01 Jan 1956

Light Gage Cold-formed Steel Design Manual

American Iron and Steel Institute

LIGHT GAGE COLD-FORMED STEEL DESIGN MANUAL

1956 EDITION

AMERICAN IRON AND STEEL INSTITUTE
150 EAST FORTY-SECOND STREET
NEW YORK 17, N. Y.

CCFSS LIBRARY American Iron and Steel
18 5 * 44 Institute LIGHT GAGE COLD-FORMED
c2 STEEL DESIGN MANUAL

DATE	ISSUED TO

LIGHT GAGE COLD-FORMED STEEL DESIGN MANUAL

1956 EDITION

PRICE—ONE DOLLAR

AMERICAN IRON AND STEEL INSTITUTE
150 EAST FORTY-SECOND STREET
NEW YORK 17, N. Y.

PREFACE

The "Specification for the Design of Light Gage Steel Structural Members," first published by American Iron and Steel Institute in April 1946, has gained both national and international recognition. It is accepted as the design standard for cold-formed steel structural members in: The Basic Building Code sponsored by the Building Officials Conference of America, the National Building Code of the National Board of Fire Underwriters, the Southern Standard Building Code sponsored by the Southern Building Code Congress and the Uniform Building Code of the International Conference of Building Officials. This design standard is now used wholly or partly by most of the U. S. cities having building codes.

In this second edition, the Specification and Manual have been rewritten and enlarged. The order of presentation of the material has been changed, and substantive changes and additions have been made, in keeping with rapid technical developments and to reflect the results of the continuing program of research conducted at Cornell University since 1939 under the sponsorship of the American Iron and Steel Institute.

In Part I, the ASTM material references in Section 1 have been brought up to date. In Section 2, the provisions for determining effective design widths have been simplified (2.3.1); design provisions for multiple-stiffened elements (2.3.1.2) and intermediate stiffeners (2.3.2.2) have been added; the maximum allowable flat width ratios have, in certain instances, been increased (2.3.3.a); a maximum depth to thickness ratio for webs has been established (2.3.4). In Section 3, provisions for angle struts (3.2.c), for unbraced channels and Z-beams (3.3), for bending in webs (3.4.2 and 3.4.3), for wind and earthquake stresses (3.8) and for design of tubular compression members (3.9) have been added; provisions to avoid web crippling (3.5) have been improved. In Section 4 new provisions for welds and their spacing have been added, as have provisions for bolted connections. In Section 5, bracing requirements for channel and Z-shaped beams (5.2) have been added. In Section 6, "Tests for Special Cases" has been clarified.

In Part II, methods of determining the lateral bracing value of wall materials have been added. Part II also contains a new and complete list of the symbols used in the Specification.

In Parts III and IV the illustrative examples and the tables and charts have been clarified and revised to conform with the foregoing changes. The structural properties of "hat" sections and inverted U sections have been included.

For the designing engineer this Manual will provide information for determining the safe load capacities and deflections of sections made of any combination of flat elements formed from sheet or strip steel less than 3/16 in. thick. The tables of structural properties will be useful not only in designing the sections shown but also for approximating the corresponding properties of comparable formed sections.

The provisions of the Design Specification and the data contained in this Manual give accurate results for carbon and low-alloy steels. *They do not apply to non-ferrous metals* whose modulus of elasticity is substantially different from that of steel.

Designers are by no means limited to the use of sections listed in the Tables of Part IV of this Manual. The flexibility of the forming processes and the great variety of shapes which may be formed of sheet and strip steel are such that substantial economies can often be effected, or particular end use requirements met, by the use of special sections. However, the designer should seek the advice of manufacturers or fabricators before specifying special sections.

TABLE OF CONTENTS

Part I

SPECIFICATION FOR THE DESIGN OF LIGHT GAGE COLD-FORMED STEEL STRUCTURAL MEMBERS

Part II
SUPPLEMENTARY INFORMATION

Part III
ILLUSTRATIVE EXAMPLES

Part IV

CHARTS AND TABLES OF STRUCTURAL PROPERTIES

APPENDIX

PART I

SPECIFICATION FOR THE DESIGN OF LIGHT GAGE COLD-FORMED STEEL STRUCTURAL MEMBERS

1956 Edition

SECTION 1. GENERAL

1.1—SCOPE

This Specification shall apply to the design of structural members cold formed to shape from sheet or strip steel less than 3/16 inch thick and used for load-carrying purposes in buildings.

Nothing herein is intended to conflict with provisions of the Specifications issued by the American Institute of Steel Construction for the Design, Fabrication, and Erection of Structural Steel for Buildings nor with the Standard Specifications for Open-Web Steel Joist Construction as issued by the Steel Joist Institute.

1.2—MATERIAL

Except as otherwise provided herein, steel shall conform to the following Standard Specifications of the American Society for Testing Materials, as amended to date:

Heavy Gage Structural Quality Flat Hot-Rolled Carbon-Steel Sheets, ASTM designation: A245

Light Gage Structural Quality Flat Rolled Carbon-Steel Sheets, ASTM designation: A246

Hot-Rolled Carbon-Steel Strip of Structural Quality, ASTM designation: A303

The terms C, B, and A when used herein to designate grades of steel refer to grades provided by the above-listed ASTM Specifications.

Steel of higher strength than is covered by the above-mentioned ASTM specifications may be used at the unit stresses hereinafter specified for "other" grades of steel provided the design is based upon the published minimum properties of such higher strength steel as certified by the manufacturer's test reports. It is the intent of this Specification to permit the use of special high-strength steels of suitable properties for purposes coming within the scope of this Specification, but not to permit the use of carbon steels at unit stresses higher than those specified in Section 3 for Grade C material unless the steel is produced to and identified with a published specification which specifically establishes its qualifications and properties.

SECTION 2. DESIGN PROCEDURE

2.1—PROCEDURE

All computations for safe load, stress, deflection and the like shall be in accordance with conventional methods of structural design except as otherwise specified herein.

2.2—DEFINITIONS

Where the following terms appear in this Specification they shall have the meaning herein indicated:

(a) *Stiffened Compression Elements.* The term "stiffened compression elements" shall mean flat compression elements (i.e., plane compression flanges of flexural members and plane webs and flanges of compression members) of which *both* edges parallel to the direction of stress are stiffened by connection to a stiffening means (i.e., web, flange, stiffening lip, intermediate stiffener, or the like) conforming to the requirements of Section 2.3.2.*

(b) *Unstiffened Compression Elements.* Any flat element which is stiffened at only *one* edge parallel to the direction of stress shall be considered an "unstiffened" element.

(c) *Multiple-Stiffened Elements.* A multiple-stiffened element is an element that is stiffened between webs, or between a web and an edge, by means of intermediate stiffeners which are parallel to the direction of stress and which conform to the requirements of Section 2.3.2.2. A *sub-element* is the portion between adjacent stiffeners or between web and intermediate stiffener or between edge and intermediate stiffener.*

(d) *Flat-Width Ratio.* The flat-width ratio, w/t, is the ratio of the flat width, w, exclusive of edge fillets, of a single flat element to the thickness, t, of such element. In the case of sections such as I-, T-, channel- and Z-shaped sections, the width, w, is the width of the flat projection of flange from web, exclusive of fillets and of any stiffening lip that may be at the outer edge of the flange. In the case of *multiple-web* sections such as hat-, U- or box-shaped sections, the width, w, is the flat width of flange between adjacent webs, exclusive of fillets.

(e) *Effective Design Width.* Where the flat width, w, of an element is reduced for design purposes, the reduced design width, b, is termed the "effective width," or the "effective design width." This "effective design width" is determined in accordance with Sections 2.3.1 and 2.3.5.*

2.3—PROPERTIES OF SECTIONS

Properties of sections (cross-sectional area, moment of inertia, section modulus, radius of gyration, etc.) shall be determined in accordance with conventional methods of structural design. Properties shall be based on the full cross-section of the members (or net section where the use of a net section is customary) except where the use of a reduced cross-section, or "effective design width," is required by the provisions of Sections 2.3.1 and 2.3.5 of this Specification.**

---

* Charts 1 and 2 in Part IV illustrate several types of stiffened elements and their Effective Cross Sections.

** Tables 1 to 9 in Part IV list properties of many sections.

2.3.1 Properties of Stiffened Compression Elements

Effective Design Width—In computing properties of sections of flexural members and in computing values of "Q" (Section 3.6.1) for compression members, the flat width, w, of any stiffened compression element having a flat-width ratio larger than $(w/t)_{lim}$ as hereinafter defined shall be considered as being reduced for design purposes to an *effective design width,* b or b′, determined in accordance with the provisions of Sections 2.3.1.1 or 2.3.1.2, whichever is applicable, and subject to the limitations of Section 2.3.5 where applicable. That portion of the total width which is considered removed to arrive at the effective design width shall be located symmetrically about the center line of the element.*

2.3.1.1 Elements Without Intermediate Stiffeners

The effective design widths of compression elements which are not subject to the provisions of Section 2.3.1.2 governing multiple-stiffened elements shall be determined from the following formulas: **

For load determination:

Flanges are fully effective $(b = w)$ up to $(w/t)_{lim} = 3790/\sqrt{f}$ (Table 2.3.1. A lists values of $(w/t)_{lim}$)

For flanges with w/t larger than $(w/t)_{lim}$

$$(b/t) = \frac{7590}{\sqrt{f}} \left(1 - \frac{1900}{(w/t)\ \sqrt{f}} \right)$$

For deflection determination: †

Flanges are fully effective up to $(w/t)_{lim} = 5160/\sqrt{f}$

For flanges with w/t larger than $(w/t)_{lim}$

$$(b/t) = \frac{10320}{\sqrt{f}} \left(1 - \frac{2580}{(w/t)\ \sqrt{f}} \right)$$

In the above,

w/t = flat-width ratio

b = effective design width

f = actual unit stress in the compression element computed on the basis of the effective design width.**

---

* Charts 1 and 2 in Part IV illustrate several types of stiffened elements and their effective cross sections. Charts 3A to 3D show effective design widths for stiffened elements.

** The determination of effective design width may be facilitated by the use of Charts 3A to 3D inclusive. Table 2.3.1.1. B shows effective design width for a unit stress of 18,000 psi. It is to be noted that where the flat-width ratio exceeds $(w/t)_{lim}$ the properties of the section must frequently be determined by successive approximations, since the unit stress and the effective design width are interdependent.

† Explanation of the differences for deflection and safe load determinations appears in Part II under "Stiffened Compression Elements — Reduced Cross Section."

TABLE 2.3.1.1. A

Maximum Width/Thickness Ratios, $(w/t)_{lim}$, at which Compression Elements are Fully Effective $(b/t = w/t)$

Stress, f, p.s.i.	For Load Determination	For Deflection Determination
6,000	48.9	66.6
8,000	42.4	57.7
10,000	37.9	51.6
12,000	34.6	47.1
13,500	32.6	44.4
15,000	30.9	42.1
16,500	29.5	40.2
18,000	28.2	38.5
20,000	26.8	36.5
22,000	25.6	34.8
24,000	24.5	33.3
27,000	23.1	31.4
30,000	21.9	29.8
36,000	20.0	27.2
40,000	19.0	25.8
Other	$3790/\sqrt{f}$	$5160/\sqrt{f}$

TABLE 2.3.1.1. B

Ratio b/t of Effective Design Width to Thickness of Stiffened Compression Elements, for f = 18,000 lbs. per sq. in.

w/t	b/t for		w/t	b/t for	
	Load Determination	Deflection Determination		Load Determination	Deflection Determination
28.2*	28.2	28.2	120	49.9	64.6
30	29.9	30.0	140	50.8	66.4
35	33.7	35.0	160	51.6	67.7
38.5**	35.8	38.5	180	52.1	68.7
40	36.5	39.9	200	52.6	69.5
45	38.8	44.0	225	53.0	70.3
50	40.5	47.3	250	53.4	71.0
55	42.0	50.0	275	53.7	71.5
60	43.2	52.3	300	53.9	72.0
70	45.1	55.8	350	54.3	72.7
80	46.6	58.4	400	54.6	73.2
90	47.7	60.5	450	54.8	73.6
100	48.6	62.1	500	55.0	74.0

* $(w/t)_{lim}$ for load determination, see Section 2.3.1.

** $(w/t)_{lim}$ for deflection determination, see Section 2.3.1.

2.3.1.2 Multiple-Stiffened Elements

Where the flat-width ratio of a sub-element of a multiple-stiffened element does not exceed 60, the effective design width, b, of such sub-element shall be determined in accordance with the provisions of Section 2.3.1.1. Where such flat-width ratio exceeds 60, the effective design width, b′, of the sub-element shall be determined from the following formula:

$$\frac{b'}{t} = \frac{b}{t} - 0.10 \left(\frac{w}{t} - 60 \right)$$

where

w/t = flat-width ratio of sub-element

b = effective design width determined in accordance with the provisions of Section 2.3.1.1

b' = effective design width of sub-element to be used in design computations

For computing the effective structural properties of a member having multiple-stiffened elements, the area of any intermediate stiffener shall be considered reduced to an effective area as given by the expression

$$A_{eff} = A_{full}\,(b/w), \text{ or}$$
$$A_{eff} = A_{full}\,(b'/w),$$

whichever is applicable. In the above expressions, A_{eff} and A_{full} refer only to the area of the stiffener section, exclusive of any portion of adjacent elements.

The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.

2.3.2 Stiffeners for Compression Elements

2.3.2.1 Edge Stiffeners

In order that a flat compression element may be considered a "stiffened compression element" it shall be stiffened along each longitudinal edge parallel to the direction of stress by a web, lip, or other stiffening means, having the following minimum moment of inertia:

$$I_{min} = 1.83t^4 \sqrt{(w/t)^2 - 144} \qquad \text{(See Table 2.3.2.1)}$$

where w/t = flat-width ratio of stiffened element, and

I_{min} = minimum allowable moment of inertia of stiffener (of any shape) about its own centroidal axis parallel to the stiffened element.*

Where the stiffener consists of a simple lip bent at right angles to the stiffened element, the required over-all depth d of such lip may be determined with satisfactory accuracy by the following formula:

$$d = 2.8t \sqrt[6]{(w/t)^2 - 144} \qquad \text{(See Table 2.3.2.1)}$$

A simple lip shall not be used as an edge stiffener for any element having a flat-width ratio greater than 60.

---

* Table 8 in Part IV lists properties of simple lip stiffeners.

TABLE 2.3.2.1

Minimum Properties of Stiffeners for Compression Elements

w/t	I	d	w/t	I	d
12 or less	0	0	20	$29.3t^4$	7.1t
			25	$40.2t^4$	7.8t
13	$9.2t^4$	4.8t	30	$50.4t^4$	8.5t
14	$13.2t^4$	5.4t	40	$69.9t^4$	9.4t
16	$19.4t^4$	6.2t	50	$89.0t^4$	10.2t
18	$24.6t^4$	6.7t	60	$107.6t^4$	10.9t
			Over 60	$1.83t^3w$	

2.3.2.2 Intermediate Stiffeners

In order that a flat compression element may be considered a "multiple stiffened element," it shall be stiffened between webs, or between a web and an edge, by means of intermediate stiffeners parallel to the direction of stress, and the moment of inertia of each such intermediate stiffener shall be not less than twice the minimum allowable moment of inertia specified for edge stiffeners in Section 2.3.2.1. The following limitations also shall apply:

(a) If the spacing of stiffeners between two webs is such that the flat-width ratio of the sub-element between stiffeners is larger than $(w/t)_{lim}$ (Section 2.3.1) only two intermediate stiffeners (those nearest each web) shall be considered effective.

(b) If the spacing of stiffeners between a web and an edge stiffener is such that the flat-width ratio of the sub-element between stiffeners is larger than $(w/t)_{lim}$ (Section 2.3.1) only one intermediate stiffener shall be considered effective.

(c) If intermediate stiffeners are spaced so closely that the flat-width ratio between stiffeners does not exceed $(w/t)_{lim}$ (Section 2.3.1) all the stiffeners may be considered effective. In computing the flat-width ratio of the entire multiple-stiffened element, such element shall be considered as replaced by an element without intermediate stiffeners whose width w_s is the *whole* width between webs or from web to edge stiffener, and whose equivalent thickness t_s is determined as follows:

$$t_s = \sqrt[3]{\frac{12\ I_s}{w_s}}$$

where I_s = moment of inertia of the full area of the multiple-stiffened element, including the intermediate stiffeners, about its own centroidal axis.

2.3.3 Maximum Allowable Flat-Width Ratios

Maximum allowable overall flat-width ratios, w/t, disregarding intermediate stiffeners and taking as t the actual thickness of the element, shall be as follows:

(a) Stiffened compression element having *one* longitudinal edge connected to a web or flange element, the other stiffened by:
Simple lip bent at right angle to the element 60
Any other kind of stiffener 90

(b) Stiffened compression element with *both* longitudinal edges connected to a web or flange element (U-type or box-type sections) 500

(c) Unstiffened compression element 60

Note: Unstiffened compression elements that have flat-width ratios exceeding approximately 30 and stiffened compression elements that have flat-width ratios exceeding approximately 250 are likely to develop noticeable deformation at the full allowable working stresses, without detriment to the ability of the member to carry design loads.

Stiffened elements having flat-width ratios larger than 500 may be used with safety to support loads, but substantial deformation of such elements under load may occur and may render inapplicable the design formulas of this Specification.

(d) Unusually Wide Flanges: Where a flange of a flexural member is unusually wide and it is desired to limit the maximum amount of curling or movement of the flange toward the neutral axis, the following formula applies to compression and tension flanges, either stiffened or unstiffened:

$$w_{max} = \sqrt{\frac{1{,}800{,}000th}{f_{av}}} \times \sqrt[4]{\frac{100\ c}{h}}$$

where

w_{max} = the width, in inches, of flange projecting beyond the web; or half of the distance between webs for box- or U-type beams.

t = thickness of flange in inches.

h = depth of beam in inches.

c = the amount of curling in inches.*

f_{av} = the *average* stress in the full, unreduced flange width. [Where members are designed by the *effective design width* procedure, the average stress = the maximum stress x (the ratio of the effective design width to the actual width).]

2.3.4 Maximum Allowable Web Depth

The ratio h/t of the webs of flexural members shall not exceed **150**
where

h = clear distance between flanges, in.

t = thickness of web, in.

Where a web consists of two or more sheets, the h/t ratio of the individual sheets shall not exceed 150

2.3.5 Unusually Short Spans Supporting Concentrated Loads

Where the span of the beam is less than 30 w′ (w′ as defined below) and it carries one concentrated load, or several loads spaced farther apart than 2 w′,

* The amount of curling that can be tolerated will vary with different kinds of sections and must be established by the designer. Amount of curling in the order of 5% of the depth of the section is usually not considered excessive.

the effective design width of any flange, whether in tension or compression, shall be limited to the following:

TABLE 2.3.5

Short, Wide Flanges

Maximum Allowable Ratio of Effective Design Width to Actual Width

L/w'	Ratio	L/w'	Ratio
30	1.00	14	0.82
25	0.96	12	0.78
20	0.91	10	0.73
18	0.89	8	0.67
16	0.86	6	0.55

In Table 2.3.5 above:

L = full span for simple spans; or the distance between inflection points for continuous beams; or twice the length of cantilever beams.

w' = width of flange projection beyond the web for I-beam and similar sections or half the distance between webs for box- or U-type sections.

For flanges of I-beams and similar sections stiffened by lips at the outer edges, w' shall be taken as the sum of the flange projection beyond the web plus the depth of the lip.

SECTION 3. ALLOWABLE DESIGN STRESSES

The maximum allowable unit stresses to be used in design shall be as follows:

3.1—BASIC DESIGN STRESS

Tension on the net section of tension members, and tension and compression, f_b, on the extreme fibers of flexural members shall not exceed the values specified below except as otherwise specifically provided herein.

Grade of Steel	Min. Yield Point lbs per sq. in.	f_b (lbs per sq. in.)
C	33,000	18,000
B	30,000	16,500
A	25,000	13,500

Other f_b = Specified minimum yield point/1.85

For special provisions for members resisting wind or earthquake loads see Section 3.8, Wind or Earthquake Stresses.

3.2—COMPRESSION ON UNSTIFFENED ELEMENTS

Compression, f_c, in pounds per square inch, on flat unstiffened elements:

(a) For w/t not greater than 12, $f_c = f_b$

(b) For w/t greater than 12 but not over 30:

$$f_c = (1.67\, f_b - 5430) - (1/18)\ (f_b - 8150)\ w/t$$

(Values of f_c in accordance with formula are given in Table 3.2(b))

(c) For w/t over 30 but not over 60*:

For Angle Struts: $f_c = 7{,}330{,}000/(w/t)^2$

For All Other Sections: $f_c = 12{,}600 - 148.5(w/t)$

In the above formulas, w/t = flat-width ratio as defined in Section 2.2.

TABLE 3.2(b)

Allowable Design Stresses on Unstiffened Elements—Section 3.2(b)
ASTM A245, A246, and A303 Grades of Steel

For w/t Ratios From 12 to 30

w/t	Grade C	Grade B	Grade A
12	18,000	16,500	13,500
14	16,910	15,580	12,910
16	15,810	14,650	12,310
18	14,720	13,720	11,720
20	13,630	12,790	11,130
22	12,530	11,860	10,530
24	11,440	10,940	9,940
26	10,340	10,010	9,340
28	9,250	9,080	8,750
30	8,150	8,150	8,150

TABLE 3.2(c)

Allowable Design Stresses on Unstiffened Elements—Section 3.2(c)

For w/t Ratios from 30 to 60 (All Grades of Steel)

Ratio w/t	Allowable f_c for		Ratio w/t	Allowable f_c for	
	Angle Struts	Other Sections		Angle Struts	Other Sections
30	8150	8150	46	3460	5770
32	7160	7850	48	3180	5470
34	6340	7550	50	2930	5180
36	5660	7250	52	2710	4880
38	5080	6960	54	2510	4580
40	4580	6660	56	2340	4280
42	4160	6360	58	2180	3990
44	3790	6070	60	2040	3690

* Unstiffened compression elements having ratios of w/t exceeding approximately 30 may show noticeable distortion of the free edges under allowable compressive stress without detriment to the ability of the member to support load.

For ratios of w/t exceeding approximately 60 distortion of the flanges is likely to be so pronounced as to render the section structurally undesirable unless load and stress are limited to such a degree as to render such use uneconomical.

3.3—LATERALLY UNBRACED SINGLE WEB BEAMS

To prevent lateral buckling, the maximum compression stress f'_c, in pounds per square inch, on extreme fibers of compression flanges of laterally unsupported straight I-, Z- or channel-shaped flexural members, (not including multiple-web deck, U- and closed box-type members and curved or arch members) shall not exceed the allowable stress as specified in Sections 3.1 or 3.2 nor the following maximum stresses:

(a) For I- or channel-shaped sections: $f'_c = \dfrac{250{,}000{,}000}{(L/r_y)^2}$

(b) For Z-shaped sections: $f'_c = \dfrac{125{,}000{,}000}{(L/r_y)^2}$

where

L = the unbraced length of the member, and

r_y = the radius of gyration of the entire section of the member about its gravity axis parallel to the web.

3.4—ALLOWABLE STRESSES IN WEBS OF BEAMS

3.4.1 Shear Stresses in Webs

The maximum average shear stress, v, in pounds per square inch, on the gross area of a flat web shall not exceed:

$$v = \frac{64{,}000{,}000}{(h/t)^2} \text{ with a maximum of } 2/3\ f_b.$$

where

t = web thickness, in.

h = clear distance between flanges, in.

f_b = basic working stress as specified in Section 3.1, psi.

Where the web consists of two or more sheets, each sheet shall be considered as a separate member carrying its share of the shear.

(Values in accordance with the above formula are given in Table 3.4.1.)

TABLE 3.4.1

Maximum Allowable Shear in Flat Webs

$$v = \frac{64{,}000{,}000}{(h/t)^2} \text{ with maximum of } 2/3\ f_b$$

h/t	v
60	17,780
70	13,060
73	12,000 (maximum for Grade C)
80	10,000
90	7,900
100	6,400
110	5,290
120	4,440
130	3,790
140	3,270
150	2,840

3.4.2 Bending Stress in Webs

The compressive stress f_w in pounds per square inch in the flat web of a beam due to bending in its plane, shall not exceed f_b nor shall it exceed:

$$f_w = \frac{520{,}000{,}000}{(h/t)^2}$$

where h = clear distance between flanges, in.

3.4.3 Combined Bending and Shear Stresses in Webs

For webs subject to both bending and shear stresses, the member shall be so proportioned that such stresses do not exceed the allowable values specified in Sections 3.4.1 and 3.4.2 and that the quantity

$(f'_b/f_w)^2 + (v'/v)^2$ does not exceed unity,

where

$$f_w = \frac{520{,}000{,}000}{(h/t)^2}$$

$$v = \frac{64{,}000{,}000}{(h/t)^2}$$

f'_b = actual compressive stress at junction of flange and web, psi.

v' = actual average shear stress, i.e., shear force per web divided by web area, psi.

3.5—WEB CRIPPLING OF BEAMS

To avoid crippling of flat webs of beams, concentrated loads and reactions shall not exceed the values of P_{max} given below.

(a) Beams of Grade C steel having single unreinforced webs with inside corner radius equal to or less than the thickness of the sheet:

(1) For end reactions or for concentrated loads on the outer ends of cantilevers

$$P_{max} = 100t^2\,[980 + 42(B/t) - 0.22(B/t)(h/t) - 0.11\,(h/t)]$$

For other grades of steel and other corner radii, the value P_{max} given by the above formula is to be multiplied by

$$k(1.15 - 0.15n)(1.33 - 0.33k)$$

(2) For reactions of interior supports or for concentrated loads located anywhere on the span

$$P_{max} = 100t^2\,[3050 + 23(B/t) - 0.09(B/t)(h/t) - 5(h/t)]$$

For other grades of steel and other corner radii, the value P_{max} given by the above formula is to be multiplied by

$$k(1.06 - 0.06n)\,(1.22 - 0.22k)$$

(b) For I-beams made of two channels connected back to back or for similar sections which provide a high degree of restraint against rotation of the web, such as I sections made by welding two angles to a channel:

(1) For end reactions or for concentrated loads on the outer ends of cantilevers

$$P_{max} = t^2 f_b(7.4 + 0.93\sqrt{B/t})$$

(2) For reactions of interior supports or for concentrated loads located anywhere on the span

$$P_{max} = t^2 f_b(11.1 + 2.41\sqrt{B/t})$$

In all of the above P_{max} represents the load or reaction for one solid web sheet connecting top and bottom flanges. For webs consisting of two or more such sheets, P_{max} shall be computed for each individual sheet and the results added to obtain the allowable load or reaction for the composite web.

For loads located close to ends of beams, provision (a-2) and (b-2) apply, provided that for cantilevers the distance from the free end to the nearest edge of bearing, and for a load close to an end support, the clear distance from edge of end bearing to nearest edge of load bearing, is larger than 1.5 h. Otherwise provisions (a-1) and (b-1) apply.

In the above formulas,

P_{max} = allowable concentrated load or reactions, lbs.
t = web thickness, in.
B = actual length of bearing, in inches, except that in the above formulas the value of "B" shall not be taken greater than "h".
h = clear distance between flanges, in.
f_b = basic allowable design stress, psi. (Section 3.1)
k = $f_b/18{,}000$
n = Ratio of inside bend radius divided by web thickness

3.6—AXIALLY LOADED COMPRESSION MEMBERS

3.6.1 Unit Stress

The average axial stress, P/A, in compression members, shall not exceed the values of F_a, as follows: *

For Grade C Steel

L/r equal to or less than $132/\sqrt{Q}$: $F_a = 15{,}300Q - 0.437Q^2(L/r)^2$

L/r equal to or greater than $132/\sqrt{Q}$: $F_a = \dfrac{134{,}000{,}000}{(L/r)^2}$

For other Grades of Steel

L/r equal to or less than $\dfrac{24{,}000}{\sqrt{f_y}\ \sqrt{Q}}$: $F_a = 0.464Qf_y - \left(\dfrac{2Qf_yL/r}{100{,}000}\right)^2$

L/r equal to or greater than $\dfrac{24{,}000}{\sqrt{f_y}\ \sqrt{Q}}$: $F_a = \dfrac{134{,}000{,}000}{(L/r)^2}$

In the above formulas,

P = total load, lbs.;
A = full, unreduced cross-sectional area of the member, in.^{2};
F_a = maximum allowable average axial stress in compression, psi.;
L = unsupported length of member, in.;**
r = radius of gyration of full, unreduced cross-section, in.;
f_y = yield point of steel, psi.;
Q = a factor determined as follows:

(a) For members composed entirely of *stiffened* elements, "Q" is the ratio between the effective design area, as determined from the effective design widths of such elements, and the full or gross area of the cross-section. The effective design area used in determining Q is to be based upon the basic design stress f_b as defined in Section 3.1.

* Values of F_a for different Q and L/r values appear in Chart 4.

** L = For continuous compression chords of trusses with rigid welded connections at panel points, the value of L to be used in computing L/r in the plane of the truss is 3/4 the distance between panel points.

(b) For members composed entirely of *unstiffened* elements, "Q" is the ratio between the allowable compression stress f_c for the weakest element of the cross-section (the element having the largest flat-width ratio) and the basic design stress f_b; where f_c is as defined in Section 3.2 and f_b is as defined in 3.1.

(c) For members composed of both *stiffened and unstiffened* elements the factor "Q" is the product of a stress factor Q_s computed as outlined in (b) above and an area factor Q_a computed as outlined in (a) above, except that the stress upon which Q_a is to be based shall be that value of the unit stress f_c which is used in computing Q_s; and the effective area to be used in computing Q_a shall include the full area of all unstiffened elements. (See Examples in Part III.)

3.6.2 Maximum Slenderness Ratio

The maximum allowable ratio L/r of unsupported length, L, to radius of gyration, r, of compression members shall be as follows:

(a) Columns, and other primary compression members, except as provided otherwise in this Section 120

The slenderness ratio L/r of a main compression member may exceed 120, but not 200, provided its unit stress under full design load does not exceed the following fraction of that stipulated under Section 3.6.1 $\quad 1.6 - (L/200r)$

(b) Load-bearing wall studs 160

The slenderness ratio L/r of a load-bearing wall stud may exceed 160, but not 200, provided its unit stress under full design load does not exceed the following fraction of that stipulated under Section 3.6.1 $\quad 2.6 - (L/100r)$

(c) Secondary members 200

Exception: During construction only, L/r may exceed the foregoing limits but shall not exceed 300.

If members which are temporarily unbraced during construction are to act as permanent load-carrying members in the completed structure they must be so braced prior to completion of the structure as to reduce the L/r ratio to a value not exceeding that given in (a), (b), or (c) above, whichever may apply.

3.7—COMBINED AXIAL AND BENDING STRESSES

Members subject to both axial compression and bending stresses shall be so proportioned that the quantity

$$\frac{f_a}{F_a} + \frac{f'_b}{F_b} \text{ shall not exceed unity,}$$

where

F_a = maximum axial unit stress in compression that is permitted by this Specification where axial stress only exists. (Section 3.6.1)

F_b = maximum bending unit stress in compression that is permitted by this Specification where bending stress only exists. (Section 3.1 and 3.2)

f_a = axial unit stress = axial load divided by full cross-sectional area of member, $\frac{P.}{A}$

f'_b = bending unit stress = bending moment divided by section modulus of member, $\frac{M}{S}$, noting that for members having stiffened compression elements the section modulus shall be based upon the effective design widths of such elements.

3.8—WIND OR EARTHQUAKE STRESSES

3.8.1 Wind or Earthquake only

Members and assemblies subject only to stresses produced by wind or earthquake forces may be proportioned for unit stresses 33⅓ per cent greater than those specified for dead and live load stresses. A corresponding increase may be applied to the allowable unit stresses in connections and details.

3.8.2 Combined Forces

Members and assemblies subject to stresses produced by a combination of wind or earthquake and other loads may be proportioned for unit stresses 33⅓ percent greater than those specified for dead and live load stresses, provided the section thus required is not less than that required for the combination of dead load and live load. A corresponding increase may be applied to the allowable unit stresses in connections and details.

3.9—CYLINDRICAL TUBULAR COMPRESSION MEMBERS

The ratio, D/t, of mean diameter to wall thickness of a cylindrical tubular compression member shall not exceed $3{,}300{,}000/f_y$. For such members, the allowable unit stress P/A under axial load shall be as prescribed by Section 3.6.1, with $Q = 1$.

SECTION 4. CONNECTIONS

4.1—GENERAL

Connections shall be designed to transmit the maximum stress in the connected member with proper regard for eccentricity. In the case of members subject to reversal of stress, except if caused by wind or earthquake loads, the connection shall be proportioned for the sum of the stresses.

4.2—WELDS

4.2.1 Fusion Welds

For all grades of steel, fusion welds shall be proportioned so that the unit stresses therein do not exceed 13,600 psi in shear on the throat of fillet or plug welds. The allowable unit stress in tension or compression on butt welds shall be the same as prescribed for the base metal being joined, provided the weld penetrates 100% of the section. Stresses due to eccentricity of loading, if any, shall be combined with the primary stresses; and the combined unit stresses shall not exceed the values given above.

Stresses in a fillet weld shall be considered as shear on the throat for any direction of the applied stress. Neither plug nor slot welds shall be assigned any value in resistance to any stresses other than shear.

All fusion welding shall comply with the provisions of the Standard Code for Arc and Gas Welding in Building Construction of the American Welding Society, of latest edition, except as otherwise specified herein and excepting such provisions of that Code as are clearly not applicable to material of the thicknesses to which this Specification applies.

4.2.2 Resistance Welds

In Sheets joined by spot welding, the design strength per spot shall be as follows:

Thickness of Thinnest Outside Sheet, In.	Design Strength per Spot, lbs.
.010	50
.020	125
.030	225
.040	350
.050	525
.060	725
.080	1075
.094	1375
.109	1650
.125	2000
.155	2700
.185	3300

(The above values are based upon the American Welding Society's Recommended Practice for Resistance Welding. They are applicable for all structural grades of low carbon steel, up to a yield point of 70,000 lb. per sq. in. and are based on a factor of safety of approximately two and one-half. The welding procedure shall conform to that set forth in the Recommended Practice published by the American Welding Society.)

4.3—WELDS CONNECTING TWO CHANNELS TO FORM AN I-SECTION FOR USE AS A BEAM

The required tension strength of welds connecting two channels to form an I-beam shall be determined from the following formula:

$$S_w = \frac{mqs}{2c}$$

where

S_w = required strength of weld in tension, lbs.

s = longitudinal spacing of welds, in.

c = vertical distance between the two rows of welds near or at top and bottom flanges, in.

q = intensity of load, on beam, lbs. per lin. in.
(For method of determination, see below)

m = distance of shear center of channel from mid-plane of the web, in.
For simple channels without stiffening lips at the outer edges,

$$m = \frac{w^2}{2w + h/3}$$

For C-shaped channels with stiffening lips at the outer edges,

$$m = \frac{wht}{4I_x}\left[wh + 2d\,(h - d)\right]$$

w = projection of flanges from inside face of web, in. (For channels with flanges of unequal width, w shall be taken as the width of the wider flange.)

h = depth of channel or beam, in.

d = depth of lip, in.

I_x = moment of inertia of one channel about its centroidal axis normal to the web, in.^4

The intensity of load, q, is obtained by dividing the magnitude of concentrated loads or reactions by the length of bearing or by longitudinal spacing of welds, s, whichever is larger. For beams designed for "uniformly distributed load," the intensity q shall be taken equal to three times the intensity of the uniformly distributed design load.

The required strength of welds depends upon the intensity of the load directly at the weld. Therefore, if uniform diameter and spacing of welds are used over the whole length of the beam, the necessary strength of the welds shall be determined at the point of maximum local load intensity. In cases where this procedure would result in uneconomically close spacing either one of the following methods may be adopted: (a) the weld spacing may be varied along the beam according to the variation of the load intensity; or (b) reinforcing cover plates may be welded to the flanges at points where concentrated loads occur. The required strength *in shear* of the welds connecting these plates to the flanges shall then be determined from the formula for S_w specified herein but "c" shall then represent the depth of the beam.

4.4—SPACING OF CONNECTIONS IN COMPRESSION ELEMENTS

The spacing, in line of stress, of welds, rivets, or bolts connecting a compression cover plate or sheet to a non-integral stiffener or other element shall not exceed

(a) that which is required to transmit the shear between the connected parts on the basis of the design strength per connection specified elsewhere herein; nor

(b) $s = 6000t/\sqrt{f}$, where s is spacing in inches, t is the thickness of the cover plate or sheet in inches, and f is the design stress in the cover plate or sheet in pounds per square inch; nor

(c) three times the total flat width, w, of the narrowest unstiffened compression element in that portion of the cover plate or sheet which is tributary to the welds, but need not be less than thirty-six times the thickness of such element unless closer spacing is required by (a) or (b) of this Section 4.4.

In the case of intermittent fillet welds parallel to the direction of stress the spacing shall be taken as the clear distance between welds plus one-half inch. In all other cases the spacing shall be taken as the center to center distance between connections.

Exception: The requirements of this Section 4.4 do not apply to cover sheets which act only as sheathing material and are not considered as load-carrying elements.

4.5—BOLTED CONNECTIONS

The following requirements govern bolted connections of light gage steel structural members:

4.5.1 Minimum Spacing and Edge Distance in Line of Stress

The clear distance between bolts which are arranged in rows parallel to the direction of force, also the distance from the center of any bolt to that end or other boundary of the connecting member towards which the pressure of the bolt is directed, shall not be less than 1½ d nor less than $P/f_b t$

where

d = diameter of bolt, in.
P = force transmitted by bolt, lbs.
t = thickness of thinnest connected sheet, in.
f_b = basic design stress, as defined elsewhere, psi.

4.5.2 Tension Stress on Net Section

The tension stress on the net section of a bolted connection shall not exceed f_b nor shall it exceed

$$(0.1 + 3d/s) f_b$$

where

s = spacing of bolts perpendicular to line of stess, in. In the case of a single bolt, s = width of sheet.
d and f_b are as previously defined.

4.5.3 Bearing Stress in Bolted Connections

The bearing stress on the area ($d \times t$) shall not exceed 3.5 f_b

4.5.4 Shearing Stress on Bolts

The allowable value for shear on unfinished bolts is 10,000 lbs per sq. in.

SECTION 5. BRACING REQUIREMENTS

Structural members and assemblies of light gage steel construction shall be adequately braced in accordance with good engineering practice. The following provisions cover certain special cases and conditions.

5.1—WALL STUDS

The safe load-carrying capacity of a stud may be computed on the basis that wall material or sheathing (attached to the stud) furnishes adequate lateral support to the stud in the plane of the wall, provided the wall material and its attachments to the stud comply with the following requirements:

(a) Wall material or sheathing must be attached to both faces or flanges of the studs being braced.

(b) The maximum spacing of attachments of wall material to the stud being braced shall not exceed "a_{max}" as determined from the formula:

$$a_{max} = \frac{8\ E\ I_2\ k}{A^2\ f_y{}^2}$$

where k is the test value as defined in (d) and the other terms are as defined in (c) and (d).

The slenderness ratio of the stud between attachments, a/r_2, shall not exceed $L/2r_1$. Therefore, the spacing of attachments shall not exceed that specified above nor shall it exceed:

$$a_{max} = \frac{L\ r_2}{2\ r_1}$$

where L = length of stud, in.
r_1 = radius of gyration of stud about its axis parallel to wall = $\sqrt{I_1/A}$, in.
r_2 = radius of gyration of stud about its axis perpendicular to wall = $\sqrt{I_2/A}$, in.

(c) The minimum modulus of elastic support, k, to be exerted laterally by the wall material and its attachments in order to brace the stud, shall be not less than —

for Steel of Grade C: $k = \dfrac{4.5\ a\ A^2}{I_2}$;

for Grade B: $k = \dfrac{3.7\ a\ A^2}{I_2}$;

for Grade A: $k = \dfrac{2.6\ a\ A^2}{I_2}$;

for steel of grade other than Grades A, B, and C:

$$k = \frac{f_y^2\ a\ A^2}{240{,}000{,}000\ I_2}$$

where

f_y = yield point of steel in the studs, psi.

a = actual spacing of attachments of wall material to stud measured along the length of stud, (a = 1 for continuous attachment), in.

A = area of cross section of stud, $in.^2$

I_2 = moment of inertia of cross section of stud about its axis perpendicular to wall, $in.^4$

k = spring constant or modulus of elastic support of wall material (on each [one] side of stud) plus attachment, i.e., k = F/y where F is the force in pounds which produces an elongation of y inches of a strip of wall material of width "a" and of length equal to the distance between adjacent studs, lbs per in.*

(d) The lateral force, F, which each single attachment of the wall material shall be capable of exerting on the stud in the plane of the wall (in order to prevent lateral buckling of the stud) shall not be less than:

$$F_{min} = \frac{keP}{2\sqrt{EI_2k/a} - P} \quad *$$

where

k = modulus of elastic support of the wall material and its attachments as determined from tests, i.e., the value of "k" used in the formula to determine the maximum allowable spacing of attachments in (b), lbs per in.

$e = \dfrac{\text{stud length in inches}}{240}$

P = design load on stud, lbs.

I_2 = moment of inertia of stud about its axis perpendicular to the wall, $in.^4$

a = actual spacing of attachments measured along stud, in. (a = 1 inch for continuous attachment).

E = Modulus of Elasticity = 29,500,000 psi.

* Whether a given wall material or means of attachment satisfies the requirements of this Section may be established by the test procedure described in Part II of this Manual. Also included in Part II are a number of "k" values determined by the test procedure described, for several common types of wall sheathing.

5.2—CHANNEL AND Z-SECTIONS USED AS BEAMS

The following provisions for the bracing, against twist, of channel and Z-sections used as beams apply only when (a) neither flange is connected to deck or sheathing material in such a manner as to effectively restrain lateral deflection of the connected flange and (b) such members are loaded in the plane of the web.*

5.2.1 Spacing of Braces

Braces shall be attached both to the top and bottom flanges of the sections at the ends and at intervals not greater than one-quarter of the span length in such a manner as to prevent tipping at the ends and lateral deflection of either flange in either direction at intermediate braces. If one-third or more of the total load on the beams is concentrated over a length of one-twelfth or less of the span of the beam, an additional brace shall be placed at or near the center of this loaded length.

5.2.2 Design of Braces

Each intermediate brace, at top and bottom flange, shall be designed to resist a lateral force P_b determined as follows:

(a) For a uniformly loaded beam, $P_b = 1.5K$ times the load within a distance 0.5a each side of the brace.

(b) For concentrated loads $P_b = 1.0K$ times the concentrated load. P within a distance 0.3a each side of the brace, plus a force F determined from the following formula, for each such concentrated load P located farther than 0.3a, but not farther than *a* from the brace:

$$F = \frac{1.0}{0.7}\left(1 - \frac{x}{a}\right) P\ K$$

In the above formulas:

For channels:

$K = m/h$, where

m = distance from shear center to mid-plane of the web, as specified in Section 4.3, in.

h = depth of channel, in.

For Z-sections:

$K = I_{xy}/I_x$, where

I_{xy} = product of inertia of full section about centroidal axes parallel and perpendicular to web, $in.^4$

I_x = moment of inertia of full section about centroidal axis perpendicular to web, $in.^4$

For channels and Z-sections:

x = distance from concentrated load P to brace, in.

a = length of bracing interval, in.

End braces shall be designed for half of the above forces.

Braces shall be designated as to avoid local crippling at the points of attachment to the member.

---

* When only one flange is connected to a deck or sheathing material to effectively restrain lateral deflection of the connected flange, bracing may or may not be needed to prevent twisting of the member, depending upon the dimensions of the member and span and upon whether the unconnected flange is in compression or tension.

5.2.3 Allowable Stresses

For channel and Z-beams intermediately braced according to the requirements of Sections 5.2.1 and 5.2.2, the maximum compression stress f'_c shall be that specified in Section 3.3, except that the length of the bracing interval, *a*, shall be used instead of the length L in th formulas of that Section.

5.3—LATERALLY UNBRACED BOX BEAMS

For closed box-type sections used as beams the ratio of the laterally unsupported length, L, to the distance between the webs of section shall not exceed 75.

SECTION 6. TESTS FOR SPECIAL CASES

6.1—GENERAL

Where elements, assemblies, or details of structural members formed from sheet or strip steel are such that calculation of their safe load-carrying capacity or deflection cannot be made in accordance with the provisions of Sections 2 through 5 of this Specification, their structural performance shall be established from test procedure as specified in Section 6.2.

6.2—TEST PROCEDURE*

It is recommended that tests for the purposes defined in Section 6.1 be conducted in accordance with the following procedure.

(a) Where practicable, evaluation of test results shall be made on the basis of the mean values resulting from tests of not fewer than three identical specimens, provided the deviation of any individual test result from the mean value obtained from all tests does not exceed $\pm 10\%$. If such deviation from the mean exceeds 10%, at least three more tests of the same kind shall be made. The average of the three lowest values of all tests made shall then be regarded as the result of the series of tests.

(b) Determinations of allowable load-carrying capacity shall be made on the basis that the member, assembly, or connection shall be capable of sustaining during the test without failure a total load, including the weight of the test specimen, equal to twice the live load plus twice the dead load. Furthermore, harmful local distortions shall not develop during the test at a total load, including the weight of the test specimen, equal to the dead load plus one and one half times the live load. For members and assemblies subjected to wind or earthquake loads, appropriate modification of the foregoing factors shall be made in accordance with Section 3.8.

(c) In evaluating test results, due consideration must be given to any differences that may exist between the yield point of the material from which the tested sections are formed and the minimum yield point specified for the material which the manufacturer intends to use.

(d) Tests shall be made by an independent testing laboratory or by a manufacturer's testing laboratory.

* The test procedures and test factors specified in Section 6.2 are not applicable to confirmatory tests of members and assemblies whose properties have been calculated according to Sections 2 through 5; for the latter, the Specification provides generally a safety factor of 1.85.

PART II

SUPPLEMENTARY INFORMATION

1. BASIC LIST OF SYMBOLS

A = cross-sectional area, in.^{2}

a = actual spacing of wall material attachments along stud, in. (Section 5.1)
= length of bracing interval, in. (Section 5.2.2)

B = length of bearing, in. (Section 3.5)
= breadth in tabulated sections, in. (Tables of section properties)

b = effective design width of stiffened elements, in.

b' = effective design width of stiffened sub-element, in. (Section 2.3.1.2)

c = distance from neutral axis to extreme fiber, in.
= amount of flange curling, in. (Section 2.3.3)
= spacing between two rows of welds joining webs of two channels, in. (Section 4.3)

D = mean diameter of cylindrical tubes, in. (Section 3.9)
= depth of tabulated sections, in. (Tables of section properties)

d = depth of stiffening lip, in.
= diameter of bolt, in. (Sections 4.5.1, 4.5.2)

E = modulus of elasticity, 29,500,000 psi.

F_a = maximum average axial stress in compression permitted where axial stress alone exists, psi. (Sections 3.6.1, 3.7)

F_b = maximum bending stress in compression permitted where bending stress alone exists, psi. (Section 3.7)

f = stress, psi.

f_a = axial load divided by cross-sectional area, psi. (Section 3.7)

f_{av} = average stress in flanges subject to curling, psi. (Section 2.3.3)

f_b = basic allowable design stress, psi.

f_c = maximum allowable compressive stress in unstiffened elements, psi.

f'_c = maximum allowable compressive stress in flanges of beams subject to lateral buckling, psi. (Section 3.3)

f_w = maximum allowable compressive stress in the flat web of a beam, psi. (Sections 3.4.2, 3.4.3)

f_y = yield stress, psi.

h = depth of beams and channels, in. (Sections 2.3.3, 4.3, 5.2.2)
= clear distance between flanges, in. (Sections 2.3.4, 3.4.1, 3.4.2, 3.5)

I = moment of inertia, in.^{4}

I_s = moment of inertia of a multiple stiffened element, in.^{4} (Section 2.3.2.2)

k = $f_b/18{,}000$ (Section 3.5)

k = modulus of elastic support of wall material and its attachments, lbs. per in. (Section 5.1)

P = force or concentrated load, lb.

Q = form factor, i.e. stress and/or area factor used to modify beam and column loads. (Section 3.6.1)

q = intensity of loading along beam, lb. per linear in. (Section 4.3)

L = length, in.

M = bending moment, in.-lb.
m = distance of shear center of a channel from the mid-plane of its web, in. (Sections 4.3, 5.2.2)
n = inside bend radius divided by thickness, in. (Section 3.5)
R = inside bend radius, in.
r = radius of gyration, in.
S = section modulus (I/c), in.^{3}
S_w = required strength of weld in tension, lb. (Section 4.3)
s = spacing of welds, rivets or bolts, in. (Sections 4.3, 4.4, 4.5.2)
t = thickness of element, in.
t_s = equivalent thickness of a multiple stiffened element, in. (Section 2.3.2.2)
V = total shear force, lb.
v = maximum allowable average shear stress in a web, psi. (Sections 3.4.1, 3.4.3)
v′ = shear force divided by web cross-sectional area, psi. (Section 3.4.3)
W = total load, lb.
w = flat width of element, in.
= projection of flange from inside face of web, in. (Section 4.3 only)

2. EXPLANATORY COMMENTS ON THE DESIGN SPECIFICATION

The Design Specification recognizes two types of compression elements, as follows:

Unstiffened elements, which are flat elements having one unstiffened edge parallel to the direction of stress; for example, the compression flange of a plain channel or I-section having no lip or stiffener at the outstanding edges.

Stiffened elements, those in which both edges parallel to the direction of stress are stiffened by connection to a web, flange, stiffening lip or other stiffening elements; for example, the top (compression) flange of a hat- or inverted U-section, or the lipped flange of a channel or I-section. See Chart 1.

Light gage steel members having *stiffened* elements are far more effective in supporting load than *unstiffened* members of comparable dimensions. The strength-to-weight ratio of the former may easily be twice that of the latter. Hence, economic reasons dictate the use of stiffened compression elements where feasible.

UNSTIFFENED COMPRESSION ELEMENTS — REDUCED WORKING STRESSES

When dealing with unstiffened compression elements, cognizance must be taken of the fact that such elements will develop sudden buckling at predictable unit stresses. Those stresses may be less than the yield strength of the steel when the flat-width ratio w/t is greater than 12. Therefore, to avoid such buckling, the design of unstiffened elements with w/t exceeding 12 is based on a reduced allowable unit stress. This allowable unit stress decreases with increasing w/t. The values for these allowable stresses are given in Tables 3.2b and 3.2c of Section 3.2 of the Specification. These reduced allowable unit stresses are used with the structural properties of the full, unreduced cross-section of the members. They are reflected in Tables 2, 3 and 5.

STIFFENED COMPRESSION ELEMENTS — REDUCED CROSS-SECTION

Stiffened compression elements behave quite differently from *unstiffened* compression elements under load. Under low compressive stress, the entire area and width of the stiffened element is effective. As the unit stress increases, particularly in elements where the flat-width ratio is high, the portion in the center of the element (most remote from the stiffened edges) tends to develop slight buckling waves and becomes relatively less effective in resisting compressive stresses than the portions adjacent to the stiffeners. Therefore, under the design procedure of the Specification, safe load determinations for sections having *stiffened* compression elements are made by considering that only the portions of any such element adjacent to its stiffened edges are structurally effective; the properties of such sections are based upon this reduced *effective* portion. See Charts 3A and 3B.

It will be observed, in Section 2.3.1 of the Design Specification, that the values of effective width that are prescribed for safe load determination are different from those prescribed for deflection (or stiffness) determination. The reason for this is that, for accuracy, the computation of deflection should be based upon the width which is effective under the stress caused by the actual applied load. On the other hand, since effective width varies inversely with the stress, the computation of safe load-carrying capacity must be based upon the width which is effective at a theoretical stress 1.85 (the factor of safety) times the stress actually caused by the load. Accordingly, the load formulas and Charts 3A and 3B actually give b/t ratios for unit stresses 1.85 times those shown; whereas, the deflection formulas and Charts 3C and 3D give the b/t ratios for the actual stresses.

VARIABLE PROPERTIES OF SECTIONS HAVING STIFFENED ELEMENTS

The width of stiffened compression elements which is considered effective varies not only with the ratio of width to thickness of the element but also with the unit stress in the element. Since the actual unit stress in a flexural member is a function of the bending moment, the effective width, as well as the moment of inertia, varies with the bending moment. If the accompanying unit compression stress is known, the effective width and the section properties can be calculated directly. This is the case for allowable load determinations if the distance from the neutral axis to the compression fiber is larger than that to the tension fiber, in which case the allowable compression stress is known to be f_b. If the reverse is the case, it is the allowable tension stress which is equal to f_b and the compression stress depends on the position of the neutral axis. This position depends on the effective width of the compression flange which, however, depends in turn on the compression stress. A similar situation, where the compression stress is not known in advance, obtains when computing moments of inertia for determining deflections under design loads. Such calculations require a series of successive approximations which is illustrated in Examples Nos. 6, 7, 14 and 16. The fact that some section properties depend on the magnitude of the compression stress is reflected in Tables 1, 4 and 7.

COMPRESSION MEMBERS

In the case of compression members (columns and struts) the procedure for considering the reduced strength of unstiffened elements and the reduced effective width of stiffened elements has been simplified by the introduction of a form factor, "Q." (Q is defined in Section 3.6.1, Design Specification.)

In some compression members excessive local deformation of certain component elements can occur at loads considerably below the maximum load which the member as a whole can carry. The Specification provides a margin of safety against such excessive local deformation.

The important point for the designer to bear in mind in considering all of the foregoing is that all of these apparent departures from usual practice are really only mechanical devices intended to facilitate the application of variable stresses to *unstiffened* elements and variable effective widths to *stiffened* elements.

3. 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 center line 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 $= L \cdot t$, where L is the total length of all line elements.

The moment of inertia of the section, I, is found from the relation:

$I = I' \cdot t$, where I′ is the moment of inertia of the center line of the steel sheet. The section modulus is computed as usual by dividing I or $I' \cdot t$ by the distance from the neutral axis to *the extreme fiber,* not to the center line 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.

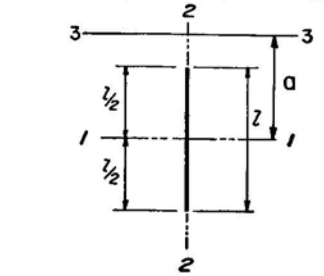
When the flat width, w, of a stiffened compression element is reduced for design purposes, the effective design width, b, is used directly to compute the total effective length L_{eff} of the line elements, as shown in Example No. 15.

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 formulas on the facing page.

The formulas 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:

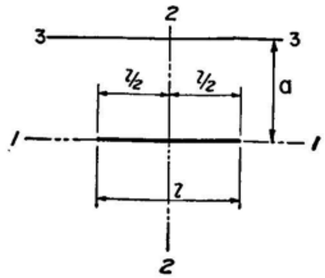
1. The moment of inertia of a straight actual element about its longitudinal axis is considered negligible.
2. The moment of inertia of a straight actual element inclined to the axes of reference is slightly larger than that of the corresponding line element, but for elements of like length the error 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. (See Table 9, showing *actual* properties of a 90° corner.) When the mean radius of a circular element

PROPERTIES OF LINE ELEMENTS



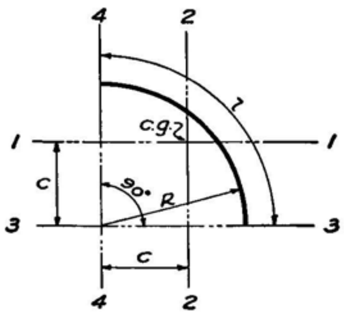
$$I_1 = \frac{l^3}{12} \qquad I_2 = 0$$

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



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

$$I_3 = la^2$$

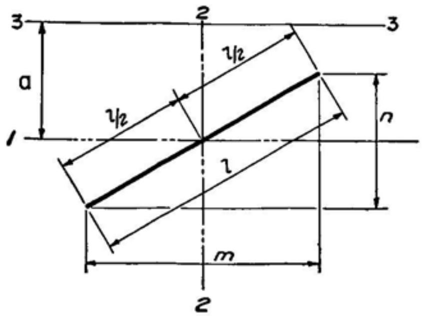


$$l = 1.57 \ R$$

$$c = 0.637 \ R$$

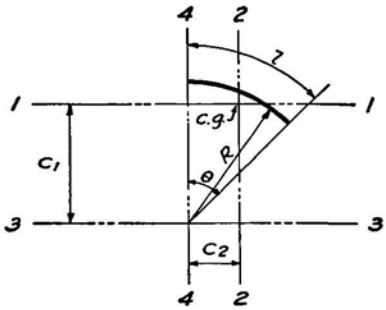
$$I_1 = I_2 = 0.149 \ R^3$$

$$I_3 = I_4 = 0.785 \ R^3$$



$$I_1 = \frac{ln^2}{12} \qquad I_2 = \frac{lm^2}{12}$$

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



θ (expressed in radians) $= 0.01745 \times \theta$ (expressed in degrees and decimals thereof).

$$l = \theta R$$

$$c_1 = \frac{R \sin \theta}{\theta} \qquad c_2 = \frac{R(1 - \cos \theta)}{\theta}$$

$$I_1 = \left[\frac{\theta + (\sin \theta)(\cos \theta)}{2} - \frac{(\sin \theta)^2}{\theta}\right] R^3 \qquad I_3 = \left[\frac{\theta + (\sin \theta)(\cos \theta)}{2}\right] R^3$$

$$I_2 = \left[\frac{\theta - (\sin \theta)(\cos \theta)}{2} - \frac{(1 - \cos \theta)^2}{\theta}\right] R^3 \qquad I_4 = \left[\frac{\theta - (\sin \theta)(\cos \theta)}{2}\right] R^3$$

is over four times its thickness, as for tubular sections and for sheets with circular corrugations, the error in using linear arc properties practically disappears.

Examples No. 13 to 16, inclusive, illustrate the application of the linear method.

4. WALL-BRACED STUDS

It is standard practice in the design of stud framing to recognize the lateral support afforded by the collateral materials attached to the studs to form partitions or walls.

Wall-braced panels in which the columns or studs are cold-formed, lightweight shapes of sheet or strip steel now have increasingly wide application for structural purposes. In such panels the collateral wall material also provides the steel column or stud with lateral support in the plane of the wall. Design criteria to establish the necessary qualifications of collateral materials which are to serve as lateral bracing of wall studs are established in those provisions. Example No. 11 illustrates the use of the Design Specification.

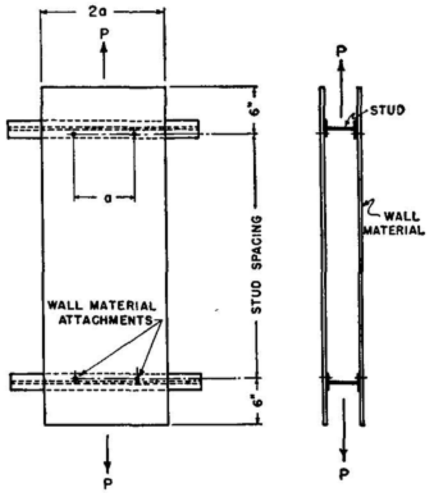
Wall materials generally are selected for properties other than their ability to provide lateral support to the stud members to which they are attached. For that reason, they generally are capable of providing a degree of elastic support far in excess of the minimum needed for bracing the stud. In determining the maximum allowable spacing of the attachments of the wall material to the studs which it braces, the actual k value of the wall material (as determined by tests described under the heading that follows) should be used in Formula 5.1 (b), when it exceeds the minimum k values necessary to afford adequate lateral bracing, Section 5.1 (c).

The effectiveness of the attachments usually is the critical design factor affecting the lateral bracing qualifications of the wall material.

TESTING OF WALL MATERIALS FOR LATERAL BRACING VALUE

Where wall material is utilized as lateral bracing of wall stud members in accordance with Section 5 of this Specification, its minimum properties necessary to perform that function adequately may be established by the following testing procedure.

Specimens — Test specimens shall be made up from two strips of the wall material, the width of each being equal to twice the spacing (along the stud) of the attachments which are to be used. Where continuous attachment of the wall sheathing to the studs is contemplated, the width of specimen shall be not less than 6 inches. Two short pieces of studs of the type to be used shall be attached to the wall material at a distance equal to the stud spacing of the actual installation. See sketch below. The attachments shall be located symmetrically with respect to the center lines of the specimen of wall material. In order to simulate conditions under which wall material is continuous and is attached to studs without joints, the wall material shall protrude 6 inches beyond the stud on each side. In order to simulate conditions under which wall material is not continuous, a joint shall be made by attaching to the stud on its free edge a strip of wall material 6 inches wide and by joining it to the wall material that connects the studs in the manner proposed for use in the structure. The number of specimens to be tested should be as specified in Section 6 of the Design Specification.



Wall Material Test Assembly

Testing Procedure—The test specimen assemblies described above shall be tested in tension in a standard testing machine. By means of suitable connections, the loads shall be applied concentrically to each assembly, through the studs.

An initial load P_o of the order of 50 lb. shall be applied to the assembly. The load shall then be increased by uniform increments. At each increment the distances between the attachments in each of the four pairs shall be measured to the nearest thousandth of an inch. For specimens with continuous attachment the distance between the studs proper shall be measured at each increment. This procedure shall be continued until the specimen fails.

Evaluation of Tests—The elongations corresponding to each load shall be computed by subtracting the original distance between attachments at the load P_o from the distance measured at any particular load. The elongations so obtained for the four pairs of attachments shall be averaged for any particular load, and a curve of the average elongations (in inches) versus applied load (in pounds) shall be drawn. From this curve the elongation y_1 corresponding to the reference load $P_1 = 0.75\ P_{ultimate}$ shall be obtained. The modulus of support of one attachment, k, is then determined for individual attachments from

$$k = \frac{P_1 - P_o}{4y_1}$$

and for continuous attachment from

$$k = \frac{P_1 - P_o}{2y_1 b}$$

where, b = the width of the wall material specimen, in inches, and the other factors are as described above.

The k value so determined shall be equal to or larger than the minimum value of k required in Section 5.1 (c).

The strength of one attachment, F, shall be determined from the ultimate test load, P_{ult}, for one individual attachment from the formula:

$$F = \frac{P_{ult}}{4}$$

and for continuous attachment from

$$F = \frac{P_{ult}}{2b}$$

where, b = width of continuously bonded wall material in inches.

k Values as Determined by Tests — The tabulation below indicates the approximate magnitude of k values of a few common types of wall sheathing materials, tested in accordance with the procedure outlined above using one type of steel stud and two alternate types of attachment.

		Range of k Values
1/2″	Standard Density Wood and Cane Fiber Insulating Boards	290 — 603
1/2″	Paper Base Insulating Board	915 — 1460
3/8″	Gypsum Board Sheathing	775 — 1535
3/16″	Medium Density Compressed Wood Fiber Board	2010 — 4560
5/32″	High Density Compressed Wood Fiber Board	3960 — 7560

The above values are indicative only; the k values for any specific construction will depend upon the particular type of wall sheathing material and the method of attachment employed, also the type of steel stud to which attachment is made.

5. RIBBED STEEL ROOF DECK

Steel roof decks are designed to support insulation and roofing while carrying specified roof loads. Therefore, the commonly used types have decidedly unsymmetrical cross sections with a flat deck (wide compression flange) for support of the roofing material and shallow longitudinal ribs for stiffness and strength. In the case of an unbalanced section having less metal in the tension flange than in the compression flange, the basic unit design stress, f_b, will limit the stress in tension. However, the actual (and lower) unit stress in compression will govern the effective design width of the section and thus the structural properties of the deck. Hence, since the structural properties vary with the (unknown) actual compression stress, which is less than the (known) tension or allowable basic design stress, the actual compression stress can be determined precisely only by successive trials. (This is illustrated in Example 6, Part III.)

Since most ribbed steel roof decks have similar profiles, with shallow longitudinal ribs spaced not over 6″ on center, and are used on spans having end supports spaced not over 10′ apart, a simplified design procedure, described below, has been developed. This design procedure is adequate for application to conventional ribbed steel roof decks only because of their similarity in cross sections.

While actual cross-sectional dimensions of the commercially available ribbed steel roof decks vary somewhat, it has been found sufficiently accurate to use the following limiting values for the effective design width of the wide top compression flange between ribs:

Thickness of Deck Metal	Width of Top Flange Effective
#18 U. S. Standard Gage	3/4 of clear flat width of flange
#20 U. S. Standard Gage	5/8 of clear flat width of flange
#22 U. S. Standard Gage	1/2 of clear flat width of flange

This approach greatly simplifies the design or checking of designs of ribbed steel roof deck.

As explained above, the tabulated effective width values are only applicable to steel roof decks which have unbalanced cross sections with flat top deck and shallow longitudinal ribs spaced not over 6" on center; they cannot be applied to other sections formed of light gage steel.

6. THICKNESS LIMITATIONS

In the design of light steel members, proper proportioning of the component parts, consistent with their width to thickness ratio and their unit stress, is the factor of importance to assure safe structural performance; the thickness of the metal itself is, per se, not a critical factor. Members formed of extremely thin steel will function satisfactorily, if designed in accordance with the procedure prescribed in the Design Specification and in this Manual.

No minimum thickness limitations are necessary to assure sound structural behavior. In building construction, however, it is not unusual to specify certain minimum thickness limitations based upon prevailing practices, practical considerations, and experience in handling standard products in the field. The following provision for light steel constructions used within buildings, based on Bulletin V Steel Regulations, published by American Iron and Steel Institute, is typical of regulatory procedure:

> Thickness: Steel used to form individual structural (load-carrying) members, shall be of thickness not less than 18 U. S. Standard Gage, provided that the use of material of less thickness may be allowed upon the submission of test data from approved authorities verifying the structural behavior of the members formed from such material.
>
> Exception: Steel used to form load-carrying panels, including ribbed steel roof deck constructions, shall be of thickness not less than 22 U. S. Standard Gage.

7. PROTECTION OF FORMED LIGHT GAGE STEEL MEMBERS

Formed light gage steel members are normally protected against corrosion by the application of manufacturer's standard shop coat of paint, or by the use of galvanized sheets. Steel members protected in this fashion have a long record of satisfactory performance. It is usually not practicable nor does it represent standard practice to paint these constructions after their installation, unless for decorative purposes where they are exposed to view, or where exposed to the weather. (Shop painting is, of course, unnecessary when the members are porcelain enameled or have other special protective coating or treatment.)

The excellent record of performance of light gage steel construction covers tremendous quantities of formed light gage steel sections, such as cellular floors, roof decks, joists, standard steel building, etc. Examples of its durability can be found in constructions going back to the mid-nineteenth century. With the advent of continuous rolling mills and their increased ability to turn out steel sheets rapidly, structural members formed of sheets have been utilized in machines, mechanical equipment, airplanes and building construction. In many cases these structural members have been subjected to conditions of severe exposure, shock, vibration and loading, far more severe than the conditions usually prevailing in building construction.

In 1940-42, the Pittsburgh Testing Laboratory conducted a survey to determine whether the painting practices of the industry had provided effective protection to light gage steel members. Fifty installations were inspected in locations where severe climatic conditions prevail. The light gage steel members inspected were generally in excellent condition, all were structurally sound, and, quote, "appeared to justify the conclusion that commercial protective paint coatings applied to light steel structural members of the type included in this survey, provide effective protection to the steel in actual practice and that such light steel structural members may be expected to retain their structural properties during the life of the building, when enclosed within the confines of the building." Copies of the Laboratory's report, entitled, "The Durability of Lightweight Types of Steel Construction," are available from American Iron and Steel Institute upon request.

Another report of an independent study made by the Engineering Research Institute of the University of Michigan also is available; it confirms the foregoing conclusions. Details of that investigation are published in University of Michigan Bulletin No. 30, "Durability of Light Weight Steel Construction."

PART III

ILLUSTRATIVE EXAMPLES

The following examples are intended to illustrate the application of various provisions of the Design Specification, and the use of the Tables of Properties and Charts appearing in this Manual. In the Table below are shown the Table, Chart or Section of the Design Specification used in each of the design examples.

DESCRIPTION OF ILLUSTRATIVE EXAMPLES

EXAMPLE No. 1

Beam strength and deflection (using Table 4).

GIVEN: 1. STEEL: Grade C ($f_b = 18{,}000$ psi)

2. SPAN: 12′—0″; beam continuously braced along top flanges

3. LOADS:

LIVE LOAD:	120 lbs. per lin. ft.
DEAD LOAD, incl. weight of beam:	40 lbs. per lin. ft.
TOTAL LOAD:	160 lbs. per lin. ft.

4. Maximum deflection caused by live load: Not to exceed 1/360 of span or 0.40 in.

REQUIRED: A stiffened type of I-section which will satisfy both beam strength and beam deflection requirements.

SOLUTION:

(a) Beam strength requirement:

$$\text{Applied Moment, } M = \frac{160 \times 12^2 \times 12}{8} = 34{,}560 \text{ in.-lbs.}$$

$$\text{Required } S = \frac{M}{f_b} = \frac{34{,}560}{18{,}000} = 1.92 \text{ in.}^3$$

(b) Deflection requirement:

$$\text{Required } I = \frac{5WL^3}{384\,E\triangle} = \left[\frac{5\text{x}\ (12\text{x}120)\text{x}(12\text{x}12)^3}{384 \text{ x } 29{,}500{,}000 \text{ x } 0.40}\right] = 4.73 \text{ in.}^4$$

From Table 4 the following sections can be selected:

$6 \times 5 \times$ No. 16 ga. $S_x = 2.61$ in.^{3}, $I_x = 8.02$ in.^{4}, wt $= 4.92$ lbs per ft.
or $5 \times 4 \times$ No. 14 ga. $S_x = 2.24$ in.^{3}, $I_x = 5.60$ in.^{4}, wt $= 5.06$ lbs per ft.

EXAMPLE No. 2

Beam strength and deflection (using Table 5).

GIVEN: Same as Example No. 1

REQUIRED: An unstiffened type of I section which will satisfy the requirements of Example No. 1.

(a) Beam strength requirement:

$$\text{Applied Moment, } M = \frac{160 \times 12^2 \times 12}{8} = 34{,}560 \text{ in.-lbs.}$$

1st Trial:

Try $7 \times 3 \times$ No. 16 ga. double channel section (Table No. 5)

$I_x = 7.08$ in.^{4}, $S_x = 2.02$ in.^{3}

$$f_c = \frac{M}{S_x} = \frac{34{,}560}{2.02} = 17{,}100 \text{ psi}$$

This section is unsatisfactory since the beam stress is greater than the allowable beam stress, $f_c = 13{,}460$ psi.

2nd Trial:

Try $7 \times 3 \times$ No. 14 ga. double channel section (Table 5)

$I_x = 9.08$ in.^{4}, $S_x = 2.60$ in.^{3}

$$f_c = \frac{34{,}560}{2.60} = 13{,}290 \text{ psi}$$

This section is satisfactory since the beam stress is less than the allowable beam stress, $f_c = 15{,}190$ psi.

(b) Beam deflection requirement:

$$\text{deflection} = \Delta = \frac{5\ WL^3}{384\ E\ I} = \frac{5\ (12 \times 120)\ (12 \times 12)^3}{384 \times 29{,}500{,}000 \times 9.08} = 0.21 \text{ in.}$$

This section is satisfactory since the live load deflection is less than 0.40 in.

EXAMPLE No. 3

Beam strength and deflection (using Table 6).

GIVEN: 1. STEEL: Grade C ($f_b = 18{,}000$ psi)

2. SPAN: 6′—0″; section continuously braced laterally.

3. LOADS:

LIVE LOAD — 120 lbs. per lin. ft.

DEAD LOAD — beam weight

4. Maximum deflection caused by live load: not to exceed L/240 or 0.30 in.

REQUIRED: A double angle section which will satisfy beam strength and deflection requirements.

SOLUTION:

(a) Beam strength requirements:

$$\text{Live load moment} = M = \frac{120 \times 6^2 \times 12}{8} = 6480 \text{ in.-lbs.}$$

From table 6 it is seen that the lightest sections which will carry the specified total load weigh approximately 5 lbs. per foot.

$$\text{Approximate dead load moment} = \frac{5 \times 6^2 \times 12}{8} = 270 \text{ in.-lbs.}$$

Total design moment $= 6480 + 270 = 6750$ in.-lbs.

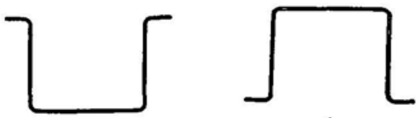
(b) Beam deflection requirements:

$$\text{Required I} = \frac{5 \times (120 \times 6) \times (6 \times 12)^3}{384 \times 29{,}500{,}000 \times 0.30} = 0.394 \text{ in.}^4$$

From Table 6 the following sections can be selected to satisfy both of these requirements:

Joined Legs Up	Joined Legs Down
4 x 4 x No. 10 ga.	3 x 3 x No. 12 ga.
3 x 3 x No. 10 ga.	2½ x 2½ x No. 10 ga.
2½ x 2½ x No. 10 ga.	

EXAMPLE No. 4



a b

Beam strength and deflection (using Table 7).

GIVEN: 1. STEEL: Grade C ($f_b = 18{,}000$ psi)

2. SPAN: L = 12 ft., section continuously braced laterally.

3. LOAD:

LIVE LOAD: 400 lbs. per lin. ft.

DEAD LOAD, incl. weight of beam 40 lbs. per lin. ft.

4. Maximum deflection caused by live load shall not exceed L/240 of the span, or 0.60 in.

REQUIRED: A hat section which will satisfy both beam strength and beam deflection requirements.

SOLUTION:

(a) Beam strength requirement:

$$\text{Applied Moment, M} = \frac{440 \text{ x } 12^2 \text{ x } 12}{8} = 95{,}040 \text{ in.-lbs.}$$

$$\text{Required Section Modulus} = \frac{\text{Applied Moment}}{f_b} = \frac{95{,}040}{18{,}000} = 5.28 \text{ in.}^3$$

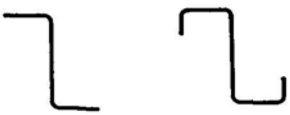
If the member is to be used with the closed flange in tension (figure a above), the section in Table 7 having the following nominal dimension will satisfy the beam strength requirement for the designated live and dead loads:
from 10 x 15 x No. 10 ga. to 10 x 10 x No. 12 ga.
also 8 x 12 x No. 10 ga. and 8 x 12 x No. 12 ga.
If the member is to be used with the closed flanges in compression, (figure b above), the beam strength requirements will be satisfied by the following sections:
from 10 x 15 x No. 10 ga. to 10 x 10 x No. 12 ga.
and 8 x 12 x No. 10 ga.

(b) Beam deflection requirement:

$$\text{Required I} = \frac{5 \text{ x } (400 \text{ x } 12) \text{ x } (12 \text{ x } 12)^3}{384 \text{ x } 29{,}500{,}000 \text{ x } .60} = 10.4 \text{ in.}^4$$

From Table 7 it is seen that all of the above listed sections will meet the deflection limitations.

EXAMPLE No. 5



Beam strength, web shear and web crippling (using Table 1 or 2).

GIVEN: 1. STEEL: Specified minimum yield 50,000. (f_b = 27,000 psi)
2. SPAN: L = 16 ft.
3. LOAD: 1500 lb. concentrated load at midspan
4. DEFLECTION: No limitation

REQUIRED: 1. The lightest Z-beam of adequate strength selected from Table 1 or 2, f_b = 27,000 psi.
2. Check adequacy, web in shear, for the above loading and the weight of the beam. (Section 3.4.1)
3. A check to investigate whether 1″ of bearing at both ends of the beam will be sufficient to assure against web crippling at these points. (Section 3.5)
4. The maximum allowable spacing of lateral bracing for the section selected. (Sections 3.3 and 5.2)

SOLUTION: 1. Beam strength requirement:

Applied Moment exclusive of weight of beam =

$$\frac{PL}{4} = \frac{1500 \times 16 \times 12}{4} = 72{,}000 \text{ in.-lbs.}$$

$$\text{Required section modulus} = \frac{72{,}000}{27{,}000} = 2.67 \text{ in.}^3$$

The following Zee's (with stiffened flanges) of Table 1, of nominal dimensions as follows, will meet the beam strength requirement for the designated load:

from 12 x 3½ x No. 10 ga. to 9 x 3¼ x No. 14 ga.
8 x 3 x No. 10 ga. and 8 x 3 x No. 12 ga.
7 x 2¾ x No. 10 ga. and 7 x 2¾ x No. 12 ga.
also 6 x 2½ x No. 10 ga.

The following Zee (with unstiffened flanges) of Table 2, of nominal dimensions as follows, will meet the beam strength requirement for the designated load:

8 x 2 No. 10 ga.

However, the beam strength must provide for the dead weight of the selected section in addition to that of the concentrated load. Investigating this feature confirms that all these sections will satisfy the beam strength requirement.

A satisfactory section so selected is the 7 x 2¾ x No. 12 ga. Zee with stiffened flanges. It is this section which will be investigated further.

2. Web Shear Requirement:

In accordance with Section 3.4.1 of the Design Specification maximum average shear stress, $v = \dfrac{64{,}000{,}000}{(h/t)^2}$ but shall not exceed $(2/3)\ f_b$.

$$v = \frac{64{,}000{,}000}{(6.79/.105)^2} = 15{,}300 \text{ psi which is less than}$$

2(27,000)/3 and therefore governs. The total allowable shear on web is, therefore, $v = 6.79 \times 0.105 \times 15{,}300$ psi = 10,900 lbs.

Since this value is substantially greater than the actual shear occasioned by the combination of concentrated load and beam weight, i.e., 789 lbs., the section is satisfactory from this standpoint.

3. Check end bearing:

In accordance with Section 3.5 (a) (1) of the Design Specification:

$$\begin{aligned} P_{max} &= 100t^2[980 + 42\,(B/t) - 0.22(B/t)(h/t) - 0.11\,(h/t)] \\ &= 100(0.105)^2\,[980 + 42\,(1/0.105) - 0.22\,(1/0.105) \\ &\quad (6.415/0.105) - 0.11\,(6.415/0.105)] = 1370 \text{ lbs.} \end{aligned}$$

This value, however, must be corrected to reflect the effect of a radius of bend greater than that of the thickness of the web and a minimum yield steel of 50,000 psi which is specified. Doing

this in accord with the aforementioned Section 3.5(a)(1):
Allowable reaction $= k\ (1.15 - 0.15\ n)(1.33 - 0.33\ k)\ P_{max}$
$= 1.500\ (1.15 - 0.15 \times 1.79)(1.33 - 0.33 \times 1.500) \times 1370$
$= 1.095 \times 1370 = 1500$ lbs.

$$\text{where } k = \frac{f_b}{18{,}000} = \frac{27{,}000}{18{,}000} = 1.5$$

$$n = \frac{\text{inside corner radius}}{t} = \frac{0.1875}{0.105} = 1.79$$

The value of 1500 lbs. is considerably greater than the reaction of 789 lbs. realized at each end of the beam under the loads imposed. It is apparent that the section is satisfactory in this respect.

4. Beam lateral brace spacing requirements:
The maximum allowable lateral brace spacing for the selected section as determined by the Design Specification, Section 3.3(b) is:

$$L = r_y \sqrt{\frac{125{,}000{,}000}{f'_c}}$$

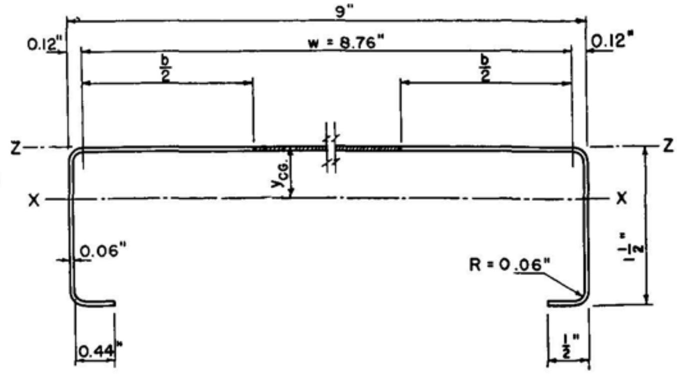
$$f'_c = \frac{M}{S_x} = \frac{73{,}890}{2.98} = 24{,}795 \text{ psi.}$$

$$r_y = 1.30 \text{ in (from Table 1)}$$

$$L = 1.30 \sqrt{\frac{125{,}000{,}000}{24{,}795}} = 92.3 \text{ in.}$$

However, Section 5.2.1 states that top and bottom flanges must be braced at the ends and at intervals not to exceed one quarter of the span, i.e., $16 \times 12/4 = 48$ in. This specification also requires that a similar brace shall be provided at the point of application of a concentrated load. Therefore, a brace will be provided every 4 feet.

EXAMPLE No. 6



GIVEN: **Beam strength (using Chart 3A).**

1. STEEL: Grade C ($f_b = 18{,}000$)
2. Section illustrated above.

REQUIRED: The resisting moment of this section as governed by bending stress.

SOLUTION:

1st Approximation:

For a first approximation assume a compressive bending stress in the top compressive element of 10,000 psi. (It is obvious that because of the larger amount of metal in the compressive flange the neutral axis will be closer to the top flange and hence the stress in the top flange will be considerably less than 18,000 psi when the bottom tension flange is so stressed.) From this an effective design width is determined in accord with Section 2.3.1.1 of the Design Specifications as follows:

$\frac{w}{t} = \frac{8.76}{0.06} = 146$. From Chart 3A, the effective design width, b, exclusive of corners $= 66.2t = 66.2 \times 0.06 = 3.97$ in.

Total effective width $= 3.97 + 2(0.06) = 4.09$ in., web to web.

Cross-sectional properties (assuming square corners)

Element	Area A (in.^{2})	y Dist. from top fiber (in.)	(A)y (in.^{3})	A(y)^2 (in.^{4})
Top Flange	$4.09 \times 0.06 = 0.2454$	0.03	0.0074	0.0002
Webs	$2 \times 1.50 \times 0.06 = 0.1800$	0.75	0.1350	0.1012
Bottom Flanges	$2 \times 0.44 = 0.06 = 0.0528$	1.47	0.0776	0.1141
Summation	0.4782		0.2200	0.2155

$$+ I_{cg} \text{ of webs} = \frac{2 \times 0.06 \times (1.5)^3}{12} = 0.0338$$

$$0.2493 = I_z$$

$$-0.478 \times 0.460^2 = -0.1012$$

$$\text{Moment of inertia: } I_x = 0.1481$$

Distance of neutral axis from top fiber:

$$y_{cg} = \frac{0.2200}{0.4782} = 0.460 \text{ in.}$$

Since the stress in any element will vary directly as its distance from the neutral axis and since the bottom, tensile elements are stressed to 18,000 psi the compressive stress in the top flange will be:

$$f'_c = \frac{0.460}{1.500 - 0.460} \times 18{,}000 = 7960 \text{ psi}$$

Due to the position of the neutral axis the compressive stress is considerably less than the assumed 10,000 psi, when the tensile stress in the bottom flange is equal to the working stress, 18,000 psi. Hence, it is desirable to correct the original assumption for the compressive stress. This will affect the effective area of the top flange and thereby the position of the neutral axis. It is therefore desirable to locate the neutral axis accurately by successive approximations.

2nd Approximation:

Guided by the determination in the first approximation:
assume $y = 0.44$ in.
Then the compressive stress in the top flange:

$$f_c = \frac{18{,}000 \times 0.44}{1.50 - 0.44} = 7470 \text{ psi}$$

From Chart 3A, for this f_c and $w/t = 146$:
$b = 74.0t - 74.0 \times 0.06 = 4.440$
Total effective width $= 4.440 + (2 \times 0.06) = 4.56$ in.
Check assumed position of neutral axis (assuming square corners):

Element	Area A (in.^{2})	y (in.)	(A)y (in.^{3})
Top Flange	$4.56 \times 0.06 = 0.2736$	0.03	0.0082
Webs	$2 \times 1.50 \times 0.06 = 0.1800$	0.75	0.1350
Bottom Flange	$2 \times 0.44 \times 0.06 = 0.0528$	1.47	0.0776
Summation	0.5064		0.2208

Distance of axis from top fiber: $y_{cg} = \dfrac{0.2208}{0.5064} = 0.436$ in.

3rd Approximation:

On the basis of the approximation above assume
$y = 0.434$ in.

$$f_c = \frac{18{,}000 \times 0.434}{1.500 - 0.434} = 7330 \text{ psi}$$

From Chart 3A, for this f_c and $w/t = 146$:
$b = 74.9t = 74.9 \times 0.06 = 4.49$ in.
Total effective width $= 4.49 + 2 \times 0.06 = 4.61$ in.

Cross-sectional properties (assuming square corners):

Element	Area A (in.^{2})	y (in.)	(A)y (in.^{3})	(A)y^2 (in.^{4})
Top Flange	$4.61 \times 0.06 = 0.2766$	0.03	0.0083	0.0002
Webs	$2 \times 1.50 \times 0.06 = 0.1800$	0.75	0.1350	0.1012
Bottom Flanges	$2 \times 0.44 \times 0.06 = 0.0528$	1.47	0.0776	0.1141
Summation	0.5094		0.2209	0.2155

$$+ I_{cg} \text{ (of webs)} = \frac{2 \times 0.06 \times (1.5)^3}{12} = 0.0338$$

$$0.2493 = I_z$$

$$-0.5094 \times 0.434^2 = -0.0959$$

$$\text{Moment of inertia: } I_x = 0.1534 \text{ in.}^4$$

Distance of axis from top fiber:

$$y_{cg} = \frac{0.2209}{0.5094} = 0.434 \text{ in. (This checks with value assumed)}$$

$$\text{Section Modulus: } S_x = \frac{I_x}{c} = \frac{0.1534}{1.500 - 0.434} = 0.144 \text{ in.}^3$$

Resisting moment $M = 0.144 \times 18{,}000 = 2590$ in.-lbs.

EXAMPLE No. 7

Beam deflection (using Chart 3C).

GIVEN: Same as Example No. 6.

REQUIRED: Moment of Inertia for computing deflection. (Section 2.3.1). Section loaded to capacity Resisting Moment as determined in Example No. 6.

SOLUTION:

1st Approximation:

Since the top flange stress for load determination is 7330 psi as found in Example No. 6, that stress will be used for a first approximation in computing the effective width of the top flange for use in deflection determinations.

$w/t = 146$ (as in Example No. 6).

From Chart 3C, the effective design width, b, exclusive of corners:

$$= 95.3t = 95.3 \times 0.06 = 5.72 \text{ in.}$$

Total effective width $= 5.72 + (2 \times 0.06) = 5.84$ in. web to web.

Properties computed in the same manner as in Example No. 6 are:

Moment of Inertia: $I_x = 0.164 \text{ in.}^4$

Distance of neutral axis from top fiber: $y_{cg} = 0.383$ in.

Stress in top flange: $f_c = \dfrac{2590 \times 0.383}{0.164} = 6050$ psi.

2nd Approximation:

Guided by determination of the first approximation, it is assumed that the compressive stress in the top flange is 5600 psi.

From Chart 3C, $b_, = 105.3\ t = 6.32$ in.

Total effective width $= 6.32 + (2 \times 0.06) = 6.44$ in.

Corresponding properties are:

Distance of neutral axis from top fiber: $y_{cg} = 0.362$ in.

Stress in top flange: $f_c = \dfrac{2590 \times 0.362}{0.1683} = 5570$ psi.

3rd Approximation:

On the basis of the determination above, assume $f_c = 5570$ psi.

Following Section 2.3.1.1, Design Specification, b/t is calculated:

$\dfrac{b}{t} = \dfrac{10{,}320}{\sqrt{5570}}\left[1 - \dfrac{2580}{146\sqrt{5570}}\right] = 105.7$ (or this value could also be found using Chart 3C).

The effective design width, $b = 105.7 \times 0.06 = 6.34$ in.

Total effective width $= 6.34 + (2 \times 0.06) = 6.46$ in.

Cross-sectional properties (assuming square corners):

Element	Area A (in.^{2})	y (in.)	(A)y (in.^{3})	(A)y^2 (in.^{4})
Top Flange	$6.46 \times 0.06 = 0.3876$	0.03	0.0116	0.0003
Webs	$2 \times 1.50 \times 0.06 = 0.1800$	0.75	0.1350	0.1012
Bottom Flanges	$2 \times 0.44 \times 0.06 = 0.0528$	1.47	0.0776	0.1141
Summation	0.6204		0.2242	0.2156

$$+ \ I_{cg} \text{ (of webs)} = \frac{2 \times 0.06 \times (1.5)^3}{12} = 0.0338$$

$$0.2494 = I_z$$

$$-\,0.6204 \times 0.362^2 = -\,0.0813$$

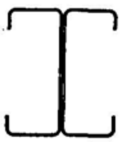
$$\text{Moment of Inertia: } I_x = 0.1681 \text{ in.}^4$$

Stress in top flange:

$$f_c = \frac{2590 \times 0.362}{0.1681} = 5570 \text{ psi (checks assumed stress)}$$

A comparison of the value of 0.1681 in.^{4} for the moment of inertia, I (for deflection) with the value of 0.1534 in.^{4} which was used in the stress calculations of Example No. 6 shows that the deflection, as determined, will actually be approximately 10% less than if computed on the basis of the same moment of inertia used in the stress calculations. It will also be noted that the first approximation of the deflection determinations gave a value of I which was within approximately 2% of the correct value.

EXAMPLE No. 8



Axially loaded primary compression member (using Table 4 and Chart 4).

GIVEN:

1. STEEL: Grade C ($f_b = 18{,}000$ psi)
2. Unsupported Length, L= 12′—6″ = 150″
3. AXIAL LOAD = P = 30,000 lb.

REQUIRED: An appropriate section as shown in the sketch.

For a column or primary support, L/r must not exceed 120 (Section 3.6.2), except as provided otherwise in that section. Therefore the allowable unit stress for the member under investigation is: $P/A = 15{,}300\,Q - 0.437Q^2 \left(\frac{L}{r}\right)^2$ in accordance with Section 3.6.1.

Since r_y is less than r_x for all double channel (stiffened) sections in Table 4, r_y will govern. Since r_y must equal at least 150/120 or 1.25 in., the section, according to Table 4, must be among those of nominal dimensions from 12 x 7 x No. 10 ga. to 7 x 5½ x No. 16 ga. inclusive. For these sections r_y ranges from 1.26 in. min. to 1.60 in. max.

For all the sections cited above, Q ranges from 0.588 to 0.923 inclusive.

The allowable unit stress, P/A, therefore will not be greater than $[(15{,}300 \times 0.923) - 0.437 \times 0.923^2 \times (150/1.60)^2]$ or 10,850 psi and will not be less than $[15{,}300 \times 0.588 - 0.437 \times 0.588^2 \times (150/1.26)^2]$ or 6855 psi. (These values can be obtained directly from Chart 4). The average of these values, 8850 psi, may be used for tentatively selecting a section which must then be tested by application of the formula:

$$\text{allowable unit stress} = 15{,}300 \times Q - 0.437Q^2(L/r)^2$$

Assuming P/A value of 8850 psi for tentative selection:

$$\text{Area of section } A = \frac{30{,}000}{8{,}850} \text{ (approx.)} = 3.39 \text{ sq. in. (approx.)}$$

A section whose area most nearly approximates 3.39 sq. in. is the double channel section of nominal dimensions 7 x 5½ x No. 10 ga., with an area = 3.54 sq. in.; Q = 0.923; r_y = 1.29.

Checking this section:

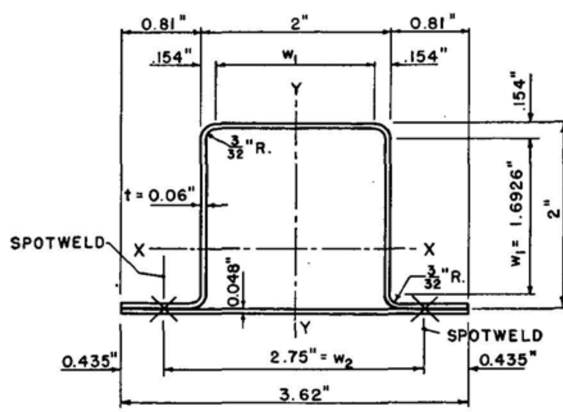
From Chart 4, with Q = 0.923 and L/r = 150/1.29 = 116.3 the corresponding P/A value is 9088 psi.

Allowable axial load:

$$P = \text{Area} \times \text{Allowable unit stress} = 3.54 \times 9088 = 32{,}170 \text{ lbs.}$$

The section tentatively selected is therefore suitable for use. Other sections falling either side of this section in Table 4, if considered desirable for use, may be similarly checked.

EXAMPLE No. 9a



Axially loaded compression member — allowable load, grade C steel (using Chart 3B and Chart 4).

GIVEN:

1. STEEL: Grade C: f_b = 18,000 psi
   f_y = 33,000 psi
2. Section consisting of a hat and flat plate, as shown.

REQUIRED: Allowable axial load, P, for the section indicated if (I) length $L = 10$ ft., (II) length $L = 6$ ft.

SOLUTION: From Table 7 the following full section properties of the hat section are obtained for a 2 x 2 x No. 16 ga. hat section:

$A = 0.437$ in.^{2} $\quad I_y = 0.444$ in.^{4}

$I'_x = 0.265$ in.^{4} $\quad y = 1.04$ in.

By calculation the properties of the 18 ga. plate are:

$A = 3.62 \times 0.048 = 0.174$

$I_x = 0.000$

$I_y = 0.048\ (3.62^3)/12 = 0.190$

Calculating the combined properties:

Element	Area A (in.^{2})	y Distance from bottom fiber (in.)	(A)y (in.^{3})	(A)y^2 (in.^{4})
Hat section	0.437	1.088	0.475	0.517
Plate	0.174	0.024	0.004	—
Summation	0.611		0.479	0.517

$$\text{Distance of axis from bottom fiber} = y_{cg} = \frac{0.479}{0.611} = 0.785 \text{ in.}$$

$$\text{I Hat Section} = 0.265 \text{ in.}^4$$

$$\text{Summation } (A)y^2 = \underline{0.517}$$

$$0.782$$

$$-\text{ Summation } A(y_{cg}^2) = -0.611 \times 0.785^2 = -\underline{0.377}$$

$$0.405 \text{ in.}^4$$

$$I_y = 0.444 + 0.190 = 0.634$$

$$r_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{0.405}{0.611}} = 0.81 \text{ in.}$$

It is assumed that, by means of spot welding at appropriate intervals, the flat plate is sufficiently attached to the hat so that the lines of welds are also lines of stiffening. (See Section 4.4 of the Design Specification.)

The value of Q (Section 3.6.1, Design Specification) is determined as follows:

$$\frac{w_1}{t_1} = \frac{1.692}{0.06} = 28.20$$

$$\frac{w_2}{t_2} = \frac{2.750}{0.048} = 57.29$$

Effective design widths for $f_b + 18{,}000$ psi according to Chart 3B or Section 2.3.1.1, Design Specification are:

$b_1 = w_1 = 1.692$ in.

$b_2 = 42.59t = 42.59 \times 0.048 = 2.044$ in.

Then $A_{eff} = 0.611 - [(2.750 - 2.044) \times 0.048]$

$= 0.611 - 0.034 = 0.577$ in.^{2} •

Since the member is composed entirely of stiffened elements:

$$Q = \frac{\text{Effective Area}}{\text{Gross Area}} = \frac{0.577}{0.611} = 0.944$$

(I) Length L = 10 ft.

Slenderness ratio $= \frac{L}{r_x} = \frac{120}{0.81} = 148$

In accordance with Section 3.6.1 Design Specification and as can be determined from Chart 4, the unit stress is computed:

$\frac{132}{\sqrt{Q}} = \frac{132}{\sqrt{0.944}} = 136$ and since L/r is greater than 136:

$P/A = \frac{134{,}000{,}000}{(L/r)^2} = \frac{134{,}000{,}000}{(148)^2} = 6118$ psi acting on the gross section and:

$P = 6118 \times 0.611 = 3738$ lbs. which is the allowable load for a secondary member.

But, assuming this is a primary compression member and since L/r exceeds 120, in accordance with Section 3.6.2(a) of the Design Specification.

$$P' = [1.6 - (L/200r)]\ P$$
$$= \left[1.6 - \frac{120}{200 \times 0.81}\right] 3445$$
$$= (0.86) \times 3738 = 3215 \text{ lbs.}$$

(II) Length L = 6 ft.

$\frac{L}{r_x} = \frac{72}{0.81} = 88.9$

In accordance with Section 3.6.1 and as can be determined from Chart 4, the unit stress is computed:

$\frac{132}{\sqrt{Q}} = \frac{132}{\sqrt{0.944}} = 136$ and since L/r is less than 136:

$$P/A = 15{,}300Q - 0.437Q^2\ (L/r)^2$$
$$= 15{,}300\ (.944) - 0.437\ (.944^2)\ (88.9^2)$$
$= 11{,}366$ psi acting on the gross section and:

$P = 11{,}366 \times 0.611 = 6945$ lbs. which is the allowable load for a main or secondary member since L/r is less than 120.

EXAMPLE No. 9b

Axially loaded compression member — allowable load, grade A steel (using Chart 3B and Chart 4).

GIVEN:

1. STEEL: Grade "A" ($f_b = 13{,}500$ psi; $f_y = 25{,}000$ psi)
2. Section as in Example 9a.

REQUIRED: Allowable axial load, P, for the section indicated, if (I) Length, L = 10 ft; (II) Length, L = 6 ft.

SOLUTION: The full section properties and the flat width ratios are:

$A = 0.611$ in.^{2}; $r_x = 0.81$ in; $\frac{w_1}{t_1} = 28.2$; $\frac{w_2}{t_2} = 57.3$

For $f_b = 13{,}500$ psi the effective design widths for the material indicated are from Chart 3B.

$b_1 = w_1 = 28.2\ t_1 = 1.692$ in.

$b_2 = 46.7\ t_2 = 46.7 \times 0.048 = 2.242$ in. Thus the effective area:

$$A_{eff} = 0.611 - [(2.750 - 2.242) \times 0.048]$$
$$= 0.611 - 0.024 = 0.587 \text{ in. and}$$
$$Q = \frac{0.587}{0.611} = 0.961$$

(I) Length, $L = 10$ ft.

Slenderness ratio: $L/r_y = 120/0.81 = 148$

In accordance with Section 3.6.1 Design Specification and as can be determined from Chart 4, the unit stress is computed:

$$\frac{24{,}000}{\sqrt{f_y}\ \sqrt{Q}} = \frac{24{,}000}{\sqrt{25{,}000}\ \ \sqrt{0.961}} = \frac{24{,}000}{158.1 \times 0.98} = 154.9$$

Since L/r is less than 154.9:

$$P/A = 0.464\ Q\ f_y - \frac{4.0Q^2\ (f_y)^2\ (L/r)^2}{10{,}000{,}000{,}000}$$

$= 11{,}148 - 5057 = 6090$ psi

and:

$P = 6090 \times 0.611 = 3720$ lbs. which is the allowable load for a secondary member.

But if this is a main compression member, since L/r exceeds 120, in accordance with Section 3.6.2 (a) of the Design Specification.

$$P' = [1.6 - (L/200r)]\ P$$
$$= \left[1.6 - \frac{120}{200 \times 0.81}\right]\ 3720$$
$$= 0.86 \times 3720 = 3200 \text{ lbs.}$$

(II) Length, $L = 6$ ft.

$L/r_x = 72/0.81 = 88.9$

In accordance with Section 3.6.1 and as can be determined from Chart 4, the unit stress is computed:

$$\frac{24{,}000}{\sqrt{f_y}\ \sqrt{Q}} = \frac{24{,}000}{\sqrt{25{,}000}\ \sqrt{0.961}} = 154.9$$

Since L/r is less than 154.9:

$$P/A = 0.464\ Q\ f_y - \frac{4.0Q^2\ (f_y^2)\ (L/r)^2}{10{,}000{,}000{,}000}$$
$$= 11{,}148 - 1824 = 9324 \text{ psi}$$

and:

$P = 9324 \times 0.611 = 5700$ lbs. which is the allowable load for a main or secondary member since L/r is less than 120.

EXAMPLE No. 9c

Axially loaded compression member — allowable load, steel 50,000 yield point (using Chart 3B and Chart 4).

GIVEN:

1. STEEL: Specified minimum yield 50,000 psi
   ($f_b = 27{,}027$ psi)
2. Section as in Example No. 9a

REQUIRED: Allowable axial load, P, for the section indicated if (I) length, L = 10 ft; (II) Length, L = 6 ft.

SOLUTION: The full section properties and the flat width ratios are:

$$A = 0.611 \text{ in.}^2;\ r_x = 0.81 \text{ in.};\ \frac{w_1}{t_1} = 28.2;\ \frac{w_2}{t_2} = 57.3$$

For $f_b = 27{,}027$ psi the effective design widths of the material indicated are from Chart 3B:

$$b_1 = 27.24t_1 = 27.24 \times 0.06 = 1.63 \text{ in.}$$
$$b_2 = 36.94t_2 = 36.94 \times 0.048 = 1.77 \text{ in.}$$

thus the effective area:

$$A_{eff} = 0.611 - [3 \times (1.693 - 1.63)\ 0.06] - [(2.750 - 1.77)\ 0.048]$$
$$= 0.611 - 0.011 - 0.047 = 0.553 \text{ in.}^2$$

Since the member is composed entirely of stiffened elements:

$$Q = \frac{0.553}{0.611} = 0.905$$

(I) Length, L = 10 ft.

Slenderness ratio: $L/r_x = 120/0.81 = 148$

In accordance with Section 3.6.1 of the Design Specification, or as could be determined from Chart 4, the unit stress is computed:

$$\frac{24{,}000}{\sqrt{f_y}\ \sqrt{Q}} = \frac{24{,}000}{\sqrt{50{,}000}\ \sqrt{0.905}} = 113$$

and since L/r is greater than the above:

$$P/A = \frac{134{,}000{,}000}{(L/r)^2} = \frac{134{,}000{,}000}{(148)^2}$$
$$= 6118 \text{ psi acting on the gross area of the section and:}$$

$P = 6118 \times 0.611 = 3738$ lbs. which is the allowable load for a secondary member.

But, if this is a main compression member and since L/r exceeds 120, in accordance with Section 3.6.2 (a) of the Design Specification.

$$P' = [1.6 - (L/200r)]\ P$$
$$= \left[1.6 - \frac{120}{200 \times 0.81}\right] 3738$$
$$= 0.86 \times 3738 = 3215 \text{ lbs.}$$

(II) Length, L = 6 ft.

$$L/r_x = 72/0.81 = 88.9$$

In accordance with Section 3.6.1 and as can be determined from Chart 4, the unit stress is computed:

$$\frac{24{,}000}{\sqrt{f}\ \sqrt{Q}} = \frac{24{,}000}{\sqrt{50{,}000}\ \sqrt{0.905}} = 113$$

Since L/r is less than 113:

$$P/A = 0.464\ Q\ f_y - \frac{4.0Q^2\ \ (f_y^2)\ \ (L/r)^2}{10{,}000{,}000{,}000}$$

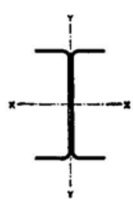
$$= 21{,}000 - 4710 = 16{,}290 \text{ psi}$$

and:

$P = 16{,}290 \times 0.611 = 9950$ lbs. which is the allowable load for a main or secondary member since L/r is less than 120.

NOTE: Comparison of results of Examples No. 9a, b and c shows that the yield strength of the steel used has a significant influence on the strength of columns with relatively low L/r ratios, but has very little effect on relatively slender columns.

EXAMPLE No. 10



Column subjected to combined axial and bending stresses (using Table 5).

GIVEN:

1. STEEL: Grade C (f_b = 18,000 psi.)
2. LENGTH 8′-4″ (Section is adequately braced about the Y-Y axis, unbraced about the X-X axis.)
3. AXIAL LOAD: P = 10,000 lbs.
4. APPLIED MOMENT: about the X-X axis: M_x = 25,000 in.-lbs.

REQUIRED: A column (Table 5) or primary member adequate to support axial load and bending.

SOLUTION: For a column or primary support, L/r must not exceed 120 except as otherwise provided for in Section 3.6.2, Design Specification in Part I. The column is adequately braced about the Y-Y axis, consequently r_x will govern and must be at least 100/120 or 0.833 in. The section must therefore be selected from among those sections which have r_x values greater than 0.833 in.

If bending stresses alone were present, the required section modulus, S_x, would be

$$\frac{25{,}000}{18{,}000} = 1.39 \text{ in.}^3$$

For initial trial selection, a section having about twice this section modulus is taken: 2×1.39 in.^{3} = 2.78 in.^{3} The section whose section modulus, S_x, most nearly approximates this value is the section of nominal dimensions 6 x 3 No. 12 ga. for which S_x = 2.80 in.^{3},

$Q = 0.871$; r_x = 2.17 in., f_x = 18,000 psi; area = 1.80 sq. in.

Checking this section (Section 3.6, Design Specification):

Allowable axial unit stress, $F_a = 15{,}300 \times Q - 0.437Q^2(L/r_x)^2$
$= (15{,}300 \times 0.871) - [0.437 \times (.871^2)\ (100/2.17)^2]$
$= 13{,}326 - 704 = 12{,}620$ psi (or see Chart 4)

Allowable bending unit stress in compression $= F_b = f_b = 18{,}000$ psi

$$\text{Axial unit stress, } f_a = \frac{\text{axial load}}{\text{Area of Section}} = P/A = \frac{10{,}000}{1.80} = 5560 \text{ psi}$$

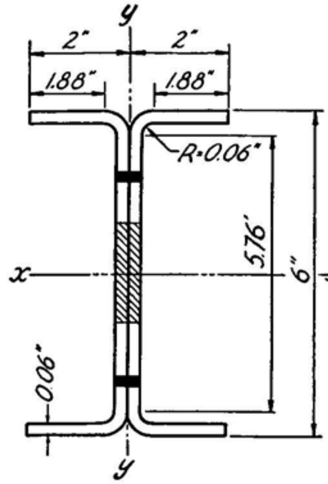
$$\text{Bending unit stress, } f'_b = \frac{\text{Applied Moment}}{\text{Section Modulus}} = \frac{25{,}000}{2.80} = 8930 \text{ psi.}$$

Checking the above with Section 3.7, of the Design Specification:

$$\frac{f_a}{F_a} + \frac{f'_b}{F_b} = \frac{5556}{12{,}620} + \frac{8930}{18{,}000} = 0.440 + 0.496 = 0.936$$
$$= 0.933$$

Since 0.936 does not exceed unity, the section tentatively selected is suitable for use. Other sections near this in Table 5 may be checked similarly. In this example, the first section selected for investigation was the lightest section that appeared to qualify.

EXAMPLE No. 11



Wall stud braced by wall-sheathing — axial compression member (stiffened and unstiffened elements).

GIVEN:

1. STEEL: Specified Minimum Yield: 40,000 psi ($f_b = 21{,}622$ psi)
2. LENGTH: $L = 15$ feet.
3. A section consisting of a stud faced on both flanges by wall sheathing of adequate strength to brace said section.

REQUIRED: (a) The allowable design axial load and (b), the maximum spacing of wall sheathing attachments.

SOLUTION:

(a) Allowable design axial load:

In accordance with Section 3.6.1, of the Design Specification, the cross-sectional properties are to be computed on the basis of the full section, hence

$A = 1.159$ in.^{2} $\qquad I_y = 0.641$ in.^{4}
$I_x = 6.04$ in.^{4} $\qquad r_y = 0.74$ in.
$r_x = 2.28$ in.

The value of Q, (Section 3.6.1), is determined as follows:

The member is composed of both stiffened and unstiffened elements. Paragraph (c) of Section 3.6.1. therefore applies.

For the flanges, $w/t = \dfrac{1.88}{0.06} = 31.3$

Since the flanges are unstiffened elements, from Section 3.2:

$f_c = 12{,}600 - (148.5 \times 31.3) = 7950$ psi, whence: $Q_s = \dfrac{7950}{21{,}622} = 0.368$

For the webs (stiffened elements):

$$w/t = \frac{5.76}{0.06} = 96$$

b/t, (Section 2.3.1.1, and Chart 3A), based on $f = 7950$ psi, equals 66.3.
Since: $w - b = (96 - 66.3)\ t = 29.7 \times 0.06 = 1.78$ in., the corresponding area reduction (for two webs) is: $1.78 \times 0.06 \times 2 = 0.214$ in.^{2}
The net area of the stud is: $1.159 - 0.214 = 0.945$ in.^{2}

Thus: $Q_a = \dfrac{0.945}{1.159} = 0.816$

Then: $Q = Q_s \times Q_a = (0.368 \times 0.816) = 0.300$

According to Section 5.1 (b) the slenderness ratio about the Y-Y axis shall not exceed half of that about the X-X axis; hence the latter governs and

$$\frac{L}{r_x} = \frac{15 \times 12}{2.28} = 78.9$$

From the Column Design Curves, Chart 4, with

$$Q_{eff} = Q\ \frac{f_y}{33{,}000} = 0.300 \times \frac{40{,}000}{33{,}000} = 0.364$$

$P/A = 5210$ psi
and: $P = 5210 \times 1.159 = 6040$ lbs.

(b) Spacing of wall sheathing attachments:

Allowable spacing of wall-sheathing attachments (assuming a test-determined value of $k = 1000$) from Section 5.1 (b):

$$a = \frac{8\ EI_2 k}{A^2\ f_y^2} = \frac{8 \times 29{,}500{,}000 \times 0.641 \times 1000}{(1.159)^2 \times (40{,}000)^2} = 70.4 \text{ in.}$$

Section 5.1 (b) further limits the spacing of wall-sheathing attachments such that the maximum allowable spacing becomes:

$$a_{max} = \frac{L\ r_2}{2\ r_1} = \frac{15(12) \times 0.74}{2 \times 2.28} = 29.2 \text{ in.}$$

Therefore assume that a spacing, a = 30 in. will be chosen which exceeds the computed value by less than 3% and results in uniform spacing of attachments along the stud.

Section 5.1, (d) further states that each attachment will have to be capable of resisting a force not less than F_{min} determined as follows:

$$F_{min} = \frac{keP}{2\sqrt{EI_2k/a} - P} = \frac{1000 \times 0.75 \times 6040}{2\sqrt{29{,}500{,}000 \times 0.641 \times 1000/30} - 6040}$$
$$= 104 \text{ lbs.}$$

USE OF TABLES FOR OTHER GRADES OF STEEL

See description of charts and tables on page 63.

The beam strength properties, S_x and S_y, are in many cases identical in value for the two grades of steel covered. For those sections these same values apply for any grade of steel. Wherever there is a difference between the value tabulated under $f_b = 18{,}000$ psi, and the corresponding value tabulated under $f_b = 27{,}000$ psi, a value for any other grade of steel can be obtained by straight-line interpolation or extrapolation based on the two tabulated values. So long as the extrapolation is not extended to values of f_b lower than 13,500 psi (Grade A steel) or above 30,000 psi (steel with a specified minimum yield of 55,500 psi) the interpolated or extrapolated values will be satisfactorily accurate.

EXAMPLE No. 12a

(Table 4, Double Channels with Stiffened Flanges.)

GIVEN:

1. STEEL: Grade A ($f_b = 13{,}500$ psi)
2. Section from Table 4, nominal dimensions 7 x 5½ x No. 16 ga.

REQUIRED: Find the Section Modulus about the Major Axis, S_x.

SOLUTION:

From Table 4: $S_x = 3.33$ in.^{3} ($f_b = 18{,}000$) and 3.18 in.^{3} ($f_b = 27{,}000$)

$$S_x \text{ (for } f_b = 13{,}500) = 3.33 + \left((3.18 - 3.33)\frac{(13{,}500 - 18{,}000)}{27{,}000 - 18{,}000}\right)$$
$$= 3.33 + 0.08 = 3.41 \text{ in.}^3$$

The deflection properties, I_x and I_y, do not vary with the grade of steel used and therefore no interpolation or extrapolation is required.

The column form factor Q, is determined in exactly the same manner as beam strength properties, as explained above, and the limits of extrapolation for satisfactory accuracy are the same.

EXAMPLE No. 12b

(Table 1, Zee with Stiffened Flanges.)

GIVEN:

1. STEEL: Specified minimum yield 40,000 psi ($f_b = 21{,}622$ psi)
2. Section from Table 1, Zee with nominal dimensions 9 x 3¼ x No. 14 ga.

REQUIRED: The column form factor, Q.

SOLUTION: From Table 1: Q = 0.672 for f_b = 18,000 and 0.603 for f_b = 27,000

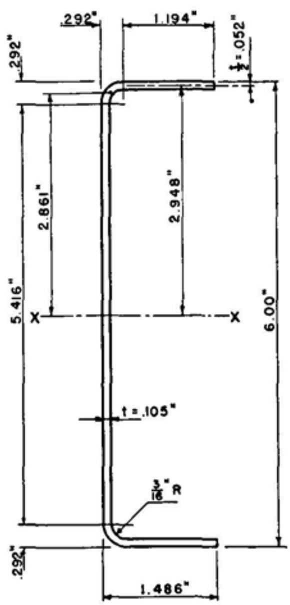
Hence, interpolating by the procedure discussed on Page 64:

Q for f_b = 21,622 psi =

$$0.672 + \left((0.603 - 0.672) \times \frac{(21{,}622 - 18{,}000)}{27{,}000 - 18{,}000}\right)$$

$$= 0.672 - 0.028 = 0.644$$

EXAMPLE No. 13



Beam strength using linear method.

GIVEN:

1. STEEL: Any Grade.
2. SECTION: 6 x 1½ (nominal) x No. 12 ga. channel with Unstiffened Flanges.

REQUIRED: The Section Modulus, S_x, measured about the Major Axis of this section.

SOLUTION: Properties of 90° Corner: (See Page 25)

Rad. of center of corner $R = 0.1875 + \frac{0.105}{2} = 0.240$ in.

Length of Arc, $L = 1.57 \times 0.240 = 0.377$ in.

I′ of corner about its centroidal axis is negligible.

Distance of C.G. from the center of radius: $c = 0.637 \times 0.240 = 0.153$ in.
Flat Width of Flange: $1.486 - 0.292 = 1.194$ in.
Flat Width of Web: $6.000 - (2 \times 0.292) = 5.416$ in.

Distance from X-X Axis to Center of Flange: $3.00 - \dfrac{0.105}{2} = 2.948$ in.

Distance from X-X Axis to C.G. of Corner: $\dfrac{5.416}{2} + 0.153 = 2.861$ in.

COMPUTATION of Linear I'_x:

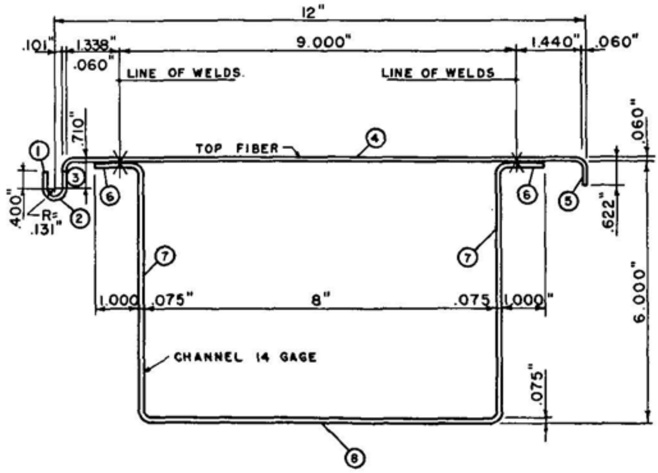
WEB:	$1/12 \times (5.416)^3$	$= 13.24$ in.^{3}
CORNERS:	$2 \times 0.377 \times (2.861)^2$	$= 6.17$
FLANGES:	$2 \times 1.194 \times (2.948)^2$	$= 20.75$
	I'_x	$= 40.16$ in.^{3}

Actual $I_x = I'_x \times t = 40.16 \times 0.105 = 4.22$ in.^{4}

$$S_x = \frac{4.22}{3.00} = 1.41 \text{ in.}^3$$

These values are identical with those in Table 2 of the Manual for this section.

EXAMPLE No. 14



Composite Section, Computation of Properties.

It is proposed to investigate one of a family of sections to determine section modulus, moment of inertia, allowable reactions, and necessary welding.

GIVEN:

1. A section consisting of a channel shape of 14 gage material and a flat plate consisting of 16 gage material, dimensioned and assembled as in sketch.
2. STEEL: Grade C ($f_b = 18{,}000$ psi)
3. Flat plate is in compression, and channel shaped element is on tension side.

REQUIRED: 1. The section modulus to be used in beam strength investigations. (Section 2.3.1)

2. The moment of inertia to be used in beam deflection investigations when loaded to capacity as determined by (1) preceding. (Section 2.3.1)
3. The allowable reaction for 3″ of bearing for (A) an end reaction and for (B) an interior reaction where the panel is continuous over the support. (Section 3.5)
4. The maximum longitudinal weld spacing to allow the line of welds to be considered a line of stiffening for the flat plate. (Section 4.4)
5. The maximum allowable shear using the weld spacing as determined by (4) preceding. (Section 4.2.2)

SOLUTION: 1. Determination of section modulus "S" for use in beam strength calculations:
(Using linear method, and assuming square corners)

1st APPROXIMATION:

A. Flat Plate:

For a first approximation assume a compression stress of 18,000 psi in the top plate. According to Section 2.3.1.1, Design Specification:

$\frac{w}{t} = \frac{9.000}{0.060} = 150$. From Chart 3A the effective design width of the flat plate between welds $= 52 \times 0.060 = 3.120$ in. Total effective design width top flange $= 3.120 + 1.338^* + 1.440^* = 5.898$ in.

* NOTE: Since the w/t ratios, 1.338/0.060 and 1.440/0.060, do not exceed $(w/t)_{lim} = 3790/\sqrt{f} = 3790/\sqrt{18{,}000} = 28.2$ no reduction in effective width is required.

Element No.	L Effective Length (in.)	y (Dist. from top fiber) (in.)	(L)y (in²)	(L)y² (in³)
1	0.400	0.510	0.204	0.104
2	0.412	0.794	0.327	0.260
3	0.710	0.355	0.252	0.089
4	5.898	0.030	0.177	0.005
5	0.622	0.311	0.193	0.060
Summation	8.042		1.153	0.518

Distance of axis from top fibre $= y_{cg} = \frac{1.153}{8.042} = 0.143$ in.

Linear I's of elements about their respective c.g.'s.

$$\begin{aligned}
I'_1 &= 0.400^3/12 &&= 0.005 \\
I'_2 &= 2 \times 0.149 \times 0.143^3 &&= 0.001 \\
I'_3 &= 0.701^3/12 &&= 0.030 \\
I'_5 &= 0.622^3/12 &&= 0.020 \\
\text{Summation} & &&= 0.056 \\
\text{Summation } (L)\ y^2 & &&= 0.518 \\
& && \quad 0.574
\end{aligned}$$

$$I'_{top\ fiber} = 0.574$$

Minus Summation (L) $y^2_{cg} = -8.042 \times 0.143^2 = -0.164$ in.^{3}

Linear I′ Flat Plate $= 0.410$ in.^{3}

Actual I Flat Plate $= 0.060 \times 0.410 = 0.025$ in.^{4}

B. Channel Section:

Element No.	Length L (in.)	y (Dist. from top fiber) (in.)	L(y) (in.^{2})	L(y^2) (in.^{3})
6	2 × 1.000 = 2.00	0.098	0.196	0.019
7	2 × 6.000 = 12.00	3.060	36.720	112.363
8	8.00	6.022	48.176	290.116
Summation	22.00		85.092	402.498

Distance of axis from top fiber $= y_{cg} = \dfrac{85.092}{22.00} = 3.868$ in.

Linear I's of elements about their own axes:

$2I'_7 = 2(6.000^3/12) = 36.000$ in.^{3}

Summation L $(y_{cg}^2) = 402.498$

$I'_{top\ fiber} = 438.498$ in.^{3}

Minus Summation L $(y_{cg}^2) = -22.00 \times 3.868^2 = -329.151$ in.^{3}

Linear I′ channel section $= 109.347$ in.^{3}

Actual I channel section $= 109.347 \times 0.075 = 8.201$ in.^{4}

C. Composite Section:

Element	Area (in.^{2})	y Dist. from top fiber (in.)	(A)y (in^3)	(A)y^2 (in.^{4})
Flat Plate	8.042 × 0.060 = 0.483	0.143	0.069	0.010
Channel Sect.	22.00 × 0.075 = 1.650	3.868	6.382	24.686
Summation	2.133		6.451	24.696

Distance of axis from top fiber $= y_{cg} = 6.451/2.133 = 3.024$ in.

NOTE: Since distance of top compression fiber from neutral axis is greater than 1/2 depth $(6.060/2 = 3.030)$ a compression stress of 18,000 psi will govern as assumed and no further approximation will be required.

I Flat Plate $= 0.025$ in.^{4}

I Channel $= 8.201$

Summation A $(y^2) = 24.696$

$I_{top\ fiber} = 32.922$ in.^{4}

Minus Summation (A) $y_{cg}^2 = -(2.133)\,3.024^2 = -19.505$

Moment of Inertia, I $= 13.417$ in.^{4}

Section Modulus, $S = 13.417/3.036 = 4.419$ in.^{3}

Resisting moment $= 18{,}000 \times 4.419 = 79{,}540$ in.-lbs.

2. Determination of moment of inertia "I" for use in beam deflection calculations: (Using linear method, and assuming square corners.)

Bending moment $= 79{,}540$ in.-lbs.

A. Flat Plate

For a first approximation assume an effective width of flat plate based on a compression stress of 18,000 psi.

$\frac{w}{t} = \frac{9.000}{.060} = 150$. From Chart 3C, the effective design width of the top flange between welds $= 65 \times 0.060 = 3.900$ in.

Total effective design width of top flange $= 3.900 + 1.338^* + 1.440^* = 6.678$ in.

* NOTE: Since the w/t ratios, 1.338/0.060 and 1.440/0.060, do not exceed $(W/t)_{lim} = 5160/\sqrt{f} = 5160/\sqrt{18{,}000} = 38.5$, no reduction in effective width is required.

Element No.	L Effective Length (in.)	y Dist. from top fiber (in.)	(L)y (in.^{2})	(L)y^2 (in.^{3})
1	0.400	0.510	0.204	0.104
2	0.412	0.794	0.327	0.260
3	0.710	0.355	0.252	0.089
4	6.678	0.030	0.200	0.006
5	0.622	0.311	0.193	0.060
Summation	8.822		1.176	0.519

Distance of axis from top fiber $= y_{cg} = 1.176/8.822 = 0.133$ in.

Linear I's of element about their respective c.g.'s:

$$I'_1 = 0.400^3/12 = 0.005 \text{ in.}^3$$
$$I'_2 = 2 \times 0.149 \times 0.131^3 = 0.001$$
$$I'_3 = 0.710^3/12 = 0.030$$
$$I'_5 = 0.622^3/12 = 0.020$$
$$\text{Summation} = 0.056 \text{ in.}^3$$
$$\text{Summation } (L)\ y^2 = 0.519$$
$$0.575 = 0.575 \text{ in.}^3$$
$$\text{Minus summation } (L)\ y_{cg}^2 = -8.822 \times 0.133^2 = -0.156$$
$$\text{Linear } I'_x \text{ Flat Plate} = 0.419 \text{ in.}^3$$

Actual I_x Flat Plate $= 0.419 \times 0.060 = 0.025$ in.^{4}

B. Channel:

Same as determination of y_{cg}, I′ and I, as contained under Procedure 1.B. Beam Strength calculations.

C. Composite Section:

Element	Area A (in.)	Dist. from top fiber (in.)	(A)y (in.^{3})	(A)y^2 (in.^{4})
Flat Plate $8.822 \times 0.060 =$	0.529	0.133	0.070	0.009
Channel	1.650	3.868	6.382	24.686
Summation	2.179		6.452	24.695

Distance of axis from top fiber $= 6.452/2.179 = 2.961$ in.

$$\text{I Flat Plate} = .025$$
$$\text{I Channel} = 8.201$$
$$\text{Summation } (A)\ y^2 = 24.695$$
$$32.921$$

$$I_{top\ fiber} = 32.921 \text{ in.}^4$$

$$\text{Minus Summation (A) } y^2_{cg} = -2.179 \times 2.961^2 = -19.104$$

$$\text{First Approximation I} = 13.817 \text{ in.}^4$$

Check Stress in flat plate:

$$f_c = \frac{79{,}540 \times 2.961}{13.817} = 17{,}050 \text{ psi.}$$

SECOND APPROXIMATION

A. Flat Plate

Assume $f_c = 17{,}000$ psi.

From Chart 3C, $b/t = 66$

$b = 66 \times 0.060 = 3.960$ in.

Effective design width top flange $= 3.960 + 1.338 + 1.440 = 6.738$ in.

Element No.	L Effective Length (in.)	y Dist. from top fiber (in.)	(L)y (in.^{2})	L(y)^2 (in.^{3})
1	0.400	0.510	0.204	0.104
2	0.412	0.794	0.327	0.260
3	0.710	0.355	0.252	0.089
4	6.738	0.030	0.202	0.006
5	0.622	0.311	0.193	0.060
Summation	8.882		1.178	0.519

$$\text{Distance of axis from top fiber} = y_{cg} = \frac{1.178}{8.884} = .133 \text{ in.}$$

Summation of Linear I's $= 0.056$

Summation of $(L)y^2 = 0.519$

0.575 $\qquad I'_{top\ fiber} = 0.575$ in.^{3}

Minus Summation $(L)y^2_{cg} = 8.842 \times 0.133^2 = -0.157$

Linear I'_x Flat Plate $= 0.418$ in.^{3}

Actual I_x Flat Plate $= 0.060 \times 0.418 = 0.025$ in.^{4}

B. Channel Section

Same as determination of y_{cg}, I'_x and I_x as contained under procedure 1.B Beam Strength Calculations.

C. Composite Section

Element	Area A (in.^{2})	y Dist. from top fiber (in.)	A (y) in.^{3}	A(y)^2 (in.^{4})
Flat Plate 8.884(0.060) =	0.533	0.133	0.071	0 009
Channel	1.650	3.868	6.382	24.686
Summation	2.183		6.453	24.695

Distance of axis from top fiber $= 6.453/2.183 = 2.956$

I Flat Plate $= 0.025$

I Channel $= 8.201$

Summation $A(y)^2 = 24.695$

32.921 $\qquad I_{top\ fiber} = 32.921$

Minus Summation A $(y_{cg}^2) = 2.183 \times 2.956^2 = -19.075$

Second Approximation I $= 13.846$ in.^{4}

Check Stress in Flat Plate

$$f_c = \frac{79{,}540 \times 2.956}{13.846} = 16{,}980 \text{ psi.}$$

Since this checks closely with the previous assumption that $f_c = 17{,}000$ psi., no further approximations are necessary and the moment of inertia to be used in deflection computations is 13.85 in.^{4}.

3. (A) Determination of end reaction for 3″ bearing. From Section 3.5(a) of Design Specification:

Allowable reaction per web =

$$P = 100t^2\,[980 + 42B/t - 0.33(B/t)\;(h/t) - 0.11\;(h/t)] =$$

$$100 \times 0.075^2\,[980 + \frac{42 \times 3}{0.075} - \frac{0.22 \times 3 \times 6}{0.075^2} - \frac{0.11 \times 6}{0.075}] = 1095 \text{ lbs.}$$

Since there are 2 webs per panel the total reaction $2P = 2 \times 1095 = 2190$ lbs.

(B) For *interior reaction* for 3″ bearing with the panel continuous over the support:

$$P = 100t^2\,[3050 + 23\,\frac{B}{t} - 0.09\,\frac{(B)}{t}\frac{(h)}{t} - 5\,\frac{h}{t}\,] = 1850 \text{ lbs.}$$

Since there are 2 webs per panel the total allowable reaction $= 2P$ $= 2 \times 1850 = 3700$ lbs.

4. Determination of maximum longitudinal weld spacing to allow the lines of welds to be considered lines of stiffening. From Section 4.4(b) of Design Specification:

$$\text{Spacing} = \frac{6000t}{\sqrt{f_b}} = \frac{6000 \times 0.060}{\sqrt{18{,}000}} = 2.68 \text{ in. Use } 2\tfrac{5}{8} \text{ in.}$$

5. Determination of maximum allowable shear using weld spacing of 2⅝ in. (2.625 in.).

To satisfy Section 4.4(a) of Design Specification:

$$V = \frac{2\;I\;F}{(\text{Spacing}) \times Q}$$

where: V = Total shear.

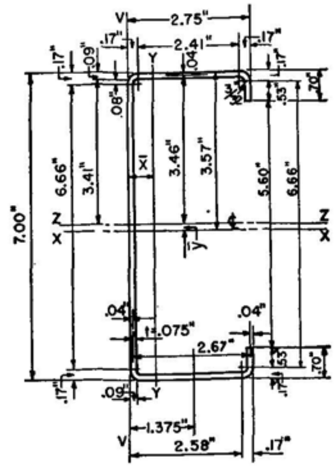
I = Moment of inertia as determined in beam strength calculations.

F = Weld value = 725 lbs. per weld for 16 gage steel, according to Section 4.2.2.

Q = Statical moment of flat plate, or channel, about the neutral axis of the composite section.

$$V = \frac{2 \times 13.417 \times 725}{2.625 \times 1.401} = 5290 \text{ lbs.}$$

EXAMPLE No. 15



Column properties and beam strength using the linear method.

GIVEN:

1. STEEL: Grade C ($f_b = 18{,}000$ psi).
2. SECTION: 7 $\times$ 2¾ (nominal) $\times$ No. 14 ga. channel with stiffened flanges.

REQUIRED:

(a) Column properties.
(b) Strength in bending about the Major Axis.

SOLUTION:

Properties of the 90° corner. From page 25, with

Rad. of center of corner: $R = 0.095 + \frac{0.075}{2} = 0.133$ in.

Length of Arc, $L = 1.57 \times 0.133 = 0.209$, say 0.21 in.

Distance center of radius to C.G. of corner:

$c = 0.637 \times 0.13 = 0.084$, say 0.08 in.

I' of corner about its centroidal axis is negligible.

Compute other dimensions and distances from Z-Z axis (center line of section) and from V-V axis (back of web) to center of gravity of elements as shown on sketch.

(a) Column Properties:

1. Computation of I_x

Web	$1/12 \times 6.66^3 =$	24.62 in.³
Straight Part of Lips	$1/12(6.66^3 - 5.60^3) =$	9.98
Four Corners	$4 \times 0.21 \times (3.41)^2 =$	9.77
Flanges	$2 \times 2.41 \times (3.46)^2 =$	57.70
For Full Section ($y = 0$): $I'_z = I'_x =$		102.07 in.³

$I_x = I_z \times t = 102.07 \times 0.075 = 7.655$ in.⁴ (Table 1, conventional $I_x = 7.660$ in.⁴, full section).

2. Computation I_y.

Element	Length L (in.)	x (in.)	(L) x (in.^{2})	(L) x^2 (in.^{3})	I′ about c.g. of elements
Web	6.66	0.04	0.27	0.01	0
Straight part of Lips	1.06	2.71	2.87	7.78	0
2 Near Corners	0.42	0.09	0.04	0	0
2 Far Corners	0.42	2.66	1.12	2.98	0
Flanges	4.82	1.375	6.63	9 12	$2 \times 1/12 \times 2.41^3 =$ 2.33
Summation	13.38		10.93	19.89	2.33

$x = \dfrac{10.93}{13.38} = 0.817$ in. (Table 1, $x = 0.8150$)

$I'_v = 19.89 + 2.33 = 22.22$ in.^{3}

$I'_y = I'_v - (L)x^2_{cg} = 22.22 - (13.38 \times 0.817^2) = 13.29$ in.^{3}

$I_y = I'_y \times t = 13.29 \times 0.075 = 0.997$ in.^{4} (Table 1, $I_y = 1.00$ in.^{4})

3. Full Section Properties:

$A = L \times t = 13.38 \times 0.075 = 1.003$ in.^{2} (Table 1, $A = 1.003$ in.^{2})

$$r_x = \sqrt{\frac{I_x}{L}} = \sqrt{\frac{102.07}{13.38}} = 2.76 \text{ in. (Table 1, } r_x = 2.76 \text{ in.)}$$

$$r_y = \sqrt{\frac{I'_y}{L}} = \sqrt{\frac{13.29}{13.38}} = 0.997 \text{ in. (Table 7, } r_y = 0.999 \text{ in.)}$$

4. Computation of Q (Section 3.6.1, Design Specification)

Flanges: $w/t = \dfrac{2.41}{0.075} = 32.1$, $b/t = 31.6$ (Chart 3A)

Deduction $= 2(32.1 - 31.6) \times 0.075 = 0.08$ in.

Web: $w/t = \dfrac{6.66}{0.075} = 88.8$, $b/t = 47.5$ (Chart 3B)

Deduction $= (88.8 - 47.5) \times 0.075 = 3.09$ in.

Total Effective Length: $L_{eff} = 13.38 - 0.08 - 3.09 = 10.21$ in.

$Q = \dfrac{L_{eff}}{L} = \dfrac{10.21}{13.38} = 0.763$ (Table 1, $Q = 0.763$)

(b) Strength in bending about major axis:

The deduction from the flat width of the compression flange is

$(32.1 - 31.6) \times 0.075 = 0.04$ (see above)

The reduced I'_z is then: $102.07 - (0.04 \times 3.46^2) = 101.59$ in.^{3}

Total effective length is: $L_{eff} = 13.38 - 0.04 = 13.34$ in.

The distance $\bar{y}$ of the X-X axis (through the center of gravity of the reduced section) from the Z-Z axis, is found from the equation:

$13.34(\bar{y}) = 0.04 \times 3.46 = 0.138$; $\bar{y} = 0.01$ in.

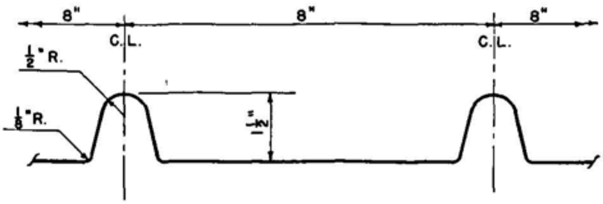
Therefore $I'_x = I'_z - (13.34 \times 0.01^2) = 101.59 - 0.00 = 101.59$ in.^{3}

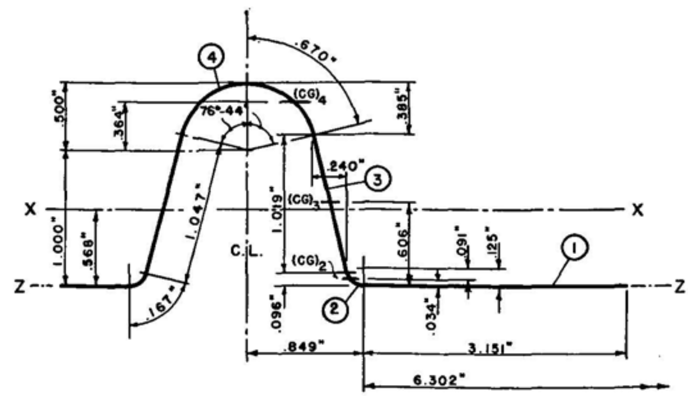
Distance from X-X axis to the extreme fiber is: $3.50 + 0.01 = 3.51$ in.

$S_x = \dfrac{I'_x \times t}{3.51} = \dfrac{101.59 \times 0.075}{3.51} = 2.171$ in.^{3} (Table 1, $S_x = 2.17$ in.^{3})

$M_{max} = f_b \times S_x = 18{,}000 \times 2.171$ in.$^3 = 39{,}080$ in.-lb.

EXAMPLE No. 16





Beam strength using linear method.

GIVEN:

1. STEEL: Specified minimum yield = 30,000 psi (f_b = 16,216 psi).
2. A 24 gage wall siding as shown.

REQUIRED: Strength in bending per foot of width about the minor axis when flat elements are in compression.

SOLUTION:

Properties of Circular Elements: From page 25.

Element No. 2:

Radius: $R = 0.125$ in.
Angle θ: $= 76° - 44' = 1.339$ Radians
Sin $= 0.973$ Cos $= 0.229$

The distance of the center of gravity of the arc with central angle from the arc center is:

$$c_1 = \frac{R \text{ Sin } \theta}{\theta} = \frac{(0.125)\ (0.973)}{1.339} = 0.091 \text{ in.}$$

$$I' = \left(\frac{\theta + \text{Sin}\theta \ \text{Cos}\theta}{2} - \frac{(\text{Sin}\theta)^2}{\theta}\right) R^3$$

$$= \left(\frac{1.339 + (0.973)(0.229)}{2} - \frac{(0.973)^2}{1.339}\right) (0.125)^3 = \text{Negligible}$$

Element No. 4

Radius: $R = 0.500$ in.

Angle θ: Same as for Element No. 2 (see above)

$$c_1 = \frac{(0.500)(0.973)}{1.339} = 0.363 \text{ in.}$$

$$I' = \left(\frac{1.339 + (0.973)(0.229)}{2} - \frac{(0.973)^2}{1.339}\right)(0.500)^3 = 0.009 \text{ in.}^3$$

For an approximation use an effective width of element No. 1 based on $f = 16{,}216$ psi.

$$\frac{w}{t} = \frac{6.302}{0.024} = 262$$

From Chart 3A, the effective design width, b, exclusive of corners $= 56t = 1.344$ in.

Cross-sectional Properties:

Element		Length L (in.)	Distance from Axis: y (in.)	(L) y (in.^{2})
1	$\frac{1.344}{2} =$	0.675	0	0
2		0.167	0.034	0.006
3		1.047	0.606	0.634
4		0.670	1.364	0.914
Summation:		2.556		1.554

Distance of neutral axis from Z axis:

$$y_{cg} = \frac{1.554}{2.556} = 0.608 \text{ in.}$$

2nd Approximation:

Guided by the determination in the first approximation:

Assume $y_{cg} = 0.580$ in.

Then the compressive stress in Element No. 1 is:

$$f_c = \frac{0.580}{0.920} \times 16{,}216 = 10{,}220 \text{ psi}$$

From Chart 3A, for this f_c and $w/t = 262$: $b/t = 70$; thus, the effective design width, b, $= 70 \times 0.024 = 1.680$ in.

Element		Length L (in.)	y Distance from Z axis: y (in.)	(L) y (in.^{2})
1	$\frac{1.680}{2} =$	0.840	0	0
2		0.167	0.034	0.006
3		1.047	0.606	0.634
4		0.670	1.364	0.914
Summation		2.724		1.554

Distance of X axis from Z axis: $y_{cg} = \frac{1.554}{2.724} = 0.570$ in.

3rd Approximation:

On the basis of the determination above assume y = 0.568 in.

$$f_c = \frac{0.568}{0.932} \times 16216 = 9890 \text{ psi}$$

From Chart 3A, for this f_c and w/t = 262, b/t = 71; thus, the effective design width, b = 71 × 0.024 = 1.704 in.

Element	L (in.)	y (in.)	(L) y (in.^{2})	(L) y^2 (in.^{3})	I' of elements about their own axes
1	$\frac{1.704}{2} = 0.852$	0	0	0	
2	0.167	0.034	0.006	0	
3	1.047	0.606	0.634	0 384	$\frac{1.047 \times 1.019^2}{12} = 0.091$
4	0.670	1.364	0.914	1.247	See above = 0.009
Summation	2.736		1 554	1.631	0.100 in.^{3}

$$(1/2)\ I'_z = 1.631 + 0.100 = 1.731 \text{ in.}^3$$

$$y_{cg} = \frac{1.554}{2.736}\ 0.568 \text{ in. (Checks the assumed value)}$$

Compute I'_x for one-half of the 8 in. width:

$(1/2)\ I'_x = (1/2)\ I_z - L\ (y_{cg})^2 = 1.731 - (2.736 \times 0.568^2) = .848$ in.^{3}

$I'_x \quad = 0.848 \times 2 = 1.696$ in.^{3}

$I_x \quad = I'_x \cdot t = 1.696 \times 0.024 = 0.0407$ in.^{4}

$S_x \quad = \frac{0.0407}{0.932} = 0.0437$ in.^{3}

$M_{max} = f_b \times S_x = 16216 \times 0.0437 = 710$ in.-lbs.

PART IV

CHARTS AND TABLES OF STRUCTURAL PROPERTIES

PART IV

CHARTS AND TABLES OF STRUCTURAL PROPERTIES

This section contains charts for determination of effective widths, column design curves, and tables of properties of a selected series of cold-formed sections.

The effective width charts of the first edition of this manual have been replaced by new charts drawn to conform to the revised provisions of the Specification for effective width determination. There are two charts for effective width determination for safe load calculations and two for effective width determination for deflection calculations. One of each pair of charts covers the entire range of flat-width ratios permitted under the specification. The other includes only the smaller flat-width ratios and is drawn to a larger scale.

The chart of column curves, Chart 4, is the same as in the previous edition.

The tables have been completely rearranged and some of the tables of the first edition have been combined in order to avoid repetition. The channel and zee sections are substantially the same as those of the previous edition, but both effective and full section properties are now given in the same table. Properties of the effective cross section of sections having stiffened compression elements have been recomputed in accordance with the revised effective width provisions of the Specification. In some instances, it has been found that for purposes of deflection computation the full section properties can be used without substantial error, and the variable moments of inertia which appeared in some of the tables of the first edition have been omitted.

Table 7, Properties of Hat Sections, is completely new, and has been included because of the growing popularity of sections of this kind. Although the group of hat sections listed in Table 7 is by no means comprehensive, an effort has been made to include a sufficient range of sizes to permit the designer to interpolate with a reasonable degree of accuracy in making preliminary estimates of the properties of sections of other dimensions than those shown.

In the following tables:

f_b is the basic design stress.

S_x and S_y are section moduli about the X-X and Y-Y axes, respectively.

I_x and I_y are moments of inertia of the section about the X-X and Y-Y axes, respectively.

r_x and r_y are radii of gyration about the X-X and Y-Y axes, respectively, based on the full theoretical outline of the section as specified in Section 3.6, Design Specification.

Q is the column form factor defined in Section 3.6.1, Design Specification.

The properties in the tables have been tabulated both for Grade C steel (33,000 lbs per sq. in. specified minimum yield point, $f_b = 18{,}000$ lbs per sq. in.) and for a high strength steel having a specified minimum yield point of 50,000 psi and $f_b = 27{,}000$ psi.

Interpolation and Extrapolation

Properties which vary with the basic design stress f_b, may be found for other grades of steel by straight line interpolation and extrapolation provided

f_b is not less than 13,500 psi nor more than 30,000 psi. The procedure for such determination may be expressed by the following formula:

$$F_f = F_{18} + \left((F_{27} - F_{18}) \frac{f_b - 18000}{27{,}000 - 18{,}000} \right), \text{ in which}$$

F_f is the property desired.

F_{18} and F_{27} are the corresponding properties for $f_b = 18{,}000$ and $f_b = 27{,}000$ psi respectively.

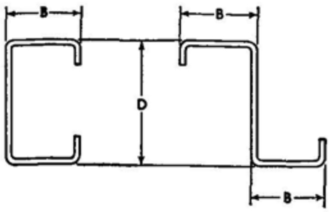
f_b is the basic allowable design stress for which the interpolated property F_f is desired.

In using the above formula careful attention must be given to the sign of each quantity shown.

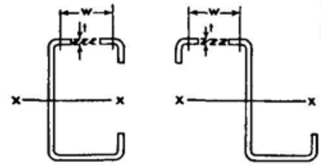
The following properties are subject to straight line interpolation and extrapolation in the manner outlined above

Table	Properties
1	S_x, Q
2	Q, f_c
3	M_{max}, Q, f_c
4	S_x, Q
5	Q, f_c
6	M_{max}, Q, f_c

TABLE 1



CHANNEL OR ZEE WITH STIFFENED FLANGES

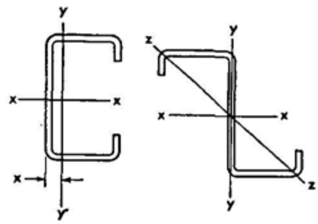


Effective section for beam strength about the x-x axis where w/t of compression flange exceeds 28.2 for f_b = 18,000 psi and 23.1 for f_b = 27,000 psi.

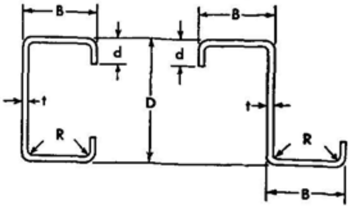
Nominal				Beam Strength		Properties of Full Section							
				Effective S_x		Area	Axis x-x			Axis y-y			
				C or Z		C or Z	C or Z			C			
Size D x B	Gage	t	Wt./Ft.	f_b = 18,000 psi	f_b = 27,000 psi		I_x	S_x	r_x	I_y	S_y	r_y	x
In.	No.	In.	Lbs.	$In.^3$	$In.^3$	Sq. In.	$In.^4$	$In.^3$	In.	$In.^4$	$In.^3$	In.	In.
12 x 3½	10	0.135	9.42	9.37	9.37	2.70	56.2	9.37	4.56	4.03	1.55	1.22	0.910
	12	0.105	7.31	7.31	7.22	2.10	43.8	7.31	4.57	3.09	1.18	1.21	0.883
10 x 3½	10	0.135	8.48	7.30	7.30	2.43	36.5	7.30	3.87	3.81	1.53	1.25	1.00
	12	0.105	6.57	5.70	5.63	1.89	28.5	5.70	3.89	2.93	1.16	1.25	0.975
	14	0.075	4.67	3.89	3.71	1.34	20.5	4.10	3.91	2.02	0.783	1.23	0.926
9 x 3¼	10	0.135	7.77	6.04	6.04	2.23	27.2	6.04	3.49	3.08	1.35	1.17	0.965
	12	0.105	5.95	4.66	4.64	1.71	21.0	4.66	3.50	2.26	0.965	1.15	0.910
	14	0.075	4.28	3.28	3.14	1.23	15.3	3.40	3.53	1.63	0.689	1.15	0.888
	16	0.060	3.40	2.47	2.35	0.975	12.2	2.70	3.53	1.26	0.527	1.14	0.859
8 x 3	10	0.135	6.97	4.83	4.83	2.00	19.3	4.83	3.11	2.33	1.11	1.08	0.901
	12	0.105	5.40	3.78	3.78	1.55	15.1	3.78	3.12	1.79	0.840	1.07	0.873
	14	0.075	3.89	2.71	2.61	1.12	11.1	2.76	3.15	1.29	0.600	1.08	0.851
	16	0.060	3.08	2.06	1.95	0.885	8.79	2.20	3.15	0.997	0.458	1.06	0.822
7 x 2¾	10	0.135	6.17	3.75	3.75	1.77	13.1	3.75	2.72	1.71	0.893	0.982	0.837
	12	0.105	4.86	2.98	2.98	1.39	10.4	2.98	2.74	1.38	0.723	0.996	0.837
	14	0.075	3.50	2.17	2.11	1.00	7.66	2.19	2.76	1.00	0.517	0.999	0.815
	16	0.060	2.77	1.67	1.59	0.795	6.10	1.74	2.77	0.773	0.393	0.986	0.786
6 x 2½	10	0.135	5.37	2.81	2.81	1.54	8.42	2.81	2.34	1.21	0.700	0.885	0.774
	12	0.105	4.23	2.24	2.24	1.21	6.72	2.24	2.35	0.983	0.570	0.900	0.774
	14	0.075	3.10	1.68	1.65	0.891	5.04	1.68	2.38	0.756	0.440	0.921	0.780
	16	0.060	2.46	1.30	1.25	0.705	4.01	1.34	2.39	0.583	0.333	0.910	0.751
5 x 2	10	0.135	4.43	1.88	1.88	1.27	4.69	1.88	1.92	0.651	0.480	0.715	0.644
	12	0.105	3.50	1.51	1.51	1.00	3.76	1.51	1.94	0.534	0.394	0.729	0.643
	14	0.075	2.53	1.12	1.12	0.726	2.80	1.12	1.96	0.390	0.283	0.733	0.622
	16	0.060	2.00	0.890	0.877	0.573	2.23	0.891	1.97	0.298	0.212	0.721	0.594
	18	0.048	1.61	0.707	0.680	0.461	1.80	0.722	1.98	0.244	0.173	0.727	0.594
4 x 2	10	0.135	3.96	1.38	1.38	1.14	2.76	1.38	1.56	0.601	0.466	0.727	0.712
	12	0.105	3.14	1.11	1.11	0.900	2.22	1.11	1.57	0.493	0.383	0.740	0.712
	14	0.075	2.27	0.832	0.832	0.651	1.67	0.832	1.60	0.361	0.276	0.745	0.689
	16	0.060	1.79	0.665	0.654	0.513	1.33	0.665	1.61	0.277	0.206	0.735	0.660
	18	0.048	1.44	0.528	0.506	0.413	1.08	0.540	1.62	0.226	0.169	0.740	0.660
3½ x 2	10	0.135	3.73	1.15	1.15	1.07	2.01	1.15	1.37	0.571	0.458	0.730	0.753
	12	0.105	2.95	0.927	0.927	0.847	1.62	0.927	1.38	0.469	0.376	0.744	0.753
	14	0.075	2.14	0.699	0.699	0.613	1.22	0.699	1.41	0.344	0.271	0.750	0.729
	16	0.060	1.68	0.559	0.550	0.483	0.979	0.560	1.42	0.264	0.203	0.740	0.699
	18	0.048	1.36	0.444	0.425	0.389	0.795	0.455	1.43	0.216	0.166	0.745	0.699
3 x 1¾	12	0.105	2.59	0.679	0.679	0.742	1.02	0.679	1.17	0.319	0.300	0.655	0.689
	14	0.075	1.82	0.509	0.509	0.523	0.764	0.509	1.21	0.219	0.196	0.647	0.635
	16	0.060	1.47	0.416	0.416	0.423	0.624	0.416	1.22	0.181	0.162	0.654	0.635
	18	0.048	1.16	0.331	0.321	0.331	0.498	0.332	1.23	0.137	0.119	0.642	0.604

NOTE: The effective section moduli in bending about the y-y axis have not been tabulated. When one of the webs acts as a compression flange the section modulus should be calculated on the basis of its effective width as provided in Section 2.3 of the Design Specification. When the web acts as a tension flange the section modulus of the full section is effective.
For all of the sections listed in this table the moment of inertia, I_x, of the full section may be used in deflection calculations without appreciable error.

TABLE 1



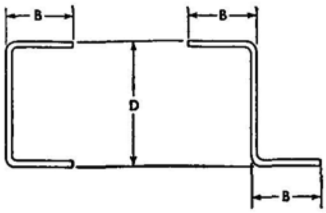
CHANNEL OR ZEE
WITH
STIFFENED FLANGES



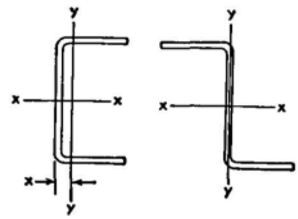
Properties of Full Section (cont'd)					Column Form Factor		Dimensions of Sections							
Axis y-y			Axis z-z	Product of Inertia	Q									
Z			Z	Z	C or Z		C or Z							
I_y	S_y	r_y	r min.	I_{xy}	f_b = 18,000 psi	f_b = 27,000 psi	D	B	d	t	R	Wt./Ft.	Gage	Size D x B
In.⁴	In.³	In.	In.	In.⁴			In.	In.	In.	In.	In.	Lbs.	No.	In.
6.25	1.82	1.52	1.06	13.1	0.750	0.702	12.0	3.50	1.0	0.135	3/16	9.42	10	12 x 3½
4.72	1.37	1.50	1.04	10.1	0.687	0.637	12.0	3.50	0.9	0.105	3/16	7.31	12	
6.25	1.82	1.60	1.07	10.8	0.818	0.769	10.0	3.50	1.0	0.135	3/16	8.48	10	10 x 3½
4.72	1.37	1.58	1.05	8.36	0.754	0.702	10.0	3.50	0.9	0.105	3/16	6.57	12	
3.16	0.914	1.54	1.02	5.74	0.632	0.563	10.0	3.50	0.7	0.075	3/32	4.67	14	
5.14	1.62	1.52	1.00	8.51	0.851	0.809	9.0	3.25	1.0	0.135	3/16	7.77	10	9 x 3¼
3.67	1.15	1.47	0.968	6.33	0.783	0.735	9.0	3.25	0.8	0.105	3/16	5.95	12	
2.59	0.808	1.45	0.958	4.56	0.672	0.603	9.0	3.25	0.7	0.075	3/32	4.28	14	
1.98	0.614	1.42	0.940	3.55	0.588	0.520	9.0	3.25	0.6	0.060	3/32	3.40	16	
3.94	1.34	1.40	0.920	6.29	0.885	0.835	8.0	3.00	0.9	0.135	3/16	6.97	10	8 x 3
2.96	1.01	1.38	0.903	4.85	0.820	0.773	8.0	3.00	0.8	0.105	3/16	5.40	12	
2.10	0.708	1.37	0.893	3.50	0.719	0.648	8.0	3.00	0.7	0.075	3/32	3.89	14	
1.59	0.537	1.34	0.876	2.72	0.633	0.563	8.0	3.00	0.6	0.060	3/32	3.08	16	
2.94	1.10	1.29	0.838	4.49	0.923	0.874	7.0	2.75	0.8	0.135	3/16	6.17	10	7 x 2¾
2.35	0.873	1.30	0.837	3.61	0.860	0.813	7.0	2.75	0.8	0.105	3/16	4.86	12	
1.67	0.614	1.29	0.828	2.61	0.763	0.698	7.0	2.75	0.7	0.075	3/32	3.50	14	
1.26	0.464	1.26	0.810	2.03	0.684	0.613	7.0	2.75	0.6	0.060	3/32	2.77	16	
2.13	0.874	1.17	0.755	3.07	0.961	0.918	6.0	2.50	0.7	0.135	3/16	5.37	10	6 x 2½
1.71	0.697	1.19	0.754	2.48	0.904	0.857	6.0	2.50	0.7	0.105	3/16	4.23	12	
1.30	0.527	1.21	0.760	1.88	0.813	0.756	6.0	2.50	0.7	0.075	3/32	3.10	14	
0.981	0.397	1.18	0.744	1.46	0.742	0.672	6.0	2.50	0.6	0.060	3/32	2.46	16	
1.17	0.607	0.960	0.625	1.68	0.993	0.962	5.0	2.00	0.7	0.135	3/16	4.43	10	5 x 2
0.947	0.487	0.971	0.624	1.37	0.950	0.906	5.0	2.00	0.7	0.105	3/16	3.50	12	
0.670	0.341	0.961	0.615	0.999	0.857	0.810	5.0	2.00	0.6	0.075	3/32	2.53	14	
0.499	0.254	0.934	0.598	0.772	0.799	0.744	5.0	2.00	0.5	0.060	3/32	2.00	16	
0.406	0.205	0.938	0.598	0.629	0.735	0.668	5.0	2.00	0.5	0.048	3/32	1.61	18	
1.17	0.606	1.02	0.614	1.32	1.000	0.998	4.0	2.00	0.7	0.135	3/16	3.96	10	4 x 2
0.947	0.486	1.03	0.612	1.07	0.993	0.966	4.0	2.00	0.7	0.105	3/16	3.14	12	
0.670	0.341	1.01	0.603	0.787	0.925	0.883	4.0	2.00	0.6	0.075	3/32	2.27	14	
0.499	0.254	0.987	0.587	0.610	0.873	0.818	4.0	2.00	0.5	0.060	3/32	1.79	16	
0.406	0.205	0.991	0.586	0.498	0.808	0.737	4.0	2.00	0.5	0.048	3/32	1.44	18	
1.17	0.606	1.05	0.598	1.13	1.000	1.000	3.5	2.00	0.7	0.135	3/16	3.73	10	3½ x 2
0.947	0.486	1.06	0.596	0.924	1.000	0.989	3.5	2.00	0.7	0.105	3/16	2.95	12	
0.670	0.341	1.05	0.587	0.680	0.958	0.921	3.5	2.00	0.6	0.075	3/32	2.14	14	
0.499	0.254	1.02	0.572	0.530	0.912	0.859	3.5	2.00	0.5	0.060	3/32	1.68	16	
0.406	0.205	1.02	0.571	0.432	0.849	0.776	3.5	2.00	0.5	0.048	3/32	1.36	18	
0.669	0.394	0.949	0.529	0.611	1.000	1.000	3.0	1.75	0.7	0.105	3/16	2.59	12	3 x 1¾
0.429	0.251	0.906	0.508	0.430	0.984	0.953	3.0	1.75	0.5	0.075	3/32	1.82	14	
0.351	0.204	0.911	0.506	0.354	0.948	0.910	3.0	1.75	0.5	0.060	3/32	1.47	16	
0.257	0.149	0.881	0.491	0.272	0.899	0.837	3.0	1.75	0.4	0.048	3/32	1.16	18	

DIMENSIONS: Equipment and forming practices vary with different manufacturers, resulting in minor variations in some of these dimensions. These minor variations do not affect the published properties. Consult the manufacturer for actual weight per foot and actual dimensions.

TABLE 2



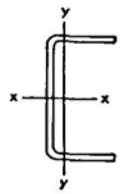
CHANNEL OR ZEE
WITH
UNSTIFFENED FLANGES



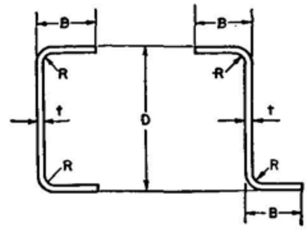
Nominal		t	Wt./Ft.	Properties of Full Section										
				Area	Axis x-x			Axis y-y						
				[OR Z	[OR Z			[				Z		
Size D x B	Gage				I_x	S_x	r_x	I_y	S_y	r_y	x	I_y	S_y	r_y
In.	No.	In.	Lbs.	Sq. In.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.	In.^{4}	In.^{3}	In.
8 x 2	10	0.135	5.38	1.55	12.9	3.24	2.89	0.465	0.293	0.548	0.383	0.685	0.360	0.666
	12	0.105	4.23	1.21	10.3	2.57	2.91	0.379	0.236	0.559	0.376	0.548	0.283	0.672
	14	0.075	3.08	0.884	7.66	1.91	2.94	0.294	0.178	0.577	0.373	0.416	0.209	0.686
	16	0.060	2.44	0.699	6.02	1.50	2.93	0.211	0.132	0.549	0.343	0.293	0.153	0.647
7 x 1½	10	0.135	4.39	1.26	7.54	2.15	2.45	0.170	0.148	0.368	0.260	0.249	0.186	0.445
	12	0.105	3.50	1.00	6.18	1.77	2.48	0.160	0.131	0.399	0.268	0.229	0.160	0.478
	14	0.075	2.53	0.725	4.54	1.30	2.50	0.113	0.0930	0.395	0.248	0.157	0.110	0.465
	16	0.060	1.99	0.572	3.54	1.01	2.49	0.077	0.0669	0.368	0.221	0.105	0.0777	0.428
6 x 1½	10	0.135	3.91	1.12	5.12	1.70	2.13	0.164	0.146	0.382	0.283	0.249	0.186	0.471
	12	0.105	3.13	0.898	4.22	1.40	2.17	0.155	0.130	0.415	0.293	0.229	0.160	0.505
	14	0.075	2.26	0.650	3.10	1.03	2.19	0.109	0.0918	0.410	0.272	0.157	0.110	0.491
	16	0.060	1.78	0.512	2.42	0.806	2.17	0.075	0.0660	0.383	0.243	0.105	0.0777	0.453
	18	0.048	1.43	0.409	1.94	0.646	2.18	0.059	0.0519	0.378	0.234	0.081	0.0603	0.444
5 x 1¼	12	0.105	2.58	0.741	2.38	0.953	1.79	0.087	0.0884	0.343	0.252	0.132	0.111	0.422
	14	0.075	1.84	0.528	1.71	0.683	1.80	0.053	0.0563	0.316	0.214	0.076	0.0684	0.380
	16	0.060	1.47	0.422	1.37	0.547	1.80	0.041	0.0439	0.311	0.202	0.058	0.0523	0.369
	18	0.048	1.15	0.331	1.06	0.423	1.79	0.027	0.0307	0.284	0.176	0.037	0.0360	0.334
4 x 1⅛	12	0.105	2.17	0.623	1.33	0.663	1.46	0.071	0.0779	0.337	0.265	0.113	0.101	0.426
	14	0.075	1.61	0.462	1.02	0.512	1.49	0.058	0.0611	0.355	0.262	0.089	0.0759	0.439
	16	0.060	1.26	0.362	0.792	0.396	1.48	0.039	0.0430	0.327	0.230	0.057	0.0523	0.399
	18	0.048	1.01	0.289	0.635	0.318	1.48	0.030	0.0337	0.322	0.220	0.044	0.0404	0.390
3 x 1⅛	12	0.105	1.80	0.518	0.658	0.439	1.13	0.065	0.0750	0.354	0.308	0.113	0.101	0.467
	14	0.075	1.35	0.387	0.515	0.344	1.15	0.054	0.0590	0.372	0.306	0.089	0.0759	0.480
	16	0.060	1.05	0.302	0.398	0.265	1.15	0.036	0.0416	0.344	0.270	0.057	0.0523	0.436
	18	0.048	0.841	0.241	0.319	0.213	1.15	0.028	0.0326	0.339	0.259	0.044	0.0403	0.426
2 x 1⅛	12	0.105	1.44	0.413	0.250	0.250	0.779	0.056	0.0702	0.369	0.374	0.113	0.100	0.522
	14	0.075	1.09	0.312	0.200	0.200	0.800	0.047	0.0556	0.387	0.370	0.089	0.0758	0.534
	16	0.060	0.843	0.242	0.154	0.154	0.799	0.031	0.0392	0.360	0.330	0.057	0.0523	0.487
	18	0.048	0.673	0.193	0.124	0.124	0.802	0.024	0.0308	0.356	0.318	0.044	0.0403	0.476

NOTE: The effective section moduli in bending about the y-y axis have not been tabulated. When one of the webs acts as a compression flange the section modulus should be calculated on the basis of its effective width as provided in Section 2.3 of the Design Specification. When the web acts as a tension flange the section modulus of the full section is effective.

TABLE 2



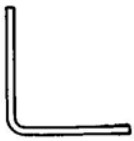
CHANNEL OR ZEE
WITH
UNSTIFFENED FLANGES



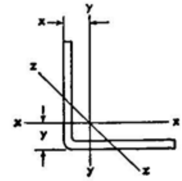
Properties of Full Section (cont'd)		Allowable Beam Stress f_c		Column Form Factor Q		Dimensions of Sections						
Axis z-z Z	Product of Inertia Z	[OR Z		[OR Z		[OR Z						
r min.	I_{xy}	f_b = 18,000 psi	f_b = 27,000 psi	f_b = 18,000 psi	f_b = 27,000 psi	D	B	t	R	Wt./Ft.	Gage	Size D x B
In.	In.	psi	psi			In.	In.	In.	In.	Lbs.	No.	In.
0.503	2.04	17,960	26,890	0.850	0.784	8.0	1.97	0.135	3/16	5.38	10	8 x 2
0.503	1.62	15,800	22,790	0.693	0.619	8.0	1.99	0.105	3/16	4.23	12	
0.510	1.22	10,880	13,740	0.448	0.354	8.0	2.03	0.075	3/32	3.08	14	
0.483	0.896	8,330	8,450	0.323	0.218	8.0	1.94	0.060	3/32	2.44	16	
0.352	0.899	18,000	27,000	0.891	0.823	7.0	1.40	0.135	3/16	4.39	10	7 x 1½
0.370	0.791	18,000	27,000	0.806	0.740	7.0	1.49	0.105	3/16	3.50	12	
0.360	0.554	15,190	21,570	0.595	0.522	7.0	1.46	0.075	3/32	2.53	14	
0.333	0.394	13,460	18,280	0.485	0.411	7.0	1.38	0.060	3/32	1.99	16	
0.363	0.768	18,000	27,000	0.947	0.887	6.0	1.40	0.135	3/16	3.91	10	6 x 1½
0.380	0.676	18,000	27,000	0.871	0.807	6.0	1.49	0.105	3/16	3.13	12	
0.370	0.474	15,190	21,570	0.650	0.573	6.0	1.46	0.075	3/32	2.26	14	
0.343	0.338	13,460	18,280	0.533	0.453	6.0	1.38	0.060	3/32	1.78	16	
0.336	0.264	10,730	13,070	0.405	0.315	6.0	1.36	0.048	3/32	1.43	18	
0.319	0.386	18,000	27,000	0.933	0.873	5.0	1.24	0.105	3/16	2.58	12	5 x 1¼
0.291	0.243	17,460	25,950	0.784	0.715	5.0	1.15	0.075	3/32	1.84	14	
0.282	0.188	15,730	22,650	0.655	0.582	5.0	1.13	0.060	3/32	1.47	16	
0.257	0.130	14,290	19,870	0.545	0.471	5.0	1.05	0.048	3/32	1.15	18	
0.311	0.276	18,000	27,000	0.990	0.951	4.0	1.17	0.105	3/16	2.17	12	4 x 1⅛
0.318	0.215	17,010	25,080	0.853	0.785	4.0	1.21	0.075	3/32	1.61	14	
0.291	0.150	15,730	22,650	0.736	0.660	4.0	1.13	0.060	3/32	1.26	16	
0.284	0.117	13,590	18,520	0.598	0.513	4.0	1.11	0.048	3/32	1.01	18	
0.317	0.205	18,000	27,000	1.000	1.000	3.0	1.17	0.105	3/16	1.80	12	3 x 1⅛
0.324	0.160	17,010	25,080	0.929	0.878	3.0	1.21	0.075	3/32	1.35	14	
0.298	0.112	15,730	22,650	0.824	0.752	3.0	1.13	0.060	3/32	1.05	16	
0.291	0.0871	13,590	18,520	0.681	0.590	3.0	1.11	0.048	3/32	0.841	18	
0.305	0.134	18,000	27,000	1.000	1.000	2.0	1.17	0.105	3/16	1.44	12	2 x 1⅛
0.312	0.106	17,010	25,080	0.945	0.929	2.0	1.21	0.075	3/32	1.09	14	
0.291	0.0737	15,730	22,650	0.874	0.834	2.0	1.13	0.060	3/32	0.843	16	
0.284	0.0576	13,590	18,520	0.752	0.671	2.0	1.11	0.048	3/32	0.673	18	

DIMENSIONS: Equipment and forming practices vary with different manufacturers, resulting in minor variations in some of these dimensions. These minor variations do not affect the published properties. Consult the manufacturer for actual weight per foot and actual dimensions.

TABLE 3



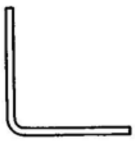
EQUAL LEG ANGLE
WITH
UNSTIFFENED LEGS



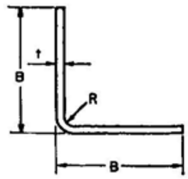
Nominal		t	Wt./Ft.	Area	Properties of Full Section						Beam Strength f_b = 18,000 psi		
					Axis x-x and Axis y-y				Axis z-z		$M_{max.}$		$M_{max.}$
Size	Gage				I	S	r	x = y	I	r	f_c	Comp. x–L–x Tension	Comp. x–Γ–x Tension
In.	No.	In.	Lbs.	Sq. In.	In.^{4}	In.^{3}	In.	In.	In.^{4}	In.	psi	In.-Lb.	In.-Lb.
4 x 4	10	0.135	3.66	1.05	1.715	0.582	1.28	1.07	0.662	0.794	9,660	5,630	10,480
3 x 3	10	0.135	2.72	0.781	0.712	0.324	0.955	0.819	0.271	0.589	13,720	4,450	5,840
	12	0.105	2.16	0.620	0.586	0.262	0.972	0.817	0.224	0.601	10,230	2,680	4,710
2½ x 2½	10	0.135	2.25	0.646	0.407	0.223	0.793	0.694	0.153	0.487	15,740	3,520	4,020
	12	0.105	1.79	0.515	0.338	0.182	0.811	0.692	0.128	0.499	12,840	2,330	3,270
2 x 2	10	0.135	1.78	0.511	0.204	0.141	0.632	0.569	0.0756	0.385	17,770	2,510	2,540
	12	0.105	1.43	0.410	0.173	0.116	0.649	0.567	0.0643	0.396	15,440	1,790	2,090
	14	0.075	1.08	0.311	0.144	0.092	0.680	0.570	0.0555	0.423	10,260	940	1,650
	16	0.060	0.840	0.241	0.104	0.069	0.658	0.545	0.0404	0.409	7,880	540	1,240

The allowable bending moments shown in this Table apply only when the sections are adequately braced laterally.

Where the vertical legs of the angles are in compression, M_{max} is based on the values of f_c (see 3.2 of Design Specification) indicated; where the vertical legs of the angles are in tension M_{max} is based on f_b (tension) since the compression stress is always less than f_c for the sections listed.



EQUAL LEG ANGLE
WITH
UNSTIFFENED LEGS

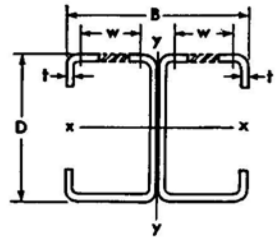


Beam Strength (cont'd) f_b = 27,000 psi			Column Form Factor Q		Dimensions of Sections					
	M max.	**M** max.								
f_c	Comp. Tension	Comp. Tension	f_b = 18,000 psi	f_b = 27,000 psi	**B**	**t**	**R**	Wt./Ft.	Gage	Size
psi	In.-Lb.	In.-Lb.			In.	In.	In.	Lbs.	No.	In.
11020	6410	15720	0.537	0.408	4.01	0.135	3/16	3.66	10	4 x 4
18770	6090	8750	0.762	0.695	3.01	0.135	3/16	2.72	10	3 x 3
12110	3170	7070	0.568	0.448	3.05	0.105	3/16	2.16	12	
22650	5060	6030	0.875	0.839	2.51	0.135	3/16	2.25	10	2½ x 2½
17090	3110	4900	0.713	0.633	2.55	0.105	3/16	1.79	12	
26530	3750	3810	0.987	0.983	2.01	0.135	3/16	1.78	10	2 x 2
22080	2560	3140	0.858	0.818	2.05	0.105	3/16	1.43	12	
12160	1110	2470	0.570	0.450	2.14	0.075	3/32	1.08	14	
7880	540	1860	0.437	0.292	2.06	0.060	3/32	0.840	16	

DIMENSIONS: Equipment and forming practices vary with different manufacturers, resulting in minor variations in some of these dimensions. These minor variations do not affect the published properties. Consult the manufacturer for actual weight per foot and actual dimensions.

TABLE 4

2 CHANNELS
WITH
STIFFENED FLANGES
BACK-TO-BACK

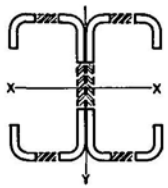


Effective section for beam strength about the x-x axis where w/t of compression flange exceeds 28.2 for $f_b = 18{,}000$ psi and 23.1 for $f_b = 27{,}000$ psi.

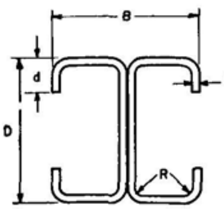
Nominal		t	Wt./Ft.	Beam Strength			Properties of Full Section			
				Effective				Axis x-x		
				S_x		S_y	Area			
Size D x B	Gage			$f_b = 18{,}000$ psi	$f_b = 27{,}000$ psi	$f_b = 18{,}000$ $f_b = 27{,}000$		I_x	S_x	r_x
In.	No.	In.	Lbs.	In.^{3}	In.^{3}	In.^{3}	Sq. In.	In.^{4}	In.^{3}	In.
12 x 7	10	0.135	18.8	18.7	18.7	3.58	5.40	112.0	18.7	4.56
	12	0.105	14.6	14.6	14.4	2.70	4.20	87.7	14.6	4.57
10 x 7	10	0.135	17.0	14.6	14.6	3.58	4.86	73.0	14.6	3.87
	12	0.105	13.1	11.4	11.3	2.70	3.78	57.0	11.4	3.89
	14	0.075	9.34	7.78	7.42	1.81	2.68	41.0	7.78	3.91
9 x 6½	10	0.135	15.5	12.1	12.1	3.17	4.46	54.3	12.1	3.49
	12	0.105	11.9	9.32	9.28	2.26	3.42	41.9	9.32	3.50
	14	0.075	8.56	6.56	6.28	1.60	2.46	30.6	6.56	3.53
	16	0.060	6.80	4.95	4.69	1.22	1.95	24.3	4.95	3.53
8 x 6	10	0.135	13.9	9.66	9.66	2.63	4.00	38.6	9.66	3.11
	12	0.105	10.8	7.56	7.56	1.98	3.10	30.2	7.56	3.12
	14	0.075	7.78	5.42	5.21	1.40	2.24	22.1	5.42	3.15
	16	0.060	6.16	4.11	3.91	1.06	1.77	17.6	4.11	3.15
7 x 5½	10	0.135	12.3	7.50	7.50	2.14	3.54	26.2	7.50	2.72
	12	0.105	9.72	5.96	5.96	1.71	2.78	20.9	5.96	2.74
	14	0.075	7.00	4.35	4.21	1.21	2.00	15.3	4.35	2.76
	16	0.060	5.54	3.33	3.18	0.919	1.59	12.2	3.33	2.77
6 x 5	10	0.135	10.7	5.62	5.62	1.71	3.08	16.8	5.62	2.34
	12	0.105	8.46	4.48	4.48	1.37	2.42	13.4	4.48	2.35
	14	0.075	6.20	3.35	3.29	1.04	1.78	10.1	3.35	2.38
	16	0.060	4.92	2.61	2.50	0.785	1.41	8.02	2.61	2.39
5 x 4	10	0.135	8.86	3.76	3.76	1.18	2.54	9.38	3.76	1.92
	12	0.105	7.00	3.02	3.02	0.950	2.00	7.53	3.02	1.94
	14	0.075	5.06	2.24	2.24	0.671	1.45	5.60	2.24	1.96
	16	0.060	4.00	1.78	1.75	0.500	1.15	4.45	1.78	1.97
	18	0.048	3.22	1.41	1.36	0.406	0.922	3.61	1.41	1.98
4 x 4	10	0.135	7.92	2.76	2.76	1.18	2.28	5.51	2.76	1.56
	12	0.105	6.28	2.22	2.22	0.950	1.80	4.44	2.22	1.57
	14	0.075	4.54	1.66	1.66	0.670	1.30	3.33	1.66	1.60
	16	0.060	3.58	1.33	1.31	0.500	1.03	2.66	1.33	1.61
	18	0.048	2.88	1.06	1.01	0.406	0.826	2.16	1.06	1.62
3½ x 4	10	0.135	7.46	2.30	2.30	1.18	2.14	4.01	2.30	1.37
	12	0.105	5.90	1.85	1.85	0.950	1.69	3.24	1.85	1.38
	14	0.075	4.28	1.40	1.40	0.670	1.23	2.45	1.40	1.41
	16	0.060	3.36	1.12	1.10	0.500	0.966	1.96	1.12	1.42
	18	0.048	2.72	0.888	0.850	0.406	0.778	1.59	0.888	1.43
3 x 3½	12	0.105	5.18	1.36	1.36	0.767	1.48	2.04	1.36	1.17
	14	0.075	3.64	1.02	1.02	0.491	1.05	1.53	1.02	1.21
	16	0.060	2.94	0.832	0.831	0.402	0.846	1.25	0.832	1.22
	18	0.048	2.32	0.661	0.643	0.294	0.662	0.995	0.661	1.23

The properties of this Table apply to flexural computations only when the channels are adequately joined together.

TABLE 4



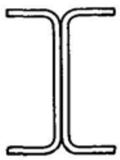
2 CHANNELS
WITH
STIFFENED FLANGES
BACK-TO-BACK



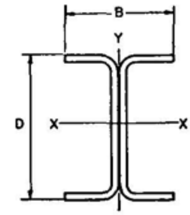
Properties of Full Section (cont'd)			Column Form Factor		Dimensions of Sections							
Axis y-y			Q									
I_y	S_y	r_y	f_b = 18,000 psi	f_b = 27,000 psi	D	B	d	t	R	Wt./Ft.	Gage	Size D x B
In.⁴	In.³	In.			In.	In.	In.	In.	In.	Lbs.	No.	In.
12.52	3.58	1.52	0.750	0.702	12.0	7.0	1.0	0.135	3/16	18.8	10	12 x 7
9.45	2.70	1.50	0.687	0.637	12.0	7.0	0.9	0.105	3/16	14.6	12	
12.52	3.58	1.60	0.818	0.769	10.0	7.0	1.0	0.135	3/16	17.0	10	10 x 7
9.45	2.70	1.58	0.754	0.702	10.0	7.0	0.9	0.105	3/16	13.1	12	
6.33	1.81	1.54	0.632	0.563	10.0	7.0	0.7	0.075	3/32	9.34	14	
10.31	3.17	1.52	0.851	0.809	9.0	6.5	1.0	0.135	3/16	15.5	10	9 x 6½
7.34	2.26	1.47	0.783	0.735	9.0	6.5	0.8	0.105	3/16	11.9	12	
5.19	1.60	1.45	0.672	0.603	9.0	6.5	0.7	0.075	3/32	8.56	14	
3.96	1.22	1.42	0.588	0.520	9.0	6.5	0.6	0.060	3/32	6.80	16	
7.90	2.63	1.41	0.885	0.835	8.0	6.0	0.9	0.135	3/16	13.9	10	8 x 6
5.94	1.98	1.38	0.820	0.773	8.0	6.0	0.8	0.105	3/16	10.8	12	
4.20	1.40	1.37	0.719	0.648	8.0	6.0	0.7	0.075	3/32	7.78	14	
3.19	1.06	1.34	0.633	0.563	8.0	6.0	0.6	0.060	3/32	6.16	16	
5.90	2.14	1.29	0.923	0.874	7.0	5.5	0.8	0.135	3/16	12.3	10	7 x 5½
4.72	1.71	1.30	0.860	0.813	7.0	5.5	0.8	0.105	3/16	9.72	12	
3.33	1.21	1.29	0.763	0.698	7.0	5.5	0.7	0.075	3/32	7.00	14	
2.53	0.919	1.26	0.684	0.613	7.0	5.5	0.6	0.060	3/32	5.54	16	
4.26	1.71	1.18	0.961	0.918	6.0	5.0	0.7	0.135	3/16	10.7	10	6 x 5
3.42	1.37	1.19	0.904	0.857	6.0	5.0	0.7	0.105	3/16	8.46	12	
2.60	1.04	1.21	0.813	0.756	6.0	5.0	0.7	0.075	3/32	6.20	14	
1.96	0.785	1.18	0.742	0.672	6.0	5.0	0.6	0.060	3/32	4.92	16	
2.36	1.18	0.962	0.993	0.962	5.0	4.0	0.7	0.135	3/16	8.86	10	5 x 4
1.90	0.950	0.973	0.950	0.906	5.0	4.0	0.7	0.105	3/16	7.00	12	
1.34	0.671	0.961	0.857	0.810	5.0	4.0	0.6	0.075	3/32	5.06	14	
1.00	0.500	0.934	0.799	0.744	5.0	4.0	0.5	0.060	3/32	4.00	16	
0.813	0.406	0.939	0.735	0.668	5.0	4.0	0.5	0.048	3/32	3.22	18	
2.35	1.18	1.02	1.000	0.998	4.0	4.0	0.7	0.135	3/16	7.92	10	4 x 4
1.90	0.950	1.03	0.993	0.966	4.0	4.0	0.7	0.105	3/16	6.28	12	
1.34	0.670	1.02	0.925	0.883	4.0	4.0	0.6	0.075	3/32	4.54	14	
1.00	0.500	0.987	0.873	0.818	4.0	4.0	0.5	0.060	3/32	3.58	16	
0.812	0.406	0.992	0.808	0.737	4.0	4.0	0.5	0.048	3/32	2.88	18	
2.35	1.18	1.05	1.000	1.000	3.5	4.0	0.7	0.135	3/16	7.46	10	3½ x 4
1.90	0.950	1.06	1.000	0.989	3.5	4.0	0.7	0.105	3/16	5.90	12	
1.34	0.670	1.05	0.958	0.921	3.5	4.0	0.6	0.075	3/32	4.28	14	
1.00	0.500	1.02	0.912	0.859	3.5	4.0	0.5	0.060	3/32	3.36	16	
0.812	0.406	1.02	0.849	0.776	3.5	4.0	0.5	0.048	3/32	2.72	18	
1.34	0.767	0.951	1.000	1.000	3.0	3.5	0.7	0.105	3/16	5.18	12	3 x 3½
0.860	0.491	0.906	0.984	0.953	3.0	3.5	0.5	0.075	3/32	3.64	14	
0.703	0.402	0.912	0.948	0.910	3.0	3.5	0.5	0.060	3/32	2.94	16	
0.515	0.294	0.881	0.899	0.837	3.0	3.5	0.4	0.048	3/32	2.32	18	

DIMENSIONS: Equipment and forming practices vary with different manufacturers, resulting in minor variations in some of these dimensions. These minor variations do not affect the published properties. Consult the manufacturer for actual weight per foot and actual dimensions.

TABLE 5

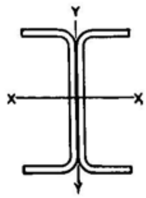


2 CHANNELS
WITH
UNSTIFFENED FLANGES
BACK-TO-BACK

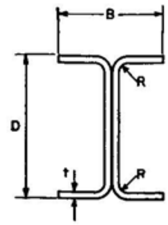


Nominal				Properties of Full Section						
					Axis x-x			Axis y-y		
Size D x B	Gage	t	Wt./Ft.	Area	I_x	S_x	r_x	I_y	S_y	r_y
In.	No.	In.	Lbs.	Sq. In.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.
8 x 4	10	0.135	10.8	3.10	25.8	6.48	2.89	1.38	0.704	0.669
	12	0.105	8.46	2.42	20.6	5.14	2.91	1.10	0.555	0.674
	14	0.075	6.16	1.77	15.3	3.82	2.94	0.834	0.411	0.687
	16	0.060	4.88	1.40	12.0	3.00	2.93	0.586	0.303	0.647
7 x 3	10	0.135	8.78	2.52	15.1	4.30	2.45	0.510	0.363	0.450
	12	0.105	7.00	2.00	12.4	3.54	2.48	0.464	0.313	0.481
	14	0.075	5.06	1.45	9.08	2.60	2.50	0.315	0.215	0.466
	16	0.060	3.98	1.14	7.08	2.02	2.49	0.211	0.152	0.429
6 x 3	10	0.135	7.82	2.24	10.2	3.40	2.13	0.509	0.362	0.476
	12	0.105	6.26	1.80	8.44	2.80	2.17	0.464	0.312	0.508
	14	0.075	4.52	1.30	6.20	2.06	2.19	0.315	0.215	0.492
	16	0.060	3.56	1.02	4.84	1.61	2.17	0.211	0.152	0.454
	18	0.048	2.86	0.818	3.88	1.29	2.18	0.162	0.119	0.445
5 x 2½	12	0.105	5.16	1.48	4.76	1.91	1.79	0.268	0.217	0.425
	14	0.075	3.68	1.06	3.42	1.37	1.80	0.154	0.134	0.382
	16	0.060	2.94	0.844	2.74	1.09	1.80	0.116	0.103	0.370
	18	0.048	2.30	0.662	2.12	0.846	1.79	0.074	0.071	0.335
4 x 2¼	12	0.105	4.34	1.25	2.66	1.33	1.46	0.229	0.195	0.429
	14	0.075	3.22	0.924	2.04	1.02	1.49	0.180	0.148	0.441
	16	0.060	2.52	0.724	1.58	0.792	1.48	0.116	0.103	0.400
	18	0.048	2.02	0.578	1.27	0.636	1.48	0.088	0.079	0.390
3 x 2¼	12	0.105	3.60	1.04	1.32	0.878	1.13	0.228	0.195	0.470
	14	0.075	2.70	0.774	1.03	0.688	1.15	0.180	0.148	0.481
	16	0.060	2.10	0.604	0.796	0.530	1.15	0.115	0.102	0.437
	18	0.048	1.68	0.482	0.638	0.426	1.15	0.088	0.079	0.427
2 x 2¼	12	0.105	2.88	0.826	0.500	0.500	0.779	0.227	0.194	0.525
	14	0.075	2.18	0.624	0.400	0.400	0.800	0.179	0.148	0.536
	16	0.060	1.69	0.484	0.308	0.308	0.799	0.115	0.102	0.488
	18	0.048	1.35	0.386	0.248	0.248	0.802	0.088	0.079	0.477

TABLE 5



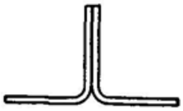
2 CHANNELS
WITH
UNSTIFFENED FLANGES
BACK-TO-BACK



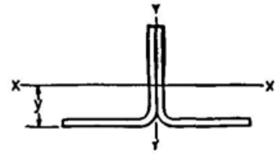
Allowable Beam Stress f_c		Column Form Factor Q		Dimensions of Sections						
f_b = 18,000 psi	f_b = 27,000 psi	f_b = 18,000 psi	f_b = 27,000 psi	D In.	B In.	t In.	R In.	Wt./Ft. Lbs.	Gage No.	Size D x B In.
17960	26890	0.850	0.784	8.0	3.934	0.135	3/16	10.8	10	8 x 4
15800	22790	0.693	0.619	8.0	3.972	0.105	3/16	8.46	12	
10880	13740	0.448	0.354	8.0	4.052	0.075	3/32	6.16	14	
8330	8450	0.323	0.218	8.0	3.882	0.060	3/32	4.88	16	
18000	27000	0.891	0.823	7.0	2.810	0.135	3/16	8.78	10	7 x 3
18000	27000	0.806	0.740	7.0	2.972	0.105	3/16	7.00	12	
15190	21570	0.595	0.522	7.0	2.928	0.075	3/32	5.06	14	
13460	18280	0.485	0.411	7.0	2.758	0.060	3/32	3.98	16	
18000	27000	0.947	0.887	6.0	2.810	0.135	3/16	7.82	10	6 x 3
18000	27000	0.871	0.807	6.0	2.972	0.105	3/16	6.26	12	
15190	21570	0.650	0.573	6.0	2.928	0.075	3/32	4.52	14	
13460	18280	0.533	0.453	6.0	2.758	0.060	3/32	3.56	16	
10730	13070	0.405	0.315	6.0	2.722	0.048	3/32	2.86	18	
18000	27000	0.933	0.873	5.0	2.472	0.105	3/16	5.16	12	5 x 2½
17460	25950	0.784	0.715	5.0	2.302	0.075	3/32	3.68	14	
15730	22650	0.655	0.582	5.0	2.258	0.060	3/32	2.94	16	
14290	19870	0.545	0.471	5.0	2.098	0.048	3/32	2.30	18	
18000	27000	0.990	0.951	4.0	2.346	0.105	3/16	4.34	12	4 x 2¼
17010	25080	0.853	0.785	4.0	2.428	0.075	3/32	3.22	14	
15730	22650	0.736	0.660	4.0	2.258	0.060	3/32	2.52	16	
13590	18520	0.598	0.513	4.0	2.222	0.048	3/32	2.02	18	
18000	27000	1.000	1.000	3.0	2.346	0.105	3/16	3.60	12	3 x 2¼
17010	25080	0.929	0.878	3.0	2.428	0.075	3/32	2.70	14	
15730	22650	0.824	0.752	3.0	2.258	0.060	3/32	2.10	16	
13590	18520	0.681	0.590	3.0	2.222	0.048	3/32	1.68	18	
18000	27000	1.000	1.000	2.0	2.346	0.105	3/16	2.88	12	2 x 2¼
17010	25080	0.945	0.929	2.0	2.428	0.075	3/32	2.18	14	
15730	22650	0.874	0.834	2.0	2.258	0.060	3/32	1.69	16	
13590	18520	0.752	0.671	2.0	2.222	0.048	3/32	1.35	18	

DIMENSIONS: Equipment and forming practices vary with different manufacturers, resulting in minor variations in some of these dimensions. These minor variations do not affect the published properties. Consult the manufacturer for actual weight per foot and actual dimensions.

TABLE 6



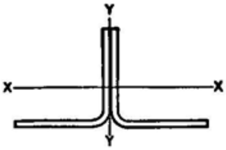
2 EQUAL LEG ANGLES
BACK-TO-BACK
UNSTIFFENED LEGS



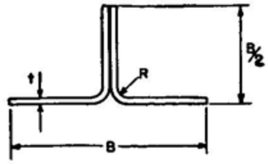
Nominal (One Angle)			BEAM STRENGTH							DEFLECTION	
			Section Modulus Based on Full Theoretical Outline	f_b = 18,000 psi			f_b = 27,000 psi			Any Grade of Steel	
					$M_{max.}$			$M_{max.}$			
Size	Gage	Thickness	S_x	f_c	Comp. / Tension	Comp. / Tension	f_c	Comp. / Tension	Comp. / Tension	y	I_x
In.	No.	In.	In.^{3}	psi	In.-Lbs.	In.-Lbs.	psi	In.-Lbs.	In.-Lbs.	In.	In.^{4}
4 x 4	10	.135	1.164	9660	11260	20960	11020	12820	31440	1.069	3.430
3 x 3	10	.135	0.648	13720	8900	11680	18770	12180	17500	0.819	1.424
	12	.105	0.524	10230	5360	9420	12110	6340	14140	0.817	1.172
2½ x 2½	10	.135	0.446	15740	7040	8040	22650	10120	12060	0.694	0.814
	12	.105	0.364	12840	4660	6540	17090	6220	9800	0.692	0.676
2 x 2	10	.135	0.282	17770	5020	5080	26530	7500	7620	0.569	0.408
	12	.105	0.232	15440	3580	4180	22080	5120	6280	0.567	0.346
	14	.075	0.183	10260	1880	3300	12160	2220	4940	0.570	0.288
	16	.060	0.137	7880	1080	2480	7880	1080	3720	0.546	0.208

The properties of Table 6 may be used for flexural computations only when the angles are adequately joined and adequately braced laterally. Q is the column factor (Sec. 3.6.1, Design Specification).

Where the vertical legs of the angles are in compression, M_{max} is based on the values of f_c (Sec. 3.2 of Design Specification) indicated; where the vertical legs of the angles are in tension M_{max} is based on f_b (tension) since the compression stress is always less than f_c for the sections listed.



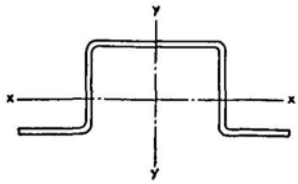
2 EQUAL LEG ANGLES
BACK-TO-BACK
UNSTIFFENED LEGS



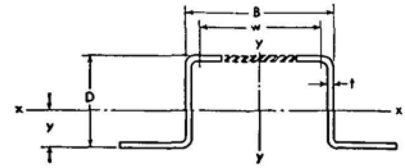
COLUMN PROPERTIES					DIMENSIONS				
Full Theoretical Outline			Q		B	Thickness t	Radius R	Weight per Ft.	Nominal Size
Area	r_x	r_y	f_b = 18,000 psi	f_b = 27,000 psi					
Sq. In.	In.	In.			In.	In.	In.	Lbs.	In.
2.10	1.28	1.67	0.537	0.408	8.030	.135	3/16	7.32	4 x 4
1.56	0.955	1.26	0.762	0.695	6.030	.135	3/16	5.44	3 x 3
1.24	0.972	1.27	0.568	0.448	6.110	.105	3/16	4.32	
1.29	0.793	1.05	0.875	0.839	5.030	.135	3/16	4.50	2½ x 2½
1.03	0.811	1.07	0.713	0.633	5.110	.105	3/16	3.58	
1.02	0.632	0.850	0.987	0.983	4.030	.135	3/16	3.56	2 x 2
0.820	0.649	0.862	0.858	0.818	4.110	.105	3/16	2.86	
0.622	0.680	0.887	0.570	0.450	4.276	.075	3/32	2.16	
0.482	0.658	0.855	0.437	0.292	4.128	.060	3/32	1.68	

DIMENSIONS: Equipment and forming practices vary with different manufacturers, resulting in minor variations in some of these dimensions. These minor variations do not affect the published properties. Consult the manufacturer for actual weight per foot and actual dimensions.

TABLE 7



HAT SECTIONS

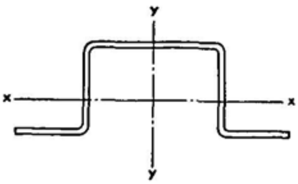


Effective section for beam strength about the x-x axis where w/t of compression flange exceeds 28.2 for $f_b = 18{,}000$ psi and 23.1 for $f_b = 27{,}000$ psi.

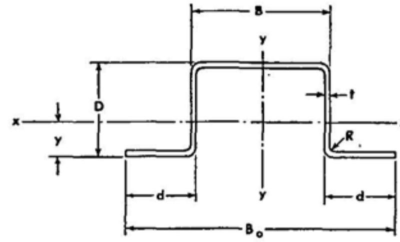
Nominal Size D x B	Nominal Gage	t	Wt./Ft.	For Beam Strength Calculations: Comp. / Tension, f_b = 18,000 psi, S_x	Comp. / Tension, f_b = 18,000 psi, S^1_x	Comp. / Tension, f_b = 27,000 psi, S_x	Comp. / Tension, f_b = 27,000 psi, S^1_x	For Deflection Calculations: Comp. / Tension, Full Section, I_x	Comp. / Tension, f_b = 18,000 psi, I_x	Comp. / Tension, f_b = 27,000 psi, I_x	comp. / Tension, I^1_x	For Column Strength, Full (solid) Section: Area	r_x	r_y
In.	No.	In.	Lbs.	In.³	In.³	In.³	In.³	In.⁴	In.⁴	In.⁴	In.⁴	Sq. In.	In.	In.
10 x 15	10	0.135	17.40	9.99	10.9	9.71	10.9	70.6	62.3	58.5	70.6	5.12	3.71	6.50
	12	0.105	13.3	6.98	7.92	6.78	7.92	52.1	44.0	40.3	52.1	3.91	3.65	6.45
10 x 10	12	0.105	11.5	6.90	7.46	6.72	7.46	45.2	41.0	39.1	45.2	3.39	3.65	4.50
	14	0.075	8.06	4.28	4.87	4.17	4.87	30.3	25.9	23.9	30.3	2.37	3.58	4.45
10 x 5	14	0.075	6.79	4.18	4.36	4.10	4.36	24.1	23.1	22.6	24.1	2.00	3.48	2.35
8 x 12	10	0.135	14.2	7.12	7.58	6.94	7.58	38.2	35.1	33.6	38.2	4.17	3.02	5.26
	12	0.105	10.8	4.94	5.43	4.80	5.43	28.0	24.6	23.1	28.0	3.18	2.97	5.20
8 x 8	12	0.105	9.38	4.86	5.12	4.75	5.12	24.3	22.8	22.2	24.3	2.76	2.97	3.64
	14	0.075	6.53	2.99	3.29	2.91	3.29	16.1	14.3	13.5	16.1	1.92	2.90	3.58
8 x 4	14	0.075	5.51	2.89	2.95	2.84	2.95	12.8	12.6	12.4	12.8	1.62	2.81	1.90
	16	0.060	4.34	2.15	2.24	2.10	2.24	9.89	9.49	9.30	9.89	1.28	2.78	1.88
6 x 9	10	0.135	11.0	4.64	4.79	4.54	4.79	17.4	16.7	16.4	17.4	3.23	2.32	4.03
	12	0.105	8.37	3.20	3.38	3.12	3.38	12.7	11.8	11.3	12.7	2.44	2.28	3.96
6 x 6	12	0.105	7.24	3.12	3.20	3.06	3.20	10.9	10.6	10.5	10.9	2.13	2.27	2.79
	14	0.075	5.00	1.90	2.01	1.85	2.01	7.21	6.71	6.48	7.21	1.47	2.21	2.52
6 x 3	14	0.075	4.24	1.80	1.81	1.78	1.81	5.71	5.71	5.71	5.71	1.25	2.14	1.46
	16	0.060	3.32	1.33	1.35	1.31	1.35	4.38	4.33	4.30	4.38	0.977	2.12	1.43
	18	0.048	2.62	0.994	1.03	0.975	1.03	3.38	3.28	3.22	3.38	0.770	2.10	1.42
4 x 6	10	0.135	7.76	2.57	2.58	2.54	2.58	5.78	5.77	5.76	5.78	2.28	1.59	2.84
	12	0.105	5.81	1.76	1.79	1.73	1.79	4.20	4.11	4.07	4.20	1.71	1.57	2.75
4 x 4	12	0.105	5.09	1.69	1.69	1.67	1.69	3.59	3.59	3.59	3.59	1.50	1.55	1.97
	14	0.075	3.47	1.02	1.03	1.00	1.03	2.35	2.31	2.29	2.35	1.02	1.52	2.12
4 x 2	14	0.075	2.96	0.910	0.910	0.910	0.910	1.84	1.84	1.84	1.84	0.871	1.45	1.15
	16	0.060	2.30	0.686	0.686	0.683	0.686	1.40	1.40	1.40	1.40	0.677	1.44	1.12
	18	0.048	1.80	0.511	0.513	0.505	0.513	1.08	1.08	1.08	1.08	0.530	1.43	0.970
3 x 4½	10	0.135	6.15	1.69	1.69	1.68	1.69	2.64	2.64	2.64	2.64	1.81	1.21	2.28
	12	0.105	4.56	1.16	1.16	1.14	1.16	1.92	1.92	1.92	1.92	1.34	1.20	2.16
3 x 3	12	0.105	4.02	1.08	1.08	1.08	1.08	1.63	1.63	1.63	1.63	1.18	1.17	1.58
	14	0.075	2.71	0.656	0.658	0.649	0.658	1.07	1.07	1.07	1.07	0.796	1.16	1.45
3 x 1½	14	0.075	2.32	0.518	0.518	0.518	0.518	0.830	0.830	0.830	0.830	0.684	1.10	0.838
	16	0.060	1.79	0.410	0.410	0.410	0.410	0.631	0.631	0.631	0.631	0.527	1.09	0.789
	18	0.048	1.39	0.320	0.320	0.320	0.320	0.484	0.484	0.484	0.484	0.410	1.09	0.759
2 x 4	12	0.105	3.67	0.689	0.689	0.685	0.689	0.746	0.746	0.746	0.746	1.08	0.832	1.98
	14	0.075	2.45	0.388	0.392	0.382	0.392	0.466	0.460	0.457	0.466	0.721	0.804	1.82
2 x 2	14	0.075	1.94	0.345	0.345	0.345	0.345	0.351	0.351	0.351	0.351	0.571	0.784	1.07
	16	0.060	1.49	0.256	0.256	0.255	0.256	0.265	0.265	0.265	0.265	0.437	0.779	1.01
2 x 1	16	0.060	1.28	0.183	0.183	0.183	0.183	0.204	0.204	0.204	0.204	0.377	0.736	0.606
	18	0.048	0.990	0.147	0.147	0.147	0.147	0.157	0.157	0.157	0.157	0.290	0.736	0.562
1½ x 3	12	0.105	2.95	0.444	0.444	0.444	0.444	0.337	0.337	0.337	0.337	0.868	0.623	1.63
	14	0.075	1.94	0.258	0.258	0.256	0.258	0.215	0.215	0.215	0.215	0.571	0.613	1.45
1½ x 1½	14	0.075	1.56	0.192	0.192	0.192	0.192	0.158	0.158	0.158	0.158	0.459	0.588	0.893
	16	0.060	1.18	0.154	0.154	0.154	0.154	0.120	0.120	0.120	0.120	0.347	0.588	0.822
1½ x ¾	16	0.060	1.03	0.102	0.102	0.102	0.102	0.091	0.091	0.091	0.091	0.302	0.549	0.489
	18	0.048	0.780	0.083	0.083	0.083	0.083	0.071	0.071	0.071	0.071	0.230	0.554	0.478

NOTE: S^1_x and I^1_x in this Table are properties of full (unreduced) section, and are applicable as indicated. The effective section moduli in bending about the y-y axis have not been tabulated. When one of the webs acts as a compression flange the section modulus should be calculated on the basis of its effective width as provided in Section 2 3 of the Design Specification. When the web acts as a tension flange the section modulus of the full section is effective.

TABLE 7



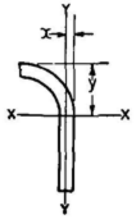
HAT SECTIONS



Column Form Factor Q		Additional Properties of Full (Unreduced) Sections			Dimensions of Sections								
f_b = 18,000 psi	f_b = 27,000 psi	S_y	I_y	y	D	B	B_o	d	t	R	Wt./Ft.	Gage	Size D x B
		In.³	In.⁴	In.	In.	In.	In.	In.	In.	In.	Lbs.	No.	In.
0.623	0.548	23.3	216.4	6.45	10.00	15.00	18.61	1.94	0.135	3/16	17.40	10	10 x 15
0.519	0.454	18.2	163.0	6.58	10.0	15.0	17.89	1.55	0.105	3/16	13.3	12	
0.590	0.517	10.6	68.55	6.06	10.0	10.0	12.89	1.55	0.105	3/16	11.5	12	10 x 10
0.441	0.382	7.84	47.02	6.23	10.0	10.0	11.99	1.07	0.075	3/32	8.06	14	
0.506	0.442	3.16	11.06	5.53	10.0	5.0	6.99	1.07	0.075	3/32	6.79	14	10 x 5
0.728	0.648	14.8	115.5	5.03	8.0	12.0	15.61	1.94	0.135	3/16	14.2	10	8 x 12
0.617	0.544	11.6	86.0	5.16	8.0	12.0	14.89	1.55	0.105	3/16	10.8	12	
0.696	0.616	6.70	36.46	4.74	8.0	8.0	10.89	1.55	0.105	3/16	9.38	12	8 x 8
0.531	0.462	4.93	24.63	4.91	8.0	8.0	9.99	1.07	0.075	3/32	6.53	14	
0.599	0.528	1.96	5.857	4.34	8.0	4.0	5.99	1.07	0.075	3/32	5.51	14	8 x 4
0.510	0.446	1.61	4.536	4.42	8.0	4.0	5.62	0.87	0.060	3/32	4.34	16	
0.860	0.784	8.33	52.51	3.63	6.0	9.0	12.61	1.94	0.135	3/16	11.0	10	6 x 9
0.753	0.675	6.45	38.34	3.75	6.0	9.0	11.89	1.55	0.105	3/16	8.31	12	
0.836	0.755	3.72	16.53	3.42	6.0	6.0	8.89	1.55	0.105	3/16	7.24	12	6 x 6
0.661	0.583	2.33	9.31	3.58	6.0	6.0	7.99	1.07	0.075	3/32	5.00	14	
0.727	0.652	1.06	2.64	3.15	6.0	3.0	4.99	1.07	0.075	3/32	4.24	14	6 x 3
0.633	0.561	0.868	2.01	3.23	6.0	3.0	4.62	0.87	0.060	3/32	3.32	16	
0.543	0.478	0.714	1.55	3.29	6.0	3.0	4.34	0.72	0.048	3/32	2.62	18	
0.974	0.942	3.84	18.45	2.24	4.0	6.0	9.61	1.94	0.135	3/16	7.76	10	4 x 6
0.924	0.863	2.90	12.88	2.35	4.0	6.0	8.89	1.55	0.105	3/16	5.81	12	
0.987	0.939	1.68	5.80	2.12	4.0	4.0	6.89	1.55	0.105	3/16	5.09	12	4 x 4
0.857	0.776	1.53	4.57	2.27	4.0	4.0	5.99	1.07	0.075	3/32	3.47	14	
0.888	0.824	0.582	1.16	1.98	4.0	2.0	3.99	1.07	0.075	3/32	2.96	14	4 x 2
0.809	0.739	0.470	0.850	2.05	4.0	2.0	3.62	0.87	0.060	3/32	2.30	16	
0.722	0.649	0.300	0.502	2.10	4.0	2.0	3.34	0.72	0.048	3/32	1.80	18	
1.000	0.989	2.32	9.41	1.57	3.0	4.5	8.11	1.94	0.135	3/16	6.15	10	3 x 4½
0.982	0.955	1.69	6.26	1.66	3.0	4.5	7.39	1.55	0.105	3/16	4.56	12	
1.000	1.000	1.01	2.97	1.49	3.0	3.0	5.89	1.55	0.105	3/16	4.02	12	3 x 3
0.968	0.907	0.673	1.68	1.62	3.0	3.0	4.99	1.07	0.075	3/32	2.71	14	
0.975	0.928	0.275	0.480	1.40	3.0	1.5	3.49	1.07	0.075	3/32	2.32	14	3 x 1½
0.916	0.855	0.210	0.328	1.46	3.0	1.5	3.12	0.87	0.060	3/32	1.79	16	
0.841	0.776	0.166	0.236	1.51	3.0	1.5	2.84	0.72	0.048	3/32	1.39	18	
0.994	0.972	1.22	4.21	1.08	2.0	4.0	6.89	1.55	0.105	3/16	3.67	12	2 x 4
0.932	0.894	0.798	2.39	1.19	2.0	4.0	5.99	1.07	0.075	3/32	2.45	14	
1.000	1.000	0.325	0.649	0.982	2.0	2.0	3.99	1.07	0.075	3/32	1.94	14	2 x 2
1.000	0.977	0.245	0.444	1.04	2.0	2.0	3.62	0.87	0.060	3/32	1.49	16	
1.000	0.982	0.106	0.139	0.886	2.0	1.0	2.62	0.87	0.060	3/32	1.28	16	2 x 1
0.974	0.928	0.078	0.092	0.929	2.0	1.0	2.34	0.72	0.048	3/32	0.99	18	
1.000	1.000	0.785	2.31	0.742	1.5	3.0	5.89	1.55	0.105	3/16	2.95	12	1½ x 3
0.985	0.957	0.480	1.20	0.831	1.5	3.0	4.99	1.07	0.075	3/32	1.94	14	
1.000	1.000	0.210	0.366	0.675	1.5	1.5	3.49	1.07	0.075	3/32	1.56	14	1½ x 1½
1.000	1.000	0.151	0.235	0.720	1.5	1.5	3.12	0.87	0.060	3/32	1.18	16	
1.000	1.000	0.072	0.086	0.609	1.5	0.75	2.37	0.87	0.060	3/32	1.03	16	1½ x ¾
1.000	0.995	0.050	0.053	0.646	1.5	0.75	2.09	0.72	0.048	3/32	0.78	18	

DIMENSIONS: Equipment and forming practices vary with different manufacturers, resulting in minor variations in some of these dimensions. These minor variations do not affect the published properties. Consult the manufacturer for actual weight per foot and actual dimensions.

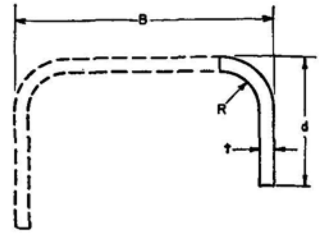
TABLE 8



ONE FLANGE STIFFENER

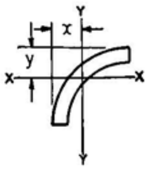
(Which Includes One 90° Corner)

PROPERTIES AND DIMENSIONS

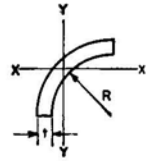


Stock Width of Blank Taken at t/3 Distance From Inner Surface.

Nominal	Thickness	Depth	PROPERTIES				Area	DIMENSIONS					Nominal
			X-X AXIS		Y-Y AXIS			Max. Flange	Blank Width	Thickness	Radius	Depth	
Gage	t	d	I_x	y	I_y	x		B		t	R	d	Gage
No.	In.	In.	$In.^4$	In.	$In.^4$	In.	Sq. In.	In.	In.	In.	In.	In.	No.
10	.135	1.0	.01255	.4737	.000796	.1005	.14554	3.87	1.043	.135	3/16	1.0	10
		.9	.00916	.4250	.000759	.1039	.13204	3.25	0.943	.135	3/16	.9	
		.8	.00643	.3766	.000719	.1080	.11854	2.81	0.843	.135	3/16	.8	
		.7	.00430	.3286	.000674	.1133	.10504	2.53	0.743	.135	3/16	.7	
		.6	.00269	.2811	.000621	.1200	.09154	2.37	0.643	.135	3/16	.6	
		.5	.00153	.2346	.000556	.1291	.07804	2.30	0.543	.135	3/16	.5	
		.4	.00076	.1896	.000474	.1420	.06454	2.27	0.443	.135	3/16	.4	
12	.105	.9	.00735	.4205	.000475	.0850	.10337	4.28	0.957	.105	3/16	.9	12
		.8	.00518	.3719	.000453	.0886	.09287	3.34	0.857	.105	3/16	.8	
		.7	.00348	.3237	.000428	.0932	.08237	2.66	0.757	.105	3/16	.7	
		.6	.00219	.2761	.000398	.0992	.07187	2.22	0.657	.105	3/16	.6	
		.5	.00126	.2292	.000362	.1072	.06137	1.98	0.557	.105	3/16	.5	
		.4	.00064	.1836	.000314	.1185	.05087	1.88	0.457	.105	3/16	.4	
14	.075	.9	.00493	.4351	.000081	.0475	.07031	6.79	0.918	.075	3/32	.9	14
		.8	.00348	.3855	.000076	.0487	.06281	4.94	0.818	.075	3/32	.8	
		.7	.00234	.3361	.000072	.0502	.05531	3.50	0.718	.075	3/32	.7	
		.6	.00148	.2869	.000067	.0522	.04781	2.46	0.618	.075	3/32	.6	
		.5	.00086	.2379	.000062	.0549	.04031	1.77	0.518	.075	3/32	.5	
		.4	.00044	.1894	.000055	.0589	.03281	1.40	0.418	.075	3/32	.4	
16	.060	.8	.00283	.3836	.000048	.0400	.05044	7.50	0.825	.060	3/32	.8	16
		.7	.00191	.3341	.000045	.0414	.04444	5.19	0.725	.060	3/32	.7	
		.6	.00121	.2848	.000043	.0432	.03844	3.46	0.625	.060	3/32	.6	
		.5	.00071	.2357	.000040	.0456	.03244	2.23	0.525	.060	3/32	.5	
		.4	.00036	.1871	.000036	.0492	.02644	1.48	0.425	.060	3/32	.4	
18	.048	.7	.00155	.3325	.000030	.0344	.03567	7.96	0.731	.048	3/32	.7	18
		.6	.00099	.2831	.000028	.0360	.03087	5.19	0.631	.048	3/32	.6	
		.5	.00058	.2340	.000026	.0382	.02607	3.20	0.531	.048	3/32	.5	
		.4	.00030	.1853	.000024	.0414	.02127	1.87	0.431	.048	3/32	.4	



ONE 90° CORNER
PROPERTIES AND DIMENSIONS



Stock Width of Blank Taken at $t/3$ Distance From Inner Surface.

Nominal	Thickness	PROPERTIES		Area	Blank Width	DIMENSIONS		Nominal
		Moment of Inertia				Thickness	Radius Inside	
Gage	t	$I_x = I_y$	$x=y$			t	R	Gage
No.	In.	$In.^4$	In.	Sq. In.	In.	In.	In.	No.
10	.135	.0003889	.1564	.05407	.3652	.135	.1875	10
12	.105	.0002408	.1373	.03958	.3495	.105	.1875	12
14	.075	.0000301	.0829	.01546	.1865	.075	.0938	14
16	.060	.0000193	.0734	.01166	.1787	.060	.0938	16
18	.048	.0000128	.0658	.00888	.1724	.048	.0938	18
20	.036	.00000313	.0464	.00452	.1170	.036	.0625	20

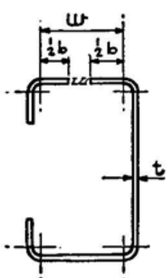
TABLE 10

Standard Weights and Thickness of Carbon Steel Sheets

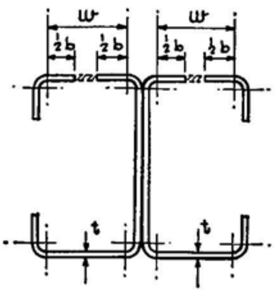
U. S. Standard Gage Number	Weight in lbs. per sq. ft.	Approximate Thickness, in.
10	5.625	0.1345
12	4.375	.1046
14	3.125	.0747
16	2.500	.0598
18	2.000	.0478
20	1.500	.0359
22	1.250	.0299
24	1.000	.0239
26	.750	.0179
28	.625	.0149
30	.500	.0120

U. S. Standard Gage is a weight gage, the basis of each gage number being the weight per square foot. The thicknesses shown in the above table are based upon a weight of 41.820 pounds per square foot per inch thickness. In the other tables in this manual the thicknesses have been rounded out to three decimal places.

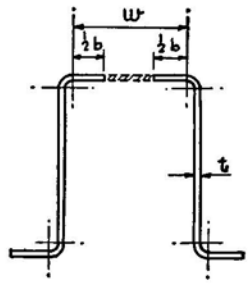
EFFECTIVE CROSS SECTIONS OF MEMBERS IN BENDING



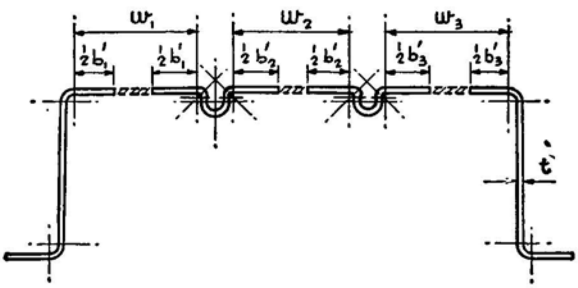
Lipped Channel
or
"C" Section



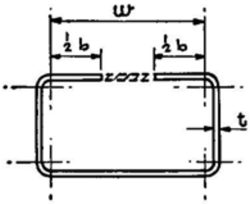
Double Channel I-Beam
with Stiffened Flanges



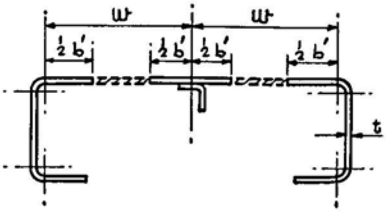
Hat Section



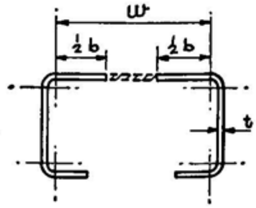
Multiple Stiffened Hat Section



Box-Type Section



Multiple Stiffened
Inverted "U" Section

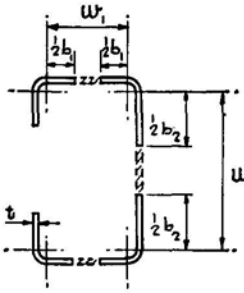


Inverted "U" Section

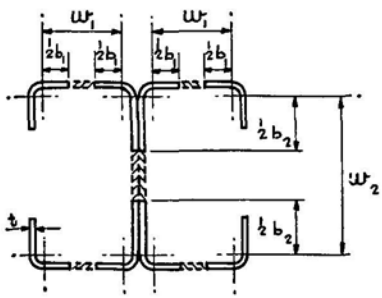
EFFECTIVE DESIGN AREA FOR DETERMINING "Q" FOR CROSS SECTIONS OF MEMBERS IN COMPRESSION

(See Section 3.6.1)

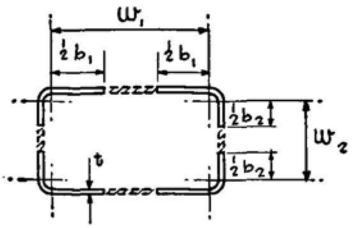
$$Q = \frac{\text{Total Area} - \text{Shaded Area}}{\text{Total Area}} \quad \left(\begin{array}{l}\text{WHERE ALL ELEMENTS ARE} \\ \text{STIFFENED AT BOTH EDGES}\end{array}\right)$$



Lipped Channel
or
"C" Section



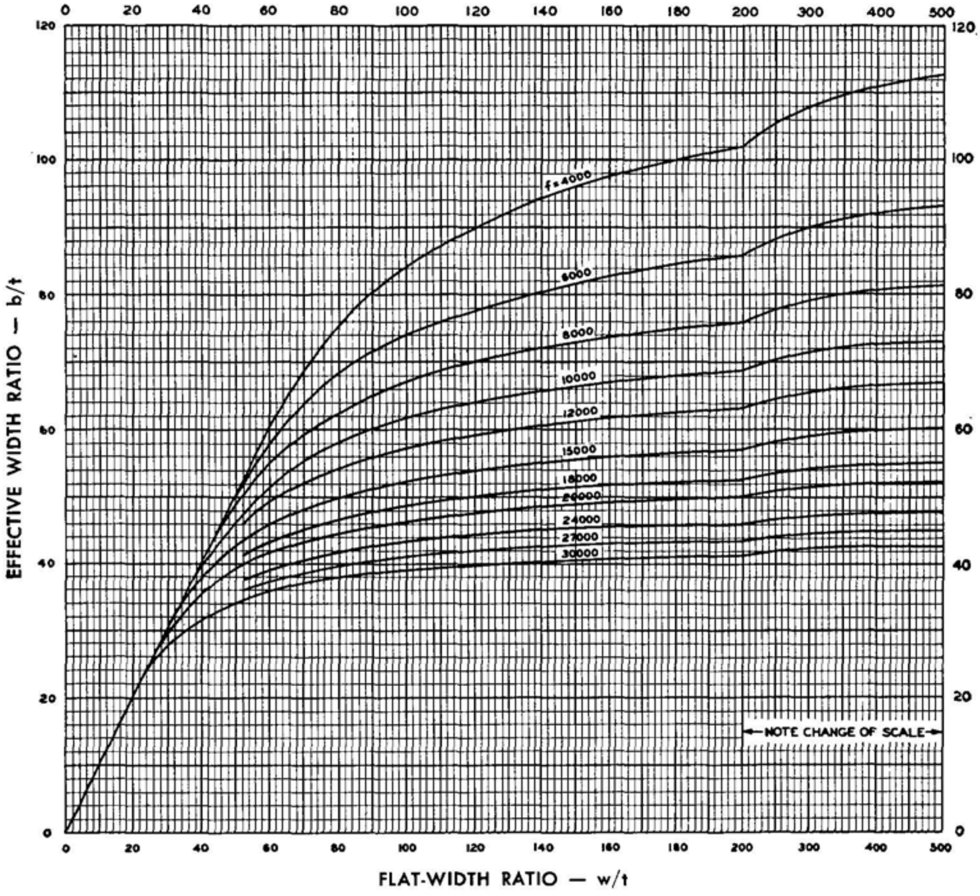
Double Channel I-Beam
with Stiffened Flanges



Box-Type Section

CHART 3A

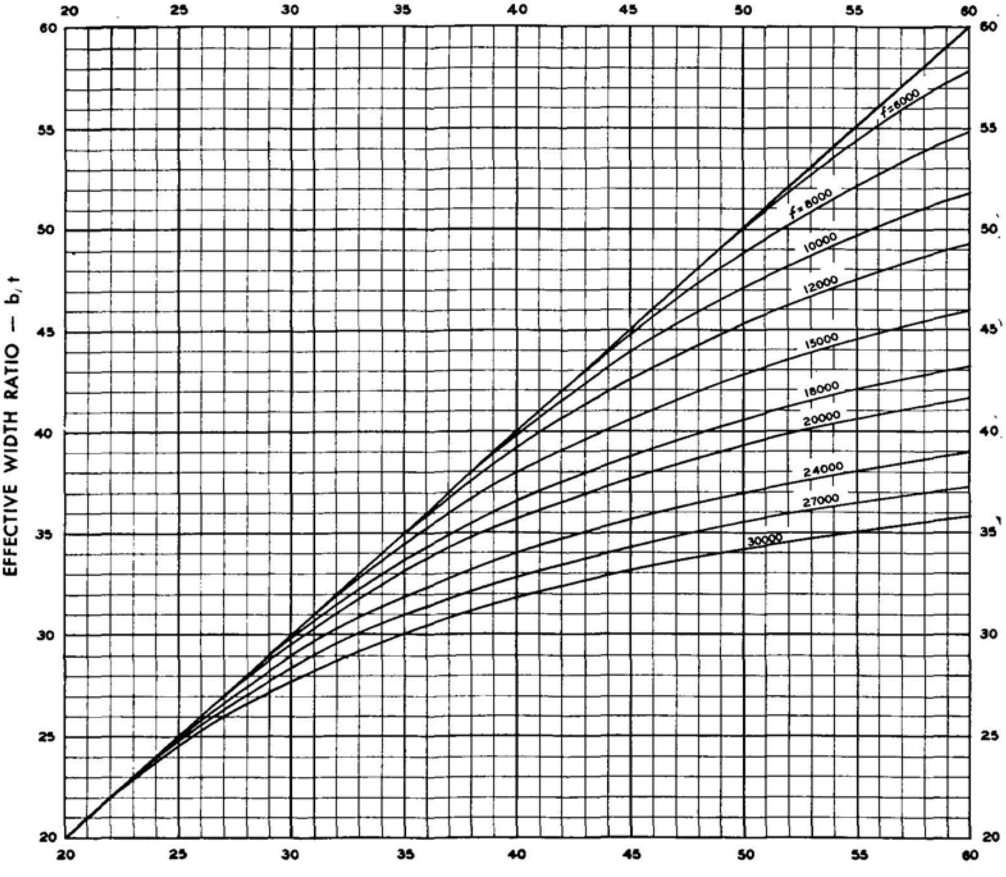
EFFECTIVE WIDTH FOR SAFE LOAD DETERMINATION



COMPRESSION ELEMENTS STIFFENED ALONG BOTH EDGES (Large w/t Ratios)

CHART 3B

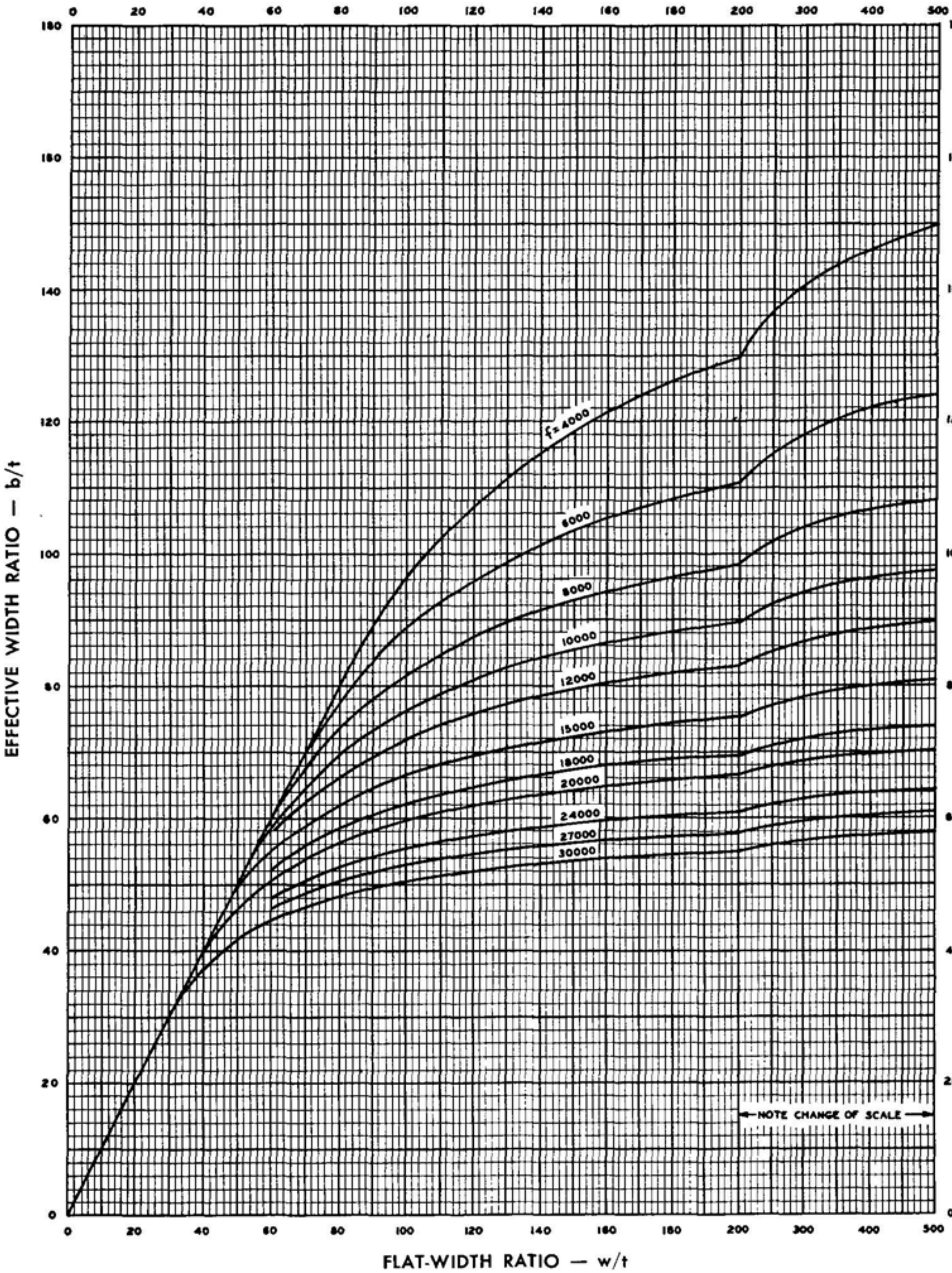
EFFECTIVE WIDTH FOR SAFE LOAD DETERMINATION



COMPRESSION ELEMENTS STIFFENED ALONG BOTH EDGES (Small w/t Ratios)

CHART 3C

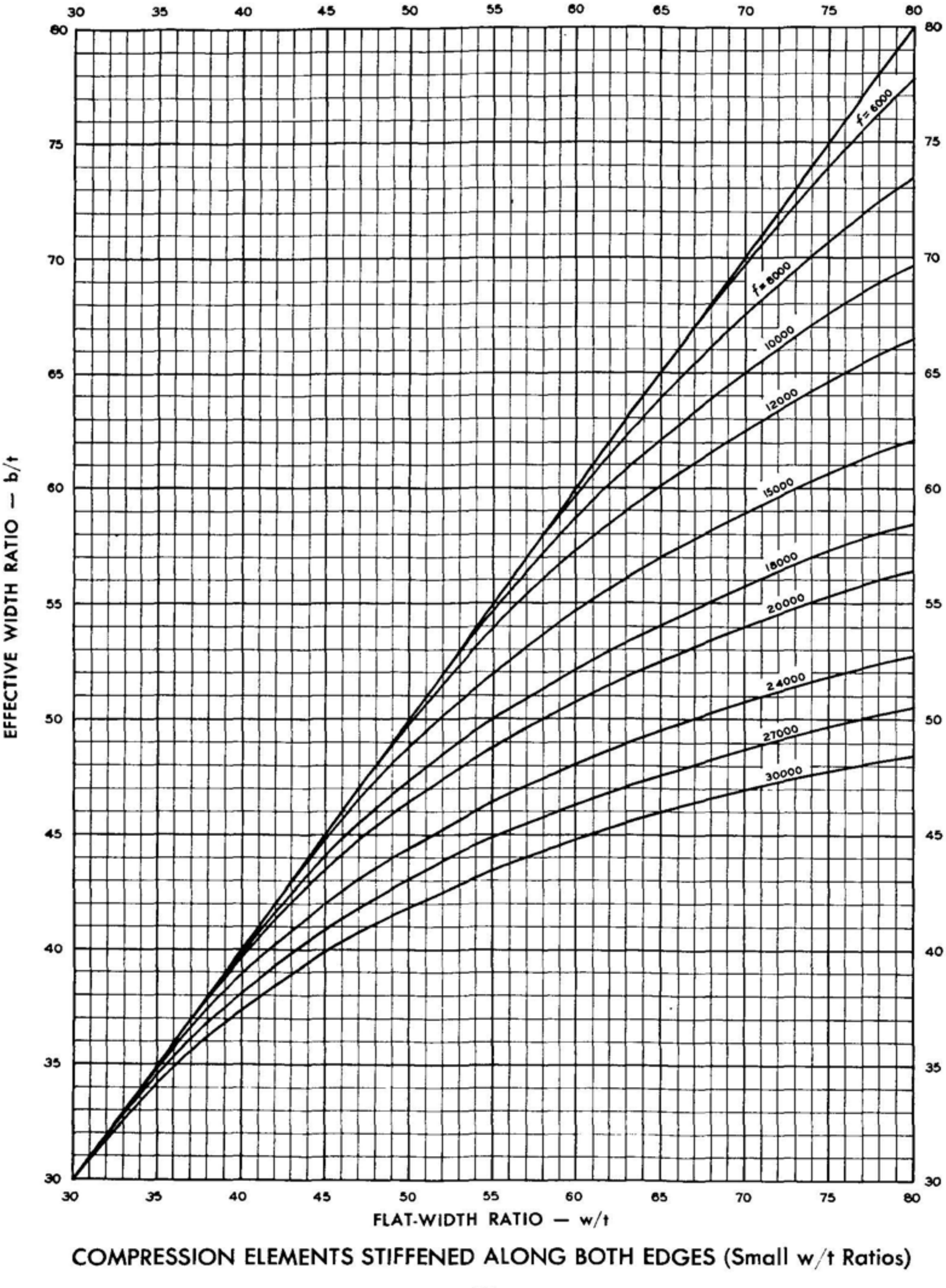
EFFECTIVE WIDTH FOR DEFLECTION DETERMINATION



COMPRESSION ELEMENTS STIFFENED ALONG BOTH EDGES (Large w/t Ratios)

CHART 3D

EFFECTIVE WIDTH FOR DEFLECTION DETERMINATION



COMPRESSION ELEMENTS STIFFENED ALONG BOTH EDGES (Small w/t Ratios)

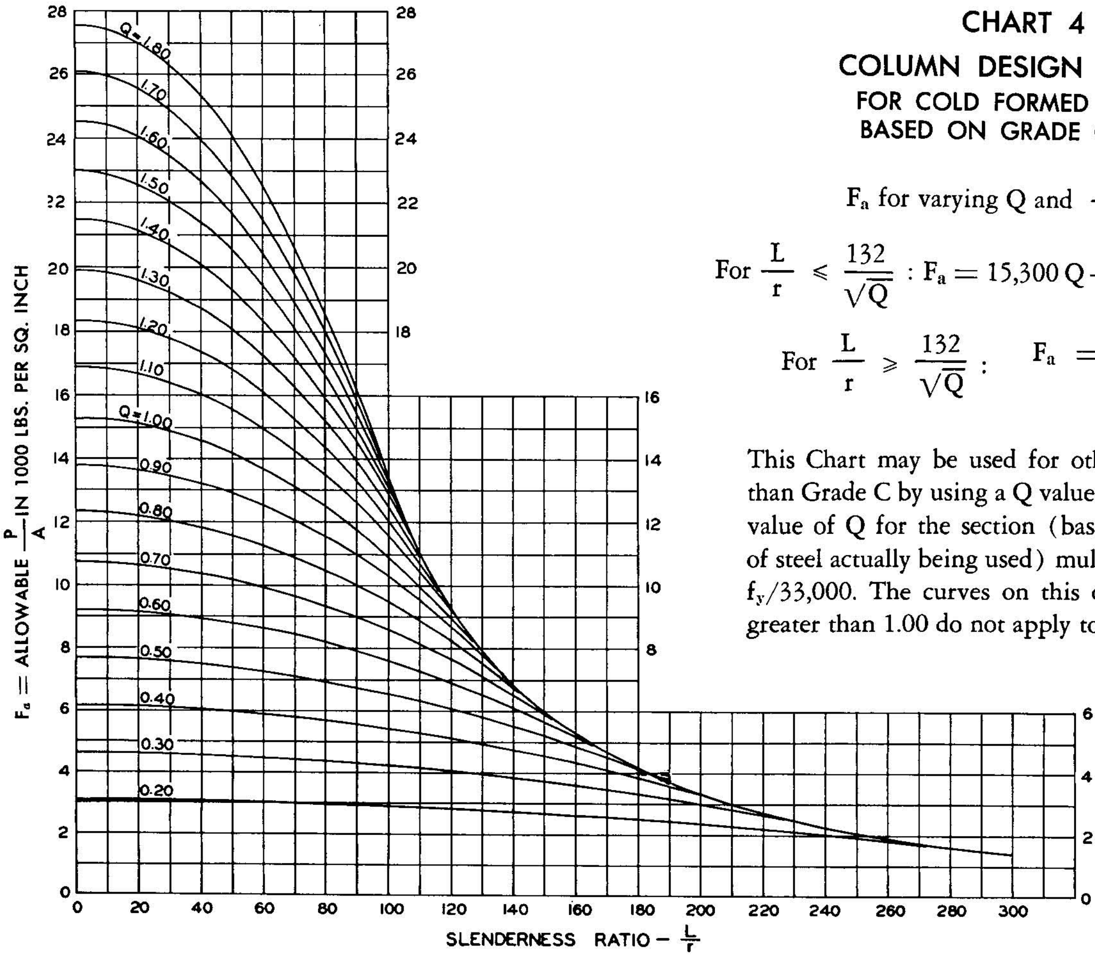


CHART 4

COLUMN DESIGN CURVES

FOR COLD FORMED SHAPES BASED ON GRADE C STEEL

F_a for varying Q and $\frac{L}{r}$ values

For $\frac{L}{r} \leqslant \frac{132}{\sqrt{Q}}$: $F_a = 15{,}300\,Q - 0.437\,Q^2 \left(\frac{L}{r}\right)^2$;

For $\frac{L}{r} \geqslant \frac{132}{\sqrt{Q}}$: $F_a = \dfrac{134{,}000{,}000}{\left(\frac{L}{r}\right)^2}$

This Chart may be used for other grades of steel than Grade C by using a Q value equal to the actual value of Q for the section (based upon the grade of steel actually being used) multiplied by the ratio $f_y/33{,}000$. The curves on this chart for Q values greater than 1.00 do not apply to Grade C Steel.

APPENDIX

The references listed below cover some of the research investigations that provide the background for the Design Specification.

(1) Stress Distribution in and Equivalent Width of Flanges of Wide, Thin-Wall Steel Beams, by George Winter, N.A.C.A. Technical Note No. 784, 1940.*

(2) The Durability of Lightweight Types of Steel Construction, American Iron and Steel Institute, 1942.

(3) Durability of Light Weight Steel Construction, by J. H. Cissel and W. E. Quinsey, University of Michigan Engineering Research Bulletin No. 30, 1942.

(4) Crushing Strength of Thin Steel Webs, by George Winter and R. H. J. Pian, Cornell University Engineering Experiment Station Bulletin No. 35, Part 1, 1946.

(5) Strength of Thin Steel Compression Flanges, by George Winter, Trans. ASCE, vol. 112, p. 527, 1947.**

(6) Discussion by George Winter of Strength of Beams as Determined by Lateral Buckling, by Karl de Vries, Trans. ASCE, vol. 112, p. 1272, 1947.

(7) Light Gage Steel Columns in Wall-Braced Panels, by G. G. Green, George Winter, and T. R. Cuykendall, Cornell University Engineering Experiment Station Bulletin No. 35, Part 2, 1947.

(8) Performance of Thin Steel Compression Flanges, by George Winter, International Association for Bridge & Structural Engineering, Third Congress, Liege, Preliminary Publication, p. 317, 1948.*

(9) Buckling of Trusses and Rigid Frames, by George Winter, P. T. Hsu, B. Koo, and M. H. Loh, Cornell University Engineering Experiment Station Bulletin No. 36, 1948.

(10) Discussion by George Winter of Stability of Thin-Walled Compression Members, by H. D. Wimer, Jr., Trans. ASCE, vol. 114, p. 553, 1949.

(11) Performance of Laterally Loaded Channel Beams, by George Winter, W. Lansing and R. B. McCalley, Jr., Research, Engineering, Structures Supplement (Colston Papers, vol. II), p. 49, London, 1949.*

(12) Performance of Compression Plates as Parts of Structural Members, by George Winter. (Same reference as 11, above, p. 179.*)

(13) Light-gage (Thin-walled) Steel Structures for Buildings in the U.S.A., by George Winter, International Association for Bridges & Structural Engineering, Fourth Congress, Preliminary Publication, p. 523, Cambridge — London, 1952.

(14) Supplement to (13) by George Winter, Same Congress, Final Publication, p. 261, 1953.

(15) Unsymmetrical Bending of Beams With and Without Lateral Bracing, by Lev Zetlin and George Winter, Proc. ASCE, vol. 81, Paper No. 774, 1955.

(16) Tests on Bolted Connections in Light Gage Steel, by George Winter, Proc. ASCE, vol. 82, Paper No. 920, 1956.

* Also available in Four Papers on the Performance of Thin Walled Steel Structures, Cornell University Engineering Experiment Station Reprint No. 33, 1950.

** Also available with supplements in Cornell University Engineering Experiment Station Reprint No. 32, 1947.

PUBLICATIONS AVAILABLE

Fire Protection Through Modern Building Codes

Building Code Modernization

Bulletin I	—	General Code Considerations
Bulletin II	—	Code Contents and Arrangement
Bulletin III	—	Building Classification and Fire Protection
Bulletin IV	—	Exit Requirements
Bulletin V	—	Steel Regulations
Bulletin VI	—	Steel-Framed Wall Assemblies
Bulletin VII	—	Steel Piles for Foundations of Buildings
Bulletin VIII	—	Ventilating and Heating Ducts
Bulletin IX	—	Modern Steel Constructions with Fire Resistive Ceilings
Bulletin X	—	Exterior Walls of Buildings — Fire Resistance Requirements
Bulletin XI	—	Non-Combustible Construction Versus Fire Hazards

Building Codes and Fire Safety

What Size Buildings

Building Code Regulations and Modern Building Construction

Cornell Bulletin No. 35/1 — Crushing Strength of Thin Steel Webs

Cornell Bulletin No. 35/2 — Light Gage Steel Columns in Wall-Braced Panels

Cornell Bulletin Reprint No. 32 — Strength of Thin Steel Compression Flanges

Light Gage Cold-Formed Steel Design Manual

The Durability of Lightweight Types of Steel Construction

Memo — Panel Construction for Exterior Walls

Memo — Automobile Parking Structures

Pipe in American Life

Radiant Panel Heating with Steel Pipe

Steel Pipe Snow Melting and Ice Removal Systems

Steel Electrical Raceways

Diagonal Tension in Reinforced Concrete Beams

Measurement of the Distribution of Tensile and Bond Stresses Along Reinforcing Bars

The Effect of Compressive Reinforcement on the Plastic Flow of Reinforced Concrete Beams

Spacing of Spliced Bars in Tension Pull-Out Specimens

The Effect of Sustained Overload on the Strength and Plastic Flow of Reinforced Concrete Beams