DRAFT FOR DEVELOPMENT

DD ENV 1993-1-1:1992

Eurocode 3: Design of steel structures —

Part 1.1: General rules and rules for buildings —

(together with United Kingdom National Application Document)

UDC 624.92.014.2:624.07

Cooperating organizations

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This Draft for Development, having been prepared under the direction of the Technical Sector Board for Building and Civil Engineering (B/-), was published under the authority of the Standards Board and comes into effect on 15 November 1992

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Contents

National foreword

This publication comprises the English language version of ENV 1993-1-1:1992 Eurocode 3: *Design of Steel Structures — Part 1.1: General rules and rules for buildings,* as published by the European Committee for Standardization (CEN), plus the National Application Document (NAD) to be used with the ENV on the design of buildings to be constructed in the United Kingdom (UK).

ENV 1993-1-1:1992 results from a programme of work sponsored by the European Commission to make available a common set of rules for the design of building and civil engineering works.

An ENV is made available for provisional application, but does not have the status of a European Standard. The aim is to use the experience gained to modify the ENV so that it can be adopted as a European Standard.

The values for certain parameters in the ENV Eurocodes may be set by CEN members so as to meet the requirements of national regulations. These parameters are designated by $\boxed{}$ in the ENV.

During the ENV period reference should be made to the supporting documents listed in the National Application Document (NAD).

The purpose of the NAD is to provide essential information, particularly in relation to safety, to enable the ENV to be used for buildings constructed in the UK. The NAD takes precedence over corresponding provisions in the ENV.

The Building Regulations 1991, Approved Document A 1992, (published December 1991) identifies ENV 1993-1-1:1992 as appropriate guidance, when used in conjunction with the NAD, for the design of steel buildings.

Compliance with ENV 1993-1-1:1992 and the NAD does not in itself confer immunity from legal obligations.

Users of this document are invited to comment on its technical content, ease of use and any ambiguities or anomalies. These comments will be taken into account when preparing the UK national response to CEN on the question of whether the ENV can be converted to an EN.

Comments should be sent in writing to BSI, 2 Park Street, London W1A 2BS quoting the document reference, the relevant clause and, where possible, a proposed revision, within 2 years of the issue of this document.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to xxii, the ENV title page, pages 2 to 270, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

National Application Document

for use in the UK with ENV 1993-1-1:1991

Contents of National Application Document

Introduction

This National Application Document (NAD) has been prepared under the direction of the Technical Sector Board for Building and Civil Engineering. It has been developed from:

a) a textual examination of ENV 1993-1-1:1992;

b) a parametric calibration against BS 5950, supporting standards and test data;

c) trial calculations.

1 Scope

This NAD provides information to enable ENV 1993-1-1:1992 (EC3-1.1) to be used for the design of buildings to be constructed in the UK.

2 References

2.1 Normative references

This National Application Document incorporates, by reference, provisions from specific editions of other publications. These normative references are cited at the appropriate points in the text and the publications are listed on page xix. Subsequent amendments to, or revisions of, any of these publications apply to this National Application Document only when incorporated in it by updating or revision.

2.2 Informative references

This National Application Document refers to other publications that provide information or guidance. Editions of these publications current at the time of issue of this standard are listed on page xix, but reference should be made to the latest editions.

3 Partial safety factors, combination factors and other values

a) The values for partial safety factors (γ) should be those given in Table 1 and Table 2 of this NAD.

b) The values for combination factors (ψ) should be those given in Table 3 and Table 4 of this NAD.

c) The value of the reduction factor ψ_{vec} should be taken as 0.7.

Table 1 — Partial safety factors (γ factors)

Reference in EC3-1.1	Definition	Symbol	Condition	Value	
				Boxed EC3	UK
2.3.2.2(1)	Partial safety factors for accidental actions	γ_A	Accidental	1.00	1.05
2.3.2.2(3)	Partial safety factors for permanent actions in accidental design situation	γ_{GA}	Favourable	1.00	0.90
		γ_{GA}	Unfavourable	1.00	1.05
2.3.3.1(1)	Partial safety factors for permanent actions	$\gamma_{G, inf}$	Favourable	1.00	1.00
		$\gamma_{G, sup}$	Unfavourable	1.35	1.35
2.3.3.1(1)	Partial safety factors for variable action	$\gamma_{Q, inf}$	Favourable	0.00	0.00
		$\gamma_{Q, sup}$	Unfavourable	1.50	1.50
		$\gamma_{Q, sup}$	2 or more combined	1.50	1.50
2.3.3.1(3)	Partial safety factors for permanent action	$\gamma_{G, inf}$	Favourable part	1.10	1.10
		$\gamma_{G, sup}$	Unfavourable part	1.35	1.35
		$\gamma_{G, inf}$	Favourable and unfavourable parts	1.00	1.00

Table 1 — Partial safety factors (γ factors)

<table>
<tr><th rowspan="2">Reference in EC3-1.1</th><th rowspan="2">Definition</th><th rowspan="2">Symbol</th><th rowspan="2" colspan="2">Condition</th><th colspan="2">Value</th></tr>
<tr><th>Boxed EC3</th><th>UK</th></tr>
<tr><td rowspan="4">5.1.1</td><td rowspan="4">Partial safety factors for steel</td><td>γ_{M0}</td><td colspan="2">Resistance of Class 1, 2 or 3 cross-sections</td><td>1.10</td><td>1.05</td></tr>
<tr><td>γ_{M1}</td><td colspan="2">Resistance of Class 4 cross-sections</td><td>1.10</td><td>1.05</td></tr>
<tr><td>γ_{M1}</td><td colspan="2">Resistance of a member to buckling</td><td>1.10</td><td>1.05</td></tr>
<tr><td>γ_{M2}</td><td colspan="2">Resistance of net section at bolt holes</td><td>1.25</td><td>1.20</td></tr>
<tr><td rowspan="4">6.1.1</td><td rowspan="4">Partial safety factors for connections</td><td>γ_{Mb}</td><td colspan="2">Bolts</td><td>1.25</td><td>1.35</td></tr>
<tr><td>γ_{Mr}</td><td colspan="2">Rivets</td><td>1.25</td><td>1.35</td></tr>
<tr><td>γ_{Mp}</td><td colspan="2">Pins</td><td>1.25</td><td>1.35</td></tr>
<tr><td>γ_{Mw}</td><td colspan="2">Welds</td><td>1.25</td><td>1.35</td></tr>
<tr><td rowspan="3">6.5.8.1</td><td rowspan="3">Partial safety factors for slip resistance</td><td>$\gamma_{Ms.ult}$</td><td colspan="2">Ultimate limit state</td><td>1.25</td><td>1.20</td></tr>
<tr><td>$\gamma_{Ms.ser}$</td><td colspan="2">Serviceability limit state</td><td>1.10</td><td>1.35</td></tr>
<tr><td>$\gamma_{Ms.ult}$</td><td colspan="2">Ultimate limit state with oversize or slotted holes</td><td>1.40</td><td>1.35</td></tr>
<tr><td>9.3.2</td><td>Partial safety factors for fatigue loading</td><td>γ_{Ff}</td><td colspan="2">Fatigue loading</td><td>1.00</td><td>1.00</td></tr>
<tr><td>9.3.4</td><td>Partial safety factors for fatigue strength</td><td>γ_{Mf}</td><td colspan="2">Fatigue strength</td><td>—</td><td>See Table 2</td></tr>
<tr><td>C.2.5</td><td>γ factors for brittle fracture</td><td>γ_{C1}</td><td colspan="2">C1</td><td>1.00</td><td>1.00</td></tr>
<tr><td rowspan="3">C.2.5</td><td rowspan="3">γ factor for brittle fracture</td><td rowspan="3">γ_{C2}</td><td rowspan="3">C2</td><td>Fe 430 or Fe E 275</td><td>1.50</td><td>1.20</td></tr>
<tr><td>Fe 510 or Fe E 355</td><td>1.50</td><td>1.10</td></tr>
<tr><td>All other grades</td><td>1.50</td><td>1.50</td></tr>
<tr><td>K.1</td><td>Partial safety factor for joint resistance</td><td>γ_{Mj}</td><td colspan="2">Hollow section lattice girder connections</td><td>1.10</td><td>1.05</td></tr>
</table>

Table 2 — Partial safety factors for fatigue strength

Inspection and access	"Fail-safe" components	Non-"fail-safe" components
Periodic inspection[a] and maintenance Accessible joint detail	1.0	1.0
Periodic inspection[a] and maintenance Poor accessibility	1.0	1.0

[a] See **9.3.1**(2) of EC3-1.1 concerning inspection.

Table 3 — Combination factors (ψ factors)

Variable action[a]		ψ_0	ψ_1	ψ_2
Imposed floor loads	Dwellings	0.5	0.4	0.2
	Office and stores	0.7	0.6	0.3
	Parking	0.7	0.7	0.6
Wind loads		0.7	0.2	0
Imposed roof loads[b]		0.7	0.2	0
Crane loads[c]	Vertical	0.7	0.6	0.3
	Horizontal			
	0.9 (vertical + horizontal)			

[a] For the purpose of EC3-1.1 these four categories of variable actions should be treated as separate and independent variable actions.
[b] Local drifting of snow on roofs should be treated as an accidental action [see **6.1.1** c)].
[c] The most onerous of the three specified alternatives should be treated as a single variable action.

Table 4 — Combination factors for accidental loads

Variable action		ψ_1 or ψ_2 for use in A.3 and A.4
Imposed floor loads	Dwellings	0.35[a]
	Offices	0.35[a]
	Stores	1.0
	Parking	0.35[a]
Wind loads[b]		0.35
Imposed roof loads		0.35
Crane loads[c]	Vertical	1.00
	Horizontal	0.00

[a] Where the variable action is of a persistent or quasi-permanent nature, the ψ factor should be taken as 1.0.
[b] The full value obtained from CP 3:Chapter V-2:1972 should be multiplied by 0.35.
[c] The values given in this table assume that the crane is stationary. The vertical load to which the combination factor is applied is the static load value.

4 Loading codes

The loading codes to be used are:

BS 648:1964, *Schedule of weights of building materials.*

BS 6399, *Loading for buildings.*

BS 6399-1:1984, *Code of practice for dead and imposed loads.*

BS 6399-3:1988, *Code of practice for imposed roof loads.*

CP 3, *Code of basic data for the design of buildings.*

CP 3:Chapter V, *Loading.*

CP 3:Chapter V-2:1972, *Wind loads.*

In using these documents with EC 3-1.1 the following modifications should be noted.

a) The imposed floor loads of a building should be treated as one variable action to which the reduction factors given in BS 6399-1:1984 are applicable.

b) The wind loading should be taken as 90 % of the value obtained from CP 3:Chapter V-2:1972.

5 Reference standards

The supporting standards to be used, including materials specifications and standards for construction, are listed in Table 5 to Table 14.

Table 5 — Reference standard 1. Weldable structural steel

Topic	EC3-1.1 calls up	UK supporting standard
Hot rolled	EN 10025	BS EN 10025 and BS 4360
	prEN 10113	BS EN 10113 and BS 4360
	prEN 10210-1	BS 4360
Cold formed	prEN 10219-1	BS 6363

Table 6 — Reference standard 2. Dimensions of sections and plates

Topic	EC3-1.1 calls up	UK supporting standard
Hot rolled sections excluding structural hollow sections	EN 10025	BS EN 10025
	EN . . . . . [**B.2.2.1**(2)]	BS 4
	EN . . . . . [**B.2.2.1**(3)]	BS 4
	EN . . . . . [**B.2.2.1**(4)]	BS 4848-5
	EN . . . . . [**B.2.2.1**(5)]	BS 4
	EN . . . . . [**B.2.2.1**(6)]	BS 4
	EN . . . . . [**B.2.2.1**(7)]	BS 4848-4
	ISO 657-1 and ISO 657-2	ISO 657-1 and ISO 657-2
	EN . . . . . [**B.2.2.1**(9)]	BS 4360
	EN . . . . . [**B.2.2.1**(10)]	BS 4360
	EN . . . . . [**B.2.2.1**(11)]	BS 4360
Hot rolled structural hollow sections	prEN 10210-2	BS 4848-2
	ISO 657-14	ISO 657-14
Cold finished structural hollow sections	prEN 10219-2-2	BS 6363
	ISO 4019	ISO 4019

Table 7 — Reference standard 2. Dimensions of sections and plates: tolerances

Topic	EC3-1.1 calls up	UK supporting standard
Hot rolled sections excluding structural hollow sections	prEN 10034	BS 4
	prEN 10056	BS 4848-4
	EN . . . . . [**B.2.3.1**(3)]	BS 4
	EN . . . . . [**B.2.3.1**(4)]	BS 4
	EN . . . . . [**B.2.3.1**(5)]	BS 4848-5
	EN . . . . . [**B.2.3.1**(6)]	BS 4
	EN . . . . . [**B.2.3.1**(7)]	BS 4360
	EN . . . . . [**B.2.3.1**(8)]	BS 4360
Structural hollow sections	prEN 10210-2	BS 4848-2
	prEN 10219-2	BS 6363
Plates and flats	EN 10029	BS EN 10029
	EN . . . . . [**B.2.3.4**(2)]	BS 4360
	EN . . . . . [**B.2.3.4**(3)]	BS 4360

Table 8 — Reference standard 3. Bolts, nuts and washers: non-pre-loaded bolts

Topic	EC3-1.1 calls up	UK supporting standards
Bolts	EN 24014	BS EN 24014, BS 3692, BS 4190, BS 4933
	EN 24016	BS EN 24016, BS 3692, BS 4190, BS 4933
	EN 24017	BS EN 24017, BS 3692, BS 4190, BS 4933
	EN 24018	BS EN 24018, BS 3692, BS 4190, BS 4933
	ISO 7411	BS 4395
	ISO 7412	BS 4395
Nuts	EN 24032	BS EN 24032, BS 3692, BS 4190
	EN 24034	BS EN 24034, BS 3692, BS 4190
	ISO 7413	BS 3692, BS 4190
	ISO 4775	BS 4395
	ISO 7414	BS 4395
Washers	ISO 7089	ISO 7089 BS 4320
	ISO 7090	ISO 7090 BS 4320
	ISO 7091	ISO 7091 BS 4320
	ISO 7415	ISO 7415
	ISO 7416	ISO 7416

Table 9 — Reference standard 3. Bolts, nuts and washers: pre-loaded bolts

Topic	EC3-1.1 calls up	UK supporting standard
Bolts	ISO 7411	BS 4395-1 and BS 4395-2
Nuts	ISO 4775	BS 4395-1 and BS 4395-2
Washers	ISO 7415	BS 4395-1 and BS 4395-2
	ISO 7416	BS 4395-1 and BS 4395-2

Table 10 — Reference standard 4. Welding consumables

EC3-1.1 calls up	UK supporting standards
EN . . . . . [**B.2.5**(1)]	BS 639, BS 2901, BS 2926, BS 4105, BS 4165 and BS 7084

Table 11 — Reference standard 5. Rivets

EC3-1.1 calls up	UK supporting standard
EN . . . . . [**B.2.6**(1)]	BS 4620

Table 12 — Reference standards 6 to 9. Execution standards

EC3-1.1 calls up	UK supporting standard
EN . . . . . [**B.2.7**(1)]	BS 5950-2, BS 4604-1 and BS 4604-2, BS 5135, BS 5531

Table 13 — Reference standard 10. Corrosion protection

EC3-1.1 calls up	UK supporting standard
EN . . . . . [**B.2.8**(1)]	BS 5493

Table 14 — Directly referenced supporting standards

EC3-1.1 calls up	UK supporting standards
ISO 8930	ISO 8930
ISO 6707-1	ISO 6707-1
prEN 10025	BS EN 10025
prEN 10113	BS EN 10113
EN . . . . . [**6.6.1**(2)]	BS 5135

6 Additional recommendations

6.1 Guidance on EC3-1.1

NOTE **6.1.1** to **6.1.6** should be followed when designing in accordance with EC3-1.1.

6.1.1 *Chapter 2. Basis of design*

a) *Clause **2.1**(2)*

Structural integrity. Design rules to provide structural integrity by limiting the effects of accidental damage are given in Annex A.

b) *Clause **2.2.2.3***

Temperature. Where, in the design of a structure, it is necessary to take account of changes in temperature it may be assumed that in the UK the average temperature of internal steelwork varies from – 5 °C to + 35 °C. The actual range, however, depends on the location, type and purpose of the structure and special consideration may be necessary for structures in other conditions.

c) *Clause **2.3.2.2***

Accidental design situation. When designing for the accidental situation in Table 2.1 of EC3-1.1 the values of ψ_1, ψ_2 and A_k should be determined from Annex A.

NOTE The values of ψ_1 and ψ_2 are also given in Table 4.

The accidental load A_k (34 kN/m^2, see **A.4**), should be multiplied by a γ_A factor of 1.05.

The γ_{GA} factor should be taken as 1.05, except where the dead load is considered as consisting of unfavourable and favourable parts, in which case the favourable part should be multiplied by a γ_{GA} factor of 0.9 and the unfavourable part should be multiplied by a γ_{GA} factor of 1.05.

d) *Clause **2.5***

Fire resistance. Pending the issue of ENV 1993-1-2 (Eurocode 3-1.2), BS 5950-8:1990 should be used.

6.1.2 ***Chapter 3. Materials***

a) *Clause* ***3.2***

Grade A steels are not covered in EC3-1.1. They are not included in the harmonized text of EN 10025 and appear only in Annex D of BS EN 10025:1990.

Pending the superseding of grade A in UK practice by untested grade B, grade A may be used up to the maximum thickness given in Table 15 for the conditions and temperatures given in Table 15. However, if the conditions differ such that reference to Annex C of EC3-1.1 is necessary, then grade A steels should not be used.

The recommendations of this clause do not apply to grade Fe 430A base plates subject to compression only. Grade Fe 430A base plates transmitting moments to the foundation should not exceed the thickness limits for grade Fe 430A in Table 15.

b) *Clause* ***3.2.2.3***

Maximum thickness. The maximum thickness should not exceed the value given in Table 15. Where the steel is subjected to temperatures other than those given or where the steel grade or thickness used is not covered by Table 15 then Annex C of EC3-1.1 may be used with a γ_C factor for condition C2 of 1.2 for Fe 430 and Fe E 275 steel, 1.1 for Fe 510 and Fe E 355 steel and 1.5 for all other grades.

Crane girder loads. For crane girders under normal use, the loading rate to be used in calculations for brittle fracture should be taken as R1 (see **C.2.2** of EC3-1.1).

6.1.3 ***Chapter 5. Ultimate limit state***

a) *Table 5.2.1*

In continuous framing, with elastic global analysis, rigid connections need not be full-strength. Similarly in continuous framing with rigid-plastic global analysis, full-strength connections need not be rigid (but see also **6.4.3.2**(3) of EC3-1.1).

In rigid-plastic global analysis, where full-strength connections are not needed to resist the internal forces and moments, partial-strength connections may be introduced provided they are remote from plastic hinge locations.

b) *Clause* ***5.2.3.4***

Columns in simple framing. Pending the issue of Annex H of EC3-1.1 interim design rules for columns in simple framing are given in Annex B of this NAD.

c) *Clause* ***5.4.8***

As an alternative to the formulae in **5.4.8** of EC3-1.1, the theoretical reduced plastic resistance moment of a cross section in the presence of axial force may be used.

NOTE Formulae for such values are given in some section property tables commercially available from steel producers and suppliers.

Table 15 — Maximum thickness for statically loaded structural elements

Dimensions in millimetres

Steel grade and quality	Maximum thickness for lowest service temperature of			
	– 5 °C: Internal		– 15 °C: External	
	S1[a]	S2[a]	S1[a]	S2[a]
BS EN 10025[b]				
Fe 430 A	50	25	30	15
Fe 430 B	120	32	81	23
Fe 430 C	250	82	235	57
Fe 430 D	250	250	250	150
Fe 510 A	40	20	25	12
Fe 510 B	60	20	43	13
Fe 510 C	150	43	115	31
Fe 510 D	250	117	250	79
Fe 510 DD[d]	250	168	250	142
BS EN 10113[d]				
Fe E 275 KG[e]	250	250	250	250
Fe E 275 KT	250	250	250	250
Fe E 355 KG[e]	250	168	250	142
Fe E 355 KT	250	250	250	250

[a] Service conditions.
S1: either
— non-welded, or
— in compression.
S2: as welded, in tension.
In both cases this table assumes loading rate R1 and consequences of failure condition C2 defined in Annex C of EC3-1.1.
For full details of service conditions, refer to Annex C of EC3-1.1.

[b] For rolled sections over 100 mm thick, the minimum Charpy V-notch energy specified in BS EN 10025 is subject to agreement. For thicknesses up to 150 mm, a minimum value of 27 J at the relevant specified test temperature is necessary; a minimum value of 23 J at the relevant specified test temperature is necessary for thicknesses over 150 mm up to 250 mm.

[c] For steel grade Fe 510 DD conforming to BS EN 10025, the specified minimum Charpy V-notch energy value is 40 J at – 20 °C. The entries in this row assume an equivalent value of 27 J at – 30 °C.

[d] For steels of delivery condition N conforming to BS EN 10113-2 over 150 mm thick and for steels of delivery condition TM conforming to BS EN 10113-3 over 150 mm thick for long products and over 63 mm thick for flat products, the minimum Charpy V-notch energy specified in BS EN 10113-1 is subject to agreement. For thicknesses up to 150 mm, a minimum value of 27 J is necessary and a minimum value of 23 J is necessary for thicknesses over 150 mm up to 250 mm. The test temperature should be – 30 °C for KG quality steel and – 50 °C for KT quality steel.

[e] For steel of quality KG conforming to BS EN 10113-1, the specified minimum values of Charpy V-notch energy go down to 40 J at – 20 °C. The entries in this row assume an equivalent value of 27 J at – 30 °C.

d) *Clause* ***5.5.1***

Maximum slenderness. The value of λ should not exceed the following:

1) for members resisting loads other than wind loads — 180;
2) for members resisting self weight and wind loads only — 250;
3) for any member normally acting as a tie but subject to reversal of stress resulting from the action of wind — 350.

A member with slenderness greater than 180 should be checked for self weight deflection using the method in **4.7.3.2** of BS 5950-1:1990.

Buckling length. Where no guidance is given in EC3-1.1, the nominal effective lengths for a strut given in **4.7.2** of BS 5950-1:1990 should be used.

e) *Clause* ***5.5.2***

Effective length factor.

1) When calculating the elastic critical moment a value of k (see Annex F of EC3-1.1) less than 0.7 may be used for a member only where it can be demonstrated that the stiffness of the connecting members and of the connections to be used would justify such a value. In all other cases the value of k should not be taken as less than 0.7.

2) For normal loading conditions where no guidance is given in EC3-1.1, the recommendations in **4.3.5** of BS 5950-1:1990 for the effective length of beams and cantilevers with normal loading conditions may be used to determine the value of k. The effective length, L_E, referred to in BS 5950-1:1990 is equivalent to the kL term used in Annex F of EC3-1.1. For destabilizing loads see *Load position below.*

Load position. For loads above or below the shear centre, the effective length factors in 1) and 2) above should be used, in association with the appropriate value of z_g.

Buckling resistance moment for single angles. The buckling resistance moment for a single angle should be taken from **4.3.8** of BS 5950-1:1990.

f) *Clause* ***5.5.4***

Appendix G of BS 5950-1:1990 should be used for the design of restrained members with an unrestrained compression flange.

g) *Clause* ***5.7.6***

Design of diagonal, tension and torsional stiffeners. **4.5.6, 4.5.7** and **4.5.8** of BS 5950-1:1990 should be used for the design of diagonal, tension and torsional stiffeners respectively. Bearing stiffeners should be designed in accordance with EC3-1.1.

6.1.4 *Chapter 6. Connections subject to static loading*

a) *Clause* ***6.4.3.2***

When allowing for overstrength effects by checking whether the design resistance of the full-strength connection is at least 1.2 times the design plastic resistance of the members, the value γ_{Mb} for bolts in tension should be taken as 1.2.

The rotation capacity of a connection adjacent to a haunch need not be checked provided that the connection is capable of resisting the maximum moments and forces that would result if one or more of the plastic hinges located in the members are overstrength, due to the relevant members having an actual yield strength 1.2 times the specified value.

The rotation capacity need not be checked in a full-strength connection immediately adjacent to the last hinge to form, provided that this can be clearly identified.

b) *Clause* ***6.5.5***

Bearing resistance. The values for bearing resistance given in Table 6.5.3 of EC3-1.1 may result in larger deformations in joints than those normally accepted in the UK. Unless such deformation is acceptable, the bearing stresses on the parent material should be limited to $0.85(f_u + f_y)/\gamma_{Mb}$.

c) *Clause* ***6.5.8.1(3)***

Load combination. The load combination for the serviceability limit state should be taken as the rare combination defined in **2.3.4**(2) of EC3-1.1.

d) *Clause* ***6.5.8.2***

Pre-loading force. For high strength bolts conforming to BS 4395-1:1969 and BS 4395-2:1969, with controlled tightening in conformity with BS 4604-1:1970 and BS 4604-2:1970, the design pre-loading force, $F_{p.Cd}$, to be used in design calculations should be that given in BS 4604-1:1970 and BS 4604-2:1970.

e) *Clause* ***6.5.8.4***

Fasteners conforming to BS 4395-2:1969 should not be subjected to externally applied tension.

f) *Clause* ***6.6.4(7)***

Weld ductility. The welds should be designed for the full design resistance of the weakest element, not 80 % of the design resistance.

g) *Clause* ***6.6.5.2***

Throat size. The throat thickness should not be taken as more than 0.7 times the leg length (see Figure 6.6.6 of EC3-1.1).

h) *Clause* ***6.6.8(5)***

Connecting welds. **6.6.8**(5) in EC3-1.1 assumes that the axial force, N_{Sd}, in the plate is equal to its resistance, based on its effective breadth, b_{eff}. In practice where the axial force is less than this resistance the welds should have a design resistance per unit length equal to N_{Sd}/b_{eff}, provided that the same size of weld extends across the full width of the plate.

6.1.5 *Chapter 9. Fatigue*

a) *Clause* ***9.1.2***

General. For crane supporting structures reference should be made to BS 2573-1:1983, BS 466:1984, BS 2573-2:1980 and the crane manufacturer's publications for loading and frequency details.

6.1.6 *Annex L. Column bases*

a) *Clause* ***L.1***

Thickness. The thickness of the base plate should not be less than the thickness of the column flange which it supports.

Bearing strength. When calculating the bearing strength, f_j, of the joint, the γ_c factor should be taken as 1.5.

6.2 Recommendations on subjects not covered in EC3-1.1

6.2.1 *Design of purlins and slide rails*

As an alternative to the general rules in EC3-1.1 purlins and side rails may be designed using the empirical rules given in BS 5950-1:1990.

6.2.2 *Web openings*

Pending the issue of Annex N of EC3 the design of beams with web openings, other than those required for fasteners, should be in accordance with **4.15** of BS 5950-1:1990.

6.2.3 *Cased columns*

Cased columns and beams may be designed using the rules given in **4.14** of BS 5950-1:1990.

6.2.4 *Eccentrically connected T-sections and channels*

a) *General.* All eccentrically connected members should be designed in accordance with the principles given in **6.5.2.3**(1) and **6.6.10**(1) of EC3-1.1. The following application rules satisfy these principles for eccentrically connected T-sections and channel sections.

b) *Tension resistance.* The tension resistance of a member may be determined in accordance with **5.4.3** of EC3-1.1 provided that the effective net area of the cross section, A_{net}, is determined from the following recommendations.

For single T-sections connected only through the flange and channel sections connected through the web the effective net area, A_{net}, should be taken as the effective net area of the connected element plus half the area of the outstanding elements.

c) *Buckling resistance.* The member buckling resistance may be determined in accordance with **5.5.1** of EC3-1.1 provided that the slenderness, λ, is determined from the following recommendations.

1) Single channels: for a single channel connected only by its web, the connection should be by two or more rows of symmetrically placed fasteners or an equivalent weld and the slenderness for buckling about the minor axis should be determined from **4.7.10.4** of BS 5950-1:1990.

2) Single T-sections: for a single T-section connected only by its flange the connection should be by two or more rows of symmetrically placed fasteners or an equivalent weld and the slenderness for buckling about the axis parallel to the flange should be determined from **4.7.10.5** of BS 5950-1:1990.

Annex A (normative) General recommendations for structural integrity

A.1 Introduction

All structures should be designed using the principles given in **2.1** of EC3-1.1. This annex gives application rules which satisfy the principle of structural integrity given in **2.1**(2) of EC3-1.1. These application rules apply to buildings.

For the purposes of this provision, it may be assumed that substantial permanent deformation of members and their connections is acceptable.

A.2 Tying forces

A.2.1 *Recommendations for all buildings*

Every building frame should be effectively tied together at each principal floor and roof level. All columns should be effectively restrained in two directions approximately at right angles at each principal floor or roof which they support.

This anchorage may be provided by either beams or tie members. Where possible these should be arranged in continuous lines as close as practicable to the columns and to each edge. At re-entrant corners the peripheral tie should be anchored into the steel framework.

Ties may be either steel members or steel reinforcement embedded in concrete or masonry provided that they are properly anchored to the steel framework.

Steel members provided for other purposes may be utilized as ties. When they are checked as ties other loading may be ignored. Beams designed to carry the floor or roof loading will generally be suitable provided that their end connections are capable of resisting tension.

All ties and their end connections should be of a standard of robustness commensurate with the structure of which they form a part and should have a design tension resistance of not less than 75 kN at floors or 40 kN at roof level.

Ties are not required at a roof level where steelwork supports cladding weighing not more than 0.7 kN/m^2 and carries roof loads only.

Where a building is provided with expansion joints, each section between expansion joints should be treated as a separate building for the purpose of this clause.

A.2.2 *Additional recommendations for tall multi-storey buildings*

Local or national regulations may stipulate that tall multi-storey buildings be designed to localize accidental damage.

Steel-framed buildings which satisfy the recommendations of **A.2.1** may be assumed to conform to this requirement provided that the five additional conditions given below are met.

A tall multi-storey building which is required to be designed to localize accidental damage but which does not satisfy these five additional conditions should be checked as recommended in **A.3**.

a) *Bracing*. The bracing or shear walls should be so distributed throughout the building that no substantial portion of the structural framework is solely reliant on a single plane of bracing in each direction.

b) *Tying*. The ties described in **A.2.1** should be arranged in continuous lines wherever practicable throughout each floor and roof level in two directions approximately at right angles. These and their connections should be checked for the following design tensile forces, which need not be considered as additive to other forces.

1) Generally: $0.5 w_f s_t L_a$ for any internal ties and $0.25 w_f s_t L_a$ for edge ties but not less than 75 kN for floors or 40 kN at roof level

where

w_f is the total factored dead and imposed load per unit area of floor or roof;

s_t is the mean transverse spacing of the ties;

L_a is the greatest distance in the direction of the tie under consideration between the centres of adjacent lines of supporting columns, frames or walls.

2) At the periphery: ties anchoring columns at the periphery of a floor or roof should be checked for the greater of:

— the force given in item b) 1) and

— 1 % of the design vertical load in the column at that level.

c) *Columns.* All column splices should be capable of resisting a design tensile force of not less than two-thirds of the design vertical load applied to the column from the floor level next below the splice.

Where the steel framework is not of continuous construction in at least one direction, the columns should be carried through at each beam-to-column connection.

d) *Integrity.* Any beam which carries a column should be checked, together with the members which support it, for localization of damage as recommended in **A.3**.

e) *Floor units.* Where precast concrete or other heavy floor or roof units are used they should be effectively anchored in the direction of their span either to each other over a support or directly to their supports as recommended in BS 8110-1:1985 and BS 8110-2:1985.

A.3 Localization of damage

At the accidental limit state, where recommended in **A.2**, the effect of the removal of any single column or beam carrying a column should be assessed for each storey of a building in turn. Where the removal of one of these members would result in collapse of any area greater than 70 m^2 or 15 % of the area of the storey, that member should be designed as a key element as recommended in **A.4**.

In this check the appropriate value of ψ of the ordinary wind load and of the ordinary imposed load should be considered together with the dead load, except that in the case of buildings used predominantly for storage, or where the imposed load is of a persistent nature, the full imposed load should be used. The combination factors, ψ_1 and ψ_2, for accidental loads are given in Table 4. The γ_{GA} factor should be taken as 1.05 except where the dead load is considered as consisting of unfavourable and favourable parts, in which case the favourable part should be multiplied by a γ_{GA} factor of 0.9 and the unfavourable part should be multiplied by a γ_{GA} factor of 1.05.

A.4 Design of key elements

Key elements or members are single structural elements which support a floor or roof area of more than 70 m^2 or 15 % of the area of the storey.

Any other steel member or other structural component which provides lateral restraint vital to the stability of a key element should itself also be designed as a key element for the same accidental loading.

Where it is recommended in **A.3** that a member be designed as a key element, the accidental load, A_k, should be chosen having particular regard to the importance of the key element and the consequences of failure and should not be less than 34 kN/m^2. The accidental load, A_k, should be multiplied by a γ_A factor of 1.05.

Accidental loads should be applied to members from appropriate directions together with the reactions from other building components attached to the member which are subject to the same loading but limited to the ultimate strength of these components or their connections.

In designing for the accidental situation the member should be designed for the accidental load in combination with the dead and imposed loads [see **2.3.2.2**(2) of EC3-1.1]. The combination factors for use with loads are given in Table 4.

Annex B (normative)
Application rules for columns in simple framing

B.1 General

The application rules in **B.2** to **B.5** apply to columns in structures of simple framing and are intended as application rules for use within the UK.

B.2 Pattern loading

Pattern loading need not normally be considered in simple framing. However, unbalanced loading due to variations in span or actual loading should be taken into account.

B.3 Buckling length of column

Provided that the nominal moments obtained as described in **B.5** are the only applied moments the geometrical slenderness ratio of the column, λ_{LT}, should be determined from Annex F of EC3-1.1 with the C_1 factor taken as 1.0.

B.4 Eccentricities

The eccentricity of the beam end reactions or other loads should be as follows.

a) For a beam supported on a cap plate, the load should be taken as acting at the face of the column, or edge of packing if used, towards the span of the beam.

b) For a roof truss supported on a cap plate, eccentricity may be neglected provided simple connections are used which do not develop significant moments adversely affecting the structure.

c) In all other cases the load should be taken as acting at a distance from the face of the steel column towards the span of the beam equal to 100 mm, or at the centre of the length of stiff bearing, whichever gives the greater eccentricity.

B.5 Unbalanced loading

Where columns are subject to unbalanced loading, they should be designed for the resulting moments. In multi-storey buildings where the columns are effectively continuous at each floor level, the net moment at one level should be divided between the column lengths above and below that level in proportion to the stiffness coefficient, (I/L), of each length.

The moments due to the eccentricities given in **B.4** should be assumed to have no effect at the levels above and below the level at which they are applied.

List of references (see clause 2)

Normative references

BSI standards publications

BRITISH STANDARDS INSTITUTION, London

BS 466:1984, *Specification for power driven overhead travelling cranes, semi-goliath and goliath cranes for general use.*

BS 648:1964, *Schedule of weights of building materials.*

BS 2573, *Rules for the design of cranes.*

BS 2573-1:1983, *Specification for classification, stress calculations and design criteria for structures.*

BS 2573-2:1980, *Specification for classification, stress calculations and design of mechanisms.*

BS 4395, *Specification for high strength friction grip bolts and associated nuts and washers for structural engineering.*

BS 4395-1:1969, *General grade.*

BS 4395-2:1969, *Higher grade bolts and nuts and general grade washers.*

BS 4604, *Specification for the use of high strength friction grip bolts in structural steelwork. Metric series.*

BS 4604-1:1970, *General grade.*

BS 4604-2:1970, *Higher grade (parallel shank).*

BS 5950, *Structural use of steelwork in building.*

BS 5950-1:1990, *Code of practice for design in simple and continuous construction: hot rolled sections.*

BS 5950-8:1990, *Code of practice for fire resistant design.*

BS 6399, *Loading for buildings.*

BS 6399-1:1984, *Code of practice for dead and imposed loads.*

BS 6399-3:1988, *Code of practice for imposed roof loads.*

BS 8110, *Structural use of concrete.*

BS 8110-1:1985, *Code of practice for design and construction.*

BS 8110-2:1985, *Code of practice for special circumstances.*

CP 3, *Code of basic data for the design of buildings.*

CP 3:Chapter V, *Loading.*

CP 3:Chapter V-2:1972, *Wind loads.*

Informative references

BSI standards publications

BRITISH STANDARDS INSTITUTION, London

BS 4, *Structural steel sections.*

BS 4-1:1980, *Specification for hot-rolled sections.*

BS 639:1986, *Specification for covered carbon and carbon manganese steel electrodes for manual metal-arc welding.*

BS 2901, *Filler rods and wires for gas-shielded arc welding.*

BS 2901-1:1983, *Ferritic steels.*

BS 2901-2:1990, *Specification for stainless steels.*

BS 2901-3:1990, *Specification for copper and copper alloys.*

BS 2901-4:1990, *Specification for aluminium and aluminium alloys and magnesium alloys.*

BS 2901-5:1990, *Specification for nickel and nickel alloys.*

BS 2926:1984, *Specification for chromium and chromium-nickel steel electrodes for manual metal-arc welding.*

BS 3692:1967, *Specification for ISO metric precision hexagon bolts, screws and nuts. Metric units.*

BS 4105:1990, *Specification for liquid carbon dioxide, industrial.*

BS 4165:1984, *Specification for electrode wires and fluxes for the submerged arc welding of carbon steel and medium-tensile steel.*

BS 4190:1967, *Specification for ISO metric black hexagon bolts, screws and nuts.*

BS 4320:1968, *Specification for metal washers for general engineering purposes. Metric series.*

BS 4360:1990, *Specification for weldable structural steels.*

BS 4620:1970, *Specification for rivets for general engineering purposes.*

BS 4848, *Hot-rolled structural steel sections.*

BS 4848-4:1972, *Equal and unequal angles.*

BS 4848-5:1980, *Flats.*

BS 4933:1973, *Specification for ISO metric black cup and countersunk head bolts and screws with hexagon nuts.*

BS 5135:1984, *Specification for arc welding of carbon and carbon manganese steels.*

BS 5493:1977, *Code of practice for protective coating of iron and steel structures against corrosion.*

BS 5531:1988, *Code of practice for safety in erecting structural frames.*

BS 5950, *Structural use of steelwork in building.*

BS 5950-2:1992, *Specification for materials, fabrication and erection: hot-rolled sections.*

BS 5950-3, *Design in composite construction.*

BS 5950-3.1:1990, *Code of practice for design of simple and continuous composite beams.*

BS 5950-4:1982, *Code of practice for design of floors with profiled steel sheeting.*

BS 5950-5:1987, *Code of practice for design of cold formed sections.*

BS 5950-7:1992, *Specification for materials and workmanship: cold formed sections.*

BS 6363:1983, *Specification for welded cold formed steel structural hollow sections.*

BS 7084:1989, *Specification for carbon and carbon-manganese steel tubular cored welding electrodes.*

BS EN 10025:1990, *Specification for hot rolled products of non-alloy structural steels and their technical delivery conditions.*

BS EN 10029:1991, *Specification for tolerances on dimensions, shape and mass for hot rolled steel plates.*

BS EN 10113, *Hot-rolled products in weldable fine grain structural steels.*

BS EN 10113-1:1992, *General delivery conditions.*

BS EN 10113-2:1992, *Delivery conditions for normalized steels.*

BS EN 10113-3:1992, *Delivery conditions for thermomechanical rolled steels.*

BS EN 24014:1992, *Hexagon head bolts. Product grades A and B.*

BS EN 24016:1992, *Hexagon head bolts. Product grade C.*

BS EN 24017:1992, *Hexagon head screws. Product grades A and B.*

BS EN 24018:1992, *Hexagon head screws. Product grade C.*

BS EN 24032:1992, *Hexagon nuts, style 1. Product grades A and B.*

BS EN 24034:1992, *Hexagon nuts. Product grade C.*

ISO standards publications

INTERNATIONAL ORGANIZATION FOR STANDARDIZATION (ISO), GENEVA. (All publications are available from BSI Sales.)

ISO 657-1:1989, *Hot-rolled steel sections — Part 2: Equal-leg angles — Dimensions.*

ISO 657-2:1989, *Hot-rolled steel sections — Part 2: Unequal-leg angles — Dimensions.*

ISO 657-14:1982, *Hot-rolled steel sections — Part 14: Hot-finished structural hollow sections — Dimensions and sectional properties.*

ISO 4019:1982, *Cold-finished steel structural hollow sections — Dimensions and sectional properties.*

ISO 6707-1:1989, *Building and civil engineering — Vocabulary — Part 1: General terms.*

ISO 7089:1983, *Plain washers — Normal series — Product grade A.*

ISO 7091:1983, *Plain washers — Normal series — Product grade C.*

ISO 7415:1984, *Plain washers for high-strength structural bolting, hardened and tempered.*

ISO 7416:1984, *Plain washers, chamfered, hardened and tempered for high-strength structural bolting.*

ISO 8930:1987, *General principles for reliability of structures — List of equivalent terms.*

EUROPEAN PRESTANDARD

PRÉNORME EUROPÉENNE

EUROPÄISCHE VORNORM

ENV 1993-1-1

April 1992

UDC 624.92.014.2:624.07

Descriptors: Buildings, steel structures, computation, building codes, rules of calculation

English version

Eurocode 3: Design of steel structures — Part 1.1: General rules and rules for buildings

Calcul des structures en acier
Partie 1.1: Règles générales et règles pour les bâtiments

Bemessung und Konstruktion von Stahlbauten
Teil 1.1: Allgemeine Bemessungsregeln, Bemessungsregeln für den Hochbau

This European Prestandard (ENV) was approved by CEN on 1992-04-24 as a prospective standard for provisional application. The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into a European Standard (EN).

CEN members are required to announce the existence of this ENV in the same way as for an EN and to make the ENV available promptly at national level in an appropriate form. It is permissible to keep conflicting national standards in force (in parallel to the ENV) until the final decision about the possible conversion of the ENV into an EN is reached.

CEN members are the national standards bodies of Austria, Belgium, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.

CEN

European Committee for Standardization
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart 36, B-1050 Brussels

Ref. No. ENV 1993-1-1:1992 E

Foreword

0.1 Objectives of the Eurocodes

(1) The Structural Eurocodes comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.

(2) They are intended to serve as reference documents for the following purposes:

a) As a means to prove compliance of building and civil engineering works with the essential requirements of the Construction Products Directive (CPD)

b) As a framework for drawing up harmonised technical specifications for construction products.

(3) They cover execution and control only to the extent that is necessary to indicate the quality of the construction products, and the standard of the workmanship, needed to comply with the assumptions of the design rules.

(4) Until the necessary set of harmonised technical specifications for products and for methods of testing their performance is available, some of the Structural Eurocodes cover some of these aspects in informative annexes.

0.2 Background to the Eurocode Programme

(1) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonized technical rules for the design of building and civil engineering works which would initially serve as an alternative to the different rules in force in the various Member States and would ultimately replace them. These technical rules became known as the “Structural Eurocodes”.

(2) In 1990, after consulting their respective Member States, the CEC transferred the work of further development, issue and updates of the Structural Eurocodes to CEN, and the EFTA Secretariat agreed to support the CEN work.

(3) CEN Technical Committee CEN/TC 250 is responsible for all Structural Eurocodes.

0.3 Eurocode programme

(1) Work is in hand on the following Structural Eurocodes, each generally consisting of a number of parts:

EN 1991	Eurocode 1	*Basis of design and actions on structures*
EN 1992	Eurocode 2	*Design of concrete structures*
EN 1993	Eurocode 3	*Design of steel structures*
EN 1994	Eurocode 4	*Design of composite steel and concrete structures*
EN 1995	Eurocode 5	*Design of timber structures*
EN 1996	Eurocode 6	*Design of masonry structures*
EN 1997	Eurocode 7	*Geotechnical design*
EN 1998	Eurocode 8	*Design of structures for earthquake resistance*

In addition the following may be added to the programme:

EN 1999	Eurocode 9	*Design of aluminium structures*

(2) Separate sub-committees have been formed by CEN/TC250 for the various Eurocodes listed above.

(3) This part of the Structural Eurocode for Design of Steel Structures, which had been finalised and approved for publication under the direction of CEC, is being issued by CEN as a European Prestandard (ENV) with an initial life of three years.

(4) This Prestandard is intended for experimental practical application in the design of the building and civil engineering works covered by the scope as given in **1.1.2** and for the submission of comments.

(5) After approximately two years CEN members will be invited to submit formal comments to be taken into account in determining future action.

(6) Meanwhile feedback and comments on this Prestandard should be sent to the Secretariat of sub-committee CEN/TC250/SC3 at the following address:

BSI Standards
2 Park Street
London W1A 2BS
England

or to your national standards organisation.

0.4 National Application Documents

(1) In view of the responsibilities of authorities in member countries for the safety, health and other matters covered by the essential requirements of the CPD, certain safety elements in this ENV have been assigned indicative values which are identified by $\boxed{}$. The authorities in each member country are expected to assign definitive values to these safety elements.

(2) Many of the harmonized supporting standards, including the Eurocodes giving values for actions to be taken into account and measures required for fire protection, will not be available by the time this Prestandard is issued. It is therefore anticipated that a National Application Document (NAD) giving definitive values for safety elements, referencing compatible supporting standards and providing national guidance on the application of this Prestandard, will be issued by each member country or its Standards Organisation.

(3) It is intended that this Prestandard is used in conjunction with the NAD valid in the country where the building or civil engineering works are located.

0.5 Matters specific to this Prestandard

0.5.1 General

(1) The scope of Eurocode 3 is defined in **1.1.1** and the scope of this Part of Eurocode 3 is defined in **1.1.2**. Additional Parts of Eurocode 3 which are planned are indicated in **1.1.3**; these will cover additional technologies or applications, and will complement and supplement this Part.

(2) In using this Prestandard in practice, particular regard should be paid to the underlying assumptions and conditions given in **1.3**.

(3) In developing this Prestandard, background documents have been prepared, which give commentaries on, and justifications for, some of the provisions in the Prestandard.

0.5.2 Use of Annexes

(1) The nine chapters of this Prestandard are complemented by a number of Annexes, some normative and some informative.

(2) The normative annexes have the same status as the chapters to which they relate. Most have been introduced by moving some of the more detailed Application Rules, which are needed only in particular cases, out of the main part of the text to aid its clarity.

0.5.3 Concept of Reference Standards

(1) In using this Prestandard reference needs to be made to various CEN and ISO standards. These are used to define the product characteristics and processes which have been assumed to apply in formulating the design rules.

(2) This Prestandard mentions 10 "Reference Standards" which are detailed in normative Annex B. Each Reference Standard makes reference to the whole or, or part of, a number of CEN and/or ISO standards. Where any referenced CEN or ISO standard is not yet available, the National Application Document should be consulted for the standard to be used instead. It is assumed that only those grades and qualities given in normative Annex B will be used for buildings and civil engineering works designed to this Prestandard.

0.5.4 Weldable structural steel

(1) An important product standard quoted in the defined Reference Standard for weldable structural steels is EN 10025, in which grades Fe 360, Fe 430 and Fe 510 are relevant.

(2) However, EN 10025 also contains other steel grades besides these three weldable grades. It has been recognised that even for these three steel grades, which past experience has shown to be weldable, the specifications in EN 10025 are such that within the tolerance limits for the chemical analysis, steels could be supplied that might prove to be difficult to weld. Therefore in referring to EN 10025 in normative Annex B, an additional requirement has been included in **B.2.1.1**(2) concerning weldability of the steel, which should be quoted when steels to EN 10025 are ordered.

(3) The means for achieving adequate weldability has not been specified in this Prestandard. However, EN 10025 offers the definition of Carbon Equivalent Values (CEV) that can be negotiated with the steel suppliers to ensure adequate weldability.

0.5.5 Partial safety factors for resistances

(1) This Prestandard gives general rules for the design of steel structures which relate to limit states of members such as fracture in tension, failure by instability phenomena or rupture of the connections.

(2) It also gives particular rules related to the design of buildings such as rules for frames, beams, lattice girders and beam-to-column connections.

(3) Most of the rules have been calibrated against test results in order to obtain consistent values of the partial safety factors for resistance γ_M.

(4) In order to avoid a large variety of γ_M values, two categories were selected:

$\gamma_{M1} = 1,1$	to be applied to resistances related to the yield strength f_y (eg for all instability phenomena)
$\gamma_{M2} = 1,25$	to be applied to resistances related to the ultimate tensile strength f_u (eg net section strength in tension or bolt and weld resistances).

(5) However, for the particular cases of hot-rolled I beams with Class 1 cross-sections that are bent about the strong axis and are not subject to failure through instability phenomena, and of members in tension where the cross-section verification against yielding governs the design, it has been found from calibration studies using data from European steel producers, that the statistical distribution of geometrical tolerances and yield strengths would justify reducing the γ_{M1} factor from 1,1 to 1,0. In view of this finding, category γ_{M0} was introduced to allow member countries to choose either $\gamma_{M0} = 1,1$ or $\gamma_{M0} = 1,0$.

0.5.6 Fabrication and erection

(1) Chapter 7 of this Prestandard is intended to indicate some minimum standards of workmanship and normal tolerances that have been assumed in deriving the design rules given in the Prestandard.

(2) It also indicates to the designer the information relating to a particular structure that needs to be supplied in order to define the execution requirements.

(3) In addition it defines normal clearances and other practical details which the designer needs to use in calculations.

0.5.7 Design assisted by testing

(1) Chapter 8 is not required in the course of routine design, but is provided for use in the special circumstances in which it may become appropriate.

(2) Only the Principles to be followed are outlined. More detailed guidance appears in the Application Rules given in informative Annex Y.

0.5.8 Fatigue resistance

(1) Chapter 9 has been included in this Prestandard under the category of “General Rules”. Its inclusion does not imply that fatigue is likely to be a design criterion for the majority of building structures.

(2) It is anticipated that the principal role of Chapter 9 will be as general rules that can be referred to in subsequent parts of this Eurocode.

(3) However, its inclusion does also make possible the application of this Prestandard to that minority of special building structures where it is necessary to consider the effects of repeated fluctuations of stresses.

Contents

1 Introduction

1.1 Scope

1.1.1 Scope of Eurocode 3

(1) Eurocode 3 applies to the design of buildings and civil engineering works in steel. It is subdivided into various separate parts, see **1.1.2** and **1.1.3**.

(2) This Eurocode is only concerned with the requirements for resistance, serviceability and durability of structures. Other requirements, e.g. concerning thermal or sound insulation are not considered.

(3) Execution[1)] is covered to the extent that is necessary to indicate the quality of the construction materials and products which should be used and the standard of workmanship on site needed to comply with the assumptions of the design rules. Generally, the rules related to execution and workmanship are to be considered as minimum requirements which may have to be further developed for particular types of buildings or civil engineering works[1)] and methods of construction[1)].

(4) Eurocode 3 does not cover the special requirements of seismic design. Rules related to such requirements are provided in ENV 1998 Eurocode 8 "Design of structures for earthquake resistance"[2)] which complements or adapts the rules of Eurocode 3 specifically for this purpose.

(5) Numerical values of the actions on buildings and civil engineering works to be taken into account in the design are not given in Eurocode 3. They are provided in ENV 1991 Eurocode 1 "Basis of design and actions on structures"[2)] which is applicable to all types of construction[1)].

1.1.2 Scope of Part 1.1 of Eurocode 3

(1) Part 1.1 of Eurocode 3 gives a general basis for the design of buildings and civil engineering works in steel.

(2) In addition, Part 1.1 gives detailed rules which are mainly applicable to ordinary buildings. The applicability of these rules may be limited, for practical reasons or due to simplifications; their use and any limits of applicability are explained in the text where necessary.

(3) The following subjects are dealt with in this initial version of Eurocode 3-1.1:

• Chapter 1:	Introduction	
• Chapter 2:	Basis of design	
• Chapter 3:	Materials	
• Chapter 4:	Serviceability limit states	
• Chapter 5:	Ultimate limit states	
• Chapter 6:	Connections subject to static loading	
• Chapter 7:	Fabrication and erection	
• Chapter 8:	Design assisted by testing	
• Chapter 9:	Fatigue	
• Annex B:	Reference standards	(normative)
• Annex C:	Design against brittle fracture	(informative)
• Annex E:	Buckling length of a compression member	(informative)
• Annex F:	Lateral-torsional buckling	(informative)
• Annex J:	Beam-to-column connections	(normative)
• Annex K:	Hollow section lattice girder connections	(normative)
• Annex L:	Column bases	(normative)
• Annex M:	Alternative method for fillet welds	(normative)
• Annex Y:	Guidelines for loading tests	(informative)

1) For the meaning of this term, see **1.4.1**(2)

2) At present at the draft stage.

(4) Additional Annexes are already available or under preparation, for incorporation into Part 1.1 at an appropriate stage, after approval of their contents, as follows:

- Annex D: The use of steel grade Fe E 460 etc
- Annex K: Hollow section lattice girder connections — revised version including multi-planar joints.
- Annex Z: Determination of design resistance from tests

(5) Further Annexes which have been proposed for future inclusion in Part 1.1 are as follows:

- Annex G: Design for torsion resistance
- Annex H: Modelling of building structures for analysis
- Annex J: Beam-to-column connections — extended version
- Annex N: Openings in webs
- Annex S: The use of stainless steel

(6) Chapter 1 and Chapter 2 are common to all Structural Eurocodes, with the exception of some additional clauses which are specific to individual Eurocodes.

(7) This Part 1.1 does not cover:

- resistance to fire
- particular aspects of special types of buildings
- particular aspects of special types of civil engineering works (such as bridges, masts and towers or offshore platforms)
- cases where special measures may be necessary to limit the consequences of accidents.

1.1.3 Further Parts of Eurocode 3

(1) This Part 1.1 of Eurocode 3 will be supplemented by further Parts 2, 3 etc. which will complement or adapt it for particular aspects of special types of buildings and civil engineering works, special methods of construction and certain other aspects of design which are of general practical importance.

(2) Further Parts of Eurocode 3 which, at present, are being prepared or are planned include the following:

- Part 1.2 Fire resistance
- Part 1.3 Cold formed thin gauge members and sheeting
- Part 2 Bridges and plated structures
- Part 3 Towers, masts and chimneys
- Part 4 Tanks, silos and pipelines
- Part 5 Piling
- Part 6 Crane structures
- Part 7 Marine and maritime structures
- Part 8 Agricultural structures

1.2 Distinction between Principles and Application Rules

(1) Depending on the character of the individual clauses, distinction is made in this Eurocode between Principles and Application Rules.

(2) The Principles comprise:

- general statements and definitions for which there is no alternative, as well as
- requirements and analytical models for which no alternative is permitted unless specifically stated.

(3) *The Principles are printed in roman type.*

(4) The Application Rules are generally recognised rules which follow the Principles and satisfy their requirements.

(5) It is permissible to use alternative design rules different from the Application Rules given in the Eurocode, provided that it is shown that the alternative rule accords with the relevant Principles and is at least equivalent with regard to the resistance, serviceability and durability achieved by the structure.

(6) *The Application Rules are printed in italics. This is an Application Rule.*

1.3 Assumptions

(1) The following assumptions apply:

- Structures are designed by appropriately qualified and experienced personnel.
- Adequate supervision and quality control is provided in factories, in plants and on site.
- Construction is carried out by personnel having the appropriate skill and experience.
- The construction materials and products are used as specified in this Eurocode or in the relevant material or product specifications.
- The structure will be adequately maintained.
- The structure will be used in accordance with the design brief.

(2) The design procedures are valid only when the requirements for execution and workmanship given in Chapter 7 are also complied with.

(3) Numerical values identified by $\boxed{}$ are given as indications. Other values may be specified by Member States.

1.4 Definitions

1.4.1 Terms common to all Structural Eurocodes

(1) Unless otherwise stated in the following, the terminology used in International Standard ISO 8930 applies.

(2) The following terms are used in common for all Structural Eurocodes with the following meanings:

- **Construction works:** Everything that is constructed or results from construction operations[3]. This term covers both building and civil engineering works. It refers to the complete construction comprising both structural and non-structural elements.
- **Execution:** The activity of creating a building or civil engineering works. The term covers work on site; it may also signify the fabrication of components off site and their subsequent erection on site.

  NOTE In English "construction" may be used instead of "execution" in certain combinations of words where there is no ambiguity (e.g. "during construction").
- **Structure:** Organized combination of connected parts designed to provide some measure of rigidity[4]. This term refers to load carrying parts.
- **Type of building or civil engineering works:** Type of "construction works" designating its intended purpose, e.g. dwelling house, industrial building, road bridge.

  NOTE "Type of construction works" is not used in English.
- **Form of structure:** Structural type designating the arrangement of structural elements, e.g. beam, triangulated structure, arch, suspension bridge.
- **Construction material:** A material used in construction work, e.g. concrete, steel, timber, masonry.
- **Type of construction:** Indication of principal structural material, e.g. reinforced concrete construction, steel construction, timber construction, masonry construction.
- **Method of construction:** Manner in which the construction will be carried out, e.g. cast in place, prefabricated, cantilevered.
- **Structural system:** The load bearing elements of a building or civil engineering works and the way in which these elements are assumed to function, for the purpose of modelling.

(3) The equivalent terms in various languages are given in Table 1.1.

[3] This definition accords with International Standard ISO 6707-1.

[4] International Standard ISO 6707-1 gives the same definition but adds "or a construction works having such an arrangement". In the Structural Eurocodes this addition is not used, in order to facilitate unambiguous translation.

1.4.2 Special terms used in this Part 1.1 of Eurocode 3

(1) The following terms are used in Part 1.1 of Eurocode 3 with the following meanings:

- **Frame:** Portion of a structure, comprising an assembly of directly connected structural elements, designed to act together to resist load. This term refers to both rigid-jointed frames and triangulated frames. It covers both plane frames and three-dimensional frames.
- **Sub-frame:** A frame which forms part of a larger frame, but is treated as an isolated frame in a structural analysis.
- **Type of framing:** Terms used to distinguish between frames which are either:
  - **Semi-continuous,** in which the structural properties of the connections need explicit consideration in the global analysis.
  - **Continuous,** in which only the structural properties of the members need be considered in the global analysis.
  - **Simple,** in which the joints are not required to resist moments.
- **Global analysis:** The determination of a consistent set of internal forces and moments in a structure, which are in equilibrium with a particular set of actions on the structure.
- **System length:** Distance between two adjacent points at which a member is braced against lateral displacement in a given plane, or between one such point and the end of the member.
- **Buckling length:** System length of an otherwise similar member with pinned ends, which has the same buckling resistance as a given member.
- **Designer:** Appropriately qualified and experienced person responsible for the structural design.

1.5 S.I. units

(1) S.I. units shall be used in accordance with ISO 1000.

(2) For calculations, the following units are recommended:

• forces and loads	:	kN, kN/m, kN/m^2
• unit mass	:	kg/m^3
• unit weight	:	kN/m^3
• stresses and strengths	:	N/mm^2 (= MN/m^2 or MPa)
• moments (bending ....)	:	kNm.

Table 1.1 — List of equivalent terms in various languages

English	Français	Deutsch	Italiano	Nederlands	Español
Construction works	Construction	Bauwerk	Costruzione	Bouwwerk	Construcción
Execution	Exécution	(Bau-)Ausführung	Esecuzione	Uitvoering	Ejecución
Structure	Structure	Tragwerk	Struttura	Draagconstructie	Estructura
Type of building or civil engineering works	Nature de construction	Art des Bauwerks	Tipo di costruzione	Type bouwwerk	Natureleza de la construcción
Form of structure	Type de structure	Art des Tragwerks	Tipo di struttura	Type draagconstructie	Tipo de estructura
Construction material	Matériau de construction	Baustoff; Werkstoff*) (*nur im Stahlbau)	Materiale da costruzione	Constructie materiaal	Material de construcción
Type of construction	Mode de construction	Bauweise	Sistema costruttivo	Bouwwijze	Modo de construcción
Method of construction	Procédé d’ exécution	Bauverfahren	Procedimento esecutivo	Bouwmethode	Procedimiento de ejecución
Structural system	Système structural	Tragsystem	Sistema strutturale	Constructief systeem	Sistema estructural

1.6 Symbols used in Part 1.1 of Eurocode 3

1.6.1 Latin upper case letters

A	Accidental action
A	Area
B	Bolt force
C	Capacity; Fixed value; Factor
D	Damage (fatigue assessment)
E	Modulus of elasticity
E	Effect of actions
F	Action
F	Force
G	Permanent action
G	Shear modulus
H	Total horizontal load or reaction
I	Second moment of area
K	Stiffness factor (l/L)
L	Length; Span; System length
M	Moment in general
M	Bending moment
N	Axial force
Q	Variable action
R	Resistance; Reaction
S	Internal forces and moments (with subscripts d or k)
S	Stiffness (shear, rotational ... stiffness with subscripts v, j ...)
T	Torsional moment; Temperature
V	Shear force; Total vertical load or reaction
W	Section modulus
X	Value of a property of a material

1.6.2 Greek upper case letters

Δ	Difference in ... (precedes main symbol)

1.6.3 Latin lower case letters

a	Distance; Geometrical data
a	Throat thickness of a weld
a	Area ratio
b	Width; Breadth
c	Distance; Outstand
d	Diameter; Depth; Length of diagonal
e	Eccentricity; Shift of centroidal axis
e	Edge distance; End distance
f	Strength (of a material)
g	Gap; Width of a tension field
h	Height
i	Radius of gyration; Integer
k	Coefficient; Factor
l (or ℓ or L)	Length; Span; Buckling length[a]

[a] l (lower case L) can be replaced by L or by ℓ (handwritten) for certain lengths or to avoid confusion with 1 (numeral) or I (upper case i)

n	Ratio of normal forces or normal stresses
n	Number of ...
p	Pitch; Spacing
q	Uniformly distributed force
r	Radius; Root radius
s	Staggered pitch; Distance
t	Thickness
uu	Major axis
vv	Minor axis
xx, yy, zz	Rectangular axes

1.6.4 Greek lower case letters

α	(alpha)	Angle; Ratio; Factor
α		Coefficient of linear thermal expansion
β	(beta)	Angle; Ratio; Factor
γ	(gamma)	Partial safety factor; Ratio
δ	(delta)	Deflection; Deformation
ε	(epsilon)	Strain; Coefficient = $[235/f_y]^{0,5}$ (f_y in N/mm^2)
η	(eta)	Coefficient (in Annex E)
θ	(theta)	Angle; Slope
λ	(lambda)	Slenderness ratio; Ratio
μ	(mu)	Slip factor; Factor
ν	(nu)	Poisson's ratio
ρ	(rho)	Reduction factor; Unit mass
σ	(sigma)	Normal stress
τ	(tau)	Shear stress
ϕ	(phi)	Rotation; Slope; Ratio
χ	(chi)	Reduction factor (for buckling)
ψ	(psi)	Stress ratio; Reduction factor
ψ		Factors defining representative values of variable actions.

1.6.5 Subscripts

A	Accidental; Area
a	Average (yield strength)
a, b....	First, second .... alternative
b	Basic (yield strength)
b	Bearing; Buckling
b	Bolt; Beam; Batten
C	Capacity; Consequences
c	Cross section
c	Concrete; column
com	Compression
cr	Critical
d	Design; Diagonal
dst	Destablizing
E	Effect of actions (with d or k)
E	Euler

eff	Effective
e	Effective (with further subscript)
eℓ	Elastic
ext	External
f	Flange; Fastener
g	Gross
G	Permanent action
h	Height; Higher
h	Horizontal
i	Inner
inf	Inferior; Lower
i, j, k	Indices (replace by numeral)
j	Joint
k	Characteristic
ℓ	Lower
L	Long
LT	Lateral-torsional
M	Material
M	(Allowing for) bending moment
m	Bending; Mean
max	Maximum
min	Minimum
N	(Allowing for) axial force
n	Normal
net	Net
nom	Nominal
o	Hole; Initial; Outer
o	Local buckling
o	Point of zero moment
ov	Overlap
p	Plate; Pin; Packing
p	Preloading (force)
p	Partial; Punching shear
pℓ	Plastic
Q	Variable action
R	Resistance
r	Rivet; Restraint
rep	Representative
S	Internal force; Internal moment
s	Tensile stress (area)
s	Slip; Storey
s	Stiff; Stiffener
ser	Serviceability
stb	Stabilizing
sup	Superior; Upper
t (or ten[*)])	Tension; Tensile
t (or tor[*)])	Torsion

u	Major axis of cross-section
u	Ultimate (tensile strength)
ult	Ultimate (limit state)
V	(Allowing for) shear force
v	Shear; Vertical
v	Minor axis of cross-section
vec	Vectorial effects
w	Web; Weld; Warping
x	Axis along member; Extension
y	Yield
y	Axis of cross-section
z	Axis of cross-section
σ	Normal stress
τ	Shear stress
$\perp$	Perpendicular
//	Parallel

1.6.6 Use of subscripts in Part 1.1 of Eurocode 3

(1) Strengths and properties of steel materials are nominal values, treated as characteristic values but written as below:

f_y	yield strength	[rather than f_{yk}]
f_u	ultimate strength	[rather than f_{uk}]
E	modulus of elasticity	[rather than E_k]

(2) To avoid ambiguity, subscripts are given in full in this Eurocode, but some may be omitted in practice where ambiguity is not caused by their omission.

(3) Where symbols with multiple subscripts are needed, they have been assembled in the following sequence:

- main parameter: *eg. M, N, β*
- variant type: *eg. pℓ, eff, b, c*
- sense: *eg. t, v*
- axis: *eg. y, z*
- location: *eg. 1, 2, 3*
- nature: *eg. R, S*
- level: *eg. d, k*
- index: *eg. 1, 2, 3*

(4) Dots are used to separate subscripts into pairs of characters, except as follows:

- Subscripts with more than one character are not sub-divided.
- Combinations Rd, Sd etc. are not sub-divided.

(5) Where two variant type subscripts are needed to describe a parameter, they may be separated by a comma:

eg. M, ψ

1.6.7 Conventions for member axes

(1) In general the convention for member axes is:

x-x — along the member
y-y — axis of the cross-section
z-z — axis of the cross-section

(2) For steel members, the conventions used for cross-section axes are:

- generally:
  y-y — cross-section axis parallel to the flanges
  z-z — cross-section axis perpendicular to the flanges
- for angle sections:
  y-y — axis parallel to the smaller leg
  z-z — axis perpendicular to the smaller leg
- where necessary:
  u-u — major axis (where this does not coincide with the yy axis)
  v-v — minor axis (where this does not coincide with the zz axis)

(3) The symbols used for dimensions and axes of rolled steel sections are indicated in Figure 1.1.

(4) *For rolled steel sections, section properties were formerly tabulated in Reference Standards with the following convention for cross-section axes:*

x — cross-section axis parallel to the flanges or the smaller leg.
y — cross-section axis perpendicular to the flanges or the smaller leg.

(5) The convention used for subscripts which indicate axes for moments is:

"Use the axis about which the moment acts."

(6) *For example, for an I-section a moment acting in the plane of the web is denoted* M_y *because it acts about the cross-section axis parallel to the flanges.*

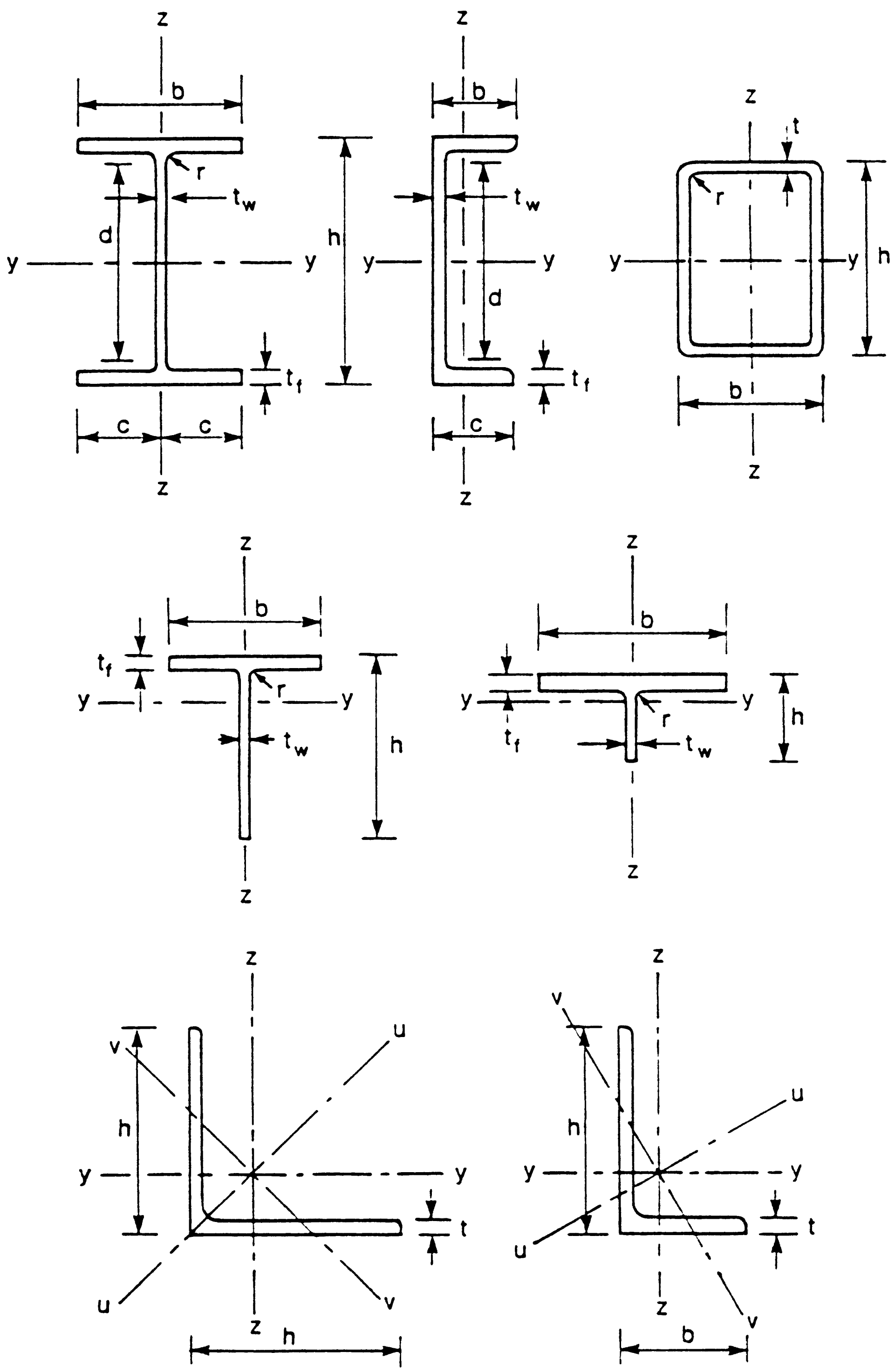

Figure 1.1 — Dimensions and axes of sections

2 Basis of design

2.1 Fundamental requirements

(1) A structure shall be designed and constructed in such a way that:

- with acceptable probability, it will remain fit for the use for which it is required, having due regard to its intended life and its cost, and
- with appropriate degrees of reliability, it will sustain all actions and other influences likely to occur during execution and use and have adequate durability in relation to maintenance costs.

(2) A structure shall also be designed in such a way that it will not be damaged by events like explosions, impact or consequences of human errors, to an extent disproportionate to the original cause.

(3) *The potential damage should be limited or avoided by appropriate choice of one or more of the following:*

- *avoiding, eliminating or reducing the hazards which the structure is to sustain*
- *selecting a structural form which has low sensitivity to the hazards considered*
- *selecting a structural form and design that can survive adequately the accidental removal of an individual element*
- *tying the structure together*

(4) The above requirements shall be met by the choice of suitable materials, by appropriate design and detailing and by specifying control procedures for production, construction and use as relevant for the particular project.

2.2 Definitions and classifications

2.2.1 Limit states and design situations

2.2.1.1 *Limit states*

(1) Limit states are states beyond which the structure no longer satisfies the design performance requirements.

Limit states are classified into:

- ultimate limit states
- serviceability limit states.

(2) Ultimate limit states are those associated with collapse, or with other forms of structural failure which may endanger the safety of people.

(3) States prior to structural collapse which, for simplicity, are considered in place of the collapse itself are also classified and treated as ultimate limit states.

(4) *Ultimate limit states which may require consideration include:*

- *loss of equilibrium of the structure or any part of it, considered as a rigid body,*
- *failure by excessive deformation, rupture, or loss of stability of the structure or any part of it, including supports and foundations.*

(5) Serviceability limit states correspond to states beyond which specified service criteria are no longer met.

(6) *Serviceability limit states which may require consideration include:*

- *deformations or deflections which adversely affect the appearance or effective use of the structure (including the proper functioning of machines or services) or cause damage to finishes or non-structural elements*
- *vibration which causes discomfort to people, damage to the building or its contents, or which limits its functional effectiveness.*

2.2.1.2 *Design situations*

(1) Design situations are classified as:

- persistent situations corresponding to normal conditions of use of the structure
- transient situations, for example during construction or repair
- accidental situations.

2.2.2 Actions

2.2.2.1 *Definitions and principal classification*[5)]

(1) An action (F) is:

- a force (load) applied to the structure (direct action), or
- an imposed deformation (indirect action); for example, temperature effects or settlement.

(2) Actions are classified:

i) by their variation in time:

- permanent actions (G), e.g. self-weight of structures, fittings, ancilliaries and fixed equipment
- variable actions (Q), e.g. imposed loads, wind loads or snow loads
- accidental actions (A), e.g. explosions or impact from vehicles

ii) by their spatial variation:

- fixed actions, e.g. self-weight [but see **2.3.2.3**(2) for structures very sensitive to variations in self-weight]
- free actions, which result in different arrangements of actions, e.g. movable imposed loads, wind loads, snow loads.

(3) *Supplementary classifications relating to the response of the structure are given in the relevant clauses.*

2.2.2.2 *Characteristic values of actions*

(1) Characteristic values F_k are specified:

- in ENV 1991 Eurocode 1 or other relevant loading codes, or
- by the client, or the designer in consultation with the client, provided that the minimum provisions specified in the relevant loading standards or by the competent authority are observed.

(2) For permanent actions where the coefficient of variation is large or where the actions are likely to vary during the life of the structure (e.g. for some superimposed permanent loads), two characteristic values are distinguished, an upper ($G_{k,sup}$) and a lower ($G_{k,inf}$). Elsewhere a single characteristic value (G_k) is sufficient.

(3) *The self-weight of the structure may, in most cases, be calculated on the basis of the nominal dimensions and mean unit masses.*

(4) For variable actions the characteristic value (Q_k) corresponds to either:

- the upper value with an intended probability of not being exceeded, or the lower value with an intended probability of not being reached, during some reference period, having regard to the intended life of the structure or the assumed duration of the design situation, or
- the specified value.

(5) For accidental actions the characteristic value A_k (when relevant) generally corresponds to a specified value.

2.2.2.3 *Representative values of variable actions*[6)]

(1) The main representative value is the characteristic value Q_k.

(2) Other representative values are related to the characteristic value Q_k by means of a factor ψ_i.

These values are defined as:

- combination value: $\psi_0 Q_k$ (see **2.3.2.2**)
- frequent value: $\psi_1 Q_k$ (see **2.3.4**)
- quasi-permanent value: $\psi_2 Q_k$ (see **2.3.4**)

(3) Supplementary representative values are used for fatigue verification and dynamic analysis.

(4) The factors ψ_0, ψ_1 and ψ_2 are specified:

- in ENV 1991 Eurocode 1 or other relevant loading standards, or
- by the client, or the designer in consultation with the client, provided that the minimum provisions specified in the relevant loading standards or by the competent authority are observed.

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5) Fuller definitions of the classification of actions will be found in ENV 1991 Eurocode 1.
6) Fuller definitions of representative values will be found in ENV 1991 Eurocode 1.

2.2.2.4 ***Design values of actions***

(1) The design value F_d of an action is expressed in general terms as:

$$F_d = \gamma_F F_k \qquad (2.1)$$

where γ_F is the partial safety factor for the action considered — taking account of, for example, the possibility of unfavourable deviations of the actions, the possibility of inaccurate modelling of the actions, uncertainties in the assessment of effects of actions and uncertainties in the assessment of the limit state considered.

(2) Specific examples of the use of γ_F are:

$G_d = \gamma_G G_k$

$Q_d = \gamma_Q Q_k$ or $\gamma_Q \psi_i Q_k$

$A_d = \gamma_A A_k$ (if A_d is not directly specified)

(3) The upper and lower design values of permanent actions are expressed as follows:

- where only a single characteristic value G_k is used [see **2.2.2.2**(2)] then:

  $G_{d,sup} = \gamma_{G,sup} G_k$

  $G_{d,inf} = \gamma_{G,inf} G_k$

- where upper and lower characteristic values of permanent actions are used [see **2.2.2.2**(2)] then:

  $G_{d,sup} = \gamma_{G,sup} G_{k,sup}$

  $G_{d,inf} = \gamma_{G,inf} G_{k,inf}$

  where $G_{k,inf}$ is the lower characteristic value of the permanent action

  $G_{k,sup}$ is the upper characteristic value of the permanent action

  $\gamma_{G,inf}$ is the lower value of the partial safety factor for the permanent action

  $\gamma_{G,sup}$ is the upper value of the partial safety factor for the permanent action

2.2.2.5 ***Design values of the effects of actions***

(1) The effects of actions (E) are responses (for example, internal forces and moments, stresses, strains) of the structure to the actions. Design values of the effects of actions (E_d) are determined from the design values of the actions, geometrical data and material properties when relevant:

$$E_d = E(F_d, a_d \ldots.) \qquad (2.2)$$

where a_d is defined in **2.2.4**.

2.2.3 Material properties

2.2.3.1 ***Characteristic values***

(1) A material property is represented by a characteristic value X_k which in general corresponds to a fractile in the assumed statistical distribution of the particular property of the material, specified by relevant standards and tested under specified conditions.

(2) In certain cases a nominal value is used as the characteristic value.

(3) Material properties for steel structures are generally represented by nominal values used as characteristic values.

(4) A material property may have two characteristic values, the upper value and the lower value. In most cases only the lower value need be considered. However, higher values of the yield strength, for example, should be considered in special cases where overstrength effects may produce a reduction in safety.

2.2.3.2 ***Design values***

(1) The design value X_d of a material property is generally defined as:

$$X_d = X_k/\gamma_M$$

where γ_M is the partial safety factor for the material property.

(2) For steel structures, the design resistance R_d is generally determined directly from the characteristic values of the material properties and geometrical data:

$$R_d = R\,(X_k, a_k, \ldots)/\gamma_M \qquad (2.3)$$

where γ_M is the partial safety factor for the resistance.

(3) The design value R_d may be determined from tests. Guidance is given in Chapter 8.

2.2.4 Geometrical data

(1) Geometrical data are generally represented by their nominal values:

$$a_d = a_{nom} \tag{2.4}$$

(2) In some cases the geometrical design values are defined by:

$$a_d = a_{nom} + \Delta a \tag{2.5}$$

The values of Δa are given in the appropriate clauses.

(3) *For imperfections to be adopted in the global analysis of the structure, see* ***5.2.4****.*

2.2.5 Load arrangements and load cases[7)]

(1) A load arrangement identifies the position, magnitude and direction of a free action.

(2) A load case identifies compatible load arrangements, sets of deformations and imperfections considered for a particular verification.

2.3 Design requirements

2.3.1 General

(1) It shall be verified that no relevant limit state is exceeded.

(2) All relevant design situations and load cases shall be considered.

(3) Possible deviations from the assumed directions or positions of actions shall be considered.

(4) Calculations shall be performed using appropriate design models (supplemented, if necessary, by tests) involving all relevant variables. The models shall be sufficiently precise to predict the structural behaviour, commensurate with the standard of workmanship likely to be achieved, and with the reliability of the information on which the design is based.

2.3.2 Ultimate limit states

2.3.2.1 ***Verification conditions***

(1) When considering a limit state of static equilibrium or of gross displacements or deformations of the structure, it shall be verified that:

$$E_{d,dst} \leq E_{d,stb} \tag{2.6}$$

where $E_{d,dst}$ is the design effect of the destabilizing actions

and $E_{d,stb}$ is the design effect of the stabilizing actions.

(2) When considering a limit state of rupture or excessive deformation of a section, member or connection (fatigue excluded) it shall be verified that:

$$S_d \leq R_d \tag{2.7}$$

where S_d is the design value of an internal force or moment (or of a respective vector of several internal forces or moments)

and R_d is the corresponding design resistance,
each taking account of the respective design values of all structural properties.

(3) When considering a limit state of transformation of the structure into a mechanism, it shall be verified that a mechanism does not occur unless actions exceed their design values, taking account of the respective design values of all structural properties.

(4) When considering a limit state of stability induced by second-order effects, it shall be verified that instability does not occur unless actions exceed their design values, taking account of the respective design values of all structural properties. In addition, sections shall be verified according to (2) above.

(5) When considering a limit state of rupture induced by fatigue, it shall be verified that the design value of the damage indicator D_d does not exceed unity, see Chapter 9.

[7)] Detailed rules on load arrangements and load cases are given in ENV 1991 Eurocode 1.

(6) When considering effects of actions, it shall be verified that:

$$E_d \leq C_d \quad (2.8)$$

where E_d is the design value of the particular effect of actions being considered
and C_d is the design capacity for that effect of actions.

2.3.2.2 *Combinations of actions*

(1) For each load case, design values E_d for the effects of actions shall be determined from combination rules involving the design values of actions given in Table 2.1.

Table 2.1 — Design values of actions for use in the combination of actions

Design situation	Permanent actions G_d	Variable actions Q_d		Accidental actions A_d
		Leading variable action	Accompanying variable action	
Persistent and Transient	$\gamma_G G_k$	$\gamma_Q Q_k$	$\psi_0 \gamma_Q Q_k$	
Accidental (if not specified differently elsewhere)	$\gamma_{GA} G_k$	$\psi_1 Q_k$	$\psi_2 Q_k$	$\gamma_A A_k$ (if A_d is not specified directly)

(2) The design values given in Table 2.1 shall be combined using the following rules (given in symbolic form):[8)]

- Persistent and transient design situations for verifications other than those relating to fatigue (fundamental combinations):

$$\sum_j \gamma_{G,j}\, G_{k,j} + \gamma_{Q,1}\, Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\, \psi_{0,i}\, Q_{k,i} \quad (2.9)$$

- Accidental design situations (if not specified differently elsewhere):

$$\sum_j \gamma_{GA,j}\, G_{k,j} + A_d + \psi_{1,1}\, Q_{k,1} + \sum_{i>1} \psi_{2,i}\, Q_{k,i} \quad (2.10)$$

where:

$G_{k,j}$ are the characteristic values of the permanent actions
$Q_{k,1}$ is the characteristic value of one of the variable actions
$Q_{k,i}$ are the characteristic values of the other variable actions
A_d is the design value (specified value) of the accidental action
$\gamma_{G,j}$ is the partial safety factor for the permanent action $G_{k,j}$
$\gamma_{GA,j}$ is as $\gamma_{G,j}$ but for accidental design situations
$\gamma_{Q,i}$ is the partial safety factor for the variable action $Q_{k,i}$
and ψ_0, ψ_1, ψ_2 are factors defined in **2.2.2.3**.

(3) Combinations for accidental design situations either involve an explicit accidental action A or refer to a situation after an accidental event (A = 0). Unless specified otherwise, $\gamma_{GA} = \boxed{1,0}$ may be used.

(4) In expressions (2.9) and (2.10), indirect actions shall be introduced where relevant.

(5) For fatigue, see Chapter 9.

(6) Simplified combinations for building structures are given in **2.3.3.1**.

8) Detailed rules on combinations of actions are given in ENV 1991 Eurocode 1.

2.3.2.3 ***Design values of permanent actions***

(1) In the various combinations defined above, those permanent actions that increase the effect of the variable actions (i.e. produce unfavourable effects) shall be represented by their upper design values and those that decrease the effect of the variable actions (i.e. produce favourable effects) by their lower design values [see **2.2.2.4**(3)].

(2) Where the results of a verification may be very sensitive to variations of the magnitude of a single permanent action from place to place in the structure, this action shall be treated as consisting of separate unfavourable and favourable parts. This applies in particular to the verification of static equilibrium, see **2.3.2.4**.

(3) Where a single permanent action is treated as consisting of separate unfavourable and favourable parts, allowance may be made for the relationship between these parts by adopting special design values [see **2.3.3.1**(3) for building structures].

(4) *Except for the cases mentioned in (2), the whole of each permanent action should be represented throughout the structure by either its lower or its upper design value, whichever gives the more unfavourable effect.*

(5) *For continuous beams and frames, the same design value of the self-weight of the structure [evaluated as in **2.2.2.2(3)**] may be applied to all spans, except for cases involving the static equilibrium of cantilevers (see **2.3.2.4**).*

2.3.2.4 ***Verification of static equilibrium***

(1) For the verification of static equilibrium, destabilizing (unfavourable) actions shall be represented by upper design values and stabilizing (favourable) actions by lower design values [see **2.3.2.1**(1)].

(2) For stabilizing effects, only those actions which can reliably be assumed to be present in the situation considered shall be included in the relevant combination.

(3) *Variable actions should be applied where they increase the destabilizing effects but omitted where they would increase the stabilizing effects.*

(4) *Account should be taken of the possibility that non-structural elements might be omitted or removed.*

(5) Permanent actions shall be represented by appropriate design values, depending on whether the destabilizing and stabilizing effects result from:

- the unfavourable and the favourable parts of a single permanent action, see (9) below, and/or
- different permanent actions, see (10) below.

(6) *The self-weights of any unrelated structural or non-structural elements made of different construction materials should be treated as different permanent actions.*

(7) *The self-weight of a homogeneous structure should be treated as a single permanent action consisting of separate unfavourable and favourable parts.*

(8) *The self-weights of essentially similar parts of a structure (or of essentially uniform non-structural elements) may also be treated as separate unfavourable and favourable parts of a single permanent action.*

(9) For building structures, the special partial safety factors given in **2.3.3.1**(3) apply to the unfavourable and the favourable parts of each single permanent action, as envisaged in **2.3.2.3**(2).

(10) For building structures, the normal partial safety factors given in **2.3.3.1**(1) apply to permanent actions other than those covered by (9).

(11) *For closely bounded or closely controlled permanent actions, smaller ratios of partial safety factors may apply in the other Parts of Eurocode 3.*

(12) Where uncertainty of the value of a geometrical dimension significantly affects the verification of static equilibrium, this dimension shall be represented in this verification by the most unfavourable value that it is reasonably possible for it to reach.

2.3.3 Partial safety factors for ultimate limit states

2.3.3.1 ***Partial safety factors for actions on building structures***

(1) For the persistent and transient design situations the partial safety factors given in Table 2.2 shall be used.

Table 2.2 — Partial safety factors for actions on building structures for persistent and transient design situations

	Permanent actions (γ_G)	Variable actions (γ_Q)	
		Leading variable action	Accompanying variable actions
Favourable effect $\gamma_{F,inf}$	$\boxed{1{,}0^{*)}}$	**)	**)
Unfavourable effect $\gamma_{F,sup}$	$\boxed{1{,}35^{*)}}$	$\boxed{1{,}5}$	$\boxed{1{,}5}$

*) See also **2.3.3.1**(3)
**) See Eurocode 1; in normal cases for building structures $\gamma_{Q,inf} = 0$.

(2) For accidental design situations to which expression (2.10) applies, the partial safety factors for the variable actions are taken as equal to 1,0.

(3) Where, according to **2.3.2.3**(2), a single permanent action needs to be considered as consisting of unfavourable and favourable parts, the favourable part may, as an alternative, be multiplied by:

$$\gamma_{G,inf} = \boxed{1{,}1}$$

and the unfavourable part by:

$$\gamma_{G,sup} = \boxed{1{,}35}$$

provided that applying $\gamma_{G,inf} = \boxed{1{,}0}$ both to the favourable part and to the unfavourable part does not give a more unfavourable effect.

(4) *Where the components of a vectorial effect can vary independently, favourable components (eg. the longitudinal force) should be multiplied by a reduction factor:*

$$\psi_{vec} = \boxed{0{,}8}$$

(5) *For building structures, as a simplification, expression (2.9) may be replaced by whichever of the following combinations gives the larger value:*

- *considering only the most unfavourable variable action:*

$$\sum_j \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} \tag{2.11}$$

- *considering all unfavourable variable actions:*

$$\sum_j \gamma_{G,j} G_{k,j} + 0{,}9 \sum_{i \geq 1} \gamma_{Q,i} Q_{k,i} \tag{2.12}$$

2.3.3.2 ***Partial safety factors for resistances***

(1) Partial safety factors for resistances are given in the relevant clauses in Chapters 5 and 6.

(2) Where structural properties are determined by testing see Chapter 8.

(3) For fatigue verifications see Chapter 9.

2.3.4 Serviceability limit states

(1) It shall be verified that:

$$E_d \leq C_d \text{ or } E_d \leq R_d \tag{2.13}$$

where:

C_d is a nominal value or a function of certain design properties of materials related to the design effect of actions considered, and

E_d is the design effect of actions, determined on the basis of one of the combinations defined below.

The required combination is identified in the particular clause for each serviceability verification, see **4.2.1**(4) and **4.3.1**(4).

(2) Three combinations of actions for serviceability limit states are defined by the following expressions:

Rare combination:

$$\sum_j G_{k,j} + Q_{k,1} + \sum_{i>1} \psi_{0,i} Q_{k,i} \qquad (2.14)$$

Frequent combination:

$$\sum_j G_{k,j} + \psi_{1,1} Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i} \qquad (2.15)$$

Quasi-permanent combination:

$$\sum_j G_{k,j} + \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \qquad (2.16)$$

where the notation is defined in **2.3.2.2**(2)

(3) Where simplified compliance rules are given in the relevant clauses dealing with serviceability limit states, detailed calculations using combinations of actions are not required.

(4) Where the design considers compliance of serviceability limit states by detailed calculations, simplified expressions may be used for building structures.

(5) *For building structures, as a simplification, expression (2.14) for the rare combination may be replaced by whichever of the following combinations gives the larger value:*

- *considering only the most unfavourable variable action:*

$$\sum_j G_{k,j} + Q_{k,1} \qquad (2.17)$$

- *considering all unfavourable variable actions:*

$$\sum_j G_{k,j} + 0{,}9 \sum_{i\geq 1} Q_{k,i} \qquad (2.18)$$

These two expressions may also be used as a substitute for expression (2.15) for the frequent combination.

(6) Values of γ_M shall be taken as 1,0 for all serviceability limit states, except where stated otherwise in particular clauses.

2.4 Durability

(1) To ensure an adequately durable structure, the following inter-related factors shall be considered:

- the use of the structure
- the required performance criteria
- the expected environmental conditions
- the composition, properties and performance of the materials
- the shape of members and the structural detailing
- the quality of workmanship and level of control
- the particular protective measures
- the likely maintenance during the intended life.

(2) The internal and external environmental conditions shall be estimated at the design stage to assess their significance in relation to durability and to enable adequate provisions to be made for protection of the materials.

2.5 Fire resistance

(1) For fire resistance, refer to ENV 1993-1-2 Eurocode 3-1.2[9)].

3 Materials

3.1 General

(1) The material properties given in this Chapter are nominal values to be adopted as characteristic values in design calculations.

(2) Other material properties are given in the relevant Reference Standards defined in normative Annex B.

3.2 Structural steel

3.2.1 Scope

(1) This Part 1.1 of Eurocode 3 covers the design of structures fabricated from steel material conforming to Reference Standard 1, see normative Annex B.

(2) It may also be used for other structural steels, provided that adequate data exist to justify the application of the relevant design and fabrication rules. Test procedures and test evaluation shall conform with Chapters 2 and 8 of this Part 1.1 and the test requirements shall align with those required in Reference Standard 1.

(3) For high strength steel refer to normative Annex D[9)].

3.2.2 Material properties for hot rolled steel

3.2.2.1 *Nominal values*

(1) The nominal values of the yield strength f_y and the ultimate tensile strength f_u for hot rolled steel are given in Table 3.1 for steel grades Fe 360, Fe 430 and Fe 510 in accordance with EN 10025 and steel grades Fe E 275 and Fe E 355 in accordance with prEN 10113.

Table 3.1 — Nominal values of yield strength f_y and ultimate tensile strength f_u for structural steel to EN 10025 or prEN 10113

Nominal steel grade	Thickness t mm[a]			
	t ≤ 40 mm		40 mm < t ≤ 100 mm[b]	
	f_y (N/mm²)	f_u (N/mm²)	f_y (N/mm²)	f_u (N/mm²)
EN 10025:				
Fe 360	235	360	215	340
Fe 430	275	430	255	410
Fe 510	355	510	335	490
prEN 10113:				
Fe E 275	275	390	255	370
Fe E 355	355	490	335	470

[a] t is the nominal thickness of the element.
[b] 63 mm for plates and other flat products in steels of delivery condition TM to prEN 10113-3

(2) The nominal values in Table 3.1 may be adopted as characteristic values in calculations.

(3) As an alternative, the values specified in EN 10025 and prEN 10113 for a larger range of thicknesses may be used.

(4) Similar values may be adopted for hot finished structural hollow sections.

(5) For high strength steel refer to normative Annex D[9)].

[9)] In preparation

3.2.2.2 ***Plastic analysis***

(1) Plastic analysis (see **5.2.1.4**) may be utilised in the global analysis of structures or their elements provided that the steel complies with the following additional requirements:

- the ratio of the specified minimum ultimate tensile strength f_u to the specified minimum yield strength f_y satisfies:

  $f_u / f_y \geq 1{,}2$

- the elongation at failure on a gauge length of $5{,}65 \sqrt{A_o}$ (where A_o is the original cross section area) is not less than 15 %
- the stress-strain diagram shows that the ultimate strain ε_u corresponding to the ultimate tensile strength f_u is at least 20 times the yield strain ε_y corresponding to the yield strength f_y.

(2) *The steel grades listed in Table 3.1 may be accepted as satisfying these requirements.*

3.2.2.3 ***Fracture toughness***

(1) The material shall have sufficient fracture toughness to avoid brittle fracture at the lowest service temperature expected to occur within the intended life of the structure.

(2) *In normal cases of welded or non-welded members in building structures subject to static loading or fatigue loading (but not impact loading), no further check against brittle fracture is necessary if the conditions given in Table 3.2 are satisfied.*

(3) *For high strength steel refer to normative Annex D.*

(4) *For all other cases reference should be made to informative Annex C.*

3.2.3 Material properties for cold formed steel

(1) The nominal values of the yield strength and the ultimate tensile strength (to be adopted as characteristic values in calculations) for cold formed steel are specified in ENV 1993-1-3 Eurocode 3-1.3[10)].

(2) The average yield strength of cold finished structural hollow sections shall be determined as specified in Figure 5.5.2.

3.2.4 Dimensions, mass and tolerances

(1) The dimensions and mass of all rolled steel sections, plates and structural hollow sections, and their dimensional and mass tolerances, shall conform with Reference Standard 2, see normative Annex B.

3.2.5 Design values of material coefficients

(1) The material coefficients to be adopted in calculations for the steels covered by this Eurocode shall be taken as follows:

• modulus of elasticity	$E = 210\ 000$ N/mm^2
• shear modulus	$G = E/2(1 + \nu)$
• Poisson's ratio	$\nu = 0{,}3$
• coefficient of linear thermal expansion	$\alpha = 12 \times 10^{-6}$ per °C
• unit mass	$\rho = 7\ 850$ kg/m^3

10) In preparation

Table 3.2 — Maximum thickness for statically loaded structural elements without reference to informative Annex C

Steel grade and quality	Maximum thickness (mm) for lowest service temperature of					
	0 °C		– 10 °C		– 20 °C	
Service condition	S1	S2	S1	S2	S1	S2
EN 10025(1):						
Fe 360 B	150	41	108	30	74	22
Fe 360 C	250	110	250	75	187	53
Fe 360 D	250	250	250	212	250	150
Fe 430 B	90	26	63	19	45	14
Fe 430 C	250	63	150	45	123	33
Fe 430 D	250	150	250	127	250	84
Fe 510 B	40	12	29	9	21	6
Fe 510 C	106	29	73	21	52	16
Fe 510 D	250	73	177	52	150	38
Fe 510 DD(2)	250	128	250	85	250	59
prEN 10113:(3)						
Fe E 275 KG(4)	250	250	250	192	250	150
Fe E 275 KT	250	250	250	250	250	250
Fe E 355 KG(4)	250	128	250	85	250	59
Fe E 355 KT	250	250	250	250	250	150

Service conditions(5):

S1 Either:
- non-welded, or
- in compression

S2 As welded, in tension

In both cases this table assumes loading rate R1 and consequences of failure condition C2, see informative Annex C.

Notes:

(1) For rolled sections over 100 mm thick, the minimum Charpy V-notch energy specified in EN 10025 is subject to agreement. For thicknesses up to 150 mm, a minimum value of 27 J at the relevant specified test temperature is required and 23 J for thicknesses over 150 mm up to 250 mm.

(2) For steel grade Fe 510 DD to EN 10025, the specified minimum Charpy V-notch energy value is 40 J at – 20 °C. The entries in this row assume an equivalent value of 27 J at – 30 °C.

(3) For steels of delivery condition N to prEN 10113-2 over 150 mm thick and for steels of delivery condition TM to prEN 10113-3 over 150 mm thick for long products and over 63 mm thick for flat products, the minimum Charpy V-notch energy specified in prEN 10113 is subject to agreement. For thicknesses up to 150 mm, a minimum value of 27 J is required and 23 J for thicknesses over 150 mm up to 250 mm. The test temperature should be – 30 °C for KG quality steel and – 50 °C for KT quality steel.

(4) For steel of quality KG to prEN 10113, the specified minimum values of Charpy V-notch energy go down to 40 J at – 20 °C. The entries in this row assume an equivalent value of 27 J at – 30 °C.

(5) For full details of service conditions, refer to informative Annex C.

3.3 Connecting devices

3.3.1 General

(1) Connecting devices shall be suitable for their specified use.

(2) *Suitable connecting devices include bolts, friction grip fasteners, rivets and welds, each to the appropriate Reference Standard, see normative Annex B.*

3.3.2 Bolts, nuts and washers

3.3.2.1 *General*

(1) Bolts, nuts and washers shall conform with Reference Standard 3, see normative Annex B.

(2) Bolts of grades lower than 4.6 or higher than 10.9 shall not be used unless test results prove their acceptability in a particular application.

(3) The nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} (to be adopted as characteristic values in calculations) are given in Table 3.3.

Table 3.3 — Nominal values of yield strength f_{yb} and ultimate tensile strength f_{ub} for bolts

Bolt grade	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1 000

3.3.2.2 *Preloaded bolts*

(1) High strength bolts may be used as preloaded bolts with controlled tightening, if they conform with the requirements for preloaded bolts in Reference Standard 3.

(2) Other suitable types of high strength bolts may also be used as preloaded bolts with controlled tightening, when agreed between the client, the designer and the competent authority.

3.3.3 Other types of preloaded fasteners

(1) Other suitable types of high strength fasteners (such as high strength swaged fasteners) may also be used as preloaded fasteners, when agreed between the client, the designer and the competent authority, provided that they have similar mechanical properties to those required for preloaded bolts and are capable of being reliably tightened to appropriate specified initial preloads.

3.3.4 Rivets

(1) The material properties, dimensions and tolerances of steel rivets shall conform with Reference Standard 5, see normative Annex B.

3.3.5 Welding consumables

(1) All welding consumables shall conform with Reference Standard 4, see normative Annex B.

(2) The specified yield strength, ultimate tensile strength, elongation at failure and minimum Charpy V-notch energy value of the filler metal, shall all be either equal to, or better than, the corresponding values specified for the steel grade being welded.

4 Serviceability limit states

4.1 Basis

(1) Serviceability limit states, see also **2.2.1.1**, for steelwork are:

- deformations or deflections which adversely affect the appearance or effective use of the structure (including the proper functioning of machines or services)
- vibration, oscillation or sway which causes discomfort to the occupants of a building or damage to its contents
- deformations, deflections, vibration, oscillation or sway which causes damage to finishes or non-structural elements.

(2) To avoid exceeding these limits, it is necessary to limit deformations, deflections and vibrations.

(3) *Except when specific limiting values are agreed between the client, the designer and the competent authority, the limiting values given in this Chapter should be applied.*

(4) *When plastic global analysis is used for the ultimate limit state, the possibility that plastic redistribution of forces and moments would also occur at the serviceability limit state should be investigated. This should be permitted only where it can be shown that it will not be repeated. It should also be taken into account in calculating the deformations.*

(5) Where preloaded bolts are used in Category B connections [see **6.5.3.1**(3)] the requirements given in **6.5.8** for slip-resistance at the serviceability limit state shall be satisfied.

4.2 Deflections

4.2.1 Requirements

(1) Steel structures and components shall be so proportioned that deflections are within the limits agreed between the client, the designer and the competent authority as being appropriate to the intended use and occupancy of the building and the nature of the materials to be supported.

(2) *Recommended limits for deflections are given in* ***4.2.2****. In some cases more stringent limits (or exceptionally, less stringent limits) will be appropriate to suit the use of the building or the characteristics of the cladding materials or to ensure the proper operation of lifts etc.*

(3) *The values given in* ***4.2.2*** *are empirical values. They are intended for comparison with the results of calculations and should not be interpreted as performance criteria.*

(4) *The design values given in* ***2.3.4*** *for the rare combination should be used in connection with all limiting values given in section* ***4.2****.*

(5) *The deflections should be calculated making due allowance for any second order effects, the rotational stiffness of any semi-rigid joints and the possible occurrence of any plastic deformations at the serviceability limit state.*

4.2.2 Limiting values

(1) *The limiting values for vertical deflections given below are illustrated by reference to the simply supported beam shown in Figure 4.1, in which:*

$$\delta_{max} = \delta_1 + \delta_2 \Pi \delta_0 \qquad (4.1)$$

where δ_{max} *is the sagging in the final state relative to the straight line joining the supports.*

δ_0 *is the pre-camber (hogging) of the beam in the unloaded state, (state 0).*

δ_1 *is the variation of the deflection of the beam due to the permanent loads immediately after loading, (state 1).*

and δ_2 *is the variation of the deflection of the beam due to the variable loading plus any time dependent deformations due to the permanent load, (state 2).*

(2) *For buildings, the recommended limits for vertical deflections are given in Table 4.1, in which L is the span of the beam. For cantilever beams, the length L to be considered is twice the projecting length of the cantilever.*

(3) *For crane gantry girders and runway beams, the horizontal and vertical deflections should be limited according to the use and class of the equipment.*

(4) *For buildings the recommended limits for horizontal deflections at the tops of the columns are:*

- *Portal frames without gantry cranes:* $h/150$
- *Other single storey buildings:* $h/300$
- *In a multistorey building:*
  - *In each storey* $h/300$
  - *On the structure as a whole* $h_o/500$

where h *is the height of the column or of the storey*

and h_o *is the overall height of the structure.*

4.2.3 Ponding

(1) *To ensure the correct discharge of rainwater from a flat or nearly flat roof, the design of all roofs with a slope of less than 5 % should be checked to ensure that rainwater cannot collect in pools. In this check, due allowance should be made for possible construction inaccuracies and settlements of foundations, deflections of roofing materials, deflections of structural members and the effects of precamber. This also applies to floors of car parks and other open sided structures.*

(2) *Precambering of beams may reduce the likelihood of rainwater collecting in pools, provided that rainwater outlets are appropriately located.*

(3) *Where the roof slope is less than 3 % additional calculations should be made to check that collapse cannot occur due to the weight of water:*

- *either collected in pools which may be formed due to the deflection of structural members or roofing material*
- *or retained by snow.*

Table 4.1 — Recommended limiting values for vertical deflections

Conditions	Limits (see Figure 4.1)	
	δ_{max}	δ_2
Roofs generally	L/200	L/250
Roofs frequently carrying personnel other than for maintenance	L/250	L/300
Floors generally	L/250	L/300
Floors and roofs supporting plaster or other brittle finish or non-flexible partitions	L/250	L/350
Floors supporting columns (unless the deflection has been included in the global analysis for the ultimate limit state)	L/400	L/500
Where δ_{max} can impair the appearance of the building	L/250	—

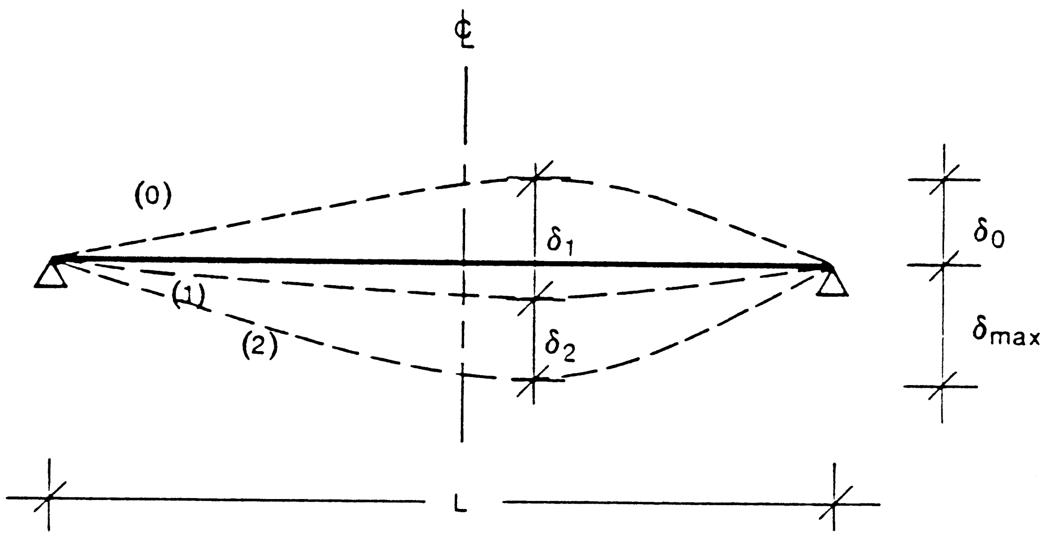


Figure 4.1 — Vertical deflections to be considered

4.3 Dynamic effects

4.3.1 Requirements

(1) Suitable provisions shall be made in the design for the effects of imposed loads which can induce impact, vibration, etc.

(2) *The dynamic effects to be considered at the serviceability limit state are vibration caused by machines and oscillation caused by harmonic resonance.*

(3) *The natural frequencies of structures or parts of structures should be sufficiently different from those of the excitation source to avoid resonance.*

(4) *The design values given in **2.3.4** for the frequent combination should be used in connection with all limiting values given in section **4.3**.*

4.3.2 Structures open to the public

(1) The oscillation and vibration of structures on which the public can walk shall be limited to avoid significant discomfort to users.

(2) *In the case of floors over which people walk regularly, such as the floors of dwellings, offices and the like, the lowest natural frequency of the floor construction should not be lower than 3 cycles/second. This condition will be satisfied if the instantaneous total deflection* $\delta_1 + \delta_2$ *(as defined in* ***4.2.2*** *but calculated using the frequent combination) is less than 28 mm. These limits may be relaxed where justified by high damping values.*

(3) *In the case of a floor which is jumped or danced on in a rhythmical manner, such as the floor of a gymnasium or dance hall, the lowest natural frequency of that floor should not be less than 5 cycles/second. This condition will be satisfied if the deflection calculated as above is not greater than 10 mm.*

(4) *If necessary, a dynamic analysis may be carried out to show that the accelerations and frequencies which would be produced would not be such as to cause significant discomfort to users or damage to equipment.*

4.3.3 Wind — excited oscillations

(1) Unusually flexible structures, such as very slender tall buildings or very large roofs, and unusually flexible elements, such as light tie rods, shall be investigated under dynamic wind loads both for vibrations in plane and also for vibrations normal to the wind direction.

(2) *Such structures should be examined for:*

- *gust induced vibrations*
- *vortex induced vibrations*

(3) *See also ENV 1991 Eurocode 1*[11].

5 Ultimate limit states

5.1 Basis

5.1.1 General

(1) Steel structures and components shall be so proportioned that the basic design requirements for the ultimate limit state given in Chapter 2 are satisfied.

(2) The partial safety factor γ_M shall be taken as follows:

- resistance of Class 1, 2 or 3 cross-section:[a] $\gamma_{M0} = \boxed{1,1}$
- resistance of Class 4 cross-section:[a] $\gamma_{M1} = \boxed{1,1}$
- resistance of member to buckling: $\gamma_{M1} = \boxed{1,1}$
- resistance of net section at bolt holes: $\gamma_{M2} = \boxed{1,25}$
- resistance of connections: see Chapter 6

[a] For classification of cross-sections see **5.3**

5.1.2 Frame design

(1) Frames shall be checked for:

- resistance of cross-sections (**5.4**)
- resistance of members (**5.5**)
- resistance of connections (Chapter 6)
- frame stability (**5.2.6**)
- static equilibrium (**2.3.2.4**)

[11] *In preparation*

(2) *When checking the resistance of cross-sections and members of a frame, each member may be treated as isolated from the frame, with forces and moments applied to each end as determined from the frame analysis. The conditions of restraint at each end should be determined by considering the member as part of the frame and should be consistent with the type of analysis (see **5.2.1** and **5.2.2**) and mode of failure (see **5.2.6**).*

5.1.3 Tension members

(1) Tension members shall be checked for:

- resistance of cross-sections (**5.4.3**)

5.1.4 Compression members

(1) Compression members shall be checked for:

- resistance of cross-sections (**5.4.4**)
- resistance to buckling (**5.5.1**)

5.1.5 Beams

(1) Members subject to bending shall be checked for:

- resistance of cross-sections (**5.4**)
- resistance to lateral-torsional buckling (**5.5.2**)
- resistance to shear buckling (**5.6**)
- resistance to flange-induced buckling (**5.7.7**)
- resistance to web crippling (**5.7.1**)

5.1.6 Members with combined axial force and moment

(1) Members subject to combined axial force and moment shall be checked for:

- resistance of cross-sections to the combined effects (**5.4.8**)
- resistance of members to the combined effects (**5.5.3** and **5.5.4**)
- the criteria for beams (**5.1.5**)
- the criteria for tension members (**5.1.3**) or compression members (**5.1.4**) as appropriate

5.1.7 Joints and connections

(1) Joints and connections shall satisfy the requirements specified in Chapter 6.

5.1.8 Fatigue

(1) Where repeated fluctuating loads are applied to a structure, its resistance to fatigue shall be checked.

(2) For hot-rolled steelwork and for hot finished and cold-finished structural hollow sections, the requirements given in Chapter 9 shall be satisfied.

(3) *For cold-formed steelwork, the design rules given in ENV 1993-1-3 Eurocode 3-1.3*[12)] *cover only structures which are predominantly statically loaded. Cold-formed steelwork should not be used for structures in which fatigue predominates, unless adequate data for the fatigue assessment are available which demonstrate that the fatigue resistance is sufficient.*

(4) *For building structures a fatigue check is not normally required, except for:*

- *members supporting lifting appliances or rolling loads,*
- *members supporting vibrating machinery,*
- *members subject to wind-induced oscillations,*
- *members subject to crowd-induced oscillations.*

[12)] In preparation

5.2 Calculation of internal forces and moments

5.2.1 Global analysis

5.2.1.1 ***Methods of analysis***

(1) The internal forces and moments in a statically determinate structure shall be obtained using statics.

(2) The internal forces and moments in a statically indeterminate structure may generally be determined using either:

a) elastic global analysis (**5.2.1.3**)

b) plastic global analysis (**5.2.1.4**)

(3) Elastic global analysis may be used in all cases.

(4) Plastic global analysis may be used only where the member cross-sections satisfy the requirements specified in **5.2.7** and **5.3.3** and the steel material satisfies the requirements specified in **3.2.2.2**.

(5) *When the global analysis is carried out by applying the loads in a series of increments, it may be assumed to be sufficient, in the case of building structures, to adopt simultaneous proportional increases of all loads.*

5.2.1.2 ***Effects of deformations***

(1) The internal forces and moments may generally be determined using either:

a) first order theory, using the initial geometry of the structure.

b) second order theory, taking into account the influence of the deformation of the structure.

(2) First order theory may be used for the global analysis in the following cases:

a) braced frames (**5.2.5.3**)

b) non-sway frames (**5.2.5.2**)

c) design methods which make indirect allowances for second-order effects (**5.2.6**).

(3) Second order theory may be used for the global analysis in all cases.

5.2.1.3 ***Elastic global analysis***

(1) Elastic global analysis shall be based on the assumption that the stress-strain behaviour of the material is linear, whatever the stress level.

(2) This assumption may be maintained for both first-order and second-order elastic analysis, even where the resistance of a cross-section is based on its plastic resistance, see **5.3.3**.

(3) Following a first-order elastic analysis, the calculated bending moments may be modified by redistributing up to 15 % of the peak calculated moment in any member, provided that:

a) the internal forces and moments in the frame remain in equilibrium with the applied loads, and

b) all the members in which the moments are reduced have Class 1 or Class 2 cross-sections (see **5.3**).

(4) The design assumptions for the connections shall satisfy the requirements specified in **5.2.2**.

5.2.1.4 ***Plastic global analysis***

(1) Plastic global analysis may be carried out using either:

- Rigid-Plastic methods.
- Elastic-Plastic methods.

(2) The following methods of Elastic-Plastic analysis may be used:

- Elastic — Perfectly Plastic
- Elasto-plastic

(3) When plastic global analysis is used, lateral restraint shall be provided at all plastic hinge locations at which plastic hinge rotation may occur under any load case.

(4) *The restraint should be provided within a distance along the member from the theoretical plastic hinge location not exceeding half the depth of the member.*

(5) *Rigid-Plastic methods should not be used for second-order analysis, except as specified in* ***5.2.6.3****.*

(6) *In "Rigid-Plastic" analysis elastic deformations of the members and the foundations are neglected and plastic deformations are assumed to be concentrated at plastic hinge locations.*

(7) *In "Elastic — Perfectly Plastic" analysis, it is assumed that the cross-section remains fully elastic until the plastic resistance moment is reached and then becomes fully plastic. Plastic deformations are assumed to be concentrated at the plastic hinge locations.*

(8) *In "Elasto-plastic" analysis, the bi-linear stress-strain relationship indicated in Figure 5.2.1 may be used for the grades of structural steel specified in Chapter 3. Alternatively, a more precise relationship may be adopted. The cross-section remains fully elastic until the stress in the extreme fibres reaches the yield strength. As the moment continues to increase, the section yields gradually as plasticity spreads across the cross-section and plastic deformations extend partially along the member.*

(9) *To avoid possible computational difficulties when using a computer for elasto-plastic analysis, the alternative bi-linear stress-strain relationship indicated in Figure 5.2.2 may be used if necessary.*

(10) *When elastic-plastic analysis is carried out, it may be assumed to be sufficient, in the case of building structures, to apply the loads in a series of increments, stopping when the full design load is reached, and to use the resulting internal forces and moments to check the resistances of the cross-sections and the buckling resistances of the members.*

(11) *In the case of building structures, it is not normally necessary to consider the effects of alternating plasticity.*

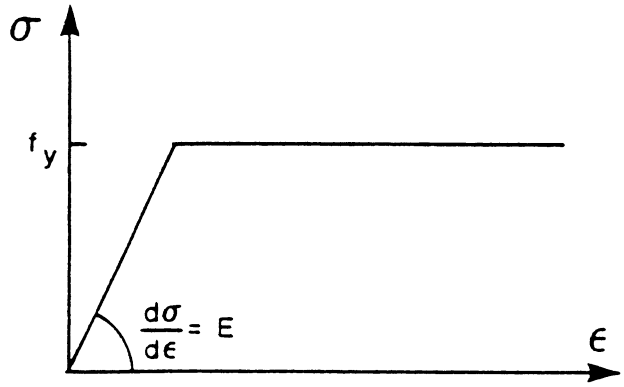


Figure 5.2.1 — Bi-linear stress-strain relationship

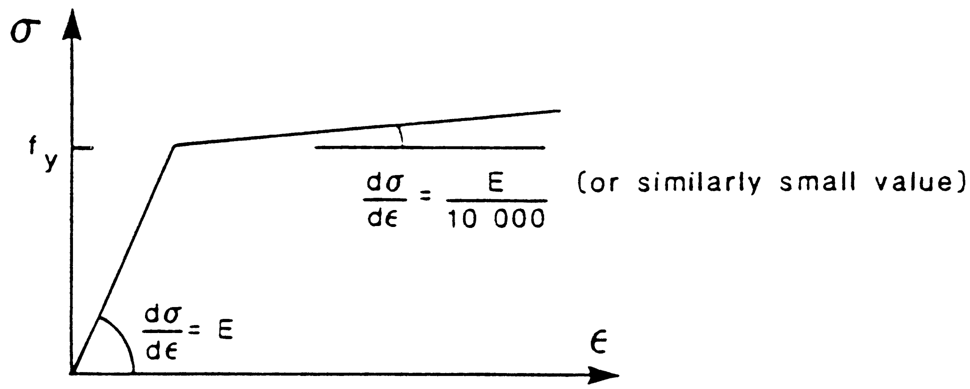


Figure 5.2.2 — Alternative bi-linear stress-strain relationship (for use in elasto-plastic analysis only)

5.2.2 Design assumptions

5.2.2.1 *Basis*

(1) The assumptions made in the global analysis of the structure shall be consistent with the anticipated type of behaviour of the connections.

(2) The assumptions made in the design of the members shall be consistent with (or conservative in relation to) the method used for the global analysis and with the anticipated type of behaviour of the connections.

(3) *Table 5.2.1* shows the type *of connections required for different types of framing, depending on the method of global analysis used.*

(4) *The requirements for the various types of connections are given in **6.4.2** and **6.4.3***

(5) *For classification of beam-to-column connections as rigid or semi-rigid see **6.9.6**.*

(6) *When it is necessary to calculate the elastic critical load for failure of a frame in a sway mode, account should be taken of the effects of any semi-rigid connections, irrespective of whether elastic analysis or plastic analysis is used for the global analysis of the frame.*

(7) *Where semi-rigid connections are used, the initial value of the rotational stiffness (see **6.9.6**) should be used when calculating elastic critical loads or buckling lengths.*

5.2.2.2 *Simple framing*

(1) *In simple framing the connections between the members may be assumed not to develop moments. In the global analysis, members may be assumed to be effectively pin connected.*

(2) *The connections should satisfy the requirements for nominally pinned connections, either:*

a) *as given in **6.4.2.1**.*

b) *as given in **6.4.3.1***

5.2.2.3 *Continuous framing*

(1) *Elastic analysis should be based on the assumption of full continuity, with rigid connections which satisfy the requirements given in **6.4.2.2**.*

(2) *Rigid-Plastic analysis should be based on the assumption of full continuity, with full strength connections which satisfy the requirements given in **6.4.3.2**.*

(3) *Elastic-Plastic analysis should be based on the assumption of full continuity with rigid full-strength connections which satisfy the requirements given in both **6.4.2.2** and **6.4.3.2**.*

5.2.2.4 *Semi-continuous framing*

(1) *Elastic analysis should be based on reliably predicted design moment-rotation or force-displacement characteristics for the connections used.*

(2) *Rigid-Plastic analysis should be based on the design moment resistances of connections which have been demonstrated to have sufficient rotation capacity, see **6.4.3** and **6.9.5**.*

(3) *Elastic-Plastic analysis should be based on the design moment-rotation characteristics of the connections, see **6.9.2**.*

Table 5.2.1 — Design assumptions

Type of framing	Method of global analysis	Types of connections
Simple	Pin joints	Nominally pinned (**6.4.2.1**) Nominally pinned (**6.4.3.1**)
Continuous	Elastic	Rigid (**6.4.2.2**) Nominally pinned (**6.4.3.1**)
	Rigid-Plastic	Full-strength (**6.4.3.2**) Nominally pinned (**6.4.3.1**)
	Elastic-Plastic	Full-strength — Rigid (**6.4.3.2** and **6.4.2.2**) Nominally pinned (**6.4.3.1** and **6.4.2.1**)
Semi-continuous	Elastic	Semi-rigid (**6.4.2.3**) Rigid (**6.4.2.2**) Nominally pinned (**6.4.2.1**)
	Rigid-Plastic	Partial-strength (**6.4.3.3**) Full-strength (**6.4.3.2**) Nominally pinned (**6.4.3.1**)
	Elastic-Plastic	Partial-strength — Semi-rigid (**6.4.3.3** and **6.4.2.3**) Partial-strength — Rigid (**6.4.3.3** and **6.4.2.2**) Full-strength — Semi-rigid (**6.4.3.2** and **6.4.2.3**) Full-strength — Rigid (**6.4.3.2** and **6.4.2.2**) Nominally pinned (**6.4.3.1** and **6.4.2.1**)

5.2.3 Structural systems

5.2.3.1 *Structures*

(1) The extent of global analysis required depends on the form of structure, as follows:

a) **Simple structural elements:**

Single-span beams and individual tension or compression members are statically determinate. Triangulated frames may be statically determinate or statically indeterminate.

b) **Continuous beams and non-sway frames:**

Continuous beams and frames in which sway effects are negligible, or are eliminated by suitable means (see **5.2.5**), shall be analysed under appropriate arrangements of the variable loads to determine those combinations of internal forces and moments which are critical for verifying the resistance of the individual members and of the connections.

c) **Sway frames:**

Sway frames (see **5.2.5**) shall be analysed under those arrangements of the variable loads which are critical for failure in a sway mode. In addition, sway frames shall also be analysed for the non-sway mode as described in b).

(2) The initial sway imperfections specified in **5.2.4.3** — and member imperfections where necessary, see **5.2.4.2**(4) — shall be included in the global analysis of all frames.

5.2.3.2 *Sub-frames*

(1) For the global analysis, the structure may be sub-divided into a number of sub-frames, provided that:

- the structural interaction between the sub-frames is reliably modelled.
- the arrangement of the sub-frames is appropriate for the structural system used.
- account is taken of possible adverse effects of interaction between the sub-frames.

5.2.3.3 *Stiffness of bases*

(1) Account shall be taken of the deformation characteristics of the bases or other foundations to which columns have moment-resisting connections. Appropriate rotational stiffness values shall be adopted in all methods of global analysis other than the rigid-plastic method.

(2) Where an actual pin or rocker is used, the rotational stiffness of the foundation shall be taken as zero.

(3) *Optionally, appropriate rotational stiffness values may also be adopted to represent the semi-rigid nature of nominally pinned bases.*

5.2.3.4 *Simple framing*

(1) *Suitable methods of modelling structures with simple framing are given in Annex H*[13].

5.2.3.5 *Continuous framing*

(1) *Suitable sub-frames for global analysis of rigid-jointed frames are given in Annex H*[13].

5.2.3.6 *Semi-continuous framing*

(1) *Suitable sub-frames may also be used for the global analysis of structures with semi-continuous framing, see Annex H*[13].

5.2.4 Allowance for imperfections

5.2.4.1 *Basis*

(1) Appropriate allowances shall be incorporated to cover the effects of practical imperfections, including residual stresses and geometrical imperfections such as lack of verticality, lack of straightness, lack of fit and the unavoidable minor eccentricities present in practical connections.

(2) Suitable equivalent geometric imperfections may be used, with values which reflect the possible effects of all types of imperfection.

(3) The effects of imperfections shall be taken into account in the following cases:

a) Global analysis

b) Analysis of bracing systems

c) Member design

5.2.4.2 *Method of application*

(1) Imperfections shall be allowed for in the analysis by including appropriate additional quantities, comprising frame imperfections, member imperfections and imperfections for analysis of bracing systems.

(2) The effects of the frame imperfections given in **5.2.4.3** shall be included in the global analysis of the structure. The resulting forces and moments shall be used for member design.

(3) The effects of the imperfections given in **5.2.4.4** shall be included in the analysis of bracing systems. The resulting forces shall be used for member design.

(4) The effects of member imperfections (see **5.2.4.5**) may be neglected when carrying out the global analysis of frames, except in sway frames (see **5.2.5.2**) in the case of members which are subject to axial compression, which have moment-resisting connections and in which:

$$\bar{\lambda} > 0{,}5\ [A f_y / N_{Sd}]^{0{,}5} \tag{5.1}$$

where N_{Sd} is the design value of the compressive force

and $\bar{\lambda}$ is the in-plane non-dimensional slenderness (see **5.5.1.2**) calculated using a buckling length equal to the system length.

[13] To be prepared at a later stage.

5.2.4.3 ***Frame imperfections***

(1) The effects of imperfections shall be allowed for in frame analysis by means of an equivalent geometric imperfection in the form of an initial sway imperfection ϕ determined from:

$$\phi = k_c\, k_s\, \phi_o \tag{5.2}$$

with $\phi_o = 1/200$

$k_c = [0{,}5 + 1/n_c]^{0,5}$ but $k_c \leq 1{,}0$

and $k_s = [0{,}2 + 1/n_s]^{0,5}$ but $k_s \leq 1{,}0$

where n_c is the number of columns per plane

and n_s is the number of storeys.

(2) Columns which carry a vertical load N_{Sd} of less than 50 % of the mean value of the vertical load per column in the plane considered, shall not be included in n_c.

(3) Columns which do not extend through all the storeys included in n_s shall not be included in n_c. Those floor levels and roof levels which are not connected to all the columns included in n_c shall not be included when determining n_s.

NOTE Where more than one combination of n_c and n_s satisfies these conditions, any such combination can safely be used.

(4) These initial sway imperfections apply in all horizontal directions, but need only be considered in one direction at a time.

(5) The possible torsional effects on the structure of anti-symmetric sways, on two opposite faces, shall also be considered.

(6) *If more convenient, the initial sway imperfection may be replaced by a closed system of equivalent horizontal forces, see Figure 5.2.3.*

(7) *In beam-and-column building frames, these equivalent horizontal forces should be applied at each floor and roof level and should be proportionate to the vertical loads applied to the structure at that level, see Figure 5.2.4.*

(8) *The horizontal reactions at each support should be determined using the initial sway imperfection and not the equivalent horizontal forces. In the absence of actual horizontal loads, the net horizontal reaction is zero.*

5.2.4.4 ***Imperfections for analysis of bracing systems***

(1) The effects of imperfections shall be allowed for in the analysis of bracing systems which are required to provide lateral stability within the length of beams or compression members, by means of an equivalent geometric imperfection of the members to be restrained, in the form of an initial bow imperfection:

$$e_o = k_r L/500. \tag{5.3}$$

where L is the span of the bracing system

and $k_r = [0{,}2 + 1/n_r]^{0,5}$ but $k_r \leq 1{,}0$

in which n_r is the number of members to be restrained.

(2) *For convenience, the initial bow imperfections of the members to be restrained by a bracing system, may be replaced by the equivalent stabilizing force shown in Figure 5.2.5.*

(3) *Where the bracing system is required to stabilize a beam, the force N in Figure 5.2.5 should be obtained from:*

$$N = M/h \tag{5.4}$$

where M is the maximum moment in the beam

and h is the overall depth of the beam.

(4) At points where beams or compression members are spliced, it shall also be verified that the bracing system is able to resist a local force equal to $k_r N/100$ applied to it by each beam or compression member which is spliced at that point, and to transmit this force to the adjacent points at which that beam or compression member is restrained, see Figure 5.2.6.

(5) When checking for this local force, any external loads acting on the bracing system shall also be included, but the forces arising from the imperfection given in (1) may be omitted.

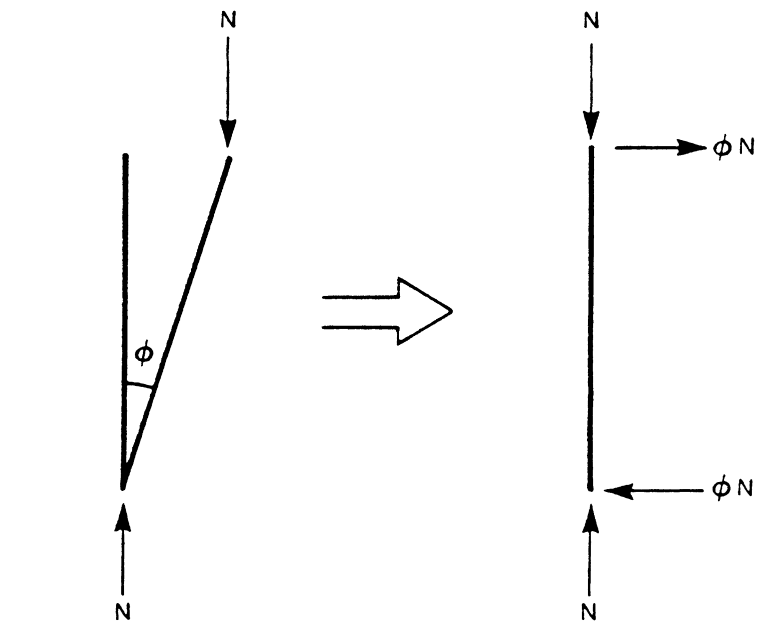


Figure 5.2.3 — Replacement of initial sway imperfections by equivalent horizontal forces

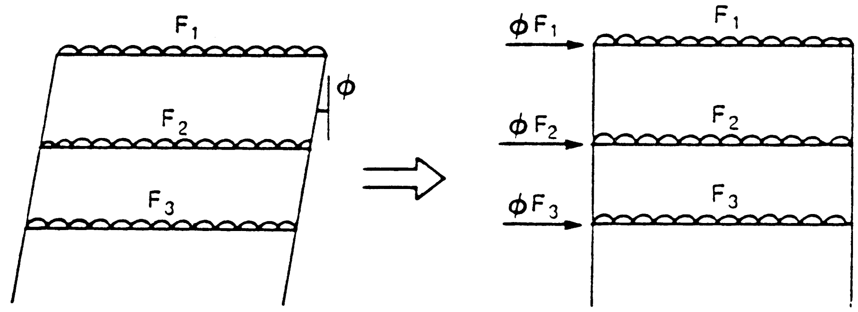


Figure 5.2.4 — Equivalent horizontal forces

5.2.4.5 ***Member imperfections***

(1) Normally the effects of imperfections on member design shall be incorporated by using the appropriate buckling formulae given in this Eurocode.

(2) Alternatively, for a compression member, the initial bow imperfection specified in **5.5.1.3** may be included in a second order analysis of the member.

(3) Where it is necessary (according to **5.2.4.2**) to allow for member imperfections in the global analysis, the imperfections specified in **5.5.1.3** shall be included and second order global analysis shall be used.

5.2.5 Sway stability

5.2.5.1 ***Sway stiffness***

(1) All structures shall have sufficient stiffness to limit lateral sway. This may be supplied by:

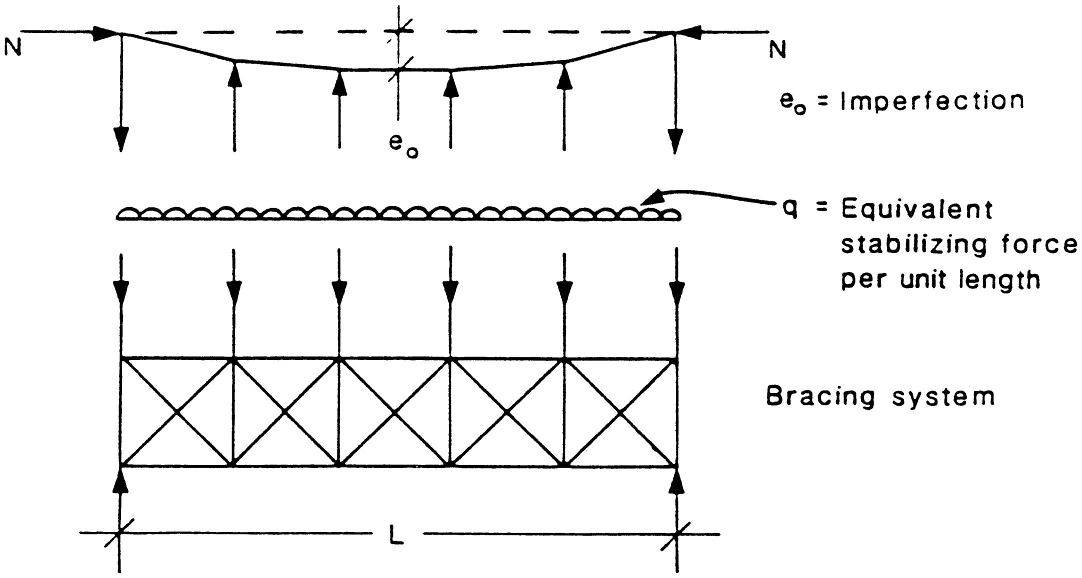
a) the sway stiffness of bracing systems, which may be:

- triangulated frames
- rigid-jointed frames
- shear walls, cores and the like

b) the sway stiffness of the frames, which may be supplied by one or more of the following:

- triangulation
- the stiffness of the connections
- cantilever columns

(2) Semi-rigid connections may be used, provided that they can be demonstrated to provide sufficient reliable rotational stiffness (see **6.9.4**) to satisfy the requirements for sway-mode frame stability, see **5.2.6**.



The force N is assumed uniform within the span L of the bracing system. For non-uniform force this is slightly conservative.

For a single restrained member:

for $\delta_q \leq \frac{L}{2500}$: $q = \frac{N}{50L}$

for $\delta_q > \frac{L}{2500}$: $q = \frac{N}{60L}[1 + \alpha]$

where δ_q is the in-plane deflection of the bracing system due to q plus any external loads

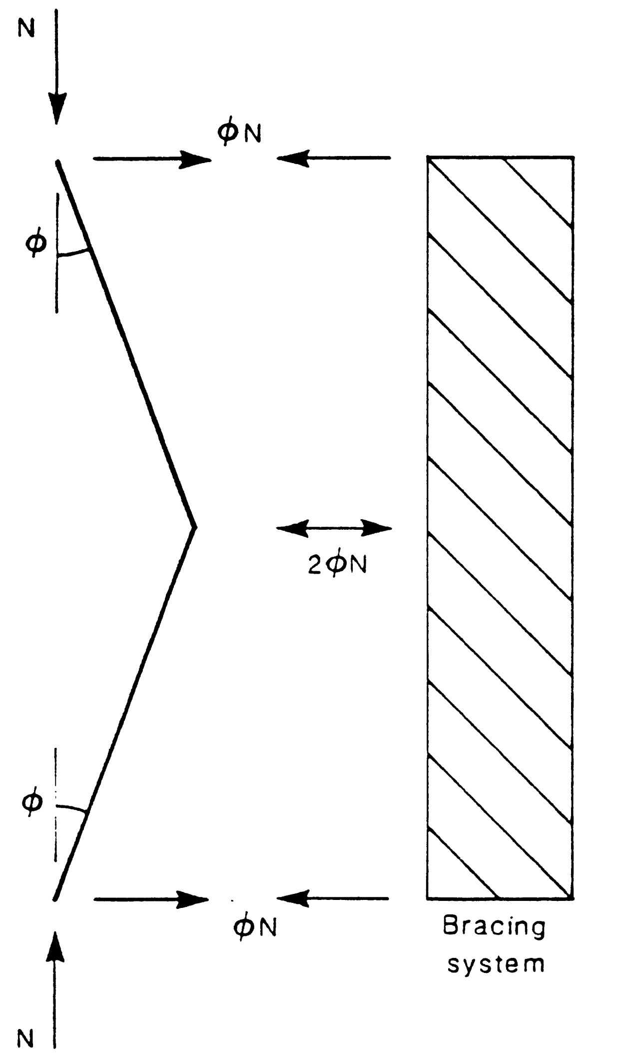
and $\alpha = 500\,\delta_q/L$ but $\alpha \geq 0{,}2$

For multiple restrained members:

for $\delta_q \leq \frac{L}{2500}$: $q = \frac{\Sigma N}{60L}[k_r + 0{,}2]$

for $\delta_q > \frac{L}{2500}$: $q = \frac{\Sigma N}{60L}[k_r + \alpha]$

Figure 5.2.5 — Equivalent stabilizing force



$\phi = k_r\, \phi_0 \quad : \quad \phi_0 = 1/200$

$2\phi N = k_r\, N/100$

Figure 5.2.6 — Bracing forces at splices in compression elements

5.2.5.2 ***Classification as sway or non-sway***

(1) A frame may be classified as non-sway if its response to in-plane horizontal forces is sufficiently stiff for it to be acceptably accurate to neglect any additional internal forces or moments arising from horizontal displacements of its nodes.

(2) Any other frame shall be classified as a sway frame and the effects of the horizontal displacements of its nodes taken into account in its design, see **5.2.1.2**.

(3) *A frame may be classified as non-sway for a given load case if the elastic critical load ratio V_{Sd}/V_{cr} for that load case satisfies the criterion:*

$$V_{Sd}/V_{cr} \leq 0,1 \tag{5.5}$$

where V_{Sd} *is the design value of the total vertical load*

and V_{cr} *is its elastic critical value for failure in a sway mode.*

(4) *Beam-and-column type plane frames in building structures with beams connecting each column at each storey level (see Figure 5.2.7) may be classified as non-sway for a given load case if the following criterion is satisfied. When first order theory is used, the horizontal displacements in each storey due to the design loads (both horizontal and vertical), plus the initial sway imperfection (see* **5.2.4.3***) applied in the form of equivalent horizontal forces, should satisfy the criterion:*

$$\left[\frac{\delta}{h}\right]\left[\frac{V}{H}\right] \leq 0,1 \tag{5.6}$$

where δ *is the horizontal displacement at the top of the storey, relative to the bottom of the storey*

h *is the storey height*

H *is the total horizontal reaction at the bottom of the storey*

and V *is the total vertical reaction at the bottom of the storey.*

(5) *For sway frames, the requirements for frame stability given in* **5.2.6** *should also be satisfied.*

5.2.5.3 *Classification as braced or unbraced*

(1) A frame may be classified as braced if its sway resistance is supplied by a bracing system with a response to in-plane horizontal loads which is sufficiently stiff for it to be acceptably accurate to assume that all horizontal loads are resisted by the bracing system.

(2) *A steel frame may be classified as braced if the bracing system reduces its horizontal displacements by at least 80 %.*

(3) A braced frame may be treated as fully supported laterally.

(4) The effects of the initial sway imperfections (see **5.2.4.3**) in the braced frame shall be taken into account in the design of the bracing system.

(5) *The initial sway imperfections (or the equivalent horizontal forces, see* **5.2.4.3***) plus any horizontal loads applied to a braced frame, may be treated as affecting only the bracing system.*

(6) *The bracing system should be designed to resist:*

- *any horizontal loads applied to the frames which it braces,*
- *any horizontal or vertical loads applied directly to the bracing system,*
- *the effects of the initial sway imperfections (or the equivalent horizontal forces) from the bracing system itself and from all the frames which it braces.*

(7) *Where the bracing system is a frame or sub-frame, it may itself be either sway or non-sway, see* **5.2.5.2***.*

(8) *When applying the criterion given in* **5.2.5.2***(3) to a frame or sub-frame acting as a bracing system, the total vertical load acting on all the frames which it braces should also be included.*

(9) *When applying the criterion given in* **5.2.5.2***(4) to a frame or sub-frame acting as a bracing system, the total horizontal and vertical load acting on all the frames which it braces should also be included, plus the initial sway imperfection applied in the form of the equivalent horizontal forces from the bracing system itself and from all the frames which it braces.*

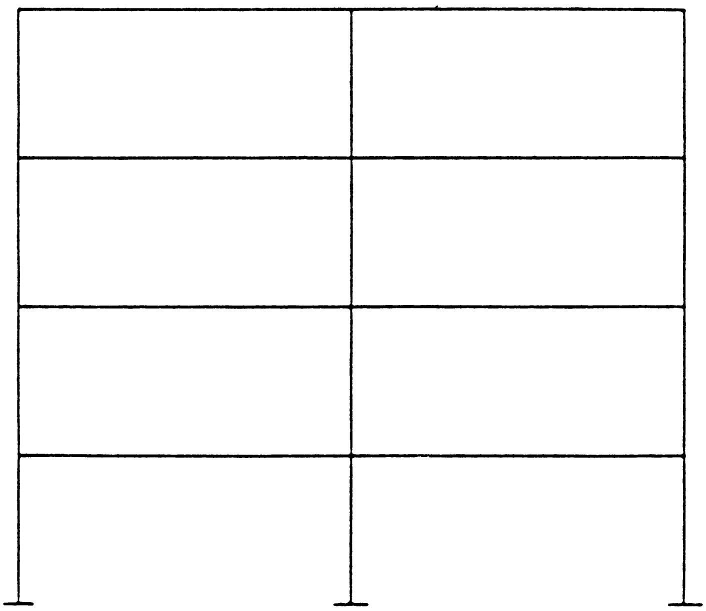


Figure 5.2.7 — Building frame with beams connecting each column at each storey level

5.2.6 Frame stability

5.2.6.1 *General*

(1) All frames shall have adequate resistance to failure in a sway mode. However, where the frame is shown to be a non-sway frame, see **5.2.5.2**, no further sway mode verification is required.

(2) All frames, including sway frames, shall also be checked for adequate resistance to failure in non-sway modes.

(3) *A check should be included for the possibility of local storey-height failure modes.*

(4) Frames with non-triangulated pitched roofs shall also be checked for snap-through buckling.

(5) The use of rigid-plastic analysis with plastic hinge locations in the columns shall be limited to cases where it can be demonstrated that the columns are able to form hinges with sufficient rotation capacity, see **5.2.7**.

5.2.6.2 *Elastic analysis of sway frames*

(1) When elastic global analysis is used, the second order effects in the sway mode shall be included, either directly by using second order elastic analysis, or indirectly by using one of the following alternatives:

a) first order elastic analysis, with amplified sway moments.

b) first order elastic analysis, with sway-mode buckling lengths.

(2) *When second order elastic global analysis is used, in-plane buckling lengths for the non-sway mode may be used for member design.*

(3) *In the amplified sway moments method, the sway moments found by a first order elastic analysis should be increased by multiplying them by the ratio:*

$$\frac{1}{1 - V_{Sd}/V_{cr}} \qquad (5.7)$$

where V_{Sd} *is the design value of the total vertical load*

and V_{cr} *is its elastic critical value for failure in a sway mode.*

(4) The amplified sway moments method should not be used when the elastic critical load ratio V_{Sd}/V_{cr} *is more than 0,25.*

(5) *Sway moments are those associated with the horizontal translation of the top of a storey relative to the bottom of that storey. They arise from horizontal loading and may also arise from vertical loading if either the structure or the loading is asymmetrical.*

(6) As an alternative to determining V_{Sd}/V_{cr} *direct the following approximation may be used in beam-and-column type frames as described in* ***5.2.5.2****(4):*

$$\frac{V_{Sd}}{V_{cr}} = \left(\frac{\delta}{h}\right)\left(\frac{V}{H}\right) \tag{5.8}$$

where δ, h, H and V are as defined ***5.2.5.2****(4).*

(7) *When the amplified sway moments method is used, in-plane buckling lengths for the non-sway mode may be used for member design.*

(8) *When first order elastic analysis, with sway-mode in-plane buckling lengths, is used for column design, the sway moments in the beams and the beam-to-column connections should be amplified by at least 1,2 unless a smaller value is shown to be adequate by analysis.*

5.2.6.3 *Plastic analysis of sway frames*

(1) When plastic global analysis is used, allowance shall be made for the second order effects in the sway mode.

(2) *This should generally be done by using second order elastic-plastic analysis, see* ***5.2.1.4****.*

(3) *However, as an alternative, rigid-plastic analysis with indirect allowance for second-order effects, as given in (4) below, may be adopted in the following cases:*

a) *Frames one or two storeys high in which either:*

- *no plastic hinge locations occur in the columns, or*
- *the columns satisfy* ***5.2.7****.*

b) *Frames with fixed bases, in which the sway failure mode involves plastic hinge locations in the columns at the fixed bases only, see Figure 5.2.8, and the design is based on an incomplete mechanism in which the columns are designed to remain elastic at the calculated plastic hinge moment.*

(4) In the cases given in (3), V_{Sd}/V_{cr} *should not exceed 0,20 and all the internal forces and moments should be amplified by the ratio given in* ***5.2.6.2****(3).*

(5) *In-plane buckling lengths for the non-sway mode may be used for member design. These should be determined with due allowance for the effects of plastic hinges.*

5.2.7 Column requirements for plastic analysis

(1) In frames it is necessary to ensure that where plastic hinges are required to form in members which are also under compression, adequate rotation capacity is available.

(2) *This criterion may be assumed to be satisfied when elastic-plastic global analysis is used, provided that the cross-sections satisfy the requirements given in* ***5.3.3****.*

(3) *When plastic hinge locations occur in the columns of frames designed using first order rigid-plastic analysis, the columns should satisfy the following:*

- *in braced frames:*

$$\bar{\lambda} \leq 0{,}40\,[Af_y/N_{Sd}]^{0,5} \tag{5.9}$$

- *in unbraced frames:*

$$\bar{\lambda} \leq 0{,}32\,[Af_y/N_{Sd}]^{0,5} \tag{5.10}$$

(4) *where $\bar{\lambda}$ is the in-plane non-dimensional slenderness (see* ***5.5.1.2****) calculated using a buckling length equal to the system length.*

(5) *In frames designed using first order rigid-plastic global analysis, columns containing plastic hinge locations should also be checked for resistance to in-plane buckling, using buckling lengths equal to their system lengths.*

(6) *Except for the method outlined in* ***5.2.6.3****(3) b), first order rigid-plastic global analysis should not be used for unbraced frames with more than two storeys.*

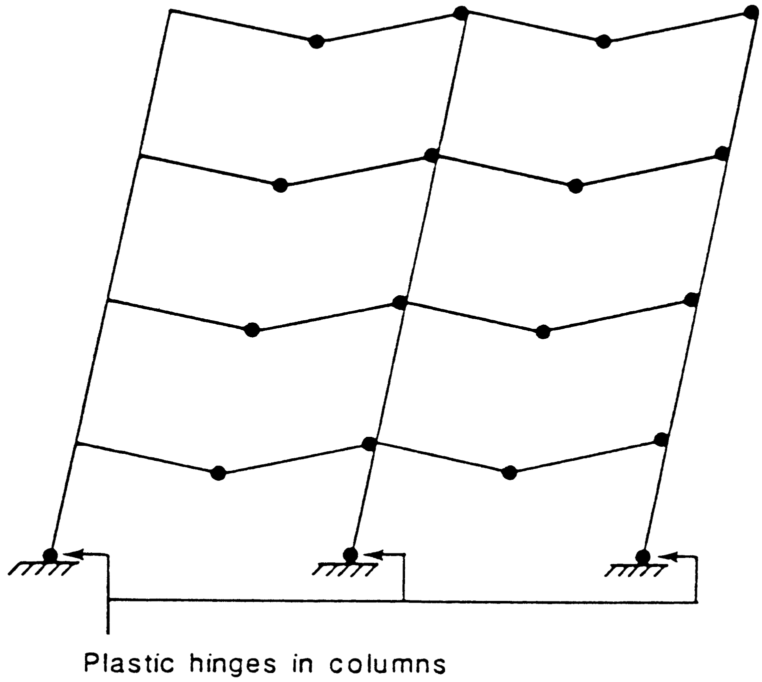


Figure 5.2.8 — Sway mechanism involving plastic hinges in columns at bases only

5.3 Classification of cross-sections

5.3.1 Basis

(1) When plastic global analysis is used, the members shall be capable of forming plastic hinges with sufficient rotation capacity to enable the required redistribution of bending moments to develop.

(2) When elastic global analysis is used, any class of cross-section may be used for the members, provided that the design of the members takes into account the possible limits on the resistance of cross-sections due to local buckling.

5.3.2 Classification

(1) Four classes of cross-sections are defined, as follows:

- Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required for plastic analysis.
- Class 2 cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity.
- Class 3 cross-sections are those in which the calculated stress in the extreme compression fibre of the steel member can reach its yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- Class 4 cross-sections are those in which it is necessary to make explicit allowances for the effects of local buckling when determining their moment resistance or compression resistance.

(2) Effective widths may be used in Class 4 cross-sections to make the necessary allowances for reductions in resistance due to the effects of local buckling, see **5.3.5**.

(3) The classification of a cross-section depends on the proportions of each of its compression elements.

(4) Compression elements include every element of a cross-section which is either totally or partially in compression, due to axial force or bending moment, under the load combination considered.

(5) The various compression elements in a cross-section (such as a web or a flange) can, in general, be in different classes.

(6) A cross-section is normally classified by quoting the highest (least favourable) class of its compression elements.

(7) Alternatively the classification of a cross-section may be defined by quoting both the flange classification and the web classification.

(8) *The limiting proportions for Class 1, 2, and 3 compression elements should be obtained from Table 5.3.1. An element which fails to satisfy the limits for Class 3 should be taken as Class 4.*

5.3.3 Cross-section requirements for plastic global analysis

(1) At plastic hinge locations, the cross-section of the member which contains the plastic hinge shall have an axis of symmetry in the plane of loading.

(2) At plastic hinge locations, the cross-section of the member which contains the plastic hinge shall have a rotation capacity of not less than the required rotation at that plastic hinge location.

(3) *To satisfy the above requirement, the required rotations should be determined from a rotation analysis.*

(4) For building structures in which the required rotations are not calculated, all members containing plastic hinges shall have Class 1 cross-sections at the plastic hinge location.

(5) *Where the cross-sections of the members vary along their length, the following additional criteria should be satisfied:*

a) *Adjacent to plastic hinge locations, the thickness of the web should not be reduced for a distance along the beam from the plastic hinge location of at least 2d, where d is the clear depth of the web at the plastic hinge location.*

b) *Adjacent to plastic hinge locations, the compression flange should be Class 1 for a distance along the beam from the plastic hinge location of not less than the greater of:*

- *2d, where d is as defined in a)*
- *the distance to the point at which the moment in the beam has fallen to 0,8 times the plastic moment resistance at the point concerned.*

c) *Elsewhere the compression flange should be Class 1 or Class 2 and the web should be Class 1, Class 2 or Class 3.*

5.3.4 Cross-section requirements when elastic global analysis is used

(1) When elastic global analysis is used, the role of cross-section classification is to identify the extent to which the resistance of a cross-section is limited by its local buckling resistance.

(2) When all the compression elements of a cross-section are Class 2, the cross-section may be taken as capable of developing its full plastic resistance moment.

(3) When all the compression elements of a cross-section are Class 3, its resistance may be based on an elastic distribution of stresses across the cross-section, limited to the yield strength at the extreme fibres.

(4) *When yielding first occurs on the tension side of the neutral axis, the plastic reserves of the tension zone may be utilised when determining the resistance of a Class 3 cross-section, using the method given in ENV 1993-1-3 Eurocode 3-1.3*[14)].

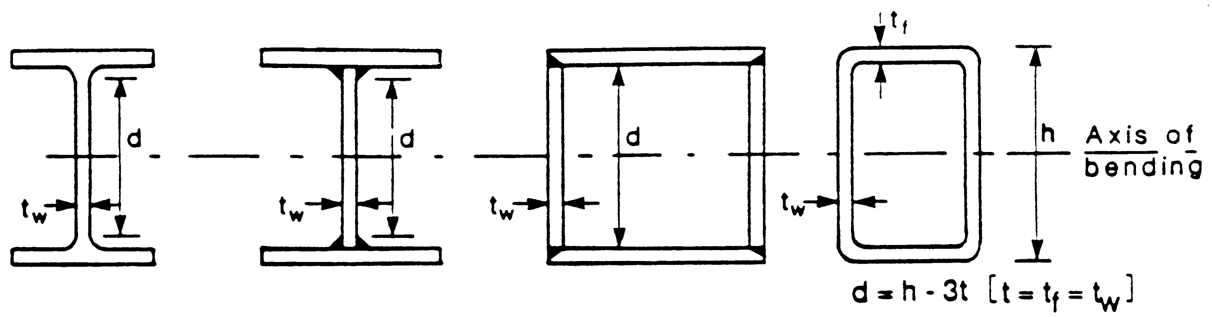
(5) *The resistance of a cross-section with a Class 2 compression flange but a Class 3 web may alternatively be determined by treating the web as an effective Class 2 web with a reduced effective area, using the method given in ENV 1994-1-1 Eurocode 4-1.1*[14)].

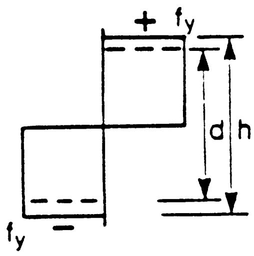
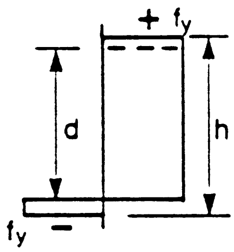
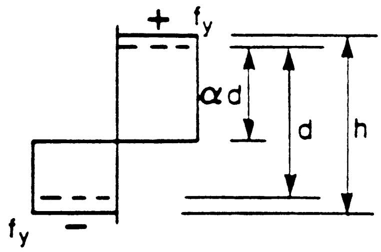
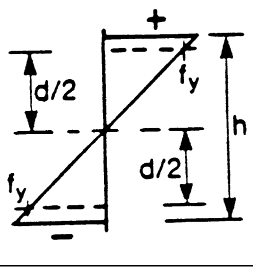
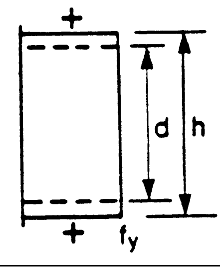
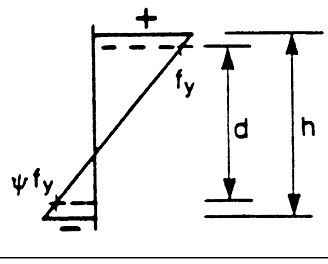
(6) When any of the compression elements of a cross-section is Class 4 the cross-section shall be designed as a Class 4 cross-section, see **5.3.5**.

[14)] In preparation.

Table 5.3.1 — Maximum width-to-thickness ratios for compression elements (Sheet 1)

a) Webs: (internal elements perpendicular to axis of bending)

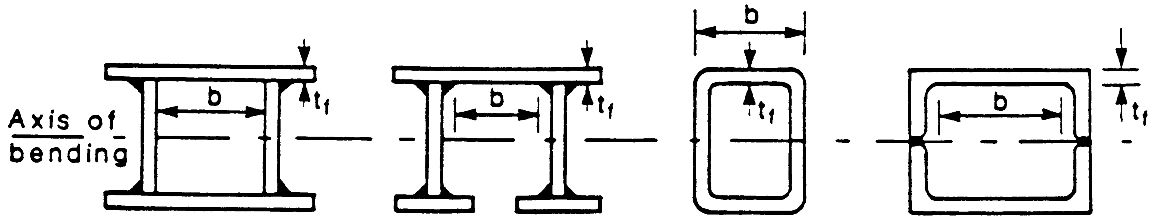


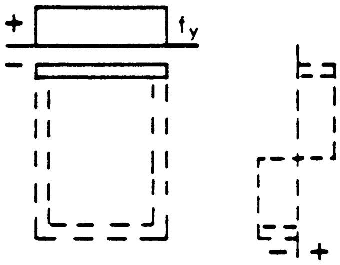
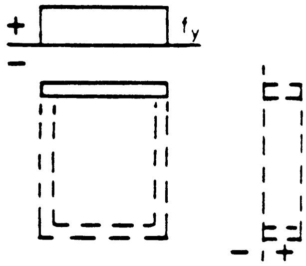
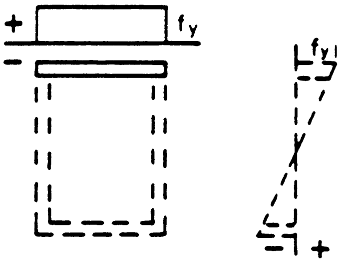
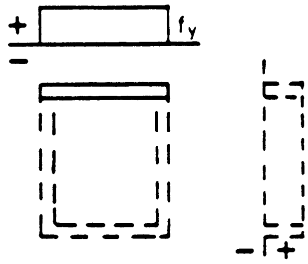
Class	Web subject to bending	Web subject to compression	Web subject to bending and compression
Stress distribution in element (compression positive)			
1	$d/t_W \leq 72\varepsilon$	$d/t_W \leq 33\varepsilon$	when $\alpha > 0{,}5$: $d/t_W \leq 396\varepsilon/(13\alpha - 1)$ when $\alpha \leq 0{,}5$: $d/t_W \leq 36\varepsilon/\alpha$
2	$d/t_W \leq 83\varepsilon$	$d/t_W \leq 38\varepsilon$	when $\alpha > 0{,}5$: $d/t_W \leq 456\varepsilon/(13\alpha - 1)$ when $\alpha \leq 0{,}5$: $d/t_W \leq 41{,}5\varepsilon/\alpha$
Stress distribution in element (compression positive)			
3	$d/t_W \leq 124\varepsilon$	$d/t_W \leq 42\varepsilon$	when $\psi > -1$: $d/t_W \leq 42\varepsilon/(0{,}67 + 0{,}33\psi)$ when $\psi \leq -1$: $d/t_W \leq 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355
	ε	1	0,92	0,81

Table 5.3.1 — Maximum width-to-thickness ratios for compression elements (Sheet 2)

b) <u>Internal flange elements:</u> (internal elements parallel to axis of bending)

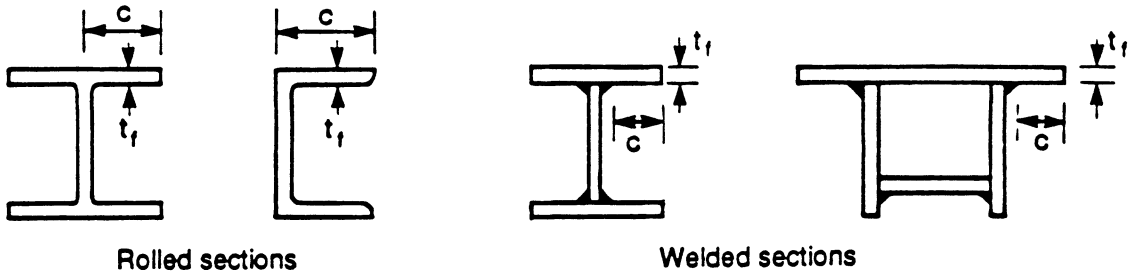


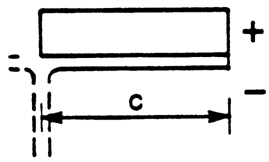
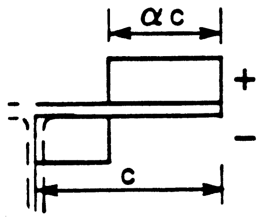
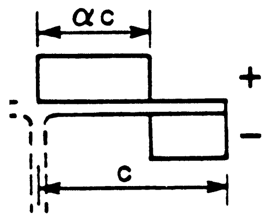
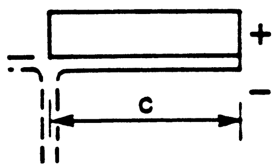
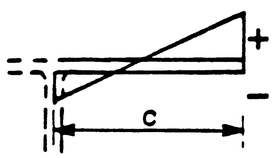
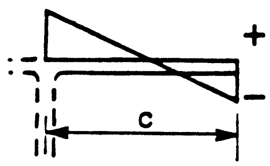
Class	Type	Section in bending	Section in compression
Stress distribution in element and across section (compression positive)			
1	Rolled Hollow Section Other	$(b-3t_f)/t_f \leq 33\varepsilon$ $b/t_f \leq 33\varepsilon$	$(b-3t_f)/t_f \leq 42\varepsilon$ $b/t_f \leq 42\varepsilon$
2	Rolled Hollow Section Other	$(b-3t_f)/t_f \leq 38\varepsilon$ $b/t_f \leq 38\varepsilon$	$(b-3t_f)/t_f \leq 42\varepsilon$ $b/t_f \leq 42\varepsilon$
Stress distribution in element and across section (compression positive)			
3	Rolled Hollow Section Other	$(b-3t_f)/t_f \leq 42\varepsilon$ $b/t_f \leq 42\varepsilon$	$(b-3t_f)/t_f \leq 42\varepsilon$ $b/t_f \leq 42\varepsilon$

$\varepsilon = \sqrt{235/f_y}$	f_Y	235	275	355
	ε	1	0,92	0,81

Table 5.3.1 — Maximum width-to-thickness ratios for compression elements (Sheet 3)

c) Outstand flanges:



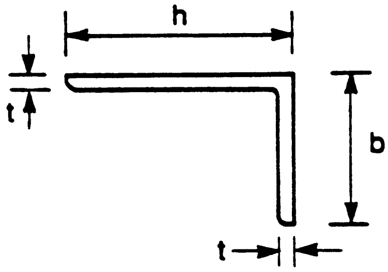
Class	Type of section	Flange subject to compression	Flange subject to compression and bending: Tip in compression	Flange subject to compression and bending: Tip in tension
Stress distribution in element (compression positive)				
1	Rolled	$c/t_f \le 10\varepsilon$	$c/t_f \le \frac{10\varepsilon}{\alpha}$	$c/t_f \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
	Welded	$c/t_f \le 9\varepsilon$	$c/t_f \le \frac{9\varepsilon}{\alpha}$	$c/t_f \le \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	Rolled	$c/t_f \le 11\varepsilon$	$c/t_f \le \frac{11\varepsilon}{\alpha}$	$c/t_f \le \frac{11\varepsilon}{\alpha\sqrt{\alpha}}$
	Welded	$c/t_f \le 10\varepsilon$	$c/t_f \le \frac{10\varepsilon}{\alpha}$	$c/t_f \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in element (compression positive)				
3	Rolled	$c/t_f \le 15\varepsilon$	$c/t_f \le 23\varepsilon\sqrt{k_\sigma}$	
	Welded	$c/t_f \le 14\varepsilon$	$c/t_f \le 21\varepsilon\sqrt{k_\sigma}$ For k_σ see Table 5.3.3	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355
	ε	1	0,92	0,81

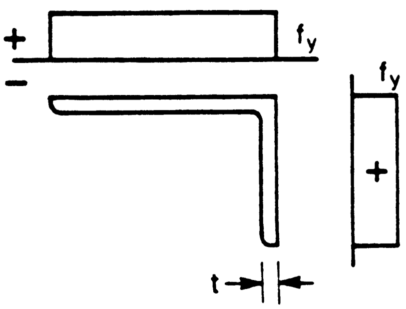
Table 5.3.1 — Maximum width-to-thickness ratios for compression elements (Sheet 4)

d) Angles:

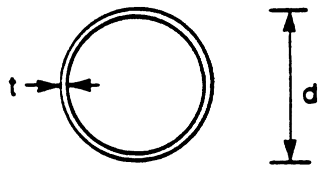
Refer also to c) "Outstand flanges" (see Sheet 3).



(Does not apply to angles in continuous contact with other components)

Class	Section in compression
Stress distribution across section (compression positive)	
3	$\frac{h}{t} \le 15\varepsilon$: $\frac{b+h}{2t} \le 11{,}5\varepsilon$

e) Tubular sections:



Class	Section in bending and/or compression
1	$d/t \le 50\varepsilon^2$
2	$d/t \le 70\varepsilon^2$
3	$d/t \le 90\varepsilon^2$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355
	ε	1	0,92	0,81
	ε^2	1	0,85	0,66

5.3.5 Effective cross-section properties of Class 4 cross-sections

(1) The effective cross-section properties of Class 4 cross-sections shall be based on the effective widths of the compression elements [see **5.3.2**(2)].

(2) *The effective widths of flat compression elements should be obtained using Table 5.3.2 for internal elements and Table 5.3.3 for outstand elements.*

(3) *As an approximation, the reduction factor ρ may be obtained as follows:*

- *when* $\bar{\lambda}_p \le 0{,}673$: $\rho = 1$
- *when* $\bar{\lambda}_p > 0{,}673$: $\rho = (\bar{\lambda}_p - 0{,}22)/\bar{\lambda}_p^{\,2}$ *(5.11)*

where $\overline{\lambda_p}$ *is the plate slenderness given by:*

$$\overline{\lambda}_p = [f_y / \sigma_{cr}]^{0,5} = (\bar{b}/t) / (28,4\, \epsilon \sqrt{k_\sigma})$$

in which t *is the relevant thickness*

σ_{cr} *is the critical plate-buckling stress*

$k\sigma$ *is the buckling factor corresponding to the stress ratio* ψ *from Table 5.3.2 or Table 5.3.3 as appropriate*

and $\bar{b}$ *is the appropriate width (see Table 5.3.1) as follows:*

$\bar{b}$ *= d for webs*

$\bar{b}$ *= b for internal flange elements (except RHS)*

$\bar{b}$ *= b – 3t for flanges of RHS*

$\bar{b}$ *= c for outstand flanges*

$\bar{b}$ *= (b + h)/2 for equal-leg angles*

$\bar{b}$ *= h or (b + h)/2 for unequal-leg angles*

(4) *To determine the effective widths of flange elements, the stress ratio* ψ *used in Table 5.3.2 or Table 5.3.3 may be based on the properties of the gross cross-section.*

(5) *To determine the effective width of a web, the stress ratio* ψ *used in Table 5.3.2 may be obtained using the effective area of the compression flange but the gross area of the web.*

(6) *Generally the centroidal axis of the effective cross-section will shift by a dimension e compared to the centroidal axis of the gross cross-section, see Figure 5.3.1 and Figure 5.3.2. This should be taken into account when calculating the properties of the effective cross-section.*

(7) *When the cross-section is subject to an axial force, the method given in* ***5.4.8.3*** *should be used to take account of the additional moment* ΔM *given by:*

$$\Delta M = N\, e_N \qquad (5.12)$$

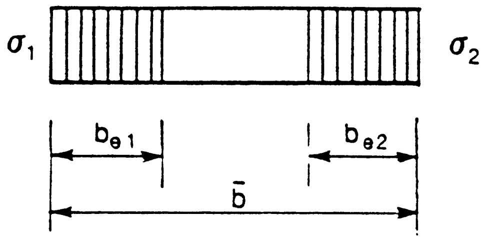
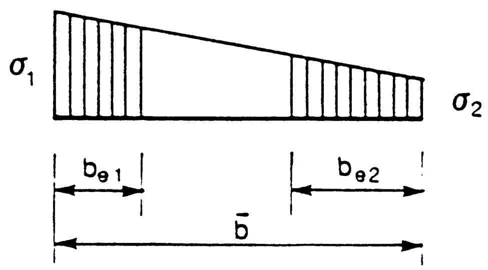
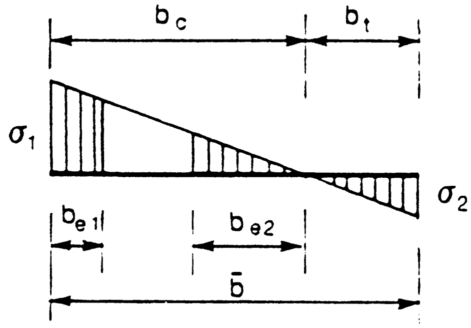
where e_N *is the shift of the centroidal axis when the effective cross-section is subject to uniform compression, see Figure 5.3.1.*

and N is positive for compression.

(8) *Except as given in (9), for greater economy the plate slenderness* $\overline{\lambda_p}$ *of an element may be determined using the maximum calculated compressive stress* $\sigma_{com.Ed}$ *in that element in place of the yield strength* f_y*, provided that* $\sigma_{com.Ed}$ *is determined using the effective widths* b_{eff} *of all the compression elements. This procedure generally requires an iterative calculation in which* ψ *is determined again at each step from the stresses calculated on the effective cross-section defined at the end of the previous step, including the stresses from the additional moment* ΔM.

(9) *However, when verifying the design buckling resistance of a member using section* ***5.5****, the plate slenderness* $\overline{\lambda_p}$ *of an element should always be based on its yield strength* f_y *when calculating the values of* A_{eff}, e_N *and* W_{eff}.

Table 5.3.2 — Internal compression elements

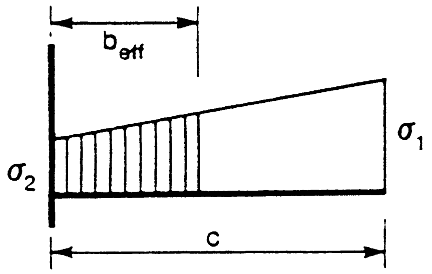
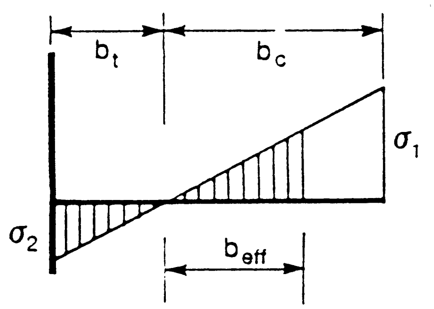
Stress distribution (compression positive)	**Effective width b_{eff}**
	<u>$\psi = 1$:</u> $b_{eff} = \rho \bar{b}$ $b_{e1} = 0.5b_{eff}$ $b_{e2} = 0,5b_{eff}$
	<u>$1 > \psi \geq 0$:</u> $b_{eff} = \rho \bar{b}$ $b_{e1} = \frac{2b_{eff}}{5 - \psi}$ $b_{e2} = b_{eff} - b_{e1}$
	<u>$\psi < 0$:</u> $b_{eff} = \rho\, b_c = \rho\ \bar{b}/(1 - \psi)$ $b_{e1} = 0,4b_{eff}$ $b_{e2} = 0,6b_{eff}$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -\varkappa$	-1	$-1 > \psi > -\ 2$
Buckling factor $k\sigma$	4,0	$\frac{8,2}{1,05 + \psi}$	7,81	$7,81 - 6,29\,\psi\ + 9,78\,\psi^2$	23,9	$5,98(1 - \psi)^2$

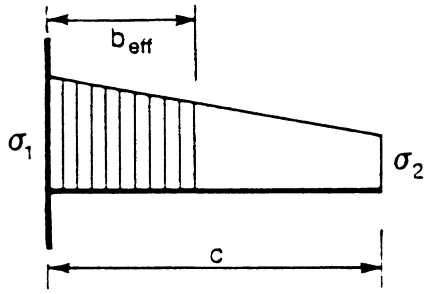
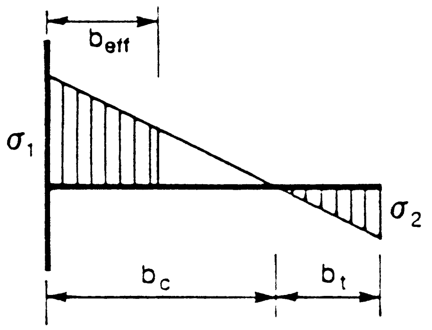
Alternatively, for $1 \geq \psi \geq -1$:

$$k_\sigma = \frac{16}{\left[(1+\psi)^2 + 0,112(1-\psi)^2\right]^{0,5} + (1+\psi)}$$

Table 5.3.3 — Outstanding compression elements

Stress distribution (compression positive)	Effective width b_{eff}
	<u>$1 > \psi \geq 0$:</u> $b_{eff} = \rho c$
	<u>$\psi < 0$:</u> $b_{eff} = \rho b_c = \rho c/(1 - \psi)$

$\psi = \sigma_2/\sigma_1$	1	0	– 1	$1 \geq \psi \geq -1$
Buckling factor k_σ	0,43	0,57	0,85	$0{,}57 - 0{,}21\,\psi + 0{,}07\,\psi^2$

	<u>$1 > \psi \geq 0$:</u> $b_{eff} = \rho c$
	<u>$\psi < 0$:</u> $b_{eff} = \rho b_c = \rho c/(1 - \psi)$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi\ \psi - 1$	– 1
Buckling factor k_σ	0,43	$\dfrac{0{,}578}{\psi + 0{,}34}$	1,70	$1{,}7 - 5\,\psi + 17{,}1\,\psi^2$	23,8

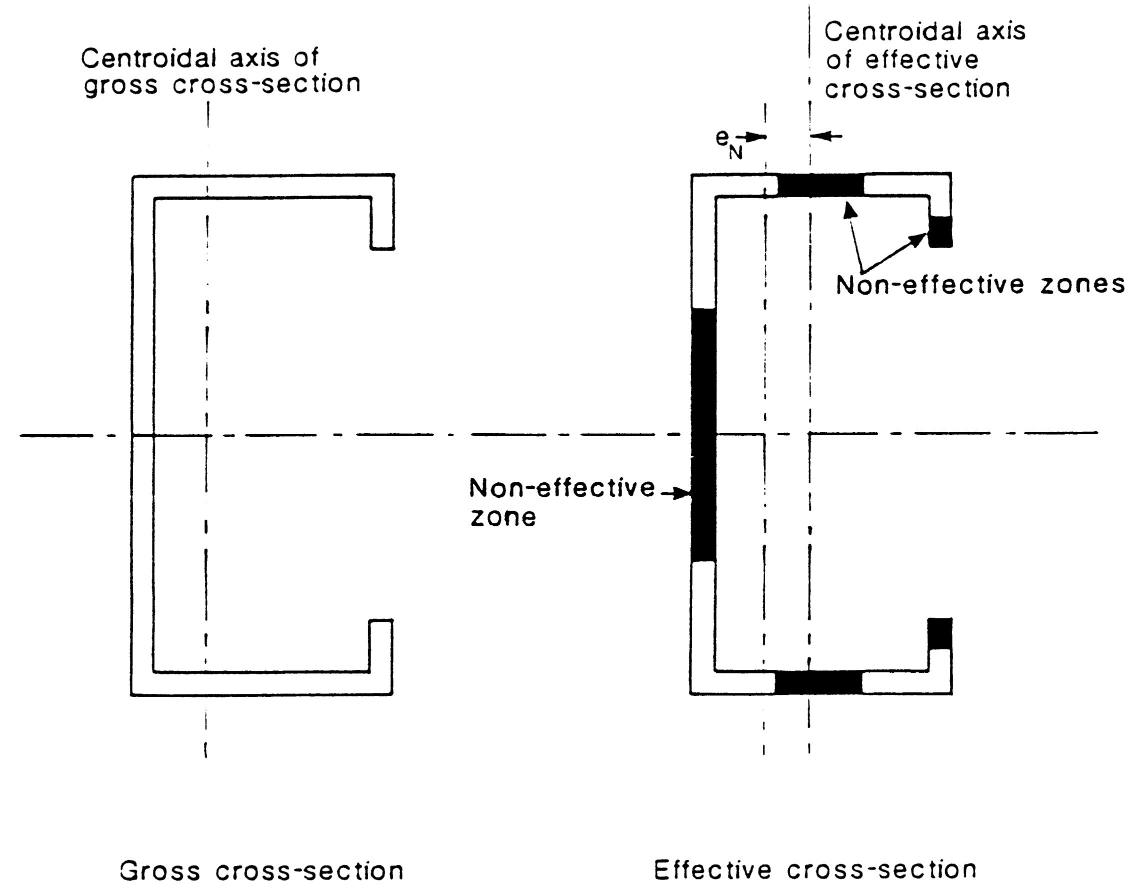


Figure 5.3.1 — Class 4 cross-sections — axial force

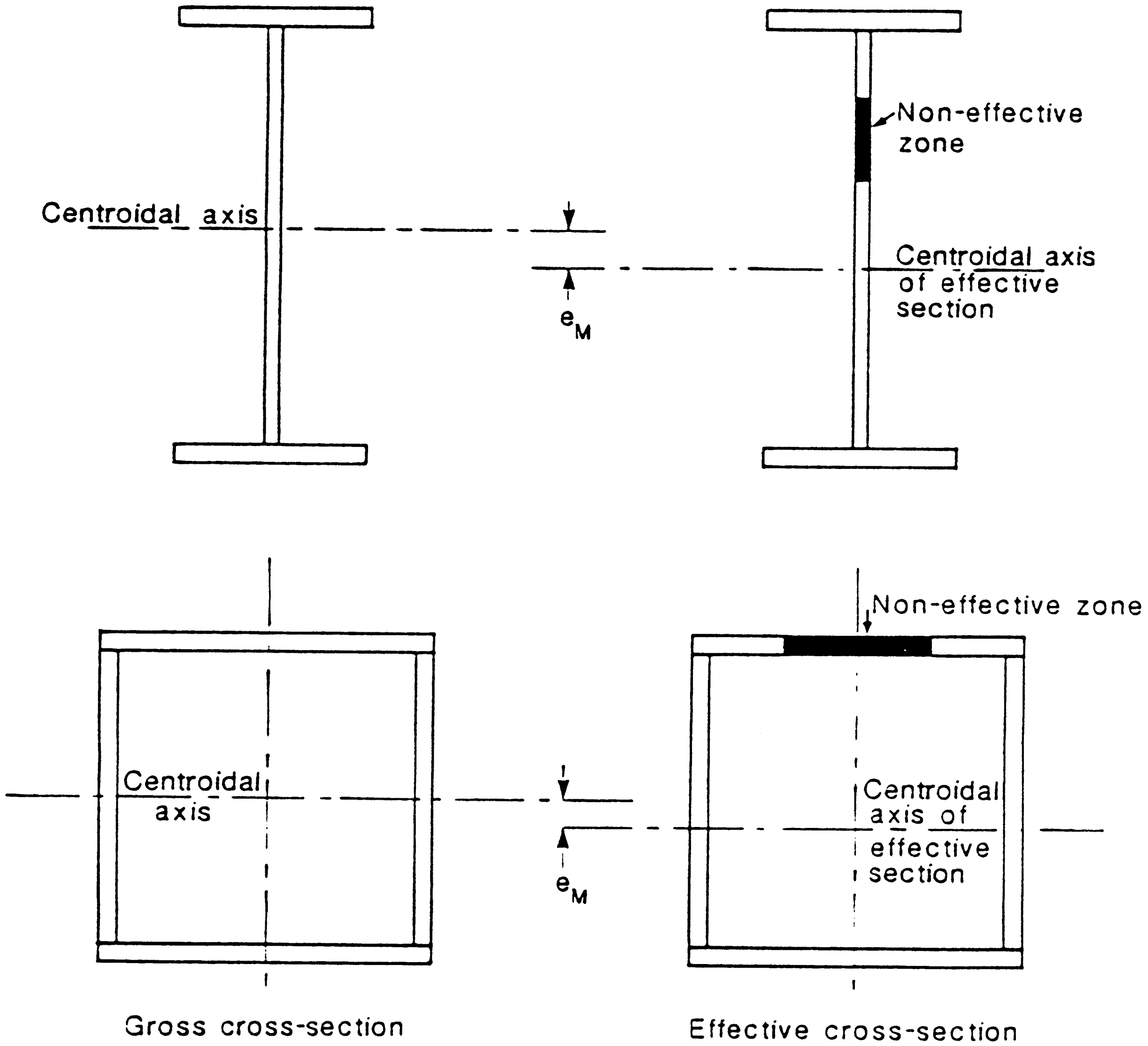


Figure 5.3.2 — Class 4 cross-sections — bending moment

5.3.6 Effects of transverse forces on webs

(1) The effects of significant transverse compressive stresses on the local buckling resistance of a web shall be taken into account in design. Such stresses may arise from transverse forces on a member and at member intersections.

(2) *The presence of significant transverse compressive stresses may effectively reduce the maximum values of the depth-to-thickness ratios d/t_w for Class 1, Class 2 and Class 3 webs below those given in Table 5.3.1, depending on the spacing of any web stiffeners.*

(3) *A recognised method of verification should be used. Reference may be made to the application rules for stiffened plating given in ENV 1993-2 Eurocode 3-2*[15].

5.4 Resistance of cross-sections

5.4.1 General

(1) This clause covers the resistance of member cross-sections, which may be limited by:

- the plastic resistance of the gross cross-section

[15] In preparation.

- the resistance of the net section at holes for fasteners
- shear lag effects
- local buckling resistance
- shear buckling resistance.

(2) *The plastic resistance of a cross-section may be verified by finding a stress distribution which equilibrates the internal forces and moments without exceeding the yield strength, provided that this stress distribution is feasible, considering the associated plastic deformations.*

(3) In addition to the requirements given in this clause, the buckling resistance of the member shall also be verified, see **5.5**.

(4) *Where appropriate, frame stability should also be verified, see **5.2.1.2** and **5.2.6**.*

5.4.2 Section properties

5.4.2.1 *Gross cross-section*

(1) The properties of the gross cross-section shall be determined using the specified dimensions. Holes for fasteners need not be deducted, but allowance shall be made for larger openings. Splice materials and battens shall not be included.

5.4.2.2 *Net area*

(1) The net area of a member or element cross-section shall be taken as its gross area less appropriate deductions for all holes and other openings.

(2) When calculating net section properties, the deduction for a single fastener hole shall be the gross cross-sectional area of the hole in the plane of its axis. For countersunk holes, appropriate allowance shall be made for the countersunk portion.

(3) Provided that the fastener holes are not staggered, the total area to be deducted for fastener holes shall be the maximum sum of the sectional areas of the holes in any cross-section perpendicular to the member axis.

(4) When the fastener holes are staggered, the total area to be deducted for fastener holes shall be the greater of:

a) the deduction for non-staggered holes given in (3)

b) the sum of the sectional areas of all holes in any diagonal or zig-zag line extending progressively across the member or part of the member, less $s^2t/(4p)$ for each gauge space in the chain of holes, see Figure 5.4.1

where s is the staggered pitch, the spacing of the centres of two consecutive holes in the chain measured parallel to the member axis.

p is the spacing of the centres of the same two holes measured perpendicular to the member axis.

and t is the thickness.

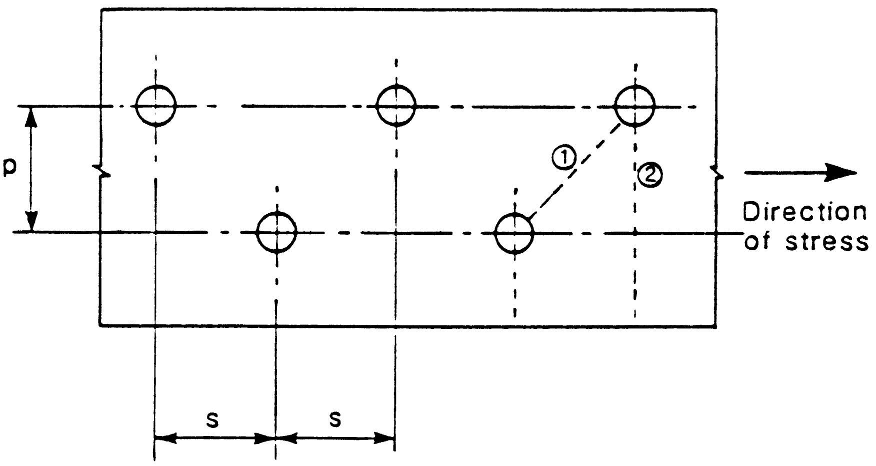


Figure 5.4.1 — Staggered holes

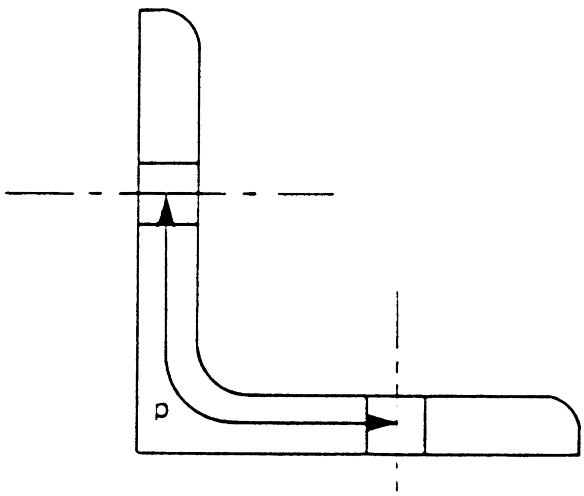


Figure 5.4.2 — Angles with holes in both legs

(5) In an angle or other member with holes in more than one plane, the spacing p shall be measured along the centre of thickness of the material, see Figure 5.4.2.

5.4.2.3 *Shear lag effects*

(1) *Shear lag effects in flanges may be neglected provided that:*

- *for outstand elements:* $c \leq L_o/20$
- *for internal elements:* $b \leq L_o/10$

where L_o *is the length between points of zero moment.*

b *is the breadth*

and c *is the outstand*

(2) *When these limits are exceeded an effective breadth of flange should be taken.*

(3) *The calculation of effective breadths of flanges is covered in ENV 1993-1-3 Eurocode 3-1.3*[16] *and ENV 1993-2 Eurocode 3-2*[16].

[16] In preparation.

5.4.3 Tension

(1) For members in axial tension, the design value of the tensile force N_{sd} at each cross-section shall satisfy:

$$N_{Sd} \leq N_{t.Rd} \tag{5.13}$$

where $N_{t.Rd}$ is the design tension resistance of the cross-section, taken as the smaller of:

a) the design plastic resistance of the gross cross-section

$$N_{p\ell.Rd} = Af_y/\gamma_{M0}$$

b) the design ultimate resistance of the net cross-section at holes for fasteners

$$N_{u.Rd} = 0{,}9\ A_{net}\ f_u/\gamma_{M2}$$

(2) In Category C connections designed to be slip-resistant at the ultimate limit state (see **6.5.3.1**), the design plastic resistance of the net section at holes for fasteners $N_{net.Rd}$ shall not be taken as more than:

$$N_{net.Rd} = A_{net}\ f_y/\gamma_{M0} \tag{5.14}$$

(3) *For angles connected through one leg, see also **6.5.2.3** and **6.6.10**. Similar consideration should also be given to other types of sections connected through outstands such as T-sections and channels.*

(4) Where ductile behaviour is required, the design plastic resistance $N_{p\ell.Rd}$ shall be less than the design ultimate resistance of the net section at fastener holes $N_{u.Rd}$, that is:

$$N_{u.Rd} \geq N_{p\ell.Rd} \tag{5.15}$$

This will be satisfied if:

$$0{,}9[A_{net}/A] \geq [f_y/f_u]\ [\gamma_{M2}/\gamma_{M0}]$$

5.4.4 Compression

(1) For members in axial compression, the design value of the compressive force N_{Sd} at each cross-section shall satisfy:

$$N_{Sd} \leq N_{c.Rd} \tag{5.16}$$

where $N_{c.Rd}$ is the design compression resistance of the cross-section, taken as the smaller of:

a) the design plastic resistance of the gross section

$$N_{pl.Rd} = Af_y/\gamma_{M0}$$

b) the design local buckling resistance of the gross section

$$N_{o.Rd} = A_{eff}f_y/\gamma_{M1}$$

where A_{eff} is the effective area of the cross-section, see **5.3.5**.

(2) The design compression resistance of the cross-section $N_{c.Rd}$ may be determined as follows:

Class 1, 2 or 3 cross-sections:	$N_{c.Rd} = Af_y/\gamma_{M0}$
Class 4 cross-sections:	$N_{c.Rd} = A_{eff}f_y/\gamma_{M1}$

(3) *In the case of unsymmetrical Class 4 sections, the method given in **5.4.8.3** should be used to allow for the additional moment ΔM due to the eccentricity of the centroidal axis of the effective section, see **5.3.5(7)**.*

(4) In addition, the buckling resistance of the member shall also be verified, see **5.5.1**.

(5) Fastener holes need not be allowed for in compression members, except for oversize and slotted holes.

5.4.5 Bending moment

5.4.5.1 *Basis*

(1) In the absence of shear force, the design value of the bending moment M_{Sd} at each cross-section shall satisfy:

$$M_{Sd} \leq M_{c.Rd} \tag{5.17}$$

where $M_{c.Rd}$ is the design moment resistance of the cross-section, taken as the smallest of:

a) the design plastic resistance moment of the gross section

$$M_{p\ell.Rd} = W_{p\ell}f_y/\gamma_{M0}$$

b) the design local buckling resistance moment of the gross section

$M_{o.Rd} = W_{eff} f_y / \gamma_{M1}$

where W_{eff} is the effective section modulus (see **5.3.5**).

c) the design ultimate resistance moment of the net section at bolt holes $M_{u.Rd}$, see **5.4.5.3**.

(2) For a Class 3 cross-section the design moment resistance of the gross section shall be taken as the design elastic resistance moment given by:

$$M_{e\ell.Rd} = W_{e\ell} f_y / \gamma_{M0} \tag{5.18}$$

(3) Refer to **5.4.7** for combinations of bending moment and shear force.

(4) In addition, the resistance of the member to lateral-torsional buckling shall also be verified, see **5.5.2**.

5.4.5.2 ***Bending about one axis***

1) In the absence of shear force, the design moment resistance of a cross-section without holes for fasteners may be determined as follows:

Class 1 or 2 cross-sections: $M_{c.Rd} = W_{p\ell} f_y / \gamma_{M0}$

Class 3 cross-sections: $M_{c.Rd} = W_{e\ell} f_y / \gamma_{M0}$

Class 4 cross-sections: $M_{c.Rd} = W_{eff} f_y / \gamma_{M1}$

5.4.5.3 ***Holes for fasteners***

(1) Fastener holes in the tension flange need not be allowed for, provided that for the tension flange:

$$0{,}9\ [A_{f.net}/A_f] \geq [f_y/f_u]\ [\gamma_{M2}/\gamma_{M0}] \tag{5.19}$$

(2) When $A_{f.net}/A_f$ is less than this limit, a reduced flange area may be assumed which satisfies the limit.

(3) Fastener holes in the tension zone of the web need not be allowed for, provided that the limit given in (1) is satisfied for the complete tension zone comprising the tension flange plus the tension zone of the web.

(4) Fastener holes in the compression zone of the cross-section need not be allowed for, except for oversize and slotted holes.

5.4.5.4 ***Bi-axial bending***

(1) For bending about both axes, the methods given in **5.4.8** shall be used.

5.4.6 Shear

(1) The design value of the shear force V_{Sd} at each cross-section shall satisfy:

$$V_{Sd} \leq V_{p\ell.Rd} \tag{5.20}$$

where $V_{p\ell.Rd}$ is the design plastic shear resistance given by:

$V_{p\ell.Rd} = A_v\ (f_y/\sqrt{3})/\gamma_{M0}$

where A_v is the shear area.

(2) The shear area A_v may be taken as follows:

a) rolled I and H sections, load parallel to web	$A - 2bt_f + (t_w + 2r)t_f$
b) rolled channel sections, load parallel to web	$A - 2bt_f + (t_w + r)t_f$
c) welded I, H and box sections, load parallel to web	$\Sigma(dt_w)$
d) welded I, H, channel and box sections, load parallel to flanges	$A - \Sigma(dt_w)$
e) rolled rectangular hollow sections of uniform thickness:	
load parallel to depth	$Ah/(b + h)$
load parallel to breadth	$Ab/(b + h)$
f) circular hollow sections and tubes of uniform thickness	$2A/\pi$
g) plates and solid bars	A

where
- A is the cross-section area
- b is the overall breadth
- d is the depth of the web
- h is the overall depth

	r	is the root radius
	t_f	is the flange thickness
and	t_w	is the web thickness

(3) *For other cases A_v should be determined analogously.*

(4) *For simplicity, the value of A_v for a rolled I, H or channel section, load parallel to web, may be taken as $1,04ht_w$.*

(5) *In appropriate cases the formulae in (2) may be applied to components of a built-up section.*

(6) *If the web thickness is not constant, t_w should be taken as the minimum thickness.*

(7) In addition the shear buckling resistance shall also be verified as specified in **5.6** when:

- for an unstiffened web:

$d/t_w > 69\varepsilon$

- for a stiffened web:

$$d/t_w > 30\varepsilon\sqrt{k_\tau}$$

where k_τ is the buckling factor for shear, see **5.6.3**

and $\varepsilon = [235/f_y]^{0,5}$ (f_y in N/mm²)

(8) Fastener holes need not be allowed for in shear verifications provided that:

$$A_{v.net} \geq (f_y/f_u)\, A_v \tag{5.21}$$

(9) When $A_{v.net}$ is less than this limit, an effective shear area of $(f_u/f_y)\, A_{v.net}$ may be assumed.

The block shear criterion given in **6.5.2.2** shall also be verified at the ends of a member.

5.4.7 Bending and shear

(1) The theoretical plastic resistance moment of a cross-section is reduced by the presence of shear. For small values of the shear force this reduction is so small that it is counter-balanced by strain hardening and may be neglected. However, when the shear force exceeds half the plastic shear resistance, allowance shall be made for its effect on the plastic resistance moment.

(2) *Provided that the design value of the shear force V_{Sd} does not exceed 50 % of the design plastic shear resistance $V_{p\ell.Rd}$ no reduction need be made in the resistance moments given by **5.4.5.2**.*

(3) *When V_{Sd} exceeds 50 % of $V_{p\ell.Rd}$ the design resistance moment of the cross-section should be reduced to $M_{V.Rd}$ the reduced design plastic resistance moment allowing for the shear force, obtained as follows:*

a) *for cross-sections with equal flanges, bending about the major axis:*

$$M_{V.Rd} = \left[W_{p\ell} - \frac{\rho A_v^2}{4\ t_w}\right] f_y/\gamma_{M0} \quad \text{but } M_{V.Rd} \leq M_{c.Rd} \tag{5.22}$$

where $\rho = (2V_{Sd}/V_{p\ell.Rd} - 1)^2$

b) *for other cases: $M_{V.Rd}$ should be taken as the design plastic resistance moment of the cross-section, calculated using a reduced strength $(1 - p)\, f_y$ for the shear area, but not more than $M_{c.Rd}$.*

NOTE Paragraph (3) applies to Class 1, 2, 3 and 4 cross-sections. The appropriate value of $M_{c.Rd}$ should be used, see **5.4.5.2**.

5.4.8 Bending and axial force

5.4.8.1 *Class 1 and 2 cross-sections*

(1) For class 1 and 2 cross-sections, the criterion to be satisfied in the absence of shear force is:

$$M_{Sd} \leq M_{N.Rd} \tag{5.23}$$

where $M_{N.Rd}$ is the reduced design plastic resistance moment allowing for the axial force.

(2) For a plate without bolt holes, the reduced design plastic resistance moment is given by:

$M_{N.Rd} = M_{p\ell.Rd}\,[1 - (N_{Sd}/N_{p\ell.Rd})^2]$

and the criterion becomes:

$$\frac{M_{Sd}}{M_{pl.Rd}} + \left[\frac{N_{Sd}}{N_{pl.Rd}}\right]^2 \leq 1 \tag{5.24}$$

(3) In flanged sections, the reduction of the theoretical plastic resistance moment by the presence of small axial forces is counter-balanced by strain hardening and may be neglected. However, for bending about the y-y-axis, allowance shall be made for the effect of the axial force on the plastic resistance moment when the axial force exceeds half the plastic tension resistance of the web, or a quarter of the plastic tension resistance of the cross-section, whichever is smaller. Similarly, for bending about the z-z-axis, allowance shall be made for the effect of the axial force when it exceeds the plastic tension resistance of the web.

(4) *For cross-sections without bolt holes, the following approximations may be used for standard rolled I or H sections:*

$$M_{Ny.Rd} = M_{p\ell.y.Rd}(1 - n)/(1 - 0{,}5a) \text{ but } M_{Ny.Rd} \leq M_{p\ell.y.Rd} \tag{5.25}$$

for $n \leq a$: $M_{Nz.Rd} \leq M_{p\ell.z.Rd}$

$$\text{for } n > a: \quad M_{Nz.Rd} = M_{pl.z.Rd}\left[1 - \left[\frac{n - a}{1 - a}\right]^2\right] \tag{5.26}$$

where $n = N_{Sd}/N_{p\ell.Rd}$

and $a = (A\text{-}2bt_f)/A$ *but* $a \leq 0{,}5$

(5) *The expressions given in (4) may also be used for welded I or H sections with equal flanges.*

(6) *The approximations given in (4) may be further simplified (for standard rolled I or H sections only) to:*

$$M_{Ny.Rd} = 1{,}11 M_{p\ell.y.Rd}\,(1 - n) \text{ but } M_{Ny.Rd} \leq M_{p\ell.y.Rd} \tag{5.27}$$

for $n \leq 0{,}2$: $M_{Nz.Rd} \leq M_{p\ell.z.Rd}$

$$\text{for } n > 0{,}2: \quad M_{Nz.Rd} = 1{,}56 M_{p\ell.z.Rd}\,(1 - n)(n + 0{,}6) \tag{5.28}$$

(7) *For cross-sections without bolt holes, the following approximations may be used for rectangular structural hollow sections of uniform thickness:*

$$M_{Ny.Rd} = M_{pt.y.Rd}\,(1 - n)/(1 - 0{,}5a_w) \quad \text{but } M_{Ny.Rd} \leq M_{p\ell.y.Rd} \tag{5.29}$$

$$M_{Nz.Rd} = M_{p\ell.z.Rd}\,(1 - n)/(1 - 0{,}5a_f) \quad \text{but } M_{Nz.Rd} \leq M_{p\ell.z.Rd} \tag{5.30}$$

where $a_w = (A - 2bt)/A$ *but* $a_w \leq 0{,}5$

and $a_f = (A - 2ht)/A$

(8) *The expressions given in (7) may also be used for welded box sections with equal flanges and equal webs, by taking:*

$a_w = (A\text{-}2bt_f)/A$ *but* $a_w \leq 0{,}5$

$a_f = (A\text{-}2ht_w)/A$ *but* $a_f \leq 0{,}5$

(9) *The approximations given in (7) may be further simplified for standard rectangular structural hollow sections of uniform thickness, as follows:*

- *for a square section:*

$$M_{N.Rd} = 1{,}26 M_{p\ell.Rd}\,(1 - n) \quad \text{but } M_{N.Rd} \leq M_{p\ell.Rd} \tag{5.31}$$

- *for a rectangular section:*

$$M_{Ny.Rd} = 1{,}33 M_{p\ell.y.Rd}\,(1 - n) \quad \text{but } M_{Ny.Rd} \leq M_{p\ell.y.Rd} \tag{5.32}$$

$$M_{Nz.Rd} = M_{p\ell.z.Rd}\,(1 - n)/(0{,}5 + ht/A) \quad \text{but } M_{Nz.Rd} \leq M_{p\ell.z.Rd} \tag{5.33}$$

(10) *For cross-sections without bolt holes, the following approximation may be used for circular tubes of uniform thickness:*

$$M_{N.Rd} = 1{,}04 M_{p\ell.Rd}\,(1 - n^{1{,}7}) \quad \text{but } M_{N.Rd} \leq M_{p\ell.Rd} \tag{5.34}$$

(11) *For bi-axial bending the following approximate criterion may be used:*

$$\left[\frac{M_{y.Sd}}{M_{Ny.Rd}}\right]^{\alpha} + \left[\frac{M_{z.Sd}}{M_{Nz.Rd}}\right]^{\beta} \leq 1 \tag{5.35}$$

in which α and β are constants, which may conservatively be taken as unity, otherwise as follows:

- *I and H sections:*
  $\alpha = 2; \beta = 5n$ *but* $\beta \geq 1$
- *circular tubes:*
  $\alpha = 2; \beta = 2$
- *rectangular hollow sections:*

$$\alpha = \beta = \frac{1,66}{1 - 1,13n^2} \quad \text{but } \alpha = \beta \leq 6$$

- *solid rectangles and plates:*
  $\alpha = \beta = 1,73 + 1,8n^3$

where $n = N_{Sd}/N_{p\ell.Rd}$

(12) *As a further conservative approximation, the following criterion may be used:*

$$\frac{N_{Sd}}{N_{p\ell.Rd}} + \frac{M_{y.Sd}}{M_{p\ell.y.Rd}} + \frac{M_{z.Sd}}{M_{p\ell.z.Rd}} \leq 1 \tag{5.36}$$

5.4.8.2 *Class 3 cross-sections*

(1) In the absence of shear force, Class 3 cross-sections will be satisfactory if the maximum longitudinal stress $\sigma_{x.Ed}$ satisfies the criterion:

$$\sigma_{x.Ed} \leq f_{yd} \tag{5.37}$$

where $f_{yd} = f_y/\gamma_{M0}$

(2) *For cross-sections without holes for fasteners, the above criterion becomes:*

$$\frac{N_{Sd}}{Af_{yd}} + \frac{M_{y.Sd}}{W_{e\ell.y}\, f_{yd}} + \frac{M_{z.Sd}}{W_{e\ell.z}\, f_{yd}} \leq 1 \tag{5.38}$$

5.4.8.3 *Class 4 cross-sections*

(1) In the absence of shear force, Class 4 cross-sections will be satisfactory if the maximum longitudinal stress $\sigma_{x.Ed}$ calculated using the effective widths of the compression elements [see **5.3.2**(2)] satisfies the criterion:

$$\sigma_{x.Ed} \leq f_{yd} \tag{5.39}$$

where $f_{yd} = f_y/\gamma_{M1}$

(2) *For cross-sections without holes for fasteners, the above criterion becomes:*

$$\frac{N_{Sd}}{A_{eff}\, f_{yd}} + \frac{M_{y.Sd} + N_{Sd}\, e_{Ny}}{W_{eff.y}\, f_{yd}} + \frac{M_{z.Sd} + N_{Sd}\, e_{Nz}}{W_{eff.z}\, f_{yd}} \leq 1 \tag{5.40}$$

where A_{eff} *is the effective area of the cross-section when subject to uniform compression.*

W_{eff} *is the effective section modulus of the cross-section when subject only to moment about the relevant axis.*

e_N *is the shift of the relevant centroidal axis when the cross-section is subject to uniform compression.*

5.4.9 Bending, shear and axial force

(1) When the shear force exceeds half the plastic shear resistance, allowance shall be made for the effect of both shear force and axial force on the reduced plastic resistance moment.

(2) *Provided that the design value of the shear force V_{Sd} does not exceed 50 % of the design plastic shear resistance $V_{p\ell.Rd}$ no reduction need be made in combinations of moment and axial force that meet the criteria in **5.4.8**.*

(3) *When V_{Sd} exceeds 50 % of $V_{p\ell.Rd}$ the design resistance of the cross-section to combinations of moment and axial force should be calculated using a reduced yield strength $(1 - \rho)f_y$ for the shear area, where $\rho = (2V_{Sd}/V_{p\ell.Rd} - 1)^2$.*

5.4.10 Transverse forces on webs

(1) In the absence of shear force, the web of a member subject to a transverse force in the plane of the web, see Figure 5.4.3, in addition to any combination of moment and axial force on the cross-section, shall at all points satisfy the following yield criterion:

$$\left[\frac{\sigma_{x.Ed}}{f_{yd}}\right]^2 + \left[\frac{\sigma_{z.Ed}}{f_{yd}}\right]^2 - \left[\frac{\sigma_{x.Ed}}{f_{yd}}\right]\left[\frac{\sigma_{z.Ed}}{f_{yd}}\right] \leq 1 \qquad (5.41)$$

where $\sigma_{x.Ed}$ is the design value of the local longitudinal stress due to moment and axial force at the point

$\sigma_{z.Ed}$ is the design value of the stress at the same point due to the transverse force

and $f_{yd} = f_y/\gamma_{M0}$

In expression (5.41) above, $\sigma_{x.Ed}$ and $\sigma_{z.Ed}$ shall each be taken as positive for compression and negative for tension.

(2) *When the moment resistance is based on a plastic distribution of stresses in the cross-section, the above criterion may be assumed to be satisfied when:*

$$\left[\frac{\sigma_{xm.Ed}}{f_{yd}}\right]^2 + \left[\frac{\sigma_{z.Ed}}{f_{yd}}\right]^2 - k\left[\frac{\sigma_{xm.Ed}}{f_{yd}}\right]\left[\frac{\sigma_{z.Ed}}{f_{yd}}\right] \leq 1 - \beta_m \qquad (5.42)$$

where $\sigma_{xm.Ed}$ is the design value of the mean longitudinal stress in the web

$\beta_m = M_{w.Sd}/M_{pl.w.Rd}$

$M_{w.Sd}$ is the design value of the moment in the web

$M_{p\ell.w.Rd} = 0,25 t_w d^2 f_y/\gamma_{M0}$

and k is obtained as follows:

for $\sigma_{xm.Ed}/\sigma_{z.Ed} \leq 0$: $k = 1 - \beta_m$

for $\sigma_{xm.Ed}/\sigma_{z.Ed} > 0$:

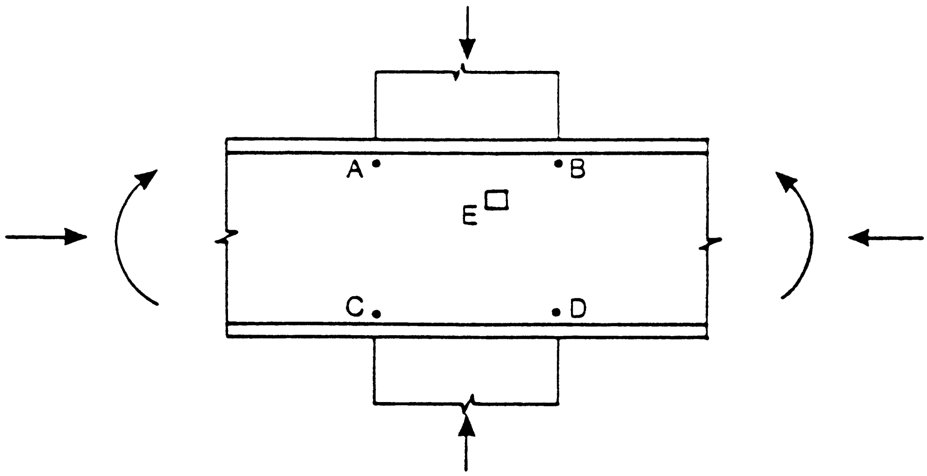
- *if $\beta_m \leq 0,5$: $k = 0,5\,(1 + \beta_m)$*
- *if $\beta_m > 0,5$: $k = 0,5\,(1 - \beta_m)$*

(3) *Provided that the design value of the shear force V_{Sd} does not exceed 50 % of the design plastic shear resistance $V_{p\ell.Rd}$, the criterion given in (2) may be adopted without any modification to allow for shear.*

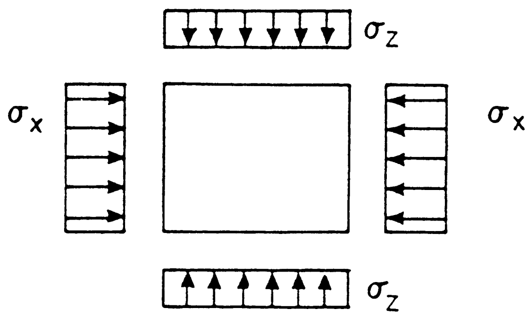
(4) *When V_{Sd} exceeds 50 % of $V_{p\ell.Rd}$ the yield criterion given in (1) should be modified to:*

$$\left[\frac{\sigma_{x.Ed}}{f_{yd}}\right]^2 + \left[\frac{\sigma_{z.Ed}}{f_{yd}}\right]^2 - \left[\frac{\sigma_{x.Ed}}{f_{yd}}\right]\left[\frac{\sigma_{z.Ed}}{f_{yd}}\right] \leq 1 - \rho \qquad (5.43)$$

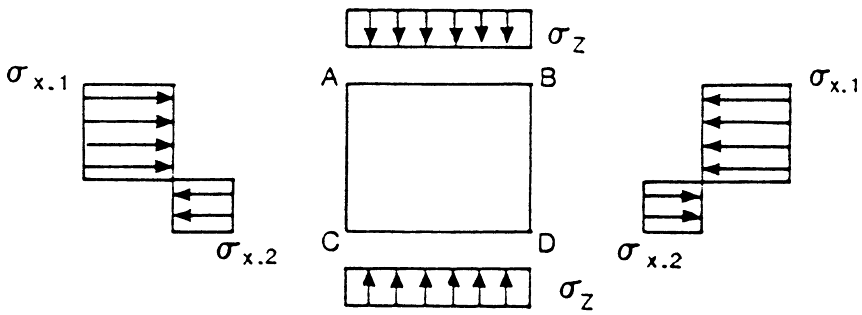
where $\rho = (2V_{Sd}/V_{p\ell.Rd} - 1)^2$



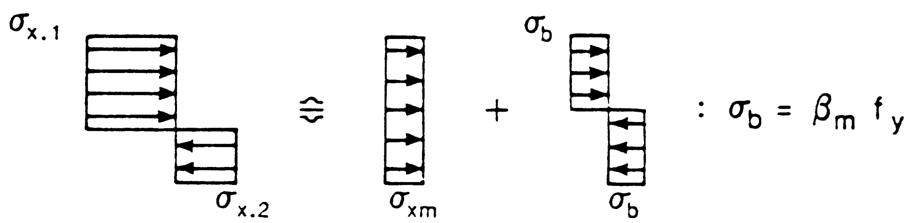
(a)Layout



(b) Stresses in element E



(c) Stresses in panel ABCD



(d) Approximate equivalent longitudinal stresses

Figure 5.4.3 — Stresses in web panel due to bending moment, axial force and transverse force

(5) *When V_{Sd} exceeds 50 % of $V_{p\ell.Rd}$ and the moment resistance is based on a plastic distribution of stresses in the cross-section, the following approximate criterion may be adopted:*

$$\left[\frac{\sigma_{xm.Ed}}{f_{yd}}\right]^2 + \left[\frac{\sigma_{z.Ed}}{f_{yd}}\right]^2 - k\left[\frac{\sigma_{xm.Ed}}{f_{yd}}\right]\left[\frac{\sigma_{z.Ed}}{f_{yd}}\right] \leq 1 - \beta_m - \rho \tag{5.44}$$

where k and β_m are as defined in (2).

(6) *The effective value of the transverse stress $\sigma_{z.Ed}$ due to a point load should be determined by assuming it to be uniformly distributed over a length s equal to the lesser of the depth d of the web and the spacing a of the transverse web stiffeners.*

(7) *The effective value of the transverse stress $\sigma_{z.Ed}$ due to a load distributed over a length between transverse web stiffeners less than their spacing a should similarly be determined by assuming it to be distributed over a length s determined as in (6).*

(8) *The effects of transverse compressive forces on the local buckling resistance of the web should be checked, see* ***5.3.6****.*

(9) *In addition the crippling resistance and the buckling resistance of the web should be checked, see* ***5.7.4*** *and* ***5.7.5****.*

5.5 Buckling resistance of members

5.5.1 Compression members

5.5.1.1 *Buckling resistance*

(1) The design buckling resistance of a compression member shall be taken as:

$$N_{b.Rd} = \chi\, \beta_A A f_y / \gamma_{M1} \tag{5.45}$$

where β_A = 1 for Class 1, 2 or 3 cross-sections

β_A = A_{eff}/A for Class 4 cross-sections

and χ is the reduction factor for the relevant buckling mode.

(2) For hot rolled steel members with the types of cross-section commonly used for compression members, the relevant buckling mode is generally "flexural" buckling.

(3) In some cases the "torsional" or "flexural-torsional" modes may govern. Reference may be made to ENV 1993-1-3 Eurocode 3-1.3[17].

5.5.1.2 *Uniform members*

(1) For constant axial compression in members of constant cross-section, the value of χ for the appropriate non-dimensional slenderness $\bar{\lambda}$, may be determined from:

$$\chi = \frac{1}{\phi + [\phi^2 - \bar{\lambda}^2]^{0,5}} \quad \text{but } \chi \leq 1 \tag{5.46}$$

where ϕ = $0,5\,[1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2]$

α is an imperfection factor

$\bar{\lambda}$ = $[\beta_A A f_y / N_{cr}]^{0,5} = (\lambda/\lambda_1)\,[\beta_A]^{0,5}$

λ is the slenderness for the relevant buckling mode

λ_1 = $\pi\,[E/f_y]^{0,5} = 93,9\varepsilon$

ε = $[235/f_y]^{0,5}$ (f_y in N/mm^2)

and N_{cr} is the elastic critical force for the relevant buckling mode.

(2) The imperfection factor α corresponding to the appropriate buckling curve shall be obtained from Table 5.5.1.

[17] In preparation

Table 5.5.1 — Imperfection factors

Bucking curve	a	b	c	d
Imperfection factor α	0,21	0,34	0,49	0,76

(3) Values of the reduction factor χ for the appropriate non-dimensional slenderness $\bar{\lambda}$ may be obtained from Table 5.5.2.

(4) Alternatively, uniform members may be verified using second order analysis, see **5.5.1.3**(4) and **5.5.1.3**(6).

Table 5.5.2 — Reduction factors χ

$\bar{\lambda}$	Buckling curve			
	a	b	c	d
0,2	1,0000	1,0000	1,0000	1,0000
0,3	0,9775	0,9641	0,9491	0,9235
0,4	0,9528	0,9261	0,8973	0,8504
0,5	0,9243	0,8842	0,8430	0,7793
0,6	0,8900	0,8371	0,7854	0,7100
0,7	0,8477	0,7837	0,7247	0,6431
0,8	0,7957	0,7245	0,6622	0,5797
0,9	0,7339	0,6612	0,5998	0,5208
1,0	0,6656	0,5970	0,5399	0,4671
1,1	0,5960	0,5352	0,4842	0,4189
1,2	0,5300	0,4781	0,4338	0,3762
1,3	0,4703	0,4269	0,3888	0,3385
1,4	0,4179	0,3817	0,3492	0,3055
1,5	0,3724	0,3422	0,3145	0,2766
1,6	0,3332	0,3079	0,2842	0,2512
1,7	0,2994	0,2781	0,2577	0,2289
1,8	0,2702	0,2521	0,2345	0,2093
1,9	0,2449	0,2294	0,2141	0,1920
2,0	0,2229	0,2095	0,1962	0,1766
2,1	0,2036	0,1920	0,1803	0,1630
2,2	0,1867	0,1765	0,1662	0,1508
2,3	0,1717	0,1628	0,1537	0,1399
2,4	0,1585	0,1506	0,1425	0,1302
2,5	0,1467	0,1397	0,1325	0,1214
2,6	0,1362	0,1299	0,1234	0,1134
2,7	0,1267	0,1211	0,1153	0,1062
2,8	0,1182	0,1132	0,1079	0,0997
2,9	0,1105	0,1060	0,1012	0,0937
3,0	0,1036	0,0994	0,0951	0,0882

5.5.1.3 ***Non-uniform members***

(1) Tapered members and members with changes of cross-section within their length may be verified using second order analysis, see (4) and (6).

(2) Alternatively, simplified methods of analysis may be based on modifications of the basic procedure for uniform members.

(3) *No one method is preferred. Any recognised method may be used provided that it can be demonstrated to be conservative.*

(4) Second order analysis of a member shall incorporate the appropriate equivalent initial bow imperfection given in Figure 5.5.1 corresponding to the relevant buckling curve, depending on the method of analysis and type of cross-section verification.

(5) The equivalent initial bow imperfections given in Figure 5.5.1 shall also be used where it is necessary (according to **5.2.4.5**) to include member imperfections in the global analysis.

(6) When the imperfections given in Figure 5.5.1 are used, the resistances of the cross-sections shall be verified as specified in **5.4**, but using γ_{M1} in place of γ_{M0}.

5.5.1.4 *Flexural buckling*

(1) For flexural buckling the appropriate buckling curve shall be determined from Table 5.5.3.

(2) Sections not contained in Table 5.5.3 shall be classified analogously.

(3) The slenderness λ shall be taken as follows:

$$\lambda = \ell / i \tag{5.47}$$

where i is the radius of gyration about the relevant axis, determined using the properties of the gross cross-section.

(4) Cold formed structural hollow sections shall be verified using either:

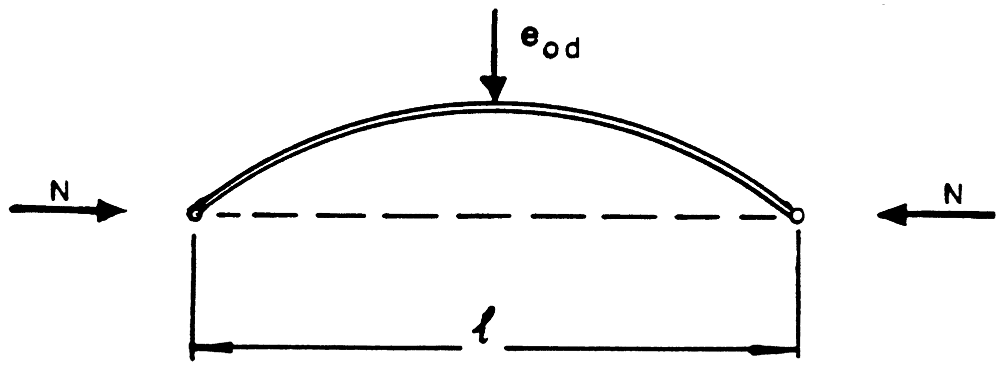
a) the basic yield strength f_{yb} of the flat sheet material out of which the member is made by cold-forming, with buckling curve b.

b) The average yield strength f_{ya} of the member after cold-forming, determined in conformity with the definition given in Figure 5.5.2, with buckling curve c.

5.5.1.5 *Buckling length*

(1) The buckling length ℓ of a compression member with both ends effectively held in position laterally, may conservatively be taken as equal to its system length L.

(2) Alternatively the buckling length ℓ may be determined using informative Annex E.



Cross-section		Method of global analysis	
Method used to verify resistance	Section type and axis	Elastic or Rigid — Plastic or Elastic — Perfectly plastic	Elasto-plastic (plastic zone method)
Elastic [**5.4.8.2**]	Any	$\alpha(\bar{\lambda} - 0{,}2)\, k\gamma\, W_{e\ell}/A$	—
Linear plastic [**5.4.8.1**(12)]	Any	$\alpha(\bar{\lambda} - 0{,}2)\, k\gamma\, W_{p\ell}/A$	—
Non-linear plastic [**5.4.8.1**(1) to (11)]	I-section yy-axis	$1{,}33\alpha(\bar{\lambda} - 0{,}2)\, k\gamma\, W_{p\ell}/A$	$\alpha(\bar{\lambda} - 0{,}2)\, k\gamma\, W_{p\ell}/A$
	I-section zz-axis	$2{,}0\, k\gamma\, e_{eff}/\varepsilon$	$k\gamma\, e_{eff}/\varepsilon$
	Rectangular hollow section	$1{,}33\alpha(\bar{\lambda} - 0{,}2)\, k\gamma\, W_{p\ell}/A$	$\alpha(\bar{\lambda} - 0{,}2)\, k_\gamma\, W_{p\ell}/A$
	Circular hollow section	$1{,}5\, k\gamma\, e_{eff}/\varepsilon$	$k\gamma\, e_{eff}/\varepsilon$

$k\gamma = (1 - k_\delta) + 2k_\delta\bar{\lambda}$ but $k_\gamma \geq 1{,}0$

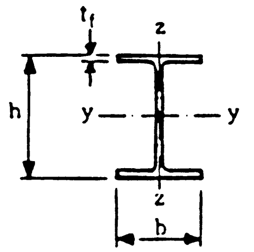
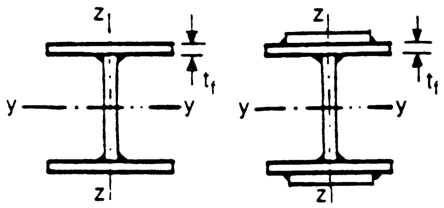
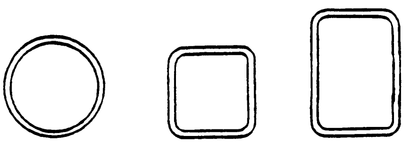
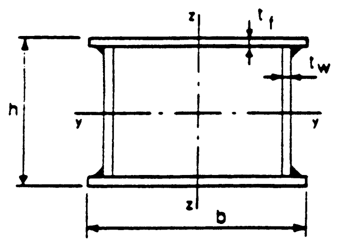
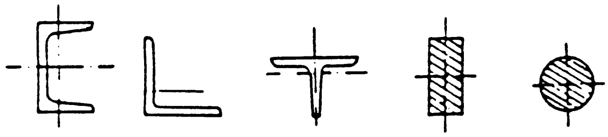
Buckling curve	σ	e_{eff}	k_δ			
			$\gamma_{M1} = 1{,}05$	$\gamma_{M1} = 1{,}10$	$\gamma_{M1} = 1{,}15$	$\gamma_{M1} = 1{,}20$
a	0,21	ℓ/600	0,12	0,23	0,33	0,42
b	0,34	ℓ/380	0,08	0,15	0,22	0,28
c	0,49	ℓ/270	0,06	0,11	0,16	0,20
d	0,76	ℓ/180	0,04	0,08	0,11	0,14

Non-uniform members:

Use value of $W_{e\ell}/A$ or $W_{p\ell}/A$ at centre of buckling length ℓ

Figure 5.5.1 — Design values of equivalent initial bow imperfection $e_{o,d}$

Table 5.5.3 — Selection of buckling curve for a cross-section

Cross section	Limits	Buckling about axis	Buckling curve
Rolled I-sections 	h/b > 1,2: $t_f \leq$ 40 mm	y – y z – z	a b
	40 mm < $t_f \leq$ 100 mm	y – y z – z	b c
	h/b $\leq$ 1,2: $t_f \leq$ 100 mm	y – y z – z	a b
	t_f > 100 mm	y – y z – z	d d
Welded I-sections 	$t_f \leq$ 40 mm	y – y z – z	b c
	t_f > 40 mm	y – y z – z	c d
Hollow sections 	hot rolled	any	a
	cold formed — using f_{yb}[a]	any	b
	cold formed — using f_{ya}[a]	any	c
Welded box sections 	generally (except as below)	any	b
	thick welds and b/t_f < 30 h/t_W < 30	y – y z – z	c c
U-, L-, T- and solid sections 		any	c

[a] See **5.5.1.4**(4) and Figure 5.5.2

Average yield strength:
The average yield strength f_{ya} may be determined from full size section tests or as follows:

$$f_{ya} = f_{yb} + (knt^2/A_g)(f_u - f_{yb})$$

where:

f_{yb}, f_u is the tensile yield strength and tensile ultimate strength of the basic material as defined
t below (N/mm²)
A_g is the material thickness (mm)
k is the gross cross-sectional area (mm²)
is the coefficient depending on the type of forming:

- k = 7 for cold rolling
- k = 5 for other methods of forming

n is the number of 90° bend in the section with an internal radius < 5t (fractions of 90° bends should be counted as fractions of n)

and f_{ya} should not exceed f_u or 1,2 f_{yb}

The increase in yield strength due to cold working should not be utilised for members which are welded, annealed, galvanised (after forming) or subject to heat treatment after forming which may produce softening.

Basic material:
Basic material is the flat sheet material out of which sections are made by cold-forming.

Figure 5.5.2 — Average yield strength f_{ya} of cold formed structural hollow sections

5.5.2 Lateral-torsional buckling of beams

(1) The design buckling resistance moment of a laterally unrestrained beam shall be taken as:

$$M_{b.Rd} = \chi_{LT}\, \beta_W\, W_{pl.y} f_y / \gamma_{M1} \qquad (5.48)$$

where $\beta_W = 1$ for Class 1 or Class 2 cross-sections
$\beta_W = W_{el.y}/W_{pl.y}$ for Class 3 cross-sections
$\beta_W = W_{eff.y}/W_{pl.y}$ for Class 4 cross-sections
and χ_{LT} is the reduction factor for lateral-torsional buckling.

(2) The value of χ_{LT} for the appropriate non-dimensional slenderness $\bar{\lambda}_{LT}$ may be determined from:

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \bar{\lambda}_{LT}^2]^{0,5}} \quad \text{but } \chi_{LT} \leq 1 \qquad (5.49)$$

in □□□□□φ

$$\phi_{LT} = 0,5\,[1 + \alpha_{LT}(\bar{\lambda}_{LT} - 0,2) + \bar{\lambda}_{LT}^2]$$

(3) *The values of the imperfection factor α_{LT} for lateral torsional buckling should be taken as:*

$\alpha_{LT} = 0,21$ for rolled sections

$\alpha_{LT} = 0,49$ for welded sections

(4) *Values of the reduction factor χ_{LT} for the appropriate non-dimensional slenderness $\bar{\lambda}_{LT}$ may be obtained from Table 5.5.2 with $\bar{\lambda} = \bar{\lambda}_{LT}$ and $\chi = \chi_{LT}$, using:*

- *for rolled sections:*

curve a ($\alpha = 0,21$)

- *for welded sections:*

curve c ($\alpha = 0,49$)

(5) The value of $\bar{\lambda}_{LT}$ may be determined from:

$$\bar{\lambda}_{LT} = [\beta_W W_{p\ell.y} f_y/M_{cr}]^{0,5} = [\lambda_{LT}/\lambda_1]\ [\beta_W]^{0,5}$$

where $\lambda_1 = \pi\ [E/f_y]^{0,5} = 93,9\varepsilon$

$\varepsilon = [235/f_y]^{0,5}$ (f_y in N/mm^2)

and M_{cr} is the elastic critical moment for lateral-torsional buckling.

(6) Information for the calculation of M_{cr} (or for the direct calculation of λ_{LT}) is given in informative Annex F.

(7) Where the non-dimensional slenderness $\bar{\lambda}_{LT} \leq 0,4$ no allowance for lateral-torsional buckling is necessary.

(8) A beam with full restraint does not need to be checked for lateral-torsional buckling.

5.5.3 Bending and axial tension

(1) Members subject to combined bending and axial tension shall be checked for resistance to lateral-torsional buckling, treating the axial force and bending moment as a vectorial effect, see **2.3.3.1**(4).

(2) *Where the axial force and bending moment can vary independently, the design value of the axial tension should be multiplied by a reduction factor for vectorial effects:*

$$\psi_{vec} = \boxed{0,8}$$

(3) *The not calculated stress $\sigma_{com.Ed}$ (which can exceed f_y) in the extreme compression fibre due to the vectorial effects should be determined from:*

$$\sigma_{com.Ed} = M_{Sd}/W_{com} - \psi_{vec}\ N_{T.Sd}/A \qquad (5.50)$$

where W_{com} *is the elastic section modulus for the extreme compression fibre*

and $N_{T.Sd}$ *is the design value of the axial tension*

(4) *The verification should be carried out using an effective design internal moment $M_{eff.Sd}$ obtained from:*

$$M_{eff.Sd} = W_{com}\sigma_{com.Ed}$$

(5) *The design buckling resistance moment $M_{b.Rd}$ should be obtained using **5.5.2**.*

5.5.4 Bending and axial compression

(1) Members with Class 1 and Class 2 cross-sections subject to combined bending and axial compression shall satisfy:

$$\frac{N_{Sd}}{\chi_{min}\ A\ f_y/\gamma_{M1}} + \frac{k_y\ M_{y.Sd}}{W_{pl.y}\ f_y/\gamma_{M1}} + \frac{k_z\ M_{z.Sd}}{W_{pl.z}\ f_y/\gamma_{M1}} \leq 1 \qquad (5.51)$$

in which:

$$k_y = 1 - \frac{\mu_y\ N_{Sd}}{\chi_y\ A\ f_y} \quad \text{but } k_y \leq 1,5$$

$$\mu_y = \bar{\lambda}_y\ (2\beta_{My} - 4) + \left[\frac{W_{pl.y} - W_{el.y}}{W_{el.y}}\right] \quad \text{but } \mu_y \leq 0,90$$

$$k_z = 1 - \frac{\mu_z\ N_{Sd}}{\chi_z\ A\ f_y} \quad \text{but } k_z \leq 1,5$$

$$\mu_z = \bar{\lambda}_z\ (2\beta_{Mz} - 4) + \left[\frac{W_{pl.z} - W_{el.z}}{W_{el.z}}\right] \quad \text{but } \mu_z \leq 0,90$$

X_{min} is the lesser of χ_y and χ_z

where χ_y and χ_z are the reduction factors from **5.5.1** for the y-y and z-z axes respectively

and β_{My} and β_{Mx} are equivalent uniform moment factors for flexural buckling, see (7).

(2) Members with Class 1 and Class 2 cross-sections for which lateral-torsional buckling is a potential failure mode shall also satisfy:

$$\frac{N_{Sd}}{\chi_z\, A\, f_y/\gamma_{M1}} + \frac{k_{LT}\, M_{y.Sd}}{\chi_{LT}\, W_{pl.y}\, f_y/\gamma_{M1}} + \frac{k_z\, M_{z.Sd}}{W_{pl.z}\, f_y/\gamma_{M1}} \leq 1 \quad (5.52)$$

in which:

$$k_{LT} = 1 - \frac{\mu_{LT}\, N_{Sd}}{\chi_z\, A\, f_y} \quad \text{but } k_{LT} \leq 1$$

$$\mu_{LT} = 0{,}15\, \bar{\lambda}_z\, \beta_{M.LT} - 0{,}15 \quad \text{but } \mu_{LT} \leq 0{,}90$$

(3) where $\beta_{M.LT}$ is an equivalent uniform moment factor for lateral-torsional buckling, see (7).
Members with Class 3 cross-sections subject to combined bending and axial load shall satisfy:

$$\frac{N_{Sd}}{\chi_{min}\, A\, f_y/\gamma_{M1}} + \frac{k_y\, M_{y.Sd}}{W_{el.y}\, f_y/\gamma_{M1}} + \frac{k_z\, M_{z.Sd}}{W_{el.z}\, f_y/\gamma_{M1}} \leq 1 \quad (5.53)$$

where k_y, k_z and χ_{min} are as in (1)

$\mu_y = \bar{\lambda}_y (2\, \beta_{My} - 4)$ but $\mu_y \leq 0{,}90$

and $\mu_z = \bar{\lambda}_z (2\, \beta_{Mz} - 4)$ but $\mu_z \leq 0{,}90$

(4) Members with Class 3 cross-sections for which lateral-torsional buckling is a potential failure mode shall also satisfy:

$$\frac{N_{Sd}}{\chi_z\, A\, f_y/\gamma_{M1}} + \frac{k_{LT}\, M_{y.Sd}}{\chi_{LT}\, W_{el.y}\, f_y/\gamma_{M1}} + \frac{k_z\, M_{z.Sd}}{W_{el.z}\, f_y/\gamma_{M1}} \leq 1 \quad (5.54)$$

(5) Members with Class 4 cross-sections subject to combined bending and axial load shall satisfy:

$$\frac{N_{Sd}}{\chi_{min}\, A_{eff}\, f_y/\gamma_{M1}} + \frac{k_y\, M_{y.Sd} + N_{Sd}\, e_{N.y}}{W_{eff.y}\, f_y/\gamma_{M1}} + \frac{k_z\, M_{z.Sd} + N_{Sd}\, e_{N.z}}{W_{eff.z}\, f_y/\gamma_{M1}} \leq 1 \quad (5.56)$$

where k_y, k_z and χ_{min} are as in (1)

μ_y and μ_z are as in (3)

and A_{eff}, $W_{eff.y}$, $W_{eff.z}$, e_{Ny} and $e_{N.z}$ are as in **5.4.8.3**.

(6) Members with Class 4 cross-sections for which lateral-torsional buckling is a potential failure mode shall also satisfy:

$$\frac{N_{Sd}}{\chi_z\, A_{eff}\, f_y/\gamma_{M1}} + \frac{k_{LT}\, M_{y.Sd} + N_{Sd}\, e_{N.y}}{\chi_{LT}\, W_{eff.y}\, f_y/\gamma_{M1}} + \frac{k_z\, M_{z.Sd} + N_{Sd}\, e_{N.z}}{W_{eff.z}\, f_y/\gamma_{M1}} \leq 1 \quad (5.57)$$

(7) The equivalent uniform moment factors $\beta_{M.y}$, $\beta_{M.z}$ and $\beta_{M.LT}$ shall be obtained from Figure 5.5.3 according to the shape of the bending moment diagram between the relevant braced points as follows:

factor:	moment about axis:	points braced in direction:
$\beta_{M.y}$	y-y	z-z
$\beta_{M.z}$	z-z	y-y
$\beta_{M.LT}$	y-y	y-y

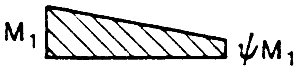
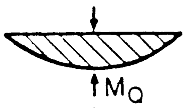
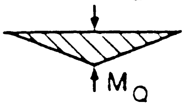
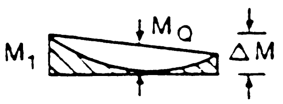
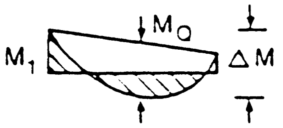
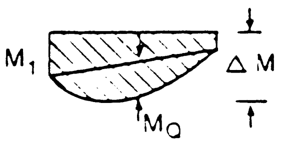
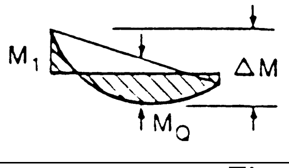
Moment diagram	Equivalent uniform moment factor β_M
end moments  $-1 \le \psi \le 1$	$\beta_{M,\psi} = 1{,}8 - 0{,}7\psi$
moments due to in-plane lateral loads 	$\beta_{M,Q} = 1{,}3$
	$\beta_{M,Q} = 1{,}4$
moments due to in-plane lateral loads plus end moments 	$\beta_M = \beta_{M,\psi} + \frac{M_Q}{\Delta M}(\beta_{M,Q} - \beta_{M,\psi})$
	$M_Q = \mid \max M \mid$ due to lateral load only
 	$\Delta M \begin{cases} \mid \max M \mid & \text{for moment diagram without change of sign} \\ \mid \max M \mid + \mid \min M \mid & \text{for moment diagram with change of sign} \end{cases}$

Figure 5.5.3 — Equivalent uniform moment factors

5.6 Shear buckling resistance

5.6.1 Basis

(1) Webs with d/t_w greater than 69ε for an unstiffened web, or $30\varepsilon\sqrt{k_\tau}$ [see **5.4.6**(7)] for a stiffened web, shall be checked for resistance to shear buckling.

(2) The shear buckling resistance of a web depends on the depth-to-thickness ratio d/t_w and the spacing of any intermediate web stiffeners.

(3) The shear buckling resistance may also depend on the anchorage of tension fields by end stiffeners or by flanges. The anchorage provided by flanges is reduced by longitudinal stresses due to bending moment and axial load.

(4) All webs with d/t_w greater than 69ε shall be provided with transverse stiffeners at the supports.

5.6.2 Design methods

(1) *For webs without intermediate transverse stiffeners and for webs with transverse stiffeners only, the shear buckling resistance may be verified using either:*

a) *the simple post-critical method (see **5.6.3**), or*

b) *the tension field method (see **5.6.4**).*

(2) *Alternatively, the methods given in Part 2 of Eurocode 3 may be adopted.*

(3) *The simple post-critical method can be used for webs of I-section girders, with or without intermediate transverse stiffeners, provided that the web has transverse stiffeners at the supports.*

(4) *The tension field method may be used for webs with transverse stiffeners at the supports plus intermediate transverse stiffeners, provided that adjacent panels or end posts provide anchorage for the tension fields. However it should not be used where:*

$a/d < 1{,}0$

where a *is the clear spacing between transverse stiffeners*

and d *is the depth of the web*

(5) *Where the transverse stiffeners are widely spaced, the tension field method becomes over-conservative. It is not recommended for use where:*

$a/d > 3{,}0$

(6) *For both methods, intermediate transverse stiffeners should be checked as specified in* **5.6.5** *and welds should be checked as specified in **5.6.6**.*

(7) *For webs with longitudinal stiffeners refer to ENV 1993-2 Eurocode 3-2*[18].

5.6.3 Simple post-critical method

(1) *In the simple post-critical method, the design shear buckling resistance $V_{ba.Rd}$ should be obtained from:*

$$V_{ba.Rd} = d\, t_w\, \tau_{ba}/\gamma_{M1} \qquad (5.58)$$

where τ_{ba} is the simple post-critical shear strength.

(2) *The simple post-critical shear strength τ_{ba} should be determined as follows:*

a) *when $\bar{\lambda}_w \leq 0{,}8$:*

$$\tau_{ba} = (f_{yw}/\surd 3)$$

b) *when $0{,}8 < \bar{\lambda}_w < 1{,}2$:*

$$\tau_{ba} = [1 - 0{,}625\,(\bar{\lambda}_w - 0{,}8)]\,(f_{yw}/\surd 3)$$

c) *when $\bar{\lambda}_w \geq 1{,}2$:*

$$\tau_{ba} = [0{,}9/\bar{\lambda}_w]\,(f_{yw}/\surd 3)$$

in which $\bar{\lambda}_w$ is the web slenderness given by:

$$\bar{\lambda}_w = [\,(f_{yw}/\surd 3)/\tau_{cr}\,]^{0,5} = \frac{d/t_w}{37{,}4\,\varepsilon\sqrt{k_\tau}}$$

where τ_{cr} *is the elastic critical shear strength*

and k_τ *is the buckling factor for shear.*

(3) *The buckling factor for shear k_τ is given by the following:*

- *For webs with transverse stiffeners at the supports but no intermediate transverse stiffeners:*

  $k_\tau = 5{,}34$

[18] In preparation.

- *For webs with transverse stiffeners at the supports and intermediate transverse stiffeners with $a/d < 1$:*

  $k_\tau = 4 + 5{,}34/(a/d)^2$

- *For webs with transverse stiffeners at the supports and intermediate transverse stiffeners with $a/d \geq 1$:*

  $k_\tau = 5{,}34 + 4/(a/d)^2$

5.6.4 Tension field method

5.6.4.1 *Shear buckling resistance*

(1) *In the tension field method, the design shear buckling resistance $V_{bb.Rd}$ should be obtained from:*

$$V_{bb.Rd} = [(d\, t_w\, \tau_{bb}) + 0{,}9\,(g\, t_w\, \sigma_{bb} \sin \phi)]/\gamma_{M1} \qquad (5.59)$$

where σ_{bb} *is the strength of the tension field, obtained from:*

$$\sigma_{bb} = [f_{yw}^{\,2} - 3\tau_{bb}^{\,2} + \psi^2]^{0,5} - \psi$$

in which ψ is $1{,}5\tau_{bb} \sin 2\phi$

where ϕ *is the inclination of the tension field*

g *is the width of the tension field, see Figure 5.6.1*

and τ_{bb} *is the initial shear buckling strength.*

(2) *The initial shear buckling strength τ_{bb} should be determined as follows: when $\bar{\lambda}_w \leq 0{,}8$:*

a) $\tau_{bb} = (f_{yw}/\sqrt{3})$

b) *when $0{,}8 < \bar{\lambda}_w < 1{,}25$:*

$$\tau_{bb} = [1 - 0{,}8\,(\bar{\lambda}_w - 0{,}8)]\,(f_{yw}/\sqrt{3})$$

c) *when $\bar{\lambda}_w \geq 1{,}25$:*

$$\tau_{bb} = [1/\bar{\lambda}_w^{\,2}]\,(f_{yw}/\sqrt{3})$$

In which $\bar{\lambda}_w$ is as given in **5.6.3***(2).*

(3) *The width of the tension field g is given by:*

$$g = d \cos \phi - (a - s_c - s_t) \sin \phi$$

where s_c and s_t are the anchorage lengths of the tension field along the compression and tension flanges respectively, obtained from:

$$s = \frac{2}{\sin \phi}\left[\frac{M_{Nf.Rk}}{t_w\, \sigma_{bb}}\right]^{0,5} \quad \text{but } s \leq a$$

where $M_{Nf.Rk}$ is the reduced plastic resistance moment of the flange.

(4) *When calculating the plastic resistance moment of a flange, any lips or flange stiffeners should be neglected. The reduced plastic resistance moment $M_{Nf.Rk}$ allowing for the longitudinal force $N_{f.Sd}$ in the flange (due to the moment M_{Sd} and any axial force N_{Sd} in the member), is given by:*

$$M_{Nf.Rk} = 0{,}25\, b t_f^{\,2} f_{yf}\, [1 - [N_{f.Sd}/(b t_f\, f_{yf}/\gamma_{M0})]^2] \qquad (5.60)$$

where b and t_f are the width and thickness of the relevant flange.

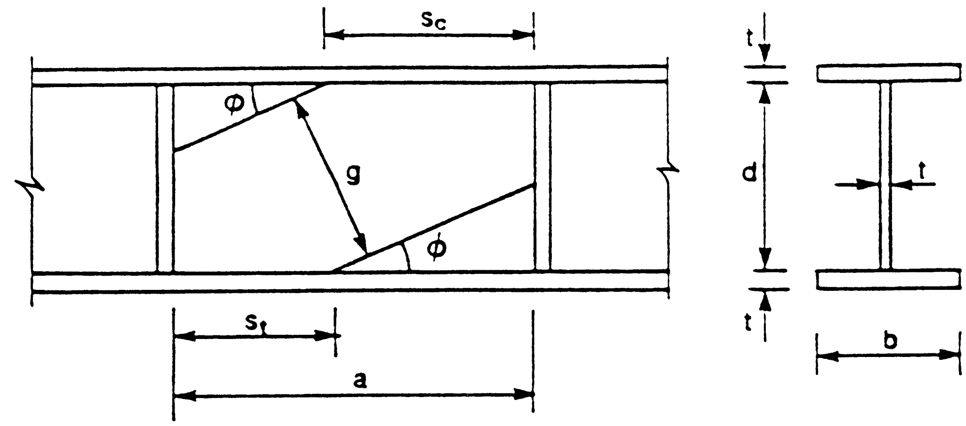


Figure 5.6.1 — Tension field geometry

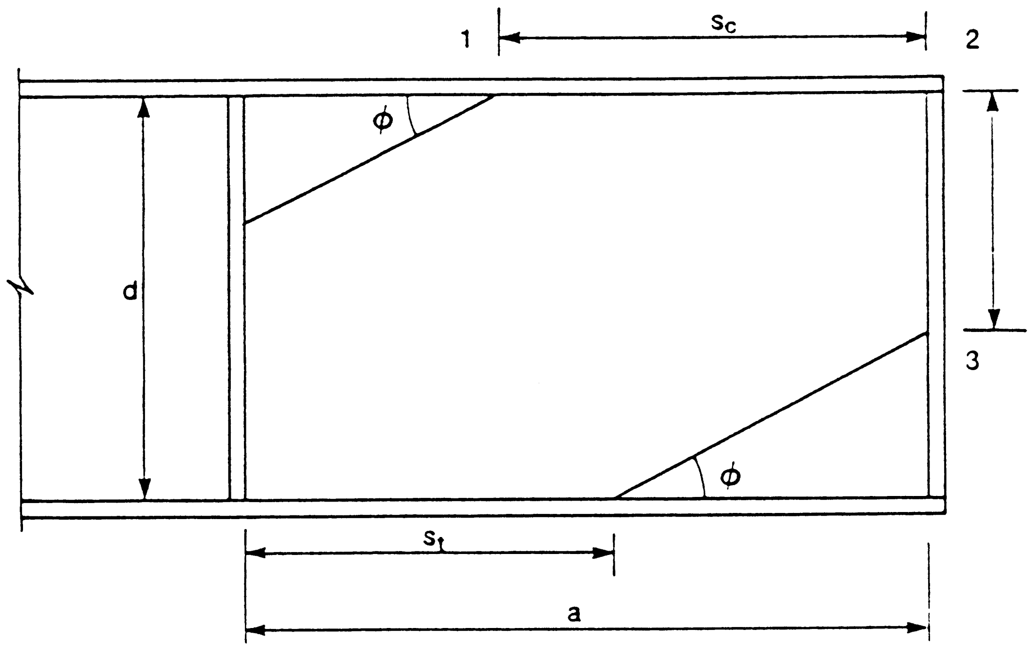


Figure 5.6.2 — End panel geometry

5.6.4.2 ***Inclination of the tension field***

(1) *The inclination of the tension field ϕ varies between a minimum of $\Theta/2$ and a maximum of Θ where Θ is the slope of the panel diagonal given by:*

$\Theta = \arctan(d/a)$

(2) *The minimum value $\Theta/2$ applies when the flanges are fully utilised in resisting the bending moment in the member. The maximum value of Θ applies to the complete tension field condition with $s = a$.*

(3) *The appropriate value of ϕ in any other case is the value (between the limits $\Theta/2$ and Θ), which gives the maximum value of the design shear buckling resistance $V_{bb.Rd}$.*

(4) *Any other value of ϕ (between the limits $\Theta/2$ and Θ) is conservative. As an approximation $\phi = \Theta/1,5$ may be assumed. Alternatively, iteration may be used to find the optimum value of ϕ.*

5.6.4.3 ***End panels***

(1) *Unless a suitable end post is supplied to anchor the tension field, end panels should be designed using the simple post-critical method given in* ***5.6.3***.

(2) *When a suitable end post which satisfies the criterion given in (4) is used, the design shear buckling resistance should be determined as given in* ***5.6.4.1***, *except that the anchorage length s_c should be obtained from (3), see Figure 5.6.2.*

(3) *When a single plate end post of breadth b_s and thickness t_s is used, the anchorage length s_c should be determined from:*

$$s_c = \frac{2}{\sin\phi}\left[\frac{M_{p\ell.1} + M_{p\ell.2}}{2t_w\,\sigma_{bb}}\right]^{0,5} \quad \text{but } s_c \leq a \tag{5.61}$$

where $M_{p\ell.1} = 0{,}25bt_f^2 f_{yf}\,[1 - [N_{f1}/(bt_f f_{yf})]^2]$

$N_{f1} = g\,t_w\,\sigma_{bb}\cos\phi$

$M_{p\ell.2}$ *is the lesser of* M_{Nf} *and* M_{Ns}

$M_{Nf} = 0{,}25\,bt_f^2 f_{yf}\,[1 - [F_{bb}/(bt_f f_{yf})]^2]$

$M_{Ns} = 0{,}25\,b_s\,t_s^2 f_{ys}\,[1 - [N_{s2}/(b_s\,t_s\,f_{ys})]^2]$

$F_{bb} = t_w\,s_s\,\sigma_{bb}\cos^2\phi$

$N_{s2} = t_w\,s_c\,\sigma_{bb}\sin^2\phi$

and $s_s = d - (a - s_t)\tan\phi$

(4) *A single plate end post required to resist the tension field anchorage force F_{bb} should satisfy the criterion:*

$$M_{p\ell.2} + M_{p\ell.3} \geq 0{,}5F_{bb}s_s \tag{5.62}$$

where $M_{p\ell.3} = 0{,}25\,b_st_s^2 f_{ys}\,[1 - [N_{s3}/(b_st_sf_{ys})]^2]$

and $N_{s3} = V_{Sd} - \tau_{bb}t_w\,(d - s_s)$

(5) *If an end post does not satisfy the criterion in (4), an increased value of ϕ may be adopted such that the anchorage length s_s is reduced sufficiently for the criterion to be satisfied, provided that a reduced value of the shear buckling resistance is then determined for the end panel corresponding to this increased value of ϕ.*

5.6.4.4 ***End post details***

(1) *The welds connecting the end post to the top flange should be designed to resist $M_{p\ell.2}$, F_{bb} and N_{s2}.*

(2) *A twin-stiffener type of end post may be used as an alternative to the single plate type shown in Figure 5.6.2, provided that the design expressions given in* ***5.6.4.3*** *are adjusted accordingly.*

5.6.5 Intermediate transverse stiffeners

(1) *For both the simple post-critical method and the tension field method, the compression force N_s in an intermediate transverse stiffener should be obtained from:*

$$N_s = V_{Sd} - d\,t_w\,\tau_{bb}/\gamma_{M1} \quad \text{but } N_s \geq 0 \tag{5.63}$$

in which τ_{bb} is the initial shear buckling strength from ***5.6.4.1****(2); the lower value of τ_{bb} for the two panels adjacent to the stiffener should be used.*

(2) *The buckling resistance of the stiffeners should be determined as specified in* ***5.7.6***.

(3) *The second moment of area of an intermediate transverse stiffener should satisfy the following:*

if $a/d < \sqrt{2}$:

$$I_s \geq 1{,}5d^3t_w^3/a^2 \tag{5.64}$$

if $a/d \geq \sqrt{2}$:

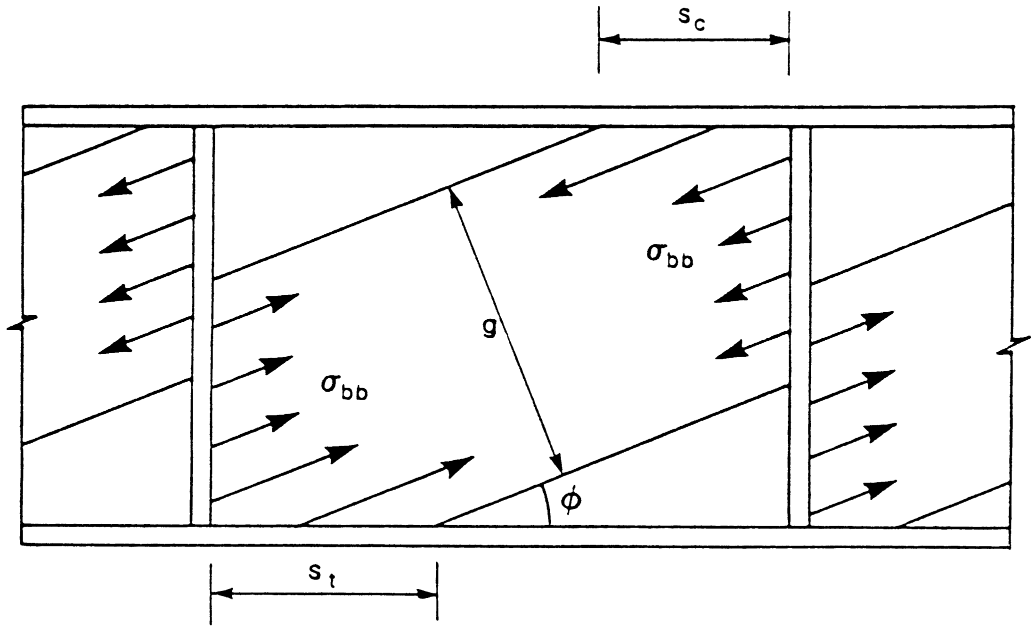
$$I_s \geq 0{,}75dt_w^3 \tag{5.65}$$

5.6.6 Welds

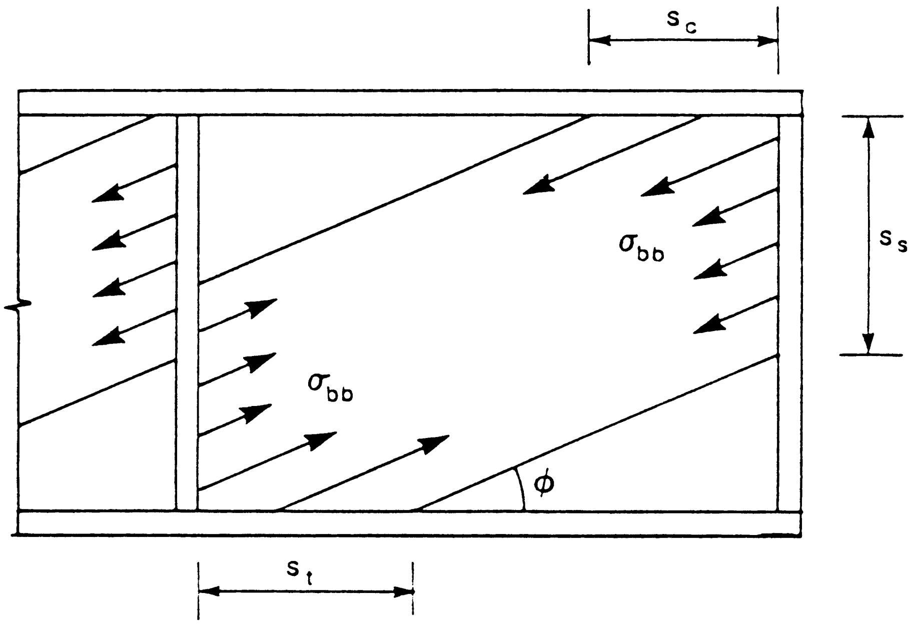
(1) *The forces used to check the web-to-flange welds shall be compatible with the stress fields in the web panels according to the method used to determine the shear buckling resistance.*

(2) *The design of the web-to-stiffener welds should also be consistent with the design assumptions for the web panels.*

(3) *The tensile stresses in the web panels for the tension field method are indicated in Figure 5.6.3.*



(a) Internal panel



(b) End panel

Figure 5.6.3 — Tension field stresses

5.6.7 Interaction between shear force, bending moment and axial force

5.6.7.1 *General*

(1) *Provided that the flanges can resist the whole of the design values of the bending moment and axial force in the member, the design shear resistance of the web need not be reduced to allow for the moment and axial force in the member, except as given in **5.6.4.1**(4) for the tension field method.*

(2) *For the procedure in other cases, refer to:*

- ***5.6.7.2** for the simple post-critical method.*
- ***5.6.7.3** for the tension field method.*

5.6.7.2 *Simple post-critical method*

(1) *The cross-section may be assumed to be satisfactory, without investigating the effect of the shear force on the design moment resistance, if both the following criteria are satisfied:*

$$M_{Sd} \leq M_{f.Rd} \tag{5.66a}$$

and $$V_{Sd} \leq V_{ba.Rd} \tag{5.66b}$$

where $M_{f.Rd}$ *is the design plastic moment resistance of a cross-section consisting of the flanges only, taking account of the effective width* b_{eff} *of the compression flange, see **5.3.5**.*

and $V_{ba.Rd}$ *is the design shear buckling resistance from **5.6.3**.*

When an axial force N_{Sd} *is also applied, the value of* $M_{f.Rd}$ *should be reduced accordingly, see **5.4.8**.*

(2) *Provided that* V_{Sd} *does not exceed 50 % of* $V_{ba.Rd}$ *the design resistance of the cross-section to bending moment and axial force need not be reduced to allow for the shear force.*

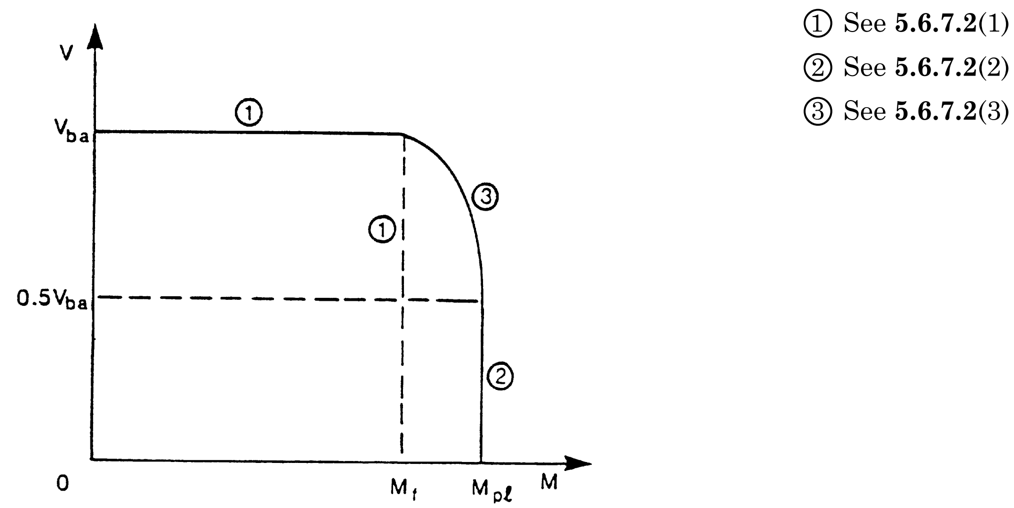
(3) *When* V_{Sd} *exceeds 50 % of* $V_{ba.Rd}$ *the following criterion should be satisfied:*

$$M_{Sd} \leq M_{f.Rd} + (M_{p\ell.Rd} - M_{f.Rd})\,[1 - (2\,V_{Sd}/V_{ba.Rd} - 1)^2] \tag{5.67}$$

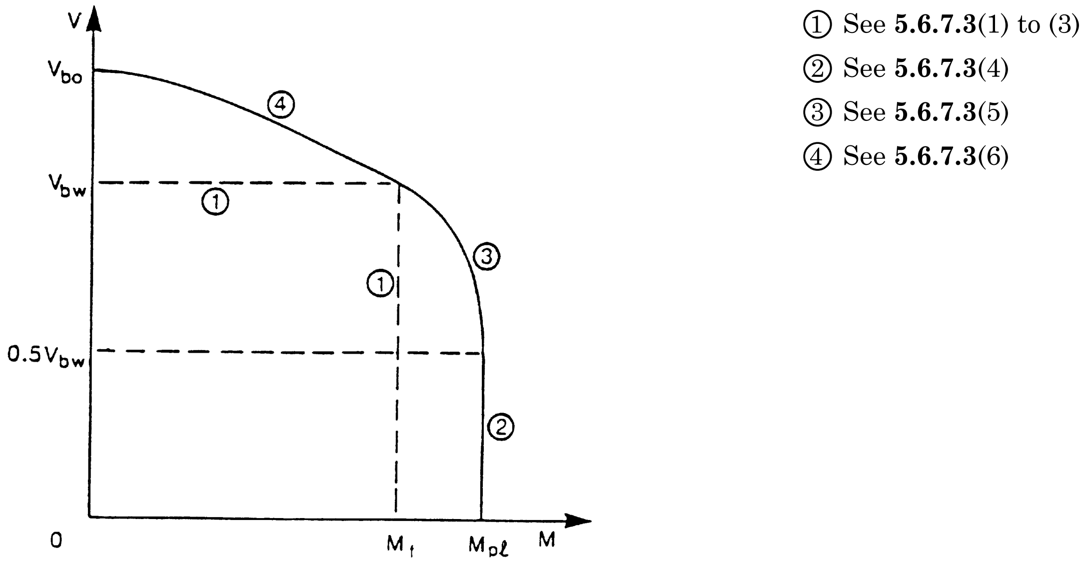
If an axial force N_{Sd} *is also applied, then* $M_{p\ell.Rd}$ *should be replaced by the reduced plastic resistance moment* $M_{N.Rd}$ *(see **5.4.8**).*

NOTE Paragraph (3) applies to Class 1, 2, 3 and 4 cross-sections, provided that the design resistance appropriate for that class of cross-section in the absence of shear force is not exceeded.

(4) *The interaction between shear force and bending moment is illustrated in Figure 5.6.4(a).*



(a) Simple post-critical method



(b) Tension field method

Figure 5.6.4 — Interaction of shear buckling resistance and moment resistance

5.6.7.3 *Tension field method*

(1) *The cross-section may be assumed to be satisfactory, without investigating the effect of the shear force on the design moment resistance, if both the following criteria are satisfied:*

$$M_{Sd} \leq M_{f.Rd} \tag{5.68a}$$

and

$$V_{Sd} \leq V_{bw.Rd} \tag{5.68b}$$

where M_{sd} *and* V_{Sd} *are each taken as the maximum respective value within the panel between adjacent transverse web stiffeners*

$M_{f.Rd}$ *is as given in* ***5.6.7.2(1)***

and $V_{bw.Rd}$ *is the "web only" shear buckling resistance.*

When an axial force N_{Sd} *is also applied, the value of* $M_{f.Rd}$ *should be reduced accordingly, see* ***5.4.8***.

(2) *The "web only" shear buckling resistance* $V_{bw.Rd}$ *is the specific value of* $V_{bb.Rd}$ *from* ***5.6.4*** *for the case where the flanges are resisting a moment* M_{Sd} *equal to* $M_{f.Rd}$ *and consequently in* ***5.6.4.1****(4) the reduced plastic resistance moment of the flange* $M_{Nf.Rk} = 0$.

(3) *For a section with equal flanges and no axial force,* $V_{bw.Rd}$ *should be calculated assuming:*

$s_c = s_t = 0$

and $\phi = \Theta/2$

(4) *Provided that* V_{Sd} *does not exceed 50 % of* $V_{bw.Rd}$ *the design resistance of the cross-section to bending moment and axial force need not be reduced to allow for the shear force.*

(5) *When* V_{Sd} *exceeds 50 % of* $V_{bw.Rd}$ *but does not exceed* $V_{bw.Rd}$ *the following criterion should be satisfied:*

$$M_{Sd} \leq M_{f.Rd} + (M_{pt.Rd} - M_{f.Rd})\,[1 - (2V_{Sd}/V_{bw.Rd} - 1)^2] \qquad (5.69)$$

When an axial force N_{Sd} *is also applied, then* $M_{p\ell.Rd}$ *should be replaced by the reduced plastic resistance moment* $M_{N.Rd}$ *(see* ***5.4.8****).*

NOTE Paragraph (5) applies to Class 1, 2, 3 and 4 cross-sections, provided that the design resistance appropriate for that class of cross-section in the absence of shear force is not exceeded.

(6) *When* V_{Sd} *exceeds* $V_{bw.Rd}$ *the following criterion should be satisfied:*

$$V_{Sd} \leq V_{bb.Rd} \qquad (5.70)$$

where $V_{bb.Rd}$ *is obtained from* ***5.6.4.1****, taking account of* M_{Sd} *and* N_{Sd} *in* ***5.6.4.1****(4).*

(7) *The interaction between shear force and bending moment is illustrated in Figure 5.6.4(b).*

(8) *In this figure,* $V_{bo.Rd}$ *is the specific value of* $V_{bb.Rd}$ *for the case where* $M_{Sd} = 0$.

5.7 Resistance of webs to transverse forces

5.7.1 Basis

(1) The resistance of an unstiffened web to transverse forces applied through a flange, is governed by one of the following modes of failure:

- crushing of the web close to the flange, accompanied by plastic deformation of the flange,
- crippling of the web in the form of localised buckling and crushing of the web close to the flange, accompanied by plastic deformation of the flange.
- buckling of the web over most of the depth of the member

(2) A distinction is made between two types of load application, as follows:

- Forces applied through one flange and resisted by shear forces in the web, see Figure 5.7.1(a).
- Forces applied to one flange and transferred through the web directly to the other flange, see Figure 5.7.1(b).

(3) *Where forces are applied through one flange and resisted by shear forces in the web, the resistance of the web to transverse forces should be taken as the smaller of:*

- *the crushing resistance (see* ***5.7.3****).*
- *the crippling resistance (see* ***5.7.4****).*

(4) *Where forces are applied to one flange and transferred through the web directly to the other flange, the resistance of the web to transverse forces should be taken as the smaller of:*

- *the crushing resistance (see* ***5.7.3****).*
- *the buckling resistance (see* ***5.7.5****).*

(5) *Where, in a practical case, the details are such that there is doubt over which mode governs, all three modes should be considered.*

(6) *In addition the effect of the transverse force on the moment resistance of the member should be considered, see* ***5.3.6*** *and* ***5.4.10****.*

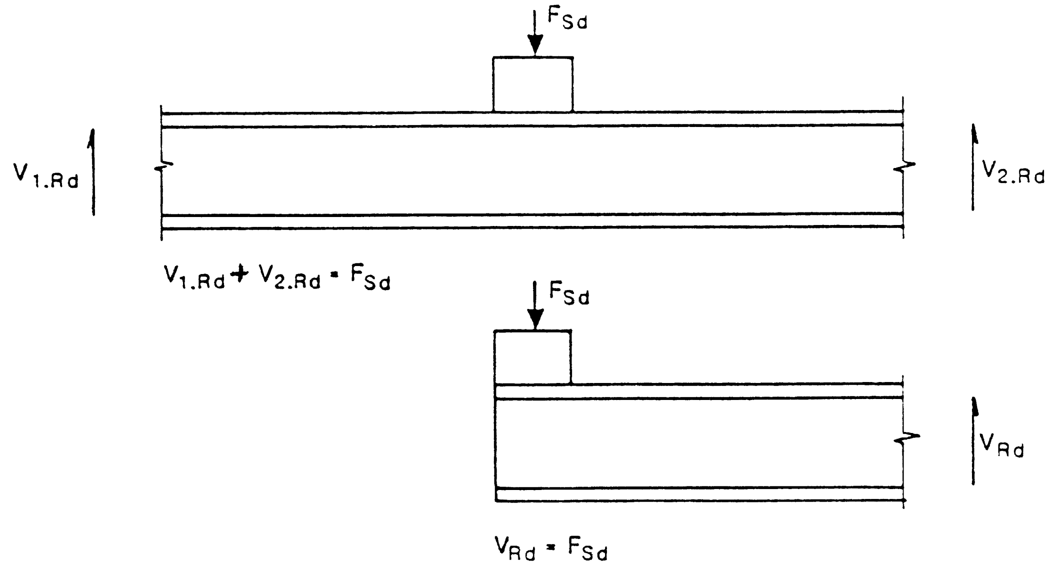
(7) *The crippling resistance of a stiffened web between the locations of transverse web stiffeners, is basically similar to that of an unstiffened web, with some increase due to the presence of the stiffeners.*

5.7.2 Length of stiff bearing

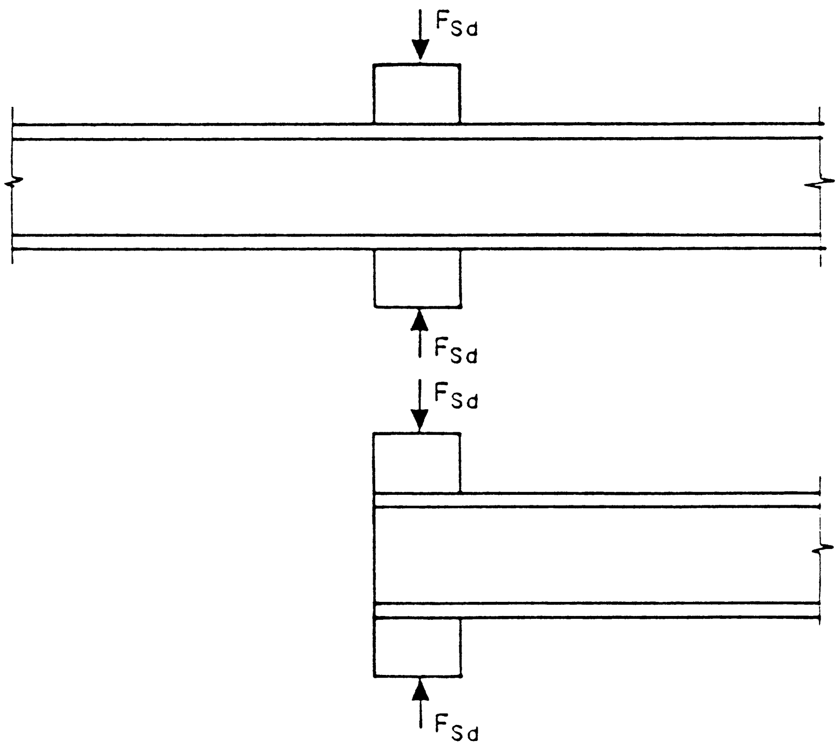
(1) The length of stiff bearing on the flange is the distance over which the applied force is effectively distributed.

(2) The resistance of the web to transverse forces is influenced by the length of stiff bearing.

(3) *The length of stiff bearing s_s should be determined by dispersion of load through solid steel material which is properly fixed in place at a slope of 1 : 1, see Figure 5.7.2. No dispersion should be taken through loose packs.*



(a) Forces resisted by shear in the web



(b) Forces transmitted directly through the web

Figure 5.7.1 — Forces applied through a flange

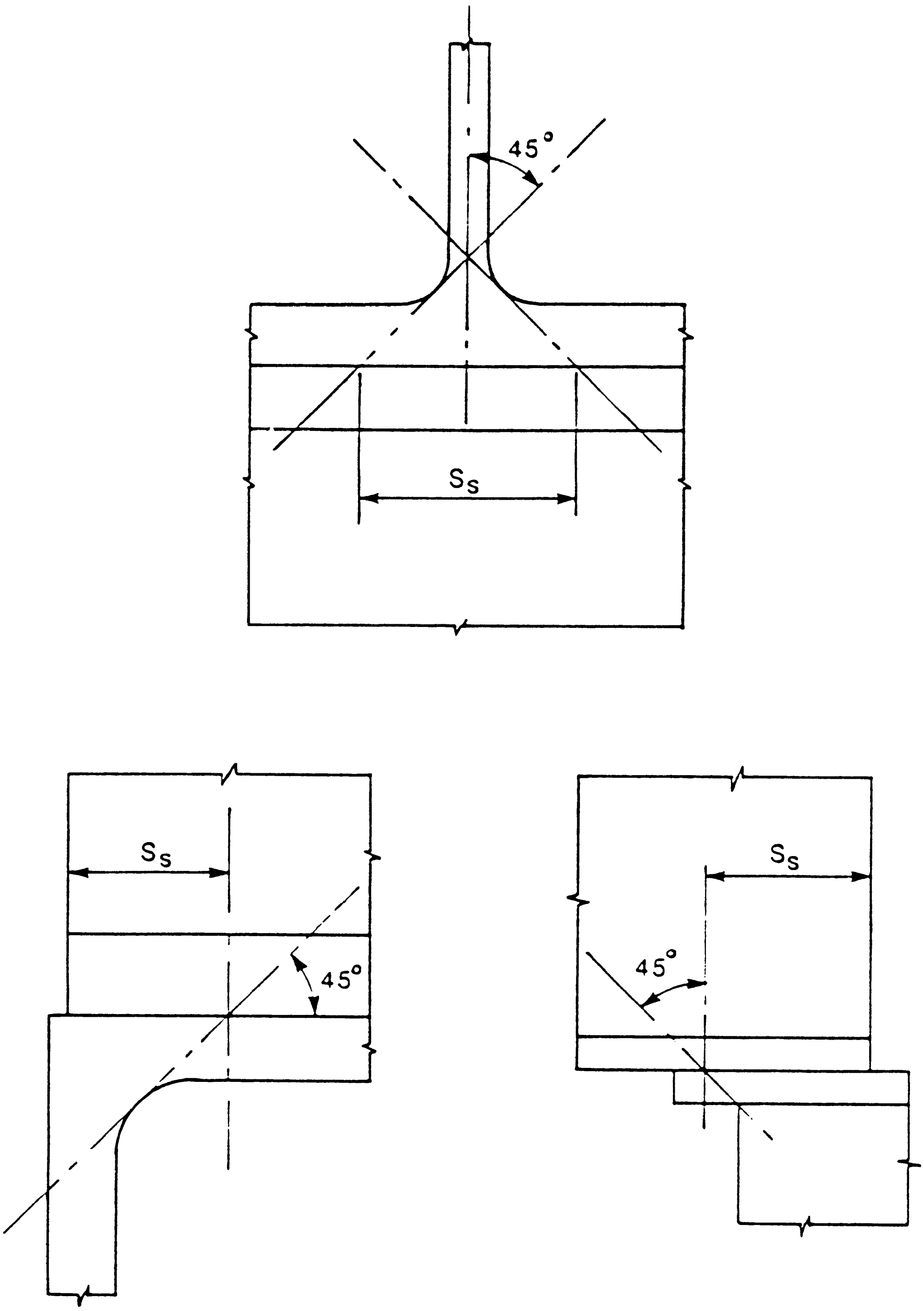


Figure 5.7.2 — Length of stiff bearing

5.7.3 Crushing resistance

(1) *The design crushing resistance $R_{y.Rd}$ of the web of an I, H or U section should be obtained from:*

$$R_{y.Rd} = (s_s + s_y)\, t_w\, f_{yw}/\gamma_{M1} \tag{5.71}$$

in which s_y is given by:

$$s_y = 2t_f\,(b_f/t_w)^{0,5}\,[f_{yf}/f_{yw}]^{0,5}\,[1-(\sigma_{f.Ed}/f_{yf})^2]^{0,5} \tag{5.72}$$

but b_f should not be taken as more than $25t_f$

where $\sigma_{f.Ed}$ is the longitudinal stress in the flange.

(2) *For a rolled I, H or U section s_y may alternatively be obtained from:*

$$s_y = \frac{2{,}5\,(h-d)\,[1-(\sigma_{f.Ed}/f_{yf})^2\,]^{0,5}}{(1+0{,}8\,s_s/(h-d))} \tag{5.73}$$

(3) *At the end of a member s_y should be halved.*

(4) *For wheel loads from cranes, transmitted through a crane rail bearing on a flange but not welded to it, the design crushing resistance of the web $R_{y.Rd}$ should be taken as:*

$$R_{y.Rd} = s_y\, t_w\, f_{yw}/\gamma_{M1} \tag{5.74}$$

in which:

$$s_y = k_R\left[\frac{I_f + I_R}{t_w}\right]^{1/3}\,[1-(\sigma_{f.Ed}/f_{yf})^2\,]^{0,5} \tag{5.75}$$

or more approximately:

$$s_y = 2(h_R + t_f)\,[1-(\sigma_{f.Ed}/f_{yf})^2]^{0,5} \tag{5.76}$$

where h_R *is the height of the crane rail*

I_f *is the second moment of area of the flange about its horizontal centroidal axis*

I_R *is the second moment of area of the crane rail about its horizontal centroidal axis*

and k_R *is a constant taken as follows:*

- *when the crane rail is mounted directly on the flange, $k_R = 3{,}25$*
- *when a suitable resilient pad not less than 5 mm thick is interposed between the crane rail and the beam flange: $k_R = 4{,}0$*

5.7.4 Crippling resistance

(1) *The design crippling resistance $R_{a.Rd}$ of the web of an I, H or U section should be obtained from:*

$$R_{a.Rd} = 0{,}5t_w^2(Ef_{yw})^{0,5}\,[(t_f/t_w)^{0,5} + 3(t_w/t_f)(s_s/d)]/\gamma_{M1} \tag{5.77}$$

where s_s *is the length of stiff bearing from* ***5.7.2****(3)*

but s_s/d *should not be taken as more than 0,2.*

(2) *Where the member is also subject to bending moments, the following criteria should be satisfied:*

$$F_{Sd} \le R_{a.Rd} \tag{5.78a}$$

$$M_{Sd} \le M_{c.Rd} \tag{5.78b}$$

and $$\frac{F_{Sd}}{R_{a.Rd}} + \frac{M_{Sd}}{M_{c.Rd}} \le 1{,}5 \tag{5.78c}$$

5.7.5 Buckling resistance

(1) *The design buckling resistance $R_{b.Rd}$ of the web of an I, H or U section should be obtained by considering the web as a virtual compression member with an effective breadth b_{eff} obtained from:*

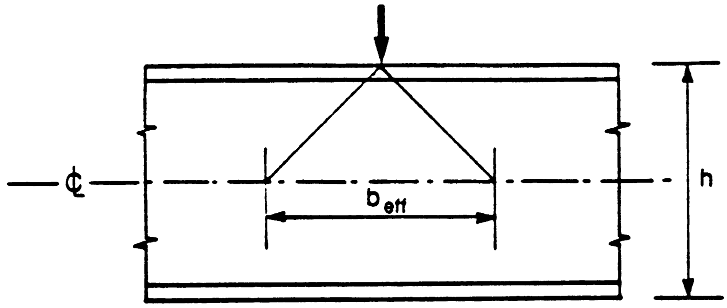
$$b_{eff} = [h^2 + s_s^2]^{0,5} \tag{5.79}$$

(2) *Near the ends of a member (or at openings in the web) the effective breadth b_{eff} should not be taken as greater than the breadth actually available, measured at mid-depth, see Figure 5.7.3.*

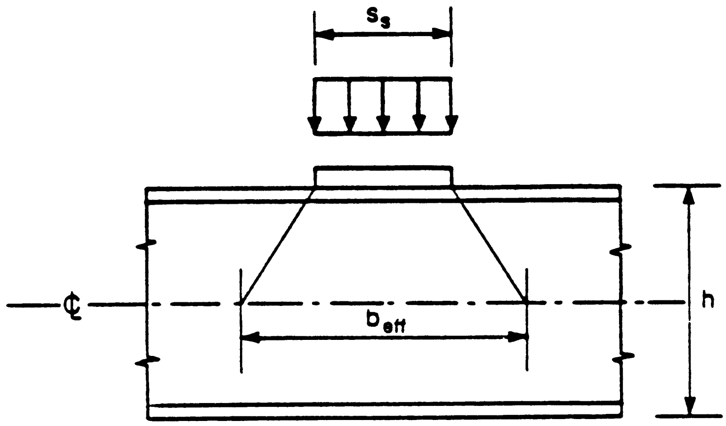
(3) *The buckling resistance should be determined from* ***5.5.1*** *using buckling curve c and $\beta_A = 1$.*

(4) *The buckling length of the virtual compression member should be determined from the conditions of lateral and rotational restraint at the flanges at the point of load application.*

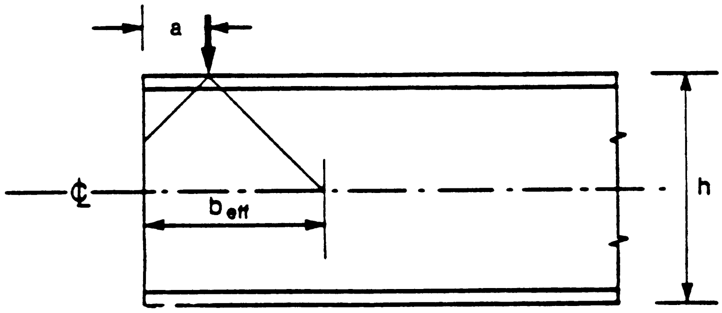
(5) *The flange through which the load is applied should normally be restrained in position at the point of load application. Where this is not practicable, a special buckling investigation should be carried out.*



$b_{eff} = h$

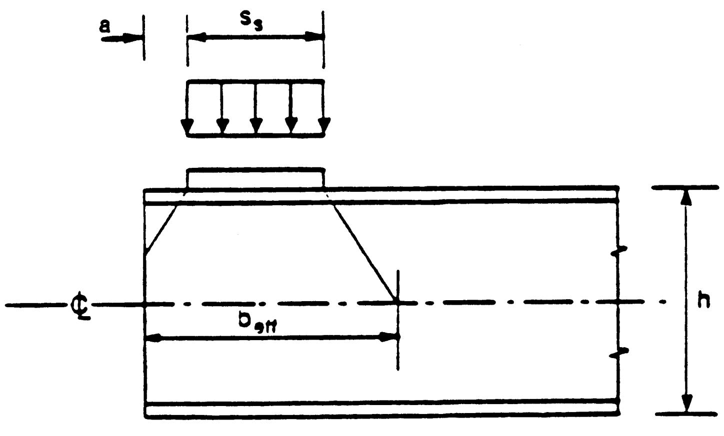


$b_{eff} = \left[h^2 + s_s^2\right]^{1/2}$



$b_{eff} = \dfrac{h}{2} + a$

but $b_{eff} \leq h$



$b_{eff} = \dfrac{1}{2}\left[h^2 + s_s^2\right]^{1/2} + a + \dfrac{s_s}{2}$

but $b_{eff} \leq \left[h^2 + s_s^2\right]^{1/2}$

Figure 5.7.3 — Effective breadth for web buckling resistance

5.7.6 Transverse stiffeners

(1) *When checking the buckling resistance, the effective cross-section of a stiffener should be taken as including a width of web plate equal to 30εt_w, arranged with 15εt_w each side of the stiffener, see Figure 5.7.4. At the ends of the member (or openings in the web) the dimension of 15εt_w should be limited to the actual dimension available.*

(2) *The out-of-plane buckling resistance should be determined from* ***5.5.1****, using buckling curve c and a buckling length ℓ of not less than 0,75d, or more if appropriate for the conditions of restraint.*

(3) *End stiffeners and stiffeners at internal supports should normally be double sided and symmetric about the centreline of the web.*

(4) *Stiffeners at locations where significant external forces are applied should preferably be symmetric.*

(5) *Where single sided or other asymmetric stiffeners are used, the resulting eccentricity should be allowed for, using clause* ***5.5.4****.*

(6) *In addition to checking the buckling resistance, the cross-section resistance of a load bearing stiffener should also be checked adjacent to the loaded flange. The width of web plate included in the effective cross-section should be limited to s_y (see* ***5.7.3****) and allowance should be made for any openings cut in the stiffener to clear the web-to-flange welds.*

(7) *For intermediate transverse stiffeners it is only necessary to check the buckling resistance, provided that they are not subject to external loads.*

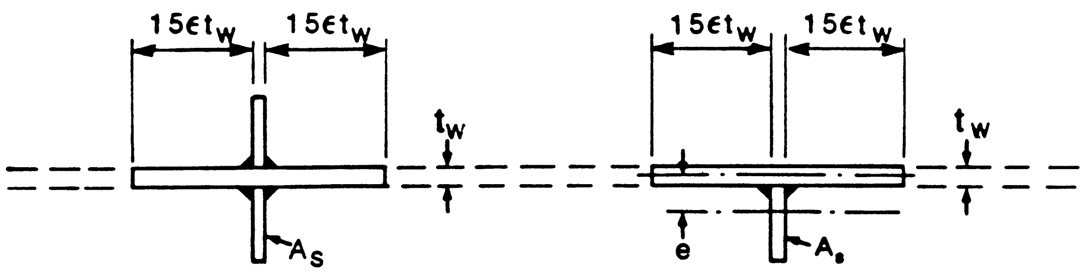


Figure 5.7.4 — Effective cross-section of stiffeners

5.7.7 Flange induced buckling

(1) To prevent the possibility of the compression flange buckling in the plane of the web, the ratio d/t_w of the web shall satisfy the following criterion:

$$d/t_w = k\,(E/f_{yf})[A_w/A_{fc}]^{0,5} \tag{5.80}$$

where A_w is the area of the web

A_{fc} is the area of the compression flange

and f_{yf} is the yield strength of the compression flange.

(2) *The value of the factor k should be taken as follows:*

Class 1 flanges: 0,3

Class 2 flanges: 0,4

Class 3 or Class 4 flanges: 0,55

(3) *When the girder is curved in elevation, with the compression flange on the concave face, the criterion should be modified to:*

$$d/t_w \le \frac{k(E/f_{yf})\ [A_w/A_{fc}]^{0,5}}{[1 + dE/(3rf_{yf})]^{0,5}} \tag{5.81}$$

where r is the radius of curvature of the compression flange.

(4) *When the girder has transverse web stiffeners, the limiting value of d/t_w may be increased accordingly.*

5.8 Triangulated structures

5.8.1 General

(1) Triangulated structures such as lattice girders and triangulated bracing subject to predominantly static loading may be analysed by assuming that the member ends are nominally pin-jointed.

(2) The buckling resistance of the compression members in such structures may be determined from **5.5.1** for compression members or **5.5.4** bending and axial compression. The buckling length may be determined from **5.8.2**. For built-up compression members, see section **5.9**.

(3) *For the design of angles as web members, see **5.8.3**.*

(4) *For the design of lattice towers and masts see ENV 1993-3 Eurocode 3-3*[19].

5.8.2 Buckling length of members

(1) For chord members generally and for out-of-plane buckling of web members, the buckling length ℓ shall be taken as equal to the system length L, unless a smaller value is justified by analysis.

(2) Web members may be designed for in-plane buckling using a buckling length smaller than the system length, provided that the chords supply appropriate end restraint and the end connections supply appropriate fixity (at least 2 bolts if bolted).

(3) *Under these conditions, in normal triangulated structures the buckling length ℓ of web members for in-plane buckling may be taken as 0,9L, except for angle sections.*

(4) *For angle sections used as web members in compression see **5.8.3**.*

5.8.3 Angles as web members in compression

(1) *Provided that the chords supply appropriate end restraint to the web members and the end connections of the web members supply appropriate fixity (at least two bolts if bolted), the eccentricities may be neglected and end fixities allowed for in the design of angles used as web members in compression, by using an effective slenderness ratio $\bar{\lambda}_{eff}$ obtained as follows:*

for buckling about the v-v axis: $$\bar{\lambda}_{eff.v} = 0{,}35 + 0{,}7\bar{\lambda}_v \tag{5.82}$$

for buckling about the y-y axis: $$\bar{\lambda}_{eff.y} = 0{,}50 + 0{,}7\bar{\lambda}_y \tag{5.83}$$

for buckling about the z-z axis: $$\bar{\lambda}_{eff.z} = 0{,}50 + 0{,}7\bar{\lambda}_z \tag{5.84}$$

*where $\bar{\lambda}$ is as defined in **5.5.1.2** and the axes are as defined in Figure 1.1.*

(2) *This modified slenderness ratio $\bar{\lambda}_{eff}$ should be used with buckling curve c in **5.5.1** to determine the buckling resistance.*

(3) *When only single bolts are used for end connections of angle web members or when the end connection has poor stiffness, the eccentricity should be taken into account using **5.5.4** and the buckling length ℓ should be taken as equal to the system length L.*

5.9 Built-up compression members

5.9.1 Basis

(1) Built-up compression members consisting of two or more main components connected together at intervals to form a single compound member shall be designed incorporating an equivalent geometric imperfection comprising an initial bow e_o of not less than $\ell/500$.

(2) The deformation of the compound member shall be taken into account in determining the internal forces and moments in the main components, internal connections and any subsidiary components such as lacings or battens.

(3) The design of the main and subsidiary components shall be checked using the methods given in **5.4** and **5.5**. The design of the internal connections shall be checked using Chapter 6.

(4) *The design procedures given in **5.9.2** to **5.9.5** apply only to built-up members with two main components, except where it is explicitly stated that they can be applied to members with more than two main components.*

[19] To be prepared at a later stage

(5) *In addition to the axial forces, allowance should also be made for any other forces or moments applied to the member, such as the effects of the self-weight or wind resistance of the member.*

5.9.2 Laced compression members

5.9.2.1 *Application*

(1) The design procedure given in this sub-clause is for a design compressive force N_{Sd} applied to a built-up member consisting of two similar parallel chords of uniform cross section, with a fully triangulated system of lacing which is uniform throughout the length of the member.

(2) The chords may be solid members or may themselves be laced or battened in the perpendicular plane.

(3) *Where variations on the above are necessary, the procedure should be supplemented or modified as appropriate.*

5.9.2.2 *Constructional details*

(1) Where possible, single lacing systems on opposite sides of the main components shall be corresponding systems as shown in Figure 5.9.1(a), arranged so that one is in the shadow of the other.

(2) Single lacing systems on opposite sides of the main components shall not be mutually opposed in direction as shown in Figure 5.9.1(b), unless the resulting torsional deformation of the main components can be accepted.

(3) Tie panels shall be provided at the ends of lacing systems, at points where the lacing is interrupted and at connections with other members.

(4) *Tie panels may take the form of battens conforming with **5.9.3.2**; alternatively cross braced panels of similar rigidity may be used.*

(5) Except for these tie panels, if other components perpendicular to the longitudinal axis of the member are combined with double intersection lacing systems [see Figure 5.9.2(a)], or single intersection lacing systems mutually opposed in direction on opposite sides of the main components [see Figure 5.9.2(b)], the resulting internal forces produced in the lacings due to the continuity of the main components shall be determined and allowed for in the design of the lacings and their end connections.

(6) The lacings shall be positively connected to the main components, either by fasteners or by welding.

5.9.2.3 *Second moment of area*

(1) *The effective second moment of area l_{eff} of a laced compression member with two main components should be taken as:*

$$l_{eff} = 0{,}5\, h_o^2\, A_f \tag{5.85}$$

where A_f *is the cross-sectional area of one chord*

and h_o *is the distance between centroids of chords.*

5.9.2.4 *Chord forces at mid-length*

(1) *The chord force $N_{f.Sd}$ at mid-length should be determined from:*

$$N_{f.Sd} = 0{,}5\, N_{Sd} + M_s/h_o \tag{5.86}$$

where $M_s = N_{Sd} e_o/(1 - N_{Sd}/N_{cr} - N_{Sd}/S_v)$

$e_o = \ell/500$ *(see **5.9.1**)*

$N_{cr} = \pi^2 E l_{eff} / \ell^2$

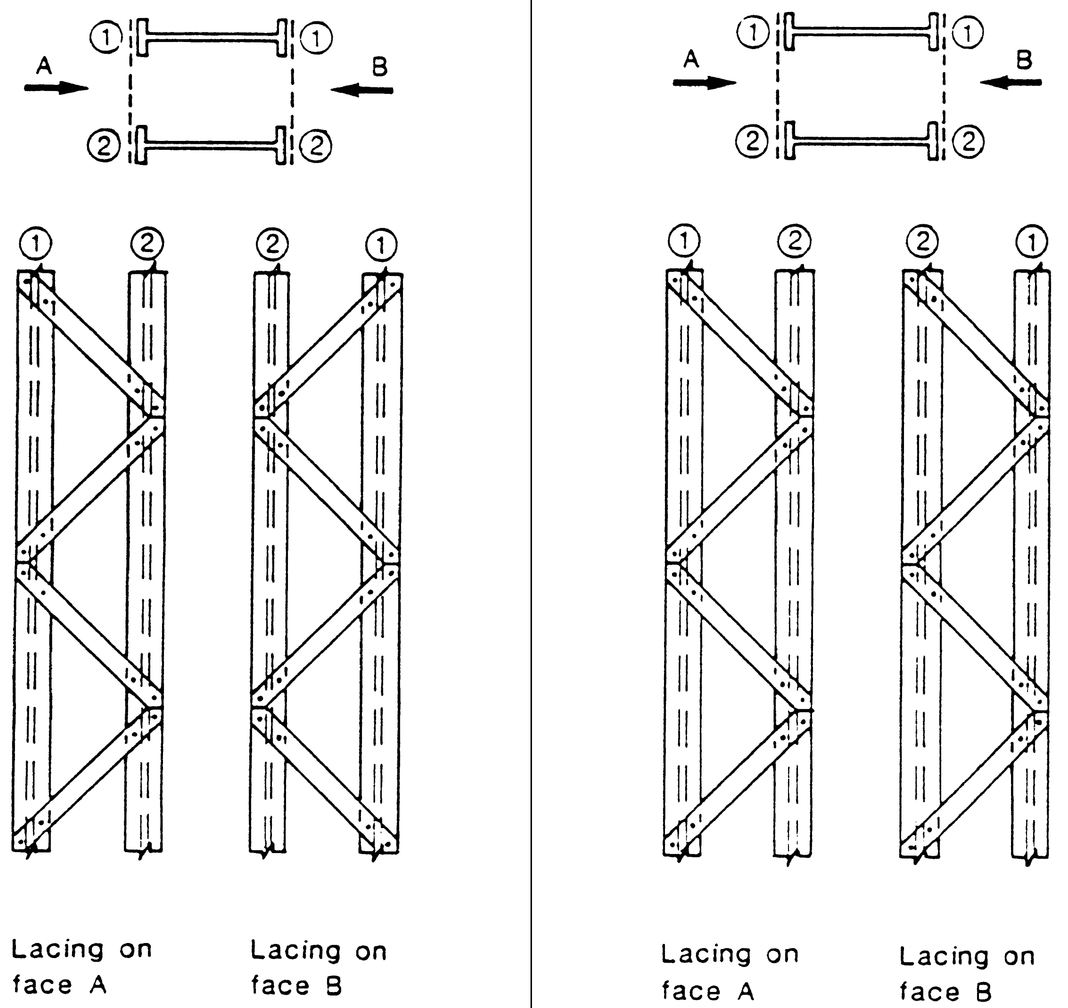
and S_v *is the shear stiffness of the lacings (the shear force required to produce unit shear deformation).*

(2) *Values of S_v for various lacing systems are given in Figure 5.9.3.*

5.9.2.5 *Buckling resistance of chords*

(1) *The buckling length of a chord in the plane of a lacing system should be taken as the system length a between lacing connections.*

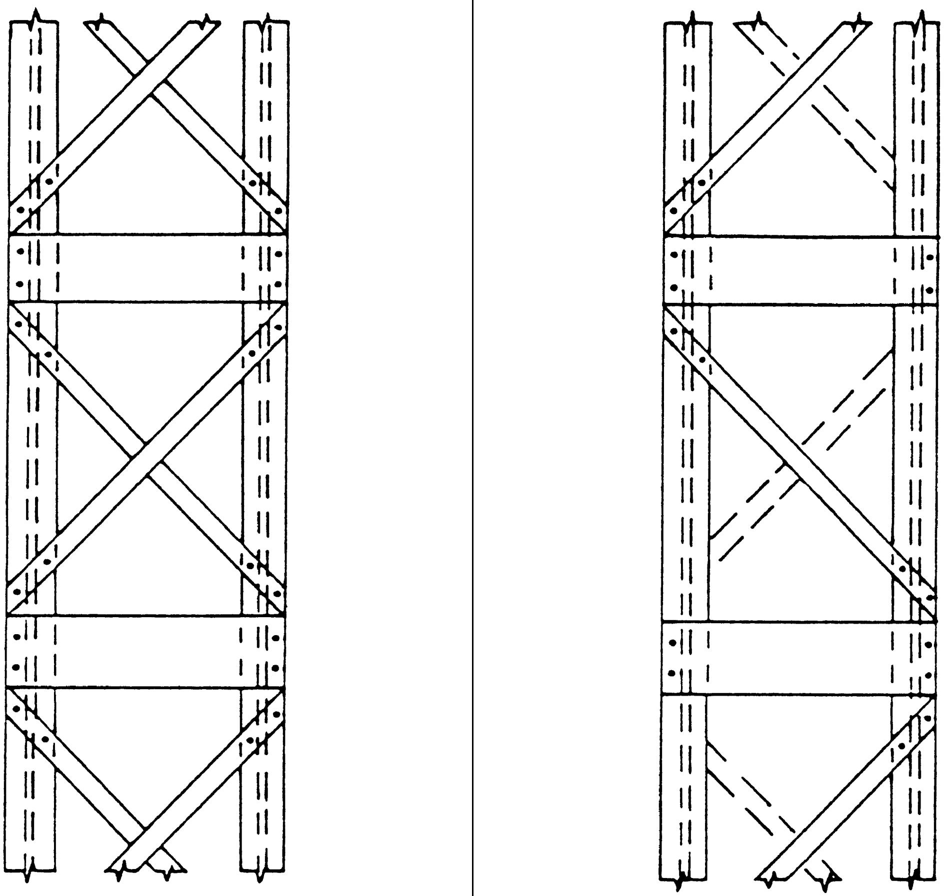
(2) *In a member with four chords made of equal-leg angles with lacing in both directions, the buckling length ℓ for buckling about the weakest axis depends on the arrangement of the lacings, see Figure 5.9.4.*



(a) Corresponding lacing systems
(Recommended system)

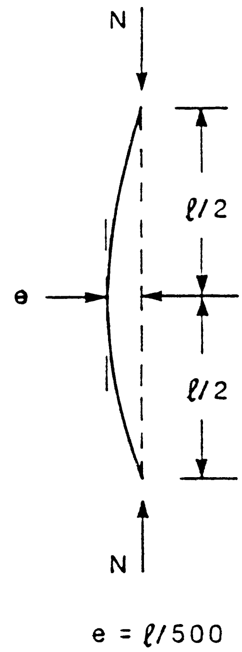
(b) Mutually opposed lacing systems
(Not recommended)

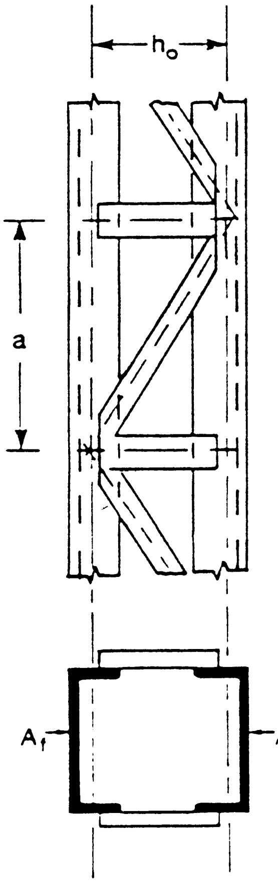
Figure 5.9.1 — Single lacing systems on opposite sides of main components

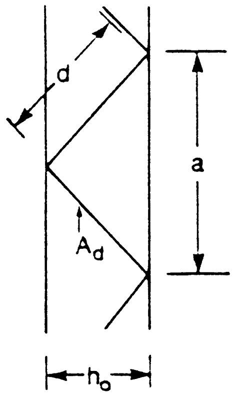
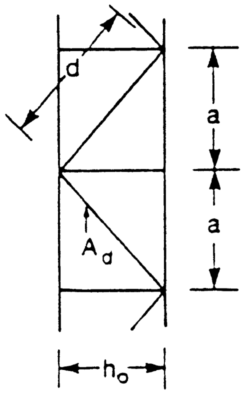
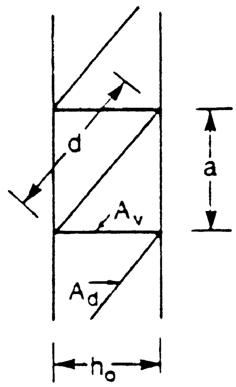


(a) Double intersection lacing system	(b) Mutually opposed lacing system (not recommended)

Figure 5.9.2 — Lacing systems combined with other components perpendicular to the longitudinal axis of the member





System	S_v
	$\dfrac{n\,EA_d a h_o^2}{2d^3}$
	$\dfrac{n\,EA_d a h_o^2}{d^3}$
	$\dfrac{n\,EA_d a h_o^2}{d^3\left[1+\dfrac{A_d h_o^3}{A_v d^3}\right]}$

n is the number of planes of lacings.

A_d and A_y refer to a single plane.

Figure 5.9.3 — Laced compression member

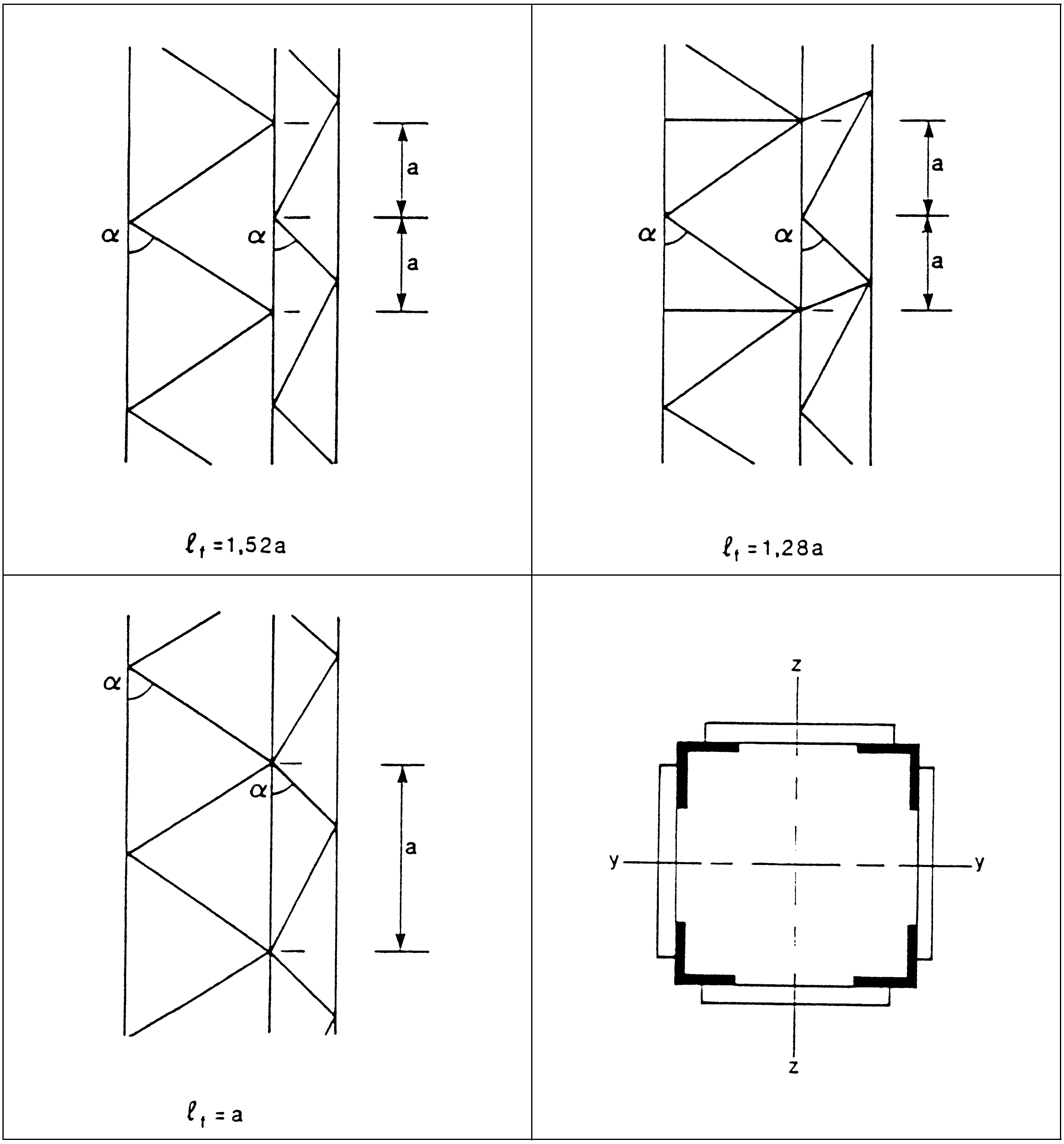


Figure 5.9.4 — Buckling lengths of angle chords in laced members

5.9.2.6 ***Lacing forces***

(1) *The lacing forces adjacent to the ends of the member should be derived from the internal shear force V_s taken as:*

$$V_s = \pi M_s/\ell \qquad (5.87)$$

with M_s from **5.9.2.4**

The force N_d in a diagonal lacing is given by:

$$N_d = \frac{V_s\, d}{n h_0} \qquad (5.88)$$

with d, n and h_0 from Figure 5.9.3.

5.9.3 Battened compression members

5.9.3.1 ***Application***

(1) The design procedure given in this sub-clause is for a design compressive force N_{Sd} applied to a built-up member consisting of two similar parallel chords of uniform cross-section, spaced apart and inter-connected by means of battens, which are rigidly connected to the chords and uniformly spaced throughout the length of the member.

(2) The chords may be solid members or may themselves be laced or battened in the perpendicular plane.

(3) *Where variations on the above are necessary the procedure should be supplemented or modified as appropriate.*

5.9.3.2 ***Constructional details***

(1) Battens shall be supplied at each end of the member.

(2) *Battens should also be supplied at intermediate points where loads are applied or lateral restraint is supplied.*

(3) *Intermediate battens should be supplied to divide the length of the member into at least 3 panels. There should be at least 3 panels between points which are taken as laterally restrained in the plane of the battens. As far as possible, the intermediate battens should be spaced and proportioned uniformly throughout the length of the member.*

(4) *Where parallel planes of battens are supplied, the battens in each plane should be arranged opposite each other.*

(5) *When S_v is evaluated disregarding the flexibility of the batten plates themselves [see* ***5.9.3.4****(3)], the width of an end batten along the length of the member should not be less than h_o, and the width of an intermediate batten should not be less than 0,5 h_o, where h_o is the distance between the centroids of the chords.*

(6) *Unless the flexibility of the batten plates is explicitly taken into account in the evaluation of S_v, the battens should also satisfy:*

$$\frac{n\, I_b}{h_o} \geq 10\, \frac{I_f}{a} \qquad (5.89)$$

where	I_b	*is the in-plane second moment of area of one batten*
	I_f	*is the in-plane second moment of area of one chord*
	h_o	*is the distance between centroids of chords*
	a	*is the system length between centrelines of battens*
and	n	*is the number of planes of battens.*

5.9.3.3 ***Second moment of area***

(1) *The effective in-plane second moment of area l_{eff} of a battened compression member with two main components should be taken as:*

$$l_{eff} = 0{,}5 h_o^2 A_f + 2\mu l_f \tag{5.90}$$

with μ obtained from the following:

$\lambda \leq 75$: $\quad \mu = 1$

$75 < \lambda < 150$: $\quad \mu = 2 - \lambda/75$

$\lambda \geq 150$: $\quad \mu = 0$

in which $\lambda = \ell/i_o$

where A_f *is the cross-sectional area of one chord*

l_f *is the second moment of area of one chord*

h_o *is the distance between centroids of chords*

i_o $= [0{,}5\, l_1/A_f]^{0,5}$

and l_f *is the value of l_{eff} with $\mu = 1$*

5.9.3.4 ***Chord forces at mid-length***

(1) *The chord force $N_{f.Sd}$ at mid-length should be determined from:*

$$N_{f.Sd} = 0{,}5(N_{Sd} + M_s h_o A_f / l_{eff}) \tag{5.91}$$

where $M_s = N_{Sd} e_o / (1 - N_{Sd}/N_{cr} - N_{Sd}/S_v)$

$e_o = \ell/500$ *(see **5.9.1**)*

$N_{cr} = \pi^2 E l_{eff} / \ell^2$

(2) *Provided that the criterion in **5.9.3.2(6)** is satisfied, the shear stiffness S_v should be taken as:*

$$S_v = 2\pi^2 E l_{eff} / a^2 \tag{5.92}$$

(3) *When the criterion in **5.9.3.2(6)** is not satisfied, the flexibility of the batten plates should be taken into account by obtaining S_v from:*

$$S_v = \frac{24 EI_f}{a^2 \left[1 + \dfrac{2 I_f}{n I_b} \cdot \dfrac{h_o}{a}\right]} \quad \text{but } S_v \leq \frac{2\pi^2 EI_f}{a^2} \tag{5.93}$$

5.9.3.5 ***Buckling resistance of chords***

(1) *The buckling length of a chord in the plane of the battens should be taken as the system length a between centrelines of battens.*

5.9.3.6 ***Moments and shears due to battening***

(1) *The battens, their connections to the chords and the chords themselves should be checked for the moments and forces in the end panel indicated in Figure 5.9.5, in which the internal shear force V_s is taken as:*

$$V_s = \pi M_s / \ell \tag{5.94}$$

*with M_s from **5.9.3.4**.*

(2) *For the purpose of this check, the axial force in each chord may be taken as $0{,}5N_{Sd}$ even when there are only three panels in the length of the member.*

(3) *In the case of chords with unsymmetric cross-sections (such as channels) the reduced plastic resistance moments for use in the expression given in **5.4.8.1(11)** may be taken as the mean of the values for positive and negative bending moments for the purpose of this check.*

5.9.4 Closely spaced built-up members

(1) *Built-up compression members such as those shown in Figure 5.9.6, with main components in contact or closely spaced and connected through packing plates, need not be treated as battened members provided that they are connected together by bolts or welds at a spacing of not more than 15* i_{min}, *where* i_{min} *is the minimum radius of gyration of a main component.*

(2) *The interconnecting bolts or welds should be designed to transmit the longitudinal shear between the main components derived from the internal shear force* V_s.

(3) V_s *may be taken as 2,5 % of the axial force in the member. Alternatively* V_s *may be determined as in* ***5.9.3.6***.

(4) *The longitudinal shear per interconnection may be taken as* $0{,}25 V_s\, a/i_{min}$ *where a is the system length of the main components centre-to-centre of interconnections.*

5.9.5 Star-battened angle members

(1) *Built-up compression members consisting of two similar angle members connected by pairs of battens in two perpendicular planes as shown in Figure 5.9.7, may be checked for buckling about the y-y axis as a single integral member, provided that the buckling lengths in the two perpendicular planes y-y and z-z are equal and provided that the spacing of pairs of battens is not more than 70* i_{min} *where* i_{min} *is the minimum radius of gyration of one angle.*

(2) *In the case of unequal-leg angles it may be assumed that:*

$$i_y = i_o / 1{,}15 \qquad (5.95)$$

where i_o *is the minimum radius of gyration of the built-up member.*

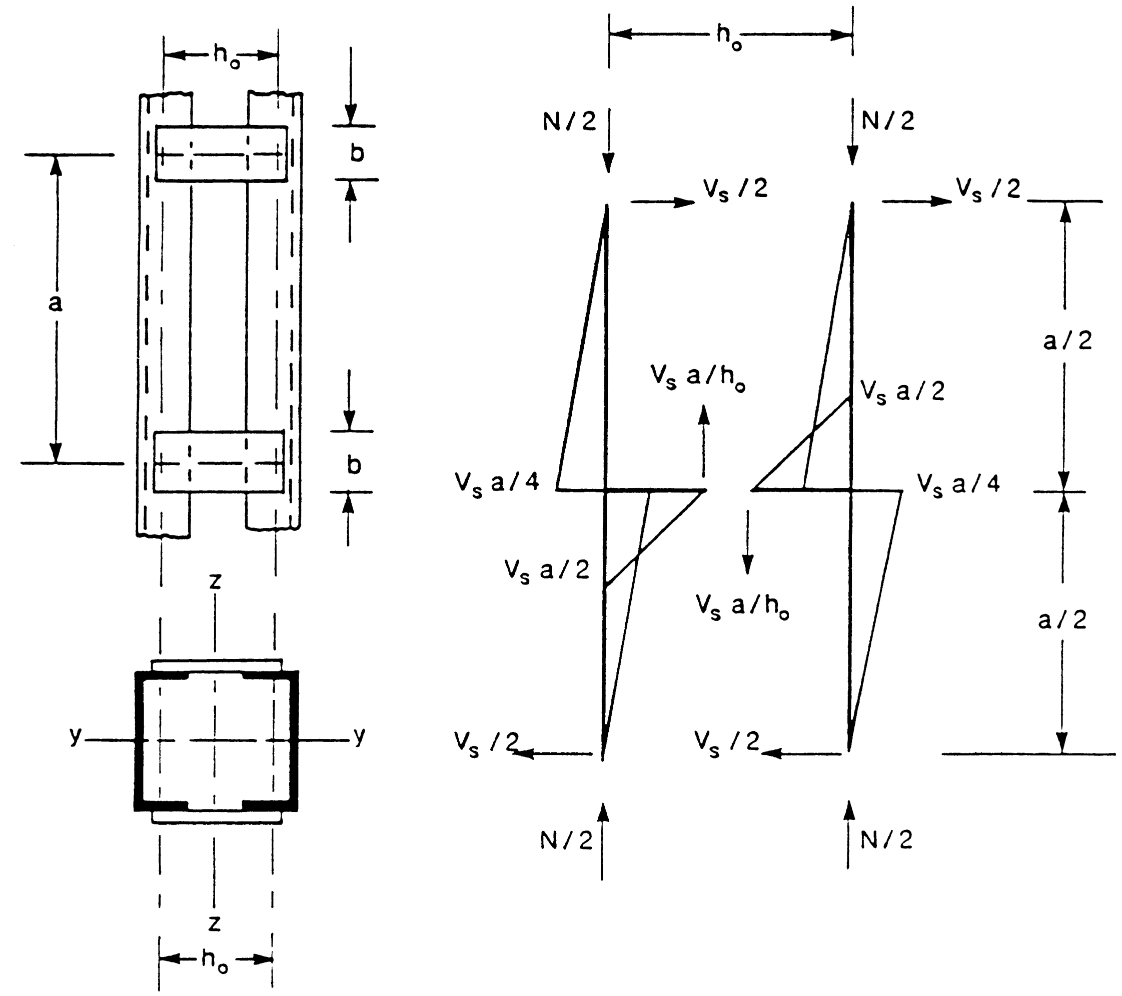


Figure 5.9.5 — Battened compression member

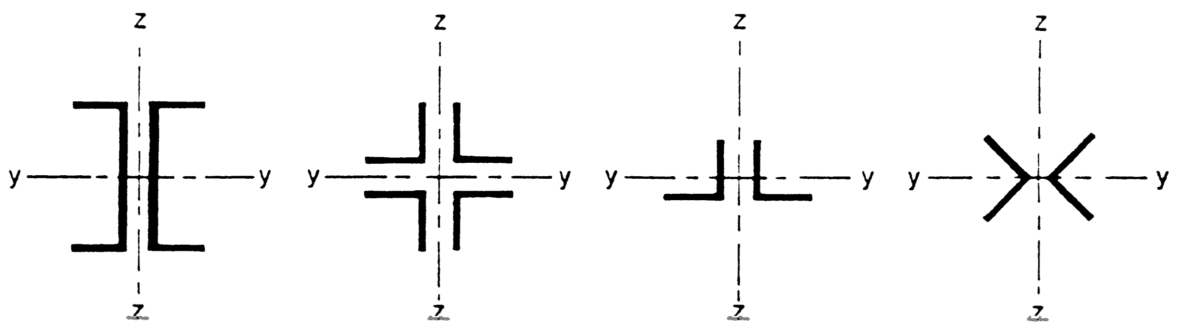


Figure 5.9.6 — Closely spaced built-up members

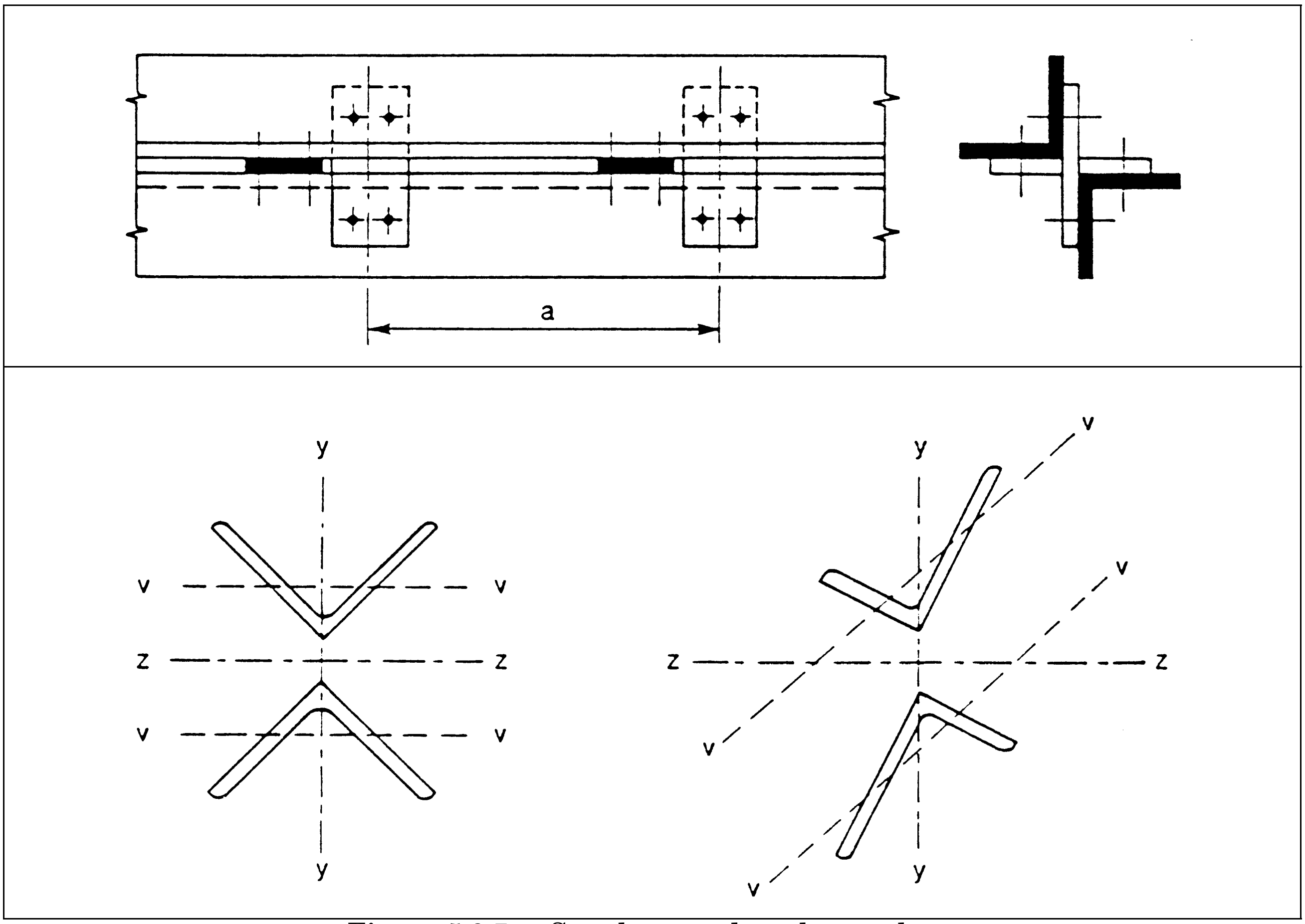


Figure 5.9.7 — Star-battened angle members

6 Connections subject to static loading

6.1 Basis

6.1.1 Introduction

(1) All connections shall have a design resistance such that the structure remains effective and is capable of satisfying all the basic design requirements given in Chapter 2.

(2) The partial safety factor γ_M shall be taken as follows:

- resistance of bolted connections: $\gamma_{Mb} = \boxed{1,25}$
- resistance of rivetted connections: $\gamma_{Mr} = \boxed{1,25}$
- resistance of pin connections: $\gamma_{Mp} = \boxed{1,25}$
- resistance of welded connections: $\gamma_{Mw} = \boxed{1,25}$
- slip resistance: see **6.5.8.1**
- resistance of joints in hollow section lattice girders: see Annex K
- resistance of members and cross-sections:

  γ_{M0}, γ_{M1} and γ_{M2} see **5.1.1**

(3) Connections subject to fatigue shall also satisfy the requirements given in Chapter 9.

6.1.2 Applied forces and moments

(1) The forces and moments applied to connections at the ultimate limit state shall be determined by global analysis conforming with Chapter 5.

(2) These applied forces and moments shall include:

- second order effects;
- the effects of imperfections, see **5.2.4**;
- the effects of connection flexibility in the case of semi-rigid connections, see **6.9**.

6.1.3 Resistance of connections

(1) The resistance of a connection shall be determined on the basis of the resistances of the individual fasteners or welds.

(2) Linear-elastic analysis shall generally be used in the design of the connection. Alternatively non-linear analysis of the connection may be employed provided that it takes account of the load deformation characteristics of all the components of the connection.

(3) If the design model is based on yield lines, the adequacy of this model shall be demonstrated on the basis of physical tests.

6.1.4 Design assumptions

(1) Connections may be designed by distributing the internal forces and moments in whatever rational way is best, provided that:

a) the assumed internal forces and moments are in equilibrium with the applied forces and moments,

b) each element in the connection is capable of resisting the forces or stresses assumed in the analysis,

c) the deformations implied by this distribution are within the deformation capacity of the fasteners or welds and of the connected parts, and

d) the deformations assumed in any design model based on yield lines are based on rigid body rotations (and in-plane deformations) which are physically possible.

(2) In addition, the assumed distribution of internal forces shall be realistic with regard to relative stiffnesses within the joint. The internal forces will seek to follow the path with the greatest rigidity. This path shall be clearly identified and consistently followed throughout the design of the connection.

(3) Residual stresses and stresses due to tightening of fasteners and due to ordinary accuracy of fit-up need not normally be allowed for.

6.1.5 Fabrication and erection

(1) Ease of fabrication and erection shall be considered in the design of all joints and splices.

(2) *Attention should be paid to:*

- *the clearances necessary for safe erection,*
- *the clearances needed for tightening fasteners,*
- *the need for access for welding,*
- *the requirements of welding procedures, and*
- *the effects of angular and length tolerances on fit-up.*

(3) *Attention should also be paid to the requirements for:*

- *subsequent inspection,*
- *surface treatment, and*
- *maintenance.*

NOTE For detailed rules on fabrication and erection see Chapter 7.

6.2 Intersections

(1) Members meeting at a joint shall normally be arranged with their centroidal axes intersecting at a point.

(2) Where there is eccentricity at intersections this shall be taken into account, except in the case of particular types of structures where it has been demonstrated that it is not necessary.

(3) *In the case of joints with angles or tees connected by at least two bolts at each connection, the setting out lines for the bolts in the angles and tees may be substituted for the centroidal axes for the purpose of intersection at the joints.*

6.3 Joints loaded in shear subject to vibration and/or load reversal

(1) Where a joint loaded in shear is subject to impact or significant vibration, either welding or else bolts with locking devices, preloaded bolts, injection bolts or other types of bolt which effectively prevent movement shall be used.

(2) Where slipping is not acceptable in a joint because it is subject to reversal of shear load (or for any other reason), either preloaded bolts in a slip-resistant connection (Category B or C as appropriate, see **6.5.3**), fitted bolts or welding shall be used.

(3) For wind and/or stability bracings, bolts in bearing type connections (Category A in **6.5.3**) may normally be used.

6.4 Classification of connections

6.4.1 General

(1) The structural properties of all connections shall be such as to achieve the assumptions made in the analysis of the structure and in the design of the members.

(2) Connections may be classified:

- by rigidity, see **6.4.2**.
- by strength, see **6.4.3**.

(3) *The types of connections should conform with Table 5.2.1 depending on the member design assumptions and the method of global analysis, see **5.2.2**.*

6.4.2 Classification by rigidity

6.4.2.1 *Nominally pinned connections*

(1)A nominally pinned connection shall be so designed that it cannot develop significant moments which might adversely affect members of the structure.

(2) *Nominally pinned connections should be capable of transmitting the forces calculated in design and should be capable of accepting the resulting rotations.*

6.4.2.2 *Rigid connections*

(1) A rigid connection shall be so designed that its deformation has no significant influence on the distribution of internal forces and moments in the structure, nor on its overall deformation.

(2) *The deformations of rigid connections should be such that they do not reduce the resistance of the structure by more than 5 %.*

(3) *Rigid connections should be capable of transmitting the forces and moments calculated in design.*

6.4.2.3 *Semi-rigid connections*

(1) A connection which does not meet the criteria for a rigid connection or a nominally pinned connection given in **6.4.2.2**(1) and **6.4.2.1**(1) shall be classified as a semi-rigid connection.

(2) *Semi-rigid connections should provide a predictable degree of interaction between members, based on the design moment-rotation characteristics of the joints.*

(3) *Semi-rigid connections should be capable of transmitting the forces and moments calculated in design.*

6.4.3 Classification by strength

6.4.3.1 *Nominally pinned connections*

(1) A nominally pinned connection shall be capable of transmitting the calculated design forces, without developing significant moments which might adversely affect members of the structure.

(2) *The rotation capacity of a nominally pinned connection should be sufficient to enable all the necessary plastic hinges to develop under the design loads.*

6.4.3.2 *Full-strength connections*

(1) The design resistance of a full strength connection shall be not less than that of the member connected.

(2) *Where the rotation capacity of a full-strength connection is limited, overstrength effects should be taken into account. If the design resistance of the connection is at least 1,2 times the design plastic resistance of the member, the rotation capacity of the connection need not be checked.*

(3) *The rigidity of a full-strength connection should be such that, under the design loads, the rotations at the necessary plastic hinges do not exceed their rotation capacities.*

6.4.3.3 *Partial-strength connections*

(1) The design resistance of a partial-strength connection shall not be less than that necessary to transmit the calculated design forces and moments, but may be less than that of the member connected.

(2) The rotation capacity of a partial-strength connection which occurs at a plastic hinge location shall not be less than that needed to enable all the necessary plastic hinges to develop under the design loads.

(3) *The rotation capacity of a connection may be demonstrated by experimental evidence. Experimental demonstration is not required when using details which experience has proved have adequate properties.*

(4) *The rigidity of a partial-strength connection should be such that the rotation capacity of none of the necessary plastic hinges is exceeded under the design loads.*

6.5 Connections made with bolts, rivets or pins

6.5.1 Positioning of holes for bolts and rivets

6.5.1.1 *Basis*

(1) The positioning of holes for bolts and rivets shall be such as to prevent corrosion and local buckling and to facilitate the installation of the bolts or rivets.

(2) The positioning of the holes shall also be in conformity with the limits of validity of the rules used to determine the design resistances of the bolts and rivets.

6.5.1.2 *Minimum end distance*

(1) The end distance e_1 *from the centre of a fastener hole to the adjacent end of any part, measured in the direction of load transfer (see Figure 6.5.1), should be not less than* $1,2d_o$*, where* d_o *is the hole diameter, see* ***7.5.2***.

(2) *The end distance should be increased if necessary to provide adequate bearing resistance, see* ***6.5.5*** *and* ***6.5.6***.

6.5.1.3 *Minimum edge distance*

(1) *The edge distance* e_2 *from the centre of a fastener hole to the adjacent edge of any part, measured at right angles to the direction of load transfer (see Figure 6.5.1), should normally be not less than* $1,5d_o$.

(2) *The edge distance may be reduced to not less than* $1,2d_o$ *provided that the design bearing resistance is reduced accordingly, see* ***6.5.5*** *and* ***6.5.6***.

6.5.1.4 *Maximum end and edge distances*

(1) *Where the members are exposed to the weather or other corrosive influences, the maximum end or edge distance should not exceed 40 mm + 4t, where t is the thickness of the thinner outer connected part.*

(2) *In other cases the end or edge distance should not exceed 12t or 150 mm, whichever is the larger.*

The edge distance should also not exceed the maximum to satisfy local buckling requirements for an outstand element. This requirement does not apply to fasteners interconnecting the components of tension members. The end distance is not affected by this requirement.

6.5.1.5 *Minimum spacing*

(1) *The spacing* p_1 *between centres of fasteners in the direction of load transfer (see Figure 6.5.1), should be not less than* $2,2d_o$*. This spacing should be increased if necessary to provide adequate bearing resistance, see* ***6.5.5*** *and* ***6.5.6***.

(2) *The spacing* p_2 *between rows of fasteners, measured perpendicular to the direction of load transfer (see Figure 6.5.1), should normally be not less than* $3,0d_o$*. This spacing may be reduced to* $2,4d_o$ *provided that the design bearing resistance is reduced accordingly, see* ***6.5.5*** *and* ***6.5.6***.

6.5.1.6 ***Maximum spacing in compression members***

(1) *The spacing p_1 of the fasteners in each row and the spacing p_2 between rows of fasteners, should not exceed the lesser of 14t or 200 mm. Adjacent rows of fasteners may be symmetrically staggered, see Figure 6.5.2.*

(2) *The centre-to-centre spacing of fasteners should also not exceed the maximum width which satisfies local buckling requirements for an internal element, see* ***5.3.4****.*

6.5.1.7 ***Maximum spacing in tension members***

(1) *In tension members the centre-to-centre spacing $p_{1,i}$ of fasteners in inner rows may be twice that given in* ***6.5.1.6****(1) for compression members, provided that the spacing $p_{1,o}$ in the outer row along each edge does not exceed that given in* ***6.5.1.6****(1), see Figure 6.5.3.*

(2) *Both of these values may be multiplied by 1,5 in members not exposed to the weather or other corrosive influences.*

6.5.1.8 ***Slotted holes***

(1) *The minimum distance e_3 from the axis of a slotted hole to the adjacent end or edge of any part (see Figure 6.5.4) should not be less than $1,5d_o$.*

(2) *The minimum distance e_4 from the centre of the end radius of a slotted hole to the adjacent end or edge of any part (see Figure 6.5.4) should not be less than $1,5d_o$.*

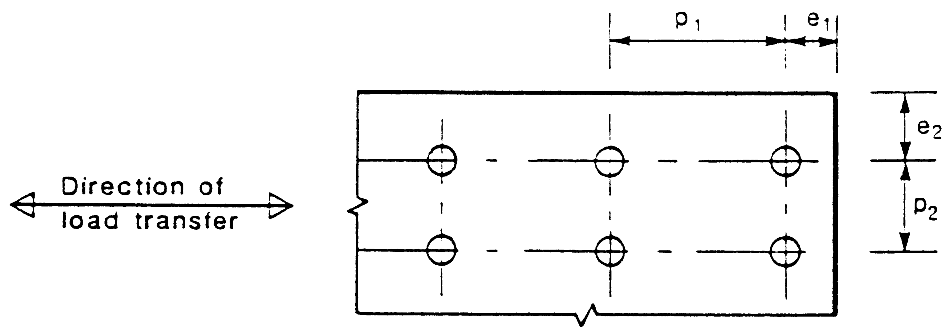


Figure 6.5.1 — Symbols for spacing of fasteners

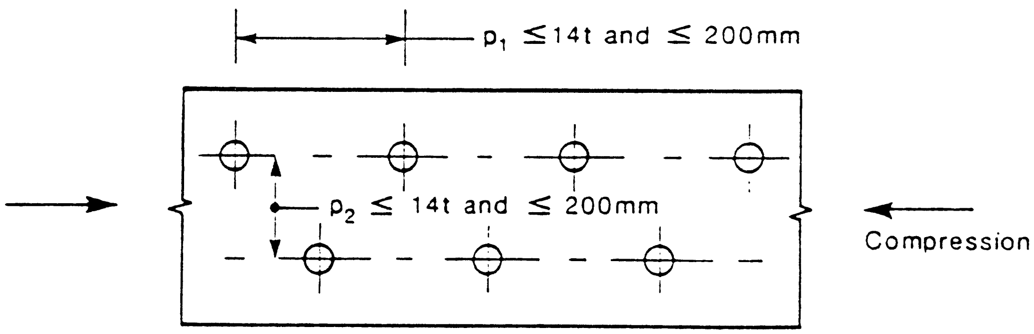


Figure 6.5.2 — Staggered spacing — compression

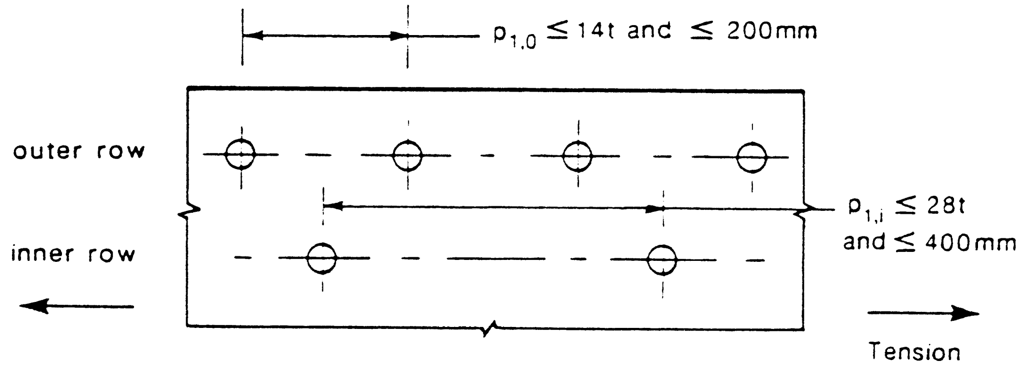


Figure 6.5.3 — Spacing in tension members

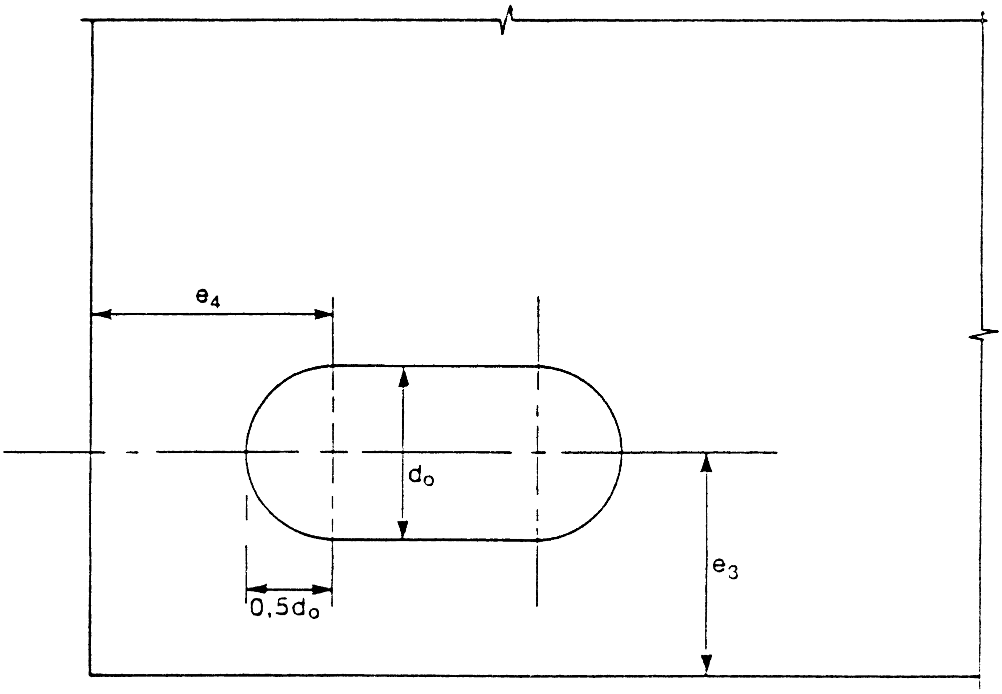


Figure 6.5.4 — End and edge distances for slotted holes

6.5.2 Deductions for fastener holes

6.5.2.1 *General*

(1) In the design of connections in compression members, no deduction for fastener holes is normally required except for oversize or slotted holes.

(2) In the design of connections in other types of member the provisions given in **5.4.3**, **5.4.5.3**(3) and **5.4.6**(8) apply for tension, bending moment and shear force respectively.

6.5.2.2 *Design shear rupture resistance*

(1) "Block shear" failure at a group of fastener holes near the end of a beam web or a bracket, see Figure 6.5.5, shall be prevented by using appropriate hole spacing. This mode of failure generally consists of tensile rupture along the line of fastener holes on the tension face of the hole group, accompanied by gross section yielding in shear at the row of fastener holes along the shear face of the hole group, see Figure 6.5.5.

(2) *The design value of the effective resistance to block shear* $V_{eff.Rd}$ *should be determined from:*

$$V_{eff.Rd} = (f_Y/\sqrt{3})\, A_{v.eff}/\gamma_{M0} \qquad (6.1)$$

where $A_{v.eff}$ *is the effective shear area*

(3) *The effective shear area* $A_{v.eff}$ *should be determined as follows:*

$$A_{v.eff} = t\, L_{v.eff}$$

where $L_{v.eff} = L_v + L_1 + L_2$ *but* $L_{v.eff} \le L_3$

in which $L_1 = a_1$ *but* $L_1 \le 5\,d$

$L_2 = (a_2 - k\, d_{o.t})\, (f_u/f_y)$

and $L_3 = L_v + a_1 + a_3$ *but* $L_3 < (L_v + a_1 + a_3 - nd_{o.v})\, (f_u/f_y)$

where a_1, a_2, a_3 *and* L_v *are as indicated in Figure 6.5.5*

d *is the nominal diameter of the fasteners*

$d_{o.t}$ *is the hole size for the tension face, generally the hole diameter, but for horizontally slotted holes the slot length should be used*

$d_{o.v}$ *is the hole size for the shear face, generally the hole diameter, but for vertically slotted holes the slot length should be used*

n *is the number of fastener holes on the shear face*

t *is the thickness of the web or bracket*

and k *is a coefficient with values as follows:*

- *for a single row of bolts:* $k = 0{,}5$
- *for two rows of bolts:* $k = 2{,}5$

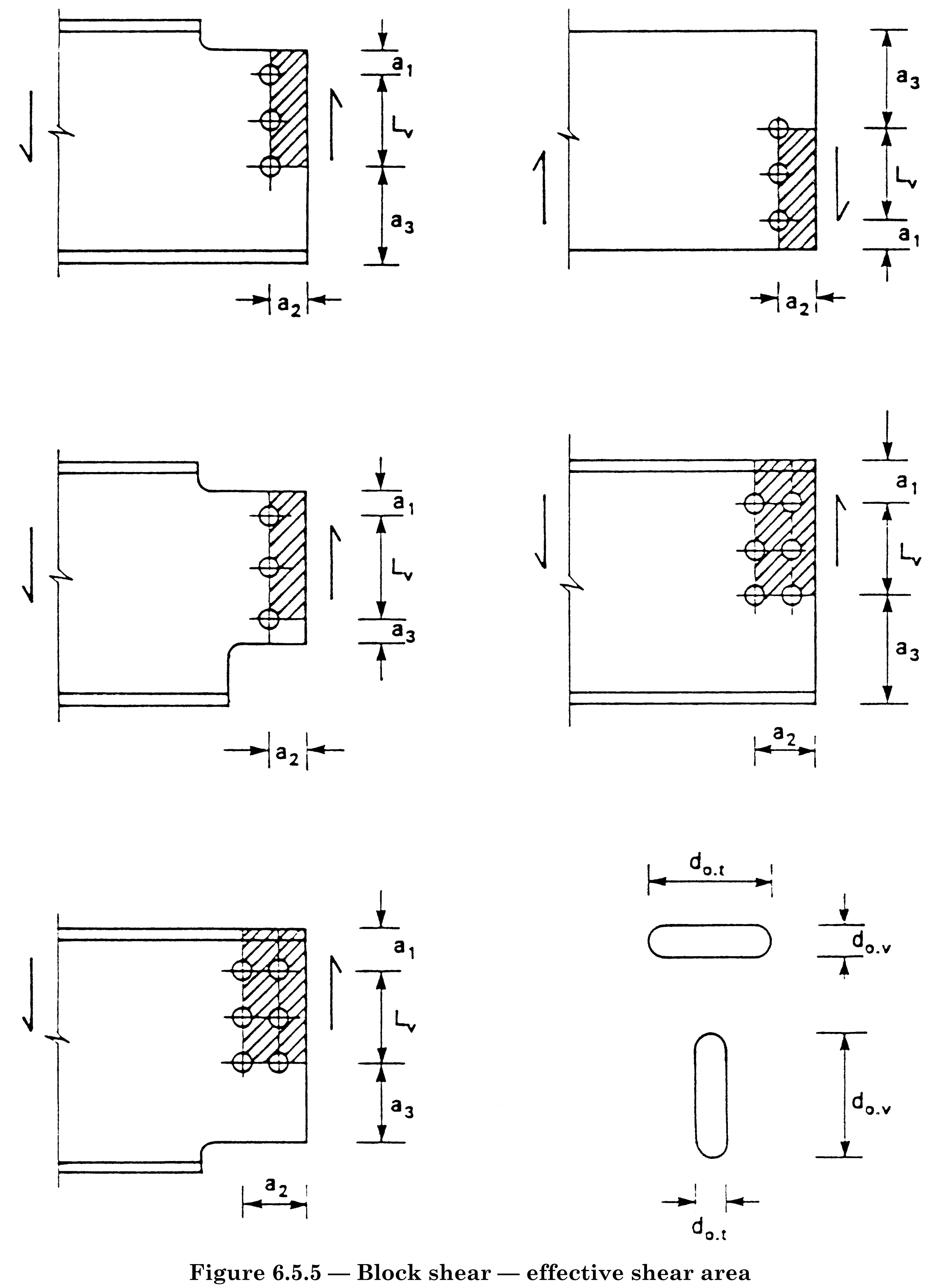


Figure 6.5.5 — Block shear — effective shear area

6.5.2.3 ***Angles connected by one leg***

(1) In the case of unsymmetrical or unsymmetrically connected members such as angles connected by one leg, the eccentricity of fasteners in end connections and the effects of the spacing and edge distances of the bolts shall be taken into account when determining the design resistance.

(2) *Angles connected by a single row of bolts in one leg, see Figure 6.5.6, may be treated as concentrically loaded and the design ultimate resistance of the net section determined as follows*:

with 1 bolt:

$$N_{u.Rd} = \frac{2{,}0\ (e_2 - 0{,}5 d_o)\ t\ f_u}{\gamma_{M2}} \qquad (6.2)$$

with 2 bolts:

$$N_{u.Rd} = \frac{\beta_2\ A_{net}\ f_u}{\gamma_{M2}} \qquad (6.3)$$

with 3 bolts:

$$N_{u.Rd} = \frac{\beta_3\ A_{net}\ f_u}{\gamma_{M2}} \qquad (6.4)$$

where β_2 and β_3 are reduction factors dependent on the pitch p_1 as given in Table 6.5.1.

For intermediate values of p_1 the value of β may be determined by linear interpolation.

and A_{net} is the net area of the angle. For an unequal-leg angle connected by its smaller leg, A_{net} should be taken as equal to the net section area of an equivalent equal-leg angle of leg size equal to that of the smaller leg.

(3) *The design buckling resistance of a compression member, see* ***5.5.1****, should be based on the gross cross-sectional area, but should not be taken as more than the design resistance of the cross-section given in (2).*

Table 6.5.1 — Reduction factors β_2 and β_3

Pitch	p_1	$\leq$ 2,5 d_o	$\geq$ 5,0 d_o
2 bolts	β_2	0,4	0,7
3 bolts or more	β_3	0,5	0,7

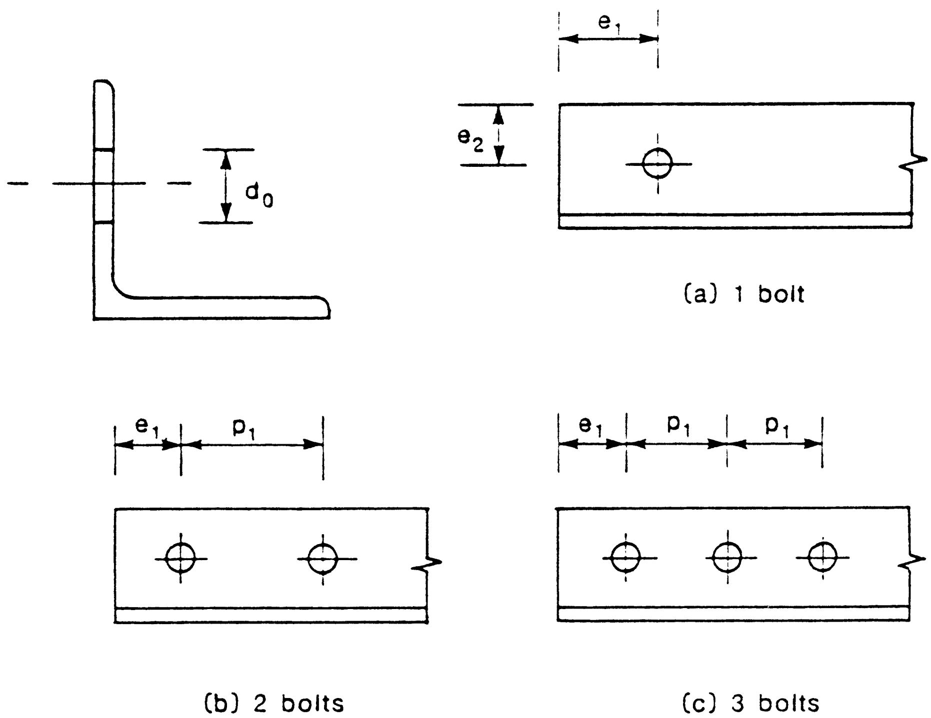


Figure 6.5.6 — Connections of angles

6.5.3 Categories of bolted connections

6.5.3.1 *Shear connections*

(1) The design of a bolted connection loaded in shear shall conform with one of the following categories, see Table 6.5.2.

(2) **Category A: Bearing type**

In this category ordinary bolts (manufactured from low carbon steel) or high strength bolts, from grade 4.6 up to and including grade 10.9, shall be used. No preloading and special provisions for contact surfaces are required. The design ultimate shear load shall not exceed the design shear resistance nor the design bearing resistance, obtained from **6.5.5**.

(3) **Category B: Slip-resistant at serviceability limit state**

In this category preloaded high strength bolts with controlled tightening in conformity with Reference Standard 8 shall be used. Slip shall not occur at the *serviceability* limit state. The combination of actions to be considered shall be selected from **2.3.4** depending on the load cases where resistance to slip is required. The design serviceability shear load should not exceed the design slip resistance, obtained from **6.5.8**. The design ultimate shear load shall not exceed the design shear resistance nor the design bearing resistance, obtained from **6.5.5**.

(4) **Category C: Slip-resistant at ultimate limit state**

In this category preloaded high strength bolts with controlled tightening in conformity with Reference Standard 8 shall be used. Slip shall not occur at the *ultimate* limit state. The design ultimate shear load shall not exceed the design slip resistance obtained from **6.5.8** nor the design bearing resistance obtained from **6.5.5**.

In addition, at the ultimate limit state the design plastic resistance of the net section at bolt holes $N_{net.Rd}$ (see **5.4.3**) shall be taken as:

$$N_{net.Rd} = A_{net} f_y/\gamma_{M0} \tag{5.14}$$

Table 6.5.2 — Categories of bolted connections

Category	Criteria	Remarks
Shear connections		
A bearing type	$F_{v.Sd} \leq F_{v.Rd}$ $F_{v.Sd} \leq F_{b.Rd}$	No preloading required. All grades from 4.6 to 10.9.
B slip-resistant at serviceability	$F_{v.Sd.ser} \leq F_{s.Rd.ser}$ $F_{v.sd} \leq F_{v.Rd}$ $F_{v.sd} \leq F_{b.Rd}$	Preloaded high strength bolts. No slip at the serviceability limit state.
C slip-resistant at ultimate	$F_{v.Sd} \leq F_{s.Rd}$ $F_{v.Sd} \leq F_{b.Rd}$	Preloaded high strength bolts. No slip at the ultimate limit state.
Tension capacity		
Category	**Creterion**	**Remarks**
D non-preloaded	$F_{t.Sd} \leq F_{t.Rd}$	No preloading required. All grades from 4.6 to 10.9.
E preloaded	$F_{t.Sd} \leq F_{t.Rd}$	Preloaded high strength bolts.

Key:

$F_{v.Sd.ser}$ = design shear force per bolt for the serviceability limit state
$F_{v.Sd}$ = design shear force per bolt for the ultimate limit state
$F_{v.Rd}$ = design shear resistance per bolt
$F_{b.Rd}$ = design bearing resistance per bolt
$F_{s.Rd.ser}$ = design slip resistance per bolt at the serviceability limit state
$F_{s.Rd}$ = design slip resistance per bolt at the ultimate limit state
$F_{t.sd}$ = design tensile force per bolt for the ultimate limit state
$F_{t.Rd}$ = design tension resistance per bolt

6.5.3.2 *Tension connections*

(1) The design of a bolted connection loaded in tension shall conform with one of the following categories, see Table 6.5.2.

(2) **Category D: Connections with non-preloaded bolts**

In this category ordinary bolts (manufactured from low carbon steel) or high strength bolts up to and including grade 10.9 shall be used. No preloading is required. This category shall not be used where the connections are frequently subjected to variations of tensile loading. However, they may be used in connections designed to resist normal wind loads.

(3) **Category E: Connections with preloaded high strength bolts**

In this category preloaded high strength bolts with controlled tightening in conformity with Reference Standard 8 shall be used. Such preloading improves fatigue resistance. However, the extent of the improvement depends on detailing and tolerances.

(4) For tension connections of both Categories D and E no special treatment of contact surfaces is necessary, except where connections of Category E are subject to both tension and shear
(combination E-B or E-C).

6.5.4 Distribution of forces between fasteners

(1) The distribution of internal forces between fasteners at the ultimate limit state shall be proportional to the distance from the centre of rotation, see Figure 6.5.7(a), in the following cases:

- Category C slip-resistant connections
- Other shear connections where the design shear resistance $F_{v.Rd}$ of a fastener is less than the design bearing resistance $F_{b.Rd}$.

(2) *In other cases the distribution of internal forces between fasteners at the ultimate limit state may be either as in (1) or else plastic, see Figure 6.5.7. Any reasonable distribution may be assumed provided that it satisfies the requirements given in **6.1.4**.*

(3) *In a lap joint, the same bearing resistance in any particular direction should be assumed for each fastener.*

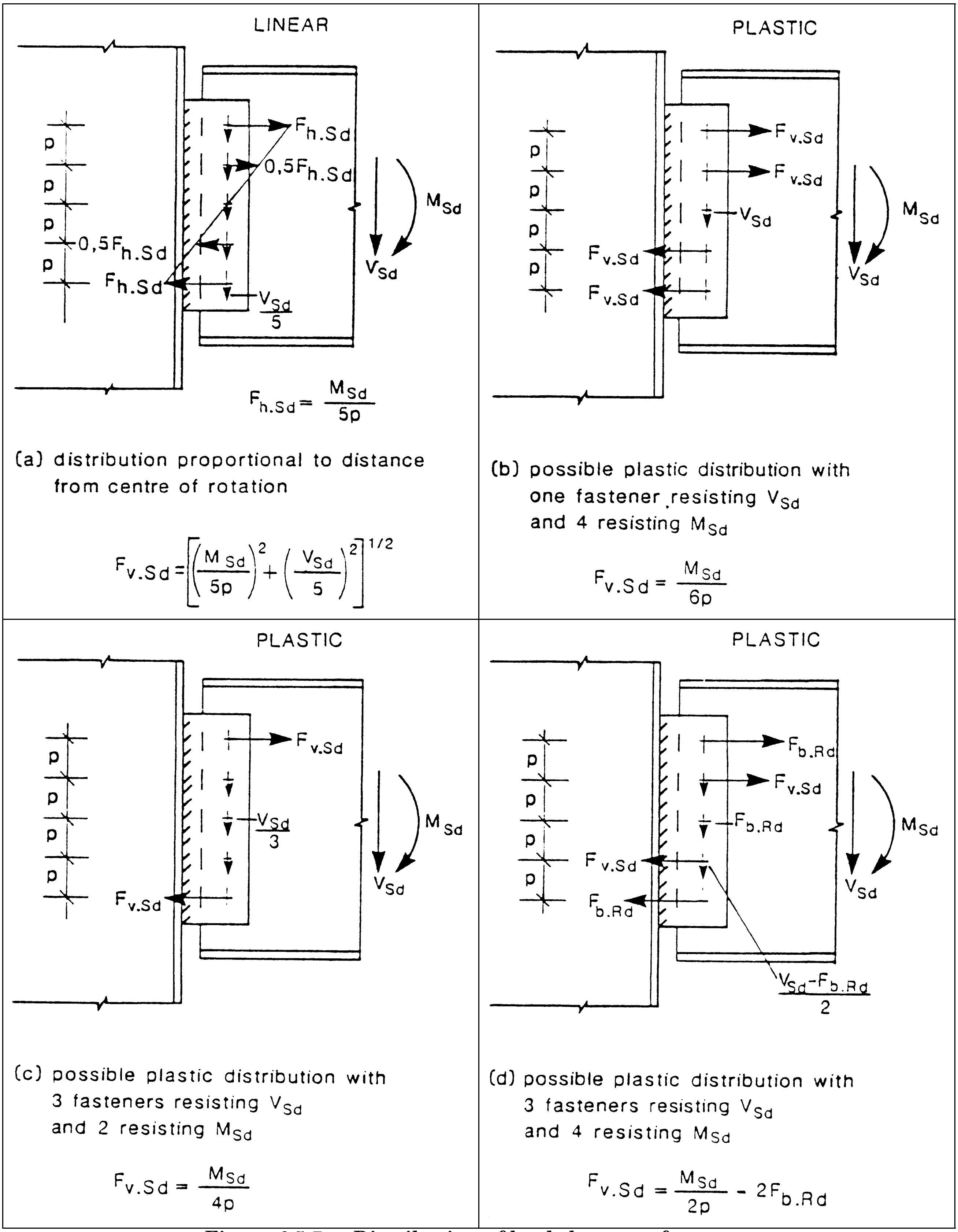


Figure 6.5.7 — Distribution of loads between fasteners

6.5.5 Design resistances of bolts

(1) The design resistances given in this clause apply to standard manufactured bolts of strength grades from grade 4.6 up to and including grade 10.9, which conform with Reference Standard 3, see normative Annex B. Nuts and washers shall also conform with Reference Standard 3 and shall have the corresponding specified strengths.

(2) At the ultimate limit state the design shear force $F_{v.Sd}$ on a bolt shall not exceed the lesser of:

- the design shear resistance $F_{v.Rd}$
- the design bearing resistance $F_{b.Rd}$

both as given in Table 6.5.3.

(3) The design tensile force $F_{t.Sd}$, inclusive of any force due to prying action, shall not exceed the design tension resistance $B_{t.Rd}$ of the bolt-plate assembly.

(4) The design tension resistance of the bolt-plate assembly $B_{t.Rd}$ shall be taken as the smaller of the design tension resistance $F_{t.Rd}$ given in Table 6.5.3 and the design punching shear resistance of the bolt head and the nut, $B_{p.Rd}$ obtained from:

$$B_{p.Rd} = 0{,}6\,\pi\, d_m\, t_p\, f_u/\gamma_{Mb} \qquad (6.5)$$

where t_p is the thickness of the plate under the bolt head or the nut

and d_m is the mean of the across points and across flats dimensions of the bolt head or the nut, whichever is smaller

(5) Bolts subject to both shear force and tensile force shall in addition satisfy the following:

$$\frac{F_{v.Sd}}{F_{v.Rd}} + \frac{F_{t.Sd}}{1{,}4\,F_{t.Rd}} \leq 1{,}0 \qquad (6.6)$$

(6) The design resistances for tension and for shear through the threaded portion given in Table 6.5.3 are restricted to bolts manufactured in conformity with Reference Standard 3. For other items with cut threads, such as holding-down bolts or tie rods fabricated from round steel bars where the threads are cut by the steelwork fabricator and not by a specialist bolt manufacturer, the relevant values from Table 6.5.3 shall be reduced by multiplying them by a factor of 0,85.

(7) The values for design shear resistance $F_{v.Rd}$ given in Table 6.5.3 apply only where the bolts are used in holes with nominal clearances not exceeding those for standard holes as specified in **7.5.2**(1).

(8) M12 and M14 bolts may also be used in 2 mm clearance holes provided that:

- for bolts of strength grade 4.8, 5.8, 6.8 or 10.9 the design shear resistance $F_{v.Rd}$ is taken as 0,85 times the value given in Table 6.5.3.
- the design shear resistance $F_{v.Rd}$ (reduced as above if applicable) is not less than the design bearing resistance $F_{b.Rd}$.

(9) The values for the design bearing resistance in Table 6.5.3 apply only where the edge distance e_2 is not less than 1,5 d_o and the spacing p_2 measured transverse to the load direction is at least 3,0 d_o.

(10) If e_2 is reduced to 1,2 d_o and/or p_2 is reduced to 2,4 d_o, then the bearing resistance $F_{b.Rd}$ should be reduced to 2/3 of the value given in Table 6.5.3. For intermediate values 1,2 $d_o < e_2 \leq 1{,}5\, d_o$ and/or 2,4 $d_o \leq p_2 \leq 3 d_o$ the value of $F_{b.Rd}$ may be determined by linear interpolation.

(11) *For bolts in standard clearance holes (see **7.5.2**), conservative values of the design bearing resistance $F_{b.Rd}$, based on the bolt diameter d, may be obtained from Table 6.5.4.*

Table 6.5.3 — Design resistance for bolts

Shear resistance per shear plane:
if the shear plane passes through the threaded portion of the bolt:
• for strength grades 4.6, 5.6 and 8.8:
$F_{v.Rd} = \dfrac{0,6\ f_{ub}\ A_s}{\gamma_{Mb}}$
• for strength grades 4.8, 5.8, and 10.9:
$F_{v.Rd} = \dfrac{0,5\ f_{ub}\ A_s}{\gamma_{Mb}}$
if the shear plane passes through the unthreaded portion of the bolt:
$F_{v.Rd} = \dfrac{0,6\ f_{ub}\ A}{\gamma_{Mb}}$

Bearing resistance: $F_{b.Rd} = \dfrac{2,5\ \alpha\ f_u\ d\ t}{\gamma_{Mb}}$ where α is the smallest of: $\dfrac{e_1}{3d_o}\ ;\ \dfrac{p_1}{3d_o} - \dfrac{1}{4}\ ;\ \dfrac{f_{ub}}{f_u}$ or 1,0.
Tension resistance: $F_{t.Rd} = \dfrac{0,9\ f_{ub}\ A_s}{\gamma_{Mb}}$
A is the gross cross-section area of bolt A_s is the tensile stress area of bolt d is the bolt diameter d_o is the hole diameter
See also Table 6.5.4 for values of design bearing resistance based on bolt diameter.

Table 6.5.4 — Design bearing resistance — based on bolt diameter

Conservative values for bolts in standard clearance holes (see **7.5.2**) with γ_{Mb} = 1,25

Nominal bearing class	Minimum dimensions		Design bearing resistance $F_{b.Rd}$
	e_1	p_1	
low	1,7d	2,5d	1,0 $f_u dt$[a]
medium	2,5d	3,4d	1,5 $f_u dt$[a]
high	3,4d	4,3d	2,0 $f_u dt$[a]

[a] but $F_{b.Rd} \leq 2,0\ f_{ub}dt$

6.5.6 Design resistance of rivets

(1) At the ultimate limit state the design shear force $F_{v.Sd}$ on a rivet shall not exceed the lesser of:

- the design shear resistance $F_{v.Rd}$
- the design bearing resistance $F_{b.Rd}$

both as given in Table 6.5.5.

(2) Riveted connections shall be designed to transfer forces essentially in shear. If tension is necessary to satisfy equilibrium, the design tensile force $F_{t.Sd}$ shall not exceed the design tension resistance $F_{t.Rd}$ given in Table 6.5.5.

(3) Rivets subject to both shear and tensile forces shall in addition satisfy the following:

$$\frac{F_{v.Sd}}{F_{v.Rd}} + \frac{F_{t.Sd}}{1,4\ F_{t.Rd}} \leq 1,0 \qquad (6.6)$$

Table 6.5.5 — Design resistances for rivets

Shear resistance per shear plane: $F_{v.Rd} = \frac{0,6\ f_{ur}\ A_o}{\gamma_{Mr}}$
Bearing resistance: $F_{b.Rd} = \frac{2,5\ \alpha\ f_u\ d_o t}{\gamma_{Mr}}$ where α is the smallest of: $\frac{e_1}{3d_o}$; $\frac{p_1}{3d_o} - \frac{1}{4}$; $\frac{f_{ur}}{f_u}$ or 1,0
Tension resistance: $F_{t.Rd} = \frac{0,6\ f_{ur}\ A_o}{\gamma_{Mr}}$
A_o is the area of the rivet hole d_o is the diameter of the rivet hole f_{ur} is the specified ultimate tensile strength of the rivet.

(4) The values for the design bearing resistance $F_{b.Rd}$ in Table 6.5.5 apply only where the edge distance e_2 is not less than 1,5 d_o and the spacing p_2 measured transverse to the load direction is at least 3,0 d_o.

(5) For smaller values of e_2 and/or p_2 the same reduction of $F_{b.Rd}$ should be applied as given in **6.5.5**(10) for bolts.

(6) For grade Fe 360 the "as driven" value of f_{ur} may be taken as 400 N/mm^2.

(7) *As a general rule, the grip length of a rivet should not exceed 4,5d for hammer riveting and 6,5d for press riveting.*

6.5.7 Countersunk bolts and rivets

(1) The design tension resistance $F_{t.Rd}$ of a countersunk bolt or rivet shall be taken as 0,7 times the design tension resistance given in Table 6.5.3 or Table 6.5.5 respectively.

(2) The angle and depth of countersinking shall conform with Reference Standard 3, otherwise the tension resistance shall be adjusted accordingly.

(3) The design bearing resistance $F_{b.Rd}$ of a countersunk bolt or rivet shall be calculated as specified in **6.5.5** or **6.5.6** respectively, with half the depth of the countersink deducted from the thickness t of the relevant part joined.

6.5.8 High strength bolts in slip-resistant connections

6.5.8.1 *Slip resistance*

(1) The design slip resistance of a preloaded high-strength bolt shall be taken as:

$$F_{s.Rd} = \frac{k_s\, n\, \mu}{\gamma_{Ms}} F_{p.Cd} \qquad (6.7)$$

where $F_{p.Cd}$ is the design preloading force, given in **6.5.8.2**

μ is the slip factor, see **6.5.8.3**

and n is the number of friction interfaces.

(2) The value of k_s shall be taken as follows:

- where the holes in all the plies have standard nominal clearances as specified in **7.5.2**(1):
  $k_s = 1{,}0$
- for oversize holes, as specified in **7.5.2**(6), or short slotted holes, as specified in **7.5.2**(9):
  $k_s = 0{,}85$
- for long slotted holes, as specified in **7.5.2**(10):
  $k_s = 0{,}7$

(3) For bolts in standard nominal clearance holes and for bolts in slotted holes with the axis of the slot perpendicular to the direction of load transfer, the partial safety factor for slip resistance γ_{Ms} be taken as:

$\gamma_{Ms.ult} = \boxed{1{,}25}$ for the ultimate limit state,

$\gamma_{Ms.ser} = \boxed{1{,}10}$ for the serviceability limit state.

(4) Connections with bolts in oversize holes or in slotted holes with the axis of the slot parallel to the direction of load transfer, shall be designed as Category C, slip-resistant at the ultimate limit state.

In this case, the partial safety factor for slip resistance shall be taken as:

$\gamma_{Ms.ult} = \boxed{1{,}40}$

6.5.8.2 *Preloading*

(1) For high strength bolts conforming with Reference Standard 3, with controlled tightening in conformity with Reference Standard 8, the design preloading force $F_{p.Cd}$, to be used in the design calculations, shall be taken as:

$$F_{p.Cd} = 0{,}7\, f_{ub}\, A_s \qquad (6.8)$$

(2) Where other types of preloaded bolts or other types of preloaded fasteners are used, the design preloading force $F_{p.Cd}$ shall be agreed between the client, the designer and the competent authority.

6.5.8.3 *Slip factor*

(1) The design value of the slip factor μ is dependent on the specified class of surface treatment as given in Reference Standard 8. The value of μ should be taken as follows:

$\mu = 0{,}50$ for class A surfaces

$\mu = 0{,}40$ for class B surfaces

$\mu = 0{,}30$ for class C surfaces

$\mu = 0{,}20$ for class D surfaces

(2) The classification of any surface treatment shall be based on tests on specimens representative of the surfaces used in the structure using the procedure set out in Reference Standard 8.

(3) Provided that the contact surfaces have been treated in conformity with Reference Standard 8 the following surface treatments may be classified without further testing:

In class A: — surfaces blasted with shot or grit, with any loose rust removed, no pitting;
— surfaces blasted with shot or grit, and spray-metallized with aluminium;
— surfaces blasted with shot or grit, and spray-metallized with a zinc-based coating certified to provide a slip factor of not less than 0,5;

In class B: — surfaces blasted with shot or grit, and painted with an alkali-zinc silicate paint to produce a coating thickness of 50–80 μm.

In class C: — surfaces cleaned by wire brushing or flame cleaning, with any loose rust removed;

In class D: — surfaces not treated.

6.5.8.4 *Combined tension and shear*

(1) If a slip-resistant connection is subjected to an applied tensile force F_t in addition to the shear force F_v tending to produce slip, the slip resistance per bolt shall be taken as follows:

- Category B: Slip-resistant at serviceability:

$$F_{s.Rd.ser} = \frac{k_s\, n\, \mu\, (F_{p.Cd} - 0{,}8\, F_{t.Sd.ser})}{\gamma_{Ms.ser}} \tag{6.9}$$

- Category C: Slip-resistant at ultimate:

$$F_{s.Rd} = \frac{k_s\, n\, \mu\, (F_{p.Cd} - 0{,}8\, F_{t.Sd})}{\gamma_{Ms.ult}} \tag{6.10}$$

(2) If, in a moment connection, the applied tensile force is counterbalanced by a contact force on the compression side, no reduction of the slip resistance is required.

6.5.9 Prying forces

(1) Where fasteners are required to carry an applied tensile force, they shall be proportioned to also resist the additional force due to prying action, where this can occur, see Figure 6.5.8.

(2) The prying forces depend on the relative stiffness and geometrical proportions of the parts of the connection, see Figure 6.5.9.

(3) *If the effect of the prying force is taken advantage of in the design of the parts, then the prying force should be determined by a suitable analysis analogous to that incorporated in the application runs given in normative Annex J for beam-to-column connections.*

6.5.10 Long joints

(1) Where the distance L_j between the centres of the end fasteners in a joint, measured in the direction of the transfer of force (see Figure 6.5.10), is more than 15 d, where d is the nominal diameter of the bolts or rivets, the design shear resistance $F_{v.Rd}$ of all the fasteners calculated as specified in **6.5.5** or **6.5.6** as appropriate shall be reduced by multiplying it by a reduction factor β_{Lf}, given by:

$$\beta_{Lf} = 1 - \frac{L_j - 15\, d}{200\, d} \tag{6.11}$$

but $\beta_{Lf} \leq 1{,}0$ and $\beta_{Lf} \geq 0{,}75$

(2) This provision does not apply where there is a uniform distribution of force transfer over the length of the joint, e.g. the transfer of shear force from the web of a section to the flange.

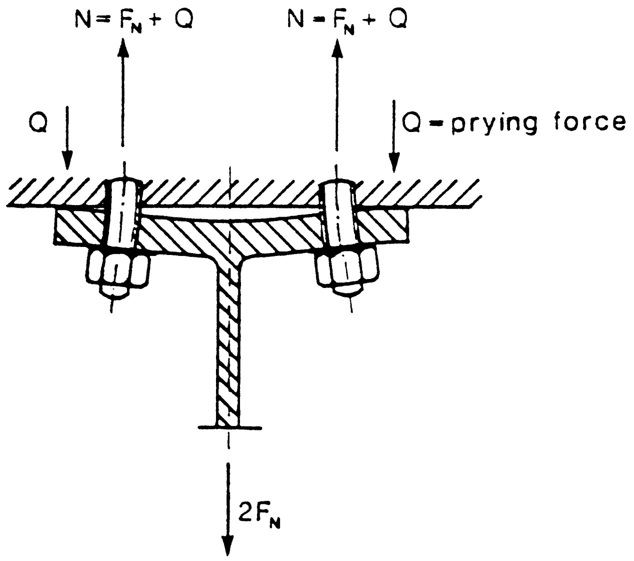


Figure 6.5.8 — Prying forces

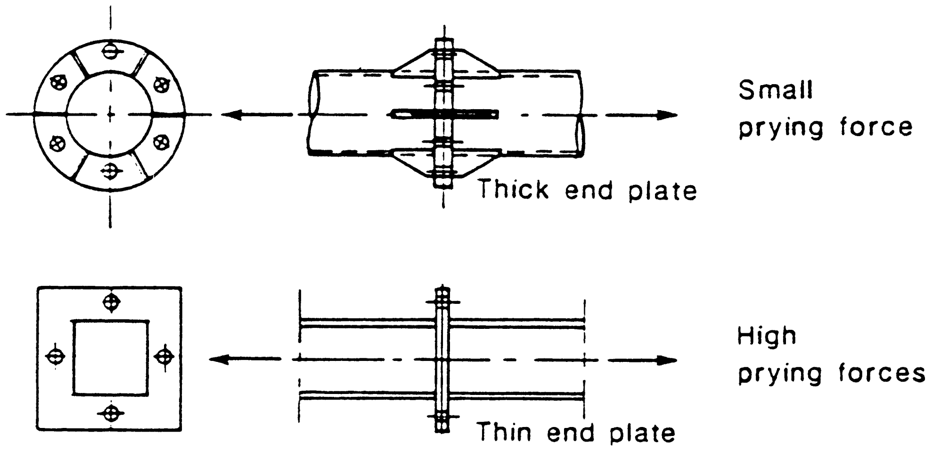


Figure 6.5.9 — Effect of details on prying forces

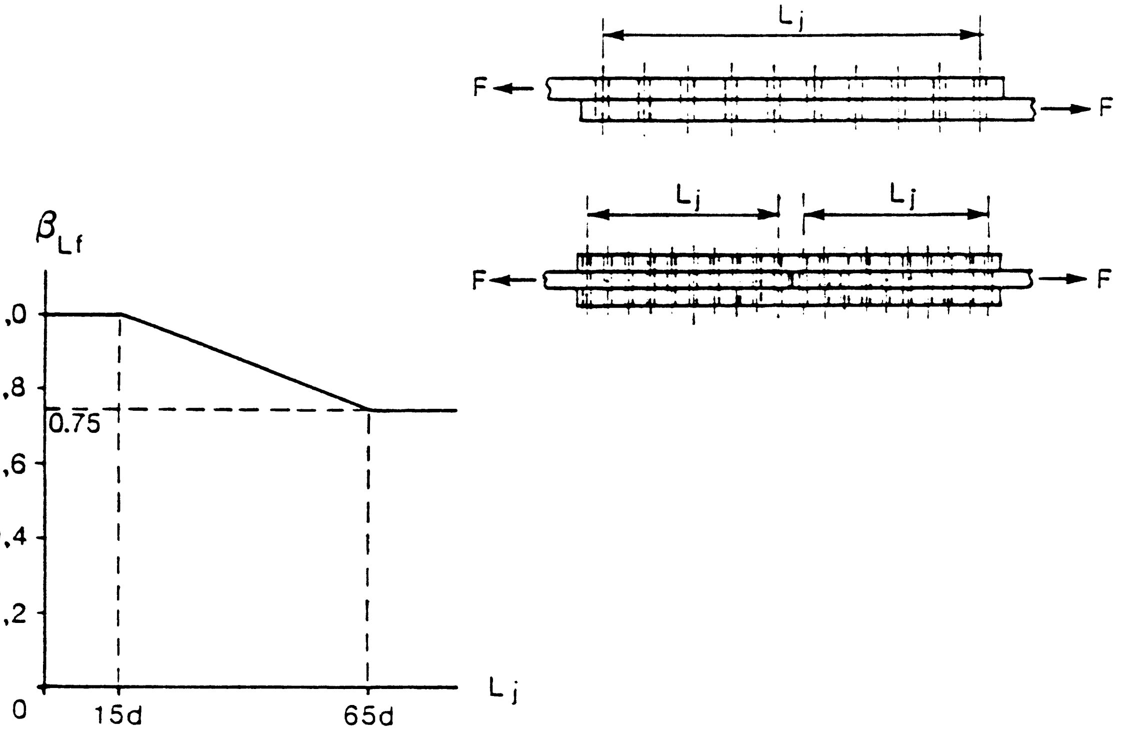


Figure 6.5.10 — Long joints

6.5.11 Single lap joints with one bolt

(1) In single lap joints of flats with only one bolt, see Figure 6.5.11, the bolt shall be provided with washers under both the head and the nut to avoid pull-out failure.

(2) The bearing resistance $F_{b.Rd}$ determined in accordance with **6.5.5** shall be limited to:

$$F_{b.Rd} \leq 1,5\ f_u dt/\gamma_{Mb} \tag{6.12}$$

NOTE Single rivets should not be used in single lap joints.

(3) *In the case of high strength bolts, grades 8.8 or 10.9, hardened washers should be used for single lap joints of flats with only one bolt, even where the bolts are not preloaded.*

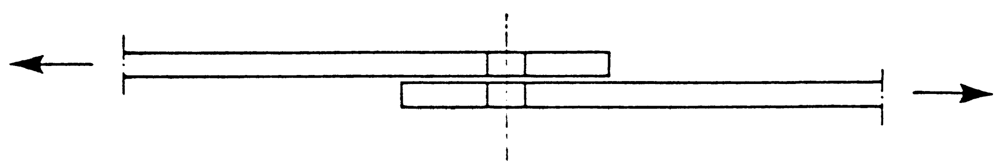


Figure 6.5.11 — Single lap joint with one bolt

6.5.12 Fasteners through packings

(1) Where bolts or rivets transmitting load in shear and bearing pass through packings of total thickness t_p greater than one-third of the nominal diameter d, the design shear resistance $F_{v.Rd}$ calculated as specified in **6.5.5** or **6.5.6** as appropriate, shall be reduced by multiplying it by a reduction factor β_p given by:

$$\beta_p = \frac{9d}{8d + 3t_p} \quad \text{but} \quad \beta_p \leq 1 \tag{6.13}$$

(2) *For double shear connections with packings on both sides of the splice, t_p should be taken as the thickness of the thicker packing.*

(3) *Any additional fasteners required due to the application of the reduction factor β_p may optionally be placed in an extension of the packing.*

6.5.13 Pin connections

6.5.13.1 *Scope*

(1) This clause applies to pin connections where free rotation is required. Pin connections in which no rotation is required may be designed as single bolted connections, see **6.5.5** and **6.5.11**.

6.5.13.2 *Pin holes and pin plates*

(1) The geometry of plates in pin connections shall be in accordance with the dimensional requirements given in Table 6.5.6.

(2) At the ultimate limit state the design force N_{Sd} in the plate shall not exceed the design bearing resistance given in Table 6.5.7.

(3) Pin plates provided to increase the net area of a member or to increase the bearing resistance of a pin shall be of sufficient size to transfer the design force from the pin into the member and shall be arranged to avoid eccentricity.

6.5.13.3 *Design of pins*

(1) The bending moments in a pin should be calculated as indicated in Figure 6.5.12.

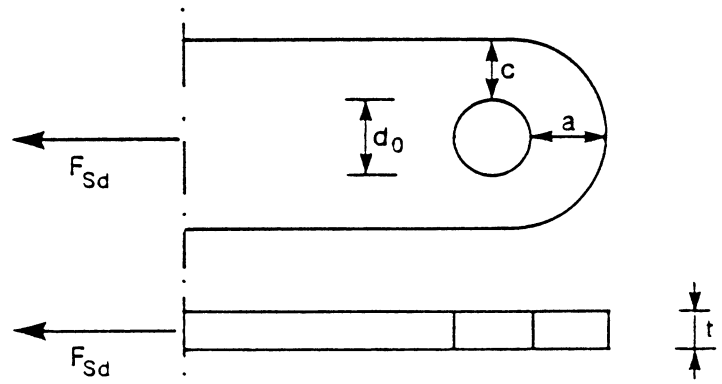
(2) At the ultimate limit state the design forces and moments in a pin shall not exceed the relevant design resistances given in Table 6.5.7.

Table 6.5.7 — Design resistances for pin connections

Criterion	Resistance
Shear of the pin	$F_{v.Rd} = 0{,}6\ A\ f_{up}/\gamma_{Mp}$
Bending of the pin	$M_{Rd} = 0{,}8\ W_{e\ell} f_{yp}/\gamma_{Mp}$
Combined shear and bending of the pin	$\left[\frac{M_{Sd}}{M_{Rd}}\right]^2 + \left[\frac{F_{v.Sd}}{F_{v.Rd}}\right]^2 \leq 1$
Bearing of the plate and the pin	$F_{b.Rd} = 1{,}5\ t\ d\ f_y/\gamma_{Mp}$

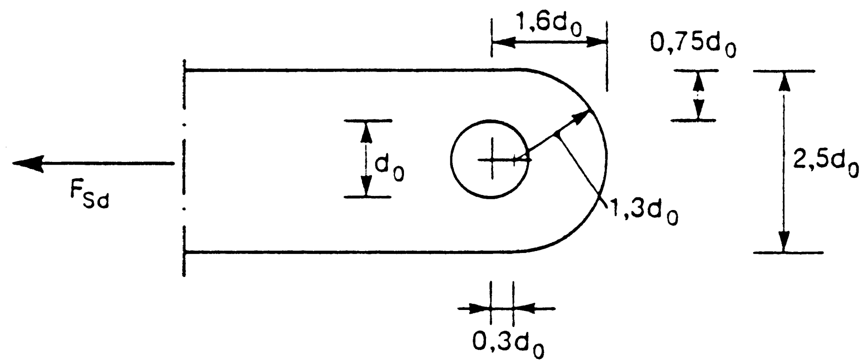
Table 6.5.6 — Geometrical conditions for plates in pin connections

Type A: Given thickness t

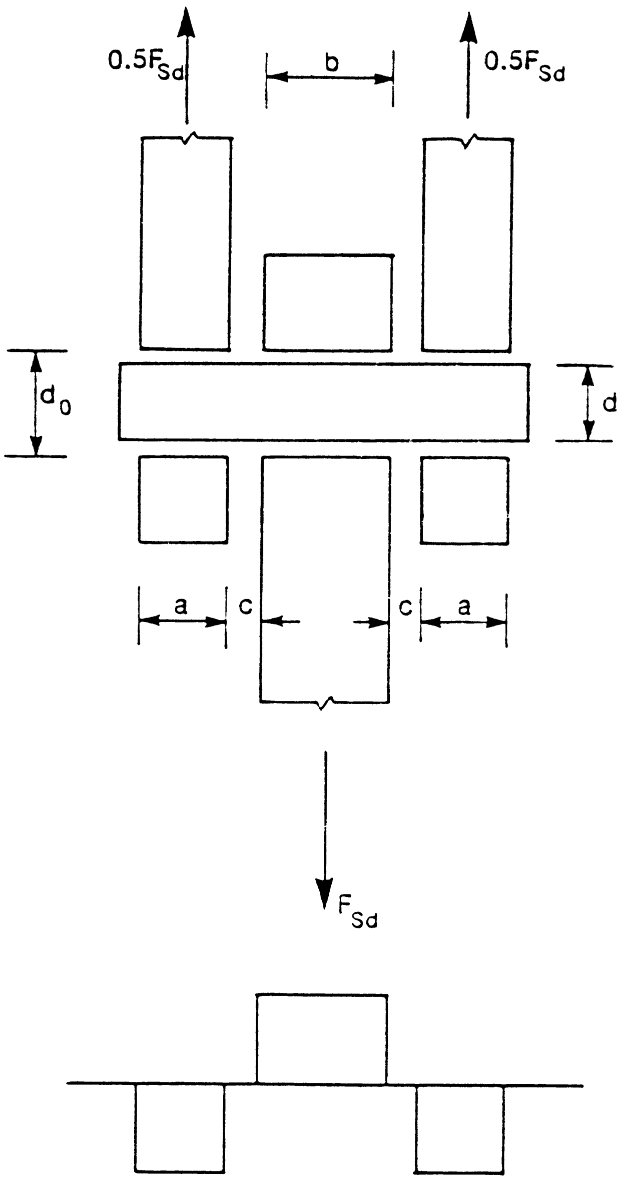


$$a \geq \frac{F_{Sd}\,\gamma_{Mp}}{2tf_y} + \frac{2d_0}{3} \quad : \quad c \geq \frac{F_{Sd}\,\gamma_{Mp}}{2tf_y} + \frac{d_0}{3}$$

Type B: Given geometry



$$t \geq 0{,}7 \left[\frac{F_{Sd}\,\gamma_{Mp}}{f_y}\right]^{1/2} \quad : \quad d_0 \leq 2{,}5t$$



$$M_{Sd} = \frac{F_{Sd}}{8}(b + 4c + 2a)$$

Figure 6.5.12 — Bending moment in a pin

6.6 Welded connections

6.6.1 General

(1) Connections made by welding shall comform with the relevant requirements concerning materials and execution specified in Chapter 3 and Chapter 7.

(2) The provisions of section **6.6** apply to:

- Weldable structural steels meeting the requirements of section **3.2** and Chapter 7.
- Welding by an arc welding process, defined in accordance with EN..... "*Welding processes*"[20] as follows:
  - 111 — metal-arc welding with covered electrodes
  - 114 — flux-cored arc welding (without gas shield)
  - 12 — submerged arc welding
  - 131 — MIG (metal inert gas) welding
  - 135 — MAG (metal active gas) welding
  - 136 — flux-cored wire metal-arc welding (with active gas shield)
  - 141 — TIG (tungsten inert gas) welding
- Material thicknesses of 4 mm and over. For welds in thinner material refer to ENV 1993-1-3 Eurocode 3-1.3[20].
- Joints in which the weld metal is compatible with the parent metal in terms of mechanical properties.

(3) Welds subject to fatigue shall also satisfy the requirements given in Chapter 9.

6.6.2 Geometry and dimensions

6.6.2.1 *Type of weld*

(1) For the purpose of this Eurocode, welds shall generally be classified as:

- fillet welds,
- slot welds,
- butt welds,
- plug welds, or
- flare groove welds.

(2) Butt welds may be either:

- full penetration butt welds, or
- partial penetration butt welds.

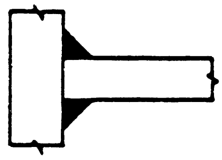
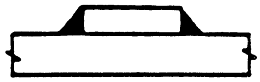
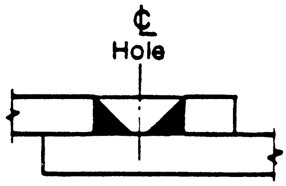
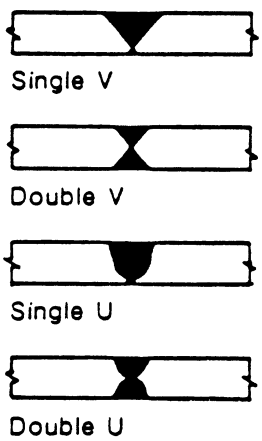
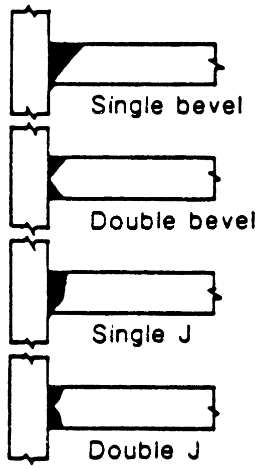
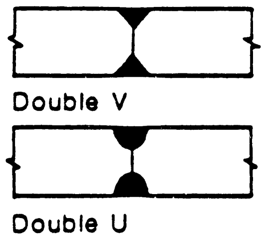
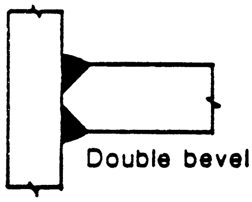
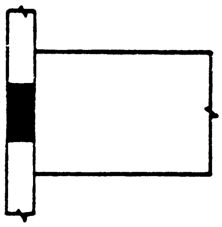
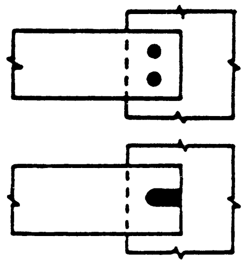
(3) Both slot welds and plug welds may be in either:

- circular holes, or
- elongated holes.

(4) *Weld classification is illustrated in Table 6.6.1.*

[20] In preparation

Table 6.6.1 — Common types of welded joints

Type of weld	Type of joint		
	Butt joint	Tee-butt joint	Lap joint
Fillet weld			
Slot weld			
Full penetration butt weld[a]			
Partial penetration butt weld[a]			
Plug weld			
Flare groove welds	See Figure 6.6.3 and Figure 6.6.4		

[a] Butt welds can sometimes be formed without cutting any weld preparation in either component

6.6.2.2 *Fillet welds*

(1) Fillet welds may be used for connecting parts where the fusion faces form an angle of between 60° and 120°.

(2) Smaller angles than 60° are also permitted. However, in such cases the weld shall be considered to be a partial penetration butt weld.

(3) For angles over 120°, fillet welds shall not be relied upon to transmit forces.

(4) *Fillet welds should not terminate at corners of parts or members, but should be returned continuously, full sized, around the corner for a length equal to twice the leg size of the weld, wherever such a return can be made in the same plane.*

(5) *End returns should be indicated on the drawings.*

(6) Fillet welds may be continuous or intermittent.

(7) Intermittent fillet welds shall not be used in corrosive conditions.

(8) In an intermittent fillet weld, the clear unconnected gaps between the ends of each length of weld (see Figure 6.6.1) shall not be more than the smallest of:

a) 200 mm;

b) 12 times the thickness of the thinner part when the part connected is in compression;

c) 16 times the thickness of the thinner part when the part connected is in tension;

d) one-quarter of the distance between stiffeners, when used to connect stiffeners to a plate or other part subjected to compression or shear.

(9) In an intermittent fillet weld, the clear unconnected gap shall be measured between the ends of welds on opposing sides or on the same side, whichever is shorter.

(10) In any run of intermittent fillet welds there shall always be a length of weld at each end of the part connected.

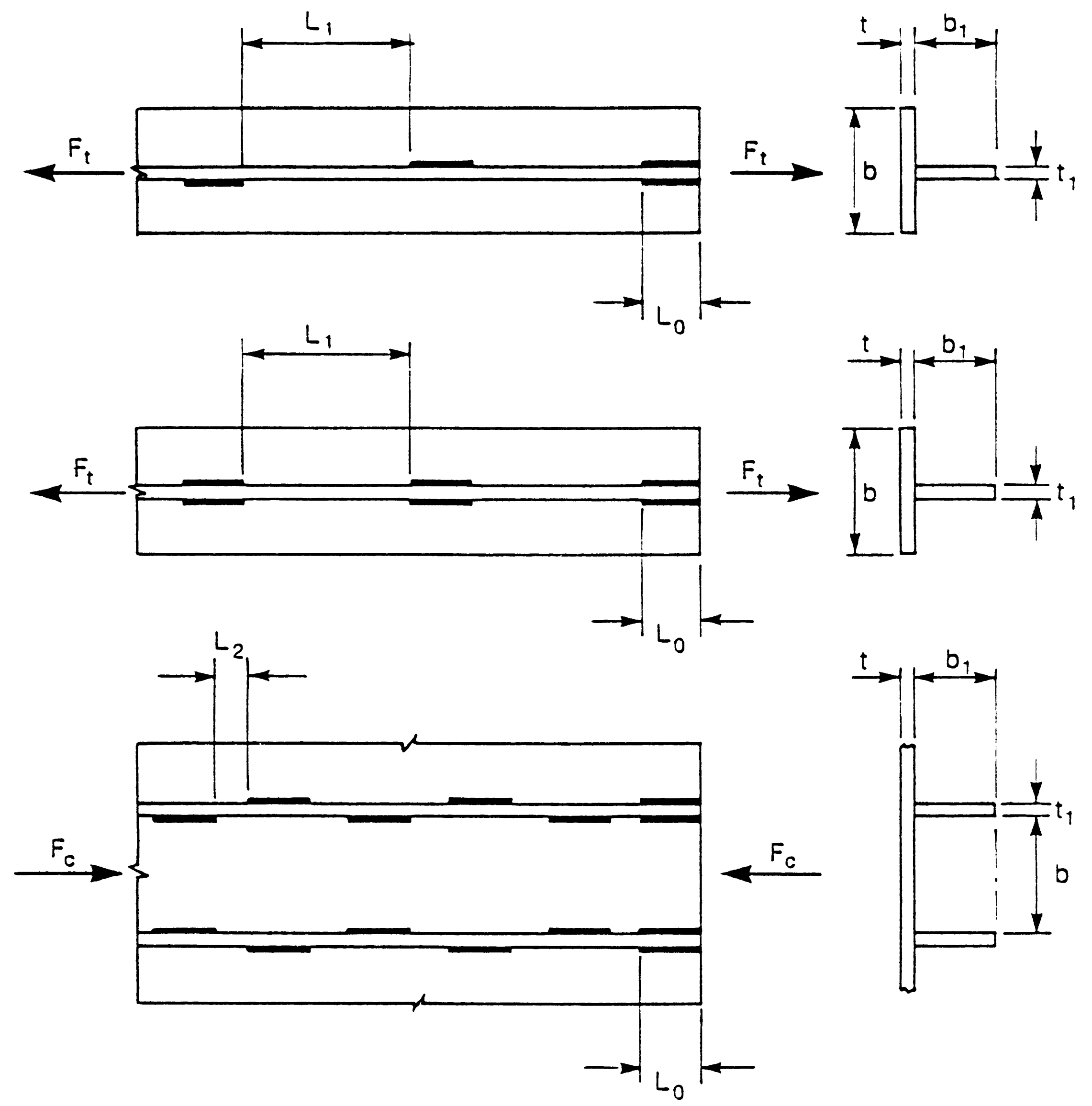
(11) In a fabricated member in which plates are connected by means of intermittent fillet welds, a continuous fillet weld shall be provided on each side of the plate for a length at each end equal to at least three-quarters of the width of the narrower plate concerned (see Figure 6.6.1).

(12) A single fillet weld shall not be used to transmit a bending moment about the longitudinal axis of the weld if it produces tension at the root of the weld, nor to transmit a significant tensile force perpendicular to the longitudinal axis of the weld in situations which would effectively produce such a bending moment.

(13) *A fillet weld may be used as part of a weld group around the perimeter of a structural hollow section, see Figure 6.6.2(a), but should not be used in the situation indicated in Figure 6.6.2(c).*

(14) When a single fillet weld is used to transmit a force perpendicular to its longitudinal axis, the eccentricity of the weld (relative to the line of action of the force to be resisted) shall be taken into account.

(15) *There is normally no eccentricity of this nature in welded connections of structural hollow sections.*



$L_0 \geq 0{,}75b$ or $0{,}75b_1$ — whichever is smaller
$L_1 \leq 16t$ or $16t_1$ or 200 mm — whichever is smallest
$L_2 \leq 12t$ or $12t_1$ or $0{,}25b$ or 200 mm — whichever is smallest

Figure 6.6.1 — Intermittent fillet welds

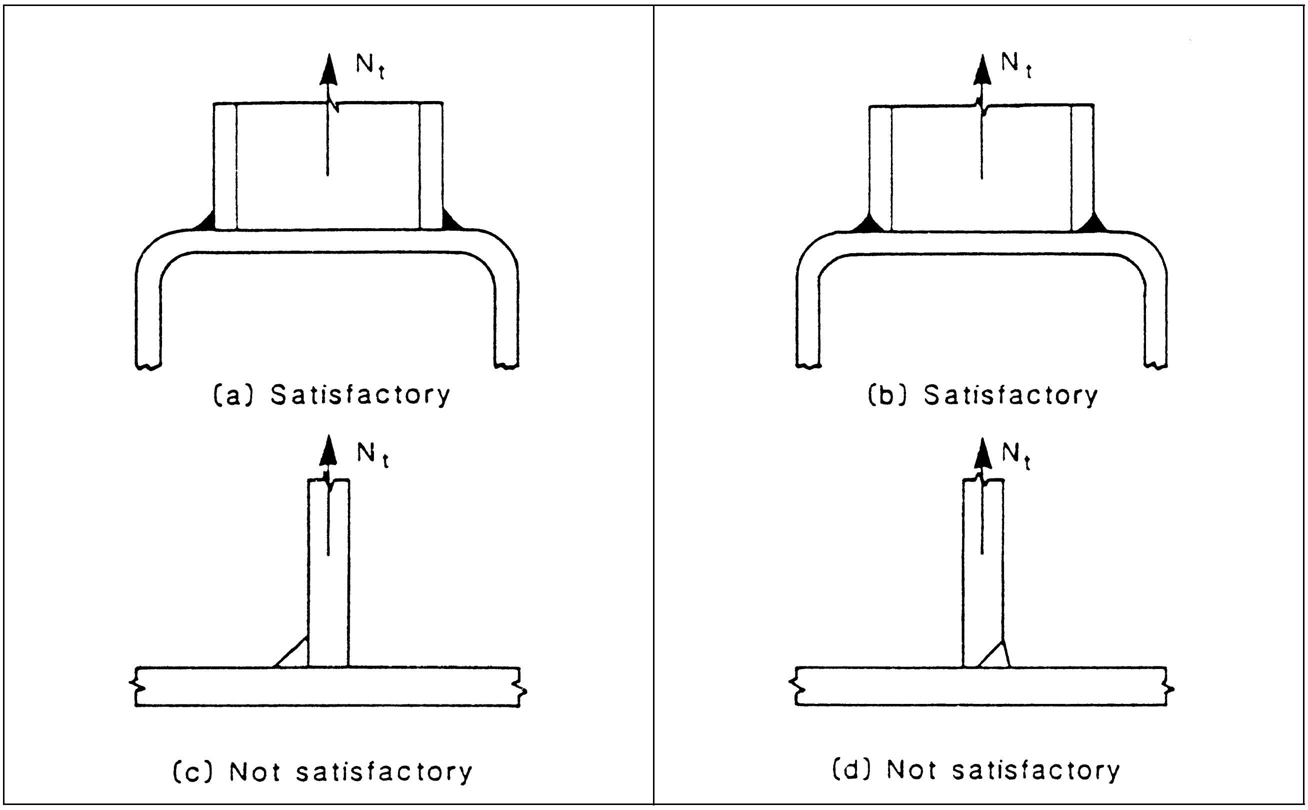


Figure 6.6.2 — Single fillet welds and single-sided partial penetration butt welds

6.6.2.3 ***Slot welds***

(1) Slot welds, comprising fillet welds in circular or elongated holes, may be used only to transmit shear or to prevent the buckling or separation of lapped parts.

(2) The diameter of a circular hole, or width of an elongated hole, for a slot weld shall not be less than four times the thickness of the part containing it.

(3) The ends of elongated holes shall be semi-circular, except for those ends which extend to the edge of the part concerned.

6.6.2.4 ***Butt welds***

(1) A full penetration butt weld is defined as a butt weld that has complete penetration and fusion of weld and parent metal throughout the thickness of the joint.

(2) A partial penetration butt weld is defined as a butt weld that has joint penetration which is less than the full thickness of the parent material.

(3) A single-sided partial penetration butt weld shall not be used to transmit a bending moment about the longitudinal axis of the weld if it produces tension at the root of the weld, nor to transmit a significant tensile force perpendicular to the longitudinal axis of the weld in situations which would effectively produce such a bending moment.

(4) *A single sided partial penetration butt weld may be used as part of a weld group around the perimeter of a structural hollow section, see Figure 6.6.2(b), but should not be used in the situation indicated in Figure 6.6.2(d).*

(5) When a single-sided partial penetration weld is used to transmit a force perpendicular to its longitudinal axis, the eccentricity of the weld (relative to the line of action of the force to be resisted) shall be taken into account.

(6) *There is normally no eccentricity of this nature in welded connections of structural hollow sections.*

(7) Intermittent butt welds shall not be used.

6.6.2.5 ***Plug welds***

(1) Plug welds, comprising welds which fill circular or elongated holes, shall not be used to resist externally applied tension, but they may be used:

- to transmit shear, or
- to prevent the buckling or separation of lapped parts, or
- to inter-connect the components of built-up members.

(2) The diameter of a circular hole, or width of an elongated hole, for a slot weld shall be at least 8 mm more than the thickness of the part containing it.

(3) The ends of elongated holes shall either be semi-circular or else shall have corners which are rounded to a radius of not less than the thickness of the part containing the slot, except for those ends which extend to the edge of the part concerned.

(4) The thickness of a plug weld in material up to 16 mm thick shall be equal to the thickness of the material. The thickness of a plug weld in material over 16 mm thick shall be at least half the thickness of the material and not less than 16 mm.

(5) The centre to centre spacing of plug welds shall not exceed the value necessary to prevent local buckling.

6.6.2.6 ***Flare groove welds***

(1) In rectangular structural hollow sections the effective throat thickness of flare-V and flare-bevel-groove welds (see Figure 6.6.3) shall be determined by means of trial welds for each set of procedural conditions.

(2) The trial welds shall be sectioned and measured to establish welding techniques that will ensure that the design throat thickness is achieved in production.

(3) For solid bars the same procedure shall be used to determine the effective throat thickness of flare-groove welds, when fitted flush to the surface of the solid section of the bars (see Figure 6.6.4).

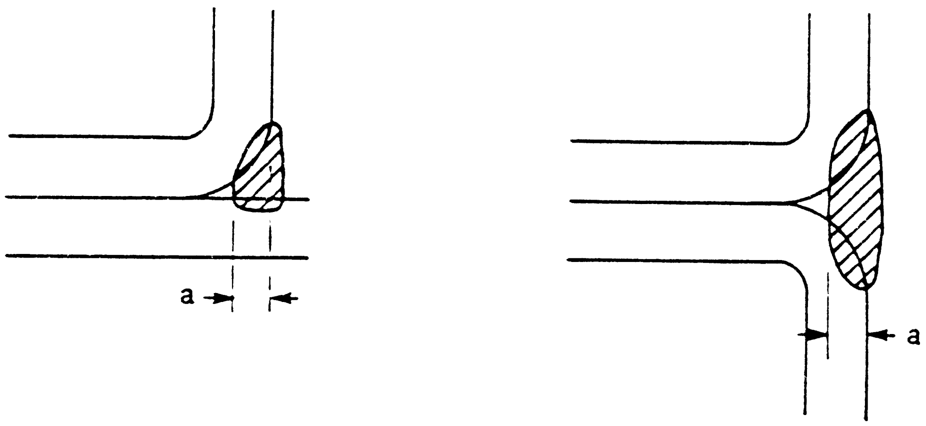


Figure 6.6.3 — Effective throat of flare groove welds in rectangular structural hollow sections

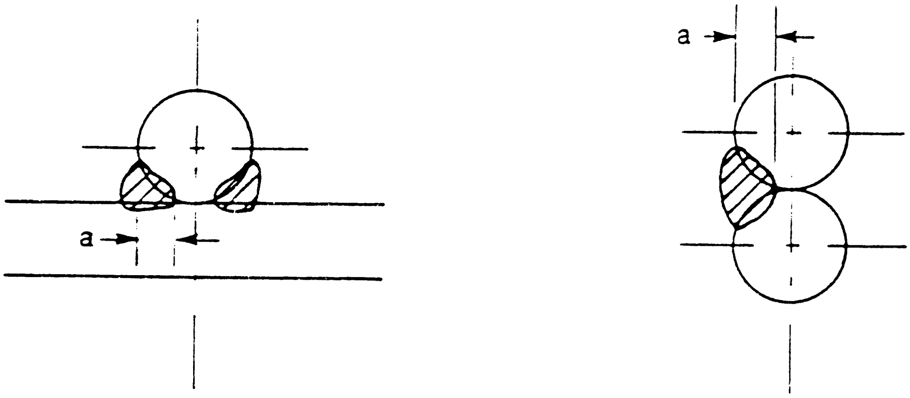


Figure 6.6.4 — Effective throat of flare groove welds in solid sections

6.6.3 Lamellar tearing

(1) Joint details causing through-thickness stresses originating from welding carried out under conditions of restraint shall be avoided whenever possible.

(2) Where such details are unavoidable, appropriate measures shall be taken to minimise the possibility of lamellar tearing.

(3) *If tensile stresses perpendicular to the surface of the part (due to external loads or due to residual welding stresses) occur in a flat part more than 15 mm thick, then the combination of the welding procedure, the through-thickness properties of the material and the joint detail, see Figure 6.6.5, should be such as to avoid lamellar tearing.*

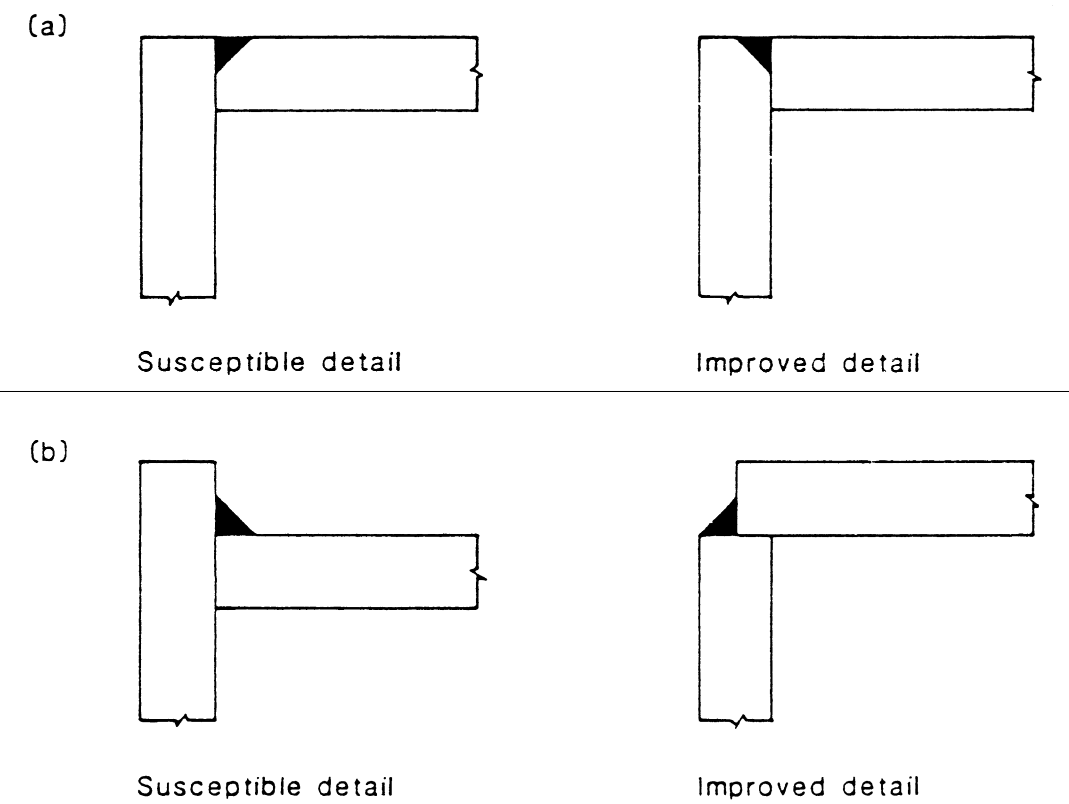


Figure 6.6.5 — Avoiding lamellar tearing

6.6.4 Distribution of forces

(1) The distribution of forces in a welded connection may be calculated on the assumption of either elastic or plastic behaviour in conformity with **6.1.3** and **6.1.4**.

(2) Normally it is acceptable to assume a simplified load distribution within the welds.

(3) Residual stresses and stresses not participating in the transfer of load need not be included when checking the resistance of a weld. This applies specifically to the normal stress parallel to the axis of a weld.

(4) Welded connections shall be designed to have adequate deformation capacity.

(5) In joints where plastic hinges may form, the welds shall be designed to provide at least the same design resistance as the weakest of the connected parts.

(6) In other joints where deformation capacity for joint rotation is required due to the possibility of excessive straining, the welds require sufficient strength not to rupture before general yielding in the adjacent parent material.

(7) *In general this will be satisfied if the design resistance of the weld is not less than 80 % of the design resistance of the weakest of the connected parts.*

6.6.5 Design resistance of a fillet weld

6.6.5.1 *Effective length*

(1) The effective length of a fillet weld shall be taken as the overall length of the full-size fillet, including end returns. Provided that the weld is full size throughout this length, no reduction in effective length need be made for either the start or the termination of the weld.

(2) *Welds with effective lengths shorter than 40 mm or 6 times the throat thickness, whichever is larger, should be ignored for transmission of forces.*

(3) Where the stress distribution along a weld is significantly influenced by the stiffness of the members or parts joined, the non-uniformity of the stress distribution may be neglected, provided that the design resistance is correspondingly reduced.

(4) *The effective breadths of welded joints designed to transfer transverse loads to an unstiffened flange of an I, H or box section should be reduced as specified in **6.6.8**.*

(5) *The design resistances of welds in long joints should be reduced as specified in **6.6.9**.*

6.6.5.2 *Throat thickness*

(1) The throat thickness, a, of a fillet weld shall be taken as the height of the largest triangle which can be inscribed within the fusion faces and the weld surface, measured perpendicular to the outer side of this triangle, see Figure 6.6.6.

(2) *The throat thickness of a fillet weld should not be less than 3 mm.*

(3) In determining the resistance of a deep penetration fillet weld, account may be taken of its additional throat thickness, see Figure 6.6.7, provided that it is shown by preliminary trials that the required penetration can consistently be achieved.

(4) *In the case of a fillet weld made by an automatic submerged arc process, the throat thickness may be increased by 20 % or 2 mm, whichever is smaller, without resorting to preliminary trials.*

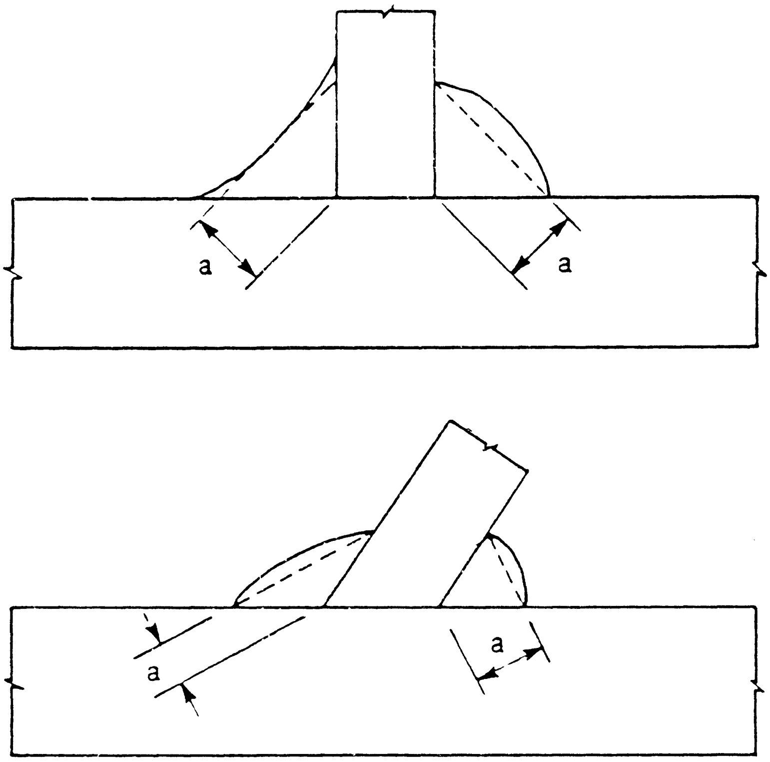


Figure 6.6.6 — Throat thickness of a fillet weld

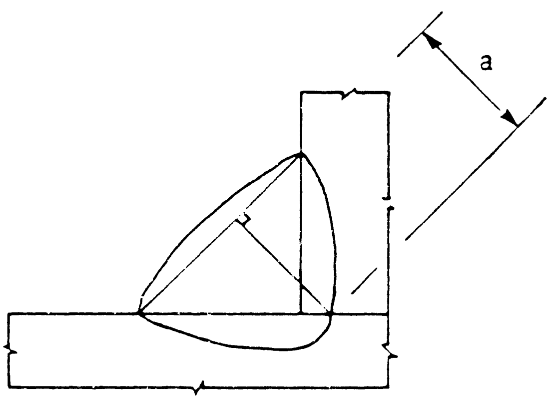


Figure 6.6.7 — Throat thickness of a deep penetration fillet weld

6.6.5.3 ***Resistance per unit length***

(1) The design resistance per unit length of a fillet weld shall be determined using either the method given below, or else the alternative method given in normative Annex M.

(2) The resistance of a fillet weld may be assumed to be adequate if, at every point in its length, the resultant of all the forces per unit length transmitted by the weld does not exceed its design resistance $F_{w.Rd}$.

(3) Independent of the orientation of the weld, the design resistance per unit length $F_{w.Rd}$ shall be determined from:

$$F_{w.Rd} = f_{vw.d}\ a \qquad (6.14)$$

where $f_{vw.d}$ is the design shear strength of the weld.

(4) The design shear strength $f_{vw.d}$ of the weld shall be determined from:

$$f_{vw.d} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_{Mw}} \tag{6.15}$$

where f_u is the nominal ultimate tensile strength of the weaker part joined.

and β_w is the appropriate correlation factor.

(5) *The value of the correlation factor β_w should be taken as follows:*

Steel grade	*Ultimate tensile strength f_u*	*Correlation factor β_w*
EN 10025:		
Fe 360	*360 N/mm²*	*0,8*
Fe 430	*430 N/mm²*	*0,85*
Fe 510	*510 N/mm²*	*0,9*
prEN 10113:		
Fe E 275	*390 N/mm²*	*0,8*
Fe E 355	*490 N/mm²*	*0,9*

(6) *For intermediate values of f_u, the value of β_w may be determined by linear interpolation.*

6.6.6 Design resistance of butt welds

6.6.6.1 *Full penetration butt welds*

(1) The design resistance of a full penetration butt weld shall be taken as equal to the design resistance of the weaker of the parts joined, provided that the weld is made with a suitable electrode (or other welding consumable) which will produce all-weld tensile specimens having both a minimum yield strength and a minimum tensile strength not less than those specified for the parent metal.

6.6.6.2 *Partial penetration butt welds*

(1) The resistance of a partial penetration butt weld shall be determined as for a deep penetration fillet weld, see **6.6.5**.

(2) The throat thickness of a partial penetration butt weld shall be taken as the depth of penetration that can consistently be achieved.

(3) The throat thickness that can consistently be achieved may be determined by preliminary trials.

(4) *Where the weld preparation is of the U, V, J or bevel type, see Figure 6.6.8, the throat thickness should be taken as the nominal depth of preparation minus 2 mm, unless a larger value is shown to be justified by preliminary trials.*

6.6.6.3 *Tee-butt joints*

(1) The resistance of a tee-butt joint, consisting of a pair of partial penetration butt welds reinforced by superimposed fillet welds, may be determined as for a full penetration butt weld (see **6.6.6.1**) if the total nominal throat thickness, exclusive of the unwelded gap, is not less than the thickness t of the part forming the stem of the tee joint, provided that the unwelded gap is not more than (t/5) or 3 mm, whichever is less, see Figure 6.6.9(a).

(2) The resistance of a tee-butt joint which does not meet the requirements given in (1) shall be determined as for a deep penetration fillet weld, see **6.6.5**. The throat thickness shall be determined in conformity with the provisions for both fillet welds (see **6.6.5.2**) and partial penetration butt welds (see **6.6.6.2**).

(3) *The throat thickness should be taken as the nominal throat thickness minus 2 mm [see Figure 6.6.9(b)], unless a larger value is shown to be justified by preliminary trials.*

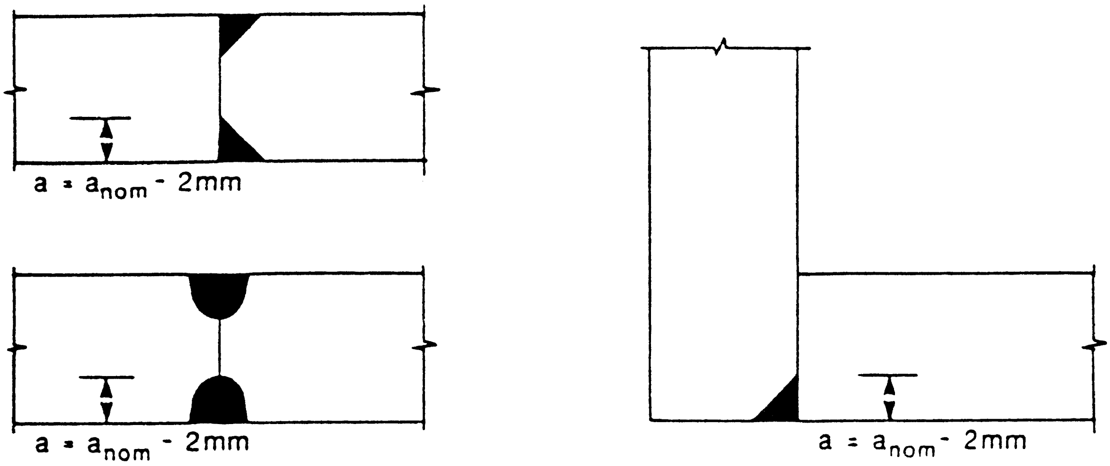


Figure 6.6.8 — Partial penetration butt welds

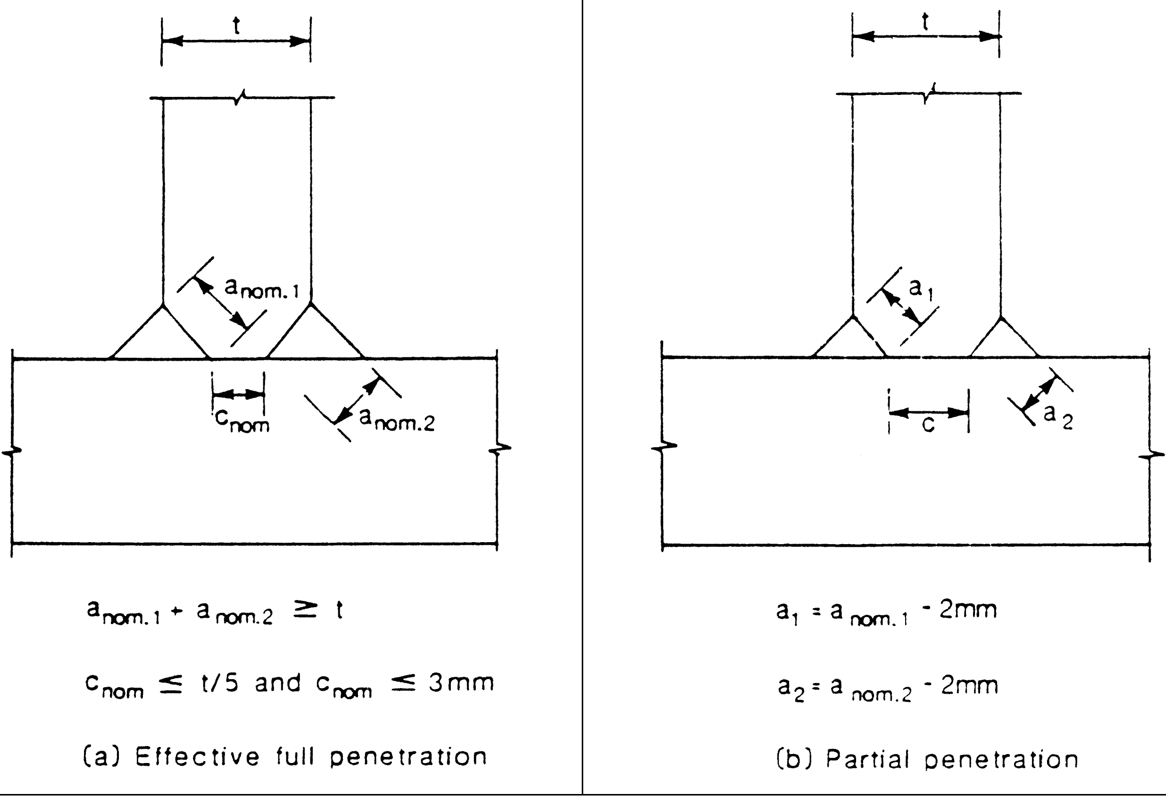


Figure 6.6.9 — Tee-butt welds

6.6.7 Design resistance of plug welds

(1) The design resistance $F_{w.Rd}$ of a plug weld (see **6.6.2.5**) shall be taken as $f_{vw.d}A_w$, where $f_{vw.d}$ is the design shear strength of a weld given in **6.6.5.3**(4).

(2) The effective area A_w of a plug weld shall be taken as the area of the hole.

(3) Slot welds (see **6.6.2.3**) shall be considered as fillet welds. The design resistance of a slot weld shall be determined from **6.6.5**.

6.6.8 Joints to unstiffened flanges

(1) In a tee-joint of a plate to an unstiffened flange of an I, H or a box section, a reduced effective breadth shall be taken into account both for the parent material and for the welds, see Figure 6.6.10.

(2) *For an I or H sectlon the effective breadth b_{eff} should be obtained from:*

$$b_{eff} = t_w + 2r + 7t_f \quad \text{but} \quad b_{eff} \leq t_w + 2r + 7(t_f^2/t_p)(f_y/f_{yp}) \qquad (6.16)$$

where f_Y is the design strength of the member

and f_{Yp} is the design strength of the plate.

(3) *If b_{eff} is less than 0,7 times the full breadth, the joint should be stiffened.*

(4) *For a box section the effective breadth b_{eff} should be obtained from:*

$$b_{eff} = 2t_w + 5t_f \quad \text{but} \quad b_{eff} \leq 2t_w + 5(t_f^2/t_p)\,(f_y/f_{yp}) \qquad (6.17)$$

(5) The welds connecting the plate to the flange shall have a design resistance per unit length not less than the design resistance per unit width of the plate.

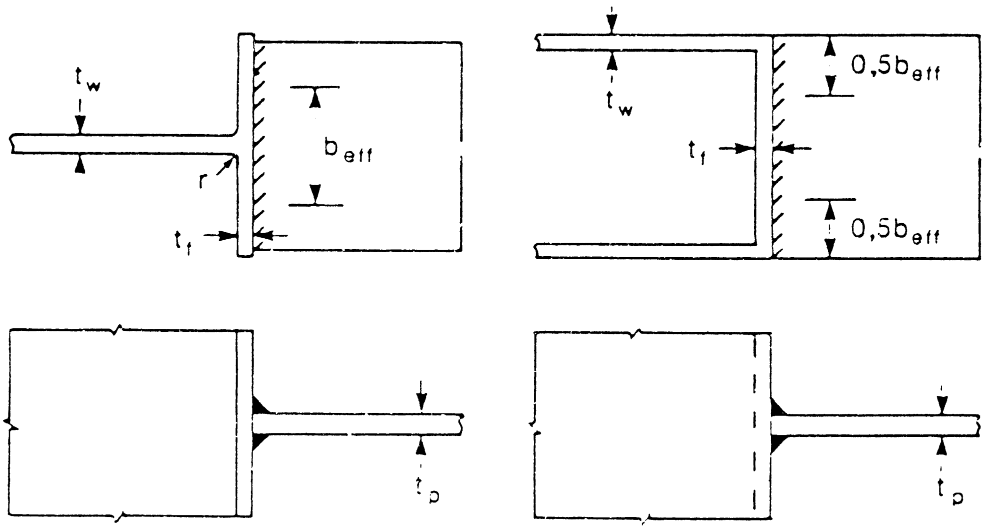


Figure 6.6.10 — Effective breadth of an unstiffened tee-joint

6.6.9 Long joints

(1) In lap joints the design resistance of a fillet weld shall be reduced by multiplying it by a reduction factor β_{Lw} to allow for the effects of non-uniform distribution of stress along its length.

(2) This provision does not apply when the stress distribution along the weld corresponds to the stress distribution in the adjacent base metal, as, for example, in the case of a weld connecting the flange and the web of a plate girder.

(3) *Generally in lap joints longer than 150a the reduction factor β_{Lw} should be taken as $\beta_{Lw.1}$ given by:*

$$\beta_{Lw.1} = 1{,}2 - 0{,}2L_j/(150a) \qquad (6.18)$$

but $\beta_{Lw.1} \leq 1{,}0$

where L_j is the overall length of the lap in the direction of the force transfer.

(4) *For fillet welds longer than 1,7 metres connecting transverse stiffeners in plated members, the reduction factor β_{Lw} may be taken as $\beta_{Lw.2}$ given by:*

$$\beta_{Lw.2} = 1{,}1 - L_w/17 \qquad (6.19)$$

but $\beta_{Lw.2} \leq 1{,}0$ and $\beta_{Lw.2} \geq 0{,}6$

where L_j is the length of the weld (in metres).

6.6.10 Angles connected by one leg

(1) In angles connected by one leg, the eccentricity of welded lap joint end connections may be allowed for by adopting an effective cross-sectional area and then treating the member as concentrically loaded.

(2) *For an equal-leg angle, or an unequal-leg angle connected by its larger leg, the effective area may be taken as equal to the gross area.*

(3) *For an unequal-leg angle connected by its smaller leg, the effective area should be taken as equal to the gross cross-sectional area of an equivalent equal-leg angle of leg size equal to that of the smaller leg, when determining the design resistance of the cross-section, see **5.4.3** and **5.4.4**. However when determining the design buckling resistance of a compression member, see **5.5.1**, the actual gross cross-sectional area should be used.*

6.7 Hybrid connections

(1) When different forms of fasteners are used to carry a shear load or when welding and fasteners are used in combination, see Figure 6.7.1, then one form of connector shall normally be designed to carry the total load.

(2) As an exception to this provision, preloaded high-strength bolts in connections designed as slip-resistant at the ultimate limit state (Category C in **6.5.3.1**) may be assumed to share load with welds, provided that the final tightening of the bolts is carried out after the welding is complete.

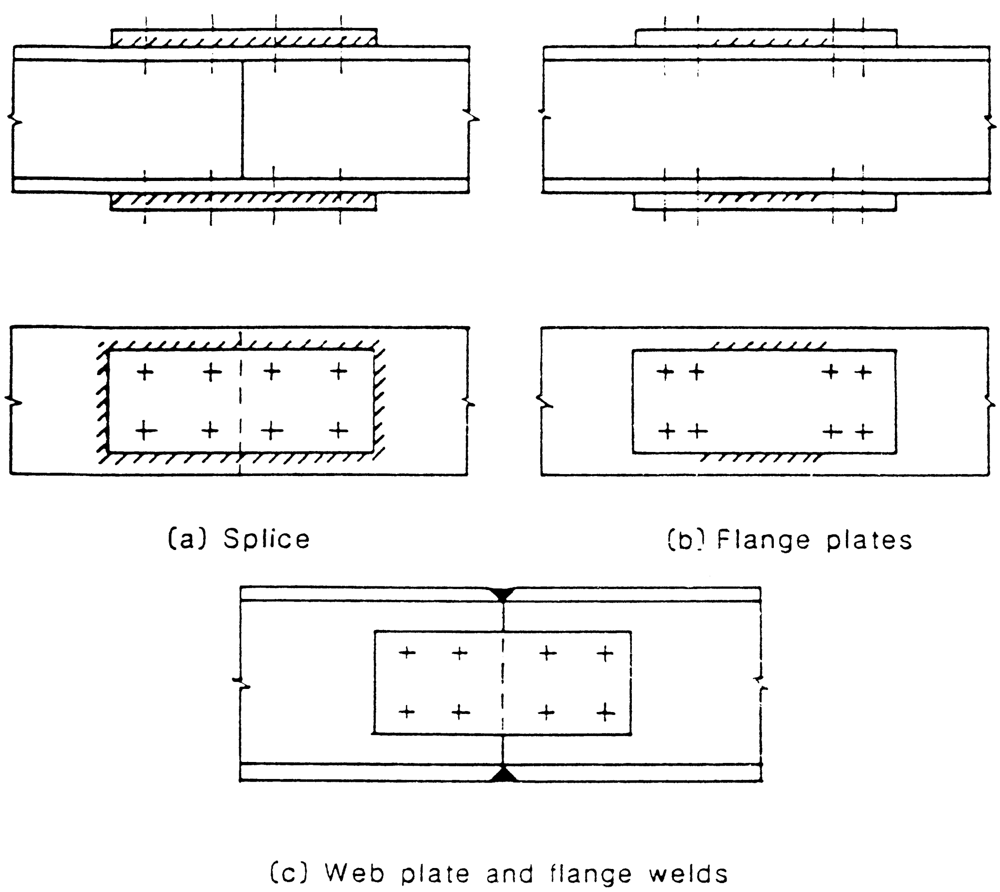


Figure 6.7.1 — Hybrid connections

6.8 Splices

6.8.1 General

(1) This section applies to the design of joints within the length of a member or other structural part.

(2) Splices shall be designed to hold the connected members in place.

(3) Wherever practicable the members shall be arranged so that the centroidal axis of any splice material coincides with the centroidal axis of the member. If eccentricity is present then the resulting forces shall be taken into account.

6.8.2 Splices in compression members

(1) Where the members are not prepared for full contact in bearing, splice material shall be provided to transmit the internal forces and moments in the member at the spliced section, including the moments due to applied eccentricity, initial imperfections and second order deformations.

(2) Where the members are prepared for full contact in bearing, the splice shall be designed to provide continuity of stiffness about both axes and to resist any tension where moments are present for any reason, including those given in (1).

(3) The alignment of the abutting ends shall be maintained by cover plates or other means. The splice material and its fastenings shall be proportioned to carry a force at the abutting ends, acting in any direction perpendicular to the axis of the member, of not less than 2,5 % of the compressive force in the member.

6.8.3 Splices in tension members

(1) A splice in a member or part subject to tension shall be designed to transmit all the moments and forces to which the member of part is subjected at that point.

6.9 Beam-to-column connections

6.9.1 Basis

(1) The design moment resistance M_{Rd} of a beam-to-column connection shall not be less than the applied design moment M_{Sd}.

(2) The moment-rotation characteristics of a beam-to-column connection shall be consistent with the assumptions made in the global analysis of the structure and with the assumptions made in the design of the members, see **5.2.2.1**.

6.9.2 Moment-rotation characteristic

(1) The determination of the design moment-rotation characteristics of beam-to-column connections shall be based on theory supported by experimental evidence.

(2) As an approximation to the real behaviour, a beam-to-column connection may be represented by a rotational spring connecting the centre lines of the column and the connected beam at the point of intersection, as indicated in Figure 6.9.1.

(3) Generally the actual moment-rotation characteristic of a beam-to-column connection is non-linear.

(4) An approximate design moment-rotation characteristic may be derived from a more precise characteristic by adopting any appropriate curve, including a linearised approximation (e.g. bi-linear or tri-linear), provided that the approximate curve lies wholly below the more precise characteristic, see Figure 6.9.2.

(5) A design moment-rotation characteristic (see Figure 6.9.3) shall define three main properties, as follows:

- moment resistance (see **6.9.3**)
- rotational stiffness (see **6.9.4**)
- rotation capacity (see **6.9.5**)

(6)*When using elastic global analysis it is not necessary to consider the rotation capacity of rigid or se mi-rigid connections, see* **6.4.2**.

(7) In certain cases the moment-rotation behaviour of a beam-to-column connection includes some initial rotation due to bolt slip or lack of fit, as indicated in Figure 6.9.4. Where this occurs, an initial hinge rotation ϕ_o shall also be included in the design moment-rotation characteristic, see Figure 6.9.4(b).

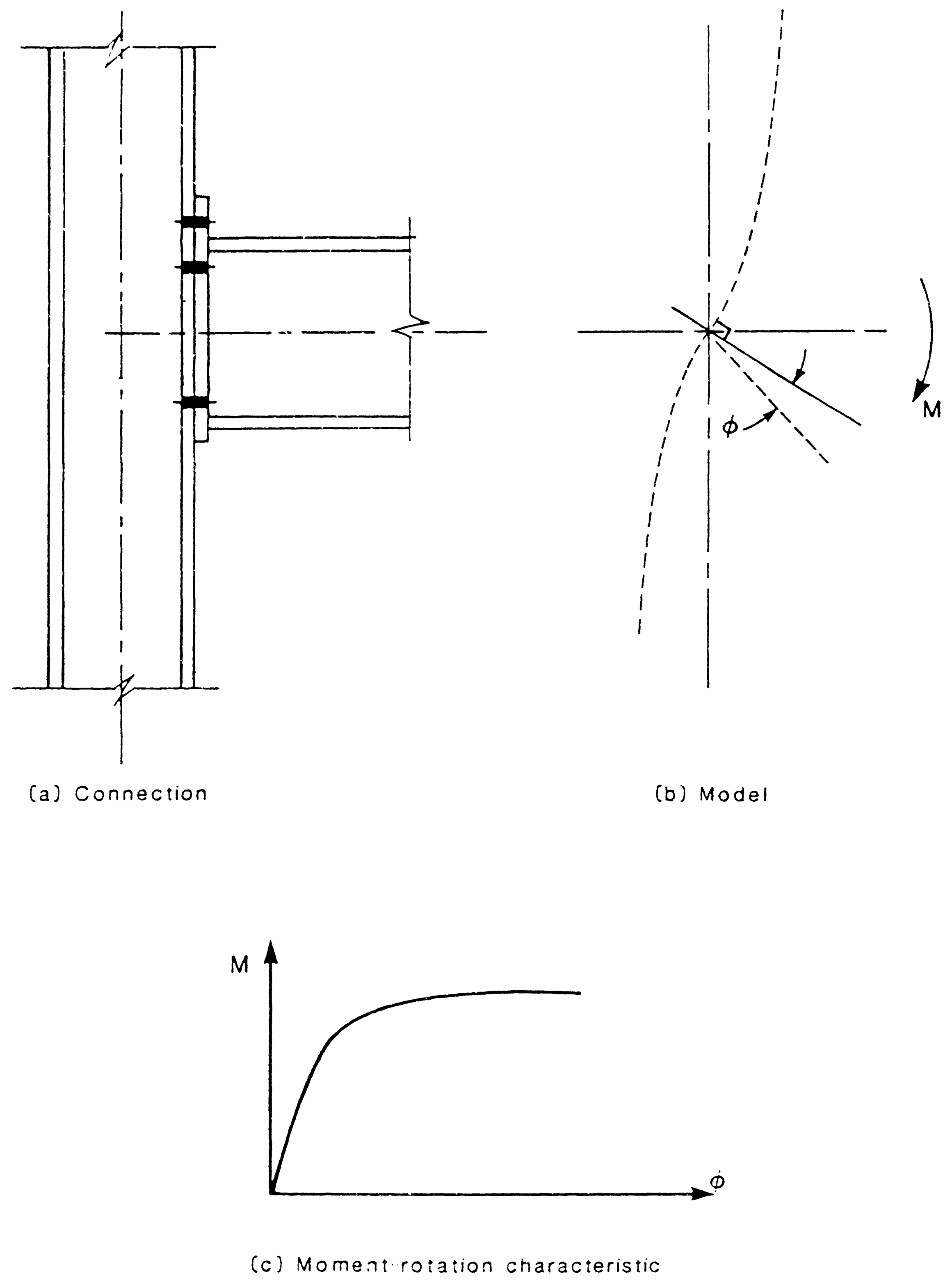


Figure 6.9.1 — Modelling a connection as a rotational spring

6.9.3 Moment resistance

(1) The design moment resistance M_{Rd} is equal to the peak value of the design moment-rotation characteristic.

6.9.4 Rotational stiffness

(1) Full benefit may be taken of a non-linear design moment-rotation characteristic by using incremental calculation procedures.

(2) Except as indicated in (1), the rotational stiffness S_j shall be taken as the secant stiffness as indicated in Figure 6.9.5.

(3) Different values of the secant stiffness may be used, depending on the design moment M_{Sd} for the load case and limit state under consideration, see Figure 6.9.6.

6.9.5 Rotation capacity

(1) The design rotation capacity ϕ_{Cd} of a beam-to-column connection shall be taken as the rotation achieved at the maximum design moment resistance of the connection, see Figure 6.9.7.

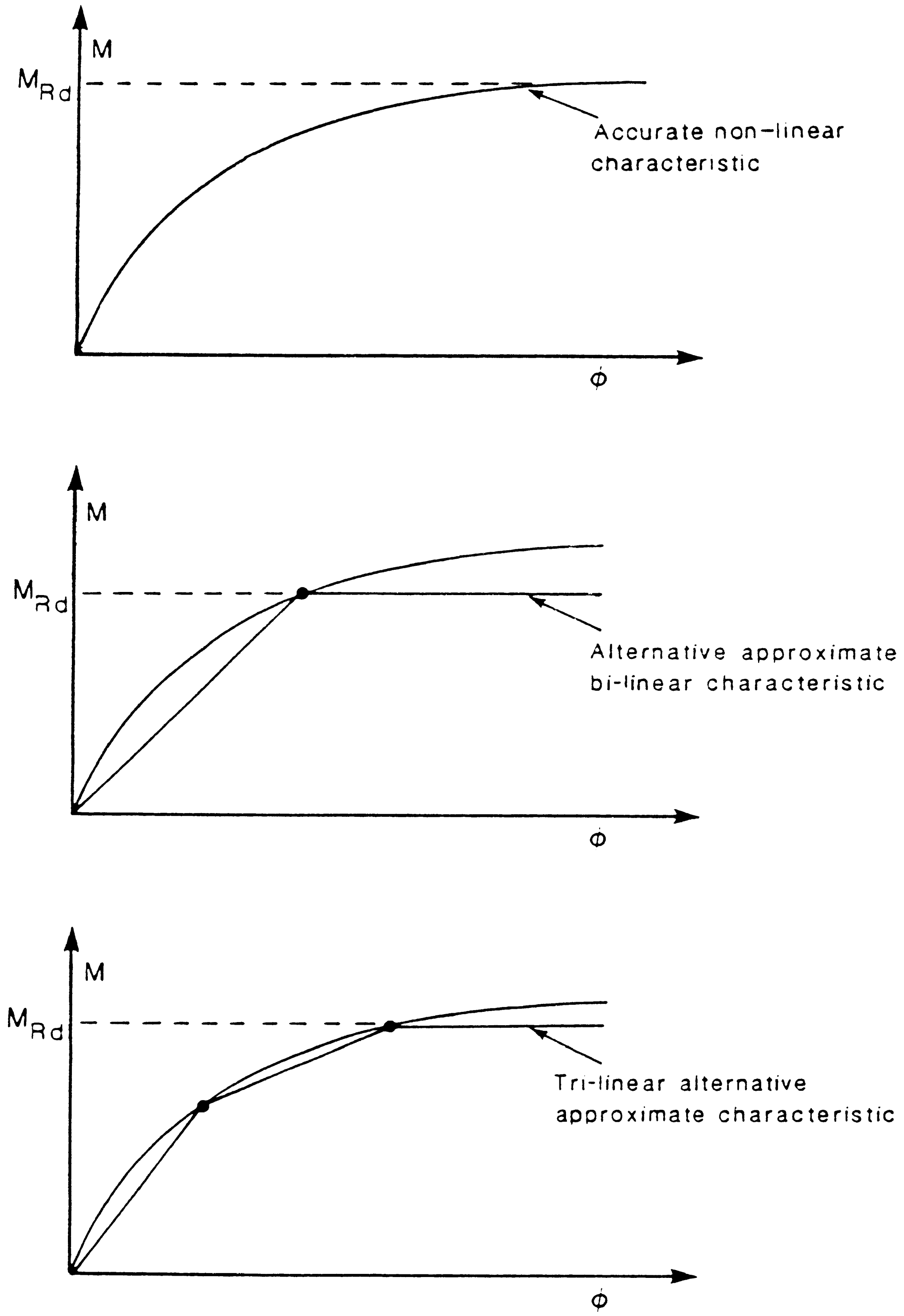
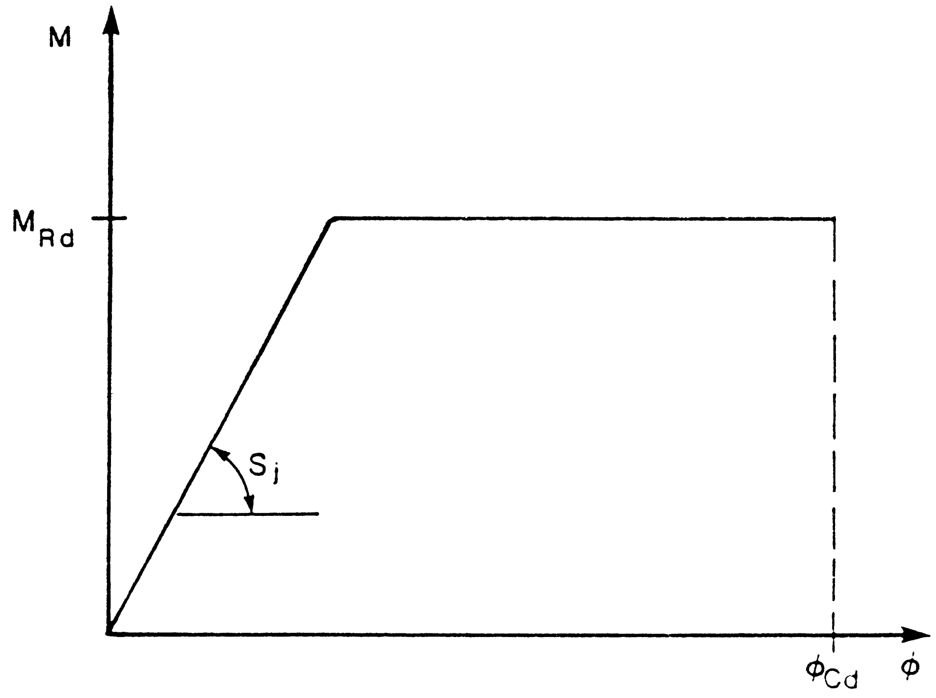


Figure 6.9.2 — Derivation of approximate moment-rotation characteristics

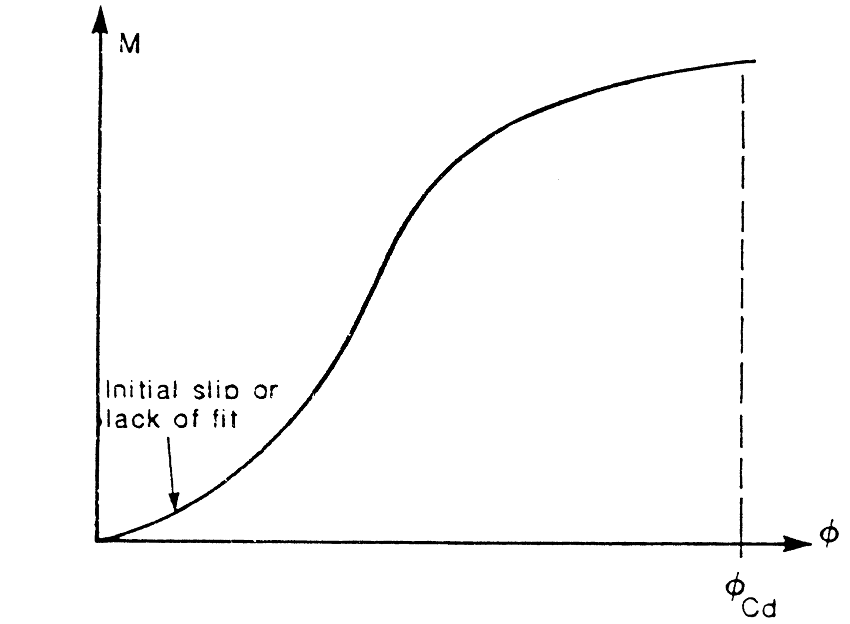


M_{Rd} = Design moment resistance

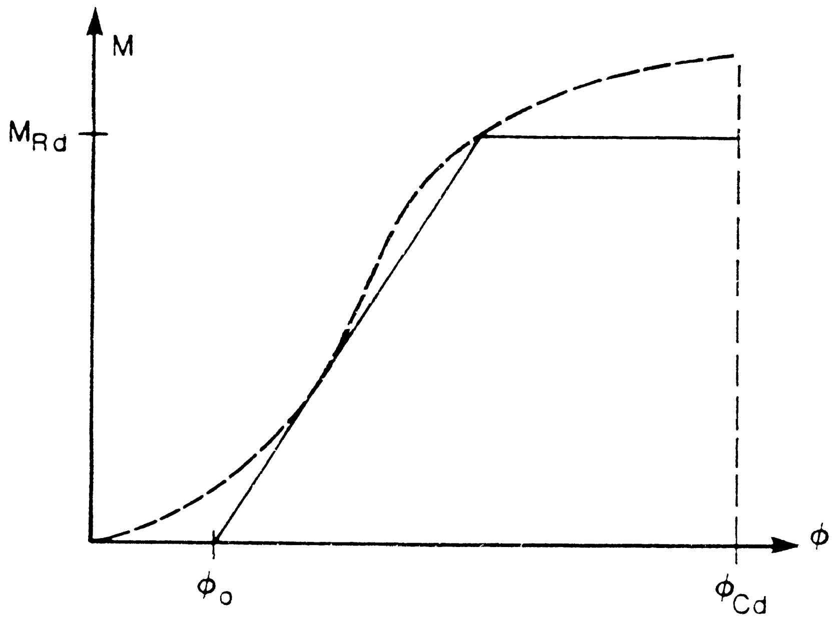
S_j = Rotational stiffness

ϕ_{Cd} = Design rotation capacity

Figure 6.9.3 — Properties of a design moment-rotation characteristic



(a) Actual moment-rotation characteristic



(b) Design moment-rotation characteristic

Figure 6.9.4 — Moment-rotation characteristic with an initial hinge rotation

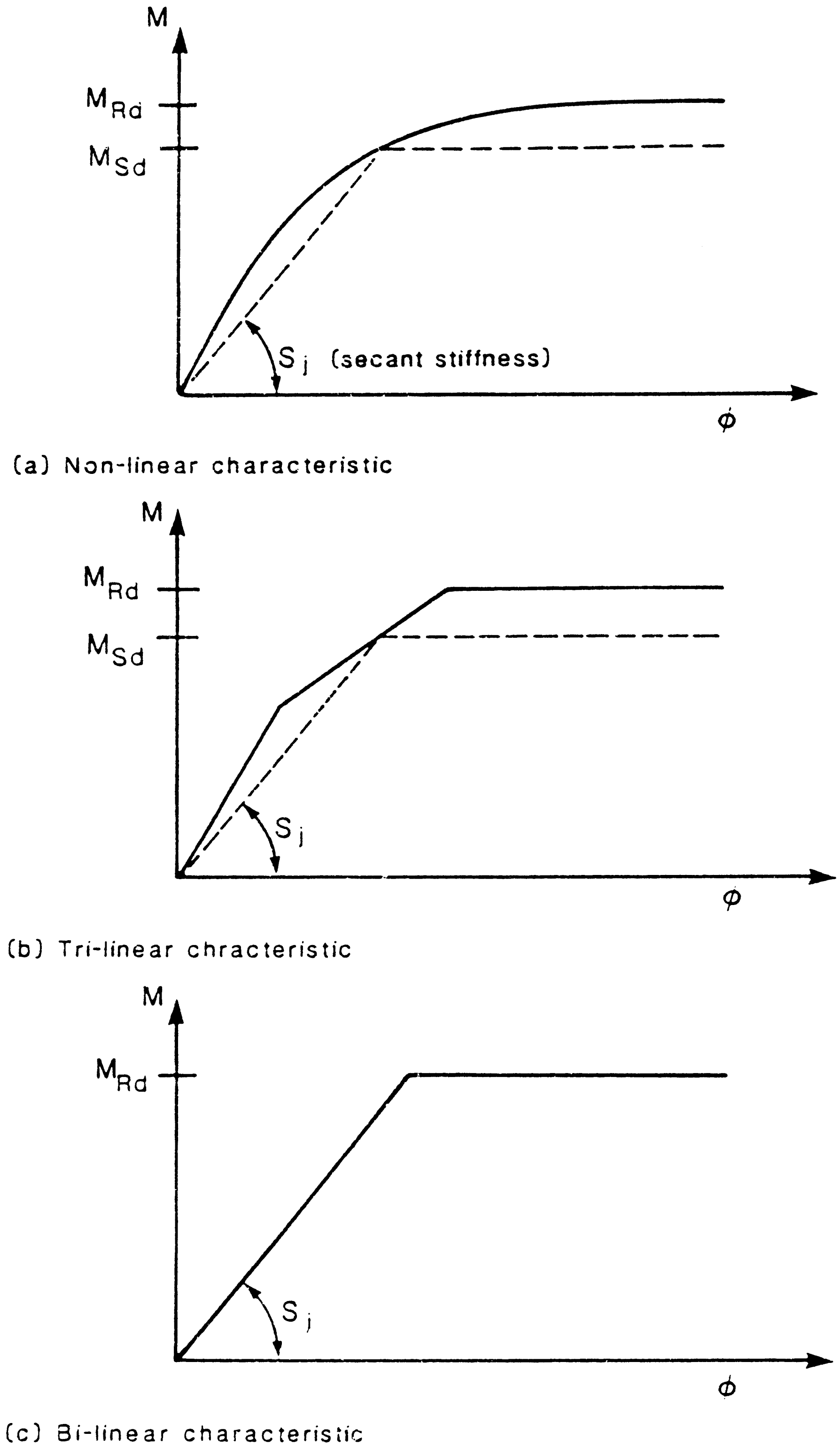


Figure 6.9.5 — Rotational stiffness S_j

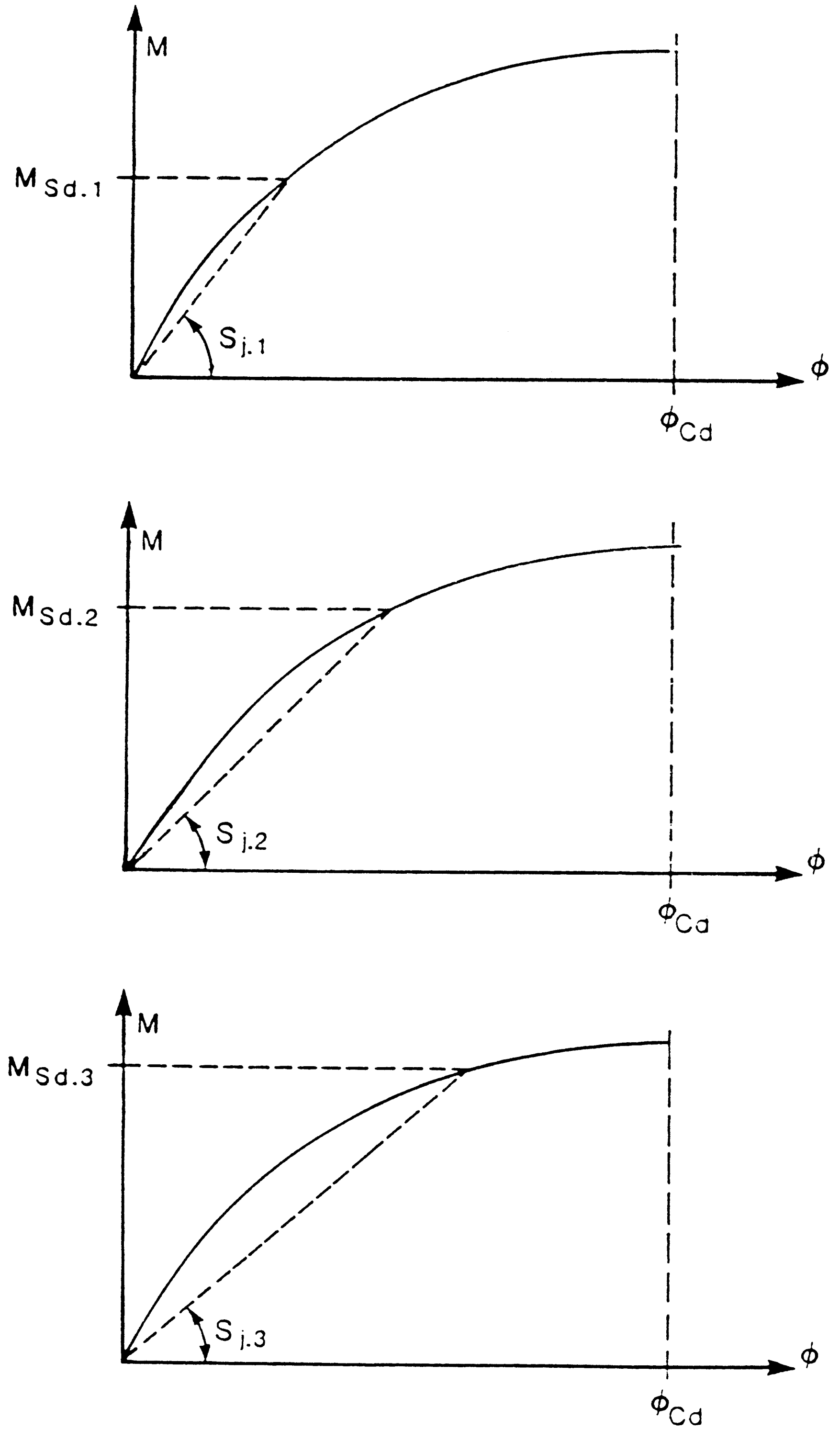


Figure 6.9.6 — Variation of rotational stiffness with applied moment

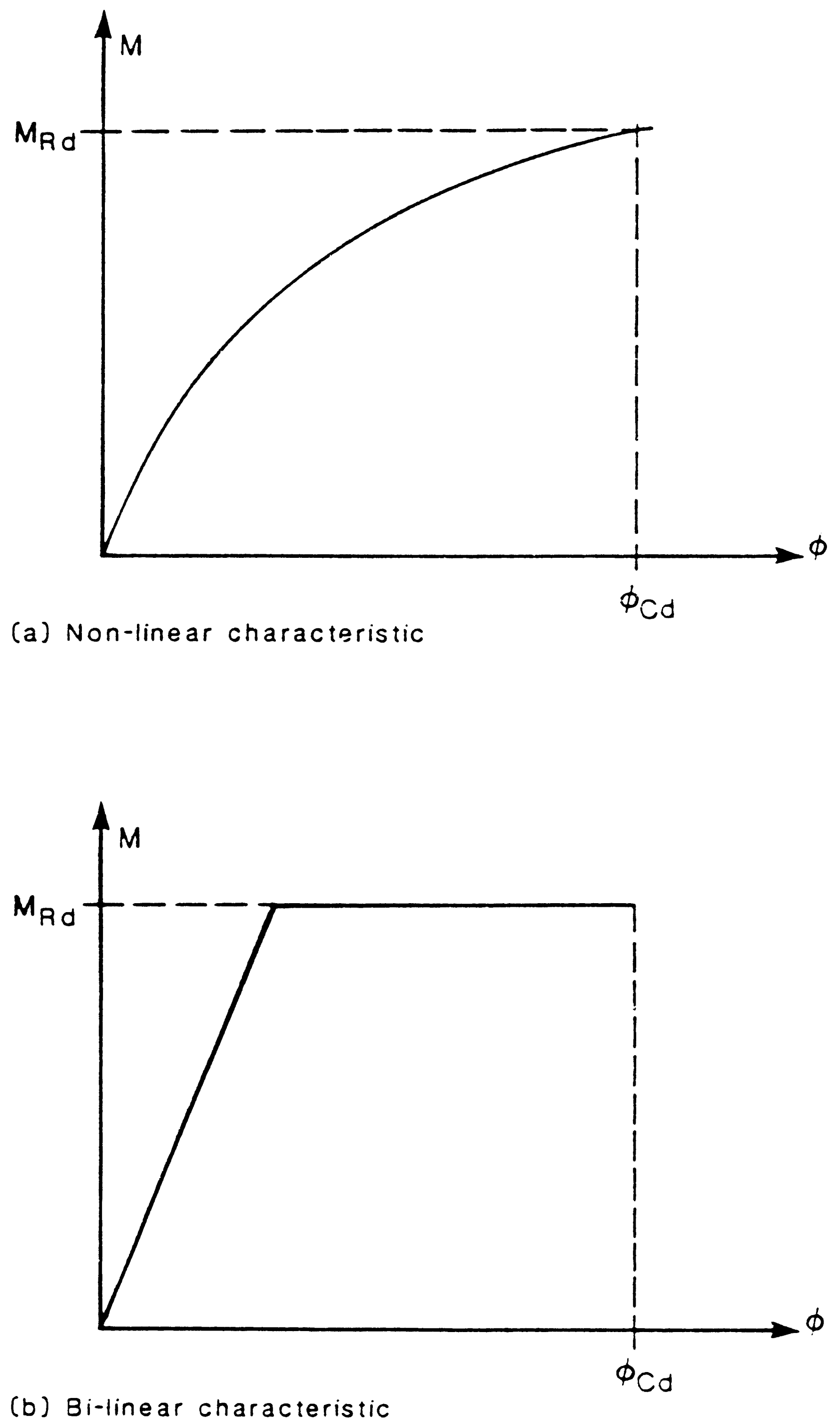


Figure 6.9.7 — Rotation capacity ϕ_{Cd}

6.9.6 Classification of beam-to-column connections

6.9.6.1 *Basis*

(1) Beam-to-column connections may be classified:

- by rotational stiffness, see **6.9.6.2**.
- by moment resistance, see **6.9.6.3**.

6.9.6.2 *Rotational stiffness*

(1) The rotational stiffness of a beam-to-column connection may be classified as:

- nominally pinned, see **6.4.2.1**.
- rigid, see **6.4.2.2**.
- semi-rigid, see **6.4.2.3**.

(2) A beam-to-column connection may be classified as rigid or nominally pinned on the basis of particular or general experimental evidence, or significant experience of previous satisfactory performance in similar cases or by calculations based on test evidence.

(3) A beam-to-column connection may be classified as nominally pinned if its rotational stiffness S_j *(based on a moment rotation characteristic representative of its actual anticipated behaviour) satisfies the condition:*

$$S_j \leq 0{,}5\ EI_b/L_b \tag{6.20}$$

where S_j *is the secant rotational stiffness of the connection, see* ***5.9.4***.

I_b *is the second moment of area of the connected beam.*

L_b *is the length of the connected beam.*

(4) *A beam-to-column connection in a braced frame, or in an unbraced frame which satisfies the condition specified in (5), may be considered to be rigid compared to the connected beam, if the rising portion of its moment-rotation characteristic lies above the solid line on the appropriate diagram in Figure 6.9.8.*

(5) *The line given in Figure 6.9.8(a) for an unbraced frame may be used only for frames in which every storey satisfies:*

$$K_b/K_c \geq 0{,}1 \tag{6.21}$$

in which K_b *is the mean value of* I_b/L_b *for all the beams at the top of that storey*

and K_c *is the mean value of* I_c/L_c *for all the columns in that storey*

where I_b *is the second moment of area of a beam*

I_c *is the second moment of area of a column*

L_b *is the span of a beam (centre-to-centre of columns)*

L_c *is the storey height for a column*

(6) *If the rising portion of its moment-rotation characteristic lies below the appropriate line in Figure 6.9.8, a beam-to-column connection should be classified as semi-rigid, unless it also satisfies the requirements for a nominally pinned connection.*

(7) *Connections which are classified as rigid or nominally pinned, may optionally be treated as semi-rigid.*

6.9.6.3 *Moment resistance*

(1) With respect to the design moment resistance, beam-to-column connections may be classified as:

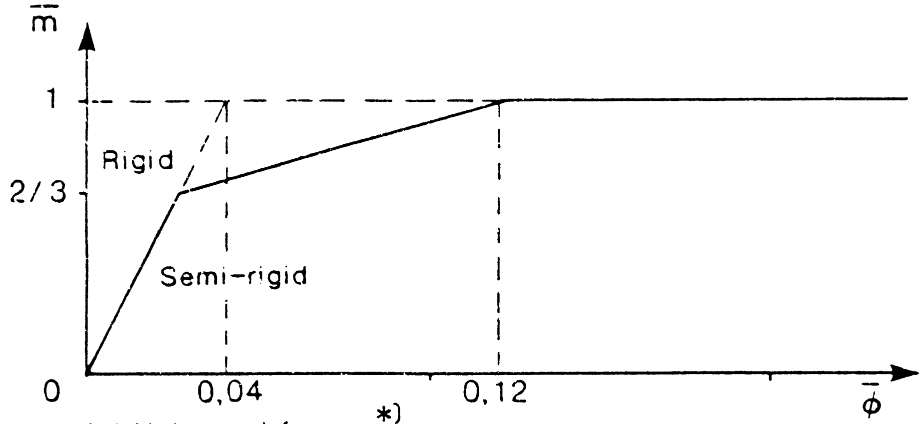
- nominally pinned, see **6.4.3.1**.
- full-strength, see **6.4.3.2**.
- partial-strength, see **6.4.3.3**.

(2) *A beam-to-column connection may be classified as nominally pinned if its design moment resistance* M_{Rd} *is not greater than 0,25 times the design plastic moment resistance of the connected beam* $M_{p\ell.Rd}$*, provided that it also has sufficient rotation capacity.*

(3) *A beam-to-column connection may be classified as full-strength if its design moment resistance* M_{Rd} *is at least equal to the design plastic moment resistance of the connected beam* $M_{p\ell.Rd}$*, provided that it also has sufficient rotation capacity.*

(4) *If the design moment resistance* M_{Rd} *of a beam-to-column connection is at least 1,2 times the design plastic moment resistance of the member* $M_{p\ell.Rd}$*, the rotation capacity of the connection need not be checked.*

(5) *A beam-to-column connection should be classified as partial-strength if its design moment resistance* M_{Rd} *is less than* $M_{p\ell.Rd}$*.*

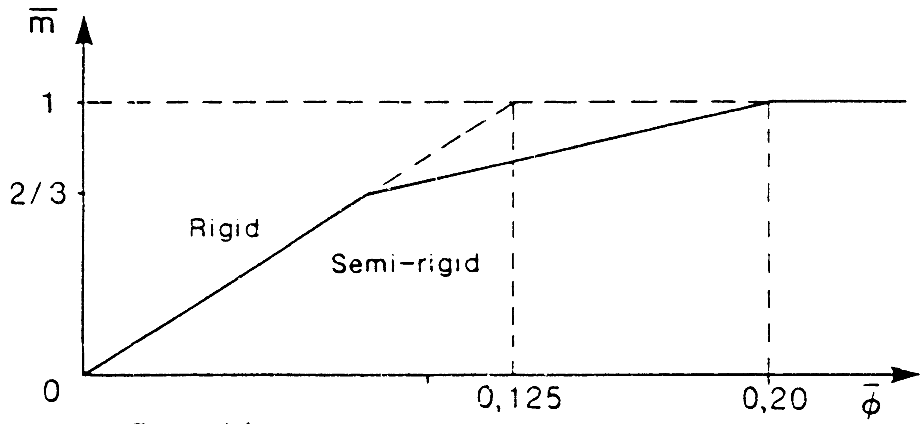


(a) Unbraced frames *)

when $\bar{m} \leq 2/3$: $\bar{m} = 25\,\bar{\phi}$

when $2/3 < \bar{m} \leq 1{,}0$: $\bar{m} = (25\,\bar{\phi} + 4)/7$

*) but see also 6.9.6.2(5)



(b) Braced frames

when $\bar{m} \leq 2/3$: $\bar{m} = 8\,\bar{\phi}$

when $2/3 < \bar{m} \leq 1{,}0$: $\bar{m} = (20\,\bar{\phi} + 3)/7$

$$\bar{m} = \frac{M}{M_{p\ell.Rd}} \quad : \quad \bar{\phi} = \frac{E I_b \phi}{L_b M_{p\ell Rd}}$$

Figure 6.9.8 — Recommended classification boundaries for rigid beam-to-column connections

6.9.6.4 *Classification of moment-rotation characteristics*

(1) *The classification of typical moment-rotation characteristics for beam-to-column connections with respect to both rotational stiffness and moment resistance, is illustrated in Figure 6.9.9.*

(2) *The moment-rotation characteristics indicated in Figure 6.9.9 are shown as non-linear for clarity. The figure is equally valid for hi-linear or tri-linear characteristics.*

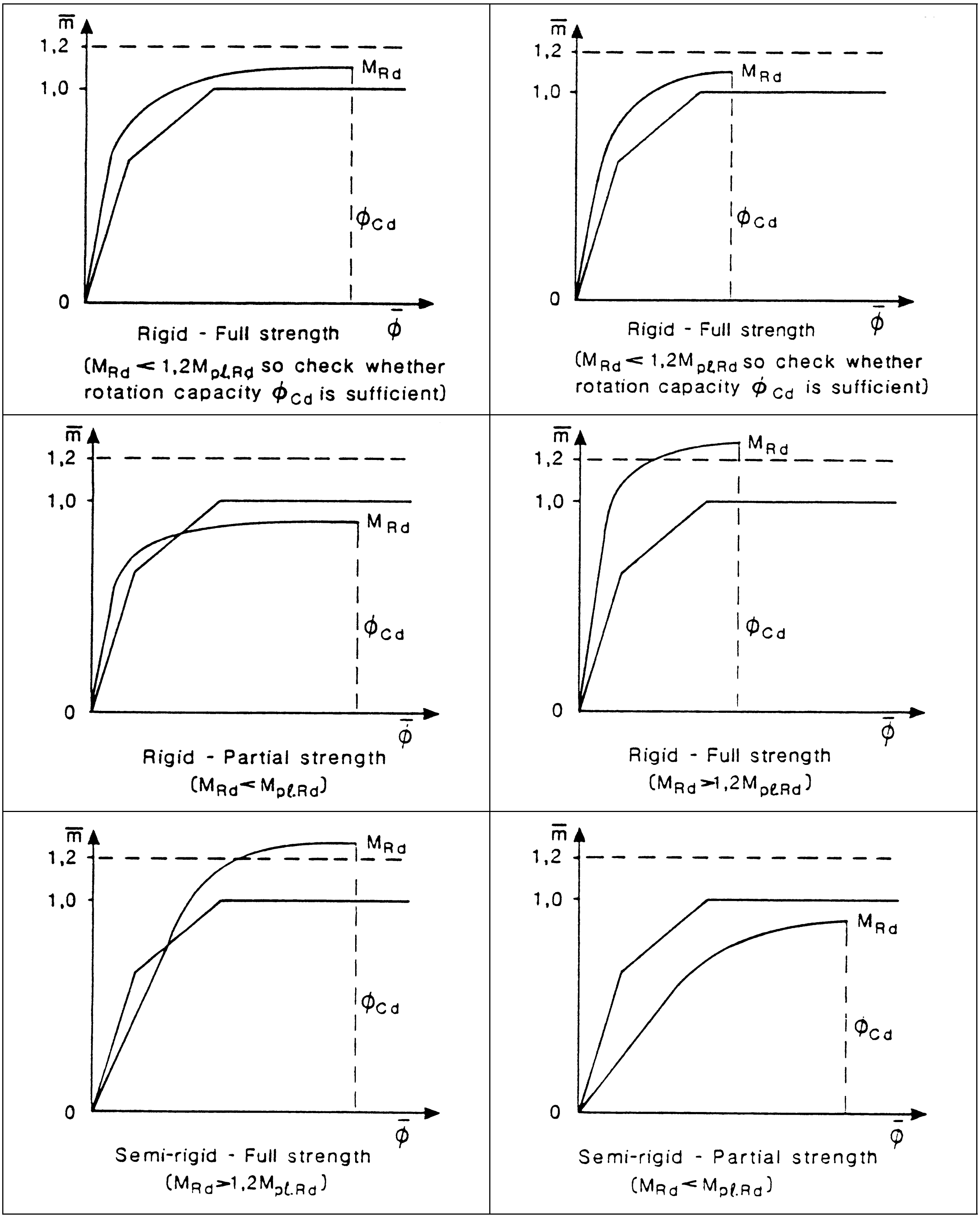


Figure 6.9.9 — Examples of classification of moment-rotation characteristics for beam-to-column connections

6.9.7 Calculated properties

6.9.7.1 *Moment resistance*

(1) The moment resistance of a beam-to-column connection depends on the resistance of the three critical zones indicated in Figure 6.9.10, as follows:

- Tension zone
- Compression zone
- Shear zone.

(2) The design moment resistance shall be determined taking account of the following criteria:

a) Tension zone:
  - Yielding of the column web.
  - Yielding of the beam web.
  - Yielding of the column flange.
  - Yielding of the connection material (e.g. end plate).
  - Weld failure.
  - Bolt failure.

b) Compression zone:
  - Crushing of the column web.
  - Buckling of the column web.

c) Shear zone:
  - Shear failure of the column web panel.

(3) The design resistance of the compression zone may be influenced by local second order effects caused by normal stresses in the column due to the frame behaviour.

(4) Except as indicated in (3), the design resistances of the critical zones of the connection may be assumed to be unaffected by stresses due to the frame behaviour.

(5) The design moment resistance of a beam-to-column connection shall be taken as the smaller of the resistances of the tension zone and the compression zone, (reduced if necessary so that the design shear resistance of the column web panel is not exceeded), multiplied by the distance between their centres of resistance.

(6) *Where the design resistance of the shear zone is greater than or equal to the smaller of the design resistances of the tension zone and the compression zone, no further check on the shear resistance of the column web panel is required.*

6.9.7.2 *Rotational stiffness*

(1) The calculated rotational stiffness of a beam-to-column connection shall be based on the flexibilities of the components in the critical zones.

6.9.7.3 *Rotation capacity*

(1) The validity of calculation procedures used to determine rotation capacity shall be verified from test evidence.

(2) The calculated rotation capacity of a beam-to-column connection shall be determined from the plastic deformation capacity of the same critical zone which governs in the calculation of the design moment resistance of the connection.

6.9.8 Application rules

(1) *The principles for the design of beam-to-column connections given in section **6.9** can be satisfied by following the detailed application rules given in normative Annex J.*

(2) *The design of other types of connection not covered in normative Annex J should be based on similar application rules conforming to the principles given in section **6.9**.*

(3) *Alternative application rules can also be used provided that:*

- *they accord with the same principles, and*
- *it can be demonstrated that they lead to at least the same safety level*

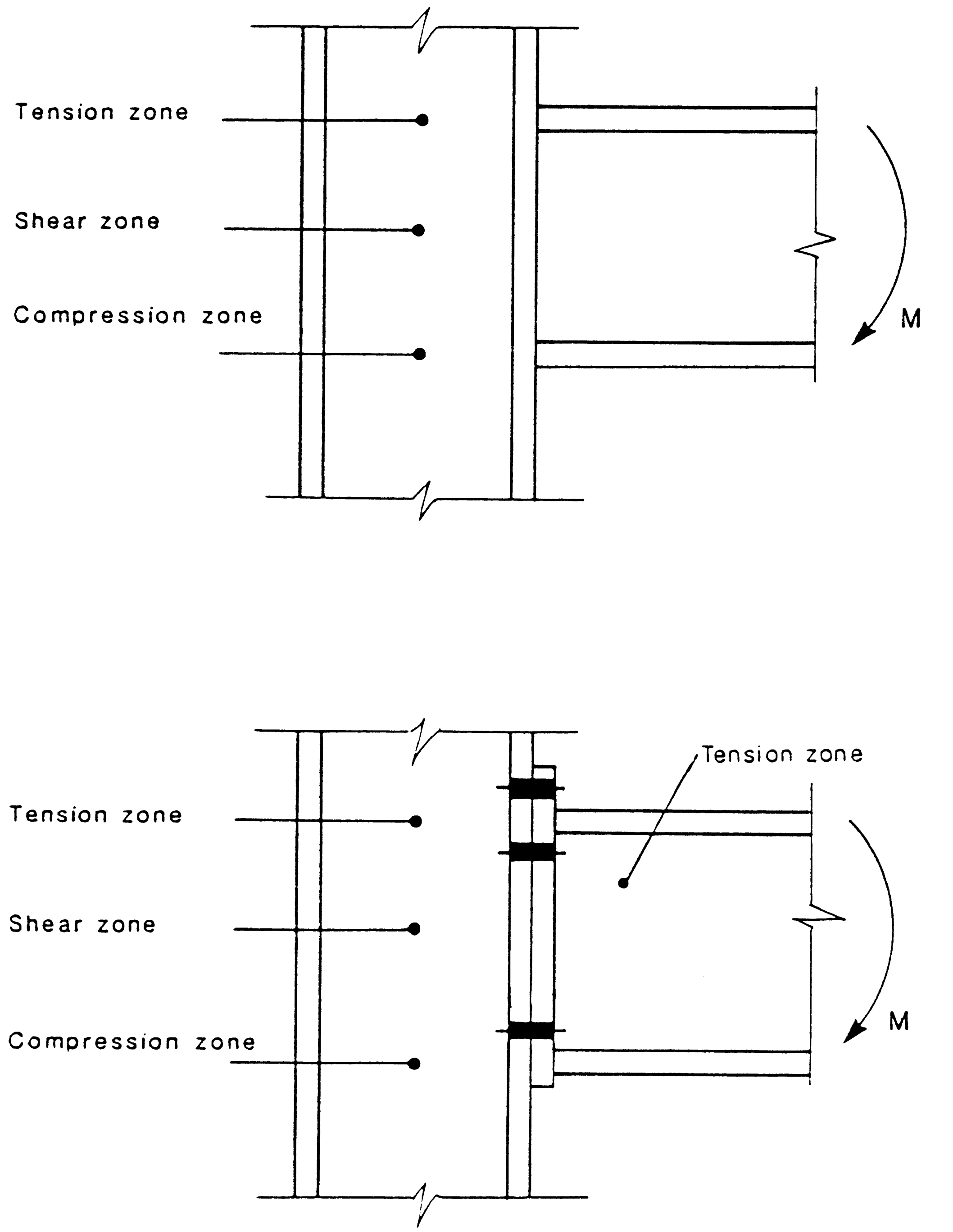


Figure 6.9.10 — Critical zones in beam-to-column connections

6.10 Hollow section lattice girder joints

6.10.1 Design resistance

(1) The design resistances of joints between hollow sections shall be based on the following criteria as applicable:

a) chord face failure.

b) chord web (or wall) failure by yielding or instability.

c) chord shear failure.

d) chord punching shear failure.

e) brace failure with reduced effective width.

f) local buckling failure.

(2) The welds shall be designed to have sufficient resistance and ductility to allow redistribution of non-uniform stress distributions and to allow redistribution of secondary bending moments.

6.10.2 Application rules

(1) *The principles for the design of hollow section lattice girder joints given in section **6.10** can be satisfied by following the detailed application rules given in normative Annex K.*

(2) *Alternative application rules can also be used provided that:*

- *they accord with the same principles, and*
- *it can be demonstrated that they lead to at least the same safety level.*

6.11 Column bases

6.11.1 Base plates

(1) Columns shall be provided with base plates capable of distributing the compressive forces in the compressed parts of the column over a bearing area, such that the bearing pressure on the foundation does not exceed the design strength of the joint.

(2) The design strength of the joint between the base plate and the foundation shall be determined taking account of the material properties and dimensions of both the grout and the concrete foundation.

6.11.2 Holding down bolts

(1) Holding down bolts shall be provided if necessary to resist the effects of the design loads. They shall be designed to resist tension due to uplift forces and tension due to bending moments as appropriate.

(2) When calculating the tension forces due to bending moments, the lever arm shall not be taken as more than the distance between the centroid of the bearing area on the compression side and the centroid of the bolt group on the tension side, taking the tolerances on the positions of the holding down bolts into account.

(3) Holding down bolts shall either be anchored into the foundation by a hook or by a washer plate or by some other appropriate load distributing member embedded in the concrete.

(4) If no special elements for resisting the shear force are provided, such as block or bar shear connectors, it shall be demonstrated that sufficient resistance to transfer the shear force between the column and the foundation is provided by one of the following:

- the frictional resistance of the joint between the base plate and the foundation
- the shear resistance of the holding down bolts
- the shear resistance of the surrounding part of the foundation

6.11.3 Application rules

(1) *The principles for the design of column bases given in section **6.11** can be satisfied by following the detailed application rules given in normative Annex L.*

(2) *Alternative application rules can also be used provided that:*

- *they accord with the same principles, and*
- *it can be demonstrated that they lead to at least the same safety level.*

7 Fabrication and erection

7.1 General

7.1.1 Scope

(1) This chapter specifies the minimum standards of workmanship required for fabrication and erection to ensure that the design assumptions of this Eurocode are satisfied and hence that the intended level of structural safety can be attained.

(2) The minimum requirements apply to structures which are predominantly statically loaded. Higher standards of workmanship and more rigorous levels of inspection and testing may be necessary for structures in which fatigue predominates, depending on the design details and the required fatigue strength (see Chapter 9) or for other reasons.

(3) Any supplementary requirements specific to particular structures shall be stated in the Project Specification.

7.1.2 Requirements

(1) Provided that all structural steel materials, fasteners and welding consumables conform with the requirements specified in Chapter 3, the workmanship shall be in conformity with the following Reference Standards:

- Reference Standard No. 6: Fabrication of structural steelwork
- Reference Standard No. 7: Erection of structural steelwork
- Reference Standard No. 8: Installation of preloaded bolts
- Reference Standard No. 9: Welding of structural steelwork

NOTE For details of Reference Standards 6 to 9 see normative Annex B.

(2) If any alternative or additional materials are used, the requirements specified in (1) shall be supplemented as necessary to ensure a similar level of safety.

7.2 Project specification

(1) The designer shall provide, or adopt, a Project Specification containing details of all the requirements for materials, fabrication and erection necessary to ensure compliance with the design assumptions relevant to the particular structure.

(2) The Project Specification shall contain adequate details of any special requirements for:

- fabrication
- erection
- inspection
- acceptance.

(3) The Project Specification shall cover all relevant requirements arising from the requirements of sections **7.3** to **7.7** of this Chapter.

(4) The Project Specification may include drawings in addition to text.

(5) The Project Specification may supplement the requirements of the Reference Standards but it shall not relax their technological requirements and it shall not supersede the minimum requirements specified in this Chapter.

(6) Once approved the Project Specification shall not be altered without the agreement of the designer and of the authority responsible for inspection.

(7) *As far as possible the requirements in the Project Specification should be specified using the Reference Standards.*

7.3 Fabrication restrictions

(1) It is necessary to avoid or eliminate hardened material in the following situations:

- when design is based on plastic analysis, within a distance along the member equal to the depth of the member, either side of each plastic hinge location.
- when fatigue predominates, where detail categories 140 or 160 (see Chapter 9) are used in design.
- where design for seismic actions or accidental actions relies on plastic deformation.

(2) Where any of the situations listed in (1) occurs, the locations required to be free from hardened material shall be identified in the Project Specification.

(3) At locations required to be free of hardened material, the restrictions specified in Reference Standard 6 shall be applied to the following:

a) flame cut or sheared edges

b) punched holes

c) hard marking

d) temporary welded attachments

e) surface repair by welding

NOTE Condition e) affects the supply conditions for the material, see Reference Standard 1.

(4) *All locations where restrictions on hardening are required should be clearly indicated on the drawings.*

7.4 Preparation of material

(1) Any necessary straightening or shaping shall be done using methods that do not reduce the properties of the material below those specified.

(2) Steelwork that has been galvanised shall be re-straightened or re-shaped if necessary to satisfy the specified tolerance limits.

(3) All surfaces and edges shall be free from defects likely to impair the effectiveness of the surface protection system specified in the Project Specification.

(4) The standards of flatness necessary at contact bearing surfaces to transmit the design forces shall be specified.

(5) Any special treatment required at cut-outs shall be specified in the Project Specification.

7.5 Bolted connections

7.5.1 Holes

(1) Holes for bolts may be drilled or punched unless specified otherwise.

(2) Where drilled holes are required they may be sub-punched and reamed.

(3) Where countersunk holes are required, the angle of countersinking shall correspond to that of standard countersunk bolts as specified in Reference Standard 3, unless special non-standard countersunk bolts are specified.

(4) *Care should be taken that the depth of countersinking is sufficient to accommodate the head of the bolt. Where this would involve countersinking into more than one ply the action to be taken should be stated in the Project Specification.*

(5) Slotted holes shall either be punched in one operation or else formed by punching or drilling two round holes and completed by high quality flame cutting and dressing to ensure that the bolt can freely travel the full length of the slot.

7.5.2 Clearances in holes for fasteners

(1) Except for fitted bolts or where low-clearance or oversize holes are specified, the nominal clearance in standard holes shall be:

- 1 mm for M12 and M14 bolts
- 2 mm for M16 to M24 bolts
- 3 mm for M27 and larger bolts.

(2) Holes with smaller clearances than standard holes may be specified.

(3) Holes with 2 mm nominal clearance may also be specified for M12 and M14 bolts, provided that the design meets the requirements specified in **6.5.5**(8).

(4) Unless special clearances are specified, the clearance for fitted bolts shall be as specified in Reference Standard 6.

(5) Oversize and slotted holes may be used for slip-resistant connections only where specified.

(6) The nominal clearance in oversize holes for slip-resistant connections shall be:

- 3 mm for M12 bolts
- 4 mm for M14 to M22 bolts
- 6 mm for M24 bolts
- 8 mm for M27 and larger bolts.

(7) Oversize holes in the outer ply of a slip-resistant connection shall be covered by hardened washers.

(8) Holes for holding down bolts may be oversize holes with clearances as specified in the Project Specification, provided that these holes are covered by cover plates of appropriate dimensions and thickness. The holes in the cover plates shall not be larger than standard holes.

(9) The nominal sizes of short slotted holes for slip resistant connections shall be not greater than:

- (d + 1) mm by (d + 4) mm for M12 and M14 bolts
- (d + 2) mm by (d + 6) mm for M16 to M22 bolts
- (d + 2) mm by (d + 8) mm for M24 bolts
- (d + 3) mm by (d + 10) mm for M27 and larger bolts

where d is the nominal bolt diameter in mm.

(10) The nominal sizes of long slotted holes for slip resistant connections shall be not greater than:

- (d + 1) mm by 2,5d for M12 and M14 bolts
- (d + 2) mm by 2,5d for M16 to M24 bolts
- (d + 3) mm by 2,5d for M27 and larger bolts

(11) Long slots in an outer ply shall be covered by cover plates of appropriate dimensions and thickness. The holes in the cover plates shall not be larger than standard holes.

(12) The sizes required for long slotted holes for movement joints shall be specified. Slots in an outer ply shall be covered by cover plates of appropriate dimensions and thickness.

7.5.3 Bolts

(1) Where design is based on bolts with unthreaded shanks in the shear plane, appropriate measures shall be specified to ensure that, after allowing for tolerances, neither the threads nor the thread run-out will be in the shear plane.

(2) Bolts with threads up to the head may be used except where prohibited by the Project Specification.

(3) The length of a non-preloaded bolt shall be such that, after allowing for tolerances:

- the threaded shank will protrude beyond the nut after tightening, and
- at least one full thread (in addition to the thread run-out) will remain clear between the nut and the unthreaded part of the shank.

(4) The length of a preloaded bolt shall be such that, after allowing for tolerances:

- the threaded shank will protrude beyond the nut after tightening, and
- at least four full threads (in addition to the thread run-out) will remain clear between the nut and the unthreaded part of the shank.

7.5.4 Nuts

(1) For structures subject to vibration, measures shall be taken to avoid any loosening of the nuts.

(2) *If non-preloaded bolts are used in structures subject to vibrations, the nuts should be secured by locking devices or other mechanical means.*

(3) *The nuts of preloaded bolts may be assumed to be sufficiently secured by the normal tightening procedure.*

7.5.5 Washers

(1) Washers are not required for non-preloaded bolts except as follows:

- A taper washer shall be used where the surface is inclined at more than 3° to a plane perpendicular to the bolt axis.
- Washers shall be used where this is necessary due to a requirement, specified in the Project Specification, to use a longer bolt in order to keep the bolt threads out of a shear plane or out of a fitted hole.

(2) Hardened washers shall be used for preloaded bolts as follows:

- A hardened washer shall be used under the bolt head or the nut, whichever is to be rotated.
- A hardened washer shall also be used under the non-rotated component (bolt head or nut) where specified in the Project Specification.
- A hardened taper washer shall be used if necessary to ensure that the rotated component bears on a surface perpendicular to the bolt axis.

- A hardened taper washer shall be used under the non-rotated component where the surface is inclined at more than 3° to a plane perpendicular to the bolt axis.

7.5.6 Tightening of bolts

(1) Non-preloaded bolts shall be tightened sufficiently to ensure that sufficient contact is achieved between the parts assembled.

(2) *It is not necessary to tighten non-preloaded bolts to a predetermined value. However as an indication, the tightening required should be:*

- *that which can be achieved by one man using a normal podger spanner, or*
- *up to the point where an impact wrench first starts to impact.*

(3) Preloaded bolts shall be tightened in conformity with Reference Standard 8. The Project Specification shall specify which of the methods given in the Reference Standard may be used.

7.5.7 Slip resistant contact surfaces

(1) Where a particular surface condition is required at friction interfaces in bolted joints, the surface condition required shall be specified in the Project Specification, see **6.5.8.3**.

(2) If steel packing plates are used in a slip-resistant joint, it shall be ensured that their contact surfaces are also prepared to the specified condition.

7.5.8 Fit of contact surfaces

(1) Unless smaller values are specified in the Project Specification, the maximum step between adjacent surfaces in a joint (see Figure 7.1) shall not exceed:

- 2 mm when using non-preloaded bolts
- 1 mm when using preloaded bolts.

(2) *When using preloaded bolts, the designer should consider allowing for the possible effects of lack of fit as an alternative to imposing smaller tolerances.*

(3) Steel packing plates shall be provided where necessary to ensure that the remaining step does not exceed the specified limit.

(4) *Unless a greater value is specified the minimum thickness of a steel packing plate should be:*

- *2 mm in indoor conditions, if not exposed to corrosive influences*
- *4 mm in outdoor conditions or if exposed to corrosive influences*

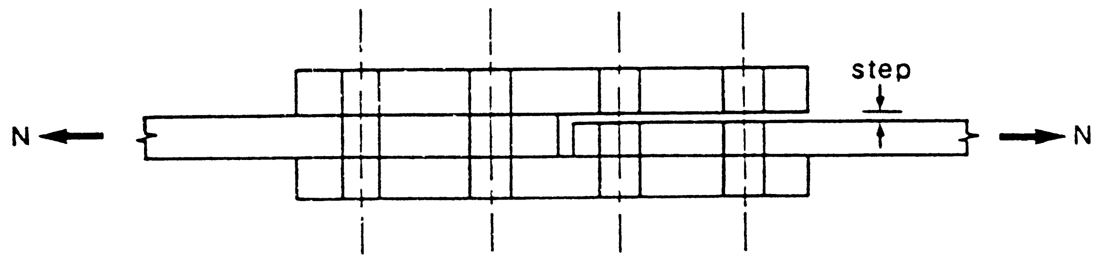


Figure 7.1 — Maximum step between adjacent surfaces

7.6 Welded connections

(1) Assembly and welding shall be carried out in such a way that the final dimensions are within the appropriate tolerances.

(2) The Project Specification shall include appropriate details of any welded connections which require:

- special welding procedures
- special levels of quality control
- special inspection procedures
- special test procedures.

(3) Welding may be carried out on site unless prohibited by the Project Specification.

(4) *The drawings should indicate clearly whether butt welds are intended to be full penetration butt welds or partial penetration butt welds. In the case of partial penetration butt welds, the required throat size should be specified.*

7.7 Tolerances

7.7.1 Types of tolerances

(1) "Normal" tolerances are the basic limits for dimensional deviations necessary:

- to satisfy the design assumptions for statically loaded structures.
- to define acceptable tolerances for building structures in the absence of any other requirements.

(2) "Special" tolerances are more stringent tolerances necessary to satisfy the design assumptions:

- for structures other than normal building structures.
- for structures in which fatigue predominates.

(3) "Particular" tolerances are more stringent tolerances necessary to satisfy functional requirements of particular structures or structural components, related to:

- attachment of other structural or non-structural components
- shafts for lifts (elevators)
- tracks for overhead cranes
- other criteria such as clearances
- alignment of external face of a building

7.7.2 Application of tolerances

(1) All tolerance values specified in section **7.7** shall be treated as "normal" tolerances.

(2) "Normal" tolerances apply to conventional single-storey and multi-storey steel framed structures of residential, administrative, commercial and industrial buildings except where "special" or "particular" tolerances are specified.

(3) Any special or particular tolerances required shall be detailed in the Project Specification.

(4) *Any special or particular tolerances required should also be indicated on the relevant drawings.*

7.7.3 Normal erection tolerances

(1) The unloaded steel structure, as erected, shall satisfy the criteria specified in Table 7.1 within the specified tolerance limits, see Figure 7.2.1 and Figure 7.2.2.

(2) Each criterion given in the tables shall be considered as a separate requirement, to be satisfied independently of any other tolerance criteria.

(3) The erection tolerances specified in Table 7.1 apply to the following reference points:

- For a column, the actual centre point of the column at each floor level and at the base, excluding any base- plate or cap-plate.
- For a beam, the actual centre point of the top surface at each end of the beam, excluding any endplate.

Table 7.1 — Normal tolerances after erection

Criterion	Permitted deviation
Deviation of distance between adjacent columns	± 5 mm
Inclination of a column in a multi-storey building between adjacent floor levels	0,002h where h is the storey height
Deviation of location of a column in a multi-storey building at any floor level, from a vertical line through the intended location of the column base	$0{,}0035\Sigma h/n^{0{,}5}$ where Σh is the total height from the base to the floor level concerned and n is the number of storeys from the base to the floor level concerned
Inclination of a column in a single storey building, (not supporting a crane gantry) other than a portal frame	0,0035h where h is the height of the column
Inclination of the columns of a portal frame (not supporting a crane gantry)	Mean: 0,002h Individual: 0,010h

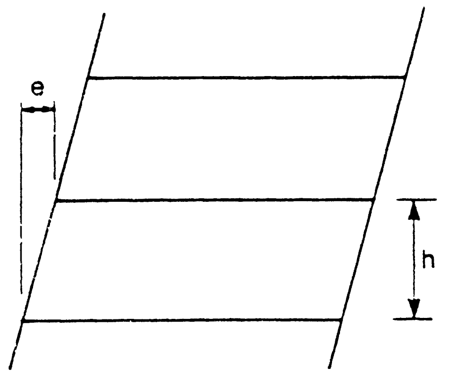
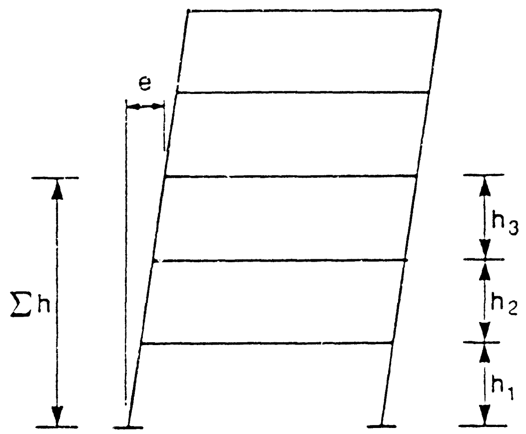
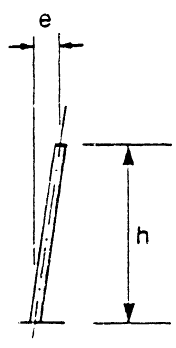
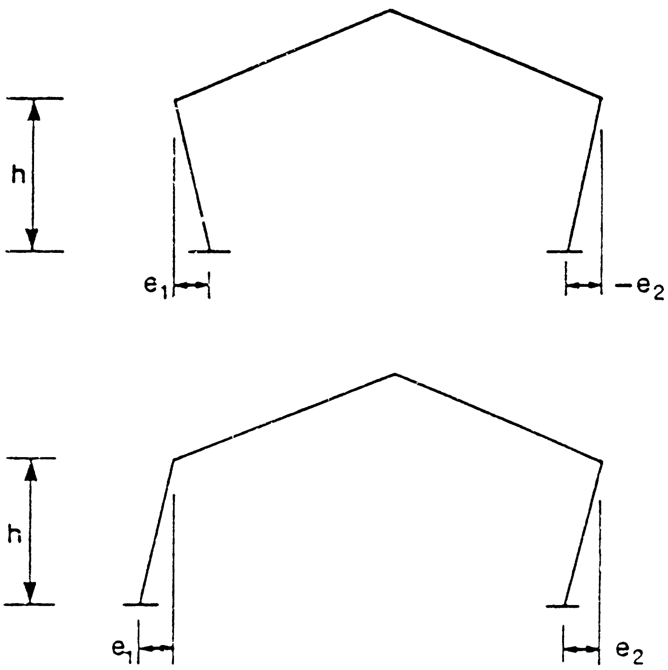
Criterion	Description	Permitted deviation
Inclination of a column between adjacent floor levels		$e \leq 0{,}002h$
Location of a column at any floor level, from a vertical line through the intended location of the column base		$e \leq \dfrac{0{,}0035\Sigma h}{n^{1/2}}$
Inclination of a column in a single-storey building, without a crane gantry, other than in a portal frame		$e \leq 0{,}0035h$

Figure 7.2.1 — Normal tolerances after erection — Part 1

Criterion: Inclination of columns of a portal frame not supporting a crane gantry.

Description:



Permitted deviation:

Inclination of individual columns:

$e_1 \leq 0{,}010h : e_2 \leq 0{,}010h$

Mean inclination of a frame:

$$\frac{e_1 + e_2}{2} \leq 0{,}002h \text{ where } e_1 \geq e_2$$

Figure 7.2.2 — Normal tolerances after erection — Part 2

7.7.4 Fabrication tolerances

(1) The normal fabrication tolerances shall be the normal fabrication tolerances for building structures specified in Reference Standard 6.

(2) The straightness tolerances specified in Table 7.2 have been assumed in the derivation of the design rules for the relevant type of member. Where the curvature exceeds these values, the additional curvature shall be allowed for in the design calculations.

Table 7.2 — Straightness tolerances incorporated in design rules

Criterion	Permitted deviation
Straightness of a column (or other compression member) between points which will be laterally restrained on completion of erection	± 0,001L generally ± 0,002L for members with hollow cross-sections where L is the length between points which will be laterally restrained
Straightness of the compression flange of a beam, relative to the weak axis, between points which will be laterally restrained on completion of erection	± 0,001L generally ± 0,002L for members with hollow cross-sections where L is the length between points which will be laterally restrained

7.7.5 Position of holding down bolts

(T olerances shall be specified for the positional deviations of the holding down bolts which will enable the tolerance limits for erection of steelwork to be satisfied.

(2) Tolerances shall be specified for the levels of the holding down bolts which enable the specified tolerances to be satisfied for the following criteria:

- the level of the baseplate
- the thickness of the bedding material under the baseplate
- the protrusion of the bolt through the nut
- the number of threads clear below the nut.

(3) The deviations of the spacings between individual bolts within the group of holding down bolts for each member shall not exceed the following:

- for bolts rigidly cast in, between centres of bolts: ± 5 mm
- for bolts set in sleeves, between centres of sleeves: ± 10 mm.

7.8 Inspection and testing

(1) The requirements for inspection and testing shall be those for the normal level of inspection and testing specified in the relevant Reference Standards, unless special inspection requirements are specified.

(2) The criteria for acceptance shall be the normal criteria for acceptance specified in the relevant Reference Standards, unless special acceptance criteria are specified.

8 Design assisted by testing

8.1 Basis

(1) The provisions in this Chapter give guidance to designers who may become involved with experimental assessments.

(2) When the calculation models available are not sufficient for a particular structure or structural component, experimental assessment shall be undertaken in place of design by calculation or to supplement design by calculation.

(3) Experimental verification may also be undertaken where the rules for design by calculation given in this Eurocode would lead to uneconomic results. However, the conservative assumptions in the specified calculation models (which are intended to account for unfavourable calculation influences not explicitly considered in the specified calculation models) shall not be by-passed.

(4) The planning, execution, evaluation and documentation of tests shall be in accordance with the minimum requirements stated in this Chapter.

(5) *Because circumstances and test facilities vary greatly, the test procedures should be agreed in advance by all concerned.*

8.2 Planning of tests

(1) The experimental assessment shall be based on tentative calculation models, which may be incomplete, but which relate one or several relevant variables to the structural behaviour under consideration, such that basic tendencies are adequately predicted. The experimental assessment shall then be confined to the evaluation of correction terms in the tentative calculation model.

(2) If the prediction of the relevant calculation models or of the failure mode to be expected in the tests is extremely doubtful, the test plan shall be developed on the basis of accompanying pilot tests.

(3) Prior to the execution of tests, a test plan shall be drawn up by the designer and the testing organisation. This shall contain the objective of the tests and all the instructions and other specifications necessary for the selection or production of the test specimens, the execution of the tests and the test evaluation.

(4) *Reference should be made to informative Annex Y for guidance in preparing the test plan.*

(5) The test plan shall deal with the following items:

a) Scope of information required from the tests (e.g. required parameters and range of validity).

b) Description of all properties of the members considered which may influence the behaviour at a limit state, (e.g. form of the member, stiffness, steel grade and quality and relevant material properties, geometrical and structural parameters and their tolerances, parameters influenced by fabrication and erection procedures).

c) Specifications on the properties of the test specimen (e.g. sampling procedures, specification for dimensions, material and fabrication of prototypes, number of specimens, number of subsets, restraints).

d) Description of the actions to which the members are required to react and demonstrate the properties referred to in b), (e.g. load arrangements, load cases, load combinations).

e) Specifications on the loading and environmental conditions in the test (e.g. loading points, loading methods, loading path in time and space, temperatures).

f) Modes of failure and tentative calculation models with the corresponding relevant variables, see **8.2**(1).

g) Testing arrangements (including measures to ensure sufficient strength and stiffness of the loading and supporting rigs and clearance for deflections etc).

h) Determination of the monitoring points and methods for observation and recording (e.g. time histories of strains, forces, deflections).

i) Determination of the type and control of load application (stress-controlled, strain-controlled etc).

k) Required accuracy of measurements and measuring devices.

(6) All details on the sampling or manufacturing of the test specimens shall be reported and measurements shall be carried out on these test specimen before the execution of tests starts, in order to demonstrate that the test plan has been fulfilled, otherwise it shall be revised.

8.3 Execution of tests

(1) The performance of experimental assessments shall be entrusted only to organisations where the staff is sufficiently knowledgeable and experienced in the planning, execution and evaluation of tests.

(2) The testing laboratory shall be adequately equipped and the testing organisation shall ensure careful management and documentation of all tests.

8.4 Test evaluation

(1) The test evaluation shall take account of the random character of all data.

(2) *This test evaluation should be carried out using the method given in Annex Z*[21]*).*

[21]) To be prepared at a later stage

8.5 Documentation

(1) The following documentation shall be provided in the test report:

- the test plan (including any revisions),
- descriptions and specifications for all test specimens,
- details of the testing arrangements,
- details of the execution of the tests, and
- the test results which are necessary for the test evaluation.

9 Fatigue

9.1 General

9.1.1 Basis

(1) The aim of designing a structure against the limit state of fatigue is to ensure, with an acceptable level of probability, that its performance is satisfactory during its entire design life, such that the structure is unlikely to fail by fatigue or to require repair of damage caused by fatigue.

(2) The required safety level shall be obtained by applying the appropriate partial safety factors (see **9.3**).

9.1.2 Scope

(1) This Chapter presents a general method for the fatigue assessment of structures and structural elements which are subjected to repeated fluctuations of stresses.

(2) The fatigue assessment procedures assume that the structure also conforms with the other limit state requirements of this Eurocode.

(3) The fatigue assessment procedures given in this Chapter are applicable when all structural steel materials, fasteners and welding consumables conform with the requirements specified in Chapter 3.

9.1.3 Limitations

(1) For fatigue assessment, all nominal stresses [see **9.1.5**(7)] shall be within the elastic limits of the material. The range of the design values of such stresses shall not exceed 1,5 f_y for normal stresses or 1,5 $f_y/\sqrt{3}$ for shear stresses.

(2) The fatigue strengths specified in this Chapter are applicable to structures with suitable corrosion protection, subjected only to mildly corrosive environments, such as normal atmospheric conditions (pit depth $\leq$ 1 mm).

(3) The fatigue assessment procedures given in this Chapter are applicable only to structures subjected to temperatures not exceeding 150 °C.

9.1.4 Necessity for fatigue assessment

(1) No fatigue assessment is normally required for building structures except as follows:

a) Members supporting lifting appliances or rolling loads.

b) Members subject to repeated stress cycles from vibrating machinery.

c) Members subject to wind-induced oscillations.

d) Members subject to crowd-induced oscillations.

(2) No fatigue assessment is required when any of the following conditions is satisfied:

a) The largest nominal stress range $\Delta\sigma$ satisfies:

$$\gamma_{Ff}\,\Delta\sigma \leq 26/\gamma_{Mf}\ \text{N/mm}^2. \qquad (9.1)$$

b) The total number of stress cycles N satisfies:

$$N \leq 2 \times 10^6 \left[\frac{36/\gamma_{Mf}}{\gamma_{Ff}\ \Delta\sigma_{E.2}}\right]^3 \qquad (9.2)$$

where $\Delta\sigma_{E.2}$ is the equivalent constant amplitude stress range in N/mm².

c) For a detail for which a constant amplitude fatigue limit $\Delta\sigma_D$ is specified, the largest stress range (nominal or geometric as appropriate) $\Delta\sigma$ satisfies:

$$\gamma_{Ff}\,\Delta\sigma \leq \Delta\sigma_D/\gamma_{Mf} \tag{9.3}$$

9.1.5 Definitions

(1) **Fatigue:** Damage in a structural part, through gradual crack propogation caused by repeated stress fluctuations.

(2) **Fatigue loading:** A set of typical load events described by the positions of loads, their intensities and their relative frequencies of occurrence.

(3) **Loading event:** A defined loading sequence applied to the structure and giving rise to a stress history.

(4) **Equivalent constant amplitude fatigue loading:** Simplified constant amplitude loading representing the fatigue effects of actual variable amplitude loading events.

(5) **Stress history:** A record, or a calculation, of the stress variation at a particular point in a structure during a load event.

(6) **Stress range:** The algebraic difference between the two extremes of a particular stress cycle forming part of a stress history. ($\Delta\sigma = \sigma_{max} - \sigma_{min}$ or $\Delta\tau = \tau_{max} - \tau_{min}$).

(7) **Nominal stress:** A stress in the parent material adjacent to a potential crack location, calculated in accordance with simple elastic strength of materials theory, excluding all stress concentration effects.

(8) **Modified nominal stress:** A nominal stress increased by an appropriate stress concentration factor, to allow for a geometric discontinuity which has not been taken into account in the classification of a particular constructional detail.

(9) **Geometric stress:** The maximum principal stress in the parent material, adjacent to the weld toe, taking into account stress concentration effects due to the overall geometry of a particular constructional detail, but excluding local stress concentration effects due to weld geometry and discontinuities in the weld and the adjacent parent metal.

NOTE The geometric stress is also known as the "hot spot stress".

(10) **"Rainflow" method and "reservoir" method:** Particular methods of producing a stress-range spectrum from a given stress history.

NOTE They are two versions of the same basic method.

(11) **Stress-range spectrum:** Histogram of the frequency of occurrence for all stress ranges of different magnitudes recorded or calculated for a particular loading event.

(12) **Design spectrum:** The total of all stress-range spectra relevant to the fatigue assessment, see Figure 9.1.1.

(13) **Equivalent constant amplitude stress range:** The constant-amplitude stress range that would result in the same fatigue life as for the spectrum of variable-amplitude stress ranges, when the comparison is based on a Miner's summation.

(14) *For convenience, the equivalent constant amplitude stress range may be related to a total number of 2 million variable amplitude stress range cycles.*

(15) **Fatigue life:** The total number of cycles of stress variation predicted to cause fatigue failure.

(16) **Miner's summation:** A linear cumulative damage calculation based on the Palmgren-Miner rule.

(17) **Constant amplitude fatigue limit:** The limiting stress range value above which a fatigue assessment is necessary.

(18) **Detail category:** The designation given to a particular welded or bolted detail, in order to indicate which fatigue strength curve is applicable for the fatigue assessment.

(19) **Fatigue strength curve:** The quantitative relationship relating fatigue failure to stress range and number of stress cycles, used for the fatigue assessment of a category of constructional detail, see Figure 9.1.2.

(20) **Design life:** The reference period of time for which a structure is required to perform safely with an acceptable probability that failure by fatigue cracking will not occur.

(21) **Cut-off limit:** Limit below which stress ranges of the design spectrum do not contribute to the calculated cumulative damage.

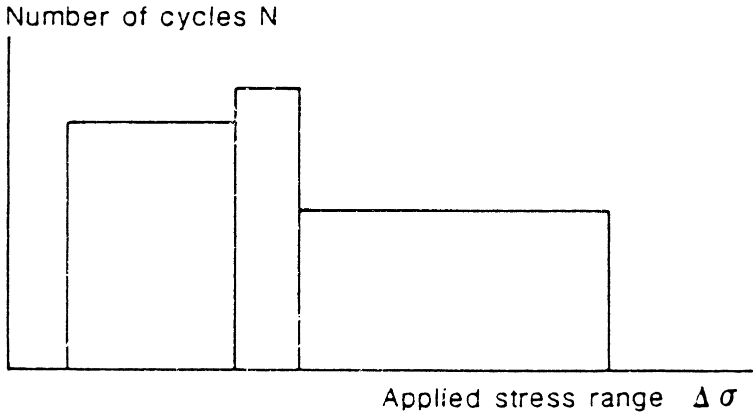


Figure 9.1.1 — Design spectrum

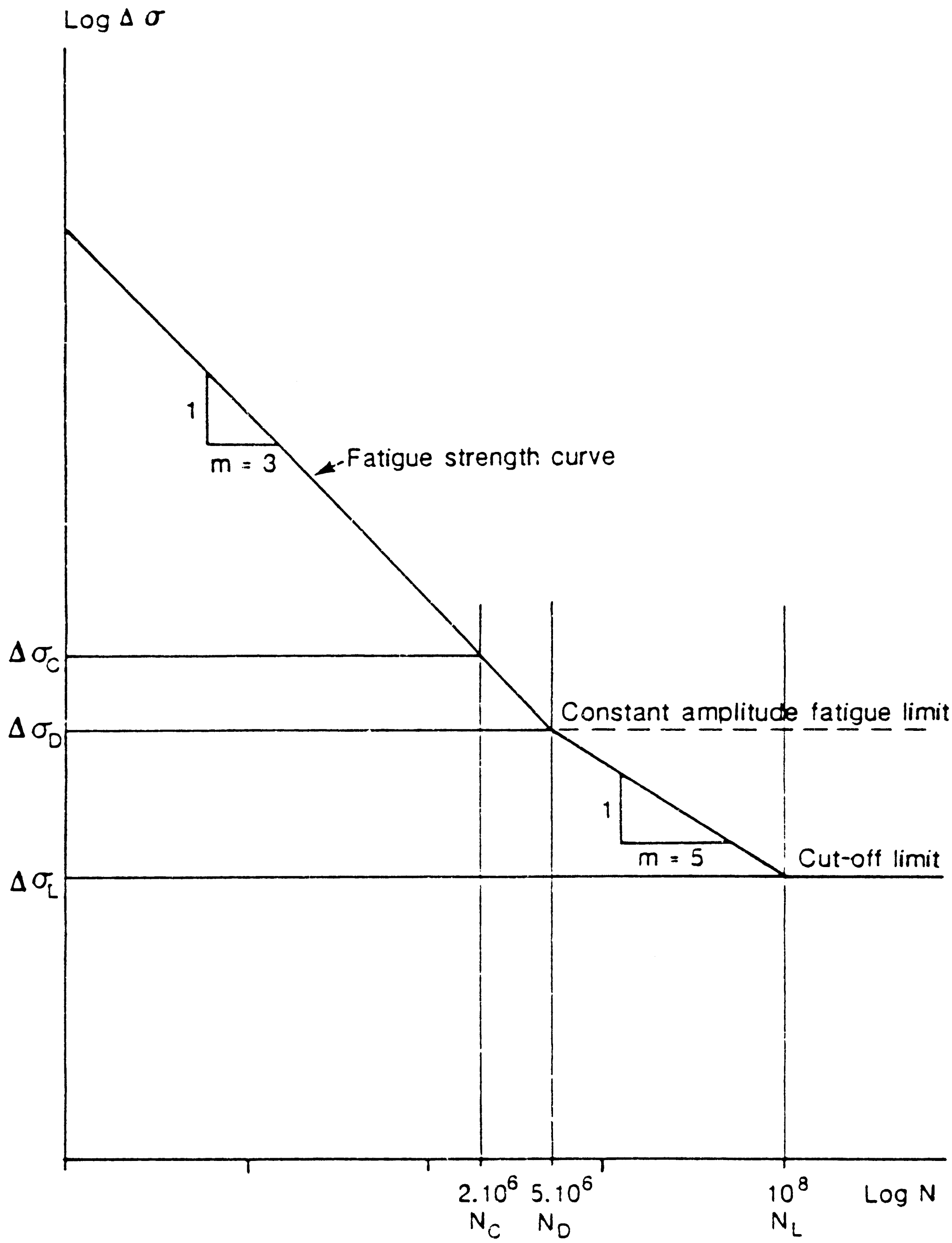


Figure 9.1.2 — Fatigue strength curve

9.1.6 Symbols

γ_{Ff}	Partial safety factor for fatigue loads.
γ_{Mf}	Partial safety factor for fatigue strength.
σ_{max}, σ_{min}	Maximum and minimum values of the fluctuating stresses in a stress cycle.
$\Delta\sigma$	Nominal stress range (normal stress).
$\Delta\sigma_D$	Constant amplitude fatigue limit.
$\Delta\sigma_R$	Fatigue strength (normal stress).
$\Delta\sigma_C$	Reference value of the fatigue strength at 2 million cycles (normal stress).
$\Delta\sigma_E$	Equivalent constant amplitude stress range (normal stress).
$\Delta\sigma_{E.2}$	Equivalent constant amplitude stress range (normal stress) for 2 million cycles.
$\Delta\sigma_L$	Cut-off limit.
$\Delta\tau$	Nominal stress range (shear stress).
$\Delta\tau_R$	Fatique strength (shear stress).
$\Delta\tau_E$	Equivalent constant amplitude stress range (shear stress).
$\Delta\tau_{E.2}$	Equivalent constant amplitude stress range (shear stress) for 2 million cycles.
$\Delta\tau_C$	Reference value of the fatigue strength at 2 million cycles (shear stress).
m	Slope constant of a fatigue strength curve, with values of 3 and/or 5.
n_i	Number of cycles of stress range $\Delta\sigma_i$.
N	Number (or total number) of stress range cycles.
N_i	Number of cycles of stress range $\gamma_{Ff}\gamma_{Mf}\Delta\sigma_i$ to cause failure.
N_C	Number of cycles (2 million) at which the reference value of the fatigue strength is defined.
N_D	Number of cycles (5 million) at which the constant amplitude fatigue limit is defined.
N_L	Number of cycles (100 million) at which the cut-off limit is defined.
log	Logarithm to base 10.

9.2 Fatigue loading

(1) The fatigue loading shall be obtained from ENV 1991 Eurocode 1[22] or other relevant loading standard.

(2) The loading used for the fatigue assessment shall be a characteristic value which represents the anticipated service loading throughout the required design life of the structure with a sufficient, defined, reliability.

(3) The fatigue loading may comprise different loading events which are defined by complete loading sequences of the structure, each characterised by their relative frequency of occurrence as well as their magnitude and geometrical position.

(4) Dynamic effects shall be considered when the response of the structure contributes to the modification of the design spectrum.

(5) In the absence of more accurate information, the dynamic amplification factors used for the static limit state may be employed.

(6) The effect of a loading event shall be represented by its stress history, see **9.1.5**(5).

(7) *The load models used for fatigue assessment of such structures as bridges and cranes should take into account the possible changes in use, such as growth of traffic or changes in the loading rate.*

(8) *Allowance should also be made for such future changes where it is necessary to base a fatigue assessment on a measured stress history.*

(9) *Simplified design calculations may be based on an equivalent fatigue loading, representing the fatigue effects of the full spectrum of loading events.*

(10) *The equivalent fatigue loading may vary with the dimensions and location of the structural element.*

[22] In preparation

9.3 Partial safety factors

9.3.1 General

(1) The values of the partial safety factors to be used shall be agreed between the client, the designer and competent public authority as being appropriate, considering:

- the ease of access for inspection or repair and likely frequency of inspection and maintenance,
- the consequences of failure.

(2) *Inspection may detect fatigue cracks before subsequent damage is caused. Such inspection is visual unless specified otherwise in the Project Specification.*

NOTE In-service inspection is not a requirement of Eurocode 3-1.1 and, if it is required, it should be subject to agreement.

(3) *In any circumstances, the possibility of general failure without any pre-warning conditions is not tolerable.*

(4) *Difficulties of access for inspection or repair may be such as to make the detection or the repair of cracks impractical. The client should be made aware of this so that measures to perform inspection may be taken.*

9.3.2 Partial safety factors for fatigue loading

(1) To take account of uncertainties in the fatigue response analysis, the design stress ranges for the fatigue assessment procedure shall incorporate a partial safety factor γ_{Ff}.

(2) The partial safety factor γ_{Ff} covers the uncertainties in estimating:

- the applied load levels,
- the conversion of these loads into stresses and stress ranges,
- the equivalent constant amplitude stress range from the design stress range spectrum,
- the design life of the structure, and the evolution of the fatigue loading within the required design life of the structure.

(3) *The fatigue loading given in ENV 1991 Eurocode 1*[23] *already incorporates an appropriate value of the partial safety factor* γ_{Ff}.

Unless otherwise stated in subsequent Parts of Eurocode 3, or in the relevant loading standard, a value of $\gamma_{Ff} = \boxed{1,0}$ *may be applied to the fatigue loading.*

9.3.3 Partial safety factors for fatigue strength

(1) In the fatigue assessment procedure, in order to take account of uncertainties in the fatigue resistance, the design value of the fatigue strength shall be obtained by dividing by a partial safety factor γ_{Mf}.

(2) The factor γ_{Mf} covers the uncertainties of the effects of:

- the size of the detail,
- the dimensions, shape and proximity of the discontinuities,
- local stress concentrations due to welding uncertainties.
- variable welding processes and metallurgical effects.

9.3.4 Recommended values of γ_{Mf}

(1) The recommended values given in this clause assume that Quality Assurance procedures are applied to ensure that the fabricated constructional details comply with the relevant quality requirements for structures subjected to fatigue as defined in Reference Standard 9, see normative Annex B.

(2) Concerning the consequences of failure, two possible situations may arise as follows:

- "fail-safe" structural components with reduced consequences of failure, such that the local failure of one component does not result in failure of the structure.
- non "fail-safe" structural components where local failure of one component leads rapidly to failure of the structure.

(3) *Recommended values of the partial safety factor* γ_{Mf} *are given in Table 9.3.1. These values should be applied to the fatigue strength.*

[23] *In preparation*

(4) *Where values of γ_{Ff} other than* $\boxed{1{,}0}$ *are applied to the fatigue loading, the γ_{Mf} values may need corresponding adjustment.*

Table 9.3.1 — Partial safety factor for fatigue strength γ_{Mf}

Inspection and access	"Fail-safe" components	Non "fail-safe" components
Periodic inspection and maintenance. Accessible joint detail.	$\boxed{1{,}00}$	$\boxed{1{,}25}$
Periodic inspection and maintenance. Poor accessibility.	$\boxed{1{,}15}$	$\boxed{1{,}35}$
See **9.3.1**(2) concerning inspection.		

9.4 Fatigue stress spectra

9.4.1 Calculation of stresses

(1) Stresses shall be determined by an elastic analysis of the structure under fatigue loading. Dynamic response of the structure or impact effect shall be considered when appropriate.

9.4.2 Stress range in parent material

(1) Depending upon the fatigue assessment carried out, either nominal stress ranges or geometric stress ranges shall be evaluated.

(2) When determining the stress at a detail, stresses arising from joint eccentricity and imposed deformations, secondary stresses due to joint stiffness, stress redistribution due to buckling and shear lag, and the effects of prying (see Chapter 6) shall be taken into account.

9.4.3 Stress range for welds

(1) In load-carrying partial penetration or fillet welded joints, the forces transmitted by a unit length of weld shall be resolved into components transverse and parallel to the longitudinal axis of the weld.

(2) The fatigue stresses in the weld shall be taken as:

- a normal stress σ_w transverse to the axis of the weld
- a shear stress τ_w longitudinal to the axis of the weld.

(3) The stresses σ_w and τ_w may be obtained by dividing the relevant component of the force transmitted per unit length of weld, by the throat size a.

(4) Alternatively σ_w and τ_w may be obtained by using the method given in normative Annex M and taking:

$$\sigma_w = [\sigma_\perp^2 + \tau_\perp^2]^{0,5} \quad \text{and} \quad \tau_w = \tau \qquad (9.4)$$

9.4.4 Design stress range spectrum

(1) The stress history due to a loading event shall be reduced to a stress range spectrum by employing a soundly based method of cycle counting.

(2) For a particular detail, the total of all stress range spectra, caused by all loading events, shall be compiled to produce the design stress range spectrum to be used for the fatigue assessment.

(3) *The design stress range spectrum for a typical detail or structural element may be derived from the stress history obtained by appropriate tests or by numerical evaluations based on the theory of elasticity.*

(4) *For many applications the "rainflow" or "reservoir" stress cycle counting methods are appropriate for use in conjunction with the Palmgren-Miner summation.*

(5) *Different components of a structure may have different stress range spectra.*

9.5 Fatigue assessment procedures

9.5.1 General

(1) The safety verification shall be carried out either:

- in terms of cumulative damage by comparing the applied damage to the limiting damage, or
- in terms of the equivalent stress range by comparing it with the fatigue strength for a given number of stress cycles.

(2) For a particular class of constructional detail, the stresses to be considered may be normal stresses or shear stresses or both.

(3) When a constructional detail is defined in the detail classification tables (Table 9.8.1 to Table 9.8.7) the nominal stress range shall be used, see **9.5.2**.

(4) The effects of geometric discontinuities which are not part of the constructional detail itself, such as holes, cut-outs or re-entrant corners shall be taken into account separately, either by a special analysis or by the use of appropriate stress concentration factors, to determine the modified nominal stress range.

(5) When a constructional detail differs from a detail defined in the detail classification tables by the presence of a geometric discontinuity in the detail itself, the geometric stress range shall be used, see **9.5.3**.

(6) For constructional details not included in the detail classification tables, the geometric stress range shall be used, see **9.5.3**.

9.5.2 Fatigue assessment based on nominal stress ranges

9.5.2.1 *Constant amplitude loading*

(1) For constant amplitude loading the fatigue assessment criterion is:

$$\gamma_{Ff}\ \Delta\sigma \leq \Delta\sigma_R / \gamma_{Mf} \tag{9.5}$$

where $\Delta\sigma$ is the nominal stress range

and $\Delta\sigma_R$ is the fatigue strength for the relevant detail category (see **9.8**) for the total number of stress cycles N during the required design life.

9.5.2.2 *Variable amplitude loading*

(1) For variable amplitude loading defined by a design spectrum, the fatigue assessment shall be based on Palmgren-Miner rule of cumulative damage.

(2) If the maximum stress range due to the variable amplitude loading is higher than the constant amplitude fatigue limit then one of the following types of fatigue assessment shall be made:

a) Cumulative damage, see (3).

b) Equivalent constant amplitude, see (7).

(3) A cumulative damage assessment may be made using:

$$D_d \leq 1 \qquad \text{where} \quad D_d = \Sigma \frac{n_i}{N_i} \tag{9.6}$$

in which n_i is the number of cycles of stress range $\Delta\sigma_i$ during the required design life

N_i is the number of cycles of stress range $\gamma_{Ff}\gamma_{Mf}\Delta\sigma_i$ to cause failure, for the relevant detail category, see **9.8**.

(4) Cumulative damage calculations shall be based on one of the following:

a) a fatigue strength curve with a single slope constant m = 3,

b) a fatigue strength curve with double slope constants (m = 3 and m = 5), changing at the constant amplitude fatigue limit.

c) a fatigue strength curve with double slope constants (m = 3 and m = 5), and a cut-off limit at N = 100 million cycles,

d) in the case described in **9.6.2.2**(2), a fatigue strength curve with a single slope constant m = 5 and a cut-off limit at N = 100 million cycles.

(5) *C ase c) is the most general. Stress ranges below the cut-off limit may be neglected.*

(6) When using case c) with a constant amplitude fatigue limit $\Delta\sigma_D$ *at 5 million cycles,* N_i *may be calculated as follows:*

- *if* $\gamma_{Ff}\ \Delta\sigma_i\ \geq \Delta\sigma_D/\gamma_{Mf}$:

$$N_i = 5 \times 10^6 \left[\frac{\Delta\sigma_D/\gamma_{Mf}}{\gamma_{Ff}\Delta\sigma_i} \right]^3 \tag{9.7}$$

- *if* $\Delta\sigma_D/\gamma_{Mf} > \gamma_{Ff}\ \Delta\sigma_i\ \geq \Delta\sigma_L/\gamma_{Mf}$:

$$N_i = 5 \times 10^6 \left[\frac{\Delta\sigma_D/\gamma_{Mf}}{\gamma_{Ff}\Delta\sigma_i} \right]^5 \tag{9.8}$$

- $\gamma_{Ff}\ \Delta\sigma_i < \Delta\sigma_L/\gamma_{Mf}$:

$$N_i = \infty \tag{9.9}$$

(7) An equivalent constant amplitude fatigue assessment may be made by checking the criterion:

$$\gamma_{Ff}\ \Delta\sigma_E \leq \Delta\sigma_R/\gamma_{Mf} \tag{9.10}$$

where $\Delta\sigma_E$ is the equivalent constant amplitude stress range which, for the given number of cycles, leads to the same cumulative damage as the design spectrum.

and $\Delta\sigma_R$ is the fatigue strength for the relevant detail category (see **9.8**), for the same number of cycles as used to determine $\Delta\sigma_E$.

(8) *A conservative assumption may be adopted in evaluating* $\Delta\sigma_E$ *and* $\Delta\sigma_R$ *by using a fatigue strength curve of unique slope constant m = 3.*

(9) *More generally,* $\Delta\sigma_E$ *may be calculated taking into account the double slope fatigue strength curve and the cut-off limit, as defined in Figure 9.1.2.*

(10) *Alternatively, an equivalent constant amplitude fatigue assessment may be made by checking the specific criterion:*

$$\gamma_{Ff}\ \Delta\sigma_{E.2} \leq \Delta\sigma_C/\gamma_{Mf} \tag{9.11}$$

where $\Delta\sigma_{E.2}$ *is the equivalent constant amplitude stress range for 2 million cycles, and*

$\Delta\sigma_C$ *is the reference value of the fatigue strength at 2 million cycles for the relevant detail category, see* ***9.8***.

9.5.2.3 ***Shear stress ranges***

(1) Nominal shear stress ranges, $\Delta\tau$, shall be treated similarly to nominal normal stress ranges, but using a single slope constant m = 5.

(2) For shear stresses, N_i *may be calculated as follows:*

- *if* $\gamma_{Ff}\ \Delta\tau_i\ \geq \Delta\tau_L/\gamma_{Mf}$:

$$N_i = 2 \times 10^6 \left[\frac{\Delta\tau_C/\gamma_{Mf}}{\gamma_{Ff}\ \Delta\tau_i} \right]^5 \tag{9.12}$$

- *if* $\gamma_{Ff}\ \Delta\tau_i < \Delta\tau_L/\gamma_{Mf}$:

$$N_i = \infty \tag{9.13}$$

9.5.2.4 ***Combination of normal and shear stress ranges***

(1) In the case of a combination of normal and shear stresses the fatigue assessment shall consider their combined effects.

(2) If the equivalent nominal shear stress range is less than 15 % of the equivalent nominal normal stress range, the effects of the shear stress range may be neglected.

(3) At locations other than weld throats, if the normal and shear stresses induced by the same loading event vary simultaneously, or if the plane of the maximum principal stress does not change significantly in the course of a loading event, the maximum principal stress range may be used.

(4) If, at the same location, normal and shear stresses vary independently, the components of damage for normal and shear stresses shall be assessed separately using the Palmgren-Miner rule, then combined using the criterion:

$$D_{d.\sigma} + D_{d.\tau} \leq 1 \tag{9.14}$$

in which $D_{d.\sigma} = \Sigma(n_i/N_i)$ for normal stress ranges $\Delta\sigma_i$

and $D_{d.\tau} = \Sigma(n_i/N_i)$ for shear stress ranges ΔT_i

(5) When using equivalent constant amplitude stress ranges, this criterion generally becomes:

$$\left[\frac{\gamma_{Ff}\ \Delta\sigma_E}{\Delta\sigma_R/\gamma_{Mf}}\right]^3 + \left[\frac{\gamma_{Ff}\ \Delta\tau_E}{\Delta\tau_R/\gamma_{Mf}}\right]^5 \leq 1 \tag{9.15}$$

(5) *Alternatively, an equivalent constant amplitude fatigue assessment may be made using the specific criterion:*

$$\left[\frac{\gamma_{Ff}\ \Delta\sigma_{E.2}}{\Delta\sigma_C/\gamma_{Mf}}\right]^3 + \left[\frac{\gamma_{Ff}\ \Delta\tau_{E.2}}{\Delta\tau_C/\gamma_{Mf}}\right]^5 \leq 1 \tag{9.16}$$

(6) Stress ranges in welds shall be determined as specified in **9.4.3**. The components of damage for normal and shear stresses shall be assessed separately using the Palmgren-Miner rule, then combined using the criterion:

$$D_{d.\sigma} + D_{d.\tau} \leq 1 \tag{9.17}$$

in which $D_{d.\sigma} = \Sigma(n_i/N_i)$ for stress ranges of the normal stress σ_w defined in **9.4.3**.

and $D_{d.\tau} = \Sigma(n_i/N_i)$ for stress ranges of the shear stress τ_w defined in **9.4.3**.

9.5.3 Fatigue assessments based on geometric stress ranges

(1) The geometric stress is the maximum principal stress in the parent material adjacent to the weld toe taking into account only the overall geometry of the joint, excluding local stress concentration effects due to the weld geometry and discontinuities at the weld toe.

(2) The maximum value of the geometric stress range shall be found, investigating various locations at the weld toe around the welded joint or the stress concentration area.

(3) The geometric stresses may be determined using stress concentration factors obtained from parametric formulae within their domains of validity, a finite element analysis or an experimental model.

(4) A fatigue assessment based on the geometric stress range, shall be treated similarly to the assessments given in **9.5.2**, but replacing the nominal stress range by the geometric stress range.

(5) The fatigue strength to be used in assessments based on geometric stress ranges shall be determined by reference to **9.6.3**.

9.6 Fatigue strength

9.6.1 General

(1) The fatigue strength is defined for normal stresses by a series of log $\Delta\sigma_R$ – log N curves, each applying to a typical detail category. Each detail category is designated by a number which represents, in N/mm^2, the reference value $\Delta\sigma_C$ of the fatigue strength at 2 million cycles, see Figure 9.6.1. The values used are rounded values, corresponding to the detail categories given in Table 9.6.1.

(2) The fatigue strength curves for nominal normal stresses are defined by:

$$\log N = \log a - m \log \Delta\sigma_R \tag{9.18}$$

where

$\Delta\sigma_R$ is the fatigue strength

N is the number of stress range cycles

m is the slope constant of the fatigue strength curves, with values of 3 and/or 5.

log a is a constant which depends on the related part of the slope, see **9.6.2.1**.

(3) Similar fatigue strength curves are used for shear stresses, see Figure 9.6.2 and Table 9.6.2.

(4) *The curves are based on representative experimental investigations and thus include the effects of:*

- *local stress concentrations due to the weld geometry,*
- *size and shape of acceptable discontinuities,*
- *the stress direction,*
- *residual stresses,*
- *metallurgical conditions,*
- *in some cases, the welding process and post-weld improvement procedures.*

(5) When test data are used to determine the appropriate detail category for a particular constructional detail, the value of the stress range $\Delta\sigma_R$ corresponding to a value of N of 2 million cycles shall be calculated for a 75 % confidence interval of 95 % probability of survival for log N, taking into account the standard deviation and the sample size. The number of data points (not lower than 10) shall be considered in the statistical analysis.

(6) Proper account shall be taken of the fact that residual stresses are low in small scale samples. The resulting fatigue strength curve shall be corrected to allow for the greater effect of residual stresses in full scale structures.

(7) *The level of acceptable discontinuities are defined in Reference Standard 9, see normative Annex B.*

(8) Separate fatigue strength curve definitions are given for:

- Classified details, for which the nominal stress range procedure applies, see **9.6.2**.
- Non-classified details, for which the geometrical stress range procedure applies, see **9.6.3**.

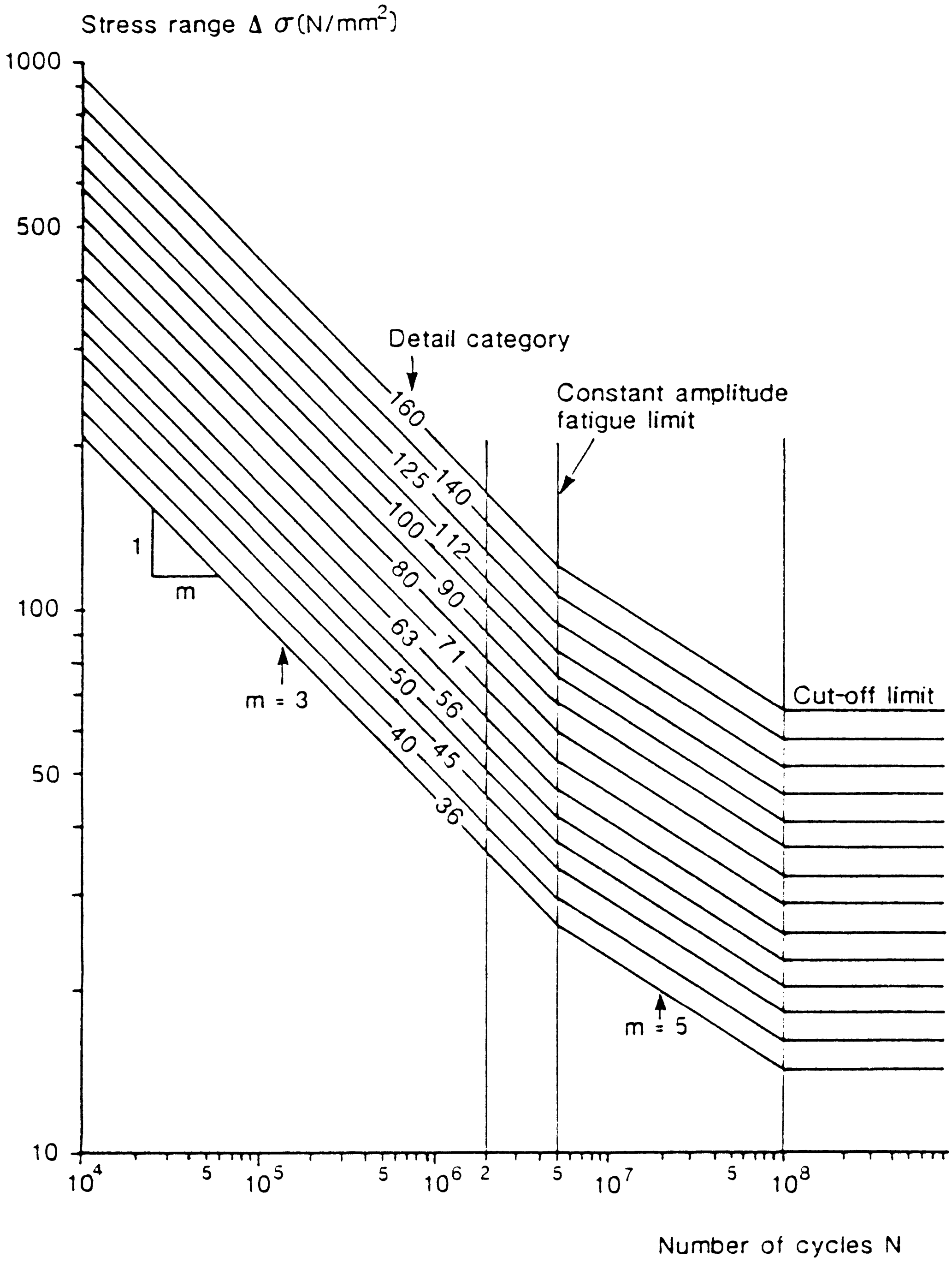


Figure 9.6.1 — Fatigue strength curves for normal stress ranges

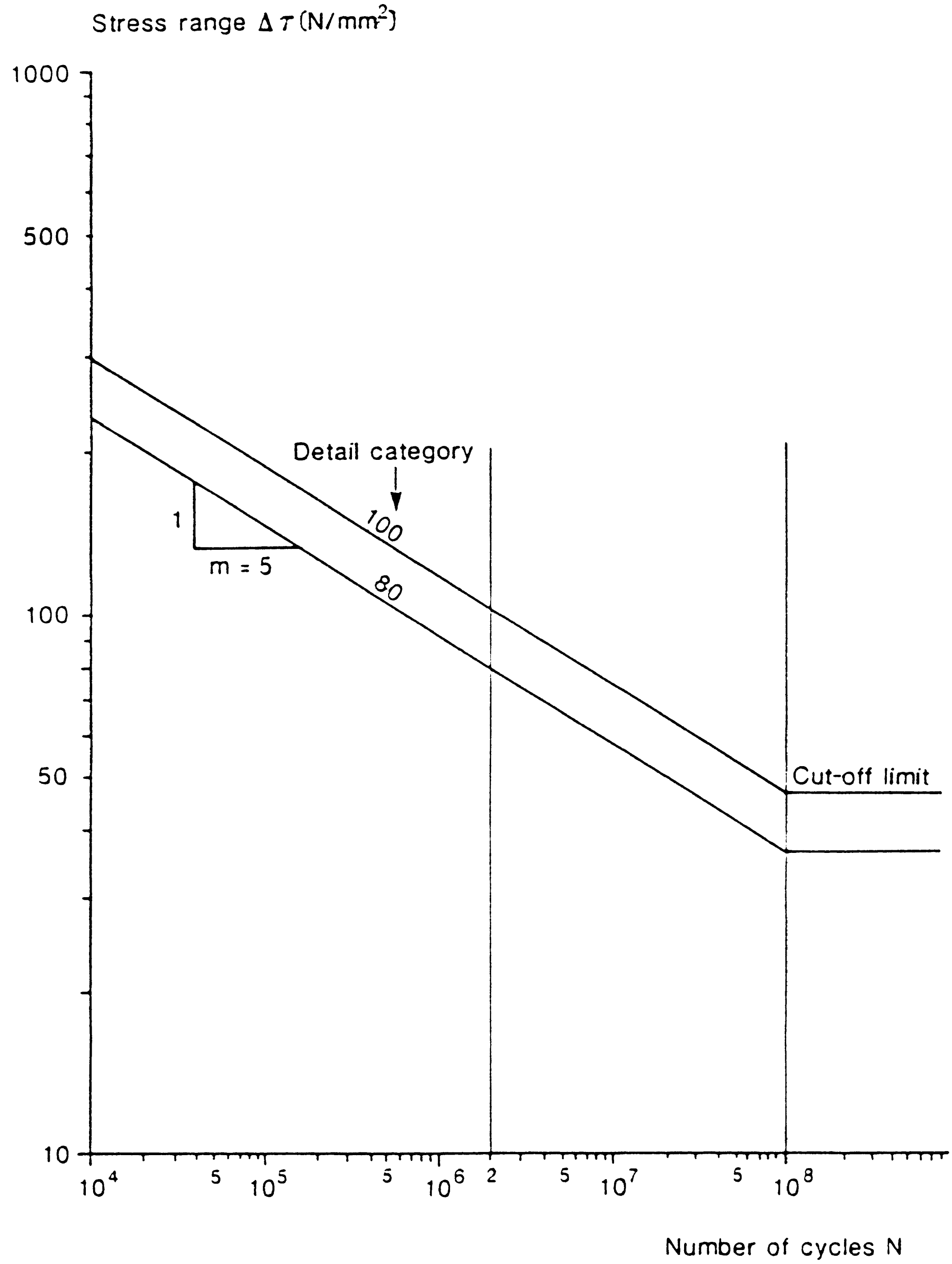


Figure 9.6.2 — Fatigue strength curves for shear stress ranges

9.6.2 Fatigue strength curves for classified details

9.6.2.1 *Fatigue strength curves for open sections*

(1) The detail categories to be used for various typical constructional details for open sections are given in 5 tables as follows:

Table 9.8.1: Non-welded details.

Table 9.8.2: Welded built-up sections.

Table 9.8.3: Transverse butt welds.

Table 9.8.4: Welded attachments with non-load carrying welds.

Table 9.8.5: Welded joints with load-carrying welds.

(2) In Table 9.8.1 onwards, the arrows in the diagrams indicate the location and direction of the stresses to which the relevant fatigue strengths apply.

(3) The detail category used to designate a particular fatigue strength curve corresponds to the reference value (in N/mm²) of the fatigue strength at 2 million cycles, $\Delta\sigma_C$ or $\Delta\tau_C$ as appropriate.

(4) Fatigue strength curves for nominal normal stress ranges for a number of typical detail categories are given in Figure 9.6.1. The constant amplitude fatigue limit corresponds to the fatigue strength for 5 million cycles and the cut-off limit corresponds to the fatigue strength for 100 million cycles.

(5) The corresponding values for calculating the fatigue strength are given in Table 9.6.1.

Table 9.6.1 — Numerical values for fatigue strength curves for normal stress ranges

Detail category	**log a for N < 10⁸**		**Stress range at constant amplitude fatigue limit**	**Stress range at cut-off limit**
	$N \leq 5 \times 10^6$	$N \geq 5 \times 10^6$	$(N = 5 \times 10^6)$	$(N = 10^8)$
$\Delta\sigma_C$ (N/mm²)	(m = 3)	(m = 5)	$\Delta\sigma_D$ (N/mm²)	$\Delta\sigma_L$ (N/mm²)
160	12,901	17,036	117	64
140	12,751	16,786	104	57
125	12,601	16,536	93	51
112	12,451	16,286	83	45
100	12,301	16,036	74	40
90	12,151	15,786	66	36
80	12,001	15,536	59	32
71	11,851	15,286	52	29
63	11,701	15,036	46	26
56	11,551	14,786	41	23
50	11,401	14,536	37	20
45	11,251	14,286	33	18
40	11,101	14,036	29	16
36	10,951	13,786	26	14

(6) Fatigue strength curves for nominal shear stress ranges are given in Figure 9.6.2. They have a single slope constant of m = 5. There is no constant amplitude fatigue limit for these curves but the cut-off limit at 100 million cycles applies as for nominal normal stress ranges.

(7) The corresponding values for calculating the fatigue strength are given in Table 9.6.2.

(8) Detail category 100 is for parent metal, full penetration butt welds and for bearing type fitted bolts in shear.

(9) Detail category 80 is for fillet welds and for partial penetration butt welds in shear.

Table 9.6.2 — Numerical values for fatigue strength curves for shear stress ranges

Detail category $\Delta\tau_C$ (N/mm²)	**log a for N < 10⁸** (m = 5)	**Stress range at cut-off limit (N = 10⁸)** $\Delta\tau_L$ (N/mm²)
100	16,301	46
80	15,801	36

9.6.2.2 ***Fatigue strength curves for hollow sections***

(1) The fatigue strength curves to be used in conjunction with the hollow section details shown in Table 9.8.6, are those given in Figure 9.6.1. They have double slope constants of m = 3 and m = 5.

(2) The fatigue strength curves to be used in conjunction with the hollow section joint details for lattice girders shown in Table 9.8.7, are given in Figure 9.6.3. They have a single slope constant of m = 5.

(3) The corresponding values for numerical calculations of the fatigue strength are given in Table 9.6.3.

(4) The throat thickness of a fillet weld shall not be less than the wall thickness of the hollow section member which it connects.

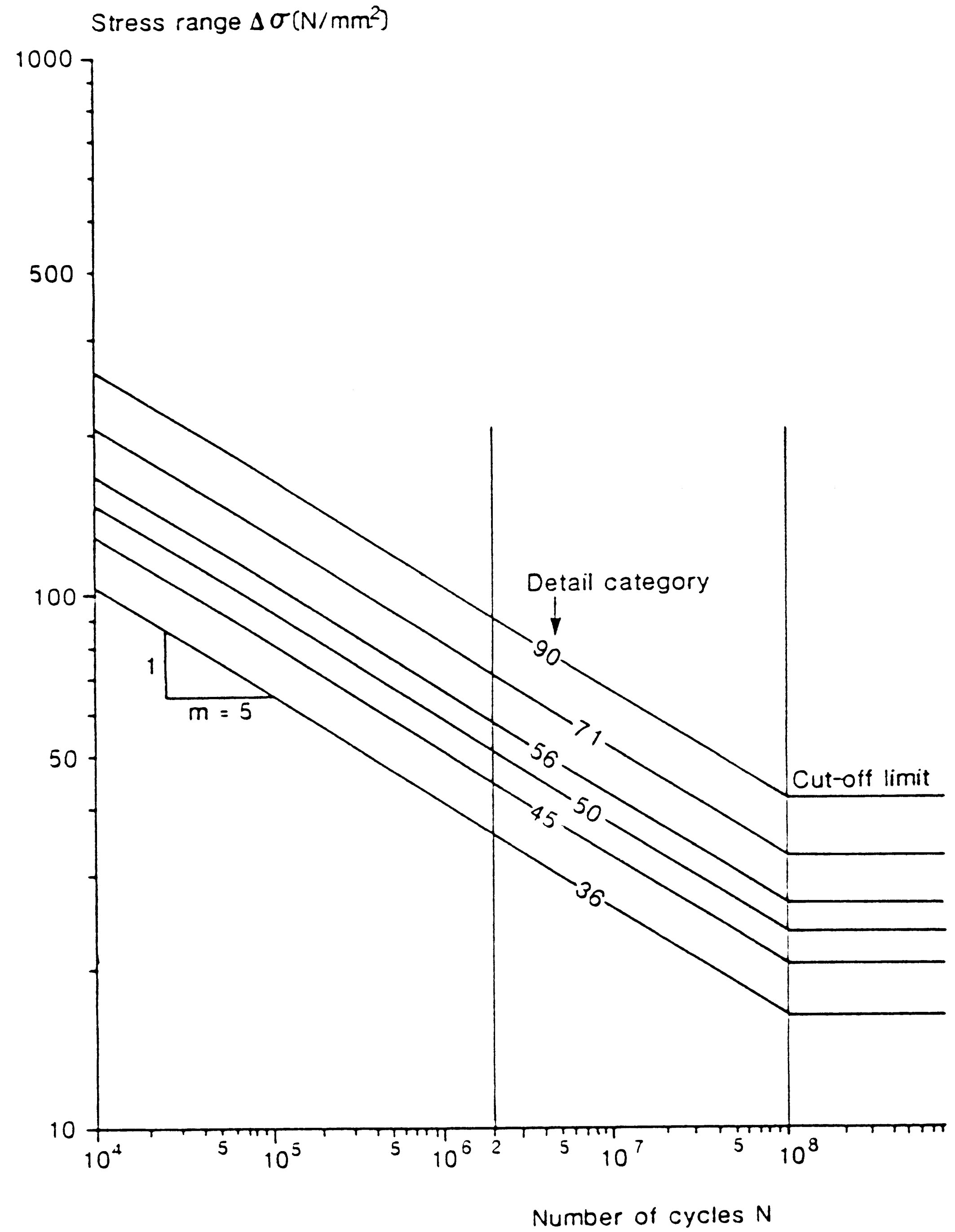


Figure 9.6.3 — Fatigue strength curves for joints in tubular lattice girders

Table 9.6.3 — Numerical values for fatigue strength curves for hollow sections

Detail category $\Delta\sigma_C$ (N/mm²)	log a for N < 10^8 (m = 5)	Stress range at cut-off limit (N = 10^8) $\Delta\sigma_L$ (N/mm²)
90	16,051	41
71	15,551	32
56	15,051	26
50	14,801	23
45	14,551	20
36	14,051	16

(5) The member forces may be analysed neglecting the effect of eccentricities and joint stiffness, assuming hinged connections, provided that the effects of secondary bending moments on stress ranges are considered.

(6) In the absence of rigorous stress analysis and modelling of the joint, the effects of secondary bending moments may be taken into account by multiplying the stress ranges due to axial member forces by appropriate coefficients as follows:

- for joints in lattice girders made from circular hollow sections, see Table 9.6.4.
- for joints in lattice girders made from rectangular hollow sections, see Table 9.6.5.

(7) For clarification of the terminology used in Table 9.6.4 and Table 9.6.5, see Table 9.8.7.

Table 9.6.4 — Coefficients to account for secondary bending moments in joints of lattice girders made from circular hollow sections

Type of joint		Chords	Verticals	Diagonals
Gap joints	K type	1,5	1,0	1,3
	N type	1,5	1,8	1,4
Overlap joints	K type	1,5	1,0	1,2
	N type	1,5	1,65	1,25

Table 9.6.5 — Coefficients to account for secondary bending moments in joints of lattice girders made from rectangular hollow sections

Type of joint		Chords	Verticals	Diagonals
Gap joints	K type	1,5	1,0	1,5
	N type	1,5	2,2	1,6
Overlap joints	K type	1,5	1,0	1,3
	N type	1,5	2,0	1,4

9.6.3 Fatigue strength curves for non-classified details

(1) The fatigue assessment of all constructional details not included in Table 9.8.1 to Table 9.8.7 and of all hollow section members and tubular joints with wall thicknesses greater than 12,5 mm, shall be carried out using the procedure based on geometric stress ranges, given in **9.5.3**.

(2) The fatigue strength curves to be used for fatigue assessments based on geometric stress ranges, shall be:

a) For full penetration butt welds:

- Category 90, in Figure 9.6.1, when both weld profile and permitted weld defects acceptance criteria are satisfied.
- Category 71, in Figure 9.6.1, when only permitted weld defects acceptance criteria are satisfied.

b) For load carrying partial penetration butt welds and fillet welds:

- Category 36, in Figure 9.6.1, or alternatively a fatigue strength curve obtained from adequate fatigue test results.

c) For stress ranges in welds see **9.4.3**.

9.7 Fatigue strength modifications

9.7.1 Stress range in non-welded or stress relieved details

(1) In non-welded details or stress relieved welded details, the effective stress range to be used in the fatigue assessment shall be determined by adding the tensile portion of the stress range and 60 % of the compressive portion of the stress range.

9.7.2 Influence of thickness

(1) The fatigue strength depends on the thickness of the parent metal in which a potential crack may initiate and propogate.

(2) The variation of fatigue strength with thickness shall be taken into account for material thicknesses greater than 25 mm by reducing the fatigue strength using:

$$\Delta\sigma_{R.t} = \Delta\sigma_R \, (25/t)^{0,25} \qquad (9.19)$$

with $t > 25$ mm

(3) When the material thickness of the constructional detail is less than 25 mm the fatigue strength shall be taken as that for a thickness of 25 mm.

(4) This reduction for thickness shall be applied only to structural details with welds transverse to the direction of the normal stresses.

(5) Where the detail category in the classification tables already varies with thickness, the above correction for thickness shall not be applied.

9.7.3 Modified fatigue strength curves

(1) Test data for certain details do not fit the fatigue strength curves given in Figure 9.6.1. In order to avoid any non-conservative conditions, such details are allocated to one detail category lower than their fatigue strength at 2 million cycles would otherwise indicate.

(2) These details are identified by an asterisk in Table 9.8.1 to Table 9.8.5. The classification of such details may be increased by one detail category in Table 9.6.1, provided that modified fatigue strength curves are adopted in which the constant amplitude fatigue limit is taken as the fatigue strength at 10 million cycles for $m = 3$, see Figure 9.7.1.

(3) The numerical values necessary for calculating a modified value of fatigue strength are given in Table 9.7.1.

Table 9.7.1 — Numerical values for modified fatigue strength curves for normal stress ranges

Detail category	log a for $N < 10^8$		Stress range at constant amplitude fatigue limit	Stress range at cut-off limit
(Nominal)	$N \leq 10^7$ ($m = 3$)	$N \geq 10^7$ ($m = 5$)	($N = 10^7$) $\Delta\sigma_D$ (N/mm^2)	($N = 10^8$) $\Delta\sigma_L$ (N/mm^2)
50*	11,551	14,585	33	21
45*	11,401	14,335	29	18
36*	11,101	13,835	23	15

9.8 Classification tables

(1) The classification of the constructional details listed in Table 9.8.1 to Table 9.8.7 has been established on the basis of stresses along the direction indicated by the arrow for potential cracks on the surface of the parent metal, or for the case of weld throat cracking, on the stress calculated in the weld throat.

(2) The stresses shall be calculated using the gross or net section of the loaded member as appropriate.

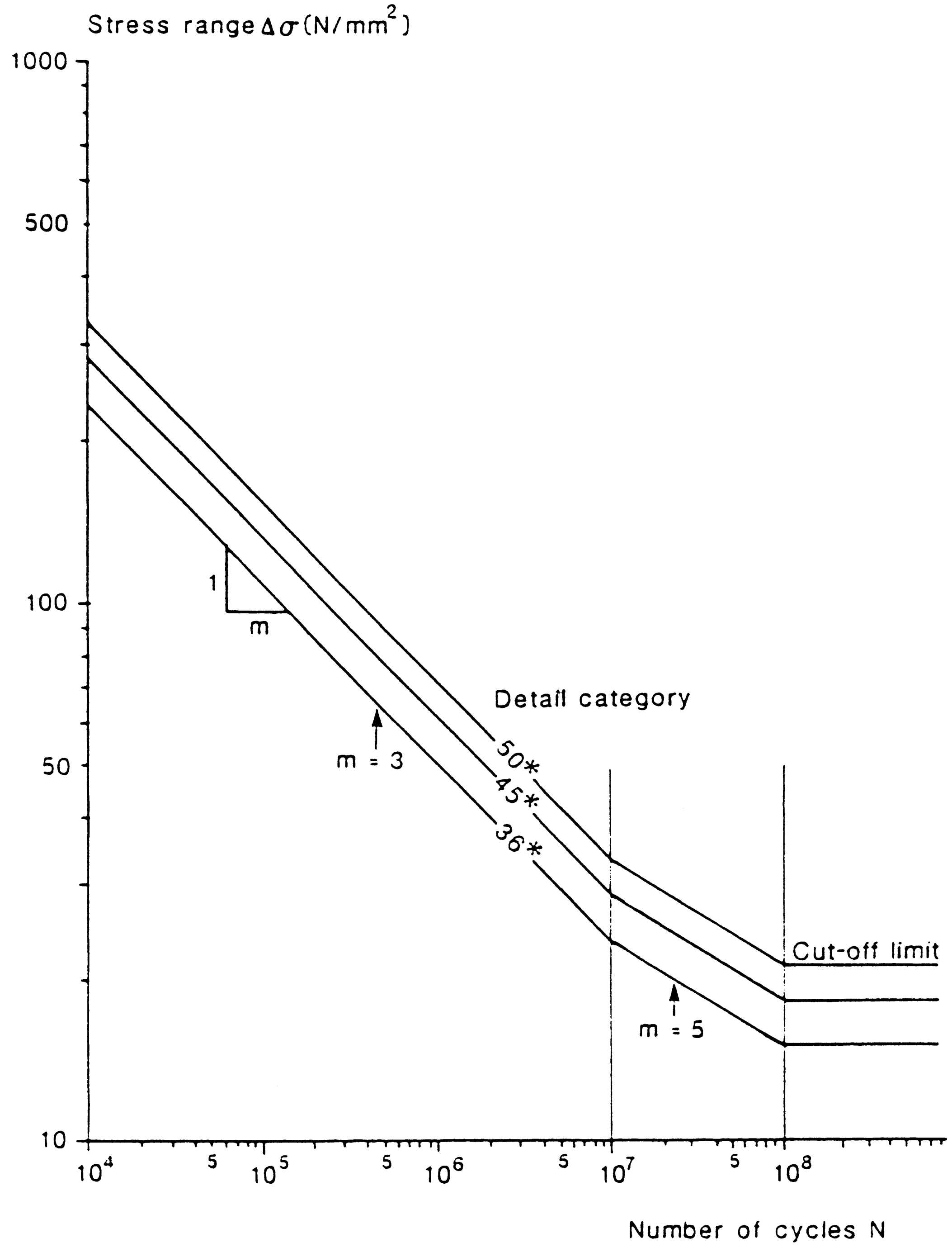


Figure 9.7.1 — Modified fatigue strength curve

Table 9.8.1 — Non-welded details — sheet 1 of 2

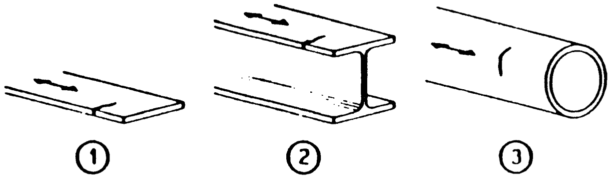
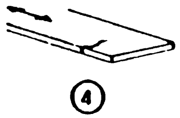
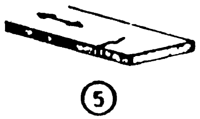
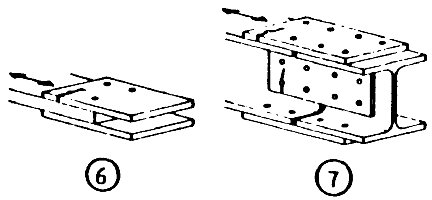
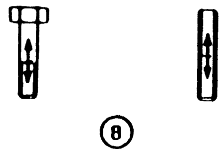
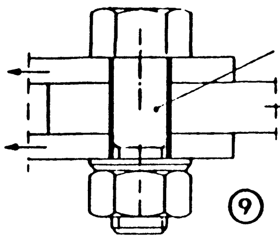
Detail category	Constructional details	Description	Requirement
160		Rolled and extruded products ① Plates and flats. ② Rolled sections. ③ Seamless hollow sections (see Table 9.8.6 and Table 9.8.7.	① to ③: Sharp edges, surface and rolling flaws to be improved by grinding.
140		Sheared or gas cut plates ④ Machine gas cut or sheared material with no drag lines.	④ All visible signs of edges discontinuities should be removed.
125		⑤ Manually gas cut material or material with machine gas cut edges with shallow and regular drag lines.	⑤ Subsequent dressed to remove all edge discontinuities. ④ and ⑤ — No repair by weld refill. — Re-entrant corners (slope < 1 : 4) or aperture should be improved by grinding for any visible defects. — At apertures the design stress area should be taken as the net cross section area.

Table 9.8.1 — Non-welded details — sheet 2 of 2

Detail category	Constructional details	Description	Requirement
112		Bolted connections ⑥ Unsupported one-sided connections shall be avoided or else effects of eccentricities taken into account when calculating stresses. ⑦ Beam splices or bolted cover plates.	⑥ and ⑦ — Stresses to be calculated in the gross section for slip resistant connections or in the net section for all other connections.
36[a]		⑧ Bolts and threaded rods in tension. For preloaded bolts, the stress range in the bolt depends upon the level of preload and the geometry of the connection.	⑧ — Tensile stresses to be calculated using the tensile stress area of the bolt.
100 m = 5		Bolts in single or double shear ⑨ Fitted bolt of bearing type made of high strength steel as defined in chapter 3 (bolts grade 8.8 and 10.9).	⑨ — Design shear stress evaluated using the shank area of the bolt. — Only bearing type fitted bolts are covered by this detail category.

[a] See clause **9.7.3**

Table 9.8.2 — Welded built-up sections — sheet 1 of 2

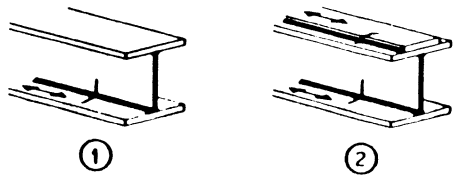
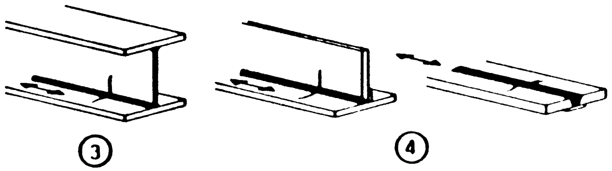
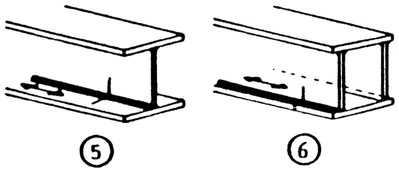
Detail category	Constructional details	Description	Requirement
125		<u>Continuous longitudinal welds</u> ① Automatic butt welds carried out from both sides. If a specialist inspection demonstrates that longitudinal welds are free from significant flaws, category 140 may be used. ② Automatic fillet welds. Cover plate ends shall be verified using detail ⑤ in Table 9.8.5.	① and ② — No stop/start position is permitted except when the repair is performed by a specialist and inspection carried out to verify the proper execution of the repair.
112		③ Automatic fillet or butt welds carried out from both sides but containing stop/start positions. ④ Automatic butt welds made from one side only, with a backing bar, but without stop/start positions.	④ — When this detail contains stop/start positions use category 100.
100		⑤ Manual fillet or butt welds. ⑥ Manual or automatic butt welds carried out from one side only, particularly for box girders.	⑥ — A very good fit between the flange and web plates is essential. Prepare the web edge such that the root face is adequate for the achievement of regular root penetration without break-out.

Table 9.8.2 — Welded built-up sections — sheet 2 of 2

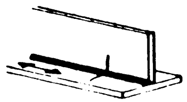
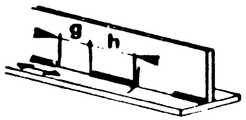
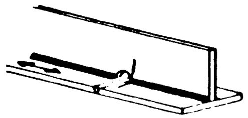
Detail category	Constructional details	Description	Requirement
100	 ⑦	⑦ Repaired automatic or manual fillet or butt welds.	⑦ — Improvement methods which are adequately verified may restore the original category.
80	 ⑧	<u>Intermittent longitudinal welds</u> ⑧ Stitch or tack welds not subsequently covered by a continuous weld.	⑧ — Intermittent fillet weld with gap ratio g/h ⩽ 2.5.
71	 ⑨	⑨ Ends of continuous welds at cope holes.	⑨ — Cope hole not to be filled with weld metal.

Table 9.8.3 — Transverse butt welds — sheet 1 of 2

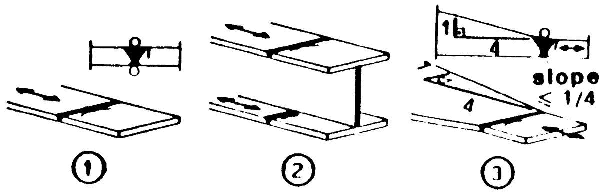
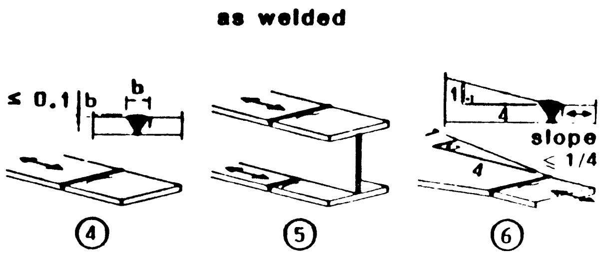
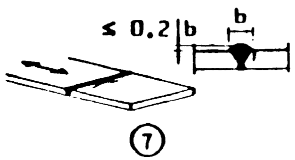
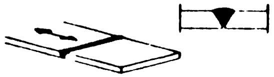
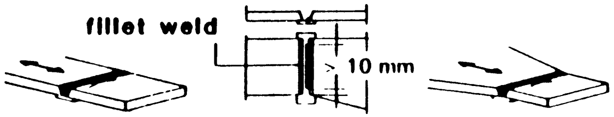
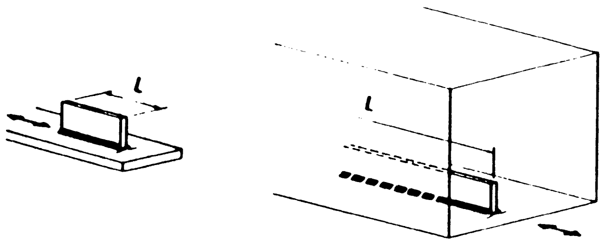
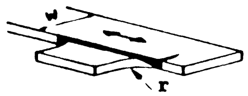
Detail category	Constructional details	Description	Requirement
112		<u>Without backing bar</u> ① Transverse splices in plates, flats and rolled sections. ② Flange splices in plate girders before assembly. ③ Transverse splices in plates or flats tapered in width or in thickness where the slope is not greater than 1 : 4.	① and ② — Details ① and ② may be increased to Category 125 when high quality welding is achieved and is verified to satisfy the tolerances of Reference Standard 9 — Quality level 3. ①, ② and ③ — All welds ground flush to plate surface parallel to direction of the arrow.
90		④ Transverse splices in plates or flats. ⑤ Transverse splices in rolled sections or welded plate girders. ⑥ Transverse splices in plates or flats tapered in width or in thickness where the slope is not greater than 1 : 4.	④, ⑤ and ⑥ — The height of the weld convexity to be not greater than 10 % of the weld width, with smooth transitions to plate surface. — Welds made in flat position.
80		⑦ Transverse splices in plates, flats, rolled sections or plate girders.	⑦ — The height of the weld convexity to be not greater than 20 % of the weld width. ① to ⑦ — Weld run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. — Welds made from two sides.

Table 9.8.3 — Transverse butt welds — sheet 2 of 2

Detail category	Constructional details	Description	Requirement
36[a]	 ⑧	⑧ Butt welds made from one side only.	⑧ — Without backing strip.
71	 ⑨ ⑩	With backing strip ⑨ Transverse splice. ⑩ Transverse butt weld tapered in width or in thickness where the slope is not greater than 1 : 4.	⑨ and ⑩ — The fillet weld which attaches the backing strip to terminate more then 10 mm from the edges of the stressed plate.
50	 ⑪	⑪ Transverse butt weld on a permanent backing strip.	⑪ — When backing strip fillet welds are terminated closer than 10 mm to the plate edge, or when good fit cannot be guaranteed.

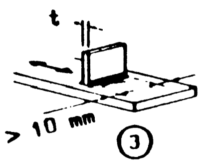
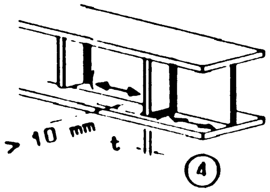
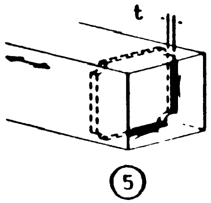
[a] See clause **9.7.3**

Table 9.8.4 — Welded attachements with non-load carrying welds — sheet 1 of 2

Detail category	Constructional details		Description	Requirement
80	$L \leqslant 50$ mm	 ①	Longitudinal attachments ① The Detail Category varies according to the length of the attachment, L.	
71	$50 < L \leqslant 100$ mm			
50[a]	$L > 100$ mm			
90	$\frac{1}{3} \leqslant \frac{r}{w}$ $r > 150$mm	 ②	② Gusset plate, welded to the edge of plate or beem flange.	② — Smooth transition radius, r, formed by initially machining or gas cutting the gusset plate before welding and then subsequently grinding the weld area parallel to the direction of the arrow.
71	$\frac{1}{6} \leqslant \frac{r}{w} < \frac{1}{3}$			
45[a]	$\frac{r}{w} < \frac{1}{6}$			

[a] See clause **9.7.3**

Table 9.8.4 — Welded attachments with non-load carrying welds — sheet 2 of 2

<table>
<tr><th>Detail category</th><th colspan="2">Constructional details</th><th>Description</th><th>Requirement</th></tr>
<tr><td>80</td><td>t ⩽ 12 mm</td><td rowspan="2"></td><td rowspan="2"><u>Transverse attachments</u>
③ Welds which terminate more than 10 mm from the edge of the plate.

④ Vertical stiffeners welded to a beam or a plate girder.

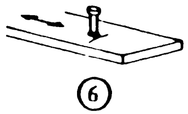
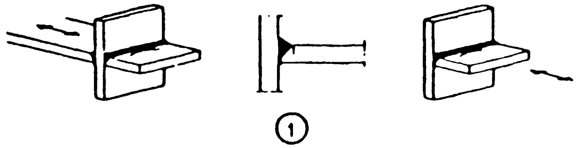
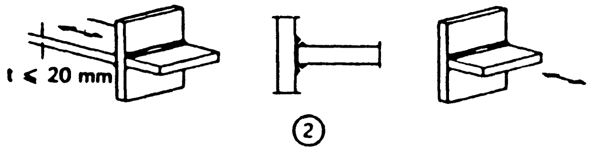
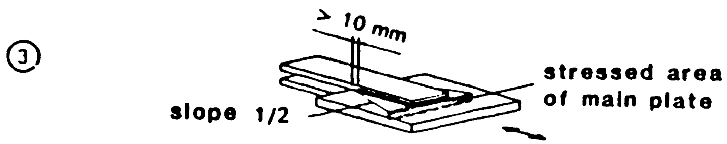
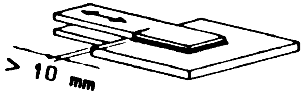
⑤ Diaphragms of box girders welded to the flange or web.</td><td rowspan="2">④
— The stress range should be calculated using principal stresses if the stiffener terminates in the web.</td></tr>
<tr><td>71</td><td>t > 12 mm</td></tr>
<tr><td>80</td><td colspan="2"></td><td>⑥ The effect of welded shear connectors on base material.</td><td></td></tr>
</table>

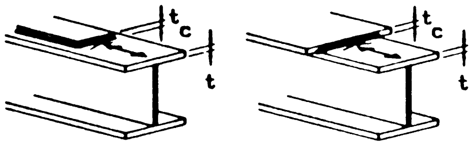
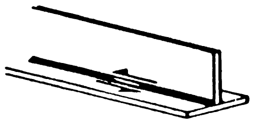
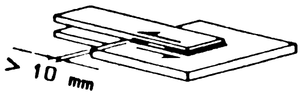
Table 9.8.5 — Welded joints with load-carrying welds — sheet 1 of 3

Detail category	Constructional details	Description	Requirement
71		Cruciform joints ① Full penetration butt weld.	① — Inspected and found free from discontinuities outside the tolerances of Reference Standard 9 — Quality Level 3.
36[a]		② Partial penetration tee-butt joint or fillet welded joint and effective full penetration in tee-butt joint as defined in Figure 6.6.9(a).	② — Two fatigue assessments are required. Firstly, root cracking is evaluated according to **9.4.3**, taking Category 36[a] for σ_w and Category 80 for τ_w. Secondly, toe cracking is evaluated by determining the stress range in the load-carrying plates, Category 71. ① and ② — The misalignment of the load-carrying plates should not exceed 15 % of the thickness of the intermediate plate.
63		Overlapped welded joints ③ Fillet welded lap joint.	③ — Stress in the main plate to be calculated on the basis of area shown in the sketch.

[a] See clause **9.7.3**

Table 9.8.5 — Welded joints with load-carrying welds — sheet 2 of 3

<table>
<tr><th>Detail category</th><th colspan="2">Constructional details</th><th>Description</th><th>Requirement</th></tr>
<tr><td>45[a]</td><td colspan="2">
④</td><td><u>Overlapped welded joints</u>
④ Fillet welded lap joint.</td><td>④
— Stress to be calculated in the overlapping plate elements.

③ and ④
— Weld terminations more then 10 mm from plate edge.
— Shear cracking in the weld should be verified using detail ⑦.</td></tr>
<tr><td>50[a]</td><td>t and t_c
⩽ 20 mm</td><td rowspan="2">
⑤</td><td rowspan="2"><u>Cover plates in beams and plate girders</u>
⑤ End zones of single or multiple welded cover plates, with or without frontal weld.</td><td rowspan="2">⑤
— When the cover plate is wider than the flange, a frontal weld, carefully ground to remove undercut, is necessary.</td></tr>
<tr><td>36[a]</td><td>t or t_c
> 20 mm</td></tr>
<tr><td>80
m = 5</td><td colspan="2">
⑥

⑦</td><td><u>Welds in shear</u>
⑥ Continuous fillet welds transmitting a shear flow, such as web to flange welds in plate girders. For continuous full penetration butt weld in shear use category 100.

⑦ Fillet welded lap joint.</td><td>⑥
— Stress range to be calculated from the weld throat area.

⑦
— Stress range to be calculated from the weld throat area considering the total length of the weld.
— Weld terminations more than 10 mm from plate edge.</td></tr>
<tr><td colspan="5">[a] See clause **9.7.3**</td></tr>
</table>

Table 9.8.5 — Welded joints with load-carrying welds — sheet 3 of 3

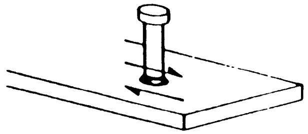
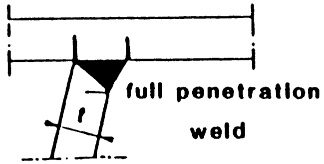
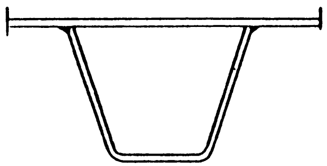
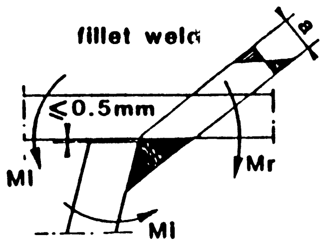
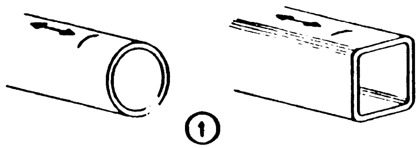
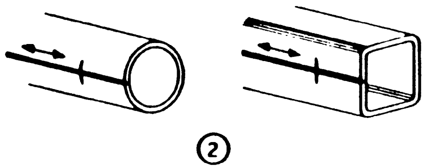
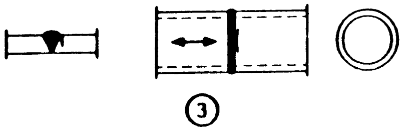
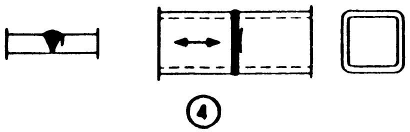
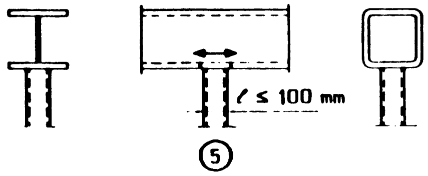
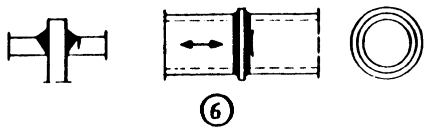
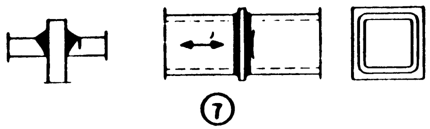
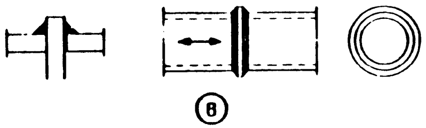
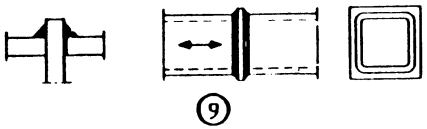
Detail category	Constructional details		Description	Requirement
80 m = 5	 ⑧		Welds in shear ⑧ Stud connectors (failure in the weld or heat effected zone).	⑧ — The shear stress to be calculated on the nominal cross-section of the stud.
71		 ⑨	Trapezoidal stiffener to deck plate welds ⑨ With fillet weld or full or partial penetration butt weld.	⑨ — For a full penetration butt weld, the bending stress range shall be calculated on the basis of the thickness of the stiffener. — For fillet weld or a partial penetration butt weld, the bending stress range shall be calculated on the basis of the throat thickness of the weld, or the thickness of the stiffener if smaller.
50				

Table 9.8.6 — Hollow sections — sheet 1 of 2([a])

Detail category	Constructional details	Description	Requirement
160		<u>Rolled and extruded products</u> ① Non-welded elements.	① — Sharp edges and surface flaws to be improved by grinding.
140		<u>Continuous longitudinal welds</u> ② Automatic longitudinal seam welds (for all other cases, see Table 9.8.2).	② — No stop/start positions, end free from defects outside the tolerances of Reference Standard 9 — Quality Level 3.
71		<u>Transverse butt welds</u> ③ Butt-welded end-to-end connection of circular hollow sections.	③ and ④ — Height of the weld convexity less then 10 % of the weld width, with smooth transitions to the plate surface. — Welds made in flat position and inspected and found free from defects outside the tolerances of Reference Standard 9 — Quality Level 3. — Details with wall thicknesses greater then 8 mm may be classified two Category higher.
56		④ Butt-welded end-to-end connection of rectangular hollow sections.	

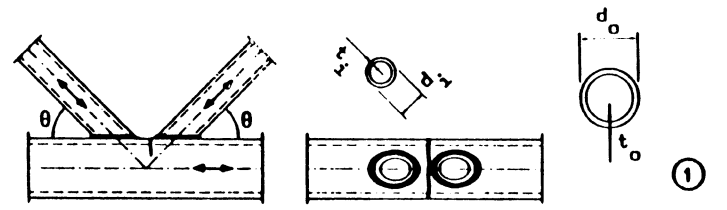
[a] (t ⩽ 12.5 mm)

Table 9.8.6 — Hollow sections — sheet 2 of 2([a])

Detail category	Constructional details	Description	Requirement
71		Welded attachments ⑤ Circular or rectangular hollow section, fillet welded to another section.	⑤ — Non-load-carrying welds. — Section width parallel to stress direction ⩽ 100 mm. — All other cases, see Table 9.8.4.
50		— Welded splices ⑥ Circular hollow sections, butt welded end-to-end with an intermediate plate.	⑥ and ⑦ — Load-carrying welds. — Welds inspected and found free from defects outside the tolerances of Reference Standard 9 — Quality Level 3. — Details with wall thickness greater than 8 mm may be classified one Detail Category higher.
45		⑦ Rectangular hollow sections, butt welded end-to-end with an intermediate plate.	
40		⑧ Circular hollow sections, fillet welded end-to-end with an intermediate plate.	⑧ and ⑨ — Load-carrying welds. — Wall thickness less than 8 mm.
36		⑨ Rectangular hollow sections, fillet welded end-to-end with an intermediate plate.	

[a] (t ⩽ 12.5 mm)

Table 9.8.7 — Lattice girder joints — sheet 1 of 2 (m = 5)

<table>
<tr><th>Detail category</th><th colspan="2">Constructional details</th><th>Description</th><th>Requirement</th></tr>
<tr><td>90</td><td>$\frac{t_o}{t_i} \geqslant 2.0$</td><td rowspan="2"></td><td rowspan="4"><u>Joints with gap</u> (*)
① Circular hollow sections, K and N joints.

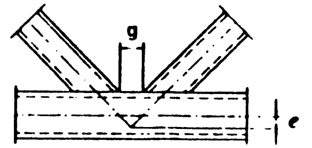
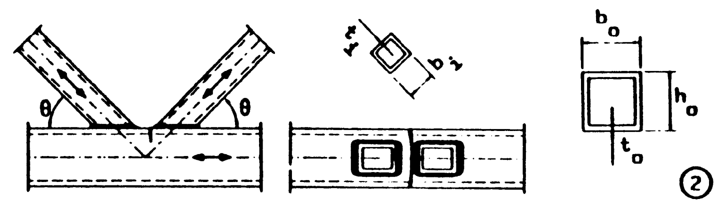
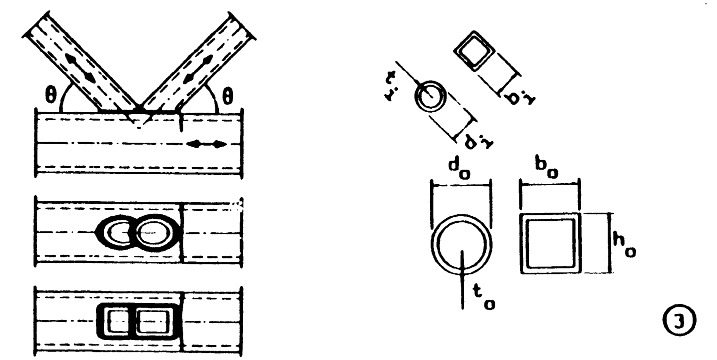
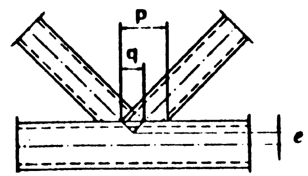
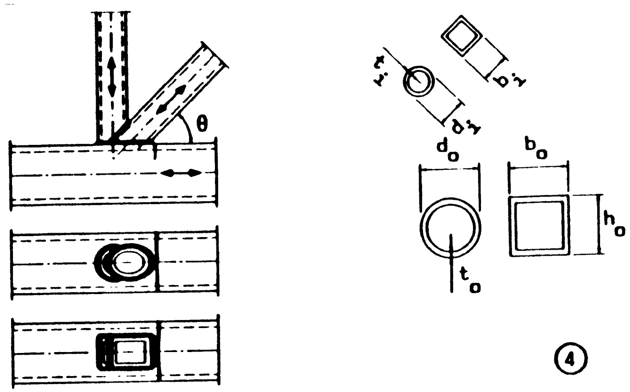
② Rectangular hollow sections, K and N joints.</td><td rowspan="4">
②
— $0.5\ (b_o - b_i) \leqslant g \leqslant 1.1\ (b_o - b_i)$
— $g \geqslant 2\ t_o$
g: gap
e: positive eccentricity</td></tr>
<tr><td>45</td><td>$\frac{t_o}{t_i} = 1.0$</td></tr>
<tr><td>71</td><td>$\frac{t_o}{t_i} \geqslant 2.0$</td><td rowspan="2"></td></tr>
<tr><td>36</td><td>$\frac{t_o}{t_i} = 1.0$</td></tr>
<tr><td>71</td><td>$\frac{t_o}{t_i} \geqslant 1.4$</td><td rowspan="2"></td><td rowspan="2"><u>Joints with overlap</u> (*)
③ K-joint.</td><td rowspan="2">③ and ④
— Overlap between 30 % and 100 %.

$\frac{q}{p} \cdot 100$: overlap
e: negative eccentricity</td></tr>
<tr><td>56</td><td>$\frac{t_o}{t_i} = 1.0$</td></tr>
<tr><td colspan="5">— Note that the braces and the chords require separate fatigue assessments.
— (*) For intermediate t_o/t_i values, use linear interpolation between nearest Detail Categories.</td></tr>
</table>

Table 9.8.7 — Lattice girder joints — sheet 2 of 2 (m = 5)

Detail category	Constructional details		Description	Requirement
71	$\frac{t_o}{t_i} \geq 1.4$		<u>Joints with overlap</u> (*) ④ N-joints.	① to ④ — t_o, $t_i \leqslant 12.5$ mm — $35^o \leqslant \theta \leqslant 50^o$ — $b_o/t_o \leqslant 25$ — $d_o/t_o \leqslant 25$ — $0.4 \leqslant b_i/b_o \leqslant 1.0$ — $0.25 \leqslant d_i/d_o \leqslant 1.0$ — $b_o \leqslant 200$ mm — $d_o \leqslant 300$ mm — $-0.5\ h_o \leqslant e \leqslant 0.25\ h_o$ — $0.5\ d_o \leqslant e \leqslant 0.25\ d_o$ — Out of plane eccentricity: $\leqslant 0.02\ b_o$ or $\leqslant 0.02\ d_o$ — Fillet welds are permitted in braces with wall thicknesses $\leqslant 8$ mm — For wall thicknesses greater than 12.5 mm see clause **9.6.3**.
50	$\frac{t_o}{t_i} = 1.0$			

— Note that the braces and the chords require separate fatigue assessments.
— (*) For intermediate t_o/t_i values, use linear interpolation between nearest Detail Categories.

Annex B (normative)
Reference standards

B.1 Scope

(1) This Part 1.1 of Eurocode 3 mentions 10 Reference Standards. They define the product standards and execution standards which apply to steel structures designed in accordance with Eurocode 3-1.1.

B.2 Definitions

B.2.1 *Reference Standard 1: "Weldable structural steel"*

(1) European Standard EN 10025 "*Hot rolled products of non-alloy structural steels — Technical delivery conditions*" grades Fe 360, Fe 430 and Fe 510 only.

(2) European Standard prEN 10113 "*Hot rolled products in weldable fine grain structural steels*" grades Fe E 275 and Fe E 355 only.

(3) For prEN 10113 grades Fe E 420 and Fe E 460 refer to Annex D[24)].

(4) European Standard prEN 10210-1 "*Hot finished steel hollow sections: Part 1 Technical delivery requirements*"[24)].

(5) European Standard prEN 10219-1 "*Cold formed structural steel hollow sections: Part 1 Non-alloy and fine grain steels*"[24)].

(6) It shall be ensured that the weldability of the material is sufficient for the purpose for which it is required.

(7) For cold formed thin gauge members and sheeting refer to prENV 1993-1-3 Eurocode 3-1.3[24)].

B.2.2 *Reference Standard 2: "Dimensions of sections and plates"*

B.2.2.1 *Hot rolled sections, other than structural hollow sections*

(1) The Euronorms for sections listed in European Standard EN 10025 modified as follows:

- excluding tolerances
- including the "corresponding national standards" for hot rolled sections listed in Annex B of EN 10025 (but excluding tolerances).

(2) European Standard EN...... "*Hot rolled tapered flange and parallel flange channels — dimensions and tolerances*" (when available).

(3) European Standard EN...... "*Hot rolled tees — Dimensions and tolerances*" (when available)

(4) European Standard EN...... "*Hot rolled bulb flats — Dimensions and tolerances*" (when available).

(5) European Standard EN...... "*Hot rolled I and H sections — Dimensions*" (when available).

(6) European Standard EN...... "*Hot rolled split tees — Dimensions and tolerances*" (when available).

(7) European Standard EN...... "*Hot rolled equal leg and unequal leg angles — Dimensions*" (when available).

(8) International Standard ISO 657 "*Hot rolled steel sections*": Part 1 "*Equal leg angles*" and Part 2 "*Unequal leg angles*".

(9) European Standard EN...... "*Hot rolled flat, square and round steel bars — Dimensions*" (when available).

(10) European Standard EN...... "*Hot rolled square steel bars — Dimensions*" (when available).

(11) European Standard EN...... "*Hot rolled round steel bars — Dimensions*" (when available).

B.2.2.2 *Hot rolled structural hollow sections*

(1) European Standard prEN 10210-2 "*Hot finished steel hollow sections: Part 2 Dimensions and tolerances*"[24)].

(2) International Standard ISO 657 "*Hot rolled steel sections*": Part 14 "*Hot finished structural hollow sections, dimensional and sectional properties*", as follows:

- except that steel is to be to EN 10025

[24)] In preparation

B.2.2.3 *Cold finished structural hollow sections*

(1) European Standard prEN 10219-2 "*Cold formed structural steel hollow sections: Part 2 Dimensions and tolerances*"[25)].

(2) International Standard ISO 4019 "*Cold finished steel structural hollow sections — Dimensions and sectional properties*".

B.2.2.4 *Cold formed sections, other than structural hollow sections*

(1) For cold formed thin gauge members and sheeting refer to prENV 1993-1-3 Eurocode 3-1.3[25)].

B.2.3 ***Tolerances***

B.2.3.1 *Hot rolled sections, other than structural hollow sections*

(1) European Standard prEN 10034 "*Structural steel I and H sections — Tolerances on shape and dimensions*"[25)].

(2) European Standard prEN 10056 "*Structural steel equal leg and unequal leg angles — Tolerances on shape and dimensions*"[25)].

(3) European Standard EN ...... "*Hot rolled tapered flange and parallel flange channels — Dimensions and tolerances*" (when available).

(4) European Standard EN ...... "*Hot rolled tees — Dimensions and tolerances*" (when available).

(5) European Standard EN ...... "*Hot rolled bulb flats — Dimensions and tolerances*" (when available).

(6) European Standard EN ...... "*Hot rolled split tees — Dimensions and tolerances*" (when available).

(7) European Standard EN ...... "*Hot rolled square steel bars — Tolerances*" (when available).

(8) European Standard EN ...... "*Hot rolled round steel bars — Tolerances*" (when available).

B.2.3.2 *Structural hollow sections*

(1) European Standard prEN 10210-2 "*Hot finished steel hollow sections — Part 2 Dimensions and tolerances*"[25)].

(2) European Standard prEN 10219-2 "*Cold formed structural steel hollow sections — Part 2 Dimensions and tolerances*"[25)].

B.2.3.3 *Cold formed sections, other than structural hollow sections*

(1) For cold formed thin gauge members and sheeting refer to prENV 1993-1-3 Eurocode 3-1.3[25)].

B.2.3.4 *Plates and flats*

(1) European Standard EN 10029 "*Tolerances on dimensions, shape and mass for hot rolled steel plates 3 mm thick or above*" as follows:

- Class A tolerances

(2) European Standard EN..... "*Tolerance requirements for wide flats*" (when available).

(3) European Standard EN..... "*Tolerance requirements for flat bars*" (when available).

B.2.4 ***Reference Standard 3: "Bolts, nuts and washers"***

B.2.4.1 *Non-preloaded bolts*

(1) Bolts to European Standards EN 24014, EN 24016, EN 24017 or EN 24018, nuts to EN 24032, EN 24034 or ISO 7413, washers to ISO 7089, ISO 7090 or ISO 7091.

(2) Bolts to International Standard ISO 7411, nuts to ISO 4775, washers to ISO 7415 or ISO 7416.

(3) Bolts to International Standard ISO 7412, nuts to ISO 7414, washers to ISO 7415 or ISO 7416.

B.2.4.2 *Preloaded bolts*

(1) Bolts to International Standard ISO 7411, nuts to ISO 4775, washers to ISO 7415 or ISO 7416.

B.2.5 ***Reference Standard 4: "Welding Consumables"***

(1) European Standard EN ..... "*Welding consumables*" (when available).

B.2.6 ***Reference Standard 5: "Rivets"***

(1) European Standard EN ..... "*Structural steel rivets*" (when available).

[25)] In preparation

B.2.7 ***Reference Standards 6 to 9: "Execution standards"***

(1) European Standard EN ..... "*Execution of steel structures*" Part 1 "*General rules and rules for buildings*"[26)].

B.2.8 ***Reference Standard 10: "Corrosion protection"***

(1) European Standard EN ..... "*Corrosion protection*" (when available).

Annex C (informative)
Design against brittle fracture

C.1 Resistance to brittle fracture

(1) Brittle fracture is characterized as failure of a structural element without plastic deformation. The failure mode is mainly dependent on the following:

- steel strength grade
- thickness of the material
- loading speed
- lowest service temperature
- material toughness
- type of structural element

(2) The required steel grade can be determined by considering the factors listed above. The choice depends on the toughness of the material and the requirements in terms of fracture mechanics. The criterion is expressed in terms of the test temperature at which the Charpy V-notch energy has a minimum value of 27 Joules.

(3) The procedure which follows determines the lowest service temperature for a given grade and thickness of steel, depending on the service conditions, the loading rate and the consequences of failure.

(4) The steel grades in this procedure refer to material conforming with EN 10025 or prEN 10113.

(5) *The procedure given in this Annex should not be applied for service temperatures below – 40 °C.*

C.2 Calculation procedure

C.2.1 ***Service Conditions***

(1) Three levels of severity are defined, with stress levels calculated using the characteristic values of the actions and a partial safety factor $\gamma_F = 1,0$ as follows:

- S1: Either:
  - no welding, or
  - as-welded, with local tensile stresses not exceeding 0,2 times the yield strength, or
  - full stress-relief post-weld heat treatment, with local tensile stresses (including any geometric stress concentration) not exceeding 0,67 times the yield strength.
- S2: As-welded condition, with either:
  - local tensile stresses in the range 0,2 to 0,67 times the yield strength, or
  - post-weld heat treated stress concentrations with local stresses up to twice the yield strength.
- S3: Complex geometry stress concentration regions, either:
  - as-welded with local tensile stresses in the range 0,67 to 2 times the yield strength, or
  - post-weld heat treated with local stresses in the range of 2 to 3 times the yield strength,

  but in all cases below plastic collapse.

(2) Table 3.2 is based on stress levels S1 and S2.

C.2.2 ***Loading rate***

(1) Two loading rates are defined as follows:

- R1: Normal static or slow loading, applicable to structures subjected to self weight, floor loading, vehicular loading, wind and wave loading and lifting loads.

[26)] In preparation

• R2: Impact loading, applicable to high strain rate, explosive or crash conditions.

(2) Table 3.2 is based on loading rate R1.

C.2.3 *Consequences of failure*

(1) Two conditions are defined as follows:

• C1: Non-critical members or joints, where failure would be restricted to local effects without serious consequences (eg. redundant members).

• C2: Fracture critical members or joints, where local failure would cause complete structural collapse with serious consequences to life or very high cost.

(2) Table 3.2 is based on condition C2.

C.2.4 *Nominal yield strength*

(1) The nominal value of the lower yield strength $f_{y\tau}$, reduces with thickness and may be obtained from:

$$f_{y\tau} = f_{yo} - 0{,}25\ (t/t_1)\ (f_{yo}/235) \qquad \text{(C.1)}$$

where f_{yo} is the base value of $f_{y\tau}$ (in N/mm^2)

t is the thickness (in mm)

and t_1 = 1 mm

(2) The base value of the mean lower yield strength f_{yo} (for use in Annex C only) may be determined from Table C.1.

Table C.1 — Base value of the mean lower yield strength

Grade of steel	Fe 360	Fe 430	Fe 510
f_{yo} (N/mm^2)	235	275	355

C.2.5 *Parameters*

(1) The values of the constants to be adopted for categories S, R and C shall be obtained from Table C.2.

Table C.2 — Values of constants

Stress category	**S1**	**S2**	**S3**
k_a k_b k_c	0,18 0,40 0,03	0,18 0,15 0,03	0,10 0,07 0,04
Loading rate	R1	R2	
Value of k_d	10^{-3}	1,0	
Consequences of failure	C1	C2	
Value of γ_c	$\boxed{1{,}0}$	$\boxed{1{,}5}$	

(2) Details of the Charpy V-notch test temperature T_{cv} for standard notch-toughness grades of steel to EN 10025 are given in Table C.3.

(3) Relevant details of the Charpy V-notch test temperature T_{cv} for steel to prEN 10113 are also given in Table C.3.

Table C.3 — Charpy V-notch test temperature T_{cv}

Notch toughness quality	Specified values			Nominal value of T_{cv} (°C) that may be assumed to give 27 Joules for thickness t (mm)	
	Test temperature (°C)	Minimum energy (J) for thickness t (mm)			
		> 10 ≤ 150[1)]	> 150 ≤ 250[1)]	> 150[1)]	≤ 150 ≤ 250[1)]
EN 10025					
B	+ 20	27	23	+ 20	+ 25
C	0	27	23	+ 0	+ 5
D	– 20	27	23	– 20	– 15
DD	– 20	40	33	– 30[2)]	– 25[2)]
prEN 10113					
KG	– 20	40	33	– 30[2)]	– 25[2)]
KT	– 50	27	23	– 50	– 45

Notes:
1. The value shall be agreed with the steel producer for rolled sections to EN 10025 with a nominal thickness over 100 mm, for steels of delivery condition N to prEN 10113-2 with a nominal thickness over 150 mm and for steels of delivery condition TM to prEN 10113-3 with a nominal thickness over 150 mm for long products or over 63 mm for flat products.
2. These values are assumed to be equivalent to a Charpy V-notch energy of 40 J at – 20 °C, or 33 J at – 20 °C for steel over 150 mm up to 250 mm thick.

C.2.6 *Calculations*

(1) The required fracture toughness K_{1C} shall be obtained from:

$$K_{1C} = [\gamma_c \alpha]^{0,55}\, f_y\, \ell\, t^{0,5}/1,226 \tag{C.2}$$

in which:

$$\alpha = 1/[k_a + k_b\, \ell n(t/t_1) + k_c\, (t/t_1)^{0,5}]$$

(2) The minimum service temperature T_{min} shall be obtained from:

$$T_{min} = 1,4\, T_{cv} + 25 + \beta + (83 - 0,08\, f_{y\,\ell})\, [k_d]^{0,17} \tag{C.3}$$

in which:

$$\beta = 100\, (\ell n\, K_{1C} - 8,06)$$

Annex E (informative) Buckling length of a compression member

E.1 Basis

(1) The buckling length ℓ of a compression member is the length of an otherwise similar member with "pinned ends" (ends restrained against lateral movement but free to rotate in the plane of buckling) which has the same buckling resistance.

(2) In the absence of better information, the theoretical buckling length for elastic critical buckling may conservatively be adopted.

(3) An equivalent buckling length may be used to relate the buckling resistance of a member subject to non-uniform loading to that of an otherwise similar member subject to uniform loading.

(4) An equivalent buckling length may also be used to relate the buckling resistance of a non-uniform member to that of a uniform member under similar conditions of loading and restraint.

E.2 Columns in building frames

(1) The buckling length ℓ of a column in a non-sway mode may be obtained from Figure E.2.1.

(2) The buckling length ℓ of a column in a sway mode may be obtained from Figure E.2.2.

(3) For the theoretical models shown in Figure E.2.3 the distribution factors η_1 and η_2 are obtained from:

$$\eta_1 = K_c/(K_c + K_{11} + K_{12}) \tag{E.1}$$

$$\eta_2 = K_c/(K_c + K_{21} + K_{22}) \tag{E.2}$$

where K_c is the column stiffness coefficient l/L and K_{ij} is the effective beam stiffness coefficient

(4) These models may be adapted to the design of continuous column, by assuming that each length of column is loaded to the same value of the ratio (N/N_{cr}). In the general case where (N/N_{cr}) varies, this leads to a conservative value of ℓ/L for the most critical length of column.

(5) For each length of a continuous column the assumption made in (4) may be introduced by using the model shown in Figure E.2.4 and obtaining the distribution factors η_1 and η_2 from:

$$\eta_1 = \frac{K_c + K_1}{K_c + K_1 + K_{11} + K_{12}} \tag{E.3}$$

$$\eta_2 = \frac{K_c + K_2}{K_c + K_2 + K_{21} + K_{22}} \tag{E.4}$$

where K_1 and K_2 are the stiffness coefficients for the adjacent lengths of column.

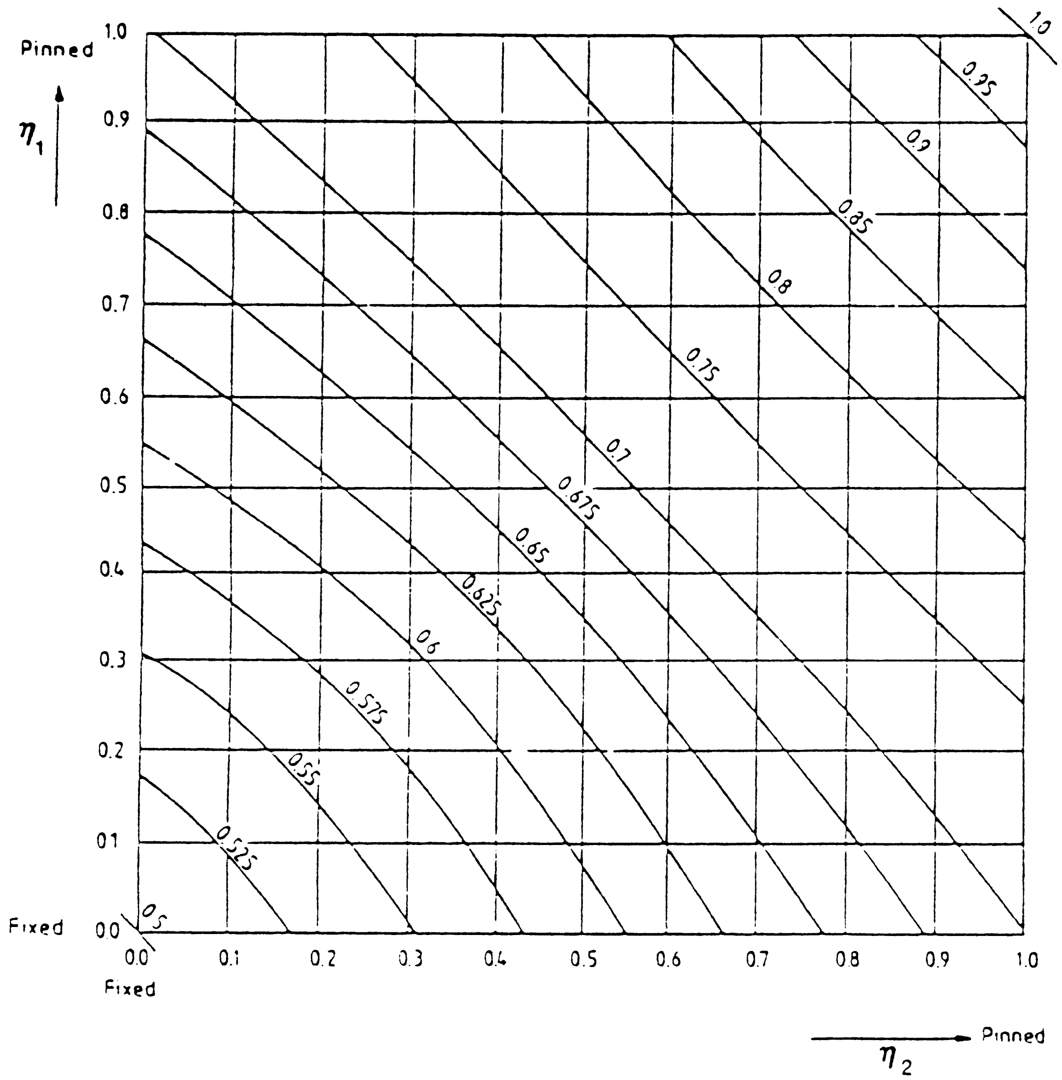


Figure E.2.1 — Buckling length ratio ℓ/L for a column in a non-sway mode

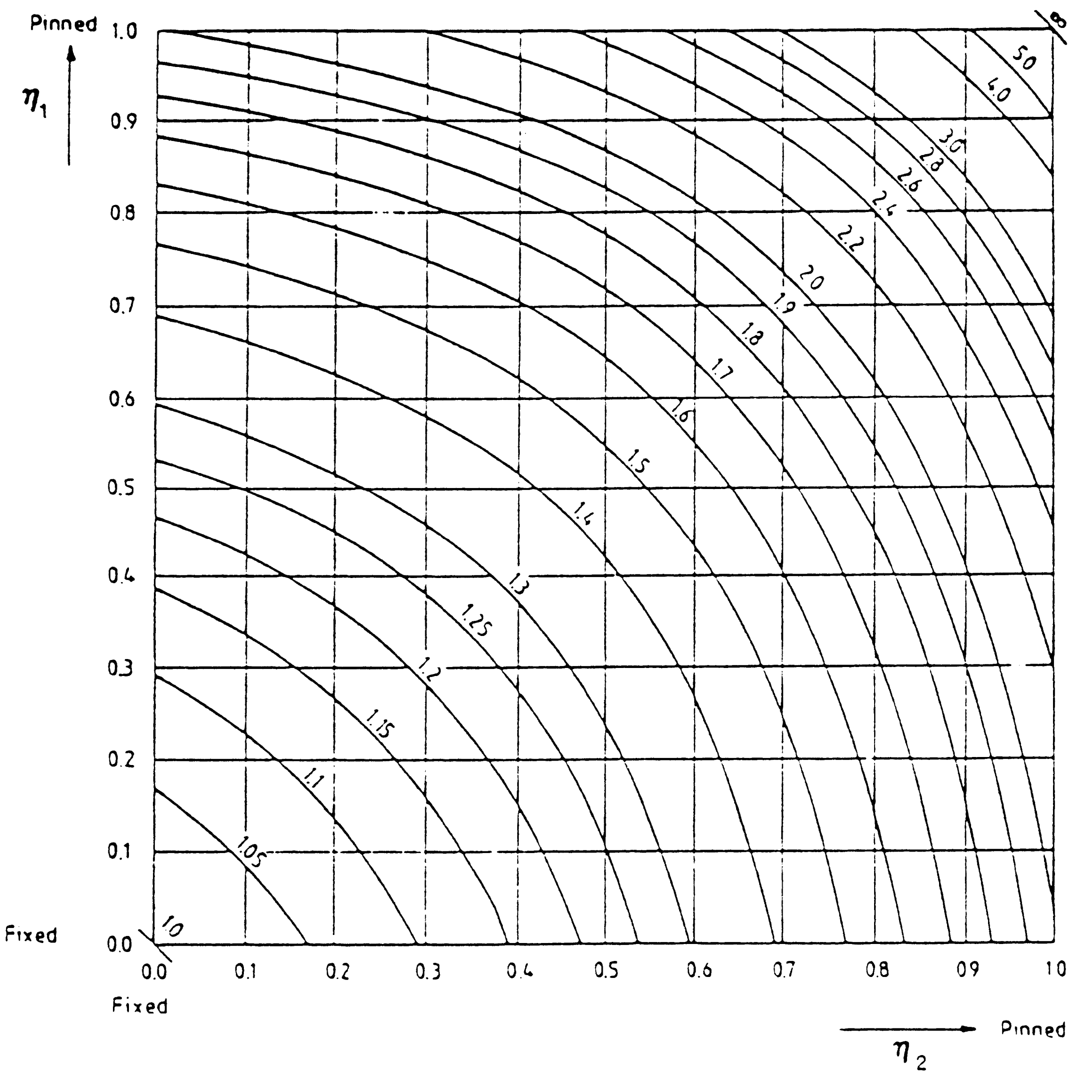


Figure E.2.2 — Buckling length ratio ℓ/L for a column in a sway mode

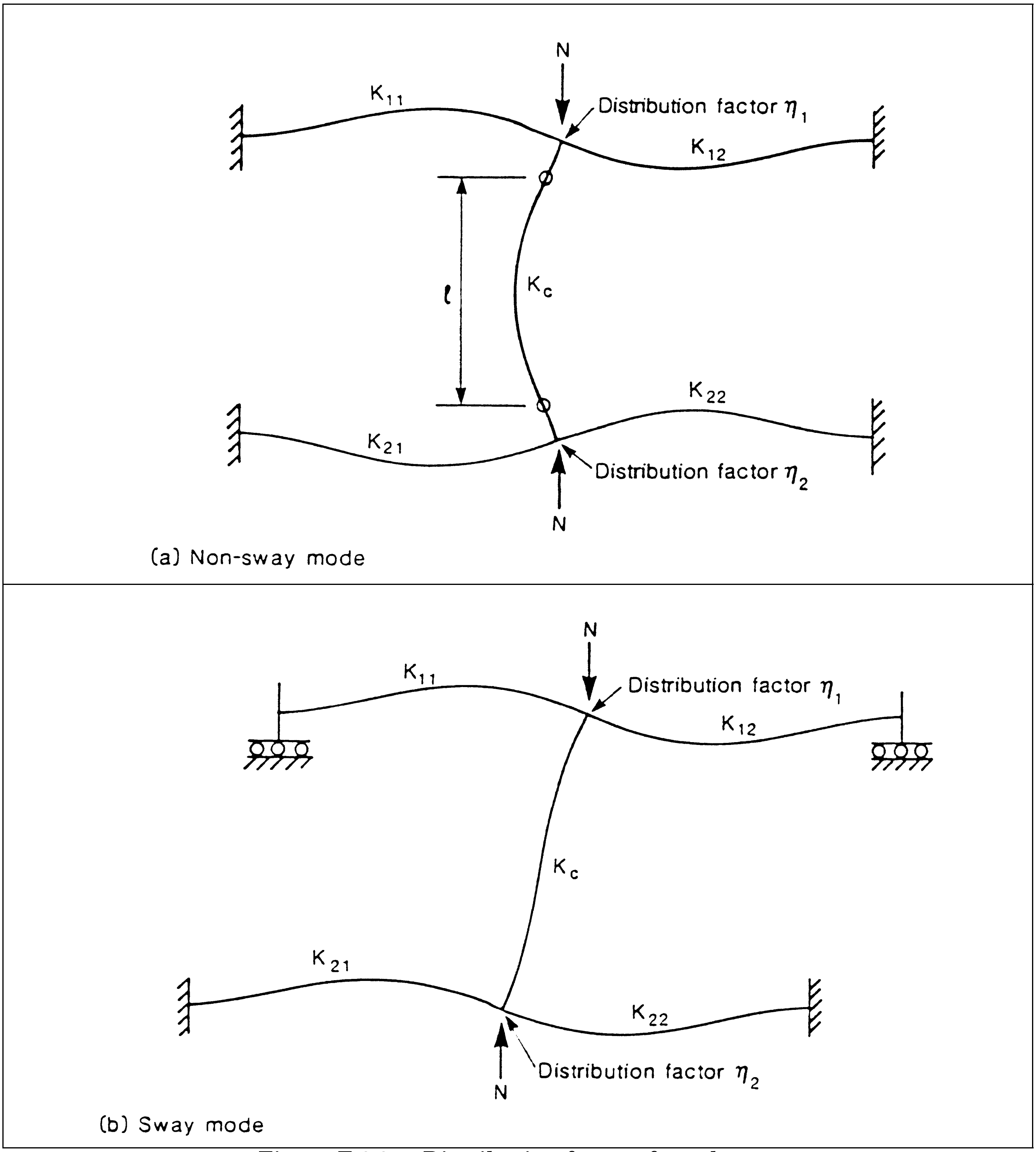


Figure E.2.3 — Distribution factors for columns

(6) Where the beams are not subject to axial forces, their effective stiffness coefficients may be determined by reference to Table E.1, provided that they remain elastic under the design moments.

Table E.1 — Effective stiffness coefficient for a beam

Conditions of rotational restraint at far end of beam	Effective beam stiffness coefficient K (provided that beam remains elastic)
Fixed at far end	1,0 I/L
Pinned at far end	0,75 I/L
Rotation as at near end (double curvature)	1,5 I/L
Rotation equal and opposite to that at near end (single curvature)	0,5 I/L
General case. Rotation θ_a at near end and θ_b at far end	$(1 + 0,5\ \theta_b/\theta_a)$ I/L

(7) *For building frames with concrete floor slabs, provided that the frame is of regular layout and the loading is uniform, it is normally sufficiently accurate to assume that the effective stiffness coefficients of the beams are as shown in Table E.2.*

Table E.2 — Effective stiffness coefficient for a beam in a building frame with concrete floor slabs

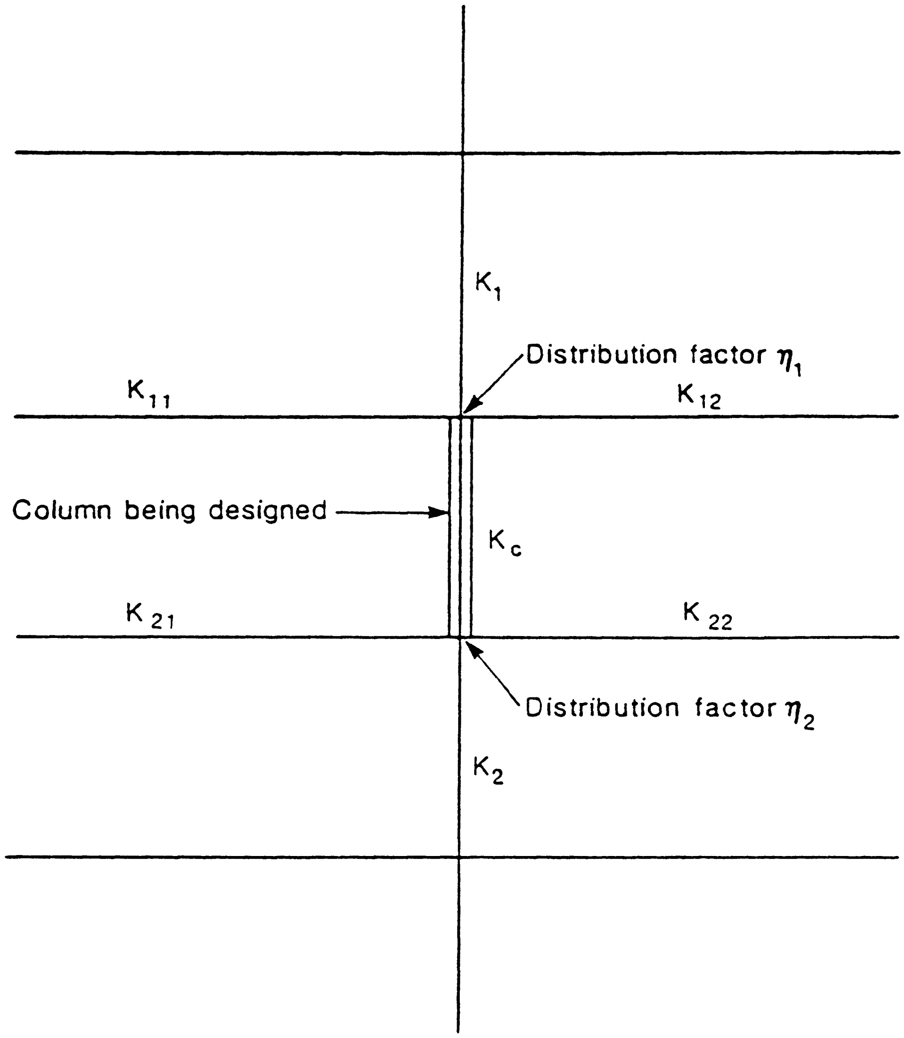
Loading conditions for the beam	Non-sway mode	Sway mode
Beams directly supporting concrete floor slabs	1,0 I/L	1,0 I/L
Other beams with direct loads	0,75 I/L	1,0 I/L
Beams with end moments only	0,5 I/L	1,5 I/L

(8) *Where, for the same load case, the design moment in any of the beams exceeds $W_{el} f_y/\gamma_{M0}$, the beam should be assumed to be pinned at the point or points concerned.*

(9) *Where a beam has nominally pinned connections, it should be assumed to be pinned at the point or points concerned.*

(10) *Where a beam has semi-rigid connections, its effective stiffness coefficient should be reduced accordingly.*

(11) *Where the beams are subject to axial forces, their effective stiffness coefficients should be adjusted accordingly. Stability functions may be used. As a simple alternative, the increased stiffness coefficient due to axial tension may be neglected and the effects of axial compression may be allowed for by using the conservative approximations given in Table E.3.*



$$\eta_1 = \frac{K_c + K_1}{K_c + K_1 + K_{11} + K_{12}}$$

$$\eta_2 = \frac{K_c + K_2}{K_c + K_2 + K_{21} + K_{22}}$$

Figure E.2.4 — Distribution factors for continuous columns

Table E.3 — Approximate formulae for reduced beam stiffness coefficients due to axial compression

Conditions of rotational restraint at far end of beam	Effective beam stiffness coefficient K (provided that beam remains elastic)
Fixed	1,0 I/L $(1 - 0{,}4\ N/N_E)$
Pinned	0,75 I/L $(1 - 1{,}0\ N/N_E)$
Rotation as at near end (double curvature)	1,5 I/L $(1 - 0{,}2\ N/N_E)$
Rotation equal and opposite to that at near end (single curvature)	0,5 I/L $(1 - 1{,}0\ N/N_E)$
In this table $N_E = \pi^2\ EI/L^2$	

(12) *The following empirical expressions may be used as conservative approximations instead of reading values from Figure E.2.1 and Figure E.2.2:*

a) *non-sway mode (Figure E.2.1)*

$$\ell/L = 0{,}5 + 0{,}14\ (\eta_1 + \eta_2) + 0{,}055\ (\eta_1 + \eta_2)^2 \qquad (E.5)$$

or alternatively:

$$\ell/L = \left[\frac{1 + 0{,}145\ (\eta_1 + \eta_2) - 0{,}265\ \eta_1\ \eta_2}{2 - 0{,}364\ (\eta_1 + \eta_2) - 0{,}247\ \eta_1\ \eta_2}\right] \qquad (E.6)$$

b) *sway mode (Figure E.2.2)*

$$\ell/L = \left[\frac{1 - 0{,}2\ (\eta_1 + \eta_2) - 0{,}12\ \eta_1\ \eta_2}{1 - 0{,}8\ (\eta_1 + \eta_2) + 0{,}6\ \eta_1\ \eta_2}\right]^{0,5} \qquad (E.7)$$

Annex F (informative) Lateral torsional buckling

F.1 Elastic critical moment

F.1.1 *Basis*

(1) The elastic critical moment for lateral-torsional buckling of a beam of uniform symmetrical cross-section with equal flanges, under standard conditions of restraint at each end, loaded through its shear centre and subject to uniform moment is given by:

$$M_{cr} = \frac{\pi^2 EI_z}{L^2}\left[\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z}\right]^{0,5} \qquad (F.1)$$

where $G = \dfrac{E}{2(1+\upsilon)}$

I_t is the torsion constant

I_w is the warping constant

I_z is the second moment of area about the minor axis

and L is the length of the beam between points which have lateral restraint.

(2) The standard conditions of restraint at each end are:

- restrained against lateral movement
- restrained against rotation about the longitudinal axis
- free to rotate in plan

F.1.2 ***General formula for cross-sections symmetrical about the minor axis***

(1) In the case of a beam of uniform cross-section which is symmetrical about the minor axis, for bending about the major axis the elastic critical moment for lateral-torsional buckling is given by the general formula:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \left\{ \left[\left(\frac{k}{k_w} \right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} + [C_2 z_g - C_3 z_j]^2 \right]^{0,5} - [C_2 z_g - C_3 z_j] \right\} \quad \text{(F.2)}$$

where C_1, C_2 and C_3 are factors depending on the loading and end restraint conditions

k and k_w are effective length factors

$z_g = z_a - z_s$

$z_j = z_s - 0,5 \int_A (y^2 + z^2)\, z\, dA/I_y$

z_a is the coordinate of the point of load application

z_s is the coordinate of the shear centre

NOTE See **F.1.2**(7) and (8) for sign conventions and **F.1.4**(2) for approximations for z_j

(2) The effective length factors k and k_w vary from 0,5 for full fixity to 1,0 for no fixity, with 0,7 for one end fixed and one end free.

(3) *The factor k refers to end rotation on plan. It is analogous to the ratio ℓ/L for a compression member.*

(4)*The factor k_w refers to end warping. Unless special provision for warping fixity is made, k_w should be taken as 1,0.*

(5) Values of C_1, C_2 and C_3 are given in Table F.1.1 and Table F.1.2 for various load cases, as indicated by the shape of the bending moment diagram over the length L between lateral restraints. Values are given corresponding to various values of k.

(6) *For cases with k = 1,0 the value of C_1 for any ratio of end moment loading as indicated in Table F.1.1, is given approximately by:*

$$C_1 = 1,88 - 1,40\,\psi + 0,52\psi^2 \text{ but } C_1 \leq 2,70 \quad \text{(F.3)}$$

(7) The sign convention for determining z_j, see Figure F.1.1, is:

- z is positive for the compression flange
- z_j is positive when the flange with the larger value of I_z is in compression at the point of largest moment.

(8) The sign convention for determining z_g is:

- for gravity loads z_g is positive for loads applied above the shear centre
- in the general case z_g is positive for loads acting towards the shear centre from their point of application.

F.1.3 ***Beams with uniform doubly symmetric cross-sections***

(1) For doubly symmetric cross-sections $z_j = 0$, thus:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \left\{ \left[\left(\frac{k}{k_w} \right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} + [C_2 z_g]^2 \right]^{0,5} - C_2 z_g \right\} \quad \text{(F.4)}$$

(2) For end-moment loading $C_2 = 0$ and for transverse loads applied at the shear centre $z_g = 0$. For these cases:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \left[\left(\frac{k}{k_w} \right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} \right]^{0,5} \quad \text{(F.5)}$$

(3) When $k = k_w = 1{,}0$ (no end fixity):

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{L^2} \left[\frac{I_w}{I_z} + \frac{L^2 G I_t}{\pi^2 E I_z} \right]^{0,5} \tag{F.6}$$

Table F.1.1 — Values of factors C_1, C_2 and C_3 corresponding to values of factor k: End moment loading

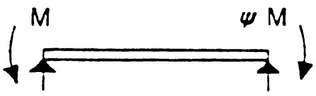
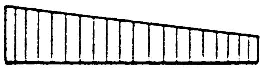
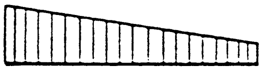
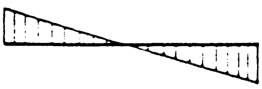
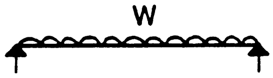
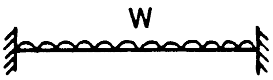
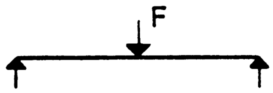
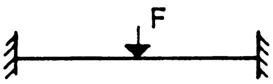
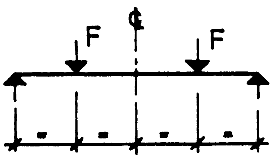
Loading and support conditions	Bending moment diagram	Value of k	Values of factors		
			C_1	C_2	C_3
	$\psi = +1$ 	1,0 0,7 0,5	1,000 1,000 1,000	—	1,000 1,113 1,144
	$\psi = +3/4$ 	1,0 0,7 0,5	1,141 1,270 1,305	—	0,998 1,565 2,283
	$\psi = +1/2$ 	1,0 0,7 0,5	1,323 1,473 1,514	—	0,992 1,556 2,271
	$\psi = +1/4$ 	1,0 0,7 0,5	1,563 1,739 1,788	—	0,977 1,531 2,235
	$\psi = 0$ 	1,0 0,7 0,5	1,879 2,092 2,150	—	0,939 1,473 2,150
	$\psi = -1/4$ 	1,0 0,7 0,5	2,281 2,538 2,609	—	0,855 1,340 1,957
	$\psi = -1/2$ 	1,0 0,7 0,5	2,704 3,009 3,093	—	0,676 1,059 1,546
	$\psi = -3/4$ 	1,0 0,7 0,5	2,927 3,009 3,093	—	0,366 0,575 0,837
	$\psi = -1$ 	1,0 0,7 0,5	2,752 3,063 3,149	—	0,000 0,000 0,000

Table F.1.2 — Values of factors C_1, C_2 and C_3 corresponding to values of factor k: Transverse loading cases

Loading and support conditions	Bending moment diagram	Values of k	Values of factors		
			C_1	C_2	C_3
		1,0 0,5	1,132 0,972	0,459 0,304	0,525 0,980
		1,0 0,5	1,285 0,712	1,562 0,652	0,753 1,070
		1,0 0,5	1,365 1,070	0,553 0,432	1,730 3,050
		1,0 0,5	1,565 0,938	1,267 0,715	2,640 4,800
		1,0 0,5	1,046 1,010	0,430 0,410	1,120 1,890

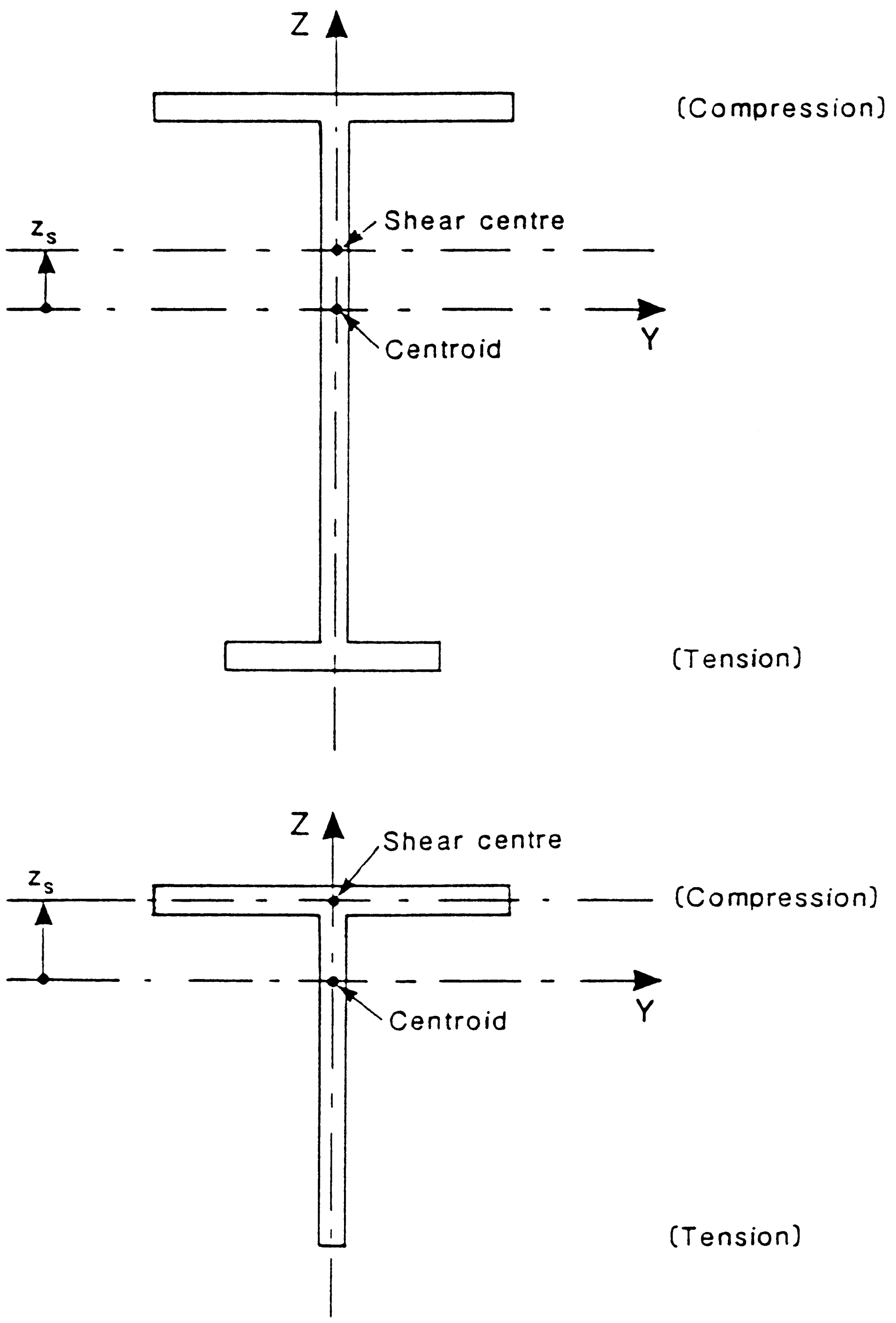


Figure F.1.1 — Sign convention for determining z_j

F.1.4 *Beams with uniform monosymmetric cross-sections with unequal flanges*

(1) For an I-section with unequal flanges:

$$I_w = \beta_f (1 - \beta_f)\, I_z\, h_s^2 \qquad (F.7)$$

where $\beta_f = \dfrac{I_{fc}}{I_{fc} + I_{ft}}$

I_{fc} is the second moment of area of the compression flange about the minor axis of the section

I_{ft} is the second moment of area of the tension flange about the minor axis of the section

and h_s is the distance between the shear centres of the flanges.

(2) *The following approximations for z_j can be used:*

when $\beta_f > 0{,}5$:

$$z_j = 0{,}8\,(2\beta_f\ \Pi\ 1)\, h_s/2 \qquad (F.8)$$

when $\beta_f < 0{,}5$:

$$z_j = 1{,}0\,(2\beta_f\ \Pi\ 1)\, h_s/2 \qquad (F.9)$$

for sections with a lipped compression flange:

$$z_j = 0{,}8\,(2\beta_f\ \Pi\ 1)\,(1 + h_L/h)\, h_s/2 \text{ when } \beta_f > 0{,}5 \qquad (F.10)$$

$$z_j = 1{,}0\,(2\beta_f\ \Pi\ 1)\,(1 + h_L/h)\, h_s/2 \text{ when } \beta_f < 0{,}5 \qquad (F.11)$$

where h_L *is the depth of the lip*

F.2 Slenderness

F.2.1 *General*

(1) The slenderness ratio $\bar{\lambda}_{LT}$ for lateral-torsional buckling is given by:

$$\bar{\lambda}_{LT} = [\lambda_{LT} / \lambda_1]\, [\beta_w]^{0,5} \qquad (F.12)$$

where $\lambda_1 = \pi\ [E/f_y]^{0.5} = 93{,}9\,\varepsilon$

$\varepsilon = [235/f_y]^{0,5}$ (f_y in N/mm^2)

$\beta_w = 1$ for Class 1 or Class 2 cross-sections

$\beta_w = W_{e\ell.y}/W_{p\ell.y}$ for Class 3 cross-sections

and $\beta_w = W_{eff.y}/W_{p\ell.y}$ for Class 4 cross sections

(2) The geometrical slenderness ratio λ_{LT} for lateral-torsional buckling is given for all classes of cross-section, by:

$$\lambda_{LT} = [\pi^2\, EW_{p\ell.y}/M_{cr}]^{0,5} \qquad (F.13)$$

F.2.2 *Beams with uniform doubly symmetric cross-sections*

(1) For cases with $z_g = 0$ (end-moment loading or transverse loads applied at the shear centre) and $k = kw = 1{,}0$ (no end fixity), the value of λ_{LT} can be obtained from:

$$\lambda_{LT} = \frac{L\left[\dfrac{W_{p\ell.y}^2}{I_z\, I_w}\right]^{0,25}}{(C_1)^{0,5}\left[1 + \dfrac{L^2 G I_t}{\pi^2 E I_w}\right]^{0,25}} \qquad (F.14)$$

which can also be written:

$$\lambda_{LT} = \frac{L/i_{LT}}{(C_1)^{0,5}\left[1 + \dfrac{(L/a_{LT})^2}{25{,}66}\right]^{0,25}} \qquad (F.15)$$

where $a_{LT} = (I_w/I_t)^{0.5}$

(2) For a plain I or H section (without lips):

$$I_w = I_z\, h_s^2/4 \tag{F.16}$$

where $h_s = h - t_f$

(3) *For a doubly symmetric cross-section, the value of i_{LT} is given by:*

$$i_{LT} = [I_z\, I_w / W_{p\ell.y}^2]^{0,25} \tag{F.17}$$

or with a slight approximation by:

$$i_{LT} = [I_z/(A - 0,5\, t_w\, h_s)]^{0,5} \tag{F.18}$$

(4) *For rolled I or H sections conforming with Reference Standard 2, the following conservative approximations can be used:*

$$\lambda_{LT} = \frac{L/i_{LT}}{(C_1)^{0,5}\left[1 + \frac{1}{20}\left[\frac{L/i_{LT}}{h/t_f}\right]^2\right]^{0,25}} \tag{F.19}$$

or

$$\lambda_{LT} = \frac{0,9\; L/i_z}{(C_1)^{0,5}\left[1 + \frac{1}{20}\left[\frac{L/i_z}{h/t_f}\right]^2\right]^{0,25}} \tag{F.20}$$

(5) *For any plain I or H section with equal flanges, the following approximation is conservative:*

$$\lambda_{LT} = \frac{L/i_z}{(C_1)^{0,5}\left[1 + \frac{1}{20}\left[\frac{L/i_z}{h/t_f}\right]^2\right]^{0,25}} \tag{F.21}$$

(6) *Cases with $k < 1,0$ and/or $k_w < 1,0$ can be included by using:*

$$\lambda_{LT} = \frac{kL\left[\frac{W_{p\ell.y}^2}{I_z\, I_w}\right]^{0,25}}{(C_1)^{0,5}\left[\left[\frac{k}{k_w}\right]^2 + \frac{(kL)^2\, G\, I_t}{\pi^2\, E\, I_w}\right]^{0,25}} \tag{F.22}$$

or

$$\lambda_{LT} = \frac{kL/i_{LT}}{(C_1)^{0,5}\left[\left[\frac{k}{k_w}\right]^2 + \frac{(kL/a_{LT})^2}{25,66}\right]^{0,25}} \tag{F.23}$$

or for standard rolled I or H sections:

$$\lambda_{LT} = \frac{kL/i_{LT}}{(C_1)^{0,5}\left[\left[\frac{k}{k_w}\right]^2 + \frac{1}{20}\left[\frac{kL/i_{LT}}{h/t_f}\right]^2\right]^{0,25}} \tag{F.24}$$

or $$\lambda_{LT} = \frac{0,9\ kL/i_z}{(C_1)^{0,5}\left[\left[\frac{k}{k_w}\right]^2 + \frac{1}{20}\left[\frac{kL/i_z}{h/t_f}\right]^2\right]^{0,25}} \tag{F.25}$$

or for any plain I or H section with equal flanges:

$$\lambda_{LT} = \frac{kL/i_z}{(C_1)^{0,5}\left[\left[\frac{k}{k_w}\right]^2 + \frac{1}{20}\left[\frac{kL/i_z}{h/t_f}\right]^2\right]^{0,25}} \tag{F.26}$$

(7) *Unless special provision for warping fixity is made, k_w should be taken as 1,0.*

(8) *Cases with transverse loading applied above the shear centre (z_g > 0,0) or below the shear centre (z_g < 0,0) can be included by using:*

$$\lambda_{LT} = \frac{kL\left[\frac{W_{pl.y}^2}{I_z\ I_w}\right]^{0,25}}{(C_1)^{0,5}\left\{\left[\left[\frac{k}{k_w}\right]^2 + \frac{(kL)^2\ GI_t}{\pi^2\ EI_w} + (C_2\ z_g)^2\ \frac{I_z}{I_w}\right]^{0,5} - C_2\ z_g\left[\frac{I_z}{I_w}\right]^{0,5}\right\}^{0,5}} \tag{F.27}$$

or alternatively:

$$\lambda_{LT} = \frac{kL/i_{LT}}{(C_1)^{0,5}\left\{\left[\left[\frac{k}{k_w}\right]^2 + \frac{(kL/a_{LT})^2}{25,66} + \left[\frac{2C_2\ z_g}{h_s}\right]^2\right]^{0,5} - \frac{2C_2\ z_g}{h_s}\right\}^{0,5}} \tag{F.28}$$

or for standard rolled I or H sections:

$$\lambda_{LT} = \frac{kL/i_{LT}}{(C_1)^{0,5}\left\{\left[\left[\frac{k}{k_w}\right]^2 + \frac{1}{20}\left[\frac{kL/i_{LT}}{h/t_f}\right]^2 + \left[\frac{2C_2\ z_g}{h_s}\right]^2\right]^{0,5} - \frac{2C_2\ z_g}{h_s}\right\}^{0,5}} \tag{F.29}$$

or alternatively:

$$\lambda_{LT} = \frac{0,9\ kL/i_z}{(C_1)^{0,5}\left\{\left[\left[\frac{k}{k_w}\right]^2 + \frac{1}{20}\left[\frac{kL/i_z}{h/t_f}\right]^2 + \left[\frac{2C_2\ z_g}{h_s}\right]^2\right]^{0,5} - \frac{2C_2\ z_g}{h_s}\right\}^{0,5}} \tag{F.30}$$

or for any plain I or H section with equal flanges:

$$\lambda_{LT} = \frac{kL/i_z}{(C_1)^{0,5}\left\{\left[\left[\frac{k}{k_w}\right]^2 + \frac{1}{20}\left[\frac{kL/i_z}{h/t_f}\right]^2 + \left[\frac{2C_2\, z_g}{h_s}\right]^2\right]^{0,5} - \frac{2C_2\, z_g}{h_s}\right\}^{0,5}} \qquad \text{(F.31)}$$

Annex J (normative)
Beam-to-column connections

J.1 Scope

J.1.1 *Types of connections covered*

(1) *This Annex gives application rules for the design of beam-to-column connections, following the principles given section* ***6.9****.*

(2) *Both the beam and the column are assumed to be I or H sections.*

(3) *The beam is assumed to be connected to the flange of the column.*

(4) *The types of connections covered are shown in Figure J.1.1 as follows:*

- *Welded connections.*
- *Bolted connections with extended end plates.*
- *Bolted connections with flush end plates.*

(5) *The column web may have:*

- *Stiffeners in line with both flanges of the beam.*
- *Stiffeners in line with one flange of the beam.*
- *No stiffeners in line with the beam flanges.*

(6) *In addition, the column web may be reinforced by:*

- *Diagonal stiffeners.*
- *A supplementary web plate.*

(7) *In bolted connections, column flanges may be reinforced by the use of backing plates.*

(8) *Methods are given for the determination of the following characteristics of a beam-to-column connection:*

- *Moment resistance.*
- *Rotational stiffness.*
- *Rotation capacity.*

J.1.2 *Other types of connection*

(1) *The methods given in this Annex can also be applied to beam-to-beam connections.*

(2) *Parts of the methods can also be applied to the relevant parts of other types of connections.*

(3) *These application rules do not cover connections in which the beam is to be connected to the web of the column.*

(4) *These application rules should not be applied to members with sections other than I or H sections.*

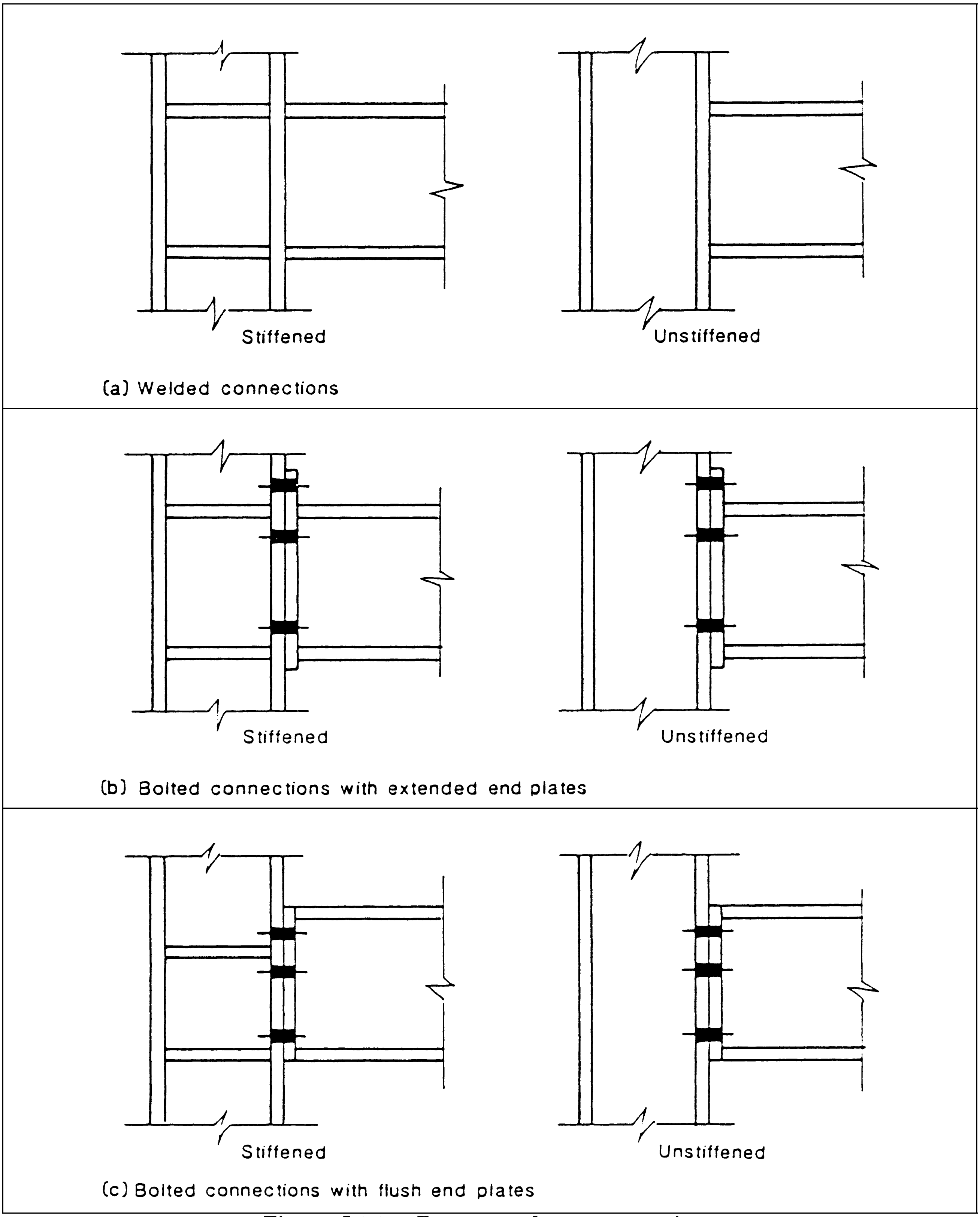


Figure J.1.1 — Beam-to-column connections

J.2 Welded beam-to-column connections

J.2.1 *Moment resistance*

(1) *The moment resistance of a welded beam-to-column connection depends on:*

- *the resistance of the tension zone (see **J.2.3**)*
- *the resistance of the compression zone (see **J.2.4**)*
- *the resistance of the shear zone (see **J.2.5**).*

J.2.2 *Supplementary web plates*

(1) *A supplementary web plate, see Figure J.2.1, may be used to increase the resistance of a column web in:*

- *tension, see **J.2.3.2***
- *compression, see **J.2.4.1***
- *shear, see **J.2.5.1**.*

(2) *The steel grade of the supplementary web plate should be similar to that of the column.*

(3) *The breadth b_s should be such that the welds connecting the supplementary web plate extend to the toe of the root radius, see Figure J.2.1.*

(4) *The length ℓ_s should be such that the supplementary web plate extends throughout the effective width of web in tension and compression, see Figure J.2.1.*

(5) *The thickness t_s should be not less than the column web thickness t_{wc}.*

(6) *The supplementary web plate should be welded all round (see Figure J.2.1). The welds should have a throat thickness a as follows:*

a) *where the supplementary web plate is required to increase the resistance of the web to shear or compression:*

$$a \geq t{\neq}/\sqrt{2} \qquad (J.1)$$

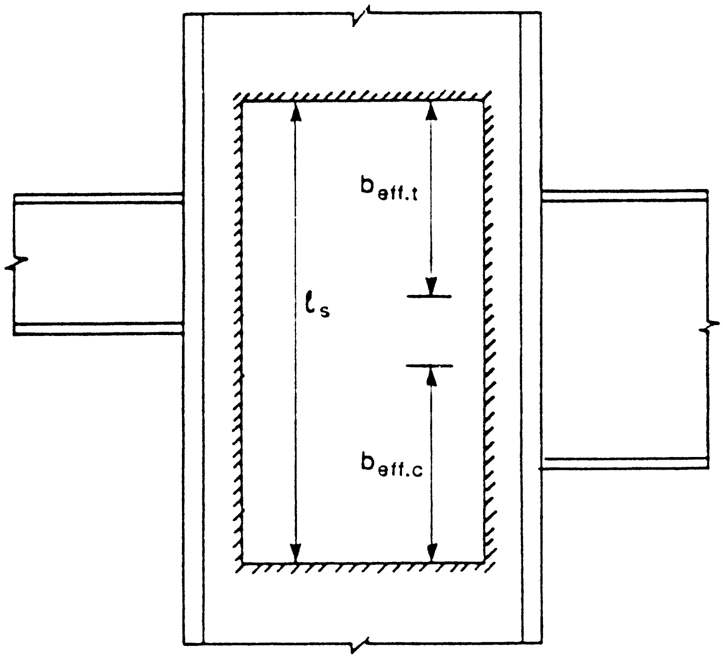
b) *where the supplementary web plate is required to increase the resistance of the web to tension, see **J.2.3.2(4)**:*

- *longitudinal butt welds:*

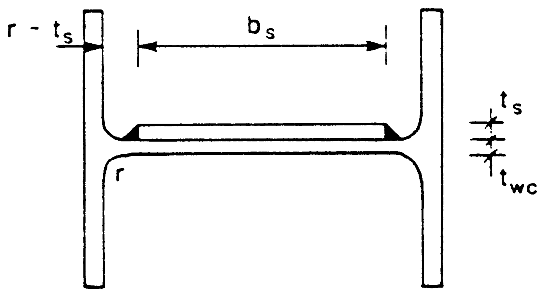
$$a \geq t_s \qquad (J.2)$$

- *transverse welds and longitudinal fillet welds:*

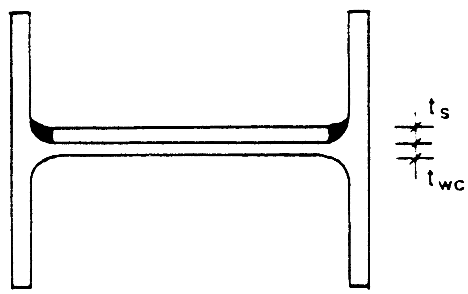
$$a \geq t_s/\sqrt{2} \qquad (J.3)$$



(a) Layout



(b) Cross-section with longitudinal fillet welds



(c) Cross-section with longitudinal butt welds

Figure J.2.1 — Supplementary web plate

(7) *When the breadth b_s of a supplementary web plate exceeds $40\varepsilon t_s$, a row of plug welds or bolts should be used to ensure proper co-operation between the supplementary web plate and the column web, see Figure J.2.2. The following requirements apply:*

$e_1 \leq 40\varepsilon t_s$

$e_2 \leq 40\varepsilon t_s$

$p \leq 40\varepsilon t_s$

$d_o \geq t_s$

where e_1 is *the end distance of the holes*

e_2 is *the edge distance of the holes*

p is *the pitch of the holes*

d_o is *the diameter of the holes*

and $\varepsilon = [235/f_y]^{0,5}$ (f_y *in* N/mm^2)

J.2.3 *Resistance of tension zone*

J.2.3.1 *Unstiffened column flange*

(1) *The design resistance of the unstiffened flange of a column subject to a transverse tensile force (see Figure J.2.3) is given by:*

- *for a rolled I or H section column:*

$$F_{t.Rd} = [f_{yb}\, t_{fb}\, (t_{wc} + 2r_c) + 7\, f_{yc}\, t_{fc}^2]/\gamma_{M0} \quad (J.4)$$

but $$F_{t.Rd} \leq f_{yb}\, t_{fb}\, [t_{wc} + 2r_c + 7\, t_{fc}]/\gamma_{M0} \quad (J.5)$$

- *for a welded I or H section column:*

$$F_{t.Rd} = [f_{yb}\, t_{fb}\, (t_{wc} + 2\sqrt{2}\, a_c) + 7\, f_{yc}\, t_{fc}^2]/\gamma_{M0} \quad (J.6)$$

but $$F_{t.Rd} \leq f_{yb}\, t_{fb}\, [t_{wc} + 2\sqrt{2}\, a_c + 7\, t_{fc}]/\gamma_{M0} \quad (J.7)$$

(2) *If the design resistance $F_{t.Rd}$ obtained from (1) does not satisfy the following condition, the joint should be stiffened:*

$$F_{t.Rd} \geq 0,7\, f_{yb}\, t_{fb}\, b_{fb}/\gamma_{M0} \quad (J.8)$$

where b_{fb} is the width of the beam flange.

(3) *The welds connecting the beam flange to the column should be designed to develop the full design resistance of the beam flange $f_{yb}\, t_{fb}\, b_{fb}/\gamma_{M0}$.*

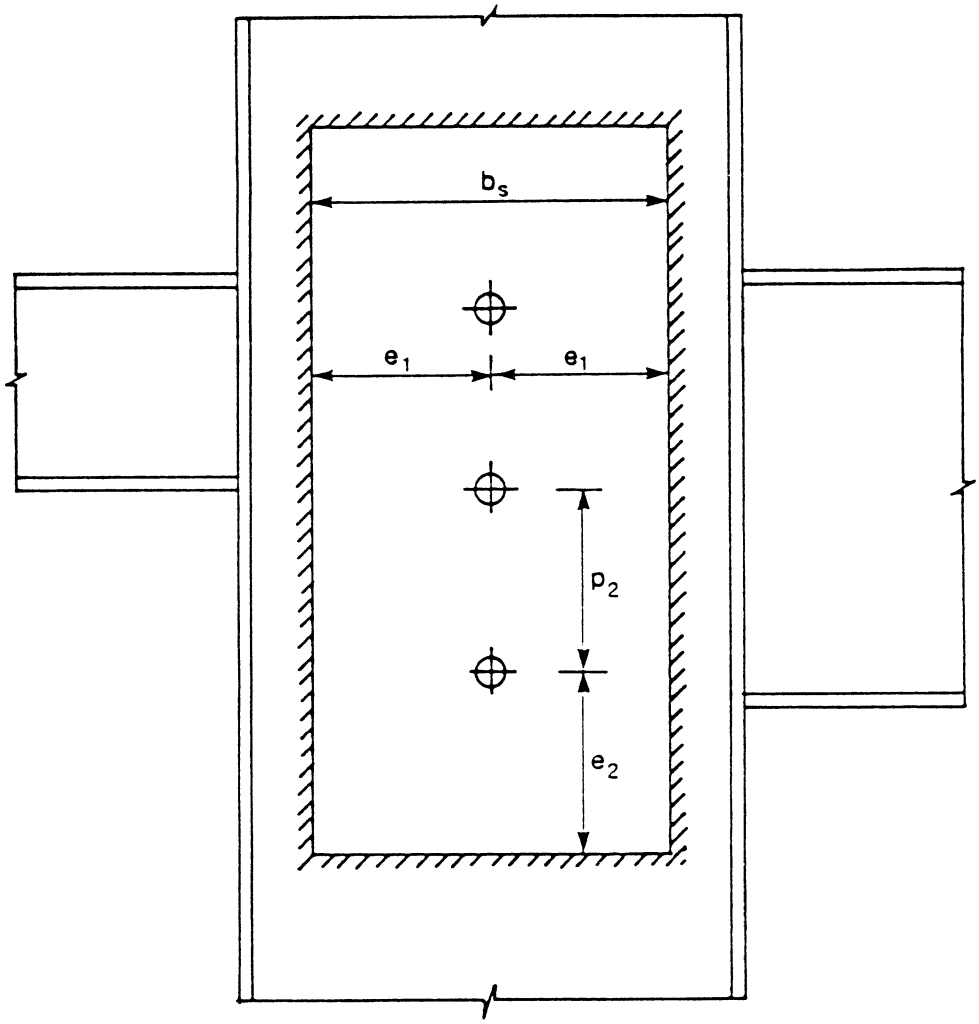


Figure J.2.2 — Spacing of plug welds or bolts for supplementary web plate

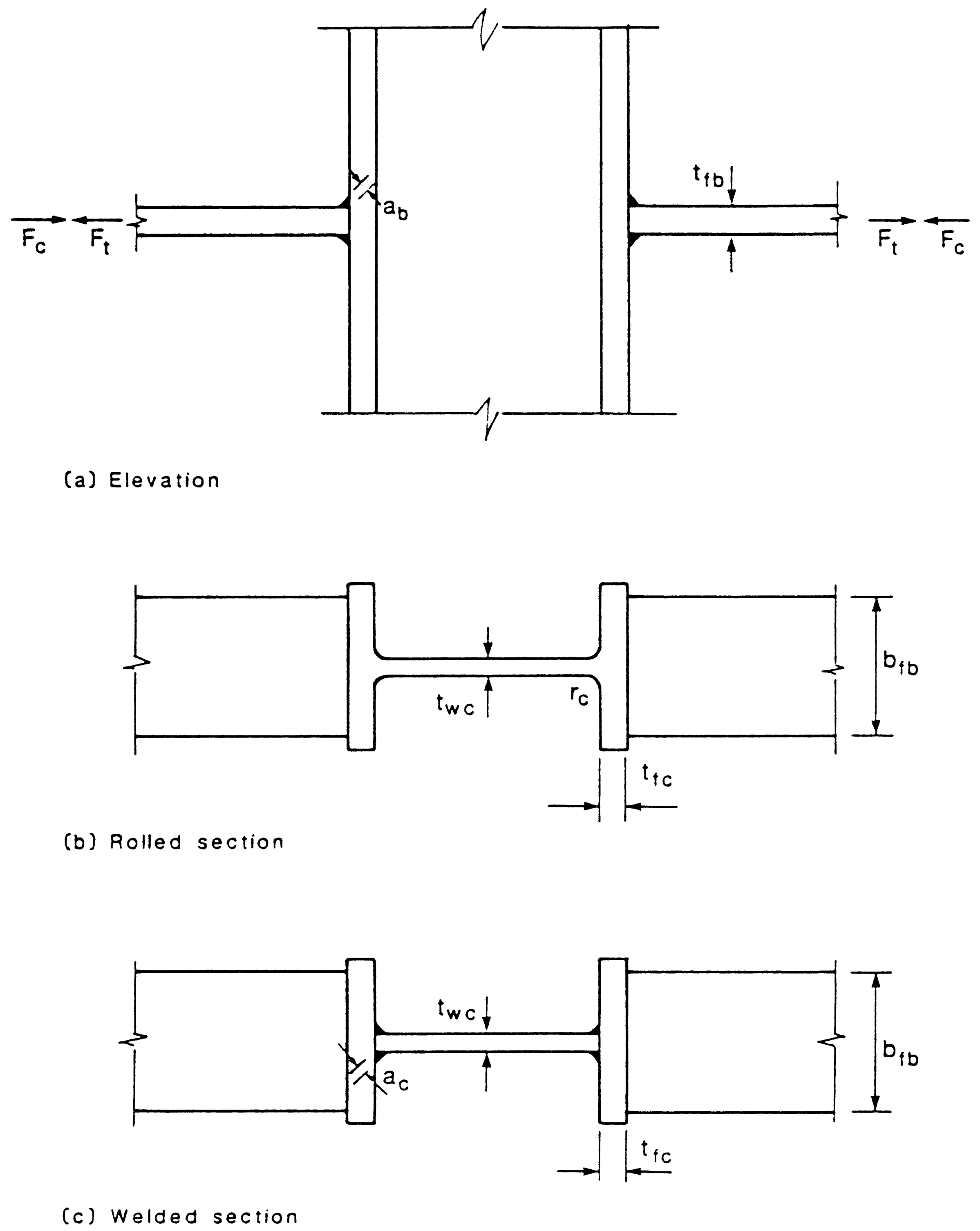


Figure J.2.3 — Transverse force on an unstiffened column

J.2.3.2 *Unstiffened column web*

(1) *The design resistance of an unstiffened column web subject to a transverse tensile force is given by:*

$$F_{t.Rd} = f_{yc}\, t_{wc}\, b_{eff}/\gamma_{M0} \qquad (J.9)$$

(2) *In a welded connection, the effective width of the column web, see Figure J.2.3, is given by:*

- *for a rolled I or H section column:*

$$b_{eff} = t_{fb} + 2\sqrt{2}\,a_b + 5(t_{fc} + r_c) \qquad (J.10)$$

- *for a welded I or H section column:*

$$b_{eff} = t_{fb} + 2\sqrt{2}\,a_b + 5(t_{fc} + \sqrt{2}\,a_c) \qquad (J.11)$$

(3) *An unstiffened column web may be strengthened by adding a supplementary web plate, see* ***J.2.2***.

(4) *The design tension resistance of a column web with a supplementary web plate conforming with* ***J.2.2*** *depends on the throat thickness of the longitudinal welds connecting the supplementary web plate, see* ***J.2.2(6)*** *b). The effective thickness of the web $t_{w.eff}$ may be taken as follows:*

- *when the longitudinal welds are butt welds with a throat thickness $a \geq t_s$:*
  - *with one supplementary web plate:*

$$t_{w.eff} = 1{,}5t_{wc} \qquad (J.12)$$

  - *with supplementary web plates both sides:*

$$t_{w.eff} = 2{,}0t_{wc} \qquad (J.13)$$

- *when the longitudinal welds are fillet welds with a throat thickness $a \geq t_s/\sqrt{2}$, then for either one or two supplementary web plates:*

$$t_{w.eff} = 1{,}4t_{wc} \qquad (J.14)$$

J.2.3.3 *Stiffened column*

(1) *The design resistance of a stiffened column subject to a transverse tensile force is at least equal to the design resistance of the beam flange, provided that the stiffeners meet the following requirements:*

a) *The thickness of the stiffeners should not be smaller than the flange thickness of the beam.*

b) *If the steel grade of the stiffeners is lower than that of the beam, the adequacy of the stiffeners to resist the transverse forces applied by the beam flanges should also be verified.*

c) *The welds between the stiffeners and the column flanges should be designed to resist the transverse forces applied by the beam flanges.*

d) *The welds between the stiffeners and the column web should be designed to resist the forces to be transferred into the column web from the beam flanges.*

J.2.4 *Resistance of compression zone*

J.2.4.1 *Unstiffened column web*

(1) *The design crushing resistance of an unstiffened column web subject to a transverse compression force is given by:*

$$F_{c.Rd} = f_{yc}\, t_{wc}\, [1{,}25 - 0{,}5\,\gamma_{M0}\,\sigma_{n.Ed}/f_{yc}]\, b_{eff}/\gamma_{M0} \qquad (J.15)$$

but $$F_{c.Rd} \leq f_{yc}\, t_{wc}\, b_{eff}/\gamma_{M0} \qquad (J.16)$$

where $\sigma_{n.Ed}$ *is the maximum compressive normal stress in the web of the column due to axial force and bending.*

(2) *In a welded connection, the effective width of the column web, see Figure J.2.3, is given by:*

- *for a rolled I or H section column:*

$$b_{eff} = t_{fb} + 2\sqrt{2}\,a_b + 5\,(t_{fc} + r_c) \qquad (J.10)$$

- *for a welded I or H section column:*

$$b_{eff} = t_{fb} + 2\sqrt{2}\,a_b + 5\,(t_{fc} + \sqrt{2}\,a_c) \qquad (J.11)$$

(3) *In addition the resistance of the column web to buckling in a column mode, as indicated in Figure J.2.4, should be verified using* ***5.7.5***.

(4) *The sway mode shown in Figure J.2.4(b) should normally be prevented by constructional restraints.*

(5) *An unstiffened column web may be strengthened by adding a supplementary web plate conforming with* ***J.2.2***.

(6) *In calculating the design crushing resistance of a column web with a supplementary web plate, the effective thickness of the web may be taken as 1,5t_{wc} when one supplementary web plate is added or 2,0t_{wc} when supplementary web plates are added both sides of the web.*

J.2.4.2 *Stiffened column web*

(1) *The design resistance of a stiffened column web subject to a transverse compression force is at least equal to the design resistance of the beam flange, provided that the stiffeners meet the requirements specified in **J.2.3.3(1)**.*

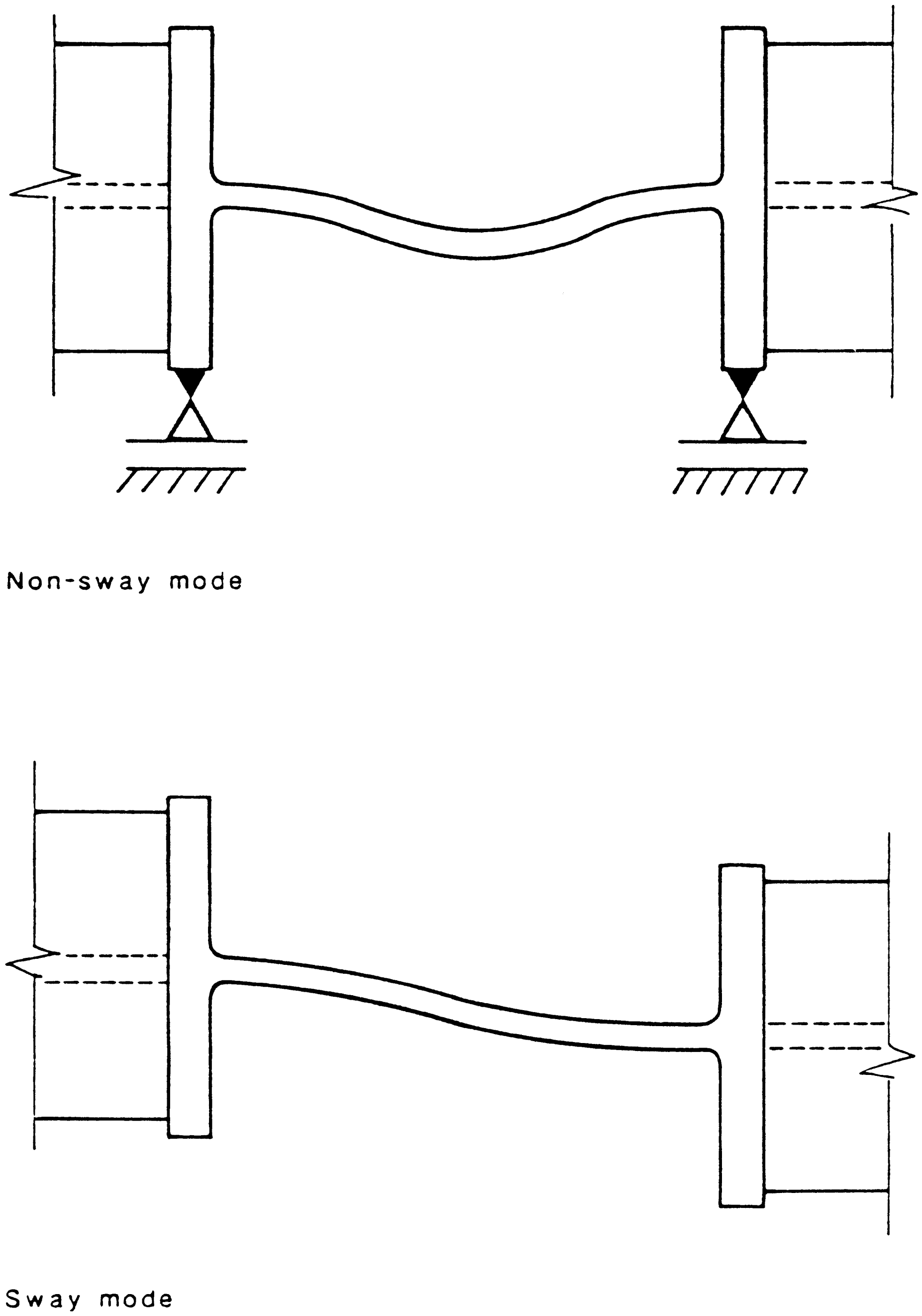


Figure J.2.4 — Column buckling modes of an unstiffened web

J.2.5 *Resistance of shear zone*

J.2.5.1 *Unstiffened column web panel*

(1) *The design plastic shear resistance of an unstiffened column web panel subject to a shear force (see Figure J.2.5) is given by:*

$$V_{p\ell.Rd} = [f_{yc}\, A_v / \sqrt{3}] / \gamma_{M0} \qquad (J.17)$$

where A_v *is the shear area of the column as given in* ***5.4.6****(2).*

(2) *In addition the shear buckling resistance should be checked, if necessary, see* ***5.4.6****(7).*

(3) *An unstiffened column web can be strengthened by adding a supplementary web plate conforming with* ***J.2.2****.*

(4) *In calculating the design shear resistance of a web panel with a supplementary web plate, its shear area* A_v *may be increased by* $b_s t_{wc}$*. No further increase in* A_v *should be made if supplementary web plates are added both sides of the web.*

J.2.5.2 *Stiffened column web panel*

(1) *When diagonal web stiffeners (see Figure J.2.6) are used to increase the shear resistance of a column web, they should be designed to resist the tensile and compressive forces transmitted to the column by the flanges of the beams.*

(2) *The welds between the stiffeners and the column flanges should be designed to resist the forces in the stiffeners.*

(3) *The welds between the stiffeners and the column web should be treated as nominal.*

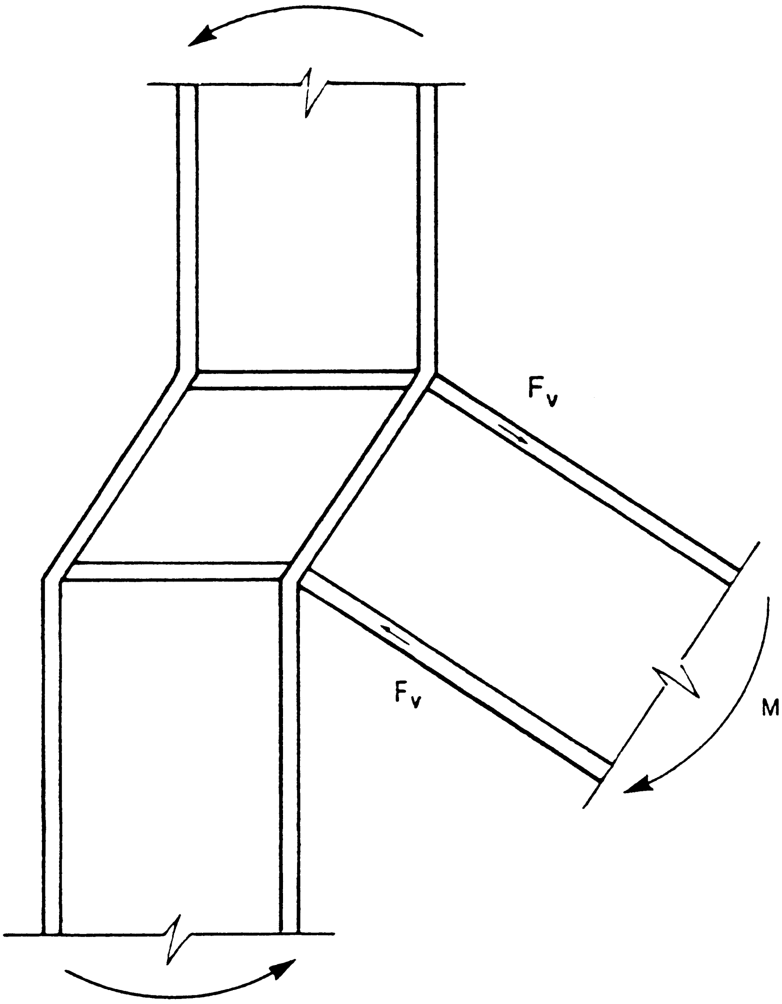


Figure J.2.5 — Unstiffened column web panel subject to a shear force

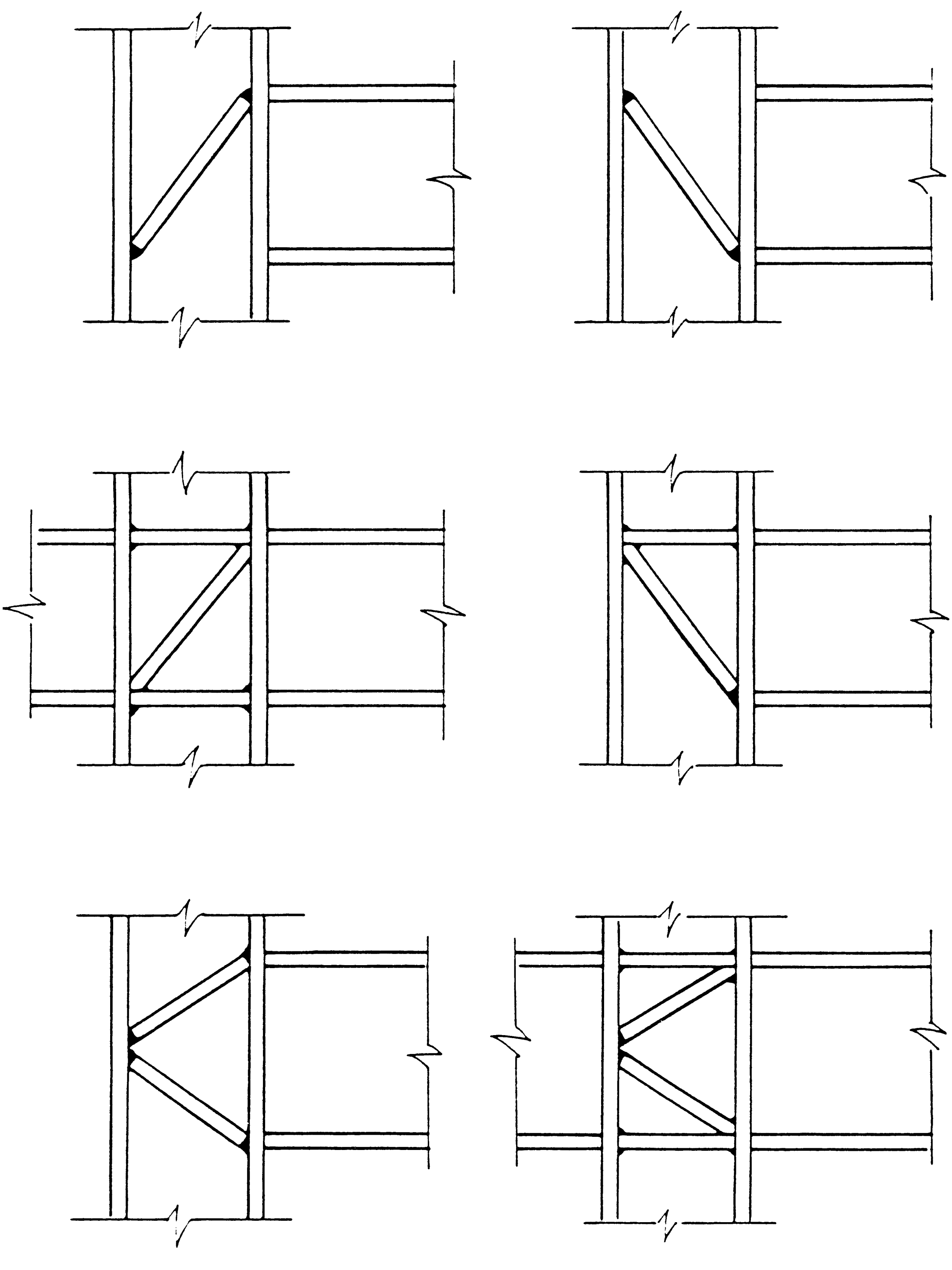


Figure J.2.6 — Column web panels with diagonal stiffeners

J.2.6 *Rotational stiffness*

(1) *The rotational stiffness of a welded beam-to-column connection may be obtained from:*

$$S_j = \frac{E\,(h_b - t_{fb})^2\, t_{wc}}{\sum \frac{1}{k_i}\left[\frac{F_i}{F_{i.Rd}}\right]^2} \qquad (J.18)$$

where S_j *is the secant stiffness with respect to a particular moment M in the connection* ($M \leq M_{Rd}$)

k_i *is the stiffness factor for component i*

F_i *is the force in component i of the connection due to the moment M, but not less than* $F_{i.Rd}/1,5$.

$F_{i.Rd}$ *is the design resistance of component i of the connection*

(2) *In an unstiffened welded connection, the stiffness factors* k_i *should be taken as follows:*

Column web, shear zone: $k_1 = 0,24$

Column web, tension zone: $k_2 = 0,8$

Column web, compression zone: $k_3 = 0,8$

(3) *For any stiffened component, the relevant stiffness factor* k_i *should be taken as infinity.*

(4) *A welded connection in which the column web is stiffened in both the tension zone and the compression zone may be assumed to be a rigid connection (see* ***6.4.2.2****).*

J.2.7 *Rotation capacity*

(1) *An unstiffened welded beam-to-column connection, designed in conformity with the application rules given in this annex, may be assumed to have a rotation capacity* ϕ_{Cd} *of 0,015 radians.*

(2) *A full-strength welded beam-to-column connection may be assumed to have adequate rotation capacity for plastic analysis.*

(3) *A welded beam-to-column connection in which the moment resistance is governed by the resistance of the shear zone may be assumed to have adequate rotation capacity for plastic analysis.*

(4) *A welded beam-to-column connection in which the column is stiffened in both the tension zone and the compression zone may be assumed to have adequate rotation capacity for plastic analysis, even if it is not full-strength.*

(5) *A welded beam-to-column connection in which the column is stiffened in the tension zone but not in the compression zone, may be assumed to have adequate rotation capacity for plastic analysis.*

(6) *In a welded beam-to-column connection in which the column is stiffened in the compression zone but not in the tension zone, when the moment resistance is not governed by the resistance of the shear zone, see (3) above, the rotation capacity* ϕ_{Cd} *may be determined from:*

$$\phi_{Cd} = 0,025 h_c / h_b \qquad (J.19)$$

J.3 Bolted beam-to-column connections

J.3.1 *Limitations*

(1) *In section* ***J.3*** *the following limitations apply:*

- *All bolted beam-to-column connections are assumed to have only 2 bolts in each bolt-row.*
- *The projecting portion of an extended end plate is assumed to have only one row of bolts.*
- *The projecting portion of an extended end plate is assumed to be unstiffened.*

(2) *Parts of the methods given in section* ***J.3*** *can also be applied to the relevant parts of other types of connection.*

(3) *The predicted rotational stiffness at the serviceability limit state is reasonably accurate, but the predicted rotational stiffness at the ultimate limit state is low in some cases.*

J.3.2 ***Moment resistance***

(1) *The moment resistance of a bolted beam-to-column connection depends on:*

- *The resistance of the tension zone, see* ***J.3.4****.*
- *The resistance of the compression zone, see* ***J.3.5****.*
- *The resistance of the shear zone, see* ***J.3.6****.*

(2) *Except as specified in (3), the moment resistance of a bolted beam-to-column connection should be obtained using Procedure J.3.1.*

(3) *The moment resistance of a bolted beam-to-column connection which is required to be full strength, may also be determined by using Procedure J.3.1 or alternatively by using Procedure J.3.2.*

Procedure J.3.1	**Sheet 1**
Moment resistance of a bolted beam-to-column connection — plastic distribution of bolt forces.	
(1)	Determine the potential resistance of the column flange in the tension zone, see **J.3.4.1** to **J.3.4.3**.
(2)	Determine the potential resistance of the beam end plate in the tension zone, see **J.3.4.4**.
(3)	Using the values obtained in Steps (1) and (2), obtain the effective resistance for each individual row of bolts in the tension zone, see **J.3.4.5**.
(4)	Except in the case of a full-strength connection, if the design value of the effective resistance for any individual bolt-row exceeds $1{,}8B_{t.Rd}$, where $B_{t.Rd}$ is as given in **J.3.3**(3), change the design of the connection (for example by using stronger bolts), unless it can be shown that the effective resistance of that bolt-row will be omitted (or reduced below $1{,}8B_{t.Rd}$) in Step (10).
(5)	From Step (3), determine the total effective resistance of all the bolt-rows in the tension zone.
(6)	Determine the resistance of the column web in the tension zone, see **J.3.4.6** to **J.3.4.7**.
(7)	Determine the resistance of the column web in the compression zone, see **J.3.5**.
(8)	Determine the resistance of the column web in the shear zone, see **J.3.6**.
(9)	Adopt the smallest of the design values obtained in Steps (5) to (8) as the resistance of the weakest zone.
(10)	If the total of the effective resistance of the bolt-rows in the tension zone obtained in Step (5) is greater than the resistance of the weakest zone obtained in Step (9), reduce it by omitting or reducing the effective resistances of successive bolt rows, starting with the row nearest to the centre of compression, until the effective resistance of the remaining bolt rows is equal to the resistance of the weakest zone.
(11)	Adopt a reduced tension zone containing only those bolt rows which remain after completing Step (10).
(12)	Re-check the resistance of the column web in the reduced tension zone, see **J.3.4.6** to **J.3.4.7**.
(13)	If the value obtained in Step (12) is less than the total effective resistance of the bolt-rows in the reduced tension zone, adopt it as the new value of the resistance of the weakest zone and return to Step (10).
(14)	Check the resistance of the tension zone of the beam web adjacent to the beam end plate in the same way as for the column web, Step (12).
(15)	If the value obtained in Step (14) is less than the total effective resistance of the bolt-rows in the reduced tension zone, adopt it as the new value of the resistance of the weakest zone and return to Step (10).

Procedure J.3.1	**Sheet 2**
(16) Determine the design value of the moment resistance of the connection M_{Rd} based on the bolt-rows in the reduced tension zone, from:	
$M_{Rd} = \Sigma\ [F_{ti.Rd}\ h_i]$	(J.20)
where $F_{ti.Rd}$ is the design value of the effective resistance of an individual row of bolts	
and h_i is the distance from that bolt-row to the centre of resistance of the compression zone.	
(17) Ensure that the resistance of the welds between the beam flange and the end plate satisfies **J.3.4.4**(6).	

Procedure J.3.2	**Sheet 1**
Moment resistance of a bolted beam-to-column connection — distribution of bolt forces proportional to distance from centre of compression.	

(1)	Adopt a distribution of bolt forces in which the resistance of each individual bolt-row in the tension zone of the compression zone and the maximum bolt-row force is $2,0B_{t.Rd}$, where $B_{t.Rd}$ is as given in **J.3.3**(3).
(2)	Using the values from Step (1), determine the total effective resistance of all the bolt-rows in the tension zone.
(3)	Determine the resistance of the column web in the tension zone, see **J.3.4.6** to **J.3.4.7**.
(4)	Determine the resistance of the column web in the compression zone, see **J.3.5**.
(5)	Determine the resistance of the column web in the shear zone, see **J.3.6**.
(6)	Adopt the smallest of the design values obtained in Steps (2) to (5) as the resistance of the weakest zone.
(7)	If the total effective resistance of all the bolt-rows in the tension zone obtained in Step (2) is greater than the resistance of the weakest zone obtained in Step (6), reduce the force in each individual bolt-row pro rata so that the total force for all the bolt-rows is equal to the resistance of the weakest zone.
(8)	For the column flange, ensure that the sum of the bolt-row forces from Step (7) for each group of bolt-rows (or for all the bolt-rows for an unstiffened flange) does not exceed $2M_{p\ell.Rd}/m$ for the relevant effective length of column flange from **J.3.4.1** or **J.3.4.3**, where $M_{p\ell.Rd}$ and m are as defined in **J.3.3**(3).
(9)	If necessary to satisfy Step (8), reduce the force in each individual bolt-row pro rata.
(10)	For the column flange, ensure that the maximum bolt-row force from Step (9) in any bolt-row not adjacent to a column stiffener does not exceed $2M_{p\ell.Rd}/m$ for an effective length of column flange equal to the lesser of 4 m + 1,25e or 2 πm, where e is as defined in **J.3.3**(3).
(11)	If necessary to satisfy Step (10), reduce the force in each individual bolt-row pro rata.
(12)	For the beam end plate, ensure that the sum of the bolt-row forces from Step (11) for each group of bolt-rows does not exceed $2M_{p\ell.Rd}/m$ for the relevant effective length of end plate from **J.3.4.4**, using the relevant values of $M_{p\ell.Rd}$ and m for the end plate.
(13)	If necessary to satisfy Step (12), reduce the force in each individual bolt-row pro rata.
(14)	For the beam end plate, ensure that the maximum bolt-row force from Step (13) in any bolt-row not adjacent to a stiffener or flange connected to the beam end plate, does not exceed $2M_{p\ell.Rd}/m$ for an effective length of end plate equal to the lesser of 4 m + 1,25e or 2 πm.
(15)	If necessary to satisfy Step (14), reduce the force in each individual bolt-row pro rata.
(16)	For the column web, ensure that the maximum bolt-row force from Step (15) in any bolt-row not adjacent to a column stiffener does not exceed the resistance of the column web in the tension zone, see **J.3.4.6**, for an effective width of column web equal to the effective length of column flange from Step (10).
(17)	If necessary to satisfy Step (16), reduce the force in each individual bolt-row pro rata.
(18)	Check the resistance of the tension zone of the beam web adjacent to the beam end plate in the same way as for the column web, see **J.3.4.6** to **J.3.4.7**, considering both the whole of each group of bolt-rows and the critical individual bolt-row from Step (14).
(19)	If necessary to satisfy Step (18), reduce the force in each individual bolt-row pro rata.

Procedure J.3.2	**Sheet 2**
(20) Determine the design value of the moment resistance of the connection M_{Rd} from:	

$$M_{Rd} = F_{t1.Rd} \frac{\sum h_i^2}{h_1} \qquad (J.21)$$

where $F_{t1.Rd}$ is the design value of the effective resistance of the bolt-row farthest from the centre of resistance of the compression zone.

h_1 is the distance from the farthest bolt-row to the centre of resistance of the compression zone.

and h_i is the distance from any bolt-row to the centre of resistance of the compression zone.

J.3.3 *Equivalent T-stub*

(1) *The tension resistance of the column flange and of the beam end-plate are given in terms of equivalent T-stubs, see Figure J.3.1.*

(2) *The resistance of a T-stub may be governed by:*

- *the resistance of the flange*
- *the resistance of the bolts*
- *the resistance of the web, and*
- *the resistance of the web-to-flange welds, in the case of a welded T-stub.*

(3) *The design tension resistance of a T-stub flange should be taken as the smallest value for the three possible modes of the failure indicated in Figure J.3.2 as follows:*

Mode 1: *Complete yielding of flange:*

$$F_{t.Rd} = \frac{4M_{p\ell.Rd}}{m} \qquad (J.22)$$

Mode 2: *Bolt failure with yielding of flange:*

$$F_{t.Rd} = \frac{2M_{p\ell.Rd} + n\Sigma B_{t.Rd}}{m + n} \qquad (J.23)$$

Mode 3: *Bolt failure only:*

$$F_{t.Rd} = \Sigma B_{t.Rd} \qquad (J.24)$$

where $M_{p\ell.Rd} = 0{,}25\ell t_y^2 f_y/\gamma_{M0}$ *(J.25)*

$B_{t.Rd}$ *is the design tension resistance of a single bolt-plate assembly, obtained from* ***6.5.5****(4).*

$\Sigma B_{t.Rd}$ *is the total value for all the bolt in the T-stub.*

$n = e_{min}$ *but* $n \leq 1{,}25\,m$

and ℓ*, m and e are as indicated in Figure J.3.1.*

(4) *The relationship between connection geometry and mode of failure is indicated in Figure J.3.3, in which:*

$$\beta = \frac{4M_{p\ell.Rd}}{m\Sigma B_{t.Rd}}$$

and $\lambda = n/m$

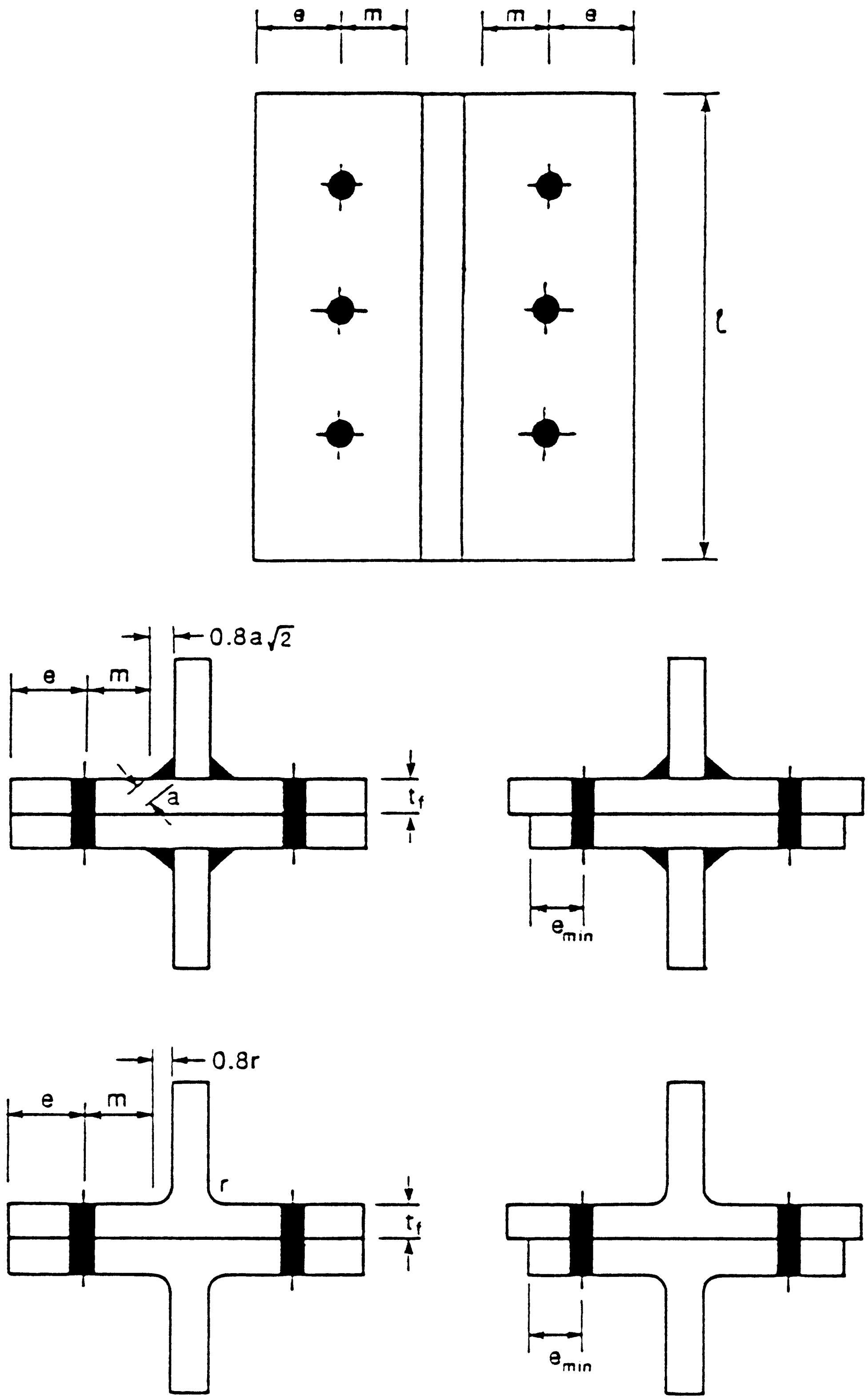


Figure J.3.1 — Dimensions of a T-stub

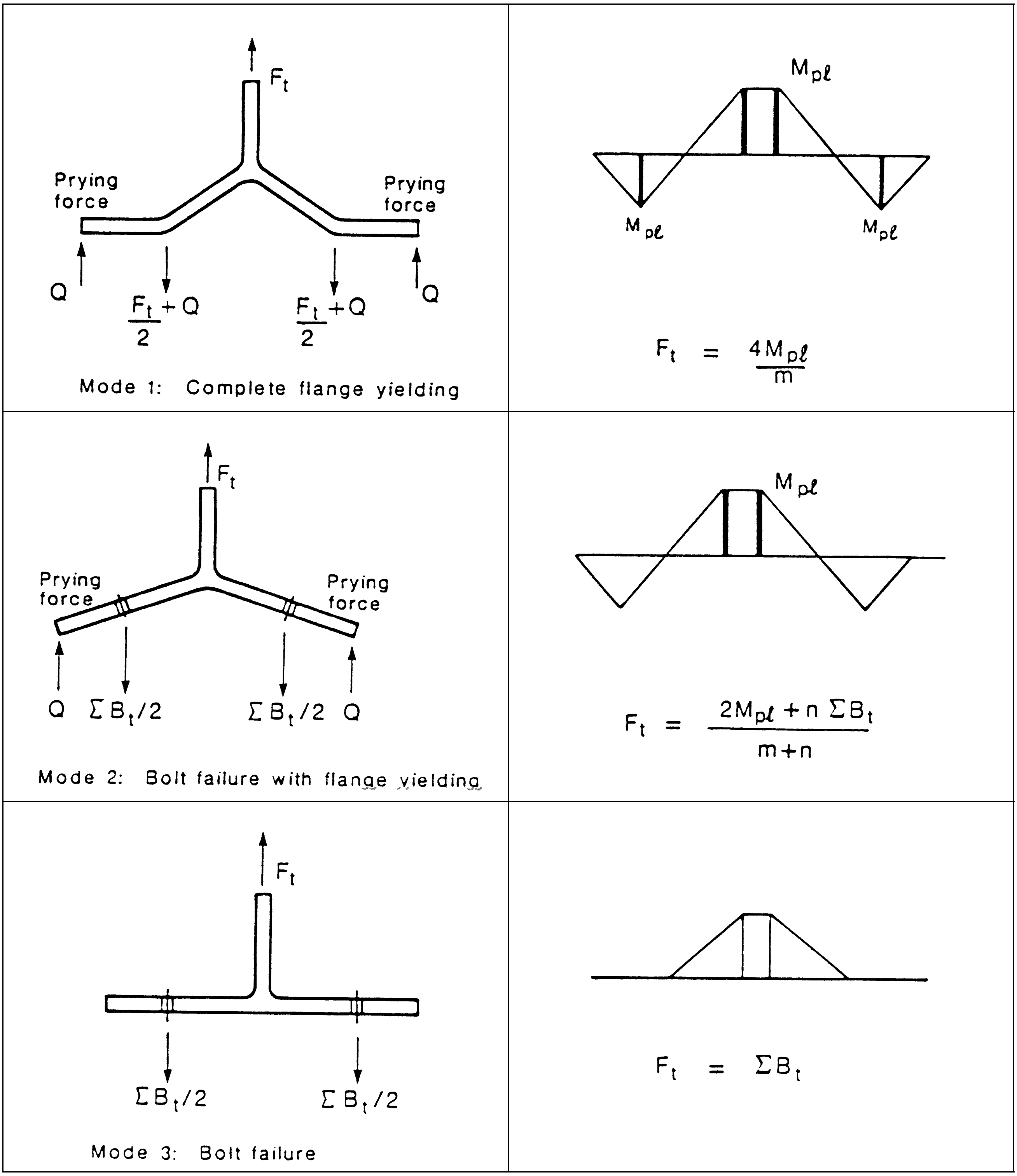


Figure J.3.2 — Failure modes of a T-stub flange

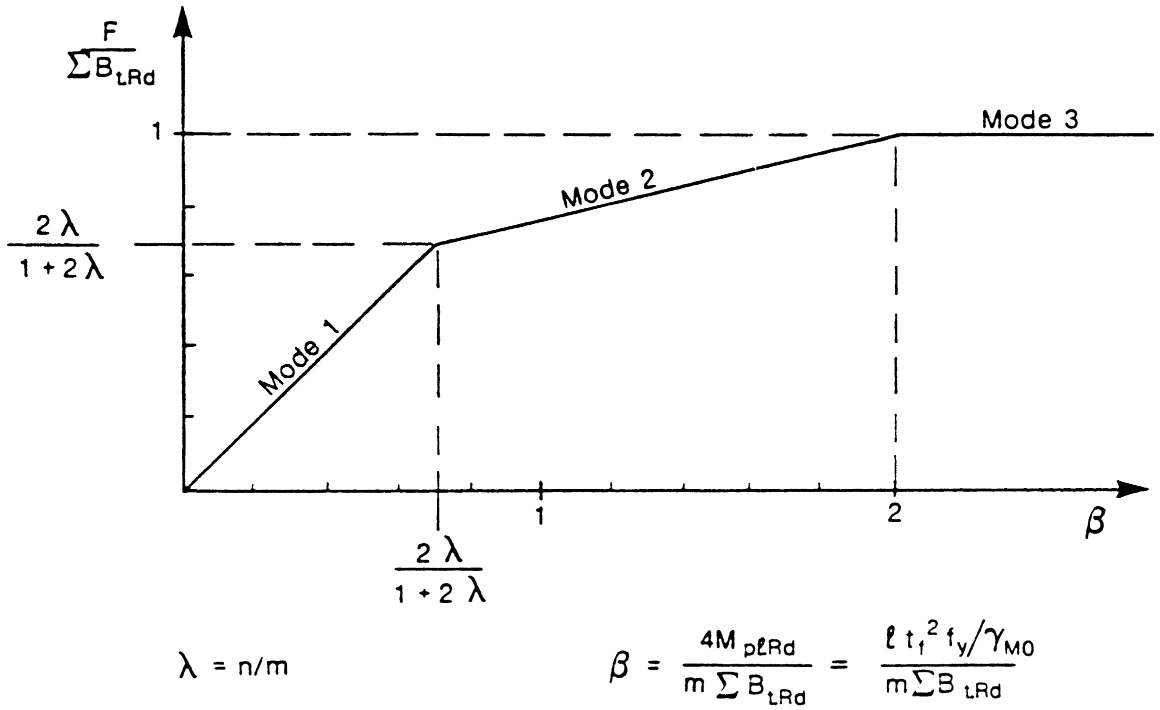


Figure J.3.3 — Effect of connection geometry on failure mode of a T-stub flange

J.3.4 ***Resistance of tension zone***

J.3.4.1 *Unstiffened column flange*

(1) *The tension zone of an unstiffened column flange should be considered to act as a series of equivalent T-stubs with a total length equal to the total effective length $\Sigma\ell_{eff}$ of the bolt pattern in the tension zone of the connection, as indicated in Figure J.3.4.*

(2) *The effective length ℓ_{eff} for each row of bolts should be taken as the smallest of the following values for the respective case:*

a) ***for inner bolts:***

$\ell_{eff.a} = p$	*[see Figure J.3.4(a)]*	*(J.26)*
$\ell_{eff.a} = 4\,m + 1{,}25e$	*[see Figure J.3.4(b)]*	*(J.27)*
$\ell_{eff.a} = 2\pi m$	*[see Figure J.3.4(c)]*	*(J.28)*

b) ***for end bolts:***

$\ell_{eff.b} = 0{,}5p + 2\,m + 0{,}625e$	*[see Figure J.3.4(a)]*	*(J.29)*
$\ell_{eff.b} = 4\,m + 1{,}25e$	*[see Figure J.3.4(b)]*	*(J.30)*
$\ell_{eff.b} = 2\pi m$	*[see Figure J.3.4(c)]*	*(J.31)*

(3) *When the compressive normal stress $\sigma_{n.Ed}$ in the column flange due to the axial force and bending moment in the column exceeds 180 N/mm² at the location of the tension zone, the possible reduction of the design moment resistance of the column flange should be allowed for by multiplying the value of $M_{p\ell.Rd}$ in **J.3.3**(3) by a reduction factor k_r obtained as follows:*

- *when $\sigma_{n.Ed} \leq 180$ N/mm²:*

$k_r = 1$

- *when 180 N/mm² $< \sigma_{n.Ed} \leq f_y$:*

$$k_r = \frac{2f_y - 180 - \sigma_{n.Ed}}{2f_y - 360} \quad \text{but } k_r \leq 1 \qquad (J.32)$$

where $\sigma_{n.Ed}$ and f_y are in N/mm².

(4) *The mode of failure and the maximum potential design resistance should be determined by considering all the bolt-rows in the tension zone as a single group acting together in a single equivalent T-stub.*

(5) *For this purpose, the equivalent T-stub should be assumed to be in equilibrium with another similar T-stub. The minimum value of e for the column flange or the beam end-plate should be used to determine n but the actual value of e for the column flange should be used to determine ℓ_{eff}.*

(6) *The actual effective design resistance for each bolt row, allowing for compatibility with the tension zone of the beam end-plate, should be determined as described in **J.3.4.5**.*

J.3.4.2 *Column flange with backing plates*

(1) *Column flanges may be reinforced by adding loose backing plates as indicated in Figure J.3.5.*

(2) *The width of a backing plate b_{bp} should not be less than the distance from the edge of the flange to the toe of the root radius or weld fillet.*

(3) *The length of a backing plate should not be less than the total effective length for the bolt pattern in the tension zone of the connection and should be such that it extends not less than 2d beyond the last bolt at each end.*

(4) *The design tension resistance of a column flange reinforced with backing plates, should be taken as the smallest value for the three possible modes failure [see **J.3.3**(3)] as follows:*

Mode 1: *Complete yielding of flange and backing plate:*

$$F_{t.Rd} = \frac{4M_{p\ell.Rd} + 2M_{bp.Rd}}{m} \qquad (J.33)$$

Mode 2: *Bolt failure with yielding of flange only:*

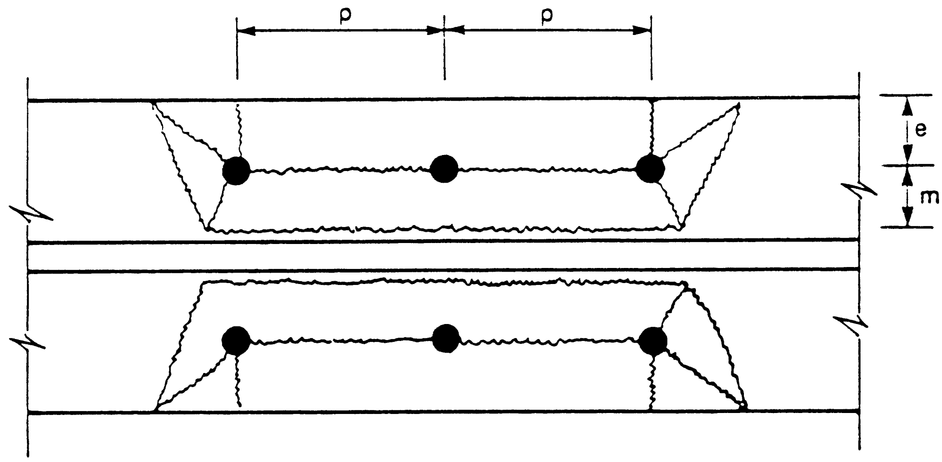
$$F_{t.Rd} = \frac{2M_{p\ell.Rd} + n\Sigma B_{t.Rd}}{m + n} \qquad (J.23)$$

Mode 3: *Bolt failure only:*

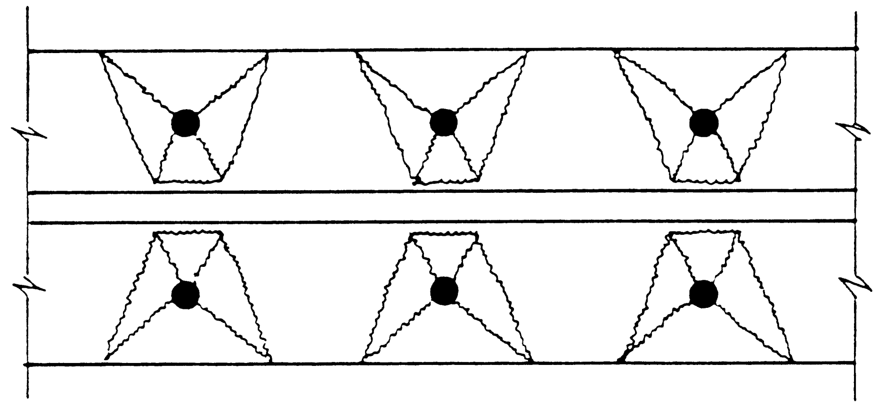
$$F_{t.Rd} = \Sigma B_{t.Rd} \qquad (J.24)$$

where $M_{bp.Rd}$ is the design moment resistance of a backing plate given by:

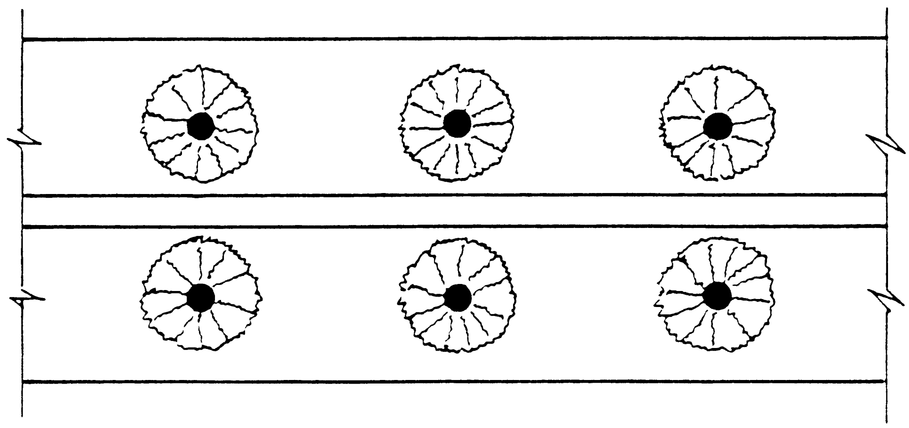
$$M_{bp.Rd} = 0{,}25\, \ell_{eff}\, t_{bp}^2\, f_{y.bp}/\gamma_{M0} \qquad (J.25)$$



(a) Combined bolt group pattern



(b) Separate bolt patterns



(c) Circles around each bolt

Figure J.3.4 — Yield line patterns for an unstiffened flange

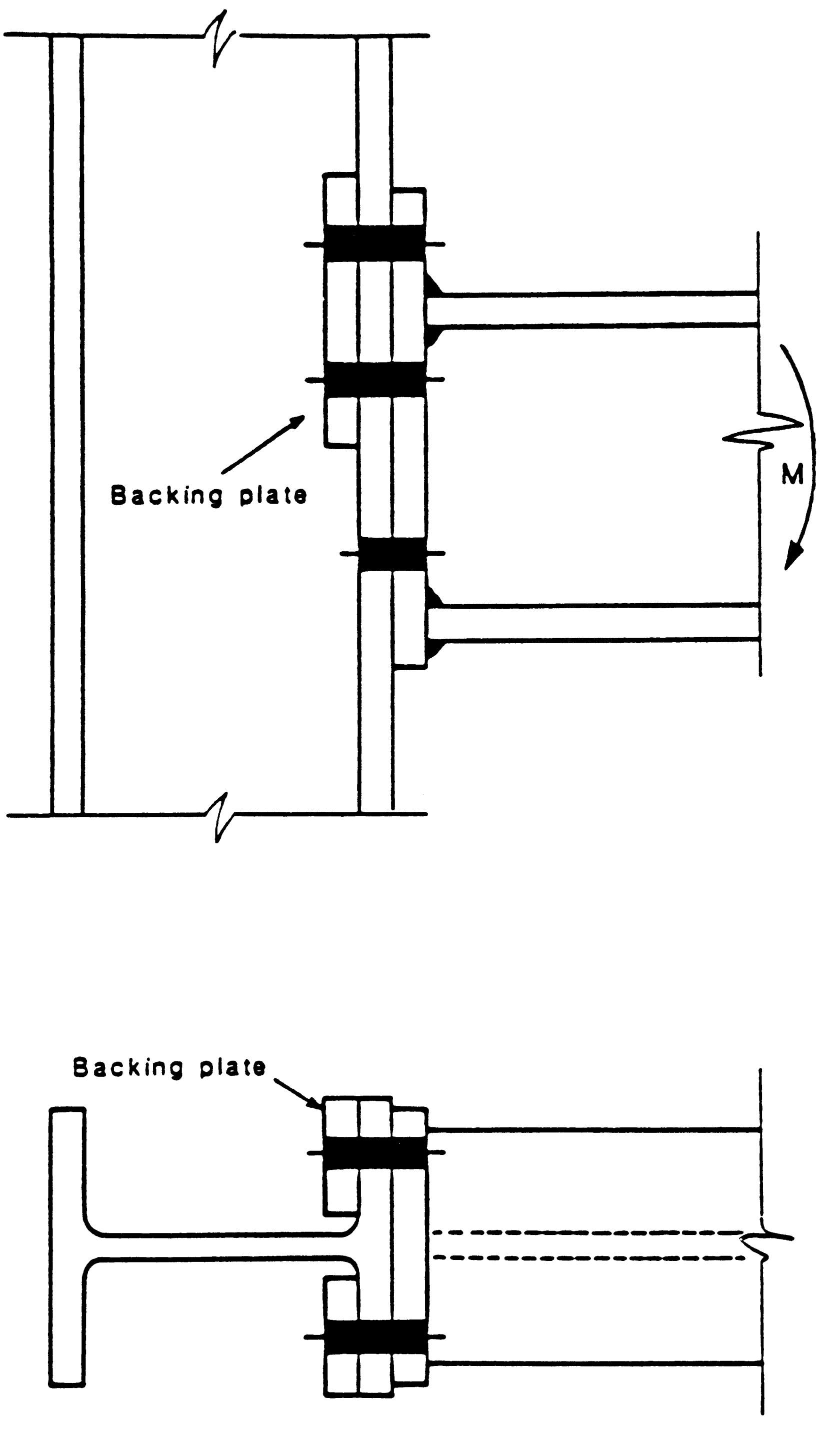


Figure J.3.5 — Column flange with backing plates in the tension zone

J.3.4.3 *Stiffened column flange*

(1) *The tension zone of a stiffened column flange should be considered to act as a series of equivalent T-stubs with a total length equal to the total effective length of the bolt pattern in the tension zone, as indicated in Figure J.3.6.*

(2) *The effective length ℓ_{eff} for each row of bolts should be taken as the smallest of the following values for the respective case:*

a) ***for bolts adjacent to a stiffener:***

$$\ell_{eff.a} = \alpha m \qquad (J.35)$$

$$\ell_{eff.a} = 2\pi m \qquad (J.28)$$

b) ***for other inner bolts:***

$$\ell_{eff.b} = p \qquad (J.36)$$

$$\ell_{eff.b} = 4m + 1{,}25e \qquad (J.30)$$

$$\ell_{eff.b} = 2\pi m \qquad (J.31)$$

c) ***for other end bolts:***

$$\ell_{eff.c} = 0{,}5p + 2m + 0{,}625e \qquad (J.37)$$

$$\ell_{eff.c} = 4m + 1{,}25e \qquad (J.38)$$

$$\ell_{eff.c} = 2\pi m \qquad (J.39)$$

where the ratio α is obtained from Figure J.3.7.

(3) *When the compressive normal stress $\sigma_{n.Ed}$ in the column flange due to the axial force and bending moment in the column exceeds 180 N/mm^2 at the location of the tension zone, the reduction factor k_r should be applied as in* ***J.3.4.1****(3).*

(4) *The groups of bolt-rows each side of a stiffener should be treated as separate overlapping equivalent T-stubs. The mode of failure and the maximum potential design resistance should be determined separately for each such group of bolt-rows.*

(5) *For this purpose each equivalent T-stub should be assumed to be in equilibrium with another similar T-stub. The minimum value of e for the column flange or the beam end plate should be used to determine n but the actual value of e for the column flange should be used to determine ℓ_{eff}.*

(6) *The actual effective design resistance for each bolt row, allowing for compatibility with the tension zone of the beam end-plate, should be determined as described in* ***J.3.4.5****.*

(7) *The stiffeners should meet the requirements specified in* ***J.2.3.3****(1).*

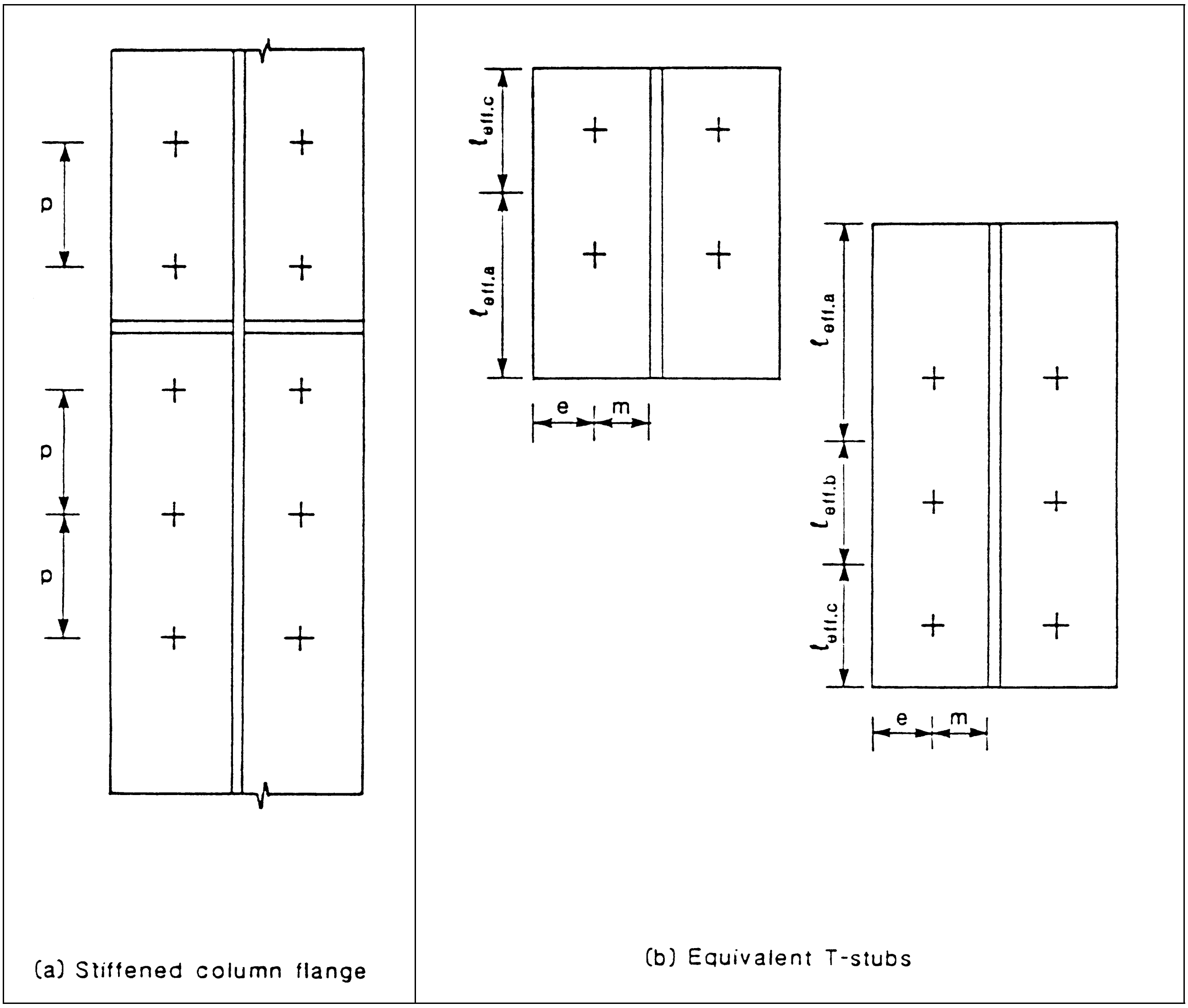


Figure J.3.6 — Effective lengths of equivalent T-stub flanges representing a stiffened column flange

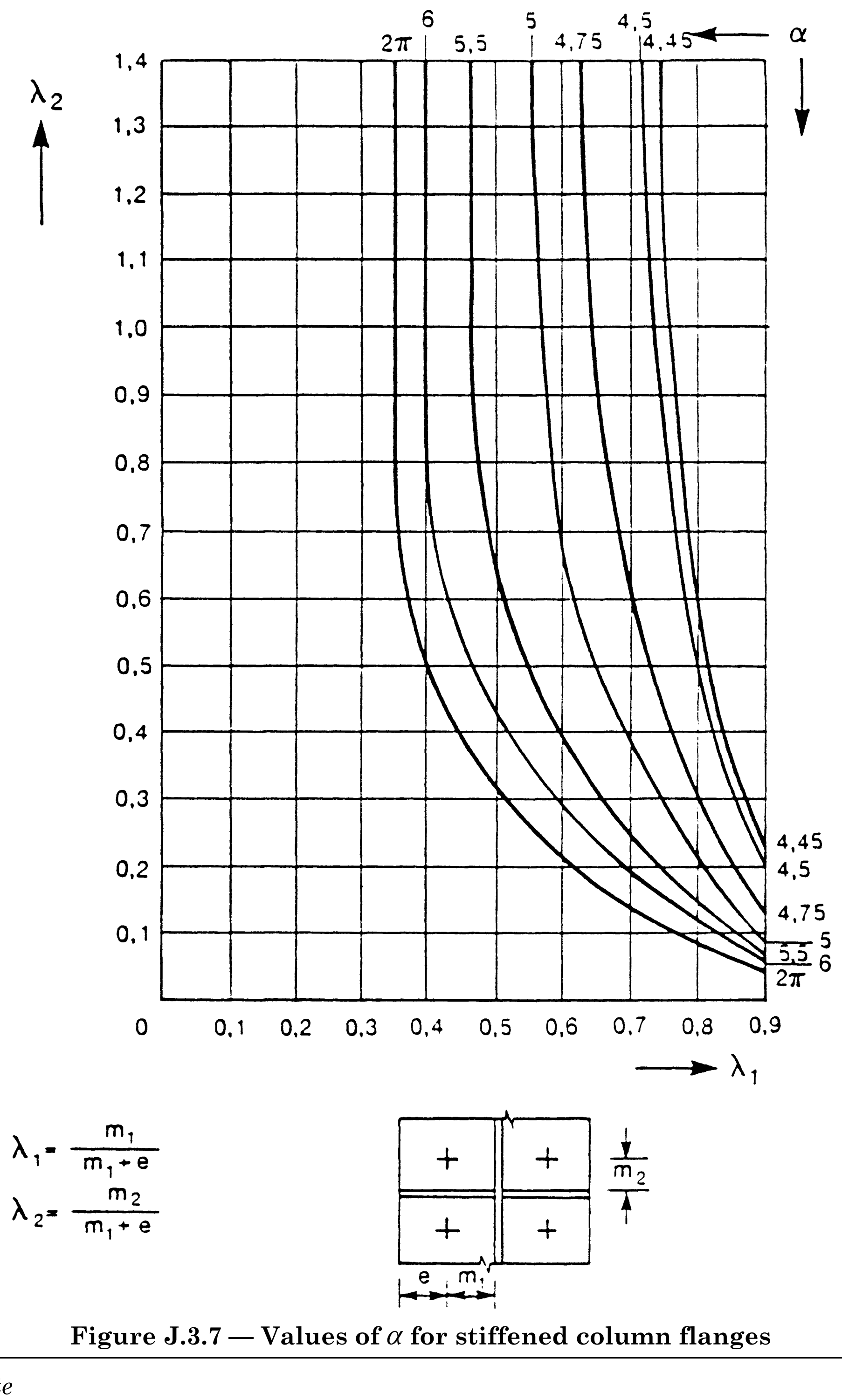


Figure J.3.7 — Values of α for stiffened column flanges

J.3.4.4 *End plate*

(1) *The tension zone of a beam end plate should be considered to act as a series of equivalent T-stubs with a total length equal to the total effective length of the bolt pattern in the tension zone, as indicated in Figure J.3.8.*

(2) *The effective length ℓ_{eff} for each row of bolts should be taken as the smallest of the following values for the respective case:*

a) ***for bolts outside tension flange of beam:***

$$\ell_{eff.a} = 0{,}5b_p \qquad (J.40)$$

$$\ell_{eff.a} = 0{,}5w + 2m_x + 0{,}625e_x \qquad (J.41)$$

$$\ell_{eff.a} = 4m_x + 1{,}25e_x \qquad (J.42)$$

$$\ell_{eff.a} = 2\pi m \qquad (J.43)$$

b) ***for first row of bolts below tension flange of beam:***

$$\ell_{eff.b} = \alpha m \qquad (J.44)$$

$$\ell_{eff.b} = 2\pi m \qquad (J.31)$$

c) ***for other inner bolts:***

$$\ell_{eff.c} = p \qquad (J.45)$$

$$\ell_{eff.c} = 4m + 1{,}25e \qquad (J.38)$$

$$\ell_{eff.c} = 2\pi m \qquad (J.39)$$

d) ***for other end bolts:***

$$\ell_{eff.d} = 0{,}5p + 2m + 0{,}625e \qquad (J.46)$$

$$\ell_{eff.d} = 4m + 1{,}25e \qquad (J.47)$$

$$\ell_{eff.d} = 2\pi m \qquad (J.48)$$

where the ratio α is obtained from Figure J.3.7.

(3) *The groups of bolt-rows each side of any stiffeners connected to the end plate should be treated as separate overlapping equivalent T-stubs. In extended end plates, the groups of bolt-rows above and below the tension flange of the beam should also be treated as separate overlapping equivalent T-stubs. The mode of failure and the maximum potential design resistance should be determined separately for each such group of bolt-rows.*

(4) *For this purpose each equivalent T-stub should be assumed to be in equilibrium with another similar T-stub. The minimum value of e for the end plate or the column flange should be used to determine n but the actual value of e for the end plate should be used to determine ℓ_{eff}.*

(5) *The actual effective design resistance for each bolt row, allowing for compatibility with the tension zone of the column flange, should be determined as described in* ***J.3.4.5***.

(6) *To ensure that the welds between the beam flange and the end plate have sufficient deformation capacity, they should be designed to resist the effects of a moment equal to the smaller of:*

- *the design plastic moment resistance of the beam $M_{p\ell.Rd}$*
- *Y times the design moment resistance of the connection.*

where $Y = 1{,}4$ *for a braced frame* (J.49)

or $Y = 1{,}7$ *for an unbraced frame.* (J.50)

J.3.4.5 *Effective resistance of bolt-rows*

(1) *The maximum potential design resistance of the column flange is generally not the same as the maximum potential design resistance of the beam end plate.*

(2) *In order to determine the actual design resistance of the tension zone, a compatible distribution of bolt-row forces should be obtained in which, for each row of bolts, there is equilibrium between its contributions to the design resistances of the column flange and the beam end-plate.*

(3) *The effective design resistances for the individual bolt-rows should be obtained using Procedure J.3.3.*

(4) *It may be assumed that the effective design resistance for each bolt-row acts at the centre-line of that bolt-row.*

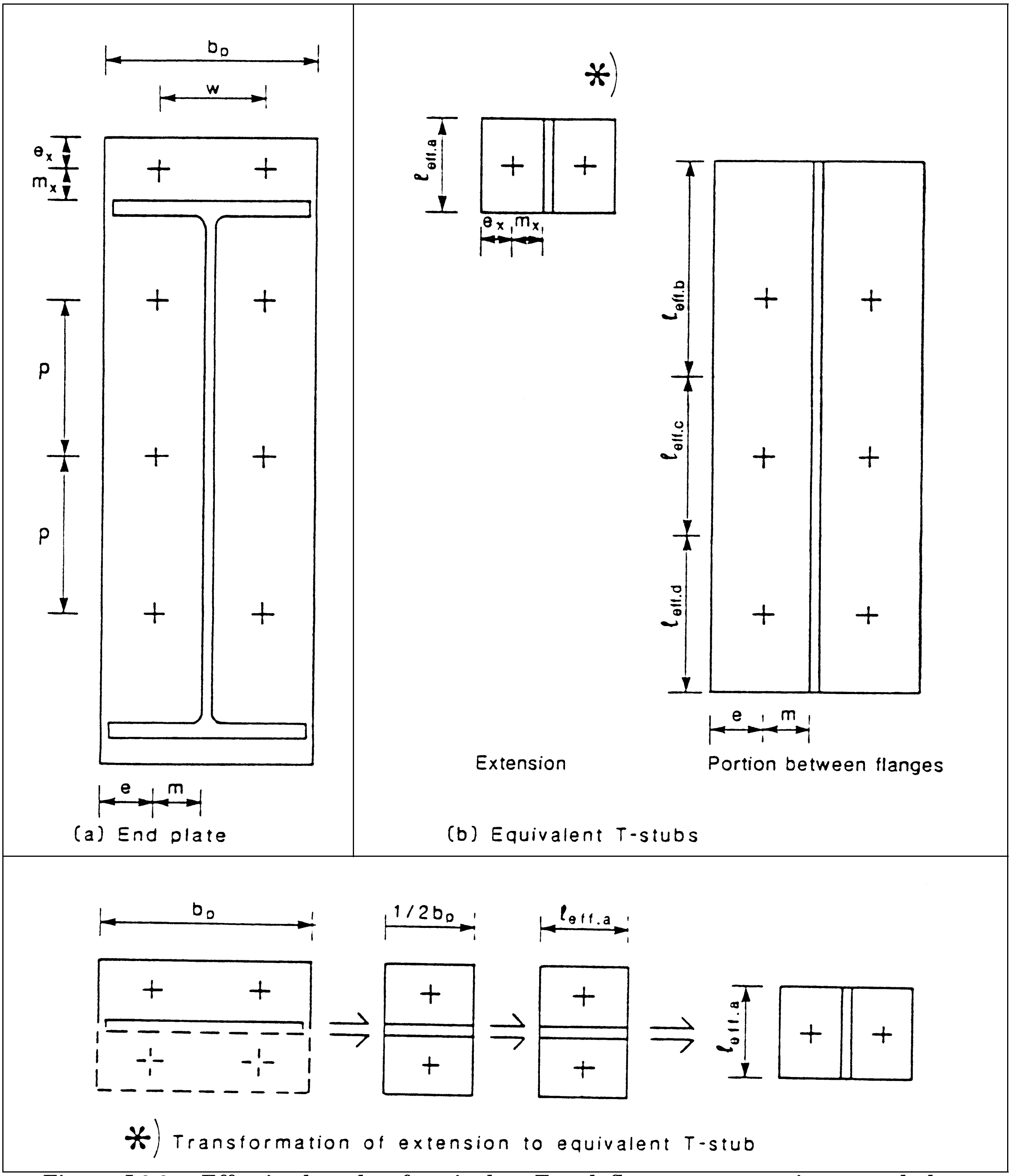


Figure J.3.8 — Effective lengths of equivalent T-stub flanges representing an end plate

J.3.4.6 *Unstiffened column web*

(1) *The design resistance of an unstiffened column web subject to a transverse tensile force is given by:*

$$F_{t.Rd} = f_{yc}\, t_{wc}\, b_{eff}/\gamma_{M0} \tag{J.9}$$

(2) *In a bolted connection, the effective width of the column web in tension should be taken as equal to the total effective length of the bolt pattern in the tension zone of the connection, obtained from* ***J.3.4.1***.

(3) *An unstiffened column web may be strengthened by adding a supplementary web plate conforming with* ***J.2.2****, see* ***J.2.3.2****(4).*

J.3.4.7 *Stiffened column web*

(1) *The design resistance of a stiffened column web subject to a transverse tensile force is at least equal to the design resistance of the beam flange, provided that the stiffeners meet the requirements specified in* ***J.2.3.3****(1).*

Procedure J.3.3

Effective design resistances for bolt-rows.

(1) Recalculate the potential design resistance of a column flange, successively omitting the lowest bolt-row. For a stiffened column flange, recalculate the potential design resistance separately for each relevant group of bolt-rows.

(2) Recalculate the potential design resistance of each group of bolt-rows in the beam end-plate, successively omitting the lowest bolt-row.

(3) Take the reduction in resistance due to the omission of a bolt-row in Steps (1) and (2) as its contribution to the total potential design resistance of the flange or end-plate.

(4) For each bolt-row, determine the difference between the potential design resistances of the column flange and the beam end-plate, obtained in Step (3).

(5) Starting from the highest bolt-row, redistribute the resistance values from Step (3) to minimise the differences found in Step (4), provided that:

- resistance is redistributed only within the same group of bolt-rows (i.e. not past a flange or a stiffener)
- the resistance for any individual bolt-row is limited to that obtained using an effective length of $4m + 1{,}25e$ or $2\pi m$, whichever is smaller.

(6) Reduce the values from Step (5) to obtain equilibrium between the design resistances of the column flange and the beam end-plate.

(7) Adopt the resistance values from Step (6) as the effective design resistances for the individual bolt-rows.

J.3.5 ***Resistance of compression zone***

J.3.5.1 *Unstiffened column web*

(1) *The design crushing resistance of an unstiffened column web subject to a transverse compression force is given by:*

$$F_{c.Rd} = f_{Yc}\, t_{wc}\, [1{,}25 - 0{,}5\, \gamma_{M0}\, \sigma_{n.Ed}/f_{Yc}]\, b_{eff}/\gamma_{M0} \qquad (J.15)$$

but $F_{c.Rd} \leq f_{Yc}\, t_{wc}\, b_{eff}/\gamma_{M0}$ *(J.16)*

where $\sigma_{n.Ed}$ *is the maximum compressive normal stress in the web of the column due to axial force and bending.*

(2) *In a bolted connection, the effective width of the column web in compression is given by:*

- *for a rolled I or H section column:*

$$b_{eff} = t_{fb} + 2\sqrt{2}\, a_p + 2t_p + 5(t_{fc} + r_c) \qquad (J.51)$$

- *for a welded I or H section column:*

$$b_{eff} = t_{fb} + 2\sqrt{2}\, a_p + 2t_p + 5(t_{fc} + \sqrt{2}\, a_c) \qquad (J.52)$$

(3) *In addition the resistance of the column web to buckling in a column mode, as indicated in Figure J.2.4, should be verified using* ***5.7.5****.*

(4) *The sway mode shown in Figure J.2.4(b) should normally be prevented by constructional restraints.*

(5) *An unstiffened column web can be strengthened by adding a supplementary web plate conforming with* ***J.2.2****, see* ***J.2.4.1****(6).*

J.3.5.2 *Stiffened column web*

(1) *The design resistance of a stiffened column web subject to a transverse compression force is at least equal to the design resistance of the beam flange, provided that the stiffeners meet the requirements specified in* ***J.2.3.3****(1).*

J.3.6 ***Resistance of shear zone***

J.3.6.1 *Unstiffened column web panel*

(1) *The design plastic shear resistance of an unstiffened column web panel subject to a shear force (see Figure J.2.5) is given by:*

$$V_{p\ell.Rd} = [f_{yc}\, A_v/\sqrt{3}]/\gamma_{M0} \qquad (J.17)$$

where A_v *is the shear area of the column as given in* ***5.4.6****(2).*

(2) *In addition the shear buckling resistance should be checked, see* ***5.4.6****(7).*

(3) *An unstiffened column web may be strengthened by adding a supplementary web plate conforming with* ***J.2.2****.*

(4) *In calculating the design shear resistance of a web panel with a supplementary web plate, its shear area* A_v *may be increased by* $b_s t_{wc}$*. No further increase in* A_v *should be made if supplementary web plates are added on both sides of the web.*

J.3.6.2 *Stiffened column web panel*

(1) *When diagonal web stiffeners (see Figure J.2.6) are used to increase the shear resistance of a column web, they should be designed to resist the tensile and compressive forces transmitted to the column by the flanges of the beams.*

(2) *The welds between the stiffeners and the column flanges should be designed to resist the forces in the stiffeners.*

(3) *The welds between the stiffeners and the column web should be treated as nominal.*

J.3.7 ***Rotational stiffness***

(1) *The rotational stiffness of a bolted end-plate beam-to-column connection may be approximated by:*

$$S_j = \frac{E h_1^2 t_{wc}}{\sum \frac{\mu_i}{k_i} \left[\frac{F_i}{F_{i.Rd}} \right]^2} \qquad (J.53)$$

where:

- S_j *is the secant stiffness with respect to a particular moment M in the connection* ($M \le M_{Rd}$)
- M_{Rd} *is the design moment resistance of the connection*
- h_1 *is the distance from the first bolt-row below the tension flange of the beam, to the centre of resistance of the compression zone, except as noted in (8)*
- μ_1 *is the modification factor, see (5) and (6) below*
- k_i *is the stiffness factor for component i, see (2) to (4)*
- F_i *is the force in component i of the connection due to the moment M.*
- $F_{i.Rd}$ *is the design resistance of component i of the connection*

For components 2 to 6 the value of F_i *should not be taken as less than* $F_{i.Rd}/1,5$.

(2) *In an unstiffened connection the stiffness factors* k_i *should be taken as follows:*

Column web, shear zone: $k_1 = 0,24$

Column web, tension zone: $k_2 = 0,8$

Column web, compressive zone: $k_3 = 0,8$

Column flange, tension zone:

$$k_4 = \frac{t_{fc}^3}{4m^2 t_{wc}}$$

Bolts, tension zone:

$$k_5 = \frac{2A_s}{\ell_b t_{wc}}$$

End plate, tension zone:

$$k_6 = \frac{t_e^3}{12\lambda_2 m^2 t_{wc}}$$

but

$$k_6 \ge \frac{t_e^3}{4m^2 t_{wc}}$$

where ℓ_b *is the elongation length of the bolt, which may be taken as the total grip length (thickness of material plus washers) plus half the sum of the height of the bolt head and the height of the nut.*

and λ_2 *is as defined in Figure J.3.7.*

(3) *Where the column has a stiffener in the tension zone:*

$$k_4 = \frac{t_{fc}^3}{12\lambda_2 m^2 t_{wc}} \quad \text{but} \quad k_4 \ge \frac{t_{fc}^3}{4m^2 t_{wc}}$$

(4) *For any other stiffened component, the relevant stiffness factor should be taken as infinity.*

(5) *For i = 1, 2 or 3 the modification factor* μ_i *should be taken as 1.*

(6) *For i = 4, 5 or 6 the modification factor* μ_i *should be obtained from:*

$$\mu_1 = \frac{h_1 F_{1.Rd}}{M_{Rd}}$$

where $F_{1.Rd}$ *is the force in the first row of bolts below the tension flange of the beam, corresponding to design moment resistance* M_{Rd}*, except as noted in (8).*

(7) *In an extended end plate connection the rotational stiffness S_{je} based on the end plate extension should also be calculated, and the larger value S_j or S_{je} should be adopted as the rotational stiffness of the connection.*

(8) *When calculating S_{je} the distance h_1 should be measured from the bolt-row in the end plate extension to the centre of resistance of the compression zone and $F_{1.Rd}$ should be taken as the force in that bolt-row corresponding to M_{Rd}. The stiffness factor k_6 should then be taken as:*

$$k_6 = \frac{t_e^3}{4m_x^2\, t_{wc}}$$

where m_x is as defined in Figure J.3.8

(9) *A bolted end-plate connection may be assumed to be a rigid connection when both of the following, conditions are satisfied:*

a) *The column has web stiffeners in both the tension zone and the compression zone.*

b) *The moment resistance is determined using Procedure J.3.2*

J.3.8 *Rotation capacity*

(1) *A bolted beam-to-column connection in which the moment resistance is governed by the resistance of the shear zone may be assumed to have adequate rotation capacity for plastic analysis.*

(2) *A bolted beam-to-column connection in which the moment resistance is governed by the resistance of the tension zone, may be assumed to have adequate rotation capacity for plastic analysis if adequate deformation capacity is available throughout the tension zone, either in the column flange or in the beam end-plate.*

(3) *The criterion given in (2) may be assumed to be satisfied if, for each bolt-row, the resistance of at least one component (column flange or beam end-plate) is governed by Mode 1 failure, see **J.3.3**. This condition is met if, for each bolt-row, whichever component gives the lower value of β also satisfies the criterion:*

$$\beta \le \frac{2\lambda}{1 + 2\lambda} \tag{J.54}$$

*in which β and λ are as defined in **J.3.3**(4).*

(4) *If Mode 2 failure governs, that is if the lower value of β satisfies the condition:*

$$\frac{2\lambda}{1 + 2\lambda} < \beta < 2 \tag{J.55}$$

then the rotation capacity ϕ_{Cd} may be obtained from:

$$\phi_{Cd} = \frac{10{,}6 - 4\beta_{cr}}{1{,}3\ h_1} \tag{J.56}$$

where h_1 *is the distance (in mm) from the first bolt-row below the tension flange of the beam to the centre of resistance of the compression zone, except as noted in (5).*

and β_{cr} *is the value of β for the component with the lower value of $F_{t.Rd}/\Sigma B_{t.Rd}$ [see **J.3.3**(4)].*

(5) *The criteria given in (2) to (4) also apply to extended end plate connections, provided that the end plate extension has sufficient deformation capacity. This may be assumed to be satisfied if Mode 1 failure governs in the end plate extension. In an extended end plate connection, the distance h_1 in expression (J.56) should be measured from the bolt-row in the end plate extension to the centre of resistance of the compression zone, but the end plate extension should be excluded in determining β_{cr}.*

(6) *Unless the connection is classified as full-strength (as defined in **6.4.3.2**) the lower value of β should not exceed 1,8.*

Annex K (normative) Hollow section lattice girder connections

K.1 General

(1) *This Annex gives detailed application rules to determine the static resistances of uniplanar joints in lattice structures composed of rectangular, circular or square hollow sections, or combinations of these with open sections.*

(2) *The static resistances of the joints are expressed in terms of maximum design axial resistances for the brace members.*

(3) *These rules are valid for both hot finished hollow sections as defined in* ***3.2.2*** *and for cold formed hollow sections as defined in* ***3.2.3****.*

(4) *The nominal yield strength of hot finished hollow sections and the nominal yield strength of the basic material of cold formed hollow sections should not exceed 355 N/mm*^{2}*.*

(5) *The requirements given in* ***6.10.1*** *should be satisfied.*

(6) *The nominal wall thickness of hollow sections should be limited to a minimum of 2,5 mm.*

(7) *The nominal wall thickness of a hollow section chord should not be greater than 25 mm unless special measures have been taken to ensure that the through thickness properties of the material will be adequate.*

(8) *The partial safety factor for joint resistance should be taken as:*

$$\gamma_{Mj} = \boxed{1,1}$$

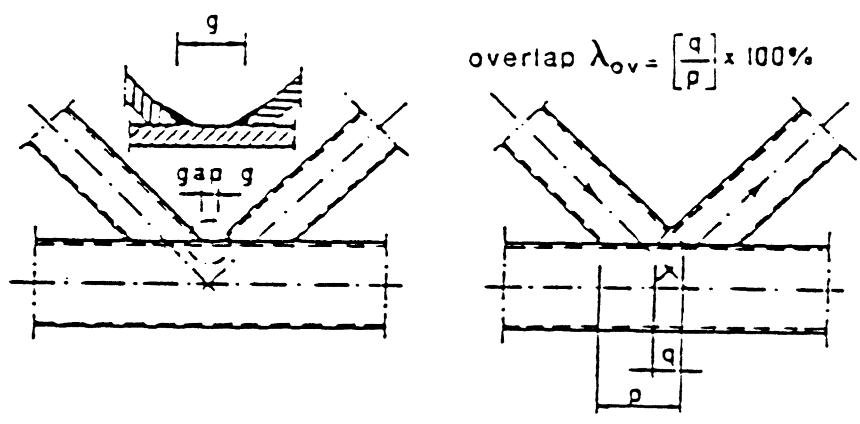
K.2 Definitions

(1) *In this Annex, a uniplanar joint in a lattice structure means a connection between members which are situated in a single plane and which transmit primarily axial forces.*

(2) *The gap g is defined as the distance, measured along the length of the connecting face of the chord, between the toes of the adjacent brace members, see Figure K.1(a).*

(3) *The overlap* λ_{ov} *is defined as (q/p) × 100 % as shown in Figure K.1(b).*

(4) *The symbols used in the tables in this Annex are defined in* ***K.9****.*



(a) Definition of gap (b) Definition of overlap

Figure K.1 — Gap and overlap of joints

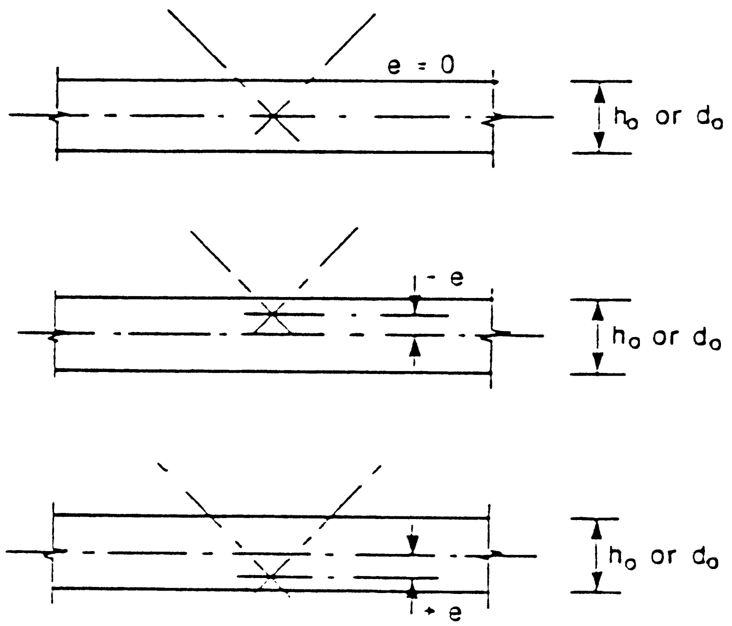


Figure K.2 — Eccentricity of joints

K.3 Field of application

(1) *The application rules given in this Annex may be used only where all the following conditions are satisfied:*

a) *The members should have Class 1 or Class 2 cross-sections.*

b) *The angles between the chords and the brace members and between adjacent brace members should not be less than 30°.*

c) *Moments resulting from eccentricities may be neglected in calculating the resistance of the joint, provided that the eccentricities are within the following limits:*

- $-0{,}55\, d_o \leq e \leq 0{,}25\, d_o$ *(K.1a)*
- $-0{,}55\, h_o \leq e \leq 0{,}25\, h_o$ *(K.1b)*

where e *is the eccentricity, as defined in Figure K.2*

d_o *is the diameter of the chord*

h_o *is the depth of the chord, in the plane of the lattice girder.*

(2) *The members that meet at a joint should have their ends prepared in such a way that their cross-sectional shape is not modified.*

(3) *In gap type joints, the gap between the brace members should not be less than* $(t_1 + t_2)$*, to ensure that the clearance is adequate for forming satisfactory welds.*

(4) *In overlap type joints, the overlap should be sufficient to ensure that the interconnection of the brace members is adequate for satisfactory shear transfer from one brace to the other.*

(5) *Where overlapping brace members have different thicknesses, the thinner member should overlap the thicker member.*

(6) *Where overlapping brace members are of different strength grades, the member with the lower yield strength should overlap the member with the higher yield strength.*

K.4 Analysis

(1) *The axial force distribution in a lattice girder may be determined on the assumption that the members are connected by pinned joints.*

(2) *Secondary moments in the joints, caused by the actual bending stiffness of the joints, may be neglected, provided that:*

- *the joint geometry is within the range of validity specified in Table K.6.1, Table K.7.1 or Table K.8.1 as appropriate, and*
- *the ratio of the system length to the members depth in the plane of the girder is not less than:*
  - *12 for chord members, and*
  - *24 for brace members.*

(3) *Eccentricities within the limits given, in section* ***K.3*** *may be neglected.*

(4) *For fatigue assessments see Chapter 9.*

K.5 Welds

(1) *In welded joints, the connection should normally be established around the entire perimeter of the hollow section by means of a butt weld, a fillet weld, or combinations of the two. However in partially overlapping joints the hidden part of the connection need not be welded.*

(2) *The design resistance of the weld per unit length of the perimeter should not normally be less than the design tension resistance of the cross-section of the member per unit length of the perimeter.*

(3) *The required throat thickness should be determined from* ***6.6.5****.*

(4) *The criterion given in (2) will be satisfied if the throat thickness of a fillet weld satisfies the following:*

- *for steel to EN 10025:*
  - *for Fe 360:* $a/t \geq 0{,}84\alpha$ (K.2a)
  - *for Fe 430:* $a/t \geq 0{,}87\alpha$ (K.2b)
  - *for Fe 510:* $a/t \geq 1{,}01\alpha$ (K.2c)
- *for steel to prEN 10113:*
  - *for Fe E 275:* $a/t \geq 0{,}91\alpha$ (K.2d)
  - *for Fe E 355:* $a/t \geq 1{,}05\alpha$ (K.2e)

When $\gamma_{Mj} = 1{,}1$ *and* $\gamma_{Mw} = 1{,}25$ *the value of* α *is 1,0. Otherwise* α *should be determined from:*

$$\alpha = \frac{1{,}1}{\gamma_{Mj}} \times \frac{\gamma_{Mw}}{1{,}25} \qquad (K.3)$$

(5) *The criterion given in (2) may be waived where smaller weld sizes can be justified with regard both to resistance and to deformation capacity and/or rotation capacity.*

K.6 Welded joints between circular hollow sections

(1) *The design values of the internal axial forces both in the brace members and in the chords at the ultimate limit state should not exceed the design resistances of the members determined from Chapter 5.*

(2) *The design values of the internal axial forces in the brace members at the ultimate limit state should also not exceed the design resistances of the joints.*

(3) *Provided that the geometry of the joints is within the range of validity given in Table K.6.1, the design resistances of the joints should be determined using the formulae given in Table K.6.2.*

(4) *For joints outside the range of validity given in Table K.6.1 a more detailed analysis should be made. This analysis should also take account of the secondary moments in the joints caused by the bending stiffness of the joints.*

Table K.6.1 — Range of validity for welded joints between circular hollow sections

$0{,}2 \leq \frac{d_i}{d_o} \leq 1{,}0$
$5 \leq \frac{d_i}{2t_i} \leq 25$
$5 \leq \frac{d_o}{2t_o} \leq 25$ $5 \leq \frac{d_o}{2t_o} \leq 20$ for X-joints
$\lambda_{ov} \geq 25\ \%$
$g \geq t_1 + t_2$

K.7 Welded joints between hollow section brace members and square or rectangular hollow section chords

K.7.1 *General*

(1) *The design values of the internal axial forces both in the brace members and in the chords at the ultimate limit state should not exceed the design resistances of the members determined from Chapter 5.*

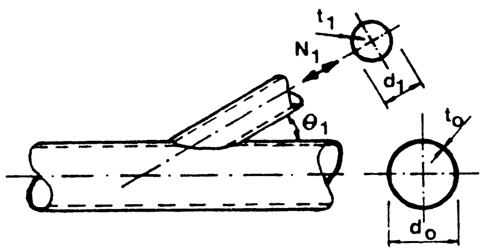
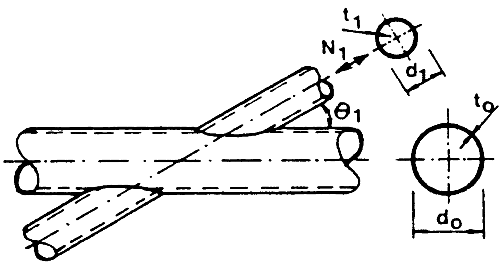
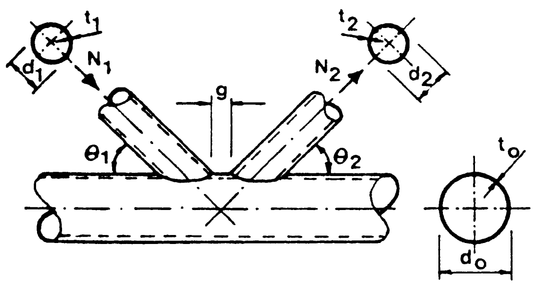
(2) *The design values of the internal axial forces in the brace members at the ultimate limit state should also not exceed the design resistances of the joints.*

K.7.2 *Square or circular brace members and square chords*

(1) *Provided that the geometry of the joints is within the range of validity given in Table K.7.1, the design resistances of the joints should be determined using the formulae given in Table K.7.2.*

(2) *For joints outside the range of validity given in Table K.7.1 refer to clause* ***K.7.3****.*

Table K.6.2 — Design resistances of welded joints between circular hollow sections

Type of joint	Design resistance (i = 1 or 2)
T and Y joints	Chord plastification
	$N_{1.Rd} = \frac{f_{yo}\ t_o^2}{\sin\theta_1}(2,8 + 14,2\beta^2)\ \gamma^{0,2}\ k_p \left[\frac{1,1}{\gamma_{Mj}}\right]$
X joints	Chord plastification
	$N_{1.Rd} = \frac{f_{yo}\ t_o^2}{\sin\theta_1}\ \frac{5,2}{(1 - 0,81\beta)}\ k_p \left[\frac{1,1}{\gamma_{Mj}}\right]$
K and N gap or overlap joints	Chord plastification
	$N_{1.Rd} = \frac{f_{yo}\ t_o^2}{\sin\theta_1}[1,8 + 10,2\ d_1/d_o]\ k_p\ k_g \left[\frac{1,1}{\gamma_{Mj}}\right]$ $N_{2.Rd} = \frac{\sin\theta_1}{\sin\theta_2}\ N_{1.Rd}$
T, Y and X joints and K, N and KT joints with a gap When $d_i \leq d_o - 2t_o$	Punching shear $N_{i.Rd} = \frac{f_{yo}}{\sqrt{3}}\ t_o\ \pi\ d_i\ \frac{1 + \sin\theta_i}{2\sin^2\theta_i} \left[\frac{1,1}{\gamma_{Mj}}\right]$
Functions	
$k_p = 1,0$ $k_p = 1 - 0,3\ n_p\ (1 + n_p)$ but $k_p \leq 1,0$	for $n_p \leq 0$ (tension) for $n_p > 0$ (compression)
$k_g = \gamma^{0,2}\left[1 + \frac{0,024\gamma^{1,2}}{\exp(0,5g/t_o - 1,33) + 1}\right]$	(see Figure K.3)

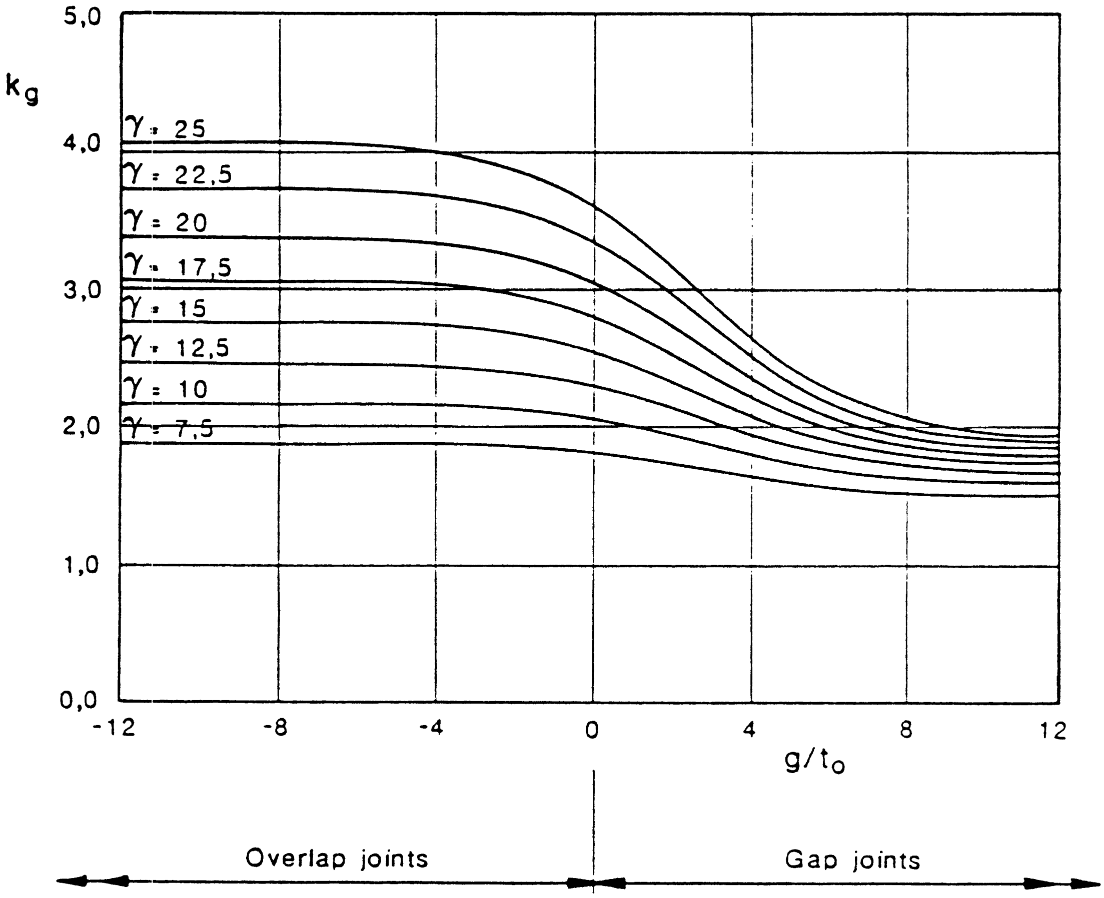


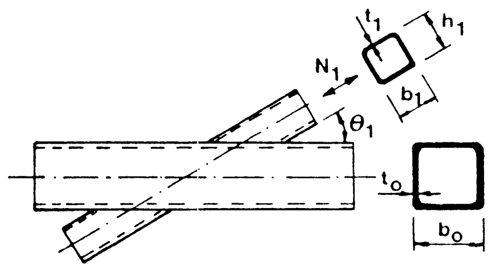
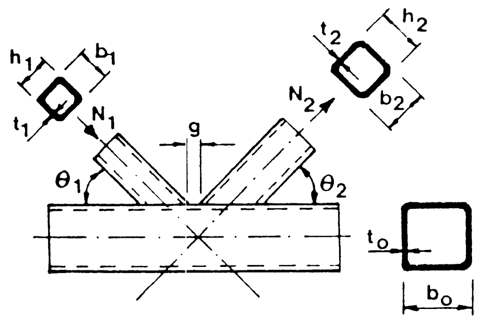
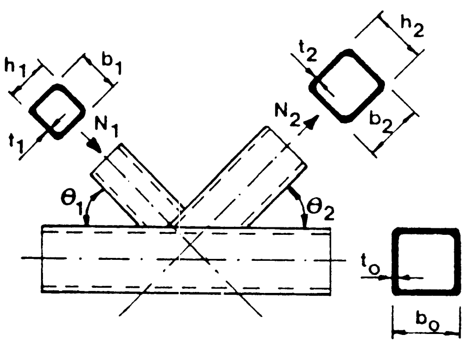
Figure K.3 — Values of factor k_g

Table K.7.1 — Range of validity for welded joints between square or circular hollow section brace members and square hollow section chords[a]

Type of joint	Joint parameters [i = 1 or 2, j = overlapped brace]					
	$\frac{b_i}{b_o}$ or $\frac{d_i}{b_o}$	$\frac{b_i}{t_i}$ or $\frac{d_i}{t_i}$ Compression	$\frac{b_i}{t_i}$ or $\frac{d_i}{t_i}$ Tension	$\frac{b_o}{t_o}$	$\frac{b_1 + b_2}{2\,b_1}$ or $\frac{b_i}{b_j}$ and $\frac{t_i}{t_j}$	Gap or Overlap
T, Y or X joint	$0{,}25 \leq \frac{b_i}{b_o} \leq 0{,}85$	$\frac{b_i}{t_i} \leq 1{,}25 \sqrt{\frac{E}{f_{yi}}}$		$10 \leq \frac{b_o}{t_o} \leq 35$		
K gap joint N gap joint	$\frac{b_i}{b_o} \geq 0{,}1 + 0{,}01 \frac{b_o}{t_o}$ and $\frac{b_i}{b_o} \geq 0{,}35$	and $\frac{b_i}{t_i} \leq 35$	$\frac{b_i}{t_i} \leq 35$	$15 \leq \frac{b_o}{t_o} \leq 35$	$0{,}6 \leq \frac{b_1 + b_2}{2b_1} \leq 1{,}3$	$\frac{g}{b_o} \geq 0{,}5(1 - \beta)$ but $\frac{g}{b_o} \leq 0{,}5(1 - \beta)$ and $g \geq t_1 + t_2$
K overlap joint N overlap joint	$\frac{b_i}{b_o} \geq 0{,}25$	$\frac{b_i}{t_i} \leq 1{,}1 \sqrt{\frac{E}{f_{yi}}}$		$\frac{b_o}{t_o} \leq 40$	$\frac{t_i}{t_j} \leq 1{,}0$ $\frac{b_i}{b_j} \geq 0{,}75$	$25\ \% \leq \lambda_{ov} \leq 100\ \%$
Circular brace member	$0{,}4 \leq \frac{d_i}{b_o} \leq 0{,}8$	$\frac{d_i}{t_i} \leq 1{,}5 \sqrt{\frac{E}{f_{yi}}}$	$\frac{d_i}{t_i} \leq 50$	As above but with d_i replacing b_i		

[a] NOTE Outside these parameter ranges the resistance of the joint may be determined as for a joint with a rectangular chord section, see clause **K.7.3.**

Table K.7.2 — Design resistances of welded joints square or circular hollow section brace members and square hollow sections chords

Type of joint	Design resistance (i = 1 or 2, j = overlapped brace)
T, Y and X joints	Chord face yielding $\beta \leq 0,85$
	$N_{1.Rd} = \dfrac{f_{yo}\, t_o^2}{(1-\beta)\sin\theta_1}\left[\dfrac{2\beta}{\sin\theta_1} + 4\,(1-\beta)^{0,5}\right] k_n \left[\dfrac{1,1}{\gamma_{Mj}}\right]$
K and N gap joints	Chord face yielding $\beta \leq 1,0$
	$N_{i.Rd} = \dfrac{8,9\, f_{yo}\, t_o^2}{\sin\theta_i}\left[\dfrac{b_1 + b_2}{2b_o}\right]\gamma^{0,5}\, k_n \left[\dfrac{1,1}{\gamma_{Mj}}\right]$
K and N overlap joints[a]	Effective width $25\ \% \leq \lambda_{ov} < 50\ \%$
	$N_{i.Rd} = f_{yi}\, t_i \left[\dfrac{\lambda_{ov}}{50}(2h_i - 4t_i) + b_{eff} + b_{e.ov}\right]\left[\dfrac{1,1}{\gamma_{Mj}}\right]$
	Effective width $50\ \% \leq \lambda_{ov} \leq 80\ \%$
	$N_{i.Rd} = f_{yi}\, t_i\, [2h_i - 4t_i + b_{eff} + b_{e.ov}]\left[\dfrac{1,1}{\gamma_{Mj}}\right]$
	Effective width $\lambda_{ov} \geq 80\ \%$
	$N_{i.Rd} = f_{yi}\, t_i\, [2h_i - 4t_i + b_i + b_{e.ov}]\left[\dfrac{1,1}{\gamma_{Mj}}\right]$
Circular braces	Multiply the above resistances by $\pi/4$. Replace b_1 and h_1 by d_1 and replace b_2 and h_2 by d_2.
Functions	
for $n \leq 0$ (tension): $k_n = 1,0$	for $n \geq 0$ (compression): $k_n = 1,3 - \dfrac{0,4n}{\beta}$ but $k_n \leq 1,0$
$b_{eff} = \dfrac{10}{b_o/t_o}\,\dfrac{f_{yo}\, t_o}{f_{yi}\, t_i}\, b_i$ but $b_{eff} \leq b_i$	$b_{e.ov} = \dfrac{10}{b_j/t_j}\,\dfrac{f_{yj}\, t_j}{f_{yi}\, t_i}\, b_i$ but $b_{e.ov} \leq b_i$

[a] Only the overlapping brace need be checked. The brace member efficiency (ie. the design resistance of the joint divided by the design plastic resistance of the brace) for the overlapped brace should be taken as not more than that of the overlapping brace.

K.7.3 ***Rectangular sections***

(1) *The design resistances of joints between rectangular hollow sections, and of joints between square hollow sections outside the range of validity of Table K.7.1, should be based on the following criteria as applicable:*

a) *Plastic failure of the chord face or the chord cross section.*

b) *Crack initiation leading to rupture of the bracings from the chord (punching shear).*

c) *Cracking in the welds or in the bracings (effective width).*

d) *Chord wall bearing or local buckling under the compression bracing.*

e) *Local buckling in the compressive areas of the members.*

f) *Shear failure of the chord.*

(2) *Figure K.4(a) to (f) illustrates the modes of failure relevant to criteria a) to f) given in (1).*

K.8 Welded joints between hollow section brace members and an I or H section chord

(1) *The design values of the internal axial forces both in the brace members and in the chords at the ultimate limit state should not exceed the design resistances of the members determined from Chapter 5.*

(2) *In gap-type joints, the design resistances of the chords allowing for the shear force transferred between the brace members by the chords should be determined from* ***5.4.9****, neglecting the associated secondary moments, as follows:*

- *when* $V_{Sd}/V_{p\ell.Rd} \leq 0{,}5$: $N_{o.Rd} = f_{yo}\, A_o/\gamma_{M0}$ *(K.4)*
- *when* $V_{Sd}/V_{p\ell.Rd} > 0{,}5$ *but* $V_{Sd}/V_{p\ell.Rd} \leq 1{,}0$:

$$N_{o.Rd} = f_{yo}\,[A_o - A_v\,(2V_{Sd}/V_{p\ell.Rd} - 1)^2]/\gamma_{M0} \qquad (K.5)$$

(3) *The design values of the internal axial forces in the brace members at the ultimate limit state should also not exceed the design resistances of the joints.*

(4) *Provided that the geometry of the joints is within the range of validity given in Table K.8.1, the design resistances of the joints should be determined using the formulae given in Table K.8.2.*

(5) *For joints outside the range of validity given in Table K.8.1 a more detailed analysis should be made. This analysis should also take account of the secondary moments in the joints caused by the bending stiffness of the joints.*

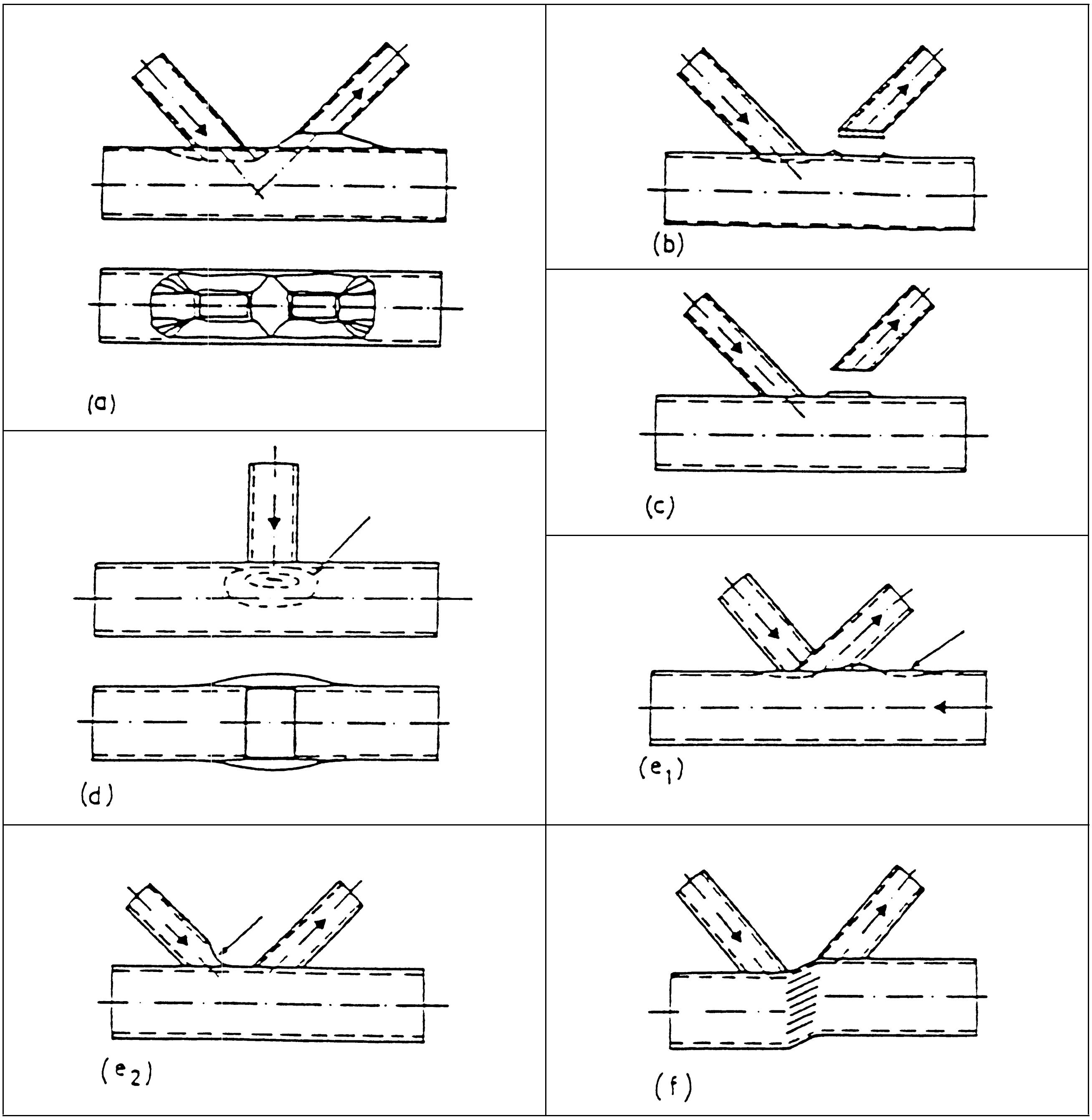
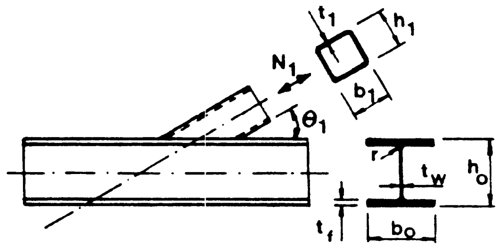
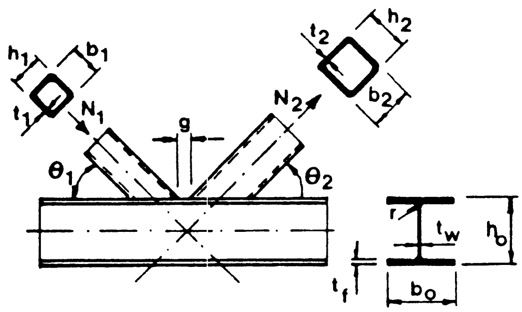
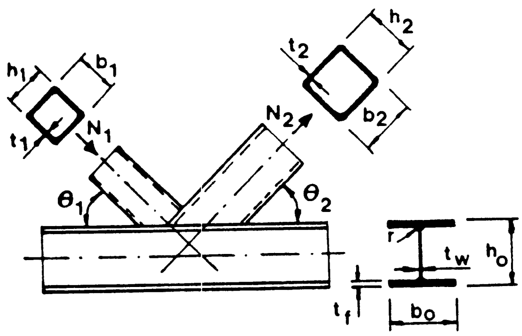


Figure K.4 — Modes of failure — rectangular sections

Table K.8.1 — Range of validity for welded joints between hollow section brace members and I or H section chords

Type of joint	Joint parameter [i = 1 or 2, j = overlapped brace]					
	$\frac{h_i}{b_i}$	$\frac{b_j}{b_i}$	$\frac{d_w}{t_w}$	$\frac{b_o}{t_o}$	$\frac{b_i}{t_i}, \frac{h_i}{t_i}, \frac{d_i}{t_i}$	
					Compression	**Tension**
X-joint	$0{,}5 \leq \frac{h_i}{b_i} \leq 2{,}0$		$\frac{d_w}{t_w} \leq 1{,}2 \sqrt{\frac{E}{f_{yo}}}$ and $d_w \leq 400$ mm	$\frac{b_o}{t_o} \leq 0{,}75 \sqrt{\frac{E}{f_{yo}}}$	$\frac{h_i}{t_i} \leq 1{,}1 \sqrt{\frac{E}{f_{yi}}}$ $\frac{b_i}{t_i} \leq 1{,}1 \sqrt{\frac{E}{f_{yi}}}$ $\frac{d_i}{t_i} \leq 1{,}5 \sqrt{\frac{E}{f_{yi}}}$	$\frac{h_i}{t_i} \leq 35$ $\frac{b_i}{t_i} \leq 35$ $\frac{d_i}{t_i} \leq 50$
T-joint Y-joint	$\frac{h_i}{b_i} = 1{,}0$		$\frac{d_w}{t_w} \leq 1{,}5 \sqrt{\frac{E}{f_{yo}}}$ and $d_w \leq 400$ mm			
K — gap joint N — gap joint						
K — overlap joint N — overlap joint	$0{,}5 \leq \frac{h_i}{b_i} \leq 2{,}0$	$\frac{b_j}{b_i} \geq 0{,}75$				

Table K.8.2 — Design resistances of welded joints between hollow section brace members and I or H section chords

<table>
<tr><th>Type of joint</th><th colspan="2">Design resistance (i = 1 or 2, j = overlapped brace)</th></tr>
<tr><td>T, Y and X joints</td><td colspan="2">Chord web yielding</td></tr>
<tr><td rowspan="3"></td><td colspan="2">$N_{1.Rd} = \frac{f_{yo}\, t_w\, b_w}{\sin \theta_1} \left[\frac{1,1}{\gamma_{Mj}}\right]$</td></tr>
<tr><td colspan="2">Effective width</td></tr>
<tr><td colspan="2">$N_{1.Rd} = 2 f_{y1}\, t_1\, b_{eff} \left[\frac{1,1}{\gamma_{Mj}}\right]$</td></tr>
<tr><td>K and N gap joints</td><td>Chord web stability</td><td rowspan="4">No effective width check required if:
$g/t_f \geq 20 - 28\beta$
$\beta \leq 1,0 - 0,03\gamma$ and
$0,75 \leq d_1/d_2 \leq 1,33$ for CHS
$0,75 \leq b_1/b_2 \leq 1,33$ for RHS</td></tr>
<tr><td rowspan="5"></td><td>$N_{i.Rd} = \frac{f_{yo}\, t_w\, b_w}{\sin \theta_1} \left[\frac{1,1}{\gamma_{Mj}}\right]$</td></tr>
<tr><td>Effective width</td></tr>
<tr><td>$N_{i.Rd} = 2 f_{yi}\, t_i\, b_{eff} \left[\frac{1,1}{\gamma_{Mj}}\right]$</td></tr>
<tr><td colspan="2">Chord shear</td></tr>
<tr><td colspan="2">$N_{i.Rd} = \frac{f_{yo}\, A_v}{\sqrt{3}\, \sin \theta_i} \left[\frac{1,1}{\gamma_{Mj}}\right]$</td></tr>
<tr><td>K and N overlap joints[a]</td><td colspan="2">Effective width $25\ \% \leq \lambda_{ov} < 50\ \%$</td></tr>
<tr><td rowspan="5"></td><td colspan="2">$N_{i.Rd} = f_{yi}\, t_i \left[\frac{\lambda_{ov}}{50}(2h_i - 4t_i) + b_{eff} + b_{e.ov}\right] \left[\frac{1,1}{\gamma_{Mj}}\right]$</td></tr>
<tr><td colspan="2">Effective width $50\ \% \leq \lambda_{ov} < 80\ \%$</td></tr>
<tr><td colspan="2">$N_{i.Rd} = f_{yi}\, t_i\, [2h_i - 4t_i + b_{eff} + b_{e.ov}] \left[\frac{1,1}{\gamma_{Mj}}\right]$</td></tr>
<tr><td colspan="2">Effective width $\lambda_{ov} \geq 80\ \%$</td></tr>
<tr><td colspan="2">$N_{i.Rd} = f_{yi}\, t_i\, [2h_i - 4t_i + b_i + b_{e.ov}] \left[\frac{1,1}{\gamma_{Mj}}\right]$</td></tr>
<tr><td colspan="3">Functions</td></tr>
<tr><td>RHS $\begin{cases} b_w = \frac{h_i}{\sin \theta_i} + 5(t_f + r) \\ b_w \leq 2t_i + 10(t_f + r) \end{cases}$
CHS $\begin{cases} b_w = \frac{d_i}{\sin \theta_i} + 5(t_f + r) \\ b_w \leq 2t_i + 10(t_f + r) \end{cases}$</td><td colspan="2">$A_v = A_o - (2 - \alpha) b_o\, t_f + (t_w + 2r) t_f$
for RHS $\alpha = \left[\frac{1}{1 + \frac{4g^2}{3t_f^2}}\right]^{0,5}$
for CHS brace $\alpha = 0$</td></tr>
<tr><td>$b_{eff} = t_w + 2r + 7 \frac{f_{yo}}{f_{yi}} t_f$ but $b_{eff} \leq b_i$</td><td colspan="2">$b_{e.ov} = \frac{10}{b_j/t_j} \frac{f_{yj}\, t_j}{f_{yi}\, t_i} b_i$ but $b_{e.ov} \leq b_i$</td></tr>
<tr><td colspan="3">[a] Only the overlapping brace need be checked. The brace member efficiency (i.e. the design resistance of the joint divided by the design plastic resistance of the brace) for the overlapped brace should be taken as not more than that of the overlapping brace.</td></tr>
</table>

K.9 Symbols used in tables

A_i	*is*	*the cross-sectional area of member i*
A_v	*is*	*the shear area of the chord*
E	*is*	*the elastic modulus of steel*
N_i	*is*	*the axial force in member i*
$N_{i.Rd}$	*is*	*the design resistance of the joint for the axial force in member i*
a	*is*	*the throat thickness of a fillet weld*
b_i	*is*	*the external width of a square or rectangular hollow section member i (i = 0, 1 or 2)*
b_{eff}	*is*	*the effective width for a brace to chord connection*
$b_{e.ov}$	*is*	*the effective width for an overlapping brace to overlapped brace connection*
b_w	*is*	*the effective width for the web of the chord*
d_i	*is*	*the diameter of a circular hollow section member i (i = 0, 1 or 2)*
d_w	*is*	*the depth of the web of an I or H section chord*
e	*is*	*the eccentricity of a joint*
f_{yi}	*is*	*the design value of the yield strength of member i (i = 0, 1 or 2)*
g	*is*	*the gap between the braces of a K or N joint*
h_i	*is*	*the external depth of a section, member i (i = 0, 1 or 2)*
i	*is*	*the integer subscript used to designate a member of a joint, i = 0 denoting a chord and i = 1 and 2 the brace members. In joints with two braces, i = 1 normally denotes the compression brace and i = 2 the tension brace*
i, j	*are*	*integer subscripts used to denote respectively the overlapping brace member and the overlapped brace member.*
k_g, k_p	*are*	*factors defined in Table K.6.2*
k_n	*is*	*a factor defined in Table K.7.2*
n	=	σ_o / f_{yo}
n_p	=	σ_p / f_{yo}
r_o	*is*	*the root radius of an I or H section chord*
t_i	*is*	*the wall thickness of member i (i = 0, 1 or 2)*
t_f	*is*	*the flange thickness of an I or H section*
t_w	*is*	*the web thickness of an I or H section*
α	*is*	*the factor giving the effectiveness of the chord flange for shear*
β	*is*	*the mean brace to chord diameter or width ratio* $\left[\frac{d_1}{d_o}, \quad \frac{d_1 + d_2}{2\, d_o}, \quad \frac{b_1}{b_o} \text{ or } \frac{b_1 + b_2}{2\, b_o}\right]$
γ	*is*	*the ratio of the chord width or diameter to twice its wall thickness* $\left[\frac{d_o}{2\, t_o} \text{ or } \frac{b_o}{2\, t_o}\right]$
Θ_i	*is*	*the included angle between the chord and a brace member i (i = 1 or 2)*
λ_{ov}	*is*	*the overlap ratio, expressed as a percentage* ($\lambda_{ov} = (q/p) \times 100\ \%$)
σ_o	*is*	*the maximum compressive stress in the chord at the joint, due to axial force and bending moment*
σ_p	*is*	*the value of* σ_o *excluding the stress due to the horizontal components of the forces in the braces at that joint*
CHS	*is*	*used as an abbreviation for "circular hollow section"*

RHS   *is*   *used as an abbreviation for "rectangular hollow section", which in this context also includes a square hollow section.*

K, N, T, X, Y and KT joints are abbreviated descriptions for the types of joints shown in Figure K.5.

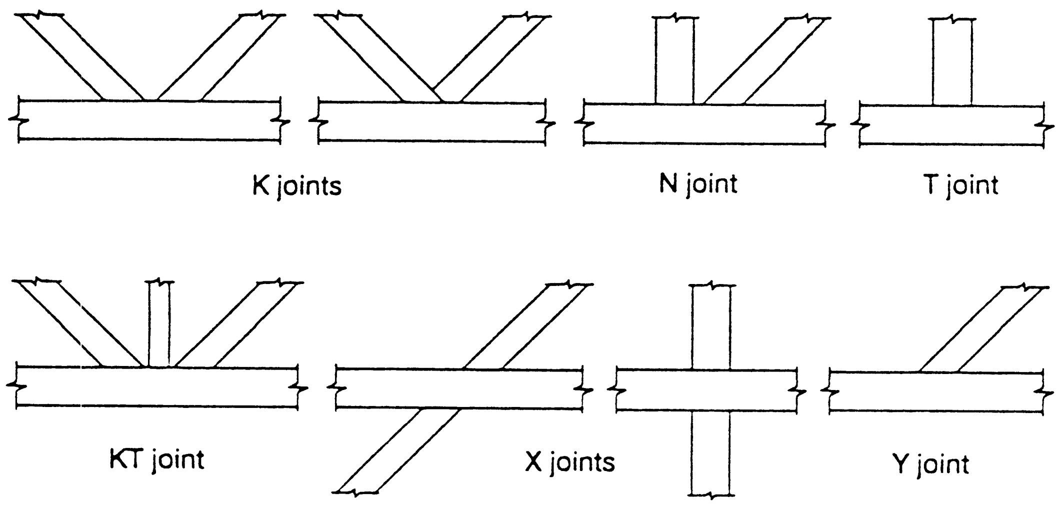


Figure K.5 — Types of joints

Annex L (normative)
Design of column bases

L.1 Base plates

(1) *Columns should be provided with adequate steel base plates to distribute the compression forces in compressed parts of the column over a bearing area, such that the bearing pressure does not exceed the design strength f_j of the joint (grout and concrete).*

(2) *The resistance moment m_{Rd} per unit length of a yield line in the base plate, either in the compression region or in the tension region, should be taken as:*

$$m_{Rd} = \frac{t^2 f_y}{6 \gamma_{M0}} \qquad (L.1)$$

(3) *The forces transferred to the foundation from the compression elements of the column should be assumed to be spread uniformly by the base plate as shown in Figure L.1(a). The pressure on the resulting bearing area should not exceed the bearing strength f_j of the joint and the additional bearing width c should not exceed:*

$$c = t \left[\frac{f_y}{3 f_j \gamma_{M0}} \right]^{0,5} \qquad (L.2)$$

where t *is the thickness of the steel base plate*

and f_y *is the yield strength of the steel base plate material*

(4) *Where the projection of the base plate is less than c the effective bearing area should be assumed to be as indicated in Figure L.1(b).*

(5) *Where the projection of the base plate exceeds c the additional projection should be neglected, see Figure L.1(c).*

(6) *The bearing strength of the joint f_j should be determined from:*

$$f_j = \beta_j k_j f_{cd} \tag{L.3}$$

where:	β_j	*is*	*the joint coefficient, which may be taken as 2/3 provided that the characteristic strength of the grout is not less than 0,2 times the characteristic strength of the concrete foundation and the thickness of the grout is not greater than 0,2 times the smallest width of the steel base plate*
	k_i	*is*	*the concentration factor*
	f_{cd}	*is*	*the design value of the concrete cylinder compressive strength of the concrete given by:* $f_{cd} = f_{ck}/\gamma_c$
in which	f_{ck}	*is*	*the characteristic cylinder compressive strength of the concrete determined in conformity with ENV 1992-1-1 Eurocode 2-1.1*
and	γ_c	*is*	*the partial safety factor for concrete material properties given in Eurocode 2-1.1.*

(7) The concentration factor k_j may be taken as 1,0 or otherwise as:

$$k_j = \left[\frac{a_1\ b_1}{ab}\right]^{0,5} \tag{L.4}$$

where	a and b	are the dimensions of the base plate
and	a_1 and b_1	are the dimensions of the effective area, as indicated in Figure L.2.

(8) For a_1 the least of the following should be taken:

- $a_1 = a + 2\ a_r$ (L.5a)
- $a_1 = 5\ a$ (L.5b)
- $a_1 = a + h$ (L.5c)
- $a_1 = 5\ b_1$ but $a_1 \ngeq a$ (L.5d)

(9) For b_1 the least of the following should be taken:

- $b_1 = b + 2\ b_r$ (L.6a)
- $b_1 = 5\ b$ (L.6b)
- $b_1 = b + h$ (L.6c)
- $b_1 = 5\ a_1$ but $b_1 \geq b$ (L.6d)

(10) When the column base is placed on a concrete slab, due account should be taken of the moment resistance and the punching resistance of the concrete slab.

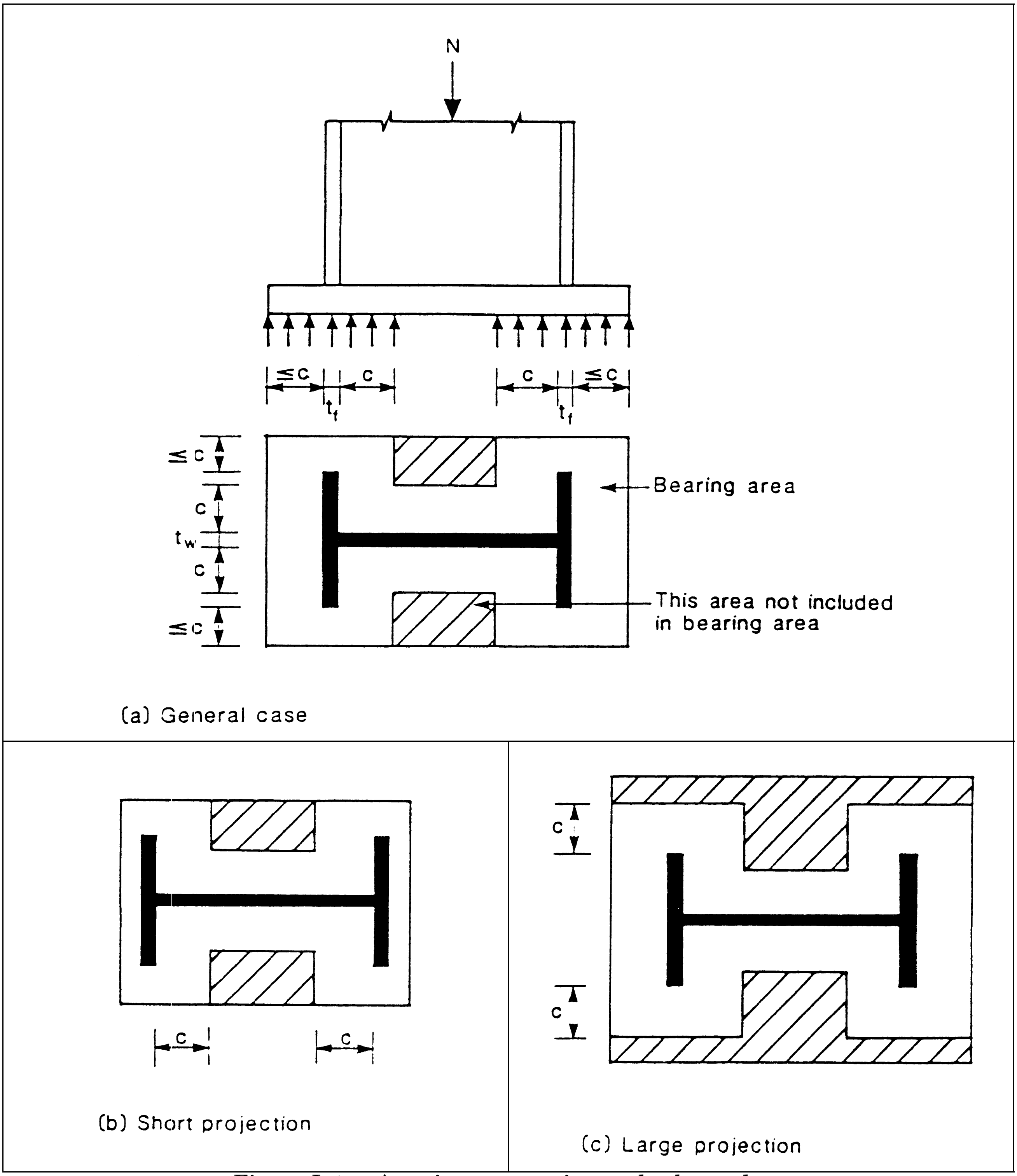


Figure L.1 — Area in compression under base plate

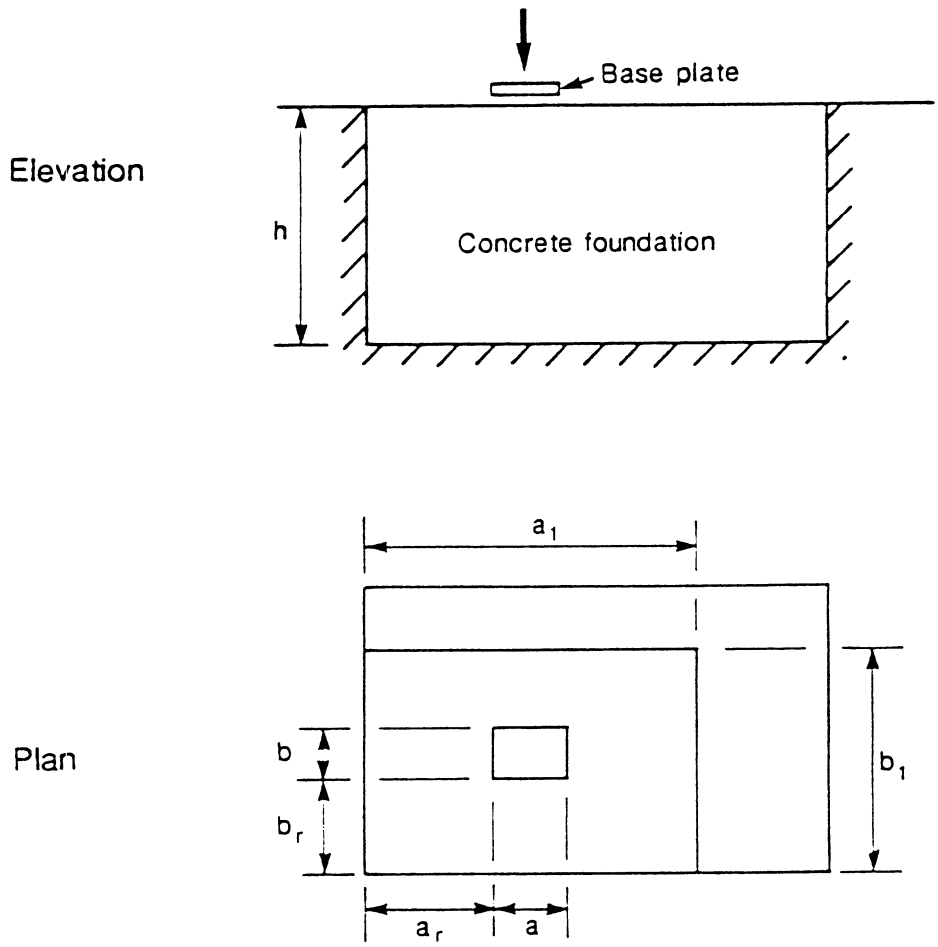


Figure L.2 — Column base

L.2 Holding down bolts

(1) *Holding down bolts should be designed to resist the effects of the design loads. They should provide resistance to tension due to uplift forces and bending moments where appropriate.*

(2) *If no special elements for resisting shear are provided, such as block or bar shear connectors, it should be demonstrated that either the shear resistance of the holding down bolts or the friction resistance of the base plate is sufficient to transfer the design shear force.*

(3) *When calculating the tension forces in the holding down bolts due to bending moments, the lever arm should not be taken as more than the distance between the centroid of the bearing area on the compression side and the centroid of the bolt group, taking the tolerances on the positions of the holding down bolts into account.*

(4) *The design resistance of the holding down bolts should be determined from* ***6.5.5***.

(5) *Holding down bolts should either be anchored into the foundation by:*

- *a hook [Figure L.3(a)], or*
- *a washer plate [Figure L.3(b)], or*
- *some other appropriate load distributing member embedded in the concrete, or*
- *some other fixing which has been adequately tested and approved by the designer, the client and the competent authority.*

(6) *The anchorage of holding down bolts should be in accordance with the relevant clauses in ENV 1992-1-1 Eurocode 2-1.1.*

(7) *When the bolts are provided with a hook, the anchorage length should be such as to prevent bond failure before yielding of the bolt. The anchorage length should be calculated in accordance with the relevant clauses in Eurocode 2. This type of anchorage should not be used for bolts with a specified yield strength higher than 300 N/mm*^{2}.

(8) *When the holding down bolts are provided with a washer plate or other load distributing member, no account should be taken of the contribution of bond. The whole of the force should be transferred through the load distributing device.*

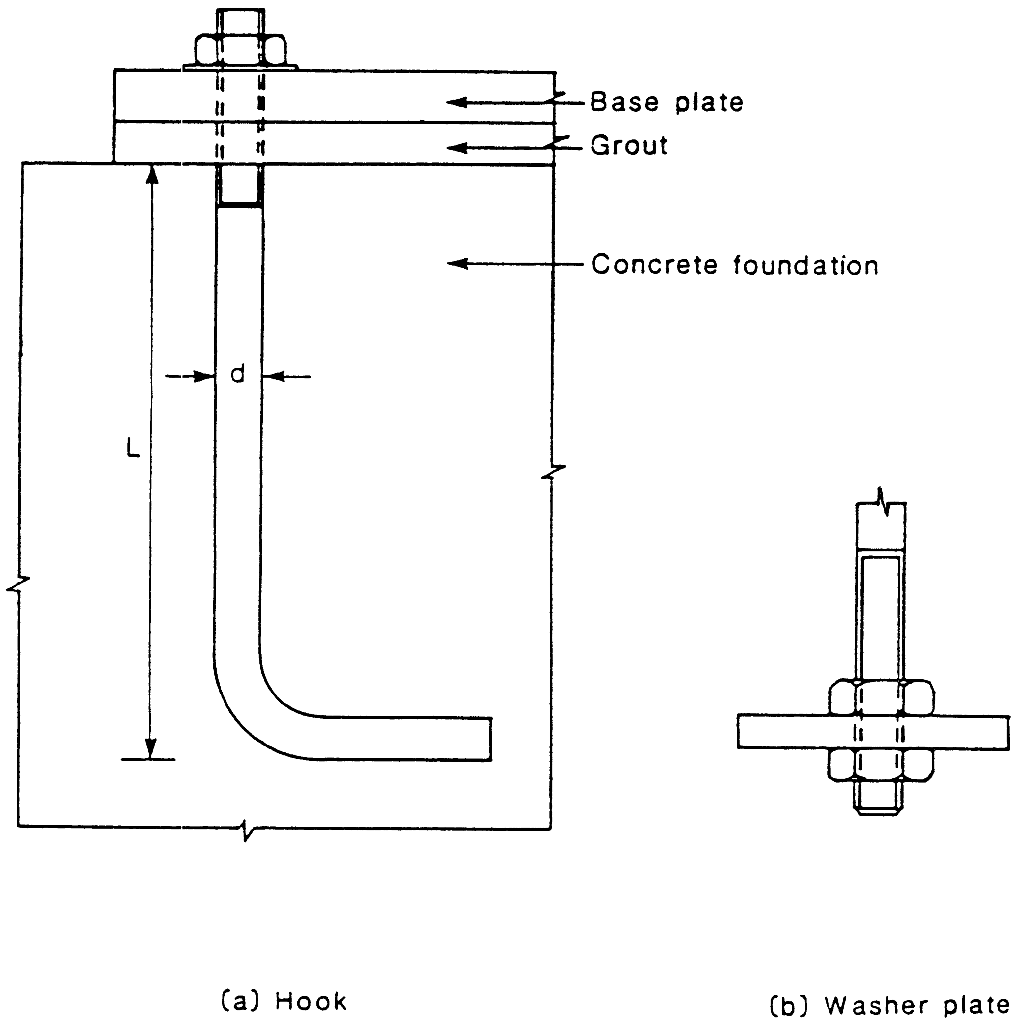


Figure L.3 — Anchorage of holding down bolts

Annex M (normative)
Alternative method for fillet welds

(1) *The resistance of a fillet weld may be verified by the following method as an alternative to the method given in* ***6.6.5.3***.

(2) *In this method, the forces transmitted by a unit length of weld are resolved into components parallel and transverse to the longitudinal axis of the weld and normal and transverse to the plane of its throat.*

(3) *A uniform distribution of stress is assumed on the throat section of the weld, leading to the normal stresses and shear stresses shown in Figure M.1, as follows:*

$\sigma_{\perp}$ *is the normal stress perpendicular to the throat*

$\sigma_{\parallel}$ *is the normal stress parallel to the axis of the weld*

$\tau_{\perp}$ *is the shear stress (in the plane of the throat) perpendicular to the axis of the weld*

$\tau_{\parallel}$ *is the shear stress (in the plane of the throat) parallel to the axis of the weld.*

(4) *The normal stress $\sigma_{\parallel}$ parallel to the axis is not considered when verifying the resistance of the weld.*

(5) *The resistance of the fillet weld will be sufficient if the following are both satisfied:*

$$[\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2)]^{0,5} \leq f_u/(\beta_w \gamma_{Mw}) \quad \text{(M1)}$$

$$\text{and} \quad \sigma_{\perp} \leq f_u/\gamma_{Mw}$$

*where f_u and β_w are as defined in **6.6.5.3**.*

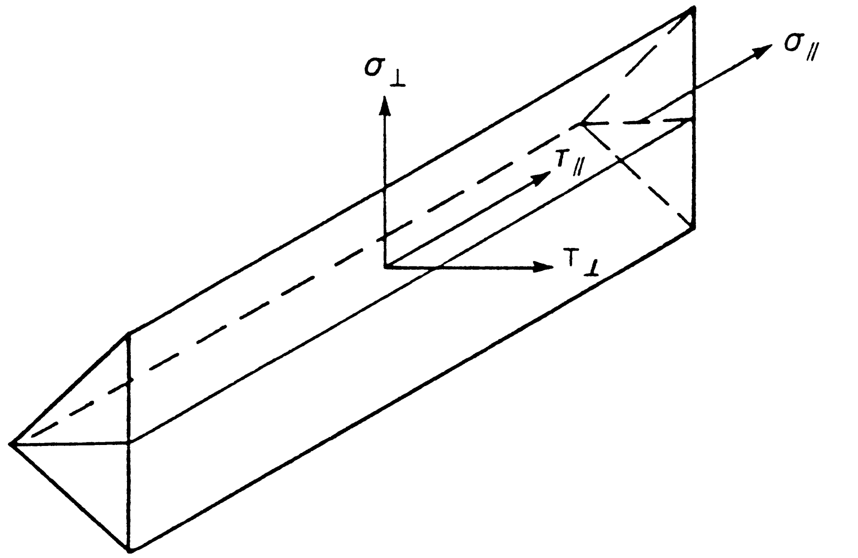


Figure M.1 — Stresses on the throat section of a fillet weld

Annex Y (informative)
Guidelines for loading tests

Y.1 General

(1) Testing may be undertaken when:

a) the calculation models specified in Chapters 4 to 6 are not sufficient for a particular structure or structural component or may lead to uneconomic results [see tests (1) and (2) below];

b) the design resistance of a component or structure is to be established from a knowledge of its ultimate resistance [see test (3) below];

c) confirmation is required of the consistency of production of components or structures originally justified by testing [see test (4) below];

d) The actual performance of an existing structure is to be established because its resistance is in question [see test (1) below].

(2) To meet these situations a basis is presented for four types of tests:

i) an acceptance test for confirmation of general structural behaviour (see **Y.4.1**);

ii) a strength test against the required ultimate loads (see **Y.4.2**);

iii) a test to failure, to determine the ultimate resistance and mode of failure (see **Y.4.3**);

iv) a check test to establish consistency of production (see **Y.4.4**).

(3) These test procedures are intended for steel structures only.

(4) *For cold-formed steel sheeting and members standard testing procedures have been developed which are specified in ENV 1993-1-3 Eurocode 3-1.3*[27].

(5) *For structures of composite construction in steel and concrete reference should be made to ENV 1994-1-1 Eurocode 4-1.1*[27].

(6) Testing of scale models or of items subject to fluctuating loads which could cause fatigue to become a design criterion is not covered by this Annex.

Y.2 Test conditions

(1) The design of the test rig shall be such that the loading system adequately simulates the magnitude and distribution of the loading and allows the specimen to perform in a manner representative of service conditions.

(2) *The specimen should be free to deflect under load. Lateral and torsional restraints should be representative of those in service.*

(3) Care shall be taken to avoid inadvertent eccentricities at the points of application of the test loads and at the supports.

(4) Load and deflection measurements shall be controlled as closely as practicable. The loading system shall be able to follow the movements of the specimen without interruption or abnormal restraint.

(5) *Deflections should be measured at sufficient points of high movement to ensure that the maximum value is determined. The anticipated magnitude of such deflections should be estimated in advance. Generous allowances should be made for movement beyond the elastic range.*

(6) *In some situations it may be desirable to determine the magnitude of stresses in a specimen. This may be demonstrated qualitatively by means of brittle coatings or quantitatively by measurements of strain. Such information should be considered supplementary to the overall behaviour as determined by deflections.*

Y.3 General test procedures

(1) Where the self weight of the specimen is not representative of the actual permanent load in service, allowance for the difference shall be made in the calculation of the test loads to be applied.

(2) Prior to any test, preliminary loading (not exceeding the characteristic values of the relevant loads) may be applied and then removed, in order to bed down the test specimen onto the test rig.

(3) Loading shall be applied in a number of regular increments (not less than 5) at regular intervals in each phase. Sufficient time shall be allowed between each increment for the specimens to reach stationary equilibrium. After each increment the specimen shall be carefully examined for signs of rupture, yield or buckling.

(4) *A running plot should be maintained of loading against the principal deflection. When this indicates significant non-linearity, then the load increments should be reduced.*

(5) On the attainment of maximum load for either acceptance or strength tests, this load shall be maintained at a constant value for at least 1 hour. Readings of load and deflection shall be taken at intervals of 15 minutes and the loading shall be maintained constant until there is no significant increase in deflection during a 15 minute period and at least 1 hour has elapsed.

(6) Unloading shall be completed in regular decrements, with deflection readings taken at each stage and again when the unloading is complete.

(7) Where test results are used to establish or confirm the behaviour of similar structures or components the properties of the steel used in the relevant items shall be established by coupon tests to validate comparisons between tests carried out on different specimens or at different times.

(8) *Coupons should either be cut from the same sections or plates or else recovered from unyielded areas of the specimen after test.*

Y.4 Specific test procedures

Y.4.1 *Acceptance test*

(1) This test is intended as a non-destructive test for confirming structural performance. For acceptance, the assembly shall prove capable of sustaining the test loading given in (3).

(2) It should be recognised that such loading applied to certain structures may cause permanent local distortions. Such effects do not necessarily indicate structural failure in an acceptance test, but the possibility of their occurrence should be agreed before testing.

[27] In preparation

(3) *The test load for an acceptance test should be:*

- $\boxed{1,0}$ × *(actual self-weight present during test),*
- $\boxed{1,15}$ × *(remainder of the permanent load) and*
- $\boxed{1,25}$ × *(variable loads)*

(4) The assembly shall satisfy the following criteria:

a) it shall demonstrate substantially linear behaviour under test loading

b) on removal of the test load the residual deflection should not exceed 20 % of the maximum recorded.

(5) If the conditions given in (4) are not satisfied the test may be repeated once only. The assembly shall demonstrate substantially linear behaviour under this second application of the test loading and the new residual deflection shall not exceed 10 % of the maximum recorded during the second test.

Y.4.2 *Strength test*

(1) The strength test is used to confirm the calculated resistance of a structure or component.

(2) Where a number of items are to be constructed to a common design, and one or more prototypes are tested to confirm their strength, the others may be accepted without further tests provided they are similar in all relevant respects to the prototype, see **Y.4.4**.

(3) *Before carrying out the strength test the specimen should first be submitted to and satisfy the acceptance test described in **Y.4.1**.*

(4) The test load for a strength test shall be based on the calculated design load for the ultimate limit state as given in Chapter 2 for the appropriate combination of permanent and variable loads.

(5) The resistance of the assembly under test will be dependent on the material properties. The actual yield strengths of all the steel materials in the assembly shall be determined from coupon tests.

(6) The value of the averaged yield strength f_{ym} taken from such tests shall be determined with due regard to the importance of each element in the assembly.

(7) The test load $F_{test.s}$ (including self weight) shall be determined from:

$$F_{test.s} = \gamma_{M1}\, F_{Sd.ult}(f_{ym}/f_y) \qquad \text{(Y.1)}$$

where $F_{Sd.ult}$ is the design load for the ultimate limit state.

(8) At this load there shall be no failure by buckling or rupture of any part of the specimen.

(9) On removal of the test load the deflection shall reduce by at least 20 %.

Y.4.3 *Test to failure*

(1) The objective of a test to failure is to determine the design resistance from the actual ultimate resistance.

(2) It is only from a test to failure that the actual mode of failure and resistance of an specimen can be determined. Where the specimen is not required for use it may be advantageous to secure this additional information after a strength test.

(3) *In this situation it is still desirable to carry out the load cycling of the acceptance and strength tests. An estimate should be made of the anticipated ultimate resistance as a basis for such tests.*

(4) *Before a test to failure, the specimen should first satisfy the strength test described in **Y.4.2**. Where the ultimate resistance has been estimated its value should be reviewed in the light of the specimen's behaviour in the strength test.*

(5) During a test to failure the loading shall first be applied in increments up to the strength test load, as specified in **Y.4.2**. Subsequent load increments shall then be determined from consideration of the principal plot.

(6) The test load resistance $F_{test.R}$ shall be determined as that load at which the specimen is unable to sustain any further increases in load.

(7) *At this load, gross permanent distortion is likely to have occurred and in some cases gross deformation may define the test limit.*

(8) Not less than three tests shall be carried out on nominally identical specimens.

(9) If the deviation of any individual test result from the mean value obtained from all the tests exceeds 10 %, at least six tests shall be carried out. The determination of the design resistance F_{Rd} shall then be carried out in accordance with the statistical method given in Annex Z[28].

(10) When the deviation from the mean does not exceed 10 %, the design resistance may be determined from (11) to (14).

(11) Provided that there is a ductile failure, the design resistance F_{Rd} may be determined from:

$$F_{Rd} = 0{,}9F_{test.R.min}\,(f_y/f_{ym})/\gamma_{M1} \tag{Y.2}$$

where $F_{test.R.min}$ is the minimum test result

and f_{ym} is the averaged yield strength, see **Y.4.2**(6).

(12) In the case of a sudden ("brittle") rupture type failure the design resistance may be determined from:

$$F_{Rd} = 0{,}9F_{test.R.min}(f_y/f_{um})/\gamma_{M1} \tag{Y.3}$$

where f_{um} is the averaged ultimate tensile strength, determined as for f_{ym}, see **Y.4.2**(6).

(13) In the case of a sudden ("brittle") buckling type failure the design resistance shall be determined from:

$$F_{Rd} = 0{,}75F_{test.R.min}(f_y/f_{ym})/\gamma_{M1} \tag{Y.4}$$

(14) In the case of a ductile buckling type failure in which the relevant slenderness λ can be reliably assessed, the design resistance may [as an alternative to (11)] be determined from:

$$F_{Rd} = 0{,}9F_{test.R.min}[(\chi\, f_y)/(\chi_m\, f_{ym})]/\gamma_{M1} \tag{Y.5}$$

where χ is the reduction factor for the relevant buckling curve (see **5.5.1**)

and χ_m is the value of χ when the yield strength is f_{ym}.

Y.4.4 ***Check tests***

(1) Where a component or assembly is designed on the basis of strength tests or tests to failure as described in **Y.4.2** and **Y.4.3** and a production run is carried out of such items, an appropriate number of samples (not less than two) shall be selected from each production batch at random.

(2) The samples should be carefully examined to ensure they are similar in all respects to the prototype tested, particular attention being given to the following:

a) dimensions of components and connections;

b) tolerance and workmanship

c) quality of steel used, checked with reference to mill certificates.

(3) Where it is not possible to determine either the variations or the effect of variations from the prototype, an acceptance test shall be carried out as a check test.

(4) In this check test, the deflections shall be measured at the same positions as in the acceptance test of the prototype. The maximum measured deflection shall not exceed 120 % of the deflection recorded during the acceptance test on the prototype and the residual deflection should not be more than 105 % of that recorded for the prototype.

Y.4.5 ***Testing to determine strength functions and model factors***

(1) Strength functions and model factors may be evaluated from the results of appropriate series of tests to failure.

(2) The determination of the design value for the strength shall be in accordance with the evaluation procedure given in Annex Z[28].

[28] In preparation

Y.4.6 *Other test procedures*

(1) For certain structural components specific test procedures are given in the relevant Eurocode Annex or product standard.

(2) *Examples are:*

- *stub-column tests for cold-formed sections*,
- *slip factor tests for slip-resistant bolted connections*,
- *testing of semi-rigid connections and*,
- *shear connector tests for composite construction*.

(3) Similar specific procedures, conforming to the Principles given in Chapter 8 and compatible with the guidance given in this Annex, may be developed and agreed between the client, the designer and the competent authority.

National annex NA (informative) Committees responsible

The preparation of the National Application Document for use in the UK with ENV 1993-1-1:1992 was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/31, Structural use of steel, upon which the following bodies were represented:

British Constructional Steelwork Association
British Industrial Fasteners Federation
British Railways Board
British Steel Industry
Cold Rolled Sections Association
Department of the Environment (Building Research Establishment)
Department of the Environment (Construction Directorate)
Department of the Environment (Property Services Agency)
Department of Transport
Health and Safety Executive
Institution of Civil Engineers
Institution of Structural Engineers
Royal Institution of British Architects
Steel Construction Institute
The Welding Institute

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DRAFT FOR DEVELOPMENT

DD ENV 1993-1-3:2001
Incorporating Corrigendum No. 1

Eurocode 3: Design of steel structures —

Part 1.3: General rules — Supplementary rules for cold formed thin gauge members and sheeting

(together with United Kingdom National Application Document)

ICS 91.010.30; 91.080.10

Committees responsible for this Draft for Development

The preparation of this Draft for Development was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/31, Structural use of steel, upon which the following bodies were represented:

British Constructional Steelwork Association

Cold Rolled Sections Association

Confederation of British Forgers

Department of the Environment, Transport and the Regions

Department of the Environment, Transport and the Regions — Construction Directorate

Department of the Environment, Transport and the Regions — Highways Agency

Health and Safety Executive

Institution of Civil Engineers

Institution of Structural Engineers

Steel Construction Institute

UK Steel Association

Welding Institute

This Draft for Development, having been prepared under the direction of the Sector Committee for Building and Civil Engineering was published under the authority of the Standards Committee and comes into effect on 15 July 2001

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Contents

National foreword

This publication has been prepared by Subcommittee B/525/31 and is the English language version of ENV 1993-1-3:1996, *Eurocode 3: Design of steel structures — Part 1.3: General rules — Supplementary rules for cold formed thin gauge members and sheeting* incorporating its corrigendum of October 1997, as published by the European Committee for Standardization (CEN). This Draft for Development also includes the United Kingdom (UK) National Application Document (NAD) to be used with the ENV in the design of buildings to be constructed in the UK.

ENV 1993-1-3:1996 results from a programme of work sponsored by the European Commission to make available a common set of rules for the design of building and civil engineering works.

This publication should not be regarded as a British Standard.

An ENV is made available for provisional application, but does not have the status of a European Standard. The aim is to use the experience gained to modify the ENV so that it can be adopted as a European Standard.

The value for certain parameters in the ENV Eurocodes may be set by CEN members so as to meet the requirements of national regulations. These parameters are designated by □ (boxed values) in the ENV.

During the ENV period of validity, reference should be made to the supporting documents listed in the NAD.

The purpose of the NAD is to provide essential information, particularly in relation to safety, to enable the ENV to be used for buildings constructed in the UK. The NAD takes precedence over corresponding provisions in the ENV.

The Building Regulations 1991, Approved Document A 1992 (published December 1991)[1)], draws designers' attention to the potential use of ENV Eurocodes as an alternative approach to Building Regulation compliance. ENV 1993-1-3:1996 has been thoroughly examined over a period of several years and is considered to offer such an alternative approach, when used in conjunction with this NAD.

Compliance with DD ENV 1993-1-3:2001 does not of itself confer immunity from legal obligations.

Users of this document are invited to comment on its technical content, ease of use and any ambiguities or anomalies. These comments will be taken into account when preparing the UK national response to CEN on the question of whether the ENV can be converted into an EN.

Comments should be sent in writing to BSI, 389 Chiswick High Road, London W4 4AL, quoting the document reference, the relevant clause and, where possible, proposed revised wording.

This document does not purport to include all the necessary provisions of a contract. Users of this document are responsible for its correct application.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to xvi, the ENV title page, pages 2 to 128 and a back cover.

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National Application Document

for use in the UK with ENV 1993-1-3:1996

Contents of National Application Document

Introduction

This National Application Document (NAD) has been prepared by Subcommittee B/525/31. It has been developed from:

a) a textual examination of ENV 1993-1-3:1996;

b) calibration against UK practice, supporting standards and test data.

NOTE *Design of cold formed steel sections and sheeting to Eurocode 3:Part 1.3* [1] gives a series of worked examples based on ENV 1993-1-3:1996 and this NAD.

It should be noted that this NAD, in common with ENV 1993-1-3 and supporting CEN standards, uses a comma (,) where a decimal point (.) would be traditionally used in the UK.

1 Scope

This NAD provides information required to enable ENV 1993-1-3:1996 to be used for the fire resistant design of buildings to be constructed in the UK.

2 Normative references

The following normative documents contain provisions, which, through reference in this text, constitute provisions of this NAD. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. For undated references, the latest edition of the publication referred to applies.

BS 648:1964 (all parts), *Schedule of weights of building materials.*

BS 6399-1:1996, *Loadings for buildings — Part 1: Code of practice for dead and imposed loads.*

BS 6399-3:1988, *Loadings for buildings — Part 3: Code of practice for imposed roof loads.*

CP 3:Chapter V:Part 2:1972, *Code of basic data for the design of buildings — Loading — Wind loads.*

3 Partial safety factors and other factors

3.1 Material factors

The values for the partial safety factor γ_M for use with ENV 1993-1-3:1996 should be as given in Table 1 of this NAD.

The values for the load factors for acceptance tests are given in **6.10** of this NAD.

Table 1 — Partial safety factors (γ_M)

Reference in ENV 1993-1-3	Definition	Symbol	Condition	Value	
				Boxed ENV value	Value for UK use
2.2(3)P	Partial safety factor for verification at the ultimate limit state.	γ_{M0}	Resistance of cross-section where failure is caused by yielding.	1,10	1,05
		γ_{M1}	Resistance of members and sheeting where failure is caused by buckling.	1,10	1,05
		γ_{M2}	Resistance of net section of bolt holes.	1,25	1,20
2.3(3)P	Partial safety factor for verifications at serviceability limit state.	$\gamma_{M,ser}$	—	1,00	1,00
8.4(6)P	Partial safety factor for calculating the design resistance of mechanical fasteners.	γ_{M2}	Bolts	1,25	1,35
			Rivets	1,25	1,35
			Pins	1,25	1,35
			Spot welds	1,25	1,35
			Lap welds	1,25	1,35
10.2.2.1(1)	Partial safety factor for steel liner trays restrained by sheeting.	γ_{M2}	Wide flange in compression	1,25	1,20
10.2.2.2(1)	Partial safety factor for steel liner trays restrained by sheeting.	γ_{M2}	Wide flange in tension	1,25	1,20
A.6.4	Partial factor for difference in behaviour under test conditions and service conditions.	γ_{sys}	—	1,0	1,0

4 Loading codes

The loading codes to be used are:

BS 648:1964 (all parts), *Schedule of weights of building materials*.

BS 6399-1:1996, *Loadings for buildings — Code of practice for dead and imposed loads*.

BS 6399-3:1988, *Loadings for buildings — Code of practice for imposed roof loads*.

CP3:Chapter V:Part 2:1972, *Code of basic data for the design of buildings — Loading — Wind loads*.

In using these documents with ENV 1993-1-3:1996, the following modifications should be noted.

a) The imposed floor loads of a building should be treated as one variable action to which the reduction factors given in clause **5** of BS 6399-1:1984 are applicable.

b) The wind loading should be taken as 90 % of the value obtained from clause **4.3** of CP3:Chapter V:Part 2:1972.

NOTE Although it is intended that BS 6399-2 will eventually replace CP3:Chapter V:Part 2, wind loads for structures designed in accordance with ENV 1993-1-3:1996 should continue to be determined in accordance with CP3:Chapter V:Part 2 rather than in accordance with BS 6399-2 until such time as CP3:Chapter V:Part 2 is withdrawn. In such cases, local wind pressure and suction need not be considered in the design of purlins and sheeting rails.

c) The design for structural integrity should follow the provisions in **6.2**a) of this NAD.

d) Reference should be made to clause **12** of BS 6399-1:1996 for the determination of accidental loads.

5 Reference standards

Where ENs are directly referred to by ENV 1993-1-3:1996, the appropriate BS ENs should be used. The remaining supporting standards to be used for construction with cold formed thin gauge members and sheeting designed in accordance with ENV 1993-1-3:1996 are given in Table 2 of this NAD.

Table 2 — Directly referenced supporting standards in ENV 1993-1-3

ENV 1993-1-3:1996 calls up	UK supporting standard
BS EN 10149-2	BS EN 10149-2
BS EN 10149-3	BS EN 10149-3
ENV 1090-2	DD ENV 1090-2[a] BS 5950-7
ENV 1991-1	BS 6399, CP3:Chapter V:Part 2[a]
ENV 1993-1-1	DD ENV 1993-1-1:1992
ISO 4997	ISO 4997
ISO 1000	ISO 1000 (BS 5555)

[a] Currently in preparation.

[a] See the note in clause **4** of this NAD.

6 Additional recommendations

6.1 Chapter 1 General

a) *1.1 Scope*

Cold formed thin gauge members may be either open or closed and should be made up of flat elements bounded either by free edges or by bends with included angles not exceeding 135° and internal radii not exceeding $5t$ where t is the material thickness. ENV 1993-1-3:1995 does not apply to cold formed structural hollow sections complying with EN 10219, for which reference should be made to ENV 1993-1-1:1992.

The designer responsible for the overall stability of the structure should be clearly identified. This designer should ensure the compatibility of the structural design and detailing between all those structural parts and components that are needed for overall stability, even if some or all of the structural design and detailing of those structural parts and components is carried out by another designer.

b) *1.1(3)*

The detailed design of stressed-skin constructions should be in accordance with BS 5950-9.

c) *1.1(5)*

The limitations that do not apply to design assisted by testing are:

1) b/t ratio;

2) thickness;

3) material properties.

d) *1.5(3)*

System lines of flanges means mid-lines of flanges.

e) *1.7.4(2)*

To simplify the design rules for torsional and torsional-flexural buckling in **6.2.3** of ENV 1993-1-3:1996, the convention for member axes differs from that used in ENV 1993-1-1:1992 and it may also change depending on the design situation.

6.2 Chapter 2 Basis of design

a) *2.1 General*

1) Structures constructed using cold formed thin gauge members and sheeting should be designed to fulfil, with due regard to economy, their intended function and should sustain the design loads for their intended life. The design should also facilitate fabrication, erection and future maintenance.

2) A structure should also be designed so that it should not be damaged by events, explosions, impact or the consequences of human error, to an extend disproportionate to the original cause. Design rules to provide structural integrity by limiting the effects of accidental damage are given in Annex A of the NAD for ENV 1993-1-1:1992.

In construction where vertical loads are resisted by an assembly of closely spaced elements, (e.g. cold formed steel framing) the tying members should be distributed to ensure that the entire assembly is effectively tied. In such cases the forces for anchoring the vertical elements at the periphery should be based on the spacing of the elements or taken as 1 % of the factored vertical load in the element without applying the minimum value of 75 kN or 40 kN to the individual elements, provided that each tying member and its connections are designed to resist the appropriate loading.

NOTE 1 The above recommendations should be met by the choice of suitable materials, by appropriate design and detailing and by specifying control procedures for production, construction and use as relevant for the particular project

NOTE 2 Further guidance on methods of reducing the sensitivity of buildings to disproportionate collapse in the event of an accident are given in Approved Document A of the Building Regulations [2].

b) *2.1(4)P*

The values of partial factors given in this NAD should be adopted for Construction Clauses I, II and III.

c) *2.2(1)P*

1) Where it is necessary to take account of changes in temperature in the design of a structure, it may be assumed that in the UK the average temperature of internal steelwork varies from –5 °C to +35 °C. The actual range, however, depends on the location, type and purpose of the structure and special consideration may be necessary for structures in other environments.

2) When designing for the accidental situation in Table 2.1 of ENV 1993-1-1:1992, the values of ψ_1 and ψ_2 should be determined from Table 4 of the NAD for ENV 1993-1-1:1992. For the determination of the accidental load (A_k), reference should be made to BS 6399-1 where appropriate.

The accidental total A_k should be multiplied by a γ_A factor of 1,05 and the γ_{GA} factor should be taken as 1,05, except where the dead load is considered to consist of unfavourable and favourable parts. In this case, the favourable part should be multiplied by a γ_{GA} factor of 0,9 and the unfavourable part should be multiplied by a factor of 1,05.

6.3 Chapter 3 Properties of materials and cross-sections

a) *3.1.1(7)P*

Although the real value for the modulus of elasticity for cold formed steel is less, the value of 210 000 N/m^2 should be used because the formulae have been developed and calibrated using this value.

b) *2.1.2(3)P*

For cross-sections which are not fully effective, the increase in yield strength due to cold forming may be calculated using the recommendations given in BS 5950-5.

c) *3.1.2(7)P*

The increase in yield strength due to cold working should not be utilized for members which undergo welding, annealing, galvanizing or any other heat treatment after forming that may produce softening.

d) *3.1.3(1)P*

For steel sheeting, the upper limit for the nominal core thickness (t_{cor}) may be taken as 8.0 mm. No lower limit is needed for the sheeting providing that it can be demonstrated to have adequate resistance to denting from construction and maintenance traffic. ENV 1993-1-1:1992 may be used for steel with a nominal core thickness t_{cor} exclusive of zinc or organic coating greater than 5 mm. No lower limit is necessary for the nominal core thickness of members.

e) *313(3)P amd 313(4)P*

The design thickness should be obtained from the following expression:

$t = k \times t_{cor}$

where

k is obtained from Table 3 of this NAD.

f) *3.1.3(5)*

The nominal core thickness (t_{cor}) should be calculated using the following expression:

$t_{cor} = t_{nom} - t_{coating}$

where

t_{nom} is the nominal thickness;

$t_{coating}$ is the thickness of the coating (i.e. zinc, paint, etc.)

Table 3 — Values of factor *k*

Nominal thickness t_{cor}	Members		Sheeting	
	Normal tolerances	Special tolerances	Normal tolerances	Special tolerances
$1{,}0 \leq t_{nom}$	1,0	—[a]	1,0	—[a]
$0{,}6 \leq t_{nom} < 1{,}0$	0,95	1,0	1,0	—[a]
$0{,}4 \leq t_{nom} < 0{,}6$	0,9	0,95	0,95	1,0
[a] Not applicable.				

6.4 Chapter 4 local buckling

a) *4.2(5)*

Alternative 1 should be used and Alternative 2 should be ignored.

NOTE Alternative 1 is based on the ultimate limit state.

b) *4.2(6)*

Alternative 2 should be used and Alternative 1 should be ignored.

NOTE Alternative 2 is based on the serviceability limit state.

c) *4.2(9)*

Values of effective section properties obtained without iteration may be used. Alternatively, the iteration process may be used if it gives a larger value.

NOTE Under some circumstances iteration may give a smaller value. If this is the case, the initial value may be used.

d) *4.3.2.1(1)*

Provided that the conditions given in **4.3.2.1**(2)P of ENV 1993-1-3:1996 and modified by the conditions given in **6.4**c) of this NAD are met, the effectiveness of a multiple edge fold stiffener may be determined from the procedures given in **4.3.2.1**(3) of ENV 1993-1-3:1996.

e) *4.3.2.1(2)P*

It is not necessary to limit c such that:

$c \leq 0{,}2b_p$

f) *Figure 4.7*

For the figure showing one stiffener, $S_{eff,4}$ should read $S_{eff,n}$, and for the figure showing two stiffeners, $S_{eff,6}$ should read $S_{eff,n}$.

g) *4.3.2.2(8) and 4.3.2.2(9)*

The value of χ may be obtained without iteration. Alternatively, the iteration process may be used if it gives a higher value.

h) *4.3.2.3(3)*

The values of 0,5 and 1,0 for χ should be changed to 0,45 and 0,9 respectively.

i) 4.3.3.1

The general procedure in **4.3.3.2** of ENV 1993-1-3:1996 and the simplified procedure in **4.3.3.3** of ENV 1993-1-3:1996 do not include the effects of flange curling and give inaccurate results for members subject to bending actions. These procedures should not be used for beams where the ratio of b/t is greater than 300.

j) *4.3.4.2 Flanges with intermediate stiffeners*

The procedure in **4.3.4.2** of ENV 1993-1-3:1996 does not include the effects of flange curling and gives inaccurate results for sheeting subjected to bending actions. This procedure should not be used for beams where the ratio of b/t is greater than 300.

k) *4.3.4.2(3)*

The expression for b_e is incorrect and should be replaced with the following correct expression:

$$b_e = 2b_{p,1} + b_{p,2} + 2b_s$$

l) *Figure 4.3*

When considering this figure, it should be noted that c_{eff} is not necessarily equal to c.

6.5 Chapter 5 Resistance of cross-sections

a) *5.1 General*

Attention is drawn to the fact that the following four different cross-sectional properties are used in ENV 1993-1-3:1996:

1) gross cross-sectional properties;

2) effective area;

3) effective section modulus about the major axis;

4) effective section modulus about the minor axis.

b) *Figure 5.1*

In Figure 5.1, the applied axial force N_{Sd} is assumed to act at the centroid of the gross cross-section, whereas the resistance to axial force N_{Rd} is assumed to act at the centroid of the effective cross-section. Thus the force N_{Sd} should be treated as being applied at an eccentricity equal to the shift e_N of the centroid.

c) *5.4.2 Partial plastic resistance*

If the section conforms to the requirements for a class 1 cross-section given in clause **5.3** of ENV 1993-1-1:1992, the method given in **5.2.3.1** of BS 5950-5:1998 for calculating the plastic bending category may be used.

d) *5.4.2(2)*

A bilinear distribution is shown in the right hand diagram in Figure 5.3.

e) *5.4.2(4)*

Alternatively, the provisions given in **7.2** may be demonstrated by calculation in appropriate cases.

f) *5.4.3(1)P*

The effects of shear lag should be taken into account if the length L_m is less than $20b_0$ for simply-supported beams with a uniformly-distributed load, and less than $50b_0$ for all other cases.

g) *Table 5.1 Reduction factors β_i for shear lag*

It should be noted that L_m is the distance between points of contraflexure.

h) 5.6(2)P

The definitions for e_{Ny} and e_{Nz} are incorrect — e_{Ny} and e_{Nz} are shifts of the y-y and z-z centroid axes respectively, under axial loading.

i) *5.8(5)P*

When using Table 5.2, the value of f_{bv} should be calculated using the formulae for webs without stiffening at the support, irrespective of whether stiffeners are present or not.

j) *5.8(6)*

The correct interpretation for k_r is:

$$k_r = 5,34 + \frac{2,10}{t}\left[\frac{I_s}{s_d}\right]^{1/3}$$

k) *5.9.2*

For I-beams with restraint against web rotation, the method given in **5.3** of BS 5950-5:1998 may be used.

6.6 Chapter 6 Buckling resistance

a) *Expression 6.4a*

Expression 6.4a is incorrect and should be replaced by the following expression

$\bar{\lambda} = (f_{yb}/\sigma_{cr})^{0,5}[\beta_A]^{0,5}$

b) *6.2.3 Torsional buckling and torsional-flexural buckling*

All sections should be checked for torsional, torsional-flexural and flexural buckling.

c) *6.2.3(1)P*

An example of a point-symmetric open cross-section is a Z-section with similar flanges.

d) *6.2.3(7)P*

For non-symmetrical cross-sections, the maximum stress should be determined by second-order analysis.

e) *6.3(1)P*

Alternatively, M_{cr} can be obtained from **6.6.2.2** of BS 5950-5:1998, its value being that of M_E.

f) *6.4 Distortional buckling*

For distortional buckling, reference should be made to AS/NZS 4600:1996 or a geometrical non-linear analysis could be carried out using suitable initial imperfections.

g) *6.5 Bending and axial compression*

All members subject to combined bending and axial compression should be designed in accordance with the recommendations given in **6.4** of BS 5950-5:1998.

6.7 Chapter 7 Serviceability limit states

a) *7.3 Deflections*

The designs for deflections should follow the provisions in clause **4**, in particular, Table 4.1 and **4.2.3** of ENV 1993-1-1:1992.

b) *7.3(3)*

There is no limit to the deflection of purlins, provided the provisions in **4.2.3** of ENV 1993-1-1:1992 are complied with in respect of the supporting structure.

c) *7.4(2)*

Serviceability limits for sheeting should be obtained from BS 5427-1 and BS 5950-6.

6.8 Chapter 8 Joints and connections

NOTE Design guidance for butt and V-flared welds is given in **6.6.2**(6) of ENV 1993-1-1:1992,

a) *Table 8.4 Bearing resistance*

The bearing resistance given in Table 8.4 can only be used if washers are used under both the head and nut of the bolt.

b) *8.4(10)*

If both $F_{t,Rd}$ and $F_{v,Rd}$ are obtained by calculation, the following equation may be used:

$$\frac{F_{t,Sd}}{F_{t,Rd}} + \frac{F_{v,Sd}}{1,4F_{v,Rd}} \leq 1$$

where

$F_{v,Sd} \leq F_{v,Rd}$.

If either $F_{v,Sd}$ or $F_{v,Rd}$ is obtained by testing, the linear equation given by expression 8.2 should be used.

Alternatively, combined tension and shear testing may be carried out.

c) *8.5(7)P*

The thickness t of the test specimen should be the same as the thickness of the specimens used in practice.

d) *8.6.2*

The values of $L_{w,e}$ and $L_{w,s}$ used in expressions 8.4a and 8.4b respectively for calculating the design resistance $F_{w,Rd}$ of a fillet weld, should not exceed the width of the connected part or sheet, b.

e) *8.6.3(5)P*

1) The minimum distance measured parallel to the direction of force transfer, from the centreline of an arc spot weld to the nearest edge of an adjacent weld or to the end of the connected part towards which the force is directed, should not be less than the value of e_{min} given by the following.

If $\dfrac{f_u}{f_y} \geq 1,15$

$$e_{min} = 1,8 \times \frac{F_{wSd}}{tf_u/\gamma_{M2}}$$

If $\dfrac{f_u}{f_y} \leq 1,15$

$$e_{min} = 2,1 \times \frac{F_{wSd}}{tf_u/\gamma_{M2}}$$

2) The minimum distance from the centreline of a circular arc spot weld to the end or edge of the connected sheet should not be less than $1,5d_w$ where d_w is the visible diameter of the arc spot weld.

3) The minimum clear distance between an elongated arc spot weld and the end of the sheet and between the weld and the edge of the sheet should not be less than $1,0d_w$.

f) *8.6.3(6)*

This clause should be replaced by the following.

The design shear resistance $F_{W,Rd}$ of a circular arc spot weld should be determined using the following expression:

$$F_{w,Rd} = \frac{\pi}{4} d_s^2 \times 0{,}625 \frac{f_{uw}}{\gamma_{M2}}$$

where

f_{uw} is the minimum ultimate tensile strength of the welding electrodes.

$F_{W,Rd}$ should not be taken as more than the peripheral resistance given by the following expression:

$$\text{If} \leq 18 \times \left(\frac{420}{f_u}\right)^{0,5}$$

$$F_{w,Rd} = 1{,}5 d_p \Sigma t\, f_u/\gamma_{m2}$$

The minimum distance from the centreline of a circular arc spot weld to the end or edge of the connected sheet should be not less than $1{,}5d_w$, where d_w is the visible diameter of the arc spot weld.

The minimum clear distance between an elongated arc spot weld and the end of the sheet and between the weld and the edge of the sheet should be not less than $1{,}0d_w$.

$$\text{If } 18 \times \left(\frac{420}{f_u}\right)^{0,5} < \frac{d_p}{\Sigma t} < 30 \times \left(\frac{420}{f_u}\right)^{0,5}$$

$$F_{W,Rd} = 27(420/f_u)^{0,5} (\Sigma t)^2 f_u/\gamma_{M2}$$

$$\text{If } \frac{d_p}{\Sigma t} \geq 30 \times \left(\frac{420}{f_u}\right)^{0,5}$$

$$F_w = 0{,}9\, d_p \Sigma \mathrm{t}\, f_u/\gamma_{M2}$$

g) *8.6.3(7)*

This clause should be replaced by the following.

The interface diameter d_s of an arc spot weld (see Figure 8.6), should be obtained from the following expression:

$$d_s = 0{,}7d_w - 1{,}5\Sigma t$$

where

d_s $\leq 0{,}55d_w$;

d_w is the visible diameter of the arc spot weld (see Figure 8.6).

h) *8.6.3(9)*

This clause should be replaced by the following.

The design shear resistance $F_{W,Rd}$ of an elongation arc spot weld should be determined from the following expression:

$$F_{W,Rd} = \left[\left(\frac{\pi}{4}\right) \times d_s^2 + L_w \times d_s\right] \times \left[0{,}625 \times \frac{f_{uw}}{\gamma_{M2}}\right]$$

where

$F_{W,Rd}$ should not be taken as more than the peripheral resistance given by:

$F_{W,Rd} = (0,5\ L_w + 1,67\ d_p)\Sigma t\ f_u/\gamma_{M2}$

where

L_w is the length of the elongated arc spot weld, measured as shown in Figure 8.7.

6.9 Particular applications

a) *10.1.3.4(2)*

Alternatively, the characteristics may be determined by calculation.

b) *Figure 10.7*

The orientation of the members is not dependent on the number of spans (i.e. the left hand diagram is not related to the right hand diagram).

c) *10.3 Stressed skin design*

The detailed design of stressed-skin construction should also be in accordance with BS 5950-9.

6.10 Annex A Testing procedures

a) *Table A.1 Number of tests*

When the general shape of the buckling curve is obtained from prior knowledge, a smaller number of tests may be carried out, providing it contains a significant number of tests at $\bar{\lambda} = 1,0$.

b) *A.3.4(2)*

Lateral means in any direction at a right angle to the longitudinal axis.

c) *A.4.1 Acceptance tests*

The values for the load factors for acceptance tests for use with ENV 1993-1-3:1996 should be taken as equal to the sum of:

1) 1,0 × the actual self-weight present during the test;

2) one of the following as appropriate:

i) 1,25 × (the imposed load) + 1,15 × (the remainder of the permanent load);

ii) 1,15 × (the remainder of the permanent load) + 1,25 × (the wind load);

iii) 1,25 × (the wind uplift) – 1,0 × (the remainder of the permanent load);

iv) 1,15 × (the remainder of the permanent load) + 1,0 × (the imposed load and the wind load).

d) *A.4.1(6)*

On the attainment of the acceptance test load, it should be maintained at a near constant value to allow repeat measurements for the detection of possible creep. The loads and deflections should be measured at regular checking intervals of at least 5 min. The loading should be adjusted to remain constant until there is no significant increase in deflection during at least three checking intervals subsequent to the attainment of the acceptance test load.

e) *A.5.2.3 Interpretation of test results*

The factor of 0,9 need not be applied if the procedure in **A.6** of ENV 1993-1-3:1996 is followed. Figure A.9 and A.10 are for illustration only. The procedure can be applied to sheeting.

f) *A.6.2(6) Resistance adjustment coefficient*

If $f_{yb,obs} > f_{yb}$,

$a = 1,0$ should be used in all cases.

g) *A.6.3 Characteristic values*

Replace expression A.11 with the following expression:

$$R_k = 1{,}1 \times (R_m - k_s)$$

where

$$R_k \leq R_m$$

h) *A.6.3.3 Characteristic values based on a small number of tests*

For calculation of the characteristic value of resistance (R_k) the value of η_k should be taken as 0,9 for all modes of failure

If two or three tests are performed, the characteristic value of resistance should be obtained from the following expression:

$$R_k = \eta_k \times R_{min}$$

Bibliography

BS 5427-1, *Code of practice for the use of profiled sheet for roof and wall cladding on buildings — Part 1: Design*.

BS 5950-6, *Structural use of steelwork in building — Part 6: Code of practice for design of light gauge profiled steel sheeting*.

BS 5950-7, *Structural use of steelwork in building — Part 7: Specification for materials and workmanship: cold formed sections*.

BS EN 10149-2, *Specification for hot-rolled products made of high yield strength steels for cold forming — Part 2: Delivery conditions for thermomechanically rolled steels*.

BS EN 10149-3, *Specification for hot-rolled products made of high yield strength steels for cold forming — Part 3: Delivery conditions for normalized or normalized rolled steels*.

BS EN 10219, *Cold formed welded structural sections of non-alloy and fine grain steels*.

ISO 1000, *SI units and recommendations for the use of their multiples and of certain other units*.

ISO 4997, *Cold-reduced steel sheet of structural quality*.

[1] COUCHMAN, G.H. *Design of cold formed steel sections and sheeting to Eurocode 3:Part 1.3:1999*[2]. ISBN 1 85942 086 9.

[2] GREAT BRITAIN. The Building Regulations 1991, Approved Document A 1992. London: The Stationery Office[3].

[2] Available from The Steel Construction Institute, Silwood Park, Ascot, Berkshire SL5 7QN.
[3] Available from the Stationery Office, PO Box 29, St Crispins House, Duke Street, Norwich NR3 1GN.

EUROPEAN PRESTANDARD
PRÉNORME EUROPÉENNE
EUROPÄISCHE VORNORM

ENV 1993-1-3:1996/AC

October 1997
Octobre 1997
Oktober 1997

English version
Version Française
Deutsche Fassung

Eurocode 3: Design of steel structures - Part 1-3: General rules - Supplementary rules for cold formed thin gauge members and sheeting

Eurocode 3: Calcul des structures en acier - Partie 1-3: Règles générales - Règles supplémentaires pour les éléments minces formés à froid - Produits longs et produits plats

Eurocode 3: Bemessung und Konstruktion von Stahlbauten - Teil 1-3: Allgemeine Regeln - Ergänzende Regeln für kaltgeformte dünnwandige Bauteile und Bleche

This corrigendum becomes effective on 2 October 1997 for incorporation in the official English version of the ENV.

Ce corrigendum prendra effet le 2 octobre 1997 pour incorporation dans la version anglaise officielle de la ENV.

Die Berichtigung tritt am 2.Oktober 1997 in Kraft zur Einarbeitung der offiziellen Englischen Fassung der ENV einzufügen.

S

EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

Central Secretariat: rue de Stassart, 36 B-1050 Brussels

Ref. No. ENV 1993-1-3:1996/AC:1997 E

The following editorial corrections should be made to ENV 1993-1-3.

Page 24, clause 3.5, below 3.5(1) and above table 3.3, insert paragraph (2) reading:

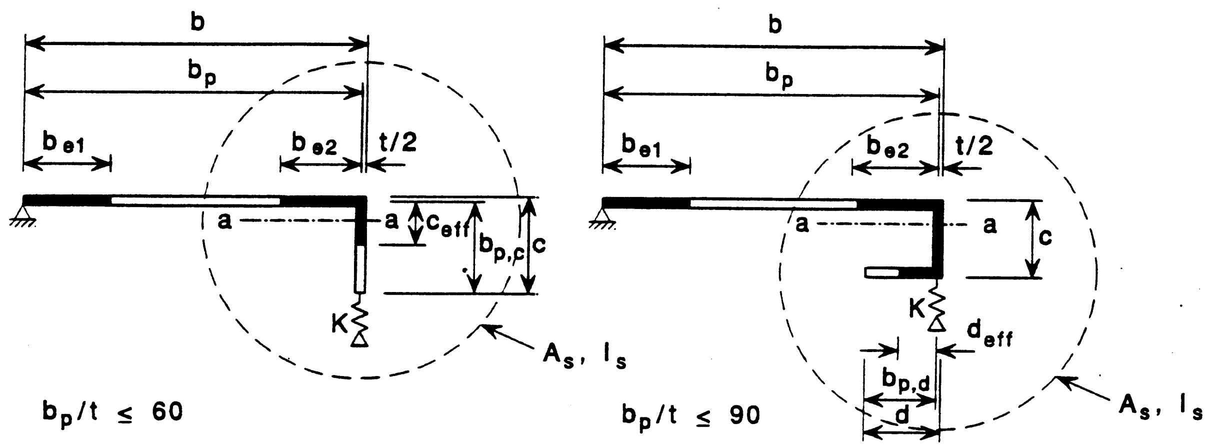
(2) The mutual influence of multiple stiffeners should be taken into account.

Page 27, table 4.2, row 4, column 4, replace 1 by −1.

Page 27, table 4.2, row 5, last column should read:

$$0{,}57 - 0{,}21\psi + 0{,}07\psi^2$$

Page 30, figure 4.2 should be replaced by:



a) single edge fold **b) double edge fold**

Figure 4.2: Edge stiffeners

Page 31, paragraph 4.3.2.2(4), expression (4.10c) should read:

$$k_\sigma = 0{,}5 + 0{,}83 \times \sqrt[3]{(b_{p,c}/b_p - 0{,}35)^2} \quad \ldots (4.10c)$$

Page 33, paragraph 4.3.2.3(2), line 4 should read:

ρ should be obtained from 4.2(5) with $\sigma_{com,Ed}$ equal to $\chi f_{yb}/\gamma_{M1}$, so that:

Page 33, paragraph 4.3.2.2(10), replace expression (4.14) by two lines reading:

$$A_{s,red} = \chi A_s \left[\frac{f_{yb}/\gamma_{M1}}{\sigma_{com,Ed}} \right] \quad \text{but} \quad A_{s,red} \leq A_s \qquad \ldots (4.14)$$

in which $\sigma_{com,Ed}$ is the calculated stress at the centreline of the stiffener.

Page 33, paragraph 4.3.2.2(11), modify line 2 to read:

by using a reduced thickness $t_{red} = tA_{s,red}/A_s$ for all the elements included in A_s.

Page 33, paragraph 4.3.2.3(4), replace expression (4.19) by two lines reading:

$$A_{s,red} = \chi A_s \left[\frac{f_{yb}/\gamma_{M1}}{\sigma_{com,Ed}} \right] \quad \text{but} \quad A_{s,red} \leq A_s \qquad \ldots (4.19)$$

in which $\sigma_{com,Ed}$ is the calculated stress at the centreline of the stiffener.

Page 33, paragraph 4.3.2.3(5), modify line 2 to read:

by using a reduced thickness $t_{red} = tA_{s,red}/A_s$ for all the elements included in A_s.

Page 35, paragraph 4.3.3.2(9), replace expression (4.23) by two lines reading:

$$A_{s,red} = \chi A_s \left[\frac{f_{yb}/\gamma_{M1}}{\sigma_{com,Ed}} \right] \quad \text{but} \quad A_{s,red} \leq A_s \qquad \ldots (4.23)$$

in which $\sigma_{com,Ed}$ is the calculated stress at the centreline of the stiffener.

Page 35, paragraph 4.3.3.2(10), modify line 2 to read:

by using a reduced thickness $t_{red} = tA_{s,red}/A_s$ for all the elements included in A_s.

Page 35, paragraph 4.3.3.3(3), line 1 should start:

The effective widths $b_{1,e2}$ and $b_{2,e1}$ should .....

and line 3 should read:

to $\chi f_{yb}/\gamma_{M1}$, so that:

Page 37, paragraph 4.3.3.3(5), replace expression (4.28) by two lines reading:

$$A_{s,red} = \chi A_s \left[\frac{f_{yb}/\gamma_{M1}}{\sigma_{com,Ed}} \right] \quad \text{but} \quad A_{s,red} \leq A_s \qquad \text{... (4.28)}$$

in which $\sigma_{com,Ed}$ is the calculated stress at the centreline of the stiffener.

Page 37, paragraph 4.3.3.3(6), modify line 2 to read:

by using a reduced thickness $t_{red} = tA_{s,red}/A_s$ for all the elements included in A_s.

Page 45, paragraph 5.4.1(1)P, expression (5.3b) should read:

$$M_{c,Rd} = f_{ya} W_{et} / \gamma_{M0} \qquad \text{... (5.3b)}$$

Page 53, paragraph 5.8(6)P, expression (5.15a) should read:

$$\bar{\lambda}_w = \sqrt{\frac{f_{yb}/\sqrt{3}}{\tau_{cr}}} \equiv \frac{s_w}{t} \sqrt{\frac{12(1 - \nu^2) f_{yb}}{\sqrt{3}\, \pi^2 E k_\tau}} \qquad \text{... (5.15a)}$$

Page 57, paragraph 5.9.3(2), delete line 6 reading:

s_s is the actual length of stiff bearing;

Page 58, paragraph 5.9.3(4), insert after line 8:

where:

s_s is the actual length of stiff bearing;

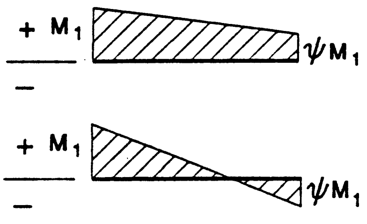
Page 60, paragraph 5.9.4(2), expression (5.24) should read:

$$\kappa_{a,s} = 1{,}45 - 0{,}05\, e_{max}/t \quad \text{but} \quad \kappa_{a,s} \leq 0{,}95 + 35\,000 t^2 e_{min}/(b_d^2 s_p) \qquad \text{... (5.24)}$$

Page 68, paragraph 6.3(1)P, line 4 should read:

in which χ_{LT} is obtained from the following:

Page 71, table 6.4, column 1, row 2, replace the second and third sketches by:



Page 77, figure 8.2 should be replaced by:

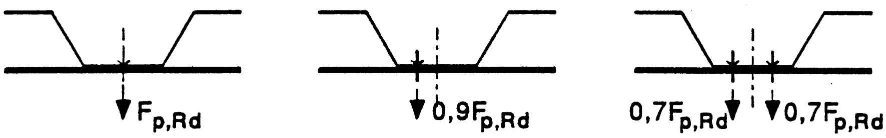


Figure 8.2: Reduction of tension resistance due to the position of fasteners

Page 81, table 8.4, row 2, line 2 should read:

$F_{b,Rd} \quad = \quad 2{,}5\,dtf_u/\gamma_{M2} \quad$ but $\quad F_{b,Rd} \leq (e_1 t/1{,}2)(f_u/\gamma_{M2})$

Page 82, table 8.5, row 2, lines 3 to 5 should read:

$F_{tb,Rd} = 2{,}7\sqrt{t}\;d_s f_u/\gamma_{M2}$ [with t in mm]

- if $t_1 > 2{,}5t$:

$F_{tb,Rd} = 2{,}7\sqrt{t}\;d_s f_u/\gamma_{M2}$ but $F_{tb,Rd} \leq 0{,}7 d_s^2 f_u/\gamma_{M2}$ and $F_{tb,Rd} \leq 3{,}1\,t d_s f_u/\gamma_{M2}$

Page 84, paragraph 8.6.3(3)P should read:

Arc spot welds shall have an interface diameter d_s of not less than 10 mm.

Page 101, paragraph 10.1.5.2(10), expression (10.18) should read:

$$C_{D,A} = \frac{h^2}{(1/K_A + 1/K_B) - 4(1-\nu^2)h^2(h_d + e)/(Et^3)} \qquad \text{... (10.18)}$$

Page 105, paragraph 10.2.2.2(1), in step 2 the last line should read:

I_a is the second moment of area of the wide flange, about its own centroid, see figure 10.8.

Page 105, paragraph 10.2.2.2(2), the first sentence should read:

The effects of shear lag need not be considered if $L / b_{u,eff} \geq 20$.

Page 109, paragraph 10.3.5(4), lines 2 to 4 should read:

$$T_{v,Rd} = 6E \sqrt[4]{I_a (t/b_u)^9} \quad \ldots (10.22)$$

where:

I_a is the second moment of area of the wide flange, about its own centroid, see figure 10.8;

EUROPEAN PRESTANDARD

PRÉNORME EUROPÉENNE

EUROPÄISCHE VORNORM

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English version

Eurocode 3: Design of steel structures - Part 1-3: General rules - Supplementary rules for cold formed thin gauge members and sheeting

Eurocode 3: Calcul des structures en acier - Partie 1-3: Règles générales - Règles supplémentaires pour les éléments minces formés à froid - Produits longs et produits plats

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This European Prestandard (ENV) was approved by CEN on 1993-06-04 as a prospective standard for provisional application. The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into an European Standard (EN).

CEN members are required to announce the existance of this ENV in the same way as for an EN and to make the ENV available promptly at national level in an appropriate form. It is permissible to keep conflicting national standards in force (in parallel to the ENV) until the final decision about the possible conversion of the ENV into an EN is reached.

CEN members are the national standards bodies of Austria, Belgium, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.

CEN

European Committee for Standardization
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart,36 B-1050 Brussels

Ref. No. ENV 1993-1-3:1996 E

Contents

Foreword

Objectives of the Eurocodes

(1) The "Structural Eurocodes" comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.

(2) They cover execution and control only to the extent that is necessary to indicate the quality of the construction products, and the standard of the workmanship, needed to comply with the assumptions of the design rules.

(3) Until the necessary set of harmonized technical specifications for products and for methods of testing their performance is available, some of the Structural Eurocodes cover some of these aspects in informative annexes.

Background to the Eurocode programme

(4) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonized technical rules for the design of building and civil engineering works which would initially serve as an alternative to the different rules in force in the various member states and would ultimately replace them. These technical rules became known as the "Structural Eurocodes".

(5) In 1990, after consulting their respective member states, the CEC transferred the work of further development, issue and updating of the Structural Eurocodes to CEN, and the EFTA Secretariat agreed to support the CEN work.

(6) CEN Technical Committee CEN/TC 250 is responsible for all Structural Eurocodes.

Eurocode programme

(7) Work is in hand on the following Structural Eurocodes, each generally consisting of a number of parts:

EN 1991	Eurocode 1	Basis of design and actions on structures;
EN 1992	Eurocode 2	Design of concrete structures;
EN 1993	Eurocode 3	Design of steel structures;
EN 1994	Eurocode 4	Design of composite steel and concrete structures;
EN 1995	Eurocode 5	Design of timber structures;
EN 1996	Eurocode 6	Design of masonry structures;
EN 1997	Eurocode 7	Geotechnical design;
EN 1998	Eurocode 8	Design provisions for earthquake resistance of structures;
EN 1999	Eurocode 9	Design of aluminium alloy structures.

(8) Separate sub-committees have been formed by CEN/TC 250 for the various Eurocodes listed above.

(9) This Part 1.3 of Eurocode 3 is published by CEN as a European Prestandard (ENV) with an initial life of three years.

(10) This Prestandard is intended for experimental application and for the submission of comments.

(11) After approximately two years CEN members will be invited to submit formal comments to be taken into account in determining future actions.

(12) Meanwhile feedback and comments on this Prestandard should be sent to the secretariat of CEN/TC 250/SC 3 at the following address:

BSI Standards
British Standards House
389 Chiswick High Road
London W4 4AL
England

or to your national standards organization.

National Application Documents (NAD's)

(13) In view of the responsibilities of the authorities in member countries for safety, health and other matters covered by the essential requirements of the Construction Products Directive (CPD), certain safety elements in this ENV have been assigned indicative values which are identified by ▭ ("boxed values"). The authorities in each member country are expected to review the "boxed values" and <u>may</u> substitute alternative definitive values for these safety elements for use in national application.

(14) Some of the supporting European or International Standards might not be available by the time this Prestandard is issued. It is therefore anticipated that a National Application Document (NAD) giving any substitute definitive values for safety elements, referencing compatible supporting standards and providing guidance on the national application of this Prestandard, will be issued by each member country or its Standards Organization.

(15) It is intended that this Prestandard is used in conjunction with the NAD valid in the country where the building or civil engineering works is located.

Matters specific to this Prestandard

(16) The Parts of ENV 1993 that are currently envisaged are:

ENV 1993-1-1	General rules and rules for buildings;
ENV 1993-1-2	Supplementary rules for structural fire design;
ENV 1993-1-3	Supplementary rules for cold formed thin gauge members and sheeting;
ENV 1993-1-4	Supplementary rules for stainless steels;
ENV 1993-2	Bridges and plated structures;
ENV 1993-3	Towers, masts and chimneys;
ENV 1993-4	Silos, tanks and pipelines;
ENV 1993-5	Piling;
ENV 1993-6	Crane supporting structures;
ENV 1993-7	Marine and maritime structures;
ENV 1993-8	Agricultural structures.

(17) Work on this Part 1.3 of Eurocode 3 was initiated by the Commission of the European Communities. The work was carried out in collaboration with a working group of the European Convention for Constructional Steelwork (ECCS) and a draft was issued in 1990 as a "Draft Eurocode 3 : Annex A".

(18) With the transfer of work on Structural Eurocodes to CEN, the responsibility for completing this document passed to CEN Technical Committee CEN/TC 250, sub-committee CEN/TC 250/SC 3.

(19) In this Part 1.3 of Eurocode 3, a distinction is made between three construction classes using cold formed thin gauge members and sheeting. The boxed values of the partial factors given in Part 1.3 are recommended values for Construction Class I and Construction Class II.

1 General

1.1 Scope

(1)P This Part 1.3 of ENV 1993 deals with the design of steel structures comprising cold formed thin gauge members and sheeting. It gives supplementary provisions for structural applications using cold formed steel products made from coated or uncoated thin gauge hot or cold rolled sheet or strip. It is intended to be used for the design of buildings and civil engineering works in conjunction with ENV 1993-1-1.

NOTE: The execution of steel structures comprised of cold formed thin gauge members and sheeting is covered in ENV 1090-2.

(2)P Methods are given for determining the load bearing capacity and serviceability of elements and connections under predominantly static loads. These design methods apply to steel members and profiled steel sheets that have been cold formed by such processes as cold-rolled forming or press-braking. They are also applicable for the design of profiled steel sheeting for composite steel and concrete slabs at the construction stage, see ENV 1994-1-1.

(3)P Methods are also given for stressed-skin design using steel sheeting as a structural diaphragm.

(4)P This Part 1.3 of ENV 1993 gives methods for design by calculation and for design assisted by testing.

NOTE: In the field of cold formed members and sheeting, many standard products are commonly used for which design by calculation might not lead to economical solutions, so it is frequently desirable to use design assisted by testing. Appropriate test methods are given in annex A.

(5) The methods for design by calculation apply only within stated ranges of material properties and geometrical proportions for which sufficient experience and test evidence is available. These limitations do not apply to design assisted by testing.

(6) This Part does not apply to cold formed structural hollow sections, for which reference should be made to ENV 1993-1-1.

1.2 Distinction between principles and application rules

(1)P Depending on the character of the individual paragraphs, a distinction is made in this Part between principles and application rules.

(2)P The principles comprise:

- general or definitive statements for which there is no alternative;
- requirements and analytical models for which no alternative is permitted unless specifically stated.

(3) The principles are identified by the letter P following the paragraph number.

(4)P The application rules are generally recognized rules that follow the principles and satisfy their requirements. Alternative design rules different from the application rules given in the Eurocode may be used, provided that it is shown that the alternative rule accords with the relevant principles and has at least the same reliability.

(5) In this Part the application rules are identified by a number in brackets, as in this paragraph.

1.3 Normative references

This European Prestandard incorporates, by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Prestandard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies.

EN 10002 *Metallic materials - Tensile testing:*

Part 1: *Method of test (at ambient temperature)*;

EN 10025 *Hot rolled products of non-alloy structural steels - Technical delivery conditions*;

EN 10113 *Hot rolled products in weldable fine grain structural steels:*

Part 2: *Delivery conditions for normalized/normalized rolled steels*;

Part 3: *Delivery conditions for thermomechanical rolled steels*;

EN 10143 *Continuously hot-dip metal coated steel sheet and strip - Tolerances on dimensions and shape*;

EN 10147 *Specification for continuously hot-dip zinc coated structural steel sheet - Technical delivery conditions*;

prEN 10149 *Hot rolled flat products made of high yield strength steels for cold forming:*

Part 2: *Delivery conditions for normalized/normalized rolled steels*;

Part 3: *Delivery conditions for thermomechanical rolled steels*;

EN 10155 *Structural steels with improved atmospheric corrosion resistance - Technical delivery conditions*;

ENV 1090 *Execution of steel structures:*

Part 2: *Rules for cold formed thin gauge members and sheeting*;

ENV 1991 *Eurocode 1: Basis of design and actions on structures:*

Part 1: *Basis of design*;

ENV 1993 *Eurocode 3: Design of steel structures:*

Part 1.1: *General rules : General rules and rules for buildings*;

ENV 1994 *Eurocode 4: Design of composite steel and concrete structures:*

Part 1.1: *General rules : General rules and rules for buildings*;

ISO 1000 *SI units*;

ISO 4997 *Cold reduced steel sheet of structural quality*.

1.4 Definitions

Supplementary to ENV 1993-1-1, for the purposes of this Part 1.3 of ENV 1993, the following definitions apply:

1.4.1 basic material: The flat sheet steel material out of which cold formed sections and profiled sheets are made by cold forming.

1.4.2 basic yield strength: The tensile yield strength of the basic material.

1.4.3 diaphragm action: Structural behaviour involving in-plane shear in the sheeting.

1.4.4 liner tray: Profiled sheet with large lipped edge stiffeners, suitable for interlocking with adjacent liner trays to form a plane of ribbed sheeting that is capable of supporting a parallel plane of profiled sheeting spanning perpendicular to the span of the liner trays.

1.4.5 partial restraint: Restriction of the lateral or rotational movement, or the torsional or warping deformation, of a member or plane element, that increases its buckling resistance in a similar way to a spring support, but to a lesser extent than a rigid support.

1.4.6 relative slenderness: A normalized slenderness ratio.

NOTE: In ENV 1993-1-1 relative slenderness is termed "non-dimensional slenderness".

1.4.7 restraint: Restriction of the lateral or rotational movement, or the torsional or warping deformation, of a member or plane element, that increases its buckling resistance to the same extent as a rigid support.

1.4.8 stressed-skin design: A design method that allows for the contribution made by diaphragm action in the sheeting to the stiffness and strength of a structure.

1.4.9 support: A location at which a member is able to transfer forces or moments to a foundation, or to another member or other structural component.

1.5 Symbols

(1) In addition to those given in ENV 1993-1-1, the following main symbols are used:

C Rotational spring stiffness;

K Linear spring stiffness;

θ Rotation.

(2) In addition to those given in ENV 1993-1-1, the following subscripts are used:

d Developed;

red Reduced;

spn Span;

sup Support;

TF Torsional-flexural.

(3) In addition to those used in ENV 1993-1-1, the following major symbols are used:

b_p Notional flat width of plane element;

h_w Web height, measured between system lines of flanges;

s_w Slant height of a web, measured between midpoints of corners.

(4) Further symbols are defined where they first occur.

1.6 Units

(1)P S.I. units shall be used in accordance with ISO 1000.

(2) The following units are recommended for use in calculations:

- forces and loads: kN, kN/m, kN/m^2;
- unit mass: kg/m^3;
- unit weight: kN/m^3;
- stresses and strengths: N/mm^2 ($\equiv$ MN/m^2 or MPa);
- bending moments: kNm;
- torsional moments: kNm.

1.7 Terminology

1.7.1 Form of sections

(1) Cold formed members and profiled sheets are steel products made from coated or uncoated hot rolled or cold rolled flat products. Within the permitted tolerances, they have a constant thickness over their entire length and may have either a constant or a variable cross-section.

NOTE: These products are obtained solely by cold forming, for example profiled by a rolling machine or formed by a press or press brake.

(2) The cross-sections of cold formed members and profiled sheets essentially comprise a number of curved elements joined by plane elements.

(3) Typical forms of sections for cold formed members include:

- single open sections as shown in figure 1.1(a);
- open built-up sections as shown in figure 1.1(b);
- closed built-up sections as shown in figure 1.1(c).

(4) Examples of cross-sections for cold formed members and sheets are illustrated as follows:

- compression members and tension members, in figure 1.2(a);
- beams and other members subject to bending, in figure 1.2(b);
- profiled sheets and liner trays, in figure 1.2(c).

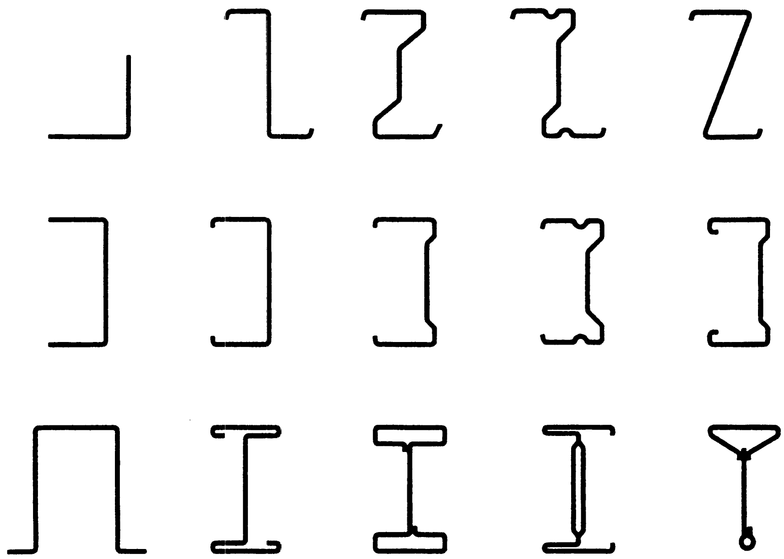
(5) Cross-sections of cold formed members and sheets can either be unstiffened or incorporate longitudinal stiffeners in their webs or flanges, or in both.

1.7.2 Form of stiffeners

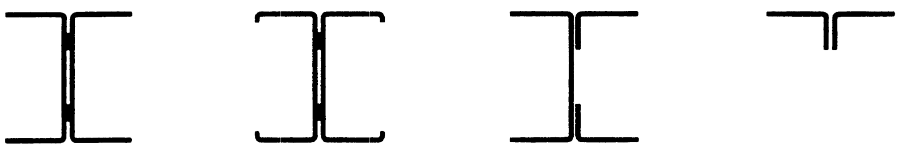
(1) Typical forms of stiffeners for cold formed members and sheets include:

- folds and bends, see figure 1.3(a);
- folded or curved grooves, see figure 1.3(b);
- other sections, bolted, rivetted or welded on, see figure 1.3(c).

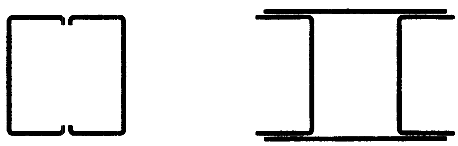
(2) Longitudinal flange stiffeners can be either edge stiffeners or intermediate stiffeners.



a) Single open sections



b) Open built-up sections



c) Closed built-up sections

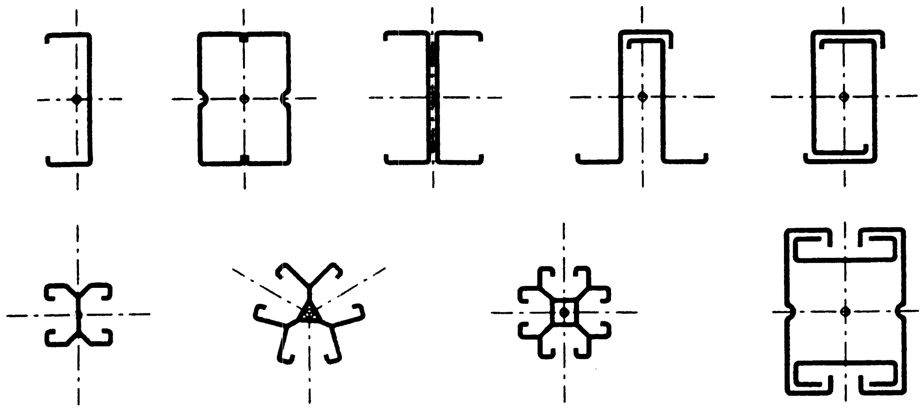
Figure 1.1: Typical forms of sections for cold formed members

(3) Typical edge stiffeners include:

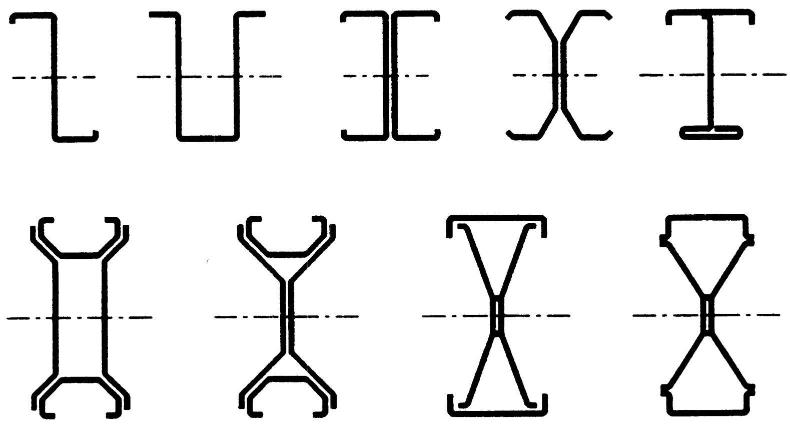
- single edge fold stiffeners or lips, see figure 1.4(a);
- double edge fold stiffeners, see figure 1.4(b).

(4) Typical intermediate longitudinal stiffeners are illustrated as follows:

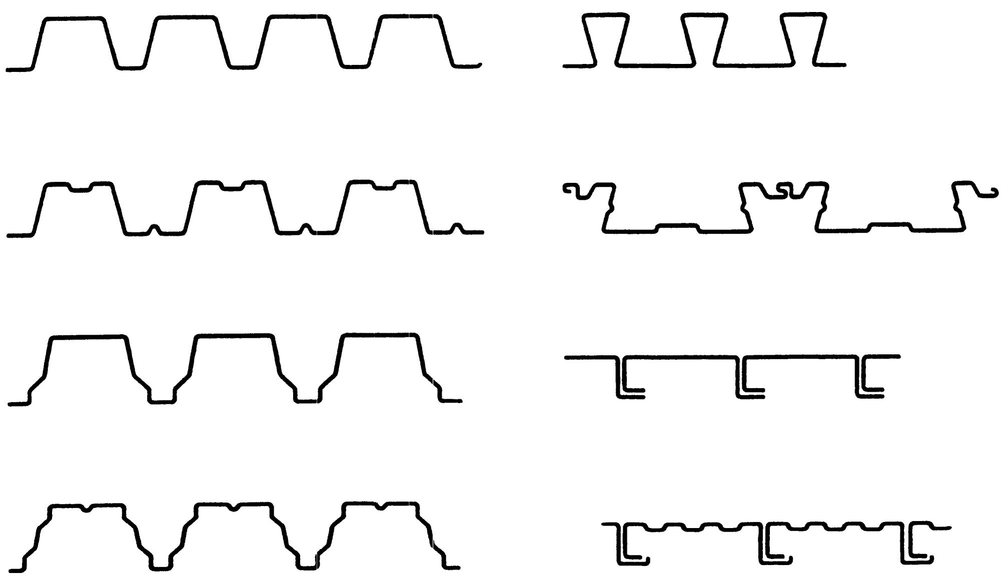
- for flanges, in figure 1.5(a);
- for webs in figure 1.5(b).



a) Compression members and tension members

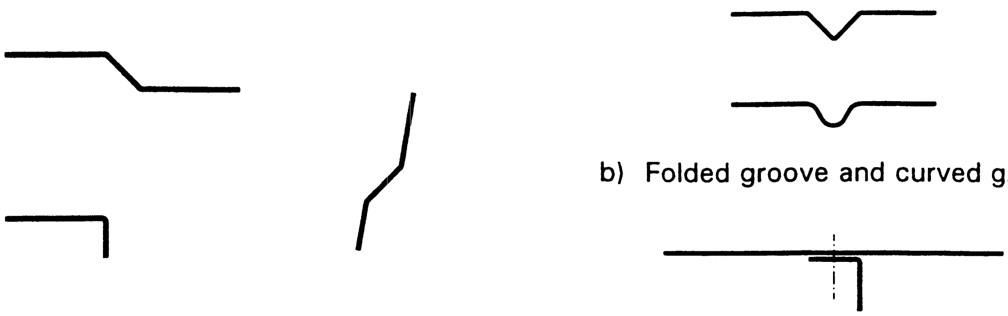


b) Beams and other members subject to bending



c) Profiled sheets and liner trays

Figure 1.2: Examples of cold formed members and profiled sheets



a) Folds and bends

b) Folded groove and curved groove

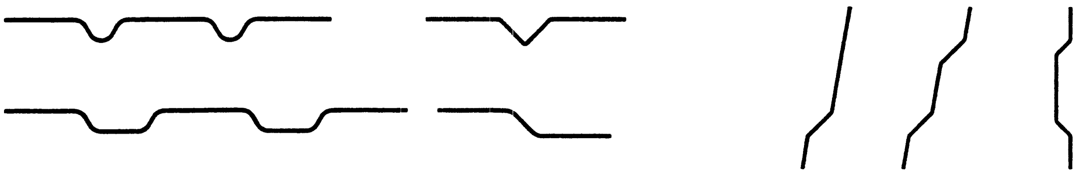
c) Bolted angle stiffener

Figure 1.3: Typical forms of stiffeners for cold formed members and sheeting

a) Single edge fold stiffeners

b) Double edge fold stiffeners

Figure 1.4: Typical edge stiffeners



a) Intermediate flange stiffeners

b) Intermediate web stiffeners

Figure 1.5: Typical intermediate longitudinal stiffeners

1.7.3 Cross-section dimensions

(1) Overall dimensions of cold formed thin gauge members and sheeting, including overall width b, overall height h, internal bend radius r and other external dimensions denoted by symbols without subscripts, such as a, c or d, are measured to the face of the material, unless stated otherwise.

(2) Unless stated otherwise, the other cross-sectional dimensions of cold formed thin gauge members and sheeting, denoted by symbols with subscripts, such as b_d, h_w or s_w, are measured either to the midline of the material, the system line of the element or the midpoint of the corner.

(3) In the case of sloping elements, such as webs of trapezoidal profiled sheets, the slant height s is measured parallel to the slope.

(4) The developed height of a web is measured along its midline, including any web stiffeners.

(5) The developed width of a flange is measured along its midline, including any intermediate stiffeners.

1.7.4 Convention for member axes

(1) In the Structural Eurocodes the general convention for member axes is:

- x - x along the member;
- y - y axis of the cross-section;
- z - z axis of the cross-section.

(2) For cold formed steel members the following axis convention is used in this Part 1.3 of ENV 1993:

- for monosymmetric cross-sections:
  - y - y the axis of symmetry of the cross-section;
  - z - z the other principal axis of the cross-section;
- otherwise:
  - y - y major axis;
  - z - z minor axis.
- where necessary:
  - u - u axis perpendicular to the height (if this does not coincide with y - y or z - z);
  - v - v axis parallel to the height (if this does not coincide with y - y or z - z).

NOTE: This differs from the axis convention used in ENV 1993-1-1 in order to give rules for torsional-flexural buckling that can be applied consistently to all cross-sections.

(3) The use of u - u and v - v axes is illustrated in figure 1.6.

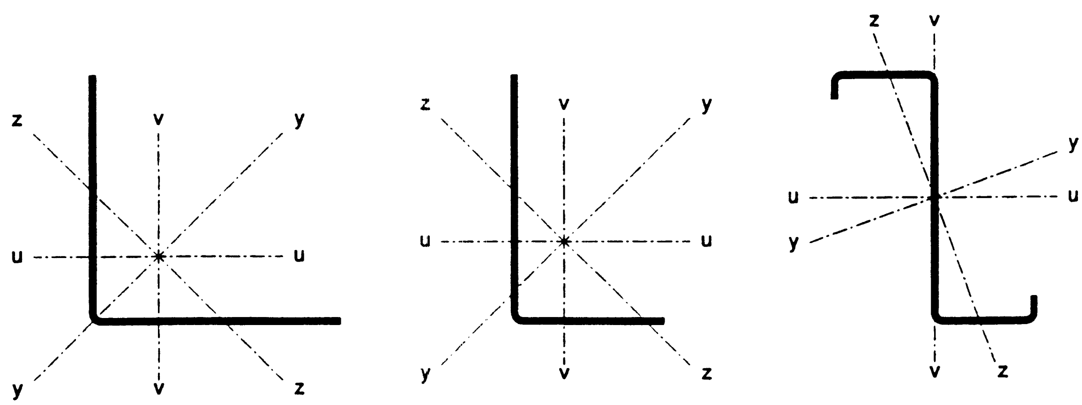


Figure 1.6: Axis convention

(4) For profiled sheets and liner trays the following axis convention is used in this Part 1.3 of ENV 1993:

- y - y axis parallel to the plane of sheeting;
- z - z axis perpendicular to the plane of sheeting.

2 Basis of design

2.1 General

(1)P For the purpose of differentiating levels of reliability, a distinction may be made between three "construction classes" defined as follows:

- **Construction Class I**: Construction where cold formed thin gauge members and sheeting are designed to contribute to the overall strength and stability of a structure;
- **Construction Class II**: Construction where cold formed thin gauge members and sheeting are designed to contribute to the strength and stability of individual structural elements;
- **Construction Class III**: Construction where cold formed sheeting is used as an element that only transfers loads to the structure.

(2)P The methods for design by calculation and for design assisted by testing given in this Part 1.3 of ENV 1993 may be adopted for all construction classes.

(3)P Appropriate partial factors shall be adopted for ultimate limit states and serviceability limit states.

(4)P The values of the partial factors given in this Part 1.3 of ENV 1993 shall be adopted for Construction Class I and Construction Class II.

2.2 Ultimate limit states

(1)P The principles for ultimate limit states given in Sections 2 and 5 in Part 1.1 of ENV 1993 shall also be applied to cold formed thin gauge members and sheeting.

(2) The application rules for ultimate limit states given in Sections 2 and 4 of Part 1.1 of ENV 1993 should also be applied, except where different application rules are given in this Part 1.3.

(3)P For verifications by calculation at ultimate limit states the partial factor γ_M shall be taken as follows:

- resistance of cross-sections where failure is caused by yielding:

$$\gamma_{M0} = \boxed{1,1}$$

- resistance of members and sheeting where failure is caused by buckling:

$$\gamma_{M1} = \boxed{1,1}$$

- resistance of net sections at bolt holes:

$$\gamma_{M2} = \boxed{1,25}$$

(4)P For values of γ_M for resistances of connections, see Section 8 of this Part 1.3.

2.3 Serviceability limit states

(1)P The principles for serviceability limit states given in Sections 2 and 4 of Part 1.1 of ENV 1993 shall also be applied to cold formed thin gauge members and sheeting, see 7.1(1)P in this Part 1.3.

(2) The application rules for serviceability limit states given in Sections 2 and 4 of Part 1.1 of ENV 1993 should also be applied, except where different application rules are given in Section 7 of this Part 1.3.

(3)P For verifications at serviceability limit states the partial factor $\gamma_{M,ser}$ shall be taken as follows:

$$\gamma_{M,ser} = \boxed{1,0}$$

2.4 Design assisted by testing

(1)P Verifications for ultimate limit states or serviceability limit states that rely on the results of testing shall be in accordance with Section 9.

(2) Test specimens for sheet profiles should normally comprise at least two complete corrugations, but a test specimen may comprise just one complete corrugation, provided that the stiffness of the corrugations is sufficient.

2.5 Durability

(1)P To ensure adequate durability of cold formed components under conditions relevant to both their intended use and their intended life, the following inter-related factors shall be taken into account at the design stage:

- the intended use of the structure;
- the required performance criteria;
- the expected environmental conditions;
- the composition, properties and performance of the materials;
- the effects of connecting different materials together;
- the shape of the members and the structural detailing;
- the quality of the workmanship and the level of control;
- the particular protective measures;
- the likely maintenance during the intended life.

(2)P The internal and external environmental conditions shall be estimated at the design stage in order to assess their significance in relation to durability and enable adequate provisions to be made for the protection of the materials.

(3) Special attention should be given to cases in which different materials are intended to act compositely, if these materials are such that electrochemical phenomena might produce conditions leading to corrosion.

(4) The environmental conditions prevailing from the time of manufacture, including those during transport and storage on site, should be taken into account.

3 Properties of materials and cross-sections

3.1 Structural steel

3.1.1 General

(1)P All steels used for cold formed members and profiled sheets shall be suitable for cold forming and welding. Steels used for members and sheets to be galvanized shall also be suitable for galvanizing.

(2)P The methods for design by calculation given in this Part 1.3 of ENV 1993 may be used for structural steels conforming with the European Standards and International Standards listed in table 3.1.

Table 3.1: Nominal values of basic yield strength f_{yb} and ultimate tensile strength f_u

Type of steel	Standard	Grade	f_{yb} N/mm^2	f_u N/mm^2
Hot rolled steel sheet of structural quality	EN 10025	S 235 S 275 S 355	235 275 355	360 430 510
Hot rolled steel sheet of high yield strength of structural quality	EN 10113: Part 2	S 275 N S 355 N S 420 N S 460 N	275 355 420 460	370 470 520 550
	EN 10113: Part 3	S 275 M S 355 M S 420 M S 460 M	275 355 420 460	360 450 500 530
Cold reduced steel sheet of structural quality	ISO 4997	CR 220 CR 250 CR 320	220 250 320	300 330 400
Continuous hot dip zinc coated carbon steel sheet of structural quality	EN 10147	Fe E 220 G Fe E 250 G Fe E 280 G Fe E 320 G Fe E 350 G	220 250 280 320 350	300 330 360 390 420
High yield strength steels for cold forming	prEN 10149: Part 2	S 315 MC S 355 MC S 420 MC S 460 MC S 500 MC S 550 MC	315 355 420 460 500 550	390 430 480 520 550 600
	prEN 10149: Part 3	S 260 NC S 315 NC S 355 NC S 420 NC	260 315 355 420	370 430 470 530

(3)P These design methods may also be applied to other structural steels with similar strength and toughness properties, provided that all of the following conditions are satisfied:

a) the steel satisfies the requirements for chemical analysis, mechanical tests and other control procedures to the extent and in the manner prescribed in the standards that are listed in table 3.1;

b) the ratio of the specified minimum ultimate tensile strength f_u to the specified minimum basic yield strength f_{yb} is not less than 1,2;

c) the steel is supplied either:

- to another recognized standard for structural steel sheet;
- with mechanical properties and chemical composition at least equivalent to one of the steel grades that are listed in table 3.1.

(4)P The nominal values of the basic yield strength f_{yb} and ultimate tensile strength f_u given in table 3.1 shall be adopted as characteristic values in design calculations. For other steels the characteristic values shall be based on the results of tensile tests carried out in accordance with EN 10002-1.

(5) It should be assumed that the properties of steel in compression are the same as those in tension.

(6)P Where the yield strength is specified using the symbol f_y the average yield strength f_{ya} may be used, if the conditions given in 3.1.2 are satisfied, otherwise the basic yield strength f_{yb} shall be used. Where the yield strength is specified using the symbol f_{yb} the basic yield strength f_{yb} shall be used.

(7)P For the steels covered by this Part 1.3 of ENV 1993, the other material properties to be used in design shall be taken as follows:

- modulus of elasticity: $E = 210\,000$ N/mm²;
- shear modulus: $G = E/2(1+\nu)$ N/mm²;
- Poisson's ratio: $\nu = 0{,}3$;
- coefficient of linear thermal elongation: $\alpha = 12 \times 10^{-6}$ 1/K;
- unit mass $\rho = 7850$ kg/m³.

3.1.2 Average yield strength

(1)P The average yield strength f_{ya} of a cross-section due to cold working may be determined from the results of full size tests in accordance with Section 9.

(2)P Alternatively the increased average yield strength f_{ya} may be determined by calculation using:

$$f_{ya} = f_{yb} + (f_u - f_{yb})knt^2/A_g \quad \text{but} \quad f_{ya} \leq (f_u + f_{yb})/2 \qquad \text{... (3.1)}$$

where:

A_g is the gross cross-sectional area;

k is a numerical coefficient that depends on the type of forming as follows:

- $k = 7$ for cold rolling;
- $k = 5$ for other methods of forming;

n is the number of 90° bends in the cross-section with an internal radius $r \leq 5t$ (fractions of 90° bends should be counted as fractions of n);

t is the nominal core thickness t_{cor} of the steel material before cold forming, exclusive of zinc or organic coatings, see 3.1.3.

(3)P The increased yield strength due to cold forming shall be taken into account only as follows:

- in axially loaded members in which the effective cross-sectional area A_{eff} equals the gross area A_g;
- in other cases in which it can be shown that the effects of cold forming lead to an increase in the load carrying capacity.

(4) In determining A_{eff} the yield strength f_y should be taken as f_{yb}.

(5) The average yield strength f_{ya} may be utilized in determining:

- the cross-section resistance of an axially loaded tension member;
- the cross-section resistance and the buckling resistance of an axially loaded compression member with a fully effective cross-section;
- the moment resistance of a cross-section with fully effective flanges.

(6) To determine the moment resistance of a cross-section with fully effective flanges, the cross-section may be subdivided into m nominal plane elements, such as flanges. Expression (3.1) may then be used to obtain values of increased yield strength $f_{y,i}$ separately for each nominal plane element i, provided that:

$$\sum_{i=1}^{m} A_{g,i} f_{y,i} / \sum_{i=1}^{m} A_{g,i} \leq f_{ya} \qquad \text{... (3.2)}$$

where:

$A_{g,i}$ is the gross cross-sectional area of nominal plane element i.

(7)P The increase in yield strength due to cold forming shall not be utilized for members that are subjected to heat treatment after forming at more than 520°C for more than one hour.

(8) Special attention should be paid to the fact that some heat treatments (especially annealing) might induce a reduced yield strength lower than the basic yield strength f_{yb}.

3.1.3 Thickness

(1)P The provisions for design by calculation given in this Part 1.3 of ENV 1993 may be used only for steel within the following ranges of nominal core thickness t_{cor} exclusive of zinc or organic coatings:

- for sheeting: $0{,}5\,\text{mm} \leq t_{cor} \leq 4{,}0\,\text{mm}$;
- for members: $1{,}0\,\text{mm} \leq t_{cor} \leq 8{,}0\,\text{mm}$.

(2)P Thicker or thinner material may also be used, provided that the load bearing capacity is determined by design assisted by testing in accordance with Section 9.

(3)P Because the design provisions for cold formed members and sheeting given in this Part 1.3 have been developed on the basis of thickness tolerances that are approximately half the tolerance values specified as "normal tolerances" in EN 10143, if larger tolerances are used the nominal values of thickness t_{nom} shall be adjusted to maintain the equivalent reliability.

(4) For continuously hot-dip metal coated members and sheeting of nominal thickness $t_{nom} < 1{,}5\,\text{mm}$ supplied with negative tolerances equal to the "special tolerances (S)" given in EN 10143, the design thickness t may be taken as equal to the nominal core thickness t_{cor}.

(5) In the case of continuously hot-dip metal coated steel sheet and strip conforming with EN 10147, the nominal core thickness t_{cor} may be taken as $t_{nom} - t_{zin}$ where t_{nom} is the nominal sheet thickness and t_{zin} is the total thickness of zinc coating, including both surfaces.

NOTE: For the usual Z 275 zinc coating, $t_{zin} = 0{,}04\,\text{mm}$.

3.2 Connecting devices

3.2.1 Bolt assemblies

(1)P Bolts, nuts and washers shall conform to the requirements given in ENV 1993-1-1.

3.2.2 Other types of mechanical fastener

(1)P The following additional types of mechanical fasteners may be used:

- self-tapping screws;
- cartridge-fired pins;
- blind rivets.

(2)P Self tapping screws may be:

- thread-forming self-tapping screws;
- thread-cutting self-tapping screws;
- self-drilling self-tapping screws.

(3) For details concerning suitable self-tapping screws, cartridge-fired pins and blind rivets reference should be made to ENV 1090: Part 2.*)

3.2.3 Welding consumables

(1)P Welding consumables shall conform to the requirements given in ENV 1993-1-1.

3.3 Section properties

3.3.1 General

(1)P Section properties shall be calculated according to normal good practice, taking due account of the sensitivity of the properties of the overall cross-section to any approximations used, see 3.3.4, and their influence on the predicted resistance of the member.

(2)P The effects of local buckling shall be taken into account by using effective cross-sections as specified in Section 4.

3.3.2 Gross cross-section

(1)P The properties of the gross cross-section shall be determined using the specified nominal dimensions. In calculating gross cross-sectional properties, holes for fasteners need not be deducted but allowance shall be made for large openings. Plates that are used solely in splices or as battens shall not be included.

3.3.3 Net area

(1)P The net area of a member cross-section, or of an element of a cross-section, shall be taken as its gross area minus appropriate deductions for all fastener holes and other openings.

(2)P In deducting holes for fasteners, the nominal hole diameter shall be used, not the fastener diameter.

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*) This document is in preparation.

(3) For countersunk holes, the area to be deducted should be the gross cross-sectional area of the hole, including the countersunk portion, in the plane of its axis.

(4)P Provided that the fastener holes are not staggered, the area to be deducted from the gross cross-sectional area shall be the maximum sum of the sectional areas of the fastener holes in any cross-section perpendicular to the direction of direct stress in the member.

(5)P Where the fastener holes are staggered, the area to be deducted shall be the greater of:

a) The deduction for non-staggered holes given in (4)P;

b) The sum of the sectional areas of all holes in any diagonal or zigzag line extending progressively across the member or element, see figure 3.1, minus an allowance for each gauge space p in the chain of holes. This allowance shall be taken as $0{,}25 s^2 t/p$ but not more than $0{,}6 s t$, where:

- p is the gauge space, i.e. the distance measured perpendicular to the direction of load transfer, between the centres of two consecutive holes in the chain;
- s is the staggered pitch, i.e. the distance, measured parallel to the direction of load transfer, between the centres of the same two holes;
- t is the thickness of the holed material.

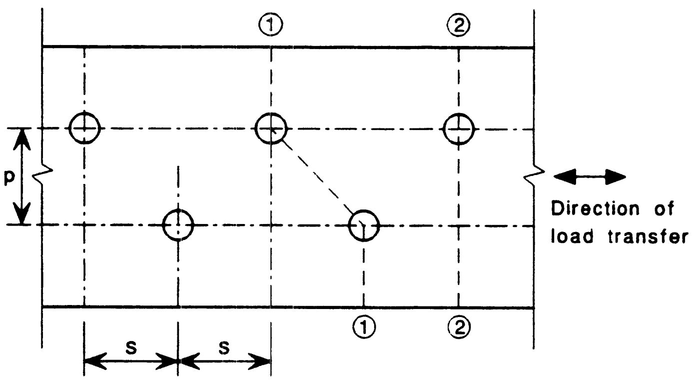


Figure 3.1: Staggered holes and appropriate sections

(6)P For cross-sections such as angles with holes in more than one plane, the spacing p shall be measured along the centre of thickness of the material, see figure 3.2.

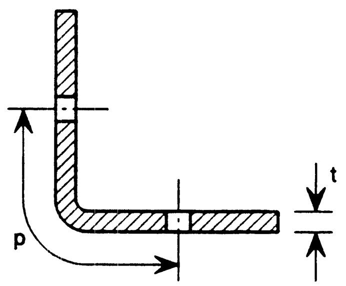


Figure 3.2: Angles with holes in both legs

(7)P In a built-up member where the critical chains of holes in each component part do not correspond with the critical chain of holes for the member as a whole, the resistances of any fasteners joining the parts between such chains of holes shall be taken into account in determining the resistance of the member.

NOTE: No general rules can be given for continuously perforated members because the resistance is influenced by the form and pattern of the perforations.

3.3.4 Influence of rounded corners

(1)P In cross-sections with rounded corners, the notional flat widths b_p of the plane elements shall be measured from the midpoints of the adjacent corner elements as indicated in figure 3.3.

(2)P In cross-sections with rounded corners, the calculation of section properties shall be based upon the actual geometry of the cross-section.

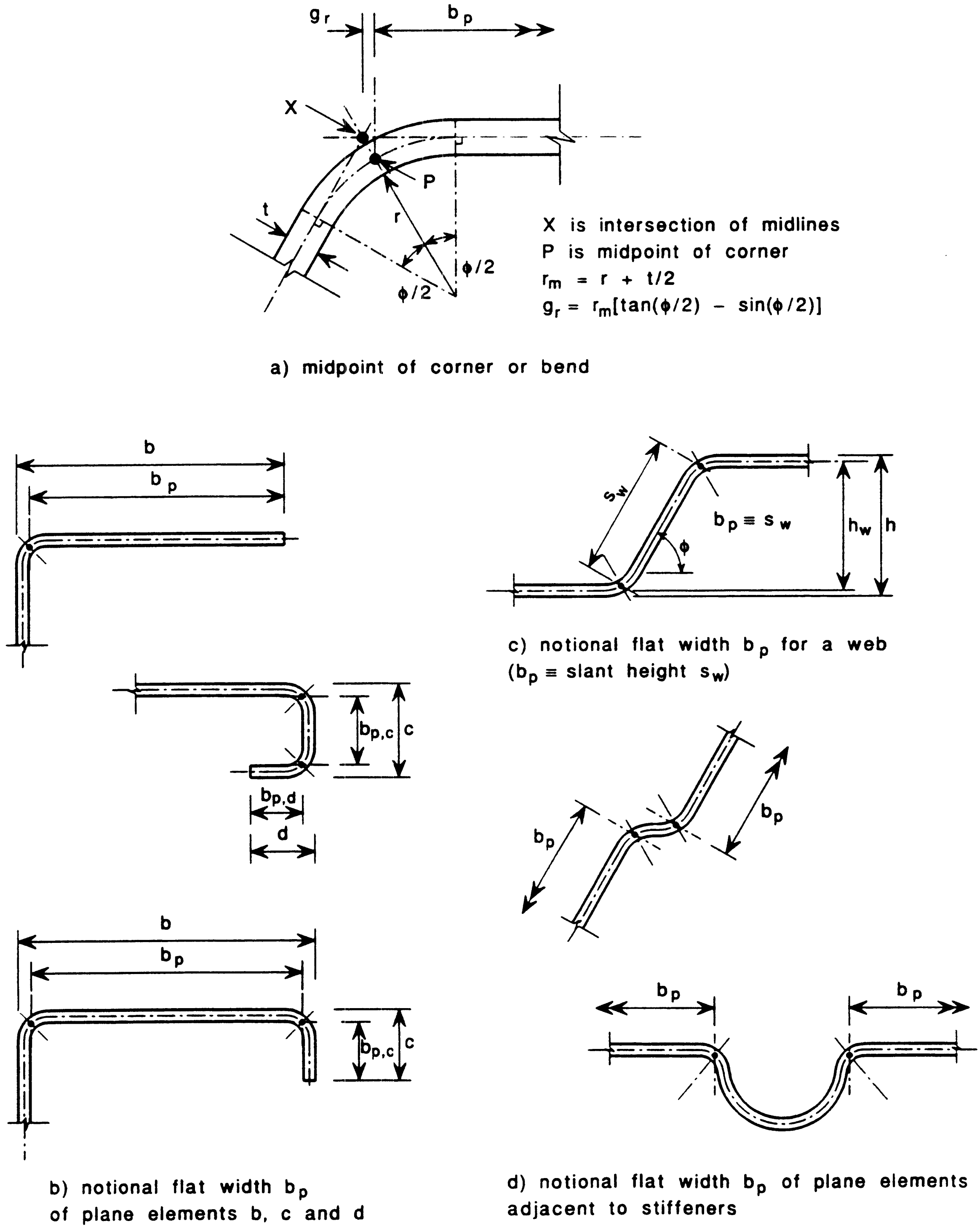


Figure 3.3: Notional widths of plane elements b_p allowing for corner radii

(3) The influence of rounded corners with internal radius $r \leq 5t$ and $r \leq 0{,}15b_p$ on section properties may be neglected, and the cross-section may be assumed to consist of plane elements with sharp corners.

(4) If the internal radius r exceeds the limits given in (3) the influence of rounded corners on section properties should be allowed for. This may be done with sufficient accuracy by reducing the properties calculated for an otherwise similar cross-section with sharp corners, see figure 3.4, using the following approximations:

$$A_g \approx A_{g,sh}(1 - \delta) \qquad \text{... (3.3a)}$$

$$I_g \approx I_{g,sh}(1 - 2\delta) \qquad \text{... (3.3b)}$$

$$I_w \approx I_{w,sh}(1 - 4\delta) \qquad \text{... (3.3c)}$$

with:

$$\delta = 0{,}43 \sum_{j=1}^{n} r_j \Big/ \sum_{i=1}^{m} b_{p,i} \qquad \text{... (3.3d)}$$

where:

- A_g is the area of the gross cross-section;
- $A_{g,sh}$ is the value of A_g for a cross-section with sharp corners;
- $b_{p,i}$ is the notional flat width of plane element i for a cross-section with sharp corners;
- I_g is the second moment of area of the gross cross-section;
- $I_{g,sh}$ is the value of I_g for a cross-section with sharp corners;
- I_w is the warping constant of the gross cross-section;
- $I_{w,sh}$ is the value of I_w for a cross-section with sharp corners;
- m is the number of plane elements;
- n is the number of curved elements;
- r_j is the internal radius of curved element j.

(5) The reductions given by expression (3.3) may also be applied in calculating the effective section properties A_{eff}, $I_{y,eff}$, $I_{z,eff}$ and $I_{w,eff}$, provided that the notional flat widths of the plane elements are measured to the points of intersection of their midlines.

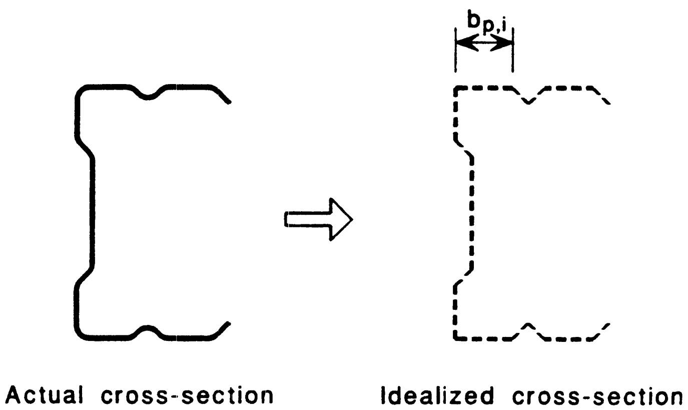


Figure 3.4: Approximate allowance for rounded corners

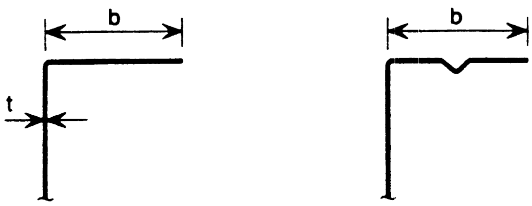
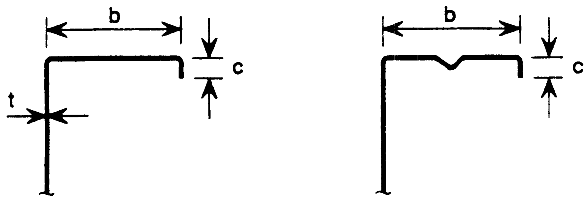
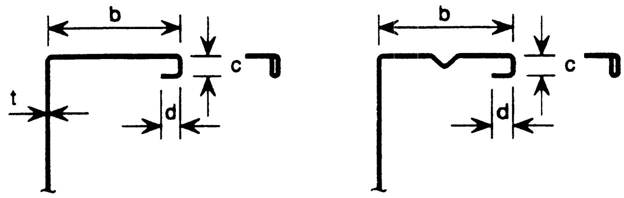
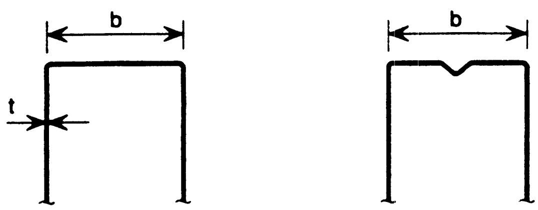
3.4 Geometrical proportions

(1)P The provisions for design by calculation given in this Part 1.3 of ENV 1993 shall not be applied to cross-sections outside the range of width-to-thickness ratios for which sufficient experience and verification by testing is available.

(2) The maximum width-to-thickness ratios b/t and h/t given in table 3.2 may be assumed to represent the field for which sufficient experience and verification by testing is already available.

(3) Cross-sections with larger width-to-thickness ratios may also be used, provided that their resistance at ultimate limit states and their behaviour at serviceability limit states are verified by testing in accordance with Section 9.

Table 3.2: Maximum width-to-thickness ratios

Element of cross-section	Maximum value
	$b/t \leq 50$
	$b/t \leq 60$
	$b/t \leq 90$
	$b/t \leq 500$
	$45° \leq \phi \leq 90°$ $h/t \leq 500 \sin\phi$

(4) In order to provide sufficient stiffness and to avoid primary buckling of the stiffener itself, the sizes of stiffeners should be within the following ranges:

$$0,2 \leq c/b \leq 0,6 \qquad \ldots (3.4a)$$

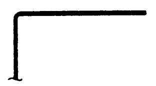
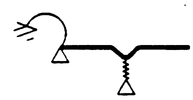
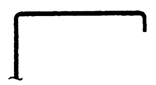
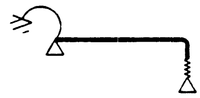
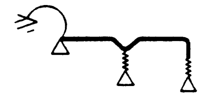
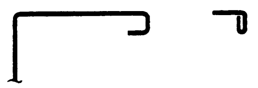
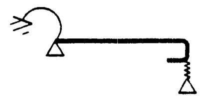
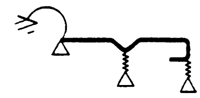
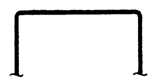
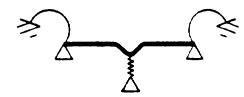
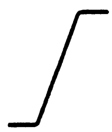
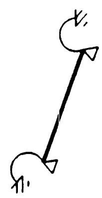
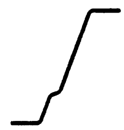
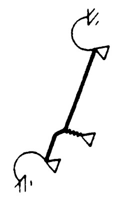
$$0,1 \leq d/b \leq 0,3 \qquad \ldots (3.4b)$$

in which the dimensions b, c and d are as indicated in table 3.2.

3.5 Modelling for cross-section analysis

(1) The elements of a cross-section may be modelled for analysis as indicated in table 3.3.

Table 3.3: Modelling of elements of a cross-section

Type of element	Model	Type of element	Model
			
			
			
			
			

4 Local buckling

4.1 General

(1)P The effects of local buckling shall be taken into account in determining the resistance and stiffness of cold formed members and sheeting.

(2)P This may be done by using effective cross-sectional properties, calculated on the basis of the effective widths of those elements that are prone to local buckling.

(3)P The possible shift of the centroidal axis of the effective cross-section relative to the centroidal axis of the gross cross-section shall be taken into account.

(4) In determining resistance to local buckling, the yield strength f_y should be taken as f_{yb}.

(5) In determining the resistance of a cross-section, the effective width of a compression element should be based on the compressive stress $\sigma_{com,Ed}$ in the element when the cross-section resistance is reached.

(6) For serviceability verifications, the effective width of a compression element should be based on the compressive stress $\sigma_{com,Ed,ser}$ in the element under the serviceability limit state loading.

4.2 Plane elements without stiffeners

(1)P The effective widths of compression elements shall be obtained from table 4.1 for doubly supported compression elements or table 4.2 for outstand compression elements.

(2)P The notional flat width b_p of a plane element shall be determined as specified in 3.3.4. In the case of plane elements in a sloping web, the appropriate slant height shall be used.

NOTE: In ENV 1993-1-1 the symbol $\bar{b}$ is used for the notional flat width of a plane element.

(3)P The reduction factor ρ used in tables 4.1 and 4.2 to determine b_{eff} shall be based on the largest compressive stress $\sigma_{com,Ed}$ in the relevant element (calculated on the basis of the effective cross-section and taking account of possible second order effects), when the resistance of the cross-section is reached.

(4) If $\sigma_{com,Ed} = f_{yb}/\gamma_{M1}$ the reduction factor ρ should be obtained from the following:

- if $\bar{\lambda}_p \leq 0{,}673$: $\rho = 1{,}0$ ... (4.1a)
- if $\bar{\lambda}_p > 0{,}673$: $\rho = (1{,}0 - 0{,}22/\bar{\lambda}_p)/\bar{\lambda}_p$ ... (4.1b)

in which the plate slenderness $\bar{\lambda}_p$ is given by:

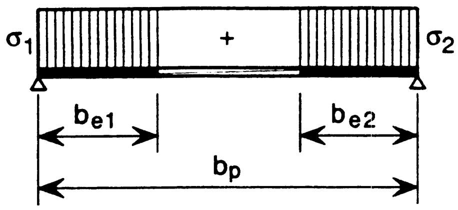
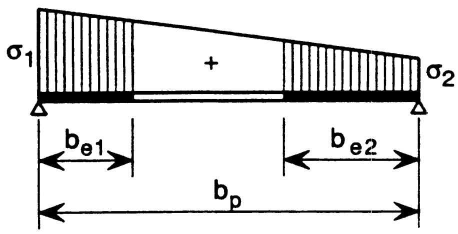
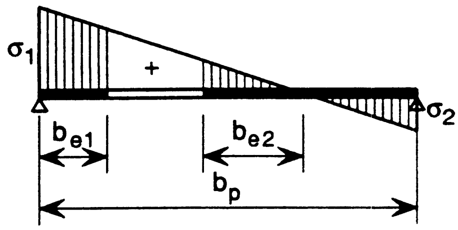
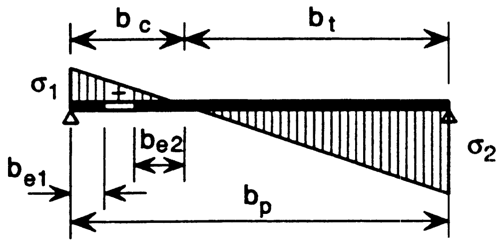
$$\bar{\lambda}_p = \sqrt{\frac{f_{yb}}{\sigma_{cr}}} \equiv \frac{b_p}{t}\sqrt{\frac{12(1-\nu^2)f_{yb}}{\pi^2 E k_\sigma}} \cong 1{,}052\,\frac{b_p}{t}\sqrt{\frac{f_{yb}}{E k_\sigma}} \cong \frac{b_p/t}{28{,}4\,\varepsilon\sqrt{k_\sigma}} \quad \text{... (4.2)}$$

where:

k_σ is the relevant buckling factor from table 4.1 or 4.2;

ε is the ratio $\sqrt{235/f_{yb}}$ with f_{yb} in N/mm^2.

Table 4.1: Doubly supported compression elements

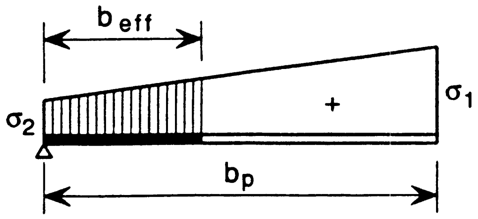
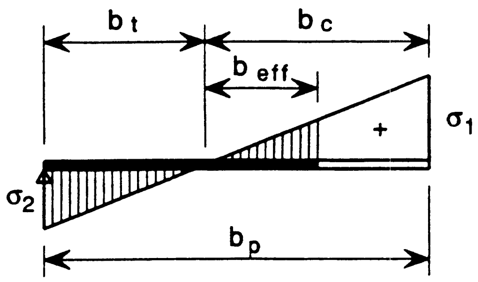
<table>
<tr><th>Stress distribution
[compression positive]</th><th>Effective width b_{eff}</th></tr>
<tr><td>
</td><td>$\psi = +1$:
$b_{eff} = \rho b_p$
$b_{e1} = 0{,}5 b_{eff}$
$b_{e2} = 0{,}5 b_{eff}$</td></tr>
<tr><td>
</td><td>$+1 > \psi \geq 0$:
$b_{eff} = \rho b_p$
$b_{e1} = \dfrac{2 b_{eff}}{5 - \psi}$
$b_{e2} = b_{eff} - b_{e1}$</td></tr>
<tr><td>
</td><td>$0 > \psi \geq -1$:
$b_{eff} = \rho b_c$
$b_{e1} = 0{,}4 b_{eff}$
$b_{e2} = 0{,}6 b_{eff}$</td></tr>
<tr><td>
</td><td>$\psi < -1$:
$b_{eff} = \rho b_c$
$b_{e1} = 0{,}4 b_{eff}$
$b_{e2} = 0{,}6 b_{eff}$</td></tr>
</table>

$\psi = \sigma_2 / \sigma_1$	+1	$+1 > \psi > 0$	0	$0 > \psi > -1$	-1	$-1 > \psi > -3$
Buckling factor k_σ	4,0	$\dfrac{8{,}2}{1{,}05 + \psi}$	7,81	$7{,}81 - 6{,}29\psi + 9{,}78\psi^2$	23,9	$5{,}98(1 - \psi)^2$

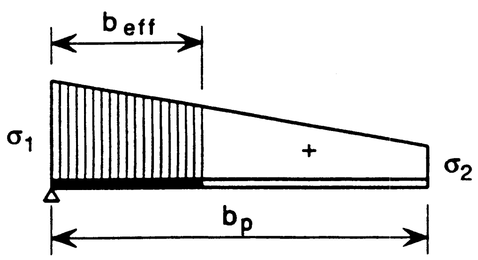
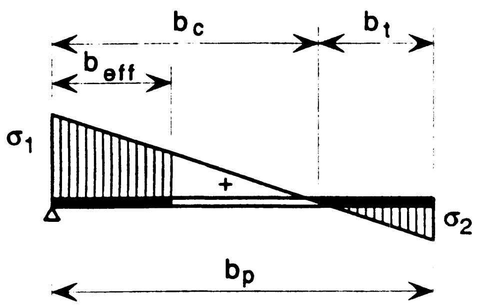
Alternatively, for $+1 \geq \psi \geq -1$:

$$k_\sigma = \frac{16}{\left[(1 + \psi)^2 + 0{,}112(1 - \psi)^2\right]^{0{,}5} + (1 + \psi)}$$

Table 4.2: Outstand compression elements

Stress distribution [compression positive]	Effective width b_{eff}
	$+1 > \psi \geq 0$: $b_{eff} = \rho b_p$
	$\psi < 0$: $b_{eff} = \rho b_c$

$\psi = \sigma_2 / \sigma_1$	+1	0	1	$+1 \geq \psi \geq -1$
Buckling factor k_σ	0,43	0,57	0,85	$0,57 - 0,21\psi = 0,07\psi^2$

Stress distribution [compression positive]	Effective width b_{eff}
	$+1 > \psi \geq 0$: $b_{eff} = \rho b_p$
	$\psi < 0$: $b_{eff} = \rho b_c$

$\psi = \sigma_2 / \sigma_1$	+1	$+1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$\frac{0,578}{\psi + 0,34}$	1,70	$1,70 - 5\psi + 17,1\psi^2$	23,8

(5) If $\sigma_{com,Ed} < f_{yb}/\gamma_{M1}$ the reduction factor ρ should be determined as follows:

- **Alternative 1**: Use expressions (4.1a) and (4.1b) but replace the plate slenderness $\bar{\lambda}_p$ by the reduced plate slenderness $\bar{\lambda}_{p,red}$ given by:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\frac{\sigma_{com,Ed}}{f_{yb}/\gamma_{M1}}} \qquad \ldots (4.3)$$

- **Alternative 2**: Replace expressions (4.1a) and (4.1b) by expressions (4.4a) and (4.4b) as follows:
  - if $\bar{\lambda}_{p,red} \leq 0{,}673$: $\rho = 1{,}0$ ... (4.4a)
  - if $\bar{\lambda}_{p,red} > 0{,}673$:

$$\rho = \frac{1 - 0{,}22/\bar{\lambda}_{p,red}}{\bar{\lambda}_{p,red}} + 0{,}18\frac{\bar{\lambda}_p - \bar{\lambda}_{p,red}}{\bar{\lambda}_p - 0{,}6} \quad \text{but} \quad \rho \leq 1{,}0 \qquad \ldots (4.4b)$$

(6) For effective widths at serviceability limit states ρ should be determined as follows:

- **Alternative 1**: Use expressions (4.1a) and (4.1b) but replace the ultimate limit states plate slenderness $\bar{\lambda}_p$ by the serviceability limit states plate slenderness $\bar{\lambda}_{p,ser}$ given by:

$$\bar{\lambda}_{p,ser} = \bar{\lambda}_p \sqrt{\frac{\sigma_{com,Ed,ser}}{f_{yb}}} \qquad \ldots (4.5)$$

where:

$\sigma_{com,Ed,ser}$ is the largest compressive stress in the relevant element (calculated on the basis of the effective cross-section) under the serviceability limit state loading.

- **Alternative 2**: Use expressions (4.4a) and (4.4b) but replace the reduced plate slenderness $\bar{\lambda}_{p,red}$ by the serviceability limit states plate slenderness $\bar{\lambda}_{p,ser}$ from expression (4.5).

(7) In determining the effective width of a flange element subject to stress gradient, the stress ratio ψ used in tables 4.1 and 4.2 may be based on the properties of the gross cross-section.

(8) In determining the effective width of a web element the stress ratio ψ used in table 4.1 may be obtained using the effective area of the compression flange but the gross area of the web.

(9) Optionally the effective section properties may be refined by repeating (7) and (8) iteratively, but using the effective cross-section already found in place of the gross cross-section.

(10) In the case of webs of trapezoidal profiled sheets under stress gradient, the simplified method given in 4.3.4 may be used.

4.3 Plane elements with edge or intermediate stiffeners

4.3.1 General

(1)P The design of compression elements with edge or intermediate stiffeners shall be based on the assumption that the stiffener behaves as a compression member with continuous partial restraint, with a spring stiffness that depends on the boundary conditions and the flexural stiffness of the adjacent plane elements.

(2) The spring stiffness of a stiffener should be determined by applying a unit load per unit length u as illustrated in figure 4.1. The spring stiffness K per unit length may be determined from:

$$K = u/\delta \qquad \text{... (4.6)}$$

where:

δ is the deflection of the stiffener due to the unit load u.

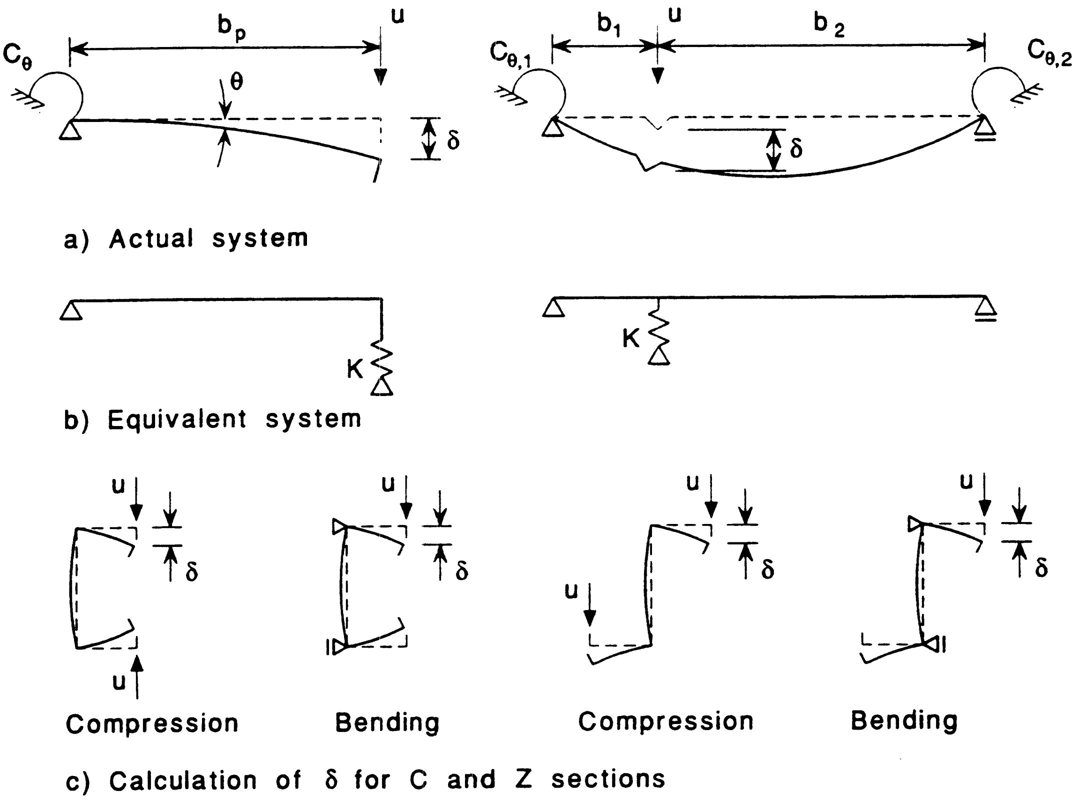


Figure 4.1: Determination of spring stiffness

(3) In determining the values of the rotational spring stiffnesses C_θ, $C_{\theta,1}$ and $C_{\theta,2}$ from the geometry of the cross-section, account should be taken of the possible effects of other stiffeners that exist on the same element, or on any other element of the cross-section that is subject to compression.

(4) For an edge stiffener, the deflection δ should be obtained from:

$$\delta = \theta\, b_p + \frac{u\, b_p^{\,3}}{3} \times \frac{12\left(1 - \nu^2\right)}{E t^3} \qquad \text{... (4.7)}$$

with:

$$\theta = u\, b_p / C_\theta$$

(5) In the case of the edge stiffeners of lipped C-sections and lipped Z-sections, C_θ should be determined with the unit loads u applied as shown in figure 4.1(c).

(6) For an intermediate stiffener, as a conservative alternative the values of the rotational spring stiffnesses $C_{\theta,1}$ and $C_{\theta,2}$ may be taken as equal to zero, and the deflection δ may be obtained from:

$$\delta = \frac{u\, b_1^2\, b_2^2}{3(b_1 + b_2)} \times \frac{12(1-\nu^2)}{E t^3} \qquad \text{... (4.8)}$$

(7) The reduction factor χ for the flexural buckling resistance of a stiffener should be obtained from 6.2.1(2)P using buckling curve a_0 (imperfection factor $\alpha = 0{,}13$) for the relative slenderness $\bar{\lambda}$ from:

$$\bar{\lambda} = \sqrt{f_{yb} / \sigma_{cr,s}} \qquad \text{... (4.9)}$$

where:

$\sigma_{cr,s}$ is the elastic critical stress for the stiffener from 4.3.2, 4.3.3 or 4.3.4.

4.3.2 Plane elements with edge stiffeners

4.3.2.1 Conditions

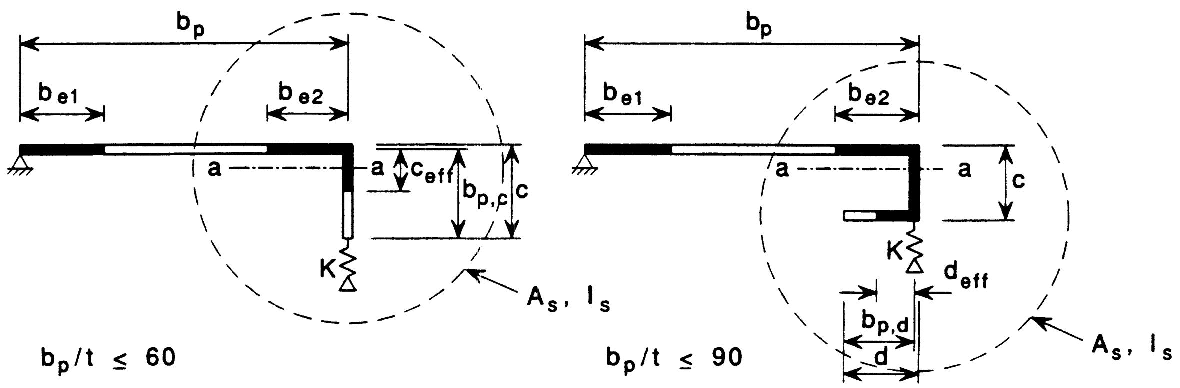
(1) An edge stiffener may be either a single edge fold or a double edge fold as illustrated in figure 4.2.

(2)P An edge stiffener shall not be taken into account in determining the resistance of the plane element to which it is attached unless the following conditions are met:

- the angle between the stiffener and the plane element is not less than 45° and not more than 135°;
- the outstand c is not less than $0{,}2b_p$, where b_p and c are as shown in figure 4.2;
- the ratio b_p / t is not more than 60 for a single edge fold stiffener, or 90 for a double edge fold stiffener.

(3) If the criteria in (1) and (2)P are met, the effectiveness of the stiffener may be determined from either:

- the general procedure given in 4.3.2.2;
- the simplified procedure given in 4.3.2.3.



a) single edge fold

b) double edge fold

Figure 4.2: Edge stiffeners

4.3.2.2 General procedure

(1) The cross-section of an edge stiffener should be taken as comprising the effective portions of the stiffener, element c or elements c and d as shown in figure 4.2, plus the adjacent effective portion of the plane element b_p.

(2) The procedure, which is illustrated in figure 4.3, should be carried out in steps as follows:

- **Step 1**: Obtain an initial effective cross-section for the stiffener using effective widths determined by assuming that the stiffener gives full restraint and that $\sigma_{com,Ed} = f_{yb}/\gamma_{M1}$, see (3) to (5);
- **Step 2**: Use the initial effective cross-section of the stiffener to determine the reduction factor for flexural buckling, allowing for the effects of the continuous spring restraint, see (6) and (7);
- **Step 3**: Iterate to refine the value of the reduction factor for buckling of the stiffener, see (8) and (9).

(3) Initial values of the effective widths b_{e1} and b_{e2} shown in figure 4.2 should be determined from clause 4.2 by assuming that the plane element b_p is doubly supported, see table 4.1.

(4) Initial values of the effective widths c_{eff} and d_{eff} shown in figure 4.2 should be obtained as follows:

a) for a single edge fold stiffener:

$$c_{eff} = \rho b_{p,c} \qquad \text{... (4.10a)}$$

with ρ obtained from 4.2(4), except using a value of the buckling factor k_σ given by the following:

- if $b_{p,c}/b_p \leq 0{,}35$:

$$k_\sigma = 0{,}5 \qquad \text{... (4.10b)}$$

- if $0{,}35 < b_{p,c}/b_p \leq 0{,}6$:

$$k_\sigma = 0{,}5 - 0{,}83 \times \sqrt[3]{(b_{p,c}/b_p - 0{,}35)^2} \qquad \text{... (4.10c)}$$

b) for a double edge fold stiffener:

$$c_{eff} = \rho b_{p,c} \qquad \text{... (4.10d)}$$

with ρ obtained from 4.2(4) with a buckling factor k_σ for a doubly supported element from table 4.1;

$$d_{eff} = \rho b_{p,d} \qquad \text{... (4.10e)}$$

with ρ obtained from 4.2(4) with a buckling factor k_σ for an outstand element from table 4.2.

(5) The effective cross-sectional area of the edge stiffener A_s should be obtained from:

$$A_s = t(b_{e2} + c_{eff} + d_{eff}) \qquad \text{... (4.11)}$$

(6) The elastic critical buckling stress $\sigma_{cr,s}$ for an edge stiffener should be obtained from:

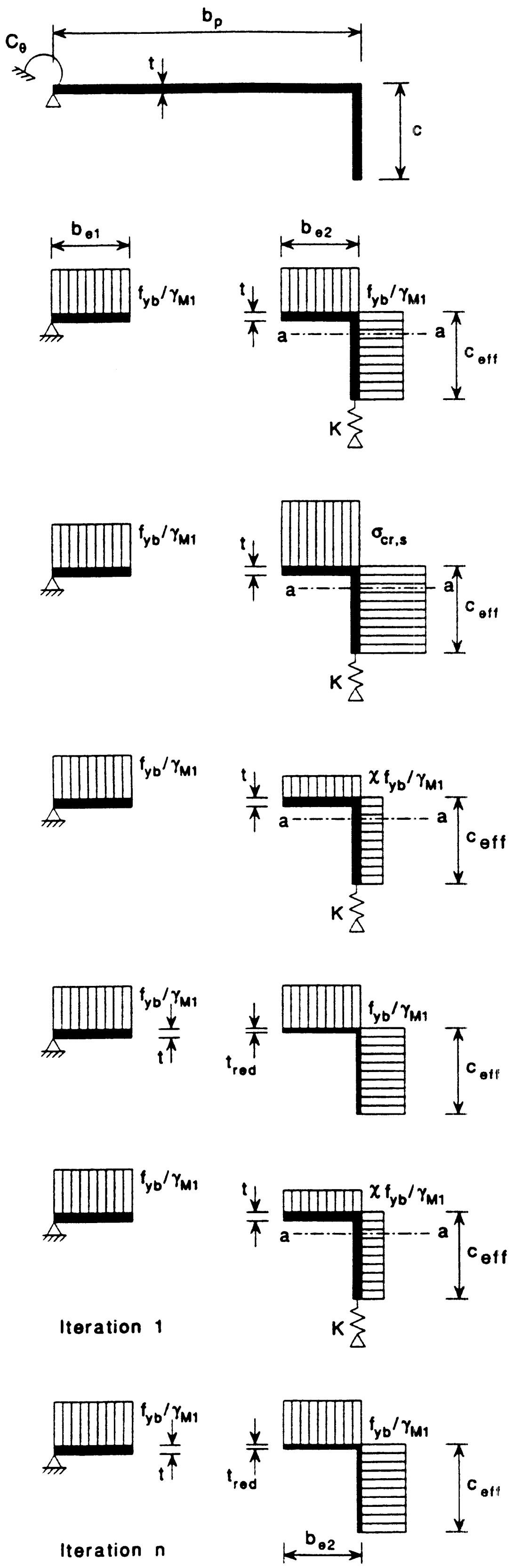
$$\sigma_{cr,s} = \frac{2\sqrt{KEI_s}}{A_s} \qquad \text{... (4.12)}$$

where:

K is the spring stiffness per unit length, see 4.3.1(2).

I_s is the effective second moment of area of the stiffener, taken as that of its effective area A_s about the centroidal axis a - a of its effective cross-section, see figure 4.2.

(7) The reduction factor χ for the flexural buckling resistance of an edge stiffener should be obtained from the value of $\sigma_{cr,s}$ using the method given in 4.3.1(7).



a) Gross cross-section and boundary conditions

b) **Step 1**: Effective cross-section for $K = \infty$ based on $\sigma_{com,Ed} = f_{yb}/\gamma_{M1}$

c) **Step 2**: Elastic critical stress $\sigma_{cr,s}$ for effective area of stiffener A_s from step 1

d) Reduced strength $\chi f_{yb}/\gamma_{M1}$ for effective area of stiffener A_s, with reduction factor χ based on $\sigma_{cr,s}$

e) Reduced thickness t_{red} for effective width of stiffener, with reduction factor χ based on $\sigma_{cr,s}$

f) **Step 3**: Repeat step 2 using reduced effective area of stiffener $A_{s,red}$ from previous iteration, continuing until $\chi_n \approx \chi_{(n-1)}$ but $\chi_n \leq \chi_{(n-1)}$

g) Adopt an effective cross-section with b_{e2}, c_{eff} and reduced thickness t_{red} corresponding to χ_n

Figure 4.3: Compression resistance of a flange with an edge stiffener

(8) If $\chi < 1$ it may optionally be refined iteratively, starting the iteration with modified values of ρ obtained using 4.2(5) with $\sigma_{com,Ed}$ equal to $\chi f_{yb}/\gamma_{M1}$, so that:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi} \quad \text{... (4.13)}$$

(9) If iteration is carried out, it should be continued until the current value of χ is approximately equal to, but not more than, the previous value.

(10) The reduced effective area of the stiffener $A_{s,red}$ allowing for flexural buckling should be taken as:

$$A_{s,red} = \chi A_s \quad \text{... (4.14)}$$

(11) In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = \chi t$ for all the elements included in A_s.

(12) The effective section properties at serviceability limit states should be based on the design thickness t.

4.3.2.3 Simplified procedure

(1) As an alternative to the general procedure given in 4.3.2.2, the following simplified procedure may be used to determine the reduced effective area $A_{s,red}$ of an edge stiffener as shown in figure 4.2.

(2) The effective cross-sectional area of the edge stiffener A_s should be obtained from:

$$A_s = t(b_{e2} + c_{eff} + d_{eff}) \quad \text{... (4.15)}$$

in which the effective widths b_{e2}, c_{eff} and d_{eff} should be obtained as in 4.3.2.2(3) and (4), except that ρ should be obtained from 4.2(5) with $\sigma_{com,Ed}$ equal to $\chi f_y/\gamma_{M1}$, so that:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi} \quad \text{... (4.16)}$$

(3) The reduction factor χ may be taken as equal to 0,5 if:

$$I_s \geq 0{,}31(1{,}5 + h/b_p)(f_{yb}/E)^2(b_p/t)^3 A_s^2 \quad \text{... (4.17)}$$

otherwise the reduction factor χ may be taken as approximately equal to 1,0 if:

$$I_s \geq 4{,}86(1{,}5 + h/b_p)(f_{yb}/E)^2(b_p/t)^3 A_s^2 \quad \text{... (4.18)}$$

where:

b_p is the notional flat width of the plane outstand element, see figure 4.2;

h is the overall depth of the adjacent web;

I_s is the effective second moment of area of the edge stiffener, taken as that of its effective area A_s about the centroidal axis a - a of its effective cross-section, see figure 4.2.

(4) The reduced effective area of the stiffener $A_{s,red}$ allowing for flexural buckling should be taken as:

$$A_{s,red} = \chi A_s \quad \text{... (4.19)}$$

(5) In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = \chi t$ for all the elements included in A_s.

(6) The effective section properties at serviceability limit states should be based on the design thickness t for all values of I_s.

4.3.3 Plane elements with intermediate stiffeners

4.3.3.1 Conditions

(1) Intermediate stiffeners may be formed by grooves or bends.

(2) The stiffeners should be equally shaped and not more than two in number.

(3) If the criteria in (1) and (2) are met the effectiveness of the stiffener may be determined from either:

- the general procedure given in 4.3.3.2;
- the simplified procedure given in 4.3.3.3.

4.3.3.2 General procedure

(1) The cross-section of an intermediate stiffener should be taken as comprising the stiffener itself plus the adjacent effective portions of the adjacent plane elements $b_{p,1}$ and $b_{p,2}$ shown in figure 4.4.

(2) The procedure, which is illustrated in figure 4.5, should be carried out in steps as follows:

- **Step 1**: Obtain an initial effective cross-section for the stiffener using effective widths determined by assuming that the stiffener gives full restraint and that $\sigma_{com,Ed} = f_{yb}/\gamma_{M1}$, see (3) and (4);
- **Step 2**: Use the initial effective cross-section of the stiffener to determine the reduction factor for flexural buckling, allowing for the effects of the continuous spring restraint, see (5) and (6);
- **Step 3**: Iterate to refine the value of the reduction factor for buckling of the stiffener, see (7) and (8).

(3) Initial values of the effective widths $b_{1,e2}$ and $b_{2,e1}$ shown in figure 4.4 should be determined from 4.2 by assuming that the plane elements $b_{p,1}$ and $b_{p,2}$ are doubly supported, see table 4.1.

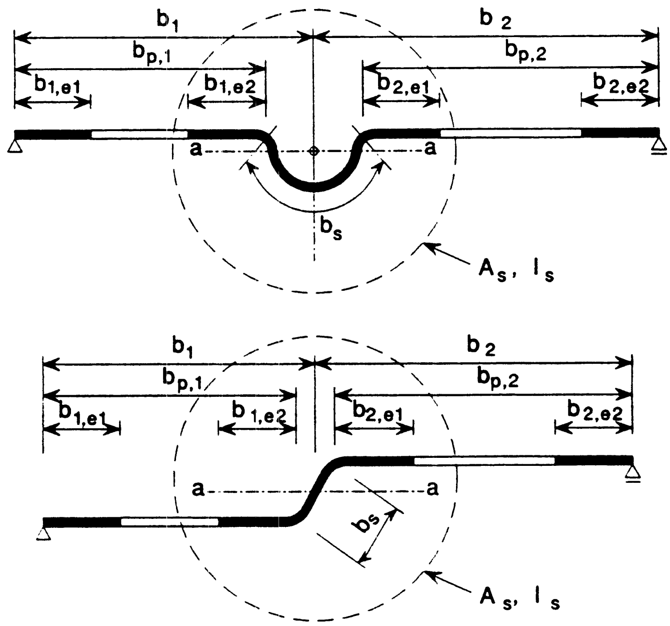


Figure 4.4: Intermediate stiffeners

(4) The effective cross-sectional area of an intermediate stiffener A_s should be obtained from:

$$A_s = t(b_{1,e2} + b_{2,e1} + b_s) \qquad \dots (4.20)$$

in which the stiffener width b_s is as shown in figure 4.4.

(5) The critical buckling stress $\sigma_{cr,s}$ for an intermediate stiffener should be obtained from:

$$\sigma_{cr,s} = \frac{2\sqrt{KEI_s}}{A_s} \qquad \dots (4.21)$$

where:

K is the spring stiffness per unit length, see 4.3.1(2).

I_s is the effective second moment of area of the stiffener, taken as that of its effective area A_s about the centroidal axis a - a of its effective cross-section, see figure 4.4.

(6) The reduction factor χ for the flexural buckling resistance of an intermediate stiffener should be obtained from the value of $\sigma_{cr,s}$ using the method given in 4.3.1(7).

(7) If $\chi < 1$ it may optionally be refined iteratively, starting the iteration with modified values of ρ obtained using 4.2(5) with $\sigma_{com,Ed}$ equal to $\chi f_{yb}/\gamma_{M1}$, so that:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi} \qquad \dots (4.22)$$

(8) If iteration is carried out, it should be continued until the current value of χ is approximately equal to, but not more than, the previous value.

(9) The reduced effective area of the stiffener $A_{s,red}$ allowing for flexural buckling should be taken as:

$$A_{s,red} = \chi A_s \qquad \dots (4.23)$$

(10) In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = \chi t$ for all the elements included in A_s.

(11) The effective section properties at serviceability limit states should be based on the design thickness t.

4.3.3.3 Simplified procedure

(1) As an alternative to the general procedure given in 4.3.3.2, the following simplified procedure may be used to determine the reduced effective area $A_{s,red}$ of an intermediate stiffener as shown in figure 4.4.

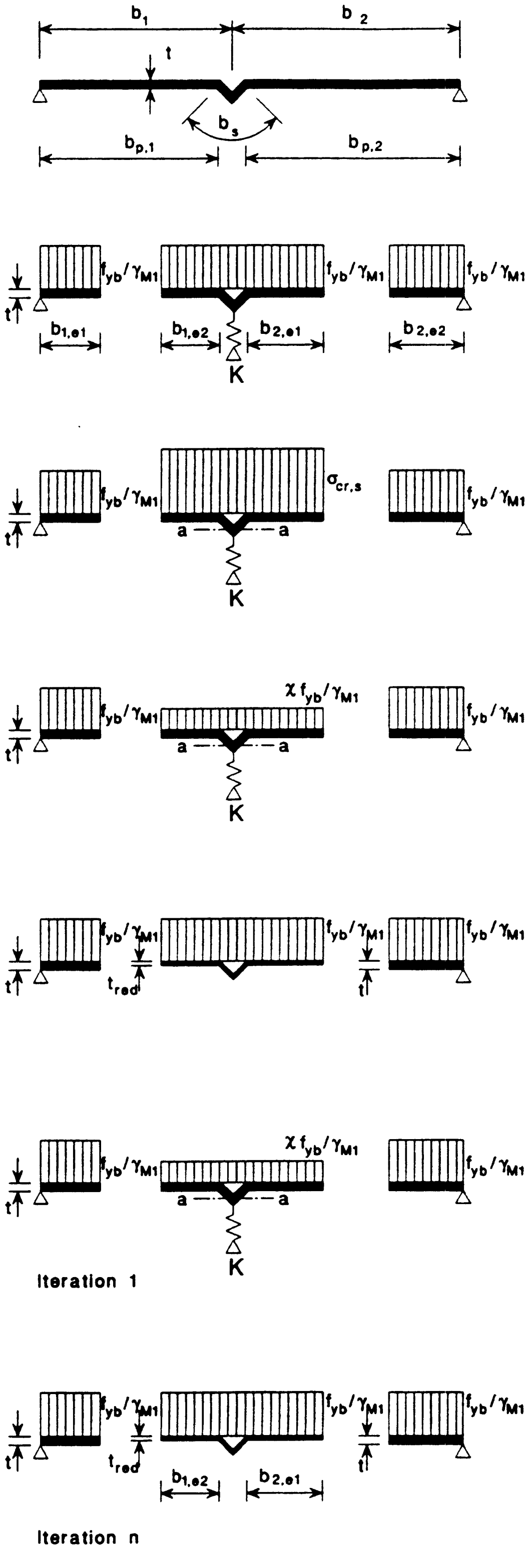
(2) The effective cross-sectional area of an intermediate stiffener A_s should be obtained from:

$$A_s = t(b_{1,e2} + b_{2,e1} + b_s) \qquad \dots (4.24)$$

in which the effective widths $b_{1,e2}$ and $b_{2,e1}$ and the stiffener width b_s are as shown in figure 4.4.

(3) The effective widths $b_{1,e2}$ and $b_{1,e1}$ should be determined from 4.2 with a buckling factor k_σ for a doubly supported element from table 4.1, using a value of ρ obtained from 4.2(5) with $\sigma_{com,Ed}$ equal to $\chi f_y/\gamma_{M1}$, so that:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi} \qquad \dots (4.25)$$



a) Gross cross-section and boundary conditions

b) **Step 1**: Effective cross-section for $K = \infty$ based on $\sigma_{com,Ed} = f_{yb}/\gamma_{M1}$

c) **Step 2**: Elastic critical stress $\sigma_{cr,s}$ for effective area of stiffener A_s from step 1

d) Reduced strength $\chi f_{yb}/\gamma_{M1}$ for effective area of stiffener A_s, with reduction factor χ based on $\sigma_{cr,s}$

e) Reduced thickness t_{red} for effective width of stiffener, with reduction factor χ based on $\sigma_{cr,s}$

f) **Step 3**: Repeat step 2 using reduced effective area of stiffener $A_{s,red}$ from previous iteration, continuing until $\chi_n \approx \chi_{(n-1)}$ but $\chi_n \leq \chi_{(n-1)}$

g) Adopt an effective cross-section with $b_{1,e2}$, $b_{2,e1}$ and reduced thickness t_{red} corresponding to χ_n

Figure 4.5: Compression resistance of a flange with an intermediate stiffener

(4) The reduction factor χ may be taken as equal to 0,5 if:

$$I_s \geq 0{,}016\,(f_{yb}/E)^2\,(b_o/t)^3 A_s^2 \quad \text{... (4.26)}$$

otherwise the reduction factor χ may be taken as approximately equal to 1,0 if:

$$I_s \geq 0{,}24\,(f_{yb}/E)^2\,(b_o/t)^3 A_s^2 \quad \text{... (4.27)}$$

where:

b_o = $b_1 + b_2$ (see figure 4.4);

I_s is the effective second moment of area of the stiffener, taken as that of its effective area A_s about the centroidal axis a - a of its effective cross-section, see figure 4.4.

(5) The reduced effective area of the stiffener $A_{s,red}$ allowing for flexural buckling should be taken as:

$$A_{s,red} = \chi A_s \quad \text{... (4.28)}$$

(6) In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = \chi t$ for all the elements included in A_s.

(7) The effective section properties at serviceability limit states should be based on the design thickness t for all values of I_s.

4.3.4 Trapezoidal sheeting profiles with intermediate stiffeners

4.3.4.1 General

(1) This sub-clause 4.3.4 should be used for trapezoidal profiled sheets, in association with 4.3.2 for flanges with intermediate flange stiffeners and 4.3.3 for webs with intermediate stiffeners.

(2) Interaction between the buckling of intermediate flange stiffeners and intermediate web stiffeners should also be taken into account using the method given in 4.3.4.4.

4.3.4.2 Flanges with intermediate stiffeners

(1) If it is subject to uniform compression, the effective cross-section of a flange with intermediate stiffeners should be assumed to consist of the reduced effective areas $A_{s,red}$ of up to two intermediate stiffeners and two strips of width $0{,}5b_{eff}$ adjacent to the edges supported by webs, see figure 4.6.

(2) For one central flange stiffener, the elastic critical buckling stress $\sigma_{cr,s}$ should be obtained from:

$$\sigma_{cr,s} = \frac{4{,}2\,k_w E}{A_s}\sqrt{\frac{I_s t^3}{4 b_p^2 (2 b_p + 3 b_s)}} \quad \text{... (4.29)}$$

where:

b_p is the notional flat width of plane element shown in figure 4.6;

b_s is the stiffener width, measured around the perimeter of the stiffener, see figure 4.6;

k_w is a coefficient that allows for partial rotational restraint of the stiffened flange by the webs or other adjacent elements, see (5) and (6);

and A_s and I_s are as defined in 4.3.3.2.

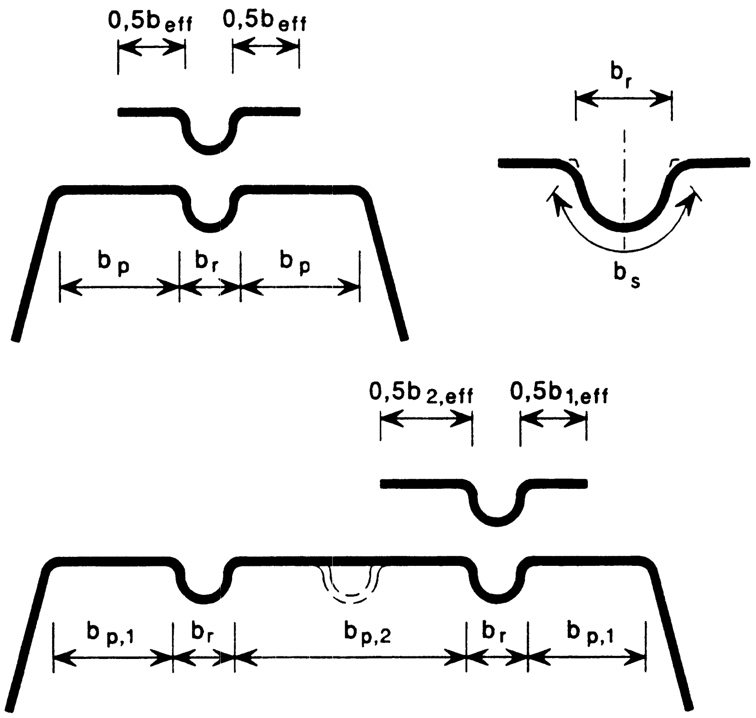


Figure 4.6: Compression flange with one, two or three stiffeners

(3) For two symmetrically placed flange stiffeners, the elastic critical buckling stress $\sigma_{cr,s}$ should be obtained from:

$$\sigma_{cr,s} = \frac{4{,}2\, k_w E}{A_s} \sqrt{\frac{I_s t^3}{8 b_1^2 (3 b_e - 4 b_1)}} \qquad \dots (4.30)$$

with:

$$b_e = 2b_{p,1} + b_{p,2} - 2b_s$$

$$b_1 = b_{p,1} + 0{,}5 b_r$$

where:

$b_{p,1}$ is the notional flat width of an outer plane element, as shown in figure 4.6;

$b_{p,2}$ is the notional flat width of the central plane element, as shown in figure 4.6;

b_r is the overall width of a stiffener, see figure 4.6.

(4) If there are three stiffeners, the one in the middle should be assumed to be ineffective.

(5) The value of k_w may be calculated from the compression flange buckling wavelength ℓ_b as follows:

- if $\ell_b / s_w \geq 2$:

$$k_w = k_{wo} \qquad \dots (4.31a)$$

- if $\ell_b / s_w < 2$:

$$k_w = k_{wo} - (k_{wo} - 1)[2\ell_b / s_w - (\ell_b / s_w)^2] \qquad \dots (4.31b)$$

where:

s_w is the slant height of the web, see figure 3.3(c).

(6) Alternatively, the rotational restraint coefficient k_w may conservatively be taken as equal to 1,0 corresponding to a pin-jointed condition.

(7) The values of ℓ_b and k_{wo} may be determined from the following:

- for a compression flange with one intermediate stiffener:

$$\ell_b = 3{,}07 \sqrt[4]{I_s b_p^2 (2b_p + 3b_s)/t^3} \quad \text{... (4.32)}$$

$$k_{wo} = \sqrt{\frac{s_w + 2b_d}{s_w + 0{,}5b_d}} \quad \text{... (4.33)}$$

with:

$$b_d = 2b_p + b_s$$

- for a compression flange with two or three intermediate stiffeners:

$$\ell_b = 3{,}65 \sqrt[4]{I_s b_1^2 (3b_e - 4b_1)/t^3} \quad \text{... (4.34)}$$

$$k_{wo} = \sqrt{\frac{(2b_e + s_w)(3b_e - 4b_1)}{b_1(4b_e - 6b_1) + s_w(3b_e - 4b_1)}} \quad \text{... (4.35)}$$

(8) The reduced effective area of the stiffener $A_{s,red}$ allowing for flexural buckling should be taken as:

$$A_{s,red} = \chi A_s \quad \text{... (4.36)}$$

(9) If the webs are unstiffened, the reduction factor χ should be obtained directly from $\sigma_{cr,s}$ using the method given in 4.3.1(7).

(10) If the webs are also stiffened, the reduction factor χ should be obtained using the method given in 4.3.1(7), but with the modified elastic critical stress $\sigma_{cr,mod}$ given in 4.3.4.4.

(11) In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = \chi t$ for all the elements included in A_s.

(12) The effective section properties at serviceability limit states should be based on the design thickness t.

4.3.4.3 Webs with up to two intermediate stiffeners

(1) The effective cross-section of the compression zone of a web (or other element of a cross-section that is subject to stress gradient) should be assumed to consist of the reduced effective areas $A_{s,red}$ of up to two intermediate stiffeners, a strip adjacent to the compression flange and a strip adjacent to the centroidal axis of the effective cross-section, see figure 4.7.

(2) The effective cross-section of a web as shown in figure 4.7 should be taken to include:

a) a strip of width $s_{eff,1}$ adjacent to the compression flange;

b) the reduced effective area $A_{s,red}$ of each web stiffener, up to a maximum of two;

c) a strip of width $s_{eff,n}$ adjacent to the effective centroidal axis;

d) the part of the web in tension.

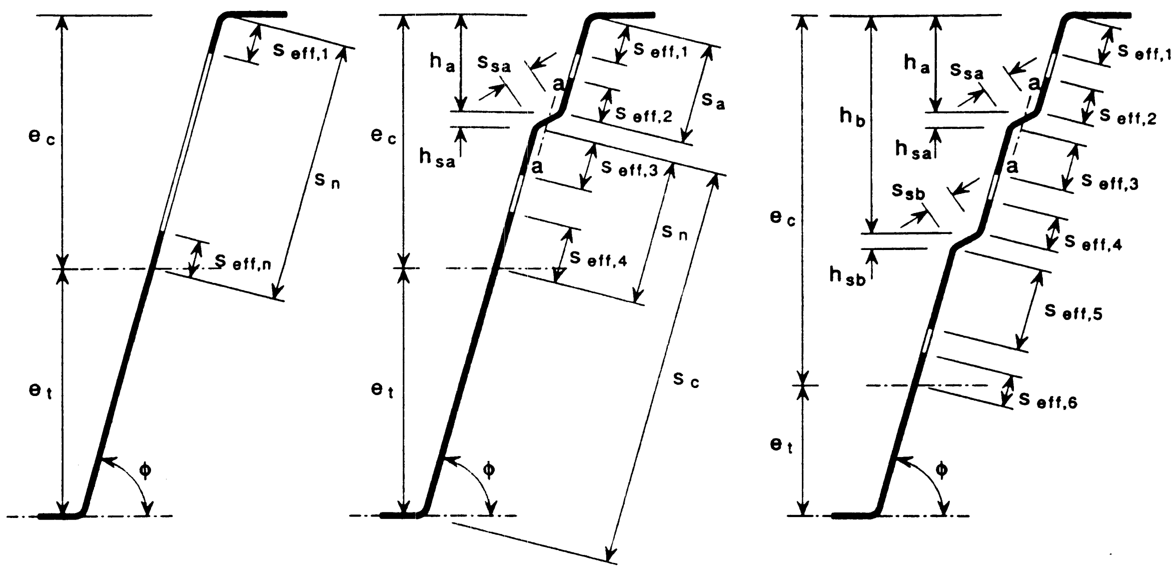


Figure 4.7: Effective cross-sections of webs of trapezoidal profiled sheets

(3) The effective areas of the stiffeners should be obtained from the following:

- for a single stiffener, or for the stiffener closer to the compression flange:

$$A_{sa} = t(s_{eff,2} + s_{eff,3} + s_{sa}) \quad \text{... (4.37)}$$

- for a second stiffener:

$$A_{sb} = t(s_{eff,4} + s_{eff,5} + s_{sb}) \quad \text{... (4.38)}$$

in which the dimensions $s_{eff,1}$ to $s_{eff,n}$ and s_{sa} and s_{sb} are as shown in figure 4.7.

(4) Initially the location of the effective centroidal axis should be based on the effective cross-sections of the flanges but the gross cross-sections of the webs. In this case the basic effective width $s_{eff,0}$ should be obtained from:

$$s_{eff,0} = 0{,}76t\sqrt{E/(\gamma_{M1}\sigma_{com,Ed})} \quad \text{... (4.39)}$$

where:

$\sigma_{com,Ed}$ is the stress in the compression flange when the cross-section resistance is reached.

(5) If the web is not fully effective, the dimensions $s_{eff,1}$ to $s_{eff,n}$ should be determined as follows:

$$s_{eff,1} = s_{eff,0} \quad \text{... (4.40a)}$$

$$s_{eff,2} = (1 + 0{,}5h_a/e_c)s_{eff,0} \quad \text{... (4.40b)}$$

$$s_{eff,3} = [1 + 0{,}5(h_a + h_{sa})/e_c]s_{eff,0} \quad \text{... (4.40c)}$$

$$s_{eff,4} = (1 + 0{,}5h_b/e_c)s_{eff,0} \quad \text{... (4.40d)}$$

$$s_{eff,5} = [1 + 0{,}5(h_b + h_{sb})/e_c]s_{eff,0} \quad \text{... (4.40e)}$$

$$s_{eff,n} = 1{,}5s_{eff,0} \quad \text{... (4.40f)}$$

where:

e_c is the distance from the effective centroidal axis to the system line of the compression flange, see figure 4.7;

and the dimensions h_a, h_b, h_{sa} and h_{sb} are as shown in figure 4.7.

(6) The dimensions $s_{eff,1}$ to $s_{eff,n}$ should initially be determined from (5) and then revised if the relevant plane element is fully effective, using the following:

- in an unstiffened web, if $s_{eff,1} + s_{eff,n} \geq s_n$ the entire web is effective, so revise as follows:

$$s_{eff,1} = 0{,}4 s_n \quad \text{... (4.41a)}$$

$$s_{eff,n} = 0{,}6 s_n \quad \text{... (4.41b)}$$

- in stiffened web, if $s_{eff,1} + s_{eff,2} \geq s_a$ the whole of s_a is effective, so revise as follows:

$$s_{eff,1} = s_a / (2 + 0{,}5 h_a / e_c) \quad \text{... (4.42a)}$$

$$s_{eff,2} = s_a (1 + 0{,}5 h_a / e_c) / (2 + 0{,}5 h_a / e_c) \quad \text{... (4.42b)}$$

- in a web with one stiffener, if $s_{eff,3} + s_{eff,n} \geq s_n$ the whole of s_n is effective, so revise as follows:

$$s_{eff,3} = s_n [1 + 0{,}5(h_a + h_{sa}) / e_c] / [2{,}5 + 0{,}5(h_a + h_{sa}) / e_c] \quad \text{... (4.43a)}$$

$$s_{eff,n} = 1{,}5 s_n / [2{,}5 + 0{,}5(h_a + h_{sa}) / e_c] \quad \text{... (4.43b)}$$

- in a web with two stiffeners:

  - if $s_{eff,3} + s_{eff,4} \geq s_b$ the whole of s_b is effective, so revise as follows:

$$s_{eff,3} = s_b [1 + 0{,}5(h_a + h_{sa}) / e_c] / [2 + 0{,}5(h_a + h_{sa} + h_b) / e_c] \quad \text{... (4.44a)}$$

$$s_{eff,4} = s_b (1 + 0{,}5 h_b / e_c) / [2 + 0{,}5(h_a + h_{sa} + h_b) / e_c] \quad \text{... (4.44b)}$$

  - if $s_{eff,5} + s_{eff,n} \geq s_n$ the whole of s_n is effective, so revise as follows:

$$s_{eff,5} = s_n [1 + 0{,}5(h_b + h_{sb}) / e_c] / [2{,}5 + 0{,}5(h_b + h_{sb}) / e_c] \quad \text{... (4.45a)}$$

$$s_{eff,n} = 1{,}5 s_n / [2{,}5 + 0{,}5(h_b + h_{sb}) / e_c] \quad \text{... (4.45b)}$$

(7) For a single stiffener, or for the stiffener closer to the compression flange in webs with two stiffeners, the elastic critical buckling stress $\sigma_{cr,sa}$ should be determined using:

$$\sigma_{cr,sa} = \frac{1{,}05 k_f E \sqrt{I_s t^3 s_1}}{A_{sa} s_2 (s_1 - s_2)} \quad \text{... (4.46a)}$$

in which s_1 is given by the following:

- for a single stiffener:

$$s_1 = 0{,}9 (s_a + s_{sa} + s_c) \quad \text{... (4.46b)}$$

- for the stiffener closer to the compression flange, in webs with two stiffeners:

$$s_1 = s_a + s_{sa} + s_b + 0{,}5(s_{sb} + s_c) \quad \text{... (4.46c)}$$

with:

$$s_2 = s_1 - s_a - 0{,}5 s_{sa} \quad \text{... (4.46d)}$$

where:

k_f is a coefficient that allows for partial rotational restraint of the stiffened web by the flanges;

I_s is the second moment of area of a stiffener cross-section comprising the fold width s_{sa} and two adjacent strips, each of width $s_{eff,1}$, about its own centroidal axis parallel to the plane web elements, see figure 4.8. In calculating I_s the possible difference in slope between the plane web elements on either side of the stiffener may be neglected.

(8) In the absence of a more detailed investigation, the rotational restraint coefficient k_f may conservatively be taken as equal to 1,0 corresponding to a pin-jointed condition.

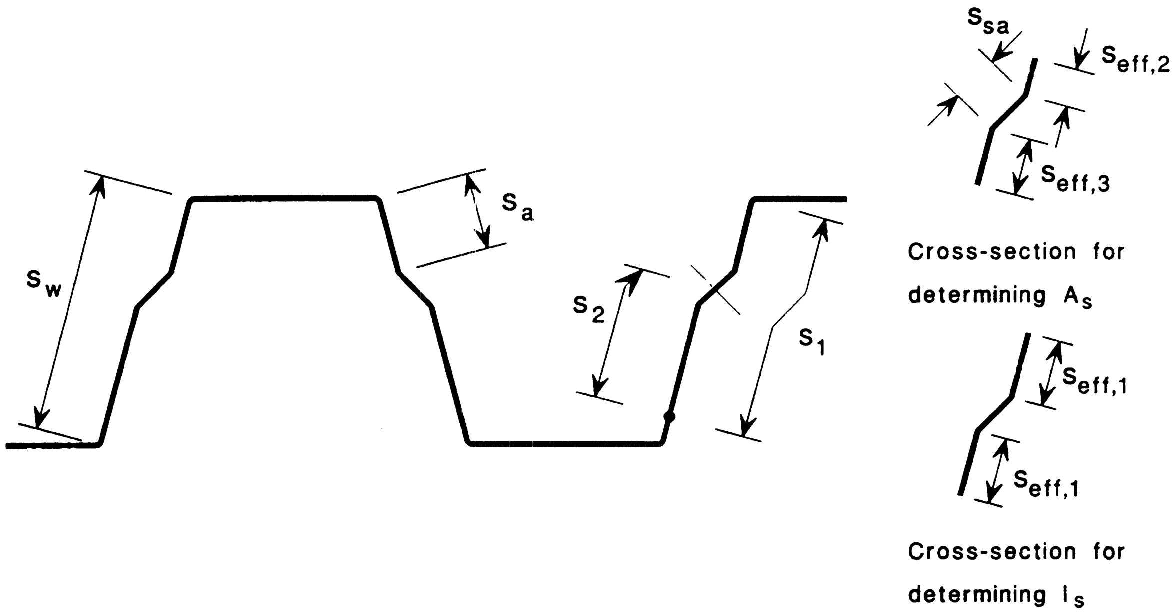


Figure 4.8: Web stiffeners for trapezoidal profiled sheeting

(9) For a single stiffener in compression, or for the stiffener closer to the compression flange in webs with two stiffeners, the reduced effective area $A_{sa,red}$ should be determined from:

$$A_{sa,red} = \chi A_{sa}/[1 - (h_a + 0,5h_{sa})/e_c] \quad \text{but} \quad A_{sa,red} \leq A_{sa} \qquad \ldots (4.47)$$

(10) If the flanges are unstiffened, the reduction factor χ should be obtained directly from $\sigma_{cr,sa}$ using the method given in 4.3.1(7).

(11) If the flanges are also stiffened, the reduction factor χ should be obtained using the method given in 4.3.1(7), but with the modified elastic critical stress $\sigma_{cr,mod}$ given in 4.3.4.4.

(12) For a single stiffener in tension, the reduced effective area $A_{sa,red}$ should be taken as equal to A_{sa}.

(13) For webs with two stiffeners, the reduced effective area $A_{sb,red}$ for the second stiffener, should be taken as equal to A_{sb}.

(14) In determining effective section properties, the reduced effective area $A_{sa,red}$ should be represented by using a reduced thickness $t_{red} = \chi t$ for all the elements included in A_{sa}.

(15) The effective section properties at serviceability limit states should be based on the design thickness t.

(16) Optionally, the effective section properties may be refined iteratively by basing the location of the effective centroidal axis on the effective cross-sections of the webs determined by the previous iteration and the effective cross-sections of the flanges determined using the reduced thickness t_{red} for all the elements included in the flange stiffener areas A_s. This iteration should be based on an increased basic effective width $s_{eff,0}$ obtained from:

$$s_{eff,0} = 0,95t\sqrt{E/(\gamma_{M1}\sigma_{com,Ed})} \qquad \ldots (4.48)$$

4.3.4.4 Sheeting with flange stiffeners and web stiffeners

(1) In the case of sheeting with intermediate stiffeners in the flanges and in the webs, see figure 4.9, interaction between the flexural buckling of the flange stiffeners and the web stiffeners should be allowed for by using a modified elastic critical stress $\sigma_{cr,mod}$ for both types of stiffeners, obtained from:

$$\sigma_{cr,mod} = \frac{\sigma_{cr,sa}}{\left[\left(\sigma_{cr,sa}/\sigma_{cr,s}\right)^4 + \left(1 - (h_a + 0{,}5h_{sa})/e_c\right)^4\right]^{0,25}} \quad \text{... (4.49)}$$

where:

$\sigma_{cr,s}$ is the elastic critical stress for an intermediate flange stiffener, see 4.3.4.2(2) for a flange with a single stiffener or 4.3.4.2(3) for a flange with two stiffeners;

$\sigma_{cr,sa}$ is the elastic critical stress for a single web stiffener, or the stiffener closer to the compression flange in webs with two stiffeners, see 4.3.4.3(7).

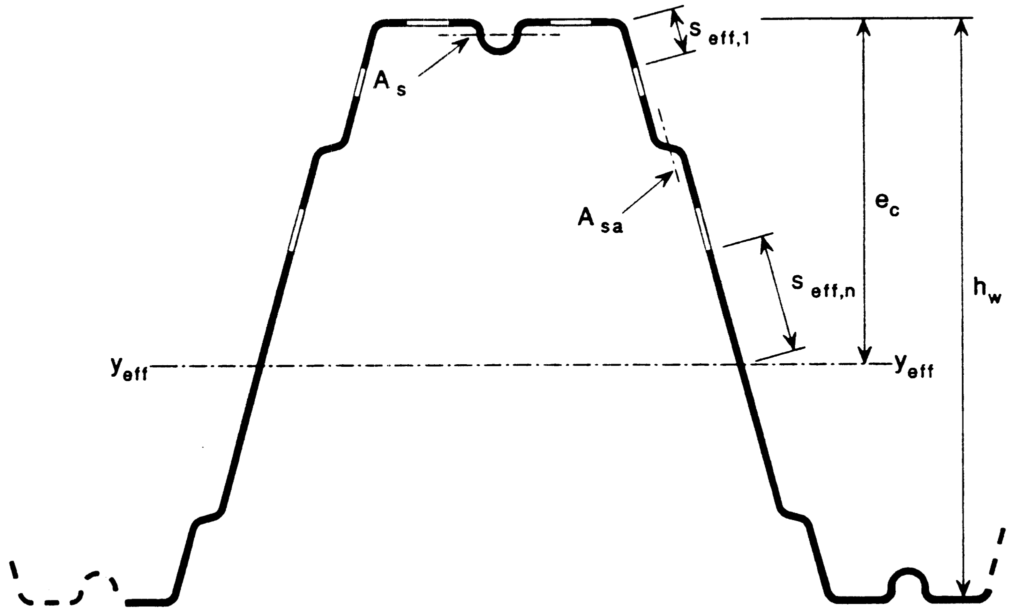


Figure 4.9: Trapezoidal profiled sheeting with flange stiffeners and web stiffeners

5 Resistance of cross-sections

5.1 General

(1)P The design values of the internal forces and moments at each cross-section shall not exceed the design values of the corresponding resistances.

(2)P The design resistance of a cross-section shall be determined either by calculation, using the methods given in this Section 5, or by design assisted by testing, in accordance with Section 9.

(3)P For design by calculation, the resistance of the cross-section shall be determined for:

- axial tension, as given in 5.2;
- axial compression, as given in 5.3;
- bending moment, as given in 5.4;
- combined bending and axial tension, as given in 5.5;
- combined bending and axial compression, as given in 5.6;
- torsional moment, as given in 5.7;
- shear force, as given in 5.8;
- local transverse forces, as given in 5.9;
- combined bending moment and shear force, as given in 5.10;
- combined bending moment and local transverse force, as given in 5.11.

(4) Design assisted by testing may be used instead of design by calculation for any of these resistances.

NOTE: Design assisted by testing is particularly likely to be beneficial for cross-sections with relatively high b_p/t ratios, e.g. in relation to inelastic behaviour, web crippling or shear lag.

(5)P For design by calculation, the effects of local buckling shall be taken into account by using effective section properties determined as specified in Section 4.

(6)P The buckling resistance of members shall be verified as specified in Section 6.

(7)P The effects of frame instability shall be taken into account as specified in ENV 1993-1-1.

(8)P In members with cross-sections that are susceptible to cross-sectional distortion, account shall be taken of possible lateral buckling of compression flanges and lateral bending of flanges generally, see 6.4.

5.2 Axial tension

(1)P The design tension resistance of a cross-section $N_{t,Rd}$ shall be determined from:

$$N_{t,Rd} = f_{ya} A_g / \gamma_{M0} \quad \text{but} \quad N_{t,Rd} \leq F_{n,Rd} \qquad \dots (5.1)$$

where:

A_g is the gross area of the cross-section;

$F_{n,Rd}$ is the net-section resistance from 8.4 for the appropriate type of mechanical fastener;

f_{ya} is the average yield strength, see 3.1.2.

(2) The tension resistance of an angle connected through one leg, or other types of section connected through outstands, should be determined as specified in ENV 1993-1-1.

5.3 Axial compression

(1)P The design compression resistance of a cross-section $N_{c,Rd}$ shall be determined from the following:

- if its effective area A_{eff} is less than its gross area A_g:

$$N_{c,Rd} = f_{yb} A_{eff} / \gamma_{M1} \quad \text{... (5.2a)}$$

- if its effective area A_{eff} is equal to its gross area A_g:

$$N_{c,Rd} = f_{ya} A_g / \gamma_{M0} \quad \text{... (5.2b)}$$

where:

A_{eff} is the effective area of the cross-section, obtained from Section 4 by assuming a uniform compressive stress $\sigma_{com,Ed}$ equal to f_{yb}/γ_{M1};

f_{ya} is the average yield strength, see 3.1.2.

f_{yb} is the basic yield strength.

(2)P The internal axial force in a member shall be taken as acting at the centroid of its gross cross-section.

(3)P The resistance of a cross-section to axial compression shall be assumed to act at the centroid of its effective cross-section. If this does not coincide with the centroid of its gross cross-section, the shift e_N of the centroidal axes (see figure 5.1) shall be taken into account, using the method given in 5.6.

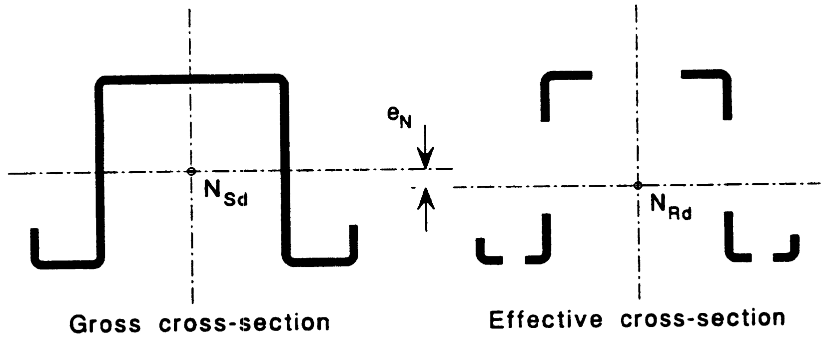


Figure 5.1: Effective cross-section under compression

5.4 Bending moment

5.4.1 General

(1)P The moment resistance of a cross-section for bending about a principal axis shall be obtained from the following:

- if the effective section modulus W_{eff} is less than the gross elastic section modulus $W_{e\ell}$:

$$M_{c,Rd} = f_y W_{eff} / \gamma_{M1} \quad \text{... (5.3a)}$$

- if the effective section modulus W_{eff} is equal to the gross elastic section modulus $W_{e\ell}$:

$$N_{c,Rd} = f_{ya} W_{e\ell} / \gamma_{M0} \quad \text{... (5.3b)}$$

where:

f_y is the yield strength as defined in 3.1.1(6)P.

(2)P The effective section modulus W_{eff} shall be based on an effective cross-section that is subject only to bending moment about the relevant principal axis, with a maximum stress $\sigma_{max,Ed}$ equal to f_{yb} / γ_{M1}, allowing for the effects of local buckling as specified in Section 4. Where shear lag is relevant, allowance shall also be made for its effects as specified in 5.4.3.

(3) The stress ratio $\psi = \sigma_2 / \sigma_1$ used to determine the effective portions of the web may be obtained by using the effective area of the compression flange but the gross area of the web, see figure 5.2.

(4)P If yielding occurs first at the compression edge of the cross-section, unless the conditions given in 5.4.2(5)P are met the value of W_{eff} shall be based on a linear distribution of stress across the cross-section.

(5)P For biaxial bending the following criterion shall be satisfied:

$$M_{y,Sd} / M_{cy,Rd} + M_{z,Sd} / M_{cz,Rd} \leq 1 \qquad \text{... (5.4)}$$

where:

$M_{y,Sd}$ is the applied bending moment about the major axis;

$M_{z,Sd}$ is the applied bending moment about the minor axis;

$M_{cy,Rd}$ is the resistance of the cross-section if subject only to moment about the y - y axis;

$M_{cz,Rd}$ is the resistance of the cross-section if subject only to moment about the z - z axis.

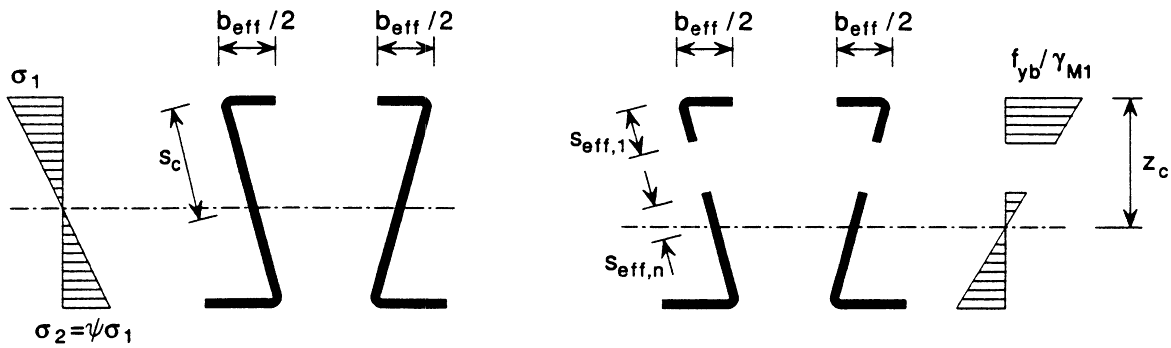


Figure 5.2: Effective cross-section for resistance to bending moments

5.4.2 Partially plastic resistance

(1)P Provided that bending moment is applied only about one principal axis of the cross-section, and provided that yielding occurs first at the tension edge, plastic reserves in the tension zone may be utilized without any strain limit until the maximum compressive stress $\sigma_{com,Ed}$ reaches f_{yb} / γ_{M1}.

(2) In this case, the effective partially plastic section modulus $W_{pp,eff}$ should be based on a stress distribution that is bilinear in the tension zone but linear in the compression zone.

(3) In the absence of a more detailed analysis, the effective width b_{eff} of an element subject to stress gradient may be obtained using 4.2 by basing b_c on the bilinear stress distribution, but ignoring the shape of the stress distribution in determining ψ.

(4) If redistribution of bending moments is assumed in the global analysis, it should be demonstrated from the results of tests in accordance with Section 9 that the provisions given in 7.2 are satisfied.

(5)P Plastic reserves may also be utilized in the compression zone, up to the strain specified in (6)P, provided that all the following conditions are satisfied:

a) Bending moment is applied only about one principal axes of the cross-section;

b) The member is not subject to torsion, or to torsional, torsional-flexural or lateral-torsional buckling;

c) Distortion of compressed parts of the cross-section is prevented;

d) The angle between any web and the vertical does not exceed 30 °;

e) The slant height s_c of the compressed portion of the web satisfies:

$$s_c / t \quad \leq \quad 1{,}11\sqrt{E/f_{yb}} \qquad [\cong \quad 33{,}18\varepsilon] \qquad \dots (5.5)$$

(6)P The compressive strain $\varepsilon_{com,Ed}$ shall not exceed $C_y \, \varepsilon_y / \gamma_{M1}$, where $\varepsilon_y = f_{yb}/E$, see figure 5.3, and the factor C_y is obtained from the following:

- for doubly supported compression elements without intermediate stiffeners:

  - if $s_c/t \leq 1{,}11\sqrt{E/f_{yb}}$ $\quad [\cong \quad 33{,}18\varepsilon]$:

    $C_y = 3$

  - if $s_c/t \leq 1{,}29\sqrt{E/f_{yb}}$ $\quad [\cong \quad 38{,}56\varepsilon]$:

    $C_y = 1$

  - if $1{,}11\sqrt{E/f_{yb}} < s_c/t < 1{,}29\sqrt{E/f_{yb}}$ $\quad$ [i.e. if $33{,}18\varepsilon < s_c/t < 38{,}56\varepsilon$]:

$$C_y = 3 - \frac{\left((s_c/t)\sqrt{f_{yb}/E} - 1{,}11\right)}{0{,}09} \qquad \left[\cong \; 3 - \frac{\left(s_c/t - 33{,}18\varepsilon\right)}{2{,}69\varepsilon}\right]$$

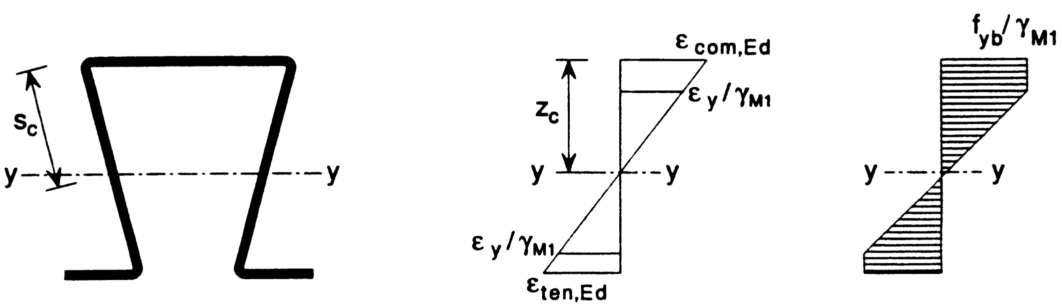
- for outstand elements:

  $C_y = 1$

- for elements with edge or intermediate stiffeners:

  $C_y = 1$

(7) In this case, the partially plastic section modulus W_{pp} should be based on a stress distribution that is bilinear in both the tension zone and the compression zone, as indicated in figure 5.3.



Effective cross-section for bending about the y-y axis

Design strain distribution $\varepsilon_{com,Ed} \leq C_y \varepsilon_y / \gamma_{M1}$

Design stress distribution

Figure 5.3: Partially plastic moment resistance

5.4.3 Effects of shear lag

(1)P The effects of shear lag shall be taken into account in flanges of flexural members if the length L_m between points of zero moment is less than $20 b_o$, where b_o is the width of flange contributing to shear lag, as shown in figure 5.4.

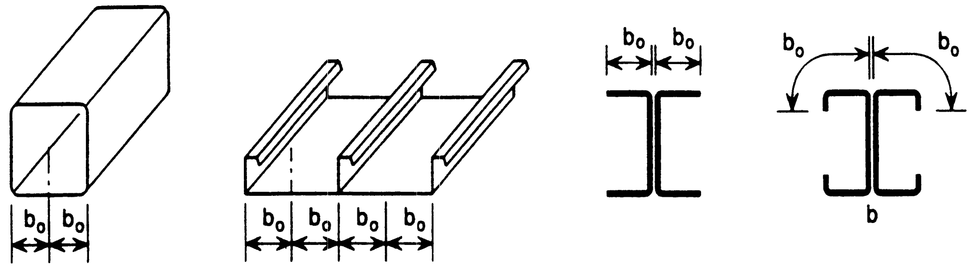


Figure 5.4: Width b_o contributing to shear lag

(2) In the absence of better information the following method may be adopted:

- for tension elements, replace b_o by b_{eff} given by:

$$b_{eff} = \beta_i b_o \quad \text{... (5.6)}$$

- for compression elements, replace the reduction factor ρ for local buckling (see 4.2(4)) by:

$$\rho_L = \beta_i^{\eta} \rho \quad \text{... (5.7a)}$$

in which:

- for stiffened flanges:

$$\eta = b_o / L_m \quad \text{... (5.7b)}$$

- for unstiffened flanges:

$$\eta = (b_o / L_m) / \delta \quad \text{... (5.7c)}$$

with:

$$\delta = \frac{2 b_o}{t} \sqrt{\frac{f_y}{E}} \cong \frac{b_o / t}{14,95 \varepsilon} \quad \text{but} \quad \delta \geq 1,0 \quad \text{... (5.8)}$$

where:

β_i is the appropriate value of the reduction factor for shear lag given in table 5.1.

NOTE: Further information is given in ENV 1993-2.*)

(3) For flanges with intermediate stiffeners b_o should be taken as half of the developed width b_d of the stiffened element, see figure 5.5.

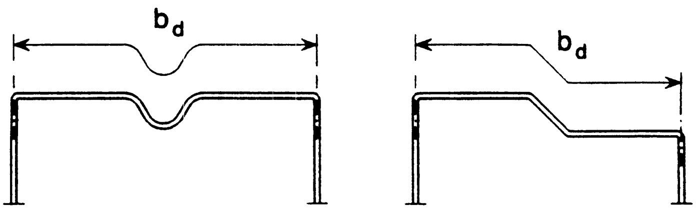
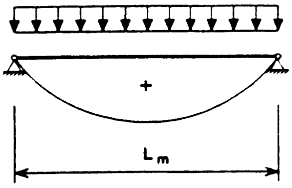
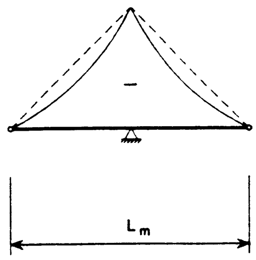
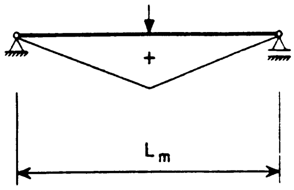
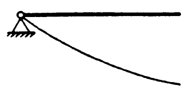
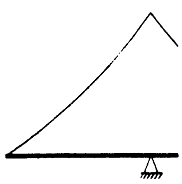


Figure 5.5: Developed width b_d of flanges with intermediate stiffeners

*) This document is in preparation.

Table 5.1: Reduction factors β_i for shear lag

Case and moment diagram	Reduction factor β_i
Span moment in simple or continuous beam - with uniformly distributed load 	for $b_o/L_m \geq 1/20$: $\beta_1 = \dfrac{1}{1 + 6{,}4(b_o/L_m)^2}$ for $b_o/L_m < 1/20$: $\beta_1 = 1{,}0$
Internal support of continuous beam or cantilever 	for $b_o/L_m \geq 1/20$: $\beta_2 = \dfrac{1}{1 + 6{,}0(b_o/L_m) + 1{,}6(b_o/L_m)^2}$ for $b_o/L_m < 1/50$: $\beta_2 = 1{,}0$ for $1/50 \leq b_o/L_m \leq 1/20$: $\beta_2 = 1{,}155 - 7{,}76(b_o/L_m)$
Span moment in simple or continuous beam - with central point load 	for $b_o/L_m \geq 1/20$: $\beta_3 = \dfrac{1}{1 + 4{,}0(b_o/L_m) + 3{,}2(b_o/L_m)^2}$ for $b_o/L_m < 1/50$: $\beta_3 = 1{,}0$ for $1/50 \leq b_o/L_m \leq 1/20$: $\beta_3 = 1{,}115 - 5{,}74(b_o/L_m)$
End support of beam 	$\beta_0 = (0{,}55 + 0{,}025 L_m/b_o)\beta_1$ but $\beta_0 \leq \beta_1$
Cantilever 	$\beta_0 = 1{,}0$

(4) As a simplification, in continuous beams the lengths L_m between points of zero moment may be replaced by the effective lengths L_e shown in figure 5.6, provided that no span is longer than 1,5 times an adjacent span, and no cantilever is longer than half the adjacent span.

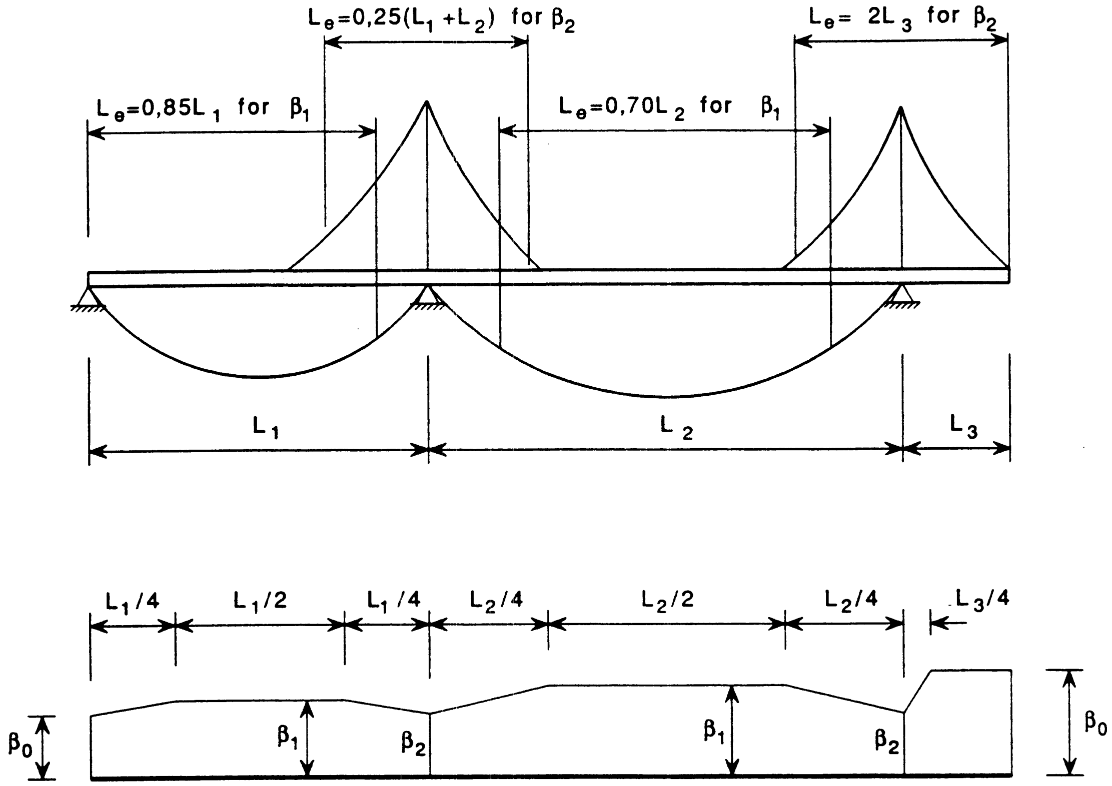


Figure 5.6: Simplified assumptions for continuous beams

5.5 Combined tension and bending

(1)P Cross-sections subject to combined axial tension N_{Sd} and bending moments $M_{y,Sd}$ and $M_{z,Sd}$ shall satisfy the criterion:

$$\frac{N_{Sd}}{f_y A_g / \gamma_M} + \frac{M_{y,Sd}}{f_y W_{eff,y,ten} / \gamma_M} + \frac{M_{z,Sd}}{f_y W_{eff,z,ten} / \gamma_M} \leq 1 \qquad \text{... (5.9a)}$$

where:

$W_{eff,y,ten}$ is the effective section modulus for maximum tensile stress if subject only to moment about the y - y axis;

$W_{eff,z,ten}$ is the effective section modulus for maximum tensile stress if subject only to moment about the z - z axis;

and $\gamma_M = \gamma_{M0}$ if $W_{eff} = W_{el}$ for each axis about which a bending moment acts, otherwise $\gamma_M = \gamma_{M1}$.

(2)P If $W_{eff,y,ten} \geq W_{eff,y,com}$ or $W_{eff,z,ten} \geq W_{eff,z,com}$ (where $W_{eff,y,com}$ and $W_{eff,z,com}$ are the effective section moduli for the maximum compressive stress in a effective cross-section that is subject only to moment about the relevant axis), the following criterion shall also be satisfied:

$$\frac{M_{y,Sd}}{f_y W_{eff,y,com} / \gamma_M} + \frac{M_{z,Sd}}{f_y W_{eff,z,com} / \gamma_M} - \frac{\psi_{vec} N_{Sd}}{f_y A_g / \gamma_M} \leq 1 \qquad \text{... (5.9b)}$$

in which ψ_{vec} is the factor for vectorial effects defined in ENV 1993-1-1.

5.6 Combined compression and bending

(1)P Cross-sections subject to combined axial compression N_{Sd} and bending moments $M_{y,Sd}$ and $M_{z,Sd}$ shall satisfy the criterion:

$$\frac{N_{Sd}}{f_y A_{eff} / \gamma_M} + \frac{M_{y,Sd} + \Delta M_{y,Sd}}{f_y W_{eff,y,com} / \gamma_M} + \frac{M_{z,Sd} + \Delta M_{z,Sd}}{f_y W_{eff,z,com} / \gamma_M} \leq 1 \qquad \text{... (5.10a)}$$

in which A_{eff} is as defined in 5.3, $W_{eff,y,com}$ and $W_{eff,z,com}$ are as defined in 5.5 and $\gamma_M = \gamma_{M0}$ if $A_{eff} = A_g$, otherwise $\gamma_M = \gamma_{M1}$.

(2)P The additional moments $\Delta M_{y,Sd}$ and $\Delta M_{z,Sd}$ due to shifts of the centroidal axes shall be taken as:

$$\Delta M_{y,Sd} = N_{Sd}\, e_{Ny}$$

$$\Delta M_{z,Sd} = N_{Sd}\, e_{Nz}$$

in which e_{Ny} and e_{Nz} are the shifts of the centroidal axes in the y and z directions, see 5.3(3)P.

(3)P If $W_{eff,y,com} \geq W_{eff,y,ten}$ or $W_{eff,z,com} \geq W_{eff,z,ten}$ the following criterion shall also be satisfied:

$$\frac{M_{y,Sd} + \Delta M_{y,Sd}}{f_y W_{eff,y,ten} / \gamma_M} + \frac{M_{z,Sd} + \Delta M_{z,Sd}}{f_y W_{eff,z,ten} / \gamma_M} - \frac{\psi_{vec} N_{Sd}}{f_y A_g / \gamma_M} \leq 1 \qquad \text{... (5.10b)}$$

in which $W_{eff,y,ten}$, $W_{eff,z,ten}$ and ψ_{vec} are as defined in 5.5.

5.7 Torsional moment

(1)P Where loads are applied eccentric to the shear centre of the cross-section, the effects of torsion shall be taken into account.

NOTE: As far as practicable, torsional moments are best avoided or reduced by restraints, because they substantially reduce the load bearing capacity, especially with open sections.

(2) The centroidal axis and shear centre to be used in determining the effects of the torsional moment, should be taken as those of the effective cross-section for the bending moment due to the relevant load.

(3) The direct stresses due to the axial force N_{Sd} and the bending moments $M_{y,Sd}$ and $M_{z,Sd}$ should be based on the respective effective cross-sections used in 5.2 to 5.4. The shear stresses due to transverse shear forces, the shear stress due to uniform (St. Venant) torsion and the direct stresses and shear stresses due to warping, should all be based on the properties of the gross cross-section.

(4) In cross-sections subject to torsion, the following conditions should be satisfied:

$$\sigma_{tot,Ed} \leq f_y / \gamma_M \qquad \text{... (5.11a)}$$

$$\tau_{tot,Ed} \leq \left(f_y / \sqrt{3}\right) / \gamma_{M0} \qquad \text{... (5.11b)}$$

$$\sqrt{\sigma_{tot,Ed}^2 + 3\tau_{tot,Ed}^2} \leq 1{,}1 f_y / \gamma_M \qquad \text{... (5.11c)}$$

where:

$\sigma_{tot,Ed}$ is the total direct stress, calculated on the relevant effective cross-section;

$\tau_{tot,Ed}$ is the total shear stress, calculated on the gross cross-section.

and $\gamma_M = \gamma_{M0}$ if $W_{eff} = W_{e\ell}$ for each axis about which a bending moment acts, otherwise $\gamma_M = \gamma_{M1}$.

(5) The total direct stress $\sigma_{tot,Ed}$ and the total shear stress $\tau_{tot,Ed}$ should by obtained from:

$$\sigma_{tot,Ed} = \sigma_{N,Ed} + \sigma_{My,Ed} + \sigma_{Mz,Ed} + \sigma_{w,Ed} \qquad \text{... (5.12a)}$$

$$\tau_{tot,Ed} = \tau_{Vy,Ed} + \tau_{Vz,Ed} + \tau_{t,Ed} + \tau_{w,Ed} \qquad \text{... (5.12b)}$$

where:

$\sigma_{My,Ed}$ is the direct stress due to the bending moment $M_{y,Sd}$;

$\sigma_{Mz,Ed}$ is the direct stress due to the bending moment $M_{z,Sd}$;

$\sigma_{N,Ed}$ is the direct stress due to the axial force N_{Sd};

$\sigma_{w,Ed}$ is the direct stress due to warping;

$\tau_{Vy,Ed}$ is the shear stress due to the transverse shear force $V_{y,Sd}$;

$\tau_{Vz,Ed}$ is the shear stress due to the transverse shear force $V_{z,Sd}$;

$\tau_{t,Ed}$ is the shear stress due to uniform (St. Venant) torsion;

$\tau_{w,Ed}$ is the shear stress due to warping.

5.8 Shear force

(1)P The shear resistance of the web $V_{w,Rd}$ shall be taken as the lesser of the shear buckling resistance $V_{b,Rd}$ and the plastic shear resistance $V_{p\ell,Rd}$.

(2) The plastic shear resistance $V_{p\ell,Rd}$ should be checked in the case of a web without longitudinal stiffeners if $s_w/t \leq 72\varepsilon(f_{yb}/f_y)(\gamma_{M0}/\gamma_{M1})$ or generally if $\bar{\lambda}_w \leq 0{,}83(f_{yb}/f_y)(\gamma_{M0}/\gamma_{M1})$.

(3)P The shear buckling resistance $V_{b,Rd}$ shall be determined from:

$$V_{b,Rd} = (h_w/\sin\phi)\,t\,f_{bv}/\gamma_{M1} \qquad \text{... (5.13)}$$

where:

f_{bv} is the shear buckling strength;

h_w is the web height between the midlines of the flanges, see figure 3.3(c);

ϕ is the slope of the web relative to the flanges.

(4)P The plastic shear resistance $V_{p\ell,Rd}$ shall be determined from:

$$V_{p\ell,Rd} = (h_w/\sin\phi)\,t\,(f_y/\sqrt{3})/\gamma_{M0} \qquad \text{... (5.14)}$$

(5)P The shear buckling strength f_{bv} for the appropriate value of the relative web slenderness $\bar{\lambda}_w$ shall be obtained from table 5.2.

Table 5.2: Shear buckling strength f_{bv}

Relative web slenderness	Web without stiffening at the support	Web with stiffening at the support [1]
$\bar{\lambda}_w < 1{,}40$	$0{,}48 f_{yb}/\bar{\lambda}_w$	$0{,}48 f_{yb}/\bar{\lambda}_w$
$\bar{\lambda}_w \geq 1{,}40$	$0{,}67 f_{yb}/\bar{\lambda}_w^2$	$0{,}48 f_{yb}/\bar{\lambda}_w$
[1] Stiffening at the support, such as cleats, arranged to prevent distortion of the web and designed to resist the support reaction.		

(6)P The relative web slenderness $\bar{\lambda}_w$ shall be obtained from the following:

$$\bar{\lambda}_w = \sqrt{\frac{f_{yb}/\sqrt{3}}{\tau_{cr}}} \equiv \frac{b_p}{t}\sqrt{\frac{12(1-\nu^2)f_{yb}}{\sqrt{3}\,\pi^2 E k_\tau}} \qquad \text{... (5.15a)}$$

- for webs without longitudinal stiffeners:

$$\bar{\lambda}_w = 0{,}346\,\frac{s_w}{t}\sqrt{\frac{f_{yb}}{E}} \cong \frac{s_w/t}{86{,}4\,\varepsilon} \qquad \text{... (5.15b)}$$

- for webs with longitudinal stiffeners, see figure 5.7:

$$\left.\begin{aligned} \bar{\lambda}_w &= 0{,}346\,\frac{s_d}{t}\sqrt{\frac{5{,}34}{k_\tau}\frac{f_{yb}}{E}} \cong \frac{s_d/t}{86{,}4\,\varepsilon}\sqrt{\frac{5{,}34}{k_\tau}} \\ \text{but } \bar{\lambda}_w &\geq 0{,}346\,\frac{s_p}{t}\sqrt{\frac{f_{yb}}{E}} \cong \frac{s_p/t}{86{,}4\,\varepsilon} \end{aligned}\right\} \qquad \text{... (5.15c)}$$

with:

$$k_\tau = 5{,}34 + \frac{2{,}10}{t}\sqrt[3]{\frac{I_s}{s_d}}$$

where:

I_s is the second moment of area of the longitudinal stiffener as defined in 4.3.4.3(7), about the axis a - a as indicated in figure 5.7;

s_d is the total developed slant height of the web, as indicated in figure 5.7;

s_p is the slant height of the largest plane element in the web, see figure 5.7;

s_w is the slant height of the web, as shown in figure 5.7, between the midpoints of the corners, see figure 3.3(c).

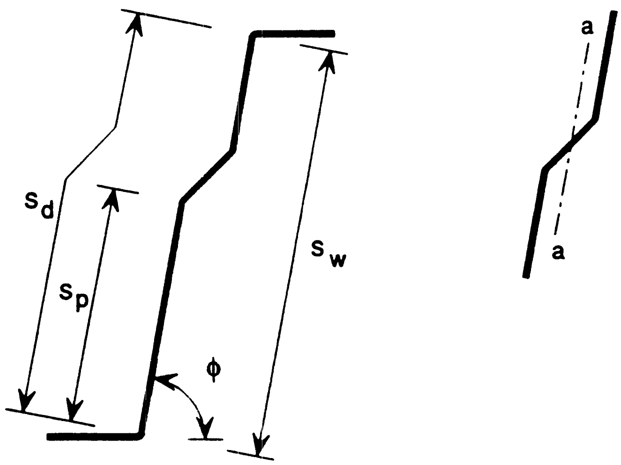


Figure 5.7: Longitudinally stiffened web

5.9 Local transverse forces

5.9.1 General

(1)P To avoid crushing, crippling or buckling in a web subject to a support reaction or other local transverse force applied through the flange, the transverse force F_{Sd} shall satisfy:

$$F_{Sd} \leq R_{w,Rd} \qquad \dots (5.16)$$

where:

$R_{w,Rd}$ is the local transverse resistance of the web.

(2)P The local transverse resistance of a web $R_{w,Rd}$ shall be obtained as follows:

a) for an unstiffened web:

- for a cross-section with a single web: from 5.9.2;
- for any other case, including sheeting: from 5.9.3;

b) for a stiffened web: from 5.9.4.

(3) Where the local load or support reaction is applied through a cleat that is arranged to prevent distortion of the web and is designed to resist the local transverse force, the local resistance of the web to the transverse force need not be considered.

(4) In beams with I-shaped cross-sections built up from two channels, or with similar cross-sections in which two components are interconnected through their webs, the connections between the webs should be located as close as practicable to the flanges of the beam.

5.9.2 Cross-sections with a single unstiffened web

(1) For a cross-section with a single unstiffened web, see figure 5.8, the local transverse resistance of the web may be determined as specified in (2), provided that the cross-section satisfies the following criteria:

$$h_w/t \leq 200 \qquad \dots (5.17a)$$

$$r/t \leq 6 \qquad \dots (5.17b)$$

$$45° \leq \phi \leq 90° \qquad \dots (5.17c)$$

where:

h_w is the web height between the midlines of the flanges;

r is the internal radius of the corners;

ϕ is the slope of the web relative to the flanges [degrees].

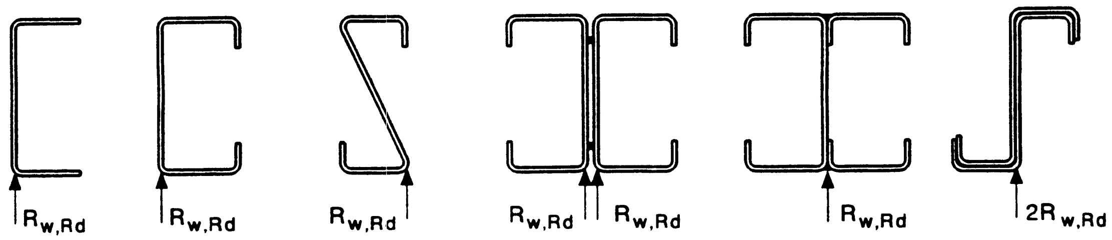


Figure 5.8: Examples of cross-sections with a single web

(2) For cross-sections that satisfy the criteria specified in (1), the local transverse resistance of a web $R_{w,Rd}$ may be determined as follows:

a) for a single local load or support reaction, see figure 5.9(a):

i) $c \leq 1{,}5\, h_w$ clear from a free end:

- for a cross-section with stiffened flanges:

$$R_{w,Rd} = k_1 k_2 k_3 [9{,}04 - (h_w/t)/60] [1 + 0{,}01 (s_s/t)] t^2 f_{yb}/\gamma_{M1} \quad \text{... (5.18a)}$$

- for a cross-section with unstiffened flanges:

  - if $s_s/t \leq 60$:

$$R_{w,Rd} = k_1 k_2 k_3 [5{,}92 - (h_w/t)/132] [1 + 0{,}01 (s_s/t)] t^2 f_{yb}/\gamma_{M1} \quad \text{... (5.18b)}$$

  - if $s_s/t > 60$:

$$R_{w,Rd} = k_1 k_2 k_3 [5{,}92 - (h_w/t)/132] [0{,}71 + 0{,}015 (s_s/t)] t^2 f_{yb}/\gamma_{M1} \quad \text{... (5.18c)}$$

ii) $c > 1{,}5\, h_w$ clear from a free end:

- if $s_s/t \leq 60$:

$$R_{w,Rd} = k_3 k_4 k_5 [14{,}7 - (h_w/t)/49{,}5] [1 + 0{,}007 (s_s/t)] t^2 f_{yb}/\gamma_{M1} \quad \text{... (5.18d)}$$

- if $s_s/t > 60$:

$$R_{w,Rd} = k_3 k_4 k_5 [14{,}7 - (h_w/t)/49{,}5] [0{,}75 + 0{,}011 (s_s/t)] t^2 f_{yb}/\gamma_{M1} \quad \text{... (5.18e)}$$

b) for two opposing local transverse forces closer together than $1{,}5\, h_w$, see figure 5.9(b):

i) $c \leq 1{,}5\, h_w$ clear from a free end:

$$R_{w,Rd} = k_1 k_2 k_3 [6{,}66 - (h_w/t)/64] [1 + 0{,}01 (s_s/t)] t^2 f_{yb}/\gamma_{M1} \quad \text{... (5.18f)}$$

ii) $c > 1{,}5\, h_w$ clear from a free end:

$$R_{w,Rd} = k_3 k_4 k_5 [21{,}0 - (h_w/t)/16{,}3] [1 + 0{,}0013 (s_s/t)] t^2 f_{yb}/\gamma_{M1} \quad \text{... (5.18g)}$$

(3) The values of the constants k_1 to k_5 should be determined as follows:

$k_1 = (1{,}33 - 0{,}33\, k)$

$k_2 = (1{,}15 - 0{,}15\, r/t)$ but $k_2 \geq 0{,}50$ and $k_2 \leq 1{,}0$

$k_3 = 0{,}7 + 0{,}3\, (\phi/90)^2$

$k_4 = (1{,}22 - 0{,}22\, k)$

$k_5 = (1{,}06 - 0{,}06\, r/t)$ but $k_5 \leq 1{,}0$

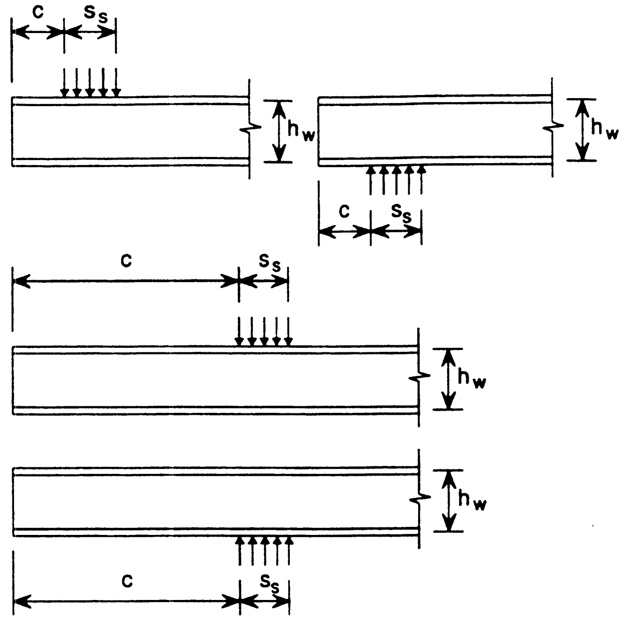
where:

$k = f_{yb}/228$ [with f_{yb} in N/mm^2];

s_s is the actual length of stiff bearing. In the case of two equal and opposite local transverse forces distributed over unequal bearing lengths, the smaller value of s_s should be used.

(i) $c \leq 1,5h_w$

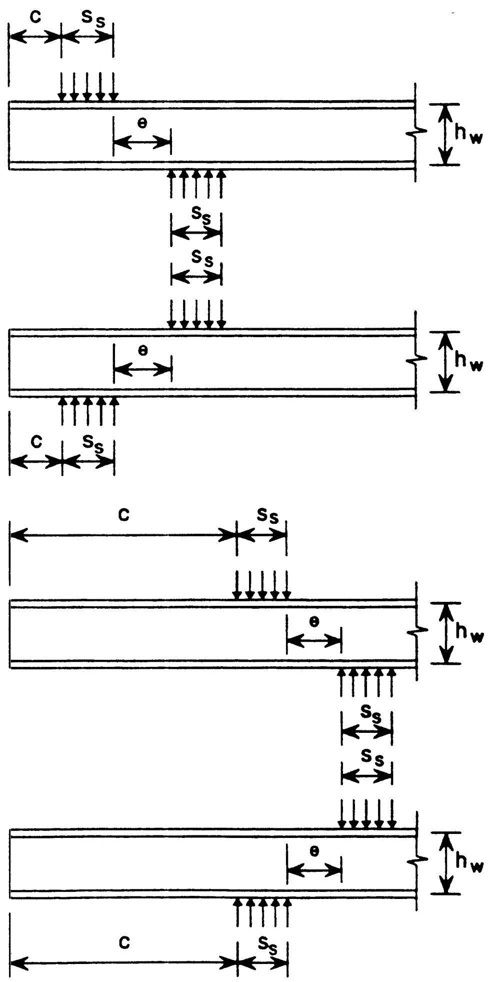
(ii) $c > 1,5h_w$



(a) Single load or support reaction

(i) $c \leq 1,5h_w$

(ii) $c > 1,5h_w$



(b) Opposing forces with $e < 1,5h_w$

Figure 5.9: Local loads and supports — cross-sections with a single web

5.9.3 Cross-sections with two or more unstiffened webs

(1) In cross-sections with two or more webs, including sheeting, see figure 5.10, the local transverse resistance of an unstiffened web should be determined as specified in (2), provided that both of the following conditions are satisfied:

- the clear distance c from the actual bearing length for the support reaction or local load to a free end, see figure 5.11, is at least 40 mm;
- the cross-section satisfies the following criteria:

$$r/t \leq 10 \quad \text{... (5.19a)}$$

$$h_w/t \leq 200 \sin\phi \quad \text{... (5.19b)}$$

$$45° \leq \phi \leq 90° \quad \text{... (5.19c)}$$

where:

h_w is the web height between the midlines of the flanges;

r is the internal radius of the corners;

ϕ is the slope of the web relative to the flanges [degrees].

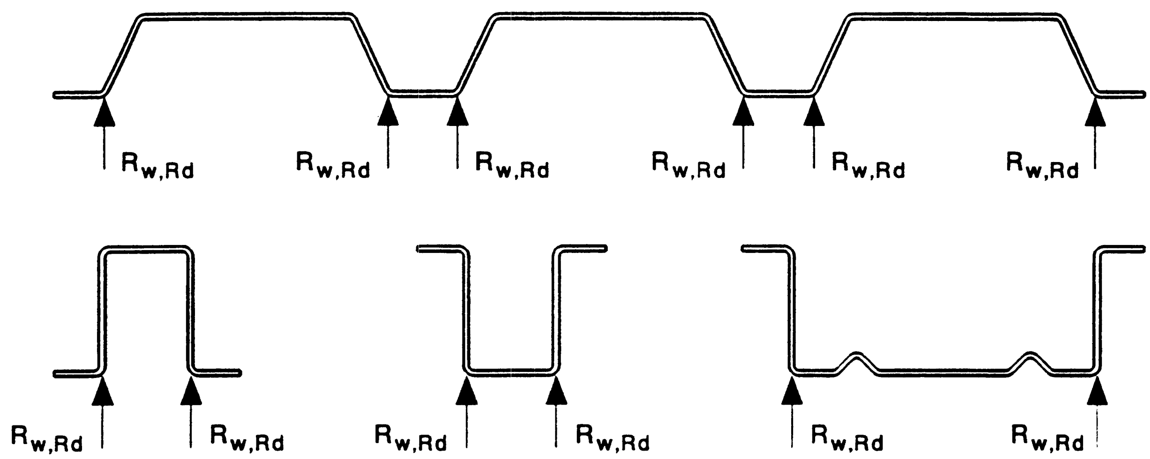


Figure 5.10: Examples of cross-sections with two or more webs

(2) Where both of the conditions specified in (1) are satisfied, the local transverse resistance $R_{w,Rd}$ per web of the cross-section should be determined from:

$$R_{w,Rd} = \alpha t^2 \sqrt{f_{yb} E}\left(1 - 0,1\sqrt{r/t}\right)\left[0,5 + \sqrt{0,02\,\ell_a/t}\right]\left(2,4 + (\phi/90)^2\right)/\gamma_{M1} \quad \text{... (5.20)}$$

where:

ℓ_a is the effective bearing length for the relevant category, see (3);

s_s is the actual length of stiff bearing;

α is the coefficient for the relevant category, see (3).

(3) The values of ℓ_a and α should be obtained from (4) and (5) respectively. The relevant category (1 or 2) should be based on the clear distance e between the local load and the nearest support, or the clear distance c from the support reaction or local load to a free end, see figure 5.11, as follows:

a) Category 1, see figure 5.11(a):

- local load applied with $e \leq 1{,}5\,h_w$ clear from the nearest support;
- local load applied with $c \leq 1{,}5\,h_w$ clear from a free end;
- reaction at end support with $c \leq 1{,}5\,h_w$ clear from a free end.

b) Category 2, see figure 5.11(b):

- local load applied with $e > 1{,}5\,h_w$ clear from the nearest support;
- local load applied with $c > 1{,}5\,h_w$ clear from a free end;
- reaction at end support with $c > 1{,}5\,h_w$ clear from a free end;
- reaction at internal support.

(4) The value of the effective bearing length ℓ_a should be obtained from the following:

a) for Category 1: $\ell_a = 10\,\text{mm}$ ... (5.21a)

b) for Category 2:

- $\beta_V \leq 0{,}2$: $\ell_a = s_s$ ... (5.21b)
- $\beta_V \geq 0{,}3$: $\ell_a = 10\,\text{mm}$ ... (5.21c)
- $0{,}2 < \beta_V < 0{,}3$: Interpolate linearly between the values of ℓ_a for 0,2 and 0,3.

with:

$$\beta_V = \frac{|V_{Sd,1}| - |V_{Sd,2}|}{|V_{Sd,1}| + |V_{Sd,2}|}$$

in which $|V_{Sd,1}|$ and $|V_{Sd,2}|$ are the absolute values of the transverse shear forces on each side of the local load or support reaction, and $|V_{Sd,1}| \geq |V_{Sd,2}|$.

(5) The value of the coefficent α should be obtained from the following:

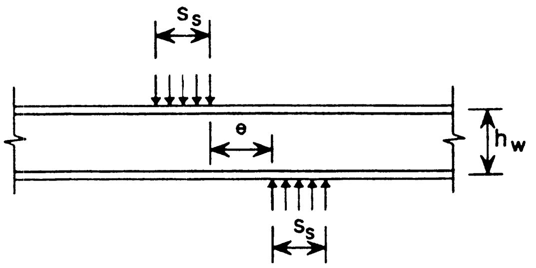
a) for Category 1:

- for sheeting profiles: $\alpha = 0{,}075$ ... (5.22a)
- for liner trays and hat sections: $\alpha = 0{,}057$ ... (5.22b)

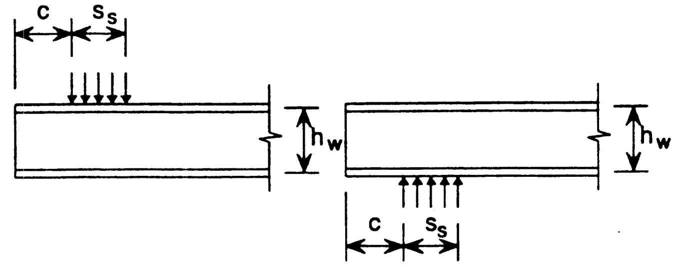
b) for Category 2:

- for sheeting profiles: $\alpha = 0{,}15$ ... (5.22c)
- for liner trays and hat sections: $\alpha = 0{,}115$ ... (5.22d)

(i) $e \leq 1,5h_w$

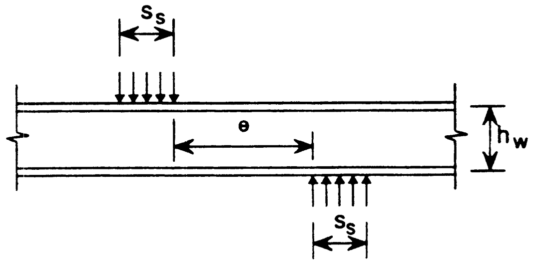


(ii) $c \leq 1,5h_w$

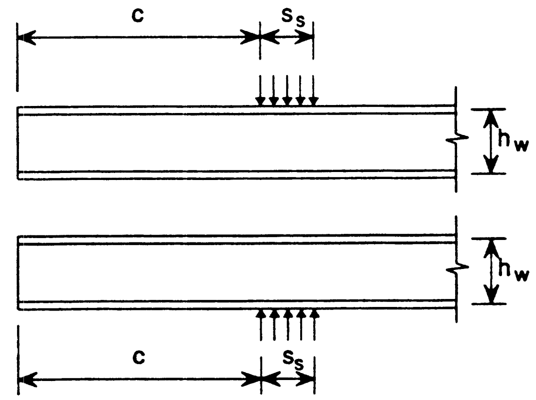


(a) Category 1

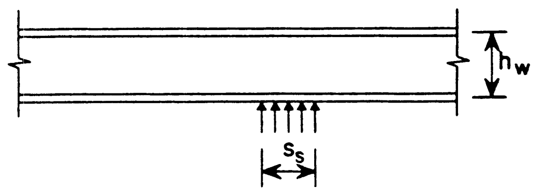
(i) $e > 1,5h_w$



(ii) $c > 1,5h_w$



(iii) Intermediate support



(b) Category 2

Figure 5.11: Local loads and supports — cross-sections with two or more webs

5.9.4 Stiffened webs

(1) The local transverse resistance of a stiffened web may be determined as specified in (2) for cross-sections with longitudinal web stiffeners folded in such a way that the two folds in the web are on opposite sides of the system line of the web joining the points of intersection of the midline of the web with the midlines of the flanges, see figure 5.12, that satisfy the condition:

$$2 < e_{max}/t < 12 \qquad \text{... (5.23)}$$

where:

e_{max} is the larger eccentricity of the folds relative to the system line of the web.

(2) For cross-sections with stiffened webs satisfying the conditions specified in (1), the local transverse resistance of a stiffened web may be determined by multiplying the corresponding value for a similar unstiffened web, obtained from 5.9.2 or 5.9.3 as appropriate, by the factor $\kappa_{a,s}$ given by:

$$\kappa_{a,s} = 1,45 - 0,05\, e_{max}/t \quad \text{but} \quad \kappa_{a,s} \leq 0,95 + 35\,000\, t^2 e_{min}/(b_1{}^2 h_p) \qquad \text{... (5.24)}$$

where:

b_d is the developed width of the loaded flange, see figure 5.12;

e_{min} is the smaller eccentricity of the folds relative to the system line of the web;

s_p is the slant height of the plane web element nearest to the loaded flange, see figure 5.12.

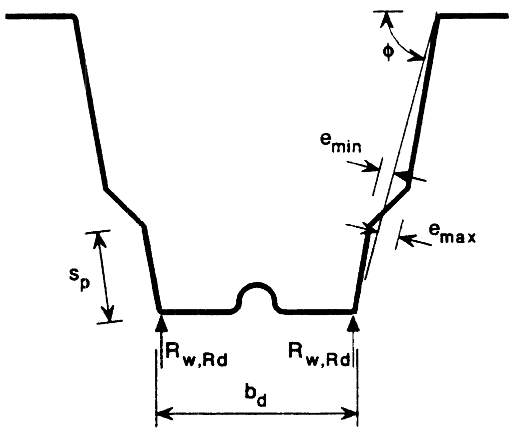


Figure 5.12: Stiffened webs

5.10 Combined shear force and bending moment

(1)P Cross-sections subject to the combined action of a bending moment M_{Sd} and a shear force V_{Sd} shall satisfy:

$$\left[\frac{M_{Sd}}{M_{c,Rd}}\right]^2 + \left[\frac{V_{Sd}}{V_{w,Rd}}\right]^2 \leq 1 \qquad \text{... (5.25)}$$

where:

$M_{c,Rd}$ is the moment resistance of the cross-section given in 5.4.1(1)P;

$V_{w,Rd}$ is the shear resistance of the web given in 5.8(1)P.

5.11 Combined bending moment and local load or support reaction

(1)P Cross-sections subject to the combined action of a bending moment M_{Sd} and a transverse force due to a local load or support reaction F_{Sd} shall satisfy the following:

$$M_{Sd}/M_{c,Rd} \leq 1 \quad \dots (5.26a)$$

$$F_{Sd}/R_{w,Rd} \leq 1 \quad \dots (5.26b)$$

$$\frac{M_{Sd}}{M_{c,Rd}} + \frac{F_{Sd}}{R_{w,Rd}} \leq 1,25 \quad \dots (5.26c)$$

where:

$M_{c,Rd}$ is the moment resistance of the cross-section given in 5.4.1(1)P;

$R_{w,Rd}$ is the appropriate value of the local transverse resistance of the web from 5.9.

S

6 Buckling resistance

6.1 General

(1)P The design values of the internal forces and moments in each member shall not exceed its design buckling resistance to:

- axial compression, as given in 6.2;
- bending moments, as given in 6.3;
- combined bending and axial compression, as given in 6.5.

(2)P In members with cross-sections that are susceptible to cross-sectional distortion, account shall be taken of possible lateral buckling of compression flanges and lateral bending of flanges generally, see 6.4.

(3)P The effects of local buckling shall be taken into account by using effective section properties determined as specified in Section 4.

(4)P The internal axial force in a member shall be taken as acting at the centroid of its gross cross-section.

(5) The resistance of a member to axial compression should be assumed to act at the centroid of its effective cross-section. If this does not coincide with the centroid of its gross cross-section, moments corresponding to the shift of the centroidal axes (see figure 6.1) should be taken into account, using the method given in 6.5.

(6)P Frame instability shall be taken into account as specified in ENV 1993-1-1.

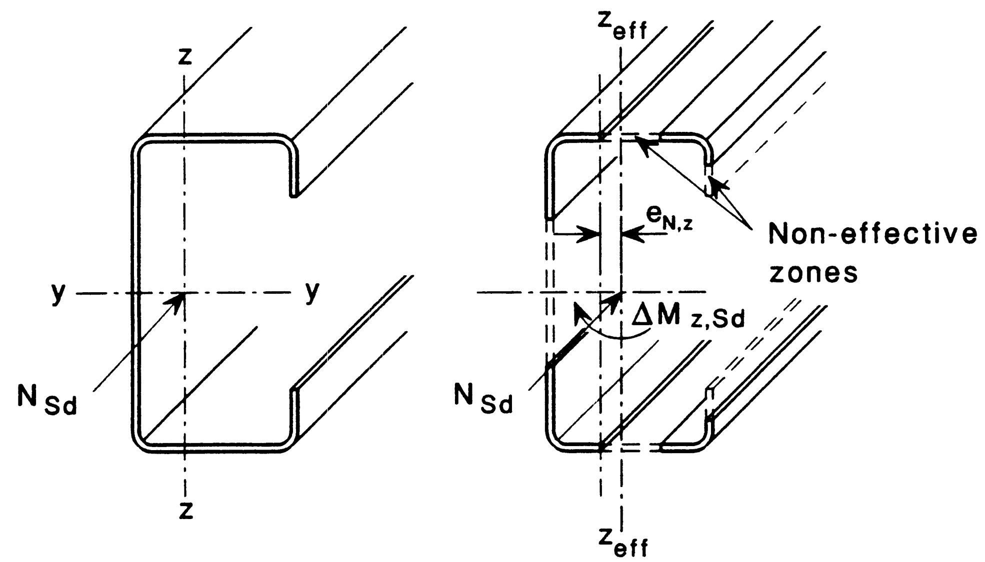


Figure 6.1: Shift of centroidal axis

6.2 Axial compression

6.2.1 Design buckling resistance

(1)P Unless determined by a second-order analysis of the member, see 6.2.2(6)P, the design buckling resistance for axial compression $N_{b,Rd}$ shall be obtained from:

$$N_{b,Rd} = \chi A_{eff} f_y / \gamma_{M1} \cong \chi \beta_A A_g f_y / \gamma_{M1} \qquad \ldots (6.1)$$

where:

- A_{eff} is the effective area of the cross-section, obtained from Section 4 by assuming a uniform compressive stress $\sigma_{com,Ed}$ equal to f_{yb}/γ_{M1} ;
- A_g is the area of the gross cross-section;
- χ is the appropriate value of the reduction factor for buckling resistance.

in which the reduction factor β_A is given by:

$$\beta_A = A_{eff} / A_g$$

(2)P The reduction factor χ for buckling resistance shall be determined from:

$$\chi = \frac{1}{\phi + [\phi^2 - \bar{\lambda}^2]^{0,5}} \quad \text{but} \quad \chi \leq 1,0 \qquad \ldots (6.2a)$$

with:

$$\phi = 0,5[1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2] \qquad \ldots (6.2b)$$

where:

- α is an imperfection factor, depending on the appropriate buckling curve;
- $\bar{\lambda}$ is the relative slenderness for the relevant buckling mode.

(3)P The lowest value of χ for flexural buckling of the member about any relevant axis, or for torsional or torsional-flexural buckling, shall be used.

(4)P The imperfection factor α corresponding to the appropriate buckling curve shall be obtained from table 6.1.

Table 6.1: Imperfection factor α

Buckling curve	a_0	a	b	c
α	0,13	0,21	0,34	0,49

6.2.2 Flexural buckling

(1)P The design buckling resistance $N_{b,Rd}$ for flexural buckling shall be obtained from 6.2.1 using the appropriate buckling curve from table 6.2 according to the type of cross-section and axis of buckling.

(2)P The buckling curve for a cross-section not included in table 6.2 may be obtained by analogy.

(3)P The buckling resistance of a closed built-up cross-section shall be determined using either:

- buckling curve b in association with the basic yield strength f_{yb} of the flat sheet material out of which the member is made by cold forming;
- buckling curve c in association with the average yield strength f_{ya} of the member after cold forming, determined as specified in 3.1.2, provided that $\beta_A = 1,0$.

(4)P The relative slenderness $\bar{\lambda}$ for flexural buckling about a given axis ($\bar{\lambda}_y$ or $\bar{\lambda}_z$) shall be determined from the following:

$$\bar{\lambda} = (\lambda / \lambda_1)[\beta_A]^{0,5} \quad \text{... (6.3a)}$$

with:

$$\lambda = \ell / i \quad \text{... (6.3b)}$$

$$\lambda_1 = \pi [E/f_y]^{0,5} \quad \text{... (6.3c)}$$

where:

ℓ is the buckling length for flexural buckling about the relevant axis (ℓ_y or ℓ_z);

i is the radius of gyration about the corresponding axis (i_y or i_z), based on the properties of the gross cross-section.

(5) Reference should be made to ENV 1993-1-1 for information on determining the buckling length ℓ for flexural buckling of a compression member, from its system length L.

(6)P As an alternative to (1)P, the design buckling resistance $N_{b,Rd}$ for flexural buckling may be obtained from a second-order analysis of the member as specified in ENV 1993-1-1, based on the properties of the effective cross-section obtained from Section 4.

6.2.3 Torsional buckling and torsional-flexural buckling

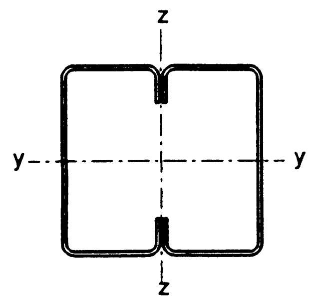
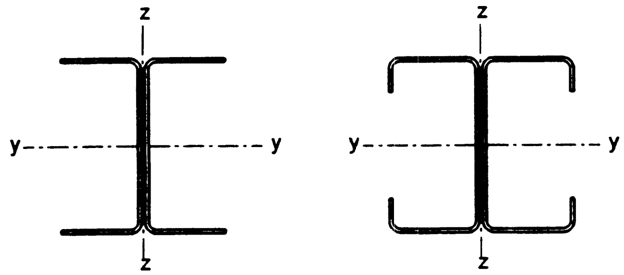
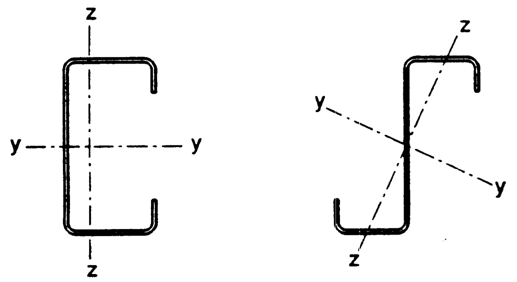
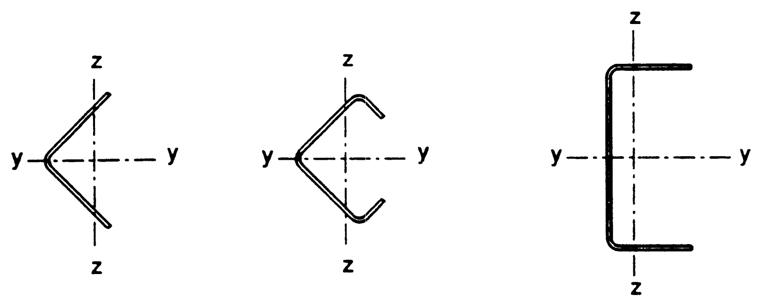
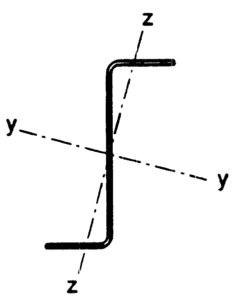
(1)P For members with point-symmetric open cross-sections, account shall be taken of the possibility that the resistance of the member to torsional buckling might be less than its resistance to flexural buckling.

(2)P For members with mono-symmetric open cross-sections, see figure 6.2, account shall be taken of the possibility that the resistance of the member to torsional-flexural buckling might be less than its resistance to flexural buckling.

(3)P For members with non-symmetric open cross-sections, account shall be taken of the possibility that the resistance of the member to either torsional or torsional-flexural buckling might be less than its resistance to flexural buckling.

(4)P The design buckling resistance $N_{b,Rd}$ for torsional or torsional-flexural buckling shall be obtained from 6.2.1 using buckling curve b.

Table 6.2: Appropriate buckling curve for various types of cross-section

Type of cross-section		Buckling about axis	Buckling curve
	if f_{yb} is used	any	b
	if f_{ya} is used *)	any	c
		y - y	a
		z - z	b
		any	b
  or other cross-section		any	c
*) The average yield strength f_{ya} should not be used unless $A_{eff} = A_g$			

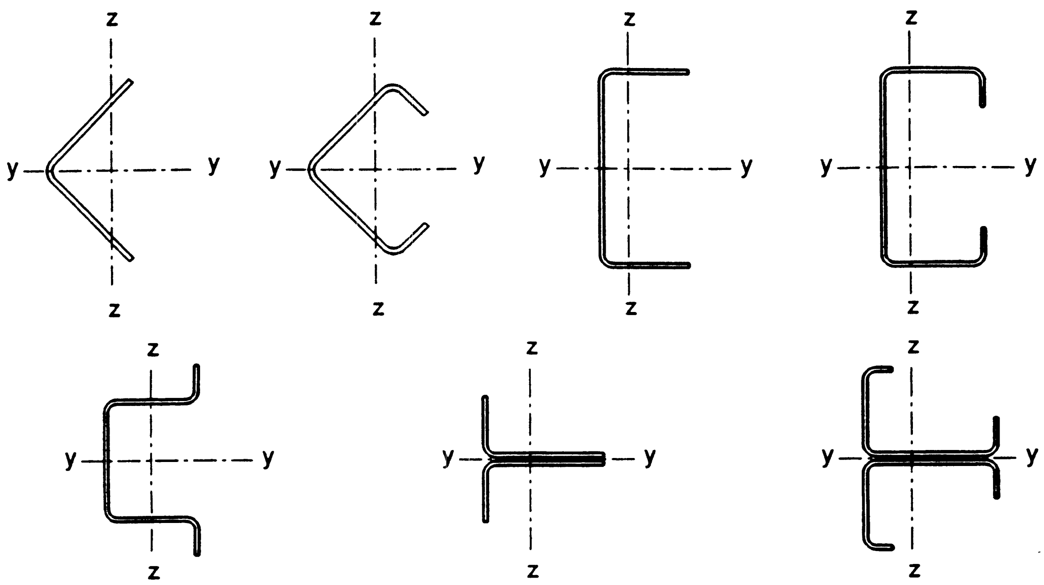


Figure 6.2: Cross-sections susceptible to torsional-flexural buckling

(5)P The relative slenderness $\bar{\lambda}$ for torsional or torsional-flexural buckling shall be obtained from:

$$\bar{\lambda} = (f_{yb} / \sigma_{cr})[\beta_A]^{1/2} \qquad \dots (6.4a)$$

with:

$$\sigma_{cr} = \sigma_{cr,TF} \quad \text{but} \quad \sigma_{cr} \leq \sigma_{cr,T} \qquad \dots (6.4b)$$

where:

$\sigma_{cr,T}$ is the elastic critical stress for torsional buckling, see (6)P;

$\sigma_{cr,TF}$ is the elastic critical stress for torsional-flexural buckling, see (7)P.

(6)P The elastic critical stress $\sigma_{cr,T}$ for torsional buckling shall be determined from:

$$\sigma_{cr,T} = \frac{1}{A_g i_o^2} \left[G I_t + \frac{\pi^2 E I_w}{\ell_T^2} \right] \qquad \dots (6.5a)$$

with:

$$i_o^2 = i_y^2 + i_z^2 + y_o^2 \qquad \dots (6.5b)$$

where:

G is the shear modulus;

I_t is the torsion constant of the gross cross-section;

I_w is the warping constant of the gross cross-section;

i_y is the radius of gyration of the gross cross-section about the y - y axis;

i_z is the radius of gyration of the gross cross-section about the z - z axis;

ℓ_T is the buckling length of the member for torsional buckling;

y_0 is the distance from the shear centre to the centroid of the gross cross-section.

(7)P For cross-sections that are symmetrical about the y - y axis, the elastic critical stress $\sigma_{cr,TF}$ for torsional-flexural buckling shall be determined from:

$$\sigma_{cr,TF} = \frac{1}{2\beta}\left[(\sigma_{cr,y} + \sigma_{cr,T}) - \sqrt{(\sigma_{cr,y} + \sigma_{cr,T})^2 - 4\beta\sigma_{cr,y}\sigma_{cr,T}}\right] \quad \ldots (6.6)$$

with:

$$\sigma_{cr,y} = \pi^2 E/(\ell_y/i_y)^2$$

$$\beta = 1 - (y_o/i_o)^2$$

where:

ℓ_y is the buckling length for flexural buckling about the y - y axis.

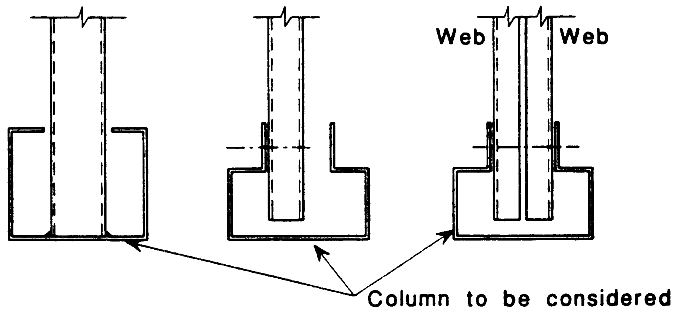
(8)P The buckling length ℓ_T for torsional or torsional-flexural buckling shall be determined taking into account the degree of torsional and warping restraint at each end of the system length L_T.

(9) Normal practical connections should not be assumed to provide full torsional or warping restraint and therefore the theoretical values of ℓ_T/L_T (1,0 for "torsion fixed, warping free" or 0,5 for "torsion fixed, warping fixed") should not normally be used directly in design.

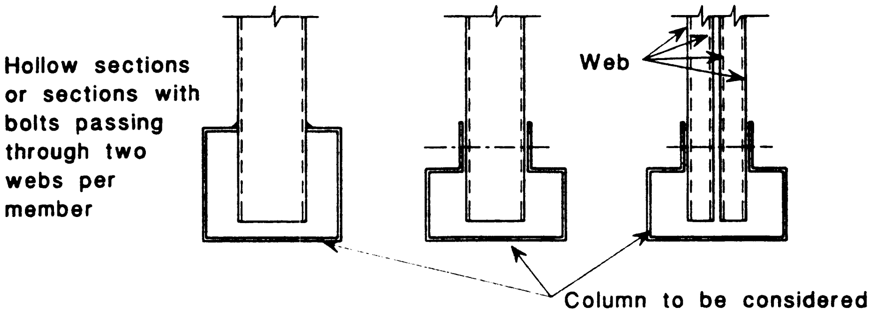
(10) For practical connections at each end, the value of ℓ_T/L_T may be taken as follows:

- 1,0 for connections that provide partial restraint against torsion and warping, see figure 6.3(a);
- 0,7 for connections that provide significant restraint against torsion and warping, see figure 6.3(b).

(11) Improved values of ℓ_T/L_T may be used where this is justified by tests in accordance with Section 9.



a) connections capable of giving partial torsional and warping restraint



b) connections capable of giving significant torsional and warping restraint

Figure 6.3: Torsional and warping restraint from practical connections

6.3 Lateral-torsional buckling of members subject to bending

(1)P The design buckling resistance moment of a member that is susceptible to lateral-torsional buckling shall be determined from:

$$M_{b,Rd} = \chi_{LT} W_{eff} f_{yb} / \gamma_{M1} \quad \text{... (6.7)}$$

in which $\bar{\lambda}_{LT}$ is obtained from the following:

- if $\bar{\lambda}_{LT} \leq 0,4$:

$$\chi_{LT} = 1,0 \quad \text{... (6.8a)}$$

- if $\bar{\lambda}_{LT} > 0,4$:

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \bar{\lambda}_{LT}^2]^{0,5}} \quad \text{... (6.8b)}$$

with:

$$\phi_{LT} = 0,5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0,2\right) + \bar{\lambda}_{LT}^2\right] \quad \text{... (6.9a)}$$

$$\bar{\lambda}_{LT} = [f_y W_{eff} / M_{cr}]^{0,5} \quad \text{... (6.9b)}$$

$\alpha_{LT} = 0,21$ [buckling curve a in table 6.1]

where:

M_{cr} is the elastic critical moment of the gross cross-section, for lateral-torsional buckling about the relevant axis;

W_{eff} is the section modulus of the effective cross-section, if subject only to moment about the relevant axis.

NOTE: Information for the calculation of M_{cr} is given in annex F of ENV 1993-1-1.

(2) This method should not be used for U-sections and similar sections that have a significant angle between the principal axes of the effective cross-section, compared to those of the gross cross-section.

6.4 Distortional buckling

(1)P Distortional buckling shall be taken into account where it constitutes the critical failure mode.

(2) The effects of distortional buckling should be allowed for in cases such as those indicated in figures 6.4(a), (b) and (c), if the lowest elastic critical stress for a distortional buckling mode, evaluated by examining the various possible deformation modes, is lower than the elastic critical stresses for local and overall buckling, as indicated in figure 6.5.

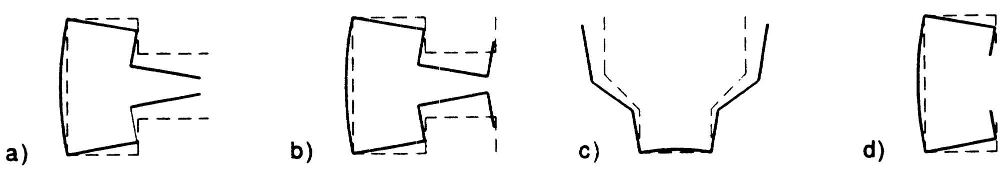


Figure 6.4: Examples of distortional buckling modes

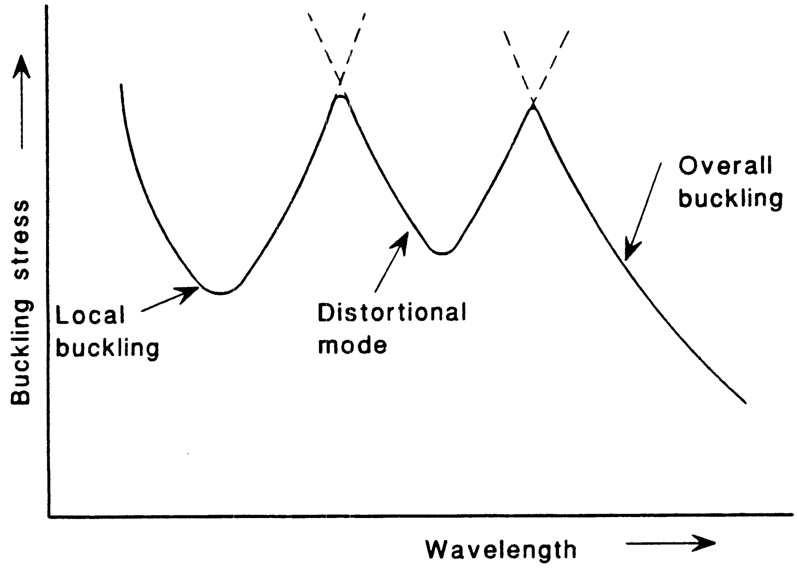


Figure 6.5: Elastic critical buckling stresses for various failure modes

(3) For elements with edge or intermediate stiffeners as indicated in figure 6.4(d), no further allowance need be made for distortional buckling if the effective area of the stiffener is reduced as specified in 4.3.

6.5 Bending and axial compression

6.5.1 General

(1)P All members subject to combined bending and axial compression shall satisfy the criterion:

$$\frac{N_{Sd}}{\chi_{min} f_{yb} A_{eff} / \gamma_{M1}} + \frac{\kappa_y (M_{y,Sd} + \Delta M_{y,Sd})}{f_{yb} W_{eff,y,com} / \gamma_{M1}} + \frac{\kappa_z (M_{z,Sd} + \Delta M_{z,Sd})}{f_{yb} W_{eff,z,com} / \gamma_{M1}} \leq 1 \qquad \text{... (6.10)}$$

where:

A_{eff}	is	the effective area of an effective cross-section that is subject only to axial compression, see figure 6.6(a);
$W_{eff,y,com}$	is	the effective section modulus for the maximum compressive stress in an effective cross-section that is subject only to moment about the y - y axis, see figure 6.6(b);
$W_{eff,z,com}$	is	the effective section modulus for the maximum compressive stress in an effective cross-section that is subject only to moment about the z - z axis, see figure 6.6(c);
$\Delta M_{y,Sd}$	is	the additional moment due to possible shift of the centroidal axis in the y direction, see 5.6(2)P;
$\Delta M_{z,Sd}$	is	the additional moment due to possible shift of the centroidal axis in the z direction, see 5.6(2)P;
χ_y	is	the reduction factor from 6.2 for buckling about the y - y axis;
χ_z	is	the reduction factor from 6.2 for buckling about the z - z axis;
χ_{min}	is	the lesser of χ_y and χ_z.

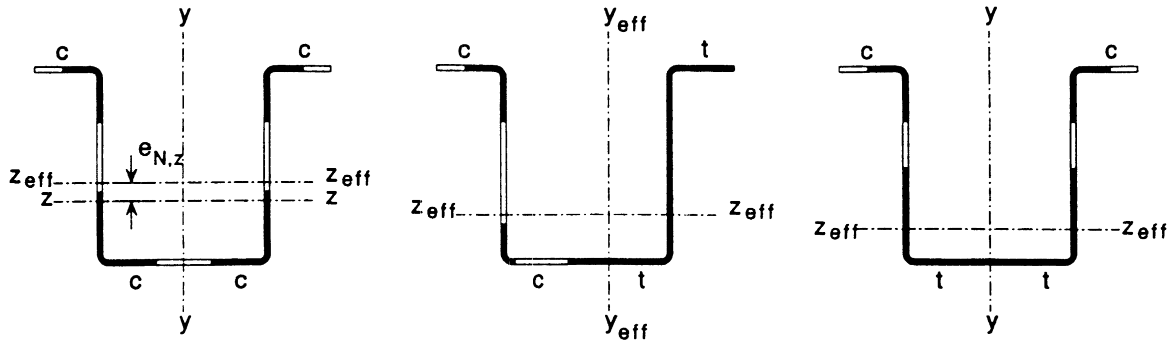


Figure 6.6: Calculation of effective section properties

(2)P The factors κ_y and κ_z in expression (6.10) shall be obtained from:

$$\kappa_y = 1 - \frac{\mu_y N_{Sd}}{\chi_y f_{yb} A_{eff}} \quad \text{but } \kappa_y \leq 1,50 \qquad \dots (6.11a)$$

$$\kappa_z = 1 - \frac{\mu_z N_{Sd}}{\chi_z f_{yb} A_{eff}} \quad \text{but } \kappa_z \leq 1,50 \qquad \dots (6.11b)$$

with:

$$\mu_y = \bar{\lambda}_y (2\beta_{M,y} - 4) \quad \text{but } \mu_y \leq 0,90$$

$$\mu_z = \bar{\lambda}_z (2\beta_{M,z} - 4) \quad \text{but } \mu_z \leq 0,90$$

where:

$\beta_{M,y}$ is the equivalent uniform moment factor for buckling about the y - y axis;

$\beta_{M,z}$ is the equivalent uniform moment factor for buckling about the z - z axis;

NOTE: The expressions for μ_y and μ_z can result in negative values.

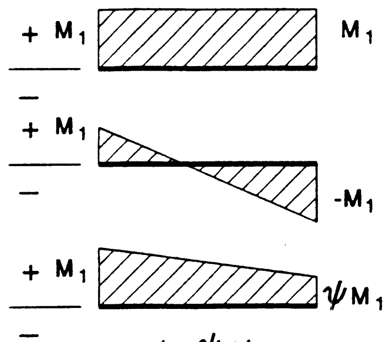
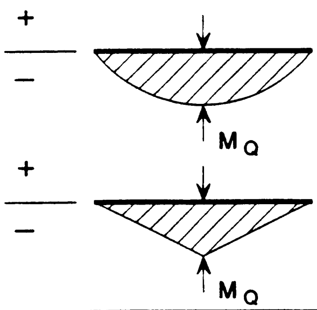
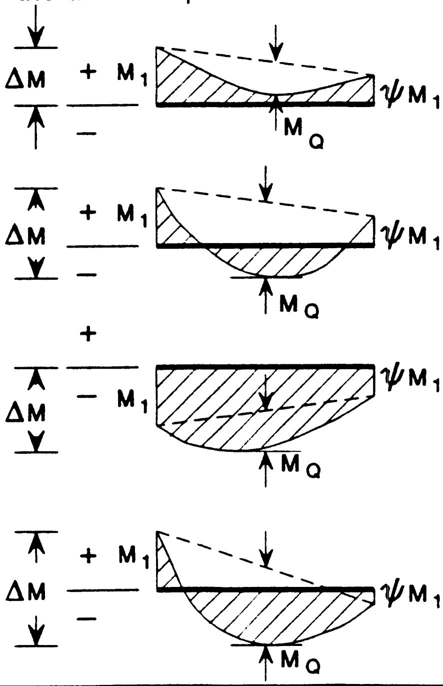
(3)P The equivalent uniform moment factors $\beta_{M,y}$ and $\beta_{M,z}$ shall be based on the shape of the bending moment diagram about the relevant axis, between points that are braced in the relevant direction, as given in table 6.3. The bending moments taken into account shall include the additional moments $\Delta M_{y,Sd}$ and $\Delta M_{z,Sd}$ due to possible shift of the centroidal axes.

(4)P The equivalent uniform moment factors $\beta_{M,y}$ and $\beta_{M,z}$ shall be determined from table 6.4.

Table 6.3: Relevant axes for determining β_M factors

Factor	Diagram of bending moments applied about axis:	Lateral buckling about axis:	System length taken between points braced in direction:
$\beta_{M,y}$	y - y	y - y	z - z
$\beta_{M,z}$	z - z	z - z	y - y
$\beta_{M,LT}$	y - y	z - z	y - y

Table 6.4: Equivalent uniform moment factors

Moment diagram	Equivalent uniform moment factor β_M
End moments 	$\beta_{M,\psi} = 1{,}8 - 0{,}7\psi$
Moments due to in-plane lateral loads. 	$\beta_{M,Q} = 1{,}3$ $\beta_{M,Q} = 1{,}4$
Moments due to in-plane lateral loads plus end moments 	$\beta_M = \beta_{M,\psi} + \frac{M_Q}{\Delta M}(\beta_{M,Q} - \beta_{M,\psi})$ where: M_Q = $\lvert \max M \rvert$ due to lateral load only ΔM = $\lvert \max M \rvert$ for moment diagram without change of sign ΔM = $\lvert \max M \rvert + \lvert \min M \rvert$ for moment diagram with change of sign

6.5.2 Bending and axial compression with lateral-torsional buckling

(1)P Members that are susceptible to lateral-torsional buckling shall also satisfy the criterion:

$$\frac{N_{Sd}}{\chi_{lat} f_{yb} A_{eff} / \gamma_{M1}} + \frac{\kappa_{LT}(M_{y,Sd} + \Delta M_{y,Sd})}{\chi_{LT} f_{yb} W_{eff,y,com} / \gamma_{M1}} + \frac{\kappa_z (M_{z,Sd} + \Delta M_{z,Sd})}{f_{yb} W_{eff,z,com} / \gamma_{M1}} \leq 1 \qquad \text{... (6.12)}$$

where:

χ_{LT} is the reduction factor for lateral-torsional buckling, see 6.3.

(2)P Generally the reduction factor χ_{lat} shall be taken as equal to χ_z. However if torsional-flexural buckling (see 6.2.3) or distortional buckling (see 6.3) are potential failure modes, χ_{lat} shall be taken as equal to the smallest of χ_z and the values of χ for torsional-flexural or distortional buckling.

(3)P The factor κ_{LT} in expression (6.12) shall be obtained from:

$$\kappa_{LT} = 1 - \frac{\mu_{LT} N_{Sd}}{\chi_z f_{yb} A_{eff}} \quad \text{but} \quad \kappa_{LT} \leq 1,0 \qquad \text{... (6.13a)}$$

with:

$$\mu_{LT} = 0,15 \bar{\lambda}_{lat} \beta_{M,LT} - 0,15 \quad \text{but} \quad \mu_{LT} \leq 0,90 \qquad \text{... (6.13b)}$$

(4)P The equivalent uniform moment factor for lateral-torsional buckling $\beta_{M,LT}$ shall be based on the shape of the bending moment diagram about the y - y axis, between points that are braced in the y - y direction, as also given in table 6.3. The bending moments taken into account shall include the additional moment $\Delta M_{y,Sd}$ due to possible shift of the centroidal axis.

(5)P The equivalent uniform moment factor $\beta_{M,LT}$ shall be determined from table 6.4.

7 Serviceability limit states

7.1 General

(1)P The principles for serviceability limit states given in Section 4 of ENV 1993-1-1 shall also be applied to cold formed thin gauge members and sheeting.

(2) The application rules given in Section 4 of ENV 1993-1-1 should also be applied to cold formed thin gauge members and sheeting, except as modified by the supplementary application rules in this Section 7.

(3) The design values for the characteristic (rare) load combination, see ENV 1991-1, should be used in serviceability limit states verifications for plastic deformation and for deflection calculations.

NOTE: In ENV 1993-1-1 the characteristic (rare) combination is termed the "rare combination".

(4) The properties of the effective cross-section for serviceability limit states obtained from Section 4 should be used in all serviceability limit state calculations for cold formed thin-gauge members and sheeting.

(5) The effective second moment of area I_{eff} may be taken as variable along the span. Alternatively a uniform value may be used, based on the maximum span moment due to serviceability loading.

7.2 Plastic deformation

(1) In order to avoid excessive plastic deformation under service conditions, if redistribution of internal moments and forces is used in the global analysis for ultimate limit states, it should be ensured that no significant plastic deformations are liable to appear under serviceability loading.

(2) In such cases, the combination of support moment and support reaction at an internal support should not exceed 0,9 times the combined design resistance, determined using $\gamma_{M,ser}$.

(3) The combined design resistance may be determined from 5.11, but using the effective cross-section for serviceability limit states and $\gamma_{M,ser}$. Alternatively the design resistance may be determined from tests in accordance with Section 9, by dividing the characteristic resistance R_k by $\gamma_{M,ser}$.

NOTE: Appropriate testing procedures are given in annex A.

7.3 Deflections

(1) Deflections should be limited to values that would not adversely affect the appearance or effective use of the structure, or damage finishes or non-structural elements.

(2) The deflections may be calculated assuming elastic behaviour.

(3) The calculated deflection of a purlin in the direction perpendicular to the roof surface, due to variable gravity loads, should not exceed $L/180$, where L is the span of the purlin.

7.4 Sheeting

(1) The provisions given in 7.2 should also be adopted for the design of sheeting.

(2) The provisions given in 7.3(1) and (2) should also be adopted for the design of sheeting, but the deflection limit given in 7.3(3) does not apply to sheeting.

8 Joints and connections

8.1 General

8.1.1 Design assumptions

(1)P Joints shall be designed on the basis of a realistic assumption of the distribution of internal forces and moments, having regard to relative stiffnesses within the joint. This distribution shall correspond with direct load paths through the elements of the joint. It shall be ensured that equilibrium is maintained with the applied external forces and moments.

(2)P Allowance may be made for the ductility of steel in facilitating the redistribution of internal forces generated within a joint. Accordingly, residual stresses and stresses due to tightening of fasteners and normal accuracy of fit-up need not be considered.

(3)P Ease of fabrication and erection shall be taken into account in the design of the details of connections and splices. Attention shall be paid to the clearances necessary for tightening of fasteners, the requirements of welding procedures, and the need for subsequent inspection, surface treatment and maintenance.

8.1.2 Intersections

(1)P Members meeting at a joint shall normally be arranged so that their centroidal axes intersect at a point.

(2)P If there is eccentricity at intersections, the members and connections shall be designed to accommodate the moments that result.

(3) In the case of bolted framing of angles and tees, the setting out lines for the bolts may be used instead of the centroidal axes for the purpose of intersection at the joint.

8.1.3 Connections subject to impact, vibration or load reversal

(1)P Where a connection is subject to impact or vibration, either preloaded bolts, bolts with locking devices or welding shall be used.

(2)P Where a connection that is loaded in shear is subject to reversal of stress (unless such stress is due solely to wind) or where for some special reason slipping of bolts is not acceptable, either preloaded bolts, fitted bolts or welding shall be used.

8.2 Requirements for joints

8.2.1 Joints in simple framing

(1)P In simple framing, the joints between the members shall have nominally pinned connections that:

- are capable of transmitting the forces calculated in the global analysis;
- are able to sustain the resulting rotations;
- do not develop significant moments adversely affecting members of the structure.

8.2.2 Joints in continuous framing

(1)P In continuous framing, the joints between the members shall be capable of transmitting the forces and moments calculated in the global analysis.

(2) If elastic global analysis is used, the rigidity of a moment-resisting joint should not be less than that of the connected member.

(3) In the case of plastic global analysis, the moment resistance of a moment-resisting joint that is located at, or adjacent to, a plastic hinge location, should not be less than the moment resistance of the cross-section of the connected member. In addition, the joint should have sufficient rotation capacity.

8.2.3 Joints in semi-continuous framing

(1)P In semi-continuous framing, the joints between the members shall be capable of providing a predictable degree of interaction. The moment-resisting joints shall be capable of resisting the internal moments developed by the joints themselves, in addition to the other internal forces and moments at the joints.

(2) The moment-resisting joints should have sufficient rigidity to develop the moments calculated in the global analysis, but sufficient flexibility to avoid developing larger moments than they can resist.

(3) If the design value of the moment resistance of a joint is less than that of the connected member, it should be demonstrated that the rotation capacity of the joint is sufficient to allow the necessary redistribution of internal moments and forces to take place.

8.3 Splices and end connections of members subject to compression

(1)P Splices and end connections in members that are subject to compression, shall either have at least the same resistance as the cross-section of the member, or be designed to resist an additional bending moment due to the second-order effects within the member, in addition to the internal compressive force N_{Sd} and the internal moments $M_{y,Sd}$ and $M_{z,Sd}$ obtained from the global analysis.

(2) In the absence of a second-order analysis of the member, this additional moment ΔM_{Sd} should be taken as acting about the cross-sectional axis that gives the smallest value of the reduction factor χ for flexural buckling, see 6.2.1(2)P, with a value determined from:

$$\Delta M_{Sd} = N_{Sd} \left[\frac{1}{\chi} - 1 \right] \frac{W_{eff}}{A_{eff}} \sin\frac{\pi a}{\ell} \qquad \dots (8.1)$$

where:

- A_{eff} is the effective area of the cross-section;
- a is the distance from the splice or end connection to the nearer point of contraflexure;
- ℓ is the buckling length of the member between points of contraflexure, for buckling about the relevant axis;
- W_{eff} is the section modulus of the effective cross-section for bending about the relevant axis.

(3) Splices and end connections should be designed in such a way that load can be transmitted to the effective portions of the cross-section.

(4) If the constructional details at the ends of a member are such that the line of action of the internal axial force cannot be clearly identified, a suitable eccentricity should be assumed and the resulting moments should be taken into account in the design of the member, the end connections and the splice, if there is one.

8.4 Connections with mechanical fasteners

(1)P Connections with mechanical fasteners shall be compact in shape. The positions of the fasteners shall be arranged to provide sufficient room for satisfactory assembly and maintenance.

(2)P The shear forces on individual mechanical fasteners in a connection may be assumed to be equal, provided that:

- the fasteners have sufficient ductility;
- shear is not the critical failure mode.

(3)P The fasteners to be used shall have known and documented resistances.

(4)P The resistances of individual mechanical fasteners shall be determined either by calculation or from the results of tests in accordance with Section 9. For design by calculation, the resistances of mechanical fasteners subject to static loads shall be determined from:

- table 8.1 for blind rivets;
- table 8.2 for self-tapping screws;
- table 8.3 for cartridge fired pins;
- table 8.4 for bolts.

(5)P In tables 8.1 to 8.4 the meanings of the symbols shall be taken as follows:

A	is	the gross cross-sectional area of a bolt;
A_s	is	the tensile stress area of a bolt;
A_{net}	is	the net cross-sectional area of the connected part;
d	is	the nominal diameter of the fastener;
d_o	is	the nominal diameter of the hole;
d_w	is	the diameter of the washer or the head of the fastener;
e_1	is	the end distance from the centre of the fastener to the adjacent end of the connected part, in the direction of load transfer, see figure 8.1;
e_2	is	the edge distance from the centre of the fastener to the adjacent edge of the connected part, in the direction perpendicular to the direction of load transfer, see figure 8.1;
f_{ub}	is	the ultimate tensile strength of the bolt material;
$f_{u,sup}$	is	the ultimate tensile strength of the supporting member into which a screw is fixed;
n	is	the number of sheets that are fixed to the supporting member by the same screw or pin;
p_1	is	the spacing centre-to-centre of fasteners in the direction of load transfer, see figure 8.1;
p_2	is	the spacing centre-to-centre of fasteners in the direction perpendicular to the direction of load transfer, see figure 8.1;
t	is	the thickness of the thinner connected part or sheet;
t_1	is	the thickness of the thicker connected part or sheet;
t_{sup}	is	the thickness of the supporting member into which a screw or a pin is fixed.

(6)P The partial factor γ_M for calculating the design resistances of mechanical fasteners shall be taken as:

$$\gamma_{M2} = \boxed{1{,}25}$$

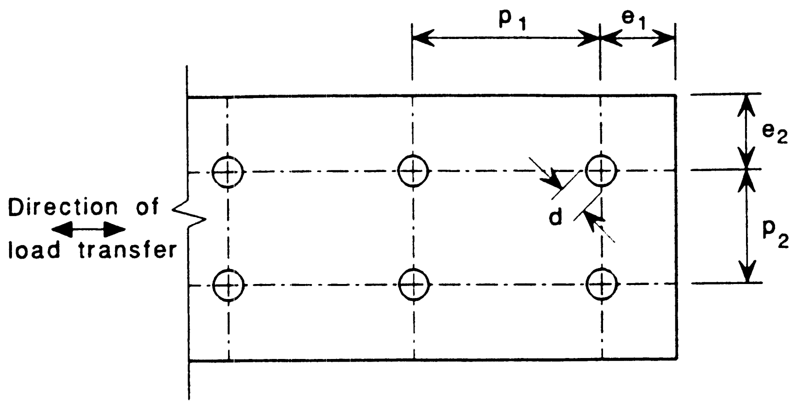


Figure 8.1: End distance, edge distance and spacings for fasteners and spot welds

(7)P The resistance of a connection shall preferably be based on tests in accordance with Section 9. Alternatively, the resistance of a connection subject to static loads may be determined from tables 8.1 to 8.4, provided that the limits on dimensions of fasteners and thicknesses of sheets stated in the tables are satisfied.

(8) If the pull-out resistance $F_{o,Rd}$ of a fastener is smaller than its pull-through resistance $F_{p,Rd}$ the deformation capacity should be determined from the results of tests in accordance with Section 9.

(9) The pull-through resistances given in tables 8.2 and 8.3 for self-tapping screws and cartridge fired pins should be reduced if the fasteners are not located centrally in the troughs of the sheeting. If attachment is at a quarter point, the design resistance should be reduced to $0{,}9F_{p,Rd}$ and if there are fasteners at both quarter points, the resistance should be taken as $0{,}7F_{p,Rd}$ per fastener, see figure 8.2.

(10) For a fastener loaded in combined shear and tension, if either the shear resistance $F_{v,Rd}$ or the tension resistance $F_{t,Rd}$ have been determined by testing, the resistance to combined shear and tension should also be verified on the basis of tests in accordance with Section 9. Provided that both $F_{t,Rd}$ and $F_{v,Rd}$ are determined by calculation on the basis of tables 8.1 to 8.4, the resistance of the fastener to combined shear and tension may be verified using:

$$F_{t,Sd}/F_{t,Rd} + F_{v,Sd}/F_{v,Rd} \leq 1 \qquad \ldots (8.2)$$

(11) The limit state of gross distortion may be assumed to be satisfied if the design resistance is obtained from tables 8.1 to 8.4, provided that the fastening is through a flange not more than 150 mm wide.

(12) The diameter of pre-drilled holes for screws should be in accordance with the manufacturer's guidelines. These guidelines should be based on following criteria:

- the applied torque should be just higher than the threading torque;
- the applied torque should be lower than the thread stripping torque or head-shearing torque;
- the threading torque should be smaller than 2/3 of the head-shearing torque.

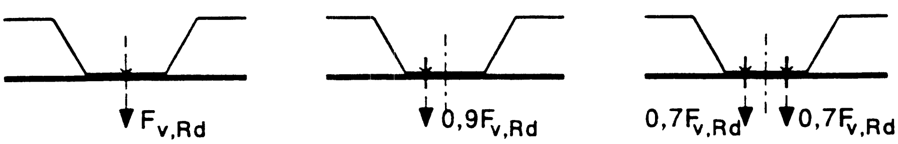


Figure 8.2: Reduction of tension resistance due to the position of fasteners

Table 8.1: Design resistances for blind rivets [1)]

Rivets loaded in shear:
Bearing resistance: $F_{b,Rd} = \alpha f_u d t / \gamma_{M2}$ in which α is given by the following: - if $t = t_1$: $\alpha = 3{,}2\sqrt{t/d}$ but $\alpha \leq 2{,}1$ - if $t_1 \geq 2{,}5t$: $\alpha = 2{,}1$ - if $t < t_1 \leq 2{,}5t$: obtain α by linear interpolation. Net-section resistance: $F_{n,Rd} = A_{net} f_u / \gamma_{M2}$ Shear resistance: Shear resistance $F_{v,Rd}$ to be determined by testing.
Conditions: $F_{v,Rd} \geq 1{,}2 F_{b,Rd}$ and $F_{v,Rd} \geq 1{,}2 F_{n,Rd}$
Rivets loaded in tension: [2)]
Pull-through resistance: Pull-through resistance $F_{p,Rd}$ to be determined by testing. Pull-out resistance: Not relevant for rivets. Tension resistance: Tension resistance $F_{t,Rd}$ to be determined by testing.
Conditions: $F_{t,Rd} \geq n F_{p,Rd}$
Range of validity: [3)]
$e_1 \geq 3d$ $p_1 \geq 3d$ $2{,}6\,\text{mm} \leq d \leq 6{,}4\,\text{mm}$ $e_2 \geq 1{,}5d$ $p_2 \geq 3d$
[1)] In this table it is assumed that the thinnest sheet is next to the preformed head of the blind rivet. [2)] Blind rivets are not usually used in tension. [3)] Blind rivets may be used beyond this range of validity if the resistance is determined from the results of tests in accordance with Section 9.

Table 8.2: Design resistances for self-tapping screws [1)]

Screws loaded in shear:

Bearing resistance:

$$F_{b,Rd} = \alpha f_u d t / \gamma_{M2}$$

in which α is given by the following:

- if $t = t_1$: $\alpha = 3{,}2\sqrt{t/d}$ but $\alpha \leq 2{,}1$
- if $t_1 \geq 2{,}5t$: $\alpha = 2{,}1$
- if $t < t_1 \leq 2{,}5t$: obtain α by linear interpolation.

Net-section resistance:

$$F_{n,Rd} = A_{net} f_u / \gamma_{M2}$$

Shear resistance:

Shear resistance $F_{v,Rd}$ to be determined by testing.

Conditions:

$F_{v,Rd} \geq 1{,}2 F_{b,Rd}$ and $F_{v,Rd} \geq 1{,}2 F_{n,Rd}$

Screws loaded in tension:

Pull-through resistance: [2)]

- for static loads: $F_{p,Rd} = d_w t f_u / \gamma_{M2}$
- for screws subject to repeated wind loads: $F_{pr,Rd} = 0{,}5 d_w t f_u / \gamma_{M2}$

Pull-out resistance:

$$F_{o,Rd} = 0{,}65 d t_{sup} f_{u,sup} / \gamma_{M2}$$

Tension resistance:

Tension resistance $F_{t,Rd}$ to be determined by testing.

Conditions:

$F_{t,Rd} \geq n F_{p,Rd}$ and $F_{t,Rd} \geq F_{o,Rd}$

Range of validity: [3)]

Generally:	$e_1 \geq 3d$	$p_1 \geq 3d$	$3{,}0\,\text{mm} \leq d \leq 8{,}0\,\text{mm}$
	$e_2 \geq 1{,}5d$	$p_2 \geq 3d$	
For tension:	$0{,}5\,\text{mm} \leq t \leq 1{,}5\,\text{mm}$	and	$t_1 \geq 0{,}9\,\text{mm}$

1) In this table it is assumed that the thinnest sheet is next to the head of the screw.

2) These values assume that the washer has sufficient rigidity to prevent it from being deformed appreciably or pulled over the head of the fastener.

3) Self-tapping screws may be used beyond this range of validity if the resistance is determined from the results of tests in accordance with Section 9.

Table 8.3: Design resistances for cartridge fired pins

Pins loaded in shear:

Bearing resistance:

$F_{b,Rd} = 3{,}2 f_u d t / \gamma_{M2}$

Net-section resistance:

$F_{n,Rd} = A_{net} f_u / \gamma_{M2}$

Shear resistance:

Shear resistance $F_{v,Rd}$ to be determined by testing.

Conditions:

$F_{v,Rd} \geq 1{,}5 F_{b,Rd}$ and $F_{v,Rd} \geq 1{,}5 F_{n,Rd}$

Pins loaded in tension:

Pull-through resistance: [1]

- for static loads: $F_{p,Rd} = d_w t f_u / \gamma_{M2}$
- for repeated wind loads: $F_{pr,Rd} = 0{,}5 d_w t f_u / \gamma_{M2}$

Pull-out resistance:

Pull-out resistance $F_{o,Rd}$ to be determined by testing.

Tension resistance:

Tension resistance $F_{t,Rd}$ to be determined by testing.

Conditions:

$F_{o,Rd} \geq n F_{p,Rd}$

$F_{t,Rd} \geq F_{o,Rd}$

Range of validity: [2]

Generally:	$e_1 \geq 4{,}5d$	$3{,}7\,\text{mm} \leq d \leq 6{,}0\,\text{mm}$
	$e_2 \geq 4{,}5d$	for $d = 3{,}7\,\text{mm}$: $t_{sup} \geq 4{,}0\,\text{mm}$
	$p_1 \geq 4{,}5d$	for $d = 4{,}5\,\text{mm}$: $t_{sup} \geq 6{,}0\,\text{mm}$
	$p_2 \geq 4{,}5d$	for $d = 5{,}2\,\text{mm}$: $t_{sup} \geq 8{,}0\,\text{mm}$
For tension:	$0{,}5\,\text{mm} \leq t \leq 1{,}5\,\text{mm}$	$t_{sup} \geq 6{,}0\,\text{mm}$

1) These values assume that the washer has sufficient rigidity to prevent it from being deformed appreciably or pulled over the head of the fastener.

2) Cartridge fired pins may be used beyond this range of validity if the resistance is determined from the results of tests in accordance with Section 9.

Table 8.4: Design resistances for bolts

Bolts loaded in shear:

Bearing resistance:

$F_{b,Rd} = 2{,}5 f_u d t / \gamma_{M2}$ but $F_{b,Rd} \leq f_u e_1 t / 1{,}2 / \gamma_{M2}$

Net-section resistance:

$F_{n,Rd} = (1 + 3r(d_o/u - 0{,}3)) A_{net} f_u / \gamma_{M2}$ but $F_{n,Rd} \leq A_{net} f_u / \gamma_{M2}$

with:

r = [number of bolts at the cross-section] / [total number of bolts in the connection]

$u = 2e_2$ but $u \leq p_2$

Shear resistance:

- for strength grades 4.6, 5.6 and 8.8:

  $F_{v,Rd} = 0{,}6 f_{ub} A_s / \gamma_{M2}$
- for strength grades 4.8, 5.8, 6.8 and 10.9:

  $F_{v,Rd} = 0{,}5 f_{ub} A_s / \gamma_{M2}$

Conditions:

$F_{v,Rd} \geq 1{,}2 F_{b,Rd}$ and $F_{v,Rd} \geq 1{,}2 F_{n,Rd}$

Bolts loaded in tension:

Pull-through resistance:

Pull-through resistance $F_{p,Rd}$ to be determined by testing.

Pull-out resistance:

Not relevant for bolts.

Tension resistance:

$F_{t,Rd} = 0{,}9 f_{ub} A_s / \gamma_{M2}$

Conditions:

$F_{t,Rd} \geq n F_{p,Rd}$

Range of validity: [1)]

$e_1 \geq 1{,}5d$	$p_1 \geq 3d$	$t \geq 1{,}25$ mm	Minimum bolt size: M6
$e_2 \geq 1{,}5d$	$p_2 \geq 3d$		Strength grades: 4.6 - 10.9

1) Bolts may be used beyond this range of validity if the resistance is determined from the results of tests in accordance with Section 9.

8.5 Spot welds

(1)P Spot welds may be used with as-rolled or galvanized parent material up to 4,0 mm thick, provided that the thinner connected part is not more than 3,0 mm thick.

(2) Spot welds may be either resistance welded or fusion welded.

(3)P The design resistance $F_{v,Rd}$ of a spot weld loaded in shear shall be determined using table 8.5.

(4)P In table 8.5 the meanings of the symbols shall be taken as follows:

A_{net} is the net cross-sectional area of the connected part;

t is the thickness of the thinner connected part or sheet [mm];

t_1 is the thickness of the thicker connected part or sheet;

and the end and edge distances e_1 and e_2 and the spacings p_1 and p_2 are as defined in 8.4(4)P.

(5)P The partial factor γ_M for calculating the design resistances of spot welds shall be taken as:

$\gamma_{M2} = \boxed{1,25}$

Table 8.5: Design resistances for spot welds

Spot welds loaded in shear:
Tearing and bearing resistance: - if $t \leq t_1 \leq 2,5t$: $F_{tb,Rd} = 2,7\sqrt{3}\,t\,d_s f_u / \gamma_{M2}$ - if $t_1 > 2,5t$: $F_{tb,Rd} = 2,7\sqrt{3}\,t\,d_s f_u / \gamma_{M2}$ but $F_{tb,Rd} \leq 0,7 d_s^2 f_u / \gamma_{M2}$ and $F_{tb,Rd} \leq 3,1\,t\,d_s f_u / \gamma_{M2}$ End resistance: $F_{e,Rd} = 1,4\,t\,e_1 f_u / \gamma_{M2}$ Net section resistance: $F_{n,Rd} = A_{net} f_u / \gamma_{M2}$ Shear resistance: $F_{v,Rd} = \frac{\pi}{4} d_s^2 f_u / \gamma_{M2}$
Conditions: $F_{v,Rd} \geq 1,25 F_{tb,Rd}$ and $F_{v,Rd} \geq 1,25 F_{e,Rd}$ and $F_{v,Rd} \geq 1,25 F_{n,Rd}$
Range of validity:
$2d_s \leq e_1 \leq 6d_s$ $3d_s \leq p_1 \leq 8d_s$ $e_2 \leq 4d_s$ $3d_s \leq p_2 \leq 6d_s$

(6)P The interface diameter d_s of a spot weld shall be determined from the following:

- for fusion welding: $d_s = 0{,}5t + 5\,\text{mm}$ ... (8.3a)

- for resistance welding: $d_s = 5\sqrt{t}$ [with t in mm] ... (8.3b)

(7)P The value of d_s actually produced by the welding procedure shall be verified by shear tests in accordance with Section 9, using single-lap test specimens as shown in figure 8.3.

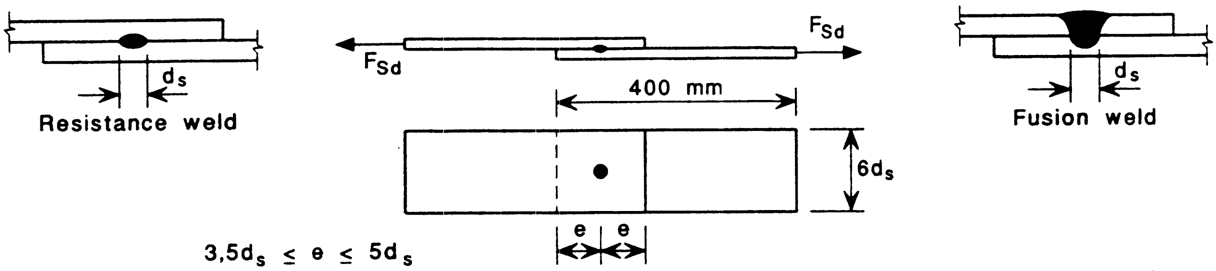


Figure 8.3: Test specimen for shear tests of spot welds

8.6 Lap welds

8.6.1 General

(1)P This clause 8.6 shall be used for the design of arc-welded lap welds where the parent material is 4,0 mm thick or less. For thicker parent material, lap welds shall be designed using ENV 1993-1-1.

(2)P The weld size shall be chosen such that the resistance of the connection is governed by the thickness of the connected part or sheet, rather than the weld.

(3) The requirement in (2)P may be assumed to be satisfied if the throat size of the weld is at least equal to the thickness of the connected part or sheet.

(4)P The partial factor γ_M for calculating the design resistances of lap welds shall be taken as:

$\gamma_{M2} = \boxed{1{,}25}$

8.6.2 Fillet welds

(1)P The design resistance $F_{w.Rd}$ of a fillet-welded connection shall be determined from the following:

- for a side fillet that comprises one of a pair of side fillets:

$F_{w,Rd} = tL_{w,s}(0{,}9 - 0{,}45L_{w,s}/b)f_u/\gamma_{M2}$ ... (8.4a)

- for an end fillet:

$F_{w,Rd} = tL_{w,e}(1 - 0{,}3L_{w,e}/b)f_u/\gamma_{M2}$ [for one weld] ... (8.4b)

where:

b	is	the width of the connected part or sheet, see figure 8.4;
$L_{w,e}$	is	the effective length of the end fillet weld, see figure 8.4;
$L_{w,s}$	is	the effective length of a side fillet weld, see figure 8.4.

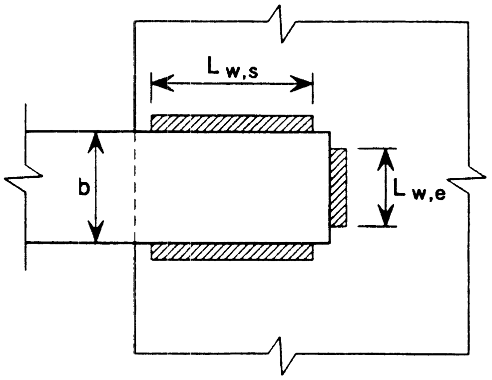


Figure 8.4: Fillet welded lap connection

(2)P If a combination of end fillets and side fillets is used in the same connection, its total resistance shall be taken as equal to the sum of the resistances of the end fillets and the side fillets.

(3)P The effective length L_w of a fillet weld shall be taken as the overall length of the full-size fillet, including end returns. Provided that the weld is full size throughout this length, no reduction in effective length need be made for either the start or termination of the weld.

(4) Fillet welds with effective lengths less than 8 times the thickness of the thinner connected part should not be designed to transmit any forces.

8.6.3 Arc spot welds

(1)P Arc spot welds shall not be designed to transmit any forces other than in shear.

(2)P Arc spot welds shall not be used through connected parts or sheets with a total thickness Σt of more than 4 mm, or where the thinnest connected part or sheet is more than 4 mm thick.

(3)P Arc spot welds shall have an effective diameter d_{eff} of not less than 10 mm.

(4)P If the connected part or sheet is less than 0,7 mm thick, a weld washer shall be used, see figure 8.5.

(5)P Arc spot welds shall have adequate end and edge distances.

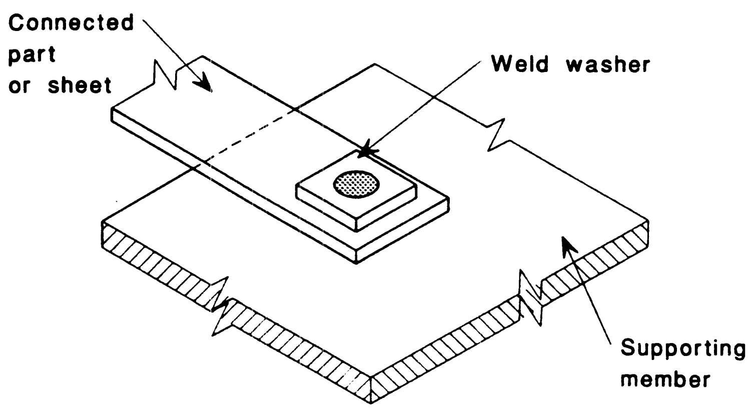


Figure 8.5: Arc spot weld with weld washer

(6) The design shear resistance $F_{w,Rd}$ of a circular arc spot weld should be determined as follows:

$$F_{w,Rd} = (\pi/4) d_s^2 \times 0,5 f_{uw} / \gamma_{M2} \qquad \dots (8.5a)$$

where:

f_{uw} is the ultimate tensile strength of the welding electrodes;

but $F_{w,Rd}$ should not be taken as more than the peripheral resistance given by the following:

- if $d_p / \Sigma t \leq 24\varepsilon$:

$$F_{w,Rd} = 1,33 d_p \Sigma t f_u / \gamma_{M2} \qquad \dots (8.5b)$$

- if $24\varepsilon < d_p/\Sigma t < 41,5\varepsilon$:

$$F_{w,Rd} = 0,17(d_p + 164\varepsilon\Sigma t)\Sigma t f_u / \gamma_{M2} \qquad \dots (8.5c)$$

- if $d_p/\Sigma t \geq 41,5\varepsilon$:

$$F_{w,Rd} = 0,84 d_p \Sigma t f_u / \gamma_{M2} \qquad \dots (8.5d)$$

(7) The interface diameter d_s of an arc spot weld, see figure 8.6, should be obtained from:

$$d_s = 0,7 d_w - 1,5 \Sigma t \qquad \dots (8.6)$$

where:

d_w is the visible diameter of the arc spot weld, see figure 8.6.

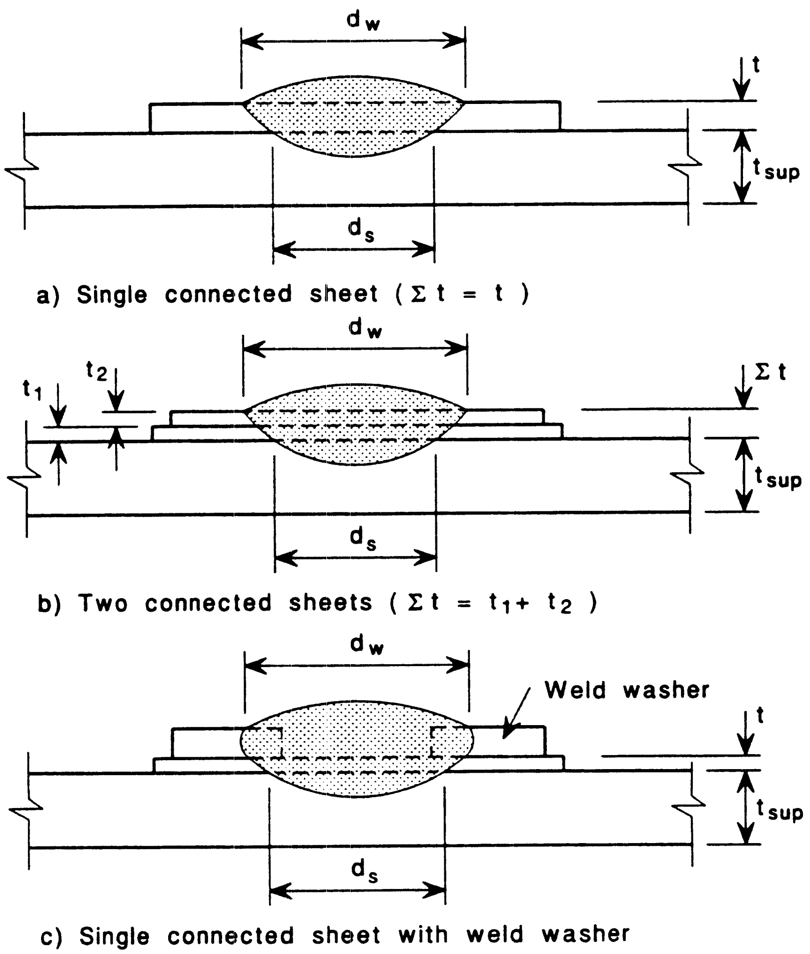


Figure 8.6: Arc spot welds

(8) The effective peripheral diameter d_p of an arc spot weld should be obtained as follows:

- for a single connected sheet or part of thickness t:

$$d_p = d_w - t \qquad \text{... (8.7a)}$$

- for multiple connected sheets or parts of total thickness Σt:

$$d_p = d_w - 2\Sigma t \qquad \text{... (8.7b)}$$

(9) The design shear resistance $F_{w,Rd}$ of an elongated arc spot weld should be determined from:

$$F_{w,Rd} = [(\pi/4)d_s^2 + L_w d_s] \times 0{,}5 f_{uw} / \gamma_{M2} \qquad \text{... (8.8a)}$$

but $F_{w,Rd}$ should not be taken as more than the peripheral resistance given by:

$$F_{w,Rd} = (0{,}4 L_w + 1{,}33 d_p) \Sigma t f_u / \gamma_{M2} \qquad \text{... (8.8b)}$$

where:

L_w is the length of the elongated arc spot weld, measured as shown in figure 8.7.

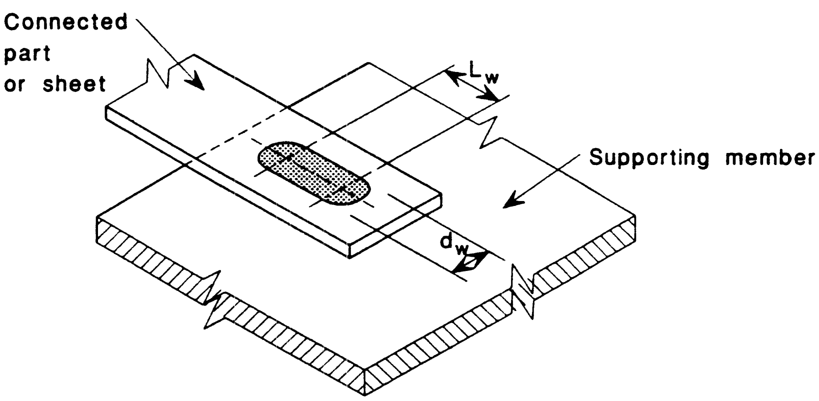


Figure 8.7: Elongated arc spot weld

9 Design assisted by testing

9.1 Basis

(1)P This Section 9 shall be used to apply the principles for design assisted by testing given in Section 8 of ENV 1993-1-1, to the specific requirements of cold formed thin gauge members and sheeting.

(2) Testing may be undertaken under any of the following circumstances:

a) if the properties of the steel are unknown;

b) if it is desired to take account of the actual properties of the cold formed member or sheet;

c) if adequate analytical procedures are not available for designing a component by calculation alone;

d) if realistic data for design cannot otherwise be obtained;

e) if it is desired to check the performance of an existing structure or structural component;

f) if it is desired to build a number of similar structures or components on the basis of a prototype;

g) if confirmation of consistency of production is required;

h) if it is desired to determine the effects of interaction with other structural components;

i) if it is desired to determine the effects of the lateral or torsional restraint supplied by other components;

j) if it is desired to prove the validity and adequacy of an analytical procedure;

k) if it is desired to produce resistance tables based on tests, or on a combination of testing and analysis;

l) if it is desired to take into account practical factors that might alter the performance of a structure, but are not addressed by the relevant analysis method for design by calculation.

(3) Testing as a basis for tables of load carrying capacity should be in accordance with 9.3.

NOTE: Information is given in annex A on procedures for:

- tests on profiled sheets and liner trays;
- tests on cold formed members;
- tests on structures and portions of structures;
- tests on beams torsionally restrained by sheeting;
- evaluation of test results to determine design values.

(4) Tensile testing of steel should be carried out in accordance with EN 10002-1. Testing of other steel properties should be carried out in accordance with the relevant European Standards.

(5) Testing of fasteners and connections should be carried out in accordance with the relevant European Standard or International Standard if there is one.

NOTE: Pending availability of an appropriate European or International Standard, guidance on testing procedures for fasteners can be obtained from:

ECCS Publication No. 21 (1983): *European recommendations for steel construction: the design and testing of connections in steel sheeting and sections*;

ECCS Publication No. 42 (1983): *European recommendations for steel construction: mechanical fasteners for use in steel sheeting and sections.*

9.2 Conditions

(1)P The planning, execution, evaluation and documentation of tests shall be in accordance with the minimum requirements specified in this Section 9.

(2)P The performance of experimental assessments shall be entrusted only to organisations where the staff is sufficiently knowledgeable and experienced in the planning, execution and evaluation of tests.

(3)P The testing laboratory shall be adequately equipped and the testing organisation shall ensure careful management and documentation of all tests.

(4)P The application of any test result shall be consistent with the particular conditions used in the test.

(5)P Tests shall simulate the behaviour of the member, sheeting or assembly under practical conditions, and the loading, support and constraint conditions used in the test shall model those that apply in practice.

NOTE: Good practice guides define conventional practical conditions for specific applications.

(6) The rate of load application should be such that the behaviour can be considered to be quasi-static.

(7) A comprehensive record of load-deformation behaviour should be made, containing an adequate number of readings for each variable monitored.

(8) Testing may be carried out using any of the following methods:

- incremental loading;
- continuously variable and continuously monitoring machines;
- a test rig with continuously variable applicators, such as air bags or vacuum boxes.

(9) For incremental loading, the increments should be determined from the expected load-deformation behaviour and their number should be sufficient to give a full record of the behaviour of the test specimen. The deformations at critical points should be measured at each increment of the loading.

(10) During load application, up to attainment of the service load, the load may be removed and then reapplied. For this purpose the service load may be taken as equal to the serviceability limit state design load for the characteristic (rare) combination as defined in ENV 1991-1. Above the service load, the loading should be held constant at each increment until any time-dependent deformations due to plastic behaviour have become negligible.

NOTE: In ENV 1993-1-1 the characteristic (rare) combination is termed the "rare combination".

(11)P A test report shall be prepared giving the following information:

a) a specification of the test;

b) a diagram indicating the geometry of the structure or component;

c) a diagram indicating the positions of the loading points and locations of the measuring devices;

d) details of the loading method and procedure;

e) the actual dimensional measurements of the structure or component;

f) the deflections and strains measured in the test, with the corresponding stage of loading or unloading;

g) a record of all other observations from the test.

(12) The test report should be supplemented by an evaluation of the resistance of the structure or component.

9.3 Load tables based on testing

9.3.1 General

(1) Load tables giving the load carrying capacity of specific structural components may be either based completely on the results of tests, or based on a combination of testing and rational analysis.

(2) Such load tables may represent the performance of a member when used within a specific structural system in which its behaviour is influenced by interaction with cladding and other structural components.

(3) If the performance of a system relies on the stabilizing effect of associated materials, such as sheeting on roof purlin systems, load tables based on testing should clearly state the necessary conditions of validity in terms of the associated materials and the methods of fixing them.

(4) In preparing load tables, account should be taken of the possibility that relevant serviceability limit state criteria, rather than the ultimate limit state design resistance, might govern the load carrying capacity.

(5) The tests should verify that, under the serviceability limit state characteristic (rare) combination, see ENV 1991-1, the member has no significant local deformation and no significant permanent deformation.

NOTE: In ENV 1993-1-1 the characteristic (rare) combination is termed the "rare combination".

9.3.2 Tables based completely on testing

(1) If the load tables are based completely on tests, these tests should adequately cover the whole range of geometries and loading conditions to be included in the load tables, and the support conditions and connections used in the tests should correspond with those stated in the load tables.

(2) Extrapolation should generally be avoided. However, limited extrapolation may be used where this can be justified on the basis of a specific and appropriate analysis of the test results, provided that it can be demonstrated that this extrapolation does not lead to conditions in which a different failure mode is likely.

S

9.3.3 Tables based on combined testing and analysis

(1) As an alternative to 9.3.2, load tables may be based on a rational analysis assisted by testing. The mathematical model of the resistance should take account of all failure modes that are possible within the range of the load tables. This mathematical model should be validated by testing.

(2) The validation of the mathematical model may be by means of full scale tests on a completely representative portion of a structure, comprising the structural components and connections, together with the associated materials and the methods of fixing them to be used in service.

(3) Alternatively the mathematical model may be validated by carrying out separate tests on all members, connections and other structural components to determine their strength and stiffness, and the rotational restraint given to members by the cladding. This analysis should also take account of all failure modes that are possible within the range of the load tables. If this is in doubt, sufficient full scale tests as described in (2) should be carried out to remove the doubt.

(4) In comparing the results of a test with those of the mathematical model, the actual thickness and yield strength of the critical component should be used.

(5)P Appropriate safety factors shall be applied. The mathematical model may be adjusted to achieve compliance with this requirement.

NOTE: Information on appropriate procedures is given in annex A.

10 Particular applications

10.1 Beams restrained by sheeting

10.1.1 General

(1) The provisions given in this clause 10.1 may be applied to purlins of Z, C, Σ or similar shaped cross-section with continuous full lateral restraint to one flange. The purlins may be designed by calculation, by testing in accordance with Section 9, or by a combination of calculation and testing.

(2) These provisions may also be applied to cold formed members used as side rails, floor beams and other similar types of beam that are similarly restrained by sheeting.

(3) Side rails may be designed on the basis that wind pressure has a similar effect on them to gravity loading on purlins, and that wind suction acts on them in a similar way to uplift loading on purlins.

(4) Full continuous lateral restraint may be supplied by trapezoidal steel sheeting or other profiled steel sheeting with sufficient stiffness, continuously connected to the top flange of the purlin through the troughs of the sheets. In other cases (for example, fastening through the crests of the sheets) the degree of restraint should either be validated by experience, or determined from tests in accordance with Section 9.

(5) Unless alternative support arrangements can be justified from the results of tests in accordance with Section 9, the purlin should have support details, such as cleats, that prevent rotation and lateral displacement at its supports. The effects of forces in the plane of the sheeting, that are transmitted to the supports of the purlin, should be taken into account in the design of the support details.

(6) The behaviour of a laterally restrained purlin should be modelled as outlined in figure 10.1. The connection of the purlin to the sheeting may be assumed to partially restrain the twisting of the purlin. This partial torsional restraint may be represented by a rotational spring with a spring stiffness C_D. The stresses in the free flange, not directly connected to the sheeting, should then be calculated by superposing the effects of in-plane bending and the effects of torsion, including lateral bending due to cross-sectional distortion.

(7) Where the free flange of a single span purlin is in compression under uplift loading, allowance should also be made for the amplification of the stresses due to torsion and distortion.

10.1.2 Calculation methods

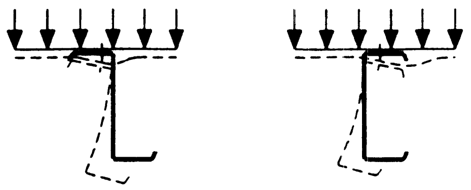
(1) Unless a second order analysis is carried out, the method given in 10.1.3 and 10.1.4 should be used to allow for the tendency of the free flange to move laterally (thus inducing additional stresses) by treating it as a beam subject to a lateral load $q_{h,Fd}$, see figure 10.1.

(2) For use in this method, the rotational spring should be replaced by an equivalent lateral linear spring of stiffness K. In determining K the effects of cross-sectional distortion should also be allowed for. For this purpose, the free flange may be treated as a compression member subject to a non-uniform axial force, with a continuous lateral spring support of stiffness K.

(3) If the free flange of a purlin is in compression due to in-plane bending (for example, due to uplift loading in a single span purlin), the resistance of the free flange to lateral buckling should also be verified.

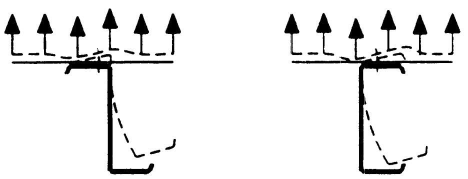
(4) For a more precise calculation, a second order analysis should be carried out, using values of the rotational spring stiffness C_D obtained from 10.1.5.2. Allowance should be made for the effects of an initial bow imperfection of $L/500$ in the free flange, where L is the span.

(5) A second order analysis using the rotational spring stiffness C_D obtained from 10.1.5.2 may also be used if lateral restraint is not supplied or if its effectiveness cannot be proved.



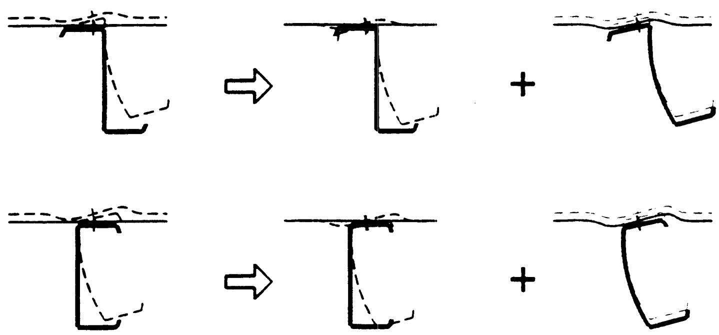
Z or C section purlin with its upper flange connected to sheeting:

- gravity loading.



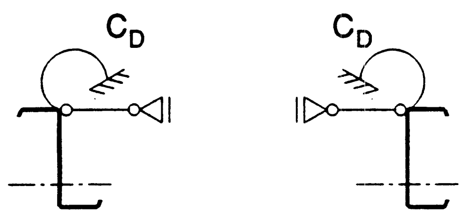
Z or C section purlin with its upper flange connected to sheeting:

- uplift loading.



Total deformation split into two parts:

- torsion and lateral bending;
- in-plane bending.

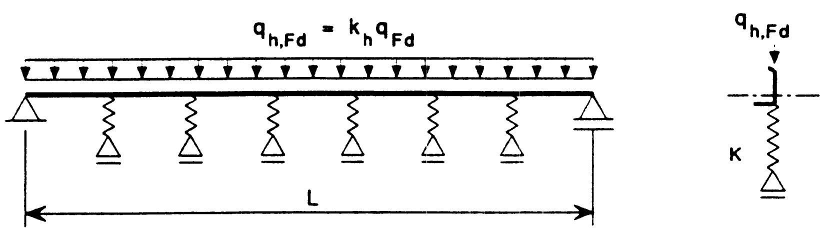


Model purlin as laterally braced with rotational spring restraint C_D from the sheeting.

As a simplification, replace the rotational spring C_D by a lateral spring of stiffness K.

Simplified calculation model used in 10.1.4.

Free flange of purlin modelled as beam on elastic foundation.



Model representing effects of torsion and lateral bending (including cross-section distortion) on single span purlin with uplift loading.

Figure 10.1: Modelling laterally braced purlins rotationally restrained by sheeting

10.1.3 Design criteria

10.1.3.1 Single span purlins

(1) For gravity loading, a single span purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1. If it is subject to axial compression, it should also satisfy the criteria for stability of the free flange given in 10.1.4.2.

(2) For uplift loading, a single span purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1 and the criteria for stability of the free flange given in 10.1.4.2.

10.1.3.2 Purlins continuous over two spans

(1) The moments due to gravity loading in a purlin that is physically continuous over two spans without overlaps or sleeves, may be either be obtained by calculation or based on the results of tests.

(2) If the moments are calculated they should be determined using elastic global analysis. The purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1. For the moment at the internal support, the criteria for stability of the free flange given in 10.1.4.2 should also be satisfied.

(3) Alternatively the moments may be determined using the results of tests in accordance with Section 9 on the moment-rotation behaviour of the purlin over the internal support.

NOTE: Appropriate testing procedures are given in annex A.

(4) The design value of the resistance moment at the supports $M_{\mathrm{sup,Rd}}$ for a given value of the load per unit length q_{Fd}, should be obtained from the intersection of two curves representing the design values of:

- the moment-rotation characteristic at the support, obtained by testing in accordance with Section 9;
- the theoretical relationship between the support moment $M_{\mathrm{sup,Sd}}$ and the corresponding plastic hinge rotation ϕ_{Ed} in the purlin over the support.

(5) The span moments should then be calculated from the value of the support moment.

(6) The following expressions may be used for a purlin with two equal spans:

$$\phi_{\mathrm{Ed}} = \frac{L}{12\,EI_{\mathrm{eff}}}\left[q_{\mathrm{Fd}}L^2 - 8M_{\mathrm{sup,Sd}}\right] \quad \ldots (10.1)$$

$$M_{\mathrm{spn,Sd}} = \frac{\left(q_{\mathrm{Fd}}L^2 - 2M_{\mathrm{sup,Rd}}\right)^2}{8q_{\mathrm{Fd}}L^2} \quad \ldots (10.2)$$

where:

I_{eff} is the effective second moment of area for the moment $M_{\mathrm{spn,Sd}}$;

L is the span;

$M_{\mathrm{spn,Sd}}$ is the maximum moment in the span.

(7) The expressions for a purlin with two unequal spans should be obtained by analysis.

(8) The maximum span moment $M_{\mathrm{spn,Sd}}$ in the purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1. Alternatively the resistance moment in the span may be determined by testing in accordance with Section 9, using single span tests with a span comparable to the distance between the points of contraflexure in the span.

10.1.3.3 Two-span continuous purlins with uplift loading

(1) The moments due to uplift loading in a purlin that is physically continuous over two spans without overlaps or sleeves, should be determined using elastic global analysis.

(2) The moment over the internal support should satisfy the criteria for cross-section resistance given in 10.1.4.1. Because the support reaction is a tensile force, no account need be taken of its interaction with the support moment.

(3) The moments in the spans should satisfy the criteria for stability of the free flange given in 10.1.4.2.

10.1.3.4 Purlins with continuity given by overlaps or sleeves

(1) The moments in purlins in which continuity over two or more spans is given by overlaps or sleeves at internal supports, should be determined taking into account the effective section properties of the cross-section and the effects of the overlaps or sleeves.

(2) Tests in accordance with Section 9 should be carried out on the support details to determine:

- the flexural stiffness of the overlapped or sleeved part;
- the moment-rotation characteristic for the overlapped or sleeved part;
- the resistance of the overlapped or sleeved part to combined support reaction and moment;
- the resistance of the non-overlapped unsleeved part to combined shear force and bending moment.

(3) For gravity loading, the purlin should satisfy the following criteria:

- at internal supports, the resistance to combined support reaction and moment determined by testing;
- near supports, the resistance to combined shear force and bending moment determined by testing;
- in the spans, the criteria for cross-section resistance given in 10.1.4.1;
- if the purlin is subject to axial compression, the criteria for stability of the free flange given in 10.1.4.2.

(4) For uplift loading, the purlin should satisfy the following criteria:

- at internal supports, the resistance to combined support reaction and moment determined by testing, taking into account the fact that the support reaction is a tensile force in this case;
- near supports, the resistance to combined shear force and bending moment determined by testing;
- in the spans, the criteria for stability of the free flange given in 10.1.4.2.

10.1.3.5 Serviceability criteria

(1) The serviceability criteria relevant to purlins given in Section 7 should also be satisfied.

10.1.4 Design resistance

10.1.4.1 Resistance of cross-sections

(1) For a purlin subject to axial force and transverse load the resistance of the cross-section should be verified as indicated in figure 10.2 by superposing the stresses due to:

- the in-plane bending moment $M_{y,Sd}$;
- the axial force N_{Sd};
- a lateral load $q_{h,Fd}$ acting on the free flange, due to torsion and lateral bending, see (3).

(2) The maximum stresses in the cross-section should satisfy the following:

- restrained flange:

$$\sigma_{max,Ed} = \frac{M_{y,Sd}}{W_{eff,y}} + \frac{N_{Sd}}{A_{eff}} \leq f_y/\gamma_M \qquad \text{... (10.3a)}$$

- free flange:

$$\sigma_{max,Ed} = \frac{M_{y,Sd}}{W_{eff,y}} + \frac{N_{Sd}}{A_{eff}} + \frac{M_{fz,Sd}}{W_{fz}} \leq f_y/\gamma_M \qquad \text{... (10.3b)}$$

where:

A_{eff} is the effective area of the cross-section for uniform compression;

f_y is the yield strength as defined in 3.1.1(6)P;

$M_{fz,Sd}$ is the bending moment in the free flange due to the lateral load $q_{h,Fd}$;

$W_{eff,y}$ is the effective section modulus of the cross-section for bending about the y - y axis;

W_{fz} is the gross elastic section modulus of the free flange plus 1/6 of the web height, for bending about the z - z axis;

and $\gamma_M = \gamma_{M0}$ if $A_{eff} = A_{el}$ or if $W_{eff,y} = W_{el,y}$ and $N_{Sd} = 0$, otherwise $\gamma_M = \gamma_{M1}$.

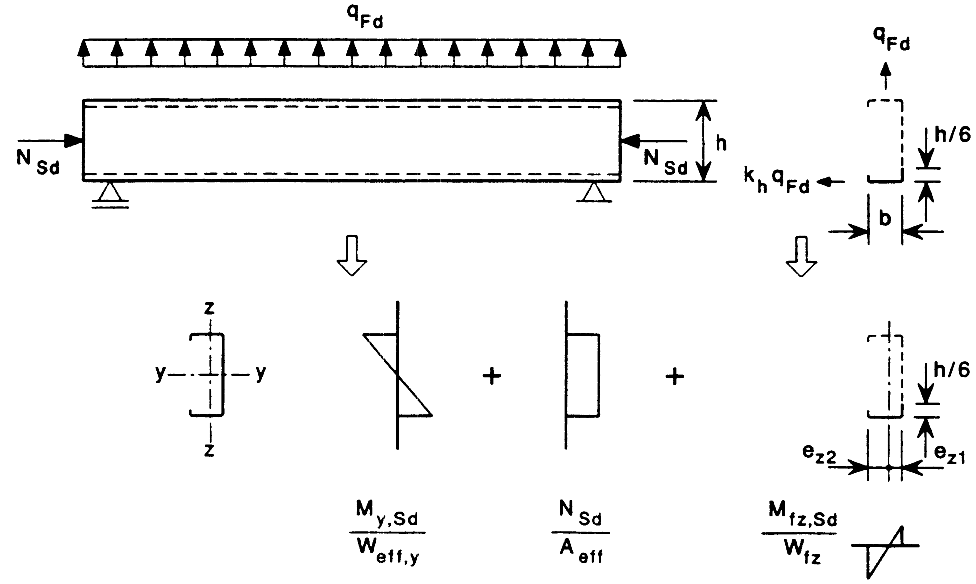


Figure 10.2: Superposition of stresses

(3) The lateral load $q_{h,Fd}$ acting on the free flange, due to torsion and lateral bending, should be obtained from:

$$q_{h,Fd} = k_h q_{Fd} \qquad \dots (10.4)$$

(4) The coefficient k_h should be obtained as indicated in figure 10.3 for common types of cross-section.

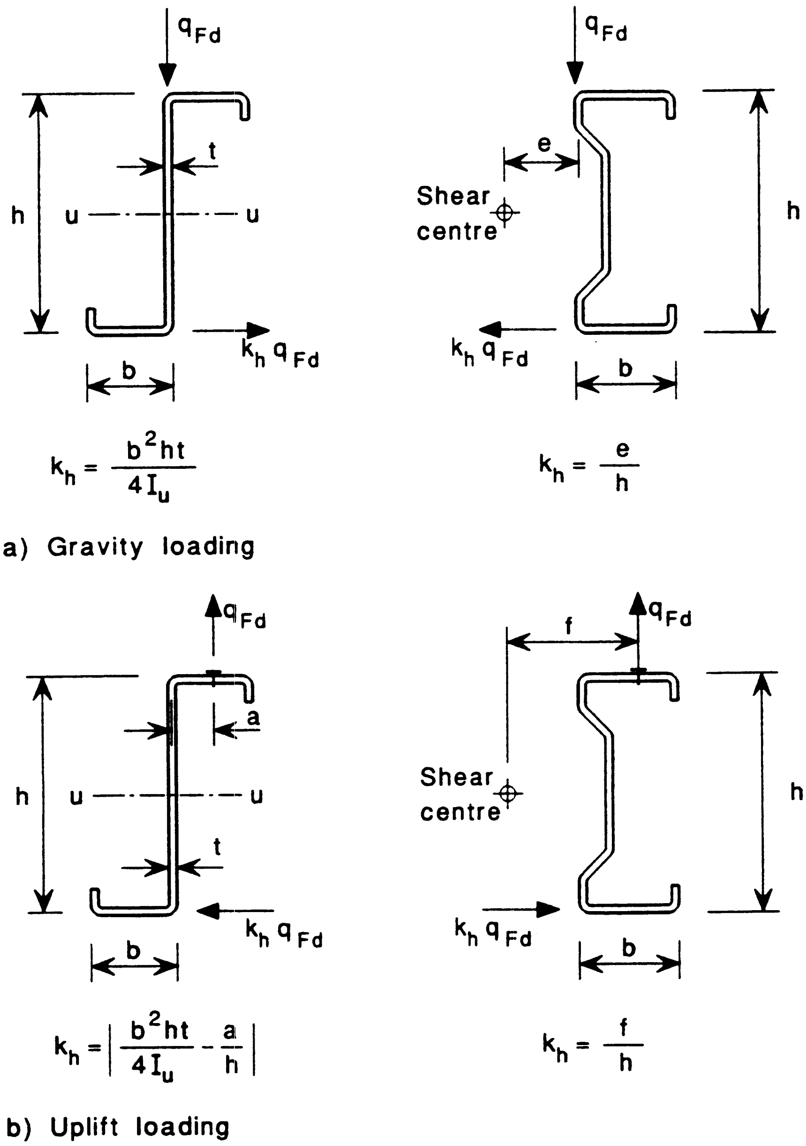


Figure 10.3: Conversion of torsion into lateral bending of the free flange

(5) The lateral bending moment $M_{fz,Sd}$ should be taken as equal to zero if the free flange is in tension, otherwise $M_{fz,Sd}$ should be determined from:

$$M_{fz,Sd} = \beta_R M_{0,fz,Sd} \qquad \dots (10.5)$$

where:

$M_{0,fz,Sd}$ is the initial lateral bending moment in the free flange without any spring support;

β_R is a correction factor for the effective spring support.

(6) The initial lateral bending moment in the free flange $M_{0,fz,Sd}$ should be determined from table 10.1 for the critical locations in the span, at supports, at anti-sag bars and between anti-sag bars.

(7) The correction factor β_R for the relevant location and boundary conditions, should be determined from table 10.1, using the value of the coefficient R of the spring support given by:

$$R = \frac{K L_a^4}{\pi^4 E I_{fz}} \qquad \dots (10.6)$$

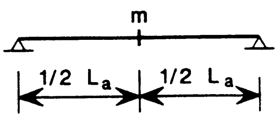
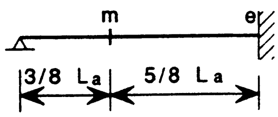
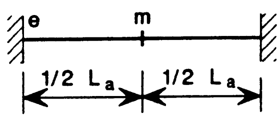
where:

I_{fz} is the second moment of area of the gross cross-section of the free flange plus 1/6 of the web height, for bending about the z - z axis;

K is the lateral spring stiffness per unit length from 10.1.5.1;

L_a is the distance between anti-sag bars, or if none are present, the span L of the purlin.

Table 10.1: Values of initial moment $M_{0,fz,Sd}$ and correction factor β_R

System	Location	$M_{0,fz,Sd}$	β_R
	m	$\frac{1}{8} q_{h,Fd} L_a^2$	$\beta_R = \frac{1 - 0{,}0225R}{1 + 1{,}013R}$
	m	$\frac{9}{128} q_{h,Fd} L_a^2$	$\beta_R = \frac{1 - 0{,}0141R}{1 + 0{,}416R}$
	e	$-\frac{1}{8} q_{h,Fd} L_a^2$	$\beta_R = \frac{1 + 0{,}0314R}{1 + 0{,}396R}$
	m	$\frac{1}{24} q_{h,Fd} L_a^2$	$\beta_R = \frac{1 - 0{,}0125R}{1 + 0{,}198R}$
	e	$-\frac{1}{12} q_{h,Fd} L_a^2$	$\beta_R = \frac{1 + 0{,}0178R}{1 + 0{,}191R}$

10.1.4.2 Buckling resistance of free flange

(1) If the free flange is in compression, its buckling resistance should be verified using:

$$\frac{1}{\chi}\left[\frac{M_{y,Sd}}{W_{eff,y}} + \frac{N_{Sd}}{A_{eff}}\right] + \frac{M_{fz,Sd}}{W_{fz}} \leq f_{yb}/\gamma_{M1} \qquad \dots (10.7)$$

in which χ is the reduction factor for flexural buckling of the free flange, obtained from 6.2.1(2)P using buckling curve a (imperfection factor $\alpha = 0{,}21$) for the relative slenderness λ_{fz} given in (2).

(2) The relative slenderness $\bar{\lambda}_{fz}$ for flexural buckling of the free flange should be determined from:

$$\bar{\lambda}_{fz} = \frac{\ell_{fz}/i_{fz}}{\lambda_1} \qquad \text{... (10.8)}$$

with:

$$\lambda_1 = \pi\left[E/f_{yb}\right]^{0,5}$$

where:

- ℓ_{fz} is the buckling length for the free flange from (3) to (7);
- i_{fz} is the radius of gyration of the gross cross-section of the free flange plus 1/6 of the web height, about the z - z axis.

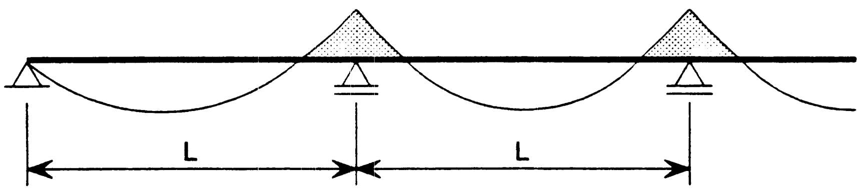
(3) For gravity loading, provided that $0 \leq R \leq 200$, the buckling length of the free flange for a variation of the compressive stress over the length L as shown in figure 10.4 may be obtained from:

$$\ell_{fz} = \eta_1 L_a\left(1 + \eta_2 R^{\eta_3}\right)^{\eta_4} \qquad \text{... (10.9)}$$

where:

- L_a is the distance between anti-sag bars, or if none are present, the span L of the purlin;
- R is as given in 10.1.4.1(7);

and η_1 to η_4 are coefficients that depend on the number of anti-sag bars, as given in table 10.2.



[Dotted areas show regions in compression]

Figure 10.4: Varying compressive stress in free flange for gravity load cases

Table 10.2: Coefficients η_i

Number of anti-sag bars per span	η_1	η_2	η_3	η_4
0	0,526	22,8	2,12	- 0,108
1	0,622	66,7	2,68	- 0,084
2 or 3	0,713	62,7	2,75	- 0,084
more than 3	1,000	30,4	2,28	- 0,108

(4) For gravity loading, if there are more than three equally spaced anti-sag bars, the buckling length need not be taken as greater than the value for two anti-sag bars, with $L_a = L/3$.

(5) If the compressive stress over the length L is almost constant, due to the application of a relatively large axial force, the buckling length should be determined using the values of η_i from table 10.2 for the case shown as more than three anti-sag bars per span, but the actual spacing L_a.

(6) For uplift loading, provided that $0 \leq R_0 \leq 200$, the buckling length of the free flange for variations of the compressive stress over the length L_0 as shown in figure 10.5, may be obtained from:

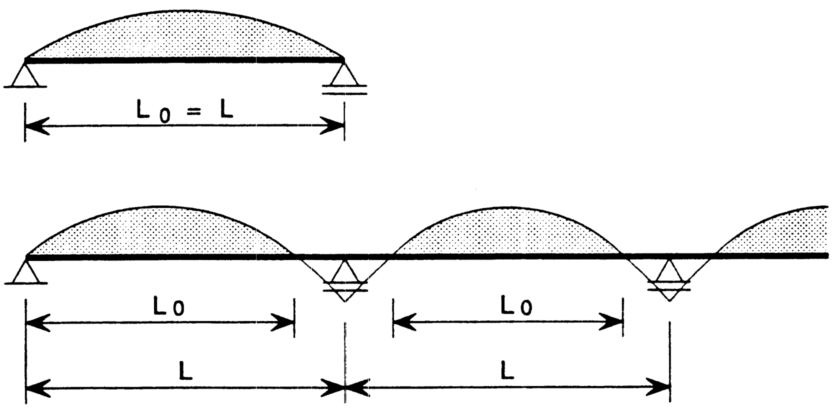
$$\ell_{fz} = 0{,}7L_0(1 + 13{,}1 R_0^{1,6})^{-0,125} \quad \text{... (10.10a)}$$

with:

$$R_0 = \frac{K L_0^4}{\pi^4 E I_{fz}} \quad \text{... (10.10b)}$$

in which I_{fz} and K are as defined in 10.1.4.1(7).

(7) For uplift loading, if the free flange is effectively held in position laterally at intervals by anti-sag bars, the buckling length may conservatively be taken as that for a uniform moment, determined as in (4).



[Dotted areas show regions in compression]

Figure 10.5: Varying compressive stress in free flange for uplift cases

10.1.5 Rotational restraint given by the sheeting

10.1.5.1 Lateral spring stiffness

(1) The lateral spring support given to the free flange of the purlin by the sheeting should be modelled as a lateral spring acting at the free flange, see figure 10.1. The total lateral spring stiffness K per unit length should be determined from:

$$\frac{1}{K} = \frac{1}{K_A} + \frac{1}{K_B} + \frac{1}{K_C} \quad \text{... (10.11)}$$

where:

K_A is the lateral stiffness corresponding to the rotational stiffness of the connection between the sheeting and the purlin;

K_B is the lateral stiffness due to distortion of the cross-section of the purlin;

K_C is the lateral stiffness due to the flexural stiffness of the sheeting.

(2) Normally it may be assumed to be safe as well as realistic to neglect $1/K_C$ because K_C is very large compared to K_A and K_B. The value of K should then be obtained from:

$$K = \frac{1}{\left(1/K_A + 1/K_B\right)} \quad \text{... (10.12)}$$

(3) The value of $\left(1/K_A + 1/K_B\right)$ may be obtained either by testing in accordance with Section 9, or by calculation.

NOTE: Appropriate testing procedures are given in annex A.

(4) The lateral spring stiffness K per unit length may be determined by calculation using:

$$\frac{1}{K} = \frac{4(1-\nu^2)\,h^2\,(h_d + e)}{E\,t^3} + \frac{h^2}{C_D} \quad \text{... (10.13)}$$

in which the dimension e is determined as follows:

- for cases bringing the purlin into contact with the sheeting at the purlin web:

  $e = a$

- for cases bringing the purlin into contact with the sheeting at the tip of the purlin flange:

  $e = 2a + b$

where:

a	is	the distance from the sheet-to-purlin fastener to the purlin web, see figure 10.6;
b	is	the width of the purlin flange connected to the sheeting, see figure 10.6;
C_D	is	the total rotational spring stiffness from 10.1.5.2;
h	is	the overall height of the purlin;
h_d	is	the developed height of the purlin web, see figure 10.6.

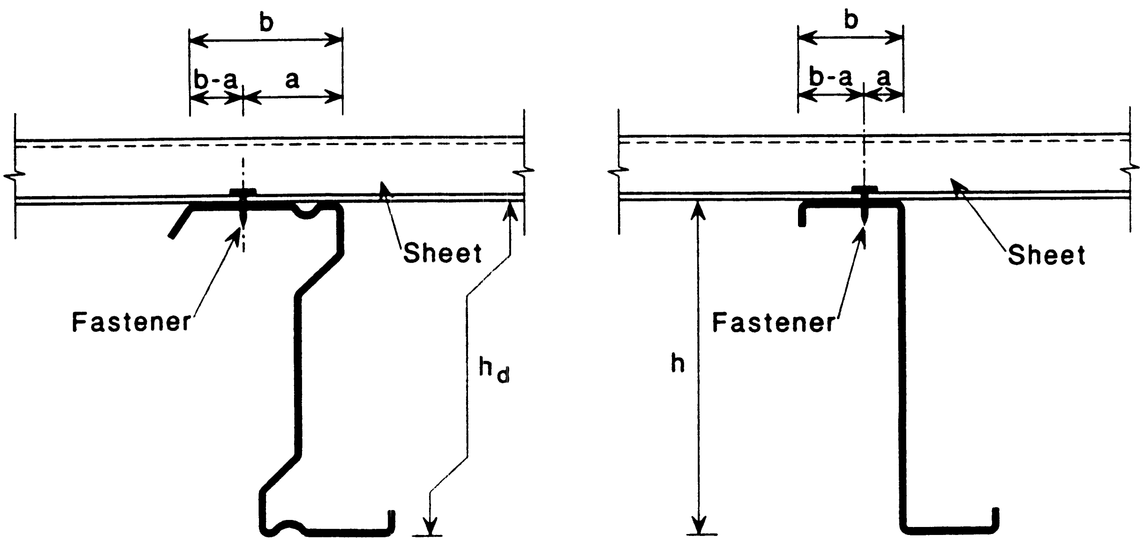


Figure 10.6: Purlin and attached sheeting

10.1.5.2 Rotational spring stiffness

(1) The rotational restraint given to the purlin by the sheeting that is connected to its top flange, should be modelled as a rotational spring acting at the top flange of the purlin, see figure 10.1. The total rotational spring stiffness C_D should be determined from:

$$C_D = \frac{1}{(1/C_{D,A} + 1/C_{D,C})} \quad \text{... (10.14)}$$

where:

$C_{D,A}$ is the rotational stiffness of the connection between the sheeting and the purlin;

$C_{D,C}$ is the rotational stiffness corresponding to the flexural stiffness of the sheeting.

(2) Generally $C_{D,A}$ may be calculated as given in (5) and (7). Alternatively $C_{D,A}$ may be obtained by testing, see (9).

(3) The value of $C_{D,C}$ may be taken as the minimum value obtained from calculational models of the type shown in figure 10.7, taking account of the rotations of the adjacent purlins and the degree of continuity of the sheeting, using:

$$C_{D,C} = m / \theta \quad \text{... (10.15)}$$

where:

I_{eff} is the effective second moment of area per unit width of the sheeting;

m is the applied moment per unit width of sheeting, applied as indicated in figure 10.7;

θ is the resulting rotation, measured as indicated in figure 10.7 [radians].

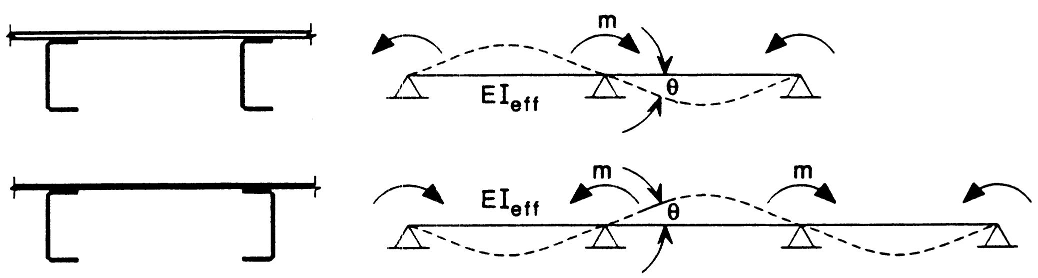


Figure 10.7: Model for calculating $C_{D,C}$

(4) Alternatively a conservative value of $C_{D,C}$ may be obtained from:

$$C_{D,C} = \frac{k E I_{eff}}{s} \quad \text{... (10.16)}$$

in which k is a numerical coefficient, with values as follows:

- for single span sheets: $k = 2$
- for sheets that are continuous over two or more spans: $k = 4$

where:

s is the spacing of the purlins.

(5) Provided that the sheet-to-purlin fasteners are positioned centrally on the flange of the purlin, the value of $C_{D,A}$ for trapezoidal sheeting connected to the top flange of the purlin may be determined as follows:

- if $b_a \leq 125$:

$$C_{D,A} = C_{100} \left[\frac{b_a}{100} \right]^2 \quad \text{... (10.17a)}$$

- if $125 \leq b_a \leq 200$:

$$C_{D,A} = 1{,}25\, C_{100} \left[\frac{b_a}{100} \right] \quad \text{... (10.17b)}$$

where:

b_a is the width of the purlin flange [in mm];

C_{100} is a rotation coefficient, representing the value of $C_{D,A}$ if $b_a = 100$ mm.

(6) Provided that there is no insulation between the sheeting and the purlins, the value of the rotation coefficient C_{100} may be obtained from table 10.3.

(7) Alternatively $C_{D,A}$ may be taken as equal to $130p$ [Nm/m/radian], where p is the number of sheet-to-purlin fasteners per metre length of purlin (but not more than one per rib of sheeting), provided that:

- the flange width b of the sheeting through which it is fastened does not exceed 120 mm;
- the nominal core thickness t of the sheeting is at least 0,66 mm;
- the distance a or $b - a$ between the centreline of the fastener and the centre of rotation of the purlin (depending on the direction of rotation), as shown in figure 10.6, is at least 25 mm.

(8) If the effects of cross-section distortion (which is not included in the rotational spring stiffness C_D) have to be taken into account, see 10.1.5.1, it may be assumed to be realistic to neglect $C_{D,C}$, because the spring stiffness is mainly influenced by the value of $C_{D,A}$ and the cross-section distortion.

(9) Alternatively, values of $C_{D,A}$ may be obtained from a combination of testing and calculation.

(10) If the value of $(1/K_A + 1/K_B)$ is obtained by testing, the values of $C_{D,A}$ for gravity loading and for uplift loading should be determined from:

$$C_{D,A} = \frac{h^2}{(1/K_A + 1/K_B) - 4(1 - \nu^2)\, h^2 (h_d + e)/E t^3} \quad \text{... (10.18)}$$

in which e, h and h_d are as defined in 10.1.5.1(4).

Table 10.3: Rotation coefficient C_{100} for trapezoidal steel sheeting

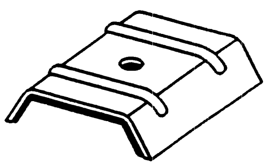
Positioning of sheeting		Sheet fastened through		Pitch of fasteners		Washer diameter	C_{100}	$b_{T,max}$
Positive	Negative	Trough	Crest	$e = b_R$	$e = 2b_R$	[mm]	[kNm/m]	[mm]
For gravity loading:								
×		×		×		22	5,2	40
×		×			×	22	3,1	40
	×		×	×		K_a	10,0	40
	×		×		×	K_a	5,2	40
	×	×		×		22	3,1	120
	×	×			×	22	2,0	120
For uplift loading:								
×		×		×		16	2,6	40
×		×			×	16	1,7	40

Key:

b_R is the corrugation width [185 mm maximum];

b_T is the width of the sheeting flange through which it is fastened to the purlin.

K_a indicates a steel saddle washer as shown below with $t \geq 0,75$ mm

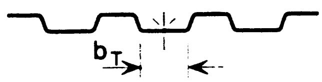


The values in this table are valid for:

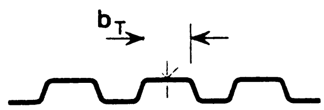
- sheet fastener screws of diameter: $\phi = 6,3$ mm;
- steel washers of thickness: $t_w \geq 1,0$ mm;
- sheeting of nominal core thickness: $t \geq 0,66$ mm.

Sheet fastened:

- through the trough:



- through the crest:



10.2 Liner trays restrained by sheeting

10.2.1 General

(1) Liner trays should be large channel-type sections, with two narrow flanges, two webs and one wide flange, generally as shown in figure 10.8. The two narrow flanges should be laterally restrained by attached profiled steel sheeting.

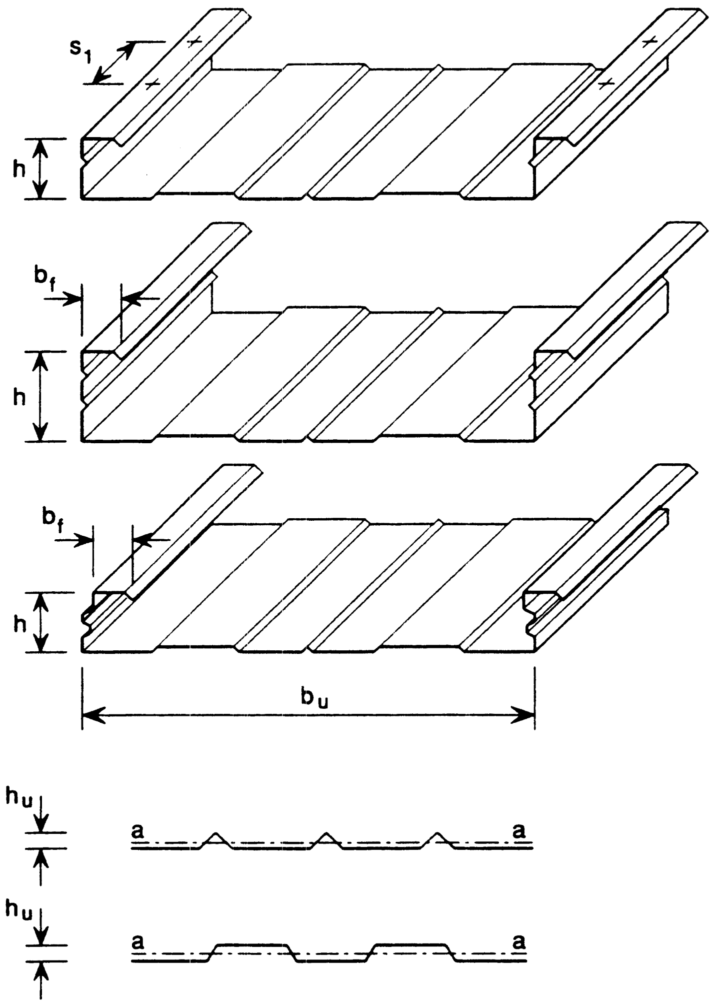


Figure 10.8: Typical geometry for liner trays

(2) The resistance of the webs of liner trays to shear forces and to local transverse forces should be obtained using 5.8 to 5.11, but using the value of $M_{c,Rd}$ given by (3) or (4).

(3) The moment resistance $M_{c,Rd}$ of a liner tray may be obtained using 10.2.2 provided that:

- the geometrical properties are within the range given in table 10.4;
- the depth h_u of the corrugations of the wide flange does not exceed $h/8$, where h is the overall depth of the liner tray.

(4) Alternatively the moment resistance of a liner tray may be determined by testing in accordance with Section 9, provided that it is ensured that the local behaviour of the liner tray is not affected by the testing equipment.

NOTE: Appropriate testing procedures are given in annex A.

Table 10.4: Range of validity of 10.2.2

0,75 mm	$\leq$	t_{nom}	$\leq$	1,5 mm
30 mm	$\leq$	b_f	$\leq$	60 mm
60 mm	$\leq$	h	$\leq$	200 mm
300 mm	$\leq$	b_u	$\leq$	600 mm
		I_a / b_u	$\leq$	10 mm^4 / mm
		s_1	$\leq$	1000 mm

10.2.2 Moment resistance

10.2.2.1 Wide flange in compression

(1) The moment resistance of a liner tray with its wide flange in compression should be determined using the step-by-step procedure outlined in figure 10.9 as follows:

- **Step 1:** Determine the effective areas of all compression elements of the cross-section, based on values of the stress ratio $\psi = \sigma_2 / \sigma_1$ obtained using the effective widths of the compression flanges but the gross areas of the webs;

- **Step 2:** Find the centroid of the effective cross-section, then obtain the moment resistance $M_{c,Rd}$ from:

$$M_{c,Rd} = W_{eff,min} f_{yb} / \gamma_{M2} \qquad \text{... (10.19)}$$

with:

$$W_{eff,min} = I_{y,eff} / z_c \quad \text{but} \quad W_{eff,min} \leq I_{y,eff} / z_t$$

$$\gamma_{M2} = \boxed{1,25}$$

where z_c and z_t are as indicated in figure 10.9.

NOTE: The use of γ_{M2} in this expression rather than γ_{M0} is necessary for calibration.

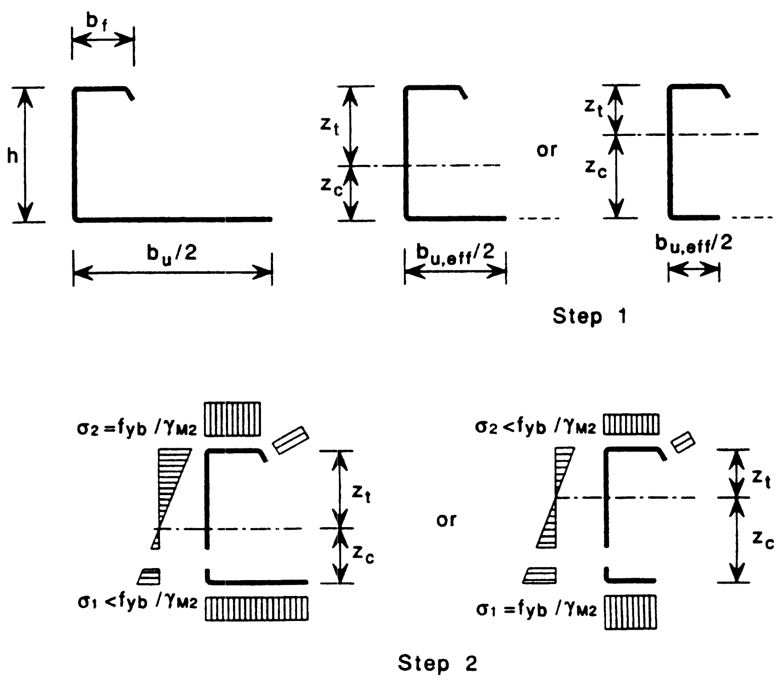


Figure 10.9: Determination of moment resistance — wide flange in compression

10.2.2.2 Wide flange in tension

(1) The moment resistance of a liner tray with its wide flange in tension should be determined using the step-by-step procedure outlined in figure 10.10 as follows:

- **Step 1:** Locate the centroid of the gross cross-section;
- **Step 2:** Obtain the effective width of the wide flange $b_{u,eff}$, allowing for possible flange curling, from:

$$b_{u,eff} = \frac{53{,}3 \times 10^{10}\, e_o^2\, t^3\, t_{eq}}{h\, L\, b_u^3} \qquad \text{... (10.20)}$$

where:

b_u is the overall width of the wide flange;

e_o is the distance from the centroidal axis of the gross cross-section to the centroidal axis of the narrow flanges;

h is the overall depth of the liner tray;

L is the span of the liner tray;

t_{eq} is the equivalent thickness of the wide flange, given by:

$$t_{eq} = (12\, I_a / b_u)^{1/3}$$

I_a is the second moment of area of the wide flange, about its own centroid.

- **Step 3:** Determine the effective areas of all the compression elements, based on values of the stress ratio $\psi = \sigma_2 / \sigma_1$ obtained using the effective widths of the flanges but the gross areas of the webs;

- **Step 4:** Find the centroid of the effective cross-section, then obtain the buckling resistance moment $M_{b,Rd}$ using:

$$M_{b,Rd} = \beta_b\, W_{eff,com} f_{yb} / \gamma_{M2} \quad \text{but} \quad M_{b,Rd} \leq W_{eff,t} f_{yb} / \gamma_{M2} \qquad \text{... (10.21)}$$

with:

$$W_{eff,com} = I_{y,eff} / z_c$$

$$W_{eff,t} = I_{y,eff} / z_t$$

in which the correlation factor β_b is given by the following:

- if $s_1 \leq 300\,\text{mm}$:

$$\beta_b = 1{,}0$$

- if $300\,\text{mm} \leq s_1 \leq 1000\,\text{mm}$:

$$\beta_b = 1{,}15 - s_1 / 2000$$

where:

s_1 is the longitudinal spacing of fasteners supplying lateral restraint to the narrow flanges, see figure 10.8;

$\gamma_{M2} = \boxed{1{,}25}$

NOTE: The use of γ_{M2} in this expression rather than γ_{M0} and γ_{M1} is necessary for calibration.

(2) The effects of shear lag need not be considered if $L / b_{u,eff} \leq 20$. Otherwise a reduced value of ρ should be determined as specified in 5.4.3.

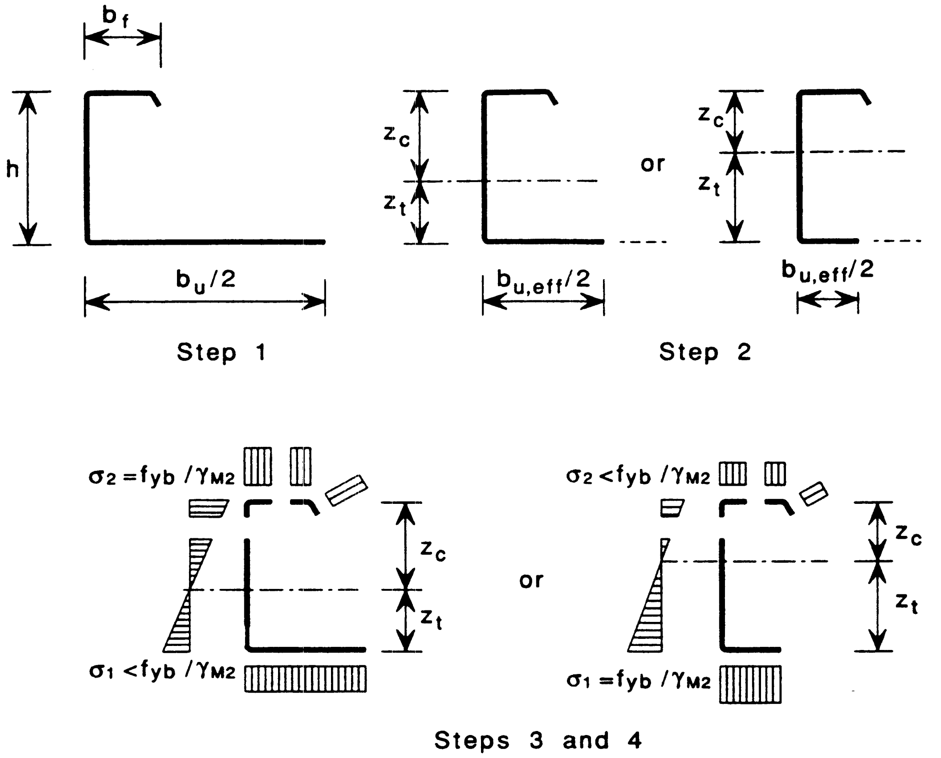


Figure 10.10: Determination of moment resistance — wide flange in tension

(3) Flange curling need not be taken into account in determining deflections at serviceability limit states.

(4) As a simplified alternative, the moment resistance of a liner tray with an unstiffened wide flange may be approximated by taking the same effective area for the wide flange in tension as for the two narrow flanges in compression combined.

10.3 Stressed skin design

10.3.1 General

(1)P The interaction between structural members and sheeting panels that are designed to act together as parts of a combined structural system, may be allowed for as described in this clause 10.3.

(2)P The provisions given in this clause shall be applied only to sheet diaphragms that are made of steel.

(3)P Diaphragms may be formed from profiled sheeting used as roof or wall cladding or for floors. They may also be formed from wall or roof structures based upon liner trays.

NOTE: Guidance on the verification of such diaphragms can be obtained from:

ECCS Publication No. 88 (1995): *European recommendations for the application of metal sheeting acting as a diaphragm.*

10.3.2 Diaphragm action

(1)P In stressed skin design, advantage may be taken of the contribution that diaphragms of sheeting used as roofing, flooring or wall cladding make to the overall stiffness and strength of the structural frame, by means of their stiffness and strength in shear.

(2) Roofs and floors may be treated as deep plate girders extending throughout the length of a building, resisting transverse in-plane loads and transmitting them to end gables, or to intermediate stiffened frames. The panel of sheeting may be treated as a web that resists in-plane transverse loads in shear, with the edge members acting as flanges that resist axial tension and compression forces, see figures 10.11 and 10.12.

(3) Similarly, rectangular wall panels may be treated as bracing systems that act as shear diaphragms to resist in-plane forces.

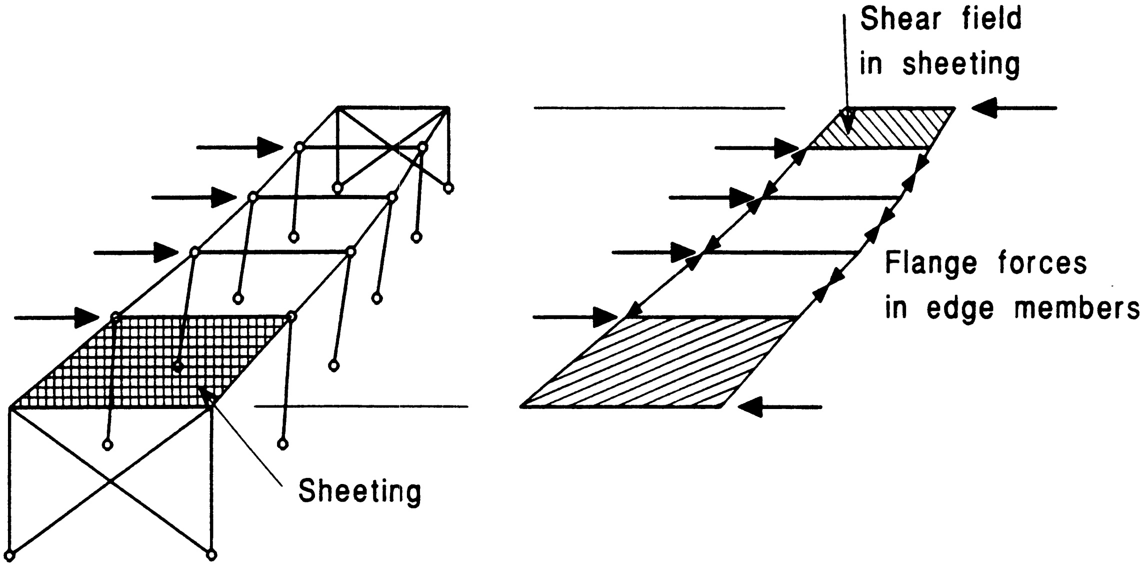


Figure 10.11: Stressed skin action in a flat-roof building

10.3.3 Necessary conditions

(1)P Methods of stressed skin design that utilize sheeting as an integral part of a structure, may be used only under the following conditions:

- the use made of the sheeting, in addition to its primary purpose, is limited to the formation of shear diaphragms to resist structural displacement in the plane of that sheeting;
- the diaphragms have longitudinal edge members to carry flange forces arising from diaphragm action;
- the diaphragm forces in the plane of a roof or floor are transmitted to the foundations by means of braced frames, further stressed-skin diaphragms, or other methods of sway resistance;
- suitable structural connections are used to transmit diaphragm forces to the main steel framework and to join the edge members acting as flanges;
- the sheeting is treated as a structural component that cannot be removed without proper consideration;
- the project specification, including the calculations and drawings, draws attention to the fact that the building is designed to utilize stressed skin action.

(2)P Stressed skin design shall be used predominantly in low-rise buildings, or in the floors and facades of high-rise buildings.

(3)P Stressed skin diaphragms shall be used predominantly to resist wind loads, snow loads and other loads that are applied through the sheeting itself. They may also be used to resist small transient loads, such as surge from light overhead cranes or hoists on runway beams, but may not be used to resist permanent external loads, such as those from plant.

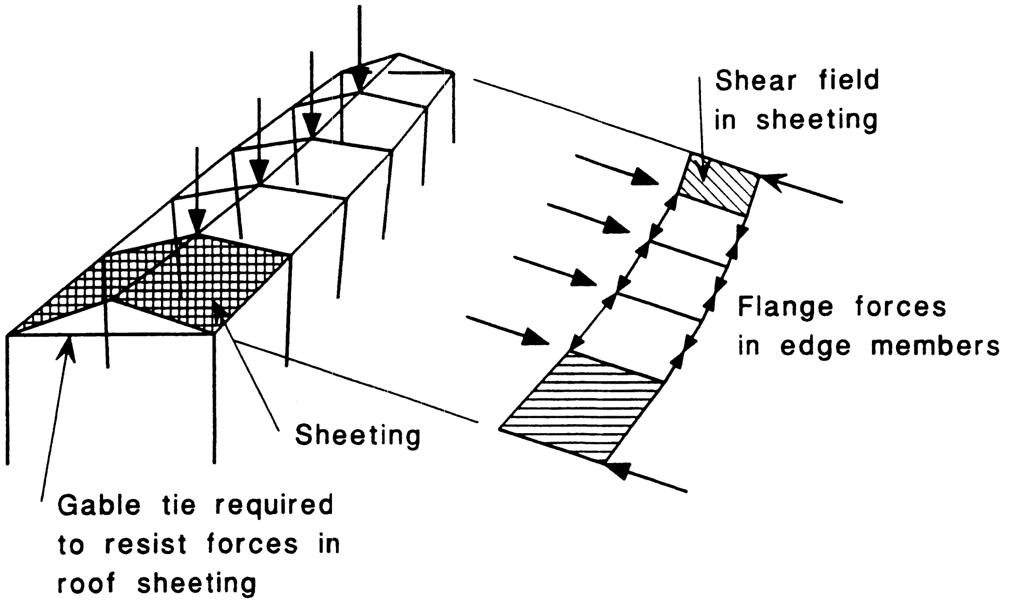


Figure 10.12: Stressed skin action in a pitched roof building

10.3.4 Profiled steel sheet diaphragms

(1)P In a profiled steel sheet diaphragm, see figure 10.13, both ends of the sheets shall be attached to the supporting members by means of self-tapping screws, cartridge fired pins, welding, bolts or other fasteners of a type that will not work loose in service, pull out, or fail in shear before causing tearing of the sheeting. All such fasteners shall be fixed directly through the sheeting into the supporting member, for example through the troughs of profiled sheets, unless special measures are taken to ensure that the connections effectively transmit the forces assumed in the design.

(2)P The seams between adjacent sheets shall be fastened by rivets, self-drilling screws, welds, or other fasteners of a type that will not work loose in service, pull out, or fail in shear before causing tearing of the sheeting. The spacing of such fasteners shall not exceed 500 mm.

(3)P The distances from all fasteners to the edges and ends of the sheets shall be adequate to prevent premature tearing of the sheets.

(4) Small randomly arranged openings, up to 3% of the relevant area, may be introduced without special calculation, provided that the total number of fasteners is not reduced. Openings up to 15% of the relevant area may be introduced if justified by detailed calculations. Areas that contain larger openings should be split into smaller areas, each with full diaphragm action.

(5)P All sheeting that also forms part of a stressed-skin diaphragm shall first be designed for its primary purpose in bending. To ensure that any deterioration of the sheeting would be apparent in bending before the resistance to stressed skin action is affected, it shall then be verified that the shear stress due to diaphragm action does not exceed $0{,}25 f_{yb}/\gamma_{M1}$.

(6)P The shear resistance of a stressed-skin diaphragm shall be based on the least tearing strength of the seam fasteners or the sheet-to-member fasteners parallel to the corrugations or, for diaphragms fastened only to longitudinal edge members, the end sheet-to-member fasteners. The calculated shear resistance for any other type of failure shall exceed this minimum value by at least the following:

- for failure of the sheet-to-purlin fasteners under combined shear and wind uplift, by at least 40%;
- for any other type of failure, by at least 25%.

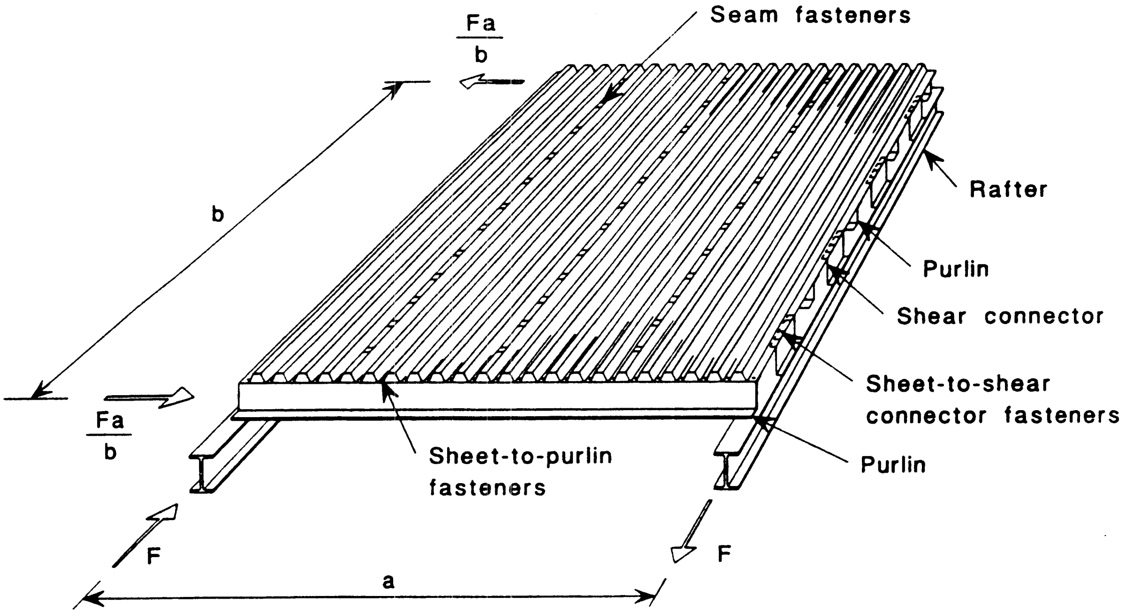


Figure 10.13: Arrangement of an individual panel

10.3.5 Steel liner tray diaphragms

(1) Liner trays used to form shear diaphragms should have stiffened wide flanges.

(2) Liner trays in shear diaphragms should be inter-connected by seam fasteners through the web at a spacing e_s of not more than 300 mm by seam fasteners (normally blind rivets) located at a distance e_u from the wide flange of not more than 30 mm, all as shown in figure 10.14.

(3) An accurate evaluation of deflections due to fasteners may be made using a similar procedure to that for trapezoidal profiled sheeting.

(4) The shear flow $T_{v,Sd}$ due to ultimate limit states design loads should not exceed $T_{v,Rd}$ given by:

$$T_{v,Rd} = 0{,}2\,E\,\sqrt[4]{I_1\,(t/b_u)^9} \qquad \text{... (10.22)}$$

where:

I_1 is the second moment of area of the wide flange;

b_u is the overall width of the wide flange.

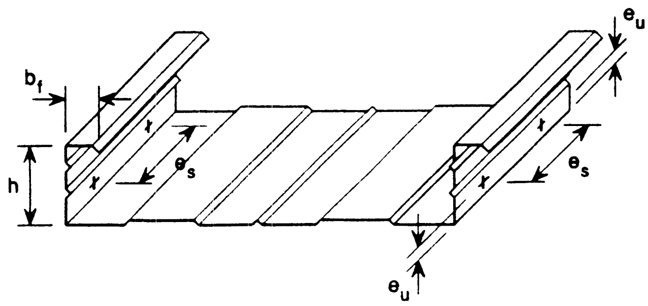


Figure 10.14: Location of seam fasteners

(5) The shear flow $T_{v,ser}$ due to serviceability design loads should not exceed $T_{v,Cd}$ given by:

$$T_{v,Cd} = S_v / 375 \qquad \dots (10.23)$$

where:

S_v is the shear stiffness of the diaphragm, per unit length of the span of the liner trays.

(6) The shear stiffness S_v per unit length may be obtained from:

$$S_v = \frac{\alpha L b_u}{e_s (b - b_u)} \qquad \dots (10.24)$$

where:

L is the overall length of the shear diaphragm (in the direction of the span of the liner trays);

b is the overall width of the shear diaphragm ($b = \Sigma\, b_u$);

α is the stiffness factor.

(7) The stiffness factor α may be derived from tests in accordance with Section 9. Alternatively, in the absence of test results, α may conservatively be taken as equal to 2000 N/mm.

10.4 Perforated sheeting

(1) Perforated sheeting may be designed by calculation, provided that the rules for non-perforated sheeting are modified by introducing the effective thicknesses given below.

NOTE: These calculation rules tend to give rather conservative values. More economical solutions might be obtained from design assisted by testing, see Section 9.

(2) Provided that $0{,}2 \leq d/a \leq 0{,}8$ gross section properties may be calculated using 3.3.2, but replacing t by $t_{a,eff}$ obtained from:

$$t_{a,eff} = 1{,}18t\left(1 - 0{,}9d/a\right) \qquad \dots (10.25)$$

where:

d is the diameter of the perforations;

a is the spacing between the centres of the perforations.

(3) Provided that $0{,}2 \leq d/a \leq 1{,}0$ effective section properties may be calculated using Section 4, but replacing t by $t_{b,eff}$ obtained from:

$$t_{b,eff} = t\sqrt[3]{1{,}18\left(1 - d/a\right)} \qquad \dots (10.26)$$

(4) The resistance of a single unstiffened web to local transverse forces may be calculated using 5.9, but replacing t by $t_{c,eff}$ obtained from:

$$t_{c,eff} = t\left[1 - \left(d/a\right)^2 s_{per}/s_w\right]^{3/2} \qquad \dots (10.27)$$

where:

s_{per} is the slant height of the perforated portion of the web;

s_w is the total slant height of the web.

Annex A [informative]

Testing procedures

A.1 General

(1) This annex A gives appropriate standardized testing and evaluation procedures for a number of tests that are commonly required in practice, as a basis for harmonization of future testing.

(2) It is, however, recognized that most existing test data have been obtained on the basis of tests that differ to some extent from these procedures.

(3) So that the existing data can continue to be used, and to allow sufficient time for transition to harmonized procedures after adequate trial implementation, these testing procedures are presented as an informative annex, covering:

- tests on profiled sheets and liner trays, see A.2;
- tests on cold formed members, see A.3;
- tests on structures and portions of structures, see A.4;
- tests on beams torsionally restrained by sheeting, see A.5;
- evaluation of test results to determine design values, see A.6.

A.2 Tests on profiled sheets and liner trays

A.2.1 General

(1) Although these test procedures are presented in terms of profiled sheets, similar test procedures based on the same principles may also be used for liner trays.

(2) Loading may be applied through air bags or in a vacuum chamber or by steel or timber cross beams arranged to simulate uniformly distributed loading.

(3) To prevent spreading of corrugations, transverse ties or other appropriate test accessories such as timber blocks may be applied to the test specimen. Some examples are given in figure A.1.

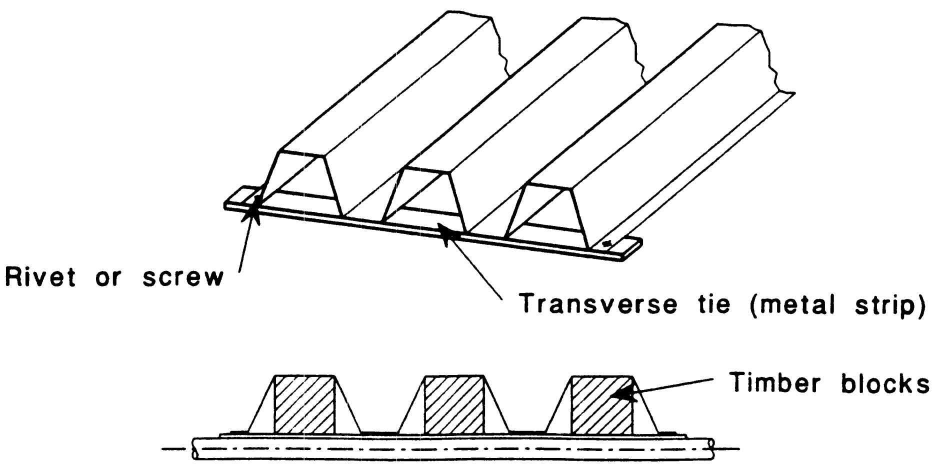


Figure A.1: Examples of appropriate test accessories

(4) For uplift tests, the test set-up should realistically simulate the behaviour of the sheeting under practical conditions. The type of connections between the sheet and the supports should be the same as in the connections to be used in practice.

(5) To give the results a wide range of applicability, hinged and roller supports should preferably be used, to avoid any influence of torsional restraint at the supports on the test results,

(6) It should be ensured that the direction of the loading remains perpendicular to the initial plane of the sheet throughout the test procedure.

(7) To eliminate the deformations of the supports, the deflections at both ends of the test specimen should also be measured.

(8) The test result should be taken as the maximum value of the loading applied to the specimen either coincident with failure or immediately prior to failure as appropriate.

A.2.2 Single span test

(1) A test set-up equivalent to that shown in figure A.2 may be used to determine the midspan moment resistance (in the absence of shear force) and the effective flexural stiffness.

(2) The span should be chosen such that the test results represent the moment resistance of the sheet.

(3) The moment resistance should be determined from the test result.

(4) The flexural stiffness should be determined from a plot of the load-deflection behaviour.

A.2.3 Double span test

(1) The test set-up shown in figure A.3 may be used to determine the resistance of a sheet that is continuous over two or more spans to combinations of moment and shear at internal supports, and its resistance to combined moment and support reaction for a given support width.

(2) The loading should preferably be uniformly distributed (applied using an air bag or a vacuum chamber, for example).

(3) Alternatively any number of line loads (transverse to the span) may be used, arranged to produce internal moments and forces that are appropriate to represent the effects of uniformly distributed loading. Some examples of suitable arrangements are shown in figure A.4.

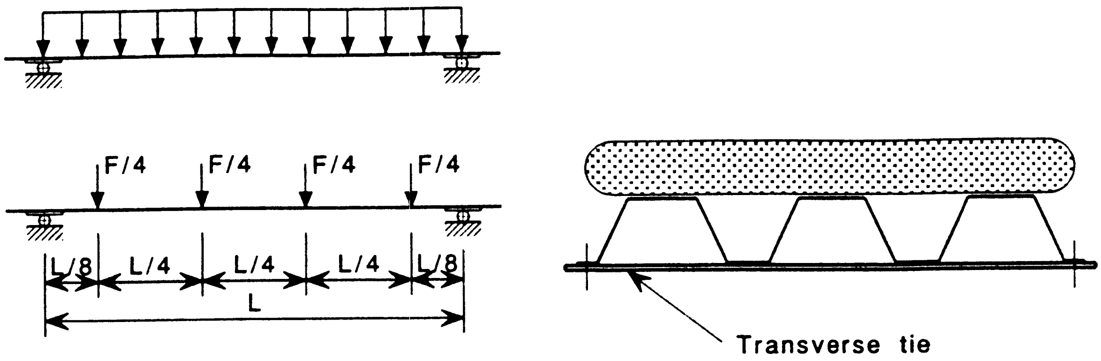
A.2.4 Internal support test

(1) As an alternative to A.2.3, the test set-up shown in figure A.5 may be used to determine the resistance of a sheet that is continuous over two or more spans to combinations of moment and shear at internal supports, and its resistance to combined moment and support reaction for a given support width.

(3) The test span s used to represent the portion of the sheet between the points of contraflexure each side of the internal support, in a sheet continuous over two equal spans L may be obtained from:

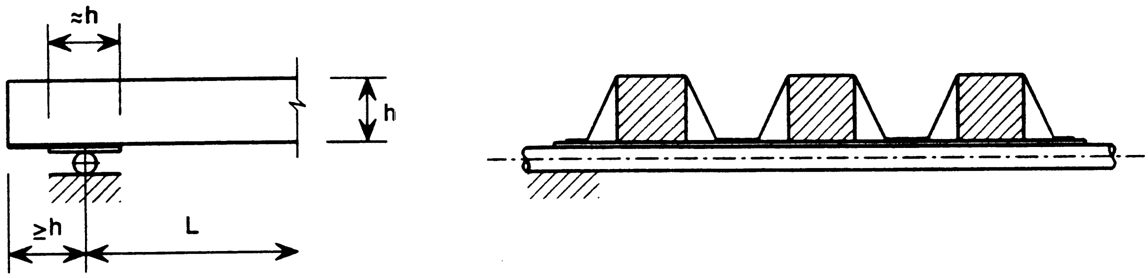
$$s = 0{,}4L \qquad \ldots \text{(A.1)}$$

(3) If plastic redistribution of the support moment is expected, the test span s should be reduced to represent the appropriate ratio of support moment to shear force.

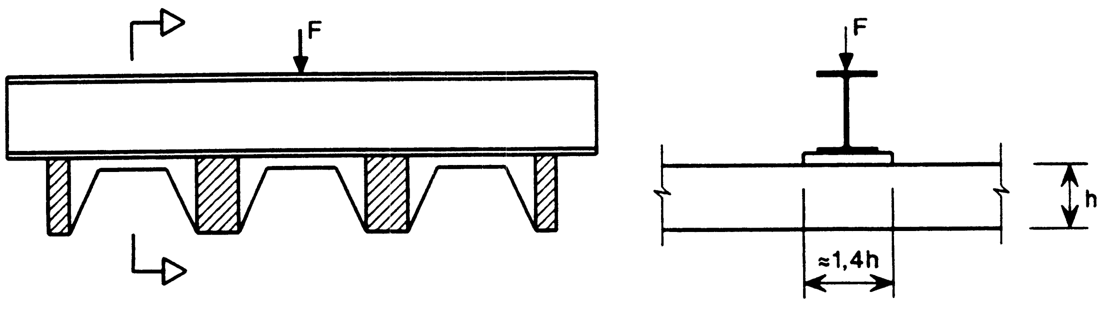


a) Uniformly distributed loading and an example of alternative equivalent line loads

b) Distributed loading applied by an airbag (alternatively by a vacuum test rig)



c) Example of support arrangements for preventing distortion



d) Example of method of applying a line load

Figure A.2: Test set-up for single span tests

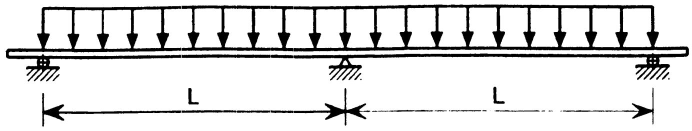


Figure A.3: Test setup for double span tests

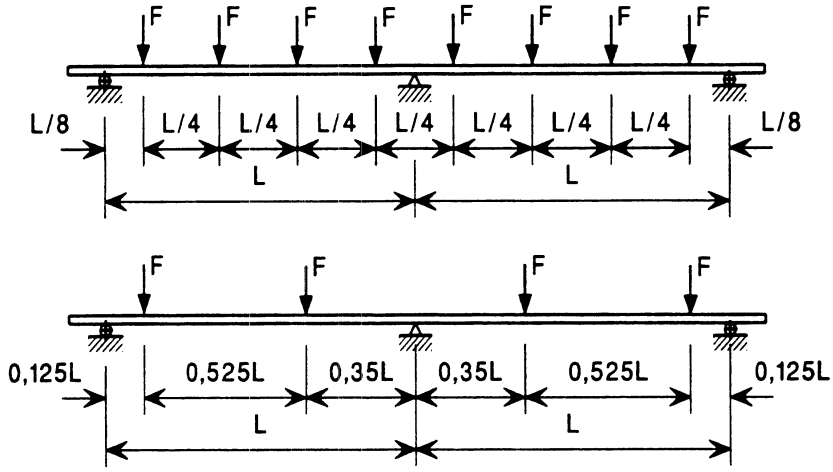


Figure A.4: Examples of suitable arrangements of alternative line loads

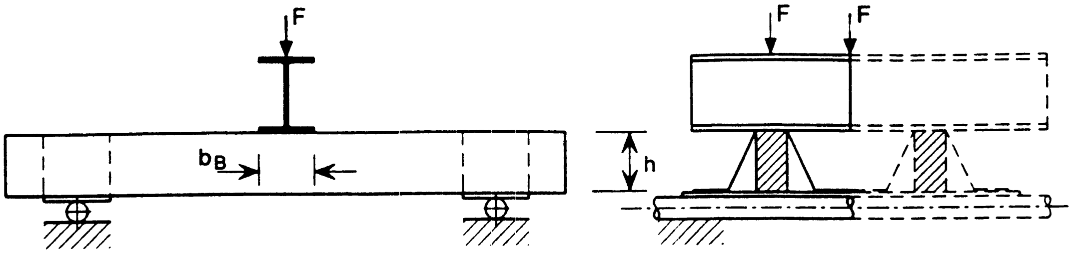
(4) The width b_B of the beam used to apply the test load should be selected to represent the actual support width to be used in practice.

(5) Each test result may be used to represent the resistance to combined bending moment and support reaction (or shear force) for a given span and a given support width. To obtain information about the interaction of bending moment and support reaction, tests should be carried out for several different spans.

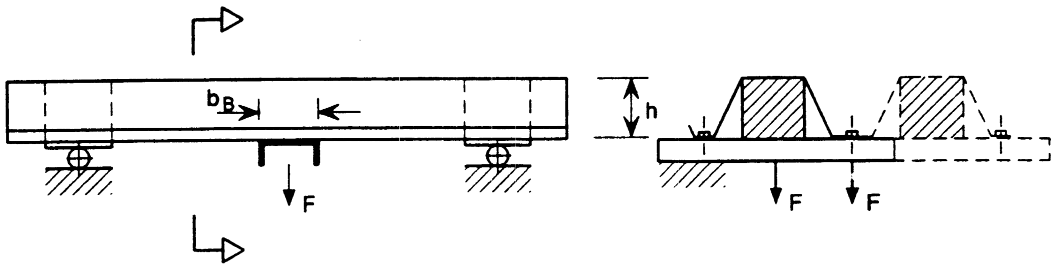
A.2.5 End support test

(1) The test set-up shown in figure A.6 may be used to determine the shear resistance of a sheet at an end support.

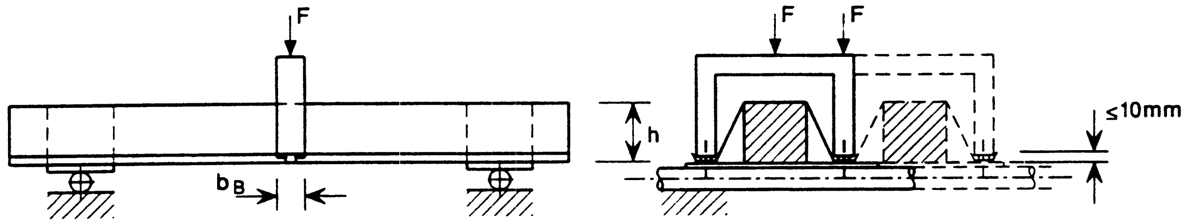
(2) Separate tests should be carried out to determine the shear resistance of the sheet for different lengths u from the contact point at the inner edge of the end support, to the actual end of the sheet, see figure A.6.



a) Internal support under gravity loading

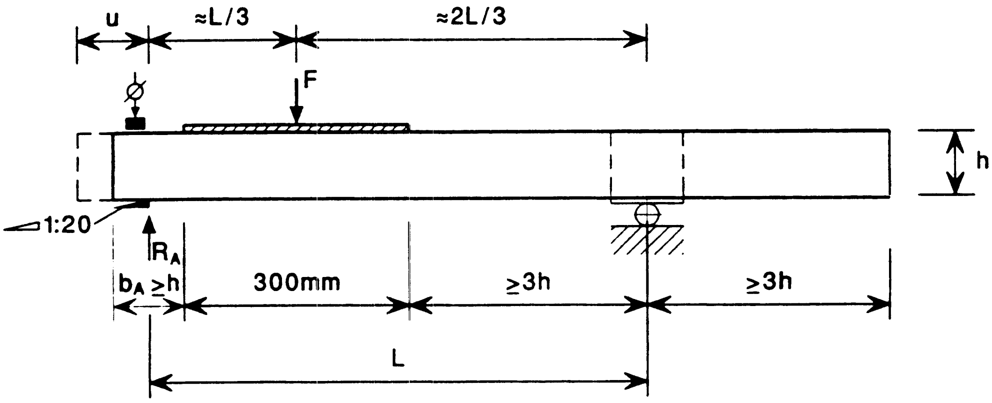


b) Internal support under simulated uplift loading



c) Internal support with loading applied to tension flange

Figure A.5: Test set-up for internal support tests



Key:

b_A = support width

u = length from internal edge of end support to end of sheet

Figure A.6: Test set-up for end support tests

A.3 Tests on cold formed members

A.3.1 General

(1) Each test specimen should be similar in all respects to the component or structure that it represents.

(2) The supporting devices used for tests should preferably provide end conditions that closely reproduce those supplied by the connections to be used in service. Where this cannot be achieved, less favourable end conditions that decrease the load carrying capacity or increase the flexibility should be used, as relevant.

(3) The devices used to apply the test loads should reproduce the way that the loads would be applied in service. It should be ensured that they do not offer more resistance to transverse deformations of the cross-section than would be available in the event of an overload in service. It should also be ensured that they do not localize the applied forces onto the lines of greatest resistance.

(4) If the given load combination includes forces on more than one line of action, each increment of the test loading should be applied proportionately to each of these forces.

(5) At each stage of the loading, the displacements or strains should be measured at one or more principal locations on the structure. Readings of displacements or strains should not be taken until the structure has completely stabilized after a load increment.

(6) Failure of a test specimen should be considered to have occurred in any of the following cases:

- at collapse or fracture;
- if a crack begins to spread in a vital part of the specimen;
- if the displacement is excessive.

(7) The test result should be taken as the maximum value of the loading applied to the specimen either coincident with failure or immediately prior to failure as appropriate.

(8) The accuracy of all measurements should be compatible with the magnitude of the measurement concerned and should in any case not exceed $\pm 1\%$ of the value to be determined.

(9) The measurements of the cross-sectional geometry of the test specimen should include:

- the overall dimensions (width, depth and length) to an accuracy of $\pm 1{,}0$ mm;
- widths of plane elements of the cross-section to an accuracy of $\pm 1{,}0$ mm;
- radii of bends to an accuracy of $\pm 1{,}0$ mm;
- inclinations of plane elements to an accuracy of $\pm 2{,}0°$;
- angles between flat surfaces to an accuracy of $\pm 2{,}0°$;
- locations and dimensions of intermediate stiffeners to an accuracy of $\pm 1{,}0$ mm;
- the thickness of the material to an accuracy of $\pm 0{,}01$ mm.

(10) All other relevant parameters should also be measured, such as:

- locations of components relative to each other;
- locations of fasteners;
- the values of torques etc. used to tighten fasteners.

A.3.2 Full cross-section compression tests

A.3.2.1 Stub column test

(1) Stub column tests may be used to allow for the effects of local buckling in thin gauge cross-sections, by determining the value of the ratio $\beta_A = A_{eff}/A_g$ and the location of the effective centroidal axis.

(2) If local buckling of the plane elements governs the resistance of the cross-section, the specimen should have a length of at least 3 times the width of the widest plate element.

(3) The lengths of specimens with perforated cross-sections should include at least 5 pitches of the perforations, and should be such that the specimen is cut to length midway between two perforations.

(4) In the case of a cross-section with edge or intermediate stiffeners, it should be ensured that the length of the specimen is not less than the expected buckling lengths of the stiffeners.

(5) If the overall length of the specimen exceeds 20 times the least radius of gyration of its gross cross-section i_{min}, intermediate lateral restraints should be supplied at a spacing of not more than $20\,i_{min}$.

(6) Before testing, the tolerances of the cross-sectional dimensions of the specimen should be checked to ensure that they are within the permitted deviations.

(7) The cut ends of the specimen should be flat, and should be perpendicular to its longitudinal axis.

(8) An axial compressive force should be applied to each end of the specimen through pressure pads at least 30 mm thick, that protrude at least 10 mm beyond the perimeter of the cross-section.

(9) The test specimen should be placed in the testing machine with a ball bearing at each end. There should be small drilled indentations in the pressure pads to receive the ball bearings. The ball bearings should be located in line with the centroid of the calculated effective cross-section. If the calculated location of this effective centroid proves not to be correct, it may be adjusted within the test series.

(10) In the case of open cross-sections, possible spring-back may be corrected.

(11) Stub column tests may be used to determine the compression resistance of a cross-section. In interpreting the test results, the following parameters should be treated as variables:

- the thickness;
- the ratio b_p/t;
- the ratio f_u/f_{yb};
- the location of the centroid of the effective cross-section;
- imperfections in the shape of the elements of the cross-section;
- the method of cold rolling (for example increasing the yield strength by introducing a deformation that is subsequently removed).

A.3.2.2 Member buckling test

(1) Member buckling tests may be used to determine the resistance of compression members with thin gauge cross-sections to overall buckling (including flexural buckling, torsional buckling and torsional-flexural buckling) and the interaction between local buckling and overall buckling.

(2) The method of carrying out the test should be generally as given for stub column tests in A.3.2.1.

(3) A series of tests on axially loaded specimens may be used to determine the appropriate buckling curve for a given type of cross-section and a given grade of steel, produced by a specific process. The values of relative slenderness $\bar{\lambda}$ to be tested and the minimum number of tests n at each value, should be as given in table A.1.

Table A.1: Relative slenderness values and numbers of tests

$\bar{\lambda}$	0,2	0,5	0,7	1,0	1,3	1,6	2,0	3,0
n	3	5	5	5	5	5	5	5

(4) Similar tests may also be used to determine the effect of introducing intermediate restraints on the torsional buckling resistance of a member.

(5) For the interpretation of the test results the following parameters should be taken into account:

- the parameters listed for stub column tests in A.3.2.1(11);
- overall lack of straightness imperfections;
- type of end or intermediate restraint (flexural, torsional or both).

A.3.3 Full cross-section tension test

(1) This test may be used to determine the average yield strength f_{ya} of the cross-section.

(2) The specimen should have a length of at least 5 times the width of the widest plane element in the cross-section.

(3) The load should be applied through end supports that ensure a uniform stress distribution across the cross-section.

(4) The failure zone should occur at a distance from the end supports of not less than the width of the widest plane element in the cross-section.

A.3.4 Full cross-section bending test

(1) This test may be used to determine the moment resistance and rotation capacity of a cross-section.

(2) The specimen should have a length of at least 15 times its greatest lateral dimension. The spacing of lateral restraints to the compression flange should not be less than the spacing to be used in service.

(3) A pair of point loads should be applied to the specimen to produce a length under uniform bending moment at midspan of at least $0,2 \times$ (span) but not more than $0,33 \times$ (span). These loads should be applied through the shear centre of the cross-section. If necessary, local buckling of the specimen should be prevented at the points of load application, to ensure that failure occurs within the central portion of the span. The deflection should be measured at the load positions, at midspan and at the ends of the specimen.

(4) In interpreting the test results, the following parameters should be treated as variables:

- the thickness;
- the ratio b_p/t;
- the ratio f_u/f_y;
- differences between restraints used in the test and those available in service;
- the support conditions.

A.4 Tests on structures and portions of structures

A.4.1 Acceptance test

(1) This acceptance test may be used as a non-destructive test to confirm the structural performance of a structure or portion of a structure.

(2) The test load for an acceptance test should be taken as equal to the sum of:

- $\boxed{1,0}$ × (the actual self-weight present during the test);
- $\boxed{1,15}$ × (the remainder of the permanent load);
- $\boxed{1,25}$ × (the variable loads).

but need not be taken as more than the mean of the total ultimate limit state design load and the total serviceability limit state design load for the characteristic (rare) load combination.

(3) Before carrying out the acceptance test, preliminary bedding down loading (not exceeding the characteristic values of the loads) may optionally be applied, and then removed.

(4) The structure should first be loaded up to a load equal to the total characteristic load. Under this load it should demonstrate substantially elastic behaviour. On removal of this load the residual deflection should not exceed 20% of the maximum recorded. If these criteria are not satisfied this part of the test procedure should be repeat. In this repeat load cycle, the structure should demonstrate substantially linear behaviour up to the characteristic load and the residual deflection should not exceed 10% of the maximum recorded.

(5) During the acceptance test, the loads should be applied in a number of regular increments at regular time intervals and the principal deflections should be measured at each stage. When the deflections show significant non-linearity, the load increments should be reduced.

(6) On the attainment of the acceptance test load, the load should be maintained for at least one hour and deflection measurements should be taken to establish whether the structure is subject to any time-dependent deformations, such as deformations of fasteners or deformations arising from creep in the zinc layer.

(7) Unloading should be completed in regular decrements, with deflection readings taken at each stage.

(8) The structure should prove capable of sustaining the acceptance test load, and there should be no significant local distortion or defects likely to render the structure unserviceable after the test.

A.4.2 Strength test

(1) This strength test may be used to confirm the calculated load carrying capacity of a structure or portion of a structure. Where a number of similar items are to be constructed to a common design, and one or more prototypes have been submitted to and met all the requirements of this strength test, the others may be accepted without further testing provided that they are similar in all relevant respects to the prototypes.

(2) Before carrying out a strength test the specimen should first pass the acceptance test detailed in A.4.1.

(3) The load should then be increased in increments up to the strength test load and the principal deflections should be measured at each stage. The strength test load should be maintained for at least one hour and deflection measurements should be taken to establish whether the structure is subject to creep.

(4) Unloading should be completed in regular decrements with deflection readings taken at each stage.

(5) The total test load (including self-weight) for a strength test F_{str} should be determined from the total design load F_{Sd} specified for ultimate limit state verifications by calculation, using:

$$F_{str} = \mu_F F_{Sd} \qquad \dots (A.2)$$

in which μ_F is the load adjustment coefficient.

(6) The load adjustment coefficient μ_F should take account of variations in the load carrying capacity of the structure, or portion of a structure, due to the effects of variation in the material yield strength, local buckling, overall buckling and any other relevant parameters or considerations.

(7) Where a realistic assessment of the load carrying capacity of the structure, or portion of a structure, can be made using the provisions of this Part 1.3 of ENV 1993 for design by calculation, or another proven method of analysis that takes account of all buckling effects, the load adjustment coefficient μ_F may be taken as equal to the ratio of (the value of the assessed load carrying capacity based on the averaged basic yield strength f_{ym}) compared to (the corresponding value based on the nominal basic yield strength f_{yb}).

(8) The value of f_{ym} should be determined from the measured basic strength $f_{yb,obs}$ of the various components of the structure, or portion of a structure, with due regard to their relative importance.

(9) If realistic theoretical assessments of the load carrying capacity cannot be made, the load adjustment coefficient μ_F should be taken as equal to the resistance adjustment coefficient μ_R defined in A.6.2.

(10) Under the test load there should be no failure by buckling or rupture in any part of the specimen.

(11) On removal of the test load, the deflection should be reduced by at least 20%.

A.4.3 Prototype failure test

(1) A test to failure may be used to determine the real mode of failure and the true load carrying capacity of a structure or assembly. If the prototype is not required for use, it may optionally be used to obtain this additional information after completing the strength test described in A.4.2.

(2) Alternatively a test to failure may be carried out to determine the true design load carrying capacity from the ultimate test load. As the acceptance and strength test procedures should preferably be carried out first, an estimate should be made of the anticipated design load carrying capacity as a basis for such tests.

(3) Before carrying out a test to failure, the specimen should first pass the strength test described in A.4.2. Its estimated design load carrying capacity may then be adjusted based on its behaviour in the strength test.

(4) During a test to failure, the loading should first be applied in increments up to the strength test load. Subsequent load increments should then be based on an examination of the plot of the principal deflections.

(5) The ultimate load carrying capacity should be taken as the value of the test load at that point at which the structure or assembly is unable to sustain any further increase in load.

NOTE: At this point gross permanent distortion is likely to have occurred. In some cases gross deformation might define the test limit.

A.4.4 Identification test

(1) An identification test may be used to:

- verify load bearing behaviour relative to analytical design models;
- quantify parameters derived from design models, such as strength or stiffness of members or joints.

A.5 Tests on beams torsionally restrained by sheeting

A.5.1 General

(1) These test procedures may be used for beams that are partially restrained against torsional displacement, by means of trapezoidal profiled steel sheeting or other suitable cladding.

(2) These procedures may be used for purlins, side rails, floor beams and other similar types of beams that have relevant restraint conditions.

A.5.2 Internal support test

A.5.2.1 Test set-up

(1) The test set-up shown in figure A.7 may be used to determine the resistance of a beam that is continuous over two or more spans, to combinations of bending moment and shear force at internal supports.

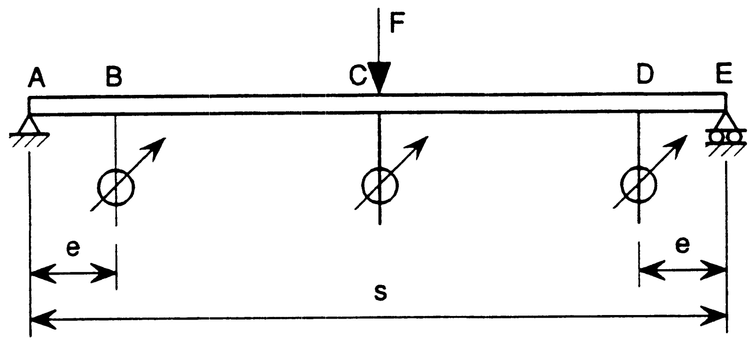


Figure A.7: Test set-up for internal support tests

(2) The supports at **A** and **E** should be hinged and roller supports respectively. At these supports, rotation about the longitudinal axis of the beam may be prevented, for example by means of cleats.

(3) The method of applying the load at **C** should correspond with the method to be used in service.

NOTE: In many cases this will mean that lateral displacement of both flanges is prevented at **C**.

(4) In order to eliminate the displacements of the supports, the vertical deflections should be measured at the point of load application **C** and at two points **B** and **D** located at a distance e from each support.

(5) The test span s should be chosen to produce combinations of bending moment and shear force that represent those expected to occur in practical application under the design load for the relevant limit state.

(6) For double span beams of span L subject to uniformly distributed loads, the test span s should normally be taken as equal to $0{,}4L$. However, if plastic redistribution of the support moment is expected, the test span s should be reduced to represent the appropriate ratio of support moment to shear force.

A.5.2.2 Execution of tests

(1) In addition to the general rules for testing, the following specific aspects should be taken into account.

(2) Testing should continue beyond the peak load and the recording of the deflections should be continued either until the applied load has reduced to between 10% and 15% of its peak value or until the deflection has reached a value 6 times the maximum elastic displacement.

A.5.2.3 Interpretation of test results

(1) The actual measured test results $R_{obs,i}$ should be adjusted as specified in A.6.2 to obtain adjusted values $R_{adj,i}$ related to the nominal basic yield strength f_{yb} and design thickness t of the steel, see 3.1.3.

(2) For each value of the test span s the support reaction R should be taken as the mean of the adjusted values of the peak load F_{max} for that value of s. The corresponding value of the support moment M should then be determined from:

$$M = \frac{sR}{4} \qquad \text{... (A.3)}$$

(3) The pairs of values of M and R for each value of s should be plotted as shown in figure A.8. Pairs of values for intermediate combinations of M and R may then be determined by linear interpolation.

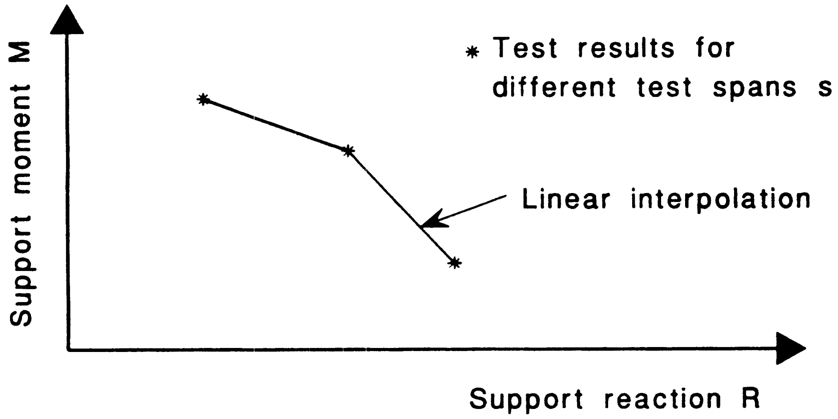


Figure A.8: Relation between support moment and support reaction

(4) The net deflection at the point of load application **C** in figure A.7 should be obtained from the gross measured values by deducting the mean of the corresponding deflections measured at the points **B** and **D** located at a distance e from the support points **A** and **E**, see figure A.7.

(5) For each test the applied load should be plotted against the corresponding net deflection, see figure A.9. From this plot, the rotation θ should be obtained for a range of values of the applied load using:

$$\theta = \frac{2(\delta_{p\ell} - \delta_{e\ell})}{0{,}5\,s - e} \qquad \text{... (A.4)}$$

where:

$\delta_{e\ell}$ is the net deflection for a given load on the rising part of the curve, before F_{max};

$\delta_{p\ell}$ is the net deflection for the same load on the falling part of the curve, after F_{max};

s is the test span;

e is the distance between a deflection measurement point and a support, see figure A.7.

(6) The relationship between M and θ should then be plotted for each test at a given test span s corresponding to a given value of beam span L as shown in figure A.10. The design M - θ characteristic for the moment resistance of the beam over an internal support should then be taken as equal to 0,9 times the mean value of M for all the tests corresponding to that value of the beam span L.

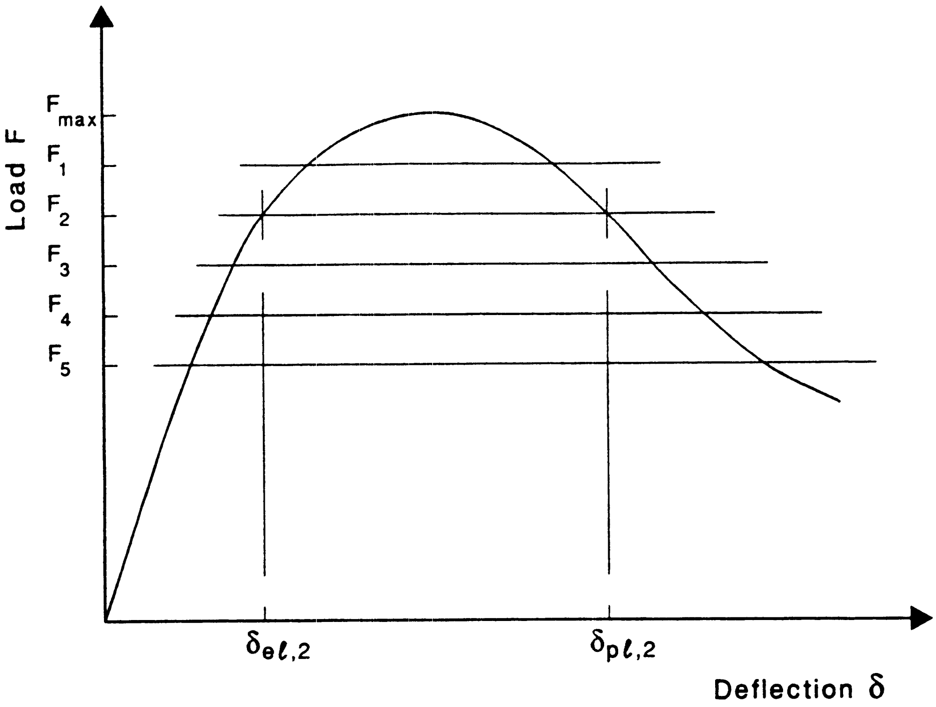


Figure A.9: Relationship between load and net deflection

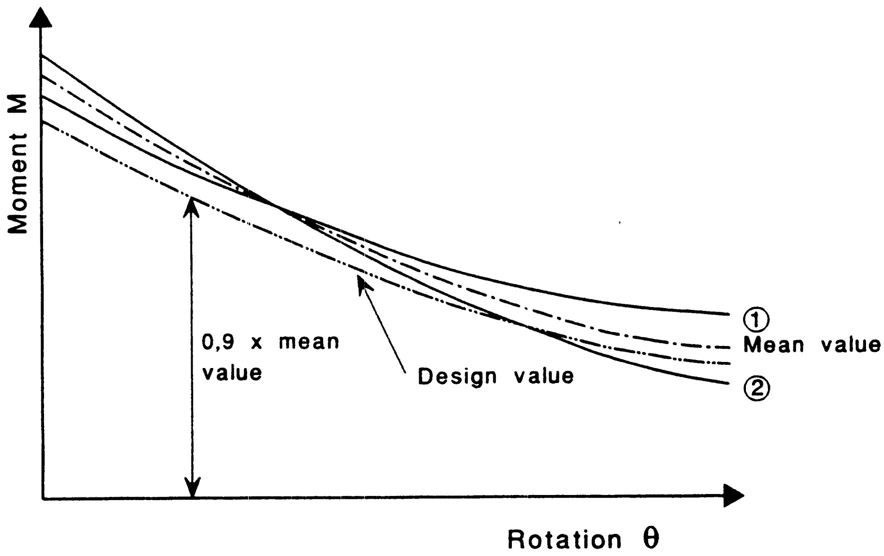


Figure A.10: Derivation of the design moment-rotation characteristic

A.5.3 Determination of torsional restraint

(1) The test set-up shown in figure A.11 may be used to determine the amount of torsional restraint given by adequately fastened sheeting or by another member perpendicular to the span of the beam.

(2) This test set-up covers two different contributions to the total amount of restraint as follows:

a) The lateral stiffness K_A per unit length corresponding to the rotational stiffness of the connection between the sheeting and the beam;

b) The lateral stiffness K_B per unit length due to distortion of the cross-section of the purlin.

(3) The combined restraint per unit length may be determined from:

$$\left(1/K_A + 1/K_B\right) = \delta/F \qquad \ldots (A.5)$$

where:

F is the load per unit length of the test specimen to produce a lateral deflection of $h/10$;

h is the overall depth of the specimen;

δ is the lateral displacement of the top flange in the direction of the load F.

(4) In interpreting the test results, the following parameters should be treated as variables:

- the number of fasteners per unit length of the specimen;
- the type of fasteners;
- the flexural stiffness of the beam, relative to its thickness;
- the flexural stiffness of the bottom flange of the sheeting, relative to its thickness;
- the positions of the fasteners in the flange of the sheeting;
- the distance from the fasteners to the centre of rotation of the beam;
- the overall depth of the purlin;
- the possible presence of insulation between the beam and the sheeting.

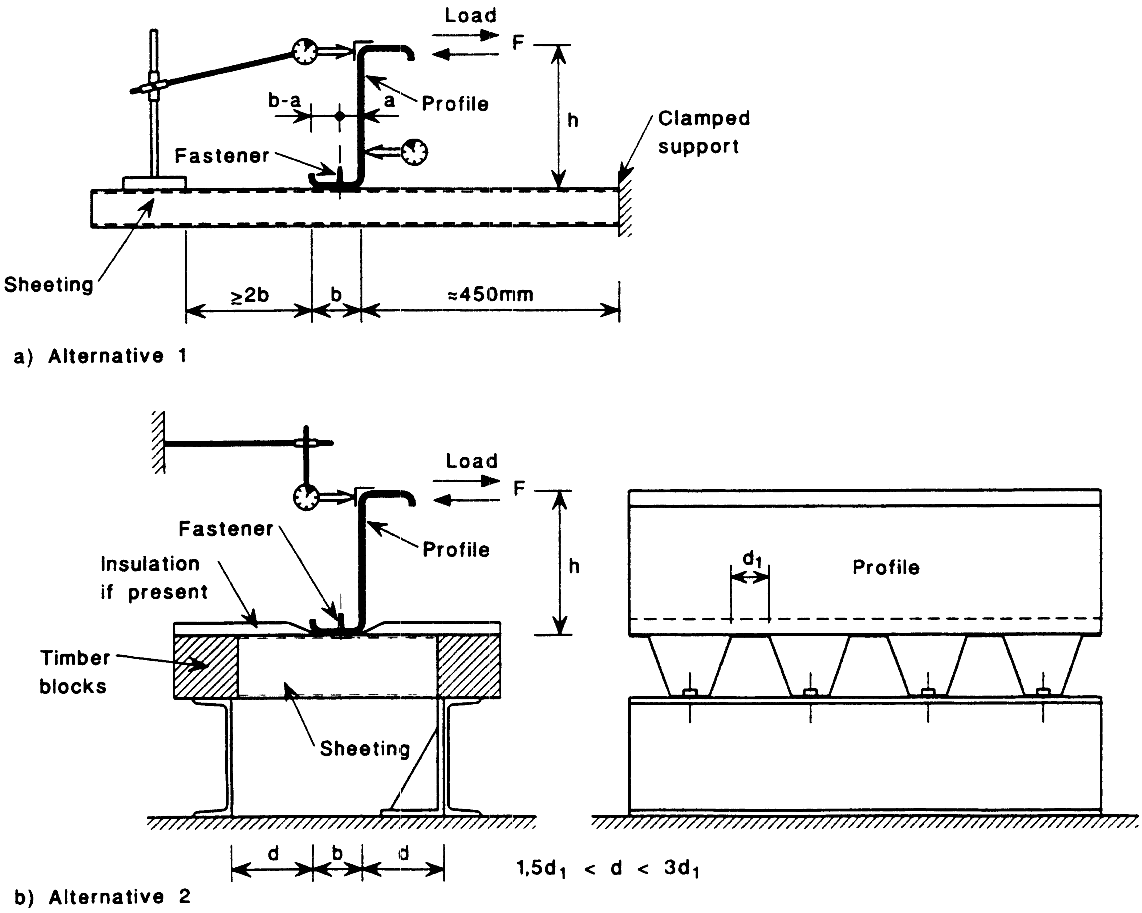


Figure A.11: Experimental determination of spring stiffnesses K_A and K_B

A.6 Evaluation of test results

A.6.1 General

(1) A specimen under test should be regarded as having failed if the applied test loads reach their maximum values, or if the gross deformations exceed specified limits.

(2) The gross deformations of members should generally satisfy:

$$\delta \leq L/50 \qquad \text{... (A.6)}$$

$$\phi \leq 1/50 \qquad \text{... (A.7)}$$

where:

δ is the maximum deflection of a beam of span L;

ϕ is the sway angle of a structure.

(3) In the testing of connections, or of components in which the examination of large deformations is necessary for accurate assessment (for example, in evaluating the moment-rotation characteristics of sleeves), no limit need be placed on the gross deformation during the test.

(4) An appropriate margin of safety should be available between a ductile failure mode and possible brittle failure modes. As brittle failure modes do not usually appear in large scale tests, additional detail tests should be carried out where necessary.

NOTE: This is often the case for connections.

A.6.2 Adjustment of test results

(1) Test results should be appropriately adjusted to allow for variations between the actual measured properties of the test specimens and their nominal values.

(2) The actual measured basic yield strength $f_{yb,obs}$ should not deviate by more than $\pm 25\%$ from the nominal basic yield strength f_{yb}.

(3) The actual measured material thickness t_{obs} should not exceed the design thickness t based on the nominal material thickness t_{nom} (see 3.1.3) by more than 12%.

(4) Adjustments should be made in respect of the actual measured values of the material thickness t_{obs} and the basic yield strength $f_{yb,obs}$ for all tests, except where the design expression that uses the test results also uses the actual measured value of the thickness or yield strength of the material, as appropriate.

(5) The adjusted value $R_{adj,i}$ of the test result for test i should be determined from the actual measured test result $R_{obs,i}$ using:

$$R_{adj,i} = R_{obs,i} / \mu_R \qquad \text{... (A.8)}$$

in which μ_R is the resistance adjustment coefficient given by:

$$\mu_R = \left(\frac{f_{yb,obs}}{f_{yb}}\right)^{\alpha} \left(\frac{t_{obs}}{t}\right)^{\beta} \qquad \text{... (A.9)}$$

(6) The exponent α for use in expression (A.9) should be obtained as follows:

- if $f_{yb,obs} \leq f_{yb}$: $\alpha = 0$
- if $f_{yb,obs} > f_{yb}$:
  - generally: $\alpha = 1$
  - for profiled sheets (or liner trays) in which compression elements have such large b_p / t ratios that local buckling is clearly the failure mode: $\alpha = 0{,}5$

(7) The exponent β for use in expression (A.9) should be obtained as follows:

- if $t_{obs} \leq t$: $\beta = 1$
- if $t_{obs} > t$:
  - for tests on profiled sheets or liner trays: $\beta = 2$
  - for tests on members, structures or portions of structures:
    - if $b_p / t \leq (b_p / t)_{lim}$: $\beta = 1$
    - if $b_p / t > 1{,}5(b_p / t)_{lim}$: $\beta = 2$
    - if $(b_p / t)_{lim} < b_p / t < 1{,}5(b_p / t)_{lim}$: obtain β by linear interpolation.

in which the limiting width-to thickness ratio $(b_p / t)_{lim}$ given by:

$$(b_p / t)_{lim} = 0{,}64 \sqrt{\frac{E k_\sigma}{f_{yb}}} \times \sqrt{\frac{f_{yb} / \gamma_{M1}}{\sigma_{com,Ed}}} \cong 19{,}1\, \varepsilon \sqrt{k_\sigma} \times \sqrt{\frac{f_{yb} / \gamma_{M1}}{\sigma_{com,Ed}}} \quad \text{... (A.10)}$$

where:

b_p	is	the notional flat width of a plane element;
k_σ	is	the relevant buckling factor from table 4.1 or 4.2;
$\sigma_{com,Ed}$	is	the largest calculated compressive stress in that element, when the resistance of the cross-section is reached.

A.6.3 Characteristic values

A.6.3.1 General

(1) Characteristic values may be determined statistically, provided that there are at least 4 test results.

NOTE: A larger number is generally preferable, particularly if the scatter is relatively wide.

(2) If the number of test results available is 3 or less, the method given in A.6.3.3 may be used.

(3) The characteristic minimum value should be determined using the following provisions. If the characteristic maximum value or the characteristic mean value is required, it should be determined by using appropriate adaptations of the provisions given for the characteristic minimum value.

(4) The characteristic value of a resistance R_k determined on the basis of at least 4 tests may be obtained from:

$$R_k = R_m - ks \quad \text{... (A.11)}$$

where:

s is the standard deviation;

k is the appropriate coefficient from table A.2;

R_m is the mean value of the adjusted test results R_{adj}.

(5) The standard deviation s may be determined using:

$$s = \sqrt{\frac{\sum_{i=1}^{n} \left(R_{adj,i} - R_m\right)^2}{n-1}} \equiv \sqrt{\frac{\sum_{i=1}^{n} \left(R_{adj,i}\right)^2 - \frac{1}{n}\left(\sum_{i=1}^{n} R_{adj,i}\right)^2}{n-1}} \quad \text{... (A.12)}$$

where:

$R_{adj,i}$ is the adjusted test result for test i;

n is the number of tests.

Table A.2: Values of the coefficient *k*

n	4	5	6	8	10	20	30	∞
k	2,63	2,33	2,18	2,00	1,92	1,76	1,73	1,64

A.6.3.2 Characteristic values for families of tests

(1) A series of tests carried out on a number of otherwise similar structures, portions of structures, members, sheets or other structural components, in which one or more parameters is varied, may be treated as a single family of tests, provided that they all have the same failure mode. The parameters that are varied may include cross-sectional dimensions, spans, thicknesses and material strengths.

(2) The characteristic resistances of the members of a family may be determined on the basis of a suitable design expression that relates the test results to all the relevant parameters. This design expression may either be based on the appropriate equations of structural mechanics, or determined on an empirical basis.

(3) The design expression should be modified to predict the mean measured resistance as accurately as practicable, by adjusting the coefficients to optimize the correlation.

NOTE: Information on this process is given in annex Z*) of ENV 1993-1-1.

(4) In order to calculate the standard deviation s each test result should first be normalized by dividing it by the corresponding value predicted by the design expression. If the design expression has been modified as specified in (3), the mean value of the normalized test results will be unity. The number of tests n should be taken as equal to the total number of tests in the family.

(5) For a family of at least four tests, the characteristic resistance R_k should then be obtained from expression (A.11) by taking R_m as equal to the value predicted by the design expression, and using the value of k from table A.2 corresponding to a value of n equal to the total number of tests in the family.

*) This annex is in preparation.

A.6.3.3 Characteristic values based on a small number of tests

(1) If only one test is carried out, then the characteristic resistance R_k corresponding to this test should be obtained from the adjusted test result R_{adj} using:

$$R_k = 0,9\,\eta_k R_{adj} \quad \ldots \text{(A.13)}$$

in which η_k should be taken as follows, depending on the failure mode:

- yielding failure: $\eta_k = 0,9$;
- gross deformation: $\eta_k = 0,9$;
- local buckling: $\eta_k = 0,8$;
- overall instability: $\eta_k = 0,7$.

(2) For a family of two or three tests, provided that each adjusted test result $R_{adj,i}$ is within $\pm 10\%$ of the mean value R_m of the adjusted test results, the characteristic resistance R_k should be obtained using:

$$R_k = \eta_k R_m \quad \ldots \text{(A.14)}$$

(3) The characteristic values of stiffness properties (such as flexural or rotational stiffness) may be taken as the mean value of at least two tests, provided that each test result is within $\pm 10\%$ of the mean value.

A.6.4 Design values

(1) The design value of a resistance R_d should be derived from the corresponding characteristic value R_k determined by testing, using:

$$R_d = R_k / \gamma_M / \gamma_{sys} \quad \ldots \text{(A.15)}$$

where:

γ_M is the partial factor for resistance;

γ_{sys} is a partial factor for differences in behaviour under test conditions and service conditions.

(2) For a family of at least four tests, the value of γ_M may be determined using statistical methods.

NOTE: Information on an appropriate method is given in annex Z*) of ENV 1993-1-1.

(3) Alternatively γ_M may be taken as equal to the appropriate value of γ_M for design by calculation given in Section 2 or Section 8 of this Part 1.3.

(4) The appropriate value for γ_{sys} should be agreed between the client, the designer, the testing organisation and the competent authority.

(5) For sheeting and for other well defined standard testing procedures (including A.3.2.1 stub column tests, A.3.3 tension tests and A.3.4 bending tests) γ_{sys} may be taken as equal to 1,0.

(6) For other types of tests in which possible instability phenomena, or modes of behaviour, of structures or structural components might not be covered sufficiently by the tests, the value of γ_{sys} should be assessed taking into account the actual testing conditions, in order to achieve the necessary reliability.

A.6.5 Serviceability

(1) The provisions given in Section 7 should be satisfied.

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*) This annex is in preparation.

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DRAFT FOR DEVELOPMENT

DD ENV 1993-1-2:2001

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Eurocode 3: Design of steel structures —

Part 1.2: General rules — Structural fire design

(together with United Kingdom National Application Document)

ICS 13.220.20; 91.010.30; 91.080.10

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British Constructional Steelwork Association

Cold Rolled Sections Association

Confederation of British Forgers

Department of the Environment, Transport and the Regions

Department of the Environment, Transport and the Regions — Construction Directorate

Department of the Environment, Transport and the Regions — Highways Agency

Health and Safety Executive

Institution of Civil Engineers

Institution of Structural Engineers

Steel Construction Institute

UK Steel Association

Welding Institute

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Contents

National foreword

This publication has been prepared by Subcommittee B/525/31 in cooperation with B/525/4 and is the English language version of ENV 1993-1-2:1995, *Eurocode 3: Design of steel structures — Part 1.2: General rules — Structural fire design*, as published by the European Committee for Standardization (CEN). This Draft for Development also includes the United Kingdom (UK) National Application Document (NAD) to be used with the ENV in the design of buildings to be constructed in the UK.

ENV 1993-1-2:1995 results from a programme of work sponsored by the European Commission to make available a common set of rules for the design of building and civil engineering works.

This publication should not be regarded as a British Standard.

An ENV is made available for provisional application, but does not have the status of a European Standard. The aim is to use the experience gained to modify the ENV so that it can be adopted as a European Standard.

The values for certain parameters in the ENV Eurocodes may be set by CEN Members so as to meet the requirements of national regulations. These parameters are designated by □ (boxed values) in the ENV.

During the ENV period of validity, reference should be made to the supporting documents listed in the NAD.

The purpose of the NAD is to provide essential information, particularly in relation to safety, to enable the ENV to be used for buildings constructed in the UK. The NAD takes precedence over corresponding provisions in the ENV.

The Building Regulations 1991, Approved Document A 1992 (published December 1991)[1)], draws designers' attention to the potential use of ENV Eurocodes as an alternative approach to Building Regulation compliance. ENV 1993-1-2:1995 has been thoroughly examined over a period of several years and is considered to offer such an alternative approach, when used in conjunction with this NAD.

Compliance with DD ENV 1993-1-2:2001 does not of itself confer immunity from legal obligations.

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National Application Document

for use in the UK with ENV 1993-1-2:1995

Contents of National Application Document

Introduction

This National Application Document (NAD) has been prepared by Subcommittee B/525/31. It has been developed from:

a) a textual examination of ENV 1993-1-2:1995;

b) a parametric calibration against BS 5950-8, supporting standards and test data;

c) trial calculations.

It should be noted that this NAD, in common with ENV 1993-1-2 and supporting CEN standards, uses a comma (,) where a decimal point (.) would be traditionally used in the UK.

1 Scope

This NAD provides information required to enable ENV 1993-1-2:1995 *Eurocode 3 — Design of steel structures — Part 1.2: General rules — Structural fire design*, to be used for the fire resistant design of buildings to be constructed in the UK. ENV 1993-1-2:1995 is intended to be used in conjunction with DD ENV 1991-2-2:1996 and DD ENV 1993-1-1:1992, which refer to British Standards for the values of mechanical and thermal loads (actions).

2 Normative references

The following normative documents contain provisions which, through reference in this text, constitute provisions of this NAD. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. For undated references, the latest edition of the publication referred to applies.

BS 476-20:1987, *Fire tests on building materials and structures — Part 20: Method for determination of the fire resistance of elements of construction (general principles).*

BS 476-21:1987, *Fire tests on building materials and structures — Part 21: Methods for determination of the fire resistance of loadbearing elements of construction.*

BS 5555:1993, *Specification for SI units and recommendations for the use of their multiples and certain other units.*

DD ENV 1991-2-2:1996, *Basis of design and actions on structures — Part 2-2: Actions on structures exposed to fire (together with United Kingdom National Application Document).*

DD ENV 1993-1-1:1992, *Eurocode 3: Design of steel structures — Part 1-1: General rules and rules for buildings (together with United Kingdom National Application Document).*

DD ENV 1993-1-3, *Eurocode 3: Design of steel structures — Part 1-3: General rules — Supplementary rules for cold formed thin gauge members and sheeting (together with United Kingdom National Application Document).*

DD ENV 1994-1-2, *Eurocode 4: Design of composite steel and concrete structures — Part 1-2: General rules — Structural fire design (together with United Kingdom National Application Document).*

3 Mechanical loading, partial factors, combination factors and other values

a) Pending the issue of EN 1993-1-1, the mechanical actions, partial factors, combination factors and other values should be determined from clauses **3** and **4** of the NAD for ENV 1993-1-1:1992.

b) The partial factors for the fire situation should be taken from Table 1 and the combination factors should be taken from Table 2.

4 Thermal loading, partial factors, combination factors and other values

a) The thermal actions should be obtained from the NAD of ENV 1991-2-2.

b) The value of $h_{net,d}$ referred to in ENV 1993-1-2:1995, **4.2.5.1**(2) should be obtained from ENV 1991-2-2, **4.2.1** making the following modifications:

1) a value of 0,45 should be used in ENV 1991-2-2:1996, **4.2.1**(2) for the factor $\gamma_{n,r}$ in accordance with the NAD of DD ENV 1991-2-2;

2) a value of 0,8 should be used in ENV 1991-2-2:1996, **4.2.1**(3) for the emissivity of steel ε_m, giving a value of 0,64 for ε_{res}.

c) The values for the adaptation factors κ_1 and κ_2 should be taken from Table 3

Table 1 — Partial factors (γ factors) for the fire situation

Reference in ENV 1993-1-2	Description	Symbol	Condition	Value	
				Boxed ENV value	Value for UK use
2.3(2)P	Partial factor for the fire situation	$\gamma_{M,fi}$	For thermal properties	1,0	1,0
2.3(3)P	Partial factor for the fire situation	$\gamma_{M,fi}$	For mechanical properties	1,0	1,0
2.4.3(4)	Partial factor for permanent action in accidental design situation	γ_{GA}	Fire	1,0	1,0[a]
[a] This value of γ_{GA} is different from the value in the NAD of DD ENV 1993-1-1:1992.					

5 Reference standards

Where ENs are referred to, appropriate BS ENs should be used. The remaining supporting standards which should be used are listed in Table 4.

Table 2 — Combination factors for the fire situation

Action	$\Psi_{1,1}$
Imposed floor loads in buildings:	
— storage;	0,9
— escape stairs and lobbies;	0,9
— all other areas.	0,7
Imposed roof loads (including snow loads)	0
Wind loads:	
— height to eaves up to 8 m;	0
— height to eaves greater than 8 m.	0,3
NOTE Plant load should be taken as a permanent load.	

6 Additional recommendations

Table 3 — Adaptation factors

Reference in ENV 1993-1-2	Description	Symbol	Condition	Value	
				ENV value	Value for UK use
4.2.3.3(8)	The adaptation factor for non-uniform temperature distribution across a cross-section	κ_1	For a beam exposed on all four sides	1,0	1,0
4.2.3.3(8)	The adaptation factor for non-uniform temperature distribution across a cross-section	κ_1	For a beam exposed on three sides with a composite or concrete slab on side four	0,7	0,7
4.2.3.3(9)	The adaptation factor for non-uniform temperature distribution along a beam	κ_2	At the supports of a statically indeterminate beam	0,85	0,85
			In all other cases	1,0	1,0

Table 4 — Directly referenced supporting standards

Reference in ENV 1993-1-2	UK supporting standards
PrEN ISO 834	BS 476-20:1987
PrENV yyy 5-1	BS 476-20:1987 BS 476-21:1987[a]
PrENV yyy 5-2	
PrENV yyy 5-3	
ENV 1991-2-2	DD ENV 1991-2-2
ENV 1993-1-1	DD ENV 1993-1-1
ENV 1993-1-3	DD ENV 1993-1-3
ENV 1994-1-2	DD ENV 1994-1-2
ISO 1000	BS 5555:1993

[a] Further information can be obtained from *Fire protection for structural steel in buildings* [1].

6.1 Chapter 1 General

a) *1.1 Scope*

ENV 1993-1-2:1995 may be used to determine the resistance of stainless steel members subjected to the accidental situation of exposure to fire.

NOTE The performance of stainless steel in fire is usually significantly better than that of mild steel.

6.2 Chapter 2 Basic Principles and Rules

a) *2.1(2)P*

Provided fire protection materials satisfy the recommendations given in ENV 1993-1-2:1995, **3.3.2** the reduction factor k_y given in ENV 1993-1-2:1995, Table 3.1 may be used to calculate the member resistance. In other cases, the reduction factor k_x in ENV 1993-1-2:1995, Table 3.1 should be used.

NOTE Information on fire protection materials that satisfy the recommendations in ENV 1993-1-2:1995, **3.3.2** can be found in the publication *Fire protection for structural steel in buildings* [1].

b) *2.4.4(4)*

In general, the effects of thermal expansion may be neglected. However, consideration should be given to the effect of thermal expansion on bracing members. Further guidance is given in **6.4**b).

6.3 Chapter 4 Structural Fire Design

a) *4.2.1(5)P*

Alternatively, for the design of bolts and welds in the fire situation, the strength reduction factor may conservatively be taken as 80 % of the temperature-dependent value of $k_{x,\theta}$ given in ENV 1993-1-2:1995, Table 3.1.

b) *4.2.2(4)*

Contrary to ENV 1993-1-2:1995, **4.2.2**(4) in a fire design situation all members should be classified as for normal temperature design using the following expression for ε:

$\varepsilon = [(235/f_y)]^{0,5}$

where

f_y is the steel yield strength given in ENV 1993-1-1:1992, **3.2.2.1**.

c) *4.2.3.1*

For tension members, $k_{y,\theta}$ should be replaced in the expression by the temperature-dependent value of $k_{x,\theta}$ given in Table 3.1 of ENV 1993-1-2:1995.

d) *4.2.3.2(2)*

As an alternative to expression (4.6) in ENV 1993-1-2:1995, $\bar{\lambda}_{\theta,\max}$ may be conservatively calculated as:

$\bar{\lambda}_{\theta,\max} = 1.2\ \bar{\lambda}$

where

$\bar{\lambda}$ is the relative slenderness for a normal temperature design.

e) *4.2.3.2(4)*

For a column in a steel frame in which each storey comprises a separate fire compartment with sufficient fire resistance, the buckling length ℓ_{fi} should be taken as 0,7L where the column is continuous at both ends. Where the column is continuous at one end only, the buckling length should be taken as 0,85L. ENV 1993-1-2:1995, Figure 4.1 as modified by **6.3**f) gives the buckling lengths of columns in braced frames.

f) *Figure 4.1 Buckling lengths ℓ_{fi} of columns in braced frames*

The buckling length $\ell_{fi,2}$ of the column in the intermediate storey should be taken as 0,7L_2 and the buckling length $\ell_{fi,4}$ of the top storey column should be taken as 0,85L_4.

g) *4.2.3.3(5)*

The constant 1,2 in expression (**4.11**) of ENV 1993-1-2:1995 should be replaced by the parameter K whose value is given by the following expression:

$$K = \frac{\lambda_{L,T,\theta,com}}{3} + 0,87$$

where

$\lambda_{L,T,\theta,com}$ is given in ENV 1993-1-2:1995, **4.2.3.3**(6) as modified by **6.3**h).

h) *4.2.3.3(6)*

Contrary to ENV 1993-1-2:1995, **4.2.3.3**(6) the value of the non-dimensional slenderness $\bar{\lambda}_{L,T,\theta,com}$ for the temperature $\theta_{a,com}$ should be taken as the value of the normal temperature non-dimensional slenderness $\bar{\lambda}_{LT}$.

i) *4.2.4*

The critical temperature method is an alternative to that given in ENV 1993-1-2:1995, **4.2.3** but cannot be used for the following cases:

— columns;

— tension members;

— unrestrained beams with $\bar{\lambda}_{L,T,fi} > 0,4$.

It can only be used for fully restrained beams, unrestrained beams where $\bar{\lambda}_{LT,fi}$ is not greater than 0,4 and members with Class 4 cross-sections.

ENV 1993-1-2 uses the term "degree of utilization", μ_0, which allows the applied load on a member to be directly related to the critical temperature and is determined from the following expression:

$$\mu_0 = \frac{E_{fi,d}}{R_{fi,d,0}}$$

The variables in the above expression are as defined in ENV 1993-1-2:1995, **4.2.3.5**.

For a beam bending with a uniform temperature distribution over its cross-section and along its length, $R_{fi,d,0}$ is the design moment of resistance of the member at ambient temperature.

For a beam bending with a non-uniform temperature distribution over its cross-section and along its length, $R_{fi,d,0}$ is determined by dividing the member's design moment of resistance at ambient temperature by the adaptation factors κ_1 and κ_2.

j) *4.2.4(4)*

When calculating the degree of utilization, the value of $R_{fi,d,0}$ is determined by dividing the member resistance at normal temperature by both adaptation factors κ_1 and κ_2.

k) *4.2.4(6)*

The method described in this clause may be used to design members constructed from Class 4 open sections.

6.4 Recommendations on subjects not covered in ENV 1993-1-2:1995

a) *Re-use of steel after a fire*

It may be possible to re-use steel after a fire. The guidance in BS 5950-8:1990, appendix C should be followed.

b) *Bracing members*

Bracing members provided stability to the structure in the fire design situation and should be distributed throughout the building so no substantial portion of the structural frame is solely reliant on a single plane of bracing in each of two directions approximately at right angles. Where the stability of the structure is solely dependent on a single plane of bracing or where the bracing systems are located adjacent to a single fire compartment, the temperature in the columns and compressive members forming part of the bracing system should not exceed 450 °C.

c) *Water-filled structures*

The design of water-filled structures should follow the guidance in BS 5950-8:1990, **4.7**.

NOTE Further information may be found in *Fire and steel construction — Water cooled hollow columns* [2].

d) *Portal frames*

The design of portal frames in the fire situation should follow the guidance in BS 5950-8:1990, **4.5**.

NOTE Further information may be found in *Fire and steel construction: The behaviour of steel portal frames in boundary conditions* [3].

e) *Beams with shelf angles*

The fire resistance for beams with shelf angles should be determined in accordance with the guidance in BS 5950-8:1990, appendix E.

f) *Fire resisting walls*

The guidance in BS 5950-8:1990, **4.10** should be followed for the design of fire resisting walls in the fire situation.

g) *Roofs*

Where a roof spans across a fire resisting compartment wall where it is required that strips of the roof should be fire protected on the underside of both sides of the compartment wall, care should be taken to fire stop any gaps between the top of the wall and the underside of the roof cladding to accommodate differential thermal movement in fire. Where practicable, combustible insulation should also be fire stopped along the line of the wall.

h) *Ceilings*

In addition to ENV 1993-1-2:1995, **4.2.5.3** the guidance in BS 5950-8:1990, **4.12** should also be followed for the design of ceilings in the fire situation

Bibliography

BS 5950-8:1990, *Structural use of steelwork in building — Code of practice for fire resistant design.*

[1] Association of Specialist Fire Protection Contractors and Manufacturers Limited, Steel Construction Institute. *Fire protection for structural steel in buildings* (Revised Second Edition), 1992[2)].

[2] BOND, G.V.L. *Fire and steel construction — Water cooled hollow columns*. Steel Construction Institute, 1975[2)].

[3] NEWMAN, G.M. *Fire and steel construction — The behaviour of steel portal frames in boundary conditions* (Second Edition), Steel Construction Institute, 1990[2)].
ISBN 1 870004 49 3

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EUROPEAN PRESTANDARD

PRÉNORME EUROPÉENNE

EUROPÄISCHE VORNORM

ENV 1993-1-2

September 1995

ICS 13.220.20; 91.040.00; 91.080.10

Descriptors: buildings, steel construction, structural steels, design , safety requirements, accident prevention, fire protection, fire resistance, mechanical properties, thermodynamic properties, computation, mechanical strength

English version

Eurocode 3 - Design of steel structures - Part 1-2: General rules - Structural fire design

Eurocode 3 - Calcul des structures en acier - Partie 1-2: Règles générales - Calcul du comportement au feu

Bemessung und Konstruktion von Stahlbauten - Teil 1-2: Allgemeine Regeln - Tragwerksbemessung für den Brandfall

This European Prestandard (ENV) was approved by CEN on 1993-11-05 as a prospective standard for provisional application. The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into an European Standard (EN).

CEN members are required to announce the existance of this ENV in the same way as for an EN and to make the ENV available promptly at national level in an appropriate form. It is permissible to keep conflicting national standards in force (in parallel to the ENV) until the final decision about the possible conversion of the ENV into an EN is reached.

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CEN

European Committee for Standardization
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart,36 B-1050 Brussels

Ref. No. ENV 1993-1-2:1995 E

Contents

Foreword

Objectives of the Eurocodes

(1) The "Structural Eurocodes" comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.

(2) They cover execution and control only to the extent that is necessary to indicate the quality of the construction products, and the standard of the workmanship, needed to comply with the assumptions of the design rules.

(3) Until the necessary set of harmonized technical specifications for products and for methods of testing their performance is available, some of the Structural Eurocodes cover some of these aspects in informative annexes.

Background to the Eurocode programme

(4) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonized technical rules for the design of building and civil engineering works which would initially serve as an alternative to the different rules in force in the various member states and would ultimately replace them. These technical rules became known as the "Structural Eurocodes".

(5) In 1990, after consulting their respective member states, the CEC transferred the work of further development, issue and updating of the Structural Eurocodes to CEN, and the EFTA Secretariat agreed to support the CEN work.

(6) CEN Technical Committee CEN/TC 250 is responsible for all Structural Eurocodes.

Eurocode programme

(7) Work is in hand on the following Structural Eurocodes, each generally consisting of a number of parts:

EN 1991	Eurocode 1	Basis of design and actions on structures;
EN 1992	Eurocode 2	Design of concrete structures;
EN 1993	Eurocode 3	Design of steel structures;
EN 1994	Eurocode 4	Design of composite steel and concrete structures;
EN 1995	Eurocode 5	Design of timber structures;
EN 1996	Eurocode 6	Design of masonry structures;
EN 1997	Eurocode 7	Geotechnical design;
EN 1998	Eurocode 8	Design provisions for earthquake resistance of structures;
EN 1999	Eurocode 9	Design of aluminium alloy structures.

(8) Separate sub-committees have been formed by CEN/TC 250 for the various Eurocodes listed above.

(9) This Part 1.2 of Eurocode 3 is published by CEN as a European Prestandard (ENV) with an initial life of three years.

(10) This Prestandard is intended for experimental application and for the submission of comments.

(11) After approximately two years CEN members will be invited to submit formal comments to be taken into account in determining future actions.

(12) Meanwhile feedback and comments on this Prestandard should be sent to the secretariat of CEN/TC 250/SC 3 at the following address:

BSI Standards
British Standards House
389 Chiswick High Road
London W4 4AL
England

or to your national standards organization.

National Application Documents (NAD's)

(13) In view of the responsibilities of the authorities in member countries for safety, health and other matters covered by the essential requirements of the Construction Products Directive (CPD), certain safety elements in this ENV have been assigned indicative values which are identified by $\boxed{}$ ("boxed values"). The authorities in each member country are expected to review the "boxed values" and may substitute alternative definitive values for these safety elements for use in national application.

(14) Some of the supporting European or International Standards might not be available by the time this Prestandard is issued. It is therefore anticipated that a National Application Document (NAD) giving any substitute definitive values for safety elements, referencing compatible supporting standards and providing guidance on the national application of this Prestandard, will be issued by each member country or its Standards Organization.

(15) It is intended that this Prestandard is used in conjunction with the NAD valid in the country where the building or civil engineering works is located.

Matters specific to this Prestandard

(16) Work on those parts of the Structural Eurocodes covering fire resistance was initiated by the CEC and a first draft of this document was issued in 1990 as a draft "Eurocode 3 : Part 10".

(17) With the transfer of work on Structural Eurocodes to CEN, the responsibility for completing this document passed to CEN Technical Committee CEN/TC 250, sub-committee CEN/TC 250/SC 3.

(18) The scope of Eurocode 3 is defined in 1.1.1 of ENV 1993-1-1. Additional Parts of Eurocode 3 that are planned are indicated in 1.1.3 of ENV 1993-1-1.

(19) The general objectives of fire protection are to limit risks with respect to the individual and society, neighbouring property and, where required, directly exposed property, in the case of fire.

(20) The Structural Eurocodes deal with specific aspects of passive fire protection in terms of designing structures and parts thereof for adequate load-bearing capacity and for limiting fire spread as relevant.

(21) Required functions and levels of performance are generally specified by national authorities - mostly in terms of standard fire resistance rating. Where fire safety engineering for assessing passive and active measures is accepted, requirements by authorities may be less prescriptive and allow alternative strategies.

(22) This Part 1.2, together with ENV 1991-2-2, gives the supplements to ENV 1993-1-1 that are necessary so that structures designed according to this set of Structural Eurocodes may also comply with structural fire resistance requirements.

(23) Supplementary requirements concerning, for example:

- the possible installation and maintenance of sprinkler systems;
- conditions of occupancy of the building or fire compartment;
- the use of approved insulation and coating materials, including their maintenance;

are not given in this document, because they are subject to specification by national authorities.

(24) A method is included in this ENV for applying deformation criteria to the load-bearing structure where the means of protection, or the design criteria for separating members, require such consideration (see 2.1(2), 3.2.1(6), 4.2.1(6) and 4.2.2(5)). However no specific provisions are given for its application. It is intended that where any such provision is considered necessary, it should be included in the NAD.

(25) A full analytical procedure for structural fire design would take into account the behaviour of the structural system at elevated temperatures, the potential heat exposure and the beneficial effects of active fire protection systems, together with the uncertainties associated with these three features and the importance of the structure (consequences of failure).

(26) At the present time it is possible to undertake a procedure for determining adequate performance that incorporates some, if not all, of these parameters and to demonstrate that the structure, or its components, will give adequate performance in a real building fire. However, the principal current procedure in European countries is one based on results from standard fire resistance tests. The grading systems in national regulations that call for specific periods of fire resistance, take into account (though not explicitly) the features and uncertainties described above.

(27) Due to the limitations of the test method, further tests or analyses may be used. Nevertheless, the results of standard fire tests form the bulk of the input to calculation methods for structural fire design. This Prestandard therefore deals in the main with design for the standard fire resistance.

(28) Simple calculation models for steel structures are given in this document and accordingly tabulated data are not included. It is expected that tables and other design aids based on the calculation methods given in this ENV 1993-1-2 will be prepared by interested external organisations.

1 General

1.1 Scope

(1)P This Part 1.2 of ENV 1993 deals with the design of steel structures for the accidental situation of fire exposure and is intended to be used in conjunction with ENV 1993-1-1 and ENV 1991-2-2. This Part 1.2 only identifies differences from, or supplements to, normal temperature design.

(2)P This document deals only with passive methods of fire protection. Active methods are not covered.

(3)P This Part 1.2 applies to structures that, for reasons of general fire safety, are required to avoid premature collapse of the structure in exposure to fire (load-bearing function).

(4)P This Part 1.2 gives principles and application rules (see 1.2) for designing structures for specified requirements in respect of the aforementioned function and the levels of performance.

(5)P This document only applies to structures, or parts of structures, that are within the scope of ENV 1993-1-1 and are designed accordingly.

(6)P The methods given in this document may also be applied to cold-formed thin gauge steel members and sheeting within the scope of ENV 1993-1-3.

(7)P For the fire resistance of composite steel and concrete structures see ENV 1994-1-2.

(8)P The methods given in this document are applicable to any steel grade for which material properties are available.

(10)P The steel properties given in this document apply to steel grades S 235, S 275 and S 355 of EN 10025 and to all steel grades of EN 10113, EN 10155, EN 10210-1 and EN 10219-1.

1.2 Distinction between principles and application rules

(1)P Depending on the character of the individual clauses, distinction is made in this Part between principles and application rules.

(2)P The principles comprise:

- general statements and definitions for which there is no alternative;
- requirements and analytical models for which no alternative is permitted unless specifically stated.

(3) The principles are identified by the letter P following the paragraph number.

(4)P The application rules are generally recognised rules which follow the principles and satisfy their requirements. It is permissible to use alternative design rules different from the application rules given in the Eurocode, provided that it is shown that the alternative rule accords with the relevant principles and have at least the same reliability.

(5) In this Part the application rules are identified by a number in brackets, as in this paragraph.

1.3 Normative references

This European Prestandard incorporates, by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Prestandard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies.

EN 10025 *Hot rolled products of non-alloy structural steels: Technical delivery conditions;*

EN 10113 *Hot rolled products in weldable fine grade structural steels:*

Part 1: *General delivery conditions*;

Part 2: *Delivery conditions for normalized/normalized rolled steels*;

Part 3: *Delivery conditions for thermomechanically rolled steels*;

EN 10155 *Structural steels with improved atmospheric corrosion resistance - Technical delivery conditions*;

EN 10210 *Hot finished structural hollow sections of non-alloy and fine grain structural steels:*

Part 1: *Technical delivery conditions*;

EN 10219 *Cold formed welded structural hollow sections of non-alloy and fine grain structural steels:*

Part 1: *Technical delivery conditions*;

pr EN ISO 834 *Fire resistance: General requirements*;

pr ENV yyy5 *Fire tests on elements of building construction:*

Part 1: *Test method for determining the contribution to the fire resistance of structural members: by horizontal protective membranes*;

Part 2 *Test method for determining the contribution to the fire resistance of structural members: by vertical protective membranes*;

Part 4: *Test method for determining the contribution to the fire resistance of structural members: by applied protection to steel structural elements*;

ENV 1991 *Eurocode 1. Basis of design and actions on structures:*

Part 2.2: *Actions on structures exposed to fire*;

ENV 1993 *Eurocode 3. Design of steel structures:*

Part 1.1: *General rules : General rules and rules for buildings*;

Part 1.3: *General rules : Supplementary rules for cold formed thin gauge steel members and sheeting*;

ENV 1994 *Eurocode 4. Design of composite steel and concrete structures:*

Part 1.2: *General rules : Structural fire design*;

ISO 1000 *SI units*.

1.4 Definitions

For the purposes of this Part 1.2 of ENV 1993, the following definitions apply:

1.4.1 configuration factor: Solid angle within which the radiating environment can be seen from a particular point on the member surface, divided by 2π.

1.4.2 convective heat transfer coefficient: Convective heat flux to the member related to the difference between the bulk temperature of gas bordering the relevant surface of the member and the temperature of that surface.

1.4.3 critical temperature of structural steel: For a given load level, the temperature at which failure is expected to occur in a structural steel element for a uniform temperature distribution.

1.4.4 design fire: A specified fire development assumed for design purposes.

1.4.5 effective yield strength: For a given temperature, the stress level at which the stress-strain relationship of steel is truncated to provide a yield plateau.

1.4.6 external member: Structural member located outside the building, that may be exposed to fire through openings in the building enclosure.

1.4.7 fire compartment: A space within a building, extending over one or several floors, that is enclosed by separating members such that fire spread beyond the compartment is prevented during the relevant fire exposure.

1.4.8 fire protection material: A material that has been shown, by fire resistance tests, to be capable of remaining in position and of providing adequate thermal insulation for the fire resistance period under consideration.

1.4.9 fire resistance: The ability of a structure, a part of a structure or a member to fulfil its required functions (load bearing function and/or separating function) for a specified fire exposure and for a specified period of time.

NOTE: For steel members only the load bearing function applies.

1.4.10 fire wall: A wall separating two spaces (generally two buildings) that is designed for fire resistance and structural stability, including resistance to horizontal loading such that, in case of fire and failure of the structure on one side of the wall, fire spread beyond the wall is avoided.

1.4.11 global structural analysis (for fire): An analysis of the entire structure, when either the entire structure, or only parts of it, are exposed to fire. Indirect fire actions are considered throughout the structure.

1.4.12 indirect fire actions: Thermal expansions, thermal deformations or thermal gradients causing internal forces and moments.

1.4.13 load bearing criterion: A criterion by which the ability of a structure or member to sustain specified actions, during the relevant fire, is assessed.

1.4.14 load bearing function: The ability of a structure or a member to sustain specified actions during the relevant fire, according to defined criteria.

1.4.15 member analysis (for fire): The thermal and mechanical analysis of a structural member exposed to fire in which the member is considered as isolated, with appropriate support and boundary conditions. Indirect fire actions are not considered, except those resulting from thermal gradients.

1.4.16 net heat flux: Energy per unit time and surface area absorbed by members.

1.4.17 normal temperature design: Ultimate limit state design for ambient temperatures according to ENV 1993-1-1 for the fundamental combination according to ENV 1991-1.

1.4.18 protected members: Members for which measures are taken to reduce the temperature rise in the member due to fire.

1.4.19 section factor: For a steel member, the ratio between the exposed surface area and the volume of steel; for an enclosed member, the ratio between the internal surface area of the exposed encasement and the volume of steel.

1.4.20 separating member: Structural or non-structural member (wall or floor) forming part of the enclosure of a fire compartment.

1.4.21 standard fire exposure: Exposure to furnace gases with a temperature that varies with time according to the standard temperature-time curve.

1.4.22 standard fire resistance: The ability of a structure or part of it (usually only members) to fulfil required functions (load-bearing function and/or separating function), for the standard fire exposure for a stated period of time.

> **NOTE:** Standard fire resistance requirements are normally expressed in terms of periods of time, such as 30, 60 or more minutes.

1.4.23 standard temperature-time curve: The nominal temperature-time curve given in ENV 1991-2-2.

1.4.24 structural members: The load-bearing members of a structure, including bracings.

1.4.25 sub-assembly analysis (for fire): The structural analysis of parts of the structure exposed to fire, in which the respective part of the structure is considered as isolated, with appropriate support and boundary conditions. Indirect fire actions within the sub-assembly are considered, but no time-dependent interaction with other parts of the structure.

> **NOTE 1:** Where the effects of indirect fire actions within the sub-assembly are negligible, sub-assembly analysis is equivalent to member analysis.

> **NOTE 2:** Where the effects of indirect fire actions between sub-assemblies are negligible, sub-assembly analysis is equivalent to global structural analysis.

1.4.26 support and boundary conditions: Restraint conditions and applied forces and moments assumed at the supports and boundaries of a structure or part of a structure for the purposes of structural analysis.

1.4.27 temperature analysis: The procedure of determining the temperature development in members on the basis of the thermal actions (net heat flux) and the thermal material properties of the members and of protective surfaces, where relevant.

1.4.28 temperature-time curves: Gas temperatures in the environment of member surfaces as a function of time. They may be:

- **nominal**: Conventional curves, adopted for classification or verification of fire resistance, such as the standard time-temperature curve;
- **parametric**: Determined on the basis of fire models and the specific physical parameters defining the conditions in the fire compartment.

1.4.29 thermal actions: Actions on the structure described by the net heat flux to the members.

1.5 Symbols

1.5.1 Supplementary to ENV 1993-1-1, the following symbols are used:

A_m	is	the surface area of a member per unit length;
A_p	is	the area of the inner surface of the fire protection material per unit length of the member;
E_a	is	the modulus of elasticity of steel for normal temperature design;
$E_{a,\theta}$	is	the slope of the linear elastic range for steel at elevated temperature θ_a ;
$E_{fi,d}$	is	the design effect of actions in the fire situation;
$R_{d,\theta}$	is	the design resistance at uniform elevated material temperature θ ;
$R_{fi,d}$	is	the design resistance in the fire situation;
$R_{fi,d,t}$	is	the design value of a resistance in the fire situation, at time t ;
T	is	the temperature [K] (cf θ temperature [°C]);
V	is	the volume of a member per unit length;
$X_{fi,d}$	is	the design material property in the fire situation;
X_k	is	the characteristic value of a material property;
$X_{k,\theta}$	is	the characteristic value of a material property at elevated temperature θ ;
c	is	the specific heat [J/kgK];
d_p	is	the thickness of fire protection material;
$f_{p,\theta}$	is	the proportional limit for steel at elevated temperature θ_a ;
$f_{y,\theta}$	is	the effective yield strength of steel at elevated temperature θ_a ;
$\dot{h}_{net,d}$	is	the design value of the net heat flux per unit area;
k_θ	is	the relative value of a strength or deformation property of steel at elevated temperature θ_a ;
ℓ	is	the length at 20 °C ;
$\Delta\ell$	is	the temperature induced expansion;
t	is	the time in fire exposure [minutes];
Δt	is	the time interval [seconds];
η_{fi}	is	the reduction factor for design load level in the fire situation;
θ	is	the temperature [°C] (cf T temperature [K]);
κ	is	the adaptation factor;
λ	is	the thermal conductivity [W/mK];
μ_0	is	the degree of utilisation at time $t = 0$.

1.5.2 Supplementary to ENV 1993-1-1, the following subscripts are used:

a	steel;
c	connection;
fi	value relevant for the fire situation;
m	member;
p	fire protection material;
t	dependent on time;
θ	dependent on temperature.

1.5.3 Additional symbols are used in annexes C and D. These are defined where they first occur.

1.6 Units

(1)P SI units shall be used in conformity with ISO 1000.

(2) Supplementary to ENV 1993-1-1, the following units are recommended for use in calculations:

- area	:	m^2 ;
- insulation thickness	:	m;
- temperature	:	°C;
- absolute temperature	:	K;
- temperature difference	:	K;
- specific heat	:	J/kgK;
- coefficient of thermal conductivity	:	W/mK.

2 Basic principles and rules

2.1 Performance requirements

(1)P Where mechanical resistance in the case of fire is required, steel structures shall be designed and constructed in such a way that they maintain their load bearing function during the relevant fire exposure.

(2)P Deformation criteria shall be applied where the means of protection, or the design criteria for separating members, require consideration of the deformation of the load bearing structure.

2.2 Actions

(1)P The thermal and mechanical actions shall be obtained from ENV 1991-2-2.

(2) Where rules given in this Part 1.2 of ENV 1993 are valid only for the standard fire exposure, this is identified in the relevant clauses.

2.3 Design values of material properties

(1)P Design values of thermal and mechanical material properties $X_{fi,d}$ are defined as follows:

- thermal properties for thermal analysis:
  - if an increase of the property is favourable for safety:

$$X_{fi,d} = X_{k,\theta} / \gamma_{M,fi} \tag{2.1a}$$

  - if an increase of the property is unfavourable for safety:

$$X_{fi,d} = \gamma_{M,fi} X_{k,\theta} \tag{2.1b}$$

- strength and deformation properties for structural analysis:

$$X_{fi,d} = k_\theta X_k / \gamma_{M,fi} \tag{2.1c}$$

where:

$X_{k,\theta}$ is the characteristic value of a material property in fire design, generally dependent on the material temperature, see section 3;

X_k is the characteristic value of a strength or deformation property (generally f_k or E_k) for normal temperature design to ENV 1993-1-1;

k_θ is the reduction factor for a strength or deformation property $(X_{k,\theta} / X_k)$, dependent on the material temperature, see 3.2.1;

$\gamma_{M,fi}$ is the partial safety factor for the relevant material property, for the fire situation.

(2)P For thermal properties of steel, the partial safety factor for the fire situation shall be taken as:

$$\gamma_{M,fi} = \boxed{1,0}$$

(3)P For mechanical properties of steel, the partial safety factor for the fire situation shall be taken as:

$$\gamma_{M,fi} = \boxed{1,0}$$

2.4 Assessment methods

2.4.1 General

(1)P The model of the structural system adopted for design to this Part 1.2 of ENV 1993 shall reflect the expected performance of the structure in fire exposure.

(2)P The structural analysis for the fire situation may be carried out using one of the following:

- global structural analysis, see 2.4.2;
- analysis of portions of the structure, see 2.4.3;
- member analysis, see 2.4.4.

(3) For verifying standard fire resistance requirements, a member analysis is sufficient.

(4)P As an alternative to the use of calculation models, design may be based on the results of tests.

2.4.2 Global structural analysis

(1)P Global structural analysis for the fire situation shall be carried out, taking into account the relevant failure mode in fire exposure, the temperature-dependent material properties and member stiffnesses.

(2)P It shall be verified that, for the relevant duration of fire exposure t :

$$E_{fi,d} \leq R_{fi,d,t} \tag{2.2}$$

where:

$E_{fi,d}$ is the design effect of actions for the fire situation, determined in accordance with ENV 1991-2-2, including the effects of thermal expansions and deformations;

$R_{fi,d,t}$ is the corresponding design resistance at elevated temperatures.

2.4.3 Analysis of portions of the structure

(1)P As an alternative to global structural analysis of the entire structure for various fire situations, structural analysis of subassemblies comprising appropriate portions of the structure may be carried out in accordance with 2.4.2.

(2) The reactions at supports and internal forces and moments at boundaries of subassemblies applicable at time $t = 0$ may be assumed to remain unchanged throughout the fire exposure.

(3) As an alternative to carrying out a global structural analysis for the fire situation at time $t = 0$, the reactions at supports and internal forces and moments at boundaries of subassemblies may be obtained from a global structural analysis for normal temperature design by using:

$$E_{fi,d} = \eta_{fi} E_d \tag{2.3}$$

where:

E_d is the design value of the corresponding force or moment for normal temperature design, for the fundamental combination of actions given by expression (2.9) in ENV 1993-1-1;

η_{fi} is the reduction factor for the design load level for the fire situation.

(4) The reduction factor for the design load level for the fire situation η_{fi} is given by:

$$\eta_{fi} = \frac{\gamma_{GA} G_k + \psi_{1,1} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \qquad (2.4)$$

where:

$Q_{k,1}$ is the principal variable load;

γ_{GA} is the partial factor for permanent actions in accidental design situations;

$\psi_{1,1}$ is the combination factor for frequent values, see table 9.3 in ENV 1991-1.

NOTE: Figure 2.1 shows the variation of the reduction factor η_{fi} with the load ratio $Q_{k,1} / G_k$ for different values of the factor $\psi_{1,1}$ for γ_{GA} = 1,0 with γ_G = 1,35 and γ_Q = 1,5.

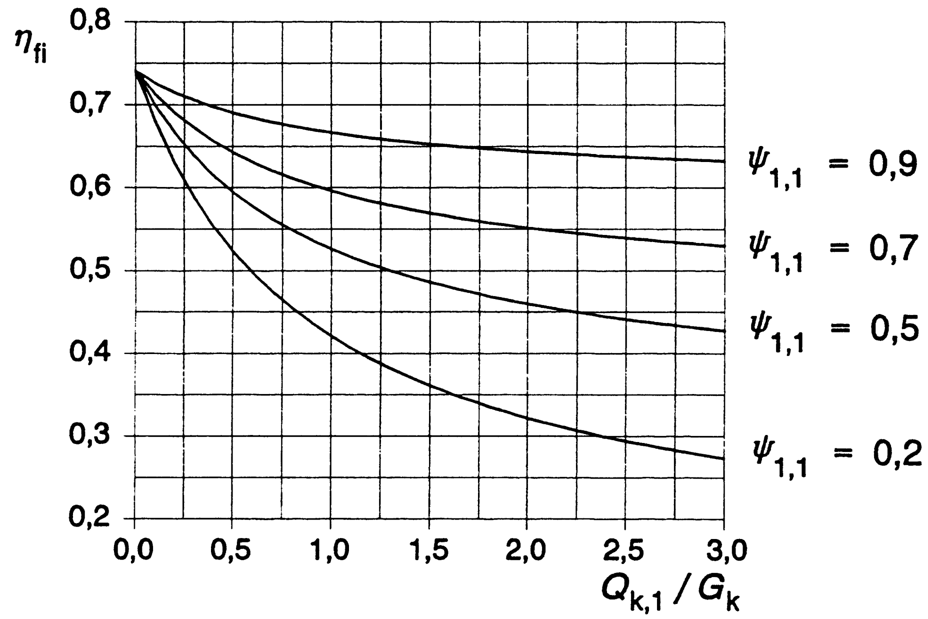


Figure 2.1: Variation of the reduction factor η_{fi} with the load ratio $Q_{k,1} / G_k$

2.4.4 Member analysis

(1)P As an alternative to global structural analysis, individual members may be analysed for the fire situation. The restraint conditions at supports and ends of members applicable at time t = 0 may generally be assumed to remain unchanged throughout the fire exposure. Where different conditions apply, this is identified in the relevant provisions.

(2) The internal forces and moments at supports and ends of members applicable at time t = 0 may be assumed to remain unchanged throughout the fire exposure.

(3) As an alternative to carrying out a global structural analysis for the fire situation at time t = 0, the internal forces and moments at supports and ends of members may be obtained from a global structural analysis for normal temperature design by using expression (2.3).

(4) Only the effects of thermal deformations resulting from thermal gradients across the cross-section need be considered. The effects of thermal expansions of the members may be neglected.

3 Material properties

3.1 General

(1)P The thermal and mechanical properties of steel shall be determined from the following. For materials not included herein, reference shall be made to the relevant CEN product standard or European Technical Approval (ETA).

(2)P The values of material properties given in section 3 shall be treated as characteristic values, see 2.3(1).

(3)P The mechanical properties of steel at 20 °C shall be taken as those given in ENV 1993-1-1 for normal temperature design.

3.2 Mechanical properties of steel

3.2.1 Strength and deformation properties

(1)P For heating rates between 2 and 50 K/min, the strength and deformation properties of steel at elevated temperatures shall be obtained from the stress-strain relationship given in figure 3.1.

(2) This relationship should be used to determine resistance to tension, compression, moment or shear.

(3) Table 3.1 gives the reduction factors, relative to the appropriate value at 20 °C, for the stress-strain relationship for steel at elevated temperatures given in figure 3.1, as follows:

- effective yield strength, relative to yield strength at 20 °C: $k_{y,\theta} = f_{y,\theta} / f_y$
- proportional limit, relative to yield strength at 20 °C: $k_{p,\theta} = f_{p,\theta} / f_y$
- slope of linear elastic range, relative to slope at 20 °C: $k_{E,\theta} = E_{a,\theta} / E_a$

(4) The variation of these three reduction factors with temperature is illustrated in figure 3.2.

(5)P Alternatively, for temperatures below 400 °C, the stress-strain relationship specified in (1) may be extended by the strain-hardening option given in annex B, provided that the proportions of the cross-section are not such that local buckling is liable to prevent attainment of the increased strain and that the member is adequately restrained to prevent buckling.

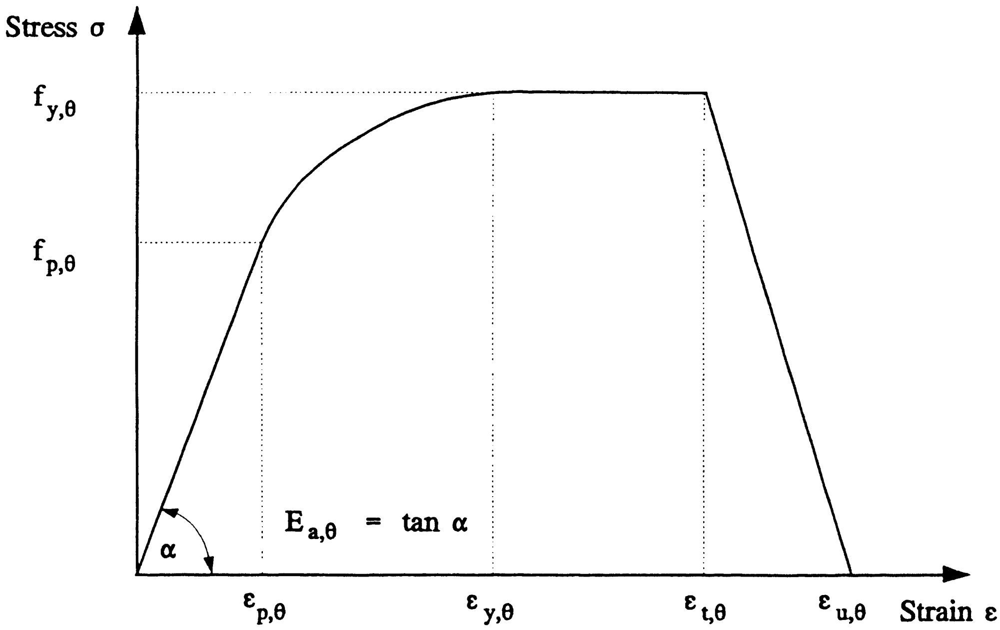
(6) Table 3.1 also gives values of a modified reduction factor $k_{x,\theta}$ for use in place of $k_{y,\theta}$ where it is necessary to satisfy deformation criteria.

3.2.2 Unit mass

(1)P The unit mass of steel ρ_a may be considered to be independent of the steel temperature. The following value may be taken:

$$\rho_a = 7850\,\text{kg/m}^3$$

Strain range	Stress σ	Tangent modulus
$\varepsilon \leq \varepsilon_{p,\theta}$	$\varepsilon E_{a,\theta}$	$E_{a,\theta}$
$\varepsilon_{p,\theta} < \varepsilon < \varepsilon_{y,\theta}$	$f_{p,\theta} - c + (b/a)\left[a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2\right]^{0,5}$	$\dfrac{b(\varepsilon_{y,\theta} - \varepsilon)}{a\left[a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2\right]^{0,5}}$
$\varepsilon_{y,\theta} \leq \varepsilon \leq \varepsilon_{t,\theta}$	$f_{y,\theta}$	0
$\varepsilon_{t,\theta} < \varepsilon < \varepsilon_{u,\theta}$	$f_{y,\theta}\left[1 - (\varepsilon - \varepsilon_{t,\theta})/(\varepsilon_{u,\theta} - \varepsilon_{t,\theta})\right]$	-
$\varepsilon = \varepsilon_{u,\theta}$	0,00	-
Parameters	$\varepsilon_{p,\theta} = f_{p,\theta}/E_{a,\theta}$ $\quad \varepsilon_{y,\theta} = 0,02$	$\varepsilon_{t,\theta} = 0,15$ $\quad \varepsilon_{u,\theta} = 0,20$
Functions	$a^2 = (\varepsilon_{y,\theta} - \varepsilon_{p,\theta})(\varepsilon_{y,\theta} - \varepsilon_{p,\theta} + c/E_{a,\theta})$ $b^2 = c(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a,\theta} + c^2$ $c = \dfrac{(f_{y,\theta} - f_{p,\theta})^2}{(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a,\theta} - 2(f_{y,\theta} - f_{p,\theta})}$	



Key:

$f_{y,\theta}$	is	the effective yield strength;
$f_{p,\theta}$	is	the proportional limit;
$E_{a,\theta}$	is	the slope of the linear elastic range;
$\varepsilon_{p,\theta}$	is	the strain at the proportional limit;
$\varepsilon_{y,\theta}$	is	the yield strain;
$\varepsilon_{t,\theta}$	is	the limiting strain for yield strength;
$\varepsilon_{u,\theta}$	is	the ultimate strain.

Figure 3.1: Stress-strain relationship for steel at elevated temperatures.

Table 3.1: Reduction factors for stress-strain relationship of steel at elevated temperatures.

Steel temperature θ_a	Reduction factors at temperature θ_a relative to the value of f_y or E_a at 20 °C			
	Reduction factor (relative to f_y) for effective yield strength $k_{y,\theta} = f_{y,\theta}/f_y$	Modified factor (relative to f_y) for satisfying deformation criteria $k_{x,\theta} = f_{x,\theta}/f_y$	Reduction factor (relative to f_y) for proportional limit $k_{p,\theta} = f_{p,\theta}/f_y$	Reduction factor (relative to E_a) for the slope of the linear elastic range $k_{E,\theta} = E_{a,\theta}/E_a$
20 °C	1,000	1,000	1,000	1,000
100 °C	1,000	1,000	1,000	1,000
200 °C	1,000	0,922	0,807	0,900
300 °C	1,000	0,845	0,613	0,800
400 °C	1,000	0,770	0,420	0,700
500 °C	0,780	0,615	0,360	0,600
600 °C	0,470	0,354	0,180	0,310
700 °C	0,230	0,167	0,075	0,130
800 °C	0,110	0,087	0,050	0,090
900 °C	0,060	0,051	0,0375	0,0675
1000 °C	0,040	0,034	0,0250	0,0450
1100 °C	0,020	0,017	0,0125	0,0225
1200 °C	0,000	0,000	0,0000	0,0000

NOTE: For intermediate values of the steel temperature, linear interpolation may be used.

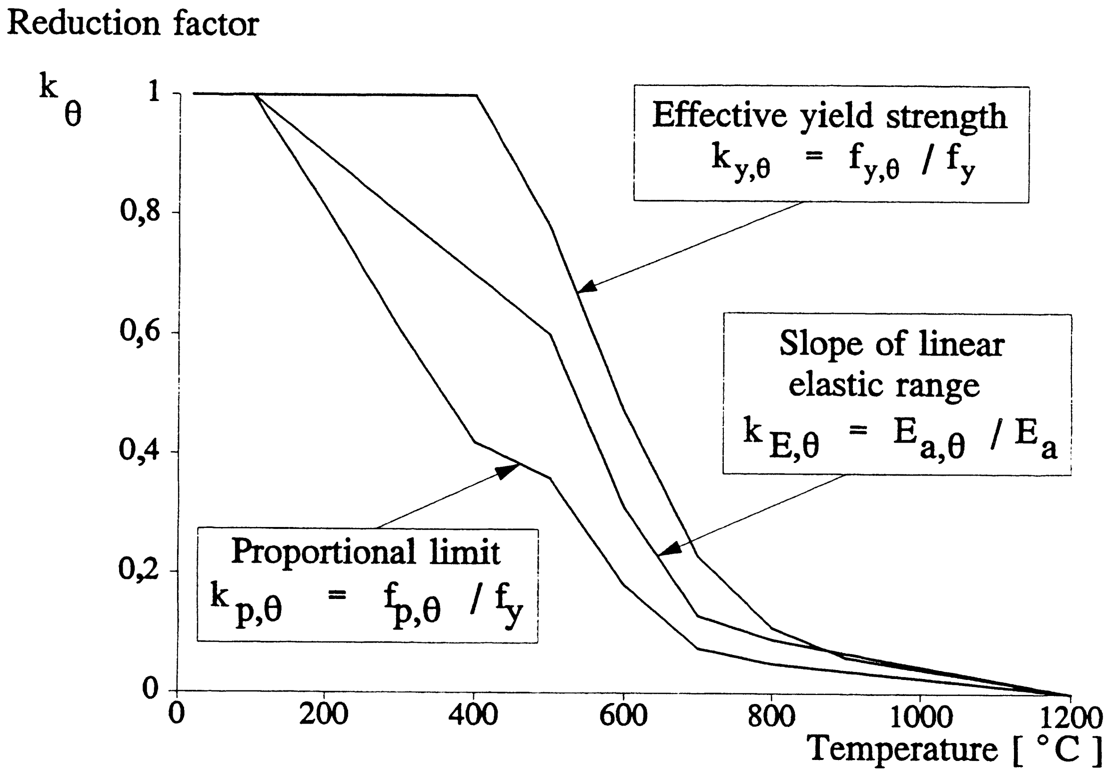


Figure 3.2: Reduction factors for the stress-strain relationship of steel at elevated temperatures

3.3 Thermal properties

3.3.1 Steel

3.3.1.1 Thermal elongation

(1)P The thermal elongation of steel $\Delta\ell / \ell$ may be determined from the following:

- for $20\,°C \leq \theta_a < 750\,°C$:

$$\Delta\ell / \ell = 1{,}2 \times 10^{-5}\,\theta_a + 0{,}4 \times 10^{-8}\,\theta_a^2 - 2{,}416 \times 10^{-4} \qquad (3.1a)$$

- for $750\,°C \leq \theta_a \leq 860\,°C$:

$$\Delta\ell / \ell = 1{,}1 \times 10^{-2} \qquad (3.1b)$$

- for $860\,°C < \theta_a \leq 1200\,°C$:

$$\Delta\ell / \ell = 2 \times 10^{-5}\,\theta_a - 6{,}2 \times 10^{-3} \qquad (3.1c)$$

where:

ℓ is the length at 20 °C;

$\Delta\ell$ is the temperature induced expansion;

θ_a is the steel temperature [°C].

(2) The variation of the thermal elongation with temperature is illustrated in figure 3.3.

(3)P In simple calculation models (see 4.2) the relationship between thermal elongation and steel temperature may be considered to be constant. In this case the elongation may be determined from:

$$\Delta\ell / \ell = 14 \times 10^{-6}\,(\theta_a - 20) \qquad (3.1d)$$

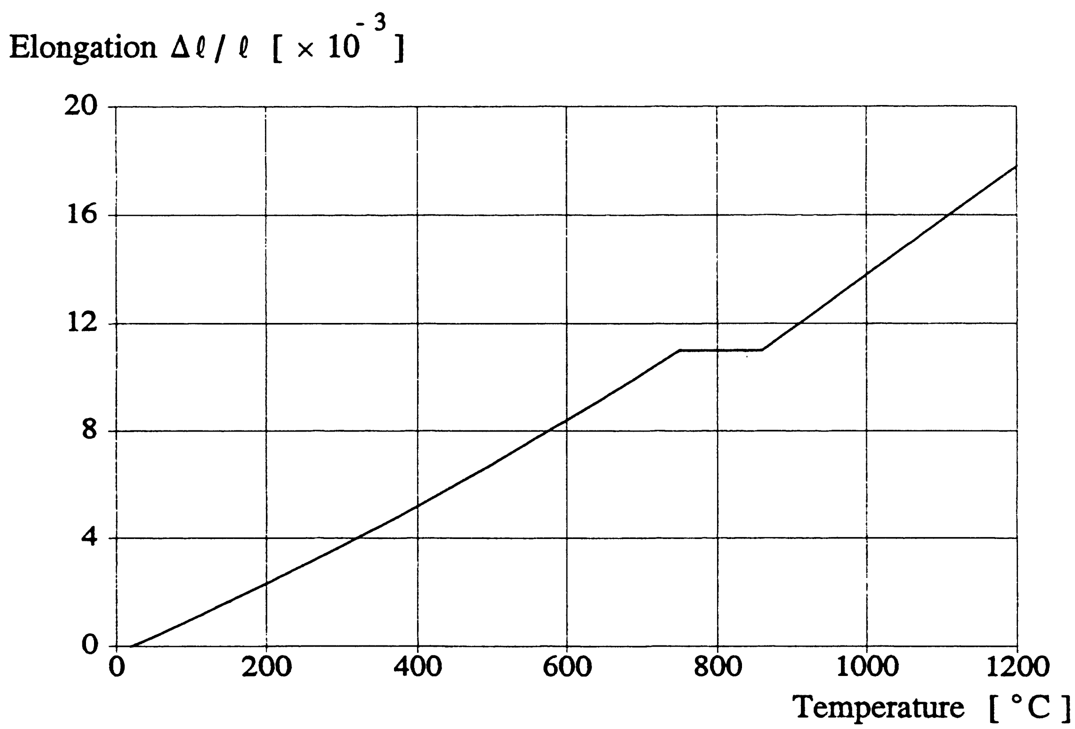


Figure 3.3: Thermal elongation of steel as a function of the temperature

3.3.1.2 Specific heat

(1)P The specific heat of steel c_a may be determined from the following:

- for $20\,°C \leq \theta_a < 600\,°C$:

$$c_a = 425 + 7{,}73 \times 10^{-1}\,\theta_a - 1{,}69 \times 10^{-3}\,\theta_a^2 + 2{,}22 \times 10^{-6}\,\theta_a^3 \text{ J/kgK} \tag{3.2a}$$

- for $600\,°C \leq \theta_a < 735\,°C$:

$$c_a = 666 + \frac{13002}{738 - \theta_a} \text{ J/kgK} \tag{3.2b}$$

- for $735\,°C \leq \theta_a < 900\,°C$:

$$c_a = 545 + \frac{17820}{\theta_a - 731} \text{ J/kgK} \tag{3.2c}$$

- for $900\,°C \leq \theta_a \leq 1200\,°C$:

$$c_a = 650 \text{ J/kgK} \tag{3.2d}$$

where:

θ_a is the steel temperature [°C].

(2) The variation of the specific heat with temperature is illustrated in figure 3.4.

(3)P In simple calculation models (see 4.2) the specific heat may be considered to be independent of the steel temperature. In this case the following value may be taken:

$$c_a = 600 \text{ J/kgK} \tag{3.2e}$$

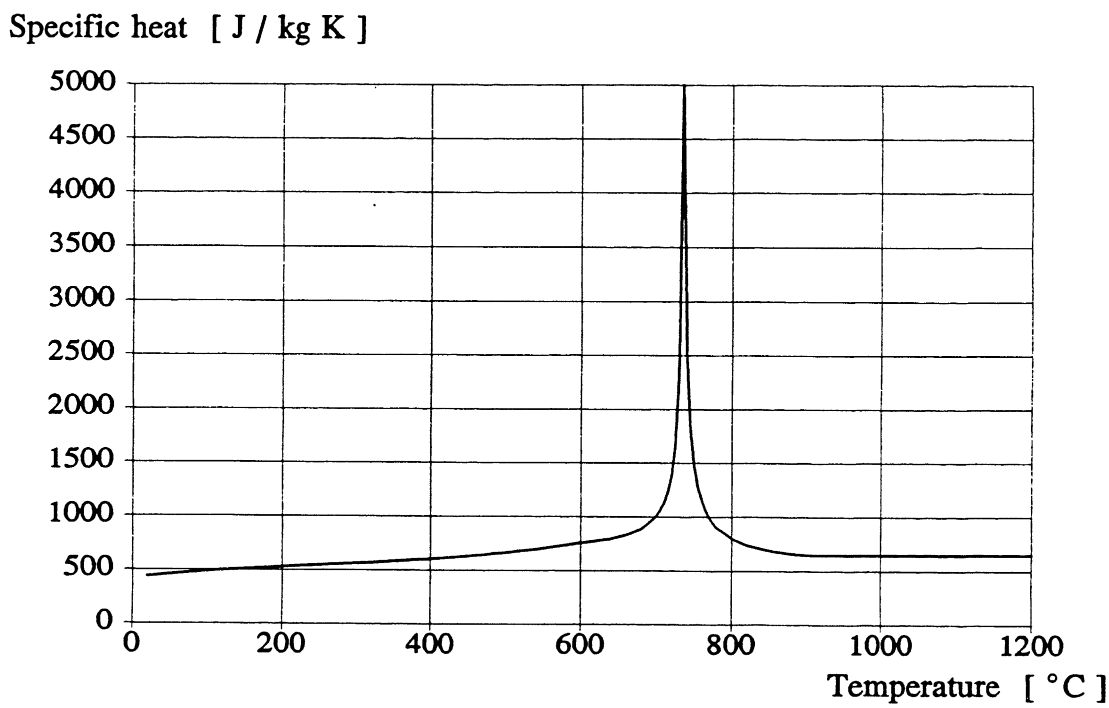


Figure 3.4: Specific heat of steel as a function of the temperature

3.3.1.3 Thermal conductivity

(1)P The thermal conductivity of steel λ_a may be determined from the following:

- for $20\,°C \leq \theta_a < 800\,°C$:

$$\lambda_a = 54 - 3{,}33 \times 10^{-2}\,\theta_a \text{ W/mK} \qquad (3.3a)$$

- for $800\,°C \leq \theta_a \leq 1200\,°C$:

$$\lambda_a = 27{,}3 \text{ W/mK} \qquad (3.3b)$$

where:

θ_a is the steel temperature [°C].

(2) The variation of the thermal conductivity with temperature is illustrated in figure 3.5.

(3)P In simple calculation models (see 4.2) the thermal conductivity may be considered to be independent of the steel temperature. In this case the following value may be taken:

$$\lambda_a = 45 \text{ W/mK} \qquad (3.3c)$$

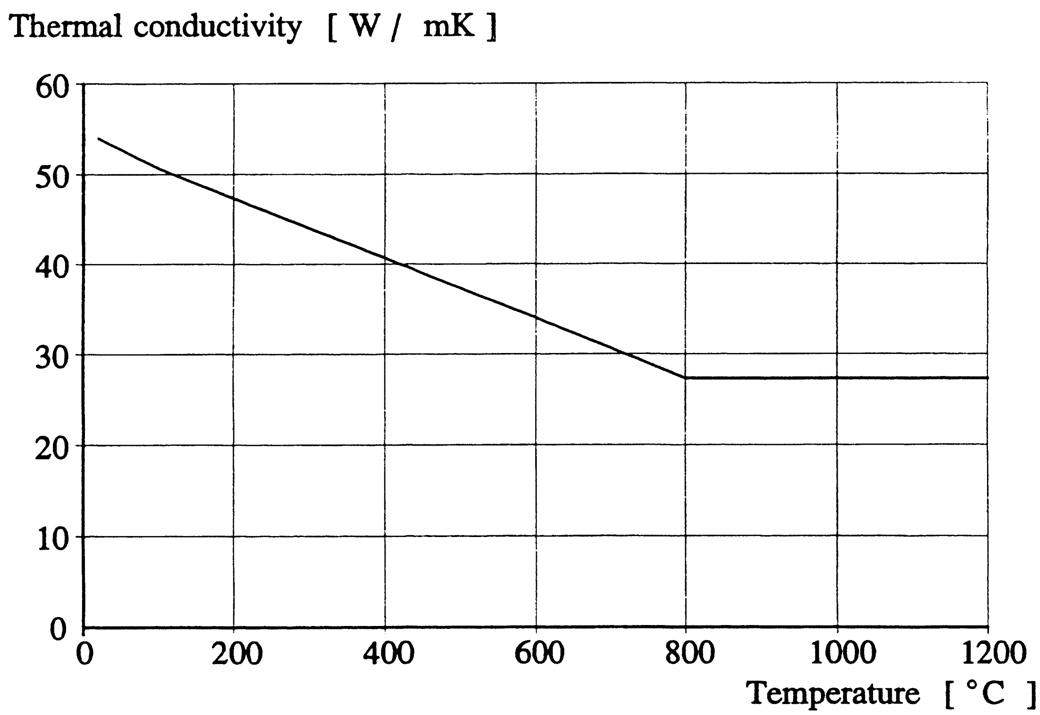


Figure 3.5: Thermal conductivity of steel as a function of the temperature

3.3.2 Fire protection materials

(1)P The properties and performance of fire protection materials shall be assessed using the test procedures given in pr ENV yyy5-1, pr ENV yyy5-2 or pr ENV yyy5-4 as appropriate.

NOTE: This assumes that these standards will include a requirement that the fire protection materials shall remain coherent and cohesive to their supports throughout the relevant fire exposure.

4 Structural fire design

4.1 General

(1)P Steelwork may be either:

- unprotected;
- insulated by fire protection material;
- protected by heat screens;
- protected by any other method that limits the temperature rise of the steel.

NOTE: Examples of other methods include water filling or partial protection in walls and floors.

(2)P The assessment of structural behaviour in a fire design situation shall be based on one of the following approaches, or on a combination of them:

- simple calculation models applied to individual members;
- advanced calculation models;
- testing.

(3)P Simple calculation models are simplified design methods which give conservative results.

(4)P Advanced calculation models are design methods in which engineering principles are applied in a realistic manner to specific applications.

(5)P Where no simple calculation model is given, it is necessary to use either a design method based on an advanced calculation model or a method based on test results.

4.2 Simple calculation models

4.2.1 General

(1)P The load-bearing function of a steel member shall be assumed to be maintained after a time t in a given fire if:

$$E_{\mathrm{fi,d}} \leq R_{\mathrm{fi,d,t}} \tag{4.1}$$

where:

$E_{\mathrm{fi,d}}$ is the design effect of actions for the fire design situation, according to ENV 1991-2-2;

$R_{\mathrm{fi,d,t}}$ is the corresponding design resistance of the steel member, for the fire design situation, at time t.

(2)P The design resistance $R_{\mathrm{fi,d,t}}$ at time t shall be determined for the temperature distribution in the cross-section by modifying the design resistance for normal temperature design to ENV 1993-1-1, to take account of the mechanical properties of steel at elevated temperatures, see 4.2.3.

NOTE: In 4.2.3 $R_{\mathrm{fi,d,t}}$ becomes $M_{\mathrm{fi,t,Rd}}$, $N_{\mathrm{fi,t,Rd}}$ etc (separately or in combination) and the corresponding values of $M_{\mathrm{fi,Ed}}$, $N_{\mathrm{fi,Ed}}$ etc represent $E_{\mathrm{fi,d}}$.

(3)P Alternatively, by using a uniform temperature distribution, the verification may be carried out in the temperature domain, see 4.2.4.

(4) Net-section failure at fastener holes need not be considered, provided that there is a fastener in each hole, because the steel temperature is lower at connections due to the presence of additional material.

(5)P The resistance of connections between members need not be checked provided that the thermal resistance $(d_f/\lambda_f)_c$ of the fire protection of the connection is not less than the minimum value of the thermal resistance $(d_f/\lambda_f)_m$ of the fire protection of any of the steel members joined by that connection, where:

d_f is the thickness of the fire protection material — take $d_f = 0$ for unprotected members;

λ_f is the effective thermal conductivity of the fire protection material.

(6) Where the means of protection, or the design criteria for separating members, require the consideration of deformation criteria, see 2.1(2), the verification should be carried out as specified in 4.2.3, but substituting the reduction factors $k_{y,\theta}$ and $k_{y,\theta,max}$ for effective yield strength at temperatures θ_a and $\theta_{a,max}$ by modified reduction factors $k_{x,\theta}$ and $k_{x,\theta,max}$, see 3.2.1 and table 3.1.

4.2.2 Classification of cross-sections

(1) In a fire design situation, the classification of cross-sections as defined in 5.3 of ENV 1993-1-1 should take due account of the stress-strain relationship of steel at the relevant steel temperature.

(2) A compression member may be classified as for normal temperature design, without any change.

(3) A simply supported beam with a composite or concrete slab on the compression flange, but exposed on the other three sides, may be classified as for normal temperature design, without any change.

(4) Any other member may be classified as for normal temperature design, but using a modified value of ε in table 5.3.1 of ENV 1993-1-1, given by:

$$\varepsilon = [(235/f_y)(k_{E,\theta}/k_{y,\theta})]^{0,5} \tag{4.2}$$

(5) When considering deformation criteria, see 2.1(2), a member may be classified as for normal temperature design, without any change.

4.2.3 Resistance

4.2.3.1 Tension members

(1)P The design resistance $N_{fi,t,Rd}$ at time t of a tension member with a non-uniform temperature distribution across the cross-section may be determined from:

$$N_{fi,t,Rd} = \sum_{i=1}^{n} A_i k_{y,\theta,i} f_y/\gamma_{M,fi} \tag{4.3}$$

where:

A_i is an elemental area of the cross-section with a temperature θ_i;

$k_{y,\theta,i}$ is the reduction factor for the yield strength of steel at temperature θ_i, see 3.2.1;

θ_i is the temperature in the elemental area A_i.

(2) The design resistance $N_{fi,t,Rd}$ at time t of a tension member with a non-uniform temperature distribution may conservatively be taken as equal to the design resistance $N_{fi,\theta,Rd}$ of a tension member with a uniform steel temperature θ_a equal to the maximum steel temperature $\theta_{a,max}$ reached at time t.

(3) The design resistance $N_{fi,\theta,Rd}$ of a tension member with a uniform temperature θ_a should be determined from:

$$N_{fi,\theta,Rd} = k_{y,\theta} N_{Rd} [\gamma_{M,1} / \gamma_{M,fi}] \quad (4.4)$$

where:

$k_{y,\theta}$	is	the reduction factor for the yield strength of steel at temperature θ_a, see 3.2.1;
N_{Rd}	is	the design resistance of the gross cross-section $N_{p\ell,Rd}$ for normal temperature design, according to 5.4.3 of ENV 1993-1-1.

4.2.3.2 Compression members with Class 1, Class 2 or Class 3 cross-sections

(1) The design buckling resistance $N_{b,fi,t,Rd}$ at time t of a compression member with a Class 1, Class 2 or Class 3 cross-section should be determined from:

$$N_{b,fi,t,Rd} = [\chi_{fi} / 1,2] A \, k_{y,\theta,max} f_y / \gamma_{M,fi} \quad (4.5)$$

where:

χ_{fi}	is	the reduction factor for flexural buckling in the fire design situation;
$k_{y,\theta,max}$	is	the reduction factor from 3.2.1 for the yield strength of steel at the maximum steel temperature $\theta_{a,max}$ reached at time t.

NOTE: The constant 1,2 in this expression is a correction factor that allows for a number of effects, including the difference in the strain at failure compared to $\varepsilon_{y,\theta}$. The value is empirical.

(2) The value of χ_{fi} should be taken as the lesser of the values of $\chi_{y,fi}$ and $\chi_{z,fi}$ determined as given in 5.5.1 of ENV 1993-1-1, except using:

- buckling curve c, irrespective of the type of cross-section or the axis of buckling;
- the buckling length ℓ_{fi} for the fire design situation in place of ℓ;
- the non-dimensional slenderness $\bar{\lambda}_{\theta,max}$ for the temperature $\theta_{a,max}$, given by:

$$\bar{\lambda}_{\theta,max} = \bar{\lambda} [k_{y,\theta,max} / k_{E,\theta,max}]^{0,5} \quad (4.6)$$

where:

$k_{y,\theta,max}$	is	the reduction factor from 3.2.1 for the yield strength of steel at the maximum steel temperature $\theta_{a,max}$ reached at time t;
$k_{E,\theta,max}$	is	the reduction factor from 3.2.1 for the slope of the linear elastic range at the maximum steel temperature $\theta_{a,max}$ reached at time t.

(3) The buckling length ℓ_{fi} of a column for the fire design situation should generally be determined as for normal temperature design. However, in a braced frame the buckling length ℓ_{fi} of a column length may be determined by considering it as fixed in direction at continuous or semi-continuous connections to the column lengths in the fire compartments above and below, provided that the fire resistance of the building components that separate these fire compartments is not less than the fire resistance of the column.

(4) In the case of a steel frame in which each storey comprises a separate fire compartment with sufficient fire resistance, in an intermediate storey the buckling length of a column $\ell_{fi} = 0,5L$ and in the top storey the buckling length $\ell_{fi} = 0,7L$, where L is the system length in the relevant storey, see figure 4.1.

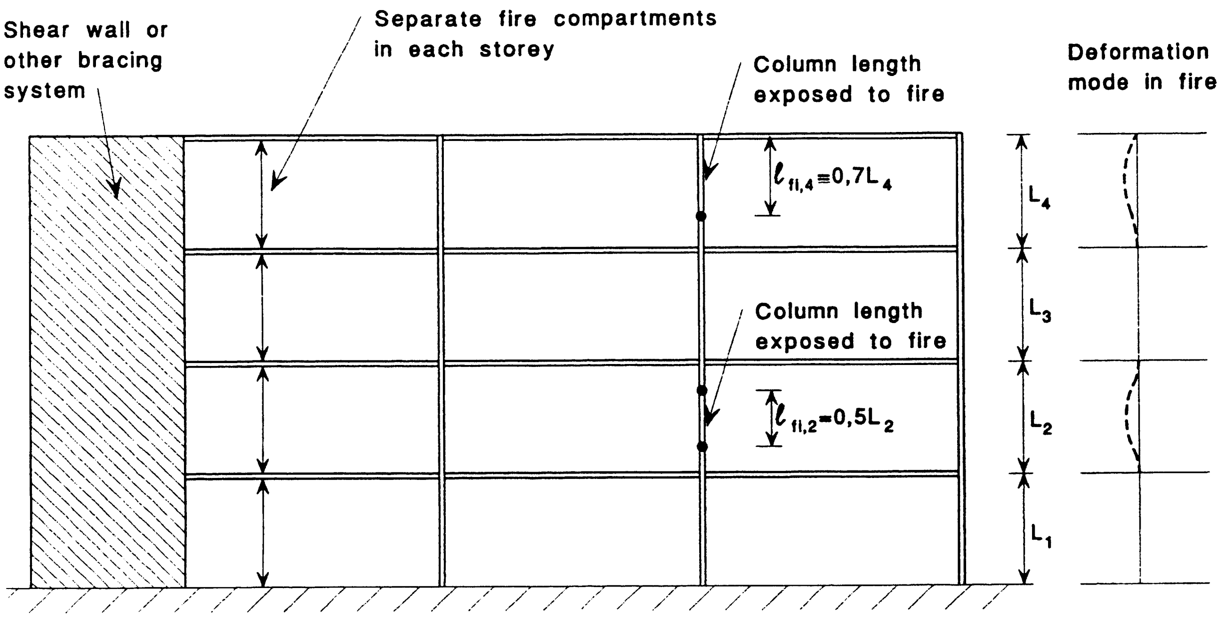


Figure 4.1: Buckling lengths ℓ_{fi} of columns in braced frames

4.2.3.3 Beams with Class 1 or Class 2 cross-sections

(1)P The design moment resistance $M_{fi,t,Rd}$ at time t of a Class 1 or Class 2 cross-section with a non-uniform temperature distribution across the cross-section may be determined from:

$$M_{fi,t,Rd} = \sum_{i=1}^{n} A_i z_i k_{y,\theta,i} f_{y,i} / \gamma_{M,fi} \tag{4.7}$$

where:

- z_i is the distance from the plastic neutral axis to the centroid of the elemental area A_i;
- $f_{y,i}$ is the nominal yield strength f_y for the elemental area A_i taken as positive on the compression side of the plastic neutral axis and negative on the tension side;
- A_i and $k_{y,\theta,i}$ are as defined in 4.2.3.1(1).

(2)P The plastic neutral axis of a Class 1 or Class 2 cross-section with a non-uniform temperature distribution is the axis perpendicular to the plane of bending that satisfies the following criterion:

$$\sum_{i=1}^{n} A_i k_{y,\theta,i} f_{y,i} = 0 \tag{4.8}$$

(3) Alternatively, the design moment resistance $M_{fi,t,Rd}$ at time t of a Class 1 or Class 2 cross-section in a member with a non-uniform temperature distribution, may conservatively be determined from:

$$M_{fi,t,Rd} = M_{fi,\theta,Rd} / \kappa_1 \kappa_2 \tag{4.9}$$

where:

- $M_{fi,\theta,Rd}$ is the design moment resistance of the cross-section for a uniform temperature θ_a equal to the maximum temperature $\theta_{a,max}$ reached in the cross-section at time t;
- κ_1 is an adaptation factor for non-uniform temperature across the cross-section, see (8);
- κ_2 is an adaptation factor for non-uniform temperature along the beam, see (9).

(4)P The design moment resistance $M_{fi,\theta,Rd}$ of a Class 1 or Class 2 cross-section with a uniform temperature θ_a may be determined from:

$$M_{fi,\theta,Rd} = k_{y,\theta}[\gamma_{M,1}/\gamma_{M,fi}]M_{Rd} \quad (4.10)$$

where:

M_{Rd} is the plastic moment resistance of the gross cross-section $M_{p\ell,Rd}$ for normal temperature design, according to 5.4.5 of ENV 1993-1-1 or the reduced moment resistance for normal temperature design, allowing for the effects of shear if necessary, according to 5.4.7 of ENV 1993-1-1;

$k_{y,\theta}$ is the reduction factor for the yield strength of steel at temperature θ_a, see 3.2.1

(5) Provided that the non-dimensional slenderness $\bar{\lambda}_{LT,\theta,com}$ for the maximum temperature in the compression flange $\theta_{a,com}$ reached at time t does not exceed 0,4 no allowance need be made for lateral-torsional buckling. Where $\bar{\lambda}_{LT,\theta,com} > 0,4$ the design buckling resistance moment $M_{b,fi,t,Rd}$ at time t of a laterally unrestrained beam with a Class 1 or Class 2 cross-section should be determined from:

$$M_{b,fi,t,Rd} = [\chi_{LT,fi}/1,2]\, W_{p\ell,y} k_{y,\theta,com} f_y / \gamma_{M,fi} \quad (4.11)$$

where:

$\chi_{LT,fi}$ is the reduction factor for lateral-torsional buckling in the fire design situation;

$k_{y,\theta,com}$ is the reduction factor from 3.2.1 for the yield strength of steel at the maximum temperature in the compression flange $\theta_{a,com}$ reached at time t.

NOTE 1: The constant 1,2 in this expression is a correction factor that allows for a number of effects. The value of 1,2 is the same as the empirically determined value for compression members.

NOTE 2: Conservatively $\theta_{a,com}$ can be assumed to be equal to the maximum temperature $\theta_{a,max}$.

(6) The value of $\chi_{LT,fi}$ should be determined as given in 5.5.2 of ENV 1993-1-1, except using the non-dimensional slenderness $\bar{\lambda}_{LT,\theta,com}$ for the temperature $\theta_{a,com}$ given by:

$$\bar{\lambda}_{LT,\theta,com} = \bar{\lambda}_{LT}[k_{y,\theta,com}/k_{E,\theta,com}]^{0,5} \quad (4.12)$$

where:

$k_{E,\theta,com}$ is the reduction factor from 3.2.1 for the slope of the linear elastic range at the maximum steel temperature in the compression flange $\theta_{a,com}$ reached at time t.

(7) The design shear resistance $V_{fi,t,Rd}$ at time t of a Class 1 or Class 2 cross-section with a non-uniform temperature distribution may be determined from:

$$V_{fi,t,Rd} = k_{y,\theta,max} V_{Rd}[\gamma_{M,1}/\gamma_{M,fi}]/\kappa_1\kappa_2 \quad (4.13)$$

where:

V_{Rd} is the shear resistance of the gross cross-section for normal temperature design, according to 5.4.6 of ENV 1993-1-1.

(8) The value of the adaptation factor κ_1 for non-uniform temperature distribution across a cross-section should be taken as follows:

- for a beam exposed on all four sides: $\kappa_1 = 1,0$;
- for a beam exposed on three sides, with a composite or concrete slab on side four: $\kappa_1 = \boxed{0,70}$.

(9) The value of the adaptation factor κ_2 for non-uniform temperature distribution along a beam should be taken as follows:

- at the supports of a statically indeterminate beam: $\kappa_2 = \boxed{0,85}$;
- in all other cases: $\kappa_2 = 1,0$.

4.2.3.4 Beams with Class 3 cross-sections

(1) The design moment resistance $M_{fi,t,Rd}$ at time t of a Class 3 cross-section with a non-uniform temperature distribution should be determined from:

$$M_{fi,t,Rd} = k_{y,\theta,max} M_{Rd} [\gamma_{M,1} / \gamma_{M,fi}] / \kappa_1 \kappa_2 \qquad (4.14)$$

where:

M_{Rd} is the elastic moment resistance of the gross cross-section $M_{el,Rd}$ for normal temperature design, according to 5.4.5 of ENV 1993-1-1 or the reduced moment resistance allowing for the effects of shear if necessary according to 5.4.7 of ENV 1993-1-1;

$k_{y,\theta,max}$ is the reduction factor for the yield strength of steel at the maximum steel temperature $\theta_{a,max}$ reached at time t, see 3.2.1;

κ_1 is an adaptation factor for non-uniform temperature in a cross-section, see 4.2.3.3(8);

κ_2 is an adaptation factor for non-uniform temperature along the beam, see 4.2.3.3(9).

(2) Provided that the non-dimensional slenderness $\bar{\lambda}_{LT,\theta,com}$ for the maximum temperature in the compression flange $\theta_{a,com}$ reached at time t does not exceed 0,4 no allowance need be made for lateral-torsional buckling. Where $\bar{\lambda}_{LT,\theta,com} > 0,4$ the design buckling resistance moment $M_{b,fi,t,Rd}$ at time t of a laterally unrestrained beam with a Class 3 cross-section should be determined from:

$$M_{b,fi,t,Rd} = [\chi_{LT,fi} / 1,2] \, W_{el,y} k_{y,\theta,com} f_y / \gamma_{M,fi} \qquad (4.15)$$

where:

$\chi_{LT,fi}$ is as given in 4.2.3.3(6).

NOTE 1: The constant 1,2 in this expression is a correction factor that allows for a number of effects. The value of 1,2 is the same as the empirically determined value for compression members.

NOTE 2: Conservatively $\theta_{a,com}$ can be assumed to be equal to the maximum temperature $\theta_{a,max}$.

(3) The design shear resistance $V_{fi,t,Rd}$ at time t of a Class 3 cross-section with a non-uniform temperature distribution may be determined from:

$$V_{fi,t,Rd} = k_{y,\theta,max} V_{Rd} [\gamma_{M,1} / \gamma_{M,fi}] / \kappa_1 \kappa_2 \qquad (4.16)$$

where:

V_{Rd} is the shear resistance of the gross cross-section for normal temperature design, according to 5.4.6 of ENV 1993-1-1.

4.2.3.5 Members with Class 1, 2 or 3 cross-sections, subject to bending and axial compression

(1) The design buckling resistance $R_{fi,t,d}$ at time t of a member subject to combined bending and axial compression should be verified by satisfying expressions (5.51) and (5.52) of ENV 1993-1-1 for a member with a Class 1 or Class 2 cross-section, or expressions (5.53) and (5.54) of ENV 1993-1-1 for a member with a Class 3 cross-section, using the modified values given in (2) and (3).

(2) The modified values of the internal forces and moments should be taken as:

$$M_{y,Sd} = M_{y,fi,Ed} \quad (4.17a)$$

$$M_{z,Sd} = M_{z,fi,Ed} \quad (4.17b)$$

$$N_{Sd} = N_{fi,Ed} \quad (4.17c)$$

(3) The resistance terms should be modified by using:

- $[\chi_{y,fi}/1,2]$ in place of χ_y, where $\chi_{y,fi}$ is as defined in 4.2.3.2(2);
- $[\chi_{z,fi}/1,2]$ in place of χ_z, where $\chi_{z,fi}$ is as defined in 4.2.3.2(2);
- $[\chi_{LT,fi}/1,2]$ in place of χ_{LT}, where $\chi_{LT,fi}$ is as defined in 4.2.3.3(6);
- $k_{y,\theta,max} f_y$ in place of f_y, where $k_{y,\theta,max}$ is as defined in 4.2.3.2(1);
- $\gamma_{M,fi}$ in place of γ_{M1}.

4.2.4 Critical temperature

(1)P As an alternative to 4.2.3, verification may be carried out in the temperature domain.

(2) Except when considering deformation criteria, the critical steel temperature $\theta_{a,cr}$ at time t for a uniform temperature distribution may be determined for any degree of utilisation μ_0 at time $t = 0$ using:

$$\theta_{a,cr} = 39,19\,\ell n \left[\frac{1}{0,9674\,\mu_0^{3,833}} - 1 \right] + 482 \quad (4.18)$$

(3) Values of $\theta_{a,cr}$ for values of μ_0 from 0,22 to 0,80 are given in table 4.1.

(4) For members with Class 1, Class 2 or Class 3 cross-sections and for all tension members, the degree of utilisation μ_0 at time $t = 0$ may be obtained from:

$$\mu_0 = E_{fi,d} / R_{fi,d,0} \quad (4.19)$$

where:

$R_{fi,d,0}$ is the value of $R_{fi,d,t}$ for time $t = 0$, from 4.2.3;

$E_{fi,d}$ and $R_{fi,d,t}$ are as defined in 4.2.1(1).

(5) Alternatively for tension members, and for beams where lateral-torsional buckling is not a potential failure mode, μ_0 may conservatively be obtained from:

$$\mu_0 = \eta_{fi} [\gamma_{M,fi} / \gamma_{M1}] \quad (4.20)$$

where:

η_{fi} is the reduction factor defined in 2.4.3(3).

(6) For members with Class 4 cross-sections, other than tension members, it may be assumed that 4.2.1(1) is satisfied if at time t the steel temperature θ_a at all cross-sections is not more than 350 °C.

Table 4.1: Critical temperature $\theta_{a,cr}$ for values of the utilisation factor μ_0

μ_0	$\theta_{a,cr}$	μ_0	$\theta_{a,cr}$	μ_0	$\theta_{a,cr}$
0,22	711	0,42	612	0,62	549
0,24	698	0,44	605	0,64	543
0,26	685	0,46	598	0,66	537
0,28	674	0,48	591	0,68	531
0,30	664	0,50	585	0,70	526
0,32	654	0,52	578	0,72	520
0,34	645	0,54	572	0,74	514
0,36	636	0,56	566	0,76	508
0,38	628	0,58	560	0,78	502
0,40	620	0,60	554	0,80	496

4.2.5 Steel temperature development

4.2.5.1 Unprotected internal steelwork

(1) For an equivalent uniform temperature distribution in the cross-section, the increase of temperature $\Delta\theta_{a,t}$ in an unprotected steel member during a time interval Δt may be determined from:

$$\Delta\theta_{a,t} = \frac{A_m / V}{c_a \rho_a} \dot{h}_{net,d} \, \Delta t \tag{4.21}$$

where:

A_m / V	is	the section factor for unprotected steel members;
A_m	is	the exposed surface area of the member per unit length;
V	is	the volume of the member per unit length;
c_a	is	the specific heat of steel, from 3.3.1.2 [J/kgK];
$\dot{h}_{net,d}$	is	the design value of the net heat flux per unit area [W/m^2];
Δt	is	the time interval [seconds];
ρ_a	is	the unit mass of steel, from 3.2.2(1) [kg/m^3].

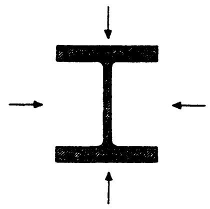
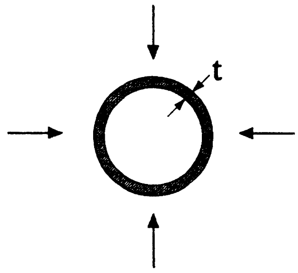
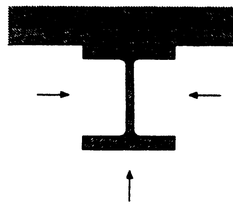
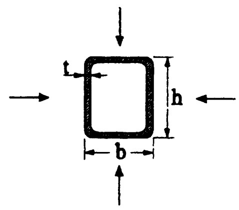
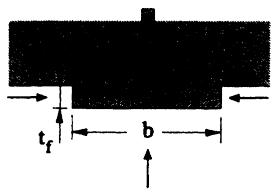
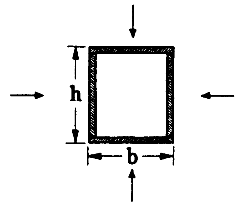
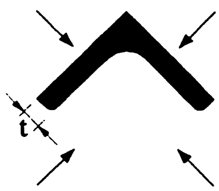
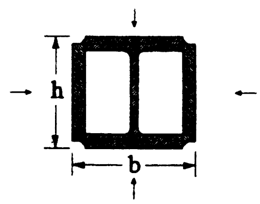
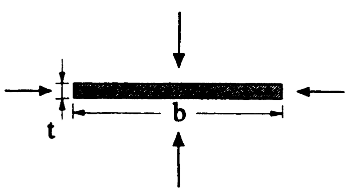
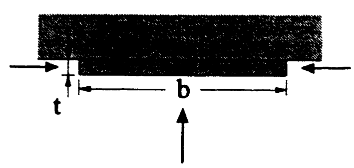
(2) The value of $\dot{h}_{net,d}$ should be obtained from ENV 1991-2-2 using $\varepsilon_f = 0,8$ and $\varepsilon_m = 0,625$ leading to $\varepsilon_{res} = 0,5$, where ε_f, ε_m and ε_{res} are as defined in ENV 1991-2-2.

(3) The value of Δt should not be taken as more than 5 seconds.

(4) In expression (4.21) the value of the section factor A_m / V should not be taken as less than $10\,\mathrm{m}^{-1}$.

(5) Some expressions for calculating design values of the section factor A_m / V for unprotected steel members are given in table 4.2.

Table 4.2: Section factor A_m / V for unprotected steel members.

Open section exposed to fire on all sides: $\frac{A_m}{V} = \frac{\text{perimeter}}{\text{cross-section area}}$ 	Tube exposed to fire on all sides: $A_m / V = 1 / t$ 
Open section exposed to fire on three sides: $\frac{A_m}{V} = \frac{\text{surface exposed to fire}}{\text{cross-section area}}$ 	Hollow section (or welded box section of uniform thickness) exposed to fire on all sides: If $t \ll$ b: $A_m / V \approx 1 / t$ 
I-section flange exposed to fire on three sides: $A_m / V = (b + 2t_f) / (bt_f)$ If $t \ll$ b: $A_m / V \approx 1 / t_f$ 	Welded box section exposed to fire on all sides: $\frac{A_m}{V} = \frac{2(b + h)}{\text{cross-section area}}$ 
Angle (or any open section of uniform thickness) exposed to fire on all sides: $A_m / V = 2 / t$ 	I-section with box reinforcement, exposed to fire on all sides: $\frac{A_m}{V} = \frac{2(b + h)}{\text{cross-section area}}$ 
Flat bar exposed to fire on all sides: $A_m / V = 2(b + t) / (bt)$ If $t \ll b$: $A_m / V \approx 2 / t$ 	Flat bar exposed to fire on three sides: $A_m / V = (b + 2t) / (bt)$ If $t \ll b$: $A_m / V \approx 1 / t$ 

4.2.5.2 Internal steelwork insulated by fire protection material

(1) For a uniform temperature distribution in a cross-section, the temperature increase $\Delta\theta_{a,t}$ of an insulated steel member during a time interval Δt may be obtained from:

$$\Delta\theta_{a,t} = \frac{\lambda_p A_p/V}{d_p c_a \rho_a} \frac{(\theta_{g,t} - \theta_{a,t})}{(1 + \phi/3)} \Delta t - (e^{\phi/10} - 1)\Delta\theta_{g,t} \quad \text{but} \quad \Delta\theta_{a,t} \geq 0 \qquad (4.22)$$

with:

$$\phi = \frac{c_p \rho_p}{c_a \rho_a} d_p A_p/V$$

where:

A_p/V	is	the section factor for steel members insulated by fire protection material;
A_p	is	the appropriate area of fire protection material per unit length of the member;
V	is	the volume of the member per unit length;
c_a	is	the specific heat of steel, from 3.3.1.2 [J/kgK];
c_p	is	the specific heat of the fire protection material [J/kgK];
d_p	is	the thickness of the fire protection material [m];
Δt	is	the time interval [seconds];
$\theta_{a,t}$	is	the steel temperature at time t;
$\theta_{g,t}$	is	the ambient gas temperature at time t;
$\Delta\theta_{g,t}$	is	the increase of the ambient gas temperature during the time interval Δt;
λ_p	is	the thermal conductivity of the fire protection material [W/mK];
ρ_a	is	the unit mass of steel, from 3.2.2 [kg/m^3];
ρ_p	is	the unit mass of the fire protection material [kg/m^3].

(2) The values of c_p, λ_p and ρ_p should be determined as specified in 3.3.2.

(3) The value of Δt should not be taken as more than 30 seconds.

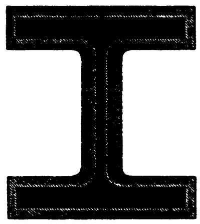
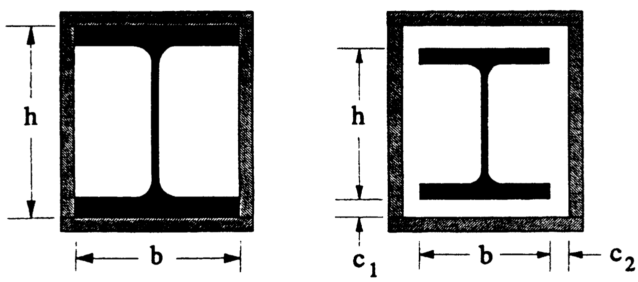
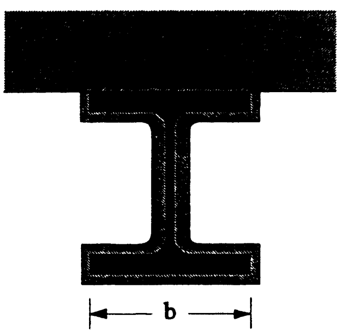
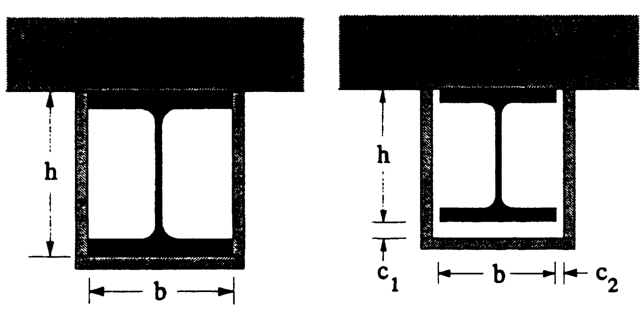
(4) The area A_p of the fire protection material should generally be taken as the area of its inner surface, but for hollow encasement with a clearance around the steel member the same value as for hollow encasement without a clearance may be adopted.

(5) Some design values of the section factor A_p/V for insulated steel members are given in table 4.3.

(6) For moist fire protection materials the calculation of the steel temperature increase $\Delta\theta_a$ may be modified to allow for a time delay in the rise of the steel temperature when it reaches 100 °C. This delay time should be determined by a method conforming with pr ENV yyy5-4.

(7) Alternatively, the uniform temperature of an insulated steel member after a given time duration of standard fire exposure may be obtained using design flow charts derived in conformity with pr ENV yyy5-4.

Table 4.3: Section factor A_p / V for steel members insulated by fire protection material

Sketch	Description	Section factor (A_p / V)
	Contour encasement of uniform thickness	$\dfrac{\text{steel perimeter}}{\text{steel cross-section area}}$
	Hollow encasement[1] of uniform thickness	$\dfrac{2(b + h)}{\text{steel cross-section area}}$
	Contour encasement of uniform thickness, exposed to fire on three sides	$\dfrac{\text{steel perimeter} - b}{\text{steel cross-section area}}$
	Hollow encasement[1] of uniform thickness, exposed to fire on three sides	$\dfrac{2h + b}{\text{steel cross-section area}}$

[1] The clearance dimensions c_1 and c_2 should not normally exceed $h/4$.

4.2.5.3 Internal steelwork in a void that is protected by heat screens

(1)P The provisions given below apply to both of the following cases:

- steel members in a void that is bordered by a floor on top and by a horizontal heat screen below, and
- steel members in a void that is bordered by vertical heat screens on both sides,

provided in both cases that there is a gap between the heat screen and the member. They do not apply if the heat screen is in direct contact with the member.

(2)P The properties and performance of the heat screens shall be determined using a test procedure conforming with pr ENV yyy5-1 or pr ENV yyy5-2 as appropriate.

(3)P The temperature development in the void in which the steel members are situated shall be determined from a standard fire test conforming with pr ENV yyy5-1 or pr ENV yyy5-2 as appropriate.

(4) For internal steelwork protected by heat screens, the calculation of the steel temperature increase $\Delta\theta_a$ should be based on the methods given in 4.2.5.1 or 4.2.5.2 as appropriate, taking the ambient gas temperature $\theta_{g,t}$ as equal to the gas temperature in the void.

(5) As an alternative to the procedure given in 4.2.5.1, $\Delta\theta_a$ may be calculated using values of the convective and radiative heat transfer coefficients α_c and α_r determined from tests conforming with pr ENV yyy5-1.

4.2.5.4 External steelwork

(1)P The temperature in external steelwork shall be determined taking into account:

- the radiative heat flux from the fire compartment
- the radiative heat flux and the convective heat flux from the flames emanating from openings
- the radiative and convective heat loss from the steelwork to the ambient atmosphere
- the sizes and locations of the structural members.

(2)P Heat screens may be provided on one, two or three sides of an external steel member in order to protect it from radiative heat transfer.

(3) Heat screens should be either:

- directly attached to that side of the steel member that it is intended to protect, or
- large enough to fully screen that side from the expected radiative heat flux.

(4) Heat screens should be non-combustible and have a fire resistance of at least EI 30 according to pr ISO EN 834.

(5) The temperature in external steelwork protected by heat screens should be determined as specified in (1), assuming that there is no radiative heat transfer to those sides that are protected by heat screens.

(6) Calculations may be based on steady state conditions resulting from a stationary heat balance using the methods given in annex C.

(7) Design using annex C of this Part 1-2 of ENV 1993 should be based on the model given in annex C of ENV 1991-2-2 describing the compartment fire conditions and the flames emanating from openings, on which the calculation of the radiative and convective heat fluxes should be based.

4.3 Advanced calculation models

4.3.1 Basis

(1)P Advanced calculation models may be used for individual members, for sub-assemblies or for entire structures.

(2)P Advanced calculation methods may be used with any type of cross-section.

(3)P Advanced calculation methods shall provide a realistic analysis of structures exposed to fire. They shall be based on fundamental physical behaviour in such a way as to lead to a reliable approximation of the expected behaviour of the relevant structural component under fire conditions.

(4)P Advanced calculation methods may include separate calculation models for the determination of:

- the development and distribution of the temperature within structural members (thermal response model);
- the mechanical behaviour of the structure or of any part of it (mechanical response model).

(5)P Any potential failure modes not covered by the advanced calculation method (including local buckling and failure in shear) shall be eliminated by appropriate means.

(6)P Advanced calculation methods may be used in association with any heating curve, provided that the material properties are known for the relevant temperature range.

(7)P The validity of any specific advanced calculation method for a particular situation shall be agreed between the client, the designer and the competent authority.

4.3.2 Thermal response

(1)P Advanced calculation methods for thermal response shall be based on the acknowledged principles and assumptions of the theory of heat transfer.

(2)P The thermal response model shall consider:

- the relevant thermal actions specified in ENV 1991-2-2;
- the variation of the thermal properties of the material with the temperature, see 3.3.

(3) The effects of non-uniform thermal exposure and of heat transfer to adjacent building components may be included where appropriate.

(4) The influence of any moisture content and of any migration of the moisture within the fire protection material may conservatively be neglected.

4.3.3 Mechanical response

(1)P Advanced calculation methods for mechanical response shall be based on the acknowledged principles and assumptions of the theory of structural mechanics, taking into account the changes of mechanical properties with temperature.

(2)P The effects of thermally induced strains and stresses both due to temperature rise and due to temperature differentials, shall be considered.

(3)P Where relevant, the mechanical response of the model shall also take account of:

- the combined effects of mechanical actions, geometrical imperfections and thermal actions;
- the temperature dependent mechanical properties of the material, see 3.2;
- geometrical non-linear effects;
- the effects of non-linear material properties, including the beneficial effects of loading and unloading on the structural stiffness.

(4) Provided that the stress-strain relationships given in 3.2 are used, the effects of transient thermal creep need not be given explicit consideration.

(5)P The deformations at ultimate limit state implied by the calculation method shall be limited as necessary to ensure that compatibility is maintained between all parts of the structure.

(6) If necessary, the design should be based on the ultimate limit state beyond which the calculated deformations of the structure would cause failure due to the loss of adequate support to one of the members.

Annex A [informative]

Stress-strain relationships at elevated temperatures (no strain-hardening)

(1) The stress strain relationship specified in 3.2.1 is evaluated for steel grades S 235, S 275, S 355 and S 460 in tables A.1 to A.4 respectively. The variation of this relationship with temperature is illustrated in figures A.1 to A.4 for steel grades S 235, S 275, S 355 and S 460 respectively.

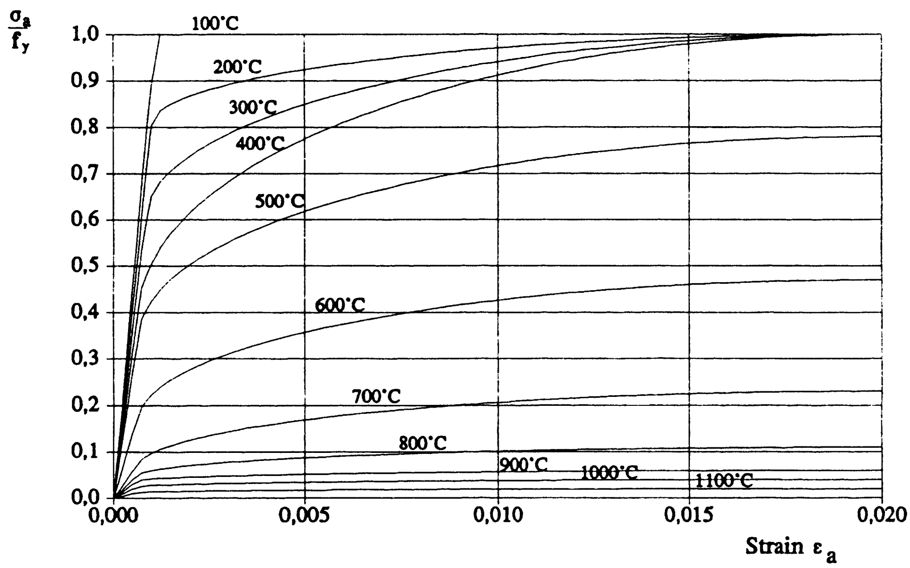


Figure A.1: Variation of stress-strain relationship with temperature for grade S 235 steel (strain-hardening not included)

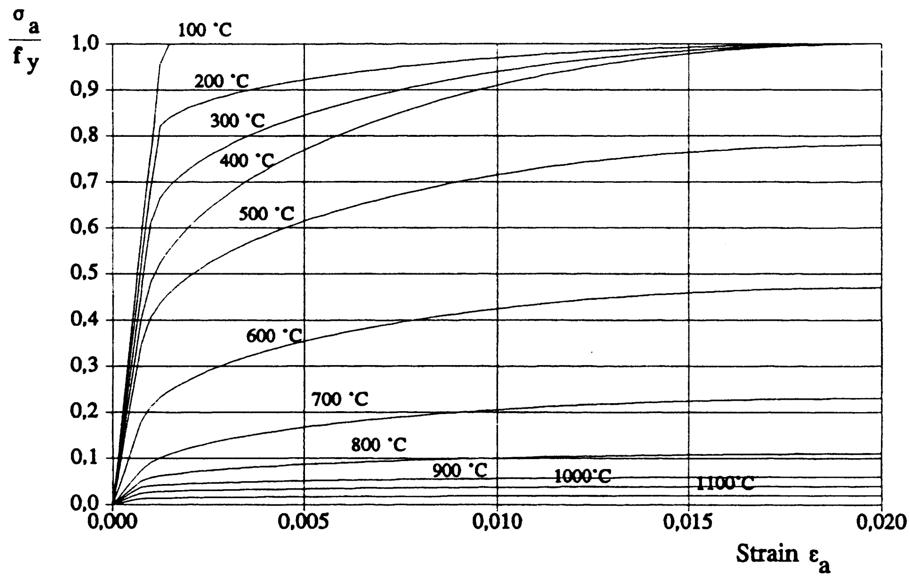


Figure A.2: Variation of stress-strain relationship with temperature for grade S 275 steel (strain-hardening not included)

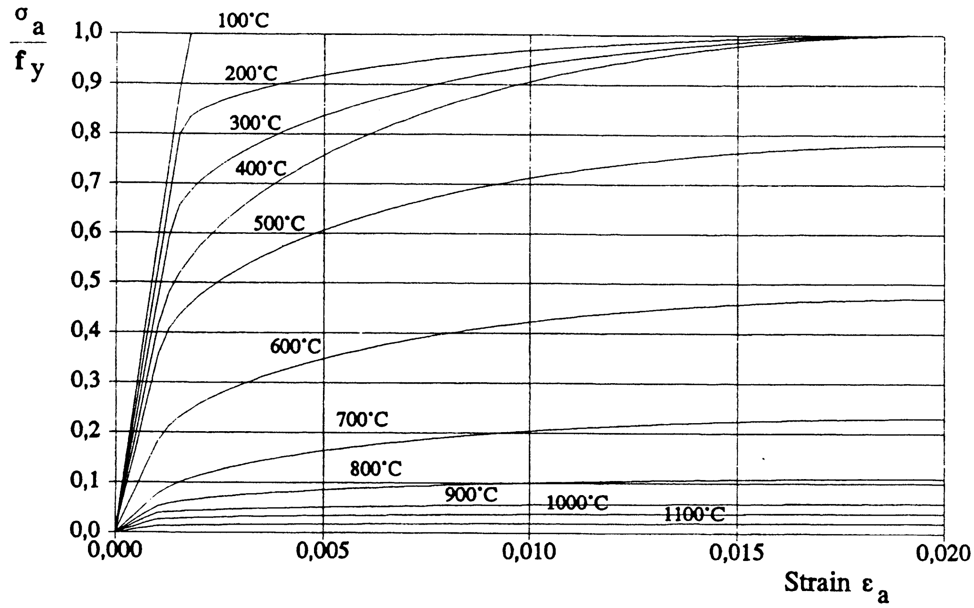


Figure A.3: Variation of stress-strain relationship with temperature for grade S 355 steel (strain-hardening not included)

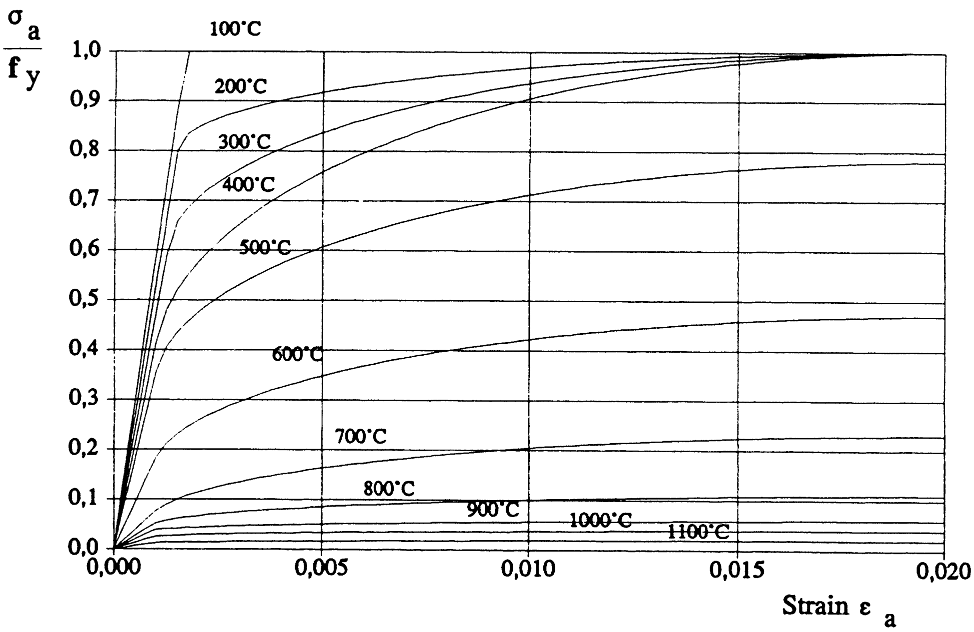


Figure A.4: Variation of stress-strain relationship with temperature for grade S 460 steel (strain-hardening not included)

Table A.1: Stress-strain relationship at elevated temperatures for grade S 235 steel

Strain	Effective yield strength at elevated temperature, relative to yield strength at 20 °C. $k_{y,\theta} = f_{y,\theta}/f_y$							
	Steel temperature θ_a [°C]							
	100	200	300	400	500	600	700	800
0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000
0,0005	0,447	0,402	0,357	0,313	0,268	0,139	0,058	0,040
0,0010	0,894	0,804	0,652	0,505	0,424	0,223	0,097	0,060
0,0015	1,000	0,849	0,705	0,569	0,470	0,254	0,113	0,066
0,0020	1,000	0,867	0,738	0,614	0,502	0,276	0,125	0,071
0,0025	1,000	0,880	0,763	0,650	0,528	0,295	0,135	0,074
0,0030	1,000	0,892	0,785	0,681	0,551	0,310	0,143	0,078
0,0035	1,000	0,901	0,804	0,708	0,570	0,324	0,151	0,080
0,0040	1,000	0,910	0,821	0,733	0,588	0,336	0,157	0,083
0,0045	1,000	0,917	0,836	0,755	0,604	0,347	0,163	0,085
0,0050	1,000	0,924	0,849	0,775	0,618	0,357	0,169	0,087
0,0055	1,000	0,931	0,862	0,794	0,632	0,367	0,174	0,089
0,0060	1,000	0,937	0,873	0,811	0,644	0,375	0,179	0,091
0,0065	1,000	0,942	0,884	0,827	0,656	0,383	0,183	0,092
0,0070	1,000	0,947	0,894	0,842	0,666	0,391	0,187	0,094
0,0075	1,000	0,952	0,903	0,856	0,676	0,397	0,191	0,095
0,0080	1,000	0,956	0,912	0,868	0,685	0,404	0,194	0,097
0,0085	1,000	0,960	0,920	0,880	0,694	0,410	0,197	0,098
0,0090	1,000	0,964	0,928	0,892	0,702	0,416	0,201	0,099
0,0095	1,000	0,967	0,935	0,902	0,710	0,421	0,203	0,100
0,0100	1,000	0,971	0,941	0,912	0,717	0,426	0,206	0,101
0,0110	1,000	0,977	0,953	0,930	0,730	0,435	0,211	0,103
0,0120	1,000	0,982	0,964	0,945	0,741	0,443	0,215	0,104
0,0130	1,000	0,986	0,972	0,959	0,750	0,449	0,219	0,106
0,0140	1,000	0,990	0,980	0,970	0,758	0,455	0,222	0,107
0,0150	1,000	0,993	0,986	0,979	0,765	0,460	0,224	0,108
0,0160	1,000	0,996	0,991	0,987	0,771	0,463	0,226	0,109
0,0170	1,000	0,998	0,995	0,993	0,775	0,466	0,228	0,109
0,0180	1,000	0,999	0,997	0,997	0,778	0,468	0,229	0,110
0,0190	1,000	1,000	0,999	0,999	0,779	0,470	0,230	0,110
0,0200	1,000	1,000	1,000	1,000	0,780	0,470	0,230	0,110

Table A.2: Stress-strain relationship at elevated temperatures for grade S 275 steel

Strain	Effective yield strength at elevated temperature, relative to yield strength at 20 °C. $k_{y,\theta} = f_{y,\theta}/f_y$							
	Steel temperature θ_a [°C]							
	100	200	300	400	500	600	700	800
0,0000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000
0,0005	0,382	0,344	0,305	0,267	0,229	0,118	0,050	0,034
0,0010	0,764	0,687	0,611	0,482	0,407	0,212	0,091	0,058
0,0015	1,000	0,840	0,691	0,553	0,459	0,247	0,109	0,065
0,0020	1,000	0,861	0,728	0,602	0,494	0,270	0,122	0,070
0,0025	1,000	0,876	0,756	0,640	0,522	0,290	0,132	0,074
0,0030	1,000	0,888	0,779	0,672	0,545	0,306	0,141	0,077
0,0035	1,000	0,898	0,798	0,701	0,565	0,320	0,148	0,080
0,0040	1,000	0,907	0,816	0,726	0,583	0,333	0,155	0,082
0,0045	1,000	0,915	0,831	0,749	0,600	0,344	0,161	0,085
0,0050	1,000	0,922	0,845	0,770	0,615	0,354	0,167	0,087
0,0055	1,000	0,929	0,858	0,789	0,628	0,364	0,172	0,089
0,0060	1,000	0,935	0,870	0,806	0,641	0,373	0,177	0,090
0,0065	1,000	0,941	0,881	0,823	0,653	0,381	0,182	0,092
0,0070	1,000	0,946	0,892	0,838	0,664	0,389	0,186	0,094
0,0075	1,000	0,950	0,901	0,852	0,674	0,396	0,190	0,095
0,0080	1,000	0,955	0,910	0,865	0,683	0,402	0,193	0,096
0,0085	1,000	0,959	0,918	0,878	0,692	0,409	0,197	0,098
0,0090	1,000	0,963	0,926	0,889	0,701	0,414	0,200	0,099
0,0095	1,000	0,967	0,933	0,900	0,708	0,420	0,203	0,102
0,0100	1,000	0,970	0,940	0,910	0,716	0,425	0,205	0,102
0,0110	1,000	0,976	0,952	0,928	0,729	0,434	0,210	0,104
0,0120	1,000	0,981	0,963	0,944	0,740	0,442	0,215	0,105
0,0130	1,000	0,986	0,972	0,958	0,750	0,449	0,218	0,107
0,0140	1,000	0,990	0,980	0,969	0,758	0,455	0,222	0,108
0,0150	1,000	0,993	0,986	0,979	0,765	0,459	0,224	0,108
0,0160	1,000	0,996	0,991	0,985	0,769	0,462	0,226	0,109
0,0170	1,000	0,997	0,995	0,992	0,775	0,466	0,228	0,110
0,0180	1,000	0,999	0,998	0,997	0,778	0,468	0,229	0,110
0,0190	1,000	1,000	0,999	0,999	0,779	0,470	0,230	0,110
0,0200	1,000	1,000	1,000	1,000	0,780	0,470	0,230	0,110

Table A.3: Stress-strain relationship at elevated temperatures for grade S 355 steel

Strain	Effective yield strength at elevated temperature, relative to yield strength at 20 °C. $k_{y,\theta} = f_{y,\theta}/f_y$							
	Steel temperature θ_a [°C]							
	100	200	300	400	500	600	700	800
0,0000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000
0,0005	0,296	0,266	0,237	0,207	0,177	0,092	0,038	0,027
0,0010	0,592	0,532	0,473	0,414	0,355	0,183	0,077	0,052
0,0015	0,887	0,799	0,657	0,520	0,435	0,230	0,100	0,062
0,0020	1,000	0,848	0,706	0,576	0,476	0,258	0,114	0,067
0,0025	1,000	0,866	0,739	0,619	0,507	0,279	0,126	0,072
0,0030	1,000	0,880	0,765	0,654	0,532	0,296	0,135	0,075
0,0035	1,000	0,892	0,786	0,685	0,554	0,312	0,144	0,078
0,0040	1,000	0,902	0,805	0,712	0,574	0,325	0,151	0,081
0,0045	1,000	0,910	0,822	0,736	0,591	0,337	0,158	0,083
0,0050	1,000	0,918	0,837	0,758	0,607	0,348	0,164	0,086
0,0055	1,000	0,925	0,851	0,778	0,621	0,359	0,169	0,088
0,0060	1,000	0,932	0,864	0,797	0,635	0,368	0,174	0,090
0,0065	1,000	0,938	0,876	0,814	0,647	0,377	0,179	0,091
0,0070	1,000	0,943	0,886	0,830	0,659	0,385	0,183	0,093
0,0075	1,000	0,948	0,896	0,845	0,669	0,399	0,187	0,094
0,0080	1,000	0,953	0,906	0,859	0,679	0,406	0,191	0,096
0,0085	1,000	0,957	0,915	0,872	0,689	0,412	0,195	0,097
0,0090	1,000	0,961	0,923	0,884	0,697	0,417	0,198	0,098
0,0095	1,000	0,965	0,930	0,896	0,705	0,423	0,201	0,099
0,0100	1,000	0,969	0,937	0,906	0,713	0,428	0,204	0,101
0,0110	1,000	0,975	0,950	0,925	0,726	0,437	0,209	0,102
0,0120	1,000	0,981	0,961	0,942	0,738	0,441	0,214	0,104
0,0130	1,000	0,985	0,971	0,956	0,748	0,448	0,218	0,106
0,0140	1,000	0,989	0,979	0,968	0,757	0,454	0,221	0,107
0,0150	1,000	0,993	0,985	0,978	0,764	0,459	0,224	0,108
0,0160	1,000	0,995	0,991	0,986	0,770	0,463	0,226	0,109
0,0170	1,000	0,997	0,995	0,992	0,774	0,466	0,228	0,109
0,0180	1,000	0,999	0,998	0,997	0,778	0,468	0,229	0,110
0,0190	1,000	1,000	0,999	0,999	0,779	0,470	0,230	0,110
0,0200	1,000	1,000	1,000	1,000	0,780	0,470	0,230	0,110

Table A.4: Stress-strain relationship at elevated temperatures for grade S 460 steel

Strain	Effective yield strength at elevated temperature, relative to yield strength at 20 °C. $k_{y,\theta} = f_{y,\theta}/f_y$							
	Steel temperature θ_a [°C]							
	100	200	300	400	500	600	700	800
0,0000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000
0,0005	0,228	0,205	0,183	0,160	0,137	0,071	0,030	0,021
0,0010	0,457	0,411	0,365	0,320	0,274	0,142	0,059	0,041
0,0015	0,685	0,616	0,548	0,465	0,395	0,205	0,087	0,057
0,0020	0,913	0,815	0,669	0,537	0,449	0,239	0,104	0,064
0,0025	1,000	0,850	0,712	0,587	0,485	0,263	0,117	0,069
0,0030	1,000	0,868	0,743	0,627	0,514	0,283	0,127	0,073
0,0035	1,000	0,882	0,769	0,661	0,538	0,300	0,137	0,076
0,0040	1,000	0,893	0,790	0,691	0,560	0,315	0,145	0,079
0,0045	1,000	0,903	0,809	0,718	0,579	0,328	0,152	0,082
0,0050	1,000	0,912	0,825	0,742	0,596	0,340	0,158	0,084
0,0055	1,000	0,920	0,841	0,764	0,611	0,351	0,164	0,086
0,0060	1,000	0,927	0,854	0,784	0,626	0,361	0,170	0,088
0,0065	1,000	0,933	0,867	0,802	0,639	0,370	0,175	0,090
0,0070	1,000	0,939	0,879	0,819	0,651	0,379	0,180	0,092
0,0075	1,000	0,945	0,890	0,835	0,663	0,387	0,184	0,094
0,0080	1,000	0,950	0,900	0,850	0,673	0,395	0,188	0,095
0,0085	1,000	0,954	0,909	0,864	0,683	0,402	0,192	0,096
0,0090	1,000	0,959	0,918	0,877	0,692	0,408	0,196	0,098
0,0095	1,000	0,963	0,926	0,889	0,701	0,414	0,199	0,099
0,0100	1,000	0,967	0,933	0,900	0,709	0,420	0,202	0,100
0,0110	1,000	0,974	0,947	0,921	0,723	0,430	0,208	0,102
0,0120	1,000	0,979	0,959	0,938	0,736	0,439	0,213	0,104
0,0130	1,000	0,984	0,969	0,953	0,747	0,446	0,217	0,105
0,0140	1,000	0,989	0,977	0,966	0,756	0,453	0,221	0,107
0,0150	1,000	0,992	0,984	0,977	0,763	0,458	0,223	0,108
0,0160	1,000	0,995	0,990	0,985	0,769	0,462	0,226	0,109
0,0170	1,000	0,997	0,994	0,992	0,774	0,466	0,228	0,109
0,0180	1,000	0,999	0,998	0,996	0,777	0,468	0,229	0,110
0,0190	1,000	0,999	0,999	1,000	0,779	0,470	0,230	0,110
0,0200	1,000	1,000	1,000	1,000	0,780	0,470	0,230	0,110

Annex B [normative]

Strain-hardening of steel at elevated temperatures

(1) For temperatures below 400 °C , the alternative strain-hardening option mentioned in 3.2.1(5) may be used as follows:

- for $0{,}02 < \varepsilon < 0{,}04$:

$$\sigma_a = 50(f_{u,\theta} - f_{y,\theta})\varepsilon + 2f_{y,\theta} - f_{u,\theta} \tag{B.1a}$$

- for $0{,}04 \leq \varepsilon \leq 0{,}15$:

$$\sigma_a = f_{u,\theta} \tag{B.1b}$$

- for $0{,}15 < \varepsilon < 0{,}20$:

$$\sigma_a = f_{u,\theta}[1 - 20(\varepsilon - 0{,}15)] \tag{B.1c}$$

- for $\varepsilon \geq 0{,}20$:

$$\sigma_a = 0{,}00 \tag{B.1d}$$

where:

$f_{u,\theta}$ is the ultimate strength at elevated temperature, allowing for strain-hardening.

(2) The alternative stress-strain relationship for steel, allowing for strain hardening, is illustrated in figure B.1.

(3) The ultimate strength at elevated temperature, allowing for strain hardening, should be determined as follows:

- for $\theta_a < 300\,°C$:

$$f_{u,\theta} = 1{,}25 f_{y,\theta} \tag{B.2a}$$

- for $300\,°C \leq \theta_a < 400\,°C$:

$$f_{u,\theta} = f_{y,\theta}(2 - 0{,}0025\theta a) \tag{B.2b}$$

- for $\theta_a \geq 400\,°C$:

$$f_{u,\theta} = f_{y,\theta} \tag{B.2c}$$

(4) The variation of the alternative stress-strain relationship with temperature is illustrated in figure B.2.

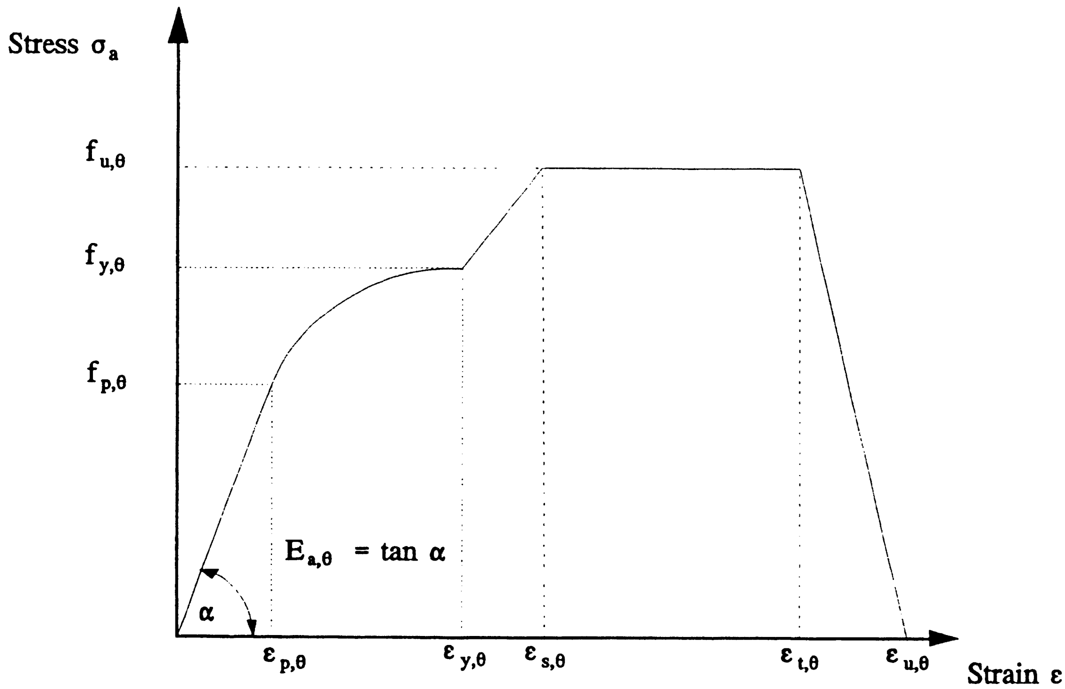


Figure B.1: Alternative stress-strain relationship for steel allowing for strain-hardening

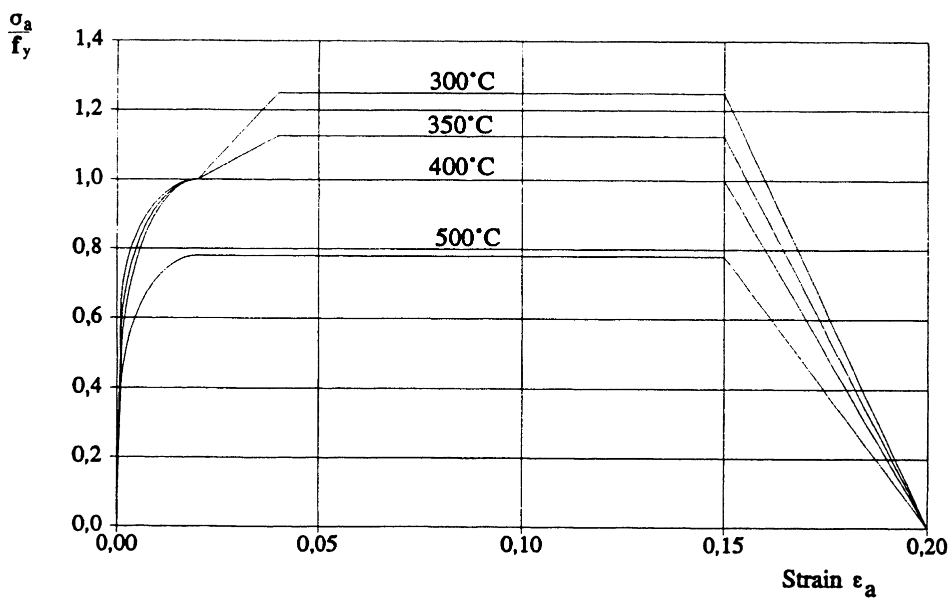


Figure B.2: Alternative stress-strain relationships for steel at elevated temperatures, allowing for strain hardening

Annex C [normative]

Heat transfer to external steelwork.

C.1 General

C.1.1 Basis

(1) In this annex C, the fire compartment is assumed to be confined to one storey only. All windows or other similar openings in the fire compartment are assumed to be rectangular.

(2) Annex C of ENV 1991-2-2 should be used to determine the temperature of the compartment fire, the dimensions and temperatures of the flames projecting from the openings, and the radiation and convection parameters.

(3) A distinction should be made between members not engulfed in flame and members engulfed in flame, depending on their locations relative to the openings in the walls of the fire compartment.

(4) A member that is not engulfed in flame should be assumed to receive radiative heat transfer from all the openings in that side of the fire compartment and from the flames projecting from all these openings.

(5) A member that is engulfed in flame should be assumed to receive convective heat transfer from the engulfing flame, plus radiative heat transfer from the engulfing flame and from the fire compartment opening from which it projects. The radiative heat transfer from other flames and from other openings may be neglected.

C.1.2 Member dimensions and faces

(1) The convention used for the dimensions d_1 and d_2 of a member and the notation used to identify its four faces are indicated in figure C.1.

C.1.3 Heat balance

(1) For a member not engulfed in flame, the average temperature of the steel member T_m [K] should be determined from the solution of the following heat balance:

$$\sigma T_m{}^4 + \alpha T_m = \Sigma I_z + \Sigma I_f + 293\alpha \tag{C.1}$$

where:

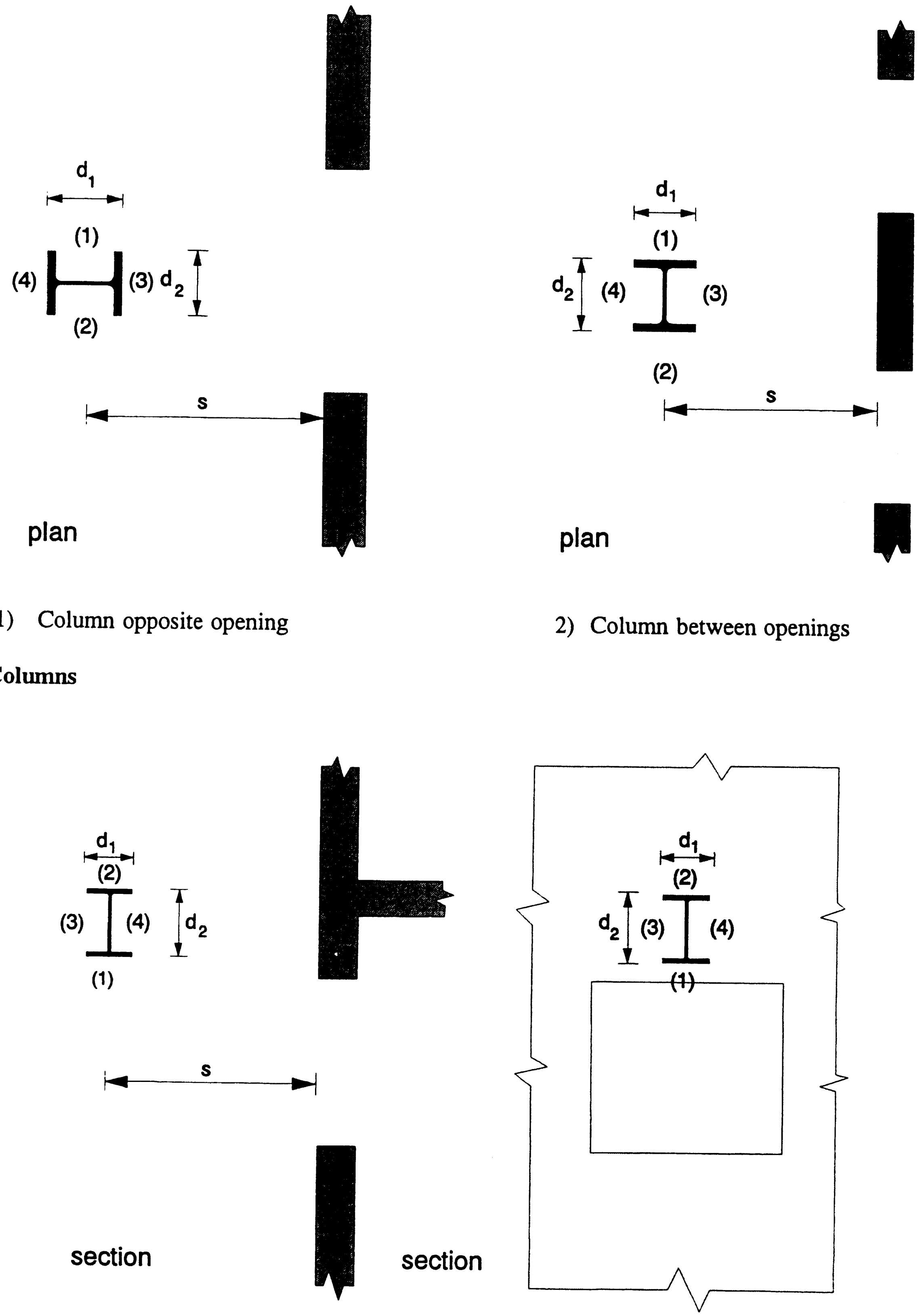
σ is the Stefan Boltzmann constant [$56{,}7 \times 10^{-12}$ kW/m^{2}K^4];

α is the convective heat transfer coefficient [kW/m^{2}K];

I_z is the radiative heat flux from a flame [kW/m^2];

I_f is the radiative heat flux from an opening [kW/m^2].

(2) The convective heat transfer coefficient α should be obtained from annex C of ENV 1991-2-2 for the 'no forced draught' or the 'forced draught' condition as appropriate, using an effective cross-sectional dimension $d = (d_1 + d_2)/2$.



1) Column opposite opening

2) Column between openings

a) **Columns**

1) Beam parallel to wall

2) Beam perpendicular to wall

b) **Beams**

Figure C.1: Member dimensions and faces

(3) For a member engulfed in flame, the average temperature of the steel member T_m [K] should be determined from the solution of the following heat balance:

$$\sigma T_m^4 + \alpha T_m = I_z + I_f + \alpha T_z \qquad \text{(C.2)}$$

where:

T_z is the flame temperature [K];

I_z is the radiative heat flux from the flame [kW/m^2];

I_f is the radiative heat flux from the corresponding opening [kW/m^2].

(4) The radiative heat flux I_z from flames should be determined according to the situation and type of member as follows:

- Columns not engulfed in flame: see C.2;
- Beams not engulfed in flame: see C.3;
- Columns engulfed in flame: see C.4;
- Beams fully or partially engulfed in flame: see C.5.

Other cases may be treated analogously, using appropriate adaptations of the treatments given in C.2 to C.5.

(5) The radiative heat flux I_f from an opening should be determined from:

$$I_f = \phi_f \varepsilon_f (1 - a_z) \sigma T_f^4 \qquad \text{(C.3)}$$

where:

ϕ_f is the overall configuration factor of the member for radiative heat transfer from that opening;

ε_f is the emissivity of the opening;

a_z is the absorptivity of the flames;

T_f is the temperature of the fire [K] from annex C of ENV 1991-2-2.

(6) The emissivity ε_f of an opening should be taken as unity, see annex C of ENV 1991-2-2.

(7) The absorptivity a_z of the flames should be determined from C.2 to C.5 as appropriate.

C.1.4 Overall configuration factors

(1) The overall configuration factor ϕ_f of a member for radiative heat transfer from an opening should be determined from:

$$\phi_f = \frac{(C_1 \phi_{f,1} + C_2 \phi_{f,2}) d_1 + (C_3 \phi_{f,3} + C_4 \phi_{f,4}) d_2}{(C_1 + C_2) d_1 + (C_3 + C_4) d_2} \quad \text{(C.4)}$$

where:

$\phi_{f,i}$ is the configuration factor of member face i for that opening, see annex D;

d_i is the cross-sectional dimension of member face i ;

C_i is the protection coefficient of member face i as follows:

- for a protected face: $C_i = 0$
- for an unprotected face: $C_i = 1$

(2) The configuration factor $\phi_{f,i}$ for a member face from which the opening is not visible should be taken as zero.

(3) The overall configuration factor ϕ_z of a member for radiative heat transfer from a flame should be determined from:

$$\phi_z = \frac{(C_1 \phi_{z,1} + C_2 \phi_{z,2}) d_1 + (C_3 \phi_{z,3} + C_4 \phi_{z,4}) d_2}{(C_1 + C_2) d_1 + (C_3 + C_4) d_2} \quad \text{(C.5)}$$

where:

$\phi_{z,i}$ is the configuration factor of member face i for that flame, see annex D.

(4) The configuration factors $\phi_{z,i}$ of individual member faces for radiative heat transfer from flames may be based on equivalent rectangular flame dimensions. The dimensions and locations of equivalent rectangles representing the front and sides of a flame for this purpose should be determined as given in C.2 for columns and C.3 for beams. For all other purposes, the flame dimensions from annex C of ENV 1991-2-2 should be used.

(5) The configuration factor $\phi_{z,i}$ for a member face from which the flame is not visible should be taken as zero.

(6) A member face may be protected by a heat screen, see 4.2.5.4. A member face that is immediately adjacent to the compartment wall may also be treated as protected, provided that there are no openings in that part of the wall. All other member faces should be treated as unprotected.

C.2 Column not engulfed in flame

C.2.1 Radiative heat transfer

(1) A distinction should be made between a column located opposite an opening and a column located between openings, see figure C.2.

(2) If the column is opposite an opening, see figure C.3, the radiative heat flux I_z from the flame should be determined from:

$$I_z = \phi_z \varepsilon_z \sigma T_z^4 \tag{C.6}$$

where:

ϕ_z is the overall configuration factor of the column for heat from the flame, see C.1.4;

ε_z is the emissivity of the flame, see C.2.2;

T_z is the flame temperature [K] from C.2.3.

(3) If the column is between openings, see figure C.4, the total radiative heat flux I_z from the flames on each side should be determined from:

$$I_z = (\phi_{z,m} \varepsilon_{z,m} + \phi_{z,n} \varepsilon_{z,n}) \sigma T_z^4 \tag{C.7}$$

where:

$\phi_{z,m}$ is the overall configuration factor of the column for heat from flames on side m, see C.1.4;

$\phi_{z,n}$ is the overall configuration factor of the column for heat from flames on side n, see C.1.4;

$\varepsilon_{z,m}$ is the total emissivity of the flames on side m, see C.2.2;

$\varepsilon_{z,n}$ is the total emissivity of the flames on side n, see C.2.2.

C.2.2 Flame emissivity

(1) If the column is opposite an opening, the flame emissivity ε_z should be determined from the expression for ε given in annex C of ENV 1991-2-2, using the flame thickness λ at the level of the top of the openings. Provided that there is no awning or balcony above the opening λ may be taken as follows:

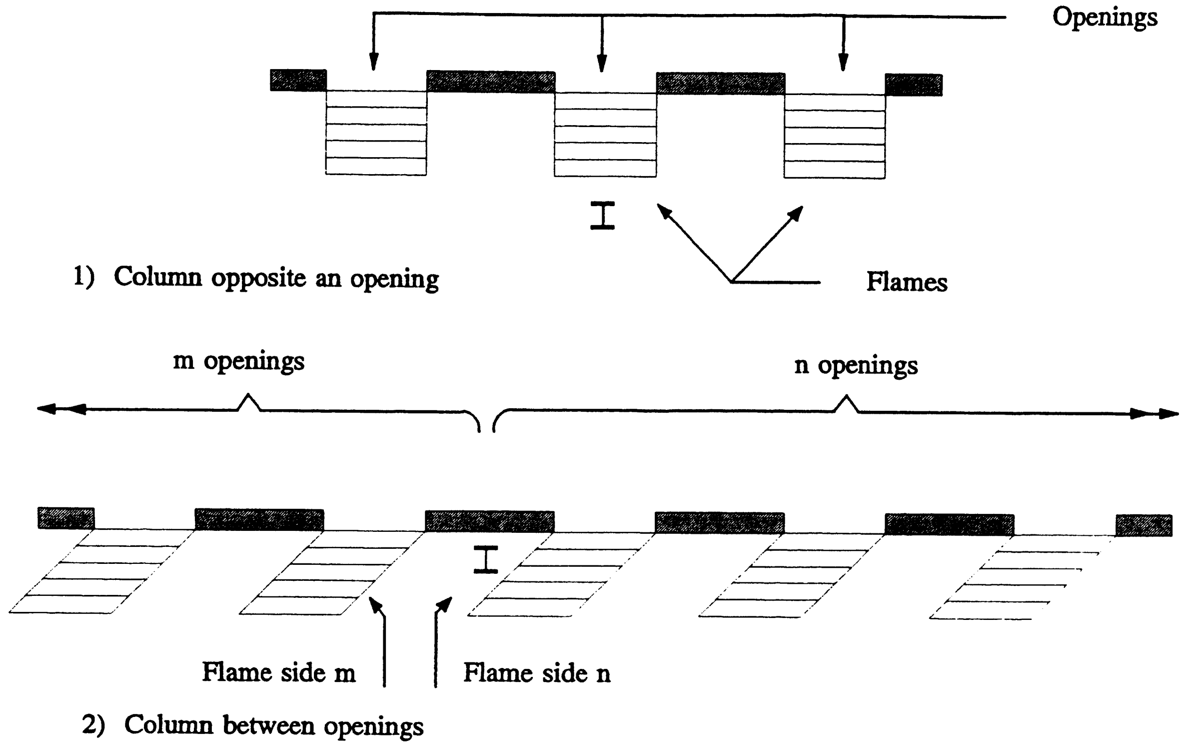
- for the 'no forced draught' condition:

$$\lambda = 2h/3 \tag{C.8a}$$

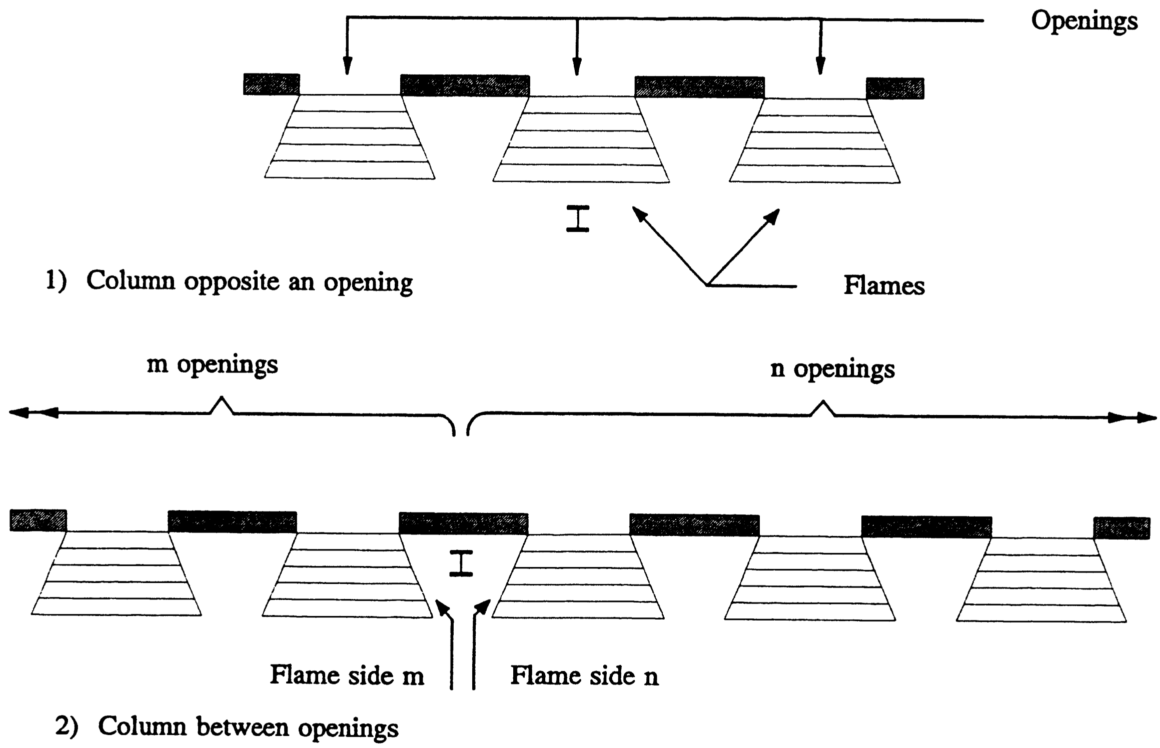
- for the 'forced draught' condition:

$$\lambda = x \quad \text{but } \lambda \leq hx/z \tag{C.8b}$$

where h, x and z are as given in annex C of ENV 1991-2-2.

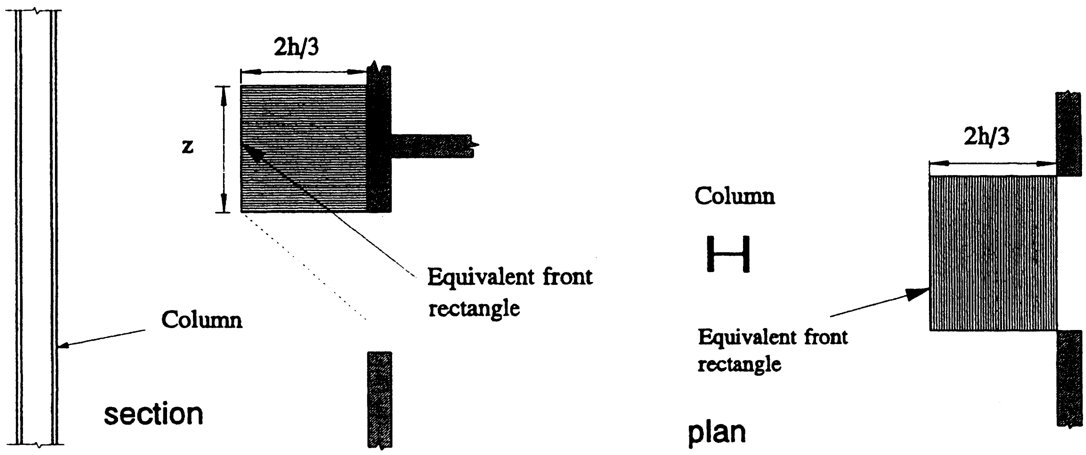


a) 'No forced draught' condition

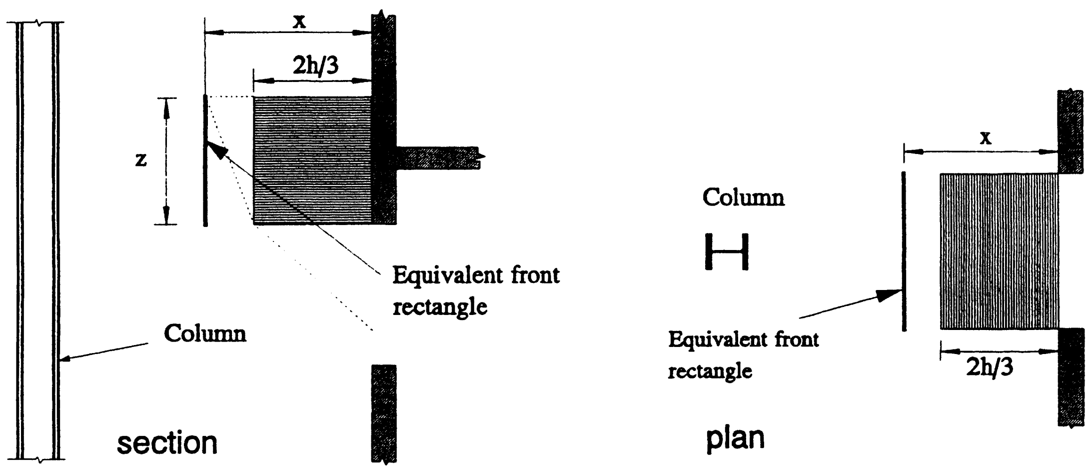


b) 'Forced draught' condition

Figure C.2: Column positions

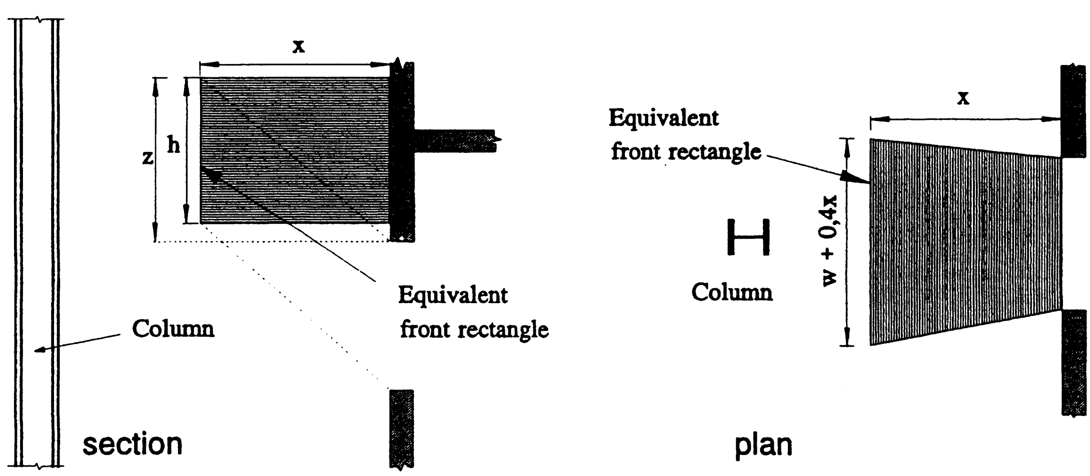


1) wall above and h < 1,25w



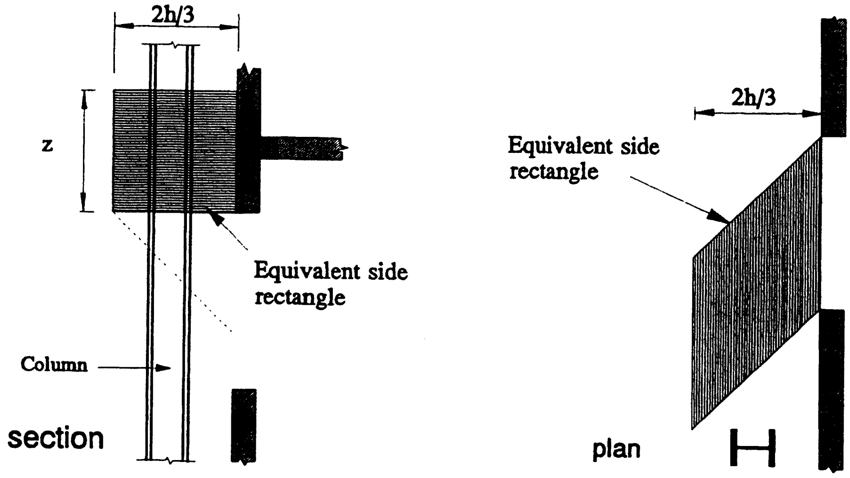
2) wall above and h > 1,25w or no wall above

a) 'No forced draught'

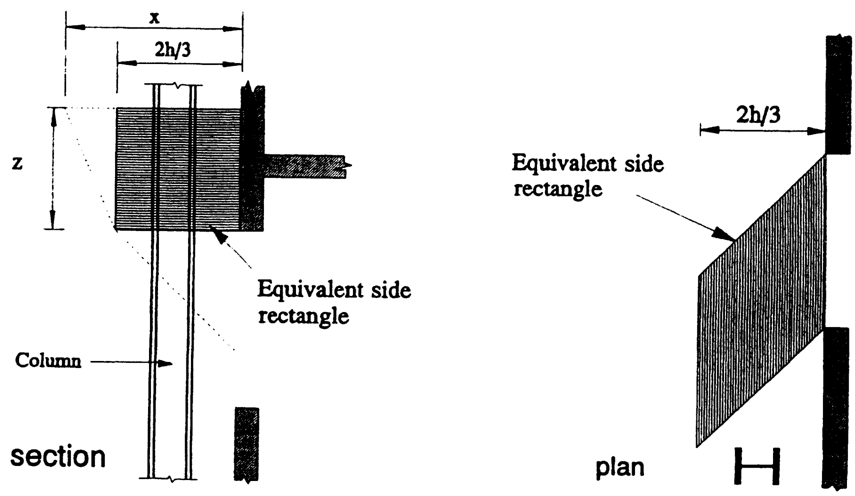


b) 'Forced draught'

Figure C.3: Column opposite opening

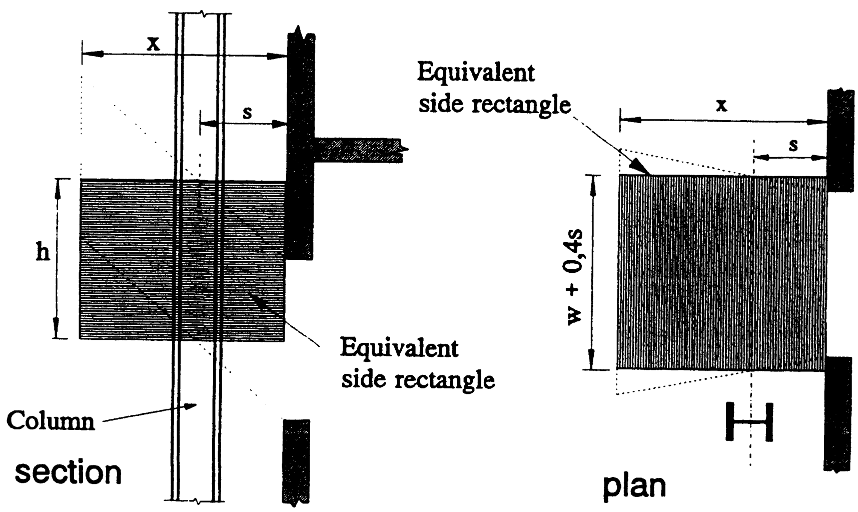


1) wall above and h < 1,25w



2) wall above and h > 1,25w or no wall above

a) **'No forced draught'**



b) **'Forced draught'**

Figure C.4: Column between openings

(2) If the column is between two openings, the total emissivities $\varepsilon_{z,m}$ and $\varepsilon_{z,n}$ of the flames on sides m and n should be determined from the expression for ε given in annex C of ENV 1991-2-2 using a value for the total flame thickness λ as follows:

- for side m: $$\lambda = \sum_{i=1}^{m} \lambda_i \tag{C.9a}$$

- for side n: $$\lambda = \sum_{i=1}^{n} \lambda_i \tag{C.9b}$$

where:

m is the number of openings on side m;

n is the number of openings on side n;

λ_i is the flame thickness for opening i.

(3) The flame thickness λ_i should be taken as follows:

- for the 'no forced draught' condition:

$$\lambda_i = w_i \tag{C.10a}$$

- for the 'forced draught' condition:

$$\lambda_i = w_i + 0{,}4s \tag{C.10b}$$

where:

w_i is the width of the opening;

s is the horizontal distance from the centreline of the column to the wall of the fire compartment, see figure C.1.

C.2.3 Flame temperature

(1) The flame temperature T_z should be taken as the temperature at the flame axis obtained from the expression for T_z given in annex C of ENV 1991-2-2, for the 'no forced draught' condition or the 'forced draught' condition as appropriate, at a distance ℓ from the opening, measured along the flame axis, as follows:

- for the 'no forced draught' condition:

$$\ell = h/2 \tag{C.11a}$$

- for the 'forced draught' condition:

  - for a column opposite an opening:

$$\ell = 0 \tag{C.11b}$$

  - for a column between openings ℓ is the distance along the flame axis to a point at a horizontal distance s from the wall of the fire compartment. Provided that there is no awning or balcony above the opening:

$$\ell = sX/x \tag{C.11c}$$

where X and x are as given in annex C of ENV 1991-2-2.

C.2.4 Flame absorptivity

(1) For the 'no forced draught' condition, the flame absorptivity a_z should be taken as zero.

(2) For the 'forced draught' condition, the flame absorptivity a_z should be taken as equal to the emissivity ε_z of the relevant flame, see C.2.2.

C.3 Beam not engulfed in flame

C.3.1 Radiative heat transfer

(1) Throughout C.3 it is assumed that the level of the bottom of the beam is not below the level of the top of the openings in the fire compartment.

(2) A distinction should be made between a beam that is parallel to the external wall of the fire compartment and a beam that is perpendicular to the external wall of the fire compartment, see figure C.5.

(3) If the beam is parallel to the external wall of the fire compartment, the average temperature of the steel member T_m should be determined for a point in the length of the beam directly above the centre of the opening. For this case the radiative heat flux I_z from the flame should be determined from:

$$I_z = \phi_z \varepsilon_z \sigma T_z^4 \tag{C.12}$$

where:

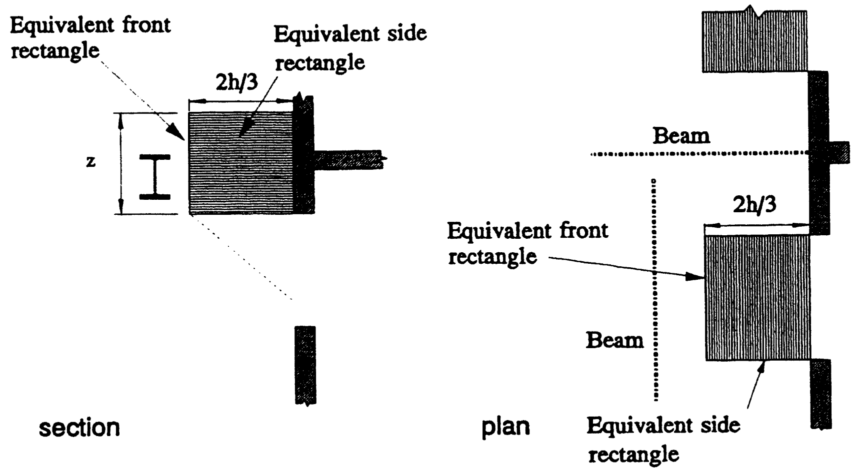
ϕ_z	is	the overall configuration factor for the flame directly opposite the beam, see C.1.4;
ε_z	is	the flame emissivity, see C.3.2;
T_z	is	the flame temperature from C.3.3 [K].

(4) If the beam is perpendicular to the external wall of the fire compartment, the average temperature in the beam should be determined at a series of points every 100 mm along the length of the beam. The average temperature of the steel member T_m should then be taken as the maximum of these values. For this case the radiative heat flux I_z from the flames should be determined from:

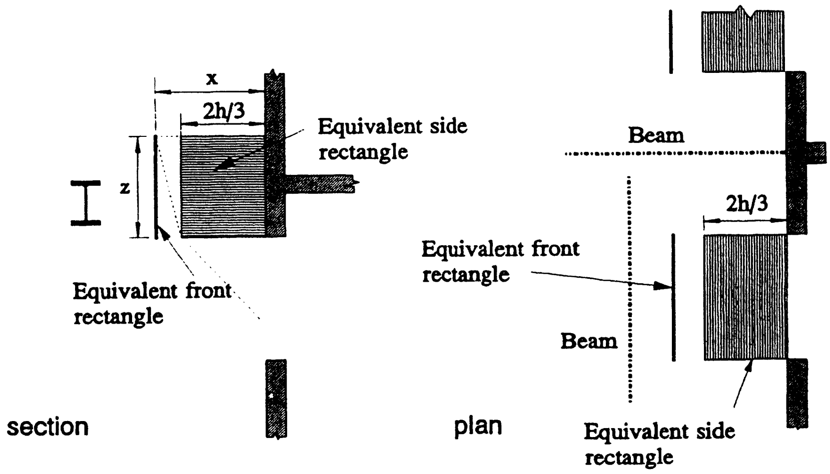
$$I_z = (\phi_{z,m} \varepsilon_{z,m} + \phi_{z,n} \varepsilon_{z,n}) \sigma T_z^4 \tag{C.13}$$

where:

$\phi_{z,m}$	is	the overall configuration factor of the beam for heat from flames on side m, see C.3.2;
$\phi_{z,n}$	is	the overall configuration factor of the beam for heat from flames on side n, see C.3.2;
$\varepsilon_{z,m}$	is	the total emissivity of the flames on side m, see C.3.3;
$\varepsilon_{z,n}$	is	the total emissivity of the flames on side n, see C.3.3;
T_z	is	the flame temperature [K], see C.3.4.

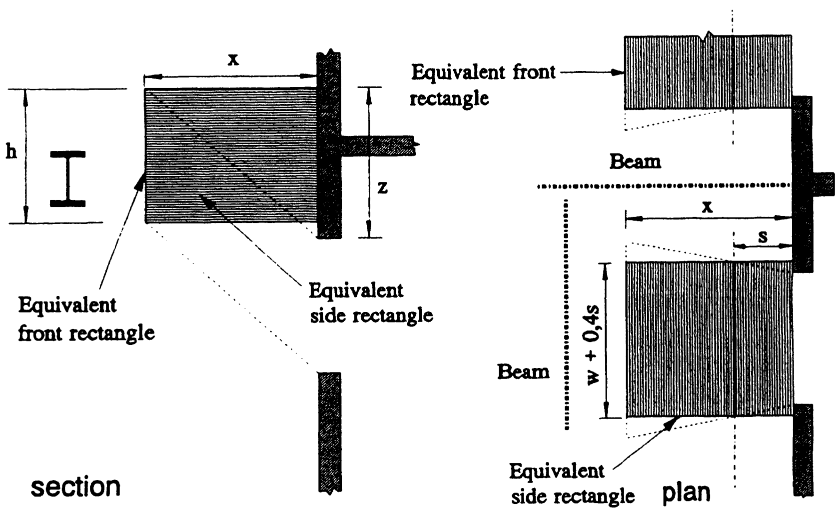


1) wall above and h < 1,25w



2) wall above and h > 1,25w or no wall above

a) **'No forced draught'**



b) **'Forced draught'**

Figure C.5: Beam not engulfed in flame

C.3.2 Flame emissivity

(1) If the beam is parallel to the external wall of the fire compartment, above an opening, the flame emissivity ε_z should be determined from the expression for ε given in annex C of ENV 1991-2-2, using a value for the flame thickness λ at the level of the top of the openings. Provided that there is no awning or balcony above the opening λ may be taken as follows:

- for the 'no forced draught' condition:

$$\lambda = 2h/3 \tag{C.14a}$$

- for the 'forced draught' condition:

$$\lambda = x \quad \text{but} \quad \lambda \leq hx/z \tag{C.14b}$$

where h, x and z are as given in annex C of ENV 1991-2-2

(2) If the beam is perpendicular to the external wall of the fire compartment, between two openings, the total emissivities $\varepsilon_{z,m}$ and $\varepsilon_{z,n}$ of the flames on sides m and n should be determined from the expression for ε given in annex C of ENV 1991-2-2 using a value for the flame thickness λ as follows:

- for side m:

$$\lambda = \sum_{i=1}^{m} \lambda_i \tag{C.15a}$$

- for side n:

$$\lambda = \sum_{i=1}^{n} \lambda_i \tag{C.15b}$$

where:

m is the number of openings on side m;

n is the number of openings on side n;

λ_i is the width of opening i.

(3) The flame thickness λ_i should be taken as follows:

- for the 'no forced draught' condition:

$$\lambda_i = w_i \tag{C.16a}$$

- for the 'forced draught' condition:

$$\lambda_i = w_i + 0{,}4s \tag{C.16b}$$

where:

w_i is the width of the opening;

s is the horizontal distance from the wall of the fire compartment to the point under consideration on the beam, see figure C.5.

C.3.3 Flame temperature

(1) The flame temperature T_z should be taken as the temperature at the flame axis obtained from the expression for T_z given in annex C of ENV 1991-2-2, for the 'no forced draught' or 'forced draught' condition as appropriate, at a distance ℓ from the opening, measured along the flame axis, as follows:

- for the 'no forced draught' condition:

$$\ell = h/2 \tag{C.17a}$$

- for the 'forced draught' condition:
  - for a beam parallel to the external wall of the fire compartment, above an opening:

$$\ell = 0 \tag{C.17b}$$

  - for a beam perpendicular to the external wall of the fire compartment, between openings ℓ is the distance along the flame axis to a point at a horizontal distance s from the wall of the fire compartment. Provided that there is no awning or balcony above the opening:

$$\ell = sX/x \tag{C.17c}$$

where X and x are as given in annex C of ENV 1991-2-2.

C.3.4 Flame absorptivity

(1) For the 'no forced draught' condition, the flame absorptivity a_z should be taken as zero.

(2) For the 'forced draught' condition, the flame absorptivity a_z should be taken as equal to the emissivity ε_z of the relevant flame, see C.3.2.

C.4 Column engulfed in flame

(1) The radiative heat flux I_z from the flames should be determined from:

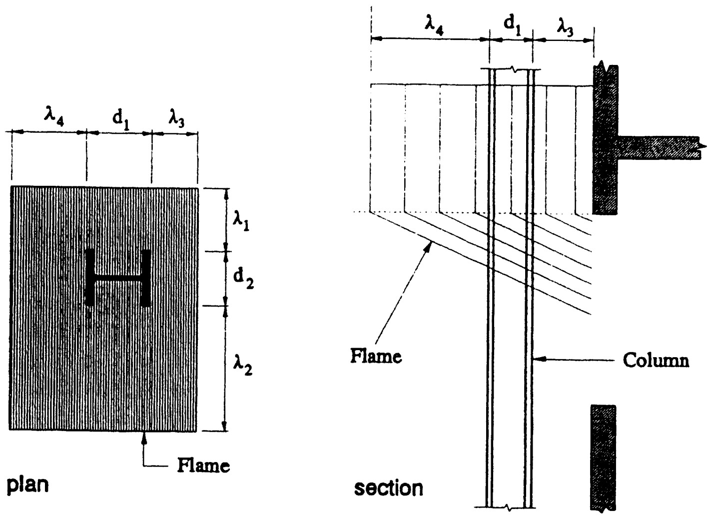
$$I_z = \frac{(I_{z,1} + I_{z,2})d_1 + (I_{z,3} + I_{z,4})d_2}{2(d_1 + d_2)} \tag{C.18}$$

with:

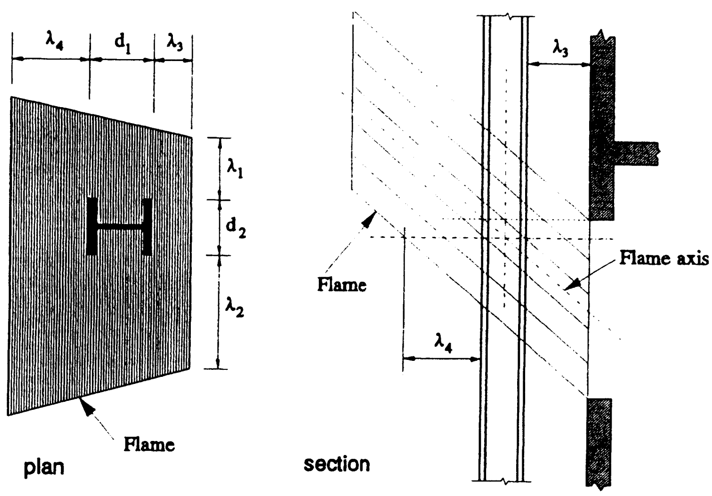
$$I_{z,1} = C_1 \varepsilon_{z,1} \sigma T_z^4$$
$$I_{z,2} = C_2 \varepsilon_{z,2} \sigma T_z^4$$
$$I_{z,3} = C_3 \varepsilon_{z,3} \sigma T_o^4$$
$$I_{z,4} = C_4 \varepsilon_{z,4} \sigma T_z^4$$

where:

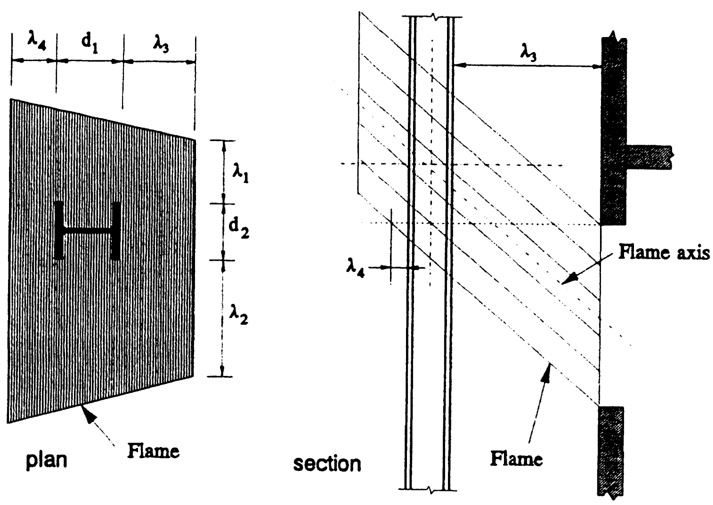
$I_{z,i}$	is	the radiative heat flux from the flame to column face i;
$\varepsilon_{z,i}$	is	the emissivity of the flames with respect to face i of the column;
i	is	the column face indicator (1), (2), (3) or (4);
C_i	is	the protection coefficient of member face i, see C.1.4;
T_z	is	the flame temperature [K];
T_o	is	the flame temperature at the opening [K] from annex C of ENV 1991-2-2.



a) 'No forced draught' condition



1) Flame axis intersects column axis below top of opening



2) Flame axis intersects column axis above top of opening

b) 'Forced draught' condition

Figure C.6: Column engulfed in flame

(2) The emissivity of the flames $\varepsilon_{z,i}$ for each of the faces 1, 2, 3 and 4 of the column should be determined from the expression for ε given in annex C of ENV 1991-2-2, using a flame thickness λ equal to the dimension λ_i indicated in figure C.6 corresponding to face i of the column.

(3) For the 'no forced draught' condition the values of λ_i at the level of the top of the opening should be used, see figure C.6(a).

(4) For the 'forced draught' condition, if the level of the intersection of the flame axis and the column centreline is below the level of the top of the opening, the values of λ_i at the level of the intersection should be used, see figure C.6(b)(1). Otherwise the values of λ_i at the level of the top of the opening should be used, see figure C.6(b)(2), except that if $\lambda_4 < 0$ at this level, the values at the level where $\lambda_4 = 0$ should be used.

(5) The flame temperature T_z should be taken as the temperature at the flame axis obtained from the expression for T_z given in annex C of ENV 1991-2-2 for the 'no forced draught' or 'forced draught' condition as appropriate, at a distance ℓ from the opening, measured along the flame axis, as follows:

- for the 'no forced draught' condition:

$$\ell = h/2 \tag{C.19a}$$

- for the 'forced draught' condition, ℓ is the distance along the flame axis to the level where λ_i is measured. Provided that there is no balcony or awning above the opening:

$$\ell = (\lambda_3 + 0{,}5d_1)X/x \quad \text{but } \ell \leq 0{,}5hX/z \tag{C.19b}$$

where h, X, x and z are as given in annex C of ENV 1991-2-2.

(6) The absorptivity a_z of the flames should be determined from:

$$a_z = \frac{\varepsilon_{z,1} + \varepsilon_{z,2} + \varepsilon_{z,3}}{3} \tag{C.20}$$

where $\varepsilon_{z,1}$, $\varepsilon_{z,2}$ and $\varepsilon_{z,3}$ are the emissivities of the flame for column faces 1, 2, and 3.

C.5 Beam fully or partially engulfed in flame

C.5.1 Radiative heat transfer

C.5.1.1 General

(1) Throughout C.5 it is assumed that the level of the bottom of the beam is not below the level of the top of the adjacent openings in the fire compartment.

(2) A distinction should be made between a beam that is parallel to the external wall of the fire compartment and a beam that is perpendicular to the external wall of the fire compartment, see figure C.7.

(3) If the beam is parallel to the external wall of the fire compartment, its average temperature T_m should be determined for a point in the length of the beam directly above the centre of the opening.

(4) If the beam is perpendicular to the external wall of the fire compartment, the value of the average temperature should be determined at a series of points every 100 mm along the length of the beam. The maximum of these values should then be adopted as the average temperature of the steel member T_m .

(5) The radiative heat flux I_z from the flame should be determined from:

$$I_z = \frac{(I_{z1} + I_{z2})\, d_1 + (I_{z3} + I_{z4})\, d_2}{2\,(d_1 + d_2)} \tag{C.21}$$

where:

$I_{z,i}$ is the radiative heat flux from the flame to beam face i;

i is the beam face indicator (1), (2), (3) or (4).

C.5.1.2 'No forced draught' condition

(1) For the 'no forced draught' condition, a distinction should be made between those cases where the top of the flame is above the level of the top of the beam and those where it is below this level.

(2) If the top of the flame is above the level of the top of the beam:

$$I_{z,1} = C_1\, \varepsilon_{z,1}\, \sigma T_o^4 \tag{C.22a}$$

$$I_{z,2} = C_2\, \varepsilon_{z,2}\, \sigma T_{z,2}^4 \tag{C.22b}$$

$$I_{z,3} = C_3\, \varepsilon_{z,3}\, \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \tag{C.22c}$$

$$I_{z,4} = C_4\, \varepsilon_{z,4}\, \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \tag{C.22d}$$

where:

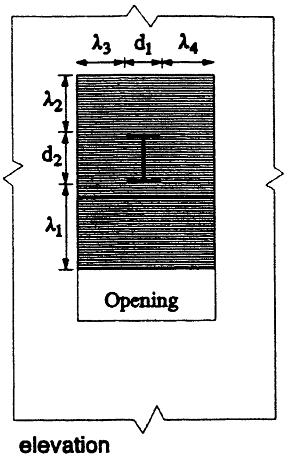
$\varepsilon_{z,i}$ is the emissivity of the flame with respect to face i of the beam, see C.5.2;

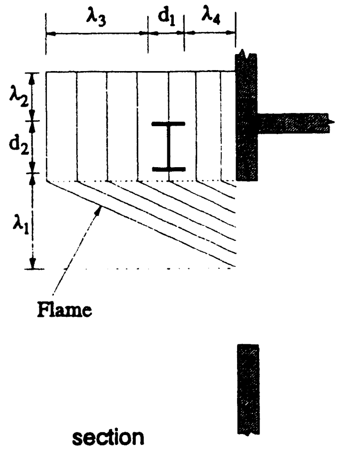
T_o is the temperature at the opening [K] from annex C of ENV 1991-2-2;

$T_{z,1}$ is the flame temperature [K] from annex C of ENV 1991-2-2, level with the bottom of the beam;

$T_{z,2}$ is the flame temperature [K] from annex C of ENV 1991-2-2, level with the top of the beam.

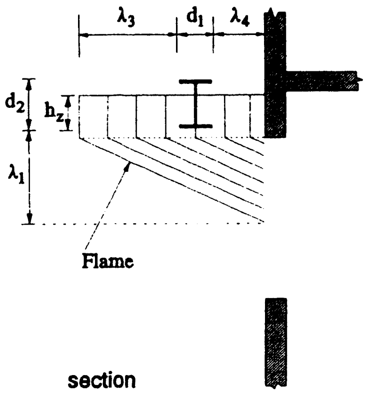
(3) In the case of a beam parallel to the external wall of the fire compartment C_4 may be taken as zero if the beam is immediately adjacent to the wall, see figure C.7.

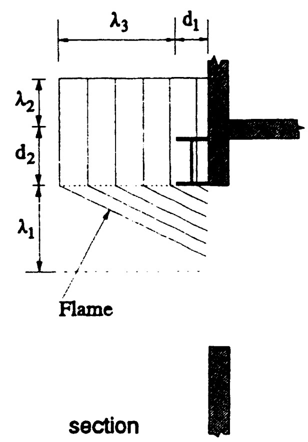




1) Beam perpendicular to wall

2) Beam parallel to wall

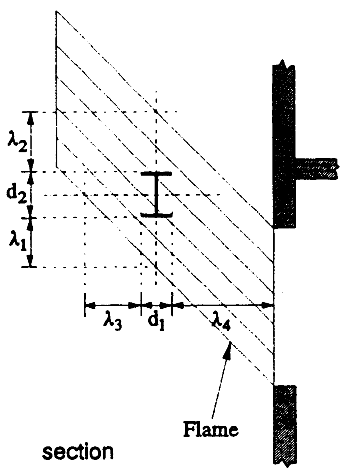


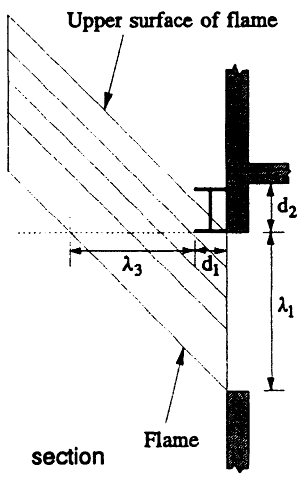


3) Top of flame below top of beam

4) Beam immediately adjacent to wall

a) 'No forced draught' condition





1) Beam not adjacent to wall

2) Beam immediately adjacent to wall

b) 'Forced draught' condition

Figure C.7: Beam engulfed in flame

(4) If the top of the flame is below the level of the top of the beam:

$$I_{z,1} = C_1 \varepsilon_{z,1} \sigma T_o^4 \quad \text{(C.23a)}$$

$$I_{z,2} = 0 \quad \text{(C.23b)}$$

$$I_{z,3} = (h_z / d_2) C_3 \varepsilon_{z,3} \sigma (T_{z,1}^4 + T_x^4)/2 \quad \text{(C.23c)}$$

$$I_{z,4} = (h_z / d_2) C_4 \varepsilon_{z,4} \sigma (T_{z,1}^4 + T_x^4)/2 \quad \text{(C.23d)}$$

where:

T_x is the flame temperature at the flame tip [813 K];

h_z is the height of the top of the flame above the bottom of the beam.

C.5.1.3 'Forced draught' condition

(1) For the 'forced draught' condition, in the case of beams parallel to the external wall of the fire compartment a distinction should be made between those immediately adjacent to the wall and those not immediately adjacent to it, see figure C.7.

(2) For a beam parallel to the wall, but not immediately adjacent to it, or for a beam perpendicular to the wall:

$$I_{z,1} = C_1 \varepsilon_{z,1} \sigma T_o^4 \quad \text{(C.24a)}$$

$$I_{z,2} = C_2 \varepsilon_{z,2} \sigma T_{z,2}^4 \quad \text{(C.24b)}$$

$$I_{z,3} = C_3 \varepsilon_{z,3} \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \quad \text{(C.24c)}$$

$$I_{z,4} = C_4 \varepsilon_{z,4} \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \quad \text{(C.24d)}$$

(3) If the beam is parallel to the wall and immediately adjacent to it, only the bottom face should be taken as engulfed in flame but one side and the top should be taken as exposed to radiative heat transfer from the upper surface of the flame, see figure C.7(b)(2). Thus:

$$I_{z,1} = C_1 \varepsilon_{z,1} \sigma T_o^4 \quad \text{(C.25a)}$$

$$I_{z,2} = \phi_{z,2} C_2 \varepsilon_{z,2} \sigma T_{z,2}^4 \quad \text{(C.25b)}$$

$$I_{z,3} = \phi_{z,3} C_3 \varepsilon_{z,3} \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \quad \text{(C.25c)}$$

$$I_{z,4} = 0 \quad \text{(C.25d)}$$

where $\phi_{z,i}$ is the configuration factor relative to the upper surface of the flame, for face i of the beam, from annex D.

C.5.2 Flame emissivity

(1) The emissivity of the flame ε_{zi} for each of the faces 1, 2, 3 and 4 of the beam should be determined from the expression for ε given in annex C of ENV 1991-2-2, using a flame thickness λ equal to the dimension λ_i indicated in figure C.7 corresponding to face i of the beam.

C.5.3 Flame absorptivity

(1) The absorptivity of the flame a_z should be determined from:

$$a_z = 1 - e^{-0,3h} \quad \text{(C.26)}$$

Annex D [informative]

Configuration factor

(1) The configuration factor ϕ is defined in 1.4.1. It measures the fraction of the total radiative heat leaving a given radiating surface that arrives at a given receiving surface. Its value depends on the size of the radiating surface, on the distance from the radiating surface to the receiving surface and on their relative orientation.

(2) In this annex all radiating surfaces are assumed to be rectangular in shape. They comprise the windows and other openings in fire compartment walls and the equivalent rectangular surfaces of flames, see C.1.4.

(3) In calculating the configuration factor for a given situation, a rectangular envelope should first be drawn around the cross-section of the member receiving the radiative heat transfer, as indicated in figure D.1. The value of ϕ should then be determined for the mid-point P of each face of this rectangle.

(4) The configuration factor for each receiving surface should be determined as the sum of the contributions from each of the zones on the radiating surface (normally four) that are visible from the point P on the receiving surface, as indicated in figures D.2 and D.3. These zones should be defined relative to the point X where a horizontal line perpendicular to the receiving surface meets the plane containing the radiating surface. No contribution should be taken from zones such as the shaded zones on figure D.3 that are not visible from the point P.

(5) If the point X lies outside the radiating surface, the effective configuration factor should be determined by adding the contributions of the two rectangles extending from X to the farther side of the radiating surface, then subtracting the contributions of the two rectangles extending from X to the nearer side of the radiating surface.

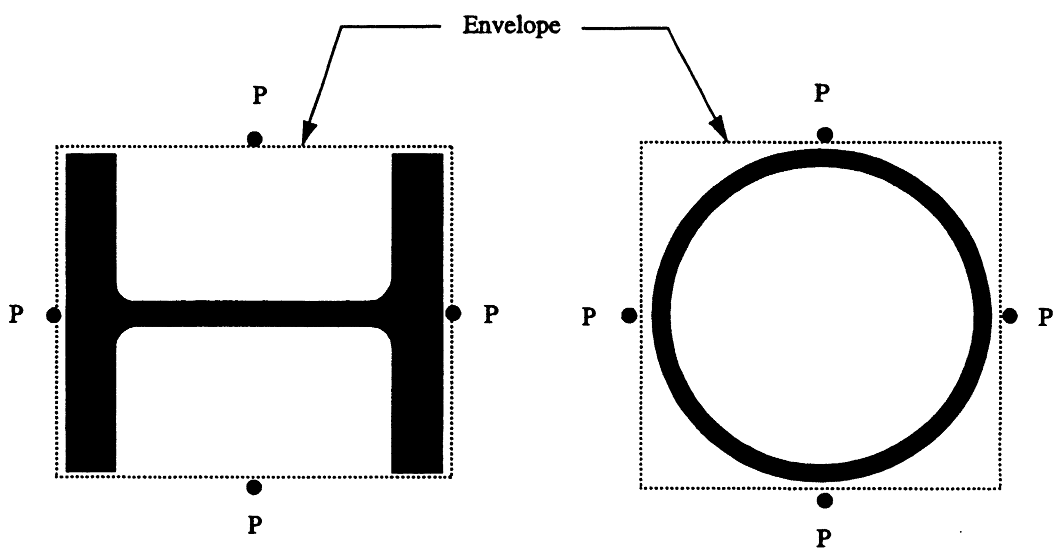


Figure D.1: Envelope of receiving surfaces

(6) The contribution of each zone should be determined as follows:

a) receiving surface parallel to radiating surface:

$$\phi = \frac{1}{2\pi}\left[\frac{a}{(1+a^2)^{0,5}}\tan^{-1}\left[\frac{b}{(1+a^2)^{0,5}}\right] + \frac{b}{(1+b^2)^{0,5}}\tan^{-1}\left[\frac{a}{(1+b^2)^{0,5}}\right]\right] \quad \text{(D.1)}$$

with:

$a = h/s$

$b = w/s$

where:

s is the distance from P to X;

h is the height of the zone on the radiating surface;

w is the width of that zone.

b) receiving surface perpendicular to radiating surface:

$$\phi = \frac{1}{2\pi}\left[\tan^{-1}(a) - \frac{1}{(1+b^2)^{0,5}}\tan^{-1}\left[\frac{a}{(1+b^2)^{0,5}}\right]\right] \quad \text{(D.2)}$$

c) receiving surface in a plane at angle θ to the radiating surface:

$$\phi = \frac{1}{2\pi}\left[\tan^{-1}(a) - \frac{(1-b\cos\theta)}{(1+b^2-2b\cos\theta)^{0,5}}\tan^{-1}\left[\frac{a}{(1+b^2-2b\cos\theta)^{0,5}}\right]\right.$$

$$\left. + \frac{a\cos\theta}{(a^2+\sin^2\theta)^{0,5}}\left[\tan^{-1}\left[\frac{(b-\cos\theta)}{(a^2+\sin^2\theta)^{0,5}}\right] + \tan^{-1}\left[\frac{\cos\theta}{(a^2+\sin^2\theta)^{0,5}}\right]\right]\right] \quad \text{(D.3)}$$

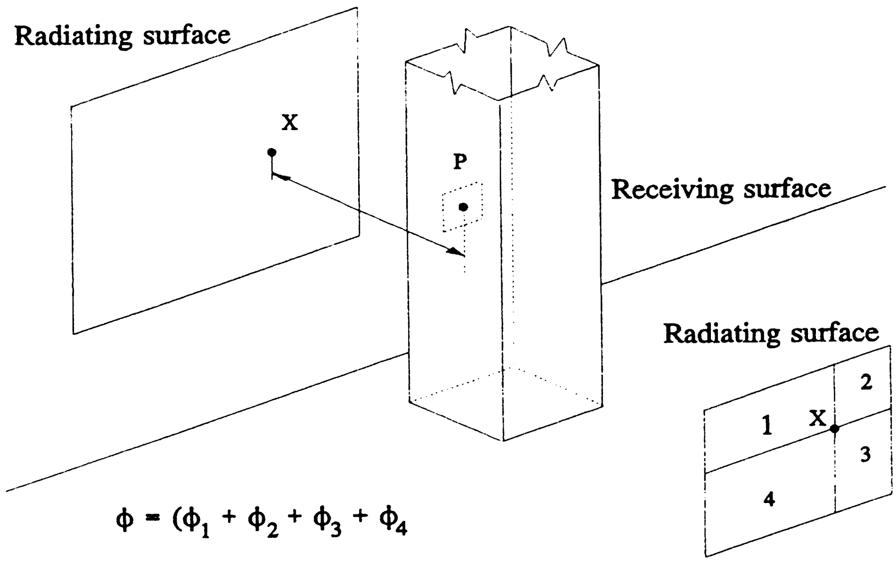


Figure D.2: Receiving surface in a plane parallel to that of the radiating surface

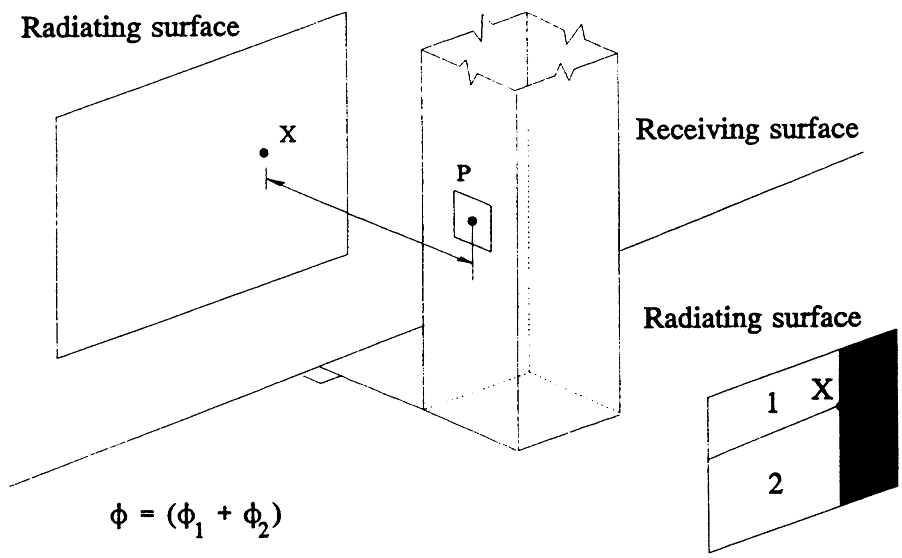


Figure D.3: Receiving surface perpendicular to the plane of the radiating surface

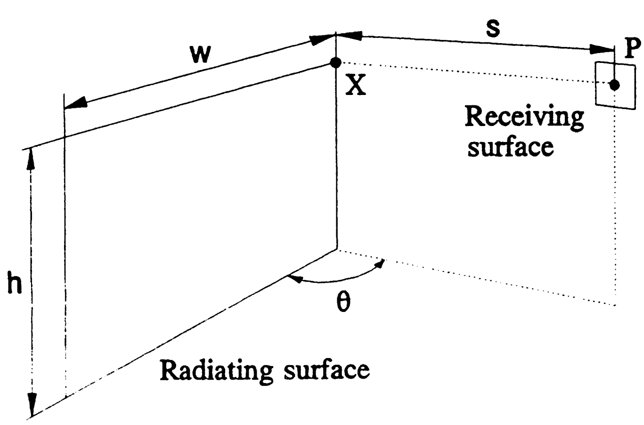


Figure D.4: Receiving surface in a plane at angle θ to that of the radiating surface

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