4.7.2 In-plane frame buckling The elastic buckling load factor (λ_c) of a rigid-jointed frame shall be determined by using—

(a) one of the approximate methods of Clauses 4.7.2.1 and 4.7.2.2; or

(b) a rational elastic buckling analysis of the whole frame.

NOTE: The value of λ_c depends on the load set.

4.7.2.1 *Rectangular frames with all members braced* In a rectangular frame with regular loading and negligible axial forces in the beams, the braced member buckling load (N_{omb}) for each column shall be determined in accordance with Clauses 4.6.2, 4.6.3.3 and 4.6.3.4.

The elastic buckling load factor (λ_m) for each column shall be determined as follows:

$$\lambda_m = \frac{N_{omb}}{N^*}$$

The elastic buckling load factor (λ_c) for the whole frame shall be taken as the lowest of all the λ_m values.

4.7.2.2 *Rectangular frames with sway members* In a rectangular frame with regular loading and negligible axial forces in the beams, the member buckling load (N_{oms}) for each column shall be determined in accordance with Clauses 4.6.2, 4.6.3.3 and 4.6.3.4.

The elastic buckling load factor (λ_{ms}) for each storey shall be determined as follows:

$$\lambda_{ms} = \frac{\sum\left(\frac{N_{oms}}{L}\right)}{\sum\left(\frac{N^*}{L}\right)}$$

where

N^* = member design axial force, with tension taken as negative, and the summation includes all columns within a storey.

The elastic buckling load factor (λ_c) for the whole frame shall be taken as the lowest of all the λ_{ms} values.

SECTION 5 MEMBERS SUBJECT TO BENDING

5.1 DESIGN FOR BENDING MOMENT A member bent about the section major principal x-axis which is analysed by the elastic method (see Clause 4.4) shall satisfy—

$M^*_x \leq \phi M_{sx}$, and

$M^*_x \leq \phi M_{bx}$

where

M^*_x = the design bending moment about the x-axis determined in accordance with Clause 4.4.

ϕ = the capacity factor (see Table 3.4)

M_{sx} = the nominal section moment capacity, as specified in Clause 5.2, for bending about the x-axis

M_{bx} = the nominal member moment capacity, as specified in Clause 5.3 or 5.6, for bending about the x-axis.

A member bent about the section minor principal y-axis which is analysed by the elastic method (see Clause 4.4) shall satisfy—

$M^*_y \leq \phi M_{sy}$

where

M^*_y = the design bending moment about the y-axis determined in accordance with Clause 4.4.

M_{sy} = the nominal section moment capacity, as specified in Clause 5.2, for bending about the y-axis.

A member which is analysed by the plastic method (see Clause 4.5) shall be compact at all sections where plastic hinges may form (see Clause 5.2.3), shall have full lateral restraint as specified in Clause 5.3.2, and its web shall satisfy Clause 5.10.6. The member shall satisfy—

$M^* \leq \phi M_s$

where

M^* = the design bending moment determined in accordance with Clause 4.5, and

M_s = the nominal section moment capacity as specified in Clause 5.2.1.

A member whose deflections are constrained to a non-principal plane shall be analysed as specified in Clause 5.7.1, and shall satisfy Clause 8.3.4.

A member which is bent about a non-principal axis and whose deflections are unconstrained shall be analysed as specified in Clause 5.7.2, and shall satisfy Clauses 8.3.4 and 8.4.5.

A member subjected to combined bending and shear shall satisfy the requirements of this Clause and Clause 5.12.

A member subjected to combined bending and axial compression or tension shall satisfy Section 8.

5.2 SECTION MOMENT CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

5.2.1 General The nominal section moment capacity (M_s) shall be calculated as follows:

$$M_s = f_y Z_e$$

where the effective section modulus (Z_e) shall be as specified in Clauses 5.2.3, 5.2.4, or 5.2.5 as appropriate.

5.2.2 Section slenderness For a section with flat compression plate elements, the section slenderness (λ_s) shall be taken as the value of the plate element slenderness (λ_e) for the element of the cross-section which has the greatest value of λ_e/λ_{ey} —

where

$$\lambda_e = \left(\frac{b}{t}\right)\sqrt{\left(\frac{f_y}{250}\right)}$$

λ_{ey} = the plate element yield slenderness limit (see Table 5.2)

b = the clear width of the element outstand from the face of the supporting plate element or the clear width of the element between the faces of supporting plate elements

t = the element thickness.

The section plasticity and yield slenderness limits (λ_{sp}) and (λ_{sy}) respectively shall be taken as the values of the element slenderness limits (λ_{ep}) and (λ_{ey}) respectively given in Table 5.2 for the element of the cross-section which has the greatest value of λ_e/λ_{ey}.

For circular hollow sections, the section slenderness (λ_s) shall be calculated as follows:

$$\lambda_s = \left(\frac{d_o}{t}\right)\left(\frac{f_y}{250}\right)$$

where d_o is the outside diameter of the section. The section plasticity and yield slenderness limits (λ_{sp}) and (λ_{sy}) respectively shall be taken as the values of the element slenderness limits (λ_{ep}) and (λ_{ey}) respectively given in Table 5.2.

5.2.3 Compact sections For sections which satisfy $\lambda_s \leq \lambda_{sp}$, the effective section modulus (Z_e) shall be the lesser of S or $1.5Z$, where S and Z are the plastic and elastic section moduli respectively, determined in accordance with Clause 5.2.6.

5.2.4 Non-compact sections For sections which satisfy $\lambda_{sp} < \lambda_s \leq \lambda_{sy}$, the effective section modulus (Z_e) shall be calculated as follows:

$$Z_e = Z + \left[(\lambda_{sy} - \lambda_s)(Z_c - Z)\right]$$

where Z_c is the effective section modulus (Z_e) for a compact section specified in Clause 5.2.3.

TABLE 5.2
VALUES OF PLATE ELEMENT SLENDERNESS LIMITS

Plate element type	Longitudinal edges supported	Residual stresses (see Notes)	Plasticity limit (λ_{ep})	Yield limit (λ_{ey})	Deformation limit (λ_{ed})
Flat (Uniform compression)	One	SR HR LW,CF HW	10 9 8 8	16 16 15 14	35 35 35 35
Flat (Maximum compression at unsupported edge, zero stress or tension at supported edge)	One	SR HR LW,CF HW	10 9 8 8	25 25 22 22	— — — —
Flat (Uniform compression)	Both	SR HR LW,CF HW	30 30 30 30	45 45 40 35	90 90 90 90
Flat (Compression at one edge, tension at the other)	Both	Any	82	115	—
Circular hollow sections		SR HR,CF LW HW	50 50 42 42	120 120 120 120	— — — —

NOTES:

1 SR —stress relieved
HR —hot-rolled or hot-finished
CF —cold formed
LW—lightly welded longitudinally
HW—heavily welded longitudinally

2 Welded members whose compressive residual stresses are less than 40 MPa may be considered to be lightly welded.

5.2.5 Slender sections For sections with flat plate elements in uniform compression which satisfy $\lambda_s > \lambda_{sy}$, the effective section modulus (Z_e) shall be calculated either as follows:

$$Z = z\left(\frac{\lambda_{sy}}{\lambda_s}\right)$$

or for the effective cross-section determined by omitting from each flat compression element the width in excess of the width corresponding to λ_{sy}.

For a section whose slenderness is determined by the value calculated for a flat plate element with maximum compression at an unsupported edge and zero stress or tension at the other edge and which satisfies $\lambda_s > \lambda_{sy}$, the effective section modulus (Z_e) shall be calculated as follows:

$$Z_e = z\left(\frac{\lambda_{sy}}{\lambda_s}\right)^2$$

For circular hollow sections which satisfy $\lambda_s > \lambda_{sy}$, the effective section modulus shall b
taken as the lesser of—

$$Z_e = Z\sqrt{\left(\frac{\lambda_{sy}}{\lambda_s}\right)} \text{ and}$$

$$Z_e = Z\left(\frac{2\lambda_{sy}}{\lambda_s}\right)^2$$

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For elements where $\lambda_e > \lambda_{ed}$ in which λ_{ed} is the deformation slenderness limit given i
Table 5.2, noticeable deformations may occur under service loading.

5.2.6 Elastic and plastic section moduli For sections without holes, or for sections with holes that reduce either of the flange areas by not more than $100\{1 - [f_y/(0.85f_u)]\}$%, the elastic and plastic section moduli may be calculated using the gross section.

For sections with holes that reduce either of the flange areas by more th $100\{1 - [f_y/(0.85f_u)]\}$%, the elastic and plastic section moduli shall be calculated using either—

(a) (A_n/A_g) times the value for the gross section, in which A_n is the sum of the net area of the flanges and the gross area of the web, and A_g the gross area of the section; o

(b) the net section.

When net areas are calculated, any deductions for fastener holes shall be made i accordance with Clause 9.1.10.

5.3 MEMBER CAPACITY OF SEGMENTS WITH FULL LATERAL RESTRAINT

5.3.1 Member capacity The nominal member moment capacity (M_b) of a segment wi full lateral restraint shall be taken as the nominal section moment capacity (M_s) (se Clause 5.2) of the critical section (see Clause 5.3.3).

A segment in a member subjected to bending is the length between adjacent cross-sections which are fully or partially restrained (see Clauses 5.4.2.1 and 5.4.2.2), or the lengt between an unrestrained end (see Clause 5.4.1) and the adjacent cross-section which i fully or partially restrained.

5.3.2 Segments with full lateral restraint

5.3.2.1 *General* A segment may be considered to have full lateral restraint if it satisfie one of the following clauses: Clause 5.3.2.2, 5.3.2.3 or Clause 5.3.2.4, or if its nominal member moment capacity (M_b) calculated in accordance with Clause 5.6 is not less tha the nominal section moment capacity (M_s) (see Clause 5.2) at the critical section (se Clause 5.3.3).

5.3.2.2 *Segments with continuous lateral restraints* A segment with continuous lateral restraints may be considered to have full lateral restraint, provided that—

(a) both ends are fully or partially restrained (see Clauses 5.4.2.1, 5.4.2.2, 5.4.3.1, and 5.4.3.2); and

(b) the continuous restraints act at the critical flange (see Clause 5.5), and satisfy Clause 5.4.3.1.

5.3.2.3 *Segments with intermediate lateral restraints* A segment with intermediate lateral restraints (see Clauses 5.4.2.4 and 5.4.3.1) which divide the segment into a series of sub-segments may be considered to have full lateral restraint, provided that—

(a) both ends are fully or partially restrained (see Clauses 5.4.2.1, 5.4.2.2, 5.4.3.1 and 5.4.3.2); and

(b) the length (L) of each sub-segment satisfies Clause 5.3.2.4; and

(c) the lateral restraints act at the critical flange (see Clause 5.5), and satisfy Clause 5.4.3.1.

5.3.2.4 *Segments with full or partial restraints at both ends* A segment with full or partial restraints at both ends (see Clauses 5.4.2.1, 5.4.2.2, 5.4.3.1 and 5.4.3.2) may be considered to have full lateral restraint, provided its length (L) satisfies—

$$\frac{L}{r_y} \le 80 + 50\beta_m)\sqrt{\left(\frac{250}{f_y}\right)} \quad \text{if the segment is of equal flanged I-section; or}$$

$$\frac{L}{r_y} \le (60 + 40\beta_m)\sqrt{\left(\frac{250}{f_y}\right)} \quad \text{if the segment is an equal flanged channel; or}$$

$$\frac{L}{r_y} \le (80 + 50\beta_m)\left[\sqrt{\left(\frac{2\varrho A d_f}{2.5Z_{ex}}\right)}\right]\sqrt{\left(\frac{250}{f_y}\right)} \quad \text{if the segment is of I-section with}$$

unequal flanges; or

$$\frac{L}{r_y} \le (1800 + 1500\beta_m)\left(\frac{b_f}{b_w}\right)\left(\frac{250}{f_y}\right) \quad \text{if the segment is of rectangular or square}$$

hollow section; or

$$\frac{L}{t} \le (210 + 175\beta_m)\left[\sqrt{\left(\frac{b_2}{b_1}\right)}\right]\left(\frac{250}{f_y}\right) \quad \text{if the segment is of angle section.}$$

where

A	=	area of cross-section
b_f, b_w	=	the flange width and web depth, respectively
b_1, b_2	=	the greater and lesser leg lengths, respectively
d_f	=	the distance between flange centroids
I_{cy}	=	the second moment of area of the compression flange about the section minor y-axis
I_y	=	the second moment of area of the section about the section minor principal y-axis
r_y	=	the radius of gyration about the minor principal y-axis
t	=	the thickness of an angle section
Z_e	=	the effective section modulus (see Clause 5.2)
ϱ	=	I_{cy}/I_y

The ratio β_m shall be taken as one of the following as appropriate:

(a) −1.0; or

(b) −0.8 for segments with transverse loads; or

(c) the ratio of the smaller to the larger end moments in the length (L), (positive when the segment is bent in reverse curvature and negative when bent in single curvature) for segments without transverse loads.

5.3.3 Critical section The critical section in a segment shall be taken as the cross-section which has the largest value of the ratio of the design bending moment (M^*) to the nominal section capacity in bending (M_s) (see Clause 5.2).

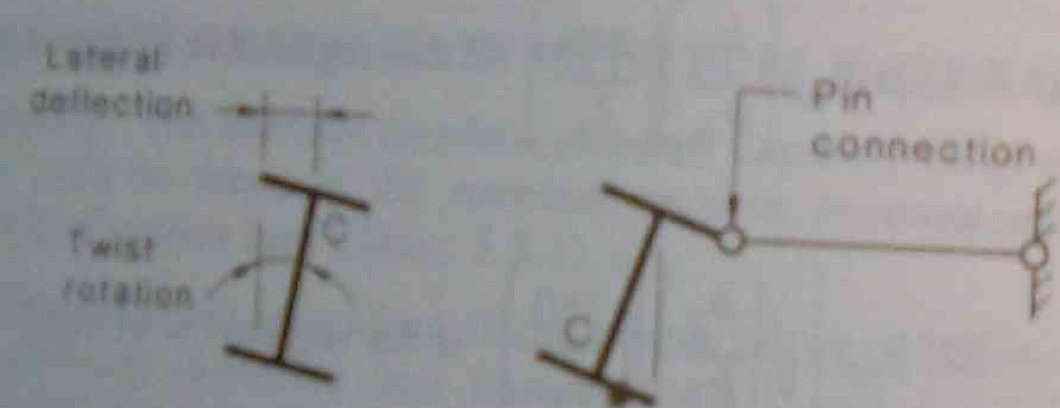
5.4 RESTRAINTS

5.4.1 General A cross-section may be considered to be fully, partially, rotationally or laterally restrained if its restraints satisfy the appropriate requirements of Clause 5.4.2.

Restraints against lateral deflection, twist rotation, or lateral rotation may be considered to be effective if they satisfy the appropriate requirements of Clause 5.4.3.

The members and connections of restraint systems shall be designed to transfer the appropriate forces and bending moments specified in Clause 5.4.3, together with any other forces or bending moments which may act simultaneously, from the points where the forces or bending moments arise to anchorage or reaction points.

Any cross-section of a member which does not satisfy any of Clauses 5.4.2.1 to 5.4.2.4 shall be considered to be unrestrained, as for example in Figure 5.4.1, unless the member capacity in bending is determined by the method of design by buckling analysis (see Clause 5.6.4).



No critical flange restraint, no twist restraint

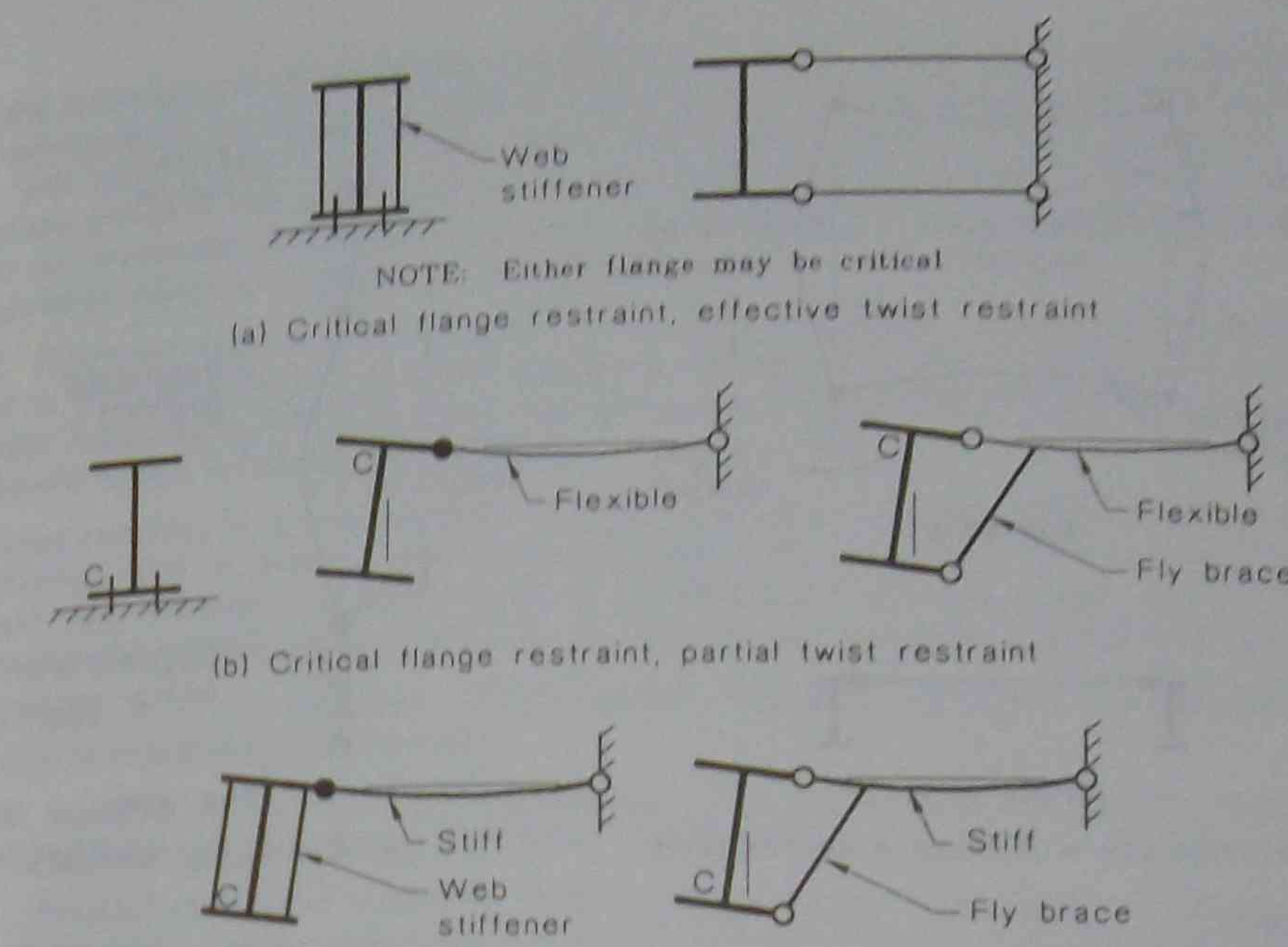
FIGURE 5.4.1 UNRESTRAINED CROSS-SECTION

5.4.2 Restraints at a cross-section

5.4.2.1 *Fully restrained* A cross-section of a member may be considered to be fully restrained if either—

(a) the restraint or support effectively prevents lateral deflection of the critical flange (see Clause 5.5), and effectively prevents twist rotation of the section, as for example in Figure 5.4.2.1(a); or partially prevents twist rotation of the section, as for example in Figure 5.4.2.1(b).

(b) the restraint or support effectively prevents lateral deflection of some other point in the cross-section, and effectively prevents twist rotation of the section, as for example in Figure 5.4.2.1(c).



(a) Critical flange restraint, effective twist restraint

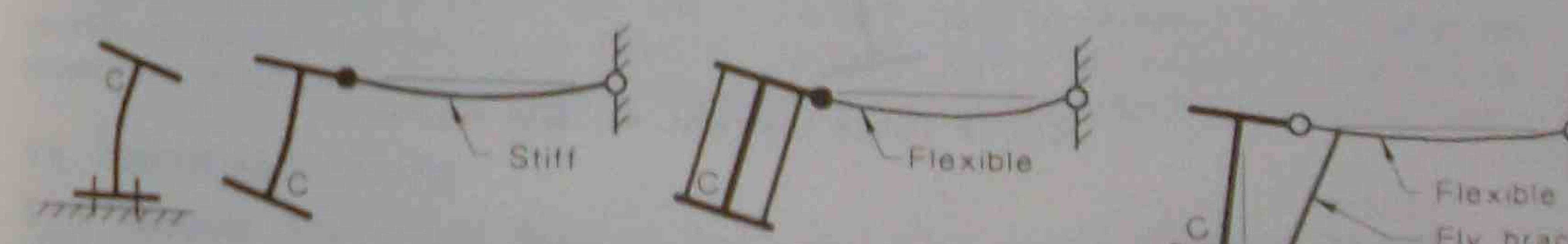
(b) Critical flange restraint, partial twist restraint

(c) Non-critical flange restraint, effective twist restraint

LEGEND:
O = pin connection
● = moment connection
C = critical flange

FIGURE 5.4.2.1 FULLY RESTRAINED CROSS-SECTIONS

5.4.2.2 *Partially restrained* A cross-section of a member may be considered to be partially restrained if the restraint or support effectively prevents lateral deflection of some point in the cross-section other than the critical flange, and partially prevents twist rotation of the section, as for example in Figure 5.4.2.2.



Non-critical flange restraint, partial twist restraint

LEGEND:
O = pin connection
● = moment connection
C = critical flange

FIGURE 5.4.2.2 PARTIALLY RESTRAINED CROSS-SECTIONS

5.4.2.3 *Rotationally restrained* A cross-section of a member which may be considered to be fully or partially restrained may be considered to be rotationally restrained when the restraint or support provides significant restraint against lateral rotation of the critical flange (see Clause 5.5) out of the plane of bending, as for example in Figure 5.4.2.3.

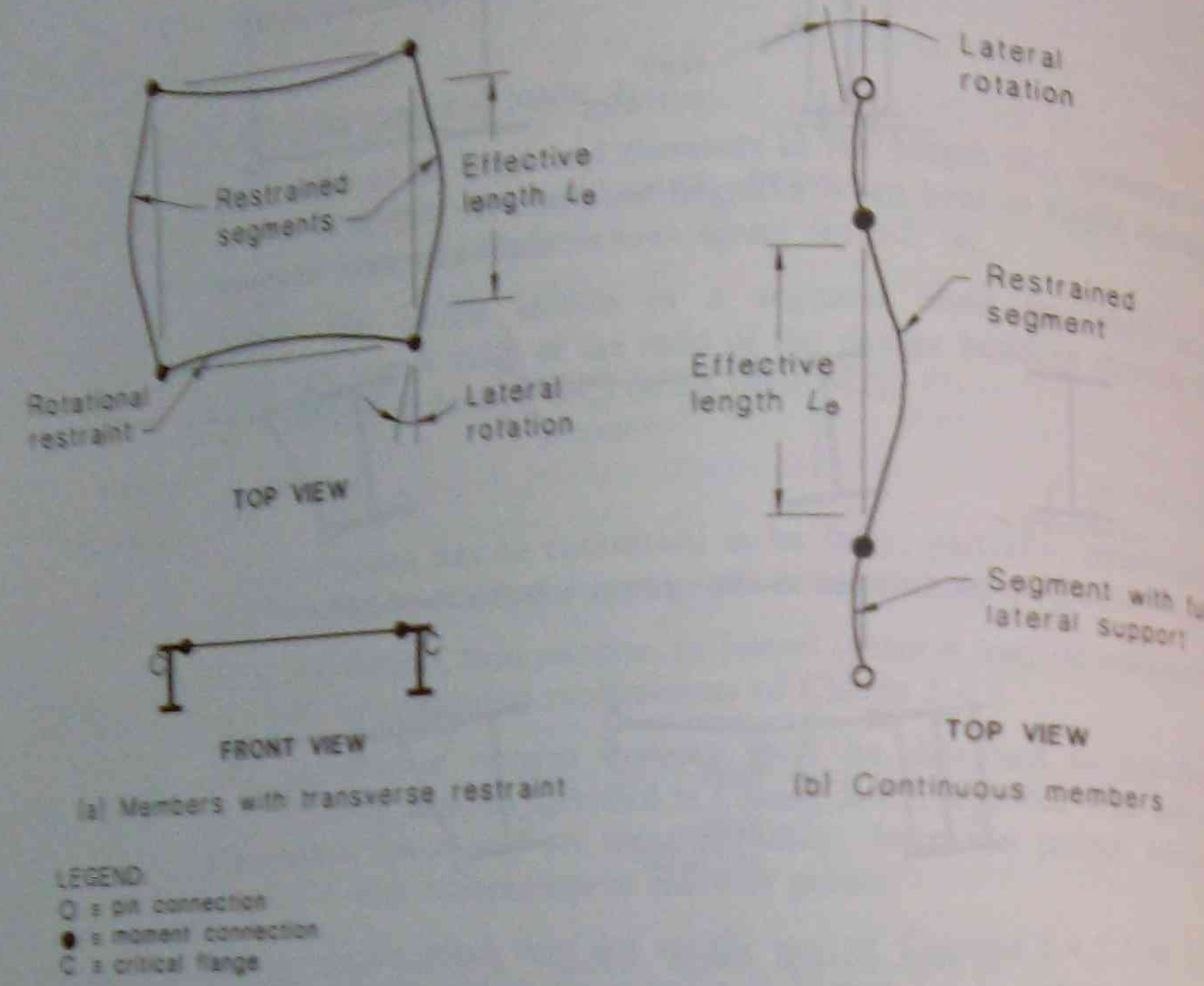


FIGURE 5.4.2.3 ROTATIONALLY RESTRAINED CROSS-SECTIONS

5.4.2.4 *Laterally restrained* A cross-section of a segment whose ends are fully or partially restrained may be considered to be laterally restrained when the restraint effectively prevents lateral deflection of the critical flange (see Clause 5.5) but is ineffective in preventing twist rotation of the section, as for example in Figure 5.4.2.4. Cross-sections in member segments with one end unrestrained shall not be considered to be laterally restrained.

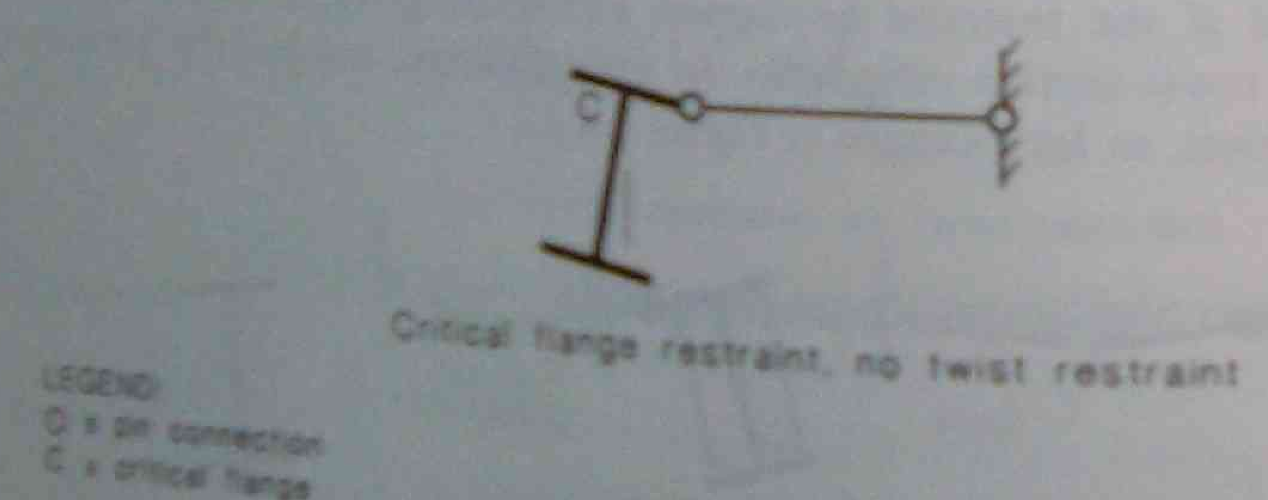


FIGURE 5.4.2.4 LATERALLY RESTRAINED CROSS-SECTION

5.4.3 Restraining elements

5.4.3.1 *Restraint against lateral deflection* The lateral restraint at any cross-section considered to be fully, partially or laterally restrained in terms of Clause 5.4.2 shall be designed to transfer a transverse force acting at the critical flange (see Clause 5.5) equal to 0.025 times the maximum force in the critical flanges of the adjacent segments or sub-segments, except where the restraints are more closely spaced than is required to ensure that $M^* = \phi M_b$.

When the restraints are more closely spaced, then a lesser force may be designed for. The actual arrangement of restraints shall be assumed to be equivalent to a set of restraints which will ensure that $M^* = \phi M_b$. Each equivalent restraint shall correspond to an appropriate group of the actual restraints. This group shall then be designed as a whole to transfer the transverse force of 0.025 times the maximum force in the critical flanges of the equivalent adjacent segments or sub-segments.

5.4.3.2 *Restraint against twist rotation* A torsional restraint at a cross-section may be deemed to provide effective restraint against twist rotation if it is designed to transfer a transverse force equal to 0.025 times the maximum force in the critical flange from any unrestrained flange to the lateral restraint.

A torsional restraint at a cross-section may be deemed to provide partial restraint against twist rotation if it is able to provide an elastic restraint against twist rotation without rotational slip. Flexible elements such as unstiffened webs may form part of such a restraint provided that they are connected in such a way as to prevent rotational slip.

Any restraint at a cross-section which permits rotational slip shall be deemed to be ineffective in restraining twist rotation.

NOTE: Guidance on the effects of the stiffness of a torsional restraint on the resistance to lateral buckling is given in Paragraph H5.1 of Appendix H.

5.4.3.3 *Parallel restrained members* When a series of parallel members is restrained by a line of restraints, each restraining element shall be designed to transfer a transverse force equal to the sum of 0.025 times the flange force from the connected member and 0.0125 times the sum of the flange forces in the connected members beyond, except that no more than seven members need be considered.

5.4.3.4 *Restraint against lateral rotation* A rotational restraint at a cross-section which is considered to be fully or partially restrained (see Clauses 5.4.2.1, 5.4.2.2 and 5.4.2.3) may be deemed to provide restraint against lateral rotation out of the plane of bending, providing its flexural stiffness in the plane of rotation is comparable with the corresponding stiffness of the restrained member.

NOTE: Guidance on the effects of the stiffness of a rotational restraint on the resistance to lateral buckling is given in Paragraph H5.2 of Appendix H.

A segment which has full lateral restraint (see Clause 5.3.2) may be deemed to provide rotational restraint to an adjacent segment which is laterally continuous with it.

A segment which does not have full lateral restraint shall be assumed to be unable to provide rotational restraint to an adjacent segment, unless the member resistance is determined by the method of design by buckling analysis in accordance with Clause 5.6.4.

5.5 CRITICAL FLANGE

5.5.1 General The critical flange at any cross-section is the flange which in the absence of any restraint at that section would deflect the farther during buckling.

The critical flange may be determined by an elastic buckling analysis (see Clause 5.6.4) or as specified in Clauses 5.5.2 and 5.5.3.

5.5.2 Segments with both ends restrained The critical flange at any section of a segment restrained at both ends shall be the compression flange.

5.5.3 Segments with one end unrestrained When gravity loads are dominant, the critical flange of a segment with one end unrestrained shall be the top flange.

When wind loads are dominant, the critical flange shall be the exterior flange in the case of external pressure or internal suction, and shall be the interior flange in the case of internal pressure or external suction.

5.6 MEMBER CAPACITY OF SEGMENTS WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments fully or partially restrained at both ends

5.6.1.1 *Open sections with equal flanges*

(a) *Segments of constant cross-section* The nominal member moment capacity (M_b) shall be calculated as follows:

$$M_b = \alpha_m \alpha_s M_s \leq M_s \qquad \ldots (5.6.1.1(1))$$

where

α_m = a moment modification factor

α_s = a slenderness reduction factor

M_s = the nominal section moment capacity determined in accordance with Clause 5.2 for the gross section.

The moment modification factor (α_m) shall be taken as one of the following—

(i) 1.0; or

(ii) a value obtained from Table 5.6.1; or

(iii) $$\alpha_m = \frac{1.7 M_m^*}{\sqrt{\left[(M_2^*)^2 + (M_3^*)^2 + (M_4^*)^2\right]}} \leq 2.5$$

where

M^*_m = maximum design bending moment in the segment

M^*_2, M^*_4 = design bending moments at the quarter points of the segment

M^*_3 = design bending moment at the midpoint of the segment; or

(iv) a value obtained from an elastic buckling analysis in accordance with Clause 5.6.4.

except that for sub-segments formed by intermediate lateral restraints in segments fully or partially restrained at both ends, the sub-segment moment distribution shall be used instead of the segment moment distribution when using (ii) or (iii).

The slenderness reduction factor (α_s) shall be determined as follows:

$$\alpha_s = 0.6\left\{\sqrt{\left[\left(\frac{M_s}{M_{oa}}\right)^2 + 3\right]} - \left(\frac{M_s}{M_{oa}}\right)\right\} \qquad \ldots (5.6.1.1(2))$$

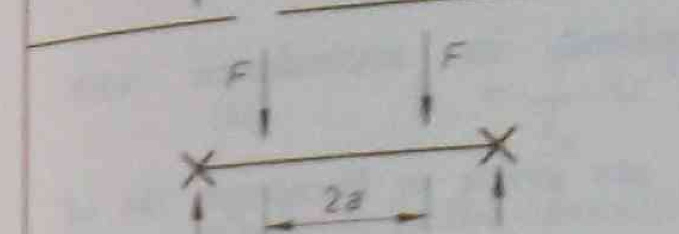
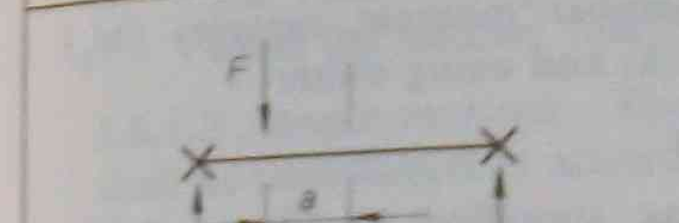
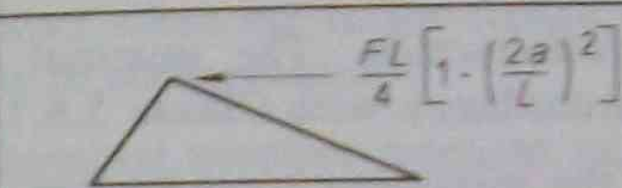
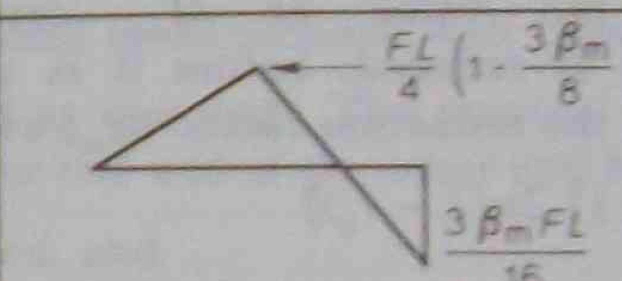
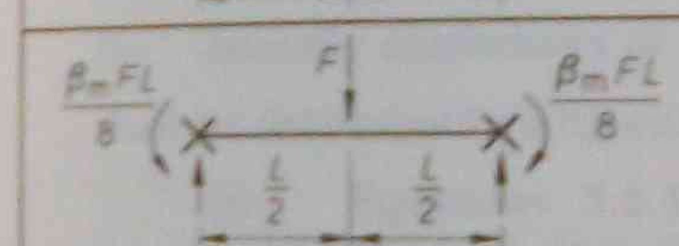
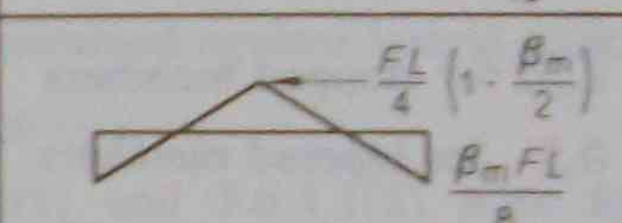
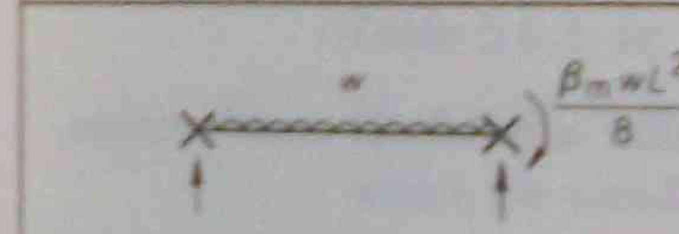
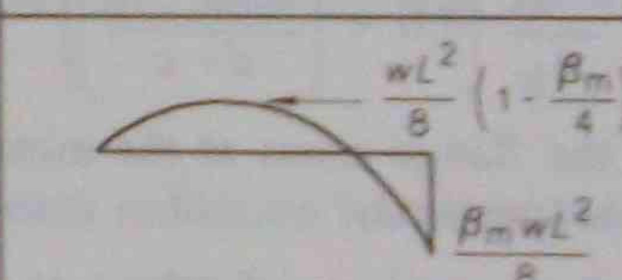
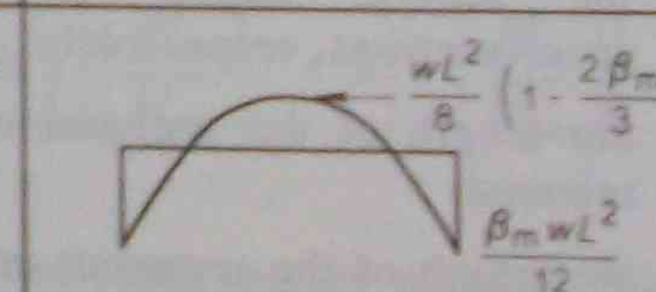
in which M_{oa} shall be taken as either—

(A) $M_{oa} = M_o$, where M_o is the reference buckling moment; or

(B) the value determined from an elastic buckling analysis in accordance with Clause 5.6.4.

TABLE 5.6.1

MOMENT MODIFICATION FACTORS (α_m) FOR SEGMENTS FULLY OR PARTIALLY RESTRAINED AT BOTH ENDS

Beam segment	Moment distribution	Moment modification factor (α_m)	Range
		$1.75 + 1.05\beta_m + 0.3\beta_m^2$ 2.5	$-1 \leq \beta_m \leq 0.6$ $0.6 < \beta_m \leq 1$
		$1.0 + 0.35\left(1 - \frac{2a}{L}\right)^2$	$0 \leq \frac{2a}{L} \leq 1$
		$1.35 + 0.4\left(\frac{2a}{L}\right)^2$	$0 \leq \frac{2a}{L} \leq 1$
		$1.35 + 0.15\beta_m$ $-1.2 + 3.0\beta_m$	$0 \leq \beta_m < 0.9$ $0.9 \leq \beta_m \leq 1$
		$1.35 + 0.36\beta_m$	$0 \leq \beta_m \leq 1$
		$1.13 + 0.10\beta_m$ $-1.25 + 3.5\beta_m$	$0 \leq \beta_m \leq 0.7$ $0.7 \leq \beta_m \leq 1$
		$1.13 + 0.12\beta_m$ $-2.38 + 4.8\beta_m$	$0 \leq \beta_m \leq 0.75$ $0.75 \leq \beta_m \leq 1$
		1.00	
		1.75	
		3.50	

NOTE: × = full or partial restraint

The reference buckling moment (M_o) shall be determined as follows:

$$M_o = \sqrt{\left\{\left(\frac{\pi^2 EI_y}{L_e^2}\right)\left[GJ + \left(\frac{\pi^2 EI_w}{L_e^2}\right)\right]\right\}} \qquad \ldots (5.6.1.1(3))$$

where

E, G = the elastic moduli (see Clause 1.4)

I_y, J, and I_w = section constants (see Clause 1.4)

L_e = the effective length determined in accordance with Clause 5.6.3.

NOTE: Values of E and G and expressions for J and I_w are given in Paragraph H4 of Appendix H.

(b) *Segments of varying cross-section* The nominal member moment capacity (M_b) shall be determined in accordance with Clause 5.6.1.1(a) and using either—

(i) the properties of the minimum cross-section; or

(ii) the properties of the critical cross-section as specified in Clause 5.3.1 provided that the value of M_{oa} determined in accordance with Clause 5.6.1.1(a) is reduced, before it is used in Equation 5.6.1.1(2), by multiplying it by the reduction factor (α_{st}) as follows:

$$\alpha_{st} = 1.0 - [1.2r_r(1 - r_s)]$$

where

r_r = L_r/L for stepped members

= 0.5 for tapered members

r_s = $\dfrac{A_{fm}}{A_{fc}}\left[0.6 + \left(\dfrac{0.4d_m}{d_c}\right)\right]$

A_{fm}, A_{fc} = the flange areas at the minimum and critical cross-sections respectively

d_m, d_c = the section depths at the minimum and critical cross-sections, respectively

L_r = the length of the segment over which the cross-section is reduced

L = the length of the segment; or

(iii) the method of design by buckling analysis (see Clause 5.6.4).

5.6.1.2 *I-sections with unequal flanges* The nominal member moment capacity (M_b) shall be determined in accordance with Clause 5.6.1.1(a), except that the reference buckling moment (M_o) shall be determined by using either—

(a) $M_o = \sqrt{\left(\dfrac{\pi^2 EI_y}{L_e^2}\right)}\left\{\sqrt{\left[(GJ) + \left(\dfrac{\pi^2 EI_w}{L_e^2}\right) + \left(\dfrac{\beta_x^2}{4}\dfrac{\pi^2 EI_y}{L_e^2}\right)\right]} + \dfrac{\beta_x}{2}\sqrt{\left(\dfrac{\pi^2 EI_y}{L_e^2}\right)}\right\}$; or

(b) the method of design by buckling analysis (see Clause 5.6.4).

The monosymmetry section constant (β_x) shall be determined using either—

(i) $\beta_x = 0.8d_f\left[\left(\dfrac{2I_{cy}}{I_y}\right) - 1\right]$

where

d_f = the distance between flange centroids

I_{cy} = the second moment of area of the compression flange about the section minor principal y-axis;

or

(ii) $\beta_x = \dfrac{1}{I_x}\int(x^2y + y^3)dA - 2y_o$

where y_o is the coordinate of the shear centre (see Reference H6.11 of Appendix H). The values of β_x are positive when the larger flange is in compression, and negative when the smaller flange is in compression.

5.6.1.3 *Angle sections* The nominal member moment capacity (M_b) of an angle section shall be determined in accordance with Clause 5.6.1.1(a) using $I_w = 0$.

5.6.1.4 *Hollow sections* The nominal member moment capacity (M_b) of a rectangular hollow section shall be determined in accordance with Clause 5.6.1.1(a) using $I_w = 0$.

5.6.2 Segments unrestrained at one end The nominal member moment capacity (M_b) of a segment unrestrained at one end and at the other end both—

(a) fully or partially restrained; and

(b) laterally continuous or restrained against lateral rotation

shall be determined using either—

(i) Equations 5.6.1.1(1) and 5.6.1.1(2) with M_{oa} equal to the value of M_o obtained from Equation 5.6.1.1(3), and the appropriate value of α_m given in Table 5.6.2; or

(ii) $M_b = \alpha_s M_s \le M_s$

where the slenderness reduction factor (α_s) shall be determined as follows:

$$\alpha_s = 0.6\left\{\sqrt{\left[\left(\frac{M_s}{M_{ob}}\right)^2 + 3\right]} - \left(\frac{M_s}{M_{ob}}\right)\right\}$$

where M_s is the nominal section moment capacity determined in accordance with Clause 5.2 for the gross section, and M_{ob} is determined by an elastic buckling analysis in accordance with Clause 5.6.4.

The reference buckling moment (M_o) shall be determined as follows:

$$M_o = \sqrt{\left\{\left(\frac{\pi^2 EI_y}{L_e^2}\right)\left[GJ + \left(\frac{\pi^2 EI_w}{L_e^2}\right)\right]\right\}} \quad \ldots (5.6.1.1(3))$$

where

E, G = the elastic moduli (see Clause 1.4)

I_y, J and I_w = section constants (see Clause 1.4)

L_e = the effective length determined in accordance w… Clause 5.6.3.

NOTE: Values of E and G and expressions for J and I_w are given in Paragraph … Appendix H.

(b) *Segments of varying cross-section* The nominal member moment capacity … shall be determined in accordance with Clause 5.6.1.1(a) and using either—

(i) the properties of the minimum cross-section; or

(ii) the properties of the critical cross-section as specified in Clause 5… provided that the value of M_{oa} determined in accordance … Clause 5.6.1.1(a) is reduced, before it is used in Equation 5.6.1.1(2), … multiplying it by the reduction factor (α_{st}) as follows:

$$\alpha_{st} = 1.0 - [1.2r_r(1 - r_s)]$$

where

r_s = L_r/L for stepped members

= 0.5 for tapered members

r_r = $\frac{A_{fm}}{A_{fc}}\left[0.6 + \left(\frac{0.4d_m}{d_c}\right)\right]$

A_{fm}, A_{fc} = the flange areas at the minimum and critical cross-sections respectively

d_m, d_c = the section depths at the minimum and critical cross-sections, respectively

L_r = the length of the segment over which the cross-section is reduced

L = the length of the segment; or

(iii) the method of design by buckling analysis (see Clause 5.6.4).

5.6.1.2 *Sections with unequal flanges* The nominal member moment capacity … shall be determined in accordance with Clause 5.6.1.1(a), except that the reference buckling moment (M_o) shall be determined by using either—

(a) $$M_o = \sqrt{\left(\frac{\pi^2 EI_y}{L_e^2}\right)}\left\{\sqrt{\left[GJ + \left(\frac{\pi^2 EI_w}{L_e^2}\right) + \left(\frac{\beta_x^2}{4}\frac{\pi^2 EI_y}{L_e^2}\right)\right]} + \frac{\beta_x}{2}\sqrt{\left(\frac{\pi^2 EI_y}{L_e^2}\right)}\right\}; \text{ or}$$

(b) the method of design by buckling analysis (see Clause 5.6.4).

The monosymmetry section constant (β_x) shall be determined using either—

(i) $$\beta_x = 0.8d_f\left[\left(\frac{2I_{cy}}{I_y}\right) - 1\right]$$

where

d_f = the distance between flange centroids

I_{cy} = the second moment of area of the compression flange about the section minor principal y-axis;

or

(ii) $$\beta_x = \frac{1}{I_x}\int(x^2y + y^3)\mathrm{d}A - 2y_o$$

where y_o is the coordinate of the shear centre (see Reference H6.11 of Appendix H). The values of β_x are positive when the larger flange is in compression, and negative when the smaller flange is in compression.

5.6.1.3 *Angle sections* The nominal member moment capacity (M_b) of an angle section shall be determined in accordance with Clause 5.6.1.1(a) using $I_w = 0$.

5.6.1.4 *Hollow sections* The nominal member moment capacity (M_b) of a rectangular hollow section shall be determined in accordance with Clause 5.6.1.1(a) using $I_w = 0$.

5.6.2 Segments unrestrained at one end The nominal member moment capacity (M_b) of a segment unrestrained at one end and at the other end both—

(a) fully or partially restrained; and

(b) laterally continuous or restrained against lateral rotation

shall be determined using either—

(i) Equations 5.6.1.1(1) and 5.6.1.1(2) with M_{oa} equal to the value of M_o obtained from Equation 5.6.1.1(3), and the appropriate value of α_m given in Table 5.6.2; or

(ii) $M_b = \alpha_s M_s \leq M_s$

where the slenderness reduction factor (α_s) shall be determined as follows:

$$\alpha_s = 0.6\left\{\sqrt{\left[\left(\frac{M_s}{M_{ob}}\right)^2 + 3\right]} - \left(\frac{M_s}{M_{ob}}\right)\right\}$$

where M_s is the nominal section moment capacity determined in accordance with Clause 5.2 for the gross section, and M_{ob} is determined by an elastic buckling analysis in accordance with Clause 5.6.4.

TABLE 5.6.2

MOMENT MODIFICATION FACTORS (α_m) FOR SEGMENTS UNRESTRAINED AT ONE END

Member segment	Moment distribution	Moment modification factor (α_m)
		0.25
		1.25
		2.25

NOTE: X = Full or partial restraint.

5.6.3 Effective length The effective length (L_e) of a segment or sub-segment shall be determined as follows:

$$L_e = k_t k_l k_r L$$

where

k_t = a twist restraint factor given in Table 5.6.3(1)

k_l = a load height factor given in Table 5.6.3(2)

k_r = a lateral rotation restraint factor given in Table 5.6.3(3)

and the length (L) shall be taken as either—

(a) the segment length, for segments without intermediate restraints, or for segments unrestrained at one end, with or without intermediate lateral restraints; or

(b) the sub-segment length, for sub-segments formed by intermediate lateral restraints (see Clauses 5.4.2.4 and 5.4.3.1) in a segment which is fully or partially restrained at both ends.

The lateral rotation restraint factor (k_r) shall only be taken as less than unity when effective rotational restraints, complying with Clause 5.4.3.4, act at one or both ends of a segment which is fully, or partially restrained at both ends. The lateral rotation restraint factor shall be taken as unity for all segments which are unrestrained at one end.

In Tables 5.6.3(1), 5.6.3(2) and 5.6.3(3),

d_1 = clear depth between flanges ignoring fillets or welds

n_w = number of webs

t_f = thickness of critical flange

t_w = thickness of web

F = fully restrained

L = laterally restrained

P = partially restrained

U = unrestrained

and two of the symbols F, L, P, U are used to indicate the conditions at the two ends. (For F, L, P and U restraint requirements, see Clause 5.4.3.)

TABLE 5.6.3(1)

TWIST RESTRAINT FACTORS (k_t)

Restraint arrangement	Factor (k_t)
FF, FL, LL, FU	1.0
FP, PL, PU	$1 + \dfrac{\left[\left(\dfrac{d_1}{L}\right)\left(\dfrac{t_f}{2t_w}\right)^3\right]}{n_w}$
PP	$1 + \dfrac{\left[2\left(\dfrac{d_1}{L}\right)\left(\dfrac{t_f}{2t_w}\right)^3\right]}{n_w}$

TABLE 5.6.3(2)

LOAD HEIGHT FACTORS (k_l) FOR GRAVITY LOADS

Longitudinal position of the load	Restraint arrangement	Load height position	
		Shear centre	Top flange
Within segment	FF,FP,FL,PP,PL,LL FU,PU	1.0 1.0	1.4 2.0
At segment end	FF,FP,FL,PP,PL,LL FU,PU	1.0 1.0	1.0 2.0

TABLE 5.6.3(3)

LATERAL ROTATION RESTRAINT FACTORS (k_r)

Restraint arrangement	Ends with lateral rotation restraints (see Clause 5.4.3.4)	Factor (k_r)
FU, PU	Any	1.0
FF, FP, FL, PP, PL, LL	None	1.0
FF, FP, PP	One	0.85
FF, FP, PP	Both	0.70

5.6.4 Design by buckling analysis When a member is designed by this method, the elastic buckling bending moment (M_{ob}) at the most critical section of the member shall be determined by using the results of an elastic flexural-torsional buckling analysis. This analysis shall take proper account of the member support, restraint, and loading conditions.

The value of M_{oa} to be used in Clause 5.6.1.1(a) shall be taken as follows:

$$M_{oa} = \frac{M_{ob}}{\alpha_m}$$

The moment modification factor (α_m) shall be determined by using either (a) or (b) given below—

(a) Clause 5.6.1.1(a); or

(b) the value given by $\alpha_m = \dfrac{M_{os}}{M_{oo}}$

where

M_{os} = the elastic buckling moment for a segment, fully restrained at both ends, which is unrestrained against lateral rotation and loaded at the shear centre

M_{oo} = the reference elastic buckling moment given by Equation 5.6.1.1(3) with $l_e = l$.

NOTE: Summaries and approximations of the results of elastic buckling analyses are given in Appendix H and in the references given in Paragraph H6.

5.7 BENDING IN A NON-PRINCIPAL PLANE

5.7.1 Deflections constrained to a non-principal plane When the deflection of a member is constrained to a non-principal plane by continuous lateral restraints which prevent lateral deflection, then the forces exerted by the restraints shall be determined, and the principal axis bending moments acting on the member shall be calculated from these forces and the applied forces by a rational analysis.

The calculated principal axis bending moments shall satisfy the requirements of Clause 8.3.4.

5.7.2 Deflections unconstrained When the deflections of a member loaded in a non-principal plane are unconstrained, the principal axis bending moments shall be calculated by a rational analysis.

The calculated principal axis bending moments shall satisfy Clauses 8.3.4 and 8.4.5.

5.8 SEPARATORS AND DIAPHRAGMS If separators or diaphragms are used to connect two or more I-section members or channels placed side by side to act together and in the distribution of external loads between them, the separators and diaphragms shall meet the following requirements:

(a) Separators, made up of spacers and through bolts, shall not be used to transmit forces between the members, other than those due to transverse forces (if any) and a design transverse force (Q^*), taken as not less than 0.025 times the maximum design force occurring in the most heavily loaded compression flange of any member forming the unit. The design transverse force (Q^*) shall be taken as shared equally between the separators.

(b) Diaphragms shall be used where external vertical as well as transverse forces are to be transmitted from one member to another. The diaphragms and their fastenings shall be proportioned to distribute the forces applied to them and in addition, to resist the design transverse force (Q^*) specified above, and resulting shear forces. The design transverse force (Q^*) shall be taken as shared equally between the diaphragms.

5.9 DESIGN OF WEBS

5.9.1 General The geometry and arrangement of beam webs, including any transverse or longitudinal stiffeners, shall satisfy Clause 5.10.

A web subject to shear force shall satisfy Clause 5.11.

A web subject to shear force and bending moment shall satisfy Clause 5.12.

A web subject to bearing load shall satisfy Clause 5.13.

Load-bearing stiffeners and end posts shall satisfy Clause 5.14.

Intermediate transverse stiffeners shall satisfy Clause 5.15.

Longitudinal stiffeners shall satisfy Clause 5.16.

5.9.2 Definition of web panel A web panel of thickness (t_w) shall be considered to extend over an unstiffened area of a web plate with longitudinal dimension (s) and clear transverse dimension (d_p). The web panel may be bounded by flanges, transverse or longitudinal stiffeners, or free edges.

5.9.3 Minimum thickness of web panel Unless a rational analysis would warrant a lesser value, the thickness of a web panel shall satisfy Clauses 5.10.1, 5.10.4, 5.10.5 and 5.10.6.

5.10 ARRANGEMENT OF WEBS

5.10.1 Unstiffened webs The thickness of an unstiffened web bounded on both longitudinal sides by flanges shall not be less than—

$$\left(\frac{d_1}{180}\right)\sqrt{\left(\frac{f_y}{250}\right)}$$

where d_1 is the clear depth of the web between flanges, ignoring fillets or welds.

The thickness of an unstiffened web bounded on one longitudinal side by a free edge shall not be less than—

$$\left(\frac{d_1}{90}\right)\sqrt{\left(\frac{f_y}{250}\right)}$$

where d_1 is the clear depth of the web, ignoring fillets or welds.

5.10.2 Load bearing stiffeners Load bearing stiffeners shall be provided in pairs where the design compressive bearing forces applied through a flange by loads or reactions exceed the design bearing capacity (ϕR_b) of the web alone specified in Clause 5.13.2, or when required to form an end post (Clause 5.15.2.2).

5.10.3 Side reinforcing plates Additional side reinforcing plates may be provided to augment the strength of the web. Proper account shall be taken of any lack of symmetry. The proportion of shear force assumed to be resisted by such plates shall be limited by the amount of horizontal shear which can be transmitted through the fasteners to the web and to the flanges.

5.10.4 Transversely stiffened webs The thickness of a web transversely stiffened but without longitudinal stiffeners shall not be less than—

(a) $\left(\dfrac{d_1}{200}\right)\sqrt{\left(\dfrac{f_y}{250}\right)}$ when $1.0 \leq s/d_1 \leq 3.0$; or

(b) $\left(\frac{s}{200}\right)\sqrt{\left(\frac{f_y}{250}\right)}$ when $0.74 < s/d_1 \le 1.0$; or

(c) $\left(\frac{d_1}{270}\right)\sqrt{\left(\frac{f_y}{250}\right)}$ when $s/d_1 \le 0.74$.

All web lengths for which $s/d_p > 3.0$ shall be considered to be unstiffened, where d_p is the greatest panel depth in the length.

5.10.5 Webs with longitudinal and transverse stiffeners The thickness of a web with a set of longitudinal stiffeners placed on one or both sides of the web at a distance ... from the compression flange shall not be less than—

(a) $\left(\frac{d_1}{250}\right)\sqrt{\left(\frac{f_y}{250}\right)}$ when $1.0 \le s/d_1 \le 2.4$; or

(b) $\left(\frac{s}{250}\right)\sqrt{\left(\frac{f_y}{250}\right)}$ when $0.74 \le s/d_1 \le 1.0$; or

(c) $\left(\frac{d_1}{340}\right)\sqrt{\left(\frac{f_y}{250}\right)}$ when $s/d_1 < 0.74$.

The thickness of a web with an additional set of longitudinal stiffeners placed on one or both sides of the web at the neutral axis shall be not less than—

$\left(\frac{d_1}{400}\right)\sqrt{\left(\frac{f_y}{250}\right)}$ when $s/d_1 \le 1.5$.

5.10.6 Webs of members designed plastically The web thickness of a member assumed to contain a plastic hinge shall not be less than $(d_1/82)\sqrt{(f_y/250)}$.

Load bearing stiffeners shall be provided when a bearing load or shear force acts within $d_1/2$ of a plastic hinge location and the design bearing load or design shear force exceeds 0.1 times the design shear yield capacity (ϕV_w) of the member specified in Clause 5.11.4. These stiffeners shall be located within a distance $d_1/2$ on either side of the hinge location and shall be designed in accordance with Clause 5.14 to carry the greater of the design bearing load or the design shear force considered as a bearing load.

If the stiffeners are flat plates, their slenderness (λ_s) as defined in Clause 5.2.2 using the stiffener yield stress (f_{ys}) shall be less than the plasticity limit (λ_{sp}) specified in Clause 5.2.2.

5.10.7 Openings in webs Except for a castellated member, an opening in a web may be unstiffened provided that the greatest internal dimension of the opening (L_w) satisfies either—

(a) $L_w/d_1 \le 0.10$ for webs without longitudinal stiffeners; or

(b) $L_w/d_1 \le 0.33$ for longitudinally stiffened webs,

provided that the longitudinal distance between boundaries of adjacent openings is at least three times the greatest internal dimension of the opening.

In addition, not more than one unstiffened opening shall be provided at any cross-section unless a rational analysis shows that stiffeners are not necessary.

The design of a castellated member or a member with stiffened openings shall be based on a rational analysis.

5.11 SHEAR CAPACITY OF WEBS

5.11.1 Shear capacity A web subject to a design shear force (V^*) shall satisfy—

$$V^* \le \phi V_v$$

where

ϕ = the capacity factor (see Table 3.4)

V_v = the nominal shear capacity of the web determined from either Clause 5.11.2 or Clause 5.11.3.

5.11.2 Approximately uniform shear stress distribution The nominal shear capacity (V_v) of a web where the shear stress distribution is approximately uniform shall be taken as—

$$V_v = V_u$$

where V_u is the nominal shear capacity of a web with a uniform shear stress distribution given below:

(a) When the maximum web panel depth to thickness ratio d_p/t_w satisfies—

$$\frac{d_p}{t_w} \le \frac{82}{\sqrt{\left(\frac{f_y}{250}\right)}}$$

the nominal shear capacity of the web (V_u) shall be taken as—

$$V_u = V_w$$

where the nominal shear yield capacity of the web (V_w) is specified in Clause 5.11.4.

(b) when the maximum web panel depth to thickness ratio d_p/t_w satisfies—

$$\frac{d_p}{t_w} > \frac{82}{\sqrt{\left(\frac{f_y}{250}\right)}}$$

the nominal shear capacity (V_u) of the web shall be taken as—

$$V_u = V_b$$

where the nominal shear buckling capacity of the web (V_b) is specified in Clause 5.11.5.

5.11.3 Non-uniform shear stress distribution The nominal shear capacity (V_v) of a web with a non-uniform shear stress distribution, such as in a member with unequal flanges, varying web thickness or holes not used for fasteners, shall be calculated as follows:

$$V_v = \frac{2V_u}{0.9 + \left(\frac{f^*_{vm}}{f^*_{va}}\right)} \le V_u$$

where

V_w = the nominal shear capacity of a web with a uniform shear stress distribution determined in accordance with Clause 5.11.2

f^*_{vm}, f^*_{va} = the maximum and average design shear stresses in the web determined by a rational elastic analysis.

For a circular hollow section, V_v shall be taken as the nominal shear yield capacity (V_w) specified in Clause 5.11.4.

5.11.4 Shear yield capacity The nominal shear yield capacity (V_w) of a web shall be calculated as follows:

$$V_w = 0.6 f_y A_w$$

where A_w is the gross sectional area of the web.

The nominal shear yield capacity (V_w) of a circular hollow section shall be calculated as follows:

$$V_w = 0.36 f_y A_e$$

where the effective sectional area (A_e) shall be taken as the gross area of the circular hollow section provided either that there are no holes larger than those required for fasteners, or that the net area is greater than 0.9 times the gross area, or otherwise as the net area.

5.11.5 Shear buckling capacity

5.11.5.1 *Unstiffened web* The nominal shear buckling capacity (V_b) for an unstiffened web or a web considered to be unstiffened (see Clause 5.10.4) shall be calculated as follows:

$$V_b = \alpha_v V_w \le V_w$$

where

$$\alpha_v = \left[\frac{82}{\left(\frac{d_p}{t_w}\sqrt{\frac{f_y}{250}}\right)}\right]^2$$

5.11.5.2 *Stiffened web* The nominal shear buckling capacity (V_b) for a stiffened web with $s/d_p \le 3.0$ shall be calculated as follows:

$$V_b = \alpha_v \alpha_d \alpha_f V_w \le V_w$$

where

$$\alpha_v = \left[\frac{82}{\left(\frac{d_p}{t_w}\sqrt{\frac{f_y}{250}}\right)}\right]^2 \left[\frac{0.75}{\left(\frac{s}{d_p}\right)^2} + 1.0\right] \le 1.0 \text{ when } 1.0 \le s/d_p \le 3.0$$

$$\alpha_v = \left[\frac{82}{\left(\frac{d_p}{t_w}\sqrt{\frac{f_y}{250}}\right)}\right]^2 \left[\frac{1}{\left(\frac{s}{d_p}\right)^2} + 0.75\right] \le 1.0 \text{ when } s/d_p \le 1.0, \text{ and}$$

$$\alpha_d = 1 + \frac{1 - \alpha_v}{1.15\alpha_v\sqrt{\left[1 + \left(\frac{s}{d_p}\right)^2\right]}}; \text{ or}$$

$\alpha_d = 1.0$ when required by Clause 5.15.2.2, and

d_p = the depth of the deepest web panel.

Values of the product $\alpha_v \alpha_d$ are given in Table 5.11.5.2.

The flange restraint factor (α_f) shall be taken as either—

(a) $\alpha_f = 1.0$; or

(b) $$\alpha_f = 1.5 - \frac{0.6}{\sqrt{\left[1 + \left(\frac{40 b_{fo} t_f^2}{d_1^2 t_w}\right)^2\right]}}$$

for webs without longitudinal stiffeners, in which b_{fo} is the least of all of the following:

(i) $\dfrac{12 t_f}{\sqrt{(f_y/250)}}$;

(ii) the distance from the mid-plane of the web to the nearer edge of the flange (taken as zero if there is no flange outstand);

(iii) half the clear distance between the webs if there are two or more webs; or

(c) shall be determined from a rational buckling analysis.

NOTE: Guidance on the shear buckling capacity of a web which contains an axial load is given in Appendix I.

5.12 INTERACTION OF SHEAR AND BENDING

5.12.1 General The nominal web shear capacity (V_{vm}) in the presence of bending moment shall be calculated using the provisions of either Clause 5.12.2 or 5.12.3.

5.12.2 Proportioning method When the bending moment is assumed to be resisted only by the flanges and the design bending moment (M^*) satisfies—

$$M^* \le \phi M_f$$

where M_f is the nominal moment capacity calculated for the flanges alone and determined as follows:

$$M_f = A_{fm} d_f f_y$$

where

A_{fm} = the lesser of the flange effective areas, determined using Clause 6.2.2 for the compression flange and the lesser of A_{fg} and $0.85 A_{fn} f_u / f_y$ for the tension flange

A_{fg} = the gross area of the flange

A_{fn} = the net area of the flange

d_f = the distance between flange centroids,

the member shall satisfy—

$$V^* \le \phi V_{vm}$$

where

$V_{vm} = V_v$

and V_v is the nominal web shear capacity determined either from Clause 5.11.2 or Clause 5.11.3.

TABLE 5.11.5.2

VALUES OF $\alpha_v \alpha_d$

$\left(\frac{d_p}{t_w}\right)\sqrt{\left(\frac{f_y}{250}\right)}$	$\frac{s}{d_p}$ 0.3	0.5	0.75	1.0	1.25	1.5	1.75	2.0	2.5	3.0	[illegible]
90	1.000	1.000	1.000	1.000	1.000	1.000	1.000	0.991	0.952	0.927	[illegible]
100	1.000	1.000	1.000	1.000	0.998	0.946	0.907	0.877	0.833	0.803	[illegible]
110	1.000	1.000	1.000	0.989	0.919	0.866	0.825	0.792	0.744	0.711	[illegible]
120	1.000	1.000	1.000	0.930	0.859	0.805	0.762	0.728	0.677	0.642	[illegible]
130	1.000	1.000	1.000	0.883	0.812	0.757	0.713	0.678	0.625	0.587	[illegible]
140	1.000	1.000	0.960	0.846	0.775	0.719	0.674	0.638	0.583	0.544	[illegible]
150	1.000	1.000	0.926	0.816	0.745	0.689	0.643	0.606	0.550	0.510	[illegible]
160	1.000	1.000	0.898	0.792	0.721	0.664	0.617	0.579	0.522	0.481	[illegible]
170	1.000	1.000	0.875	0.772	0.701	0.643	0.596	0.558	0.499	0.458	[illegible]
180	1.000	0.997	0.855	0.755	0.684	0.626	0.578	0.539	0.480	0.438	[illegible]
190	1.000	0.974	0.839	0.740	0.669	0.611	0.563	0.524	0.464	0.421	
200	1.000	0.955	0.825	0.728	0.657	0.598	0.550	0.511	0.450	0.407	
210	1.000	0.939	0.813	0.718							
220	1.000	0.924	0.803	0.709							
230	1.000	0.912	0.793	0.701							
240	1.000	0.901	0.785	0.694							
250	1.000	0.891	0.778	0.687							
260	1.000	0.883	0.772	0.682							
270	1.000	0.875	0.767	0.677							

5.12.3 Shear and bending interaction method When the bending moment is assumed to be resisted by the whole of the cross-section, the member shall be designed for combined bending and shear, and shall satisfy—

$V^* \le \phi V_{vm}$

where

$V_{vm} = V_v$ for $M^* \le 0.75\phi M_s$; or

$= V_v\left[2.2 - \left(\frac{1.6M^*}{\phi M_s}\right)\right]$ for $0.75\phi M_s \le \phi M_s$

where

V_v = the nominal shear capacity of a web in shear alone (see Clause 5.11.1)

M_s = the nominal section moment capacity determined in accordance with and 5.2

NOTE: Guidance on stiffened web panels required to resist bending moment, shear, axial and transverse loading is given in Appendix I.

5.13 COMPRESSIVE BEARING ACTION ON THE EDGE OF A WEB

5.13.1 Dispersion of force to web Where a force is applied to a flange either as a point load or through a stiff bearing of length (b_s), it shall be considered as dispersed uniformly through the flange at a slope of 1:2.5 to the surface of the flange, as shown in Figure 5.13.1.1, or to the top of the flat portion of the web for rectangular and square hollow sections to AS 1163, as shown in Figure 5.13.1.3. The stiff bearing length is that length which cannot deform appreciably in bending. The dispersion of load to the flange shall be taken at a slope of 1:1 through solid material, as shown in Figure 5.13.1.2.

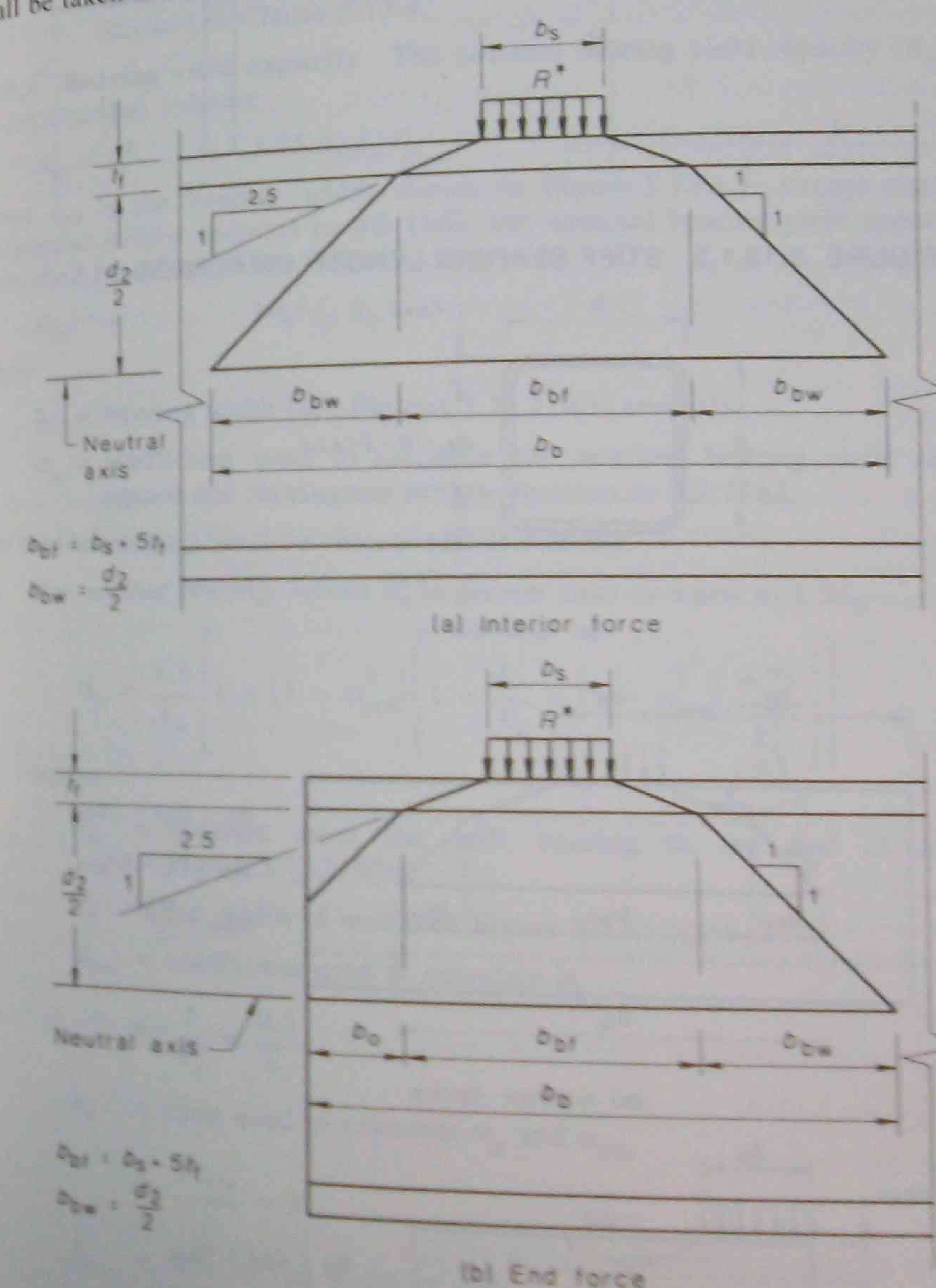


FIGURE 5.13.1.1 DISPERSIONS OF FORCE THROUGH FLANGE AND WEB

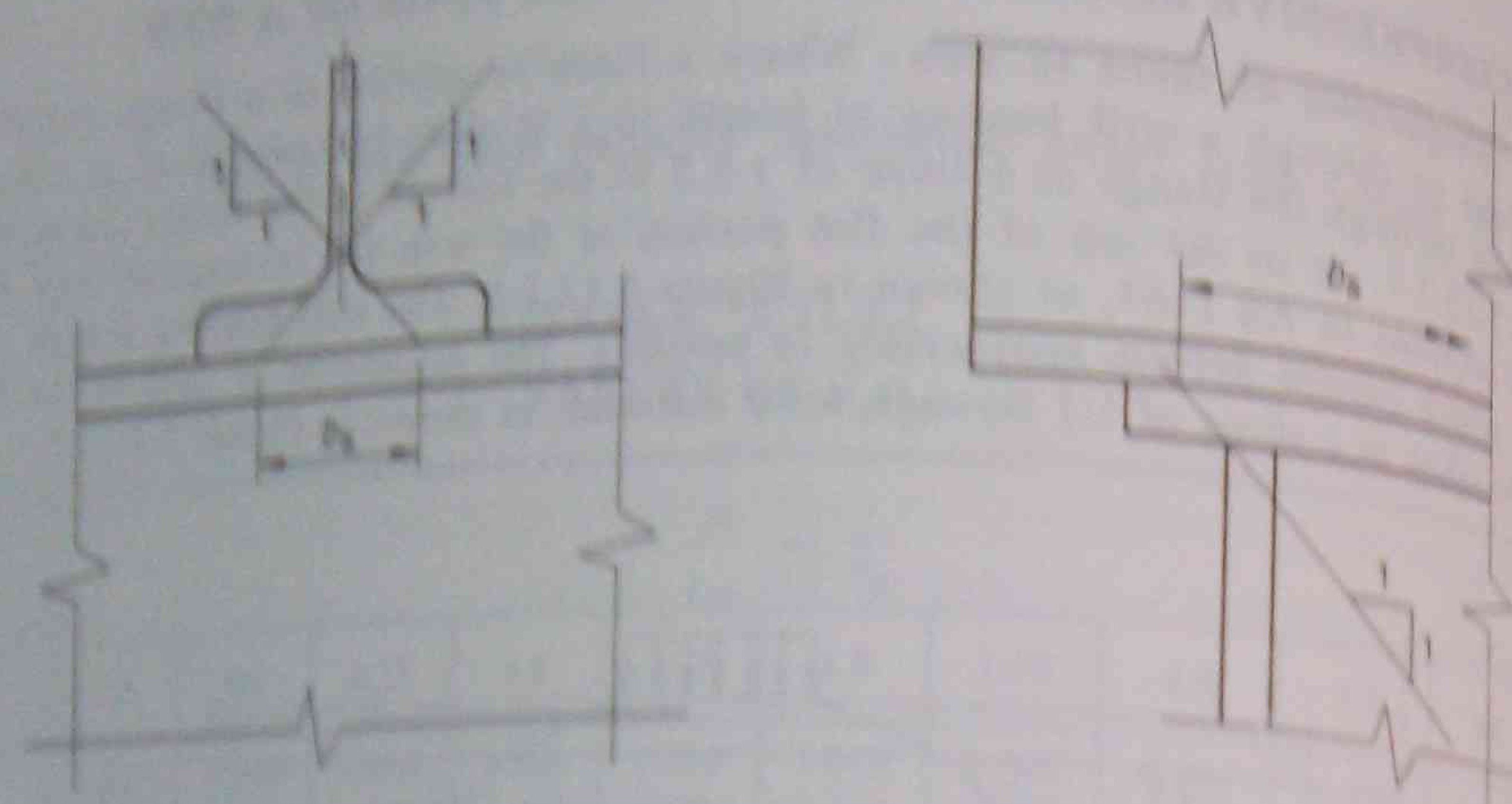


FIGURE 5.13.1.2 STIFF BEARING LENGTH ON FLANGE

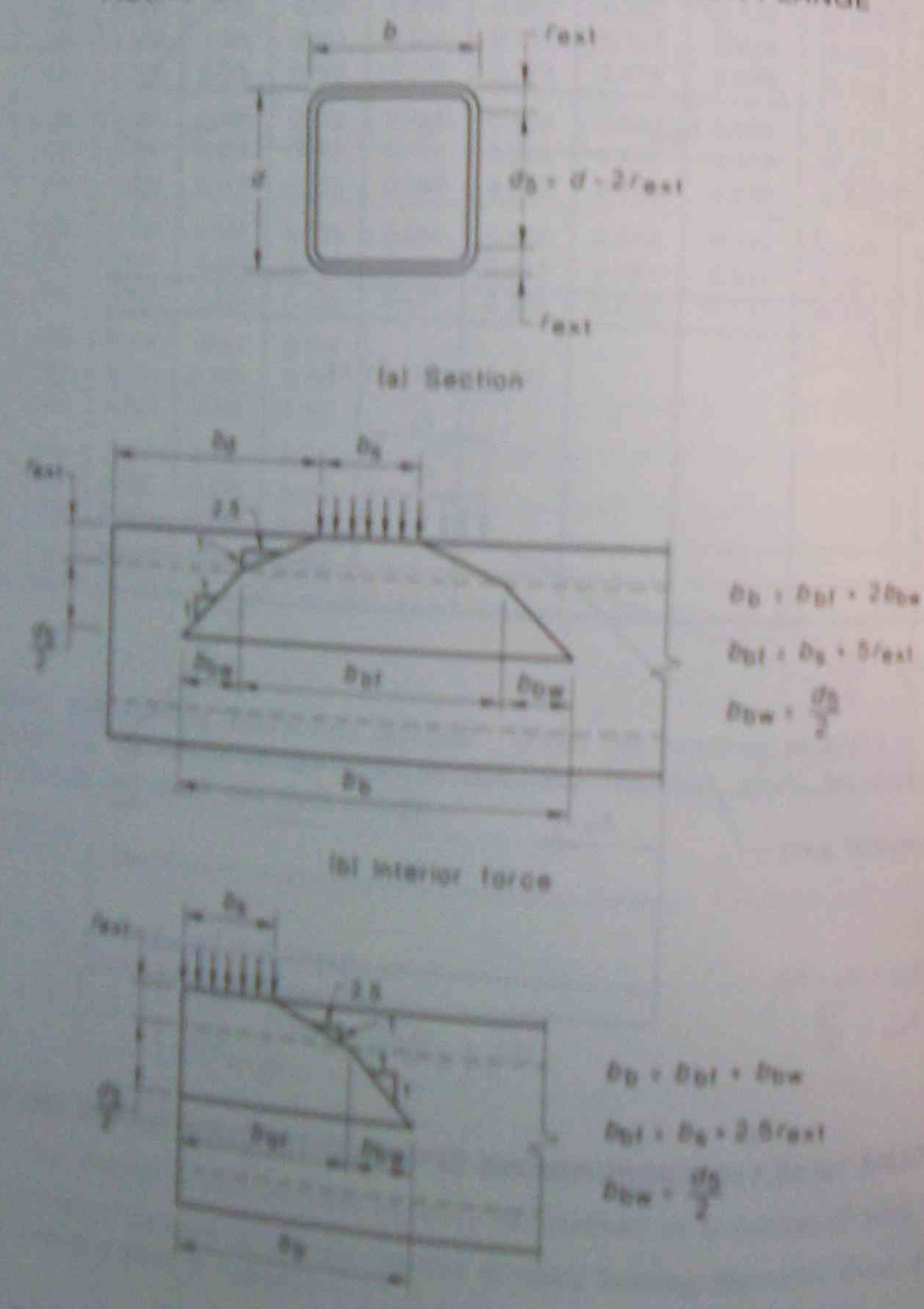


FIGURE 5.13.1.3 RECTANGULAR AND SQUARE HOLLOW SECTIONS — DISPERSION OF FORCE THROUGH FLANGE, RADIUS AND WEB

5.13.2 Bearing capacity The design bearing force (R^*) on a web shall satisfy—

$$R^* \leq \phi R_b$$

where

ϕ = the capacity factor (see Table 3.4)

R_b = the nominal bearing capacity of the web under concentrated or patch loading, which shall be taken as the lesser of its nominal bearing yield capacity (R_{by}) defined in Clause 5.13.3, and its nominal bearing buckling capacity (R_{bb}) defined in Clause 5.13.4.

5.13.3 Bearing yield capacity The nominal bearing yield capacity (R_{by}) of a web shall be calculated as follows:

$$R_{by} = 1.25\, b_{bf} t_w f_y$$

where b_{bf} is the bearing width shown in Figure 5.13.1.1, except that for square and rectangular hollow sections to AS 1163, the nominal bearing yield capacity (R_{by}) of both webs shall be calculated as follows:

$$R_{by} = 2 b_b t f_y \alpha_p$$

where

b_b = bearing width (see Figures 5.13.1.3(b) and (c))

α_p = coefficient used to calculate the nominal bearing yield capacity (R_{by}) for square and rectangular hollow sections to AS 1163.

The coefficient (α_p) shall be determined as follows:

(a) For interior bearing, where b_d is greater than or equal to $1.5d_5$—

$$\alpha_p = \frac{0.5}{k_s}\left[1 + \left(1 - \alpha_{pm}^2\right)\left(1 + \frac{k_s}{k_v} - \left(1 - \alpha_{pm}^2\right)\frac{0.25}{k_v^2}\right)\right]$$

where

b_d = distance from the stiff bearing to the end of the member (see Figure 5.13.1.3(b))

d_5 = flat width of web (see Figure 5.13.1.3(a))

α_{pm} = coefficient used to calculate α_p

$$= \frac{1}{k_s} + \frac{0.5}{k_v}$$

k_s = ratio used to calculate α_p and α_{pm}

$$= \frac{2r_{ext}}{t} - 1$$

k_v = ratio of flat width of web (d_5) to thickness (t) of section

$$= \frac{d_5}{t}$$

r_{ext} = outside radius of section (see Figure 5.13.1.3(a)).

The bearing width (b_b) shall be calculated as follows:

$b_b = b_s + 5r_{ext} + d_5$

(b) For end bearing, where b_d is less than $1.5d_5$—

$$\alpha_p = \sqrt{2 + k_s^2} - k_s$$

The bearing width (b_b) shall be calculated as follows:

$$b_b = b_s + 2.5r_{ext} + \frac{d_5}{2}$$

NOTE: Guidance on the nominal yield capacity of a stiffened web in bearing in the apresence of bending moment and axial load is given in Appendix I.

5.13.4 Bearing buckling capacity The nominal bearing buckling capacity (R_{bb}) of a web without transverse stiffeners shall be taken as the axial load capacity determinted in accordance with Section 6 using $\alpha_b = 0.5$ and $k_f = 1.0$ for a compression member of area $t_w b_b$ and slenderness ratio $Le/r = 2.5d_1/t_w$ where b_b is the total bearing width obtained by dispersions at a slope of 1:1 from b_{bf} to the neutral axis, if available, as shown in Figure 5.13.1.1, except that for square and rectangular hollow sections to AS 1163, the slenderness ratio $L_e/r = 3.5d_5/t_w$ for interior bearing ($b_d \geq 1.5d_5$) and $L_e/r = 3.8d_5/t_w$ for end bearing ($b_d < 1.5d_5$ and b_b is as shown in Figure 5.13.1.3.

NOTE: Guidance on the nominal bearing buckling capacity (R_{bb}) of a stiffened web with a bearing load between the stiffeners in the presence of bending moment and axial load is given in Appendix I.

5.13.5 Combined bending and bearing of rectangular and square hollow sections Rectangular and square hollow sections to AS 1163 subjected to combined bending and bearing force shall satisfy Clauses 5.2, 5.13.2, and either—

$$1.2\left(\frac{R^*}{\phi R_b}\right) + \left(\frac{M^*}{\phi M_s}\right) \leq 1.5 \text{ for } \frac{b_s}{b} \geq 1.0 \text{ and } \frac{d_1}{t_w} \leq 30;$$

or

$$0.8\left(\frac{R^*}{\phi R_b}\right) + \left(\frac{M^*}{\phi M_s}\right) \leq 1.0 \text{ otherwise}$$

where

ϕ = capacity factor (see Table 3.4)

R_b = nominal bearing capacity of a web specified in Clause 54.13.2

M_s = nominal section moment capacity determined in accordance with Clause 5.2

b_s = stiff bearing length

b = total width of section

5.14 DESIGN OF LOAD BEARING STIFFENERS

5.14.1 Yield capacity When a load bearing stiffener is required, it shall satisfy—

$$R^* \leq \phi R_{sy}$$

where

R^* = the design bearing force or design reaction, including the effects of any shear forces applied directly to the stiffener

ϕ = the capacity factor (see Table 3.4)

R_{sy} = the nominal yield capacity of the stiffened web

$= R_{by} + A_s f_{ys}$

R_{by} = the nominal bearing yield capacity (see Clause 5.13.3)

A_s = the area of the stiffener in contact with the flange

f_{ys} = the yield stress of the stiffener.

5.14.2 Buckling capacity When a load bearing stiffener is required, it shall satisfy—

$$R^* \leq \phi R_{sb}$$

where

ϕ = the capacity factor (see Table 3.4)

R_{sb} = the nominal buckling capacity of the stiffened web, determined in accordance with Section 6 using $\alpha_b = 0.5$ and $k_f = 1.0$ for a compression member whose radius of gyration is taken about the axis parallel to the web.

The effective section of the compression member shall be taken as the area of the stiffener, together with a length of web on each side of the centreline not greater than the lesser of—

$$\frac{17.5t_w}{\sqrt{\left(\frac{f_y}{250}\right)}} \text{ and } \frac{s}{2}, \text{ if available.}$$

The effective length (L_e) of the compression member used in calculating the buckling capacity (R_{sb}) shall be determined as either—

$$L_e = 0.7d_1$$

where the flanges are restrained by other structural elements against rotation in the plane of the stiffener, or—

$$L_e = d_1$$

if either of the flanges is not so restrained.

5.14.3 Outstand of stiffeners Unless the outer edge of a flat stiffener is continuously stiffened, the stiffener outstand from the face of a web (b_{es}) shall satisfy—

$$b_{es} \leq \frac{15t_s}{\sqrt{\left(\frac{f_{ys}}{250}\right)}}$$

where

t_s = the thickness of the stiffener

f_{ys} = the yield stress of the stiffener used in design.

5.14.4 Fitting of load bearing stiffeners A load bearing stiffener shall be fitted to provide a tight and uniform bearing against the loaded flange, unless welds are provided between the flange and stiffener for the purpose of transmitting the concentrated force or reaction. Where a point of concentrated force is directly over a support, this provision shall apply to both flanges.

Load bearing stiffeners shall be provided with sufficient welds or bolts to transmit their share of the design bearing force or design reaction (R^*) to the web.

5.14.5 Design for torsional end restraint When load bearing stiffeners are the sole means of providing torsional end restraint at the supports of a member, the second moment of area of a pair of stiffeners (I_s) about the centreline of the web shall be such that—

$$I_s \geq \frac{\alpha_t}{1000}\frac{d^3 t_f R^*}{F^*}$$

where

α_t $= \dfrac{230}{(L_e/r_y)} - 0.60$ and $0 \leq \alpha_t \leq 4$

R^* = the design reaction at the bearing

F^* = the total design load on the member between supports

t_f = the thickness of the critical flange (see Clause 5.5)

(L_e/r_y) = the load bearing stiffener slenderness ratio used in Clause 5.14.2.

5.15 DESIGN OF INTERMEDIATE TRANSVERSE WEB STIFFENERS

5.15.1 General Intermediate transverse web stiffeners shall extend between each flange and shall terminate no further from a flange than four times the web thickness.

NOTE: Intermediate stiffeners may be provided on one or both sides of a web.

5.15.2 Spacing

5.15.2.1 *Interior panels* The spacing (s) of intermediate web stiffeners which define internal panels shall satisfy Clause 5.10.4 or Clause 5.10.5.

5.15.2.2 *End panels* An end panel shall be provided with an end post which satisfies Clause 5.15.9, unless the width (s) of the end panel is reduced so that its shear buckling capacity (V_b) calculated by using $\alpha_d = 1.0$ in Clause 5.11.5.2 satisfies Clauses 5.11.1 and 5.12.

5.15.3 Minimum area An intermediate web stiffener not subject to external loads or moments shall have an area A_s which satisfies—

$$A_s \geq 0.5\gamma A_w (1 - \alpha_v)\left(\frac{V^*}{\phi V_v}\right)\left\{\left(\frac{s}{d_p}\right) - \frac{\left(\frac{s}{d_p}\right)^2}{\sqrt{\left[1 + \left(\frac{s}{d_p}\right)^2\right]}}\right\}$$

where

α_v = the value determined in accordance with Clause 5.11.5.2

γ = 1.0 for a pair of stiffeners

= 1.8 for a single angle stiffener

= 2.4 for a single plate stiffener.

5.15.4 Buckling capacity An intermediate web stiffener shall satisfy—

$$V^* \leq \phi(R_{sb} + V_b)$$

where

ϕ = capacity factor (see Table 3.4)

V_b = nominal shear buckling capacity specified in Clause 5.11.5.2 for a stiffened web using $\alpha_d = 1.0$ and $\alpha_f = 1.0$

R_{sb} = nominal buckling capacity of the intermediate stiffener determined in accordance with Clause 5.14.2.

The effective length (L_e) of the compression member used in calculating R_{sb} shall be taken as—

$$L_e = d_1$$

5.15.5 Minimum stiffness An intermediate web stiffener not subject to external loads or moments shall have a minimum second moment of area (I_s) about the centreline of the web such that—

$$I_s \geq 0.75 d_1 t_w^3 \text{ for } \frac{s}{d^1} \leq \sqrt{2}\text{; and}$$

$$I_s \geq \frac{1.5 d_1^3 t_w^3}{s^2} \text{ for } \frac{s}{d^1} > \sqrt{2}$$

5.15.6 Outstand of stiffeners The outstand (b_{es}) of an intermediate web stiffener shall satisfy Clause 5.14.3.

5.15.7 External forces

5.15.7.1 *Increase in stiffness* Where an intermediate stiffener is used to transfer design forces (F_n^*) normal to the web or design moments ($M^* + F_p^* e$) acting normal to the web (including moments $F_p^* e$ caused by any eccentric force F_p^* parallel to the web), the minimum value of I_s in Clause 5.15.5 shall be increased by—

$$\frac{d_1^4 \{2F_n^* + [(M^* + F_p^* e)/d_1]\}}{\phi E d_1 t_w}$$

5.15.7.2 *Increase in strength* When an intermediate stiffener is required to carry a transverse load parallel to the web, it shall be designed as a load bearing stiffener in accordance with Clause 5.14.

5.15.8 Connection of intermediate stiffeners to web The web connections of intermediate transverse stiffeners not subject to external loading shall be designed to resist a design shear force per unit length, in kilonewtons per millimetre (kN/mm), of not less than—

$$\frac{0.0008(t_w)^2 f_y}{b_{es}}$$

where b_{es} is the outstand width of the stiffener from the face of the web, in millimetres, and t_w is the web thickness, in millimetres.

5.15.9 End posts When an end post is required by Clause 5.15.2.2, it shall be formed by a load bearing stiffener and a parallel end plate. The load bearing stiffener shall be designed in accordance with Clause 5.14, and shall be no smaller than the end plate. The area of the end plate (A_{ep}) shall satisfy—

$$A_{ep} \geq \frac{d_1[(V^*/\phi) - \alpha_v V_w]}{8 e f_y}$$

where

α_v is given in Clause 5.11.5.2

V_w is given in Clause 5.11.4

e = the distance between the end plate and load bearing stiffener.

5.16 DESIGN OF LONGITUDINAL WEB STIFFENERS

5.16.1 General Longitudinal web stiffeners shall be continuous or shall extend between and be attached to transverse web stiffeners.

5.16.2 Minimum stiffness When a longitudinal stiffener is required at a distance $0.2d_1$ from the compression flange, it shall have a second moment of area (I_s) about the face of the web such that—

$$I_s \geq 4 d_2 t_w^3 \left[1 + \frac{4A_s}{d_2 t_w}\left(1 + \frac{A_s}{d_2 t_w}\right)\right]$$

where A_s is the area of the stiffener.

When a second longitudinal stiffener is required at the neutral axis of the section, it shall have a second moment of area (I_s) about the face of the web such that—

$$I_s \geq d_2 t_w^3$$

SECTION 6 MEMBERS SUBJECT TO AXIAL COMPRESSION

6.1 DESIGN FOR AXIAL COMPRESSION A concentrically loaded member subject to a design axial compression force (N^*) shall satisfy both—

$N^* \leq \phi N_s$, and

$N^* \leq \phi N_c$

where

ϕ = the capacity factor (see Table 3.4)

N_s = the nominal section capacity determined in accordance with Clause 6.2

N_c = the nominal member capacity determined in accordance with Clause 6.3.

6.2 NOMINAL SECTION CAPACITY

6.2.1 General The nominal section capacity (N_s) of a concentrically loaded compression member shall be calculated as follows:

$$N_s = k_f A_n f_y$$

where

k_f = the form factor given in Clause 6.2.2.

A_n = the net area of the cross-section, except that for sections with penetrations or unfilled holes that reduce the section area by less than $100\{1 - [f_y/(0.85 f_u)]\}$%, the gross area may be used. Deductions for fastener holes shall be made in accordance with Clause 9.1.10.

6.2.2 Form factor The form factor (k_f) shall be calculated as follows:

$$k_f = \frac{A_e}{A_g}$$

where

A_e = the effective area

A_g = the gross area of the section.

The effective area (A_e) shall be calculated from the gross area by summing the effective areas of the individual elements, whose effective widths are specified in Clause 6.2.4.

6.2.3 Plate element slenderness The slenderness (λe) of a flat plate element shall be calculated as follows:

$$\lambda_e = \frac{b}{t}\sqrt{\left(\frac{f_y}{250}\right)}$$

where

b = the clear width of the element outstand from the face of the supporting plate element, or the clear width of the element between the faces of the supporting plate elements.

t = the thickness of the plate.

For circular hollow sections, the element slenderness (λ_e) shall be calculated as follows:

$$\lambda_e = \left(\frac{d_o}{t}\right)\left(\frac{f_y}{250}\right)$$

where

d_o = the outside diameter of the section

t = the wall thickness of the section.

6.2.4 Effective width The effective width (b_e) of a flat plate element of clear width (b), or the effective outside diameter (d_e) of a circular hollow section of outside diameter (d_o), shall be calculated from the value of the element slenderness (λ_e) given in Clause 6.2.3 and the element yield slenderness limit (λ_{ey}) given in Table 6.2.4.

The effective width (b_e) for a flat plate element shall be calculated as follows:

$$b_e = b\left(\frac{\lambda_{ey}}{\lambda_e}\right) \le b$$

The effective outside diameter (d_e) for a circular hollow section shall be the lesser of—

$$d_e = d_o\sqrt{\left(\frac{\lambda_{ey}}{\lambda_e}\right)} \le d_o, \text{ and}$$

$$d_e = d_o\left(\frac{3\lambda_{ey}}{\lambda_e}\right)^2$$

Alternatively, the effective width (b_e) for a flat plate element may be obtained from the following:

$$b_e = b\left(\frac{\lambda_{ey}}{\lambda_e}\right)\sqrt{\left(\frac{k_b}{k_{bo}}\right)} \le b$$

where k_b is the elastic buckling coefficient for the element.

For a flat plate element supported along both longitudinal edges—

k_{bo} = 4.0

and for a flat plate element supported along one longitudinal edge (outstand)—

k_{bo} = 0.425

The elastic buckling coefficient (k_b) for the flat plate element shall be determined from a rational elastic buckling analysis of the whole member as a flat plate assemblage.

TABLE 6.2.4

VALUES OF PLATE ELEMENT YIELD SLENDERNESS LIMIT

Plate element type	Longitudinal edges supported	Residual stresses (see Notes)	Yield slenderness limit (λ_{ey})
Flat	One (Outstand)	SR	16
		HR	16
		LW, CF	15
		HW	14
	Both	SR	45
		HR	45
		LW, CF	40
		HW	35
Circular hollow sections		SR	82
		HR, CF	82
		LW	82
		HW	82

NOTES:

1 SR —stress relieved
HR —hot-rolled or hot-finished
CF —cold-formed
LW —lightly welded longitudinally
HW —heavily welded longitudinally

2 Welded members whose compressive residual stresses are less than 40 MPa may be considered to be lightly welded.

6.3 NOMINAL MEMBER CAPACITY

6.3.1 Definitions For the purpose of this Clause, the definitions below apply.

Geometrical slenderness ratio—the geometrical slenderness ratio (L_e/r), taken as the effective length (L_e), specified in Clause 6.3.2, divided by the radius of gyration (r) computed for the gross section about the relevant axis.

Length—the actual length (L) of an axially loaded member, taken as the length centre-to-centre of intersections with supporting members, or the cantilevered length in the case of free-standing members.

6.3.2 Effective length The effective length (L_e) of a compression member shall be determined as follows:

$$L_e = k_e L$$

where k_e is the member effective length factor determined in accordance with Clause 4.6.3.

6.3.3 Nominal capacity of a member of constant cross-section The nominal member capacity (N_c) of a member of constant cross-section shall be determined as follows:

$$N_c = \alpha_c N_s \le N_s$$

where

N_s = the nominal section capacity, determined in accordance with Clause 6.2

α_c = the member slenderness reduction factor

$$= \xi\left\{1 - \sqrt{\left[1 - \left(\frac{90}{\xi\lambda}\right)^2\right]}\right\}$$

$$\xi = \frac{\left(\frac{\lambda}{90}\right)^2 + 1 + \eta}{2\left(\frac{\lambda}{90}\right)^2}$$

$\lambda = \lambda_n + \alpha_a \alpha_b$

$\eta = 0.00326(\lambda - 13.5) \geq 0$

$$\lambda_n = \left(\frac{L_e}{r}\right)\sqrt{(k_f)}\sqrt{\left(\frac{f_y}{250}\right)}$$

$$\alpha_a = \frac{2100(\lambda_n - 13.5)}{\lambda_n^2 - 15.3\lambda_n + 2050}$$

α_b = the appropriate member section constant given in Table 6.3.3(1) or 6.3.3(2)

k_f = the form factor determined in accordance with Clause 6.2.2.

Alternatively, values of the member slenderness reduction factor (α_c) may be obtained directly from Table 6.3.3(3) using the value of the modified member slenderness (λ_n) and the appropriate member section constant (α_b) given in Table 6.3.3(1) or 6.3.3(2).

6.3.4 Nominal capacity of a member of varying cross-section The nominal member capacity (N_c) of a member of varying cross-section shall be determined using the provisions of Clause 6.3.3 provided that the following are satisfied—

(a) The nominal section capacity (N_s) is the minimum value for all cross-sections along the length of the member; and

(b) The modified member slenderness (λ_n) given in Clause 6.3.3 is replaced by the following:

$$\lambda_n = 90\sqrt{\left(\frac{N_s}{N_{om}}\right)}$$

where N_{om} is the elastic flexural buckling load of the member in axial compression determined using a rational elastic buckling analysis.

6.4 LACED AND BATTENED COMPRESSION MEMBERS

6.4.1 Design forces If a compression member composed of two or more main components which are parallel is intended to act as a single member, the main components and their connections shall be proportioned to resist a design transverse shear force (V^*) applied at any point along the length of the member in the most unfavourable direction. The design transverse shear force (V^*) shall be calculated as follows:

$$V^* = \frac{\pi\left(\frac{N_s}{N_c} - 1\right)N^*}{\lambda_n} \geq 0.01N^*$$

where

N_s = the nominal section capacity of the compression member given by Clause 6.2

N_c = the nominal member capacity of the compression member given by Clause 6.3

N^* = the design axial force applied to the compression member

λ_n = the modified member slenderness.

The modified member slenderness (λ_n) of a battened compression member shall be determined using Clauses 6.4.3.2 and 6.3.3.

TABLE 6.3.3(1)
VALUES OF MEMBER SECTION CONSTANT (α_b) FOR k_f = 1.0

Compression member section constant (α_b)	Section description
-1.0	— Hot-formed RHS and CHS — Cold-formed (stress relieved) RHS and CHS
-0.5	— Cold-formed (non-stress relieved) RHS and CHS
0	— Hot-rolled UB and UC sections (flange thickness up to 40 mm) — Welded H and I sections fabricated from flame-cut plates — Welded box sections
0.5	— Tees flame-cut from universal sections, and angles — Hot-rolled channels — Welded H and I sections fabricated from as-rolled plates (flange thickness up to 40 mm) — Other sections not listed in this Table
1.0	— Hot-rolled UB and UC sections (flange thickness over 40 mm) — Welded H and I sections fabricated from as-rolled plates (flange thickness over 40 mm)

TABLE 6.3.3(2)
VALUES OF MEMBER SECTION CONSTANT (α_b) FOR k_f < 1.0

Compression member section constant (α_b)	Section description
-0.5	— Hot-formed RHS and CHS — Cold-formed RHS and CHS (stress relieved) — Cold-formed RHS and CHS (non-stress relieved)
0	— Hot-rolled UB and UC sections (flange thickness up to 40mm) — Welded box sections
0.5	— Welded H and I sections (flange thickness up to 40 mm)
1.0	— Other sections not listed in this Table

TABLE 6.3.3(3)

VALUES OF MEMBER SLENDERNESS REDUCTION FACTOR (α_c)

Modified member slenderness (λ_n)	Compression member section constant (α_b)				
	−1.0	−0.5	0	0.5	1.0
0	1.000	1.000	1.000	1.000	1.000
5	1.000	1.000	1.000	1.000	1.000
10	1.000	1.000	1.000	1.000	1.000
15	1.000	0.998	0.995	0.992	0.990
20	1.000	0.989	0.978	0.967	0.956
25	0.997	0.979	0.961	0.942	0.923
30	0.991	0.968	0.943	0.917	0.888
35	0.983	0.955	0.925	0.891	0.853
40	0.973	0.940	0.905	0.865	0.818
45	0.959	0.924	0.884	0.837	0.782
50	0.944	0.905	0.861	0.808	0.747
55	0.927	0.885	0.836	0.778	0.711
60	0.907	0.862	0.809	0.746	0.676
65	0.886	0.837	0.779	0.714	0.642
70	0.861	0.809	0.748	0.680	0.609
75	0.835	0.779	0.715	0.646	0.576
80	0.805	0.746	0.681	0.612	0.545
85	0.772	0.711	0.645	0.579	0.516
90	0.737	0.675	0.610	0.547	0.487
95	0.700	0.638	0.575	0.515	0.461
100	0.661	0.600	0.541	0.485	0.435
105	0.622	0.564	0.508	0.457	0.412
110	0.584	0.528	0.477	0.431	0.389
115	0.546	0.495	0.448	0.406	0.368
120	0.510	0.463	0.421	0.383	0.348
125	0.476	0.434	0.395	0.361	0.330
130	0.445	0.406	0.372	0.341	0.313
135	0.416	0.381	0.350	0.322	0.297
140	0.389	0.357	0.330	0.304	0.282
145	0.364	0.336	0.311	0.288	0.268
150	0.341	0.316	0.293	0.273	0.255
155	0.320	0.298	0.277	0.259	0.242
160	0.301	0.281	0.263	0.246	0.231
165	0.283	0.265	0.249	0.234	0.220
170	0.267	0.251	0.236	0.222	0.210
175	0.252	0.238	0.224	0.212	0.200
180	0.239	0.225	0.213	0.202	0.192
185	0.226	0.214	0.203	0.193	0.183
190	0.214	0.203	0.193	0.184	0.175
195	0.204	0.194	0.185	0.176	0.168
200	0.194	0.185	0.176	0.168	0.161
205	0.184	0.176	0.168	1.161	0.154
210	0.176	0.168	0.161	0.154	0.148
215	0.167	0.161	0.154	0.148	0.142
220	0.160	0.154	0.148	0.142	0.137
225	0.153	0.147	0.142	0.137	0.132
230	0.146	0.141	0.136	0.131	0.127
235	0.140	0.135	0.131	0.126	0.122

(continued)

Modified member slenderness (λ_n)	Compression member section constant (α_b)				
	−1.0	−0.5	0	0.5	1.0
240	0.134	0.130	0.126	0.122	0.118
245	0.129	0.125	0.121	0.117	0.114
250	0.124	0.120	0.116	0.113	0.110
255	0.119	0.116	0.112	0.109	0.106
260	0.115	0.111	0.108	0.105	0.102
265	0.110	0.107	0.104	0.102	0.099
270	0.106	0.103	0.101	0.098	0.096
275	0.102	0.100	0.097	0.095	0.092
280	0.099	0.096	0.094	0.092	0.089
285	0.095	0.093	0.091	0.089	0.087
290	0.092	0.090	0.088	0.086	0.084
295	0.089	0.087	0.085	0.083	0.081
300	0.086	0.084	0.082	0.081	0.079
305	0.083	0.082	0.080	0.078	0.077
310	0.081	0.079	0.077	0.076	0.074
315	0.078	0.077	0.075	0.074	0.072
320	0.076	0.074	0.073	0.071	0.070
340	0.067	0.066	0.065	0.064	0.063
370	0.057	0.056	0.055	0.054	0.054
400	0.049	0.048	0.047	0.047	0.046
450	0.039	0.038	0.038	0.037	0.037
500	0.031	0.031	0.031	0.031	0.030
550	0.026	0.026	0.026	0.025	0.025
600	0.022	0.022	0.022	0.021	0.021

6.4.2 Laced compression members

6.4.2.1 *Slenderness ratio of a main component* The maximum slenderness ratio $(L_e/r)_c$ of a main component, based on its minimum radius of gyration and the length between consecutive points where lacing is attached, shall not exceed the lesser of 50 or 0.6 times the slenderness ratio of the member as a whole.

6.4.2.2 *Slenderness ratio of a laced compression member* The slenderness ratio shall be calculated by assuming that the main components act as an integral member but shall not be taken as less than $1.4(L_e/r)_c$.

6.4.2.3 *Lacing angle* The angle of inclination of the lacing to the longitudinal axis of the member shall be within the following limits:

(a) 50° to 70° for single lacing.

(b) 40° to 50° for double lacing.

6.4.2.4 *Effective length of a lacing element* The effective length of a lacing element shall be taken as the distance between the inner welds or fasteners for single lacing, and 0.7 times this distance for double lacing which is connected by welds or fasteners.

6.4.2.5 *Slenderness ratio limit of a lacing element* The slenderness ratio of a lacing element shall not exceed 140.

6.4.2.6 *Mutually opposed lacing* Single lacing systems mutually opposed in direction on opposite sides of two main components shall not be used unless allowance is made for the resulting torsional effects.

Double lacing systems and single lacing systems mutually opposed in direction on opposite sides of two main components shall not be combined with members or diaphragms perpendicular to the longitudinal axis of the compression member, except for tie plates as specified in Clause 6.4.2.7, unless all actions resulting from the deformation of the compression member are calculated and allowed for in design.

6.4.2.7 *Tie plates* Tie plates shall be provided at the ends of the lacing system, at points where the lacing system is interrupted, and at connections with other members. End tie plates shall have a width measured along the axis of the member of not less than the perpendicular distance between the centroids of their connections to the main components. Intermediate tie plates shall have a width of not less than three-quarters of this distance.

A tie plate and its connections shall be treated as battens for design purposes (see Clause 6.4.3). The thickness of a tie plate shall not be less than 0.02 times the distance between the innermost lines of welds or fastenings, except where the tie plate is effectively stiffened at the free edges. In the latter case, the edge stiffeners shall have a slenderness ratio less than 170.

6.4.3 Battened compression member

6.4.3.1 *Slenderness ratio of a main component* The maximum slenderness ratio $(L_e/r)_c$ of a main component, based on its minimum radius of gyration and the length between consecutive points where battens are attached, shall not exceed the lesser of 50, or 0.6 times the slenderness ratio of the member as a whole determined using Clause 6.4.3.2.

6.4.3.2 *Slenderness ratios of battened compression member* The slenderness ratio $(L_e/r)_b$ of a battened compression member about the axis normal to the plane of the battens shall be calculated as follows:

$$\left(\frac{L_e}{r}\right)_{bm} = \sqrt{\left[\left(\frac{L_e}{r}\right)_m^2 + \left(\frac{L_e}{r}\right)_c^2\right]}$$

where

$\left(\frac{L_e}{r}\right)_m$ = the slenderness ratio of the whole member about the above axis calculated by assuming that the main components act as an integral member

$\left(\frac{L_e}{r}\right)_c$ = the maximum slenderness ratio of the main component, determined in accordance with Clause 6.4.3.1.

The slenderness ratio $(L_e/r)_{bp}$ of a battened compression member about the axis parallel to the plane of the battens shall be taken as not less than $1.4(L_e/r)_c$.

6.4.3.3 *Effective length of a batten* The effective length of an end batten shall be taken as the perpendicular distance between the centroids of the main components. The effective length of an intermediate batten shall be taken as 0.7 times the perpendicular distance between the centroids of the main components.

6.4.3.4 *Maximum slenderness ratio of a batten* The slenderness ratio of a batten shall not exceed 180.

6.4.3.5 *Width of a batten* The width of an end batten shall be not less than the greater of the distance between the centroids of the main components and twice the width of the narrower main component.

The width of an intermediate batten shall be not less than the greater of half the distance between the main components and twice the width of the narrower main component.

6.4.3.6 *Thickness of a batten* The thickness of a batten shall be not less than 0.02 times the minimum distance between the innermost lines of welds or fasteners, except where the batten is effectively stiffened at the free edges. In this case, the edge stiffeners shall have a slenderness ratio of not greater than 170, where the radius of gyration is taken about the axis parallel to the member axis.

6.4.3.7 *Loads on battens* The batten and its connections shall be designed to transmit simultaneously to the main components a design longitudinal shear force (V_l^*) calculated as follows:

$$V_l^* = \frac{V^* s_b}{n_b d_b}$$

and a design bending moment (M^*) calculated as follows:

$$M^* = \frac{V^* s_b}{2 n_b}$$

where

V^* = the design transverse shear force specified in Clause 6.4.1

s_b = the longitudinal centre-to-centre distance between the battens

n_b = the number of parallel planes of battens

d_b = the lateral distance between the centroids of the welds or fasteners.

6.5 COMPRESSION MEMBERS BACK TO BACK

6.5.1 Components separated

6.5.1.1 *Application* This Clause applies to compression members composed of two angle, channel or tee-section components discontinuously separated back to back by a distance not exceeding that required for the end gusset connection. If such a member is designed as a single integral member, then it shall comply with Clauses 6.5.1.2 to 6.5.1.5.

6.5.1.2 *Configuration* The configuration of the main components shall be of similar sections arranged symmetrically with their corresponding rectangular axes aligned.

6.5.1.3 *Slenderness* The slenderness of the compression member about the axis parallel to the connected surfaces shall be calculated in accordance with Clause 6.4.3.2.

6.5.1.4 *Connection* The main components shall be interconnected by fasteners. Where the components are connected together, the member shall be designed as a battened compression member in accordance with Clause 6.4.3. The main components shall be connected at intervals so that the member is divided into at least three bays of approximately equal length. At the ends of the member, the main components shall be connected by not less than two fasteners in each line along the length of the member, or by equivalent welds.

6.5.1.5 *Design forces* The interconnecting fasteners shall be designed to transmit a design longitudinal shear force between the components induced by the transverse shear force (V^*) given in Clause 6.4.1. The design longitudinal shear force V^*_l per connection shall be taken as follows:

$$V_l^* = 0.25 V^* \left(\frac{L_e}{r}\right)_c$$

where $(L_e/r)_c$ is the slenderness ratio of the main component between the interconnections.

6.5.2 Components in contact

6.5.2.1 *Application* This Clause applies to compression members composed of two angle, channel or tee-section components back-to-back or separated by continuous steel packing. If such a member is designed as a single integral member, then it shall comply with Clauses 6.5.2.2 to 6.5.2.5.

6.5.2.2 *Configuration* The main components shall be of similar sections arranged symmetrically with their corresponding rectangular axes aligned.

6.5.2.3 *Slenderness* The slenderness of the compression member about the axis parallel to the connected surfaces shall be calculated in accordance with Clause 6.4.3.2.

6.5.2.4 *Connection* The main components shall be connected at intervals so that the member is divided into at least three bays of approximately equal length. At the ends of the member, the main components shall be interconnected by not less than two fasteners in each line along the length of the member, or by equivalent welds.

6.5.2.5 *Design forces* The interconnecting fasteners or welds shall be designed to transmit a longitudinal shear force between the components induced by the transverse shear force (V^*) in accordance with Clause 6.4.1. The design longitudinal shear force V^*_1 per connection shall be as specified in Clause 6.5.1.5.

6.6 RESTRAINTS

6.6.1 Restraint systems The members and the connections of restraining systems required to brace compression members and reduce their effective lengths shall be determined by analysing the structure for its design loads, including any notional horizontal forces (see Clause 3.2.4), from the points where the forces arise to anchorage or reaction points, and by designing the members and connections as specified in Clauses 6.6.2 and 6.6.3.

6.6.2 Restraining members and connections At each restrained cross-section of a compression member, the restraining members and their connections which are required to brace the compression member shall be designed for the greater of—

(a) the restraining member forces specified in Clause 6.6.1; and

(b) 0.025 times the maximum axial compression force in the member at the position of the restraint,

except where the restraints are more closely spaced than is required to ensure that—

$$N^* = \phi N_c$$

When the restraint spacing is less, then a lesser force may be designed for. The actual arrangement of restraints shall be assumed to be equivalent to a set of restraints which will ensure that $N^* = \phi N_c$. Each equivalent restraint shall correspond to an appropriate group of the actual restraints. This group shall then be designed as a whole to transfer the transverse force determined for the position of the equivalent restraint.

6.6.3 Parallel braced compression members When a series of parallel compression members is restrained by a line of restraints, each restraining element shall be designed to transfer the transverse force specified in Clause 6.6.2, except that 0.025 times the axial compression force shall be replaced by the sum of 0.025 times the axial force in the connected compression member and 0.0125 times the sum of the axial forces in the connected compression members beyond, with no more than seven members considered in the summation.

SECTION 7 MEMBERS SUBJECT TO AXIAL TENSION

7.1 DESIGN FOR AXIAL TENSION A member subject to a design axial tension force (N^*) shall satisfy—

$$N^* \leq \phi N_t$$

where

ϕ = the capacity factor, see Table 3.4

N_t = the nominal section capacity in tension determined in accordance with Clause 7.2.

7.2 NOMINAL SECTION CAPACITY The nominal section capacity of a tension member shall be taken as the lesser of—

$$N_t = A_g f_y; \text{ and}$$

$$N_t = 0.85 k_t A_n f_u$$

where

A_g = the gross area of the cross-section

f_y = the yield stress used in design

k_t = the correction factor for distribution of forces determined in accordance with Clause 7.3

A_n = the net area of the cross-section, obtained by deducting from the gross area the sectional area of all penetrations and holes, including fastener holes. The deduction for all fastener holes shall be made in accordance with Clause 9.1.10. For threaded rods, the net area shall be taken as the tensile stress area of the threaded portion, as defined in AS 1275.

f_u = the tensile strength used in design.

7.3 DISTRIBUTION OF FORCES

7.3.1 End connections providing uniform force distribution Where for design purposes it is assumed that the tensile force is distributed uniformly to a tension member, the end connections shall satisfy both the following:

(a) The connections shall be made to each part of the member and shall be symmetrically placed about the centroidal axis of the member.

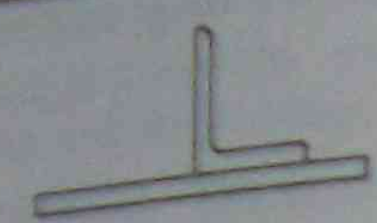
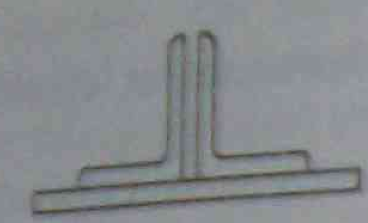
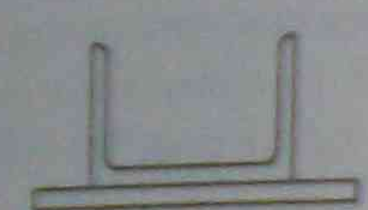
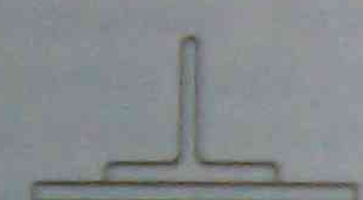
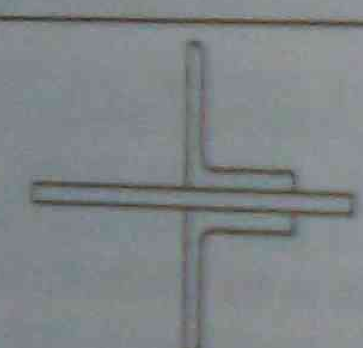
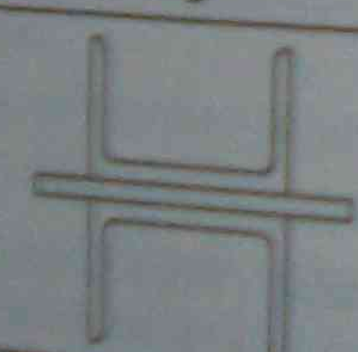
(b) Each part of the connection shall be proportioned to transmit at least the maximum design force carried by the connected part of the member.

For connections satisfying these requirements, the value of k_t shall be taken as 1.0.

7.3.2 End connections providing non-uniform force distribution If the end connections of a tension member do not satisfy the requirements of Clause 7.3.1, then the member shall be designed to comply with Section 8 using $k_t = 1.0$, except that Clause 7.2 may be used for the following members:

(a) *Eccentrically connected angles, channels and tees* Eccentrically connected angles, channels and tees may be designed in accordance with Clause 7.2, using the appropriate value of k_t given in Table 7.3.2.

TABLE 7.3.2
CORRECTION FACTOR (k_t)

Configuration case		Correction factor (k_t)
(i)		0.75 for unequal angles connected by the short leg 0.85 otherwise
(ii)		As for Case (i)
(iii)		0.85
(iv)		0.90
(v)		1.0
(vi)		1.0
(vii)		1.0

(b) *I-sections or channels connected by both flanges only* A symmetrical rolled or built-up member of solid I-section or channel section connected by both flanges only may be designed in accordance with Clause 7.2 using a value of k_t equal to 0.85 provided that—

(i) the length between the first and last rows of fasteners in the connection or, when the member is welded, the length of longitudinal weld provided to each side of the connected flanges shall be not less than the depth of the member; and

(ii) each flange connection shall be proportioned to transmit at least half of the maximum design force carried by the connected member.

7.4 TENSION MEMBERS WITH TWO OR MORE MAIN COMPONENTS

7.4.1 General A tension member composed of two or more main components intended to act as a single member shall comply with Clauses 7.4.2 to 7.4.5.

7.4.2 Design forces for connections If a tension member is composed of two or more main components, the connections between the components shall be proportioned to resist the internal actions arising from the external design forces and bending moments (if any). The design forces for lacing bars, and the design forces and bending moments (if any) for battens, shall be considered as divided equally among the connection planes parallel to the direction of force.

7.4.3 Tension member composed of two components back-to-back A tension member composed of two flats, angles, channels or tees, discontinuously connected back-to-back either in contact or separated by a distance not exceeding that required for the end gusset connection, shall comply with the following:

(a) *Where the components are separated* They shall be connected either—

(i) together at regular intervals along their length by welding, or bolting, so that the slenderness ratio of the individual components between connections does not exceed 300; or

(ii) by connections which comply with Clauses 6.5.1.4 and 6.5.1.5.

(b) *Where component members are in contact back-to-back* They shall be connected together as required by Clauses 6.5.2.4 and 6.5.2.5.

7.4.4 Laced tension member A tension member composed of two components connected by lacing shall comply with Clause 6.4.2 except as follows:

(a) The slenderness ratio of the lacing elements shall not exceed 210.

(b) The slenderness ratio of a main component based on its minimum radius of gyration and the length between consecutive points where lacing is attached shall not exceed 300.

For tie plates, the requirements of Clause 6.4.2.7 shall be satisfied except that the thickness of tie plates shall be not less than 0.017 times the distance between the innermost lines of connections.

7.4.5 Battened tension member A tension member composed of two components connected by battens shall comply with Clause 6.4.3 except as follows:

(a) The spacing of battens shall be such that the maximum slenderness ratio of each main component, based on its minimum radius of gyration and the length between consecutive battens, does not exceed 300.

(b) Battens attached by bolts shall be connected by not less than two bolts and Clause 6.4.3.7 shall not apply.

(c) Batten plates shall have a thickness of not less than 0.017 times the distance between the innermost lines of connections.

(d) Intermediate battens shall have a width of not less than half the effective width of end batten plates.

7.5 MEMBERS WITH PIN CONNECTIONS

The nominal capacity of a pin connection shall be determined in accordance with Clause 9.5. A pin connection in a tension member shall comply with the following additional requirements:

(a) The thickness of an unstiffened element containing a hole for a pin connection shall be greater than or equal to 0.25 times the distance from the edge of the hole to the edge of the element measured at right angles to the axis of the member. This limit does not apply to the internal plies where the connected elements are clamped together by external nuts.

(b) The net area beyond a hole for a pin, parallel to or within 45° of the axis of the member, shall be greater than or equal to the net area required for the member.

(c) The sum of the areas at a hole for a pin, perpendicular to the axis of the member, shall be greater than or equal to 1.33 times the net area required for the member.

(d) Pin plates provided to increase the net area of a member or to increase the bearing capacity of a pin shall be arranged to avoid eccentricity and shall be proportioned to distribute the load from the pin into the member.

SECTION 8 MEMBERS SUBJECT TO COMBINED ACTIONS

8.1 GENERAL A member subject to combined axial and bending actions shall be proportioned so that its design actions specified in Clause 8.2, in combination with the nominal section and member capacities (see Sections 5, 6 and 7), satisfy Clauses 8.3 and 8.4. For plastic design (see Clause 4.5), only the requirements of Clause 8.4.3 need to be satisfied.

Eccentrically loaded double-bolted or welded angles in trusses shall be proportioned to satisfy Clause 8.3, and either Clause 8.4.5 or Clause 8.4.6.

8.2 DESIGN ACTIONS For checking the section capacity at a section, the design axial force (N^*), which may be tension or compression, shall be the force at the section, and the design bending moments (M^*_x, M^*_y) shall be the bending moments at the section about the major *x*- and minor *y*- principal axes, respectively.

For checking the member capacity, the design axial force (N^*) shall be the maximum axial force in the member, and the design bending moments (M^*_x, M^*_y) shall be the maximum bending moments in the member.

M^*_x, M^*_y are the design bending moments resulting from frame action and transverse loading on the member, and include the second order design bending moments resulting from the design loads acting on the structure and its members in their displaced and deformed configuration.

The design bending moments (M^*_x, M^*_y) shall be determined from one of the following methods of analysis:

(a) *First-order linear elastic analysis*—by modifying the first-order design bending moments, by using the appropriate moment amplification factors determined in accordance with Clause 4.4.2.

(b) *Second-order elastic analysis*—in which the design bending moments (M^*) are obtained either directly, or by modifying the second-order end moments by using the moment amplification factors determined in accordance with Appendix E.

(c) *First-order plastic analysis*—in which the design bending moments (M^*) are obtained directly for frames where the elastic buckling load factor (λ_c) satisfies $\lambda_c \geq 5$ and the requirements of Clause 4.5.4 are satisfied.

(d) *Second-order plastic analysis*—in which the design bending moments (M^*) are obtained directly for frames where the elastic buckling load factor (λ_c) satisfies $\lambda_c < 5$.

(e) *Advanced structural analysis*—in which the design bending moments (M^*_x, M^*_y) are obtained directly in accordance with Appendix D, in which case only the section capacity requirements of Clause 8.3 and the connection requirements of Section 9 need to be satisfied.

8.3 SECTION CAPACITY

8.3.1 General The member shall satisfy Clauses 8.3.2, 8.3.3 and 8.3.4, as appropriate:

(a) For bending about the major principal *x*-axis only, sections at all points along the member shall have sufficient capacity to satisfy Clause 8.3.2.

(b) For bending about the minor principal *y*-axis only, sections at all points along the member shall have sufficient capacity to satisfy Clause 8.3.3.

(c) For bending about a non-principal axis, or bending about both principal axes, sections at all points along the member shall have sufficient capacity to satisfy Clause 8.3.4.

In this Section—

M_{sx}, M_{sy} = the nominal section moment capacities about the x- and y-axes respectively, determined in accordance with Clause 5.2

N_s = the nominal section axial load capacity determined in accordance with Clause 6.2 for axial compression, or Clause 7.2 for axial tension (for which $N_s = N_t$).

8.3.2 Uniaxial bending about the major principal x-axis Where uniaxial bending occurs about the major principal x-axis, the following shall be satisfied:

$$M_x^* \leq \phi M_{rx}$$

where

ϕ = the capacity factor (see Table 3.4)

M_{rx} = the nominal section moment capacity, reduced by axial force (tension or compression)

$$= M_{sx}\left(1 - \frac{N^*}{\phi N_s}\right)$$

Alternatively, for doubly symmetric I-sections and rectangular and square hollow sections to AS 1163, which are compact, as defined in Clause 5.2.3, M_{rx} may be calculated by one of the following as appropriate:

(a) For compression members where k_f is equal to 1.0 and for tension members—

$$M_{rx} = 1.18M_{sx}\left(1 - \frac{N^*}{\phi N_s}\right) \leq M_{sx}$$

(b) For compression members where k_f is less than 1.0—

$$M_{rx} = M_{sx}\left(1 - \frac{N^*}{\phi N_s}\right)\left[1 + 0.18\left(\frac{82 - \lambda_w}{82 - \lambda_{wy}}\right)\right] \leq M_{sx}$$

where λ_w and λ_{wy} are the values of λ_e and λ_{ey} for the web (see Clause 6.2.3 and Table 6.2.4).

8.3.3 Uniaxial bending about the minor principal y-axis Where uniaxial bending occurs about the minor principal y-axis, the design bending moment (M^*_y) about the minor principal y-axis shall satisfy—

$$M^*_y \leq \phi M_{ry}$$

where

ϕ = the capacity factor (see Table 3.4)

M_{ry} = the nominal section moment capacity reduced by the axial tensile or compressive force

$$= M_{sy}\left[1 - \frac{N^*}{\phi N_s}\right]$$

Alternatively, M_{ry} may be calculated by one of the following as appropriate:

(a) $$M_{ry} = 1.19\, M_{sy}\left[1 - \left(\frac{N^*}{\phi N_s}\right)^2\right] \leq M_{sy}$$

for doubly symmetric I-sections which are compact, as defined in Clause 5.2.3.

(b) $$M_{ry} = 1.18\, M_{sy}\left[1 - \left(\frac{N^*}{\phi N_s}\right)\right] \leq M_{sy}$$

for rectangular or square hollow sections to AS 1163 which are compact as defined in Clause 5.2.3.

8.3.4 Biaxial bending Where biaxial bending occurs, the design tensile or compressive force (N^*) and the design bending moments (M^*_x) and (M^*_y) about the major principal x-axis and minor principal y-axis shall satisfy—

$$\frac{N^*}{\phi N_s} + \frac{M_x^*}{\phi M_{sx}} + \frac{M_y^*}{\phi M_{sy}} \leq 1$$

Alternatively, for doubly symmetric I-sections and rectangular and square hollow sections to AS 1163, which are compact as defined in Clause 5.2.3, sections at all points along the member shall satisfy—

$$\left(\frac{M_x^*}{\phi M_{rx}}\right)^\gamma + \left(\frac{M_y^*}{\phi M_{ry}}\right)^\gamma \leq 1$$

where M_{rx} and M_{ry} shall be calculated in accordance with Clauses 8.3.2 and 8.3.3 respectively, and

$$\gamma = 1.4 + \left(\frac{N^*}{\phi N_s}\right) \leq 2.0$$

8.4 MEMBER CAPACITY

8.4.1 General The member shall satisfy Clauses 8.4.2, 8.4.3 and 8.4.4, as appropriate:

(a) For a member bent about the major principal x-axis only and where there is sufficient restraint to prevent lateral buckling, or for a member bent about the minor principal y-axis only, the member shall satisfy the in-plane requirements of Clause 8.4.2 for a frame analyzed elastically, or Clause 8.4.3 for a frame analyzed plastically.

(b) For a member bent about the major principal x-axis only and with insufficient restraint to prevent lateral buckling, the member shall satisfy both the in-plane requirements of Clause 8.4.2 and the out-of-plane requirements of Clause 8.4.4.

(c) For a member bent about a non-principal axis, or bent about both principal axes, the member shall satisfy the biaxial bending requirements of Clause 8.4.5.

8.4.2 In-plane capacity—elastic analysis

8.4.2.1 *Application* This Clause applies to a member analyzed using an elastic method in accordance with Clause 4.4, or to a member in a statically determinate structure.

8.4.2.2 *Compression members* A member bent about a principal axis shall have sufficient in-plane capacity to satisfy the following:

$$M^* \le \phi M_i$$

where

M^* = the design bending moment about the principal axis

ϕ = the capacity factor (see Table 3.4)

M_i = the nominal in-plane member moment capacity

$$= M_s\left(1 - \frac{N^*}{\phi N_c}\right)$$

M_s = the nominal section moment capacity determined in accordance with Clause 5.2 for bending about the same principal axis as the design bending moment

N^* = the design axial compressive force

N_c = the nominal member capacity in axial compression determined in accordance with Clause 6.3 for buckling about the same principal axis, with the effective length factor (k_e) taken as 1.0 for both braced and sway members, unless a lower value is calculated for braced members from Clause 4.6.3.2, 4.6.3.3 or Clause 4.6.3.5, provided Clause 6.1 is satisfied for N_c calculated using l_e determined in accordance with Clause 4.6.3.

Alternatively, for doubly symmetric I-sections and rectangular and square hollow sections to AS 1163, which are compact as defined in Clause 5.2.3, and where the form factor (k_f) determined in accordance with Clause 6.2.2 is unity, M_i may be calculated as follows:

$$M_i = M_s\left\{\left[1 - \left(\frac{1 + \beta_m}{2}\right)^3\right]\left(1 - \frac{N^*}{\phi N_c}\right) + 1.18\left(\frac{1 + \beta_m}{2}\right)^3\sqrt{\left(1 - \frac{N^*}{\phi N_c}\right)}\right\}$$

$\le M_{rx}$ or M_{ry} as appropriate

where

β_m = the ratio of the smaller to the larger end bending moment, taken as positive when the member is bent in reverse curvature for members without transverse load, or

= the value determined in accordance with Clause 4.4.2.2 for members with transverse load

M_{rx} or M_{ry} = the nominal section moment capacity about the appropriate principal axis determined in accordance with Clause 8.3.

8.4.2.3 *Tension members* A member subject to a design axial tensile force (N^*) and a design bending moment (M^*) shall satisfy Clause 8.3.

8.4.3 In-plane capacity— plastic analysis

8.4.3.1 *Application* This Clause applies only to compact doubly symmetric I-section members. When the distribution of moments in a frame is determined using a plastic method of analysis in accordance with Clause 4.5, then the design axial compressive force (N^*) in any member of the frame which is assumed to contain a plastic hinge shall satisfy the member slenderness requirements of Clause 8.4.3.2, and the web slenderness requirements of Clause 8.4.3.3.

The design plastic moment capacity reduced by axial force (tension or compression) for compact doubly symmetric I-sections shall be as specified in Clause 8.4.3.4.

8.4.3.2 *Member slenderness* The design axial compressive force (N^*) in every member assumed to contain a plastic hinge shall satisfy the following:

$$\frac{N^*}{\phi N_s} \le \left[\frac{0.60 + 0.40\beta_m}{\sqrt{(N_s/N_{oL})}}\right]^2 \quad \text{when} \quad \frac{N^*}{\phi N_s} \le 0.15,$$

and

$$\frac{N^*}{\phi N_s} \le \frac{1 + \beta_m - \sqrt{(N_s/N_{oL})}}{1 + \beta_m + \sqrt{(N_s/N_{oL})}} \quad \text{when} \quad \frac{N^*}{\phi N_s} > 0.15,$$

where

β_m = the ratio of the smaller to the larger end bending moment, taken as positive when the member is bent in reverse curvature

N_s = the nominal section capacity in axial compression determined in accordance with Clause 6.2

$$N_{oL} = \frac{\pi^2 EI}{L^2}$$

I = the second moment of area for the axis about which the design moment acts

L = the actual length of the member.

A member for which—

$$\frac{N^*}{\phi N_s} > 0.15, \text{ and}$$

$$\frac{N^*}{\phi N_s} > \frac{1 + \beta_m - \sqrt{(N_s/N_{oL})}}{1 + \beta_m + \sqrt{(N_s/N_{oL})}}$$

shall not contain plastic hinges, although it shall be permissible to design the member as an elastic member in a plastically analyzed structure to satisfy the requirements of Clause 8.4.2.

8.4.3.3 *Web slenderness* The design axial compressive force (N^*) in every member assumed to contain a plastic hinge shall satisfy the following:

$$\frac{N^*}{\phi N_s} \le 0.60 - \left[\frac{d_1}{t}\,\frac{\sqrt{(f_y/250)}}{137}\right] \quad \text{for webs where} \quad 45 \le \frac{d_1}{t}\sqrt{\left(\frac{f_y}{250}\right)} \le 82$$

or

$$\frac{N^*}{\phi N_s} \le 1.91 - \left[\frac{d_1}{t}\frac{\sqrt{(f_y/250)}}{27.4}\right] \le 1.0 \quad \text{for webs where} \quad 25 < \frac{d_1}{t}\sqrt{\left(\frac{f_y}{250}\right)} < 45$$

or

$$\frac{N^*}{\phi N_s} \le 1.0 \quad \text{for webs where} \quad 0 \le \frac{d_1}{t}\sqrt{\left(\frac{f_y}{250}\right)} \le 25$$

Members which have webs for which $(d_1/t)\sqrt{(f_y/250)}$ exceeds 82 shall not contain plastic hinges, although it shall be permissible to design such a member as an elastic member in a plastically analyzed structure to satisfy the requirements of Clause 8.4.2.

8.4.3.4 *Plastic moment capacity* The design plastic moment capacity (ϕM_{pr}) reduced for axial force (tension or compression) shall be calculated as follows:

$$\phi M_{prx} = 1.18\phi M_{sx}\left(1 - \frac{N^*}{\phi N_s}\right) \le \phi M_{sx}$$

for members bent about the major principal x-axis, and

$$\phi M_{pry} = 1.19\phi M_{sy}\left[1 - \left(\frac{N^*}{\phi N_s}\right)^2\right] \le \phi M_{sy}$$

for members bent about the minor principal y-axis

where M_{sx} and M_{sy} are the nominal section moment capacities determined in accordance with Clauses 5.2.1 and 5.2.3.

8.4.4 Out-of-plane capacity

8.4.4.1 *Compression members* A member subject to a design axial compressive force (N^*) and a design bending moment (M_x^*) about its major principal x-axis, and which may buckle laterally, shall satisfy Clause 8.4.2 and also the following:

$$M_x^* \le \phi M_{ox}$$

where

ϕ = the capacity factor (see Table 3.4)

M_{ox} = the nominal out-of-plane member moment capacity

$$= M_{bx}\left(1 - \frac{N^*}{\phi N_{cy}}\right)$$

M_{bx} = the nominal member moment capacity of the member without full lateral restraint and bent about the major principal x-axis, determined in accordance with Clause 5.6 using a moment modification factor (α_m) appropriate to the distribution of design bending moments along the member

N_{cy} = the nominal member capacity in axial compression, determined in accordance with Clause 6.3 for buckling about the minor principal y-axis.

Alternatively, for members without transverse loads which are of compact doubly symmetric I-section (see Clause 5.2.3), are fully or partially restrained at both ends, and have a form factor (k_f) of unity determined in accordance with Clause 6.2.2, M_{ox} may be calculated as follows:

$$M_{ox} = \alpha_{bc} M_{bxo}\sqrt{\left[\left(1 - \frac{N^*}{\phi N_{cy}}\right)\left(1 - \frac{N^*}{\phi N_{oz}}\right)\right]}$$

where

$$\frac{1}{\alpha_{bc}} = \frac{1 - \beta_m}{2} + \left(\frac{1 + \beta_m}{2}\right)^3\left(0.4 - 0.23\frac{N^*}{\phi N_{cy}}\right)$$

M_{bxo} = the nominal member moment capacity without full lateral restraint and with a uniform distribution of design bending moment so that α_m is unity, determined in accordance with Clause 5.6

N_{cy} = the nominal member capacity in axial compression, determined in accordance with Clause 6.3 for buckling about the minor principal y-axis

β_m = the ratio of the smaller to the larger end bending moment, taken as positive when the member is bent in reverse curvature

N_{oz} = the nominal elastic torsional buckling capacity of the member, calculated as follows:

$$N_{oz} = \frac{GJ + (\pi^2 E I_w / L_z^2)}{(I_x + I_y)/A}$$

E, G = the elastic moduli

A, I_w, I_x, I_y, and J = the section constants

L_z = the distance between partial or full torsional restraints

NOTE: Values of E and G, and expressions for I_w and J are given in Appendix H.

8.4.4.2 *Tension members* A member subject to a design axial tension force (N^*) and a design bending moment (M_x^*) about its major principal x-axis, and which may buckle laterally, shall satisfy the following:

$$M_x^* \le \phi M_{ox}$$

where

ϕ = the capacity factor (see Table 3.4)

M_{ox} = the nominal out-of-plane member moment capacity

$$= M_{bx}\left(1 + \frac{N^*}{\phi N_t}\right) \le M_{rx}$$

M_{bx} = the nominal member moment capacity defined in Clause 8.4.4.1

N_t = the nominal section capacity in axial tension determined in accordance with Clause 7.2

M_{rx} = the nominal section moment capacity reduced by axial force determined in accordance with Clause 8.3.2.

8.4.5 Biaxial bending capacity

8.4.5.1 *Compression members* A member subject to a design axial compressive force (N^*) and design bending moments (M_x^*) and (M_y^*) about the major x- and minor y- principal axes respectively shall satisfy the following:

$$\left(\frac{M_x^*}{\phi M_{cx}}\right)^{1.4} + \left(\frac{M_y^*}{\phi M_{iy}}\right)^{1.4} \leq 1$$

where

ϕ = the capacity factor (see Table 3.4)

M_{cx} = the lesser of the nominal in-plane member moment capacity (M_{ix}) and the nominal out-of-plane member moment capacity (M_{ox}) for bending about the major principal x-axis, determined in accordance with Clauses 8.4.2 and 8.4.4 respectively

M_{iy} = the nominal in-plane member moment capacity, determined in accordance with Clause 8.4.2, for bending about the minor principal y-axis.

8.4.5.2 *Tension members* A member subject to a design axial tension force (N^*) and design bending moments (M_x^*) and (M_y^*) about the major x- and minor y- principal axes respectively shall satisfy the following:

$$\left(\frac{M_x^*}{\phi M_{tx}}\right)^{1.4} + \left(\frac{M_y^*}{\phi M_{ry}}\right)^{1.4} \leq 1$$

where

ϕ = the capacity factor (see Table 3.4)

M_{tx} = the lesser of the nominal section moment capacity (M_{rx}) reduced by axial tension and the nominal out-of-plane member moment capacity (M_{ox}) determined in accordance with Clauses 8.3.2 and 8.4.4.2 respectively

M_{ry} = the nominal section moment capacity reduced by axial tension, determined in accordance with Clause 8.3.3.

8.4.6 Eccentrically loaded double bolted or welded single angles in trusses Single angle web compression members in trusses which are connected with at least two bolts or welded at their ends and loaded through one leg (see Figure 8.4.6) shall be designed to satisfy Clause 8.3 and either Clause 8.4.5 or the following:

$$\frac{N^*}{\phi N_{ch}} + \frac{M_h^*}{\phi M_{bx}\cos\alpha} \leq 1$$

where

N^* = the design axial compression force in the member

M^*_h = the design bending moment acting about the rectangular h-axis parallel to the loaded leg

ϕ = the capacity factor (see Table 3.4)

N_{ch} = the nominal member capacity in axial compression, determined in accordance with Clause 6.3, of a single angle compression member buckling with $L_e = L$ about the rectangular h-axis parallel to the loaded leg

M_{bx} = the nominal member moment capacity, determined in accordance with Clause 5.6, for an angle without full lateral support, bent about the major principal x-axis using a factor α_m appropriate to the distribution of design bending moment along the member

α = the angle between x- and h- axes.

For equal leg angles, where $L/t \leq (210 + 175\beta_m)(250/f_y)$, M_{bx} may be taken as M_{sx},

where

M_{sx} = the nominal section moment capacity about the x-principal axis, determined in accordance with Clause 5.2

L = the member length

t = the thickness of the angle.

For other equal leg angles, M_{bx} may be determined by using Clause 5.6.1.1 with—

$$M_o = \left(\frac{525\,t}{L}\right)\left(\frac{250}{f_y}\right) M_s$$

The design end bending moment (M^*_h) shall be calculated from a rational elastic analysis of the truss, or shall be taken as not less than N^*_e, resulting from the out-of-plane eccentricity (e) of the design axial force (N^*) in the member,

where

e = $(c_h - \frac{t}{2})$, for angles on the same side of the truss chord

= $(e_c + e_t)$, for angles on opposite sides of the truss chord.

(see Figure 8.4.6).

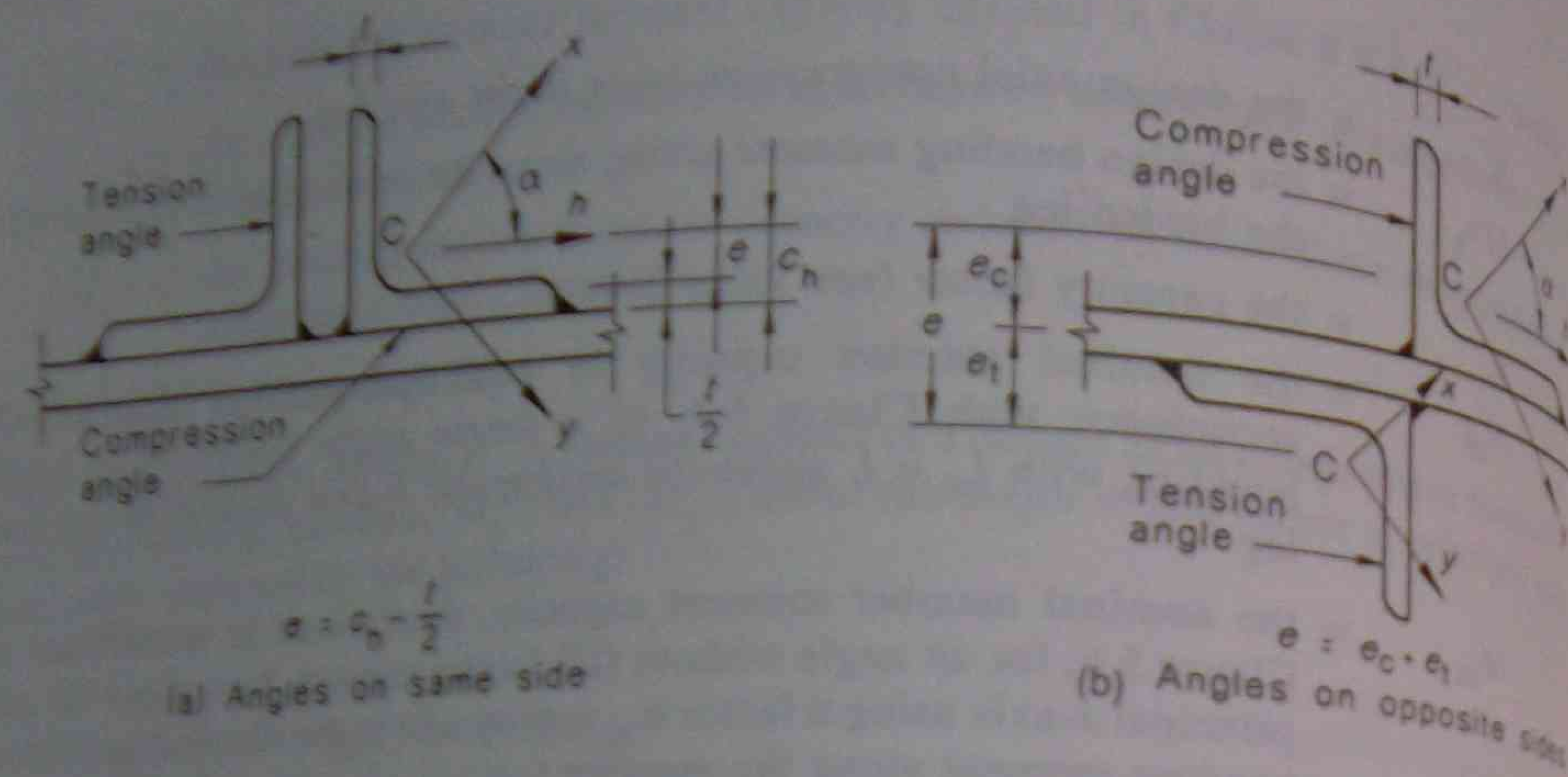


FIGURE 8.4.6 SINGLE ANGLES LOADED THROUGH ONE LEG

SECTION 9 CONNECTIONS

9.1 GENERAL

9.1.1 Requirements for connections Connection elements consist of connection components (cleats, gusset plates, brackets, connecting plates) and connectors (bolts, pins and welds). The connections in a structure shall be proportioned so as to be consistent with the assumptions made in the analysis of the structure and to comply with this Section. Connections shall be capable of transmitting the calculated design action effects.

9.1.2 Classification of connections

9.1.2.1 *Connections in rigid construction* The connections shall comply with Clause 4.2.2. The joint deformations shall be such that they have no significant influence on the distribution of action effects nor on the overall deformation of the frame.

9.1.2.2 *Connections in semi-rigid construction* The connections shall comply with Clause 4.2.3. Connections between members in semi-rigid construction shall provide a predictable degree of interaction between members, based on the actual action-deformation characteristics of the connection as determined experimentally.

9.1.2.3 *Connections in simple construction* The connections shall comply with Clause 4.2.4. Connections between members in simple construction shall be capable of deforming to provide the required rotation at the connection. The connections shall not develop a level of restraining bending moment which adversely affects any part of the structure. The rotation capacity of the connection shall be provided by the detailing of the connection and shall have been demonstrated experimentally. The connection shall be considered as subject to reaction shear forces acting at an eccentricity appropriate to the connection detailing.

9.1.2.4 *Connections in structures analyzed by the plastic method* Connections in structures analyzed by the plastic method shall comply with Clause 4.5.3, in addition to the requirements of this Section.

9.1.3 Design of connections Each element in a connection shall be designed so that the structure is capable of resisting all design actions. The design capacities of each element shall be not less than the calculated design action effects. For earthquake load combinations the connection shall be designed for the calculated design action effects, shall exhibit the required ductility and shall comply with Section 13.

Connections and the adjacent areas of members shall be designed by distributing the design action effects so that they comply with the following requirements:

(a) The distributed design action effects are in equilibrium with the design action effects acting on the connection.

(b) The deformations in the connection are within the deformation capacities of the connection elements.

(c) All of the connection elements and the adjacent areas of members are capable of resisting the design action effects acting on them.

(d) The connection elements shall remain stable under the design action effects and deformations.

Design shall be on the basis of a recognized method supported by experimental evidence.

Residual actions due to the installation of bolts need not be considered.

9.1.4 Minimum design actions on connections Connections carrying design action effects, except for lacing connections and connections to sag rods, purlins and girts, shall be designed to transmit the greater of—

(a) the design action in the member; and

(b) the minimum design action effects expressed either as the value or the factor times the member design capacity for the minimum size of member required by the strength limit state, specified in Items (i) to (vii) below:

(i) Connections in rigid construction—a bending moment of 0.5 times the member design moment capacity.

(ii) Connections to beams in simple construction—a shear force of 40 kN.

(iii) Connections at the ends of tension or compression members—a force of 0.3 times the member design capacity, except that for threaded rod acting as a bracing member with turnbuckles, the minimum tension force shall be equal to the member design capacity.

(iv) Splices in members subject to axial tension—a force of 0.3 times the member design capacity in tension.

(v) Splices in members subject to axial compression—for ends prepared for full contact in accordance with Clause 14.4.4.2, it shall be permissible to carry compressive actions by bearing on contact surfaces. When members are prepared for full contact to bear at splices, there shall be sufficient fasteners to hold all parts securely in place. The fasteners shall be sufficient to transmit a force of 0.15 times the member design capacity in axial compression.

In addition, splices located between points of effective lateral support shall be designed for the design axial force (N^*) plus a design bending moment not less than the design bending moment (M^*)

where

$$M^* = \frac{\delta N^* L_s}{1000}$$

δ = appropriate amplification factor δ_b or δ_s determined in accordance with Clause 4.4

L_s = distance between points of effective lateral support.

When members are not prepared for full contact, the splice material and its fasteners shall be arranged to hold all parts in line and shall be designed to transmit a force of 0.3 times the member design capacity in axial compression.

(vi) Splices in flexural members—a bending moment of 0.3 times the member design capacity in bending. This provision shall not apply to splices designed to transmit shear force only.

A splice subjected to a shear force only shall be designed to transmit the design shear force together with any bending moment resulting from the eccentricity of the force with respect to the centroid of the connector group.

(vii) Splices in members subject to combined actions—a splice in a member subject to a combination of design axial tension or design axial compression and design bending moment shall satisfy (iv), (v) and (vi) simultaneously.

For earthquake load combinations, the design action effects specified in this Clause may need to be increased to meet the required behaviour of the steel frame and shall comply with Section 13.

9.1.5 Intersections Members or components meeting at a joint shall be arranged to transfer the design actions between the parts and, wherever practicable, with their centroidal axes meeting at a point. Where there is eccentricity at joints, the members and components shall be designed for the design bending moments which result.

The disposition of fillet welds to balance the design actions about the centroidal axis or axes for end connections of single angle, double angle and similar type members is not required for statically loaded members but is required for members and connection components subject to fatigue loading.

Eccentricity between the centroidal axes of angle members and the gauge lines for their bolted end connections may be neglected in statically loaded members, but must be considered in members and connection components subject to fatigue loading.

9.1.6 Choice of fasteners Where slip in the serviceability limit state must be avoided in a connection, high-strength bolts in a friction-type joint (bolting category 8.8/TF), fitted bolts or welds shall be used.

Where a joint is subject to impact or vibration, high-strength bolts in a friction-type joint (bolting category 8.8/TF), locking devices or welds shall be used.

9.1.7 Combined connections When non-slip fasteners (such as high-strength bolts in a friction-type connection or welds) are used in a connection in conjunction with slip-type fasteners (such as snug-tight bolts, or tensioned high-strength bolts in bearing-type connections), all of the design actions shall be assumed to be carried by the non-slip fasteners.

Where a mixture of non-slip fasteners is used, sharing of the design actions may be assumed. However, when welding is used in a connection in conjunction with other non-slip fasteners—

(a) any design actions initially applied directly to the welds shall not be assumed to be distributed to fasteners added after the application of the design actions; and

(b) any design actions applied after welding shall be assumed to be carried by the welds.

9.1.8 Prying forces Where bolts are required to carry a design tensile force, the bolts shall be proportioned to resist any additional tensile force due to prying action.

9.1.9 Connection components Connection components (cleats, gusset plates, brackets, etc) other than connectors shall have their capacities assessed using the provisions of Sections 5, 6, 7 and 8 as applicable.

9.1.10 Deductions for fastener holes

9.1.10.1 *Hole area* In calculating the deductions to be made for holes for fasteners (including countersunk holes), the gross areas of the holes in the plane of their axes shall be used.

9.1.10.2 *Holes not staggered* For holes that are not staggered, the area to be deducted shall be the maximum sum of the areas of the holes in any cross-sections at right angles to the direction of the design action in the member.

9.1.10.3 *Staggered holes* When holes are staggered, the area to be deducted shall be the greater of—

(a) the deduction for non-staggered holes; or

(b) the sum of the areas of all holes in any zig-zag line extending progressively across the member or part of the member, less ($s_p^2 t/4s_g$) for each gauge space in the chain of holes

where

s_p = staggered pitch, the distance measured parallel to the direction of the design action in the member, centre-to-centre of holes in consecutive lines, (see Figure 9.1.10.3(1))

t = thickness of the holed material

s_g = gauge, the distance, measured at right angles to the direction of the design action in the member, centre-to-centre of holes in consecutive lines, (see Figure 9.1.10.3(1)). For sections such as angles with holes in both legs, the gauge shall be taken as the sum of the back marks to each hole, less the leg thickness (see Figure 9.1.10.3(2)).

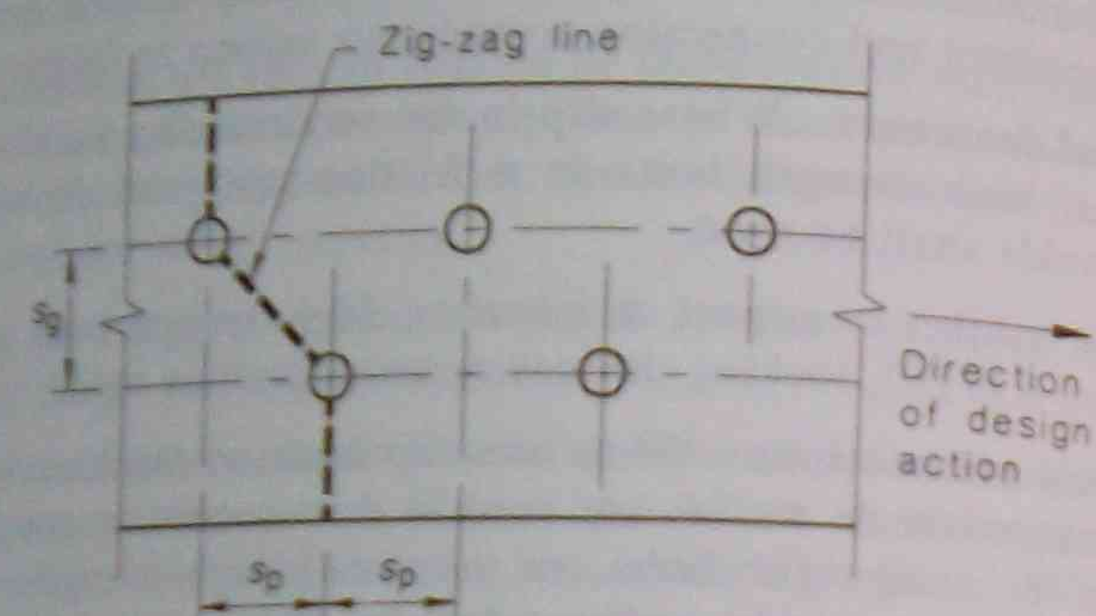


FIGURE 9.1.10.3(1) STAGGERED HOLES

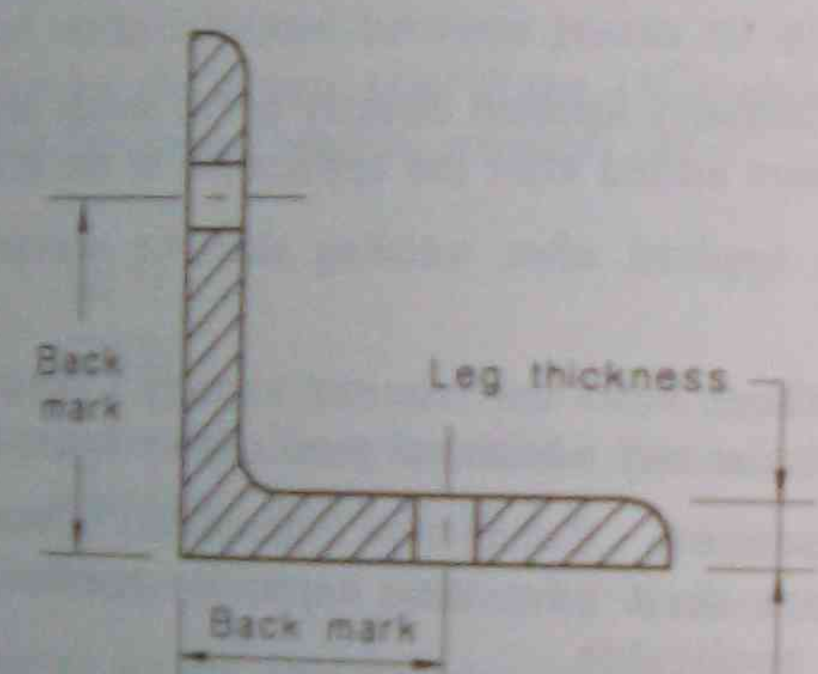


FIGURE 9.1.10.3(2) ANGLES WITH HOLES IN BOTH LEGS

9.1.11 Hollow section connections When design actions from one member are applied to a hollow section at a connection, consideration shall be given to the local effects on the hollow section.

9.2 DEFINITIONS

For the purpose of this Section, the definitions below apply.

Bearing-type connection—connection effected using either snug-tight bolts, or high-strength bolts tightened to induce a specified minimum bolt tension, in which the design action is transferred by shear in the bolts and bearing on the connected parts at the strength limit state.

Friction-type connection—connection effected using high-strength bolts tightened to induce a specified minimum bolt tension such that the resultant clamping action transfers the design shear forces at the serviceability limit state acting in the plane of the common contact surfaces by the friction developed between the contact surfaces.

Full tensioning—a method of installing and tensioning a bolt in accordance with Clauses 15.2.4 and 15.2.5.

In-plane loading—loading for which the design forces and bending moments are in the plane of the connection, such that the design action effects induced in the connection components are shear forces only.

Non-slip fasteners—fasteners which do not allow slip to occur between connected plates or members at the serviceability limit state so that the original alignment and relative positions are maintained.

Out-of-plane loading—loading for which the design forces or bending moments result in design action effects normal to the plane of the connection.

Pin—an unthreaded fastener manufactured out of round bar.

Prying force—additional tensile force developed as a result of the flexing of a connection component in a connection subjected to tensile force. External tension force reduces the contact pressure between the component and the base, and bending in part of the component develops a prying force near the edge of the connection component.

Snug tight—the tightness of a bolt achieved by a few impacts of an impact wrench or by the full effort of a person using a standard podger spanner.

9.3 DESIGN OF BOLTS

9.3.1 Bolts and bolting category The bolts and bolting categories listed in Table 9.3.1 shall be designed in accordance with this Clause and Clause 9.4.

Other grades of bolts conforming to AS 1110, AS 1111 and AS 1559 may be designed in accordance with the provisions of this Clause and Clause 9.4.

TABLE 9.3.1

BOLTS AND BOLTING CATEGORY

Bolting category	Bolt standard	Bolt grade	Method of tensioning	Minimum tensile strength (f_{uf}) (see Note 2) Bolting
4.6/S	AS 1111	4.6	Snug tight	400
8.8/S	AS 1252	8.8	Snug tight	830
8.8/TB	AS 1252	8.8	Full tensioning	830
8.8/TF (See Note 1)	AS 1252	8.8	Full tensioning	830

NOTES:

1 Special category used in connections where slip in the serviceability limit state is to be restricted (see Clauses 3.5.5 and 9.1.6).

2 f_{uf} is the minimum tensile strength of the bolt as specified in an Australian Standard.

9.3.2 Bolt strength limit states

9.3.2.1 *Bolt in shear* A bolt subject to a design shear force (V_f^*) shall satisfy—

$$V_f^* \leq \phi V_f$$

where

ϕ = capacity factor (see Table 3.4)

V_f = nominal shear capacity of a bolt.

The nominal shear capacity of a bolt (V_f) shall be calculated as follows:

$$V_f = 0.62 f_{uf} k_r (n_n A_c + n_x A_o)$$

where

f_{uf} = minimum tensile strength of the bolt as specified in an Australian Standard (see Table 9.3.1)

k_r = reduction factor given in Table 9.3.2.1 to account for the length of a bolted lap connection (L_j). For all other connections, $k_r = 1.0$

n_n = number of shear planes with threads intercepting the shear plane

A_c = minor diameter area of the bolt as defined in AS 1275

n_x = number of shear planes without threads intercepting the shear plane

A_o = nominal plain shank area of the bolt.

TABLE 9.3.2.1

REDUCTION FACTOR FOR A BOLTED LAP CONNECTION (k_r)

Length mm	$L_j < 300$	$300 \leq L_j \leq 1300$	$L_j > 1300$
k_r	1.0	$1.075 - L_j/4000$	0.75

9.3.2.2 *Bolt in tension* A bolt subject to a design tension force (N^*_{tf}) shall satisfy—

$$N^*_{tf} \leq \phi N_{tf}$$

where

ϕ = capacity factor (see Table 3.4)

N_{tf} = nominal tension capacity of a bolt.

The nominal tension capacity of a bolt (N_{tf}) shall be calculated as follows:

$$N_{tf} = A_s f_{uf}$$

where A_s is the tensile stress area of a bolt as specified in AS 1275.

9.3.2.3 *Bolt subject to combined shear and tension* A bolt required to resist both design shear (V^*_f) and design tension forces (N^*_{tf}) at the same time shall satisfy—

$$\left(\frac{V^*_f}{\phi V_f}\right)^2 + \left(\frac{N^*_{tf}}{\phi N_{tf}}\right)^2 \leq 1.0$$

where

ϕ = capacity factor (see Table 3.4)

V_f = nominal shear capacity calculated in accordance with Clause 9.3.2.1

N_{tf} = nominal tension capacity calculated in accordance with Clause 9.3.2.2

9.3.2.4 *Ply in bearing* A ply subject to a design bearing force (V^*_b) due to a bolt in shear shall satisfy—

$$V^*_b \leq \phi V_b$$

where

ϕ = capacity factor (see Table 3.4)

V_b = nominal bearing capacity of a ply.

The nominal bearing capacity of a ply (V_b) shall be calculated as follows:

$$V_b = 3.2 d_f t_p f_{up} \quad 9.3.2.4(1)$$

provided that, for a ply subject to a component of force acting towards an edge, the nominal bearing capacity of a ply (V_b) shall be the lesser of that given by Equation 9.3.2.4(1) and that given by Equation 9.3.2.4(2)—

$$V_b = a_e t_p f_{up} \quad 9.3.2.4(2)$$

where

d_f = diameter of the bolt

t_p = thickness of the ply

f_{up} = tensile strength of the ply

a_e = minimum distance from the edge of a hole to the edge of a ply, measured in the direction of the component of a force, plus half the bolt diameter. The edge of a ply shall be deemed to include the edge of an adjacent bolt hole.

9.3.2.5 *Filler plates* For connections in which filler plates exceed 6 mm in thickness but are less than 20 mm in thickness, the nominal shear capacity of a bolt (V_f) specified in Clause 9.3.2.1 shall be reduced by 15 percent. For multi-shear plane connections with more than one filler plate through which a bolt passes, the reduction shall be determined using the maximum thickness of filler plate on any shear plane through which the bolt passes.

9.3.3 Bolt serviceability limit state

9.3.3.1 *Design* For friction-type connections (bolting category 8.8/TF) in which slip in the serviceability limit state is required to be limited, a bolt subjected only to a design shear force (V^*_{sf}) in the plane of the interfaces shall satisfy—

$$V^*_{sf} \leq \phi V_{sf}$$

where

ϕ = capacity factor (see Clause 3.5.5)

V_{sf} = nominal shear capacity of a bolt, for a friction-type connection.

The nominal shear capacity of a bolt (V_{sf}) shall be calculated as follows:

$$V_{sf} = \mu n_{ei} N_{ti} k_h$$

where

μ = slip factor as defined in Clause 9.3.3.2

n_{ei} = number of effective interfaces

N_{ti} = minimum bolt tension at installation as specified in Clause 15.2.5.1.

k_h = factor for different hole types, as specified in Clause 14.3.5.2.

= 1.0 for standard holes

= 0.85 for short slotted and oversize holes

= 0.70 for long slotted holes.

The strength limit state shall be separately assessed in accordance with Clause 9.3.2.

9.3.3.2 *Contact surfaces* Where the surfaces in contact are clean 'as-rolled' surfaces, the slip factor (μ) shall be taken as 0.35. If any applied finish, or other surface condition including a machined surface, is used, the slip factor shall be based upon test evidence. Tests performed in accordance with the procedure specified in Appendix J shall be deemed to provide satisfactory test evidence.

A connection involving 8.8/TF bolting category shall be identified as such, and the drawings shall clearly indicate the surface treatment required at such a connection and whether masking of the connection surfaces is required during painting operations (see Clause 14.3.6.3).

9.3.3.3 *Combined shear and tension* Bolts in a connection for which slip in the serviceability limit state must be limited, which are subject to a design tension force (N^*_{tf}) shall satisfy—

$$\left(\frac{V^*_{sf}}{\phi V_{sf}}\right) + \left(\frac{N^*_{tf}}{\phi N_{tf}}\right) \leq 1.0$$

where

V^*_{sf} = design shear force on the bolt in the plane of the interfaces

N^*_{tf} = design tension force on the bolt

ϕ = capacity factor (see Clause 3.5.5)

V_{sf} = nominal shear capacity of the bolt as specified in Clause 9.3.3.1

N_{tf} = nominal tension capacity of the bolt.

The nominal tension capacity of the bolt (N_{tf}) shall be taken as—

$N_{tf} = N_{ti}$

where N_{ti} is the minimum bolt tension at installation as specified in Clause 15.2.5.1.

The strength limit state shall also be separately assessed in accordance with Clause 9.3.2.3.

9.4 ASSESSMENT OF THE STRENGTH OF A BOLT GROUP

9.4.1 Bolt group subject to in-plane loading The design actions in a bolt group shall be determined by an analysis based on the following assumptions:

(a) The connection plates shall be considered to be rigid and to rotate relative to each other about a point known as the instantaneous centre of the bolt group.

(b) In the case of a bolt group subject to a pure couple only, the instantaneous centre of rotation coincides with the bolt group centroid.

In the case of a bolt group subject to an in-plane shear force applied at the group centroid, the instantaneous centre of notation is at infinity and the design shear force is uniformly distributed throughout the group.

In all other cases, either the results of independent analyses for a pure couple alone and for an in-plane shear force applied at the bolt group centroid shall be superposed, or a recognized method of analysis shall be used.

(c) The design shear force in each bolt shall be assumed to act at right angles to the radius from the bolt to the instantaneous centre, and shall be taken as proportional to that radius.

Each bolt shall satisfy the requirements of Clause 9.3.2.1 using the capacity factor (ϕ) for a bolt group (see Table 3.4) and the ply in bearing shall satisfy Clause 9.3.2.4.

9.4.2 Bolt group subject to out-of-plane loading The design actions in any bolt in a bolt group subject to out-of-plane loading shall be determined in accordance with Clause 9.1.3.

Each bolt shall comply with Clauses 9.3.2.1, 9.3.2.2 and 9.3.2.3 using the capacity factor (ϕ) for a bolt group (see Table 3.4), and the ply in bearing shall comply with Clause 9.3.2.4.

9.4.3 Bolt group subject to combinations of in-plane and out-of-plane loadings The design actions in any bolt in a bolt group shall be determined in accordance with Clauses 9.4.1 and 9.4.2.

Each bolt shall comply with Clauses 9.3.2.1, 9.3.2.2 and 9.3.2.3 using the capacity factor (ϕ) for a bolt group (see Table 3.4), and the ply in bearing shall comply with Clause 9.3.2.4.

9.5 DESIGN OF A PIN CONNECTION

9.5.1 Pin in shear A pin subject to a design shear force (V^*_f) shall satisfy—

$V^*_f \leq \phi V_f$

where

ϕ = capacity factor (see Table 3.4)

V_f = nominal shear capacity of the pin.

The nominal shear capacity of a pin (V_f) shall be calculated as follows:

$V_f = 0.62 f_{yp} n_s A_p$

where

f_{yp} = yield stress of the pin

n_s = number of shear planes

A_p = cross-sectional area of the pin

9.5.2 Pin in bearing A pin subject to a design bearing force (V^*_b) shall satisfy—

$V^*_b \leq \phi V_b$

where

ϕ = capacity factor (see Table 3.4)

V_b = nominal bearing capacity of the pin.

The nominal bearing capacity of a pin (V_b) shall be calculated as follows:

$V_b = 1.4 f_{yp} d_f t_p k_p$

where

f_{yp} = yield stress of the pin

d_f = pin diameter

t_p = connecting plate thickness(es)

k_p = 1.0 for pins without rotation, or

= 0.5 for pins with rotation.

9.5.3 Pin in bending A pin subject to a design bending moment (M^*) shall satisfy—

$M^* \leq \phi M_p$

where

ϕ = capacity factor (see Table 3.4)

M_p = nominal moment capacity of the pin.

The nominal moment capacity of a pin (M_p) shall be calculated as follows:

$$M_p = f_{yp}S$$

where

f_{yp} = yield stress of the pin

S = plastic section modulus of the pin.

9.5.4 Ply in bearing A ply subject to a design bearing force (V_b^*) due to a pin in shear shall satisfy Clause 9.3.2.4.

9.6 DESIGN DETAILS FOR BOLTS AND PINS

9.6.1 Minimum pitch The distance between centres of fastener holes shall be not less than 2.5 times the nominal diameter of the fastener (d_f).

NOTE: The minimum pitch may also be affected by Clause 9.3.2.4.

9.6.2 Minimum edge distance The minimum edge distance shall be as specified in Table 9.6.2, where d_f is the nominal diameter of the fastener. The edge distance shall be the distance from the nearer edge of a hole to the physical edge of a plate or rolled section, plus half the fastener diameter (d_f).

TABLE 9.6.2

MINIMUM EDGE DISTANCE

Sheared or hand flame cut edge	Rolled plate, flat bar or section: machine flame cut, sawn or planed edge	Rolled edge of a rolled flat bar or section
$1.75d_f$	$1.50d_f$	$1.25d_f$

NOTE: The edge distance may also be affected by Clause 9.3.2.4.

9.6.3 Maximum pitch The maximum distance between centres of fasteners shall be the lesser of $15t_p$ (where t_p = thickness of thinner ply connected) or 200 mm. However, in the following cases, the maximum distances shall be as follows:

(a) For fasteners which are not required to carry design actions in regions not liable to corrosion—the lesser of $32t_p$ or 300 mm.

(b) For an outside line of fasteners in the direction of the design action—the lesser of $(4t_p + 100)$ mm, or 200 mm.

9.6.4 Maximum edge distance The maximum distance from the centre of any fastener to the nearest edge of parts in contact with one another shall be 12 times the thickness of the thinnest outer connected ply under consideration, but shall not exceed 150 mm.

9.6.5 Holes Holes for bolts shall comply with Clause 14.3.5 and holes for pins shall comply with Clause 14.3.7.

9.7 DESIGN OF WELDS

9.7.1 Scope

9.7.1.1 *General* Welding shall comply with AS 1554.1, AS 1554.2 or AS 1554.5, as appropriate.

9.7.1.2 *Weld types* For the purpose of this Standard, welds shall be butt, fillet, slot or plug welds, or compound welds.

9.7.1.3 *Weld quality* Weld quality shall be either SP or GP as specified in AS 1554.1, except that where a higher quality weld is required by Clause 11.1.5, weld quality conforming with AS 1554.5 shall be used. Weld quality shall be specified on the design drawings.

9.7.2 Complete and incomplete penetration butt welds

9.7.2.1 *Definitions* For the purpose of this Clause, the definitions below apply.

Complete penetration butt weld—a butt weld in which fusion exists between the weld and parent metal throughout the complete depth of the joint.

Incomplete penetration butt weld—a butt weld in which fusion exists over less than the complete depth of the joint.

Prequalified weld preparation—a joint preparation prequalified in terms of AS 1554.1.

9.7.2.2 *Size of weld* The size of a complete penetration butt weld, other than a complete penetration butt weld in a T-joint or a corner joint, and the size of an incomplete penetration butt weld shall be the minimum depth to which the weld extends from its face into a joint, exclusive of reinforcement.

The size of a complete penetration butt weld for a T-joint or a corner joint shall be the thickness of the part whose end or edge butts against the face of the other part.

9.7.2.3 *Design throat thickness* Design throat thickness shall be as follows:

(a) *Complete penetration butt weld* The design throat thickness for a complete penetration butt weld shall be the size of the weld.

(b) *Incomplete penetration butt weld* The design throat thickness for an incomplete penetration butt weld shall be as follows:

(i) Prequalified preparation for incomplete penetration butt weld except as otherwise provided in (iii), as specified in AS 1554.1.

(ii) Non-prequalified preparation for incomplete penetration butt weld except as provided in (iii)—

(A) where $\theta < 60°$ ... $(d - 3)$ mm, for single V weld; $[(d_3 + d_4) - 6]$ mm, for double V weld.

(B) where $\theta > 60°$ ... d mm, for single V weld; $(d_3 + d_4)$ mm, for double V weld.

where

d = depth of preparation (d_3 and d_4 are the values of d for each side of the weld)

θ = angle of preparation.

(iii) For an incomplete penetration butt weld made by an automatic arc welding process for which it can be demonstrated by means of a macro test on a production weld that the required penetration has been achieved, an increase in design throat thickness up to the depth of preparation may be allowed. If the macro test shows penetration beyond the depth of preparation, an increase in design throat thickness up to that shown in Figure 9.7.3.4 may be allowed.

NOTE: It is only necessary to specify the design throat thickness required, leaving the fabricator to determine the welding procedure necessary to achieve the specified design throat thickness.

9.7.2.4 *Effective length* The effective length of a butt weld shall be the length of the continuous full size weld.

9.7.2.5 *Effective area* The effective area of a butt weld shall be the product of the effective length and the design throat thickness.

9.7.2.6 *Transition of thickness or width* Butt welded joints between parts of different thickness or unequal width that are subject to tension shall have a smooth transition between surfaces or edges. The transition shall be made by chamfering the thicker part or by sloping the weld surfaces or by any combination of those, as shown in Figure 9.7.2.6.

The transition slope between the parts shall not exceed 1:1. However, the provisions of Section 11 require a lesser slope than this or a curved transition between the parts for some fatigue detail categories.

9.7.2.7 *Strength assessment of a butt weld* The assessment of a butt weld for the strength limit state shall be as follows:

(a) *Complete penetration butt weld* The design capacity of a complete penetration butt weld shall be taken as equal to the nominal capacity of the weaker part of the parts joined, multiplied by the appropriate capacity factor (ϕ) for butt welds given in Table 3.4, provided that the welding procedures are qualified in accordance with AS 1554.1.

The butt weld shall be made using a welding consumable which will produce butt tensile test specimens in accordance with AS 2205.2.1 for which the minimum strength is not less than that given in Table 2.1 for the parent material.

(b) *Incomplete-penetration butt weld* The design capacity of an incomplete-penetration butt weld shall be calculated as for a fillet weld (see Clause 9.7.3.10) using the design throat thickness determined in accordance with Clause 9.7.2.3(b).

9.7.3 Fillet welds

9.7.3.1 *Size of a fillet weld* The size of a fillet weld shall be specified by the leg lengths. The leg lengths shall be defined as the lengths (t_{w1}, t_{w2}) of the sides lying along the legs of a triangle inscribed within the cross-section of the weld (see Figures 9.7.3.1(a) and (b)). When the legs are of equal length, the size shall be specified by a single dimension (t_w).

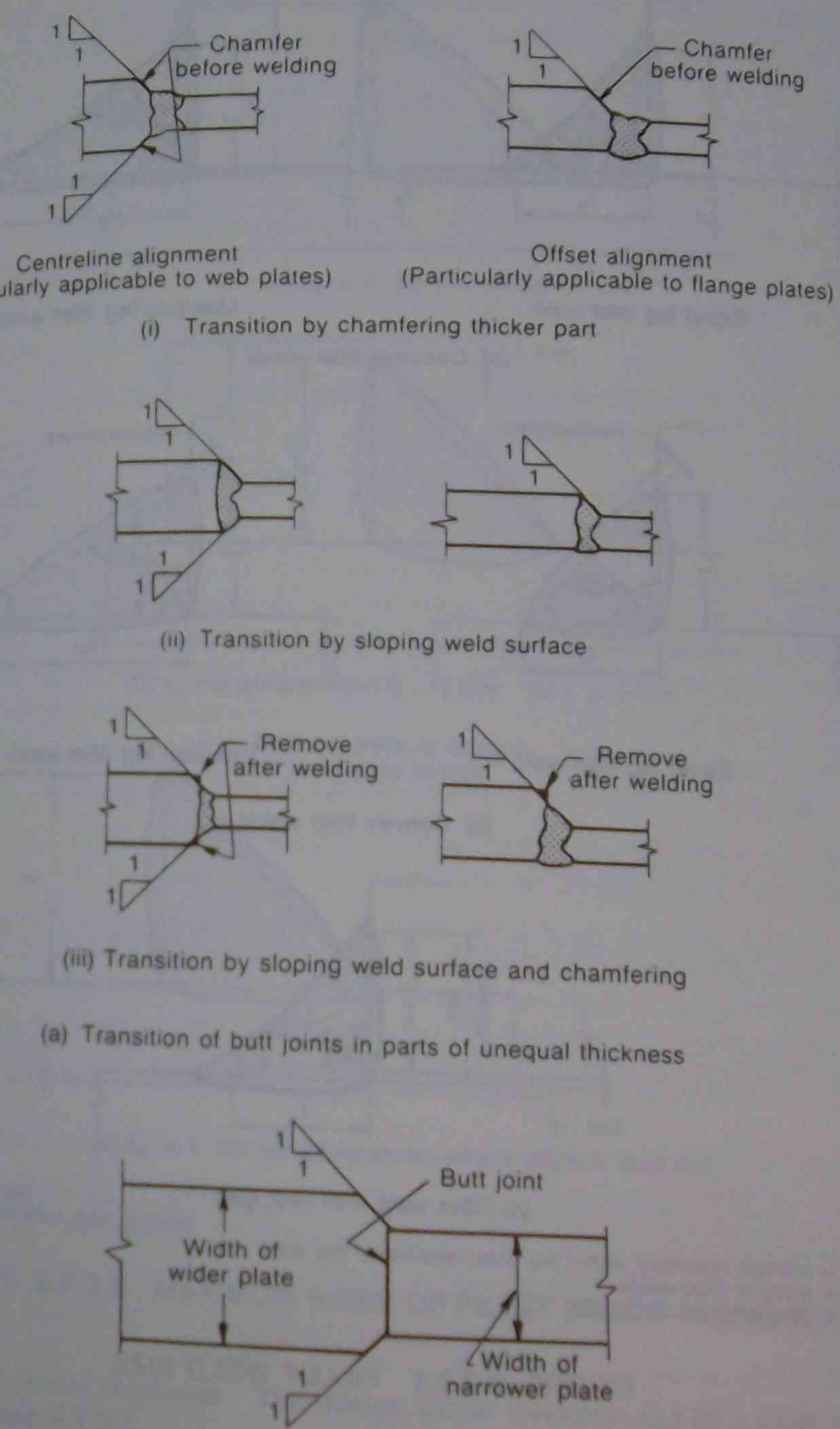
Where there is a root gap, the size (t_w) shall be given by the lengths of the legs of the inscribed triangle reduced by the root gap as shown in Figure 9.7.3.1(c).

NOTE: The preferred sizes of a fillet weld less than 15 mm are 3, 4, 5, 6, 8, 10 and 12 mm.

9.7.3.2 *Minimum size of a fillet weld* The minimum size of a fillet weld, other than a fillet weld used to reinforce a butt weld, shall conform with Table 9.7.3.2, except that the size of the weld need not exceed the thickness of the thinner part joined.

TABLE 9.7.3.2
MINIMUM SIZE OF A FILLET WELD

Thickness of thickest part (t) millimetres	Minimum size of a fillet weld (t_w) millimetres
$t \le 7$	3
$7 < t \le 10$	4
$10 < t \le 15$	5
$15 < t$	6



Centreline alignment (Particularly applicable to web plates)

Offset alignment (Particularly applicable to flange plates)

(i) Transition by chamfering thicker part

(ii) Transition by sloping weld surface

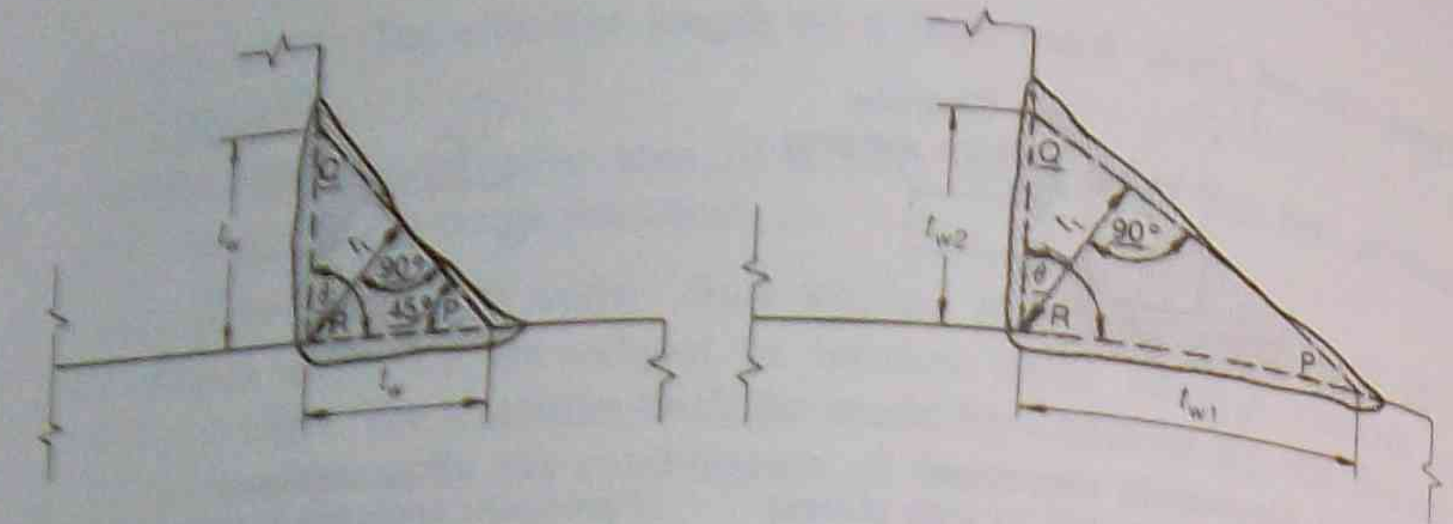
(iii) Transition by sloping weld surface and chamfering

(a) Transition of butt joints in parts of unequal thickness

(b) Transition of butt joints in parts of unequal width—Transition by chamfering wider part

NOTE: Transition slopes shown in (a) and (b) are the maximum permitted.

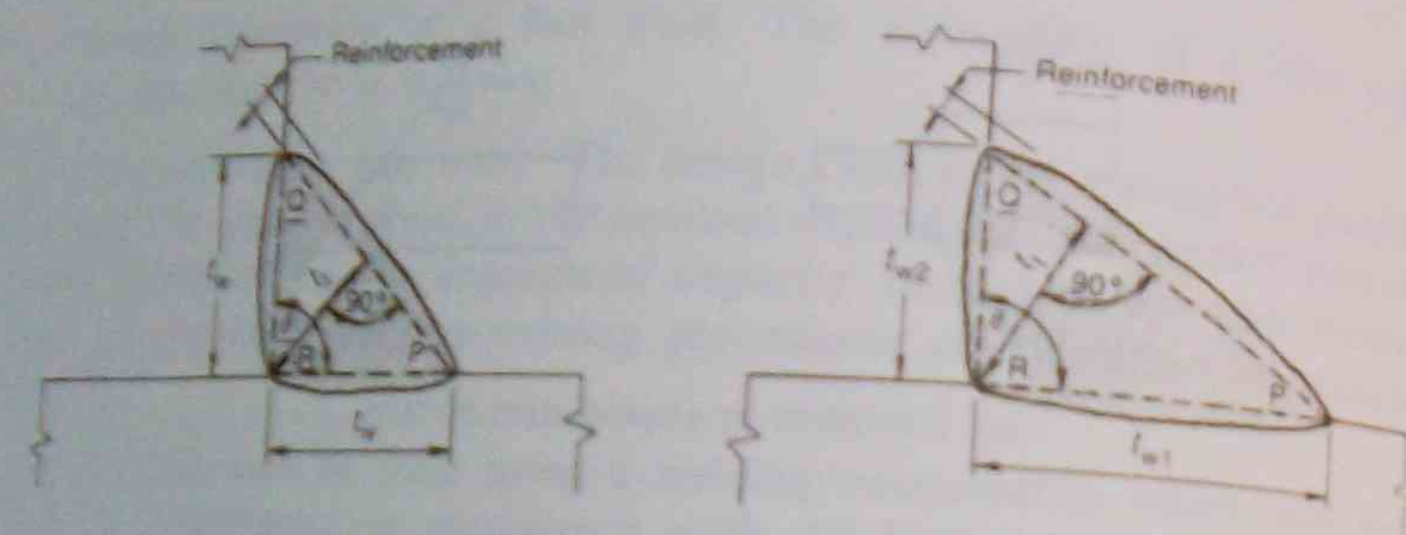
FIGURE 9.7.2.6 TRANSITIONS OF THICKNESS OR WIDTH FOR BUTT WELDS SUBJECT TO TENSION



Equal leg fillet weld

Unequal leg fillet weld

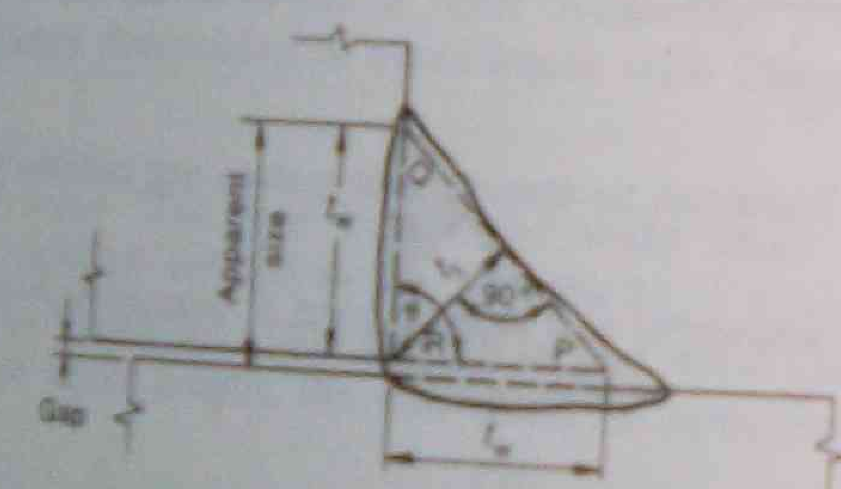
(a) Concave fillet welds



Equal leg fillet weld

Unequal leg fillet weld

(b) Convex fillet welds



(c) Fillet weld with root gap

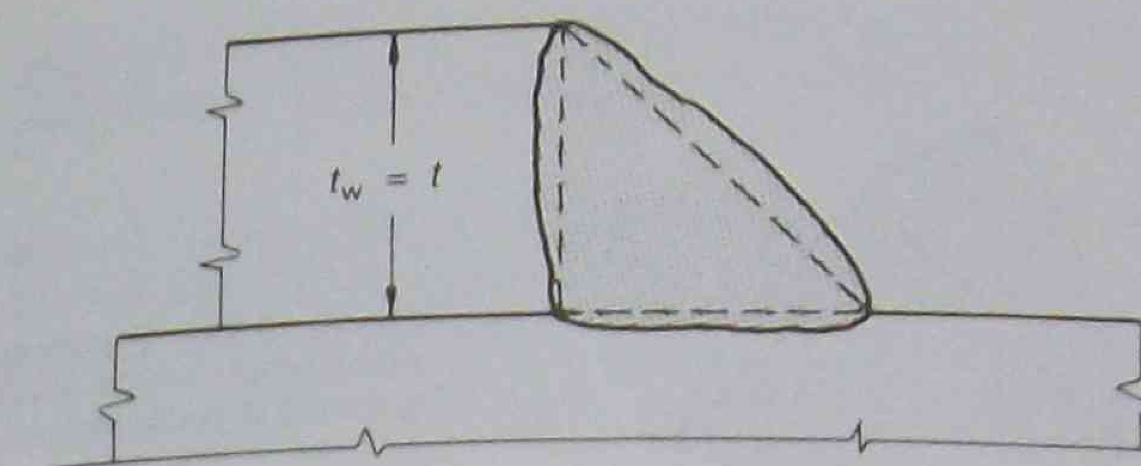
LEGEND:
PQR = triangle inscribed within the cross-section of the weld
t_w, t_{w1}, t_{w2} = sizes of fillet welds
t_t = design throat thickness

FIGURE 9.7.3.1 FILLET WELD SIZE

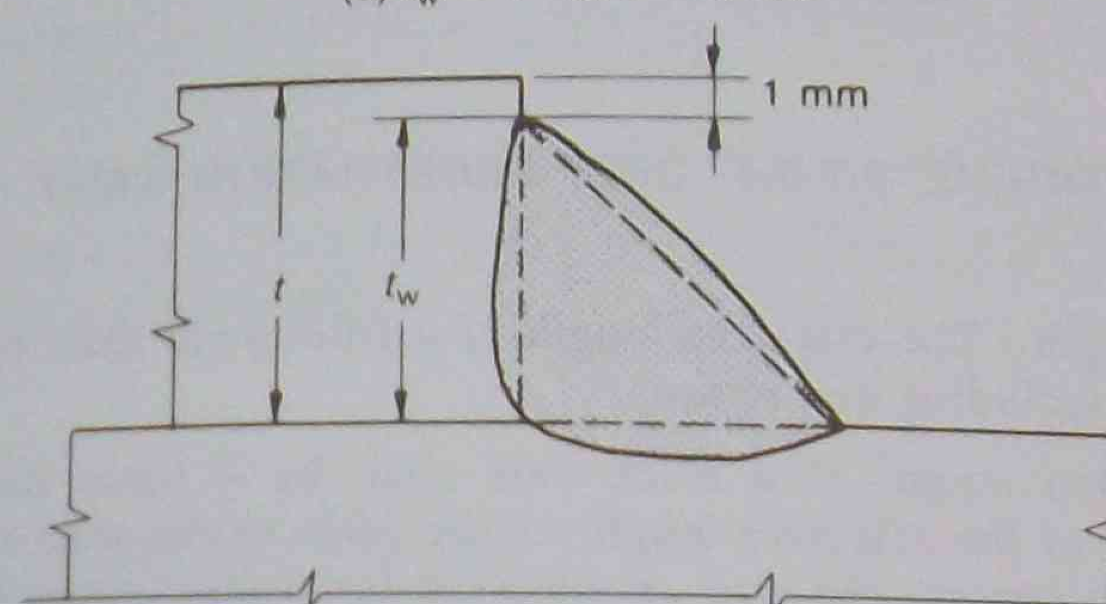
9.7.3.3 *Maximum size of a fillet weld along an edge* The maximum size of a fillet weld along an edge of material shall be—

(a) for material less than 6 mm in thickness, the thickness of the material (see Figure 9.7.3.3(a));

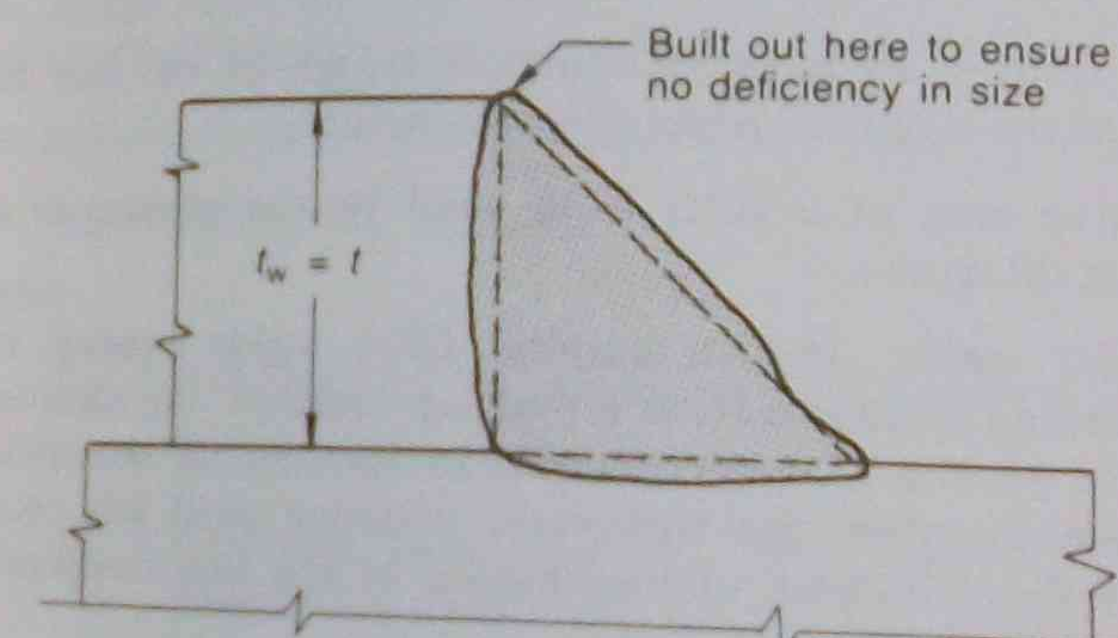
(b) for material 6 mm or more in thickness (see Figure 9.7.3.3(b)), unless the weld is designated on the drawing to be built out to obtain the design throat thickness (see Figure 9.7.3.3(c)), 1 mm less than the thickness of the material.



(a) $t_w = t$ for $t < 6$ mm



(b) t_w not greater than $(t - 1)$ mm for $t \geq 6$ mm



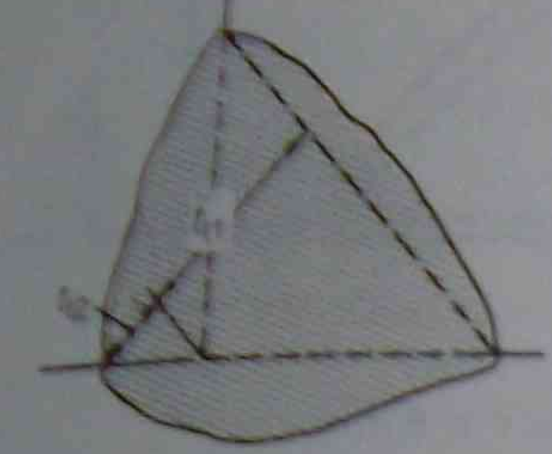
(c) $t_w = t$ for all thicknesses where edge is built out

LEGEND:
t_w = size of fillet weld
t = thickness of thinner part joined

FIGURE 9.7.3.3 MAXIMUM SIZES OF FILLET WELDS ALONG EDGES

9.7.3.4 *Design throat thickness* The design throat thickness (t_t) of a fillet weld shall be as shown in Figure 9.7.3.1.

For a weld made by an automatic arc welding process, an increase in design throat thickness may be allowed as shown in Figure 9.7.3.4, provided that it can be demonstrated by means of a macro test on a production weld that the required penetration has been achieved. Where such penetration is achieved, the size of weld required may be correspondingly reduced to give the specified design throat thickness.



Design throat thickness for deep penetration welds made by automatic processes:
$t_t = t_{t1} + 0.85t_{t2}$

FIGURE 9.7.3.4 DEEP PENETRATION WELD

9.7.3.5 *Effective length* The effective length of a fillet weld shall be the overall length of the full-size fillet, including end returns.

The minimum effective length of a fillet weld shall be 4 times the size of the weld. However, if the ratio of the effective length of the weld to the size of the weld does not comply with this requirement, the size of the weld for design purposes shall be taken as 0.25 times the effective length. The minimum length requirement shall also apply to lap joints.

Any segment of intermittent fillet weld shall have an effective length of not less than 40 mm or 4 times the nominal size of the weld, whichever is the greater.

9.7.3.6 *Effective area* The effective area of a fillet weld shall be the product of the effective length and the design throat thickness.

9.7.3.7 *Transverse spacing of fillet welds* If two parallel fillet welds connect two components in the direction of the design action to form a built-up member, the transverse distance between the welds shall not exceed $32t_p$, except that in the case of intermittent fillet welds at the ends of a tension member, the transverse distance shall not exceed either $16t_p$ or 200 mm, where t_p is the thickness of the thinner of the two components connected.

It shall be permissible to use fillet welds in slots and holes in the direction of the design action in order to satisfy this Clause.

9.7.3.8 *Intermittent fillet welds* Except at the ends of a built-up member, the clear spacing between the lengths of consecutive collinear intermittent fillet welds shall not exceed the lesser of—

(a) for elements in compression . . . $16t_p$ and 300 mm; and

(b) for elements in tension . . . . $24t_p$ and 300 mm.

9.7.3.9 *Built-up members—intermittent fillet welds* If intermittent fillet welds connect components forming a built-up member, the welds shall comply with the following requirements:

(a) At the ends of a tension or compression component of a beam, or at the ends of a tension member, when side fillets are used alone, they shall have a length along each joint line at least equal to the width of the connected component. If the connected component is tapered, the length of weld shall be the greater of—

(i) the width of the widest part; and

(ii) the length of the taper.

(b) At the cap plate or baseplate of a compression member, welds shall have a length along each joint line of at least the maximum width of the member at the contact face.

(c) Where a beam is connected to the face of a compression member, the welds connecting the compression member components shall extend between the levels of the top and bottom of the beam and in addition—

(i) for an unrestrained connection, a distance (d) below the lower face of the beam; and

(ii) for a restrained connection, a distance (d) above and below the upper and lower faces of the beam,

where d is the maximum cross-sectional dimension of the compression member.

9.7.3.10 *Strength limit state for fillet weld* A fillet weld subject to a design force per unit length of weld (v^*_w) shall satisfy—

$$v^*_w \le \phi v_w$$

where

ϕ = capacity factor (see Table 3.4)

v_w = nominal capacity of a fillet weld per unit length.

The design force per unit length (v^*_w) shall be the ectorial sum of the design forces per unit length on the effective area of the weld.

The nominal capacity of a fillet weld per unit length (v_w) shall be calculated as follows:

$$v_w = 0.6 f_{uw} t_t k_r$$

where

f_{uw} = nominal tensile strength of weld metal (see Table 9.7.3.10(1))

t_t = design throat thickness

k_r = reduction factor given in Table 9.7.3.10(2) to account for the length of a welded lap connection (L_w). For all other connection types, k_r = 1.0.

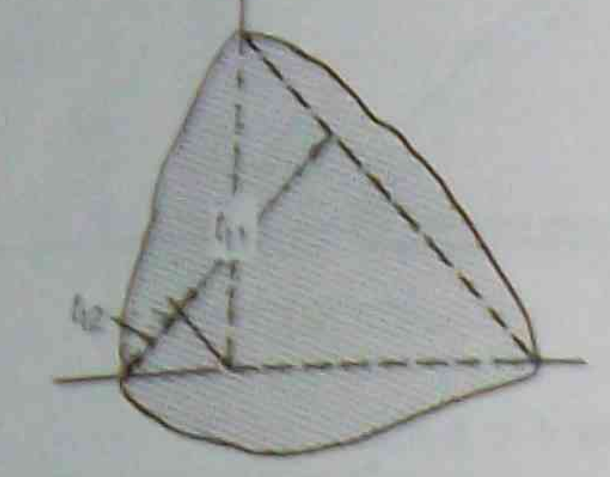
TABLE 9.7.3.10(1)
NOMINAL TENSILE STRENGTH OF WELD METAL (f_{uw})

Manual metal arc electrode (AS 1553.1)	Submerged arc (AS 1858.1) Flux cored arc (AS 2203) Gas metal arc (AS 2717.1)	Nominal tensile strength of weld metal (f_{uw}) Bolting
E41XX	W40X (see Note)	410
E48XX	W50X	480

NOTE: Not included in AS 2717.1.

TABLE 9.7.3.10(2)
REDUCTION FACTOR FOR A WELDED LAP CONNECTION (k_r)

Length of weld (L_w) metres	$L_w \le 1.7$	$1.7 < L_w \le 8.0$	$L_w > 8.0$
k_r	1.00	$1.10 - 0.06L_w$	0.62



Design throat thickness for deep penetration welds made by automatic processes:
$t_t = t_{t1} + 0.85t_{t2}$

FIGURE 9.7.3.4 DEEP PENETRATION WELD

9.7.3.5 *Effective length* The effective length of a fillet weld shall be the overall leng of the full-size fillet, including end returns.

The minimum effective length of a fillet weld shall be 4 times the size of the we However, if the ratio of the effective length of the weld to the size of the weld does comply with this requirement, the size of the weld for design purposes shall be taken 0.25 times the effective length. The minimum length requirement shall also apply to joints.

Any segment of intermittent fillet weld shall have an effective length of not less th 40 mm or 4 times the nominal size of the weld, whichever is the greater.

9.7.3.6 *Effective area* The effective area of a fillet weld shall be the product of th effective length and the design throat thickness.

9.7.3.7 *Transverse spacing of fillet welds* If two parallel fillet welds connect tw components in the direction of the design action to form a built-up member, the transver distance between the welds shall not exceed $32t_p$, except that in the case of intermitte fillet welds at the ends of a tension member, the transverse distance shall not exce either $16t_p$ or 200 mm, where t_p is the thickness of the thinner of the two compone connected.

It shall be permissible to use fillet welds in slots and holes in the direction of the desi action in order to satisfy this Clause.

9.7.3.8 *Intermittent fillet welds* Except at the ends of a built-up member, the cle spacing between the lengths of consecutive collinear intermittent fillet welds shall n exceed the lesser of—

(a) for elements in compression . . . $16t_p$ and 300 mm; and

(b) for elements in tension . . . . $24t_p$ and 300 mm.

9.7.3.9 *Built-up members—intermittent fillet welds* If intermittent fillet welds conn components forming a built-up member, the welds shall comply with the follow requirements:

(a) At the ends of a tension or compression component of a beam, or at the ends of tension member, when side fillets are used alone, they shall have a length alo each joint line at least equal to the width of the connected component. If th connected component is tapered, the length of weld shall be the greater of—

(i) the width of the widest part; and

(ii) the length of the taper.

(b) At the cap plate or baseplate of a compression member, welds shall have a length along each joint line of at least the maximum width of the member at the contact face.

(c) Where a beam is connected to the face of a compression member, the welds connecting the compression member components shall extend between the levels of the top and bottom of the beam and in addition—

(i) for an unrestrained connection, a distance (d) below the lower face of the beam; and

(ii) for a restrained connection, a distance (d) above and below the upper and lower faces of the beam,

where d is the maximum cross-sectional dimension of the compression member.

9.7.3.10 *Strength limit state for fillet weld* A fillet weld subject to a design force per unit length of weld (v^*_w) shall satisfy—

$$v^*_w \le \phi v_w$$

where

ϕ = capacity factor (see Table 3.4)

v_w = nominal capacity of a fillet weld per unit length.

The design force per unit length (v^*_w) shall be the ectorial sum of the design forces per unit length on the effective area of the weld.

The nominal capacity of a fillet weld per unit length (v_w) shall be calculated as follows:

$$v_w = 0.6 f_{uw} t_t k_r$$

where

f_{uw} = nominal tensile strength of weld metal (see Table 9.7.3.10(1))

t_t = design throat thickness

k_r = reduction factor given in Table 9.7.3.10(2) to account for the length of a welded lap connection (L_w). For all other connection types, k_r = 1.0.

TABLE 9.7.3.10(1)

NOMINAL TENSILE STRENGTH OF WELD METAL (f_{uw})

Manual metal arc electrode (AS 1553.1)	Submerged arc (AS 1858.1) Flux cored arc (AS 2203) Gas metal arc (AS 2717.1)	Nominal tensile strength of weld metal (f_{uw}) Bolting
E41XX	W40X (see Note)	410
E48XX	W50X	480

NOTE: Not included in AS 2717.1.

TABLE 9.7.3.10(2)

REDUCTION FACTOR FOR A WELDED LAP CONNECTION(k_r)

Length of weld (L_w) metres	$L_w \le 1.7$	$1.7 < L_w \le 8.0$	$L_w > 8.0$
k_r	1.00	$1.10 - 0.06L_w$	0.62

9.7.4 Plug and slot welds

9.7.4.1 *Plug and slot welds in the form of fillet welds around the circumference of the hole or slot* These plug and slot welds shall be regarded as a fillet weld with an effective length as defined in Clause 9.7.3.5, and a nominal capacity as defined in Clause 9.7.3.10. The minimum size shall be as for a fillet weld (see Clause 9.7.3.2.).

9.7.4.2 *Plug and slot welds in hole filled with weld metal* The effective shear area (A_w) of a plug or slot weld in a hole filled with weld metal shall be considered as the nominal cross-sectional area of the hole or slot in the plane of the faying surface.

Such a plug or slot weld subject to a design shear force (V^*_w) shall satisfy—

$$V^*_w \leq \phi V_w$$

where

ϕ = capacity factor (see Table 3.4)

V_w = nominal shear capacity of the weld.

The nominal shear capacity (V_w) of the weld shall be calculated as follows:

$$V_w = 0.60 f_{uw} A_w$$

9.7.4.3 *Limitations* Plug or slot welds may only be used to transmit shear in lap joints or to prevent buckling of lapped parts or to join component parts of built-up members.

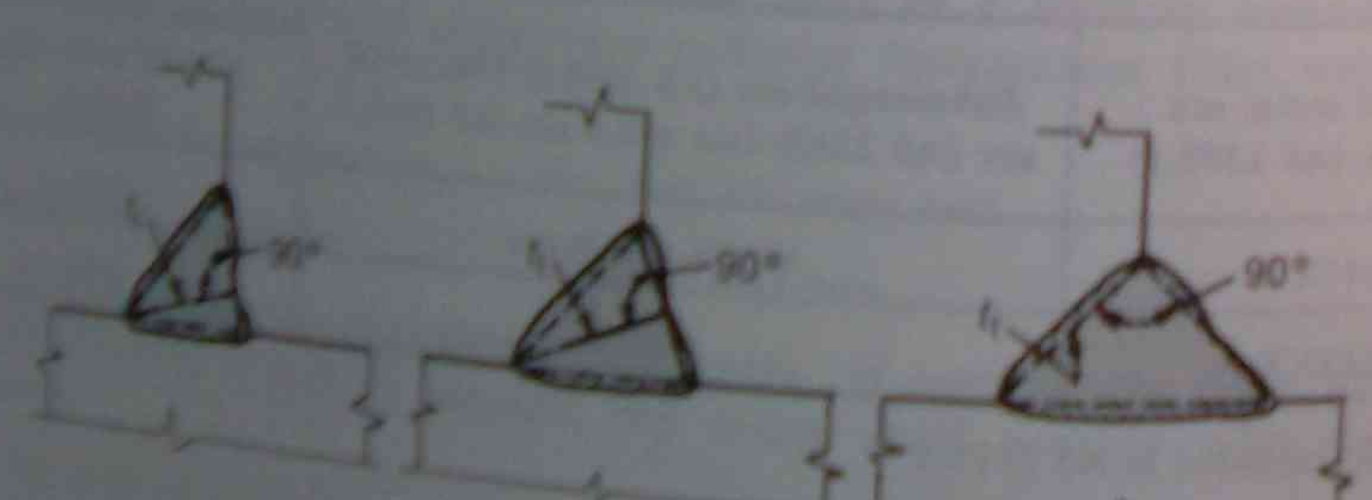
9.7.5 Compound weld

9.7.5.1 *Description of a compound weld* A compound weld, defined as a fillet weld superimposed on a butt weld, shall be as specified in AS 1101.3.

9.7.5.2 *Design throat thickness* The design throat thickness of a compound weld, for use in design calculations, shall be—

(a) for a complete penetration butt weld, the size of the butt weld without reinforcement; and

(b) for an incomplete penetration butt weld, the shortest distance from the root of the incomplete penetration butt weld to the face of the fillet weld as determined by the largest inscribed triangle in the total weld cross-section, with a maximum value equal to the thickness of the part whose end or edge butts against the face of the other part (see Figure 9.7.5.2).



NOTE: The design throat thickness (t_t) of a weld is the minimum distance from the root of a weld to its face less any reinforcement. The three sketches above illustrate this concept.

FIGURE 9.7.5.2 DESIGN THROAT THICKNESSES OF COMPOUND WELDS

9.7.5.3 *Strength limit state* The weld shall satisfy the requirements of Clause 9.7.2.7.

9.8 ASSESSMENT OF THE STRENGTH OF A WELD GROUP

9.8.1 Weld group subject to in-plane loading

9.8.1.1 *General method of analysis* The design force per unit length in a fillet weld group subject to in-plane loading shall be determined in accordance with the following:

(a) The connection plates shall be considered to be rigid and to rotate relative to each other about a point known as the instantaneous centre of rotation of the weld group.

(b) In the case of a weld group subject to a pure couple only, the instantaneous centre of rotation coincides with the weld group centroid.

In the case of a weld group subject to an in-plane shear force applied at the group centroid, the instantaneous centre of the rotation is at infinity and the design force per unit length (v^*_w) is uniformly distributed throughout the group.

In all other cases, either the results of independent analyses for a pure couple alone and for an in-plane shear force applied at the weld group centroid shall be superposed, or a recognised method of analysis shall be used.

(c) The design force per unit length (v^*_w) at any point in the fillet weld group shall be assumed to act at right angles to the radius from that point to the instantaneous centre, and shall be taken as proportional to that radius.

A fillet weld shall satisfy the requirements of Clause 9.7.3.10 at all points in the fillet weld group using the appropriate capacity factor (ϕ) for a weld group (see Table 3.4). In the case of a fillet weld group of constant throat thickness, it will be sufficient to check only that point in the group defined by the maximum value of the radius to the instantaneous centre.

9.8.1.2 *Alternative analysis* The design force per unit length in the fillet weld group may alternatively be determined by considering the fillet weld group as an extension of the connected member and proportioning the design force per unit length in the fillet weld group to satisfy equilibrium between the fillet weld group and the elements of the connected member.

A fillet weld shall satisfy the requirements of Clause 9.7.3.10 at all points in the fillet weld group using the appropriate capacity factor (ϕ) for a weld group (see Table 3.4).

9.8.2 Weld group subject to out-of-plane loading

9.8.2.1 *General method of analysis* The design force per unit length in a fillet weld group subject to out-of-plane loading shall be determined in accordance with the following:

(a) The fillet weld group shall be considered in isolation from the connected element; and

(b) The design force per unit length in the fillet weld resulting from a design bending moment shall be considered to vary linearly with the distance from the relevant centroidal axes. The design force per unit length in the fillet weld group resulting from any shear force or axial force shall be considered to be uniformly distributed over the length of the fillet weld group.

A fillet weld shall satisfy the requirements of Clause 9.7.3.10 at all points in the fillet weld group, using the appropriate capacity factor (ϕ) for a weld group (see Table 3.4).

9.8.2.2 *Alternative analysis* The design force per unit length in a fillet weld group may alternatively be determined by considering the fillet weld group as an extension of the connected member and distributing the design forces among the welds of the fillet weld group so as to satisfy equilibrium between the fillet weld group and the elements of the connected member.

A fillet weld shall satisfy the requirements of Clause 9.7.3.10 at all points in the fil weld group, using the appropriate capacity factor (ϕ) for a weld group (see Table 3.4).

9.8.3 Weld group subject to in-plane and out-of-plane loading

9.8.3.1 *General method of analysis* The design force per unit length as determined from analyses in accordance with Clauses 9.8.1.1 and 9.8.2.1 shall satisfy Clause 9.7.3 at all points in the fillet weld group, using the appropriate capacity factor (ϕ) for a weld group (see Table 3.4).

9.8.3.2 *Alternative analysis* The design force per unit length as determined from analyses in accordance with Clauses 9.8.1.2 and 9.8.2.2 shall satisfy Clause 9.7.3.10 at points in the fillet weld group, using the appropriate capacity factor (ϕ) for a weld group (see Table 3.4).

9.8.4 Combination of weld types If two or more types of weld are combined in single connection, the design capacity of the connection shall be the sum of the design capacities of each type, determined in accordance with this Section.

9.9 PACKING IN CONSTRUCTION Where packing is welded between two members and is less than 6 mm thick, or is too thin to allow provision of adequate welds or to prevent buckling, the packing shall be trimmed flush with the edges of the element subject to the design action and the size of the welds along the edges shall be increased over the required size by an amount equal to the thickness of the packing.

Otherwise the packing shall extend beyond the edges and shall be welded to the piece to which it is fitted.

SECTION 14 FABRICATION

14.1 GENERAL A fabricated item shall be liable to rejection if—

(a) the material does not satisfy the requirements of Clause 14.2; or

(b) the fabrication does not satisfy the requirements of Clause 14.3; or

(c) it does not satisfy the tolerances specified in Clause 14.4.

The fabricated item may be accepted nonetheless if—

(i) it can be demonstrated that the structural adequacy and intended use of the item are not impaired thereby; or

(ii) it passes testing in accordance with the appropriate Clauses of Section 17.

Fabricated items which do not satisfy either (i) or (ii) above and which do not satisfy either Clause 14.2, Clause 14.3 or Clause 14.4 shall be rejected.

14.2 MATERIAL

14.2.1 General All material shall satisfy the requirements of the appropriate material Standard specified in Clauses 2.2, 2.3 and 2.4.

Surface defects in the steel shall be removed using the methods specified in the appropriate Standards listed in Clause 2.2.1.

14.2.2 Identification The steel grade shall be identifiable at all stages of fabrication, or the steel shall be classed as unidentified steel and only used in accordance with Clause 2.2.3. Any marking of steelwork shall be such as to not damage the material.

14.3 FABRICATION PROCEDURES

14.3.1 Methods All material shall be straightened or formed to the specified configuration by methods that will not reduce the properties of the material below the values used in design. Steel may be bent or pressed to the required shape by either hot or cold processes.

Local application of heat or mechanical means may be used to introduce or correct camber, sweep and out-of-straight. The temperature of heated areas shall not exceed 650°C.

14.3.2 Full contact splices Full contact splices may be produced by cold saw cutting or machining.

The surfaces of such splices shall be such that, when the ends of the two members are abutted, the alignment of the members and the gap shall be within the tolerances specified in Clause 14.4.4.2.

14.3.3 Cutting Cutting may be by sawing, shearing, cropping, machining and thermal cutting processes, as appropriate.

Shearing of items over 16 mm thick shall not be carried out when the item is to be galvanized and subject to tensile force or bending moment unless the item is stress relieved subsequently.

Any cut surface not incorporated in a weld shall have a roughness not greater than the appropriate value given in Table 14.3.3. A cut surface to be incorporated in a weld shall comply with AS 1554.1.

TABLE 14.3.3

MAXIMUM CUT SURFACE ROUGHNESS

Application	Maximum roughness (CLA) µm
Normal applications, i.e. where the face and edges remain as-cut or with minor dressing	25
Fatigue applications (Detail categories as defined in Clause 11.5)	
detail category ≥ 80 MPa	12
detail category < 80 MPa	25

NOTES:

1 Roughness values may be estimated by comparison with surface replicas, such as the AWRA Flame Cut Surface replicas.

2 Suitable techniques of flame cutting are given in AWRA Technical Note 5.

3 CLA ≡ *Centre Line Average Method*.

Cut surface roughness exceeding these values shall be repaired by grinding to give a value less than the specified roughness. Grinding marks shall be parallel to the direction of the cut.

Notches and gouges, not closer than $20t$ (where t = component thickness) and not exceeding 1% of the total surface area on an otherwise satisfactory surface, are acceptable provided that imperfections greater than $t/5$ but not exceeding 2 mm in depth are removed by machining or grinding. Imperfections outside the above limits shall be repaired by welding in accordance with AS 1554.1.

A re-entrant corner shall be shaped notch free to a radius of at least 10 mm.

14.3.4 Welding Welding shall comply with AS 1554.1 or AS 1554.5 as appropriate (see Clause 11.1.5), and welding of studs shall comply with AS 1554.2.

14.3.5 Holing

14.3.5.1 *General* A round hole for a bolt shall either be machine flame cut, or drilled full size, or subpunched 3 mm undersize and reamed to size, or punched full size.

A slotted hole shall be either machine flame cut, or punched in one operation, or formed by drilling two adjacent holes and completed by machine flame cutting.

Hand flame cutting of a bolt hole shall not be permitted except as a site rectification measure for holes in column base plates.

A punched hole shall only be permitted in material whose yield stress (f_y) does not exceed 360 MPa and whose thickness does not exceed $(5600/f_y)$ mm.

14.3.5.2 *Hole size* The nominal diameter of a completed hole other than a hole in a base plate shall be 2 mm larger than the nominal bolt diameter for a bolt not exceeding 24 mm in diameter, and not more than 3 mm larger for a bolt of greater diameter.

For a hole in a base plate, the hole diameter shall be not more than 6 mm greater than the anchor bolt diameter. A special plate washer of minimum thickness 4 mm shall be used under the nut if the hole diameter is 3 mm or more larger than the bolt diameter.

(a) *Oversize or slotted hole* An oversize or slotted hole shall be permitted, provided that the following requirements are satisfied:

(i) An oversize hole shall not exceed $1.25d_f$ or $(d_f + 8)$ mm in diameter, whichever is the greater, where d_f is the nominal bolt diameter, in millimetres.

(ii) A short slotted hole shall not exceed the appropriate hole size of this Clause in width and $1.33d_f$ or $(d_f + 10)$ mm in length, whichever is the greater.

(iii) A long slotted hole shall not exceed the appropriate hole size of this Clause in width and $2.5d_f$ in length.

(b) *Limitations on use* The use of an oversize or slotted hole shall be limited so that the following requirements are satisfied:

(i) *Oversize hole* An oversize hole may be used in any or all plies of bearing-type and friction-type connections, provided hardened or plate washers are installed over the oversize hole under both the bolt head and the nut.

(ii) *Short slotted hole* A short slotted hole may be used in any or all plies of a friction-type or a bearing-type connection, provided hardened or plate washers are installed over the holes under both the bolt head and the nut.

In a friction-type connection subject to a shear force, a short slotted hole may be used without regard to the direction of loading.

In a bearing-type connection subject to a shear force, a short slotted hole may be used only where the connection is not eccentrically loaded and the bolt can bear uniformly, and where the slot is normal to the direction of the design action.

(iii) *Long slotted hole* A long slotted hole may be used only in alternate plies of either a friction-type or bearing-type connection, provided a plate washer not less than 8 mm thick is used to completely cover any long slotted hole under both the bolt head and the nut.

In a friction-type connection subject to a shear force, a long slotted hole may be used without regard to direction of loading.

In a bearing-type connection subject to a shear force, a long slotted hole may be used only where the connection is not eccentrically loaded and where the bolt can bear uniformly, and where the slot is normal to the direction of the load.

14.3.6 Bolting

14.3.6.1 *General* All bolts and associated nuts and washers shall comply with the appropriate bolt material Standard specified in Clause 2.3.1. All material within the grip of the bolt shall be steel and no compressible material shall be permitted in the grip.

The length of a bolt shall be such that at least one clear thread shows above the nut and at least one thread plus the thread run out is clear beneath the nut after tightening.

One washer shall be provided under the rotated part.

Where the slope of the surfaces of parts in contact with the bolt head or nut exceeds 1:20 with respect to a plane normal to the bolt axis, a suitably tapered washer shall be provided against the tapered surface and the non-rotating part shall be placed against the tapered washer.

The nuts used in a connection subject to vibration shall be secured to prevent loosening. (See Clause 9.1.6.)

14.3.6.2 *Tensioned bolt* A tensioned high-strength bolt when installed during fabrication shall be installed in accordance with Clauses 15.2.4 and 15.2.5. The contact surfaces of a joint using a tensioned bolt shall be prepared in accordance with Clause 14.3.6.3.

14.3.6.3 *Preparation of surfaces in contact* Preparation of surfaces in contact shall be as follows:

(a) *General* All oil, dirt, loose scale, loose rust, burrs, fins and any other defects on the surfaces of contact which will prevent solid seating of the parts in the snug-tight condition shall be removed.

NOTES:

1 If cleaning is necessary to meet these requirements, reference should be made to AS 1627.
2 A clean 'as-rolled' surface with tight mill scale is acceptable without further cleaning.
3 Snug-tight is defined in Clause 15.2.5.2.

(b) *Friction-type connection* For a friction-type connection, the contact surfaces shall be clean 'as-rolled' surfaces or equivalent and, in addition to satisfying the provisions of Item (a), shall be free from paint, lacquer, galvanizing or other applied finish unless the applied finish has been tested in accordance with Appendix J to establish the friction coefficient (see Clause 9.3.3.2).

In a non-coated connection, paint including any overspray shall be excluded from areas closer than one bolt diameter to any hole but not less than 25 mm from the edge of any hole and all areas within the bolt group.

(c) *Bearing-type connection* For a bearing-type connection, an applied finish on the contact surfaces shall be permitted.

14.3.7 Pinned connection Pins and holes shall be finished so that the forces are distributed evenly to the joint plies.

14.4 TOLERANCES

14.4.1 General The tolerance limits of this Clause shall be satisfied after fabrication is completed and any corrosion protection has been applied. Unless otherwise specified, the tolerance on all structural dimensions shall be ±2 mm.

14.4.2 Notation For the purpose of this Clause—

a_0, a_1 = out-of-square dimensions of flanges

a_2, a_3 = diagonal dimensions of a box section

b = lesser dimension of a web panel

b_f = width of a flange

d = depth of a section

d_o = overall depth of a member including out-of-square dimensions

d_1 = clear depth between flanges ignoring fillets or welds

e = web off-centre dimension

L = member length

Δ_f = out-of-flatness of a flange plate

Δ_v = deviation from verticality of a web at a support

Δ_w = out-of-flatness of a web

14.4.3 Cross-section After fabrication, the tolerances on any cross-section of a rolled section or a plate shall be those specified in AS 3678 or AS 3679, as appropriate, in respect of depth, flange width, flange thickness, web thickness, out-of-square, and web off-centre. For any built-up section, the deviations from the specified dimensions of the cross-section shall not exceed the following:

(a) Depth of a section (d) (see Figure 14.4.3(1))

for $d \leq 900$, ±3 mm

for $900 < d \leq 1800$, $\pm\left[3 + \frac{(d - 900)}{300}\right]$ mm

for $d > 1800$, ±6 mm

(b) Width of a flange (b_f) (see Figure 14.4.3(1))

for all b_f, ±6 mm

(c) Out-of-square of an individual flange (a_0 or a_1) (see Figure 14.4.3(1))

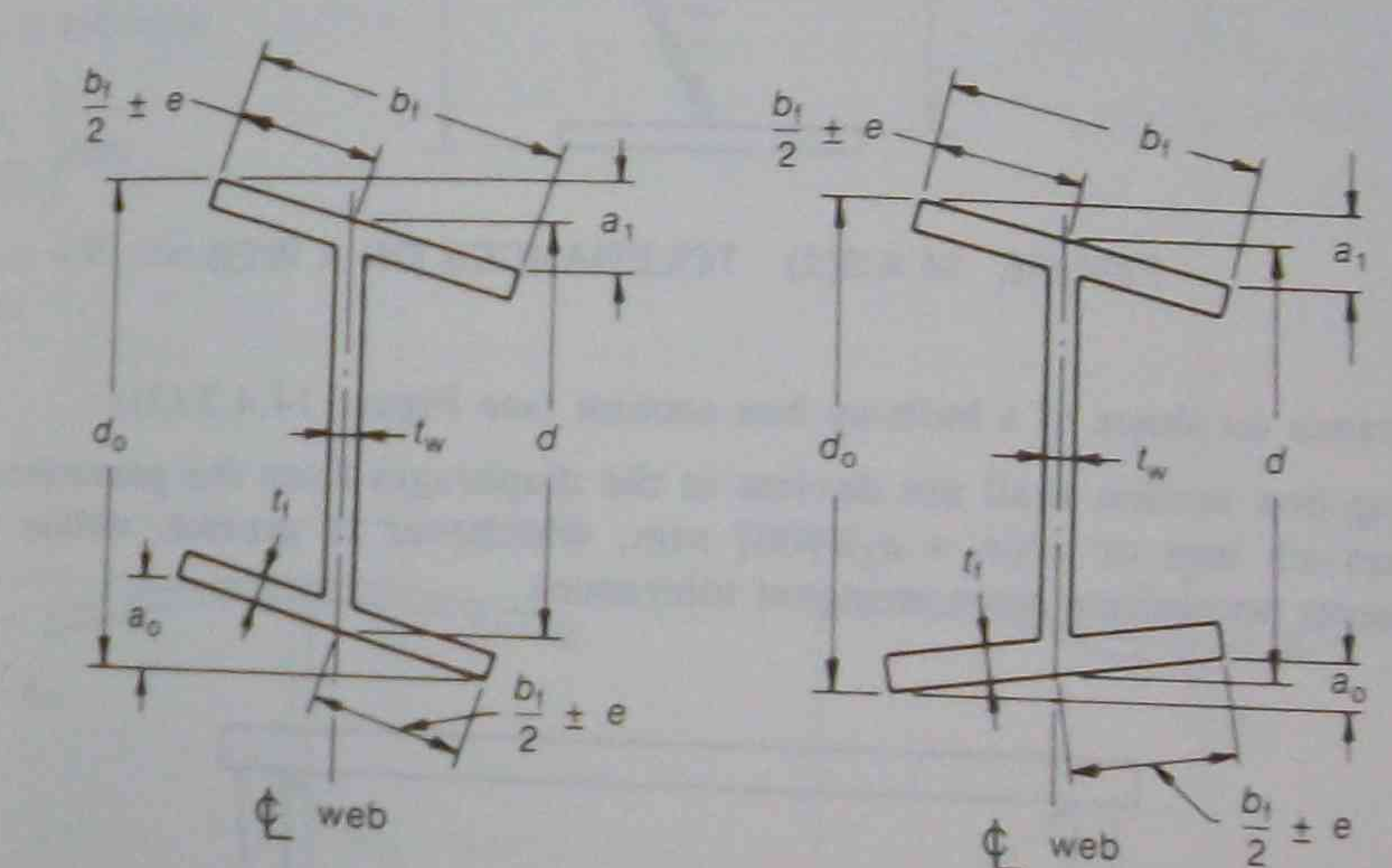
for $b_f \leq 600$ mm, ±3 mm

for $b_f > 600$ mm, $\pm\left(\frac{b_f}{200}\right)$ mm

(d) Total out-of-square of two flanges ($a_0 + a_1$) (see Figure 14.4.3(1))

for $b_f \leq 600$ mm, ±6 mm

for $b_f > 600$ mm, $\pm\left(\frac{b_f}{100}\right)$ mm



NOTES:

1 Dimension d, d_o, a_o and a_1 are measured parallel to the centreline of the web. Dimensions b_f and $(0.5b_f \pm e)$ are measured parallel to the plane of the flange.
2 Dimension d is measured at the centreline of the web.

FIGURE 14.4.3(1) TOLERANCES ON A CROSS-SECTION

(e) Out-of-flatness a of web (Δ_w) (see Figure 14.4.3(2))

d_1/150 mm
for unstiffened web,

b/100 mm
for stiffened web with intermediate stiffeners,

measured on a gauge length in the direction of d_1 or b, as appropriate.

(f) Deviation from verticality of a web at a support (Δ_v) (see Figure 14.4.3(2))

for $d \leq 900$ mm,

±3 mm

for $d > 900$ mm,

$\pm\left(\frac{d}{200}\right)$mm

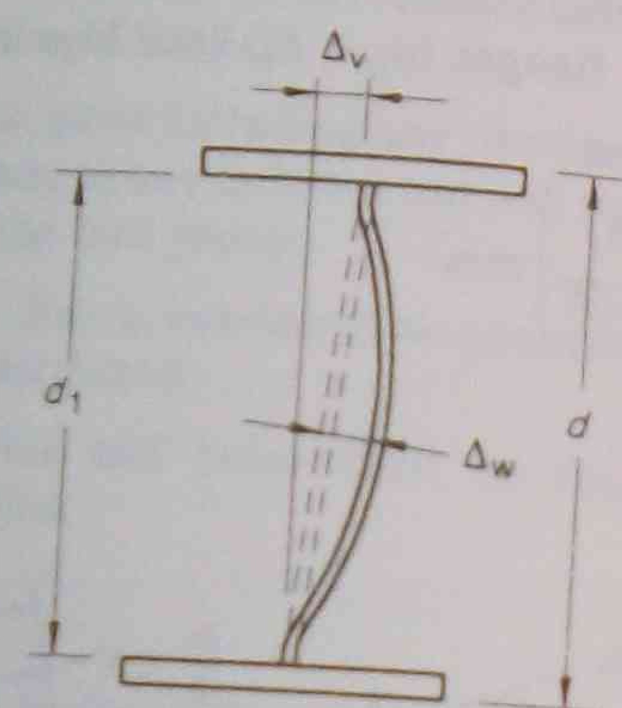


FIGURE 14.4.3(2) TOLERANCES ON A WEB

(g) Tolerance on shape of a built-up box section (see Figure 14.4.3.(3))

A built-up box section shall not deviate at the diaphragm from the prescribed shape more than ±5 mm or $\pm[(a_2 + a_3)/400]$ mm, whichever is greater, unless connection requirements necessitate more stringent tolerances.

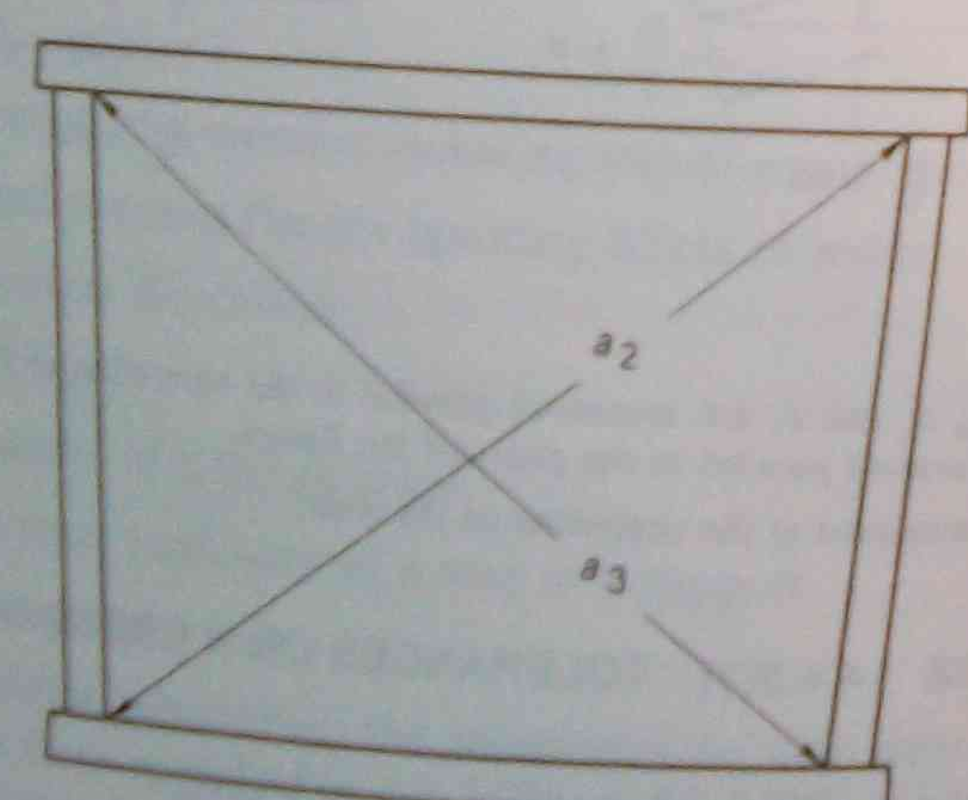


FIGURE 14.4.3(3) TOLERANCE ON SHAPE OF A BOX SECTION

(h) Off-centre of a web (e) (see Figure 14.4.3(4)) ±6 mm

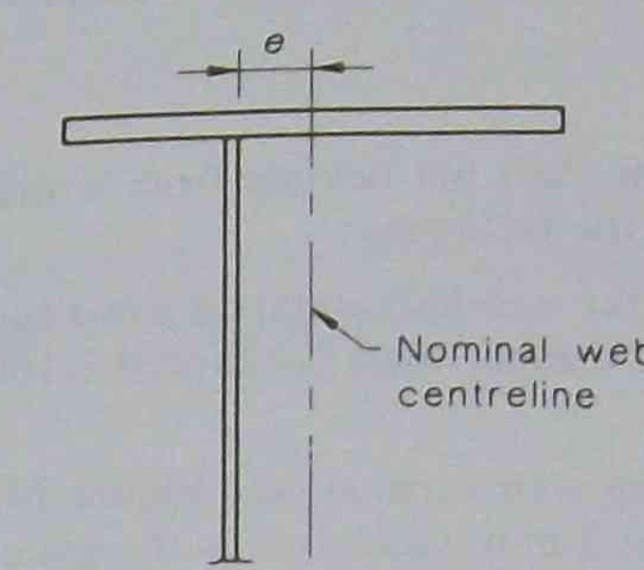


FIGURE 14.4.3(4) TOLERANCE ON OFF-CENTRE OF A WEB

(j) Out-of-flatness of a flange (Δ_f) (see Figure 14.4.3(5))

for $b_f \leq 450$ mm

$\pm\left(\frac{b_f}{150}\right)$mm

for $b_f > 450$ mm

±3 mm

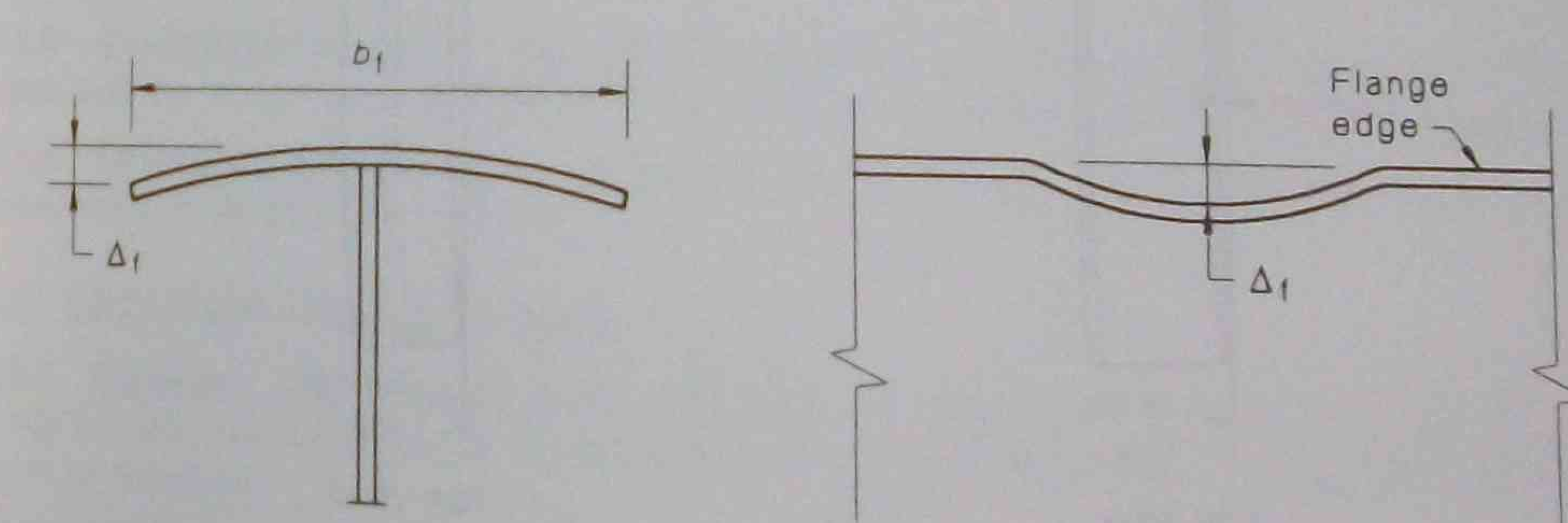


FIGURE 14.4.3(5) TOLERANCE ON OUT-OF-FLATNESS OF A FLANGE

14.4.4 Compression member

14.4.4.1 *Straightness* A member shall not deviate about either principal axis from a straight line drawn between end points by an amount exceeding $L/1000$ or 3 mm whichever is the greater.

14.4.4.2 *Full contact splice* If the ends of two butting lengths of a member, or the end of a member and the contact face of an adjoining cap plate or baseplate, are required to be in full contact, such a requirement shall be deemed to be satisfied if the bearing surfaces are prepared so that when the abutting member length or lengths are aligned to within the tolerance specified in Clause 15.3.3, the maximum clearance between the abutting surfaces shall not exceed 1 mm, and shall also not exceed 0.5 mm over at least 67% of the contact area.

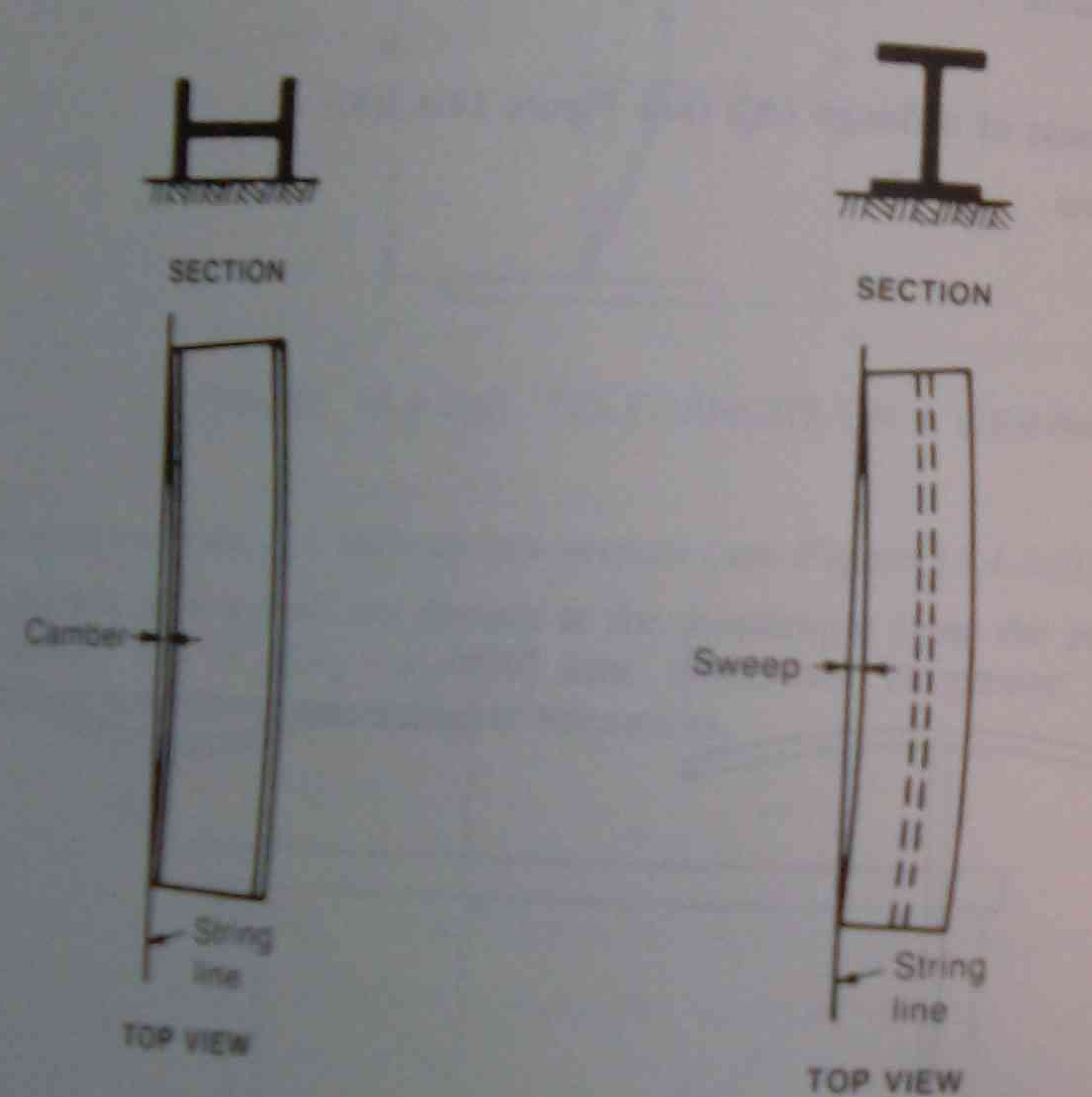
14.4.4.3 *Length* The length of a member shall not deviate from its specified length by more than ±2 mm.

14.4.5 Beam

14.4.5.1 *Straightness* A beam shall not deviate from a straight line drawn between the ends of the beam by more than the following:

(a) *Camber*—measured with the web horizontal on a test surface (see Figure 14.4.5.1(a)). The tolerance on specified camber shall not exceed $L/1000$ or 10 mm whichever is the lesser.

(b) *Sweep*—measured with the web vertical (see Figure 14.4.5.1(b)). The sweep in plan shall not exceed $L/1000$ or 3 mm whichever is the greater.



(a) Camber

(b) Sweep

FIGURE 14.4.5.1 MEASUREMENT OF CAMBER AND SWEEP

14.4.5.2 *Length* The length of a beam shall not deviate from its specified length by more than ±2 mm for lengths less than 10 m, and ±4 mm for lengths greater than 10 m.

14.4.6 Tension member

14.4.6.1 *Straightness* A member shall not deviate from a straight line drawn between end points by more than $L/500$, where L is the length between end points.

14.4.6.2 *Length* The length of a tension member shall not deviate from its specified length by more than ±2 mm for lengths less than 10 m, and ±4 mm for lengths greater than 10 m.

SECTION 15 ERECTION

15.1 GENERAL

15.1.1 Rejection of an erected item An erected item shall be liable to rejection if—

(a) the erection does not satisfy the requirements of Clause 15.2; or

(b) it does not satisfy the tolerances specified in Clause 15.3.

The erected item may be accepted nonetheless if—

(i) it can be demonstrated that the structural adequacy and intended use of the item are not impaired thereby; or

(ii) it passes testing in accordance with the appropriate clauses of Section 17.

Erected items which do not satisfy either (i) or (ii) above and which do not satisfy either Clause 15.2 or 15.3 shall be rejected.

Bolts, nuts and washers shall be liable to rejection if, in the erected structure, they do not comply with Clauses 14.3.6, 15.2.3, 15.2.4 and 15.2.5, unless it can be demonstrated that the structural adequacy and intended use of the item are not impaired thereby.

Grouting at supports which does not satisfy the requirements of Clause 15.5 shall be rejected.

15.1.2 Safety during erection During the erection of a structure, steelwork shall be made safe against erection loading, including loading due to erection equipment or its operation, and wind.

15.1.3 Equipment support Equipment supported on partly erected steelwork shall not induce actions in the steel greater than the design capacities permitted in this Standard.

15.1.4 Reference temperature Dimensions shall be set out on the basis of a reference temperature of 20 degrees Celsius.

15.2 ERECTION PROCEDURES

15.2.1 General The requirements specified in Clause 14.3 shall also be observed during the erection of the steel frame and during any modifications to the steelwork in the course of erection.

This requirement shall apply to:

(a) Full contact splices (see Clause 14.3.2).

(b) Cutting (see Clause 14.3.3).

(c) Welding (see Clause 14.3.4).

(d) Holing (see Clause 14.3.5).

(e) Bolting (see Clause 14.3.6).

Throughout the erection of the structure, the steelwork shall be securely bolted or fastened to ensure that it can adequately withstand all loadings liable to be encountered during erection, including, where necessary, those from erection plant and its operation. Any temporary bracing or temporary restraint shall be left in position until such time as erection is sufficiently advanced as to allow its safe removal.

All connections for temporary bracing and members to be provided for erection purposes shall be made in such a manner as not to weaken the permanent structure or to impair its serviceability. All welding of such connections and their removal shall be in accordance with AS 1554.1.

15.2.2 Delivery, storage and handling Members, components and fasteners shall be handled and stacked in such a way that damage is not caused to them. Means shall be provided to minimize damage to the corrosion protection on the steelwork.

All work shall be protected from damage in transit. Particular care shall be taken to stiffen free ends, prevent permanent distortion, and adequately protect all surfaces prepared for full contact splices. All bolts, nuts, washers, screws, small plates and articles generally shall be suitably packed and identified.

15.2.3 Assembly and alignment All matching holes shall align with each other so that a gauge or drift, equal in diameter to that of the bolts, shall pass freely through the assembled contact faces at right angles to them. Drifting to align holes shall be done in a manner that will not distort the metal nor enlarge the holes.

Each part of the structure shall be aligned as soon as practicable after it has been erected. Permanent connections shall not be made between members until sufficient of the structure has been aligned, levelled, plumbed and temporarily connected to ensure that members will not be displaced during subsequent erection or alignment of the remainder of the structure.

Each bolt and nut shall be assembled with at least one washer. A washer shall be placed under the rotating component. Where the slope of the surfaces of parts in contact with the bolt head or nut exceeds 1:20 with respect to a plane normal to the bolt axis, a suitable tapered washer shall be used against the sloping surface. The non-rotating component shall be placed against the tapered washer.

Bolting categories 4.6/S and 8.8/S shall be installed to the snug-tight condition specified in Clause 15.2.5.2(a).

Hardened or plate washers shall be used under both the bolt head and nut for any slotted and oversize holes as specified in Clause 14.3.5.2(b).

15.2.4 Assembly of a connection involving tensioned bolts

15.2.4.1 *Placement of a nut* The nut shall be placed so that the mark specified in AS 1252 to identify a high-strength nut is visible after tightening.

15.2.4.2 *Packing* Packing shall be provided wherever necessary to ensure that the load-transmitting plies are in effective contact when the connection is tightened to the snug-tight condition defined in Clause 15.2.5.2(a). All packing shall be steel with a surface condition similar to that of the adjacent plies.

15.2.4.3 *Tightening pattern* Snug-tightening and final tensioning of the bolts in a connection shall proceed from the stiffest part of the connection towards the free edges.

High-strength structural bolts that are to be tensioned may be used temporarily during erection to facilitate assembly, but if so used they shall not be finally tensioned until all bolts in the connection have been snug-tightened in the correct sequence.

15.2.4.4 *Retensioning* Retensioning of bolts which have been fully tensioned shall be avoided, except that if retensioning is carried out, it shall only be permitted once and only where the bolt remains in the same hole in which it was originally tensioned and with the same grip.

Retensioning of galvanized bolts shall not be permitted.

Under no circumstances shall bolts which have been fully tensioned be reused in another hole.

Touching up or retensioning of previously tensioned bolts which may have been loosened by the tensioning of adjacent bolts shall not be considered as retensioning.

15.2.5 Methods of tensioning

15.2.5.1 *General* The method of tensioning shall be in accordance with either Clause 15.2.5.2 or Clause 15.2.5.3.

In the completed connection, all bolts shall have at least the minimum bolt tension specified in Table 15.2.5.1 when all bolts in the bolt group are tightened.

TABLE 15.2.5.1

MINIMUM BOLT TENSION

Nominal diameter of bolt	Minimum bolt tension, kN
M16	95
M20	145
M24	210
M30	335
M36	490

NOTE: The minimum bolt tensions given in this Table are approximately equivalent to the minimum proof loads given in AS 1252.

15.2.5.2 *Part-turn method of tensioning* Tensioning of bolts by the part-turn method shall be in accordance with the following procedure:

(a) On assembly, all bolts in the connection shall be first tightened to a snug-tight condition to ensure that the load-transmitting plies are brought into effective contact.

Snug-tight is the tightness attained by a few impacts of an impact wrench or by the full effort of a person using a standard podger spanner.

(b) After completing snug-tightening, location marks shall be established to mark the relative position of the bolt and the nut and to control the final nut rotation.

Observation of the final nut rotation may be achieved by using marked wrench sockets, but location marks shall be permanent when required for inspection.

(c) Bolts shall be finally tensioned by rotating the nut by the amount given in Table 15.2.5.2. During the final tensioning, the component not turned by the wrench shall not rotate.

15.2.5.3 *Tensioning by use of direct-tension indication device* Tensioning of bolts using a direct-tension indication device shall be in accordance with the following procedure:

(a) The suitability of the device shall be demonstrated by testing a representative sample of not less than three bolts for each diameter and grade of bolt in a calibration device capable of indicating bolt tension. The calibration test shall demonstrate that the device indicates a tension not less than 1.05 times the minimum bolt tension specified in Table 15.2.5.1.

(b) On assembly, all bolts and nuts in the connection shall be first tightened to a snug-tight condition defined in Clause 15.2.5.2(a).

(c) After completing snug-tightening, the bolt shall be tensioned to provide the minimum bolt tension specified in Clause 15.2.5.1. This shall be indicated by the tension indication device.

NOTE: Tensioning of bolts using a direct-tension indication device should also be in accordance with the manufacturer's specification.

TABLE 15.2.5.2

NUT ROTATION FROM THE SNUG-TIGHT CONDITION

Bolt length (underside of head to end of bolt)	Disposition of outer face of bolted parts (see Notes 1, 2, 3 and 4)		
	Both faces normal to bolt axis	One face normal to bolt axis and other sloped	Both faces sloped
Up to and including 4 diameters	1/3 turn	1/2 turn	2/3 turn
Over 4 diameters but not exceeding 8 diameters	1/2 turn	2/3 turn	5/6 turn
Over 8 diameters but not exceeding 12 diameters (see Note 5)	2/3 turn	5/6 turn	1 turn

NOTES:

1 Tolerance on rotation: for 1/2 turn or less, one-twelfth of a turn (30°) over and nil under tolerance; fo 2/3 turn or more, one-eighth of a turn (45°) over and nil under tolerance.

2 The bolt tension achieved with the amount of nut rotation specified in Table 15.2.5.2 will be at lea equal to the minimum bolt tension specified in Table 15.2.5.1.

3 Nut rotation is the rotation relative to the bolt, regardless of the component turned.

4 Nut rotations specified are only applicable to connections in which all material within the grip of the bolt is steel.

5 No research has been performed to establish the turn-of-nut procedure for bolt lengths exceeding 1 diameters. Therefore, the required rotation should be determined by actual test in a suitable tensi measuring device which simulates conditions of solidly fitted steel

15.3 TOLERANCES

15.3.1 Location of anchor bolts Anchor bolts shall be restrained in position both i vertical and a horizontal direction during all setting-in operations.
Anchor bolts shall be set out in accordance with the erection drawings. They shall n vary from the positions shown on the erection drawings by more than the followi (see Figure 15.3.1.):

(a) 3 mm centre-to-centre of any two bolts within an anchor bolt group, where a anchor bolt group is defined as the set of anchor bolts which receives a sing fabricated steel member.

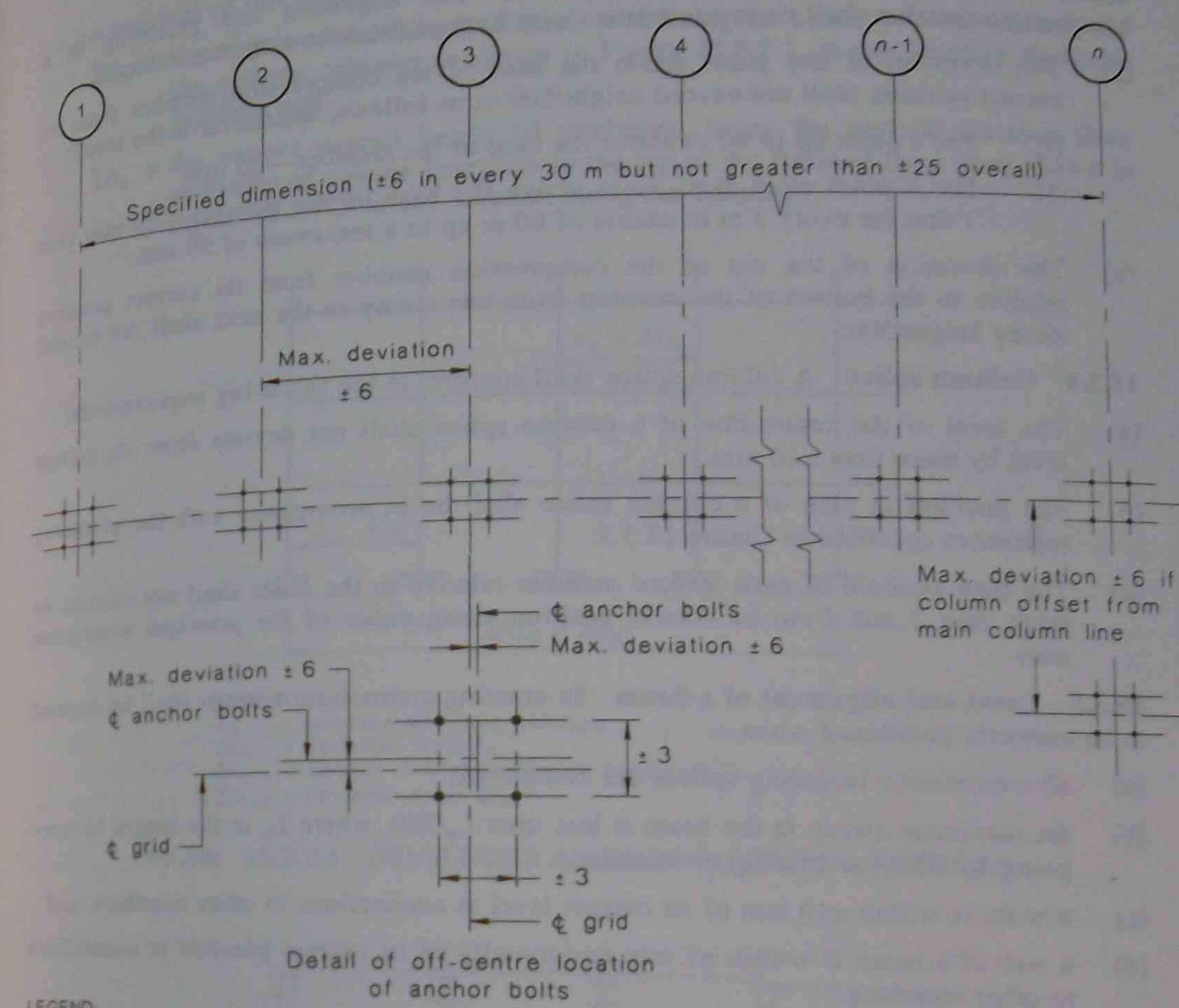
(b) 6 mm centre-to-centre of adjacent anchor bolt groups.

(c) Maximum accumulation of 6 mm per 30 000 mm along an established column li of multiple anchor bolt groups, but not to exceed a total of 25 mm. The establish column line is the actual field line most representative of the centres of the as-bu anchor bolt groups along a line of columns.

(d) 6 mm from the centre of any anchor bolt group to the established column li through that group.

Anchor bolts shall be set perpendicular to the theoretical bearing surface, threads shall b protected and free of concrete and nuts shall run freely on the threads.

The projection of the end of the anchor bolt from the theoretical bearing surface shall n be more than 25 mm longer nor 5 mm shorter than that specified.



LEGEND:
n = total number of columns

UNLESS OTHERWISE SPECIFIED DIMENSIONS IN MILLIMETRES

FIGURE 15.3.1 TOLERANCES IN ANCHOR BOLT LOCATION

15.3.2 Column base

15.3.2.1 *Position in plan* The position in plan of a steel column base shall not deviate from its correct value by more than 6 mm along either of the principal setting out axes.

15.3.2.2 *Level* The level of the underside of a steel base plate shall not deviate from its correct value by more than ±10 mm.

15.3.2.3 *Full contact* If full contact is specified, the requirements of Clause 14.4.4.2 shall be satisfied, unless shims are used to reduce the measurable gaps to values specified in Clause 14.4.4.2.

Packs, shims and other supporting devices shall be flat and of the same steel grade as the member. If such packings are to be subsequently grouted, they shall be placed so that the grout totally encloses them with a minimum cover of 50 mm.

15.3.3 Plumbing of a compression member The alignment and plumbing of a compression member shall be in accordance with both of the following requirements:

(a) The deviation of any point above the base of the compression member from its correct position shall not exceed height/500 or as follows, whichever is the lesser:

(i) For a point up to 60 m above the base of the member—25 mm.

(ii) For a point more than 60 m above the base of the member—25 mm plus 1 mm for every 3 m in excess of 60 m up to a maximum of 50 mm.

(b) The deviation of the top of the compression member from its correct position relative to the bottom of the member from one storey to the next shall not exceed storey height/500.

15.3.4 Column splice A column splice shall conform to the following requirements:

(a) The level of the centre-line of a column splice shall not deviate from its correct level by more than ±10 mm.

(b) The position in plan of a column splice shall be in accordance with the plumbing tolerances specified in Clause 15.3.3.

(c) The plan position of each spliced member relative to the other shall not deviate by more than 2 mm from its correct position along either of the principal setting-out axes.

15.3.5 Level and alignment of a beam In erecting a structure, a beam shall be deemed to be correctly positioned when—

(a) all connections including splices are completed;

(b) the maximum sweep in the beam is less than $L_b/500$, where L_b is the length between points of effective bracing or restraint;

(c) a beam is within ±10 mm of its correct level at connections to other members; and

(d) a web of a beam is within ±3 mm horizontally of its correct position at connections to other members.

15.3.6 Position of a tension member A tension member shall not deviate from its correct position relative to the members to which it is connected by more than 3 mm along any setting-out axis.

15.3.7 Overall building dimensions The overall building dimensions shall not deviate from the correct values by more than the following:

(a) Length (see Figure 15.3.7.1)

for $\Sigma L_c \leq 30$ m, $\Sigma \Delta L_c \leq \pm 20$ mm

for $\Sigma L_c > 30$ m, $\Sigma \Delta L_c \leq \pm\{20 \text{ mm} + 0.25(\Sigma L_c - 30) \text{ mm}\}$

(b) Height (see Figure 15.3.7.2)

for $\Sigma h_b \leq 30$ m, $\Sigma \Delta h_b \leq \pm 20$ mm

for $\Sigma h_b > 30$ m, $\Sigma \Delta h_b \leq \pm\{20 \text{ mm} + 0.25(\Sigma h_b - 30) \text{ mm}\}$

provided that—

(i) the distance between adjacent steel column centres (L_c) at every section does not deviate by more than ±15 mm from the correct length;

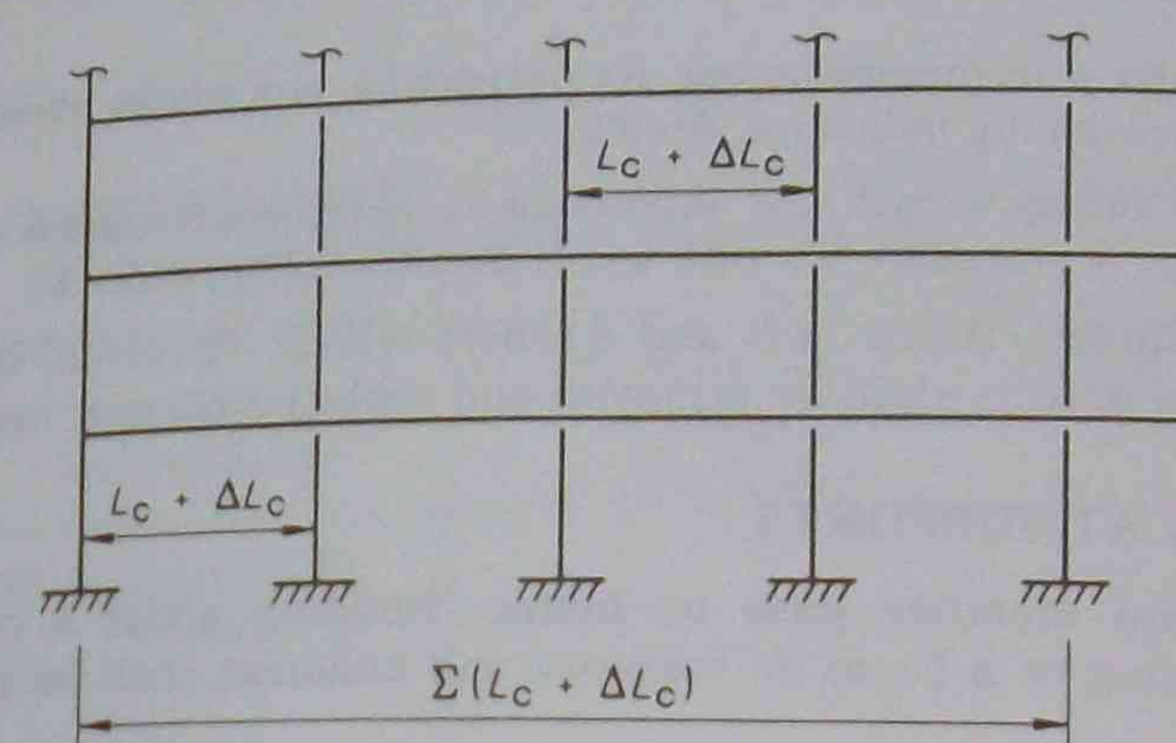
(ii) the vertical distance between tops of beams (h_b) at every section does not deviate by more than ±20 mm from the correct values; and

(iii) all other tolerances in this Section are complied with.

For the purposes of this Clause—

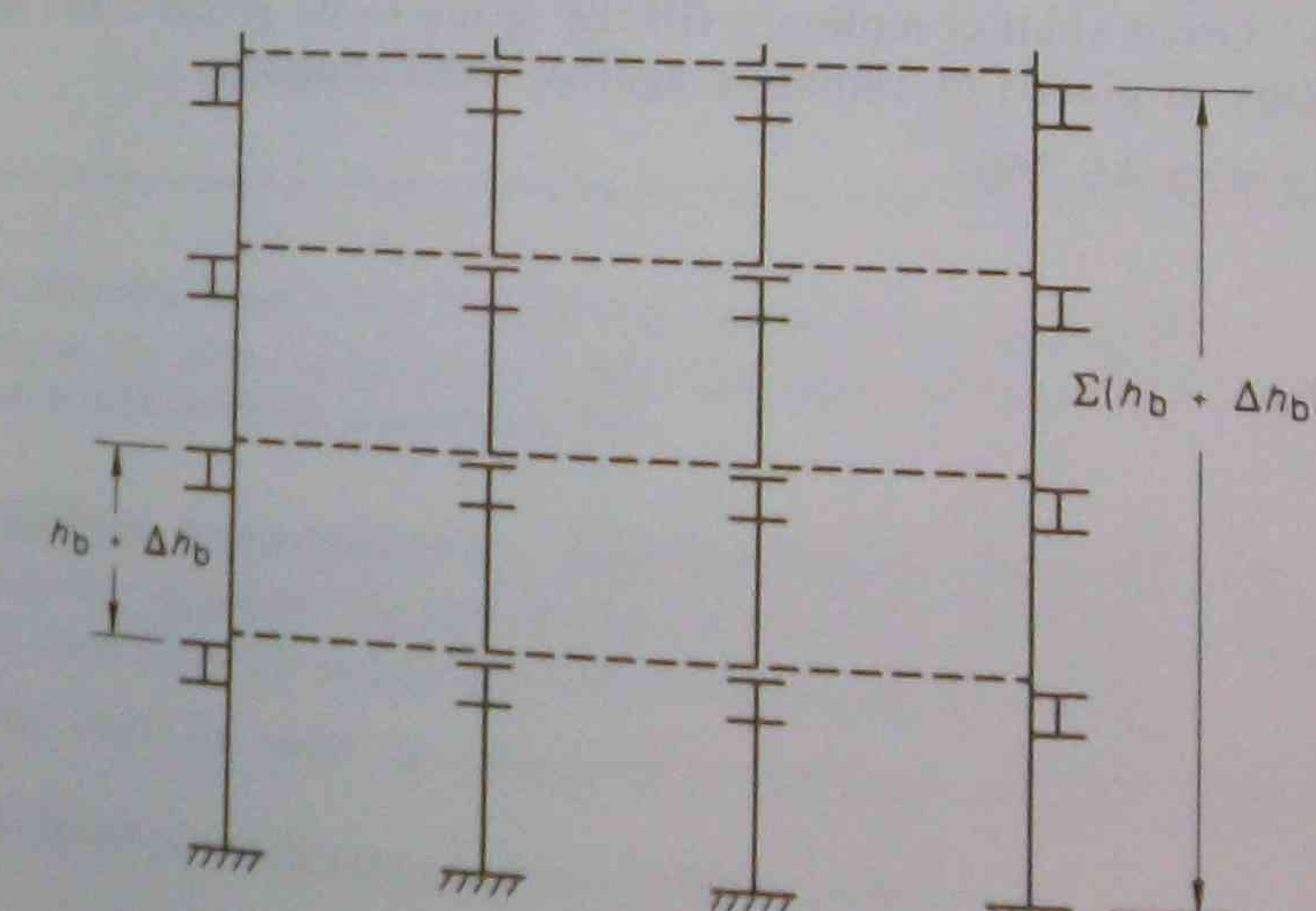
ΣL_c = the correct overall length of steelwork, being the centre-to-centre distance of the extreme columns as shown in Figure 15.3.7.1, at any location along the building, and

Σh_b = the correct overall height of steelwork, being the vertical distance from underside of column baseplate to the top of the finished floor level shown in Figure 15.3.7.2, at any location along the building.



LEGEND:
L_c = distance between columns
ΔL_c = deviation from L_c
ΣL_c = correct overall length of steelwork
$\Sigma \Delta L_c$ = deviation from ΣL_c

FIGURE 15.3.7.1 DEVIATIONS IN LENGTH (VERTICAL SECTION)



LEGEND:
h_b = distance between tops of beams
Δh_b = deviation from h_b
Σh_b = correct overall height of steelwork
$\Sigma \Delta h_b$ = deviation from Σh_b

FIGURE 15.3.7.2 DEVIATIONS IN HEIGHT (VERTICAL SECTION)

15.4 INSPECTION OF BOLTED CONNECTIONS

15.4.1 Tensioned bolts The methods of tensioning specified in Clause 15.2.5 shall comply with the following requirements:

(a) *Part-turn tensioning*—the correct part-turn from the snug-tight position shall be measured or observed.

(b) *Direct-tensioning indication device*—the minimum tension developed in the bolt shall be indicated directly by the device.

NOTES:

1 The manufacturer's recommendations for inspection procedures should be followed when using a direct-tensioning indication device.

2 The use of a torque wrench for inspection is considered suitable only to detect gross undertensioning. A procedure for such use is detailed in Appendix K.

15.4.2 Damaged items Bolts, nuts and washers which, on visual inspection, show any evidence of physical defects shall be removed and replaced by new items.

15.5 GROUTING AT SUPPORTS

15.5.1 Compression member base or beam Bedding under a compression member base plate or a bearing of a beam on masonry and concrete shall be provided by grout or mortar.

Grouting or packing shall not be carried out until a sufficient portion of the structure (in multistorey buildings, a sufficient number of bottom column lengths) has been aligned, levelled and plumbed and adequately braced by other structural members which have been levelled and are securely held by their permanent fastenings. Steel packing or levelling nuts on the anchor bolts shall be under the base plate to support the steelwork. The space under the steel shall be thoroughly cleaned and be free from moisture immediately before grouting.

15.5.2 Grouting Grout shall completely fill the space to be grouted and shall either be placed under pressure or placed by ramming against fixed supports.

Grout shall comply with AS 3600.

APPENDIX B
SUGGESTED DEFLECTION LIMITS
(Informative)

B1 SUGGESTED VERTICAL DEFLECTION LIMITS FOR BEAMS The vertical deflection of beams may be controlled using the suggested limits given in Table B1.

TABLE B1
SUGGESTED LIMITS ON CALCULATED VERTICAL DEFLECTIONS OF BEAMS

Type of beam	Deflection to be considered	Deflection limit (Δ) for span (L) (See Note 1)	Deflection limit (Δ) for cantilever (L) (See Note 2)
Beam supporting masonry partitions	The deflection which occurs after the addition or attachment of partitions	$\frac{\Delta}{L} \leq \frac{1}{500}$ where provision is made to minimize the effect of movement, otherwise $\frac{\Delta}{L} \leq \frac{1}{1000}$	$\frac{\Delta}{L} \leq \frac{1}{250}$ where provision is made to minimize the effect of movement, otherwise $\frac{\Delta}{L} \leq \frac{1}{500}$
All beams	The total deflection	$\frac{\Delta}{L} \leq \frac{1}{250}$	$\frac{\Delta}{L} \leq \frac{1}{125}$

NOTES:

1 Suggested deflection limits in this Table may not safeguard against ponding.

2 For cantilevers, the values of Δ/L given in this Table apply, provided that the effect of the rotation at the support is included in the calculation of Δ.

B2 SUGGESTED HORIZONTAL DEFLECTION LIMITS The relative horizontal deflection between adjacent frames at the eaves level of industrial buildings due to wind loads may be controlled by using the following suggested limits:

(a) Building clad in steel or aluminium sheeting without gantry cranes and without internal partitions against external walls . . . . $\frac{\text{column height}}{150}$

(b) Building with masonry walls supported by steelwork . . . . $\frac{\text{column height}}{240}$

CHAPTER 3

TIMBER STRUCTURES—DESIGN

This Chapter consists of edited extracts from AS 1720.1—*Timber Structures (known as the SAA Timber Structures Code)—Part 1: Design Methods*, including Amendment No. 1 of March 1993. This Standard sets out rules relating to the structural use of timber. Timber is a variable material and the extracts from the code provide data on different species and grades of timber and on the effects of different conditions of use on permissible stresses. Methods for designing timber members are included.

NOTE: Timber design Standards, SAA *Timber Structures Code* (AS 1720) and SAA *Timber Framing Code* (AS 1684), have as a basis classification of timber species into strength groups and sorting of timber into stress grades.

Strength Groups. A timber species or group of species for structural purposes is classified into a strength group, assigned on the basis of the mechanical properties of defect-free material or the density of the species.

In descending order of strength there are seven strength groups for unseasoned timber (S1 to S7) and eight strength groups for seasoned timber (SD1 to SD8).

Depending on the nature and amount of data on which the classification is made, a strength group may be termed 'positive' or 'provisional'.

A positive strength group is assigned on the basis of mechanical test data from five or more correctly sampled trees. A provisional strength group is assigned on the basis of density or limited mechanical test data. Provisional strength groups are shown within brackets, e.g. (S6).

Stress Grade. A piece of timber for structural purposes is classified into a stress grade, indicating primarily its basic working stress in bending and, by implication, the basic working stresses for other mechanical properties. The stress grade is designated in a form such as 'F5', which indicates that the particular piece of timber has a basic working stress in bending or approximately 5 MPa.

Designating a stress grade in this way also indicates that there is a corresponding set of values for other properties normally used in engineering design. See AS 1720 for tables of basic working stresses and moduli of elasticity.

Grading standards. Timber is sorted into grades according to the magnitude, position and extent of characteristics present in the piece. Structural grade descriptions limit all characteristics, i.e. imperfections and defects known to have a significant effect on strength, and describe material on the lower limit of that grade. Structural grade is designated in a form such as 'Structural Grade No. 2'. Structural Appearance Grades are derived from Structural Grades but have further limitations on the visual blemishes allowed in structural timbers exposed to view.

When specifying structural timber, it is preferable to nominate the required 'F' rating rather than to refer to a grade name. The following tables indicate how this may be achieved for unseasoned timber provided the strength group is known.

STANDARDS ASSOCIATION OF AUSTRALIA

Australian Standard

TIMBER STRUCTURES

PART 1: DESIGN METHODS

SECTION 1. SCOPE AND GENERAL

1.1 SCOPE. This Standard sets out the design methods for the structural use of timber which are based on the principles of structural mechanics and on data established by research. The Standard is intended for use in the design or appraisal of structural elements comprised of timber or wood products and of structures comprised substantially of timber. To this end, the Standard provides design data for sawn timber, laminated timber, timber in pole form, plywood and various types of fastenings. In addition, it provides methods of test for components or assemblies of unconventional design which may not be readily amenable to detailed analysis.

For ease of use, the simpler design situations are covered in the main body of the text. Related appendices, which form an integral part of the Standard, give acceptable procedures for detailed design situations.

1.2 REFERENCED DOCUMENTS. A list with titles of the documents referred to in this Standard is given in Appendix H.

1.3 NEW MATERIALS AND METHODS. This Standard shall not be interpreted to prevent the use of material or of methods of design or construction not specifically referred to herein. Nor is the classification of timbers into strength groups (Clause 1.4) or their grouping for joint design (Clause 4.1) to be interpreted as precluding the use of design stresses or other design data derived for a particular timber or grade of timber on the basis of authoritative research information.

NOTE: It usually will be necessary to seek approval from the Building Authority or other appropriate Regulatory Authority for the use of new materials or methods.

1.4 TIMBER QUALITY. All timber used in accordance with this Standard shall comply with the requirements of appropriate Australian Standards. The following points shall be noted:

(a) *General.* Tables 2.1 and 2.2 herein and AS 1720.2, list common species used for structural purposes together with their strength classifications and design density.

(b) *Timber classification.* Timber species are classified into seven strength groups S1 to S7 in the unseasoned condition and eight strength groups SD1 to SD8 in the seasoned condition. The timber species are also classified into six joint groups J1 to J6 if used unseasoned, and JD1 to JD6 if used seasoned. Sawn structural timber, pole timbers and plywood are classified into 12 stress grades F2 to F34 when these have been graded according to the appropriate grading Standard or other approved specification.

(c) *Stress grade and species identification.* Structural timber used in conjunction with this Standard shall have its stress grade identified.

For many purposes it may also be necessary to specify a particular species. When a particular species is specified the specification shall require that all pieces of timber be suitably identified as to species.

NOTES:

1. The design properties recommended in this Standard have been chosen on the assumption that structures of unseasoned timber that are allowed to dry will not receive their full design load until a period of air drying for at least 2 weeks has taken place. Freshly sawn timber which is unseasoned, or has recently been treated with waterborne chemicals, tends to have a reduced resistance and stiffness to sustained loads during the initial drying period.
2. Usually, only a limited number of the timber species and stress grades listed in this Standard will be readily available at any particular place and time.

(d) *Change of grade or durability.* Care shall be taken to account for any change in original grading or preservative treatment as a result of sawing or dressing. Regrading will be necessary if members are longitudinally resawn. Machining may remove preservative envelopes rendering the treatment ineffective.

(e) *Special provisions.* Design loads for timber joints and design rules for notched beams given herein are based on the assumption that there are no loose knots, severe sloping grain, gum veins, gum or rot pockets, lyctus-susceptible sapwood, holes or splits in the near vicinity of any connectors or notch roots.

(f) *Treated timber.* Timber, treated by impregnation with waterborne chemicals such as preservatives, is classified as unseasoned timber unless seasoning is specified.

NOTE: Where the material is reseasoned, regrading would normally be required.

1.5 GENERAL DESIGN CONSIDERATIONS.

1.5.1 Loads.

1.5.1.1 *General.* A structure, and any part of a structure, shall be designed for the loads specified in AS 1170 or such other loading Standard as is appropriate to the end-use of a specific structure or part of a structure.

1.5.1.2 *Load duration.* The significance of duration of loading in the design of timber structures shall be noted and particular attention paid to the term 'duration of loading'. (For the definition of this term see Clause 1.8.2 and for further information Clause 2.5.1.)

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(b) *Timber classification.* Timber species are classified into seven strength groups S1 to S7 in the unseasoned condition and eight strength groups SD1 to SD8 in the seasoned condition. The timber species are also classified into six joint groups J1 to J6 if used unseasoned, and JD1 to JD6 if used seasoned. Sawn structural timber, pole timbers and plywood are classified into 12 stress grades F2 to F34 when these have been graded according to the appropriate grading Standard or other approved specification.

(c) *Stress grade and species identification.* Structural timber used in conjunction with this Standard shall have its stress grade identified.

For many purposes it may also be necessary to specify a particular species. When a particular species is specified the specification shall require that all pieces of timber be suitably identified as to species.

NOTES:

1. The design properties recommended in this Standard have been chosen on the assumption that structures of unseasoned timber that are allowed to dry will not receive their full design load until a period of air drying for at least 2 weeks has taken place. Freshly sawn timber which is unseasoned, or has recently been treated with waterborne chemicals, tends to have a reduced resistance and stiffness to sustained loads during the initial drying period.
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(d) *Change of grade or durability.* Care shall be taken to account for any change in original grading or preservative treatment as a result of sawing or dressing. Regrading will be necessary if members are longitudinally resawn. Machining may remove preservative envelopes rendering the treatment ineffective.

(e) *Special provisions.* Design loads for timber joints and design rules for notched beams given herein are based on the assumption that there are no loose knots, severe sloping grain, gum veins, gum or rot pockets, lyctus-susceptible sapwood, holes or splits in the near vicinity of any connectors or notch roots.

(f) *Treated timber.* Timber, treated by impregnation with waterborne chemicals such as preservatives, is classified as unseasoned timber unless seasoning is specified.

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1.5.1.2 *Load duration.* The significance of duration of loading in the design of timber structures shall be noted and particular attention paid to the term 'duration of loading'. (For the definition of this term see Clause 1.8.2 and for further information Clause 2.5.1.)

1.5.2 Design methods.

1.5.2.1 *General.* A structure, or part of a structure, or an individual structural element shall be capable of sustaining the most adverse likely combination of loads. Every part of the structure shall be proportioned so that the permissible stresses determined in accordance with this Standard are not exceeded.

NOTE: Some of the clauses of this Standard have been simplified as far as practicable to permit rapid calculation and may as a consequence involve some degree of conservatism. Where appropriate, more refined design methods are given in the appendices which form an integral part of this Standard.

1.5.2.2 *Stress analysis.* All stresses shall be calculated on the basis of elastic theory in order that the requirements of this code in regard to permissible stresses may be satisfied with regard to the load effects at any particular location. For example the applied bending moment M and shear force V on a beam of rectangular cross-section shall be checked by—

$$M \leq (bd^2/6)F_b \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (1.1)$$

$$V \leq (2bd/3)F_s \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (1.2)$$

where

b and d = the breadth and depth of the member

F_b and F_s = the permissible design stresses in bending and shear.

When several materials are glued together to form a structural element, stresses may be calculated from an equivalent transformed section, where the transformation is made with respect to the moduli of elasticity.

1.5.2.3 *Experimentally based design.* Where a structure or a structural element is of an unconventional or complex nature, and it is demonstrated by the full-scale tests specified in Appendix A that requirements for strength, deformation, stability and serviceability are satisfied, the corresponding design requirements of this Standard shall be deemed also to have been satisfied.

1.5.3 Other design considerations.

1.5.3.1 *Stability.* The stability of the structure as a whole shall be investigated, and mass and anchorage shall be provided so that the structure is in overall equilibrium.

NOTE: Suitable recommendations for this purpose will be given in AS 1170.1.

1.5.3.2 *Buckling restraints.* Where there may be some doubt as to the effectiveness of buckling restraints, appropriate computations, such as those indicated in Paragraph C7, Appendix C, shall be made to check the stiffness and strength of the restraints.

1.5.3.3 *Erection and other extraneous forces.* Adequate provision shall be made to resist the lateral and other forces that can occur during the transport of structural elements, and during and after the erection of a structure.

1.5.3.4 *Secondary stresses.* Careful consideration shall be given to possible secondary stresses. Where these cannot be reduced to negligible proportions, suitable provisions in the design or some reduction in permissible primary stresses shall be made.

1.5.3.5 *Shrinkage.* When using unseasoned timber, consideration shall be given to the effects of shrinkage. Detailing of the joints shall not restrain shrinkage where splitting could render the joint ineffective. Consideration shall also be given to architectural detailing to avoid damage or unsightly appearances resulting from differential movement on structural members caused by timber shrinkage. These considerations also apply to timber which has been impregnated with waterborne chemicals and which has not been reseasoned after treatment.

NOTE: For most timbers the magnitude of shrinkage is in the range of 0.1% to 0.3% in the direction of the wood grain and 2% to 10% transverse to this direction. Information on shrinkage for specific species can be obtained from:

(a) KINGSTON, R.S.T. and RISDON C.J.E. 'Shrinkage and Density of Australian and Other South-west Pacific Woods'. Division of Forest Products Technological Paper No 13, CSIRO, 1961.

(b) BUDGEN, B. 'Shrinkage and Density of some Australian and South-east Asian Timbers'. Division of Building Research Technological Paper (Second Series) No 38, CSIRO, 1981.

1.5.3.6 *Deformations.* Timber structures shall be designed so that deformations incurred in-service do not impair the strength and serviceability of the structures or any part thereof, nor cause damage to other building components. Timber members shall have sufficient stiffness so that undesirable deflections and vibrations are avoided.

NOTES:

1. The responsibility for deflection and stiffness limits should rest with the design engineer.
2. In computing design deflections, it should be appreciated that timber is variable with respect to its structural properties. It should also be noted that the moduli given in Table 2.3 relate to design values for groups of timber. If for some reason (e.g. to specify camber) accurate predictions of deflection are required, detailed information relevant to the specific species of timber under consideration should be used.

1.5.3.7 *Timber dimensions for engineering calculations.* All engineering calculations shall be based on the minimum net cross-section. Such calculations shall not be based on the nominal cross-section.

1.5.3.8 *Timber in natural pole form.* For logs or poles complying with the quality requirements of AS 2209, the correspondence between strength groups and stress grades is as shown in Table 6.1.

1.5.3.9 *Biological deterioration.* Generally, timber under cover and in well ventilated conditions and not in contact with the ground or free water, is not subject to fungal attack. However, such timber may be subject to termite attack and to attack by other insects in parts of Australia. If conditions favourable for biological attack exist, then steps shall be taken to eliminate the hazards. This is particularly important in structures where there is no load sharing capacity, e.g. large trusses.

1.6 DESIGN AND SUPERVISION.

1.6.1 Design. The design of timber structures to which this Standard applies, including the specification of materials and any protective treatment, shall be carried out in accordance with the requirements of this Standard and the relevant documents in Appendix H.

NOTE: The design of a structure complying with this Standard should be the responsibility of a design engineer experienced in the design of such structures.

1.6.2 Supervision. The fabrication and erection of the timber structures or the parts of structures to which this Standard applies shall be supervised to ensure that all of the requirements of the design are satisfied in the completed structure.

NOTE: The supervision of fabrication and erection of timber structures should be the responsibility of a supervision engineer experienced in the fabrication and erection of such structures.

1.7 WORKMANSHIP AND MAINTENANCE.

1.7.1 General. The following requirements are intended to help ensure that a structure or element when fabricated performs, and will continue to perform, structurally in the manner intended by the designer of the structure.

1.7.2 Moisture content. When structures or elements are to be fabricated with seasoned timber in situations where dimensional stability is critical, the designer of the structure shall ascertain the average equilibrium moisture content for the environment in which the structures or elements are to be erected, and shall specify that each piece of timber used shall have an average moisture content at the time of fabrication that is within 3 percent of the equilibrium value.

NOTES:

1. Definitions used in this Standard for the moisture content of seasoned and unseasoned timber are given in Clause 1.8.2. For intermediate values of moisture content, the term 'partially seasoned timber' will be used.
2. Information on equilibrium moisture content values in timbers located in Australia can be obtained from the following references:

(a) FINIGHAN, R. 'Moisture Content Predictions for Eight Seasoned Timbers under Sheltered Outdoor Conditions in Australia and New Guinea'. Division of Forest Products Technological Paper No 44, CSIRO, 1966.

(b) BRAGG, C. 'An Equilibrium Moisture Content Survey of Timber in Queensland'. Queensland Department of Forestry Technical Paper No 40, QFD, 1986.

1.7.3 Corrosion. The designer of the structure shall take due account of any possible corrosive effects on metal connectors.

NOTE: Information on the protection of steel can be obtained from AS 2312.

1.7.4 Maintenance. Where in the opinion of the designer of a structure special maintenance is required for a structure to fulfil its intended function, then such maintenance shall be specified in relevant documents.

1.8 DEFINITIONS.

For the purpose of this Standard, the definitions given in AS O1 and those below apply.

1.8.1 Administrative definitions.

Building Authority or other Regulatory Authority—body having statutory powers to control the design and erection of buildings or structures, including scaffolding, in the area in which the building or structure concerned is to be erected.

Engineer—person qualified for Corporate Membership of The Institution of Engineers, Australia.

NOTE: The definition of engineer does not require that the person be a Corporate Member of The Institution of Engineers.

1.8.2 Technical definitions.

Basic working stress—stress appropriate to an arbitrarily chosen, but constant, basic reference set of conditions. It is derived from the known strength properties of a timber, due allowance having been made for such factors as material variability, long-duration loading, grade of timber, and a safety factor.

Basic working load for connectors—load appropriate to an arbitrarily chosen, but constant, basic reference set of conditions. It is derived from the known strength properties of the timber-connector system, due allowance having been made for such factors as material variability, long-duration loading, grade of timber, and a safety factor.

Collapse-susceptible timber—timber for which the shrinkage values before and after reconditioning differ by more than 2 percent.

NOTE: Information on shrinkage values can be obtained from:

(a) KINGSTON, R.S.T. and RISDON C.J.E. 'Shrinkage and Density of Australian and Other South-west Pacific Woods'. Division of Forest Products Technological Paper No 13, CSIRO, 1961.

(b) BUDGEN, B. 'Shrinkage and Density of some Australian and South-east Asian Timbers'. Division of Building Research Technological Paper (Second Series) No 38, CSIRO, 1981.

Corewood—timber adjacent to or including the pith, that is of density less than 80 percent that of the density of mature trees.

NOTE: For plantation grown softwoods, corewood may be avoided by excluding all timber within a radius of 50 mm from the pith, that has a ring width greater than 6 mm.

Duration of loading—period during which a member, a structural element or a complete structure is stressed as a consequence of the loads applied.

NOTES:

1. For the purposes of interpretation in the use of load-duration factors in this Standard, see Clause 2.5.1.
2. The strength properties of timber under load are time dependent.

In-grade verification—verification of the design properties assigned to stress graded timber. Where applicable, these properties shall be evaluated in accordance with AS 4063.

NOTE: Where AS 4063 is employed to assign design properties to stress graded timber, the stress grading procedures should be subjected to a continuing quality-control program.

Permissible stress—maximum stress to be used in the design of an element of a structure. It is obtained from the basic working stress appropriately modified for the type of structure and service conditions.

Seasoned timber—wood in which the maximum moisture content anywhere within a piece does not exceed 15 percent.

NOTE: Seasoned timber is sometimes referred to as 'dry' or 'air-dried' timber. It includes kiln-dried timber.

Stress grade—classification of timber for structural purposes by means of either visual or machine grading to indicate the basic working stresses and stiffnesses to be used for structural design purposes.

NOTE: The stress grade is designated in a form such as 'F7' which indicates that, for such a grade of material, the basic working stress in bending is approximately 7 MPa.

Unseasoned timber—wood in which the average moisture content of each piece exceeds 25 percent.

NOTE: Unseasoned timber is sometimes referred to as 'green' timber.

1.9 NOTATION.

Except where specifically defined in a particular clause, the quantity symbols and factors used in this Standard are listed in Appendix I.

1.10 UNITS.

Unless otherwise stated, the units of measurement used in this Standard are in accordance with the International System of Units (SI).

NOTE: In general N (newton), mm (millimetre) and MPa (megapascal) are appropriate units to be used.

SECTION 2. BASIC PROPERTIES OF STRUCTURAL TIMBER

2.1 GENERAL. Permissible stresses for structural timber shall be obtained through modifying basic working values by factors appropriate to the service conditions. This general procedure applies to all types of structural timber, including sawn timber, laminated timber, natural round timber and plywood.

2.2 STRUCTURAL CLASSIFICATIONS. Tables 2.1 and 2.2 list the structural classifications and design densities (for computing dead loads) of timber species and species groups that are commonly used in Australia.

The data given in Tables 2.1 and 2.2 are taken from other Standards, in particular AS 2082, AS 2209, AS 2269, AS 2858 and AS 2878; any changes to these Standards shall be taken to supersede the data cited herein. In addition, any stress grades evaluated through in-grade testing of full size structural material shall be taken to supersede all the above information.

NOTES:

1. The density of unseasoned timber depends on its moisture content which reduces as the timber dries. The values given in Tables 2.1 and 2.2 have been computed on the basis that the percentage saturation of the timber is 45 and 80 percent for softwoods and hardwoods respectively.
2. The values of density given in Tables 2.1 and 2.2 do not represent average values for the species indicated; they are intended for use in computing the dead loading imposed by timber.
3. The moduli of elasticity given in Table 2.3 are intended to represent average values except where species mixtures or species with high variability are concerned; in the latter case, the cited moduli of elasticity are less than the average values.
4. A more extensive list of timber species and species groups will be given in AS 1720.2.

2.3 BASIC WORKING STRESSES AND MODULUS OF ELASTICITY.

2.3.1 Basic working stresses parallel to grain, and shear stresses in beams. These basic working stresses are given in Table 2.3 for the various stress grades.

2.3.2 Basic working stress in compression perpendicular to the grain and shear stress at joint details. These basic working stresses are given in Table 2.4 for each strength group and are applicable to all stress grades within the strength group.

2.3.3 Basic working stress in compression at an angle to the grain. The basic working stress in compression at angles to the grain other than 0° and 90° shall be denoted by F'_θ and shall be calculated from the Hankinson formula

$$F'_\theta = (F'_c \times F'_p)/(F'_c \sin^2\theta + F'_p \cos^2\theta) \quad \ldots \text{(2.1)}$$

where

θ = the angle between the direction of the load and the direction of the grain.

2.3.4 Modulus of elasticity and rigidity. Design values of the modulus of elasticity and rigidity are given in Table 2.3.

NOTE: It should be noted that the modulus of elasticity, for the various stress grades given in Table 2.3, refers to the average modulus of elasticity for the stress-graded timbers that are grouped together within a stress grade. Therefore, when a better estimate for deflection is required, the modulus of elasticity values derived solely from Table 2.3 for a given stress grade should either be conservatively modified or accurate values should be obtained from in-grade verification.

2.3.5 Basic working stresses in tension (softwoods only) The basic working stresses in tension, for the various F-grades given in Table 2.3 for softwood timbers, shall be multiplied by the factor 0.85.

2.4 DESIGN.

2.4.1 Permissible stresses. Permissible stresses for structural timber, whether sawn or laminated construction, or in pole form, shall be obtained by multiplying the basic working stresses given in Clause 2.3 by modification factors such as those given in Clause 2.5 as are appropriate to the service conditions.

For example, F_b the permissible stress in bending is given by—

$$F_b = kF'_b \quad \ldots \text{(2.2)}$$

where

k = the product of the relevant modification factors, such as those in Clause 2.5, as are appropriate to the particular service conditions for which the structural member is being designed.

NOTES:

1. As an example, the factor k for the design bending stress of a solid timber beam is typically given by $k = k_1k_2k_3k_4$...
2. For convenience, the modification factors are collated and referenced in Appendix I.

2.4.2 Deflections. Deflection calculations shall take into account the modification factors in Clause 2.5.1.2.

2.5 MODIFICATION FACTORS.

2.5.1 Duration of load.

2.5.1.1 *Effect on strength.* In order to determine permissible design stresses, the basic working stresses shall be multiplied by the appropriate duration of load factor k_1 from Table 2.5. This factor is shown graphically in Figure 2.1.

In checking the strength of a structural element, all load combinations must be considered.

For any given combination of loads of differing duration, the factor k_1 to be used is that appropriate to the load which is of the shortest duration. In Table 2.5 the effective duration of a peak load refers to the cumulative duration for which the peak load occurs.

For the purposes of interpretation in the selection of load-duration factors in this Standard, the following shall apply:

(a) Dead loads, and live loads which are removed or replaced at regular intervals such that the structure remains fully loaded for a substantial proportion of its life, are to be considered 'permanent loads'.

(b) Live loads (such as those due to vehicles or people) that act on floors, and are applied at frequent but irregular intervals such that the structure is unloaded, or loaded well below the allowable maximum, for most of each day, are to be considered 'loads of five months duration'.

(c) Live loads, such as those arising during erection and maintenance, and at infrequent crowd loading, applied for periods of a few days and at infrequent intervals, are to be considered 'loads of five days duration'.

(d) Gust wind loads with a long return period, such as those referred to in AS 1170.2, and impact loads such as those caused by falling weights or snatch lifts, are to be considered 'loads of five second duration'.

2.5.1.2 *Effect on stiffness.* For members in bending and compression or for members in tension, the calculated short-term deformation shall be multiplied by the appropriate creep factor j_2 or j_3, as given in Table 2.6 and illustrated graphically in Figures 2.2 and 2.3.

Values intermediate between those given in Table 2.6 may be obtained through an interpolation involving the logarithm of time, and a linear function of initial moisture content as shown in Figures 2.2 and 2.3.

When several types of load act on a timber member, the maximum deformation shall be taken to be equal to the sum of the deformations computed for each type of load acting alone.

The modification factors j_2 and j_3 given in Table 2.6 are not applicable to collapse susceptible hardwoods (see Clause 1.8.2) when their initial moisture content is above 25%. For these timbers the creep factors may be considerably greater than the values shown.

NOTES:

1. The loads to be considered in computing deflections are not only the peak loads used for strength checks, but all loads that act during the life of the structure. In general, peak values of live load are not of a permanent nature; accordingly if a designer wishes to compute the long term deformations of a structure he must first estimate the portion of the load that is permanently or semi-permanently applied, and then use an appropriate creep factor.
2. Where there is a recovery period of more than ten times that of the applied load, the creep component of deformation may be assumed to be totally recovered.

2.5.2 Moisture condition. Depending on the initial moisture content of the timber and the moisture content at time of loading and throughout its life, the basic working stresses shall be modified as follows:

(a) *Unseasoned timber.* Where unseasoned timber is used, the basic working stresses shall be those in Tables 2.3 and 2.4 appropriate to the stress grade and strength group of the unseasoned timber as indicated in Tables 2.1 and 2.2.

(b) *Unseasoned timber partly dry before use.* Where unseasoned timber is used under normal conditions of temperature and humidity and will not be subject to its full design load until it has partly seasoned, i.e. to below 25% moisture content, the basic working stresses for unseasoned timber may be increased by multiplying by the factor k_4 given in Table 2.7.

(c) *Seasoned timber.*

(i) Where seasoned timber is used, the basic working stresses shall be those in Tables 2.3 and 2.4 appropriate to the stress grade and strength group of the timber in the seasoned condition as indicated in Tables 2.1 and 2.2.

(ii) Where seasoned timber is subjected to conditions in which its average moisture content for a 12-month period is expected to exceed 15%, the basic working stresses shall be decreased by multiplying by the factor k_5 determined as the greater of:

$$k_5 = 1 - \left[\frac{EMC - 15}{10}\right]\left[1 - \frac{F'_{(\text{unseasoned})}}{F'_{(\text{seasoned})}}\right] \quad \text{(2.3(a))}$$

and

$$k_5 = \frac{F'_{(\text{unseasoned})}}{F'_{(\text{seasoned})}} \quad \ldots\ldots\ldots\ldots\ldots \text{(2.3(b))}$$

where

EMC = the highest value of the annual average moisture content (per-cent) that the timber will attain in service

$F'_{(\text{seasoned})}$ = the basic working stress for the seasoned material

$F'_{(\text{unseasoned})}$ = the basic working stress for material of the same grade in the unseasoned condition.

2.5.3 Temperature. For covered timber structures under ambient conditions, no modification to the basic working stresses need be made for the effect of temperature except that where seasoned timber is used in structures erected in the coastal regions of Queensland north of latitude 25°S and all other regions of Australia north of latitude 16°S, the basic working stresses shall be multiplied by a factor k_6 of 0.9.

NOTE: Information on the effects of high temperatures can be obtained from:

MEYER, R.W. and KELLOG, R.M. *Structural Use of Wood in Adverse Environments*. Van Nostrand, 1982.

2.5.4 Length and position of bearing. For rectangular bearing areas for bearings of length less than 150 mm and with the bearing surface 75 mm or more from the end of a piece of timber, the basic working stress in bearing perpendicular to the grain given in Table 2.4 may be multiplied by the appropriate factor k_7 in Table 2.8, the length of bearing being measured parallel to the grain of the loaded member.

For circular bearing areas the effective bearing length shall be taken as being equal to the diameter of the bearing area.

TABLE 2.1

STRENGTH CLASSIFICATIONS AND DESIGN DENSITY FOR SOME COMMON GROUPS OF TIMBER

Species group	Moisture condition	Strength group[1]	Joint group[2]	Stress grade: Structural timber[3] Structural No 1	Structural No 2	Structural No 3	Structural No 4	Structural No 5	Structural plywood[4]	Pole timber[5]	Design density[6] kg/m³
Mixed Australian hardwoods (excluding rainforest species) from S.A. and southern N.S.W.	Unseasoned	S4	J3	F14	F11	F8	F7	—	—	F11	1050
	Seasoned	SD4	JD3	F22	F17	F14	F11	—	F17	—	650
Ash-type Eucalypts from NSW Highlands, Victoria and Tasmania	Unseasoned	S4	J3	F14	F11	F8	F7	—	—	F17	1050
	Seasoned	SD4	JD3	F22	F17	F14	F11	—	F17	—	650
Non-ash-type Eucalypts from Qld and N.S.W.	Unseasoned	S3	J2	F17	F14	F11	F8	—	—	F22	1150
	Seasoned	SD3	JD2	F27	F22	F17	F14	—	F22	—	750
Rainforest species	Unseasoned	S7	J4	F7	F5	F4	—	—	—	F8	800
	Seasoned	SD7	JD4	F11	F8	F7	F5	—	F8	—	500
Mixed *pinus* species (Australian grown)	Unseasoned	—	—	—	—	—	—	—	—	—	850
	Seasoned	SD7	JD4	F11	F8	F7	F5	—	F8	—	550
Mixed softwood species (excl. *pinus* species)	Unseasoned	—	—	—	—	—	—	—	—	—	850
	Seasoned	SD8	JD4	F8	F7	F5	—	—	F7	—	500
Imported softwoods (unidentified)	Unseasoned	S7	J6	F7	F5	F4	—	—	—	—	850
	Seasoned	SD8	JD6	F8	F7	F5	F4	—	F7	—	400

NOTES:

1. For classification into strength groups—see AS 2878.
2. For joint strength—see AS 1649.
3. For mechanical stress grades—see AS 1748.
4. For structural plywood—see AS 2269.
5. For timber poles—see AS 2209.
6. For use only in computing dead load due to mass in timber.

TABLE 2.2

STRENGTH CLASSIFICATIONS AND DESIGN DENSITY FOR SOME COMMON SPECIES OF TIMBER

Species	Moisture condition	Strength group[1]	Joint group[2]	Stress grade: Structural timber[3] Structural No 1	Structural No 2	Structural No 3	Structural No 4	Structural No 5	Structural plywood[4]	Pole timber[5]	Design density[6] kg/m³
ash, alpine	Unseasoned	S4	J3	F14	F11	F8	F7	—	—	F17	1 050
	Seasoned	SD4	JD3	F22	F17	F14	F11	—	F17	—	650
ash, mountain	Unseasoned	S4	J3	F14	F11	F8	F7	—	—	F17	1 050
	Seasoned	SD3	JD3	F27	F22	F17	F14	—	F22	—	650
ash, silvertop	Unseasoned	S3	J2	F17	F14	F11	F8	—	—	F22	1 100
	Seasoned	SD3	JD2	F27	F22	F17	F14	—	F22	—	850
balau	Unseasoned	S2	J2	F22	F17	F14	F11	—	—	F27	1 150
	Seasoned	SD3	JD2	F27	F22	F17	F14	—	F22	—	900
blackbutt	Unseasoned	S2	J2	F22	F17	F14	F11	—	—	F27	1 150
	Seasoned	SD2	JD2	F34	F27	F22	F17	—	F27	—	900
box, brush	Unseasoned	S3	J2	F17	F14	F11	F8	—	—	F22	1 150
	Seasoned	SD3	JD2	F27	F22	F17	F14	—	F22	—	900
box, grey, coast	Unseasoned	S1	J1	F27	F22	F17	F14	—	—	F34	1 200
	Seasoned	SD1	JD1	F34	F34	F27	F22	—	F34	—	1 100
brown barrel	Unseasoned	S4	J3	F14	F11	F8	F7	—	—	F17	1 100
	Seasoned	SD4	JD3	F22	F17	F14	F11	—	F17	—	750
chengal	Unseasoned	S1	J2	F27	F22	F17	F14	—	—	F34	1 150
	Seasoned	SD2	JD2	F34	F27	F22	F17	—	F27	—	950
fir, Douglas, North America	Unseasoned	S5	J4	F11	F8	F7	F5	F4	—	F14	710
	Seasoned	SD5	JD4	F14	F11	F8	F7	F5	F14	—	550
fir, Douglas, elsewhere	Unseasoned	S6	J5	F8	F7	F5	F4	—	—	F11	710
	Seasoned	SD6	JD5	F14	F11	F8	F7	F5	F11	—	550
gum, blue, southern	Unseasoned	S3	J2	F17	F14	F11	F8	—	—	F22	1 150
	Seasoned	SD2	JD2	F34	F27	F22	F17	—	F27	—	1 000
gum, blue, Sydney	Unseasoned	S3	J2	F22	F17	F14	F11	—	—	F27	1 100
	Seasoned	SD3	JD2	F27	F22	F17	F14	—	F22	—	850
gum, red, river	Unseasoned	S5	J2	F11	F8	F7	F5	—	—	F14	1 150
	Seasoned	SD5	JD2	F17	F14	F11	F8	—	F14	—	900
gum, rose	Unseasoned	S3	J2	F17	F14	F11	F8	—	—	F22	1 100
	Seasoned	SD4	JD2	F22	F17	F14	F11	—	F17	—	750

(continued)

TABLE 2.2 (continued)

Species	Moisture condition	Strength group[1]	Joint group[2]	Stress grade							Design density[6]
				Structural timber[3]							
				Structural No 1	Structural No 2	Structural No 3	Structural No 4	Structural No 5	Structural plywood[4]	Pole timber[5]	kg/m³
gum, spotted	Unseasoned	S2	J1	F22	F17	F14	F11	—	—	F27	1 200
	Seasoned	SD2	JD1	F34	F27	F22	F17	—	F27	—	1 100
hardwood, Johnston River	Unseasoned	S2	J1	F22	F17	F14	F11	—	—	F27	1 150
	Seasoned	SD3	JD1	F27	F22	F17	F14	—	F22	—	950
Hemlock, western	Unseasoned	S6	J4	F8	F7	F5	F4	—	—	F11	750
	Seasoned	SD6	JD4	F14	F11	F8	F7	F5	F11	—	500
Hem-fir[7]	Unseasoned	S7	J5	F7	F5	F4	—	—	—	F8	750
	Seasoned	SD7	JD5	F11	F8	F7	F5	F4	F8	—	550
ironbark, grey	Unseasoned	S1	J1	F27	F22	F17	F14	—	—	F34	1 250
	Seasoned	SD1	JD1	—	F34	F27	F22	—	F34	—	1 100
ironbark, red, narrow-leaved	Unseasoned	S2	J1	F22	F17	F14	F11	—	—	F27	1 250
	Seasoned	SD3	JD1	F27	F22	F17	F14	—	F22	—	1 050
jarrah	Unseasoned	S4	J2	F14	F11	F8	F7	—	—	F17	1 100
	Seasoned	SD4	JD2	F22	F17	F14	F11	—	F17	—	800
kapur	Unseasoned	S3	J2	F17	F14	F11	F8	—	—	F22	1 100
	Seasoned	SD4	JD2	F22	F17	F14	F11	—	F22	—	750
karri	Unseasoned	S3	J2	F17	F14	F11	F8	—	—	F22	1 150
	Seasoned	SD2	JD2	F34	F27	F22	F17	—	F27	—	900
kempas	Unseasoned	S2	J1	F22	F17	F14	F11	—	—	F27	1 100
	Seasoned	SD2	JD2	F34	F27	F22	F17	—	F27	—	900
kwila (Merbau)	Unseasoned	S2	J2	F22	F17	F14	F11	—	—	F27	1 150
	Seasoned	SD3	JD2	F34	F27	F22	F17	—	F27	—	850
lumbayu, Chengkulang	Unseasoned	S5	J3	F11	F8	F7	F5	—	—	F14	1 100
	Seasoned	SD5	JD3	F17	F14	F11	F8	—	F14	—	750
mahogany, red	Unseasoned	S2	J1	F22	F17	F14	F11	—	—	F27	1 200
	Seasoned	SD3	JD1	F27	F22	F17	F14	—	F22	—	950
marri	Unseasoned	S3	J2	F17	F14	F11	F8	—	—	F22	1 100
	Seasoned	SD3	JD2	F27	F22	F17	F14	—	F22	—	850
meranti, dark red[7]	Unseasoned	S5	J4	F11	F8	F7	F5	—	—	F14	1 100
	Seasoned	SD6	JD4	F14	F11	F8	F7	—	F11	—	[illegible]

TABLE 2.2 (continued)

Species	Moisture condition	Strength group[1]	Joint group[2]	Stress grade							Design density[6]
				Structural timber[3]							
				Structural No 1	Structural No 2	Structural No 3	Structural No 4	Structural No 5	Structural plywood[4]	Pole timber[5]	kg/m³
mersawa	Unseasoned	S6	J3	F8	F7	F5	F4	—	—	F11	1 050
	Seasoned	SD6	JD3	F14	F11	F8	F7	—	F11	—	700
messmate	Unseasoned	S3	J3	F17	F14	F11	F8	—	—	F22	1 100
	Seasoned	SD3	JD3	F27	F22	F17	F14	—	F22	—	750
oak, tulip brown	Unseasoned	S2	J2	F22	F17	F14	F11	—	—	F27	1 150
	Seasoned	SD2	JD2	F34	F27	F22	F17	—	F27	—	900
pine, cypress, white	Unseasoned	S5	J3	—	—	F7	F5	F4	—	F14	850
	Seasoned	SD6	JD3	—	—	—	—	—	F11	—	700
pine, hoop	Unseasoned	S6	J4	F8	F7	F5	F4	—	—	F11	800
	Seasoned	SD5	JD4	F17	F14	F11	F8	F7	F14	—	550
pine, radiata (Australia and New Zealand)	Unseasoned	S6	J4	—	—	—	—	—	—	F11	800
	Seasoned	SD6	JD4	F14	F11	F8	F7	F5	F11	—	550
pine, slash	Unseasoned	S5	J3	F11	F8	F7	F5	F4	—	F14	850
	Seasoned	SD5	JD3	F17	F14	F11	F8	F7	F14	—	650
spruce-pine-fir[7]	Unseasoned	—	—	—	—	—	—	—	—	—	700
	Seasoned	SD7	JD5	F8	F8	F7	F5	F4	F8	—	500
stringybark brown	Unseasoned	S3	J2	F17	F14	F11	F8	—	—	F22	1 100
	Seasoned	SD3	JD2	F27	F22	F17	F14	—	F22	—	850
stringybark yellow	Unseasoned	S3	J2	F17	F14	F11	F8	—	—	F22	1150
	Seasoned	SD3	JD2	F27	F22	F17	F14	—	F22	—	900
tallowwood	Unseasoned	S2	J1	F22	F17	F14	F11	—	—	F27	1 200
	Seasoned	SD2	JD2	F34	F27	F22	F17	—	F27	—	1 000
turpentine	Unseasoned	S3	J2	F17	F14	F11	F8	—	—	F22	1 050
	Seasoned	SD3	JD2	F27	F22	F17	F14	—	F22	—	950
wandoo	Unseasoned	S2	J1	F22	F17	F14	F11	—	—	F27	1 250
	Seasoned	SD3	JD1	F27	F22	F17	F14	—	F22	—	1 100

NOTES:

1. For classification into strength groups—see AS 2878.
2. For joint strength—see AS 1649.
3. For mechanical stress grades—see AS 1748.
4. For structural plywood—see AS 2269.
5. For timber poles—see AS 2209.
6. For use only in computing dead load due to mass in timber.
7. Species mixture.

TABLE 2.3
BASIC WORKING STRESSES AND STIFFNESS FOR STRUCTURAL TIMBER

Stress grade	Basic working stress, MPa				Short duration modulus of elasticity* (E)	Short duration modulus of rigidity (G)
	Bending (F'_b)	Tension parallel to grain (F'_t)	Shear in beams (F'_s)	Compression parallel to grain (F'_c)		
F34	34.5	20.7	2.45	26.0	21 500	[illegible]
F27	27.5	16.5	2.05	20.5	18 500	[illegible]
F22	22.0	13.2	1.70	16.5	16 000	[illegible]
F17	17.0	10.2	1.45	13.0	14 000	[illegible]
F14	14.0	8.4	1.25	10.2	12 000	[illegible]
F11	11.0	6.6	1.05	8.4	10 500	[illegible]
F8	8.6	5.2	0.85	6.6	9 100	[illegible]
F7	6.9	4.1	0.70	5.2	7 900	[illegible]
F5	5.5	3.3	0.60	4.1	6 900	[illegible]
F4	4.3	2.6	0.50	3.3	6 100	[illegible]
F3	3.4	2.0	0.45	2.6	5 200	[illegible]
F2	2.7	1.6	0.35	2.1	4 500	[illegible]

* The modulus of elasticity includes an allowance of about 5 percent for shear deformation.

NOTE: For the basic working stresses in tension for softwoods **only**, refer to Clause 2.3.5.

TABLE 2.4
BASIC WORKING STRESSES FOR COMPRESSION PERPENDICULAR TO GRAIN AND SHEAR AT JOINTS

Strength group		Basic working stress, MPa	
Unseasoned	Seasoned	Compression perpendicular to grain (F'_p)	Shear at joints details (F'_{sj})
—	SD1	10.4	4.15
—	SD2	9.0	3.45
—	SD3	7.8	2.95
S1	SD4	6.6	2.45
S2	SD5	5.2	2.05
S3	SD6	4.1	1.70
S4	SD7	3.3	1.45
S5	SD8	2.6	1.25
S6	—	2.1	1.05
S7	—	1.7	0.85

TABLE 2.5
DURATION OF LOAD FACTOR FOR STRENGTH

Type of load	Effective duration of peak load	Multiplying factor (k_1)	
		Basic stresses for solid timber	Basic working loads for laterally loaded connectors*
Instantaneous	5 seconds	1.75	2.00
Standard test	5 minutes	1.75	1.75
Short term	5 hours	1.70	1.50
Medium term	5 days	1.65	1.35
Long term	5 months	1.40	1.20
Permanent	50+ years	1.00	1.00

* For connectors loaded in withdrawal and for the strength of steel in connectors, k_1 = 1.00.

TABLE 2.6
DURATION OF LOAD FACTOR FOR DEFLECTION

Initial moisture content*	For bending, compression and shear members (j_2)		For tension members (j_3)	
%	Load duration ≤ 1 day	Load duration ≥ 1 year	Load duration ≤ 1 day	Load duration ≥ 1 year
≤ 15	1	2	1	1
≥ 25	1	3	1	1.5

* Moisture content at the time of load application.

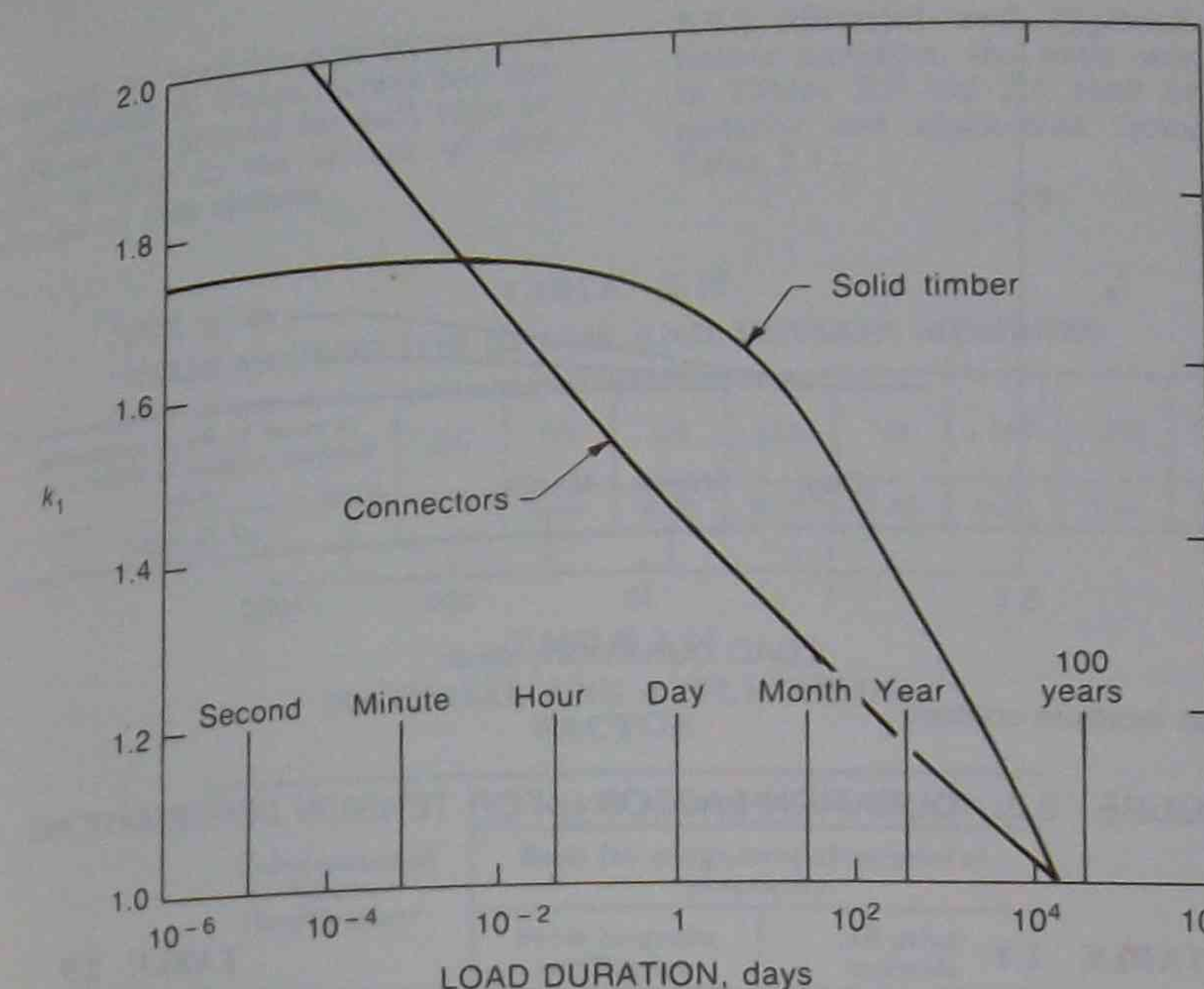
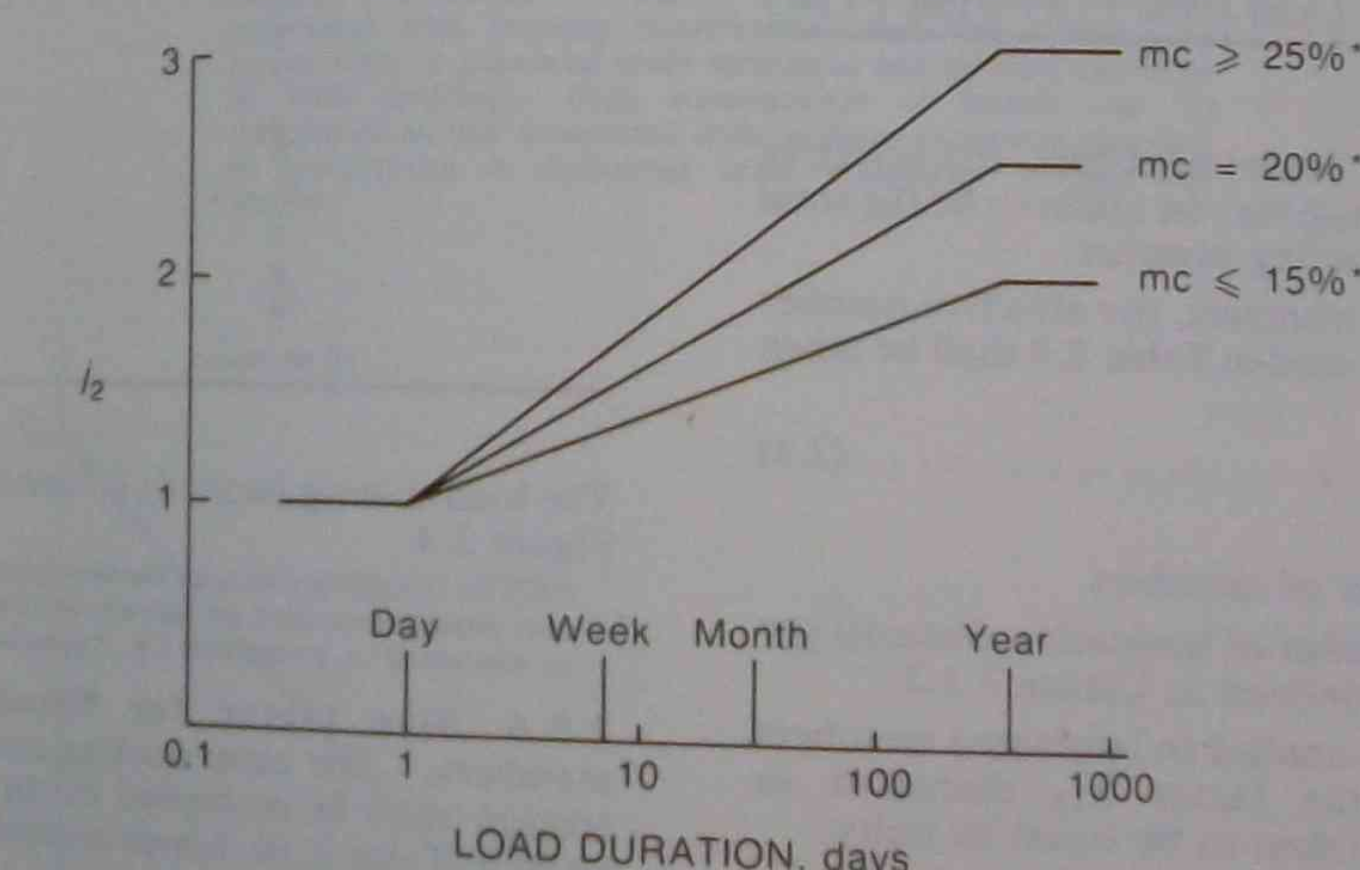


FIGURE 2.1. DURATION OF LOAD FACTOR k_1



*Initial moisture content

FIGURE 2.2. DURATION FACTOR j_2 FOR BENDING AND COMPRESSION DEFORMATIONS

2.5.5 Load sharing.

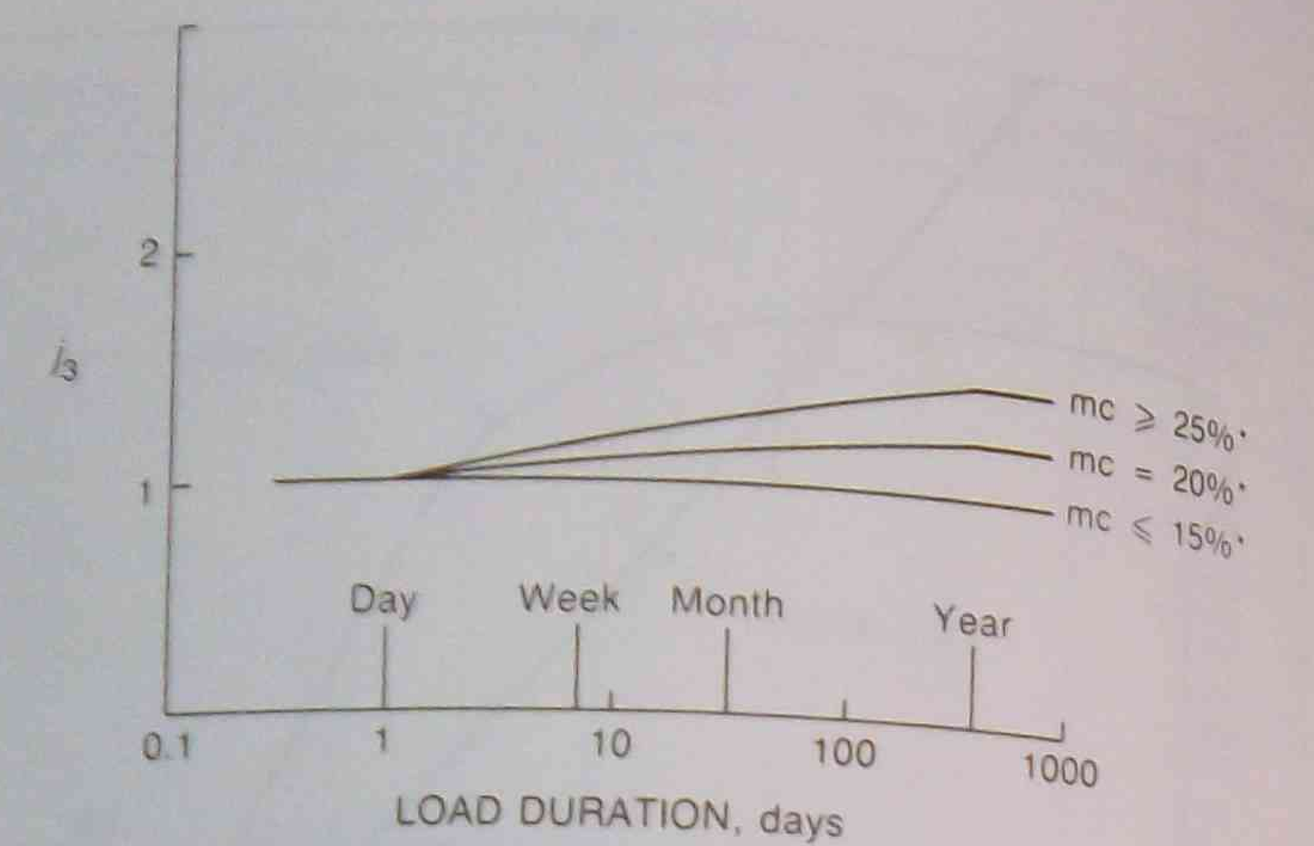
2.5.5.1 *General.* When a structural system consists of parallel acting elements that interact to assist each other, then the basic working stresses may be increased by the appropriate load sharing factor.

2.5.5.2 *Parallel structural systems.* For structural systems comprised of two or more elements effectively connected so that all of the elements are constrained to the same deformation, the load sharing factor k_8 may be obtained from Table 2.9, and applied to the basic working stresses for bending and compression.

If the effective number of elements is not an exact integer, then a suitable value of k_8 may be derived by linear interpolation.

TABLE 2.7
PARTIAL SEASONING FACTOR

Least dimension of member	38 mm or less	50 mm	75 mm	100 mm or more
Value of k_4	1.15	1.10	1.05	1.00



*Initial moisture content

FIGURE 2.3 DURATION FACTOR j_3 FOR TENSION DEFORMATIONS

TABLE 2.8
LENGTH OF BEARING FACTOR

Length of bearing of member	12	25	50	75	125	150 or more
Value of k_7	1.85	1.60	1.30	1.15	1.05	1.00

Except for laminated timber members, the effective number of elements (n_{eff}) may be taken to be the total number of members acting together.

For laminated timber members, the effective number of elements (n_{eff}) to be used in Table 2.9 shall be taken as—

$$n_{eff} = n_m \times n_L \quad \text{(2.4)}$$

where

n_m = total number of members

n_L = effective number of lamination elements per member as defined in Clause 7.3.2.

Where the factor k_8 is applied to laminated members acting in parallel, the factor k_{21} discussed in Clause 7.3.2 shall be taken to be equal to unity.

2.5.5.3 *Grid systems.* Where constructions are such that three or more members act together to support either an overlying set of members usually laid at right angles to the supporting members or a structural sheathing material, a load sharing factor k_9 may be applied to the basic working stress for bending, in beams. This factor is given by the equation:

$$k_9 = k_8 + (k_8 - k_0)\,[1.0 - 2(s/L)] \quad \text{(2.5)}$$

but not less than k_0, where

s = the centre-to-centre spacing of the supporting members

L = span of the supporting member

k_8 = load sharing factor for parallel structural systems (see Clause 2.5.5.2)

k_0 = 1.0 for solid timber

= k_{21} for glued-laminated timber (Clause 7.3.2)

TABLE 2.9
PARALLEL SUPPORT FACTOR

Effective number of elements carrying common load (n_{eff})	Factor k_8
1	[illegible]
2	[illegible]
3	[illegible]
4	[illegible]
5	[illegible]
6	[illegible]
7	[illegible]
8	[illegible]
9	[illegible]
10 or more	[illegible]

The load sharing factor k_9 is illustrated graphically in Figure 2.4.

NOTE: In addition to load sharing characteristics, ... also provide a method for laterally distributing concentrated ... as described in Paragraph C8, Appendix C.

2.5.6 Size factor for flexural and tension members. The basic working stress in bending ... tension shall be multiplied by the size factor k_{11} given in Table 2.10. Linear interpolation may be ... for intermediate sizes.

For beams of depth d greater than 1500 mm, the value of k_{11} shall be taken to be given by—

$$k_{11} = (300/d)^{0.167}$$

NOTE: The size factor for beams refers to beams of ... or glulam. For built up beams the size factor shall be ... the individual components; an example of this would ... tension flange of a box beam.

2.5.7 Stability factor. In the design of ... structural members, a factor k_{12} is used to ... the effects of slenderness on strength. It is defined ...

$$F = k_{12}F_o$$

where

F = the nominal design stress

F_o = the value of F if the structural member ... completely stable.

The factor k_{12} depends on both material factors and the slenderness coefficient S. These factors and the slenderness coefficient are defined for each type of slender structural member in the section of this Standard appropriate to that element.

2.5.8 Material and application factor For all timber members, the basic working stresses given in Tables 2.3 and 2.4 shall be multiplied by the material and application factor k_2, as shown in Table 2.11.

TABLE 2.10
SIZE FACTOR FOR BEAMS AND TENSION MEMBERS

Maximum depth of beam or twice width of tension member mm	300	375	500	625	750	1 000	1 250	1 500
Value of k_{11}	1	0.96	0.92	0.89	0.86	0.82	0.79	0.77

TABLE 2.11
MATERIAL AND APPLICATION FACTOR

Consequence of failure classification*	Material and application factor (k_2) — Basis for assignment of structural properties: From in-grade verification	All other methods
Normal	1.0	1.0
High	0.9	0.7

* Normal consequence of failure can be interpreted as that associated with housing construction, secondary framing in commercial or industrial scale structures and primary elements in farm buildings. High consequence of failure can be interpreted as that associated with primary structural elements in commercial or industrial scale structures, bridges and similar.

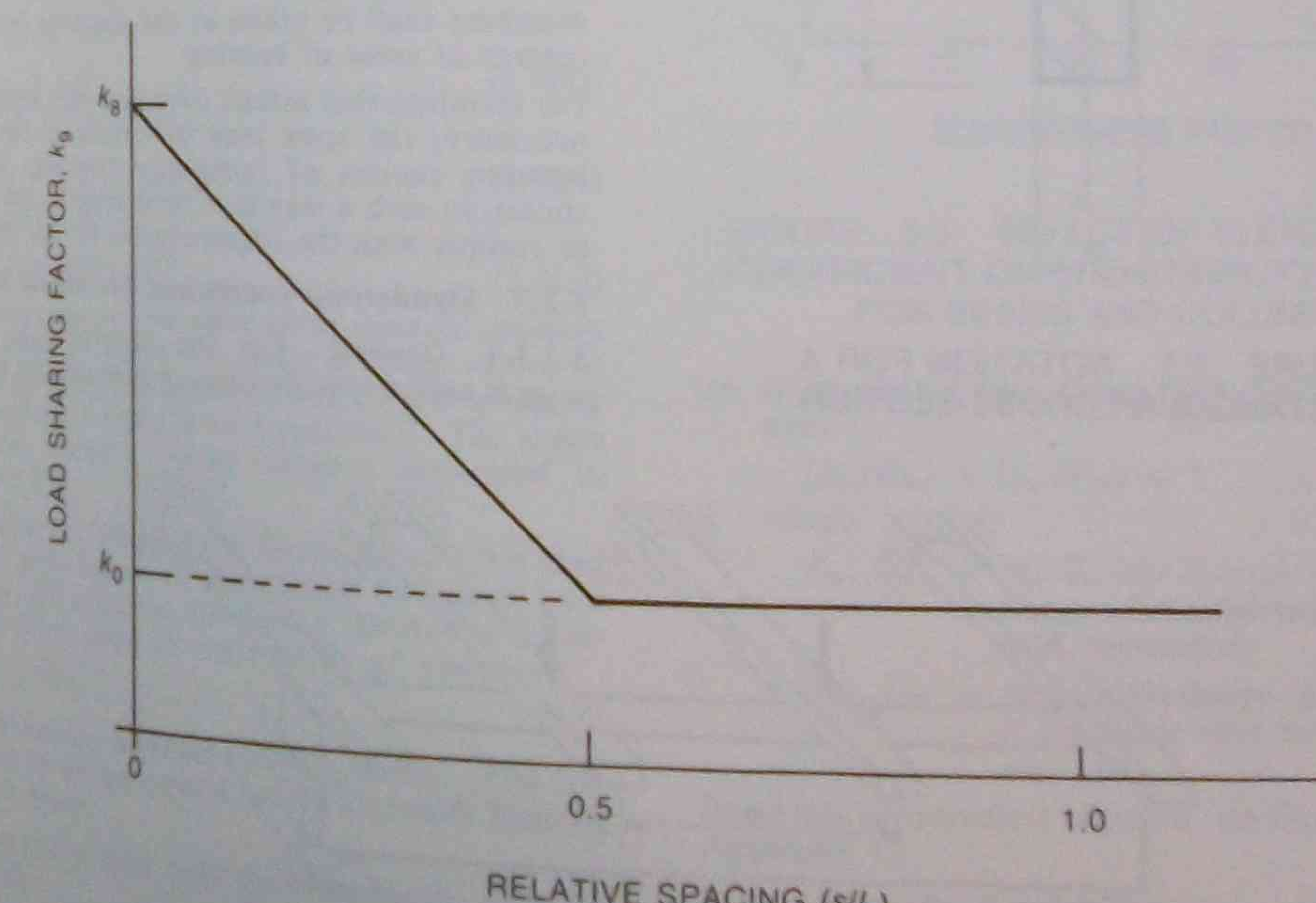


FIGURE 2.4 LOAD SHARING FACTOR k_9 FOR GRID SYSTEMS

SECTION 3. DESIGN OF BASIC STRUCTURAL MEMBERS

3.1 GENERAL. This Section shall be applied in conjunction with the clauses of Section 2. This Section applies to the design of basic structural members such as columns, beams and ties. In particular many of the design parameters given refer to members of rectangular cross-section, for which the notation used is shown in Figure 3.1. The corresponding parameters for members of less usual shape are given in Appendix C. Special design requirements related to the use of pole timbers, glued-laminated construction and plywood are given in later sections. Clauses for the design of more complex structural elements are given in Appendix C. These include clauses related to—

(a) the design of spaced columns (Paragraph C6);
(b) buckling restraint systems (Paragraph C7);
(c) grid systems (Paragraph C8);
(d) notched beams (Paragraph C9);
(e) notched columns (Paragraph C10); and
(f) notched tension members (Paragraph C11).

NOTE: In beam design deflection considerations will usually govern member sizes (see Clauses 1.5.3.6, 2.3, 2.4 and 2.5).

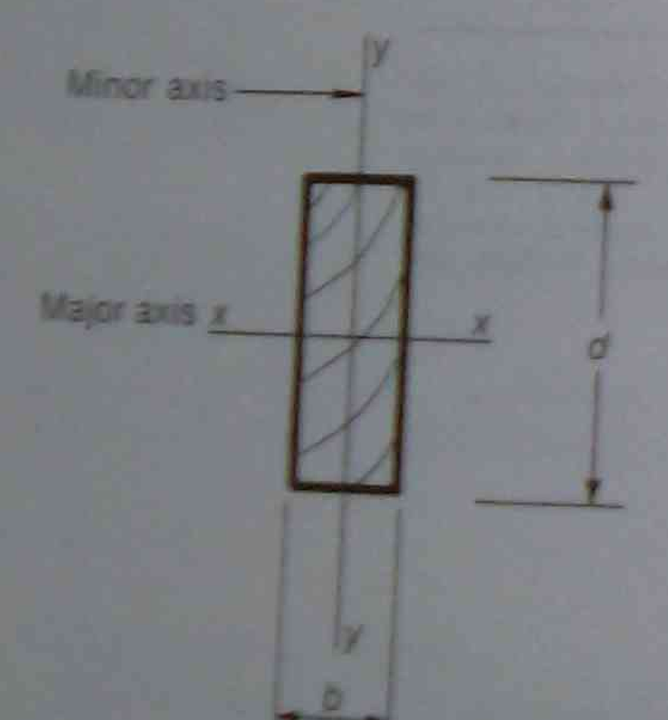


FIGURE 3.1 NOTATION FOR A RECTANGULAR CROSS-SECTION

3.2 BEAM DESIGN.

3.2.1 Maximum stresses. Calculated values of stresses in a beam shall not exceed the permissible stresses in bending F_b, in shear F_s, and in compression perpendicular to the grain F_p, determined in accordance with Clause 2.4 for sawn and laminated timber and for poles. Due regard shall be paid to the effective span and lateral stability, and to an acceptable deflection. (See also Clause 3.5 and Paragraph C... combined bending and axial loading, Paragraph C7, Appendix C for the design of ... and torsional restraints.)

When calculating a shear force in a beam, loads within a distance of the support of 1.5 times the ... of the beam from the inside face of the support ... be disregarded, except in the design of notched ... (Paragraph C9, Appendix C).

For unnotched beams, the permissible stress ... calculated by the following equations:

(a) In bending—

$$F_b = k_1k_2k_4k_5k_6k_8k_{11}k_{12}F_b'$$

(b) In shear—

$$F_s = k_1k_2k_4k_5k_6F_s'$$

(c) In compression perpendicular to the grain—

$$F_p = k_1k_2k_4k_5k_6k_7F_p'$$

where the factors k_1 to k_{11} are given in Section 2 ... k_{12} is the stability factor defined in Equation 3.8 ... Clause 3.2.4). For beams in grid systems the ... sharing factor k_8 is replaced by k_9.

3.2.2 Effective span. The effective span of ... members shall be taken as the distance between ... centres of areas of bearing.

For members that extend over bearings longer than necessary, the span may be taken as the distance between centres of imaginary bearings which ... chosen in such a way that their lengths are adequate to comply with the requirements of this Standard.

3.2.3 Slenderness coefficient for lateral buckling.

3.2.3.1 *General.* For the general case, and several useful specific cases, equations for evaluating the slenderness coefficient are given in Paragraph C3, Appendix C. For the special cases of solid beams of rectangular cross-section, the simple approximations given in Clause 3.2.3.2 may be used.

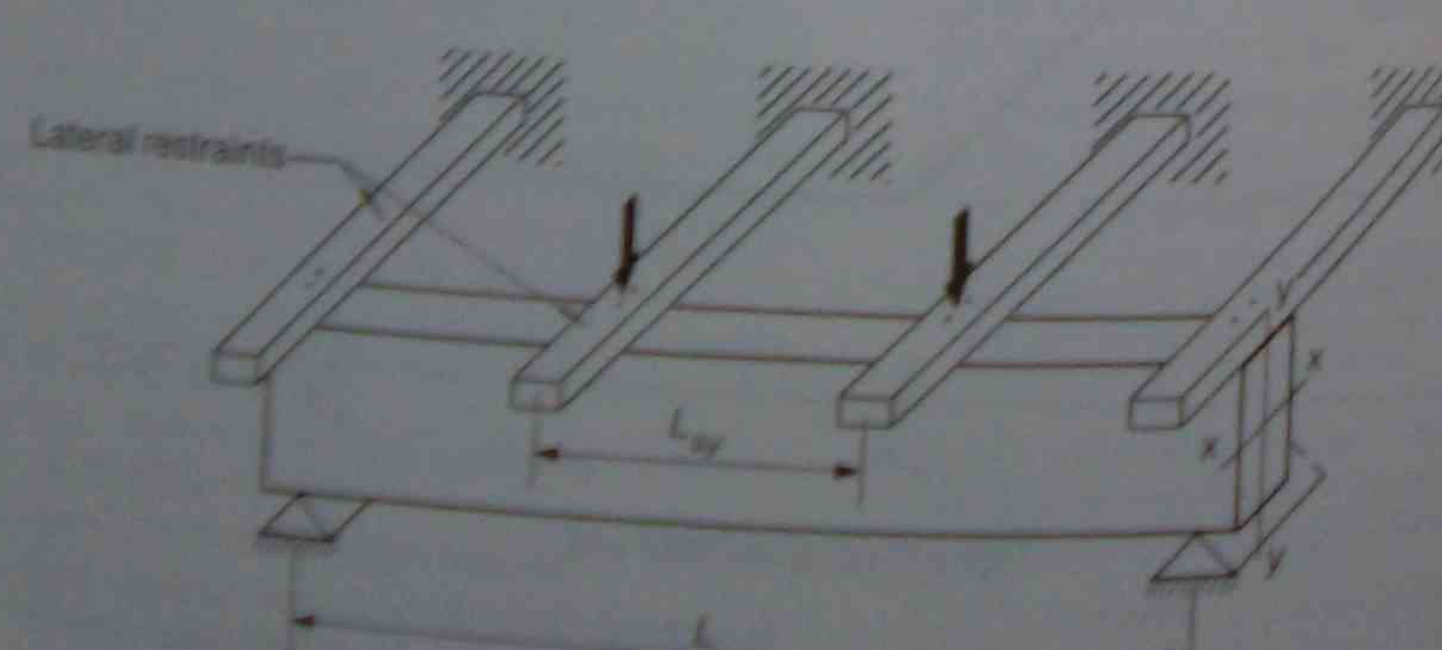


FIGURE 3.2 NOTATION FOR BEAM RESTRAINTS

3.2.3.2 *Beams of rectangular cross-section.* For beams of rectangular cross-section, the slenderness coefficients may be taken as follows:

(a) *Beams that bend only about their major axis.* For discrete restraint systems that effectively restrain the compression flange of the beam at points L_{ay} apart, the slenderness coefficient, denoted by S_1, may be taken to be—

$$S_1 = 1.25(L_{ay}d/b^2)^{1/2} \quad \ldots\ldots (3.4)$$

For restraint systems that are continuous along the compression flange of the beam, the slenderness coefficient may be taken to be—

$$S_1 = 0.0 \quad \ldots\ldots (3.5)$$

For restraint systems that are continuous along the tension flange of the beam, and in addition the load is applied to the tension flange, the slenderness coefficient may be taken to be—

$$S_1 = 2.5d/b \quad \ldots\ldots (3.6)$$

(b) *Beams that bend only about their minor axis.* For all cases, the slenderness coefficient, denoted by S_2, may be taken to be—

$$S_2 = 0.0 \quad \ldots\ldots (3.7)$$

(c) *Beams that bend about both axes.* The design of such beams described in Clause 3.2.5, is based on an interaction of the two special cases for bending about single axes only, and hence no special definition of slenderness is required for this case.

3.2.4 Stability factor. The stability factor k_{12} for modification of the basic working stress in bending shall be given by—

(a) For $\varrho S \leqslant 10$—

$$k_{12} = 1.0 \quad \ldots\ldots (3.8(a))$$

(b) For $10 \leqslant \varrho S \leqslant 20$—

$$k_{12} = 1.5 - 0.05\varrho S \quad \ldots\ldots (3.8(b))$$

(c) For $\varrho S \geqslant 20$—

$$k_{12} = 200/(\varrho S)^2 \quad \ldots\ldots (3.8(c))$$

where a conservative value of the material constant ϱ is given in Table 3.1; more accurate values of ϱ are given by Equations C1 and C2 and tabulated in Tables C1 and C2 of Appendix C. The shape of the stability factor curve is illustrated in Figure 3.3.

For large beams, where a size factor $k_{11} < 1.0$ is used either for solid beams or the tension flanges of built up beams, the material constant ϱ inserted in Equations 3.8 above may be replaced by ϱ^* where—

$$\varrho^* = \varrho\sqrt{k_{11}} \quad \ldots\ldots (3.9)$$

3.2.5 Allowable nominal bending stress. The following are the design criteria for the allowable bending stress in a beam:

(a) Beam that is bent only about its major axis (the x-axis)—

$$f_{bx}/F_{bx} \leqslant 1 \quad \ldots\ldots (3.10)$$

(b) Beam that is bent only about its minor axis (the y-axis)

$$f_{by}/F_{by} \leqslant 1 \quad \ldots\ldots (3.11)$$

TABLE 3.1
MATERIAL CONSTANT ϱ FOR BEAMS

Stress grade	Material constant ϱ	
	Seasoned timber	Unseasoned timber
F34	1.23	1.32
F27	1.18	1.27
F22	1.13	1.22
F17	1.08	1.17
F14	1.04	1.14
F11	1.00	1.09
F8	0.95	1.05
F7	0.91	1.01
F5	0.88	0.97
F4	0.84	0.93
F3	0.80	0.90
F2	0.78	0.87

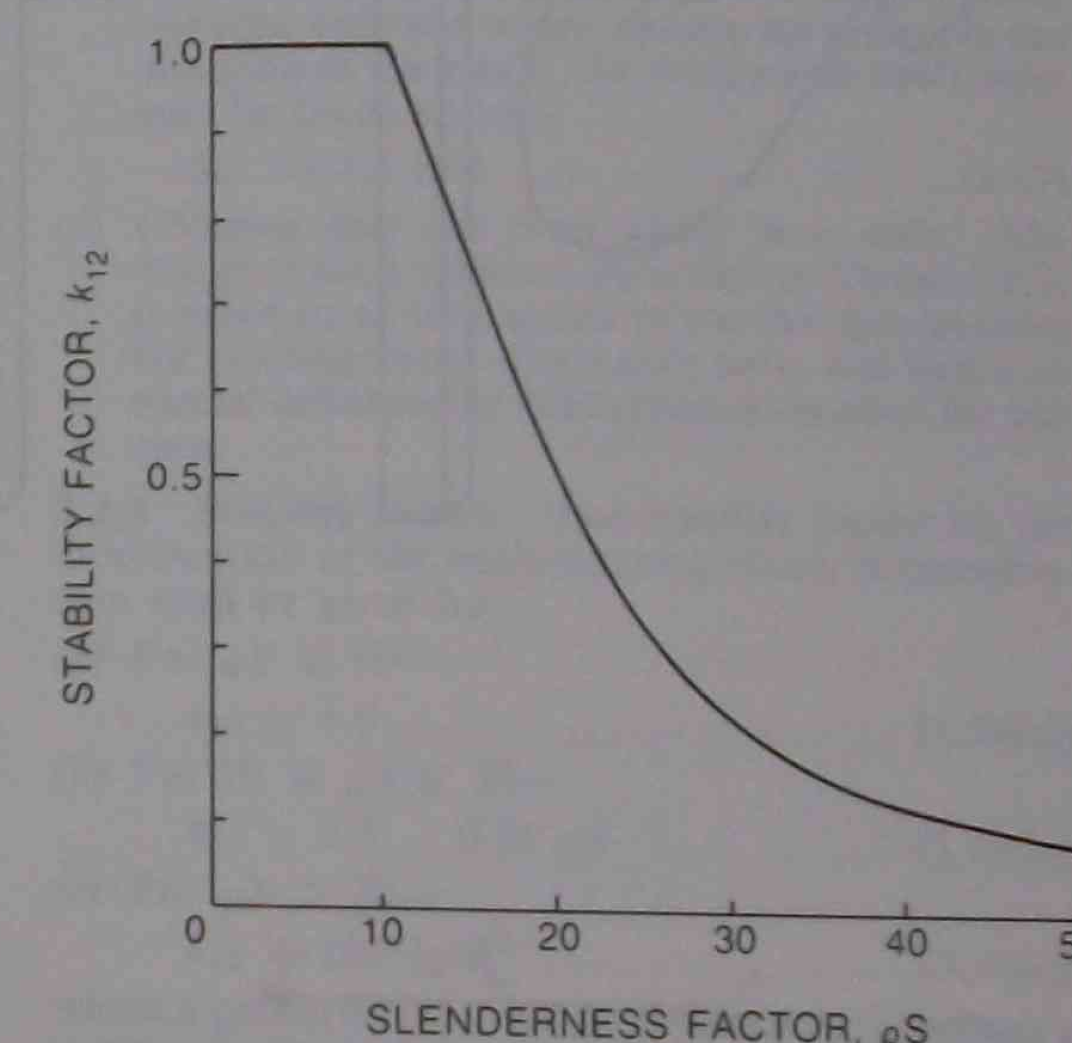


FIGURE 3.3. EFFECT OF SLENDERNESS COEFFICIENT ON THE STABILITY FACTOR FOR BEAMS AND COLUMNS

(c) Beam that is bent about both major and minor axes—

$$(f_{bx}/F_{bx}) + (f_{by}/F_{by}) \leqslant 1 \quad \ldots\ldots (3.12)$$

where

f_{bx}, f_{by} = M_x/Z_x, M_y/Z_y = calculated bending stresses about the major and minor axes respectively

F_{bx}, F_{by} = permissible design values of f_{bx}, f_{by} if the beam were bent about only one axis.

For a less conservative criterion, see Equation C14 of Appendix C.

3.2.6 Strength of notched beams. Clauses for the design strength of notched beams are given in Paragraph C9, Appendix C.

3.2.7 Concentrated loads and partial area loads on grid systems. Clauses to assist in the design of floor grid systems to resist concentrated and partial area loads are given in Paragraph C8, Appendix C.