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AISI Manual Cold-Formed Steel Design 2002 Edition

American Iron and Steel Institute

AISI MANUAL

Cold-Formed Steel Design

2002 Edition

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SECTION GUIDE

American
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AISI MANUAL

Cold-Formed Steel Design Manual

2002 EDITION

The material contained herein has been developed by the American Iron and Steel Institute 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.

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

PREFACE

The 2002 edition of the *Cold-Formed Steel Design Manual* consists of six Parts. This information is supplemental to the 2001 edition of the *North American Specification for the Design of Cold -Formed Steel Structural Members*. Each part in the *Design Manual* should be used in conjunction with the *Specification, Commentary* and the other parts, where appropriate.

Part I, Dimensions and Properties contains (a) information regarding the availability and properties of steels referenced in the *Specification*, (b) tables of section properties (c) formulas and examples of calculations of 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, Connections contains (a) tables to aid in connection design, and (b) connection example problems.

Part V, Supplementary Information contains (a) 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 (b) other information intended to assist users of cold-formed steel.

Part VI, Test Procedures contains (a) test methods for cold-formed steel, (b) a bibliography of other pertinent test methods, and (c) an example problem.

In addition to updating the *Manual* for conformance with the 2001 edition of the *North American Specification*, the following improvements or additions have been made:

- Standard studs and tracks produced by members of Steel Stud Manufacturers Association are included in Parts I through IV.
- Four new comprehensive design examples are added,
  (a) C-Section with Openings – ASD and LRFD in Part II,
  (b) Unbraced Equal Leg Angle With Lips – Compression in Part III,
  (c) I-Section – Built-Up from Channels in Part III, and
  (d) Bolted Connection with Consideration of Shear Lag in Part IV.
- A table of cross references between the *Specification* provisions and the corresponding illustrative examples is provided in Part V.
- Numerical designations have been added to the titles of test procedures in Part VI.
- The following three new test procedures are included in Part VI:
  (a) AISI TS-4-02, Standard Test Methods for Determining the Tensile and Shear Strength of Screws,
  (b) AISI TS-6-02, Standard Procedures for Panel and Anchor Structural Tests and Commentary on the Standard Procedures,

(c) AISI TS-8-02, Base Test Method for Purlins Supporting a Standing Seam Roof System.

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

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

Product Classifications

Of the referenced steels, 5 are for plate and bar, 1 is for plate, 2 are for sheet and strip, 5 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:

Product Classification - Hot-Rolled Steel

Width, w in.	Thickness, t in.			
	0.2300 ≤ t	0.2031 ≤ t ≤ 0.2299	0.1800 ≤ t ≤ 0.2030	0.0470 ≤ t ≤ 0.1799
w ≤ 3-1/2	bar	bar	strip	strip
3-1/2 < w ≤ 6	bar	bar	strip	strip
6 < w ≤ 8	bar	strip	strip	strip
8 < w ≤ 12	plate (1)	strip	strip	strip
12 < w ≤ 48	plate (2)	sheet	sheet	sheet
48 < w	plate (3)	plate (3)	plate (3)	sheet

(1) Strip, only when ordered in coils.
(2) Sheet, only when ordered in coils.
(3) Sheet, only when ordered in coils.
Maximum width 74 inches.

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

Product Classification - Cold-Rolled Sheet Steel

Width, w in.	Thickness, t in.	
	Carbon Steel	HSLA steel
2 ≤ w ≤ 12 12 < w	0.014 ≤ t ≤ 0.082 0.014 ≤ t	0.019 ≤ t ≤ 0.082 0.020 ≤ t

Structural Properties

The structural properties significant to cold-formed steel structures are listed in the table of referenced steels, and include yield point, tensile strength, elongation in 2 inches, and the ratio of tensile strength to yield point. 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 point is an indication of the ability of the material to redistribute stress.

1.2 Summary Of Scope And Principle Tensile Properties, ASTM Specifications For Referenced Steels

Table 1.2

Summary Of Scope And Principle 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-00a 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 where improved notch toughness is important. These shall apply only when specified by the purchaser in the order. When the steel is to be welded, a welding procedure suitable for the grade of steel and intended use or service will be utilized.	Plates and Bars	--	36	58 / 80	23	1.61
A242/A242M-00a 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 will be utilized.	Plates and Bars $t \le 3/4$ in.	--	50	70	21	1.40

Table 1.2 (continued)

Summary Of Scope And Principle 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-00 This specification covers four 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 will be utilized.	Plate	A B C D	24 27 30 33	45 / 60 50 / 65 55 / 75 60 / 80	30 28 25 23	1.88 1.85 1.83 1.82
A500-99 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 maximum periphery of 64 in. [1626 mm] and a maximum wall of 0.625 in. [15.88 mm]. Grade D requires heat treatments. *Note:* 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-00 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 12 in. [305 mm] wide, bars to 3 1/2 in. [90 mm] thick, and Group 1 and 2 shapes; Grade 55 for plates to 1 in. [25.4 mm] thick and to 12 in. [305 mm] wide, bars to 3 in. [75 mm] thick, and Group 1 and 2 shapes. 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 will be utilized.	Plates and Bars	50 55	50 55	70 / 100 70 / 100	21 20	1.40 1.27

Table 1.2 (continued)

Summary Of Scope And Principle 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-00a 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 will 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-00a 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 will be utilized.	Plates and Bars $t \le 4$ in.	--	50	70	21	1.40

Table 1.2 (continued)

Summary Of Scope And Principle 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)	$\frac{F_{u(min)}}{F_{y(min)}}$
A606-98 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	50	70	22	1.40
		Hot Rolled -Annealed or Normalized	45	65	22	1.44
		Cold Rolled	45	65	22	1.44
A653/A653M-00 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 strength in both structural steel (SS) and high-strength low-alloy (HSLAS). HSLAS sheets are available as Type A or Type B. HSLAS Type A is intended for applications where improved formability is required compared to SS. HSLAS Type B is intended for applications where improved formability is required compared to HSLAS Type A. 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 I	50	65	12	1.30
		50 Class 3	50	70	12	1.40
		80	80	82	-	1.03
		HSLAS Type A				
		40	40	50	22	1.25
		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 Type B				
		40	40	50	24	1.25
		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 1.2 (continued)

Summary Of Scope And Principle 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)	$\frac{F_{u(min)}}{F_{y(min)}}$
A792/A792M-99 This specification covers 55% aluminum-zinc alloy-coated steel sheet in coils and cut lengths coated by the hot-dip process. The aluminum-zinc alloy composition by weight is nominally 55% aluminum, 1.6% silicon, and the balance zinc. The product is intended for applications requiring corrosion resistance or heat resistance or both. Aluminum-zinc alloy-coated sheet is available as Commercial Steel (CS), Forming Steel (FS), Drawing Steel (DS), High Temperature Steel (HTS), and Structural Steel (SS).	Sheet	SS 33 37 40 50 Class 1 80	 33 37 40 50 80	 45 52 55 65 82	 20 18 16 12 -	 1.36 1.41 1.38 1.30 1.03
A847-99a 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 the 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. [15.88 mm], and in seamless with a maximum periphery of 32 in. [813 mm] and a maximum wall of 0.500 in. [12.70 mm].	Round and Shaped Tubing	-	50	70	19	1.40

Table 1.2 (continued)

Summary Of Scope And Principle 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)	$F_{u(min)}$ / $F_{y(min)}$
A875/A875M-99 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	--	--
		HSLAS Type A				
		50	50	60	20	1.20
		60	60	70	16	1.17
		70	70	80	12	1.14
		80	80	90	10	1.12
		HSLAS Type B				
		50	50	60	22	1.20
		60	60	70	18	1.17
		70	70	80	14	1.14
		80	80	90	12	1.12
A1003/A1003M-00 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.	Sheet	ST33H	33	-	10	1.08*
		ST37H	37	-	10	1.08*
		ST40H	40	-	10	1.08*
		ST50H	50	-	10	1.08*
		ST33L	33	-	3	-
		ST37L	37	-	3	-
		ST40L	40	-	3	-
		ST50L	50	-	3	-
* $(F_u/F_y)_{minimum}$						

Table 1.2 (continued)

Summary Of Scope And Principle 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-00 This specification covers cold-rolled structural, high-strength low-alloy, and high-strength low-alloy with improved formability steel sheet, in coils and cut lengths. The product is produced in a number of designations including structural steel (SS), high-strength low-alloy steel (HSLAS), and high-strength low-alloy steel with improved formability (HSLAS-F). 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 may 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
		80	80	82	--	--
		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	1.12

Table 1.2 (continued)

Summary Of Scope And Principle 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)	$F_{u(min)}$ / $F_{y(min)}$
A1011/A1011M-00 This specification covers hot-rolled structural, high-strength low-alloy, and high-strength low-alloy with improved formability 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), and high-strength low-alloy steel with improved formability (HSLAS-F). 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 may be treated to achieve inclusion control.	Sheet	SS				
		30	30	49	25-21*	1.63
		33	33	52	23-18*	1.62
		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	45	60	19-13*	1.33
		50	50	65	17-11*	1.30
		55	55	70	15-9*	1.27
		HSLAS Class 1				
		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
* Specified value varies with thickness range.		80	80	90	18-16*	1.12

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.

Two different naming conventions are used throughout the *Manual*. Standard stud and track sections are identified using the SSMA naming convention. Sections other than studs and tracks are identified by a convention developed by AISI for use in this manual.

2.1.1 SSMA Stud and Track Section Nomenclature

Standard studs and tracks produced by members of the Steel Stud Manufacturers Association (SSMA) are identified in this *Manual* by the SSMA identification code. The codes are formed by concatenating the following information:

1. Depth in 1/100th inches. For 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 =Stud (C-Section with Lips), T = Track (C-Section without Lips)
3. Flange Width in 1/100th inches
4. "-"
5. Minimum base material thickness (95% of design thickness) in 1/1000th inches

For example, a section with the designation 600S162-54 is a stud (C-section with lips), with a depth of 6 inches, a flange width of 1 5/8 inches and a minimum thickness of 0.054 inches. 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.

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 inches

For example, a section with the designation 9CS3x075 is a C-Section with stiffened lips, with a depth of 9 inches, a flange width of 3 inches and a thickness of 0.075 inches. 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 section.

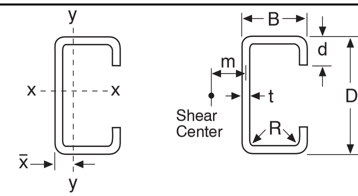
(f) Section dimensions are defined in the figures provided in each table. Section properties are defined in the *Specification*, Symbols and Definitions.

2.3 Gross Section Property Tables

Table I - 1

Gross Section Properties

C-Sections With Lips

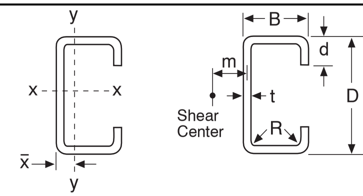


ID	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
12CS4x105	12.000	4.000	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 (continued)

Gross Section Properties

C-Sections With Lips

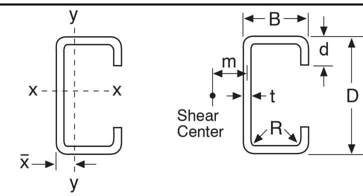


ID	Dimensions							Properties of Full Section													
								Axis x-x			Axis y-y										
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
9CS2.5x105	9.000	2.500	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 (continued)

Gross Section Properties

C-Sections With Lips



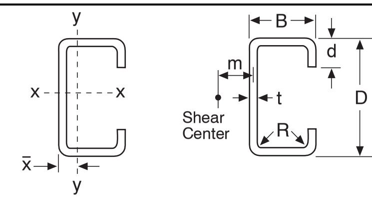
ID	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
7CS2.5x105	7.000	2.500	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

SSMA 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	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.
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

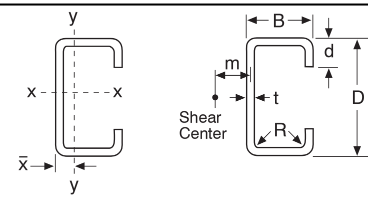
* h/t of web > 200; stiffeners are required.

Table I - 2 (continued)

Gross Section Properties

SSMA 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	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

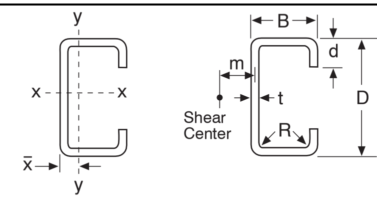
* h/t of web > 200; stiffeners are required.

Table I - 2 (continued)

Gross Section Properties

SSMA Studs

C-Sections With Lips



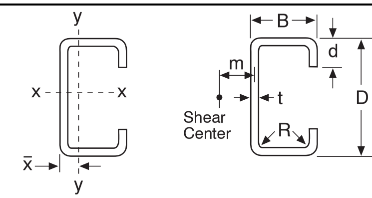
ID	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	x	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
600S162-97	6.000	1.625	0.1017	0.500	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
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

Table I - 2 (continued)

Gross Section Properties

SSMA Studs

C-Sections With Lips



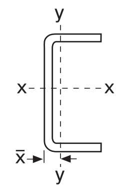
ID	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
362S200-68	3.625	2.000	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
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

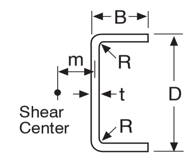
Table I - 3

Gross Section Properties

SSMA Tracks

C–Sections Without Lips





ID	Dimensions						Properties of Full Sections												
							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

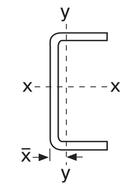
* h/t of web > 200; stiffeners are required.

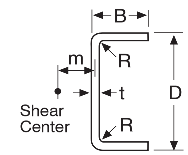
Table I - 3 (continued)

Gross Section Properties

SSMA Tracks

C–Sections Without Lips





ID	Dimensions						Properties of Full Sections												
							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.
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
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

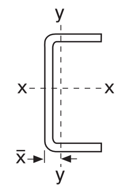
* h/t of web > 200; stiffeners are required.

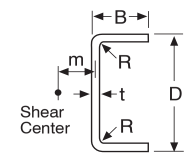
Table I - 3 (continued)

Gross Section Properties

SSMA Tracks

C–Sections Without Lips





ID	Dimensions						Properties of Full Sections													
							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.
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
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

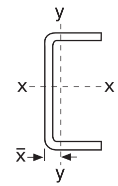
* h/t of web > 200; stiffeners are required.

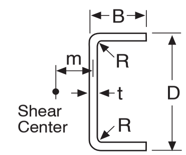
Table I - 3 (continued)

Gross Section Properties

SSMA Tracks

C–Sections Without Lips





ID	Dimensions						Properties of Full Sections													
							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.
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
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

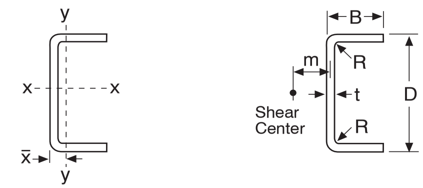
* h/t of web > 200; stiffeners are required.

Table I - 3 (continued)

Gross Section Properties

SSMA Tracks

C–Sections Without Lips



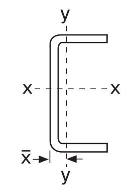
ID	Dimensions						Properties of Full Sections												
							Axis x-x			Axis y-y									
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
362T200-68	3.875	2.000	0.0713	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
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

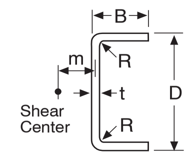
Table I - 3 (continued)

Gross Section Properties

SSMA Tracks

C–Sections Without Lips



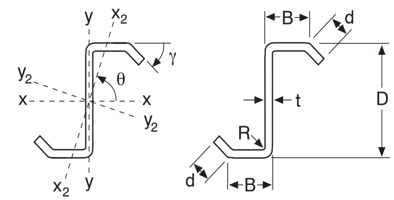


ID	Dimensions						Properties of Full Sections													
							Axis x-x			Axis y-y										
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
350T125-68	3.750	1.250	0.0713	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
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

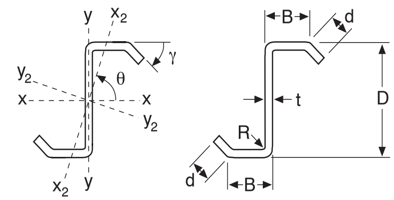


ID	Dimensions								Properties of Full Section												
									Axis x-x			Axis y-y									
	D	B	t	d	γ	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	I_{xy}	I_{x2}	I_{y2}	r_{min}	θ	J	C_w
	in.	in.	in.	in.	deg	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{4}	in.^{4}	in.^{4}	in.	deg	in.^{4}	in.^{6}
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 (continued)

Gross Section Properties

Z-Sections With Lips

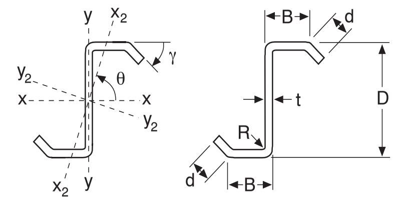


ID	Dimensions								Properties of Full Section													
	D	B	t	d	γ	R	Area	wt/ft	Axis x-x			Axis y-y			I_{xy}	I_{x2}	I_{y2}	r_{min}	θ	J	C_w
									I_x	S_x	r_x	I_y	S_y	r_y							
	in.	in.	in.	in.	deg	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{4}	in.^{4}	in.^{4}	in.	deg	in.^{4}	in.^{6}
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 (continued)

Gross Section Properties

Z-Sections With Lips

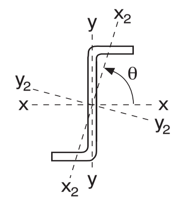


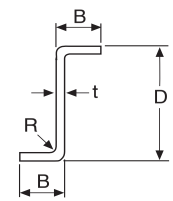
ID	Dimensions								Properties of Full Section													
									Axis x-x			Axis y-y										
	D	B	t	d	γ	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	I_{xy}	I_{x2}	I_{y2}	r_{min}	θ	J	C_w
	in.	in.	in.	in.	deg	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{4}	in.^{4}	in.^{4}	in.	deg	in.^{4}	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



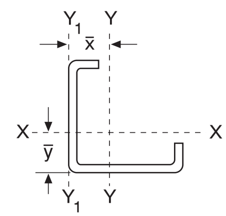


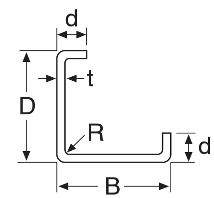
ID	Dimensions						Properties of Full Section												
	D	B	t	R	Area	wt/ft	Axis x-x			Axis y-y			I_{xy}	I_{x2}	I_{y2}	r_{min}	θ	J	C_w
							I_x	S_x	r_x	I_y	S_y	r_y							
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{4}	in.^{4}	in.^{4}	in.	deg	in.^{4}	in.^{6}
8ZU1.25x105	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

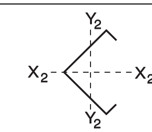
Table I - 6

Gross Section Properties

Equal Leg Angles With Lips





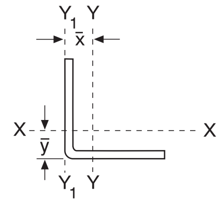


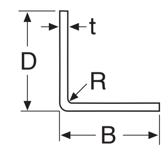
ID	Size		t	d	R	Area	wt/ft	Properties of Full Section												
								Axis x-x and y-y				r_{y1}	J	C_w	r_o					
	D	B						I_x	S_x	r	$\bar{x} = \bar{y}$					I_{y2}	r_{y2}	j	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.^{4}	in.	in.	in.
4LS4x135	4.000	4.000	0.135	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

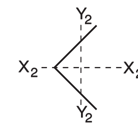
Table I - 7

Gross Section Properties

Equal Leg Angles Without Lips





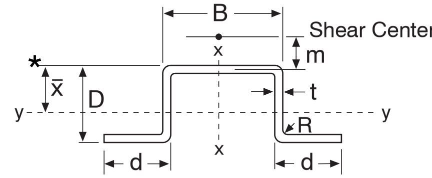


ID	Size		t	R	Area	wt/ft	Properties of Full Section												
							Axis x-x and Axis y-y				r_{y1}	J	C_w	r_o				
	D	B					I_x	S_x	r	$\bar{x} = \bar{y}$					I_{y2}	r_{y2}	j	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.^{4}	in.	in.	in.
4LU4x135	4.000	4.000	0.135	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	$\bar{x}$	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.
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 at times used for architectural application; however, they are fundamentally structural products. As such, the structural capabilities (strength and stiffness) are determined using the *Specification*, which also provides base steel specifications. Design may use either Allowable Strength Design (ASD) or Load and Resistance Factor Design (LRFD) techniques. The usual deck products are roof deck, form deck and composite floor deck. In addition to supporting gravity and wind uplift loads, steel deck profiles can also serve to resist in-plane loading i.e. diaphragm loads. The most common types of steel deck are discussed in Section 2.4.1.

One of the important items 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 thought 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 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.

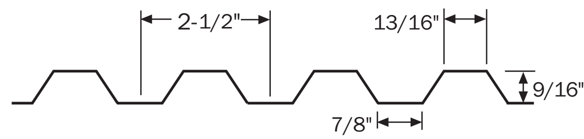
(b) Narrow Rib Deck (NR): Narrow rib deck is used as roof decking in climatic areas where minimal roof insulation is required. The narrow rib provides a small span that thin rigid insulation can bridge. Narrow rib deck is the least efficient structurally but has the advantage of easy attachment of roofing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

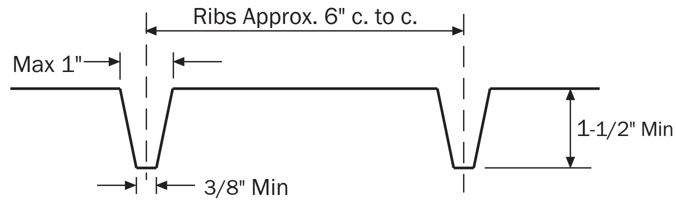
(d) Wide Rib Deck (WR): Wide rib deck is used as roof deck in very hot or cold climates, where the thick insulation used can span the wide flutes. It is the most structurally efficient cross section of the roof deck profiles.

(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.

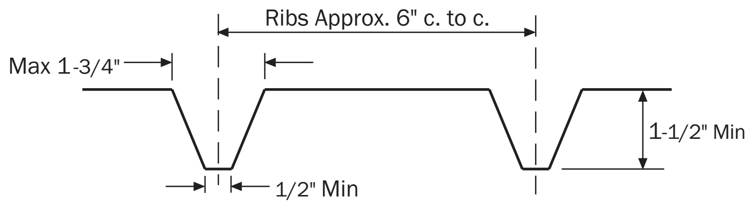
In addition to these industry standard profiles, manufacturers produce proprietary steel deck profiles that provide special functional enhancements, such as acoustical absorption capability and cellular raceway features.



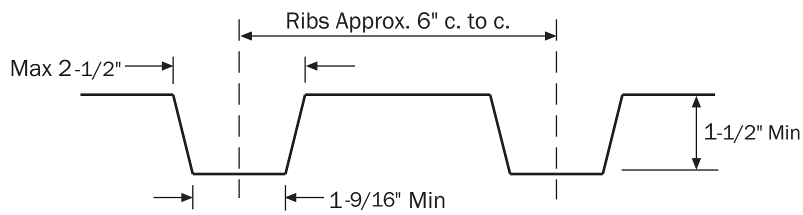
(a) 9/16" Form Deck (Representative)



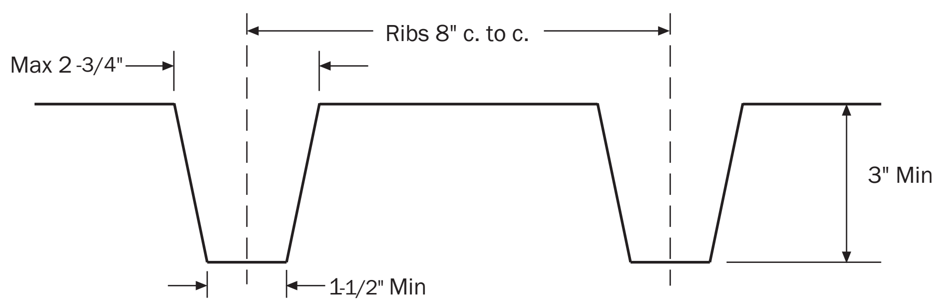
(b) Narrow Rib Deck Type NR



(c) Intermediate Rib Deck Type IR



(d) Wide Rib Deck Type WR



(e) Deep Rib Deck Type 3DR

Figure 2.4-1 Deck Profiles

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 will vary.

2.4.2 Maximum Spans

Recommended Maximum Spans for Construction and Maintenance Loads

Type*		Span Condition	Maximum Recommended Spans	
			Span ft-in.	Roof Deck Cantilever ft-in.
Narrow Rib Deck	NR22	1	3′-10″	
	NR22	2 or more	4′-9″	1′-0″
	NR20	1	4′-10″	
	NR20	2 or more	5′-11″	1′-2″
	NR18	1	5′-11″	
	NR18	2 or more	6′-11″	1′-7″
Intermediate Rib Deck	IR22	1	4′-6″	
	IR22	2 or more	5′-6″	1′-2″
	IR20	1	5′-3″	
	IR20	2 or more	6′-3″	1′-5″
	IR18	1	6′-2″	
	IR18	2 or more	7′-4″	1′-10″
Wide Rib Deck	WR22	1	5′-6″	
	WR22	2 or more	6′-6″	1′-11″
	WR20	1	6′-3″	
	WR20	2 or more	7′-5″	2′-4″
	WR18	1	7′-6″	
	WR18	2 or more	8′-10″	2′-10″
Deep Rib Deck	3DR22	1	11′-0″	
	3DR22	2 or more	13′-0″	3′-5″
	3DR20	1	12′-6″	
	3DR20	2 or more	14′-8″	3′-11″
	3DR18	1	15′-0″	
	3DR18	2 or more	17′-8″	4′-9″

* Deck section properties are provided in Section 2.4.3

Construction and maintenance loads

Spans are governed by a maximum stress of 26 ksi and a maximum deflection of L/240 with a 200 pound concentrated load at midspan on a 12 in. wide section of deck.

If the designer contemplates loads of greater magnitude, spans shall be decreased or the thickness of the steel deck increased as required.

All loads shall be distributed by appropriate means to prevent damage to the completed assembly during construction.

Do not walk or stand on deck until it is fastened in accordance with the Steel Deck Institute Design Manual, Publication No. 30.

Cantilever loads

Construction phase load of 10 psf on adjacent span and cantilever plus 200 pound load at end of cantilever with a stress limit of 26 ksi for ASD.

Service load of 45 psf on adjacent span and cantilever plus 100 pound load at end of cantilever with a stress limit of 20 ksi.

Deflection limited to 1/240 of adjacent span for interior span and deflection at end of cantilever to 1/120 of overhang.

Notes:

1. Adjacent span: Limited to those spans shown in Section 3.4 of the SDI Roof Deck Specifications. In those instances where the adjacent span is less than 3 times the cantilever span, the individual manufacturer should be consulted for the appropriate cantilever span.
2. Sidelaps must be attached at the end of the cantilever and at a maximum of 12 inches on center from the end.
3. No permanent suspended loads are to be supported by the steel deck.
4. The deck must be completely attached to the supports and at the sidelaps before any load is applied to the cantilever.

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 tables below, the I and S_t values given are for compression on the top; The S_b values given are for compression on the bottom. The weight provided is for dead load calculations; it should not be used as a basis for ordering. The values given are based on steel with a yield strength 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/ft^2		I in.^{4}/ft	S_t in.^{3}/ft	S_b in.^{3}/ft
			Painted	Galvanized			
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
1-1/2″ Narrow Rib Deck	NR22	0.0295	1.7	1.8	0.099	0.089	0.098
	NR20	0.0358	2.0	2.1	0.128	0.111	0.118
	NR18	0.0474	2.7	2.8	0.181	0.152	0.157
1-1/2″ Intermediate Rib Deck	IR22	0.0295	1.6	1.7	0.108	0.102	0.110
	IR20	0.0358	1.9	2.0	0.139	0.127	0.134
	IR18	0.0474	2.6	2.7	0.196	0.173	0.177
1-1/2″ Wide Rib Deck	WR22	0.0295	1.7	1.8	0.152	0.182	0.184
	WR20	0.0358	2.0	2.1	0.198	0.226	0.237
	WR18	0.0474	2.7	2.8	0.284	0.307	0.316
3″ Deep Rib Deck	3DR22	0.0295	1.9	2.0	0.551	0.321	0.369
	3DR20	0.0358	2.3	2.4	0.714	0.400	0.449
	3DR18	0.0474	3.1	3.2	1.036	0.550	0.594
	3DR16	0.0598	3.9	4.0	1.399	0.714	0.748

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: Area = L x t, 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′ x 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′ x 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

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

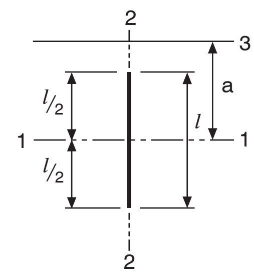
3.2 Properties of Line Elements

3.2.1 Straight Line Elements

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

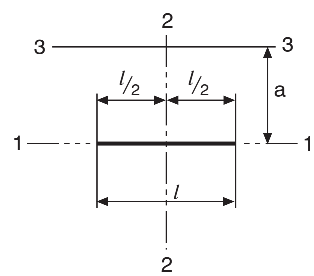
$$I_1 = \frac{l^3}{12}, \qquad I_2 = 0$$

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



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

$$I_3 = la^2$$

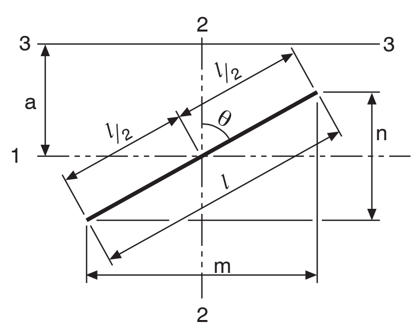


$$I_1 = \left[\frac{\cos^2\theta}{12}\right]l^3 = \frac{ln^2}{12}$$

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

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

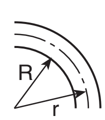
$$I_3 = la^2 + \frac{ln^2}{12} = l\left(a^2 + \frac{n^2}{12}\right)$$



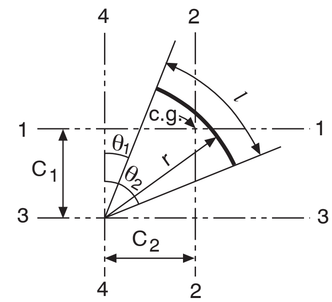
3.2.2 Circular Line Elements

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

R = inside radius
r = median radius
θ (expressed in radians) = 0.01745 θ



$= \dfrac{\pi\theta}{180}$ (expressed in degrees and decimals thereof)



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

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

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

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

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

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

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

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

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

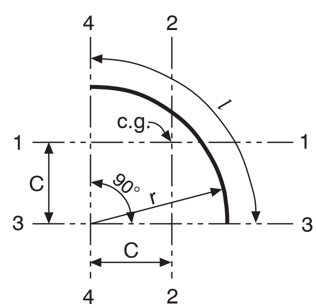
$c = 0.637\ r$

$I_1 = I_2 = 0.149\ r^3$

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

$I_3 = I_4 = 0.785\ r^3$

$I_{34} = 0.5\ r^3$

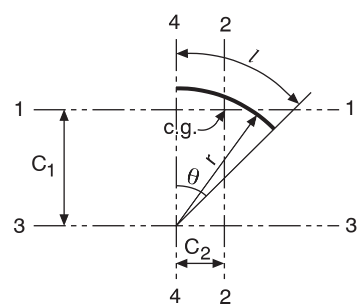


Case II: $\theta_1 = 0$; $\theta_2 = \theta$

$l = \theta r$

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

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



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

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

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

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

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
   $a = A' - [r + t/2 + \alpha(r + t/2)]$*
   $\bar{a} = A' - [t/2 + \alpha t/2]$
   $c = \alpha[C' - (r + t/2)]$
   $\bar{c} = \alpha[C' - t/2]$
   $u = \pi r/2 = 1.57r$

2. Cross-Sectional area
   $A = t[2a + u + \alpha(2c + 2u)]$

3. Distance between centroid and centerlines of webs

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

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

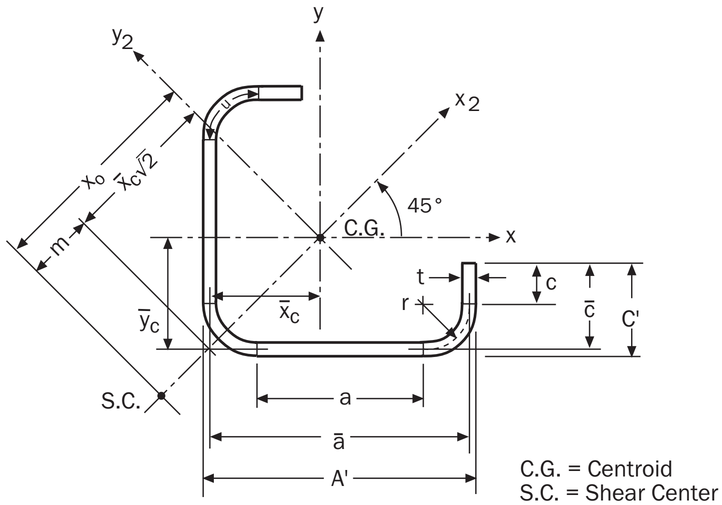


Figure 3.3.1-1
Equal Leg Angle (Singly Symmetric) With Lips

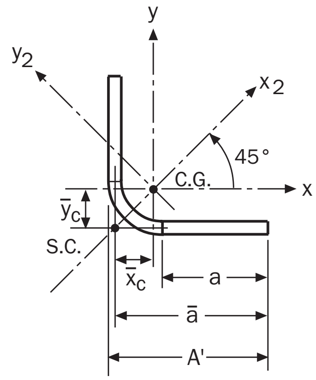


Figure 3.3.1-2
Equal Leg Angle (Singly Symmetric)
Without Lips

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

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

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

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

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

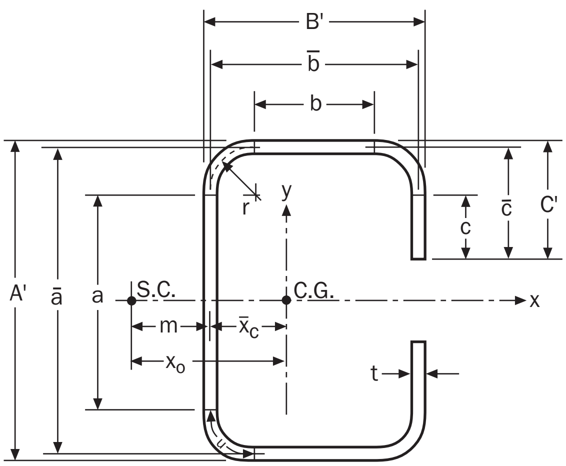


Figure 3.3.2-1
C-Section (Singly Symmetric) With Lips

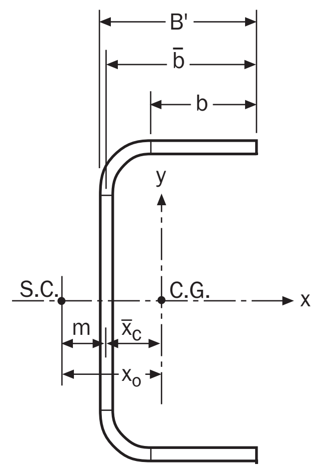
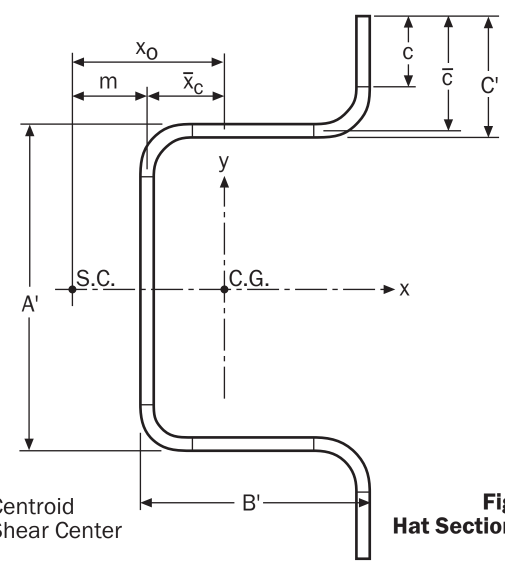


Figure 3.3.2-2
C-Section (Singly Symmetric) Without Lips



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

Figure 3.3.2-3
Hat Section (Singly Symmetric)

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

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

3. Moment of inertia about x-axis
   Channel:

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

   Hat Section:

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

4. Distance between centroid and web centerline

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

5. Distance between centroid and outside of web

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

6. Moment of inertia about y-axis

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

7. Distance between shear center and web centerline

   a) Channel:

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

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

c) Hat Section:

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

8. Distance between centroid and shear center
$x_o = -(\bar{x}_c + m)$*

9. St. Venant torsion constant

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

10. Warping constant
a) Channel:

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

b) Hat section:

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

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

b) Hat section: $\beta_l = 2\bar{c}t(\bar{b} - \bar{x}_c)^3 + \frac{2}{3}t(\bar{b} - \bar{x}_c)\,[(\bar{a}/2 + \bar{c})^3 - (\bar{a}/2)^3]$

14. Parameter used in determination of elastic critical moment

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

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

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

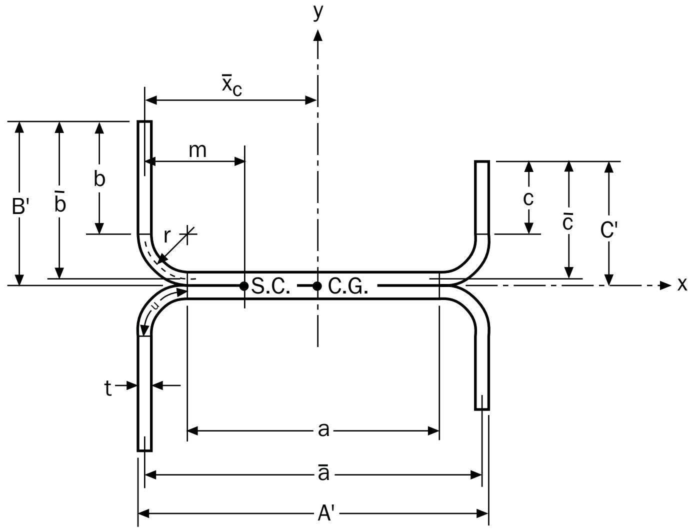


Figure 3.3.3-1
I-Section With Unequal Flanges (Singly Symmetric)

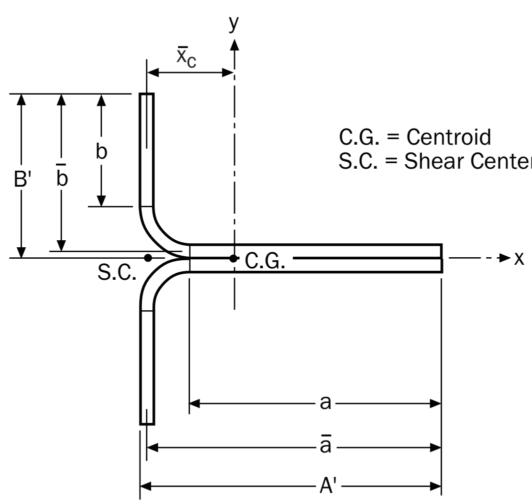


Figure 3.3.3-2
T-Section (Singly Symmetric)

1. Basic parameters

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

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

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

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

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

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

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

2. Cross-sectional area

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

* For I-Sections, $\alpha = 1.0$; for T-Sections, $\alpha = 0$.

3. Moment of inertia about x-axis

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

4. Distance between centroid and longer flange centerline

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

5. Distance between centroid and outside of longer flange

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

6. Moment of inertia about y-axis

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

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

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

9. St. Venant torsion constant

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

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:

Unlipped I-Sections and T-Sections:

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

For double symmetric, lipped I-Sections:

$\bar{c}$ = length of lip, see Figure 3.3.3-1

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

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

11. Parameter used in determination of elastic critical moment

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

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

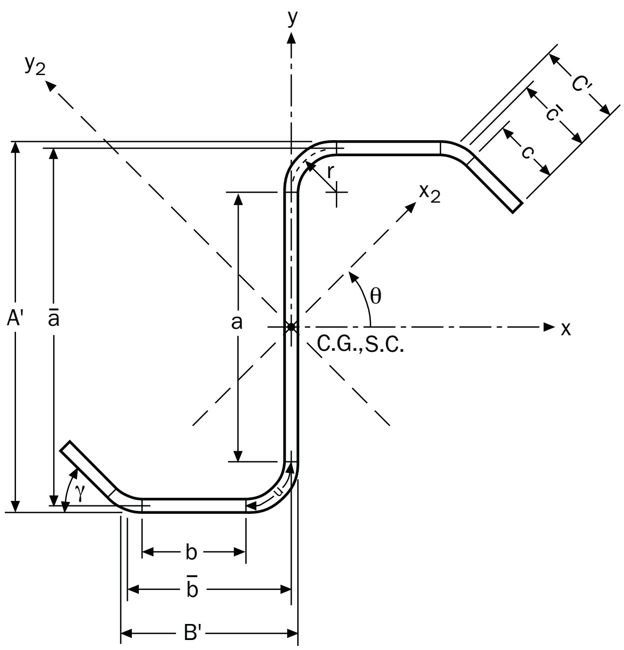


Figure 3.3.4-1
Z-Section (Point Symmetric) With Lips

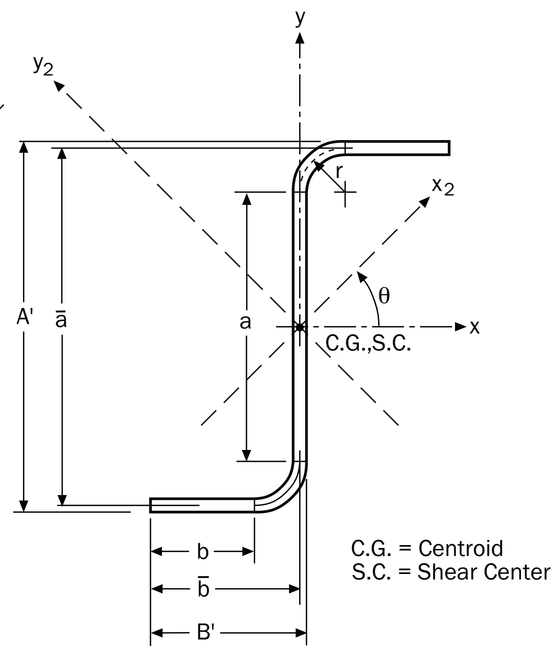


Figure 3.3.4-2
Z-Section (Point Symmetric) Without Lips

1. Basic Parameters
   a $= A' - (2r + t)$
   $\bar{a}$ $= A' - t$
   b $= B' - [r + t/2 + \alpha(r + t/2)\tan(\gamma/2)]$*
   $\bar{b}$ $= B' - [t/2 + (\alpha t/2)\tan(\gamma/2)]$
   c $= \alpha[C' - (r + t/2)\tan(\gamma/2)]$
   $\bar{c}$ $= \alpha[C' - (t/2)\tan(\gamma/2)]$
   u_1 $= \pi r/2 = 1.57r$
   u_2 $= \gamma r$ where γ is in radians

2. Cross-sectional area
   $A = t[a + 2b + 2u_1 + \alpha(2c + 2u_2)]$

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

3. Moment of inertia about x-axis

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

4. Moment of inertia about y-axis

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

5. Product of inertia (See note below)

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

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

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

12. Warping constant

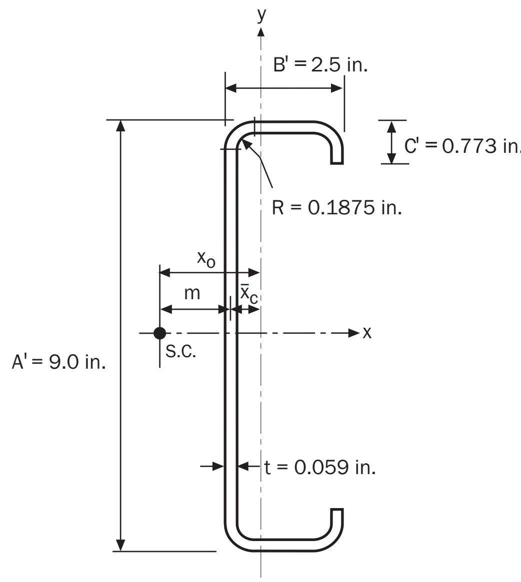
$$C_w = \frac{t}{12}\left\{\frac{\bar{a}^2\bar{b}^3(2\bar{a} + \bar{b}) + \alpha\left[\begin{array}{l}\bar{b}^2\left(4\bar{c}^4 + 16\bar{b}\bar{c}^3 + 6\bar{a}^3\bar{c} + 4\bar{a}^2\bar{b}\bar{c} + 8\bar{a}\bar{c}^3\right)\\ + 6\bar{a}\bar{b}\bar{c}^2\left(\bar{a} + \bar{b}\right)\left(2\bar{b}\sin\gamma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

3.4 Gross Section Properties - Example Problems

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

$r = R + t/2$

$= 0.1875 + 0.059/2 = 0.217$ in.

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

$= 9.000 - [(2)(0.217) + 0.059] = 8.507$ in.

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

$= 9.000 - 0.059 = 8.941$ in.

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

$= 2.500 - [0.217 + 0.059/2 + 1.0(0.217 + 0.059/2)] = 2.007$ in.

$\bar{b} = B' - [t/2 + \alpha t/2]$

$= 2.500 - [0.059/2 + (1.0)(0.059/2)] = 2.441$ in.

$$c = \alpha[C' - (r + t/2)]$$
$$= 1.0[0.773 - (0.217 + 0.059/2)] = 0.527 \text{ in.}$$
$$\bar{c} = \alpha[C' - (t/2)]$$
$$= 1.0[0.773 - (0.059/2)] = 0.744 \text{ in.}$$
$$u = \pi r/2$$
$$= \pi(0.217)/2 = 0.341 \text{ in.}$$

b. Cross-section area

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

c. Moment of inertia about the x-axis

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

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

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

d. Distance between centroid and web centerline

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

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

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

e. Moment of inertia about the y-axis

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

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

$$= 0.118\,\{2.990 + 0.674 + 0.0036 + 1.0\,[3.140 + 1.903 + 0.0015]\} - 0.330$$

$$= 0.698 \text{ in.}^4$$

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

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

$$= -(0.612 + 1.048)$$

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

2. Torsional properties

a. St. Venant torsional constant

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

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

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

b. Warping constant

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

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

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

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

c. Parameter used in determination of elastic critical moment

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

$$= -\left[\frac{(0.059)(0.612)(8.941)^3}{12} + (0.059)(0.612)^3(8.941)\right]$$

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

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

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

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

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

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

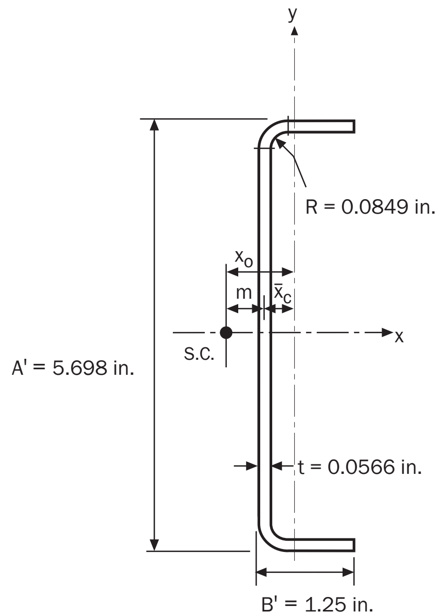
$$= 3.242 \text{ in.}^5$$

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

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

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

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



Given:

1. Section: SSMA 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.

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

$= 5.698 - [(2)(0.113) + 0.0566] = 5.415$ in.

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

$= 5.698 - 0.0566 = 5.641$ in.

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

$= 1.250 - [0.113 + 0.0566/2 + 0.0] = 1.109$ in.

$\bar{b} = B' - [t/2 + \alpha t/2]$

$= 1.250 - [0.0566/2 + 0.0] = 1.222$ in.

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

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

$u = \pi r/2$

$= \pi(0.113)/2 = 0.177$ in.

b. Cross-section area

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

$= 0.0566[5.415 + (2)(1.109) + (2)(0.177) + 0.0]$

$= 0.452$ in.^{2}

c. Moment of inertia about the x-axis

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

$$= (2)(0.0566)\left\{\begin{array}{l} 0.0417(5.415)^3 + 1.109(5.415/2 + 0.113)^2 \\ + 0.177[5.415/2 + 0.637(0.113)]^2 + 0.149(0.113)^3 + 0.0 \end{array}\right\}$$

$= 1.90$ in.^{4}

d. Distance between centroid and web centerline

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

$$= \frac{(2)(0.0566)}{0.452}\{1.109(1.109/2 + 0.113) + (0.177)(0.363)(0.113) + 0.0\}$$

$= 0.187$ in.

e. Moment of inertia about the y-axis

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

$$= (2)(0.0566)\left\{\begin{array}{l} 1.109(1.109/2 + 0.113)^2 + (1.109)^3/12 + 0.356(0.113)^3 \\ + 0.0 \end{array}\right\} - (0.452)(0.187)^2$$

$= 0.0531$ in.^{4}

f. Distance between shear center and web centerline

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

$$= 1.222\left[\frac{(3)(5.641)^2(1.222) + 0.0}{(5.641)^3 + (6)(5.641)^2(1.222) + 0.0}\right]$$

$= 0.345$ 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

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

$$= \frac{(5.641)^2(1.222)^2(0.0566)}{12}\left\{\frac{(2)(5.641)^3(1.222) + (3)(5.641)^2(1.222)^2 + 0.0}{(6)(5.641)^2(1.222) + (5.641 + 0.0)^3 - 0.0}\right\}$$
$$= 0.316 \text{ in.}^6$$

c. Parameter used in determination of elastic critical moment

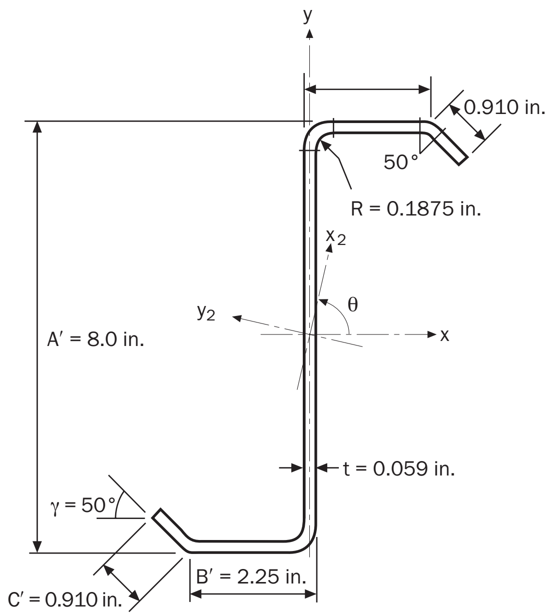
$$\beta_w = -\left[\frac{t\bar{x}_c\bar{a}^3}{12} + t\bar{x}_c^3\bar{a}\right]$$
$$= -\left[\frac{(0.0566)(0.187)(5.641)^3}{12} + (0.0566)(0.187)^3(5.641)\right]$$
$$= -0.1604 \text{ in.}^5$$

$$\beta_f = \frac{t}{2}\left[(\bar{b} - \bar{x}_c)^4 - \bar{x}_c^4\right] + \frac{t\bar{a}^2}{4}\left[(\bar{b} - \bar{x}_c)^2 - \bar{x}_c^2\right]$$
$$= \frac{0.0566}{2}\left[(1.222 - 0.187)^4 - (0.187)^4\right] + \frac{(0.0566)(5.641)^2}{4}\left[(1.222 - 0.187)^2 - 0.187^2\right]$$
$$= 0.4990 \text{ in.}^5$$

$$\beta_l = \alpha\left\{2\bar{c}t(\bar{b} - \bar{x}_c)^3 + \frac{2}{3}t(\bar{b} - \bar{x}_c)\left[(\bar{a}/2)^3 - (\bar{a}/2 - \bar{c})^3\right]\right\}$$
$$= 0.0 \text{ in.}^5$$

$$j = \frac{1}{2I_y}(\beta_w + \beta_f + \beta_l) - x_o$$
$$= \frac{1}{(2)(0.0531)}(-0.1604 + 0.4990 + 0.0) - (-0.532)$$
$$= 3.72 \text{ in.}$$

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.

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

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

$= 8.000 - [(2)(0.217) + 0.059] = 7.507$ in.

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

$= 8.000 - 0.059 = 7.941$ in.

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

$= 2.250 - [0.217 + 0.059/2 + 1.0(0.217 + 0.059/2)\tan(0.8727/2)] = 1.889$ in.

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

$= 2.250 - [0.059/2 + (1.0)(0.059/2)\tan(0.8727/2)] = 2.207$ in.

$$c = \alpha[C' - (r + t/2)\tan(\gamma/2)]$$
$$= 1.0[0.910 - (0.217 + 0.059/2)\tan(0.8727/2)] = 0.795 \text{ in.}$$
$$\bar{c} = \alpha[C' - (t/2)\tan(\gamma/2)]$$
$$= 1.0[0.910 - (0.059/2)\tan(0.8727/2)] = 0.896 \text{ in.}$$
$$u_1 = \pi r/2$$
$$= \pi(0.217)/2 = 0.341 \text{ in.}$$
$$u_2 = \gamma r$$
$$= (0.8727)(0.217) = 0.189 \text{ in.}$$

b. Cross-section area

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

c. Moment of inertia about the x-axis

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

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

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

d. Moment of inertia about the y axis

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

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

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

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

e. Product of inertia

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

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

$$= 0.118\ \{8.712 + 0.0051 + 0.1007 + 7.211 - 0.0002 - 0.0206 + 1.636\}$$

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

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

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

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

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

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

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

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

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

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

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

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

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

b. Warping constant

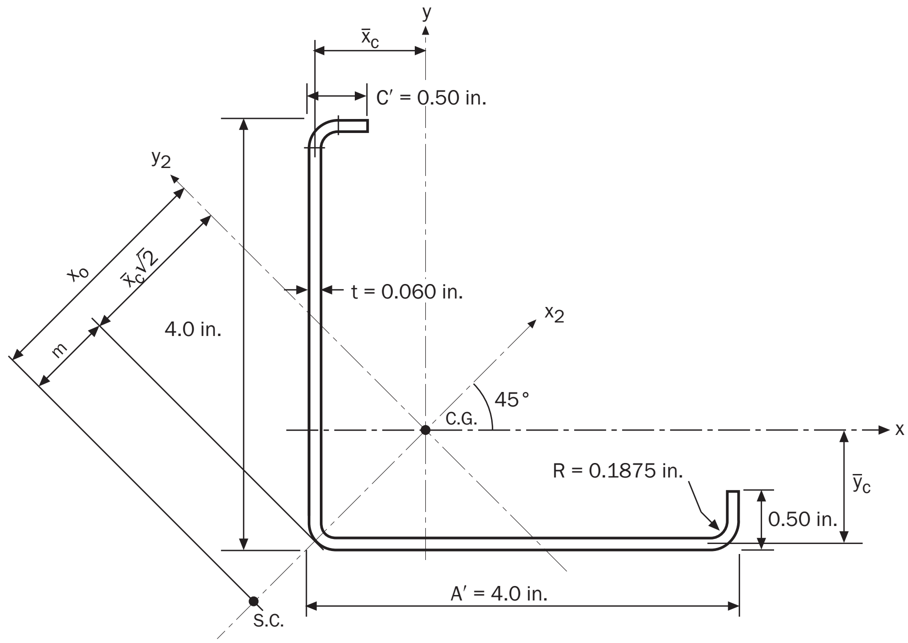
$$C_w = \frac{t}{12}\left\{\frac{\bar{a}^2\bar{b}^3(2\bar{a} + \bar{b}) + \alpha\left[\begin{array}{l}\bar{b}^2\left[4\bar{c}^4 + 16\bar{b}\bar{c}^3 + 6\bar{a}^3\bar{c} + 4\bar{a}^2\bar{b}\bar{c} + 8\bar{a}\bar{c}^3\right] \\ + 6\bar{a}\bar{b}\bar{c}^2(\bar{a} + \bar{b})\left[2\bar{b}\sin\gamma + \bar{a}\cos\gamma\right]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

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

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

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

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.

α = 1.0 (section has stiffener lips)

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

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

$= 4.000 - [0.218 + 0.060/2 + 1.0(0.218 + 0.060/2)] = 3.504$ in.

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

$= 4.000 - [0.060/2 + (1.0)(0.060)/2] = 3.940$ in.

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

$= (1.0)[0.500 - (0.218 + 0.060/2)] = 0.252$ in.

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

$= (1.0)[0.500 - 0.060/2] = 0.470$ in.

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

b. Cross-section area

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

$$A = 0.060\big[(2)(3.504) + 0.342 + (1.0)\big((2)(0.252) + (2)(0.342)\big)\big]$$

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

c. Distance between centroid and centerlines of webs

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

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

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

d. Moment of inertia about x and y axes

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

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

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

e. Product of inertia

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

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

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

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

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

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

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

b. St. Venant torsion constant

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

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

c. Warping constant

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

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

d. Distance from centroid to shear center

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

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

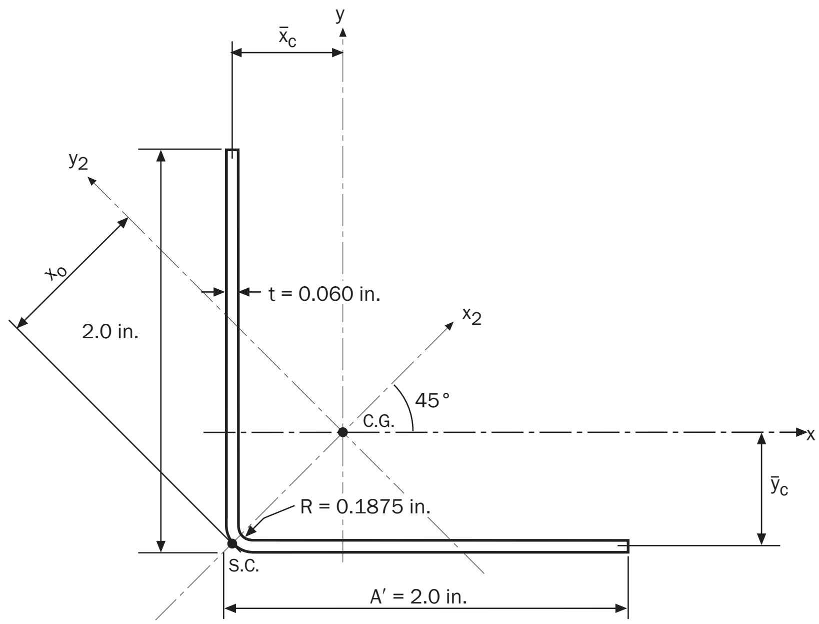
e. Parameter used to determine elastic critical moment

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

$$= \frac{\sqrt{2}\,(0.060)}{(48)(0.396)}\left[(3.940)^4 + (4)(3.940)^3(0.470) - (6)(3.940)^2(0.470)^2 + (0.470)^4\right] - (-1.634)$$

$$= 3.13 \text{ in.}$$

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.

α = 0.0 (section does not have stiffener lips)

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

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

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

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

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

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

$= 0.0$ in.

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

$= 0.0$ in.

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

b. Cross-section area

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

c. Distance between centroid and centerlines of webs

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

$$= \frac{0.060}{0.231}\left\{(1.752)\left(\frac{1.752}{2} + 0.218\right) + (0.342)(0.363)(0.218) + 0.0\right\}$$

$= 0.505$ in.

d. Moment of inertia about x and y axes

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

$$= 0.060\left\{\begin{matrix} 1.752\left(\frac{1.752}{2} + 0.218\right)^2 + \frac{(1.752)^3}{12} \\ + (0.342)[(0.363)(0.218)]^2 + (0.149)(0.218)^3 + 0.0 \end{matrix}\right\} - (0.231)(0.505)^2$$

$= 0.0940$ in.^{4}

e. Product of inertia

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

$$= 0.060\{-0.137(0.218)^3 + 0.342[0.363(0.218)]^2 + 0.0\} - (0.231)(0.505)(0.505)$$

$= -0.0589$ in.^{4}

f. Moment of inertia about y_2 axis

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

2. Torsional properties

a. Distance between shear center and centerline of square corner

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

$m = 0.000$ in.

b. St. Venant torsional constant

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

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

$$= 0.000277 \text{ in.}^4$$

c. Warping constant

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

$$= 0.000 \text{ in.}^6$$

d. Distance from centroid to shear center

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

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

$$= -0.714 \text{ in.}$$

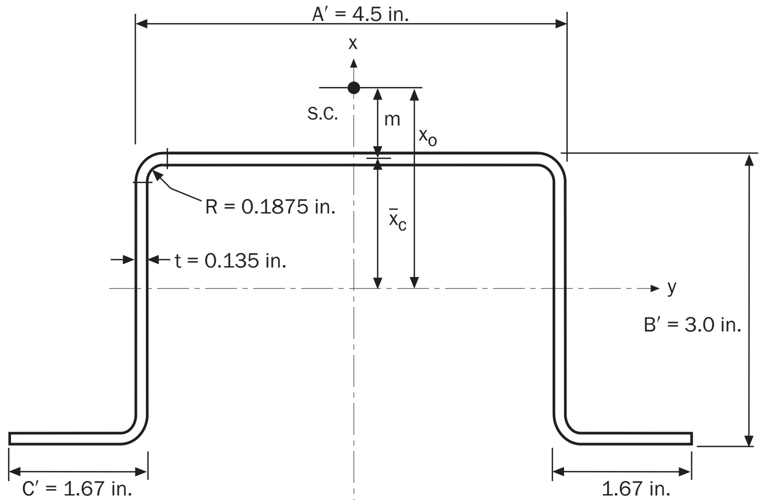
e. Parameter used to determine elastic critical moment

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

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

$$= 1.47 \text{ in.}$$

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′ = 4.5 in.

B′ = 3.00 in.

C′ = 1.670 in.

t = 0.135 in.

R = 0.1875 in.

α = 1.0

r = R + t/2

= 0.1875 + 0.135/2 = 0.255 in.

a = A′- (2r + t)

= 4.5 - [(2)(0.255) + 0.135] = 3.855 in.

$\bar{a}$ = A′- t

= 4.50 - 0.135 = 4.365 in.

b = B′- [r + t/2 + α(r + t/2)]

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

$\bar{b}$ = B′- [t/2 + αt/2]

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

c = α[C′- (r + t/2)]

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

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

$= 1.0[1.670 - (0.135/2)] = 1.603$ in.

$u = \pi r/2$

$= \pi(0.255)/2 = 0.401$ in.

b. Cross-section area

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

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

$= 1.737$ in.^{2}

c. Moment of inertia about the x axis

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

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

$= 0.270\{2.389 + 11.22 + 1.752 + 0.0025 + 1.0\ [0.2040 + 13.05 + 2.076 + 0.0025]\}$

$= 8.29$ in.^{4}

d. Distance between centroid and web centerline

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

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

$= 0.1554\{3.374 + 0.0371 + 1.0\ [\ 1.112 + 3.862]\}$

$= 1.303$ in.

e. Moment of inertia about the y axis

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

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

$$= 0.270\left\{4.833 + 1.088 + 0.0059 + 1.0\,[11.06 + 3.082 + 0.0025]\right\} - 2.949$$

$$= 2.470 \text{ in.}^4$$

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

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

$$= -(1.303 + 1.182)$$

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

2. Torsional properties

a. St. Venant torsional constant

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

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

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

b. Warping constant

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

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

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

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

c. Parameter used in determination of elastic critical moment

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

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

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

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

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

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

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

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

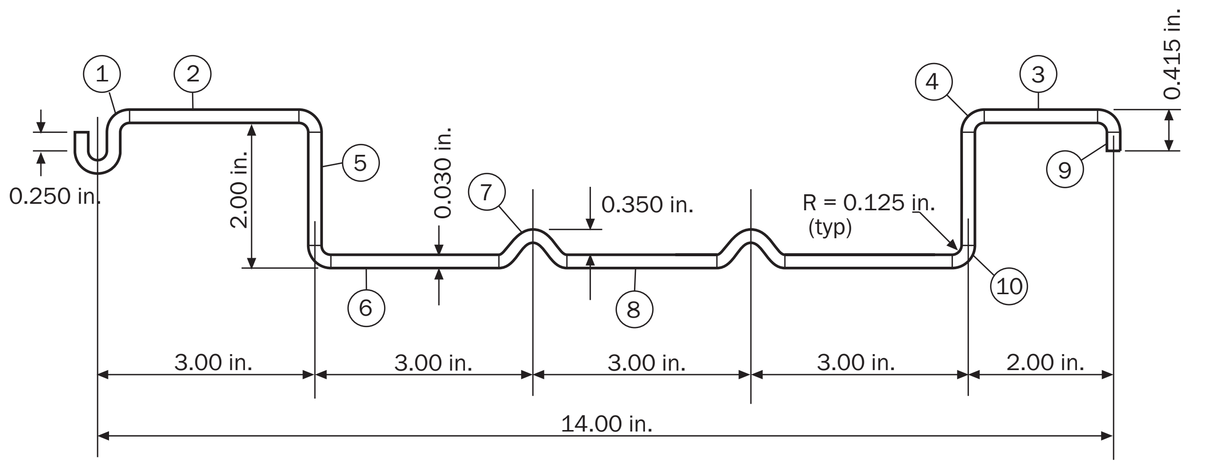
$$= 7.814 \text{ in.}^5$$

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

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

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

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)

$y = 2.00 + (0.125 - 0.089) = 1.964$ 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., $\theta = 45° = 0.785$ rad.

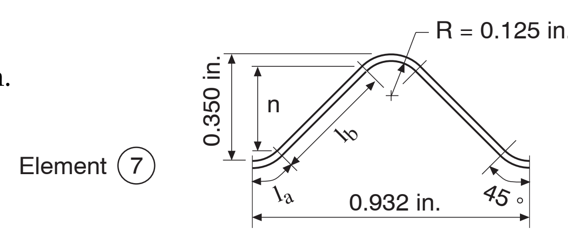
$c_1 = r\sin\theta/\theta = (0.140)(0.707)/0.785 = 0.126$ in.

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

$= 0.350 - 0.082 = 0.268$ 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.

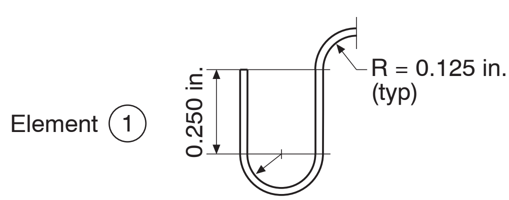
	L (in.)	y (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 about own axis (in.^{3})
Upper Radius	0.220	0.161	0.035	0.0057	----
Straight Segments	(2)(0.379) = 0.758	0.000	0.000	-----	0.0045
Lower Radii	0.220	-0.161	- 0.035	0.0057	----
Sum Σ	1.198		0.000	0.0114	0.0045

$$y_{cg} = \Sigma Ly/\Sigma L$$
$$= 0.000/1.198 = 0.000 \text{ in. (at centerline as expected)}$$
$$\Sigma I'_x = \Sigma Ly^2 + \Sigma I'_1 - y_{cg}^2\Sigma L$$
$$= 0.0114 + 0.0045 - (0.000)^2(1.198) = 0.0159 \text{ in.}^3$$

Distance of c.g. from top of panel

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

3. Element 1



	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 Σ	1.160		0.372	0.1476	0.0034

$$y_{cg} = \Sigma Ly/\Sigma L$$
$$= 0.372/1.160 = 0.321 \text{ in.}$$

$$\Sigma I'_x = \Sigma Ly^2 + \Sigma I'_1 - y_{cg}{}^2 \Sigma L$$
$$= 0.1476 + 0.0034 - (0.321)^2(1.160) = 0.0314 \text{ in.}^3$$

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 Σ	19.512		23.135	41.335	0.914

$$\bar{y} = \Sigma Ly/\Sigma L$$
$$= 23.135/19.512 = 1.186 \text{ in. from top fiber}$$
$$I_x = [\Sigma Ly^2 + \Sigma I'_x - \bar{y}^2 \Sigma L]t$$
$$= \left[41.335 + 0.914 - (1.186)^2(19.512)\right] 0.030 = 0.444 \text{ in.}^4$$
$$S_{ft} = I_x/\bar{y}$$
$$= 0.444/1.186 = 0.374 \text{ in.}^3$$
$$S_{fb} = I_x/(d - \bar{y})$$
$$= 0.444/(2.030 - 1.186) = 0.526 \text{ in.}^3$$
$$A = (\Sigma L)t = (19.512)(0.030) = 0.585 \text{ in.}^2$$

3.5 Effective Section Properties

Effective section properties are based on the effective width concept. For plate elements it is assumed that the total load in a plate element is carried by a fictitious effective width subject to a uniformly distributed stress equal to the maximum edge stress in the element while eliminating the remainder of the plate element. This concept eliminates the need to consider the non-uniform distribution of stress over the entire width of the plate. The non-uniform distribution of stress occurs in cold-formed steel design because of the consideration of post-buckling strength in the member elements. The use of post-buckling strength behavior complicates member design, but does permit more efficient use of steel.

Equations are presented in the *Specification* for the calculation of effective width of plate elements. The effective width varies depending upon the magnitude of the stress level, the distribution of stress, and the geometrical properties of the element being considered (flat width, thickness and whether the element is stiffened or unstiffened, etc.).

In many situations the calculation of section properties is iterative in nature. For example, if one considers a given beam profile, the stresses in the beam could be calculated initially using the full section properties of the beam. Using these properties, the effective widths of the compression elements are determined. Using the effective widths a new beam centroid is determined and new section properties are calculated. Using the newly determined properties, stresses are recalculated and the procedure is repeated until convergence is satisfied.

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

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.

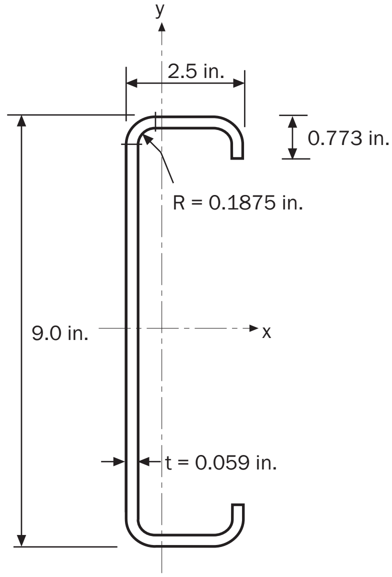
For uniformly stressed sections, i.e. column sections, the effective widths do not vary with location of the neutral axis, thus iteration is not required.

3.6 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 3.4 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 mdulus, 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 B4.2

$w = b = 2.007$ in.

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

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-1)

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

Compute k of the flange based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \leq t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4.2\text{-}10)$$

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

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

$d = c = 0.527$ in.

$\theta = 90$ degrees

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

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

$R_I = I_s/I_a \le 1$ (*Eq.* B4.2-9)
$= 0.000720/0.00166 = 0.434 < 1$ OK

$$n = \left[0.582 - \frac{w/t}{4S}\right] \ge \frac{1}{3}$$ (*Eq.* B4.2-11)

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

$= 0.295 < 1/3 \therefore n = 1/3$
$D = 0.773$ in.
$D/w = 0.773/2.007 = 0.385 < 0.8$ OK (From Table B4.2)

$$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \le 4$$ (From Table B4.2)

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

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

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

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

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

$\rho = (1 - 0.22/\lambda) / \lambda$ (*Eq.* B2.1-3)
$= (1 - 0.22/0.955)/ 0.955 = 0.806$
$b = \rho w$ (*Eq.* B2.1-2)
$= (0.806)(2.007) = 1.618$ 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)
$f = 55\,[4.500 - 0.059/2 - 0.217]/ 4.500 = 51.99$ ksi
$k = 0.43$

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

$$= 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{8.93}\right)^2 = 143.8 \text{ ksi}$$

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

$$= \sqrt{\frac{51.99}{143.8}} = 0.601 < 0.673 \therefore \text{ lip is not subject to local buckling}$$

$d'_s = d = 0.527$ in.
$d_s = d'_s(R_I)$ (*Eq.* B4.2-7)
$= (0.527)(0.434) = 0.229$ 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)

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

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

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

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

$b_e = (0.639)(8.507) = 5.436$ 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)

$= 5.436/(3 + 1) = 1.359$ in.

For $\psi > 0.236$

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

$b_1 + b_2 \le w/2$

$1.359 + 2.718 = 4.077 < 8.507/2 = 4.254$ $\therefore$ 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:

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

$= (0.059/2) + 0.217 + 1.359 + (0.177/2) = 1.694$ 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 Σ	14.075		66.730	425.229	51.325

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

$= 66.730/14.075 = 4.741$ in. below top fiber

$I_x = [\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2\Sigma L]\, t$

$= [51.325 + 425.229 - (4.741)^2(14.075)]\,(0.059) = 9.451$ 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

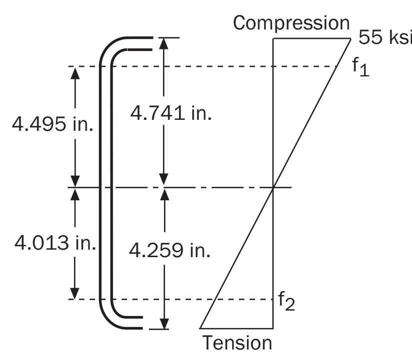
The change in neutral axis location will change the stress gradient and consequently the maximum stress in the stiffener slightly. This may cause a minor change in the effective width of the stiffener. Neglect in this case.

c. Web

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

f_1 = 55(4.741 - 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)

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

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

ρ = (1 - 0.22/1.380)/1.380 = 0.609 (*Eq.* B2.1-3)

b_e = (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. ∴ 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 Σ	13.679		65.962	423.746	51.309

$\bar{y} = \Sigma Ly/\Sigma L$
$= 65.962/13.679 = 4.822$ in. below top fiber
$I_x = [\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]\ t$
$= [51.309 + 423.746 - (4.822)^2(13.679)]\ (0.059) = 9.263$ in.^{4}
$S_e = I_x / \bar{y}$
$= 9.263/4.822 = 1.92$ 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_x 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.
$I_x = 9.18$ in.^{4}
$S_e = 1.89$ 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

$$S = 1.28\sqrt{29500/37.25} = 36.02 \qquad (Eq.\ B4\text{-}1)$$

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

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \qquad (Eq.\ B4.2\text{-}10)$$

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

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

$$R_I = I_s/I_a = 0.000720/0.00113 = 0.637 \qquad (Eq.\ B4.2\text{-}9)$$

$$n = \left[0.582 - \frac{w/t}{4S}\right] \ge \frac{1}{3} \qquad (Eq.\ B4.2\text{-}11)$$

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

$= 0.346 > 1/3$ OK

$$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \le 4 \qquad \text{(From Table B4.2)}$$

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

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

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

$$\rho = (1 - 0.22/0.746)/0.746 = 0.945 \qquad (Eq.\ B2.1\text{-}3)$$

$$b = \rho w = (0.945)(2.007) = 1.897 \text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

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

$$f = 37.25 \text{ ksi}$$

$$k = 0.43$$

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

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

$$d'_s = w = 0.527 \text{ in.}$$

$$d = d'_s(R_I) = (0.527)(0.637) = 0.336 \text{ in.} \qquad (Eq.\ B4.2\text{-}7)$$

c. Web: from Section B2.3

$$f = 37.25 \text{ ksi}$$

$$k = 4.0$$

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

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

$$\rho = (1 - 0.22/2.695)/2.695 \qquad (Eq.\ B2.1\text{-}3)$$
$$= 0.341$$

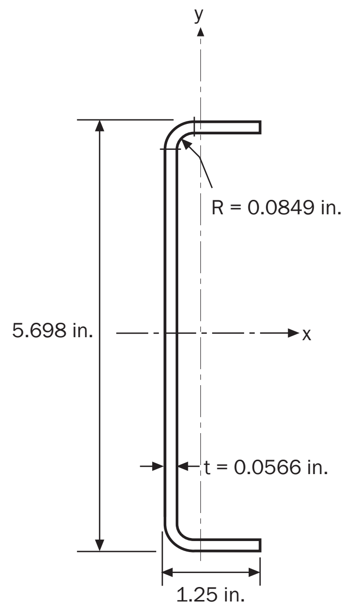
$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$
$$= (0.341)(8.507) = 2.901 \text{ in.}$$

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 Σ	8.731

$$A_e = t\Sigma L$$
$$= (0.059)(8.731) = 0.515 \text{ in.}^2$$

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



Given:

1. Steel: F_y = 33 ksi
2. Section: SSMA 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-3)

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 in.

w/t = 1.109/0.0566 = 19.59 < 60 OK (Section B1.1(a)(3))

k = 0.43

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

$$= 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{19.59}\right)^2 = 29.87 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{30.93}{29.87}} = 1.018 > 0.673 \therefore \text{ flange is subject to local buckling}$$

ρ = (1 - 0.22/λ) / λ (*Eq.* B2.1-3)
= (1 - 0.22/1.018)/ 1.018 = 0.770

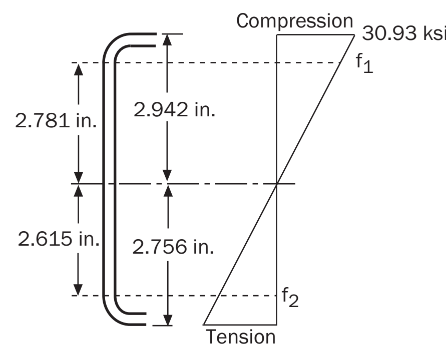
$b = \rho w$ (*Eq.* B2.1-2)

$= (0.770)(1.109)$

$= 0.854$ 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 Σ	7.732		22.747	85.211	13.232



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

$= 22.747/7.732 = 2.942$ in. below top fiber

$I_x = [\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2\Sigma L]\ t$

$= [13.232 + 85.211 - (2.942)^2(7.732)]\ (0.0566)$

$= 1.784$ in.^{4}

$S_e = I_x / \bar{y} = 1.784/2.942 = 0.606$ in.^{3}

Check Web:

By similar triangles

$$f_1 = \left(\frac{2.942 - 0.113 - 0.0566/2}{2.942}\right)(30.93) = 29.44 \text{ ksi}$$

$$f_2 = -\left(\frac{5.698 - 2.942 - 0.113 - 0.0566/2}{2.942}\right)(30.93) = -27.49 \text{ ksi}$$

$\psi = |f_2/f_1| = |-27.49/29.44| = 0.934$ (*Eq.* B2.3-1)

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

$= 4 + 2(1 + 0.934)^3 + 2(1 + 0.934)$

$= 22.34$

$w = 5.698 - 2(0.113) - 0.0566 = 5.415$ 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)

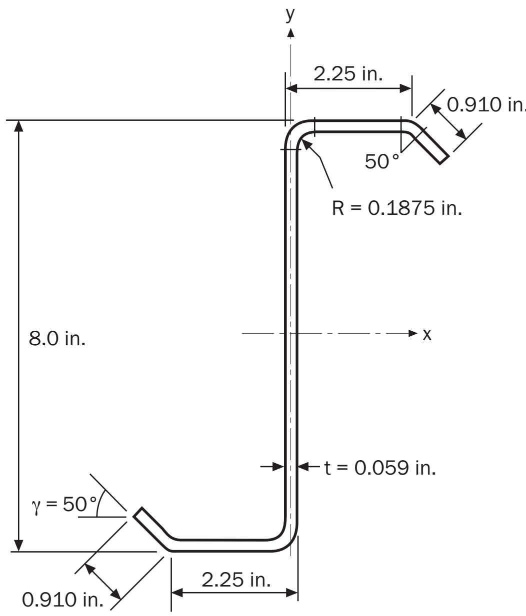
$= 5.415/(3 + 0.934) = 1.376$ in.

$b_2 = b_e/(1 + \psi) - b_1$ (*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
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 effect 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 B4.2

$w = b = 1.889$ in.

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

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-1)

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

Compute k of flange based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \leq t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4.2\text{-}10)$$

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

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

$d = c = 0.795$ in.

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

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

$R_I = I_s/I_a \leq 1$ (*Eq.* B4.2-9)

$= 0.00145/0.00156 = 0.929$

$$n = \left[0.582 - \frac{w/t}{4S}\right] \geq \frac{1}{3} \quad (\textit{Eq. } B4.2\text{-}11)$$

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

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

$D = 0.910$ in.

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

$$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \leq 4 \quad \text{(From Table B4.2)}$$

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

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

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

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (\textit{Eq. } B2.1\text{-}4)$$

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

$\rho = (1 - 0.22/\lambda) / \lambda$ (*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_3 (by similar triangles)

$f = f_3 = 55\,[4.000 - 0.059/2 - 0.217(1-\cos(50°))]/ 4.000 = 53.5$ ksi

$k = 0.43$

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

$$= 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{13.5}\right)^2 = 62.9 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (\textit{Eq. } B2.1\text{-}4)$$

$$= \sqrt{\frac{53.5}{62.9}} = 0.922 > 0.673 \;\therefore\; \text{lip is subject to local buckling}$$

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

$= (1 - 0.22/0.922)/ 0.922 = 0.826$

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

$= (0.826)(0.795) = 0.657$ in.

$d_s = d'_s(R_I)$ (*Eq.* B4.2-7)

$= (0.657)(0.929) = 0.610$ 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)

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

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

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

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

$b_e = (0.707)(7.507) = 5.307$ 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	0.610	0.340	0.207	0.071	0.011
Bottom Lip	0.795	7.589	6.033	45.786	0.025
Sum Σ	13.480		55.614	319.162	35.295

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

$= 55.614/13.480 = 4.126$ in. below top fiber

I_x = $[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2\Sigma L]\ t$
= $[35.295 + 319.162 - (4.126)^2(13.480)]\ (0.059) = 7.37$ in.^{4}

Second iteration with new neutral axis location

The calculated neutral axis location (4.126 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

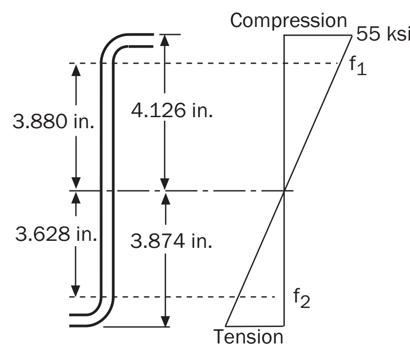
The change in neutral axis location will change the stress gradient and consequently the maximum stress in the stiffener slightly. This may cause a minor change in the effective width of the stiffener. Neglect in this case.

c. Web

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

f_1 = $55(4.126 - 0.059 - 0.1875)/4.126$
= 51.71 ksi

f_2 = $-55(8.000 - 4.126 - 0.059 - 0.1875)/4.126$
= - 48.36 ksi



ψ = $|f_2 / f_1| = |-48.36/51.71| = 0.935$ (*Eq.* B2.3-1)

k = $4 + 2\,(1 + 0.935)^3 + 2\,(1 + 0.935) = 22.36$ (*Eq.* B2.3-2)

$$F_{cr} = 22.36\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{127.2}\right)^2 = 36.85 \text{ ksi}$$ (*Eq.* B2.1-5)

$$\lambda = \sqrt{\frac{51.71}{36.85}} = 1.185 > 0.673 \therefore \text{ web may be subject to local buckling}$$ (*Eq.* B2.1-4)

ρ = $(1 - 0.22/1.185)/1.185 = 0.687$ (*Eq.* B2.1-3)

b_e = $(0.687)(7.507) = 5.157$ in. (*Eq.* B2.1-2)

b_1 = $b_e / (3 + \psi)$ (*Eq.* B2.3-3)
= $5.157 / (3 + 0.935) = 1.311$ in.

For $\psi > 0.236$

b_2 = $b_e / 2 = 5.157/2 = 2.579$ in. (*Eq.* B2.3-4)

$b_1 + b_2 = 1.311 + 2.579 = 3.890$ in.

Width of compression block

$4.126 - 0.059 - 0.1875 = 3.880$ in. < 3.890 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:

$\bar{y}$ = 4.126 in. below top fiber

I_x = 7.37 in.^{4}

S_e = $I_x/\bar{y}$

$S_e = 7.37/4.126 = 1.79 \text{ in.}^3$

$M_n = S_e F_y$ (*Eq.* C3.1.1-1)

$= (1.79)(55) = 98.5$ kip-in.

2. Effective moment of inertia, I_x, at a service load of 60% of M_n; M = (0.60)(98.5) = 59.1 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 B4.2

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-1)

$= 1.28\sqrt{29500/33} = 38.3$

$w/t = 32.0 > 0.328\,S \Rightarrow$ check effective width of flange

Compute flange k based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4.2\text{-}10)$$

$$= 399(0.059)^4\left[\frac{32.0}{38.3} - 0.328\right]^3 \le (0.059)^4\left[115\frac{32.0}{38.3} + 5\right]$$

$= 0.000632 \text{ in.}^4 < 0.00123 \text{ in.}^4 \therefore I_a = 0.000632 \text{ in.}^4$

$R_I = I_s/I_a \le 1$ (*Eq.* B4.2-9)

$= 0.00145/0.000632 = 2.29 \therefore R_I = 1.0$

$$n = \left[0.582 - \frac{w/t}{4S}\right] \ge \frac{1}{3} \quad (Eq.\ B4.2\text{-}11)$$

$$= \left[0.582 - \frac{32.0}{(4)(38.3)}\right] \ge \frac{1}{3}$$

$= 0.373 > 1/3 \therefore n = 0.373$

$$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \le 4 \quad \text{(From Table B4.2)}$$

$$= \left(4.82 - \frac{(5)(0.910)}{1.889}\right)(1.0)^{0.373} + 0.43 = 2.84 < 4 \text{ OK}$$

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

$$= 2.84\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{32.0}\right)^2 = 73.95 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{33}{73.95}} = 0.668 < 0.673 \therefore \text{flange is not subject to local buckling}$$

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)

$f = 33\,[4.000 - 0.059/2 - 0.217(1\text{-}\cos(50°))]/4.000 = 32.12$ ksi

$k = 0.43$

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

$$F_{cr} = 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{13.5}\right)^2 = 62.91 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{32.12}{62.91}} = 0.715 > 0.673 \therefore \text{ lip is subject to local buckling}$$

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

$= (1 - 0.22/0.715)/0.715 = 0.968$

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

$= (0.968)(0.795) = 0.770$ in.

$d_s = d'_s(R_I)$ (*Eq.* B4.2-7)

$= (0.770)(1.0) = 0.770$ 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. 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	1.889	0.030	0.057	0.002	---
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	0.770	0.402	0.310	0.124	0.022
Bottom Lip	0.795	7.589	6.033	45.786	0.025
Sum Σ	13.910		55.725	319.216	35.306

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

$= 55.725/13.910 = 4.006$ in. below top fiber

$I_x = [\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2\Sigma L]\, t$

$= [35.306 + 319.216 - (4.006)^2(13.910)](0.059) = 7.75$ in.^{4}

$S_e = I_x / \bar{y}$

$= 7.75 / 4.006 = 1.93$ in.^{3}

$M = S_e f$

$= (1.93)(33) = 63.7$ kip-in. > 59.1 kip-in. $\therefore$ maximum stress at service level is lower.

Try f = 33(59.1/63.7) = 30.6 ksi (by proportioning)

Recomputing I_x using a maximum stress of 30.6 ksi using the same approach as above (calculations not shown) we obtain:

$\bar{y}$ = 4.002 in. below top fiber

I_x = 7.76 in.^{4}

S_e = 1.94 in.^{3}

M = $S_e f$

= (1.94)(30.6) = 59.4 kip-in. ≅ 59.1 kip-in. ∴ solution has converged satisfactorily

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. It can be inferred by comparison with the calculations above that any reduction in the effective width of the stiffener lip will be insignificant at a stress level of 25.9 ksi.

Check web

$k = 4.0$

$$F_{cr} = 4.0\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{127.2}\right)^2 = 6.59 \text{ ksi} \qquad (Eq.\ \text{B2.1-5})$$

$$\lambda = \sqrt{\frac{25.9}{6.59}} = 1.982 > 0.673 \ \therefore \text{ web is not fully effective} \qquad (Eq.\ \text{B2.1-4})$$

$$\rho = (1 - 0.22/1.982)/1.982 = 0.449 \qquad (Eq.\ \text{B2.1-3})$$

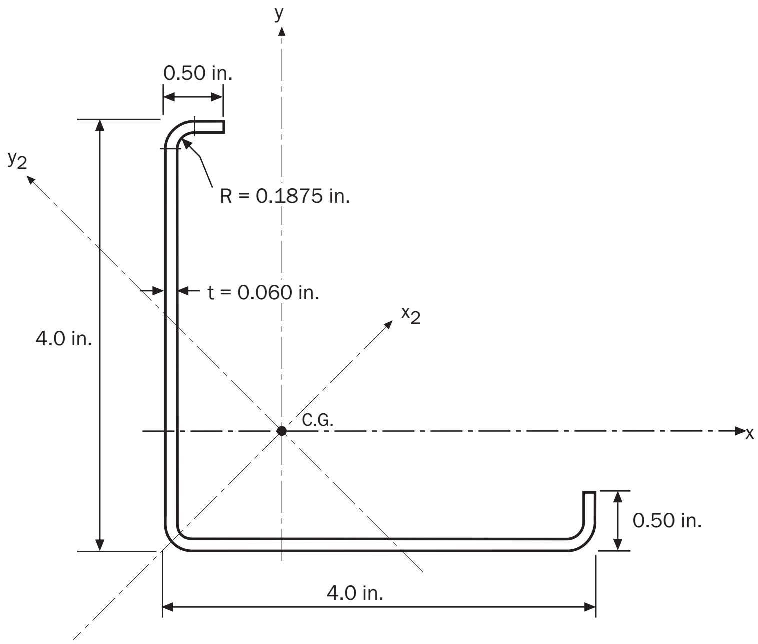
$$b_e = (0.449)(7.507) = 3.371 \text{ in.} \qquad (Eq.\ \text{B2.1-2})$$

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 in.^{2}

A_e = 0.822 - (7.507 - 3.371)(0.059) = 0.578 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:

1. Effective area, A_e, at a uniform compression stress of 14.7 ksi, as required in Example III-4

Solution:

See Example I-4 for basic parameters. Treat each leg as a uniformly compressed element with an edge stiffener (Section B4.2).

a. Legs

$w = 3.504$ in.

$t = 0.060$ in.

$w/t = 3.504/0.060 = 58.4$

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-1)

$= 1.28\sqrt{29500/14.7}$

$= 57.3$

w/t > 0.328S, therefore check effective width of leg

Compute k of flange based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4.2\text{-}10)$$

$$= 399(0.060)^4\left[\frac{58.4}{57.3} - 0.328\right]^3 \le (0.060)^4\left[115\frac{58.4}{57.3} + 5\right]$$

$= 0.00171 \text{ in.}^4 > 0.00158 \text{ in.}^4 \;\therefore I_a = 0.00158 \text{ in.}^4$

$d = c = 0.252$ in.

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

$$= \frac{(0.252)^3(0.060)\sin^2(90°)}{12} = 0.0000800 \text{ in.}^4$$

$R_I = I_s/I_a \le 1$ (*Eq.* B4.2-9)

$= 0.0000800 / 0.00158 = 0.0506$

$n = \left[0.582 - \dfrac{w/t}{4S}\right] \ge \dfrac{1}{3}$ (*Eq.* B4.2-11)

$= \left[0.582 - \dfrac{58.4}{(4)(57.3)}\right] \ge \dfrac{1}{3}$

$= 0.327 < 1/3 \therefore n = 1/3$

$D = 0.500$ in.

$D/w = 0.500/3.504 = 0.143 < 0.25$

$k = 3.57(R_I)^n + 0.43 \le 4$ (From Table B4.2)

$= 3.57(0.0506)^{\frac{1}{3}} + 0.43 = 1.75$

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

$d_s = d_s'(R_I)$ (*Eq.* B4.2-7)

$= (0.252)(0.0506) = 0.013$ in.

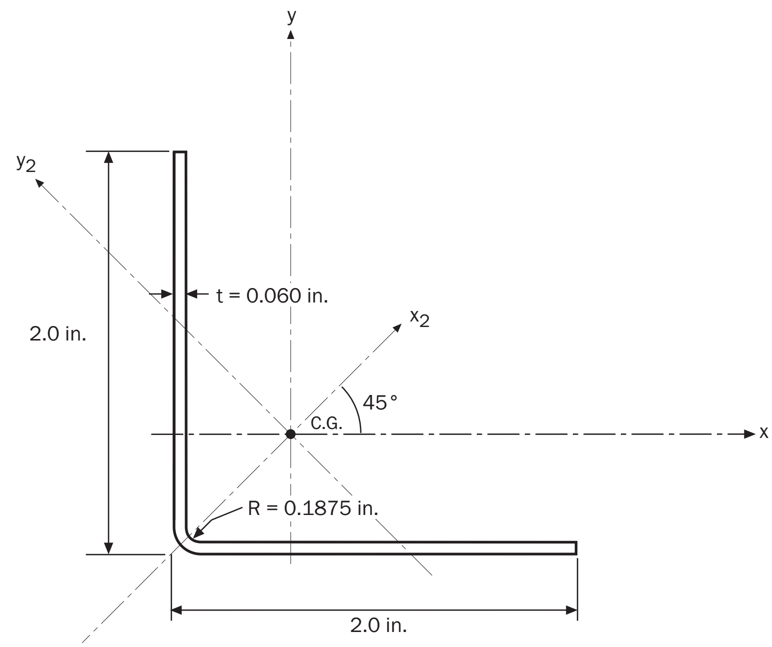
Summing the effective widths,

$A_e = t[2(b + d_s) + 3u]$

$= 0.060[2(2.663 + 0.013) + (3)(0.342)]$

$= 0.383$ 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 web as unstiffened element with stress gradient (Section B3.2).

k = 0.43

w = 1.752 in.

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

$$= 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.060}{1.752}\right)^2 = 13.45 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{33}{13.45}} = 1.566 > 0.673 \therefore \text{ leg is subject to local buckling}$$

$$\rho = (1 - 0.22/\lambda)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$

$$= (1 - 0.22/1.566)/1.566 = 0.549$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$b = (0.549)(1.752) = 0.962$ 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	0.962	1.272	1.224	1.557	0.074
corner	0.342	1.891	0.647	1.223	0.002
flange	1.752	1.970	3.451	6.799	----
Sum Σ	3.056		5.322	9.579	0.076

$\bar{y} = \Sigma Ly/\Sigma L = 5.322/3.056 = 1.741$ in. below top fiber

$$I_x = [\Sigma I_x' + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$$

$$= [0.076 + 9.579 - (1.741)^2(3.056)]0.060 = 0.0235 \text{ in.}^4$$

$$S_t = \frac{I_x}{y_t} = \frac{0.0235}{1.741} = 0.0135 \text{ in.}^3$$

$$S_b = \frac{I_x}{y_b} = \frac{0.0235}{2.000 - 1.741} = 0.0907 \text{ in.}^3$$

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.

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

$$= 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.060}{1.752}\right)^2 = 13.45 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (\textit{Eq. } \text{B2.1-4})$$

$$= \sqrt{\frac{12.0}{13.45}} = 0.945 > 0.673 \therefore \text{ leg is subject to local buckling}$$

$$\rho = (1 - 0.22/\lambda)/\lambda \quad (\textit{Eq. } \text{B2.1-3})$$

$$= (1 - 0.22/0.945)/0.945$$

$$= 0.812$$

$$b = \rho w = (0.812)(1.752) = 1.423 \text{ in.} \quad (\textit{Eq. } \text{B2.1-2})$$

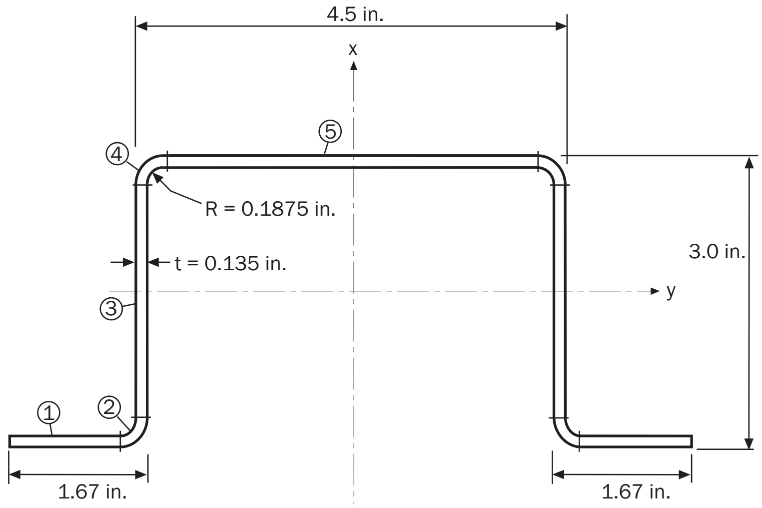
Effective area

$$A_e = t\Sigma L$$

$$= (0.060)[(2)(1.423) + 0.342]$$

$$= 0.191 \text{ in.}^2$$

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 (Section B1.2-(a)). Assumed fully effective

Element 5:

w/t = 3.855/0.135 = 28.56 < 500 OK (Section B1.1-(a)-(2))

k = 4.0 (fully stiffened element)

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

$$= 4\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{28.56}\right)^2 = 130.8 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{50}{130.8}} = 0.618 < 0.673 \therefore \text{ flange is not subject to local buckling}$$

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

$= 3.855$ 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 Σ	12.865		17.639	40.282	2.185

Distance of neutral axis from top fiber,

$\bar{x} = \Sigma Lx/\Sigma L = 17.639/12.865 = 1.371$ in.

$I_y = [\Sigma Lx^2 + \Sigma I'_y - \bar{x}^2\Sigma L]t$

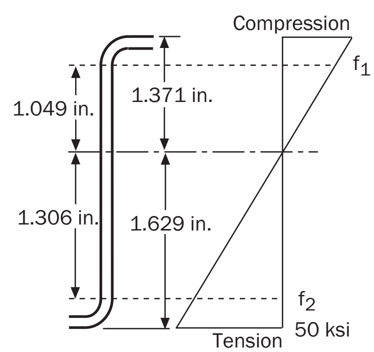
$= [40.282 + 2.185 - (1.371)^2(12.865)](0.135)$

$= 2.469$ 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



$f_1 = (1.049/1.629)(50) = 32.20$ ksi (compression)

$f_2 = -(1.306/1.629)(50) = -40.09$ ksi (tension)

$\psi = |f_2/f_1| = |-40.09/32.20| = 1.245$ (*Eq.* B2.3-1)

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

$= 4 + 2(1+ 1.245)^3 + 2(1 +1.245)$

$k = 31.12$

$$F_{cr} = 31.12\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{17.44}\right)^2 = 2728 \text{ ksi}$$ (*Eq.* B2.1-5)

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

$$= \sqrt{\frac{32.20}{2728}} = 0.109 < 0.673$$

$b = w$ (*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.

$$\lambda_1 = \frac{1.11}{\sqrt{F_y/E}} \quad (Eq.\ C3.1.1\text{-}2)$$

$$= \frac{1.11}{\sqrt{50/29500}} = 26.96$$

$$\lambda_2 = \frac{1.28}{\sqrt{F_y/E}} \quad (Eq.\ C3.1.1\text{-}3)$$

$$= \frac{1.28}{\sqrt{50/29500}} = 31.09$$

w/t of compression flange = 28.56

For $26.96 = \lambda_1 < w/t < \lambda_2 = 31.09$

$$C_y = 3 - 2\left(\frac{w/t - \lambda_1}{\lambda_2 - \lambda_1}\right)$$

$$= 3 - 2\left(\frac{28.56 - 26.96}{31.09 - 26.96}\right) = 2.23$$

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. Using the equations from Reck, Pekoz and Winter, "Inelastic Strength of Cold-Formed Steel Beams", Journal of the Structural Division, November 1975, ASCE:

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.

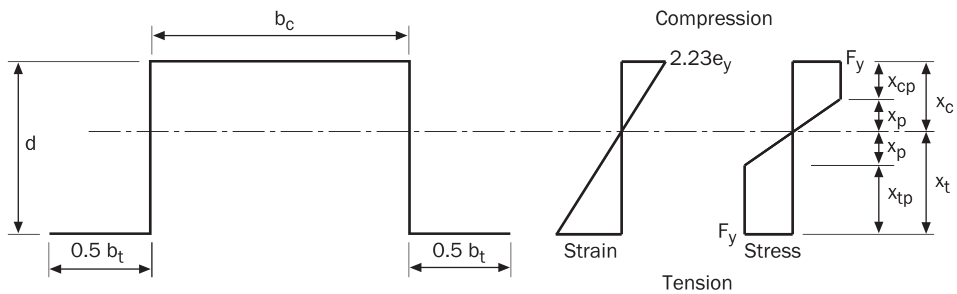
$x_c = (1/4)(b_t - b_c + 2d)$

$= (1/4)[3.340 - 4.500 + 2(3.000)] = 1.210$ 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.
$x_{cp} = x_c - x_p$
$x_{cp} = 1.210 - 0.543 = 0.667$ in.
$x_{tp} = x_t - x_p = 1.790 - 0.543 = 1.247$ in.



Summing moments of stresses in component plates:

$$M_n = F_y t\left\{b_c x_c + 2x_{cp}\left[x_p + \frac{x_{cp}}{2}\right] + \frac{4}{3}x_p^2 + 2x_{tp}\left[x_p + \frac{x_{tp}}{2}\right] + b_t x_t\right\}$$

$$M_n = (50.0)(0.135)\left\{\begin{array}{c}(4.500)(1.210) + (2)(0.667)\left[0.543 + \frac{0.667}{2}\right] + \frac{4}{3}(0.543)^2 \\ + 2(1.247)\left[0.543 + \frac{1.247}{2}\right] + (3.340)(1.790)\end{array}\right\}$$

$M_n = 107.3$ 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 torsional-flexural buckling.

(2) The effect of cold-forming is not included in determining the yield point, 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)

$w/t = 3.855/0.135$
$= 28.56$
$k = 4.0$

$F_{cr} = 4.0\dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{1}{28.56}\right)^2 = 130.8$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{50}{130.8}} = 0.618 < 0.673$ (flange is fully effective) (*Eq.* B2.1-4)

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

$= 3.855$ in.

Element 3: Uniformly Compressed Element with an Edge Stiffener (Section B4.2)

$w = 2.355$ in.

$w/t = 2.355/0.135 = 17.4 < 60$ OK (Section B1.1(a)(1))

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-1)

$= 1.28\sqrt{29500/50} = 31.1 \therefore w/t \geq 0.328S \Rightarrow$ check effective width of element

Compute k of element based on stiffener lip (element 1) properties.

$I_a = 399t^4\left[\dfrac{w/t}{S} - 0.328\right]^3 \leq t^4\left[115\dfrac{w/t}{S} + 5\right]$ (*Eq.* B4.2-10)

$= 399(0.135)^4\left[\dfrac{17.4}{31.1} - 0.328\right]^3 \leq (0.135)^4\left[115\dfrac{17.4}{31.1} + 5\right]$

$= 0.00164 \text{ in.}^4 < 0.0230 \text{ in.}^4 \therefore I_a = 0.00164 \text{ in.}^4$

$d = 1.348$ in.

$\theta = 90$ degrees

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

$= (1.348)^3 (0.135) \sin^2(90°)/12 = 0.0276 \text{ in.}^4$

$R_I = I_s/I_a \leq 1$ (*Eq.* B4.2-9)

$= 0.0276/0.00164 = 16.8 > 1 \therefore R_I = 1$

$D = 1.67$ in.

$D/w = 1.67/2.355 = 0.71 < 0.8$ OK (From Table B4.2)

$k = \left(4.82 - \dfrac{5D}{w}\right)(R_I)^n + 0.43 \leq 4$ (From Table B4.2)

$= \left(4.82 - \dfrac{(5)(1.67)}{2.355}\right)(1)^n + 0.43 = 1.70 < 4$ OK

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

$= 1.70\dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{1}{17.4}\right)^2 = 150$ ksi

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

$= \sqrt{\dfrac{50}{150}} = 0.577 < 0.673 \therefore$ element is not subject to local buckling

$b = w$

$= 2.355$ in.

Element 1: Uniformly Compressed Unstiffened Element (Section B3.1 and B4.2)

$w/t = 1.348/0.135 = 9.99$

$k = 0.43$

$$F_{cr} = 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{9.99}\right)^2 = 115 \text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{50}{115}} = 0.659 < 0.673 \therefore \text{ element is not subject to local buckling}$$

$$d'_s = b = w \qquad (Eq.\ B2.1\text{-}1)$$

$$= 1.348 \text{ in.}$$

$$d_s = d'_s(R_I) \qquad (Eq.\ B4.2\text{-}7)$$

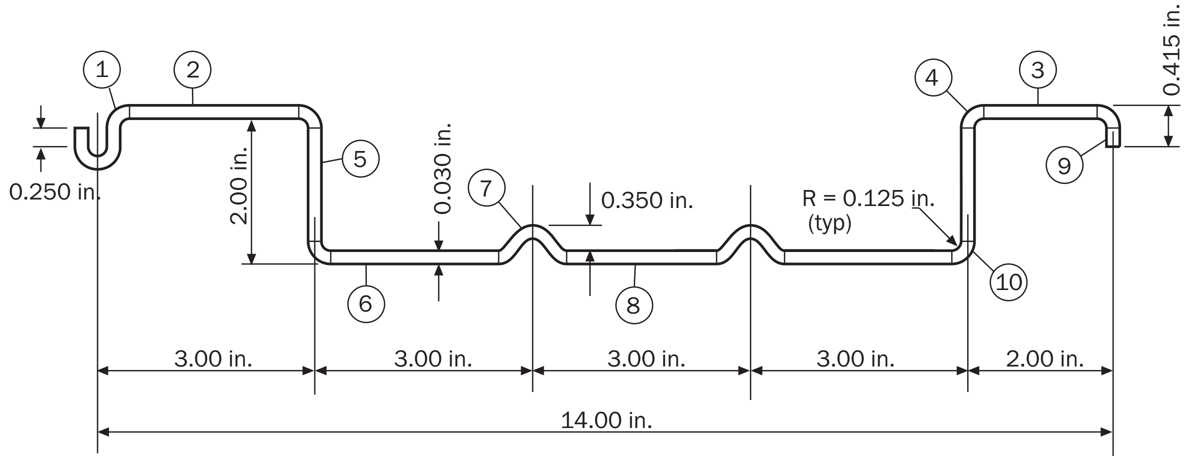
$$= (1.348)(1.0) = 1.348 \text{ in.}$$

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 Σ	12.865

$A_e = t\Sigma L = (0.135)(12.865) = 1.74 \text{ in.}^2$

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)

w = 0.250 in.

k = 0.43

f = F_y (Conservative assumption for preliminary check)

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

$$= 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{0.250}\right)^2 = 165.1\text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$= \sqrt{\dfrac{50}{165.1}} = 0.550 < 0.673$ ∴ flange is not subject to local buckling, so no need to calculate more precisely

b = w = 0.250 in.

Stiffened Flat Elements of Element 1 with Stress Gradient from Section B2.3

By inspection, the stress level will be the same as that of 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.

$A'_s = A = (1.160)(0.030) = 0.0348 \text{ in.}^2$

Element 2 from Section B4.2(a)

$w = 3.000 - 3(0.125 + 0.030/2) = 2.580 \text{ in.}$

$f = F_y = 50 \text{ ksi}$

$$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/50} = 31.09 \qquad (\textit{Eq. B4-1})$$

$w/t = 2.580/0.030 = 86.0 > 0.328S \therefore$ check effective width

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \qquad (\textit{Eq. B4.2-10})$$

$$= 399(0.030)^4\left[\frac{86.0}{31.09} - 0.328\right]^3 \le (0.030)^4\left[115\frac{86.0}{31.09} + 5\right]$$

$$= 0.00468 \text{ in.}^4 > 0.000262 \text{ in.}^4 \therefore I_a = 0.000262 \text{ in.}^4$$

$I_s = I'_1 t = (0.0314)(0.030) = 0.000942 \text{ in.}^4$ (from Example I-7)

$$R_I = I_s/I_a \le 1 \qquad (\textit{Eq. B4.2-9})$$

$$= 0.000942/0.000262 = 3.6 > 1.0 \therefore R_I = 1.0$$

$$n = \left[0.582 - \frac{w/t}{4S}\right] \ge \frac{1}{3} \qquad (\textit{Eq. B4.2-11})$$

$$= \left[0.582 - \frac{86.0}{(4)(31.09)}\right] \ge \frac{1}{3}$$

$$= -0.110 < 1/3 \therefore n = 1/3$$

$$k = 3.57(R_I)^n + 0.43 \le 4 \qquad \text{(From Table B4.2)}$$

$$= 3.57(1.0)^{\frac{1}{3}} + 0.43 = 4.0$$

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

$$= 4.0\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{2.580}\right)^2 = 14.42 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (\textit{Eq. B2.1-4})$$

$$= \sqrt{\frac{50.0}{14.42}} = 1.862 > 0.673 \therefore \text{ element is subject to local buckling}$$

$$\rho = (1 - 0.22/\lambda)/\lambda \qquad (\textit{Eq. B2.1-3})$$

$$= (1 - 0.22/1.862)/1.862 = 0.474$$

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

$= (0.474)(2.580) = 1.223$ in.

$A_s = A'_s(R_I) = (1.160)(0.030)\ (1.0) = 0.0348\ \text{in.}^2$ (*Eq.* B4.2-8)

Element 9 from Section B3.2(a)

$w = 0.415 - 0.030 - 0.125 = 0.260$ in.

$k = 0.43$

$f < F_y$, use F_y as a conservative value

$$F_{cr} = 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{0.260}\right)^2 = 152.6\ \text{ksi} \quad (\textit{Eq. } B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}}$$

$$= \sqrt{\frac{50.0}{152.6}} = 0.572 < 0.673 \ \therefore \text{ element is not subject to local buckling} \quad (\text{Eq. B2.1-4})$$

$d'_s = b = w = 0.260$ in. (*Eq.* B2.1-1)

Element 3 from Section B4.2(a)

$w = 2.000 - 2(0.125 + 0.030/2) = 1.720$ in.

$f = F_y = 50$ ksi

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/50} = 31.09$ (*Eq.* B4-1)

$w/t = 1.720/0.030 = 57.33 > 0.328S \ \therefore$ check effective width

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (\textit{Eq. } B4.2\text{-}10)$$

$$= 399(0.030)^4\left[\frac{57.33}{31.09} - 0.328\right]^3 \le (0.030)^4\left[115\frac{57.33}{31.09} + 5\right]$$

$= 0.00113\ \text{in.}^4 > 0.000176\ \text{in.}^4 \ \therefore I_a = 0.000176\ \text{in.}^4$

$I_s = (d^3 t \sin^2\theta)/12 = (0.260)^3(0.030)\sin^2(90°)/12 = 0.000044\ \text{in.}^4$ (*Eq.* B4-2)

$R_I = I_s/I_a \le 1$ (*Eq.* B4.2-9)

$= 0.000044 / 0.000176 = 0.25$

$$n = \left[0.582 - \frac{w/t}{4S}\right] \ge \frac{1}{3} \quad (\textit{Eq. } B4.2\text{-}11)$$

$$= \left[0.582 - \frac{57.33}{(4)(31.09)}\right] \ge \frac{1}{3}$$

$= 0.121 < 1/3 \ \therefore n = 1/3$

$D/w = 0.415/1.720 = 0.241 < 0.25$

$k = 3.57(0.25)^{\frac{1}{3}} + 0.43 = 2.68$ (From Table B4.2)

$$F_{cr} = 2.68\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{1.720}\right)^2 = 21.74\ \text{ksi} \quad (\textit{Eq. } B2.1\text{-}5)$$

$\lambda = \sqrt{\dfrac{f}{F_{cr}}} = \sqrt{\dfrac{50.0}{21.74}} = 1.517 > 0.673 \therefore$ 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)

$= (0.564)(1.720) = 0.970$ in.

$d_s = d'_s(R_I) = (0.260)(0.25) = 0.065$ in. (*Eq.* B4.2-7)

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	1.223	0.015	0.018	---	---
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 Σ	17.210		23.041	41.315	0.913

$\bar{y} = \Sigma Ly/\Sigma L = 23.041/17.210 = 1.339$ in.; below mid depth as assumed

$I_x = [\Sigma Ly^2 + \Sigma I'_x - \bar{y}^2 \Sigma L]t$

$= \left[41.315 + 0.913 - (1.339)^2(17.210)\right](0.030) = 0.341 \text{ in.}^4$

$S_{xt} = I_x/y_{cg} = 0.341/1.339 = 0.255 \text{ in.}^3$

$M_n = S_e F_y = 0.255(50) = 12.8$ kip-in (*Eq.* C3.1.1-1)

Element 5 from Section B2.3(a): check assumption that element is fully effective

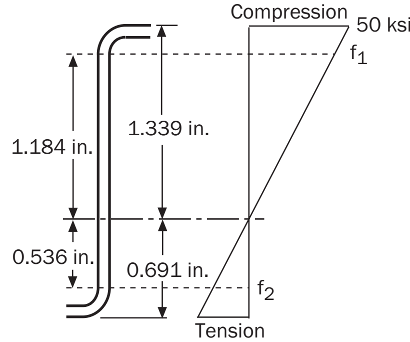
$y_{cg} = 1.339$ in.

$f_1 = [(1.339 - 0.125 - 0.030)/1.339](50)$

$= 44.21$ ksi

$f_2 = -[(2.030 - 0.125 - 0.030 - 1.339)/1.339](50)$

$= -20.01$ ksi



$\psi = |f_2/f_1| = |-20.01/44.21| = 0.453$ (*Eq.* B2.3-1)

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

$= 4 + 2(1+0.453)^3 + 2(1+0.453) = 13.04$

$f = f_1$

$w = 2.030 - (2)(0.125 + 0.030) = 1.720$ in.

$$F_{cr} = 13.04\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{1.720}\right)^2 = 105.8 \text{ ksi}$$ (*Eq.* B2.1-5)

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

$$\lambda = \sqrt{\frac{44.21}{105.8}} = 0.646 < 0.673$$

$b_e = w = 1.720$ in. (*Eq.* B2.1-1)

$h_o/b_o = 2.030/2.030 = 1.00 < 4.0$

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

$= 1.720/(3+0.453) = 0.498$ in.

For $\psi > 0.236$

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

$= 1.720/2 = 0.860$ in.

$w_c = 1.339 - 0.030 - 0.125 = 1.184$ in. (compression portion of web)

$b_1+b_2 = 0.498 + 0.860 = 1.358$ in. > 1.184 in.

Thus element 5 is fully effective so properties above are correct.

2. Moment of inertia for deflection determination - compression on top, $M_s = 0.6M_n = 7.68$ 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 I_{eff}, first approximation, 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 B4.2(a)

$w = 3.000 - 3(0.125 + 0.030/2) = 2.580$ in.

$f = 30$ ksi

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/30} = 40.14$ (*Eq.* B4-1)

$w/t = 2.580/0.030 = 86.0 > 0.328S \therefore$ 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.* B4.2-10)

$$= 399(0.030)^4\left[\frac{86.0}{40.14} - 0.328\right]^3 \le (0.030)^4\left[115\frac{86.0}{40.14} + 5\right]$$

I_a = 0.00193 in.^{4} > 0.000204 in.^{4} $\therefore$ I_a = 0.000204 in.^{4}

I_s = 0.000942 in.^{4}

R_I = $I_s/I_a \le 1$, by inspection, R_I = 1.0 (*Eq.* B4.2-9)

k = $3.57(1.0)^n + 0.43 = 4.0$ (From Table B4.2)

$$F_{cr} = 4.0\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{2.580}\right)^2 = 14.42 \text{ ksi} \quad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{30.0}{14.42}} = 1.442 > 0.673 \therefore \text{ element is subject to local buckling} \quad (Eq.\ B2.1\text{-}4)$$

$$\lambda_c = 0.256 + 0.328(w/t)\sqrt{F_y/E} \quad (Eq.\ B2.1\text{-}11)$$

$$= 0.256 + 0.328(2.580/0.030)\sqrt{50/29500} = 1.417$$

for $\lambda \ge \lambda_c$,

$$\rho = (0.41 + 0.59\sqrt{F_y/f_d} - 0.22/\lambda)/\lambda \quad (Eq.\ B2.1\text{-}10)$$

$$= (0.41 + 0.59\sqrt{50/30} - 0.22/1.442)/1.442 = 0.707$$

b = ρw (*Eq.* B2.1-2)

= (0.707)(2.580) = 1.824 in.

A_s = $A'_s(R_I)$ = (0.0348)(1.0) = 0.0348 in.^{2} (*Eq.* B4.2-8)

Elements 3 and 9 from Section B4.2(a)

w = 2.000 - 2(0.125 + 0.030/2) = 1.720 in.

f = 30 ksi

$$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/30} = 40.14 \quad (Eq.\ B4\text{-}1)$$

w/t = 1.720/0.030 = 57.33 > 0.328S $\therefore$ check effective width

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4.2\text{-}10)$$

$$= 399(0.030)^4\left[\frac{57.33}{40.14} - 0.328\right]^3 \le (0.030)^4\left[115\frac{57.33}{40.14} + 5\right]$$

= 0.000430 in.^{4} > 0.000137 in.^{4} $\therefore$ I_a = 0.000137 in.^{4}

R_I = I_s/I_a = 0.000044/0.000137 = 0.321 (*Eq.* B4.2-9)

$$n = \left[0.582 - \frac{57.33}{(4)(40.14)}\right] \ge \frac{1}{3} \quad (Eq.\ B4.2\text{-}11)$$

= 0.225 < 1/3 $\therefore$ n = 1/3

D/w = 0.415/1.72 = 0.241 < 0.25 $\therefore$

k = $3.57(0.321)^{\frac{1}{3}} + 0.43 = 2.87$ (From Table B4.2)

$$F_{cr} = 2.87\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{1.720}\right)^2 = 23.28 \text{ ksi} \quad (Eq.\ B2.1\text{-}5)$$

$\lambda \quad = \sqrt{\dfrac{f}{F_{cr}}} = \sqrt{\dfrac{30.0}{23.28}} = 1.135 > 0.673 \therefore$ element is subject to local buckling (*Eq.* B2.1-4)

$\lambda_c \quad = 0.256 + 0.328(1.720/0.030)\sqrt{50/29500} = 1.030$ (*Eq.* B2.1-11)

for $\lambda \geq \lambda_c$

$\rho \quad = (0.41 + 0.59\sqrt{50/30} - 0.22/1.135) / 1.135 = 0.862$ (*Eq.* B2.1-10)

$b \quad = \rho w = (0.862)(1.720) = 1.483$ in. (*Eq.* B2.1-2)

$d_s \quad = d'_s(R_I) = (0.260)(0.321) = 0.084$ in. (*Eq.* B4.2-7)

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	1.824	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	0.084	0.197	0.017	0.003	---
10	0.440	1.964	0.864	1.697	0.001
Sum Σ	18.343		23.062	41.316	0.913

$\bar{y} \quad = \Sigma Ly/\Sigma L = 23.062/18.343 = 1.257$ in.

$I_e \quad = [\Sigma Ly^2 + \Sigma I'_x - \bar{y}^2 \Sigma L]t$

$\quad = [41.316 + 0.913 - (1.257)^2(18.343)](0.030) = 0.397$ in.^{4}

$S_e \quad = I_e/\bar{y} = 0.397/1.257 = 0.316$ in.^{3}

$M_n \quad = S_e f = (0.316)(30) = 9.48$ kip-in $> M_s = 7.68$ kip-in.

For the second approximation, estimate the compression stress f in the top fibers of the section at M = 7.68 kip-in. by extrapolation:

M = 12.8 kip-in. at f = 50 ksi

M = 9.48 kip-in. at f = 30 ksi

for M = 7.68 kip-in.:

$(12.8 - 9.48)/(50 - 30) = (9.48 - 7.68)/(30 - f)$

f = 19.2 ksi

For the second approximation, assume a compression stress of f = 19.2 ksi in the top fiber of the section.

Element 2 from Section B4.2(a)

w = 2.580 in.

f = 19.2 ksi

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/19.2} = 50.17$ (*Eq.* B4-1)

$w/t = 2.580/0.030 = 86.0 > 0.328S \therefore$ check local buckling

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4.2\text{-}10)$$

$$I_a = 399(0.030)^4\left[\frac{86.0}{50.17} - 0.328\right]^3 \le (0.030)^4\left[115\frac{86.0}{50.17} + 5\right]$$

$= 0.000861\text{ in.}^4 > 0.000164\text{ in.}^4 \therefore I_a = 0.000164\text{ in.}^4$

$I_s = 0.000942\text{ in.}^4$

$R_I = I_s/I_a \le 1$, by inspection $R_I = 1.0$ (*Eq.* B4.2-9)

$k = 3.57(1.0)^n + 0.43 = 4.0$ (From Table B4.2)

$$F_{cr} = 4.0\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{2.580}\right)^2 = 14.42\text{ ksi} \quad (Eq.\ B2.1\text{-}5)$$

$\lambda = \sqrt{\frac{19.2}{14.42}} = 1.154 > 0.673 \therefore$ element is subject to local buckling (*Eq.* B2.1-4)

$\lambda_c = 1.417$ (from above)

for $0.673 < \lambda < \lambda_c$

$\rho = (1.358 - 0.461/\lambda) / \lambda$ (*Eq.* B2.1-9)

$= (1.358 - 0.461 /1.154)/1.154 = 0.831$

$b = \rho w = (0.831)(2.580) = 2.144$ in. (*Eq.* B2.1-2)

$A_s = A'_s(R_I) = (0.0348)(1.0) = 0.0348\text{ in.}^2$ (*Eq.* B4.2-8)

Elements 3 and 9 from Section B4.2(a)

$w = 1.720$ in.

$f = 19.2$ ksi

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/19.2} = 50.17$ (*Eq.* B4-1)

$w/t = 1.720/0.030 = 57.33 > 0.328S \therefore$ check local buckling

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4.2\text{-}10)$$

$$= 399(0.030)^4\left[\frac{57.33}{50.17} - 0.328\right]^3 \le (0.030)^4\left[115\frac{57.33}{50.17} + 5\right]$$

$= 0.000175\text{ in.}^4 > 0.000110\text{ in.}^4 \therefore I_a = 0.000110\text{ in.}^4$

$R_I = I_s/I_a = 0.000044 / 0.000110 = 0.400$ (*Eq.* B4.2-9)

$$n = \left[0.582 - \frac{57.33}{(4)(50.17)}\right] \ge \frac{1}{3} \quad (Eq.\ B4.2\text{-}11)$$

$= 0.296 < 1/3 \therefore n = 1/3$

$k = 3.57(0.400)^{\frac{1}{3}} + 0.43 = 3.06$ (From Table B4.2)

$F_{cr} = 3.06\dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{0.030}{1.720}\right)^2 = 24.82$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{f}{F_{cr}}} = \sqrt{\dfrac{19.2}{24.82}} = 0.880 > 0.673 \therefore$ element is subject to local buckling (*Eq.* B2.1-4)

$\lambda_c = 1.030$ (*Eq.* B2.1-11)

for $0.673 < \lambda < \lambda_c$

$\rho = (1.358 - 0.461/\lambda)/\lambda$ (*Eq.* B2.1-9)

$= (1.358 - 0.461/0.880)/0.880 = 0.948$

$b = \rho w = (0.948)(1.720) = 1.631$ in. (*Eq.* B2.1-2)

$d_s = d'_s(R_I) = (0.260)(0.400) = 0.104$ in. (*Eq.* B4.2-7)

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	2.144	0.015	0.032	---	---
3	1.631	0.015	0.024	---	---
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 Σ	18.831		23.074	41.317	0.913

$\bar{y} = \Sigma Ly/\Sigma L = 23.074/18.831 = 1.225$ in.

$I_e = [\Sigma Ly^2 + \Sigma I'x - \bar{y}^2\Sigma L]t$

$= [41.317 + 0.913 - (1.225)^2(18.831)](0.030) = 0.419$ in.^{4}

$S_e = I_e/\bar{y} = 0.419/1.225 = 0.342$ in.^{3}

$M_n = S_e f = (0.342)(19.2) = 6.57$ kip-in $< M_s = 7.68$ kip-in.

For the third approximation, estimate the compression stress f in the top fibers of the section at M = 7.68 kip-in. by interpolation:

$M = 9.48$ kip-in. at $f = 30$ ksi

$M = 6.57$ kip-in. at $f = 19.2$ ksi

for M =7.68 kip-in.:

$(9.48 - 6.57)/(30 - 19.2) = (9.48 - 7.68)/(30 - f)$

$f = 23.3$ ksi

Repeating the previous calculations (not shown) with f =23.3 ksi gives:

I_e = 0.409 in.^{4}

S_e = 0.330 in.^{3}

M = $S_e f$ = (0.330)(23.3) = 7.70 kip-in. ≅ 7.68 kip-in.

Therefore, the effective moment of inertia is:

I_e = 0.409 in.^{4}

3. Section modulus, S_e, for nominal flexural strength - compression on bottom

Since the neutral axis may be closer to the compression flange than to the tension flange, the compression stress 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 in.^{2} (from Example I-7)

A_s = (1.198)(0.030) = 0.0359 in.^{2}

b_o = 3.000 + 3.000 + 3.000 - (2)(0.125 + 0.030/2) = 8.720 in.

h = 2.030 - 2(0.125 + 0.030) = 1.720 in.

$I_{sp} = I'_x t + A_s y^2$

$= (0.0159)(0.030) + (0.0359)(0.380/2 - 0.030/2)^2 = 0.00158$ in.^{4}

$$\gamma = \frac{10.92 I_{sp}}{b_o t^3} \qquad (Eq.\ B5.1.1\text{-}4)$$

$$= \frac{(10.92)(0.00158)}{(8.720)(0.030)^3} = 73.3$$

$$\delta = \frac{A_s}{b_o t} \qquad (Eq.\ B5.1.1\text{-}5)$$

$$= \frac{0.0359}{(8.72)(0.030)} = 0.137$$

$$\beta = (1 + \gamma(n + 1))^{1/4} \qquad (Eq.\ B5.1.1\text{-}3)$$

$$= (1 + 73.3(2 + 1))^{1/4} = 3.86$$

$$k_d = \frac{(1 + \beta^2)^2 + \gamma(1 + n)}{\beta^2(1 + \delta(n + 1))} \qquad (Eq.\ B5.1.1\text{-}2)$$

$$= \frac{(1 + 3.86^2)^2 + 73.3(1 + 2)}{3.86^2(1 + 0.137(2 + 1))} = 22.5$$

$b_o/h = 8.720/1.72 = 5.07 > 1.0$, therefore:

$$R = \frac{11 - b_o/h}{5} \geq \frac{1}{2} \qquad (Eq.\ B5.1\text{-}8)$$

$$= \frac{11 - 8.720/1.720}{5} = 1.186$$

$$Rk_d = (1.186)(22.5) = 26.7$$

$$k_{loc} = 4(n + 1)^2 \qquad (Eq.\ B5.1.1\text{-}1)$$

$$= 4(2 + 1)^2 = 36.0$$

$k = \min(Rk_d, k_{loc}) = \min(26.7, 36.0) = 26.7 \ \therefore$ distortional buckling controls

$$F_{cr} = k\frac{\pi^2 E}{12(1 - \mu^2)}\left(\frac{t}{b_o}\right)^2 \qquad (Eq.\ B5.1\text{-}5)$$

$$= 26.7\frac{\pi^2(29500)}{12(1 - 0.3^2)}\left(\frac{0.030}{8.720}\right)^2 = 8.43 \text{ ksi}$$

$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{50}{8.43}} = 2.435 > 0.673 \ \therefore$ element is subject to buckling (*Eq.* B5.1-4)

$$\rho = (1 - 0.22/\lambda) / \lambda \qquad (Eq.\ B5.1\text{-}3)$$

$$= (1 - 0.22/2.435)/2.435 = 0.374$$

$$b_e = \rho\left(\frac{A_g}{t}\right) \qquad (Eq.\ B5.1\text{-}1)$$

$$= 0.374\left(\frac{0.278}{0.030}\right) = 3.466 \text{ in.}$$

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 Σ	13.726		11.739	18.837	0.882

$\bar{y} = \Sigma Ly/\Sigma L = 11.739/13.726 = 0.855$ in. < 1.015 in. (above centerline)

$y_c = 2.030 - 0.855 = 1.175$ in.

Therefore, compression stress controls as assumed.

I_e = $[\Sigma Ly^2 + \Sigma I'_x - \bar{y}^2\Sigma L]t$

= $\left[18.837 + 0.882 - (0.855)^2(13.726)\right](0.030) = 0.291$ in.^{4}

S_{et} = $I_e/\bar{y} = 0.291/0.855 = 0.340$ in.^{3}

S_{eb} = $0.291/1.175 = 0.248$ in.^{3}

M_n = $\min(S_{et}, S_{eb})F_y = 0.248(50) = 12.4$ kip-in. (*Eq.* C3.1.1-1)

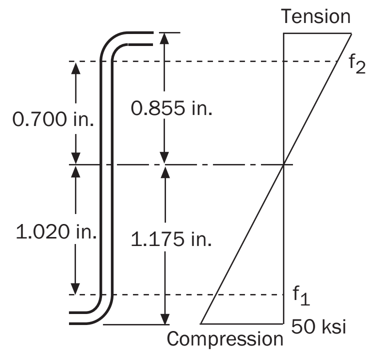
Element 5 from Section B2.3(a): check assumption that element is fully effective

f_1 = $[(1.175 - 0.125 - 0.030)/1.175](50)$

= 43.40 ksi

f_2 = $-[(0.855 - 0.125 - 0.030)/1.175](50)$

= -29.79 ksi



ψ = $|f_2/f_1| = |-29.79/43.40| = 0.686$ (*Eq.* B2.3-1)

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

= $4 + 2(1+ 0.686)^3 + 2(1 +0.686) = 16.96$

f = f_1

w = 1.720 in.

F_{cr} = $16.96\dfrac{\pi^2(29500)}{12(1 - 0.3^2)}\left(\dfrac{0.030}{1.720}\right)^2 = 137.6$ ksi (*Eq.* B2.1-5)

λ = $\sqrt{\dfrac{f}{F_{cr}}}$ (*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.

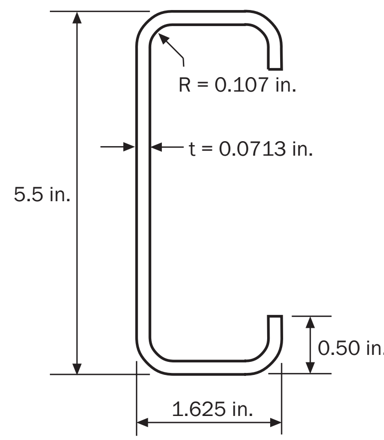
w_c = $1.175 - 0.030 - 0.125 = 1.020$ in. (compression portion of web)

$b_1+b_2 = 0.467 + 0.860 = 1.327$ in. > 1.020 in.

Thus element 5 is fully effective so properties above are correct.

3.7 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: SSMA Stud 550S162-68 as shown above
4. Section to be used as a beam and is fully braced against lateral-torsional buckling

Required:

Determine the nominal flexural strength, M_n, considering the increase in strength resulting from the cold work of forming, using the provisions of Section A7.2.

Solution:

1. Check the limitations

In order to use Eq. A7.2-1 for computing the average tensile yield point for the beam flange, the geometry of the section and yield point 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}

$$B_c = 3.69(F_{uv}/F_{yv}) - 0.819\,(F_{uv}/F_{yv})^2 - 1.79 \qquad (Eq.\ A7.2\text{-}3)$$

$$= 3.69(1.36) - 0.819(1.36)^2 - 1.79 = 1.714$$

$$m = 0.192(F_{uv}/F_{yv}) - 0.068 \qquad (Eq.\ A7.2\text{-}4)$$

$$= 0.192(1.36) - 0.068 = 0.193$$

$$F_{yc} = B_cF_{yv}/(R/t)^m \qquad (Eq.\ A7.2\text{-}2)$$

$$= 1.714(33)/(1.50)^{0.193} = 52.31 \text{ ksi}$$

3. Calculation of F_{ya}

$r = R + t/2 = 0.107 + 0.0713/2 = 0.143$ in.

Cross-sectional area of corner = $(\pi/2)(0.143)(0.0713) = 0.0160$ in.^{2}

Total corner cross-sectional area of the controlling flange

$= (0.0160)(2) = 0.0320$ 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

$A_{flange} = 0.0320 + (1.268)(0.0713) = 0.122$ in.^{2}

$C = 0.0320/0.122 = 0.262$

$F_{ya} = CF_{yc} + (1 - C)F_{yf}$ (*Eq.* A7.2-1)

$= (0.262)(52.31) + (1- 0.262)(33) = 38.06$ 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.

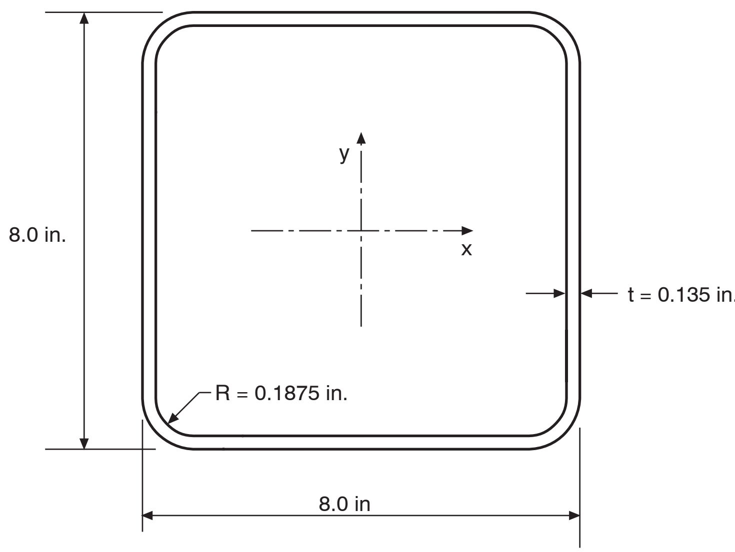
5. Calculation of M_n

$M_n = S_eF_y$ (*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 midspan
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 = [8.0 - (2)(0.135)]/2 = 3.865 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 $0.71\left[8.0 - (2)(0.135)\right] = 5.488$ in.

The effective flat width of the flanges is:

b_{max} = 5.488 - 2R = 5.488 - (2)(0.1875) = 5.113 in.

Recalculate properties using effective compression and tension flange widths of 5.113 in. (governed by Section B1.1).

I_x = 33.43 in.^{4}

S_e = 8.36 in.^{3}

M_n = (8.36)(50) = 418 kip-in. CONTROLS

3. Determination of the ASD flexural allowable design strength:

$M \leq M_n/\Omega_b$ (*Eq.* A4.1.1-1)

$\Omega_b = 1.67$

$M \leq 418/1.67 = 250$ kip-in.

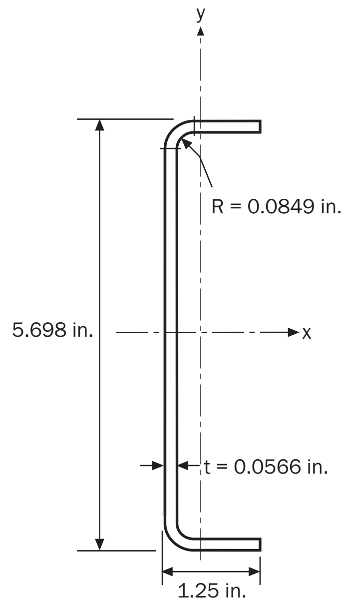
4. Determination of the LRFD flexural design strength:

$M_u \leq \phi_b M_n$ (*Eq.* A5.1.1-1)

$\phi_b = 0.95$

$M_u \leq (0.95)(418) = 397$ kip-in.

Example I-17: Flange Curling



Given:

1. Steel: F_y = 33 ksi
2. Section: SSMA Track 550T125-54 as shown above
3. Compression flange braced against lateral buckling

Required:

Determine the amount of curling of the compression flange at a maximum flexural compressive stress of 30.93 ksi, as used in Example II-3.

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.

$$w_f = \sqrt{0.061tdE/f_{av}}\ \sqrt[4]{(100c_f/d)} \qquad (Eq.\ B1.1\text{-}1)$$

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.

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PART II
BEAM DESIGN
FOR USE WITH THE
2001 EDITION OF THE
NORTH AMERICAN
SPECIFICATION FOR THE DESIGN OF
COLD-FORMED STEEL STRUCTURAL MEMBERS

PART II - BEAM DESIGN

SECTION 1 - BENDING

1.1 Notes On The Tables

(a) With the exception of the SSMA 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* or the information on laterally unbraced compression flanges in Part V should also be taken into account where applicable.

(d) Tabulated section properties are shown to three significant figures, while 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 to II-6 inclusive were computed using the yield points listed in the tables, except where a value of F_{ya} is given in Tables II-2 and II-3, the provisions of Section A7 of the *Specification* for strength increase from cold work of forming have been used. This increased strength has also been considered in Tables II-8a and II-8b. Sections were considered eligible for the cold work of forming increase in yield point 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 values in the columns labeled M_{web}, M_{flange} and M_{lip} in Tables II-1 through II-5 are the highest nominal moments at which the web, flange and lip (if applicable) respectively are fully effective. These values may be used to determine if each of the elements is fully effective at a given nominal moment. These values are only meaningful where they do not exceed M_{nxo} for the section and yield point in question.

(i) Tables II-7a, II-8a and II-9a incorporate factors of safety and are valid for ASD use only. Tables II-7b, II-8b and II-9b incorporate resistance factors and are valid for LRFD use only. The values given in all other tables must be modified by factors of safety (ASD) or resistance factors (LRFD). See the appropriate *Specification* sections for more information.

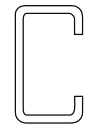
(j) The effects of standard factory punchouts in SSMA studs have been included in Tables II-2, II-8a, and II-8b. These punchouts are considered in SSMA studs with flange widths of 1.625 in. or less. Standard punchout sizes are 1.5 in. by 4.5 in. for sections with depths of 3.5 inches or more and 0.75 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 "*".

(k) 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 point. Dashes in other columns indicate that the section is not available in the listed grade of steel.

1.2 Beam Property Tables

Table II - 1

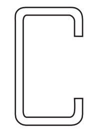
Beam Properties
C-Sections With Lips



Section	F_y = 33 ksi				F_y = 55 ksi				Maximum 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.	M_{lip} kip-in.
12CS4x105	14.4	246	7.45	47.5	14.4	368	6.70	46.3	414	234	159
12CS4x085	7.63	187	5.65	38.1	7.63	264	4.79	36.5	230	135	97.9
12CS4x070	4.25	143	4.32	31.0	4.25	189	3.44	29.6	134	81.8	64.3
12CS3.5x105	14.4	234	7.10	43.8	14.4	358	6.51	43.4	423	286	201
12CS3.5x085	7.63	183	5.53	35.6	7.63	261	4.74	34.9	235	166	123
12CS3.5x070	4.25	142	4.29	29.2	4.25	188	3.42	28.2	138	101	80.8
12CS2.5x105	14.4	196	5.93	36.3	14.4	309	5.62	36.0	156	425	353
12CS2.5x085	7.63	152	4.61	29.4	7.63	232	4.22	28.8	82.3	244	210
12CS2.5x070	4.25	120	3.64	23.8	4.25	169	3.07	23.4	45.8	148	134
10CS4x105	15.1	192	5.82	31.0	17.5	286	5.19	30.3	404	183	125
10CS4x085	9.25	145	4.40	25.0	9.25	220	4.01	23.5	224	106	76.8
10CS4x070	5.15	115	3.48	20.2	5.15	158	2.87	19.1	131	64.2	50.5
10CS4x065	4.12	105	3.18	18.4	4.12	140	2.54	17.7	107	53.1	43.3
10CS3.5x105	15.1	183	5.54	28.5	17.5	277	5.04	28.2	417	223	157
10CS3.5x085	9.25	142	4.31	23.1	9.25	214	3.89	22.6	232	129	96.3
10CS3.5x070	5.15	111	3.38	19.0	5.15	158	2.86	18.1	136	79.1	63.1
10CS3.5x065	4.12	102	3.10	17.6	4.12	139	2.53	16.7	112	65.7	54.0
10CS2.5x105	15.1	154	4.66	23.3	17.5	256	4.66	23.3	450	373	298
10CS2.5x085	9.25	125	3.79	19.0	9.25	204	3.70	19.0	251	217	180
10CS2.5x070	5.15	104	3.14	15.7	5.15	153	2.78	15.7	147	133	117
10CS2.5x065	4.12	96.2	2.92	14.6	4.12	134	2.43	14.6	121	110	99.8
10CS2x105	15.1	137	4.15	20.7	17.5	221	4.02	20.7	159	461	434
10CS2x085	9.25	109	3.29	16.9	9.25	172	3.13	16.7	83.7	267	256
10CS2x070	5.15	86.3	2.61	13.9	5.15	136	2.47	13.6	46.5	163	161
10CS2x065	4.12	78.9	2.39	12.8	4.12	124	2.26	12.5	37.2	134	136
9CS2.5x105	15.1	133	4.02	18.1	19.5	221	4.02	18.1	439	321	257
9CS2.5x085	9.88	108	3.27	14.7	10.3	176	3.19	14.7	243	187	156
9CS2.5x070	5.76	89.4	2.71	12.2	5.76	136	2.47	12.2	143	114	101
9CS2.5x065	4.60	83.1	2.52	11.3	4.60	121	2.21	11.3	116	95.0	86.2
9CS2.5x059	3.44	74.7	2.26	10.3	3.44	104	1.89	10.3	90.2	74.6	70.3
8CS4x105	15.1	143	4.34	18.6	19.5	211	3.83	18.1	389	137	93.1
8CS4x085	9.88	108	3.26	15.0	11.7	162	2.95	14.1	216	79.2	57.5
8CS4x070	6.52	84.8	2.57	12.1	6.52	127	2.31	11.3	126	48.1	37.9
8CS4x065	5.22	77.4	2.34	11.1	5.22	112	2.03	10.4	103	39.8	32.4
8CS4x059	3.90	68.7	2.08	9.93	3.90	95.1	1.73	9.41	79.6	31.2	26.6

Table II - 1 (continued)

Beam Properties
C-Sections With Lips



Section	F_y = 33 ksi				F_y = 55 ksi				Maximum 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.	M_{lip} kip-in.
8CS3.5x105	15.1	136	4.11	16.9	19.5	205	3.72	16.8	405	165	117
8CS3.5x085	9.88	105	3.19	13.8	11.7	157	2.86	13.5	225	96.2	71.7
8CS3.5x070	6.52	82.3	2.49	11.4	6.52	124	2.26	10.7	133	58.9	47.1
8CS3.5x065	5.22	75.3	2.28	10.5	5.22	112	2.04	9.84	109	48.9	40.3
8CS3.5x059	3.90	67.2	2.04	9.41	3.90	95.5	1.74	8.90	83.9	38.4	33.0
8CS2.5x105	15.1	113	3.41	13.6	19.5	188	3.41	13.6	431	272	218
8CS2.5x085	9.88	91.8	2.78	11.1	11.7	149	2.71	11.1	240	159	132
8CS2.5x070	6.52	76.0	2.30	9.21	6.52	115	2.10	9.21	142	97.2	85.9
8CS2.5x065	5.22	70.7	2.14	8.57	5.22	105	1.91	8.57	117	80.7	73.3
8CS2.5x059	3.90	63.5	1.92	7.79	3.90	93.0	1.69	7.79	90.3	63.4	59.8
8CS2x105	15.1	99.1	3.00	12.0	19.5	165	3.00	12.0	465	379	351
8CS2x085	9.88	80.8	2.45	9.79	11.7	135	2.45	9.79	261	222	210
8CS2x070	6.52	66.9	2.03	8.11	6.52	111	2.03	8.11	155	137	135
8CS2x065	5.22	62.2	1.89	7.54	5.22	104	1.89	7.54	127	113	115
8CS2x059	3.90	56.6	1.72	6.86	3.90	93.3	1.70	6.86	98.6	85.3	92.7
7CS4x105	13.3	121	3.66	13.7	19.5	176	3.21	13.4	380	115	78.5
7CS4x085	9.88	90.4	2.74	11.1	12.8	135	2.46	10.4	210	66.8	48.5
7CS4x070	6.70	71.0	2.15	8.97	7.53	106	1.93	8.32	123	40.6	32.0
7CS4x065	5.78	64.8	1.96	8.17	6.02	96.7	1.76	7.64	101	33.6	27.4
7CS4x059	4.49	57.4	1.74	7.34	4.49	83.1	1.51	6.88	77.4	26.3	22.5
7CS2.5x105	13.3	93.7	2.84	9.94	19.5	156	2.84	9.94	429	226	181
7CS2.5x085	9.88	76.5	2.32	8.11	12.8	124	2.26	8.11	239	132	110
7CS2.5x070	6.70	63.4	1.92	6.72	7.53	95.8	1.74	6.72	141	81.0	71.6
7CS2.5x065	5.78	59.0	1.79	6.25	6.02	87.3	1.59	6.25	116	67.3	61.2
7CS2.5x059	4.49	53.0	1.61	5.69	4.49	78.4	1.42	5.69	89.9	52.9	49.9
6CS4x105	11.3	99.2	3.01	9.64	18.8	144	2.62	9.44	368	94.7	64.5
6CS4x085	9.18	74.1	2.25	7.81	12.8	110	2.00	7.34	204	55.0	40.0
6CS4x070	6.70	58.1	1.76	6.36	8.65	86.1	1.57	5.87	119	33.5	26.4
6CS4x065	5.78	52.9	1.60	5.78	7.12	78.5	1.43	5.40	97.3	27.8	22.6
6CS4x059	4.76	46.9	1.42	5.19	5.31	69.5	1.26	4.83	74.9	21.7	18.5
6CS2.5x105	11.3	76.0	2.30	6.91	18.8	127	2.30	6.91	425	184	147
6CS2.5x085	9.18	62.1	1.88	5.65	12.8	101	1.83	5.65	236	107	89.5
6CS2.5x070	6.70	51.5	1.56	4.69	8.65	77.7	1.41	4.69	140	65.9	58.3
6CS2.5x065	5.78	48.0	1.45	4.36	7.12	70.7	1.29	4.36	115	54.8	49.8
6CS2.5x059	4.76	43.2	1.31	3.97	5.31	63.4	1.15	3.97	88.7	43.1	40.6

Table II - 1 (continued)

Beam Properties
C-Sections With Lips

Section	F_y = 33 ksi				F_y = 55 ksi				Maximum 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.	M_{lip} kip-in.
4CS4x105	7.10	59.9	1.81	3.87	11.8	85.7	1.56	3.81	340	57.3	38.8
4CS4x085	5.82	44.5	1.35	3.16	9.69	65.2	1.19	2.98	187	33.4	24.2
4CS4x070	4.83	34.7	1.05	2.60	8.05	50.6	0.921	2.38	109	20.4	16.0
4CS4x065	4.50	31.6	0.956	2.35	7.46	46.0	0.837	2.19	88.5	16.9	13.7
4CS4x059	4.10	27.9	0.844	2.11	6.15	40.6	0.739	1.95	68.0	13.3	11.3
4CS2.5x105	7.10	44.1	1.34	2.67	11.8	73.5	1.34	2.67	415	107	85.3
4CS2.5x085	5.82	36.3	1.10	2.20	9.69	59.1	1.08	2.20	228	62.9	52.3
4CS2.5x070	4.83	30.3	0.917	1.83	8.05	45.4	0.825	1.83	134	38.8	34.2
4CS2.5x065	4.50	28.2	0.855	1.71	7.46	41.3	0.750	1.71	109	32.3	29.3
4CS2.5x059	4.10	25.5	0.771	1.56	6.15	36.9	0.671	1.56	84.3	25.4	23.9
4CS2x105	7.10	37.9	1.15	2.30	11.8	63.2	1.15	2.30	430	137	155
4CS2x085	5.82	31.3	0.947	1.89	9.69	52.1	0.947	1.89	242	80.2	93.4
4CS2x070	4.83	26.1	0.791	1.58	8.05	43.5	0.791	1.58	145	47.5	55.8
4CS2x065	4.50	24.3	0.737	1.47	7.46	40.4	0.734	1.47	119	38.8	45.9
4CS2x059	4.10	22.2	0.673	1.35	6.15	36.1	0.656	1.35	93.3	29.6	35.2

Notes:

1. Shear and moment strengths given are nominal strengths. To obtain the design strength, these values must be modified by factors of safety (ASD) or resistance factors (LRFD).

2. M_{web}, M_{flange} and M_{lip} are the highest nominal moments at which the web, flange and lip, respectively, are fully effective.

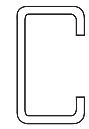
Table II - 2

Beam Properties
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi, F_u = 45 ksi					F_y = 50 ksi, F_u = 65 ksi					Maximum Effective Moment [2]		
	V_n [1] kips	F_{ya} [3] ksi.	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	F_{ya} [3] ksi	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.	M_{lip} kip-in.
1200S250-97	13.0	-	181	5.50	34.0	13.0	-	252	5.04	33.8	134	310	203
1200S250-68	4.43	-	115	3.50	24.0	4.43	-	150	3.01	23.6	45.7	127	97.7
1200S250-54*	2.20	-	81.9	2.48	18.8	2.20	-	107	2.15	18.4	22.7	72.2	63.5
1200S200-97	13.0	-	162	4.90	30.4	13.0	-	233	4.66	30.2	120	419	299
1200S200-68	4.43	-	106	3.22	21.4	4.43	-	148	2.96	20.9	41.0	167	138
1200S200-54*	2.20	-	80.0	2.43	16.7	2.20	-	104	2.07	16.3	20.4	94.4	87.6
1200S162-97	11.9	-	143	4.33	27.0	11.9	-	205	4.09	26.7	107	491	335
1200S162-68	4.43	-	92.9	2.82	19.0	4.43	-	132	2.65	18.4	36.5	189	142
1200S162-54*	2.20	-	69.6	2.11	14.7	2.20	-	95.7	1.91	14.3	18.2	105	86.5
1000S250-97	14.1	37.72	164	4.36	21.8	15.8	56.17	235	4.18	21.8	371	268	163
1000S250-68	5.35	36.36	110	3.03	15.8	5.35	-	138	2.77	15.7	135	113	82.5
1000S250-54	2.66	-	75.1	2.28	12.7	2.66	-	94.0	1.88	12.7	72.5	65.2	55.7
1000S250-43*	1.34	-	53.4	1.62	10.2	-	-	-	-	-	41.5	37.8	38.7
1000S200-97	14.1	-	128	3.87	19.3	15.8	-	187	3.74	19.3	136	342	242
1000S200-68	5.35	-	86.0	2.61	13.9	5.35	-	121	2.42	13.7	46.0	137	112
1000S200-54	2.66	-	65.5	1.98	11.0	2.66	-	85.3	1.71	10.8	22.8	77.5	71.7
1000S200-43*	1.34	-	48.5	1.47	8.60	-	-	-	-	-	11.5	42.3	46.8
1000S162-97	10.3	-	112	3.39	17.0	11.5	-	163	3.27	17.0	119	401	271
1000S162-68	5.35	-	75.1	2.28	12.3	5.35	-	108	2.16	12.0	40.5	155	116
1000S162-54	2.66	-	56.8	1.72	9.63	2.66	-	78.6	1.57	9.39	20.2	86.1	70.8
1000S162-43*	1.34	-	43.0	1.30	7.52	-	-	-	-	-	10.2	49.2	45.4
800S250-97	14.1	37.72	120	3.19	12.8	17.4	56.17	172	3.05	12.8	362	195	119
800S250-68	6.75	36.36	80.7	2.22	9.26	6.75	-	103	2.06	9.24	131	82.9	60.6
800S250-54	3.35	-	56.5	1.71	7.47	3.35	-	76.3	1.53	7.38	71.0	47.9	41.0
800S250-43	1.68	-	43.3	1.31	6.02	-	-	-	-	-	40.9	27.8	28.5
800S200-97	14.1	38.83	109	2.80	11.2	17.4	57.61	161	2.80	11.2	382	281	186
800S200-68	6.75	37.16	75.6	2.04	8.14	6.75	55.44	109	1.96	8.14	138	116	90.6
800S200-54	3.35	36.34	59.7	1.64	6.57	3.35	-	74.9	1.50	6.57	74.1	66.3	60.1
800S200-43	1.68	-	42.7	1.29	5.30	-	-	-	-	-	42.4	36.3	40.9
800S200-33*	0.758	-	26.8	0.813	4.10	-	-	-	-	-	19.3	16.4	22.2
800S162-97	7.72	40.08	97.3	2.43	9.71	9.50	-	121	2.43	9.71	139	311	208
800S162-68	5.39	-	57.3	1.74	7.09	5.39	-	83.2	1.66	7.07	46.8	121	89.3
800S162-54	3.35	-	44.0	1.33	5.70	3.35	-	61.4	1.23	5.60	23.2	67.4	55.2
800S162-43	1.68	-	33.6	1.02	4.50	-	-	-	-	-	11.7	38.6	35.6
800S162-33*	0.758	-	23.4	0.710	3.38	-	-	-	-	-	5.29	19.5	21.5

Table II - 2 (continued)

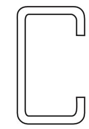
Beam Properties
SSMA Studs
C-Sections With Lips



Section	F_y = 33 ksi, F_u = 45 ksi					F_y = 50 ksi, F_u = 65 ksi					Maximum Effective Moment [2]		
	V_n [1] kips	F_{ya} [3] ksi.	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	F_{ya} [3] ksi	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.	M_{lip} kip-in.
800S137-97	7.72	41.25	88.7	2.15	8.60	9.50	-	107	2.15	8.60	123	330	213
800S137-68	5.39	-	50.9	1.54	6.30	5.39	-	73.4	1.47	6.29	41.6	127	82.7
800S137-54	3.35	-	38.9	1.18	5.08	3.35	-	54.2	1.08	4.97	20.7	69.7	48.6
800S137-43	1.68	-	29.6	0.896	4.00	-	-	-	-	-	10.4	39.2	30.1
800S137-33*	0.758	-	20.5	0.622	3.00	-	-	-	-	-	4.72	20.6	17.6
600S250-97	11.1	37.72	81.5	2.16	6.50	16.8	56.17	116	2.06	6.50	354	132	80.9
600S250-68	6.96	36.36	54.8	1.51	4.73	8.56	-	69.3	1.39	4.72	126	56.3	41.3
600S250-54	4.38	-	38.2	1.16	3.82	4.52	-	53.4	1.07	3.77	67.9	32.7	28.0
600S250-43	2.27	-	30.3	0.918	3.08	-	-	-	-	-	38.9	19.0	19.5
600S200-97	11.1	38.83	72.6	1.87	5.61	16.8	57.61	108	1.87	5.61	373	187	125
600S200-68	6.96	37.16	50.8	1.37	4.10	8.56	55.44	73.0	1.32	4.10	132	77.5	60.9
600S200-54	4.38	36.34	40.2	1.11	3.32	4.52	-	50.8	1.02	3.32	71.0	44.6	40.5
600S200-43	2.27	-	28.8	0.873	2.68	-	-	-	-	-	41.1	24.5	27.6
600S200-33	1.02	-	20.4	0.618	2.06	-	-	-	-	-	19.8	11.1	16.1
600S162-97	4.02	40.08	64.1	1.60	4.80	6.09	59.24	94.7	1.60	4.80	384	228	143
600S162-68	3.74	38.07	44.7	1.17	3.52	4.61	56.63	65.9	1.16	3.53	131	91.7	63.7
600S162-54	3.02	37.07	35.3	0.954	2.86	3.12	55.32	50.6	0.916	2.86	67.4	51.9	40.6
600S162-43	1.98	36.30	27.9	0.767	2.32	-	-	-	-	-	36.7	30.1	27.1
600S162-33	1.02	-	19.1	0.577	1.79	-	-	-	-	-	19.5	15.2	17.2
600S137-97	4.02	41.25	57.6	1.40	4.19	6.09	60.78	84.9	1.40	4.19	150	234	148
600S137-68	3.74	38.94	40.2	1.03	3.09	4.61	-	51.5	1.03	3.09	50.8	91.5	58.3
600S137-54	3.02	-	27.5	0.832	2.52	3.12	-	38.9	0.777	2.52	25.0	50.5	34.6
600S137-43	1.98	-	21.3	0.645	2.04	-	-	-	-	-	12.6	28.5	21.6
600S137-33	1.02	-	15.0	0.455	1.55	-	-	-	-	-	5.70	15.0	12.8
550S162-68	3.29	38.07	39.6	1.04	2.86	4.05	56.63	58.4	1.03	2.86	129	81.2	56.4
550S162-54	2.67	37.07	31.3	0.845	2.32	3.01	55.32	44.9	0.811	2.32	66.5	45.9	36.0
550S162-43	1.92	36.30	24.7	0.681	1.88	-	-	-	-	-	36.5	26.7	24.1
550S162-33	1.12	-	16.9	0.512	1.46	-	-	-	-	-	19.4	13.5	15.2
400S200-68	5.14	37.16	29.5	0.795	1.59	7.79	55.44	42.4	0.766	1.59	125	45.1	35.4
400S200-54	4.17	36.34	23.5	0.646	1.29	5.39	-	29.4	0.589	1.29	65.5	26.1	23.6
400S200-43	2.78	-	16.8	0.509	1.05	-	-	-	-	-	37.0	14.3	16.1
400S200-33	1.56	-	12.0	0.363	0.805	-	-	-	-	-	19.1	6.49	9.50
400S162-68	1.43	-	21.7	0.658	1.33	2.17	-	32.4	0.648	1.32	0.00	48.7	35.0
400S162-54	1.51	-	17.3	0.526	1.08	1.96	-	24.9	0.498	1.07	0.00	27.5	22.1
400S162-43	1.30	-	13.7	0.417	0.870	-	-	-	-	-	0.00	16.0	14.6
400S162-33	0.952	-	9.86	0.299	0.669	-	-	-	-	-	0.00	8.12	9.03

Table II - 2 (continued)

Beam Properties
SSMA Studs
C-Sections With Lips



Section	F_y = 33 ksi, F_u = 45 ksi					F_y = 50 ksi, F_u = 65 ksi					Maximum Effective Moment [2]		
	V_n [1] kips	F_{ya} [3] ksi.	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	F_{ya} [3] ksi	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.	M_{lip} kip-in.
400S137-68	1.43	-	18.7	0.568	1.14	2.17	-	27.9	0.558	1.14	0.00	50.5	31.7
400S137-54	1.51	-	15.0	0.453	0.934	1.96	-	21.4	0.428	0.929	0.00	28.1	19.1
400S137-43	1.30	-	11.8	0.359	0.755	-	-	-	-	-	0.00	16.1	12.1
400S137-33	0.952	-	8.55	0.259	0.580	-	-	-	-	-	0.00	8.57	7.25
362S200-68	4.61	37.16	25.9	0.698	1.27	6.99	55.44	37.3	0.673	1.27	123	39.6	31.1
362S200-54	3.75	36.34	20.6	0.568	1.03	5.39	-	25.9	0.517	1.03	64.0	22.9	20.8
362S200-43	2.78	-	14.8	0.448	0.836	-	-	-	-	-	36.0	12.6	14.2
362S200-33	1.64	-	10.5	0.319	0.642	-	-	-	-	-	18.9	5.72	8.40
362S162-68	1.06	-	19.1	0.579	1.05	1.61	-	28.7	0.574	1.05	0.00	43.4	31.0
362S162-54	1.13	-	15.4	0.467	0.857	1.63	-	22.2	0.444	0.854	0.00	24.6	19.7
362S162-43	1.08	-	12.3	0.372	0.694	-	-	-	-	-	0.00	14.3	13.0
362S162-33	0.834	-	8.84	0.268	0.535	-	-	-	-	-	0.00	7.27	8.08
362S137-68	1.06	-	16.4	0.498	0.902	1.61	-	24.7	0.493	0.902	0.00	45.0	28.1
362S137-54	1.13	-	13.3	0.402	0.740	1.63	-	19.1	0.381	0.737	0.00	25.1	16.9
362S137-43	1.08	-	10.6	0.320	0.600	-	-	-	-	-	0.00	14.4	10.8
362S137-33	0.834	-	7.66	0.232	0.463	-	-	-	-	-	0.00	7.68	6.49
350S162-68	0.947	-	18.2	0.551	0.965	1.44	-	27.5	0.549	0.965	0.00	41.6	29.7
350S162-54	1.01	-	14.7	0.447	0.789	1.52	-	21.3	0.426	0.787	0.00	23.6	18.9
350S162-43	1.01	-	11.8	0.357	0.640	-	-	-	-	-	0.00	13.7	12.5
350S162-33	0.779	-	8.49	0.257	0.494	-	-	-	-	-	0.00	6.98	7.76
250S162-68	0.548	38.07	13.7	0.360	0.450	0.830	56.63	20.2	0.357	0.450	121	28.1	19.5
250S162-54	0.596	37.07	11.0	0.296	0.370	0.903	55.32	15.7	0.284	0.370	59.5	16.1	12.6
250S162-43	0.631	36.30	8.73	0.240	0.302	-	-	-	-	-	31.4	9.43	8.50
250S162-33	0.638	-	5.94	0.180	0.235	-	-	-	-	-	16.3	4.80	5.38
250S137-68	0.548	38.94	12.0	0.309	0.386	0.830	57.76	17.8	0.308	0.386	126	29.3	17.7
250S137-54	0.596	37.78	9.62	0.255	0.318	0.903	56.24	13.7	0.244	0.318	60.1	16.7	10.8
250S137-43	0.631	36.88	7.56	0.205	0.261	-	-	-	-	-	30.6	9.68	7.01
250S137-33	0.638	-	5.20	0.158	0.203	-	-	-	-	-	15.2	5.21	4.31

Notes:

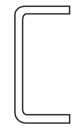
* Web h/t > 200, therefore bearing stiffeners are required.

1. Shear and moment strengths given are nominal strengths. To obtain design strengths, these values must be modified by factors of safety (ASD) or resistance factors (LRFD).

2. M_{web}, M_{flange} and M_{lip} are the highest nominal moments at which the web, flange and lip, respectively, are fully effective.

3. Where values are given for F_{ya}, flexural strength is calculated using the strength increase from cold working, based on an average yield point of F_{ya}.

Table II - 3

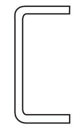


Beam Properties
SSMA Tracks
C-Sections Without Lips

Section	F_y = 33 ksi, F_u = 45 ksi				F_y = 50 ksi, F_u = 65 ksi				Maximum 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.
1200T200-97	12.6	142	4.30	29.8	12.6	191	3.82	29.0	104	85.1
1200T200-68	4.34	78.7	2.38	19.3	4.34	103	2.06	18.0	34.8	27.0
1200T200-54*	2.17	52.2	1.58	14.1	2.17	67.5	1.35	13.0	17.2	13.0
1200T150-97	12.6	132	4.00	26.0	12.6	181	3.62	25.7	93.8	137
1200T150-68	4.34	75.3	2.28	17.6	4.34	99.3	1.99	16.6	31.8	42.8
1200T150-54*	2.17	50.5	1.53	13.0	2.17	65.7	1.31	12.0	15.8	20.5
1200T125-97	12.6	122	3.69	24.1	12.6	172	3.44	23.7	86.9	184
1200T125-68	4.34	72.8	2.21	16.2	4.34	96.7	1.93	15.7	29.5	56.2
1200T125-54*	2.17	49.2	1.49	12.3	2.17	64.3	1.29	11.5	14.7	26.6
1000T200-97	14.1	113	3.43	19.0	15.2	154	3.08	18.6	108	65.0
1000T200-68	5.22	63.9	1.94	12.5	5.22	84.2	1.68	11.8	35.9	20.6
1000T200-54	2.60	42.8	1.30	9.23	2.60	55.6	1.11	8.56	17.7	9.94
1000T200-43*	1.32	28.4	0.861	6.72	-	-	-	-	8.84	4.90
1000T150-97	14.1	104	3.17	16.4	15.2	145	2.90	16.4	103	109
1000T150-68	5.22	60.9	1.85	11.3	5.22	81.1	1.62	10.8	34.7	33.8
1000T150-54	2.60	41.2	1.25	8.43	2.60	53.9	1.08	7.88	17.1	16.0
1000T150-43*	1.32	27.6	0.837	6.19	-	-	-	-	8.56	7.83
1000T125-97	14.1	95.9	2.91	15.1	15.2	138	2.75	15.1	94.7	147
1000T125-68	5.22	58.8	1.78	10.5	5.22	78.7	1.58	10.2	32.1	44.9
1000T125-54	2.60	40.1	1.22	7.96	2.60	52.8	1.06	7.48	15.9	21.2
1000T125-43*	1.32	27.0	0.819	5.88	-	-	-	-	7.99	10.3
800T200-97	14.1	82.2	2.49	11.2	17.4	117	2.35	10.8	112	47.3
800T200-68	6.54	49.2	1.49	7.30	6.54	65.5	1.31	7.05	37.0	15.0
800T200-54	3.26	33.3	1.01	5.50	3.26	43.6	0.872	5.15	18.1	7.23
800T200-43	1.65	22.3	0.676	4.04	-	-	-	-	9.04	3.57
800T200-33*	0.744	14.0	0.424	2.79	-	-	-	-	4.09	1.60
800T150-97	14.1	74.9	2.27	9.48	17.4	110	2.19	9.48	111	78.6
800T150-68	6.54	46.6	1.41	6.53	6.54	62.8	1.26	6.36	36.5	24.2
800T150-54	3.26	32.0	0.969	4.97	3.26	42.2	0.844	4.69	17.9	11.5
800T150-43	1.65	21.6	0.655	3.69	-	-	-	-	8.92	5.61
800T150-33*	0.744	13.7	0.414	2.57	-	-	-	-	4.03	2.52
800T125-97	14.1	77.8 [3]	2.06	8.61	17.4	103 [5]	2.06	8.61	107	111
800T125-68	6.54	44.8	1.36	6.00	6.54	60.8	1.22	5.96	35.6	33.4
800T125-54	3.26	31.0	0.940	4.67	3.26	41.2	0.824	4.43	17.5	15.7
800T125-43	1.65	21.1	0.640	3.48	-	-	-	-	8.73	7.60
800T125-33*	0.744	13.4	0.407	2.44	-	-	-	-	3.95	3.41

Table II - 3 (continued)

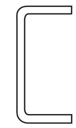
Beam Properties
SSMA Tracks
C-Sections Without Lips



Section	F_y = 33 ksi, F_u = 45 ksi				F_y = 50 ksi, F_u = 65 ksi				Maximum 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.
600T200-97	11.8	55.0	1.67	5.76	17.4	78.4	1.57	5.56	251	32.0
600T200-68	6.96	34.1	1.03	3.70	8.56	48.6	0.973	3.54	80.5	10.2
600T200-54	4.36	25.1	0.759	2.76	4.36	35.9	0.717	2.64	38.9	4.89
600T200-43	2.20	18.6	0.565	2.08	-	-	-	-	19.2	2.41
600T200-33	0.996	11.0	0.333	1.54	-	-	-	-	8.68	1.08
600T150-97	11.8	49.6	1.50	4.78	17.4	72.2	1.44	4.78	118	52.1
600T150-68	6.96	31.8	0.963	3.26	8.56	44.6	0.891	3.16	38.4	16.0
600T150-54	4.36	22.8	0.689	2.47	4.36	30.5	0.609	2.40	18.6	7.60
600T150-43	2.20	15.6	0.474	1.89	-	-	-	-	9.26	3.71
600T150-33	0.996	10.00	0.303	1.33	-	-	-	-	4.18	1.67
600T150-30	0.730	8.36	0.253	1.16	-	-	-	-	3.07	1.22
600T150-27*	0.545	7.06	0.214	1.01	-	-	-	-	2.29	0.910
600T125-97	11.8	50.8 [3]	1.35	4.28	17.4	67.4 [4]	1.35	4.28	117	73.0
600T125-68	6.96	30.2	0.916	2.97	8.56	42.9	0.858	2.93	38.1	21.9
600T125-54	4.36	22.0	0.666	2.30	4.36	29.6	0.592	2.24	18.5	10.2
600T125-43	2.20	15.2	0.461	1.77	-	-	-	-	9.20	4.97
600T125-33	0.996	9.80	0.297	1.26	-	-	-	-	4.16	2.23
600T125-30	0.730	8.22	0.249	1.09	-	-	-	-	3.05	1.63
600T125-27*	0.545	6.95	0.211	0.958	-	-	-	-	2.27	1.21
550T200-68	6.96	30.2	0.914	3.03	8.56	42.9	0.857	2.89	79.1	9.07
550T200-54	4.38	22.1	0.669	2.25	4.77	31.5	0.630	2.15	38.1	4.36
550T200-43	2.41	16.3	0.495	1.69	-	-	-	-	18.8	2.15
550T200-33	1.09	10.1	0.307	1.25	-	-	-	-	8.49	0.968
550T150-68	6.96	28.1	0.850	2.66	8.56	40.2	0.804	2.57	87.6	14.2
550T150-54	4.38	20.7	0.628	2.00	4.77	29.7	0.595	1.93	42.1	6.74
550T150-43	2.41	15.4	0.468	1.52	-	-	-	-	20.8	3.29
550T150-33	1.09	10.2	0.310	1.11	-	-	-	-	9.37	1.48
550T150-30	0.798	8.29	0.251	0.994	-	-	-	-	6.87	1.08
550T150-27	0.595	6.85	0.208	0.892	-	-	-	-	5.12	0.807
550T125-68	6.96	26.6	0.807	2.41	8.56	38.4	0.769	2.38	38.8	19.3
550T125-54	4.38	19.7	0.597	1.86	4.77	26.7	0.535	1.81	18.8	9.05
550T125-43	2.41	13.7	0.417	1.43	-	-	-	-	9.31	4.39
550T125-33	1.09	8.90	0.270	1.03	-	-	-	-	4.21	1.97
550T125-30	0.798	7.47	0.226	0.897	-	-	-	-	3.09	1.44
550T125-27	0.595	6.32	0.192	0.786	-	-	-	-	2.30	1.07

Table II - 3 (continued)

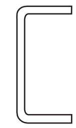
Beam Properties
SSMA Tracks
C-Sections Without Lips



Section	F_y = 33 ksi, F_u = 45 ksi				F_y = 50 ksi, F_u = 65 ksi				Maximum 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.
400T200-68	5.50	19.5	0.591	1.49	8.33	27.4	0.549	1.41	73.5	6.06
400T200-54	4.38	14.1	0.426	1.09	5.39	19.8	0.397	1.04	35.1	2.91
400T200-43	2.78	10.3	0.311	0.810	-	-	-	-	17.2	1.43
400T200-33	1.50	7.25	0.220	0.581	-	-	-	-	7.77	0.646
400T150-68	5.50	18.1	0.548	1.29	8.33	25.6	0.513	1.24	83.5	9.29
400T150-54	4.38	13.2	0.399	0.960	5.39	18.7	0.374	0.918	39.7	4.41
400T150-43	2.78	9.68	0.293	0.719	-	-	-	-	19.4	2.15
400T150-33	1.50	6.88	0.208	0.519	-	-	-	-	8.77	0.968
400T150-30	1.10	6.03	0.183	0.458	-	-	-	-	6.43	0.708
400T150-27	0.823	5.08	0.154	0.409	-	-	-	-	4.79	0.528
400T125-68	5.50	17.1	0.517	1.15	8.33	24.4	0.488	1.13	89.7	12.4
400T125-54	4.38	12.6	0.381	0.882	5.39	17.9	0.359	0.849	42.6	5.83
400T125-43	2.78	9.30	0.282	0.666	-	-	-	-	20.8	2.83
400T125-33	1.50	6.63	0.201	0.484	-	-	-	-	9.38	1.27
400T125-30	1.10	5.82	0.177	0.427	-	-	-	-	6.87	0.929
400T125-27	0.823	5.14	0.156	0.380	-	-	-	-	5.13	0.692
400T125-18*	0.242	2.32	0.0701	0.241	-	-	-	-	1.50	0.202
362T200-68	4.97	17.1	0.519	1.20	7.53	24.0	0.480	1.14	71.6	5.37
362T200-54	3.97	12.3	0.372	0.879	5.39	17.3	0.345	0.832	34.1	2.58
362T200-43	2.78	8.92	0.270	0.649	-	-	-	-	16.7	1.27
362T200-33	1.64	6.28	0.190	0.464	-	-	-	-	7.53	0.573
362T150-68	4.97	15.9	0.481	1.03	7.53	22.4	0.449	0.993	82.1	8.19
362T150-54	3.97	11.5	0.349	0.769	5.39	16.3	0.325	0.734	38.9	3.88
362T150-43	2.78	8.42	0.255	0.574	-	-	-	-	19.0	1.90
362T150-33	1.64	5.95	0.180	0.414	-	-	-	-	8.56	0.853
362T150-30	1.22	5.21	0.158	0.364	-	-	-	-	6.27	0.624
362T150-27	0.910	4.60	0.140	0.323	-	-	-	-	4.68	0.465
362T125-68	4.97	15.0	0.453	0.921	7.53	21.3	0.427	0.907	88.7	10.9
362T125-54	3.97	11.0	0.333	0.705	5.39	15.6	0.312	0.678	41.9	5.12
362T125-43	2.78	8.08	0.245	0.531	-	-	-	-	20.4	2.48
362T125-33	1.64	5.74	0.174	0.384	-	-	-	-	9.21	1.11
362T125-30	1.22	5.03	0.153	0.339	-	-	-	-	6.75	0.816
362T125-27	0.910	4.45	0.135	0.301	-	-	-	-	5.03	0.608
362T125-18	0.267	2.10	0.0637	0.189	-	-	-	-	1.47	0.177

Table II - 3 (continued)

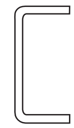
Beam Properties
SSMA Tracks
C-Sections Without Lips



Section	F_y = 33 ksi, F_u = 45 ksi				F_y = 50 ksi, F_u = 65 ksi				Maximum 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.
350T200-68	4.79	16.4	0.496	1.11	7.26	22.9	0.458	1.05	71.0	5.15
350T200-54	3.83	11.7	0.355	0.814	5.39	16.5	0.329	0.770	33.8	2.47
350T200-43	2.78	8.49	0.257	0.600	-	-	-	-	16.5	1.22
350T200-33	1.64	5.97	0.181	0.428	-	-	-	-	7.45	0.549
350T150-68	4.79	15.1	0.459	0.957	7.26	21.4	0.428	0.919	81.6	7.84
350T150-54	3.83	11.0	0.332	0.711	5.39	15.5	0.310	0.679	38.6	3.71
350T150-43	2.78	8.01	0.243	0.531	-	-	-	-	18.8	1.81
350T150-33	1.64	5.66	0.172	0.382	-	-	-	-	8.49	0.816
350T150-30	1.26	4.95	0.150	0.336	-	-	-	-	6.22	0.597
350T150-27	0.943	4.37	0.132	0.298	-	-	-	-	4.64	0.445
350T125-68	4.79	14.3	0.433	0.851	7.26	20.4	0.407	0.839	88.3	10.4
350T125-54	3.83	10.5	0.317	0.651	5.39	14.9	0.297	0.626	41.6	4.89
350T125-43	2.78	7.69	0.233	0.490	-	-	-	-	20.3	2.37
350T125-33	1.64	5.46	0.165	0.354	-	-	-	-	9.14	1.06
350T125-30	1.26	4.78	0.145	0.312	-	-	-	-	6.70	0.779
350T125-27	0.943	4.22	0.128	0.277	-	-	-	-	5.00	0.581
350T125-18	0.277	2.03	0.0615	0.174	-	-	-	-	1.46	0.169
250T200-68	3.38	10.7	0.324	0.548	5.12	14.8	0.296	0.517	64.9	3.48
250T200-54	2.71	7.53	0.228	0.396	4.10	10.4	0.209	0.371	30.5	1.67
250T200-43	2.17	5.37	0.163	0.288	-	-	-	-	14.8	0.820
250T200-33	1.64	3.71	0.112	0.203	-	-	-	-	6.67	0.369
250T150-68	3.38	9.88	0.299	0.465	5.12	13.8	0.276	0.445	76.4	5.20
250T150-54	2.71	7.05	0.214	0.342	4.10	9.84	0.197	0.324	35.6	2.46
250T150-43	2.17	5.07	0.154	0.252	-	-	-	-	17.2	1.20
250T150-33	1.64	3.52	0.107	0.180	-	-	-	-	7.75	0.540
250T150-30	1.33	3.06	0.0929	0.157	-	-	-	-	5.68	0.395
250T150-27	1.10	2.69	0.0815	0.139	-	-	-	-	4.24	0.295
250T125-68	3.38	9.28	0.281	0.409	5.12	13.1	0.262	0.402	84.2	6.83
250T125-54	2.71	6.70	0.203	0.310	4.10	9.42	0.188	0.297	39.1	3.20
250T125-43	2.17	4.86	0.147	0.231	-	-	-	-	18.8	1.55
250T125-33	1.64	3.40	0.103	0.166	-	-	-	-	8.48	0.696
250T125-30	1.33	2.96	0.0896	0.145	-	-	-	-	6.21	0.509
250T125-27	1.10	2.60	0.0788	0.129	-	-	-	-	4.63	0.380
250T125-18	0.392	1.46	0.0443	0.0778	-	-	-	-	1.36	0.111

Table II - 3 (continued)

Beam Properties
SSMA Tracks
C-Sections Without Lips



Section	F_y = 33 ksi, F_u = 45 ksi				F_y = 50 ksi, F_u = 65 ksi				Maximum 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.
162T125-33	1.06	1.92	0.0583	0.0656	-	-	-	-	7.60	0.418
162T125-30	0.956	1.66	0.0504	0.0574	-	-	-	-	5.57	0.306
162T125-27	0.866	1.45	0.0440	0.0505	-	-	-	-	4.15	0.228
162T125-18	0.484	0.831	0.0252	0.0297	-	-	-	-	1.22	0.0667

Notes:

* Web h/t > 200, therefore bearing stiffeners are required.

1. Shear and moment strengths given are nominal strengths. To obtain design strengths, these values must be modified by factors of safety (ASD) or resistance factors (LRFD).

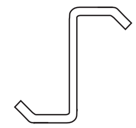
2. M_{web} and M_{flange} are the highest nominal moments at which the web and flange, respectively, are fully effective.

3. Flexural properties were calculated incorporating strength increase from cold work of forming with F_{ya} = 37.72 ksi

4. Flexural properties were calculated incorporating strength increase from cold work of forming with F_{ya} = 54.14 ksi

5. Flexural properties were calculated incorporating strength increase from cold work of forming with F_{ya} = 51.83 ksi

Table II - 4

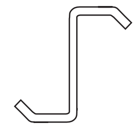


Beam Properties
Z-Sections With Lips

Section	F_y = 33 ksi				F_y = 55 ksi				Maximum 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.	M_{lip} kip-in.
12ZS3.25x105	14.4	241	7.29	43.7	14.4	364	6.61	43.7	424	325	261
12ZS3.25x085	7.63	190	5.75	35.5	7.63	266	4.84	35.5	242	193	168
12ZS3.25x070	4.25	148	4.49	29.3	4.25	194	3.53	29.2	148	120	114
12ZS2.75x105	14.4	216	6.54	40.0	14.4	339	6.16	39.8	172	377	331
12ZS2.75x085	7.63	168	5.10	32.4	7.63	241	4.38	31.8	90.7	223	208
12ZS2.75x070	4.25	133	4.05	26.3	4.25	180	3.27	25.9	50.4	139	138
12ZS2.25x105	14.4	195	5.92	36.3	14.4	309	5.62	36.0	156	481	433
12ZS2.25x085	7.63	152	4.60	29.4	7.63	239	4.35	28.7	82.2	285	242
12ZS2.25x070	4.25	120	3.64	23.8	4.25	183	3.33	23.2	45.7	162	141
10ZS3.25x105	15.1	188	5.69	28.4	17.5	282	5.13	28.4	416	253	204
10ZS3.25x085	9.25	148	4.48	23.1	9.25	217	3.95	23.1	238	150	131
10ZS3.25x070	5.15	115	3.50	19.1	5.15	163	2.97	18.7	145	93.9	89.3
10ZS3.25x065	4.12	107	3.23	17.8	4.12	145	2.64	17.2	121	78.7	77.7
10ZS3.25x059	3.08	96.1	2.91	16.1	3.08	125	2.27	15.7	96.1	59.8	64.6
10ZS2.75x105	15.1	171	5.17	25.9	17.5	282	5.12	25.9	423	321	274
10ZS2.75x085	9.25	139	4.21	21.0	9.25	213	3.87	21.0	243	191	175
10ZS2.75x070	5.15	115	3.47	17.4	5.15	161	2.93	17.4	149	120	119
10ZS2.75x065	4.12	105	3.17	16.2	4.12	143	2.61	16.2	125	96.5	103
10ZS2.75x059	3.08	93.0	2.82	14.7	3.08	124	2.25	14.7	99.2	72.4	84.2
10ZS2.25x105	15.1	154	4.66	23.3	17.5	249	4.53	23.3	179	391	353
10ZS2.25x085	9.25	122	3.70	18.9	9.25	194	3.53	18.8	93.9	232	197
10ZS2.25x070	5.15	97.4	2.95	15.6	5.15	150	2.72	15.3	52.1	132	115
10ZS2.25x065	4.12	89.2	2.70	14.4	4.12	130	2.37	14.1	41.7	106	93.3
10ZS2.25x059	3.08	79.1	2.40	13.0	3.08	114	2.07	12.8	31.1	80.1	71.0
9ZS2.25x105	15.1	132	4.02	18.1	19.5	221	4.02	18.1	443	367	326
9ZS2.25x085	9.88	108	3.27	14.7	10.3	180	3.27	14.7	254	220	184
9ZS2.25x070	5.76	89.1	2.70	12.2	5.76	145	2.63	12.2	155	126	108
9ZS2.25x065	4.60	82.9	2.51	11.3	4.60	129	2.35	11.3	129	101	87.4
9ZS2.25x059	3.44	75.1	2.27	10.3	3.44	110	2.00	10.3	100	76.1	66.7
8ZS3.25x105	15.1	139	4.23	16.9	19.5	208	3.79	16.9	400	187	152
8ZS3.25x085	9.88	110	3.33	13.8	11.7	160	2.91	13.8	228	112	97.6
8ZS3.25x070	6.52	85.5	2.59	11.4	6.52	128	2.32	10.9	139	69.9	66.6
8ZS3.25x065	5.22	79.0	2.39	10.6	5.22	117	2.12	10.1	116	58.6	57.9
8ZS3.25x059	3.90	71.0	2.15	9.62	3.90	100	1.82	9.25	91.7	44.5	48.1

Table II - 4 (continued)

Beam Properties
Z-Sections With Lips



Section	F_y = 33 ksi				F_y = 55 ksi				Maximum 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.	M_{lip} kip-in.
8ZS2.75x105	15.1	126	3.82	15.3	19.5	208	3.78	15.3	410	236	202
8ZS2.75x085	9.88	103	3.11	12.4	11.7	156	2.84	12.4	235	141	130
8ZS2.75x070	6.52	84.8	2.57	10.3	6.52	123	2.24	10.3	144	88.5	88.1
8ZS2.75x065	5.22	77.3	2.34	9.56	5.22	114	2.07	9.56	120	71.4	76.0
8ZS2.75x059	3.90	68.6	2.08	8.69	3.90	99.7	1.81	8.62	95.5	53.6	62.5
8ZS2.25x105	15.1	112	3.41	13.6	19.5	187	3.41	13.6	420	311	277
8ZS2.25x085	9.88	91.6	2.77	11.1	11.7	153	2.77	11.1	239	186	156
8ZS2.25x070	6.52	75.7	2.30	9.18	6.52	123	2.24	9.18	148	107	91.7
8ZS2.25x065	5.22	70.4	2.13	8.54	5.22	110	2.00	8.54	124	85.9	74.5
8ZS2.25x059	3.90	63.8	1.93	7.76	3.90	98.4	1.79	7.76	98.8	64.7	56.9
7ZS2.25x105	13.3	93.5	2.83	9.92	19.5	156	2.83	9.92	408	259	232
7ZS2.25x085	9.88	76.3	2.31	8.09	12.8	127	2.31	8.09	235	155	131
7ZS2.25x070	6.70	63.1	1.91	6.70	7.53	102	1.86	6.70	145	89.0	76.8
7ZS2.25x065	5.78	58.7	1.78	6.23	6.02	91.4	1.66	6.23	121	71.6	62.4
7ZS2.25x059	4.49	53.2	1.61	5.67	4.49	81.7	1.49	5.66	96.5	54.0	47.7
6ZS2.25x105	11.3	75.8	2.30	6.89	18.8	126	2.30	6.89	400	210	189
6ZS2.25x085	9.18	61.9	1.88	5.63	12.8	103	1.88	5.63	230	126	107
6ZS2.25x070	6.70	51.3	1.55	4.66	8.65	83.2	1.51	4.66	141	72.3	62.7
6ZS2.25x065	5.78	47.8	1.45	4.34	7.12	74.1	1.35	4.34	118	58.2	51.0
6ZS2.25x059	4.76	43.3	1.31	3.95	5.31	66.2	1.20	3.95	93.4	43.9	39.0
4ZS2.25x070	4.83	30.0	0.910	1.82	8.05	48.6	0.884	1.82	130	42.4	37.5
4ZS2.25x065	4.50	28.0	0.848	1.70	7.46	43.3	0.787	1.70	108	34.2	30.5
4ZS2.25x059	4.10	25.4	0.771	1.55	6.15	38.6	0.702	1.55	85.0	25.8	23.3
3.5ZS1.5x070	4.14	18.6	0.563	0.985	6.90	31.0	0.563	0.985	130	56.6	48.9
3.5ZS1.5x065	3.86	17.3	0.525	0.919	6.42	28.9	0.525	0.919	107	47.3	40.0
3.5ZS1.5x059	3.51	15.8	0.479	0.838	5.86	26.3	0.479	0.838	82.4	36.3	30.9

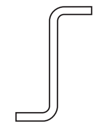
Notes:

1. Shear and moment strengths given are nominal strengths. To obtain design strengths, these values must be modified by factors of safety (ASD) or resistance factors (LRFD).

2. M_{web}, M_{flange} and M_{lip} are the highest nominal moments at which the web, flange and lip, respectively, are fully effective.

Table II - 5

Beam Properties
Z-Sections Without Lips



Section	F_y = 33 ksi				F_y = 50 ksi				Maximum 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	56.3	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	30.2	0.914	2.77	9.47	43.8	0.876	2.77	49.0	27.6
6ZU1.25x060	4.93	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	7.01
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	16.7	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	0.293	0.647	3.88	13.8	0.276	0.624	30.8	3.90
4ZU1.25x036	1.77	6.75	0.205	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	25.9	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	11.1	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	3.41
3.625ZU1.25x036	1.77	5.83	0.177	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

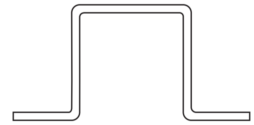
Notes:

1. Shear and moment strengths given are nominal strengths. To obtain design strengths, these values must be modified by factors of safety (ASD) or resistance factors (LRFD).

2. M_{web} and M_{flange} are the highest nominal moments at which the web and flange, respectively, are fully effective.

Table II - 6

Beam Properties
Hat-Sections Without Lips



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	11.5	104	3.16	34.1	146	2.93	10.9	158	3.16
6HU6x105	22.5	97.3	2.95	10.4	98.4	2.98	34.1	144	2.88	10.3	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	1.00	15.4	0.468	7.76	23.4	0.468	1.00	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. To obtain design strengths, these values must be modified by factors of safety (ASD) or resistance factors (LRFD).

2. Y-axis is horizontal.

1.3 Calculation of L_u

For members bent about the centroidal axis perpendicular to the web, calculation of lateral 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 formulae. All terms are as defined in Section C3.1.2 of the *Specification.*

(a) For singly-, doubly-, and point symmetric sections:

$$L_u = \left\{ \frac{GJ}{2C_1} + \left[\frac{C_2}{C_1} + \left(\frac{GJ}{2C_1} \right)^2 \right]^{0.5} \right\}^{0.5}$$

where

(1) For singly- and doubly-symmetric sections:

$$C_1 = \frac{7.72}{AE}\left[\frac{K_yF_yS_f}{C_b\pi r_y}\right]^2$$

$$C_2 = \frac{\pi^2EC_w}{(K_t)^2}$$

(2) For point-symmetric sections:

$$C_1 = \frac{30.9}{AE}\left[\frac{K_yF_yS_f}{C_b\pi r_y}\right]^2$$

$$C_2 = \frac{\pi^2EC_w}{(K_t)^2}$$

(b) For I- 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:

$$L_u = \sqrt{\frac{0.36C_b\pi^2EdI_{yc}}{F_yS_f}}$$

(2) For point-symmetric Z-sections:

$$L_u = \sqrt{\frac{0.18C_b\pi^2EdI_{yc}}{F_yS_f}}$$

(c) For closed box members:

$$L_u = \frac{0.36C_b\pi}{F_yS_f}\sqrt{EGJI_y} \qquad (Eq.\ C3.1.2.2\text{-}1)$$

1.4 Notes on the Charts

(a) With the exception of the SSMA 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 SSMA studs have been included in Charts II-2a and II-2b. These punchouts are considered in SSMA 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 points listed in the charts, except for the SSMA sections (Tables II-2a and II-2b), the provisions of Section A7 of the *Specification* for strength increase from cold work of forming have been used where the section was eligible for the increase. Sections were considered eligible for the cold work of forming increase in yield point 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=1.67 (ASD).

(g) To obtain LRFD design values, the nominal strengths at the extreme left side of each curve, in the horizontal region where the section is not subject to lateral-torsional buckling, must be multiplied by ϕ=0.95. To the right of the horizontal region of each curve, in the sloped region where the section is subject to lateral-torsional buckling, the nominal strengths must be multiplied by ϕ=0.90.

(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) Unbraced lengths in these charts are arbitrarily limited to a maximum of 40 times the depth of the section.

(j) 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.

1.5 Beam Charts

Chart II-1a

Nominal Flexural Strength
C-Sections with Lips, (F_y = 33 ksi, C_b = 1)

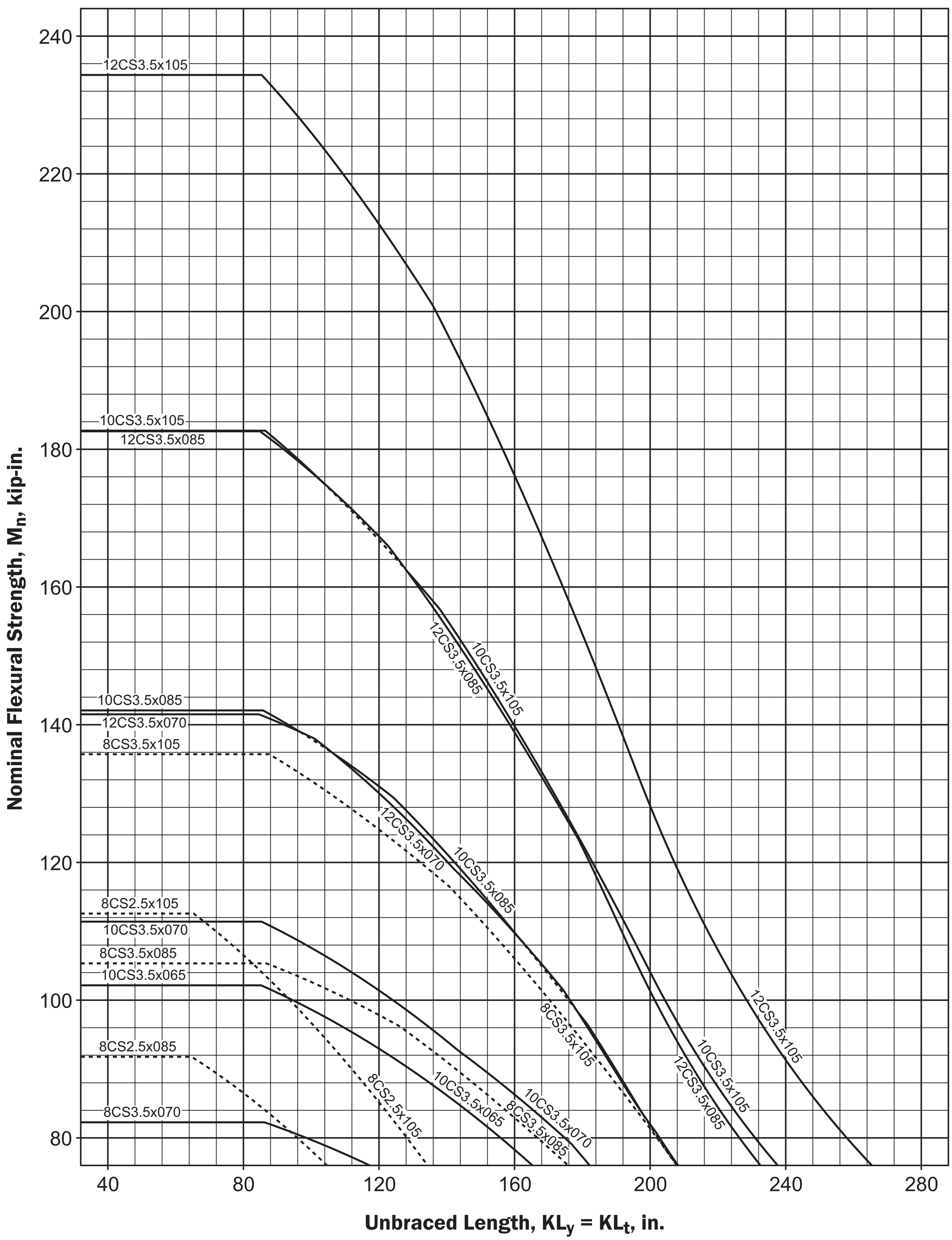


Chart II-1a (continued)

Nominal Flexural Strength C-Sections with Lips, (F_y = 33 ksi, C_b = 1)

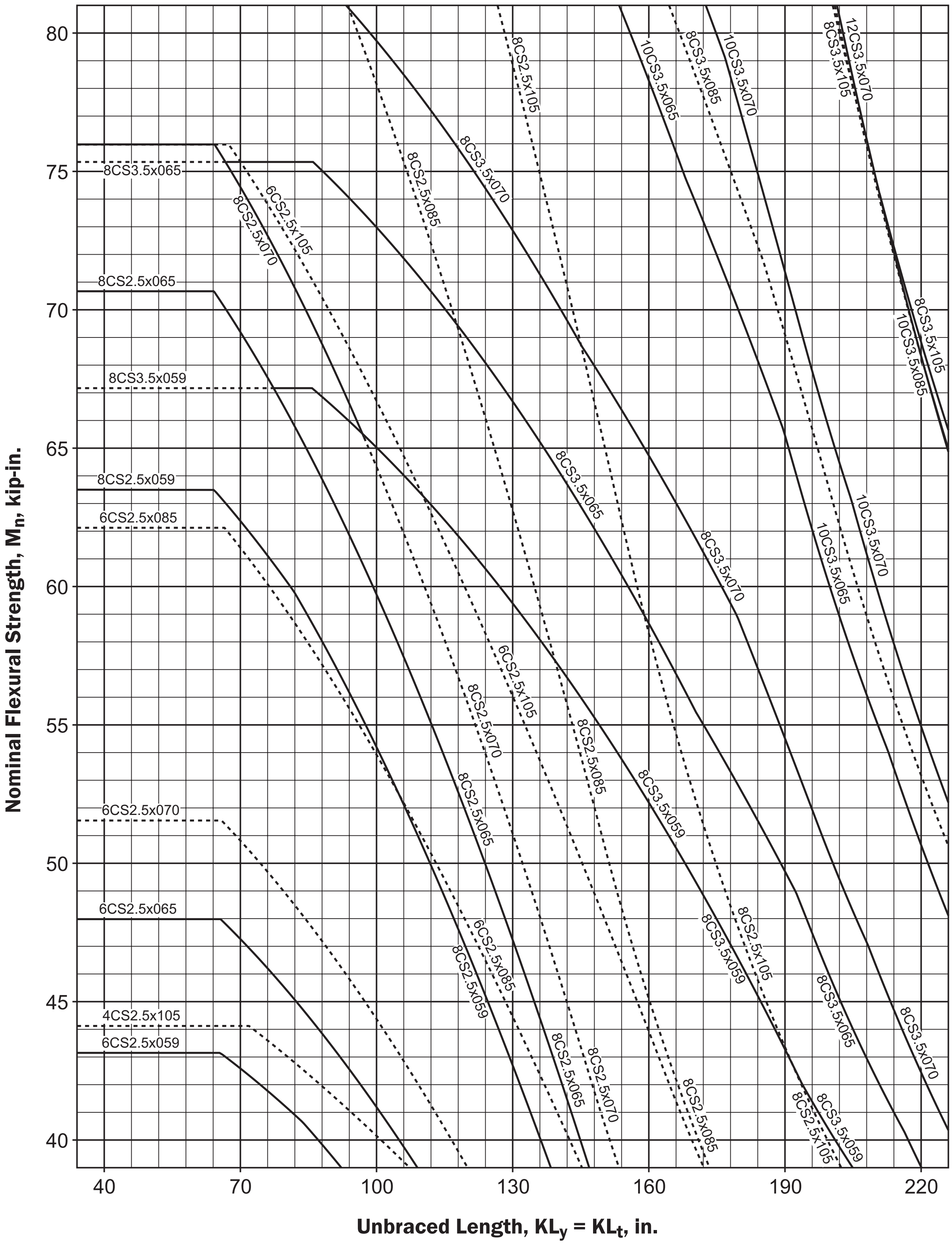


Chart II-1a (continued)

Nominal Flexural Strength
C-Sections with Lips, (F_y = 33 ksi, C_b = 1)

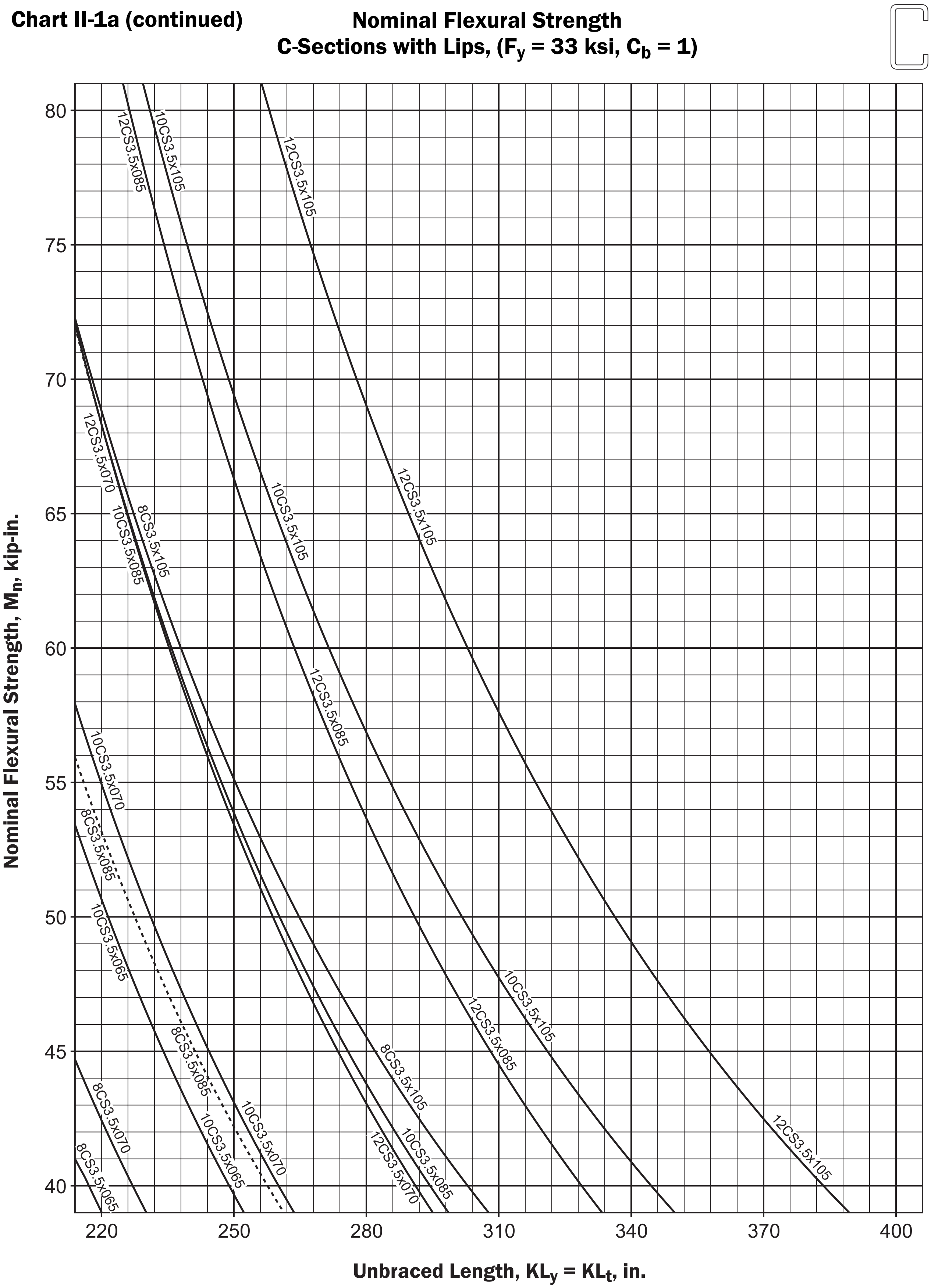


Chart II-1a (continued)

Nominal Flexural Strength
C-Sections with Lips, (F_y = 33 ksi, C_b = 1)

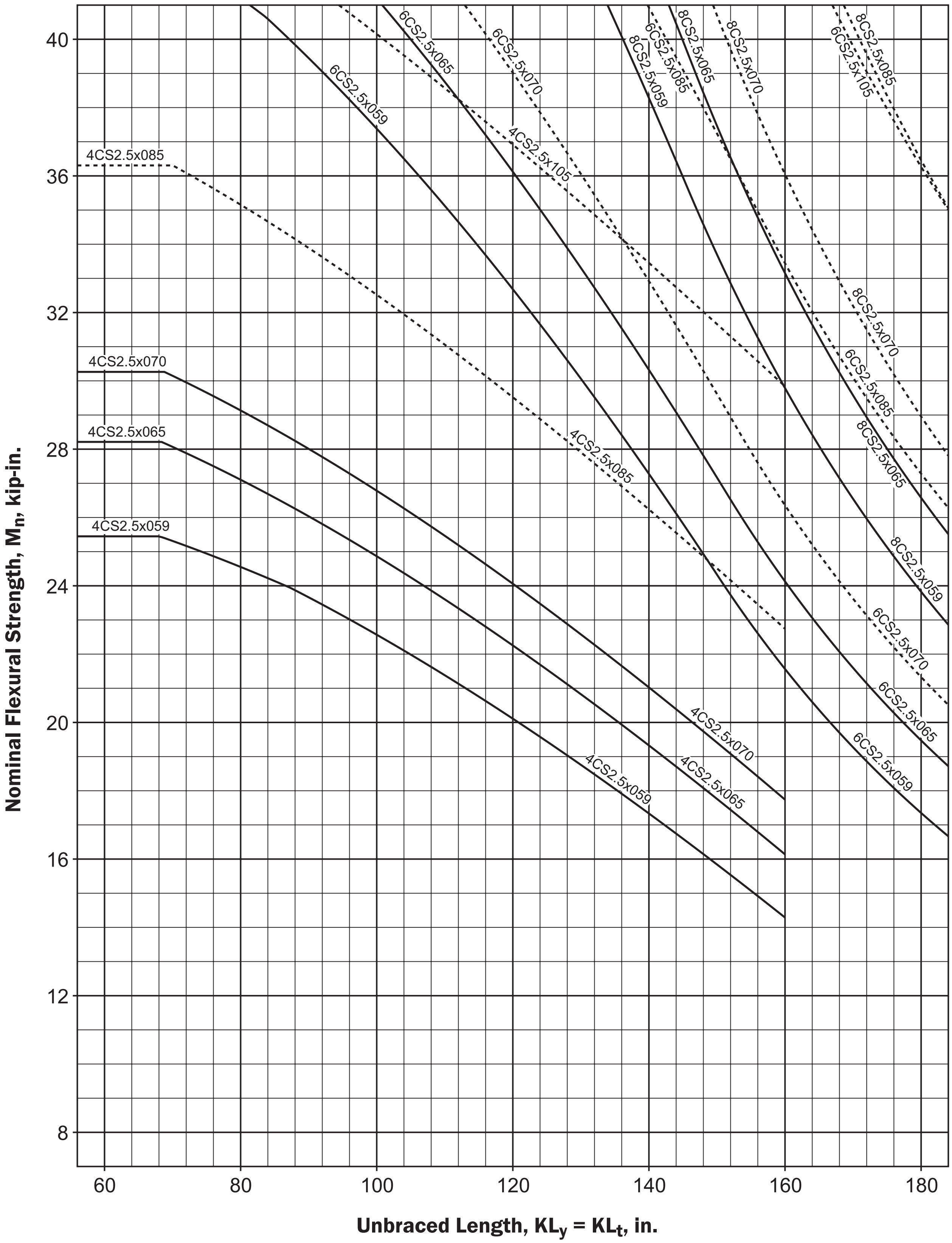


Chart II-1a (continued)

Nominal Flexural Strength
C-Sections with Lips, (F_y = 33 ksi, C_b = 1)

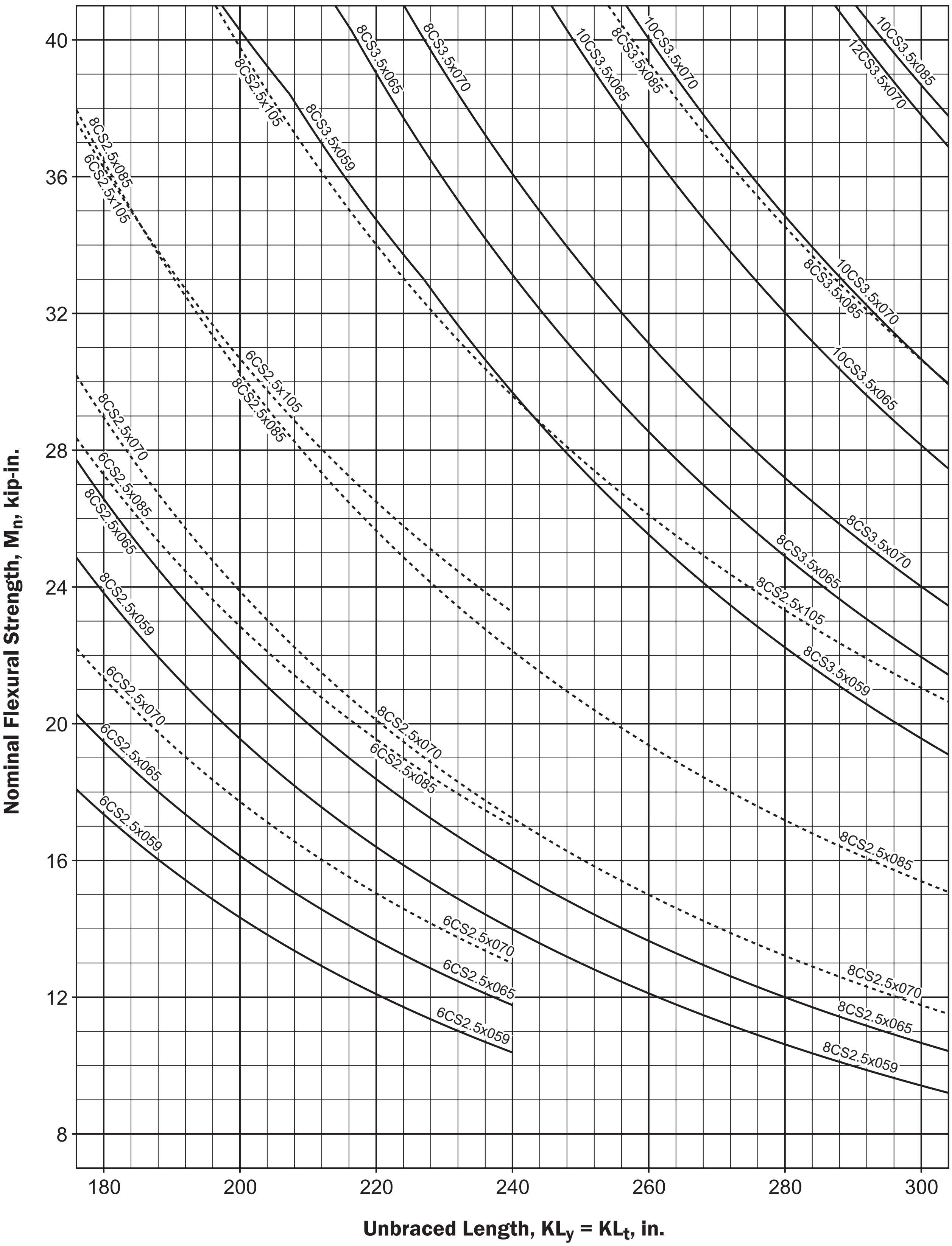


Chart II-1a (continued)

Nominal Flexural Strength
C-Sections with Lips, (F_y = 33 ksi, C_b = 1)

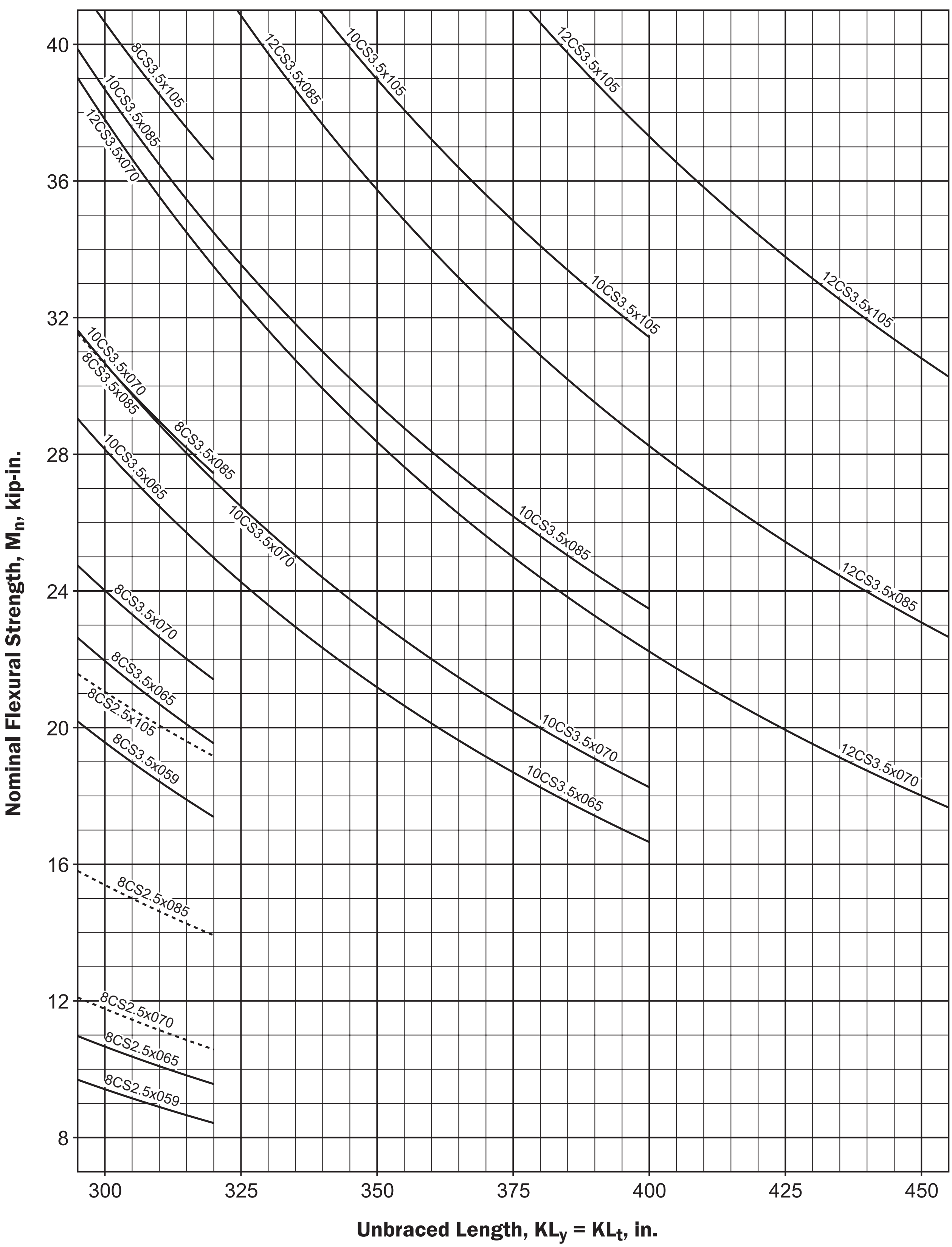


Chart II-1b

Nominal Flexural Strength
C-Sections with Lips, (F_y = 55 ksi, C_b = 1)

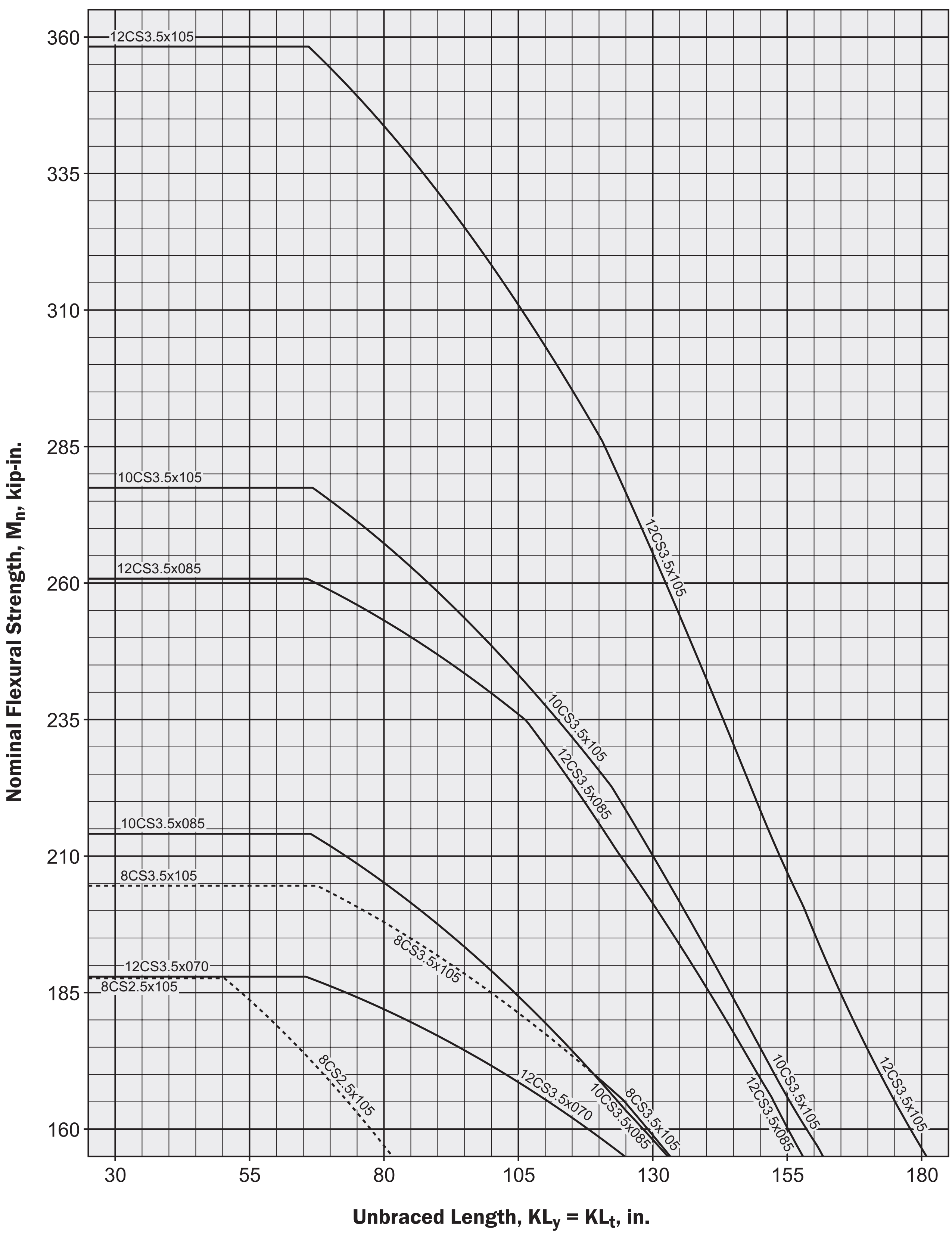


Chart II-1b (continued)

Nominal Flexural Strength
C-Sections with Lips, (F_y = 55 ksi, C_b = 1)

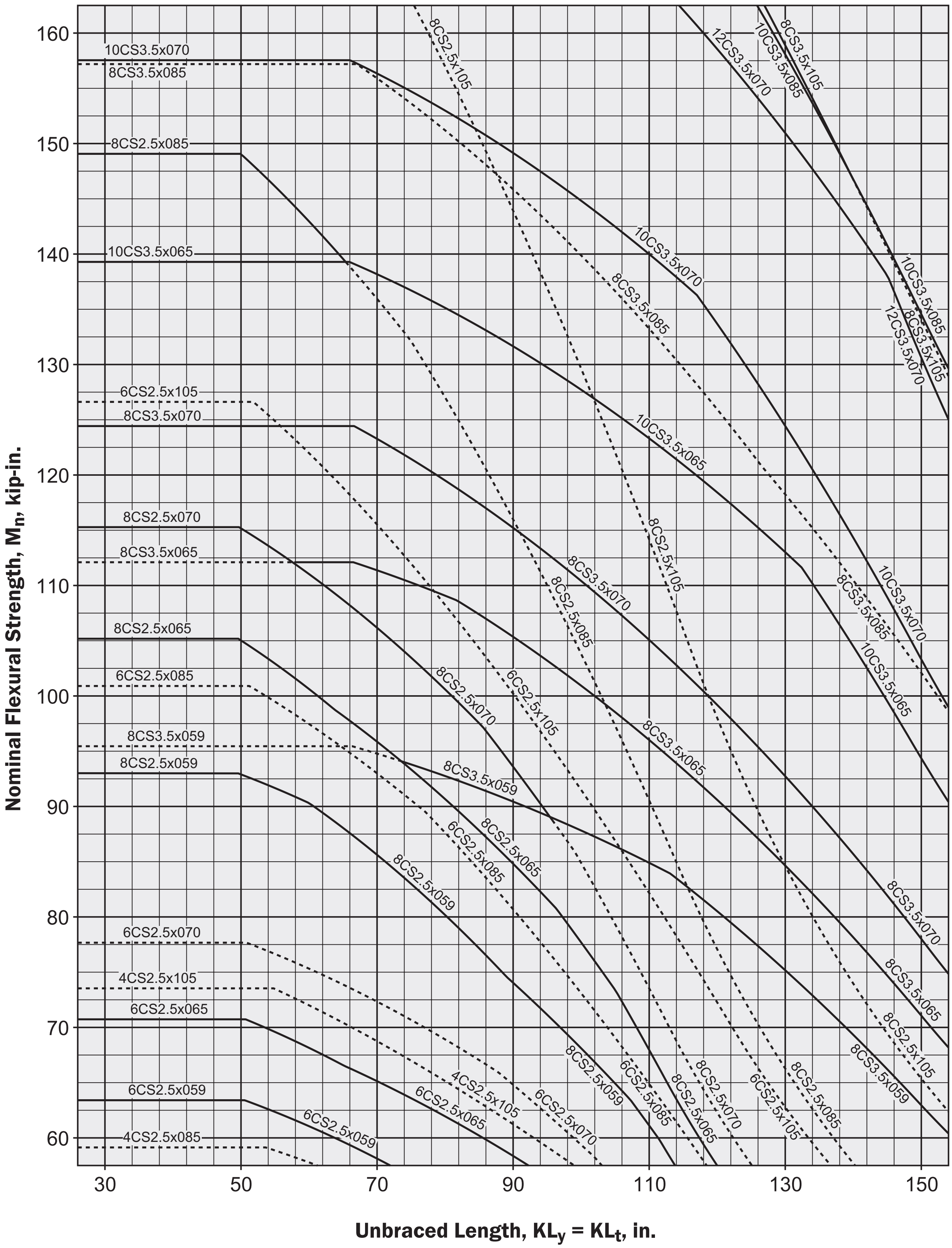


Chart II-1b (continued)

Nominal Flexural Strength
C-Sections with Lips, (F_y = 55 ksi, C_b = 1)

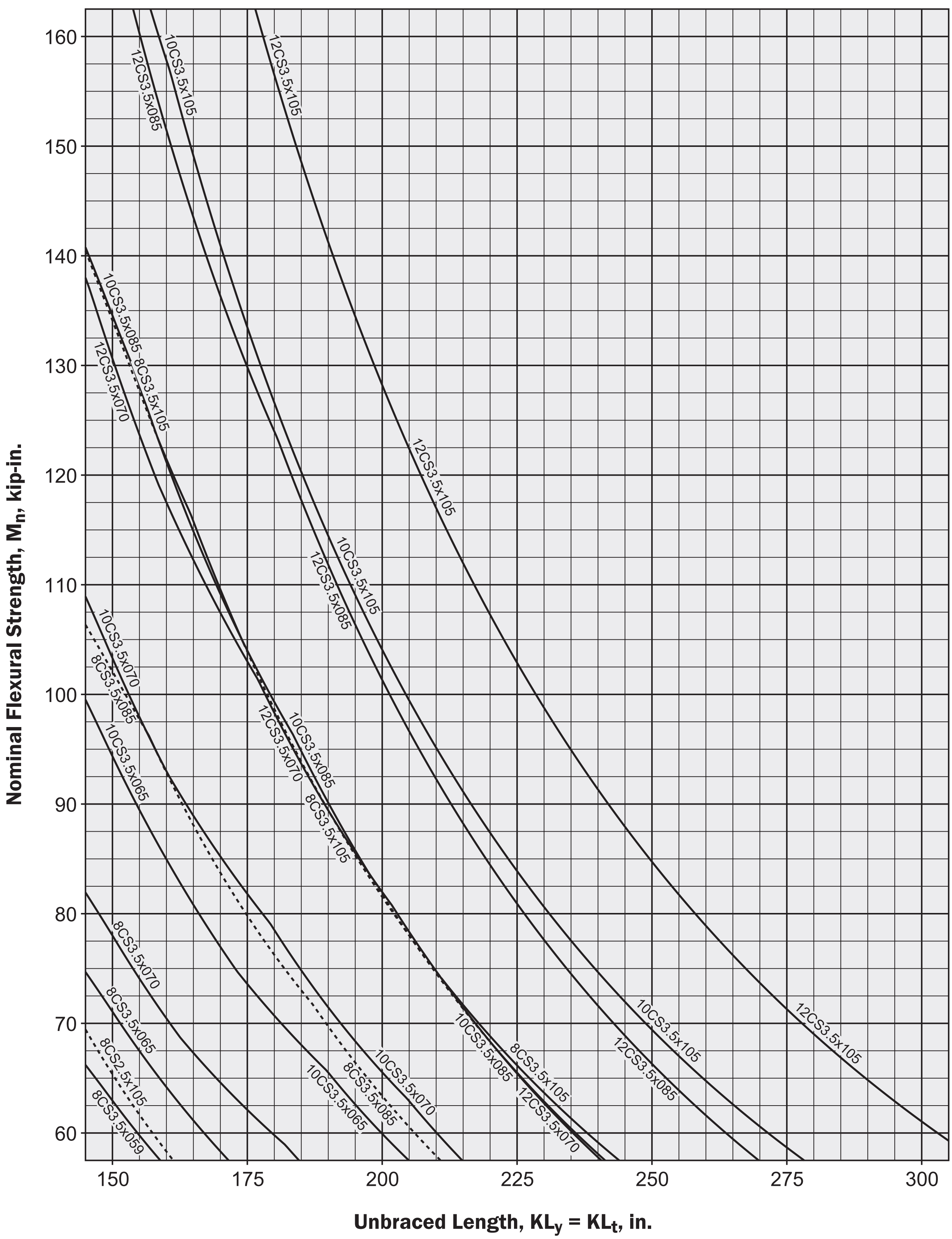


Chart II-1b (continued)

Nominal Flexural Strength C-Sections with Lips, (F_y = 55 ksi, C_b = 1)

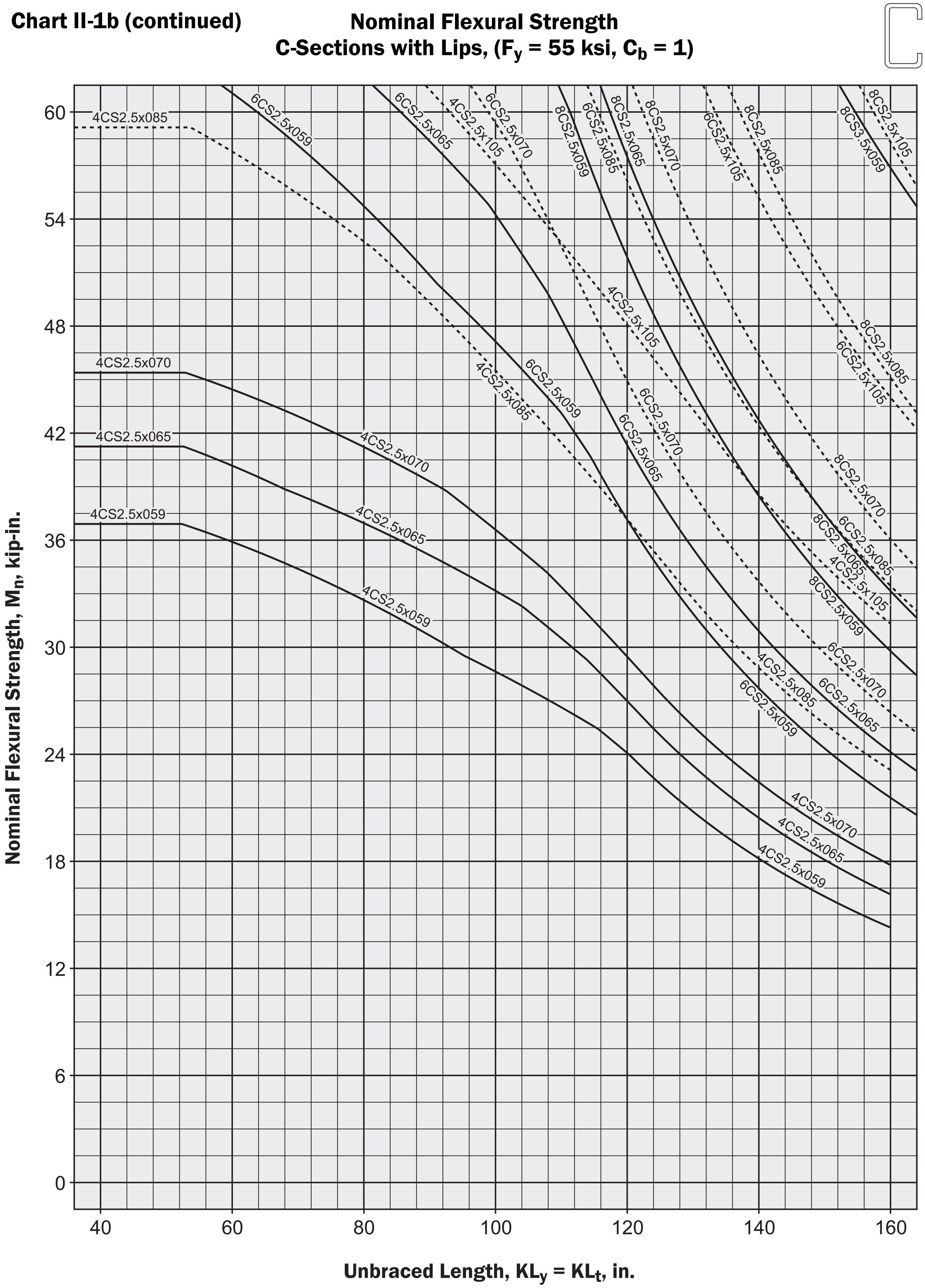


Chart II-1b (continued)

Nominal Flexural Strength
C-Sections with Lips, (F_y = 55 ksi, C_b = 1)

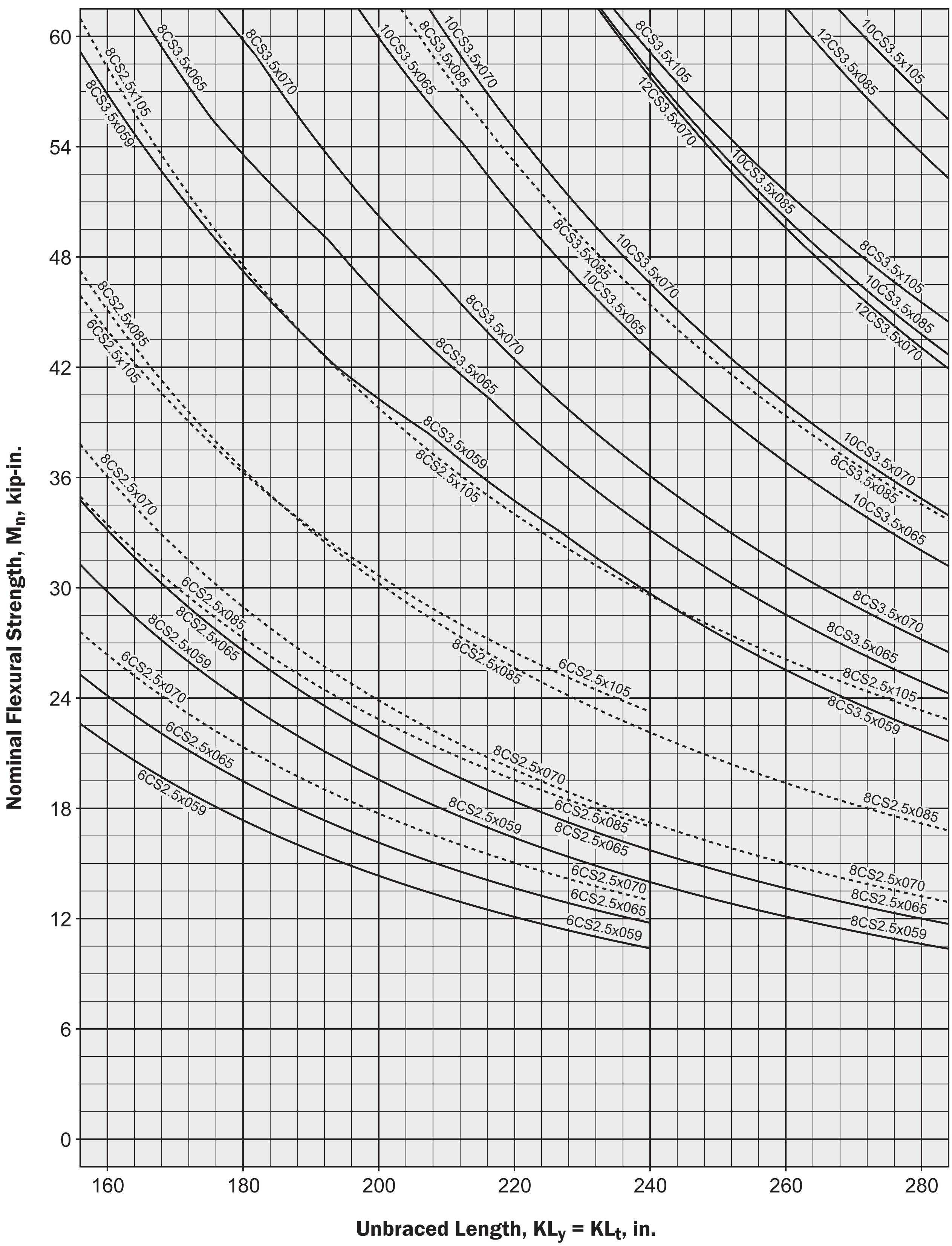


Chart II-1b (continued)

Nominal Flexural Strength
C-Sections with Lips, (F_y = 55 ksi, C_b = 1)

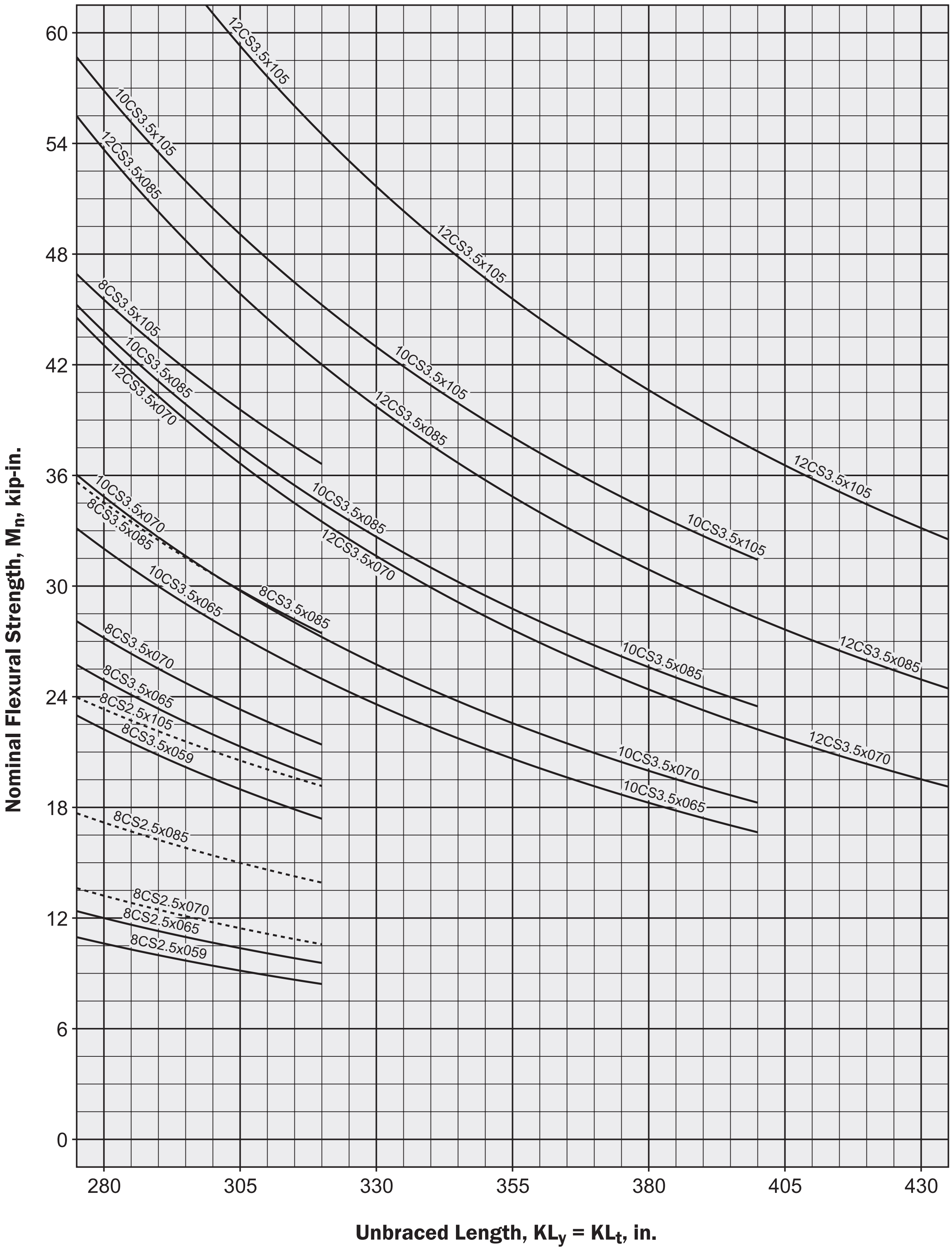


Chart II-2a

Nominal Flexural Strength
SSMA Studs - C-Sections with Lips, (F_y = 33 ksi, C_b = 1)

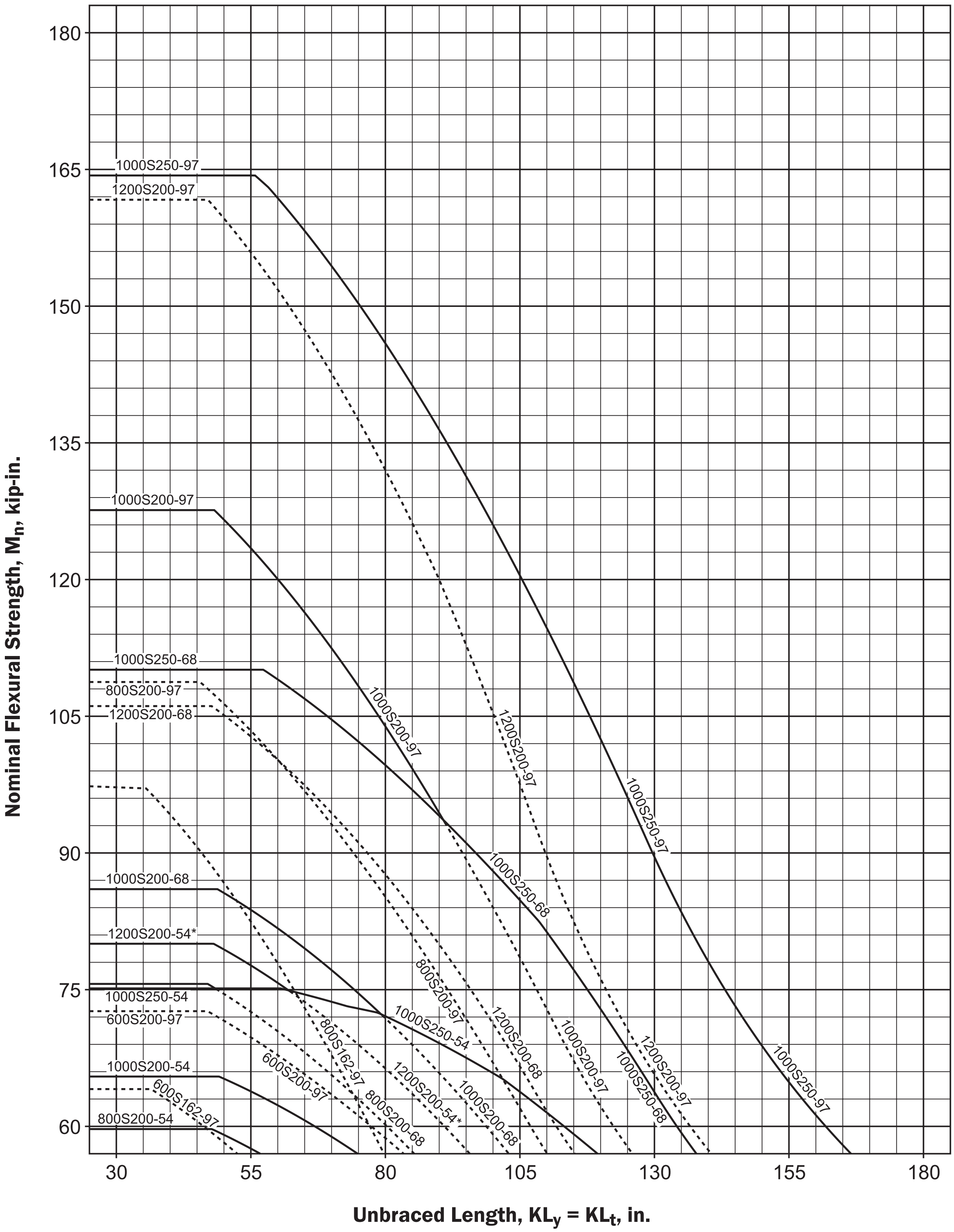


Chart II-2a (continued)

Nominal Flexural Strength
SSMA Studs - C-Sections with Lips, (F_y = 33 ksi, C_b = 1)

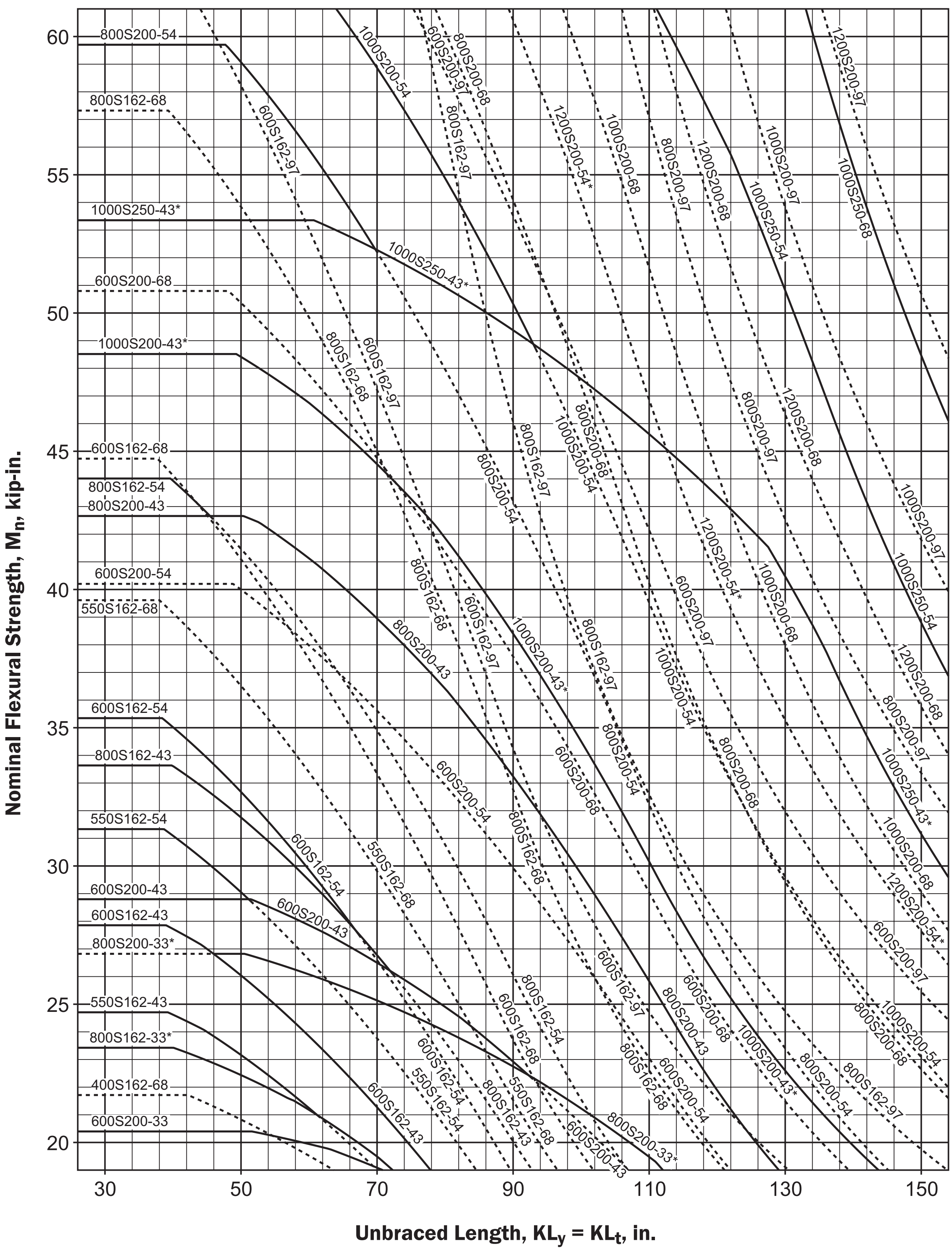


Chart II-2a (continued) **Nominal Flexural Strength**

SSMA Studs - C-Sections with Lips, (F_y = 33 ksi, C_b = 1)

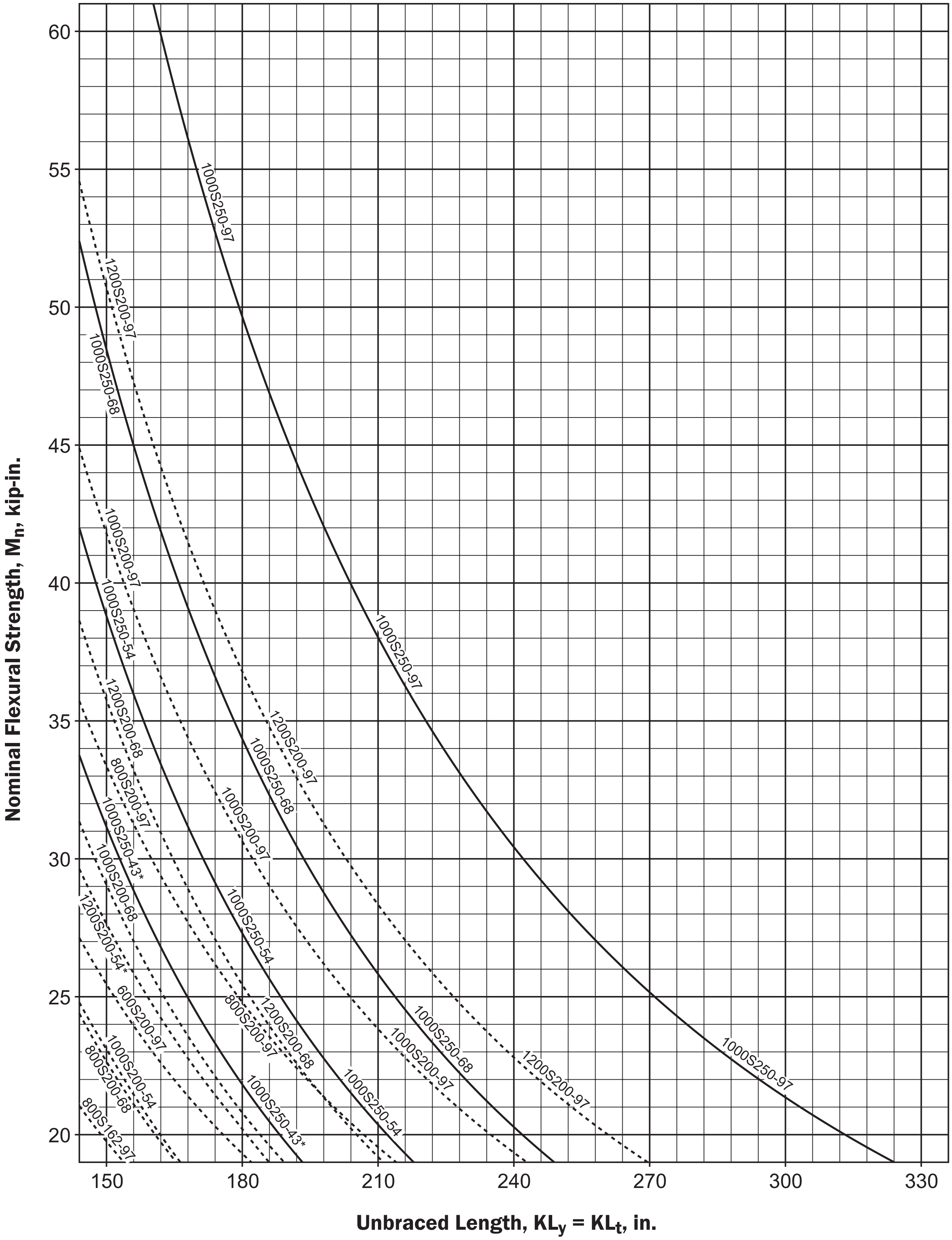


Chart II-2a (continued)

Nominal Flexural Strength
SSMA Studs - C-Sections with Lips, (F_y = 33 ksi, C_b = 1)

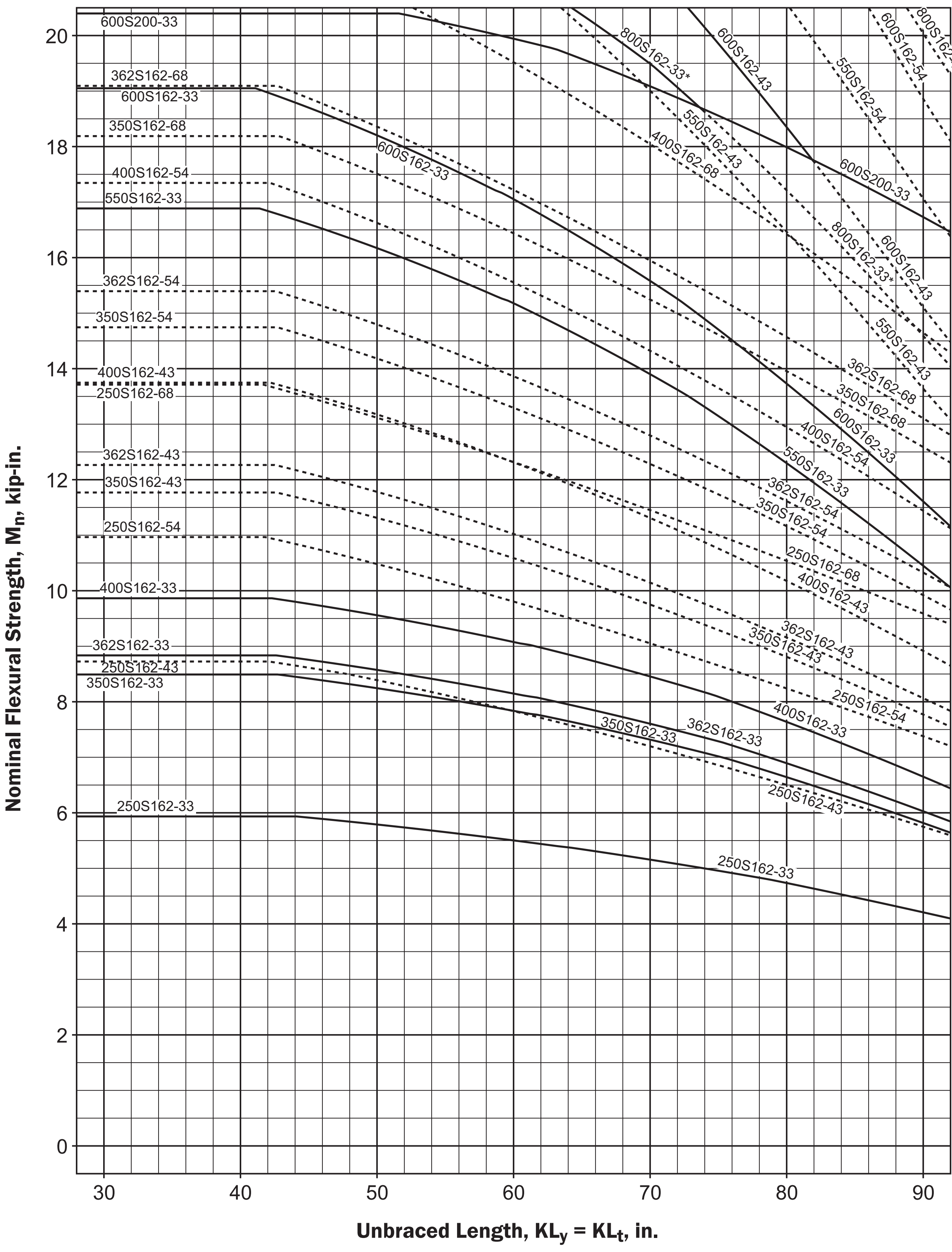


Chart II-2a (continued) **Nominal Flexural Strength**

SSMA Studs - C-Sections with Lips, (F_y = 33 ksi, C_b = 1)

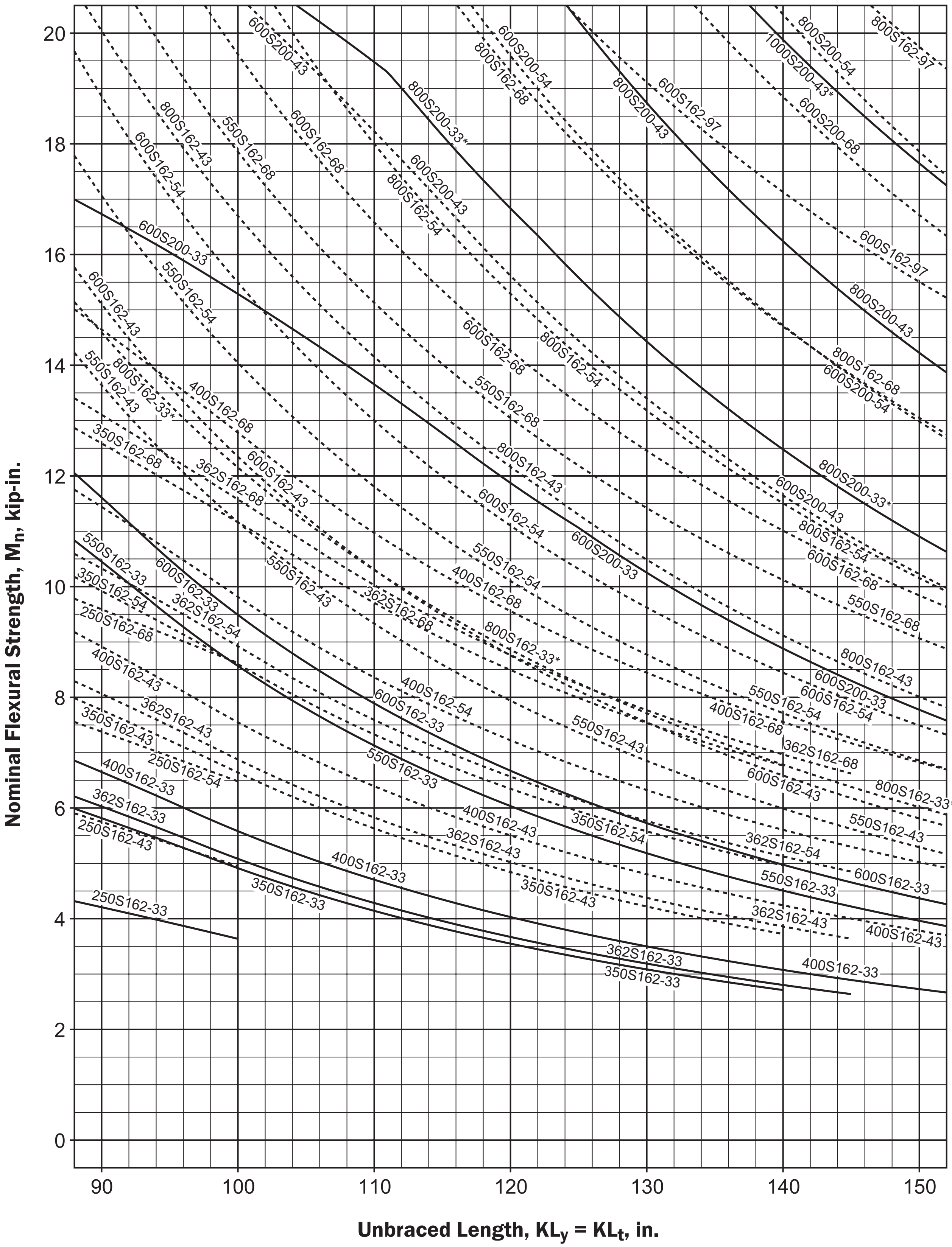


Chart II-2a (continued)

Nominal Flexural Strength
SSMA Studs - C-Sections with Lips, (F_y = 33 ksi, C_b = 1)

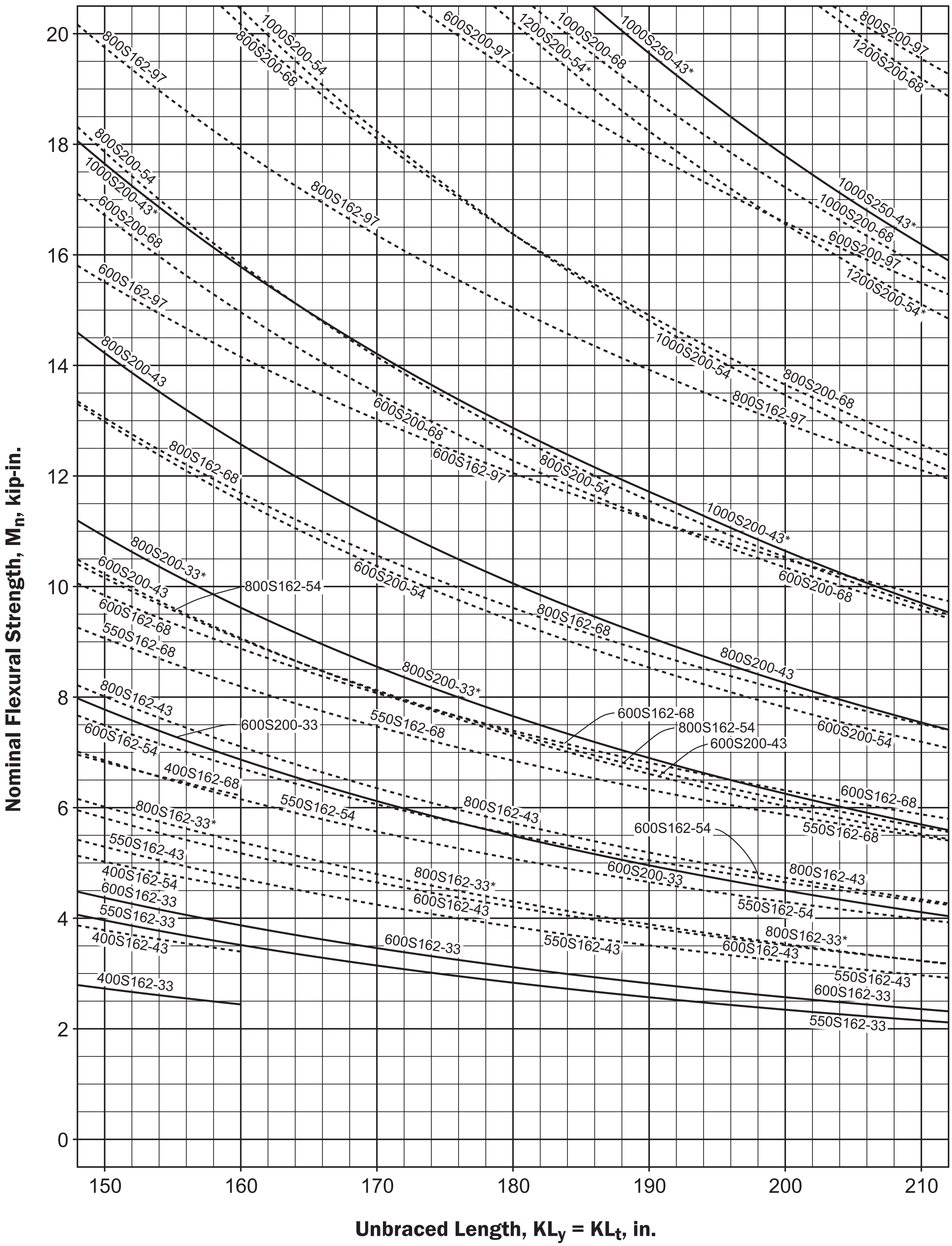


Chart II-2a (continued) **Nominal Flexural Strength**

SSMA Studs - C-Sections with Lips, (F_y = 33 ksi, C_b = 1)

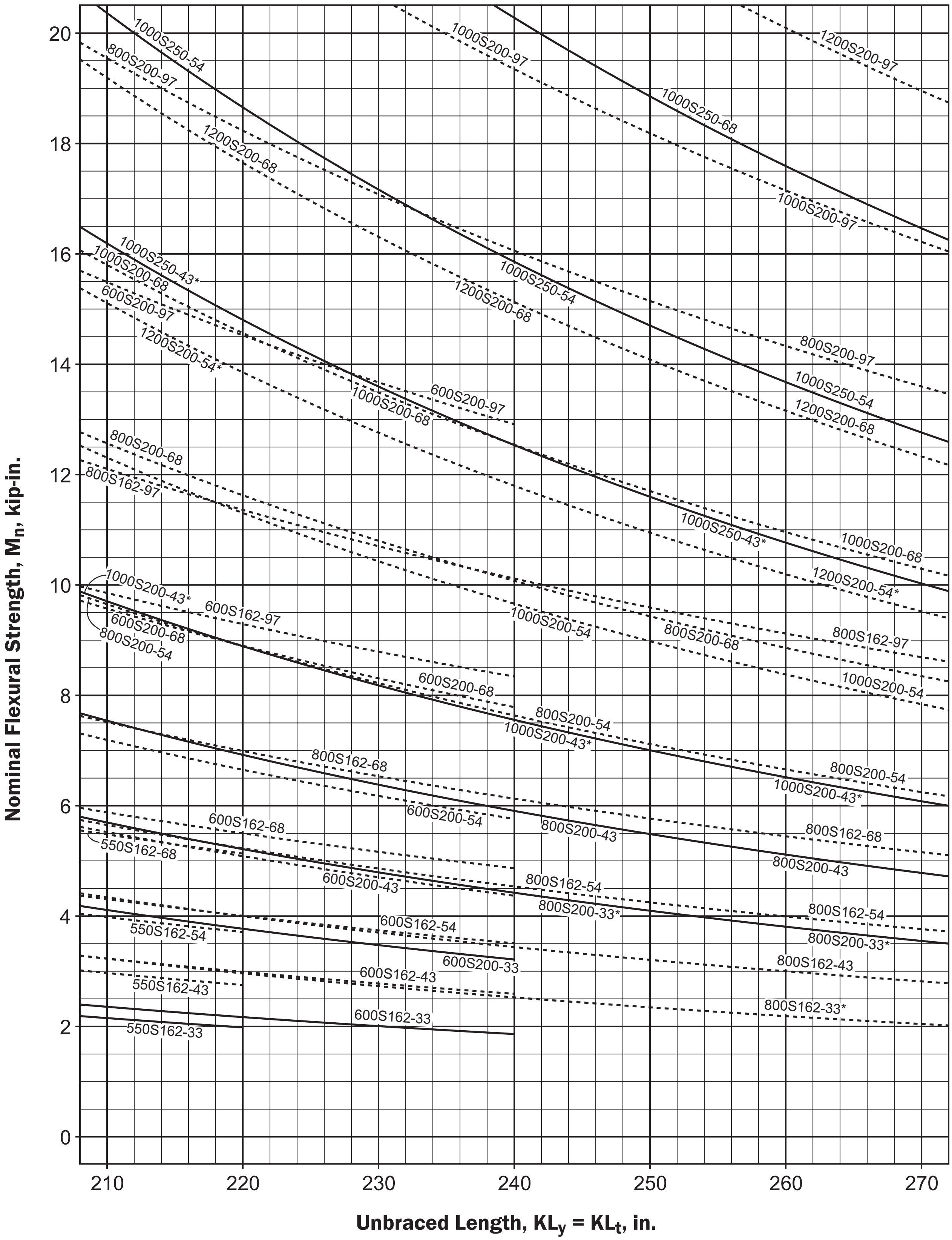


Chart II-2a (continued)

Nominal Flexural Strength
SSMA Studs - C-Sections with Lips, (F_y = 33 ksi, C_b = 1)

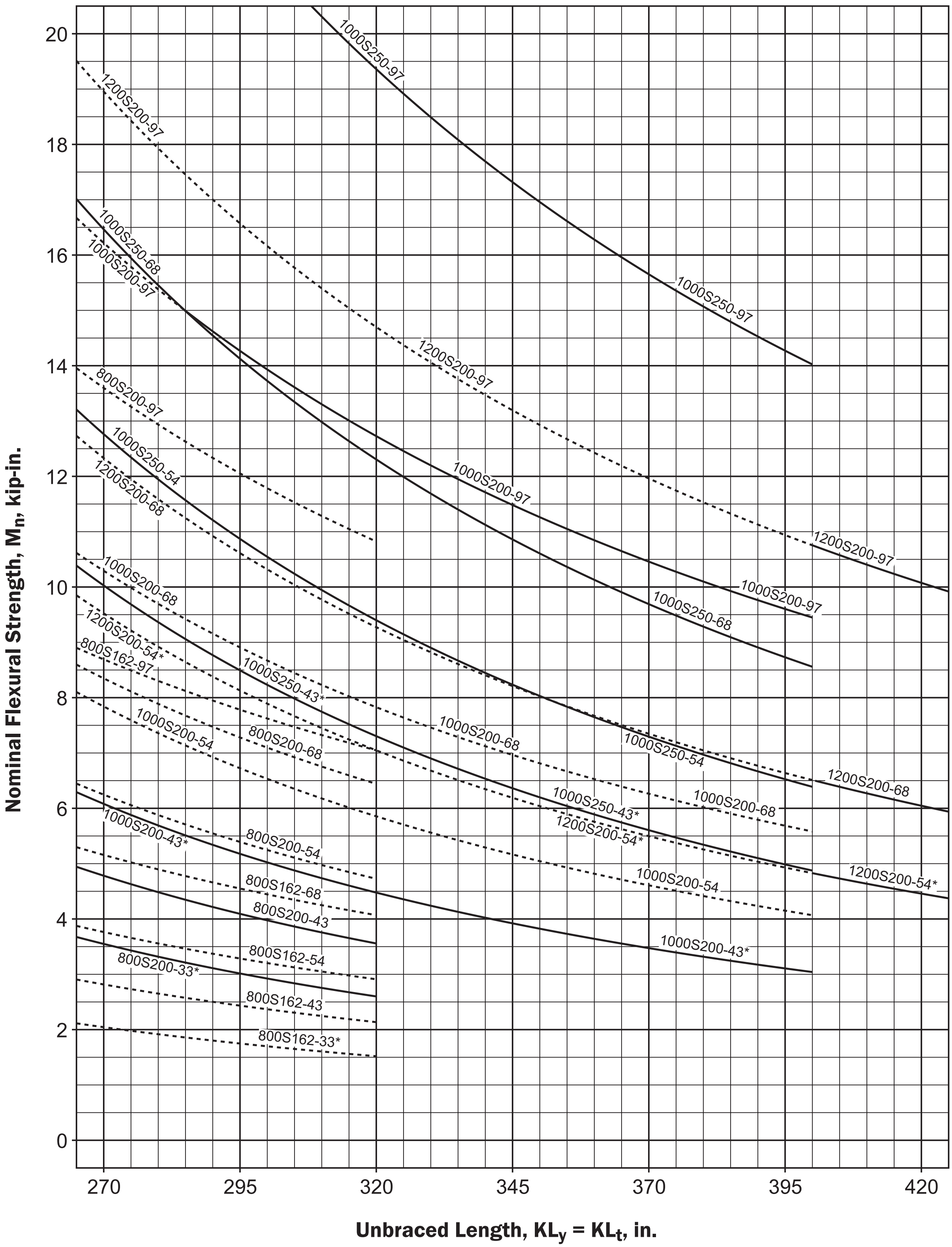


Chart II-2b

Nominal Flexural Strength
SSMA Studs - C-Sections with Lips, (F_y = 50 ksi, C_b = 1)

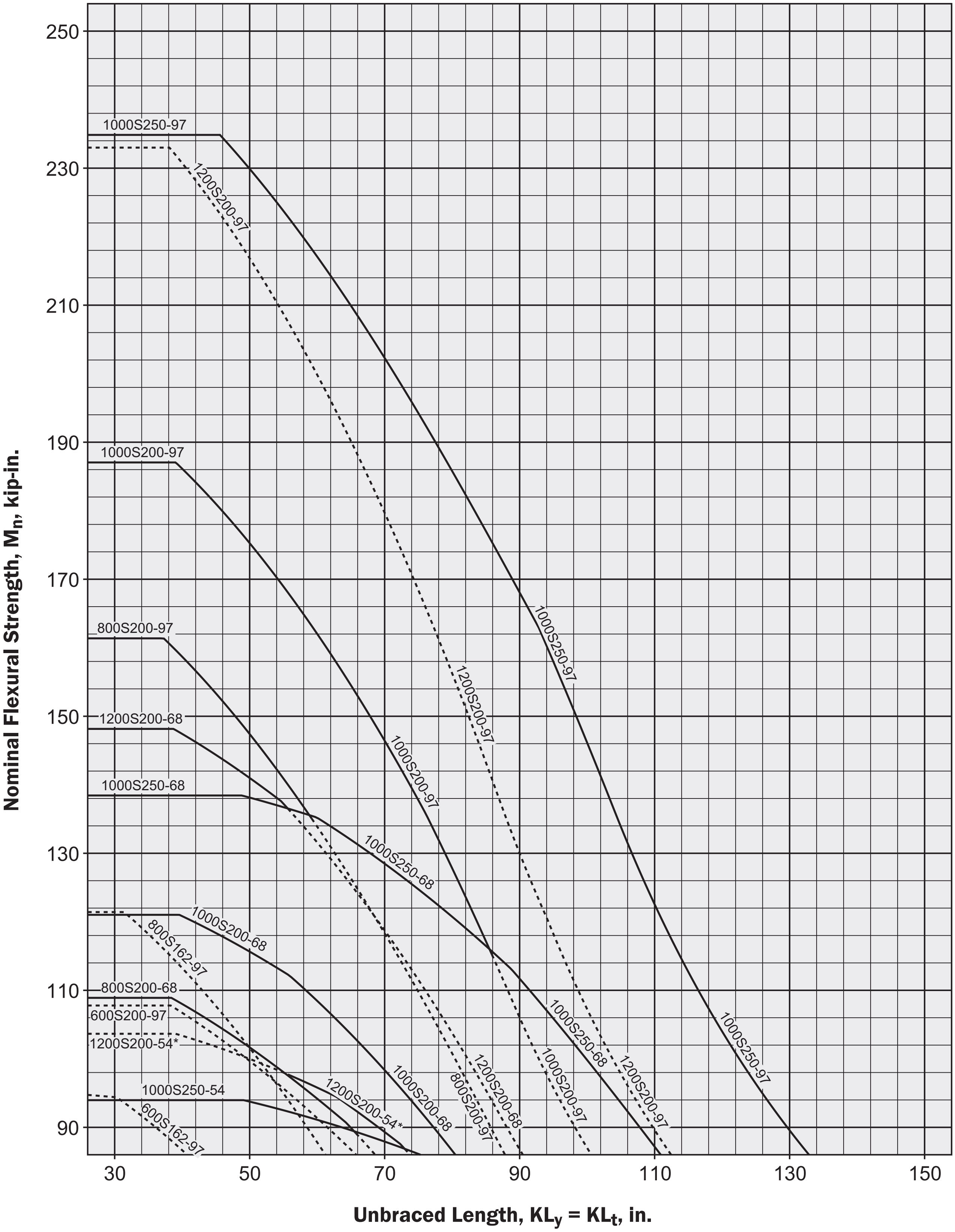


Chart II-2b (continued) **Nominal Flexural Strength**

SSMA Studs - C-Sections with Lips, (F_y = 50 ksi, C_b = 1)

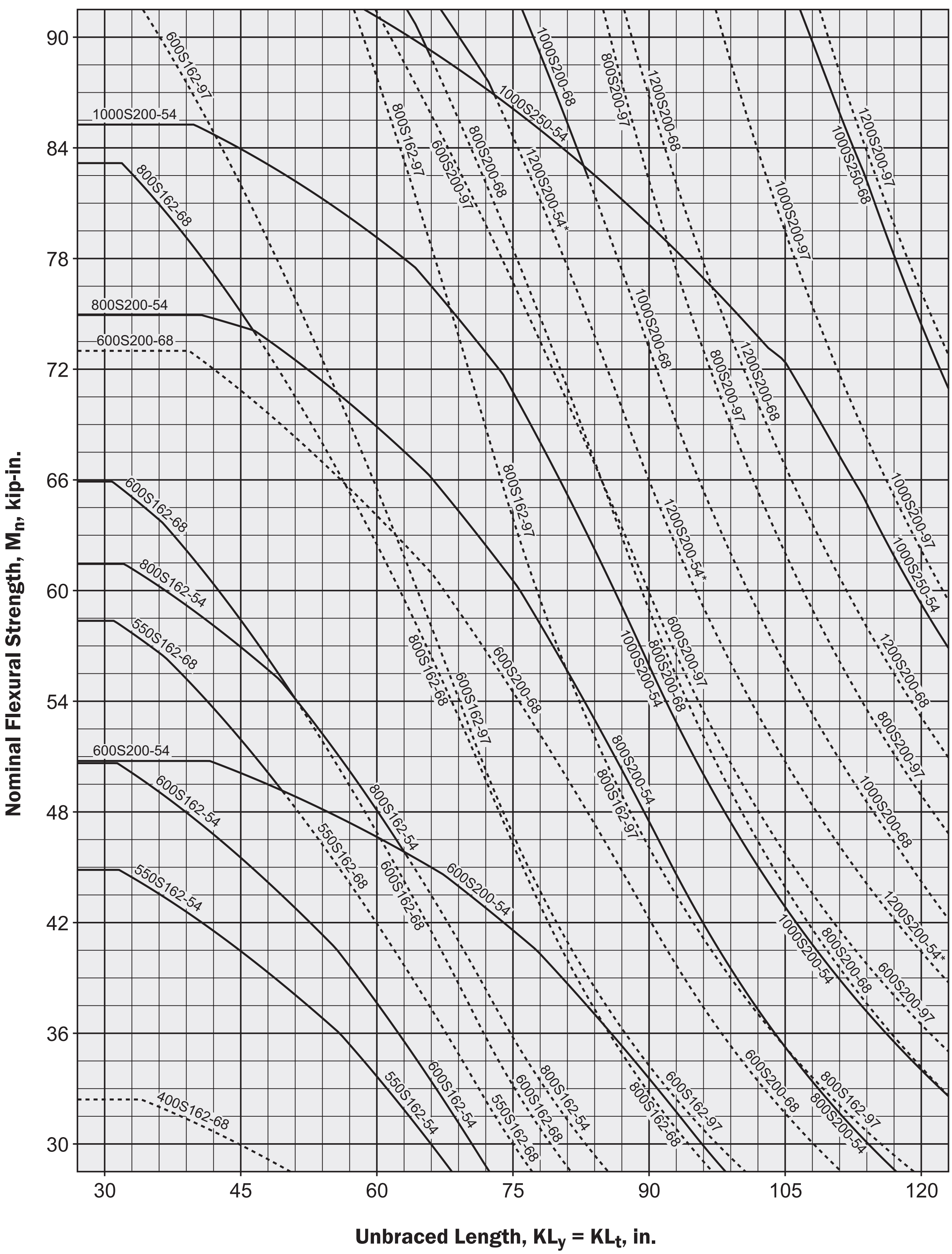


Chart II-2b (continued) **Nominal Flexural Strength**

SSMA Studs - C-Sections with Lips, (F_y = 50 ksi, C_b = 1)

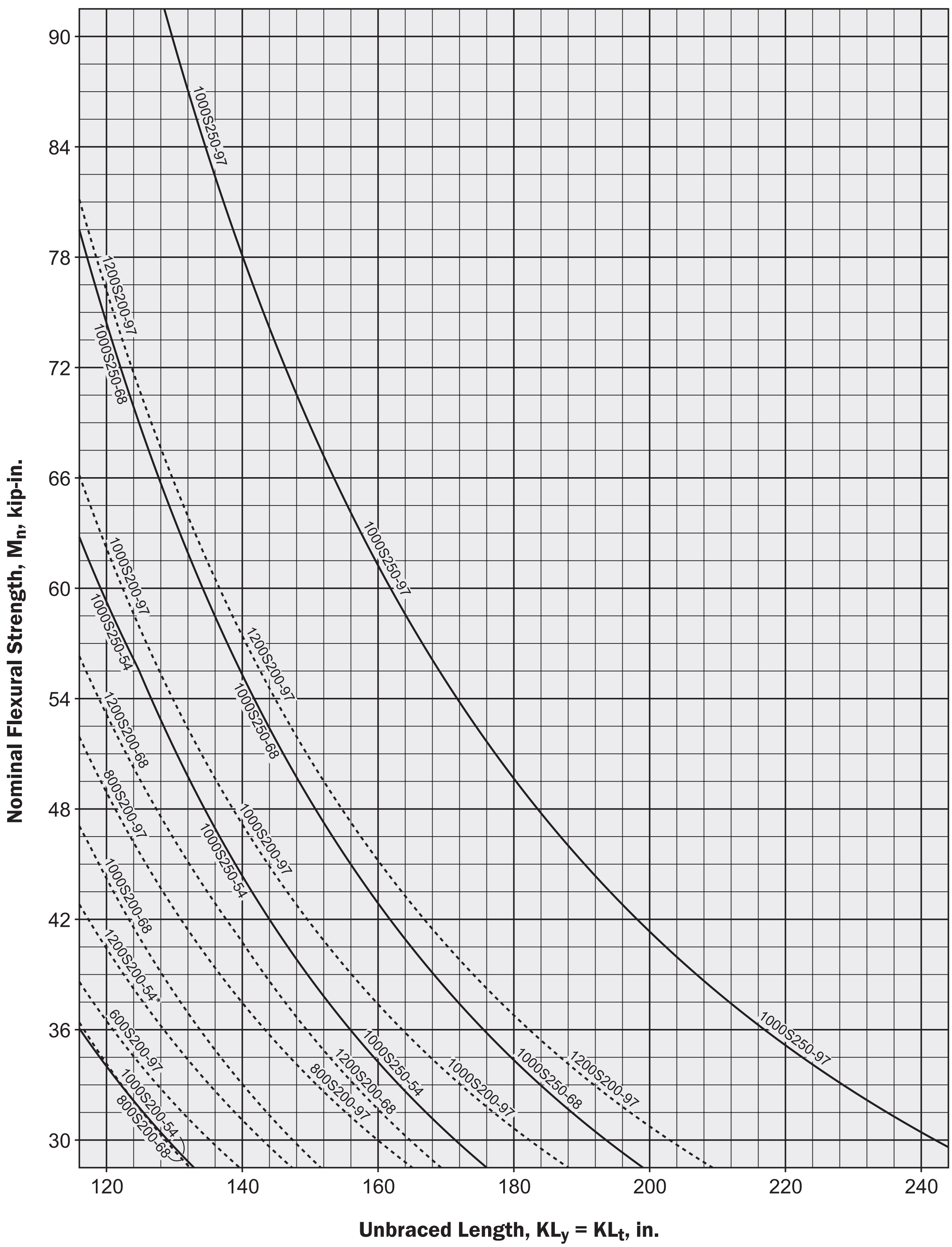


Chart II-2b (continued)

Nominal Flexural Strength
SSMA Studs - C-Sections with Lips, (F_y = 50 ksi, C_b = 1)

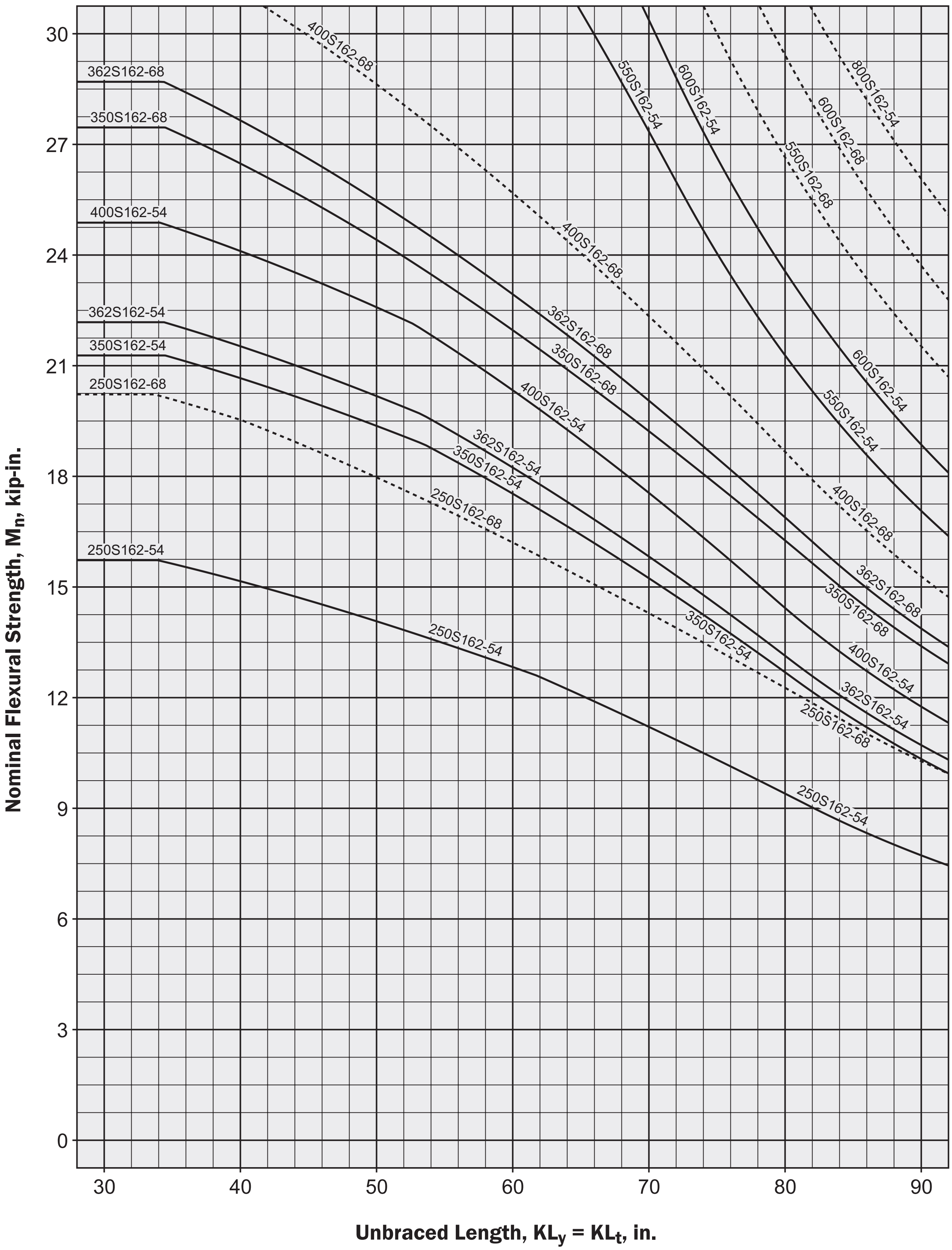


Chart II-2b (continued)

Nominal Flexural Strength
SSMA Studs - C-Sections with Lips, (F_y = 50 ksi, C_b = 1)

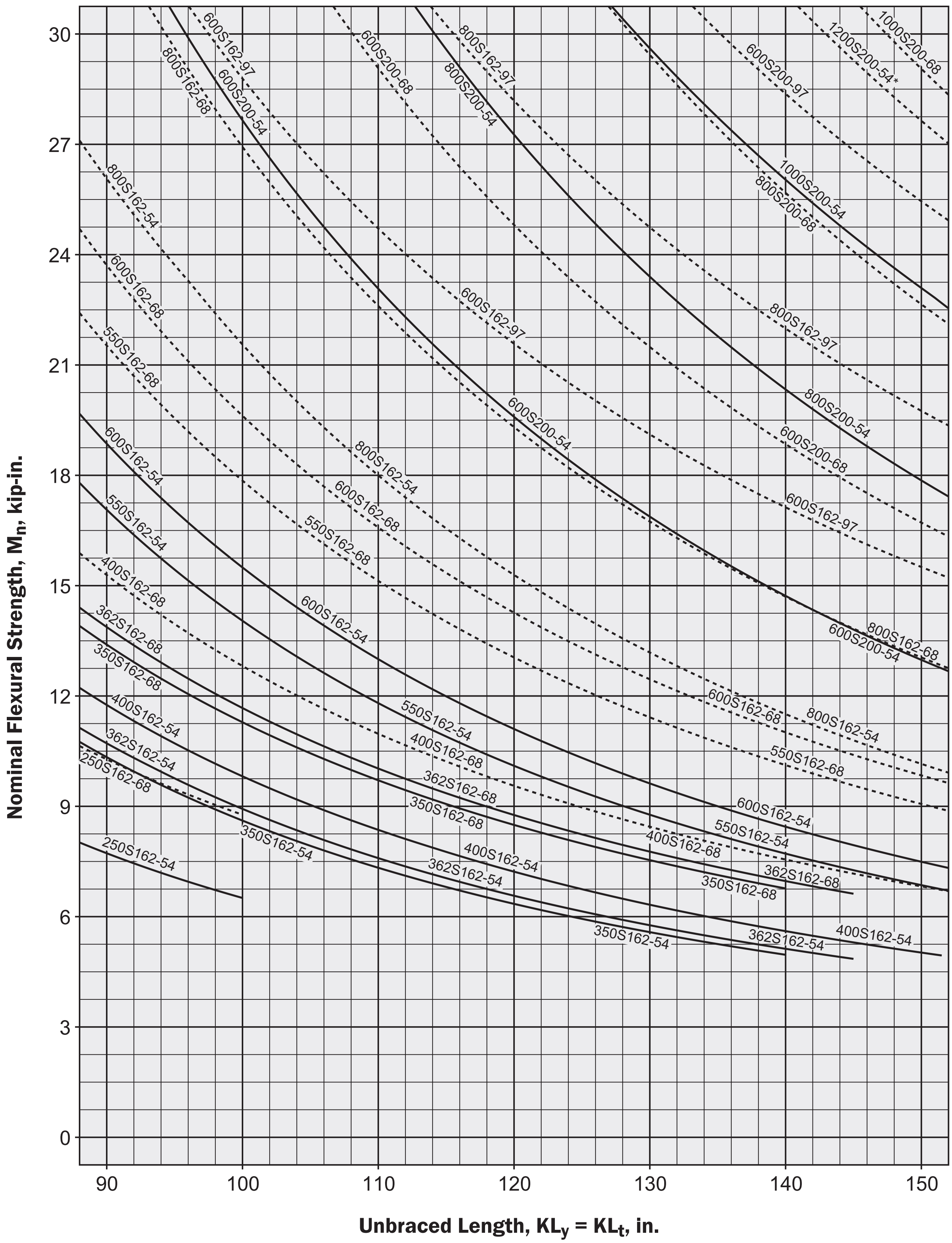


Chart II-2b (continued)

Nominal Flexural Strength
SSMA Studs - C-Sections with Lips, (F_y = 50 ksi, C_b = 1)

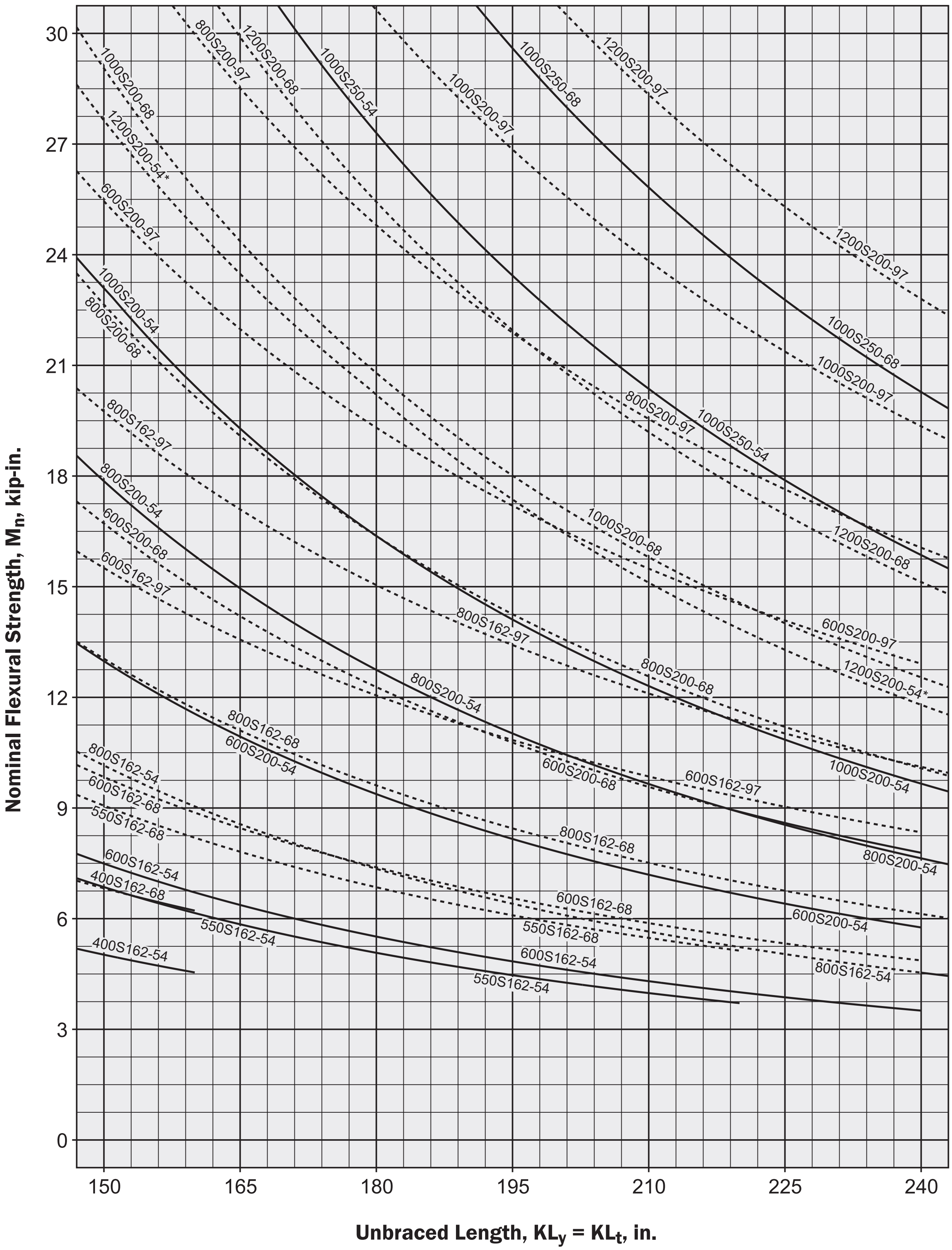


Chart II-2b (continued) **Nominal Flexural Strength**

SSMA Studs - C-Sections with Lips, (F_y = 50 ksi, C_b = 1)

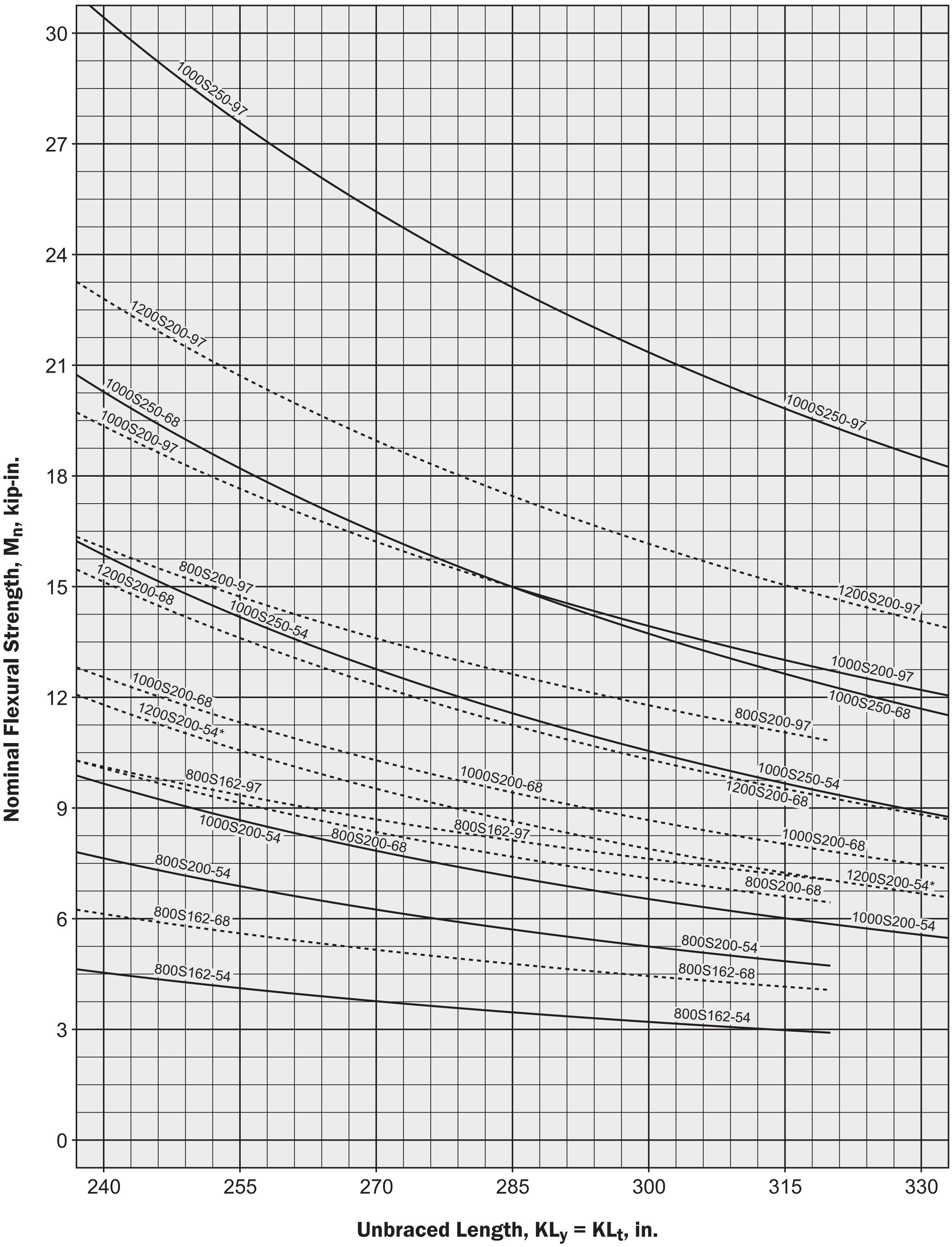


Chart II-2b (continued)

Nominal Flexural Strength
SSMA Studs - C-Sections with Lips, (F_y = 50 ksi, C_b = 1)

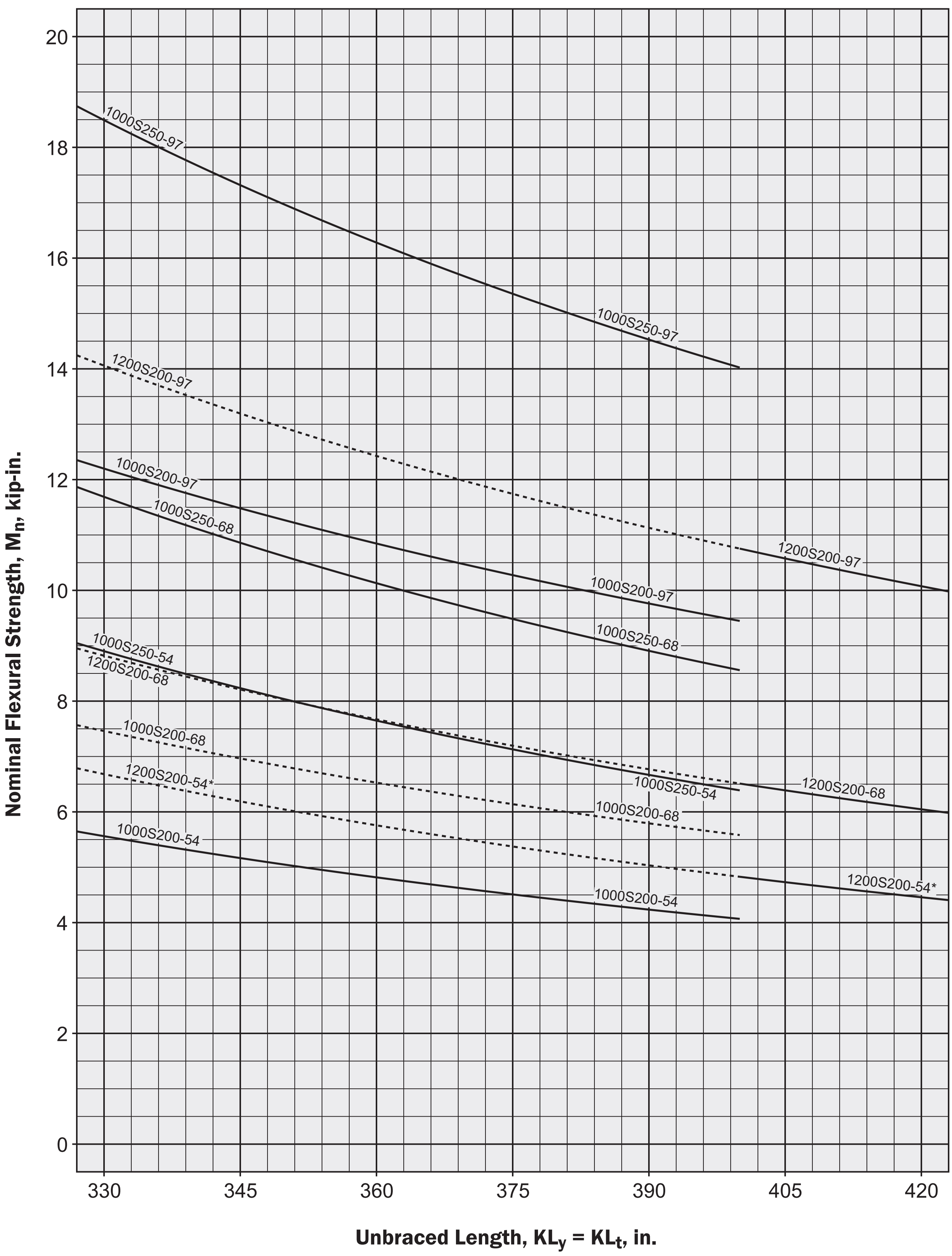


Chart II-3a

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 33 ksi, C_b = 1)

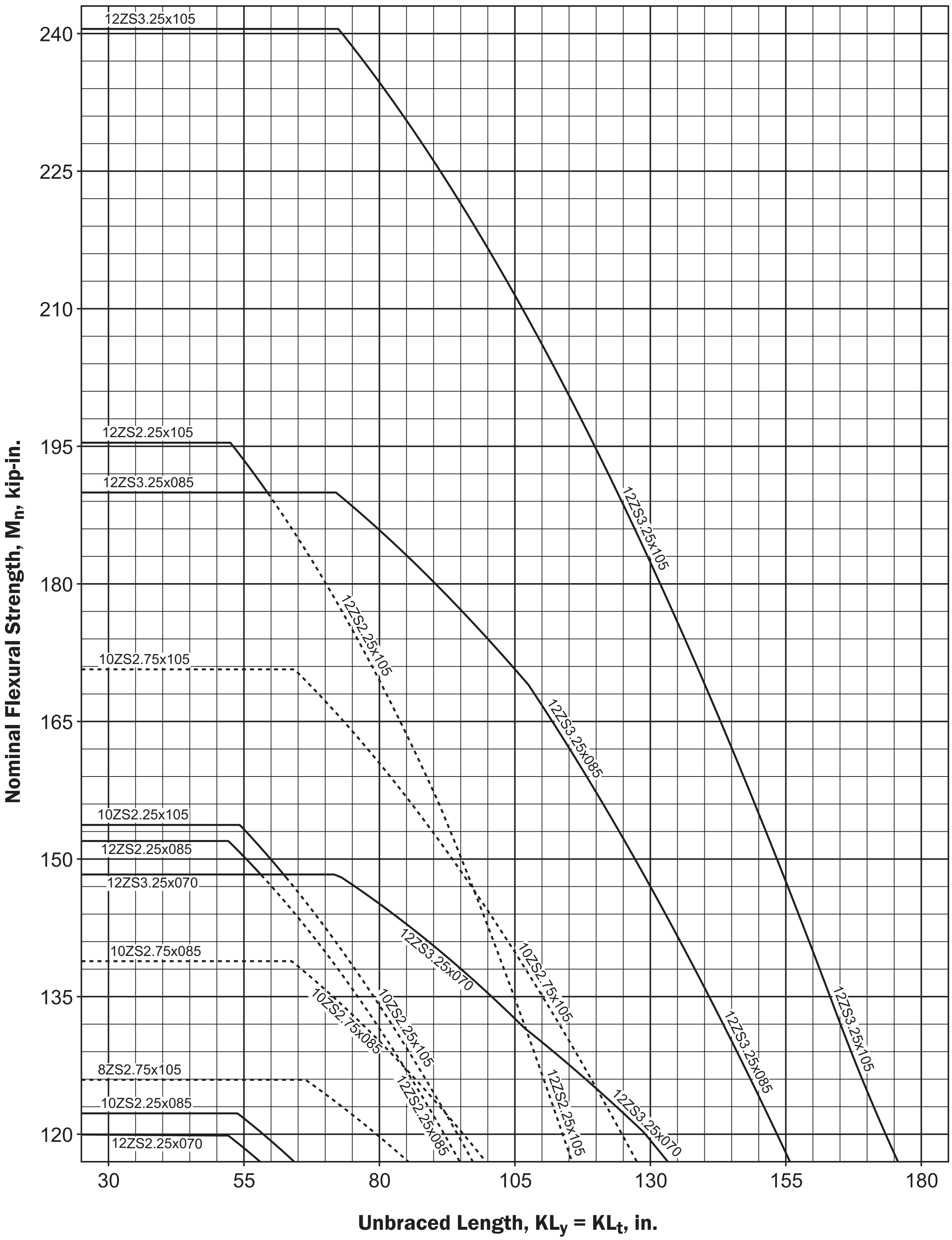


Chart II-3a (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 33 ksi, C_b = 1)

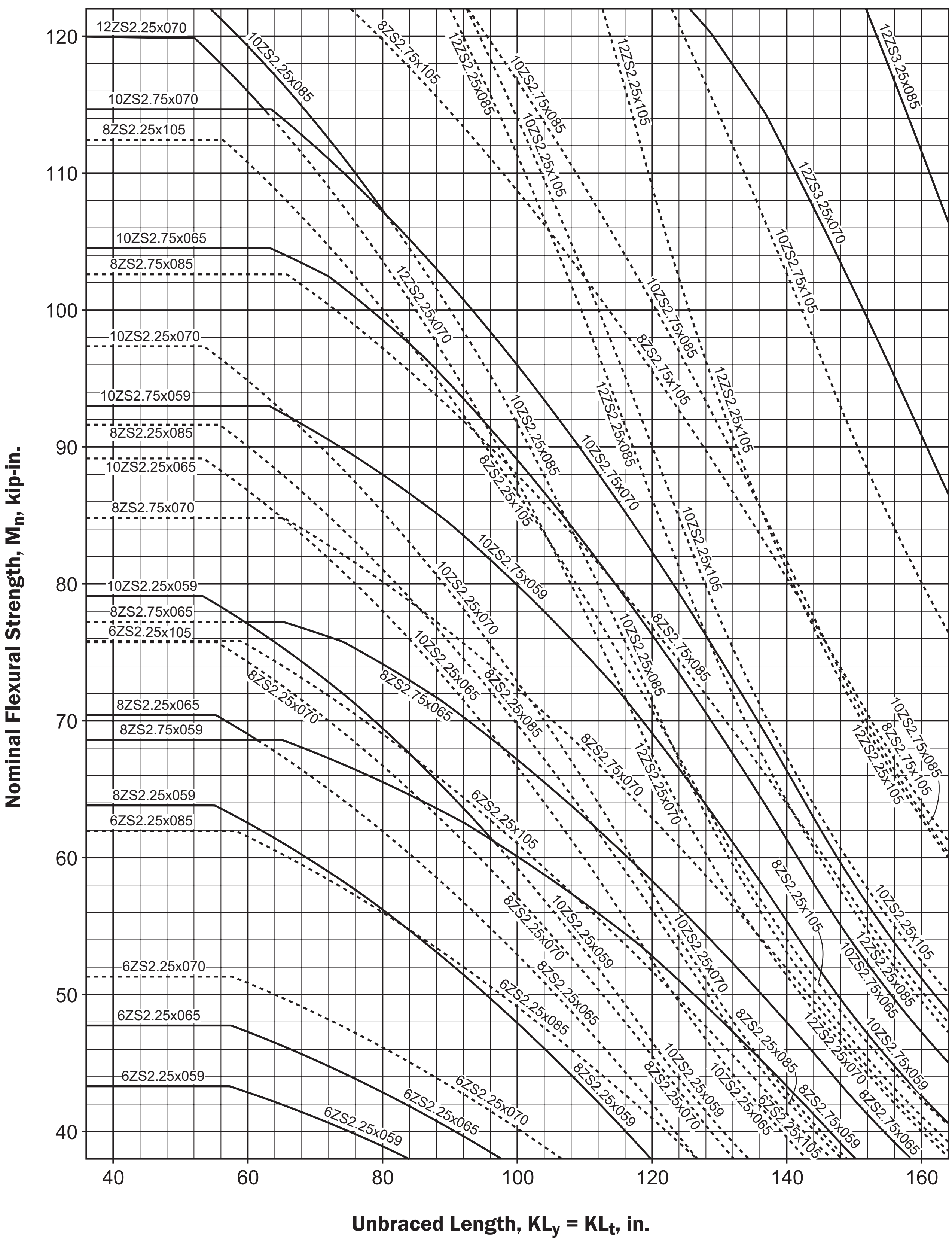


Chart II-3a (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 33 ksi, C_b = 1)

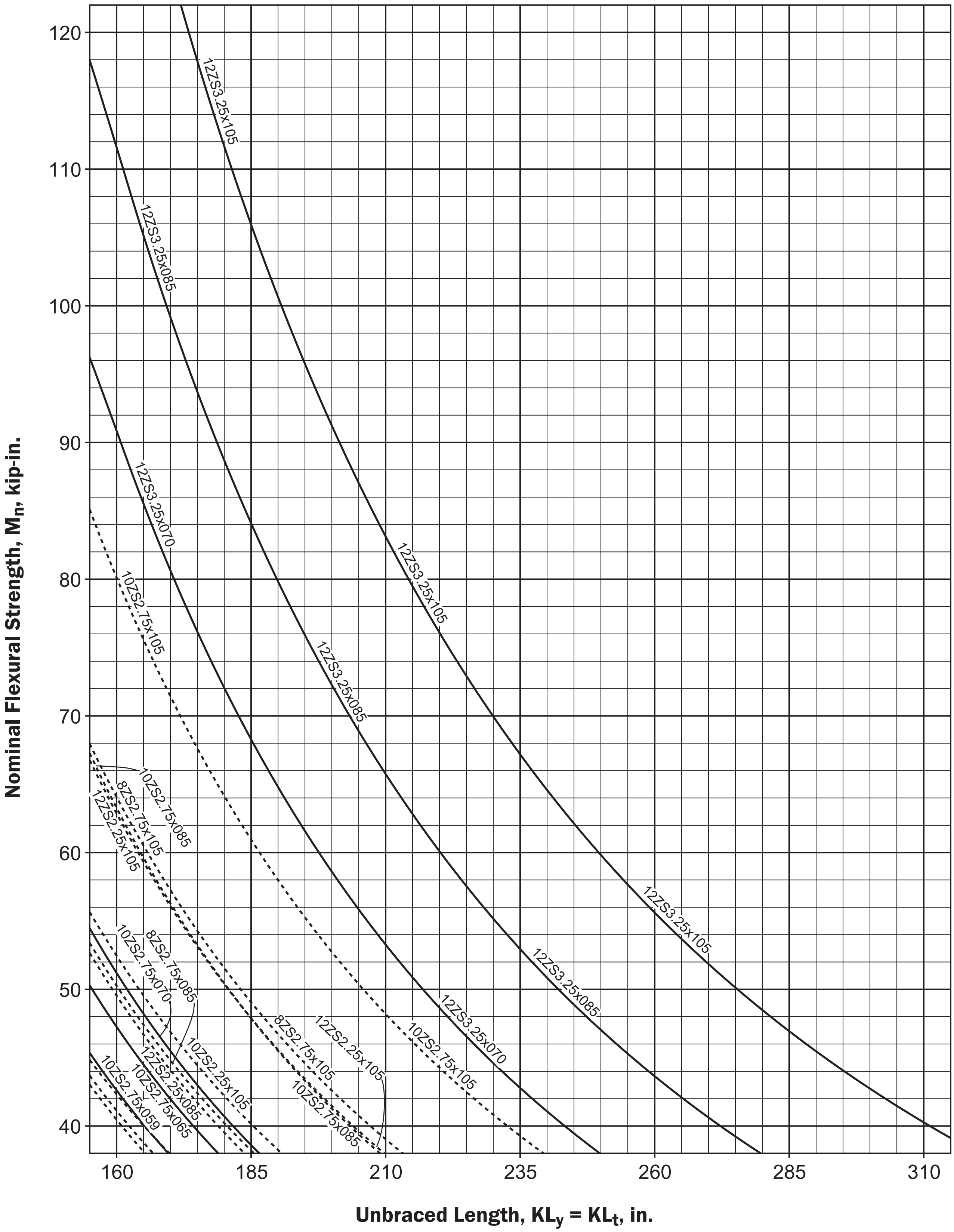


Chart II-3a (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 33 ksi, C_b = 1)

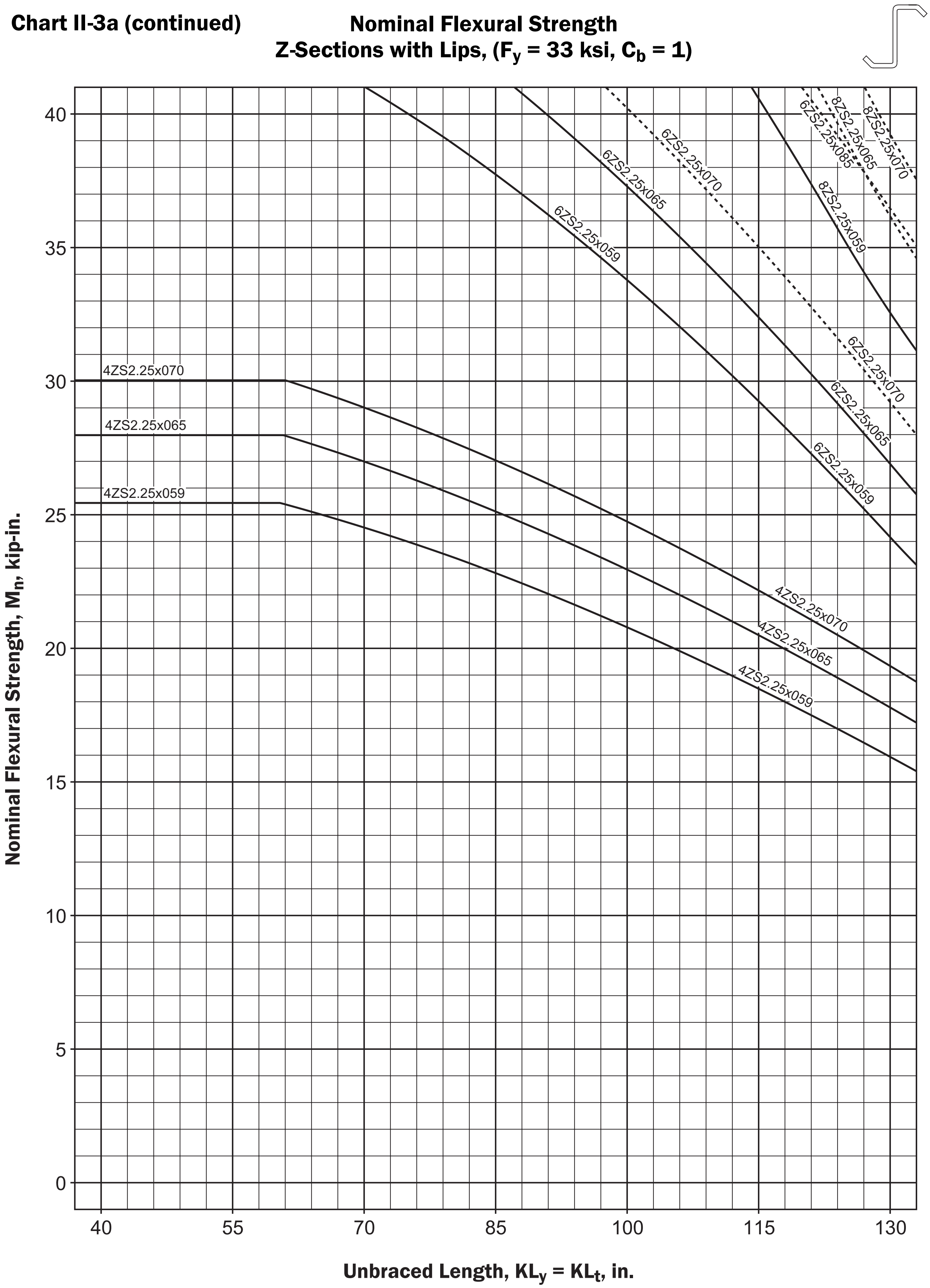
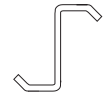


Chart II-3a (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 33 ksi, C_b = 1)



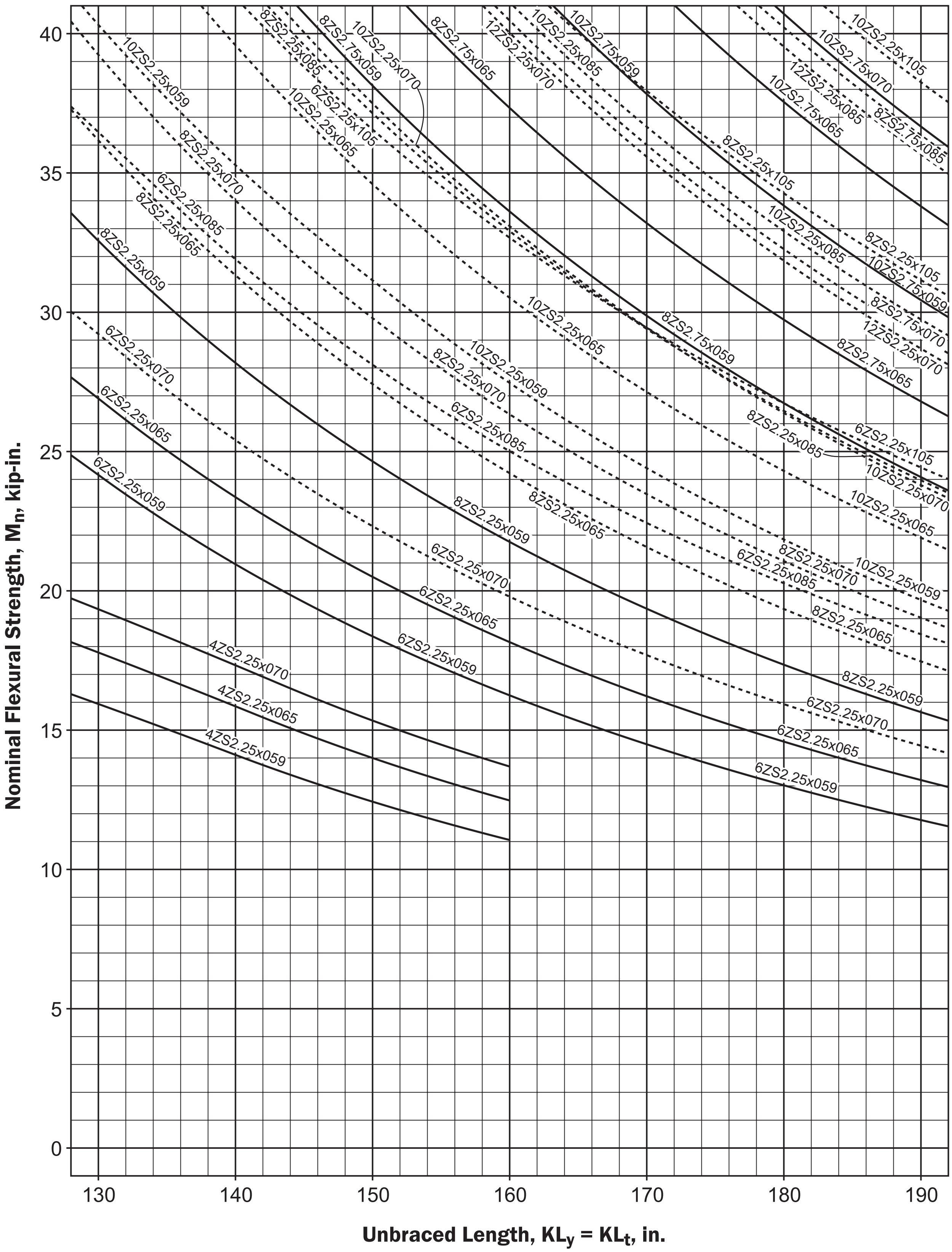


Chart II-3a (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 33 ksi, C_b = 1)

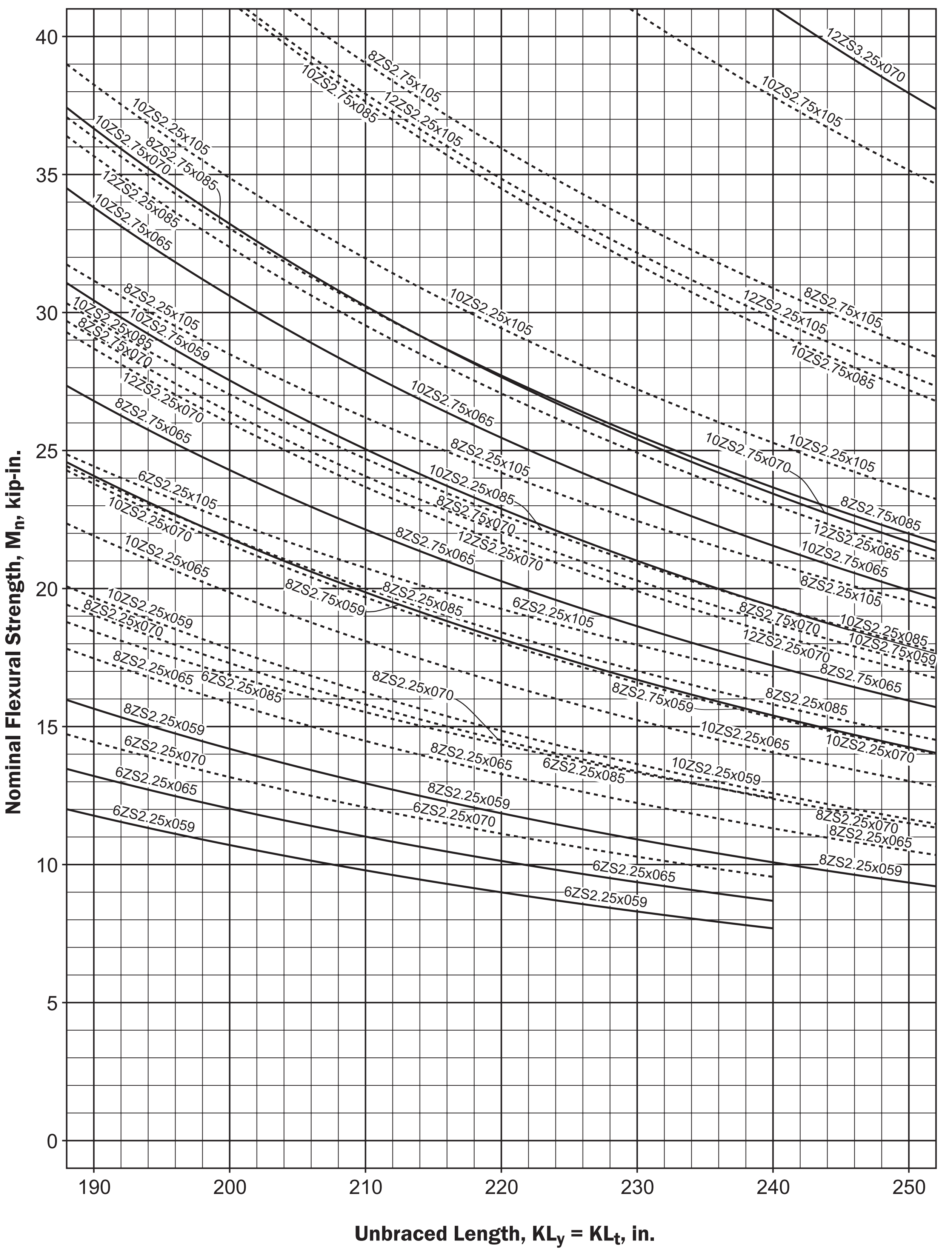


Chart II-3a (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 33 ksi, C_b = 1)

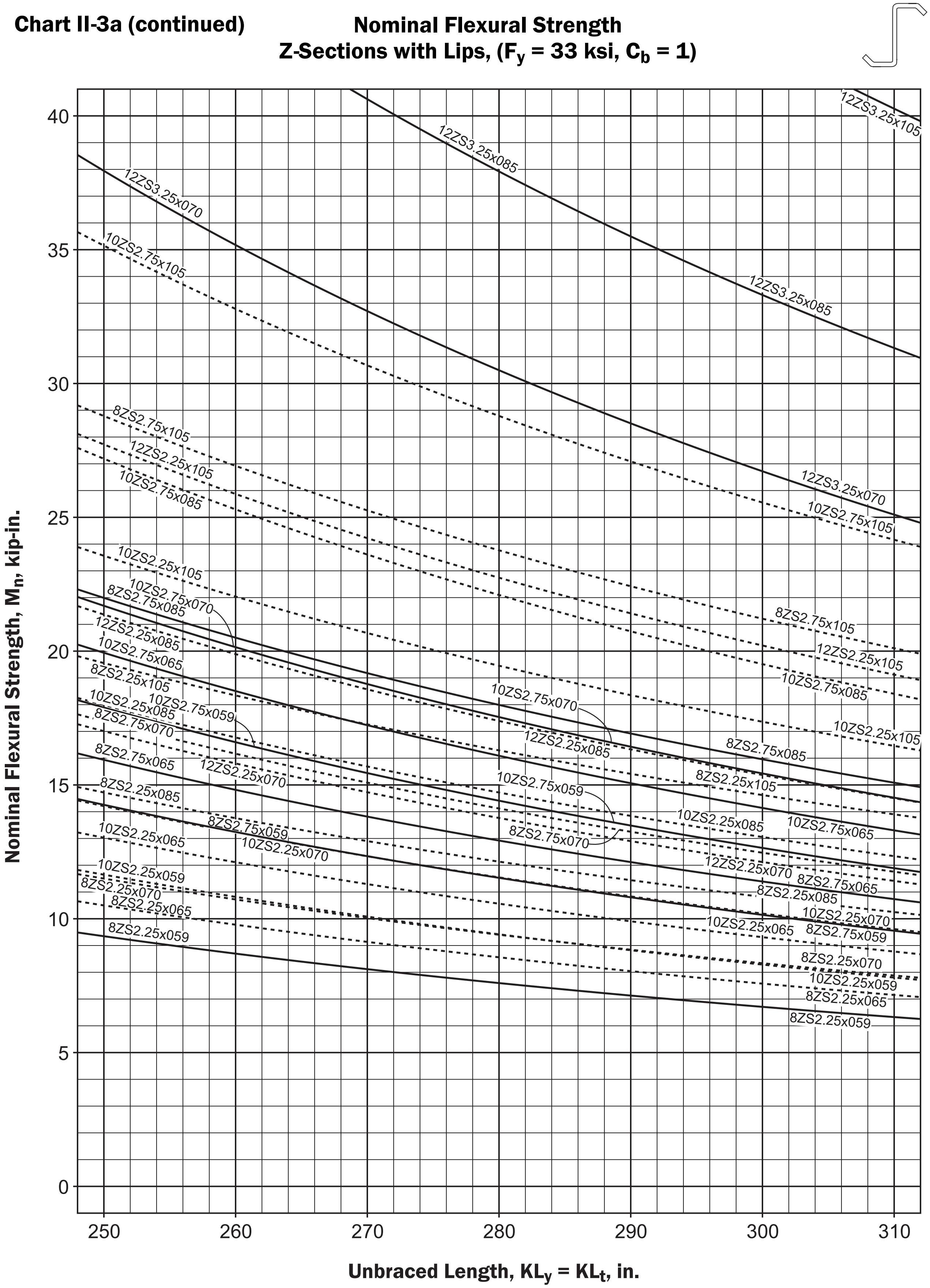


Chart II-3a (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 33 ksi, C_b = 1)

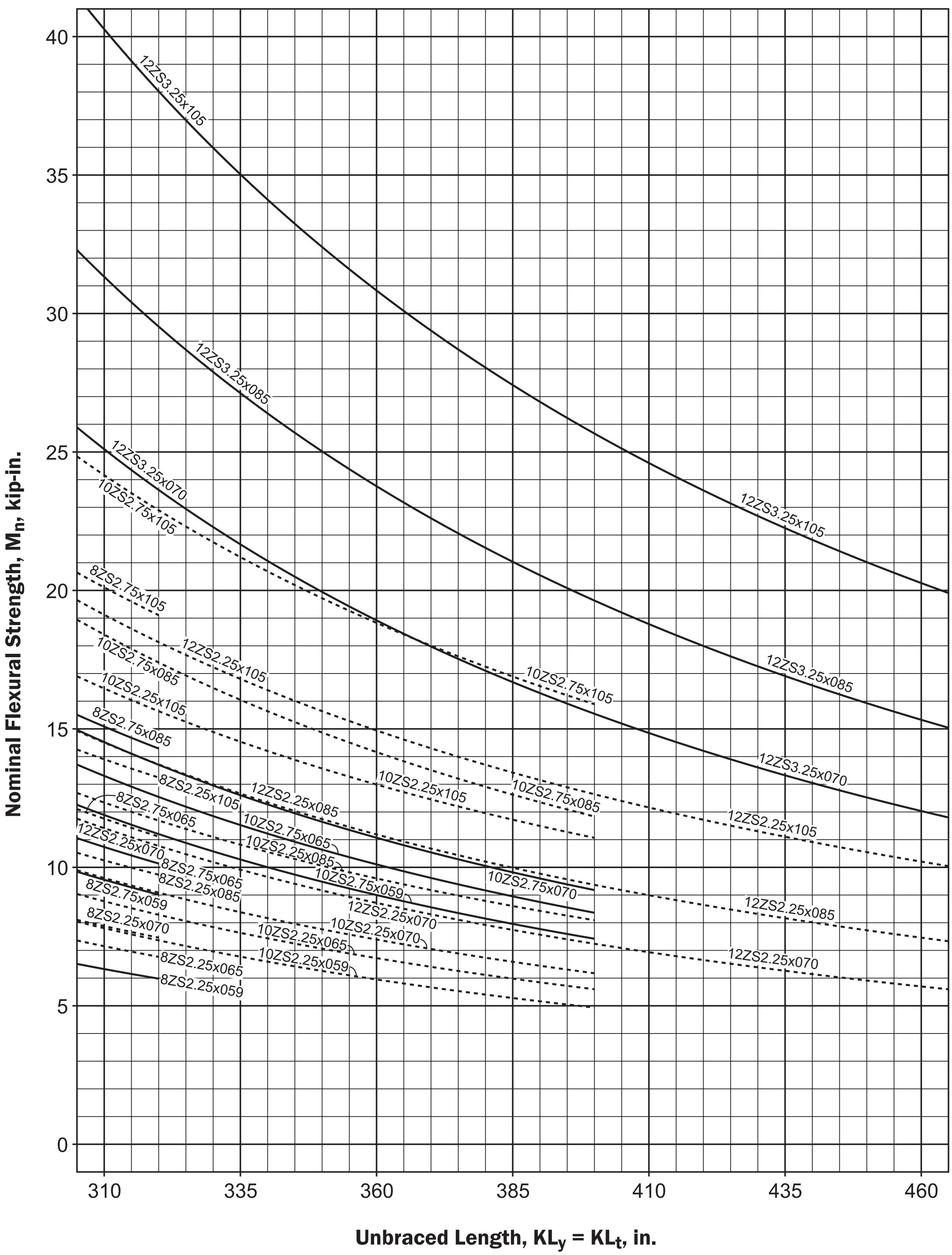


Chart II-3b

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 55 ksi, C_b = 1)

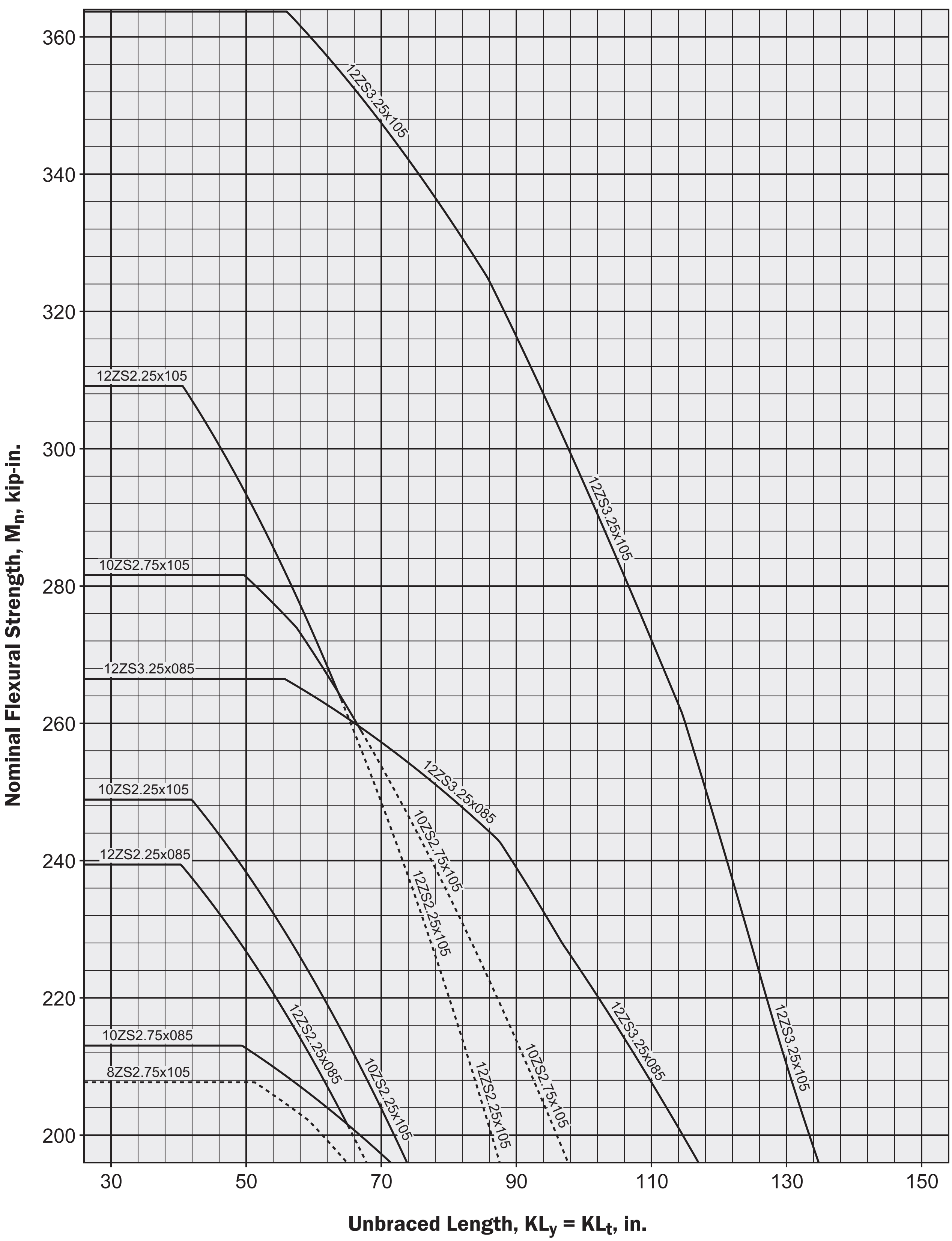


Chart II-3b (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 55 ksi, C_b = 1)

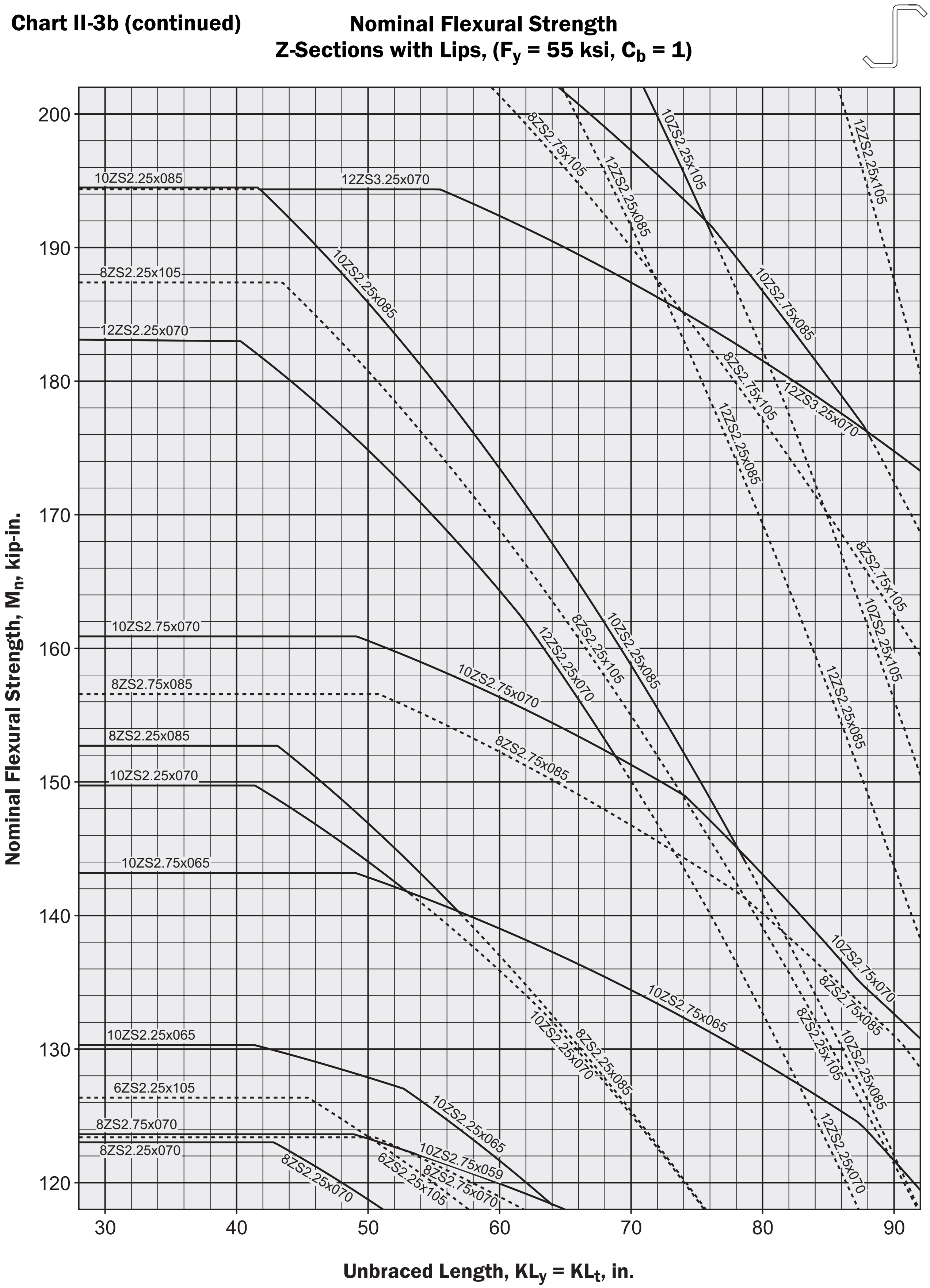


Chart II-3b (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 55 ksi, C_b = 1)

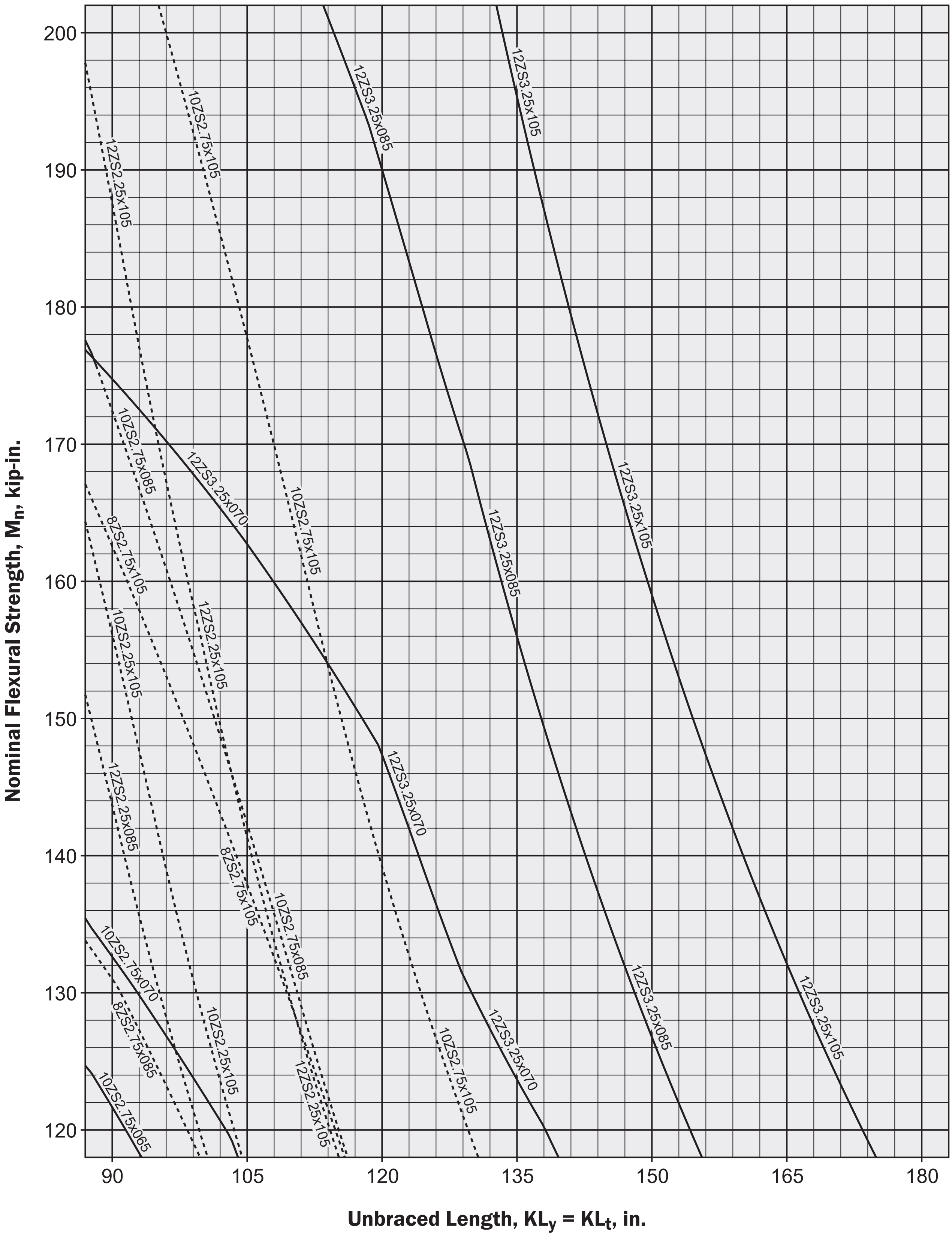
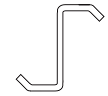


Chart II-3b (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 55 ksi, C_b = 1)



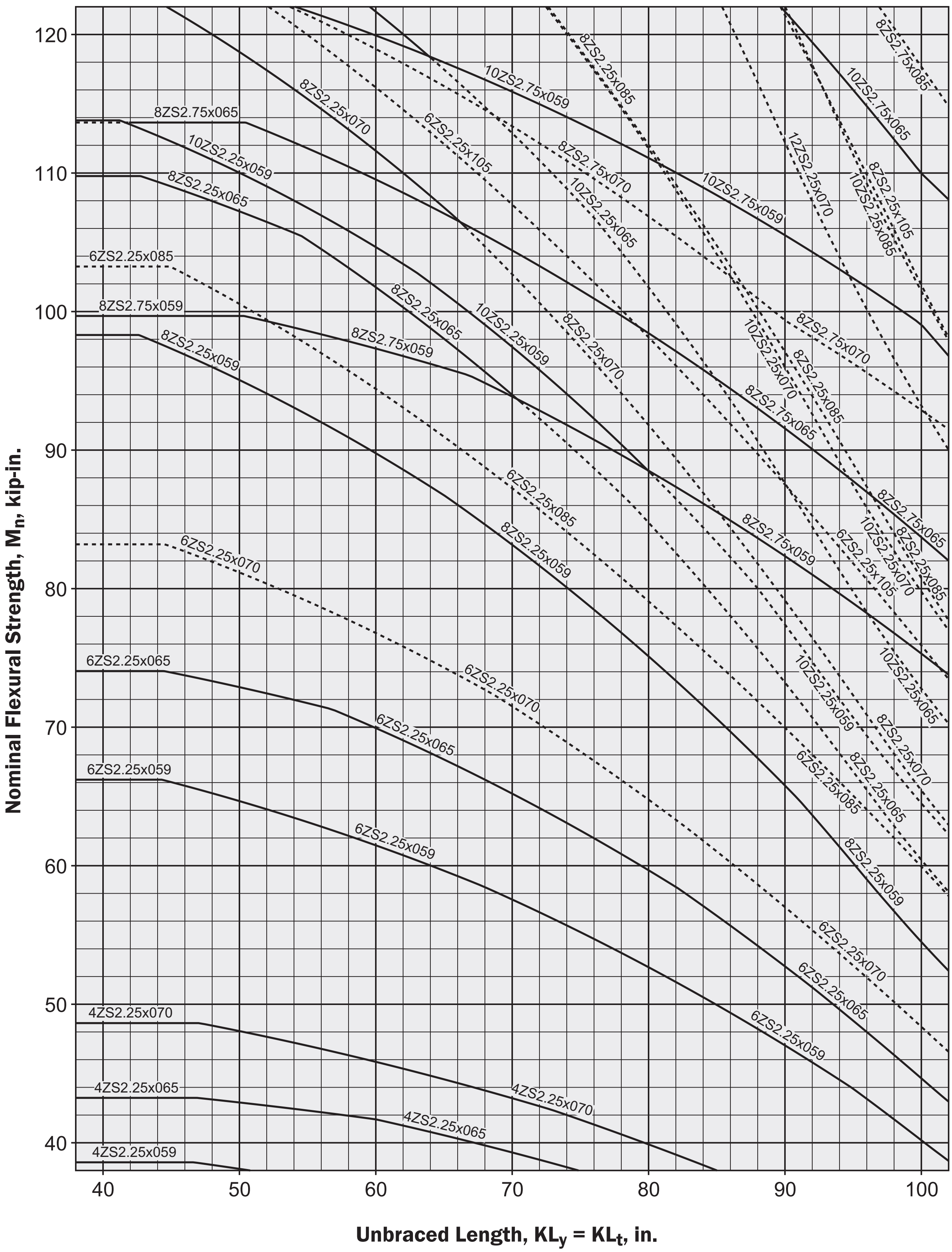
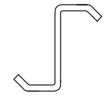


Chart II-3b (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 55 ksi, C_b = 1)



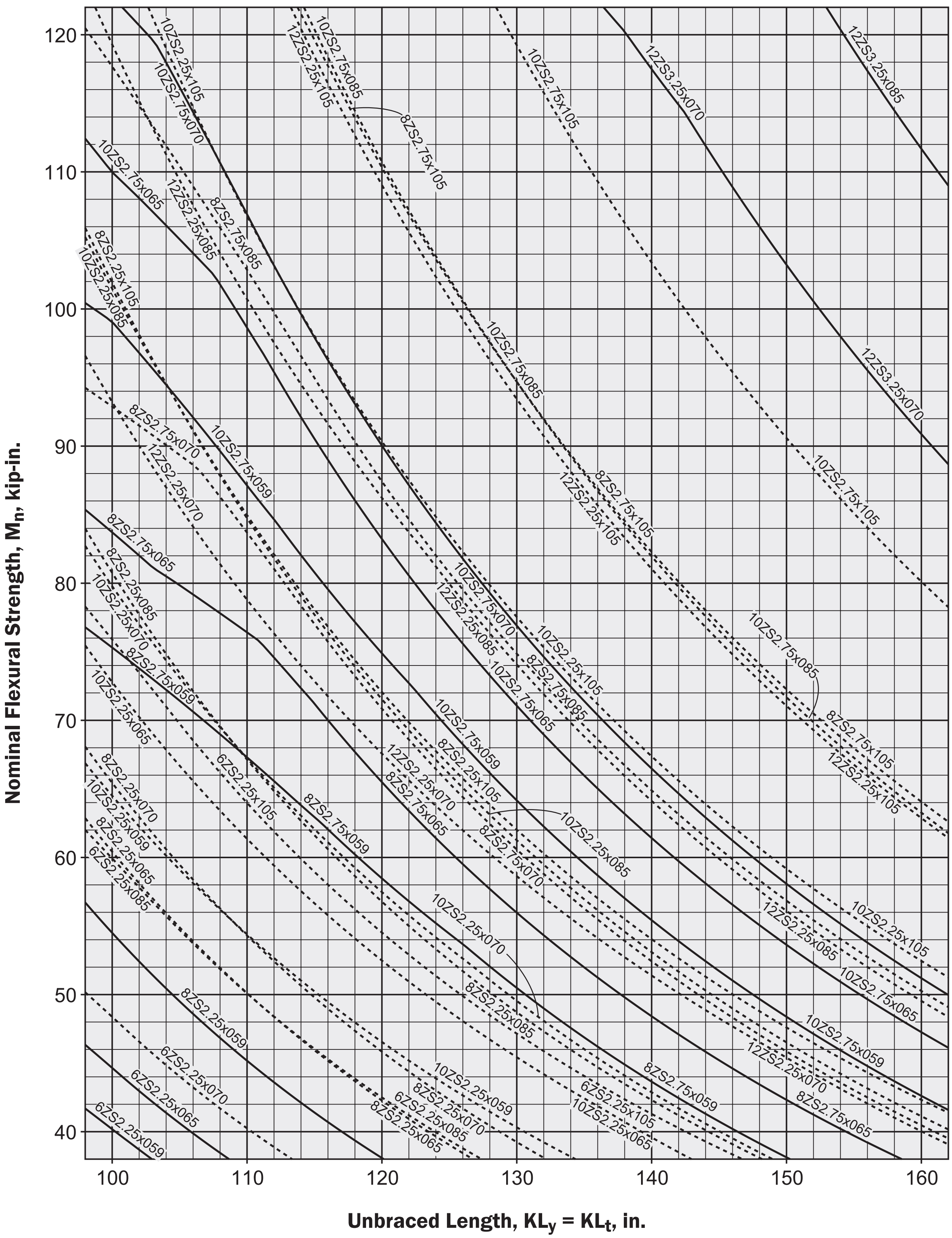


Chart II-3b (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 55 ksi, C_b = 1)

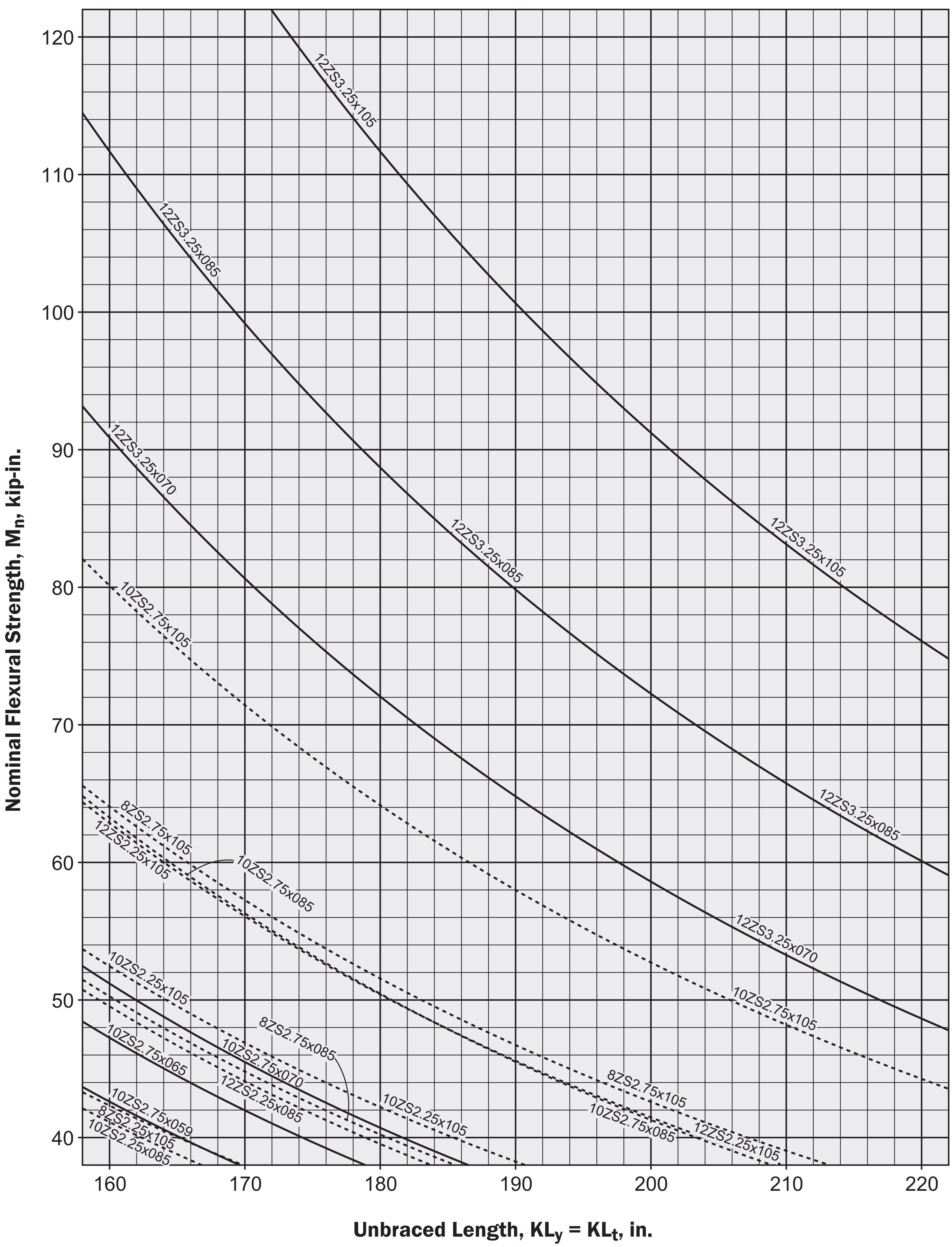


Chart II-3b (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 55 ksi, C_b = 1)

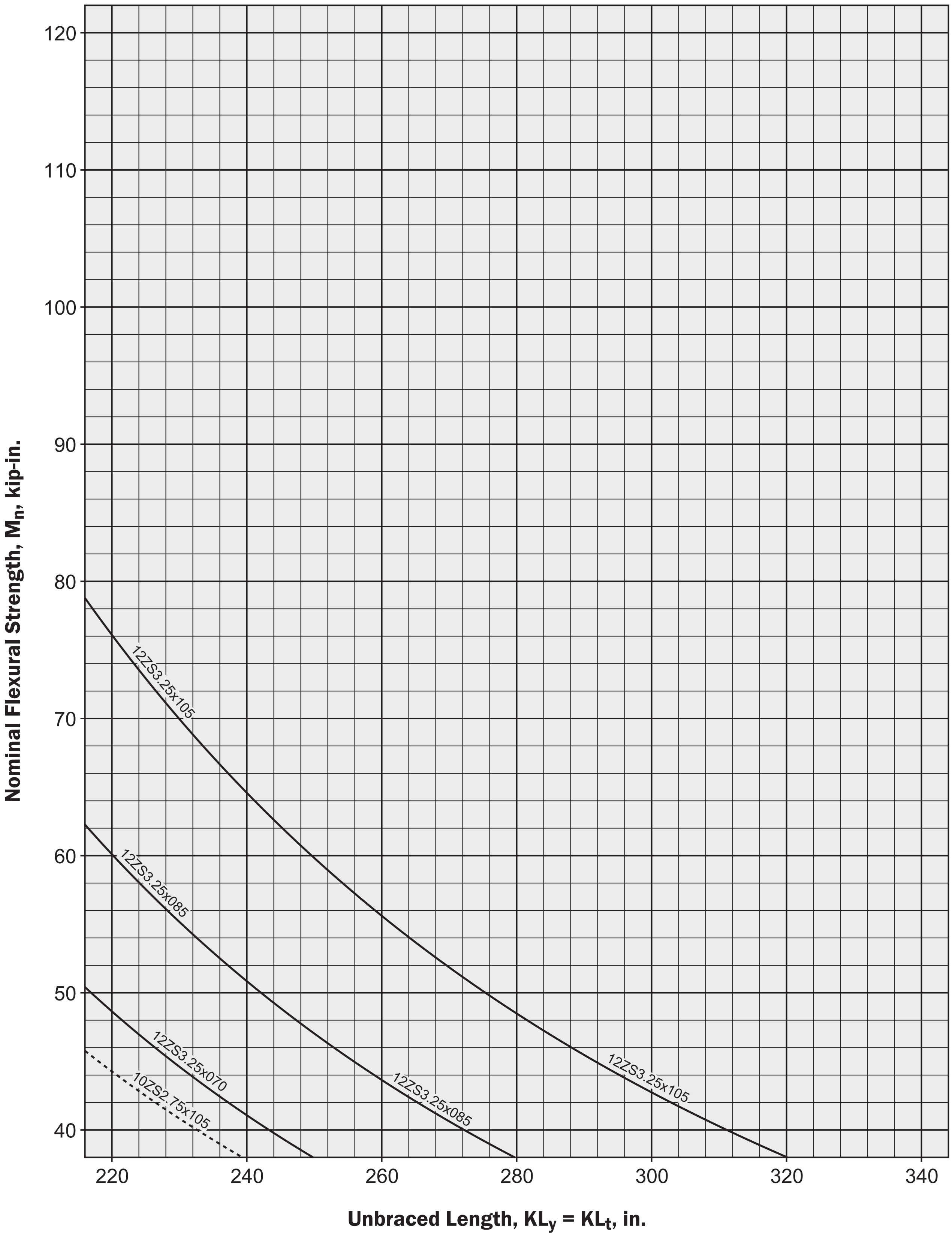


Chart II-3b (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 55 ksi, C_b = 1)

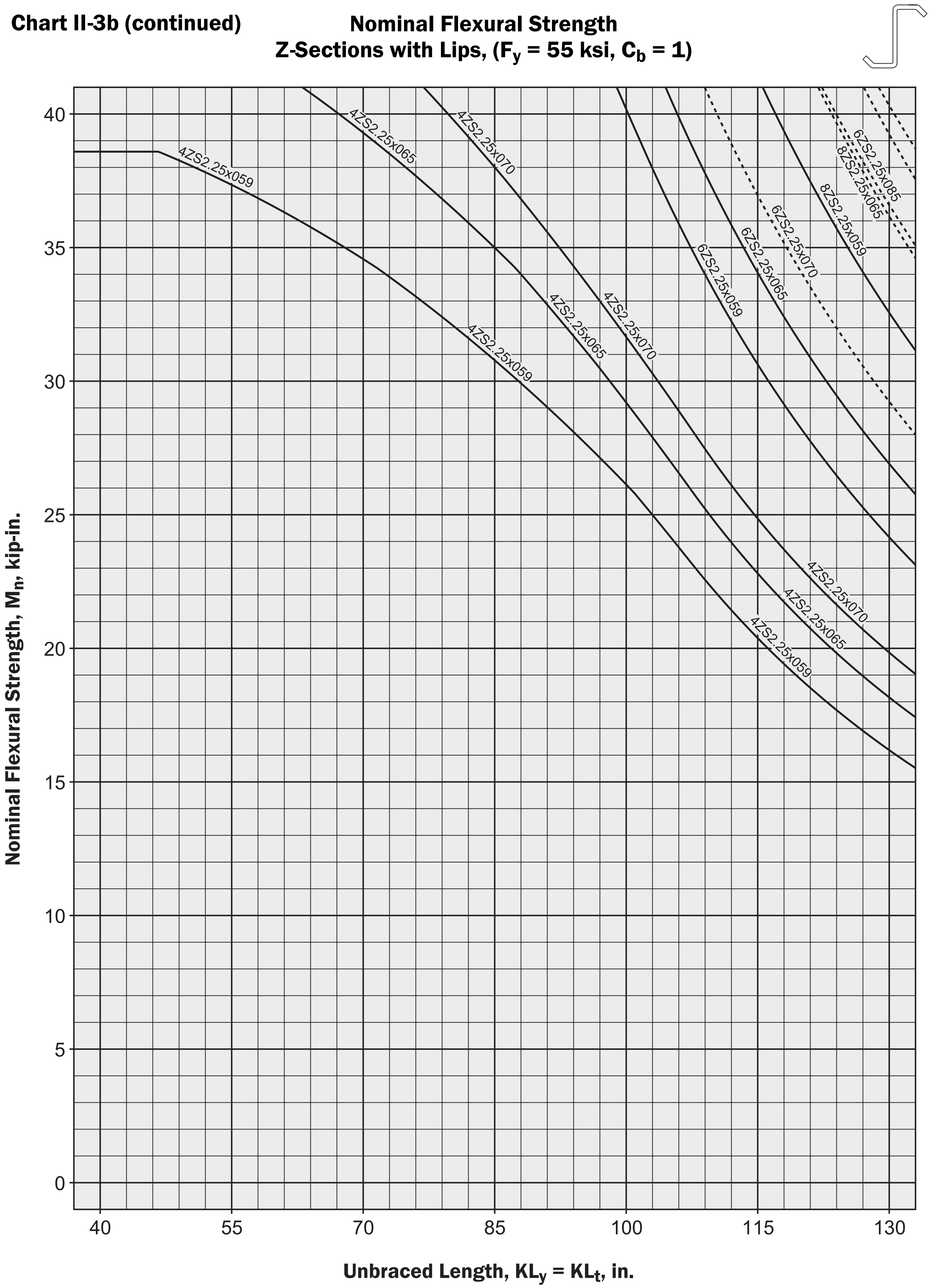
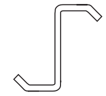


Chart II-3b (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 55 ksi, C_b = 1)



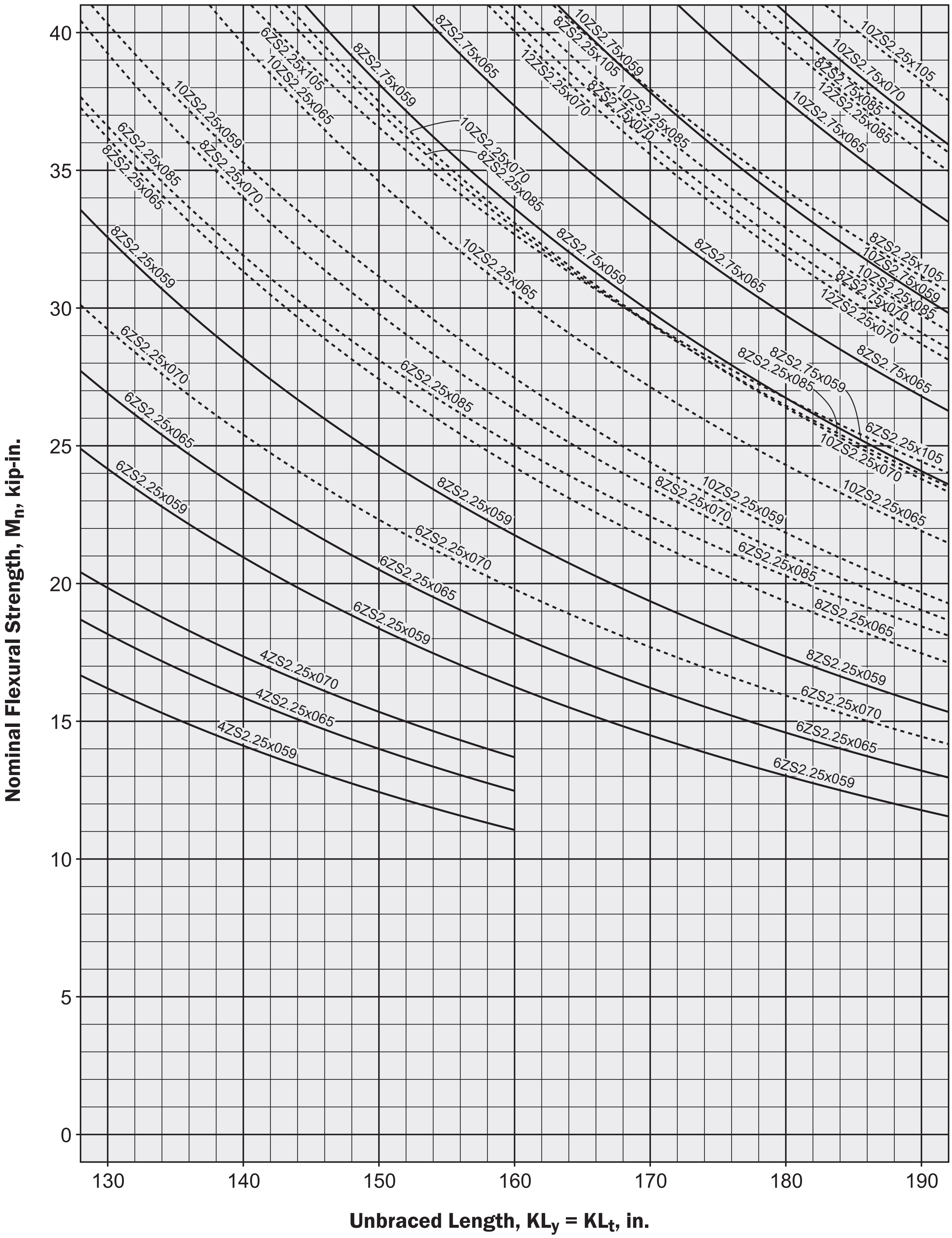
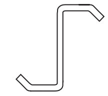


Chart II-3b (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 55 ksi, C_b = 1)



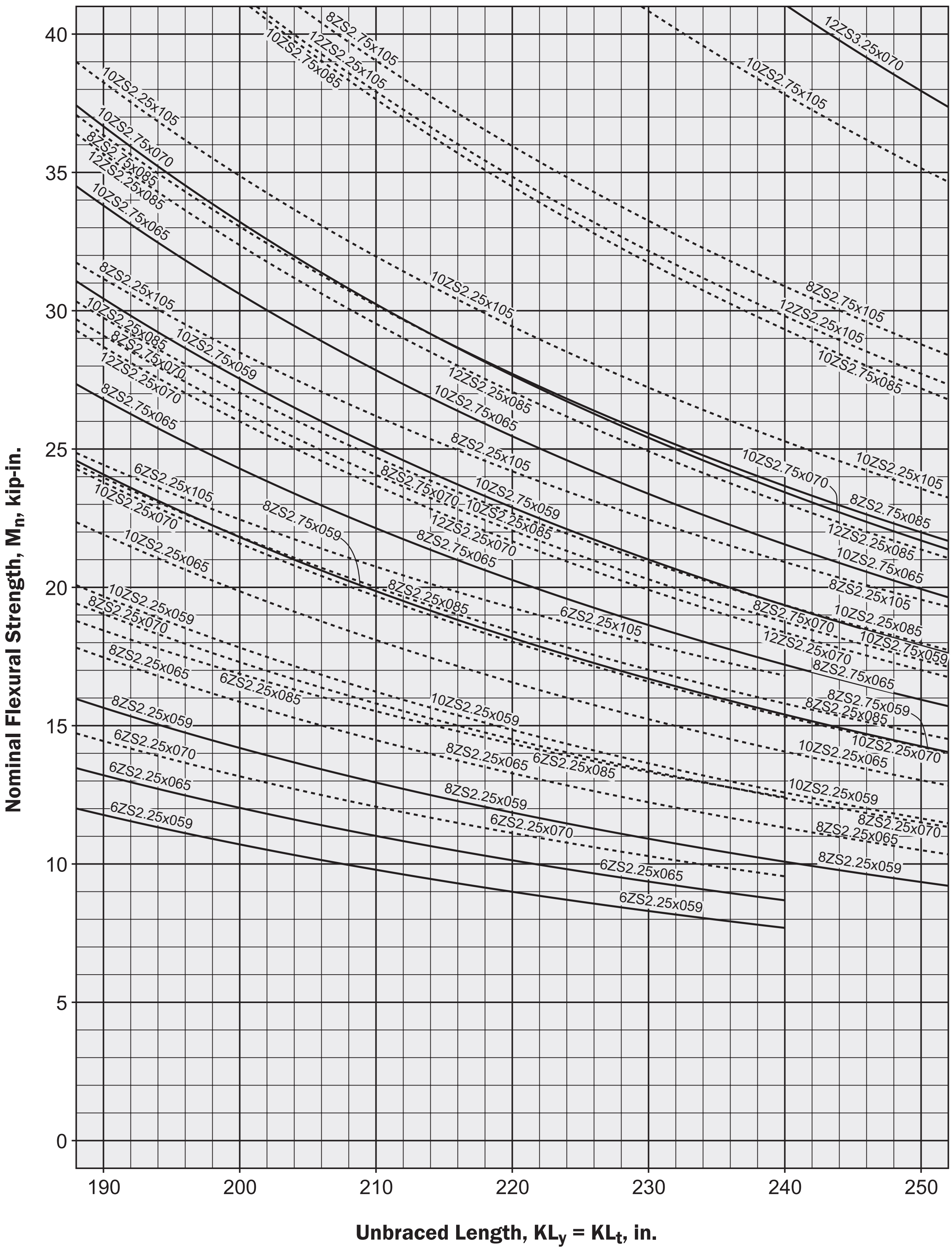
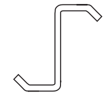


Chart II-3b (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 55 ksi, C_b = 1)



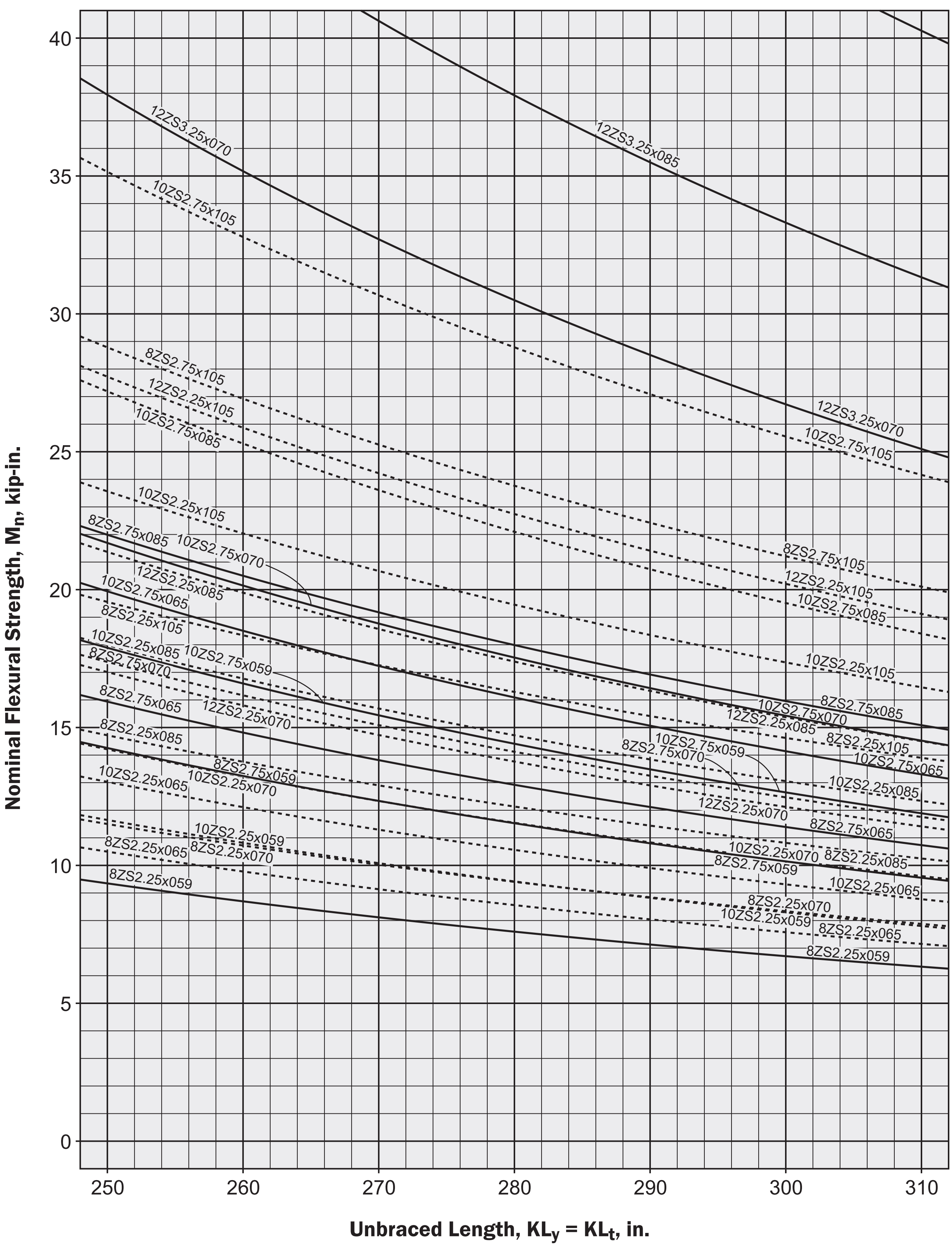


Chart II-3b (continued)

Nominal Flexural Strength
Z-Sections with Lips, (F_y = 55 ksi, C_b = 1)

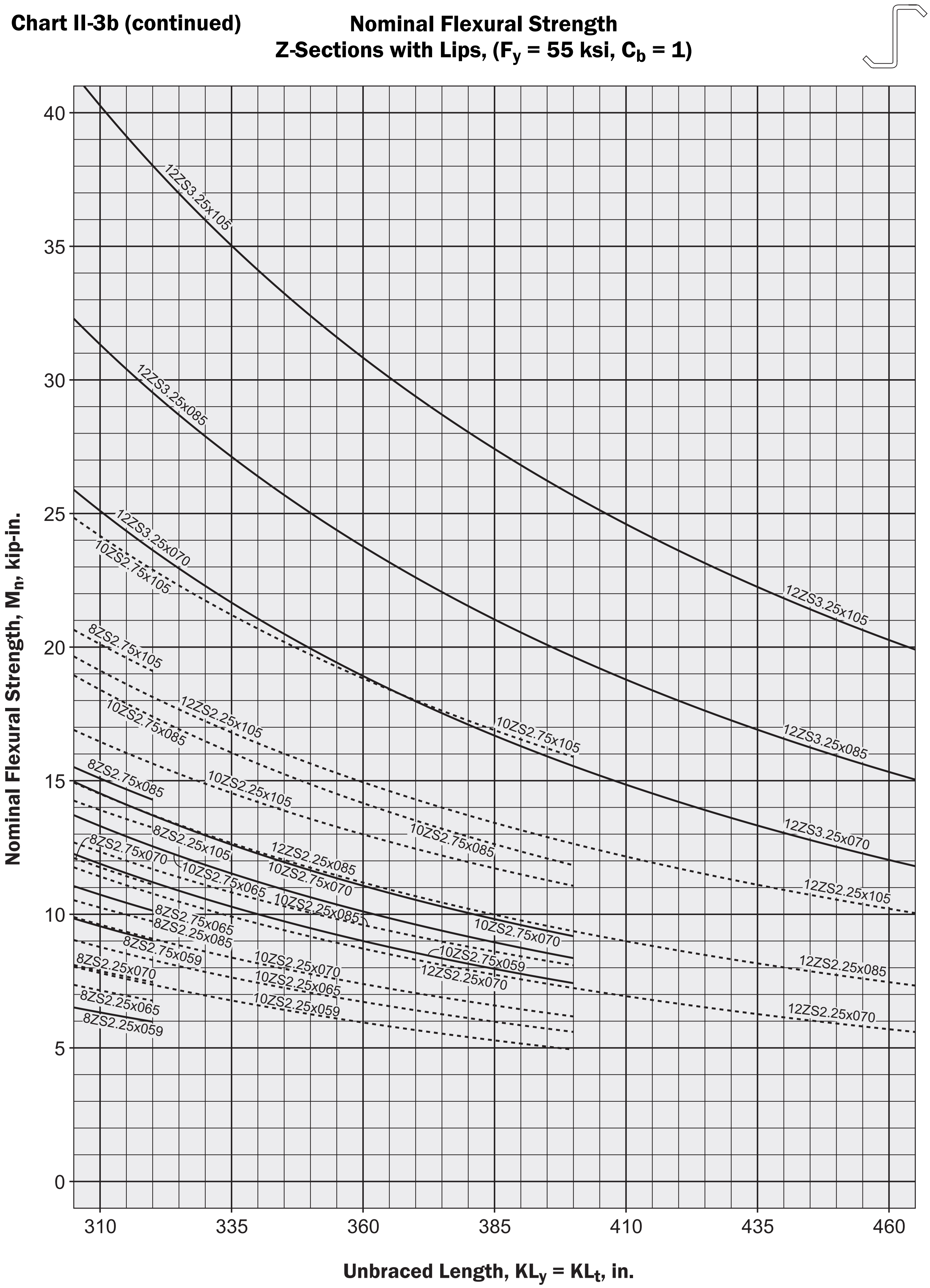


Table II - 7a

ASD - Combined Shear and Bending
C-Sections With Lips

Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
12CS3.5x105	9.02	0.00	9.02	0.00	10CS3.5x065	2.57	0.00	2.57	0.00
	8.72	36.3	8.72	55.5		2.49	15.8	2.49	21.6
	7.82	70.2	7.82	107		2.23	30.6	2.23	41.7
	6.38	99.2	6.38	152		1.82	43.3	1.82	59.0
	4.51	122	4.51	186		1.29	53.0	1.29	72.2
	2.34	136	2.34	207		0.666	59.1	0.666	80.6
	0.00	140	0.00	215		0.00	61.2	0.00	83.4
12CS3.5x085	4.77	0.00	4.77	0.00	8CS3.5x105	9.43	0.00	12.2	0.00
	4.61	28.3	4.61	40.4		9.11	21.0	11.8	31.7
	4.13	54.7	4.13	78.1		8.16	40.6	10.5	61.3
	3.37	77.3	3.37	110		6.67	57.5	8.61	86.6
	2.39	94.7	2.39	135		4.71	70.4	6.08	106
	1.23	106	1.23	151		2.44	78.5	3.15	118
	0.00	109	0.00	156		0.00	81.3	0.00	123
12CS3.5x070	2.66	0.00	2.66	0.00	8CS3.5x085	6.18	0.00	7.33	0.00
	2.57	21.9	2.57	29.1		5.97	16.3	7.08	24.4
	2.30	42.4	2.30	56.3		5.35	31.5	6.35	47.1
	1.88	59.9	1.88	79.6		4.37	44.6	5.18	66.6
	1.33	73.4	1.33	97.5		3.09	54.6	3.67	81.5
	0.688	81.8	0.688	109		1.60	60.9	1.90	90.9
	0.00	84.7	0.00	113		0.00	63.1	0.00	94.1
10CS3.5x105	9.43	0.00	10.9	0.00	8CS3.5x070	4.08	0.00	4.08	0.00
	9.11	28.3	10.6	43.0		3.94	12.7	3.94	19.3
	8.16	54.7	9.48	83.1		3.53	24.6	3.53	37.3
	6.67	77.4	7.74	117		2.88	34.8	2.88	52.7
	4.71	94.7	5.47	144		2.04	42.7	2.04	64.5
	2.44	106	2.83	160		1.06	47.6	1.06	72.0
	0.00	109	0.00	166		0.00	49.3	0.00	74.5
10CS3.5x085	5.78	0.00	5.78	0.00	8CS3.5x065	3.26	0.00	3.26	0.00
	5.58	22.0	5.58	33.2		3.15	11.7	3.15	17.4
	5.01	42.5	5.01	64.1		2.82	22.6	2.82	33.6
	4.09	60.2	4.09	90.6		2.31	31.9	2.31	47.5
	2.89	73.7	2.89	111		1.63	39.1	1.63	58.1
	1.50	82.2	1.50	124		0.844	43.6	0.844	64.8
	0.00	85.1	0.00	128		0.00	45.1	0.00	67.1
10CS3.5x070	3.22	0.00	3.22	0.00	8CS3.5x059	2.43	0.00	2.43	0.00
	3.11	17.3	3.11	24.4		2.35	10.4	2.35	14.8
	2.79	33.4	2.79	47.2		2.11	20.1	2.11	28.6
	2.28	47.2	2.28	66.7		1.72	28.4	1.72	40.4
	1.61	57.8	1.61	81.7		1.22	34.8	1.22	49.5
	0.833	64.4	0.833	91.1		0.630	38.9	0.630	55.2
	0.00	66.7	0.00	94.3		0.00	40.2	0.00	57.2

Table II - 7a (continued)

ASD - Combined Shear and Bending
C-Sections With Lips

Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
8CS2.5x105	9.43	0.00	12.2	0.00	6CS2.5x085	5.74	0.00	7.97	0.00
	9.11	17.4	11.8	29.1		5.54	9.63	7.70	15.6
	8.16	33.7	10.5	56.2		4.97	18.6	6.91	30.2
	6.67	47.7	8.61	79.4		4.06	26.3	5.64	42.7
	4.71	58.4	6.08	97.3		2.87	32.2	3.99	52.3
	2.44	65.1	3.15	109		1.49	35.9	2.06	58.4
	0.00	67.4	0.00	112		0.00	37.2	0.00	60.4
8CS2.5x085	6.18	0.00	7.33	0.00	6CS2.5x070	4.19	0.00	5.41	0.00
	5.97	14.2	7.08	23.1		4.05	7.99	5.22	12.0
	5.35	27.5	6.35	44.6		3.63	15.4	4.68	23.3
	4.37	38.9	5.18	63.1		2.96	21.8	3.82	32.9
	3.09	47.6	3.67	77.3		2.09	26.7	2.70	40.3
	1.60	53.1	1.90	86.2		1.08	29.8	1.40	44.9
	0.00	55.0	0.00	89.3		0.00	30.9	0.00	46.5
8CS2.5x070	4.08	0.00	4.08	0.00	6CS2.5x065	3.61	0.00	4.45	0.00
	3.94	11.8	3.94	17.9		3.49	7.44	4.30	11.0
	3.53	22.7	3.53	34.5		3.13	14.4	3.85	21.2
	2.88	32.2	2.88	48.8		2.55	20.3	3.14	29.9
	2.04	39.4	2.04	59.8		1.81	24.9	2.22	36.7
	1.06	43.9	1.06	66.7		0.935	27.8	1.15	40.9
	0.00	45.5	0.00	69.0		0.00	28.7	0.00	42.4
8CS2.5x065	3.26	0.00	3.26	0.00	6CS2.5x059	2.98	0.00	3.32	0.00
	3.15	11.0	3.15	16.3		2.87	6.69	3.21	9.83
	2.82	21.2	2.82	31.5		2.58	12.9	2.87	19.0
	2.31	29.9	2.31	44.5		2.10	18.3	2.35	26.8
	1.63	36.6	1.63	54.5		1.49	22.4	1.66	32.9
	0.844	40.9	0.844	60.8		0.770	25.0	0.859	36.7
	0.00	42.3	0.00	63.0		0.00	25.8	0.00	38.0
8CS2.5x059	2.43	0.00	2.43	0.00	4CS2.5x105	4.44	0.00	7.40	0.00
	2.35	9.84	2.35	14.4		4.29	6.84	7.14	11.4
	2.11	19.0	2.11	27.8		3.84	13.2	6.40	22.0
	1.72	26.9	1.72	39.4		3.14	18.7	5.23	31.1
	1.22	32.9	1.22	48.2		2.22	22.9	3.70	38.1
	0.630	36.7	0.630	53.8		1.15	25.5	1.91	42.5
	0.00	38.0	0.00	55.7		0.00	26.4	0.00	44.0
6CS2.5x105	7.04	0.00	11.7	0.00	4CS2.5x085	3.63	0.00	6.06	0.00
	6.80	11.8	11.3	19.6		3.51	5.63	5.85	9.17
	6.09	22.7	10.2	37.9		3.15	10.9	5.25	17.7
	4.98	32.2	8.29	53.6		2.57	15.4	4.28	25.0
	3.52	39.4	5.86	65.7		1.82	18.8	3.03	30.7
	1.82	43.9	3.04	73.2		0.941	21.0	1.57	34.2
	0.00	45.5	0.00	75.8		0.00	21.7	0.00	35.4

Table II - 7a (continued)

ASD - Combined Shear and Bending
C-Sections With Lips

Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
4CS2.5x070	3.02	0.00	5.03	0.00	4CS2.5x059	2.56	0.00	3.84	0.00
	2.92	4.69	4.86	7.03		2.47	3.94	3.71	5.72
	2.61	9.06	4.36	13.6		2.22	7.62	3.33	11.1
	2.13	12.8	3.56	19.2		1.81	10.8	2.72	15.6
	1.51	15.7	2.52	23.5		1.28	13.2	1.92	19.1
	0.781	17.5	1.30	26.3		0.663	14.7	0.995	21.3
	0.00	18.1	0.00	27.2		0.00	15.2	0.00	22.1
4CS2.5x065	2.81	0.00	4.66	0.00					
	2.72	4.37	4.50	6.39					
	2.43	8.45	4.04	12.4					
	1.99	11.9	3.30	17.5					
	1.41	14.6	2.33	21.4					
	0.728	16.3	1.21	23.9					
	0.00	16.9	0.00	24.7					

Notes:

1. Shear and moment strengths have been divided by the appropriate factors of safety. 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 - 7b

LRFD - Combined Shear and Bending
C-Sections With Lips

Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
12CS3.5x105	13.7	0.00	13.7	0.00	10CS3.5x065	3.91	0.00	3.91	0.00
	13.2	57.6	13.2	88.1		3.78	25.1	3.78	34.2
	11.9	111	11.9	170		3.39	48.5	3.39	66.2
	9.70	157	9.70	241		2.77	68.6	2.77	93.6
	6.86	193	6.86	295		1.96	84.0	1.96	115
	3.55	215	3.55	329		1.01	93.7	1.01	128
	0.00	223	0.00	340		0.00	97.0	0.00	132
12CS3.5x085	7.25	0.00	7.25	0.00	8CS3.5x105	14.3	0.00	18.5	0.00
	7.00	44.9	7.00	64.1		13.8	33.4	17.9	50.3
	6.28	86.7	6.28	124		12.4	64.5	16.0	97.2
	5.13	123	5.13	175		10.1	91.2	13.1	137
	3.63	150	3.63	215		7.16	112	9.25	168
	1.88	168	1.88	239		3.71	125	4.79	188
	0.00	173	0.00	248		0.00	129	0.00	194
12CS3.5x070	4.04	0.00	4.04	0.00	8CS3.5x085	9.39	0.00	11.1	0.00
	3.90	34.8	3.90	46.2		9.07	25.9	10.8	38.6
	3.50	67.2	3.50	89.3		8.13	50.0	9.65	74.7
	2.86	95.1	2.86	126		6.64	70.8	7.88	106
	2.02	116	2.02	155		4.69	86.7	5.57	129
	1.05	130	1.05	172		2.43	96.7	2.88	144
	0.00	134	0.00	179		0.00	100	0.00	149
10CS3.5x105	14.3	0.00	16.6	0.00	8CS3.5x070	6.20	0.00	6.20	0.00
	13.8	44.9	16.1	68.2		5.99	20.2	5.99	30.6
	12.4	86.8	14.4	132		5.37	39.1	5.37	59.1
	10.1	123	11.8	186		4.38	55.3	4.38	83.6
	7.16	150	8.32	228		3.10	67.7	3.10	102
	3.71	168	4.30	255		1.60	75.5	1.60	114
	0.00	174	0.00	264		0.00	78.1	0.00	118
10CS3.5x085	8.79	0.00	8.79	0.00	8CS3.5x065	4.96	0.00	4.96	0.00
	8.49	34.9	8.49	52.6		4.79	18.5	4.79	27.6
	7.61	67.5	7.61	102		4.29	35.8	4.29	53.2
	6.21	95.4	6.21	144		3.50	50.6	3.50	75.3
	4.39	117	4.39	176		2.48	62.0	2.48	92.2
	2.27	130	2.27	196		1.28	69.1	1.28	103
	0.00	135	0.00	203		0.00	71.6	0.00	106
10CS3.5x070	4.89	0.00	4.89	0.00	8CS3.5x059	3.70	0.00	3.70	0.00
	4.72	27.4	4.72	38.7		3.57	16.5	3.57	23.5
	4.24	52.9	4.24	74.8		3.20	31.9	3.20	45.3
	3.46	74.8	3.46	106		2.62	45.1	2.62	64.1
	2.45	91.7	2.45	130		1.85	55.3	1.85	78.5
	1.27	102	1.27	145		0.958	61.6	0.958	87.6
	0.00	106	0.00	150		0.00	63.8	0.00	90.7

Table II - 7b (continued)

LRFD - Combined Shear and Bending
C-Sections With Lips

Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
8CS2.5x105	14.3	0.00	18.5	0.00	6CS2.5x085	8.72	0.00	12.1	0.00
	13.8	27.7	17.9	46.1		8.42	15.3	11.7	24.8
	12.4	53.5	16.0	89.1		7.55	29.5	10.5	47.9
	10.1	75.6	13.1	126		6.17	41.7	8.57	67.8
	7.16	92.6	9.25	154		4.36	51.1	6.06	83.0
	3.71	103	4.79	172		2.26	57.0	3.14	92.6
	0.00	107	0.00	178		0.00	59.0	0.00	95.9
8CS2.5x085	9.39	0.00	11.1	0.00	6CS2.5x070	6.37	0.00	8.22	0.00
	9.07	22.6	10.8	36.7		6.15	12.7	7.94	19.1
	8.13	43.6	9.65	70.8		5.51	24.5	7.12	36.9
	6.64	61.6	7.88	100		4.50	34.6	5.81	52.2
	4.69	75.5	5.57	123		3.18	42.4	4.11	63.9
	2.43	84.2	2.88	137		1.65	47.3	2.13	71.3
	0.00	87.2	0.00	142		0.00	49.0	0.00	73.8
8CS2.5x070	6.20	0.00	6.20	0.00	6CS2.5x065	5.49	0.00	6.76	0.00
	5.99	18.7	5.99	28.3		5.30	11.8	6.53	17.4
	5.37	36.1	5.37	54.8		4.76	22.8	5.85	33.6
	4.38	51.0	4.38	77.4		3.88	32.2	4.78	47.5
	3.10	62.5	3.10	94.8		2.75	39.5	3.38	58.2
	1.60	69.7	1.60	106		1.42	44.0	1.75	64.9
	0.00	72.2	0.00	110		0.00	45.6	0.00	67.2
8CS2.5x065	4.96	0.00	4.96	0.00	6CS2.5x059	4.52	0.00	5.04	0.00
	4.79	17.4	4.79	25.9		4.37	10.6	4.87	15.6
	4.29	33.6	4.29	50.0		3.92	20.5	4.37	30.1
	3.50	47.5	3.50	70.7		3.20	29.0	3.57	42.6
	2.48	58.1	2.48	86.5		2.26	35.5	2.52	52.2
	1.28	64.8	1.28	96.5		1.17	39.6	1.31	58.2
	0.00	67.1	0.00	99.9		0.00	41.0	0.00	60.2
8CS2.5x059	3.70	0.00	3.70	0.00	4CS2.5x105	6.74	0.00	11.2	0.00
	3.57	15.6	3.57	22.9		6.52	10.8	10.9	18.1
	3.20	30.2	3.20	44.2		5.84	21.0	9.74	34.9
	2.62	42.7	2.62	62.5		4.77	29.6	7.95	49.4
	1.85	52.2	1.85	76.5		3.37	36.3	5.62	60.5
	0.958	58.3	0.958	85.3		1.75	40.5	2.91	67.5
	0.00	60.3	0.00	88.4		0.00	41.9	0.00	69.9
6CS2.5x105	10.7	0.00	17.8	0.00	4CS2.5x085	5.52	0.00	9.21	0.00
	10.3	18.7	17.2	31.1		5.34	8.93	8.89	14.5
	9.26	36.1	15.4	60.1		4.78	17.2	7.97	28.1
	7.56	51.0	12.6	85.1		3.91	24.4	6.51	39.7
	5.35	62.5	8.91	104		2.76	29.9	4.60	48.7
	2.77	69.7	4.61	116		1.43	33.3	2.38	54.3
	0.00	72.2	0.00	120		0.00	34.5	0.00	56.2

Table II - 7b (continued)

LRFD - Combined Shear and Bending
C-Sections With Lips

Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
4CS2.5x070	4.59	0.00	7.65	0.00	4CS2.5x059	3.89	0.00	5.84	0.00
	4.43	7.44	7.39	11.2		3.76	6.26	5.64	9.08
	3.97	14.4	6.62	21.6		3.37	12.1	5.06	17.5
	3.24	20.3	5.41	30.5		2.75	17.1	4.13	24.8
	2.29	24.9	3.82	37.3		1.95	20.9	2.92	30.4
	1.19	27.8	1.98	41.7		1.01	23.4	1.51	33.9
	0.00	28.7	0.00	43.1		0.00	24.2	0.00	35.1
4CS2.5x065	4.27	0.00	7.09	0.00					
	4.13	6.94	6.85	10.1					
	3.70	13.4	6.14	19.6					
	3.02	19.0	5.01	27.7					
	2.14	23.2	3.54	33.9					
	1.11	25.9	1.83	37.9					
	0.00	26.8	0.00	39.2					

Notes:

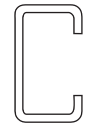
1. Shear and moment strengths have been multiplied by the appropriate resistance factors. This table is for LRFD use only.
2. Shear strengths have been calculated assuming no transverse stiffeners.
3. Linear interpolation between values is permitted.

Table II - 8a

ASD - Combined Shear and Bending
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
1200S200-97	8.15	0.00	8.15	0.00	1000S250-43*	0.836	0.00	-	-
	7.87	25.1	7.87	36.1		0.807	8.27	-	-
	7.05	48.4	7.05	69.8		0.724	16.0	-	-
	5.76	68.5	5.76	98.7		0.591	22.6	-	-
	4.07	83.8	4.07	121		0.418	27.7	-	-
	2.11	93.5	2.11	135		0.216	30.9	-	-
	0.00	96.8	0.00	140		0.00	31.9	-	-
1200S200-68	2.77	0.00	2.77	0.00	1000S200-97	8.84	0.00	9.86	0.00
	2.68	16.4	2.68	23.0		8.54	19.8	9.53	29.0
	2.40	31.8	2.40	44.4		7.66	38.2	8.54	56.0
	1.96	44.9	1.96	62.7		6.25	54.0	6.97	79.2
	1.38	55.0	1.38	76.8		4.42	66.2	4.93	97.0
	0.717	61.4	0.717	85.7		2.29	73.8	2.55	108
	0.00	63.5	0.00	88.7		0.00	76.4	0.00	112
1200S200-54*	1.38	0.00	1.38	0.00	1000S200-68	3.35	0.00	3.35	0.00
	1.33	12.4	1.33	16.1		3.23	13.3	3.23	18.8
	1.19	24.0	1.19	31.0		2.90	25.8	2.90	36.2
	0.974	33.9	0.974	43.9		2.37	36.4	2.37	51.2
	0.688	41.5	0.688	53.8		1.67	44.6	1.67	62.8
	0.356	46.3	0.356	60.0		0.866	49.8	0.866	70.0
	0.00	47.9	0.00	62.1		0.00	51.5	0.00	72.5
1000S250-97	8.84	0.00	9.86	0.00	1000S200-54	1.66	0.00	1.66	0.00
	8.54	25.5	9.53	36.4		1.60	10.1	1.60	13.2
	7.66	49.2	8.54	70.3		1.44	19.6	1.44	25.5
	6.25	69.6	6.97	99.4		1.17	27.7	1.17	36.1
	4.42	85.2	4.93	122		0.830	34.0	0.830	44.2
	2.29	95.1	2.55	136		0.430	37.9	0.430	49.3
	0.00	98.4	0.00	141		0.00	39.2	0.00	51.1
1000S250-68	3.35	0.00	3.35	0.00	1000S200-43*	0.836	0.00	-	-
	3.23	17.1	3.23	21.5		0.807	7.52	-	-
	2.90	33.0	2.90	41.4		0.724	14.5	-	-
	2.37	46.6	2.37	58.6		0.591	20.5	-	-
	1.67	57.1	1.67	71.8		0.418	25.2	-	-
	0.866	63.7	0.866	80.1		0.216	28.1	-	-
	0.00	65.9	0.00	82.9		0.00	29.1	-	-
1000S250-54	1.66	0.00	1.66	0.00	800S200-97	8.84	0.00	10.9	0.00
	1.60	11.6	1.60	14.6		8.54	16.9	10.5	25.0
	1.44	22.5	1.44	28.1		7.66	32.6	9.43	48.3
	1.17	31.8	1.17	39.8		6.25	46.1	7.70	68.3
	0.830	39.0	0.830	48.7		4.42	56.4	5.44	83.7
	0.430	43.5	0.430	54.4		2.29	62.9	2.82	93.3
	0.00	45.0	0.00	56.3		0.00	65.1	0.00	96.6

Table II - 8a (continued)



ASD - Combined Shear and Bending
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
800S200-68	4.22	0.00	4.22	0.00	800S162-54	2.09	0.00	2.09	0.00
	4.08	11.7	4.08	16.9		2.02	6.82	2.02	9.52
	3.65	22.6	3.65	32.6		1.81	13.2	1.81	18.4
	2.98	32.0	2.98	46.1		1.48	18.6	1.48	26.0
	2.11	39.2	2.11	56.5		1.05	22.8	1.05	31.9
	1.09	43.7	1.09	63.0		0.541	25.5	0.541	35.5
	0.00	45.3	0.00	65.2		0.00	26.4	0.00	36.8
800S200-54	2.09	0.00	2.09	0.00	800S162-43	1.05	0.00	-	-
	2.02	9.25	2.02	11.6		1.01	5.21	-	-
	1.81	17.9	1.81	22.4		0.910	10.1	-	-
	1.48	25.3	1.48	31.7		0.743	14.2	-	-
	1.05	31.0	1.05	38.9		0.525	17.4	-	-
	0.541	34.5	0.541	43.3		0.272	19.5	-	-
	0.00	35.8	0.00	44.9		0.00	20.1	-	-
800S200-43	1.05	0.00	-	-	800S162-33*	0.474	0.00	-	-
	1.01	6.61	-	-		0.458	3.63	-	-
	0.910	12.8	-	-		0.410	7.01	-	-
	0.743	18.1	-	-		0.335	9.92	-	-
	0.525	22.1	-	-		0.237	12.1	-	-
	0.272	24.7	-	-		0.123	13.5	-	-
	0.00	25.5	-	-		0.00	14.0	-	-
800S200-33*	0.474	0.00	-	-	600S200-97	6.91	0.00	10.5	0.00
	0.458	4.16	-	-		6.68	11.3	10.1	16.7
	0.410	8.03	-	-		5.99	21.7	9.07	32.3
	0.335	11.4	-	-		4.89	30.8	7.40	45.6
	0.237	13.9	-	-		3.46	37.7	5.24	55.9
	0.123	15.5	-	-		1.79	42.0	2.71	62.3
	0.00	16.1	-	-		0.00	43.5	0.00	64.5
800S162-97	4.82	0.00	5.94	0.00	600S200-68	4.35	0.00	5.35	0.00
	4.66	15.1	5.74	18.8		4.20	7.87	5.17	11.3
	4.18	29.1	5.14	36.4		3.76	15.2	4.63	21.9
	3.41	41.2	4.20	51.4		3.07	21.5	3.78	30.9
	2.41	50.5	2.97	63.0		2.17	26.3	2.67	37.9
	1.25	56.3	1.54	70.2		1.13	29.4	1.38	42.2
	0.00	58.3	0.00	72.7		0.00	30.4	0.00	43.7
800S162-68	3.37	0.00	3.37	0.00	600S200-54	2.74	0.00	2.82	0.00
	3.25	8.88	3.25	12.9		2.65	6.23	2.73	7.87
	2.92	17.2	2.92	24.9		2.37	12.0	2.44	15.2
	2.38	24.3	2.38	35.2		1.94	17.0	2.00	21.5
	1.68	29.7	1.68	43.1		1.37	20.8	1.41	26.3
	0.871	33.2	0.871	48.1		0.709	23.3	0.731	29.4
	0.00	34.3	0.00	49.8		0.00	24.1	0.00	30.4

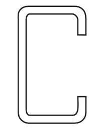
Table II - 8a (continued)

ASD - Combined Shear and Bending
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
600S200-43	1.42	0.00	-	-	600S162-33	0.638	0.00	-	-
	1.37	4.46	-	-		0.616	2.95	-	-
	1.23	8.62	-	-		0.553	5.70	-	-
	1.00	12.2	-	-		0.451	8.07	-	-
	0.708	14.9	-	-		0.319	9.88	-	-
	0.366	16.7	-	-		0.165	11.0	-	-
	0.00	17.2	-	-		0.00	11.4	-	-
600S200-33	0.638	0.00	-	-	550S162-68	2.06	0.00	2.53	0.00
	0.616	3.16	-	-		1.99	6.14	2.44	9.04
	0.553	6.11	-	-		1.78	11.9	2.19	17.5
	0.451	8.64	-	-		1.45	16.8	1.79	24.7
	0.319	10.6	-	-		1.03	20.5	1.27	30.3
	0.165	11.8	-	-		0.532	22.9	0.655	33.8
	0.00	12.2	-	-		0.00	23.7	0.00	34.9
600S162-97	2.51	0.00	3.80	0.00	550S162-54	1.67	0.00	1.88	0.00
	2.43	9.93	3.68	14.7		1.61	4.86	1.82	6.95
	2.17	19.2	3.30	28.4		1.44	9.38	1.63	13.4
	1.78	27.1	2.69	40.1		1.18	13.3	1.33	19.0
	1.26	33.2	1.90	49.1		0.833	16.2	0.940	23.3
	0.650	37.1	0.985	54.8		0.431	18.1	0.487	25.9
	0.00	38.4	0.00	56.7		0.00	18.8	0.00	26.9
600S162-68	2.34	0.00	2.88	0.00	550S162-43	1.20	0.00	-	-
	2.26	6.93	2.78	10.2		1.16	3.83	-	-
	2.03	13.4	2.49	19.7		1.04	7.40	-	-
	1.65	18.9	2.04	27.9		0.848	10.5	-	-
	1.17	23.2	1.44	34.2		0.599	12.8	-	-
	0.605	25.9	0.745	38.1		0.310	14.3	-	-
	0.00	26.8	0.00	39.5		0.00	14.8	-	-
600S162-54	1.89	0.00	1.95	0.00	550S162-33	0.698	0.00	-	-
	1.82	5.48	1.88	7.85		0.674	2.62	-	-
	1.64	10.6	1.69	15.2		0.605	5.06	-	-
	1.34	15.0	1.38	21.4		0.494	7.15	-	-
	0.945	18.3	0.973	26.3		0.349	8.76	-	-
	0.489	20.4	0.504	29.3		0.181	9.77	-	-
	0.00	21.2	0.00	30.3		0.00	10.1	-	-
600S162-43	1.24	0.00	-	-	400S162-68	0.895	0.00	1.36	0.00
	1.20	4.32	-	-		0.865	3.37	1.31	5.02
	1.07	8.34	-	-		0.775	6.50	1.17	9.70
	0.877	11.8	-	-		0.633	9.19	0.959	13.7
	0.620	14.4	-	-		0.447	11.3	0.678	16.8
	0.321	16.1	-	-		0.232	12.6	0.351	18.7
	0.00	16.7	-	-		0.00	13.0	0.00	19.4

Table II - 8a (continued)

ASD - Combined Shear and Bending
SSMA Studs
C-Sections With Lips



Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
400S162-54	0.944	0.00	1.22	0.00	362S162-33	0.521	0.00	-	-
	0.912	2.69	1.18	3.86		0.503	1.37	-	-
	0.818	5.19	1.06	7.45		0.451	2.65	-	-
	0.668	7.34	0.864	10.5		0.369	3.74	-	-
	0.472	9.00	0.611	12.9		0.261	4.58	-	-
	0.244	10.0	0.316	14.4		0.135	5.11	-	-
	0.00	10.4	0.00	14.9		0.00	5.29	-	-
400S162-43	0.809	0.00	-	-	350S162-68	0.592	0.00	0.897	0.00
	0.782	2.13	-	-		0.572	2.82	0.866	4.26
	0.701	4.12	-	-		0.513	5.45	0.777	8.22
	0.572	5.82	-	-		0.419	7.70	0.634	11.6
	0.405	7.13	-	-		0.296	9.43	0.448	14.2
	0.209	7.95	-	-		0.153	10.5	0.232	15.9
	0.00	8.23	-	-		0.00	10.9	0.00	16.4
400S162-33	0.595	0.00	-	-	350S162-54	0.633	0.00	0.947	0.00
	0.575	1.53	-	-		0.612	2.29	0.915	3.30
	0.515	2.95	-	-		0.548	4.41	0.820	6.37
	0.421	4.18	-	-		0.448	6.24	0.670	9.01
	0.298	5.12	-	-		0.317	7.65	0.473	11.0
	0.154	5.71	-	-		0.164	8.53	0.245	12.3
	0.00	5.91	-	-		0.00	8.83	0.00	12.7
362S162-68	0.663	0.00	1.00	0.00	350S162-43	0.631	0.00	-	-
	0.640	2.96	0.970	4.45		0.610	1.82	-	-
	0.574	5.72	0.869	8.59		0.547	3.52	-	-
	0.468	8.08	0.710	12.2		0.446	4.98	-	-
	0.331	9.90	0.502	14.9		0.316	6.10	-	-
	0.171	11.0	0.260	16.6		0.163	6.81	-	-
	0.00	11.4	0.00	17.2		0.00	7.05	-	-
362S162-54	0.706	0.00	1.02	0.00	350S162-33	0.487	0.00	-	-
	0.682	2.39	0.982	3.44		0.470	1.32	-	-
	0.611	4.61	0.880	6.64		0.422	2.54	-	-
	0.499	6.52	0.719	9.39		0.344	3.60	-	-
	0.353	7.98	0.508	11.5		0.243	4.40	-	-
	0.183	8.90	0.263	12.8		0.126	4.91	-	-
	0.00	9.22	0.00	13.3		0.00	5.08	-	-
362S162-43	0.676	0.00	-	-	250S162-68	0.343	0.00	0.519	0.00
	0.653	1.90	-	-		0.331	2.13	0.501	3.13
	0.585	3.67	-	-		0.297	4.11	0.449	6.05
	0.478	5.19	-	-		0.242	5.81	0.367	8.56
	0.338	6.36	-	-		0.171	7.11	0.259	10.5
	0.175	7.09	-	-		0.09	7.93	0.134	11.7
	0.00	7.34	-	-		0.00	8.21	0.00	12.1

Table II - 8a (continued)

ASD - Combined Shear and Bending
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
250S162-54	0.373	0.00	0.564	0.00	250S162-33	0.399	0.00	-	-
	0.360	1.70	0.545	2.44		0.385	0.920	-	-
	0.323	3.28	0.489	4.71		0.345	1.78	-	-
	0.263	4.64	0.399	6.66		0.282	2.51	-	-
	0.186	5.69	0.282	8.16		0.199	3.08	-	-
	0.10	6.34	0.146	9.10		0.103	3.43	-	-
	0.00	6.57	0.00	9.42		0.00	3.55	-	-
250S162-43	0.394	0.00	-	-					
	0.381	1.35	-	-					
	0.342	2.61	-	-					
	0.279	3.69	-	-					
	0.197	4.52	-	-					
	0.102	5.05	-	-					
	0.00	5.22	-	-					

Notes:

1. Shear and moment strengths have been divided by the appropriate factors of safety. 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 - 8b

LRFD - Combined Shear and Bending
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
1200S200-97	12.4	0.00	12.4	0.00	1000S250-43*	1.27	0.00	-	-
	12.0	39.8	12.0	57.3		1.23	13.1	-	-
	10.7	76.8	10.7	111		1.10	25.3	-	-
	8.75	109	8.75	157		0.898	35.8	-	-
	6.19	133	6.19	192		0.635	43.9	-	-
	3.20	148	3.20	214		0.329	49.0	-	-
	0.00	154	0.00	221		0.00	50.7	-	-
1200S200-68	4.21	0.00	4.21	0.00	1000S200-97	13.4	0.00	15.0	0.00
	4.07	26.1	4.07	36.4		13.0	31.4	14.5	46.0
	3.65	50.4	3.65	70.4		11.6	60.6	13.0	88.8
	2.98	71.3	2.98	99.5		9.50	85.7	10.6	126
	2.11	87.3	2.11	122		6.72	105	7.50	154
	1.09	97.4	1.09	136		3.48	117	3.88	172
	0.00	101	0.00	141		0.00	121	0.00	178
1200S200-54*	2.09	0.00	2.09	0.00	1000S200-68	5.08	0.00	5.08	0.00
	2.02	19.7	2.02	25.5		4.91	21.2	4.91	29.8
	1.81	38.0	1.81	49.2		4.40	40.9	4.40	57.5
	1.48	53.8	1.48	69.6		3.60	57.8	3.60	81.3
	1.05	65.9	1.05	85.3		2.54	70.8	2.54	99.6
	0.542	73.4	0.542	95.1		1.32	78.9	1.32	111
	0.00	76.0	0.00	98.5		0.00	81.7	0.00	115
1000S250-97	13.4	0.00	15.0	0.00	1000S200-54	2.52	0.00	2.52	0.00
	13.0	40.4	14.5	57.7		2.44	16.1	2.44	21.0
	11.6	78.1	13.0	112		2.19	31.1	2.19	40.5
	9.50	110	10.6	158		1.78	44.0	1.78	57.3
	6.72	135	7.50	193		1.26	53.9	1.26	70.1
	3.48	151	3.88	216		0.653	60.1	0.653	78.2
	0.00	156	0.00	223		0.00	62.2	0.00	81.0
1000S250-68	5.08	0.00	5.08	0.00	1000S200-43*	1.27	0.00	-	-
	4.91	27.1	4.91	34.0		1.23	11.9	-	-
	4.40	52.3	4.40	65.8		1.10	23.0	-	-
	3.60	74.0	3.60	93.0		0.898	32.6	-	-
	2.54	90.6	2.54	114		0.635	39.9	-	-
	1.32	101	1.32	127		0.329	44.5	-	-
	0.00	105	0.00	132		0.00	46.1	-	-
1000S250-54	2.52	0.00	2.52	0.00	800S200-97	13.4	0.00	16.5	0.00
	2.44	18.5	2.44	23.1		13.0	26.7	16.0	39.7
	2.19	35.7	2.19	44.6		11.6	51.7	14.3	76.7
	1.78	50.5	1.78	63.1		9.50	73.1	11.7	108
	1.26	61.8	1.26	77.3		6.72	89.5	8.27	133
	0.653	69.0	0.653	86.2		3.48	99.8	4.28	148
	0.00	71.4	0.00	89.3		0.00	103	0.00	153

Table II - 8b (continued)

LRFD - Combined Shear and Bending
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
800S200-68	6.41	0.00	6.41	0.00	800S162-54	3.18	0.00	3.18	0.00
	6.20	18.6	6.20	26.8		3.07	10.8	3.07	15.1
	5.56	35.9	5.56	51.7		2.75	20.9	2.75	29.2
	4.54	50.8	4.54	73.2		2.25	29.6	2.25	41.3
	3.21	62.2	3.21	89.6		1.59	36.2	1.59	50.5
	1.66	69.4	1.66	99.9		0.822	40.4	0.822	56.4
	0.00	71.9	0.00	103		0.00	41.8	0.00	58.4
800S200-54	3.18	0.00	3.18	0.00	800S162-43	1.60	0.00	-	-
	3.07	14.7	3.07	18.4		1.54	8.27	-	-
	2.75	28.4	2.75	35.6		1.38	16.0	-	-
	2.25	40.1	2.25	50.3		1.13	22.6	-	-
	1.59	49.1	1.59	61.7		0.798	27.7	-	-
	0.822	54.8	0.822	68.8		0.413	30.9	-	-
	0.00	56.7	0.00	71.2		0.00	32.0	-	-
800S200-43	1.60	0.00	-	-	800S162-33*	0.720	0.00	-	-
	1.54	10.5	-	-		0.696	5.76	-	-
	1.38	20.3	-	-		0.624	11.1	-	-
	1.13	28.7	-	-		0.509	15.7	-	-
	0.798	35.1	-	-		0.360	19.3	-	-
	0.413	39.1	-	-		0.186	21.5	-	-
	0.00	40.5	-	-		0.00	22.3	-	-
800S200-33*	0.720	0.00	-	-	600S200-97	10.5	0.00	15.9	0.00
	0.696	6.60	-	-		10.1	17.9	15.4	26.5
	0.624	12.7	-	-		9.10	34.5	13.8	51.2
	0.509	18.0	-	-		7.43	48.8	11.3	72.4
	0.360	22.1	-	-		5.25	59.8	7.96	88.7
	0.186	24.6	-	-		2.72	66.7	4.12	98.9
	0.00	25.5	-	-		0.00	69.0	0.00	102
800S162-97	7.33	0.00	9.02	0.00	600S200-68	6.61	0.00	8.13	0.00
	7.08	23.9	8.72	29.9		6.38	12.5	7.85	17.9
	6.35	46.2	7.82	57.7		5.72	24.1	7.04	34.7
	5.18	65.4	6.38	81.6		4.67	34.1	5.75	49.0
	3.67	80.1	4.51	99.9		3.30	41.8	4.07	60.1
	1.90	89.3	2.34	111		1.71	46.6	2.10	67.0
	0.00	92.5	0.00	115		0.00	48.3	0.00	69.3
800S162-68	5.12	0.00	5.12	0.00	600S200-54	4.16	0.00	4.29	0.00
	4.94	14.1	4.94	20.4		4.02	9.89	4.14	12.5
	4.43	27.2	4.43	39.5		3.61	19.1	3.72	24.1
	3.62	38.5	3.62	55.9		2.94	27.0	3.03	34.1
	2.56	47.2	2.56	68.4		2.08	33.1	2.15	41.8
	1.32	52.6	1.32	76.3		1.08	36.9	1.11	46.6
	0.00	54.5	0.00	79.0		0.00	38.2	0.00	48.2

Table II - 8b (continued)

LRFD - Combined Shear and Bending
SSMA Studs
C-Sections With Lips

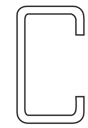
Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
600S200-43	2.15	0.00	-	-	600S162-33	0.970	0.00	-	-
	2.08	7.08	-	-		0.937	4.68	-	-
	1.86	13.7	-	-		0.840	9.05	-	-
	1.52	19.3	-	-		0.686	12.8	-	-
	1.08	23.7	-	-		0.485	15.7	-	-
	0.557	26.4	-	-		0.251	17.5	-	-
	0.00	27.4	-	-		0.00	18.1	-	-
600S200-33	0.970	0.00	-	-	550S162-68	3.13	0.00	3.85	0.00
	0.937	5.02	-	-		3.02	9.74	3.72	14.3
	0.840	9.69	-	-		2.71	18.8	3.33	27.7
	0.686	13.7	-	-		2.21	26.6	2.72	39.2
	0.485	16.8	-	-		1.56	32.6	1.92	48.0
	0.251	18.7	-	-		0.809	36.4	0.996	53.6
	0.00	19.4	-	-		0.00	37.6	0.00	55.4
600S162-97	3.82	0.00	5.78	0.00	550S162-54	2.53	0.00	2.86	0.00
	3.69	15.8	5.59	23.3		2.45	7.70	2.76	11.0
	3.31	30.4	5.01	45.0		2.19	14.9	2.48	21.3
	2.70	43.1	4.09	63.6		1.79	21.0	2.02	30.1
	1.91	52.7	2.89	77.9		1.27	25.8	1.43	36.9
	0.988	58.8	1.50	86.9		0.655	28.8	0.740	41.2
	0.00	60.9	0.00	90.0		0.00	29.8	0.00	42.6
600S162-68	3.55	0.00	4.38	0.00	550S162-43	1.82	0.00	-	-
	3.43	11.0	4.23	16.2		1.76	6.08	-	-
	3.08	21.2	3.79	31.3		1.58	11.7	-	-
	2.51	30.0	3.09	44.3		1.29	16.6	-	-
	1.78	36.8	2.19	54.2		0.911	20.3	-	-
	0.920	41.0	1.13	60.5		0.472	22.7	-	-
	0.00	42.5	0.00	62.6		0.00	23.5	-	-
600S162-54	2.87	0.00	2.96	0.00	550S162-33	1.06	0.00	-	-
	2.77	8.69	2.86	12.5		1.02	4.15	-	-
	2.49	16.8	2.56	24.1		0.919	8.02	-	-
	2.03	23.7	2.09	34.0		0.750	11.3	-	-
	1.44	29.1	1.48	41.7		0.531	13.9	-	-
	0.743	32.4	0.766	46.5		0.275	15.5	-	-
	0.00	33.6	0.00	48.1		0.00	16.0	-	-
600S162-43	1.88	0.00	-	-	400S162-68	1.36	0.00	2.06	0.00
	1.82	6.85	-	-		1.31	5.34	1.99	7.97
	1.63	13.2	-	-		1.18	10.3	1.78	15.4
	1.33	18.7	-	-		0.962	14.6	1.46	21.8
	0.942	22.9	-	-		0.680	17.9	1.03	26.7
	0.488	25.6	-	-		0.352	19.9	0.533	29.7
	0.00	26.5	-	-		0.00	20.6	0.00	30.8

Table II - 8b (continued)

LRFD - Combined Shear and Bending
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
400S162-54	1.44	0.00	1.86	0.00	362S162-33	0.792	0.00	-	-
	1.39	4.27	1.79	6.12		0.765	2.17	-	-
	1.24	8.24	1.61	11.8		0.686	4.20	-	-
	1.02	11.7	1.31	16.7		0.560	5.93	-	-
	0.718	14.3	0.929	20.5		0.396	7.27	-	-
	0.372	15.9	0.481	22.8		0.205	8.11	-	-
	0.00	16.5	0.00	23.6		0.00	8.39	-	-
400S162-43	1.23	0.00	-	-	350S162-68	0.900	0.00	1.36	0.00
	1.19	3.38	-	-		0.869	4.47	1.32	6.75
	1.07	6.53	-	-		0.779	8.64	1.18	13.0
	0.870	9.24	-	-		0.636	12.2	0.964	18.4
	0.615	11.3	-	-		0.450	15.0	0.682	22.6
	0.318	12.6	-	-		0.233	16.7	0.353	25.2
	0.00	13.1	-	-		0.00	17.3	0.00	26.1
400S162-33	0.904	0.00	-	-	350S162-54	0.962	0.00	1.44	0.00
	0.874	2.43	-	-		0.930	3.63	1.39	5.23
	0.783	4.69	-	-		0.833	7.00	1.25	10.1
	0.640	6.63	-	-		0.680	9.90	1.02	14.3
	0.452	8.12	-	-		0.481	12.1	0.720	17.5
	0.234	9.05	-	-		0.249	13.5	0.373	19.5
	0.00	9.37	-	-		0.00	14.0	0.00	20.2
362S162-68	1.01	0.00	1.53	0.00	350S162-43	0.960	0.00	-	-
	0.973	4.69	1.47	7.06		0.927	2.89	-	-
	0.872	9.07	1.32	13.6		0.831	5.59	-	-
	0.712	12.8	1.08	19.3		0.678	7.91	-	-
	0.503	15.7	0.763	23.6		0.480	9.68	-	-
	0.261	17.5	0.395	26.3		0.248	10.8	-	-
	0.00	18.1	0.00	27.3		0.00	11.2	-	-
362S162-54	1.07	0.00	1.54	0.00	350S162-33	0.740	0.00	-	-
	1.04	3.79	1.49	5.45		0.715	2.09	-	-
	0.929	7.31	1.34	10.5		0.641	4.03	-	-
	0.758	10.3	1.09	14.9		0.523	5.70	-	-
	0.536	12.7	0.772	18.2		0.370	6.99	-	-
	0.278	14.1	0.400	20.4		0.192	7.79	-	-
	0.00	14.6	0.00	21.1		0.00	8.07	-	-
362S162-43	1.03	0.00	-	-	250S162-68	0.521	0.00	0.789	0.00
	0.992	3.02	-	-		0.503	3.37	0.762	4.97
	0.889	5.83	-	-		0.451	6.51	0.683	9.60
	0.726	8.24	-	-		0.368	9.21	0.558	13.6
	0.513	10.1	-	-		0.260	11.3	0.394	16.6
	0.266	11.3	-	-		0.135	12.6	0.204	18.6
	0.00	11.7	-	-		0.00	13.0	0.00	19.2

Table II - 8b (continued)



LRFD - Combined Shear and Bending
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
250S162-54	0.566	0.00	0.858	0.00	250S162-33	0.606	0.00	-	-
	0.547	2.70	0.829	3.87		0.585	1.46	-	-
	0.490	5.21	0.743	7.47		0.525	2.82	-	-
	0.400	7.37	0.607	10.6		0.429	3.99	-	-
	0.283	9.02	0.429	12.9		0.303	4.88	-	-
	0.147	10.1	0.222	14.4		0.157	5.45	-	-
	0.00	10.4	0.00	14.9		0.00	5.64	-	-
250S162-43	0.599	0.00	-	-					
	0.579	2.15	-	-					
	0.519	4.14	-	-					
	0.424	5.86	-	-					
	0.300	7.18	-	-					
	0.155	8.01	-	-					
	0.00	8.29	-	-					

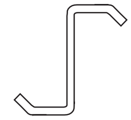
Notes:

1. Shear and moment strengths have been multiplied by the appropriate resistance factors. This table is for LRFD 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 - 9a

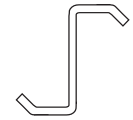
ASD - Combined Shear and Bending
Z-Sections With Lips



Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
12ZS3.25x105	9.02	0.00	9.02	0.00	10ZS2.75x105	9.43	0.00	10.9	0.00
	8.72	37.3	8.72	56.4		9.11	26.5	10.6	43.6
	7.82	72.0	7.82	109		8.16	51.1	9.48	84.3
	6.38	102	6.38	154		6.67	72.3	7.74	119
	4.51	125	4.51	189		4.71	88.5	5.47	146
	2.34	139	2.34	210		2.44	98.7	2.83	163
	0.00	144	0.00	218		0.00	102	0.00	169
12ZS3.25x085	4.77	0.00	4.77	0.00	10ZS2.75x085	5.78	0.00	5.78	0.00
	4.61	29.4	4.61	41.2		5.58	21.5	5.58	33.0
	4.13	56.8	4.13	79.7		5.01	41.6	5.01	63.7
	3.37	80.3	3.37	113		4.09	58.8	4.09	90.1
	2.39	98.4	2.39	138		2.89	72.0	2.89	110
	1.23	110	1.23	154		1.50	80.3	1.50	123
	0.00	114	0.00	159		0.00	83.1	0.00	127
12ZS3.25x070	2.66	0.00	2.66	0.00	10ZS2.75x070	3.22	0.00	3.22	0.00
	2.57	23.0	2.57	30.1		3.11	17.8	3.11	24.9
	2.30	44.4	2.30	58.2		2.79	34.3	2.79	48.2
	1.88	62.8	1.88	82.3		2.28	48.5	2.28	68.1
	1.33	76.9	1.33	101		1.61	59.5	1.61	83.4
	0.688	85.8	0.688	112		0.833	66.3	0.833	93.1
	0.00	88.8	0.00	116		0.00	68.7	0.00	96.3
12ZS2.25x105	9.02	0.00	9.02	0.00	10ZS2.75x065	2.57	0.00	2.57	0.00
	8.72	30.3	8.72	47.9		2.49	16.2	2.49	22.2
	7.82	58.5	7.82	92.6		2.23	31.3	2.23	42.9
	6.38	82.7	6.38	131		1.82	44.3	1.82	60.7
	4.51	101	4.51	160		1.29	54.3	1.29	74.3
	2.34	113	2.34	179		0.666	60.5	0.666	82.9
	0.00	117	0.00	185		0.00	62.7	0.00	85.8
12ZS2.25x085	4.77	0.00	4.77	0.00	10ZS2.75x059	1.92	0.00	1.92	0.00
	4.61	23.5	4.61	37.1		1.86	14.4	1.86	19.2
	4.13	45.5	4.13	71.6		1.66	27.9	1.66	37.0
	3.37	64.3	3.37	101		1.36	39.4	1.36	52.4
	2.39	78.8	2.39	124		0.961	48.2	0.961	64.1
	1.23	87.8	1.23	138		0.498	53.8	0.498	71.5
	0.00	90.9	0.00	143		0.00	55.7	0.00	74.1
12ZS2.25x070	2.66	0.00	2.66	0.00	10ZS2.25x105	9.43	0.00	10.9	0.00
	2.57	18.6	2.57	28.4		9.11	23.8	10.6	38.6
	2.30	35.9	2.30	54.8		8.16	46.0	9.48	74.5
	1.88	50.8	1.88	77.5		6.67	65.1	7.74	105
	1.33	62.2	1.33	95.0		4.71	79.7	5.47	129
	0.688	69.4	0.688	106		2.44	88.9	2.83	144
	0.00	71.8	0.00	110		0.00	92.0	0.00	149

Table II - 9a (continued)

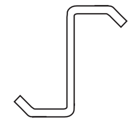
ASD - Combined Shear and Bending
Z-Sections With Lips



Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
10ZS2.25x085	5.78	0.00	5.78	0.00	8ZS2.75x070	4.08	0.00	4.08	0.00
	5.58	18.9	5.58	30.1		3.94	13.1	3.94	19.1
	5.01	36.6	5.01	58.2		3.53	25.4	3.53	36.9
	4.09	51.7	4.09	82.3		2.88	35.9	2.88	52.2
	2.89	63.4	2.89	101		2.04	44.0	2.04	64.0
	1.50	70.7	1.50	112		1.06	49.1	1.06	71.4
	0.00	73.2	0.00	116		0.00	50.8	0.00	73.9
10ZS2.25x070	3.22	0.00	3.22	0.00	8ZS2.75x065	3.26	0.00	3.26	0.00
	3.11	15.1	3.11	23.2		3.15	12.0	3.15	17.6
	2.79	29.1	2.79	44.8		2.82	23.1	2.82	34.1
	2.28	41.2	2.28	63.4		2.31	32.7	2.31	48.2
	1.61	50.5	1.61	77.6		1.63	40.1	1.63	59.0
	0.833	56.3	0.833	86.6		0.844	44.7	0.844	65.8
	0.00	58.3	0.00	89.7		0.00	46.3	0.00	68.1
10ZS2.25x065	2.57	0.00	2.57	0.00	8ZS2.75x059	2.43	0.00	2.43	0.00
	2.49	13.8	2.49	20.2		2.35	10.6	2.35	15.5
	2.23	26.7	2.23	39.1		2.11	20.6	2.11	29.9
	1.82	37.8	1.82	55.2		1.72	29.1	1.72	42.2
	1.29	46.2	1.29	67.6		1.22	35.6	1.22	51.7
	0.666	51.6	0.666	75.5		0.630	39.7	0.630	57.7
	0.00	53.4	0.00	78.1		0.00	41.1	0.00	59.7
10ZS2.25x059	1.92	0.00	1.92	0.00	8ZS2.25x105	9.43	0.00	12.2	0.00
	1.86	12.3	1.86	17.6		9.11	17.4	11.8	29.0
	1.66	23.7	1.66	34.1		8.16	33.7	10.5	56.1
	1.36	33.5	1.36	48.2		6.67	47.6	8.61	79.3
	0.961	41.0	0.961	59.1		4.71	58.3	6.08	97.2
	0.498	45.8	0.498	65.9		2.44	65.0	3.15	108
	0.00	47.4	0.00	68.2		0.00	67.3	0.00	112
8ZS2.75x105	9.43	0.00	12.2	0.00	8ZS2.25x085	6.18	0.00	7.33	0.00
	9.11	19.5	11.8	32.2		5.97	14.2	7.08	23.7
	8.16	37.7	10.5	62.2		5.35	27.4	6.35	45.7
	6.67	53.3	8.61	87.9		4.37	38.8	5.18	64.6
	4.71	65.3	6.08	108		3.09	47.5	3.67	79.1
	2.44	72.8	3.15	120		1.60	53.0	1.90	88.3
	0.00	75.4	0.00	124		0.00	54.8	0.00	91.4
8ZS2.75x085	6.18	0.00	7.33	0.00	8ZS2.25x070	4.08	0.00	4.08	0.00
	5.97	15.9	7.08	24.2		3.94	11.7	3.94	19.1
	5.35	30.7	6.35	46.8		3.53	22.7	3.53	36.8
	4.37	43.4	5.18	66.2		2.88	32.1	2.88	52.1
	3.09	53.2	3.67	81.1		2.04	39.3	2.04	63.8
	1.60	59.3	1.90	90.4		1.06	43.8	1.06	71.2
	0.00	61.4	0.00	93.6		0.00	45.4	0.00	73.7

Table II - 9a (continued)

ASD - Combined Shear and Bending
Z-Sections With Lips



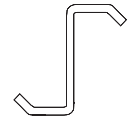
Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
8ZS2.25x065	3.26	0.00	3.26	0.00	6ZS2.25x065	3.61	0.00	4.45	0.00
	3.15	10.9	3.15	17.0		3.49	7.40	4.30	11.5
	2.82	21.1	2.82	32.9		3.13	14.3	3.85	22.2
	2.31	29.8	2.31	46.5		2.55	20.2	3.14	31.4
	1.63	36.5	1.63	57.0		1.81	24.8	2.22	38.4
	0.844	40.7	0.844	63.6		0.935	27.6	1.15	42.9
	0.00	42.2	0.00	65.8		0.00	28.6	0.00	44.4
8ZS2.25x059	2.43	0.00	2.43	0.00	6ZS2.25x059	2.98	0.00	3.32	0.00
	2.35	9.89	2.35	15.2		2.87	6.71	3.21	10.3
	2.11	19.1	2.11	29.4		2.58	13.0	2.87	19.8
	1.72	27.0	1.72	41.6		2.10	18.3	2.35	28.0
	1.22	33.1	1.22	51.0		1.49	22.5	1.66	34.4
	0.630	36.9	0.630	56.9		0.770	25.1	0.859	38.3
	0.00	38.2	0.00	58.9		0.00	25.9	0.00	39.7
6ZS2.25x105	7.04	0.00	11.7	0.00	4ZS2.25x070	3.02	0.00	5.03	0.00
	6.80	11.8	11.3	19.6		2.92	4.65	4.86	7.54
	6.09	22.7	10.2	37.8		2.61	8.99	4.36	14.6
	4.98	32.1	8.29	53.5		2.13	12.7	3.56	20.6
	3.52	39.3	5.86	65.5		1.51	15.6	2.52	25.2
	1.82	43.9	3.04	73.1		0.781	17.4	1.30	28.1
	0.00	45.4	0.00	75.7		0.00	18.0	0.00	29.1
6ZS2.25x085	5.74	0.00	7.97	0.00	4ZS2.25x065	2.81	0.00	4.66	0.00
	5.54	9.59	7.70	16.0		2.72	4.34	4.50	6.71
	4.97	18.5	6.91	30.9		2.43	8.38	4.04	13.0
	4.06	26.2	5.64	43.7		1.99	11.9	3.30	18.3
	2.87	32.1	3.99	53.5		1.41	14.5	2.33	22.4
	1.49	35.8	2.06	59.7		0.728	16.2	1.21	25.0
	0.00	37.1	0.00	61.8		0.00	16.8	0.00	25.9
6ZS2.25x070	4.19	0.00	5.41	0.00	4ZS2.25x059	2.56	0.00	3.84	0.00
	4.05	7.95	5.22	12.9		2.47	3.94	3.71	5.98
	3.63	15.4	4.68	24.9		2.22	7.62	3.33	11.6
	2.96	21.7	3.82	35.2		1.81	10.8	2.72	16.3
	2.09	26.6	2.70	43.1		1.28	13.2	1.92	20.0
	1.08	29.7	1.40	48.1		0.663	14.7	0.995	22.3
	0.00	30.7	0.00	49.8		0.00	15.2	0.00	23.1

Notes:

1. Shear and moment strengths have been divided by the appropriate factors of safety. 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 - 9b

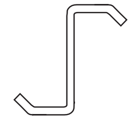
LRFD - Combined Shear and Bending
Z-Sections With Lips



Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
12ZS3.25x105	13.7	0.00	13.7	0.00	10ZS2.75x105	14.3	0.00	16.6	0.00
	13.2	59.1	13.2	89.4		13.8	42.0	16.1	69.2
	11.9	114	11.9	173		12.4	81.1	14.4	134
	9.70	162	9.70	244		10.1	115	11.8	189
	6.86	198	6.86	299		7.16	140	8.32	232
	3.55	221	3.55	334		3.71	157	4.30	258
	0.00	228	0.00	345		0.00	162	0.00	268
12ZS3.25x085	7.25	0.00	7.25	0.00	10ZS2.75x085	8.79	0.00	8.79	0.00
	7.00	46.6	7.00	65.4		8.49	34.1	8.49	52.3
	6.28	90.1	6.28	126		7.61	65.9	7.61	101
	5.13	127	5.13	179		6.21	93.2	6.21	143
	3.63	156	3.63	219		4.39	114	4.39	175
	1.88	174	1.88	244		2.27	127	2.27	195
	0.00	180	0.00	253		0.00	132	0.00	202
12ZS3.25x070	4.04	0.00	4.04	0.00	10ZS2.75x070	4.89	0.00	4.89	0.00
	3.90	36.5	3.90	47.8		4.72	28.2	4.72	39.6
	3.50	70.5	3.50	92.3		4.24	54.5	4.24	76.4
	2.86	99.6	2.86	131		3.46	77.0	3.46	108
	2.02	122	2.02	160		2.45	94.3	2.45	132
	1.05	136	1.05	178		1.27	105	1.27	148
	0.00	141	0.00	185		0.00	109	0.00	153
12ZS2.25x105	13.7	0.00	13.7	0.00	10ZS2.75x065	3.91	0.00	3.91	0.00
	13.2	48.0	13.2	76.0		3.78	25.7	3.78	35.2
	11.9	92.8	11.9	147		3.39	49.7	3.39	68.1
	9.70	131	9.70	208		2.77	70.3	2.77	96.3
	6.86	161	6.86	254		1.96	86.1	1.96	118
	3.55	179	3.55	284		1.01	96.0	1.01	132
	0.00	186	0.00	294		0.00	99.4	0.00	136
12ZS2.25x085	7.25	0.00	7.25	0.00	10ZS2.75x059	2.92	0.00	2.92	0.00
	7.00	37.3	7.00	58.8		2.82	22.9	2.82	30.4
	6.28	72.1	6.28	114		2.53	44.2	2.53	58.8
	5.13	102	5.13	161		2.07	62.5	2.07	83.1
	3.63	125	3.63	197		1.46	76.5	1.46	102
	1.88	139	1.88	220		0.756	85.4	0.756	114
	0.00	144	0.00	227		0.00	88.4	0.00	118
12ZS2.25x070	4.04	0.00	4.04	0.00	10ZS2.25x105	14.3	0.00	16.6	0.00
	3.90	29.5	3.90	45.0		13.8	37.8	16.1	61.2
	3.50	57.0	3.50	87.0		12.4	73.0	14.4	118
	2.86	80.6	2.86	123		10.1	103	11.8	167
	2.02	98.7	2.02	151		7.16	126	8.32	205
	1.05	110	1.05	168		3.71	141	4.30	228
	0.00	114	0.00	174		0.00	146	0.00	236

Table II - 9b (continued)

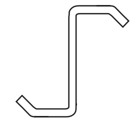
LRFD - Combined Shear and Bending
Z-Sections With Lips



Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
10ZS2.25x085	8.79	0.00	8.79	0.00	8ZS2.75x070	6.20	0.00	6.20	0.00
	8.49	30.0	8.49	47.8		5.99	20.9	5.99	30.3
	7.61	58.0	7.61	92.3		5.37	40.3	5.37	58.6
	6.21	82.1	6.21	131		4.38	57.0	4.38	82.9
	4.39	101	4.39	160		3.10	69.8	3.10	102
	2.27	112	2.27	178		1.60	77.8	1.60	113
	0.00	116	0.00	185		0.00	80.6	0.00	117
10ZS2.25x070	4.89	0.00	4.89	0.00	8ZS2.75x065	4.96	0.00	4.96	0.00
	4.72	23.9	4.72	36.8		4.79	19.0	4.79	28.0
	4.24	46.2	4.24	71.1		4.29	36.7	4.29	54.0
	3.46	65.4	3.46	101		3.50	51.9	3.50	76.4
	2.45	80.1	2.45	123		2.48	63.6	2.48	93.6
	1.27	89.3	1.27	137		1.28	70.9	1.28	104
	0.00	92.5	0.00	142		0.00	73.4	0.00	108
10ZS2.25x065	3.91	0.00	3.91	0.00	8ZS2.75x059	3.70	0.00	3.70	0.00
	3.78	21.9	3.78	32.1		3.57	16.9	3.57	24.5
	3.39	42.4	3.39	62.0		3.20	32.6	3.20	47.4
	2.77	59.9	2.77	87.6		2.62	46.1	2.62	67.0
	1.96	73.4	1.96	107		1.85	56.5	1.85	82.1
	1.01	81.8	1.01	120		0.958	63.0	0.958	91.5
	0.00	84.7	0.00	124		0.00	65.2	0.00	94.8
10ZS2.25x059	2.92	0.00	2.92	0.00	8ZS2.25x105	14.3	0.00	18.5	0.00
	2.82	19.5	2.82	28.0		13.8	27.6	17.9	46.1
	2.53	37.6	2.53	54.1		12.4	53.4	16.0	89.0
	2.07	53.1	2.07	76.5		10.1	75.5	13.1	126
	1.46	65.1	1.46	93.7		7.16	92.5	9.25	154
	0.756	72.6	0.756	104		3.71	103	4.79	172
	0.00	75.2	0.00	108		0.00	107	0.00	178
8ZS2.75x105	14.3	0.00	18.5	0.00	8ZS2.25x085	9.39	0.00	11.1	0.00
	13.8	31.0	17.9	51.1		9.07	22.5	10.8	37.5
	12.4	59.8	16.0	98.7		8.13	43.5	9.65	72.5
	10.1	84.6	13.1	140		6.64	61.5	7.88	103
	7.16	104	9.25	171		4.69	75.3	5.57	126
	3.71	116	4.79	191		2.43	84.0	2.88	140
	0.00	120	0.00	197		0.00	87.0	0.00	145
8ZS2.75x085	9.39	0.00	11.1	0.00	8ZS2.25x070	6.20	0.00	6.20	0.00
	9.07	25.2	10.8	38.4		5.99	18.6	5.99	30.2
	8.13	48.7	9.65	74.3		5.37	36.0	5.37	58.4
	6.64	68.9	7.88	105		4.38	50.9	4.38	82.6
	4.69	84.4	5.57	129		3.10	62.3	3.10	101
	2.43	94.1	2.88	143		1.60	69.5	1.60	113
	0.00	97.4	0.00	149		0.00	72.0	0.00	117

Table II - 9b (continued)

LRFD - Combined Shear and Bending
Z-Sections With Lips



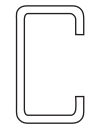
Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
8ZS2.25x065	4.96	0.00	4.96	0.00	6ZS2.25x065	5.49	0.00	6.76	0.00
	4.79	17.3	4.79	27.0		5.30	11.7	6.53	18.2
	4.29	33.5	4.29	52.2		4.76	22.7	5.85	35.2
	3.50	47.3	3.50	73.8		3.88	32.1	4.78	49.8
	2.48	58.0	2.48	90.4		2.75	39.3	3.38	61.0
	1.28	64.6	1.28	101		1.42	43.8	1.75	68.0
	0.00	66.9	0.00	104		0.00	45.4	0.00	70.4
8ZS2.25x059	3.70	0.00	3.70	0.00	6ZS2.25x059	4.52	0.00	5.04	0.00
	3.57	15.7	3.57	24.2		4.37	10.6	4.87	16.3
	3.20	30.3	3.20	46.7		3.92	20.6	4.37	31.5
	2.62	42.9	2.62	66.1		3.20	29.1	3.57	44.5
	1.85	52.5	1.85	80.9		2.26	35.6	2.52	54.5
	0.958	58.6	0.958	90.2		1.17	39.7	1.31	60.8
	0.00	60.6	0.00	93.4		0.00	41.1	0.00	62.9
6ZS2.25x105	10.7	0.00	17.8	0.00	4ZS2.25x070	4.59	0.00	7.65	0.00
	10.3	18.6	17.2	31.1		4.43	7.38	7.39	12.0
	9.26	36.0	15.4	60.0		3.97	14.3	6.62	23.1
	7.56	50.9	12.6	84.9		3.24	20.2	5.41	32.7
	5.35	62.4	8.91	104		2.29	24.7	3.82	40.0
	2.77	69.6	4.61	116		1.19	27.6	1.98	44.6
	0.00	72.0	0.00	120		0.00	28.5	0.00	46.2
6ZS2.25x085	8.72	0.00	12.1	0.00	4ZS2.25x065	4.27	0.00	7.09	0.00
	8.42	15.2	11.7	25.4		4.13	6.88	6.85	10.6
	7.55	29.4	10.5	49.0		3.70	13.3	6.14	20.6
	6.17	41.6	8.57	69.3		3.02	18.8	5.01	29.1
	4.36	50.9	6.06	84.9		2.14	23.0	3.54	35.6
	2.26	56.8	3.14	94.7		1.11	25.7	1.83	39.7
	0.00	58.8	0.00	98.0		0.00	26.6	0.00	41.1
6ZS2.25x070	6.37	0.00	8.22	0.00	4ZS2.25x059	3.89	0.00	5.84	0.00
	6.15	12.6	7.94	20.5		3.76	6.26	5.64	9.49
	5.51	24.4	7.12	39.5		3.37	12.1	5.06	18.3
	4.50	34.5	5.81	55.9		2.75	17.1	4.13	25.9
	3.18	42.2	4.11	68.5		1.95	20.9	2.92	31.8
	1.65	47.1	2.13	76.3		1.01	23.3	1.51	35.4
	0.00	48.7	0.00	79.0		0.00	24.2	0.00	36.7

Notes:

1. Shear and moment strengths have been multiplied by the appropriate resistance factors. This table is for LRFD use only.
2. Shear strengths have been calculated assuming no transverse stiffeners.
3. Linear interpolation between values is permitted.

Table II - 10

Web Crippling, P_n, kips [1,2]
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [4]				Bearing Length, N (in.) [4]					
			1	2	4	6	1	2	4	6		
12CS3.5x105	Fastened	A	1.95	2.37	2.96	3.41	3.25	3.94	4.93	5.69	1.75	0.85
		C	1.67	1.85	2.12	2.32	2.78	3.09	3.53	3.87	1.75	0.85
		D	5.32	5.76	6.37	6.85	8.87	9.59	10.6	11.4	1.75	0.85
		B	4.20	4.73	5.47	6.04	7.00	7.88	9.12	10.1	1.65	0.90
	Unfastened	A	1.95	2.37	2.96	3.41	3.25	3.94	4.93	5.69	1.85	0.80
		B	4.20	4.73	5.47	6.04	7.00	7.88	9.12	10.1	1.65	0.90
		C	1.82	1.92	2.07	2.17	3.04	3.20	3.44	3.62	1.65	0.90
		D	3.86	4.36	5.08	5.63	6.43	7.27	8.46	9.38	1.90	0.80
12CS3.5x085	Fastened	A	1.28	1.56	1.97	2.29	2.13	2.61	3.29	3.81	1.75	0.85
		B	2.67	3.03	3.54	3.93	4.45	5.05	5.89	6.54	1.65	0.90
		C	0.985	1.10	1.27	1.40	1.64	1.84	2.12	2.34	1.75	0.85
		D	3.31	3.61	4.03	4.35	5.52	6.01	6.71	7.24	1.75	0.85
	Unfastened	A	1.28	1.56	1.97	2.29	2.13	2.61	3.29	3.81	1.85	0.80
		B	2.67	3.03	3.54	3.93	4.45	5.05	5.89	6.54	1.65	0.90
		C	1.02	1.08	1.17	1.24	1.70	1.80	1.95	2.06	1.65	0.90
		D	1.95	2.22	2.61	2.91	3.25	3.71	4.35	4.85	1.90	0.80
12CS3.5x070	Fastened	A	0.861	1.06	1.35	1.57	1.44	1.77	2.25	2.62	1.75	0.85
		B	1.75	2.00	2.35	2.62	2.91	3.33	3.92	4.37	1.65	0.90
		C	0.590	0.666	0.774	0.857	0.983	1.11	1.29	1.43	1.75	0.85
		D	2.12	2.33	2.62	2.84	3.54	3.88	4.36	4.73	1.75	0.85
	Unfastened	A	0.861	1.06	1.35	1.57	1.44	1.77	2.25	2.62	1.85	0.80
		B	1.75	2.00	2.35	2.62	2.91	3.33	3.92	4.37	1.65	0.90
		C	0.580	0.619	0.673	0.714	0.967	1.03	1.12	1.19	1.65	0.90
		D	0.894	1.03	1.22	1.36	1.49	1.71	2.03	2.27	1.90	0.80
10CS3.5x105	Fastened	A	1.99	2.42	3.03	3.50	3.32	4.04	5.05	5.83	1.75	0.85
		B	4.25	4.78	5.53	6.10	7.08	7.96	9.21	10.2	1.65	0.90
		C	1.82	2.03	2.31	2.54	3.04	3.38	3.86	4.23	1.75	0.85
		D	5.55	6.01	6.65	7.15	9.26	10.0	11.1	11.9	1.75	0.85
	Unfastened	A	1.99	2.42	3.03	3.50	3.32	4.04	5.05	5.83	1.85	0.80
		B	4.25	4.78	5.53	6.10	7.08	7.96	9.21	10.2	1.65	0.90
		C	1.94	2.05	2.20	2.32	3.24	3.41	3.67	3.86	1.65	0.90
		D	3.86	4.37	5.08	5.63	6.43	7.28	8.47	9.39	1.90	0.80
10CS3.5x085	Fastened	A	1.31	1.61	2.03	2.35	2.19	2.68	3.38	3.91	1.75	0.85
		B	2.70	3.07	3.58	3.97	4.50	5.11	5.96	6.62	1.65	0.90
		C	1.10	1.23	1.42	1.56	1.83	2.05	2.36	2.60	1.75	0.85
		D	3.48	3.79	4.23	4.57	5.80	6.32	7.05	7.62	1.75	0.85
	Unfastened	A	1.31	1.61	2.03	2.35	2.19	2.68	3.38	3.91	1.85	0.80
		B	2.70	3.07	3.58	3.97	4.50	5.11	5.96	6.62	1.65	0.90
		C	1.10	1.17	1.26	1.34	1.84	1.95	2.10	2.23	1.65	0.90
		D	1.95	2.23	2.62	2.91	3.25	3.71	4.36	4.86	1.90	0.80
10CS3.5x070	Fastened	A	0.889	1.10	1.39	1.62	1.48	1.83	2.32	2.70	1.75	0.85
		B	1.77	2.02	2.38	2.66	2.95	3.37	3.97	4.43	1.65	0.90
		C	0.676	0.763	0.887	0.982	1.13	1.27	1.48	1.64	1.75	0.85
		D	2.25	2.47	2.77	3.01	3.75	4.11	4.62	5.02	1.75	0.85
	Unfastened	A	0.889	1.10	1.39	1.62	1.48	1.83	2.32	2.70	1.85	0.80
		B	1.77	2.02	2.38	2.66	2.95	3.37	3.97	4.43	1.65	0.90
		C	0.636	0.678	0.737	0.783	1.06	1.13	1.23	1.30	1.65	0.90
		D	0.895	1.03	1.22	1.36	1.49	1.72	2.03	2.27	1.90	0.80

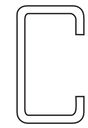
Table II - 10 (continued)

Web Crippling, P_n, kips [1,2]
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi Bearing Length, N (in.)[4] 1	2	4	6	F_y = 55 ksi Bearing Length, N (in.)[4] 1	2	4	6	Ω_w	ϕ_w
10CS3.5x065	Fastened	A	0.765	0.948	1.21	1.41	1.27	1.58	2.01	2.34	1.75	0.85
		B	1.50	1.73	2.04	2.28	2.51	2.88	3.40	3.80	1.65	0.90
		C	0.558	0.632	0.737	0.817	0.930	1.05	1.23	1.36	1.75	0.85
		D	1.90	2.09	2.36	2.56	3.17	3.48	3.93	4.27	1.75	0.85
	Unfastened	A	0.765	0.948	1.21	1.41	1.27	1.58	2.01	2.34	1.85	0.80
		B	1.50	1.73	2.04	2.28	2.51	2.88	3.40	3.80	1.65	0.90
		C	0.511	0.546	0.595	0.633	0.852	0.910	0.992	1.05	1.65	0.90
		D	0.613	0.708	0.841	0.943	1.02	1.18	1.40	1.57	1.90	0.80
8CS3.5x105	Fastened	A	2.05	2.49	3.11	3.59	3.41	4.15	5.18	5.98	1.75	0.85
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	1.99	2.22	2.53	2.77	3.32	3.69	4.22	4.62	1.75	0.85
		D	5.81	6.29	6.96	7.48	9.69	10.5	11.6	12.5	1.75	0.85
	Unfastened	A	2.05	2.49	3.11	3.59	3.41	4.15	5.18	5.98	1.85	0.80
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	2.07	2.19	2.35	2.48	3.46	3.65	3.92	4.13	1.65	0.90
		D	3.86	4.37	5.09	5.64	6.44	7.29	8.48	9.40	1.90	0.80
8CS3.5x085	Fastened	A	1.35	1.66	2.09	2.42	2.25	2.76	3.48	4.03	1.75	0.85
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.22	1.37	1.58	1.74	2.04	2.29	2.64	2.90	1.75	0.85
		D	3.67	4.00	4.46	4.82	6.12	6.67	7.44	8.03	1.75	0.85
	Unfastened	A	1.35	1.66	2.09	2.42	2.25	2.76	3.48	4.03	1.85	0.80
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.19	1.26	1.37	1.44	1.99	2.11	2.28	2.41	1.65	0.90
		D	1.95	2.23	2.62	2.92	3.26	3.72	4.36	4.86	1.90	0.80
8CS3.5x070	Fastened	A	0.919	1.14	1.44	1.68	1.53	1.89	2.40	2.79	1.75	0.85
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.772	0.871	1.01	1.12	1.29	1.45	1.69	1.87	1.75	0.85
		D	2.39	2.62	2.95	3.20	3.99	4.37	4.92	5.33	1.75	0.85
	Unfastened	A	0.919	1.14	1.44	1.68	1.53	1.89	2.40	2.79	1.85	0.80
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.698	0.744	0.809	0.859	1.16	1.24	1.35	1.43	1.65	0.90
		D	0.896	1.03	1.22	1.37	1.49	1.72	2.03	2.28	1.90	0.80
8CS3.5x065	Fastened	A	0.792	0.982	1.25	1.46	1.32	1.64	2.08	2.43	1.75	0.85
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.644	0.729	0.850	0.943	1.07	1.22	1.42	1.57	1.75	0.85
		D	2.03	2.23	2.51	2.73	3.38	3.72	4.19	4.55	1.75	0.85
	Unfastened	A	0.792	0.982	1.25	1.46	1.32	1.64	2.08	2.43	1.85	0.80
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.565	0.603	0.657	0.699	0.941	1.00	1.10	1.16	1.65	0.90
		D	0.614	0.708	0.842	0.944	1.02	1.18	1.40	1.57	1.90	0.80
8CS3.5x059	Fastened	A	0.652	0.811	1.04	1.21	1.09	1.35	1.73	2.02	1.75	0.85
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.506	0.575	0.673	0.749	0.843	0.959	1.12	1.25	1.75	0.85
		D	1.63	1.80	2.04	2.22	2.72	3.00	3.39	3.70	1.75	0.85
	Unfastened	A	0.652	0.811	1.04	1.21	1.09	1.35	1.73	2.02	1.85	0.80
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.424	0.455	0.497	0.530	0.707	0.758	0.828	0.883	1.65	0.90
		D	0.322	0.373	0.445	0.500	0.536	0.621	0.741	0.833	1.90	0.80

Table II - 10 (continued)

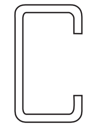
Web Crippling, P_n, kips [1,2]
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi Bearing Length, N (in.) [4] 1	2	4	6	F_y = 55 ksi Bearing Length, N (in.) [4] 1	2	4	6	Ω_w	ϕ_w
8CS2.5x105	Fastened	A	2.05	2.49	3.11	3.59	3.41	4.15	5.18	5.98	1.75	0.85
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	1.99	2.22	2.53	2.77	3.32	3.69	4.22	4.62	1.75	0.85
		D	5.81	6.29	6.96	7.48	9.69	10.5	11.6	12.5	1.75	0.85
	Unfastened	A	2.05	2.49	3.11	3.59	3.41	4.15	5.18	5.98	1.85	0.80
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	2.07	2.19	2.35	2.48	3.46	3.65	3.92	4.13	1.65	0.90
		D	3.86	4.37	5.09	5.64	6.44	7.29	8.48	9.40	1.90	0.80
8CS2.5x085	Fastened	A	1.35	1.66	2.09	2.42	2.25	2.76	3.48	4.03	1.75	0.85
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.22	1.37	1.58	1.74	2.04	2.29	2.64	2.90	1.75	0.85
		D	3.67	4.00	4.46	4.82	6.12	6.67	7.44	8.03	1.75	0.85
	Unfastened	A	1.35	1.66	2.09	2.42	2.25	2.76	3.48	4.03	1.85	0.80
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.19	1.26	1.37	1.44	1.99	2.11	2.28	2.41	1.65	0.90
		D	1.95	2.23	2.62	2.92	3.26	3.72	4.36	4.86	1.90	0.80
8CS2.5x070	Fastened	A	0.919	1.14	1.44	1.68	1.53	1.89	2.40	2.79	1.75	0.85
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.772	0.871	1.01	1.12	1.29	1.45	1.69	1.87	1.75	0.85
		D	2.39	2.62	2.95	3.20	3.99	4.37	4.92	5.33	1.75	0.85
	Unfastened	A	0.919	1.14	1.44	1.68	1.53	1.89	2.40	2.79	1.85	0.80
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.698	0.744	0.809	0.859	1.16	1.24	1.35	1.43	1.65	0.90
		D	0.896	1.03	1.22	1.37	1.49	1.72	2.03	2.28	1.90	0.80
8CS2.5x065	Fastened	A	0.792	0.982	1.25	1.46	1.32	1.64	2.08	2.43	1.75	0.85
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.644	0.729	0.850	0.943	1.07	1.22	1.42	1.57	1.75	0.85
		D	2.03	2.23	2.51	2.73	3.38	3.72	4.19	4.55	1.75	0.85
	Unfastened	A	0.792	0.982	1.25	1.46	1.32	1.64	2.08	2.43	1.85	0.80
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.565	0.603	0.657	0.699	0.941	1.00	1.10	1.16	1.65	0.90
		D	0.614	0.708	0.842	0.944	1.02	1.18	1.40	1.57	1.90	0.80
8CS2.5x059	Fastened	A	0.652	0.811	1.04	1.21	1.09	1.35	1.73	2.02	1.75	0.85
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.506	0.575	0.673	0.749	0.843	0.959	1.12	1.25	1.75	0.85
		D	1.63	1.80	2.04	2.22	2.72	3.00	3.39	3.70	1.75	0.85
	Unfastened	A	0.652	0.811	1.04	1.21	1.09	1.35	1.73	2.02	1.85	0.80
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.424	0.455	0.497	0.530	0.707	0.758	0.828	0.883	1.65	0.90
		D	0.322	0.373	0.445	0.500	0.536	0.621	0.741	0.833	1.90	0.80
6CS2.5x105	Fastened	A	2.11	2.56	3.20	3.69	3.51	4.27	5.34	6.16	1.75	0.85
		B	4.35	4.90	5.67	6.26	7.26	8.16	9.45	10.4	1.65	0.90
		C	2.19	2.43	2.78	3.05	3.65	4.06	4.63	5.08	1.75	0.85
		D	6.11	6.61	7.32	7.86	10.2	11.0	12.2	13.1	1.75	0.85
	Unfastened	A	2.11	2.56	3.20	3.69	3.51	4.27	5.34	6.16	1.85	0.80
		B	4.35	4.90	5.67	6.26	7.26	8.16	9.45	10.4	1.65	0.90
		C	2.23	2.35	2.53	2.66	3.71	3.92	4.21	4.43	1.65	0.90
		D	3.87	4.38	5.09	5.64	6.45	7.29	8.49	9.41	1.90	0.80

Table II - 10 (continued)

Web Crippling, P_n, kips [1,2]
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [4]				Bearing Length, N (in.) [4]					
			1	2	4	6	1	2	4	6		
6CS2.5x085	Fastened	A	1.40	1.71	2.16	2.50	2.33	2.85	3.60	4.17	1.75	0.85
		B	2.78	3.15	3.68	4.09	4.63	5.25	6.13	6.81	1.65	0.90
		C	1.37	1.53	1.77	1.95	2.28	2.56	2.95	3.25	1.75	0.85
		D	3.89	4.24	4.73	5.10	6.48	7.06	7.88	8.51	1.75	0.85
	Unfastened	A	1.40	1.71	2.16	2.50	2.33	2.85	3.60	4.17	1.85	0.80
		B	2.78	3.15	3.68	4.09	4.63	5.25	6.13	6.81	1.65	0.90
		C	1.29	1.37	1.48	1.57	2.16	2.29	2.47	2.62	1.65	0.90
		D	1.96	2.23	2.62	2.92	3.26	3.72	4.37	4.87	1.90	0.80
6CS2.5x070	Fastened	A	0.953	1.18	1.50	1.74	1.59	1.96	2.49	2.90	1.75	0.85
		B	1.83	2.09	2.46	2.74	3.05	3.48	4.10	4.57	1.65	0.90
		C	0.881	0.995	1.16	1.28	1.47	1.66	1.93	2.13	1.75	0.85
		D	2.56	2.80	3.15	3.42	4.26	4.67	5.25	5.69	1.75	0.85
	Unfastened	A	0.953	1.18	1.50	1.74	1.59	1.96	2.49	2.90	1.85	0.80
		B	1.83	2.09	2.46	2.74	3.05	3.48	4.10	4.57	1.65	0.90
		C	0.769	0.820	0.891	0.946	1.28	1.37	1.49	1.58	1.65	0.90
		D	0.898	1.03	1.22	1.37	1.50	1.72	2.04	2.28	1.90	0.80
6CS2.5x065	Fastened	A	0.823	1.02	1.30	1.51	1.37	1.70	2.17	2.52	1.75	0.85
		B	1.55	1.78	2.10	2.35	2.59	2.97	3.51	3.92	1.65	0.90
		C	0.742	0.841	0.980	1.09	1.24	1.40	1.63	1.81	1.75	0.85
		D	2.17	2.39	2.69	2.93	3.62	3.98	4.49	4.88	1.75	0.85
	Unfastened	A	0.823	1.02	1.30	1.51	1.37	1.70	2.17	2.52	1.85	0.80
		B	1.55	1.78	2.10	2.35	2.59	2.97	3.51	3.92	1.65	0.90
		C	0.626	0.668	0.728	0.774	1.04	1.11	1.21	1.29	1.65	0.90
		D	0.615	0.710	0.843	0.946	1.03	1.18	1.41	1.58	1.90	0.80
6CS2.5x059	Fastened	A	0.679	0.845	1.08	1.26	1.13	1.41	1.80	2.10	1.75	0.85
		B	1.25	1.44	1.71	1.92	2.09	2.41	2.86	3.20	1.65	0.90
		C	0.592	0.673	0.788	0.875	0.986	1.12	1.31	1.46	1.75	0.85
		D	1.76	1.94	2.19	2.39	2.93	3.23	3.66	3.98	1.75	0.85
	Unfastened	A	0.679	0.845	1.08	1.26	1.13	1.41	1.80	2.10	1.85	0.80
		B	1.25	1.44	1.71	1.92	2.09	2.41	2.86	3.20	1.65	0.90
		C	0.475	0.508	0.556	0.592	0.791	0.847	0.926	0.987	1.65	0.90
		D	0.322	0.373	0.446	0.501	0.537	0.622	0.743	0.835	1.90	0.80
4CS2.5x105	Fastened	A	2.18	2.65	3.31	3.82	3.63	4.42	5.52	6.37	1.75	0.85
		B	4.42	4.98	5.76	6.36	7.37	8.29	9.60	10.6	1.65	0.90
		C	2.43	2.70	3.08	3.38	4.04	4.49	5.13	5.63	1.75	0.85
		D	6.47	7.00	7.75	8.33	10.8	11.7	12.9	13.9	1.75	0.85
	Unfastened	A	2.18	2.65	3.31	3.82	3.63	4.42	5.52	6.37	1.85	0.80
		B	4.42	4.98	5.76	6.36	7.37	8.29	9.60	10.6	1.65	0.90
		C	2.41	2.55	2.73	2.88	4.02	4.24	4.56	4.80	1.65	0.90
		D	3.88	4.38	5.10	5.65	6.46	7.31	8.50	9.42	1.90	0.80
4CS2.5x085	Fastened	A	1.45	1.78	2.24	2.60	2.42	2.96	3.74	4.33	1.75	0.85
		B	2.83	3.21	3.75	4.16	4.71	5.35	6.24	6.93	1.65	0.90
		C	1.54	1.73	1.99	2.20	2.57	2.88	3.32	3.66	1.75	0.85
		D	4.15	4.52	5.05	5.45	6.92	7.54	8.41	9.08	1.75	0.85
	Unfastened	A	1.45	1.78	2.24	2.60	2.42	2.96	3.74	4.33	1.85	0.80
		B	2.83	3.21	3.75	4.16	4.71	5.35	6.24	6.93	1.65	0.90
		C	1.42	1.51	1.63	1.72	2.37	2.51	2.71	2.87	1.65	0.90
		D	1.96	2.24	2.63	2.93	3.27	3.73	4.38	4.88	1.90	0.80

Table II - 10 (continued)

Web Crippling, P_n, kips [1,2]
C-Sections With Lips

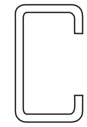
Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [4]				Bearing Length, N (in.) [4]					
			1	2	4	6	1	2	4	6		
4CS2.5x070	Fastened	A	0.995	1.23	1.56	1.82	1.66	2.05	2.60	3.03	1.75	0.85
		B	1.86	2.13	2.51	2.80	3.11	3.55	4.18	4.66	1.65	0.90
		C	1.01	1.14	1.33	1.47	1.69	1.91	2.22	2.45	1.75	0.85
		D	2.75	3.02	3.39	3.68	4.59	5.03	5.65	6.13	1.75	0.85
	Unfastened	A	0.995	1.23	1.56	1.82	1.66	2.05	2.60	3.03	1.85	0.80
		B	1.86	2.13	2.51	2.80	3.11	3.55	4.18	4.66	1.65	0.90
		C	0.854	0.911	0.990	1.05	1.42	1.52	1.65	1.75	1.65	0.90
		D	0.899	1.03	1.22	1.37	1.50	1.72	2.04	2.29	1.90	0.80
4CS2.5x065	Fastened	A	0.861	1.07	1.36	1.58	1.43	1.78	2.26	2.64	1.75	0.85
		B	1.59	1.82	2.15	2.40	2.64	3.03	3.58	4.00	1.65	0.90
		C	0.861	0.975	1.14	1.26	1.44	1.63	1.89	2.10	1.75	0.85
		D	2.35	2.58	2.91	3.16	3.92	4.30	4.85	5.27	1.75	0.85
	Unfastened	A	0.861	1.07	1.36	1.58	1.43	1.78	2.26	2.64	1.85	0.80
		B	1.59	1.82	2.15	2.40	2.64	3.03	3.58	4.00	1.65	0.90
		C	0.699	0.747	0.814	0.866	1.17	1.24	1.36	1.44	1.65	0.90
		D	0.616	0.711	0.845	0.947	1.03	1.18	1.41	1.58	1.90	0.80
4CS2.5x059	Fastened	A	0.712	0.886	1.13	1.32	1.19	1.48	1.89	2.20	1.75	0.85
		B	1.28	1.48	1.75	1.96	2.14	2.46	2.92	3.27	1.65	0.90
		C	0.695	0.790	0.925	1.03	1.16	1.32	1.54	1.71	1.75	0.85
		D	1.91	2.11	2.38	2.60	3.18	3.51	3.97	4.33	1.75	0.85
	Unfastened	A	0.712	0.886	1.13	1.32	1.19	1.48	1.89	2.20	1.85	0.80
		B	1.28	1.48	1.75	1.96	2.14	2.46	2.92	3.27	1.65	0.90
		C	0.535	0.573	0.626	0.667	0.892	0.955	1.04	1.11	1.65	0.90
		D	0.323	0.374	0.446	0.502	0.538	0.624	0.744	0.836	1.90	0.80

Notes:

1. Web crippling strengths are nominal strengths calculated without consideration of holes or other openings. To obtain design strengths, the values must be modified by factors of safety (ASD) or resistance factors (LRFD).
2. Strength reduction 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 Reaction, Opposing Loads Spaced > 1.5h
   Case C
   End Reaction, Opposing Loads Spaced ≤ 1.5h
   Case D
   Interior Reaction, Opposing Loads Spaced ≤ 1.5h
4. Linear interpolation is permitted between bearing lengths.

Table II - 11

Web Crippling, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 50 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [4]				Bearing Length, N (in.) [4]					
			1	2	4	6	1	2	4	6		
1200S200-97	Fastened	A	1.87	2.27	2.85	3.29	2.83	3.44	4.31	4.98	1.75	0.85
		B	4.10	4.62	5.35	5.91	6.21	6.99	8.10	8.96	1.65	0.90
		C	1.56	1.73	1.98	2.17	2.36	2.63	3.00	3.29	1.75	0.85
		D	5.02	5.44	6.03	6.48	7.61	8.24	9.14	9.82	1.75	0.85
	Unfastened	A	1.87	2.27	2.85	3.29	2.83	3.44	4.31	4.98	1.85	0.80
		B	4.10	4.62	5.35	5.91	6.21	6.99	8.10	8.96	1.65	0.90
		C	1.79	1.89	2.04	2.15	2.72	2.87	3.09	3.25	1.65	0.90
		D	4.33	4.90	5.71	6.33	6.55	7.42	8.65	9.59	1.90	0.80
1200S200-68	Fastened	A	0.956	1.18	1.50	1.74	1.45	1.79	2.27	2.64	1.75	0.85
		B	2.08	2.38	2.80	3.12	3.16	3.60	4.24	4.73	1.65	0.90
		C	0.636	0.718	0.833	0.922	0.964	1.09	1.26	1.40	1.75	0.85
		D	2.31	2.53	2.84	3.08	3.50	3.84	4.31	4.67	1.75	0.85
	Unfastened	A	0.956	1.18	1.50	1.74	1.45	1.79	2.27	2.64	1.85	0.80
		B	2.08	2.38	2.80	3.12	3.16	3.60	4.24	4.73	1.65	0.90
		C	0.770	0.820	0.891	0.945	1.17	1.24	1.35	1.43	1.65	0.90
		D	2.25	2.59	3.06	3.43	3.41	3.92	4.64	5.19	1.90	0.80
1000S250-97	Fastened	A	1.91	2.33	2.92	3.37	2.90	3.53	4.42	5.10	1.75	0.85
		B	4.14	4.67	5.41	5.98	6.28	7.07	8.19	9.05	1.65	0.90
		C	1.70	1.90	2.17	2.38	2.58	2.87	3.29	3.61	1.75	0.85
		D	5.25	5.68	6.30	6.77	7.95	8.61	9.55	10.3	1.75	0.85
	Unfastened	A	1.91	2.33	2.92	3.37	2.90	3.53	4.42	5.10	1.85	0.80
		B	4.14	4.67	5.41	5.98	6.28	7.07	8.19	9.05	1.65	0.90
		C	1.91	2.02	2.17	2.29	2.90	3.06	3.29	3.47	1.65	0.90
		D	4.33	4.90	5.72	6.34	6.56	7.43	8.66	9.60	1.90	0.80
1000S250-68	Fastened	A	0.986	1.22	1.55	1.80	1.49	1.84	2.34	2.72	1.75	0.85
		B	2.11	2.41	2.84	3.16	3.20	3.65	4.30	4.79	1.65	0.90
		C	0.727	0.820	0.952	1.05	1.10	1.24	1.44	1.60	1.75	0.85
		D	2.45	2.68	3.01	3.26	3.71	4.06	4.56	4.95	1.75	0.85
	Unfastened	A	0.986	1.22	1.55	1.80	1.49	1.84	2.34	2.72	1.85	0.80
		B	2.11	2.41	2.84	3.16	3.20	3.65	4.30	4.79	1.65	0.90
		C	0.842	0.897	0.975	1.03	1.28	1.36	1.48	1.57	1.65	0.90
		D	2.26	2.59	3.07	3.43	3.42	3.93	4.65	5.20	1.90	0.80
1000S250-54	Fastened	A	0.639	0.796	1.02	1.19	0.968	1.21	1.54	1.80	1.75	0.85
		B	1.36	1.57	1.87	2.09	2.06	2.38	2.83	3.17	1.65	0.90
		C	0.399	0.455	0.533	0.593	0.605	0.689	0.808	0.899	1.75	0.85
		D	1.47	1.63	1.84	2.01	2.23	2.46	2.79	3.04	1.75	0.85
	Unfastened	A	0.639	0.796	1.02	1.19	0.968	1.21	1.54	1.80	1.85	0.80
		B	1.36	1.57	1.87	2.09	2.06	2.38	2.83	3.17	1.65	0.90
		C	0.481	0.516	0.565	0.602	0.729	0.782	0.856	0.913	1.65	0.90
		D	1.48	1.72	2.06	2.31	2.25	2.61	3.11	3.51	1.90	0.80
1000S200-97	Fastened	A	1.91	2.33	2.92	3.37	2.90	3.53	4.42	5.10	1.75	0.85
		B	4.14	4.67	5.41	5.98	6.28	7.07	8.19	9.05	1.65	0.90
		C	1.70	1.90	2.17	2.38	2.58	2.87	3.29	3.61	1.75	0.85
		D	5.25	5.68	6.30	6.77	7.95	8.61	9.55	10.3	1.75	0.85
	Unfastened	A	1.91	2.33	2.92	3.37	2.90	3.53	4.42	5.10	1.85	0.80
		B	4.14	4.67	5.41	5.98	6.28	7.07	8.19	9.05	1.65	0.90
		C	1.91	2.02	2.17	2.29	2.90	3.06	3.29	3.47	1.65	0.90
		D	4.33	4.90	5.72	6.34	6.56	7.43	8.66	9.60	1.90	0.80

Table II - 11 (continued)

Web Crippling, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 50 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [4]				Bearing Length, N (in.) [4]					
			1	2	4	6	1	2	4	6		
1000S200-68	Fastened	A	0.986	1.22	1.55	1.80	1.49	1.84	2.34	2.72	1.75	0.85
		B	2.11	2.41	2.84	3.16	3.20	3.65	4.30	4.79	1.65	0.90
		C	0.727	0.820	0.952	1.05	1.10	1.24	1.44	1.60	1.75	0.85
		D	2.45	2.68	3.01	3.26	3.71	4.06	4.56	4.95	1.75	0.85
	Unfastened	A	0.986	1.22	1.55	1.80	1.49	1.84	2.34	2.72	1.85	0.80
		B	2.11	2.41	2.84	3.16	3.20	3.65	4.30	4.79	1.65	0.90
		C	0.842	0.897	0.975	1.03	1.28	1.36	1.48	1.57	1.65	0.90
		D	2.26	2.59	3.07	3.43	3.42	3.93	4.65	5.20	1.90	0.80
1000S200-54	Fastened	A	0.639	0.796	1.02	1.19	0.968	1.21	1.54	1.80	1.75	0.85
		B	1.36	1.57	1.87	2.09	2.06	2.38	2.83	3.17	1.65	0.90
		C	0.399	0.455	0.533	0.593	0.605	0.689	0.808	0.899	1.75	0.85
		D	1.47	1.63	1.84	2.01	2.23	2.46	2.79	3.04	1.75	0.85
	Unfastened	A	0.639	0.796	1.02	1.19	0.968	1.21	1.54	1.80	1.85	0.80
		B	1.36	1.57	1.87	2.09	2.06	2.38	2.83	3.17	1.65	0.90
		C	0.481	0.516	0.565	0.602	0.729	0.782	0.856	0.913	1.65	0.90
		D	1.48	1.72	2.06	2.31	2.25	2.61	3.11	3.51	1.90	0.80
800S200-97	Fastened	A	1.97	2.39	2.99	3.46	2.98	3.62	4.54	5.24	1.75	0.85
		B	4.19	4.72	5.47	6.05	6.35	7.15	8.29	9.16	1.65	0.90
		C	1.87	2.08	2.38	2.61	2.83	3.15	3.61	3.95	1.75	0.85
		D	5.50	5.96	6.60	7.10	8.33	9.02	10.0	10.8	1.75	0.85
	Unfastened	A	1.97	2.39	2.99	3.46	2.98	3.62	4.54	5.24	1.85	0.80
		B	4.19	4.72	5.47	6.05	6.35	7.15	8.29	9.16	1.65	0.90
		C	2.05	2.16	2.33	2.45	3.11	3.28	3.53	3.72	1.65	0.90
		D	4.33	4.91	5.72	6.35	6.57	7.44	8.67	9.61	1.90	0.80
800S200-68	Fastened	A	1.02	1.26	1.60	1.86	1.54	1.91	2.42	2.81	1.75	0.85
		B	2.14	2.45	2.88	3.21	3.24	3.71	4.36	4.86	1.65	0.90
		C	0.827	0.934	1.08	1.20	1.25	1.41	1.64	1.82	1.75	0.85
		D	2.60	2.85	3.20	3.47	3.94	4.31	4.84	5.25	1.75	0.85
	Unfastened	A	1.02	1.26	1.60	1.86	1.54	1.91	2.42	2.81	1.85	0.80
		B	2.14	2.45	2.88	3.21	3.24	3.71	4.36	4.86	1.65	0.90
		C	0.922	0.983	1.07	1.13	1.40	1.49	1.62	1.72	1.65	0.90
		D	2.26	2.60	3.07	3.44	3.42	3.93	4.65	5.21	1.90	0.80
800S200-54	Fastened	A	0.664	0.827	1.06	1.24	1.01	1.25	1.60	1.87	1.75	0.85
		B	1.39	1.60	1.90	2.13	2.10	2.42	2.88	3.23	1.65	0.90
		C	0.473	0.539	0.631	0.703	0.717	0.816	0.957	1.06	1.75	0.85
		D	1.58	1.75	1.98	2.16	2.40	2.65	3.00	3.27	1.75	0.85
	Unfastened	A	0.664	0.827	1.06	1.24	1.01	1.25	1.60	1.87	1.85	0.80
		B	1.39	1.60	1.90	2.13	2.10	2.42	2.88	3.23	1.65	0.90
		C	0.539	0.578	0.633	0.675	0.817	0.875	0.958	1.02	1.65	0.90
		D	1.48	1.72	2.06	2.32	2.25	2.61	3.12	3.51	1.90	0.80
800S200-43	Fastened	A	0.432	0.544	0.701	0.822	-	-	-	-	1.75	0.85
		B	0.894	1.04	1.25	1.41	-	-	-	-	1.65	0.90
		C	0.262	0.301	0.357	0.400	-	-	-	-	1.75	0.85
		D	0.959	1.07	1.22	1.34	-	-	-	-	1.75	0.85
	Unfastened	A	0.432	0.544	0.701	0.822	-	-	-	-	1.85	0.80
		B	0.894	1.04	1.25	1.41	-	-	-	-	1.65	0.90
		C	0.306	0.330	0.365	0.391	-	-	-	-	1.65	0.90
		D	0.940	1.10	1.33	1.50	-	-	-	-	1.90	0.80

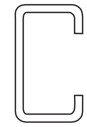
Table II - 11 (continued)

Web Crippling, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 50 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [4]				Bearing Length, N (in.) [4]					
			1	2	4	6	1	2	4	6		
800S162-97	Fastened	A	1.97	2.39	2.99	3.46	2.98	3.62	4.54	5.24	1.75	0.85
		B	4.19	4.72	5.47	6.05	6.35	7.15	8.29	9.16	1.65	0.90
		C	1.87	2.08	2.38	2.61	2.83	3.15	3.61	3.95	1.75	0.85
		D	5.50	5.96	6.60	7.10	8.33	9.02	10.0	10.8	1.75	0.85
	Unfastened	A	1.97	2.39	2.99	3.46	2.98	3.62	4.54	5.24	1.85	0.80
		B	4.19	4.72	5.47	6.05	6.35	7.15	8.29	9.16	1.65	0.90
		C	2.05	2.16	2.33	2.45	3.11	3.28	3.53	3.72	1.65	0.90
		D	4.33	4.91	5.72	6.35	6.57	7.44	8.67	9.61	1.90	0.80
800S162-68	Fastened	A	1.02	1.26	1.60	1.86	1.54	1.91	2.42	2.81	1.75	0.85
		B	2.14	2.45	2.88	3.21	3.24	3.71	4.36	4.86	1.65	0.90
		C	0.827	0.934	1.08	1.20	1.25	1.41	1.64	1.82	1.75	0.85
		D	2.60	2.85	3.20	3.47	3.94	4.31	4.84	5.25	1.75	0.85
	Unfastened	A	1.02	1.26	1.60	1.86	1.54	1.91	2.42	2.81	1.85	0.80
		B	2.14	2.45	2.88	3.21	3.24	3.71	4.36	4.86	1.65	0.90
		C	0.922	0.983	1.07	1.13	1.40	1.49	1.62	1.72	1.65	0.90
		D	2.26	2.60	3.07	3.44	3.42	3.93	4.65	5.21	1.90	0.80
800S162-54	Fastened	A	0.664	0.827	1.06	1.24	1.01	1.25	1.60	1.87	1.75	0.85
		B	1.39	1.60	1.90	2.13	2.10	2.42	2.88	3.23	1.65	0.90
		C	0.473	0.539	0.631	0.703	0.717	0.816	0.957	1.06	1.75	0.85
		D	1.58	1.75	1.98	2.16	2.40	2.65	3.00	3.27	1.75	0.85
	Unfastened	A	0.664	0.827	1.06	1.24	1.01	1.25	1.60	1.87	1.85	0.80
		B	1.39	1.60	1.90	2.13	2.10	2.42	2.88	3.23	1.65	0.90
		C	0.539	0.578	0.633	0.675	0.817	0.875	0.958	1.02	1.65	0.90
		D	1.48	1.72	2.06	2.32	2.25	2.61	3.12	3.51	1.90	0.80
800S162-43	Fastened	A	0.432	0.544	0.701	0.822	-	-	-	-	1.75	0.85
		B	0.894	1.04	1.25	1.41	-	-	-	-	1.65	0.90
		C	0.262	0.301	0.357	0.400	-	-	-	-	1.75	0.85
		D	0.959	1.07	1.22	1.34	-	-	-	-	1.75	0.85
	Unfastened	A	0.432	0.544	0.701	0.822	-	-	-	-	1.85	0.80
		B	0.894	1.04	1.25	1.41	-	-	-	-	1.65	0.90
		C	0.306	0.330	0.365	0.391	-	-	-	-	1.65	0.90
		D	0.940	1.10	1.33	1.50	-	-	-	-	1.90	0.80
600S200-97	Fastened	A	2.02	2.46	3.08	3.56	3.07	3.73	4.67	5.39	1.75	0.85
		B	4.25	4.79	5.55	6.13	6.44	7.25	8.40	9.28	1.65	0.90
		C	2.06	2.29	2.62	2.87	3.12	3.47	3.97	4.35	1.75	0.85
		D	5.79	6.27	6.95	7.47	8.77	9.49	10.5	11.3	1.75	0.85
	Unfastened	A	2.02	2.46	3.08	3.56	3.07	3.73	4.67	5.39	1.85	0.80
		B	4.25	4.79	5.55	6.13	6.44	7.25	8.40	9.28	1.65	0.90
		C	2.20	2.33	2.50	2.64	3.34	3.53	3.79	3.99	1.65	0.90
		D	4.34	4.92	5.73	6.35	6.58	7.45	8.68	9.63	1.90	0.80
600S200-68	Fastened	A	1.06	1.30	1.66	1.92	1.60	1.98	2.51	2.92	1.75	0.85
		B	2.18	2.49	2.92	3.26	3.30	3.77	4.43	4.94	1.65	0.90
		C	0.942	1.06	1.23	1.37	1.43	1.61	1.87	2.07	1.75	0.85
		D	2.77	3.04	3.41	3.70	4.20	4.60	5.17	5.60	1.75	0.85
	Unfastened	A	1.06	1.30	1.66	1.92	1.60	1.98	2.51	2.92	1.85	0.80
		B	2.18	2.49	2.92	3.26	3.30	3.77	4.43	4.94	1.65	0.90
		C	1.01	1.08	1.17	1.25	1.54	1.64	1.78	1.89	1.65	0.90
		D	2.26	2.60	3.08	3.44	3.43	3.94	4.66	5.21	1.90	0.80

Table II - 11 (continued)

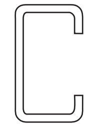
Web Crippling, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 50 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [4]				Bearing Length, N (in.) [4]					
			1	2	4	6	1	2	4	6		
600S200-54	Fastened	A	0.692	0.862	1.10	1.29	1.05	1.31	1.67	1.95	1.75	0.85
		B	1.41	1.63	1.93	2.17	2.14	2.47	2.93	3.28	1.65	0.90
		C	0.557	0.634	0.744	0.828	0.844	0.961	1.13	1.25	1.75	0.85
		D	1.71	1.88	2.14	2.33	2.59	2.86	3.24	3.53	1.75	0.85
	Unfastened	A	0.692	0.862	1.10	1.29	1.05	1.31	1.67	1.95	1.85	0.80
		B	1.41	1.63	1.93	2.17	2.14	2.47	2.93	3.28	1.65	0.90
		C	0.605	0.648	0.710	0.757	0.916	0.982	1.08	1.15	1.65	0.90
		D	1.49	1.73	2.06	2.32	2.25	2.61	3.12	3.52	1.90	0.80
600S200-43	Fastened	A	0.453	0.570	0.736	0.862	-	-	-	-	1.75	0.85
		B	0.913	1.06	1.28	1.44	-	-	-	-	1.65	0.90
		C	0.324	0.372	0.441	0.494	-	-	-	-	1.75	0.85
		D	1.05	1.17	1.34	1.47	-	-	-	-	1.75	0.85
	Unfastened	A	0.453	0.570	0.736	0.862	-	-	-	-	1.85	0.80
		B	0.913	1.06	1.28	1.44	-	-	-	-	1.65	0.90
		C	0.353	0.381	0.420	0.451	-	-	-	-	1.65	0.90
		D	0.942	1.10	1.33	1.51	-	-	-	-	1.90	0.80
600S200-33	Fastened	A	0.267	0.340	0.442	0.520	-	-	-	-	1.75	0.85
		B	0.516	0.607	0.737	0.837	-	-	-	-	1.65	0.90
		C	0.163	0.190	0.227	0.256	-	-	-	-	1.75	0.85
		D	0.577	0.648	0.750	0.828	-	-	-	-	1.75	0.85
	Unfastened	A	0.267	0.340	0.442	0.520	-	-	-	-	1.85	0.80
		B	0.516	0.607	0.737	0.837	-	-	-	-	1.65	0.90
		C	0.165	0.179	0.200	0.216	-	-	-	-	1.65	0.90
		D	0.383	0.454	0.555	0.632	-	-	-	-	1.90	0.80
600S162-97	Fastened	A	2.02	2.46	3.08	3.56	3.07	3.73	4.67	5.39	1.75	0.85
		B	4.25	4.79	5.55	6.13	6.44	7.25	8.40	9.28	1.65	0.90
		C	2.06	2.29	2.62	2.87	3.12	3.47	3.97	4.35	1.75	0.85
		D	5.79	6.27	6.95	7.47	8.77	9.49	10.5	11.3	1.75	0.85
	Unfastened	A	2.02	2.46	3.08	3.56	3.07	3.73	4.67	5.39	1.85	0.80
		B	4.25	4.79	5.55	6.13	6.44	7.25	8.40	9.28	1.65	0.90
		C	2.20	2.33	2.50	2.64	3.34	3.53	3.79	3.99	1.65	0.90
		D	4.34	4.92	5.73	6.35	6.58	7.45	8.68	9.63	1.90	0.80
600S162-68	Fastened	A	1.06	1.30	1.66	1.92	1.60	1.98	2.51	2.92	1.75	0.85
		B	2.18	2.49	2.92	3.26	3.30	3.77	4.43	4.94	1.65	0.90
		C	0.942	1.06	1.23	1.37	1.43	1.61	1.87	2.07	1.75	0.85
		D	2.77	3.04	3.41	3.70	4.20	4.60	5.17	5.60	1.75	0.85
	Unfastened	A	1.06	1.30	1.66	1.92	1.60	1.98	2.51	2.92	1.85	0.80
		B	2.18	2.49	2.92	3.26	3.30	3.77	4.43	4.94	1.65	0.90
		C	1.01	1.08	1.17	1.25	1.54	1.64	1.78	1.89	1.65	0.90
		D	2.26	2.60	3.08	3.44	3.43	3.94	4.66	5.21	1.90	0.80
600S162-54	Fastened	A	0.692	0.862	1.10	1.29	1.05	1.31	1.67	1.95	1.75	0.85
		B	1.41	1.63	1.93	2.17	2.14	2.47	2.93	3.28	1.65	0.90
		C	0.557	0.634	0.744	0.828	0.844	0.961	1.13	1.25	1.75	0.85
		D	1.71	1.88	2.14	2.33	2.59	2.86	3.24	3.53	1.75	0.85
	Unfastened	A	0.692	0.862	1.10	1.29	1.05	1.31	1.67	1.95	1.85	0.80
		B	1.41	1.63	1.93	2.17	2.14	2.47	2.93	3.28	1.65	0.90
		C	0.605	0.648	0.710	0.757	0.916	0.982	1.08	1.15	1.65	0.90
		D	1.49	1.73	2.06	2.32	2.25	2.61	3.12	3.52	1.90	0.80

Table II - 11 (continued)

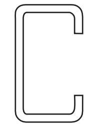
Web Crippling, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 50 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [4]				Bearing Length, N (in.) [4]					
			1	2	4	6	1	2	4	6		
600S162-43	Fastened	A	0.453	0.570	0.736	0.862	-	-	-	-	1.75	0.85
		B	0.913	1.06	1.28	1.44	-	-	-	-	1.65	0.90
		C	0.324	0.372	0.441	0.494	-	-	-	-	1.75	0.85
		D	1.05	1.17	1.34	1.47	-	-	-	-	1.75	0.85
	Unfastened	A	0.453	0.570	0.736	0.862	-	-	-	-	1.85	0.80
		B	0.913	1.06	1.28	1.44	-	-	-	-	1.65	0.90
		C	0.353	0.381	0.420	0.451	-	-	-	-	1.65	0.90
		D	0.942	1.10	1.33	1.51	-	-	-	-	1.90	0.80
600S162-33	Fastened	A	0.267	0.340	0.442	0.520	-	-	-	-	1.75	0.85
		B	0.516	0.607	0.737	0.837	-	-	-	-	1.65	0.90
		C	0.163	0.190	0.227	0.256	-	-	-	-	1.75	0.85
		D	0.577	0.648	0.750	0.828	-	-	-	-	1.75	0.85
	Unfastened	A	0.267	0.340	0.442	0.520	-	-	-	-	1.85	0.80
		B	0.516	0.607	0.737	0.837	-	-	-	-	1.65	0.90
		C	0.165	0.179	0.200	0.216	-	-	-	-	1.65	0.90
		D	0.383	0.454	0.555	0.632	-	-	-	-	1.90	0.80
550S162-68	Fastened	A	1.07	1.32	1.67	1.94	1.62	2.00	2.53	2.94	1.75	0.85
		B	2.18	2.50	2.94	3.27	3.31	3.78	4.45	4.96	1.65	0.90
		C	0.974	1.10	1.28	1.41	1.48	1.67	1.93	2.14	1.75	0.85
		D	2.82	3.09	3.47	3.76	4.27	4.68	5.26	5.70	1.75	0.85
	Unfastened	A	1.07	1.32	1.67	1.94	1.62	2.00	2.53	2.94	1.85	0.80
		B	2.18	2.50	2.94	3.27	3.31	3.78	4.45	4.96	1.65	0.90
		C	1.04	1.11	1.20	1.28	1.57	1.68	1.82	1.93	1.65	0.90
		D	2.26	2.60	3.08	3.44	3.43	3.94	4.66	5.22	1.90	0.80
550S162-54	Fastened	A	0.700	0.872	1.12	1.30	1.06	1.32	1.69	1.97	1.75	0.85
		B	1.42	1.64	1.94	2.18	2.15	2.48	2.94	3.30	1.65	0.90
		C	0.580	0.661	0.775	0.862	0.879	1.00	1.17	1.31	1.75	0.85
		D	1.74	1.92	2.18	2.38	2.64	2.91	3.30	3.60	1.75	0.85
	Unfastened	A	0.700	0.872	1.12	1.30	1.06	1.32	1.69	1.97	1.85	0.80
		B	1.42	1.64	1.94	2.18	2.15	2.48	2.94	3.30	1.65	0.90
		C	0.623	0.668	0.731	0.780	0.944	1.01	1.11	1.18	1.65	0.90
		D	1.49	1.73	2.06	2.32	2.25	2.62	3.13	3.52	1.90	0.80
550S162-43	Fastened	A	0.459	0.578	0.745	0.874	-	-	-	-	1.75	0.85
		B	0.918	1.07	1.28	1.45	-	-	-	-	1.65	0.90
		C	0.341	0.392	0.464	0.520	-	-	-	-	1.75	0.85
		D	1.07	1.20	1.37	1.50	-	-	-	-	1.75	0.85
	Unfastened	A	0.459	0.578	0.745	0.874	-	-	-	-	1.85	0.80
		B	0.918	1.07	1.28	1.45	-	-	-	-	1.65	0.90
		C	0.366	0.395	0.436	0.467	-	-	-	-	1.65	0.90
		D	0.943	1.10	1.33	1.51	-	-	-	-	1.90	0.80
550S162-33	Fastened	A	0.272	0.345	0.449	0.528	-	-	-	-	1.75	0.85
		B	0.519	0.611	0.742	0.842	-	-	-	-	1.65	0.90
		C	0.175	0.203	0.243	0.274	-	-	-	-	1.75	0.85
		D	0.594	0.668	0.772	0.852	-	-	-	-	1.75	0.85
	Unfastened	A	0.272	0.345	0.449	0.528	-	-	-	-	1.85	0.80
		B	0.519	0.611	0.742	0.842	-	-	-	-	1.65	0.90
		C	0.173	0.188	0.209	0.226	-	-	-	-	1.65	0.90
		D	0.384	0.455	0.555	0.632	-	-	-	-	1.90	0.80

Table II - 11 (continued)

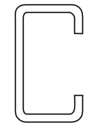
Web Crippling, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 50 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [4]				Bearing Length, N (in.) [4]					
			1	2	4	6	1	2	4	6		
400S162-68	Fastened	A	1.10	1.36	1.73	2.01	1.67	2.06	2.61	3.04	1.75	0.85
		B	2.22	2.53	2.98	3.32	3.36	3.84	4.51	5.03	1.65	0.90
		C	1.08	1.22	1.42	1.57	1.64	1.85	2.14	2.37	1.75	0.85
		D	2.98	3.26	3.67	3.97	4.51	4.94	5.55	6.02	1.75	0.85
	Unfastened	A	1.10	1.36	1.73	2.01	1.67	2.06	2.61	3.04	1.85	0.80
		B	2.22	2.53	2.98	3.32	3.36	3.84	4.51	5.03	1.65	0.90
		C	1.12	1.20	1.30	1.38	1.70	1.81	1.97	2.09	1.65	0.90
		D	2.27	2.60	3.08	3.45	3.43	3.95	4.67	5.22	1.90	0.80
400S162-54	Fastened	A	0.725	0.904	1.16	1.35	1.10	1.37	1.75	2.05	1.75	0.85
		B	1.44	1.66	1.97	2.21	2.18	2.52	2.99	3.36	1.65	0.90
		C	0.657	0.749	0.878	0.977	0.996	1.13	1.33	1.48	1.75	0.85
		D	1.86	2.05	2.32	2.53	2.81	3.11	3.52	3.84	1.75	0.85
	Unfastened	A	0.725	0.904	1.16	1.35	1.10	1.37	1.75	2.05	1.85	0.80
		B	1.44	1.66	1.97	2.21	2.18	2.52	2.99	3.36	1.65	0.90
		C	0.684	0.733	0.802	0.856	1.04	1.11	1.22	1.30	1.65	0.90
		D	1.49	1.73	2.07	2.33	2.26	2.62	3.13	3.52	1.90	0.80
400S162-43	Fastened	A	0.479	0.602	0.777	0.911	-	-	-	-	1.75	0.85
		B	0.935	1.09	1.31	1.47	-	-	-	-	1.65	0.90
		C	0.398	0.457	0.541	0.606	-	-	-	-	1.75	0.85
		D	1.16	1.29	1.47	1.62	-	-	-	-	1.75	0.85
	Unfastened	A	0.479	0.602	0.777	0.911	-	-	-	-	1.85	0.80
		B	0.935	1.09	1.31	1.47	-	-	-	-	1.65	0.90
		C	0.409	0.441	0.487	0.522	-	-	-	-	1.65	0.90
		D	0.944	1.11	1.33	1.51	-	-	-	-	1.90	0.80
400S162-33	Fastened	A	0.285	0.362	0.471	0.555	-	-	-	-	1.75	0.85
		B	0.530	0.625	0.758	0.861	-	-	-	-	1.65	0.90
		C	0.214	0.249	0.298	0.336	-	-	-	-	1.75	0.85
		D	0.650	0.731	0.846	0.934	-	-	-	-	1.75	0.85
	Unfastened	A	0.285	0.362	0.471	0.555	-	-	-	-	1.85	0.80
		B	0.530	0.625	0.758	0.861	-	-	-	-	1.65	0.90
		C	0.199	0.216	0.241	0.260	-	-	-	-	1.65	0.90
		D	0.384	0.455	0.556	0.633	-	-	-	-	1.90	0.80
362S162-68	Fastened	A	1.11	1.37	1.74	2.02	1.68	2.08	2.64	3.07	1.75	0.85
		B	2.23	2.54	2.99	3.34	3.37	3.85	4.53	5.05	1.65	0.90
		C	1.11	1.25	1.45	1.61	1.68	1.90	2.20	2.44	1.75	0.85
		D	3.02	3.31	3.72	4.03	4.58	5.02	5.64	6.11	1.75	0.85
	Unfastened	A	1.11	1.37	1.74	2.02	1.68	2.08	2.64	3.07	1.85	0.80
		B	2.23	2.54	2.99	3.34	3.37	3.85	4.53	5.05	1.65	0.90
		C	1.15	1.22	1.33	1.41	1.74	1.85	2.01	2.14	1.65	0.90
		D	2.27	2.60	3.08	3.45	3.43	3.95	4.67	5.23	1.90	0.80
362S162-54	Fastened	A	0.733	0.913	1.17	1.37	1.11	1.38	1.77	2.07	1.75	0.85
		B	1.45	1.67	1.98	2.22	2.19	2.53	3.01	3.37	1.65	0.90
		C	0.679	0.773	0.907	1.01	1.03	1.17	1.37	1.53	1.75	0.85
		D	1.89	2.09	2.36	2.58	2.86	3.16	3.58	3.91	1.75	0.85
	Unfastened	A	0.733	0.913	1.17	1.37	1.11	1.38	1.77	2.07	1.85	0.80
		B	1.45	1.67	1.98	2.22	2.19	2.53	3.01	3.37	1.65	0.90
		C	0.700	0.751	0.822	0.877	1.06	1.14	1.25	1.33	1.65	0.90
		D	1.49	1.73	2.07	2.33	2.26	2.62	3.13	3.52	1.90	0.80

Table II - 11 (continued)

Web Crippling, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 50 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [4]				Bearing Length, N (in.) [4]					
			1	2	4	6	1	2	4	6		
362S162-43	Fastened	A	0.484	0.609	0.786	0.921	-	-	-	-	1.75	0.85
		B	0.940	1.09	1.31	1.48	-	-	-	-	1.65	0.90
		C	0.414	0.476	0.563	0.630	-	-	-	-	1.75	0.85
		D	1.18	1.32	1.50	1.65	-	-	-	-	1.75	0.85
	Unfastened	A	0.484	0.609	0.786	0.921	-	-	-	-	1.85	0.80
		B	0.940	1.09	1.31	1.48	-	-	-	-	1.65	0.90
		C	0.421	0.454	0.501	0.537	-	-	-	-	1.65	0.90
		D	0.945	1.11	1.34	1.51	-	-	-	-	1.90	0.80
362S162-33	Fastened	A	0.289	0.367	0.478	0.563	-	-	-	-	1.75	0.85
		B	0.534	0.628	0.763	0.866	-	-	-	-	1.65	0.90
		C	0.225	0.262	0.313	0.353	-	-	-	-	1.75	0.85
		D	0.666	0.749	0.867	0.957	-	-	-	-	1.75	0.85
	Unfastened	A	0.289	0.367	0.478	0.563	-	-	-	-	1.85	0.80
		B	0.534	0.628	0.763	0.866	-	-	-	-	1.65	0.90
		C	0.206	0.224	0.250	0.269	-	-	-	-	1.65	0.90
		D	0.385	0.456	0.556	0.633	-	-	-	-	1.90	0.80
350S162-68	Fastened	A	1.11	1.38	1.75	2.03	1.69	2.08	2.65	3.08	1.75	0.85
		B	2.23	2.55	3.00	3.34	3.38	3.86	4.54	5.06	1.65	0.90
		C	1.12	1.26	1.47	1.62	1.70	1.92	2.22	2.46	1.75	0.85
		D	3.04	3.33	3.74	4.05	4.60	5.04	5.67	6.14	1.75	0.85
	Unfastened	A	1.11	1.38	1.75	2.03	1.69	2.08	2.65	3.08	1.85	0.80
		B	2.23	2.55	3.00	3.34	3.38	3.86	4.54	5.06	1.65	0.90
		C	1.16	1.23	1.34	1.42	1.75	1.87	2.03	2.15	1.65	0.90
		D	2.27	2.61	3.08	3.45	3.44	3.95	4.67	5.23	1.90	0.80
350S162-54	Fastened	A	0.735	0.917	1.17	1.37	1.11	1.39	1.78	2.08	1.75	0.85
		B	1.45	1.67	1.99	2.23	2.20	2.53	3.01	3.38	1.65	0.90
		C	0.687	0.782	0.917	1.02	1.04	1.18	1.39	1.55	1.75	0.85
		D	1.90	2.10	2.38	2.59	2.88	3.18	3.60	3.93	1.75	0.85
	Unfastened	A	0.735	0.917	1.17	1.37	1.11	1.39	1.78	2.08	1.85	0.80
		B	1.45	1.67	1.99	2.23	2.20	2.53	3.01	3.38	1.65	0.90
		C	0.706	0.757	0.829	0.884	1.07	1.15	1.26	1.34	1.65	0.90
		D	1.49	1.73	2.07	2.33	2.26	2.62	3.13	3.53	1.90	0.80
350S162-43	Fastened	A	0.486	0.612	0.789	0.925	-	-	-	-	1.75	0.85
		B	0.942	1.10	1.32	1.48	-	-	-	-	1.65	0.90
		C	0.419	0.482	0.570	0.638	-	-	-	-	1.75	0.85
		D	1.19	1.32	1.52	1.66	-	-	-	-	1.75	0.85
	Unfastened	A	0.486	0.612	0.789	0.925	-	-	-	-	1.85	0.80
		B	0.942	1.10	1.32	1.48	-	-	-	-	1.65	0.90
		C	0.425	0.459	0.506	0.543	-	-	-	-	1.65	0.90
		D	0.945	1.11	1.34	1.51	-	-	-	-	1.90	0.80
350S162-33	Fastened	A	0.290	0.369	0.480	0.565	-	-	-	-	1.75	0.85
		B	0.535	0.630	0.764	0.867	-	-	-	-	1.65	0.90
		C	0.229	0.266	0.319	0.359	-	-	-	-	1.75	0.85
		D	0.672	0.755	0.874	0.964	-	-	-	-	1.75	0.85
	Unfastened	A	0.290	0.369	0.480	0.565	-	-	-	-	1.85	0.80
		B	0.535	0.630	0.764	0.867	-	-	-	-	1.65	0.90
		C	0.209	0.227	0.253	0.273	-	-	-	-	1.65	0.90
		D	0.385	0.456	0.556	0.634	-	-	-	-	1.90	0.80

Table II - 11 (continued)

Web Crippling, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi, Bearing Length, N (in.) [4]				F_y = 50 ksi, Bearing Length, N (in.) [4]				Ω_w	ϕ_w
			1	2	4	6	1	2	4	6		
250S162-68	Fastened	A	1.14	1.41	1.79	2.08	1.73	2.14	2.72	3.16	1.75	0.85
		B	2.26	2.58	3.03	3.38	3.42	3.91	4.60	5.12	1.65	0.90
		C	1.21	1.37	1.59	1.76	1.84	2.07	2.41	2.66	1.75	0.85
		D	3.18	3.48	3.91	4.24	4.81	5.27	5.92	6.42	1.75	0.85
	Unfastened	A	1.14	1.41	1.79	2.08	1.73	2.14	2.72	3.16	1.85	0.80
		B	2.26	2.58	3.03	3.38	3.42	3.91	4.60	5.12	1.65	0.90
		C	1.23	1.31	1.42	1.51	1.86	1.98	2.16	2.29	1.65	0.90
		D	2.27	2.61	3.09	3.45	3.44	3.95	4.68	5.23	1.90	0.80
250S162-54	Fastened	A	0.757	0.944	1.21	1.41	1.15	1.43	1.83	2.14	1.75	0.85
		B	1.47	1.70	2.01	2.26	2.23	2.57	3.05	3.42	1.65	0.90
		C	0.753	0.857	1.01	1.12	1.14	1.30	1.52	1.69	1.75	0.85
		D	2.00	2.21	2.50	2.73	3.03	3.34	3.79	4.13	1.75	0.85
	Unfastened	A	0.757	0.944	1.21	1.41	1.15	1.43	1.83	2.14	1.85	0.80
		B	1.47	1.70	2.01	2.26	2.23	2.57	3.05	3.42	1.65	0.90
		C	0.758	0.813	0.890	0.949	1.15	1.23	1.35	1.44	1.65	0.90
		D	1.49	1.73	2.07	2.33	2.26	2.62	3.14	3.53	1.90	0.80
250S162-43	Fastened	A	0.503	0.632	0.816	0.956	-	-	-	-	1.75	0.85
		B	0.956	1.11	1.34	1.51	-	-	-	-	1.65	0.90
		C	0.467	0.537	0.636	0.712	-	-	-	-	1.75	0.85
		D	1.26	1.40	1.61	1.76	-	-	-	-	1.75	0.85
	Unfastened	A	0.503	0.632	0.816	0.956	-	-	-	-	1.85	0.80
		B	0.956	1.11	1.34	1.51	-	-	-	-	1.65	0.90
		C	0.462	0.498	0.550	0.589	-	-	-	-	1.65	0.90
		D	0.946	1.11	1.34	1.51	-	-	-	-	1.90	0.80
250S162-33	Fastened	A	0.302	0.384	0.499	0.588	-	-	-	-	1.75	0.85
		B	0.544	0.641	0.778	0.883	-	-	-	-	1.65	0.90
		C	0.262	0.305	0.365	0.411	-	-	-	-	1.75	0.85
		D	0.720	0.810	0.937	1.03	-	-	-	-	1.75	0.85
	Unfastened	A	0.302	0.384	0.499	0.588	-	-	-	-	1.85	0.80
		B	0.544	0.641	0.778	0.883	-	-	-	-	1.65	0.90
		C	0.231	0.251	0.280	0.302	-	-	-	-	1.65	0.90
		D	0.385	0.457	0.557	0.635	-	-	-	-	1.90	0.80

Notes:

1. Web crippling strengths are nominal strengths calculated without consideration of holes or other openings. To obtain design strengths, the values must be modified by factors of safety (ASD) or resistance factors (LRFD).

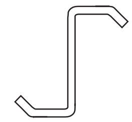
2. Strength reduction factors for standard factory punchouts for SSMA Studs are given in Tables II-13a and II-13b. Strength reduction factors for other openings must be calculated in accordance with the provisions of *Specification* Section C3.4.2.

3. Case A
   End Reaction, Opposing Loads Spaced > 1.5h
   Case B
   Interior Reaction, Opposing Loads Spaced > 1.5h
   Case C
   End Reaction, Opposing Loads Spaced ≤ 1.5h
   Case D
   Interior Reaction, Opposing Loads Spaced ≤ 1.5h

4. Linear interpolation is permitted between bearing lengths.

Table II - 12

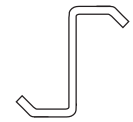
Web Crippling, P_n, kips [1]
Z-Sections With Lips



Section	Fastened or Unfastened	Case[2]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [3]				Bearing Length, N (in.) [3]					
			1	2	4	6	1	2	4	6		
12ZS3.25x105	Fastened	A	1.95	2.37	2.96	3.41	3.25	3.94	4.93	5.69	1.75	0.85
		B	4.20	4.73	5.47	6.04	7.00	7.88	9.12	10.1	1.65	0.90
		C	2.09	2.38	2.78	3.09	3.48	3.96	4.63	5.15	1.75	0.85
		D	5.61	6.02	6.61	7.06	9.35	10.0	11.0	11.8	1.85	0.80
	Unfastened	A	1.68	1.72	1.78	1.82	2.80	2.87	2.97	3.04	1.80	0.85
		B	4.20	4.73	5.47	6.04	7.00	7.88	9.12	10.1	1.65	0.90
		C	1.82	1.92	2.07	2.17	3.04	3.20	3.44	3.62	1.65	0.90
		D	3.86	4.36	5.08	5.63	6.43	7.27	8.46	9.38	1.90	0.80
12ZS3.25x085	Fastened	A	1.28	1.56	1.97	2.29	2.13	2.61	3.29	3.81	1.75	0.85
		B	2.67	3.03	3.54	3.93	4.45	5.05	5.89	6.54	1.65	0.90
		C	1.22	1.40	1.65	1.85	2.03	2.33	2.75	3.08	1.75	0.85
		D	3.41	3.68	4.07	4.36	5.68	6.13	6.78	7.27	1.85	0.80
	Unfastened	A	1.09	1.12	1.16	1.19	1.82	1.87	1.93	1.99	1.80	0.85
		B	2.67	3.03	3.54	3.93	4.45	5.05	5.89	6.54	1.65	0.90
		C	1.02	1.08	1.17	1.24	1.70	1.80	1.95	2.06	1.65	0.90
		D	1.95	2.22	2.61	2.91	3.25	3.71	4.35	4.85	1.90	0.80
12ZS3.25x070	Fastened	A	0.861	1.06	1.35	1.57	1.44	1.77	2.25	2.62	1.75	0.85
		B	1.75	2.00	2.35	2.62	2.91	3.33	3.92	4.37	1.65	0.90
		C	0.716	0.828	0.986	1.11	1.19	1.38	1.64	1.85	1.75	0.85
		D	2.12	2.30	2.56	2.76	3.53	3.84	4.27	4.60	1.85	0.80
	Unfastened	A	0.732	0.753	0.783	0.807	1.22	1.26	1.31	1.34	1.80	0.85
		B	1.75	2.00	2.35	2.62	2.91	3.33	3.92	4.37	1.65	0.90
		C	0.580	0.619	0.673	0.714	0.967	1.03	1.12	1.19	1.65	0.90
		D	0.894	1.03	1.22	1.36	1.49	1.71	2.03	2.27	1.90	0.80
12ZS2.25x105	Fastened	A	1.95	2.37	2.96	3.41	3.25	3.94	4.93	5.69	1.75	0.85
		B	4.20	4.73	5.47	6.04	7.00	7.88	9.12	10.1	1.65	0.90
		C	2.09	2.38	2.78	3.09	3.48	3.96	4.63	5.15	1.75	0.85
		D	5.61	6.02	6.61	7.06	9.35	10.0	11.0	11.8	1.85	0.80
	Unfastened	A	1.68	1.72	1.78	1.82	2.80	2.87	2.97	3.04	1.80	0.85
		B	4.20	4.73	5.47	6.04	7.00	7.88	9.12	10.1	1.65	0.90
		C	1.82	1.92	2.07	2.17	3.04	3.20	3.44	3.62	1.65	0.90
		D	3.86	4.36	5.08	5.63	6.43	7.27	8.46	9.38	1.90	0.80
12ZS2.25x085	Fastened	A	1.28	1.56	1.97	2.29	2.13	2.61	3.29	3.81	1.75	0.85
		B	2.67	3.03	3.54	3.93	4.45	5.05	5.89	6.54	1.65	0.90
		C	1.22	1.40	1.65	1.85	2.03	2.33	2.75	3.08	1.75	0.85
		D	3.41	3.68	4.07	4.36	5.68	6.13	6.78	7.27	1.85	0.80
	Unfastened	A	1.09	1.12	1.16	1.19	1.82	1.87	1.93	1.99	1.80	0.85
		B	2.67	3.03	3.54	3.93	4.45	5.05	5.89	6.54	1.65	0.90
		C	1.02	1.08	1.17	1.24	1.70	1.80	1.95	2.06	1.65	0.90
		D	1.95	2.22	2.61	2.91	3.25	3.71	4.35	4.85	1.90	0.80
12ZS2.25x070	Fastened	A	0.861	1.06	1.35	1.57	1.44	1.77	2.25	2.62	1.75	0.85
		B	1.75	2.00	2.35	2.62	2.91	3.33	3.92	4.37	1.65	0.90
		C	0.716	0.828	0.986	1.11	1.19	1.38	1.64	1.85	1.75	0.85
		D	2.12	2.30	2.56	2.76	3.53	3.84	4.27	4.60	1.85	0.80
	Unfastened	A	0.732	0.753	0.783	0.807	1.22	1.26	1.31	1.34	1.80	0.85
		B	1.75	2.00	2.35	2.62	2.91	3.33	3.92	4.37	1.65	0.90
		C	0.580	0.619	0.673	0.714	0.967	1.03	1.12	1.19	1.65	0.90
		D	0.894	1.03	1.22	1.36	1.49	1.71	2.03	2.27	1.90	0.80

Table II - 12 (continued)

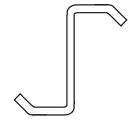
Web Crippling, P_n, kips [1]
Z-Sections With Lips



Section	Fastened or Unfastened	Case[2]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [3]				Bearing Length, N (in.) [3]					
			1	2	4	6	1	2	4	6		
10ZS2.75x105	Fastened	A	1.99	2.42	3.03	3.50	3.32	4.04	5.05	5.83	1.75	0.85
		B	4.25	4.78	5.53	6.10	7.08	7.96	9.21	10.2	1.65	0.90
		C	2.32	2.63	3.08	3.43	3.86	4.39	5.14	5.71	1.75	0.85
		D	5.98	6.42	7.04	7.52	9.97	10.7	11.7	12.5	1.85	0.80
	Unfastened	A	1.68	1.72	1.78	1.82	2.81	2.87	2.97	3.04	1.80	0.85
		B	4.25	4.78	5.53	6.10	7.08	7.96	9.21	10.2	1.65	0.90
		C	1.94	2.05	2.20	2.32	3.24	3.41	3.67	3.86	1.65	0.90
		D	3.86	4.37	5.08	5.63	6.43	7.28	8.47	9.39	1.90	0.80
10ZS2.75x085	Fastened	A	1.31	1.61	2.03	2.35	2.19	2.68	3.38	3.91	1.75	0.85
		B	2.70	3.07	3.58	3.97	4.50	5.11	5.96	6.62	1.65	0.90
		C	1.39	1.59	1.88	2.10	2.32	2.66	3.14	3.50	1.75	0.85
		D	3.68	3.97	4.39	4.71	6.13	6.62	7.31	7.85	1.85	0.80
	Unfastened	A	1.09	1.12	1.16	1.19	1.82	1.87	1.94	1.99	1.80	0.85
		B	2.70	3.07	3.58	3.97	4.50	5.11	5.96	6.62	1.65	0.90
		C	1.10	1.17	1.26	1.34	1.84	1.95	2.10	2.23	1.65	0.90
		D	1.95	2.23	2.62	2.91	3.25	3.71	4.36	4.86	1.90	0.80
10ZS2.75x070	Fastened	A	0.889	1.10	1.39	1.62	1.48	1.83	2.32	2.70	1.75	0.85
		B	1.77	2.02	2.38	2.66	2.95	3.37	3.97	4.43	1.65	0.90
		C	0.846	0.978	1.17	1.31	1.41	1.63	1.94	2.18	1.75	0.85
		D	2.32	2.52	2.81	3.03	3.87	4.21	4.68	5.04	1.85	0.80
	Unfastened	A	0.733	0.754	0.784	0.808	1.22	1.26	1.31	1.35	1.80	0.85
		B	1.77	2.02	2.38	2.66	2.95	3.37	3.97	4.43	1.65	0.90
		C	0.636	0.678	0.737	0.783	1.06	1.13	1.23	1.30	1.65	0.90
		D	0.895	1.03	1.22	1.36	1.49	1.72	2.03	2.27	1.90	0.80
10ZS2.75x065	Fastened	A	0.765	0.948	1.21	1.41	1.27	1.58	2.01	2.34	1.75	0.85
		B	1.50	1.73	2.04	2.28	2.51	2.88	3.40	3.80	1.65	0.90
		C	0.694	0.805	0.962	1.08	1.16	1.34	1.60	1.80	1.75	0.85
		D	1.94	2.11	2.36	2.55	3.24	3.52	3.93	4.25	1.85	0.80
	Unfastened	A	0.629	0.648	0.675	0.696	1.05	1.08	1.12	1.16	1.80	0.85
		B	1.50	1.73	2.04	2.28	2.51	2.88	3.40	3.80	1.65	0.90
		C	0.511	0.546	0.595	0.633	0.852	0.910	0.992	1.05	1.65	0.90
		D	0.613	0.708	0.841	0.943	1.02	1.18	1.40	1.57	1.90	0.80
10ZS2.75x059	Fastened	A	0.628	0.782	0.999	1.17	1.05	1.30	1.66	1.94	1.75	0.85
		B	1.21	1.40	1.66	1.86	2.02	2.33	2.76	3.09	1.65	0.90
		C	0.531	0.618	0.742	0.837	0.885	1.03	1.24	1.39	1.75	0.85
		D	1.53	1.67	1.87	2.03	2.55	2.79	3.12	3.38	1.85	0.80
	Unfastened	A	0.515	0.532	0.554	0.572	0.859	0.886	0.924	0.954	1.80	0.85
		B	1.21	1.40	1.66	1.86	2.02	2.33	2.76	3.09	1.65	0.90
		C	0.381	0.408	0.446	0.475	0.635	0.679	0.743	0.792	1.65	0.90
		D	0.321	0.372	0.444	0.499	0.536	0.620	0.740	0.832	1.90	0.80
10ZS2.25x105	Fastened	A	1.99	2.42	3.03	3.50	3.32	4.04	5.05	5.83	1.75	0.85
		B	4.25	4.78	5.53	6.10	7.08	7.96	9.21	10.2	1.65	0.90
		C	2.32	2.63	3.08	3.43	3.86	4.39	5.14	5.71	1.75	0.85
		D	5.98	6.42	7.04	7.52	9.97	10.7	11.7	12.5	1.85	0.80
	Unfastened	A	1.68	1.72	1.78	1.82	2.81	2.87	2.97	3.04	1.80	0.85
		B	4.25	4.78	5.53	6.10	7.08	7.96	9.21	10.2	1.65	0.90
		C	1.94	2.05	2.20	2.32	3.24	3.41	3.67	3.86	1.65	0.90
		D	3.86	4.37	5.08	5.63	6.43	7.28	8.47	9.39	1.90	0.80

Table II - 12 (continued)

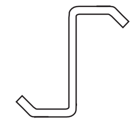
Web Crippling, P_n, kips [1]
Z-Sections With Lips



Section	Fastened or Unfastened	Case[2]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [3]				Bearing Length, N (in.) [3]					
			1	2	4	6	1	2	4	6		
10ZS2.25x085	Fastened	A	1.31	1.61	2.03	2.35	2.19	2.68	3.38	3.91	1.75	0.85
		B	2.70	3.07	3.58	3.97	4.50	5.11	5.96	6.62	1.65	0.90
		C	1.39	1.59	1.88	2.10	2.32	2.66	3.14	3.50	1.75	0.85
		D	3.68	3.97	4.39	4.71	6.13	6.62	7.31	7.85	1.85	0.80
	Unfastened	A	1.09	1.12	1.16	1.19	1.82	1.87	1.94	1.99	1.80	0.85
		B	2.70	3.07	3.58	3.97	4.50	5.11	5.96	6.62	1.65	0.90
		C	1.10	1.17	1.26	1.34	1.84	1.95	2.10	2.23	1.65	0.90
		D	1.95	2.23	2.62	2.91	3.25	3.71	4.36	4.86	1.90	0.80
10ZS2.25x070	Fastened	A	0.889	1.10	1.39	1.62	1.48	1.83	2.32	2.70	1.75	0.85
		B	1.77	2.02	2.38	2.66	2.95	3.37	3.97	4.43	1.65	0.90
		C	0.846	0.978	1.17	1.31	1.41	1.63	1.94	2.18	1.75	0.85
		D	2.32	2.52	2.81	3.03	3.87	4.21	4.68	5.04	1.85	0.80
	Unfastened	A	0.733	0.754	0.784	0.808	1.22	1.26	1.31	1.35	1.80	0.85
		B	1.77	2.02	2.38	2.66	2.95	3.37	3.97	4.43	1.65	0.90
		C	0.636	0.678	0.737	0.783	1.06	1.13	1.23	1.30	1.65	0.90
		D	0.895	1.03	1.22	1.36	1.49	1.72	2.03	2.27	1.90	0.80
10ZS2.25x065	Fastened	A	0.765	0.948	1.21	1.41	1.27	1.58	2.01	2.34	1.75	0.85
		B	1.50	1.73	2.04	2.28	2.51	2.88	3.40	3.80	1.65	0.90
		C	0.694	0.805	0.962	1.08	1.16	1.34	1.60	1.80	1.75	0.85
		D	1.94	2.11	2.36	2.55	3.24	3.52	3.93	4.25	1.85	0.80
	Unfastened	A	0.629	0.648	0.675	0.696	1.05	1.08	1.12	1.16	1.80	0.85
		B	1.50	1.73	2.04	2.28	2.51	2.88	3.40	3.80	1.65	0.90
		C	0.511	0.546	0.595	0.633	0.852	0.910	0.992	1.05	1.65	0.90
		D	0.613	0.708	0.841	0.943	1.02	1.18	1.40	1.57	1.90	0.80
10ZS2.25x059	Fastened	A	0.628	0.782	0.999	1.17	1.05	1.30	1.66	1.94	1.75	0.85
		B	1.21	1.40	1.66	1.86	2.02	2.33	2.76	3.09	1.65	0.90
		C	0.531	0.618	0.742	0.837	0.885	1.03	1.24	1.39	1.75	0.85
		D	1.53	1.67	1.87	2.03	2.55	2.79	3.12	3.38	1.85	0.80
	Unfastened	A	0.515	0.532	0.554	0.572	0.859	0.886	0.924	0.954	1.80	0.85
		B	1.21	1.40	1.66	1.86	2.02	2.33	2.76	3.09	1.65	0.90
		C	0.381	0.408	0.446	0.475	0.635	0.679	0.743	0.792	1.65	0.90
		D	0.321	0.372	0.444	0.499	0.536	0.620	0.740	0.832	1.90	0.80
8ZS2.75x105	Fastened	A	2.05	2.49	3.11	3.59	3.41	4.15	5.18	5.98	1.75	0.85
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	2.57	2.92	3.42	3.80	4.28	4.87	5.70	6.34	1.75	0.85
		D	6.39	6.86	7.52	8.03	10.6	11.4	12.5	13.4	1.85	0.80
	Unfastened	A	1.68	1.73	1.78	1.83	2.81	2.88	2.97	3.04	1.80	0.85
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	2.07	2.19	2.35	2.48	3.46	3.65	3.92	4.13	1.65	0.90
		D	3.86	4.37	5.09	5.64	6.44	7.29	8.48	9.40	1.90	0.80
8ZS2.75x085	Fastened	A	1.35	1.66	2.09	2.42	2.25	2.76	3.48	4.03	1.75	0.85
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.58	1.81	2.14	2.39	2.63	3.02	3.56	3.98	1.75	0.85
		D	3.98	4.30	4.75	5.09	6.63	7.16	7.91	8.49	1.85	0.80
	Unfastened	A	1.09	1.12	1.16	1.20	1.82	1.87	1.94	1.99	1.80	0.85
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.19	1.26	1.37	1.44	1.99	2.11	2.28	2.41	1.65	0.90
		D	1.95	2.23	2.62	2.92	3.26	3.72	4.36	4.86	1.90	0.80

Table II - 12 (continued)

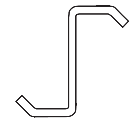
Web Crippling, P_n, kips [1]
Z-Sections With Lips



Section	Fastened or Unfastened	Case[2]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [3]				Bearing Length, N (in.) [3]					
			1	2	4	6	1	2	4	6		
8ZS2.75x070	Fastened	A	0.919	1.14	1.44	1.68	1.53	1.89	2.40	2.79	1.75	0.85
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.991	1.15	1.36	1.53	1.65	1.91	2.27	2.55	1.75	0.85
		D	2.55	2.77	3.08	3.32	4.25	4.61	5.14	5.53	1.85	0.80
	Unfastened	A	0.734	0.755	0.785	0.809	1.22	1.26	1.31	1.35	1.80	0.85
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.698	0.744	0.809	0.859	1.16	1.24	1.35	1.43	1.65	0.90
		D	0.896	1.03	1.22	1.37	1.49	1.72	2.03	2.28	1.90	0.80
8ZS2.75x065	Fastened	A	0.792	0.982	1.25	1.46	1.32	1.64	2.08	2.43	1.75	0.85
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.825	0.957	1.14	1.29	1.38	1.60	1.91	2.14	1.75	0.85
		D	2.14	2.34	2.61	2.81	3.57	3.89	4.34	4.69	1.85	0.80
	Unfastened	A	0.630	0.649	0.676	0.696	1.05	1.08	1.13	1.16	1.80	0.85
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.565	0.603	0.657	0.699	0.941	1.00	1.10	1.16	1.65	0.90
		D	0.614	0.708	0.842	0.944	1.02	1.18	1.40	1.57	1.90	0.80
8ZS2.75x059	Fastened	A	0.652	0.811	1.04	1.21	1.09	1.35	1.73	2.02	1.75	0.85
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.646	0.752	0.902	1.02	1.08	1.25	1.50	1.70	1.75	0.85
		D	1.71	1.86	2.09	2.26	2.84	3.11	3.48	3.76	1.85	0.80
	Unfastened	A	0.516	0.532	0.555	0.573	0.860	0.887	0.925	0.955	1.80	0.85
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.424	0.455	0.497	0.530	0.707	0.758	0.828	0.883	1.65	0.90
		D	0.322	0.373	0.445	0.500	0.536	0.621	0.741	0.833	1.90	0.80
8ZS2.25x105	Fastened	A	2.05	2.49	3.11	3.59	3.41	4.15	5.18	5.98	1.75	0.85
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	2.57	2.92	3.42	3.80	4.28	4.87	5.70	6.34	1.75	0.85
		D	6.39	6.86	7.52	8.03	10.6	11.4	12.5	13.4	1.85	0.80
	Unfastened	A	1.68	1.73	1.78	1.83	2.81	2.88	2.97	3.04	1.80	0.85
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	2.07	2.19	2.35	2.48	3.46	3.65	3.92	4.13	1.65	0.90
		D	3.86	4.37	5.09	5.64	6.44	7.29	8.48	9.40	1.90	0.80
8ZS2.25x085	Fastened	A	1.35	1.66	2.09	2.42	2.25	2.76	3.48	4.03	1.75	0.85
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.58	1.81	2.14	2.39	2.63	3.02	3.56	3.98	1.75	0.85
		D	3.98	4.30	4.75	5.09	6.63	7.16	7.91	8.49	1.85	0.80
	Unfastened	A	1.09	1.12	1.16	1.20	1.82	1.87	1.94	1.99	1.80	0.85
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.19	1.26	1.37	1.44	1.99	2.11	2.28	2.41	1.65	0.90
		D	1.95	2.23	2.62	2.92	3.26	3.72	4.36	4.86	1.90	0.80
8ZS2.25x070	Fastened	A	0.919	1.14	1.44	1.68	1.53	1.89	2.40	2.79	1.75	0.85
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.991	1.15	1.36	1.53	1.65	1.91	2.27	2.55	1.75	0.85
		D	2.55	2.77	3.08	3.32	4.25	4.61	5.14	5.53	1.85	0.80
	Unfastened	A	0.734	0.755	0.785	0.809	1.22	1.26	1.31	1.35	1.80	0.85
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.698	0.744	0.809	0.859	1.16	1.24	1.35	1.43	1.65	0.90
		D	0.896	1.03	1.22	1.37	1.49	1.72	2.03	2.28	1.90	0.80

Table II - 12 (continued)

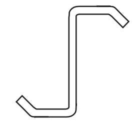
Web Crippling, P_n, kips [1]
Z-Sections With Lips



Section	Fastened or Unfastened	Case[2]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [3]				Bearing Length, N (in.) [3]					
			1	2	4	6	1	2	4	6		
8ZS2.25x065	Fastened	A	0.792	0.982	1.25	1.46	1.32	1.64	2.08	2.43	1.75	0.85
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.825	0.957	1.14	1.29	1.38	1.60	1.91	2.14	1.75	0.85
		D	2.14	2.34	2.61	2.81	3.57	3.89	4.34	4.69	1.85	0.80
	Unfastened	A	0.630	0.649	0.676	0.696	1.05	1.08	1.13	1.16	1.80	0.85
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.565	0.603	0.657	0.699	0.941	1.00	1.10	1.16	1.65	0.90
		D	0.614	0.708	0.842	0.944	1.02	1.18	1.40	1.57	1.90	0.80
8ZS2.25x059	Fastened	A	0.652	0.811	1.04	1.21	1.09	1.35	1.73	2.02	1.75	0.85
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.646	0.752	0.902	1.02	1.08	1.25	1.50	1.70	1.75	0.85
		D	1.71	1.86	2.09	2.26	2.84	3.11	3.48	3.76	1.85	0.80
	Unfastened	A	0.516	0.532	0.555	0.573	0.860	0.887	0.925	0.955	1.80	0.85
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.424	0.455	0.497	0.530	0.707	0.758	0.828	0.883	1.65	0.90
		D	0.322	0.373	0.445	0.500	0.536	0.621	0.741	0.833	1.90	0.80
6ZS2.25x105	Fastened	A	2.11	2.56	3.20	3.69	3.51	4.27	5.34	6.16	1.75	0.85
		B	4.35	4.90	5.67	6.26	7.26	8.16	9.45	10.4	1.65	0.90
		C	2.86	3.25	3.81	4.23	4.77	5.42	6.34	7.05	1.75	0.85
		D	6.86	7.36	8.08	8.63	11.4	12.3	13.5	14.4	1.85	0.80
	Unfastened	A	1.69	1.73	1.78	1.83	2.81	2.88	2.97	3.05	1.80	0.85
		B	4.35	4.90	5.67	6.26	7.26	8.16	9.45	10.4	1.65	0.90
		C	2.23	2.35	2.53	2.66	3.71	3.92	4.21	4.43	1.65	0.90
		D	3.87	4.38	5.09	5.64	6.45	7.29	8.49	9.41	1.90	0.80
6ZS2.25x085	Fastened	A	1.40	1.71	2.16	2.50	2.33	2.85	3.60	4.17	1.75	0.85
		B	2.78	3.15	3.68	4.09	4.63	5.25	6.13	6.81	1.65	0.90
		C	1.79	2.06	2.43	2.72	2.99	3.43	4.05	4.53	1.75	0.85
		D	4.32	4.67	5.16	5.53	7.20	7.78	8.60	9.22	1.85	0.80
	Unfastened	A	1.09	1.12	1.17	1.20	1.82	1.87	1.94	1.99	1.80	0.85
		B	2.78	3.15	3.68	4.09	4.63	5.25	6.13	6.81	1.65	0.90
		C	1.29	1.37	1.48	1.57	2.16	2.29	2.47	2.62	1.65	0.90
		D	1.96	2.23	2.62	2.92	3.26	3.72	4.37	4.87	1.90	0.80
6ZS2.25x070	Fastened	A	0.953	1.18	1.50	1.74	1.59	1.96	2.49	2.90	1.75	0.85
		B	1.83	2.09	2.46	2.74	3.05	3.48	4.10	4.57	1.65	0.90
		C	1.16	1.34	1.59	1.79	1.93	2.23	2.66	2.98	1.75	0.85
		D	2.81	3.05	3.39	3.66	4.68	5.08	5.66	6.10	1.85	0.80
	Unfastened	A	0.735	0.756	0.787	0.810	1.22	1.26	1.31	1.35	1.80	0.85
		B	1.83	2.09	2.46	2.74	3.05	3.48	4.10	4.57	1.65	0.90
		C	0.769	0.820	0.891	0.946	1.28	1.37	1.49	1.58	1.65	0.90
		D	0.898	1.03	1.22	1.37	1.50	1.72	2.04	2.28	1.90	0.80
6ZS2.25x065	Fastened	A	0.823	1.02	1.30	1.51	1.37	1.70	2.17	2.52	1.75	0.85
		B	1.55	1.78	2.10	2.35	2.59	2.97	3.51	3.92	1.65	0.90
		C	0.975	1.13	1.35	1.52	1.63	1.89	2.25	2.53	1.75	0.85
		D	2.38	2.59	2.89	3.12	3.96	4.31	4.81	5.20	1.85	0.80
	Unfastened	A	0.631	0.650	0.677	0.698	1.05	1.08	1.13	1.16	1.80	0.85
		B	1.55	1.78	2.10	2.35	2.59	2.97	3.51	3.92	1.65	0.90
		C	0.626	0.668	0.728	0.774	1.04	1.11	1.21	1.29	1.65	0.90
		D	0.615	0.710	0.843	0.946	1.03	1.18	1.41	1.58	1.90	0.80

Table II - 12 (continued)

Web Crippling, P_n, kips [1]
Z-Sections With Lips



Section	Fastened or Unfastened	Case[2]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N (in.) [3]				Bearing Length, N (in.) [3]					
			1	2	4	6	1	2	4	6		
6ZS2.25x059	Fastened	A	0.679	0.845	1.08	1.26	1.13	1.41	1.80	2.10	1.75	0.85
		B	1.25	1.44	1.71	1.92	2.09	2.41	2.86	3.20	1.65	0.90
		C	0.777	0.905	1.09	1.22	1.30	1.51	1.81	2.04	1.75	0.85
		D	1.91	2.08	2.33	2.53	3.18	3.47	3.89	4.21	1.85	0.80
	Unfastened	A	0.517	0.533	0.556	0.574	0.861	0.889	0.927	0.956	1.80	0.85
		B	1.25	1.44	1.71	1.92	2.09	2.41	2.86	3.20	1.65	0.90
		C	0.475	0.508	0.556	0.592	0.791	0.847	0.926	0.987	1.65	0.90
		D	0.322	0.373	0.446	0.501	0.537	0.622	0.743	0.835	1.90	0.80
4ZS2.25x070	Fastened	A	0.995	1.23	1.56	1.82	1.66	2.05	2.60	3.03	1.75	0.85
		B	1.86	2.13	2.51	2.80	3.11	3.55	4.18	4.66	1.65	0.90
		C	1.36	1.57	1.87	2.10	2.26	2.62	3.12	3.50	1.75	0.85
		D	3.12	3.39	3.77	4.06	5.20	5.65	6.29	6.77	1.85	0.80
	Unfastened	A	0.736	0.758	0.788	0.811	1.23	1.26	1.31	1.35	1.80	0.85
		B	1.86	2.13	2.51	2.80	3.11	3.55	4.18	4.66	1.65	0.90
		C	0.854	0.911	0.990	1.05	1.42	1.52	1.65	1.75	1.65	0.90
		D	0.899	1.03	1.22	1.37	1.50	1.72	2.04	2.29	1.90	0.80
4ZS2.25x065	Fastened	A	0.861	1.07	1.36	1.58	1.43	1.78	2.26	2.64	1.75	0.85
		B	1.59	1.82	2.15	2.40	2.64	3.03	3.58	4.00	1.65	0.90
		C	1.16	1.34	1.60	1.80	1.93	2.23	2.67	3.00	1.75	0.85
		D	2.66	2.89	3.23	3.48	4.43	4.82	5.38	5.81	1.85	0.80
	Unfastened	A	0.632	0.651	0.678	0.699	1.05	1.09	1.13	1.16	1.80	0.85
		B	1.59	1.82	2.15	2.40	2.64	3.03	3.58	4.00	1.65	0.90
		C	0.699	0.747	0.814	0.866	1.17	1.24	1.36	1.44	1.65	0.90
		D	0.616	0.711	0.845	0.947	1.03	1.18	1.41	1.58	1.90	0.80
4ZS2.25x059	Fastened	A	0.712	0.886	1.13	1.32	1.19	1.48	1.89	2.20	1.75	0.85
		B	1.28	1.48	1.75	1.96	2.14	2.46	2.92	3.27	1.65	0.90
		C	0.936	1.09	1.31	1.47	1.56	1.82	2.18	2.46	1.75	0.85
		D	2.15	2.35	2.63	2.85	3.58	3.91	4.38	4.74	1.85	0.80
	Unfastened	A	0.518	0.534	0.557	0.575	0.863	0.890	0.929	0.958	1.80	0.85
		B	1.28	1.48	1.75	1.96	2.14	2.46	2.92	3.27	1.65	0.90
		C	0.535	0.573	0.626	0.667	0.892	0.955	1.04	1.11	1.65	0.90
		D	0.323	0.374	0.446	0.502	0.538	0.624	0.744	0.836	1.90	0.80

Notes:

1. Web crippling strengths are nominal strengths calculated without consideration of holes or other openings. To obtain design strengths, the values must be modified by factors of safety (ASD) or resistance factors (LRFD).
2. Case A
   End Reaction, Opposing Loads Spaced > 1.5h
   Case B
   Interior Reaction, Opposing Loads Spaced > 1.5h
   Case C
   End Reaction, Opposing Loads Spaced ≤ 1.5h
   Case D
   Interior Reaction, Opposing Loads Spaced ≤ 1.5h
3. Linear interpolation is permitted between bearing lengths.

Table II-13a

Web Crippling Reduction Factor [1,2], R_c, for Interior Loading
SSMA Studs
C-Sections with Lips

Loading Condition	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 apply only to openings with the dimensions listed.
2. Linear interpolation of R_c values between distances is permitted.

Table II-13b

Web Crippling Reduction Factor [1,2], R_c, for End Loading
SSMA Studs
C-Sections with Lips

Loading Condition	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 apply only to openings with the dimensions listed.
2. Linear interpolation of R_c values between distances is permitted.

Example II-1: Four Span Continuous C-Purlins Attached to Through Fastened Roof[1] - LRFD

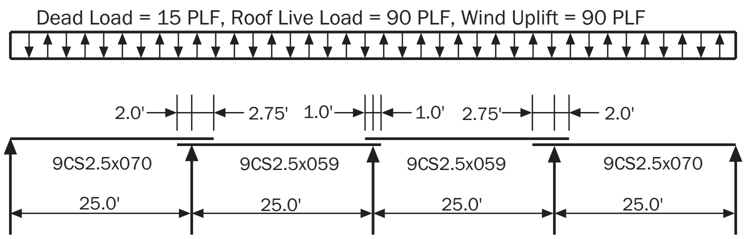


Figure 1 - Spans and Loading
Note: Lap dimensions are to connection points of purlins

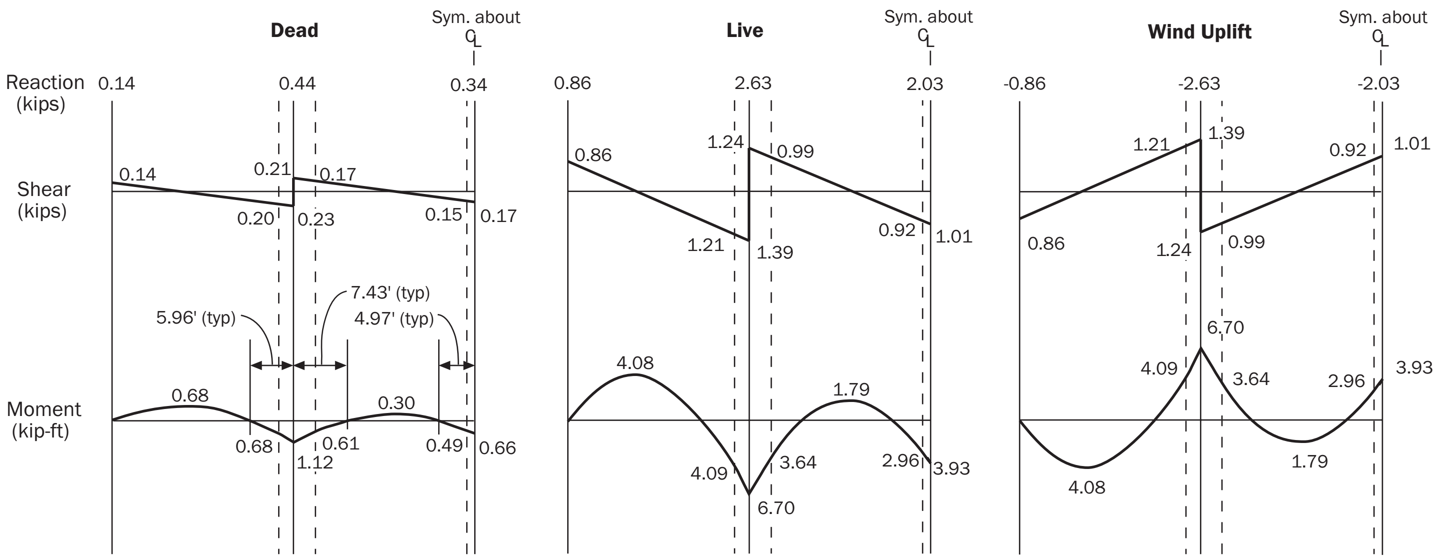


Figure 2 - Reactions, Shears and Moments
Note: Moments and forces are from unfactored nominal loads

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. Roof Slope = 0.25/12
5. Purlins are lapped back to back over supports but all face in the same direction in a given bay.
6. Discrete anchorage devices are at supports and attached to the supporting structure at every fifth purlin line.
7. Bottom flanges are bolted to supporting members with a bearing length of 5 in.

Required:

1. Check the design using LRFD with ASCE 7-98 load combinations for:
   a. Gravity Loads
   b. Uplift Loads

[1] For design of purlins supporting standing seam roof, see AISI publication *A Design Guide for Designing with Standing Seam Roof Panels.*

2. Compute the bracing anchorage forces under gravity loading.

Solution:

1. Assumptions for Analysis and Application of *Specification* Provisions

The *Specification* does not define the methods of analysis to be used; these judgements 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. It is assumed in the continuous beam analysis that 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. It is assumed that the attachment of the roof covering to the purlin provides continuous lateral and torsional support to the top flange.

e. For gravity loads, the compression (bottom) flange at and near the interior supports is assumed to be fully braced between the support and the end of the lap.

f. Under uniform gravity loading, the negative moment region between the end of the lap and the inflection point is assumed to be braced at the end of the lap and at the inflection point.

g. 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.)	9.0	9.0
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 ($in.^3$)	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)(4.08) = 7.34 kip-ft
Negative moment at end of right lap:	M_u = (1.2)(0.68) + (1.6)(4.09) = 7.36 kip-ft
Negative moment at support:	M_u = (1.2)(1.12) + (1.6)(6.70) = 12.1 kip-ft

Interior span, from left to right:

Negative moment at end of left lap:	M_u = (1.2)(0.61) + (1.6)(3.64) = 6.56 kip-ft
Maximum positive moment:	M_u = (1.2)(0.30) + (1.6)(1.79) = 3.22 kip-ft
Negative moment at end of right lap:	M_u = (1.2)(0.49) + (1.6)(2.96) = 5.32 kip-ft
Negative moment at center support:	M_u = (1.2)(0.66) + (1.6)(3.93) = 7.08 kip-ft

Design Strength

End Span:

At location of maximum positive moment, the section is assumed to be fully braced:
Calculate nominal 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 kip-in. = 11.3 kip-ft (*Eq.* C3.1.1-1)

$\phi_b M_n$ = (0.95)(11.3) = 10.7 kip-ft > 7.34 kip-ft OK (*Eq.* A5.1.1-1)

In the region of negative moment between the lap and the inflection point:

Determine the nominal strength using the distance from the inflection point to the lap as the unbraced length using Section C3.1.2.1(a).

$$F_e = \frac{C_b r_o A}{S_f}\sqrt{\sigma_{ey}\sigma_t} \quad (Eq.\ C3.1.2.1\text{-}5)$$

$L_y = L_t$ = 5.96 - 2.00 = 3.96 ft = 47.5 in.

C_b = 1.67 (Conservatively assumes linear moment diagram in this region.)

$$\sigma_{ey} = \frac{\pi^2 E}{\left(K_y L_y / r_y\right)^2} \quad (Eq.\ C3.1.2.1\text{-}8)$$

$$= \frac{\pi^2\, 29500}{\left[(1.0)(47.5)/0.890\right]^2} = 102 \text{ ksi}$$

$$\sigma_t = \frac{1}{A r_o^2}\left[GJ + \frac{\pi^2 E C_w}{\left(K_t L_t\right)^2}\right] \quad (Eq.\ C3.1.2.1\text{-}9)$$

$$= \frac{1}{1.05(3.90)^2}\left[(11300)(0.00171) + \frac{\pi^2(29500)(14.2)}{\left[(1.0)(47.5)\right]^2}\right] = 116 \text{ ksi}$$

$F_e = \dfrac{(1.67)(3.90)(1.05)}{2.71}\sqrt{(102)(116)} = 274$ kip-in. (*Eq.* C3.1.2.1-5)

$2.78F_y = (2.78)(55) = 153$ ksi

Since $F_e > 2.78\ F_y$, the section is not subject to lateral-torsional buckling and the strength is calculated using Section C3.1.1(a).

$M_n = S_eF_y = (2.47)(55) = 136$ kip-in. or 11.3 kip-ft (*Eq.* C3.1.1-1)

$\phi_bM_n = (0.95)(11.3) = 10.7$ kip-ft > 7.36 kip-ft OK (*Eq.* A5.1.1-1)

At the negative moment at the support, the section is assumed to be fully braced:
Use nominal strength based on initiation of yielding per Section C3.1.1(a), summing the strength of the two overlapped purlins:

For the interior purlin, t = 0.059 in.

$M_n = S_e F_y = (1.89)(55) = 104$ kip-in. or 8.66 kip-ft (*Eq.* C3.1.1-1)

Combined strength of purlins

$\phi_bM_n = (0.95)(11.3 + 8.66) = 19.0$ kip-ft > 12.1 kip-ft OK (*Eq.* A5.1.1-1)

Interior Span:

In the region of negative moment between the left lap and the inflection point:

Determine the nominal strength using the distance from the inflection point to the lap as the unbraced length using Section C3.1.2.1(a).

$L_y = L_t = 7.43 - 2.75 = 4.68$ ft or 56.2 in.

$C_b = 1.67$ (Conservatively assumes linear moment diagram in this region.)

$\sigma_{ey} = \dfrac{\pi^2\ 29500}{\left[(1.0)(56.2)/0.890\right]^2} = 73.0$ ksi (*Eq.* C3.1.2.1-8)

$\sigma_t = \dfrac{1}{0.881(3.90)^2}\left[(11300)(0.00102) + \dfrac{\pi^2(29500)(11.9)}{[(1.0)(56.2)]^2}\right] = 82.7$ ksi (*Eq.* C3.1.2.1-9)

$F_e = \dfrac{(1.67)(3.90)(0.881)}{2.29}\sqrt{(73.0)(82.7)} = 195$ ksi (*Eq.* C3.1.2.1-5)

$2.78F_y = (2.78)(55) = 153$ ksi

Since $F_e > 2.78\ F_y$, the section is not subject to lateral-torsional buckling and the strength is calculated using Section C3.1.1(a).

$M_n = S_eF_y = (1.89)(55) = 104$ kip-in or 8.66 kip-ft (*Eq.* C3.1.1.1-1)

$\phi_bM_n = (0.95)(8.66) = 8.23$ kip-ft > 6.56 kip-ft OK (*Eq.* A5.1.1-1)

At the location of maximum positive moment, the section is assumed to be fully braced:
Use nominal strength based on initiation of yielding per Section C3.1.1(a) calculated above.

$\phi_bM_n = (0.95)(8.66) = 8.23$ kip-ft > 3.22 kip-ft OK (*Eq.* A5.1.1-1)

In the region of negative moment between the right lap and the inflection point:

Determine the nominal strength using the distance from the inflection point to the lap as the unbraced length using Section C3.1.2.1(a).

$L = 4.97 - 1.00 = 3.97$ ft or 47.6 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.

At the negative moment at the center support, the section is assumed to be fully braced:
Use nominal strength based on initiation of yielding per Section C3.1.1(a), summing the strength of the two overlapped purlins:

Combined strength of purlins

$\phi_b M_n$ = (0.95)(8.66 + 8.66) = 16.5 kip-ft > 7.08 kip-ft OK (*Eq.* A5.1.1-1)

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)(1.24) = 2.24 kips
At end of left lap:	V_u = (1.2)(0.17) + (1.6)(0.99) = 1.79 kips
At end of right lap:	V_u = (1.2)(0.15) + (1.6)(0.92) = 1.65 kips
At center support:	V_u = (1.2)(0.17) + (1.6)(1.01) = 1.82 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. and h = 8.485 in., h/t = 121

$$1.51\sqrt{Ek_v/F_y} = 1.51\sqrt{(29500)(5.34)/55} = 80.8$$

For $\frac{h}{t} > 1.51\sqrt{E\,k_v/F_y}$:

$$F_v = \frac{\pi^2 E\,k_v}{12(1-\mu^2)(h/t)^2} \quad (Eq.\ C3.2.1\text{-}4)$$

$$= \frac{\pi^2(29500)(5.34)}{12(1-0.3^2)(8.485/0.070)^2} = 9.69\text{ ksi}$$

$$V_n = A_w F_v \quad (Eq.\ C3.2.1\text{-}1)$$

$$= (8.485)(0.070)(9.69) = 5.76\text{ kips}$$

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 kip > 2.18 kips OK (*Eq.* A5.1.1-1)

At the first interior support, sum the strength of the two overlapped purlins:

For t = 0.059 in. and h = 8.507 in., h/t = 144 > 80.8

$$F_v = \frac{\pi^2(29500)(5.34)}{12(1-0.3^2)(8.507/0.059)^2} = 6.85\text{ ksi} \quad (Eq.\ C3.2.1\text{-}4)$$

$$V_n = A_w F_v \quad (Eq.\ C3.2.1\text{-}1)$$

$$= (8.507)(0.059)(6.85) = 3.44\text{ kips}$$

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 section:

$\phi_v V_n$ = (0.95)(5.76 + 3.44) = 8.74 kips > 2.50 kips OK (*Eq*. A5.1.1-1)

Interior span:

At the first interior support, use the strength computed above:

$\phi_v V_n$ = 8.74 kip > 2.24 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 > 1.79 kips OK (*Eq*. A5.1.1-1)

At the center support, sum the strength of the two overlapped purlins:

For the combined section:

$\phi_v V_n$ = (0.95)(3.44 + 3.44) = 6.54 kips > 1.82 kips OK (*Eq*. A5.1.1-1)

c. Strength for Combined Bending and Shear (Section C3.3.2)

End Span:

$$\left(\frac{\overline{M}}{\phi_b M_{nxo}}\right)^2 + \left(\frac{\overline{V}}{\phi_v V_n}\right)^2 \leq 1.0 \qquad (Eq.\ C3.3.2\text{-}1)$$

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

At start of lap, t = 0.070 in.

$$\left(\frac{7.36}{(0.95)(11.3)}\right)^2 + \left(\frac{2.18}{(0.95)(5.76)}\right)^2 = 0.63 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.2\text{-}1)$$

At first interior support,

$$\left(\frac{12.1}{(0.95)(11.3 + 8.66)}\right)^2 + \left(\frac{2.50}{(0.95)(5.76 + 3.44)}\right)^2 = 0.49 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.2\text{-}1)$$

Interior Span:

At end of laps, t = 0.059 in. Left lap controls by inspection.

$$\left(\frac{6.56}{(0.95)(8.66)}\right)^2 + \left(\frac{1.79}{(0.95)(3.44)}\right)^2 = 0.94 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.2\text{-}1)$$

At center support,

$$\left(\frac{7.08}{(0.95)(8.66 + 8.66)}\right)^2 + \left(\frac{1.82}{(0.95)(3.44 + 3.44)}\right)^2 = 0.26 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.2\text{-}1)$$

d. Web Crippling Strength (Section C3.4)

Required Strength

By inspection, the load combination D + L_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

$$P_n = (4)(0.070)^2(55)\sin(90)\left(1 - 0.14\sqrt{\frac{0.1875}{0.070}}\right)\left(1 + 0.35\sqrt{\frac{5.0}{0.070}}\right)\left(1 - 0.02\sqrt{\frac{8.485}{0.070}}\right)$$

$= 2.57$ kips (*Eq.* C3.4.1-1)

$\phi_w P_n = (0.85)(2.57) = 2.19$ kips > 1.54 kips OK (*Eq.* A5.1.1-1)

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

$$P_n = (13)(0.070)^2(55)\sin(90)\left(1 - 0.23\sqrt{\frac{0.1875}{0.070}}\right)\left(1 + 0.14\sqrt{\frac{5.0}{0.070}}\right)\left(1 - 0.01\sqrt{\frac{8.485}{0.070}}\right)$$

$= 4.25$ kips (*Eq.* C3.4.1-1)

for t = 0.059 in.:

Check Limit: R/t = 0.1875 / 0.059 = 3.2 < 5 OK

$$P_n = (13)(0.059)^2(55)\sin(90)\left(1 - 0.23\sqrt{\frac{0.1875}{0.059}}\right)\left(1 + 0.14\sqrt{\frac{5.0}{0.059}}\right)\left(1 - 0.01\sqrt{\frac{8.507}{0.059}}\right)$$

$$= 2.96 \text{ kips} \qquad (Eq.\ C3.4.1\text{-}1)$$

At first interior support,

$$\phi_w P_n = (0.90)(4.25 + 2.96) = 6.49 \text{ kips} > 4.74 \text{ kips OK} \qquad (Eq.\ A5.1.1\text{-}1)$$

At center support,

$$\phi_w P_n = (0.90)(2.96 + 2.96) = 5.33 \text{ kips} > 3.66 \text{ kips OK} \qquad (Eq.\ A5.1.1\text{-}1)$$

e. Combined Bending and Web Crippling (Section C3.5.2(a))

$$1.07\left(\frac{\overline{P}}{\phi_w P_n}\right) + \left(\frac{\overline{M}}{\phi_b M_{nxo}}\right) \le 1.42 \qquad (Eq.\ C3.5.2\text{-}1)$$

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

ϕ_w = 0.90

ϕ_b = 0.95

At the first interior support,

$$1.07\left(\frac{4.74}{(0.90)(4.25 + 2.96)}\right) + \left(\frac{12.1}{(0.95)(11.3 + 8.66)}\right) = 1.42 \le 1.42 \text{ OK} \qquad (Eq.\ C3.5.2\text{-}1)$$

At the center support,

$$1.07\left(\frac{3.66}{(0.90)(2.96 + 2.96)}\right) + \left(\frac{7.08}{(0.95)(8.66 + 8.66)}\right) = 1.17 < 1.42 \text{ OK} \qquad (Eq.\ C3.5.2\text{-}1)$$

4. Check Uplift Loads

a. Strength for Bending Only (Section C3.1.3)

Required Strength

By inspection, load combination 0.9D + 1.6W controls.

$M_u = 0.9M_D + 1.6M_W$

End Span:

Moment near center of span: $M_u = (0.9)(0.68) + (1.6)(-4.08) = -5.92$ kip-ft

Interior Span:

Moment near center of span: $M_u = (0.9)(0.30) + (1.6)(-1.79) = -2.59$ kip-ft

Design Strength

$M_n = R\, S_e\, F_y$ (*Eq.* C3.1.3-1)

R = 0.60, assuming all 15 conditions of Section C3.1.3 are satisfied

End Span:

For t = 0.070 in.

M_n = (0.60)(2.47)(55) = 81.5 kip-in. or 6.79 kip-ft (*Eq.* C3.1.3-1)

$\phi_b M_n$ = (0.90)(6.79) = 6.11 kip-ft > 5.92 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. or 5.20 kip-ft (*Eq.* C3.1.3-1)

$\phi_b M_n$ = (0.90)(5.20) = 4.68 kip-ft > 2.59 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 that the design satisfies the *Specification* criteria for uplift.

5. Anchorage Forces

Compute the anchorage forces at the supports using Section D3.2.1 with one anchorage device at each frame line at every fifth purlin line. For each bay:

For Case (a) C-Sections

$P_L = (0.05\alpha\cos\theta - \sin\theta)W$ (*Eq.* D3.2.1-1)

α = +1 for purlins facing upward direction (lips up slope from web)

α = -1 for purlins facing downward direction (lips down slope from web)

$$\theta = \tan^{-1}\left(\frac{0.25}{12}\right) = 1.19°$$

$$W = W_u = n(L)(1.2D + 1.6L_r) = (5)(25)\left[(1.2)(15) + (1.6)(90)\right] = 20{,}250 \text{ lbs}$$

For purlins facing upward direction:

$P_L = \left(0.05(1)\cos(1.19°) - \sin(1.19°)\right)(20250) = 592$ lbs (up slope) (*Eq.* D3.2.1-1)

For purlins facing downward direction:

$P_L = \left(0.05(-1)\cos(1.19°) - \sin(1.19°)\right)(20250) = -1433$ lbs (down slope) (*Eq.* D3.2.1-1)

At interior frames, the anchorage forces from one half bay of purlins on each side of the frame line contribute to the anchorage force. The forces from alternating purlin directions in adjacent bays offset each other:

P_L = 0.5(592 - 1433) = -421 lbs (down slope)

At end frames the anchorage force is one half of the force for a whole bay and depends upon the purlin direction :

For end bays with purlins facing upward direction:

P_L = 592/2 = 296 lbs (up slope)

For end bays with purlins facing downward direction:

P_L = -1433/2 = -717 lbs (down slope)

Example II-2: Four Span Continuous Z-Purlins Attached to Through Fastened Roof[1] - ASD

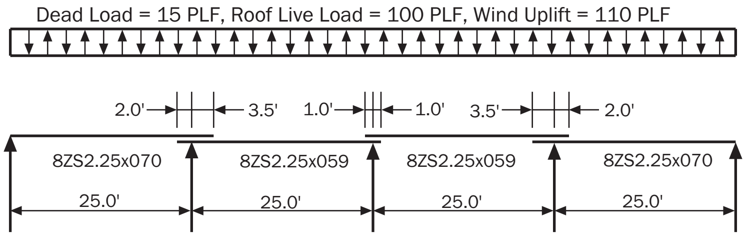


Figure 1 - Spans and Loading

Note: Lap dimensions are to connection points of purlins

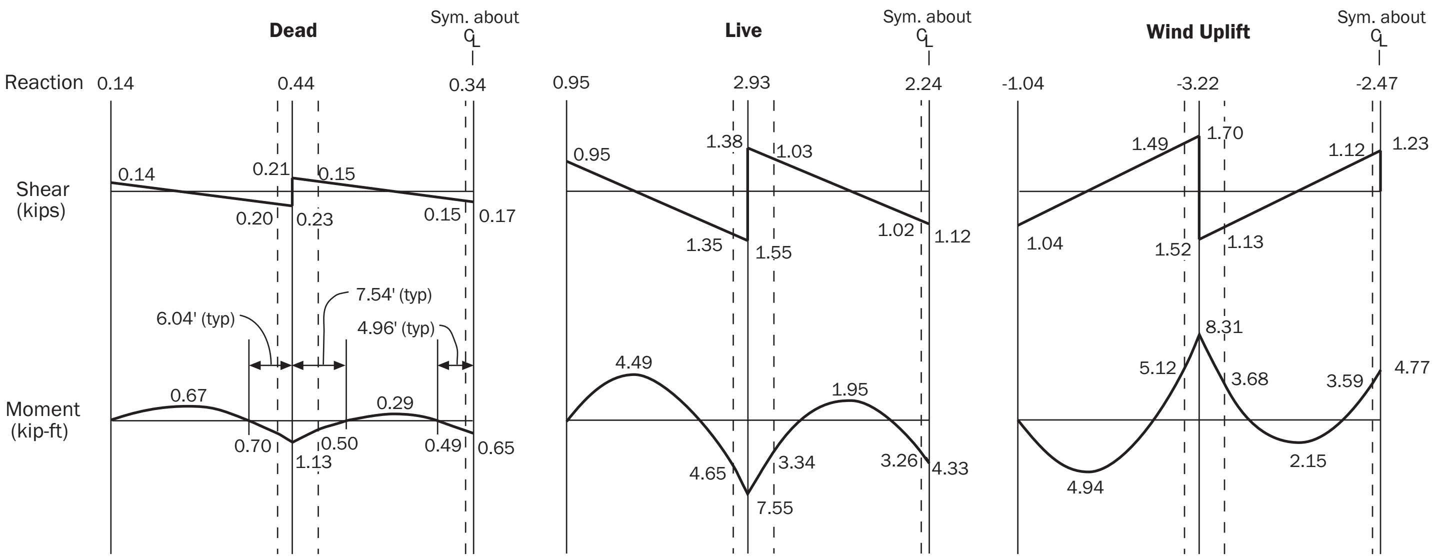


Figure 2 - Reactions, Shears and Moments

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. Roof Slope = 0.5/12; all purlins face uphill
5. No discrete bracing lines: anti-roll clips at each support at every fourth purlin line
6. Bottom flanges are bolted to a 0.25 in. thick supporting member with a bearing length of 5 in.

Required:

1. Check the design using ASD with ASCE 7-98 load combinations for:
   (a) Gravity Loads
   (b) Wind Uplift Loads
2. Compute the anchorage forces at the supports under gravity loads.

[1] For design of purlins supporting standing seam roof, see AISI publication *A Design Guide for Designing with Standing Seam Roof Panels.*

Solution:

1. Assumptions for Analysis and Application of *Specification* Provisions

The *Specification* does not define the methods of analysis to be used; these judgements 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. It is assumed in the continuous beam analysis that 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. It is assumed that the attachment of the roof covering to the purlin provides continuous lateral and torsional support to the top flange.

e. For gravity loads, the compression (bottom) flange at and near the interior supports is assumed to be fully braced between the support and the end of the lap.

f. Under uniform gravity loading, the negative moment region between the end of the lap and the inflection point is assumed to be braced at the end of the lap and the inflection point.

g. 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 *Design Manual*, the following section properties have been obtained for the two Z-sections:

Section	8ZS2.25x059	8ZS2.25x070
D (in.)	8.0	8.0
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})	1.79	2.24
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:

$M = M_D + M_{Lr}$

End Span, from left to right:

Maximum positive moment: $M = 0.67 + 4.49 = 5.16$ kip-ft

Negative moment at end of right lap: $M = 0.70 + 4.65 = 5.35$ kip-ft

Negative moment at support: $M = 1.13 + 7.55 = 8.68$ kip-ft

Interior span, from left to right:

Negative moment at end of left lap: $M = 0.50 + 3.34 = 3.84$ kip-ft

Maximum positive moment: $M = 0.29 + 1.95 = 2.24$ kip-ft

Negative moment at end of right lap: $M = 0.49 + 3.26 = 3.75$ kip-ft

Negative moment at center support: $M = 0.65 + 4.33 = 4.98$ kip-ft

Allowable Strength

End Span:

At location of maximum positive moment, the section is assumed to be fully braced.
Use nominal strength based on initiation of yielding per Section C3.1.1(a)

For the exterior purlin, t = 0.070 in.

$M_n = S_e F_y = (2.24)(55) = 123$ kip-in. $= 10.3$ kip-ft (*Eq.* C3.1.1-1)

$\frac{M_n}{\Omega_b} = \frac{10.3}{1.67} = 6.17$ kip-ft > 5.16 kip-ft OK (*Eq.* A4.1.1-1)

In the region of negative moment between the end of the lap and the inflection point:

Determine the nominal strength using the distance from the inflection point to the end of the lap as the unbraced length per Section C3.1.2.1(b).

$F_e = \frac{C_b \pi^2 E d I_{yc}}{2S_f(K_y L_y)^2}$ (*Eq.* C3.1.2.1-15)

$L = 6.04 - 2.00 = 4.04$ ft $= 48.5$ 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)

$F_e = \frac{(1.67)\pi^2(29500)(8.0)(0.640)}{(2)(2.30)(48.5)^2} = 230$ ksi (*Eq.* C3.1.2.1-15)

$2.78F_y = (2.78)(55) = 153$ ksi

Since $F_e > 2.78F_y$

$F_c = F_y$ (*Eq.* C3.1.2.1-2)

$= 55$ ksi

S_c is taken as S_e computed at a maximum stress, f, of 55 ksi. From above, $S_c = 2.24 \text{ in.}^3$

$M_n = S_c F_c$ (*Eq.* C3.1.2.1-1)

$= (2.24)(55.0) = 123$ kip-in. or 10.3 kip-ft

$\frac{M_n}{\Omega_b} = \frac{10.3}{1.67} = 6.17$ kip-ft > 5.35 kip-ft OK (*Eq.* A4.1.1-1)

In the lapped region over the support, the section is assumed to be fully braced:

Use nominal 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.

$M_n = S_e F_y = (2.24)(55) = 123$ kip-in. or 10.3 kip-ft (*Eq.* C3.1.1-1)

For the interior purlin, t = 0.059 in.

$M_n = S_e F_y = (1.79)(55) = 98.5$ kip-in. or 8.20 kip-ft (*Eq.* C3.1.1-1)

Combined strength of purlins

$$\frac{M_n}{\Omega_b} = \frac{10.3 + 8.20}{1.67} = 11.1 \text{ kip-ft} > 8.68 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

Interior Span:

In the region of negative moment between the end of the left lap and the inflection point:

Determine the nominal strength using the distance from the inflection point to the lap as the unbraced length per Section C3.1.2.1(b).

$L = 7.54 - 3.50 = 4.04$ ft or 48.5 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)

$$F_e = \frac{(1.67)\pi^2(29500)(8.0)(0.540)}{(2)(1.94)(48.5)^2} = 230 \text{ ksi} \qquad (Eq.\ C3.1.2.1\text{-}15)$$

Since $F_e > 2.78\ F_y$,

$F_c = F_y = 55$ ksi (*Eq.* C3.1.2.1-2)

S_c is taken as S_e computed at a maximum stress, f, of 55 ksi. From above, $S_c = 1.79$ in.^{3}

$M_n = S_cF_c = (1.79)(55.0) = 98.5$ kip-in. or 8.20 kip-ft (*Eq.* C3.1.2.1-1)

$$\frac{M_n}{\Omega_b} = \frac{8.20}{1.67} = 4.91 \text{ kip-ft} > 3.84 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

At the location of maximum positive moment, the section is assumed to be fully braced:

Use nominal strength based on initiation of yielding per Section C3.1.1(a) calculated above.

$$\frac{M_n}{\Omega_b} = \frac{8.20}{1.67} = 4.91 \text{ kip-ft} > 2.24 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

In the region of negative moment between the end of the right lap and the inflection point:

Determine the nominal strength using the distance from the inflection point to the lap as the unbraced length per Section C3.1.2.1(b).

$L = 4.96 - 1.00 = 3.96$ ft or 47.5 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:

Use nominal strength based on initiation of yielding per Section C3.1.1(a), summing the strength of the two overlapped purlins:

Combined strength of purlins

$$\frac{M_n}{\Omega_b} = \frac{8.20 + 8.20}{1.67} = 9.82 \text{ kip-ft} > 4.98 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

b. Strength for Shear Only (Section C3.2)

Required Allowable Strength

By inspection, the load combination D + L_r controls:

$V = V_D + V_{Lr}$

End Span, from left to right:

At left support:	$V = 0.14 + 0.95 = 1.09$ kips
At end of right lap:	$V = 0.20 + 1.35 = 1.55$ 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 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. and h = 7.485 in., h/t = 107

$$\frac{h}{t} > 1.51\sqrt{E\,k_v/F_y} = 1.51\sqrt{(29500)(5.34)/55} = 80.8$$

$$F_v = \frac{\pi^2 E k_v}{12(1-\mu^2)(h/t)^2} \qquad (Eq.\ C3.2.1\text{-}4)$$

$$= \frac{\pi^2(29500)(5.34)}{12(1-0.3^2)(7.485/0.070)^2} = 12.5 \text{ ksi}$$

$$V_n = A_w F_v \qquad (Eq.\ C3.2.1\text{-}1)$$

$$= (7.485)(0.070)(12.5) = 6.55 \text{ kips}$$

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.

$$\frac{V_n}{\Omega_v} = \frac{6.55}{1.60} = 4.09 \text{ kips} > 1.55 \text{ kips OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

At the first interior support, sum the strength of the two overlapped purlins:

For t = 0.059 in. and h = 7.507 in., h/t = 127

$$\frac{h}{t} > 1.51\sqrt{E\,k_v/F_y} = 80.8$$

$$F_v = \frac{\pi^2(29500)(5.34)}{12(1-0.3^2)(7.507/0.059)^2} = 8.80 \text{ ksi} \qquad (Eq.\ C3.2.1\text{-}4)$$

$$V_n = A_w F_v \qquad (Eq.\ C3.2.1\text{-}1)$$

$$= (7.507)(0.059)(8.80) = 3.90 \text{ kips}$$

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 section:

$$\frac{V_n}{\Omega_v} = \frac{3.90 + 6.55}{1.60} = 6.53 \text{ kips} > 1.78 \text{ kips OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

Interior span:

At first interior support, use strength computed above:

$$\frac{V_n}{\Omega_v} = 6.53 \text{ kips} > 1.59 \text{ kips OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

By inspection of the left and right laps, the left lap controls

$$\frac{V_n}{\Omega_v} = \frac{3.90}{1.60} = 2.44 \text{ kips} > 1.18 \text{ kips OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

At the center support, sum the strength of the two overlapped purlins:

For the combined section:

$$\frac{V_n}{\Omega_v} = \frac{3.90 + 3.90}{1.60} = 4.88 \text{ kips} > 1.29 \text{ kips OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

c. Strength for Combined Bending and Shear (Section C3.3.1)

End Span:

$$\left(\frac{\Omega_b M}{M_{nxo}}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2 \leq 1.0 \qquad (Eq.\ C3.3.1\text{-}1)$$

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.

$$\left(\frac{(1.67)(5.35)}{10.3}\right)^2 + \left(\frac{(1.60)(1.55)}{6.55}\right)^2 = 0.90 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.1\text{-}1)$$

At first interior support,

$$\left(\frac{(1.67)(8.68)}{10.3 + 8.20}\right)^2 + \left(\frac{(1.60)(1.78)}{6.55 + 3.90}\right)^2 = 0.69 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.1\text{-}1)$$

Interior Span:

At ends of laps, t = 0.059 in. Left lap controls by inspection.

$$\left(\frac{(1.67)(3.84)}{8.20}\right)^2 + \left(\frac{(1.60)(1.18)}{3.90}\right)^2 = 0.85 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.1\text{-}1)$$

At center support,

$$\left(\frac{(1.67)(4.98)}{8.20 + 8.20}\right)^2 + \left(\frac{(1.60)(1.29)}{3.90 + 3.90}\right)^2 = 0.33 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.1\text{-}1)$$

d. Web Crippling Strength (Section C3.4)

Required Allowable Strength

By inspection, the load combination D + L_r controls:

$P = P_D + P_{Lr}$

Supports, from left to right:

At left support: $P = 0.14 + 0.95 = 1.09$ 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 Design Strength

$$P_n = Ct^2F_y \sin\theta\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \qquad (Eq.\ C3.4.1\text{-}1)$$

where

F_y = 55 ksi
θ = 90 degrees
R = 0.1875 in.
N = bearing length = 5.0 in.

At outside 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

$$P_n = (4)(0.070)^2(55)\sin(90)\left(1 - 0.14\sqrt{\frac{0.1875}{0.070}}\right)\left(1 + 0.35\sqrt{\frac{5.0}{0.070}}\right)\left(1 - 0.02\sqrt{\frac{7.485}{0.070}}\right)$$

$$= 2.61 \text{ kips} \qquad (Eq.\ C3.4.1\text{-}1)$$

Alternately, P_n can be taken from Table II-12, 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 kips.

Using the value of P_n calculated from *Eq*. C3.4.1-1 above,

$$\frac{P_n}{\Omega_w} = \frac{2.61}{1.75} = 1.49 \text{ kips} > 1.09 \text{ kips OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

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
Ω_w = 1.65

for t = 0.070 in.:

Check Limit: R/t = 0.1875 / 0.070 = 2.7 < 5 OK

$$P_n = (13)(0.070)^2(55)\sin(90)\left(1 - 0.23\sqrt{\frac{0.1875}{0.070}}\right)\left(1 + 0.14\sqrt{\frac{5.0}{0.070}}\right)\left(1 - 0.01\sqrt{\frac{7.485}{0.070}}\right)$$

$$= 4.28 \text{ kips} \qquad (Eq.\ C3.4.1\text{-}1)$$

Alternately, P_n can be taken from Table II-12, Web Crippling, Z-Sections With Lips. Using Case B, for a 8ZS2.25x070 with a yield stress of 55 ksi P_n can be interpolated as:

$P_n = 0.5(4.03 + 4.50) = 4.27$ kips.

for t = 0.059 in.:

Check Limit: R/t = 0.1875 / 0.059 = 3.2 < 5 OK

$$P_n = (13)(0.059)^2(55)\sin(90)\left(1 - 0.23\sqrt{\frac{0.1875}{0.059}}\right)\left(1 + 0.14\sqrt{\frac{5.0}{0.059}}\right)\left(1 - 0.01\sqrt{\frac{7.507}{0.059}}\right)$$

$$= 2.98 \text{ kips} \qquad (Eq.\ C3.4.1\text{-}1)$$

Alternately, P_n can be taken from Table II-12, 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:

$P_n = 0.5(2.80 + 3.14) = 2.97$ kips.

Using the values of P_n calculated from *Eq.* C3.4.1-1 above,

At first interior support,

$$\frac{P_n}{\Omega_w} = \frac{4.28 + 2.98}{1.65} = 4.40 \text{ kips} > 3.37 \text{ kips OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

At center support,

$$\frac{P_n}{\Omega_w} = \frac{2.98 + 2.98}{1.65} = 3.61 \text{ kips} > 2.58 \text{ kips OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

e. Combined Bending and Web Crippling (Section C3.5.1(c))

$$\frac{M}{M_{no}} + 0.85\frac{P}{P_n} \le \frac{1.65}{\Omega} \qquad (Eq.\ C3.5.1\text{-}3)$$

where

M_{no} = 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 = 1.75

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 limits are satisfied

At the first interior support,

$$\frac{8.68}{10.3 + 8.20} + 0.85\frac{3.37}{4.28 + 2.98} = 0.86 < \frac{1.65}{1.75} = 0.94 \text{ OK} \qquad (Eq.\ C3.5.1\text{-}3)$$

At the center support,

$$\frac{4.98}{8.20 + 8.20} + 0.85\frac{2.58}{2.98 + 2.98} = 0.67 < 0.94 \text{ OK} \qquad (Eq.\ C3.5.1\text{-}3)$$

4. Check Uplift Loads

a. Strength for Bending Only (Section C3.1.3)

Required Allowable Strength

By inspection, the load combination D + W controls.

$M = M_D + M_w$

End Span:

Maximum Negative Moment: $M = 0.67 - 4.94 = -4.27$ kip-ft

Interior Span:

Maximum Negative Moment: $M = 0.29 - 2.15 = -1.86$ kip-ft

Allowable Strength

$M_n = R\,S_e\,F_y$ (*Eq.* C3.1.3-1)

$R = 0.70$, assuming all 15 conditions of Section C3.1.3 are satisfied

End Span:

For t = 0.070 in.

$M_n = (0.70)(2.24)(55) = 86.2$ kip-in. or 7.19 kip-ft (*Eq.* C3.1.3-1)

$\dfrac{M_n}{\Omega_b} = \dfrac{7.19}{1.67} = 4.31$ kip-ft > 4.27 kip-ft OK (*Eq.* A4.1.1-1)

Interior Span:

For t = 0.059 in.

$M_n = (0.70)(1.79)(55) = 68.9$ kip-in. or 5.74 kip-ft (*Eq.* C3.1.3-1)

$\dfrac{M_n}{\Omega_b} = \dfrac{5.74}{1.67} = 3.44$ kip-ft > 1.86 kip-ft OK (*Eq.* 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 that the design satisfies the *Specification* criteria for uplift.

5. Anchorage Forces under Gravity Loads

Compute the anchorage forces at the supports (Section D3.2.1) with anchorage devices at every fourth purlin. Assume the roof panel meets the L/360 lateral stiffness requirements of Section D3.2.1.

Case (b)(4) Multiple-Span System with Restraints at the Supports.

$$P_L = C_{tr}\left[\frac{0.053 b^{1.88} L^{0.13}}{n_p^{0.95} d^{1.07} t^{0.94}}\cos\theta - \sin\theta\right]W \quad (Eq.\ D3.2.1\text{-}5)$$

$\theta = \tan^{-1}(0.5/12) = 2.39$ degrees

$W = n(L)(W_D + W_{Lr}) = (4)(25)(15 + 100) = 11{,}500$ lbs

End Span:

$t = 0.070$ in.

$$P_L = C_{tr}\left[\frac{(0.053)(2.25^{1.88})\left[(25)(12)\right]^{0.13}}{4^{0.95} 8.0^{1.07} 0.070^{0.94}}\cos(2.39°) - \sin(2.39°)\right]11,500 \quad (Eq.\ D3.2.1\text{-}5)$$

$P_L = C_{tr}1591$

Interior Span:

$t = 0.059$ in.

$$P_L = C_{tr}\left[\frac{(0.053)(2.25^{1.88})[(25)(12)]^{0.13}}{4^{0.95}8.0^{1.07}0.059^{0.94}}\cos(2.39°) - \sin(2.39°)\right]11,500 \qquad (Eq.\ D3.2.1\text{-}5)$$

$= C_{tr}\,1952$

At outside supports, $C_{tr} = 0.63$

$P_L = 0.63(1591) = 1000$ lbs

At interior supports, average the contributions from adjacent purlins

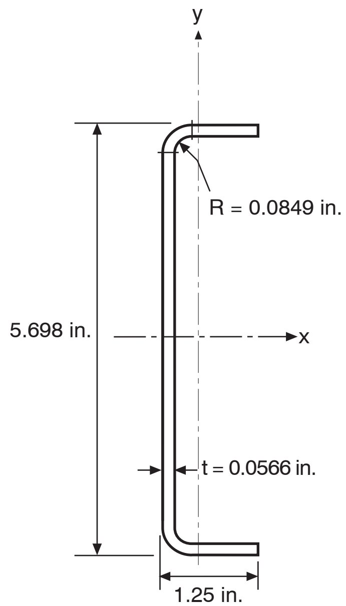
At first interior support, $C_{tr} = 0.87$

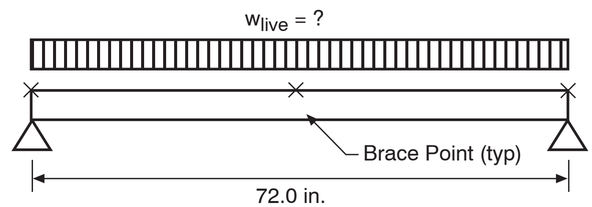
$P_L = 0.87(1591 + 1952) / 2 = 1540$ lbs

At center support, $C_{tr} = 0.81$

$P_L = 0.81(1952 + 1952) / 2 = 1580$ lbs

Example II-3: C-Section Without Lips Braced At Mid-span





Given:

1. Steel: F_y = 33 ksi
2. Section: SSMA Track 550T125-54 as shown in sketch above. The gross section properties are listed in Table I-3.
3. Simple span of 72 in.
4. Braced against twisting and lateral deflection at mid-span and ends. Ends reinforced against crippling

Required:

1. 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 flexural-torsional buckling.

1. Lateral-torsional buckling strength (Section C3.1.2.1)

For singly symmetric sections bent about the axis of symmetry,

$$F_e = \frac{C_b r_o A}{S_f}\sqrt{\sigma_{ey}\sigma_t} \qquad (\textit{Eq. } \text{C3.1.2.1-5})$$

$$\sigma_{ey} = \frac{\pi^2 E}{\left(K_y L_y / r_y\right)^2} \qquad (\textit{Eq. } \text{C3.1.2.1-8})$$

$$= \frac{\pi^2(29500)}{\left[(1.0)(36.0)/0.342\right]^2}$$

$$= 26.28 \text{ ksi}$$

$$\sigma_t = \frac{1}{A r_o^2}\left[GJ + \frac{\pi^2 E C_w}{\left(K_t L_t\right)^2}\right] \qquad (\textit{Eq. } \text{C3.1.2.1-9})$$

$$\sigma_t = \frac{1}{(0.452)(2.15)^2}\left[(11300)(0.000483) + \frac{\pi^2(29500)(0.315)}{\left[(1.0)(36.0)\right]^2}\right]$$

$= 36.48$ ksi

Calculate C_b assuming a unit loading

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \qquad (Eq.\ C3.1.2.1\text{-}10)$$

$M_{max} = \frac{wL^2}{8} = \frac{(1.0)(72.0)^2}{8} = 648.0$ kip-in. at brace point

$M_A = \frac{7wL^2}{128} = \frac{(7)(1.0)(72.0)^2}{128} = 283.5$ kip-in. at 1/4 point of unbraced segment

$M_B = \frac{3wL^2}{32} = \frac{(3)(1.0)(72.0)^2}{32} = 486.0$ kip-in. at center point of unbraced segment

$M_C = \frac{15wL^2}{128} = \frac{(15)(1.0)(72.0)^2}{128} = 607.5$ kip-in. at 3/4 point of unbraced segment

$$C_b = \frac{(12.5)(648.0)}{(2.5)(648.0) + (3)(283.5) + (4)(486.0) + (3)(607.5)} \qquad (Eq.\ C3.1.2.1\text{-}10)$$

$= 1.30$

$$F_e = \frac{(1.30)(2.15)(0.452)}{0.668}\sqrt{(26.28)(36.48)} = 58.6 \text{ ksi} \qquad (Eq.\ C3.1.2.1\text{-}5)$$

$0.56F_y = (0.56)(33.0) = 18.5$ ksi

$2.78F_y = (2.78)(33.0) = 91.7$ ksi

For $2.78F_y > F_e > 0.56F_y$:

$$F_c = \frac{10}{9}F_y\left(1 - \frac{10F_y}{36F_e}\right) \qquad (Eq.\ C3.1.2.1\text{-}3)$$

$$= \frac{10}{9}(33.0)\left(1 - \frac{(10)(33.0)}{(36)(58.6)}\right) = 30.93 \text{ ksi}$$

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}

$$w_{live} \le \frac{8M}{L^2}$$

Allowable Design Strength

$$M \le \frac{M_n}{\Omega_b} = \frac{18.7}{1.67} = 11.2 \text{ kip-in.} \qquad (Eq.\ A4.1.1\text{-}1)$$

$$w_{live} \le \frac{(8)(11.2)}{(72.0)^2} = 0.0173 \text{ kips/in.} = 207 \text{ plf}$$

LRFD

Design Strength (LRFD)

$M_u \le \phi M_n = (0.90)(18.7) = 16.8$ kip-in. (*Eq.* A5.1.1-1)

Live load factor = 1.6

$$1.6w_{live} \le \frac{(8)(16.8)}{(72.0)^2}$$

$w_{live} \le 0.0162$ kips/in. = 194 plf

3. Check Shear (Section C3.2)

$$h/t = [5.698 - 2(0.0566 + 0.0849)]/0.0566 = 95.7$$

$$\sqrt{Ek_v/F_y} = \sqrt{(29500)(5.34)/33.0} = 69.1$$

$$1.51\sqrt{Ek_v/F_y} = (1.51)(69.1) = 104.3$$

For $\sqrt{Ek_v/F_y} < h/t \le 1.51\sqrt{Ek_v/F_y}$

$$F_v = \frac{0.6\sqrt{Ek_vF_y}}{(h/t)} \quad (Eq.\ C3.2.1\text{-}3)$$

$$= \frac{0.6\sqrt{(29500)(5.34)(33)}}{(95.7)}$$

$= 14.3$ ksi

$V_n = A_wF_v$ (*Eq.* C3.2.1-1)

$= (5.415)(0.0566)(14.3) = 4.38$ kips

or per Table II-3, for a 550T125-54 with a yield stress of 33 ksi:

$V_n = 4.38$ kips

ASD

Allowable Design Strength

$$V = w_{live}L/2 \le \frac{V_n}{\Omega_v} \quad (Eq.\ A4.1.1\text{-}1)$$

$\Omega_v = 1.60$

$$V = (0.0173)(72.0)/2 = 0.62 \text{ kips} < \frac{4.38}{1.60} = 2.74 \text{ kips OK}$$

LRFD

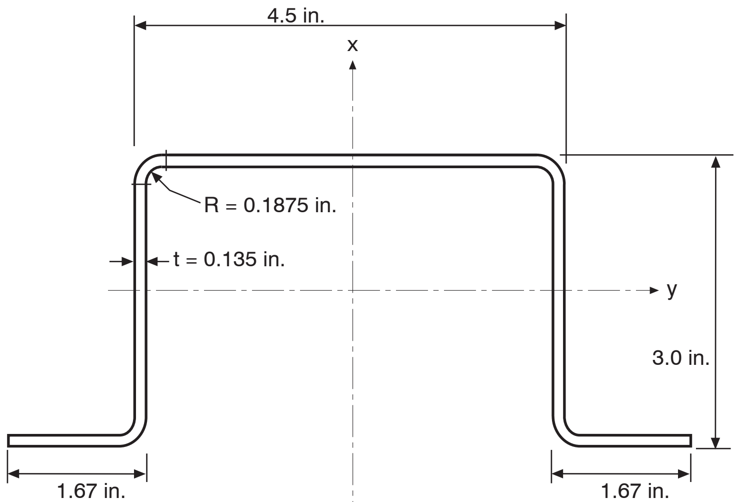
Design Strength

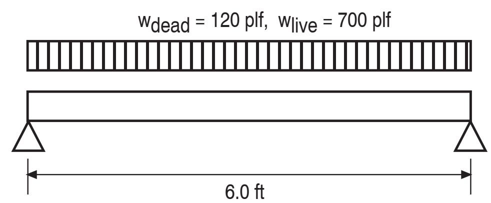
$V_u = 1.6w_{live}L/2 \le \phi_vV_n$ (*Eq.* A5.1.1-1)

$\phi_v = 0.95$

$V_u = (1.6)(0.0162)(72.0)/2 = 0.93$ kips $< (0.95)(4.38) = 4.16$ kips OK

Example II-4: 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 foot 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. Solve for M_n (Section C3.1.1)

Bending Moments

$$M_D = \frac{w_{dead}L^2}{8} = \frac{(0.120)(6)^2}{8} = 0.540 \text{ kip-ft} = 6.48 \text{ kip-in.}$$

$$M_L = \frac{w_{live}L^2}{8} = \frac{(0.700)(6)^2}{8} = 3.15 \text{ kip-ft} = 37.80 \text{ kip-in.}$$

ASD

Required Allowable Strength

$M = M_D + M_L = 6.48 + 37.80 = 44.3$ kip-in.

From Example I-13 or Table II-5, $S_e = 1.516$ in.^{3}

Nominal Flexural Strength

$M_n = S_eF_y$ (*Eq*. C3.1.1-1)

$= (1.516)(50) = 75.8$ kip-in.

Allowable Design Strength

$M < M_n/\Omega_b$ (*Eq*. A4.1.1-1)

$\Omega_b = 1.67$

$\frac{M_n}{\Omega_b} = \frac{75.8}{1.67} = 45.4$ kip-in. > 44.3 kip-in. OK

LRFD

Required Strength

$M_u = 1.2M_D + 1.6\ M_L$

$= (1.2)(6.48) + (1.6)(37.80)$

$= 68.3$ kip-in.

Nominal Flexural Strength

$M_n = S_eF_y$ (*Eq*. C3.1.1-1)

$= (1.516)(50) = 75.8$ kip-in.

Design Strength

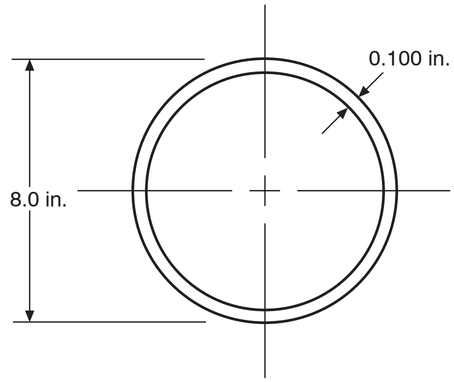
$M_u < \phi_bM_n$ (*Eq*. A5.1.1-1)

$\phi_b = 0.95$

$\phi_bM_n = (0.95)(75.8) = 72.0$ kip-in. > 68.3 kip-in. OK

The resistance factor of 0.95 is permitted per Section C3.1.1 because the compression flange is stiffened.

Example II-5: Tubular Section - Round



Given:

1. Steel: F_y = 50 ksi
2. Section: Shown in sketch above

Required:

1. ASD flexural allowable design strength
2. LRFD flexural design strength

Solution:

1. Nominal Flexural Strength (Section C6.1):

Ratio of outside diameter to wall thickness,

$D/t = 8.0/0.100 = 80.0$

$D/t < 0.441E/F_y = 0.441(29500/50) = 260$ OK

Full Section Properties

$$S_f = \pi\frac{(O.D.)^4 - (I.D.)^4}{32(O.D.)}$$

$$= \pi\frac{(8.0)^4 - (7.800)^4}{(32)(8.0)}$$

$$= 4.84 \text{ in.}^3$$

Determine the governing equation

$0.0714E/F_y = 0.0714(29500/50) = 42.1$

$0.318E/F_y = 0.318(29500/50) = 188$

Since $0.0714E/F_y < D/t < 0.318E/F_y$

$$F_c = \left[0.970 + 0.020\left(\frac{E/F_y}{D/t}\right)\right]F_y \qquad (Eq.\ C6.1\text{-}3)$$

$$F_c = \left[0.970 + 0.020\left(\frac{29500/50}{80.0}\right)\right]50 = 55.9 \text{ ksi}$$

$$M_n = F_c S_f \qquad (Eq.\ C6.1\text{-}1)$$

$M_n = (55.9)(4.84) = 271$ kip-in.

2. ASD allowable design strength

$M \le M_n/\Omega_b$ (*Eq.* A4.1.1-1)

$\Omega_b = 1.67$

$M \le 271/1.67 = 162$ kip-in.

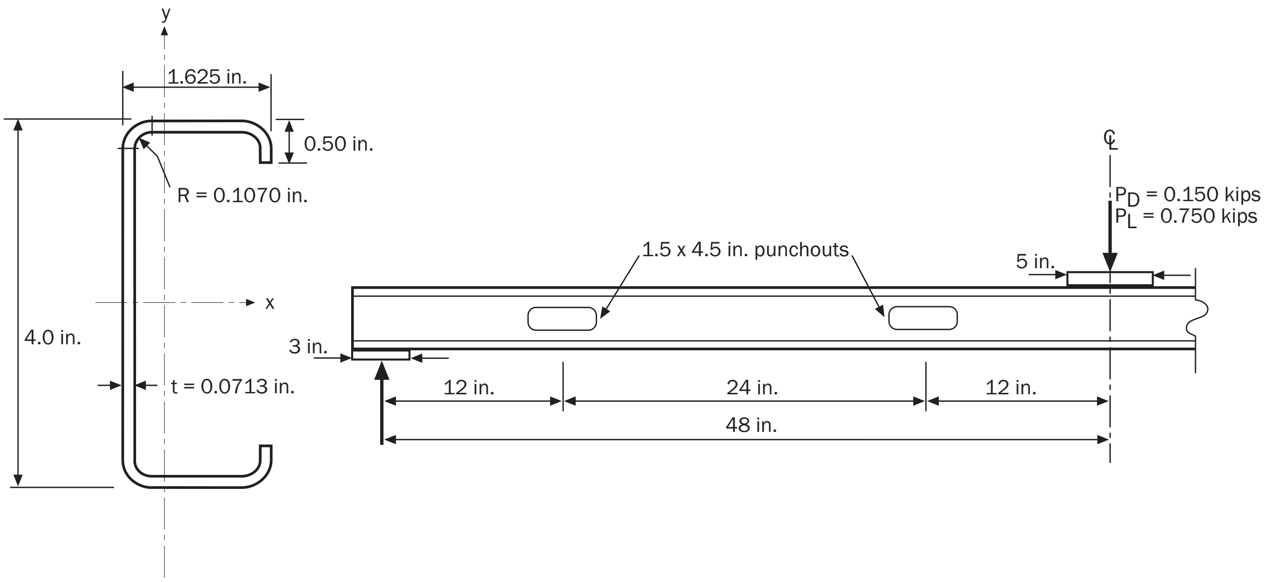
3. LRFD design strength

$M_u \le \phi_b M_n$ (*Eq.* A5.1.1-1)

$\phi_b = 0.95$

$M_u \le (0.95)(271) = 257$ kip-in.

Example II-6: 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 translation and rotation, and 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 7-98 ASD load combination D + L
2. LRFD - ASCE 7-98 LRFD load combination 1.2D + 1.6L

Neglect self weight of beam

Solution:

1. Flexural Strength

a. Required Strength

ASD Required Allowable Strength

$P = P_D + P_L = 0.150 + 0.750 = 0.900$ kips

$V = P/2 = 0.900/2 = 0.450$ kips

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

$$M = V[L/2 - (12.0 - 2.25)] = (0.450)[96.0/2 - 9.75] = 17.2 \text{ kip-in.}$$

LRFD Required Strength

$P_u = 1.2P_D + 1.6P_L = (1.2)(0.150) + (1.6)(0.750) = 1.38$ kips

$V_u = P_u/2 = 1.38/2 = 0.690$ kips

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

$$M_u = V_u[L/2 - (12.0 - 2.25)] = (0.690)[96.0/2 - 9.75] = 26.4 \text{ kip-in.}$$

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)

$S_e = S_{gross} = 0.667$ in.^{3} (calculations not shown)

$M_n = S_eF_y$

$= (0.667)(56.6) = 37.8$ 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".

$d_0 = 1.5$ in.

$b = 4.5$ in.

$h = 4.00 - 2(0.1070 + 0.0713) = 3.643$ in.

Check limits

$d_0/h = 1.5/3.643 = 0.412 < 0.7$ 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

$d_0 < 2.5$ in. OK

$b = 4.5$ in. OK

$d_0 > 9/16$ in. OK

Since $d_0/h > 0.38$, treat compression portion of web as an unstiffened element as follows:

$w = (h - d_0)/2 = (3.643 - 1.50)/2 = 1.072$ in.

$k = 0.43$

Calculate first estimate of f_1 at the top of the flat width using similar triangles with gross properties.

$$f = f_1 = 50\left(\frac{4.0/2 - 0.0713 - 0.1070}{4.0/2}\right) = 45.5 \text{ ksi}$$

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

$$= 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.0713}{1.072}\right)^2 = 50.7 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{45.5}{50.7}} = 0.947 > 0.673 \therefore \text{ web is subject to local buckling}$$

$$\rho = (1 - 0.22/\lambda) / \lambda \qquad (Eq.\ B2.1\text{-}3)$$

$$= (1 - 0.22/0.947)/ 0.947 = 0.811$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$$= (0.811)(1.072) = 0.869 \text{ in.}$$

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 to 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

$M_n = S_e F_y$

$= (0.648)(50) = 32.4$ 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. Design Strength

ASD Allowable Design Strength

$\Omega_b = 1.67$

At center, away from holes

$$\frac{M_n}{\Omega_b} = \frac{37.8}{1.67} = 22.6 \text{ kip-in.} > 21.6 \text{ kip-in. OK}$$

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

$\phi_b = 0.95$

At center, away from holes

$\phi_b M_n$ = (0.95)(37.8) = 35.9 kip-in. > 33.1 kip-in. OK

At holes nearest center

$\phi_b M_n$ = (0.95)(32.4) = 30.8 kip-in. > 26.4 kip-in. OK

2. Shear Strength

a. Required Strength

ASD Required Allowable Strength

V = 0.450 kips (from above)

LRFD Required Strength

V_u = 0.690 kips (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 $h/t < \sqrt{Ek_v/F_y}$,

$F_v = 0.60F_y$ (*Eq.* C3.2.1-2)

$= (0.60)(50) = 30$ ksi

$V_n = A_w F_v$ (*Eq.* C3.2.1-1)

$= (3.643)(0.0713)(30) = 7.79$ kips

c. Shear Strength with Holes - Section C3.2.2

Limits same as those checked above OK

$c = h/2 - d_0/2$ (*Eq.* C3.2.2-4)

$= 3.643/2 - 1.50/2 = 1.07$ in.

$c/t = 1.07/0.0713 = 15.0$

Since $5 < c/t < 54$,

$q_s = c/(54t)$ (*Eq.* C3.2.2-2)

$= 1.07/[(54)(0.0713)] = 0.278$

$V_n = q_s V_n = (0.278)(7.79) = 2.17$ 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. Design Strength

ASD Allowable Design Strength

$\Omega_v = 1.60$

$$\frac{V_n}{\Omega_v} = \frac{2.17}{1.60} = 1.36 \text{ kips} > 0.450 \text{ kips. OK}$$

LRFD Design Strength

ϕ_v = 0.95

$\phi_v V_n$ = (0.95)(2.17) = 2.06 kips > 0.690 kips. OK

3. Web Crippling Strength

a. Required Strength

ASD Required Allowable Strength

End Condition

P = V = 0.450 kips

Interior Condition

P = $P_D + P_L$ = 0.150 + 0.750 = 0.900 kips

LRFD Required Strength

End Condition

P_u = V_u = 0.690 kips

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

R/t = 0.1070/0.0713 = 1.50 < 9 OK

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:

C = 4

C_R = 0.14

C_N = 0.35

C_h = 0.02

Ω_w = 1.75

ϕ_w = 0.85

$$P_n = Ct^2F_y \sin\theta\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \qquad (Eq.\ C3.4.1\text{-}1)$$

$$= (4)(0.0713)^2(50)\sin(90)\left(1 - 0.14\sqrt{\frac{0.1070}{0.0713}}\right)\left(1 + 0.35\sqrt{\frac{3.0}{0.0713}}\right)\left(1 - 0.02\sqrt{\frac{3.643}{0.0713}}\right)$$

= 2.36 kips

Alternatively, P_n can be conservatively interpolated from Table II-11. 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
Ω_w = 1.65
ϕ_w = 0.90

$$P_n = Ct^2F_y \sin\theta\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \quad (\textit{Eq. } C3.4.1\text{-}1)$$

$$= (13)(0.0713)^2(50)\sin(90)\left(1 - 0.23\sqrt{\frac{0.1070}{0.0713}}\right)\left(1 + 0.14\sqrt{\frac{5.0}{0.0713}}\right)\left(1 - 0.01\sqrt{\frac{3.643}{0.0713}}\right)$$

= 4.79 kips

Alternatively, P_n can be conservatively interpolated from Table II-11. 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)
$R_c = 1.01 - 0.325d_0/h + 0.083x/h \leq 1.0$ (*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
$P_n = R_cP_n$ = (1.0)(2.36) = 2.36 kips

Alternatively, R_c can be extrapolated from Table II-13b. 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)

$R_c = 0.90 - 0.047d_0/h + 0.053x/h \leq 1.0$ (*Eq.* C3.4.2-2)

$= 0.90 - (0.047)(1.50)/3.643 + (0.053)(7.25)/3.643 = 0.986 < 1.0$ OK

$P_n = R_cP_n = (0.986)(4.79) = 4.72$ kips

Alternatively, P_n can be conservatively interpolated from Table II-13a. 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. Design Strength

ASD Allowable Design Strength

End Condition

$\Omega_w = 1.75$

$\frac{P_n}{\Omega_w} = \frac{2.36}{1.75} = 1.35$ kips > 0.450 kips. OK

Interior Condition

$\Omega_w = 1.65$

$\frac{P_n}{\Omega_w} = \frac{4.72}{1.65} = 2.86$ kips > 0.900 kips. OK

LRFD Design Strength

End Condition

$\phi_w = 0.85$

$\phi_w P_n = (0.85)(2.36) = 2.01$ kips > 0.690 kips. OK

Interior Condition

$\phi_w = 0.90$

$\phi_w P_n = (0.90)(4.72) = 4.25$ kips > 1.38 kips. OK

4. Combined Bending and Shear Strength

ASD

At center of beam (no holes)

$$\left(\frac{\Omega_b M}{M_{nxo}}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2 \leq 1.0 \qquad (Eq.\ C3.3.1\text{-}1)$$

$$\left(\frac{(1.67)(21.6)}{37.8}\right)^2 + \left(\frac{(1.60)(0.450)}{7.79}\right)^2 = 0.919 < 1.0 \text{ OK}$$

At edge of the hole closest to the center

$$\left(\frac{\Omega_b M}{M_{nxo}}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2 \leq 1.0 \qquad (Eq.\ C3.3.1\text{-}1)$$

$$\left(\frac{(1.67)(17.2)}{32.4}\right)^2 + \left(\frac{(1.60)(0.450)}{2.17}\right)^2 = 0.896 < 1.0 \text{ OK}$$

Alternatively, this case can be checked with Table II-8a. 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 ≤ 0.678 kips
for M = 18.7 kip-in., V ≤ 0.351 kips
for M = 17.2 kip-in., interpolating,

$$V \le 0.351 + \left(\frac{18.7 - 17.2}{18.7 - 16.8}\right)(0.678 - 0.351) = 0.609 \text{ kips} > 0.450 \text{ kips OK}$$

LRFD

At center of beam (no holes)

$$\left(\frac{\overline{M}}{\phi_b M_{nxo}}\right)^2 + \left(\frac{\overline{V}}{\phi_v V_n}\right)^2 \le 1.0 \qquad (Eq.\ C3.3.2\text{-}1)$$

$\overline{M} = M_u$
$\overline{V} = V_u$

$$\left(\frac{33.1}{(0.95)(37.8)}\right)^2 + \left(\frac{0.690}{(0.95)(7.79)}\right)^2 = 0.858 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.2\text{-}1)$$

At edge of hole closest to the center

$$\left(\frac{\overline{M}}{\phi_b M_{nxo}}\right)^2 + \left(\frac{\overline{V}}{\phi_v V_n}\right)^2 \le 1.0 \qquad (Eq.\ C3.3.2\text{-}1)$$

$$\left(\frac{26.4}{(0.95)(32.4)}\right)^2 + \left(\frac{0.690}{(0.95)(2.17)}\right)^2 = 0.848 < 1.0 \text{ OK}$$

Alternatively, this case can be checked with Table II-8b. 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 = 21.8 kip-in., V_u ≤ 1.46 kips
for M_u = 26.7 kip-in., V_u ≤ 1.03 kips
for M_u = 26.4 kip-in., interpolating,

$$V_u \le 1.03 + \left(\frac{26.7 - 26.4}{26.7 - 21.8}\right)(1.46 - 1.03) = 1.06 \text{ kips} > 0.690 \text{ kips OK}$$

5. Combined Bending and Web Crippling

Concentrated load at center of beam controls

ASD

$$1.2\left(\frac{\Omega_w P}{P_n}\right) + \left(\frac{\Omega_b M}{M_{nxo}}\right) \le 1.5 \qquad (Eq.\ C3.5.1\text{-}1)$$

$$1.2\left(\frac{(1.65)(0.900)}{4.72}\right) + \left(\frac{(1.67)(21.6)}{37.8}\right) = 1.33 < 1.5 \quad \text{OK}$$

LRFD

$$1.07\left(\frac{\overline{P}}{\phi_w P_n}\right) + \left(\frac{\overline{M}}{\phi_b M_{nxo}}\right) \le 1.42 \qquad (Eq.\ C3.5.2\text{-}1)$$

$\overline{P} = P_u$

$\overline{M} = M_u$

$$1.07\left(\frac{1.38}{(0.90)(4.72)}\right) + \left(\frac{33.1}{(0.95)(37.8)}\right) = 1.27 < 1.42 \quad \text{OK} \qquad (Eq.\ C3.5.2\text{-}1)$$

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PART III
COLUMN DESIGN
FOR USE WITH THE
2001 EDITION OF THE
NORTH AMERICAN
SPECIFICATION FOR THE DESIGN OF
COLD-FORMED STEEL STRUCTURAL MEMBERS

PART III - COLUMN DESIGN

SECTION 1 - CONCENTRICALLY LOADED COLUMNS

1.1 Notes On The Tables

(a) With the exception of the SSMA studs 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-6 inclusive were computed using the yield points 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 points listed in the respective tables.

(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 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.

(i) The calculated values in Tables III-1 through III-6 are nominal strengths. These values must be modified by a factor of safety, Ω_c, for ASD or a resistance factor ϕ_c, for LRFD. See the appropriate *Specification* section for more information.

(j) The effects of standard factory punchouts in SSMA studs have been included in Tables III-2 and III-5. These punchouts are considered in SSMA 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.

(k) Dashes in the place of data values in the P_n columns of Tables III-2 and III-3 indicate that the section is not available in the listed grade of steel. Blank data values in Tables III-4, III-5 and III-6 indicate that the section is not available in the listed grade of steel or that KL/r_y exceeds 200.

1.2 Braced Column Property Tables

Table III - 1

Braced Column Properties
C-Sections With Lips

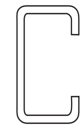
Ω_c = 1.80
ϕ_c = 0.85

Section	P_n at $f=F_y$ kips [1] F_y 33 ksi	P_n at $f=F_y$ kips [1] F_y 55 ksi	Maximum Effective Force, kips [2] P_{web}	P_{flange}	P_{lip}	Section	P_n at $f=F_y$ kips [1] F_y 33 ksi	P_n at $f=F_y$ kips [1] F_y 55 ksi	Maximum Effective Force, kips [2] P_{web}	P_{flange}	P_{lip}
12CS4x105	47.6	64.4	8.99	45.8	33.8	8CS3.5x105	44.9	62.2	16.2	52.7	39.9
12CS4x085	33.1	45.6	4.74	26.0	20.2	8CS3.5x085	32.3	44.3	8.51	30.1	23.8
12CS4x070	24.3	33.3	2.63	15.6	12.9	8CS3.5x070	23.6	32.7	4.72	18.2	15.2
12CS3.5x105	46.2	63.5	8.56	54.0	41.2	8CS3.5x065	21.1	29.1	3.77	15.0	12.9
12CS3.5x085	33.0	44.9	4.51	30.8	24.5	8CS3.5x059	18.2	25.0	2.81	11.7	10.4
12CS3.5x070	24.0	33.0	2.51	18.5	15.6	8CS2.5x105	39.6	60.7	14.2	80.9	68.8
12CS2.5x105	40.9	62.0	7.70	82.2	70.1	8CS2.5x085	29.8	44.2	7.44	46.3	40.5
12CS2.5x085	30.5	44.8	4.06	47.0	41.2	8CS2.5x070	23.0	31.4	4.13	28.0	25.6
12CS2.5x070	23.4	31.8	2.25	28.3	25.9	8CS2.5x065	20.9	27.9	3.29	23.1	21.5
10CS4x105	47.1	63.9	12.0	45.3	33.3	8CS2.5x059	18.1	24.2	2.46	18.1	17.3
10CS4x085	32.9	45.3	6.29	25.8	20.0	8CS2x105	36.2	54.9	13.2	108	103
10CS4x070	24.2	33.1	3.49	15.4	12.7	8CS2x085	27.0	41.2	6.91	62.1	59.9
10CS4x065	21.5	29.4	2.79	12.7	10.8	8CS2x070	20.7	31.8	3.83	37.7	37.4
10CS3.5x105	45.7	63.0	11.3	53.5	40.7	8CS2x065	18.7	28.8	3.06	30.9	31.4
10CS3.5x085	32.7	44.7	5.96	30.5	24.2	8CS2x059	16.5	25.1	2.28	23.3	25.0
10CS3.5x070	23.8	32.9	3.31	18.4	15.4	7CS4x105	45.7	62.5	21.7	44.0	32.0
10CS3.5x065	21.3	29.3	2.64	15.2	13.0	7CS4x085	32.2	44.6	11.4	25.1	19.2
10CS2.5x105	40.4	61.5	10.1	81.7	69.6	7CS4x070	23.8	32.7	6.28	15.0	12.3
10CS2.5x085	30.2	44.6	5.29	46.7	40.9	7CS4x065	21.2	29.1	5.02	12.4	10.5
10CS2.5x070	23.3	31.7	2.94	28.2	25.8	7CS4x059	18.2	24.9	3.74	9.63	8.46
10CS2.5x065	21.1	28.0	2.35	23.3	21.7	7CS2.5x105	39.1	60.1	17.6	80.4	68.3
10CS2x105	36.9	55.7	9.43	109	104	7CS2.5x085	29.5	43.9	9.22	46.0	40.2
10CS2x085	27.4	41.6	4.96	62.5	60.3	7CS2.5x070	22.9	31.3	5.10	27.8	25.4
10CS2x070	20.9	32.0	2.75	38.0	37.7	7CS2.5x065	20.8	27.7	4.07	23.0	21.4
10CS2x065	18.9	29.0	2.20	31.1	31.6	7CS2.5x059	18.0	24.1	3.03	18.0	17.2
9CS2.5x105	40.1	61.1	11.8	81.4	69.3	6CS4x105	45.0	61.7	28.5	43.2	31.2
9CS2.5x085	30.0	44.4	6.20	46.5	40.8	6CS4x085	31.8	44.2	14.9	24.7	18.8
9CS2.5x070	23.2	31.6	3.44	28.1	25.7	6CS4x070	23.5	32.5	8.23	14.8	12.1
9CS2.5x065	21.0	28.0	2.75	23.2	21.7	6CS4x065	21.0	28.9	6.57	12.2	10.3
9CS2.5x059	18.2	24.3	2.05	18.1	17.4	6CS4x059	18.1	24.8	4.89	9.50	8.33
8CS4x105	46.3	63.1	17.2	44.6	32.5	6CS2.5x105	38.3	59.4	22.8	79.6	67.5
8CS4x085	32.5	44.9	9.05	25.4	19.5	6CS2.5x085	29.1	43.5	11.9	45.6	39.8
8CS4x070	23.9	32.9	5.01	15.2	12.5	6CS2.5x070	22.6	31.1	6.58	27.6	25.2
8CS4x065	21.3	29.2	4.00	12.5	10.6	6CS2.5x065	20.6	27.6	5.25	22.8	21.2
8CS4x059	18.3	25.0	2.98	9.73	8.56	6CS2.5x059	17.8	24.0	3.91	17.8	17.1

Table III - 1 (continued)

Braced Column Properties
C-Sections With Lips

$\Omega_c = 1.80$
$\phi_c = 0.85$



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]		
	F_y						F_y				
	33 ksi	55 ksi	P_{web}	P_{flange}	P_{lip}		33 ksi	55 ksi	P_{web}	P_{flange}	P_{lip}
4CS4x105	41.1	58.8	51.6	39.3	27.3	4CS2x105	31.6	51.7	43.8	103	104
4CS4x085	30.2	42.7	27.8	22.7	16.7	4CS2x085	25.3	39.8	22.7	58.4	59.5
4CS4x070	22.7	31.6	15.8	13.8	11.0	4CS2x070	19.9	31.3	12.5	33.8	35.0
4CS4x065	20.3	28.2	12.8	11.4	9.38	4CS2x065	18.2	28.4	9.92	27.4	28.6
4CS4x059	17.6	24.3	9.70	8.92	7.68	4CS2x059	16.1	24.5	7.37	20.7	21.8
4CS2.5x105	34.4	56.4	47.7	76.7	64.5						
4CS2.5x085	27.6	41.9	24.7	44.0	38.3						
4CS2.5x070	21.8	30.2	13.6	26.7	24.3						
4CS2.5x065	19.9	26.9	10.8	22.1	20.6						
4CS2.5x059	17.3	23.5	8.02	17.3	16.6						

Notes:

1. Axial strengths given are nominal strengths. To obtain the design strength, these values must be modified by factors of safety (ASD) or resistance factors (LRFD).

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.

Table III - 2

Braced Column Properties
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

Section	P_n at $f=F_y$ kips [1] F_y 33 ksi	P_n at $f=F_y$ kips [1] F_y 50 ksi	Maximum Effective Force, kips [2] P_{web}	Maximum Effective Force, kips [2] P_{flange}	Maximum Effective Force, kips [2] P_{lip}
1200S250-97	37.4	49.9	6.74	60.2	41.3
1200S250-68	22.1	28.2	2.29	24.2	19.1
1200S250-54*	15.3	19.7	1.14	13.6	12.1
1200S200-97	34.0	47.2	6.35	80.4	59.3
1200S200-68	20.8	28.2	2.16	31.5	26.5
1200S200-54*	15.2	19.2	1.07	17.7	16.6
1200S162-97	25.1	35.0	0.00	80.7	56.1
1200S162-68	15.5	21.9	0.00	30.8	23.4
1200S162-54*	11.4	15.6	0.00	17.0	14.2
1000S250-97	36.9	49.4	8.74	59.8	40.8
1000S250-68	22.0	28.0	2.96	24.0	19.0
1000S250-54	15.2	19.6	1.47	13.5	12.1
1000S250-43*	11.2	-	0.739	7.76	7.92
1000S200-97	33.6	46.7	8.18	79.9	58.8
1000S200-68	20.6	28.1	2.77	31.4	26.3
1000S200-54	15.1	19.2	1.38	17.6	16.5
1000S200-43*	10.9	-	0.693	9.61	10.6
1000S162-97	24.8	34.7	0.00	80.5	55.9
1000S162-68	15.4	21.8	0.00	30.7	23.3
1000S162-54	11.4	15.5	0.00	16.9	14.2
1000S162-43*	8.55	-	0.00	9.72	9.04
800S250-97	36.3	48.7	12.2	59.1	40.1
800S250-68	21.7	27.8	4.11	23.8	18.7
800S250-54	15.1	19.5	2.04	13.4	11.9
800S250-43	11.1	-	1.02	7.71	7.86
800S200-97	32.9	46.0	11.3	79.3	58.1
800S200-68	20.4	27.9	3.82	31.2	26.1
800S200-54	15.0	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	24.4	34.2	0.00	80.0	55.4
800S162-68	15.3	21.6	0.00	30.5	23.2
800S162-54	11.3	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
800S137-97	21.9	30.4	0.00	84.3	56.0
800S137-68	13.5	19.0	0.00	31.7	21.2
800S137-54	9.93	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	35.1	47.5	19.4	57.9	38.9
600S250-68	21.4	27.4	6.45	23.4	18.3
600S250-54	14.9	19.3	3.17	13.2	11.8
600S250-43	11.0	-	1.59	7.61	7.76
600S200-97	31.7	44.8	17.7	78.1	56.9
600S200-68	20.0	27.5	5.90	30.8	25.7
600S200-54	14.8	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	23.5	33.4	0.00	79.2	54.5
600S162-68	15.0	21.3	0.00	30.3	22.9
600S162-54	11.2	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	21.0	29.5	0.00	83.4	55.1
600S137-68	13.2	18.7	0.00	31.4	20.9
600S137-54	9.79	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	14.8	21.2	0.00	30.1	22.8
550S162-54	11.1	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	19.1	26.6	11.5	29.9	24.9
400S200-54	14.4	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	14.1	20.5	0.00	29.4	22.1
400S162-54	10.8	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 (continued)

Braced Column Properties
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

Section	P_n at $f=F_y$ kips [1] F_y		Maximum Effective Force, kips [2]			Section	P_n at $f=F_y$ kips [1] F_y		Maximum Effective Force, kips [2]		
	33 ksi	50 ksi	P_{web}	P_{flange}	P_{lip}		33 ksi	50 ksi	P_{web}	P_{flange}	P_{lip}
400S137-68	12.4	17.9	0.00	30.6	20.1	350S162-68	13.5	20.0	0.00	28.9	21.5
400S137-54	9.40	13.0	0.00	16.7	11.8	350S162-54	10.6	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	18.9	26.4	13.7	29.7	24.6	250S162-68	12.9	19.5	0.00	28.5	21.1
362S200-54	14.3	18.3	6.63	16.8	15.6	250S162-54	10.4	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	13.8	20.2	0.00	29.1	21.7	250S137-68	11.1	16.8	0.00	29.7	19.2
362S162-54	10.6	14.8	0.00	16.2	13.4	250S137-54	8.97	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	12.0	17.5	0.00	30.2	19.8						
362S137-54	9.23	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:

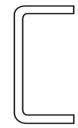
1. Axial strengths given are nominal strengths. To obtain the design strength, these values must be modified by factors of safety (ASD) or resistance factors (LRFD).

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.

Table III - 3

Braced Column Properties
SSMA Tracks
C-Sections Without Lips

$\Omega_c = 1.80$
$\phi_c = 0.85$

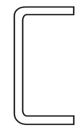


Section	P_n at $f=F_y$ kips [1] F_y		Maximum Effective Force, kips [2]		Section	P_n at $f=F_y$ kips [1] F_y		Maximum Effective Force, kips [2]	
	33 ksi	50 ksi	P_{web}	P_{flange}		33 ksi	50 ksi	P_{web}	P_{flange}
1200T200-97	29.1	37.7	5.79	19.5	600T150-97	25.7	34.3	13.3	26.7
1200T200-68	15.1	19.3	1.98	6.29	600T150-68	14.0	18.2	4.53	8.35
1200T200-54*	9.77	12.4	0.987	3.05	600T150-54	9.22	11.9	2.25	3.99
1200T150-97	27.8	36.5	5.43	28.8	600T150-43	6.06	-	1.13	1.96
1200T150-68	14.7	18.9	1.86	9.07	600T150-33	3.74	-	0.513	0.881
1200T150-54*	9.59	12.2	0.925	4.35	600T150-30	3.10	-	0.377	0.645
1200T125-97	26.1	35.2	5.24	37.3	600T150-27*	2.59	-	0.281	0.481
1200T125-68	14.4	18.6	1.79	11.5	600T125-97	24.0	33.0	12.6	35.1
1200T125-54*	9.43	12.1	0.894	5.47	600T125-68	13.7	17.9	4.28	10.8
1000T200-97	28.7	37.2	7.33	19.1	600T125-54	9.07	11.7	2.13	5.10
1000T200-68	15.0	19.2	2.50	6.15	600T125-43	5.99	-	1.07	2.49
1000T200-54	9.69	12.3	1.25	2.98	600T125-33	3.71	-	0.485	1.12
1000T200-43*	6.29	-	0.629	1.48	600T125-30	3.07	-	0.356	0.819
1000T150-97	27.4	36.0	6.80	28.4	600T125-27*	2.57	-	0.266	0.610
1000T150-68	14.6	18.8	2.32	8.93	550T200-68	14.3	18.5	5.68	5.38
1000T150-54	9.51	12.2	1.16	4.28	550T200-54	9.34	12.0	2.81	2.59
1000T150-43*	6.21	-	0.584	2.11	550T200-43	6.11	-	1.41	1.28
1000T125-97	25.7	34.8	6.54	36.9	550T200-33	3.77	-	0.638	0.573
1000T125-68	14.3	18.5	2.24	11.4	550T150-68	13.9	18.1	5.11	8.21
1000T125-54	9.36	12.0	1.11	5.39	550T150-54	9.16	11.8	2.54	3.93
1000T125-43*	6.13	-	0.562	2.64	550T150-43	6.03	-	1.28	1.93
800T200-97	28.1	36.6	9.89	18.4	550T150-33	3.73	-	0.578	0.866
800T200-68	14.8	19.0	3.37	5.93	550T150-30	3.09	-	0.424	0.634
800T200-54	9.59	12.2	1.68	2.87	550T150-27	2.58	-	0.317	0.472
800T200-43	6.24	-	0.846	1.42	550T125-68	13.6	17.8	4.81	10.6
800T200-33*	3.82	-	0.383	0.639	550T125-54	9.00	11.6	2.39	5.04
800T150-97	26.8	35.4	9.06	27.8	550T125-43	5.95	-	1.20	2.46
800T150-68	14.4	18.6	3.09	8.71	550T125-33	3.69	-	0.544	1.10
800T150-54	9.40	12.0	1.54	4.17	550T125-30	3.06	-	0.399	0.808
800T150-43	6.15	-	0.775	2.05	550T125-27	2.57	-	0.298	0.602
800T150-33*	3.78	-	0.351	0.922	400T200-68	13.7	17.8	8.40	4.53
800T125-97	25.1	34.1	8.65	36.2	400T200-54	9.04	11.7	4.13	2.18
800T125-68	14.1	18.3	2.95	11.1	400T200-43	5.96	-	2.07	1.07
800T125-54	9.25	11.9	1.47	5.29	400T200-33	3.70	-	0.935	0.482
800T125-43	6.08	-	0.740	2.58	400T150-68	13.3	17.5	8.02	7.53
800T125-33*	3.75	-	0.335	1.16	400T150-54	8.86	11.5	3.95	3.57
600T200-97	27.0	35.5	14.8	17.4	400T150-43	5.87	-	1.98	1.74
600T200-68	14.4	18.6	5.04	5.56	400T150-33	3.66	-	0.895	0.780
600T200-54	9.40	12.0	2.50	2.69	400T150-30	3.04	-	0.657	0.571
600T200-43	6.15	-	1.26	1.33	400T150-27	2.55	-	0.491	0.425
600T200-33	3.78	-	0.570	0.598					

Table III - 3 (continued)

Braced Column Properties
SSMA Tracks
C-Sections Without Lips

$\Omega_c = 1.80$
$\phi_c = 0.85$



Section	P_n at $f=F_y$ kips [1] F_y		Maximum Effective Force, kips [2]		Section	P_n at $f=F_y$ kips [1] F_y		Maximum Effective Force, kips [2]	
	33 ksi	50 ksi	P_{web}	P_{flange}		33 ksi	50 ksi	P_{web}	P_{flange}
400T125-68	13.0	17.1	7.50	10.03	350T125-68	12.6	16.8	9.11	9.70
400T125-54	8.70	11.3	3.71	4.74	350T125-54	8.54	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	13.4	17.6	9.46	4.32	250T200-68	11.9	16.5	14.8	3.68
362T200-54	8.92	11.6	4.64	2.08	250T200-54	8.36	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	13.0	17.2	9.08	7.13	250T150-68	11.6	16.1	14.4	5.91
362T150-54	8.74	11.4	4.46	3.38	250T150-54	8.18	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	12.7	16.9	8.65	9.79	250T125-68	11.2	15.8	14.1	8.17
362T125-54	8.59	11.2	4.28	4.62	250T125-54	8.03	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	13.3	17.5	9.87	4.25	162T125-33	3.24	-	2.62	0.684
350T200-54	8.88	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	13.0	17.1	9.49	6.99					
350T150-54	8.70	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. To obtain the design strength, these values must be modified by factors of safety (ASD) or resistance factors (LRFD).

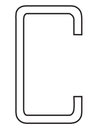
2. P_{web} and P_{flange} are the highest nominal axial compression forces at which the web and flange, respectively, are fully effective.

1.3 Nominal Axial Strength Tables - Unbraced Columns

Table III - 4

Nominal Axial Strength, P_n, kips [1]
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

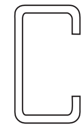


Section	KL_x ft.	F_y = 33 ksi					F_y = 55 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y= KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
12CS3.5x105	5.0	45.9	45.8	45.7	45.3	42.8	63.0	62.9	62.6	61.8	57.0
	10.0	45.2	44.9	44.4	42.8	32.5	61.6	61.0	60.1	57.0	38.4
	15.0	44.0	43.3	42.3	38.5	20.2	59.3	58.0	56.2	48.6	20.2
	20.0	42.4	41.4	39.4	32.5	12.6	56.3	54.2	50.2	38.4	12.6
	24.0	40.9	39.0	36.2	27.5		53.2	49.5	44.7	29.1	
	29.0	38.1	35.5	32.0	21.3		47.9	43.6	37.3	21.3	
	34.0	35.0	31.9	27.7	16.5		42.7	37.0	29.5	16.5	
	39.0	31.7	28.1	23.6	13.2		36.8	30.1	23.8	13.2	
12CS3.5x085	5.0	32.9	32.8	32.7	32.5	31.0	44.6	44.5	44.3	43.6	39.7
	10.0	32.4	32.2	32.0	31.0	24.4	43.4	43.0	42.2	39.7	27.9
	15.0	31.7	31.3	30.7	27.9	15.1	41.6	40.6	39.0	34.4	15.1
	20.0	30.7	29.8	28.5	24.4	9.43	39.2	37.5	35.2	27.9	9.43
	24.0	29.5	28.3	26.5	20.6		36.8	34.8	32.0	21.8	
	29.0	27.7	26.1	23.9	15.9		34.0	31.5	27.1	15.9	
	34.0	25.9	23.9	20.6	12.3		31.0	27.0	21.8	12.3	
	39.0	23.8	21.0	17.4	9.83		27.0	22.5	17.5	9.83	
12CS3.5x070	5.0	23.9	23.8	23.8	23.5	22.3	32.8	32.7	32.5	32.0	29.3
	10.0	23.5	23.3	23.1	22.2	18.2	32.0	31.6	31.1	29.2	20.3
	15.0	22.9	22.5	22.0	20.3	11.6	30.6	29.9	28.7	25.1	11.6
	20.0	22.1	21.5	20.6	18.2	7.22	28.9	27.6	25.8	20.3	7.22
	24.0	21.3	20.5	19.5	15.8		27.2	25.5	23.1	16.5	
	29.0	20.2	19.3	17.8	12.2		24.8	22.6	19.8	12.2	
	34.0	19.2	17.8	15.7	9.41		22.3	19.8	16.4	9.41	
	39.0	17.8	16.0	13.2	7.52		19.8	16.9	13.3	7.52	
10CS3.5x105	5.0	45.3	45.2	45.1	44.6	42.1	62.3	62.1	61.9	61.0	56.2
	9.0	44.5	44.2	43.8	42.5	34.3	60.9	60.3	59.5	56.9	42.0
	13.0	43.4	42.8	42.0	39.1	25.1	58.6	57.5	56.0	50.3	25.7
	17.0	41.9	41.0	39.5	34.4	16.7	55.8	53.9	51.0	42.1	16.7
	21.0	40.0	38.2	36.0	29.4		51.9	48.5	44.7	32.8	
	25.0	37.3	35.0	32.3	24.5		47.0	43.0	38.3	25.0	
	29.0	34.4	31.6	28.4	19.9		42.1	37.0	30.9	19.9	
	33.0	31.3	28.1	24.6	16.4		36.5	30.4	25.1	16.4	
10CS3.5x085	5.0	32.5	32.5	32.4	32.1	30.6	44.2	44.0	43.8	43.1	39.2
	9.0	32.1	31.9	31.7	30.8	25.3	43.0	42.5	41.9	39.7	30.2
	13.0	31.4	31.0	30.6	28.3	18.4	41.2	40.3	39.1	35.2	18.7
	17.0	30.5	29.7	28.6	25.4	12.4	38.9	37.4	35.6	30.4	12.4
	21.0	29.0	27.8	26.5	21.9		36.2	34.3	32.1	24.2	
	25.0	27.3	25.9	24.2	18.1		33.5	31.2	27.8	18.3	
	29.0	25.6	23.8	21.3	14.5		30.7	27.1	23.1	14.5	
	33.0	23.7	21.1	18.3	11.9		26.8	22.8	18.7	11.9	

Table III - 4 (continued)

Nominal Axial Strength, P_n, kips [1]
C-Sections With Lips

$\Omega_c = 1.80$
$\phi_c = 0.85$



Section	KL_x ft.	F_y = 33 ksi					F_y = 55 ksi				
		Bracing ($KL_y = KL_t$)					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
10CS3.5x070	5.0	23.7	23.6	23.5	23.3	22.0	32.5	32.4	32.3	31.7	28.9
	9.0	23.3	23.1	22.9	22.2	18.8	31.7	31.3	30.9	29.3	21.6
	13.0	22.7	22.4	22.0	20.5	13.9	30.4	29.7	28.8	25.8	14.1
	17.0	21.9	21.4	20.7	18.8	9.27	28.7	27.6	26.2	21.7	9.27
	21.0	21.0	20.3	19.5	16.5		26.8	25.2	23.3	17.8	
	25.0	20.0	19.2	18.0	13.7		24.5	22.5	20.2	13.9	
	29.0	19.0	17.8	16.1	11.0		22.1	19.8	17.2	11.0	
	33.0	17.7	16.1	14.1	8.93		19.8	17.2	14.3	8.93	
10CS3.5x065	5.0	21.1	21.1	21.0	20.8	19.6	28.9	28.8	28.7	28.2	25.7
	9.0	20.8	20.6	20.5	19.8	16.5	28.2	27.9	27.5	26.1	19.2
	13.0	20.3	20.0	19.6	18.3	12.6	27.1	26.5	25.7	23.0	12.7
	17.0	19.6	19.1	18.5	16.5	8.31	25.6	24.6	23.4	19.4	8.31
	21.0	18.7	18.1	17.2	14.7		23.8	22.5	20.8	15.8	
	25.0	17.8	16.9	15.9	12.4		21.9	20.1	17.9	12.5	
	29.0	16.7	15.8	14.4	9.85		19.7	17.6	15.4	9.85	
	33.0	15.7	14.4	12.7	8.02		17.5	15.4	12.9	8.02	
8CS3.5x105	4.0	44.5	44.5	44.4	44.0	42.1	61.6	61.4	61.2	60.6	56.8
	7.0	43.9	43.6	43.3	42.3	36.5	60.3	59.8	59.2	57.3	46.1
	10.0	42.9	42.3	41.8	40.0	28.9	58.3	57.4	56.3	52.8	32.6
	14.0	41.1	40.2	39.0	34.9	19.5	54.9	53.1	50.7	43.4	19.5
	17.0	39.4	37.7	35.9	30.7	14.5	51.4	48.2	45.1	35.9	14.5
	20.0	36.9	34.8	32.6	26.4	11.3	46.9	43.1	39.8	28.0	11.3
	24.0	33.4	30.6	28.0	21.0		40.9	35.8	30.8	21.0	
	27.0	30.5	27.4	24.6	17.4		35.7	29.8	25.3	17.4	
8CS3.5x085	4.0	32.1	32.1	32.0	31.8	30.6	43.8	43.6	43.5	43.0	39.9
	7.0	31.7	31.6	31.4	30.8	26.7	42.8	42.3	41.9	40.3	32.8
	10.0	31.1	30.8	30.5	29.0	21.6	41.2	40.4	39.5	36.6	24.0
	14.0	30.0	29.2	28.4	25.8	14.1	38.4	37.0	35.5	31.3	14.1
	17.0	28.7	27.6	26.5	23.1	10.5	35.9	34.2	32.4	26.2	10.5
	20.0	27.2	25.8	24.5	19.7	8.28	33.5	31.3	28.8	20.8	8.28
	24.0	25.0	23.3	21.2	15.5		30.0	26.5	23.2	15.5	
	27.0	23.3	20.8	18.5	12.8		26.5	22.6	19.1	12.8	
8CS3.5x070	4.0	23.4	23.4	23.3	23.2	22.2	32.3	32.2	32.1	31.7	29.5
	7.0	23.1	23.0	22.8	22.3	19.6	31.6	31.3	30.9	29.8	23.9
	10.0	22.6	22.3	22.0	21.0	16.3	30.5	29.9	29.2	27.1	17.6
	14.0	21.7	21.2	20.6	19.1	10.6	28.5	27.4	26.3	22.6	10.6
	17.0	20.8	20.1	19.5	17.3	7.84	26.7	25.2	23.7	19.2	7.84
	20.0	19.9	19.1	18.3	15.2	6.13	24.6	22.8	20.9	15.8	6.13
	24.0	18.7	17.5	16.1	11.8		21.7	19.5	17.4	11.8	
	27.0	17.5	16.0	14.3	9.76		19.6	17.1	14.7	9.76	

Table III - 4 (continued)

Ω_c = 1.80
ϕ_c = 0.85

Nominal Axial Strength, P_n, kips [1]
C-Sections With Lips

Section	KL_x ft.	F_y = 33 ksi Bracing ($KL_y = KL_t$)					F_y = 55 ksi 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	20.9	20.9	20.9	20.7	19.8	28.8	28.7	28.6	28.2	26.3
	7.0	20.6	20.5	20.4	19.9	17.4	28.1	27.9	27.6	26.6	21.3
	11.0	20.0	19.7	19.4	18.3	13.4	26.7	26.1	25.5	23.2	13.8
	14.0	19.4	18.9	18.4	16.8	9.54	25.4	24.4	23.4	20.2	9.54
	17.0	18.6	18.0	17.3	15.4	7.02	23.8	22.5	21.1	16.9	7.02
	21.0	17.4	16.5	15.7	12.9	5.09	21.4	19.6	17.8	13.2	5.09
	24.0	16.4	15.5	14.5	10.7		19.4	17.3	15.5	10.7	
	27.0	15.6	14.3	13.0	8.79		17.4	15.3	13.3	8.79	
8CS3.5x059	4.0	18.1	18.0	18.0	17.9	17.1	24.7	24.7	24.6	24.3	22.6
	7.0	17.8	17.7	17.6	17.2	15.0	24.2	24.0	23.7	22.8	18.3
	11.0	17.3	17.0	16.8	15.9	11.6	23.0	22.5	21.9	20.0	11.9
	14.0	16.7	16.4	15.9	14.6	8.31	21.9	21.1	20.2	17.4	8.31
	17.0	16.1	15.5	15.0	13.1	6.09	20.5	19.4	18.2	14.6	6.09
	21.0	15.1	14.3	13.5	11.2	4.39	18.4	16.9	15.4	11.4	4.39
	24.0	14.2	13.3	12.3	9.33		16.8	15.0	13.2	9.33	
	27.0	13.3	12.2	11.2	7.69		15.0	13.0	11.5	7.69	
8CS2.5x105	4.0	39.2	39.1	39.0	38.5	35.4	59.7	59.4	59.1	57.8	50.3
	7.0	38.4	38.1	37.7	36.3	28.2	57.7	56.9	55.9	52.3	34.3
	10.0	37.3	36.7	35.9	33.2	19.7	54.8	53.3	51.4	45.1	19.9
	13.0	35.7	34.8	33.6	29.5	12.5	51.0	48.8	45.9	37.1	12.5
	16.0	33.9	32.6	30.9	25.4		46.7	43.7	40.0	28.8	
	20.0	31.1	29.2	27.0	19.7		40.4	36.4	32.0	19.9	
	23.0	28.7	26.5	24.0	15.7		35.4	31.0	26.3	15.7	
	26.0	26.3	23.8	21.1	12.5		30.5	25.8	21.7	12.5	
8CS2.5x085	4.0	29.5	29.4	29.3	28.9	26.7	43.7	43.5	43.3	42.6	38.0
	7.0	28.9	28.7	28.4	27.2	21.2	42.6	42.2	41.6	39.4	25.8
	10.0	28.1	27.6	27.0	24.9	14.9	41.1	40.3	38.7	33.8	15.0
	13.0	26.9	26.2	25.2	22.1	9.70	38.6	36.8	34.5	27.6	9.70
	16.0	25.5	24.5	23.2	19.1		35.3	32.9	30.0	21.7	
	20.0	23.4	21.9	20.2	14.9		30.5	27.4	23.8	15.0	
	23.0	21.6	19.9	17.9	11.9		26.7	23.3	19.4	11.9	
	26.0	19.8	17.8	15.6	9.70		23.0	19.3	16.0	9.70	
8CS2.5x070	4.0	22.8	22.7	22.7	22.3	20.6	31.2	31.1	30.9	30.5	28.2
	7.0	22.4	22.2	21.9	21.0	16.3	30.5	30.3	29.9	28.8	19.9
	10.0	21.7	21.3	20.8	19.1	11.5	29.6	29.1	28.5	25.9	11.6
	13.0	20.8	20.2	19.4	16.9	7.50	28.5	27.6	26.3	21.1	7.50
	16.0	19.7	18.9	17.8	14.5		26.8	25.5	23.1	16.4	
	20.0	18.0	16.9	15.5	11.4		23.6	21.1	18.2	11.5	
	23.0	16.7	15.3	13.7	9.19		20.7	17.9	14.8	9.19	
	26.0	15.3	13.7	11.9	7.50		17.8	14.9	12.1	7.50	

Table III - 4 (continued)

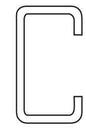
Nominal Axial Strength, P_n, kips [1]
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

Section	KL_x ft.	F_y = 33 ksi, Bracing ($KL_y = KL_t$)					F_y = 55 ksi, 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
8CS2.5x065	4.0	20.7	20.6	20.5	20.3	18.6	27.5	27.4	27.3	26.8	24.6
	7.0	20.3	20.1	19.9	19.0	14.8	26.8	26.5	26.2	25.1	18.1
	10.0	19.6	19.3	18.9	17.3	10.4	25.9	25.5	24.9	23.0	10.5
	13.0	18.8	18.3	17.6	15.3	6.78	24.9	24.2	23.3	19.1	6.78
	17.0	17.5	16.7	15.6	12.4		23.2	21.9	19.8	13.4	
	20.0	16.4	15.3	14.0	10.3		21.4	19.2	16.5	10.3	
	23.0	15.1	13.9	12.4	8.31		18.8	16.2	13.4	8.31	
	26.0	13.8	12.4	10.7	6.78		16.2	13.4	10.9	6.78	
8CS2.5x059	4.0	17.9	17.9	17.9	17.7	16.4	23.9	23.8	23.7	23.2	20.9
	7.0	17.7	17.6	17.4	16.8	13.0	23.3	23.0	22.7	21.5	15.9
	10.0	17.3	17.0	16.6	15.2	9.11	22.3	21.9	21.2	19.3	9.21
	13.0	16.6	16.1	15.5	13.4	5.96	21.2	20.4	19.6	16.7	5.96
	17.0	15.4	14.7	13.7	10.8		19.5	18.6	17.4	11.7	
	20.0	14.4	13.5	12.3	8.92		18.3	16.9	14.5	8.99	
	23.0	13.3	12.2	10.8	7.21		16.5	14.3	11.7	7.21	
	26.0	12.2	10.9	9.39	5.96		14.2	11.8	9.56	5.96	
6CS2.5x105	3.0	37.9	37.8	37.7	37.4	35.4	58.4	58.2	57.9	57.1	52.3
	5.0	37.2	37.0	36.8	35.9	31.2	56.8	56.2	55.5	53.3	42.3
	8.0	35.7	35.1	34.5	32.5	23.7	53.0	51.5	50.1	45.4	26.8
	10.0	34.4	33.5	32.6	29.8	18.6	49.7	47.6	45.6	39.3	19.0
	13.0	31.9	30.6	29.3	25.4	11.4	44.0	41.0	38.2	30.2	11.4
	15.0	30.1	28.4	26.9	22.4	8.56	39.8	36.3	33.1	24.5	8.56
	18.0	27.0	25.0	23.1	17.7		33.4	29.3	25.7	17.9	
	20.0	24.9	22.6	20.3	14.9		29.1	24.8	21.4	14.9	
6CS2.5x085	3.0	28.8	28.8	28.7	28.4	26.9	43.0	42.8	42.7	42.2	39.7
	5.0	28.4	28.2	28.0	27.3	23.6	42.1	41.8	41.4	40.3	31.8
	8.0	27.2	26.8	26.3	24.7	17.6	40.2	39.3	38.1	34.4	19.4
	10.0	26.2	25.5	24.8	22.6	13.8	37.9	36.3	34.7	29.6	13.8
	13.0	24.4	23.3	22.3	19.2	9.21	33.5	31.2	29.0	22.5	9.21
	15.0	23.0	21.7	20.5	16.8	6.96	30.4	27.7	25.1	18.1	6.96
	18.0	20.7	19.1	17.6	13.5		25.5	22.3	19.5	13.6	
	20.0	19.1	17.3	15.7	11.4		22.4	18.9	16.3	11.4	
6CS2.5x070	3.0	22.4	22.4	22.3	22.1	20.9	30.8	30.7	30.6	30.3	28.8
	5.0	22.1	21.9	21.8	21.2	18.3	30.3	30.1	29.9	29.1	24.6
	8.0	21.2	20.8	20.4	19.2	13.4	29.1	28.6	28.2	26.2	14.6
	10.0	20.4	19.9	19.3	17.5	10.3	28.1	27.4	26.4	22.9	10.3
	13.0	19.0	18.2	17.3	14.8	6.94	25.8	24.3	22.5	17.2	6.94
	15.0	17.9	16.9	15.9	12.9	5.54	23.7	21.5	19.4	13.8	5.54
	18.0	16.1	14.9	13.6	10.3		20.0	17.4	15.1	10.3	
	20.0	14.9	13.5	12.1	8.69		17.5	14.8	12.6	8.69	

Table III - 4 (continued)

Ω_c = 1.80
ϕ_c = 0.85

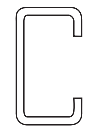


Nominal Axial Strength, P_n, kips [1]
C-Sections With Lips

Section	KL_x ft.	F_y = 33 ksi Bracing ($KL_y = KL_t$)					F_y = 55 ksi 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	20.4	20.3	20.3	20.1	19.0	27.2	27.1	27.0	26.7	25.2
	5.0	20.0	19.9	19.8	19.3	16.6	26.6	26.4	26.2	25.5	22.0
	8.0	19.2	18.9	18.6	17.4	12.1	25.5	25.1	24.6	23.2	13.1
	10.0	18.5	18.0	17.5	15.9	9.22	24.6	24.0	23.4	20.8	9.22
	13.0	17.2	16.5	15.7	13.4	6.18	23.0	21.8	20.4	15.6	6.18
	15.0	16.3	15.3	14.4	11.7	4.98	21.5	19.6	17.7	12.5	4.98
	18.0	14.7	13.5	12.4	9.25		18.1	15.8	13.6	9.26	
	20.0	13.5	12.2	11.0	7.82		15.9	13.4	11.4	7.82	
6CS2.5x059	3.0	17.7	17.7	17.7	17.5	16.8	23.7	23.6	23.5	23.2	21.6
	5.0	17.5	17.4	17.3	17.0	14.6	23.2	23.0	22.8	22.0	18.6
	8.0	17.0	16.7	16.4	15.4	10.6	22.0	21.5	21.0	19.5	11.4
	10.0	16.4	15.9	15.5	14.0	7.99	20.9	20.3	19.6	17.9	7.99
	13.0	15.2	14.6	13.9	11.8	5.33	19.3	18.5	17.7	13.7	5.33
	15.0	14.3	13.5	12.7	10.2	4.30	18.3	17.2	15.6	10.9	4.30
	18.0	12.9	11.9	10.9	8.08		16.0	13.9	12.0	8.08	
	20.0	12.0	10.8	9.69	6.81		14.0	11.8	10.0	6.81	
4CS2.5x105	2.0	34.1	34.0	33.9	33.7	32.3	55.5	55.3	55.1	54.6	51.3
	4.0	33.0	32.7	32.4	31.5	27.1	52.9	52.1	51.5	49.4	38.6
	6.0	31.3	30.6	30.1	28.4	21.0	48.8	47.2	45.8	41.5	25.2
	7.0	30.2	29.4	28.6	26.5	18.1	46.1	44.0	42.2	37.1	19.7
	9.0	27.7	26.5	25.4	22.6	13.2	40.0	37.0	34.6	28.4	13.2
	11.0	24.9	23.3	22.0	18.6	9.84	33.5	29.9	27.1	20.5	9.84
	12.0	23.4	21.6	20.2	16.7	8.75	30.3	26.4	23.6	17.5	8.75
	14.0	20.4	18.3	16.8	13.2	7.18	24.0	20.0	17.6	13.2	7.18
4CS2.5x085	2.0	27.3	27.3	27.2	27.0	26.0	41.5	41.4	41.3	40.9	39.2
	4.0	26.5	26.3	26.1	25.5	21.8	40.1	39.7	39.3	38.3	30.2
	6.0	25.3	24.8	24.4	22.9	16.5	37.9	36.7	35.7	32.6	19.3
	7.0	24.5	23.8	23.2	21.4	13.9	36.0	34.5	33.2	29.3	14.8
	9.0	22.5	21.5	20.6	18.1	9.64	31.8	29.6	27.7	22.6	9.64
	11.0	20.3	18.9	17.8	14.8	7.03	27.1	24.3	22.0	16.1	7.03
	12.0	19.1	17.6	16.3	13.2	6.18	24.8	21.5	19.1	13.7	6.18
	14.0	16.7	14.9	13.5	10.3	4.98	19.7	16.3	14.2	10.3	4.98
4CS2.5x070	2.0	21.6	21.6	21.5	21.4	20.6	30.0	29.9	29.8	29.6	28.6
	4.0	21.0	20.8	20.7	20.1	17.4	29.1	28.9	28.7	28.0	23.6
	6.0	20.1	19.7	19.3	18.3	13.3	27.9	27.4	26.9	25.2	15.2
	7.0	19.5	18.9	18.5	17.1	11.1	27.1	26.4	25.6	23.1	11.6
	9.0	18.1	17.3	16.6	14.7	7.40	24.9	23.4	21.9	17.9	7.40
	11.0	16.5	15.4	14.5	12.0	5.28	21.6	19.4	17.6	13.0	5.28
	12.0	15.6	14.4	13.5	10.7	4.60	19.7	17.3	15.5	11.0	4.60
	14.0	13.8	12.3	11.1	8.24	3.64	16.1	13.5	11.6	8.24	3.64

Table III - 4 (continued)

Ω_c = 1.80
ϕ_c = 0.85



Nominal Axial Strength, P_n, kips [1]
C-Sections With Lips

Section	KL_x ft.	F_y = 33 ksi					F_y = 55 ksi				
		Bracing (KL_y = KL_t)					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
4CS2.5x065	2.0	19.7	19.7	19.6	19.5	18.8	26.6	26.5	26.4	26.2	25.1
	4.0	19.2	19.0	18.9	18.4	15.8	25.6	25.4	25.2	24.6	21.2
	6.0	18.3	18.0	17.6	16.7	12.1	24.5	24.0	23.6	22.4	13.7
	7.0	17.8	17.3	16.9	15.6	10.2	23.8	23.2	22.7	20.9	10.6
	9.0	16.5	15.8	15.2	13.4	6.71	22.2	21.1	20.0	16.3	6.71
	11.0	15.1	14.1	13.3	11.1	4.76	19.7	17.7	16.1	11.9	4.76
	12.0	14.3	13.2	12.3	9.88	4.13	18.1	15.9	14.2	10.2	4.13
	14.0	12.6	11.4	10.3	7.58	3.25	14.8	12.4	10.8	7.58	3.25
4CS2.5x059	2.0	17.2	17.2	17.2	17.1	16.6	23.2	23.1	23.1	22.9	21.8
	4.0	16.9	16.8	16.7	16.3	14.0	22.4	22.1	21.9	21.2	18.0
	6.0	16.3	15.9	15.7	14.8	10.7	21.1	20.6	20.1	18.9	12.0
	7.0	15.8	15.4	15.0	13.9	8.98	20.3	19.7	19.1	17.8	9.26
	9.0	14.7	14.0	13.5	11.9	5.94	18.7	18.0	17.4	14.4	5.94
	11.0	13.4	12.5	11.8	9.83	4.18	17.3	15.7	14.3	10.5	4.18
	12.0	12.7	11.7	10.9	8.82	3.61	16.1	14.1	12.6	9.04	3.61
	14.0	11.3	10.1	9.22	6.81	2.81	13.2	11.0	9.59	6.81	2.81

Note:

1. Axial strengths given are nominal strengths. To obtain the design strength, these values must be modified by factors of safety (ASD) or resistance factors (LRFD).

Table III - 5

Nominal Axial Strength, P_n, kips [1]
SSMA Studs
C-Sections With Lips

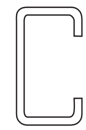
$\Omega_c = 1.80$
$\phi_c = 0.85$

Section	KL_x ft.	F_y = 33 ksi, Bracing ($KL_y = KL_t$)					F_y = 50 ksi, 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	33.8	33.3	32.7	31.2	24.0	46.6	45.6	44.4	41.2	27.6
	9.0	33.2	31.7	30.0	25.6	11.8	45.5	42.3	38.8	30.6	11.8
	14.0	32.1	28.7	25.1	17.2		43.2	36.2	29.6	17.3	
	18.0	31.0	25.6	20.6	11.8		40.8	30.6	21.9	11.8	
	23.0	29.2	21.5	15.1			37.2	23.3	15.1		
	27.0	27.5	18.1	11.8			34.0	18.3	11.8		
	32.0	25.3	14.1				29.8	14.1			
	36.0	23.3	11.8				26.5	11.8			
1200S200-68	5.0	20.6	20.3	20.0	19.1	14.8	28.0	27.6	27.1	25.5	17.3
	9.0	20.3	19.4	18.4	15.8	7.38	27.5	26.1	24.1	19.1	7.38
	14.0	19.6	17.6	15.4	10.8		26.5	22.5	18.5	10.9	
	18.0	18.8	15.8	12.8	7.38		25.1	19.1	13.8	7.38	
	23.0	17.7	13.3	9.44			22.8	14.7	9.44		
	27.0	16.7	11.2	7.38			20.8	11.5	7.38		
	32.0	15.3	8.84				18.2	8.84			
	36.0	14.2	7.38				16.2	7.38			
1200S200-54*	5.0	15.1	14.9	14.6	14.0	10.9	19.1	18.9	18.7	18.0	12.8
	9.0	14.8	14.2	13.5	11.6	5.41	18.9	18.2	17.4	14.1	5.41
	14.0	14.3	12.9	11.3	7.90		18.4	16.6	13.7	8.02	
	18.0	13.8	11.6	9.38	5.41		17.8	14.1	10.2	5.41	
	23.0	13.0	9.75	6.95			16.7	10.9	6.95		
	27.0	12.2	8.27	5.41			15.3	8.50	5.41		
	32.0	11.2	6.50	4.16			13.4	6.50	4.16		
	36.0	10.3	5.41				11.8	5.41			
1000S250-97	4.0	36.7	36.6	36.4	35.8	32.5	49.0	48.8	48.5	47.3	42.0
	8.0	36.0	35.7	34.9	32.5	22.1	47.8	47.2	45.8	42.0	23.6
	12.0	34.9	34.1	32.5	27.6	12.3	45.8	44.6	42.0	33.2	12.3
	16.0	33.4	32.1	29.4	22.1		43.4	41.5	36.5	23.6	
	20.0	31.5	29.8	25.8	16.5		40.6	37.2	30.0	16.5	
	24.0	29.4	27.1	22.1	12.3		36.5	32.3	23.6	12.3	
	28.0	27.1	24.3	18.3			32.2	27.4	18.4		
	32.0	24.6	21.5	14.9			27.9	22.7	14.9		
1000S250-68	4.0	21.9	21.8	21.7	21.4	19.9	27.9	27.8	27.7	27.2	24.8
	8.0	21.5	21.3	21.0	19.9	13.9	27.3	27.1	26.5	24.8	15.0
	12.0	21.0	20.6	19.9	17.3	7.84	26.5	25.9	24.8	20.5	7.84
	16.0	20.2	19.6	18.3	13.9		25.4	24.4	22.0	15.0	
	20.0	19.4	18.4	16.2	10.5		24.0	22.1	19.0	10.5	
	24.0	18.2	16.7	13.9	7.84		21.9	19.7	15.0	7.84	
	28.0	16.8	14.9	11.6	6.13		19.8	16.8	11.7	6.13	
	32.0	15.2	13.2	9.46			17.4	13.8	9.46		

Table III - 5 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

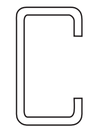


Section	KL_x ft.	F_y = 33 ksi Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	F_y = 50 ksi Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
1000S250-54	4.0	15.1	15.1	15.0	14.8	14.0	19.5	19.5	19.3	19.0	17.1
	8.0	14.9	14.8	14.6	14.0	10.3	19.1	18.9	18.5	17.1	11.2
	12.0	14.6	14.4	14.0	12.6	5.80	18.4	18.0	17.1	14.4	5.80
	16.0	14.2	13.9	13.2	10.3		17.5	16.7	15.3	11.2	
	20.0	13.7	13.2	12.0	7.80		16.4	15.3	13.6	7.80	
	24.0	13.1	12.2	10.3	5.80		15.1	13.9	11.2	5.80	
	28.0	12.3	10.9	8.61	4.53		14.0	12.3	8.73	4.53	
	32.0	11.2	9.61	7.02			12.6	10.1	7.02		
1000S250-43*	4.0	11.1	11.1	11.0	10.9	10.2					
	8.0	10.9	10.9	10.7	10.2	7.68					
	12.0	10.7	10.5	10.2	9.09	4.31					
	16.0	10.3	10.0	9.49	7.68						
	20.0	9.91	9.46	8.65	5.82						
	24.0	9.40	8.79	7.68	4.31						
	28.0	8.84	8.05	6.42	3.35						
	32.0	8.21	7.10	5.23							
1000S200-97	4.0	33.4	33.2	32.8	31.9	27.3	46.3	45.8	45.1	43.1	34.0
	8.0	32.7	31.9	30.6	27.3	14.7	44.9	43.1	40.5	34.0	14.7
	12.0	31.7	29.9	27.3	21.1		42.7	39.0	34.0	22.9	
	16.0	30.2	27.3	23.2	14.7		39.8	34.0	26.6	14.7	
	19.0	29.0	25.1	20.0	11.2		37.2	29.9	21.1	11.2	
	23.0	27.1	21.9	15.7			33.5	24.3	15.7		
	27.0	24.9	18.6	12.2			29.6	19.1	12.2		
	31.0	22.7	15.4	9.74			25.7	15.4	9.74		
1000S200-68	4.0	20.5	20.4	20.2	19.6	16.9	27.9	27.7	27.4	26.6	21.3
	8.0	20.1	19.6	18.8	16.9	9.30	27.3	26.6	25.2	21.3	9.30
	12.0	19.4	18.4	16.9	13.1		26.4	24.3	21.3	14.5	
	16.0	18.5	16.9	14.4	9.30		24.6	21.3	16.8	9.30	
	20.0	17.5	15.1	11.8	6.56		22.5	17.9	12.4	6.56	
	23.0	16.6	13.6	9.93			20.7	15.4	9.95		
	27.0	15.3	11.7	7.74			18.3	12.2	7.74		
	31.0	13.9	9.78	6.23			15.8	9.78	6.23		
1000S200-54	4.0	15.0	14.9	14.8	14.4	12.4	19.0	19.0	18.8	18.4	15.7
	8.0	14.7	14.4	13.8	12.4	6.86	18.7	18.4	17.8	15.7	6.86
	12.0	14.2	13.5	12.4	9.66		18.2	17.5	15.7	10.8	
	16.0	13.6	12.4	10.6	6.86		17.6	15.7	12.4	6.86	
	20.0	12.8	11.1	8.71	4.83		16.5	13.2	9.19	4.83	
	23.0	12.1	10.0	7.32			15.2	11.4	7.34		
	27.0	11.1	8.59	5.70			13.4	9.01	5.70		
	31.0	10.1	7.21	4.58			11.6	7.22	4.58		

Table III - 5 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85



Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					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					
	8.0	10.7	10.5	10.2	9.18	5.09					
	12.0	10.4	9.97	9.18	7.16						
	16.0	10.00	9.18	7.87	5.09						
	20.0	9.47	8.21	6.46	3.55						
	23.0	8.96	7.43	5.42							
	27.0	8.24	6.37	4.21							
	31.0	7.47	5.34	3.37							
800S200-97	4.0	32.6	32.5	32.2	31.4	27.1	45.3	45.1	44.6	42.8	34.3
	7.0	31.9	31.6	30.8	28.4	18.2	43.9	43.2	41.6	36.7	18.8
	10.0	30.9	30.3	28.8	24.3	10.2	41.9	40.5	37.5	29.1	10.2
	13.0	29.6	28.6	26.2	19.7		39.2	37.2	32.6	21.2	
	16.0	28.1	26.6	23.3	15.1		36.1	33.4	27.3	15.1	
	19.0	26.3	24.5	20.2	11.3		32.7	29.4	22.0	11.3	
	22.0	24.3	22.2	17.1	8.46		29.1	25.4	17.4	8.46	
	25.0	22.3	19.9	14.1			25.5	21.5	14.1		
800S200-68	4.0	20.2	20.1	20.0	19.4	16.9	27.6	27.4	27.2	26.5	21.6
	7.0	19.8	19.5	19.1	17.7	11.5	26.9	26.6	26.0	23.1	12.1
	10.0	19.1	18.7	17.9	15.2	6.80	26.1	25.2	23.5	18.4	6.80
	13.0	18.3	17.7	16.4	12.5		24.4	23.1	20.5	13.6	
	16.0	17.4	16.4	14.6	9.72		22.5	20.7	17.3	9.74	
	19.0	16.3	15.1	12.8	7.39		20.4	18.1	14.1	7.39	
	22.0	15.1	13.7	10.9	5.83		18.1	15.6	11.2	5.83	
	25.0	13.8	12.2	9.12			15.9	13.1	9.12		
800S200-54	4.0	14.9	14.8	14.7	14.3	12.5	18.9	18.8	18.7	18.3	16.0
	7.0	14.5	14.4	14.1	13.0	8.55	18.6	18.4	18.1	16.9	9.00
	10.0	14.1	13.8	13.2	11.2	5.06	18.1	17.8	17.1	13.7	5.06
	13.0	13.5	13.0	12.1	9.23		17.5	16.8	15.3	10.1	
	16.0	12.8	12.1	10.8	7.22		16.5	15.2	12.9	7.25	
	19.0	12.0	11.0	9.46	5.49		15.0	13.3	10.5	5.49	
	22.0	11.1	9.98	8.10	4.34		13.4	11.4	8.35	4.34	
	26.0	9.84	8.53	6.37			11.2	8.97	6.37		
800S200-43	4.0	10.8	10.8	10.7	10.5	9.29					
	7.0	10.6	10.5	10.4	9.68	6.38					
	10.0	10.3	10.1	9.79	8.38	3.76					
	13.0	9.97	9.64	9.00	6.88						
	16.0	9.50	8.96	8.06	5.39						
	19.0	8.89	8.19	7.05	4.09						
	23.0	7.99	7.10	5.71	3.00						
	26.0	7.29	6.28	4.75							

Table III - 5 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Studs
C-Sections With Lips

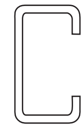
$\Omega_c = 1.80$
$\phi_c = 0.85$

Section	KL_x ft.	F_y = 33 ksi Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	F_y = 50 ksi Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
800S200-33*	4.0	7.04	7.02	6.99	6.85	6.16					
	7.0	6.93	6.87	6.77	6.37	4.50					
	10.0	6.76	6.64	6.44	5.68	2.66					
	13.0	6.54	6.34	6.01	4.81						
	16.0	6.26	5.98	5.50	3.83						
	19.0	5.94	5.56	4.91	2.89						
	23.0	5.45	4.92	4.06	2.11						
	26.0	5.04	4.42	3.37							
800S162-97	4.0	24.1	23.9	23.5	22.4	17.4	33.7	33.1	32.3	30.1	20.4
	7.0	23.6	22.9	21.7	18.8	8.88	32.6	31.0	28.7	23.0	8.88
	10.0	22.8	21.4	19.3	14.4		30.9	28.0	23.9	15.3	
	13.0	21.8	19.5	16.4	10.0		28.8	24.3	18.7	10.0	
	16.0	20.6	17.4	13.4	7.13		26.4	20.4	13.8	7.13	
	19.0	19.2	15.1	10.5			23.7	16.6	10.5		
	22.0	17.7	12.9	8.23			21.0	13.1	8.23		
	25.0	16.1	10.7	6.66			18.2	10.7	6.66		
800S162-68	4.0	15.1	15.0	14.7	14.1	11.1	21.3	21.0	20.5	19.1	13.2
	7.0	14.8	14.3	13.7	11.9	5.83	20.6	19.7	18.3	14.8	5.83
	10.0	14.3	13.5	12.2	9.25		19.5	17.8	15.3	10.1	
	13.0	13.6	12.3	10.5	6.57		18.2	15.6	12.1	6.57	
	16.0	12.9	11.1	8.64	4.71		16.6	13.2	9.06	4.71	
	19.0	12.0	9.71	6.85			14.9	10.8	6.85		
	22.0	11.1	8.34	5.41			13.2	8.61	5.41		
	25.0	10.1	7.00	4.41			11.5	7.00	4.41		
800S162-54	4.0	11.2	11.1	10.9	10.5	8.25	15.3	15.2	14.9	14.2	9.95
	7.0	11.0	10.7	10.2	8.89	4.38	14.9	14.5	13.7	11.1	4.38
	10.0	10.6	10.0	9.09	6.92		14.4	13.4	11.5	7.60	
	13.0	10.1	9.19	7.82	4.94		13.6	11.7	9.15	4.94	
	16.0	9.54	8.25	6.47	3.53		12.4	9.95	6.85	3.53	
	19.0	8.89	7.26	5.16			11.1	8.17	5.16		
	22.0	8.19	6.25	4.06			9.83	6.51	4.06		
	25.0	7.45	5.27	3.30			8.51	5.27	3.30		
800S162-43	4.0	8.42	8.35	8.22	7.86	6.21					
	7.0	8.23	8.01	7.64	6.68	3.30					
	10.0	7.95	7.52	6.83	5.21						
	13.0	7.58	6.91	5.88	3.73						
	16.0	7.14	6.21	4.87	2.65						
	19.0	6.65	5.46	3.89							
	22.0	6.12	4.70	3.05							
	25.0	5.56	3.97	2.48							

Table III - 5 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

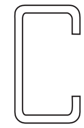


Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y= KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
800S162-33*	4.0	5.81	5.77	5.70	5.52	4.47					
	7.0	5.71	5.60	5.41	4.82	2.36					
	10.0	5.56	5.34	4.92	3.74						
	13.0	5.37	4.97	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	31.4	31.3	31.2	30.8	28.6	44.2	44.0	43.8	42.9	38.3
	5.0	30.8	30.6	30.3	29.3	23.7	43.0	42.5	41.9	39.8	29.0
	8.0	29.6	29.0	28.4	26.1	14.6	40.3	39.3	37.9	33.5	14.7
	10.0	28.4	27.6	26.7	23.7	9.40	38.0	36.5	34.6	28.8	9.40
	12.0	27.1	26.0	24.8	20.9		35.4	33.3	31.0	23.9	
	15.0	24.8	23.4	21.8	16.3		31.0	28.3	25.5	16.7	
	17.0	23.1	21.4	19.7	13.0		27.9	24.9	21.9	13.0	
	20.0	20.4	18.5	16.4	9.40		23.2	19.9	17.0	9.40	
600S200-68	3.0	19.8	19.7	19.7	19.4	18.0	27.2	27.1	27.0	26.6	24.2
	5.0	19.4	19.3	19.1	18.5	15.2	26.7	26.5	26.2	25.1	18.7
	8.0	18.6	18.3	17.9	16.4	9.94	25.5	24.8	23.9	21.0	10.0
	10.0	17.9	17.4	16.8	14.8	6.90	24.1	23.0	21.8	17.8	6.90
	13.0	16.7	15.9	15.0	12.2		21.5	20.0	18.2	13.3	
	15.0	15.7	14.8	13.7	10.5		19.7	17.9	15.9	10.8	
	17.0	14.7	13.6	12.3	8.92		17.7	15.7	13.6	8.92	
	20.0	13.1	11.7	10.3	6.90		14.8	12.6	10.5	6.90	
600S200-54	3.0	14.7	14.6	14.6	14.4	13.4	18.7	18.7	18.6	18.4	17.4
	5.0	14.4	14.3	14.2	13.7	11.2	18.5	18.3	18.2	17.7	13.7
	8.0	13.8	13.6	13.2	12.1	7.48	17.9	17.6	17.3	15.5	7.58
	10.0	13.3	12.9	12.4	10.9	5.23	17.3	16.8	16.2	13.1	5.23
	13.0	12.4	11.8	11.1	8.88		16.0	14.9	13.5	9.67	
	15.0	11.7	10.9	10.1	7.59		14.6	13.3	11.7	7.73	
	17.0	10.9	10.0	9.05	6.37		13.2	11.6	9.95	6.37	
	20.0	9.69	8.67	7.55	4.98		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						

Table III - 5 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Studs
C-Sections With Lips

$\Omega_c = 1.80$
$\phi_c = 0.85$



Section	KL_x ft.	F_y = 33 ksi, Bracing ($KL_y = KL_t$)					F_y = 50 ksi, 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
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	23.3	23.2	23.1	22.5	19.7	32.9	32.7	32.4	31.2	25.5
	5.0	22.8	22.6	22.3	20.8	14.4	31.9	31.5	30.7	27.7	15.9
	7.0	22.2	21.8	21.1	18.5	8.65	30.6	29.8	28.4	23.2	8.65
	10.0	20.9	20.3	18.9	14.4		28.0	26.6	24.0	15.9	
	12.0	19.9	19.0	17.2	11.5		25.9	24.2	20.8	11.7	
	14.0	18.7	17.6	15.3	8.65		23.7	21.6	17.5	8.65	
	17.0	16.8	15.5	12.5	5.86		20.1	17.7	12.9	5.86	
	19.0	15.5	14.0	10.6			17.7	15.3	10.6		
600S162-68	3.0	14.8	14.8	14.7	14.4	12.7	21.0	20.9	20.7	20.0	16.6
	5.0	14.5	14.4	14.2	13.3	9.46	20.4	20.1	19.7	17.9	10.6
	8.0	13.9	13.6	13.1	11.2	4.87	19.1	18.4	17.4	13.6	4.87
	10.0	13.4	12.9	12.2	9.46		17.9	17.0	15.6	10.6	
	12.0	12.7	12.1	11.2	7.74		16.6	15.4	13.6	7.91	
	15.0	11.6	10.7	9.46	5.43		14.4	12.9	10.6	5.43	
	17.0	10.8	9.81	8.31	4.39		12.9	11.2	8.71	4.39	
	19.0	9.93	8.86	7.18			11.4	9.61	7.22		
600S162-54	3.0	11.1	11.0	11.0	10.7	9.50	15.1	15.1	15.0	14.7	12.5
	5.0	10.9	10.8	10.6	9.98	7.13	14.8	14.7	14.5	13.5	8.06
	8.0	10.4	10.1	9.77	8.38	3.72	14.2	13.8	13.0	10.3	3.72
	10.0	9.97	9.61	9.07	7.13		13.5	12.7	11.6	8.06	
	12.0	9.48	9.00	8.31	5.86		12.5	11.5	10.2	6.03	
	15.0	8.65	7.99	7.09	4.14		10.8	9.60	7.99	4.14	
	17.0	8.04	7.28	6.28	3.36		9.69	8.33	6.63	3.36	
	19.0	7.41	6.57	5.45			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 - 5 (continued)

Nominal Axial Strength, P_n, kips [1]

SSMA Studs

C-Sections With Lips

Ω_c = 1.80

ϕ_c = 0.85

Section	KL_x ft.	F_y = 33 ksi, Bracing ($KL_y = KL_t$)					F_y = 50 ksi, 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
600S162-33	3.0	5.79	5.77	5.75	5.66	5.16					
	5.0	5.71	5.67	5.60	5.38	3.91					
	8.0	5.52	5.42	5.26	4.59	2.03					
	10.0	5.36	5.19	4.93	3.91						
	12.0	5.13	4.90	4.50	3.22						
	15.0	4.71	4.33	3.80	2.27						
	17.0	4.38	3.93	3.33	1.83						
	20.0	3.85	3.33	2.66							
550S162-68	3.0	14.7	14.6	14.5	14.2	12.6	20.8	20.7	20.5	19.9	16.5
	5.0	14.3	14.2	14.0	13.2	9.44	20.1	19.8	19.4	17.8	10.7
	7.0	13.9	13.6	13.2	11.9	6.11	19.2	18.6	17.8	15.1	6.11
	9.0	13.3	12.9	12.3	10.3	3.88	18.0	17.1	16.0	12.1	3.88
	11.0	12.6	12.0	11.3	8.58		16.5	15.4	14.0	9.22	
	14.0	11.4	10.6	9.64	6.11		14.2	12.7	11.0	6.11	
	16.0	10.5	9.61	8.53	4.85		12.6	11.0	9.13	4.85	
	18.0	9.61	8.60	7.46	3.88		10.9	9.24	7.59	3.88	
550S162-54	3.0	11.0	11.0	10.9	10.7	9.49	15.1	15.0	14.9	14.6	12.5
	5.0	10.8	10.7	10.5	9.95	7.16	14.7	14.6	14.3	13.5	8.15
	7.0	10.4	10.2	9.93	8.96	4.69	14.2	14.0	13.4	11.5	4.69
	9.0	9.98	9.67	9.23	7.78	3.06	13.5	12.9	12.0	9.25	3.06
	11.0	9.46	9.03	8.43	6.53		12.5	11.6	10.5	7.08	
	14.0	8.56	7.95	7.16	4.69		10.7	9.55	8.15	4.69	
	16.0	7.90	7.18	6.30	3.74		9.47	8.19	6.70	3.74	
	18.0	7.22	6.41	5.47	3.06		8.25	6.89	5.54	3.06	
550S162-43	3.0	8.31	8.28	8.23	8.07	7.18					
	5.0	8.13	8.05	7.93	7.51	5.43					
	7.0	7.87	7.72	7.49	6.75	3.59					
	9.0	7.54	7.30	6.95	5.88	2.34					
	11.0	7.14	6.81	6.34	4.96						
	14.0	6.46	5.98	5.36	3.59						
	16.0	5.96	5.40	4.69	2.86						
	18.0	5.44	4.81	4.05	2.34						
550S162-33	3.0	5.78	5.76	5.74	5.65	5.17					
	5.0	5.68	5.64	5.58	5.37	3.95					
	7.0	5.55	5.47	5.35	4.90	2.61					
	9.0	5.38	5.23	5.02	4.25	1.69					
	11.0	5.14	4.93	4.60	3.58						
	14.0	4.69	4.33	3.86	2.61						
	16.0	4.32	3.90	3.37	2.07						
	18.0	3.94	3.47	2.89	1.69						

Table III - 5 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					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
400S162-68	2.0	14.0	13.9	13.9	13.8	13.0	20.2	20.1	20.0	19.7	18.0
	4.0	13.6	13.4	13.3	12.8	10.2	19.3	19.0	18.7	17.6	12.6
	5.0	13.3	13.1	12.8	12.1	8.65	18.6	18.2	17.7	16.2	10.0
	7.0	12.5	12.1	11.7	10.5	5.60	17.0	16.3	15.5	13.1	5.60
	9.0	11.5	11.0	10.4	8.65	3.39	15.1	14.0	12.9	9.99	3.39
	10.0	11.0	10.3	9.65	7.76		14.0	12.8	11.6	8.46	
	12.0	9.78	8.90	8.10	6.13		11.9	10.4	9.06	6.17	
	14.0	8.46	7.47	6.63	4.78		9.67	7.99	6.79	4.78	
400S162-54	2.0	10.7	10.7	10.6	10.5	9.90	14.8	14.7	14.7	14.5	13.7
	4.0	10.4	10.3	10.2	9.75	7.76	14.3	14.2	14.0	13.4	9.50
	5.0	10.1	9.99	9.81	9.22	6.57	14.0	13.8	13.5	12.3	7.38
	7.0	9.56	9.27	8.97	7.98	4.37	13.0	12.4	11.8	9.90	4.37
	9.0	8.84	8.41	7.96	6.64	2.82	11.6	10.7	9.87	7.49	2.82
	10.0	8.43	7.93	7.42	5.97	2.29	10.8	9.81	8.88	6.38	2.29
	12.0	7.57	6.93	6.33	4.71		9.14	8.01	6.97	4.71	
	14.0	6.65	5.92	5.25	3.59		7.52	6.29	5.35	3.59	
400S162-43	2.0	8.19	8.17	8.14	8.06	7.58					
	4.0	7.95	7.87	7.78	7.47	5.90					
	5.0	7.78	7.65	7.52	7.06	4.95					
	7.0	7.33	7.11	6.87	6.09	3.20					
	9.0	6.78	6.44	6.09	5.03	2.16					
	10.0	6.47	6.08	5.68	4.51	1.82					
	12.0	5.81	5.32	4.83	3.51						
	14.0	5.12	4.54	4.00	2.73						
400S162-33	2.0	5.71	5.70	5.69	5.64	5.40					
	4.0	5.59	5.55	5.50	5.34	4.30					
	5.0	5.50	5.43	5.36	5.10	3.57					
	7.0	5.27	5.13	4.98	4.44	2.25					
	9.0	4.93	4.71	4.45	3.65	1.50					
	10.0	4.73	4.44	4.14	3.26	1.27					
	12.0	4.25	3.88	3.52	2.51						
	14.0	3.74	3.31	2.90	1.93						
362S162-68	2.0	13.6	13.5	13.5	13.3	12.4	19.8	19.7	19.6	19.3	17.4
	3.0	13.4	13.3	13.1	12.8	11.0	19.4	19.2	18.9	18.2	14.7
	5.0	12.7	12.4	12.1	11.3	7.82	18.0	17.5	17.0	15.3	8.86
	6.0	12.2	11.8	11.5	10.4	6.46	17.1	16.4	15.7	13.6	6.67
	8.0	11.1	10.6	10.0	8.52	4.10	15.0	13.9	12.9	10.1	4.10
	9.0	10.5	9.84	9.24	7.61	3.24	13.8	12.5	11.4	8.50	3.24
	11.0	9.18	8.36	7.65	5.92		11.3	9.79	8.57	5.99	
	12.0	8.50	7.61	6.88	5.17		10.1	8.50	7.29	5.17	

Table III - 5 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Studs
C-Sections With Lips

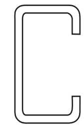
Ω_c = 1.80
ϕ_c = 0.85

Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					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	10.5	10.5	10.4	10.3	9.65	14.6	14.5	14.5	14.3	13.4
	3.0	10.3	10.3	10.2	9.95	8.60	14.4	14.2	14.1	13.8	11.2
	5.0	9.86	9.68	9.49	8.87	6.09	13.7	13.4	13.0	11.8	6.76
	6.0	9.54	9.29	9.03	8.21	4.87	13.2	12.6	12.1	10.5	4.94
	8.0	8.77	8.36	7.96	6.78	3.11	11.6	10.8	10.0	7.86	3.11
	9.0	8.33	7.85	7.38	6.02	2.61	10.7	9.80	8.94	6.64	2.61
	11.0	7.39	6.76	6.15	4.58		8.94	7.82	6.85	4.60	
	12.0	6.89	6.16	5.52	3.94		8.05	6.87	5.83	3.94	
362S162-43	2.0	8.09	8.06	8.04	7.95	7.43					
	4.0	7.80	7.71	7.61	7.28	5.64					
	5.0	7.60	7.46	7.31	6.82	4.64					
	7.0	7.08	6.82	6.56	5.76	2.88					
	8.0	6.77	6.45	6.14	5.20	2.30					
	10.0	6.08	5.65	5.23	4.08	1.61					
	11.0	5.72	5.23	4.77	3.56						
	13.0	4.96	4.38	3.87	2.64						
362S162-33	2.0	5.66	5.65	5.64	5.59	5.32					
	4.0	5.52	5.47	5.42	5.25	4.12					
	5.0	5.41	5.34	5.27	4.97	3.37					
	7.0	5.13	4.97	4.81	4.22	2.05					
	8.0	4.94	4.74	4.51	3.80	1.64					
	10.0	4.47	4.15	3.84	2.96	1.13					
	11.0	4.20	3.84	3.50	2.57						
	13.0	3.65	3.22	2.84	1.93						
350S162-68	2.0	13.3	13.2	13.2	13.0	12.1	19.6	19.5	19.4	19.1	17.2
	3.0	13.0	12.9	12.8	12.5	10.6	19.1	18.9	18.7	18.0	14.3
	5.0	12.3	12.0	11.8	10.9	7.53	17.7	17.1	16.6	14.9	8.45
	6.0	11.8	11.5	11.1	10.0	6.19	16.7	16.0	15.3	13.1	6.36
	8.0	10.7	10.1	9.62	8.15	4.03	14.4	13.3	12.3	9.54	4.03
	9.0	10.1	9.42	8.82	7.24	3.18	13.2	11.9	10.7	7.97	3.18
	11.0	8.74	7.91	7.23	5.57		10.6	9.11	7.94	5.61	
	12.0	8.05	7.16	6.46	4.84		9.36	7.83	6.73	4.84	
350S162-54	2.0	10.4	10.4	10.4	10.2	9.55	14.5	14.5	14.4	14.2	13.2
	3.0	10.3	10.2	10.1	9.85	8.46	14.3	14.1	14.0	13.7	11.0
	5.0	9.74	9.55	9.35	8.72	5.85	13.5	13.3	12.8	11.6	6.44
	6.0	9.40	9.13	8.87	8.03	4.66	13.0	12.4	11.9	10.2	4.70
	8.0	8.59	8.16	7.75	6.50	2.97	11.3	10.5	9.69	7.54	2.97
	9.0	8.13	7.62	7.13	5.72	2.50	10.4	9.44	8.58	6.21	2.50
	11.0	7.11	6.42	5.82	4.30		8.56	7.41	6.37	4.31	
	12.0	6.56	5.81	5.18	3.69		7.64	6.36	5.38	3.69	

Table III - 5 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85



Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					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
350S162-43	2.0	8.04	8.01	7.98	7.89	7.36					
	3.0	7.91	7.85	7.79	7.59	6.51					
	5.0	7.52	7.37	7.21	6.72	4.52					
	6.0	7.26	7.05	6.84	6.19	3.57					
	8.0	6.64	6.31	5.99	5.04	2.18					
	9.0	6.29	5.90	5.53	4.47	1.81					
	11.0	5.54	5.04	4.58	3.36						
	12.0	5.15	4.60	4.12	2.86						
350S162-33	2.0	5.66	5.64	5.63	5.58	5.30					
	3.0	5.59	5.56	5.53	5.43	4.79					
	5.0	5.39	5.31	5.23	4.93	3.29					
	6.0	5.26	5.13	5.01	4.56	2.56					
	8.0	4.88	4.66	4.42	3.71	1.58					
	9.0	4.65	4.36	4.08	3.27	1.28					
	11.0	4.10	3.72	3.38	2.45						
	12.0	3.81	3.40	3.03	2.11						
250S162-68	2.0	12.5	12.4	12.4	12.1	10.9	18.7	18.5	18.4	17.8	15.2
	3.0	12.1	11.9	11.8	11.3	9.12	17.8	17.4	17.0	16.0	11.6
	4.0	11.5	11.3	11.0	10.3	7.33	16.5	15.9	15.4	13.9	8.31
	5.0	10.9	10.4	10.1	9.11	5.77	15.1	14.2	13.5	11.6	5.89
	6.0	10.1	9.54	9.10	7.92	4.51	13.5	12.4	11.5	9.35	4.51
	7.0	9.25	8.57	8.05	6.76	3.67	11.8	10.5	9.59	7.34	3.67
	8.0	8.36	7.59	7.01	5.67	3.10	10.1	8.76	7.78	5.77	3.10
	9.0	7.46	6.61	6.01	4.69	2.70	8.53	7.09	6.21	4.69	2.70
250S162-54	2.0	10.1	10.1	10.00	9.83	8.83	14.1	14.0	14.0	13.7	12.1
	3.0	9.81	9.67	9.55	9.16	7.30	13.7	13.5	13.3	12.7	9.21
	4.0	9.37	9.13	8.92	8.31	5.72	13.1	12.7	12.3	11.1	6.37
	5.0	8.83	8.48	8.19	7.35	4.32	12.1	11.4	10.9	9.31	4.35
	6.0	8.21	7.75	7.38	6.36	3.24	10.9	10.1	9.36	7.47	3.24
	7.0	7.53	6.97	6.53	5.38	2.57	9.65	8.59	7.78	5.78	2.57
	8.0	6.82	6.17	5.68	4.46	2.13	8.30	7.14	6.29	4.51	2.13
	9.0	6.09	5.38	4.85	3.63	1.82	7.00	5.79	5.01	3.63	1.82
250S162-43	2.0	7.86	7.82	7.77	7.64	6.92					
	3.0	7.63	7.53	7.44	7.16	5.76					
	4.0	7.31	7.14	6.99	6.54	4.52					
	5.0	6.93	6.68	6.46	5.83	3.31					
	6.0	6.48	6.15	5.87	5.08	2.41					
	7.0	5.99	5.58	5.24	4.30	1.87					
	8.0	5.47	4.99	4.59	3.54	1.52					
	9.0	4.94	4.37	3.92	2.86	1.27					

Table III - 5 (continued)

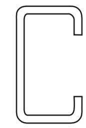
Nominal Axial Strength, P_n, kips [1]

SSMA Studs

C-Sections With Lips

Ω_c = 1.80

ϕ_c = 0.85



Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					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					

Note:

1. Axial strengths given are nominal strengths. To obtain the design strength, these values must be modified by factors of safety (ASD) or resistance factors (LRFD).

Table III - 6

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

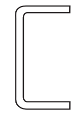
$\Omega_c = 1.80$
$\phi_c = 0.85$

Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing ($KL_y = KL_t$)					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
1200T200-97	5.0	29.0	28.4	27.8	26.2	18.8	37.3	36.2	35.1	32.2	19.7
	9.0	28.6	26.7	25.0	20.5		36.6	33.2	30.0	22.4	
	14.0	27.8	23.6	20.0	11.9		35.2	27.6	21.5	11.9	
	18.0	27.0	20.5	15.0			33.7	22.4	15.0		
	23.0	25.8	16.0	10.4			31.4	16.0	10.4		
	27.0	24.6	12.6				29.2	12.6			
	32.0	22.9	9.78				26.3	9.78			
	36.0	21.4					23.9				
1200T200-68	5.0	15.0	14.8	14.5	13.7	10.2	19.2	18.6	18.1	16.7	10.6
	9.0	14.9	14.0	13.1	11.0		18.8	17.1	15.6	11.9	
	14.0	14.5	12.5	10.7	6.98		18.1	14.5	11.5	6.98	
	18.0	14.1	11.0	8.49			17.3	11.9	8.49		
	23.0	13.4	8.91	6.19			16.2	8.95	6.19		
	27.0	12.9	7.31				15.1	7.31			
	32.0	12.0	5.81				13.7	5.81			
	36.0	11.3					12.5				
1200T200-54*	5.0	9.71	9.54	9.36	8.88	6.66	12.3	12.0	11.6	10.8	6.96
	9.0	9.60	9.04	8.52	7.17		12.1	11.0	10.1	7.79	
	14.0	9.36	8.10	7.00	4.65		11.6	9.37	7.52	4.65	
	18.0	9.10	7.17	5.61			11.2	7.79	5.61		
	23.0	8.71	5.88	4.16			10.4	5.90	4.16		
	27.0	8.33	4.87				9.77	4.87			
	32.0	7.81	3.95				8.87	3.95			
	36.0	7.36					8.10				
1000T200-97	4.0	28.6	28.3	27.9	27.0	22.3	37.0	36.4	35.8	34.0	25.6
	8.0	28.1	27.0	25.7	22.3	10.1	36.1	34.0	31.6	25.6	10.1
	12.0	27.4	24.9	22.3	15.7		34.8	30.2	25.6	15.8	
	16.0	26.4	22.3	18.1	10.1		33.0	25.6	18.9	10.1	
	20.0	25.2	19.3	13.4			30.7	20.6	13.4		
	23.0	24.2	16.6	10.8			28.9	16.8	10.8		
	27.0	22.6	13.2				26.2	13.2			
	31.0	20.9	10.6				23.3	10.6			
1000T200-68	4.0	14.9	14.8	14.6	14.1	11.9	19.0	18.8	18.5	17.6	13.6
	8.0	14.7	14.1	13.5	11.9	6.11	18.6	17.6	16.5	13.6	6.11
	12.0	14.3	13.2	11.9	8.84		17.9	15.8	13.6	8.89	
	16.0	13.8	11.9	9.90	6.11		17.0	13.6	10.3	6.11	
	20.0	13.2	10.4	7.77			15.9	11.1	7.77		
	23.0	12.7	9.24	6.47			15.0	9.37	6.47		
	27.0	12.0	7.65				13.7	7.65			
	31.0	11.1	6.38				12.3	6.38			

Table III - 6 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

$\Omega_c = 1.80$
$\phi_c = 0.85$



Section	KL_x ft.	F_y = 33 ksi, Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	F_y = 50 ksi, Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
1000T200-54	4.0	9.65	9.56	9.46	9.17	7.77	12.2	12.1	11.9	11.4	8.84
	8.0	9.51	9.17	8.79	7.77	4.14	12.0	11.4	10.6	8.84	4.14
	12.0	9.28	8.56	7.77	5.86		11.6	10.2	8.84	5.89	
	16.0	8.98	7.77	6.52	4.14		11.0	8.84	6.79	4.14	
	19.0	8.70	7.09	5.52			10.5	7.70	5.52		
	23.0	8.27	6.11	4.36			9.71	6.20	4.36		
	27.0	7.79	5.11				8.87	5.11			
	31.0	7.26	4.30				7.98	4.30			
1000T200-43*	4.0	6.27	6.21	6.15	5.97	5.07					
	8.0	6.18	5.97	5.72	5.07	2.77					
	12.0	6.03	5.58	5.07	3.86						
	16.0	5.84	5.07	4.28	2.77						
	19.0	5.66	4.64	3.65							
	23.0	5.38	4.02	2.91							
	27.0	5.07	3.39								
	31.0	4.74	2.87								
800T200-97	4.0	27.9	27.7	27.4	26.5	22.3	36.2	35.9	35.3	33.7	26.0
	7.0	27.4	26.9	26.0	23.5	12.9	35.4	34.3	32.7	28.2	12.9
	10.0	26.8	25.7	23.9	19.5		34.1	32.1	28.9	21.3	
	13.0	25.9	24.1	21.4	14.5		32.5	29.3	24.5	14.6	
	16.0	24.8	22.3	18.5	10.4		30.5	26.0	19.7	10.4	
	19.0	23.6	20.2	15.1			28.3	22.5	15.2		
	23.0	21.7	17.1	11.2			25.0	17.7	11.2		
	26.0	20.1	14.5	8.95			22.4	14.6	8.95		
800T200-68	4.0	14.7	14.6	14.4	14.0	12.0	18.7	18.6	18.3	17.5	13.8
	7.0	14.4	14.2	13.7	12.6	7.61	18.3	17.9	17.0	14.9	7.61
	10.0	14.1	13.6	12.8	10.6		17.7	16.8	15.3	11.6	
	13.0	13.7	12.9	11.5	8.38		16.9	15.4	13.1	8.39	
	16.0	13.1	12.0	10.1	6.36		15.9	13.8	10.8	6.36	
	19.0	12.5	11.0	8.63			14.8	12.1	8.68		
	23.0	11.6	9.51	6.74			13.2	9.82	6.74		
	26.0	10.8	8.38	5.66			11.9	8.39	5.66		
800T200-54	4.0	9.52	9.47	9.37	9.12	7.84	12.1	12.0	11.8	11.3	9.04
	7.0	9.38	9.23	8.95	8.22	5.11	11.8	11.5	11.0	9.71	5.11
	10.0	9.17	8.86	8.34	6.99		11.4	10.9	9.92	7.61	
	13.0	8.89	8.40	7.57	5.59		10.9	10.0	8.57	5.60	
	16.0	8.55	7.84	6.69	4.32		10.3	9.04	7.12	4.32	
	19.0	8.16	7.21	5.75			9.60	7.98	5.79		
	22.0	7.72	6.54	4.82			8.84	6.87	4.82		
	26.0	7.08	5.59	3.90			7.76	5.60	3.90		

Table III - 6 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

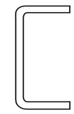
$\Omega_c = 1.80$
$\phi_c = 0.85$

Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing ($KL_y = KL_t$)					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					
	7.0	6.11	6.01	5.84	5.37	3.40					
	10.0	5.97	5.78	5.45	4.60						
	13.0	5.80	5.49	4.96	3.71						
	16.0	5.58	5.13	4.40	2.90						
	19.0	5.33	4.74	3.81							
	22.0	5.05	4.31	3.22							
	26.0	4.64	3.71	2.63							
800T200-33*	4.0	3.80	3.78	3.74	3.64	3.15					
	7.0	3.74	3.68	3.58	3.30	2.11					
	10.0	3.66	3.55	3.34	2.83						
	13.0	3.55	3.37	3.05	2.29						
	16.0	3.42	3.15	2.71	1.80						
	19.0	3.27	2.91	2.35	1.47						
	22.0	3.10	2.65	2.00							
	26.0	2.85	2.29	1.64							
800T125-97	3.0	24.9	24.3	23.7	22.1	15.2	33.8	32.5	31.3	28.2	15.9
	6.0	24.5	22.1	20.1	15.2		33.0	28.2	24.3	15.9	
	9.0	23.8	18.9	15.2	8.45		31.6	22.2	15.9	8.45	
	12.0	22.9	15.2	10.2			29.8	15.9	10.2		
	15.0	21.8	11.3	6.85			27.6	11.3	6.85		
	18.0	20.5	8.45				25.1	8.45			
	21.0	19.0					22.5				
	24.0	17.5					19.8				
800T125-68	3.0	14.0	13.7	13.4	12.7	8.98	18.1	17.6	17.0	15.6	9.51
	6.0	13.8	12.7	11.7	8.98		17.7	15.6	13.8	9.51	
	9.0	13.5	11.1	8.98	5.22		17.1	12.8	9.51	5.22	
	12.0	13.0	8.98	6.22			16.3	9.51	6.22		
	15.0	12.4	6.84	4.45			15.2	6.84	4.45		
	18.0	11.8	5.22				14.0	5.22			
	21.0	11.0					12.7				
	24.0	10.1					11.4				
800T125-54	3.0	9.21	9.03	8.86	8.38	6.20	11.8	11.5	11.1	10.3	6.53
	6.0	9.08	8.38	7.76	6.20		11.6	10.3	9.15	6.53	
	9.0	8.87	7.40	6.20	3.71		11.2	8.53	6.53	3.71	
	12.0	8.58	6.20	4.41			10.6	6.53	4.41		
	15.0	8.22	4.85	3.17			9.98	4.85	3.17		
	18.0	7.81	3.71				9.23	3.71			
	21.0	7.33					8.41				
	24.0	6.82					7.55				

Table III - 6 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85

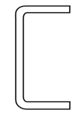


Section	KL_x	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y = KL_t)				
	ft.	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
800T125-43	3.0	6.05	5.93	5.83	5.53	4.15					
	6.0	5.97	5.53	5.14	4.15						
	9.0	5.83	4.91	4.15	2.63						
	12.0	5.65	4.15	3.06							
	15.0	5.42	3.32	2.26							
	18.0	5.15	2.63								
	21.0	4.85									
	24.0	4.52									
800T125-33*	3.0	3.73	3.66	3.60	3.42	2.59					
	6.0	3.68	3.42	3.18	2.59						
	9.0	3.60	3.05	2.59	1.68						
	12.0	3.49	2.59	1.94							
	15.0	3.35	2.10	1.47							
	18.0	3.19	1.68								
	21.0	3.00									
	24.0	2.81									
600T200-97	3.0	26.8	26.7	26.6	26.2	24.0	35.2	35.0	34.8	34.0	29.8
	5.0	26.5	26.3	25.9	24.9	19.2	34.5	34.1	33.5	31.5	21.6
	8.0	25.6	25.1	24.4	21.8	9.82	32.9	32.0	30.6	26.0	9.82
	10.0	24.9	24.2	23.1	19.2		31.6	30.2	28.2	21.6	
	13.0	23.5	22.4	20.8	14.7		29.0	27.1	24.3	14.9	
	15.0	22.4	21.1	19.2	11.2		27.1	24.8	21.6	11.2	
	18.0	20.6	19.0	16.3	7.76		24.0	21.2	17.0	7.76	
	20.0	19.3	17.5	14.0			21.8	18.8	14.1		
600T200-68	3.0	14.3	14.3	14.2	14.0	12.9	18.4	18.3	18.2	17.8	15.8
	5.0	14.1	14.0	13.9	13.3	10.6	18.1	17.9	17.6	16.6	11.8
	8.0	13.7	13.5	13.1	11.9	6.51	17.3	16.8	16.1	13.9	6.51
	10.0	13.3	13.0	12.4	10.6		16.6	15.9	14.9	11.8	
	13.0	12.7	12.1	11.2	8.55		15.3	14.3	12.8	8.66	
	15.0	12.1	11.4	10.3	7.13		14.4	13.1	11.4	7.13	
	18.0	11.2	10.3	9.03	5.47		12.8	11.3	9.29	5.47	
	20.0	10.6	9.52	8.15			11.7	10.0	8.18		
600T200-54	3.0	9.34	9.32	9.28	9.15	8.47	11.9	11.9	11.8	11.6	10.3
	5.0	9.23	9.17	9.06	8.72	7.05	11.7	11.6	11.4	10.7	7.83
	8.0	8.97	8.81	8.55	7.77	4.46	11.2	10.9	10.5	9.05	4.46
	10.0	8.73	8.49	8.11	7.00	3.29	10.8	10.3	9.66	7.75	3.29
	13.0	8.30	7.92	7.35	5.75		9.99	9.31	8.33	5.83	
	15.0	7.96	7.48	6.79	4.85		9.39	8.56	7.39	4.85	
	18.0	7.39	6.77	5.91	3.81		8.40	7.37	6.04	3.81	
	20.0	6.98	6.27	5.33	3.29		7.71	6.56	5.33	3.29	

Table III - 6 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85

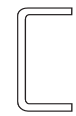


Section	KL_x ft.	F_y = 33 ksi, Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	F_y = 50 ksi, Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
600T200-43	3.0	6.11	6.09	6.07	5.99	5.55					
	5.0	6.04	6.00	5.93	5.71	4.64					
	8.0	5.87	5.77	5.60	5.09	3.01					
	10.0	5.72	5.56	5.32	4.59	2.26					
	13.0	5.44	5.19	4.82	3.78						
	15.0	5.22	4.91	4.45	3.25						
	18.0	4.86	4.45	3.88	2.59						
	20.0	4.60	4.13	3.49	2.26						
600T200-33	3.0	3.76	3.75	3.73	3.68	3.41					
	5.0	3.71	3.69	3.65	3.51	2.86					
	8.0	3.61	3.55	3.45	3.13	1.89					
	10.0	3.52	3.42	3.27	2.82	1.44					
	13.0	3.35	3.20	2.97	2.32						
	15.0	3.22	3.03	2.74	1.99						
	18.0	3.00	2.74	2.38	1.61						
	20.0	2.84	2.55	2.14	1.43						
600T125-97	3.0	23.8	23.4	22.9	21.5	15.3	32.6	31.7	30.7	28.0	16.8
	5.0	23.4	22.2	20.9	17.6	6.54	31.7	29.4	26.9	20.7	6.54
	7.0	22.8	20.6	18.3	12.9		30.5	26.3	22.0	13.2	
	10.0	21.6	17.6	13.7	6.54		28.2	20.7	14.3	6.54	
	12.0	20.6	15.3	10.2			26.2	16.8	10.2		
	14.0	19.5	12.9	7.51			24.1	13.2	7.51		
	17.0	17.6	9.05				20.8	9.05			
	19.0	16.3	7.25				18.5	7.25			
600T125-68	3.0	13.6	13.4	13.1	12.5	9.26	17.7	17.3	16.8	15.6	10.2
	5.0	13.4	12.8	12.2	10.6	4.64	17.3	16.3	15.1	12.2	4.64
	7.0	13.1	12.1	10.9	7.93		16.7	14.9	12.9	8.14	
	10.0	12.5	10.6	8.38	4.64		15.6	12.2	8.77	4.64	
	12.0	12.0	9.26	6.62			14.7	10.2	6.62		
	14.0	11.4	7.93	5.19			13.7	8.14	5.19		
	17.0	10.4	6.02				12.0	6.02			
	19.0	9.66	5.04				10.9	5.04			
600T125-54	3.0	9.00	8.88	8.73	8.33	6.41	11.6	11.3	11.1	10.3	6.98
	5.0	8.88	8.55	8.16	7.14	3.39	11.3	10.7	10.0	8.20	3.39
	7.0	8.69	8.07	7.36	5.62		11.0	9.85	8.59	5.76	
	10.0	8.31	7.14	5.89	3.39		10.3	8.20	6.15	3.39	
	12.0	8.00	6.41	4.78			9.71	6.98	4.78		
	14.0	7.64	5.62	3.77			9.07	5.76	3.77		
	17.0	7.03	4.36				8.02	4.36			
	19.0	6.59	3.67				7.28	3.67			

Table III - 6 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

$\Omega_c = 1.80$
$\phi_c = 0.85$

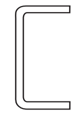


Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing ($KL_y = KL_t$)					Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
600T125-43	3.0	5.94	5.86	5.77	5.52	4.30					
	5.0	5.86	5.66	5.41	4.76	2.45					
	7.0	5.74	5.36	4.91	3.81						
	10.0	5.50	4.76	3.98	2.45						
	12.0	5.30	4.30	3.30							
	14.0	5.07	3.81	2.70							
	17.0	4.68	3.05								
	19.0	4.40	2.63								
600T125-33	3.0	3.68	3.64	3.58	3.43	2.70					
	5.0	3.63	3.51	3.36	2.97	1.58					
	7.0	3.56	3.33	3.06	2.40						
	10.0	3.41	2.97	2.50	1.58						
	12.0	3.29	2.70	2.09							
	14.0	3.15	2.40	1.73							
	17.0	2.91	1.94								
	19.0	2.74	1.69								
550T150-68	3.0	13.8	13.7	13.6	13.1	11.1	17.8	17.7	17.4	16.7	13.1
	5.0	13.5	13.4	13.0	11.9	7.24	17.4	17.1	16.4	14.4	7.24
	7.0	13.2	12.9	12.2	10.3	3.99	16.8	16.2	14.9	11.6	3.99
	9.0	12.8	12.3	11.1	8.34		16.1	15.1	13.1	8.53	
	11.0	12.3	11.5	9.95	6.21		15.1	13.8	11.1	6.21	
	14.0	11.4	10.3	7.98	3.99		13.5	11.6	8.09	3.99	
	16.0	10.7	9.32	6.52			12.4	10.0	6.52		
	18.0	10.0	8.34	5.38			11.1	8.53	5.38		
550T150-54	3.0	9.08	9.04	8.95	8.70	7.44	11.6	11.6	11.4	10.9	8.67
	5.0	8.94	8.83	8.59	7.93	5.08	11.4	11.2	10.7	9.53	5.08
	7.0	8.74	8.53	8.08	6.90	3.08	11.0	10.6	9.80	7.74	3.08
	9.0	8.48	8.14	7.44	5.70		10.5	9.91	8.67	5.84	
	11.0	8.16	7.69	6.71	4.46		9.93	9.11	7.42	4.46	
	14.0	7.59	6.90	5.49	3.08		8.92	7.74	5.57	3.08	
	16.0	7.16	6.31	4.65			8.18	6.78	4.65		
	18.0	6.70	5.70	3.95			7.40	5.84	3.95		
550T150-43	3.0	5.98	5.95	5.90	5.74	4.95					
	5.0	5.89	5.81	5.67	5.26	3.45					
	7.0	5.76	5.62	5.35	4.60	2.22					
	9.0	5.59	5.37	4.95	3.85						
	11.0	5.38	5.08	4.48	3.06						
	14.0	5.02	4.58	3.72	2.22						
	16.0	4.74	4.22	3.18							
	18.0	4.45	3.85	2.73							

Table III - 6 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85



Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					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					
	5.0	3.64	3.60	3.51	3.26	2.17					
	7.0	3.56	3.48	3.32	2.86	1.43					
	9.0	3.46	3.33	3.07	2.41						
	11.0	3.34	3.15	2.79	1.93						
	14.0	3.11	2.84	2.33	1.43						
	16.0	2.95	2.62	2.01							
	18.0	2.77	2.39	1.74							
400T125-68	2.0	12.9	12.8	12.7	12.5	11.2	17.0	16.9	16.7	16.3	13.9
	4.0	12.6	12.4	12.1	11.2	6.69	16.4	16.1	15.6	13.9	6.79
	5.0	12.3	12.1	11.7	10.3	4.34	16.0	15.6	14.8	12.3	4.34
	7.0	11.8	11.4	10.6	8.01		15.0	14.2	12.9	8.63	
	9.0	11.1	10.5	9.21	5.36		13.7	12.6	10.6	5.36	
	10.0	10.7	9.98	8.41	4.34		12.9	11.7	9.30	4.34	
	12.0	9.75	8.83	6.69	3.02		11.4	9.99	6.79	3.02	
	14.0	8.66	7.66	4.99			9.71	8.07	4.99		
400T125-54	2.0	8.64	8.61	8.57	8.42	7.62	11.2	11.2	11.1	10.8	9.33
	4.0	8.46	8.35	8.19	7.62	5.02	10.9	10.7	10.4	9.33	5.08
	5.0	8.32	8.17	7.92	7.06	3.51	10.6	10.3	9.87	8.34	3.51
	7.0	7.96	7.69	7.26	5.74		9.96	9.45	8.68	6.13	
	9.0	7.51	7.10	6.43	4.22		9.14	8.41	7.26	4.22	
	10.0	7.25	6.78	5.97	3.51		8.68	7.85	6.51	3.51	
	12.0	6.68	6.09	5.02	2.44		7.69	6.70	5.08	2.44	
	14.0	6.06	5.37	3.97			6.64	5.56	3.97		
400T125-43	2.0	5.76	5.74	5.71	5.62	5.12					
	4.0	5.64	5.58	5.47	5.12	3.47					
	5.0	5.55	5.45	5.29	4.77	2.59					
	7.0	5.33	5.14	4.86	3.93						
	9.0	5.03	4.76	4.35	3.00						
	10.0	4.87	4.55	4.08	2.59						
	12.0	4.50	4.10	3.47	1.95						
	14.0	4.10	3.62	2.85							
400T125-33	2.0	3.60	3.59	3.57	3.52	3.21					
	4.0	3.53	3.49	3.42	3.21	2.22					
	5.0	3.47	3.41	3.31	3.00	1.69					
	7.0	3.34	3.22	3.04	2.50						
	9.0	3.16	2.99	2.72	1.94						
	10.0	3.06	2.86	2.55	1.69						
	12.0	2.83	2.57	2.20	1.32						
	14.0	2.59	2.28	1.85							

Table III - 6 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85

Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					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
362T125-68	2.0	12.6	12.6	12.5	12.3	11.0	16.7	16.6	16.5	16.0	13.8
	4.0	12.3	12.1	11.8	11.0	6.47	16.0	15.7	15.3	13.8	6.61
	5.0	12.0	11.8	11.4	10.2	4.23	15.6	15.1	14.4	12.2	4.23
	7.0	11.4	10.9	10.3	7.80		14.4	13.6	12.5	8.63	
	8.0	11.0	10.5	9.76	6.47		13.7	12.7	11.5	6.61	
	10.0	10.1	9.34	8.26	4.23		12.1	10.9	9.28	4.23	
	11.0	9.55	8.72	7.35	3.50		11.2	9.95	7.87	3.50	
	13.0	8.33	7.28	5.63			9.38	7.75	5.63		
362T125-54	2.0	8.52	8.49	8.44	8.30	7.54	11.1	11.0	11.0	10.7	9.28
	4.0	8.30	8.19	8.03	7.52	4.99	10.7	10.5	10.2	9.23	5.09
	5.0	8.14	7.98	7.74	7.00	3.41	10.4	10.1	9.63	8.31	3.41
	7.0	7.72	7.44	7.03	5.70		9.61	9.10	8.37	6.14	
	8.0	7.47	7.13	6.64	4.99		9.16	8.54	7.69	5.09	
	10.0	6.90	6.43	5.81	3.41		8.15	7.33	6.32	3.41	
	11.0	6.58	6.05	5.39	2.82		7.60	6.71	5.64	2.82	
	13.0	5.90	5.28	4.50			6.47	5.47	4.50		
362T125-43	2.0	5.70	5.68	5.65	5.55	5.07					
	3.0	5.64	5.60	5.53	5.33	4.35					
	5.0	5.45	5.35	5.19	4.70	2.59					
	6.0	5.33	5.19	4.97	4.33	1.91					
	8.0	5.03	4.79	4.46	3.47						
	9.0	4.85	4.57	4.18	3.01						
	11.0	4.46	4.09	3.61	2.25						
	12.0	4.24	3.84	3.32	1.91						
362T125-33	2.0	3.57	3.56	3.54	3.48	3.18					
	3.0	3.53	3.51	3.47	3.35	2.75					
	5.0	3.42	3.36	3.26	2.95	1.70					
	6.0	3.35	3.26	3.12	2.72	1.33					
	8.0	3.16	3.02	2.80	2.21						
	9.0	3.05	2.88	2.63	1.95						
	11.0	2.81	2.58	2.26	1.50						
	12.0	2.68	2.42	2.08	1.33						
350T125-68	2.0	12.5	12.5	12.4	12.2	11.0	16.6	16.5	16.4	15.9	13.7
	3.0	12.4	12.3	12.1	11.6	9.03	16.3	16.1	15.8	14.9	10.5
	5.0	11.9	11.6	11.3	10.1	4.19	15.4	14.9	14.3	12.2	4.19
	6.0	11.6	11.2	10.8	9.03	2.91	14.8	14.2	13.3	10.5	2.91
	8.0	10.8	10.3	9.58	6.39		13.4	12.5	11.3	6.54	
	9.0	10.3	9.72	8.86	5.17		12.6	11.5	10.3	5.17	
	11.0	9.27	8.34	7.25	3.46		10.8	9.56	7.79	3.46	
	12.0	8.60	7.60	6.39	2.91		9.92	8.38	6.54	2.91	

Table III - 6 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

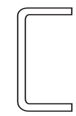
$\Omega_c = 1.80$
$\phi_c = 0.85$

Section	KL_x ft.	F_y = 33 ksi, Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	F_y = 50 ksi, Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
350T125-54	2.0	8.47	8.44	8.39	8.25	7.50	11.0	11.0	10.9	10.6	9.24
	3.0	8.37	8.31	8.21	7.90	6.36	10.9	10.7	10.6	9.96	7.24
	5.0	8.07	7.91	7.66	6.94	3.38	10.3	9.98	9.54	8.24	3.38
	6.0	7.86	7.64	7.32	6.36	2.35	9.91	9.50	8.91	7.24	2.35
	8.0	7.37	7.01	6.53	4.98		9.00	8.38	7.55	5.08	
	9.0	7.08	6.66	6.11	4.17		8.49	7.76	6.84	4.17	
	11.0	6.43	5.90	5.26	2.79		7.38	6.49	5.47	2.79	
	12.0	6.09	5.50	4.84	2.35		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	11.0	10.9	10.8	10.6	9.22	15.4	15.2	15.1	14.6	12.0
	3.0	10.7	10.6	10.4	9.82	7.67	14.9	14.6	14.3	13.2	9.20
	4.0	10.3	10.1	9.80	8.97	5.70	14.2	13.7	13.1	11.5	5.96
	5.0	9.86	9.50	9.12	8.08	3.81	13.3	12.5	11.8	9.85	3.81
	6.0	9.31	8.84	8.38	7.13	2.65	12.2	11.2	10.4	8.33	2.65
	7.0	8.70	8.12	7.54	6.26		11.0	9.92	8.99	6.86	
	8.0	8.02	7.29	6.70	5.49		9.76	8.59	7.61	5.67	
	9.0	7.25	6.47	5.89	4.70		8.52	7.21	6.26	4.71	
250T125-54	2.0	7.89	7.85	7.80	7.62	6.73	10.4	10.3	10.2	9.90	8.30
	3.0	7.73	7.63	7.52	7.16	5.47	10.1	9.92	9.71	9.05	6.33
	4.0	7.50	7.34	7.16	6.55	4.38	9.66	9.37	9.04	8.03	4.54
	5.0	7.21	6.98	6.69	5.83	3.08	9.13	8.71	8.25	6.96	3.08
	6.0	6.87	6.50	6.12	5.13	2.14	8.52	7.96	7.39	5.75	2.14
	7.0	6.42	5.97	5.53	4.48		7.84	7.15	6.43	4.68	
	8.0	5.93	5.41	4.94	3.88		7.10	6.22	5.42	3.90	
	9.0	5.43	4.85	4.36	3.29		6.25	5.27	4.52	3.29	

Table III - 6 (continued)

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85



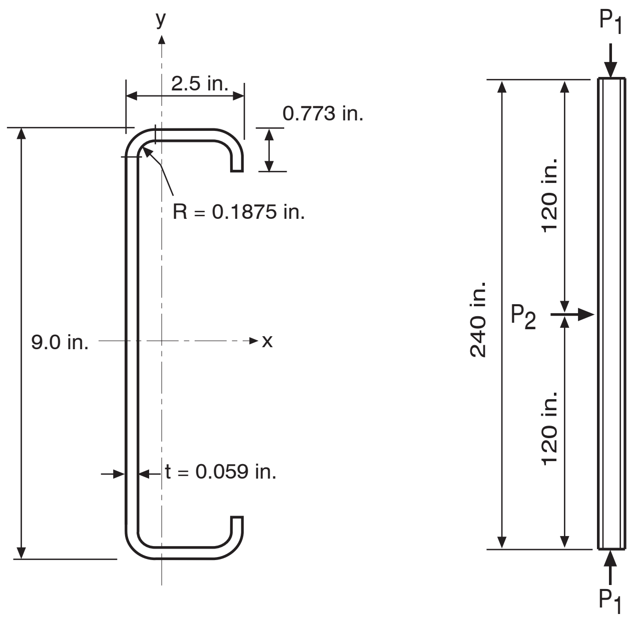
Section	KL_x ft.	F_y = 33 ksi, Bracing ($KL_y = KL_t$)					F_y = 50 ksi, Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
250T125-43	2.0	5.38	5.35	5.31	5.20	4.61					
	3.0	5.27	5.21	5.13	4.89	3.82					
	4.0	5.12	5.01	4.89	4.51	2.99					
	5.0	4.93	4.78	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. To obtain the design strength, these values must be modified by factors of safety (ASD) or resistance factors (LRFD).

SECTION 2 - EXAMPLE PROBLEMS

Example III-1: Braced C-Section With Lips - Bending And Compression



P_1 = 0.5 kips (dead)
= 2.0 kips (live)
P_2 = 0.1 kips (dead)
= 0.5 kips (live)

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 and torsional buckling
5. K_x = 1.0; L_x = 240 in.

Required:

Check the combined bending and compression strength of the section using ASD and LRFD methods with ASCE 7-98 load combinations.

Solution:

Refer to Example I-1 for derivation of geometric parameters.
Refer to Example I-8 for calculation of effective section properties.

1. Nominal flexural strength, M_n

Since the section is not subject to lateral-torsional buckling,

$M_n = S_eF_y$ (*Eq.* C3.1.1-1)

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) (*Eq.* C3.1.1-1)
= 104 kip-in.

2. Nominal axial strength, P_n

Since the member can only buckle perpendicular to the x-axis,

$$F_e = \frac{\pi^2 E}{(KL_x/r_x)^2} \quad (Eq.\ C4.1\text{-}1)$$

From Table I-1 or Example I-1

I_x = 10.30 in.^{4}

A = 0.881 in.^{2}

r_x = 3.42 in.

$$F_e = \frac{\pi^2(29500)}{\left[(1.0)(240.0)/(3.42)\right]^2} \quad (Eq.\ C4.1\text{-}1)$$

= 59.12 ksi

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \quad (Eq.\ C4\text{-}4)$$

$$= \sqrt{\frac{55}{59.12}}$$

= 0.965 < 1.5, therefore

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \quad (Eq.\ C4\text{-}2)$$

$$= \left(0.658^{(0.965)^2}\right)55 = 37.25 \text{ ksi}$$

In Example I-8, the effective area at f = 37.25 ksi was calculated as:

A_e = 0.515 in.^{2}

$P_n = A_eF_n$ (*Eq.* C4-1)

= (0.515)(37.25) = 19.2 kips

3. Required strength

$$M = \frac{P_2 L}{4}$$

$$M_{dead} = \frac{(0.10)(240.0)}{4} = 6.0 \text{ kip-in.}$$

$$M_{live} = \frac{(0.5)(240.0)}{4} = 30.0 \text{ kip-in.}$$

ASD

$M_x = M_{dead} + M_{live}$ = 6.0 + 30.0 = 36.0 kip-in.

$P = P_{dead} + P_{live}$ = 0.5 + 2.0 = 2.5 kips

LRFD

$M_{ux} = 1.2M_{dead} + 1.6M_{live}$ = (1.2)(6.0) + (1.6)(30.0) = 55.2 kip-in.

$P_u = 1.2P_{dead} + 1.6P_{live}$ = (1.2)(0.5) + (1.6)(2.0) = 3.80 kip-in.

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

$$P_{Ex} = \frac{\pi^2 E I_x}{(K_x L_x)^2} \quad (Eq.\ C5.2.1\text{-}6)$$

$$= \frac{\pi^2(29500)(10.30)}{\left[(1.0)(240.0)\right]^2} = 52.1 \text{ kips}$$

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} \qquad (Eq.\ C5.2.1\text{-}4)$$

$$= 1 - \frac{(1.80)(2.5)}{52.1} = 0.914$$

$M_y = 0.0$

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} + \frac{\Omega_b C_{my} M_y}{M_{ny}\alpha_y} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

$$\frac{(1.80)(2.5)}{19.2} + \frac{(1.67)(1.0)(36.0)}{(104)(0.914)} = 0.867 < 1.0 \text{ OK}$$

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

From Table III-1, P_{no} = 24.3 kips

$$\frac{(1.80)(2.5)}{24.3} + \frac{(1.67)(36.0)}{104} = 0.763 < 1.0 \text{ OK} \qquad (Eq.\ C5.2.1\text{-}2)$$

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)

$$\alpha_x = 1 - \frac{\overline{P}}{P_{Ex}} \qquad (Eq.\ C5.2.2\text{-}4)$$

$$\alpha_x = 1 - \frac{3.80}{52.1} = 0.927$$

$\overline{M}_y = 0.0$

$$\frac{\overline{P}}{\phi_c P_n} + \frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x} + \frac{C_{my}\overline{M}_y}{\phi_b M_{ny}\alpha_y} \leq 1.0 \qquad (Eq.\ C5.2.2\text{-}1)$$

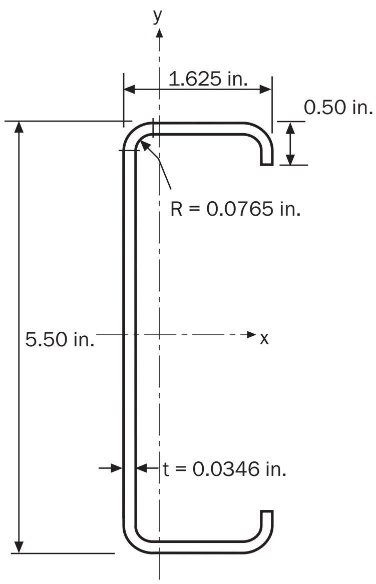
$$\frac{3.80}{(0.85)(19.2)} + \frac{(1.0)(55.2)}{(0.95)(104)(0.927)} = 0.836 < 1.0 \text{ OK}$$

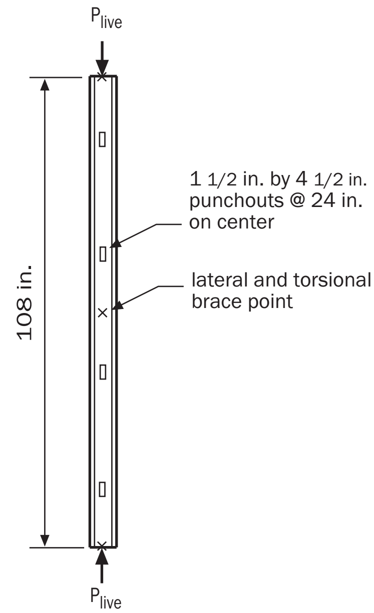
$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \leq 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

from Table III-1, P_{no} = 24.3 kips

$$\frac{3.80}{(0.85)(24.3)} + \frac{55.2}{(0.95)(104)} = 0.743 < 1.0 \text{ OK}$$

Example III-2: C-Section With Lips With Holes - Compression





$A = 0.327 \text{ in.}^2$
$r_x = 2.11$ in.
$r_y = 0.589$ in.
$J = 0.000130 \text{ in.}^4$
$C_w = 0.713 \text{ in.}^6$
$x_o = -1.11$ in.
$r_o = 2.46$ in.

Given:

1. Steel: $F_y = 33$ ksi
2. Section: 550S162-33 as shown above
3. Concentrically loaded
4. Braced for buckling about the x-axis at ends only
5. Braced for buckling about the y-axis and for torsion at ends and mid-span
6. $K_x = K_y = K_t = 1.0$

Required:

1. Permitted applied load, P_{live}, using ASD and LRFD methods using the "all steel design" approach as described in Section D4(a) of the *Specification*.

Solution:

1. Axial strength

a. Check flexural buckling (Section C4.1).

$$\frac{K_xL_x}{r_x} = \frac{(1.0)(108.0)}{2.11} = 51.2$$

$$\frac{K_yL_y}{r_y} = \frac{(1.0)(54.0)}{0.589} = 91.7$$

Since $\frac{K_yL_y}{r_y} > \frac{K_xL_x}{r_x}$, Euler buckling about the y axis will control.

$$F_e = \frac{\pi^2 E}{\left(K_yL_y/r_y\right)^2} \qquad (Eq.\ C4.1\text{-}1)$$

$$= \frac{\pi^2(29500)}{\left[(1.0)(54.0)/(0.589)\right]^2}$$

$$= 34.64 \text{ ksi}$$

b. Check torsional-flexural buckling (Section C4.2).

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

$$\beta = 1 - (x_o/r_o)^2 \qquad (Eq.\ C4.2\text{-}3)$$

$$= 1 - (-1.11/2.46)^2$$

$$\beta = 0.796$$

$$\sigma_{ex} = \frac{\pi^2 E}{(K_x L_x/r_x)^2} \qquad (Eq.\ C3.1.2.1\text{-}7)$$

$$= \frac{\pi^2(29500)}{\left[(1.0)(108.0)/2.11\right]^2}$$

$$= 111.1 \text{ ksi}$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$= \frac{1}{(0.327)(2.46)^2}\left[(11300)(0.000130) + \frac{\pi^2(29500)(0.713)}{\left[(1.0)(54.0)\right]^2}\right]$$

$$= 36.72 \text{ ksi}$$

$$F_e = \frac{1}{(2)(0.795)}\left[(111.1 + 36.72) - \sqrt{(111.1 + 36.72)^2 - (4)(0.795)(111.1)(36.72)}\right]$$

$$= 33.71 \text{ ksi}$$

c. Determine controlling buckling mode.

33.71 ksi < 34.64 ksi, therefore torsional-flexural buckling governs

F_e = 33.71 ksi

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4\text{-}4)$$

$$= \sqrt{\frac{33}{33.71}}$$

$$= 0.989 < 1.5, \text{ therefore}$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4\text{-}2)$$

$$= \left[0.658^{(0.989)^2}\right]33$$

$$= 21.91 \text{ ksi}$$

d. Compute effective area at $f = F_n$ = 21.91 ksi

Check flange as a uniformly compressed element with an edge stiffener.

w = 1.625 - 2(0.0765 + 0.0346) = 1.403 in.

w/t = 1.403/0.0346 = 40.5

$$S = 1.28\sqrt{E/f} \qquad (Eq.\ B4\text{-}1)$$

$= 1.28\sqrt{29500/21.91} = 47.0 \therefore w/t \geq 0.328S \Rightarrow$ check effective width of flange

Compute flange k based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \qquad (Eq.\ B4.2\text{-}10)$$

$$= 399(0.0346)^4\left[\frac{40.5}{47.0} - 0.328\right]^3 \le (0.0346)^4\left[115\left(\frac{40.5}{47.0}\right) + 5\right]$$

$= 0.0000869$ in.^{4} < 0.000149 in.^{4} $\therefore$ $I_a = 0.0000869$ in.^{4}

d = 0.500 - 0.0765 - 0.0346 = 0.389 in.

θ = 90 degrees = $\pi/2$ radians

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

$$= (0.389)^3\,(0.0346)\sin^2\left(\frac{\pi}{2}\right)/12 = 0.000170 \text{ in.}^4$$

$R_I = I_s/I_a \le 1$ (*Eq.* B4.2-9)

$= 0.000170/0.0000869 = 1.96 > 1$ $\therefore$ $R_I = 1.0$

D/w= 0.500/1.403 = 0.356 < 0.8 OK (From Table B4.2)

$$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \le 4 \qquad \text{(From Table B4.2)}$$

$$= \left(4.82 - \frac{(5)(0.500)}{1.403}\right)(1.0)^n + 0.43 = 3.47 < 4 \text{ OK}$$

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

$$= 3.47\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{40.5}\right)^2 = 56.41 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{21.91}{56.41}} = 0.623 < 0.673 \therefore \text{ flange is fully effective}$$

Check stiffener lip using Section B3.1

f = 21.91 ksi

k = 0.43

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

$$= 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.0346}{0.389}\right)^2 = 90.70 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{21.91}{90.70}} = 0.491 < 0.673 \therefore \text{ lip is fully effective}$$

Check web with punchout

Per Section D4, treat web as two unstiffened elements, one on each side of the 1.50 inch wide punchout.

w = [5.50 - 2(0.0765 + 0.0346) - 1.50]/2 = 1.889 in.

k = 0.43

$$F_{cr} = 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.0346}{1.889}\right)^2 = 3.846 \text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{21.91}{3.846}} = 2.387 > 0.673$$

$$\rho = (1 - 0.22/\lambda)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$

$$= (1 - 0.22/2.387)/2.387 = 0.380$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$$b = (0.380)(1.889) = 0.718 \text{ in.}$$

Compute A_e by subtracting the hole and ineffective area of the web from the gross section.

$$A_e = 0.327 - (0.0346)\left[1.50 + (2)(1.889 - 0.718)\right]$$

$$= 0.194 \text{ in.}^2$$

e. Compute P_n

$$P_n = A_eF_n \qquad (Eq.\ C4\text{-}1)$$

$$= (0.194)(21.91)$$

$$P_n = 4.25 \text{ kips}$$

Alternately, P_n may be taken from Table III-5. For the case of a 550S162-33 with a yield point of 33 ksi, unbraced length of 9.0 feet in the x-axis and braced at mid-span, P_n = 4.25 kips.

f. Compute maximum permissible applied live load, P_{live}

ASD

$$P_{live} \le \frac{P_n}{\Omega_c} \text{ where } \Omega_c = 1.80 \qquad (Eq.\ A4.1.1\text{-}1)$$

$$\le \frac{4.25}{1.80}$$

$$P_{live} \le 2.36 \text{ kips}$$

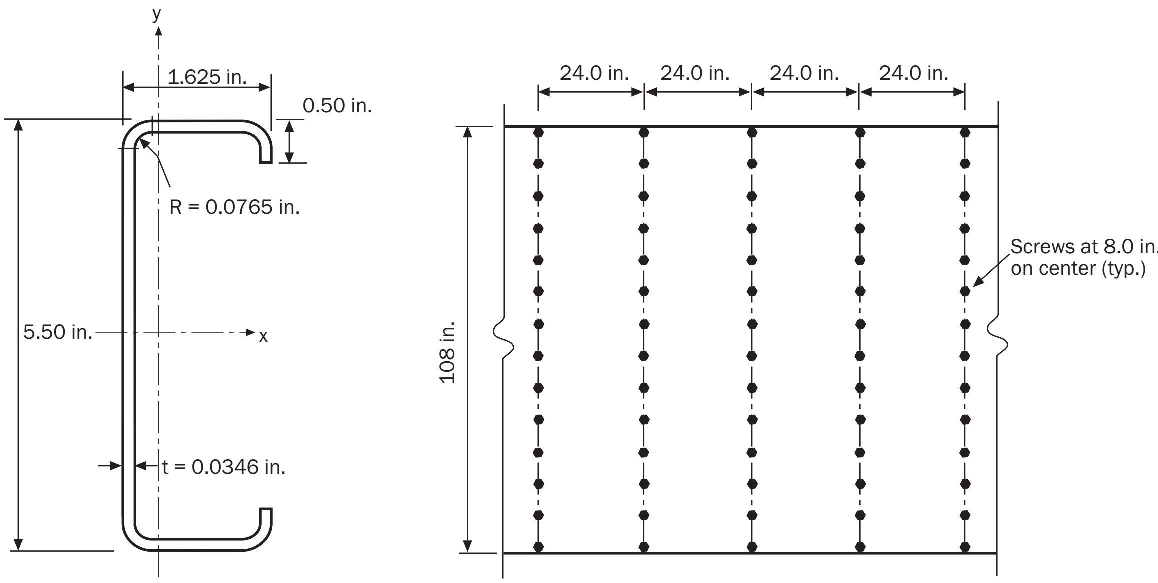
LRFD

$$1.6P_{live} \le \phi_c P_n \text{ where } \phi_c = 0.85 \qquad (Eq.\ A5.1.1\text{-}1)$$

$$P_{live} \le \frac{(0.85)(4.25)}{1.6}$$

$$\le 2.26 \text{ kips}$$

Example III-3: Sheathed Stiffened C-Stud - Bending And Compression



A	$= 0.327 \text{ in.}^2$	J	$= 0.000130 \text{ in.}^4$
I_x	$= 1.46 \text{ in.}^4$	C_w	$= 0.713 \text{ in.}^6$
r_x	$= 2.11$ in.	x_o	$= -1.11$ in.
r_y	$= 0.589$ in.	r_o	$= 2.46$ in.

Note: y-y axis is perpendicular to wall board.

Given:

1. Steel: $F_y = 33$ ksi
2. Section: 550S162-33 as shown with standard 1.5 in. by 4.5 in. punchouts at 24 in. on center
3. Stud spacing: 24 in. O.C.
4. Length: 108.0 in.
5. Cladding: On both sides, 1/2 in. gypsum board with No. 6 Type S-12 self-drilling screws at 8 in. O.C. vertically

Required:

1. The ASD axial allowable design strength
2. The permitted ASD axial load in combination with 5 psf lateral live load

   Consider the contribution of sheathing.

Solution:

1. No Lateral Load (Section D4.1)

a. Check column buckling strength between fasteners using KL of two times the fastener spacing per *Specification* Section D4.1(a).

   $KL/r_y = (2)(8.0)/0.589 = 27.17$

For flexural buckling about the y axis

$$F_e = \frac{\pi^2 E}{\left(KL/r_y\right)^2} \qquad (Eq.\ C4.1\text{-}1)$$

$$= \frac{\pi^2(29500)}{(27.17)^2} = 394 \text{ ksi}$$

Torsional-flexural buckling is not a mode of failure since the section is braced on both sides by gypsum board

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{33}{394}} = 0.289 \qquad (Eq.\ C4\text{-}4)$$

Since $\lambda_c < 1.5$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4\text{-}2)$$

$$= \left(0.658^{(0.289)^2}\right)33 = 31.9 \text{ ksi}$$

b. Check flexural and/or torsional overall column buckling strength using the lesser of σ_{CR} from Equations D4.1-2 or D4.1-3

$$\sigma_{CR} = \sigma_{ey} + \overline{Q}_a \qquad (Eq.\ D4.1\text{-}2)$$

$$\sigma_{ey} = \frac{\pi^2 E}{\left(L/r_y\right)^2} \qquad (Eq.\ D4.1\text{-}10)$$

$$= \frac{\pi^2(29500)}{\left[108/0.589\right]^2} = 8.66 \text{ ksi}$$

$$\overline{Q} = \overline{Q}_o(2 - s/s') \qquad (Eq.\ D4.1\text{-}13)$$

$\overline{Q}_o$ = 24.0 kips (from Table D4)

s = 8.0 inches

s′ = 12.0 inches

$$\overline{Q} = 24.0(2 - 8.0/12.0) = 32.0 \text{ kips} \qquad (Eq.\ D4.1\text{-}13)$$

$$\overline{Q}_a = \overline{Q}/A \qquad (Eq.\ D4.1\text{-}14)$$

$$= 32.0/0.327 = 97.9 \text{ ksi}$$

$$\sigma_{CR} = 8.66 + 97.9 = 107 \text{ ksi} \qquad (Eq.\ D4.1\text{-}2)$$

OR

$$\sigma_{CR} = \frac{1}{2\beta}\left[\left(\sigma_{ex} + \sigma_{tQ}\right) - \sqrt{\left(\sigma_{ex} + \sigma_{tQ}\right)^2 - 4\beta\sigma_{ex}\sigma_{tQ}}\right] \qquad (Eq.\ D4.1\text{-}3)$$

$$\sigma_{ex} = \frac{\pi^2 E}{\left(L/r_x\right)^2} \qquad (Eq.\ D4.1\text{-}8)$$

$$= \frac{\pi^2(29500)}{\left[108/2.11\right]^2} = 111 \text{ ksi}$$

$$\sigma_{tQ} = \sigma_t + \overline{Q}_t \qquad (Eq.\ D4.1\text{-}12)$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{L^2}\right] \qquad (Eq.\ D4.1\text{-}11)$$

$$= \frac{1}{(0.327)(2.46)^2}\left[(11300)(0.000130) + \frac{\pi^2(29500)(0.713)}{(108)^2}\right] = 9.74 \text{ ksi}$$

$\overline{Q}_t = (\overline{Q}d^2)/(4Ar_o^2)$ (*Eq*. D4.1-15)

$= (32)(5.50)^2/\left[(4)(0.327)(2.46)^2\right] = 122.3$ ksi

$\sigma_{tQ} = 9.74 + 122.3 = 132$ ksi (*Eq*. D4.1-12)

$\beta = 1 - (x_o/r_o)^2$

$= 1 - (-1.11/2.46)^2 = 0.796$

$$\sigma_{CR} = \frac{1}{(2)(0.796)}\left[(111 + 132) - \sqrt{(111 + 132)^2 - (4)(0.796)(111)(132)}\right] \quad (Eq.\ D4.1\text{-}3)$$

$= 82.7$ ksi

Use $F_e = 82.7$ ksi

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{33}{82.7}} = 0.632 < 1.5 \quad (Eq.\ C4\text{-}4)$$

therefore,

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \quad (Eq.\ C4\text{-}2)$$

$$= \left(0.658^{(0.632)^2}\right)33 = 27.9 \text{ ksi}$$

c. Check shear strain of wall material

$C_o = L/350$ (*Eq*. D4.1-22)

$= (108)/350 = 0.309$ in.

$D_o = L/700$ (*Eq*. D4.1-23)

$= 108/700 = 0.154$ in.

$E_o = L/[(d)(10{,}000)]$ (*Eq*. D4.1-24)

$= 108/[(5.50)(10{,}000)]$

$= 0.00196$ rad

Let initial trial value of F_n be the lower of F_n calculated in (a) or (b) above.

Assume $F_n = 27.9 \text{ ksi} > F_y/2$

$E' = 4EF_n(F_y - F_n)/F_y^2$ (*Eq*. D4.1-25)

$= (4)(29{,}500)(27.9)(33 - 27.9)/(33)^2 = 15400$ ksi

$G' = G(E'/E) = (11300)(15400/29500)$ (*Eq*. D4.1-26)

$= 5900$ ksi

$C_1 = (F_nC_o)/(\sigma_{ey} - F_n + \overline{Q}_a)$ (*Eq*. D4.1-17)

$$\sigma_{ey} = \frac{\pi^2E'}{\left(L/r_y\right)^2} \quad (Eq.\ D4.1\text{-}10)$$

$$= \frac{\pi^2(15400)}{\left(108/0.589\right)^2} = 4.52 \text{ ksi}$$

$\overline{Q}_a = 97.9$ ksi (calculated above)

$C_1 = (27.9)(0.309)/(4.52 - 27.9 + 97.9) = 0.116$ (*Eq*. D4.1-17)

$$E_1 = \frac{F_n\left[(\sigma_{ex} - F_n)(r_o^2E_o - x_oD_o) - F_nx_o(D_o - x_oE_o)\right]}{(\sigma_{ex} - F_n)r_o^2\left(\sigma_{tQ} - F_n\right) - (F_nx_o)^2} \text{ (absolute value)} \quad (Eq.\ D4.1\text{-}18)$$

$$\sigma_{ex} = \frac{\pi^2E'}{\left(L/r_x\right)^2} \quad (Eq.\ D4.1\text{-}8)$$

$$= \frac{\pi^2(15400)}{(108/2.11)^2} = 58.0 \text{ ksi}$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[G'J + \frac{\pi^2 E' C_w}{L^2}\right] \qquad (Eq.\ D4.1\text{-}11)$$

$$= \frac{1}{(0.327)(2.46)^2}\left[(5900)(0.000130) + \frac{\pi^2(15400)(0.713)}{(108)^2}\right] = 5.08 \text{ ksi}$$

$$\sigma_{tQ} = \sigma_t + \overline{Q}_t$$

$$= 5.08 + 122.3 = 127 \text{ ksi} \qquad (Eq.\ D4.1\text{-}12)$$

$$E_1 = \frac{27.9\left\{\begin{matrix}(58.0 - 27.9)\left[(2.46)^2(0.00196) - |\text{-}1.11|(0.154)\right] \\ - (27.9)|\text{-}1.11|\left[0.154 - |\text{-}1.11|(0.00196)\right]\end{matrix}\right\}}{(58.0 - 27.9)(2.46)^2(127 - 27.9) - \left[(27.9)|\text{-}1.11|\right]^2} \qquad (Eq.\ D4.1\text{-}18)$$

= - 0.0155, use absolute value = 0.0155

$\bar{\gamma}$ = 0.008 in./in. (from Table D4)

$\gamma = (\pi/L)[C_1 + (E_1\, d/2)]$ (*Eq.* D4.1-16)

$= [\pi/(108)][0.116 + (0.0155)(5.50)/2]$

$= 0.00461 < 0.008$ therefore flexural-torsional buckling governs the strength

By trial and error iteration, it can be shown that the shear strain will not exceed the limit of 0.008 for $F_n \le 29.9$ ksi.

Calculate A_e at f = 27.9 ksi, the smallest value of F_n.

By calculations, not shown, similar to those in Example III-2, A_e at f = 27.9 ksi is determined to be 0.187 in.^{2}

$P_n = A_e F_n$ (*Eq.* D4.1-1)

$= (0.187)(27.9) = 5.22$ kips

ASD Allowable Design Strength

$\Omega_c = 1.80$

$P = P_n/\Omega_c$

$= 5.22/1.80$

$= 2.90$ kips

2. Allowable Axial Load with 5 psf Lateral Live Load (Section D4.3)

$$M_x = \frac{wL^2}{8}$$

$$= \frac{[(0.005)(2.0)/12](108)^2}{8} = 1.22 \text{ kip-in.}$$

$$M_y = 0$$

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} + \frac{\Omega_b C_{my} M_y}{M_{ny}\alpha_y} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

Assume C_{mx} = 1.0 (braced against joint translation in the plane of loading, subject to transverse loading between supports with member ends unrestrained)

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} \qquad (Eq.\ C5.2.1\text{-}4)$$

$\Omega_c = 1.80$

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_xL_x)^2} \qquad (Eq.\ C5.2.1\text{-}6)$$

$$= \frac{\pi^2(29500)(1.46)}{(108)^2} = 36.4 \text{ kips}$$

$M_{nx} = 16.9$ kip-in. (from Table II-2)

$\Omega_b = 1.67$

$$\frac{1.80P}{5.22} + \frac{(1.67)(1.0)(1.22)}{(16.9)\left(1 - \frac{1.80P}{36.4}\right)} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

Solving for P gives:

$P \leq 2.50$ kips

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

$P_{no} = 5.83$ kip-in. (from Table III-2b)

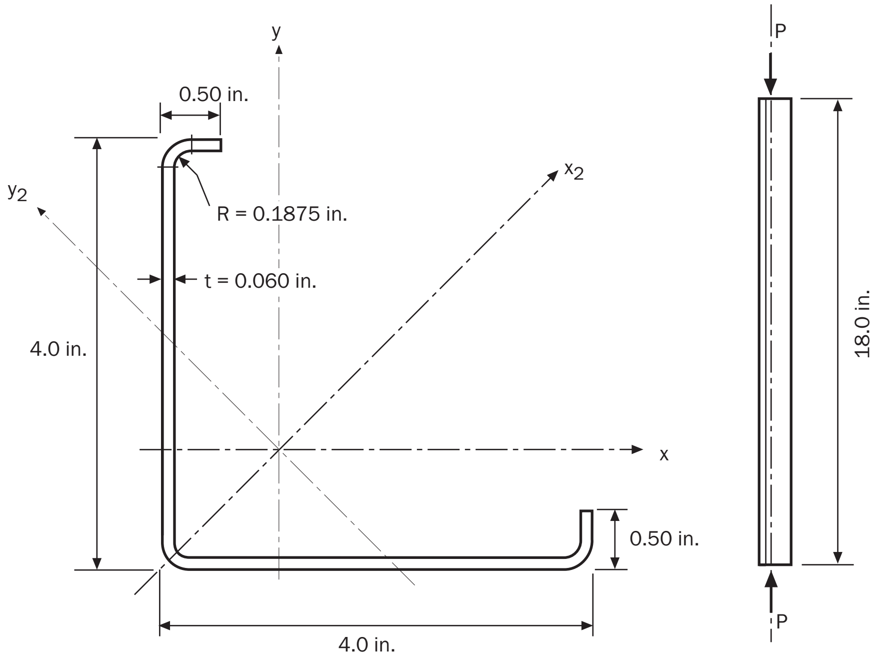
$$\frac{1.80P}{5.83} + \frac{(1.67)(1.22)}{16.9} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

Solving for P gives:

$P \leq 2.85$ kips

Therefore, $P \leq 2.50$ kips

Example III-4: Unbraced Equal Leg Angle With Lips - Compression



From Table I-5 or Example I-4

$A = 0.512 \text{ in.}^2$
$I_x = 0.960 \text{ in.}^4$
$I_{y2} = 0.397 \text{ in.}^4$
$r_{y2} = 0.881$ in.
$x_o = -1.633$ in.
$r_o = 2.53$ in.
$J = 0.000615 \text{ in.}^4$
$C_w = 0.0533 \text{ in.}^6$
$j = 3.13$ 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:

1. 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.

a. Flexural buckling (Section C4.1)

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \quad (Eq.\ C4.1\text{-}1)$$

use least radius of gyration, r_{y2}

$$F_e = \frac{\pi^2(29500)}{[(1.0)(18.0)/0.881]^2} = 697.5 \text{ ksi} \quad (Eq.\ C4.1\text{-}1)$$

b. Torsional-flexural buckling (Section C4.2)

$$F_e = \frac{1}{2\beta}\left[(\sigma_{ex2} + \sigma_t) - \sqrt{(\sigma_{ex2} + \sigma_t)^2 - 4\beta\sigma_{ex2}\sigma_t}\right] \quad (Eq.\ C4.2\text{-}1)$$

where the x_2 axis is the axis of symmetry.

$$\beta = 1 - (x_o / r_o)^2 \quad (Eq.\ C4.2\text{-}3)$$

$$= 1 - \left(\frac{-1.633}{2.53}\right)^2 = 0.583$$

$$\sigma_{ex2} = \frac{\pi^2 E}{\left(K_{x2}L_{x2}/r_{x2}\right)^2} \qquad (Eq.\ C3.1.2.1\text{-}7)$$

For the case of an equal leg angle, the radius of gyration about the axis of symmetry, I_{x2}, can be computed as:

$$I_{x2} = 2I_x - I_{y2}$$

$$= (2)(0.960) - 0.397 = 1.523 \text{ in.}^4$$

$$r_{x2} = \sqrt{\frac{I_{x2}}{A}}$$

$$= \sqrt{\frac{1.523}{0.512}} = 1.725 \text{ in.}$$

$$K_{x2} = K = 1.0$$

$$\sigma_{ex2} = \frac{\pi^2(29500)}{\left[(1.0)(18.0)/1.725\right]^2} \qquad (Eq.\ C3.1.2.1\text{-}7)$$

$$= 2674 \text{ ksi}$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_tL_t)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$= \frac{1}{(0.512)(2.53)^2}\left[(11300)(0.000615) + \frac{\pi^2(29500)(0.0533)}{\left[(1.0)(18.0)\right]^2}\right]$$

$$= 16.7 \text{ ksi}$$

$$F_e = \frac{1}{(2)(0.583)}\left[(2674 + 16.7) - \sqrt{(2674 + 16.7)^2 - (4)(0.583)(2674)(16.7)}\right] \qquad (Eq.\ C4.2\text{-}1)$$

$$= 16.7 \text{ ksi} \qquad \text{CONTROLS}$$

c. Nominal axial strength (Section C4)

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4\text{-}4)$$

$$= \sqrt{\frac{50}{16.7}} = 1.73 > 1.5$$

$$F_n = \left[\frac{0.877}{\lambda_c^2}\right]F_y \qquad (Eq.\ C4\text{-}3)$$

$$= \left[\frac{0.877}{(1.73)^2}\right]50 = 14.7 \text{ ksi}$$

$$P_n = A_eF_n \qquad (Eq.\ C4\text{-}1)$$

From Example I-11 at a uniform compression stress of 14.7 ksi,

$$A_e = 0.383 \text{ in.}^2$$

$$P_n = (0.383)(14.7) = 5.63 \text{ kips} \qquad (Eq.\ C4\text{-}1)$$

2. ASD Allowable Design Strength not considering minimum eccentricity

Ω_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(b) and C5.2 of the *Specification* include the effect 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.

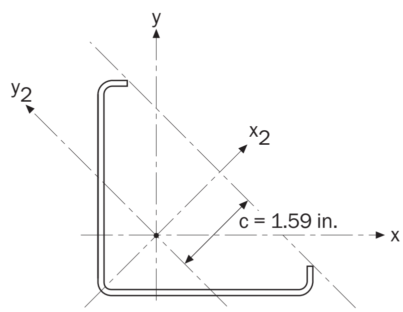
$$F_e = \frac{C_s A \sigma_{ex2}}{C_{TF} S_f}\left[j + C_s\sqrt{j^2 + r_o^2(\sigma_t/\sigma_{ex2})}\right] \qquad (Eq.\ C3.1.2.1\text{-}6)$$

$$S_{y2} = \frac{I_{y2}}{c}$$

$$= \frac{0.397}{1.59} = 0.250 \text{ in.}^3$$

σ_t = 16.7 ksi (computed above)

σ_{ex2}= 2674 ksi (computed above)



C_{TF}= 1

C_s = -1 (assume worst case - tension on shear center side of centroid)

$$F_e = \frac{(-1)(0.512)(2674)}{(1)(0.250)}\left[3.13 + (-1)\sqrt{(3.13)^2 + (2.53)^2(16.7/2674)}\right] \qquad (Eq.\ C3.1.2.1\text{-}6)$$

= 34.94 ksi

Since $2.78F_y > F_e > 0.56F_y$,

$$F_c = \frac{10}{9}F_y\left(1 - \frac{10F_y}{36F_e}\right) \qquad (Eq.\ C3.1.2.1\text{-}3)$$

$$= \frac{10}{9}(50.0)\left(1 - \frac{(10)(50.0)}{(36)(34.94)}\right) = 33.47 \text{ ksi}$$

Assume that the cross section is fully effective in bending due to the small bending moment. This assumption will be verified below.

S_c = S_{y2}

M_n = S_cF_c (*Eq.* C3.1.1-1)

= (0.250)(33.47) = 8.37 kip-in.

4. ASD Allowable Design Strength including minimum eccentricity

M_{y2} = PL/1000

= P(18.0)/1000 = 0.0180P

Assume $\frac{\Omega_c P}{P_n} > 0.15$ therefore, solve for the allowable load, P, such that equations C5.2.1-1 and C5.2.1-2 are satisfied.

C_{my2}= 1.0

$K_{y2} = K = 1.0$

$$P_{Ey2} = \frac{\pi^2 EI_{y2}}{(K_{y2}L_{y2})^2} \qquad (Eq.\ C5.2.1\text{-}7)$$

$$= \frac{\pi^2(29500)(0.397)}{\left[(1.0)(18.0)\right]^2} = 357 \text{ kips}$$

$$\alpha_{y2} = 1 - \frac{\Omega_c P}{P_{Ey2}} \qquad (Eq.\ C5.2.1\text{-}5)$$

$$= 1 - \frac{(1.80)(P)}{357}$$

$$= 1 - 0.00504P$$

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{my2} M_{y2}}{M_{ny2}\alpha_{y2}} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

$$\frac{(1.80)(P)}{5.63} + \frac{(1.67)(1.0)(0.0180P)}{(8.37)(1-0.00504P)} \leq 1.0$$

Solving for P:

$P \leq 3.09$ kips

By calculations, not shown, similar to those in Example I-11, P_{no} is calculated at $f = F_y$.

$P_{no} = 10.5$ kips

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_{y2}}{M_{ny2}} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

$$\frac{(1.80)(P)}{10.5} + \frac{(1.67)(0.0180P)}{(8.37)} \leq 1.0$$

Solving for P:

$P \leq 5.71$ kips

Therefore, equation C5.2.1-1 controls

$P \leq 3.09$ kips

Check assumption that section is fully effective at the moment of PL/1000

$$f = \frac{M_{y2}}{S_{y2}}$$

$$= \frac{0.018P}{S_{y2}} = \frac{(0.018)(3.09)}{0.250} = 0.22 \text{ ksi}$$

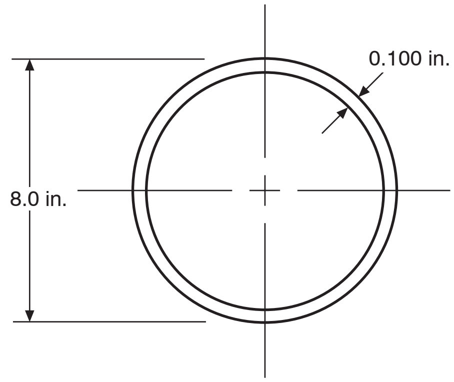
All elements of the section are fully effective at this low stress (calculations not shown); therefore, $S_e = S_{gross}$ as assumed above.

Check assumption that $\dfrac{\Omega_c P}{P_n} > 0.15$

$$\frac{\Omega_c P}{P_n} = \frac{(1.80)(3.09)}{5.63} = 0.99 > 0.15 \quad \text{OK}$$

Therefore, the ASD allowable design strength is 3.09 kips.

Example III-5: Tubular Section - Round - Bending and Compression



Given:

1. Steel: F_y = 50 ksi
2. Section: Shown in sketch above
3. Height: L = 10′-0″, simply supported at each end
4. Axial Loads: Dead = P_D = 7.5 kips, Live = P_L = 20 kips
5. Transverse Concentrated Wind Load (at midspan): P_W = 3.6 kips

Required:

Check the adequacy of the section using ASD and LRFD methods with ASCE 7-98 load combinations.

Solution:

1. Nominal Axial Strength, P_n (Section C6.2)

Ratio of outside diameter to wall thickness

$D/t = 8.0/0.100 = 80.0$

$D/t < 0.441E/F_y = 0.441(29500/50) = 260$ OK

Compute λ_c

$$I = \frac{\pi}{4}\left[(\text{O.R.})^4 - (\text{I.R.})^4\right]$$
$$= \frac{\pi}{4}\left[(4.0)^4 - (3.900)^4\right]$$
$$= 19.37 \text{ in.}^4$$

$$A = \frac{\pi}{4}\left[(\text{O.D.})^2 - (\text{I.D.})^2\right]$$
$$= \frac{\pi}{4}\left[(8.0)^2 - (7.800)^2\right]$$
$$= 2.482 \text{ in.}^2$$

$$r = \sqrt{I/A}$$
$$= \sqrt{19.37/2.482} = 2.794 \text{ in.}$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad (Eq.\ C4.1\text{-}1)$$
$$= \frac{\pi^2(29500)}{\left[(10)(12)/2.794\right]^2} = 158 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{50}{158}} = 0.563 \qquad (Eq.\ C6.2\text{-}4)$$

Since $\lambda_c \le 1.5$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C6.2\text{-}2)$$

$$= \left(0.658^{(0.563)^2}\right)50 = 43.8 \text{ ksi}$$

$$A_o = \left[\frac{0.037}{\left(DF_y\right)/(tE)} + 0.667\right]A \le A \qquad (Eq.\ C6.2\text{-}7)$$

$$= \left[\frac{0.037}{(8.000)(50)/[(0.100)(29500)]} + 0.667\right]2.482$$

$$= 2.333 \text{ in.}^2 < A$$

$$R = \frac{F_y}{2F_e} \le 1.0 \qquad (Eq.\ C6.2\text{-}6)$$

$$= \frac{50}{(2)(158)} = 0.158 < 1.0$$

$$A_e = A_o + R(A - A_o) \qquad (Eq.\ C6.2\text{-}5)$$

$$A_e = 2.333 + 0.158(2.482 - 2.333) = 2.357 \text{ in.}^2$$

$$P_n = F_nA_e \qquad (Eq.\ C6.2\text{-}1)$$

$$= (43.8)(2.357)$$

$$= 103 \text{ kips}$$

Compute P_{no} for use in Section C6.3 and C5.2

$$F_n = F_y$$

$$P_{no} = F_yA_o \qquad (Eq.\ C6.2\text{-}1)$$

$$P_{no} = (50)(2.333) = 117 \text{ kips}$$

2. Nominal Flexural Strength, M_n (from Example II-5)

$$M_n = 271 \text{ kip-in.}$$

3. Combined Bending and Compression

$$M_W = PL/4 = (3.6)(10)(12)/4 = 108 \text{ kip-in.}$$

ASD

ASCE 7-98 load combinations considered:

D + L

D + W

D + 0.75(L + W)

Controlling load combination (by inspection) is D + 0.75(L + W)

$$P = P_D + 0.75P_L = 7.5 + (0.75)(20) = 22.5 \text{ kips}$$

$$M_x = 0.75M_W = (0.75)(108) = 81.0 \text{ kip-in.}$$

$$\Omega_c = 1.80$$

$$\Omega_b = 1.67$$

$$\frac{\Omega_cP}{P_n} = \frac{(1.80)(22.5)}{103} = 0.39 > 0.15$$ therefore use Equations C5.2.1-1 and C5.2.1-2

$C_{mx} = 1.0$

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_xL_x)^2} \qquad (Eq.\ C5.2.1\text{-}6)$$

$$= \frac{\pi^2(29500)(19.37)}{\left[(1.0)(120)\right]^2} = 392 \text{ kips}$$

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} \qquad (Eq.\ C5.2.1\text{-}4)$$

$$= 1 - \frac{(1.80)(22.5)}{392} = 0.897$$

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

$$\frac{(1.80)(22.5)}{103} + \frac{(1.67)(1.0)(81.0)}{(271)(0.897)} = 0.950 < 1.0 \quad \text{OK}$$

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} \leq 1.0 \qquad (Eq.\ C5.2.1.\text{-}2)$$

$$\frac{(1.80)(22.5)}{117} + \frac{(1.67)(81.0)}{(271)} = 0.845 < 1.0 \quad \text{OK}$$

LRFD

ASCE 7-98 load combinations considered:

1.4D

1.2D + 1.6L

1.2D + 1.6W + 0.5L

0.9D + 1.6W

Controlling load combination (by inspection) is 1.2D + 1.6W + 0.5L

$\overline{P} = P_u = 1.2P_D + 0.5P_L = (1.2)(7.5) + (0.5)(20) = 19.0$ kips

$\overline{M} = M_u = 1.6M_W = (1.6)(108.0) = 173$ kip-in.

$\phi_c = 0.85$

$\phi_b = 0.95$

$$\frac{\overline{P}}{\phi_c P_n} = \frac{19.0}{(0.85)(103)} = 0.22 > 0.15 \text{ therefore use Equations C5.2.2-1 and C5.2.2-2}$$

$C_{mx} = 1.0$

$$\alpha_x = 1 - \frac{\overline{P}}{P_{Ex}} \qquad (Eq.\ C5.2.2\text{-}4)$$

$$= 1 - \frac{19.0}{392} = 0.952$$

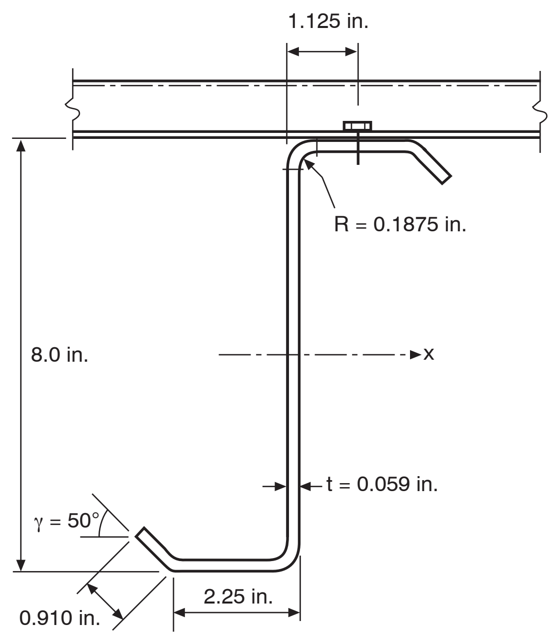
$$\frac{\overline{P}}{\phi_c P_n} + \frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x} \leq 1.0 \qquad (Eq.\ C5.2.2\text{-}1)$$

$$\frac{19.0}{(0.85)(103)} + \frac{(1.0)(173)}{(0.95)(271)(0.952)} = 0.923 < 1.0 \quad \text{OK}$$

$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} \leq 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

$$\frac{19.0}{(0.85)(117)} + \frac{173}{(0.95)(271)} = 0.863 < 1.0 \quad \text{OK}$$

Example III-6: Stiffened Z-Section With One Flange Through-Fastened To Deck Or Sheathing - Compression



Given:

1. Steel: F_y = 55 ksi
3. Span = 25 ft. = 300 in.
2. Section: 8ZS2.25x059

D = 8.0 in.

B = 2.25 in.

t = 0.059 in.

A = 0.822 in.^{2}

r_x = 3.07 in.

Required:

Compression design strength, ASD and LRFD

Solution:

1. Nominal Axial Strength, P_n - Flexural Buckling about the X-axis

$K = 1$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad (Eq.\ C4.1\text{-}1)$$

$$= \frac{\pi^2(29500)}{(300/3.07)^2} = 30.5 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4\text{-}4)$$

$$= \sqrt{\frac{55}{30.5}} = 1.34 < 1.50$$

$F_n = \left(0.658^{\lambda_c^2}\right)F_y$ (*Eq*. C4-2)

$= \left(0.658^{1.34^2}\right)(55) = 25.9$ ksi

$P_n = A_eF_n$ (*Eq*. C4-1)

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

$P_n = C_1C_2C_3AE/29500$ (*Eq*. C4.6-1)

$x = a/b$ (*Eq*. C4.6-5)

$= 1.125/2.25 = 0.50$

$C_1 = 0.79x + 0.54$ (*Eq*. C4.6-2)

$= (0.79)(0.50) + 0.54 = 0.935$

$C_2 = 1.17\alpha t + 0.93$ (*Eq*. C4.6-3)

$= (1.17)(1.0)(0.059) + 0.93 = 1.00$

$C_3 = \alpha(2.5b - 1.63d) + 22.8$ (*Eq*. C4.6-4)

$= 1\left[(2.5)(2.25) - (1.63)(8.00)\right] + 22.8 = 15.4$

$P_n = (0.935)(1.00)(15.4)(0.822)(29500)/29500$ (*Eq*. C4.6-1)

$= 11.8$ kips

3. Governing Limit State is Flexural-Torsional Buckling

$P_n =$ minimum of 11.8 kips or 15.0 kips = 11.8 kips

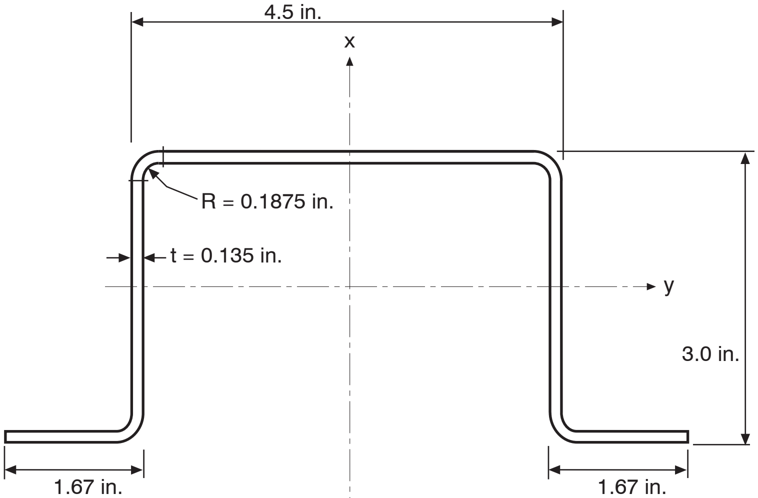
4. ASD Allowable Design Strength

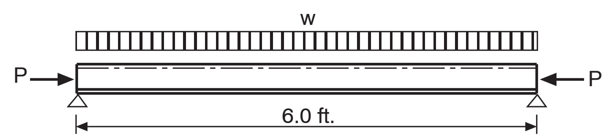
$P_n/\Omega = 11.8/1.80 = 6.56$ kips

5. LRFD Design Strength

$\phi P_n = (0.85)(11.8) = 10.0$ kips

Example III-7: Hat Section - Bending and Compression





Given:

1. Steel: F_y = 50 ksi
2. Section: 3HU4.5x135 as shown above
3. L = 6 ft., simply supported with continuous lateral and torsional bracing of the compression flange
4. Axial Loads: P_D = 2 kips, P_L = 10 kips
5. Transverse Uniform Flexural Loads: w_D = 0.090 kips/ft, w_L = 0.360 kips/ft

Required:

Check the adequacy of the section using ASD and LRFD method. Do not use inelastic reserve capacity.

Solution:

1. Bending Moments

$$M_D = \frac{w_D L^2}{8} = \frac{(0.090)(6)^2}{8} = 0.405 \text{ kip-ft.} = 4.86 \text{ kip-in.}$$

$$M_L = \frac{w_L L^2}{8} = \frac{(0.360)(6)^2}{8} = 1.620 \text{ kip-ft.} = 19.44 \text{ kip-in.}$$

2. Nominal Flexural Strength, M_n

From Example II-4

M_n = 75.8 kip-in.

3. Nominal Axial Strength, P_n

Section is free to buckle in a plane perpendicular to the flange only

$$F_e = \frac{\pi^2 E}{(KL_y/r_y)^2} \qquad (Eq.\ C4.1\text{-}1)$$

$$= \frac{\pi^2(29,500)}{\left[(1.0)(72.0)/1.192\right]^2} = 79.8 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4\text{-}4)$$

$$= \sqrt{\frac{50.0}{79.8}} = 0.792 \le 1.5$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4\text{-}2)$$

$$= \left[0.658^{(0.792)^2}\right]50 = 38.45 \text{ ksi}$$

$$P_n = A_eF_n \qquad (Eq.\ C4\text{-}1)$$

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 38.45 ksi:

$A_e = A_{gross}$ = 1.74 in.^{2}

$$P_n = (1.74)(38.45) = 66.9 \text{ kips} \qquad (Eq.\ C4\text{-}1)$$

4. Combined Compression and Bending - ASD (Section C5.2.1)

Axial Required Allowable Strength

$P = P_D + P_L = 2.0 + 10.0 = 12.0$ kips

Flexural Required Allowable Strength

$M_y = M_D + M_L = 4.86 + 19.44 = 24.3$ kip-in.

Check Ω_cP/P_n

$\Omega_c = 1.80$

$$\frac{\Omega_cP}{P_n} = \frac{(1.80)(12.0)}{66.9} = 0.323 > 0.15$$

therefore check Equations C5.2.1-1 and C5.2.1.-2.

$C_{my} = 1.0$

$$P_{Ey} = \frac{\pi^2EI_y}{\left(K_yL_y\right)^2} \qquad (Eq.\ C5.2.1\text{-}7)$$

$$= \frac{\pi^2(29500)(2.47)}{\left[(1.0)(72.0)\right]^2} = 138.7 \text{ kips}$$

$$\alpha_y = 1 - \frac{\Omega_cP}{P_{Ey}} \qquad (Eq.\ C5.2.1\text{-}5)$$

$$\alpha_y = 1 - \frac{(1.80)(12.0)}{138.7} = 0.844 \qquad (Eq.\ C5.2.1\text{-}5)$$

$M_x = 0.0$

$\Omega_b = 1.67$

$$\frac{\Omega_cP}{P_n} + \frac{\Omega_bC_{mx}M_x}{M_{nx}\alpha_x} + \frac{\Omega_bC_{my}M_y}{M_{ny}\alpha_y} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

$$\frac{(1.80)(12.0)}{66.9} + \frac{(1.67)(1.0)(24.3)}{(75.8)(0.844)} \le 1.0$$

$0.323 + 0.634 = 0.957 < 1.0$ OK

$$\frac{\Omega_cP}{P_{no}} + \frac{\Omega_bM_x}{M_{nx}} + \frac{\Omega_bM_y}{M_{ny}} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

P_{no} = A_eF_y where A_e was calculated as 1.74 in.^{2} in Example I-13
= (1.74)(50) = 87.0 kips

$$\frac{(1.80)(12.0)}{87.0} + \frac{(1.67)(24.3)}{75.8} = 0.248 + 0.535 = 0.783 < 1.0 \quad \text{OK} \qquad (Eq.\ C5.2.1\text{-}2)$$

5. Combined Compression and Bending - LRFD (Section C5.2.2)

Required Axial Strength

$\overline{P}$ = $P_u = 1.2P_D + 1.6P_L$
= (1.2)(2.0) + (1.6)(10.0) = 18.4 kips

Required Flexural Strength

$\overline{M}_y$ = $M_{uy} = 1.2M_D + 1.6M_L$
= (1.2)(4.86) + (1.6)(19.44) = 36.9 kip-in.

Check $\overline{P}/\phi_cP_n$

ϕ_c = 0.85

$$\frac{\overline{P}}{\phi_cP_n} = \frac{18.4}{(0.85)(66.9)} = 0.324 > 0.15$$

therefore check Equations C5.2.2-1 and C5.2.2-2.

C_{my} = 1.0

$$\alpha_y = 1 - \frac{\overline{P}}{P_{Ey}} \qquad (Eq.\ C5.2.2\text{-}5)$$

P_{Ey} = 138.7 kips (calculated above)

$$\alpha_y = 1 - \frac{18.4}{138.7} = 0.867 \qquad (Eq.\ C5.2.2\text{-}5)$$

$\overline{M}_x$ = 0.0

ϕ_b = 0.95 (braced beam)

$$\frac{\overline{P}}{\phi_cP_n} + \frac{C_{mx}\overline{M}_x}{\phi_bM_{nx}\alpha_x} + \frac{C_{my}\overline{M}_y}{\phi_bM_{ny}\alpha_y} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}1)$$

$$\frac{18.4}{(0.85)(66.9)} + \frac{(1.0)(36.9)}{(0.95)(75.8)(0.867)} = 0.324 + 0.591 = 0.915 < 1.0 \quad \text{OK}$$

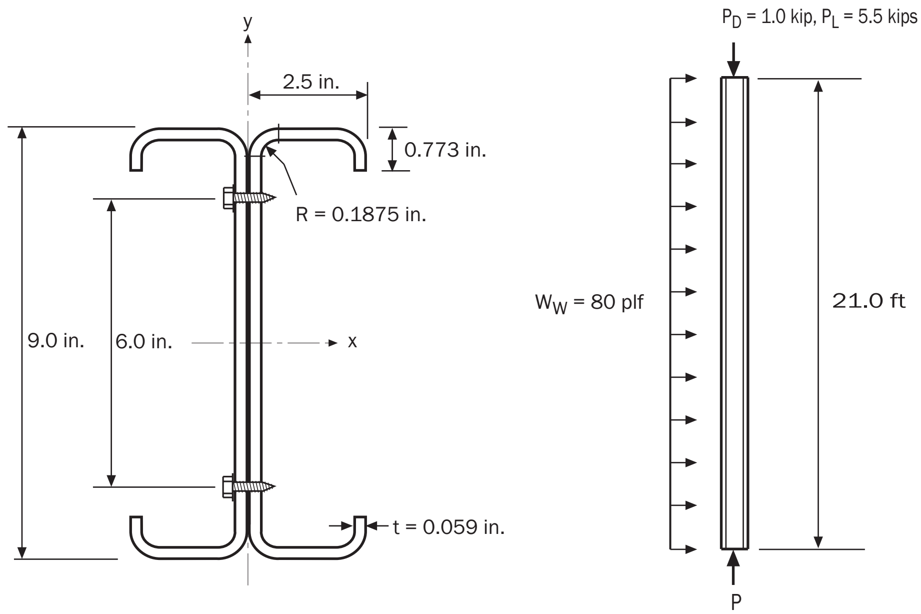
$$\frac{\overline{P}}{\phi_cP_{no}} + \frac{\overline{M}_x}{\phi_bM_{nx}} + \frac{\overline{M}_y}{\phi_bM_{ny}} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

P_{no} = 87.0 kips (calculated above)

$$\frac{18.4}{(0.85)(87.0)} + \frac{36.9}{(0.95)(75.8)} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

0.249 + 0.512 = 0.761 < 1.0 OK

Example III-8: I Section - Built-Up from Channels



Properties of each channel

$t = 0.059$ in.^{2}	$I_{xi} = 10.3$ 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.	$\overline{x}_i = 0.641$ 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 7-98 load combination D + 0.75W + 0.75L
   b. LRFD - using ASCE 7-98 load combination 1.2D + 1.6W + 0.5L

Solution:

1. Nominal Axial Strength, P_n (Section C4.5)

a. Properties of built-up section

$A = 2A_i = (2)(0.881) = 1.76$ in.^{2}

$I_x = 2I_{xi} = (2)(10.3) = 20.6 \text{ in.}^4$

$r_x = 3.42 \text{ in.}$ (same as single section)

$I_y = 2\left[I_{yi} + A_i\bar{x}_i^2\right] = 2\left[0.698 + (0.881)(0.641)^2\right] = 2.12 \text{ in.}^4$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{2.12}{1.76}} = 1.10 \text{ in.}$$

$C_w = 2C_{wi} = (2)(11.9) = 23.8 \text{ in.}^6$

$J = 2J_i = (2)(0.00102) = 0.00204 \text{ in.}^4$

$x_o = 0.0$ (distance from shear center to centroid of combined shape)

$$r_o = \sqrt{r_x^2 + r_y^2 + x_o^2} = \sqrt{3.42^2 + 1.10^2 + 0.0^2} = 3.59 \text{ in.}$$

b. x-axis flexural buckling: The buckling mode does not involve relative deformations that produce shear forces in the connectors between individual shapes, so *Equation* C4.5-1 does not apply.

$$\left(\frac{KL}{r}\right)_x = \frac{(1.0)(21.0)(12.0)}{3.42} = 73.7$$

$$F_e = \frac{\pi^2 E}{(KL/r)_x^2} \qquad (Eq.\ C4.1\text{-}1)$$

$$= \frac{\pi^2(29500)}{(73.7)^2} = 53.6 \text{ ksi}$$

c. y-axis flexural buckling: The buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, so *Equation* C4.5-1 applies.

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad (Eq.\ C4.5\text{-}1)$$

$$= \sqrt{\left(\frac{(1.0)(10.5)(12.0)}{1.10}\right)^2 + \left(\frac{36.0}{0.890}\right)^2} = 121.5 \quad \leftarrow \text{GOVERNS}$$

$a/r_i = 36.0/0.890 = 40.4 < 121.5/2$ OK

$$F_e = \frac{\pi^2 E}{(KL/r)_y^2} \qquad (Eq.\ C4.1\text{-}1)$$

$$= \frac{\pi^2(29500)}{(121.5)^2} = 19.7 \text{ ksi} \quad \leftarrow \text{CONTROLS}$$

d. Torsional buckling

$$F_e = \sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_tL_t)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$= \frac{1}{(1.76)(3.59)^2}\left[(11300)(0.00204) + \frac{\pi^2(29500)(23.8)}{\left((1.0)(10.5)(12.0)\right)^2}\right] = 20.3 \text{ ksi}$$

e. y-axis flexural buckling controls axial strength

$F_e = 19.7$ ksi

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4\text{-}4)$$

$\lambda_c = \sqrt{\frac{55.0}{19.7}} = 1.67 > 1.5$

$$F_n = \left[\frac{0.877}{\lambda_c^2}\right]F_y \qquad (\textit{Eq. } C4\text{-}3)$$

$$F_n = \left[\frac{0.877}{(1.67)^2}\right]55 = 17.3 \text{ ksi}$$

It can be shown that at an axial stress of f = 17.3 ksi, the effective area of one channel is 0.620 in^2. (calculations not shown), therefore:

$A_e = (2)(0.620) = 1.24 \text{ in.}^2$

$P_n = A_eF_n$ (*Eq*. C4-1)

$= (1.24)(17.3) = 21.5$ kips

2. Nominal Flexural Strength, M_n

a. Check maximum spacing of screws for built-up section requirements of Section D1.1

$s_{max} = L/6 \le \frac{2gT_s}{mq}$ (*Eq*. D1.1-1)

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

$P_{not} = 0.85t_cdF_{u2}$ (*Eq*. E4.4.1-1)

$= (0.85)(0.059)(0.190)(70) = 0.667$ kips

Pull-over

d_w = 0.399 in. (washer diameter)

$P_{nov} = 1.5t_1d_wF_{u1}$ (*Eq*. E4.4.2-1)

$= (1.5)(0.059)(0.399)(70) = 2.47$ kips

Screw tension

P_{ts} = 2.10 kips (from screw manufacturer)

$P_{nt} = 0.8P_{ts}$ (*Eq*. E4.4.3-1)

$= (0.8)(2.10) = 1.68$ kips

The nominal screw strength is the minimum of P_{not}, P_{nov} or P_{nt}.

$T_s = 0.667/3.0 = 0.222$ kips (ASD)

$T_s = (0.50)(0.667) = 0.334$ kips (LRFD)

Design load on beam between fasteners, q

$q = (0.75)(3)\left(\frac{0.080}{12.0}\right) = 0.0150$ kips per in. (ASD, assuming a wind load factor of 0.75)

$q = (1.6)(3)\left(\frac{0.080}{12.0}\right) = 0.0320$ kips per inch (LRFD, assuming a wind load factor of 1.6)

ASD

$$s_{max} = (21.0)(12.0)/6 \le \frac{(2)(6.0)(0.222)}{(1.05)(0.0150)} \qquad (Eq.\ D1.1\text{-}1)$$

$$s_{max} = 42.0 \text{ in.} \le 169 \text{ in.}$$

LRFD

$$s_{max} = (21.0)(12.0)/6 \le \frac{(2)(6.0)(0.334)}{(1.05)(0.0320)} \qquad (Eq.\ D1.1\text{-}1)$$

$$= 42.0 \text{ in.} \le 119 \text{ in.}$$

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 flexural strength according to Section C3.1.2.1

$$F_e = \frac{C_b r_o A}{S_f}\sqrt{\sigma_{ey}\sigma_t} \qquad (Eq.\ C3.1.2.1\text{-}5)$$

$$\sigma_{ey} = \frac{\pi^2 E}{\left(K_y L_y / r_y\right)^2} \qquad (Eq.\ C3.1.2.1\text{-}8)$$

$$= \frac{\pi^2(29500)}{\left[(1.0)(10.5)(12.0)/1.10\right]^2} = 22.2 \text{ ksi}$$

σ_t = 20.3 ksi (computed above)

C_b = 1.0

$$F_e = \frac{(1.0)(3.59)(1.76)}{(2)(2.29)}\sqrt{(22.2)(20.3)} \qquad (Eq.\ C3.1.2.1\text{-}5)$$

$$= 29.3 \text{ ksi}$$

Since $F_e < 0.56F_y = (0.56)(55) = 30.8$ ksi,

$$F_c = F_e = 29.3 \text{ ksi} \qquad (Eq.\ C3.1.2.1\text{-}4)$$

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:

$$S_c = S_{gross} = (2)(2.29) = 4.58 \text{ in.}^3$$

$$M_n = S_c F_c \qquad (Eq.\ C3.1.2.1\text{-}1)$$

$$= (4.58)(29.3) = 134 \text{ kip-in.}$$

3. Combined Compression and Bending

a. ASD - check according to Section C5.2.1

ASCE 7-98 ASD load combination D + 0.75L + 0.75W controls

Required allowable strength

$$P = P_D + 0.75P_L$$

$$= 1.0 + (0.75)(5.5) = 5.13 \text{ kips}$$

$$M_x = 0.75M_W = 0.75\frac{W_{WL}L^2}{8}$$

$$= 0.75\frac{(0.080)(21.0)^2}{8} = 3.308 \text{ kip-ft} = 39.7 \text{ kip-in.}$$

$M_y = 0$

Combined Compression and Bending

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} + \frac{\Omega_b C_{my} M_y}{M_{ny}\alpha_y} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

$$P_{Ex} = \frac{\pi^2 E I_x}{(K_x L_x)^2} \qquad (Eq.\ C5.2.1\text{-}6)$$

$$= \frac{\pi^2(29500)(20.6)}{\left[(1.0)(21.0)(12.0)\right]^2} = 94.4 \text{ kips}$$

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} \qquad (Eq.\ C5.2.1\text{-}4)$$

$$= 1 - \frac{(1.80)(5.13)}{94.4} = 0.902$$

$C_{mx} = 1.0$

$\Omega_c = 1.80$

$\Omega_b = 1.67$

$$\frac{(1.80)(5.13)}{21.5} + \frac{(1.67)(1.0)(39.7)}{(134)(0.902)} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

$0.429 + 0.549 = 0.978 < 1.0$ 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)

$$\frac{(1.80)(5.13)}{48.6} + \frac{(1.67)(39.7)}{134} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

$0.190 + 0.495 = 0.685 < 1.0$ OK

b. LRFD check according to Section C5.2.2

ASCE 7-98 LRFD load combination 1.2D + 1.6W + 0.5L controls

Required strength

$\overline{P} = P_u = 1.2P_D + 0.5P_L$

$= (1.2)(1.0) + (0.5)(5.5) = 3.95$ kips

$$\overline{M}_x = M_{ux} = 1.6\frac{W_{WL}L^2}{8}$$

$$= 1.6\frac{(0.080)(21.0)^2}{8} = 7.056 \text{ kip-ft} = 84.7 \text{ kip-in.}$$

$\overline{M}_y = 0$

Combined Compression and Bending

$$\frac{\overline{P}}{\phi_c P_n} + \frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x} + \frac{C_{my}\overline{M}_y}{\phi_b M_{ny}\alpha_y} \leq 1.0 \qquad (Eq.\ C5.2.2\text{-}1)$$

$$P_{Ex} = \frac{\pi^2 E I_x}{(K_x L_x)^2} \qquad (Eq.\ C5.2.2\text{-}6)$$

$$P_{Ex} = \frac{\pi^2(29500)(20.6)}{\left[(1.0)(21.0)(12.0)\right]^2} = 94.4 \text{ kips}$$

$$\alpha_x = 1 - \frac{\overline{P}}{P_{Ex}} \qquad (Eq.\ C5.2.2\text{-}4)$$

$$= 1 - \frac{3.95}{94.4} = 0.958$$

$$C_{mx} = 1.0$$

$$\phi_c = 0.85$$

$$\phi_b = 0.90$$

$$\frac{3.95}{(0.85)(21.5)} + \frac{(1.0)(84.7)}{(0.90)(134)(0.958)} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}1)$$

$0.216 + 0.733 = 0.949 < 1.0$ OK

$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

from above, P_{no} = 48.6 kips

$$\frac{3.95}{(0.85)(48.6)} + \frac{84.7}{(0.90)(134)} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

$0.096 + 0.702 = 0.798 < 1.0$ OK

TABLE OF CONTENTS

PART IV
CONNECTIONS
FOR USE WITH THE
2001 EDITION OF THE
NORTH AMERICAN
SPECIFICATION FOR THE DESIGN OF
COLD-FORMED STEEL STRUCTURAL MEMBERS

SECTION 1 - WELDS

Application must comply with the requirements set forth in Section E2 of the *Specification*. The *Specification* applies to the welding of parts wherein the thinnest part is 0.18 in. or less. For welded connections in which the thickness of the thinnest connected part is greater than 0.18 in., refer to the AISC specifications. Welds shall be made in accordance with AWS D1.3, except resistance welds which shall be in accordance with AWS C1.3.

1.1 Notes On The Tables

Shown in Table IV-1 are the unit nominal shear strengths for 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, the weld design shear strength is found by multiplying the nominal weld shear strength by ϕ.

Nominal shear strengths of resistance welds, "spot welds", are provided in Table IV-2. For ASD, the weld design shear strength is determined by dividing the nominal weld shear strength by Ω. For LRFD, the weld design shear strength is found by multiplying the nominal weld shear strength by ϕ.

Table IV-3 gives the nominal shear strengths for 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 design shear strength is determined by dividing the nominal weld shear strength by Ω. For LRFD, the weld design shear strength is found by multiplying the nominal weld shear strength by ϕ. The strength of the weld metal must also be checked using *Specification* equation E2.2.1-1.

Table IV-4 gives the nominal tension strengths for concentrically loaded 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. When 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 design shear strength is determined by dividing the nominal weld shear strength by Ω. For LRFD, the weld design shear strength is found by multiplying the nominal weld shear strength by ϕ. The strength of the weld metal must also be checked using *Specification* equation E2.2.2-1.

1.2 Welded Connection Design Tables

Table IV - 1

Fillet Welds
Shear of Sheet

Ω = See Table
ϕ = See Table

Nominal Shear Strength of a 1 in. long weld, P'_n, kips/in.

Sheet Thickness in.	F_u = 45 ksi						F_u = 65 ksi					
	Weld Length/Sheet Thickness, L / t Longitudinal					Transverse	Weld Length/Sheet Thickness, L / t Longitudinal					Transverse
	5	10	15	20	≥25		5	10	15	20	≥25	
	Ω						Ω					
	2.55	2.55	2.55	2.55	3.05	2.35	2.55	2.55	2.55	2.55	3.05	2.35
	ϕ						ϕ					
	0.60	0.60	0.60	0.60	0.50	0.65	0.60	0.60	0.60	0.60	0.50	0.65
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

Notes: (1) Design Strengths are:
ASD: $P'_n \times L / \Omega$
LRFD: $\phi P'_n \times L$
where L = weld length in inches
t = thickness of thinnest welded sheet
(2) Specification equation E2.4-4 must also be checked when t > 0.10 in.

Table IV - 2

Resistance ("Spot") Welds
Shear Strength

Ω = 2.35
ϕ = 0.65

Nominal Shear Strength, P_n, kips

Sheet Thickness (in.)								
0.012	0.024	0.036	0.048	0.060	0.090	0.105	0.135	0.165
0.216	0.599	1.09	1.66	2.30	4.18	5.24	7.58	9.09

Notes: Design Strengths are:
ASD: P_n / Ω
LRFD: ϕP_n

Table IV - 3

Arc Spot Welds
Shear of Single or Multiple Sheets

Nominal Shear Strength per Weld, P_n, kips

Ω = 3.05 above double line
= 2.80 above heavy line
= 2.20 below heavy line

ϕ = 0.50 above double line
= 0.55 above heavy line
= 0.70 below heavy line

Total Sheet Thickness above Shear Plane, in.	F_u = 45 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	0.497	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)** **Design Strengths are:**
ASD: P_n / Ω
LRFD: ϕP_n

(2) **The nominal shear strength given in Eq E2.2.1-1 of the Specification is not considered in Table IV-3 and must be checked.**

Table IV - 4

Arc Spot Welds
Tension

Ω= 2.50 for panel and deck
= 3.00 for other applications

ϕ = 0.60 for panel and deck
= 0.50 for other applications

Nominal Shear Strength per Weld, P_n, kips

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) Design Strengths are:
ASD: P_n / Ω
LRFD: ϕP_n

(2) The nominal tensile strength given in Eq E2.2.2-1 of the Specification is not considered in Table IV-4 and must be checked.

(3) The limitations related to weld electrode strength, F_{xx}, have not been checked in this table.

(4) Dashed values indicate that the limit $td_aF_u \le 3$ 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. 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 equals 3/16 inch or thicker refer to the AISC specification. 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 bolts 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 AISI *Specification*.

2.1 Notes On The Tables

Shown in Tables IV-5 and IV-6 are tabulated values for the nominal tension strength and the nominal shear strength for A307, A449, A325, A354 and A490 bolts. Design strengths can be found directly from the table for ASD by dividing by Ω, and for LRFD by multiplying by ϕ.

Provided in Tables IV-7a, IV-7b and IV-7c are bearing strengths under various shear conditions for steels with tensile strengths of 45 ksi and 65 ksi. The design strength for ASD can be found by dividing by Ω, and for LRFD by multiplying by ϕ.

2.2 Bolted Connection Design Tables

Table IV - 5

Bolts
Tension

Ω= See Table
ϕ = 0.75

Nominal Tension Strength, P_n, kips

ASTM Designation	F_y ksi	F_u ksi	Diameter in.	Ω	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	2.25	40.5	1.99	3.11	4.47	6.09				
			≥ 1/2	2.25	45.0					8.84	11.2	13.8	19.9
A325	92	120	≥ 1/2	2.0	90.0					17.7	22.4	27.6	39.8
A449	92	120	< 1/2	2.0	81.0	3.98	6.21	8.95	12.2				
A354 Gr. BD	130	150	< 1/2	2.0	101.0	4.96	7.75	11.2	15.2				
A490	-	150	≥ 1/2	2.0	112.5					22.1	28.0	34.5	49.7

Note: Design Strengths are:
ASD: P_n / Ω
LRFD: ϕP_n

Table IV - 6

Bolts
Shear

Ω= 2.4
ϕ = 0.65

Nominal Shear Strength, P_n, kips

ASTM Designation	Type (2)	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	<1/2	24.0	1.18	1.84	2.65	3.61				
		≥ 1/2	27.0					5.30	6.71	8.28	11.9
A325	N	≥ 1/2	54.0					10.6	13.4	16.6	23.9
	X		72.0					14.1	17.9	22.1	31.8
A449	N	< 1/2	47.0	2.31	3.60	5.19	7.07				
	X		72.0	3.53	5.52	7.95	10.8				
A354 Gr. BD	N	<1/2	59.0	2.90	4.53	6.52	8.87				
	X		90.0	4.42	6.90	9.94	13.5				
A490	N	≥ 1/2	67.5					13.3	16.8	20.7	29.8
	X		90.0					17.7	22.4	27.6	39.8

Notes: (1) Design Strengths are:
ASD: P_n / Ω
LRFD: ϕP_n
(2) Type N has threads included in a shear plane
Type X has threads exluded from all shear planes

Table IV - 7a

Bolts
Bearing on Connected Members
Inside Sheet of Double Shear Connections
Bolt Hole Deformation Not Considered
Nominal Bearing Strength, P_n, kips

Ω = 2.50
ϕ = 0.60

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	10.10	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	10.70	13.4	16.0	18.7	21.4	24.1	26.7	32.1

Table IV - 7b

Bolts
Bearing on Connected Members
Outside Sheets of Connections with Washers on Both Sides
Bolt Hole Deformation Not Considered
Nominal Bearing Strength, P_n, kips

Ω = 2.50
ϕ = 0.60

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

Table IV - 7c

Bolts
Bearing on Connected Members
Outside Sheets of Connections without Washers on Both Sides
Bolt Hole Deformation Not Considered
Nominal Bearing Strength, P_n, kips

Ω = 2.50
ϕ = 0.60

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: **Design Strengths are:**
ASD: P_n / Ω
LRFD: 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 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.

3.1 Notes On The Tables

Provided in Table IV-8a and IV-8b are the nominal shear strengths 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 Table IV-9a and IV-9b are the nominal pullout strengths 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 Table IV-10a and IV-10b are the nominal pullover strengths 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 were used in the calculations and are listed in the tables. Larger or smaller diameters will results in different strengths. 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 can be determined by interpolating within the Tables. The design strength for ASD can be found by dividing the nominal strength by Ω. The design strength for LRFD can be found by multiplying the nominal strength by ϕ.

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 - 8a

Screws
Shear of Sheet - F_u = 45 ksi

Ω= 3.0
ϕ = 0.5

Nominal Shear Strength, P_{ns}, kips

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: **Design Strengths are:**
ASD: P_{ns} / Ω
LRFD: ϕP_{ns}

Table IV - 8b

Screws
Shear of Sheet - F_u = 65 ksi

Ω = 3.0
ϕ = 0.5

Nominal Shear Strength, P_{ns}, kips

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: **Design Strengths are:**
ASD: P_{ns} / Ω
LRFD: ϕP_{ns}

Table IV - 9a

Screws
Pull-Out - F_u = 45 ksi

Ω= 3.0
ϕ = 0.5

Nominal Pullout Strength, P_{not}, kips

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 - 9b

Ω= 3.0
ϕ = 0.5

Screws
Pull-Out - F_u = 65 ksi

Nominal Pullout Strength, P_{not}, kips

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: Design Strengths are:
ASD: P_{not} / Ω
LRFD: ϕP_{not}

Table IV - 10a

Hex Head Screws
Pull-Over - F_u = 45 ksi

Ω= 3.0
ϕ = 0.5

Nominal Pullout Strength, P_{nov}, kips

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 - 10b

Hex Head Screws
Pull-Over - F_u = 65 ksi

Ω= 3.0
ϕ = 0.5

Nominal Pullout Strength, P_{nov}, kips

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

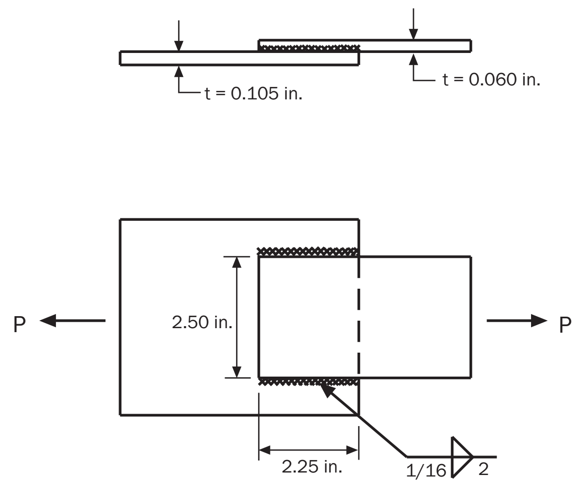
Screw Designation	Washer Head Diameter in.	Thickness of member in contact with the washer, in.						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.315	1.11	1.47	1.84	2.30	2.76	3.22	4.15
#8	0.335	1.18	1.57	1.96	2.45	2.94	3.43	4.41
#10	0.399	1.40	1.87	2.33	2.92	3.50	4.08	5.25
#12	0.415	1.46	1.94	2.43	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

Notes: Design Strengths are:
ASD: P_{nov} / Ω
LRFD: 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 kips, P_{live} = 3.0 kips
3. Detail of connection shown in sketch

Required:

Determine if longitudinal fillet welded connection is adequate to transmit the required strength P (ASD), and P_u (LRFD) using ASCE7-98 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. Strength at Weld (Section E2.4)

$L/t = 2.25/0.060 = 37.5 > 25$
For $L/t \geq 25$,

$P_n = 0.75tLF_u$ (*Eq*. E2.4-2)
$P_n = (0.75)(0.060)(2.25)(65) = 6.58$ kips/weld

Note: t = 0.060 in. < 0.10 in. Therefore, (*Eq*. E2.4-4) does not apply.

ASD

$\Omega = 3.05$
$P_n/\Omega = 6.58/3.05 = 2.16$ kips/weld
(2.16 kips/weld)(2 welds) = 4.32 kips > 4.0 kips OK

LRFD

ϕ = 0.50
ϕP_n = (0.50)(6.58) = 3.29 kips/weld
(3.29 kips/weld)(2 welds) = 6.58 kips > 6.0 kips OK

2. Tensile strength of the plate (Section C2 of Appendix A)

Yielding

$T_n = A_g F_y$ (*Eq.* C2-1)
= (2.50)(0.060)(50 ksi) = 7.50 kips

ASD

Ω_t = 1.67
T_n/Ω_t = 7.50/1.67 = 4.49 kips > 4.0 kips OK

LRFD

ϕ_t = 0.90
$\phi_t T_n$ =(0.90)(7.50) = 6.75 kips > 6.0 kips OK

Fracture away from connection

$T_n = A_n F_u$ (*Eq.* C2-2)
= (2.50)(0.060)(65) = 9.75 kips

ASD

Ω_t = 2.00
T_n/Ω_t = 9.75/2.00 = 4.88 kips > 4.0 kips OK

LRFD

ϕ_t = 0.75
$\phi_t P_n$ = (0.75)(9.75) = 7.31 kips > 6.0 kips OK

Using Connection Tables

Using Table IV-1, the design 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 ≥ 25
P'_n = 2.93 kips/inch (from Table IV-1)
P_n = (2)(2.93)(2.25) = 13.19 kips

ASD

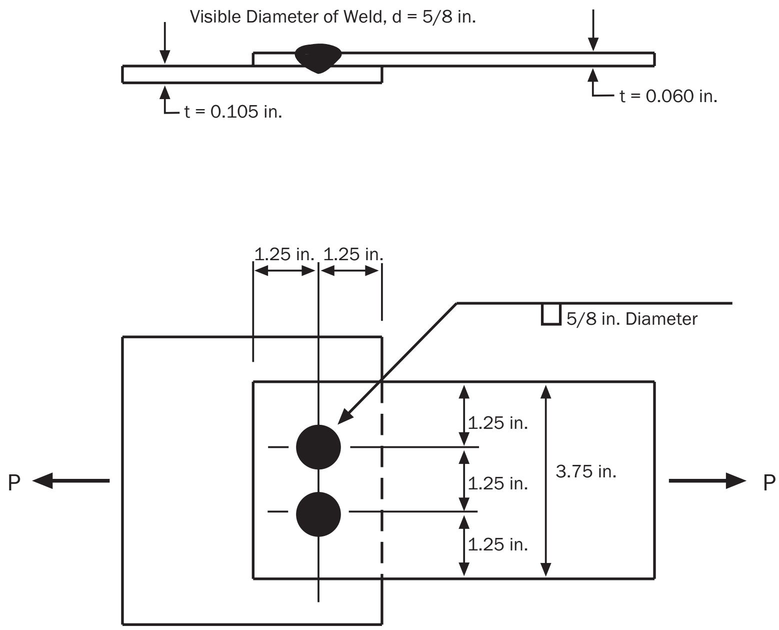
Ω = 3.05
P_n/Ω = 13.19/3.05 = 4.33 kips > 4.0 kips OK

LRFD

ϕ = 0.50
ϕP_n = (0.50)(13.19) = 6.60 > 6.0 kips OK

2. Check plate as above

Example IV-2: Flat Section With Arc Spot Welded Connection



Given:

1. Steel: $F_{sy} = F_y = 50$ ksi, $F_u = 65$ ksi
2. E60 Weld Electrode, $F_{xx} = 60$ksi
3. Detail of connection shown in sketch

Required:

Determine the allowable design strength, using ASD and LRFD.

Solution:

1. Weld Dimensions

$d = 0.625$ in.
$d_a = d - t = 0.625 - 0.060 = 0.565$ in.
$d_e = 0.7d - 1.5t \leq 0.55d$ (*Eq*. E2.2.1-5)
$= (0.70)(0.625) - (1.5)(0.060) = 0.348$ in. $< 3/8$ in. N.G.
$0.55d = 0.55(0.625) = 0.344$ in. $< 3/8$ in. N.G.

Per Section E2.2, minimum allowable effective diameter, d_e, is 3/8 in. Weld procedures must be established and welds measured to assure that a 3/8 inch effective diameter can be consistently achieved.

2. Design strength based on weld strength (Section E2.2.1(a))

$$P_n = \frac{\pi d_e^2}{4} 0.75 F_{xx} \quad (Eq.\ E2.2.1\text{-}1)$$

Using E60 electrode, $F_{xx} = 60$ ksi

$$P_n = \frac{\pi(0.375)^2}{4}(0.75)(60)(2) = 9.94 \text{ kips}$$

ASD

$\Omega = 2.55$
$P_n/\Omega = 9.94/2.55 = 3.90$ kips ← CONTROLS

LRFD

$\phi = 0.60$

$\phi P_n = (0.60)(9.94) = 5.96$ kips ← CONTROLS

3. Design strength based on sheet strength (Section E2.2.1(b))

$d_a/t = 0.565/0.060 = 9.42$

$0.815\sqrt{E/F_u} = 0.815\sqrt{29500/65} = 17.4$

Since $d_a/t < 0.815\sqrt{E/F_u}$

$P_n = 2.20 t d_a F_u$ (*Eq.* E2.2.1-2)

$= (2.20)(0.060)(0.565)(65)(2) = 9.70$ kips

ASD

$\Omega = 2.20$

$P_n/\Omega = 9.70/2.20 = 4.41$ kips

LRFD

$\phi = 0.70$

$\phi P_n = (0.70)(9.70) = 6.79$ kips

4. Design strength based on edge distance and spacing requirements (Section E2.2.1)

$F_u/F_{sy} = 65/50 = 1.3 > 1.08$

For $F_u/F_{sy} \geq 1.08$

ASD

$\Omega = 2.20$

$e_{min} = \dfrac{P\Omega}{F_u t}$ (*Eq.* E2.2.1-6a)

$\therefore P = \dfrac{e_{min} F_u t}{\Omega} = \dfrac{(1.25)(65)(0.060)}{2.20} = 2.22$ kips/weld

For 2 welds; P = 4.44 kips

LRFD

$\phi = 0.70$

$e_{min} = \dfrac{P_u}{\phi F_u t}$ (*Eq.* E2.2.1-6b)

$\therefore P_u = e_{min}\,\phi\, F_u t = (1.25)(.70)(65)(0.060) = 3.41$ kips/weld

For 2 welds, $P_u = 6.82$ kips

Edge distance shall not be less than 1.5d.

1.5d = (1.5)(0.625) = 0.94 in. < 1.25 in. OK

Clear distance between welds shall not be less than 1.0d.

1.0d = (1.0)(0.625) = 0.625 in.

Clear distance = 1.250 - (2)(0.625/2) = 0.625 in. = 0.625 in. OK

Clear distance between welds and end of member shall not be less than 1.0d.

1.0d = (1.0)(0.625) = 0.625 in.

Clear distance = 1.250 - 0.625/2 = 0.938 in. > 0.625 in. OK

Thinnest connected part, t = 0.060 in. < 0.15 in. OK

5. No weld washers required because t = 0.060 in. > 0.028 in.

6. Tensile strength of the plate (Section C2 of Appendix A)

Yielding

$T_n = A_g F_y$ (*Eq*. C2-1)

$= (3.75)(0.060)(50\text{ ksi}) = 11.3\text{ kips}$

ASD

$\Omega_t = 1.67$

$T_n/\Omega_t = 11.3/1.67 = 6.77\text{ kips}$

LRFD

$\phi_t = 0.90$

$\phi_t T_n = (0.90)(11.3) = 10.2\text{ kips}$

Fracture away from connection

$T_n = A_n F_u$ (*Eq*. C2-2)

$= (3.75)(0.060)(65) = 14.6\text{ kips}$

ASD

$\Omega_t = 2.00$

$T_n/\Omega_t = 14.6/2.00 = 7.30\text{ kips}$

LRFD

$\phi_t = 0.75$

$\phi_t P_n = (0.75)(14.6) = 11.0\text{ kips}$

Using Connection Tables

Using Table IV-3 the design strength based on weld shear and sheet shear could have been determined as follows:

1. Check weld shear as above

$P_n = 9.94\text{ kips}$

$P_n/\Omega = 9.94/2.55 = 3.90\text{ kips (ASD)}$ ← CONTROLS

$\phi P_n = (0.60)(9.94) = 5.96\text{ kips (LRFD)}$ ← CONTROLS

2. Shear of sheet, Table IV-3

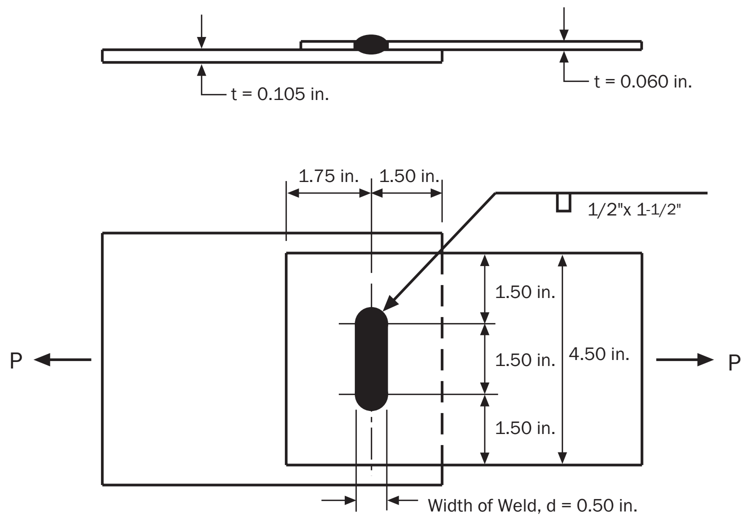
$P_n = (4.85)(2) = 9.70\text{ kips}$

$P_n/\Omega = 9.70/2.20 = 4.41\text{ kips (ASD)}$

$\phi P_n = (0.70)(9.70) = 6.79\text{ kips (LRFD)}$

3. Check edge distance and plate as above

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 = 2.9 kips (ASD), P_u = 4.1 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. Design strength based on weld strength (Section E2.3)

Weld Metal Strength

$$P_n = \left[\frac{\pi d_e^2}{4} + Ld_e\right]0.75F_{xx} \qquad (Eq.\ E2.3\text{-}1)$$

L = 1.5 in., or maximum 3d, 3(0.5) = 1.5 in. OK

$$d_a = d - t \qquad (Eq.\ E2.3\text{-}3)$$

$= 0.50 - 0.060 = 0.44$ in.

$$d_e = 0.7d - 1.5t \qquad (Eq.\ E2.3\text{-}4)$$

$= (0.7)(0.50) - (1.5)(0.060) = 0.260$ in.

$$P_n = \left[\frac{\pi(0.26)^2}{4} + (1.50)(0.26)\right](0.75)(60)$$

= 19.9 kips

ASD

Ω = 2.55

P_n/Ω = 19.9/2.55 = 7.80 kips

LRFD

ϕ = 0.60

ϕP_n = (0.60)(19.9) = 11.9 kips

Base Metal Strength

$P_n = 2.5tF_u(0.25L + 0.96d_a)$ (*Eq*. E2.3-2)

$= (2.5)(0.060)(65)\left[(0.25)(1.5) + (0.96)(0.44)\right] = 7.77$ kips

ASD

$\Omega = 2.55$

$P_n/\Omega = 7.77/2.55 = 3.05$kips > P = 2.9 kips OK ← CONTROLS

LRFD

$\phi = 0.60$

$\phi P_n = (0.60)(7.77) = 4.66$ kips > $P_u = 4.10$ kips OK ← CONTROLS

2. Determine minimum edge distance in line of force (Section E2.2.1)

$F_u/F_y = 65/50 = 1.3 > 1.08$

For $F_u/F_y > 1.08$;

$\Omega = 2.20$ (ASD)

$\phi = 0.70$ (LRFD)

ASD

$$e_{min} = \frac{P\Omega}{F_u t}$$ (*Eq*. E2.2.1-6a)

for t = 0.060

$$e_{min} = \frac{(2.9)(2.20)}{(65)(0.060)} = 1.64 \text{ in.} < 1.75 \text{ in. OK}$$

for t = 0.105

$$e_{min} = \frac{(2.9)(2.20)}{(65)(0.105)} = 0.935 \text{ in.} < 1.5 \text{ in. OK}$$

LRFD

$$e_{min} = \frac{P_u}{\phi F_u t}$$ (*Eq*. E2.2.1-6b)

for t = 0.060

$$e_{min} = \frac{4.1}{(0.70)(65)(0.060)} = 1.50 \text{ in.} < 1.75 \text{ in. OK}$$

for t = 0.105

$$e_{min} = \frac{4.1}{(0.70)(65)(0.105)} = 0.858 \text{ in.} < 1.50 \text{ in. OK}$$

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

3. Tensile Strength of the plate (Section C2 of Appendix A)

Yielding

$T_n = A_g F_y$ (*Eq*. C2-1)

$= (4.5)(0.060)(50) = 13.5$ kips

ASD

$\Omega_t = 1.67$

$T_n/\Omega_t = 13.5/1.67 = 8.08$ kips > 2.9 kips OK

LRFD

ϕ_t = 0.90

$\phi_t P_n$ = (0.90)(13.5) = 12.2 kips > 4.1 kips OK

Fracture away from connection

$T_n = A_n F_u$ (*Eq*. C2-2)

= (4.5)(0.060)(65) = 17.6 kips

ASD

Ω_t =2.00

T_n/Ω_t=17.6/2.00 = 8.80 kips > 2.9 kips OK

LRFD

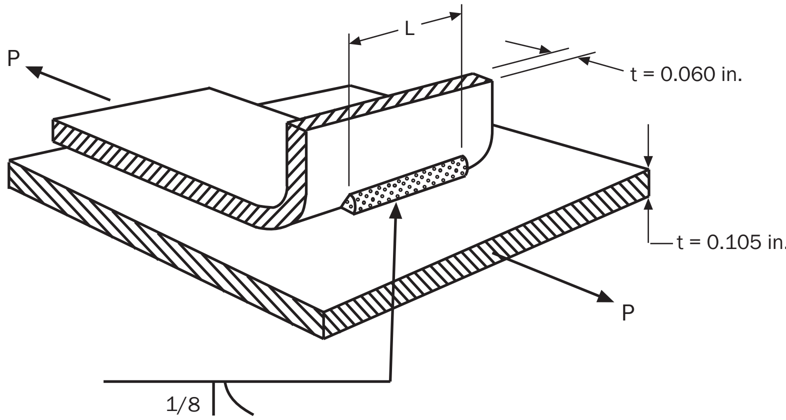
ϕ_t = 0.75

$\phi_t P_n$ = (0.75)(17.6) = 13.2 kips > 4.1 kips OK

4. 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} = 1.0 kips P_{live} = 3.0 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

$$P = P_{dead} + P_{live}$$
$$= 1.0 + 3.0 = 4.0 \text{ kips}$$

LRFD

$$P_u = 1.2\, P_{dead} + 1.6\, P_{live}$$
$$= (1.2)(1.0) + (1.6)(3.0) = 6.0 \text{ kips}$$

2. Design strength of flare-bevel groove welds, transverse loading

$$P_n = 0.833tLF_u \qquad (Eq.\ E2.5\text{-}1)$$

3. Solve for L

ASD

$$\Omega = 2.55$$
$$P \le P_n/\Omega \qquad (Eq.\ A4.1.1\text{-}1)$$
$$P \le 0.833tLF_u/\Omega$$
$$\therefore L \ge \frac{\Omega P}{0.833tF_u}$$
$$L \ge \frac{(2.55)(4.0)}{(0.833)(0.060)(65)} = 3.14 \text{ in.}$$

LRFD

$$\phi = 0.60$$
$$P_u \le \phi P_n \qquad (Eq.\ A5.1.1\text{-}1)$$
$$\le \phi 0.833tLF_u$$
$$\therefore L \ge \frac{P_u}{\phi 0.833tF_u}$$

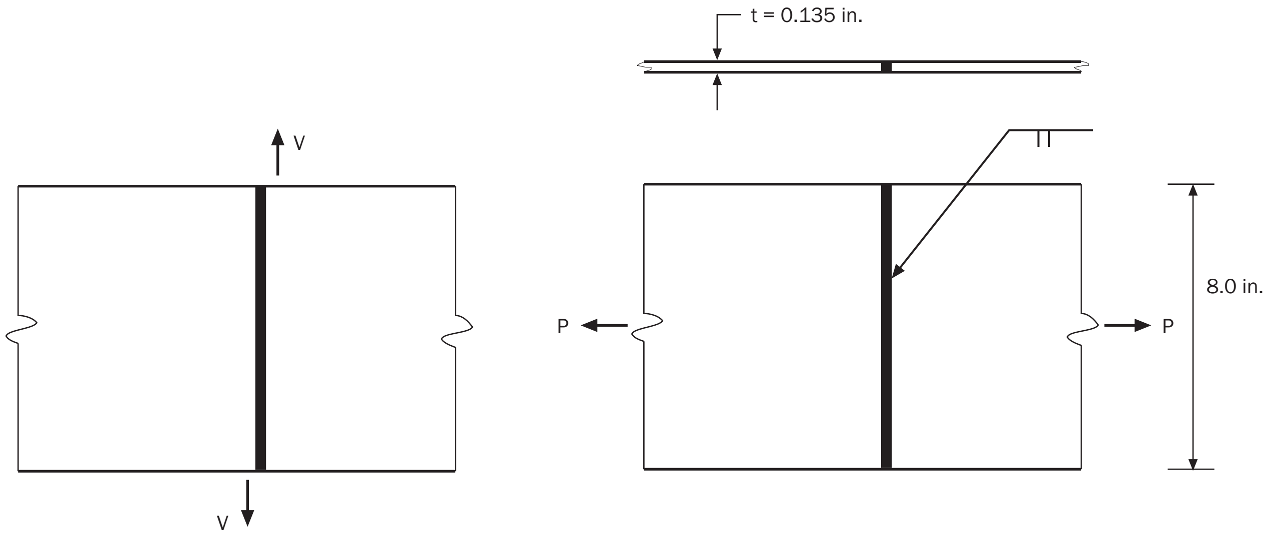
$$L \geq \frac{6.0}{(0.60)(0.833)(0.060)(65)} = 3.08 \text{ in.}$$

Equation E2.5-4 need not be checked since $t < 0.10$ in.

4. Final Design

Use 1/8 inch bevel groove weld 3 1/4 inch long.

Example IV-5: 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 design tensile strength normal to the effective area using ASD and LRFD.
2. Determine the design shear strength on the effective area using ASD and LRFD.

Solution:

1. Design tensile strength normal to the effective area (Section E2.1(a))

$P_n = Lt_eF_y$ (*Eq.* E2.1-1)

$= (8.0)(0.135)(50) = 54.0$ kips

ASD

$\Omega = 1.70$

$P_n/\Omega = 54.0/1.70 = 31.8$ kips

LRFD

$\phi = 0.90$

$\phi P_n = (0.90)(54.0) = 48.6$ kips

2. Design shear strength on the effective area (Section E2.1(b))

Weld Strength

$P_n = Lt_e 0.6F_{xx}$ (*Eq.* E2.1-2)

$= (8.0)(0.135)(0.6)(60) = 38.9$ kips

ASD

$\Omega = 1.90$

$P_n/\Omega = 38.9/1.90 = 20.5$ kips

LRFD

$\phi = 0.80$

$\phi P_n = (0.80)(38.9) = 31.1$ kips

Base Metal Strength

$P_n = Lt_eF_y/\sqrt{3}$ (*Eq.* E2.1-3)

$= (8.0)(0.135)(50)/\sqrt{3} = 31.2$ kips

ASD

$\Omega = 1.70$

$P_n/\Omega = 31.2/1.70 = 18.4$ kips

Since base metal strength governs,

$P_n/\Omega = 18.4$ kips

LRFD

$\phi = 0.90$

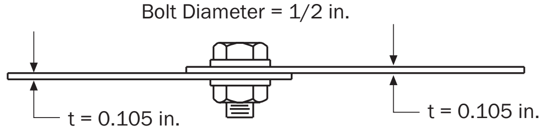
$\phi P_n = (0.90)(31.2) = 28.1$ kips

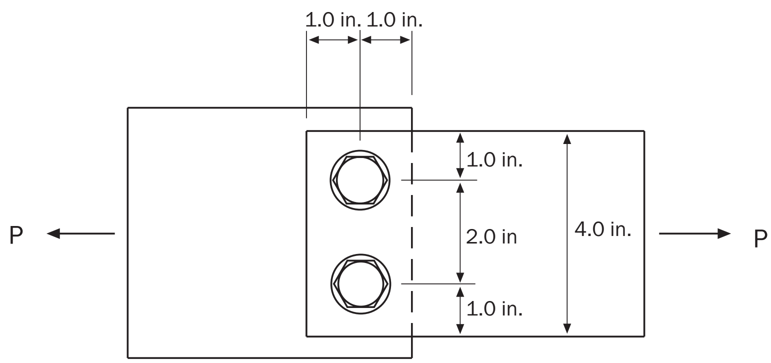
Since base metal strength governs,

$\phi P_n = 28.1$ kips

4.2 Bolt Examples

Example IV-6: Flat Section With Bolted Connection





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. Evaluate bearing without consideration of bolt hole deformation.

Required:

Determine the ASD design strength, P_n/Ω, and the LRFD design strength, ϕP_n

Solution:

Thickness of thinnest part connected, t

$t = 0.105$ in. < 3/16 in.

Therefore, Section E3 applies.

1. Design strength based on spacing and edge distance (Section E3.1)

$P_n = teF_u$ (*Eq*. E3.1-1)

$= (0.105)(1.0)(45) = 4.73$ kips/bolt

$F_u/F_y = 45/33 = 1.36 > 1.08$

For $F_u/F_y > 1.08$

$\Omega = 2.00$ (ASD)

$\phi = 0.70$ (LRFD)

ASD

$P_n/\Omega = (2)(4.73)/2.00 = 4.73$ kips

LRFD

$\phi P_n = (0.70)(2)(4.73) = 6.62$ kips

Distance between bolt hole centers must be greater than 3d.

$3d = (3)(0.50) = 1.5$ in. < 2.0 in. OK

Distance between bolt hole center and edge of connecting member must be greater than 1.5d.

$1.5d = (1.5)(0.50) = 0.75$ in. < 1.0 in. OK

2. Design strength based on tension on net section (Section C2)

Nominal tension strength shall not exceed the smallest value of T_n from Section C2:

Yielding - Section C2(a)

$A_g = (0.105)(4.0) = 0.420 \text{ in.}^2$

$T_n = A_gF_y = (0.420)(33) = 13.86$ kips (*Eq.* C2-1)

ASD

$\Omega_t = 1.67$

$T_n/\Omega_t = 13.86/1.67 = 8.30$ kips

LRFD

$\phi_t = 0.90$

$\phi_tT_n = (0.90)(13.86) = 12.5$ kips

Fracture away from connection - Section C2(b)

Since there are no holes or other reductions in area away from the connection:

$A_n = A_g = 0.420 \text{ in.}^2$

$T_n = A_nF_u = (0.420)(45) = 18.9$ kips (*Eq.* C2-2)

ASD

$\Omega_t = 2.00$

$T_n/\Omega_t = 18.9/2.00 = 9.45$ kips

LRFD

$\phi_t = 0.75$

$\phi_tT_n = (0.75)(18.9) = 14.2$ kips

Fracture at connection - Section C2(c) refers to Section E3.2

A_n - based on Table E3a in Appendix A

$A_n = 0.105[4.0 - 2(1/2 + 1/16)] = 0.302 \text{ in.}^2$

Since washers are provided under both bolt head and nut

$F_t = (0.1 + 3d/s)F_u \leq F_u$ (*Eq.* E3.2-2)

where:

$d = 0.50$ in.

$s = 4.0/2 = 2.0$ in.

$F_t = [0.1 + (3)(0.50)/2.0](45)$

$= 38.25$ ksi < 45 ksi OK

$P_n = A_nF_t$ (*Eq.* E3.2-1)

$= (0.302)(38.25) = 11.55$ kips

ASD

$\Omega = 2.22$ for single shear connection

$P_n/\Omega = 11.55/2.22 = 5.20$ kips

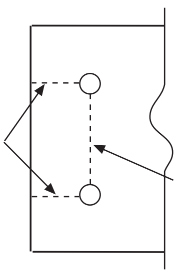
LRFD

ϕ = 0.55 for single shear connection

ϕP_n = (0.55)(11.55) = 6.35 kips

Block Shear Rupture - Section E5.3

The block shear path shown controls.



Gross area subject to shear

A_{gv} = (2)(1.0)(0.105) = 0.210 in.^{2}

Gross area subject to tension

A_{gt} = (2.0)(0.105) = 0.210 in.^{2}

Net area subject to shear

A_{nv} = 0.210 - (0.105)(2)(0.5)(0.50 + 1/16) = 0.151 in.^{2}

Net area subject to tension

A_{nt} = 0.210 - (0.105)(2)(0.5)(0.50 + 1/16) = 0.151 in.^{2}

F_uA_{nt} = (45.0)(0.151) = 6.80 kips

$0.6F_uA_{nv}$ = (0.6)(45.0)(0.151) = 4.08 kips

Since $F_uA_{nt} > 0.6F_uA_{nv}$

$R_n = 0.6F_yA_{gv} + F_uA_{nt}$ (*Eq.* E5.3-1)

= (0.6)(33.0)(0.210) + (45.0)(0.151) = 10.95 kips

ASD

Ω = 2.22 for bolted connections

R_n/Ω = 10.95/2.22 = 4.93 kips

LRFD

ϕ = 0.65

ϕP_n = (0.65)(10.95) = 7.12 kips

3. Design 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 (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)

$P_n = m_f\,CdtF_u$ (*Eq.* E3.3.1-1)

= (1.0)(3.00)(0.50)(0.105)(45) = 7.09 kips/bolt

ASD

Ω = 2.50

P_n/Ω = (2)(7.09)/2.50 = 5.67 kips

LRFD

ϕ = 0.60

ϕP_n = (0.60)(2)(7.09) = 8.51 kips

4. Design strength based on bolt shear (Section E3.4)

$P_n = A_bF_n$ (*Eq.* E3.4-1)

$A_b = (\pi/4)(0.50)^2$ = 0.196 in.^{2}

F_n = F_{nv} = 27.0 ksi (from Table E3.4-1, d ≥ 1/2 in.)
P_n = (27)(0.196) = 5.29 kips/bolt

ASD

Ω = 2.4
P_n/Ω = (2)(5.29)/2.4 = 4.41 kips

LRFD

ϕ = 0.65
ϕP_n = (0.65)(2)(5.29) = 6.88 kips

5. Determine governing limit state

Comparing the values from 1, 2, 3, and 4 above for ASD, the allowable design strength of the bolt shear controls:

P_n/Ω = 4.41 kips

Comparing the values from 1, 2, 3, and 4 above for LRFD, the design tensile strength on the net section of the connected part controls and

ϕP_n = 6.35 kips

Using Connection Tables

1. Using Table IV-7b the design strength based on bearing could have been determined as follows

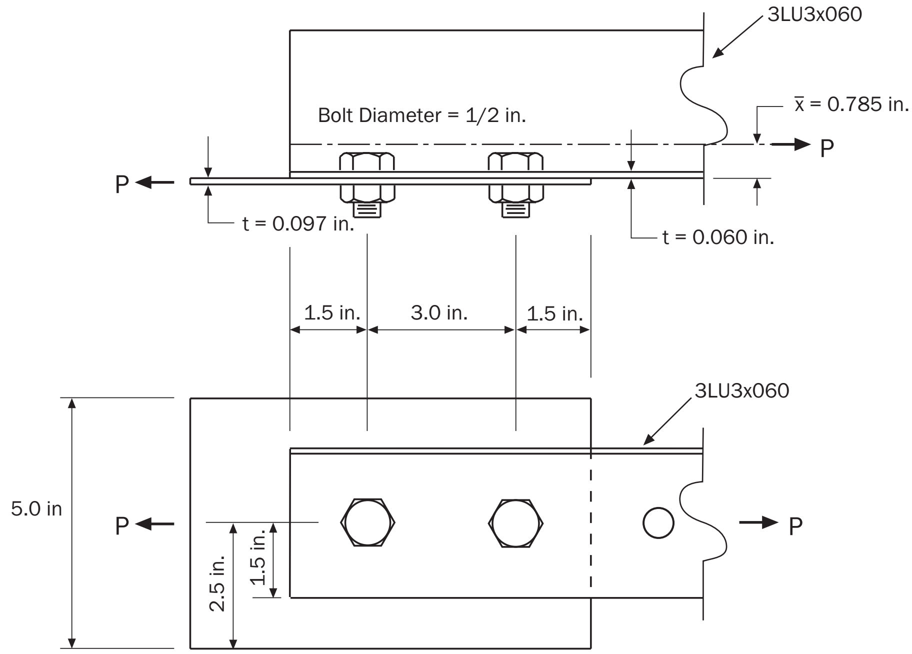
P_n = (7.09)(2) = 14.18 kips
P_a = P_n/Ω = 14.18/2.50 = 5.67 kips (ASD)
ϕP_n = (0.6)(14.18) = 8.51 kips (LRFD)

2. Using Table IV-6 the design strength based on bolt shear could have been determined as follows

P_n = (5.30)(2 bolts) = 10.6 kips
P_n/Ω = 10.6/2.4 = 4.42 kips (ASD)
ϕP_n = (0.65)(10.6) = 6.89 kips (LRFD)

3. Check net section and edge distance as above. Bolt shear controls for ASD. Tension on net section controls for LRFD.

Example IV-7: Bolted Connection with Consideration of Shear Lag



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 design strength.

Solution:

Calculate strength considering:

1. Shear, spacing and edge distance in the direction of applied force (Section E3.1)
2. Tensile strength of connected parts (Section C2)
3. Bearing strength of connected parts (Section E3.3)
4. Shear strength of bolts (Section E3.4)

1. Shear, spacing and edge distance (Section E3.1)

The thinner angle material controls by inspection.

P_n = teF_u (*Eq.* E3.1-1)

= (0.060)(1.50)(45) = 4.05 kips/bolt

F_u/F_y = 45/33 = 1.36

Since F_u/F_y > 1.08

Ω = 2.00 (ASD)

P_n/Ω= (2)(4.05)/2.00 = 4.05 kips

Distance between bolt hole centers must be greater than 3d.

3d = (3)(0.50) = 1.5 in. < 3.0 in. OK

Distance between bolt hole center and edge of connecting member must be greater than than 1.5d.

1.5d = (1.5)(0.50) = 0.75 in. < 1.5 in. OK

2. Tensile strength of connected parts (Section C2)

Angle Section

Nominal tension strength shall not exceed the smallest value of T_n from Section C2:

Yielding - Section C2(a)

$T_n = A_g F_y = (0.351)(33.0) = 11.58$ kips (*Eq*. C2-1)

$\Omega_t = 1.67$

$T_n/\Omega_t = 11.58/1.67 = 6.93$ kips

Fracture away from connection - Section C2(b)

Location of unfilled bolt hole will control.

A_n - based on Table E3a in Appendix A using 1/2 inch diameter standard holes

$A_n = 0.351 - (0.060)(0.500 + 1/16) = 0.317$ in.^{2}

$T_n = A_n F_u = (0.317)(45.0) = 14.27$ kips (*Eq*. C2-2)

$\Omega_t = 2.00$

$T_n/\Omega_t = 14.27/2.00 = 7.14$ kips

Fracture at connection

Fracture across the section - Section E3.2(3)

$P_n = A_e F_u$ (*Eq*. E3.2-8)

For an angle with two or more bolts in the line of force:

$U = 1.0 - 1.20\,\bar{x}/L < 0.9$ (*Eq*. E3.2-9)

$= 1.0 - (1.20)(0.785/3.0) = 0.686 < 0.9$ OK

$A_e = UA_n$

$= (0.686)(0.317) = 0.217$ in.^{2}

$P_n = A_e F_u$ (*Eq*. E3.2-1)

$= (0.217)(45.0) = 9.77$ kips

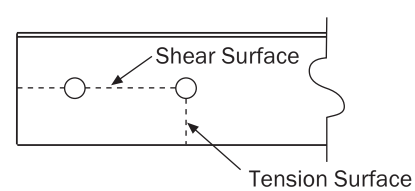
$\Omega = 2.22$ for other than flat sheet connection

$P_n/\Omega = 9.77/2.22 = 4.40$ kips

Block Shear Rupture - Section E5.3

Gross area subject to shear

$A_{gv} = (1.5 + 3.0)(0.060) = 0.270$ in.^{2}



Gross area subject to tension

$A_{gt} = (1.5)(0.060) = 0.090$ in.^{2}

Net area subject to shear

$A_{nv} = 0.270 - (0.060)(1.5)(0.50 + 1/16) = 0.219$ in.^{2}

Net area subject to tension

$A_{nt} = 0.090 - (0.060)(0.5)(0.50 + 1/16) = 0.0731$ in.^{2}

$F_u A_{nt} = (45.0)(0.0731) = 3.29$ kips

$0.6F_uA_{nv} = (0.6)(45.0)(0.219) = 5.91$ kips

Since $F_uA_{nt} < 0.6F_uA_{nv}$

$R_n = 0.6F_uA_{nv} + F_yA_{gt}$ (*Eq*. E5.3-2)

$= (0.6)(45.0)(0.219) + (33.0)(0.090) = 8.88$ kips

$\Omega = 2.22$ for bolted connections

$R_n/\Omega = 8.88/2.22 = 4.00$ kips

Block shear controls angle tensile strength

$P_n/\Omega = 4.00$ kips

Flat Plate

Nominal tension strength shall not exceed the smallest value of T_n from Section C2:

Yielding - Section C2(a)

$T_n = A_gF_y = (5.0)(0.097)(33) = 16.01$ kips (*Eq*. C2-1)

$\Omega_t = 1.67$

$T_n/\Omega_t = 16.01/1.67 = 9.59$ kips

Fracture away from connection - Section C2(b)

Since there are no holes or other reductions in the plate away from the connection:

$A_n = A_g = (5.0)(0.097) = 0.485$ in.^{2}

$T_n = A_nF_u = (0.485)(45.0) = 21.8$ kips (*Eq*. C2-2)

$\Omega_t = 2.00$

$T_n/\Omega_t = 21.8/2.00 = 10.9$ kips

Fracture at connection

Fracture across the section - Section E3.2(1-b)

A_n - based on Table E3a in Appendix A using 1/2 inch diameter standard holes

$A_n = 0.097[5.0 - (0.500 + 1/16)] = 0.430$ in.^{2}

Since washers are not provided and there are multiple bolts in the line parallel to the force:

$F_t = F_u$ (*Eq*. E3.2-5)

$P_n = A_nF_t$ (*Eq*. E3.2-1)

$= (0.430)(45.0) = 19.35$ kips

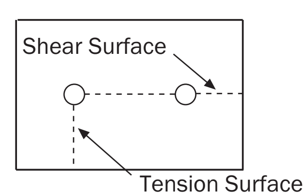
$\Omega = 2.22$ for single shear connection

$P_n/\Omega = 19.35/2.22 = 8.72$ kips

Block Shear Rupture - Section E5.3

Gross area subject to shear

$A_{gv} = (1.5 + 3.0)(0.097) = 0.437$ in.^{2}



Gross area subject to tension

$A_{gt} = (2.5)(0.097) = 0.243$ in.^{2}

Net area subject to shear

$A_{nv} = 0.437 - (0.097)(1.5)(0.50 + 1/16) = 0.355$ in.^{2}

Net area subject to tension

$A_{nt} = 0.243 - (0.097)(0.5)(0.50 + 1/16) = 0.216$ in.^{2}

$F_uA_{nt} = (45.0)(0.216) = 9.72$ kips

$0.6F_uA_{nv}$ = (0.6)(45.0)(0.355) = 9.59 kips

Since $F_uA_{nt} > 0.6F_uA_{nv}$

$R_n = 0.6F_yA_{gv} + F_uA_{nt}$ (*Eq*. E5.3-1)

= (0.6)(33.0)(0.437) + (45.0)(0.216) = 18.4 kips

Ω = 2.22 for bolted connections

R_n/Ω = 18.4/2.22 = 8.29 kips

Block shear controls plate tensile strength

P_n/Ω = 8.29 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)

$P_n = m_f CdtF_u$ (*Eq*. E3.3.1-1)

= (0.75)(3.00)(0.50)(0.060)(45.0) = 3.04 kips/bolt

Ω = 2.50

P_n/Ω = (2)(3.04)/2.50 = 2.43 kips

4. Bolt shear (Section E3.4)

$P_n = A_bF_n$ (*Eq*. E3.4-1)

$A_b = (\pi/4)(0.50)^2 = 0.196$ in.^{2}

$F_n = F_{nv}$ = 27.0 ksi (Table E3.4-1, d ≥ 1/2 in.)

P_n = (27)(0.196) = 5.29 kips/bolt (*Eq*. E3.4-1)

Ω = 2.4

P_n/Ω = (2)(5.29)/2.4 = 4.41 kips

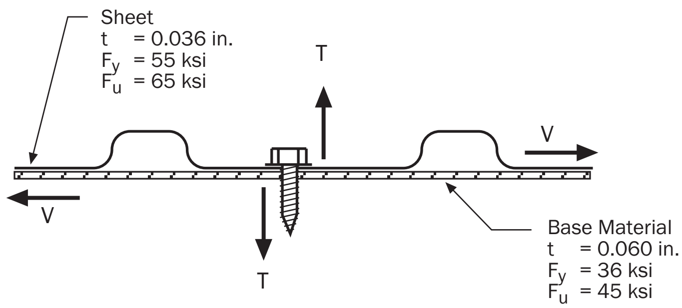
5. Determine allowable design strength

Comparing the values from 1, 2, 3, and 4 above , the allowable design strength of the bolt bearing on the angle controls:

P_n/Ω = 2.43 kips

4.3 Screw Example

Example IV-8: Screwed Connection



Given:

1. Screws: #10 Self-drilling
   d = 0.190 in.
   d_w = 0.3175 in.

   Per manufacturer's test report
   P_{ts} = 2.42 kips (screw tension strength based on tests)
   P_{ss} = 1.60 kips (screw shear strength based on tests)
2. Detail and materials as shown above
3. Minimum edge distance of top sheet is 0.75 inches

Required:

Determine the shear design strength and the tensile design strength using ASD and LRFD assuming the shear and tensile forces do not act simultaneously.

Solution:

1. Shear Design Strength (Section E4.3)

Connection shear limited by tilting and bearing (Section E4.3.1)

t_1 = 0.036 in.
t_2 = 0.060 in.
t_2/t_1 = 0.060/0.036 = 1.67, therefore interpolate between the provisions for $t_2/t_1 \le 1.0$ and $t_2/t_1 \ge 2.5$

For $t_2/t_1 \le 1.0$, P_{ns} is the smaller of

$P_{ns} = 4.2(t_2{}^3 d)^{1/2}F_{u2}$ (*Eq.* E4.3.1-1)
$= 4.2\left[(0.060)^3(0.190)\right]^{1/2}(45) = 1.21$ kips

$P_{ns} = 2.7t_1dF_{u1}$ (*Eq.* E4.3.1-2)
$= (2.7)(0.036)(0.190)(65) = 1.20$ kips

$P_{ns} = 2.7t_2dF_{u2}$ (*Eq.* E4.3.1-3)
$= (2.7)(0.060)(0.190)(45) = 1.39$ kips

$P_{ns} = \min(1.21, 1.20, 1.39) = 1.20$ kips

For $t_2/t_1 \ge 2.5$, P_{ns} is the smaller of

$P_{ns} = 2.7t_1dF_{u1}$ (*Eq.* E4.3.1-4)
$= (2.7)(0.036)(0.190)(65) = 1.20$ kips

$P_{ns} = 2.7t_2dF_{u2}$ (*Eq.* E4.3.1-5)
$= (2.7)(0.060)(0.190)(45) = 1.39$ kips

$P_{ns} = \min(1.20, 1.39) = 1.20$ kips

Since P_{ns} is equal for both cases, interpolation is unnecessary: P_{ns} = 1.20 kips.

ASD

$\Omega = 3.00$

$\frac{P_{ns}}{\Omega} = \frac{1.20}{3.0} = 0.400$ kips

LRFD

$\phi = 0.50$

$\phi P_{ns} = (0.50)(1.20) = 0.600$ kips

The connection tables IV-8a and IV-8b cannot be used because they assume that steels with identical F_u are used for both sheets, which is not the case in this example.

Connection strength limited by end distance (Appendix E4.3.2)

$P_{ns} = teF_u$ (*Eq.* E4.3.2-1)

$= (0.036)(0.75)(65) = 1.76$ kips

ASD

$\Omega = 3.00$

$\frac{P_{ns}}{\Omega} = \frac{1.76}{3.0} = 0.587$ kips

LRFD

$\phi = 0.50$

$\phi P_{ns} = (0.50)(1.76) = 0.880$ kips

Shear in screw (Section E4.3.3)

$P_{ns} = 0.8P_{ss}$ (*Eq.* E4.3.3-1)

$= (0.8)(1.60) = 1.28$ kips

ASD

$\Omega = 3.00$

$\frac{P_{ns}}{\Omega} = \frac{1.28}{3.0} = 0.427$ kips

LRFD

$\phi = 0.50$

$\phi P_{ns} = (0.50)(1.28) = 0.640$ kips

For both ASD and LRFD, tilting and bearing govern.

2. Tension Design Strength (Section E4.4)

Pull-out (Section E4.4.1)

$P_{not} = 0.85t_c dF_{u2}$ (*Eq.* E4.4.1-1)

$= (0.85)(0.060)(0.190)(45)$

$= 0.436$ kips

Pull-over (Section E4.4.2)

$P_{nov} = 1.5t_1 d_w F_{u1}$ (*Eq.* E4.4.2-1)

$= (1.5)(0.036)(0.3175)(65)$

$= 1.11$ kips

Pull-out governs

ASD

$\Omega = 3.00$

$\frac{P_n}{\Omega} = \frac{0.436 \text{ kips}}{3.0} = 0.145$ kips

LRFD

ϕ = 0.50

ϕP_n = (0.5)(0.436) = 0.218 kips

Tension in screw

$P_{nt} = 0.8Pt_s$ (*Eq.* E4.4.3-1)

= (0.8)(2.42) = 1.94 kips

ASD

Ω = 3.00

$\frac{P_{nt}}{\Omega} = \frac{1.94}{3.0} = 0.647$ kips

LRFD

ϕ = 0.50

ϕP_{nt} = (0.50)(1.94) = 0.970 kips

For both ASD and LRFD, pull-out governs.

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PART V
SUPPLEMENTARY INFORMATION
FOR USE WITH THE
2001 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	
A1.3	
A2	
A2.1	
A2.2	
A2.3	
A2.4	
A3	
A3.1	
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SECTION 2 - LATERALLY UNBRACED COMPRESSION FLANGES

There are many situations in cold-formed steel structures where a flexural member is so shaped or connected that it will not buckle laterally as a unit, but where 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.

An accurate analysis of such situations is extremely complex and beyond the scope of routine design procedures. The method outlined below is based on considerable simplifications of an exact analysis. Its results have been checked against more than 100 tests. It has been found that discrepancies rarely exceed 30 percent on the conservative to 20 percent on the unconservative side. Thus, this method allows a reasonable estimate of the design strength to be made which, if desired, can be further improved by test.

The following design procedure was developed based upon tests on individual roof panels or hat-shaped beams. These members were tested as simply supported members with two concentrated loads thus creating a region of uniform moment. Therefore, the design procedure is applicable only to an individual hat-shape type section having its free flange subjected to compression resulting from flexure. It does not apply to the following:

(1) The compression elements of roof panels interconnected by welds, mechanical fasteners or mechanical seams.

(2) A system comprised of flexural members and panels.

For ease of explanation, the design procedure is presented in the following 10 steps:

(1) Determine the location of the neutral axis and define as the "equivalent column" the portion of the beam from the extreme compression fiber to a distance

$\left(\frac{3c_c - c_t}{12c_c}\right)d$ from the extreme compression fiber.

In this expression, c_c and c_t are the distances from the neutral axis to the extreme compression and tension fibers, respectively; d is the depth of the section.

(2) Determine the distance, y_o, measured parallel to the web, from the centroid of the equivalent column to its shear center. (If the cross section of the equivalent column is of angle or T-shape, its shear center is at the intersection of web and flange; if of channel shape, the location of the shear center is obtained from Section D1.1 of the *Specification*. If the flanges of the channel are of unequal width, for an approximation take w as the mean of the two flange widths, or compute the location of the shear center by rigorous methods.)

(3) To determine the spring constant β, isolate a portion of the member one inch long, apply a force of 0.001 kip perpendicular to the web at the level of the column centroid, and compute the corresponding lateral deflection, D, of that centroid. Then the spring constant $\beta = 0.001/D$.

(4) Calculate $T_o = h/(h + 3.4y_o)$ where h is the distance from the tension flange to the centroid of the equivalent column in inches.

(5) If the flange is laterally braced at two or more points, calculate

$P_e = 290{,}000\ I/L^2$, $C = \beta L^2/P_e$, and $L' = 3.7\sqrt[4]{I(h/t)^3}$

where

I = moment of inertia of equivalent column about its gravity axis, parallel to web, $in.^4$

L = unbraced length of equivalent column, in.

If $C \leq 30$, compute
$P_{cr} = TP_e[1 + \beta L^2/(\pi^2 P_e)]$

If $C > 30$, compute
$$P_{cr} = TP_e\left(0.60 + 0.635\sqrt{\beta L^2/P_e}\right)$$

In both cases,
$T = T_o$ if $L \geq L'$
$T = LT_o/L'$ if $L < L'$

(6) If the flange is braced at less than two points, compute
$P_{cr} = T_o\sqrt{4\beta EI}$

(7) Determine the slenderness ratio of the equivalent column

$(KL/r)_{eq} = 490/\sqrt{P_{cr}/A_c}$, where A_c = cross-sectional area of equivalent column, in.^{2}

(8) From paragraph (a) of Section C4 of the Specification, compute the stress, F_n, corresponding to $(KL/r)_{eq}$

(9) The design compression bending stress is $F_{b2} = 1.15\, F_n(c_c/y_c) \leq F_y$

where
c_c = distance from neutral axis of beam to extreme compression fiber, in.
y_c = distance from neutral axis of beam to centroid of equivalent column, in.

(10) Go to Equation C3.1.2.1-1, substituting F_{b2} for F_c

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

$$\sigma_{ex} = \frac{\pi^2 E}{\left(KL/r_x\right)^2},\ \text{ksi}$$

$$\sigma_{ey} = \frac{\pi^2 E}{\left(KL/r_y\right)^2},\ \text{ksi}$$

$$\sigma_t = \frac{1}{I_p}\left[GJ + \frac{\pi^2 E C_w}{(KL)^2}\right],\ \text{ksi}$$

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 = $(1/3)\ (l_i t_1^3 + l_2 t_2^3 + \ldots + l_n t_n^3)$
C_w= warping constant of torsion of the cross section, in.^{6}
l_1 = length of cross-section middle line of segment i, in.
t_1 = 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 = $\int_0^s R_c \, ds, \text{ in.}^2$

w_o = $\int_0^s R_o \, ds, \text{ in.}^2$

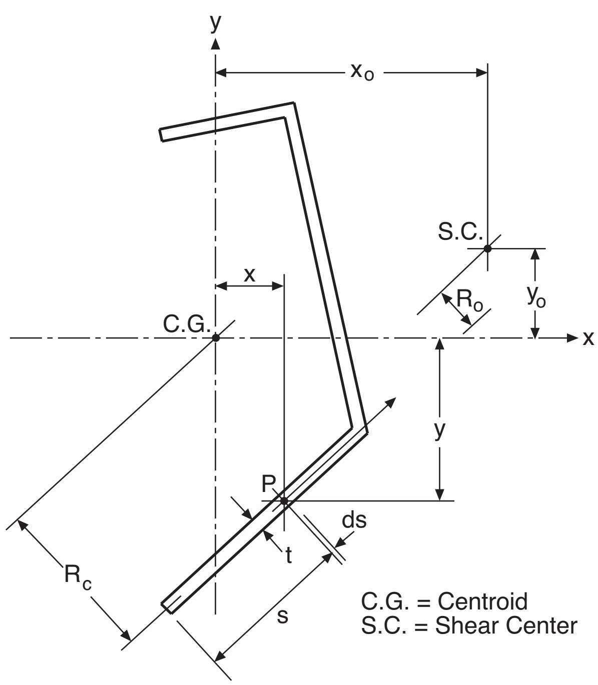

Figure 3-1
Non-Symmetric Cross-Section

x and y = the coordinates measured from the centroid to any point P along the middle line of the cross section, in.

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

General

Those taking advantage of the economies in building construction afforded by cold-formed steel structural framing are cautioned to observe the procedures outlined herein to obtain quality construction.

Designing

Building design involving cold-formed steel structural framing for floors, roofs, load-bearing walls, or curtain walls should be performed by, or under the direct supervision of, professional structural engineers registered in the state in which the project is constructed.

Detailing

Framing drawings should show size, thickness, type, and spacing of all structural members, including bridging and bracing. Details should be included for all connections either welded or screwed. Details should show the method of anchoring walls to the foundation.

Fabrication

Manufacturers of cold-formed steel structural members should maintain in-house quality control programs. Assembly of components into walls, etc., may be done on the job by the "stick built" method or off the job by the "panelized" method utilizing assembly jigs. Assemblers should follow details shown on fabrication and/or erection drawings.

Erection

Erection should be performed by experienced mechanics who follow the plans and specifications under the supervision of an experienced foreman or superintendent.

Inspection During Construction

Periodic inspections during the construction phase should be made by a professional structural engineer who either is or represents the design engineer of record. Such inspections should be made intermittently to enable the inspector to become generally familiar with the progress of the work and to observe, in general, that the work is or has been performed in such a manner that when completed, will be in general conformance with the design concept expressed in the construction documents. The inspector (as a representative of the engineer) does not have control over and is not in charge of, nor is liable in any way for methods, means, sequences, procedures, techniques, or schedules associated with the work of the project since these are the sole responsibility of the contractor. The inspector has no control over nor is responsible for safety procedures or safety training. The inspector will not control the work nor supervise the contractor or any sub-contractor.

Final Inspection

At the conclusion of the cold-formed steel construction process, a final inspection should be made by the engineer of record who should issue a statement that, to the best of his/her knowledge, information and belief, the cold-formed steel framing has been constructed in general conformance with the plans and specifications.

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PART VI
TEST PROCEDURES
FOR USE WITH THE
2001 EDITION OF THE
NORTH AMERICAN
SPECIFICATION FOR THE DESIGN OF
COLD-FORMED STEEL STRUCTURAL MEMBERS

SECTION 1 - TEST METHODS

AISI TS-1-02

ROTATIONAL-LATERAL STIFFNESS TEST METHOD FOR BEAM-TO-PANEL ASSEMBLIES

1. Scope

1.1 The purpose of this test is to determine the rotational-lateral stiffness of beam-to-panel assemblies. This test method is used primarily in determining the strength of beams connected to panels as part of a structural assembly.* The unattached "free" flange of the beam is restrained from lateral displacements and twisting by the bending stiffness of the beam elements, the connection between the "attached" flange of the beam and the panel, and the bending stiffness of the panel.

1.2 This test method applies to structural subassemblies consisting of panel, beam, and joint components, or of the joint between a wall, floor, ceiling, or roof panel and the supporting beam (purlin, girt, joist, stud).

1.3 This test method is also used to establish a limit of the displacements for avoiding joint failure.

1.4 The combined stiffness of the assembly determined by this method, K, consists of: (a) the lateral stiffness of the beam, K_a, which is a function of the geometry of the beam and geometric details of the beam-to-panel connection, (b) the local stiffness of the joint components in the immediate vicinity of the connection, K_b, which is affected by the type of fasteners, the fastener spacing used, and the geometry of the elements connected, and (c) the bending stiffness of the panel, K_c, which is a function of the moment of inertia of the panel, the beam spacing, and the beam location (edge vs interior). The latter stiffness shall be taken into account by theoretical analysis or by using the alternate test procedure described in Section 10.

1.5 For specific geometric conditions the design engineer may require duplicate testing using a new specimen with the beam orientation, or the force direction, reversed.

2. Description of Terms

2.1 *Subassembly* - A subassembly is a representative portion of a larger structural assembly consisting of a wall, floor, ceiling, or roof panel with one beam connected to the panel either continuously or at regular intervals (Figure 1).

2.2 *Panel* - The panel used in the subassembly may be made of any structural material, for example: aluminum, reinforced concrete, fiberboard, gypsum board, plastic, plywood, steel, etc. (Figure 1).

2.3 *Beam* - A beam may have an open or closed cross section. One flange of the beam is connected to the panel, and is called the "attached" flange. The other is the "unattached" flange (Figure 1).

2.4 *Joint or Connection* - A joint or connection includes the local area around a mechanical fastener, weld, or adhesively bonded area that connects the beam with the panel. The local area also includes filler material such as insulation located between the panel and the beam flange.

* *North American Specification for the Design of Cold-Formed Steel Structural Members*, Section C3.1.3

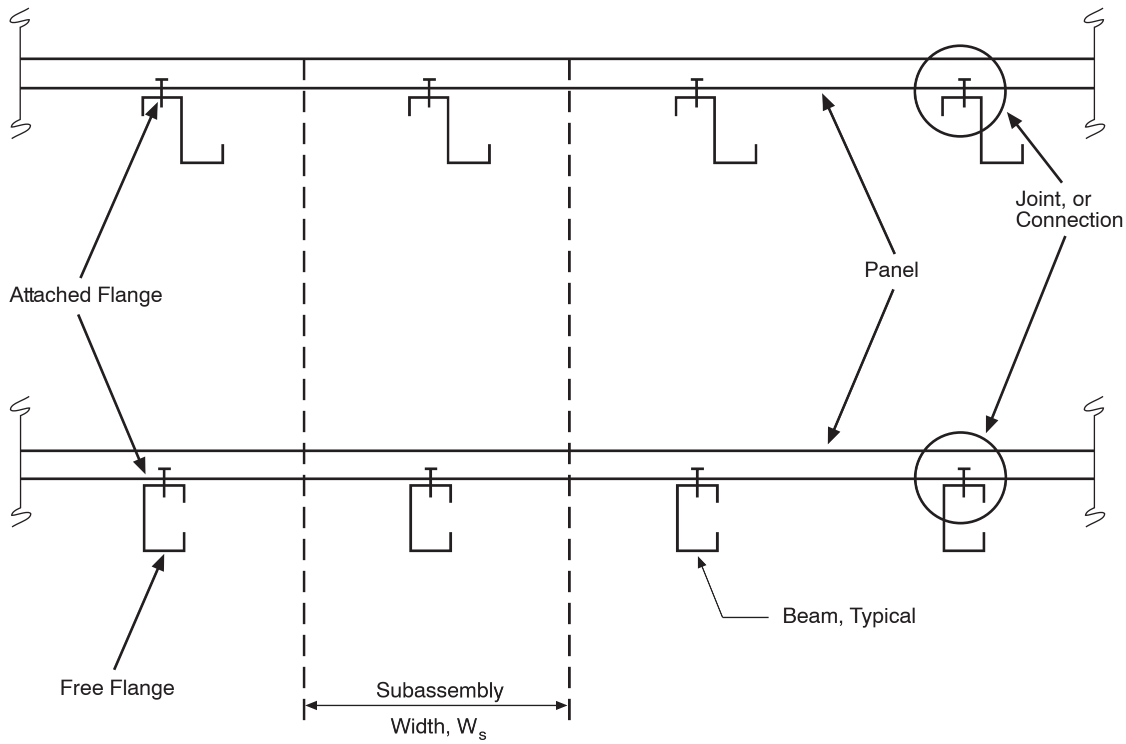

Figure 1 - Wall, Floor, Ceiling or Roof Assembly

2.5 *Lateral Load* - The total lateral load, P, in kips (kN), is applied to the unattached flange of

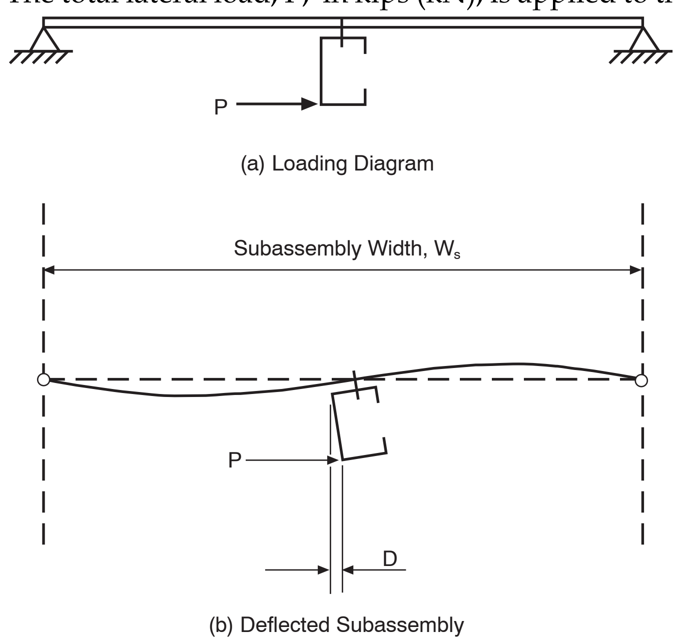

Figure 2 - Loaded and Deflected Subassembly

the beam (Figure 2) in a plane parallel to that of the original panel position.

2.6 *Lateral Deflection* - The lateral deflection (Figure 2) is the lateral displacement, D, in inches (mm), of the unattached flange due to the lateral load, P.

2.7 *Rotational-Lateral Stiffness* - The rotational-lateral stiffness, K, is equal to the total lateral load applied on the unattached flange of the test beam, divided by the length dimension of the beam, L_B (Figure 3b), and divided by the lateral deflection of the unattached flange of the beam at that load level. Thus, the units of K are: kips (kN) of lateral load per inch (mm) of beam length per inch (mm) of deflection, or k/in./in. (kN/mm/mm).

3. Materials

3.1 Components of the test specimen(s) shall be measured, and the component suppliers shall be identified.

3.2 Physical and material properties of the panel and beam shall be determined according to the latest edition of Specification ASTM E370 or other applicable standards.

4. Test Specimens

4.1 The overall panel width, W (Figure 3), of the specimen shall be such that the dial-gage support and the specimen support are each separated from the beam by a distance, W_I, not less than the largest of the following distances: (a) 1.5 times the overall panel depth P_D, (b) the overall width of the attached beam flange W_F, and (c) the fastener spacing along the flange of the beam, F_S. For ribbed panels, W_I shall also exceed two times the width of the attached flat of the panel.

4.2 The clamped width of the specimen, W_C, shall be at least equal to two times the panel depth, but not less than 2 in. (50.8 mm).

4.3 The end dimension, W_E, shall be long enough to conveniently attach a dial gage or an extensometer to the end of the panel.

4.4 The minimum overall panel width shall be equal to:

$$W = W_E + 2W_I + W_F + W_C \tag{1}$$

4.5 The minimum beam and panel length, L_B, of the test specimen shall not be less than the largest of (a) two times the maximum connector spacing, F_S, used in actual field installations, or (b) the nominal coverage width of the panel. The specimen shall contain at least two fasteners in each line of connections along the beam.

4.6 Each specimen shall be assembled under the supervision of a representative of the testing laboratory, either at the manufacturer's facilities or at the testing laboratory.

4.7 Each specimen shall be assembled from new material; i.e., materials not used in previous test specimens, and in accordance with manufacturer's specifications.

4.8 The fabrication and field installation procedures specified for the overall assembly, and the tools used, shall also be used in the specimen construction as much as possible.

4.9 Drilled or punched pilot holes in the panels or beams shall be the same as those used in field installations.

5. Test Setup

5.1 The test specimens may be tested in a horizontal or vertical position (Figure 3 and Figure 4, respectively). The zero-load readings of the deflection-measuring device(s) shall be recorded.

5.2 The clamped end of the panel shall be the only support of the test specimen.

5.3 When the test specimen panel is a hollow-core, corrugated, or trapezoidal panel, voids of the clamped regions shall be filled with filler materials such as wood, gypsum, or similar fill-

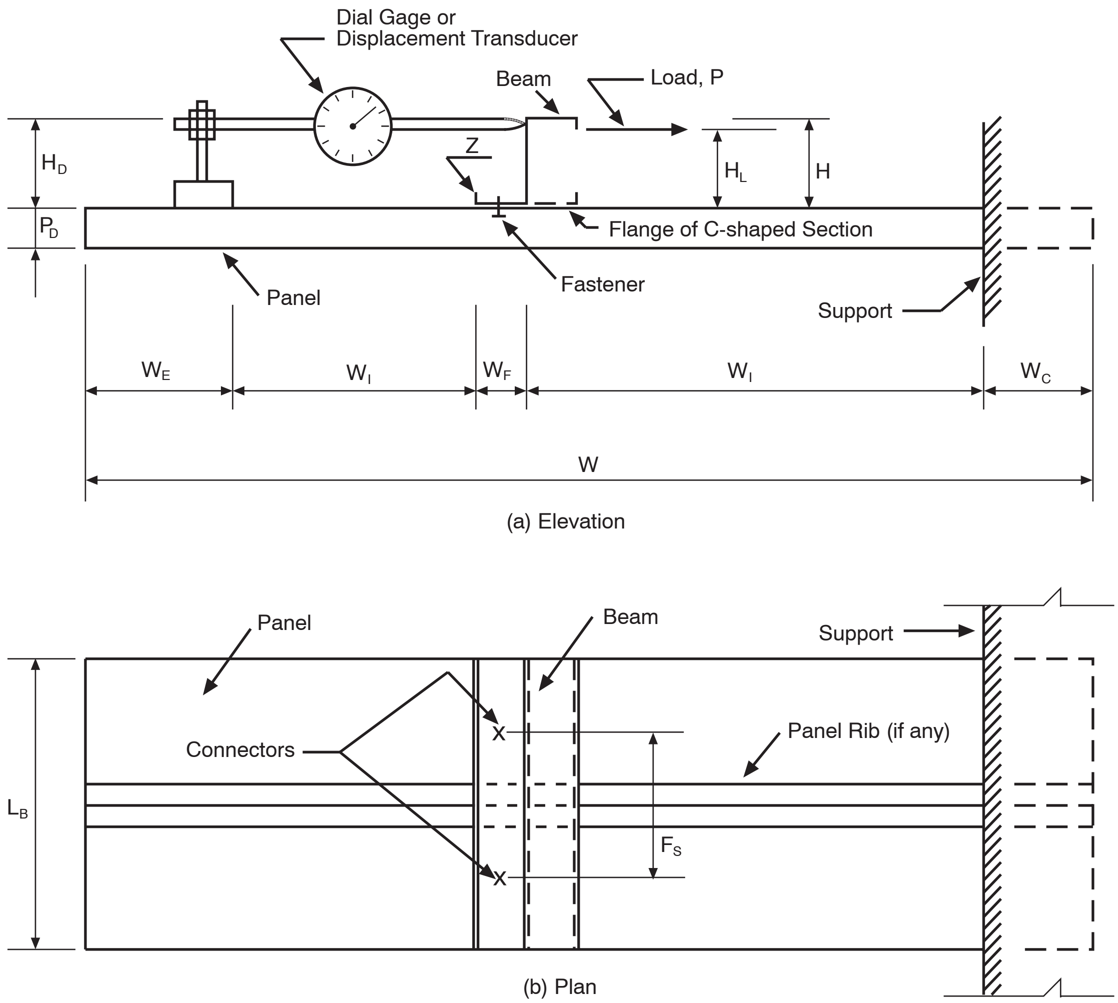

Figure 3 - Test Specimen and Horizontal Test Setup

er materials to ensure that the clamped overall depth of the panel is reasonably maintained. For foam-filled sandwich panels, if necessary, the filler material over the distance W_C may be replaced with wood, gypsum, or similar filler materials.

5.4 Loads applied to the unattached flange shall be introduced as close as possible to the extreme fiber of the beam, or at the intersection of the outer faces of the unattached flange and the web.

5.5 If the beam does not have a flat face perpendicular to the panel at the locations where the load is to be applied and the lateral displacement is to be measured, brackets are to be mechanically attached to the beam web to provide a flat surface. Figure 5 shows a typical application of a load bracket and/or dial gage bracket. The attachment of either bracket shall be accomplished such that the bracket does not stiffen the beam, or reduce its distortion.

5.6 The total lateral load applied, P, shall be distributed over several locations, if necessary, to reduce variations in the lateral deflection along the length of the unattached flange.

5.7 The load application shall be accomplished by chain or wire, and the necessary precautions shall be taken to ensure that the direction of the applied load remains essentially parallel to the original plane of the panel (Figure 5).

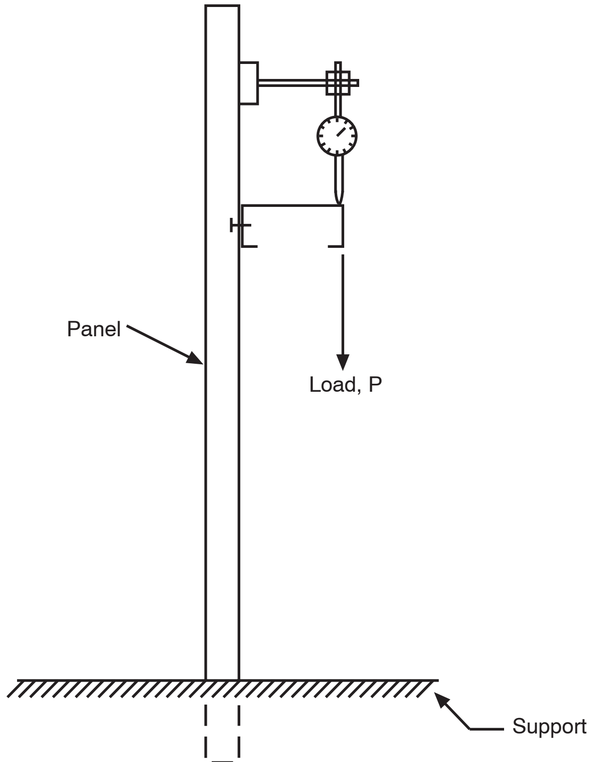

Figure 4 - Vertical Test Setup

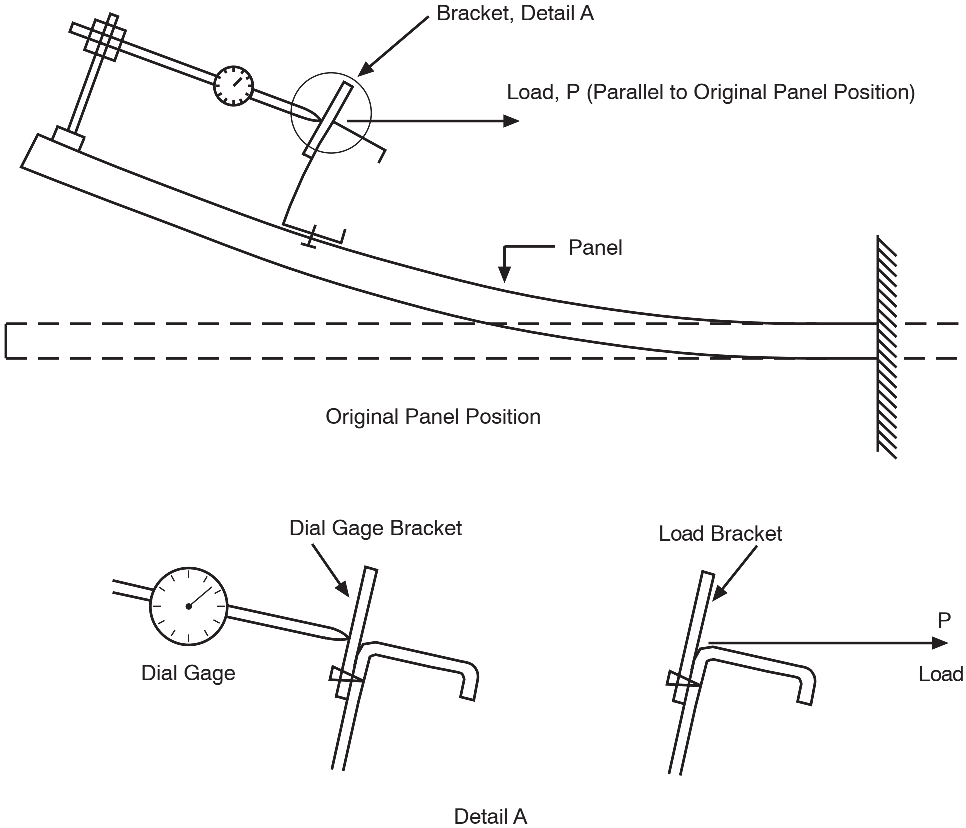

Figure 5 - Dial Gage and Load Bracket

5.8 One or more dial gages or displacement transducers shall be used to measure the lateral displacements during loading. The gages shall be arranged symmetrically about the midwidth point, and have graduations at not greater than 0.001 in. (0.0254 mm) intervals.

6. Test Procedures

6.1 The dial-gage height, H_D, and load height, H_L, as shown in Figure 3, shall be arranged such as to equal as close as possible the overall beam depth, H. Prior to loading the test specimen, the dimensions, H_D and H_L, and the dial-gage readings shall be recorded.

6.2 No preload is to be used. The load shall be applied in a direction which is critical for the intended use of the results.

6.3 The applied load shall be increased in five or more equal increments to the maximum expected value, in order to produce deflection increments of not more than 5 percent of the beam depth.

6.4 If the specimen includes fiberglass insulation or other non-metallic elements in the joint between panel and beam, the load shall be held at each increment for 5 minutes before reading the lateral movement.

6.5 After each load increment is added, and the deflection has stabilized, the load and lateral movement of the unattached flange shall be measured and recorded.

6.6 A test shall be terminated at failure (fastener pullout, fastener failure, panel buckling, panel failure, beam failure, etc.) and the mode of failure recorded, unless the design engineer has determined that the application of the rotational-lateral stiffness, K, occurs at lower load or displacement levels and that the test may be terminated earlier.

7. Number of Tests

7.1 The minimum number of tests for one set of parameters shall be three. For parametric studies using multiple values of one or more parameters a smaller number of tests may be used.

7.2 If used as part of a series of at least three tests, one test is sufficient for a specific condition of an all-metallic mechanically-fastened specimen using the same basic components, but using unique geometrical or physical-property differences such as fastener spacings, different beam or panel yield strengths, etc.

7.3 Three tests are required for any specific condition of welded or adhesively-bonded specimens, or for specimens using non-metallic materials.

7.4 When the rotational-lateral stiffness for three or more panel or beam thicknesses with otherwise identical parameters is to be determined, at least two specimens each with the minimum and the maximum thickness shall be tested. For a ratio of maximum-to-minimum thicknesses greater than 2.5, additional specimens with intermediate thicknesses must be tested. One test of every thickness may be used in accordance with Section 7.2.

7.5 When the rotational-lateral stiffness for a range of screw spacings is to be determined, the minimum number of specimens shall be as follows: For a ratio of maximum-to-minimum screw spacings equal to or less than 2, at least two specimens each with the minimum and the maximum screw spacing shall be tested. For a range of five or more different screw spacings, or for a ratio of maximum-to-minimum screw spacings greater than 2, additional specimens with intermediate spacings must be tested. One test of every screw spacing may be used in accordance with Section 7.2.

7.6 Where the rotational-lateral stiffness for a range of other panel parameters - such as yield or ultimate strength, changes in geometry, etc. - are to be determined, a number of tests similar to the requirements under Sections 7.2 through 7.5 shall be performed.

7.7 For unsymmetric or staggered fastener arrays and/or beams unsymmetric about a plane parallel to the web, duplicate tests may be required by the design engineer using new specimens with the beam orientation, or the force direction, reversed.

8. Test Evaluation Procedure

8.1 Typical load-displacement curves (P vs. D) obtained from the tests are as shown in Figure 6. For multiple tests of one set of test parameters, the curve resulting in the lowest value of K_t, as defined in Section 8.2, shall be used for the test evaluation procedure.*

8.2 The test stiffness, K_t, at any load level is determined by

$$K_t = P/D/L_B \tag{2}$$

8.3 The nominal test stiffness, K_N, shall be determined by

$$K_N = P_N/D_N/L_B \tag{3}$$

where P_N and D_N shall be determined for a point, N, such that either P_N shall be equal to 0.8 times the ultimate load, P_u, for load-displacement curves as shown in Figure 6(a), or the displacement D_N shall be equal to 0.8 times the ultimate displacement, D_u, for load-displacement curves as shown in Figure 6(b), or by a tangent drawn from the origin to the P-D curve as shown in Figure 6(c), resulting in $P_N \le 0.8P_u$ and $D_N \le 0.8D_u$.

8.4 When the design engineer specifies in advance a desired maximum lateral displacement limit of D_{NL}, the test may be discontinued when D_{NL} is reached, and K_N may be determined from P_N at D_{NL}, as long as the limits under Section 8.3 are observed and D_{NL} is not exceeded in actual design applications.

8.5 Where either H_D or H_L are not equal to the overall beam height, H, K_t and K_N shall be corrected by the factor H_DH_L/H^2.

8.6 In addition, K_t and K_N shall be adjusted by the stiffness contributions of the panel, K_c, derived from the linear-elastic displacement analysis representing the actual design applications, unless such an analysis shows that these contributions are insignificant. Alternately, the panel stiffness may be included by using the alternate test method under Section 10.

8.7 For subassemblies such as shown in Figure 2, the applied lateral test loads cause a bending moment distribution in the panel similar to that shown in Figure 7, and a lateral displacement of the unattached flange of the beam, D_c, equal to

$$D_c = \frac{PH_L{}^2W_s}{12EI} \tag{4}$$

where W_s is the width of the subassembly (Figure 2 and Figure 7), E is the modulus of elasticity of the panel material, and I is the effective moment of inertia of the panel cross section (obtained

* The test stiffness, K_t, includes the stiffness effects of the beam, K_e, and the beam-to-panel connection, K_b, but excludes the bending stiffness of the panel, K_c, and follows the relationship $K_t = (1/K_a+1/K_b)^{-1}$.

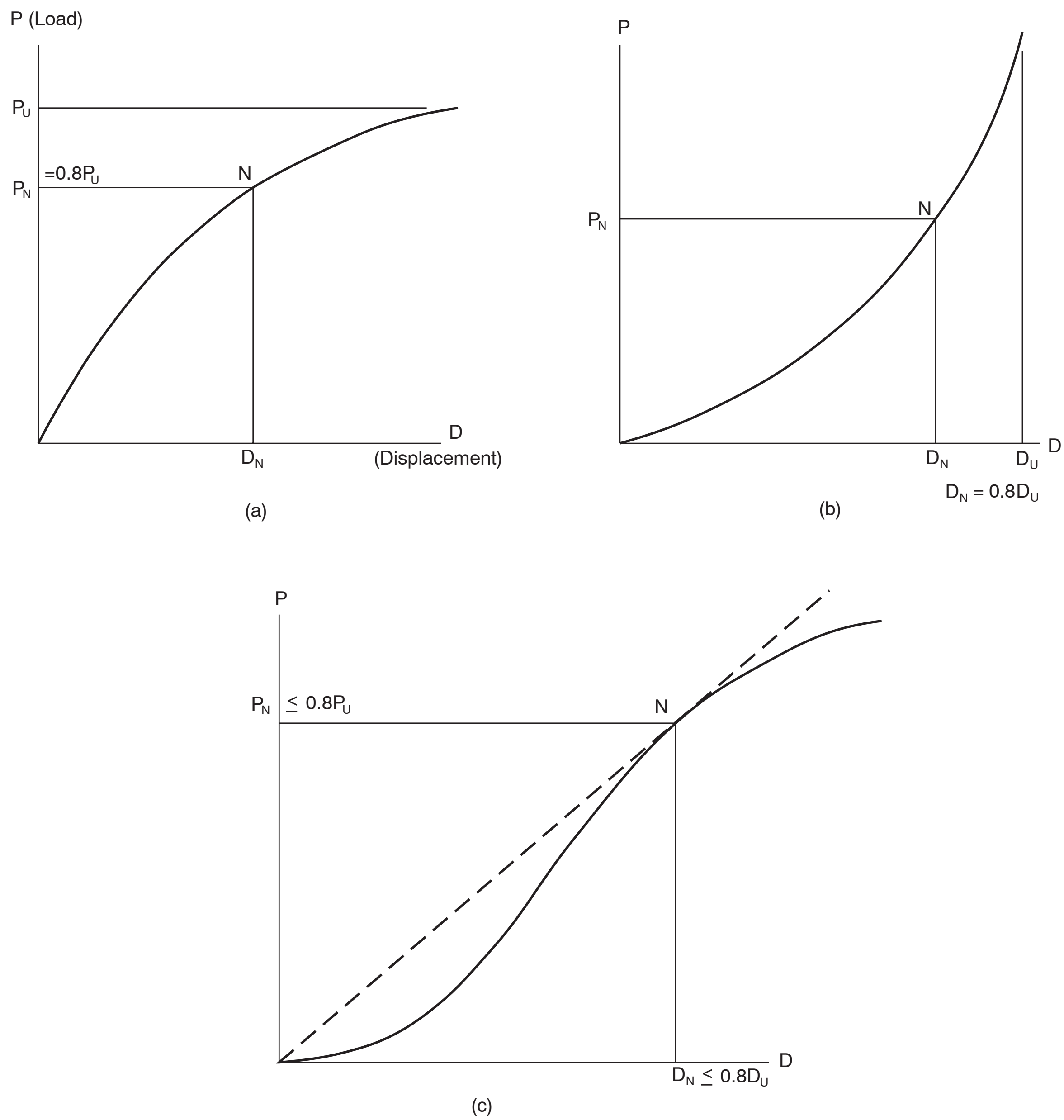

Figure 6 - Typical Load Displacement Curves

from deflection determination calculations for cold-formed metal deck panels.) The panel stiffness is equal to

$$K_c = 1/D_c \tag{5}$$

8.8 The overall rotational-lateral stiffness of the assembly shall be determined as

$$K = \left(\frac{1}{K_t} + \frac{1}{K_c}\right)^{-1} \tag{6}$$

8.9 When tests covering ranges of parameters (thickness, yield strengths, screws spacings, etc.) are conducted according to Section 7, a linear interpolation may be used to determine intermediate K values.

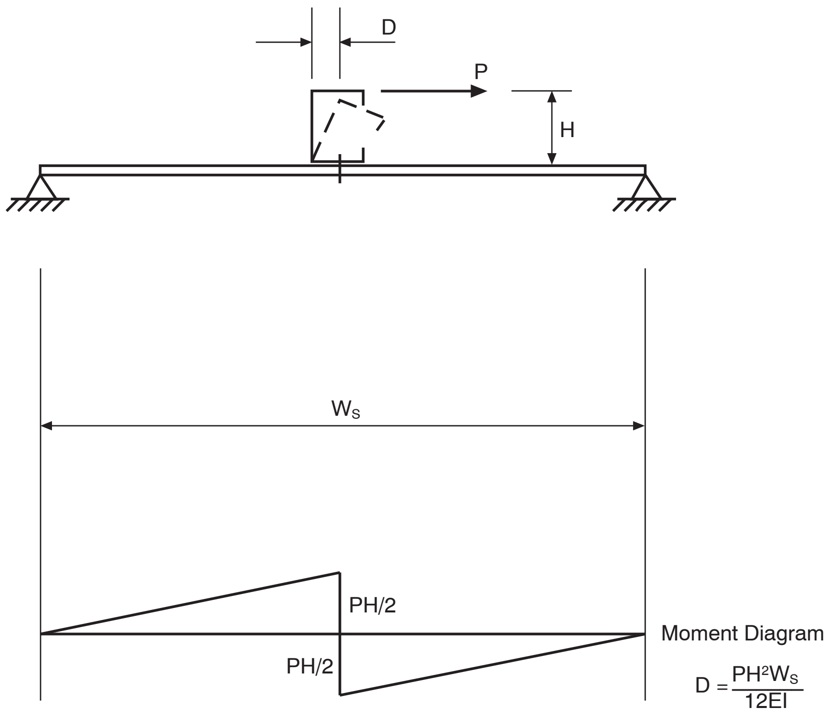

Figure 7 - Bending Moment Diagram with an Interior Beam

9. Test Report

9.1 The test report shall consist of a description of all specimen components, including drawings defining the actual and nominal geometry, material specifications, material properties test results describing the actual physical properties of each component, and the sources of supply. Differences between the actual and the nominal dimensions and material properties shall be noted in the report.

9.2 In addition, the test report shall contain a sketch or photograph of the test setup, the latest calibration date and accuracy of the equipment used, the signature of the person responsible for the tests, and a tabulation of all raw and evaluated test data.

9.3 All graphs resulting from the test evaluation procedure shall be included in the test report.

9.4 A summary statement, or tabulation, shall be included in the summary of the report to define the actual and nominal rotational-lateral stiffness derived from the tests conducted, including all limitations.

10. Alternate Rotational-Lateral Stiffness Test*

10.1 To include the panel-stiffness contribution in the test, rather than by linear-elastic analysis, the design engineer may request a test specimen and setup as shown in Figure 8 and Figure 9, respectively.

10.2 The test specimens shall be as described under Section 4 except as follows.

*This method is conservative as compared to the basic methods which analytically account for the stiffness of the panel.

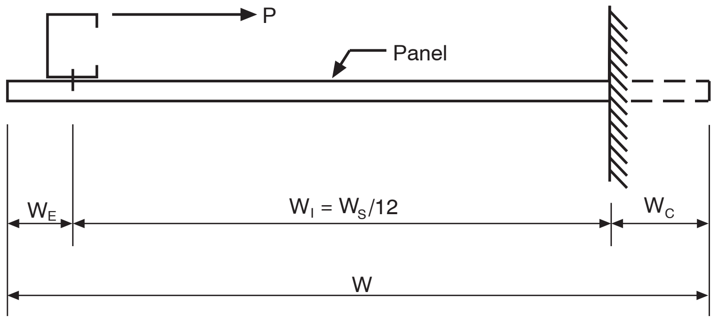

Figure 8 - Panel Width for Alternate Test

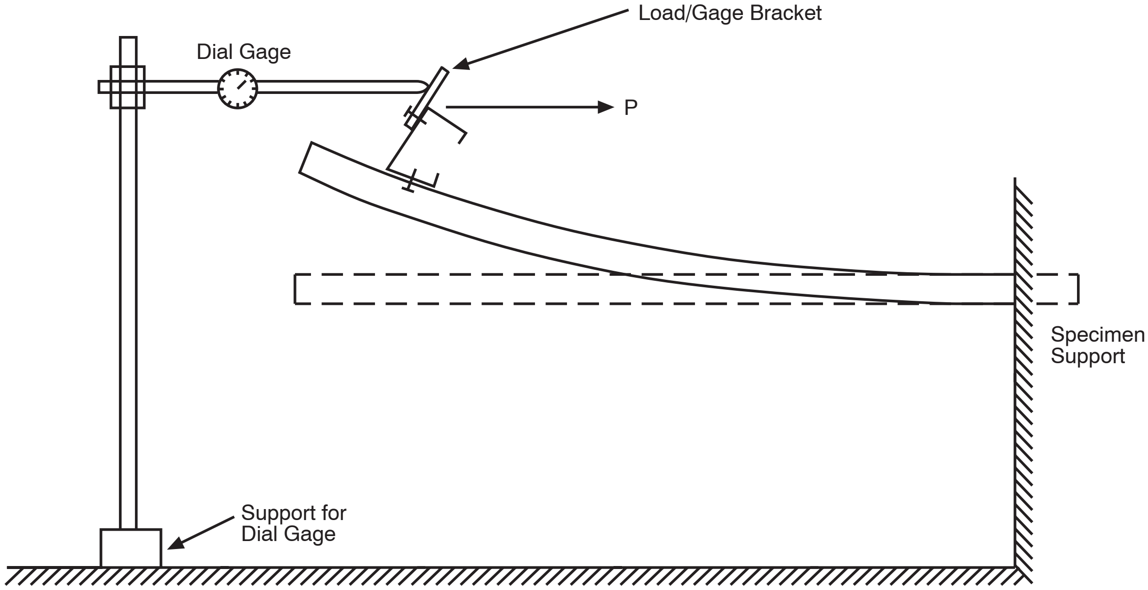

Figure 9 - Test Setup for Alternate Test

10.2.1 The minimum overall panel width of the specimen, W (Figure 8), shall be

$$W = W_E + W_I + W_C \tag{7}$$

10.2.2 The minimum end dimension, W_E, shall equal the width of the attached beam flange plus 4 in. (102 mm) to allow the development of local deformation patterns around the fasteners as they would develop in a real structure.

10.2.3 For specimens representing interior-beam subassemblies, as shown in Figures 1 and 2, the dimension W_I of the test specimen (Figure 8) shall be equal to $^1/_{12}$ of the subassembly width, W_S (Figures 1 and 2), to assure that the overall rotational-lateral stiffness contribution of the test-specimen panel is the same as that of the subassembly.

10.2.4. For other subassembly conditions, W_I shall be determined to represent the actual conditions.

10.3 The test-setup shall be as described under Section 5 except as follows.

10.3.1 The clamped support as shown in Figures 8 and 9 shall be sufficiently rigid to minimize the rotation and translation of the test specimen at the support.

10.3.2 The lateral-displacement measuring device shall be located on a support fixed relative to the clamped support of the test panel, as shown in Figure 9.

10.4 Test procedures shall be the same as described under Section 6.

10.5 The number of tests shall be determined as described in Section 7.

10.6 The test-evaluation procedure shall follow the underlying principles used to develop Section 8. The test stiffness at any load level shall be determined according to Equation 2 and the nominal test stiffness shall be determined according to Equation 3. No further adjustments are needed.

10.7 For other interior-beam spacings, for exterior-beam conditions, or for other geometrical conditions, the measured displacements shall be adjusted by a linear-elastic analysis to represent the actual field conditions, unless such an analysis shows that these displacements and their effect on K are insignificant.

AISI TS-2-02

STUB-COLUMN TEST METHOD FOR EFFECTIVE AREA OF COLD-FORMED STEEL COLUMNS

1. Scope

1.1 This test method covers the determination of the effective cross-sectional area of cold-formed steel columns. It primarily considers the effects of local buckling and residual stresses and applies to solid or perforated columns that have holes (or hole patterns) in the flat and/or curved elements of the cross section (1).[1]

1.2 The effective area is used to determine the nominal axial strength of cold-formed column sections in accordance with the *North American Specification for the Design of Cold-Formed Steel Structural Members,* hereafter called the *Specification.*

1.3 The effective area is a variable section property of columns. It reflects the effects of local buckling in relatively thin area elements caused by axial stresses, or loads. When the axial load is zero, the effective area is equal to the gross cross-sectional area; however, when an axial load is applied, the effective area may be less than the gross area. In such a case, the effective area will reduce with increasing load.

1.4 Local buckling reduces the axial load-carrying capacity that would otherwise be limited only by general yielding or overall column buckling. The amount of the reduction depends on the width-to-thickness ratio of the flat elements of the column cross section, the yield strength of the steel sheet from which the column is formed, and the size and frequency of holes or hole patterns, if present.

2. Applicable Documents

2.1 ASTM Standards:
A370 - Standard Test Methods and Definitions for Mechanical Testing of Steel Products
E4 - Standard Practices for Force Verification of Testing Machines

2.2 *North American Specification for the Design of Cold-Formed Steel Structural Members*, 2001 Edition.

3. Terminology

3.1 ASTM Definitions Standards:
E6 - Standard Terminology Relating to Methods of Mechanical Testing
IEEE/ASTM SI 10 - American National Standard for Use of the International System of Units (SI): The Modern Metric System

3.2 Description of terms specific to this standard:

Elements - Straight or curved portions of the cross section of a column or stub-column.

Local Buckling - The local buckling mode of a flat element of a column cross section, which influences the overall column-buckling behavior.

Overall Buckling - Buckling of a column as a function of its overall length.

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[1] Numbers in parentheses refer to references at the end of this test method.

Stub-Column - An axial compression member of the same cross section and material as the column for which the strength needs to be determined, but of sufficiently short length to preclude overall column buckling, if possible.

3.3 Symbols

A = the gross cross-sectional area of a column without holes or perforation, or the minimum cross-sectional area of a column with holes or perforation.

A_a = the average of all gross cross-sectional areas of the stub-columns without holes or perforations in a test unit, or the average minimum cross-sectional areas of the stub-columns with holes or perforations in a test unit.

A_e = the effective cross-sectional area of a stub-column at a load less than the ultimate test load, or the effective area of a full-length column.

A_{ei} = the effective cross-sectional area of a stub-column at load P_i.

A_{eu} = the effective cross-sectional area of a stub-column at ultimate load.

A_{eua} = the average effective cross-sectional area of a test unit of stub-columns at the ultimate axial load.

A_{euN} = the nominal effective cross-sectional area at ultimate load adjusted to the nominal thickness and the minimum specified yield strength.

A_{eu1} = the effective cross-sectional area of a stub-column with parameters of Test Unit 1 at ultimate load.

A_{eu2} = the effective cross-sectional area of a stub-column with parameters of Test Unit 2 at ultimate load.

A_N = the nominal gross cross-sectional area of a stub-column.

A_1 = the minimum gross cross-sectional area of a stub-column with parameters of Test Unit 1 at ultimate load.

A_2 = the minimum gross cross-sectional area of a stub-column with parameters of Test Unit 2 at ultimate load.

D = the axial shortening of a stub-column at load P.

D_i = the axial shortening of a stub-column at load P_i.

D_u = the axial shortening of a stub-column at load P_u.

f = the average axial stress assumed to be uniformly distributed over the effective cross-sectional area A_e.

f_i = the average axial stress assumed to be uniformly distributed over the effective cross-sectional area, A_{ei} at load P_i.

f_o = the average axial stress assumed to be uniformly distributed over the effective cross-sectional area, A_e, above which the section is not fully effective.

F_n = the nominal ultimate stress, assumed to be uniformly distributed over the effective cross section of a column as calculated from Section C4 of the *Specification*, at which flexural, torsional, torsional-flexural, or local buckling, and/or yielding, may occur.

F_u = the ultimate stress, assumed to be uniformly distributed, at which local failure occurs in a tested stub-column.

F_y = the minimum specified elastic limit or yield point of column or stub-column material.

F_{ya} = the average elastic limit or yield point of the sheet steel for a given test unit.

F_{yi} = the individual elastic limit or yield point of the sheet-steel specimens in a test unit.

F_{yN} = the minimum specified elastic limit or yield point of column or stub-column material.

i = load-displacement-reading number for a particular stub-column test (load displacement D_i at load P_i).

j = total number of load-displacement readings taken for a particular stub-column test.

L = the length of the stub-column test specimen.

L_p = the pitch of a repeating pattern of perforations along the longitudinal column axis.

n = the ratio of the effective cross-sectional area at the ultimate load to the full cross-sectional area, A_{eu}/A.

P = the applied axial compression force (column load).

P_i = the applied load at load-increment i.

P_N = the nominal failure load of a column.

P_u = the ultimate stub-column load at which local failure occurs.

P_{ua} = the average of all ultimate stub-column loads within a test unit.

r = the minimum radius of gyration of the cross-sectional area, A.

t = the nominal base-steel thickness exclusive of coating.

t_a = the average of all base-steel thicknesses within a test unit, exclusive of coating.

t_N = the nominal base-steel thickness within a test unit exclusive of coating.

W = the greatest overall width of the cross section including corner(s).

4. Significance

4.1 This test method provides requirements for testing, and equations to determine, the effective area of a cold-formed column section at ultimate load, A_{euN}, and the load- or stress-dependent effective area, A_e. These properties are used in the *Specification* to determine the ultimate and less-than-ultimate column strengths. The ultimate column strength, P_N, is the product of the minimum specified yield point, F_{yN}, or the buckling stress F_n, and the corresponding effective cross-sectional area at that stress, A_{euN}. At an applied column strength of P less than P_N, the corresponding effective cross-sectional area shall be A_e.

4.2 The test method also provides a means to observe, measure, and account for local buckling deformations when the appearance of a column section under stress must be determined.

4.3 An inherent assumption of the test method is that true stub-column behavior (which considers local buckling effects only) is achieved when overall column-buckling effects are eliminated. For this condition the ultimate test load on a stub-column, P_u, equals the product of the effective cross-sectional area at ultimate load, A_{eu}, times the stress that causes local buckling, or times the yield point of the virgin steel sheet. In case overall buckling cannot be avoided because of geometrical constraints, the critical column-buckling stress must be used.

4.4 The determination of A_e may be conducted by either one of the two following methods:

(1) The basic, more simple and conservative method:
This method is embodied in the main part of this document and is based on the measured test loads of stub-columns and their measured and tested physical and mechanical properties.

(2) An alternate and less conservative method:
This method is based on the shortening of stub-columns which occurs during testing. Also, this evaluation method requires more calculations. The results of this method lead to more accurate results for A_e, and to higher allowable axial loads at lower-than-ultimate stress levels. The evaluation procedure for this method is described in Appendix A.

5. Apparatus

5.1 The tests shall be conducted on a testing machine that complies with the requirements of ASTM E4.

5.2 Linear displacement devices for measuring lateral displacements shall have a 0.001 in. (0.0254 mm) least-reading capability.

5.3 Measuring devices for determination of the actual geometry of a test specimen shall have a 0.001 in. (0.0254 mm) least-reading capability.

5.4 If axial shortening is recorded, the measuring device shall have a 0.0001 in. (0.00254 mm) least-reading capability.

6. Test Unit

6.1 A test unit shall include a minimum of three identical stub-column specimens and a minimum of two corresponding sheet-type tensile specimens.

6.2 The specimens within a unit shall represent one type of cold-formed steel section with the same specified geometrical, physical, and chemical properties. The specimens may be taken from the same column or from different production runs provided the source of the specimens is properly identified and recorded.

6.3 If stub-column specimens are taken from different production runs, at least two corresponding sheet-type specimens must be taken and tested from each production run.

6.4 The stub-column test specimens shall be used to determine:

(1) The actual geometry of each specimen.

(2) The ultimate stub-column test load.

(3) Axial shortening at each load level if the alternate test-evaluation method described in Appendix A is used.

(4) Lateral displacements of the specimen at locations of interest (if desired).

6.5 The tensile test specimens shall be used to define the yield point of each stub-column specimen according to the requirements described in ASTM A370.

6.6 For each test specimen and test unit, the measured geometrical and tested physical properties of the individual specimens shall meet the requirements stated by the fabricator and material producer, respectively.

6.7 If the average area, thickness, or yield strength of a test unit varies by more than 20 percent from the respective nominal or specified-minimum value, the test unit is considered to be non-representative of the column section, and further evaluations of the effective area are considered to be invalid.

7. Stub-Column Specimens

The stub-column specimens shall meet length and end-flatness requirements as follows, depending on whether or not unconnected or welded endplates are used.

7.1 Stub-Column Length - The length requirements of the stub-column test specimen, L, as shown in Figures 1 and 2, are that it be (1) sufficiently short to eliminate overall column buckling effects, and (2) sufficiently long to minimize the end effects during loading, which means that its center portion be representative of the repetitive hole pattern in the full column.

7.1.1 To eliminate overall column-buckling effects, the stub-column length shall not exceed twenty times the minimum radius of gyration, r, of the cross section, A, except where necessary to meet the requirements of Sections 7.1.2 through 7.1.5.

7.1.2 For unperforated columns (Figure 1a) the stub-column length shall not be less than three times the greatest overall width of the cross section, W.

7.1.3 For perforated columns in which the pitch (gage length) of the perforation pattern, L_p, for a single hole or a group of holes, is smaller than, or equal to, the greatest overall width, W, of the cross section (Figures 1b and 1g), or for a single hole pattern with a gage length larger than the greatest overall width (Figure 1c), the specimen length shall not be less than three times the greatest overall width of the cross section, W. For widely spaced hole patterns (Figure 1c) the significant hole or hole pattern shall be located at or near the midlength of the stub column.

7.1.4 For perforated columns in which the pitch of the perforation pattern, L_p, is greater than the widest side, W, of the cross section (Figures 1d, 1e, 1f, and 1h), the specimen length shall not be less than three times the pitch of the perforation pattern.

7.1.5 For perforated sections in which the specimen end planes must pass through the normal perforation pattern (Figure 1i), a special section (Figure 1j) may be fabricated to obtain full cross-sectional surfaces at the specimen ends.

7.2 Stub-Column End Surface Preparation - The end planes of the stub-column test specimens shall be carefully cut to a flatness tolerance of plus or minus 0.002 in. (0.0508 mm). When

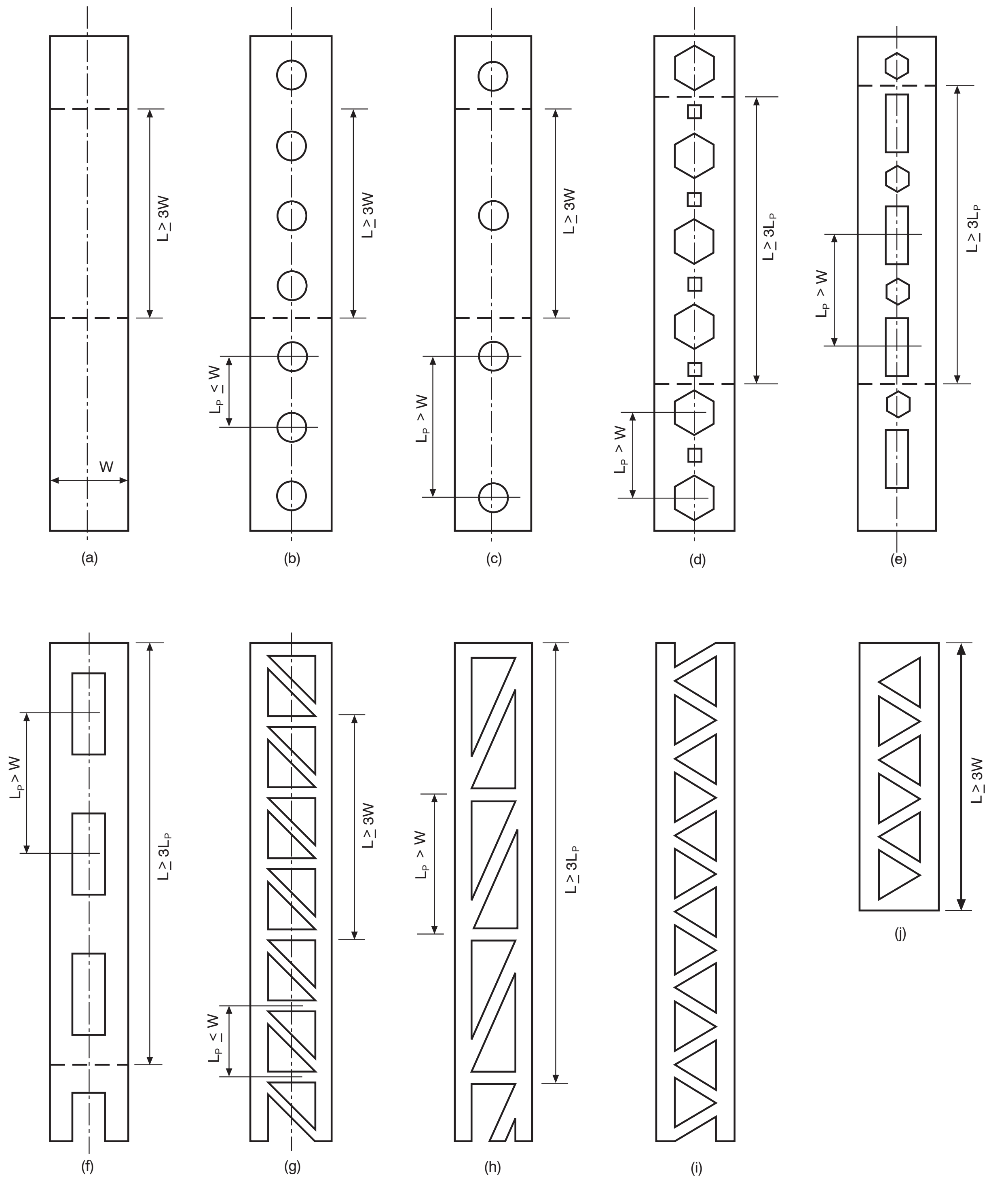


Figure 1 - Hypothetical Perforation Patterns and Suggested Stub Column Lengths

NOTES: (1) Perforations shown are in a flat portion of a member with width W
(2) L = Length of stub-column
(3) L_p = Pitch Length of Perforation Pattern

the required flatness can be achieved, welding of the stub-column ends to the endplates is not required. However, when this flatness cannot be achieved, steel endplates shall be continuously welded to both ends of the specimen so that there shall be no gap between the ends of the stub column and the endplates.

7.3 Stub-Column Specimen Source - Stub-column test specimens may be cut from the commercially fabricated column product. Alternatively, stub columns may be specially fabricated provided care is taken not to exceed the cold work of forming expected in the commercial product; however, subsequent proof tests using specimens from commercially produced columns are recommended.

7.4 Tensile Specimen Source - Longitudinal tensile specimens shall be cut from the center of the widest flat of a formed section from which the stub-column specimens have been taken. If perforations are large and frequent in all flats of the formed section, the tensile specimens may be taken from the sheet or coil material used for the fabrication of the stub-column specimens. The tensile specimens shall not be taken from parts of a previously tested stub column.

7.5 Endplate Requirements - Steel endplates shall be at least 0.5 in. (12.7 mm) thick and have a flatness tolerance of plus or minus 0.0002 in. (0.00508 mm).

8. Stub-Column Test Procedure

8.1 Vertical alignment of the stub column is essential to ensure that the applied load is uniformly distributed over the specimen end surfaces. Care should also be taken to center the specimen on the axis of the test machine.

8.1.1 Steel endplates shall be used to transfer the test loads uniformly into the stub columns (Figure 2).

8.1.2 A 0.5 in. (12.7 mm) thick layer of grout, similar to gypsum-based concrete capping compound used for fast setting, shall be placed between the stub-column endplates and the machine heads to facilitate aligning the test specimen (Figure 2).

8.2 When an axial compression load is applied to the test specimen as a result of grout expansion during curing, or if a small preload is purposely applied to ensure proper contact between the stub-column endplates and the machine heads, the load shall be treated as part of the applied test load.

8.3 The load increments applied during the test shall not exceed 10 percent of the estimated ultimate test load.

8.4 The maximum loading rate between load increments shall not exceed a corresponding applied stress rate of 3 ksi (21 MPa) of cross-sectional area per minute.

8.5 When axial shortening values are recorded, the following procedures shall be required:

(1) The change in the vertical distance between the inside surfaces of the endplates (Figure 2) shall be measured to the nearest 0.0001 in. (0.00254 mm) at each load increment for each specimen.

(2) The load increments applied during the test shall be the same for each specimen within a test unit, with a variation not to exceed one percent.

9. Calculations

9.1 For a given test unit, all individual ultimate loads, P_u, derived from the stub-column tests shall be used to calculate the average ultimate load, P_{ua}. Similarly, all individual yield points, F_{yi}, derived from the tensile tests of the same unit shall be used to calculate the average yield point of the same test unit, F_{ya}.

9.2 The effective areas A_{eua}, A_{euN}, and A_e shall be calculated as specified in Sections 9.3 through 9.6; however, the final value of these effective areas shall not exceed that of the minimum gross cross-sectional area, A.

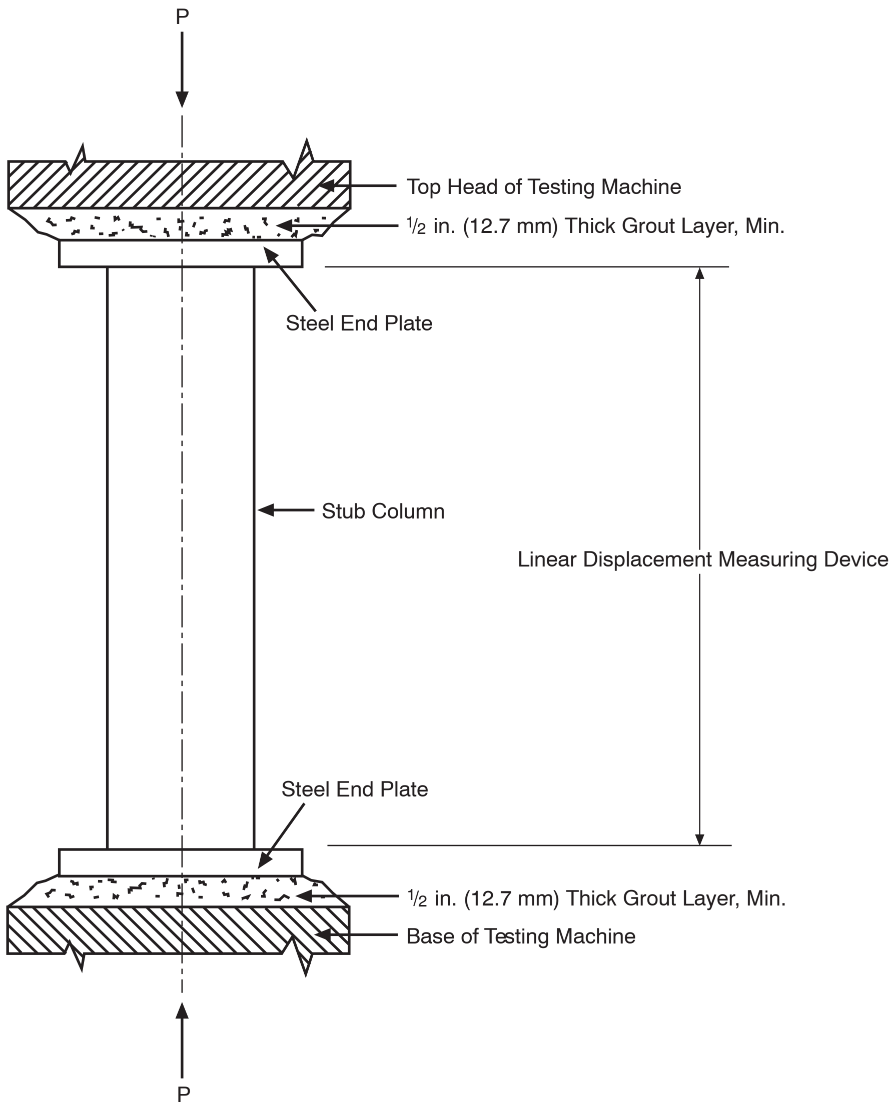


Figure 2 - Test Setup

9.3 For tests in which the length of the stub column does not exceed twenty times the minimum radius of gyration of the cross section, r, the average effective area at the ultimate load, A_{eua}, for a given test unit shall be calculated as

$$A_{eua} = P_{ua}/F_{ya}$$

9.4 For tests in which the length of the stub column exceeds twenty times the minimum radius of gyration of the cross section, the average effective area at the ultimate load shall be determined by iteration of the following equations:

$$A_{eua} = A_a - \left(A_a - \frac{P_{ua}}{F_n}\right)\left(\frac{F_n}{F_{ya}}\right)^n$$

where A_a is the average minimum area of the stub columns in the test unit, and F_n is the flexural or torsional-flexural buckling stress derived from Section C4 of the *Specification* with K = 0.5 (using the average cross-sectional properties of the test unit). The exponent n is determined as follows:

$$n = A_{eua}/A_a$$

Assuming an initial value for n equal to less than 1.0, A_{eua} can be calculated from the first equation. Using this A_{eua} in the second equation will provide a new value for n. Repeating this process will lead to convergence of the above equations and an acceptable value of A_{eua} for one specific test unit.

9.5 The value of A_{eua} for a specific test unit shall be adjusted to A_{euN}, which is the effective cross-sectional area of a column at ultimate load with a nominal cross section of A_N and a specified minimum yield point of F_{yN}. The adjustment shall be performed in one or two steps as follows.

9.5.1 If the average area of the stub columns in the test unit, A_a, or the average base steel thickness, t_a, are different from the nominal area or thickness, respectively, the effective cross-sectional area at ultimate load shall be calculated as follows:

$$A_{euN} = A_{eua}\left(\frac{A_N}{A_a}\right)$$

or

$$A_{euN} = A_{eua}\left(\frac{t_N}{t_a}\right)$$

9.5.2 If the average yield point of all stub columns in a test unit, F_{ya}, is different from the nominal yield point, F_y, the effective cross-sectional area at ultimate load shall be the lower of the two values calculated as follows:

$$A_{euN} = A_N\left[1 - \left(1 - \frac{A_{eua}}{A_N}\right)\left(\frac{F_{yN}}{F_{ya}}\right)\right]$$

or

$$A_{euN} = A_{eua}\left(\frac{F_{ya}}{F_{yN}}\right)^{0.4}$$

9.5.3 If the average area and the minimum specified yield point are different from the nominal values of a test unit, A_{euN} derived from the equation in Section 9.5.1 shall be used as A_{eua} in the equations of Section 9.5.2, which will lead to an acceptable value of A_{euN}.

9.6 The effective area at any working stress level, A_e, may be determined by

$$A_e = A_N - (A_N - A_{euN})\left(\frac{f}{F_{yN}}\right)^n$$

9.7 For a series of sections, such as in a parameter study during which only one parameter (thickness, depth, width, yield strength, etc.) is changed, interpolations between test units, or extrapolations beyond test units, shall be acceptable as described in Appendix B.

9.8 Extrapolations beyond 20 percent of the extreme parameters tested shall not be permitted.

10. Report

10.1 Documentation - The report shall include a complete record of the sources and locations of all stub-column and tensile-test specimens and shall describe whether the specimens were taken from one or several columns, one or several production runs, coil stock, or other sources.

10.2 The documentation shall include all measurements taken for each stub-column test specimen, including (1) cross-section dimensions, (2) uncoated sheet thickness, (3) longitudinal yield point, (4) end preparation procedure, (5) applicable material specification, and (6) test and evaluation procedure used.

10.3 The determination of the selected stub-column length shall be fully documented with appropriate calculations.

10.4 A description of the test setup - including the endplates, the grout layer used for alignment, and the instrumentation used to measure lateral displacements and axial shortening - shall be included.

10.5 The report shall include the load increments, rate of loading, and intermediate and ultimate loads for each stub column tested.

10.6 The report shall include complete calculations and results of the effective area, A_{euN}, for each test unit and calculations of A_e, if requested.

11. Precision

11.1 The following criteria shall be used to judge the acceptability of the test results.

11.1.1 Repeatability - Individual stub-column test results shall be considered suspect if they differ by more than 10 percent from the mean value for a test unit with at least three specimens.

11.1.2 Reproducibility - The results of tests on stub-columns conducted at two or more laboratories should agree within ten (10) percent when adjusted for differences in cross sectional dimensions and yield strength.

REFERENCES

1. T. Pekoz, “Development of a Unified Approach to the Design of Cold-Formed Steel Members”, Committee of Sheet Steel Producers, American Iron and Steel Institute, 1140 Connecticut Avenue, Washington, DC 20036, 1986.

APPENDIX A
Use of Axial Shortening Measurements In Design

A-1 Axial shortening measurements as part of thin-walled cold-formed steel stub-column tests may be used as an alternative method of determining the effective area of a column, A_e, at a certain design load or stress. This method provides a more accurate and less conservative alternative to design engineers to determine the effective area of a column section, A_e.

A-2 The calculations by this method shall be made separately for each stub-column specimen within a test unit. This shall result in a total of j calculations as a result of a total of j load-displacement tests for each test unit.

A-3 For a given specimen the effective area at ultimate load, A_{eu}, shall be calculated from Section 9.3 or 9.4 letting $A_{eua} = A_{eu}$, $A_a = A$, $F_{ya} = F_y$, and $P_{ua} = P_u$.

A-3.1 Calculations at each load-displacement reading, i, shall be conducted according to the following procedure; however, at zero load, the effective area, A_e, shall be equal to the minimum cross-sectional area, A. This provides results for the effective area at each load point:

(1) Starting with the lowest load-displacement reading, the effective area, A_i, and the assumed uniformly distributed stress f_i, shall be calculated for each reading, i, from:

$$A_{ei} = \frac{P_i D_u}{F_y D_i}$$

and

$$f_i = \frac{F_y D_i}{D_u}$$

where D_i and D_u are the axial shortening at loads P_i and P_u, respectively.

(2) If A_{ei} calculated is greater than A, A_{ei} shall be set equal to A.

(3) If A_{ei} calculated is less than A, A_{ei} shall be as calculated, and f_o, the stress above which the section is not fully effective, shall be set equal to f_{i-1}, as calculated for the previous load-displacement reading.

A.3.2 For specimens within a test unit, the lowest A_{ei} values shall be used for further evaluations.

A-4 For any load that causes a stress f higher than f_o, an exponential equation may be developed as follows:

$$A_e = A\left[1 - \left(1 - \frac{A_{eu}}{A}\right)\left(\frac{f - f_o}{F_y - f_o}\right)^b\right]$$

where

$$b = \frac{\sum_{i=1}^{j}(X_i)(Y_i) - (a)\sum_{i=1}^{j}(X_i)}{\sum_{i=1}^{j}(X_i)^2}$$

and

$$X_i = \ln\left(\frac{f_i - f_o}{F_y - f_o}\right)$$

$$Y_i = \ln\left(1 - \frac{A_{ei}}{A}\right)$$

$$a = \ln\left(1 - \frac{A_{eu}}{A}\right)$$

and ln designates the natural logarithm.

A-5 If the effective areas for a section with specified dimensions and minimum yield strength are desired, which are different from the tested specimens, the A_{eu} and A_{ei} values calculated under Section A-3 shall be normalized to the specified parameters according to Section 9.5 before the curve-fitting procedure of Section A-4 is employed. The variables A, A_{eu} and F_y should be changed to A_N, A_{euN} and F_{yN}.

A-6 All calculations pertaining to this procedure shall be included in the report, as discussed in Section 10.

APPENDIX B
Parametric Studies

B-1 For parametric studies intended to develop the effective area for a series of sections with the same basic cross section (either C, U, H, or any other shape) and the same hole pattern, but with one or more changing parameters, the required number of test units may be less than the sum of all sections with different geometries and yield strengths.

B-1.1 For a series of sections with three different values for one parameter only (dimension or nominal yield point), at least two test units shall be chosen to include the minimum and the maximum value of the changing parameter. For the third value, A_{eu} may be interpolated according to Section B-2.

B-1.2 If more than three different values for one parameter are included in a series of sections, additional units with intermediate values shall be tested such that the ratio of the changing values in adjacent units is not greater than 1.5 or be less than 0.67. For intermediate values of the changing parameter, A_{eu} may be interpolated according to Section B-2.

B-1.3 For a series of sections with the same basic cross section that includes different values for several parameters (dimensions and/or yield strength), an appropriate factorial of test units shall be established by the responsible professional engineer in accordance with the guidelines for changes in an individual parameter, and in compliance with responsible code authorities. Interpolations and extrapolations may be made as mutually agreeable, following the general guidelines set forth in Section B-2 for changes of one parameter only.

B-1.4 For a section that falls outside a series of tested members with the same basic cross section, A_{eu} may be extrapolated provided the changing parameter does not exceed a value of 20 percent below or above the respective minimum or maximum values tested in the series.

B-2 Interpolations and extrapolations are allowed as part of a parametric study, and as defined under B-1.

B-2.1 For a section with a thickness different from the thicknesses tested, but with identical overall nominal cross-sectional dimensions and minimum specified yield strength, A_{eu} for a thickness t and an area A may be calculated provided t does not exceed the limits described under Section B-1.2 and B-1.4. Under these conditions, A_{eu} may be determined by interpolation or extrapolation from the results of the nearest two test units with thicknesses t_1 and t_2, respectively.

$$A_{eu} = A\left[\frac{A_{eu1}}{A_1} + \left(\frac{A_{eu2}}{A_2} - \frac{A_{eu1}}{A_1}\right)\left(\frac{t_1 - t}{t_1 - t_2}\right)\right]$$

where A_1 and A_2 are the minimum gross cross-sectional areas, and A_{ue1} and A_{eu2} are the nominal effective cross-sectional areas for Test Units 1 and 2, respectively.

B-2.2 For a section with a yield point different from the yield points tested, but with identical cross-sectional dimensions, A_{eu} for a yield point F_y may be calculated provided F_y does not exceed the limits described under Section B-1.2 and B-1.4. Under these conditions, A_{eu} may be determined by interpolation or extrapolation from the results of the nearest two test units with yield points F_{y1} and F_{y2}, and with effective areas A_{eu1} and A_{eu2}, respectively.

$$A_{eu} = A\left[\frac{A_{eu1}}{A_1} + \left(\frac{A_{eu2}}{A_2} - \frac{A_{eu1}}{A_1}\right)\left(\frac{F_{y1} - F_y}{F_{y1} - F_{y2}}\right)\right]$$

AISI TS-3-02

STANDARD METHODS FOR DETERMINATION OF UNIFORM AND LOCAL DUCTILITY

1. Scope

This method covers the determination of uniform and local ductility from a tension test. Its primary use is as an alternative method of determining if a steel has adequate ductility as defined in the *Specification*. It is based on the method suggested by Dhalla and Winter.

2. Referenced Documents

ASTM Standard A370, "Standard Test Methods and Definitions for Mechanical Testing of Steel Products".

"*North American Specification for the Design of Cold-Formed Steel Structural Members*", 2001 Edition.

Dhalla, A. K. and Winter, G., "Steel Ductility Measurements," Journal of Structural Division, Proceedings ASCE, Vol. 100, No. ST2, February 1974.

3. Symbols

e_3 = linear elongation, in., in 3-in. (76.2 mm) gage length
e_{3e} = linear elongation, in., in 2-in. (50.8 mm) gage length not containing 1-in. (25.4 mm) length of fractured portion
e_u = linear elongation, in., at ultimate load in standard tension coupon test
ε_3 = percent elongation in 3-in. (76.2 mm) gage length
ε_{3e} = percent elongation in 2-in. (50.8 mm) gage length not containing 1-in. (25.4 mm) length of fractured portion
ε_f = percent elongation at fracture in 2-in. (50.8 mm) gage length of standard tension coupon
ε_u = percent elongation at ultimate load in standard tension coupon test
$\varepsilon_{uniform}$ = uniform percent elongation
ε_{local} = local percent elongation in 1/2 in. (12.7 mm) gage length
$\varepsilon_{1/2}$ = percent elongation in 1/2 in. (12.7 mm) gage length

4. Test Procedure

4.1 Prepare a tension coupon according to ASTM Standard A370 except that the central length of 1/2 in. (12.7 mm) uniform width of the coupon should be at least $3^1/_2$ in. (88.9 mm) long.

4.2 Scribe gage lines at 1/2-in. (12.7 mm) intervals along the entire length of the coupon.

4.3 After completion of the coupon test, measure the following two permanent plastic deformations: (a) the linear elongation in a 3-in. (76.2 mm) gage length, e_3, such that the fractured portion is included (preferably near the middle third of this 3-in. (76.2 mm) gage length); and (b) the linear elongation in a 1-in. (25.4 mm) gage length containing the fracture.

4.4 Subtract the latter from the former. This difference gives the linear elongation, e_{3e}, in a 2-in. (50.8 mm) gage length not containing the 1-in. (25.4 mm) length of the fractured portion.

4.5 From the two preceding elongation measurements, e_3 and e_{3e}, calculate the percentage elongations $\varepsilon_3 = (e_3/3) \times 100$, and $\varepsilon_{3e} = (e_{3e}/2) \times 100$. From these percentage elongations, the uniform and local ductility parameters are obtained as follows.

4.6 Since the fractured portion which includes local elongation is eliminated from ε_{3e}, it is a measure of the uniform ductility of the material. Thus

$$\varepsilon_{uniform} = \varepsilon_{3e} \tag{1}$$

4.7 The local elongation is determined over a small length which includes the fractured portion. For simplicity, this length is here assumed to be 1/2 in. (12.7 mm) which is large enough to include the necked portion of most thicknesses and type of sheet steels used, and is small enough to give valid comparison for different types of steels. Thus

$$\varepsilon_{local} = \varepsilon_{1/2} = 6\,(\varepsilon_3 - \varepsilon_{3e}) + \varepsilon_{3e} \tag{2}$$

in which 6 = the multiplication factor which converts the local elongation $(\varepsilon_3 - \varepsilon_{3e})$ measured in 3 in. (76.2 mm) to local elongation in 1/2 in. (12.7 mm) gage length.

5. Alternate Test Procedure

5.1 Prepare a standard tension coupon according to ASTM A370 with a standard 2-in. (50.8 mm) gage length.

5.2 The strain at the tensile strength, i.e., percentage strain ε_u at the peak of the stress-strain curve, is a measure of uniform ductility, because up to this strain no necking or local elongation has taken place. Therefore, to obtain the uniform ductility the stress-strain curve is plotted at least up to the maximum load or the linear elongation, e_u, at maximum load is measured directly, so that $\varepsilon_u = (e_u/2) \times 100$.

5.3 To obtain a measure of the local ductility it is necessary to measure the percentage strain at fracture ε_f, also in a 2-in. (50.8 mm) gage length. However, the strain which occurs after the maximum load has been passed (descending branch) is the necking strain, and is localized at the eventual fracture zone, thus $(\varepsilon_f - \varepsilon_u)$ is the local percentage elongation referred to in a 2-in. (50.4 mm) gage length. The following equation converts this $(\varepsilon_f - \varepsilon_u)$ into the percentage elongation in a 1/2 in. (12.7 mm) gage length:

$$\varepsilon_{local} = \varepsilon_{1/2} = \varepsilon_u + 4\,(\varepsilon_f - \varepsilon_u) \tag{3}$$

in which 4 = the multiplication factor to convert a 2-in. (50.8 mm) gage length local elongation to a 1/2 in. (12.7 mm) gage length.

AISI TS-4-02

STANDARD TEST METHODS FOR DETERMINING THE TENSILE AND SHEAR STRENGTH OF SCREWS

1. Scope

1.1 These performance test methods establish procedures for conducting tests to determine the tensile and shear strength of carbon steel screws. The screws may be thread-forming or thread-cutting, with or without a self-drilling point, and with or without washers. The intended application for these screws is to connect cold-formed sheet steel material.

1.2 These standard test methods describe mechanical tests for determining the following properties:

	Section
Tensile Strength	3.3
Single Shear Strength	3.4

1.3 These standards do not intend to address all of the safety concerns, if any, associated with their use. It is the responsibility of the user of these standards to establish appropriate safety and health practices, and determine the applicability of regulatory limitations prior to use.

2. Referenced Documents

2.1 ASTM Standards:

A370 - Standard Test Methods and Definitions for Mechanical Testing of Steel Products

E4 - Standard Practices for Force Verification of Testing Machines

F606 - Standard Test Methods for Determining the Mechanical Properties of Externally and Internally Threaded Fasteners, Washers, and Rivets

2.2 AISI Documents:

North American Specification for the Design of Cold-Formed Steel Structural Members, 2001 Edition

AISI TS-5-02 - Test Methods for Mechanically Fastened Cold-Formed Steel Connections, *Cold-Formed Steel Design Manual*, 2002 Edition

3. Test Methods

A test series shall be conducted for each screw material grade, head type, thread series and nominal diameter.

3.1 *Tensile Tests* – This test is intended to determine the ability of a screw to withstand a load when applied along the axis of the screw.

3.2 *Single Shear Test* - This test is intended to determine the ability of a screw to withstand a load applied transversely to the axis of the screw.

3.3 Tensile Tests:

3.3.1 The screw shall be tested in a holder with the load axially applied between the head and a suitable fixture, which shall have sufficient thread engagement to develop the full strength of the screw. A sample test setup is shown in Figure 1. (Note: Threads may be clamped directly by jaws of testing machine if screw shank is not crushed in so doing.)

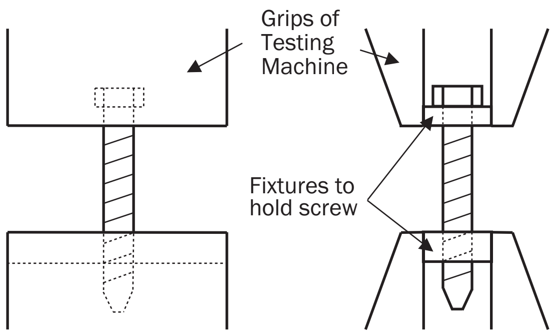


Figure 1 - Standard Tensile Test

3.3.2 The speed of testing, as determined by the rate of separation of the testing machine heads, shall be limited to the greater of 0.1 in. (2.5 mm) per minute or the separation rate caused by a loading rate of 500 pounds (2 kN) per minute.

3.3.3 The maximum load applied to the specimen, coincident with or prior to screw failure, shall be recorded as the tensile strength of the screw.

3.4 Single Shear Test

3.4.1 The specimen shall be tested using steel plates or shapes of sufficient thickness to preclude bearing failure and ensure failure through the fully-threaded section. The shear plates or shapes shall create a single-lap joint connected with one or two fasteners. If two fasteners are used, the total shear strength of the connection shall be divided by two to determine the shear strength for one screw. Suggested geometrical proportions of the test specimen are as given in Table 1, with reference to Figures 2 and 3. The test fixture shall provide for central loading across the lap joint. When the machine grips are adjustable, or when the thickness of either plate is less than 1/16 in. (2 mm), packing shims are not required for central loading.

Table 1 - Geometrical Proportions of Specimen

Screw Diameter, d in. (mm)	w in. (mm)	L in. (mm)	e in. (mm)	p in. (mm)
≤ 0.250 (6.5)	2 (50)	Min. 10 (250)	1 (25)	2 (50)
> 0.250 (6.5)	8d	Min. 10 (250)	3d > 1 (25)	3d > 2 (50)

3.4.2 The test specimen may be assembled in a shear fixture or threaded into two flat sheets. The test specimen shall be mounted in a tensile-testing machine capable of applying load at a controllable rate. The grips shall be self-aligning and care shall be taken when mounting the specimen to assure that the load will be transmitted in a straight line transversely through the test screw(s). Load shall be applied and continued until failure of the screw(s). Speed of testing, as determined by the rate of separation of the testing machine heads, shall be limited to the greater of 0.1 in. (2.5 mm) per minute or the separation rate caused by a loading rate of 500 pounds (2 kN) per minute.

3.4.3 The maximum load applied to the specimen, coincident with or prior to screw failure, shall be recorded as the shear strength of the screw.

4. Report

4.1 The objectives and purposes of the test series shall be stated at the outset of the report so that the necessary test results such as the maximum load per fastener and the mode of failure are identified.

4.2 The type of tests, the testing organization, the supervising engineer, and the dates on which the tests were conducted shall be included in the documentation.

4.3 The test specimen shall be fully documented, including:

(a) the measured dimensions and identification data of each specimen:

- thread O.D.
- thread I.D.
- threads per unit length
- head dimensions
- screw length
- manufacturer
- designation or type
- unthreaded length or imperfect threads below head
- grade of material
- drill-point diameter and length of flutes for self-drilling screws
- any other distinguishing characteristics

(b) the details of fastener installation including pre-drilling, diameter of the pilot drill if used, tightening torque, and any unique tools used in the installation,

(c) identification of the washers or washer-head data including diameter, thickness, material, and data on the sealant if present.

4.4 The test set-up shall be fully described including the type of testing machine, the specimen end grips or supports.

4.5 The test procedure shall be fully documented including the rate of loading.

4.6 In accordance with the test objectives stated by the responsible engineer, the report shall include a complete documentation of all applicable test results for each specimen such as the maximum load and the mode of failure. The report shall also include the necessary calculations for the screw design strength and safety factors/resistance factors based on the requirements specified in Section F1 of the *North American Specification for the Design of Cold-Formed Steel Structural Members*.

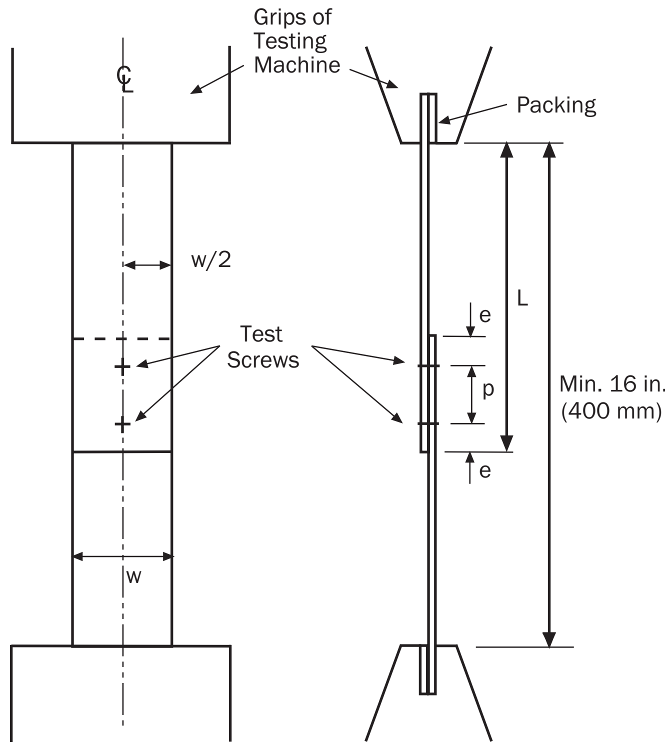


Figure 2 - Standard Lap-Joint Test - 2 Screws

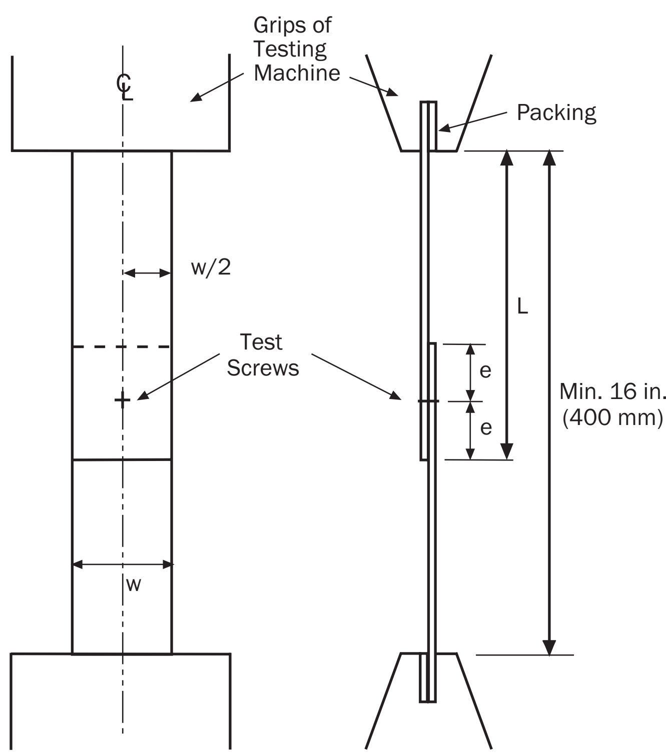


Figure 3 - Standard Lap-Joint Test - 1 Screw

AISI TS-5-02

TEST METHODS FOR MECHANICALLY FASTENED COLD-FORMED STEEL CONNECTIONS

1. Scope

1.1 These performance test methods cover the determination of the strength and deformation of mechanically fastened connections for cold-formed steel building components, and are based extensively on test methods used successfully in the past (References 1-3). Connections in which the fasteners are stressed in shear (loads applied perpendicular to the shank of the fastener) and those in which the fasteners are stressed in tension (loads applied parallel to the shank of the fastener) are included. The objective is to evaluate actual field connections using *standard* test specimens and fixtures.

1.2 For circumstances in which geometric eccentricities exclude the *standard* tests, *alternate* tests are permitted. Test procedures and data recording are generally not affected by the modified specimens and fixtures used for alternate tests. Connections of thin components (such as exterior building sheeting) to relatively thick components (such as structural frame supports) are considered as well as connections between thin components.

1.3 The mechanical fasteners considered include rivets, screws, bolts, and driven fasteners. Adhesively bonded connections and welded connections are excluded.

1.4 The mode of failure incurred during a test is critical and must be reported. Certain failure modes may prohibit the use of a connection regardless of its strength.

1.5 The subject test methods are concerned with the strength characteristics of connections in which the fasteners do not fail. It is intended that sufficient fastener strength will be provided to precipitate failure in the connected members.

2. Applicable Documents

2.1 ASTM Standards

A370 - Standard Test Methods and Definitions for Mechanical Testing of Steel Products
E4 - Standard Practices for Force Verification of Testing Machines

2.2 *North American Specification for the Design of Cold-Formed Steel Structural Members*, 2001 Edition

2.3 ANSI B18.6.4 - ASME Standard for Thread-Forming and Thread-Cutting Screws

3. Terminology

3.1 ASTM Definitions Standards

E6 - Standard Terminology Relating to Methods of Mechanical Testing
IEEE/ASTM SI 10 - American National Standard for Use of the International System of Units (SI): The Modern Metric System

3.2 Description of terms specific to this standard

Maximum load per connection - the maximum load recorded during a test.

Maximum load per fastener - the maximum connection load divided by the number of fasteners in the connection.

Load-deformation curve per fastener - the load-deformation curve for the test connection with the load values divided by the number of fasteners.

Characteristic connection strength per fastener - a statistically adjusted value for the mean maximum load per fastener measured for the test unit.

3.3 Symbols

A = elongation in a tension test
a = the width of the end supports in the alternate tension tests
a_s = shear deformation or slip
a_t = deformation in a tension test
b = the width of the troughs or flats in a profiled sheet
c = a statistical coefficient
u_s = shear flexibility per fastener
u_t = tensile (uplift) flexibility per fastener
d = the nominal diameter of a fastener
e_1 = the end distance of a fastener in a standard shear test specimen
e_2 = the fastener edge distance in an alternate shear test specimen
h = the height of the stiffening ribs in a profiled sheet
K = an empirically derived coefficient
L_s = the free strap length in a standard shear test specimen
L_a = the test span for an alternate tension test specimen
l_g = the extensometer gage length in a standard shear test
M_u = the maximum bending moment per stiffening rib
n = the number of valid tests in a test unit
P = the estimated maximum connection load per fastener
P_K = the characteristic connection strength per fastener
P_m = the mean connection strength per fastener
P_u = the maximum connection strength per fastener
F_y = the yield point
F_u = the tensile strength
s = standard deviation
p = the fastener spacing or pitch
t = the base metal sheet thickness
w = the width of the shear test specimen

4. Significance

4.1 These test methods provide the requirements for evaluating mechanically fastened connections for cold-formed steel members in buildings designed according to the *Specification* and related building codes. The continued introduction of new and different mechanical fasteners increases the need for standardized tests for cold-formed steel connections. Standard test specimens, fixtures, and procedures facilitate the exchange of information vital to understanding the behavior of a variety of fasteners with diverse properties by providing a basis for comparing strength and deformation measurements.

4.2 The test methods are applicable to mechanical fasteners currently used by the industry.

4.3 Throughout the test methods defined herein it is intended that the test specimens be fabricated as nearly as practicable to the actual generic field connection(s) or to a specific prototype with regards to material(s), assembly technique(s), and fastener installation(s).

5. Apparatus

5.1 The test methods are generally suitable for hydraulic or screw operated testing machines.

5.2 Any testing machine used shall comply with the requirements of ASTM E4, wherein the rate of loading can be controlled, constant loads maintained, and the applied load can be measured accurately to within $\pm$ 2 percent.

5.3 The test specimen support fixtures and the testing machine grips shall have the capability of maintaining a constant loading direction throughout the test.

5.4 The devices used to measure deformation shall provide an accuracy of $\pm$ 0.001 in. (0.02 mm) for shear tests and $\pm$ 0.002 in. (0.05 mm) for tension tests.

5.5 The devices used to measure the dimensions of the test specimens shall be accurate to within $\pm$ 0.0005 in. (0.01 mm) for sheet base metal thickness, $\pm$ 0.05 in. (1 mm) for sheet profile dimensions, and $\pm$ 0.005 in. (0.1 mm) for fastener dimensions.

6. Test Unit

6.1 For connections that include a single cross-section, a single nominal sheet thickness, and a single nominal tensile strength for the critical connection component a minimum of three (3) specimens shall be tested. Three (3) additional tests are required if any single test yields an maximum load that differs from the mean maximum load by more than 10 percent.

6.2 For evaluations that include one connection cross-section with several nominal values for the thickness or tensile strength of the critical connection component, at least three (3) tests shall be required for each (thickness and/or strength) value. The differences necessary to define distinct nominal values for sheet thickness and tensile strength shall be at least 0.005 in. (approximately 0.1 mm) and 20 ksi (approximately 30 N/mm^2), respectively.

6.3 Two sheet-type tension test coupons shall be tested for each thickness and strength of steel sheet used in the fabrication of the test specimens. The tension test coupon shall be taken from a flat undamaged area of the sheet component. When the sheet component is corrugated or profiled, the tension test coupon shall be oriented parallel to the corrugations or ribs. The sheet tension tests shall be conducted in accordance with ASTM A370 and the yield point, tensile strength, and percent elongation at fracture shall be measured. The average of the two respective test values shall be regarded as the yield point, tensile strength, and elongation.

7. Test Specimens and Fixtures

7.1 General

7.1.1 *Standard* shear and tension tests shall be used whenever possible. *Alternate* shear and tension tests shall only be used when the standard specimens are unsuitable for evaluating the connection properties under consideration. *Standard* or *alternate* test specimens can also be used to conduct shear tests on specimens with a single fastener. When such tests are conducted to study the influence of end distance or edge distance, the dimensions given below may be changed as required.

7.1.2 The dimensions of the test specimen components shall be measured to an accuracy of $\pm$ 0.0005 in. (0.01 mm) for sheet base metal thicknesses, $\pm$ 0.05 in. (1 mm) for sheet profile dimensions, and $\pm$ 0.005 in. (0.1 mm) for fastener dimensions.

7.1.3 Fasteners shall be placed within $\pm$ 0.05 in. (1 mm) of their specified location and affixed according to the manufacturers recommendations or the actual site practice. Special note shall be taken of the following, if applicable: (1) the diameter of predrilled holes, (2) the torque and depth control for threaded fasteners, and (3) the installation tools and cartridge types used for fired pins or impact driven fasteners.

7.2 Lap-Joint Shear Tests

7.2.1 Shear test deformations shall be obtained from extensometer readings across the lap joint to an accuracy as specified in Section 5.4.

7.2.2 The *standard* shear-test specimen (Figure 1) is a single-lap joint using two flat straps connected with 2 fasteners. Recommended geometrical proportions of the specimen shall be as given in Table 1. For tests investigating edge failure, or other special conditions, dimensions e_1, p, and W may be modified.

7.2.2.1 The test fixture (Figure 1) for the standard shear test provides for central loading across the lap joint. When the machine grips are adjustable or when the thickness of either strap is less than 1/16 in. (approximately 2 mm), packing shims are not required for central loading.

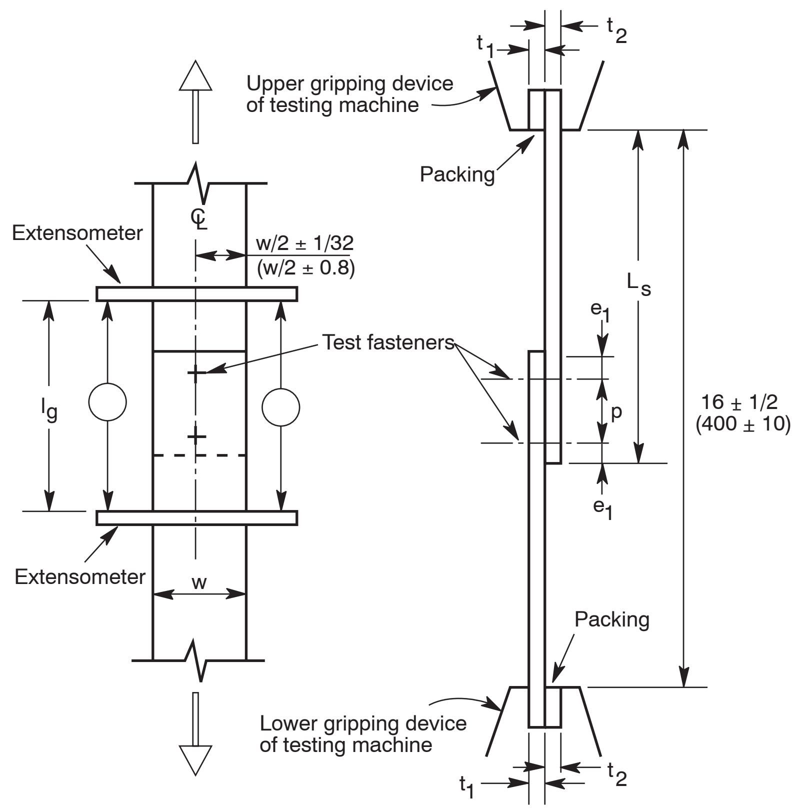


Figure 1 - Standard Lap-Joint Shear Test, units - in. (mm)

7.2.2.2 The extensometer gage length, l_g in Table 1, is sufficiently short to eliminate the influence of stretch in the specimen straps on the extensometer readings.

7.2.3 The *alternate-1* shear-test specimen (Figure 2a) is a simulated diaphragm-action connection with central loading applied in the plane in which the overlapping elements are joined by the fasteners. This specimen shall be used to determine the strength and flexibility of the diaphragm connection. The geometric proportions of the specimen are listed in Table 2, unless the actual proportions of a diaphragm are to be tested.

7.2.4 The *alternate-2* shear-test specimen (Figure 2b) is a simulated diaphragm-action connection with central loading applied in a plane different from that in which the overlapping elements are joined by the fasteners. This specimen shall be used to determine the strength and flexibility of the diaphragm connection, especially for crest-connected overlapping elements of diaphragms. The geometric proportions of the specimen are listed in Table 2, unless the actual proportions of a diaphragm are to be tested. Guides shall be machined and polished, then greased.

Table 1 Recommended Geometrical Proportions

Fastener Diameter d, in. (mm)	Specimen Dimensions, in, (mm) w	L_s	e_1	p	l_g
≤ 1/4 (6.5)	2-3/8 (60)	10-1/4 (260)	1-3/16 (30)	2-3/8 (60)	5-7/8 (150)
> 1/4 (6.5)	10d (10d)	8 + 10d (200 + 10d)	5d (5d)	10d (10d)	1-3/16 + 20d (30 + 20d)
Tolerance	+ 1/16 (+ 2)	+ 3/16 (+ 5)	+ 1/32 (+ 1)	+ 1/32 (+ 1)	+ 3/16 (+ 5)

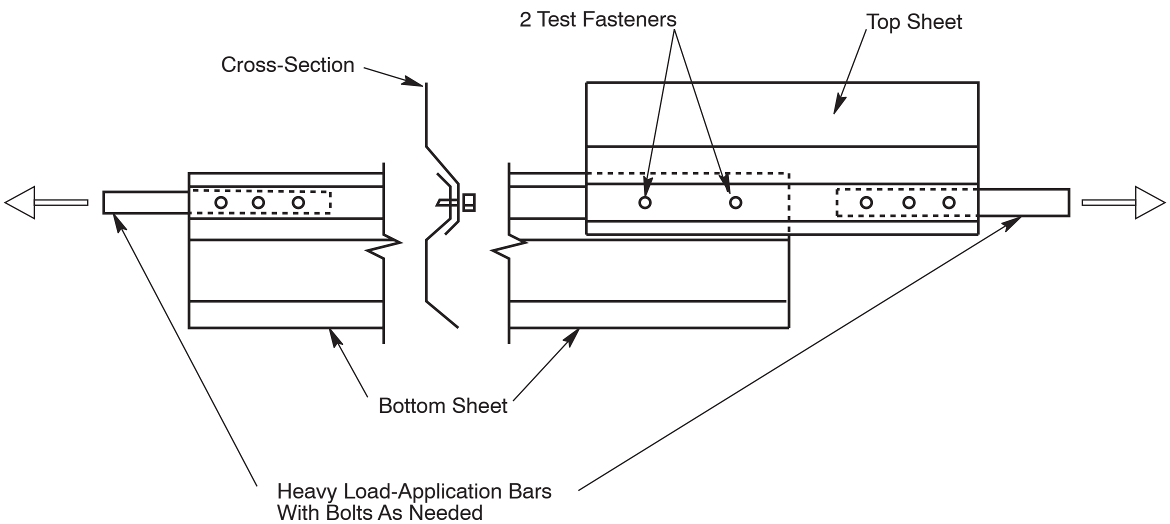


Figure 2a - Alternate-1 Lap-Joint Shear Test

7.2.4.1 The *alternate-2* test fixture (Figure 2b) is designed such that the base plate is securely attached to a level foundation beam (or slab) and loaded in a horizontal plane using a hydraulic ram or an equivalent system. A generous amount of lubricating material shall be used to reduce the friction between guide and grips.

7.3 Tension Tests

7.3.1 Loading that induces tension in the shanks of fasteners that connect relatively thin sheeting to structural support members can cause several modes of failure (also defined as Type V as discussed in Section 8.2.3 and as shown in Figure 10e). If the sheeting is pulled over the fastener heads (i.e. the fastener heads are pulled through the sheeting) the failure mode is described as pull-over or pull-through. If the fasteners are pulled free from the structural support the failure mode is described as fastener pull-out. This failure mode typically occurs when several thicknesses of sheeting are attached to a support member by a common fastener. If the sheeting undergoes very large permanent deformation prior to failure at the fasteners the failure mode is described as gross distortion.

7.3.2 The *standard* tension-test specimen for pull-over strength (Figure 3a) or for pull-out strength (Figure 3b) is specially formed from flat sheet stock used to produce the actual sheeting product under consideration. The specimen geometry serves as a generic model for

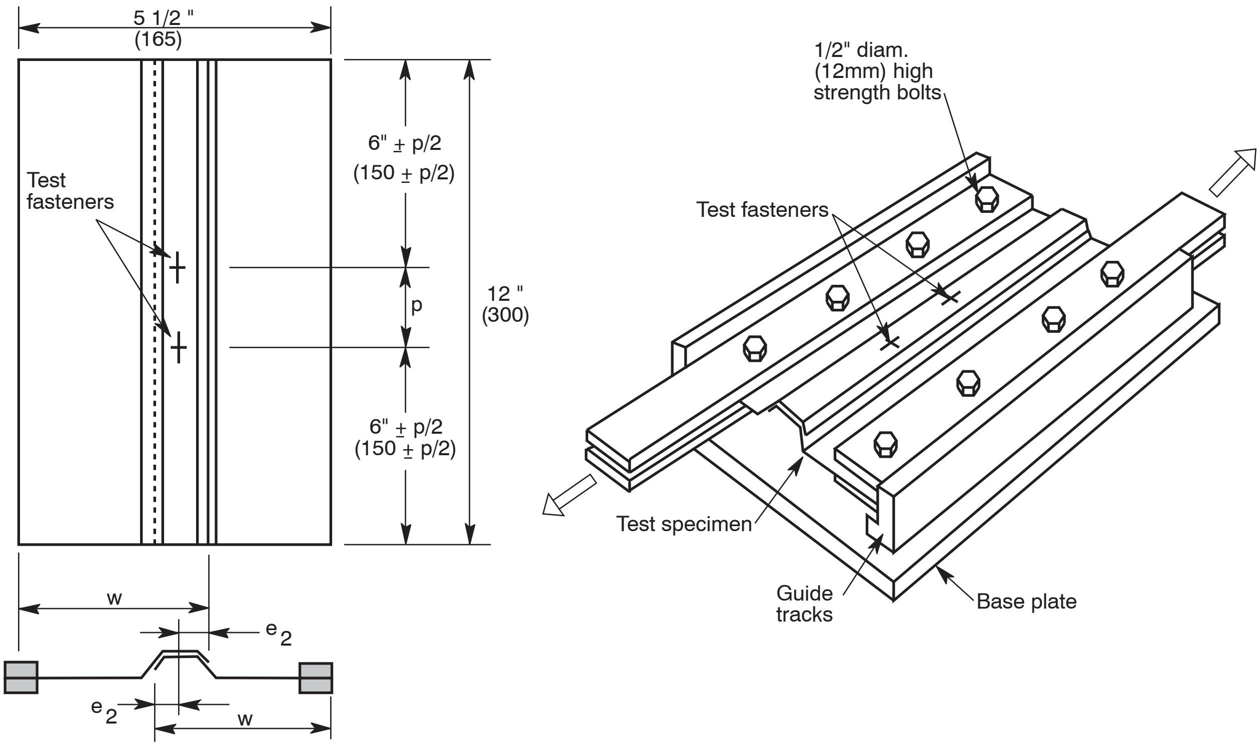


Figure 2b Alternate-2 Lap-Joint Shear Test

Table 2 - Recommended Geometrical Proportions

Fastener Diameter d, in. (mm)	Specimen Dimensions, in, (mm) w	e_2	p
≤ 1/4 (6.5)	3-5/8 (92.5)	3/8 (10)	2-3/8 (60)
> 1/4 (6.5)	3-1/4 + e_2 (82.5 + e_2)	1.5d (1.5d)	10d (10d)

profiled sheeting. The connection support member for the pull-over test shall be sufficiently thick to resist fastener pull-out with a minimal amount of symmetrical deformation from the test loads. Table 3 lists suggested support members for standard pull-over tests. For pull-out tests the actual total sheeting thickness connected to the support can be simulated by adding pieces of packing (Figure 3b).

7.3.2.1 The *standard* tension-test fixture (Figure 4) is designed for easy clamping of the test specimen and central loading along the axis of the fastener.

7.3.2.2 The influence of asymmetrical deformation of the sheeting support members on pull-over strength can be tested with the generic standard test specimen by using the modified standard tension-test fixture (Figure 5). The dimensions of the asymmetrical support member shown will simulate most cold-formed C sections, Z sections, and angles. Where the supporting member rotates, such as a C- or Z-shaped purlins or

Table 3 Recommended Support Members for Standard Pull-Over Tests

Thickness of Support Material in. (mm)	t ≥ 1/4 (6.5)	t < 1/4 (6.5)	
Type of support to be used in practice	All Types	Hot-Rolled Sections	Cold-Formed Sections, Hollow Sections, and Sheeting
Standardized Support to be used in the tests	Hot-Rolled Flat Steel: 2-3/8 x t (60 x t)	Hot-Rolled Angle: 1-5/8 x t (40 x t)	Cold-Formed Channel: 2-3/4 x 1-3/16 x t (70 x 30 x t)

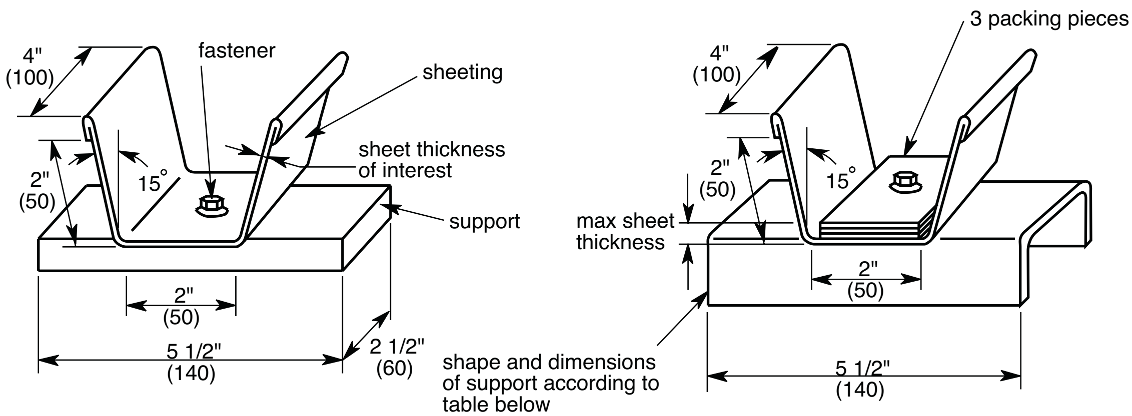


Figure 3a - Standard Pull-Over Test Specimen **Figure 3b - Standard Pull-Out Test Specimen**

girts between lateral supports in metal buildings, the prying tension shall be considered (Reference 4 and 9).

7.3.3 A specific *standard* tension-test specimen for pull-over strength of a profiled sheet (Figure 6), with width and length dimensions of 8-in. by 8-in. (200 mm by 200 mm), is cut from the sheeting under consideration and predrilled for four 1/2-in.-diameter (12 mm) bolts located 6 in. (150 mm) apart with respect to the perimeter. The specimen is cut and drilled so that the location of the test fastener on the sheet profile corresponds to the location adopted in practice and is also centrally located with respect to the four bolt holes. In cases where the prototype has flexural tensile stresses in the region of the fastener, this will augment the tensile stresses caused by the fastener pull-over test. This effect shall be included by using the alternate test method of Section 7.3.4.

7.3.3.1 The test fixture for a specific *standard* tension test (Figure 6) consists of a stiff base plate assembly with four tapped holes spaced 8 in. (200 mm) apart to match the holes in the test specimen. The test specimen is clamped to the base with four 1/2-in.-diameter (12 mm) bolts with 1-1/8-in.-diameter (29 mm) by 3/32-in.-thick (2.5 mm) washers under the bolt heads. Central loading is provided by a loading arm that is pin connected to the symmetric loading channel to which the sheeting is fastened. The loading channel can be fabricated from the member used in the actual connection or specially fabricated according to the test objectives.

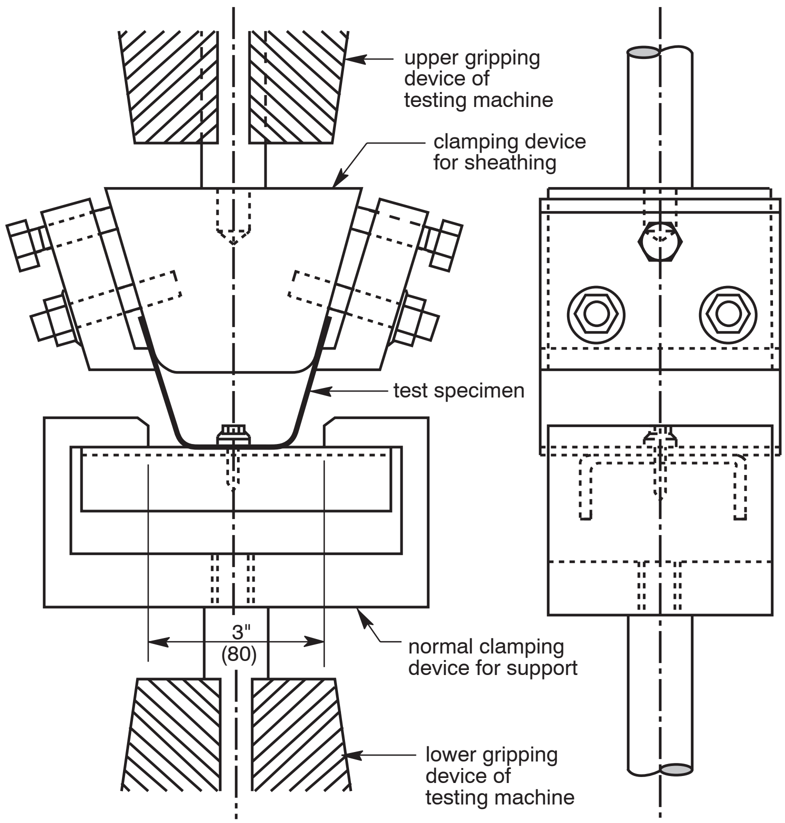


Figure 4 - Standard Tension-Test Fixture

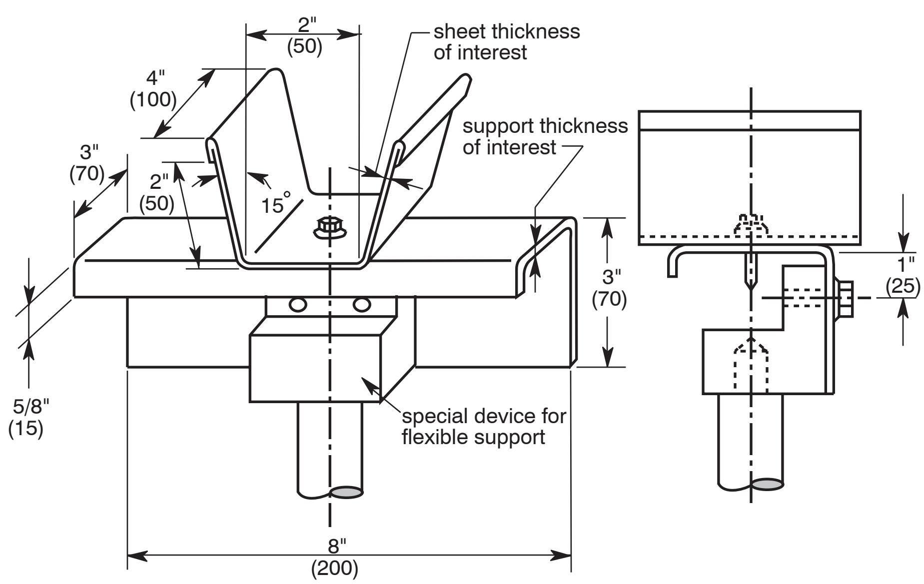


Figure 5 - Modified Standard Tension-Test Fixture for Influence of Flexible Support Members

7.3.3.2 A modification to the test fixture (Figure 6) is required to resist excessive deformation in thin flexible profiled sheeting with relatively wide flat-widths. Excessive deformation is prevented by stiffening angles attached to the base plate assembly (Figure 7). The angles are spaced a distance apart equal to the flat-width of test sheeting.

7.3.4 The *alternate* tension-test specimen for sheeting with trapezoidal cross section (Figures 8a and 8b) may be utilized whenever detailed information about the sheet deformation is required, or where the prototype has flexural tension at the fastener. The specimen consists of a segment of the test sheeting with a centrally located test fastener that connects to a loading channel similar to that used for the standard tension test. The length of the speci-

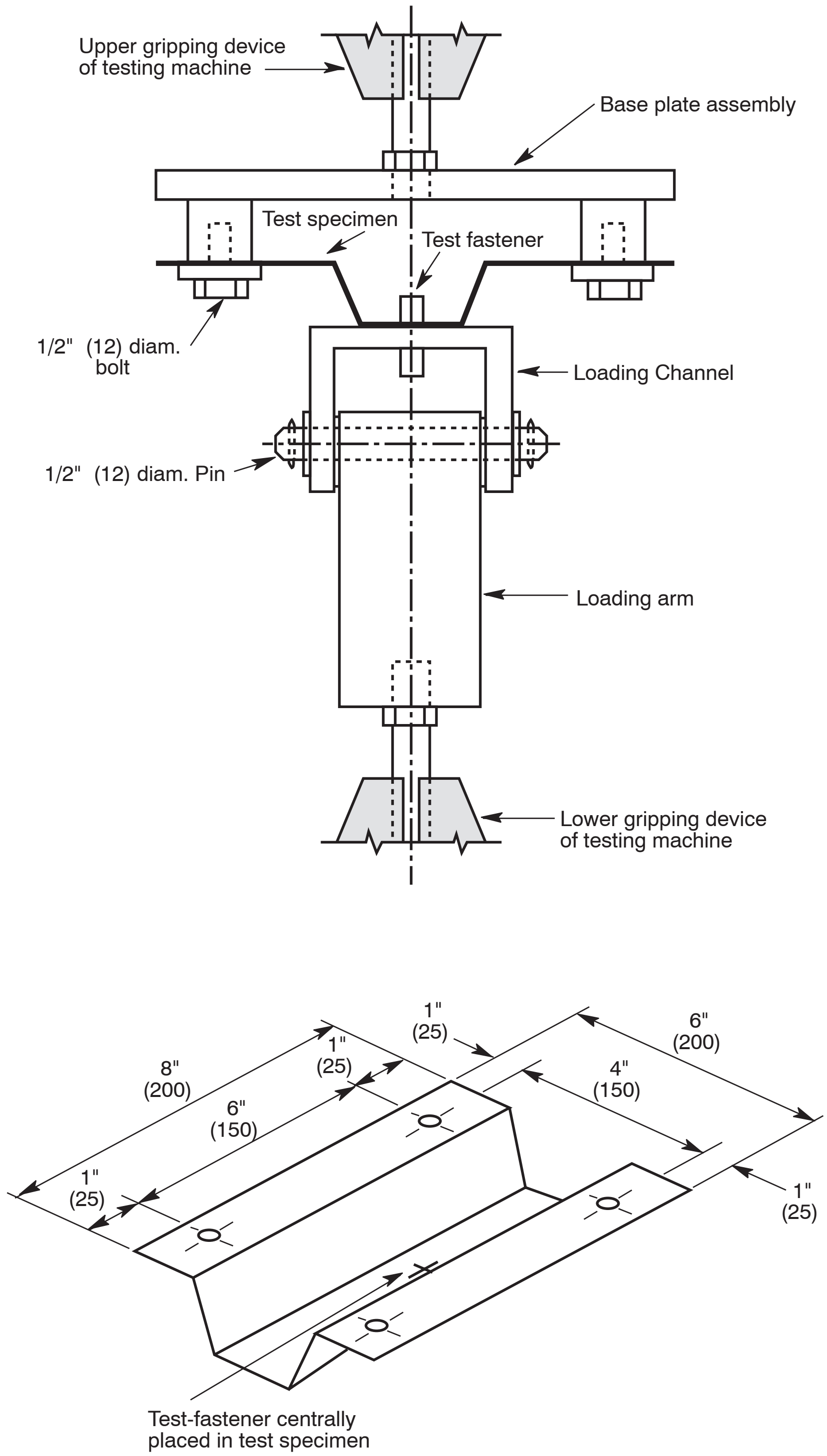


Figure 6 - Specific Standard Tension-Test Specimen and Fixture

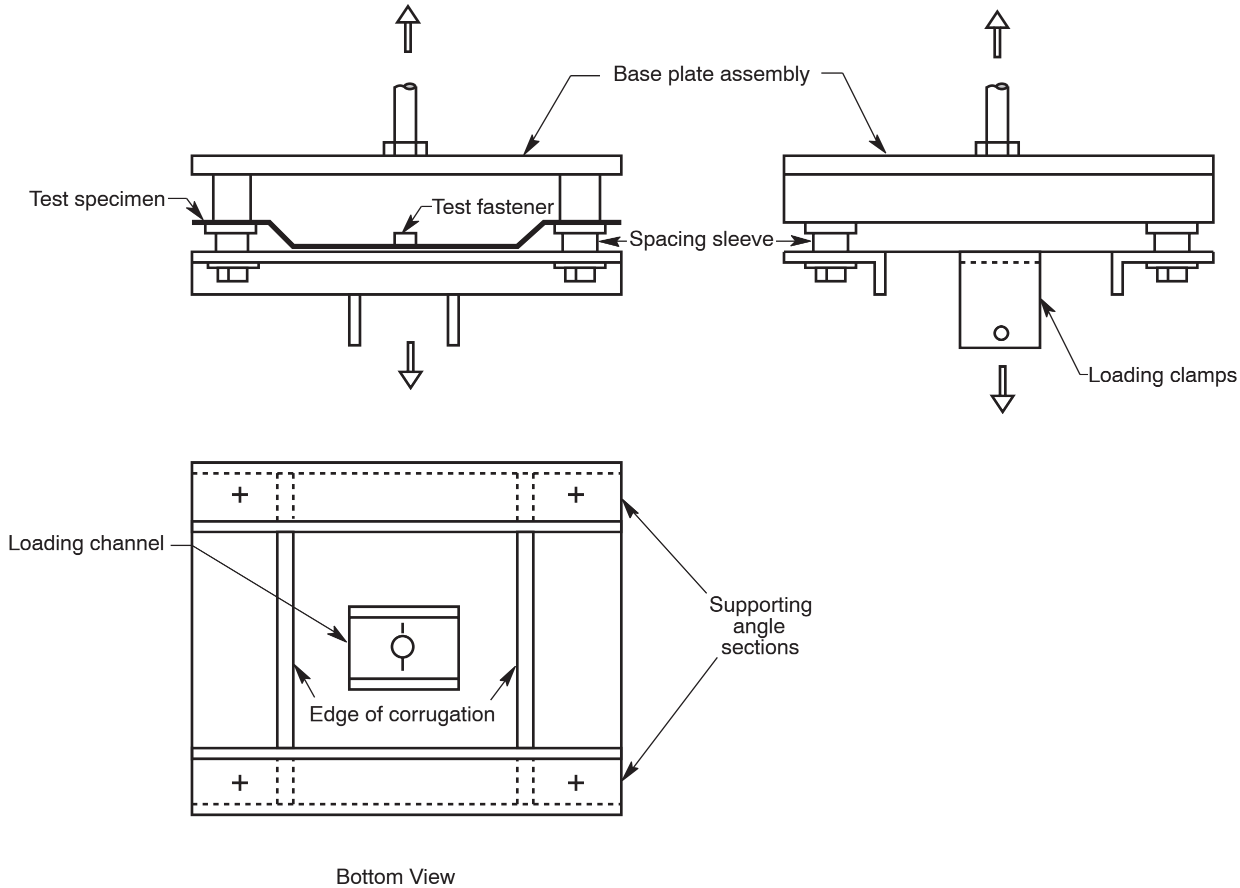


Figure 7 - Modified Specific Standard Tension-Test Specimen and Fixture With Location of Support Angles for Flexible Sheeting

men, L, shall be such that the flexural tension is at design value, and L is at least 12 times the flat-width of the corrugations, b:

$L \geq 12\ b$

The width of the specimen shall be 2 corrugations for trough fastening and 3 corrugations for crest fastening. These specimen width values are the minimum widths necessary to obtain symmetrical deformations from a centrally located fastener. Transverse stiffening straps approximately 3/4-in. (20 mm) wide by 1/16-in. (2 mm) thick shall be fastened across the specimen width to insure that the sheet profile is maintained during loading. The straps shall be located approximately $L_a/4$ from the midlength of the specimen. (The determination of L_a is dependent upon the bending strength of the sheeting and is discussed below.)

7.3.4.1 The test fixture (Figure 8c) provides for simple end supports of the test specimen and for tensile loading of a single fastener at midspan using a loading channel. The test specimen and the test fixture shall provide the following proportions:

1) a span L_a equal to or greater than 6b, and less than $3M_u/P$, to prevent premature bending failure of the test sheet,
2) a specimen width, a, less than $L_a/6$, and
3) transverse stiffening straps a distance $L_a/4$ from the midspan.

7.3.5 Where sheeting is fastened to the support by a clip (Figure 8d), or by a fastener near the edge of the underneath sheet so as to hide the fastener head (Figure 8e), the test specimen

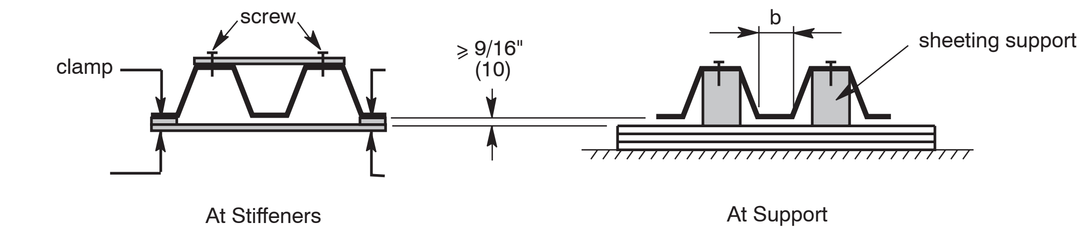


Figure 8a Cross Section (Trough Fastening)

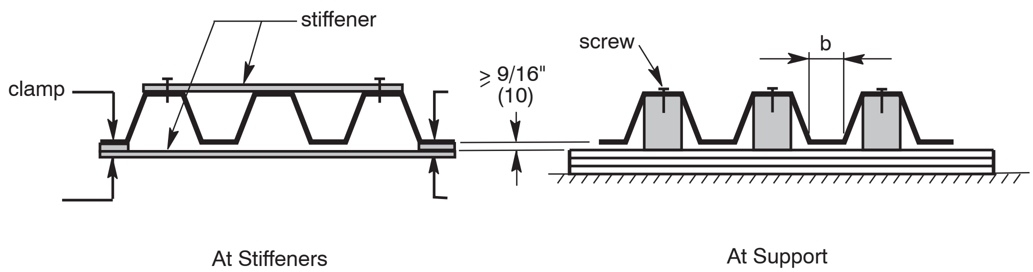


Figure 8b Cross Section (Crest Fastening)

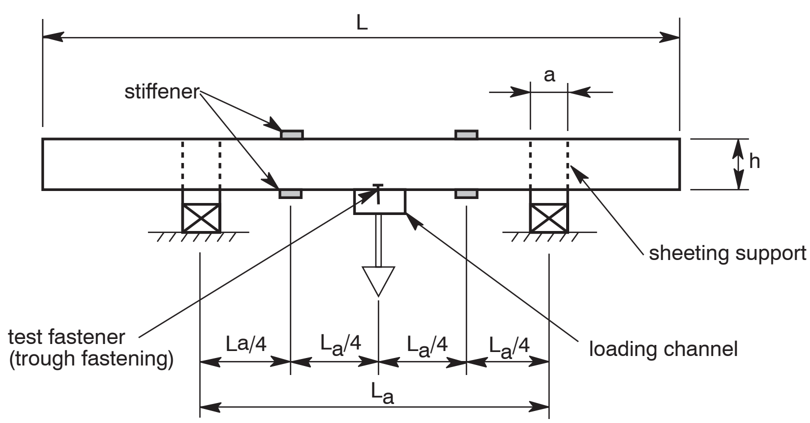


Figure 8c Test Fixture (Elevation)

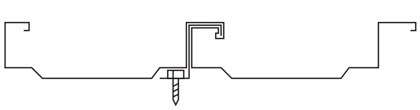


Figure 8d Clip Fastening

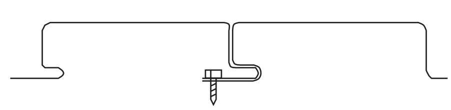


Figure 8e Hidden Fastener

Figure 8 - Alternate Tension-Test Specimen and Fixture

shall be such that the panel lap is at the center of the width. All other specimen dimensions shall meet the requirements of Section 7.3.4. The test fixture requirements at stiffeners and sheeting supports are the same as in Section 7.3.4.1.

7.3.6 A large-scale tension test capable of full-scale prototype testing of sheeting connections may also be conducted (Figure 9, schematic). The test panel shall contain two beams

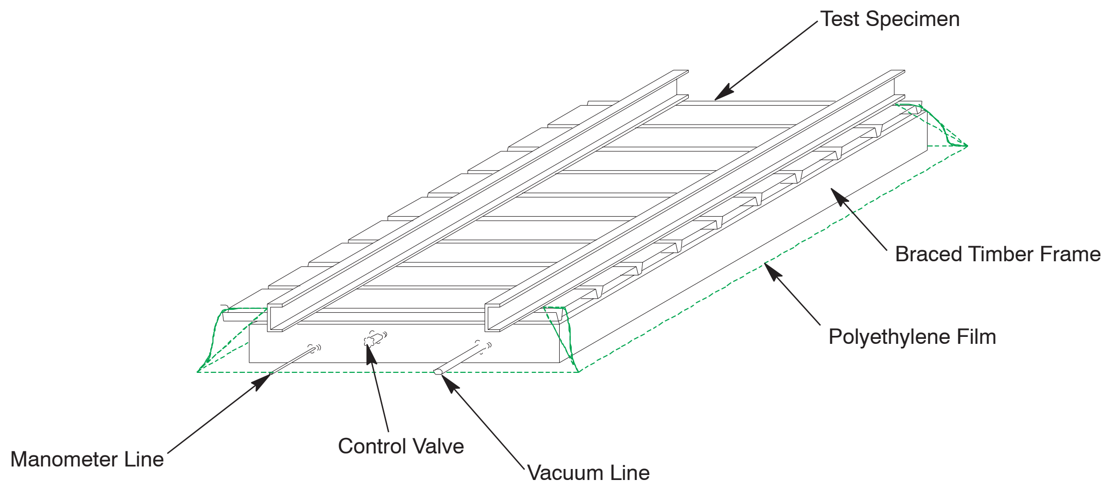


Figure 9 - Large Scale Tension Test

(purlins or girts, or similar). The test panel shall be uniformly loaded over its surface by regulating the air pressure inside the chamber below the panel. Failure can occur in the panel connections to the beams. Only the nominal tension forces, which do not include the prying forces caused by the rotations of the C- and Z-shaped purlins or girts, can be computed from the known value of the load acting on the panel surface at failure. A description of prying forces is given in Reference 9. The beam span and the purlin spacing shall match those of the prototype in order within $\pm$20 percent to properly simulate the effects of prying action. (The arrangement illustrated in Figure 9 is fully described in References 2, 4, 5, and 6.)

8. Test Procedures

8.1 General

8.1.1 The speed of testing shall not be greater than that at which the relative displacement readings can be accurately taken. End grips of the testing machine shall be in alignment with the axis of the specimen test fixtures during loading.

8.1.2 Loading shall be applied in load increments of approximately one fifth of the estimated maximum load. When the maximum load is approached, smaller increments shall be used. Each load increment shall be maintained for at least one minute (or until it has stabilized) before proceeding with the next increment. Loading shall continue until the load can not be maintained, or until one or more fasteners have failed.

8.2 Shear Tests

8.2.1 The speed of testing as determined by the rate of separation of the testing-machine heads shall be limited to the greater of 0.05 in. (approximately 1 mm) per minute, or the rate caused by a loading rate of 500 pounds (2 kN) per minute.

8.2.2 Deformation or slip measurements shall be recorded at each loading increment and at the maximum load.

8.2.3 The failure mode(s) shall be identified and recorded according to the following classifications:

Type I - End failure, or longitudinal shearing of the sheet along two approximately parallel lines (Figure 10a).

Type II - Bearing, tearing, or piling up of the thinner, or of both (equal thickness), sheet material in front of the fastener (Figure 10b).

Type III - Tension failure of one sheet in the net section (Figure 10c)

Type IV - Shearing of the fastener (Figure 10d)

Type V - Tilting and pull-out of fastener including sheet pull-over (Figure 10e).

Figure 10 shows only 1 fastener but it is also applicable for 2 or more fasteners.

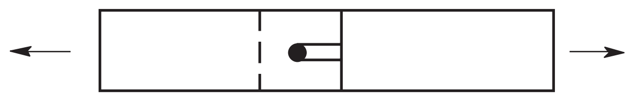


Figure 10a Type I Failure
(End Shearing Failure)

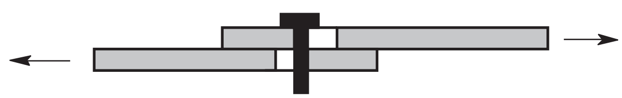


Figure 10b Type II Failure
(Bearing, Tearing, Piling Up)

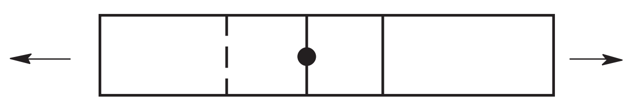


Figure 10c Type III Failure
(Tension Failure in Net Section)

Figure 10d Type IV Failure
(Shearing of Fastener)

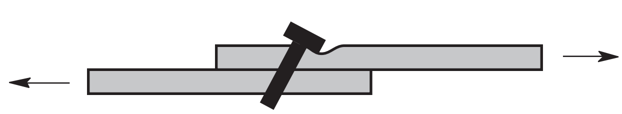


Figure 10e Type V Failure
(Tilting, Pull-Over, and Pull-Out of Fastener)

Figure 10 - Lap-Joint Shear Test Failure Modes

8.3 Tension Tests

8.3.1 The speed of testing as determined by the rate of separation of the testing machine heads shall be limited to the greater of 0.1 in. (2.5 mm) per minute or the rate caused by a loading rate of 500 pounds (approximately 2 kN) per minute.

8.3.2 When deformation measurements are necessary they shall be recorded at each loading increment and at the maximum load. If permanent-set measurements are necessary a small preload (approximately 10 percent of the anticipated maximum load) can be used. After each loading increment, the load is reduced to the preload and the permanent set is recorded.

9. Calculations

Calculations to evaluate the test results and to determine the characteristic connection strength shall be made in accordance with the procedures described elsewhere (References 7 and/or 8).

10. Report

10.1 The objectives and purposes of the test series shall be stated at the outset of the report so that the necessary test results such as the maximum load per fastener, the flexibility of the connection, and the mode of failure are identified.

10.2 The type of tests, the testing organization, and the dates on which the tests were conducted shall be included in the documentation.

10.3 The test unit shall be fully documented, including:

1) the measured dimensions of each specimen,
2) identification data for the fasteners and accessories such as washers,*
3) the details of fastener application including predrilling, tightening torque, and any unique tools used in the operation, and
4) the results of the sheet-type tension tests including yield point, tensile strength, and elongation to failure. The location and orientation of the sheet-tension coupons shall also be given.

For pull-out tests additional data shall indicate the drill-point diameter and length of flutes if self-drilling screws are used. Otherwise, the diameter of the pilot drill used shall be stated. Only new and sharp drill bits shall be used because of the tendency of resharpened, chipped and dull drill bits to make oversize holes. Washers or washer-head data shall include diameter, thickness, material, and if present the sealant data.

10.4 The test set-up shall be fully described including the testing machine, the specimen end grips or supports and the devices used to measure deformation.

10.5 The test procedure shall be fully documented including the rate of loading and the load increments.

10.6 In accordance with the test objectives stated by the responsible engineer, the report shall include a complete documentation of all applicable test results for each specimen such as the load-deformation curve, the maximum load, and the mode of failure. The report shall also in-

* Fastener data shall include the name of the manufacturer, designation or type, dimensions, number of threads, including unthreaded length or imperfect threads below head, and the major and minor diameters in the threaded region.

clude the necessary calculations for the characteristic connection strength per fastener and the connection flexibility for the test unit. Calculations for reduction of the test strength (corresponding to the specified minimum yield point of the sheeting product) shall also be included when applicable.

REFERENCES

1. ECCS Committee 17 Report 21 "European Recommendations for the Testing of Connections in Profiled Sheeting and Other Light Gauge Steel Components", May 1978
2. Fraczek, J. "Mechanical Connections in Cold-Formed Steel; Comprehensive Test Procedures and Evaluation Methods", Research Report No.359, Dept. of Structural Engineering, Cornell University, Ithaca, NY., May 1976
3. Fraczek, J. "Development of Comprehensive Test Procedures for Connections in Cold-Formed Steel and Appropriate Evaluation Methods.", Draft Report, Dept. of Structural Engineering, Cornell University, Ithaca, NY., Oct. 1974
4. LaBoube, R. A., Golovin, M., Montague, D. J., Perry, D. C., and Wilson, L. L., "Behavior of Continuous Span Purlin Systems," *Ninth International Specialty Conference on Cold-Formed Steel Structures*, St. Louis, MO., November, 1988, University of Missouri-Rolla
5. Pekoz, T., and Soroushian, D., "Behavior of C- and Z-Purlins Under Wind Uplift," *Sixth International Specialty Conference on Cold-Formed Steel Structures*, St. Louis, MO., November, 1982, University of Missouri-Rolla
6. LaBoube, R. A., "Laterally Unsupported Purlins Subjected to Uplift," Metal Building Manufacturers Association, December, 1983
7. Pekoz, T., and Hall, B., "Probabilistic Evaluation of Test Results," *Ninth International Specialty Conference on Cold-Formed Steel Structures*, St. Louis, MO., November, 1988, University of Missouri-Rolla
8. *Cold-Formed Steel Design Manual*, American Iron and of Steel Institute , 2002 Edition
9. Haussler, R. W., "Theory of Cold-Formed Steel Purlin/Girt Flexure," *Ninth International Specialty Conference On Cold-Formed Steel Structures*, St. Louis, Missouri, November, 1988, University of Missouri - Rolla

AISI TS-6-02

STANDARD PROCEDURES FOR PANEL AND ANCHOR STRUCTURAL TESTS

1. Scope

This procedure extends and provides methodology for interpretation of results of tests performed according to ASTM E1592-95.

2. Referenced Documents

2.1 ASTM Standards

E1592-95, Standard Test Method for Structural Performance of Sheet Metal Roof and Siding Systems by Uniform Static Air Pressure Difference

A370-97 Standard Test Methods and Definitions for Mechanical Testing of Steel Products

2.2 AISI Standards

North American Specification for the Design of Cold Formed Steel Structural Members, 2001 Edition

AISI TS-8-02 - Base Test Method for Purlins Supporting a Standing Seam Roof System, AISI *Cold Formed Steel Design Manual*, 2002 Edition

3. Terminology

3.1 Refer to Section 3, ASTM E1592-95.

3.2 Additional or Modified Terminology

3.2.1 *clip* - a single or multiple element device that frequently attaches to one edge of a panel and is fastened to the secondary structural members with one or more screws.

3.2.2 *field* - the area that is not included in high pressure edge strip conditions. For purposes of the test, a field condition is modeled when the pan distortions are independent of end and edge restraint.

3.2.3 *pan* - the relatively flat portion of a panel between ribs.

3.2.4 *tributary area* - the area directly supported by the structural member between adjacent supports.

3.2.5 *trim* - the sheet metal used in the finish of a building especially around openings, and at the intersection of surfaces such as roof and walls.

3.2.6 *ultimate load* - the difference in static air pressure at which failure of the specimen occurs, expressed in load per unit area, and is further defined as the point where the panel system cannot sustain additional loading.

3.2.7 *unlatching failure* - disengagement of a panel seam or anchor that occurs in an unloaded assembly due to permanent set or distortion that occurred when the assembly was loaded. This permanent set is not always detectable from readings taken normal to the panel. It is deemed to be a serviceability failure until a strength failure occurs, as defined in 3.2.6, *ultimate load*.

4. Summary of the Test Method

4.1 Refer to the requirements of Section 4, ASTM E1592-95.

5. Significance and End Use

5.1 Refer to the requirements of Section 5, ASTM E1592-95.

5.2 The end use of the procedure is the determination of allowable load carrying capacity of panels and/or their anchors under gravity or suction loading for use in a design procedure.

6. Test Apparatus

6.1 Refer to the requirements of Section 6, ASTM E1592-95.

7. Safety Precautions

7.1 Refer to the requirements of Section 7, ASTM E1592-95.

8. Test Specimens

8.1 Refer to the requirements of Section 8, ASTM E1592-95.

8.2 *Specimen Width* - Edge seals shall not contain attachments that restrict deflection of the test panel in the field in any way. No additional structural attachments that would resist deflection of the field of the test panels are permitted.

8.2.1 The test panel ribs shall be installed parallel to the long side of the test chamber.

8.3 Number of Tests

8.3.1 Tests shall use minimum thickness of support members (secondary structures) and maximum panel span. If results are to be interpolated for other values, the other extremes must be tested in order to justify an interpolation procedure.

8.3.2 Tests shall be conducted to evaluate the field condition.

9. Calibration

9.1 Refer to the requirements of Section 9, ASTM E1592-95.

10. Procedures

10.1 Refer to the requirements of Section 10, ASTM E1592-95.

11. Test Evaluation

11.1 Safety factors and resistance factors shall be determined in accordance with the procedures in Chapter F and Section C3.1.5 of the *North American Specification for the Design of Cold Formed Steel Structural Members*.

11.2 If a separate test series is performed to evaluate edge conditions and the results exceed the field case by greater than one standard deviation, a separate design allowable is permitted to be established for edge conditions.

11.3 A qualified design professional shall analyze deflections and permanent set data to assure that deflections and permanent set are acceptable at service loads.

12. Test Report

12.1 Refer to the requirements of Section 11, ASTM E1592-95.

12.2 Report the resistance factor and/or the safety factor based on the Section C3.1.5 of the *Specification* for the test results. If the factor of safety is defined, report the allowable uniform design strength of the panel system. If the allowable design strengths of the panel and anchors are determined separately, they shall be reported separately.

12.3 If intermediate values are to be calculated for different spacings of anchors or secondary structures, the basis of the interpolation shall be stated in the report. If the failure modes are different on any two tests, interpolation between these two tests is not permitted.

12.4 The design professional shall include in the report the observation as to the acceptability of deflections and permanent set data at service loads.

COMMENTARY ON THE STANDARD PROCEDURES FOR PANEL AND ANCHOR STRUCTURAL TESTS

1. Scope

The scope of the Procedure is for testing single skin panel systems. The procedure is based on ASTM E1592-95 with specific additions to define the required safety factors for a design procedure. Edge strip detail confirmation is permitted by the test method.

2. Referenced Documents

The previously developed standards, ASTM E1592-95 and the AISI TS-8-02 - Base Test Method have been used in the development of this procedure.

3. Terminology

To promote accuracy and understanding, frequently used terms need mutual understanding. This list includes the terms from ASTM E1592-95 with additions and modifications.

5. Significance and End Use

Currently, there are several organizations that have test procedures to determine product performance, but the procedures are limited to one product configuration and do not have provisions to provide the basis for a complete design procedure covering the evaluation of a safety factor for a range of product configurations. Therefore, this new Standard Procedure was developed.

6. Test Apparatus

The apparatus defined in this section is specific enough to accomplish the purpose, yet broad enough to allow many facilities to perform tests. The size of the specimen is the most important criteria. Whether or not the apparatus consists of two sections with the specimen in between is not a major issue.

Measurement of rib spread has dubious value except when seam disengagement is the failure mechanism. In that case, measurements tend to substantiate the failure mechanism.

7. Safety Precautions

In addition to other precautions, care must be exercised in taking the deflection readings required in this procedure.

8. Test Specimens

The size of a test specimen has been found to be an important element in demonstrating product performance. Minimum sizes are defined, but larger sizes are allowed. It is understood that many products are offered to the market that have insufficient usage to justify a large test program yet proof of performance to some degree is required. The procedure is developed to allow a single test with a corresponding penalty due to the reduced degree of demonstrated reliability with only a single test. The procedures of Chapter F of the *Specification* provide for the reward/penalty relationship developed with increasing number of tests and the associated coefficient of variation.

Minimum specimen size is as required in ASTM E1592-95. The minimum specimen length of 24 ft. (7.3 m) for the condition of constraint at both ends is consistent with the requirements of Factory Mutual (FM) Procedure 4471 (1995). However, in the FM tests, panels are fastened down at all edges and it is termed a field test. The details of the FM test do not meet the ASTM E1592-95 tests in many conditions. A purlin space of 5 ft. (1.5 m) requires 5 spans with both ends restrained. If one end is left free, the FM test will meet E1592-95. The application is also different in many cases because typically FM tests are run with both ends restrained and this is used as a field test. Different results may be obtained when using the three variations of panel end restraints in the test procedure that are allowed by E1592-95.

When totaling the number (n) of anchors tested for evaluation of C_p under the *Specification* Section C3.1.5, it is permissible to include all fasteners with the same tributary area as that associated with a failed anchor instead of merely totaling the number of physical tests run on a complete assembly. When totaling the number (n) of panels tested for evaluation of C_p under the *Specification* Section C3.1.5, it is permissible to include all panels with the same tributary area as that associated with a failed panel instead of merely totaling the number of physical tests run on a complete assembly

Consideration is given to the minimum spacings and material thicknesses. If allowables developed under this procedure are intended to be used in a design procedure that encompasses different secondary structural support spacings or thinner sections for anchors to attach to, the extremes must be tested in order for interpolation to be valid. This precedent is established in the AISI Base Test Method (2002) for validating the performance of purlins braced by standing seam roof panels.

10. Procedures

The procedures for loading the specimen, while not complicated, need to be defined consistent with other existing and recognized standards. A significant difference between this procedure and the AISI Base Test Method (2002) is the return to zero load after each load increment.

11. Test Evaluation

See Section C3.1.5 of the *Commentary* for the *Specification*.

12. Test Report

The definition of items to be included in the report includes the typical list of failure loads and plots of load versus deformation. Of paramount importance is the calculation of the resistance factor and safety factor of design strength or allowable design strength for panels and anchors. This procedure is an addition to those required in ASTM E1592-95. If interpolation is to be a part of the resulting design process, then appropriate interpolation procedure should be set forth in the report.

REFERENCES

Factory Mutual Research (1995) “Approval Standard for Class I Panel Roofs, Class Number 4471”, August 1995.

AISI TS-8-02 - Base Test Method for Purlins Supporting a Standing Seam Roof System, AISI *Cold-Formed Steel Design Manual*, 2002 Edition.

AISI TS-7-02

CANTILEVER TEST METHOD FOR COLD-FORMED STEEL DIAPHRAGMS

1. Tests of Framed Diaphragm Construction

Where framed wall, roof, or floor diaphragm construction is to be used for in-plane bracing, the nominal diaphragm shear strength, S_n, and the shear stiffness, G′, shall be established. These values depend on the component panel geometry, the panel width, w, support spacing, L_v, and the methods of attachment both between the panel components and on the framing members. The quality of shear transfer through the field of a diaphragm can be established by testing the assembly on a cantilever frame (see Figure 1).

2. Test Frames

The test frame dimensions a × b shall represent an approximately square zone with the frame comprised of perimeter members and interior support members similar to those intended for the construction being investigated.

Perimeter member ends shall be interconnected using angles or other devices sufficient to transfer developed axial forces into the frame supports. Interior members or purlins shall be fabricated with bolted clip angles or such other means as to minimize bending moment transfer at member ends. The test frame assemblies shall be attached to supporting devices by means appropriate for reactions and supported on rollers and other devices such that the frame possesses negligible stiffness for deflections, Δ, up to a/200. If the frame stiffness is greater than 2 percent of that for the tested assembly, the test results shall be adjusted to compensate for the frame stiffness.

3. Test Assemblies

The test assembly can be arranged to model a complete diaphragm, including conditions both on the perimeter members and over the diaphragm interior, or arranged to study selected parameters within the system.

For a complete diaphragm, the plan dimensions of the selected test frame shall be such that five or more panel elements, of width w, are required to cover the test area a × b with neither of the frame dimensions being less than eight feet (2.4 meters). The test diaphragm shall be assembled using the panels, connections, connector layouts, and spans intended for a specified construction. Where edge transfer angles or profiled end closure elements are used for shear transfer, they shall be included in the test assembly. If panel splices are used in the field assembly, at least one splice on each panel shall be provided in the test assembly.

For specific parametric studies within a system the panel edges over the test frame perimeter members may have connections spaced to prohibit failure at such locations.

For diaphragm systems where the longitudinal perimeter connections are spaced to match the interior side laps, four panel elements of width, w, may be used to cover the a x b area, provided that it is demonstrated that failure occurs at the longitudinal perimeter connections and not at the interior side laps.

The test assembly shall be instrumented to determine movements in the plane of attachment between the diaphragm and the test frame (see Figure 1). Measured deflections shall be corrected for support movements to determine the net shear deflection, Δ.

4. Testing

Prior to testing, the bare frame stiffness shall be determined by applying a force to produce a frame deflection of a/200.

The diaphragm assembly shall be loaded for a least one sequence between zero and 20 percent of the expected maximum load and unloaded for verifying the reference points on all instrumentation. The final loading sequence to P_n shall provide at least ten evenly spaced sets of deflection readings within the load range and the rate of loading shall be such that the anticipated strength is achieved in not less than 10 minutes.

The maximum diaphragm shear strength, S_n, shall be the average shear per unit length across the system at ultimate load, $S_n = P_n/b$.

The deflection data shall be combined to remove the effects of test frame support movements.

$$\Delta = \Delta_3 - [\Delta_2 + (a/b)(\Delta_1 + \Delta_4)] \qquad \text{(Eq. 1)}$$

where the Δ_i values are measured movements at the gage locations (see Figure 1). The applied loads shall be plotted against Δ (see Figure 2) and a line fitted through zero and a point on the plot at $P = 0.4\,P_n$. The resulting shear stiffness, G', is the ratio of shear per foot to the deflection per unit width at a load of $P = 0.4\,P_n$:

$$G' = \frac{P/b}{\Delta/a} = \frac{P}{\Delta}\frac{a}{b} \qquad \text{(Eq. 2)}$$

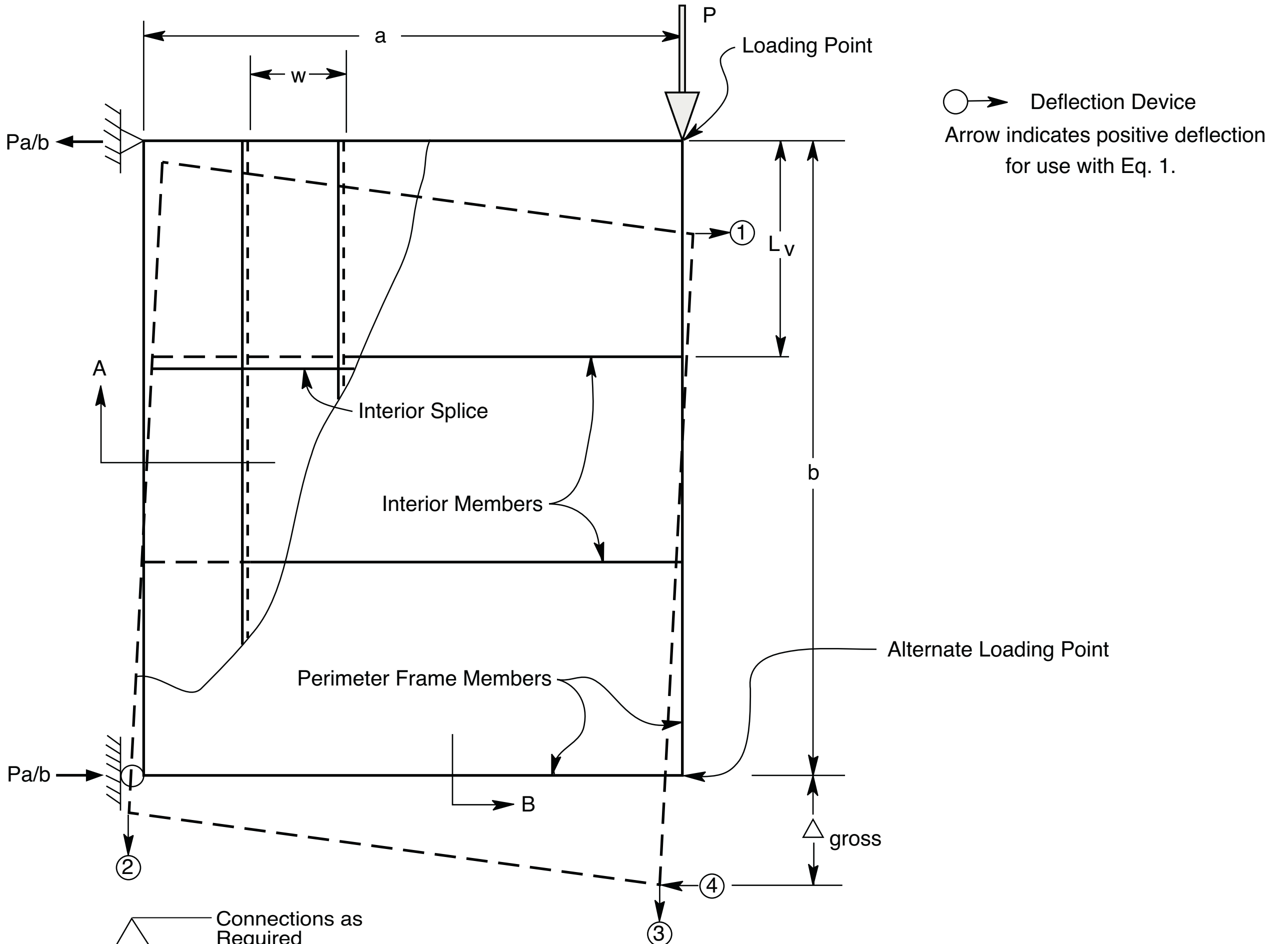
If the measured frame stiffness exceeds 2 percent of the assembly stiffness, the bare frame resistance, P_f, shall be established at a deflection corresponding to $0.4P_n$ and the P value shall be reduced by P_f for determining G'.

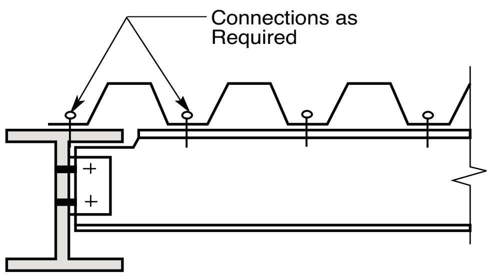
5. Minimum Number of Tests

To confirm analytical methods (1) for predicting diaphragm strength and stiffness as functions of the panel geometry, fastener arrangement, fastener types, and diaphragm details, no fewer than three confirmation tests shall be made. Tests shall be diverse in arrangement involving differing panel thicknesses, differing span lengths, and differing fastener placements within the common limits of construction. If the average ratios of the measured strength to theoretical strengths, S_n/S_t, do not agree to within 10 percent, a fourth assembly shall be tested. The theoretical strength formulas shall be modified by a scalar factor to agree with the average values from the three lowest tests.

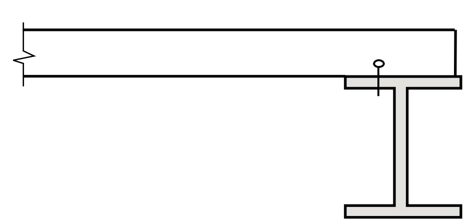
The stiffness formula likewise shall be modified to reflect the average measured stiffness.

Where a specific diaphragm construction is to be evaluated and the results limited only to that assembly, no fewer than two identical diaphragms shall be tested. If the strength of the two agree to within 10 percent, the average values of S_n and G' shall be established from the two tests. If the strength results are outside the 10% limit, a third test shall be made with the average values for S_n and G' established from the two lowest strength tests.



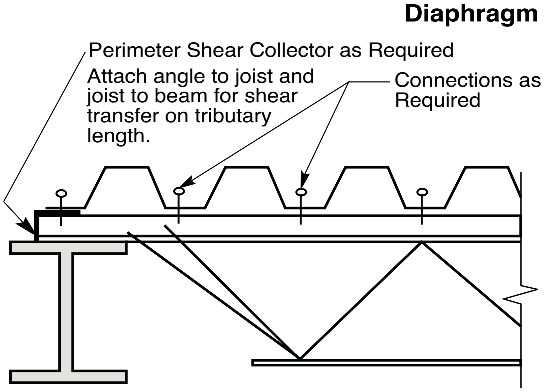


Section A

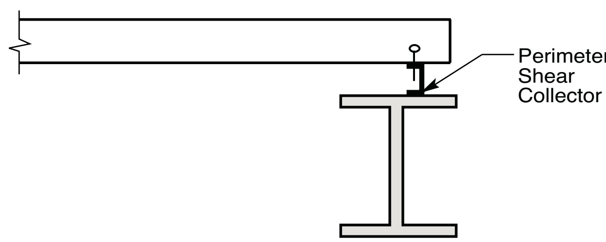


Section B

Diaphragm in the plane of the test frame



Section A



Section B

Diaphragm offset from the plane of the test frame

Figure 1 - Cantilever Frame

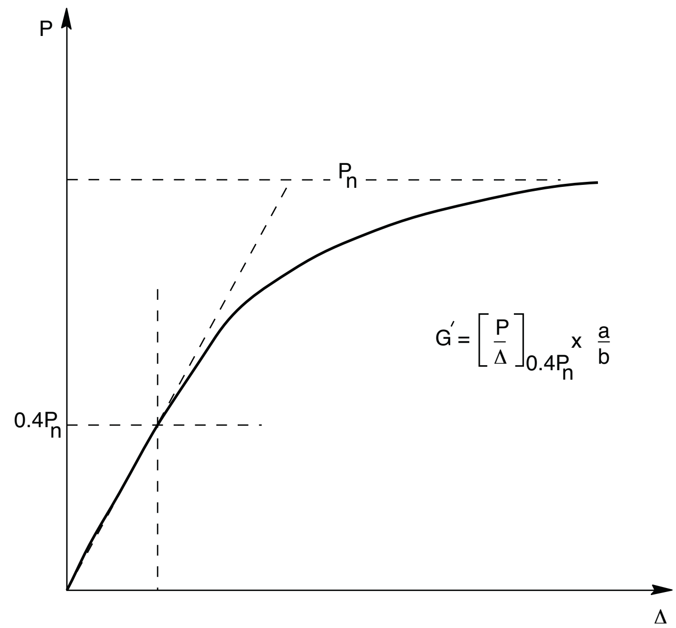


Figure 2 - Load-Deflection Curve

COMMENTARY ON THE CANTILEVER TEST METHOD FOR COLD-FORMED STEEL DIAPHRAGMS

1. Tests of Framed Diaphragm Construction

Shear diaphragms perform essentially the same functions of maintaining the shape in a building roof or wall zone as do plate girder webs in maintaining shape between their stiffeners. However, the diaphragm stiffness, G′, usually is an order of magnitude lower than that for thin-web girders.

The response of a diaphragm assembled from typical wall, roof or floor deck panels is very dependent on the panel type, panel spans, and especially on the quality of connections used. The diaphragm involves a "web" or panels, "stiffeners" or open web joists, purlins, girts and "flanges" or perimeter members. While its response may be thought of in terms of a short and deep beam, its behavior is much more related to that of truss panels having flexible diagonals.

2. Test Frames

Test frames may have perimeter members formed using various shapes including wide flange beams, C-sections, or Z-sections. The shape used must be sufficient to develop the required axial test forces and to permit proper perimeter connections to be made. Connections at the ends of perimeter members must be adequate to resist the developed axial forces and to transfer them to the support devices. Typical interior members or purlins develop only small axial forces and their end connections may be made using common clip angles. Where open-web joists are to be used, it may be necessary to attach an angle over the joist ends or to install intermediate supports to permit proper connections at the diaphragm edge.

The study of connections on the longitudinal edges of the diaphragm usually is not part of a test program. When the individual strength of these connections is known, they may be installed in sufficient numbers that they do not limit the diaphragm strength.

3. Test Assemblies

Tests are conducted to provide information on the behavior of a complete diaphragm assembly or to provide information on a specific parameter in a given system. For studies on a complete diaphragm assembly, a minimum of five panels should be used to allow reasonable force distribution across the sidelaps of the inner panels. Diaphragm tests using four panels, where the longitudinal edges are connected at the same spacing as the sidelaps, generally yield conservative measures of strength and stiffness. However, tests conducted with only four panels and having longitudinal edge connections spaced more closely than those on interior sidelaps, often yield artificially high strengths and stiffnesses.

The test assembly may represent a part of a larger diaphragm zone in a building. The assembly must be bounded by structural members for force transfer. The frame has a cross reaction equal to P(a/b) and the cross shear then is P(a/b)/a = P/b; this is equal to the shear parallel to the acting load P.

The test assembly should model typical constructions including perimeter transfer elements. In certain assemblies involving open-web joists, the lower surfaces of the diaphragm may be above the perimeter member joist supports thus limiting perimeter attachment for the diaphragm. An edge support angle can be welded to the joist end to accept frequent edge connections. Such angles must be attached to the joist ends and the joists then secured to the frame to permit proper shear transfer on the perimeter. Other block-like devices may be attached to the frame, between joists, to receive edge connections directly.

The ability to transfer forces across the longitudinal edges of a diaphragm may be assessed directly through evaluation of individual connections used. With such connectors as shear studs in concrete filled diaphragms, stud shear strengths are known and studs may be placed sufficiently close that perimeter shear failures are virtually eliminated.

4. Testing

The test sequence must be long enough to allow adequate data collection but not so short as to eliminate certain time-dependent relaxations in the system. A ten minute minimum test time has been set but the required time usually will be longer. The measured movements of the test frame support system will not be present in a field construction and are removed by the formulas indicated. In evaluation, it is of note that the diaphragm as a "beam" is in the "short-deep beam" category and that the "plane section" assumptions for longer beams do not apply. Further, the typical G' values are an order of magnitude lower than the equivalent number for thin-web girders. The beam bending correction is accounted for when the dial gages are positioned as shown in Figure 1.

The measured stiffness, G', reflects the performance of the diaphragm field as it resists shear forces. The frame itself usually has little resistance to movement prior to attaching the diaphragm. However, it is required that the bare frame be loaded with P_o to produce a deflection of a/200 where a is the width of the diaphragm. The a/200 value usually will be about one inch (25.4 mm). If the bare frame stiffness exceeds 2 percent of that for the assembly, the value for P in the stiffness equation must be reduced by P_f where P_f may be taken as $200(P_o\ \Delta/a)$. The value Δ/a is the unitless ratio of the corrected deflection at $0.4\ P_u$ to the indicated frame dimension.

The shear strength and stiffness values for diaphragms may vary in a nonlinear manner with panel length L. However, reasonably conservative intermediate design values may be found by interpolation of results from similar systems of differing lengths. Extrapolation of data may lead to erroneous results.

5. Minimum Number of Tests

The test program is intended to address two cases with one being to verify a theoretical model applicable to a range of diaphragm types and the other simply to evaluate a very specific system. For example, test programs may be needed where a new connector type is evaluated for its own shear strength over a range of material properties with comparisons made to some other connection type. Theoretical methods exist (1, 2) for finding the effects of parameter changes on the diaphragm performance and for predicting both the nominal shear strength and stiffness. In such cases, the diaphragm tests themselves are considered in the mode of verifying the established model. Should the test results suggest lower values, this section permits downward adjustments to the values from those of the parametric model.

Other specific arrangements of panels, end closure devices, overlayments, and specific connection methods may be outside the ability of existing models for predicting response. For such cases with a very specific construction application, a minimum of two tests is required to establish shear strength and stiffness.

REFERENCES

1. "Steel Deck Institute Diaphragm Design Manual, Second Edition", Steel Deck Institute, Inc., P.O. Box 9506, Canton, OH 44711, 1988
2. "Steel Deck Institute Diaphragm Design Manual, First Edition", Steel Deck Institute, Inc., P.O. Box 9506, Canton, OH 44711, 1981

AISI TS-8-02

BASE TEST METHOD FOR PURLINS SUPPORTING A STANDING SEAM ROOF SYSTEM

1. Scope

1.1 The purpose of this test is to obtain the reduction factor to be used in determining the nominal flexural strength of a purlin supporting a standing seam roof system. The reduction factor reflects the ability of a particular standing seam roof system to provide lateral and rotational support to the purlins to which it is attached. This applies to discrete lateral and torsional bracing when the sheeted flange of the purlin is the compression flange, as in gravity loading cases, and when the unsheeted flange is the compression flange, as in wind uplift cases.

1.2 This test method applies to an assembly consisting of the standing seam panel, purlin, and attachment devices used in the system being tested. The test specimen boundary conditions described in Section 6.6 apply only to standing seam roof systems for which the roof deck is positively anchored to the supporting structural system at one or more purlin or eave member lines.

1.3 Due to the many different types and construction of standing seam roof systems and their attachments, it is not practical to develop a generic method to predict the interaction of a particular standing seam roof system and supporting structure. Therefore, the amount of resisting moment which the supporting purlins can achieve can vary from the fully braced condition to the unbraced condition for a given system.

1.4 This test method provides the designer with a means of establishing a nominal flexural strength reduction factor for purlins in a simple span or continuous span, multiple purlin line, supporting a standing seam roof system, from the results of tests on a single-span, two-purlin line, sample of the system. The validity of this test method has been established by a research program at Virginia Polytechnic Institute and State University and documented in References 1 through 6.

2. Applicable Documents

2.1 ASTM Standards:
A370 - Standard Test Methods and Definitions for Mechanical Testing of Steel Products.

2.2 *North American Specification for the Design of Cold-Formed Steel Structural Members*, 2001 Edition.

3. Terminology

3.1 ASTM Definition Standards:

E6 - Standard Terminology Relating to Methods of Mechanical Testing.

IEEE/ASTM-SI-10-97 - Standard for Use of the International System of Units (SI): The Modern Metric System.

3.2 Description of terms specific to this standard:

fixed clip - a hold down clip which does not allow the roof panel to move independently of the roof substructure
insulation - glass fiber blanket or rigid board
lateral - a direction normal to the span of the purlins in the plane of the roof sheets

negative moment - a moment which causes tension in the purlin flange attached to the clips and standing seam panels

thermal block - strips of rigid insulation located directly over the purlin between clips

pan type standing seam roof - a "U" shaped panel which has vertical sides

positive moment - a moment which causes compression in the purlin flange attached to the clips and standing seam panels

rib type standing seam roof - a panel which has ribs with sloping sides and forms a trapezoidal shaped void at the side lap

sliding clip - a hold down clip which allows the roof panel to move independently of the roof substructure

standing seam roof system - a roof system in which the side laps between the roof panels are arranged in a vertical position above the roof line. The roof panel system is secured to the purlins by means of concealed hold down clips that are attached to the purlins with mechanical fasteners.

3.3 Symbols

b = flange width of the purlin
d = depth of the purlin
B = purlin spacing
F_y = design yield point
F_{yt} = measured yield point of tested purlin
L = span of the purlins tested, center to center of the supports
M_n = nominal flexural strength of a fully constrained beam, S_eF_y
$\overline{M}_{nt_{min}}$ = average flexural strength of the thinnest sections tested
$\overline{M}_{nt_{max}}$ = average flexural strength of the thickest sections tested
M_{nt} = flexural strength of a tested purlin, $S_{et}F_{yt}$
M_{ts} = failure moment for the single span purlins tested, $w_{ts}L^2/8$
p_d = weight of the specimen (force/area)
p_{ts} = failure load (force/area) of the single span system tested
P_L = lateral anchorage force in accordance with Section D3.2.1 of the *Specification*
R_t = modification factor from test, M_{ts}/M_{nt}
R = reduction factor computed for nominal purlin properties
$R_{t_{min}}$ = mean minus one standard deviation of the modification factors of the three thinnest purlins tested
$R_{t_{max}}$ = mean minus one standard deviation of the modification factors of the three thickest purlins tested
s = tributary width of the purlins tested
S_e = section modulus of the effective section
S_{et} = section modulus of the effective section of the tested member using measured dimensions and the measured yield strength
t = purlin thickness
w_{ts} = failure load (force/length) of the single span purlins tested

4. Significance

4.1 This test method provides the requirements for evaluating the resisting moment loads for cold-formed C- and Z-sections used with standing seam roof systems. This procedure is referred to as the "Base Test Method". The method is the result of extensive testing of various combinations of purlins, standing seam panels, and fastening devices. The tests were conducted over several years, benefiting from the experience provided by technical and industry experts. This procedure utilizes the results obtained from single span tests to predict the strength of multi-span conditions.

4.2 The Base Test Method shall be permitted to be used to evaluate the nominal flexural strength of C and Z-sections of multi-span, multiple purlin line, standing seam systems, with or without discrete intermediate braces.

4.3 The Base Test Method is applicable to both "rib" or "pan" type standing seam roof panels with "sliding" or "fixed" type clips.

4.4 The Base Test Method shall be conducted using standing seam roof panels, clips, fasteners, insulation, thermal blocks, discrete braces, and purlins as used in the actual standing seam roof system except as noted in Section 4.5.

4.5 Tests conducted with insulation are applicable to identical systems with thinner or no insulation.

5. Apparatus

5.1 A test chamber capable of supporting a positive or negative internal pressure differential is necessary. A rectangular frame shall be constructed of any material with sufficient strength and rigidity to provide the desired pressure differential without collapse. A typical test chamber is shown on Figure 1. Other chamber orientations shall be permitted.

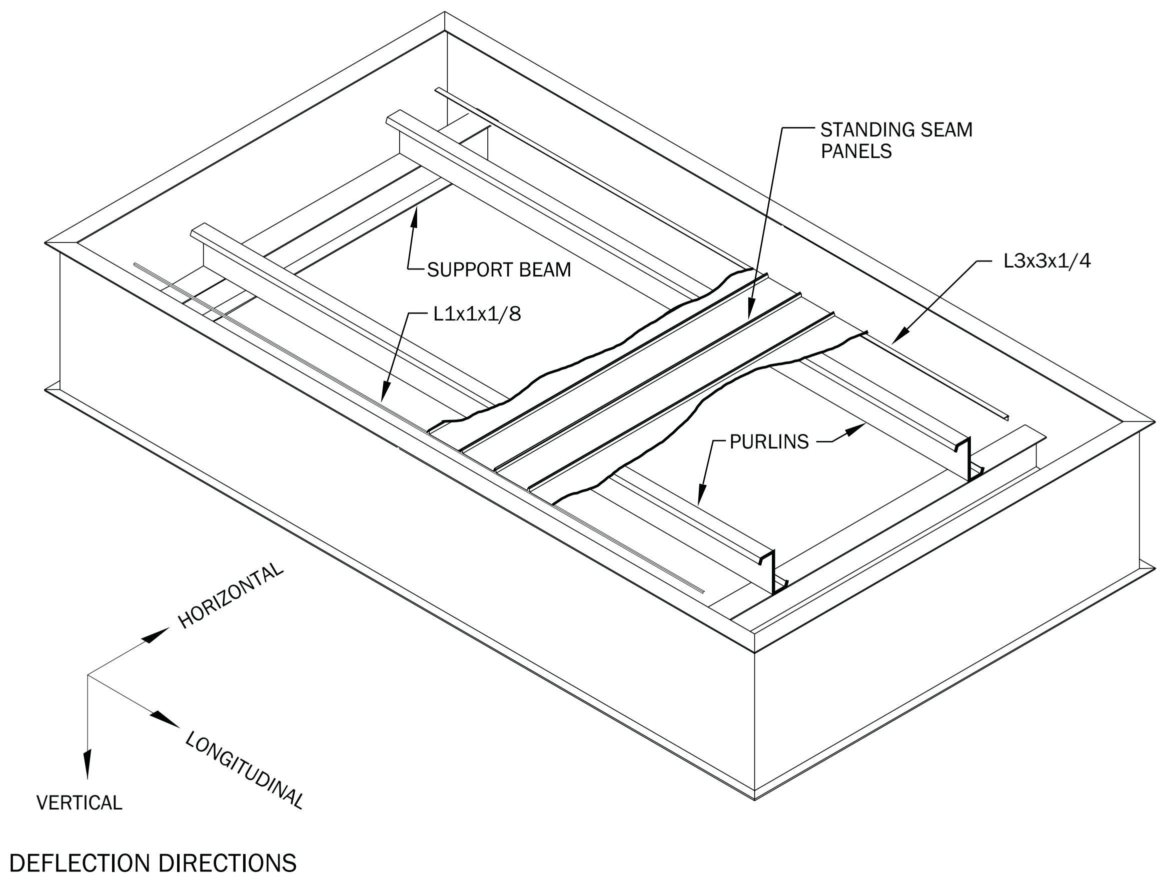


Figure 1 - Test Chamber

5.2 The length of the chamber shall be determined by the maximum length of the secondary members as required by Section 7.2. The width of the chamber shall be determined by the maximum panel length as required by Section 6.9. Allowance shall be made in the interior chamber dimensions to accommodate structural supports for the secondary members and sufficient clearance on all sides to prevent interference of the chamber wall with the test specimen as it deflects.

5.3 The height of the chamber shall be sufficient to permit assembly of the specimen and to insure adequate clearance at the maximum deflection of the specimen.

5.4 The chamber shall be sealed in a manner to prevent air leakage. All load carrying elements of the specimen or its supports shall transfer the load to the frame support; the specimen, including intermediate brace, shall not be attached to the chamber in any manner that would impede the deflection of the specimen.

5.5 The test chamber shall be sealed against air leakage by applying 6 mil (0.15 mm) maximum thickness polyethylene sheets, large enough to accommodate the system configuration and deflections. The polyethylene shall be located on the high pressure side of the panel with sufficient folds so as not to inhibit the spread of panel ribs under load. Edges of the polyethylene sheets shall be sealed against air leakage with tape or other suitable methods. Polyethylene sheets around the perimeter of the specimen shall be draped so as not to impede deflection or deformation of the specimen.

5.6 When a specimen smaller than the test chamber is tested, other panels and structure shall be installed to complete the coverage of the chamber opening. No attachment shall be made between the test specimen and this supplemental coverage.

5.7 An air pump is necessary to create the pressure differential in the chamber. The pump shall be of sufficient capacity to reach the expected test values required by the applicable specifications.

5.8 The type of air pump being used will determine the method of control. This control shall be able to regulate the pressure differential in the chamber to ± 1 psf (0.05 kPa). This can be accomplished by (a) a variable speed motor on the pump, (b) valving on the pump, or (c) variable size orifices on the chamber. It shall be permitted to use multiple pumps where very large chambers are being used. One pump connection to the chamber is satisfactory.

5.9 A minimum of two pressure differential measuring devices shall be monitored throughout the duration of the test. These devices shall be capable of measuring the pressure differential to ± 1 psf (0.05 kPa).

6. Test Specimens

6.1 Test purlins shall be supported at each end by a steel beam. The beams shall be simply supported and one of the frame end beams shall be sufficiently free to translate laterally to relieve any longitudinal catenary forces in the specimen. Purlins shall be connected to the supporting beams as recommended in the field erection drawings. Figure 1 shows the directional axes that are referred to in this test procedure.

6.2 Panel supporting clips, fasteners, and panels shall be installed as recommended in the field erection drawings.

6.3 Means of providing restraint of purlins at the support shall be as required for use in actual field application, and shall be installed as recommended on the field erection drawings.

6.4 The purlins shall be arranged either with their flanges facing in the same direction or with their flanges opposed. If the test is performed with the purlins opposed, and they are field installed with their flanges facing in the same direction, a diaphragm test must be conducted in accordance with Section 8.7.

6.5 For tests including intermediate discrete point braces, the braces used in the test shall be installed in such a manner so as not to impede the vertical deflection of the specimen.

6.6 A 1 in. x 1 in. (25 mm x 25 mm) continuous angle with a maximum thickness of 1/8 in. (3 mm) or a member of compatible stiffness shall be attached to the underside at each end of the panels to prevent separation of the panels at the ends of the seam. Fasteners shall be placed on both sides of each major rib. If the specimen is arranged with the purlin flanges facing in the same direction, a 3 in. x 3 in. (76 mm x 76 mm) continuous angle with a maximum thickness of 1/4 in. (6 mm) or a member of compatible stiffness shall be permitted to be substituted for the 1 in. x 1 in. (25 mm x 25 mm) angle at the end of the panel, corresponding to the eave of the building using the standard panel to eave fastening system (See Figure 1).

6.7 All transverse panel ends shall be left free to displace vertically under load. When the 3 in. x 3 in. (76 mm x 76 mm) eave angle is used when the purlin flanges face in the same direction, it shall be permitted to be restrained against horizontal deflection at its ends as shown in Figure 1, providing the vertical deflection is left unrestrained.

6.8 Panel joints shall not be taped and no tape shall be used to restrict panel movement.

6.9 Panel length to be used in the test shall be, as a minimum, that length which provides full engagement of the panel to purlin clip and attachment of the 1 in. x 1 in. (25 mm x 25 mm) angle at the panel ends; but a length not greater than that required to achieve zero slope of the panel at the purlin support.

6.10 The spacing of purlins being tested shall not exceed the spacing typically used with the roof system. Results from this test shall be permitted to be used in designing purlins of the same profile that are spaced closer together than the spacing used in the tests.

7. Test Procedure

7.1 A test series shall be conducted for each purlin profile, specified steel grade, and each panel system. Any variation in the characteristics or dimensions of panel or clip constitute a change in panel system. The thickness of insulation used in the test is discussed in Section 4.5. Any change in purlin shape or dimension other than thickness constitutes a change in profile. However, the lip dimension shall be permitted to vary with section thickness consistent with the member design and not constitute a change in profile.

7.2 No fewer than six tests shall be run for each combination of purlin profile and panel system. Three tests shall be conducted with the thinnest purlin of the profile and three tests shall be conducted with the thickest purlin of the profile. All tests shall be conducted using the same purlin span which shall be the same or greater than the span used in actual field conditions.

7.3 The physical and material properties shall be determined in accordance with ASTM A370 using coupons taken from the web area of the failed purlin. Coupons shall not be taken from areas where cold-working stresses could affect the results.

7.4 For gravity loading, a pressure differential load shall be applied to the system to produce a positive moment in the system. A positive moment is defined as one which causes compression in the purlin flange attached to the clips and standing seam panels. For uplift loading, a pressure differential load shall be applied to the system to produce a negative moment in the system. A negative moment is defined as one which causes tension in the purlin flange attached to the clips and standing seam panels.

7.5 An initial load equal to 5 psf (0.25 kPa) differential pressure in the direction of the test load shall be applied and removed to set the zero readings before actual system loading begins.

7.6 The system shall be loaded to failure and the mode of failure noted. Failure is the point at which the specimen will accept no further loading. The pressure differential at which the system fails shall be recorded as the failure load of the specimen. When the test must be stopped due to a flexural failure of the panel or web crippling of the purlin, it shall be permitted to exclude the test from the test program.

7.7 Vertical deflection measurements shall be taken at the mid-span of both purlins. The deck deflection in the horizontal direction shall be measured at the seam joint nearest the center of the test specimen.

7.8 Deflections and pressures shall be recorded at pressure intervals equal to a maximum of 20 percent of the anticipated failure load.

8. Test Evaluation

8.1 The single span failure load is obtained from the Base Test where a uniform load is applied until failure occurs. The computation of the failure load, w_{ts}, is dependent on the purlin orientation for Z-purlins and on the nature of the load as follows:

For Z-purlins tested for gravity loading, with flanges facing the same direction and with the top flanges of the purlins not restrained by anchorage to a point external to the panel / purlin system:

$$w_{ts} = \left(p_{ts} + p_d\right)s + 2P_L\left(\frac{d}{B}\right)$$

where,

$$P_L = 0.041\left(\frac{b^{1.5}}{d^{0.90}t^{0.60}}\right)\left(p_{ts} + p_d\right)s$$

For Z-purlins tested for gravity loading with flanges opposed and for C-sections tested for gravity loading:

$$w_{ts} = \left(p_{ts} + p_d\right)s$$

For Z-purlins or C-sections tested for uplift loading:

$$w_{ts} = \left(p_{ts} - p_d\right)s$$

The expression $2P_L(d/B)$ takes into account the effect of the overturning moment on the system due to the anchorage forces, as defined in Section D3.2.1 of the *Specification*, applied at the top flange of the purlin by the panel and resisted at the bottom flange of the purlin at the support. The expression $2P_L(d/B)$ is to be applied only to Z-sections under gravity loading when the purlin flanges are facing in the same direction, but shall not be included in those systems where discrete point braces are used when the braces are restrained from lateral movement.

8.2 From the single span failure load, w_{ts}, the maximum single span failure moment M_{ts} is calculated as:

$$M_{ts} = w_{ts}L^2 / 8$$

8.3 The single span base test moment is the maximum moment the system can resist with the purlin size used in the test. The maximum allowable moment of a roof system purlin, simple span or continuous, is limited by the results of this test. The gravity load results apply for positive moment regions in the span and uplift load results apply for negative moment regions in the span.

8.4 Using Section C3.1.1(a) of the *Specification*, the flexural strength of each tested purlin, M_{nt}, of a fully constrained beam is calculated as:

$$M_{nt} = (S_{et})(F_{yt})$$

where S_{et} is the section modulus of the effective section calculated using the measured cross-sectional dimensions and measured yield strength and F_{yt} is the measured yield strength obtained in accordance with Section 7.3.

8.5 The modification factor, R_t, is calculated for each purlin tested as:

$$R_t = M_{ts} / M_{nt}$$

8.6 For purlins of the same profile, specified steel grade and panel system as tested, the reduction factor shall be determined from the following equation:

$$R = \left(\frac{R_{t_{max}} - R_{t_{min}}}{\overline{M}_{nt_{max}} - \overline{M}_{nt_{min}}}\right)\left(M_n - \overline{M}_{nt_{min}}\right) + R_{t_{min}} \leq 1.0$$

where,

$R_{t_{min}}$ = mean minus one standard deviation of the modification factors of the three thinnest purlins tested, calculated in accordance with Section 8.5. This value may be greater than 1.0.

$R_{t_{max}}$ = mean minus one standard deviation of the modification factors of the three thickest purlins tested, calculated in accordance with Section 8.5. This value may be greater than 1.0.

M_n = nominal flexural strength of section for which R is being evaluated (S_eF_y).

$\overline{M}_{nt_{min}}$ = average flexural strength of the thinnest section tested, calculated in accordance with Section 8.4.

$\overline{M}_{nt_{max}}$ = average flexural strength of the thickest section tested, calculated in accordance with Section 8.4.

8.7 If the test is performed with the purlins opposed or with an eave member at one or more edges, the diaphragm strength and stiffness of the panel system must be tested unless the purlins are also opposed in actual field usage. The anchorage forces for the system braced in the manner tested shall be calculated in accordance with Section D3.2.1 of the *Specification*. The diaphragm strength of the panel system must be equal to or greater than the calculated brace force at the failure load of the purlin. The stiffness of the diaphragm must be such that the deflection of the diaphragm is equal to or less than the purlin span divided by 360 when subjected to the calculated brace force at the failure load of the purlin.

9. Test Report

9.1 Documentation - The report shall include who performed the test and a brief description of the system being tested.

9.2 The documentation shall include test details with a drawing showing the test fixture and indicating the components and their locations. A written description of the test setup detailing the basic concept, loadings, measurements, and assembly shall be included.

9.3 The report shall include a drawing showing the actual geometry of all specimens including material specifications and test results defining the actual material properties - material thickness, yield strength, tensile strength, and percent elongation.

9.4 The report shall include the test designation, loading increments, displacements, mode of failure, failure load, and specimen included for each test.

9.5 The report shall include a description summarizing the test program results to include specimen type, span, failure moments for the test series, and the supporting calculations.

REFERENCES

1. S. Brooks and T. Murray, "Evaluation of the Base Test Method for Predicting the Flexural Strength of Standing Seam Roof Systems Under Gravity Loading," MBMA Project 403, VPI Report No. CE/VPI-ST89/07, Metal Building Manufacturers Association, 1300 Sumner Ave., Cleveland, Ohio 44115, July 1989, Revised November 1990.
2. S. Brooks and T. Murray, "A Method for Determining the Strength of Z- and C-Purlin Supported Standing Seam Roof Systems", Proceedings of the Tenth International Specialty Conference on Cold-Formed Steel Structures, St. Louis, October 23-24, 1990, pp. 421-440.
3. L. Rayburn and T. Murray, "Base Test Method for Gravity Loaded Standing Seam Roof Systems," MBMA Project 502, VPI Report No. CE/VPI-ST90/07, Metal Building Manufacturers Association, 1300 Sumner Ave., Cleveland, Ohio 44115, December 1990.
4. T. Murray and B. Anderson, "Base Test Method for Standing Seam Roof Systems Subject to Uplift Loading - Phase I," MBMA Project 501, VPI Report No. CE/VPI-ST90/06, Metal Building Manufacturers Association, 1300 Sumner Ave., Cleveland, Ohio 44115, December 1990, Revised December 1991.
5. T. Murray and A. Pugh, "Base Test Method for Standing Seam Roof Systems Subject to Uplift Loading - Phase II," MBMA Project 602, VPI Report No. CE/VPI-ST91/17, Metal Building Manufacturers Association, 1300 Sumner Ave., Cleveland, Ohio 44115, December 1991.
6. T. Murray, "Base Test Method for Uplift Loading - Final Report," MBMA Project 501, 602 and 702, VPI Report No. CE/VPI-ST-97/10, Metal Building Manufacturers Association, 1300 Sumner Ave., Cleveland, Ohio 44115, November 1997.

SECTION 2 - 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 Extensometers

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) Test 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 Methods 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 Constructions 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 Insulation 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 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 (e-N) Fatigue Data

Joining and Fastening Test Procedures

ASTM E488 Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements

ASTM E489 Standard Test Method for Tensile Strength Properties of Metal Connector Plates

ASTM E767 Standard Test Method for Shear Strength Properties of Metal Connector Plates

General References

ASTM E631 Standard Terminology of Building Constructions

IEEE/ASTM SI 10 American National Standard for Use of the International System of Units (SI): The Modern Metric System

Other Publications:

Test Procedure for Shear Resistance of Small-Scale Framed Wall Specimens, in "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 3 - EXAMPLE PROBLEM

EXAMPLE VI-1: COMPUTING ϕ AND Ω FACTORS FROM TEST DATA

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. Determine the resistance factor, ϕ, for this assembly.

2. 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

$C_p = (1 + 1/n)m/(m-2)$ (*Eq.* F1.1-3)

where

n = number of tests = 4

$m = n-1 = 3$

$C_p = (1+1/4)3/(3-2) = 3.75$ (*Eq.* F1.1-3)

4. Compute the standard deviation of the test results

$$s = \sqrt{\frac{\sum_{i=1}^{n}\left(x_i - \overline{x}\right)^2}{n-1}}$$

$$= \sqrt{\frac{(5.6 - 5.83)^2 + (6.0 - 5.83)^2 + (5.8 - 5.83)^2 + (5.9 - 5.83)^2}{4-1}}$$

$$= 0.171$$

5. Compute the coefficient of variation of the test results, V_p

$V_p = s/R_n$

$= 0.171/5.83$

$= 0.029 < 0.065 \therefore$ 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, $\beta_{o,}$ V_Q and C_ϕ

$P_m = 1.0$ (always)

$\beta_o = 3.5$ (for connections for the United States)

$V_Q = 0.21$ (always)

$C_\phi = 1.52$ (for the United States)

8. Compute ϕ

$$\phi = C_\phi (M_m F_m P_m) e^{-\beta_o \sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_Q^2}} \qquad (Eq.\ F1.1\text{-}2)$$

$$= 1.52[(1.10)(1.0)(1.0)]e^{-3.5\sqrt{0.10^2 + 0.10^2 + (3.75)0.065^2 + 0.21^2}}$$

$$= 0.62$$

9. Compute Ω

$$\Omega = \frac{1.6}{\phi} \qquad (Eq.\ F1.2\text{-}2)$$

$$= \frac{1.6}{0.62}$$

$$= 2.6$$

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