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CODE OF PRACTICE FOR USE OF COLD-FORMED LIGHT GAUGE STEEL STRUCTURAL MEMBERS IN GENERAL BUILDING CONSTRUCTION

(*First Revision*)

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Indian Standard

CODE OF PRACTICE FOR USE OF COLD-FORMED LIGHT GAUGE STEEL STRUCTURAL MEMBERS IN GENERAL BUILDING CONSTRUCTION

(First Revision)

0. FOREWORD

0.1 This Indian Standard (First Revision) was adopted by the Indian Standards Institution on 31 January 1975, after the draft finalized by the Structural Engineering Sectional Committee had been approved by the Structural and Metals Division Council and Civil Engineering Division Council.

0.2 Cold-formed steel structural members are cold-formed in rolls or press brakes from flat steel, generally not thicker than 12·5 mm. For repetitive mass production they are formed most economically by cold-rolling, while smaller quantities of special shapes are most economically produced on press brakes. The latter process, with its great versatility of shape variation, makes this type of construction as adaptable to special requirements as reinforced concrete is in its field of use. Members are connected by spot, fillet, plug or slot welds, by screw, bolts, cold rivets or any other special devices.

0.3 This type of construction is appropriate and economical under one or more of the following conditions:

a) Where moderate loads and spans make the thicker, hot-rolled shapes uneconomical, for example, joists, purlins, girts, roof trusses, complete framing for one- and two-storey residential, commercial and industrial structures;

b) Where it is desired that load-carrying members also provide useful surfaces, for example, floor panels and roof decks, mostly installed without any shoring and wall panels; and

c) Where sub-assemblies of such members can be prefabricated in the plant, reducing site erection to a minimum of simple operations, for example, sub-assembly of panel framing up to 3×4 metres and more for structures listed in (a), standardized package shed-type utility buildings, etc.

0.4 This standard was first published in 1958 and was mainly based on 1956 edition of 'Specification for the design of cold formed steel structural members' published by American Iron and Steel Institute, New York. While revising the Indian Standards, the Sectional Committee decided that it should be brought in line with the 1968 edition of the AISI publication, as this has been the accepted practice in this country and most suitable for this type of construction.

0.5 For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS : 2-1960*. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

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1. SCOPE

1.1 This code applies to the design of structural members cold-formed to shape from carbon or low-alloy, sheet or strip steels used for load carrying purposes in buildings. It may also be used for structures other than buildings provided appropriate allowances are made for dynamic effects.

2. MATERIAL

2.1 Structural steel sheet or strip steel shall conform to IS : 1079-1973†.

2.2 Steels other than the one covered in **2.1** may be used provided such steel conforms to the chemical and mechanical requirements of IS : 1079-1973† and its weldability is guaranteed.

3. DEFINITIONS

3.0 For the purpose of this code, the following definitions shall apply.

3.1 Stiffened Compression Elements — A flat compression element, for example, a plane compression flange of a flexural member or a plane web or flange of a compression member, of which both edges parallel to the direction of stress are stiffened by a web, flange stiffening lip, intermediate stiffener, or the like conforming to the requirements of **5.2.2**.

3.2 Unstiffened Compression Elements — A flat element which is stiffened at only one edge parallel to the direction of stress.

3.3 Multiple Stiffened Elements — An element that is stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners which are parallel to the direction of stress and which conform to the requirements of **5.2.2.2**. A sub-element is the portion between adjacent stiffeners or between web and intermediate stiffener or between edge and intermediate stiffener.

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*Rules for rounding off numerical values (*revised*).

†Specification for hot rolled carbon steel sheet and strip (*third revision*).

3.4 Flat-Width Ratio — The flat-width ratio, w/t, of a single flat element, is the ratio of the flat-width, w, exclusive of edge fillets, to the thickness t. 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 shape sections, the width w is the flat-width of flange between adjacent webs, exclusive of fillets.

3.5 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, and is determined in accordance with **5.2.1** and **5.2.5**.

3.6 Thickness — The thickness t of any element or section shall be the base steel thickness, exclusive of coatings.

3.7 Torsional Flexural Buckling — A mode of buckling in which compression members can bend and twist simultaneously.

3.8 Point Symmetric Section — A section symmetrical about a point (centroid) such as a *Z* section having equal flanges.

3.9 Yield Point, F_y — It shall mean yield point or yield strength.

3.10 Stress — Force per unit area; expressed in kilogram force per square centimetre, abbreviated throughout as kgf/cm².

4. LOADS

4.1 For general guidance as to the various loads to be taken into account in the design of structures, reference should be made to IS : 800-1962* and IS : 875-1964†.

5. DESIGN PROCEDURE

5.1 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.

5.2 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 applicable) except where the use of a reduced cross section, or effective design width, is required by the provisions of **5.2.1** and **5.2.5**.

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*Code of practice for use of structural steel in general building construction (*revised*).
†Code of practice for structural safety of buildings: Loading standards (*revised*).

5.2.1 *Properties of Stiffened Compression Elements* — In computing properties of sections of flexural members and in computing values of Q (*see* **6.6.1.1**) 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_e determined in accordance with the provisions of **5.2.1.1** or **5.2.1.2** whichever is applicable, and subject to the limitations of **5.2.5** 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 centre line of the element.

5.2.1.1 *Elements without intermediate stiffeners* — The effective design widths of compression elements which are not subject to the provisions of **5.2.1.2** shall be determined from the following formulae* :

For load determination:

Flanges are fully effective ($b = w$) up to $(w/t)_{lim} = 1\,435/\sqrt{f}$

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

$$\frac{b}{t} = \frac{2\,120}{\sqrt{f}}\left[1 - \frac{465}{(w/t)\sqrt{f}}\right]$$

Exception: Flanges of closed square and rectangular tubes are fully effective ($b = w$) up to $(w/t)_{lim} = \dfrac{1\,540}{\sqrt{f}}$ for flanges with w/t larger than $(w/t)_{lim}$

$$\frac{b}{t} = \frac{2\,120}{\sqrt{f}}\left[1 - \frac{420}{(w/t)\sqrt{f}}\right]$$

When members or assemblies are subject to stresses produced by wind and earthquake forces, the effective design width b shall be determined for 0·75 times the stress caused by wind or earthquake loads alone, or 0·75 times the stress caused by wind or earthquake plus gravity loads, when use is made of the increased allowable stress permitted in **6.1.2.1** or **6.1.2.2**.

For deflection determination:

Flanges are fully effective up to $(w/t)_{lim} = 1\,850/\sqrt{f}$

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

$$\frac{b}{t} = \frac{2\,710}{\sqrt{f}}\left[1 - \frac{600}{(w/t)\sqrt{f}}\right]$$

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*It is to be noted that where the flat-width exceeds $(w/t)_{lim}$ the properties of the section shall frequently be determined by successive approximations or other appropriate methods, since the stress and the effective design width are interdependent.

Exception: Flanges of closed square and rectangular tubes are fully effective up to $(w/t)_{lim} = 1\,990/\sqrt{f}$ for flanges with w/t larger than $(w/t)_{lim}$

$$\frac{b}{t} = \frac{2\,710}{\sqrt{f}}\left[1 - \frac{545}{(w/t)\sqrt{f}}\right]$$

where

w/t = flat-width ratio,

b = effective design width in cm, and

f = actual stress in the compression element computed on the basis of the effective design width in kgf/cm².

5.2.1.2 *Multiple stiffened elements and wide stiffened elements with edge stiffeners* — Where the flat-width ratio of a sub-element of a multiple stiffened compression element or of a stiffened compression element which does not have intermediate stiffeners and which has only one longitudinal edge connected to a web does not exceed 60, the effective design width, b, of such sub-element or element shall be determined in accordance with the provisions of **5.2.1.1**. Where such flat-width ratio exceeds 60, the effective design width, b_e, of the sub-element or element shall be determined from the following formula*:

$$\frac{b_e}{t} = \frac{b}{t} - 0{\cdot}10\left(\frac{w}{t} - 60\right)$$

where

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

b = effective design width determined in accordance with the provisions of **5.2.1.1** in cm, and

b_e = effective design width of sub-element or element to be used in design computations in cm.

For computing the effective structural properties of a member having compression sub-elements or element subject to the above reduction in effective width, the area of stiffeners (edge stiffener or intermediate stiffeners†) shall be considered reduced to an effective area as follows:

For w/t between 60 and 90:

$$A_{ef} = \alpha A_{st}$$

where

$$\alpha = (3 - 2\,b_e/w) - \frac{1}{30}\left[1 - \frac{b_e}{w}\right]\frac{w}{t}$$

For w/t greater than 90:

$$A_{ef} = (b_e/w)\,A_{st}$$

*See **5.2.3**(a) for limitations on the allowable flat-width ratio of a compression element stiffened at one edge by other than a simple lip.

†See **5.2.2.2** for limitations on number of intermediate stiffeners which may be considered effective and their minimum moment of inertia.

In the above expressions, A_{ef} and A_{st} 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.

5.2.2 *Stiffeners for Compression Elements*

5.2.2.1 *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:

$$I_{Min} = 1{\cdot}83\ t^4 \sqrt{(w/t)^2 - 281\ 200/F_y} \text{ but not less than } 9{\cdot}2\ t^4$$

where

I_{Min} = minimum allowable moment of inertia of stiffener (of any shape) about its own centroidal axis parallel to the stiffened element in cm^4, and

w/t = flat-width ratio of stiffened element.

Where the stiffener consists of a simple lip bent at right angles to the stiffened element, the required overall depth d_{Min} of such lip may be determined as follows:

$$d_{Min} = 2{\cdot}8\ t \sqrt[6]{(w/t)^2 - 281\ 200/F_y} \text{ but not less than } 4{\cdot}8\ t$$

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

5.2.2.2 *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 a stiffened 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 **5.2.2.1** where w is the width of the sub-element. The following limitations shall also 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 $(w/t)_{lim}$ in **5.2.1**, 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 $(w/t)_{lim}$ in **5.2.1**, 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 $(w/t)_{lim}$ in **5.2.3**, all the stiffeners may be considered effective. Only for the purposes of 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.

5.2.3 *Maximum Allowable Overall Flat-Width Ratios* — Maximum allowable overall flat-width ratios w/t disregarding intermediate stiffeners and taking t as 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	60
Any other kind of stiffener	90

b) Stiffened compression element with both longitudinal edges connected to other stiffened elements — 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 affecting 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 formulae given in this code.

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 towards the neutral axis, the following formula applies to compression and tension flanges, either stiffened or unstiffened:

$$w_f = \sqrt{\frac{126\,500\ td}{f_{av}}} \times \sqrt[4]{\frac{100\ c_f}{d}}$$

where

w_f = the width of flange projecting beyond the web, or half of the distance between webs for box- or U-type beams;

t = flange thickness;

d = depth of beam;

c_f = the amount of curling*; and

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

f_{av} = the average stress in the full, unreduced flange-width in kgf/cm^2 (where members are designed by the effective design width procedure, the average stress equals the maximum stress multiplied by the ratio of the effective design width to the actual width).

5.2.4 *Maximum Allowable Web Depth* — The ratio h/t of the webs of flexural members shall not exceed the following limitations:

a) For members with unstiffened webs:

$$(h/t)_{Max} = 150$$

b) For members which are provided with adequate means of transmitting concentrated loads or reactions or both into the web:

$$(h/t)_{Max} = 200$$

where

h = clear distance between flanges measured along the plane of web, and

t = web thickness.

Where a web consists of two or more sheets, the h/t ratio shall be computed for individual sheets.

5.2.5 *Unusually Short Spans Supporting Concentrated Loads* — Where the span of the beam is less than 30 w_f (w_f as defined below) and it carries one concentrated load or several loads spaced farther apart than 2 w_f, the effective design width of any flange, whether in tension or compression, shall be limited to as given in Table 1.

TABLE 1 MAXIMUM ALLOWABLE RATIO OF EFFECTIVE DESIGN WIDTH TO ACTUAL WIDTH

L/w_f	Ratio	L/w_f	Ratio
(1)	(2)	(1)	(2)
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

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

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

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

6. ALLOWABLE DESIGN STRESS

6.0 General — The maximum allowable stresses to be used in design shall be as given in **6.1** to **6.8**.

6.1 Basic Design Stress — Stress on the net section of tension members, and tension and compression on the extreme fibres of flexural members shall not exceed the value F specified below, except as otherwise specifically provided herein:

$$F = 0{\cdot}60\ F_y$$

where F_y is the specified minimum yield point.

When the increase in steel strength resulting from cold work of forming is utilized in accordance with **6.1.1**, the basic design stress shall be determined as follows:

$$F = 0{\cdot}60\ F_{ya}$$

where F_{ya} is the average yield point of the full section.

Values of the basic allowable design stress F as defined above for some of the grades covered in IS : 1079-1973* are given in Table 2.

TABLE 2 BASIC ALLOWABLE DESIGN STRESS F

MINIMUM YIELD STRENGTH kgf/mm²	F kgf/cm²
21	1 250
24	1 450
30	1 800
36	2 160

6.1.1 *Utilization of Cold Work of Forming* — Allowable stresses shall be based upon the specified minimum properties of the unformed steel. Utilization, for design purposes, of any increase in steel strength that results from a cold-forming operation is permissible provided that the methods and limitations prescribed in **6.1.1.1** are observed and satisfied.

6.1.1.1 *Methods and limitations* — Utilization of cold work of forming shall be on the following basis:

a) The yield point of axially loaded compression members whe $Q=1$, and the flanges of flexural members whose proportions ar such that when treated as compression members the quantity ((*see* **6.6.1.1**) is unity, shall be determined on the basis of eith (1) full section tensile tests [*see* **9.3.1(a)**], or (2) stub column tes [*see* **9.3.1**(b)], or (3) computed as follows:

$$F_{ya} = C\ F_{yc} + (1 - C)F_{yf}$$

*Specification for hot rolled carbon steel sheet and strip (*third revision*).

where

F_{ya} = average tensile yield point of the full section of compression members, or full flange sections of flexural members;

C = ratio of the total corner area to the total cross-sectional area of the full section of compression members, or full flange sections of flexural members;

F_{yc} = tensile yield point of corners, $B_c\ F_y/(R/t)^m$. The formula does not apply where F_u/F_y is less than 1·2, R/t exceeds 7, and/or maximum included angle exceeds 120°;

F_{yf} = weighted average tensile yield point of the flat portions established in accordance with **9.3.2** or virgin yield point if tests are not made;

B_c = $3{\cdot}69\ (F_u/F_y) - 0{\cdot}819\ (F_u/F_y)^2 - 1{\cdot}79$;

m = $0{\cdot}192\ (F_u/F_y) - 0{\cdot}068$;

R = inside bend radius;

F_y = tensile yield point of virgin steel* specified in **2.1** or established in accordance with **9.3.3**; and

F_u = ultimate tensile strength of virgin steel specified in **2.1** or established in accordance with **9.3.3**.

b) The yield point of axially loaded compression members with Q less than unity, and the flanges of flexural members whose proportions are such that when treated as compression members the quantity Q (*see* **6.6.1.1**) is less than unity, may be taken as (1) the tensile yield point of the virgin steel* specified in IS : 1079-1973†, or (2) the tensile yield point of the virgin steel established in accordance with **9.3.3**, or (3) the weighted average tensile yield point of flats established in accordance with **9.3.2**.

c) The yield point of axially loaded tension members shall be determined by either method (1) or method (3) prescribed in (a) above.

d) Application of the provisions of **6.1.1.1**(a) shall be confined to the following:

1) Basic Design Stress (**6.1**),
2) Compression on Unstiffened Elements (**6.2**),
3) Laterally Unbraced Beams (**6.3**),
4) Axially Loaded Compression Members (**6.6**),
5) Combined Axial and Bending Stresses (**6.7**),
6) Cylindrical Tubular Members in Compression or Bending (**6.8**), and
7) Wall Studs (**8.1**).

*Virgin steel refers to the condition (that is coiled or straight) of the steel prior to the cold-forming operation.

†Specification for hot rolled carbon steel sheet and strip (*third revision*).

Application of all provisions of the code may be based upon the properties of the flat steel before forming or on **6.1.1.1**(b) or (c) as applicable.

e) The effect on mechanical properties of any welding that is to be applied to the member shall be determined on the basis of tests of full section specimens containing within the gauge length such welding as the manufacturer intends to use. Any necessary allowance for such effect shall be made in the structural use of the member.

6.1.2 *Wind, Earthquake, and Combined Forces*

6.1.2.1 *Wind or earthquake only* — Members and assemblies subject only to stresses produced by wind or earthquake forces may be proportioned for stresses $33\frac{1}{3}$ percent greater than those specified for dead and live load stresses. A corresponding increase may be applied to the allowable stresses in connections and details.

6.1.2.2 *Combined forces* — Members and assemblies subject to stress produced by a combination of wind or earthquake and other loads may be proportioned for unit stress $33\frac{1}{3}$ 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.

For primary and secondary members of roof assemblies and roof deck, the allowable stresses may be increased by $33\frac{1}{3}$ percent for combined stresses due to dead load, gravity live load (if any) and ponding, provided the section thus required is not less than that required for the combination of dead load and live load.

Corresponding increases may be applied to the allowable unit stresses in connections and details.

6.2 Compression on Unstiffened Elements — Compression F_c in kgf/ cm² on flat unstiffened elements:

a) For w/t not greater than $530/\sqrt{F_y}$:

$$F_c = 0{\cdot}60\ F_y$$

b) For w/t ratio greater than $530/\sqrt{F_y}$ but not greater than $1\,210/\sqrt{F_y}$* :

$$F_c = F_y\ [0{\cdot}767 - (3{\cdot}15/10^4)\ (w/t)\sqrt{F_y}]$$

c) For w/t ratio greater than $1\,210/\sqrt{F_y}$ but not greater than 25*:

$$F_c = 562\ 000/(w/t)^2$$

*When the yield point of steel is less than 2 320 kgf/cm² then for w/t ratios between $530/\sqrt{F_y}$ and 25:

$$F_c = 0{\cdot}6\ F_y - \frac{\left[\frac{w}{t} - \frac{530}{\sqrt{F_y}}\right](0{\cdot}6\,F_y - 900)}{25\,[1 - 21{\cdot}2/\sqrt{F_y}]}$$

d) For w/t ratio from 25 to 60*:

For angle struts: $F_c = 562\ 000/(w/t)^2$

For all other sections: $F_c = 1\ 390 - 20\ w/t$

In the above formulae, w/t is the flat-width ratio as defined in **3**.

6.3 Laterally Unbraced Beams — To prevent lateral buckling, the maximum compression stress F_b on extreme fibres of laterally unsupported straight flexural members† shall not exceed the allowable stress as specified in **6.1** or **6.2** nor the following maximum stresses:

a) When bending is about the centroidal axis perpendicular to the web for either I-shaped sections symmetrical about an axis in the plane of the web or symmetrical channel-shaped sections:

when $\dfrac{L^2 S_{xc}}{d\ I_{yc}}$ is greater than $\dfrac{0{\cdot}36\ \pi^2 E\ C_b}{F_y}$ but less than $\dfrac{1{\cdot}8\ \pi^2 E\ C_b}{F_y}$

$$F_b = \frac{2}{3} F_y - \frac{F_y^2}{5{\cdot}4\ \pi^2 E\ C_b} \left(\frac{L^2 S_{xc}}{d\ I_{yc}}\right)$$

when

$\dfrac{L^2 S_{xc}}{d\ I_{yc}}$ is equal to or greater than $\dfrac{1{\cdot}8\ \pi^2 E\ C_b}{F_y}$

$$F_b = 0{\cdot}6\ \pi^2 E\ C_b \frac{d\ I_{yc}}{L^2 S_{xc}}$$

b) For point-symmetrical Z-shaped sections bent about the centroidal axis perpendicular to the web:

when $\dfrac{L^2 S_{xc}}{d\ I_{yc}}$ is greater than $\dfrac{0{\cdot}18\ \pi^2 E\ C_b}{F_y}$ but less than $\dfrac{0{\cdot}9\ \pi^2 E\ C_b}{F_y}$

$$F_b = \frac{2}{3} F_y - \frac{F_y^2}{2{\cdot}7\ \pi^2 E\ C_b} \left(\frac{L^2 S_{xc}}{d\ I_{yc}}\right)$$

when $\dfrac{L^2 S_{xc}}{d\ I_{yc}}$ is equal to or greater than $\dfrac{0{\cdot}9\ \pi^2 E\ C_b}{F_y}$

$$F_b = 0{\cdot}3\ \pi^2 E\ C_b \frac{d\ I_{yc}}{L^2 S_{xc}}$$

*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.

†The provisions of this Section apply to I-, Z-, or channel-shaped flexural members (not including multiple-web deck, U- and closed-box type members and curved or arch members). The provisions of this Section do not apply to laterally unbraced compression flanges of otherwise laterally stable sections.

where

L = the unbraced length of the member;

I_{yc} = the moment of inertia of the compression portion of a section about the gravity axis of the entire section parallel to the web;

S_{xc} = Compression section modulus of entire section about major axis, I_x divided by distance to extreme compression fibre;

C_b = bending coefficient which can conservatively be taken as unity, or calculated from:

$$C_b = 1{\cdot}75 + 1{\cdot}05\left(\frac{M_1}{M_2}\right) + 0{\cdot}3\left(\frac{M_1}{M_2}\right)^2, \text{ but not more than } 2{\cdot}3.$$

Where M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length, taken about the strong axis of the members, and where M_1/M_2, the ratio of end moments is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, the ratio M_1/M_2 shall be taken as unity.

For members subject to combined axial and bending stress (*see* **6.7**), C_b, shall be 1.

E = modulus of elasticity = 2 074 000 kgf/cm²; and

d = depth of section.

6.4 Allowable Stresses in Web of Beams

6.4.1 *Shear Stresses in Webs* — The maximum average shear stress F_v, in kgf/cm², on the gross area of a flat web shall not exceed:

a) For h/t not greater than $4\,590/\sqrt{F_y}$:

$$F_v = \frac{1275\sqrt{F_y}}{h/t} \text{ with a maximum of } 0{\cdot}40\,F_y$$

b) For h/t greater than $4\,590/\sqrt{F_y}$:

$$F_v = \frac{5\,850\,000}{(h/t)^2}$$

where

t = web thickness,

h = clear distance between flanges measured along the plane of web, and

F_y = yield point in kgf/cm².

Where the web consists of two or more sheets these shall be considered as separate members carrying their share of the shear.

6.4.2 *Bending Stress in Webs* — The compressive stress F_{bw}, in kgf/cm², in the flat web of a beam due to bending in its plane, shall not exceed F nor shall it exceed:

$$F_{bw} = \frac{36\,560\,000}{(h/t)^2} \text{ kgf/cm}^2$$

6.4.3 *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 **6.4.1** and **6.4.2** and that the quantity $\sqrt{(f_{bw}/F_{bw})^2 + (f_v/F_v)^2}$ does not exceed unity:

where

f_{bw} = actual compression stress at junction of flange and web;

$F_{bw} = \dfrac{36\,560\,000}{(h/t)^2}$ kgf/cm²;

f_v = actual average shear stress, that is, shear force per web divided by webs area; and

F_v = allowable shear stress as specified in **6.4.1** except that the limit of $0{\cdot}4\ F_y$ shall not apply.

6.5 Web Crippling of Beams — To avoid crippling of unreinforced beam webs having a flat-width ratio h/t equal to or less than 150, concentrated loads and reactions shall not exceed the values of P_{Max} given below. Webs of beams for which the ratio h/t is greater than 150 shall be provided with adequate means of transmitting concentrated loads and reactions directly into the web.

a) *Beams having single unreinforced webs*:

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

For inside corner radius equal to or less than the thickness of sheet:

$$P_{Max} = 70\,t^2\,[98 + 4{\cdot}20(N/t) - 0{\cdot}022\,(N/t)\,(h/t) - 0{\cdot}011\,(h/t)] \times [1{\cdot}33 - 0{\cdot}33\,(F_y/2\,320)]\,(F_y/2\,320)$$

For other corner radii up to $4\,t$, the value P_{Max} given by the above formula shall be multiplied by $(1{\cdot}15 - 0{\cdot}15\,R/t)$.

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

For inside corner radius equal to or less than the thickness of sheet:

$$P_{Max} = 70\,t^2\,[305 + 2{\cdot}30\,(N/t) - 0{\cdot}009\,(N/t)\,(h/t) - 0{\cdot}5\,(h/t)] \times [1{\cdot}22 - 0{\cdot}22\,(F_y/2\,320)]\,(F_y/2\,320)$$

For other corner radii up to $4t$, the value P_{Max} given by the above formula is to be multiplied by $(1 \cdot 06 - 0 \cdot 06\ R/t)$.

(3) For corner radii larger than $4t$, tests shall be made in accordance with **9**.

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:

$$P_{Max} = t^2 F_y (4 \cdot 44 + 0 \cdot 558\sqrt{N/t})$$

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

$$P_{Max} = t^2 F_y (6 \cdot 66 + 1 \cdot 146\sqrt{N/t})$$

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, provisions of **6.5**(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 $1 \cdot 5h$. Otherwise provisions of **6.5**(a)(1) and (b)(2) apply.

In the above formulae,

P_{Max} = allowable concentrated load or reactions;

t = web thickness;

N = actual length of bearing, except that in the above formulae the value of N shall not be taken greater than h;

h = clear distance between flanges measured along the plane of web;

F_y = yield point; and

R = inside bend radius.

6.6 Axially Loaded Compression Members

6.6.1 *Stress*

6.6.1.1 *Shapes not subject to torsional-flexural buckling*

(a) For doubly-symmetric shapes, closed cross-sectional shapes or cylindrical sections, and any other shapes which can be shown not to be subject to torsional-flexural buckling, and for members braced against twisting, the average axial stress P/A, in compression members shall not exceed the following values of

F_{a1} except as otherwise permitted by **6.6.1.1**(b):

$K\,L/r$ less than $\dfrac{C_c}{\sqrt{Q}}$

$$F_{a1} = \frac{12}{23}\,Q\,F_y - \frac{3\,(Q\,F_y)^2}{23\,\pi^2\,E}\left(\frac{K\,L}{r}\right)^2$$

$$= 0{\cdot}522\ Q\,F_y - \left(\frac{Q\,F_y\,K\,L/r}{12\,500}\right)^2$$

$K\,L/r$ equal to or greater than $\dfrac{C_c}{\sqrt{Q}}$

$$F_{a1} = \frac{12\,\pi^2\,E}{23\,(K\,L/r)^2}$$

$$= \frac{10\,680\,000}{(K\,L/r)^2}$$

where

$C_c = \sqrt{2\,\pi^2\,E/F_y}$;
P = total load;
A = full unreduced cross-sectional area of the member;
F_{a1} = allowable average compression stress under concentric loading;
E = modulus of elasticity = 2 074 000 kgf/cm²;
K = effective length factor*;
L = unbraced length of member;
r = radius of gyration of full, unreduced cross section;
F_y = yield point of steel; and
Q = a factor determined as follows:

(1) 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 as defined in **6.1**.

(2) 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

*In frames where lateral stability is provided by diagonal bracing, shear walls, attachment to an adjacent structure having adequate lateral stability, or by floor slabs or roof decks secured horizontally by walls or bracing systems parallel to the plane of the frame, and in trusses the effective length factor K for the compression members shall be taken as unity, unless analysis shows that a smaller value may be used. The effective length KL of compression members in a frame which depends upon its own bending stiffness for lateral stability, shall be determined by a rational method and shall not be less than the actual unbraced length.

the basic design stress, F, where F_c is defined in **6.2** and F is as defined in **6.1**.

(3) 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 (2) above and an area factor Q_a computed as outlined in (1) above, except that the stress upon which Q_a is to be based shall be that value of the stress F_a 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.

b) When the factor Q is equal to unity, the steel is 2·29 mm or more in thickness and $K\,L/r$ is less than C_c:

$$F_{a1} = \frac{\left[1 - \dfrac{(K\,L/r)^2}{2\,(C_c)^2}\right] F_y}{\dfrac{5}{3} + \dfrac{3}{8}\dfrac{(K\,L/r)}{(C_c)} - \dfrac{(K\,L/r)^3}{8\,(C_c)^3}}$$

6.6.1.2 *Singly-symmetric and nonsymmetric shapes of open cross section or intermittently fastened singly-symmetrical components of built-up shapes having $Q=1{\cdot}0$ which may be subject to torsional flexural buckling* — For singly-symmetric or nonsymmetric shapes of open cross section or intermittently fastened singly-symmetrical components of built-up shapes having $Q=1{\cdot}0$ which may be subject to torsional-flexural buckling and which are not braced against twisting, the average axial stress P/A shall not exceed F_{a1} specified in **6.6.1.1** or F_{a2} given below:

For $\sigma_{TFO} > 0{\cdot}5\,F_y$:

$$F_{a2} = 0{\cdot}522\,F_y - \frac{F_y^2}{7{\cdot}67\,\sigma_{TFO}}$$

For $\sigma_{TFO} \leqslant 0{\cdot}5\,F_y$:

$$F_{a2} = 0{\cdot}522\,\sigma_{TFO}$$

where

F_{a2} = allowable average compression stress under concentric loading, and

σ_{TFO} = elastic torsional-flexural buckling stress under concentric loading which shall be determined as follows:

a) *Singly-symmetric shapes* — For members whose cross sections have one axis of symmetry (x-axis), σ_{TFO} is less than both σ_{ex} and σ_t and is equal to:

$$\sigma_{TFO} = \frac{1}{2\,\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\,\sigma_{ex}\,\sigma_t}\right]$$

where

$$\sigma_{ex} = \frac{\pi^2 E}{(K\, L/r_x)^2}$$

$$\sigma_t = \frac{1}{A r_0^2}\left[G\, J + \frac{\pi^2 E\, C_w}{(K\, L)^2}\right]$$

$\beta = 1 - (x_0/r_0)^2$,

A = cross-sectional area,

$r_0 = \sqrt{r_x^2 + r_y^2 + x_0^2}$ = polar radius of gyration of cross section about the shear centre,

r_x, r_y = radii of gyration of cross section about centroidal principal axes,

E = modulus of elasticity = 2 074 000 kgf/cm^2,

G = shear modulus = 795 000 kgf/cm^2,

K = effective length factor,

L = unbraced length of compression member,

x_0 = distance from shear centre to centroid along the principal x-axis,

J = St Venant torsion constant of the cross section, cm^4. For thin walled sections composed of n segments of uniform thickness,

$$J = (1/3)\,(l_1 t_1^3 + l_2 t_2^3 + \ldots\ldots + l_i t_i^3 \ldots + l_n t_n^3),$$

t_i = steel thickness of the member for segment i,

l_i = length of middle line of segment i, and

C_w = warping constant of torsion of the cross section.

b) *Nonsymmetric shapes* — Shapes whose cross sections do not have any symmetry, either about an axis or about a point, σ_{TFO} shall be determined by rational analysis.

Alternatively, compression members composed of such shapes may be tested in accordance with **9**.

6.6.1.3 *Singly-symmetric or nonsymmetric shapes or intermittently fastened singly-symmetrical components of built-up shapes having $Q < 1{\cdot}0$ which are subject to torsional-flexural buckling* — Compression members composed of singly-symmetric, or nonsymmetric shapes or intermittently fastened singly-symmetrical components of built-up shapes having $Q < 1{\cdot}0$ which are subject to torsional-flexural buckling and which are not braced against twisting can be conservatively proportioned by replacing F_y by QF_y in **6.6.1.2** or their strength may be determined by tests in accordance with **9**. Q is defined in **6.6.1.1**.

6.6.2 *Bracing and Secondary Members* — On the cross section of axially loaded bracing and secondary members*, when L/r ratio exceeds 120, the

*A secondary member is one upon which the integrity of the structure as a whole does not depend. For this case, K is taken as unity.

allowable compression stress under concentric loading F_{as} shall be determined as follows:

$$F_{as} = \frac{F_a}{1.3 - \dfrac{L}{400\,r}}$$

In the above formula, the maximum stress F_a shall be determined by **6.6.1.1** or **6.6.1.2** whichever is applicable.

6.6.3 *Maximum Slenderness Ratio* — The slenderness ratio $K\,L/r$ of compression members shall not exceed 200, except that during construction only, $K\,L/r$ shall not exceed 300.

6.7 Combined Axial and Bending Stresses

6.7.1 *Doubly-Symmetric Shapes or Shapes Not Subject to Torsional or Torsional-Flexural Buckling* — When subject to both axial compression and bending, doubly-symmetric shapes or shapes which are not subject to torsional or torsional-flexural buckling shall be proportioned to meet the following requirements:

$$\frac{f_a}{F_{a1}} + \frac{C_{mx}\,f_{bx}}{\left(1 - \dfrac{f_a}{F'_{ex}}\right)F_{bx}} + \frac{C_{my}\,f_{by}}{\left(1 - \dfrac{f_a}{F'_{ey}}\right)F_{by}} \leqslant 1.0$$

$$\frac{f_a}{F_{ao}} + \frac{f_{bx}}{F_{b1x}} + \frac{f_{bx}}{F_{b1y}} \leqslant 1.0$$

when

$\dfrac{f_a}{F_{a1}} \leqslant 0.15$, the following formula may be used in lieu of the above two formulae:

$$\frac{f_a}{F_{a1}} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leqslant 1.0$$

The subscripts 'x' and 'y' in the above formulae indicate the axis of bending about which a particular stress or design property applies.

6.7.2 *Singly-Symmetric Shapes or Intermittently Fastened Singly-Symmetric Components of Built-Up Shapes Having $Q = 1.0$ Which May Be Subject to Torsional-Flexural Buckling* — Singly-symmetric shapes subject to both axial compression and bending applied in the plane of symmetry shall be proportioned to meet the following four requirements as applicable:

a) $$\frac{f_a}{F_{a1}} + \frac{f_{b1}\,C_m}{F_{b1}\left[1 - \dfrac{f_a}{F'_e}\right]} \leqslant 1$$

$$\frac{f_a}{F_{ao}} + \frac{f_{b1}}{F_{b1}} \leqslant 1$$

when

$\frac{f_a}{F_{a1}} \leqslant 0.15$, the following formula may be used in lieu of the above two formulae:

$$\frac{f_a}{F_{a1}} + \frac{f_{b1}}{F_{b1}} \leqslant 1.0$$

b) If the point of application of the eccentric load is located on the side of the centroid opposite from that of the shear centre, that is, if e is positive, then the average compression stress f_a shall also not exceed F_a given below:

For $\sigma_{TF} > 0.5\ F_y$:

$$F_a = 0.522\ F_y - \frac{F_y^2}{7.67\ \sigma_{TF}}$$

For $\sigma_{TF} \leqslant 0.5\ F_y$:

$$F_a = 0.522\ \sigma_{TF}$$

where

σ_{TF} shall be determined according to the formula:

$$\frac{\sigma_{TF}}{\sigma_{TFO}} + \frac{C_{TF}\ \sigma_{b1}}{\sigma_{bT}\left(1 - \frac{\sigma_{TF}}{\sigma_e}\right)} = 1.0$$

c) Except for T- or unsymmetric I-sections, if the point of application of the eccentric load is between the shear centre and the centroid, that is, if e is negative, and if F_{a_1} is larger than F_{a_2}, then the average compression stress f_a shall also not exceed F_a given below:

$$F_a = F_{a_2} + \frac{e}{x_0}(F_{aE} - F_{a_2})$$

d) For T- and unsymmetric I-sections with negative eccentricities:

1) If the point of application of the eccentric load is between the shear centre and the centroid, and if F_{a_1} is larger than F_{a_2}, then the average compression stress f_a shall also not exceed F_a given below:

$$F_a = F_{a_2} + \frac{e}{x_0}(F_{ac} - F_{a_2})$$

2) If the point of application of the eccentric load is located on the side of the shear centre opposite from that of the centroid, then the average compression stress f_a shall also not exceed F_a given below:

$$\sigma_{TF} > 0.5\ F_y,\ F_a = 0.522\ F_y - \frac{F_y^2}{7.67\ \sigma_{TF}}$$

$$\sigma_{TF} \leqslant 0.5\ F_y,\ F_a = 0.522\ \sigma_{TF}$$

where

σ_{TF} shall be determined according to the formula:

$$\frac{\sigma_{TF}}{\sigma_{ex}} + \frac{C_{TF}}{\sigma_{bc}}\left[\frac{\sigma_{b1}}{1 - \frac{\sigma_{TF}}{\sigma_e}} - \sigma_{b2}\right] = 1{\cdot}0$$

In **6.7.2**, x and y are centroidal axes and the x-axis is the axis of symmetry whose positive direction is pointed away from the shear centre.

In **6.7**:

C_m = a coefficient whose value shall be taken as follows:

a) For compression members in frames subject to joint translation (sideway) $C_m = 0{\cdot}85$.

b) For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending

$$C_m = 0{\cdot}6 - 0{\cdot}4\frac{M_1}{M_2}, \text{ but not less than } 0{\cdot}4$$

where $\frac{M_1}{M_2}$ is the ratio of the smaller to larger moments at the ends of that portion of the memb..., unbraced in the plane of bending under consideration. $\frac{M_1}{M_2}$ is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.

c) For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports, the value of C_m may be determined by rational analysis. However, in lieu of such analysis, the following values may be used: (1) for members whose ends are restrained, $C_m = 0{\cdot}85$, and (2) for members whose ends are unrestrained, $C_m = 1{\cdot}0$.

C_{TF} = a coefficient whose value shall be taken as follows:

a) For compression members in frames subject to joint translation (sideway) $C_{TF} = 0{\cdot}85$.

b) For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending

$$C_{TF} = 0{\cdot}6 - 0{\cdot}4\frac{M_1}{M_2}$$

where $\frac{M_1}{M_2}$ is the ratio of the smaller to larger moments at the end of that portion of the members, unbraced in the plane of bending under consideration.

$\frac{M_1}{M_2}$ is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.

c = distance from the centroidal axis to the fibre with maximum compression stress, negative when the fibre is on the shear centre side of the centroid

d = depth of section

e = eccentricity of axial load with respect to the centroidal axis, negative when on the shear centre side of the centroid

F_a = maximum average compression stress

F_{aC} = average allowable compression stress determined by both requirements **6.7.2**(a) and **6.7.2**(d)(2) if the point of application of the eccentric load is at the shear centre, that is, the calculated values of f_a and F_a for $e = x_0$

F_{aE} = average allowable compression stress determined by requirements **6.7.2**(a) if the point of application of the eccentric load is at the shear centre, that is, the calculated value of f_a for $e = x_0$

F_{aO} = allowable compression stress under concentric loading determined by **6.6.1.1** for $L = 0$

F_{a1} = allowable compression stress under concentric loading according to **6.6.1.1** for buckling in the plane of symmetry

F_{a2} = allowable compression stress under concentric loading from **6.6.1.2**

F_b = maximum bending stress in compression that is permitted by this code where bending stress only exists (*see* **6.1**, **6.2** and **6.3**)

F_{b1} = maximum bending stress in compression permitted by this code where bending stress only exists and the possibility of lateral buckling is excluded (*see* **6.1** and **6.2**)

$F'_e = \frac{12\,\pi^2 E}{23\,(K L_b/r_b)^2}$ (may be increased one-third in accordance with **6.1.2**)

f_a = axial stress = axial load divided by full cross-sectional area of member P/A

f_b = maximum bending stress = bending moment divided by appropriate section modulus of member M/S, noting that for members having stiffened compression elements the section modulus shall be based upon the effective design widths of such elements

I_{xc} = moment of inertia of the compression portion of a section about its axis of symmetry

I_y = moment of inertia of the section about the y-axis

$$j = \frac{1}{2\, I_y}\left[\int_A x^3\, dA + \int_A xy^2\, dA\right] - x_0$$

where x is the axis of symmetry and y is orthogonal to x

K = effective length factor in the plane of bending

L_b = actual unbraced length in the plane of bending

$M_C = -A\sigma_{ex}\left[j + \sqrt{j^2 + r_0^2\,(\sigma_t/\sigma_{ex})}\right]$ = elastic critical moment causing compression on the shear centre side of the centroid

$M_T = -A\sigma_{ex}\left[j - \sqrt{j^2 + r_0^2\,(\sigma_t/\sigma_{ex})}\right]$ = elastic critical moment causing tension on the shear centre side of the centroid

r_b = radius of gyration about axis of bending

r_{xc} = radius of gyration about the centroidal axis parallel to the web of that portion of the I-section which is in compression when there is no axial load

S_{yc} = compression section modulus of entire section about axis normal to axis of symmetry, I_y/distance to extreme compression fibre

x_0 = x coordinate of the shear centre, negative

$\sigma_{bC} = \dfrac{M_C\, c}{I_y}$ = maximum compression bending stress caused by M_C

For I-sections with unequal flanges σ_{bc} may be approximated by

$$\frac{\pi^2 . Ed\, I_{xc}}{L^2\, S_{yc}}$$

$\sigma_{bT} = \dfrac{M_T\, c}{I_y}$ = maximum compression bending stress caused by M_T

For I-sections with unequal flanges σ_{bT} may be approximated by

$$\frac{\pi^2\, Ed\, I_{xc}}{L^2\, S_{yc}}$$

$\sigma_{b1} = \sigma_{TF}\, \dfrac{e - c}{r_y^2}$ = maximum compression bending stress in the section caused by σ_{TF}

$$\sigma_{b2} = \sigma_{TF}\, \frac{x_0\, c}{r_y^2}$$

$$\sigma_e = \frac{\pi^2\, E}{(K\, L_b/r_b)^2}$$

σ_{TF} = average elastic torsional-flexural buckling stress, that is, axial load at which torsional-flexural buckling occurs divided by the full cross-sectional area of member

A, E, r_0, r_y, σ_{ex}, σ_t, σ_{TFO} are as defined in **6.6.1.2**.

6.7.3 *Singly-Symmetric Shapes or Intermittently Fastened Singly-Symmetric Components of Built-Up Shapes Having $Q < 1{\cdot}0$ Which May Be Subject to Torsional-Flexural Buckling* — If $Q < 1{\cdot}0$ singly-symmetric shapes or intermittently fastened singly-symmetric components of built-up shapes subject to both axial compression and bending applied in the plane of symmetry can be conservatively proportioned by replacing F_y by QF_y in **6.7.2**, or their strength may be determined by tests in accordance with **9**. Q is defined in **6.6.1.1**.

6.7.4 *Singly-Symmetric Shapes Which Are Nonsymmetrically Loaded* — Singly-symmetric shapes subject to both axial compression and bending applied out of the plane of symmetry shall be designed according to **9.2**.

6.8 Cylindrical Tubular Members in Compression or Bending — For cylindrical tubular members with a ratio D/t of mean diameter to wall thickness not greater than $232\,000/F_y$, the compression stress shall not exceed the basic design stress F.

For cylindrical tubular members with a ratio D/t of mean diameter to wall thickness larger than $232\,000/F_y$ but not greater than $914\,000/F_y$ the compression stress shall not exceed

$$F_T = \frac{46\,540}{D/t} + 0{\cdot}399\ F_y$$

For compression members the allowable stress P/A under axial load shall also not exceed F_{a1} as prescribed by **6.6.1.1** for $Q = 1$.

7. CONNECTIONS

7.1 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.

7.2 Welds

7.2.1 *Fusion Welds* — Fusion welds shall be proportioned so that stresses therein do not exceed the following values:

Specified Minimum Yield Point of Lowest Strength Steel Being Joined kgf/cm²	*Permissible Stress in Shear on Throat of Fillet or Plug Welds* kgf/cm²
$\leqslant$ 2 500	955
$>$ 2 500 but $\leqslant$ 3 500	1 100
$>$ 3 500	1 250

The allowable stress in tension or compression on butt welds shall be the same as prescribed for the lower grade of the base metals being joined, provided the welds are of full penetration type and the yield strength of the filler metal is equal to or greater than the yield strength of the base metal. Stresses due to eccentricity of loading, if any, shall be combined with the primary stresses, and the combined 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.

7.2.2 *Resistance Welds* — In sheets joined by spot welding the allowable shear per spot shall be as follows:

Thickness of Thinnest Outside Sheet	*Allowable Shear Strength per Spot*	*Thickness of Thinnest Outside Sheet*	*Allowable Shear Strength per Spot*
(1)	(2)	(1)	(2)
mm	kg	mm	kg
0·25	23	2·00	489
0·50	57	2·50	625
0·80	102	2·80	750
1·00	159	3·15	909
1·25	239	5·00	1 818
1·60	330		

NOTE — The above values are based upon AWS C-1.1-66 'Recommended practices for resistance welding', issued by the American Welding Society, and apply to pulsation welding as well as spot welding. They are applicable for all structural grades of low carbon steel, and are based on a factor of safety of approximately 2·5 applied to selected values from AWS C-1.1-66 Tables 1.1 and 1.3. Values for intermediate thicknesses may be obtained by straight line interpolation. The above values may also be applied to medium carbon and low alloy steels. Spot welds in such steels give somewhat higher shear strengths than those upon which the above values are based, however, they may require special welding conditions. In all cases welding shall be performed in accordance with IS : 819-1957*.

7.3 Connecting Two Channels to Form an I-Section — The maximum permissible longitudinal spacing of welds or other connectors, S_{Max} joining two channels to form an I-section shall be:

a) *For Compression Members*:

$$S_{Max} = \frac{L\, r_{cy}}{2\, r_1}$$

where

L = unbraced length of compression members;

*Code of practice for resistance spot welding for light assemblies in mild steel.

r_{cy} = radius of gyration of one channel about its centroidal axis parallel to web; and

r_1 = radius of gyration of I-section about the axis perpendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing, if any.

b) *For Flexural Members*:

$$S_{Max} = L/6$$

In no case shall the spacing exceed the value

$$S_{Max} = \frac{2\,g\,T_s}{m\,q}$$

where

L = span of beam;

g = vertical distance between the two rows of connections near or at top and bottom flanges;

T_s = strength of connection in tension;

m = distance of shear centre of channel from mid-plane of the web, for simple channels without stiffening lips at the outer edges,

$$m = \frac{w_f^2}{2\,w_f + d/3}\text{ ; and}$$

q = intensity of load on beam (*see* **7.3.1**).

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

$$m = \frac{w_f\,dt}{4\,I_x}\left[w_f\,d + 2\,d_1\left(d - \frac{4\,d_1^2}{3\,d}\right)\right]$$

where

w_f = projection of flanges from inside face of web (for channels with flanges of unequal width, w_f shall be taken as the width of the wider flange);

d = depth of channel or beam;

d_1 = overall depth of lip; and

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

7.3.1 The intensity of load q is obtained by dividing the magnitude of concentrated loads or reactions by the length of bearing. For beams designed for a uniformly distributed load, the intensity q shall be taken equal to three times the intensity of the uniformly distributed design load. If the length of bearing of a concentrated load or reaction is smaller than the weld spacing, s, the required strength of the welds or connections closest to the load or reaction P, is

$$T_s = Pm/2\,g$$

7.3.2 The required limited spacing of connections S_{Max} depends upon the intensity of the load directly at the connection. Therefore, if uniform spacing of connections is used over the whole length of the beam, it shall be determined at the point of maximum local load intensity. In cases where this procedure would result in uneconomically close spacing either of the following methods may be adopted:

a) The connection 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 strength in shear of the connections joining these plates to the flanges shall then be used for T_s and g shall represent the depth of the beam.

7.4 Spacing of Connections in Compression Elements — The spacing s 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 in **7.2**; nor

b) $1\,680\ t/\sqrt{f}$, where t is thickness of cover plate or sheet, and f is design stress in cover plate or sheet; nor

c) three times the flat width w of the narrowest unstiffened compression element in that portion of the cover plate or sheet which is tributary to the connections, but need not be less than $1\,590\ t/\sqrt{F_y}$ if the value of F_c permitted in the unstiffened element is greater than $0{\cdot}54\ F_y$ or $1\,910\ t/\sqrt{F_y}$ if the value of F_c permitted in the unstiffened element is $0{\cdot}54\ F_y$ or less, unless closer spacing is required under (a) or (b) above.

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 13 mm. In all other cases the spacing shall be taken as centre to centre distance between connections.

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

7.5 Bolted Connections — The following requirements govern bolted connections of cold formed steel structural members.

7.5.1 *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 centre 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 1·5 d nor less than $P/(0{\cdot}6\ F_y\ t)$

where

d = diameter of bolt,
P = force transmitted by bolt,
t = thickness of thinnest connected sheet, and
F_y = yield point.

7.5.2 *Tension Stress on Net Section* — The tension stress on the net section of a bolted connection shall not exceed 0·6 F_y nor shall it exceed:

$$(1{\cdot}0 - 0{\cdot}9\, r + 3\, rd/s)\ 0{\cdot}6\ F_y$$

where

r = the force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section. If r is less than 0·2, it may be taken equal to zero;
s = spacing of bolts perpendicular to line of stress. In the case of a single bolt, s is equal to the width of sheet; and
d and F_y are defined in **7.5.1**.

7.5.3 *Bearing Stress in Bolted Connections* — The bearing stress on the area ($d \times t$) shall not exceed 2·1 F_y*.

7.5.4 *Shear Stress on Bolts* — Shear stress on the gross cross-sectional area of bolt, under dead and live load, shall not exceed the following values:

Precision and semi-precision bolts	970 kgf/cm²
Black bolts	820 kgf/cm²
Steel conforming to property class **4.6** of IS : 1367-1967†	1 060 kgf/cm²

8. BRACING REQUIREMENTS

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

8.1 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 or sheathing shall 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:

*If the ratio of tensile strength to yield point is less than 1·35, a stress equal to the specified minimum tensile strength of the material divided by 1·35 shall be used instead of F_y in applying the provisions of **7.5.1**, **7.5.2** and **7.5.3**.

†Technical supply conditions for threaded fasteners (*first revision*).

$$a_{\text{Max}} = \frac{8\,E\,I_2\,K_w}{A^2\,F_y^{\,2}}$$

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

$$a_{\text{Max}} = \frac{L\,r_2}{2\,r_1}$$

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

$$K_w = \frac{F_y^{\,2}\,a\,A^2}{8\,E\,I_2}$$

d) The lateral force in kg 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

$$P_{\text{Min}} = \frac{K_w\,P_s\,L/240}{2\sqrt{E\,I_2\,K_w/a} - P_s}$$

In the above formulae:

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

A = area of cross section of stud;

E = modulus of elasticity = 2 074 000 kgf/cm^2;

F_y = yield point of steel in stud;

I_1 = moment of inertia of cross section of stud about its axis parallel to wall;

I_2 = moment of inertia of cross section of stud about its axis perpendicular to wall;

K_w = modulus of elastic support of wall material (on each side of stud) and its attachments. This is, $K_w = P/e$ where P is the force which produces an elongation of e in a strip of wall material of width a and of length equal to the distance between adjacent studs. In paragraphs (b) and (d), K_w is the modulus actually provided as determined from tests. In paragraph (c), K_w is the minimum required for a given spacing of attachments;

L = length of stud;

P_s = design load on stud;

r_1 = radius of gyration of stud about its axis parallel to wall $= \sqrt{I_1/A}$;

r_2 = radius of gyration of stud about its axis perpendicular to wall $= \sqrt{I_2/A}$.

8.2 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*.

8.2.1 *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 centre of this loaded length.

8.2.2 *Design of Braces* — Each intermediate brace, at top and bottom flange, shall be designed to resist a lateral force P_1 determined as follows:

a) For a uniformly loaded beam $P_1 = 1{\cdot}5\ K'$ times the load within a distance $0{\cdot}5\ a$ each side of the brace.

b) For concentrated loads $P_1 = 1{\cdot}0\ K'$ times the concentrated load P within a distance $0{\cdot}3\ a$ each side of the brace, plus $\frac{1{\cdot}0}{0{\cdot}7}\left(1 - \frac{x}{a}\right) PK'$ for each such concentrated load P located farther than $0{\cdot}3\ a$ but not farther than $1{\cdot}0\ a$ from the brace.

In the above formulae:

For channels:

$$K' = m/d$$

where

m = distance from shear centre to mid-plane of the web, as specified in **7.3**; and

d = depth of channel.

For Z-sections:

$$K' = I_{xy}/I_x$$

where

I_{xy} = product of inertia of full section about centroidal axis parallel and perpendicular to web, and

I_x = moment of inertia of full section about centroidal axis perpendicular to web.

For channels and Z-sections:

x = distance from concentrated load P to brace, and

a = length of bracing interval.

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*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.

End braces shall be designed for one-half of the above forces. Braces shall be designed to avoid local crippling at the points of attachment to the member.

8.2.3 *Allowable Stresses* — For channels and Z-beams intermediately braced according to the requirements of **8.2.1** and **8.2.2** the maximum compression stress shall be that specified in **6.3**, except that the length of the bracing interval, a, shall be used instead of the length L in the formulae of **6.3**.

8.3 Laterally Unbraced Box Beams — For closed box type sections used as beams subject to bending about the major axis, the ratio of the laterally unsupported length to the distance between the webs of the section shall not exceed 175 700/F_y.

9. TEST FOR SPECIAL CASES

9.1 General

9.1.1 Where the composition or configuration of elements, assemblies, or details of cold-formed steel structural members are such that calculation of their safe load-carrying capacity or deflection cannot be made in accordance with **5** to **8** of this code, their structural performance shall be established from tests and evaluated as specified in **9.2**.

9.1.2 Tests for determination of mechanical properties of full sections to be used in **6.1.1.1** shall be made as specified in **9.3.1**.

9.1.3 Tests for determining mechanical properties of flat elements of formed sections and representative mechanical properties of virgin steel to be used in **6.1.1.1** shall be made in accordance with the provisions of **9.3.2** and **9.3.3**.

9.1.4 The provisions of **9** do not apply to light gauge steel diaphragms.

9.1.5 Tests shall be made by an independent testing laboratory or by a manufacturer's testing laboratory.

9.1.6 Tensile testing procedures shall be according to IS : 1608-1972*.

9.2 Evaluation of Tests for Determining Structural Performance — Where tests are necessary for the purposes defined in **9.1.1** they shall be evaluated 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 ±10 percent. If such deviation from the mean exceeds 10 percent

*Method for tensile testing of steel products other than sheet, strip, wire and tube.

†The test evaluation procedures and load factors specified in **9.2** are not applicable to confirmatory tests of members and assemblies whose properties can be calculated according to **5** to **8** for the latter, the code provides generally a safety factor of 5/3.

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 a total load, including the weight of the test specimen, equal to twice the live load plus one-and-a-half the dead load without failure. Where the governing design load is due in whole or part to wind, earthquake loads, or combined forces, the foregoing load factors shall be reduced by dividing by 1⅓ in accordance with **6.1.2**.

Furthermore, harmful local distortions which interfere with the proper functioning of the member or assembly or its connections shall not develop during the test at a total load, including the weight of the test specimen, equal to the dead load plus 1½ times the live load.

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

Consideration shall also be given to any variation or difference which may exist between the design thickness and the thickness of the specimens used in the tests.

9.3 Tests for Determining Mechanical Properties of Formed Section of Flat Material

9.3.1 *Full Section Tests* — These provisions are intended to apply only to the determination of the mechanical properties of full formed sections for the purposes defined in **9.1.2**. They are not to be construed as permitting the use of test procedures instead of the usual design calculations. Tests to determine mechanical properties shall be conducted in accordance with the following:

a) For tensile yield point determinations refer to **9.1.6**.

b) Compressive yield point determinations shall be made by means of compression tests* of short specimens of the section and shall be taken as the smaller value of either the maximum compressive strength of the section divided by the cross-sectional area or the stress defined by one of the following methods:

   1) For sharp yielding steel the yield point shall be determined by the autographic diagram method or by the total strain under load method,

   2) For gradual yielding steel the yield point shall be determined by the strain under load method or by the 0·2 percent offset method.

*See Appendix A for recommendations regarding details of compression testing.

When the total strain under load method is used, there shall be evidence that the yield point so determined agrees substantially with the yield point which would be determined by the 0·2 percent offset method. The methods described above shall agree in general with IS : 1608-1972*.

c) Where the principal effect of the loading to which the member will be subjected in service will be to produce bending stresses, the yield point shall be determined for the flanges only. In determining such yield point tests shall be made on specimens cut from the section. Each such specimen shall consist of one complete flange plus a portion of the web of such flat-width ratio that the value of Q for the specimen is unity.

d) For acceptance and control purposes, two full section tests shall be made from each lot of not more than 50 tonnes nor less than 30 tonnes of each section, or one test from each lot of less than 30 tonnes of each section. For this purpose a lot may be defined as that tonnage of one section that is formed in a single production run of material from one heat or blow.

e) At the option of the manufacturer, either tension or compression tests may be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the yield point of the section when subjected to the kind of stress under which the member is to be used.

9.3.2 *Tests of Flat Elements of Formed Sections* — The yield point of flats F_{yf} shall be established by means of a weighted average of the yield points of standard tensile coupons taken longitudinally from the flat portions of a representative cold-formed member. The weighted average shall be the sum of the products of the average yield point for each flat portion times the ratio of the cross-sectional area of that flat portion to the total area of flats in the cross section. The exact number of such coupons will depend on the shape of the member, that is, on the number of flats in the cross section. At least one tensile coupon shall be taken from the middle of each flat. If the actual virgin yield point exceeds the specified minimum yield point, the yield point of the flats F_{yf} shall be adjusted by multiplying the test values by the ratio of the specified minimum yield point to the actual virgin yield point.

9.3.3 *Acceptance and Control Tests for Mechanical Properties of Virgin Steel* — This provision applies to steel produced to other than IS : 1079-1973† when used in sections for which the increased yield point and ultimate strength of the steel after cold forming are computed from the virgin steel properties according to **6.1.1.1**. For acceptance and control

*Method for tensile testing of steel products (*first revision*).

†Specification for hot rolled carbon steel sheet and strip (*third revision*).

purposes, at least four tensile specimens shall be taken from each lot as defined in **9.3.1**(d) for the establishment of the representative values of the virgin tensile yield point and ultimate strength. Specimens shall be taken longitudinally from the quarter points of the width near the outer end of the coil.

APPENDIX A

COMPRESSION TESTING

(*Clause* 9.3)

A-1. It is recommended that stud column tests be made on flat-end specimens whose length is not less than three times the largest dimension of the section except that it shall be not more than 20 times the least radius of gyration.

If tests of ultimate compressive strength are to be used to check yield point for quality control purposes, the length of the section should not be less than 15 times the least radius of gyration.

It is important, in making compression tests, that care be exercised in centering the specimen in the testing machine so that the load is applied concentrically with respect to the centroidal axis of the section.

A-2. For further information regarding compression testing, reference may be made to the following publications:

ASTM E9 Standard method of compression testing of metallic materials at room temperature issued by the American Society for Testing and Materials.

Technical memoranda No. 2 and 3 of the Column Research Council.

'Notes on Compression Testing of Materials' and 'Stub-Column Test Procedures' reprinted in the Column Research Council Guide to Design Criteria for Metal Compression Members. 2nd Ed. 1966.

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