18.3
600S250-54	**19.3**	3.17	13.2	11.8
600S250-43	11.0	1.59	7.61	7.76
600S200-97	**44.8**	17.7	78.1	56.9
600S200-68	**27.5**	5.90	30.8	25.7
600S200-54	**18.9**	2.90	17.3	16.2
600S200-43	10.8	1.45	9.46	10.4
600S200-33	7.06	0.657	4.26	5.64
600S162-97	**33.4**	0.00	79.2	54.5
600S162-68	**21.3**	0.00	30.3	22.9
600S162-54	**15.3**	0.00	16.7	14.0
600S162-43	**8.45**	0.00	9.61	8.93
600S162-33	**5.84**	0.00	4.88	5.39
600S137-97	**29.5**	0.00	83.4	55.1
600S137-68	**18.7**	0.00	31.4	20.9
600S137-54	**13.4**	0.00	17.1	12.2
600S137-43	7.30	0.00	9.57	7.46
600S137-33	5.00	0.00	5.04	4.36
550S162-68	**21.2**	0.00	30.1	22.8
550S162-54	**15.3**	0.00	16.7	13.9
550S162-43	8.42	0.00	9.58	8.91
550S162-33	5.83	0.00	4.86	5.38
400S200-68	**26.6**	11.5	29.9	24.9
400S200-54	**18.5**	5.60	16.9	15.8
400S200-43	10.6	2.78	9.26	10.2
400S200-33	6.97	1.26	4.17	5.55
400S162-68	**20.5**	0.00	29.4	22.1
400S162-54	**14.9**	0.00	16.3	13.6
400S162-43	8.26	0.00	9.42	8.74
400S162-33	5.75	0.00	4.79	5.30

Table III – 2

Braced Column Properties [3]
Joists/Studs
C-Sections With Lips

F_y = 33 ksi
F_y = 50 ksi

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)

Section	P_n at $f=F_y$ kips [1]	Maximum Effective Force, kips [2]			Section	P_n at $f=F_y$ kips [1]	Maximum Effective Force, kips [2]		
		P_{web}	P_{flange}	P_{lip}			P_{web}	P_{flange}	P_{lip}
400S137-68	**17.9**	0.00	30.6	20.1	350S162-68	**20.0**	0.00	28.9	21.5
400S137-54	**13.0**	0.00	16.7	11.8	350S162-54	**14.7**	0.00	16.1	13.3
400S137-43	7.08	0.00	9.38	7.27	350S162-43	8.14	0.00	9.31	8.63
400S137-33	4.90	0.00	4.96	4.28	350S162-33	5.70	0.00	4.74	5.25
362S200-68	**26.4**	13.7	29.7	24.6	250S162-68	**19.5**	0.00	28.5	21.1
362S200-54	**18.3**	6.63	16.8	15.6	250S162-54	**14.5**	0.00	15.9	13.1
362S200-43	10.5	3.29	9.20	10.2	250S162-43	8.06	0.00	9.22	8.54
362S200-33	6.94	1.48	4.14	5.52	250S162-33	5.66	0.00	4.70	5.21
362S162-68	**20.2**	0.00	29.1	21.7	250S137-68	**16.8**	0.00	29.7	19.2
362S162-54	**14.8**	0.00	16.2	13.4	250S137-54	**12.6**	0.00	16.3	11.3
362S162-43	8.18	0.00	9.34	8.66	250S137-43	6.86	0.00	9.17	7.06
362S162-33	5.72	0.00	4.76	5.27	250S137-33	4.80	0.00	4.87	4.19
362S137-68	**17.5**	0.00	30.2	19.8					
362S137-54	**12.8**	0.00	16.5	11.6					
362S137-43	7.00	0.00	9.29	7.18					
362S137-33	4.87	0.00	4.92	4.24					

Notes:

1. Axial strengths given are nominal strengths [resistances]. To obtain the available strengths [factored resistances], these values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD).
2. P_{web}, P_{flange} and P_{lip} are the highest nominal axial compression forces at which the web, flange and lip, respectively, are fully effective.
3. The distortional buckling limit state is not considered in this table. Distortional buckling strengths are provided in Table III-5.

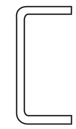
* Web h/t > 200

Table III – 3

F_y = 33 ksi
F_y = 50 ksi

Braced Column Properties
Tracks
C-Sections Without Lips

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)



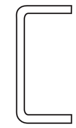
Section	P_n at $f=F_y$ kips [1]	Maximum Effective Force, kips [2]	
		P_{web}	P_{flange}
1200T200-97	**37.7**	5.79	19.5
1200T200-68	**19.3**	1.98	6.29
1200T200-54*	**12.4**	0.987	3.05
1200T150-97	**36.5**	5.43	28.8
1200T150-68	**18.9**	1.86	9.07
1200T150-54*	**12.2**	0.925	4.35
1200T125-97	**35.2**	5.24	37.3
1200T125-68	**18.6**	1.79	11.5
1200T125-54*	**12.1**	0.894	5.47
1000T200-97	**37.2**	7.33	19.1
1000T200-68	**19.2**	2.50	6.15
1000T200-54	**12.3**	1.25	2.98
1000T200-43*	6.29	0.629	1.48
1000T150-97	**36.0**	6.80	28.4
1000T150-68	**18.8**	2.32	8.93
1000T150-54	**12.2**	1.16	4.28
1000T150-43*	6.21	0.584	2.11
1000T125-97	**34.8**	6.54	36.9
1000T125-68	**18.5**	2.24	11.4
1000T125-54	**12.0**	1.11	5.39
1000T125-43*	6.13	0.562	2.64
800T200-97	**36.6**	9.89	18.4
800T200-68	**19.0**	3.37	5.93
800T200-54	**12.2**	1.68	2.87
800T200-43	6.24	0.846	1.42
800T200-33*	3.82	0.383	0.639
800T150-97	**35.4**	9.06	27.8
800T150-68	**18.6**	3.09	8.71
800T150-54	**12.0**	1.54	4.17
800T150-43	6.15	0.775	2.05
800T150-33*	3.78	0.351	0.922
800T125-97	**34.1**	8.65	36.2
800T125-68	**18.3**	2.95	11.1
800T125-54	**11.9**	1.47	5.29
800T125-43	6.08	0.740	2.58
800T125-33*	3.75	0.335	1.16
600T200-97	**35.5**	14.8	17.4
600T200-68	**18.6**	5.04	5.56
600T200-54	**12.0**	2.50	2.69
600T200-43	6.15	1.26	1.33
600T200-33	3.78	0.570	0.598
600T150-97	**34.3**	13.3	26.7
600T150-68	**18.2**	4.53	8.35
600T150-54	**11.9**	2.25	3.99
600T150-43	6.06	1.13	1.96
600T150-33	3.74	0.513	0.881
600T150-30	3.10	0.377	0.645
600T150-27*	2.59	0.281	0.481
600T125-97	**33.0**	12.6	35.1
600T125-68	**17.9**	4.28	10.8
600T125-54	**11.7**	2.13	5.10
600T125-43	5.99	1.07	2.49
600T125-33	3.71	0.485	1.12
600T125-30	3.07	0.356	0.819
600T125-27*	2.57	0.266	0.610
550T200-68	**18.5**	5.68	5.38
550T200-54	**12.0**	2.81	2.59
550T200-43	6.11	1.41	1.28
550T200-33	3.77	0.638	0.573
550T150-68	**18.1**	5.11	8.21
550T150-54	**11.8**	2.54	3.93
550T150-43	6.03	1.28	1.93
550T150-33	3.73	0.578	0.866
550T150-30	3.09	0.424	0.634
550T150-27	2.58	0.317	0.472
550T125-68	**17.8**	4.81	10.6
550T125-54	**11.6**	2.39	5.04
550T125-43	5.95	1.20	2.46
550T125-33	3.69	0.544	1.10
550T125-30	3.06	0.399	0.808
550T125-27	2.57	0.298	0.602
400T200-68	**17.8**	8.40	4.53
400T200-54	**11.7**	4.13	2.18
400T200-43	5.96	2.07	1.07
400T200-33	3.70	0.935	0.482
400T150-68	**17.5**	8.02	7.53
400T150-54	**11.5**	3.95	3.57
400T150-43	5.87	1.98	1.74
400T150-33	3.66	0.895	0.780
400T150-30	3.04	0.657	0.571
400T150-27	2.55	0.491	0.425

Table III – 3

F_y = 33 ksi
F_y = 50 ksi

Braced Column Properties
Tracks
C-Sections Without Lips

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)



Section	P_n at $f=F_y$ kips [1]	Maximum Effective Force, kips [2]		Section	P_n at $f=F_y$ kips [1]	Maximum Effective Force, kips [2]	
		P_{web}	P_{flange}			P_{web}	P_{flange}
400T125-68	**17.1**	7.50	10.03	350T125-68	**16.8**	9.11	9.70
400T125-54	**11.3**	3.71	4.74	350T125-54	**11.2**	4.50	4.58
400T125-43	5.80	1.86	2.31	350T125-43	5.72	2.26	2.22
400T125-33	3.62	0.845	1.035	350T125-33	3.59	1.02	1.00
400T125-30	3.01	0.620	0.757	350T125-30	2.98	0.750	0.727
400T125-27	2.53	0.463	0.564	350T125-27	2.51	0.560	0.542
400T125-18*	1.21	0.136	0.165	350T125-18	1.21	0.164	0.158
362T200-68	**17.6**	9.46	4.32	250T200-68	**16.5**	14.8	3.68
362T200-54	**11.6**	4.64	2.08	250T200-54	**11.0**	7.21	1.77
362T200-43	5.91	2.32	1.02	250T200-43	5.63	3.58	0.872
362T200-33	3.67	1.05	0.460	250T200-33	3.54	1.62	0.392
362T150-68	**17.2**	9.08	7.13	250T150-68	**16.1**	14.4	5.91
362T150-54	**11.4**	4.46	3.38	250T150-54	**10.8**	7.03	2.80
362T150-43	5.82	2.23	1.65	250T150-43	5.54	3.49	1.37
362T150-33	3.63	1.01	0.739	250T150-33	3.50	1.58	0.613
362T150-30	3.02	0.741	0.54	250T150-30	2.92	1.16	0.449
362T150-27	2.53	0.554	0.403	250T150-27	2.46	0.866	0.334
362T125-68	**16.9**	8.65	9.79	250T125-68	**15.8**	14.1	8.17
362T125-54	**11.2**	4.28	4.62	250T125-54	**10.7**	6.87	3.82
362T125-43	5.74	2.15	2.25	250T125-43	5.46	3.42	1.85
362T125-33	3.60	0.973	1.01	250T125-33	3.47	1.55	0.829
362T125-30	2.99	0.714	0.738	250T125-30	2.90	1.14	0.606
362T125-27	2.51	0.534	0.550	250T125-27	2.44	0.848	0.451
362T125-18	1.21	0.157	0.160	250T125-18	1.19	0.249	0.131
350T200-68	**17.5**	9.87	4.25	162T125-33	**3.24**	2.62	0.684
350T200-54	**11.5**	4.84	2.04	162T125-30	**2.73**	1.92	0.500
350T200-43	5.88	2.42	1.01	162T125-27	**2.32**	1.43	0.372
350T200-33	3.66	1.09	0.452	162T125-18	**1.15**	0.422	0.108
350T150-68	**17.1**	9.49	6.99				
350T150-54	**11.3**	4.66	3.31				
350T150-43	5.79	2.33	1.62				
350T150-33	3.62	1.05	0.725				
350T150-30	3.01	0.774	0.530				
350T150-27	2.53	0.578	0.395				

Notes:

1. Axial strengths given are nominal strengths [resistances]. To obtain the available strengths [factored resistances], these values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD).
2. P_{web} and P_{flange} are the highest nominal axial compression forces at which the web and flange, respectively, are fully effective.

* Web h/t > 200

1.3 Distortional Buckling Axial Strength Tables

Tables III-4, III-5 and III-6 provide computed distortional buckling properties and strengths under axial load for the representative purlins/girts, studs/joists and Z-Sections with lips, respectively. The values in these tables have been calculated for use with Section C4.2(a).

(a) Where a known rotational stiffness, k_ϕ, from bracing or sheathing is available, the values in the columns under the headings $k_{\phi fe}$, $\tilde{k}_{\phi fg}$, $k_{\phi we}$ and $\tilde{k}_{\phi wg}$ may be used in Eq. C4.2-6 to calculate a more exact value of F_d.

(b) The values in the column under the heading P_n are valid for cases where bracing against distortional buckling is insignificant or spaced at a length equal to or greater than L_{cr}.

Table III – 4

Distortional Buckling Properties
Purlins/Girts - Axial Strength
C-Sections With Lips

F_y = 55 ksi

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)

Section	Per Section C4.2(a)						
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d ksi	P_n[1] $_{(Lm \geq Lcr)}$ kips
12CS4x105	32.7	0.712	0.0322	0.521	0.0280	20.5	57.6
12CS4x085	35.2	0.363	0.0223	0.277	0.0195	15.3	40.2
12CS4x070	37.8	0.196	0.0158	0.154	0.0139	11.8	28.9
12CS3.5x105	30.2	0.721	0.0267	0.521	0.0328	20.9	55.4
12CS3.5x085	32.5	0.367	0.0184	0.277	0.0229	15.6	38.6
12CS3.5x070	35.0	0.198	0.0130	0.154	0.0162	12.0	27.8
12CS2.5x105	24.6	0.750	0.0175	0.521	0.0492	19.1	47.6
12CS2.5x085	26.6	0.380	0.0119	0.277	0.0342	14.2	33.1
12CS2.5x070	28.7	0.204	0.00838	0.154	0.0242	11.0	23.8
10CS4x105	31.2	0.835	0.0353	0.625	0.0177	27.5	60.3
10CS4x085	33.6	0.427	0.0244	0.332	0.0124	20.6	42.3
10CS4x070	36.1	0.231	0.0173	0.185	0.00881	16.0	30.6
10CS4x065	37.2	0.183	0.0151	0.148	0.00772	14.5	27.0
10CS3.5x105	28.8	0.845	0.0293	0.625	0.0208	29.4	58.9
10CS3.5x085	31.1	0.431	0.0202	0.332	0.0145	22.0	41.4
10CS3.5x070	33.4	0.233	0.0143	0.185	0.0103	17.0	30.0
10CS3.5x065	34.4	0.184	0.0125	0.148	0.00902	15.5	26.5
10CS2.5x105	23.5	0.877	0.0192	0.625	0.0312	29.8	52.7
10CS2.5x085	25.4	0.445	0.0131	0.332	0.0217	22.4	37.1
10CS2.5x070	27.4	0.240	0.00917	0.185	0.0153	17.3	26.9
10CS2.5x065	28.2	0.189	0.00800	0.148	0.0134	15.8	23.7
10CS2x105	20.5	0.905	0.0152	0.625	0.0411	27.2	47.3
10CS2x085	22.2	0.457	0.0103	0.332	0.0284	20.4	33.2
10CS2x070	24.0	0.245	0.00715	0.185	0.0201	15.8	24.0
10CS2x065	24.7	0.193	0.00622	0.148	0.0176	14.4	21.2
9CS2.5x105	22.9	0.960	0.0202	0.695	0.0240	37.5	54.9
9CS2.5x085	24.7	0.488	0.0138	0.369	0.0166	28.2	38.9
9CS2.5x070	26.7	0.263	0.00967	0.206	0.0118	21.9	28.3
9CS2.5x065	27.5	0.208	0.00844	0.165	0.0103	19.9	25.1
9CS2.5x059	28.6	0.153	0.00705	0.123	0.00867	17.6	21.4
8CS4x105	29.5	1.02	0.0394	0.782	0.0102	36.3	61.3
8CS4x085	31.8	0.521	0.0273	0.415	0.00709	27.2	43.4
8CS4x070	34.2	0.283	0.0193	0.232	0.00504	21.1	31.6
8CS4x065	35.2	0.224	0.0169	0.185	0.00442	19.2	27.9
8CS4x059	36.5	0.165	0.0142	0.139	0.00372	17.0	23.8
8CS3.5x105	27.3	1.03	0.0327	0.782	0.0119	40.5	60.7
8CS3.5x085	29.4	0.526	0.0226	0.415	0.00830	30.5	43.1
8CS3.5x070	31.6	0.285	0.0159	0.232	0.00590	23.7	31.4
8CS3.5x065	32.6	0.226	0.0139	0.185	0.00517	21.5	27.8
8CS3.5x059	33.8	0.167	0.0117	0.139	0.00435	19.1	23.8
8CS2.5x105	22.2	1.06	0.0214	0.782	0.0179	46.9	56.6
8CS2.5x085	24.0	0.542	0.0146	0.415	0.0124	35.4	40.4
8CS2.5x070	25.9	0.292	0.0103	0.232	0.00878	27.5	29.6
8CS2.5x065	26.7	0.231	0.00895	0.185	0.00769	25.0	26.2
8CS2.5x059	27.7	0.170	0.00748	0.139	0.00646	22.2	22.4

Table III – 4

Distortional Buckling Properties
Purlins/Girts - Axial Strength
C-Sections With Lips

F_y = 55 ksi

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)

Section	Per Section C4.2(a)						
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d ksi	P_n[1] $_{(Lm \ge Lcr)}$ kips
8CS2x105	19.4	1.09	0.0170	0.782	0.0235	46.3	52.2
8CS2x085	21.0	0.555	0.0115	0.415	0.0163	35.0	37.3
8CS2x070	22.7	0.299	0.00799	0.232	0.0115	27.2	27.3
8CS2x065	23.3	0.236	0.00696	0.185	0.0100	24.8	24.2
8CS2x059	24.3	0.173	0.00580	0.139	0.00843	21.9	20.7
7CS4x105	28.5	1.14	0.0421	0.894	0.00728	41.2	61.1
7CS4x085	30.7	0.588	0.0292	0.474	0.00508	31.0	43.5
7CS4x070	33.1	0.320	0.0206	0.265	0.00361	24.1	31.7
7CS4x065	34.0	0.253	0.0180	0.212	0.00317	21.9	28.1
7CS4x059	35.3	0.187	0.0151	0.159	0.00266	19.4	24.0
7CS2.5x105	21.5	1.19	0.0229	0.894	0.0128	58.5	57.4
7CS2.5x085	23.2	0.610	0.0156	0.474	0.00888	44.2	41.4
7CS2.5x070	25.1	0.330	0.0110	0.265	0.00629	34.4	30.5
7CS2.5x065	25.8	0.261	0.00956	0.212	0.00551	31.4	27.1
7CS2.5x059	26.8	0.192	0.00800	0.159	0.00463	27.8	23.2
6CS4x105	27.5	1.31	0.0455	1.04	0.00495	46.7	60.5
6CS4x085	29.6	0.676	0.0315	0.553	0.00345	35.2	43.2
6CS4x070	31.8	0.368	0.0223	0.309	0.00246	27.4	31.6
6CS4x065	32.7	0.292	0.0195	0.247	0.00215	24.9	28.0
6CS4x059	34.0	0.216	0.0164	0.185	0.00181	22.1	24.0
6CS2.5x105	20.7	1.37	0.0247	1.04	0.00871	72.1	57.2
6CS2.5x085	22.4	0.700	0.0169	0.553	0.00604	54.6	41.7
6CS2.5x070	24.1	0.379	0.0118	0.309	0.00428	42.7	31.0
6CS2.5x065	24.8	0.300	0.0103	0.247	0.00375	38.9	27.6
6CS2.5x059	25.8	0.221	0.00864	0.185	0.00315	34.5	23.7
4CS4x105	24.8	1.89	0.0558	1.56	0.00180	60.1	58.0
4CS4x085	26.7	0.980	0.0386	0.830	0.00125	45.4	41.9
4CS4x070	28.7	0.536	0.0273	0.463	0.000892	35.5	30.9
4CS4x065	29.6	0.426	0.0239	0.371	0.000782	32.3	27.5
4CS4x059	30.7	0.315	0.0200	0.277	0.000658	28.6	23.6
4CS2.5x105	18.7	1.96	0.0303	1.56	0.00316	105	53.5
4CS2.5x085	20.2	1.01	0.0207	0.830	0.00219	80.3	40.0
4CS2.5x070	21.8	0.549	0.0145	0.463	0.00155	63.0	30.3
4CS2.5x065	22.4	0.436	0.0127	0.371	0.00136	57.6	27.1
4CS2.5x059	23.3	0.322	0.0106	0.277	0.00114	51.1	23.5
4CS2x105	17.5	1.96	0.0232	1.56	0.00361	131	51.4
4CS2x085	19.0	1.01	0.0156	0.830	0.00248	102	39.4
4CS2x070	20.6	0.547	0.0108	0.463	0.00174	80.8	30.3
4CS2x065	21.2	0.434	0.00935	0.371	0.00152	74.1	27.3
4CS2x059	22.1	0.321	0.00777	0.277	0.00127	66.2	23.8

Note:

1. Axial strengths given are nominal strengths [resistances]. To obtain the available strengths [factored resistances], these values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD).

Table III – 5

Fy = 33 ksi
F_y = 50 ksi

Distortional Buckling Properties
Joists/Studs - Axial Strength
C-Sections With Lips

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)

Section	Per Section C4.2(a)						
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d ksi	P_n[1]$_{(Lm \geq Lcr)}$ kips
1200S250-97	19.7	0.775	0.0212	0.474	0.0749	13.0	**35.2**
1200S250-68	23.9	0.234	0.0105	0.163	0.0355	8.64	**20.1**
1200S250-54*	27.1	0.110	0.00664	0.0816	0.0219	6.69	**14.0**
1200S200-97	17.1	0.803	0.0158	0.474	0.0984	11.2	**30.7**
1200S200-68	20.9	0.241	0.00787	0.163	0.0464	7.45	**17.5**
1200S200-54*	23.7	0.112	0.00498	0.0816	0.0286	5.77	**12.1**
1200S162-97	12.9	0.938	0.0142	0.474	0.174	7.50	**23.2**
1200S162-68	15.8	0.273	0.00713	0.163	0.0815	4.92	**13.1**
1200S162-54*	17.9	0.124	0.00452	0.0816	0.0501	3.78	**9.00**
1000S250-97	18.8	0.898	0.0233	0.568	0.0475	20.7	**39.6**
1000S250-68	22.8	0.274	0.0115	0.196	0.0225	13.8	**22.9**
1000S250-54	25.9	0.129	0.00728	0.0980	0.0139	10.7	**16.0**
1000S250-43*	29.2	0.0618	0.00463	0.0496	0.00869	8.36	9.25
1000S200-97	16.4	0.929	0.0173	0.568	0.0624	18.8	**35.3**
1000S200-68	20.0	0.281	0.00862	0.196	0.0294	12.5	**20.4**
1000S200-54	22.7	0.131	0.00545	0.0980	0.0181	9.72	**14.2**
1000S200-43*	25.6	0.0629	0.00347	0.0496	0.0113	7.60	8.24
1000S162-97	12.3	1.08	0.0156	0.568	0.110	13.1	**27.2**
1000S162-68	15.1	0.316	0.00781	0.196	0.0517	8.60	**15.5**
1000S162-54	17.1	0.145	0.00495	0.0980	0.0317	6.62	**10.8**
1000S162-43*	19.4	0.0682	0.00316	0.0496	0.0197	5.15	6.23
800S250-97	17.8	1.08	0.0260	0.710	0.0272	33.7	**43.4**
800S250-68	21.6	0.332	0.0129	0.245	0.0129	22.4	**25.5**
800S250-54	24.5	0.157	0.00814	0.122	0.00796	17.3	**18.0**
800S250-43	27.6	0.0756	0.00517	0.0620	0.00498	13.6	10.4
800S200-97	15.5	1.11	0.0194	0.710	0.0357	33.1	**39.9**
800S200-68	18.9	0.340	0.00964	0.245	0.0168	22.1	**23.5**
800S200-54	21.4	0.160	0.00610	0.122	0.0104	17.1	**16.6**
800S200-43	24.2	0.0768	0.00388	0.0620	0.00648	13.4	9.56
800S200-33*	27.9	0.0331	0.00229	0.0280	0.00375	10.1	6.38
800S162-97	11.6	1.28	0.0174	0.710	0.0632	24.7	**32.0**
800S162-68	14.2	0.379	0.00873	0.245	0.0296	16.3	**18.6**
800S162-54	16.2	0.175	0.00554	0.122	0.0182	12.5	**13.0**
800S162-43	18.3	0.0828	0.00353	0.0620	0.0113	9.77	7.51
800S162-33*	21.1	0.0351	0.00208	0.0280	0.00652	7.33	4.97
800S137-97	8.59	1.56	0.0172	0.710	0.116	17.0	**24.9**
800S137-68	10.5	0.445	0.00873	0.245	0.0543	10.9	**14.1**
800S137-54	12.0	0.201	0.00556	0.122	0.0333	8.33	**9.78**
800S137-43	13.6	0.0931	0.00355	0.0620	0.0206	6.42	5.64
800S137-33*	15.7	0.0386	0.00210	0.0280	0.0118	4.78	3.70
600S250-97	16.5	1.37	0.0300	0.947	0.0132	53.6	**45.1**
600S250-68	20.1	0.427	0.0149	0.326	0.00627	35.6	**27.1**
600S250-54	22.8	0.203	0.00940	0.163	0.00388	27.6	19.3
600S250-43	25.7	0.0984	0.00597	0.0826	0.00242	21.6	11.1
600S200-97	14.4	1.41	0.0224	0.947	0.0174	59.4	**42.8**
600S200-68	17.6	0.436	0.0111	0.326	0.00820	39.5	**26.0**
600S200-54	19.9	0.206	0.00704	0.163	0.00506	30.6	**18.6**
600S200-43	22.5	0.0998	0.00448	0.0826	0.00315	23.9	10.6
600S200-33	25.9	0.0432	0.00264	0.0373	0.00183	18.0	7.18

Table III – 5 Fy = 33 ksi **F_y = 50 ksi**	**Distortional Buckling Properties Joists/Studs - Axial Strength C-Sections With Lips**					Ω_c = 1.80 (ASD) ϕ_c = 0.85 (LRFD) ϕ_c = 0.80 (LSD)	
	Per Section C4.2(a)						
Section	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d ksi	P_n[1] $_{(Lm \geq Lcr)}$ kips
600S162-97	10.8	1.60	0.0201	0.947	0.0308	50.1	**36.2**
600S162-68	13.3	0.481	0.0101	0.326	0.0144	33.0	**21.7**
600S162-54	15.1	0.224	0.00639	0.163	0.00885	25.4	**15.4**
600S162-43	17.1	0.107	0.00408	0.0826	0.00550	19.8	8.85
600S162-33	19.7	0.0455	0.00241	0.0373	0.00318	14.8	5.95
600S137-97	7.99	1.93	0.0198	0.947	0.0566	37.6	**29.6**
600S137-68	9.79	0.558	0.0101	0.326	0.0265	24.2	**17.3**
600S137-54	11.1	0.254	0.00642	0.163	0.0162	18.4	**12.2**
600S137-43	12.6	0.119	0.00410	0.0826	0.0100	14.2	6.99
600S137-33	14.6	0.0496	0.00242	0.0373	0.00577	10.6	4.64
550S162-68	13.0	0.518	0.0105	0.356	0.0116	39.5	**22.3**
550S162-54	14.8	0.241	0.00668	0.178	0.00712	30.4	**16.0**
550S162-43	16.7	0.115	0.00426	0.0901	0.00443	23.7	9.12
550S162-33	19.3	0.0493	0.00251	0.0407	0.00255	17.8	6.16
400S200-68	15.9	0.624	0.0136	0.490	0.00297	67.1	**26.0**
400S200-54	18.0	0.298	0.00862	0.245	0.00183	51.9	**19.0**
400S200-43	20.4	0.145	0.00549	0.124	0.00114	40.5	10.8
400S200-33	23.4	0.0632	0.00324	0.0559	0.000663	30.5	7.43
400S162-68	12.0	0.679	0.0123	0.490	0.00523	66.5	**23.0**
400S162-54	13.6	0.319	0.00783	0.245	0.00321	51.1	**16.8**
400S162-43	15.4	0.153	0.00499	0.124	0.00200	39.7	9.48
400S162-33	17.8	0.0660	0.00295	0.0559	0.00115	29.8	6.53
400S137-68	8.84	0.773	0.0123	0.490	0.00960	57.5	**19.7**
400S137-54	10.1	0.356	0.00786	0.245	0.00588	43.7	**14.2**
400S137-43	11.4	0.168	0.00502	0.124	0.00364	33.7	8.06
400S137-33	13.2	0.0710	0.00297	0.0559	0.00209	25.1	5.49
362S200-68	15.5	0.681	0.0143	0.540	0.00233	73.4	**25.7**
362S200-54	17.6	0.326	0.00906	0.270	0.00143	56.8	**18.9**
362S200-43	19.9	0.159	0.00577	0.137	0.000895	44.4	10.6
362S200-33	22.9	0.0693	0.00340	0.0617	0.000518	33.4	7.38
362S162-68	11.7	0.739	0.0130	0.540	0.00409	75.0	**22.7**
362S162-54	13.3	0.348	0.00823	0.270	0.00251	57.6	**16.7**
362S162-43	15.0	0.168	0.00524	0.137	0.00156	44.7	9.42
362S162-33	17.3	0.0723	0.00310	0.0617	0.000901	33.6	6.53
362S137-68	8.63	0.838	0.0130	0.540	0.00751	67.3	**19.7**
362S137-54	9.82	0.387	0.00826	0.270	0.00460	51.1	**14.4**
362S137-43	11.1	0.183	0.00527	0.137	0.00285	39.4	8.10
362S137-33	12.9	0.0775	0.00312	0.0617	0.00164	29.3	5.57
350S162-68	11.6	0.762	0.0132	0.560	0.00374	78.0	**22.6**
350S162-54	13.2	0.359	0.00837	0.280	0.00230	59.9	**16.7**
350S162-43	14.9	0.173	0.00534	0.142	0.00143	46.5	9.39
350S162-33	17.2	0.0747	0.00315	0.0639	0.000825	34.9	6.52
250S162-68	10.7	1.02	0.0156	0.783	0.00161	105	**21.1**
250S162-54	12.1	0.486	0.00990	0.392	0.000992	80.5	**15.9**
250S162-43	13.7	0.236	0.00631	0.198	0.000617	62.6	8.86
250S162-33	15.8	0.102	0.00373	0.0895	0.000356	47.0	6.29
250S137-68	7.86	1.14	0.0156	0.783	0.00296	104	**18.5**
250S137-54	8.95	0.532	0.00994	0.392	0.00182	78.6	**13.9**
250S137-43	10.2	0.254	0.00635	0.198	0.00112	60.5	7.76
250S137-33	11.7	0.109	0.00375	0.0895	0.000647	45.0	5.48

Note:

1. Axial strengths given are nominal strengths [resistances]. To obtain the available strengths [factored resistances], these values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD).

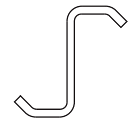
* Web h/t > 200

Table III – 6

Distortional Buckling Properties
Purlins/Girts - Axial Strength
Z-Sections With Lips

F_y = 55 ksi

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)



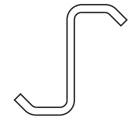
Section	Per Section C4.2(a) L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d ksi	P_n[1] $(L_m \geq L_{cr})$ kips
12ZS3.25x105	26.8	0.765	0.0320	0.521	0.0415	17.5	50.6
12ZS3.25x085	29.4	0.384	0.0215	0.277	0.0279	13.4	35.6
12ZS3.25x070	32.0	0.205	0.0150	0.154	0.0195	10.4	25.7
12ZS2.75x105	24.2	0.786	0.0268	0.521	0.0512	16.8	47.0
12ZS2.75x085	26.5	0.393	0.0179	0.277	0.0343	12.8	33.1
12ZS2.75x070	28.8	0.209	0.0124	0.154	0.0239	10.0	23.9
12ZS2.25x105	21.3	0.815	0.0221	0.521	0.0661	15.1	42.3
12ZS2.25x085	23.4	0.405	0.0148	0.277	0.0442	11.6	29.7
12ZS2.25x070	25.4	0.215	0.0102	0.154	0.0308	9.02	21.4
10ZS3.25x105	25.6	0.893	0.0351	0.625	0.0263	24.7	54.2
10ZS3.25x085	28.1	0.450	0.0236	0.332	0.0177	18.9	38.4
10ZS3.25x070	30.5	0.241	0.0164	0.185	0.0123	14.8	27.9
10ZS3.25x065	31.5	0.190	0.0142	0.148	0.0107	13.5	24.7
10ZS3.25x059	33.0	0.140	0.0118	0.111	0.00894	12.1	21.1
10ZS2.75x105	23.1	0.915	0.0293	0.625	0.0324	24.9	51.4
10ZS2.75x085	25.4	0.459	0.0197	0.332	0.0218	19.1	36.4
10ZS2.75x070	27.5	0.245	0.0136	0.185	0.0152	15.0	26.4
10ZS2.75x065	28.5	0.193	0.0118	0.148	0.0132	13.7	23.4
10ZS2.75x059	29.7	0.142	0.00982	0.111	0.0110	12.2	20.0
10ZS2.25x105	20.3	0.947	0.0243	0.625	0.0419	23.8	47.2
10ZS2.25x085	22.3	0.473	0.0162	0.332	0.0280	18.2	33.4
10ZS2.25x070	24.3	0.252	0.0112	0.185	0.0195	14.2	24.3
10ZS2.25x065	25.1	0.198	0.00970	0.148	0.0170	13.0	21.5
10ZS2.25x059	26.2	0.145	0.00805	0.111	0.0141	11.6	18.3
9ZS2.25x105	19.8	1.03	0.0256	0.695	0.0322	29.9	49.5
9ZS2.25x085	21.8	0.517	0.0171	0.369	0.0215	22.9	35.2
9ZS2.25x070	23.7	0.276	0.0118	0.206	0.0150	18.0	25.7
9ZS2.25x065	24.5	0.217	0.0102	0.165	0.0130	16.4	22.7
9ZS2.25x059	25.6	0.159	0.00848	0.123	0.0108	14.6	19.4
8ZS3.25x105	24.2	1.08	0.0392	0.782	0.0150	34.3	56.3
8ZS3.25x085	26.6	0.546	0.0264	0.415	0.0101	26.3	40.2
8ZS3.25x070	28.9	0.294	0.0183	0.232	0.00706	20.7	29.4
8ZS3.25x065	29.8	0.232	0.0159	0.185	0.00615	18.9	26.1
8ZS3.25x059	31.2	0.171	0.0132	0.139	0.00512	16.9	22.3
8ZS2.75x105	21.8	1.11	0.0328	0.782	0.0186	36.8	54.4
8ZS2.75x085	24.0	0.557	0.0220	0.415	0.0125	28.2	39.0
8ZS2.75x070	26.1	0.299	0.0152	0.232	0.00869	22.2	28.5
8ZS2.75x065	26.9	0.236	0.0132	0.185	0.00756	20.3	25.3
8ZS2.75x059	28.1	0.173	0.0110	0.139	0.00628	18.1	21.7
8ZS2.25x105	19.2	1.14	0.0271	0.782	0.0240	37.6	51.3
8ZS2.25x085	21.1	0.572	0.0181	0.415	0.0160	28.9	36.8
8ZS2.25x070	23.0	0.306	0.0125	0.232	0.0112	22.7	26.9
8ZS2.25x065	23.7	0.241	0.0108	0.185	0.00971	20.7	23.9
8ZS2.25x059	24.8	0.177	0.00900	0.139	0.00806	18.5	20.5
7ZS2.25x105	18.6	1.28	0.0290	0.894	0.0172	47.0	52.5
7ZS2.25x085	20.4	0.643	0.0194	0.474	0.0115	36.2	37.9
7ZS2.25x070	22.2	0.344	0.0134	0.265	0.00799	28.5	27.9
7ZS2.25x065	23.0	0.271	0.0116	0.212	0.00695	26.1	24.8
7ZS2.25x059	24.0	0.199	0.00962	0.159	0.00577	23.2	21.3

Table III – 6

Distortional Buckling Properties
Purlins/Girts - Axial Strength
Z-Sections With Lips

F_y = 55 ksi

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)



Section	Per Section C4.2(a)						
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d ksi	**P_n[1] $(L_m \geq L_{cr})$ kips**
6ZS2.25x105	17.9	1.46	0.0313	1.04	0.0117	58.2	52.8
6ZS2.25x085	19.7	0.735	0.0209	0.553	0.00781	44.8	38.5
6ZS2.25x070	21.4	0.395	0.0144	0.309	0.00544	35.4	28.5
6ZS2.25x065	22.1	0.312	0.0125	0.247	0.00473	32.4	25.4
6ZS2.25x059	23.1	0.229	0.0104	0.185	0.00393	28.9	21.8
4ZS2.25x070	19.3	0.568	0.0177	0.463	0.00197	52.5	28.2
4ZS2.25x065	20.0	0.450	0.0153	0.371	0.00172	48.1	25.2
4ZS2.25x059	20.9	0.331	0.0127	0.277	0.00143	43.0	21.8
3.5ZS1.5x070	11.8	0.721	0.0147	0.529	0.00357	68.3	23.3
3.5ZS1.5x065	12.1	0.568	0.0128	0.424	0.00312	62.3	20.9
3.5ZS1.5x059	12.7	0.416	0.0106	0.317	0.00260	55.3	18.2

Note:

1. Axial strengths given are nominal strengths [resistances]. To obtain the available strengths [factored resistances], these values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD).

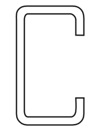
1.4 Nominal Axial Strength Tables - Unbraced Columns

Table III – 7

Nominal Axial Strength, P_n, kips [1,2]
Purlins/Girts
C-Sections With Lips

F_y = 55 ksi

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)



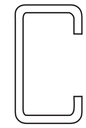
Section	KL_x ft.	Bracing ($KL_y = KL_t$)					Section	KL_x ft.	Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None			Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
12CS3.5x105	5.0	63.0†	62.9†	62.6†	61.8†	57.0†	10CS3.5x070	5.0	32.5†	32.4†	32.3†	31.7†	28.9
	10.0	61.6†	61.0†	60.1†	57.0†	38.4		9.0	31.7†	31.3†	30.9†	29.3	21.6
	15.0	59.3†	58.0†	56.2†	48.6	20.2		13.0	30.4†	29.7	28.8	25.8	14.1
	20.0	56.3†	54.2	50.2	38.4	12.6		17.0	28.7	27.6	26.2	21.7	9.27
	24.0	53.2	49.5	44.7	29.1			21.0	26.8	25.2	23.3	17.8	
	29.0	47.9	43.6	37.3	21.3			25.0	24.5	22.5	20.2	13.9	
	34.0	42.7	37.0	29.5	16.5			29.0	22.1	19.8	17.2	11.0	
	39.0	36.8	30.1	23.8	13.2			33.0	19.8	17.2	14.3	8.93	
12CS3.5x085	5.0	44.6†	44.5†	44.3†	43.6†	39.7†	10CS3.5x065	5.0	28.9†	28.8†	28.7†	28.2†	25.7
	10.0	43.4†	43.0†	42.2†	39.7†	27.9		9.0	28.2†	27.9†	27.5†	26.1	19.2
	15.0	41.6†	40.6†	39.0†	34.4	15.1		13.0	27.1†	26.5	25.7	23.0	12.7
	20.0	39.2†	37.5	35.2	27.9	9.43		17.0	25.6	24.6	23.4	19.4	8.31
	24.0	36.8	34.8	32.0	21.8			21.0	23.8	22.5	20.8	15.8	
	29.0	34.0	31.5	27.1	15.9			25.0	21.9	20.1	17.9	12.5	
	34.0	31.0	27.0	21.8	12.3			29.0	19.7	17.6	15.4	9.85	
	39.0	27.0	22.5	17.5	9.83			33.0	17.5	15.4	12.9	8.02	
12CS3.5x070	5.0	32.8†	32.7†	32.5†	32.0†	29.3†	8CS3.5x105	4.0	61.6†	61.4†	61.2†	60.6	56.8
	10.0	32.0†	31.6†	31.1†	29.2†	20.3		7.0	60.3	59.8	59.2	57.3	46.1
	15.0	30.6†	29.9†	28.7†	25.1	11.6		10.0	58.3	57.4	56.3	52.8	32.6
	20.0	28.9†	27.6	25.8	20.3	7.22		14.0	54.9	53.1	50.7	43.4	19.5
	24.0	27.2	25.5	23.1	16.5			17.0	51.4	48.2	45.1	35.9	14.5
	29.0	24.8	22.6	19.8	12.2			20.0	46.9	43.1	39.8	28.0	11.3
	34.0	22.3	19.8	16.4	9.41			24.0	40.9	35.8	30.8	21.0	
	39.0	19.8	16.9	13.3	7.52			27.0	35.7	29.8	25.3	17.4	
10CS3.5x105	5.0	62.3†	62.1†	61.9†	61.0†	56.2	8CS3.5x085	4.0	43.8†	43.6†	43.5†	43.0	39.9
	9.0	60.9†	60.3†	59.5†	56.9	42.0		7.0	42.8	42.3	41.9	40.3	32.8
	13.0	58.6	57.5	56.0	50.3	25.7		10.0	41.2	40.4	39.5	36.6	24.0
	17.0	55.8	53.9	51.0	42.1	16.7		14.0	38.4	37.0	35.5	31.3	14.1
	21.0	51.9	48.5	44.7	32.8			17.0	35.9	34.2	32.4	26.2	10.5
	25.0	47.0	43.0	38.3	25.0			20.0	33.5	31.3	28.8	20.8	8.28
	29.0	42.1	37.0	30.9	19.9			24.0	30.0	26.5	23.2	15.5	
	33.0	36.5	30.4	25.1	16.4			27.0	26.5	22.6	19.1	12.8	
10CS3.5x085	5.0	44.2†	44.0†	43.8†	43.1†	39.2	8CS3.5x070	4.0	32.3†	32.2†	32.1†	31.7†	29.5
	9.0	43.0†	42.5†	41.9†	39.7	30.2		7.0	31.6†	31.3	30.9	29.8	23.9
	13.0	41.2	40.3	39.1	35.2	18.7		10.0	30.5	29.9	29.2	27.1	17.6
	17.0	38.9	37.4	35.6	30.4	12.4		14.0	28.5	27.4	26.3	22.6	10.6
	21.0	36.2	34.3	32.1	24.2			17.0	26.7	25.2	23.7	19.2	7.84
	25.0	33.5	31.2	27.8	18.3			20.0	24.6	22.8	20.9	15.8	6.13
	29.0	30.7	27.1	23.1	14.5			24.0	21.7	19.5	17.4	11.8	
	33.0	26.8	22.8	18.7	11.9			27.0	19.6	17.1	14.7	9.76	

Table III – 7

Nominal Axial Strength, P_n, kips [1,2]
Purlins/Girts
C-Sections With Lips

F_y = 55 ksi

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)



Section	KL_x ft.	Bracing ($KL_y = KL_t$)					Section	KL_x ft.	Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None			Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
8CS3.5x065	4.0	28.8†	28.7†	28.6†	28.2†	26.3	8CS2.5x065	4.0	27.5†	27.4†	27.3†	26.8†	24.6
	7.0	28.1†	27.9†	27.6	26.6	21.3		7.0	26.8†	26.5†	26.2	25.1	18.1
	11.0	26.7	26.1	25.5	23.2	13.8		10.0	25.9	25.5	24.9	23.0	10.5
	14.0	25.4	24.4	23.4	20.2	9.54		13.0	24.9	24.2	23.3	19.1	6.78
	17.0	23.8	22.5	21.1	16.9	7.02		17.0	23.2	21.9	19.8	13.4	
	21.0	21.4	19.6	17.8	13.2	5.09		20.0	21.4	19.2	16.5	10.3	
	24.0	19.4	17.3	15.5	10.7			23.0	18.8	16.2	13.4	8.31	
	27.0	17.4	15.3	13.3	8.79			26.0	16.2	13.4	10.9	6.78	
8CS3.5x059	4.0	24.7†	24.7†	24.6†	24.3†	22.6	8CS2.5x059	4.0	23.9†	23.8†	23.7†	23.2†	20.9
	7.0	24.2†	24.0†	23.7	22.8	18.3		7.0	23.3†	23.0†	22.7†	21.5	15.9
	11.0	23.0	22.5	21.9	20.0	11.9		10.0	22.3	21.9	21.2	19.3	9.21
	14.0	21.9	21.1	20.2	17.4	8.31		13.0	21.2	20.4	19.6	16.7	5.96
	17.0	20.5	19.4	18.2	14.6	6.09		17.0	19.5	18.6	17.4	11.7	
	21.0	18.4	16.9	15.4	11.4	4.39		20.0	18.3	16.9	14.5	8.99	
	24.0	16.8	15.0	13.2	9.33			23.0	16.5	14.3	11.7	7.21	
	27.0	15.0	13.0	11.5	7.69			26.0	14.2	11.8	9.56	5.96	
8CS2.5x105	4.0	59.7†	59.4†	59.1†	57.8†	50.3	6CS2.5x105	3.0	58.4†	58.2†	57.9†	57.1	52.3
	7.0	57.7†	56.9†	55.9	52.3	34.3		5.0	56.8	56.2	55.5	53.3	42.3
	10.0	54.8	53.3	51.4	45.1	19.9		8.0	53.0	51.5	50.1	45.4	26.8
	13.0	51.0	48.8	45.9	37.1	12.5		10.0	49.7	47.6	45.6	39.3	19.0
	16.0	46.7	43.7	40.0	28.8			13.0	44.0	41.0	38.2	30.2	11.4
	20.0	40.4	36.4	32.0	19.9			15.0	39.8	36.3	33.1	24.5	8.56
	23.0	35.4	31.0	26.3	15.7			18.0	33.4	29.3	25.7	17.9	
	26.0	30.5	25.8	21.7	12.5			20.0	29.1	24.8	21.4	14.9	
8CS2.5x085	4.0	43.7†	43.5†	43.3†	42.6†	38.0	6CS2.5x085	3.0	43.0†	42.8†	42.7†	42.2†	39.7
	7.0	42.6†	42.2†	41.6†	39.4	25.8		5.0	42.1†	41.8†	41.4	40.3	31.8
	10.0	41.1†	40.3	38.7	33.8	15.0		8.0	40.2	39.3	38.1	34.4	19.4
	13.0	38.6	36.8	34.5	27.6	9.70		10.0	37.9	36.3	34.7	29.6	13.8
	16.0	35.3	32.9	30.0	21.7			13.0	33.5	31.2	29.0	22.5	9.21
	20.0	30.5	27.4	23.8	15.0			15.0	30.4	27.7	25.1	18.1	6.96
	23.0	26.7	23.3	19.4	11.9			18.0	25.5	22.3	19.5	13.6	
	26.0	23.0	19.3	16.0	9.70			20.0	22.4	18.9	16.3	11.4	
8CS2.5x070	4.0	31.2†	31.1†	30.9†	30.5†	28.2	6CS2.5x070	3.0	30.8	30.7	30.6	30.3	28.8
	7.0	30.5†	30.3†	29.9†	28.8	19.9		5.0	30.3	30.1	29.9	29.1	24.6
	10.0	29.6†	29.1	28.5	25.9	11.6		8.0	29.1	28.6	28.2	26.2	14.6
	13.0	28.5	27.6	26.3	21.1	7.50		10.0	28.1	27.4	26.4	22.9	10.3
	16.0	26.8	25.5	23.1	16.4			13.0	25.8	24.3	22.5	17.2	6.94
	20.0	23.6	21.1	18.2	11.5			15.0	23.7	21.5	19.4	13.8	5.54
	23.0	20.7	17.9	14.8	9.19			18.0	20.0	17.4	15.1	10.3	
	26.0	17.8	14.9	12.1	7.50			20.0	17.5	14.8	12.6	8.69	

Table III – 7

Nominal Axial Strength, P_n, kips [1,2]
Purlins/Girts
C-Sections With Lips

F_y = 55 ksi

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)

Section	KL_x ft.	Bracing ($KL_y = KL_t$)					Section	KL_x ft.	Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None			Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
6CS2.5x065	3.0	27.2	27.1	27.0	26.7	25.2	4CS2.5x070	2.0	30.0	29.9	29.8	29.6	28.6
	5.0	26.6	26.4	26.2	25.5	22.0		4.0	29.1	28.9	28.7	28.0	23.6
	8.0	25.5	25.1	24.6	23.2	13.1		6.0	27.9	27.4	26.9	25.2	15.2
	10.0	24.6	24.0	23.4	20.8	9.22		7.0	27.1	26.4	25.6	23.1	11.6
	13.0	23.0	21.8	20.4	15.6	6.18		9.0	24.9	23.4	21.9	17.9	7.40
	15.0	21.5	19.6	17.7	12.5	4.98		11.0	21.6	19.4	17.6	13.0	5.28
	18.0	18.1	15.8	13.6	9.26			12.0	19.7	17.3	15.5	11.0	4.60
	20.0	15.9	13.4	11.4	7.82			14.0	16.1	13.5	11.6	8.24	3.64
6CS2.5x059	3.0	23.7	23.6	23.5	23.2	21.6	4CS2.5x065	2.0	26.6	26.5	26.4	26.2	25.1
	5.0	23.2	23.0	22.8	22.0	18.6		4.0	25.6	25.4	25.2	24.6	21.2
	8.0	22.0	21.5	21.0	19.5	11.4		6.0	24.5	24.0	23.6	22.4	13.7
	10.0	20.9	20.3	19.6	17.9	7.99		7.0	23.8	23.2	22.7	20.9	10.6
	13.0	19.3	18.5	17.7	13.7	5.33		9.0	22.2	21.1	20.0	16.3	6.71
	15.0	18.3	17.2	15.6	10.9	4.30		11.0	19.7	17.7	16.1	11.9	4.76
	18.0	16.0	13.9	12.0	8.08			12.0	18.1	15.9	14.2	10.2	4.13
	20.0	14.0	11.8	10.0	6.81			14.0	14.8	12.4	10.8	7.58	3.25
4CS2.5x105	2.0	55.5†	55.3†	55.1†	54.6†	51.3	4CS2.5x059	2.0	23.2	23.1	23.1	22.9	21.8
	4.0	52.9	52.1	51.5	49.4	38.6		4.0	22.4	22.1	21.9	21.2	18.0
	6.0	48.8	47.2	45.8	41.5	25.2		6.0	21.1	20.6	20.1	18.9	12.0
	7.0	46.1	44.0	42.2	37.1	19.7		7.0	20.3	19.7	19.1	17.8	9.26
	9.0	40.0	37.0	34.6	28.4	13.2		9.0	18.7	18.0	17.4	14.4	5.94
	11.0	33.5	29.9	27.1	20.5	9.84		11.0	17.3	15.7	14.3	10.5	4.18
	12.0	30.3	26.4	23.6	17.5	8.75		12.0	16.1	14.1	12.6	9.04	3.61
	14.0	24.0	20.0	17.6	13.2	7.18		14.0	13.2	11.0	9.59	6.81	2.81
4CS2.5x085	2.0	41.5†	41.4†	41.3†	40.9†	39.2							
	4.0	40.1†	39.7	39.3	38.3	30.2							
	6.0	37.9	36.7	35.7	32.6	19.3							
	7.0	36.0	34.5	33.2	29.3	14.8							
	9.0	31.8	29.6	27.7	22.6	9.64							
	11.0	27.1	24.3	22.0	16.1	7.03							
	12.0	24.8	21.5	19.1	13.7	6.18							
	14.0	19.7	16.3	14.2	10.3	4.98							

Notes:

1. Axial strengths given are nominal strengths [resistances]. To obtain the available strengths [factored resistances], these values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD).
2. The nominal strengths [resistances] of members marked with the symbol † exceed the nominal distortional buckling strength of the member with no consideration of distortional buckling restraint from bracing or sheathing. In these cases, distortional buckling may control and the nominal strengths [resistances] listed above may be unconservative. See Table III-4 for distortional buckling strengths [resistances].

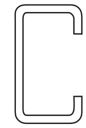
Table III – 8

F_y = 33 ksi

F_y = 50 ksi

Nominal Axial Strength, P_n, kips [1,2]
Joists/Studs
C-Sections With Lips

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)



Section	KL_x ft.	Bracing ($KL_y = KL_t$)					Section	KL_x ft.	Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None			Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
1200S200-97	5.0	46.6†	45.6†	44.4†	41.2†	27.6	1000S250-54	4.0	19.5†	19.5†	19.3†	19.0†	17.1†
	9.0	45.5†	42.3†	38.8†	30.6	11.8		8.0	19.1†	18.9†	18.5†	17.1†	11.2
	14.0	43.2†	36.2†	29.6	17.3			12.0	18.4†	18.0†	17.1†	14.4	5.80
	18.0	40.8†	30.6	21.9	11.8			16.0	17.5†	16.7†	15.3	11.2	
	23.0	37.2†	23.3	15.1				20.0	16.4†	15.3	13.6	7.80	
	27.0	34.0†	18.3	11.8				24.0	15.1	13.9	11.2	5.80	
	32.0	29.8	14.1					28.0	14.0	12.3	8.73	4.53	
	36.0	26.5	11.8					32.0	12.6	10.1	7.02		
1200S200-68	5.0	28.0†	27.6†	27.1†	25.5†	17.3	1000S250-43*	4.0	11.1†	11.1†	11.0†	10.9†	10.2†
	9.0	27.5†	26.1†	24.1†	19.1†	7.38		8.0	10.9†	10.9†	10.7†	10.2†	7.68
	14.0	26.5†	22.5†	18.5†	10.9			12.0	10.7†	10.5†	10.2†	9.09	4.31
	18.0	25.1†	19.1†	13.8	7.38			16.0	10.3†	10.0†	9.49†	7.68	
	23.0	22.8†	14.7	9.44				20.0	9.91†	9.46†	8.65	5.82	
	27.0	20.8†	11.5	7.38				24.0	9.40†	8.79	7.68	4.31	
	32.0	18.2†	8.84					28.0	8.84	8.05	6.42	3.35	
	36.0	16.2	7.38					32.0	8.21	7.10	5.23		
1200S200-54*	5.0	19.1†	18.9†	18.7†	18.0†	12.8†	1000S200-97	4.0	46.3†	45.8†	45.1†	43.1†	34.0
	9.0	18.9†	18.2†	17.4†	14.1†	5.41		8.0	44.9†	43.1†	40.5†	34.0	14.7
	14.0	18.4†	16.6†	13.7†	8.02			12.0	42.7†	39.0†	34.0	22.9	
	18.0	17.8†	14.1†	10.2	5.41			16.0	39.8†	34.0	26.6	14.7	
	23.0	16.7†	10.9	6.95				19.0	37.2†	29.9	21.1	11.2	
	27.0	15.3†	8.50	5.41				23.0	33.5	24.3	15.7		
	32.0	13.4†	6.50	4.16				27.0	29.6	19.1	12.2		
	36.0	11.8	5.41					31.0	25.7	15.4	9.74		
1000S250-97	4.0	49.0†	48.8†	48.5†	47.3†	42.0†	1000S200-68	4.0	27.9†	27.7†	27.4†	26.6†	21.3†
	8.0	47.8†	47.2†	45.8†	42.0†	23.6		8.0	27.3†	26.6†	25.2†	21.3†	9.30
	12.0	45.8†	44.6†	42.0†	33.2	12.3		12.0	26.4†	24.3†	21.3†	14.5	
	16.0	43.4†	41.5†	36.5	23.6			16.0	24.6†	21.3†	16.8	9.30	
	20.0	40.6†	37.2	30.0	16.5			20.0	22.5†	17.9	12.4	6.56	
	24.0	36.5	32.3	23.6	12.3			23.0	20.7†	15.4	9.95		
	28.0	32.2	27.4	18.4				27.0	18.3	12.2	7.74		
	32.0	27.9	22.7	14.9				31.0	15.8	9.78	6.23		
1000S250-68	4.0	27.9†	27.8†	27.7†	27.2†	24.8†	1000S200-54	4.0	19.0†	19.0†	18.8†	18.4†	15.7†
	8.0	27.3†	27.1†	26.5†	24.8†	15.0		8.0	18.7†	18.4†	17.8†	15.7†	6.86
	12.0	26.5†	25.9†	24.8†	20.5	7.84		12.0	18.2†	17.5†	15.7†	10.8	
	16.0	25.4†	24.4†	22.0	15.0			16.0	17.6†	15.7†	12.4	6.86	
	20.0	24.0†	22.1	19.0	10.5			20.0	16.5†	13.2	9.19	4.83	
	24.0	21.9	19.7	15.0	7.84			23.0	15.2†	11.4	7.34		
	28.0	19.8	16.8	11.7	6.13			27.0	13.4	9.01	5.70		
	32.0	17.4	13.8	9.46				31.0	11.6	7.22	4.58		

Table III – 8

Nominal Axial Strength, P_n, kips [1,2]

Joists/Studs

C-Sections With Lips

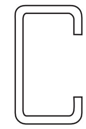
F_y = 33 ksi

F_y = 50 ksi

Ω_c = 1.80 (ASD)

ϕ_c = 0.85 (LRFD)

ϕ_c = 0.80 (LSD)



Section	KL_x ft.	Bracing ($KL_y = KL_t$)					Section	KL_x ft.	Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None			Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
1000S200-43*	4.0	10.9†	10.8†	10.7†	10.5†	9.18†	800S200-33*	4.0	7.04†	7.02†	6.99†	6.85†	6.16
	8.0	10.7†	10.5†	10.2†	9.18†	5.09		7.0	6.93†	6.87†	6.77†	6.37	4.50
	12.0	10.4†	9.97†	9.18†	7.16			10.0	6.76†	6.64†	6.44†	5.68	2.66
	16.0	10.0†	9.18†	7.87	5.09			13.0	6.54†	6.34	6.01	4.81	
	20.0	9.47†	8.21	6.46	3.55			16.0	6.26	5.98	5.50	3.83	
	23.0	8.96†	7.43	5.42				19.0	5.94	5.56	4.91	2.89	
	27.0	8.24	6.37	4.21				23.0	5.45	4.92	4.06	2.11	
	31.0	7.47	5.34	3.37				26.0	5.04	4.42	3.37		
800S200-97	4.0	45.3†	45.1†	44.6†	42.8†	34.3	800S162-97	4.0	33.7†	33.1†	32.3†	30.1	20.4
	7.0	43.9†	43.2†	41.6†	36.7	18.8		7.0	32.6†	31.0	28.7	23.0	8.88
	10.0	41.9†	40.5†	37.5	29.1	10.2		10.0	30.9	28.0	23.9	15.3	
	13.0	39.2	37.2	32.6	21.2			13.0	28.8	24.3	18.7	10.0	
	16.0	36.1	33.4	27.3	15.1			16.0	26.4	20.4	13.8	7.13	
	19.0	32.7	29.4	22.0	11.3			19.0	23.7	16.6	10.5		
	22.0	29.1	25.4	17.4	8.46			22.0	21.0	13.1	8.23		
	25.0	25.5	21.5	14.1				25.0	18.2	10.7	6.66		
800S200-68	4.0	27.6†	27.4†	27.2†	26.5†	21.6	800S162-68	4.0	21.3†	21.0†	20.5†	19.1†	13.2
	7.0	26.9†	26.6†	26.0†	23.1	12.1		7.0	20.6†	19.7†	18.3	14.8	5.83
	10.0	26.1†	25.2†	23.5†	18.4	6.80		10.0	19.5†	17.8	15.3	10.1	
	13.0	24.4†	23.1	20.5	13.6			13.0	18.2	15.6	12.1	6.57	
	16.0	22.5	20.7	17.3	9.74			16.0	16.6	13.2	9.06	4.71	
	19.0	20.4	18.1	14.1	7.39			19.0	14.9	10.8	6.85		
	22.0	18.1	15.6	11.2	5.83			22.0	13.2	8.61	5.41		
	25.0	15.9	13.1	9.12				25.0	11.5	7.00	4.41		
800S200-54	4.0	18.9†	18.8†	18.7†	18.3†	16.0	800S162-54	4.0	15.3†	15.2†	14.9†	14.2†	9.95
	7.0	18.6†	18.4†	18.1†	16.9†	9.00		7.0	14.9†	14.5†	13.7†	11.1	4.38
	10.0	18.1†	17.8†	17.1†	13.7	5.06		10.0	14.4†	13.4†	11.5	7.60	
	13.0	17.5†	16.8†	15.3	10.1			13.0	13.6†	11.7	9.15	4.94	
	16.0	16.5	15.2	12.9	7.25			16.0	12.4	9.95	6.85	3.53	
	19.0	15.0	13.3	10.5	5.49			19.0	11.1	8.17	5.16		
	22.0	13.4	11.4	8.35	4.34			22.0	9.83	6.51	4.06		
	26.0	11.2	8.97	6.37				25.0	8.51	5.27	3.30		
800S200-43	4.0	10.8†	10.8†	10.7†	10.5†	9.29	800S162-43	4.0	8.42†	8.35†	8.22†	7.86†	6.21
	7.0	10.6†	10.5†	10.4†	9.68†	6.38		7.0	8.23†	8.01†	7.64†	6.68	3.30
	10.0	10.3†	10.1†	9.79†	8.38	3.76		10.0	7.95†	7.52†	6.83	5.21	
	13.0	9.97†	9.64†	9.00	6.88			13.0	7.58†	6.91	5.88	3.73	
	16.0	9.50	8.96	8.06	5.39			16.0	7.14	6.21	4.87	2.65	
	19.0	8.89	8.19	7.05	4.09			19.0	6.65	5.46	3.89		
	23.0	7.99	7.10	5.71	3.00			22.0	6.12	4.70	3.05		
	26.0	7.29	6.28	4.75				25.0	5.56	3.97	2.48		

Table III – 8

Nominal Axial Strength, P_n, kips [1,2]
Joists/Studs
C-Sections With Lips

F_y = 33 ksi
F_y = 50 ksi

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)

Section	KL_x ft.	Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
800S162-33*	4.0	5.81†	5.77†	5.70†	5.52†	4.47
	7.0	5.71†	5.60†	5.41†	4.82	2.36
	10.0	5.56†	5.34†	4.92	3.74	
	13.0	5.37†	4.97	4.23	2.67	
	16.0	5.11†	4.47	3.50	1.88	
	19.0	4.78	3.93	2.79		
	22.0	4.39	3.38	2.18		
	25.0	3.98	2.85	1.76		
600S200-97	3.0	44.2†	44.0†	43.8†	42.9†	38.3
	5.0	43.0†	42.5	41.9	39.8	29.0
	8.0	40.3	39.3	37.9	33.5	14.7
	10.0	38.0	36.5	34.6	28.8	9.40
	12.0	35.4	33.3	31.0	23.9	
	15.0	31.0	28.3	25.5	16.7	
	17.0	27.9	24.9	21.9	13.0	
	20.0	23.2	19.9	17.0	9.40	
600S200-68	3.0	27.2†	27.1†	27.0†	26.6†	24.2
	5.0	26.7†	26.5†	26.2†	25.1	18.7
	8.0	25.5	24.8	23.9	21.0	10.0
	10.0	24.1	23.0	21.8	17.8	6.90
	13.0	21.5	20.0	18.2	13.3	
	15.0	19.7	17.9	15.9	10.8	
	17.0	17.7	15.7	13.6	8.92	
	20.0	14.8	12.6	10.5	6.90	
600S200-54	3.0	18.7†	18.7†	18.6†	18.4	17.4
	5.0	18.5	18.3	18.2	17.7	13.7
	8.0	17.9	17.6	17.3	15.5	7.58
	10.0	17.3	16.8	16.2	13.1	5.23
	13.0	16.0	14.9	13.5	9.67	
	15.0	14.6	13.3	11.7	7.73	
	17.0	13.2	11.6	9.95	6.37	
	20.0	11.0	9.31	7.68	4.98	
600S200-43	3.0	10.7†	10.7†	10.7†	10.6	9.91
	5.0	10.6	10.5	10.4	10.1	8.32
	8.0	10.2	10.1	9.84	9.04	5.48
	10.0	9.89	9.64	9.30	8.08	3.92
	13.0	9.25	8.80	8.24	6.56	
	15.0	8.71	8.16	7.49	5.56	
	18.0	7.84	7.14	6.33	4.22	
	20.0	7.23	6.45	5.57	3.58	

Section	KL_x ft.	Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
600S200-33	3.0	7.01	7.00	6.98	6.91	6.52
	5.0	6.92	6.88	6.83	6.64	5.65
	8.0	6.70	6.60	6.48	6.03	3.85
	10.0	6.50	6.36	6.17	5.51	2.71
	13.0	6.14	5.91	5.60	4.60	
	15.0	5.86	5.56	5.16	3.92	
	18.0	5.38	4.96	4.46	2.94	
	20.0	5.02	4.54	3.95	2.48	
600S162-97	3.0	32.9	32.7	32.4	31.2	25.5
	5.0	31.9	31.5	30.7	27.7	15.9
	7.0	30.6	29.8	28.4	23.2	8.65
	10.0	28.0	26.6	24.0	15.9	
	12.0	25.9	24.2	20.8	11.7	
	14.0	23.7	21.6	17.5	8.65	
	17.0	20.1	17.7	12.9	5.86	
	19.0	17.7	15.3	10.6		
600S162-68	3.0	21.0	20.9	20.7	20.0	16.6
	5.0	20.4	20.1	19.7	17.9	10.6
	8.0	19.1	18.4	17.4	13.6	4.87
	10.0	17.9	17.0	15.6	10.6	
	12.0	16.6	15.4	13.6	7.91	
	15.0	14.4	12.9	10.6	5.43	
	17.0	12.9	11.2	8.71	4.39	
	19.0	11.4	9.61	7.22		
600S162-54	3.0	15.1	15.1	15.0	14.7	12.5
	5.0	14.8	14.7	14.5	13.5	8.06
	8.0	14.2	13.8	13.0	10.3	3.72
	10.0	13.5	12.7	11.6	8.06	
	12.0	12.5	11.5	10.2	6.03	
	15.0	10.8	9.60	7.99	4.14	
	17.0	9.69	8.33	6.63	3.36	
	19.0	8.56	7.11	5.50		
600S162-43	3.0	8.37	8.34	8.29	8.12	7.19
	5.0	8.21	8.13	8.01	7.55	5.40
	8.0	7.85	7.66	7.37	6.34	2.82
	10.0	7.53	7.26	6.84	5.40	
	12.0	7.16	6.79	6.24	4.45	
	15.0	6.52	6.01	5.29	3.15	
	17.0	6.06	5.47	4.66	2.55	
	19.0	5.59	4.92	4.05	2.11	

Table III – 8

F_y = 33 ksi

F_y = 50 ksi

Nominal Axial Strength, P_n, kips [1,2]

Joists/Studs

C-Sections With Lips

Ω_c = 1.80 (ASD)

ϕ_c = 0.85 (LRFD)

ϕ_c = 0.80 (LSD)

Section	KL_x ft.	Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Section	KL_x ft.	Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
600S162-33	3.0	5.79	5.77	5.75	5.66	5.16	400S162-68	2.0	20.2	20.1	20.0	19.7	18.0
	5.0	5.71	5.67	5.60	5.38	3.91		4.0	19.3	19.0	18.7	17.6	12.6
	8.0	5.52	5.42	5.26	4.59	2.03		5.0	18.6	18.2	17.7	16.2	10.0
	10.0	5.36	5.19	4.93	3.91			7.0	17.0	16.3	15.5	13.1	5.60
	12.0	5.13	4.90	4.50	3.22			9.0	15.1	14.0	12.9	9.99	3.39
	15.0	4.71	4.33	3.80	2.27			10.0	14.0	12.8	11.6	8.46	
	17.0	4.38	3.93	3.33	1.83			12.0	11.9	10.4	9.06	6.17	
	20.0	3.85	3.33	2.66				14.0	9.67	7.99	6.79	4.78	
550S162-68	3.0	20.8	20.7	20.5	19.9	16.5	400S162-54	2.0	14.8	14.7	14.7	14.5	13.7
	5.0	20.1	19.8	19.4	17.8	10.7		4.0	14.3	14.2	14.0	13.4	9.50
	7.0	19.2	18.6	17.8	15.1	6.11		5.0	14.0	13.8	13.5	12.3	7.38
	9.0	18.0	17.1	16.0	12.1	3.88		7.0	13.0	12.4	11.8	9.90	4.37
	11.0	16.5	15.4	14.0	9.22			9.0	11.6	10.7	9.87	7.49	2.82
	14.0	14.2	12.7	11.0	6.11			10.0	10.8	9.81	8.88	6.38	2.29
	16.0	12.6	11.0	9.13	4.85			12.0	9.14	8.01	6.97	4.71	
	18.0	10.9	9.24	7.59	3.88			14.0	7.52	6.29	5.35	3.59	
550S162-54	3.0	15.1	15.0	14.9	14.6	12.5	400S162-43	2.0	8.19	8.17	8.14	8.06	7.58
	5.0	14.7	14.6	14.3	13.5	8.15		4.0	7.95	7.87	7.78	7.47	5.90
	7.0	14.2	14.0	13.4	11.5	4.69		5.0	7.78	7.65	7.52	7.06	4.95
	9.0	13.5	12.9	12.0	9.25	3.06		7.0	7.33	7.11	6.87	6.09	3.20
	11.0	12.5	11.6	10.5	7.08			9.0	6.78	6.44	6.09	5.03	2.16
	14.0	10.7	9.55	8.15	4.69			10.0	6.47	6.08	5.68	4.51	1.82
	16.0	9.47	8.19	6.70	3.74			12.0	5.81	5.32	4.83	3.51	
	18.0	8.25	6.89	5.54	3.06			14.0	5.12	4.54	4.00	2.73	
550S162-43	3.0	8.31	8.28	8.23	8.07	7.18	400S162-33	2.0	5.71	5.70	5.69	5.64	5.40
	5.0	8.13	8.05	7.93	7.51	5.43		4.0	5.59	5.55	5.50	5.34	4.30
	7.0	7.87	7.72	7.49	6.75	3.59		5.0	5.50	5.43	5.36	5.10	3.57
	9.0	7.54	7.30	6.95	5.88	2.34		7.0	5.27	5.13	4.98	4.44	2.25
	11.0	7.14	6.81	6.34	4.96			9.0	4.93	4.71	4.45	3.65	1.50
	14.0	6.46	5.98	5.36	3.59			10.0	4.73	4.44	4.14	3.26	1.27
	16.0	5.96	5.40	4.69	2.86			12.0	4.25	3.88	3.52	2.51	
	18.0	5.44	4.81	4.05	2.34			14.0	3.74	3.31	2.90	1.93	
550S162-33	3.0	5.78	5.76	5.74	5.65	5.17	362S162-68	2.0	19.8	19.7	19.6	19.3	17.4
	5.0	5.68	5.64	5.58	5.37	3.95		3.0	19.4	19.2	18.9	18.2	14.7
	7.0	5.55	5.47	5.35	4.90	2.61		5.0	18.0	17.5	17.0	15.3	8.86
	9.0	5.38	5.23	5.02	4.25	1.69		6.0	17.1	16.4	15.7	13.6	6.67
	11.0	5.14	4.93	4.60	3.58			8.0	15.0	13.9	12.9	10.1	4.10
	14.0	4.69	4.33	3.86	2.61			9.0	13.8	12.5	11.4	8.50	3.24
	16.0	4.32	3.90	3.37	2.07			11.0	11.3	9.79	8.57	5.99	
	18.0	3.94	3.47	2.89	1.69			12.0	10.1	8.50	7.29	5.17	

Table III – 8

F_y = 33 ksi
F_y = 50 ksi

Nominal Axial Strength, P_n, kips [1,2]
Joists/Studs
C-Sections With Lips

Ω_c = 1.80 (ASD)
ϕ_c = 0.85 (LRFD)
ϕ_c = 0.80 (LSD)

Section	KL_x ft.	Bracing ($KL_y = KL_t$)					Section	KL_x ft.	Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None			Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
362S162-54	2.0	14.6	14.5	14.5	14.3	13.4	350S162-43	2.0	8.04	8.01	7.98	7.89	7.36
	3.0	14.4	14.2	14.1	13.8	11.2		3.0	7.91	7.85	7.79	7.59	6.51
	5.0	13.7	13.4	13.0	11.8	6.76		5.0	7.52	7.37	7.21	6.72	4.52
	6.0	13.2	12.6	12.1	10.5	4.94		6.0	7.26	7.05	6.84	6.19	3.57
	8.0	11.6	10.8	10.0	7.86	3.11		8.0	6.64	6.31	5.99	5.04	2.18
	9.0	10.7	9.80	8.94	6.64	2.61		9.0	6.29	5.90	5.53	4.47	1.81
	11.0	8.94	7.82	6.85	4.60			11.0	5.54	5.04	4.58	3.36	
	12.0	8.05	6.87	5.83	3.94			12.0	5.15	4.60	4.12	2.86	
362S162-43	2.0	8.09	8.06	8.04	7.95	7.43	350S162-33	2.0	5.66	5.64	5.63	5.58	5.30
	4.0	7.80	7.71	7.61	7.28	5.64		3.0	5.59	5.56	5.53	5.43	4.79
	5.0	7.60	7.46	7.31	6.82	4.64		5.0	5.39	5.31	5.23	4.93	3.29
	7.0	7.08	6.82	6.56	5.76	2.88		6.0	5.26	5.13	5.01	4.56	2.56
	8.0	6.77	6.45	6.14	5.20	2.30		8.0	4.88	4.66	4.42	3.71	1.58
	10.0	6.08	5.65	5.23	4.08	1.61		9.0	4.65	4.36	4.08	3.27	1.28
	11.0	5.72	5.23	4.77	3.56			11.0	4.10	3.72	3.38	2.45	
	13.0	4.96	4.38	3.87	2.64			12.0	3.81	3.40	3.03	2.11	
362S162-33	2.0	5.66	5.65	5.64	5.59	5.32	250S162-68	2.0	18.7	18.5	18.4	17.8	15.2
	4.0	5.52	5.47	5.42	5.25	4.12		3.0	17.8	17.4	17.0	16.0	11.6
	5.0	5.41	5.34	5.27	4.97	3.37		4.0	16.5	15.9	15.4	13.9	8.31
	7.0	5.13	4.97	4.81	4.22	2.05		5.0	15.1	14.2	13.5	11.6	5.89
	8.0	4.94	4.74	4.51	3.80	1.64		6.0	13.5	12.4	11.5	9.35	4.51
	10.0	4.47	4.15	3.84	2.96	1.13		7.0	11.8	10.5	9.59	7.34	3.67
	11.0	4.20	3.84	3.50	2.57			8.0	10.1	8.76	7.78	5.77	3.10
	13.0	3.65	3.22	2.84	1.93			9.0	8.53	7.09	6.21	4.69	2.70
350S162-68	2.0	19.6	19.5	19.4	19.1	17.2	250S162-54	2.0	14.1	14.0	14.0	13.7	12.1
	3.0	19.1	18.9	18.7	18.0	14.3		3.0	13.7	13.5	13.3	12.7	9.21
	5.0	17.7	17.1	16.6	14.9	8.45		4.0	13.1	12.7	12.3	11.1	6.37
	6.0	16.7	16.0	15.3	13.1	6.36		5.0	12.1	11.4	10.9	9.31	4.35
	8.0	14.4	13.3	12.3	9.54	4.03		6.0	10.9	10.1	9.36	7.47	3.24
	9.0	13.2	11.9	10.7	7.97	3.18		7.0	9.65	8.59	7.78	5.78	2.57
	11.0	10.6	9.11	7.94	5.61			8.0	8.30	7.14	6.29	4.51	2.13
	12.0	9.36	7.83	6.73	4.84			9.0	7.00	5.79	5.01	3.63	1.82
350S162-54	2.0	14.5	14.5	14.4	14.2	13.2	250S162-43	2.0	7.86	7.82	7.77	7.64	6.92
	3.0	14.3	14.1	14.0	13.7	11.0		3.0	7.63	7.53	7.44	7.16	5.76
	5.0	13.5	13.3	12.8	11.6	6.44		4.0	7.31	7.14	6.99	6.54	4.52
	6.0	13.0	12.4	11.9	10.2	4.70		5.0	6.93	6.68	6.46	5.83	3.31
	8.0	11.3	10.5	9.69	7.54	2.97		6.0	6.48	6.15	5.87	5.08	2.41
	9.0	10.4	9.44	8.58	6.21	2.50		7.0	5.99	5.58	5.24	4.30	1.87
	11.0	8.56	7.41	6.37	4.31			8.0	5.47	4.99	4.59	3.54	1.52
	12.0	7.64	6.36	5.38	3.69			9.0	4.94	4.37	3.92	2.86	1.27

Table III – 8

F_y = 33 ksi

F_y = 50 ksi

Nominal Axial Strength, P_n, kips [1,2]

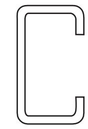
Joists/Studs

C-Sections With Lips

Ω_c = 1.80 (ASD)

ϕ_c = 0.85 (LRFD)

ϕ_c = 0.80 (LSD)



Section	KL_x ft.	Bracing ($KL_y = KL_t$)					Section	KL_x ft.	Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None			Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
250S162-33	2.0	5.56	5.54	5.52	5.45	5.05							
	3.0	5.44	5.39	5.34	5.20	4.26							
	4.0	5.28	5.19	5.10	4.82	3.32							
	5.0	5.06	4.91	4.77	4.32	2.43							
	6.0	4.79	4.56	4.35	3.76	1.76							
	7.0	4.45	4.14	3.89	3.20	1.34							
	8.0	4.07	3.71	3.42	2.66	1.06							
	9.0	3.68	3.27	2.95	2.16	0.876							

Notes:

1. Axial strengths given are nominal strengths [resistances]. To obtain the available strengths [factored resistances], these values must be modified by of safety factors (ASD) or resistance factors (LRFD, LSD).
2. The nominal strength [resistance] of members marked with the symbol † exceeds the nominal distortional buckling strength [resistance] of the member with no consideration of distortional buckling restraint from bracing or sheathing. In these cases, distortional buckling may control and the nominal strength [resistance] listed may be unconservative. See Table III-5 for distortional buckling strengths [resistances].

* Web h/t > 200

Table III – 9

F_y = 33 ksi

F_y = 50 ksi

Nominal Axial Strength, P_n, kips 1

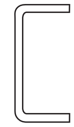
Tracks

C-Sections Without Lips

Ω_c = 1.80 (ASD)

ϕ_c = 0.85 (LRFD)

ϕ_c = 0.80 (LSD)



Section	KL_x ft.	Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Section	KL_x ft.	Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
1200T200-97	5.0	37.3	36.2	35.1	32.2	19.7	1000T200-54	4.0	12.2	12.1	11.9	11.4	8.84
	9.0	36.6	33.2	30.0	22.4			8.0	12.0	11.4	10.6	8.84	4.14
	14.0	35.2	27.6	21.5	11.9			12.0	11.6	10.2	8.84	5.89	
	18.0	33.7	22.4	15.0				16.0	11.0	8.84	6.79	4.14	
	23.0	31.4	16.0	10.4				19.0	10.5	7.70	5.52		
	27.0	29.2	12.6					23.0	9.71	6.20	4.36		
	32.0	26.3	9.78					27.0	8.87	5.11			
	36.0	23.9						31.0	7.98	4.30			
1200T200-68	5.0	19.2	18.6	18.1	16.7	10.6	1000T200-43*	4.0	6.27	6.21	6.15	5.97	5.07
	9.0	18.8	17.1	15.6	11.9			8.0	6.18	5.97	5.72	5.07	2.77
	14.0	18.1	14.5	11.5	6.98			12.0	6.03	5.58	5.07	3.86	
	18.0	17.3	11.9	8.49				16.0	5.84	5.07	4.28	2.77	
	23.0	16.2	8.95	6.19				19.0	5.66	4.64	3.65		
	27.0	15.1	7.31					23.0	5.38	4.02	2.91		
	32.0	13.7	5.81					27.0	5.07	3.39			
	36.0	12.5						31.0	4.74	2.87			
1200T200-54*	5.0	12.3	12.0	11.6	10.8	6.96	800T200-97	4.0	36.2	35.9	35.3	33.7	26.0
	9.0	12.1	11.0	10.1	7.79			7.0	35.4	34.3	32.7	28.2	12.9
	14.0	11.6	9.37	7.52	4.65			10.0	34.1	32.1	28.9	21.3	
	18.0	11.2	7.79	5.61				13.0	32.5	29.3	24.5	14.6	
	23.0	10.4	5.90	4.16				16.0	30.5	26.0	19.7	10.4	
	27.0	9.77	4.87					19.0	28.3	22.5	15.2		
	32.0	8.87	3.95					23.0	25.0	17.7	11.2		
	36.0	8.10						26.0	22.4	14.6	8.95		
1000T200-97	4.0	37.0	36.4	35.8	34.0	25.6	800T200-68	4.0	18.7	18.6	18.3	17.5	13.8
	8.0	36.1	34.0	31.6	25.6	10.1		7.0	18.3	17.9	17.0	14.9	7.61
	12.0	34.8	30.2	25.6	15.8			10.0	17.7	16.8	15.3	11.6	
	16.0	33.0	25.6	18.9	10.1			13.0	16.9	15.4	13.1	8.39	
	20.0	30.7	20.6	13.4				16.0	15.9	13.8	10.8	6.36	
	23.0	28.9	16.8	10.8				19.0	14.8	12.1	8.68		
	27.0	26.2	13.2					23.0	13.2	9.82	6.74		
	31.0	23.3	10.6					26.0	11.9	8.39	5.66		
1000T200-68	4.0	19.0	18.8	18.5	17.6	13.6	800T200-54	4.0	12.1	12.0	11.8	11.3	9.04
	8.0	18.6	17.6	16.5	13.6	6.11		7.0	11.8	11.5	11.0	9.71	5.11
	12.0	17.9	15.8	13.6	8.89			10.0	11.4	10.9	9.92	7.61	
	16.0	17.0	13.6	10.3	6.11			13.0	10.9	10.0	8.57	5.60	
	20.0	15.9	11.1	7.77				16.0	10.3	9.04	7.12	4.32	
	23.0	15.0	9.37	6.47				19.0	9.60	7.98	5.79		
	27.0	13.7	7.65					22.0	8.84	6.87	4.82		
	31.0	12.3	6.38					26.0	7.76	5.60	3.90		

Table III – 9

F_y = 33 ksi

F_y = 50 ksi

Nominal Axial Strength, P_n, kips [1]

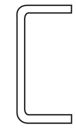
Tracks

C-Sections Without Lips

Ω_c = 1.80 (ASD)

ϕ_c = 0.85 (LRFD)

ϕ_c = 0.80 (LSD)



Section	KL_x ft.	Bracing ($KL_y = KL_t$)					Section	KL_x ft.	Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None			Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
800T200-43	4.0	6.20	6.16	6.11	5.94	5.13	800T125-43	3.0	6.05	5.93	5.83	5.53	4.15
	7.0	6.11	6.01	5.84	5.37	3.40		6.0	5.97	5.53	5.14	4.15	
	10.0	5.97	5.78	5.45	4.60			9.0	5.83	4.91	4.15	2.63	
	13.0	5.80	5.49	4.96	3.71			12.0	5.65	4.15	3.06		
	16.0	5.58	5.13	4.40	2.90			15.0	5.42	3.32	2.26		
	19.0	5.33	4.74	3.81				18.0	5.15	2.63			
	22.0	5.05	4.31	3.22				21.0	4.85				
	26.0	4.64	3.71	2.63				24.0	4.52				
800T200-33*	4.0	3.80	3.78	3.74	3.64	3.15	800T125-33*	3.0	3.73	3.66	3.60	3.42	2.59
	7.0	3.74	3.68	3.58	3.30	2.11		6.0	3.68	3.42	3.18	2.59	
	10.0	3.66	3.55	3.34	2.83			9.0	3.60	3.05	2.59	1.68	
	13.0	3.55	3.37	3.05	2.29			12.0	3.49	2.59	1.94		
	16.0	3.42	3.15	2.71	1.80			15.0	3.35	2.10	1.47		
	19.0	3.27	2.91	2.35	1.47			18.0	3.19	1.68			
	22.0	3.10	2.65	2.00				21.0	3.00				
	26.0	2.85	2.29	1.64				24.0	2.81				
800T125-97	3.0	33.8	32.5	31.3	28.2	15.9	600T200-97	3.0	35.2	35.0	34.8	34.0	29.8
	6.0	33.0	28.2	24.3	15.9			5.0	34.5	34.1	33.5	31.5	21.6
	9.0	31.6	22.2	15.9	8.45			8.0	32.9	32.0	30.6	26.0	9.82
	12.0	29.8	15.9	10.2				10.0	31.6	30.2	28.2	21.6	
	15.0	27.6	11.3	6.85				13.0	29.0	27.1	24.3	14.9	
	18.0	25.1	8.45					15.0	27.1	24.8	21.6	11.2	
	21.0	22.5						18.0	24.0	21.2	17.0	7.76	
	24.0	19.8						20.0	21.8	18.8	14.1		
800T125-68	3.0	18.1	17.6	17.0	15.6	9.51	600T200-68	3.0	18.4	18.3	18.2	17.8	15.8
	6.0	17.7	15.6	13.8	9.51			5.0	18.1	17.9	17.6	16.6	11.8
	9.0	17.1	12.8	9.51	5.22			8.0	17.3	16.8	16.1	13.9	6.51
	12.0	16.3	9.51	6.22				10.0	16.6	15.9	14.9	11.8	
	15.0	15.2	6.84	4.45				13.0	15.3	14.3	12.8	8.66	
	18.0	14.0	5.22					15.0	14.4	13.1	11.4	7.13	
	21.0	12.7						18.0	12.8	11.3	9.29	5.47	
	24.0	11.4						20.0	11.7	10.0	8.18		
800T125-54	3.0	11.8	11.5	11.1	10.3	6.53	600T200-54	3.0	11.9	11.9	11.8	11.6	10.3
	6.0	11.6	10.3	9.15	6.53			5.0	11.7	11.6	11.4	10.7	7.83
	9.0	11.2	8.53	6.53	3.71			8.0	11.2	10.9	10.5	9.05	4.46
	12.0	10.6	6.53	4.41				10.0	10.8	10.3	9.66	7.75	3.29
	15.0	9.98	4.85	3.17				13.0	9.99	9.31	8.33	5.83	
	18.0	9.23	3.71					15.0	9.39	8.56	7.39	4.85	
	21.0	8.41						18.0	8.40	7.37	6.04	3.81	
	24.0	7.55						20.0	7.71	6.56	5.33	3.29	

Table III – 9

Nominal Axial Strength, P_n, kips [1]

Tracks

C-Sections Without Lips

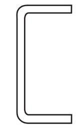
F_y = 33 ksi

F_y = 50 ksi

Ω_c = 1.80 (ASD)

ϕ_c = 0.85 (LRFD)

ϕ_c = 0.80 (LSD)



Section	KL_x ft.	Bracing ($KL_y = KL_t$)					Section	KL_x ft.	Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None			Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
600T200-43	3.0	6.11	6.09	6.07	5.99	5.55	600T125-43	3.0	5.94	5.86	5.77	5.52	4.30
	5.0	6.04	6.00	5.93	5.71	4.64		5.0	5.86	5.66	5.41	4.76	2.45
	8.0	5.87	5.77	5.60	5.09	3.01		7.0	5.74	5.36	4.91	3.81	
	10.0	5.72	5.56	5.32	4.59	2.26		10.0	5.50	4.76	3.98	2.45	
	13.0	5.44	5.19	4.82	3.78			12.0	5.30	4.30	3.30		
	15.0	5.22	4.91	4.45	3.25			14.0	5.07	3.81	2.70		
	18.0	4.86	4.45	3.88	2.59			17.0	4.68	3.05			
	20.0	4.60	4.13	3.49	2.26			19.0	4.40	2.63			
600T200-33	3.0	3.76	3.75	3.73	3.68	3.41	600T125-33	3.0	3.68	3.64	3.58	3.43	2.70
	5.0	3.71	3.69	3.65	3.51	2.86		5.0	3.63	3.51	3.36	2.97	1.58
	8.0	3.61	3.55	3.45	3.13	1.89		7.0	3.56	3.33	3.06	2.40	
	10.0	3.52	3.42	3.27	2.82	1.44		10.0	3.41	2.97	2.50	1.58	
	13.0	3.35	3.20	2.97	2.32			12.0	3.29	2.70	2.09		
	15.0	3.22	3.03	2.74	1.99			14.0	3.15	2.40	1.73		
	18.0	3.00	2.74	2.38	1.61			17.0	2.91	1.94			
	20.0	2.84	2.55	2.14	1.43			19.0	2.74	1.69			
600T125-97	3.0	32.6	31.7	30.7	28.0	16.8	550T150-68	3.0	17.8	17.7	17.4	16.7	13.1
	5.0	31.7	29.4	26.9	20.7	6.54		5.0	17.4	17.1	16.4	14.4	7.24
	7.0	30.5	26.3	22.0	13.2			7.0	16.8	16.2	14.9	11.6	3.99
	10.0	28.2	20.7	14.3	6.54			9.0	16.1	15.1	13.1	8.53	
	12.0	26.2	16.8	10.2				11.0	15.1	13.8	11.1	6.21	
	14.0	24.1	13.2	7.51				14.0	13.5	11.6	8.09	3.99	
	17.0	20.8	9.05					16.0	12.4	10.0	6.52		
	19.0	18.5	7.25					18.0	11.1	8.53	5.38		
600T125-68	3.0	17.7	17.3	16.8	15.6	10.2	550T150-54	3.0	11.6	11.6	11.4	10.9	8.67
	5.0	17.3	16.3	15.1	12.2	4.64		5.0	11.4	11.2	10.7	9.53	5.08
	7.0	16.7	14.9	12.9	8.14			7.0	11.0	10.6	9.80	7.74	3.08
	10.0	15.6	12.2	8.77	4.64			9.0	10.5	9.91	8.67	5.84	
	12.0	14.7	10.2	6.62				11.0	9.93	9.11	7.42	4.46	
	14.0	13.7	8.14	5.19				14.0	8.92	7.74	5.57	3.08	
	17.0	12.0	6.02					16.0	8.18	6.78	4.65		
	19.0	10.9	5.04					18.0	7.40	5.84	3.95		
600T125-54	3.0	11.6	11.3	11.1	10.3	6.98	550T150-43	3.0	5.98	5.95	5.90	5.74	4.95
	5.0	11.3	10.7	10.0	8.20	3.39		5.0	5.89	5.81	5.67	5.26	3.45
	7.0	11.0	9.85	8.59	5.76			7.0	5.76	5.62	5.35	4.60	2.22
	10.0	10.3	8.20	6.15	3.39			9.0	5.59	5.37	4.95	3.85	
	12.0	9.71	6.98	4.78				11.0	5.38	5.08	4.48	3.06	
	14.0	9.07	5.76	3.77				14.0	5.02	4.58	3.72	2.22	
	17.0	8.02	4.36					16.0	4.74	4.22	3.18		
	19.0	7.28	3.67					18.0	4.45	3.85	2.73		

Table III – 9

F_y = 33 ksi

F_y = 50 ksi

Nominal Axial Strength, P_n, kips [1]

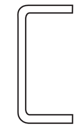
Tracks

C-Sections Without Lips

Ω_c = 1.80 (ASD)

ϕ_c = 0.85 (LRFD)

ϕ_c = 0.80 (LSD)



Section	KL_x ft.	Bracing ($KL_y = KL_t$)					Section	KL_x ft.	Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None			Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
550T150-33	3.0	3.70	3.68	3.65	3.55	3.07	362T125-68	2.0	16.7	16.6	16.5	16.0	13.8
	5.0	3.64	3.60	3.51	3.26	2.17		4.0	16.0	15.7	15.3	13.8	6.61
	7.0	3.56	3.48	3.32	2.86	1.43		5.0	15.6	15.1	14.4	12.2	4.23
	9.0	3.46	3.33	3.07	2.41			7.0	14.4	13.6	12.5	8.63	
	11.0	3.34	3.15	2.79	1.93			8.0	13.7	12.7	11.5	6.61	
	14.0	3.11	2.84	2.33	1.43			10.0	12.1	10.9	9.28	4.23	
	16.0	2.95	2.62	2.01				11.0	11.2	9.95	7.87	3.50	
	18.0	2.77	2.39	1.74				13.0	9.38	7.75	5.63		
400T125-68	2.0	17.0	16.9	16.7	16.3	13.9	362T125-54	2.0	11.1	11.0	11.0	10.7	9.28
	4.0	16.4	16.1	15.6	13.9	6.79		4.0	10.7	10.5	10.2	9.23	5.09
	5.0	16.0	15.6	14.8	12.3	4.34		5.0	10.4	10.1	9.63	8.31	3.41
	7.0	15.0	14.2	12.9	8.63			7.0	9.61	9.10	8.37	6.14	
	9.0	13.7	12.6	10.6	5.36			8.0	9.16	8.54	7.69	5.09	
	10.0	12.9	11.7	9.30	4.34			10.0	8.15	7.33	6.32	3.41	
	12.0	11.4	9.99	6.79	3.02			11.0	7.60	6.71	5.64	2.82	
	14.0	9.71	8.07	4.99				13.0	6.47	5.47	4.50		
400T125-54	2.0	11.2	11.2	11.1	10.8	9.33	362T125-43	2.0	5.70	5.68	5.65	5.55	5.07
	4.0	10.9	10.7	10.4	9.33	5.08		3.0	5.64	5.60	5.53	5.33	4.35
	5.0	10.6	10.3	9.87	8.34	3.51		5.0	5.45	5.35	5.19	4.70	2.59
	7.0	9.96	9.45	8.68	6.13			6.0	5.33	5.19	4.97	4.33	1.91
	9.0	9.14	8.41	7.26	4.22			8.0	5.03	4.79	4.46	3.47	
	10.0	8.68	7.85	6.51	3.51			9.0	4.85	4.57	4.18	3.01	
	12.0	7.69	6.70	5.08	2.44			11.0	4.46	4.09	3.61	2.25	
	14.0	6.64	5.56	3.97				12.0	4.24	3.84	3.32	1.91	
400T125-43	2.0	5.76	5.74	5.71	5.62	5.12	362T125-33	2.0	3.57	3.56	3.54	3.48	3.18
	4.0	5.64	5.58	5.47	5.12	3.47		3.0	3.53	3.51	3.47	3.35	2.75
	5.0	5.55	5.45	5.29	4.77	2.59		5.0	3.42	3.36	3.26	2.95	1.70
	7.0	5.33	5.14	4.86	3.93			6.0	3.35	3.26	3.12	2.72	1.33
	9.0	5.03	4.76	4.35	3.00			8.0	3.16	3.02	2.80	2.21	
	10.0	4.87	4.55	4.08	2.59			9.0	3.05	2.88	2.63	1.95	
	12.0	4.50	4.10	3.47	1.95			11.0	2.81	2.58	2.26	1.50	
	14.0	4.10	3.62	2.85				12.0	2.68	2.42	2.08	1.33	
400T125-33	2.0	3.60	3.59	3.57	3.52	3.21	350T125-68	2.0	16.6	16.5	16.4	15.9	13.7
	4.0	3.53	3.49	3.42	3.21	2.22		3.0	16.3	16.1	15.8	14.9	10.5
	5.0	3.47	3.41	3.31	3.00	1.69		5.0	15.4	14.9	14.3	12.2	4.19
	7.0	3.34	3.22	3.04	2.50			6.0	14.8	14.2	13.3	10.5	2.91
	9.0	3.16	2.99	2.72	1.94			8.0	13.4	12.5	11.3	6.54	
	10.0	3.06	2.86	2.55	1.69			9.0	12.6	11.5	10.3	5.17	
	12.0	2.83	2.57	2.20	1.32			11.0	10.8	9.56	7.79	3.46	
	14.0	2.59	2.28	1.85				12.0	9.92	8.38	6.54	2.91	

Table III – 9

F_y = 33 ksi

F_y = 50 ksi

Nominal Axial Strength, P_n, kips [1]

Tracks

C-Sections Without Lips

Ω_c = 1.80 (ASD)

ϕ_c = 0.85 (LRFD)

ϕ_c = 0.80 (LSD)

Section	KL_x ft.	Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
350T125-54	2.0	11.0	11.0	10.9	10.6	9.24
	3.0	10.9	10.7	10.6	9.96	7.24
	5.0	10.3	9.98	9.54	8.24	3.38
	6.0	9.91	9.50	8.91	7.24	2.35
	8.0	9.00	8.38	7.55	5.08	
	9.0	8.49	7.76	6.84	4.17	
	11.0	7.38	6.49	5.47	2.79	
	12.0	6.79	5.84	4.90	2.35	
350T125-43	2.0	5.67	5.65	5.62	5.53	5.04
	3.0	5.61	5.57	5.51	5.30	4.34
	5.0	5.41	5.31	5.15	4.66	2.59
	6.0	5.28	5.14	4.92	4.29	1.89
	8.0	4.96	4.73	4.40	3.47	
	9.0	4.78	4.50	4.11	3.01	
	11.0	4.36	4.00	3.53	2.25	
	12.0	4.14	3.73	3.24	1.89	
350T125-33	2.0	3.56	3.55	3.53	3.47	3.17
	3.0	3.52	3.50	3.46	3.33	2.73
	5.0	3.40	3.34	3.24	2.93	1.70
	6.0	3.32	3.23	3.10	2.69	1.33
	8.0	3.13	2.98	2.77	2.18	
	9.0	3.01	2.84	2.59	1.93	
	11.0	2.76	2.53	2.22	1.50	
	12.0	2.63	2.36	2.03	1.33	
250T125-68	2.0	15.4	15.2	15.1	14.6	12.0
	3.0	14.9	14.6	14.3	13.2	9.20
	4.0	14.2	13.7	13.1	11.5	5.96
	5.0	13.3	12.5	11.8	9.85	3.81
	6.0	12.2	11.2	10.4	8.33	2.65
	7.0	11.0	9.92	8.99	6.86	
	8.0	9.76	8.59	7.61	5.67	
	9.0	8.52	7.21	6.26	4.71	
250T125-54	2.0	10.4	10.3	10.2	9.90	8.30
	3.0	10.1	9.92	9.71	9.05	6.33
	4.0	9.66	9.37	9.04	8.03	4.54
	5.0	9.13	8.71	8.25	6.96	3.08
	6.0	8.52	7.96	7.39	5.75	2.14
	7.0	7.84	7.15	6.43	4.68	
	8.0	7.10	6.22	5.42	3.90	
	9.0	6.25	5.27	4.52	3.29	
250T125-43	2.0	5.38	5.35	5.31	5.20	4.61
	3.0	5.27	5.21	5.13	4.89	3.82
	4.0	5.12	5.01	4.89	4.51	2.99
	5.0	4.93	4.78	4.60	4.08	2.28
	6.0	4.72	4.50	4.28	3.63	1.73
	7.0	4.47	4.20	3.92	3.14	
	8.0	4.19	3.87	3.55	2.65	
	9.0	3.90	3.53	3.14	2.26	
250T125-33	2.0	3.42	3.40	3.38	3.31	2.94
	3.0	3.35	3.31	3.27	3.12	2.42
	4.0	3.26	3.20	3.12	2.88	1.89
	5.0	3.15	3.05	2.94	2.60	1.44
	6.0	3.01	2.88	2.74	2.31	1.14
	7.0	2.86	2.70	2.52	2.01	
	8.0	2.70	2.49	2.28	1.72	
	9.0	2.52	2.28	2.05	1.48	

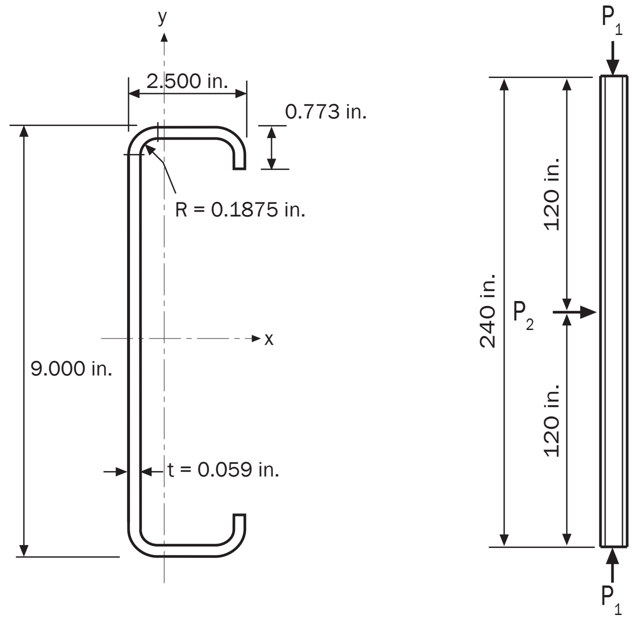
Note:

1. Axial strengths given are nominal strengths [resistances]. To obtain the available strengths [factored resistances], these values must be modified by safety factors (ASD) or resistance factors (LRFD, LSD).

* Web h/t > 200

SECTION 2 - EXAMPLE PROBLEMS

Example III-1: Braced C-Section With Lips - Bending and Compression



P_1 = 0.50 kip (dead)
= 2.0 kips (live)
P_2 = 0.10 kip (dead)
= 0.50 kip (live)

A_g = 0.881 in.^{2}
I_x = 10.3 in.^{4}
r_x = 3.42 in.

Given:

1. Steel: F_y = 55 ksi
2. Section: 9CS2.5x059 as shown above
3. Section simply supported at ends
4. Section fully braced against lateral-torsional, flexural-torsional and distortional buckling
5. K_x = 1.0; L_x = 240 in.

Required:

Verify the combined bending and compression strength of the section using ASD and LRFD methods with ASCE/SEI 7-10 load combinations.

Solution:

1. Refer to Example I-1 for derivation of geometric parameters.
2. Refer to Example I-8 for calculation of effective section properties.

1. Nominal Flexural Strength, M_n (Section C3.1)

Since the section is not subject to lateral-torsional or distortional buckling,

$$M_n = S_eF_y \qquad (Eq.\ C3.1.1\text{-}1)$$

where S_e is calculated with the extreme fibers at F_y

From Table II-1 or Example I-8, S_e = 1.89 in.^{3}

$$M_n = (1.89)(55) \qquad (Eq.\ C3.1.1\text{-}1)$$

$$= 104 \text{ kip-in.}$$

2. Nominal Axial Strength, P_n (Section C4.1)

Since the member can only buckle perpendicular to the x-axis,

$$F_e = \frac{\pi^2 E}{(KL_x/r_x)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2(29500)}{\left[(1.0)(240)/(3.42)\right]^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= 59.12 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{55}{59.12}}$$

$$= 0.965 < 1.5, \text{ therefore}$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4.1\text{-}2)$$

$$= \left(0.658^{0.965^2}\right)55 = 37.25 \text{ ksi}$$

In Example I-8, the effective area at f = 37.25 ksi was calculated as:

$$A_e = 0.515 \text{ in.}^2$$

$$P_n = A_eF_n \qquad (Eq.\ C4.1\text{-}1)$$

$$= (0.515)(37.25) = 19.2 \text{ kips}$$

3. Required Strength

$$M = \frac{P_2L}{4}$$

$$M_{dead} = \frac{(0.10)(240)}{4} = 6.00 \text{ kip-in.}$$

$$M_{live} = \frac{(0.50)(240)}{4} = 30.0 \text{ kip-in.}$$

ASD

$$M_x = M_{dead} + M_{live} = 6.0 + 30.0 = 36.0 \text{ kip-in.}$$

$$P = P_{dead} + P_{live} = 0.5 + 2.0 = 2.5 \text{ kips}$$

LRFD

$$M_{ux} = 1.2M_{dead} + 1.6M_{live} = (1.2)(6.0) + (1.6)(30.0) = 55.2 \text{ kip-in.}$$

$$P_u = 1.2P_{dead} + 1.6P_{live} = (1.2)(0.5) + (1.6)(2.0) = 3.80 \text{ kips}$$

4. Combined Compression and Bending - ASD (Section C5.2.1)

$\frac{\Omega_c P}{P_n} = \frac{(1.80)(2.5)}{19.2} = 0.234 > 0.15$, therefore use Equations C5.2.1-1 and C5.2.1-2.

$C_{mx} = 1.0$

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} \qquad (Eq.\ C5.2.1\text{-}6)$$

$$= \frac{\pi^2 (29500)(10.3)}{\left[(1.0)(240)\right]^2} = 52.1 \text{ kips}$$

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} > 0 \qquad (Eq.\ C5.2.1\text{-}4)$$

$$= 1 - \frac{(1.80)(2.5)}{52.1} = 0.914$$

$M_y = 0.0$

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} + \frac{\Omega_b C_{my} M_y}{M_{ny}\alpha_y} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

$$\frac{(1.80)(2.5)}{19.2} + \frac{(1.67)(1.0)(36.0)}{(104)(0.914)} = 0.867 < 1.0 \quad \text{OK}$$

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

From Table III-1, P_{no} = 24.3 kips

$$\frac{(1.80)(2.5)}{24.3} + \frac{(1.67)(36.0)}{104} = 0.763 < 1.0 \quad \text{OK} \qquad (Eq.\ C5.2.1\text{-}2)$$

5. Combined Compression and Bending - LRFD (Section C5.2.2)

$\overline{P} = P_u = 3.80$ kips

$\overline{M}_x = M_{ux} = 55.2$ kip-in.

$\frac{\overline{P}}{\phi_c P_n} = \frac{3.80}{(0.85)(19.2)} = 0.233 > 0.15$, therefore use Equations C5.2.2-1 and C5.2.2-2

$C_{mx} = 1.0$

P_{Ex}= 52.1 kips (computed in part 4 above)

$$\alpha_x = 1 - \frac{\overline{P}}{P_{Ex}} > 0 \qquad (Eq.\ C5.2.2\text{-}4)$$

$$= 1 - \frac{3.80}{52.1} = 0.927$$

$M_y = 0.0$

$$\frac{\overline{P}}{\phi_c P_n} + \frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x} + \frac{C_{my}\overline{M}_y}{\phi_b M_{ny}\alpha_y} \leq 1.0 \qquad (\textit{Eq. } \text{C5.2.2-1})$$

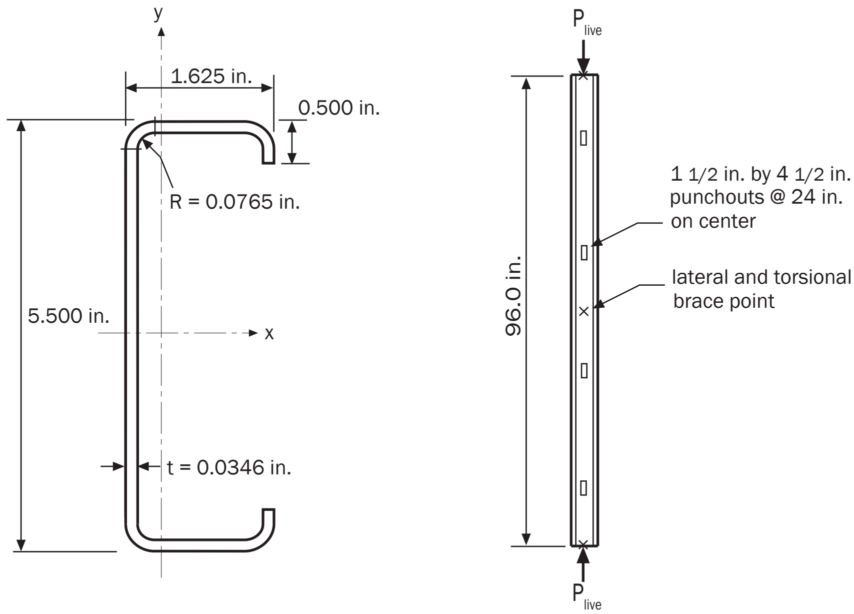
$$\frac{3.80}{(0.85)(19.2)} + \frac{(1.0)(55.2)}{(0.90)(104)(0.927)} = 0.869 < 1.0 \quad \text{OK}$$

$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \leq 1.0 \qquad (\textit{Eq. } \text{C5.2.2-2})$$

from Table III-1, P_{no} = 24.3 kips

$$\frac{3.80}{(0.85)(24.3)} + \frac{55.2}{(0.90)(104)} = 0.774 < 1.0 \quad \text{OK}$$

Example III-2: C-Section With Lips With Holes – Compression



$A = 0.327$ in.^{2}

$r_x = 2.11$ in.

$r_y = 0.589$ in.

$J = 0.000130$ in.^{4}

$C_w = 0.713$ in.^{6}

$x_0 = -1.11$ in.

$r_0 = 2.46$ in.

Given:

1. Steel: F_y = 33 ksi
2. Section: 550S162-33 as shown above
3. Concentrically loaded
4. Braced against buckling about the x-axis at ends only
5. Braced against buckling about the y-axis and for torsion at ends and mid-span
6. Braced against distortional buckling continuously
7. $K_x = K_y = K_t = 1.0$

Required:

1. Permitted applied load, P_{live}, using ASD and LRFD methods using the "all steel design" approach as described in Section D4.1 of the *Specification*.
2. Required brace strength and stiffness for the mid-span brace with P_{live} = 2.0 kips

Solution:

1. Axial Strength

a) Check flexural buckling (Section C4.1).

$$\frac{K_x L_x}{r_x} = \frac{(1.0)(96.0)}{2.11} = 45.5$$

$$\frac{K_y L_y}{r_y} = \frac{(1.0)(48.0)}{0.589} = 81.5$$

Since $\dfrac{K_y L_y}{r_y} > \dfrac{K_x L_x}{r_x}$, Euler buckling about the y-axis will control.

$$F_e = \frac{\pi^2 E}{\left(K_y L_y / r_y\right)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2 (29500)}{\left[(1.0)(48.0)/(0.589)\right]^2}$$

$$= 43.84 \text{ ksi}$$

b) Check flexural–torsional buckling (Section C4.1.2).

$$F_e = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right] \qquad (Eq.\ C4.1.2\text{-}1)$$

$$\beta = 1 - (x_o / r_o)^2 \qquad (Eq.\ C4.1.2\text{-}3)$$

$$= 1 - (-1.11/2.46)^2$$

$$= 0.796$$

$$\sigma_{ex} = \frac{\pi^2 E}{(K_x L_x / r_x)^2} \qquad (Eq.\ C3.1.2.1\text{-}11)$$

$$= \frac{\pi^2 (29500)}{\left[(1.0)(96.0)/2.11\right]^2}$$

$$= 140.7 \text{ ksi}$$

$$\sigma_t = \frac{1}{A r_o^2}\left[GJ + \frac{\pi^2 E C_w}{(K_t L_t)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$= \frac{1}{(0.327)(2.46)^2}\left[(11300)(0.000130) + \frac{\pi^2 (29500)(0.713)}{\left[(1.0)(48.0)\right]^2}\right]$$

$$= 46.27 \text{ ksi}$$

$$F_e = \frac{1}{(2)(0.796)}\left[(140.7 + 46.27) - \sqrt{(140.7 + 46.27)^2 - (4)(0.796)(140.7)(46.27)}\right] \qquad (Eq.\ C4.1.2\text{-}1)$$

$$= 42.52 \text{ ksi}$$

c) Determine controlling buckling mode.

42.52 ksi < 43.84 ksi, therefore flexural–torsional buckling governs

F_e = 42.52 ksi

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{33}{42.52}}$$

$$= 0.881 < 1.5\text{; therefore,}$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4.1\text{-}2)$$

$$= \left[0.658^{(0.881)^2}\right]33$$

$$= 23.85 \text{ ksi}$$

d) Compute effective area at $f = F_n = 23.85$ ksi

Check flange as a uniformly compressed element with an edge stiffener.

$$w = 1.625 - 2(0.0765 + 0.0346) = 1.403 \text{ in.}$$

$$w/t = 1.403/0.0346 = 40.5$$

$$S = 1.28\sqrt{E/f} \qquad (Eq.\ B4\text{-}7)$$

$$= 1.28\sqrt{29500/23.85} = 45.0\text{ ; therefore, } w/t \geq 0.328S \Rightarrow \text{check effective width of flange}$$

Compute flange k based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \leq t^4\left[115\frac{w/t}{S} + 5\right] \qquad (Eq.\ B4\text{-}8)$$

$$= 399(0.0346)^4\left[\frac{40.5}{45.0} - 0.328\right]^3 \leq (0.0346)^4\left[115\left(\frac{40.5}{45.0}\right) + 5\right]$$

$$= 0.000107 \text{ in.}^4 < 0.000156 \text{ in.}^4\text{ ; therefore, } I_a = 0.000107 \text{ in.}^4$$

$$d = 0.500 - 0.0765 - 0.0346 = 0.389 \text{ in.}$$

$$\theta = 90 \text{ degrees}$$

$$I_s = \left(d^3 t \sin^2\theta\right)/12 \qquad (Eq.\ B4\text{-}10)$$

$$= (0.389)^3(0.0346)\sin^2(90°)/12 = 0.000170 \text{ in.}^4$$

$$R_I = I_s/I_a \leq 1 \qquad (Eq.\ B4\text{-}9)$$

$$= 0.000170/0.000107 = 1.59 > 1\text{ ; therefore, } R_I = 1.0$$

$$D/w = 0.500/1.403 = 0.356 \qquad \text{(From Table B4-1)}$$

$0.25 < D/w \leq 0.8$; therefore,

$$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \leq 4 \qquad \text{(From Table B4-1)}$$

$$= \left(4.82 - \frac{(5)(0.500)}{1.403}\right)(1.0)^n + 0.43 = 3.47 < 4 \text{ OK}$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 3.47\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{40.5}\right)^2 = 56.41 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$= \sqrt{\dfrac{23.85}{56.41}}$ = 0.650 < 0.673; therefore, the flange is fully effective.

Check stiffener lip using Section B3.1

f = 23.85 ksi

k = 0.43

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 0.43\frac{\pi^2 (29500)}{12\left(1-0.3^2\right)}\left(\frac{0.0346}{0.389}\right)^2 = 90.70 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{23.85}{90.70}} = 0.513 < 0.673 \ \therefore \text{ lip is fully effective}$$

Check web with punchout

Per Section B2.2, treat web as two unstiffened elements, one on each side of the 1.50 inch wide punchout.

$$w = \left[5.50 - 2\left(0.0765 + 0.0346\right) - 1.50\right]/2 = 1.889 \text{ in.}$$

k = 0.43

$$F_{cr} = 0.43\frac{\pi^2 (29500)}{12\left(1-0.3^2\right)}\left(\frac{0.0346}{1.889}\right)^2 = 3.846 \text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{23.85}{3.846}} = 2.49 > 0.673$$

$$\rho = \left(1 - 0.22/\lambda\right)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$

$$= \left(1 - 0.22/2.49\right)/2.49 = 0.366$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$$= (0.366)(1.889) = 0.691 \text{ in.}$$

Compute A_e by subtracting the hole and ineffective area of the web from the gross section.

$$A_e = 0.327 - (0.0346)\left[1.50 + (2)(1.889 - 0.691)\right]$$

$$= 0.192 \text{ in.}^2$$

e) Compute P_n

$$P_n = A_e F_n \qquad (Eq.\ C4.1\text{-}1)$$

$$= (0.192)(23.85)$$

P_n = 4.58 kips

Alternatively, P_n may be interpolated from Table III-8. For the case of a 550S162-33 with a yield stress of 33 ksi, unbraced lengths of 9.0 feet and 7.0 feet in the x-axis and braced at mid-span, P_n = 4.25 kips and 4.90 kips, respectively. Interpolating for KL_x = 8.0 feet,

$$P_n = 4.25 + \frac{9.0 - 8.0}{9.0 - 7.0}(4.90 - 4.25) = 4.58 \text{ kips}$$

f) Compute maximum permissible applied live load, P_{live}

ASD

$$P_{live} \leq \frac{P_n}{\Omega_c} \text{ where } \Omega_c = 1.80 \quad (\textit{Eq. } \text{A4.1.1-1})$$

$$\leq \frac{4.58}{1.80}$$

$$P_{live} \leq 2.54 \text{ kips}$$

LRFD

$$1.6P_{live} \leq \phi_c P_n \text{ where } \phi_c = 0.85 \quad (\textit{Eq. } \text{A5.1.1-1})$$

$$P_{live} \leq \frac{(0.85)(4.58)}{1.6}$$

$$\leq 2.43 \text{ kips}$$

2. Required Strength and Stiffness of Mid-Span Brace (Section D3.3)

a) Calculate required brace strength for P_{live} = 2.0 kips

ASD

P_{ra} = 2.0 kips

$$P_{rb} = 0.01P_{ra} = 0.01(2.0) = 0.02 \text{ kip} \quad (\textit{Eq. } \text{D3.3-1})$$

LRFD

$$P_{ra} = 1.6P_{live} = 1.6(2.0) = 3.2 \text{ kips}$$

$$P_{rb} = 0.01P_{ra} = 0.01(3.2) = 0.032 \text{ kip} \quad (\textit{Eq. } \text{D3.3-1})$$

b) Calculate required brace stiffness for P_{live} = 2.0 kips

ASD

$$\beta_{rb} = \frac{2[4 - (2/n)]}{L_b}(\Omega P_{ra}) \quad (\textit{Eq. } \text{D3.3-2a})$$

n = 1 brace location

L_b = 48.0 in.

$\Omega = 2.00$

$$\beta_{br} = \frac{2[4 - (2/1)]}{48.0}(2.00)(2.0) = 0.333 \text{ kip per inch} \quad (\textit{Eq. } \text{D3.3-2a})$$

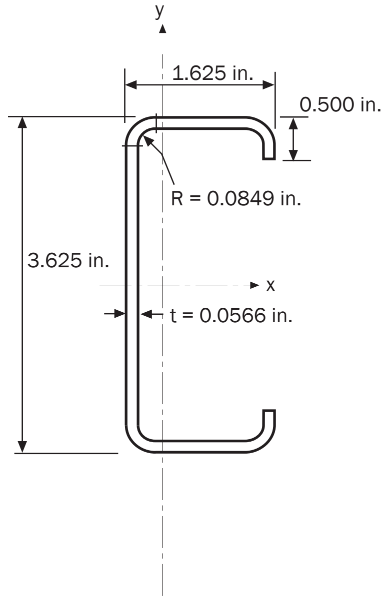
LRFD

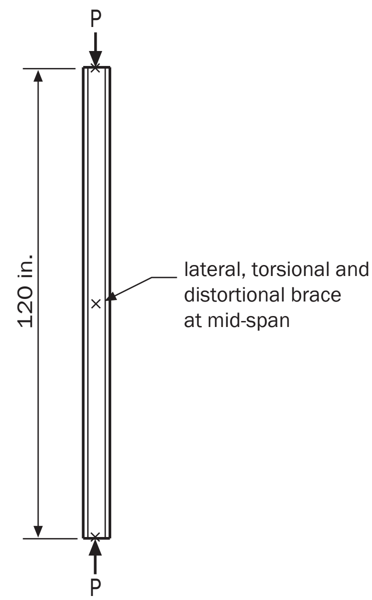
$$\beta_{rb} = \frac{2\left[4-(2/n)\right]}{L_b}\left(\frac{P_{ra}}{\phi}\right) \quad \text{(\textit{Eq}. D3.3-2b)}$$

$$\phi = 0.75$$

$$\beta_{br} = \frac{2\left[4-(2/1)\right]}{48.0}\left(\frac{3.2}{0.75}\right) = 0.356\ \text{kip per inch} \quad \text{(\textit{Eq}. D3.3-2b)}$$

Example III-3: C-Section Subject to Distortional Buckling – Compression





$A = 0.422$ in.^{2}
$r_x = 1.44$ in.
$r_y = 0.605$ in.
$J = 0.000451$ in.^{4}
$C_w = 0.457$ in.^{6}
$x_o = -1.28$ in.
$r_o = 2.02$ in.

Given:

1. Steel: $F_y = 50$ ksi
2. Section: 362S162-54 as shown above
3. Concentrically loaded
4. Braced against buckling about the x-axis at ends only
5. Braced against buckling about the y-axis and for torsion at ends and mid-span
6. $K_x = K_y = K_t = 1.0$
7. The rotational restraint providing by sheathing, k_ϕ, has been determined through testing in accordance with AISI S901-13 to be 0.0957 kip-in./rad/in.

Required:

1. Calculate the available strength using ASD and LRFD methods using the "all steel design" approach as described in Section D4.1 of the *Specification*. Consider distortional buckling.

Solution:

The available strength is the lower value calculated in accordance with sections C4.1 (nominal strength for yielding, flexural, flexural-torsional and torsional buckling) and C4.2 (distortional buckling strength).

1. Nominal Section Strength – Section C4.1

Compute the nominal axial strength using the least value of F_e for the limit states of flexural buckling (from Section C4.1.1) and flexural-torsional buckling (from Section C4.1.2).

a) Check flexural buckling (Section C4.1.1).

$$\frac{K_x L_x}{r_x} = \frac{(1.0)(120)}{1.44} = 83.3$$

$$\frac{K_y L_y}{r_y} = \frac{(1.0)(60.0)}{0.605} = 99.2$$

Since $\frac{K_y L_y}{r_y} > \frac{K_x L_x}{r_x}$, flexural buckling about the y-axis will control.

$$F_e = \frac{\pi^2 E}{\left(K_y L_y / r_y\right)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2 (29500)}{(99.2)^2} = 29.6 \text{ ksi}$$

b) Check flexural–torsional buckling (Section C4.1.2).

$$F_e = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right] \qquad (Eq.\ C4.1.2\text{-}1)$$

where

$$\beta = 1 - (x_o / r_o)^2 \qquad (Eq.\ C4.1.2\text{-}3)$$

$$= 1 - (-1.28/2.02)^2 = 0.598$$

$$\sigma_{ex} = \frac{\pi^2 E}{\left(K_x L_x / r_x\right)^2} \qquad (Eq.\ C3.1.2.1\text{-}11)$$

$$= \frac{\pi^2 (29500)}{(83.3)^2} = 42.0 \text{ ksi}$$

$$\sigma_t = \frac{1}{A r_o^2}\left[GJ + \frac{\pi^2 E C_w}{(K_t L_t)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$= \frac{1}{(0.422)(2.02)^2}\left[(11300)(0.000451) + \frac{\pi^2 (29500)(0.457)}{\left[(1.0)(60.0)\right]^2}\right] = 24.4 \text{ ksi}$$

$$F_e = \frac{1}{(2)(0.598)}\left[(42.0 + 24.4) - \sqrt{(42.0 + 24.4)^2 - (4)(0.598)(42.0)(24.4)}\right] \qquad (Eq.\ C4.1.2\text{-}1)$$

$$= 18.5 \text{ ksi}$$

c) Determine controlling buckling mode (Section C4.1).

18.5 ksi < 29.6 ksi; therefore, flexural–torsional buckling governs.

$F_e = 18.5$ ksi

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{50}{18.5}} = 1.64 > 1.5\text{; therefore,}$$

$$F_n = \left[\frac{0.877}{\lambda_c^{\,2}}\right]F_y \qquad (Eq.\ C4.1\text{-}3)$$

$$= \left[\frac{0.877}{(1.64)^2}\right]50 = 16.3\text{ ksi}$$

d) Calculate the effective area at $f = F_n = 16.3$ ksi.

It can be established by calculations not shown that at $f = F_n = 16.3$ ksi, the section effective area, $A_e = 0.414$ in.^{2}.

e) Nominal axial strength, P_n

$$P_n = A_eF_n = 0.414(16.3) = 6.75\text{ kips} \qquad (Eq.\ C4.1\text{-}1)$$

f) Available strengths

ASD allowable strength

$$\frac{P_n}{\Omega_c} = \frac{6.75}{1.80} = 3.75\text{ kips}$$

LRFD design strength

$$\phi_c P_n = 0.85(6.75) = 5.74\text{ kips}$$

2. Distortional Buckling Strength – Section C4.2

The available distortional buckling strength is calculated using Section C4.2. The strength is a function of the elastic critical distortional buckling load, P_{crd}, and the load at first yield, P_y, as defined in Equations C4.2-1 through C4.2-5.

Calculate and compare the distortional buckling capacities predicted by subsections C4.2(a) and C4.2(b)

Distortional buckling using Section C4.2(a)

Section C4.2(a) provides a precise, but involved calculation for the elastic distortional buckling stress, F_d. Tabulated geometric properties are provided in *Manual* Tables III-4 through III-6 for standard sections; however, the method is still quite involved and engineers are instead encouraged to use Section C4.2(b) with a computational analysis instead of this hand method.

The analytical model for predicting distortional buckling in C4.2(a) considers flexural-torsional buckling of the flange as a column restrained at the web/flange juncture by the available rotational stiffness from bending/buckling of the web plate. Cross-section properties of the flange itself must be calculated. The *Specification* Commentary provides formulae for these properties in Table C-C3.1.4(a)-1. These formulae may also be found in Part I, Section 3.4 of this *Manual*. Example II-5 illustrates the calculation of the required flange section properties of a similar section in detail. In the interest of brevity, the following calculations use precomputed distortional buckling coefficients taken from Table III-5.

Determine the distortional buckling strength from Table III-5, without sheathing.

If the distortional buckling unbraced length, L_m, equals or exceeds the distortional buckling half-wavelength, L_{cr}, the nominal distortional buckling strength, P_n, may be taken directly from Table III-5.

L_{cr} = 13.3 in. < 60.0 in.; therefore, (from Table III-5)

P_n = 16.7 kips (from Table III-5)

Determine the distortional buckling strength from Table III-5 considering sheathing.

If the member is sheathed and the rotational stiffness of the sheathing is known, additional strength can be calculated using *Eq*. C4.2-6. Using the given value based on testing in accordance with AISI S901-13, the rotational stiffness contributed by the sheathing, k_ϕ, is 0.0957 kip-in./rad/in.

a) Calculate F_d, the elastic distortional buckling stress

$$F_d = \frac{k_{\phi fe} + k_{\phi we} + k_\phi}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \quad (Eq.\ C4.2\text{-}6)$$

where

$k_{\phi fe}$ = 0.348 kip-in./rad/in. (from Table III-5)

$k_{\phi we}$ = 0.270 kip-in./rad/in. (from Table III-5)

$\tilde{k}_{\phi fg}$ = 0.00823 in.^{2} (from Table III-5)

$\tilde{k}_{\phi wg}$ = 0.00251 in.^{2} (from Table III-5)

$$F_d = \frac{0.348 + 0.270 + 0.0957}{0.00823 + 0.00251} = 66.5 \text{ ksi}$$

b) Calculate P_n, the nominal distortional buckling strength

$$P_{crd} = A_g F_d \quad (Eq.\ C4.2\text{-}5)$$

$$= (0.422)(66.5) = 28.1 \text{ kips}$$

$$P_y = A_g F_y = (0.422)(50) = 21.1 \text{ kips} \quad (Eq.\ C4.2\text{-}4)$$

$$\lambda_d = \sqrt{\frac{P_y}{P_{crd}}} \quad (Eq.\ C4.2\text{-}3)$$

$$= \sqrt{\frac{21.1}{28.1}} = 0.867$$

Since $\lambda_d > 0.561$

$$P_n = \left(1 - 0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right)\left(\frac{P_{crd}}{P_y}\right)^{0.6} P_y \quad (Eq.\ C4.2\text{-}2)$$

$$= \left(1 - 0.25\left(\frac{28.1}{21.1}\right)^{0.6}\right)\left(\frac{28.1}{21.1}\right)^{0.6} 21.1 = 17.6 \text{ kips}$$

Compared with the Section C4.2(a) solution without sheathing, the nominal distortional buckling strength is increased by approximately 5% by the sheathing, but flexural-torsional buckling still controls the strength of the member.

Distortional buckling using Section C4.2(b)

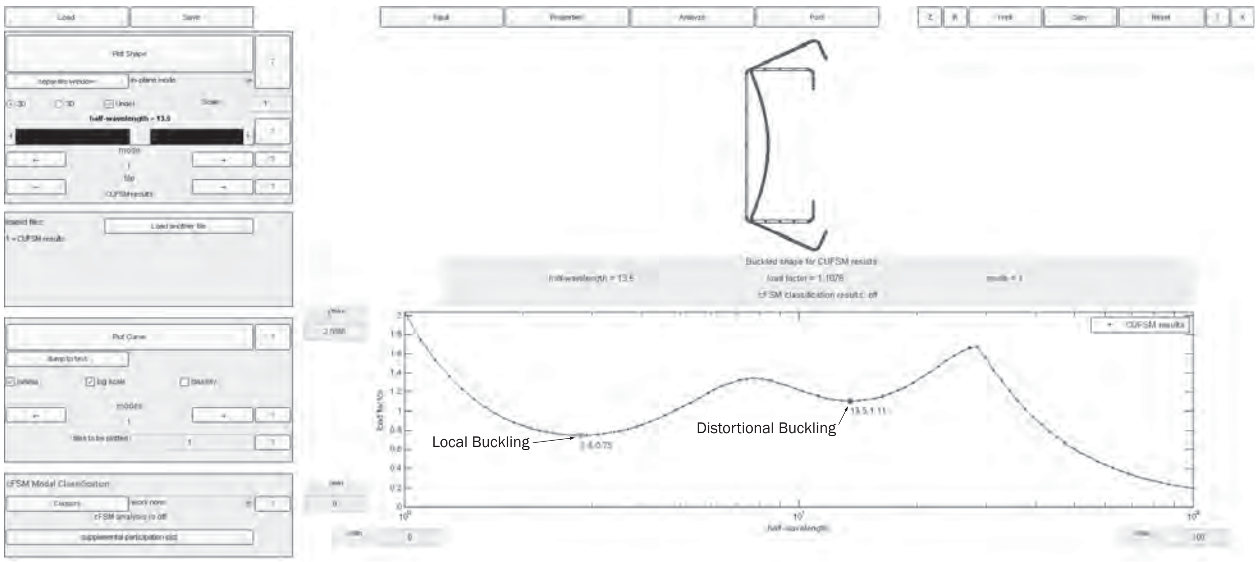
Section C4.2(b) permits the use of "rational analysis" to calculate the distortional buckling stress, F_d. The following solution uses the software program CUFSM*. Similar solutions can be obtained from other commercial programs.

The dimensions in the model shown below match the flat lengths and radii of the standard 362S162-54 section; consequently, the resulting gross section properties closely match those given in Table I-2. For the initial model, no rotational restraint from sheathing is included.

The analysis is conducted with a uniform compression yield stress of 50 ksi applied to the section. The resulting buckling curve and buckling mode shape at the distortional buckling half-wavelength are shown in the figure below. Distortional buckling is found to occur at a half-wavelength of 13.5 in. and at a load factor of 1.11.

---

* CUFSM is a free, open source program using the semi-analytical finite strip method for determination of thin-walled member stability. The program, along with tutorials, etc., may be found at www.ce.jhu.edu/bschafer/cufsm.



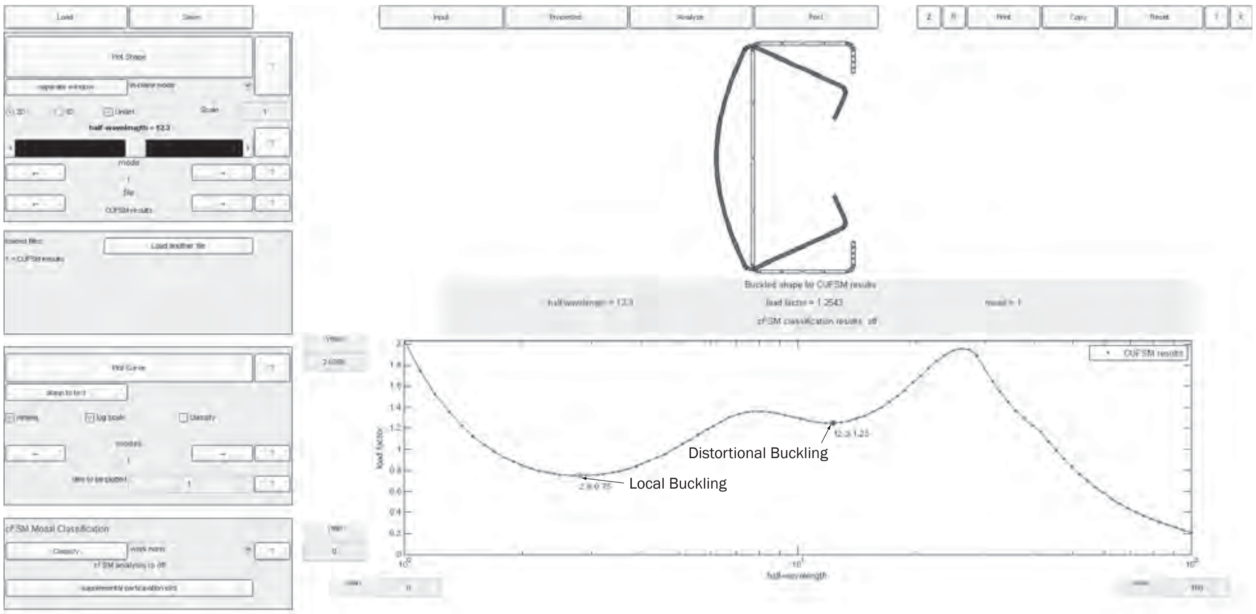
Thus, the elastic distortional buckling load, P_{crd}, is $1.11P_y$. The elastic distortional buckling stress, F_d, is calculated as:

$$F_d = \frac{P_{crd}}{A_g} = \frac{1.11P_y}{A_g}$$

$$= \frac{1.11(21.1)}{0.422} = 55.5\,\text{ksi}$$

The nominal distortional buckling strength can then be calculated from F_d using *Eqs*. C4.2-1 through C4.2-5.

If a rotational restraint of 0.0957 kip-in./rad/in. is added to the model at the mid-point of the flanges, the half-wavelength decreases to 12.3 in. and the load factor increases to 1.25, as shown in the figure below.



Thus, the elastic distortional buckling load., P_{crd}, is $1.25P_y$. The elastic distortional buckling stress, F_d, is calculated as:

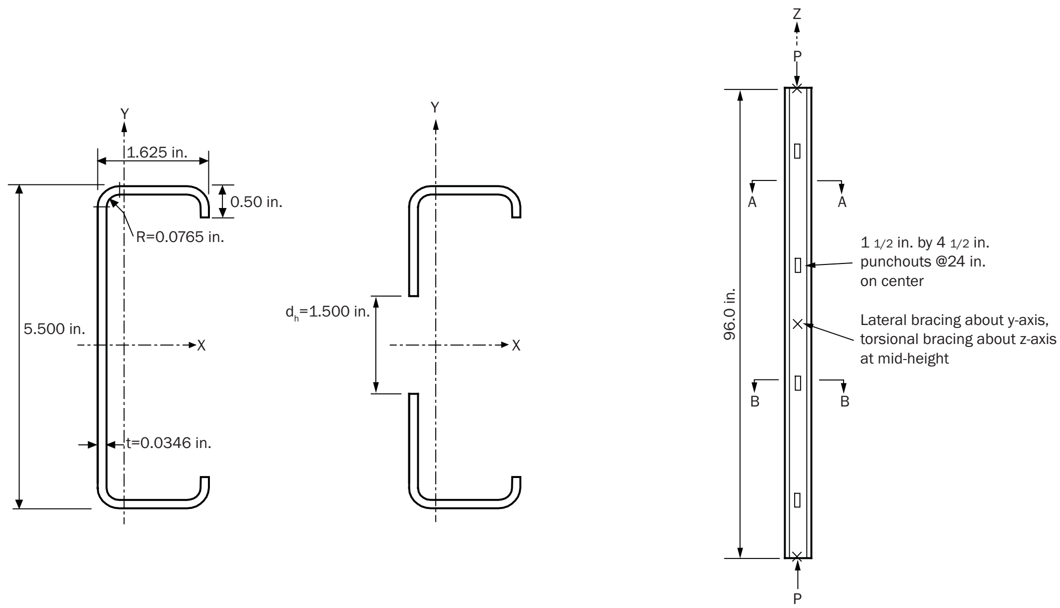
$$F_d = \frac{P_{crd}}{A_g} = \frac{1.25P_y}{A_g}$$

$$= \frac{1.25(21.1)}{0.422} = 62.5\,\text{ksi}$$

The nominal distortional buckling strength can then be calculated from F_d using *Eqs.* C4.2-1 through C4.2-5.

It can be seen that the results from the numerical analysis agree well, but not exactly, with the provisions of Section C4.2(a).

Example III-4: C-Section With Lips With Holes – Compression – Direct Strength Method



Given:

1. Steel: F_y = 33 ksi
2. Section: 550S162-33 as shown above
3. Concentrically loaded
4. Braced against x-axis global buckling at ends. Braced against lateral buckling about the y-axis and torsional buckling about the z-axis at ends and mid-span
5. $K_x = K_y = K_t = 1.0$, L_x = 96.0 in., $L_y = L_t$ = 48.0 in.
6. Cross-section properties

   Section properties for the gross section may be taken from Table I-2. Section properties for the net section may be calculated using CUFSM* by setting the cross-section thickness to zero at the hole.

A	=	0.327 in.^{2}	A_{net}	=	0.275 in.^{2}
I_x	=	1.46 in.^{4}	$I_{x,net}$	=	1.45 in.^{4}
I_y	=	0.113 in.^{4}	$I_{y,net}$	=	0.100 in.^{4}
r_x	=	2.11 in.	$r_{x,net}$	=	2.29 in.
r_y	=	0.589 in.	$r_{y,net}$	=	0.610 in.
x_o	=	-1.11 in.	$x_{o,net}$	=	-1.20 in.
r_o	=	2.46 in.	$r_{o,net}$	=	2.66 in.
J	=	0.000130 in.^{4}	J_{net}	=	0.000110 in.^{4}
C_w	=	0.713 in.^{6}	$C_{w,net}$	=	0.677 in.^{6}

* CUFSM is a free, open source program using the semi-analytical finite strip method for determination of thin-walled member stability. The program, along with tutorials, etc., may be found at www.ce.jhu.edu/bschafer/cufsm.

Required:

Available compression strengths, ϕP_n (LRFD) and $\frac{P_n}{\Omega}$ (ASD) using the "all steel design" approach as described in Section D4.1 and the Direct Strength Method of Appendix 1 of the *Specification*.

Solution:

1. Determine the Critical Elastic Column Local Buckling Load, $P_{cr\ell}$ (Appendix 1)

a) Determine the critical elastic flange local buckling stress (*Commentary* Section 1.1.2.2)

$k_{flange} = 4.0$ (Table C-B2-1)

$w = 1.625 - 2(0.0765 + 0.0346) = 1.40$ in.

$w/t = 1.40 / 0.0346 = 40.5$

$$f_{cr\ell,flange} = k_{flange}\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 = 4.0\frac{\pi^2(29500)}{12(1-0.3^2)}(40.5)^{-2} = 65.0\text{ ksi}$$ (From *Eq.* C-1.1.2-5)

b) Determine the critical elastic web local buckling stress (*Commentary* Section 1.1.2.2)

The local buckling stress in the web without considering the effect of the hole is calculated as follows:

$k_{web} = 4.0$ (Table C-B2-1)

$w = 5.500 - 2(0.0346 + 0.0765) = 5.28$ in.

$w / t = 5.28 / 0.0346 = 153$

$$f_{cr\ell,web} = k_{web}\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 = 4.0\frac{\pi^2(29500)}{12(1-0.3^2)}(153)^{-2} = 4.56\text{ ksi}$$ (From *Eq.* C-1.1.2-5)

The local buckling stress in the web considering the effect of the hole is calculated as follows in accordance with Moen and Schafer, 2009.*

Check limits of applicability based on spacing, s = 24.0 in.:

$s / L_h = 24.0 / 4.50 = 5.33 > 2$ OK

$s / h = 24.0 / 5.278 = 4.55 > 1.5$ OK

Determine the widths of unstiffened strips on either side of the hole, h_A and h_B.

$$h_A = h_B = \frac{h - d_h}{2} = \frac{5.28 - 1.50}{2} = 1.89\text{ in.}$$

$L_h / h_A = L_h / h_B = 4.50 / 1.89 = 2.38 > 1.0$; therefore, k_A and k_B are determined by:

$$k_A = k_B = 0.425 + \frac{0.20}{\left(\frac{L_h}{h_A}\right)^{0.95} - 0.6} = 0.425 + \frac{0.20}{2.38^{0.95} - 0.6} = 0.544$$

* Moen, C.D. and B.W. Schafer (2009), "Elastic Buckling of Thin Plates With Holes in Compression or Bending," *Thin-Walled Structures*, 47(12), pp. 1597-1607, 2009.

The critical stresses of the unstiffened strips are given by:

$$f_{crA} = f_{crB} = k_A \frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{h_A}\right)^2 = 0.544\frac{\pi^2 29500}{12\left(1-0.3^2\right)}\left(\frac{0.0346}{1.89}\right)^2 = 4.86\,\text{ksi}$$

The critical buckling stress of the net web section is taken as the minimum buckling stress of the two unstiffened segments, f_{crA} and f_{crB}:

$$f_{crh,net} = f_{crA} = 4.86\text{ ksi}$$

The buckling stress of the net web section is then converted to the gross web section:

$$f_{crh} = f_{crh,net}\left(1-\frac{d_h}{h}\right) = 4.86\left(1-\frac{1.50}{5.28}\right) = 3.48\text{ ksi}$$

The buckling stress of the web is the minimum stress considering buckling in the web away from the holes and buckling of the unstiffened segments adjacent to the holes:

$$f_{cr\ell,web} = \min\left(4.56, 3.48\right) = 3.48\text{ ksi}$$

c) Determine the critical elastic lip local buckling stress (*Commentary* Section 1.1.2.2)

$$k_{lip} = 0.425 \qquad \text{(Table C-B2-1)}$$

$$w = 0.500 - 0.0346 - 0.0765 = 0.389\text{ in.}$$

$$w/t = 0.389 / 0.0346 = 11.2$$

$$f_{cr\ell,lip} = k_{lip}\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 = 0.425\frac{\pi^2\left(29500\right)}{12\left(1-0.3^2\right)}\left(11.2\right)^{-2} = 90.3\text{ ksi} \qquad \text{(From } Eq.\text{ C-1.1.2-5)}$$

d) Determine the critical buckling load due to local buckling

The local buckling stress of the whole section, $f_{cr\ell}$, is taken as the minimum local buckling stress for each of the elements:

$$f_{cr\ell} = \min\left(f_{cr\ell,flange}, f_{cr\ell,web}, f_{cr\ell,lip}\right) = \min\left(65.0, 3.48, 90.3\right) = 3.48\text{ ksi}$$

The critical buckling force, $P_{cr\ell}$, is controlled by local buckling in the unstiffened strip.

$$P_{cr\ell} = A_g f_{cr\ell} = 0.327\left(3.48\right) = 1.14\text{ kips} \qquad (Eq.\text{ C-1.1.2-3})$$

2. Determine the Critical Elastic Column Distortional Buckling Load, P_{crd} (Appendix 1)

Since there is no rotational restraint provided at the flanges, $k_\phi = 0$.

The critical half-wavelength at which distortional buckling occurs and flange properties can be taken from Table III-5 for a 550S162-33 Joist/Stud with F_y = 33 ksi:

$$L_{crd} = 19.3\text{ in.}$$

$$k_{\phi fe} = 0.0493\text{ kip}$$

$$\tilde{k}_{\phi fg} = 0.00251\text{ in.}^2$$

The distortional buckling properties for the web must consider the reduction in web stiffness from the influence of the holes. The reduced thickness, t_r, is calculated by:

$$t_r = \left(1-\frac{L_h}{L_{crd}}\right)^{1/3} t = \left(1-\frac{4.50}{19.3}\right)^{1/3}\left(0.0346\right) = 0.0317\text{ in.} \qquad (Eq.\text{ C-1.1.2-2})$$

$$k_{\phi we} = \frac{Et_r^3}{6h_o\left(1-\mu^2\right)} = \frac{29500\left(0.0317\right)^3}{6\left(5.500\right)\left(1-0.3^2\right)} = 0.0313 \text{ kip} \qquad \text{(From } Eq. \text{ C4.2-7)}$$

$$L = \min(L_{cr}, L_m) = \min(19.3, 48.0) = 19.3 \text{ in.}$$

$$\tilde{k}_{\phi wg} = \left(\frac{\pi}{L}\right)^2 \frac{t_r h_o^3}{60} = \left(\frac{\pi}{19.3}\right)^2 \frac{0.0317\left(5.500\right)^3}{60} = 0.00233 \text{ in.}^2 \qquad \text{(From } Eq.\text{C4.2-8)}$$

$$f_{crd} = \frac{k_{\phi fe} + k_{\phi we} + k_{\phi}}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} = \frac{0.0493 + 0.0313 + 0}{0.00251 + 0.00233} = 16.7 \text{ ksi} \qquad \text{(From } Eq.\text{C4.2-6)}$$

$$P_{crd} = Af_{crd} = 0.327\left(16.7\right) = 5.46 \text{ kips} \qquad (Eq. \text{ C-1.1.2-6})$$

The pure distortional buckling stress calculated by hand does not include the interaction between local and distortional buckling and will therefore typically yield buckling stresses higher than that calculated using a finite strip analysis.

3. Determine Critical Elastic Column Global Buckling Load, P_{cre} (Appendix 1)

A weighted average approach as described in *Commentary* Section 1.1.2.2 is used to approximate the global elastic column global buckling load.

a) Calculate the cumulative lengths of gross and net cross-sections with respect to each unbraced length where, n, is the number of holes within the unbraced length.

$$L_{xg} = L_x - nL_h = 96.0 - 4\left(4.5\right) = 78.0 \text{ in.}$$

$$L_{xnet} = \sum_{j=1}^{n} L_{h,j} = 4\left(4.50\right) = 18.0 \text{ in.} \qquad (Eq. \text{ C-1.1.2-13})$$

$$L_{yg} = L_y - nL_h = 48.0 - 2\left(4.50\right) = 39.0 \text{ in.}$$

$$L_{ynet} = \sum_{j=1}^{n} L_{h,j} = 2\left(4.50\right) = 9.00 \text{ in.} \qquad (Eq. \text{ C-1.1.2-13})$$

$$L_{tg} = L_t - nL_h = 48.0 - 2\left(4.50\right) = 39.0 \text{ in.}$$

$$L_{tnet} = \sum_{j=1}^{n} L_{h,j} = 2\left(4.50\right) = 9.00 \text{ in.} \qquad (Eq. \text{ C-1.1.2-13})$$

b) Calculate the weighted average section properties using *Eq*. C-1.1.2-11 needed to predict flexural and flexural-torsional buckling loads.

If holes or net section regions are spaced symmetrically about the longitudinal mid-height of the column, then T = 0 and the average properties can be determined as follows:

$$I_{x,avg} = \frac{I_{xg}L_{xg} + I_{xnet}L_{xnet}}{L_x} = \frac{(1.46)(78.0) + (1.45)(18.0)}{96.0} = 1.46 \text{ in.}^4$$

$$I_{y,avg} = \frac{I_{yg}L_{yg} + I_{ynet}L_{ynet}}{L_y} = \frac{(0.113)(39.0) + (0.100)(9.00)}{48.0} = 0.111 \text{ in.}^4$$

$$r_{x,avg} = \frac{r_{xg}L_{xg} + r_{xnet}L_{xnet}}{L_x} = \frac{(2.11)(78.0)+(2.29)(18.0)}{96.0} = 2.14 \text{ in.}$$

$$r_{y,avg} = \frac{r_{yg}L_{yg} + r_{ynet}L_{ynet}}{L_y} = \frac{(0.589)(39.0)+(0.610)(9.00)}{48.0} = 0.593 \text{ in.}$$

$$J_{avg} = \frac{JL_{tg} + J_{net}L_{tnet}}{L_t} = \frac{(0.000130)(39.0)+(0.000110)(9.00)}{48.0} = 0.000126 \text{ in.}^4$$

$$x_{o,avg} = \frac{x_{o,g}L_{tg} + x_{o,net}L_{tnet}}{L_t} = \frac{(-1.11)(39.0)+(-1.20)(9.00)}{48.0} = -1.13 \text{ in.}$$

$$r_{o,avg} = \sqrt{(r_{x,avg})^2 + (r_{y,avg})^2 + (x_{o,avg})^2} = \sqrt{(2.14)^2 + (0.593)^2 + (-1.13)^2} = 2.49 \text{ in.}$$

As noted in the *Commentary* to the *Specification*, the warping torsion constant, C_w, does not follow the weighted average approach used for the other properties. Instead, the torsional warping stiffness may be approximated by using the net section properties, $C_{w,net}$. All net properties may be calculated using the software CUFSM by setting the element thickness to zero as described in Moen and Schafer.*

$$\beta = 1 - \left(\frac{x_{o,avg}}{r_{o,avg}}\right)^2 = 1 - \left(\frac{-1.13}{2.49}\right)^2 = 0.794 \qquad (\textit{Eq. } \text{C-1.1.2-15})$$

$$\sigma_{ex} = \frac{\pi^2 EI_{x,avg}}{A_g (K_x L_x)^2} = \frac{\pi^2 (29500)(1.46)}{(0.327)(1.0(96.0))^2} = 141 \text{ ksi} \qquad (\textit{Eq. } \text{C-1.1.2-16})$$

$$\sigma_{ey} = \frac{\pi^2 EI_{y,avg}}{A_g (K_y L_y)^2} = \frac{\pi^2 (29500)(0.111)}{(0.327)(1.0(48.0))^2} = 42.9 \text{ ksi}$$

$$\sigma_t = \frac{1}{A_g r_{o,avg}^2}\left[GJ_{avg} + \frac{\pi^2 EC_{w,net}}{(K_t L_t)^2}\right]$$

$$= \frac{1}{(0.327)(2.49)^2}\left[(11300)(0.000126) + \frac{\pi^2 (29500)(0.677)}{((1.0)(48.0))^2}\right] = 42.9 \text{ ksi} \qquad (\textit{Eq. } \text{C-1.1.2-17})$$

Since $\sigma_{ey} < \sigma_{ex}$, flexural buckling is controlled by buckling about the y-axis. The critical buckling load based on flexural buckling is given by:

$$P_{cre} = \sigma_{ey} A_g = 42.9(0.327) = 14.0 \text{ kips}$$

The elastic flexural-torsional buckling load is given by:

$$P_{cre} = \frac{A_g}{2\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex}+\sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right] \qquad (\textit{Eq. } \text{C-1.1.2-14})$$

$$= \frac{0.327}{2(0.794)}\left[(141+42.9) - \sqrt{(141+42.9)^2 - 4(0.794)(141)(42.9)}\right]$$

$$= 13.0 \text{ kips}$$

---

* Moen, C.D. and B.W. Schafer (2009), "Elastic Buckling of Cold-Formed Steel Columns and Beams With Holes," *Engineering Structures*, 31(12), pp. 2812-2824, 2009.

Flexural-torsional buckling controls, therefore, P_{cre} = 13.0 kips.

4. Determine the Nominal Global Buckling Strength, P_{ne} (Appendix 1.2.1.1.2)

$$P_y = A_g F_y = 0.327(33) = 10.8\text{ kips} \qquad (Eq.\ 1.2.1\text{-}4)$$

$$\lambda_c = \sqrt{\frac{P_y}{P_{cre}}} = \sqrt{\frac{10.8}{13.0}} = 0.911 \qquad (Eq.\ 1.2.1\text{-}3)$$

$\lambda_c < 1.5$, therefore:

$$P_{ne} = \left(0.658^{\lambda_c^2}\right)P_y = \left(0.658^{0.911^2}\right)10.8 = 7.63\text{ kips} \qquad (Eq.\ 1.2.1\text{-}1)$$

5. Determine the Nominal Local Buckling Strength, $P_{n\ell}$ (Appendix 1.2.1.2.2)

$$\lambda_\ell = \sqrt{\frac{P_{ne}}{P_{cr\ell}}} = \sqrt{\frac{7.63}{1.14}} = 2.59 \qquad (Eq.\ 1.2.1\text{-}7)$$

$\lambda_\ell > 0.776$; therefore,

$$P_{n\ell} = \left[1 - 0.15\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4}\right]\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4} P_{ne} = \left[1 - 0.15\left(\frac{1.14}{7.63}\right)^{0.4}\right]\left(\frac{1.14}{7.63}\right)^{0.4} 7.63 = 3.32\text{ kips} \qquad (Eq.\ 1.2.1\text{-}6)$$

Per Section 1.2.1.2.2, $P_{n\ell}$ cannot exceed P_{ynet}.

$$P_{ynet} = A_{net}F_y = 0.275(33) = 9.08\text{ kips} > 3.32\text{ kips OK} \qquad (Eq.\ 1.2.1\text{-}9)$$

6. Determine the Nominal Distortional Buckling Strength, P_{nd} (Appendix 1.2.1.3.2)

$$\lambda_d = \sqrt{\frac{P_y}{P_{crd}}} = \sqrt{\frac{10.8}{5.46}} = 1.41 \qquad (Eq.\ 1.2.1\text{-}15)$$

$$\lambda_{d2} = 0.561\left[14\left(\frac{P_y}{P_{ynet}}\right)^{0.4} - 13\right] = 0.561\left[14\left(\frac{10.8}{9.08}\right)^{0.4} - 13\right] = 1.13 \qquad (Eq.\ 1.2.1\text{-}17)$$

Since $\lambda_d > \lambda_{d2}$, Section 1.2.1.3.1 is used, and for $\lambda_d > 0.561$,

$$P_{nd} = \left[1 - 0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right]\left(\frac{P_{crd}}{P_y}\right)^{0.6} P_y = \left[1 - 0.25\left(\frac{5.46}{10.8}\right)^{0.6}\right]\left(\frac{5.46}{10.8}\right)^{0.6} 10.8 = 5.98\text{ kips} \qquad (Eq.\ 1.2.1\text{-}11)$$

7. Determine the Available Compression Strength

P_n = the least of P_{ne}, $P_{n\ell}$ and P_{nd} = 3.32 kips

The geometry of this section falls within the limits of pre-qualified columns of DSM Section 1.1.1.1; therefore, use of the higher ϕ and lower Ω of Section 1.2.1 is permitted.

LRFD design strength

$$\phi_c = 0.85$$

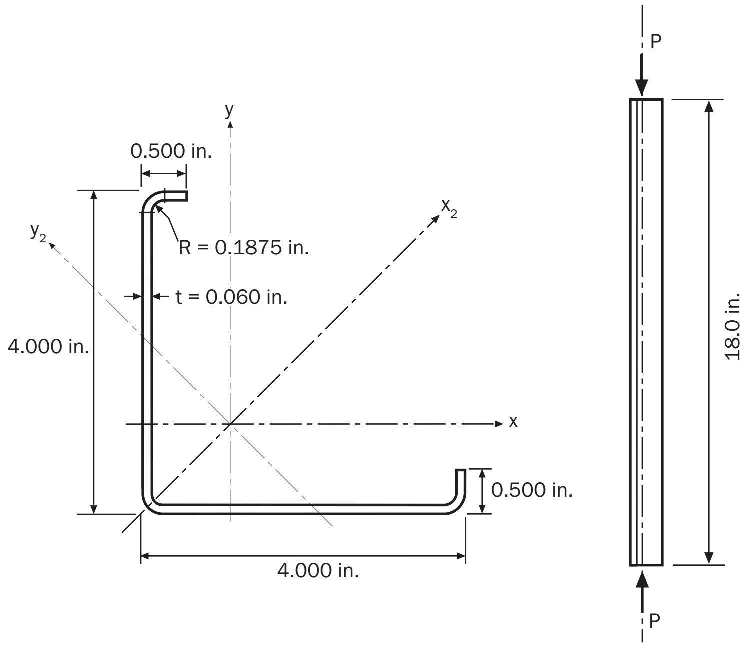
$$\phi_c P_n = 0.85(3.32) = 2.82\text{ kips}$$

ASD allowable strength

$$\Omega_c = 1.80$$

$$\frac{P_n}{\Omega_c} = \frac{3.32}{1.80} = 1.84\text{ kips}$$

Example III-5: Unbraced Equal Leg Angle With Lips – Compression



From Table I-6 or Example I-4

$A = 0.512$ in.^{2}
$I_x = 0.959$ in.^{4}
$I_{y2} = 0.397$ in.^{4}
$r_{y2} = 0.881$ in.
$x_0 = -1.63$ in.
$r_0 = 2.53$ in.
$J = 0.000615$ in.^{4}
$C_w = 0.0533$ in.^{6}
$j = 3.13$ in.

Given:

1. Steel: $F_y = 50$ ksi
2. Section: 4LS4x060 as shown above
3. Section is concentrically loaded in compression
4. $KL_x = KL_y = KL_t = 18.0$ in.

Required:

ASD allowable design strength under concentric compression loading

Solution:

1. Nominal Axial Strength, P_n (Section C4)

The equal leg angle is a singly-symmetric section; therefore, check flexural and torsional-flexural buckling. Single angles do not exhibit distortional buckling.

a) Flexural buckling (Section C4.1.1)

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

use least radius of gyration, r_{y2}

$$F_e = \frac{\pi^2(29500)}{\left[(1.0)(18.0)/0.881\right]^2} = 697.5 \text{ ksi} \qquad (Eq.\ C4.1.1\text{-}1)$$

b) Torsional-flexural buckling (Section C4.1.2)

$$F_e = \frac{1}{2\beta}\left[(\sigma_{ex2}+\sigma_t)-\sqrt{(\sigma_{ex2}+\sigma_t)^2-4\beta\sigma_{ex2}\sigma_t}\right] \qquad (\text{from } Eq.\ C4.1.2\text{-}1)$$

where the x_2 axis is the axis of symmetry.

$$\beta = 1-\left(x_o/r_o\right)^2 \qquad (Eq.\ C4.1.2\text{-}3)$$

$$= 1-\left(\frac{-1.63}{2.53}\right)^2 = 0.585$$

$$\sigma_{ex2} = \frac{\pi^2 E}{\left(K_{x2}L_{x2}/r_{x2}\right)^2} \qquad (\text{from } Eq.\ C3.1.2.1\text{-}11)$$

For the case of an equal leg angle, the radius of gyration about the axis of symmetry, r_{x2}, can be computed as:

$$I_{x2} = 2I_x - I_{y2}$$

$$= (2)(0.959)-0.397 = 1.52\text{ in.}^4$$

$$r_{x2} = \sqrt{\frac{I_{x2}}{A}}$$

$$= \sqrt{\frac{1.52}{0.512}} = 1.72\text{ in.}$$

$$K_{x2} = K = 1.0$$

$$\sigma_{ex2} = \frac{\pi^2(29500)}{\left[(1.0)(18.0)/1.72\right]^2} \qquad (Eq.\ C3.1.2.1\text{-}11)$$

$$= 2660\text{ ksi}$$

$$\sigma_t = \frac{1}{Ar_o{}^2}\left[GJ+\frac{\pi^2 EC_w}{(K_tL_t)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$= \frac{1}{(0.512)(2.53)^2}\left[(11300)(0.000615)+\frac{\pi^2(29500)(0.0533)}{\left[(1.0)(18.0)\right]^2}\right]$$

$$= 16.7\text{ ksi}$$

$$F_e = \frac{1}{(2)(0.583)}\left[(2674+16.7)-\sqrt{(2674+16.7)^2-(4)(0.583)(2674)(16.7)}\right] \qquad (Eq.\ C4.1.2\text{-}1)$$

$$= 16.7\text{ ksi} \qquad \text{CONTROLS}$$

c) Nominal axial strength (Section C4)

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{50}{16.7}} = 1.73 > 1.5$$

$$F_n = \left[\frac{0.877}{\lambda_c{}^2}\right]F_y \qquad (Eq.\ C4.1\text{-}3)$$

$$= \left[\frac{0.877}{(1.73)^2}\right]50 = 14.7\text{ ksi}$$

$P_n = A_eF_n$ (*Eq*. C4.1-1)

From Example I-11 at a uniform compression stress of 14.7 ksi,

$A_e = 0.383 \text{ in.}^2$

$P_n = (0.383)(14.7) = 5.63 \text{ kips}$ (*Eq*. C4.1-1)

2. ASD Allowable Strength not Considering Minimum Eccentricity

$\Omega_c = 1.80$

$$\frac{P_n}{\Omega_c} = \frac{5.63}{1.80} = 3.13 \text{ kips}$$

3. Nominal Flexural Strength, M_n (Section C3.1.2.1)

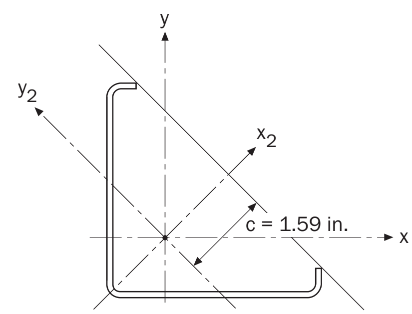
Sections C4.1(b) and C5.2.1 of the *Specification* require consideration of an eccentricity of PL/1000 about the minor axis.

The equal leg angle is a singly-symmetric section, therefore check lateral-torsional buckling about the minor principal axis, the axis perpendicular to the axis of symmetry.

$$F_e = \frac{C_sA\sigma_{ex2}}{C_{TF}S_f}\left[j + C_s\sqrt{j^2 + r_o^{\,2}(\sigma_t/\sigma_{ex2})}\right]$$ (from *Eq*. C3.1.2.1-10)

$$S_{y2} = \frac{I_{y2}}{c}$$

$$= \frac{0.397}{1.59} = 0.250 \text{ in.}^3$$



$\sigma_t = 16.7$ ksi (computed above)

$\sigma_{ex2} = 2674$ ksi (computed above)

$C_{TF} = 1$

$C_s = -1$ (assume worst case - tension on shear center side of centroid)

$$F_e = \frac{(-1)(0.512)(2674)}{(1)(0.250)}\left[3.13 + (-1)\sqrt{(3.13)^2 + (2.53)^2(16.7/2674)}\right]$$ (*Eq*. C3.1.2.1-10)

$= 34.94$ ksi

Since $2.78F_y > F_e > 0.56F_y$,

$$F_c = \frac{10}{9}F_y\left(1 - \frac{10F_y}{36F_e}\right)$$ (*Eq*. C3.1.2.1-2)

$$= \frac{10}{9}(50.0)\left(1 - \frac{(10)(50.0)}{(36)(34.94)}\right) = 33.47 \text{ ksi}$$

By calculations not shown, the section is found to be fully effective for bending about the y_2 axis with a maximum compression stress of 33.47 ksi at the ends of the lips; therefore,

$S_c = S_{y2}$

$M_n = S_cF_c = (0.250)(33.47) = 8.37$ kip-in. (*Eq*. C3.1.2.1-1)

4. ASD Allowable Strength Including Minimum Eccentricity

M_{y2} = PL/1000

$= P(18.0)/1000 = 0.0180P$

Assume $\frac{\Omega_c P}{P_n}$ > 0.15, and solve for the allowable load, P, such that equations C5.2.1-1 and C5.2.1-2 are satisfied.

C_{my2} = 1.0

K_{y2} = K = 1.0

$$P_{Ey2} = \frac{\pi^2 EI_{y2}}{\left(K_{y2}L_{y2}\right)^2} \qquad (Eq.\ C5.2.1\text{-}7)$$

$$= \frac{\pi^2(29500)(0.397)}{\left[(1.0)(18.0)\right]^2} = 357 \text{ kips}$$

$$\alpha_{y2} = 1 - \frac{\Omega_c P}{P_{Ey2}} > 0 \qquad (Eq.\ C5.2.1\text{-}5)$$

$$= 1 - \frac{(1.80)(P)}{357}$$

$$= 1 - 0.00504P$$

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{my2} M_{y2}}{M_{ny2}\alpha_{y2}} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

$$\frac{(1.80)(P)}{5.63} + \frac{(1.67)(1.0)(0.0180P)}{(8.37)(1-0.00504P)} \le 1.0$$

Solving for P:

$P \le 3.09$ kips

By calculations not shown, similar to those in Example I-11, P_{no} is calculated at f = F_y.

P_{no} = 12.3 kips

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_{y2}}{M_{ny2}} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

$$\frac{(1.80)(P)}{12.3} + \frac{(1.67)(0.0180P)}{(8.37)} \le 1.0$$

Solving for P:

$P \le 6.67$ kips

Therefore, equation C5.2.1-1 controls

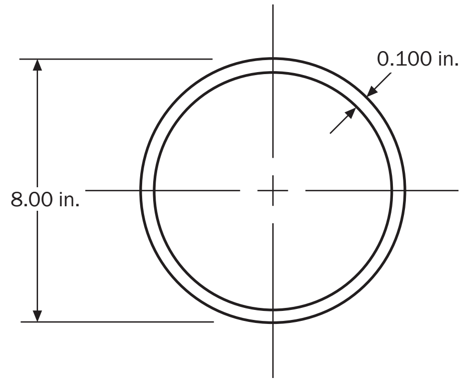
P $\le$ 3.09 kips

Check assumption that $\frac{\Omega_c P}{P_n}$ > 0.15

$$\frac{\Omega_c P}{P_n} = \frac{(1.80)(3.09)}{5.63} = 0.99 > 0.15 \quad \text{OK}$$

Therefore, the ASD allowable design strength is 3.09 kips.

Example III-6: Tubular Section - Round - Bending and Compression



Given:

1. Steel: F_y = 42 ksi
2. Section: Shown in sketch above
3. Height: L = 10.0 feet, simply supported at each end
4. Axial Loads: Dead Load: P_D = 7.5 kips, Roof Live Load: P_{Lr} = 20 kips
5. Transverse Concentrated Wind Load (at mid-span): P_W = 5.0 kips

Required:

Check the adequacy of the section using ASD and LRFD methods with ASCE/SEI 7-10 load combinations.

Solution:

1. Nominal Axial Strength, P_n (Section C4.1.5)

Ratio of outside diameter to wall thickness

$$D/t = 8.00/0.100 = 80.0$$

$$D/t < 0.441E/F_y = 0.441(29500/42) = 310 \quad \text{OK}$$

a) Compute nominal axial stress, F_n

$$I = \frac{\pi}{4}\left[(\text{Outside Radius})^4 - (\text{Inside Radius})^4\right]$$

$$= \frac{\pi}{4}\left[(4.00)^4 - (3.90)^4\right] = 19.37 \text{ in.}^4$$

$$A = \frac{\pi}{4}\left[(\text{Outside Diameter})^2 - (\text{Inside Diameter})^2\right]$$

$$= \frac{\pi}{4}\left[(8.00)^2 - (7.80)^2\right] = 2.482 \text{ in.}^2$$

$$r = \sqrt{I/A}$$

$$= \sqrt{19.37/2.482} = 2.794 \text{ in.}$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2(29500)}{\left[(10)(12)/2.794\right]^2} = 158 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{42}{158}} = 0.516 \qquad (Eq.\ C4.1\text{-}4)$$

Since $\lambda_c \leq 1.5$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4.1\text{-}2)$$

$$= \left(0.658^{(0.516)^2}\right)42 = 37.6 \text{ ksi}$$

b) Compute effective area, A_e

$$A_o = \left[\frac{0.037}{(DF_y)/(tE)} + 0.667\right]A \leq A \qquad (Eq.\ C4.1.5\text{-}2)$$

$$= \left[\frac{0.037}{(8.000)(42)/\left[(0.100)(29500)\right]} + 0.667\right]2.482$$

$$= 2.462 \text{ in.}^2 < A$$

$$R = \frac{F_y}{2F_e} \leq 1.0 \qquad (Eq.\ C4.1.5\text{-}3)$$

$$= \frac{42}{(2)(158)} = 0.133 < 1.0$$

$$A_e = A_o + R(A - A_o) \qquad (Eq.\ C4.1.5\text{-}1)$$

$$= 2.462 + 0.133(2.482 - 2.462) = 2.465 \text{ in.}^2$$

c) Compute nominal axial strengths, P_n and P_{no}

$$P_n = F_n A_e \qquad (Eq.\ C4.1\text{-}1)$$

$$= (37.6)(2.465) = 92.7 \text{ kips}$$

Compute P_{no} for use in Section C5.2

$$F_n = F_y$$

$$P_{no} = F_y A_o \qquad (Eq.\ C4.1\text{-}1)$$

$$= (42)(2.462) = 103 \text{ kips}$$

2. Nominal Flexural Strength, M_n (from Example II-8)

M_n = 233 kip-in.

3. Combined Bending and Compression

$M_W = PL/4 = (5.0)(10)(12)/4 = 150$ kip-in.

ASD

ASCE/SEI 7-10 load combinations considered:

D + L

D + 0.6W

D + 0.75(0.6W) + 0.75L_r

Controlling load combination (by inspection) is D + 0.75(0.6W) + 0.75L_r

$P = P_D + 0.75P_{Lr} = 7.5 + (0.75)(20) = 22.5$ kips

$M_x = 0.75(0.6W) = (0.75)(0.6)(150) = 67.5$ kip-in.

$\Omega_c = 1.80$

$\Omega_b = 1.67$

$\frac{\Omega_c P}{P_n} = \frac{(1.80)(22.5)}{92.7} = 0.437 > 0.15$; therefore, use Equations C5.2.1-1 and C5.2.1-2

$C_{mx} = 1.0$

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} \quad (Eq.\ C5.2.1\text{-}6)$$

$$= \frac{\pi^2(29500)(19.37)}{[(1.0)(120)]^2} = 392 \text{ kips}$$

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} > 0 \quad (Eq.\ C5.2.1\text{-}4)$$

$$= 1 - \frac{(1.80)(22.5)}{392} = 0.897$$

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} \le 1.0 \quad (Eq.\ C5.2.1\text{-}1)$$

$$\frac{(1.80)(22.5)}{92.7} + \frac{(1.67)(1.0)(67.5)}{(233)(0.897)} = 0.976 < 1.0 \quad \text{OK}$$

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} \le 1.0 \quad (Eq.\ C5.2.1\text{-}2)$$

$$\frac{(1.80)(22.5)}{103} + \frac{(1.67)(67.5)}{(233)} = 0.877 < 1.0 \quad \text{OK}$$

LRFD

ASCE/SEI 7-10 load combinations considered:

1.4D

1.2D + 1.6L_r

$1.2D + 1.0W + 0.5L_r$

$0.9D + 1.0W$

Controlling load combination (by inspection) is $1.2D + 1.0W + 0.5L_r$

$\overline{P} = P_u = 1.2P_D + 0.5P_{Lr} = (1.2)(7.5) + (0.5)(20) = 19.0 \text{ kips}$

$\overline{M} = M_u = 1.0M_W = (1.0)(150) = 150 \text{ kip-in.}$

$\phi_c = 0.85$

$\phi_b = 0.95$

$\dfrac{\overline{P}}{\phi_c P_n} = \dfrac{19.0}{(0.85)(92.7)} = 0.241 > 0.15$; therefore, use Equations C5.2.2-1 and C5.2.2-2

$C_{mx} = 1.0$

$$\alpha_x = 1 - \frac{\overline{P}}{P_{Ex}} > 0 \qquad (\textit{Eq. } \text{C5.2.2-4})$$

$$= 1 - \frac{19.0}{392} = 0.952$$

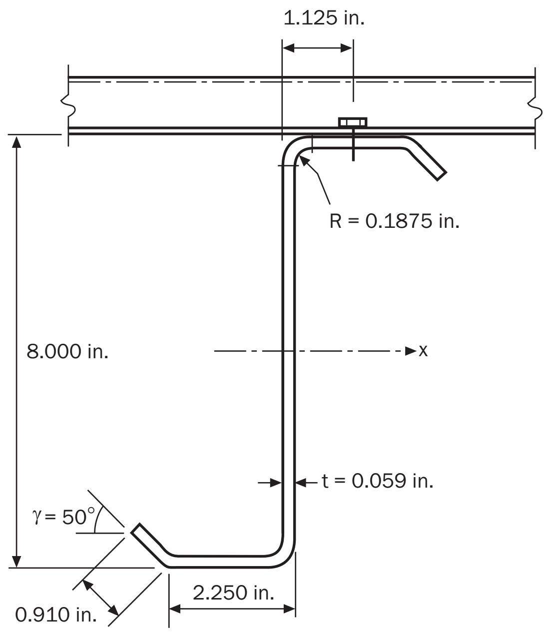
$$\frac{\overline{P}}{\phi_c P_n} + \frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x} \le 1.0 \qquad (\textit{Eq. } \text{C5.2.2-1})$$

$$\frac{19.0}{(0.85)(92.7)} + \frac{(1.0)(150)}{(0.95)(233)(0.952)} = 0.953 < 1.0 \quad \text{OK}$$

$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} \le 1.0 \qquad (\textit{Eq. } \text{C5.2.2-2})$$

$$\frac{19.0}{(0.85)(103)} + \frac{150}{(0.95)(233)} = 0.895 < 1.0 \quad \text{OK}$$

Example III-7: Stiffened Z-Section With One Flange Through-Fastened to Deck or Sheathing – Compression



Given:

1. Steel: F_y = 55 ksi
2. Span = 25.0 feet = 300 in.
3. Section: 8ZS2.25x059

   d = D = 8.000 in.

   b = B = 2.250 in.

   t = 0.059 in.

   A = 0.822 in.^{2}

   r_x = 3.07 in.
4. Through-fastened at 12 in. o.c. and assumed to be located at the center of the flange.
5. Panel has a rotational stiffness of 0.002 kip/in./in. determined by tests performed in accordance with AISI S901-13*.
6. Both flanges are restrained from lateral movement at the supports.

Required:

Available compression strengths using ASD and LRFD

Solution:

1. Nominal Axial Strength, P_n - Flexural Buckling About the X-Axis (Section C4.1.1)

K = 1

* AISI S901-13, *Rotational Lateral Test Method for Beam to Panel Assemblies*, American Iron and Steel Institute, Washington, DC, 2014.

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2(29500)}{(300/3.07)^2} = 30.5 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{55}{30.5}} = 1.34 < 1.50$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4.1\text{-}2)$$

$$= \left(0.658^{1.34^2}\right)55 = 25.9 \text{ ksi}$$

$$P_n = A_e F_n \qquad (Eq.\ C4.1\text{-}1)$$

In Example I-10, A_e was calculated as 0.578 in.^{2} at f = 25.9 ksi

$$P_n = (0.578)(25.9) = 15.0 \text{ kips}$$

2. Nominal Axial Strength, P_n - Flexural-Torsional Buckling (Section D6.1.3)

a) Check limits of applicability of Section D6.1.3

(1) $t \le 0.125$ in.; t = 0.059 in. OK

(2) $6\text{in.} \le d \le 12\text{in.}$; d = 8.00 in. OK

(3) Flanges are edge stiffened compression elements OK

(4) $70 \le d/t \le 170$; $d/t = 8.00/0.059 = 136$ OK

(5) $2.8 \le d/b \le 5$; $d/b = 8.00/2.25 = 3.56$ OK

(6) $16 \le \dfrac{\text{flat flange width}}{t} \le 50$; $\dfrac{1.889}{0.059} = 32.0$ (from Example I-3) OK

(7) Both flanges are prevented from moving laterally at the supports OK

(8) Fastener spacing ≤ 12 in. OK

Rotational lateral stiffness ≥ 0.0015 kip/in./in. OK

(9) $F_y \ge 33$ ksi OK

(10) Span length ≤ 33 ft ; L = 25 ft OK

All conditions are satisfied

b) Compute P_n

$$P_n = C_1 C_2 C_3 AE/29500 \qquad (Eq.\ D6.1.3\text{-}1)$$

α = 1 (units are inches)

$$x = a/b \qquad (Eq.\ D6.1.3\text{-}5)$$

$$= 1.125/2.25 = 0.50$$

$$C_1 = 0.79x + 0.54 \qquad (Eq.\ D6.1.3\text{-}2)$$

$$= (0.79)(0.50) + 0.54 = 0.935$$

$$C_2 = 1.17\alpha t + 0.93 \qquad (Eq.\ D6.1.3\text{-}3)$$

$$= (1.17)(1)(0.059) + 0.93 = 1.00$$

$$C_3 = \alpha(2.5b - 1.63d) + 22.8 \qquad (Eq.\ D6.1.3\text{-}4)$$

$$= 1[(2.5)(2.25) - (1.63)(8.00)] + 22.8 = 15.4$$

$$P_n = (0.935)(1.00)(15.4)(0.822)(29500)/29500 \qquad (Eq.\ D6.1.3\text{-}1)$$

$$= 11.8 \text{ kips}$$

3. Governing Limit State is Flexural-Torsional Buckling

P_n = minimum of 11.8 kips or 15.0 kips = 11.8 kips

4. Available Strength

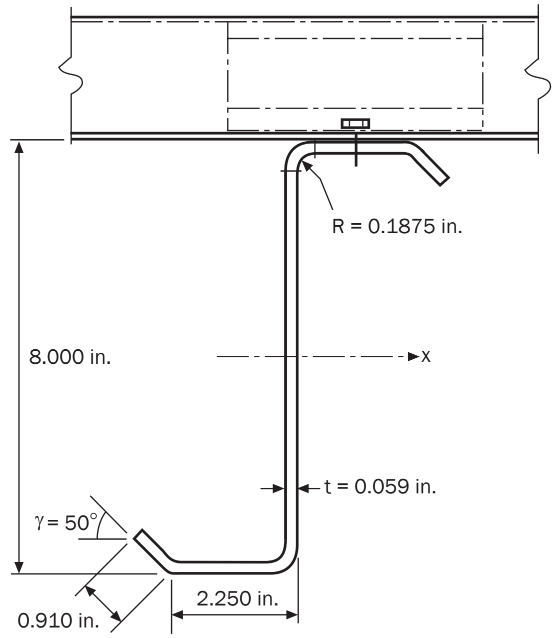
ASD

$$\frac{P_n}{\Omega} = \frac{11.8}{1.80} = 6.56 \text{ kips}$$

LRFD

$$\phi P_n = (0.85)(11.8) = 10.0 \text{ kips}$$

Example III-8: Stiffened Z-Section With One Flange Fastened to a Standing Seam Roof – Compression



Given:

1. Steel: F_y = 55 ksi
2. Span = 25.0 ft = 300 in.
3. Section: 8ZS2.25x059

   d = D = 8.000 in.

   b = B = 2.250 in.

   t = 0.059 in.

   A = 0.822 in.^{2}

   r_x = 3.07 in.
4. Reduction factor, R = 0.70, determined from uplift tests performed in accordance with AISI S908-13*
5. Both flanges are restrained from lateral movement at the supports.

Required:

Available compression strengths using ASD and LRFD

Solution:

1. Nominal Axial Strength, P_n - Flexural Buckling About the X-Axis (Section C4.1.1)

K = 1.0

* AISI S908-13, *Base Test Method for Purlins Supporting a Standing Seam Roof and the Commentary*, American Iron and Steel Institute, Washington, DC, 2014.

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \quad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2(29500)}{(300/3.07)^2} = 30.5 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \quad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{55}{30.5}} = 1.34 < 1.50$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \quad (Eq.\ C4.1\text{-}2)$$

$$= \left(0.658^{1.34^2}\right)55 = 25.9 \text{ ksi}$$

$P_n = A_e F_n$ (*Eq.* C4.1-1)

In Example I-10, A_e was calculated as 0.578 in.^{2} at f = 25.9 ksi

$P_n = (0.578)(25.9) = 15.0$ kips

2. Nominal Axial Strength, P_n - Flexural-Torsional Buckling (Section D6.1.4)

a) Check limits of applicability of Section D6.1.4 (in Appendix A)

(1) $0.054\text{ in.} \le t \le 0.125\text{ in.}$; t = 0.059 in. OK

(2) $6\text{ in.} \le d \le 12\text{ in.}$; d = 8.00 in. OK

(3) Flanges are edge stiffened compression elements OK

(4) $70 \le d/t \le 170$; $d/t = 8.00/0.059 = 136$ OK

(5) $2.8 \le d/b < 5$; $d/b = 8.00/2.25 = 3.56$ OK

(6) $16 \le \frac{\text{flat flange width}}{t} < 50$; $\frac{1.889}{0.059} = 32.0$ (from Example I-3) OK

(7) Both flanges are prevented from moving laterally at the supports OK

(8) $F_y \le 70\text{ ksi}$ OK

All conditions are satisfied.

b) Compute P_n

$d/t = 136 > 130$; therefore,

$k_{af} = 0.20$

$P_n = k_{af} R F_y A$ (*Eq.* D6.1.4-1)

$= (0.20)(0.70)(55)(0.822) = 6.33$ kips

3. Governing Limit State is Flexural-Torsional Buckling

P_n = minimum of (6.33 kips or 15.0 kips) = 6.33 kips

4. Available Strength

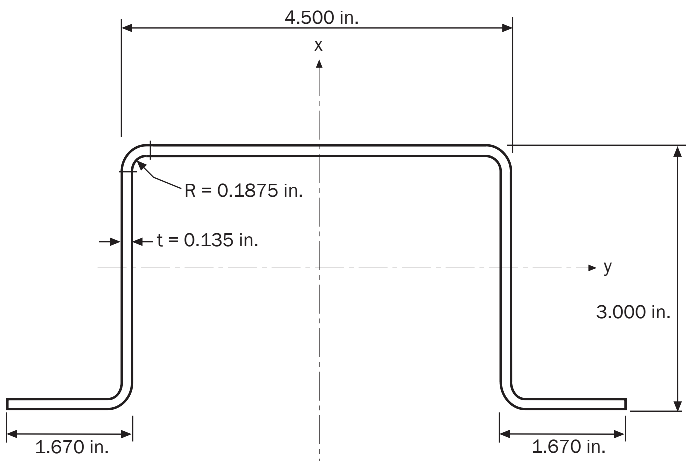
ASD

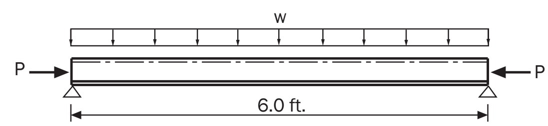
$$\frac{P_n}{\Omega} = \frac{6.33}{1.80} = 3.52 \text{ kips}$$

LRFD

$$\phi P_n = (0.85)(6.33) = 5.38 \text{ kips}$$

Example III-9: Hat Section - Bending and Compression





Given:

1. Steel: F_y = 50 ksi
2. Section: 3HU4.5x135 as shown above. From Table I-8,

   A_g = 1.74 in.^{2}

   I_y = 2.47 in.^{4}

   r_y = 1.19 in.
3. L = 6.0 feet, simply supported with continuous lateral and torsional and distortional bracing of the compression flanges.
4. Axial Loads: Dead Load: P_D = 2 kips, Live Load: P_L = 10 kips
5. Transverse Uniform Flexural Loads:

   Dead Load: w_D = 0.090 kips/ft

   Live Load: w_L = 0.360 kips/ft

Required:

Check the adequacy of the section using ASD and LRFD methods. Do not use inelastic reserve capacity.

Solution:

1. Bending Moments at Service Level

$$M_D = \frac{w_D L^2}{8} = \frac{(0.090)(6)^2}{8} = 0.405 \text{ kip-ft} = 4.86 \text{ kip-in.}$$

$$M_L = \frac{w_L L^2}{8} = \frac{(0.360)(6)^2}{8} = 1.62 \text{ kip-ft} = 19.4 \text{ kip-in.}$$

2. Nominal Flexural Strength, M_n

From Example II-7

M_n= 76.0 kip-in.

3. Nominal Axial Strength, P_n

The member is free to buckle only in the plane perpendicular to the flange.

$$F_e = \frac{\pi^2 E}{\left(KL_y/r_y\right)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2(29500)}{\left[(1.0)(72.0)/1.19\right]^2} = 79.5 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{50.0}{79.5}} = 0.793 \le 1.5$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4.1\text{-}2)$$

$$= \left[0.658^{(0.793)^2}\right]50 = 38.4 \text{ ksi}$$

$$P_n = A_eF_n \qquad (Eq.\ C4.1\text{-}1)$$

From Example I-13

A_e = 1.74 in.^{2} at a stress level of 50 ksi.

All elements are fully effective. By inspection, all elements will therefore be fully effective at the lower stress of 38.4 ksi:

A_e = A_{gross} = 1.74 in.^{2}

$$P_n = (1.74)(38.4) = 66.8 \text{ kips} \qquad (Eq.\ C4.1\text{-}1)$$

4. Combined Compression and Bending - ASD (Section C5.2.1)

a) Required axial strength

P = $P_D + P_L$ = 2.0 + 10.0 = 12.0 kips

b) Required flexural strength

M_y = $M_D + M_L$ = 4.86 + 19.4 = 24.3 kip-in.

c) Combined strength

Check $\Omega_c P/P_n$

Ω_c = 1.80

$$\frac{\Omega_c P}{P_n} = \frac{(1.80)(12.0)}{66.8} = 0.323 > 0.15\text{; therefore, check Equations C5.2.1-1 and C5.2.1-2.}$$

C_{my} = 1.0

$$P_{Ey} = \frac{\pi^2 EI_y}{\left(K_yL_y\right)^2} \qquad (Eq.\ C5.2.1\text{-}7)$$

$$= \frac{\pi^2(29500)(2.47)}{\left[(1.0)(72.0)\right]^2} = 138.7 \text{ kips}$$

$$\alpha_y = 1 - \frac{\Omega_c P}{P_{Ey}} > 0 \qquad (Eq.\ C5.2.1\text{-}5)$$

$$= 1 - \frac{(1.80)(12.0)}{138.7} = 0.844$$

$M_x = 0.0$

$\Omega_b = 1.67$

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} + \frac{\Omega_b C_{my} M_y}{M_{ny}\alpha_y} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

$$\frac{(1.80)(12.0)}{66.8} + \frac{(1.67)(1.0)(24.3)}{(76.0)(0.844)} \le 1.0$$

$0.323 + 0.633 = 0.956 < 1.0$ OK

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

$P_{no} = A_e F_y$ where A_e was calculated as 1.74 in.^{2} in Example I-13 at a stress level of 50 ksi.

$= (1.74)(50) = 87.0$ kips

$$\frac{(1.80)(12.0)}{87.0} + \frac{(1.67)(24.3)}{76.0} = 0.248 + 0.534 = 0.782 < 1.0 \quad \text{OK} \qquad (Eq.\ C5.2.1\text{-}2)$$

5. Combined Compression and Bending - LRFD (Section C5.2.2)

a) Required Axial Strength

$\bar{P} = P_u = 1.2P_D + 1.6P_L$

$= (1.2)(2.0) + (1.6)(10.0) = 18.4$ kips

b) Required Flexural Strength

$\bar{M}_y = M_{uy} = 1.2M_D + 1.6M_L$

$= (1.2)(4.86) + (1.6)(19.4) = 36.9$ kip-in.

c) Combined Strength

Check $\bar{P}/\phi_c P_n$

$\phi_c = 0.85$

$$\frac{\bar{P}}{\phi_c P_n} = \frac{18.4}{(0.85)(66.8)} = 0.324 > 0.15;$$ therefore, check Equations C5.2.2-1 and C5.2.2-2.

$C_{my} = 1.0$

$$\alpha_y = 1 - \frac{\overline{P}}{P_{Ey}} > 0 \qquad (Eq.\ C5.2.2\text{-}5)$$

P_{Ey} = 138.7 kips (calculated above)

$$\alpha_y = 1 - \frac{18.4}{138.7} = 0.867 \qquad (Eq.\ C5.2.2\text{-}5)$$

M_x = 0.0

ϕ_b = 0.90

$$\frac{\overline{P}}{\phi_c P_n} + \frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x} + \frac{C_{my}\overline{M}_y}{\phi_b M_{ny}\alpha_y} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}1)$$

$$\frac{18.4}{(0.85)(66.8)} + \frac{(1.0)(36.9)}{(0.90)(76.0)(0.867)} = 0.324 + 0.622 = 0.946 < 1.0 \quad \text{OK}$$

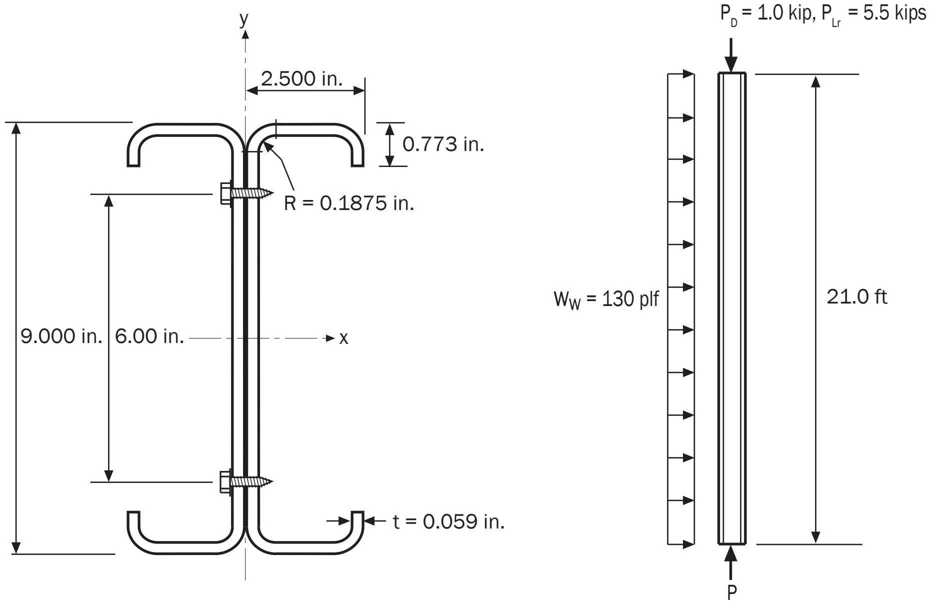
$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

P_{no} = 87.0 kips (calculated above)

$$\frac{18.4}{(0.85)(87.0)} + \frac{36.9}{(0.90)(76.0)} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

0.249 + 0.539 = 0.788 < 1.0 OK

Example III-10: I-Section - Built-Up From Channels



Gross properties of each channel

t = 0.059 in.	I_{xi} = 10.3 in.^{4}	I_{yi} = 0.698 in.^{4}
A_i = 0.881 in.^{2}	r_{xi} = 3.42 in.	r_{yi} = 0.890 in.
C_{wi} = 11.9 in.^{6}	S_{xi} = 2.29 in.^{3}	$\bar{x}_i$ = 0.641 in.
J_i = 0.00102 in.^{4}	m_i = 1.05 in.	

Given:

1. Steel: F_y = 55 ksi, F_u = 70 ksi
2. Section: Two 9CS2.5x059 back to back as shown
3. Length: 21.0 ft
4. Braced for buckling about the x-axis at the ends only
5. Braced for buckling about the y-axis and for torsion at the ends and mid-span (10.5 feet)
6. $K_x = K_y = K_t = 1.0$
7. Sections connected by pairs of #10 screws at 36 in. on center spaced 6 inches apart along the y-axis of the channel sections

Required:

1. Check members for adequacy using:
   a. ASD - using ASCE/SEI 7-10 load combination D + 0.75(0.6W) + 0.75L_r
   b. LRFD - using ASCE/SEI 7-10 load combination 1.2D + 1.0W + 0.5L_r

Solution:

1. Nominal Axial Strength, P_n (Section D1.2)

a) Properties of built-up section

$$A = 2A_i = (2)(0.881) = 1.76 \text{ in.}^2$$

$I_x = 2I_{xi} = (2)(10.3) = 20.6$ in.^{4}

r_x = 3.42 in. (same as single section)

$I_y = 2\left[I_{yi} + A_i \bar{x}_i^2\right]$

$= 2\left[0.698 + (0.881)(0.641)^2\right] = 2.12$ in.^{4}

$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{2.12}{1.76}} = 1.10$ in.

$C_w = 2C_{wi} = (2)(11.9) = 23.8$ in.^{6}

$J = 2J_i = (2)(0.00102) = 0.00204$ in.^{4}

x_o = 0.0 (distance from shear center to centroid of combined shape)

$r_o = \sqrt{r_x^2 + r_y^2 + x_o^2}$

$= \sqrt{3.42^2 + 1.10^2 + 0.0^2} = 3.59$ in.

b) X-axis flexural buckling per Section C4.1.1: The buckling mode does not involve relative deformations that produce shear forces in the connectors between individual shapes, so *Eq.* D1.2-1 does not apply.

$$\left(\frac{KL}{r}\right)_x = \frac{(1.0)(21.0)(12.0)}{3.42} = 73.7$$

$$F_e = \frac{\pi^2 E}{(KL/r)_x^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2 (29500)}{(73.7)^2} = 53.6 \text{ ksi}$$

c) Y-axis flexural buckling per Section C4.1.1: The buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, so *Eq.* D1.2-1 applies.

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

$$= \sqrt{\left(\frac{(1.0)(10.5)(12.0)}{1.10}\right)^2 + \left(\frac{36.0}{0.890}\right)^2} = 121.5$$

a/r_i = 36.0/0.890 = 40.4 < 121.5/2 = 60.8 OK

$$F_e = \frac{\pi^2 E}{(KL/r)_y^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2 (29500)}{(121.5)^2} = 19.7 \text{ ksi} \leftarrow \text{CONTROLS}$$

d) Torsional buckling per Section C4.1.2: Since the section is doubly-symmetric,

$$F_e = \sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$= \frac{1}{(1.76)(3.59)^2}\left[(11300)(0.00204) + \frac{\pi^2(29500)(23.8)}{\left[(1.0)(10.5)(12.0)\right]^2}\right] = 20.3 \text{ ksi}$$

e) Y-axis flexural buckling controls axial strength

F_e = 19.7 ksi

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{55.0}{19.7}} = 1.67 > 1.5$$

$$F_n = \left[\frac{0.877}{\lambda_c^2}\right]F_y \qquad (Eq.\ C4.1\text{-}3)$$

$$= \left[\frac{0.877}{(1.67)^2}\right]55 = 17.3 \text{ ksi}$$

It can be shown that at an axial compression stress of f = 17.3 ksi, the effective area of one channel is 0.620 in.^{2} (calculations not shown); therefore,

$A_e = (2)(0.620) = 1.24$ in.^{2}

$$P_n = A_e F_n \qquad (Eq.\ C4.1\text{-}1)$$

$$= (1.24)(17.3) = 21.5 \text{ kips}$$

2. Nominal Flexural Strength, M_n

a) Check the maximum permitted screw spacing for the built-up section using the requirements of Section D1.1

$$s_{max} = L/6 \le \frac{2gT_s}{mq} \qquad (Eq.\ D1.1\text{-}1)$$

g = 6.0 in. (screw gage)

m = 1.05 in.

Screw tension design strength, T_s, is the smaller of the screw pull-out, pull-over or tension strengths.

Pull-out

$$P_{not} = 0.85 t_c d F_{u2} \qquad (Eq.\ E4.4.1\text{-}1)$$

$$= (0.85)(0.059)(0.190)(70) = 0.667 \text{ kip}$$

Pull-over

Assuming no independent washer under the screw head, use Section E4.4.2(b)

$d'_w = d_h$ = 0.399 in. (washer head diameter)

$$P_{nov} = 1.5 t_1 d'_w F_{u1} \qquad (Eq.\ E4.4.2\text{-}1)$$

$= (1.5)(0.059)(0.399)(70) = 2.47$ kips

Screw tension

P_{ts} = 2.10 kips (from screw manufacturer)

$P_{nt} = P_{ts}$

= 2.10 kips

The nominal screw strength is the minimum of P_{not}, P_{nov} or P_{nt} = 0.667 kip.

ASD

$$T_s = \frac{0.667}{\Omega} = \frac{0.667}{3.0} = 0.222 \text{ kip}$$

LRFD

$$T_s = \phi 0.667 = (0.50)(0.667) = 0.334 \text{ kip}$$

The design load on the beam between fasteners, q, is taken as 3 times the uniformly distributed load.

$$q = (0.75)(0.6)(3)\left(\frac{0.130}{12.0}\right) = 0.0146 \text{ kips per in. (ASD, using a wind load factor of 0.75(0.6))}$$

$$q = (1.0)(3)\left(\frac{0.130}{12.0}\right) = 0.0325 \text{ kips per inch (LRFD, using a wind load factor of 1.0)}$$

ASD

$$s_{max} = (21.0)(12.0)/6 \le \frac{(2)(6.0)(0.222)}{(1.05)(0.0146)} \qquad (\textit{Eq. } \text{D1.1-1})$$

$= 42.0$ in. ≤ 174 in.

LRFD

$$s_{max} = (21.0)(12.0)/6 \le \frac{(2)(6.0)(0.334)}{(1.05)(0.0325)} \qquad (\textit{Eq. } \text{D1.1-1})$$

$= 42.0$ in. ≤ 117 in.

The spacing of 36 in. is OK for both ASD and LRFD, therefore section can be considered a built-up section in both cases.

b) Calculate the flexural strength according to Section C3.1.2.1.

$$F_e = \frac{C_b r_o A}{S_f}\sqrt{\sigma_{ey}\sigma_t} \qquad (\textit{Eq. } \text{C3.1.2.1-4})$$

$$\sigma_{ey} = \frac{\pi^2 E}{(K_y L_y / r_y)^2} \qquad (\textit{Eq. } \text{C3.1.2.1-8})$$

$$= \frac{\pi^2 (29500)}{[(1.0)(10.5)(12.0)/1.10]^2} = 22.2 \text{ ksi}$$

σ_t = 20.3 ksi (computed above)

C_b = 1.0 (conservatively)

$$F_e = \frac{(1.0)(3.59)(1.76)}{(2)(2.29)}\sqrt{(22.2)(20.3)} \qquad (\textit{Eq. } \text{C3.1.2.1-4})$$

= 29.3 ksi

Since $F_e < 0.56F_y = (0.56)(55) = 30.8$ ksi,

$F_c = F_e = 29.3$ ksi (*Eq*. C3.1.2.1-3)

It can be shown that the section is fully effective at an extreme fiber flexural stress of f = 29.3 ksi (calculations not shown), therefore:

$S_c = S_{gross} = (2)(2.29) = 4.58$ in.^{3}

$M_n = S_cF_c$ (*Eq*. C3.1.2.1-1)

$= (4.58)(29.3) = 134$ kip-in.

3. Combined Compression and Bending

a) ASD - check according to Section C5.2.1

ASCE/SEI 7-10 ASD load combination D + 0.75(0.6W) + $0.75L_r$ controls

Required strength

$P = P_D + 0.75P_{Lr}$

$= 1.0 + (0.75)(5.5) = 5.13$ kips

$$M_x = 0.75(0.6M_W) = 0.75\frac{0.6W_WL^2}{8}$$

$$= 0.75\frac{0.6(0.130)(21.0)^2}{8} = 3.23 \text{ kip-ft} = 38.7 \text{ kip-in.}$$

$M_y = 0$

Combined compression and bending

$$\frac{\Omega_cP}{P_n} + \frac{\Omega_bC_{mx}M_x}{M_{nx}\alpha_x} + \frac{\Omega_bC_{my}M_y}{M_{ny}\alpha_y} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

$\Omega_c = 1.80$

$\Omega_b = 1.67$

$$P_{Ex} = \frac{\pi^2EI_x}{(K_xL_x)^2} \qquad (Eq.\ C5.2.1\text{-}6)$$

$$= \frac{\pi^2(29500)(20.6)}{[(1.0)(21.0)(12.0)]^2} = 94.4 \text{ kips}$$

$$\alpha_x = 1 - \frac{\Omega_cP}{P_{Ex}} > 0 \qquad (Eq.\ C5.2.1\text{-}4)$$

$$= 1 - \frac{(1.80)(5.13)}{94.4} = 0.902$$

$C_{mx} = 1.0$

$$\frac{(1.80)(5.13)}{21.5} + \frac{(1.67)(1.0)(38.7)}{(134)(0.902)} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

$0.429 + 0.535 = 0.964 < 1.0$ OK

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

$P_{no} = A_e F_y$

$= (2)(24.3) = 48.6$ kips (from Table III-1)

$$\frac{(1.80)(5.13)}{48.6} + \frac{(1.67)(38.7)}{134} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

$0.190 + 0.482 = 0.672 < 1.0$ OK

b) LRFD - check according to Section C5.2.2

ASCE/SEI 7-10 LRFD load combination $1.2D + 1.0W + 0.5L_r$ controls

Required strength

$\overline{P} = P_u = 1.2P_D + 0.5P_{Lr}$

$= (1.2)(1.0) + (0.5)(5.5) = 3.95$ kips

$$\overline{M}_x = M_{ux} = 1.0\frac{W_W L^2}{8}$$

$$= 1.0\frac{(0.130)(21.0)^2}{8} = 7.17 \text{ kip-ft} = 86.0 \text{ kip-in.}$$

$\overline{M}_y = 0$

Combined compression and bending

$$\frac{\overline{P}}{\phi_c P_n} + \frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x} + \frac{C_{my}\overline{M}_y}{\phi_b M_{ny}\alpha_y} \leq 1.0 \qquad (Eq.\ C5.2.2\text{-}1)$$

$\phi_c = 0.85$

$\phi_b = 0.90$

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} \qquad (Eq.\ C5.2.2\text{-}6)$$

$$P_{Ex} = \frac{\pi^2(29500)(20.6)}{[(1.0)(21.0)(12.0)]^2} = 94.4 \text{ kips}$$

$$\alpha_x = 1 - \frac{\overline{P}}{P_{Ex}} \qquad (Eq.\ C5.2.2\text{-}4)$$

$$= 1 - \frac{3.95}{94.4} = 0.958$$

$C_{mx} = 1.0$

$$\frac{3.95}{(0.85)(21.5)} + \frac{(1.0)(86.0)}{(0.90)(134)(0.958)} \leq 1.0 \qquad (Eq.\ C5.2.2\text{-}1)$$

$0.216 + 0.744 = 0.960 < 1.0$ OK

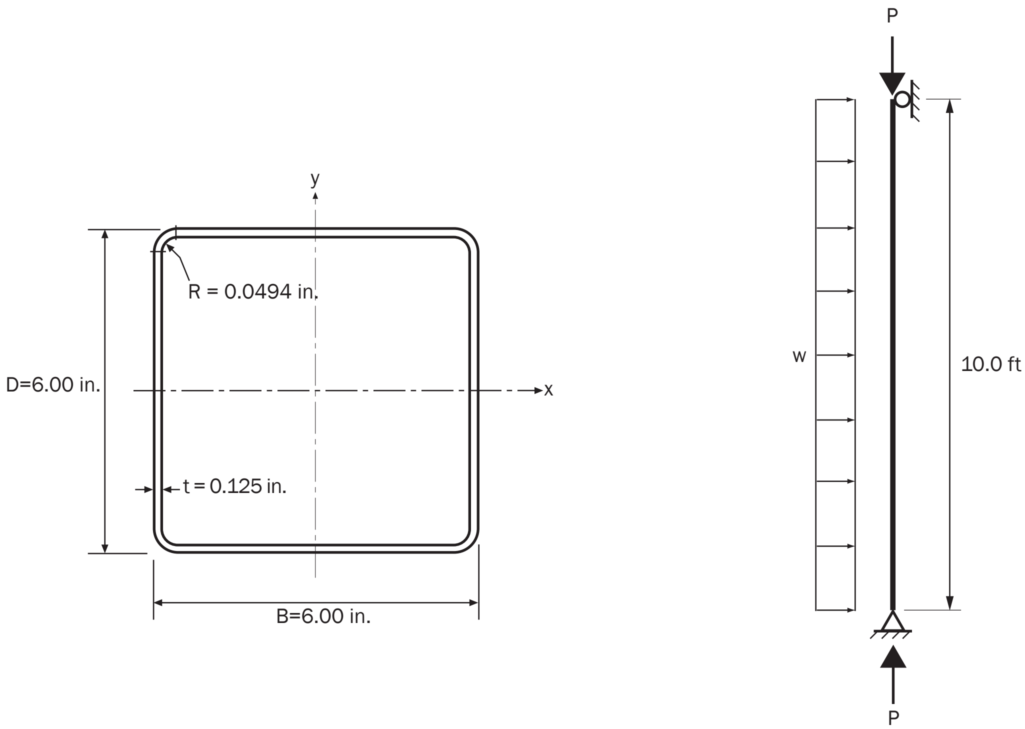
$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \leq 1.0 \qquad (\textit{Eq. } \text{C5.2.2-2})$$

from above, P_{no} = 48.6 kips

$$\frac{3.95}{(0.85)(48.6)} + \frac{86.0}{(0.90)(134)} \leq 1.0 \qquad (\textit{Eq. } \text{C5.2.2-2})$$

0.096+ 0.713 = 0.809 < 1.0 OK

Example III-11: Square HSS Section – Bending and Compression



Given:

1. Steel: ASTM A500 Grade B, F_y = 46 ksi, F_u = 58 ksi
2. Section: HSS 6x6x⅛ (See Table 1-12, 2011 AISC Steel Construction Manual)*
3. Simply supported at both ends
4. Braced about both x- and y-axis at both ends
5. $K_x = K_y = 1.0$, $L_x = L_y$ = 10.0 ft
6. Dead Load: P_D = 7.50 kips

   Live Load: P_L = 37.5 kips, w_L = 0.100 kips/ft

Required:

1. Determine the ASD allowable axial strength, P_n/Ω_c
2. Determine the LRFD design axial strength, $\phi_c P_n$
3. Compare the available strengths to those calculated per the 2010 AISC Specification†. The inside radius given above is selected to give the same flat width, w, for the flanges and webs used in the AISC calculations.
4. Verify the combined bending and compression strength of the section for the following ASCE/SEI 7-10 load combinations:

   a. ASD: D + L

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* AISC, *Steel Construction Manual – 14th Edition*, American Institute of Steel Construction, Chicago, IL, 2011.

† ANSI/AISC 360-10, *Specification for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, IL, 2011.

b. LRFD: 1.2D + 1.6L

Solution:

1. Nominal Axial Strength, P_n (Section C4.1)

Since the square tube is a doubly symmetrical closed section, it is not subject to flexural-torsional or distortional buckling. The nominal axial strength, P_n, can be computed according to *Specification* Sections C4.1(a) and C4.1.1.

a) Calculate the section properties of gross section using *Manual* Part I, Section 3.2, Properties of Line Elements.

i. Corner line elements (from Case 1 of *Manual* Part I, Section 3.2.2)

$$r = R + t/2 = 0.0494 + 0.125/2 = 0.112 \text{ in.}$$

$$l = 1.57r = (1.57)(0.112) = 0.176 \text{ in.}$$

$$c = 0.637r = (0.637)(0.112) = 0.0713 \text{ in.}$$

$$I_1 = 0.149r^3 = (0.149)(0.112)^3 = 0.000209 \text{ in.}^3$$

ii. Stiffened flange and web elements

$$w = B - 2(R+t) = 6.00 - 2(0.0494 + 0.125) = 5.651 \text{ in.}$$

iii. Gross section properties

$$A = 4(w+l)t = 4(5.651 + 0.176)(0.125) = 2.914 \text{ in.}^2$$

$$I_x = I_y = \left\{2\left[\frac{w^3}{12} + w\left(\frac{D}{2} - \frac{t}{2}\right)^2\right] + 4\left[I_1 + l\left(\frac{D}{2} - \frac{t}{2} - r + c\right)^2\right]\right\}t$$

$$= \left\{\begin{aligned} &2\left[\frac{5.651^3}{12} + 5.651\left(3.00 - \frac{0.125}{2}\right)^2\right] \\ &+4\left[0.000209 + 0.176\left(3.00 - \frac{0.125}{2} - 0.112 + 0.0713\right)^2\right]\end{aligned}\right\}0.125$$

$$= 16.69 \text{ in.}^4$$

$$r_x = r_y = \sqrt{16.69/2.914} = 2.393 \text{ in.}$$

b) Calculate the nominal flexural buckling stress, F_n.

$$\frac{K_xL_x}{r_x} = \frac{K_yL_y}{r_y} = \frac{(1.0)(10.0)(12)}{2.393} = 50.15$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 29500}{(50.15)^2} = 115.8 \text{ ksi} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{46}{115.8}} = 0.630 < 1.5 \qquad (Eq.\ C4.1\text{-}4)$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y = \left(0.658^{0.630^2}\right)46 = 38.96 \text{ ksi} \qquad (Eq.\ C4.1\text{-}2)$$

c) Calculate the effective area, A_e.

For stiffened flat elements

$w/t = 5.651/0.125 = 45.2$

$k = 4.0$

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 4.0\frac{\pi^2(29500)}{12\left(1-0.3^2\right)}\left(\frac{1}{45.2}\right)^2 = 52.20 \text{ ksi}$$

$\lambda = \sqrt{\frac{F_n}{F_{cr}}} = \sqrt{\frac{38.96}{52.20}} = 0.864 > 0.673$; therefore, flat elements are not fully effective. (*Eq.* B2.1-4)

$$\rho = (1-0.22/\lambda)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$

$$= (1-0.22/0.864)/0.864 = 0.863$$

$$b = \rho w = (0.863)(5.651) = 4.877 \text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

$$A_e = A - 4(w-b)t$$

$$= 2.914 - 4(5.651-4.877)(0.125) = 2.527 \text{ in.}^2$$

d) Calculate the nominal axial strength, P_n.

$$P_n = A_e F_n = 2.527(38.96) = 98.5 \text{ kips} \qquad (Eq.\ C4.1\text{-}1)$$

2. Available Axial Strengths

ASD

$$P \le \frac{P_n}{\Omega_c} \qquad (Eq.\ A4.1.1\text{-}1)$$

$$\Omega_c = 1.80$$

$$P \le \frac{98.5}{1.80} = 54.7 \text{ kips}$$

LRFD

$$P_u \le \phi_c P_n \qquad (Eq.\ A5.1.1\text{-}1)$$

$$\phi_c = 0.85$$

$$P_u \le 0.85(98.5) = 83.7 \text{ kips}$$

3. Comparison Between Available Axial Strengths Computed Using AISI and AISC Specifications

The available strengths determined in Parts 1 and 2 above are based on the AISI *Specification*. These values can be compared with those determined using the 2010 AISC Specification, as listed in Table 4-4 of the 14th Edition of the AISC Steel Construction Manual:

Available Strength	AISI (kips)	AISC (kips)	$\frac{\text{AISI}}{\text{AISC}}$
ASD: P_n/Ω_c	54.7	54.4	1.01
LRFD: $\phi_c P_n$	83.7	81.8	1.02

The above comparison shows that the available strengths based on the AISI and AISC specifications are practically the same, even though the design wall thickness, the design equations, and the safety and resistance factors differ as follows:

a) The AISI *Specification* uses the nominal wall thickness of 0.125 in., while the AISC Specification requires the use of a design wall thickness of 93% of the nominal thickness = 0.116 in.

b) The design equations for computing the nominal axial strengths differ. The AISI *Specification* uses the effective area multiplied by the nominal column buckling stress, while the AISC Specification uses the gross area multiplied by a reduced column buckling stress, using QF_y to replace F_y.

c) For the AISI *Specification*, Ω_c = 1.80 and ϕ_c = 0.85. For the AISC Specification, Ω_c = 1.67 and ϕ_c = 0.90.

d) The differences shown in the table above apply only to the HSS 6x6x1/8 with F_y = 46 ksi. For other sections and yield stresses, the AISI/AISC strength ratios may be slightly different.

4. Check Combined Compression and Bending – ASD (Section C5.2.1)

a) Required strength

$$M_x = M_{live} = \frac{wL^2}{8} = \frac{(0.100)(10.0)^2}{8} = 1.25 \text{ kip-ft}$$

$$P = P_{dead} + P_{live} = 7.50 + 37.5 = 45.0 \text{ kips}$$

b) From Part 1 above, P_n = 98.5 kips

$$\frac{\Omega_c P}{P_n} = \frac{1.80(45.0)}{98.5} = 0.822 > 0.15$$; therefore, use *Eqs*. C5.2.1-1 and C5.2.1-2

C_{mx} = 1.0

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} \quad (Eq.\ C5.2.1\text{-}6)$$

$$= \frac{\pi^2 (29500)(16.69)}{[(1.0)(120.0)]^2} = 337 \text{ kips}$$

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} > 0 \quad (Eq.\ C5.2.1\text{-}4)$$

$$= 1 - \frac{(1.80)(45.0)}{337} = 0.760$$

$M_y = 0.0$

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} + \frac{\Omega_b C_{my} M_y}{M_{ny}\alpha_y} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

From Example II-9;

M_{nx} = 19.0 kip-ft

$$\frac{(1.80)(45.0)}{98.5} + \frac{(1.67)(1.0)(1.25)}{(19.0)(0.760)} = 0.967 < 1.0 \quad \text{OK}$$

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

Computation of P_{no}

From Example II-9, for f = F_y = 46 ksi, b = 4.608 in.

$$A_e = A - 4(w-b)t$$

$$= 2.914 - 4(5.651 - 4.608)(0.125) = 2.393 \text{ in.}^2$$

$$P_{no} = A_e F_n = A_e F_y = (2.393)(46) = 110 \text{ kips}$$

$$\frac{(1.80)(45.0)}{110} + \frac{(1.67)(1.25)}{19.0} = 0.846 < 1.0 \quad \text{OK} \qquad (Eq.\ C5.2.1\text{-}2)$$

5. Check Combined Compression and Bending – LRFD (Section C5.2.2)

a) Required strength

$$\overline{M}_x = M_{ux} = \frac{1.6 w_{live} L^2}{8} = \frac{1.6(0.100)(10.0)^2}{8} = 2.00 \text{ kip-ft}$$

$$\overline{P} = P_u = 1.2P_{dead} + 1.6P_{live} = 1.2(7.50) + 1.6(37.5) = 69.0 \text{ kips}$$

a) From Part 1 above, P_n = 98.5 kips

$$\frac{\overline{P}}{\phi_c P_n} = \frac{69.0}{(0.85)(98.5)} = 0.824 > 0.15$$; therefore, use *Eqs*. C5.2.2-1 and C5.2.2-2

C_{mx} = 1.0

$$P_{Ex} = \frac{\pi^2 E I_x}{(K_x L_x)^2} \qquad (Eq.\ C5.2.2\text{-}6)$$

$$= \frac{\pi^2 (29500)(16.69)}{[(1.0)(120.0)]^2} = 337 \text{ kips}$$

$$\alpha_x = 1 - \frac{P}{P_{Ex}} > 0 \qquad (Eq.\ C5.2.2\text{-}4)$$

$$= 1 - \frac{69.0}{337} = 0.795$$

$\overline{M}_y = 0.0$

$$\frac{\overline{P}}{\phi_c P_n} + \frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x} + \frac{C_{my}\overline{M}_y}{\phi_b M_{ny}\alpha_y} \le 1.0 \qquad (\textit{Eq.}\ \text{C5.2.2-1})$$

From Example II-9;

M_{nx} = 19.0 kip-ft

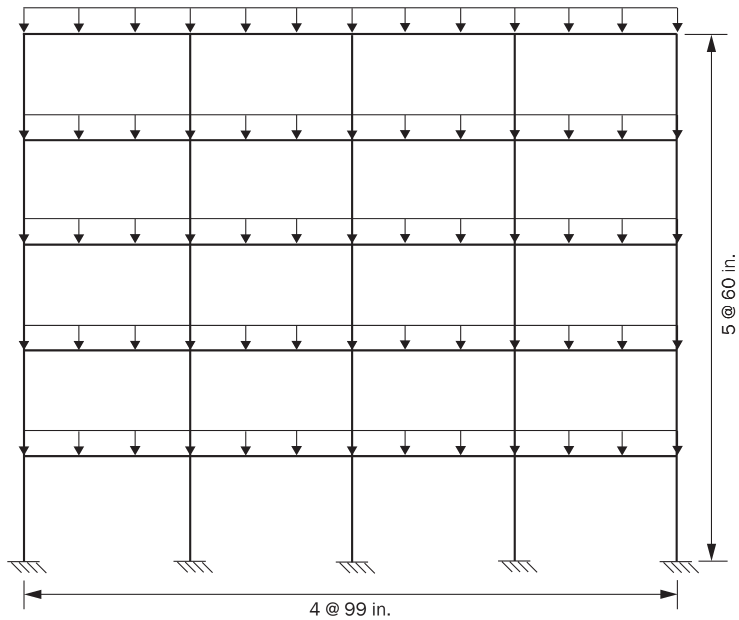
$$\frac{69.0}{(0.85)(98.5)} + \frac{(1.0)(2.0)}{(0.90)(19.0)(0.795)} = 0.971 < 1.0 \quad \text{OK}$$

$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \le 1.0 \qquad (\textit{Eq.}\ \text{C5.2.2-2})$$

P_{no} = 110 kips from Part 4 above

$$\frac{69.0}{(0.85)(110)} + \frac{2.0}{(0.90)(19.0)} = 0.855 < 1.0 \quad \text{OK} \qquad (\textit{Eq.}\ \text{C5.2.2-2})$$

Example III-12: Unbraced Frame Design by Second-Order Analysis



Given:

1. Steel: F_y = 55 ksi
2. Frame as shown above
3. All beams uniformly loaded with 35.7 lbs/in. (LRFD factored loading)
4. Column properties

   A_g = 0.936 in.^{2}

   I_x=I_y = 1.27 in.^{4}

   r_x=r_y = 1.16 in.

   S_e = 0.847 in.^{3}
5. Beam properties

   A_g = 0.780 in.^{2}

   I_x = 1.70 in.^{4}
6. Columns are not subject to local, distortional or flexural-torsional buckling.
7. Connections are semi-rigid with the following nominal rigidities:

   Beam ends: 750 kip-in./radian

   Column bases: 3000 kip-in./radian

Required:

1. Check the adequacy of the typical interior column at the ground level using the provisions of Appendix 2, *Second-Order Analysis,* with LRFD.
2. Check the adequacy of the typical interior column at the ground level using the rational effective length procedure with LRFD.

Solution:

1. Second-Order Analysis Procedure (Appendix 2)

A second-order analysis is required using 1) reduced stiffness to account for the effects of inelasticity, and 2) notional loads to account for structure out-of-plumbness. Under these conditions, the effective length factor in the plane of the frame, K_x, may be taken as 1.0.

a) Compute the reduced axial and flexural stiffnesses per *Specification* Appendix 2, Section 2.2.3

Although the *Specification* requires member stiffness reductions only for "members whose axial and flexural stiffness are considered to contribute to the lateral stability of the structure," as a practical matter, reducing the stiffness of all members in the computer analysis is much more convenient. The required modified modulus of elasticity is:

$$E^* = 0.8\tau_b E \qquad (Eq.\ 2\text{-}1)$$

where

$$\tau_b = 1.0 \text{ for } \alpha P_{ra}/P_y \le 0.5$$

$$= 4\left[\alpha P_{ra}/P_y\left(1-\alpha P_{ra}/P_y\right)\right] \text{ for } \alpha P_{ra}/P_y > 0.5$$

Assume $\alpha P_{ra}/P_y \le 0.5$ for all members; therefore τ_b = 1.0. Confirm this assumption after the analysis.

$$E^* = 0.8(1.0)29500 = 23600 \text{ ksi} \qquad (Eq.\ 2\text{-}1)$$

b) Compute the reduced connection stiffnesses per *Specification* Appendix 2, Section 2.2.3

Reduce the nominal connection stiffnesses in the computer model to 80% of their nominal value as required by Section 2.2.3.

Column bases:

$$K_{col} = 0.8(3000) = 2400 \text{ kip-in./radian}$$

Beam ends:

$$K_{beam} = 0.8(750) = 600 \text{ kip-in./radian}$$

c) Compute the notional loads per *Specification* Appendix 2, Section 2.2.4

Notional loads are lateral loads applied in the second-order analysis at each level to account for the effects of out-of-plumbness. Alternatively, the structure can be modeled with the out-of-plumb geometry. Unless the analysis software includes provisions for automatically generating the out-of-plumb geometry, the use of notional loads is usually simpler. The required notional load at each beam level is calculated as:

$$N_i = (1/240)Y_i$$

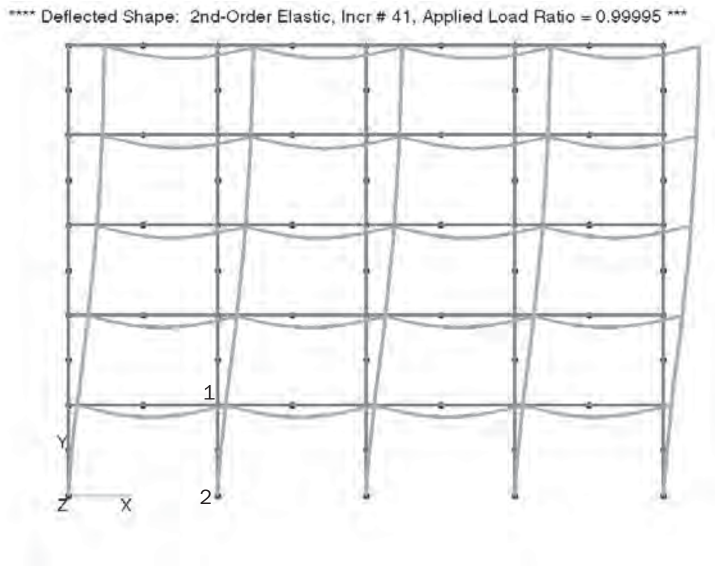
The gravity load, Y_i, at each level is:

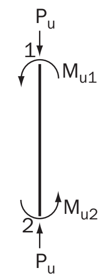
$$Y_i = 35.7(4)(99.0) = 14{,}100 \text{ lbs}$$

The notional load, N_i, at each level is:

$$N_i = (1/240)(14{,}100) = 58.8 \text{ lbs}$$

Using the specified gravity and notional loading and the reduced stiffness, the frame is analyzed using MASTAN2*. The resulting deformed shape is shown below.





The resulting axial force and moments in the most heavily loaded interior column are found to be:

P_u = 17.7 kips

M_{u1} = 8.88 kip-in. at the bottom of the column

M_{u2} = 3.55 kip-in. at the top of the column

2. Second-Order Combined Strength Check

a) Check the assumption that $P_{ra}/P_y \le 0.5$. For the member with the largest axial load:

$P_{ra} = P_u = 17.7$ kips

$\frac{P_{ra}}{P_y} = \frac{17.7}{(0.936)(55)} = 0.344 < 0.5$; therefore, assumption that τ_b = 1.0 is OK.

b) Calculate the axial strength.

Since the column is not subject to local, distortional or flexural-torsional buckling, the strength is governed by flexural column buckling.

$K_xL_x = K_yL_y = (1.0)(60.0) = 60.0$ in.

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2 29500}{(60.0/1.16)^2} = 109 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

* Ziemian, R.D. and McGuire, W, *MASTAN2*, available at www.mastan2.com.

$$= \sqrt{\frac{55}{109}} = 0.710 < 1.5 \text{; therefore,}$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4.1\text{-}2)$$

$$= \left(0.658^{0.710^2}\right)55 = 44.5 \text{ ksi}$$

$$P_n = A_eF_n \qquad (Eq.\ C4.1\text{-}1)$$

$$= (0.936)(44.5) = 41.7 \text{ kips}$$

c) Calculate the flexural strength.

Since the column is not subject to local, distortional or lateral-torsional buckling, the flexural yield strength governs.

$$M_n = S_eF_y \qquad (Eq.\ C3.1.1\text{-}1)$$

$$= (0.847)(55) = 46.6 \text{ kip-in.}$$

d) Check the combined strength.

Since the required moments were determined by a second-order analysis in accordance with Appendix 2, Section 2.1, C_m and α are both taken as 1.0; therefore, *Eq*. C5.2.2-1 will govern.

$\overline{P} = P_u$ = 17.7 kips

$\overline{M} = M_u$ = 8.88 kip-in.

$\phi_c = 0.85$

$\phi_b = 0.90$

$$\frac{\overline{P}}{\phi_c P_n} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

$$\frac{17.7}{0.85(41.7)} + \frac{8.88}{0.90(46.6)} = 0.711 \le 1.0 \text{ OK}$$

3. Effective Length Analysis

In the traditional effective length approach, in-plane effective length factors, K_x, are determined by some means, often through the use of "alignment charts", such as those published by AISC in the *Steel Construction Manual**. In this case, the presence of multiple levels of semi-rigid connections complicates this approach. Alternatively, the elastic buckling stress, F_e, can be directly determined by an elastic buckling analysis of the entire structure.

Using nominal member and connection stiffnesses and no notional loads, the smallest elastic buckling load of the structure was determined to be 1.75 times the factored load using the "elastic critical load" analysis of MASTAN2. For column 1-2, the axial load in the column under gravity load, P_u, is found to be 17.7 kips by a separate first-order analysis. The elastic buckling stress of the column in the plane of the frame, F_{ex}, can then be calculated as:

* AISC, *Steel Construction Manual – 14th Edition*, American Institute of Steel Construction Inc., Chicago, IL 2011.

$$F_{ex} = 1.75\frac{P_u}{A} = 1.75\left(\frac{17.7}{0.936}\right) = 33.1\text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{55}{33.1}} = 1.29 < 1.5\text{; therefore}$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4.1\text{-}2)$$

$$= \left(0.658^{1.29^2}\right)55 = 27.4\text{ ksi}$$

$$P_n = A_eF_n \qquad (Eq.\ C4.1\text{-}1)$$

$$= (0.936)(27.4) = 25.6\text{ kips}$$

Since there are no significant bending moments in the interior columns under this loading in the first-order analysis, an axial strength check is sufficient.

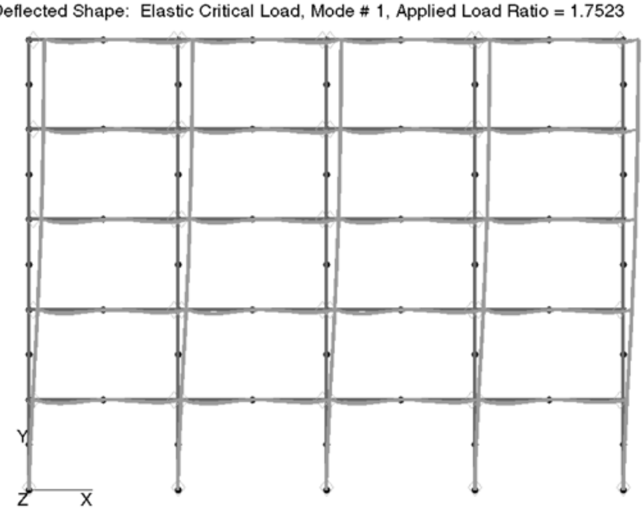
$P_u = 17.7$ kips

$\phi_c = 0.85$

$$\frac{P_u}{\phi_c P_n} \le 1.0 \qquad (Eq.\ A5.1.1\text{-}1)$$

$$\frac{17.7}{0.85(25.6)} = 0.813 \le 1.0\ \text{OK}$$

Deflected Shape: Elastic Critical Load, Mode # 1, Applied Load Ratio = 1.7523



Both methods show that the lower columns of the frame are acceptable, but the second-order analysis is less conservative in this case.

Example III-13: Braced Frame Design by Second-Order Analysis

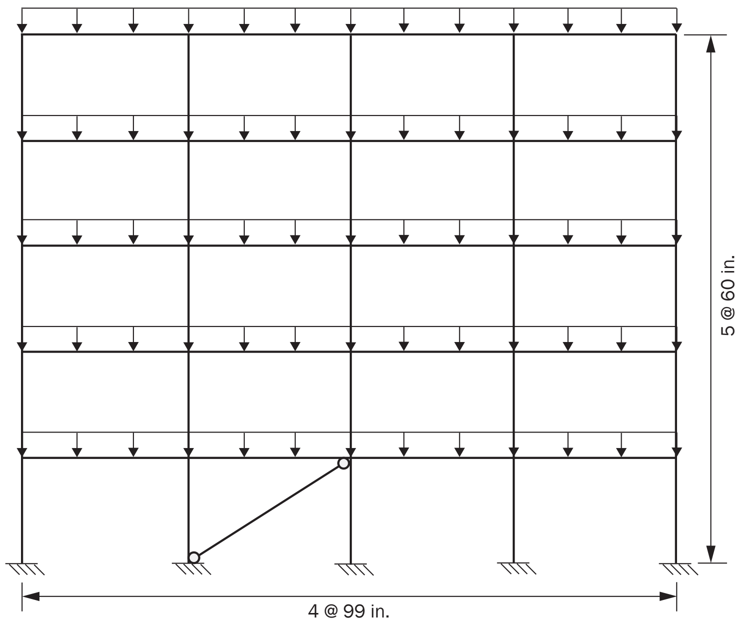


Figure 1

Geometry and Loading

Given:

1. Frame and Loading from Example III-11
2. Additional diagonal bracing as shown in Figure 1

Required:

1. Determine the axial force in a 0.180" x 2" diagonal tension strap added to the lowest tier as shown in Figure 1.
2. Verify that the area of the tension strap is adequate to limit the first level LRFD drift to less than 0.075 in.
3. Verify that the maximum LRFD second-order moments of the interior three columns will be reduced to less than 3.50 kip-in.

Solution:

Section D3.3 states that the required strength and required stiffness of the brace are permitted to be obtained by a second-order analysis in accordance with Appendix 2. Using MASTAN2*, an elastic second-order analysis of the frame is conducted identical to the one in the previous example with the exception of the added diagonal brace, where the stiffness of the diagonal brace is reduced as:

$$k_{brace} = 0.8\left(\frac{EA}{L}\right) = 0.8\left[\frac{(29500)(0.180)(2)}{\sqrt{(99)^2+(60)^2}}\right] = 73.4 \text{ kips/in.}$$

The following results are taken from the analysis:

1. The force in the brace is calculated to be 0.668 kip.
2. The first level LRFD drift is calculated to be 0.0631 in. < 0.075 in. O.K.
3. The maximum second-order moment of the three interior columns is calculated to be 3.12 kip-in. < 3.50 kip-in. O.K.

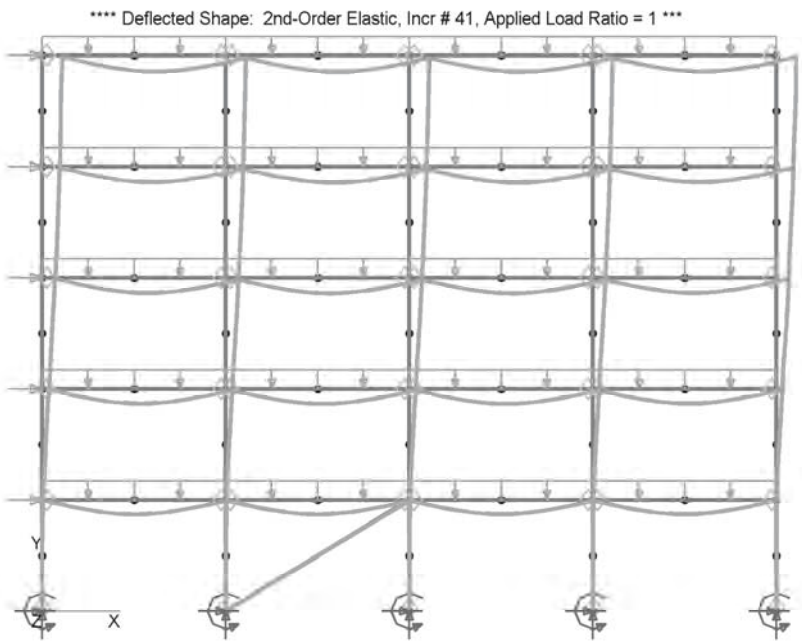
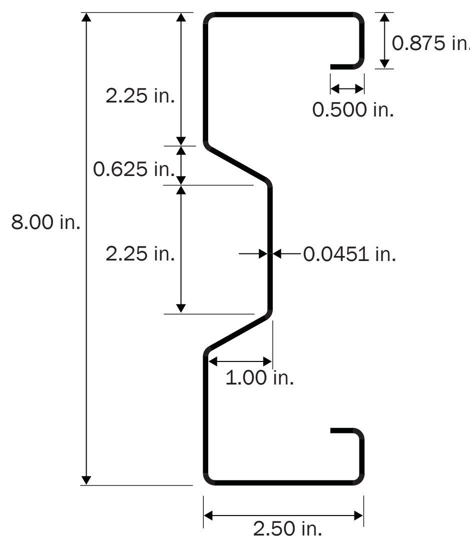


Figure 2
Deflected Shape

* Ziemian, R.D. and McGuire, W, *MASTAN2*, available at www.mastan2.com

Example III-14: Web-Stiffened C-Section by the Direct Strength Method - Compression



Given:

1. Steel: F_y = 50 ksi, F_u = 65 ksi
2. Sigma section (C-Section with web stiffener) as shown above

Required:

Calculate the ASD and LRFD available compression strengths using the Direct Strength procedure from *Specification* Appendix 1. Consider the two cases of:

1) Continuously braced against flexural, flexural-torsional and distortional buckling
2) Discretely braced against flexural, flexural-torsional and distortional buckling at a spacing of 66.0 in.

Solution:

Although the Direct Strength Method may be used for any cross-section, it is particularly well suited to this example, since the cross-section is somewhat complex and the *Specification* has no provisions applicable to the complex edge stiffeners on the flanges.

1. Perform a Finite Strip Analysis

A finite strip analysis of the cross-section is performed using a program such as CUFSM*. A pure axial stress distribution is assumed with the fibers at F_y in compression. Results from a CUFSM analysis include the axial strength under the assumed stress distribution, P_y, and a graph of the section buckling strength versus unbraced length, shown below. Examination of the mode shape for the member at a length of 66 in. shows both lateral translation associated with flexural buckling and distortion of the cross-section associated with distortional buckling; consequently, the elastic

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* Schafer, B.W., Ádány, S. "Buckling analysis of cold-formed steel members using CUFSM: conventional and constrained finite strip methods," *Eighteenth International Specialty Conference on Cold-Formed Steel Structures*, Orlando, FL. October 2006. Available at www.ce.jhu.edu/bschafer/cufsm.

buckling load at this length is used for the distortional buckling limit state check. The dashed line superimposed on the right half of the graph represents the global buckling mode isolated from other limit states. The elastic buckling load at this length from this line is used for the global buckling limit state check below. The critical buckling strengths are obtained by multiplying the yield strength, P_y, by the corresponding load factors obtained from the finite strip analysis.

From the analysis:

Yield strength

$$P_y = 37.4 \text{ kips}$$

Critical elastic local buckling strength

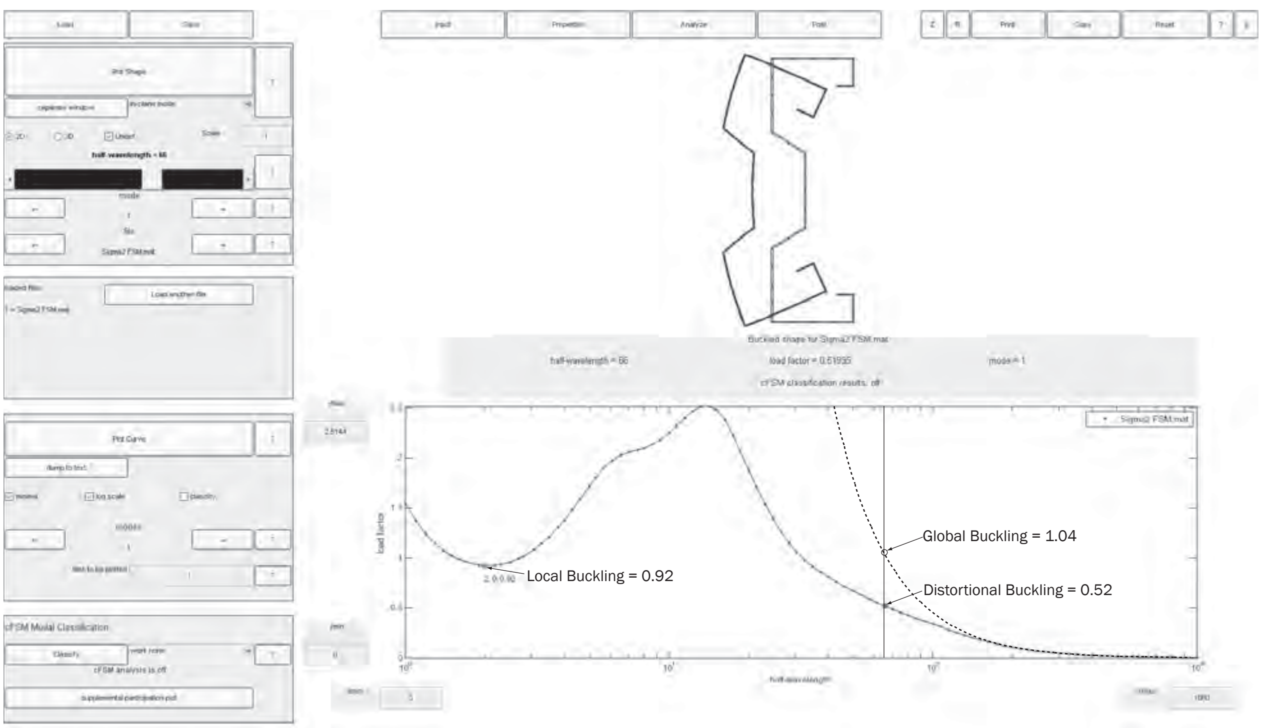
$$P_{cr\ell} = 0.92P_y = (0.92)(37.4) = 34.4 \text{ kips}$$

Critical elastic flexural buckling strength at 66.0 in.

$$P_{cre} = 1.04P_y = (1.04)(37.4) = 38.9 \text{ kips}$$

Critical elastic distortional buckling strength at 66.0 in.

$$P_{crd} = 0.52P_y = (0.52)(37.4) = 19.4 \text{ kips}$$



2. Calculate the Nominal Axial Strength

Per Section 1.2.1 of Appendix 1, take P_n as the lowest of the nominal strengths for flexural, torsional, or flexural-torsional buckling, P_{ne}, local buckling, $P_{n\ell}$ and distortional buckling, P_{nd}.

Case 1: The member is fully braced against global buckling and distortional buckling

1) Global buckling: The member is fully braced against global buckling; therefore,

$P_{ne} = P_y = 37.4$ kips

2) Local buckling:

$$\lambda_\ell = \sqrt{P_{ne}/P_{cr\ell}} \qquad (Eq.\ 1.2.1\text{-}7)$$

$$= \sqrt{37.4/34.4} = 1.04$$

Since $\lambda_\ell > 0.776$,

$$P_{n\ell} = \left(1 - 0.15\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4}\right)\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4} P_{ne} \qquad (Eq.\ 1.2.1\text{-}6)$$

$$= \left(1 - 0.15\left(\frac{34.4}{37.4}\right)^{0.4}\right)\left(\frac{34.4}{37.4}\right)^{0.4} 37.4 = 30.9\text{ kips}$$

3) Distortional buckling: The member is fully braced against distortional buckling; therefore,

$P_{nd} = P_y = 37.4$ kips

Case 2: The member is discretely braced against global buckling and distortional buckling at 66.0 in. on center.

1) Global buckling: From the finite strip analysis, at 66.0 in.,

$$\lambda_c = \sqrt{P_y/P_{cre}} \qquad (Eq.\ 1.2.1\text{-}3)$$

$$= \sqrt{37.4/38.9} = 0.981$$

Since $\lambda_c < 1.5$,

$$P_{ne} = \left(0.658^{\lambda_c^2}\right)P_y \qquad (Eq.\ 1.2.1\text{-}1)$$

$$= \left(0.658^{0.981^2}\right)37.4 = 25.0\text{ kips}$$

2) Local buckling:

$$\lambda_\ell = \sqrt{P_{ne}/P_{cr\ell}} \qquad (Eq.\ 1.2.1\text{-}7)$$

$$= \sqrt{25.0/34.4} = 0.852$$

Since $\lambda_\ell > 0.776$,

$$P_{n\ell} = \left(1 - 0.15\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4}\right)\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4} P_{ne} \qquad (Eq.\ 1.2.1\text{-}6)$$

$$= \left(1 - 0.15\left(\frac{34.4}{25.0}\right)^{0.4}\right)\left(\frac{34.4}{25.0}\right)^{0.4} 25.0 = 23.6\text{ kips}$$

3) Distortional buckling:

$$\lambda_d = \sqrt{P_y/P_{crd}} \qquad (Eq.\ 1.2.1\text{-}12)$$

$$= \sqrt{37.4/19.4} = 1.39$$

Since $\lambda_d > 0.561$,

$$P_{nd} = \left(1 - 0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right)\left(\frac{P_{crd}}{P_y}\right)^{0.6} P_y \qquad (Eq.\ 1.2.1\text{-}11)$$

$$= \left(1 - 0.25\left(\frac{19.4}{37.4}\right)^{0.6}\right)\left(\frac{19.4}{37.4}\right)^{0.6} 37.4 = 21.0 \text{ kips}$$

4) The nominal axial strengths are therefore 30.9 kips for Case 1 (braced against global and distortional buckling), governed by local buckling, and 21.0 kips for Case 2 (braced at 66.0 in on center), governed by distortional buckling.

3. Calculate the Available Strengths

Check the limitations for prequalified beams in Table 1.1.1-1 to determine the appropriate safety and resistance factors. Since there is no prequalified category for C-Sections with web stiffeners and complex lips, use the safety and resistance factors from Section A1.2(c).

Case 1: Continuously braced against global and distortional buckling

ASD - Allowable strength

$$\frac{P_n}{\Omega} = \frac{30.9}{2.00} = 15.5 \text{ kips} \qquad (Eq.\ A4.1.1\text{-}1)$$

LRFD - Design strength

$$\phi P_n = 0.80(30.9) = 24.7 \text{ kips} \qquad (Eq.\ A5.1.1\text{-}1)$$

Case 2: Discretely braced against global and distortional buckling at 66.0 in. on center

ASD - Allowable strength

$$\frac{P_n}{\Omega} = \frac{21.0}{2.00} = 10.5 \text{ kips} \qquad (Eq.\ A4.1.1\text{-}1)$$

LRFD - Design strength

$$\phi P_n = 0.80(21.0) = 16.8 \text{ kips} \qquad (Eq.\ A5.1.1\text{-}1)$$

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TABLE OF CONTENTS

PART IV
CONNECTIONS DESIGN

For Use With the
2012 Edition of the
North American Specification for the Design
of Cold-Formed Steel Structural Members

SECTION 1 - WELDS

The application of welded connections must comply with the requirements set forth in Section E2 of the *Specification*. The *Specification* applies to the welding of parts where the thinnest part is 3⁄16 in. or less. For welded connections in which the thickness of the thinnest connected part is greater than 3⁄16 in., refer to the AISC Specification[1]. Welds shall be made in accordance with AWS D1.3, except resistance welds which shall be in accordance with AWS C1.3.

The provisions governing welds are organized by weld type in Sections E2.1 through E2.7. With the exception of resistance spot welds, the welded connections are subject to the limit states of:

1. Base metal rupture
2. Weld metal rupture

These must be separately checked and the lower of the two strengths is used. For welded connections in which not all elements of the cross-section are used to transmit force, Section E6 requires the consideration of rupture of the thinnest connected part.

1.1 Notes on the Tables

Shown in Table IV-1 are the nominal shear strengths [resistances] for unit length fillet welds made on various sheet thicknesses and for sheet tensile strengths of 45 ksi and 65 ksi. The nominal weld shear strength is found by interpolating between values in the Table, then multiplying by the length of fillet weld used (adding values for longitudinal plus transverse welds). For ASD, the weld design shear strength is determined by dividing the nominal weld shear strength by Ω. For LRFD and LSD, the weld design shear strength [factored resistance] is found by multiplying the nominal weld shear strength [resistance] by ϕ.

Nominal shear strengths [resistances] of resistance welds, “spot welds”, are provided in Table IV-2. For ASD, the weld allowable shear strength is determined by dividing the nominal weld shear strength by Ω. For LRFD and LSD, the weld design shear strength [factored resistance] is found by multiplying the nominal weld shear strength [resistance] by ϕ.

Table IV-3 gives the nominal shear strengths [resistances] for sheets welded to thicker supporting members with 1/2 in., 5/8 in., and 3/4 in. arc spot welds, based on sheet strength. Values are provided for sheet tensile strengths of 45 ksi and 65 ksi. Nominal strengths are determined by interpolation based on the total sheet thickness being welded to the supporting structure. For ASD, the weld allowable shear strength is determined by dividing the nominal weld shear strength by Ω. For LRFD and LSD, the weld design shear strength [factored resistance] is found by multiplying the nominal weld shear strength [resistance] by ϕ. The strength of the weld metal must also be checked using *Specification Eq*. E2.2.2.1-1.

Table IV-4 gives the nominal shear strengths [resistances] for a sheet welded to an identical sheet with 1/2 in., 5/8 in., and 3/4 in. arc spot welds, based on sheet strength. Values are provided for sheet tensile strengths of 45 ksi. Nominal strengths [resistances] are determined by interpolation based on the thickness of one of the two identical sheets. For ASD, the weld allowable shear strength is determined by dividing the nominal weld shear strength by Ω. For LRFD and LSD, the weld design shear strength [factored resistance] is found by multiplying the nominal weld shear strength [resistance] by ϕ. The strength of the weld metal, F_{xx}, must be greater than 45 ksi.

Table IV-5 gives the nominal tensile strengths [resistances] for concentrically loaded 1/2 in., 5/8 in., and 3/4 in. arc spot welds based on sheet strength [resistance]. Values are provided for sheet tensile strengths of 45 ksi and 65 ksi. Nominal strengths [resistances] are determined by interpolation based on the total sheet thickness being welded to the supporting structure. When

[1] AISC-360, *Specification for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, IL, 2010.

used as side lap connectors within a deck system, these values must be reduced 30 percent. In other eccentric connections, these values must be reduced by 50 percent. For ASD, the weld allowable shear strength is determined by dividing the nominal weld shear strength by Ω. For LRFD and LSD, the weld design shear strength [factored resistance] is found by multiplying the nominal weld shear strength [resistance] by ϕ. The strength [resistance] of the weld metal must also be checked using *Specification Eq*. E2.2.3-1.

1.2 Welded Connection Design Tables

Table IV - 1

Fillet Welds
Shear of Sheet[1,2,3]

Ω = See Table
ϕ = See Table

Nominal Shear Strength [Resistance] of a 1 in. long weld, P′ₙ, kips/in.

Sheet Thickness in.	F_u = 45 ksi						F_u = 65 ksi					
	Ω (ASD) = 2.55					3.05 / 2.35	Ω (ASD) = 2.55					3.05 / 2.35
	ϕ (LRFD) = 0.60					0.50 / 0.65	ϕ (LRFD) = 0.60					0.50 / 0.65
	ϕ (LSD) = 0.50					0.40 / 0.60	ϕ (LSD) = 0.50					0.40 / 0.60
	Weld Length/Sheet Thickness[4], L / t Longitudinal					Trans-verse	Weld Length/Sheet Thickness[4], L / t Longitudinal					Trans-verse
	5	10	15	20	≥25		5	10	15	20	≥25	
0.036	1.54	1.46	1.38	1.30	1.22	1.62	2.22	2.11	1.99	1.87	1.76	2.34
0.048	2.05	1.94	1.84	1.73	1.62	2.16	2.96	2.81	2.65	2.50	2.34	3.12
0.060	2.57	2.43	2.30	2.16	2.03	2.70	3.71	3.51	3.32	3.12	2.93	3.90
0.075	3.21	3.04	2.87	2.70	2.53	3.38	4.63	4.39	4.14	3.90	3.66	4.88
0.090	3.85	3.65	3.44	3.24	3.04	4.05	5.56	5.27	4.97	4.68	4.39	5.85
0.105	4.49	4.25	4.02	3.78	3.54	4.73	6.48	6.14	5.80	5.46	5.12	6.83
0.135	5.77	5.47	5.16	4.86	4.56	6.08	8.34	7.90	7.46	7.02	6.58	8.78

Note: the Ω/ϕ values for the ≥25 column (first value) and the Transverse column (second value) are given in the form "≥25 / Transverse": Ω (ASD) 3.05 / 2.35; ϕ (LRFD) 0.50 / 0.65; ϕ (LSD) 0.40 / 0.60.

Notes:
1. Available Strengths [factored resistances] are:
   ASD: $P'_n \times L / \Omega$
   LRFD, LSD: $\phi P'_n \times L$
   where L = weld length in inches
   t = thickness of welded sheet
2. Check each sheet individually and use lowest value
3. *Specification* equation E2.5-7 must also be checked when t > 0.10 in.
4. t = thickness of thinnest sheet

Table IV - 2

Resistance ("Spot") Welds
Shear Strength[1]

Ω (ASD) = 2.35
ϕ (LRFD) = 0.65
ϕ (LSD) = 0.55

Nominal Shear Strength [Resistance], P_n, kips

Sheet Thickness (in.)									
0.012	0.024	0.036	0.048	0.060	0.075	0.090	0.105	0.135	0.165
0.216	0.599	1.09	1.66	2.30	3.20	4.18	5.24	7.58	9.09

Note:

1. Available Strengths [factored resistances] are:
   ASD: P_n / Ω
   LRFD, LSD: ϕP_n

Table IV - 3

Arc Spot Welds
Shear of Sheet(s) Welded
to a Thicker Supporting Member

Nominal Shear Strength [Resistance] per Weld, P_n, kips[1,2]

ASD
Ω = 3.05 above double line
= 2.80 above heavy line
= 2.20 below heavy line

LRFD
ϕ = 0.50 above double line
= 0.55 above heavy line
= 0.70 below heavy line

LSD
ϕ = 0.40 above double line
= 0.45 above heavy line
= 0.60 below heavy line

Total Sheet Thickness above Shear Plane, in.	F_u = 45 ksi Visible Diameter, in.			F_u = 65 ksi Visible Diameter, in.		
	1/2	5/8	3/4	1/2	5/8	3/4
0.015	0.497	0.576	0.695	0.662	0.833	1.00
0.020	0.842	0.874	0.920	1.04	1.10	1.33
0.025	1.18	1.32	1.36	1.57	1.63	1.68
0.030	1.40	1.77	1.90	2.02	2.28	2.34
0.035	1.61	2.04	2.48	2.33	2.95	3.11
0.040	1.82	2.32	2.81	2.63	3.35	3.98
0.045	2.03	2.58	3.14	2.93	3.73	4.54
0.050	2.23	2.85	3.47	3.22	4.11	5.01
0.055	2.42	3.10	3.78	3.50	4.48	5.47
0.060	2.61	3.36	4.10	3.78	4.85	5.92
0.065	2.80	3.60	4.41	4.04	5.21	6.37
0.070	2.98	3.85	4.71	4.30	5.56	6.81
0.075	3.16	4.08	5.01	4.56	5.90	7.24
0.080	3.33	4.32	5.31	4.80	6.23	7.66
0.085	3.49	4.54	5.60	5.04	6.56	8.08
0.090	3.65	4.77	5.88	5.28	6.89	8.49
0.095	3.81	4.98	6.16	5.50	7.20	8.90
0.100	3.96	5.20	6.44	5.72	7.51	9.30
0.105	4.11	5.41	6.70	5.93	7.81	9.68
0.110	4.25	5.61	6.97	6.13	8.10	10.1
0.115	4.38	5.81	7.23	6.33	8.39	10.4
0.120	4.51	6.00	7.48	6.52	8.67	10.8
0.125	4.64	6.19	7.73	6.70	8.94	11.2
0.130	4.76	6.37	7.98	6.88	9.20	11.5
0.135	4.88	6.55	8.22	7.05	9.46	11.9
0.140	4.99	6.72	8.45	7.21	9.71	12.2
0.145	5.10	6.89	8.68	7.36	9.95	12.5
0.150	5.20	7.05	8.91	7.51	10.2	12.9

Notes:
1. Available Strengths [factored resistances] are:
   ASD: P_n / Ω
   LRFD, LSD: ϕP_n
2. The nominal shear strength [resistance] given in Eq. E2.2.2.1-1 of the *Specification* is not considered in Table IV-3 and must be checked.

Table IV - 4

Arc Spot Welds
Shear of Sheet Welded
to an Identical Sheet

Nominal Shear Strength [Resistance] per Weld, P_n, kips[1,2]

ASD
Ω = 2.20

LRFD
ϕ = 0.70

LSD
ϕ = 0.60

Total Sheet Thickness above Shear Plane, in.	F_u = 45 ksi		
	Visible Diameter, in.		
	1/2	5/8	3/4
0.030	1.05	1.33	1.60
0.035	1.21	1.53	1.86
0.040	1.37	1.74	2.11
0.045	1.52	1.94	2.36
0.050	1.67	2.13	2.60
0.055	1.82	2.33	2.84
0.060	1.96	2.52	3.07

Notes:
1. Available Strengths [Factored Resistances] are:
   ASD: P_n / Ω
   LRFD, LSD: ϕP_n
2. F_{xx} > 45 ksi required.

Table IV - 5

Arc Spot Welds
Tension

Nominal Tensile Strength per Weld, P_n, kips[1,2,3,4]

ASD
Ω = 2.50 for panel and deck
= 3.00 for other applications

LRFD
ϕ = 0.60 for panel and deck
= 0.50 for other applications

LSD
ϕ = 0.50 for panel and deck
= 0.40 for other applications

Total Sheet Thickness, in.	F_y = 33 ksi, F_u = 45 ksi			F_y = 50 ksi, F_u = 65 ksi		
	Visible Diameter, in.			Visible Diameter, in.		
	1/2	5/8	3/4	1/2	5/8	3/4
0.015	0.487	0.613	0.738	0.639	0.804	0.969
0.020	0.643	0.810	0.977	0.844	1.06	1.28
0.025	0.795	1.00	1.21	1.04	1.32	1.59
0.030	0.944	1.19	1.45	1.24	1.57	1.90
0.035	1.09	1.38	1.68	1.43	1.81	2.20
0.040	1.23	1.57	1.90	1.62	2.06	2.50
0.045	1.37	1.75	2.12	1.80	2.29	2.79
0.050	1.51	1.92	2.34	1.98	2.53	3.08
0.055	1.64	2.10	2.56	2.15	2.76	3.36
0.060	1.77	2.27	2.77	2.32	2.98	3.64
0.065	1.89	2.44	2.98	2.48	3.20	3.91
0.070	2.01	2.60	3.19	2.65	3.41	-
0.075	2.13	2.76	3.39	2.80	3.63	-
0.080	2.25	2.92	3.59	2.95	3.83	-
0.085	2.36	3.07	3.78	3.10	4.03	-
0.090	2.47	3.22	3.98	3.24	-	-
0.095	2.58	3.37	4.17	3.38	-	-
0.100	2.68	3.51	4.35	3.52	-	-
0.105	2.78	3.66	-	3.64	-	-
0.110	2.87	3.79	-	3.77	-	-
0.115	2.96	3.93	-	3.89	-	-
0.120	3.05	4.06	-	4.01	-	-
0.125	3.14	4.18	-	-	-	-
0.130	3.22	4.31	-	-	-	-
0.135	3.30	4.43	-	-	-	-
0.140	3.37	-	-	-	-	-
0.145	3.45	-	-	-	-	-
0.150	3.51	-	-	-	-	-

Notes:
1. Available Strengths [Factored Resistances] are:
   ASD: P_n / Ω
   LRFD, LSD: ϕP_n
2. The nominal tensile strength [resistance] given in Eq. E2.2.3-1 of the *Specification* is not considered in Table IV-5 and must be checked.
3. The limitations related to weld electrode strength, F_{xx}, have not been checked in this table and must be checked per Section E2.2.3.
4. Dashed values indicate that the limit $t d_a F_u \leq 3$ kips has not been satisfied.

SECTION 2 - BOLTS

Bolts, washers and nuts approved for use with cold-formed members are listed in the *Specification* in Section E3. The application must comply with the requirements set forth in Section E3. The *Specification* applies to the bolting of cold-formed steel structural members in which the thickness of the thinnest connected part is less than 3/16 inch. For connections where the thinnest connection part is thicker, refer to the AISC Specification[2]. The area resisting failure due to shear or tension is determined by deducting the bolt hole size along the corresponding failure surface. A standard hole is defined for bolts less than 1/2 inch in diameter as the diameter of the bolt plus 1/32 in. For bolt diameters equal to or greater than 1/2 in., the standard hole size is taken as the bolt diameter plus 1/16 inch. Requirements for bolted slip critical connections are not contained in the *Specification*.

Bolted connections are subject to the limit states of

1. Shear governed by minimum spacing and minimum edge distance,
2. Rupture in net section (shear lag),
3. Bearing on the base material, and
4. Bolt strength.

Spacing and Edge Distance: Bolt spacing and edge distance provisions are found in Section E3.2. The available strengths [factored resistances] are based on shearing of connected materials between the outside bolt and the edge of the material or shearing of the connected material between bolt holes.

Rupture in net section: Rupture of the net cross-section subject to tension must be evaluated using Section E6. This section includes provisions for connections with staggered and non-staggered hole patterns as well as members in which forces are transferred by less than all of the cross-section elements.

Bearing: Section E3.3 in the *Specification* provides strength checks based on the bearing strength of the connected material. Separate checks are provided for the cases where bolt hole deformation is, and is not, considered.

Bolt Strength: The strength of bolts is evaluated using the provision of Section E3.4 in Appendix A (or Appendix B for Canada). Strengths are provided for shear, tension and the interaction of shear and tension.

2.1 Notes on the Tables

Shown in Tables IV-6 and IV-7 are tabulated values for the nominal tension strength [resistance] and the nominal shear strength [resistance] for A307, A325, A449, A354 and A490 bolts. Available strengths [factored resistances] can be found directly from the table for ASD by dividing by Ω, and for LRFD and LSD by multiplying by ϕ.

Provided in Tables IV-8a, through IV-8g are bearing strengths under various shear conditions for steels with tensile strengths of 45 ksi and 65 ksi. The available strengths [factored resistances] for ASD can be found by dividing by Ω, and for LRFD and LSD by multiplying by ϕ.

2.2 Bolted Connection Design Tables

[2] AISC-360, *Specification for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, IL, 2010.

Table IV - 6

Bolts

Tension

Nominal Tension Strength, P_n, kips[1,2]

Ω (ASD) = 2.00
ϕ (LRFD) = 0.75
ϕ (LSD)[3] = 0.80

ASTM Designation	F_y ksi	F_u ksi	Diameter in.	F_{nt} ksi	Nominal Bolt Diameter, in.							
					1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4
					Gross Area, in.[2]							
					.0491	.0767	.1104	.1503	.1963	.2485	.3068	.4418
A307	-	60	< 1/2	40.0	1.96	3.07	4.42	6.01				
			≥ 1/2	45.0					8.83	11.2	13.8	19.9
A325	92	120	≥ 1/2	90.0					17.7	22.4	27.6	39.8
A449	92	120	< 1/2	81.0	3.98	6.21	8.94	12.2				
A354 Gr. BD	130	150	< 1/2	101	4.96	7.75	11.2	15.2				
A490	-	150	≥ 1/2	113					22.2	28.1	34.7	49.9

Notes:
1. Available Strengths [Factored Resistances] are:
   ASD: P_n / Ω
   LRFD, LSD: ϕP_n
2. LSD nominal tensile stresses are rounded slightly
3. For LSD, ϕ = 0.65 for A307 with d < 0.5 in.

Table IV - 7

Bolts

Shear

Nominal Shear Strength, P_n, kips[1,2]

Ω (ASD) = 2.00
ϕ (LRFD) = 0.75
ϕ (LSD)[3] = 0.80

ASTM Designation	Type[4]	Diameter in.	F_{nv} ksi	Nominal Bolt Diameter, in.							
				1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4
				Gross Area, in.[2]							
				.0491	.0767	.1104	.1503	.1963	.2485	.3068	.4418
A307	N or X		27.0	1.33	2.07	2.98	4.06	5.30	6.71	8.28	11.9
A325	N	≥ 1/2	54.0					10.6	13.4	16.6	23.9
	X		68.0					13.3	16.9	20.9	30.0
A449	N	< 1/2	54.0	2.65	4.14	5.96	8.12				
	X		68.0	3.34	5.22	7.51	10.2				
A354 Gr. BD	N	<1/2	68.0	3.34	5.22	7.51	10.2				
	X		84.0	4.12	6.44	9.27	12.6				
A490	N	≥ 1/2	68.0					13.3	16.9	20.9	30.0
	X		84.0					16.5	20.9	25.8	37.1

Notes:
1. Available Strengths [Factored Resistances] are:
   ASD: P_n / Ω
   LRFD, LSD: ϕP_n
2. LSD nominal shear stresses are rounded slightly
3. For LSD, ϕ = 0.55 for A307 with d < 0.5 in.
4. Type N has threads included in a shear plane
   Type X has threads excluded from all shear planes

Table IV - 8a

Bolts
Bearing on Connected Members
Inside Sheet of Double Shear Connections With or Without Washers
Standard Holes
Bolt Hole Deformation Not Considered

Ω (ASD) = 2.50
ϕ (LRFD) = 0.60
ϕ (LSD) = 0.50

Nominal Bearing Strength, P_n, kips[1]

t (in.)	F_u = 45 ksi								F_u = 65 ksi							
	Nominal Bolt Diameter, in.								Nominal Bolt Diameter, in.							
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4
0.024	1.06	1.21	1.31	1.37	1.38	1.45	1.62	1.94	1.53	1.75	1.90	1.98	1.99	2.10	2.33	2.80
0.036	1.62	2.02	2.39	2.62	2.81	2.95	3.05	3.10	2.33	2.92	3.45	3.79	4.06	4.27	4.40	4.47
0.048	2.15	2.69	3.23	3.77	4.25	4.57	4.84	5.25	3.11	3.89	4.67	5.45	6.14	6.60	7.00	7.59
0.060	2.69	3.37	4.04	4.71	5.39	6.06	6.64	7.41	3.89	4.86	5.84	6.81	7.78	8.75	9.59	10.7
0.075	3.37	4.21	5.05	5.89	6.73	7.57	8.42	10.1	4.86	6.08	7.29	8.51	9.73	10.9	12.2	14.6
0.090	4.04	5.05	6.06	7.07	8.08	9.09	10.1	12.1	5.84	7.29	8.75	10.2	11.7	13.1	14.6	17.5
0.105	4.71	5.89	7.07	8.25	9.43	10.6	11.8	14.1	6.81	8.51	10.2	11.9	13.6	15.3	17.0	20.4
0.135	6.06	7.57	9.09	10.6	12.1	13.6	15.1	18.2	8.75	10.9	13.1	15.3	17.5	19.7	21.9	26.3
0.165	7.41	9.26	11.1	13.0	14.8	16.7	18.5	22.2	10.7	13.4	16.0	18.7	21.4	24.1	26.7	32.1

Note:
1. Available Strengths [Factored Resistances] are:
ASD: P_n / Ω
LRFD, LSD: ϕP_n

Table IV - 8b

Bolts
Bearing on Connected Members
Outside Sheets of Connections With Washers on Both Sides
Standard Holes
Bolt Hole Deformation Not Considered

Ω (ASD) = 2.50
ϕ (LRFD) = 0.60
ϕ (LSD) = 0.50

Nominal Bearing Strength, P_n, kips[1]

t (in.)	F_u = 45 ksi								F_u = 65 ksi							
	Nominal Bolt Diameter, in.								Nominal Bolt Diameter, in.							
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4
0.024	0.799	0.911	0.987	1.03	1.03	1.09	1.22	1.46	1.15	1.32	1.43	1.49	1.50	1.58	1.75	2.11
0.036	1.22	1.52	1.80	1.97	2.12	2.22	2.29	2.33	1.75	2.19	2.60	2.85	3.06	3.21	3.31	3.36
0.048	1.62	2.03	2.43	2.84	3.19	3.44	3.64	3.95	2.34	2.92	3.51	4.09	4.61	4.96	5.26	5.70
0.060	2.03	2.53	3.04	3.54	4.05	4.56	4.99	5.57	2.92	3.66	4.39	5.12	5.85	6.58	7.21	8.04
0.075	2.53	3.16	3.80	4.43	5.06	5.70	6.33	7.59	3.66	4.57	5.48	6.40	7.31	8.23	9.14	11.0
0.090	3.04	3.80	4.56	5.32	6.07	6.83	7.59	9.11	4.39	5.48	6.58	7.68	8.77	9.87	11.0	13.2
0.105	3.54	4.43	5.32	6.20	7.09	7.97	8.86	10.6	5.12	6.40	7.68	8.96	10.2	11.5	12.8	15.4
0.135	4.56	5.70	6.83	7.97	9.11	10.3	11.4	13.7	6.58	8.23	9.87	11.5	13.2	14.8	16.5	19.7
0.165	5.57	6.96	8.35	9.75	11.1	12.5	13.9	16.7	8.04	10.1	12.1	14.1	16.1	18.1	20.1	24.1

Note:
1. Available Strengths [Factored Resistances] are:
ASD: P_n / Ω
LRFD, LSD: ϕP_n

Table IV - 8c

Bolts
Bearing on Connected Members
Outside Sheets of Connections Without Washers on Both Sides
Standard Holes
Bolt Hole Deformation Not Considered

Ω (ASD) = 2.50
ϕ (LRFD) = 0.60
ϕ (LSD) = 0.50

Nominal Bearing Strength, P_n, kips[1]

t (in.)	F_u = 45 ksi								F_u = 65 ksi							
	Nominal Bolt Diameter, in.								Nominal Bolt Diameter, in.							
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4
0.024	0.599	0.683	0.740	0.772	0.776	0.820	0.911	1.09	0.865	0.986	1.07	1.11	1.12	1.18	1.32	1.58
0.036	0.911	1.14	1.35	1.48	1.59	1.67	1.72	1.75	1.32	1.65	1.95	2.14	2.29	2.41	2.48	2.52
0.048	1.22	1.52	1.82	2.13	2.40	2.58	2.73	2.96	1.75	2.19	2.63	3.07	3.46	3.72	3.95	4.28
0.060	1.52	1.90	2.28	2.66	3.04	3.42	3.74	4.18	2.19	2.74	3.29	3.84	4.39	4.94	5.41	6.03
0.075	1.90	2.37	2.85	3.32	3.80	4.27	4.75	5.70	2.74	3.43	4.11	4.80	5.48	6.17	6.86	8.23
0.090	2.28	2.85	3.42	3.99	4.56	5.13	5.70	6.83	3.29	4.11	4.94	5.76	6.58	7.40	8.23	9.87
0.105	2.66	3.32	3.99	4.65	5.32	5.98	6.64	7.97	3.84	4.80	5.76	6.72	7.68	8.64	9.60	11.5
0.135	3.42	4.27	5.13	5.98	6.83	7.69	8.54	10.3	4.94	6.17	7.40	8.64	9.87	11.1	12.3	14.8
0.165	4.18	5.22	6.26	7.31	8.35	9.40	10.4	12.5	6.03	7.54	9.05	10.6	12.1	13.6	15.1	18.1

Note:
1. Available Strengths [Factored Resistances] are:
   ASD: P_n/Ω
   LRFD, LSD: ϕP_n

Table IV – 8d

Bolts
Bearing on Connected Members
Single Shear or Outside Sheets of Connections Without Washers on Both Sides
Oversized or Short Slotted Hole Parallel to Applied Load
Bolt Hole Deformation Not Considered

Ω (ASD) = 2.50
ϕ (LRFD) = 0.60
ϕ (LSD) = 0.50

Nominal Bearing Strength, P_n, kips[1]

t (in.)	F_u = 45 ksi								F_u = 65 ksi							
	Nominal Bolt Diameter, in.								Nominal Bolt Diameter, in.							
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4
0.024	0.443	0.490	0.538	0.595	0.680	0.765	0.850	1.02	0.640	0.708	0.776	0.860	0.983	1.11	1.23	1.47
0.036	0.850	0.926	1.00	1.07	1.14	1.21	1.28	1.53	1.23	1.34	1.44	1.54	1.64	1.75	1.85	2.21
0.048	1.13	1.42	1.58	1.68	1.77	1.87	1.96	2.15	1.64	2.05	2.29	2.42	2.56	2.70	2.83	3.11
0.060	1.42	1.77	2.13	2.41	2.53	2.65	2.77	3.01	2.05	2.56	3.07	3.49	3.66	3.83	4.00	4.34
0.075	1.77	2.21	2.66	3.10	3.54	3.81	3.96	4.25	2.56	3.20	3.84	4.48	5.12	5.50	5.72	6.14
0.090	2.13	2.66	3.19	3.72	4.25	4.78	5.32	5.70	3.07	3.84	4.61	5.37	6.14	6.91	7.68	8.23
0.105	2.48	3.10	3.72	4.34	4.96	5.58	6.20	7.34	3.58	4.48	5.37	6.27	7.17	8.06	8.96	10.6
0.135	3.19	3.99	4.78	5.58	6.38	7.18	7.97	9.57	4.61	5.76	6.91	8.06	9.21	10.4	11.5	13.8
0.165	3.90	4.87	5.85	6.82	7.80	8.77	9.75	11.7	5.63	7.04	8.45	9.85	11.3	12.7	14.1	16.9

Note:
1. Available Strengths [Factored Resistances] are:
   ASD: P_n/Ω
   LRFD, LSD: ϕP_n

Table IV – 8e

Bolts
Bearing on Connected Members
Single Shear or Outside Sheets of Connections Without Washers on Both Sides
Short Slotted Hole Perpendicular to Applied Load
Bolt Hole Deformation Not Considered

Ω (ASD) = 2.50
ϕ (LRFD) = 0.60
ϕ (LSD) = 0.50

Nominal Bearing Strength, P_n, kips[1]

t (in.)	F_u = 45 ksi								F_u = 65 ksi							
	Nominal Bolt Diameter, in.								Nominal Bolt Diameter, in.							
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4
0.024	0.348	0.385	0.422	0.468	0.535	0.601	0.668	0.802	0.503	0.556	0.610	0.676	0.772	0.869	0.965	1.16
0.036	0.668	0.728	0.783	0.839	0.895	0.950	1.01	1.20	0.965	1.05	1.13	1.21	1.29	1.37	1.45	1.74
0.048	0.891	1.11	1.24	1.32	1.39	1.47	1.54	1.69	1.29	1.61	1.80	1.90	2.01	2.12	2.23	2.44
0.060	1.11	1.39	1.67	1.90	1.99	2.08	2.18	2.36	1.61	2.01	2.41	2.74	2.87	3.01	3.14	3.41
0.075	1.39	1.74	2.09	2.44	2.78	2.99	3.11	3.34	2.01	2.51	3.02	3.52	4.02	4.32	4.49	4.83
0.090	1.67	2.09	2.51	2.92	3.34	3.76	4.18	4.48	2.41	3.02	3.62	4.22	4.83	5.43	6.03	6.47
0.105	1.95	2.44	2.92	3.41	3.90	4.39	4.87	5.77	2.82	3.52	4.22	4.93	5.63	6.33	7.04	8.33
0.135	2.51	3.13	3.76	4.39	5.01	5.64	6.26	7.52	3.62	4.52	5.43	6.33	7.24	8.14	9.05	10.9
0.165	3.06	3.83	4.59	5.36	6.13	6.89	7.66	9.19	4.42	5.53	6.64	7.74	8.85	9.95	11.1	13.3

Note:
1. Available Strengths [Factored Resistances] are:
   ASD: P_n/Ω
   LRFD, LSD: ϕP_n

Table IV – 8f

Bolts
Bearing on Connected Members
Inside Sheets of Double Shear Connections
With or Without Washers on Both Sides
Oversized or Short Slotted Hole Parallel to Applied Load
Bolt Hole Deformation Not Considered

Ω (ASD) = 2.50
ϕ (LRFD) = 0.60
ϕ (LSD) = 0.50

Nominal Bearing Strength, P_n, kips[1]

t (in.)	F_u = 45 ksi								F_u = 65 ksi							
	Nominal Bolt Diameter, in.								Nominal Bolt Diameter, in.							
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4
0.024	0.696	0.770	0.845	0.936	1.07	1.20	1.34	1.60	1.01	1.11	1.22	1.35	1.54	1.74	1.93	2.32
0.036	1.34	1.46	1.57	1.68	1.79	1.90	2.01	2.41	1.93	2.10	2.26	2.42	2.58	2.75	2.91	3.47
0.048	1.78	2.23	2.49	2.64	2.78	2.93	3.08	3.38	2.57	3.22	3.59	3.81	4.02	4.24	4.45	4.88
0.060	2.23	2.78	3.34	3.79	3.98	4.17	4.35	4.72	3.22	4.02	4.83	5.48	5.75	6.02	6.28	6.82
0.075	2.78	3.48	4.18	4.87	5.57	5.99	6.22	6.68	4.02	5.03	6.03	7.04	8.04	8.65	8.98	9.65
0.090	3.34	4.18	5.01	5.85	6.68	7.52	8.35	8.95	4.83	6.03	7.24	8.45	9.65	10.9	12.1	12.9
0.105	3.90	4.87	5.85	6.82	7.80	8.77	9.75	11.5	5.63	7.04	8.45	9.85	11.3	12.7	14.1	16.7
0.135	5.01	6.26	7.52	8.77	10.0	11.3	12.5	15.0	7.24	9.05	10.9	12.7	14.5	16.3	18.1	21.7
0.165	6.13	7.66	9.19	10.7	12.3	13.8	15.3	18.4	8.85	11.1	13.3	15.5	17.7	19.9	22.1	26.5

Note:
1. Available Strengths [Factored Resistances] are:
   ASD: P_n/Ω
   LRFD, LSD: ϕP_n

Table IV – 8g

Bolts
Bearing on Connected Members
Inside Sheets of Double Shear Connections With or Without Washers on Both Sides
Oversized or Short Slotted Hole Perpendicular to Applied Load
Bolt Hole Deformation Not Considered

Ω (ASD) = 2.50
ϕ (LRFD) = 0.60
ϕ (LSD) = 0.50

Nominal Bearing Strength, P_n, kips[1]

t (in.)	F_u = 45 ksi								F_u = 65 ksi							
	Nominal Bolt Diameter, in.								Nominal Bolt Diameter, in.							
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4
0.024	0.570	0.630	0.691	0.765	0.875	0.984	1.09	1.31	0.823	0.910	1.00	1.11	1.26	1.42	1.58	1.90
0.036	1.09	1.19	1.28	1.37	1.46	1.55	1.65	1.97	1.58	1.72	1.85	1.98	2.11	2.25	2.38	2.84
0.048	1.46	1.82	2.04	2.16	2.28	2.40	2.52	2.76	2.11	2.63	2.94	3.12	3.29	3.47	3.64	3.99
0.060	1.82	2.28	2.73	3.10	3.26	3.41	3.56	3.86	2.63	3.29	3.95	4.48	4.70	4.92	5.14	5.58
0.075	2.28	2.85	3.42	3.99	4.56	4.90	5.09	5.47	3.29	4.11	4.94	5.76	6.58	7.07	7.35	7.90
0.090	2.73	3.42	4.10	4.78	5.47	6.15	6.83	7.33	3.95	4.94	5.92	6.91	7.90	8.88	9.87	10.6
0.105	3.19	3.99	4.78	5.58	6.38	7.18	7.97	9.44	4.61	5.76	6.91	8.06	9.21	10.4	11.5	13.6
0.135	4.10	5.13	6.15	7.18	8.20	9.23	10.3	12.3	5.92	7.40	8.88	10.4	11.8	13.3	14.8	17.8
0.165	5.01	6.26	7.52	8.77	10.0	11.3	12.5	15.0	7.24	9.05	10.9	12.7	14.5	16.3	18.1	21.7

Note:

1. Available Strengths [Factored Resistances] are:
   ASD: P_n / Ω
   LRFD, LSD: ϕP_n

SECTION 3 - SCREWS

Requirements for screw connections are listed in the *Specification* in Section E4. Application is limited to self-tapping screws with nominal screw diameters greater than or equal to 0.08 in. and less than or equal to 0.25 in. The screws must be thread forming or thread cutting, with or without a self-drilling point.

Screwed connections in shear are subject to the limit states of:
1. Tilting and Bearing,
2. Spacing and end distance, and
3. Shear in screws.

Tilting and Bearing: The strength of screws in shear can be limited by simple bearing on the connected material, or by more complex modes involving the tilting of the screw and subsequent pullout. Section E4.3.1 provides strength checks for these limit states.

Spacing and End Distance: Minimum spacing and end distances are set in Section E4.2.

Shear in Screws: Connection strength is also limited by the shear strength of the screws themselves. Section E4.3.2 requires that the nominal screw strength [resistance] be provided by the screw manufacturer or an independent testing laboratory. The *Specification* provides resistance and safety factors, but permits the use of testing to calculate more favorable factors.

Screwed connections in tension are subject to the limit states of:
1. Pull-out,
2. Pull-over, and
3. Tension in screws.

Pull-out: Screws strengths for the limit state of the screw threads pulling out of the connecting material is calculated using Section E4.4.1. The strength is limited by the thickness and material strength of the material into which the screw is anchored.

Pull-over: Screws strengths for the limit state of the top sheet of material pulling over the screw head and/or washer is calculated using Section E4.4.2. The strength is limited by the thickness and material strength of the material directly under the screw head, as well as the type of screw head.

Tension in Screws: Connection strength is also limited by the tensile strength of the screws themselves. Section E4.4.3 requires that the nominal screw strength [resistance] be provided by the screw manufacturer or an independent testing laboratory. The *Specification* provides resistance and safety factors, but permits the use of testing to calculate more favorable factors.

3.1 Notes on the Tables

Provided in Tables IV-9a through IV-9d are the nominal shear strengths [resistances] of screwed connections with designations from #6 to 1/4 inch, which connect various sheet thickness combinations. These are presented for sheets with tensile strengths of 45 ksi and 65 ksi.

Provided in Tables IV-10a through IV-10d are the nominal pull-out strengths [resistances] of screwed connections with designations from #6 to 1/4 inch, in various thicknesses of material. These are presented for sheets with tensile strengths of 45 ksi and 65 ksi.

Provided in Tables IV-11a through IV-11d are the nominal pull-over strengths [resistances] of connections with hex head and hex washer head screws with designations from #6 to 1/4 inch, in various thicknesses of material. These are presented for sheets with tensile strengths of 45 ksi and 65 ksi. ANSI/ASME standard screw head diameters are used in the calculations and are listed in the tables. Larger or smaller diameters will result in different strengths [resistances]. The hex washer head screw values are also applicable to other screws with washers of the listed diameter having a minimum thickness of 0.050 in.

The nominal strengths [resistances] can be determined by interpolating within the tables. The allowable strength for ASD can be found by dividing the nominal strength [resistance] by Ω. The design strength [factored resistance] for LRFD and LSD can be found by multiplying the nominal strength [resistance] by ϕ.

Tables are provided for both the thicknesses used for purlin/girt sections and design thicknesses used for joists/studs. The former set of tables are more convenient for interpolation. The latter set of tables provide direct solutions for joists/studs and tracks without the need for interpolation.

Note that shear and tensile strengths of the fasteners must be determined by the manufacturer through tests.

3.2 Screwed Connection Design Tables

Table IV - 9a

Screws

Nominal Shear Strength, P_{ns}, kips[1]
Shear of Sheet - F_u = 45 ksi
Purlin/Girt Thicknesses

Ω (ASD) = 3.00
ϕ (LRFD) = 0.50
ϕ (LSD) = 0.40

Screw Designation	Diameter in.	Thickness of member in contact with screw head, in.	Thickness of member not in contact with the screw head, in.						
			0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.138	0.036	0.480	0.604	0.604	0.604	0.604	0.604	0.604
		0.048	0.480	0.738	0.805	0.805	0.805	0.805	0.805
		0.060	0.480	0.738	1.01	1.01	1.01	1.01	1.01
		0.075	0.480	0.738	1.01	1.26	1.26	1.26	1.26
		0.090	0.480	0.738	1.01	1.26	1.51	1.51	1.51
		0.105	0.480	0.738	1.01	1.26	1.51	1.76	1.76
		0.135	0.480	0.738	1.01	1.26	1.51	1.76	2.26
#8	0.164	0.036	0.523	0.717	0.717	0.717	0.717	0.717	0.717
		0.048	0.523	0.805	0.956	0.956	0.956	0.956	0.956
		0.060	0.523	0.805	1.12	1.20	1.20	1.20	1.20
		0.075	0.523	0.805	1.12	1.49	1.49	1.49	1.49
		0.090	0.523	0.805	1.12	1.49	1.79	1.79	1.79
		0.105	0.523	0.805	1.12	1.49	1.79	2.09	2.09
		0.135	0.523	0.805	1.12	1.49	1.79	2.09	2.69
#10	0.190	0.036	0.563	0.831	0.831	0.831	0.831	0.831	0.831
		0.048	0.563	0.866	1.11	1.11	1.11	1.11	1.11
		0.060	0.563	0.866	1.21	1.39	1.39	1.39	1.39
		0.075	0.563	0.866	1.21	1.69	1.73	1.73	1.73
		0.090	0.563	0.866	1.21	1.69	2.08	2.08	2.08
		0.105	0.563	0.866	1.21	1.69	2.08	2.42	2.42
		0.135	0.563	0.866	1.21	1.69	2.08	2.42	3.12
#12	0.216	0.036	0.600	0.928	0.945	0.945	0.945	0.945	0.945
		0.048	0.600	0.924	1.26	1.26	1.26	1.26	1.26
		0.060	0.600	0.924	1.29	1.57	1.57	1.57	1.57
		0.075	0.600	0.924	1.29	1.80	1.97	1.97	1.97
		0.090	0.600	0.924	1.29	1.80	2.36	2.36	2.36
		0.105	0.600	0.924	1.29	1.80	2.36	2.76	2.76
		0.135	0.600	0.924	1.29	1.80	2.36	2.76	3.54
1/4 in.	0.250	0.036	0.645	1.02	1.09	1.09	1.09	1.09	1.09
		0.048	0.645	0.994	1.40	1.46	1.46	1.46	1.46
		0.060	0.645	0.994	1.39	1.82	1.82	1.82	1.82
		0.075	0.645	0.994	1.39	1.94	2.28	2.28	2.28
		0.090	0.645	0.994	1.39	1.94	2.55	2.73	2.73
		0.105	0.645	0.994	1.39	1.94	2.55	3.19	3.19
		0.135	0.645	0.994	1.39	1.94	2.55	3.19	4.10

Note:

1. Available Strengths [Factored Resistances] are:
   ASD: P_{ns} / Ω
   LRFD, LSD: ϕP_{ns}

Table IV - 9b

Screws

Nominal Shear Strength, P_{ns}, kips[1]
Shear of Sheet - F_u = 65 ksi
Purlin/Girt Thicknesses

Ω (ASD) = 3.00
φ (LRFD) = 0.50
φ (LSD) = 0.40

Screw Designation	Diameter in.	Thickness of member in contact with screw head, in.	Thickness of member not in contact with screw head, in.						
			0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.138	0.036	0.693	0.872	0.872	0.872	0.872	0.872	0.872
		0.048	0.693	1.07	1.16	1.16	1.16	1.16	1.16
		0.060	0.693	1.07	1.45	1.45	1.45	1.45	1.45
		0.075	0.693	1.07	1.45	1.82	1.82	1.82	1.82
		0.090	0.693	1.07	1.45	1.82	2.18	2.18	2.18
		0.105	0.693	1.07	1.45	1.82	2.18	2.54	2.54
		0.135	0.693	1.07	1.45	1.82	2.18	2.54	3.27
#8	0.164	0.036	0.755	1.04	1.04	1.04	1.04	1.04	1.04
		0.048	0.755	1.16	1.38	1.38	1.38	1.38	1.38
		0.060	0.755	1.16	1.62	1.73	1.73	1.73	1.73
		0.075	0.755	1.16	1.62	2.16	2.16	2.16	2.16
		0.090	0.755	1.16	1.62	2.16	2.59	2.59	2.59
		0.105	0.755	1.16	1.62	2.16	2.59	3.02	3.02
		0.135	0.755	1.16	1.62	2.16	2.59	3.02	3.89
#10	0.190	0.036	0.813	1.20	1.20	1.20	1.20	1.20	1.20
		0.048	0.813	1.25	1.60	1.60	1.60	1.60	1.60
		0.060	0.813	1.25	1.75	2.00	2.00	2.00	2.00
		0.075	0.813	1.25	1.75	2.44	2.50	2.50	2.50
		0.090	0.813	1.25	1.75	2.44	3.00	3.00	3.00
		0.105	0.813	1.25	1.75	2.44	3.00	3.50	3.50
		0.135	0.813	1.25	1.75	2.44	3.00	3.50	4.50
#12	0.216	0.036	0.867	1.34	1.36	1.36	1.36	1.36	1.36
		0.048	0.867	1.33	1.82	1.82	1.82	1.82	1.82
		0.060	0.867	1.33	1.86	2.27	2.27	2.27	2.27
		0.075	0.867	1.33	1.86	2.61	2.84	2.84	2.84
		0.090	0.867	1.33	1.86	2.61	3.41	3.41	3.41
		0.105	0.867	1.33	1.86	2.61	3.41	3.98	3.98
		0.135	0.867	1.33	1.86	2.61	3.41	3.98	5.12
1/4 in.	0.250	0.036	0.932	1.47	1.58	1.58	1.58	1.58	1.58
		0.048	0.932	1.44	2.02	2.11	2.11	2.11	2.11
		0.060	0.932	1.44	2.01	2.63	2.63	2.63	2.63
		0.075	0.932	1.44	2.01	2.80	3.29	3.29	3.29
		0.090	0.932	1.44	2.01	2.80	3.69	3.95	3.95
		0.105	0.932	1.44	2.01	2.80	3.69	4.61	4.61
		0.135	0.932	1.44	2.01	2.80	3.69	4.61	5.92

Note:
1. Available Strengths [Factored Resistances] are:
   ASD: P_{ns} / Ω
   LRFD, LSD: ϕP_{ns}

Table IV – 9c

Screws

Ω (ASD) = 3.00
ϕ (LRFD) = 0.50
ϕ (LSD) = 0.40

Nominal Shear Strength, P_{ns}, kips[1]
Shear of Sheet - F_u = 45 ksi
Joist/Stud or Track Design Thicknesses

Screw Designation	Diameter in.	Thickness of member in contact with screw head, in.	Thickness of member not in contact with the screw head, in.							
			0.0188	0.0283	0.0312	0.0346	0.0451	0.0566	0.0713	0.1017
#6	0.138	0.0188	0.181	0.315	0.315	0.315	0.315	0.315	0.315	0.315
		0.0283	0.181	0.334	0.393	0.455	0.475	0.475	0.475	0.475
		0.0312	0.181	0.334	0.387	0.457	0.523	0.523	0.523	0.523
		0.0346	0.181	0.334	0.387	0.452	0.580	0.580	0.580	0.580
		0.0451	0.181	0.334	0.387	0.452	0.672	0.756	0.756	0.756
		0.0566	0.181	0.334	0.387	0.452	0.672	0.945	0.949	0.949
		0.0713	0.181	0.334	0.387	0.452	0.672	0.945	1.20	1.20
		0.1017	0.181	0.334	0.387	0.452	0.672	0.945	1.20	1.71
#8	0.164	0.0188	0.197	0.368	0.375	0.375	0.375	0.375	0.375	0.375
		0.0283	0.197	0.364	0.432	0.503	0.564	0.564	0.564	0.564
		0.0312	0.197	0.364	0.422	0.502	0.622	0.622	0.622	0.622
		0.0346	0.197	0.364	0.422	0.493	0.689	0.689	0.689	0.689
		0.0451	0.197	0.364	0.422	0.493	0.733	0.899	0.899	0.899
		0.0566	0.197	0.364	0.422	0.493	0.733	1.03	1.13	1.13
		0.0713	0.197	0.364	0.422	0.493	0.733	1.03	1.42	1.42
		0.1017	0.197	0.364	0.422	0.493	0.733	1.03	1.42	2.03
#10	0.190	0.0188	0.212	0.406	0.434	0.434	0.434	0.434	0.434	0.434
		0.0283	0.212	0.392	0.468	0.548	0.653	0.653	0.653	0.653
		0.0312	0.212	0.392	0.454	0.544	0.720	0.720	0.720	0.720
		0.0346	0.212	0.392	0.454	0.530	0.791	0.799	0.799	0.799
		0.0451	0.212	0.392	0.454	0.530	0.789	1.04	1.04	1.04
		0.0566	0.212	0.392	0.454	0.530	0.789	1.11	1.31	1.31
		0.0713	0.212	0.392	0.454	0.530	0.789	1.11	1.57	1.65
		0.1017	0.212	0.392	0.454	0.530	0.789	1.11	1.57	2.35
#12	0.216	0.0188	0.226	0.444	0.488	0.493	0.493	0.493	0.493	0.493
		0.0283	0.226	0.418	0.502	0.592	0.743	0.743	0.743	0.743
		0.0312	0.226	0.418	0.484	0.584	0.819	0.819	0.819	0.819
		0.0346	0.226	0.418	0.484	0.565	0.855	0.908	0.908	0.908
		0.0451	0.226	0.418	0.484	0.565	0.841	1.18	1.18	1.18
		0.0566	0.226	0.418	0.484	0.565	0.841	1.18	1.49	1.49
		0.0713	0.226	0.418	0.484	0.565	0.841	1.18	1.67	1.87
		0.1017	0.226	0.418	0.484	0.565	0.841	1.18	1.67	2.67
1/4 in.	0.250	0.0188	0.244	0.491	0.543	0.571	0.571	0.571	0.571	0.571
		0.0283	0.244	0.450	0.544	0.646	0.860	0.860	0.860	0.860
		0.0312	0.244	0.450	0.521	0.633	0.918	0.948	0.948	0.948
		0.0346	0.244	0.450	0.521	0.608	0.935	1.05	1.05	1.05
		0.0451	0.244	0.450	0.521	0.608	0.905	1.29	1.37	1.37
		0.0566	0.244	0.450	0.521	0.608	0.905	1.27	1.72	1.72
		0.0713	0.244	0.450	0.521	0.608	0.905	1.27	1.80	2.17
		0.1017	0.244	0.450	0.521	0.608	0.905	1.27	1.80	3.06

Note:
1. Available Strengths [Factored Resistances] are:
   ASD: P_{ns} / Ω
   LRFD, LSD: ϕP_{ns}

Table IV – 9d

Screws

Nominal Shear Strength, P_{ns}, kips[1]
Shear of Sheet - F_u = 65 ksi
Joist/Stud or Track Design Thicknesses

Ω (ASD) = 3.00
ϕ (LRFD) = 0.50
ϕ (LSD) = 0.40

Screw Designation	Diameter in.	Thickness of member in contact with screw head, in.	Thickness of member not in contact with screw head, in.		
			0.0566	0.0713	0.1017
#6	0.138	0.0566	1.37	1.37	1.37
		0.0713	1.37	1.73	1.73
		0.1017	1.37	1.73	2.46
#8	0.164	0.0566	1.49	1.63	1.63
		0.0713	1.49	2.05	2.05
		0.1017	1.49	2.05	2.93
#10	0.190	0.0566	1.60	1.89	1.89
		0.0713	1.60	2.27	2.38
		0.1017	1.60	2.27	3.39
#12	0.216	0.0566	1.71	2.15	2.15
		0.0713	1.71	2.42	2.70
		0.1017	1.71	2.42	3.86
1/4 in.	0.250	0.0566	1.84	2.48	2.48
		0.0713	1.84	2.60	3.13
		0.1017	1.84	2.60	4.43

Note:

1. Available Strengths [Factored Resistances] are:
   ASD: P_{ns} / Ω
   LRFD, LSD: ϕP_{ns}

Table IV - 10a

Screws
Pull-Out - F_u = 45 ksi
Purlin/Girt Thicknesses

Ω (ASD) = 3.00
ϕ (LRFD) = 0.50
ϕ (LSD) = 0.40

Nominal Pull-Out Strength, P_{not}, kips[1]

Screw Designation	Diameter in.	Thickness of member not in contact with the screw head, in.						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.138	0.190	0.253	0.317	0.396	0.475	0.554	0.713
#8	0.164	0.226	0.301	0.376	0.470	0.565	0.659	0.847
#10	0.190	0.262	0.349	0.436	0.545	0.654	0.763	0.981
#12	0.216	0.297	0.397	0.496	0.620	0.744	0.868	1.12
1/4 in.	0.250	0.344	0.459	0.574	0.717	0.861	1.00	1.29

Table IV - 10b

Screws
Pull-Out - F_u = 65 ksi
Purlin/Girt Thicknesses

Ω (ASD) = 3.00
ϕ (LRFD) = 0.50
ϕ (LSD) = 0.40

Nominal Pull-Out Strength, P_{not}, kips[1]

Screw Designation	Diameter in.	Thickness of member not in contact with the screw head, in.						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.138	0.274	0.366	0.457	0.572	0.686	0.801	1.03
#8	0.164	0.326	0.435	0.544	0.680	0.815	0.951	1.22
#10	0.190	0.378	0.504	0.630	0.787	0.945	1.10	1.42
#12	0.216	0.430	0.573	0.716	0.895	1.07	1.25	1.61
1/4 in.	0.250	0.497	0.663	0.829	1.04	1.24	1.45	1.86

Note:

1. Available Strengths [Factored Resistances] are:
   ASD: P_{not} / Ω
   LRFD, LSD: ϕP_{not}

Table IV – 10c

Screws
Pull-Out - F_u = 45 ksi
Joist/Stud or Track Design Thicknesses

Ω (ASD) = 3.00
ϕ (LRFD) = 0.50
ϕ (LSD) = 0.40

Nominal Pull-Out Strength, P_{not}, kips

Screw Designation	Diameter in.	Thickness of member not in contact with the screw head, in.							
		0.0188	0.0283	0.0312	0.0346	0.0451	0.0566	0.0713	0.1017
#6	0.138	0.099	0.149	0.165	0.183	0.238	0.299	0.376	0.537
#8	0.164	0.118	0.178	0.196	0.217	0.283	0.355	0.447	0.638
#10	0.190	0.137	0.206	0.227	0.251	0.328	0.411	0.518	0.739
#12	0.216	0.155	0.234	0.258	0.286	0.373	0.468	0.589	0.840
1/4 in.	0.250	0.180	0.271	0.298	0.331	0.431	0.541	0.682	0.973

Table IV – 10d

Screws
Pull-Out - F_u = 65 ksi
Joist/Stud or Track Design Thicknesses

Ω (ASD) = 3.00
ϕ (LRFD) = 0.50
ϕ (LSD) = 0.40

Nominal Pull-Out Strength, P_{not}, kips

Screw Designation	Diameter in.	Thickness of member not in contact with the screw head, in.		
		0.0566	0.0713	0.1017
#6	0.138	0.432	0.544	0.775
#8	0.164	0.513	0.646	0.922
#10	0.190	0.594	0.748	1.07
#12	0.216	0.675	0.851	1.21
1/4 in.	0.250	0.782	0.985	1.40

Note:

1. Available Strengths [Factored Resistances] are:
   ASD: P_{not} / Ω
   LRFD, LSD: ϕP_{not}

Table IV - 11a

Hex Head Screws
Pull-Over - F_u = 45 ksi
Purlin/Girt Thicknesses

Ω (ASD) = 3.00
ϕ (LRFD) = 0.50
ϕ (LSD) = 0.40

Nominal Pull-Over Strength, P_{nov}, kips[1]

Hex Head Screws Without Washers

Screw Designation	Hex Head Diameter in.	Thickness of member in contact with the screw head, in.						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.272	0.661	0.881	1.10	1.38	1.65	1.93	2.48
#8	0.272	0.661	0.881	1.10	1.38	2.65	1.93	2.48
#10	0.340	0.826	1.10	1.38	1.72	2.07	2.41	3.10
#12	0.340	0.826	1.10	1.38	1.72	2.07	2.41	3.10
1/4 in.	0.409	0.994	1.33	1.66	2.07	2.48	2.90	3.73

Hex Washer Head Screws

Screw Designation	Washer Head Diameter in.	Thickness of member in contact with the washer head, in.						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.315	0.765	1.02	1.28	1.59	1.91	2.23	2.87
#8	0.335	0.814	1.09	1.36	1.70	2.04	2.37	3.05
#10	0.399	0.970	1.29	1.62	2.02	2.42	2.83	3.64
#12	0.415	1.01	1.34	1.68	2.10	2.52	2.94	3.78
1/4 in.	0.500	1.22	1.62	2.03	2.53	3.04	3.54	4.56

Table IV - 11b

Hex Head Screws
Pull-Over - F_u = 65 ksi
Purlin/Girt Thicknesses

Ω (ASD) = 3.00
ϕ (LRFD) = 0.50
ϕ (LSD) = 0.40

Nominal Pull-Over Strength, P_{nov}, kips[1]

Hex Head Screws Without Washers

Screw Designation	Hex Head Diameter in.	Thickness of member in contact with the washer head, in.						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.272	0.955	1.27	1.59	1.99	2.39	2.78	3.58
#8	0.272	0.955	1.27	1.59	1.99	2.39	2.78	3.58
#10	0.340	1.19	1.59	1.99	2.49	2.98	3.48	4.48
#12	0.340	1.19	1.59	1.99	2.49	2.98	3.48	4.48
1/4 in.	0.409	1.44	1.91	2.39	2.99	3.59	4.19	5.38

Hex Washer Head Screws

Screw Designation	Washer Head Diameter in.	Thickness of member in contact with the washer, in.						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.315	1.11	1.47	1.84	2.30	2.76	3.22	4.15
#8	0.335	1.18	1.57	1.96	2.45	2.94	3.43	4.41
#10	0.399	1.40	1.87	2.33	2.92	3.50	4.08	5.25
#12	0.415	1.46	1.94	2.43	3.03	3.64	4.25	5.46
1/4 in.	0.500	1.76	2.34	2.93	3.66	4.39	5.12	6.58

Note:

1. Available Strengths [Factored Resistances] are:
   ASD: P_{not} / Ω
   LRFD, LSD: ϕP_{nov}

Table IV – 11c

Hex Head Screws
Pull-Over - F_u = 45 ksi
Joist/Stud or Track Design Thicknesses

Ω (ASD) = 3.00
ϕ (LRFD) = 0.50
ϕ (LSD) = 0.40

Nominal Pull-Over Strength, P_{nov}, kips[1]

Screw Designation	Hex Head Diameter in.	Thickness of member in contact with the screw head, in.							
Hex Head Screws Without Washers									
		0.0188	0.0283	0.0312	0.0346	0.0451	0.0566	0.0713	0.1017
#6	0.272	0.345	0.520	0.573	0.635	0.828	1.04	1.31	1.87
#8	0.272	0.345	0.520	0.573	0.635	0.828	1.04	1.31	1.87
#10	0.340	0.431	0.649	0.716	0.794	1.04	1.30	1.64	2.33
#12	0.340	0.431	0.649	0.716	0.794	1.04	1.30	1.64	2.33
1/4 in.	0.409	0.519	0.781	0.861	0.955	1.25	1.56	1.97	2.81
Hex Washer Head Screws									
Screw Designation	Washer Head Diameter in.	Thickness of member in contact with the washer head, in.							
		0.0188	0.0283	0.0312	0.0346	0.0451	0.0566	0.0713	0.1017
#6	0.315	0.400	0.602	0.663	0.736	0.959	1.20	1.52	2.16
#8	0.335	0.425	0.640	0.706	0.782	1.02	1.28	1.61	2.30
#10	0.399	0.506	0.762	0.840	0.932	1.21	1.52	1.92	2.74
#12	0.415	0.527	0.793	0.874	0.969	1.26	1.59	2.00	2.85
1/4 in.	0.500	0.635	0.955	1.05	1.17	1.52	1.91	2.41	3.43

Table IV – 11d

Hex Head Screws
Pull-Over - F_u = 65 ksi
Joist/Stud or Track Design Thicknesses

Ω (ASD) = 3.00
ϕ (LRFD) = 0.50
ϕ (LSD) = 0.40

Nominal Pull-Over Strength, P_{nov}, kips[1]

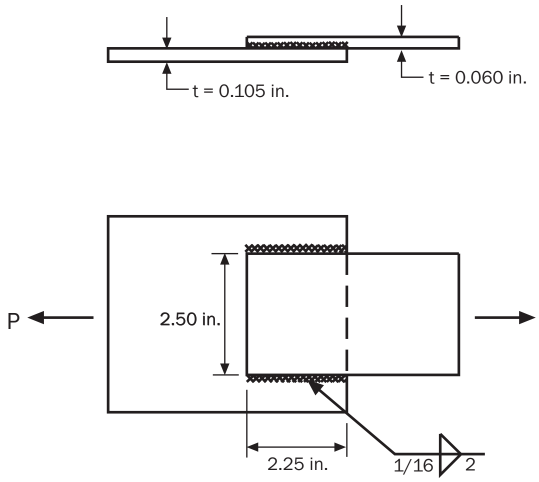
Screw Designation	Hex Head Diameter in.	Thickness of member in contact with the washer head, in.		
Hex Head Screws Without Washers				
		0.0566	0.0713	0.1017
#6	0.272	1.50	1.89	2.70
#8	0.272	1.50	1.89	2.70
#10	0.340	1.88	2.36	3.37
#12	0.340	1.88	2.36	3.37
1/4 in.	0.409	2.26	2.84	4.06
Hex Washer Head Screws				
Screw Designation	Washer Head Diameter in.	Thickness of member in contact with the washer, in.		
		0.0566	0.0713	0.1017
#6	0.315	1.74	2.19	3.12
#8	0.335	1.85	2.33	3.32
#10	0.399	2.20	2.77	3.96
#12	0.415	2.29	2.88	4.12
1/4 in.	0.500	2.76	3.48	4.96

Note: Available Strengths [Factored Resistances] are:
ASD: P_{nov} / Ω
LRFD, LSD: ϕP_{nov}

SECTION 4 - EXAMPLE PROBLEMS

4.1 Weld Examples

Example IV-1: Flat Section With Fillet Welded Lap Connection



Given:

1. Steel: F_y = 50 ksi, F_u = 65 ksi
2. Loads: P_{dead} = 1.0 kip, P_{live} = 3.0 kips
3. Detail of connection shown in sketch

Required:

Determine if the longitudinal fillet welded connection is adequate to transmit the required strength P (ASD), and P_u (LRFD) using ASCE/SEI 7-10 load combinations.

Solution:

1. Required Strength

ASD

$P = P_{dead} + P_{live}$

$= 1.0 + 3.0 = 4.0$ kips

LRFD

$P_u = 1.2P_{dead} + 1.6P_{live}$

$= 1.2(1.0)+1.6(3.0)=6.0$ kips

2. Shear Strength of Sheet at Weld (Section E2.5)

L/t = 2.25/0.060 = 37.5 > 25

For L/t ≥ 25,

Upper sheet controls by inspection

$P_{n1} = 0.75t_1LF_{u1}$ (*Eq*. E2.5-3)

$P_n = (0.75)(0.060)(2.25)(65)=6.58$ kips/weld

ASD

$\Omega = 3.05$

$$\frac{P_n}{\Omega} = \frac{6.58}{3.05} = 2.16 \text{ kips/weld}$$

$(2.16\,\text{kips/weld})(2\text{ welds}) = 4.32$ kips > 4.0 kips OK

LRFD

$\phi = 0.50$

$\phi P_n = (0.50)(6.58) = 3.29 \text{ kips/weld}$

$(3.29 \text{ kips/weld})(2 \text{ welds}) = 6.58 \text{ kips} > 6.0 \text{ kips}$ OK

3. Shear Strength of Weld

No check of weld strength Equation E2.5-7 required because t = 0.060 in. < 0.10 in.

4. Tensile Strength of the 0.060 in. Sheet (Section C2)

Yielding of the gross section

$T_n = A_g F_y$ (*Eq.* C2.1-1)

$= (2.50)(0.060)(50) = 7.50 \text{ kips}$

ASD

$\Omega_t = 1.67$

$\frac{T_n}{\Omega_t} = \frac{7.50}{1.67} = 4.49 \text{ kips} > 4.0 \text{ kips}$ OK

LRFD

$\phi_t = 0.90$

$\phi_t T_n = (0.90)(7.50) = 6.75 \text{ kips} > 6.0 \text{ kips}$ OK

Rupture of the net section away from the connection

$T_n = A_n F_u = (2.50)(0.060)(65) = 9.75 \text{ kips}$ (*Eq.* C2.2-2)

ASD

$\Omega_t = 2.00$

$\frac{T_n}{\Omega_t} = \frac{9.75}{2.00} = 4.88 \text{ kips} > 4.0 \text{ kips}$ OK

LRFD

$\phi_t = 0.75$

$\phi_t P_n = (0.75)(9.75) = 7.31 \text{ kips} > 6.0 \text{ kips}$ OK

5. Using Connection Tables

Using Table IV-1, the available strength based on sheet shear could have been determined as follows:

1.) Sheet shear, Table IV-1 for two 2.25 in. fillet welds with sheet thickness = 0.060 in.

$L/t = 2.25/0.060 = 37.5 \geq 25$

$P'_n = 2.93 \text{ kips/inch}$ (from Table IV-1)

$P_n = (2)(2.93)(2.25) = 13.19 \text{ kips}$

ASD

$\Omega = 3.05$

$\frac{P_n}{\Omega} = \frac{13.19}{3.05} = 4.32 \text{ kips} > 4.0 \text{ kips}$ OK

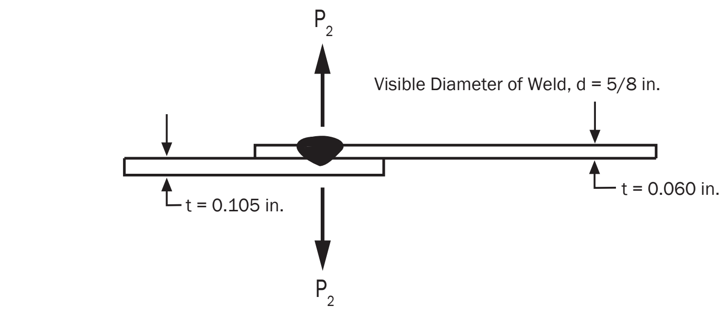
LRFD

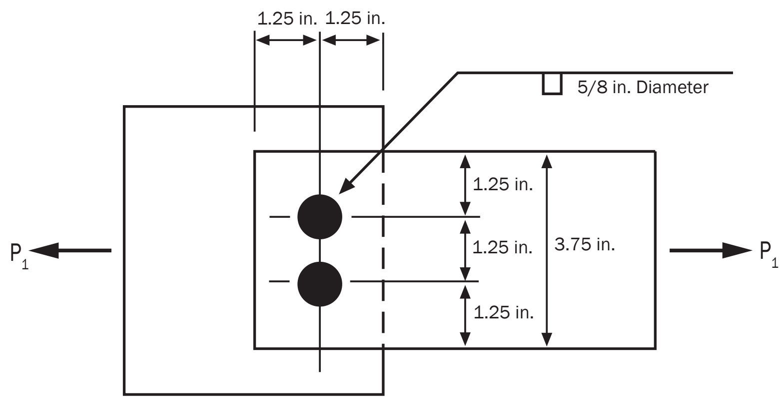
$\phi = 0.50$

$\phi P_n = (0.50)(13.19) = 6.60 \text{ kips} > 6.0 \text{ kips}$ OK

2.) Check other sheet limit states as above

Example IV-2: Flat Section With Arc Spot Welded Connection





Given:

1. Steel: $F_{sy} = F_y = 33$ ksi, $F_u = 45$ ksi
2. E60 Weld Electrode, $F_{xx} = 60$ ksi
3. Detail of connection shown in sketch

Required:

1. Determine the available shear strength, P_1, and available tensile strength, P_2, using ASD and LRFD.

2. Given $P_{1(dead)} = 0.4$ kip, $P_{1(Lr)} = 1.2$ kips, and $P_{2(wind)} = 2.0$ kips, check the connection adequacy using:

   a. ASD – using ASCE/SEI 7-10 load combination D + 0.75(0.6W) + 0.75L_r

   b. LRFD – using ASCE/SEI 7-10 load combination 1.2D + 1.0W + 0.5L_r

Solution:

1. Weld Dimensions, Minimum Edge and End Distance Check

a) Weld size and requirements (Section E2.2)

$d = 0.625$ in.

$d_a = d - t = 0.625 - 0.060 = 0.565$ in.

$d_e = 0.7d - 1.5t \leq 0.55d$ (*Eq.* E2.2.2.1-5)

$= (0.70)(0.625) - (1.5)(0.060) = 0.348$ in. $< \frac{3}{8}$ in. N.G.

$0.55d = 0.55(0.625) = 0.344$ in. $< \frac{3}{8}$ in. N.G.

Per Section E2.2, minimum allowable effective diameter, d_e, is $\frac{3}{8}$ in. Weld procedures must be established and welds measured to assure that a $\frac{3}{8}$ inch effective diameter can be consistently achieved.

b) Check plate thickness and weld washer requirement:

Thinnest connected part, t = 0.060 in. < 0.15 in. OK

No weld washers are required because t = 0.060 in. > 0.028 in.

c) Minimum edge and end distance (Section E2.2.1)

Edge distance shall not be less than 1.5d.

$$1.5\text{d} = (1.5)(0.625) = 0.938 \text{ in.} < 1.25 \text{ in. OK}$$

Clear distance between welds shall not be less than 1.0d.

$$1.0\text{d} = (1.0)(0.625) = 0.625 \text{ in.}$$

$$\text{Clear distance} = 1.250 - (2)(0.625/2) = 0.625 \text{ in.} = 0.625 \text{ in. OK}$$

Clear distance between welds and end of member shall not be less than 1.0d.

$$1.0\text{d} = (1.0)(0.625) = 0.625 \text{ in.}$$

$$\text{Clear distance } = 1.250 - 0.625/2 = 0.938 \text{ in. } > 0.625 \text{ in. OK}$$

2. Shear Strength (Section E2.2.2.1)

a) Strength based on weld strength (Section E2.2.2.1(a))

$$P_n = \frac{\pi d_e^2}{4} 0.75F_{xx} \qquad (Eq.\ \text{E2.2.2.1-1})$$

Using E60 electrode, F_{xx} = 60 ksi

$$P_n = \frac{\pi(0.375)^2}{4}(0.75)(60) = 4.97 \text{ kips / weld}$$

ASD

Ω = 2.55

$$\frac{P_n}{\Omega} = \frac{2(4.97)}{2.55} = 3.90 \text{ kips}$$

LRFD

ϕ = 0.60

$$\phi P_n = (0.60)(2)(4.97) = 5.96 \text{ kips}$$

b) Strength based on sheet strength (Section E2.2.2.1(b))

d_a/t = 0.565/0.060 = 9.42

$$0.815\sqrt{E/F_u} = 0.815\sqrt{29500/45} = 20.9$$

Since $d_a/t < 0.815\sqrt{E/F_u}$

$$P_n = 2.20td_aF_u \qquad (Eq.\ \text{E2.2.2.1-2})$$

$$= (2.20)(0.060)(0.565)(45) = 3.36 \text{ kips/weld}$$

ASD

Ω = 2.20

$$\frac{P_n}{\Omega} = \frac{(2)3.36}{2.20} = 3.05 \text{ kips} \qquad \leftarrow \text{CONTROLS}$$

LRFD

$\phi = 0.70$

$\phi P_n = (0.70)(2)(3.36) = 4.70 \text{ kips} \quad \leftarrow$ CONTROLS

3. Strength Based on Rupture (Section E6)

a) Shear rupture (Section E6.1)

$V_n = 0.6F_uA_{nv}$ (*Eq*. E6.1-1)

$F_u = 45 \text{ ksi}$

$A_{nv} = 2nte_{net}$ (*Eq*. E6.1-2)

$n = 2$

$t = 0.060$ in.

$e_{net} = 1.25 - \dfrac{0.625}{2} = 0.938 \text{ in.}$

$A_{nv} = 2(2)(0.060)(0.938) = 0.225 \text{ in.}^2$ (*Eq*. E6.1-2)

$V_n = 0.6(45)(0.225) = 6.08 \text{ kips}$ (*Eq*. E6.1-1)

ASD

$\Omega = 2.50$

$\dfrac{V_n}{\Omega} = \dfrac{6.08}{2.50} = 2.43 \text{ kips}$

LRFD

$\phi = 0.60$

$\phi V_n = 0.60(6.08) = 3.65 \text{ kips}$

b) Tension rupture (Section E6.2)

Since there are no design provisions for an arc spot weld, design provisions for bolted connections are employed as a rational engineering analysis approach per Section A1.2.

$T_n = F_uA_e$ (*Eq*. E6.2-1)

$F_u = 45 \text{ ksi}$

$A_e = U_{sl}A_{nt}$ (*Eq*. E6.2-2)

$n_b = 2$

$d_a = 0.565$ in.

$U_{sl} = 3.33d/s \le 1.0$ (*Eq*. E6.2-4)

$= 3.33\dfrac{0.565}{3.75/2} \le 1.0$

$= 1.00 \quad \text{Use } U_{sl} = 1.00$

$A_{nt} = A_g - n_bd_ht$ (*Eq*. E6.2-3)

$= 3.75(0.060) - 2(0.625)(0.060) = 0.150 \text{ in.}^2$

$A_e = (1.00)(0.150) = 0.150 \text{ in.}^2$ (*Eq*. E6.2-2)

$T_n = 45(0.150) = 6.75 \text{ kips}$ (*Eq*. E6.2-1)

ASD

$\Omega = 2.50$

$$\frac{T_n}{\Omega} = \frac{6.75}{2.50} = 2.70\,\text{kips}$$

LRFD

$\phi = 0.60$

$\phi T_n = 0.60(6.75) = 4.05\,\text{kips}$

c) Block shear rupture (Section E6.3)

By inspection, path with 2 shear legs and 1 tension leg controls

$$A_{gv} = 2(1.25)(0.060) = 0.150\,\text{in.}^2$$

$$A_{nv} = 2\left(1.25 - \frac{0.625}{2}\right)0.060 = 0.113\,\text{in.}^2$$

$$A_{nt} = (1.25 - 0.625)(0.060) = 0.0375\,\text{in.}^2$$

$U_{bs} = 1.0$

$$R_n = 0.6F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (Eq.\ E6.3\text{-}1)$$

$$= 0.6(33)(0.150) + 1.0(45)0.0375 = 4.66\,\text{kips}$$

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt} \qquad (Eq.\ E6.3\text{-}2)$$

$$= 0.6(45)(0.113) + 1.0(45)0.0375 = 4.74\,\text{kips}$$

ASD

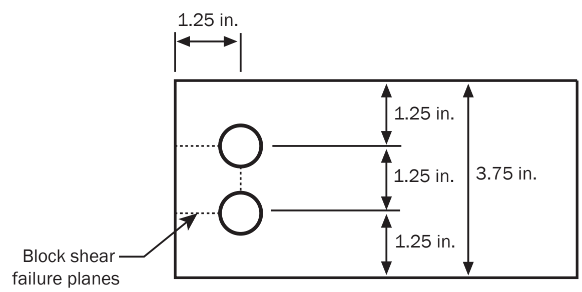
$\Omega = 2.50$

$$\frac{V_n}{\Omega} = \frac{4.66}{2.50} = 1.86\,\text{kips} \quad \text{CONTROLS}$$

LRFD

$\phi = 0.60$

$\phi V_n = 0.60(4.66) = 2.80\,\text{kips}$ CONTROLS



4. Tensile Strength of the Sheet (Section C2)

a) Yielding of the gross section (Section C2.1)

$$T_n = A_gF_y = (3.75)(0.060)(33) = 7.43\,\text{kips} \qquad (Eq.\ C2.1\text{-}1)$$

ASD

$\Omega_t = 1.67$

$$\frac{T_n}{\Omega_t} = \frac{7.43}{1.67} = 4.45\,\text{kips}$$

LRFD

$\phi_t = 0.90$

$\phi_t T_n = (0.90)(7.43) = 6.69\,\text{kips}$

b) Rupture of the net section away from connection (Section C2.2)

$$T_n = A_nF_u \qquad (Eq.\ C2.2\text{-}2)$$

$$= (3.75)(0.060)(45) = 10.1\,\text{kips}$$

ASD

Ω_t = 2.00

$$\frac{T_n}{\Omega_t} = \frac{10.1}{2.00} = 5.05\,\text{kips}$$

LRFD

ϕ_t = 0.75

$\phi_t T_n = (0.75)(10.1) = 7.58$ kips

5. Shear Strength, Corresponding to Load P_1, Using Connection Tables

Using Table IV-3 the available strength based on weld shear and sheet shear could have been determined as follows:

a) Check weld shear as in Part 2 above

P_n = 9.94 kips

$\dfrac{P_n}{\Omega}$ = 9.94/2.55 = 3.90 kips (ASD)

$\phi P_n = (0.60)(9.94) = 5.96$ kips (LRFD)

b) Shear of sheet, Table IV-3

$P_n = (3.36)(2) = 6.72$ kips

$\dfrac{P_n}{\Omega} = \dfrac{6.72}{2.20} = 3.05$ kips (ASD) ← CONTROLS

$\phi P_n = (0.70)(6.72) = 4.70$ kips (LRFD) ← CONTROLS

c) Check edge distance, rupture and sheet as above. Block shear controls.

6. Tensile Strength, Corresponding to Load P_2 (Section E2.2.3)

a) Check limits

1. $td_aF_u \le 3$ kips

   $(0.060)(0.565)(45) = 1.53 < 3$ kips OK

2. $F_{xx} \ge 60$ ksi OK

3. $F_u \le 82$ ksi OK

4. $F_{xx} > F_u$

   60 ksi > 45 ksi OK

b) Calculate P_n as the smaller value from *Eq*. E2.2.3-1 and *Eq*. E2.2.3-2

$$P_n = \frac{\pi d_e^{\,2}}{4} F_{xx} \qquad (\textit{Eq. } \text{E2.2.3-1})$$

Using E60 electrode, F_{xx} = 60 ksi

$$P_n = \frac{\pi(0.375)^2}{4}(60) = 6.63\,\text{kips/ weld}$$

$$P_n = 0.8\left(F_u/F_y\right)^2 td_aF_u \qquad (\textit{Eq. } \text{E2.2.3-2})$$

$$= 0.8(45/33)^2(0.060)(0.565)(45) = 2.27\,\text{kips/weld}$$

ASD

$\Omega = 3.00$

$\frac{P_n}{\Omega} = \frac{2(2.27)}{3.00} = 1.51\,\text{kips} \leftarrow \text{CONTROLS}$

LRFD

$\phi = 0.50$

$\phi P_n = 2(0.50)(2.27) = 2.27\text{ kips} \leftarrow \text{CONTROLS}$

7. Check Connection Strength Adequacy

a) Required strength

ASD: per ASCE/SEI 7-10 load combination D + 0.75(0.6W) + $0.75L_r$:

$P_1 = 0.4 + 0.75(1.2) = 1.30$ kips

$P_2 = 0.75(0.6)(2.0) = 0.900$ kip

For each arc-spot weld:

$T = P_2/2 = 0.900/2 = 0.450$ kip/weld

$Q = P_1/2 = 1.30/2 = 0.650$ kip/weld

LRFD: per ASCE/SEI 7-10 load combination 1.2D+1.0W+0.5Lr:

$P_{1u} = 1.2(0.4) + 0.5(1.2) = 1.08$ kips

$P_{2u} = 1.0(2.0) = 2.00$ kips

For each arc-spot weld:

$T_u = P_{2u}/2 = 2.00/2 = 1.00$ kip/weld

$Q_u = P_{1u}/2 = 1.08/2 = 0.540$ kip/weld

b) Strength check

Based on the strengths calculated per Parts 1 to 6, the connection available strengths are:

ASD: Available connection shear strength = 1.86 kips > 1.30 kips OK

Available connection tensile strength = 1.51 kips > 0.900 kip OK

Combined shear and tension on an arc spot weld per Section E2.2.4.1:

Part 2b) above, $P_{ns} = 3.36$ kips/weld; $\Omega_s = 2.20$;

Per part 6b) above, $P_{nt} = 2.27$ kips/weld, $\Omega_t = 3.00$;

$$\left(\frac{\Omega_t T}{P_{nt}}\right)^{1.5} = \left(\frac{3.00(0.450)}{2.27}\right)^{1.5} = 0.459 > 0.15$$

$$\left(\frac{\Omega_s Q}{P_{ns}}\right)^{1.5} + \left(\frac{\Omega_t T}{P_{nt}}\right)^{1.5} \le 1 \qquad (Eq.\ E2.2.4.1\text{-}1)$$

$$\left(\frac{2.20(0.650)}{3.36}\right)^{1.5} + \left(\frac{3.00(0.450)}{2.27}\right)^{1.5} = 0.736 < 1.0 \quad \text{OK}$$

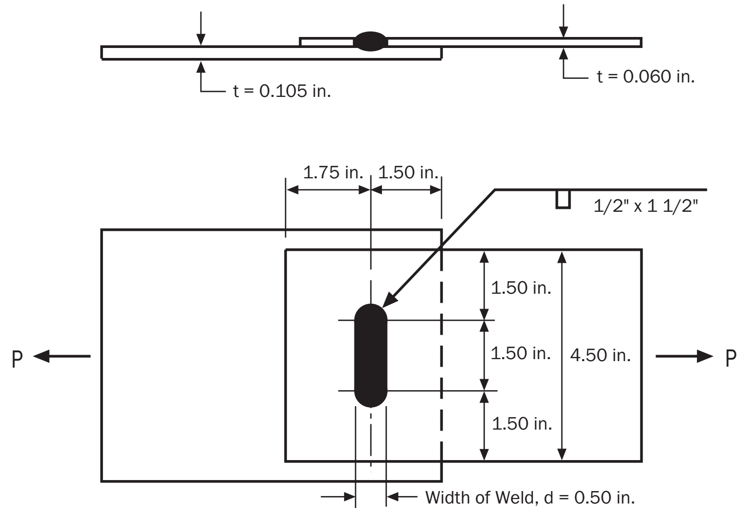
LRFD: Available connection shear strength = 2.80 kips > 1.08 kips OK
Available connection tensile strength = 2.27 kips > 2.00 kips OK
Combined shear and tension on an arc spot weld per Section E2.2.4.2:
Per part 2b) above, P_{ns} = 3.36 kips/weld; ϕ_s = 0.70;
Per part 6b) above, P_{nt} = 2.27 kips/weld, ϕ_t = 0.50;

$$\left(\frac{T_u}{\phi_t P_{nt}}\right)^{1.5} = \left(\frac{1.00}{(0.50)(2.27)}\right)^{1.5} = 0.827 > 0.15$$

$$\left(\frac{Q_u}{\phi_s P_{ns}}\right)^{1.5} + \left(\frac{T_u}{\phi_t P_{nt}}\right)^{1.5} \le 1 \qquad (\textit{Eq}.\ \text{E2.2.4.2-1})$$

$$\left(\frac{0.540}{(0.70)(3.36)}\right)^{1.5} + \left(\frac{1.00}{(0.50)(2.27)}\right)^{1.5} = 0.937 < 1.0 \quad \text{OK}$$

Example IV-3: Flat Section With Arc Seam Welded Connection



Given:

1. Steel: F_y = 50 ksi, F_u = 65 ksi
2. E60 Weld Electrode, F_{xx} = 60 ksi
3. Total Required Strength: P = 2.80 kips (ASD), P_u = 4.20 kips (LRFD)
4. Detail of connection shown in sketch
5. Connection is in flat position as required by Section E2.3

Required:

Check the ability of the connection to transmit the required strength.

Solution:

1. Minimum Edge Distance and Spacing (Section E2.3.1)

Edge distance shall not be less than 1.5 d.

$1.5d = (1.5)(0.50) = 0.75$ in. < 1.50 in. OK

Clear distance between weld and end of member shall not be less than 1.0d.

$1.0d = (1.0)(0.50) = 0.50$ in.

Clear distance = 1.50 - 0.25 = 1.25 in. > 0.50 in. OK

2. Shear Strength Based on Weld Strength (Section E2.3)

Calculate P_n as the smaller value from *Eq.* E2.3.2.1-1 and *Eq.* E2.3.2.1-2.

a) Weld metal strength

$$P_n = \left[\frac{\pi d_e^{\,2}}{4} + L d_e\right] 0.75 F_{xx} \qquad (Eq.\ E2.3.2.1\text{-}1)$$

L = 1.5 in., or maximum 3d, 3(0.5) = 1.5 in. OK

$d_a = d - t$ (*Eq.* E2.3.2.1-4)

$= 0.50 - 0.060 = 0.440$ in.

$d_e = 0.7d - 1.5t$ (*Eq.* E2.3.2.1-3)

$= (0.7)(0.50) - (1.5)(0.060) = 0.260\text{ in.}$

$$P_n = \left[\frac{\pi(0.26)^2}{4} + (1.50)(0.26)\right](0.75)(60) \quad (Eq.\ E2.3.2.1\text{-}1)$$

$= 19.9$ kips

b) Base metal strength

$$P_n = 2.5tF_u(0.25L + 0.96d_a) \quad (Eq.\ E2.3.2.1\text{-}2)$$

$= (2.5)(0.060)(65)[(0.25)(1.5) + (0.96)(0.440)] = 7.77$ kips $\leftarrow$ CONTROLS

ASD

$\Omega = 2.55$

$\frac{P_n}{\Omega} = \frac{7.77}{2.55} = 3.05\text{ kips} > 2.80\text{ kips}$ OK

LRFD

$\phi = 0.60$

$\phi P_n = (0.60)(7.77) = 4.66$ kips $> P_u = 4.20$ kips OK

3. Tensile Strength of the Plate (Section C2)

a) Yielding of the gross section

$$T_n = A_gF_y \quad (Eq.\ C2.1\text{-}1)$$

$= (4.5)(0.060)(50) = 13.5\text{ kips}$

$\Omega_t = 1.67$

$\frac{T_n}{\Omega_t} = \frac{13.5}{1.67} = 8.08\text{ kips} > 2.80\text{ kips}$ OK

LRFD

$\phi_t = 0.90$

$\phi_t T_n = (0.90)(13.5) = 12.2$ kips > 4.20 kips OK

b) Rupture of the net section away from the connection

$$T_n = A_nF_u \quad (Eq.\ C2.2\text{-}2)$$

$= (4.5)(0.060)(65) = 17.6\text{ kips}$

ASD

$\Omega_t = 2.00$

$\frac{T_n}{\Omega_t} = \frac{17.6}{2.00} = 8.80\text{ kips} > 2.80\text{ kips}$ OK

LRFD

$\phi_t = 0.75$

$\phi_t P_n = (0.75)(17.6) = 13.2$ kips > 4.20 kips OK

4. Strength Based on Rupture (Section E6)

a) Shear rupture (Section E6.1)

$$V_n = 0.6F_uA_{nv} \quad (Eq.\ E6.1\text{-}1)$$

$F_u = 65\text{ ksi}$

$$A_{nv} = 2nte_{net} \quad (Eq.\ E6.1\text{-}2)$$

n = 1

t = 0.060 in.

$$e_{net} = 1.75 - \frac{0.500}{2} = 1.50\,\text{in.}$$

$$A_{nv} = 2(1)(0.060)(1.50) = 0.180\,\text{in.}^2 \qquad (Eq.\ E6.1\text{-}2)$$

$$V_n = 0.6(65)(0.180) = 7.02\,\text{kips} \qquad (Eq.\ E6.1\text{-}1)$$

ASD

$\Omega = 2.50$

$$\frac{V_n}{\Omega} = \frac{7.02}{2.50} = 2.81\,\text{kips} > 2.80\text{ kips OK}$$

LRFD

$\phi = 0.60$

$$\phi V_n = 0.60(7.02) = 4.21\,\text{kips} > 4.20\text{ kips OK}$$

b) Tension rupture (Section E6.2)

$$T_n = F_u A_e \qquad (Eq.\ E6.2\text{-}1)$$

$F_u = 65\,\text{ksi}$

$$A_e = U_{sl} A_{nt} \qquad (Eq.\ E6.2\text{-}2)$$

$n_b = 1$

$d_h = 1.50$ in.

$U_{sl} = 1.0$

$$A_{nt} = A_g - n_b d_h t \qquad (Eq.\ E6.2\text{-}3)$$

$$= 4.50(0.060) - 1(1.50)(0.060) = 0.180\,\text{in.}^2$$

$$A_e = 1.0(0.180) = 0.180\,\text{in.}^2 \qquad (Eq.\ E6.2\text{-}2)$$

$$T_n = 65(0.180) = 11.7\,\text{kips} \qquad (Eq.\ E6.2\text{-}1)$$

ASD

$\Omega = 2.50$

$$\frac{T_n}{\Omega} = \frac{11.7}{2.50} = 4.68\,\text{kips} > 2.80\text{ kips OK}$$

LRFD

$\phi = 0.60$

$$\phi T_n = 0.60(11.7) = 7.02\,\text{kips} > 4.20\text{ kips OK}$$

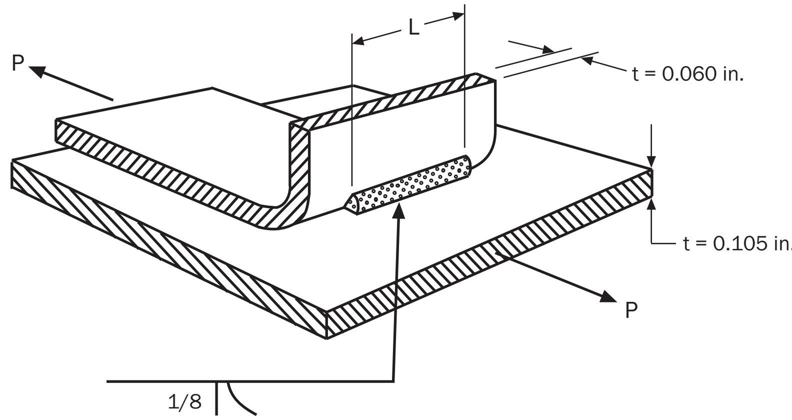
c) Block Shear (Section E6.3)

By inspection, does not control

5. Final Design

Use arc seam welded connection per sketch with E60 minimum electrode.

Example IV-4: Flat Section With Flare Bevel Groove Weld



Given:

1. Steel: $F_y = 50$ ksi, $F_u = 65$ ksi
2. Nominal Loads, $P_{dead} = 1.00$ kips $P_{live} = 3.00$ kips
3. Detail of connection shown in sketch
4. Transverse loading

Required:

Determine the required weld length, L, using ASD and LRFD.

Solution:

1. Required Strength

ASD

$$P = P_{dead} + P_{live}$$
$$= 1.00 + 3.00 = 4.00 \text{ kips}$$

LRFD

$$P_u = 1.2\,P_{dead} + 1.6\,P_{live}$$
$$= (1.2)(1.00) + (1.6)(3.00) = 6.00 \text{ kips}$$

2. Nominal Shear Strength of Flare-Bevel Groove Welds, Transverse Loading (Section E2.6)

$$P_n = 0.833tLF_u \qquad (\textit{Eq}.\ \text{E2.6-1})$$

3. Solve for L

ASD

$$\Omega = 2.55$$
$$P \le P_n/\Omega \qquad (\textit{Eq}.\ \text{A4.1.1-1})$$
$$P \le 0.833tLF_u/\Omega$$
$$\therefore L \ge \frac{\Omega P}{0.833tF_u}$$
$$L \ge \frac{(2.55)(4.00)}{(0.833)(0.060)(65)} = 3.14 \text{ in.}$$

LRFD

$\phi = 0.60$

$P_u \leq \phi P_n$ (*Eq.* A5.1.1-1)

$\leq \phi 0.833tLF_u$

$$\therefore L \geq \frac{P_u}{\phi 0.833tF_u}$$

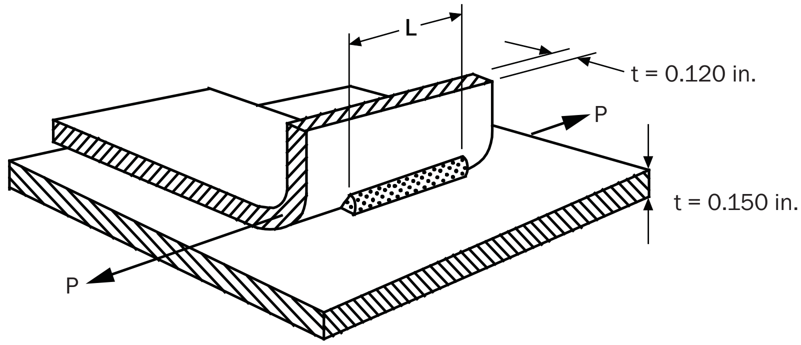
$$L \geq \frac{6.00}{(0.60)(0.833)(0.060)(65)} = 3.08 \text{ in.}$$

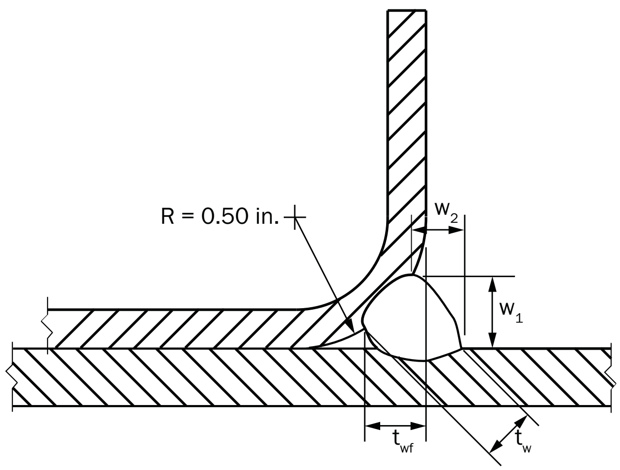
Eq. E2.6-4 need not be checked since $t < 0.10$ in.

4. Final Design

Use $\frac{1}{8}$ inch flare bevel groove weld $3\frac{1}{4}$ inch long.

Example IV-5: Flat Section With Flare Bevel Groove Weld, t > 0.10 in.





Given:

1. Steel: F_y = 60 ksi, F_u = 75 ksi
2. GMAW weld process, F_{xx} = 70 ksi.
3. Detail of connection shown in sketch.
4. w_1 = 0.188 in., w_2 = 0.125 in.
5. R = 0.50 in.
6. Lip height, h, greater than or equal to weld length, L
7. Longitudinal (shear) loading

Required:

Determine the available weld strength per inch of weld, $\frac{P_n}{\Omega}$, using ASD and ϕP_n, using LRFD .

Solution:

1. Solve for Strength Based on Shear of Weld Material (Section E2.6)

From Table E2.6-1

$t_{wf} = 0.625R = 0.625(0.50) = 0.313\,\text{in.}$

$\eta = 0.073$

$$w_f = \sqrt{w_1^2 + w_2^2} = \sqrt{0.188^2 + 0.125^2} = 0.226\,\text{in.} \qquad (Eq.\ E2.6\text{-}6)$$

$$t_w = \left[w_2 + t_{wf} - R + \sqrt{2Rw_1 - w_1^2}\right]\left(\frac{w_1}{w_f}\right) - R\eta\left(\frac{w_2}{w_f}\right)$$

$$= \left[0.125 + 0.313 - 0.50 + \sqrt{2(0.50)(0.188) - 0.188^2}\right]\left(\frac{0.188}{0.226}\right) - 0.50(0.073)\left(\frac{0.125}{0.226}\right) \qquad (Eq.\ \text{E2.6-5})$$

$$= 0.253\,\text{in.}$$

$$P_n = 0.75 t_w L F_{xx} \qquad (Eq.\ \text{E2.6-4})$$

ASD

$$P \le \frac{P_n}{\Omega} \qquad (Eq.\ \text{A4.1.1-1})$$

$$\Omega = 2.55$$

$$P \le \frac{0.75 t_w L F_{xx}}{\Omega} = \frac{0.75(0.253)(1.0)(70)}{2.55} = 5.21\,\text{kips per inch} \qquad (\text{from } Eq.\ \text{E2.6-4})$$

LRFD

$$P_u \le \phi P_n \qquad (Eq.\ \text{A5.1.1-1})$$

$$\phi = 0.60$$

$$P_u \le \phi 0.75 t_w L F_{xx} = 0.60(0.75)(0.253)(1.0)(70) = 7.97\,\text{kips per inch} \qquad (\text{from } Eq.\ \text{E2.6-4})$$

2. Solve for Strength Based on Shear of Sheet

Because $t_w > 2t$ and $h \ge L$

$$P_n = 1.50 t L F_u \qquad (Eq.\ \text{E2.6-3})$$

ASD

$$P \le \frac{P_n}{\Omega} \qquad (Eq.\ \text{A4.1.1-1})$$

$$\Omega = 2.80$$

$$P \le \frac{1.50 t L F_u}{\Omega} = \frac{1.50(0.120)(1.0)(75)}{2.80} = 4.82\,\text{kips per inch} \qquad (\text{from } Eq.\ \text{E2.6-3})$$

LRFD

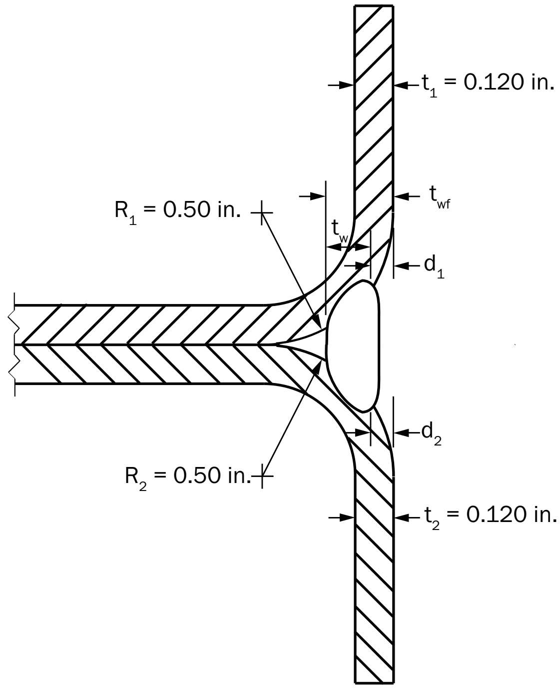
$$P_u \le \phi P_n \qquad (Eq.\ \text{A5.1.1-1})$$

$$\phi = 0.55$$

$$P_u \le \phi 1.50 t L F_u = 0.55(1.50)(0.120)(1.0)(75) = 7.43\,\text{kips per inch} \qquad (\text{from } Eq.\ \text{E2.6-3})$$

Shear of sheet controls

Example IV-6: Flat Section With Flare V Groove Weld



Given:

1. Steel: F_y = 60 ksi, F_u = 75 ksi
2. GMAW weld process, F_{xx} = 70 ksi.
2. Detail of connection shown in sketch.
3. $d_1 = d_2 = 0.125$ in.
4. $R_1 = R_2 = 0.50$ in.
5. Lip height, h, greater than or equal to L
6. Longitudinal (shear) loading

Required:

Determine the available weld strength per inch of weld, $\frac{P_n}{\Omega}$, using ASD and LRFD.

Solution:

1. Solve for Strength Based on Shear of Weld Material (Section E2.6)

$t_{wf} = 0.75R = 0.75(0.50) = 0.375\,\text{in.}$ (From Table E2.6-2)

$$\begin{aligned} t_w &= t_{wf} - d \\ &= 0.375 - 0.125 \\ &= 0.250\,\text{in.} \end{aligned}$$ (*Eq*. E2.6-7)

$P_n = 0.75 t_w L F_{xx}$ (*Eq*. E2.6-4)

ASD

$P \le \frac{P_n}{\Omega}$ (*Eq*. A4.1.1-1)

$\Omega = 2.55$

$$P \le \frac{0.75 t_w L F_{xx}}{\Omega} = \frac{0.75(0.250)(1.0)(70)}{2.55} = 5.15\,\text{kips per inch}$$

LRFD

$P_u \leq \phi P_n$ (*Eq.* A5.1.1-1)

$\phi = 0.60$

$P_u \leq \phi 0.75 t_w L F_{xx} = 0.60(0.75)(0.250)(1.0)(70) = 7.88 \text{ kips per inch}$

2. Solve for Strength Based on Shear of Sheet

Because $t_w > 2t$ and $h \geq L$

$P_n = 1.50tLF_u$ (*Eq.* E2.6-3)

ASD

$P \leq \dfrac{P_n}{\Omega}$ (*Eq.* A4.1.1-1)

$\Omega = 2.80$

$$P \leq \frac{1.50tLF_u}{\Omega} = \frac{1.50(0.120)(1.0)(75)}{2.80} = 4.82 \text{ kips per inch}$$

LRFD

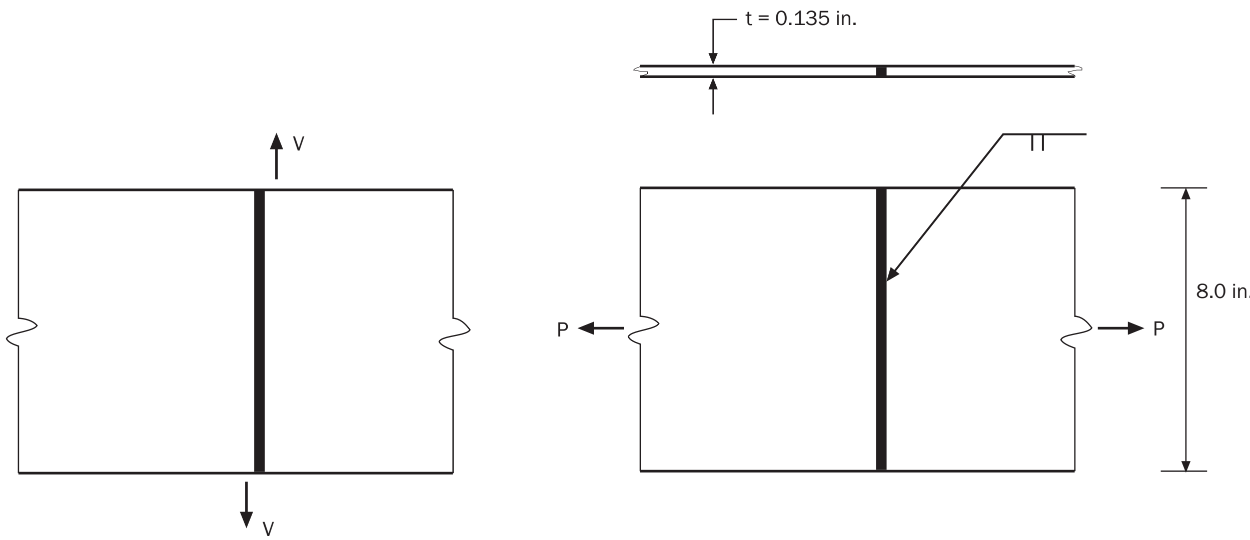
$P_u \leq \phi P_n$ (*Eq.* A5.1.1-1)

$\phi = 0.55$

$P_u \leq \phi 1.50tLF_u = 0.55(1.50)(0.120)(1.0)(75) = 7.43 \text{ kips per inch}$

Shear of sheet controls

Example IV-7: Flat Section With Groove Welded Butt Joint



Given:

1. Steel: F_y = 50 ksi
2. E60 Weld Electrode, F_{xx} = 60 ksi
3. Detail of connection shown in sketch

Required:

1. Determine the available tensile strength normal to the effective area using ASD and LRFD.
2. Determine the available shear strength on the effective area using ASD and LRFD.

Solution:

1. Available Tensile Strength Normal to the Effective Area (Section E2.1(a))

$P_n = Lt_eF_y$ (*Eq.* E2.1-1)

$= (8.0)(0.135)(50) = 54.0\,\text{kips}$

ASD

$\Omega = 1.70$

$\frac{P_n}{\Omega} = 54.0/1.70 = 31.8$ kips

LRFD

$\phi = 0.90$

$\phi P_n = (0.90)(54.0) = 48.6$ kips

2. Available Shear Strength on the Effective Area (Section E2.1(b))

Weld strength

$P_n = Lt_e0.6F_{xx}$ (*Eq.* E2.1-2)

$= 8.0(0.135)(0.6)(60) = 38.9\,\text{kips}$

ASD

$\Omega \quad = 1.90$

$$\frac{P_n}{\Omega} = \frac{38.9}{1.90} = 20.5\,\text{kips}$$

LRFD

$\phi \quad = 0.80$

$$\phi P_n = (0.80)(38.9) = 31.1\,\text{kips}$$

Base metal strength

$$P_n = Lt_eF_y \big/ \sqrt{3} \qquad (Eq.\ E2.1\text{-}3)$$

$$= (8.0)(0.135)(50) \big/ \sqrt{3} = 31.2\,\text{kips}$$

ASD

$\Omega \quad = 1.70$

$$\frac{P_n}{\Omega} = \frac{31.2}{1.70} = 18.4\,\text{kips}$$

Since base metal strength governs,

$$\frac{P_n}{\Omega} = 18.4\ \text{kips}$$

LRFD

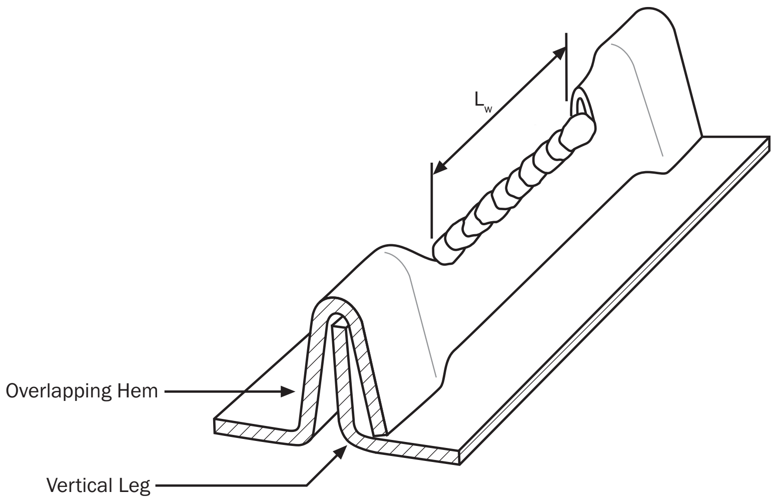
$\phi \quad = 0.90$

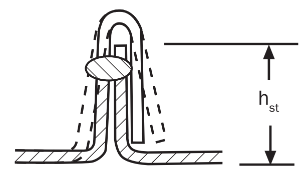
$$\phi P_n = (0.90)(31.2) = 28.1\,\text{kips}$$

Since base metal strength governs,

$$\phi P_n = 28.1\ \text{kips}$$

Example IV-8: Top Arc Seam Sidelap Weld





Cross-Section

Given:
1. Weld length: L_w = 1.5 in.
2. Base steel thickness: t = 0.0358 in.
3. Specified min. yield stress: F_{sy} = 50 ksi
4. Tensile strength: F_u = 60 ksi
5. Nominal seam height: h_{st} = 1 in.
6. Tensile strength of electrode: F_{xx} = 60 ksi
7. Weld spacing: s = 2.5 in.

Required:
ASD and LRFD available shear strengths per weld

Solution:
1. Check Limits in Section E2.4.1
a. h_{st} = 1.0 in. ≤ 1.25 in. OK
b. F_{xx} = 60 ksi ≥ 60 ksi OK
c. 0.028 in. ≤ (t = 0.0358 in.) ≤ 0.064 in. OK
d. 1.0 in. ≤ (L_w = 1.5 in.) ≤ 2.5 in.

2. Nominal Shear Strength

$$P_n = \left[4.0\left(F_u/F_{sy}\right) - 1.52\right]\left(t/L_w\right)^{0.33} L_w t F_u \qquad (\textit{Eq}.\ \text{E2.4.1-1})$$

$$= \left[4.0\left(60/50\right) - 1.52\right]\left(0.0358/1.5\right)^{0.33}(1.5)(0.0358)(60) = 3.08\,\text{kips}$$

3. Available Strength

ASD

$$\Omega = 2.60$$

$$\frac{P_n}{\Omega} = \frac{3.08}{2.60} = 1.18\,\text{kips}$$

LRFD

$$\phi = 0.60$$

$$\phi P_n = (0.60)(3.08) = 1.85\,\text{kips}$$

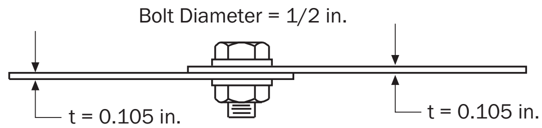
Also check Section E6 and edge distance and spacing requirements per the rupture provisions of Section E6. In addition, use *Eq.* C-E2.4.1-1 provided in the *Commentary* to determine the minimum weld spacing:

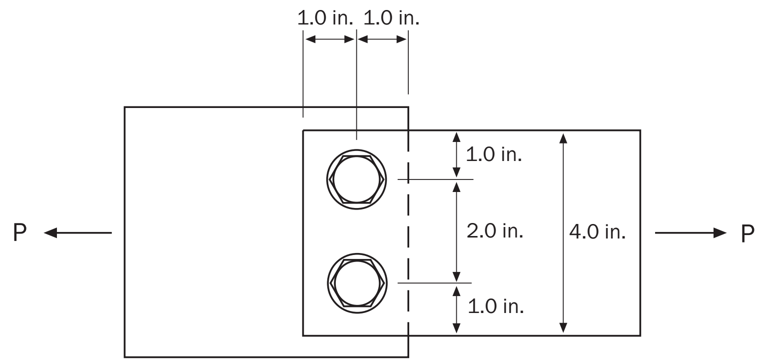
$$s = \left[6.67\left(F_u / F_{sy}\right) - 2.53\right] L_w \left(t / L_w\right)^{0.33} \qquad (\textit{Eq. } \text{C-E2.4.1-1})$$

$$= \left[6.67\left(60 / 50\right) - 2.53\right](1.5)\left(0.0358 / 1.5\right)^{0.33} = 2.39 \text{ in.} < 2.50 \text{ in. OK}$$

4.2 Bolt Examples

Example IV-9: Bolted Flat Section





Given:

1. Steel: F_y = 33 ksi, F_u = 45 ksi
2. Bolts conforming to ASTM A307 with washers under bolt head and nut
3. Detail of connection shown in sketch
4. Standard holes

Required:

Determine the ASD allowable strength, P_n/Ω, and the LRFD design strength, ϕP_n. Evaluate bearing without consideration of bolt hole deformation.

Solution:

Thickness of thinnest part connected, t

$t = 0.105$ in. < 3/16 in., therefore, Section E3 applies.

1. Minimum Spacing and Edge Distance (Sections E3 and E3.2)

Distance between bolt hole centers must be $\geq 3d$.

$3d = (3)(0.50) = 1.5 \text{ in.} < 2.0 \text{ in. OK}$

Distance between bolt hole center and edge of connecting member must be $\geq 1.5d$.

$1.5d = (1.5)(0.50) = 0.75 \text{ in.} < 1.0 \text{ in. OK}$

2. Strength Based on Tension in the Sheet at or Away From Connection (Section C2)

Available tension strength shall not exceed the smaller value of T_n from *Eq.* C2.1-1 and *Eq.* C2.2-2.

a) Yielding of the gross cross-section - Section C2.1

$A_g = (0.105)(4.0) = 0.420 \text{ in.}^2$

$T_n = A_gF_y = (0.420)(33) = 13.86 \text{ kips}$ (*Eq.* C2.1-1)

ASD

Ω_t = 1.67

$$\frac{T_n}{\Omega_t} = \frac{13.86}{1.67} = 8.30\,\text{kips}$$

LRFD

ϕ_t = 0.90

$\phi_t T_n = (0.90)(13.86) = 12.5$ kips

b) Rupture of the net section away from connection - Section C2.2

Since there are no holes or other reductions in area away from the connection:

$A_n = A_g = 0.420$ in.^{2}

$T_n = A_n F_u = (0.420)(45) = 18.9$ kips (*Eq.* C2.2-2)

ASD

Ω_t = 2.00

$$\frac{T_n}{\Omega_t} = \frac{18.9}{2.00} = 9.45\,\text{kips}$$

LRFD

ϕ_t = 0.75

$\phi_t T_n = (0.75)(18.9) = 14.2$ kips

3. Strength Based on Rupture at the Connection (Section E6)

a) Rupture of net section (Section E6.2)

A_{nt} - based on Table E3

$A_{nt} = 0.105[4.0 - 2(1/2 + 1/16)] = 0.302$ in.^{2}

Since washers are provided under both bolt head and nut and there is a single line of bolts perpendicular to the direction of force:

$U_{sl} = 3.33\,d/s \le 1.0$ (*Eq.* E6.2-4)

where:

d = 0.50 in.

s = 4.0/2 = 2.0 in.

$U_{sl} = 3.33(0.50/2.00) = 0.833 \le 1.0$

$A_e = U_{sl} A_{nt} = 0.833(0.302) = 0.252\,\text{in.}^2$ (*Eq.* E6.2-2)

$P_n = F_u A_e$ (*Eq.* E6.2-1)

$= 45(0.252) = 11.3$ kips

ASD

Ω = 2.22 for single shear connection

$$\frac{P_n}{\Omega} = \frac{11.3}{2.22} = 5.09\text{ kips}$$

LRFD

ϕ = 0.65 for single shear connection

$\phi P_n = 0.65(11.3) = 7.35$ kips

b) Block shear rupture (Section E6.3)

The block shear path shown controls.

Gross area subject to shear

$$A_{gv} = (2)(1.0)(0.105) = 0.210 \text{ in.}^2$$

Gross area subject to tension

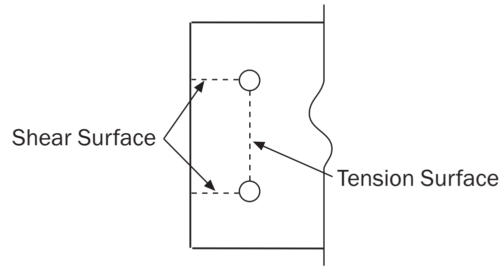
$$A_{gt} = (2.0)(0.105) = 0.210 \text{ in.}^2$$

Net area subject to shear

$$A_{nv} = 0.210 - (0.105)(2)(0.5)(0.50 + 1/16) = 0.151 \text{ in.}^2$$

Net area subject to tension

$$A_{nt} = 0.210 - (0.105)(2)(0.5)(0.50 + 1/16) = 0.151 \text{ in.}^2$$



Take R_n as the smaller value from *Eq.* E6.3-1 and *Eq.* E6.3-2.

$$R_n = 0.6F_yA_{gv} + U_{bs}F_uA_{nt} \quad (Eq.\ E6.3\text{-}1)$$

$$= (0.6)(33.0)(0.210) + 1.0(45.0)(0.151) = 11.0 \text{ kips}$$

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt} \quad (Eq.\ E6.3\text{-}2)$$

$$= (0.6)(45.0)(0.151) + 1.0(45.0)(0.151) = 10.9 \text{ kips} \leftarrow \text{CONTROLS}$$

ASD

Ω = 2.22 for bolted connections

$$\frac{R_n}{\Omega} = \frac{10.9}{2.22} = 4.91 \text{ kips}$$

LRFD

ϕ = 0.65

$$\phi P_n = (0.65)(10.9) = 7.09 \text{ kips}$$

4. Strength Based on Bearing (Section E3.3)

Since bolt hole deformation is not a consideration, Section E3.3.1 applies.

m_f = 1.0 for single shear with washers (from Table E3.3.1-2)

$d/t = 0.50/0.105 = 4.76$

Since $d/t < 10$

C = 3.0 (from Table E3.3.1-1)

$$P_n = Cm_f dtF_u \quad (Eq.\ E3.3.1\text{-}1)$$

$$= (3.00)(1.0)(0.50)(0.105)(45) = 7.09 \text{ kips/bolt}$$

ASD

Ω = 2.50

$$\frac{P_n}{\Omega} = \frac{2(7.09)}{2.50} = 5.67 \text{ kips}$$

LRFD

ϕ = 0.60

$$\phi P_n = (0.60)(2)(7.09) = 8.51 \text{ kips}$$

5. Strength Based on Bolt Shear (Appendix A, Section E3.4)

$$P_n = A_bF_n \quad (Eq.\ E3.4\text{-}1)$$

$$A_b = (\pi/4)(0.50)^2 = 0.196 \text{ in.}^2$$

$F_n = F_{nv}$ = 27.0 ksi (from Table E3.4-1, $d \geq 1/2$ in.)

$$P_n = (27.0)(0.196) = 5.29 \text{ kips/bolt}$$

ASD

$\Omega = 2.00$

$$\frac{P_n}{\Omega} = \frac{2(5.29)}{2.00} = 5.29 \text{ kips}$$

LRFD

$\phi = 0.75$ (from Table E3.4-1)

$\phi P_n = 0.75(2)(5.29) = 7.94 \text{ kips}$

6. Determine Governing Limit State

Comparing the values from Parts 2, 3, 4 and 5 above for ASD, the allowable design strength of the block shear controls:

$$\frac{P_n}{\Omega} = 4.91 \text{ kips}$$

Comparing the values from 2, 3, 4 and 5 above for LRFD, the design tensile strength on the net section of the connected part controls:

$\phi P_n = 7.09 \text{ kips}$

Using Connection Tables

1. Available Strength for Bearing Using Table IV-8b

$P_n = (7.09)(2) = 14.2 \text{ kips}$

$$\frac{P_n}{\Omega} = \frac{14.2}{2.50} = 5.68 \text{ kips (ASD)}$$

$\phi P_n = (0.6)(14.2) = 8.52 \text{ kips (LRFD)}$

2. Available Strength for Bolt Shear Using Table IV-7

$P_n = (5.30)(2 \text{ bolts}) = 10.6 \text{ kips}$

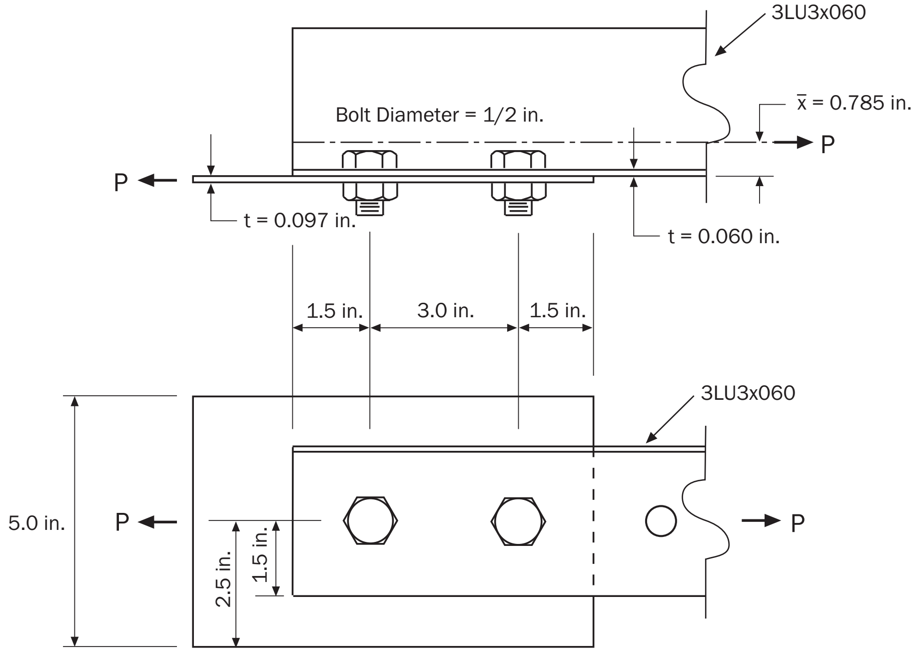
$$\frac{P_n}{\Omega} = \frac{10.6}{2.00} = 5.30 \text{ kips (ASD)}$$

$\phi P_n = (0.75)(10.6) = 7.95 \text{ kips (LRFD)}$

3. Other Checks

Check net section and edge distance as above. Check rupture as above. Block shear controls for ASD and LRFD.

Example IV-10: Bolted Angle Connection



Given:

1. Steel: ASTM A653 Grade 33: F_y = 33 ksi, F_u = 45 ksi
2. 1/2 in. diameter bolts conforming to ASTM A307 in standard holes without washers under bolt head or nut
3. Section: 3LU3x060 with gross area of 0.351 in.^{2}
4. Detail of connection shown in sketch. Note unfilled hole in angle.
5. Evaluate bearing without consideration of bolt hole deformation.

Required:

Determine the ASD allowable strength.

Solution:

Calculate strength considering:

1. Spacing and edge distance in the direction of applied force (Section E3.2)
2. Tensile strength of connected parts away from the connection (Section C2)
3. Rupture strength of connected parts at the connection (Sections E6.2 and E6.3)
4. Bearing strength of connected parts (Section E3.3)
5. Shear strength of bolts (Appendix A, Section E3.4)

1. Spacing and Edge Distance (Section E3.2)

Distance between bolt hole centers must be $\geq 3d$.

$3d = (3)(0.50) = 1.5$ in. < 3.0 in. OK

Distance between bolt hole center and edge of connecting member must be $\geq 1.5d$.

$1.5d = (1.5)(0.50) = 0.75$ in. < 1.5 in. OK

Clear distance between bolt holes must be $\geq 2d$.

$2d = (2)(0.50) = 1.0 \text{ in.} < (3.0 - 0.5) = 2.5 \text{ in. OK}$

Clear distance between edge of bolt hole and end of member must be $\geq d$.

$d = 0.5 \text{ in.} < (1.5 - 0.25) = 1.25 \text{ in. OK}$

2. Tensile Strength of Connected Parts

Angle Section

a) Tension on member away from connection (Section C2)

Nominal tensile strength shall not exceed the smallest value of T_n from Section C2:

Yielding of the gross section (Section C2.1)

$T_n = A_g F_y = (0.351)(33.0) = 11.6 \text{ kips}$ (*Eq*. C2.1-1)

$\Omega_t = 1.67$

$$\frac{T_n}{\Omega_t} = \frac{11.6}{1.67} = 6.95 \text{ kips}$$

Rupture of the net section away from the connection (Section C2.2)

Location of unfilled bolt hole will control.

A_n - based on Table E3a using 1/2 inch diameter standard holes

$A_n = 0.351 - (0.060)(0.500 + 1/16) = 0.317 \text{ in.}^2$

$T_n = A_n F_u = (0.317)(45.0) = 14.3 \text{ kips}$ (*Eq*. C2.2-2)

$\Omega_t = 2.00$

$$\frac{T_n}{\Omega_t} = \frac{14.3}{2.00} = 7.15 \text{ kips}$$

b) Rupture at the connection (Section E6)

Rupture of the net section (Section E6.2)

$T_n = F_u A_e$ (*Eq*. E6.2-1)

For an angle with two or more bolts in the line of force:

$U_{sl} = 1.0 - 1.20\bar{x}/L < 0.9$ (*Eq*. E6.2-7)

$= 1.0 - (1.20)(0.785/3.0) = 0.686 < 0.9 \text{ OK}$

$A_e = U_{sl} A_n$ (*Eq*. E6.2-2)

$= (0.686)(0.317) = 0.217 \text{ in.}^2$

$T_n = F_u A_e$ (*Eq*. E6.2-1)

$= 45(0.217) = 9.77 \text{ kips}$

$\Omega = 2.22$ for bolted connections

$$\frac{P_n}{\Omega} = \frac{9.77}{2.22} = 4.40 \text{ kips}$$

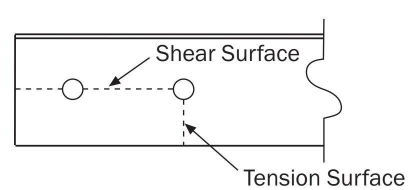
Block shear rupture (Section E6.3)

Gross area subject to shear

$A_{gv} = (1.5 + 3.0)(0.060) = 0.270 \text{ in.}^2$

Gross area subject to tension

$A_{gt} = (1.5)(0.060) = 0.0900 \text{ in.}^2$



Net area subject to shear

$A_{nv} = 0.270 - (0.060)(1.5)(0.50 + 1/16) = 0.219 \text{ in.}^2$

Net area subject to tension

$$A_{nt} = 0.0900 - (0.060)(0.5)(0.50 + 1/16) = 0.0731 \text{ in.}^2$$

Take R_n as the smaller of the values from *Eq.* E6.3-1 and *Eq.* E6.3-2.

$$R_n = 0.6F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (\textit{Eq. } \text{E6.3-1})$$

$$= (0.6)(33.0)(0.270) + 1.0(45.0)(0.0731) = 8.64 \text{ kips} \leftarrow \text{CONTROLS}$$

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt} \qquad (\textit{Eq. } \text{E6.3-2})$$

$$= (0.6)(45.0)(0.219) + 1.0(45.0)(0.0731) = 9.20 \text{ kips}$$

Ω = 2.22 for bolted connections

$$\frac{R_n}{\Omega} = \frac{8.64}{2.22} = 3.89 \text{ kips}$$

Block shear controls angle tensile strength

$$\frac{P_n}{\Omega} = 3.89 \text{ kips}$$

Flat Sheet

a) Tension on member away from connection (Section C2)

Nominal tension strength shall not exceed the smallest value of T_n from Section C2:

Yielding on the gross section (Section C2.1)

$$T_n = A_gF_y = (5.0)(0.097)(33) = 16.01 \text{ kips} \qquad (\textit{Eq. } \text{C2.1-1})$$

$$\Omega_t = 1.67$$

$$\frac{T_n}{\Omega_t} = \frac{16.01}{1.67} = 9.59 \text{ kips}$$

Rupture of the net section away from the connection (Section C2.2)

Since there are no holes or other reductions in the plate away from the connection:

$$A_n = A_g = (5.0)(0.097) = 0.485 \text{ in.}^2$$

$$T_n = A_nF_u = (0.485)(45.0) = 21.8 \text{ kips} \qquad (\textit{Eq. } \text{C2.2-2})$$

$$\Omega_t = 2.00$$

$$\frac{T_n}{\Omega_t} = \frac{21.8}{2.00} = 10.9 \text{ kips}$$

b) Rupture at the connection (Section E6)

Rupture of the net the section (Section E6.2)

A_{nt} - based on Table E3 using 1/2 inch diameter standard holes

$$A_{nt} = 0.097[5.0 - (0.500 + 1/16)] = 0.430 \text{ in.}^2$$

Since there are multiple bolts in the line parallel to the force:

$$U_{sl} = 1.0 \qquad \text{(from Table E6.2-1)}$$

$$A_e = U_{sl}A_{nt} = 1.0(0.430) = 0.430 \text{ in.}^2 \qquad (\textit{Eq. } \text{E6.2-2})$$

$$T_n = F_uA_e \qquad (\textit{Eq. } \text{E6.2-1})$$

$$= 45.0(0.430) = 19.35 \text{ kips}$$

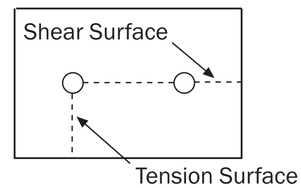
$$\Omega = 2.22$$

$$\frac{P_n}{\Omega} = \frac{19.35}{2.22} = 8.72 \text{ kips}$$

Block shear rupture (Section E6.3)

Gross area subject to shear

$$A_{gv} = (1.5 + 3.0)(0.097) = 0.437 \text{ in.}^2$$



Gross area subject to tension

$$A_{gt} = (2.5)(0.097) = 0.243 \text{ in.}^2$$

Net area subject to shear

$$A_{nv} = 0.437 - (0.097)(1.5)(0.50 + 1/16) = 0.355 \text{ in.}^2$$

Net area subject to tension

$$A_{nt} = 0.243 - (0.097)(0.5)(0.50 + 1/16) = 0.216 \text{ in.}^2$$

Take R_n as the smaller of the values from *Eq.* E6.3-1 and *Eq.* E6.3-2.

$$R_n = 0.6F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (Eq.\ E6.3\text{-}1)$$

$$= (0.6)(33.0)(0.437) + 1.0(45.0)(0.216) = 18.4 \text{ kips} \leftarrow \text{CONTROLS}$$

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt} \qquad (Eq.\ E6.3\text{-}1)$$

$$= (0.6)(45.0)(0.355) + 1.0(45.0)(0.216) = 19.3 \text{ kips}$$

Ω = 2.22 for bolted connections

$$\frac{R_n}{\Omega} = \frac{18.4}{2.22} = 8.29 \text{ kips}$$

Block shear controls sheet tensile strength

$$\frac{P_n}{\Omega} = 8.29 \text{ kips}$$

3. Bearing on Connected Parts (Section E3.3)

Since bolt hole deformation is not a consideration, Section E3.3.1 applies.

The thinner angle will control, by inspection

m_f = 0.75 for single shear without washers (from Table E3.3.1-2)

$d/t = 0.50/0.060 = 8.33$

Since $d/t < 10$

$C = 3.0$ (from Table E3.3.1-1)

$$P_n = Cm_f dtF_u \qquad (Eq.\ E3.3.1\text{-}1)$$

$$= 3.0(0.75)(0.50)(0.060)(45) = 3.04 \text{ kips/bolt}$$

$\Omega = 2.50$

$$\frac{P_n}{\Omega} = \frac{2(3.04)}{2.50} = 2.43 \text{ kips}$$

4. Bolt Shear (Appendix A, Section E3.4)

$$P_n = A_bF_n \qquad (Eq.\ E3.4\text{-}1)$$

$$A_b = (\pi/4)(0.50)^2 = 0.196 \text{ in.}^2$$

$F_n = F_{nv} = 27.0$ ksi (from Table E3.4-1, $d \geq 1/2$ in.)

$$P_n = (27)(0.196) = 5.29 \text{ kips/bolt} \qquad (Eq.\ E3.4\text{-}1)$$

$\Omega = 2.00$

$$\frac{P_n}{\Omega} = \frac{2(5.29)}{2.00} = 5.29 \text{ kips}$$

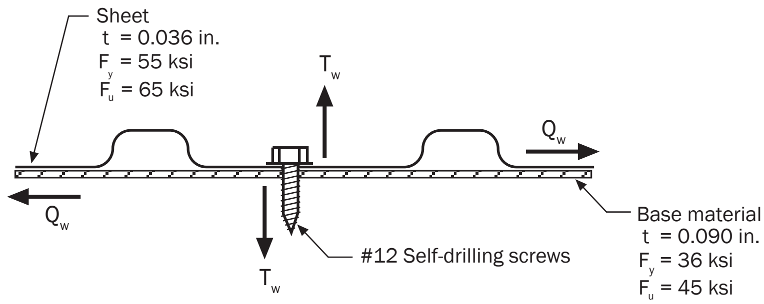
5. Determine Design Strength

Comparing the values from 2, 3, and 4 above, the allowable strength of the bolt bearing on the angle controls:

$$\frac{P_n}{\Omega} = 2.43 \text{ kips}$$

4.3 Screw Example

Example IV-11: Screwed Connection



Given:

1. Screws: #12 self-drilling spaced 12.0 in. on center
   $d = 0.216$ in.
   $d_h = 0.340$ in.
   Per manufacturer's test report
   $P_{ts} = 2.78$ kips (screw tension strength based on tests)
   $P_{ss} = 2.00$ kips (screw shear strength based on tests)
2. Detail and materials as shown above
3. Wind Forces: $T_w = 0.300$ kip, $Q_w = 0.650$ kip
4. Minimum edge distance of top sheet is 0.75 inches

Required:

Verify the shear and tensile strengths using ASD and LRFD, including interaction between shear and tension.

Solution:

1. Required Strengths

ASD

Tension: $T = 0.6T_w = 0.6(0.300) = 0.180 \text{ kip}$

Shear: $Q = 0.6Q_w = 0.6(0.650) = 0.390 \text{ kip}$

LRFD

Tension: $\overline{T} = 1.0T_w = 1.0(0.300) = 0.300 \text{ kip}$

Shear: $\overline{Q} = 1.0Q_w = 1.0(0.650) = 0.650 \text{ kip}$

2. Shear Strength (Section E4.3 and E6)

a) Connection shear limited by tilting and bearing (Section E4.3.1)

$t_1 = 0.036$ in.

$t_2 = 0.090$ in.

$t_2/t_1 = 0.090/0.036 = 2.50$

For $t_2/t_1 \geq 2.5$, P_{ns} is the smaller of

$P_{ns} = 2.7t_1dF_{u1}$ (*Eq.* E4.3.1-4)

$= (2.7)(0.036)(0.216)(65) = 1.37$ kips

$P_{ns} = 2.7t_2dF_{u2}$ (*Eq.* E4.3.1-5)

$= (2.7)(0.090)(0.216)(45) = 2.36$ kips

$P_{ns} = \min(1.37, 2.36) = 1.37$ kips

ASD

$\Omega = 3.00$

$$\frac{P_{ns}}{\Omega} = \frac{1.37}{3.0} = 0.457 \text{ kip} > 0.390 \text{ kip} \quad \text{OK} \quad \leftarrow \text{CONTROLS}$$

LRFD

$\phi = 0.50$

$$\phi P_{ns} = 0.50(1.37) = 0.685 \text{ kip} > 0.650 \text{ kip} \quad \text{OK} \quad \leftarrow \text{CONTROLS}$$

The connection tables IV-9a through IV-9d cannot be used in this case because they assume that steels with identical F_u are used for both sheets, which is not true in this example.

b) Connection strength limited by 0.75 in. clear end distance (Section E6.1)

By inspection, the thinner sheet will govern. Calculate net area per screw

$$A_{nv} = 2n\,t\,e_{net} \qquad (Eq.\ E6.1\text{-}2)$$

$$= 2(1)(0.036)(0.75) = 0.0540 \text{ in.}^2$$

$$V_n = 0.6F_uA_{nv} \qquad (Eq.\ E6.1\text{-}1)$$

$$= 0.6(65)(0.0540) = 2.11 \text{ kips}$$

ASD

$\Omega = 3.00$

$$\frac{V_n}{\Omega} = \frac{2.11}{3.0} = 0.703 \text{ kip} > 0.390 \text{ kip} \quad \text{OK}$$

LRFD

$\phi = 0.50$

$$\phi V_n = 0.50(2.11) = 1.06 \text{ kip} > 0.650 \text{ kip} \quad \text{OK}$$

c) Shear in screw (Section E4.3.2)

$P_{ns} = P_{ss}$

$= 2.00$ kips

ASD

$\Omega = 3.00$

$$\frac{P_{ns}}{\Omega} = \frac{2.00}{3.0} = 0.667 \text{ kip} > 0.390 \text{ kip} \quad \text{OK}$$

LRFD

$\phi = 0.50$

$$\phi P_{ns} = 0.50(2.00) = 1.00 \text{ kip} > 0.650 \text{ kips} \quad \text{OK}$$

For both ASD and LRFD, tilting and bearing govern shear strength.

3. Tensile Strength (Section E4.4)

a) Pull-out (Section E4.4.1)

$$P_{not} = 0.85t_cdF_{u2} \qquad (Eq.\ E4.4.1\text{-}1)$$

$$= (0.85)(0.090)(0.216)(45)$$

$$= 0.744 \text{ kip}$$

Pull-over (Section E4.4.2)

$d'_w = d_h = 0.340$ in. < ¾ in. OK

$$P_{nov} = 1.5t_1d'_wF_{u1} \qquad (Eq.\ E4.4.2\text{-}1)$$

$$= (1.5)(0.036)(0.340)(65)$$

$= 1.19$ kips

Pull-out governs

ASD

$\Omega = 3.00$

$$\frac{P_n}{\Omega} = \frac{0.744}{3.0} = 0.248 \text{ kip} > 0.180 \text{ kip OK} \quad \leftarrow \text{CONTROLS}$$

LRFD

$\phi = 0.50$

$$\phi P_n = 0.50(0.744) = 0.372 \text{ kips} > 0.300 \text{ kips OK} \quad \leftarrow \text{CONTROLS}$$

b) Tension rupture (Section E6.2)

Check the 12 in. width sheet tension rupture. By inspection, the thinner sheet will govern.

$A_{nt} = A_g - n_b d_h t$ (*Eq.* E6.2-3)

$= 12.0(0.036) - 1(0.340)(0.036) = 0.420 \text{ in.}^2$

$A_e = U_{sl} A_{nt}$ (*Eq.* E6.2-2)

$= 1.0(0.420) = 0.420 \text{ in.}^2$

$T_n = F_u A_e$ (*Eq.* E6.2-1)

$= 65(0.420) = 27.3 \text{ kips}$

ASD

$\Omega = 3.00$

$$\frac{T_n}{\Omega} = \frac{27.3}{3.0} = 9.10 \text{ kips} > 0.390 \text{ kip OK}$$

LRFD

$\phi = 0.50$

$$\phi T_n = 0.50(27.3) = 13.7 \text{ kips} > 0.650 \text{ kip OK}$$

Block shear does not apply as this is a screwed connection, per Section E6.

c) Tension in screw (Section E4.4.3)

$P_{nt} = P_{ts}$

$= 2.78$ kips

ASD

$\Omega = 3.00$

$$\frac{P_{nt}}{\Omega} = \frac{2.78}{3.0} = 0.927 \text{ kip} > 0.180 \text{ kip OK}$$

LRFD

$\phi = 0.50$

$$\phi P_{nt} = 0.50(2.78) = 1.39 \text{ kip} > 0.300 \text{ kips OK}$$

For both ASD and LRFD, pull-out governs tensile strength.

4. Combined Shear and Pull-Over (Section E4.5.1)

ASD

$$\frac{Q}{P_{ns}}+0.71\frac{T}{P_{nov}}\le\frac{1.10}{\Omega} \quad (Eq.\ E4.5.1.1\text{-}1)$$

where

$$P_{ns} = 2.7t_1dF_{u1} \quad (Eq.\ E4.5.1.1\text{-}2)$$

$$= (2.7)(0.036)(0.216)(65) = 1.36\text{ kips}$$

$$P_{nov} = 1.5t_1d_wF_{u1} \quad (Eq.\ E4.5.1.1\text{-}3)$$

$$= (1.5)(0.036)(0.340)(65) = 1.19\text{ kips}$$

$\Omega = 2.35$

$$\frac{0.390}{1.36}+0.71\frac{0.180}{1.19}\le\frac{1.10}{2.35} \quad (Eq.\ E4.5.1.1\text{-}1)$$

0.394 < 0.468 OK

LRFD

$$\frac{\overline{Q}}{P_{ns}}+0.71\frac{\overline{T}}{P_{nov}}\le 1.10\phi \quad (Eq.\ E4.5.1.2\text{-}1)$$

where

$$P_{ns} = 2.7t_1dF_{u1} \quad (Eq.\ E4.5.1.2\text{-}2)$$

$$= (2.7)(0.036)(0.216)(65) = 1.36\text{ kips}$$

$$P_{nov} = 1.5t_1d_wF_{u1} \quad (Eq.\ E4.5.1.2\text{-}3)$$

$$= (1.5)(0.036)(0.340)(65) = 1.19\text{ kips}$$

$\phi = 0.65$

$$\frac{0.650}{1.36}+0.71\frac{0.300}{1.19}\le 1.10(0.65) \quad (Eq.\ E4.5.1.2\text{-}1)$$

0.657 < 0.715 OK

5. Combined Shear and Pull-Out (Section E4.5.2)

ASD

$$\frac{Q}{P_{ns}}+\frac{T}{P_{not}}\le\frac{1.15}{\Omega} \quad (Eq.\ E4.5.2.1\text{-}1)$$

where

$$P_{ns} = 4.2\left(t_2^3d\right)^{1/2}F_{u2} \quad (Eq.\ E4.5.2.1\text{-}2)$$

$$= 4.2\left[(0.090)^3(0.216)\right]^{1/2}(45) = 2.37\text{ kips}$$

$$P_{not} = 0.85t_cdF_{u2} \quad (Eq.\ E4.5.2.1\text{-}3)$$

$$= 0.85(0.090)(0.216)(45) = 0.744\text{ kip}$$

$\Omega = 2.55$

$$\frac{0.390}{2.37}+\frac{0.180}{0.744}\le\frac{1.15}{2.55} \quad (Eq.\ E4.5.2.1\text{-}1)$$

0.407 < 0.451 OK

LRFD

$$\frac{\overline{Q}}{P_{ns}} + \frac{\overline{T}}{P_{not}} \le 1.15\phi \qquad (Eq.\ E4.5.2.2\text{-}1)$$

where

$$P_{ns} = 4.2\left(t_2^3 d\right)^{1/2} F_{u2} \qquad (Eq.\ E4.5.2.2\text{-}2)$$

$$= 4.2\left[(0.090)^3 (0.216)\right]^{1/2} (45) = 2.37 \text{ kips}$$

$$P_{not} = 0.85 t_c d F_{u2} \qquad (Eq.\ E4.5.2.2\text{-}3)$$

$$= 0.85(0.090)(0.216)(45) = 0.744 \text{ kips}$$

ϕ = 0.60

$$\frac{0.650}{2.37} + \frac{0.300}{0.744} \le 1.15(0.60) \qquad (Eq.\ E4.5.2.2\text{-}1)$$

0.678 < 0.690 OK

6. Combined Shear and Tension in Screws (Section E4.5.3)

ASD

$$\frac{Q}{P_{ss}} + \frac{T}{P_{ts}} \le \frac{1.30}{\Omega} \qquad (Eq.\ E4.5.3.1\text{-}1)$$

where

P_{ss} = 2.00 kips per manufacturer based on tests

P_{ts} = 2.78 kips per manufacturer based on tests

Ω = 3.00

$$\frac{0.390}{2.00} + \frac{0.180}{2.78} \le \frac{1.30}{3.00} \qquad (Eq.\ E4.5.3.1\text{-}1)$$

0.260 < 0.433 OK

LRFD

$$\frac{\overline{Q}}{P_{ss}} + \frac{\overline{T}}{P_{ts}} \le 1.30\phi \qquad (Eq.\ E4.5.3.2\text{-}1)$$

where

P_{ss} = 2.00 kips per manufacturer based on tests

P_{ts} = 2.78 kips per manufacturer based on tests

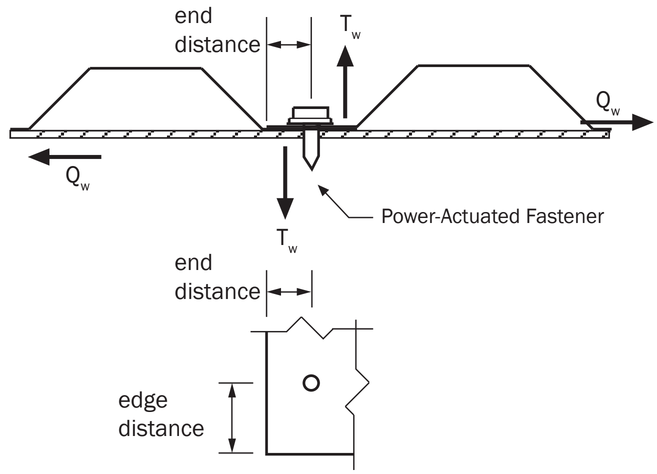
ϕ = 0.50

$$\frac{0.650}{2.00} + \frac{0.300}{2.78} \le 1.30(0.50) \qquad (Eq.\ E4.5.3.2\text{-}1)$$

0.433 < 0.650 OK

4.4 Power-Actuated Fastener Example

Example IV-12: Power-Actuated Fasteners in Shear and Tension



Given:
Sheet:

t_1 = 0.036 in.
F_{y1} = 55 ksi
F_{u1} = 65 ksi
End distance = 1.0 in.
Edge distance = 1.25 in.

Base material:

t_2 = 0.25 in.
F_{y2} = 50 ksi
F_{u2} = 65 ksi

Power-Actuated Fastener:

d = 0.180 in.
d'_w = 0.340 in.
t_w = 0.05 in.
HRC_p = 56

Nominal Wind Forces (component and cladding wind) per ASCE 7-10:

T_w = 0.190 kip
Q_w = 0.355 kip

Subsequent fasteners perpendicular to the direction of the force are spaced at 12 in. on center. The manufacturer's installation requirements stipulate that for the given thickness of base metal in this example, the fastener be driven so that the entire length of driving point, ℓ_{dp}, is located below the base metal bottom surface.

Required:
Verify the shear and tensile strengths of the power-actuated fastener connection using ASD and LRFD, including interaction between shear and tension.

Solution:

1. Required Strengths

ASD:

Tension, $T = 0.6T_w = 0.6(0.190) = 0.114$ kip
Shear, $Q = 0.6Q_w = 0.6(0.355) = 0.213$ kip

LRFD:

Tension, $T_u = 1.0T_w = 1.0(0.190) = 0.190$ kip

Shear, $Q_u = 1.0Q_w = 1.0(0.355) = 0.355$ kip

2. Applicability of Provisions (Section E5)

$t_2 = 0.25$ in. ≤ 0.75 in. OK

$t_1 = 0.036$ in. ≤ 0.06 in. OK

Since 0.106 in. < d = 0.180 in. < 0.206 in. OK

Deck component wind loads, non-diaphragm application OK

Check minimum end distance per Table E5.1-1:

Since 1.0 in. > 0.5 in. OK (Table E5.1-1)

3. Tension Strength (Section E5.2)

Check washer thickness limitation per Section E5.2:

$t_w = 0.050$ in. > 0.039 in. OK

Since the fastener is embedded such that the length ℓ_{dp} is fully located below the thickness t_2:

$d = d_s$

Since $HRC_p = 56 > 52$, use $F_{uh} = 260$ ksi

$$P_{ntp} = (d/2)^2 \pi F_{uh} = (0.180/2)^2 \pi (260) = 6.62 \text{ kips}$$ (*Eq.* E5.2.1-1)

ASD

$\Omega = 2.65$ (Section E5.2.1)

$$T \leq \frac{P_{ntp}}{\Omega} = \frac{6.62}{2.65} = 2.50 \text{ kips} > 0.114 \text{ kip} \quad \text{OK}$$

LRFD

$\phi = 0.60$ (Section E5.2.1)

$$T_u \leq \phi P_{ntp} = 0.60(6.62) = 3.97 \text{ kips} > 0.190 \text{ kip} \quad \text{OK}$$

4. Pull-Out Strength (Section E5.2.2)

Since ℓ_{dp} is located below the bottom of base material:

The nominal pull-out strength, P_{not}, determined through a test program, can be obtained from the manufacturer. Alternatively, the *Commentary* Table C-E5.2.2-1 can be consulted. Therein, for 0.177 in. < (d_s = 0.180 in.) < 0.206 in., P_{not} is given as 1.97 kips:

ASD

$\Omega = 4.00$ (Section E5.2.2)

$$T \leq \frac{P_{not}}{\Omega} = \frac{1.97}{4.00} = 0.493 \text{ kip} > 0.114 \text{ kip} \quad \text{OK}$$

LRFD

$\phi = 0.40$ (Section E5.2.2)

$$T_u \leq \phi P_{not} = 0.40(1.97) = 0.788 \text{ kip} > 0.190 \text{ kip} \quad \text{OK}$$

5. Tension Pull-Over Strength (Section E5.2.3)

$$P_{nov} = \alpha_w t_1 d'_w F_{u1} = (1.50)(0.036)(0.34)(65) = 1.19 \text{ kips}$$ (*Eq.* E5.2.3-1)

ASD

$\Omega = 3.00$ (Section E5.2.3)

$$T \le \frac{P_{nov}}{\Omega} = \frac{1.19}{3.00} = 0.397\,\text{kip} > 0.114\,\text{kip} \quad \text{GOVERNS TENSION}$$

LRFD

$\phi = 0.50$ (Section E5.2.3)

$$T_u \le \phi P_{nov} = 0.50(1.19) = 0.595\ \text{kip} > 0.190\ \text{kip} \quad \text{GOVERNS TENSION}$$

6. Shear Strength of PAF (Section E5.3.1)

Since $HRC_p = 56 > 52$, use $F_{uh} = 260$ ksi

$$P_{nsp} = 0.60(d/2)^2 \pi F_{uh} = 0.60(0.180/2)^2 \pi(260) = 3.97\ \text{kips} \qquad (Eq.\ \text{E5.3.1-1})$$

ASD

$\Omega = 2.65$ (Section E5.3.1)

$$Q \le \frac{P_{nsp}}{\Omega} = \frac{3.97}{2.65} = 1.50\,\text{kips} > 0.213\,\text{kip} \quad \text{OK}$$

LRFD

$\phi = 0.60$ (Section E5.3.1)

$$Q_u \le \phi P_{nsp} = 0.60(3.97) = 2.38\ \text{kips} > 0.355\ \text{kip} \quad \text{OK}$$

7. Shear Bearing and Tilting Strength of PAF (Section E5.3.2)

$t_2/t_1 = 0.250/0.036 = 6.94 > 2$ OK

$t_2 = 0.250$ in. > 0.125 in. OK

0.146 in. $<$ (d_s = 0.180 in.) $\cong$ 0.177 in. OK

$$P_{nbp} = \alpha_b d_s t_1 F_{u1} = (3.2)(0.180)(0.036)(65) = 1.35\ \text{kips} \qquad (Eq.\ \text{E5.3.2-1})$$

ASD

$\Omega = 2.05$ (Section E5.3.2)

$$Q \le \frac{P_{nbp}}{\Omega} = \frac{1.35}{2.05} = 0.659\,\text{kip} > 0.213\,\text{kip} \quad \text{GOVERNS SHEAR}$$

LRFD

$\phi = 0.80$ (Section E5.3.2)

$$Q_u \le \phi P_{nbp} = 0.80(1.35) = 1.08\ \text{kips} > 0.355\ \text{kips} \quad \text{GOVERNS SHEAR}$$

8. Shear Pull-Out Strength of PAF (Section E5.3.3)

0.113 in. $< t_2 = 0.250$ in. < 0.75 in. OK

0.106 in. $<$ (d_s = 0.180) in. $<$ 0.206 in. OK

As given in the problem statement, the fastener is driven through the depth larger than $0.6t_2$.

$$P_{nos} = \frac{d_{ae}^{1.8} t_2^{0.2}\left(F_{y2}E^2\right)^{1/3}}{30} = \frac{(0.180)^{1.8}(0.250)^{0.2}\left[(50)(29500)^2\right]^{1/3}}{30} = 4.06\ \text{kips} \qquad (Eq.\ \text{E5.3.3-1})$$

ASD

$\Omega = 2.55$ (Section E5.3.3)

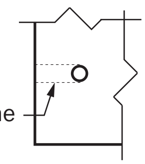
$$Q \le \frac{P_{nos}}{\Omega} = \frac{4.06}{2.55} = 1.59\,\text{kips} > 0.213\,\text{kip} \quad \text{OK}$$

LRFD

$\phi = 0.60$ (Section E5.3.3)

$$Q_u \le \phi P_{nos} = 0.60(4.06) = 2.44 \text{ kips} > 0.355 \text{ kip} \quad \text{OK}$$

9. Rupture Strength of Top Sheet (Section E6)



(a) Shear Rupture:

$d_{hole} = 0.180(1.10) = 0.20$ in. (Section E5.3.4)

$A_{nv} = (1.0 - 0.20/2)(0.036)2 = 0.0648$ in.^{2} (*Eq*. E6.1-2)

$V_n = 0.6F_uA_{nv} = 0.6(65)(0.0648) = 2.53$ kips (*Eq*. E6.1-1)

ASD

$\Omega = 3.00$ (Table E6-1)

$$Q \le \frac{V_n}{\Omega} = \frac{2.53}{3.00} = 0.843\,\text{kip} > 0.213\,\text{kip} \quad \text{OK}$$

LRFD

$\phi = 0.50$ (Table E6-1)

$$Q_u \le \phi V_n = 0.50(2.53) = 1.27 \text{ kips} > 0.355 \text{ kip} \quad \text{OK}$$

b) Tension Rupture (Section E6.2)

Check tension rupture of 12 in. width sheet. By inspection, the thinner sheet will govern.

$A_{nt} = A_g - n_bd_ht$ (*Eq*. E6.2-3)

$= 12.0(0.036) - (1)(0.200)(0.036) = 0.425\,\text{in.}^2$

$A_e = U_{sl}A_{nt}$ (*Eq*. E6.2-2)

$= 1.0(0.425) = 0.425\,\text{in.}^2$

$T_n = F_uA_e$ (*Eq*. E6.2-1)

$= 65(0.425) = 27.6\,\text{kips}$

ASD

$\Omega = 3.00$

$$\frac{T_n}{\Omega} = \frac{27.6}{3.00} = 9.20 \text{ kips} > 0.213 \text{ kip} \quad \text{OK}$$

LRFD

$\phi = 0.50$

$\phi T_n = 0.50(27.6) = 13.8$ kips > 0.355 kip OK

Block shear does not apply as this is a power-actuated fastener connection, per Section E6.

10. Combined Tension and Shear Strength (Section E5.4)

The interaction can be considered by the criteria provided in *Commentary* Section E5.4:

$$\left(\frac{T_r}{T_c}\right)^n + \left(\frac{V_r}{V_c}\right)^n \le 1.0 \qquad (Eq.\ \text{C-E5.4-1})$$

Since neither shear nor tension strength are governed by the fastener fracture strength, n = 1.

ASD:

$$\left(\frac{0.114}{0.397}\right)^1 + \left(\frac{0.213}{0.659}\right)^1 = 0.610 < 1.0 \ \text{ OK}$$

LRFD:

$$\left(\frac{0.190}{0.595}\right)^1 + \left(\frac{0.355}{1.08}\right)^1 = 0.648 < 1.0 \ \text{ OK}$$

Therefore, the connection is satisfactory as configured.

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TABLE OF CONTENTS

PART V
SUPPLEMENTARY INFORMATION

For Use With the
2012 Edition of the
North American Specification for the Design
of Cold-Formed Steel Structural Members

SECTION 1 - SPECIFICATION CROSS-REFERENCE

The table below shows where the provisions of the *Specification* are illustrated within the Example Problems in this *Manual*.

Specification Section	Example Problem
A. GENERAL PROVISIONS	
A1	
A1.1	
A1.2	I-18, III-14
A1.3	
A1.4	
A2	
A2.1	
A2.2	
A2.3	
A2.4	
A3	
A4	
A4.1	
A4.1.1	I-16, I-18, II-2, II-3, II-4, II-5, II-6, II-7, II-12, II-13, III-2, III-11, III-14, IV-4, IV-5, IV-6
A4.1.2	
A5	
A5.1	
A5.1.1	I-16, I-18, II-1, II-4, II-6, II-7, II-12, II-13, III-2, III-11, III-12, III-14, IV-4, IV-5, IV-6
A5.1.2	
A6	
A7	
A7.1	
A7.2	I-15, II-10
A8	
A9	
B. ELEMENTS	
B1	
B1.1	I-8, I-9, I-10, I-13, I-16, I-17, II-3
B1.2	I-13
B1.3	I-18
B2	III-4
B2.1	I-8, I-9, I-10, I-11, I-12, I-13, I-14 I-16, I-18, I-19, II-6, II-9, II-10, III-2, III-11
B2.2	III-2
B2.3	I-8, I-9, I-10, I-13, I-14, I-16, I-18, I-19, II-9
B2.4	II-10
B2.5	
B3	
B3.1	I-9, I-12, I-13, I-14, III-2
B3.2	I-8, I-10, I-12, I-14, I-19, II-6
B4	I-8, I-10, I-11, I-13, I-14, I-19, III-2
B5	
B5.1	I-14, I-19
B5.1.1	I-14
B5.1.2	
B5.2	
C. MEMBERS	
C1	
C2	IV-1, IV-10
C2.1	IV-1, IV-2, IV-3, IV-9, IV-10
C2.2	IV-1, IV-2, IV-3, IV-9, IV-10
C3	
C3.1	I-16, II-1, II-2, III-1
C3.1.1	I-10, I-13, I-14, I-15, I-16, I-19, II-1, II-2, II-3, II-5, II-7, II-9, II-10, II-11, III-1, III-12
C3.1.2	II-2, II-5
C3.1.2.1	II-1, II-2, II-3, II-4, II-6, III-2, III-3, III-5, III-10
C3.1.2.2	II-9
C3.1.3	II-8
C3.1.4	II-1, II-2, II-5, III-3
C3.2	II-1, II-2, II-3
C3.2.1	II-1, II-2, II-3, II-4, II-10, II-15
C3.2.2	II-10
C3.3	II-3
C3.3.1	II-2, II-3, II-10
C3.3.2	II-1, II-10, II-15
C3.4	II-1, II-2, II-3
C3.4.1	II-1, II-2, II-3, II-10
C3.4.2	II-10
C3.5	II-3
C3.5.1	II-2, II-3, II-10
C3.5.2	II-1, II-10

Specification Section	Example Problem
C3.6	II-11
C3.7	II-12
C3.7.1	II-12
C3.7.2	II-12
C3.7.3	
C3.7.4	
C4	I-13, III-5
C4.1	III-1, III-2, III-3, III-5, III-6, III-7, III-8, III-9, III-10, III-11, III-12
C4.1.1	III-1, III-2, III-3, III-5, III-6, III-7, III-8, III-9, III-10, III-11, III-12
C4.1.2	III-2, III-3, III-5, III-10
C4.1.3	
C4.1.4	
C4.1.5	III-6
C4.2	III-3, III-4
C5	
C5.1	
C5.1.1	
C5.1.2	
C5.2	III-6
C5.2.1	III-1, III-5, III-6, III-9, III-10, III-11
C5.2.2	III-1, III-6, III-9, III-10, III-11, III-12

D. STRUCTURAL ASSEMBLIES

Specification Section	Example Problem
D1	
D1.1	III-10
D1.2	III-10
D1.3	I-19
D2	
D3	
D3.1	
D3.2	
D3.2.1	
D3.3	III-2, III-13
D4	
D4.1	III-2, III-3, III-4
D5	
D6	
D6.1	
D6.1.1	II-1, II-2
D6.1.2	II-3
D6.1.3	III-7
D6.1.4	III-8
D6.2	
D6.2.1	
D6.3	
D6.3.1	
D6.3.2	

E. CONNECTIONS AND JOINTS

Specification Section	Example Problem
E1	
E2	
E2.1	IV-7
E2.2	IV-2
E2.2.1	IV-2
E2.2.2	
E2.2.2.1	IV-2
E2.2.2.2	
E2.2.3	IV-2
E2.2.4	
E2.2.4.1	IV-2
E2.2.4.2	IV-2
E2.3	IV-3
E2.3.1	IV-3
E2.3.2	
E2.3.2.1	IV-3
E2.3.2.2	
E2.4	
E2.4.1	IV-8
E2.5	IV-1
E2.6	IV-4, IV-5, IV-6
E2.7	
E3	IV-9
E3.1	
E3.2	IV-9, IV-10
E3.3	IV-9, IV-10
E3.3.1	IV-9, IV-10
E3.3.2	
E3.4	IV-9, IV-10
E4	
E4.1	IV-11
E4.2	
E4.3	
E4.3.1	IV-11
E4.3.2	
E4.4	IV-11
E4.4.1	III-10, IV-11, VI-2
E4.4.2	III-10, IV-11
E4.4.3	IV-11
E4.5	
E4.5.1	IV-11
E4.5.2	IV-11
E4.5.3	IV-11
E5	IV-12
E5.1	IV-12
E5.2	IV-12

SECTION 2 - LATERALLY UNBRACED COMPRESSION FLANGES

Previous editions of this *Manual* provided a multi-step design method for the calculation of the strength of members that will not buckle laterally while the compression flange or flanges themselves are laterally unbraced and can buckle separately by a deflection of the compression flange relative to the tension flange, accompanied by out-of-plane bending of the web and the rest of the section. An example of such a situation is the use of a hat section as a flexural member in such a manner that the "brims" are in compression. This is classified as a mode of *distortional buckling*, which is now evaluated using the provisions of the *Specification*.

The *Specification* provides both prescriptive and rational analysis methods for the consideration of distortional buckling that involves buckling of a flange and lip together. Distortional buckling modes involving out-of-plane bending of the web are not covered by the prescriptive methods; however, the Direct Strength Method provided in Appendix 1, which is permitted by *Specification* Sections C3.1.4(b) for flexural members and C4.2(b) for compression members, is generally capable of capturing this distortional mode. For this reason, the previous published design method has been removed from this edition of the *Manual*. See Example II-13 for an example of the rational analysis procedure.

SECTION 3 - TORSIONAL-FLEXURAL BUCKLING OF NON-SYMMETRICAL SHAPES

Torsional-flexural buckling of non-symmetrical sections is not covered by the *Specification*. These sections can be designed by taking F_e in Section C4 equal to σ_{TFO}.

The elastic torsional-flexural buckling stress, σ_{TFO}, is less than the smallest of the Euler buckling stresses about the x- and y- axes and the torsional buckling stress. The value of σ_{TFO} can be obtained from the following equation by trial and error:

$$\left(\frac{\sigma_{TFO}^{3}}{\sigma_{ex}\sigma_{ey}\sigma_{t}}\right)\alpha-\left(\frac{\sigma_{TFO}^{2}}{\sigma_{ey}\sigma_{t}}\right)\gamma-\left(\frac{\sigma_{TFO}^{2}}{\sigma_{ex}\sigma_{t}}\right)\beta-\left(\frac{\sigma_{TFO}^{2}}{\sigma_{ex}\sigma_{ey}}\right)+\frac{\sigma_{TFO}}{\sigma_{ex}}+\frac{\sigma_{TFO}}{\sigma_{ey}}+\frac{\sigma_{TFO}}{\sigma_{t}}=1$$

The following equation may be used for a first approximation:

$$\sigma_{TFO}=\frac{\left(\sigma_{ex}\sigma_{ey}+\sigma_{ex}\sigma_{t}+\sigma_{ey}\sigma_{t}\right)}{2\left(\sigma_{ex}\gamma+\sigma_{ey}\beta+\sigma_{t}\right)}-\frac{\sqrt{\left(\sigma_{ex}\sigma_{ey}+\sigma_{ex}\sigma_{t}+\sigma_{ey}\sigma_{t}\right)^{2}-4\left(\sigma_{ex}\sigma_{ey}\sigma_{t}\right)\left(\sigma_{ex}\gamma+\sigma_{ey}\beta+\sigma_{t}\right)}}{2\left(\sigma_{ex}\gamma+\sigma_{ey}\beta+\sigma_{t}\right)}$$

where

$$\sigma_{ex}=\frac{\pi^2 E}{\left(KL/r_x\right)^2}\text{, ksi}$$

$$\sigma_{ey}=\frac{\pi^2 E}{\left(KL/r_y\right)^2}\text{, ksi}$$

$$\sigma_{t}=\frac{1}{I_p}\left[GJ+\frac{\pi^2 EC_w}{(KL)^2}\right]\text{, ksi}$$

α = $1-(x_o/r_o)^2-(y_o/r_o)^2$

γ = $1-(y_o/r_o)^2$

β = $1-(x_o/r_o)^2$

E = modulus of elasticity = 29,500 ksi

L = unbraced length of compression member, in.

r_x = radius of gyration of cross-section about the x-axis, in.

r_y = radius of gyration of cross-section about the y-axis, in.

r_o = polar radius of gyration of cross-section about the shear center, in.

I_p = polar moment of inertia about the shear center, $in.^4$ = $Ar_o^2 = I_x + I_y + Ax_o^2 + Ay_o^2$

G = shear modulus = 11,300 ksi

J = St. Venant torsion constant of the cross-section, $in.^4$ For open sections composed of n segments of uniform thickness = $(1/3)\left(\ell_1 t_1^3+\ell_2 t_2^3+\ldots+\ell_n t_n^3\right)$

C_w= warping constant of torsion of the cross-section, $in.^6$

ℓ_i = length of cross-section middle line of segment i, in.

t_i = wall thickness of segment i, in.

x_o = distance from shear center to centroid along the principal x-axis, in.

y_o = distance from shear center to centroid along the principal y-axis, in.

For any section, the values of x_o, y_o and C_w can be computed from the following relationships (terms are defined in Figure 3-1):

$$x_o = \frac{1}{I_x}\int_0^l w_c y t \, ds, \text{ in.}$$

$$y_o = \frac{1}{I_y}\int_0^l w_c x t \, ds, \text{ in.}$$

$$C_w = \int_0^l (w_o)^2 t \, ds - \frac{1}{A}\left[\int_0^l w_o t \, ds\right]^2, \text{ in.}^6$$

where

I_x and I_y = centroidal moments of inertia of the cross-section about the principal x- and y- axes, in.^{4}

A = total area of the cross-section, in.^{2}

t = wall thickness, in.

w_c = $\int_0^s R_c ds$, in.^{2}

w_o = $\int_0^s R_o ds$, in.^{2}

x and y = the coordinates measured from the centroid to any point P along the middle line of the cross-section, in.

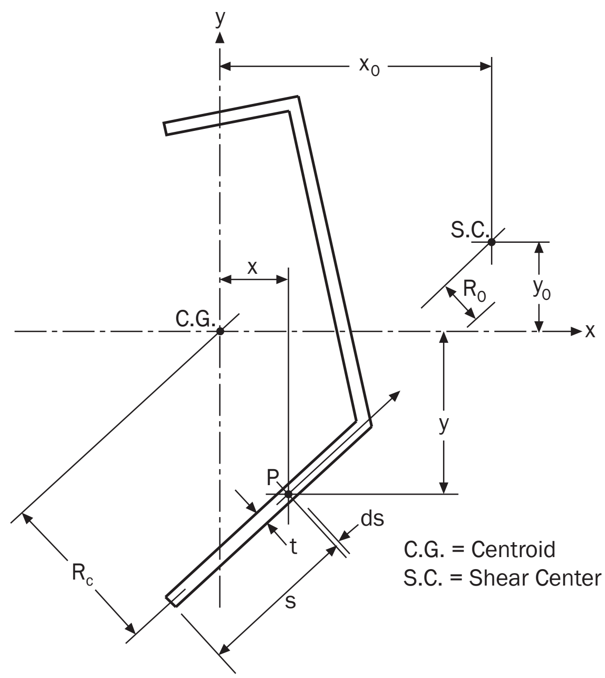

Figure 3-1
Non-Symmetrical Cross-Section

s = distance measured along the middle line of the cross-section from one end to the point P, in.

l = total length of the middle line of the cross-section, in.

R_c and R_o = perpendicular distances from the centroid (C.G.) and shear center (S.C.), respectively, to the middle line at P, in. R_c or R_o is positive if a vector tangent to the middle line at P in the direction of increasing s has a counter-clockwise moment about the C.G. or S.C. as shown in Figure 3-1

SECTION 4 - SUGGESTED COLD-FORMED STEEL STRUCTURAL FRAMING, ENGINEERING, FABRICATION, AND ERECTION PROCEDURES FOR QUALITY CONSTRUCTION

The previously published "Suggested Cold-Formed Steel Structural Framing, Engineering, Fabrication, and Erection Procedures for Quality Construction" has been superseded by the AISI *Code of Standard Practice for Cold-Formed Steel Structural Framing* which provides more detailed information on many of the same topics. In 2011, the AISI Committee on Framing Standards completed the process of updating the AISI *Code of Standard Practice* as an ANSI-approved American National Standard, which is designated as AISI S202-11.

The Preface to AISI S202-11 reads,

The American Iron and Steel Institute Committee on Framing Standards has developed this *Code of Standard Practice for Cold-Formed Steel Structural Framing (Code of Standard Practice)* to address trade practices for design, fabrication and installation of *cold-formed steel structural framing* products.

This *Code of Standard Practice* is intended to serve as a state-of-the-art guide as well as a voluntary model for establishing contractual relationships between various parties in a construction project where cold-formed steel structural materials, components, or assemblies are used. It is not intended to take precedence over the *contract, construction documents* or the use of good judgment for specific construction projects and conditions. However, these provisions are considered suitable for reference or inclusion in *contracts or construction documents* and serve as a model for that purpose.

This *Code of Standard Practice* is not applicable to *non-structural members*, including but not limited to interior drywall framing, which is addressed by ASTM C645 and C754, or *structural steel*, structural steel joists, steel deck, *metal building systems* or rack structures, which are addressed by *AISC, SJI, SDI, MBMA* and *RMI*, respectively.

AISI S202-11 is available from the American Iron and Steel Institute (www.steel.org) and Steel Framing Alliance (www.steelframing.org).

TABLE OF CONTENTS

PART VI
TEST PROCEDURES

For Use With the
2012 Edition of the
North American Specification for the Design
of Cold-Formed Steel Structural Members

SECTION 1 - BIBLIOGRAPHY

BIBLIOGRAPHY OF TEST PROCEDURES
PERTINENT TO COLD-FORMED STEEL

The following list of U.S. and Canadian publications on testing is provided for the convenience of the Manual user. No representation of correctness or completeness is implied.

ASTM Publications:

Sheet Steel, Mechanical Testing, General

ASTM A370 Standard Test Methods and Definitions for Mechanical Testing of Steel Products

ASTM E6 Standard Terminology Relating to Methods of Mechanical Testing

Sheet Steel, Mechanical Testing, Calibration and Verification

ASTM E4 Standard Practices for Force Verification of Testing Machines

ASTM E74 Standard Practice of Calibration of Force-Measuring Instruments for Verifying the Force Indication of Testing Machines

ASTM E83 Standard Practice for Verification and Classification of Extensometer Systems

Sheet Steel, Mechanical Testing, Tension

ASTM E8 Standard Test Methods for Tension Testing of Metallic Materials

ASTM E21 Standard Test Methods for Elevated Temperature Tension Tests of Metallic Materials

Sheet Steel, Mechanical Testing, Compression

ASTM E9 Standard Test Methods of Compression Testing of Metallic Materials at Room Temperature

Sheet Steel, Chemistry

ASTM E350 Standard Test Methods for Chemical Analysis of Carbon Steel, Low-Alloy Steel, Silicon Electrical Steel, Ingot Iron, and Wrought Iron

Sheet Steel, Coating Tests

ASTM E376 Standard Practice for Measuring Coating Thickness by Magnetic-Field or Eddy-Current (Electromagnetic) Testing Methods

ASTM E797 Standard Practice for Measuring Thickness by Manual Ultrasonic Pulse-Echo Contact Method

Sheet Steel, Forming Parameters

ASTM E517 Standard Test Method for Plastic Strain Ratio r for Sheet Metal

Structural Testing of Sheet Steel Assemblies

ASTM C645 Standard Specification for Nonstructural Steel Framing Members

ASTM C754 Standard Specification for Installation of Steel Framing Members to Receive Screw-Attached Gypsum Panel Products

ASTM E72 Standard Test Methods of Conducting Strength Tests of Panels for Building Construction

ASTM E73 Standard Practice for Static Load Testing of Truss Assemblies

ASTM E330 Standard Test Methods for Structural Performance of Exterior Windows, Doors, Skylights and Curtain Walls by Uniform Static Air Pressure Difference

ASTM E455	Standard Test Method for Static Load Testing of Framed Floor or Roof Diaphragm Constructions for Buildings
ASTM E564	Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings
ASTM E575	Standard Practice for Reporting Data from Structural Tests of Building Constructions, Elements, Connections, and Assemblies
ASTM E695	Standard Method for Measuring Relative Resistance of Wall, Floor, and Roof Construction to Impact Loading
ASTM E1592	Standard Test Method for Structural Performance of Sheet Metal Roof and Siding Systems by Uniform Static Air Pressure Difference

Acoustical Testing of Sheet Steel Assemblies

ASTM E90	Standard Test Method for Laboratory Measurement of Airborne Sound Transmission Loss of Building Partitions and Elements
ASTM E336	Standard Test Method for Measurement of Airborne Sound Attenuation between Rooms in Buildings
ASTM E413	Classification for Rating Sound Insulation
ASTM E492	Standard Test Method for Laboratory Measurement of Impact Sound Transmission Through Floor-Ceiling Assemblies Using the Tapping Machine

Moisture Testing of Sheet Steel Assemblies

ASTM E96	Standard Test Methods for Water Vapor Transmission of Materials
ASTM E331	Standard Test Method for Water Penetration of Exterior Windows, Skylights, Doors, and Curtain Walls by Uniform Static Air Pressure Difference
ASTM E547	Standard Test Method for Water Penetration of Exterior Windows, Skylights, Doors, and Curtain Walls by Cyclic Static Air Pressure Difference

Fire Testing of Sheet Steel Assemblies

ASTM E119	Standard Test Methods for Fire Tests of Building Construction and Materials

Welding Test Procedures

ASTM E390	Standard Reference Radiographs for Steel Fusion Welds

Fatigue Test Procedures

ASTM E466	Standard Practice for Conducting Force Controlled Constant Amplitude Axial Fatigue Tests of Metallic Materials
ASTM E467	Standard Practice for Verification of Constant Amplitude Dynamic Forces in an Axial Fatigue Testing System
ASTM E468	Standard Practice for Presentation of Constant Amplitude Fatigue Test Results for Metallic Materials
ASTM E739	Standard Practice for Statistical Analysis of Linear or Linearized Stress-Life (S-N) and Strain-Life (ε-N) Fatigue Data

Joining and Fastening Test Procedures

ASTM E488	Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements

General References

ASTM E631	Standard Terminology of Building Constructions
IEEE/ASTM SI-10	American National Standard for Metric Practice

Other Publications:

Test Procedure for Shear Resistance of Small-Scale Framed Wall Specimens:

Amir Simann and Teoman Pekoz, "Diaphragm Braced Members and Design of Wall Studs," *Journal of the Structural Division,* ASCE, January 1976.

Canadian Sheet Steel Building Institute, "Criteria for the Testing of Composite Slabs," CSSBI S2-02, March 2002.

SECTION 2 - EXAMPLE PROBLEMS

EXAMPLE VI-1: Computing ϕ And Ω Factors From Test Data Using Section F1.1(a)

Given:

1. An unusual weld configuration made up of a group of arc seam welds is tested giving the following test strengths.

Test	Strength (kips)
1.	5.60
2.	6.00
3.	5.80
4.	5.90

The failure mode is plate tearing for all tests.

Required:

1. Using Section F1.1(a), determine the resistance factor, ϕ, for this assembly.
2. Using Section F1.1(a), determine the factor of safety, Ω, for this assembly.

Solution:

1. Calculate the mean test value

R_n = (5.6 + 6.0 + 5.8 + 5.9)/4 = 5.83

2. Check maximum deviation

Test 1 controls by inspection.

(5.83-5.60)/5.83 = 0.039 < 0.15 OK

3. Compute the correction factor, C_p

$$C_p = (1 + 1/n)m/(m-2) \qquad (\textit{Eq. } \text{F1.1-4})$$

where

n = number of tests = 4

m = n-1 = 3

$$C_p = (1 + 1/4)3/(3-2) = 3.75 \qquad (\textit{Eq. } \text{F1.1-4})$$

4. Compute the standard deviation of the test results

$$s_t = \sqrt{\frac{\sum_{i=1}^{n}(x_i - \bar{x})^2}{n-1}}$$

$$= \sqrt{\frac{(5.6-5.83)^2 + (6.0-5.83)^2 + (5.8-5.83)^2 + (5.9-5.83)^2}{4-1}}$$

= 0.171

5. Compute the coefficient of variation of the test results, V_p

$$V_p = s_t/R_n \quad (Eq.\ F1.1\text{-}5)$$
$$= 0.171 / 5.83$$
$$= 0.029 < 0.065 \therefore \text{ use } 0.065$$

6. Obtain M_m, F_m, V_M, and V_F from Table F1 of the *Specification*

For Arc Seam Welds - Plate Tearing

$M_m = 1.10$

$F_m = 1.00$

$V_M = 0.10$

$V_F = 0.10$

7. Determine P_m, β_o, V_Q and C_ϕ

$P_m = 1.0$

$\beta_o = 3.5$ (for connections for the United States)

$V_Q = 0.21$ (for LRFD and LSD)

$C_\phi = 1.52$ (for LRFD)

8. Compute ϕ

$$\phi = C_\phi (M_m F_m P_m) e^{-\beta_o \sqrt{V_M{}^2 + V_F{}^2 + C_P V_P{}^2 + V_Q{}^2}} \quad (Eq.\ F1.1\text{-}2)$$
$$= 1.52\left[(1.10)(1.0)(1.0)\right] e^{-3.5\sqrt{0.10^2 + 0.10^2 + (3.75)0.065^2 + 0.21^2}}$$
$$= 0.62$$

9. Compute Ω

$$\Omega = \frac{1.6}{\phi} \quad (Eq.\ F1.2\text{-}2)$$
$$= \frac{1.6}{0.62}$$
$$= 2.6$$

EXAMPLE VI-2: Computing ϕ And Ω Factors From Test Data Using Section F1.1(b)

Given:

1. Resistance and safety factors for a special screw used within a limited range of diameters and sheet thickness values are desired. While AISI S100 provides an equation for calculating the pullout strength, this screw is assumed to have a significant difference in pull-out strength that makes it unique from typical screws currently used in the industry.

2. The desired limit state is screw pull-out. A nominal strength equation has been developed based on a rational engineering analysis by model and by tests.

Note: The S100 Section E4 design equations for screws were developed using a large database and thus are applicable for the spectrum of screw diameters and sheet thicknesses encountered in cold-formed steel construction.

Required:

1. Using Section F1.1(b), determine the resistance factor, ϕ, for the derived design equation for use with LRFD.

2. Using Section F1.2, determine the factor of safety, Ω, for the derived design equation for use with ASD.

Solution:

1. Develop and Calibrate a Rational Engineering Analysis Model for the Pull-Out Limit State.

The screw is produced in several nominal diameters, namely #8 (d = 0.16 in.), #10 (d = 0.19 in.), #12 (d = 0.21 in.), and #14 (d = 0.24 in.). The screw is intended to be used with steel thickness ranging from 0.0346 in. through 0.1013 in. For the purpose of this example, only ASTM A653 HSLAS Grade 40 (F_u = 50 ksi) material will be used for the cold formed steel parts being fastened. As such, two variable parameters are to be considered in the test program, namely t_c and d. Note that, since the specified value of F_{u2}, 50 ksi, is not being varied in the test program, the resistance and safety factors resulting from the analysis will be limited to this material in accordance with Section F1.1(b). The actual variance in F_{u2} in the tests is handled in the statistical analysis below.

The equation for screw pull-out strength, P_{not}, given in the *Specification* is:

$$P_{not} = 0.85 t_c d F_{u2} \qquad (\textit{Eq. } E4.4.1\text{-}1)$$

where

t_c = lesser of depth of penetration and thickness t_2

F_{u2} = Tensile strength of member not in contact with screw head or washer

d = nominal screw diameter

Assume that the new nominal strength equation takes a similar form, with a coefficient, C, and F_{u2} limited to a single value, 50 ksi:

$$P_{not} = C t_c d F_{u2} \qquad (\text{from } \textit{Eq. } E4.4.1\text{-}1)$$

where

C = coefficient to be determined through statistical analysis using the test data

F_{u2} = 50 ksi

Given the typically available thicknesses and the range of screw diameters noted above, there are 20 possible combinations that can be tested as shown in Table 1.

Table 1
Possible Combinations of Screw Diameter and Steel Thickness

Actual Material Thickness	Nominal Screw Diameter			
	#8	#10	#12	#14
0.0346"	1	2	3	4
0.0451"	5	6	7	8
0.0566"	9	10	11	12
0.0713"	13	14	15	16
0.1013"	17	18	19	20

Using Section F1.1(a), a minimum of 60 tests would need to be conducted (3 tests for each combination). However, conducting only 3 tests for each combination would yield conservative resistance or safety factors as a result of the correction factor, C_P, of 5.7 per AISI S100. Typically, 5 or more tests would need to be conducted to yield a more accurate set of safety and resistance factors, which would result in a total of 100 or more tests.

Using Section F1.1(b), a rational engineering analysis model (a proposed design equation for the nominal strength) is evaluated by test. Section F1.1(b) enables users to significantly reduce the required amount of testing while producing accurate resistance and safety factors if the correlation coefficient, C_c, is greater than 0.80, which indicates a good correlation between the proposed equation and the test data. If the correlation coefficient is less than 0.80 (indicating poor correlation), the user must either develop a new expression wherein C_c exceeds 0.80, or revert to Section F1.1(a) described above.

Section F1.1(b) requires that a minimum of 3 tests be conducted and for each parameter being evaluated all other parameters are to be held constant. This example includes two variable parameters, namely d and t_c. Further, Section F1.1(b) also stipulates that extrapolation outside the tested parameters is not permitted. Given these requirements, at least five tests would be required, three in which t_c is held constant and three in which d is held constant. The five required tests are highlighted in black in Table 1. However, increasing the number of tests will yield more accurate resistance and safety factors, and provided the correlation between the newly developed expression and the test data is good (i.e. $C_c > 0.8$), will result in a more accurate estimate of the nominal capacity of the screw. It is good practice to test all the extreme boundaries of the parameter ranges, further it is good practice to have coverage within the parametric range. These two practices will help ensure that the test data selected will not bias the results as required by AISI S100. Given these goals, supplementary data points are selected for testing and are highlighted in grey in Table 1. A total of 10 tests were selected. The results are shown in Table 2.

Section F1.1(b) stipulates that only one limit state may be considered at a time and the test results must reflect that limit state. As such, should another failure mode occur during testing, that test result shall not be used in the evaluation of new resistance and safety factors.

An optimization procedure, i.e. regression analysis, was used to determine the value of C of 0.98.

The calculated strength, R_n values for each test are calculated as:

$R_n = Ct_c dF_{u2}$, where C = 0.98

The individual values of C for each test, C_{test}, are calculated as:

$$C_{test} = \frac{R_t}{t_c d F_{u2}}$$

Table 2
Test Results and Analysis

t_c (in.)	d (in.)	F_{u2} (ksi)	Tested Strength, R_t (lbs)	Calculated Strength, R_n, (lbs)	C_{test}	R_t/R_n	R_tR_n	R_t^2	R_n^2
0.0349	0.165	55	299	310	0.94	0.96	92618	89295	96064
0.0575	0.165	58	533	539	0.97	0.99	287138	284320	289985
0.1018	0.165	57	920	937	0.96	0.98	861625	845684	877868
0.0701	0.185	56	765	711	1.05	1.08	544007	585918	505093
0.1018	0.185	57	1122	1051	1.05	1.07	1179019	1259612	1103582
0.0435	0.215	55	547	503	1.06	1.09	275117	298702	253394
0.0575	0.215	58	670	702	0.94	0.96	470423	449461	492362
0.0701	0.215	56	811	826	0.96	0.98	669568	657179	682189
0.0349	0.245	55	472	460	1.00	1.02	217079	222490	211799
0.1018	0.245	57	1353	1391	0.95	0.97	1882879	1831687	1935501
		Σ =	7492	7429			6479472	6524348	6447838
	Average =				0.99	1.01			

The following statistical parameters are calculated from the data above:

Average of R_t/R_n = 1.01
Standard Deviation of R_t/R_n = 0.050
Coefficient of Variation of R_t/R_n = 4.93%

Finally, the coefficient of correlation, C_c, can be calculated according to Section F1.1(c):

$$C_c = \frac{n\sum R_{t,i}R_{n,i} - \left(\sum R_{t,i}\right)\left(\sum R_{n,i}\right)}{\sqrt{n\left(\sum R_{t,i}^2\right) - \left(\sum R_{t,i}\right)^2}\sqrt{n\left(\sum R_{n,i}^2\right) - \left(\sum R_{n,i}\right)^2}} = 0.99 > 0.80$$

Since the coefficient of correlation is greater than 0.80, the new expression may be calibrated in accordance with Section F1.1(b). In determining C and the associated resistance and safety factors, the as-measured screw diameter, d and steel thickness, t_c, must be used. The as-measured tensile strength of the base steel, F_{u2}, must also be used in the evaluation of C and the resistance and safety factors.

To calibrate the newly developed expression, a similar process to Section F1.1(a) is followed. In this case, the coefficient of variation, V_P is calculated as 4.93%, but Chapter F requires that $V_P \geq 6.5\%$. Therefore, use $V_P = 0.065$.

Using *Eq*. F1.1-2 , the resistance factor is determined as follows:

$$\phi = C_\phi\left(M_m F_m P_m\right)e^{-\beta_o\sqrt{V_M{}^2 + V_F{}^2 + C_P V_P{}^2 + V_Q{}^2}} \quad (Eq.\ F1.1\text{-}2)$$

where

C_ϕ = 1.52 (for LRFD)
n = number of tests = 10
m = n-1 = 10 -1 = 9
$C_p = (1+1/n)m/(m-2)$ for $n \geq 4$ (*Eq*. F1.1-4)
$= (1+1/10)(9)/(9-2) = 1.41$

β_o = 3.5 for connections for LRFD, P_m = Average of R_t/R_n = 1.01

The remaining parameters are listed in Table F1 and are a function of the type of component. For pull-out of screw connections the parameters from Table F1 are:

M_m = 1.10, V_M = 0.10, F_m = 1.00, V_F = 0.10, V_Q = 0.21

$$\phi = 1.52[1.10(1.00)(1.01)]e^{-3.5\sqrt{0.10^2+0.10^2+1.41(0.065)^2+0.21^2}} = 0.67 \qquad (Eq.\ F1.1\text{-}2)$$

2. Compute Factor of Safety, Ω, for ASD

For Allowable Strength Design, the factor of safety is determined by using *Eq*. F1.2-2 as follows:

$$\Omega = \frac{1.6}{\phi} = \frac{1.6}{0.67} = 2.39 \qquad (Eq.\ F1.2\text{-}2)$$

The resulting design expression is therefore:

$$P_{not} = 0.98t_c d(50)$$

and the associated resistance and safety factors for LRFD and ASD, respectively, are:

ϕ = 0.67, Ω = 2.39

If the mean value of F_{u2} measured in the tests was less than the specified F_{u2}, the rupture stress in the new design expression would be limited to the mean value of the as-measured F_{u2}. As an example, if the mean tensile strength of the base steel was measured in the tests as 46 ksi for an ASTM A653 HSLAS Grade 40 steel, 46 ksi would be used as the rupture stress, resulting in the design expression:

$$P_{not} = 0.98t_c d(46)$$

However, if the mean of the tensile strength measured in the tests was greater than the specified strength, the specified tensile strength of 50 ksi would be used as the rupture stress in the calculation of the nominal pullout strength. Likewise, the specified thickness of the steel and the nominal diameter of the screw must always be used when calculating the available pullout strength of the screw.

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