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The Canadian LRFD Standard for Cold Formed Steel Design

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THE CANADIAN LRFD STANDARD FOR COLD FORMED STEEL DESIGN

by
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1. Introduction

In December, 1984 the Canadian Standards Association (CSA) published a new design standard entitled "Cold Formed Steel Structural Members", alpha-numerically designated CAN3-S136-M84 [Ref.1]. It superseded the decade-old previous edition with the same title but designated S136-1974. Publication of the 1984 edition culminated more than six years of work by the CSA Technical Committee responsible for its contents.

In contrast to the 1974 edition which was based on allowable stress design with a limit states design option, the 1984 standard is based entirely on limit states design principles also referred to as load and resistance factor design (LRFD). Although the standard has been prepared for use with SI (metric) units, the designer is able to employ any other consistent units of measurement by using the applicable general expressions provided alongside the SI expressions.

Since structural design of cold formed steel members is a relatively modern development based on extensive testing, there exists a great deal of test data and documentation to assist in the derivation of appropriate resistance expressions for various limit states (i.e. tension, compression, bending, shear, etc.). Further examination of this test data in comparison with calculated values provided evidence that modification of the previous (1974) requirements for axial compression, combined axial compression and bending, effective widths, and bolted connections was desirable. Also, recent research developments had shown that improvements were possible in the specification of requirements for web bending, web crippling, welding and screw type fasteners. Thus a number of changes have been incorporated in these areas as well. All of the changes reflect an increased understanding of the behavior of cold formed steel structures, members and elements and of cold formed steel as a structural material. The more significant aspects of CAN3-S136-M84 and some comparisons with provisions of its predecessor, and also, where appropriate, with the 1980 AISI Specification for the Design of Cold-Formed Steel Structural Members [Ref.2] will be reviewed.

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2. An LRFD Approach to Cold Formed Steel Design

Compared with allowable stress design, limit states design, or LRFD, affords a better understanding of the relation between the performance requirements of a member and its behavior under loads at the limit of structural usefulness as well as with performance under the smaller loads anticipated in service. LRFD also permits the adoption of a common format in design standards for various materials and in the codes governing end-use such as building codes. The limit states format separates the code parameters, accounting for the uncertainties associated with the determination of loads, from the design standard parameters, accounting for the uncertainties associated with the determination of member resistances. The basic format is shown in Table 1.

TABLE 1 - Limit States Format

Condition	Material Design Standard[1]		Building Code (Use Code)[2]
Ultimate Limit States	Factored Resistance	$\geqslant$	Effect of Factored Loads
Serviceability Limit States	Serviceability Limit	$\geqslant$	Effect of Specified Loads (Unfactored)

[1] Material Design Standard is unique to material being considered
[2] Use Code applies to all materials which may be used

Probabilistic methods provide a means of calibrating the uncertainties associated with the determination of both loads and resistances and some of the more germane literature on the subject is provided in References 3, 4, 5, 6, 7, and 8. In brief, the LRFD approach to cold formed steel design adopted by the technical committee responsible for CAN3-S136-M84 is given in the following section.

3. Calibrating for Structural Reliability in Limit States Design

In limit states design, structural reliability can be specified in terms of a safety index, β, which is determined in the following manner.

Define the following variables:
R = specified nominal value of the resistance
$\bar{R}$ = mean value of the actual resistance
S = specified nominal value of the load effects
$\bar{S}$ = mean value of the actual load effects
ϕ = resistance factor
α = aggregate load factor

Figure 1 shows possible frequency distributions for both load effects and structural member resistance. The figure presumes that: (1) the nominal or assumed value of resistance, R, is less than the mean actual resistance, $\bar{R}$; (2) the nominal or assumed effect of the loads, S, is greater than the mean actual effect of the loads, $\bar{S}$, (3) the factored resistance, ϕR, is slightly greater than the effect of the factored loads, αS, indicating that the code safety criterion has been met. The possibility of loads being greater than the resistance shows that there is a chance of failure, the possibility of which can be reduced to any value desired.

The data on actual loads and resistances indicates that a logarithmic transformation is warranted to transform the data into a more "normal" distribution. Figure 2 shows the frequency distribution of the algebraic difference between the load effects and the resistance, using log values, i.e. $u = \ln R - \ln S$. The area of the curve that is to the left of zero, i.e. $u < 0$, is proportionate to the probability of failure. The distribution shown on Figure 2 has a mean value approximately equal to $\bar{u} \cong \ln\bar{R} - \ln\bar{S}$ and this mean is a certain number of standard deviations away from zero. The safety index, β, is defined as the number of standard deviations the mean of this distribution is from zero.

The safety index, therefore, is directly related to the structural reliability of the design. Increasing β will increase the reliability, and decreasing β decreases the reliability. β is also directly related to the load and resistance factors used in the design.

The National Building Code of Canada [Ref. 10], with the introduction of limit states design, has defined a set of load factors, load combination factors and a set of specified minimum loads to be used in the design. The specification of these loads and load factors has fixed the position of the nominal load distribution, S, and the factored load distribution, αS. The material design standard is then obligated to specify the appropriate resistance function.

Those responsible for writing a design standard are given the loading distribution and load factors, and must calibrate the resistance factors, ϕ, such that the safety index, β, reaches a certain target value. The technical committee responsible for the S136 Standard elected to use a target safety index equal to 3.5, in keeping with a similar level used for other structural materials.

The calibration procedure used to determine the appropriate resistance factors includes a computer simulation of the expected load distribution, the expected resistance distribution and adjusts the resistance factor such that the interaction of the R and S curves produces the target β value. The calibration procedure carried out for the S136 standard has produced different resistance factors for various structural resisting mechanisms, as will be shown in a later section.

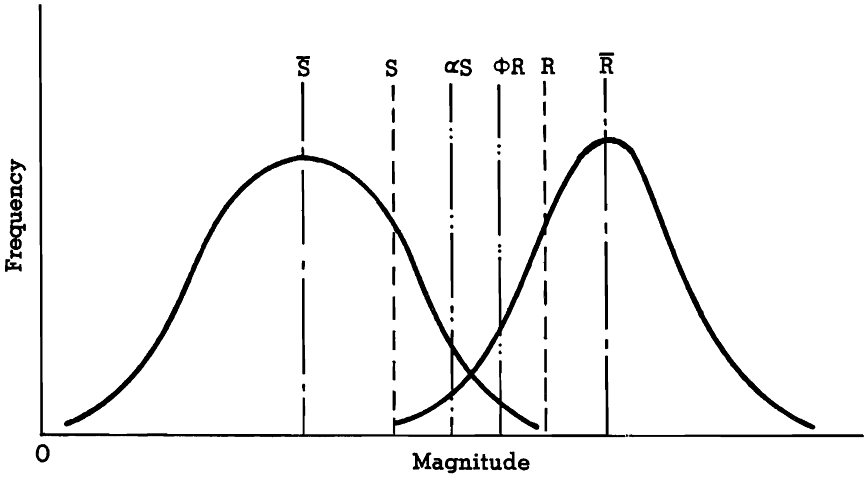

FIGURE 1 : LOAD EFFECTS AND STRUCTURAL MEMBER RESISTANCES

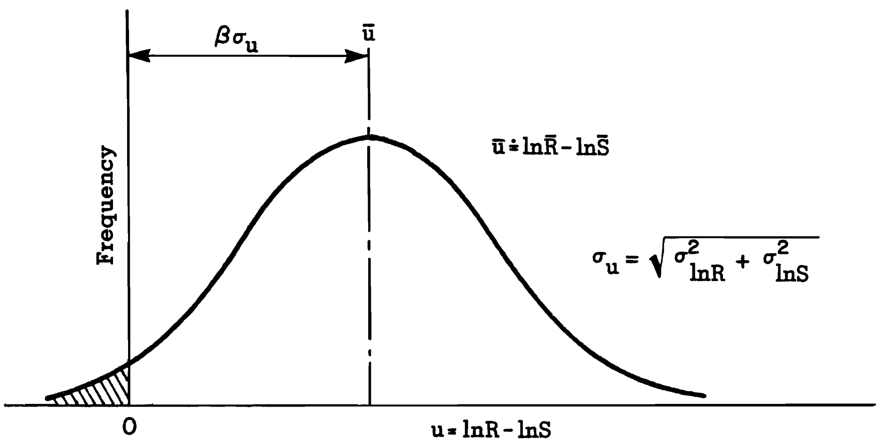

FIGURE 2: ACTUAL LOADS AND RESISTANCES

4. Analysis Level

An LRFD specification can be formatted for various analysis levels. For example, the resistance, ϕR, may express the resistance of the entire structure, an individual member, a cross-section of a member, or an individual fibre or particle. In the case of hot-rolled structural steel and in the case of reinforced concrete, formatting at the cross-sectional level has been found suitable for many applications and LRFD design standards have been developed on that basis for those materials.

The less obvious choice for cold formed steel design was between the cross-sectional level and the individual fibre level, the latter corresponding to a stress analysis format. The fibre level format had been selected for the LRFD option contained in the 1974 edition of the S136 Standard since it closely resembled the allowable stress design method. It also avoided certain complications that the adoption of the cross-sectional level format would have introduced at that time. The 1984 edition, by contrast, has adopted the cross-sectional level format so as to be consistent with the current LRFD formats for structural steel and reinforced concrete.

5. Scope and Application of CAN3-S136-M84

CAN3-S136-M84 covers the limit states design of cold formed carbon or low alloy steel structural members up to 25 mm in thickness, intended for building applications. The resistance factors which have been adopted are correlated with the loads and load factors contained in the National Building Code of Canada (1975, 1977, 1980 and 1985 editions). The load factors are repeated in the S136 standard for completeness. Where the standard prescribes design expressions or dimensional limitations which do not apply to a specific situation, it is permissible to use a rational design based on appropriate theory, test, analysis and engineering judgement.

6. Load Factors and Structural Safety Criterion

Loads to be considered in the design include dead loads (D), live loads (L), loads due to wind or earthquake (Q), and loads due to temperature changes (T) and the like, except that the latter may be omitted where it can be shown that effects due to T-loads do not adversely affect serviceability. The values of the load factors, α, are:

$\alpha_D = 1.25$ or 0.85
$\alpha_L = 1.50$ or 0
$\alpha_Q = 1.50$ or 0
$\alpha_T = 1.50$ or 0

The load combination factor, ψ, is given as:

1.00 when only one of L, Q or T act;
0.70 when two of L, Q or T act;
0.60 when all of L, Q and T act at one time.

The importance factor, γ, can be taken as 1.00 for all buildings, except for farm buildings with low human occupancy and for structures such as bulk storage buildings where it can be shown that collapse is not likely to cause injury or other serious consequences. For buildings which need to remain operational during or after a disaster, the importance factor can be raised above unity so as to create a higher safety margin.

The structural safety criterion requires that structural members be designed to have sufficient strength and stability such that:

Factored Resistance $\geqslant$ Effect of Factored Loads

$$\phi R \geqslant [\alpha_D D + \gamma\psi\{\alpha_L L + \alpha_Q Q + \alpha_T T\}]$$

There are at least sixteen load cases to be considered, although, with experience, some may be classed as redundant in specific instances.

7. Properties of Sections

Since local buckling of cold formed steel elements subjected to compression in flexural bending, axial compression, shear or bearing can occur at stress levels below the yield strength of the steel, post-buckling becomes an important consideration in the design of cold formed steel members. The well-known phenomenon of post-buckling in thin compressed plates is reflected in the effective width approach used in both Canada and the USA when computing section properties of stiffened compresssion elements. It has been long standing practice, in both countries, to compute section properties of stiffened compression elements on the basis of an effective width concept i.e., reduced section properties and full allowable stress in comparison with a reduced stress on the gross or full section as used in the design of unstiffened compression elements.

CAN3-S136-M84 uses an effective width (reduced section properties) approach for both stiffened and unstiffened compression elements, thus providing the designer with a more consistent design method.

In design, the flat width ratio of compressive elements is W except if W exceeds W_{lim}, then a reduced effective width ratio, B, is used. For strength calculations, factored loads are used; for deflection or vibration calculations, specified (unfactored) loads are used in determining B.

Compressive elements are fully effective (B=W) up to the limiting value of

$$W_{lim} = 290\sqrt{\frac{k}{f}} \qquad \left[= 0.644\sqrt{\frac{kE}{f}} \right]$$

For compressive elements with W larger than W_{lim},

$$B = 428\sqrt{\frac{k}{f}}\left[1 - \frac{93.5}{W}\sqrt{\frac{k}{f}}\right] - R \qquad \left[= 0.950\sqrt{\frac{kE}{f}}\left[1 - \frac{0.208}{W}\sqrt{\frac{kE}{f}}\right] - R\right]$$

where,
- k = 4.0 for stiffened compressive elements
- k = 0.5 for unstiffened compressive elements
- R is given by:
  - (a) R = 0 when W < 60; and
  - (b) R = 0.1W - 6 when W ⩾ 60

  When the element or sub-element is stiffened at each edge by means of a web or flange, R may be taken as zero for all values of W.

The expressions on the right in brackets, can be used with any consistent measurement units, since they are written in a non-dimensional format.

8. Member Resistances

As stated previously, all factored resistances must be equal to, or greater than, the effect of the factored loads. The factored resistance of any given member is given by the product ϕR, with ϕ being the resistance factor and R the nominal or theoretical member strength. The resistance factors specified for strength analysis in the new CSA Standard are given in Table 2.

TABLE 2 - Resistance Factors for Strength Analysis

Type of Strength	Resistance Factor
Axial tension, bending and shear:	$\phi = 0.90$
Axial Compression:	
Doubly symmetric sections, angles and wall studs:	$\phi_a = 0.90$
Bearing stiffeners and other sections:	$\phi_a = 0.75$
Web crippling in beams:	
Single unreinforced webs and deck sections:	$\phi_s = 0.80$
Other webs:	$\phi_o = 0.67$
Connections:	$\phi_c = 0.67$
Limit states determined by the tensile strength of the material:	$\phi_u = 0.75$

A number of resistance cases will be discussed in detail in this section, such as members in tension, members in bending, bending in webs, shear in webs, and web crippling of webs.

8.1 Members in Tension

Significant changes have been made in the new S136 Standard in comparison with the 1974 edition and the 1980 AISI specification. The ultimate limit state is defined as that condition in which the member suffers either uncontrolled deformation or collapse. Collapse may be caused either by rupture or instability of the steel.

Uncontrolled deformation is deemed to occur when the gross section yields over a significant length. This gives a limiting state which, while rendering the structure useless, gives some warning of impending failure and may not be the true highest load capacity. The factored resistance for this condition is given by:

$$T_r = \phi A_g F_y$$

Should yielding across the net section be exceeded, it is confined to a narrow band and the absolute overall elongation of the member is comparable to the elastic extension. Thus yielding in this zone does not lead to an ultimate limit state. Failure occurs when the stress at the net section reaches the ultimate value. As this condition is not preceeded by any gross deformation, and failure is precipitous, the resistance factor is lower than that for overall yielding. The factored resistance for this condition is given by:

$$T_r = \phi_u A_n F_u$$

The lesser of the above two values will control the design. This design procedure clearly differentiates between the two types of ultimate states, recognizing the different relationships between the yield and ultimate stresses without attempting to qualify one by the other.

A section dealing with eccentrically loaded tension members has also been added to the new CSA Standard based on the same philosophy as described above, including simple angles with unstiffened legs connected by fasteners in one leg and single channels with unstiffened flanges connected by fasteners in the web.

8.2 Members in Bending

The factored moment resistance of a member in bending equals:

$$M_r = \phi SF,$$

where ϕ is either 0.90 or 0.75, depending on the failure mode considered, S is the section modulus for either compression or tension for four different cases, and F is a stress function that depends on either the yield strength of the steel, F_y, the tensile strength of the steel, F_u, the lateral-torsional buckling stress, F_c, or the web bending stress, F_{wb}, whichever results in the lesser moment resistance.

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, F_c shall be determined as follows:

(a) when $F_b > F'/2$

$$F_c = F' - \frac{(F')^2}{4F_b}$$, but not greater than F_y; and

(b) when $F_b \leqslant F'/2$

$$F_c = F_b$$

For point symmetric Z-shaped sections bent about the centroidal axis perpendicular to the web, F_c shall be determined as follows:

(a) when $F_b > F'$

$$F_c = F' - \frac{(F')^2}{2F_b}$$, but not greater than F_y; and

(b) when $F_b \leqslant F'$

$$F_c = F_b/2$$

where,

F_b = $0.833(F_{be} + F_t)$

F_{be} = $\dfrac{2x10^6\ d\ I_{yc}\ C_b}{L^2 S_{xc}}$ $\left[= \dfrac{\pi^2 E d I_{yc} C_b}{L^2 S_{xc}} \right]$

F_t = $\dfrac{26{,}000\ A\ t^2\ C_b}{d\ S_{xc}}$ $\left[= \dfrac{GAt^2\ C_b}{3dS_{xc}} \right]$

F' = $1.11\ F_y$

C_b = bending coefficient, which may be conservatively taken as unity; or calculated as

$$1.75 + 1.05\left[\frac{M_1}{M_2}\right] + 0.3\left[\frac{M_1}{M_2}\right]^2 \leqslant 2.3$$

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 member, and where M_1/M_2, the ratio of end moments is positive when M_1 and M_2 cause reverse curvature bending, and negative when they cause 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 coeficient, C_b, shall be taken as unity. For members subjected to combined axial and bending forces, C_b shall be taken as equal to unity.

For channels and Z-shaped members with unsiffened flanges and $F_c<F_y$, the factored moment resistance shall be further limited as follows:

$$M_r = \phi S_{cf}\,\frac{91{,}700}{W^2} \qquad \left[= \phi\,\frac{0.5\pi^2 E S_{cf}}{12(1-\mu^2)W^2} \right]$$

8.3 Bending in Webs

Recently, the structural behavior of cold formed steel beam webs subjected to pure bending has been studied in detail. It was found that the postbuckling resistance of web elements under pure bending is a function of four significant parameters, namely, the web slenderness ratio (H), the tension to compression stress ratio (ß), the flat-width ratio of the flange (W) and the yield strength of the material (F_y). From an in-depth study of these parameters [Ref.11], it was found that the postbuckling resistance increases as H and F_y increase, while ß decreases. An increase of W will result in a reduction of the postbuckling resistance.

The limit stress on a beam web due to bending in its plane is computed on the basis of the effective flange area and the full web area as follows:

$$F_{wb} = \Phi \frac{183{,}000k}{H^2} \leqslant F_y \qquad \left[= \Phi \frac{0.902\ Ek}{H^2} \right]$$

where,

$$\Phi = \alpha_1\alpha_2\alpha_3\alpha_4 \geqslant 1.0$$

$$\alpha_1 = 0.017H - 0.79$$

$$\alpha_2 = \frac{0.462}{\beta} + 0.538$$

For beams with stiffened compression flanges

$$\alpha_3 = 1.16 - 0.16 \frac{W}{W_{lim}} \leqslant 1, \text{ when } \frac{W}{W_{lim}} \leqslant 2.25$$

$$\alpha_3 = 0.8, \text{ when } \frac{W}{W_{lim}} > 2.25$$

For beams with unstiffened compression flanges

$$\alpha_3 = 0.84 - 0.019 \frac{W}{W_{lim}}$$

$$\alpha_4 = \left[\frac{F_y}{406} \right] + 0.10$$

$$k = 4 + 2(1 + \beta)^3 + 2(1 + \beta)$$

$$\beta = \left[\frac{f_t}{f_c} \right]$$

8.4 Shear in Webs

In the previous edition of S136, the design expressions for determining the limiting shear stress were developed for beam webs without stiffeners. In the 1984 edition of the Standard, however, shear resistance expressions are provided for webs with and without stiffeners. These provisions are based on the results of a study of beam webs loaded primarily by shear stress [Ref. 12] and are also included in the AISI-1980 specification. Where the web consists of two or more sheets, each sheet shall be considered as a separate member carrying its share of the shear.

The factored shear resistance, V_r, of a web is determined by:

$V_r = \phi A_w F_v$

where F_v is determined as follows:

(a) when $H \leq 450\sqrt{\dfrac{k_v}{F_y}}$ $\left[\leq \sqrt{\dfrac{k_v E}{F_y}} \right]$

$F_v = 0.64\ F_y$

(b) when $450\sqrt{\dfrac{k_v}{F_y}} < H \leq 635\sqrt{\dfrac{k_v}{F_y}}$ $\left[\sqrt{\dfrac{k_v E}{F_y}} < H \leq 1.41\sqrt{\dfrac{k_v E}{F_y}} \right]$

$F_v = \dfrac{289\ \sqrt{k_v F_y}}{H}$ $\left[= \dfrac{0.641\ \sqrt{k_v F_y E}}{H} \right]$

(c) when $H > 635\sqrt{\dfrac{k_v}{F_y}}$ $\left[> 1.41\sqrt{\dfrac{k_v E}{F_y}} \right]$

$F_v = \dfrac{183{,}000 k_v}{H^2}$ $\left[= \dfrac{\pi^2 E k_v}{12(1-\mu^2)H^2} \right]$

where,
k_v = shear buckling coefficient determined as folows:

(a) for unreinforced webs, $k_v = 5.34$

(b) for beams with transverse stiffeners
$k_v = 4 + \dfrac{5.34}{(a/h)^2}$, when $a/h \leq 1.0$; and
$k_v = 5.34 + \dfrac{4}{(a/h)^2}$, when $a/h > 1.0$

8.5 Combined Bending and Shear in Webs

This part of the new Standard provides for the interaction between bending and shear in webs and their effect on the capacity of the web element. The interaction expression from the previous edition of the Standard has been included for unreinforced flat webs. In addition, a new interaction equation has been included for use with beam webs with adequate transverse stiffeners [Ref. 13] which is also used by AISI-1980.

For webs subjected to both bending and shear stresses, the member is to be proportioned such that the following limit is observed:

$$\left[\frac{M_f}{M_r} \right]^2 + \left[\frac{V_f}{V_r} \right]^2 \leq 1.0$$

For beam webs with transverse stiffeners, the member is to be proportioned such that the following limits are observed:

(a) $\frac{M_f}{M_r} \leqslant 1,$

(b) $\frac{V_f}{V_r} \leqslant 1,$ and

(c) $0.6\left[\frac{M_f}{M_r}\right] + \left[\frac{V_f}{V_r}\right] \leqslant 1.3,$ when $\frac{M_f}{M_r} > 0.5$ and $\frac{V_f}{V_r} > 0.7$

where,

$M_r = \phi S_{wb} F_{wb}$, where F_{wb} is calculated without the limit of F_y

V_r = factored shear resistance without the limit of $0.64F_y$ on F_v

8.6 Web Crippling

Considerable changes have been made in the web crippling expressions in comparision to the 1974 edition of S136. These changes were primarily based on recent tests that were carried out in both the USA [Ref. 14] and Canada [Ref. 15]. The most significant change occurred in the addition of expressions for two flange loading, which did not exist in the 1974 edition of S136. These expressions, presented in limit states format in the new S136 Standard, were adopted directly from the 1980 AISI Specification.

In addition, expressions for the design of deck sections (multiple webs) have been added to the new CSA Standard which are not contained in the 1980 AISI specification and did not exist in the 1974 edition of S136. All new web crippling expressions are based solely on testing and the limits generally placed on the various parameters have been expanded to reflect the findings of the most recent research on web crippling. Since the web crippling expressions are somewhat lengthy, only the expressions for deck sections are presented herein.

To avoid crippling of an unreinforced web of a member in bending whose slenderness ration, H, is equal to or less than 200, concentrated loads and reactions are not to exceed the value of P_r. Webs of members in bending, for which H is greater than 200, are to be provided with adquate means of transmitting concentrated loads or reactions directly into the web(s).

P_r represents the load or reaction for one solid web connecting top and bottom flanges. For webs consisting of two or more such sheets, P_r is computed for each individual sheet and the results added to obtain the limiting load or reaction for the full section.

One-flange loading or reaction occurs when the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is greater than 1.5h.

Table 3 - Deck Sections (Multiple Webs)

One-flange Loading or reaction	End	$P_r = \phi_s\, 10\, t^2 F_y (\sin\theta)\, (1 - 0.1k)$ $(1 - 0.1\sqrt{R})\, (1 + 0.005N)$ $(1 - 0.002H)$
	Interior	$P_r = \phi_s\, 18\, t^2 F_y\, (\sin\theta)\, (1 - 0.1k)$ $(1 - 0.075\sqrt{R})\, (1 + 0.005N)$ $(1 - 0.001H)$
Two-flange Loading or reaction	End	$P_r = \phi_s\, 10\, t^2 F_y\, (\sin\theta)\, (1 - 0.1k)$ $(1 - 0.1\sqrt{R})\, (1 + 0.01N)$ $(1 - 0.002H)$
	Interior	$P_r = \phi_s\, 18\, t^2 F_y\, (\sin\theta)\, (1 - 0.2k)$ $(1 - 0.03\sqrt{R})\, (1 + 0.01N)$ $(1 - 0.0015H)$

Notes:

(1) The above formulae apply to deck sections when $R \leqslant 10$, $N \leqslant 200$, $n/h \leqslant 2$, and the cell spacing of the sections does not exceed 200 mm.

(2) For single hat sections, both webs must be fastened to prevent spreading.

where,

$$C_1 = (1.49 - 0.53k) \geqslant 0.6$$

$$C_2 = 1 + \frac{H}{750} \leqslant 1.2$$

$$C_3 = \frac{1}{k} \text{ when } H \leqslant 66.5$$

$$C_3 = \left[1.1 - \frac{H}{665}\right]/k \text{ when } H > 66.5$$

$$C_4 = \left[0.98 - \frac{H}{865}\right]/k$$

$$k = F_y/230 \text{ (in MPa)}$$
$$= F_y/33 \text{ (in ksi)}$$

Unreinforced flat webs of shapes subjected to a combination of bending and web crippling shall be designed to meet the following requirements:

$$P_f/P_r + M_f/M_r \leqslant 1.3$$

Combined web crippling and bending does not need to be checked for multiple web deck sections. The stability provided by the multiple elements, as well as a long in-service history indicate that the combined action of web crippling and bending is not a problem with deck sections. Therefore, multiple web deck sections are excluded from the combined web crippling and bending criteria.

8.7 Inelastic Reserve Resistance of Members in Bending

This is a new section in S136-M84 and was taken directly from the 1980 AISI Specification but expressed in an LRFD format.

The inelastic reserve resistance of laterally supported flexural members is the additional moment which many flexural members develop over and above the yield moment, before the ultimate failure moment is reached; i.e. the inelastic reserve capacity is M(ult)-M(yield). Some hot-rolled sections, having restricted width-to-thickness ratios, can develop the full plastic moment, but the thin-walled members used in cold-formed steel construction generally are unable to reach the plastic hinge plateau: however, many sections do develop reserve capacities over and above the yield moment. A particularly favourable situation arises when the neutral axis is so located that yielding starts in the tension fibre. A detailed discussion of the inelastic reserve resistance of flexural members is given in Reference 15 which was used as the basis for this section.

8.8 Stiffeners for Beam Webs

This new section has been adopted from the 1980 AISI Specification, and provides design requirements for bearing and intermediate stiffeners.

In the case of bearing stiffeners, the resistance of the transverse stiffeners to end crushing is being considered as well as the resistance of column-type buckling of the web stiffeners.

The new expressions for determining the minimum required moment of inertia and the minimum required gross area of attached intermediate stiffeners are based on the studies summarized in Reference 13. In this reference, test data shows that even though the shear resistance expressions are based on the buckling strength of web elements rather than on tension field action, it is still necessary to provide the required moment of inertia and gross area of intermediate stiffeners.

8.9 Concentrically Loaded Compression Members

The 1974 edition of S136 and the AISI-1980 specification used the tangent modulus approach with a constant factor of safety (1.92) in the design of cold formed steel compression members. The overall column strength is reduced by the introduction of a local buckling factor that reflects the interaction of local and overall buckling in the Inelastic region only (defined by a Johnson parabola). Upon investigating the test data by DeWolf [Ref.17] it was found [Ref.18] that the effects of local buckling extended into the elastic (Euler) buckling region, neglect of which resulted in unconservative design. This prompted the S136 committee to study the problem in an effort to establish an alternative design approach. The present S136 edition uses the tangent modulus approach but the interaction effect of local buckling is included over the entire column strength curve.

A previously noted major departure in the new S136 Standard from earlier Standards is the adoption of the effective width concept for both stiffened and unstiffened sections, whereas most other Standards use the effective width concept for stiffened sections and the effective stress approach for unstiffened sections. The adoption of the effective width approach for all sections has led to a uniform formulation of the column design procedure. Only the effective area has to be determined using the basic effective width formula with the factor "k" (4.0 or 0.5) and the stress level appropriately selected. Using the gross section properties for slenderness ratio computation, the appropriate compressive stress is determined from the column curve. The effective cross-section is then re-calculated at this stress rather than at the stress level originally assumed (e.g. yield stress). The column capacity is taken to be the product of the re-calculated effective area and the overall column compressive stress.

The above design procedure applies only to members in which the resultant of all loads and moments acting on the member is equivalent to a single force acting through the centroid of the cross section in the direction of the member axis and the material is 4.5mm or less in thickness; the design of members formed from thicker material is required to comply with the provisions of CSA Standard CAN3-S16.1 [Ref.19] and Supplements thereto (i.e. structural steel design).

For members in which the maximum flat width ratio of stiffened compressive elements does not exceed 150, and for which the maximum flat width ratio of unstiffened compressive elements does not exceed 35, the compressive resistance, C_r, is determined by:

$$C_r = \phi_a A_e F_a$$

where the compressive limit stress, F_a, is determined as follows:

(a) when $F_p > F_y/2$

$$F_a = F_y - \frac{F_y^2}{4F_p}$$

(b) when $F_p \leqslant F_y/2$

$$F_a = F_p$$

For I-sections, closed cross sections, and any other sections that can be shown to be not critical in torsional buckling or not subject to torsional-flexural buckling, F_p is given by

$$F_p = 0.833F_e$$

where,

$$F_e = 2\text{x}10^6/(KL/r)^2 \qquad (= \pi^2 E/(KL/r)^2)$$

For singly symmetric open sections, such as plain and lipped channels and single or double plain and lipped angles, which may be subject to torsional-flexural buckling, F_p is given by:

$F_p = 0.833F_{st}$ or $0.833F_e$, whichever is less

where,
F_e is as applicable, and

$$F_{st} = \frac{1}{2\beta}\left[F_s + F_t - \sqrt{(F_s + F_t)^2 - 4\beta F_s F_t} \right]$$

in which,

$$F_s = 2\text{x}10^6/(KL/r)^2 \qquad (= \pi^2 E/(KL/r)^2)$$

$$F_t = \frac{1}{Ar_0^2}\left[78{,}000J + \frac{2\text{x}10^6\ C_w}{(K_tL_t)^2} \right] \qquad \left[= \frac{1}{Ar_0^2}\left[GJ + \frac{\pi^2 EC_w}{(K_tL_t)^2} \right] \right]$$

$$\beta = 1 - (x_o/r_o)^2$$

For channels, Z-shapes, and single angle sections with unstiffened flanges, the factored compressive resistance is further limited as follows:

$$C_r = \phi A \frac{91{,}700}{W^2} \qquad \left[= \frac{\phi(0.50)\pi^2 EA}{12(1-\mu^2)W^2} \right]$$

This additional limit may be waived if the channel or Z-sections are fully restrained with respect to torsion and flexural buckling about the asymmetric axis.

For point symmetric open sections, such as cruciform sections or such built-up sections which may be subject to torsional buckling and which are not braced against twisting, F_p is taken as the lesser of $0.833F_e$ or $0.833F_t$.

For the design of hollow structural section compressive members that comply with the requirements of CSA Standard CAN3-G40.20 [Ref. 9], the design is to be in accordance with CSA Standard CAN3-S16.1 and Supplements thereto.

For nonsymmetric sections whose cross sections do not have any symmetry, either about an axis or a point, and for sections formed with any stiffened elements whose flat width ratio exceeds 150 or any unstiffened elements whose flat width ratio exceeds 35, the factored compressive resistance is to be determined by rational analysis. Alternatively, compressive members composed of such sections may be tested in accordance with the appropriate testing procedure given in the Standard.

For compressive members composed of two or more sections connected together at discrete points, such as double angles and battened channels, the factored compressive resistance for buckling about the built-up member axis is computed by setting $F_p = 0.833F_e$.

where,

$$F_e = \frac{2x10^6}{[(KL/r)^2 + (a/r_1)^2]} \quad \left[= \frac{\pi^2 E}{[(KL/r)^2 + (a/r_1)^2]} \right]$$

8.10 Combined Axial Load and Bending

The interaction expressions contained in the new S136 Standard for doubly symmetric shapes are similar to those in CAN3-S16.1-M84, i.e. members are to be proportioned to meet the following requirements:

(a) $$\frac{C_f}{C_r} + \frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leqslant 1.0$$

(b) $$\frac{C_f}{C_r} + \frac{\omega_x M_{fx}}{M_{rx}\alpha_x} + \frac{\omega_y M_{fy}}{M_{ry}\alpha_y} \leqslant 1.0$$

A completely new section has been added for singly symmetric sections subject to a combination of axial load and bending about two axes. The method is based on the above interaction expressions with some modifications, taken from Reference 20.

To cover the case of singly symmetric open sections loaded with an axial load and a bending moment about the axis of symmetry, the M_{rx} term in the interaction equation is redefined. If only bending moment about the axis of symmetry was to act on the section then the allowable stress F_{bx} is based on a torsional-flexural buckling stress F_{cr} in the elastic range. A Johnson parabola defines the inelastic region. Although the formulae are somewhat time consuming to calculate, the approach is perhaps an improvement over the AISI Specification which relegates this type of member and loading to test.

For the case of an axial load acting with a moment about the axis of asymmetry, the M_{ry} term is redefined. This formulation is similar to the AISI Specification if the point of application of the load is located on the side of the centroid opposite from that of the shear centre. In this case, for example, the stiffening lips of a channel whould be in compression. For the opposite case, if the stiffening lips are in tension, and for small eccentricities, the Standard is more conservative than the AISI Specification. As the eccentricity increases the difference between the standards decreases. The approach that was adopted is intended to simplify the design procedure for members subject to both axial compression and bending. Even though the procedure has been simplified, the expressions are quite lengthy and will not be reproduced here.

8.11 Wall Studs

The provisions for the design of wall studs were taken directly from the 1980 AISI Specification and expressed in the appropriate LRFD format. Since the procedure is rather lengthy, only the design criteria and parameters are presented herein.

The factored compressive resistance of a stud may be computed on the basis that the wall material or sheathing (attached to one or both sides of the stud) furnishes adequate lateral and rotational support to the stud in the plane of the wall, provided that stud, wall material and attachments comply with the following requirements:

(a) both ends of the stud are to be braced against rotations about the stud axis and horizontal displacements perpendicular to the stud axis; however, the ends may or may not be free to rotate about both axes perpendicular to the stud axis;

(b) the sheathing is to be connected to the top and bottom members of the wall assembly to enhance the restraint provided to the stud and stabilize the overall assembly;

(c) sheathing is to retain adequate strength and stiffness for the expected service life of the wall; and

(d) supplementary steel bracing may be required for adequate structural integrity during construction and in the completed structure.

The design provisions are given to prevent three possible modes of failure. Provision #1 is for column buckling between fasteners even if one fastener is missing or otherwise ineffective. Provision #2 contains expressions for overall column buckling. Essential to these provisions is the magnitude of the shear rigidity of the wall board material. S136 gives values and an expression for determining the shear rigidity which are based on the small scale tests described in References 21 and 22. For other similar types of materials, the parameters provided can be determined using the procedures described in these references.

The effects of local buckling of multiple punched or slit flat elements of wall studs on the overall behaviour is accounted for in provision #2 through the use of the effective area, A_e, in the compressive resistance expression, C_r.

Provision #3 is a shear strain compatibility check in the wallboard to ensure that the wallboard has sufficient distortional capacity. Due to the complexity of the expressions involved, the procedure involves assuming a value of the ultimate limit stress, s, and checking whether or not the shear strain at the corresponding ultimate limit stress exceeds the allowable design shear strain of the wallboard material. In principle, the procedure is one of successive approximations, however, if the smaller of the F_a values obtained from provisions #1 and #2 is tried first and is shown to be satisfactory, then the need for iteration is eliminated. This has been substantiated in Reference 23.

9. Connections

9.1 Welded Connections

9.1.1. Arc Spot Welds: Explicit design equations for arc spot welds have been incorporated for the first time. Based on research done on behalf of the S136 Technical Committee, design expressions were developed for the shear resistance and tensile resistance of arc spot welds. These equations are dependent only on the sheet steel thickness, and are as follows:

$$V_r = \phi_c 10^3 (20t - 5)$$

$$T_r = \phi_c 10^3 (5.6t - 1)$$

Arc spot welding is a common field fastening method for sheet steel products, particularily roof deck and floor deck. The resistance equations provided have been based on limited test data and are only valid for sheet steel within the following limitations: (a) sheet steel thickness range of 0.70 mm to 1.67 mm inclusive; (b) supporting structural element having a thickness at least 2.5 times the sheet steel thickness; (c) sheet steel of weldable quality having a yield strength of 230 MPa or greater; (d) minimum edge distance of 25 mm; (e) electrodes E410XX or E480XX; and (f) the weld to be circular with a visible nominal diameter of 20 mm.

9.1.2. Fusion Welds: The requirements for fusion welded connections contain provisions formulated from research undertaken in the USA. This work was sponsored by AISI, performed by Pekoz and McGuire [Ref.24], and constitutes the provisions covering fillet welds and flare bevel groove welds. The requirements in the previous (1974) edition for these weld types directed the designer to CSA Standard W59, Welded Steel Construction (Metal-Arc Welding). Unfortunately, this standard had no explicit provisions for calculating the capacity of welds on sheet steel material thicknesses.

The work by Pekoz and McGuire investigated the welding of sheet steel and developed design provisions which were subsequently adopted by the American Welding Society Standard AWS D1.3-81, "Structural Welding Code - Sheet Steel" [Ref.25] and are also included in the S136-84. There are now design equations for predicting the capacity of: fillet welds loaded parallel and perpendicular to the direction of the weld; and flare-bevel groove welds loaded parallel and perpendicular to the direction of the weld.

9.1.3. Resistance Welds: The previous edition of S136 provided only a table of allowable shear strength values of resistance welds for different sheet thicknesses. To provide a resistance expression in a format consistent with the rest of the Standard, a linear expression for the calculation of the shear strength of resistance welds was derived from the data given in the table of values. This expression is dependent only on the sheet thickness and is valid only within a specific thickness range.

9.2 Connections made by Bolts, Screws, or Solid Rivets

A number of changes have been made in the design of connections, especially bolted connections. For bolts and solid rivets, the shear resistance of the fastener itself is given by:

$$V_r = \phi_c 0.6 A_b F_u$$

If bolt threads are in a shear plane, the aforementioned value of V_r is to be multiplied by 0.7.

For screws and special fasteners, to which the above cannot be applied, the factored shear resistance is to be taken as ϕ_c times the manufacturer's certified ultimate shear resistance in the condition specified.

The factored bearing resistance of the connected sheet for each loaded single fastener shall be determined as follows:

$$B_r = \phi_c e t F_u \leq \phi_c C d t F_u$$

where,
C = the appropriate value from Table 4

Although it is recommended that a washer be used under the end of the fastener which is turned, the values for Table 4 may be applied whether or not washers are used.

Bearing resistance is independent of whether the thread or shank bears or of any tension in the fastener.

TABLE 4 - Factor C, for Bearing Resistance of Fasteners

Ratio of bolt diameter to sheet thickness, d/t	C
$d/t \leq 10$	3
$10 < d/t < 15$	30t/d
$d/t \geq 15$	2

Where the end edge is oblique to the line of action of the force, the resistance for a single fastener is given by the lesser of the capacity of a single fastener as calculated using the above equation, or the following:

$$B_r = \phi_c(e + (e - d)\cos^2\theta)tF_u$$

For simple lap joints in tension connected by screws or hollow rivets, the sheets are free to distort, allowing the fasteners to become inclined. The factored bearing resistance of each fastener for this type of joint shall be given by the lesser of a single fastener from above or the following:

$$B_r = \phi_c C(t + t_1)dF_u/4$$

Where the force is directed away from the edge, or the group fasteners is remote from an edge, the bearing resistance of a group fasteners in which the centre-to-centre distance between fasteners at least Cd shall be equal to the sum of the individual resistances.

If the spacing is less than Cd, but not less than 2.5d, the resistance shall be reduced proportionately.

For fastener groups where the force is directed towards an edge, the factored resistance shall be the lesser of that given above and that given by the following:

(a) rectangular groups as shown in Fig. 3a
$$B_r = \phi_c[(m - 1)(g - d) + (n - 1)(s - d) + e]tF_u$$

(b) triangular groups as shown in Fig. 3b
$$B_r = \phi_c[2(m - 1)(g - d + s^2/4g) + e]tF_u$$

The above formulae represent the force required to tear out the portion bound by the failure planes ABCD, indicated in Figure 3. For other fastener patterns, the tear-out resistance shall be shown to be adequate.

10. Testing

10.1 Virgin Steel Properties: These types of tests are tensile tests to determine the mechanical properties of virgin steel or the flats of cold formed sections produced to material standards other than those recognized by the S136 Standard.

10.2 Cold Formed Steel Properties: These types of tests are full section tests to determine the modified mechanical properties of steel after cold forming for utilization of the change in strength to be permitted.

10.3 Performance Tests: These types of tests are structural performance tests to establish the limit states of structural elements or assemblies for which the composition or configuration is such that the calculation of their factored resistance or deformation cannot be made in accordance with the provisions of the Standard.

10.4 **Confirmatory Tests**: These types of tests are confirmatory tests to verify the resistance to specified factored loads of structural elements or assemblies designed in accordance with the provisions of the Standard. These tests are not to be used to establish resistances greater than those computed in accordance with the provisions of the Standard.

11. Summary

CAN3-S136-M84, Cold Formed Steel Structural Members, is a design standard available today which has incorporated the latest research into the behaviour of cold formed steel structural members. In addition to being state of the art, this standard provides the designer with a specification written in Limit States Design incorporating SI metric units, as well as a nondimensional LRFD format.

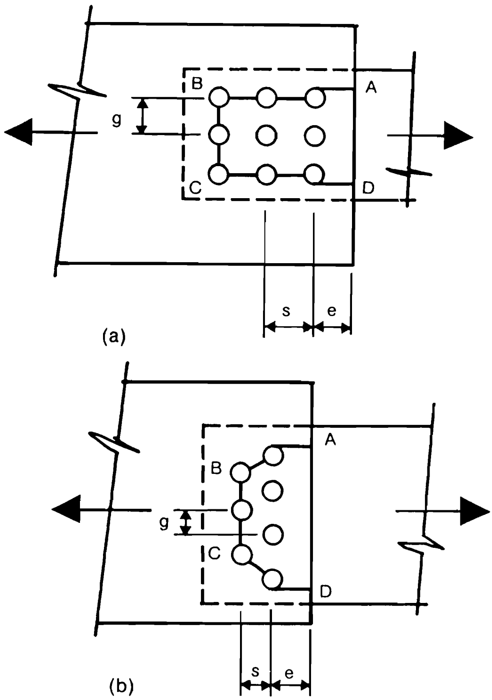

FIGURE 3: TEAR-OUT OF BOLT GROUPS

REFERENCES

1. Canadian Standards Association, "Cold Formed Steel Structural Members", CAN3-S136-M84, Rexdale, ON, 1984.

2. American Iron and Steel Institute, "Specification for the Design of Cold-Formed Steel Structural Members", 1980.

3. Allen, D.E., "Limit states design: What do we really want?", Canadian Journal of Civil Engineering, Vol. 8, No. 1, March 1981.

4. Parimi, S.R., & Lind, N.C., "Limit States Basis for Colf-Formed Steel Design", Journal of the Structural Division, ASCE, Vol. 102, No. ST3, March 1976.

5. Allen, D.E., "Limit states design - A probabilistic study", Canadian Journal of Civil Engineering, Vol. 2, No. 1, March 1975.

6. Kennedy, D.J.L., & Gad Aly, M., "Limit states design of steel structures - performance factors", Canadian Journal of Civil Engineering, Vol. 7, No. 1, March 1980.

7. Nowak A.S., & Lind, N.C., "Practical code calibration procedures", Canadian Journal of Civil Engineering, Vol. 6, No. 1, March 1979.

8. Canadian Standards Association, "Guidelines for the Development of Limit States Design", CSA Special Publication S408-1981, Rexdale, ON, 1981.

9. Canadian Standards Association, "Structural Quality Steels", CAN3-G40.21-M81, Rexdale, ON, 1981.

10. National Building Code of Canada, 1985, National Research Council of Canada, Ottawa, ON, 1985.

11. Laboube, R.A. & Yu, W.W., "Structural Behaviour of Beam Webs Subjected to Bending Stress", Final Report, Civil Engineering Study 78-1, University of Missouri-Rolla, MO., June 1978.

12. Laboube, R.A. & Yu, W.W., "Structural Behaviour of Beam Webs Subjected Primarily to Shear Stress", Final Report, Civil Engineering Study 78-1, University of Missouri-Rolla, MO., June 1978.

13. Phung, N. & Yu, W.W., "Structural Behaviour of Transversely Reinforced Beam Webs", Final Report, Civil Engineering Study 78-5, University of Missouri-Rolla, MO., June 1978.

14. Hetrakul, N. & Yu, W.W., "Structural Behavior of Beam Webs Subjected to Web Crippling and a Combination of Web Crippling and Bending", Final Report, Civil Engineering Study 78-5,

University of Missouri-Rolla, MO., June 1978.

15. Wing, B.A., "Web Crippling and the Interaction of Bending and Web crippling of Unreinforced Multi-Web Cold-Formed Steel Sections", M.A.Sc. Thesis, Department of Civil Engineering, University of Waterloo, Waterloo, ON, 1981.

16. Reck, H.P., Pekoz, T. & Winter, G., "Inelastic Strength of Cold-Formed Steel Beams", Journal of the Structural Division, American Society of Civil Engineers, Vol. 101, No. ST11, November 1975, pp. 2193-2203.

17. DeWolf, J.T., "Local and Overall Buckling of Cold-Formed Compression Members", Cornell University Report No.354, Ithaca, NY, 1973.

18. Trestain, T.W.J., "A Review of Cold Formed Steel Column Design", Report No. 81109-1 CSCC project CSS817, December, 1982.

19. Canadian Standards Association, "Steel Structures for Buildings (Limit States Design)", CAN3-S16.1-M84, Rexdale, ON, 1984.

20. Rack Manufacturers Institute, "Industrial Steel Storage Rack Manual", Pittsburgh, PA, 1980.

21. Simaan, A., "Buckling of Diaphragm-Braced Columns of Unsymmetrical Sections and Applications to Wall Studs Design", Department of Structural Engineering, Cornell University, Report No.353, August, 1973.

22. Simaan, A. & Pekoz, T., "Diaphragm-Braced Members and Design of Wall Studs", Proceedings of the American Society of Civil Engineers, Vol. 102, ST1, January 1976.

23. Kwong, J.W.C., "Parameter Study of Cold Formed Steel Wall Studs", Waterloo Department of Civil Engineering, for Professors R.M. Schuster and N.C. Lind, August, 1982.

24. Pekoz, T., & McGuire, W., "Welding of Sheet Steel", AISI Research Report SG 79-2, January, 1979.

25. American Welding Society, Inc., "Structural Welding Code - Sheet Steel", ANSI/AWS D1.3-81, December, 1981.

SYMBOLS

A	Fully effective cross sectional area of member (mm^2);
A_b	Nominal cross sectional area of a fastener (mm^2);
A_e	Effective cross sectional area of a member in compression (mm^2);
A_g	Gross cross sectional area of a member (mm^2);
A_n	Minimum net cross sectional area of a member (mm^2);
A_w	Area of web (mm^2);
a	Distance between transverse stiffeners (mm); fastener spacing (mm);
C_f	Axial compression in a member or component due to factored loads (N);
C_r	Factored compressive resistance of concentrically loaded members(N);
C_w	Warping constant of torsion (mm^6);
d	Clear perpendicular distance between the flats of the flanges (mm); diameter of a fastener (mm);
E	Young's modulus (203 000 MPa);
e	Fastener edge distance (mm);
F_e	Euler elastic buckling stress (MPa);
F_p	Reduced critical elastic buckling stress (MPa);
F_y	Tensile yield strength of virgin steel (MPa);
F_u	Tensile strength of virgin steel (MPa); tensile strength of a fastener (MPa);
F_v	Limiting stress in shear (MPa);
f	Calculated stress in a compressive element computed on the basis of the effective width (MPa);
f_c	Maximum compressive bending stress in web (MPa);
f_t	Maximum tensile bending stress in web (MPa);
g	Spacing of rows of fasteners measured perpendicular to the direction of force (mm);
H	Slenderness ratio (h/t);
h	Clear distance between the flats of flanges measures in the plane of the web (mm);
I_{yc}	Moment of inertia of the compressive portion of the fully effective cross sectional area about its gravity axis parallel to the web(s) (mm);
J	St. Venant torsion constant (mm^4);
K	Effective Length factor;
K_t	Effective length factor for torsional buckling;
KL/r	The greater of the effective slenderness ratios about the principal axes;
L	Unbraced length of member (mm);
L_t	Length of member unsupported against twisting (mm);
M_f	Moment in a member or component due to factored loads (N.mm);
M_{fx}, M_{fy}	Maximum computed moments due to factored loads occurring either at or between braced points (N.mm);
M_r	Factored moment resistance (N.mm);
M_{rx}, M_{ry}	Factored moment resistances with the possibility of lateral stability excluded (N.mm);
m	Number of fasteners in the first row parallel to the edge;

N	Ratio of bearing length to web thickness (n/t);
n	Bearing length (mm); number of rows of fasteners;
P_f	Concentrated load or reaction due to factored loads (N);
P_r	Factored web crippling resistance of members in bending (N);
R	Ratio of inside bend radius to thickness (r/t);
r	Inside bend radius (mm); radius of gyration of the fully effective cross sectional area (mm);
r_o^2	$r_x^2 + r_y^2 + x_o^2$
r_x, r_y	Radii of gyration of the fully effective cross sectional area about the centroidal principal axes (mm);
r_1	Radius of gyration of the fully effective cross sectional area of an individual section in a built-up member (mm);
S_{cf}	Compressive section modulus based on the moment of inertia of the fully effective cross sectional area (gross or net), divided by the distance from the neutral axis to the extreme compressive fibre (mm^3);
S_{wb}	Compressive section modulus based on the moment of inertia of the effective cross sectional area divided by the distance from the neutral axis to the extreme compressive fibre of the web (mm^3);
S_{xc}	Compressive section modulus of the fully effective cross sectional area about the major axis; I_x divided by the distance from the neutral axis to the extreme compressive fibre (mm^3);
s	Spacing between rows of fasteners measured parallel to the direction of force (mm);
t	Base steel nominal thickness (mm);
t_1	Thickness of the thichest connected sheet in a simple lap joint (mm);
V_f	Shear in a member or component due to factored loads (N);
W	Flat width ratio (w/t);
W_{lim}	Limiting flat width ratio for fully effective compressive elements;
x_o	Distance from shear centre to centroid of section (mm);
α_x, α_y	Amplification factors;
θ	Angle between plane of web and plane of bearing surface (degrees); angle made by the endge with the direction of load (degrees);
ϕ	Resistance factor;
ω_x, ω_y	Coefficients used to determine equivalent uniform bending stress;