British Standard

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)

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

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.*

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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 $\boxed{\quad\quad}$ 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.*

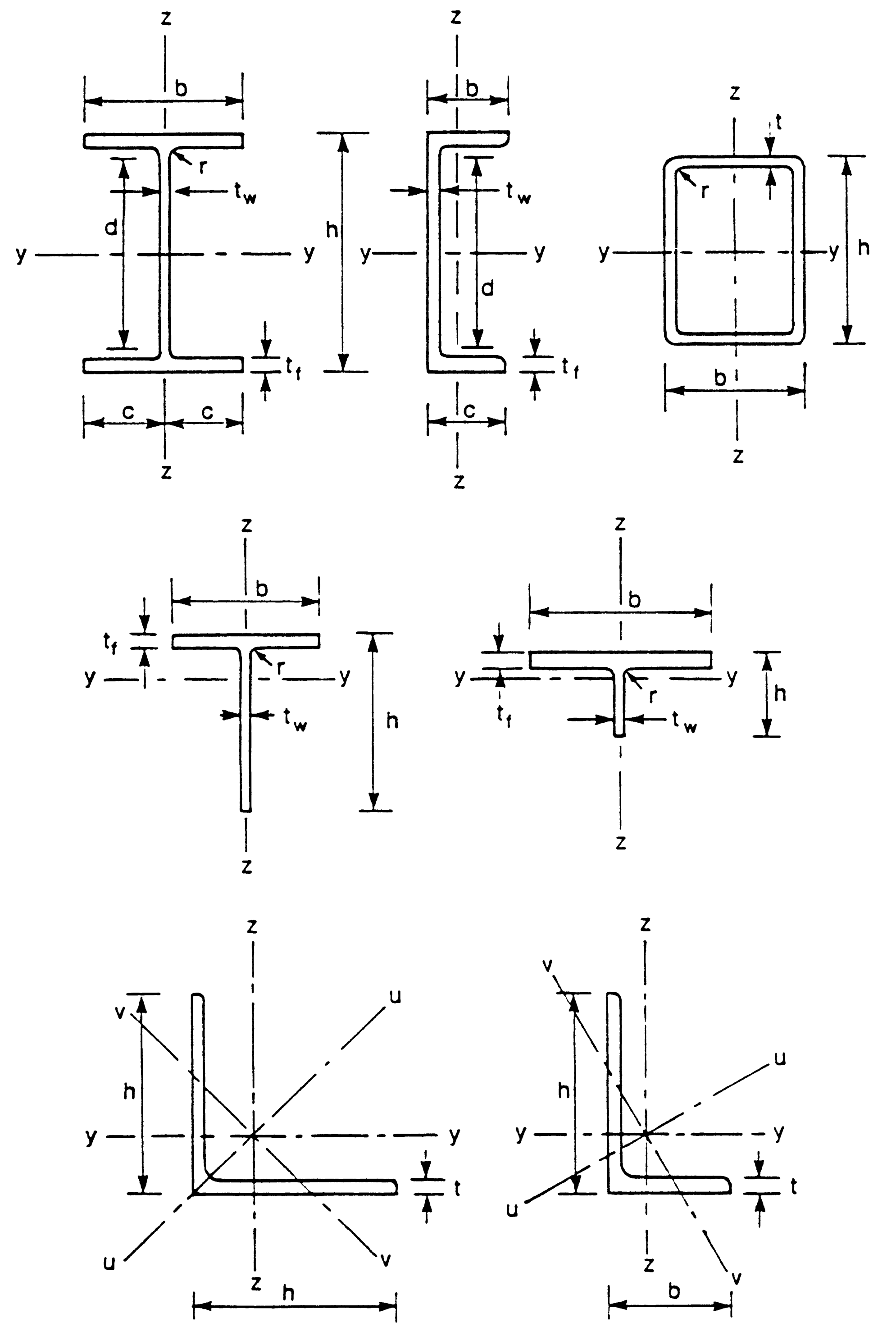


Figure 1.1 — Dimensions and axes of sections

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.

[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:

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

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:

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

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:

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

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:

$$E_d \leq C_d \tag{2.8}$$

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):

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

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

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

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:*

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

- *considering all unfavourable variable actions:*

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

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:

$$E_d \leq C_d \text{ or } E_d \leq R_d \qquad (2.13)$$

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:

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

Frequent combination:

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

Quasi-permanent combination:

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

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:*

$$\sum_j G_{k,j} + Q_{k,1} \tag{2.17}$$

- *considering all unfavourable variable actions:*

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

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]			
	$t \leq 40$ mm		$40 \text{ mm} < t \leq 100 \text{ mm}$[b]	
	f_y (N/mm^2)	f_u (N/mm^2)	f_y (N/mm^2)	f_u (N/mm^2)
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:*

$$\delta_{max} = \delta_1 + \delta_2 \; \Pi \; \delta_0 \tag{4.1}$$

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	—

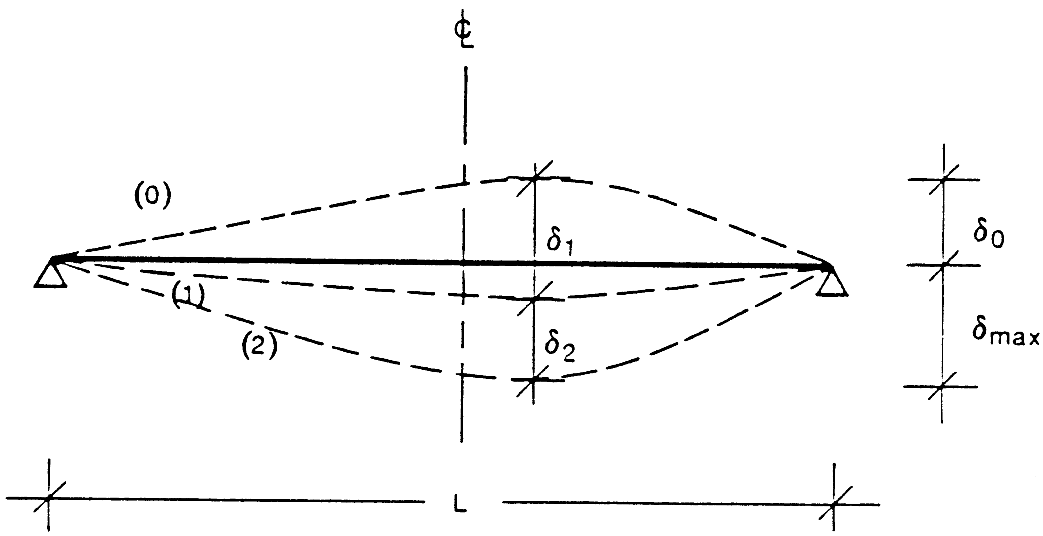


Figure 4.1 — Vertical deflections to be considered

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.*

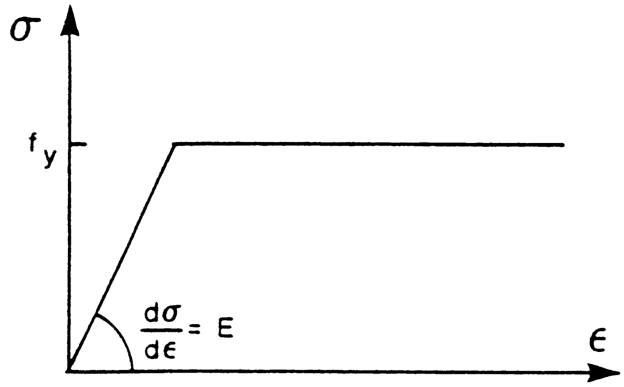


Figure 5.2.1 — Bi-linear stress-strain relationship

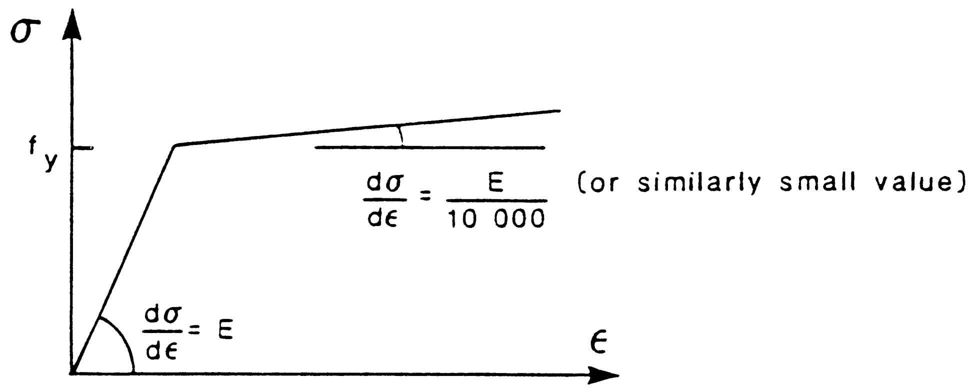


Figure 5.2.2 — Alternative bi-linear stress-strain relationship (for use in elasto-plastic analysis only)

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:

$$\bar{\lambda} > 0{,}5\ [Af_y/N_{Sd}]^{0{,}5} \qquad (5.1)$$

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:

$$\phi = k_c \, k_s \, \phi_o \qquad (5.2)$$

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:

$$e_o = k_r L/500. \qquad (5.3)$$

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:*

$$N = M/h \qquad (5.4)$$

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.

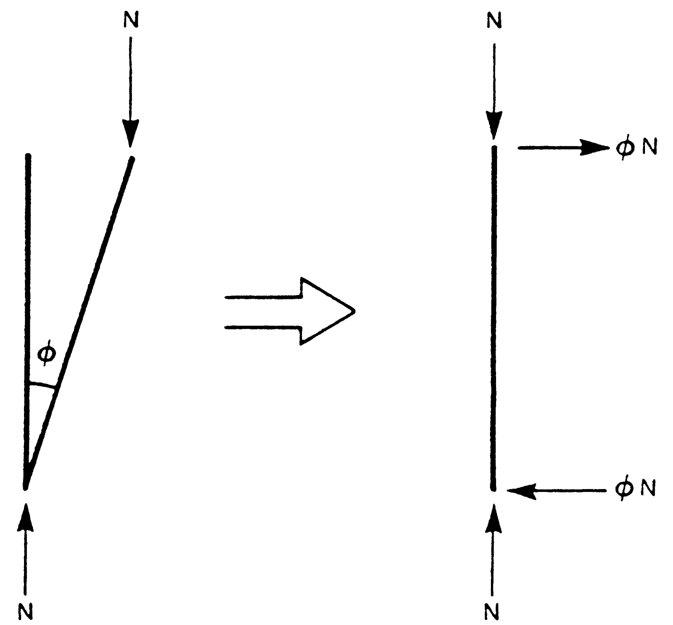


Figure 5.2.3 — Replacement of initial sway imperfections by equivalent horizontal forces

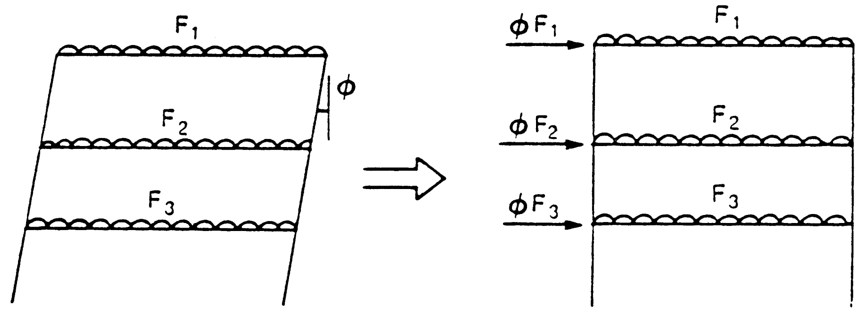


Figure 5.2.4 — Equivalent horizontal forces

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***

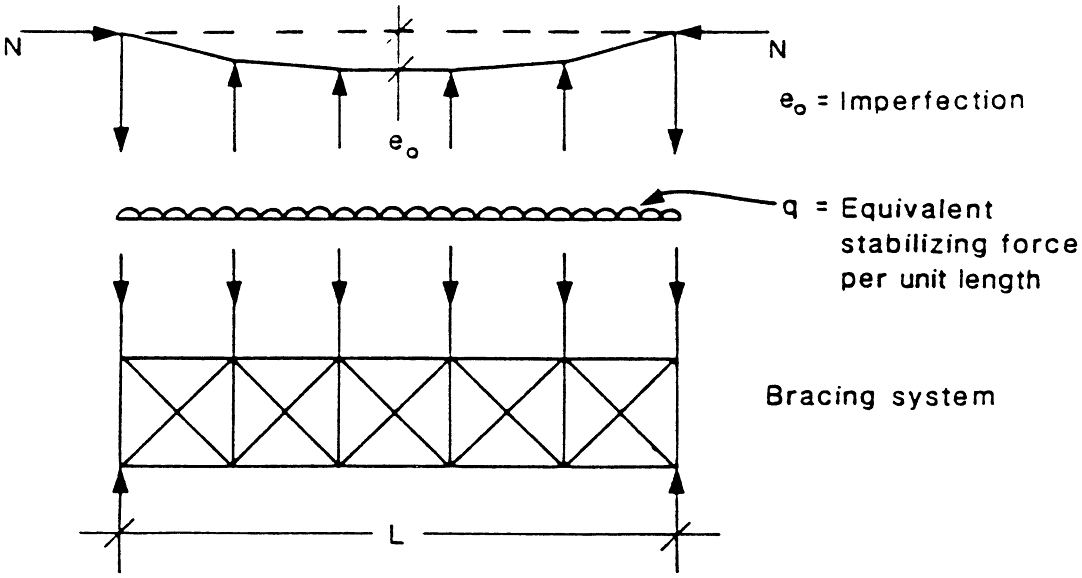
(1) All structures shall have sufficient stiffness to limit lateral sway. This may be supplied by:

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

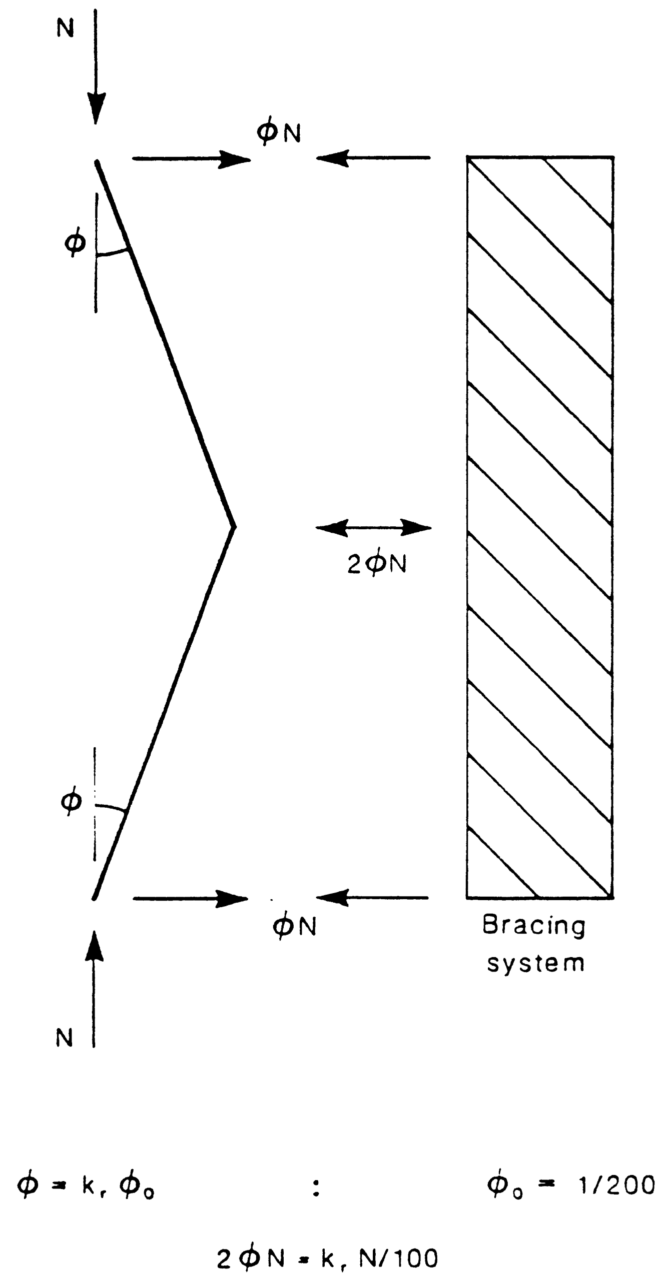


Figure 5.2.6 — Bracing forces at splices in compression elements

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.*

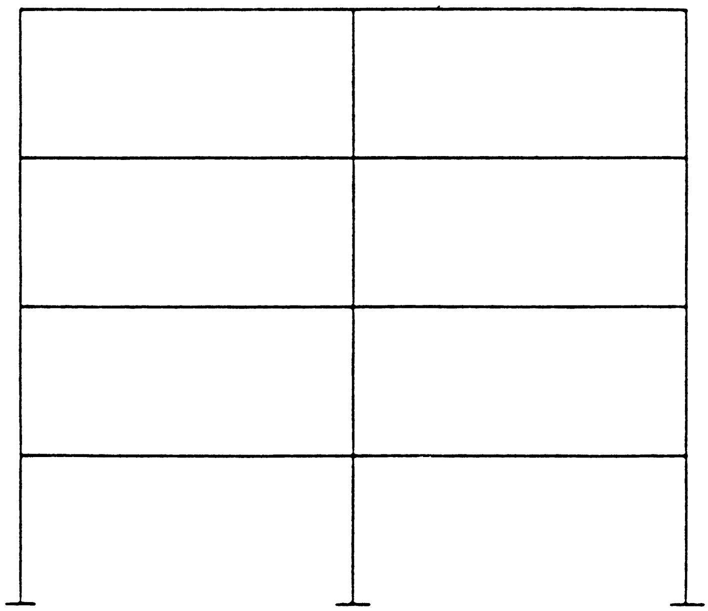


Figure 5.2.7 — Building frame with beams connecting each column at each storey level

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.*

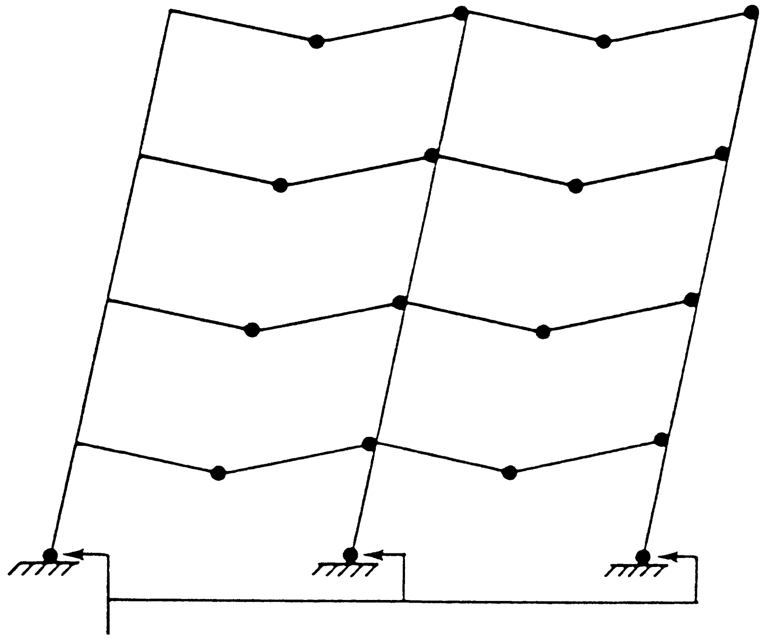


Figure 5.2.8 — Sway mechanism involving plastic hinges in columns at bases only

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].

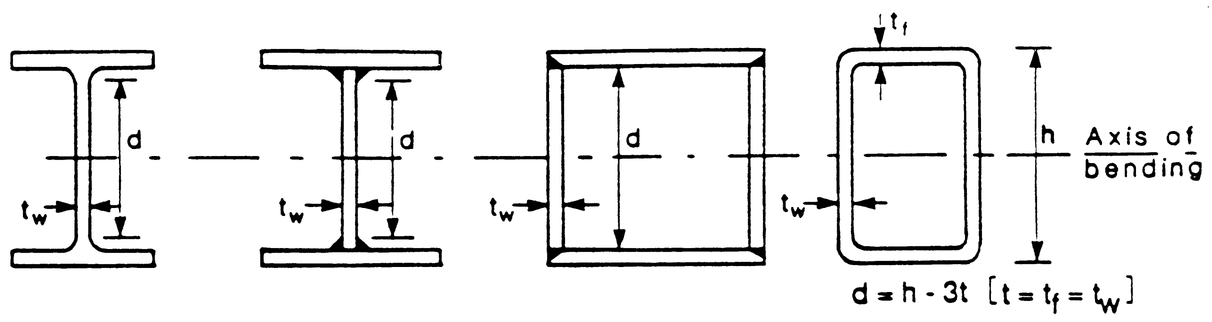
(5) *The resistance of a cross-section with a Class 2 compression flange but a Class 3 web may alternatively be determined by treating the web as an effective Class 2 web with a reduced effective area, using the method given in ENV 1994-1-1 Eurocode 4-1.1*[14].

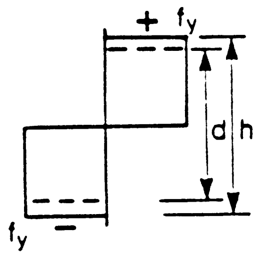
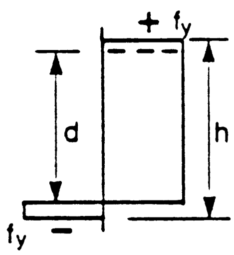
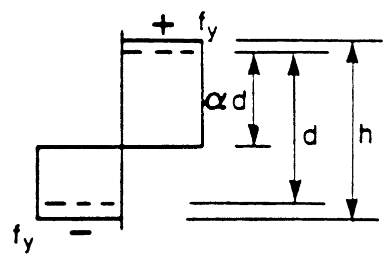
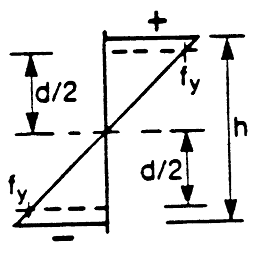
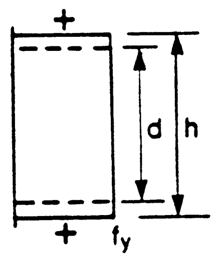
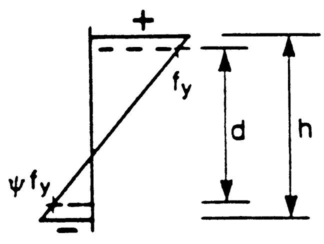
(6) When any of the compression elements of a cross-section is Class 4 the cross-section shall be designed as a Class 4 cross-section, see **5.3.5**.

[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)

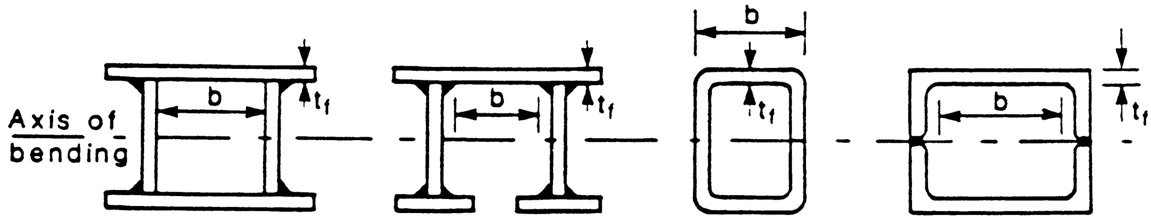


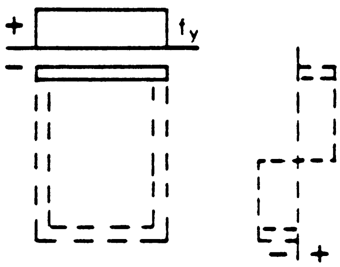
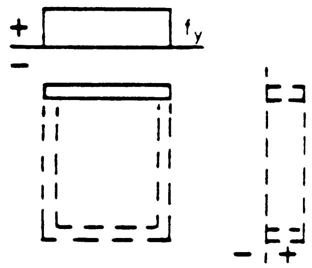
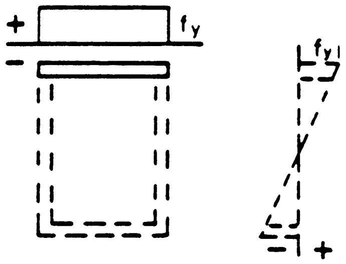
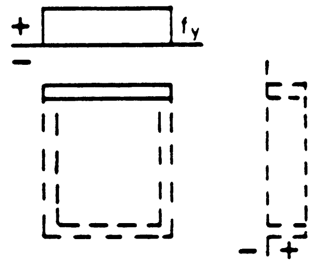
Class	Web subject to bending	Web subject to compression	Web subject to bending and compression
Stress distribution in element (compression positive)			
1	$d/t_w \leq 72\varepsilon$	$d/t_w \leq 33\varepsilon$	when $\alpha > 0{,}5$: $d/t_w \leq 396\varepsilon/(13\alpha - 1)$ when $\alpha \leq 0{,}5$: $d/t_w \leq 36\varepsilon/\alpha$
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) Internal flange elements: (internal elements parallel to axis of bending)

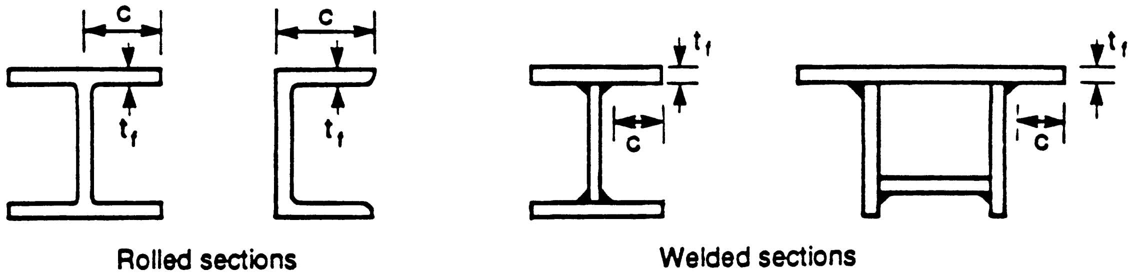


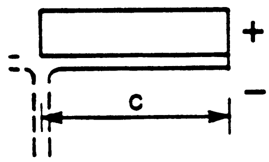
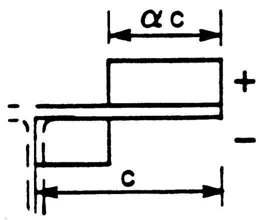
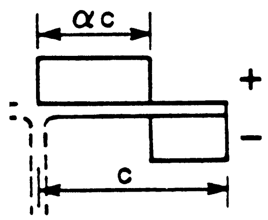
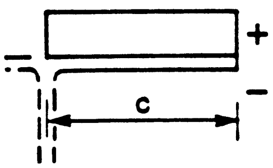
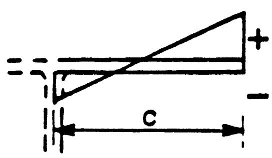
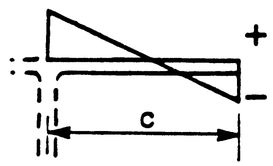
Class	Type	Section in bending	Section in compression
Stress distribution in element and across section (compression positive)			
1	Rolled Hollow Section Other	$(b-3t_f)/t_f \leq 33\varepsilon$ $b/t_f \leq 33\varepsilon$	$(b-3t_f)/t_f \leq 42\varepsilon$ $b/t_f \leq 42\varepsilon$
2	Rolled Hollow Section 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)

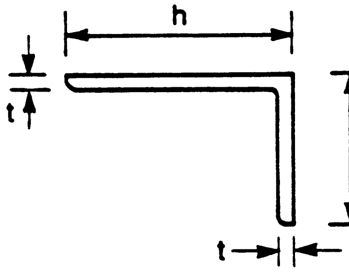
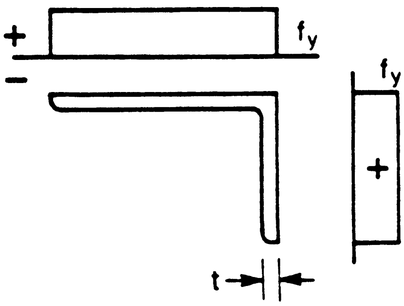
c) Outstand flanges:

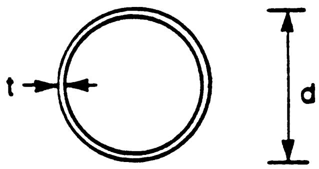


Class	Type of section	Flange subject to compression	Flange subject to compression and bending: Tip in compression	Flange subject to compression and bending: Tip in tension
Stress distribution in element (compression positive)				
1	Rolled	$c/t_f \le 10\varepsilon$	$c/t_f \le \frac{10\varepsilon}{\alpha}$	$c/t_f \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
	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	$\frac{h}{t} \leq 15\varepsilon \quad : \quad \frac{b+h}{2t} \leq 11{,}5\varepsilon$

e) Tubular sections:	
	
Class	**Section in bending and/or compression**
1	d/t $\leq 50\varepsilon^2$
2	d/t $\leq 70\varepsilon^2$
3	d/t $\leq 90\varepsilon^2$

$\varepsilon = \sqrt{235/f_y}$	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* $\bar{\lambda}_p \leq 0{,}673$: $\rho = 1$
- *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:*

$$\bar{\lambda}_p = [f_y / \sigma_{cr}]^{0,5} = (\bar{b}/t) / (28,4\, \epsilon \sqrt{k_\sigma})$$

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:*

$$\Delta M = N\, e_N \qquad (5.12)$$

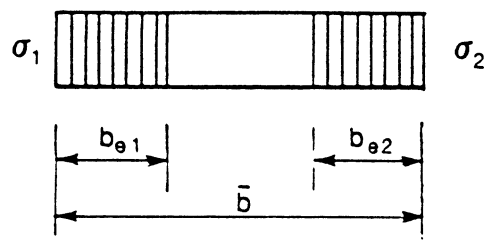
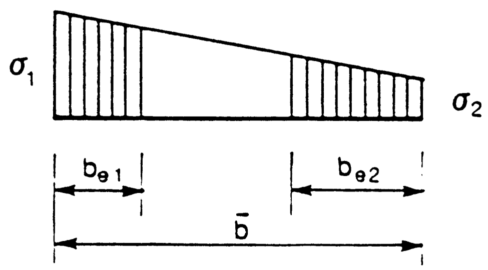
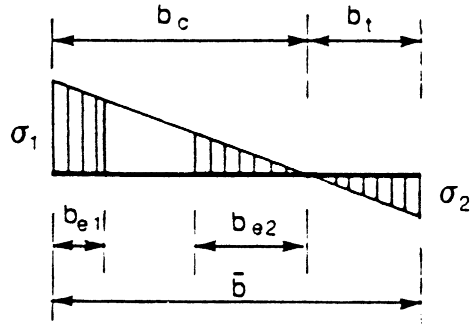
where e_N *is the shift of the centroidal axis when the effective cross-section is subject to uniform compression, see Figure 5.3.1.*

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

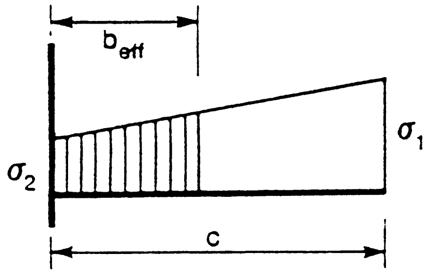
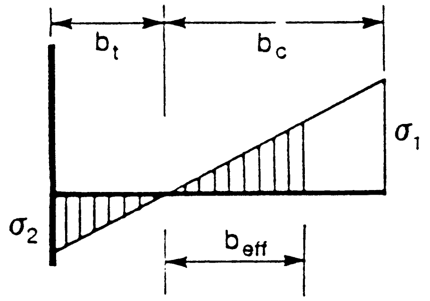
Stress distribution (compression positive)	Effective width b_{eff}
	$\psi = 1$: $b_{eff} = \rho \bar{b}$ $b_{e1} = 0.5 b_{eff}$ $b_{e2} = 0,5 b_{eff}$
	$1 > \psi \geq 0$: $b_{eff} = \rho \bar{b}$ $b_{e1} = \dfrac{2 b_{eff}}{5 - \psi}$ $b_{e2} = b_{eff} - b_{e1}$
	$\psi < 0$: $b_{eff} = \rho\, b_c = \rho\, \bar{b}/(1 - \psi)$ $b_{e1} = 0,4 b_{eff}$ $b_{e2} = 0,6 b_{eff}$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -\varkappa$	-1	$-1 > \psi > -2$
Buckling factor $k\sigma$	4,0	$\dfrac{8,2}{1,05 + \psi}$	7,81	$7,81 - 6,29\,\psi + 9,78\,\psi^2$	23,9	$5,98(1 - \psi)^2$

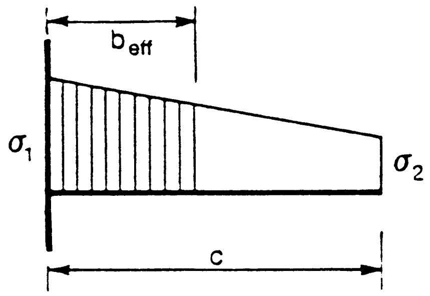
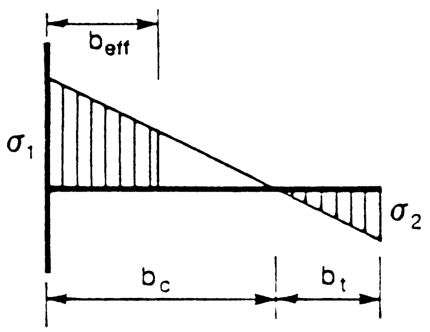
Alternatively, for $1 \geq \psi \geq -1$:

$$k_\sigma = \frac{16}{\left[(1+\psi)^2 + 0,112(1-\psi)^2\right]^{0,5} + (1+\psi)}$$

Table 5.3.3 — Outstand compression elements

<table>
<tr><th>Stress distribution
(compression positive)</th><th>Effective width b_{eff}</th></tr>
<tr><td></td><td><u>$1 > \psi \geq 0$:</u>
$b_{eff} = \rho c$</td></tr>
<tr><td></td><td><u>$\psi < 0$:</u>
$b_{eff} = \rho b_c = \rho c/(1 - \psi)$</td></tr>
</table>

$\psi = \sigma_2/\sigma_1$	1	0	– 1	$1 \geq \psi \geq - 1$
Buckling factor k_σ	0,43	0,57	0,85	$0,57 - 0,21\ \psi + 0,07\ \psi^2$

<table>
<tr><td></td><td><u>$1 > \psi \geq 0$:</u>
$b_{eff} = \rho c$</td></tr>
<tr><td></td><td><u>$\psi < 0$:</u>
$b_{eff} = \rho b_c = \rho c/(1 - \psi)$</td></tr>
</table>

$\psi = \sigma_2/\sigma_1$	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

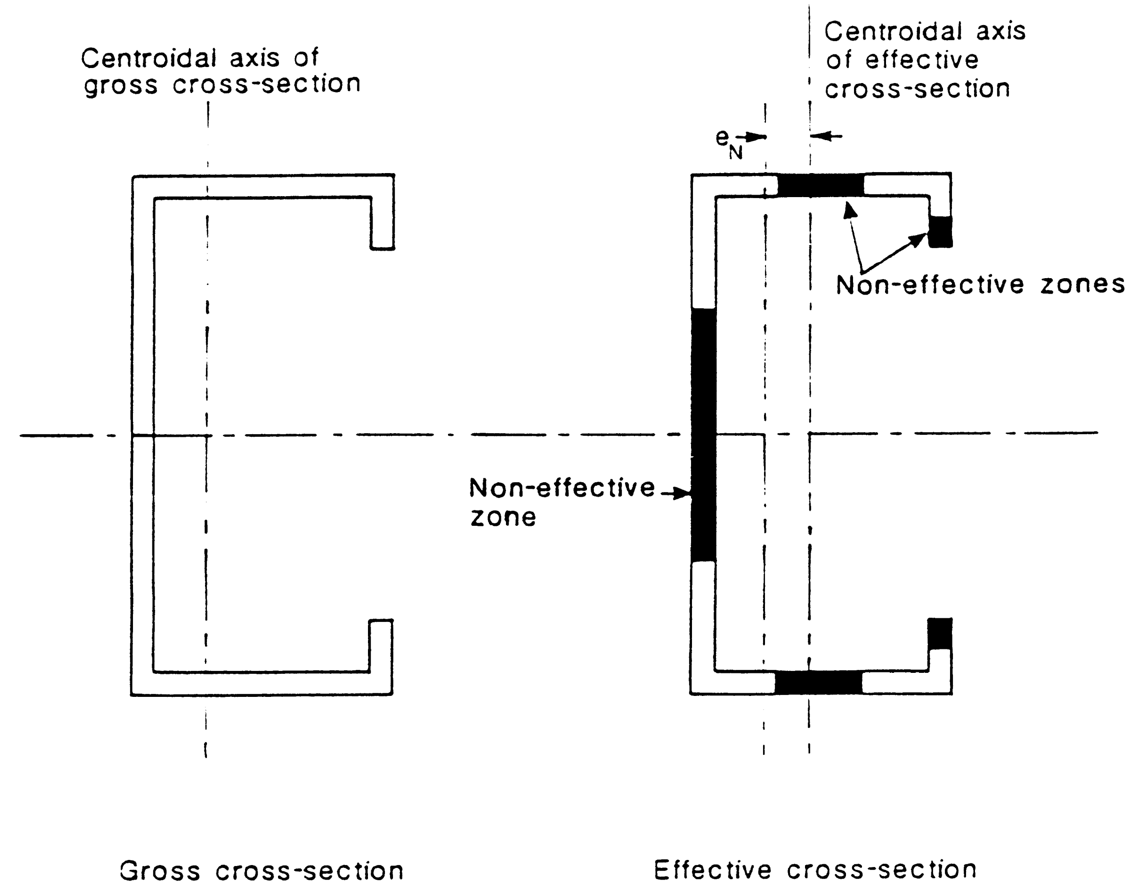


Figure 5.3.1 — Class 4 cross-sections — axial force

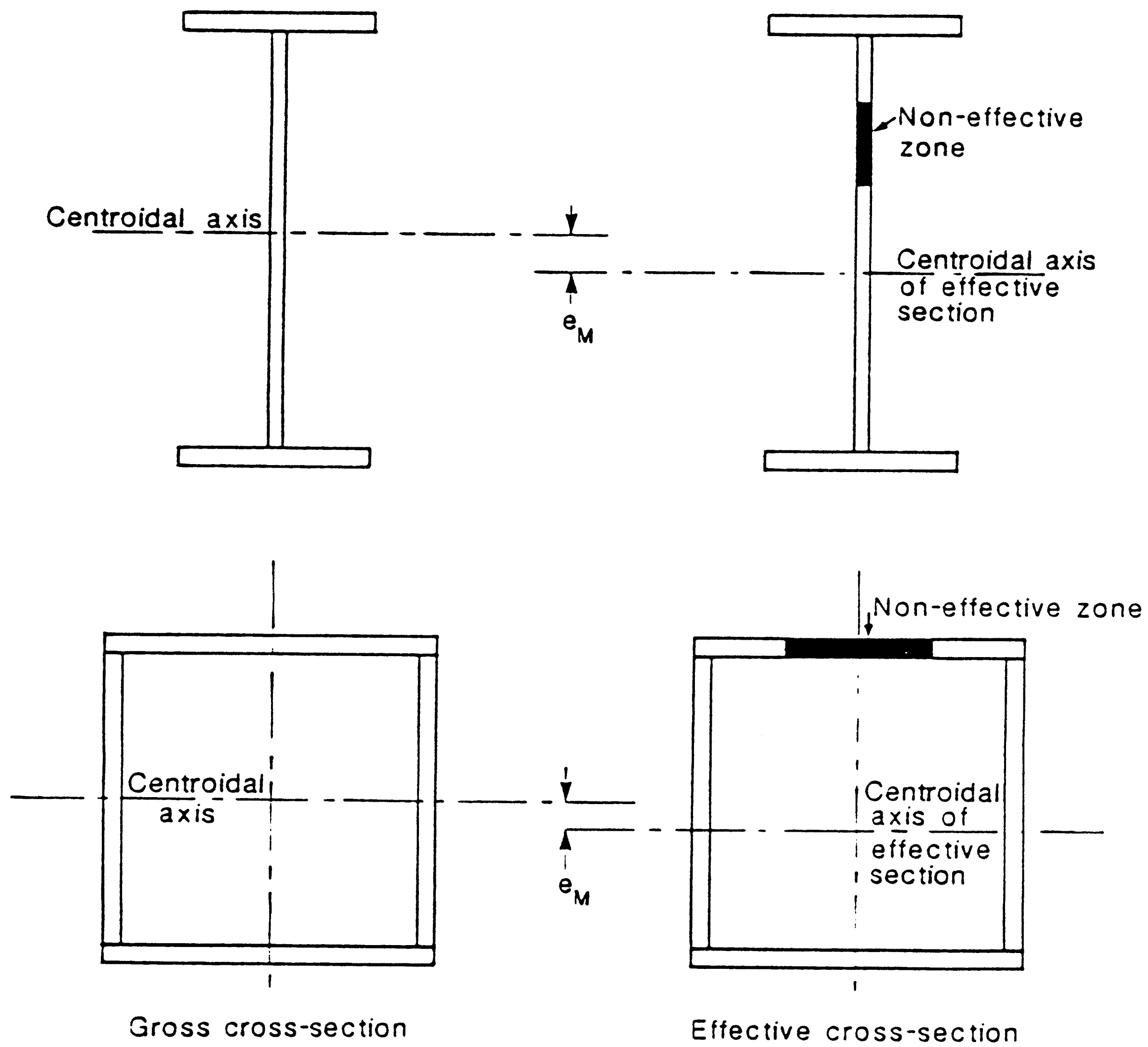


Figure 5.3.2 — Class 4 cross-sections — bending moment

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.

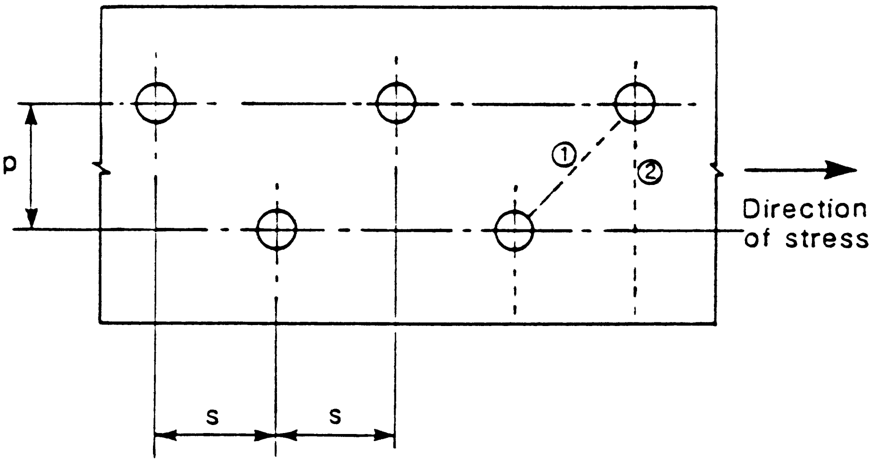


Figure 5.4.1 — Staggered holes

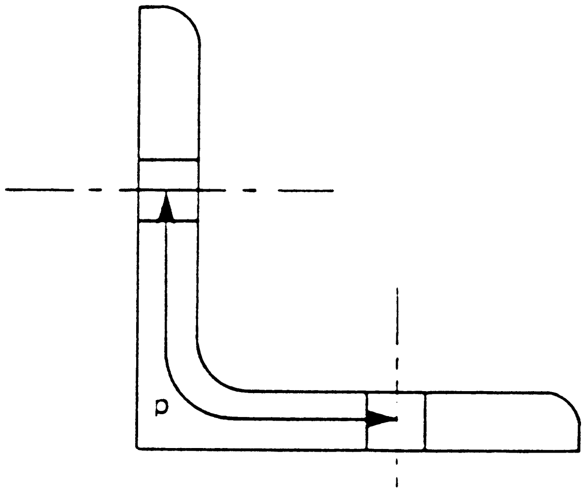


Figure 5.4.2 — Angles with holes in both legs

(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:

$$M_{e\ell.Rd} = W_{e\ell} f_y / \gamma_{M0} \qquad (5.18)$$

(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:

$$0{,}9\ [A_{f.net}/A_f] \geq [f_y/f_u]\ [\gamma_{M2}/\gamma_{M0}] \qquad (5.19)$$

(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:

$$V_{Sd} \leq V_{p\ell.Rd} \qquad (5.20)$$

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:

$$A_{v.net} \geq (f_y/f_u)\, A_v \qquad (5.21)$$

(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:*

$$M_{V.Rd} = \left[W_{pl} - \frac{\rho A_v^2}{4\, t_w}\right] f_y/\gamma_{M0} \quad \text{but } M_{V.Rd} \leq M_{c.Rd} \qquad (5.22)$$

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 $(1 - \rho)\, f_y$ 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:

$$M_{Sd} \leq M_{N.Rd} \qquad (5.23)$$

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:

$$\frac{M_{Sd}}{M_{pl.Rd}} + \left[\frac{N_{Sd}}{N_{pl.Rd}}\right]^2 \leq 1 \qquad (5.24)$$

(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:*

$M_{Ny.Rd} = M_{p\ell.y.Rd}(1 - n)/(1 - 0,5a)$ *but* $M_{Ny.Rd} \leq M_{p\ell.y.Rd}$ *(5.25)*

for $n \leq a$: $M_{Nz.Rd} \leq M_{p\ell.z.Rd}$

$$\text{for } n > a: \quad M_{Nz.Rd} = M_{pl.z.Rd}\left[1 - \left[\frac{n-a}{1-a}\right]^2\right] \qquad (5.26)$$

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:*

$M_{Ny.Rd} = 1,11M_{p\ell.y.Rd}(1 - n)$ *but* $M_{Ny.Rd} \leq M_{p\ell.y.Rd}$ *(5.27)*

for $n \leq 0,2$: $M_{Nz.Rd} \leq M_{p\ell.z.Rd}$

for $n > 0,2$: $M_{Nz.Rd} = 1,56M_{p\ell.z.Rd}(1 - n)(n + 0,6)$ *(5.28)*

(7) *For cross-sections without bolt holes, the following approximations may be used for rectangular structural hollow sections of uniform thickness:*

$M_{Ny.Rd} = M_{pt.y.Rd}(1 - n)/(1 - 0,5a_w)$ *but* $M_{Ny.Rd} \leq M_{p\ell.y.Rd}$ *(5.29)*

$M_{Nz.Rd} = M_{p\ell.z.Rd}(1 - n)/(1 - 0,5a_f)$ *but* $M_{Nz.Rd} \leq M_{p\ell.z.Rd}$ *(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:*

  $M_{N.Rd} = 1,26M_{p\ell.Rd}(1 - n)$ *but* $M_{N.Rd} \leq M_{p\ell.Rd}$ *(5.31)*

- *for a rectangular section:*

  $M_{Ny.Rd} = 1,33M_{p\ell.y.Rd}(1 - n)$ *but* $M_{Ny.Rd} \leq M_{p\ell.y.Rd}$ *(5.32)*

  $M_{Nz.Rd} = M_{p\ell.z.Rd}(1 - n)/(0,5 + ht/A)$ *but* $M_{Nz.Rd} \leq M_{p\ell.z.Rd}$ *(5.33)*

(10) *For cross-sections without bolt holes, the following approximation may be used for circular tubes of uniform thickness:*

$M_{N.Rd} = 1,04M_{p\ell.Rd}(1 - n^{1,7})$ *but* $M_{N.Rd} \leq M_{p\ell.Rd}$ *(5.34)*

(11) *For bi-axial bending the following approximate criterion may be used:*

$$\left[\frac{M_{y.Sd}}{M_{Ny.Rd}}\right]^{\alpha} + \left[\frac{M_{z.Sd}}{M_{Nz.Rd}}\right]^{\beta} \leq 1 \qquad (5.35)$$

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:*

$$\frac{N_{Sd}}{N_{p\ell.Rd}} + \frac{M_{y.Sd}}{M_{p\ell.y.Rd}} + \frac{M_{z.Sd}}{M_{p\ell.z.Rd}} \leq 1 \qquad (5.36)$$

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:

$$\sigma_{x.Ed} \leq f_{yd} \qquad (5.37)$$

where $f_{yd} = f_y/\gamma_{M0}$

(2) *For cross-sections without holes for fasteners, the above criterion becomes:*

$$\frac{N_{Sd}}{Af_{yd}} + \frac{M_{y.Sd}}{W_{el.y}\, f_{yd}} + \frac{M_{z.Sd}}{W_{el.z}\, f_{yd}} \leq 1 \qquad (5.38)$$

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:

$$\sigma_{x.Ed} \leq f_{yd} \qquad (5.39)$$

where $f_{yd} = f_y/\gamma_{M1}$

(2) *For cross-sections without holes for fasteners, the above criterion becomes:*

$$\frac{N_{Sd}}{A_{eff}\, f_{yd}} + \frac{M_{y.Sd} + N_{Sd}\, e_{Ny}}{W_{eff.y}\, f_{yd}} + \frac{M_{z.Sd} + N_{Sd}\, e_{Nz}}{W_{eff.z}\, f_{yd}} \leq 1 \qquad (5.40)$$

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 $\rho = (2V_{Sd}/V_{p\ell.Rd} - 1)^2$

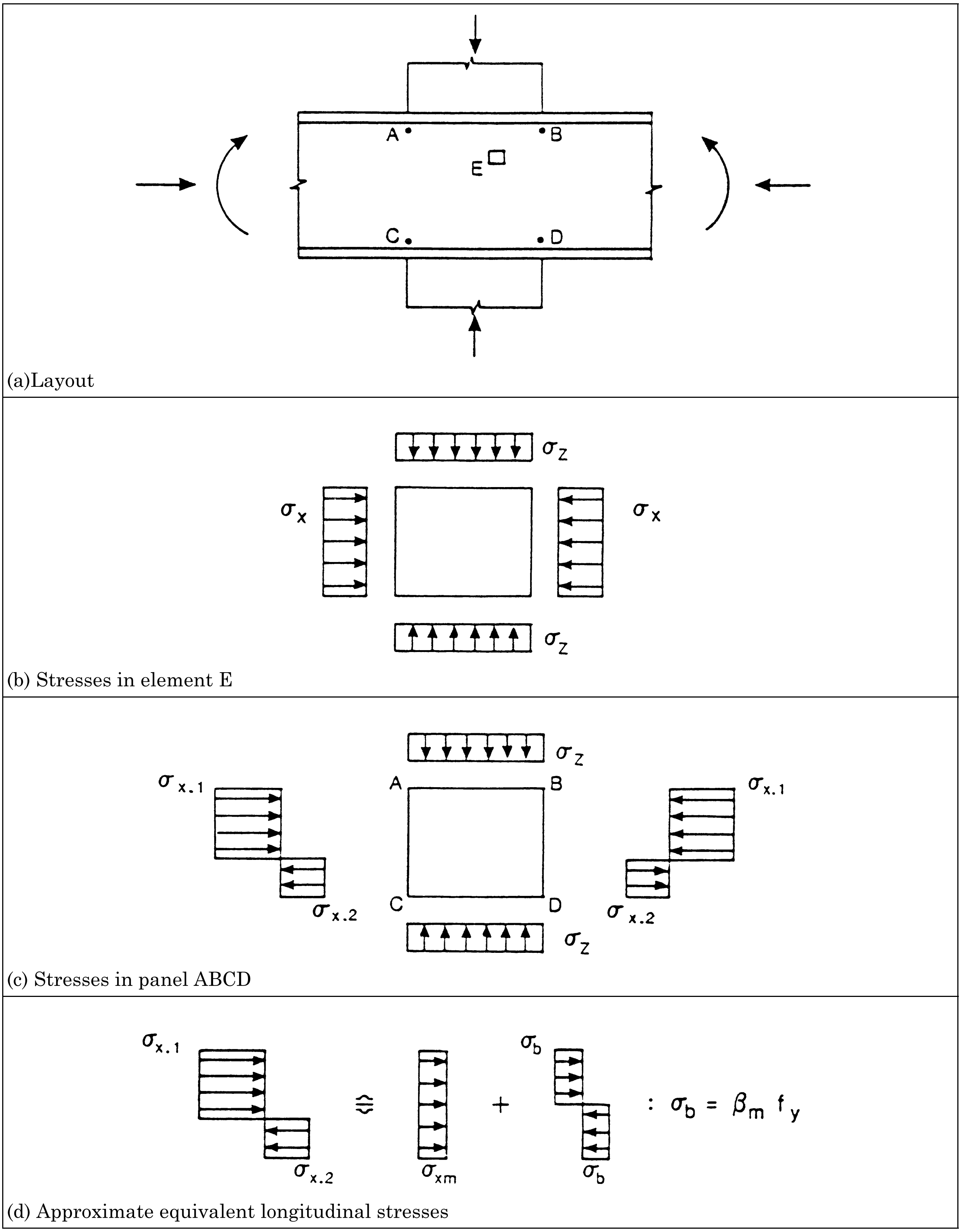


Figure 5.4.3 — Stresses in web panel due to bending moment, axial force and transverse force

(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:*

$$\left[\frac{\sigma_{xm.Ed}}{f_{yd}}\right]^2 + \left[\frac{\sigma_{z.Ed}}{f_{yd}}\right]^2 - k\left[\frac{\sigma_{xm.Ed}}{f_{yd}}\right]\left[\frac{\sigma_{z.Ed}}{f_{yd}}\right] \leq 1 - \beta_m - \rho \qquad (5.44)$$

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:

$$N_{b.Rd} = \chi\, \beta_A A f_y / \gamma_{M1} \qquad (5.45)$$

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:

$$\chi = \frac{1}{\phi + [\phi^2 - \bar{\lambda}^2]^{0,5}} \quad \text{but } \chi \leq 1 \qquad (5.46)$$

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:

$$\lambda = \ell / i \qquad (5.47)$$

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:

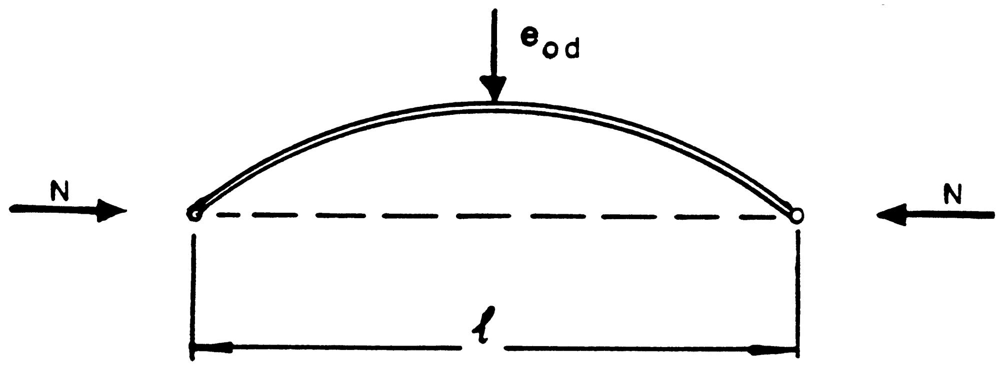
a) the basic yield strength f_{yb} of the flat sheet material out of which the member is made by cold-forming, with buckling curve b.

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 $k_\gamma \geq 1{,}0$

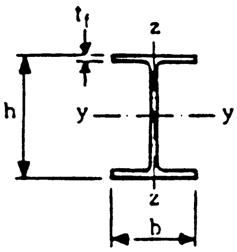
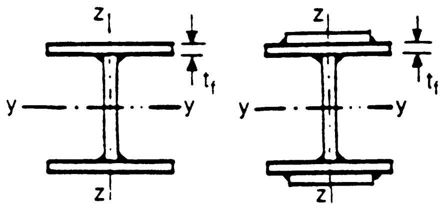
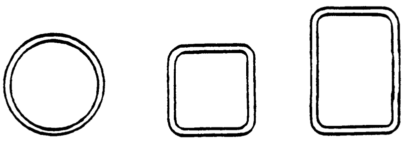
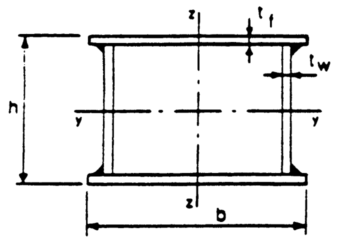
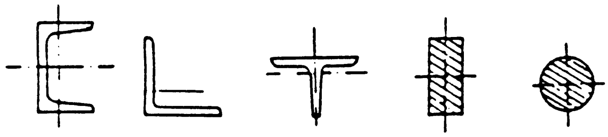
Buckling curve	σ	e_{eff}	k_δ			
			$\gamma_{M1} = 1{,}05$	$\gamma_{M1} = 1{,}10$	$\gamma_{M1} = 1{,}15$	$\gamma_{M1} = 1{,}20$
a	0,21	ℓ/600	0,12	0,23	0,33	0,42
b	0,34	ℓ/380	0,08	0,15	0,22	0,28
c	0,49	ℓ/270	0,06	0,11	0,16	0,20
d	0,76	ℓ/180	0,04	0,08	0,11	0,14

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 below (N/mm²)

t is the material thickness (mm)

A_g is the gross cross-sectional area (mm²)

k 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:

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \bar{\lambda}_{LT}^2]^{0,5}} \quad \text{but } \chi_{LT} \le 1 \qquad (5.49)$$

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/mm²)

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:

$$\frac{N_{Sd}}{\chi_z A f_y/\gamma_{M1}} + \frac{k_{LT} M_{y.Sd}}{\chi_{LT} W_{pl.y} f_y/\gamma_{M1}} + \frac{k_z M_{z.Sd}}{W_{pl.z} f_y/\gamma_{M1}} \leq 1 \tag{5.52}$$

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:

$$\frac{N_{Sd}}{\chi_{min} A f_y/\gamma_{M1}} + \frac{k_y M_{y.Sd}}{W_{el.y} f_y/\gamma_{M1}} + \frac{k_z M_{z.Sd}}{W_{el.z} f_y/\gamma_{M1}} \leq 1 \tag{5.53}$$

where k_y, k_z and χ_{min} are as in (1)

$$\mu_y = \bar{\lambda}_y (2 \beta_{My} - 4) \quad \text{but } \mu_y \leq 0,90$$

and

$$\mu_z = \bar{\lambda}_z (2 \beta_{Mz} - 4) \quad \text{but } \mu_z \leq 0,90$$

(4) Members with Class 3 cross-sections for which lateral-torsional buckling is a potential failure mode shall also satisfy:

$$\frac{N_{Sd}}{\chi_z A f_y/\gamma_{M1}} + \frac{k_{LT} M_{y.Sd}}{\chi_{LT} W_{el.y} f_y/\gamma_{M1}} + \frac{k_z M_{z.Sd}}{W_{el.z} f_y/\gamma_{M1}} \leq 1 \tag{5.54}$$

(5) Members with Class 4 cross-sections subject to combined bending and axial load shall satisfy:

$$\frac{N_{Sd}}{\chi_{min} A_{eff} f_y/\gamma_{M1}} + \frac{k_y M_{y.Sd} + N_{Sd} e_{N.y}}{W_{eff.y} f_y/\gamma_{M1}} + \frac{k_z M_{z.Sd} + N_{Sd} e_{N.z}}{W_{eff.z} f_y/\gamma_{M1}} \leq 1 \tag{5.56}$$

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:

$$\frac{N_{Sd}}{\chi_z A_{eff} f_y/\gamma_{M1}} + \frac{k_{LT} M_{y.Sd} + N_{Sd} e_{N.y}}{\chi_{LT} W_{eff.y} f_y/\gamma_{M1}} + \frac{k_z M_{z.Sd} + N_{Sd} e_{N.z}}{W_{eff.z} f_y/\gamma_{M1}} \leq 1 \tag{5.57}$$

(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

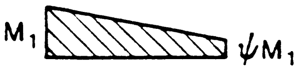
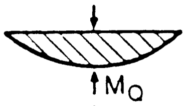
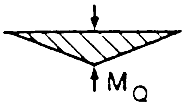
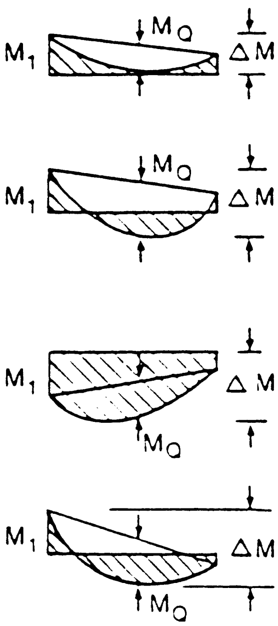
Moment diagram	Equivalent uniform moment factor β_M
end moments  $-1 \leq \psi \leq 1$	$\beta_{M,\psi} = 1{,}8 - 0{,}7\psi$
moments due to in-plane lateral loads  	$\beta_{M,Q} = 1{,}3$ $\beta_{M,Q} = 1{,}4$
moments due to in-plane lateral loads plus end moments 	$\beta_M = \beta_{M,\psi} + \frac{M_Q}{\Delta M}(\beta_{M,Q} - \beta_{M,\psi})$ $M_Q = \lvert \max M \rvert$ due to lateral load only $\Delta M \begin{cases} \lvert \max M \rvert & \text{for moment diagram without change of sign} \\ \lvert \max M \rvert + \lvert \min M \rvert & \text{for moment diagram with change of sign} \end{cases}$

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* $\bar{\lambda}_w \leq 0{,}8$:

$$\tau_{ba} = (f_{yw}/\sqrt{3})$$

b) *when* $0{,}8 < \bar{\lambda}_w < 1{,}2$:

$$\tau_{ba} = [1 - 0{,}625\,(\bar{\lambda}_w - 0{,}8)]\,(f_{yw}/\sqrt{3})$$

c) *when* $\bar{\lambda}_w \geq 1{,}2$:

$$\tau_{ba} = [0{,}9/\bar{\lambda}_w]\,(f_{yw}/\sqrt{3})$$

in which $\bar{\lambda}_w$ *is the web slenderness given by:*

$$\bar{\lambda}_w = [(f_{yw}/\sqrt{3})/\tau_{cr}]^{0,5} = \frac{d/t_w}{37{,}4\,\varepsilon\sqrt{k_\tau}}$$

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:*

$$V_{bb.Rd} = [(d\, t_w\, \tau_{bb}) + 0{,}9\,(g\, t_w\, \sigma_{bb} \sin \phi)]/\gamma_{M1} \tag{5.59}$$

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:*

$$M_{Nf.Rk} = 0{,}25\, b t_f^{\,2} f_{yf}\, [1 - [N_{f.Sd}/(b t_f\, f_{yf}/\gamma_{M0})]^2] \tag{5.60}$$

where b and t_f are the width and thickness of the relevant flange.

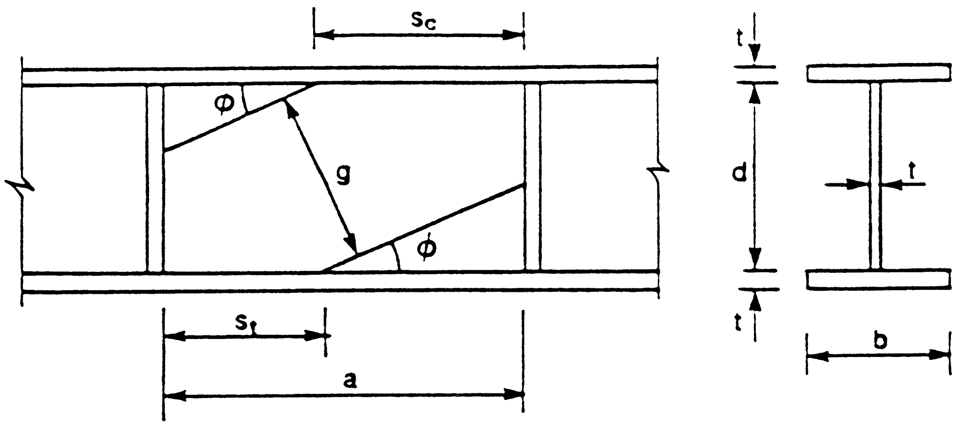


Figure 5.6.1 — Tension field geometry

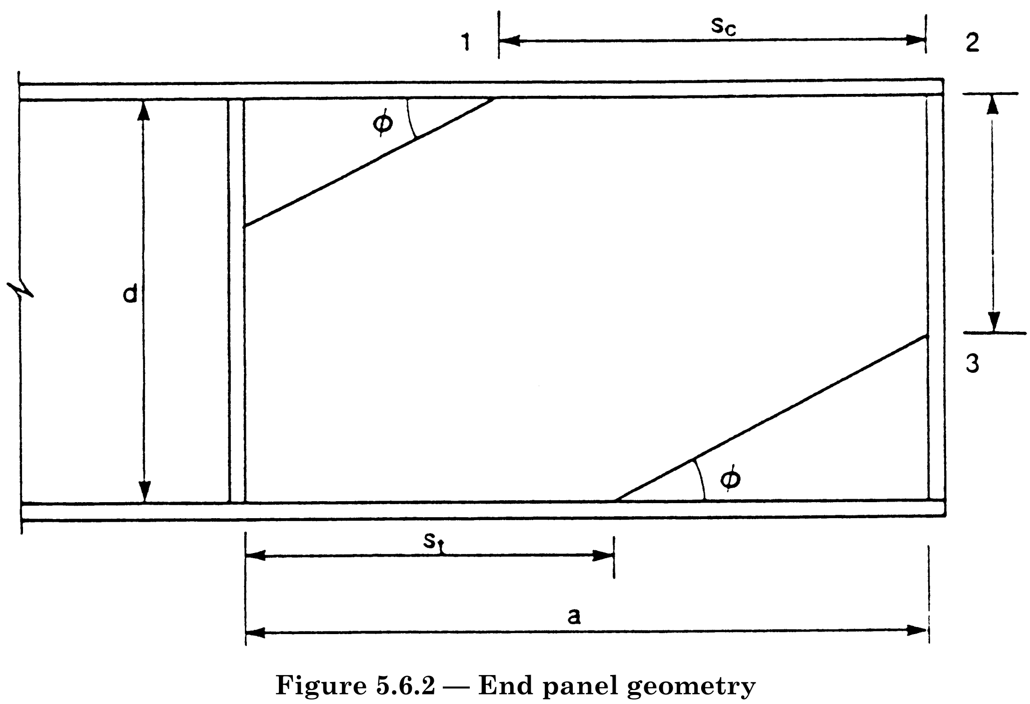


Figure 5.6.2 — End panel geometry

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:*

$$s_c = \frac{2}{\sin\phi}\left[\frac{M_{p\ell.1} + M_{p\ell.2}}{2t_w\,\sigma_{bb}}\right]^{0,5} \quad \text{but } s_c \leq a \qquad (5.61)$$

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:*

$$M_{p\ell.2} + M_{p\ell.3} \geq 0,5F_{bb}s_s \qquad (5.62)$$

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:*

$$N_s = V_{Sd} - d\,t_w\,\tau_{bb}/\gamma_{M1} \quad \text{but } N_s \geq 0 \qquad (5.63)$$

*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 $a/d < \sqrt{2}$:

$$I_s \geq 1,5d^3t_w^3/a^2 \qquad (5.64)$$

if $a/d \geq \sqrt{2}$:

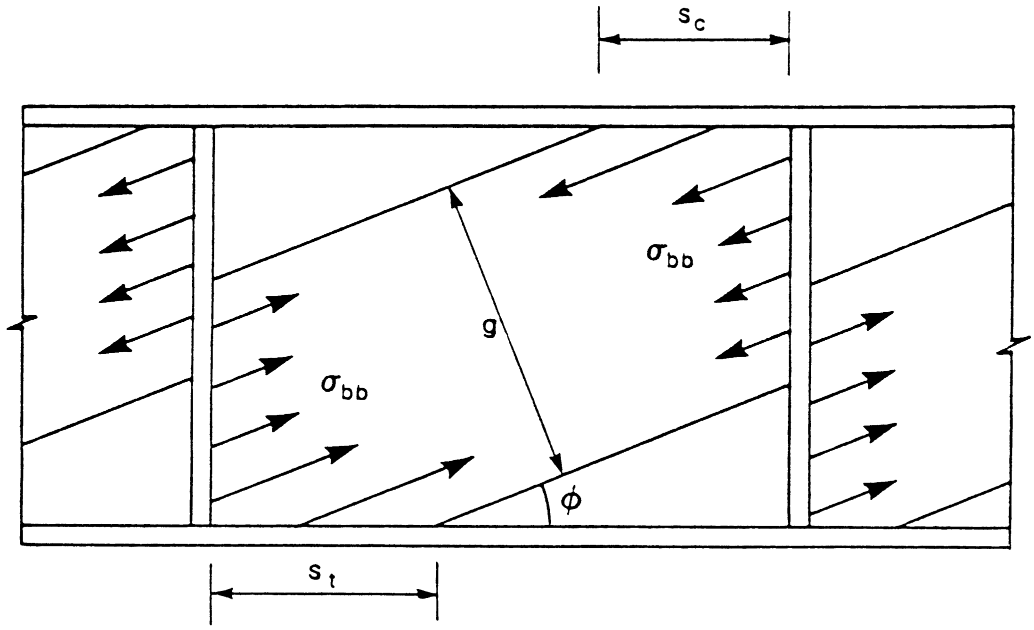
$$I_s \geq 0,75dt_w^3 \qquad (5.65)$$

5.6.6 Welds

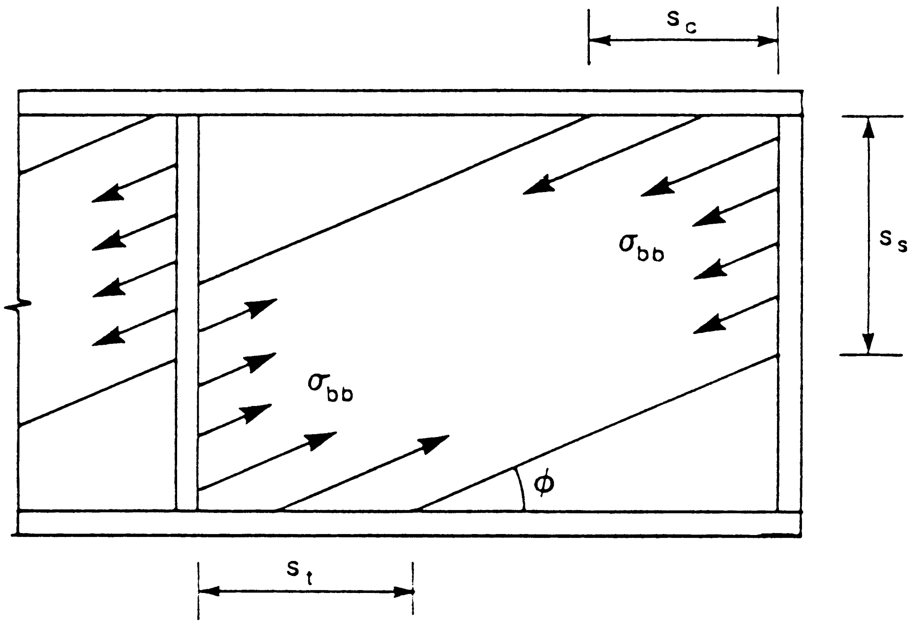
(1) *The forces used to check the web-to-flange welds shall be compatible with the stress fields in the web panels according to the method used to determine the shear buckling resistance.*

(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.*

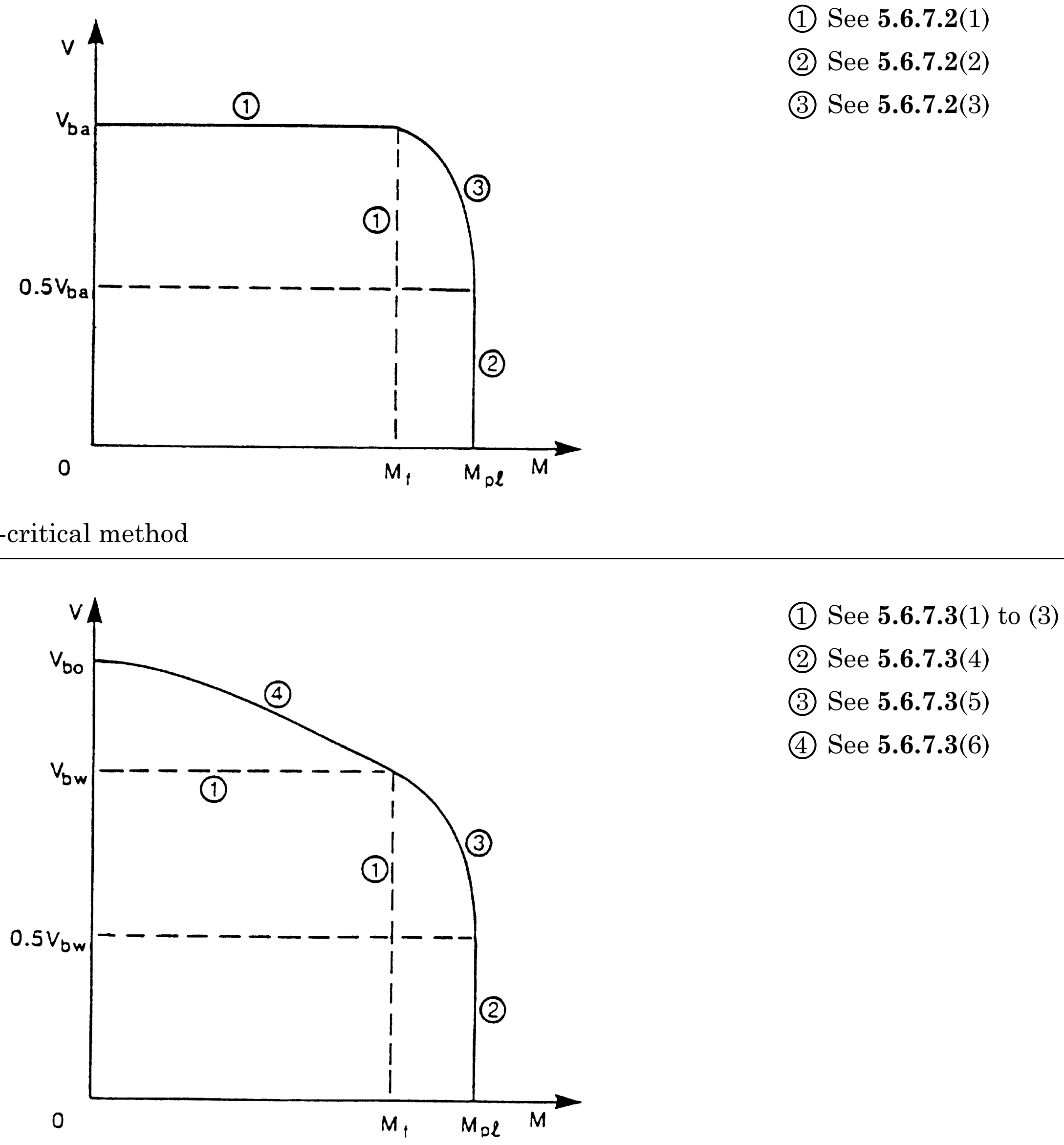
(3) *When* V_{Sd} *exceeds 50 % of* $V_{ba.Rd}$ *the following criterion should be satisfied:*

$$M_{Sd} \leq M_{f.Rd} + (M_{p\ell.Rd} - M_{f.Rd})\,[1 - (2\,V_{Sd}/V_{ba.Rd} - 1)^2] \tag{5.67}$$

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:*

$$M_{Sd} \leq M_{f.Rd} + (M_{p\ell.Rd} - M_{f.Rd})\,[1 - (2V_{Sd}/V_{bw.Rd} - 1)^2] \qquad (5.69)$$

*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**.*

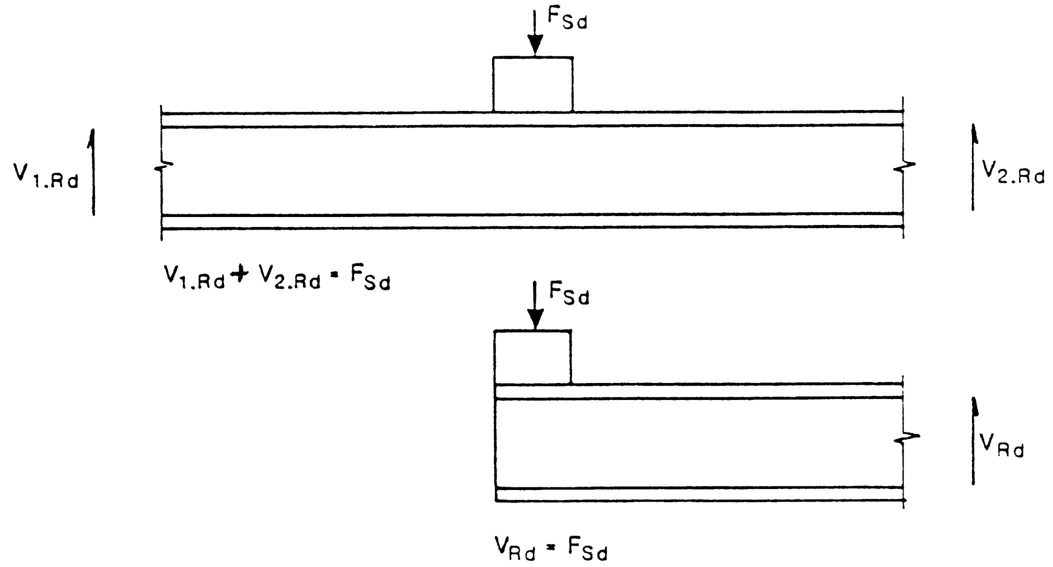
(7) *The crippling resistance of a stiffened web between the locations of transverse web stiffeners, is basically similar to that of an unstiffened web, with some increase due to the presence of the stiffeners.*

5.7.2 Length of stiff bearing

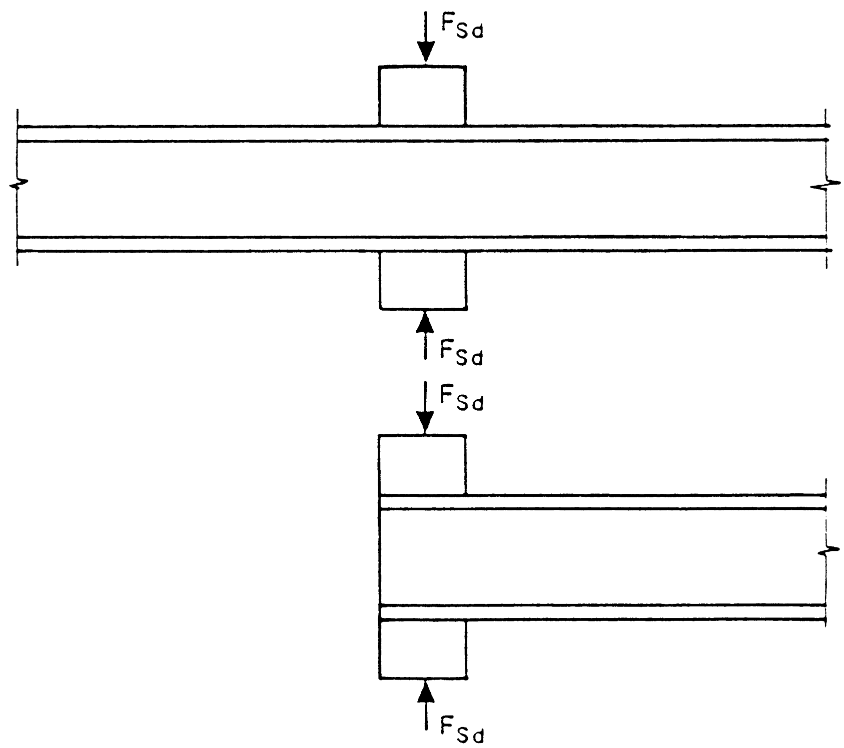
(1) The length of stiff bearing on the flange is the distance over which the applied force is effectively distributed.

(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

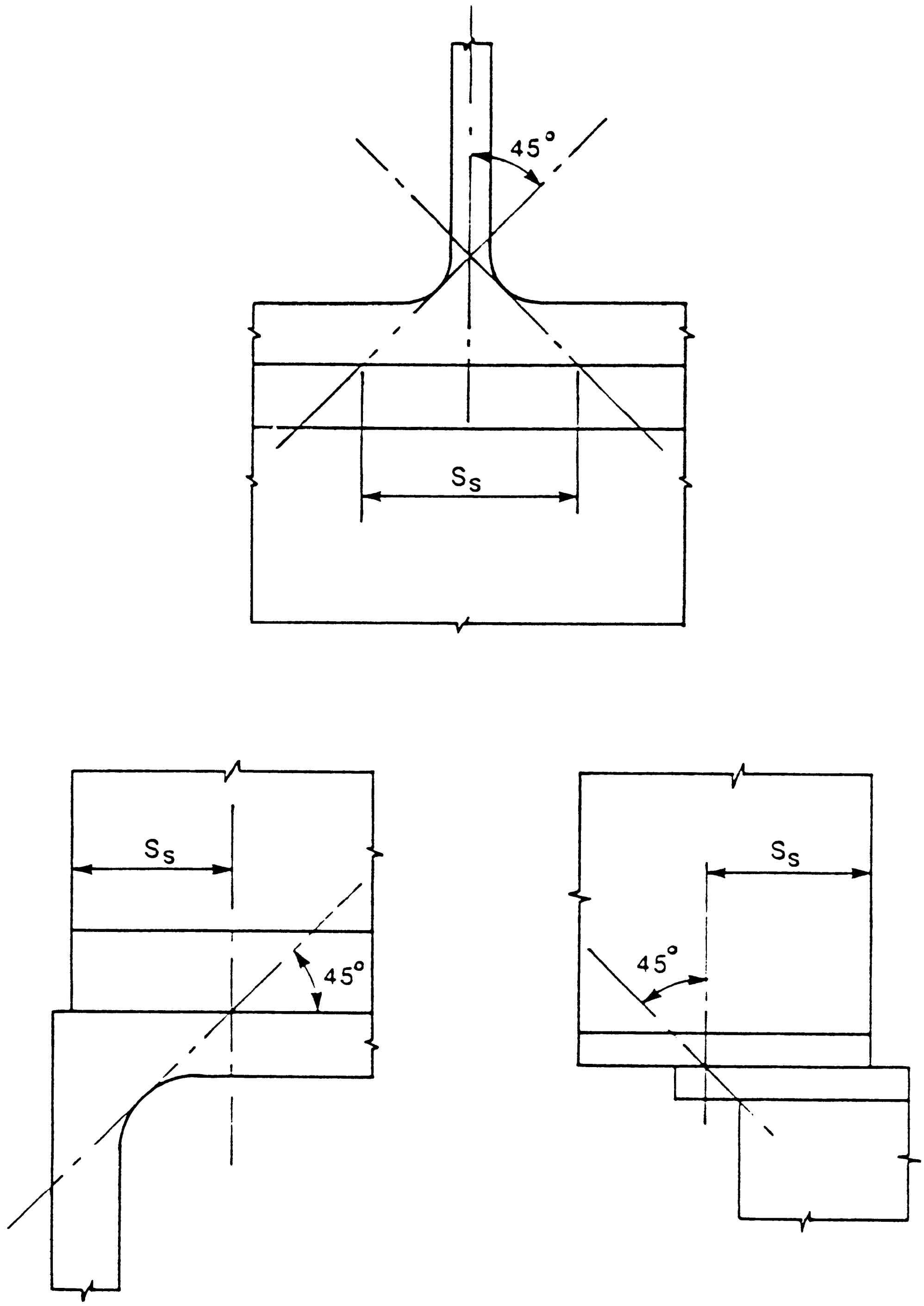


Figure 5.7.2 — Length of stiff bearing

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:*

$$R_{y.Rd} = (s_s + s_y)\, t_w\, f_{yw}/\gamma_{M1} \qquad (5.71)$$

in which s_y is given by:

$$s_y = 2t_f\, (b_f/t_w)^{0,5}\, [f_{yf}/f_{yw}]^{0,5}\, [1 - (\sigma_{f.Ed}/f_{yf})^2]^{0,5} \qquad (5.72)$$

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:*

$$s_y = \frac{2,5\ (h - d)\ [1 - (\sigma_{f.Ed}/f_{yf})^2\ ]^{0,5}}{(1 + 0,8\ s_s/(h - d))} \qquad (5.73)$$

(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:*

$$R_{y.Rd} = s_y\, t_w\, f_{yw}/\gamma_{M1} \qquad (5.74)$$

in which:

$$s_y = k_R \left[\frac{I_f + I_R}{t_w}\right]^{1/3} [1 - (\sigma_{f.Ed}/f_{yf})^2\ ]^{0,5} \qquad (5.75)$$

or more approximately:

$$s_y = 2(h_R + t_f)\ [1 - (\sigma_{f.Ed}/f_{yf})^2]^{0,5} \qquad (5.76)$$

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:*

$$R_{a.Rd} = 0,5t_w^2(Ef_{yw})^{0,5}\ [(t_f/t_w)^{0,5} + 3(t_w/t_f)(s_s/d)]/\gamma_{M1} \qquad (5.77)$$

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:*

$$F_{Sd} \leq R_{a.Rd} \qquad (5.78a)$$

$$M_{Sd} \leq M_{c.Rd} \qquad (5.78b)$$

and

$$\frac{F_{Sd}}{R_{a.Rd}} + \frac{M_{Sd}}{M_{c.Rd}} \leq 1,5 \qquad (5.78c)$$

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:*

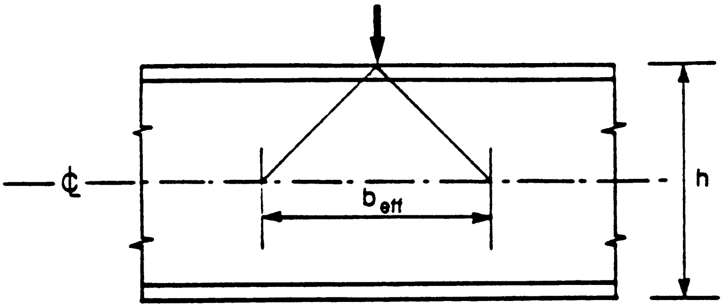
$$b_{eff} = [h^2 + s_s^2]^{0,5} \qquad (5.79)$$

(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.*

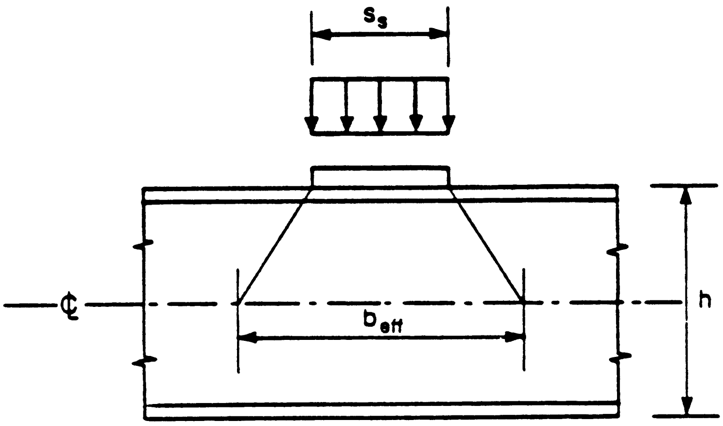
(3) *The buckling resistance should be determined from* ***5.5.1*** *using buckling curve c and $\beta_A = 1$.*

(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.*

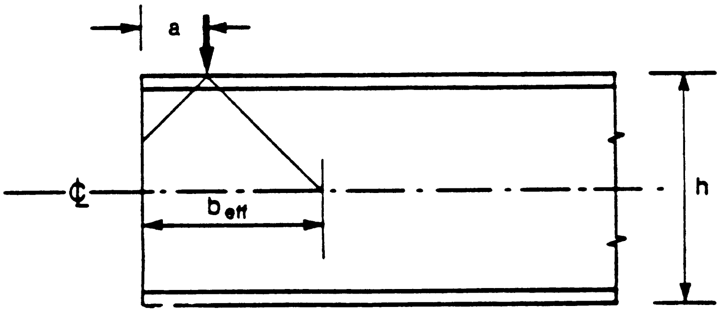
(5) *The flange through which the load is applied should normally be restrained in position at the point of load application. Where this is not practicable, a special buckling investigation should be carried out.*



$b_{eff} = h$

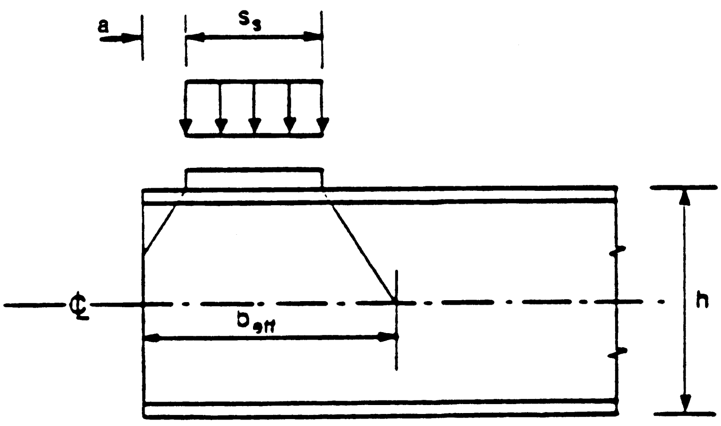


$$b_{eff} = \left[h^2 + s_s^2\right]^{1/2}$$



$$b_{eff} = \frac{h}{2} + a$$

but $b_{eff} \leq h$



$$b_{eff} = \frac{1}{2}\left[h^2 + s_s^2\right]^{1/2} + a + \frac{s_s}{2}$$

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 $30\varepsilon t_w$, arranged with $15\varepsilon t_w$ each side of the stiffener, see Figure 5.7.4. At the ends of the member (or openings in the web) the dimension of $15\varepsilon t_w$ 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.*

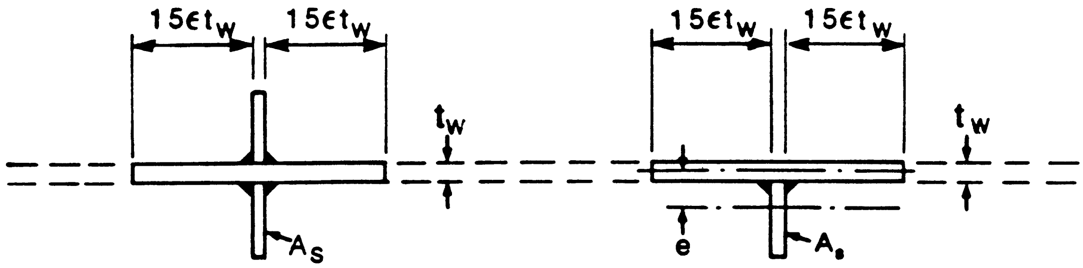


Figure 5.7.4 — Effective cross-section of stiffeners

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:*

$$d/t_w \leq \frac{k(E/f_{yf})\ [A_w/A_{fc}]^{0,5}}{[1 + dE/(3rf_{yf})]^{0,5}} \tag{5.81}$$

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: $$\bar{\lambda}_{eff.v} = 0{,}35 + 0{,}7\bar{\lambda}_v \qquad (5.82)$$

for buckling about the y-y axis: $$\bar{\lambda}_{eff.y} = 0{,}50 + 0{,}7\bar{\lambda}_y \qquad (5.83)$$

for buckling about the z-z axis: $$\bar{\lambda}_{eff.z} = 0{,}50 + 0{,}7\bar{\lambda}_z \qquad (5.84)$$

*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 I_{eff} of a laced compression member with two main components should be taken as:*

$$I_{eff} = 0{,}5\, h_o^2\, A_f \qquad (5.85)$$

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:*

$$N_{f.Sd} = 0{,}5\, N_{Sd} + M_s/h_o \qquad (5.86)$$

where $M_s = N_{Sd}e_o/(1 - N_{Sd}/N_{cr} - N_{Sd}/S_v)$

$e_o = \ell/500$ *(see **5.9.1**)*

$N_{cr} = \pi^2 E I_{eff}/\ell^2$

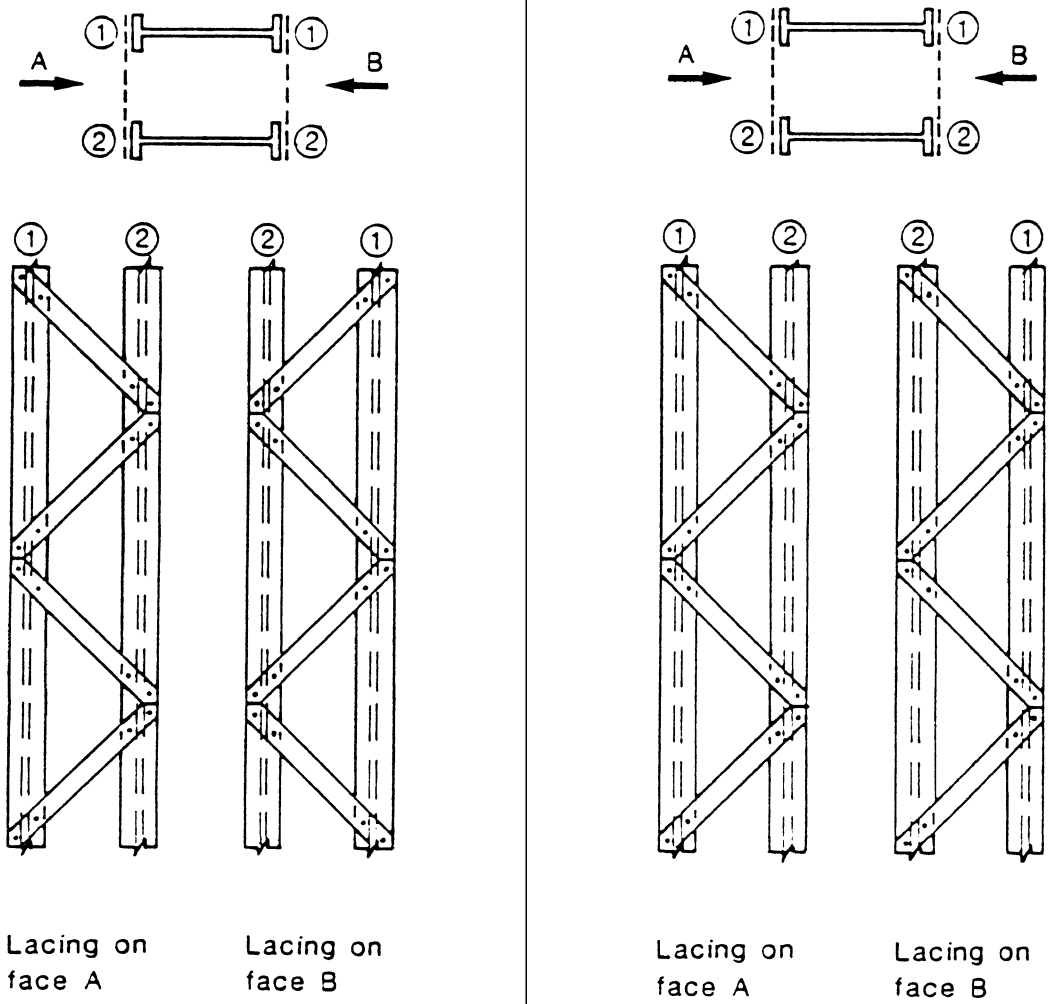
and S_v *is the shear stiffness of the lacings (the shear force required to produce unit shear deformation).*

(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.*

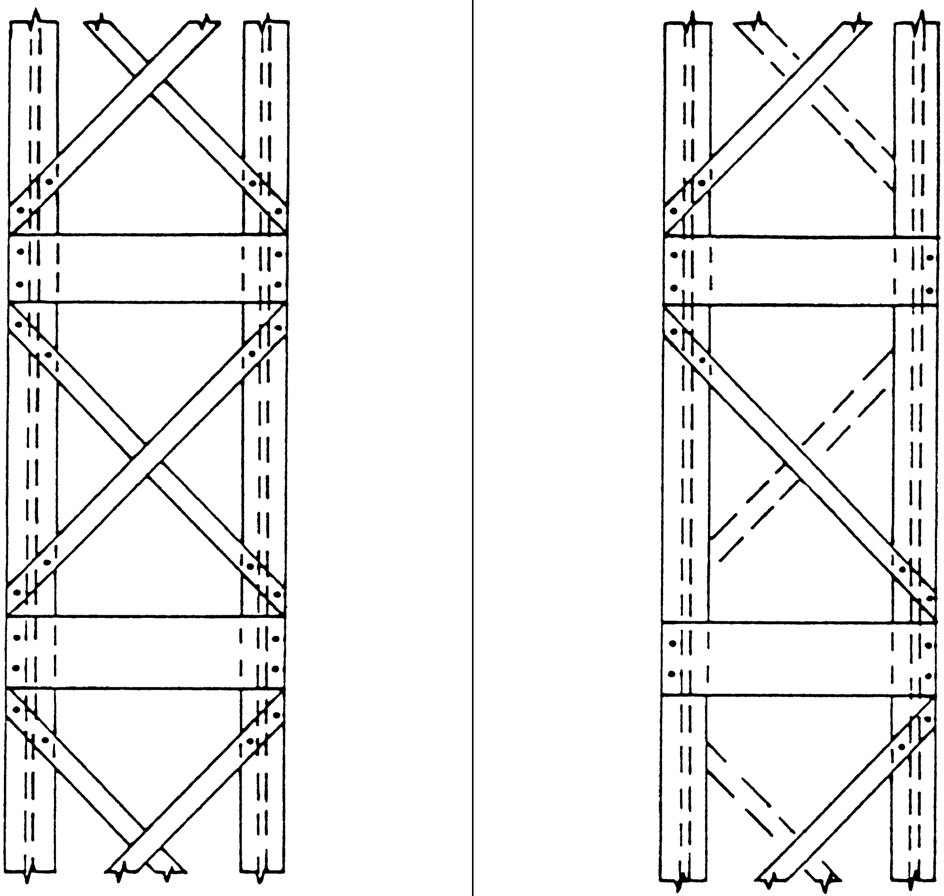
(2) *In a member with four chords made of equal-leg angles with lacing in both directions, the buckling length ℓ for buckling about the weakest axis depends on the arrangement of the lacings, see Figure 5.9.4.*



(a) Corresponding lacing systems
(Recommended system)

(b) Mutually opposed lacing systems
(Not recommended)

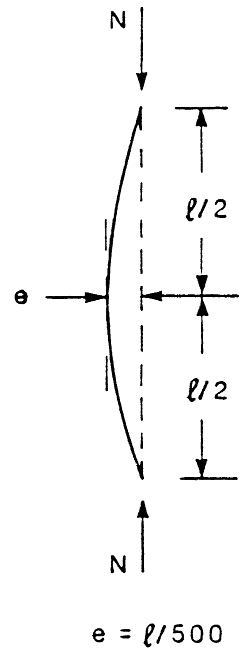
Figure 5.9.1 — Single lacing systems on opposite sides of main components

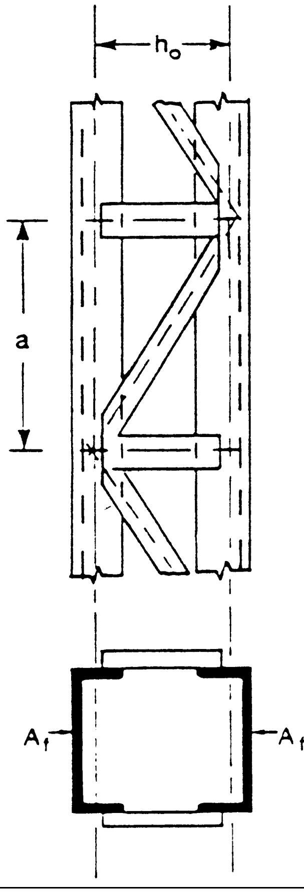


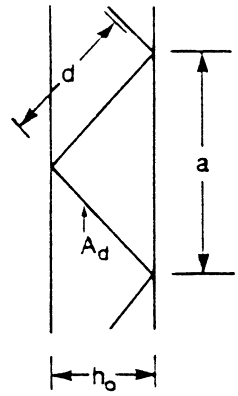
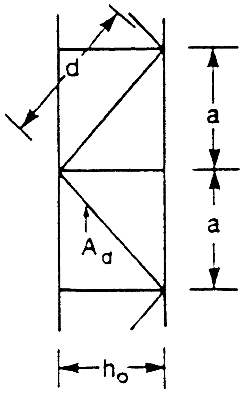
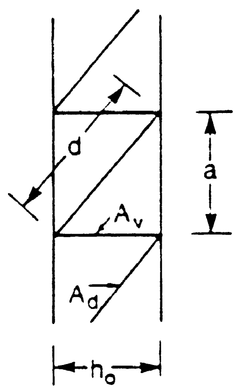
(a) Double intersection
lacing system

(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

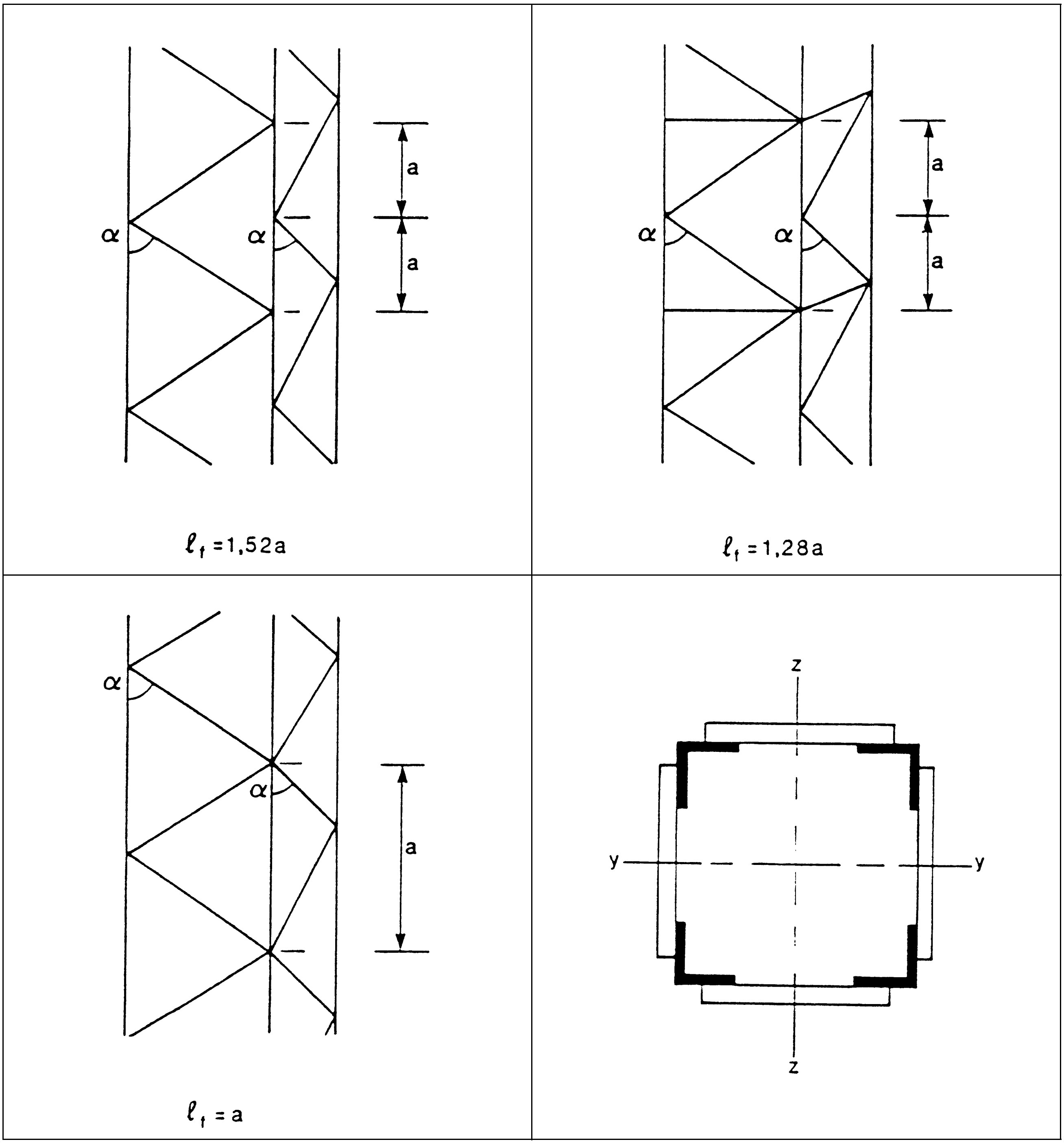


Figure 5.9.4 — Buckling lengths of angle chords in laced members

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:*

$$l_{eff} = 0{,}5h_o^2A_f + 2\mu l_f \qquad (5.90)$$

with μ *obtained from the following:*

$\lambda \leq 75$:	$\mu = 1$
$75 < \lambda < 150$:	$\mu = 2 - \lambda/75$
$\lambda \geq 150$:	$\mu = 0$

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:*

$$N_{f.Sd} = 0{,}5(N_{Sd} + M_s h_o A_f / l_{eff}) \qquad (5.91)$$

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:*

$$S_v = 2\pi^2 E l_{eff}/a^2 \qquad (5.92)$$

(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:*

$$S_v = \frac{24EI_f}{a^2\left[1 + \dfrac{2I_f}{nI_b} \cdot \dfrac{h_o}{a}\right]} \quad \text{but } S_v \leq \frac{2\pi^2 EI_f}{a^2} \qquad (5.93)$$

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:*

$$V_s = \pi M_s/\ell \qquad (5.94)$$

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* $0{,}25V_s\, a/i_{min}$ *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.*

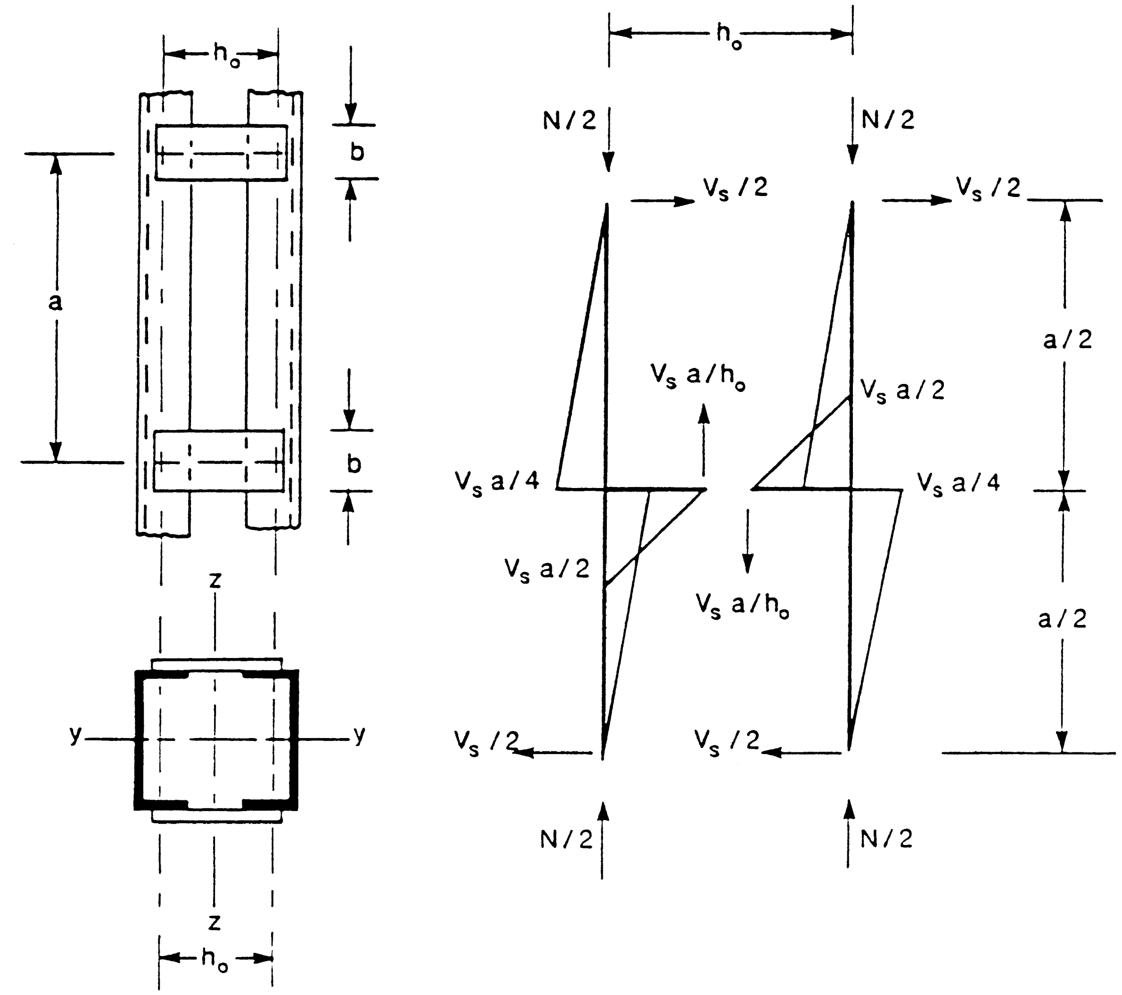


Figure 5.9.5 — Battened compression member

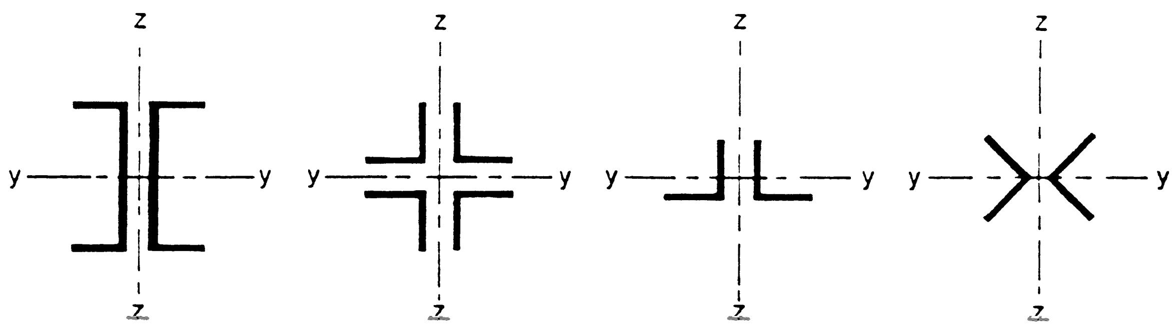


Figure 5.9.6 — Closely spaced built-up members

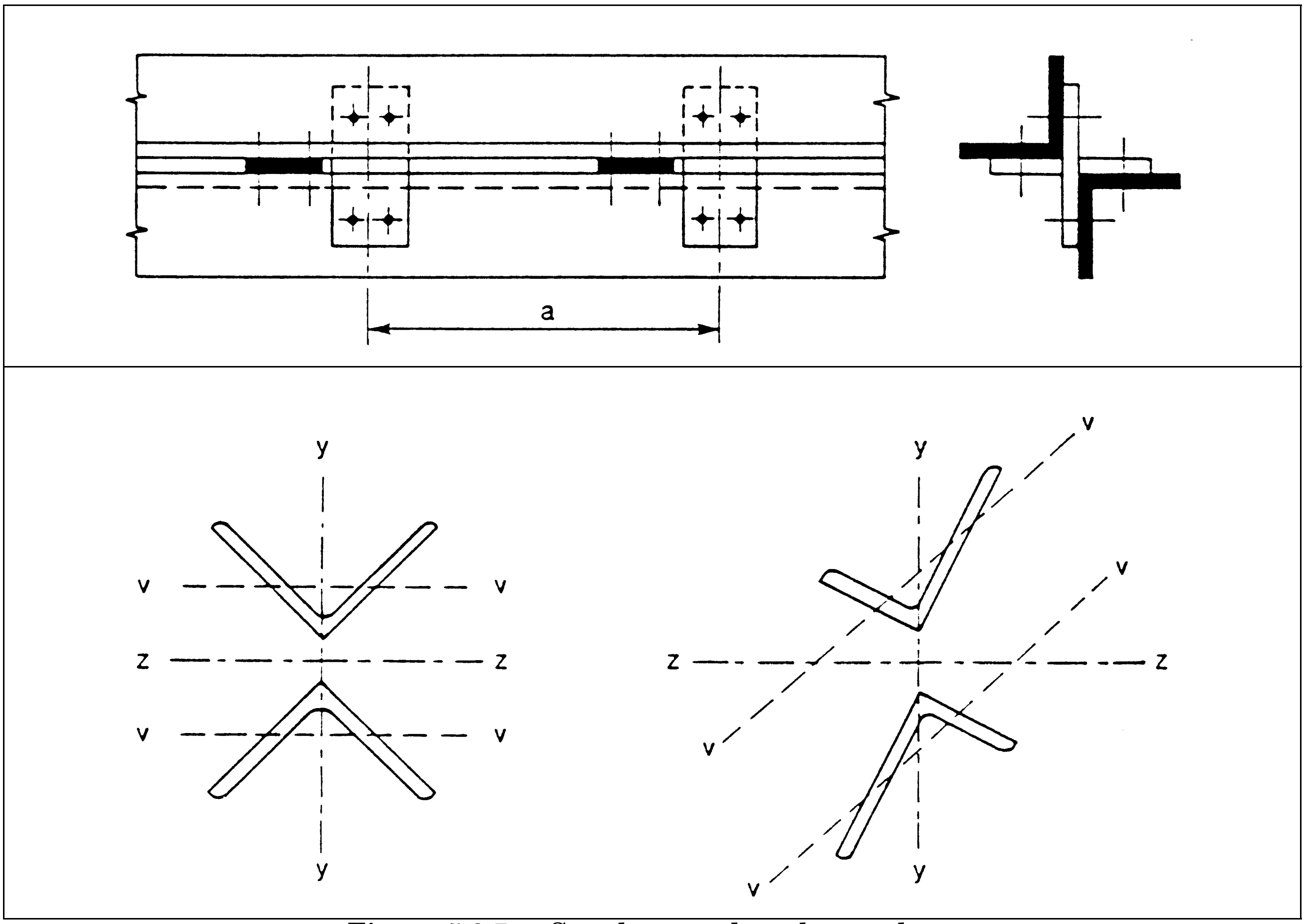


Figure 5.9.7 — Star-battened angle members

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 $1,5d_o$.*

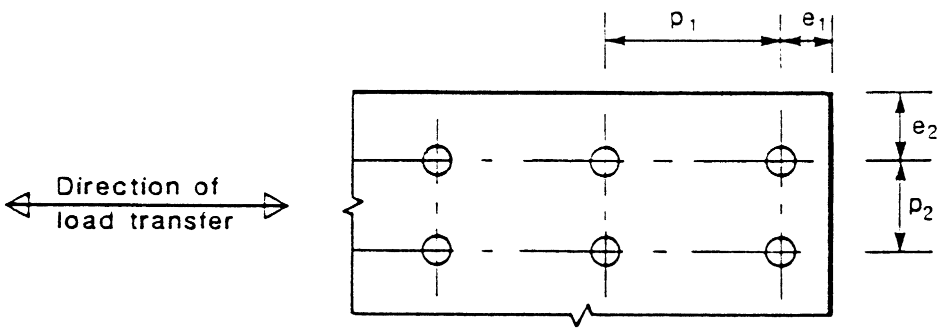


Figure 6.5.1 — Symbols for spacing of fasteners

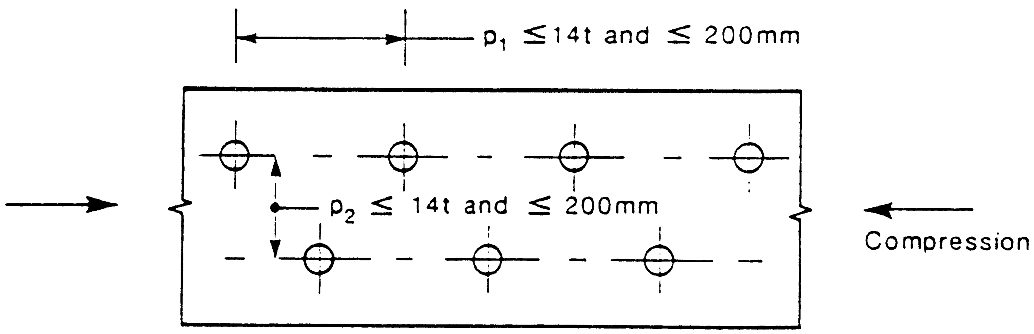


Figure 6.5.2 — Staggered spacing — compression

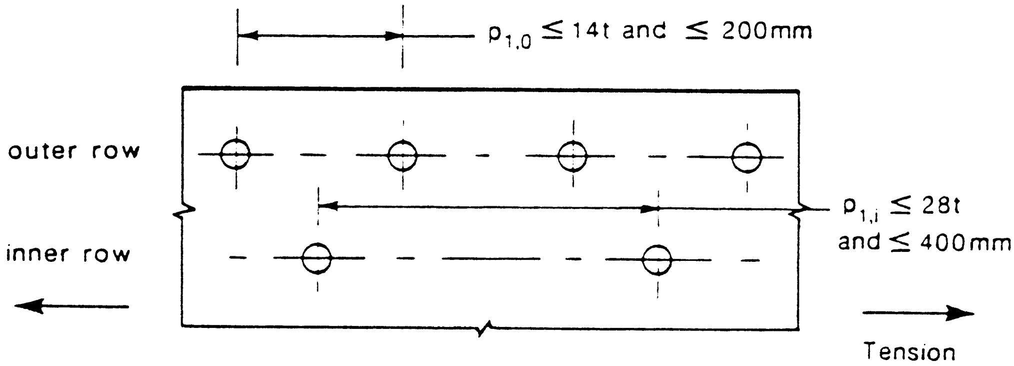


Figure 6.5.3 — Spacing in tension members

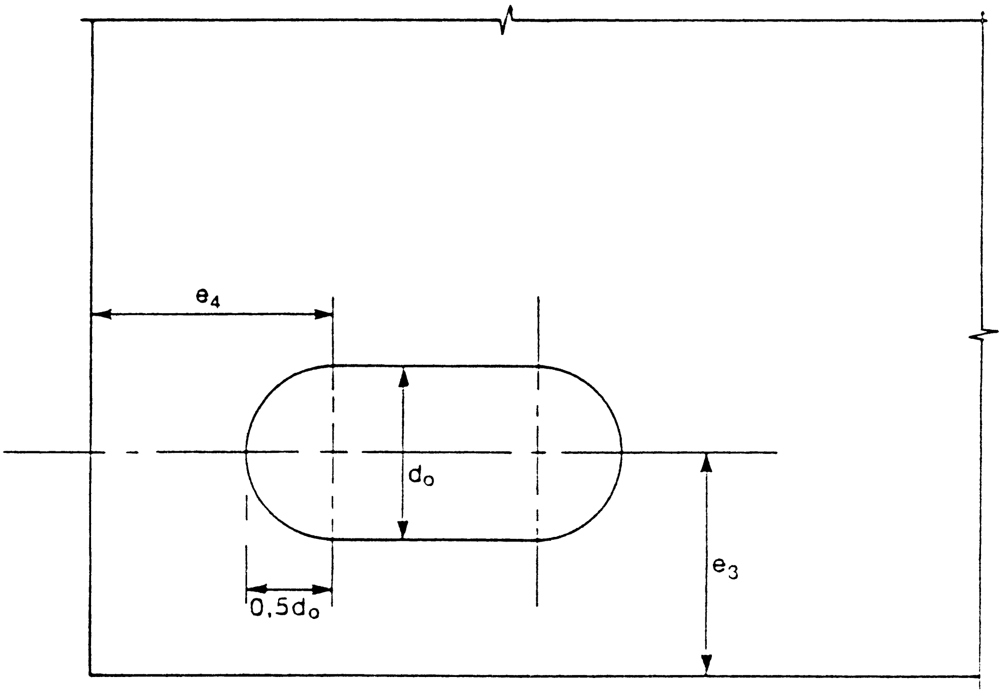


Figure 6.5.4 — End and edge distances for slotted holes

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* $L_{v.eff} \leq L_3$

in which $L_1 = a_1$ *but* $L_1 \leq 5\, d$

$L_2 = (a_2 - k\, d_{o.t})\, (f_u/f_y)$

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:* $k = 2{,}5$

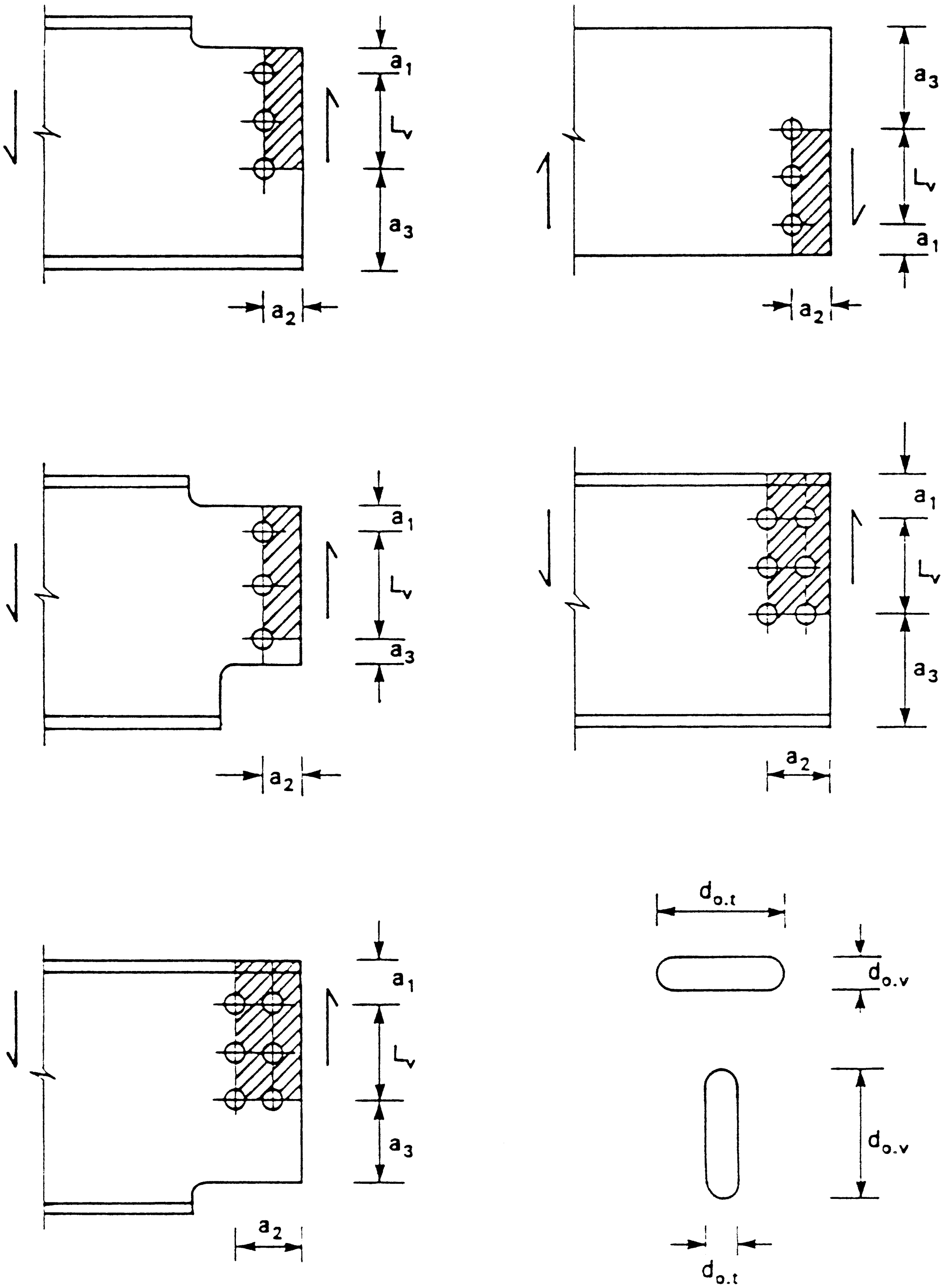


Figure 6.5.5 — Block shear — effective shear area

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:

$$N_{u.Rd} = \frac{2{,}0\ (e_2 - 0{,}5d_o)\ t\ f_u}{\gamma_{M2}} \qquad (6.2)$$

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

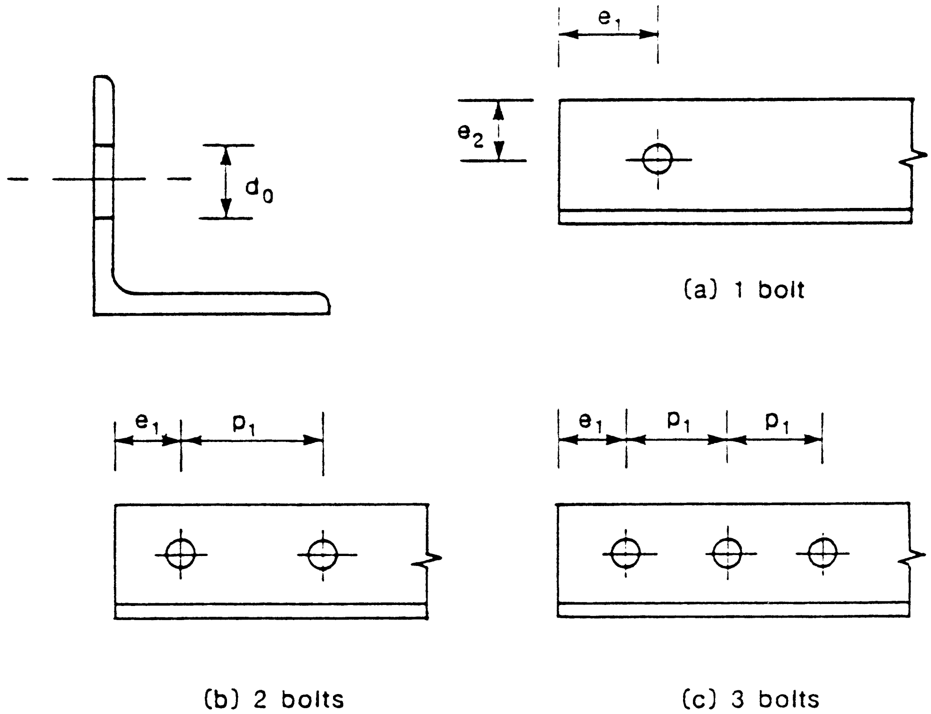


Figure 6.5.6 — Connections of angles

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:

$$N_{net.Rd} = A_{net}\, f_y/\gamma_{M0} \qquad (5.14)$$

Table 6.5.2 — Categories of bolted connections

Shear connections		
Category	**Criteria**	**Remarks**
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.*

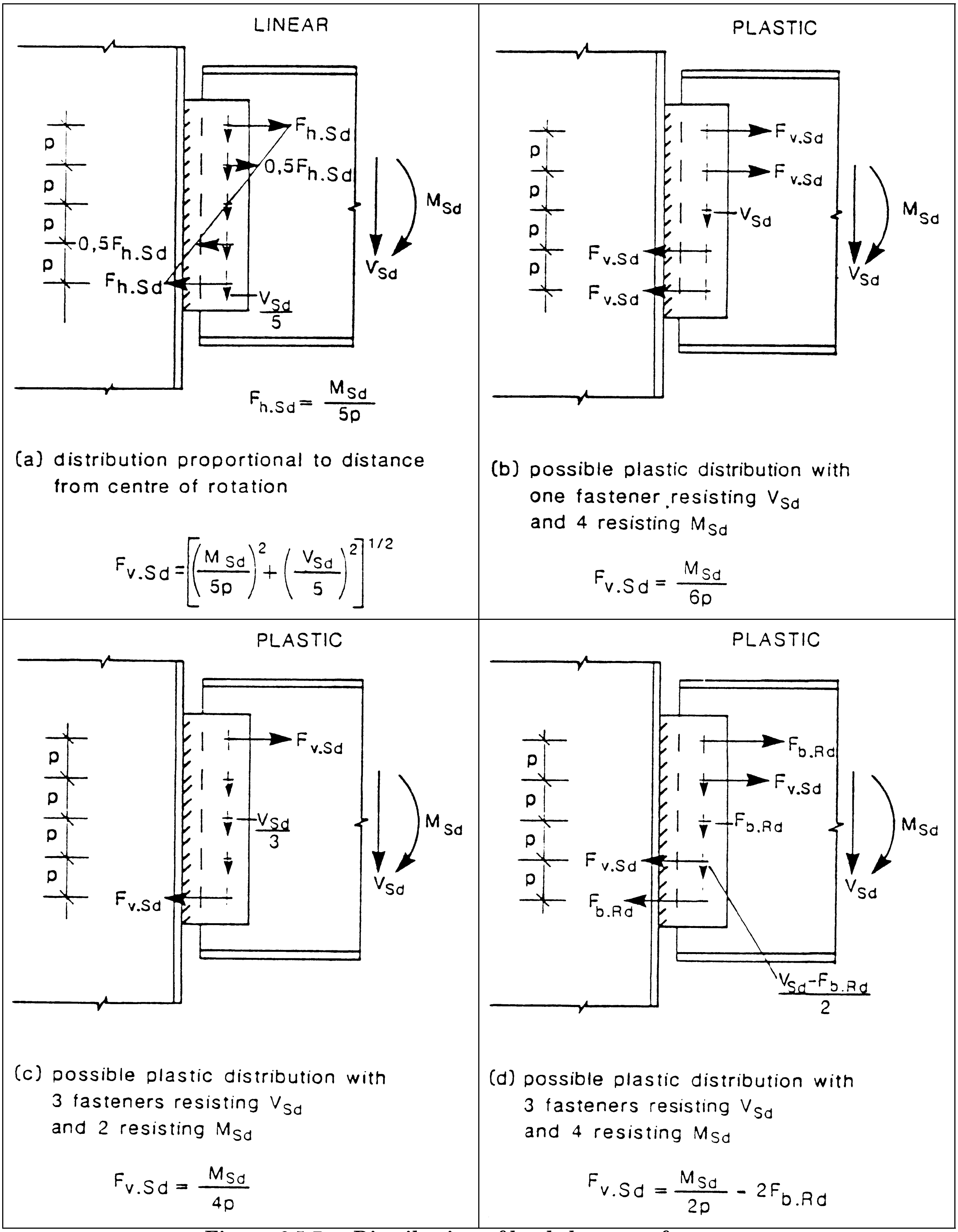


Figure 6.5.7 — Distribution of loads between fasteners

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:

$$B_{p.Rd} = 0{,}6\,\pi\,d_m\,t_p\,f_u/\gamma_{Mb} \tag{6.5}$$

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:

$$\frac{F_{v.Sd}}{F_{v.Rd}} + \frac{F_{t.Sd}}{1{,}4\,F_{t.Rd}} \leq 1{,}0 \tag{6.6}$$

(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 $d_o \leq p_2 \leq 3d_o$ 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: $F_{v.Rd} = \dfrac{0,6\ f_{ur}\ A_o}{\gamma_{Mr}}$
Bearing resistance: $F_{b.Rd} = \dfrac{2,5\ \alpha\ f_u\ d_o t}{\gamma_{Mr}}$ where α is the smallest of: $\dfrac{e_1}{3d_o}$; $\dfrac{p_1}{3d_o} - \dfrac{1}{4}$; $\dfrac{f_{ur}}{f_u}$ or 1,0
Tension resistance: $F_{t.Rd} = \dfrac{0,6\ f_{ur}\ A_o}{\gamma_{Mr}}$
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:

$$F_{s.Rd} = \frac{k_s\, n\, \mu}{\gamma_{Ms}} F_{p.Cd} \tag{6.7}$$

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:

$$F_{p.Cd} = 0,7\, f_{ub}\, A_s \tag{6.8}$$

(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:

$$F_{s.Rd.ser} = \frac{k_s\, n\, \mu\, (F_{p.Cd} - 0{,}8\, F_{t.Sd.ser})}{\gamma_{Ms.ser}} \qquad (6.9)$$

- Category C: Slip-resistant at ultimate:

$$F_{s.Rd} = \frac{k_s\, n\, \mu\, (F_{p.Cd} - 0{,}8\, F_{t.Sd})}{\gamma_{Ms.ult}} \qquad (6.10)$$

(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:

$$\beta_{Lf} = 1 - \frac{L_j - 15\, d}{200\, d} \qquad (6.11)$$

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.

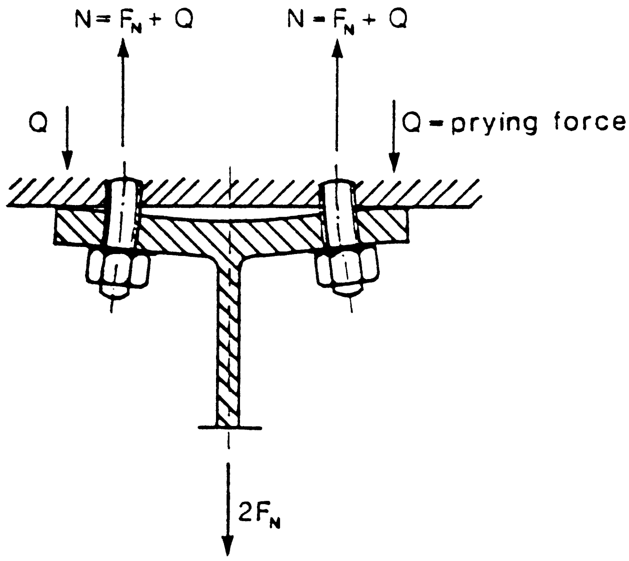


Figure 6.5.8 — Prying forces

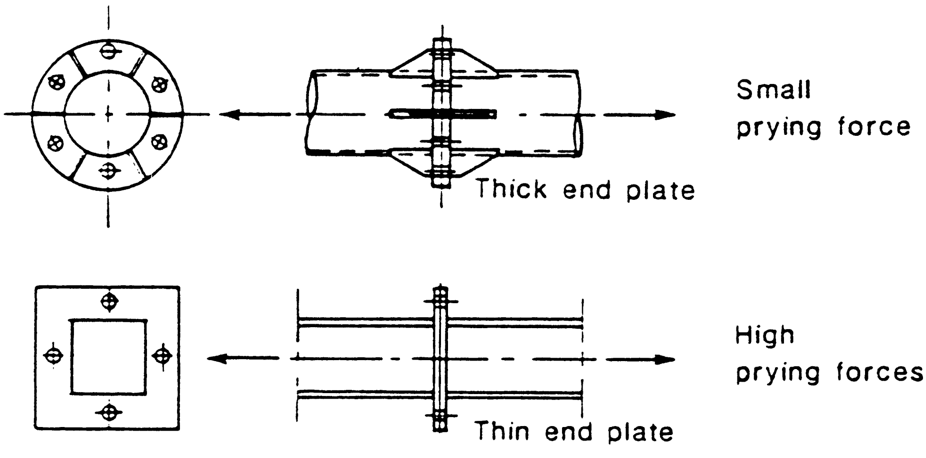


Figure 6.5.9 — Effect of details on prying forces

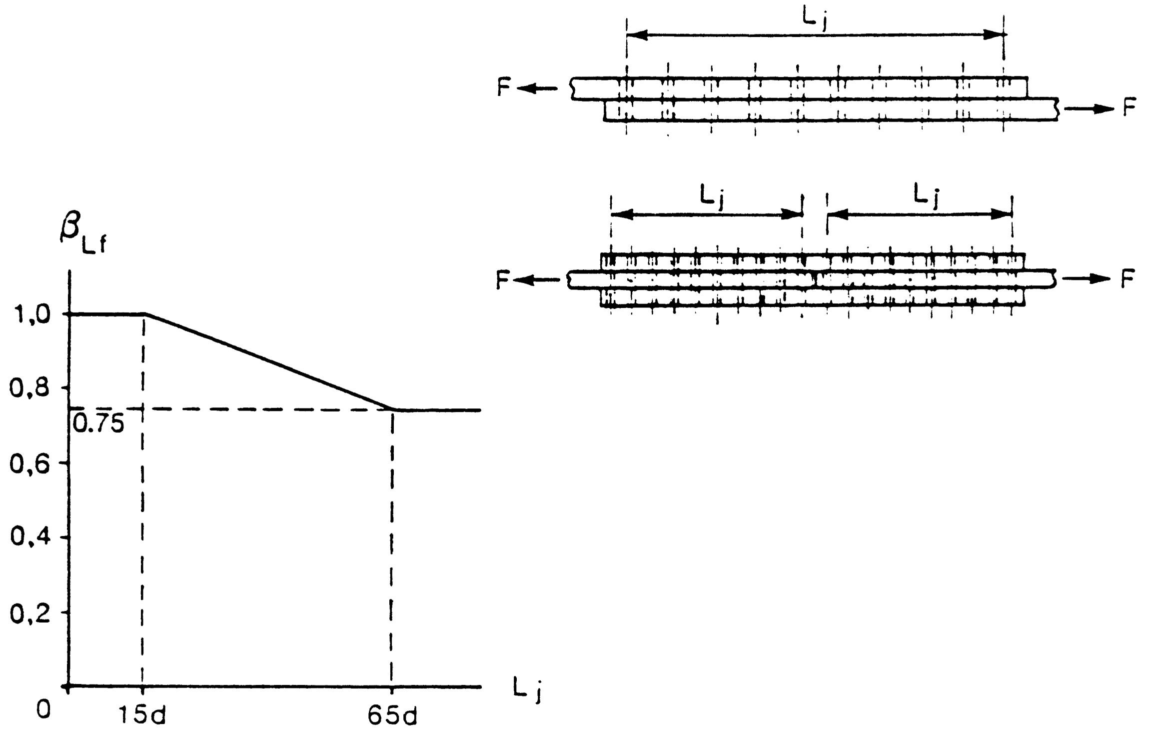


Figure 6.5.10 — Long joints

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.*

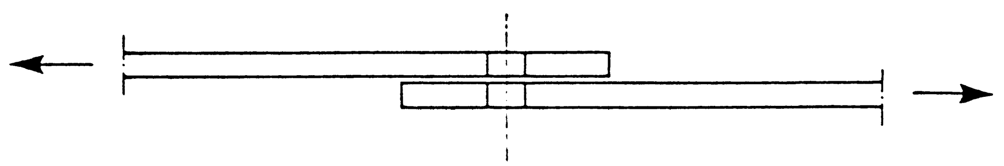


Figure 6.5.11 — Single lap joint with one bolt

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.

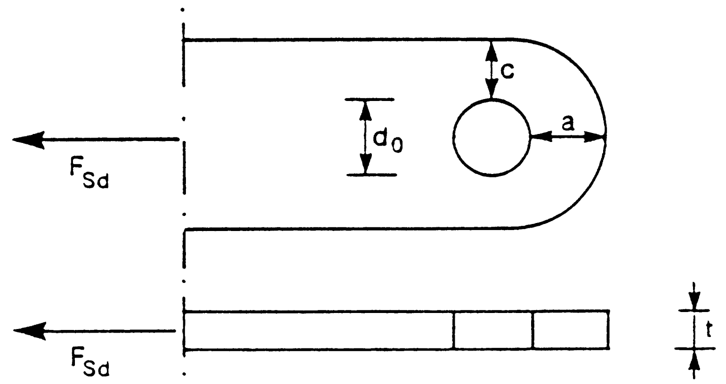
(2) At the ultimate limit state the design forces and moments in a pin shall not exceed the relevant design resistances given in Table 6.5.7.

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	$F_{b.Rd} = 1{,}5\ t\ d\ f_y/\gamma_{Mp}$

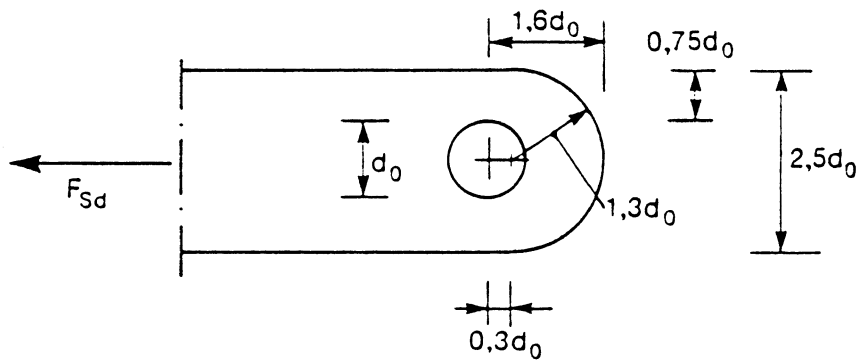
Table 6.5.6 — Geometrical conditions for plates in pin connections

Type A: Given thickness t

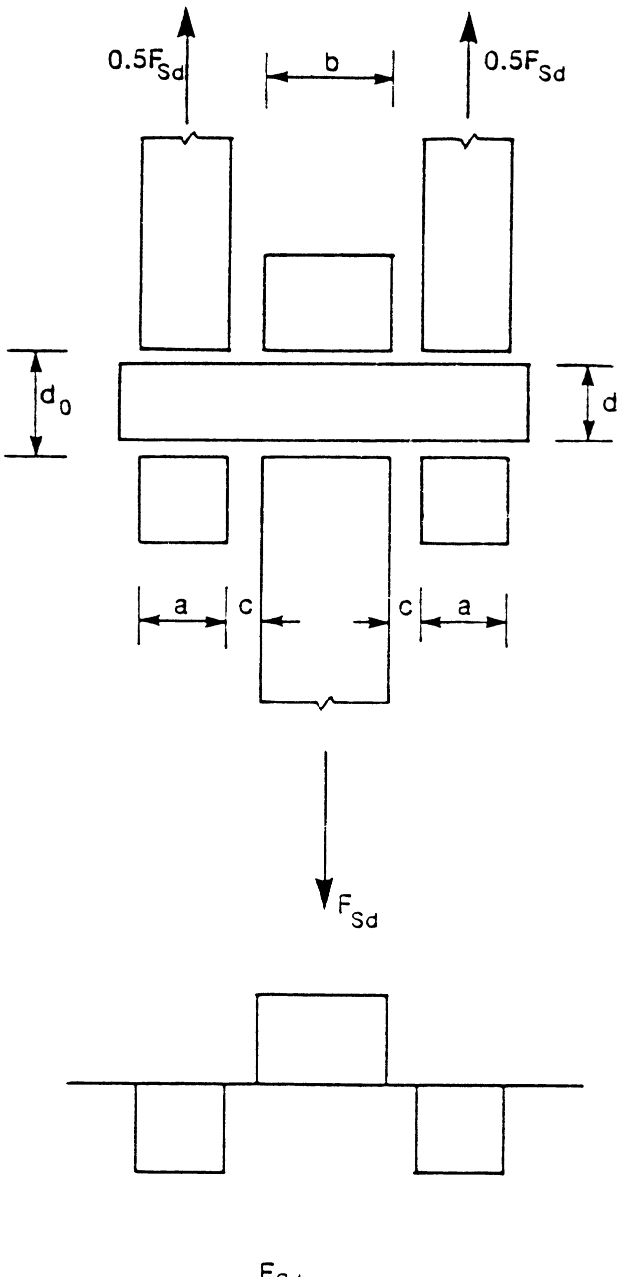


$$a \geq \frac{F_{Sd}\,\gamma_{Mp}}{2tf_y} + \frac{2d_0}{3} \quad : \quad c \geq \frac{F_{Sd}\,\gamma_{Mp}}{2tf_y} + \frac{d_0}{3}$$

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.

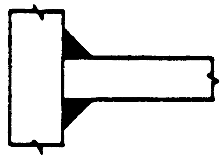
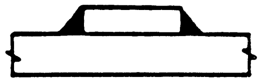
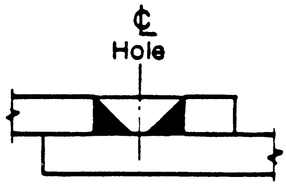
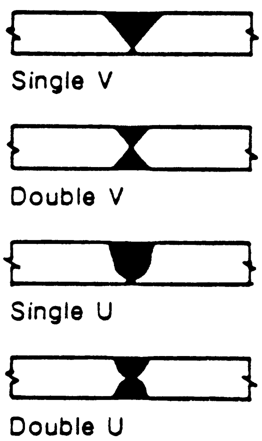
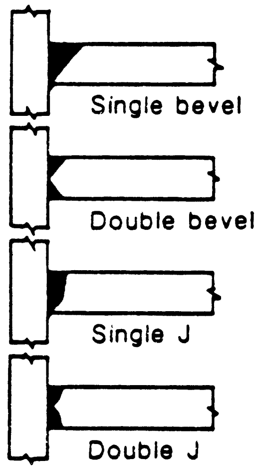
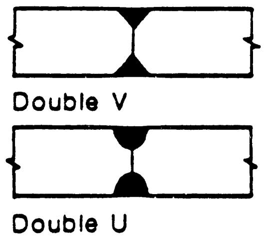
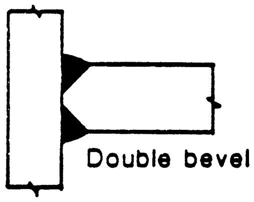
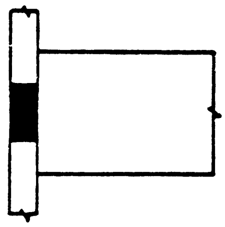
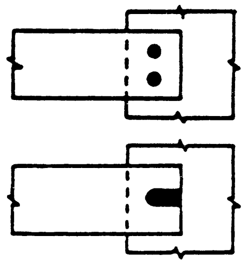
(3) Both slot welds and plug welds may be in either:

- 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.

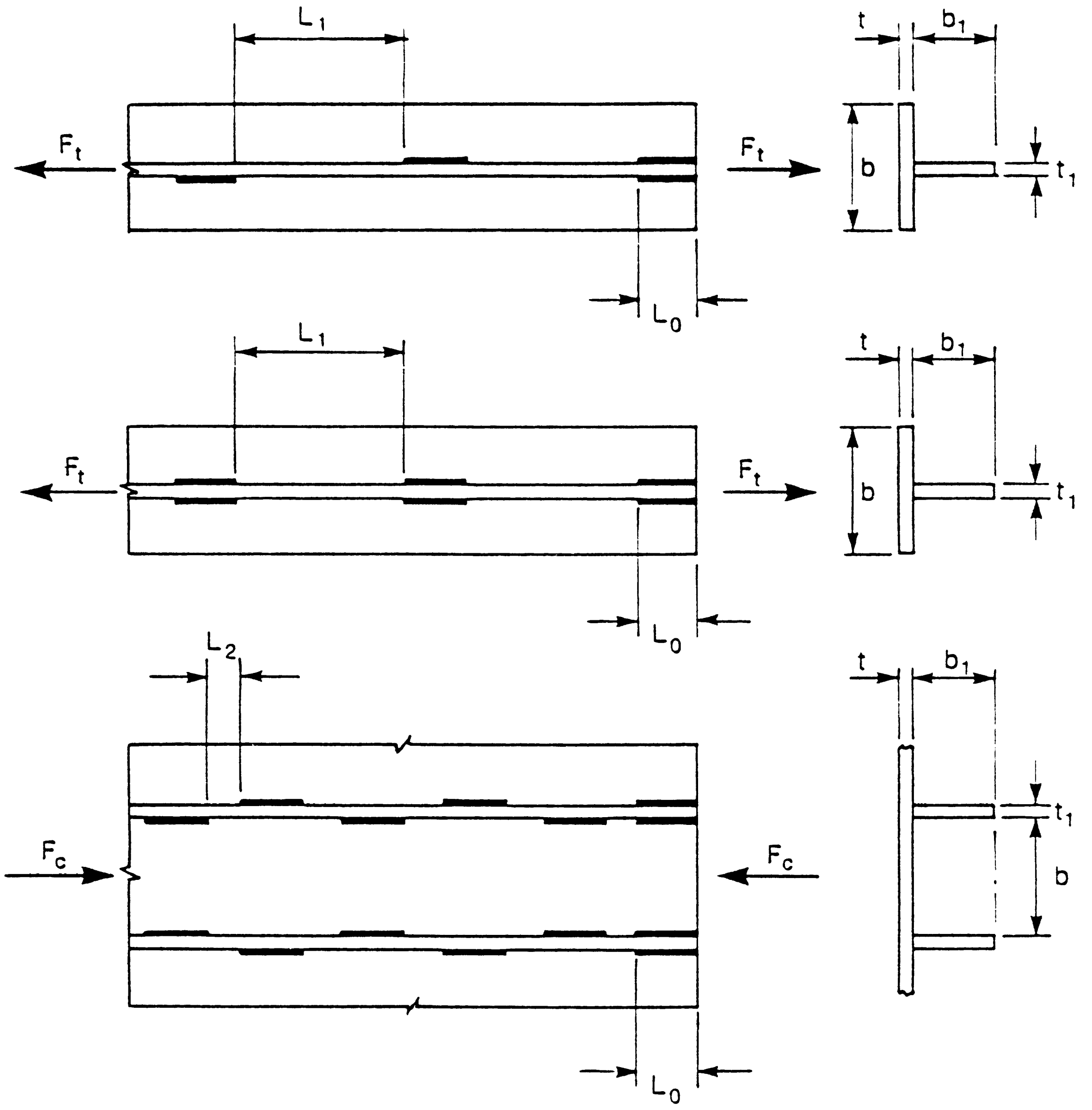
(11) In a fabricated member in which plates are connected by means of intermittent fillet welds, a continuous fillet weld shall be provided on each side of the plate for a length at each end equal to at least three-quarters of the width of the narrower plate concerned (see Figure 6.6.1).

(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

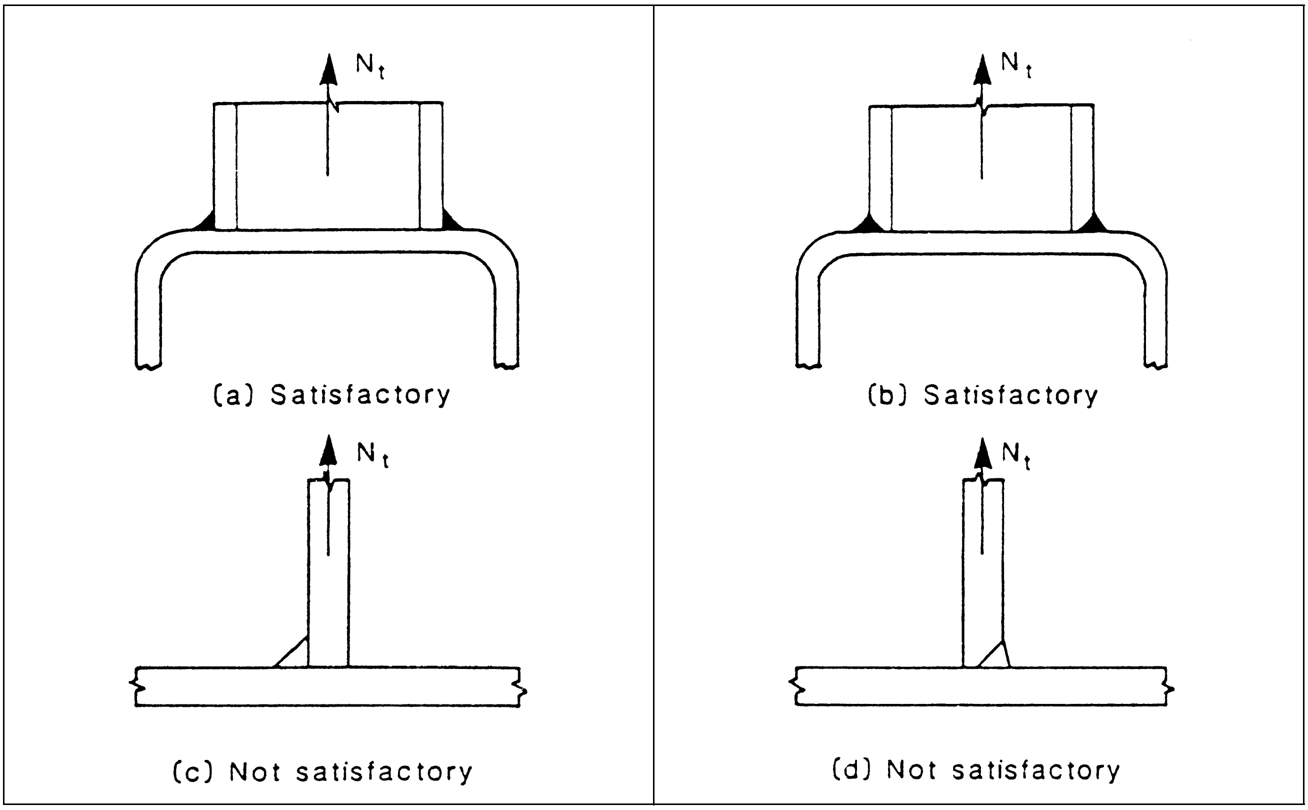


Figure 6.6.2 — Single fillet welds and single-sided partial penetration butt welds

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).

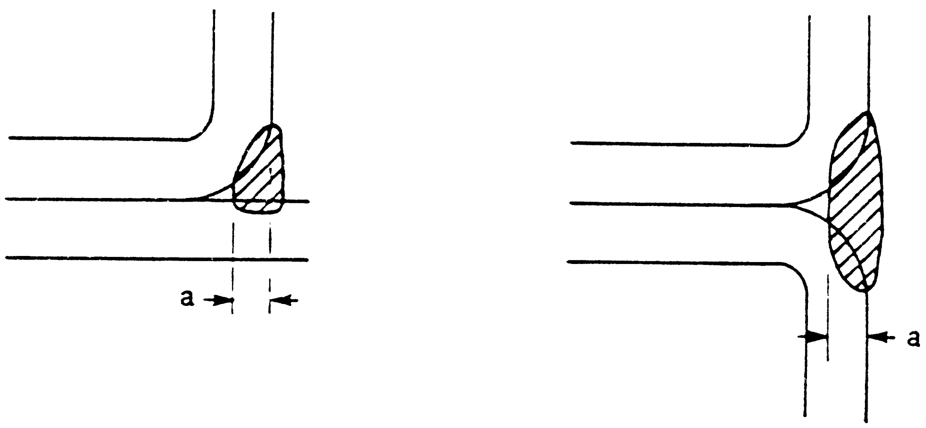


Figure 6.6.3 — Effective throat of flare groove welds in rectangular structural hollow sections

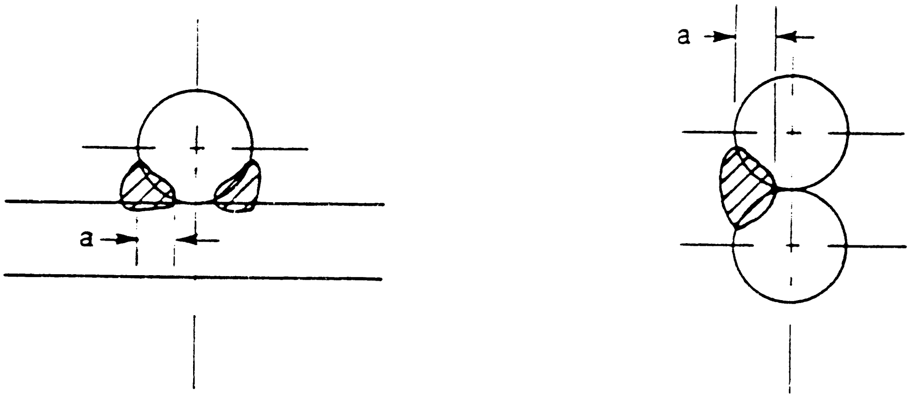


Figure 6.6.4 — Effective throat of flare groove welds in solid sections

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.*

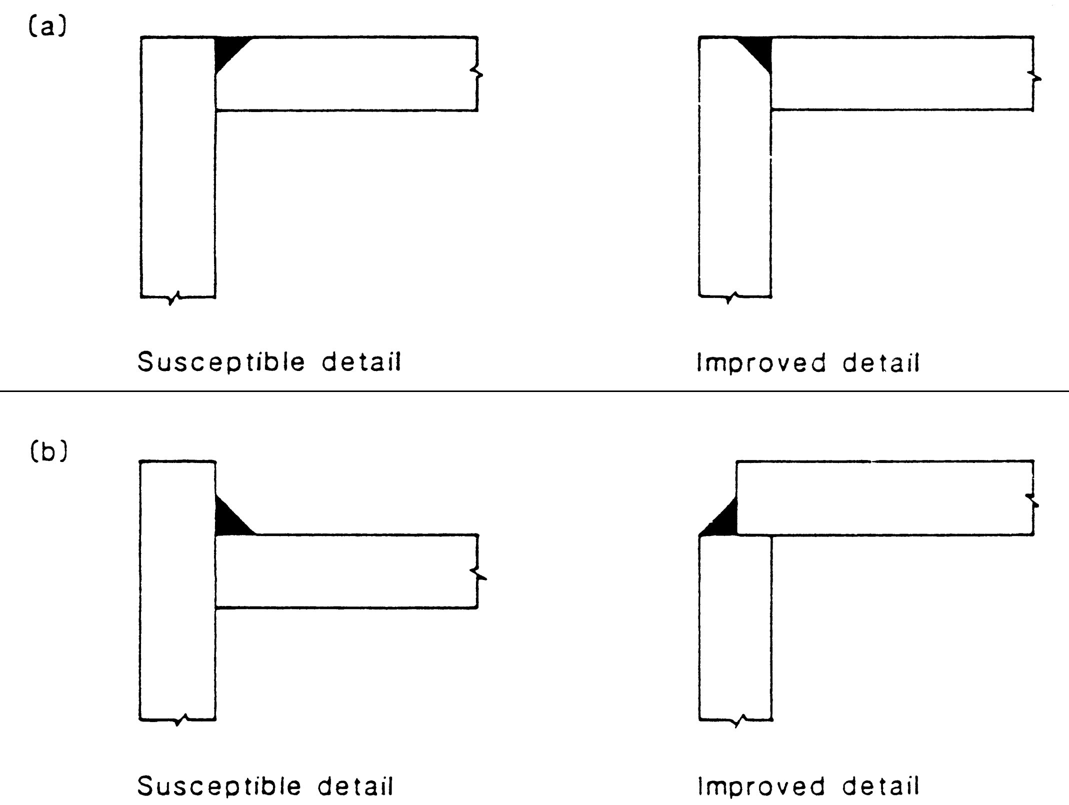


Figure 6.6.5 — Avoiding lamellar tearing

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.*

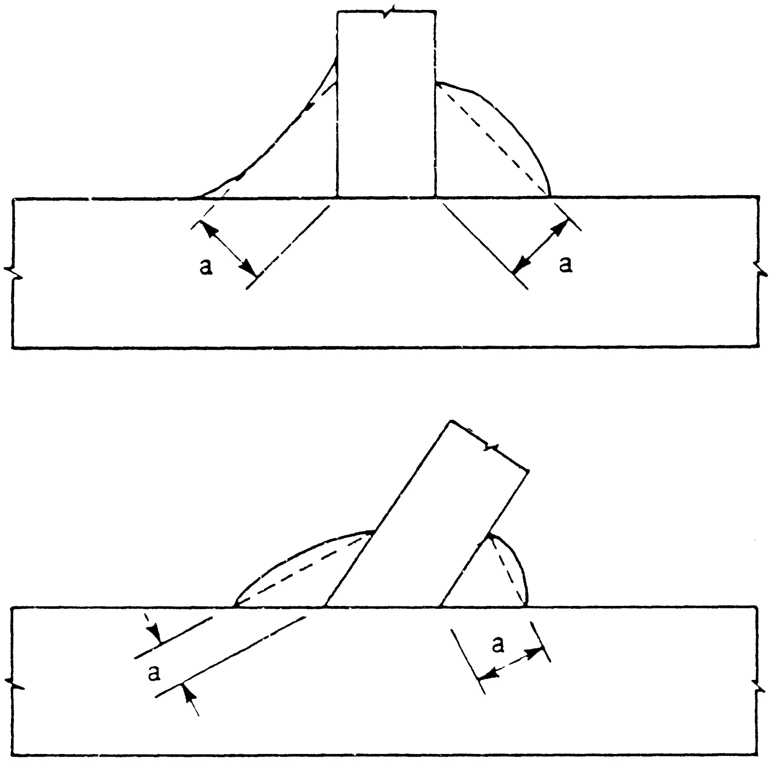


Figure 6.6.6 — Throat thickness of a fillet weld

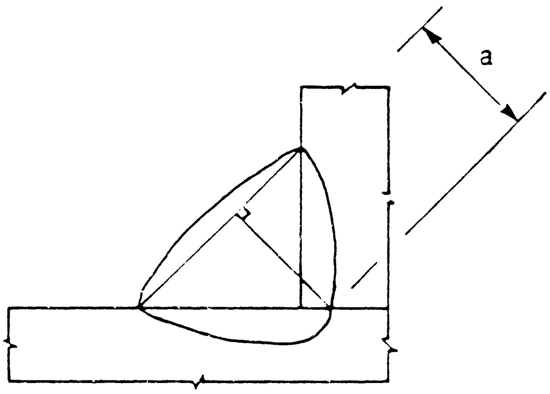


Figure 6.6.7 — Throat thickness of a deep penetration fillet weld

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:

$$F_{w.Rd} = f_{vw.d}\, a \quad (6.14)$$

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.*

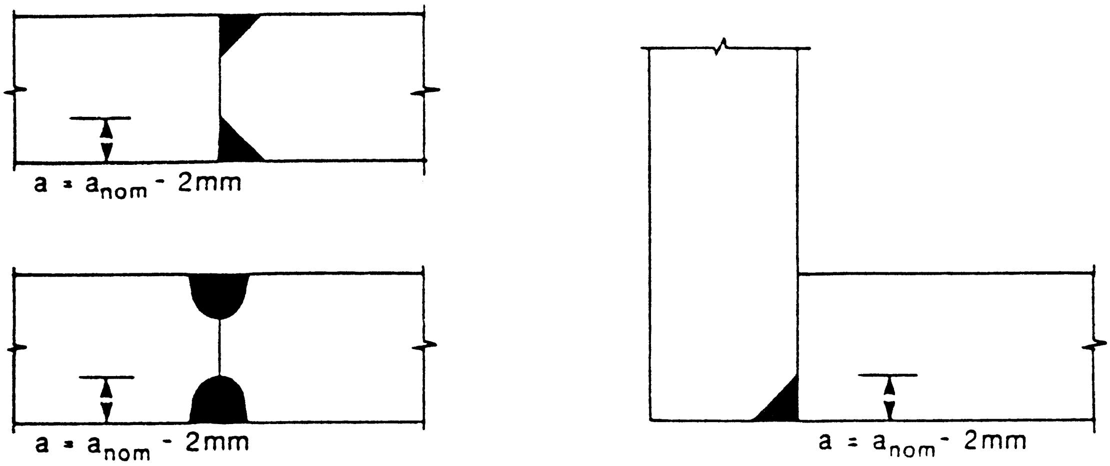


Figure 6.6.8 — Partial penetration butt welds

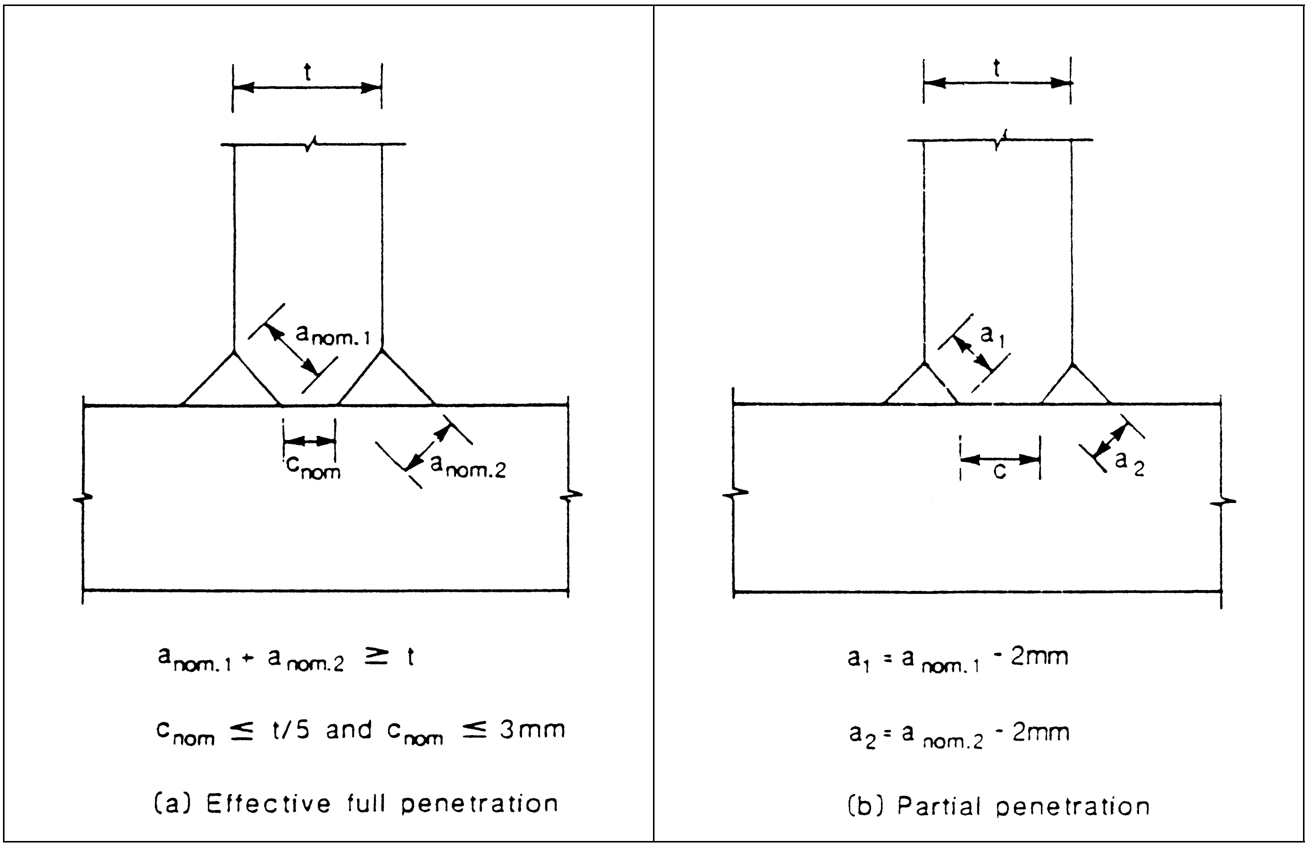


Figure 6.6.9 — Tee-butt welds

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:*

$$b_{eff} = t_w + 2r + 7t_f \quad \text{but} \quad b_{eff} \leq t_w + 2r + 7(t_f^2/t_p)(f_y/f_{yp}) \tag{6.16}$$

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:*

$$b_{eff} = 2t_w + 5t_f \quad \text{but} \quad b_{eff} \leq 2t_w + 5(t_f^2/t_p)\,(f_y/f_{yp}) \tag{6.17}$$

(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.

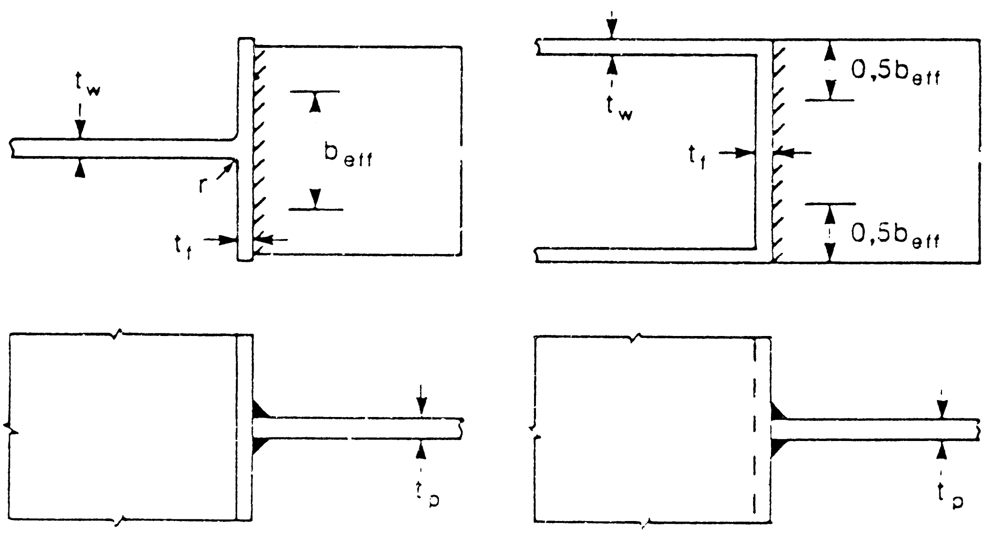


Figure 6.6.10 — Effective breadth of an unstiffened tee-joint

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:*

$$\beta_{Lw.1} = 1,2 - 0,2L_j/(150a) \tag{6.18}$$

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:*

$$\beta_{Lw.2} = 1,1 - L_w/17 \tag{6.19}$$

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.

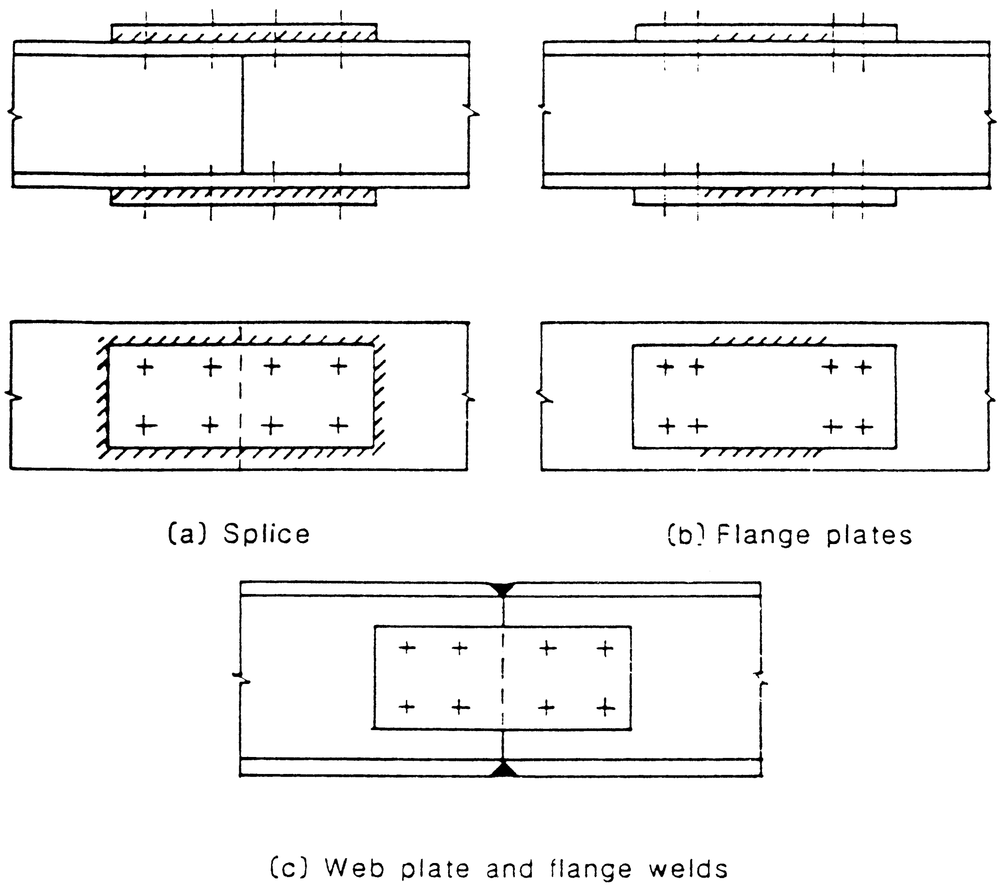


Figure 6.7.1 — Hybrid connections

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).

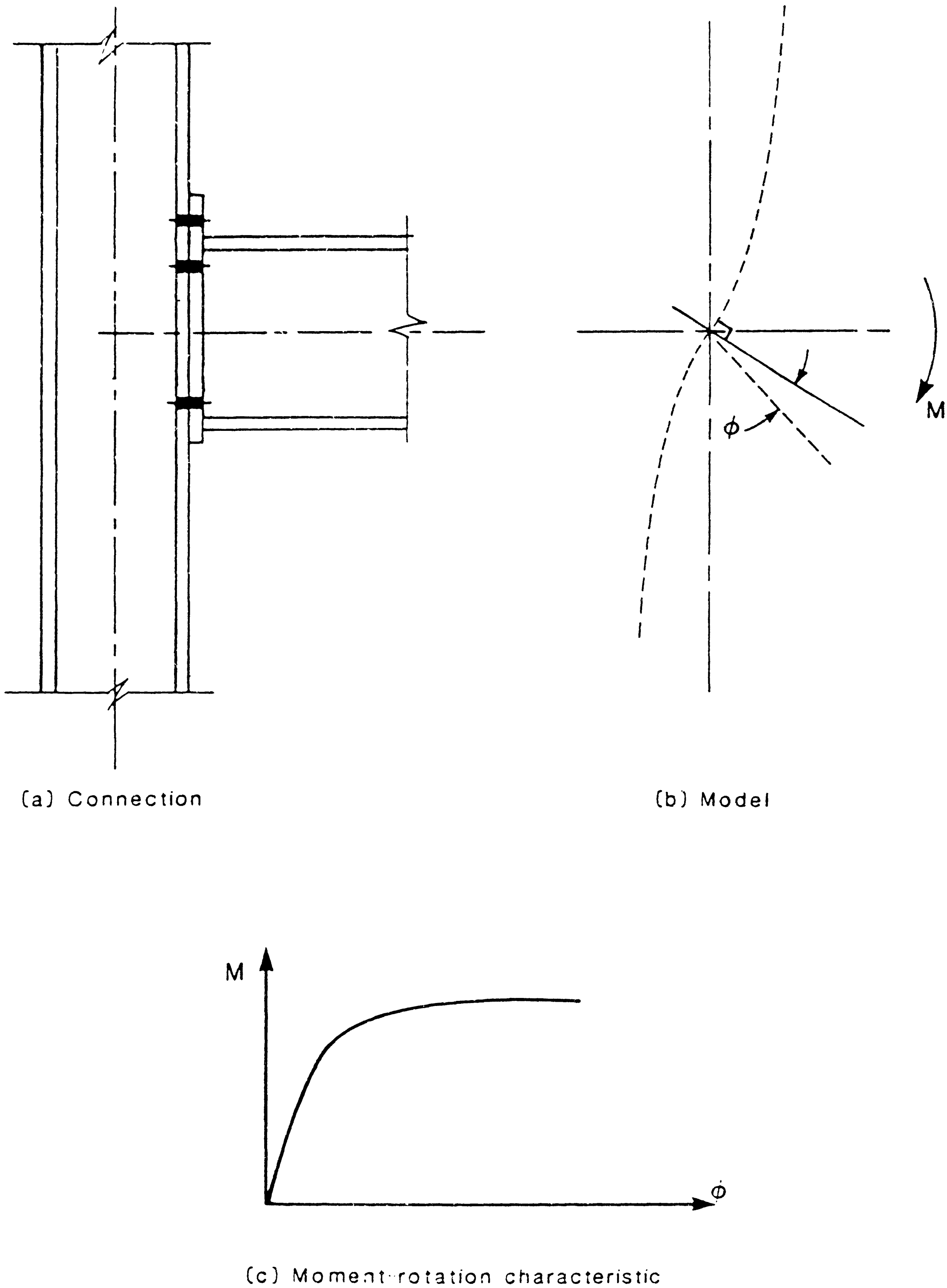


Figure 6.9.1 — Modelling a connection as a rotational spring

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.

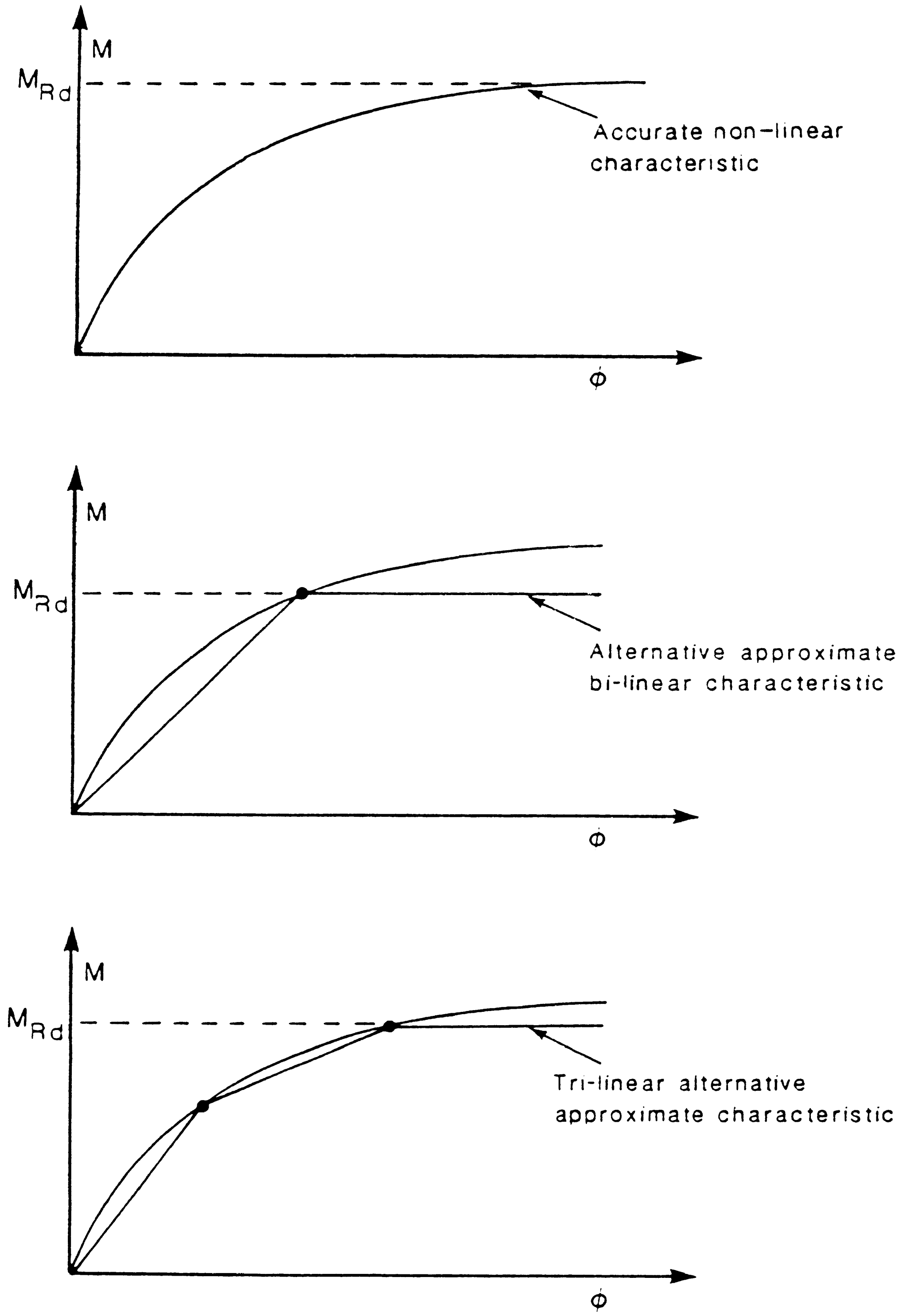
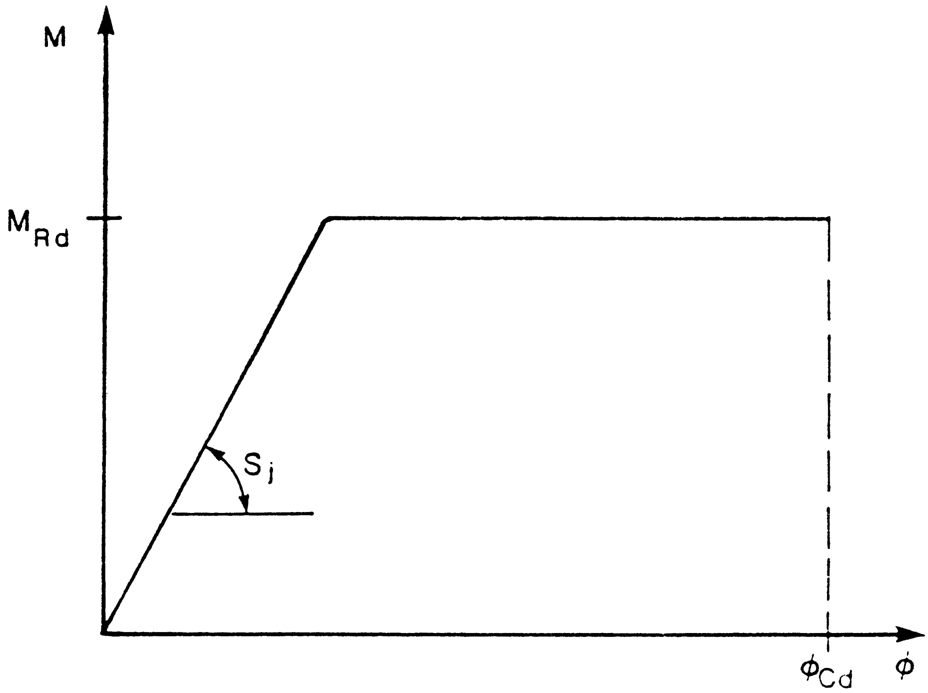


Figure 6.9.2 — Derivation of approximate moment-rotation characteristics

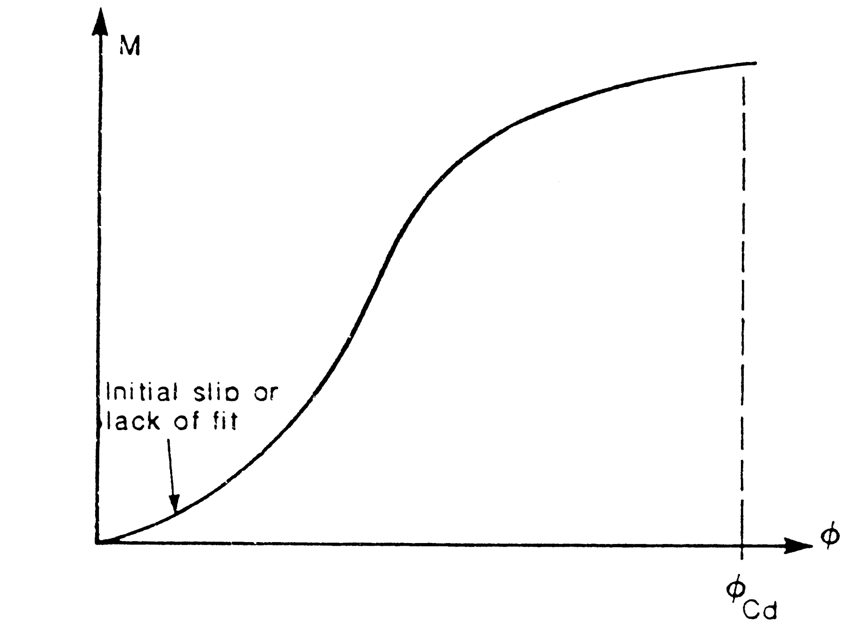


M_{Rd} = Design moment resistance

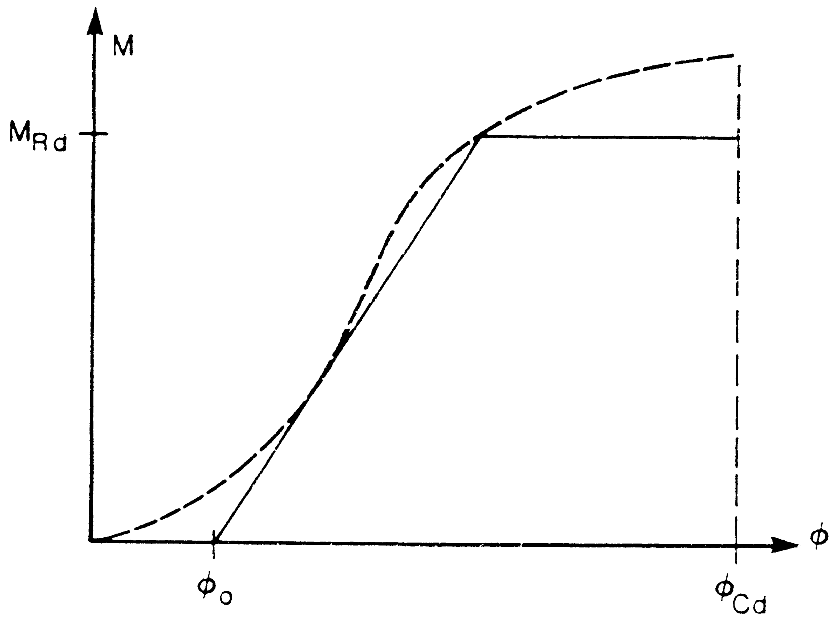
S_j = Rotational stiffness

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

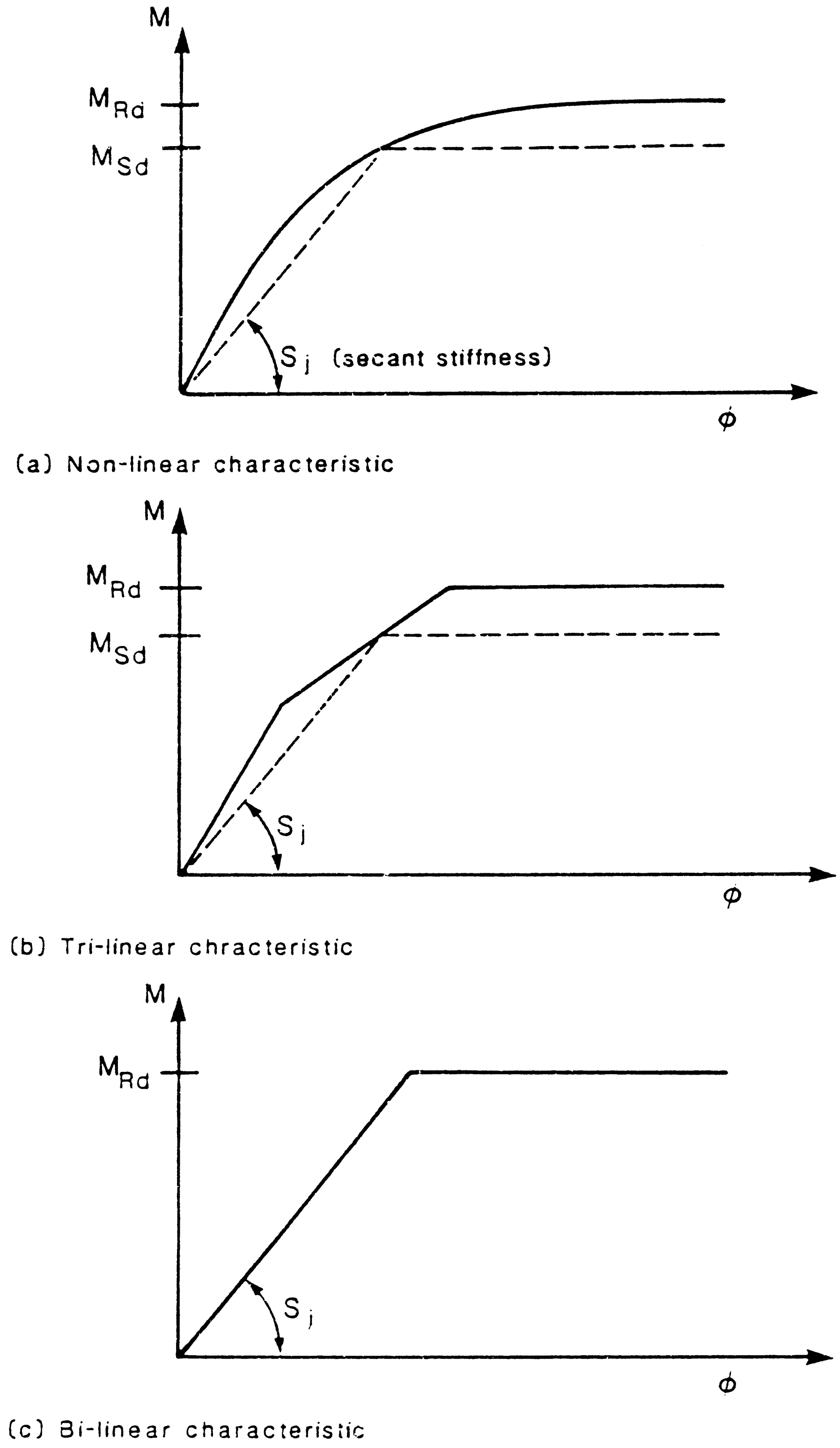


Figure 6.9.5 — Rotational stiffness S_j

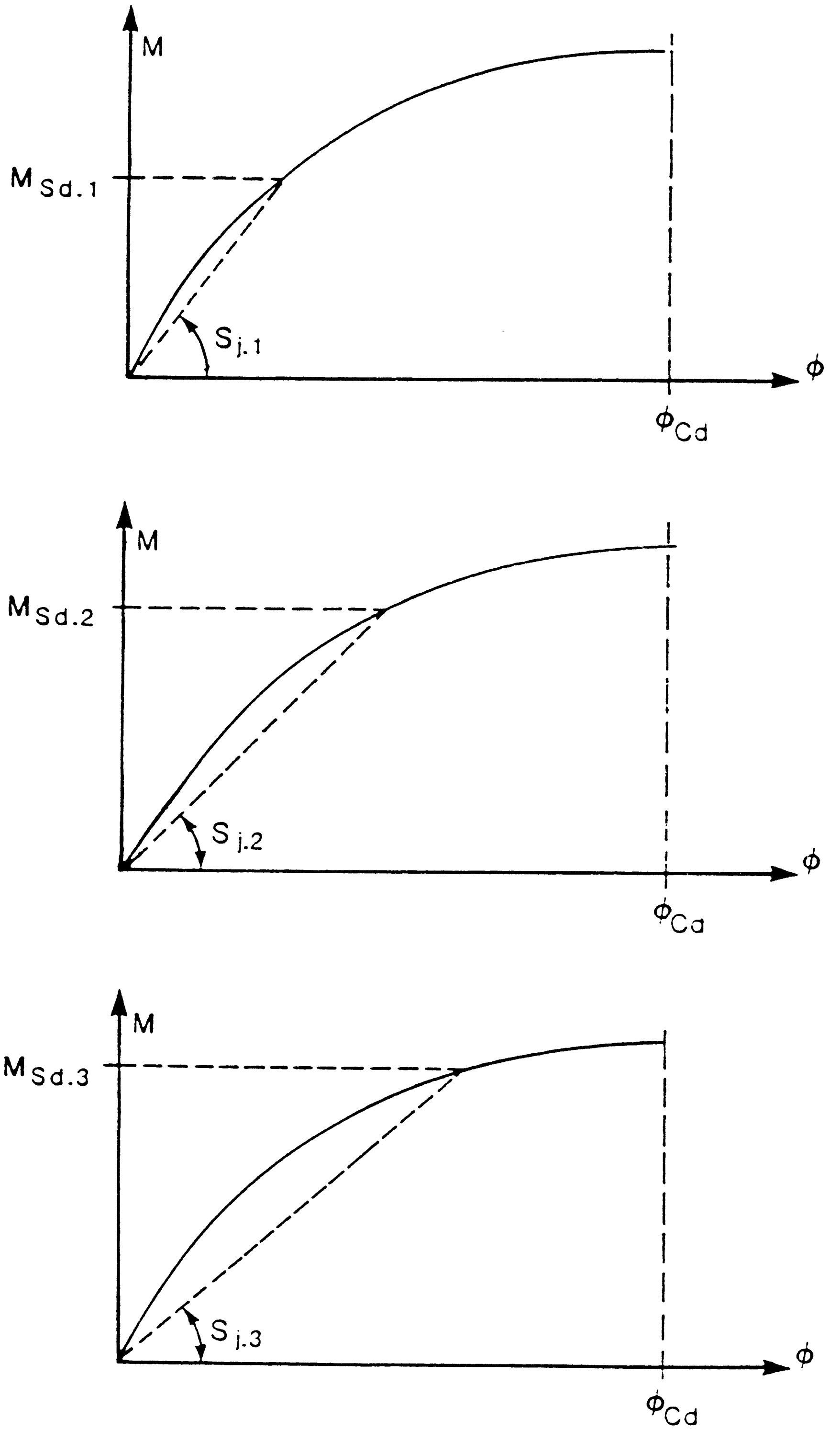


Figure 6.9.6 — Variation of rotational stiffness with applied moment

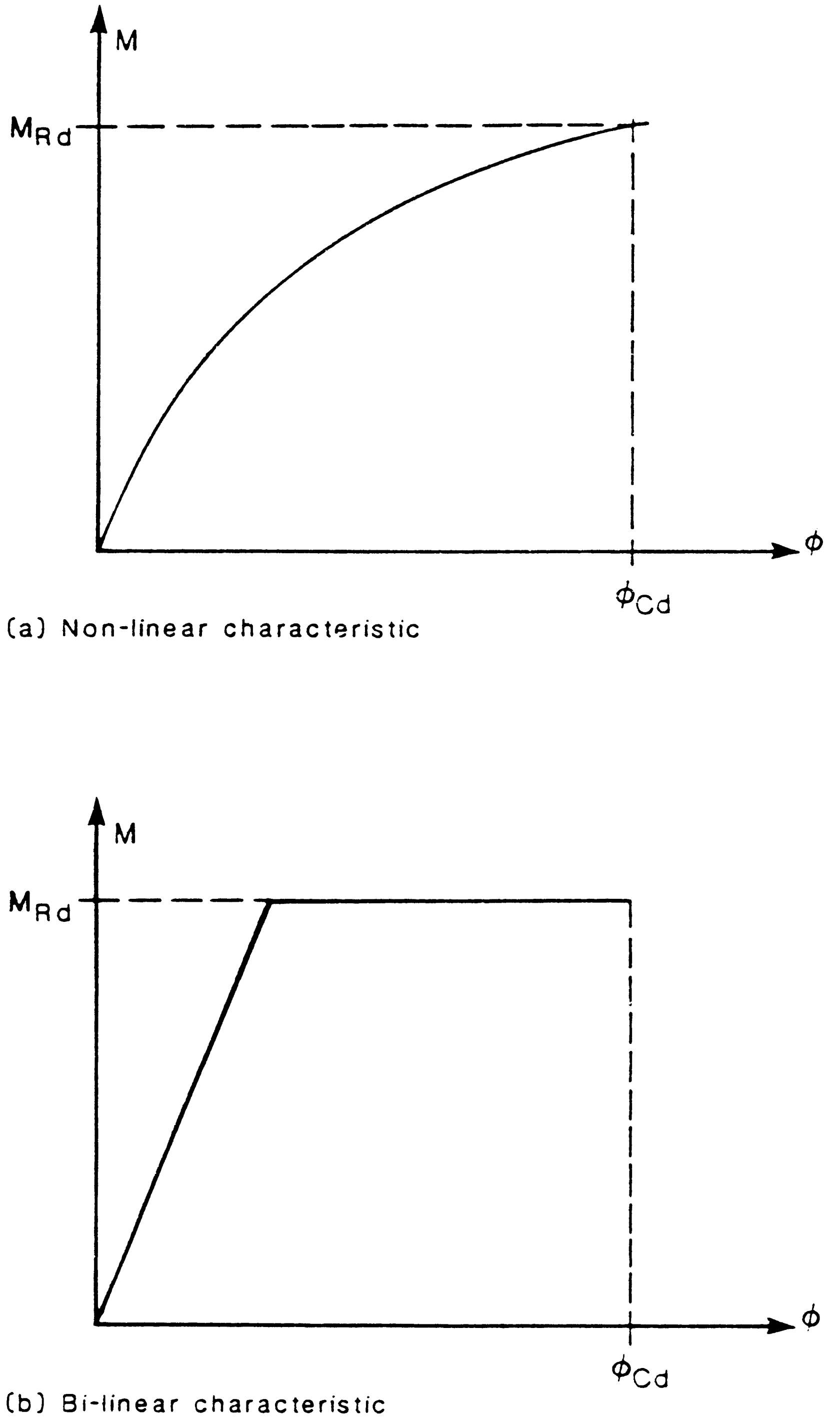


Figure 6.9.7 — Rotation capacity ϕ_{Cd}

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:

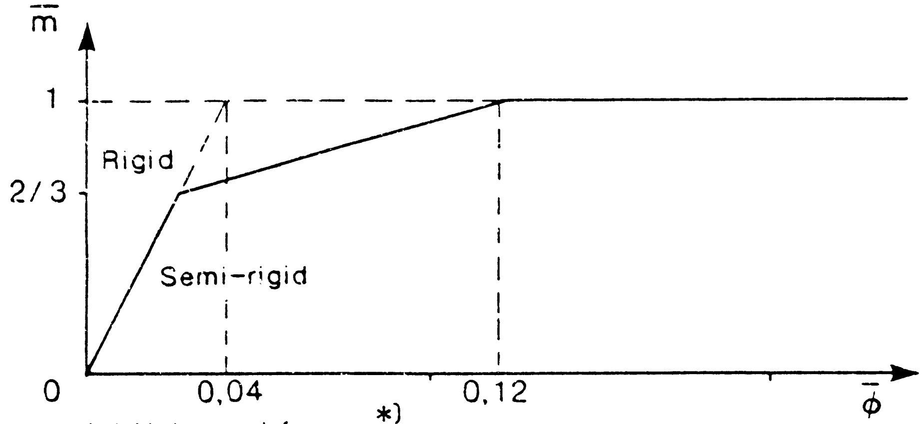
- nominally pinned, see **6.4.3.1**.
- full-strength, see **6.4.3.2**.
- partial-strength, see **6.4.3.3**.

(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* $M_{p\ell.Rd}$.

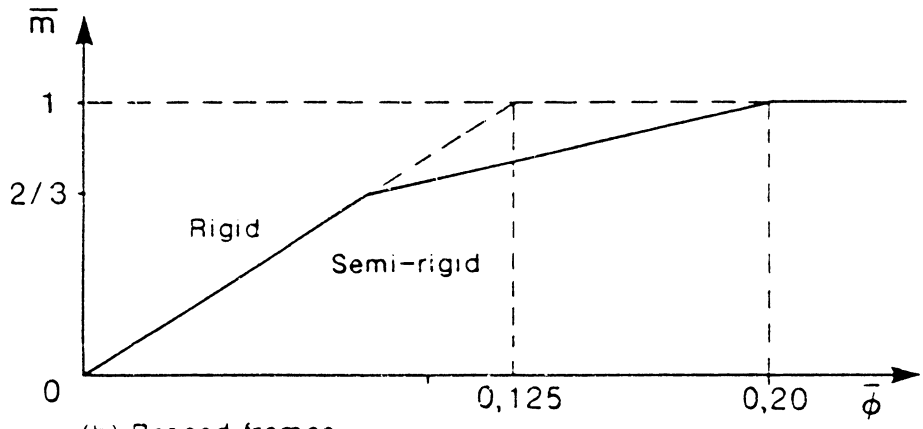


(a) Unbraced frames *)

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.*

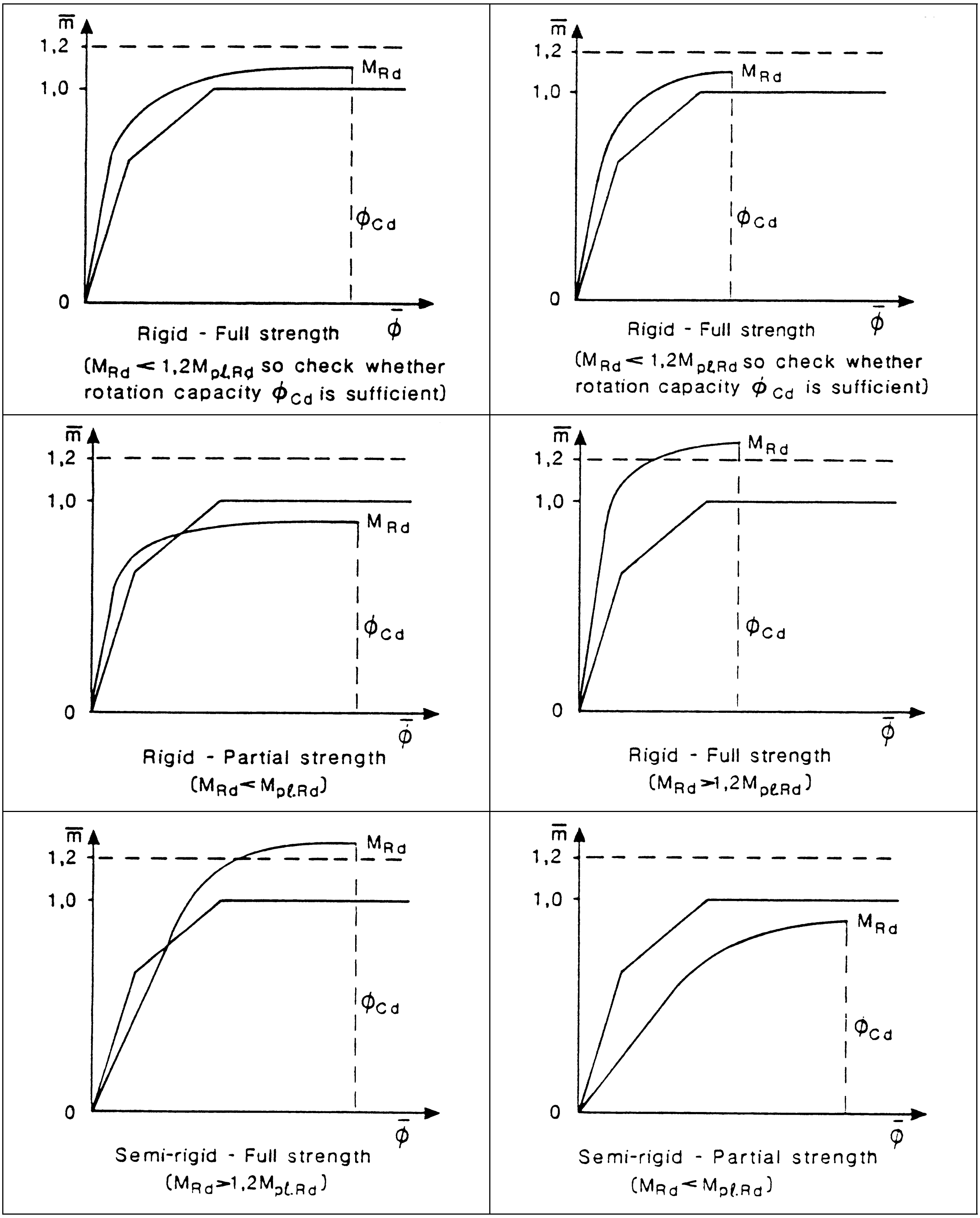


Figure 6.9.9 — Examples of classification of moment-rotation characteristics for beam-to-column connections

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*

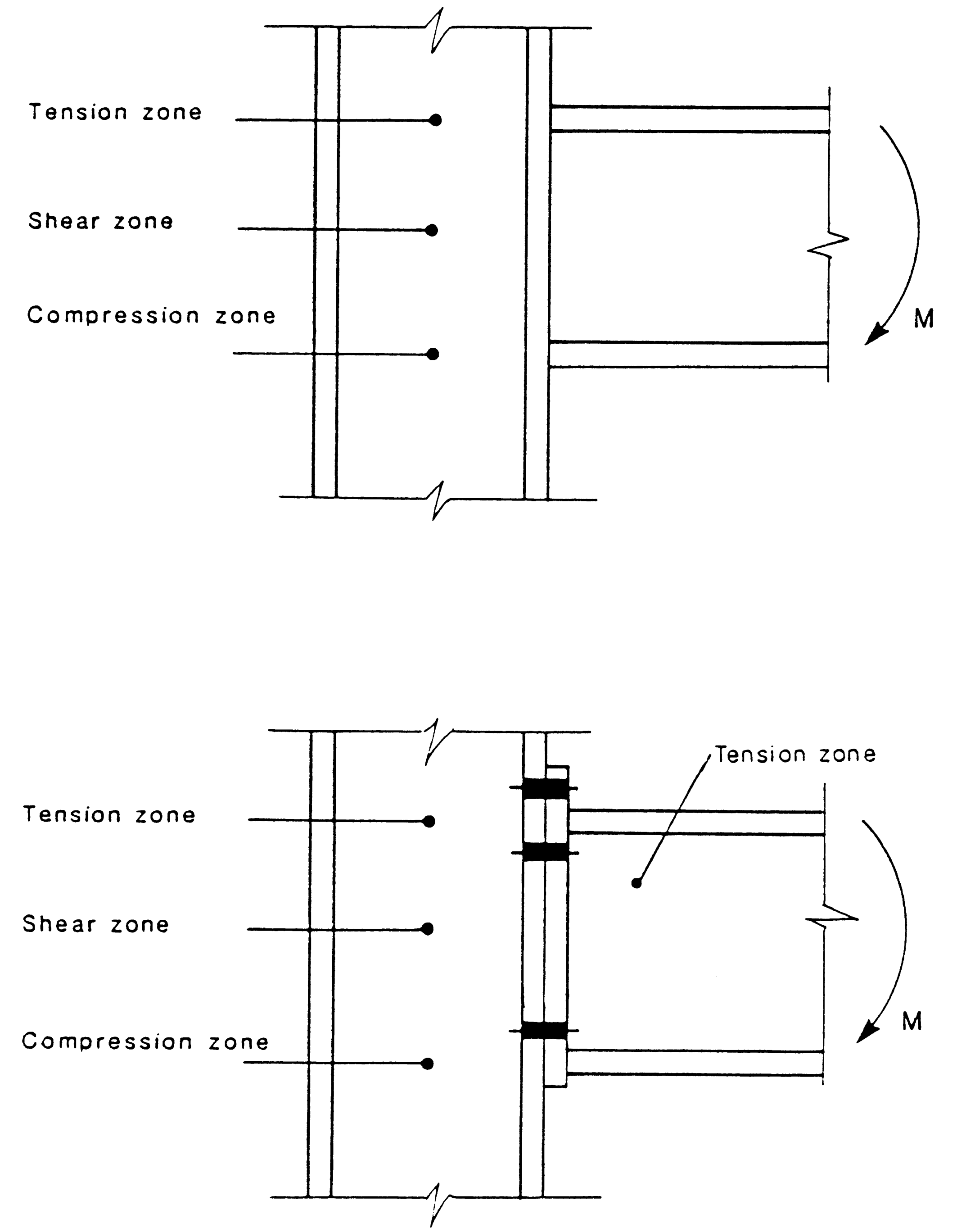


Figure 6.9.10 — Critical zones in beam-to-column connections

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*

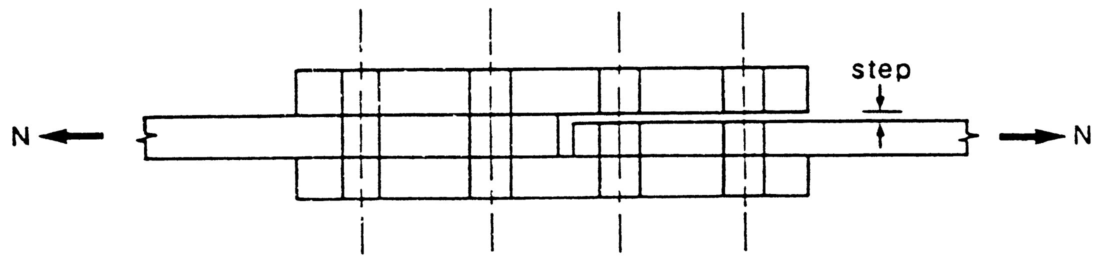


Figure 7.1 — Maximum step between adjacent surfaces

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: $0{,}010h$

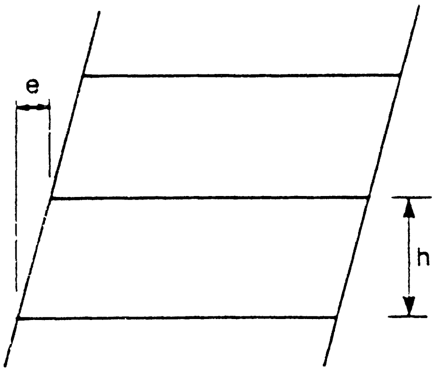
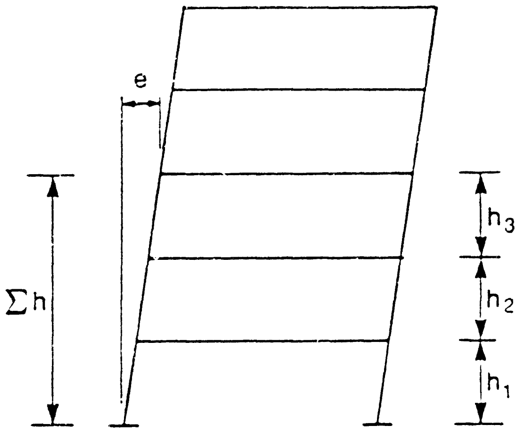
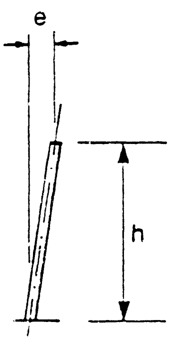
Criterion	Description	Permitted deviation
Inclination of a column between adjacent floor levels		$e \le 0{,}002h$
Location of a column at any floor level, from a vertical line through the intended location of the column base		$e \le \frac{0{,}0035\sum h}{n^{1/2}}$
Inclination of a column in a single-storey building, without a crane gantry, other than in a portal frame		$e \le 0{,}0035h$

Figure 7.2.1 — Normal tolerances after erection — Part 1

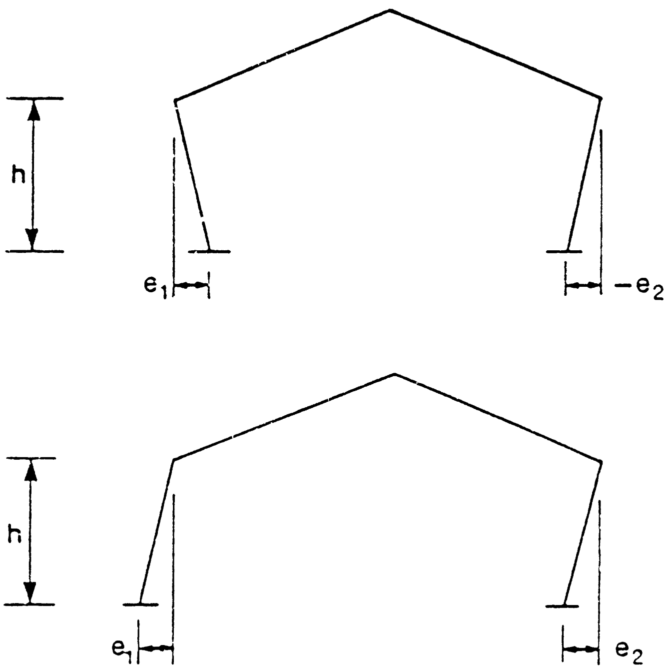
Criterion:	Inclination of columns of a portal frame not supporting a crane gantry.
Description: 	
Permitted deviation: Inclination of individual columns: $e_1 \le 0{,}010h : e_2 \le 0{,}010h$ Mean inclination of a frame: $\frac{e_1 + e_2}{2} \le 0{,}002h$ where $e_1 \ge e_2$	

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:

$$\gamma_{Ff}\,\Delta\sigma \leq 26/\gamma_{Mf}\ \text{N/mm}^2. \tag{9.1}$$

b) The total number of stress cycles N satisfies:

$$N \leq 2 \times 10^6 \left[\frac{36/\gamma_{Mf}}{\gamma_{Ff}\ \Delta\sigma_{E.2}}\right]^3 \tag{9.2}$$

where $\Delta\sigma_{E.2}$ is the equivalent constant amplitude stress range in N/mm^2.

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:

$$\gamma_{Ff}\,\Delta\sigma \le \Delta\sigma_D/\gamma_{Mf} \tag{9.3}$$

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.

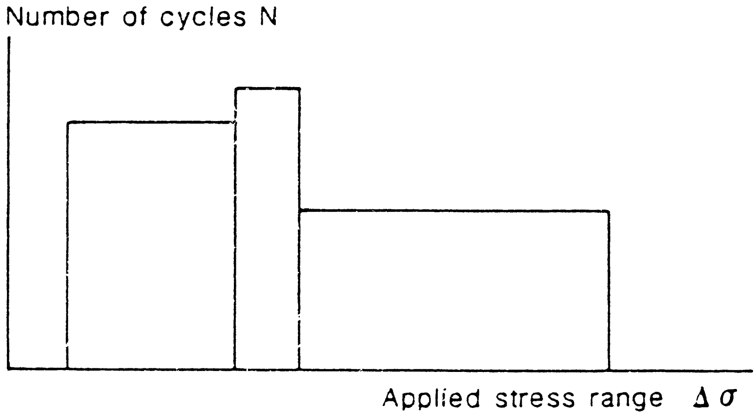


Figure 9.1.1 — Design spectrum

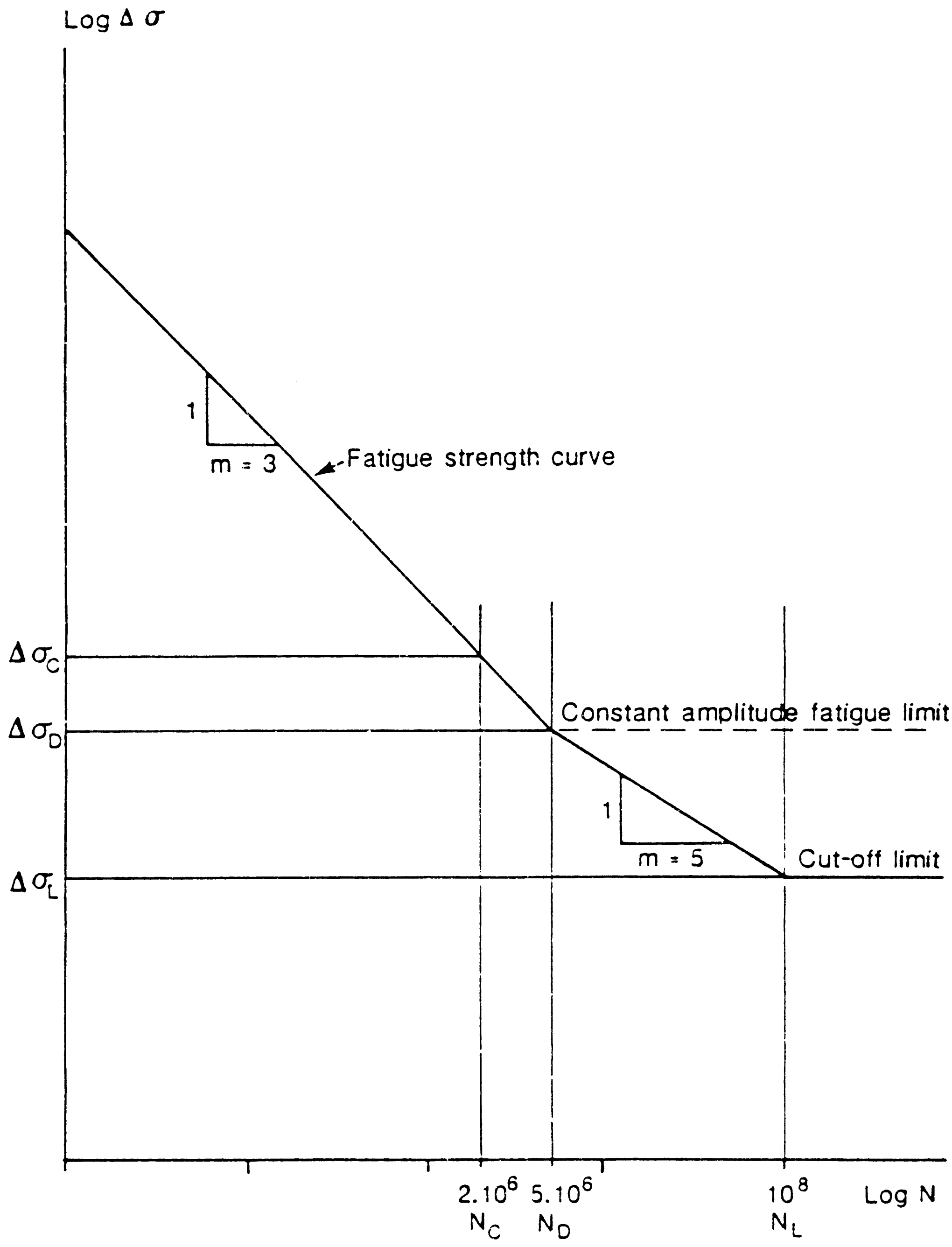


Figure 9.1.2 — Fatigue strength curve

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:

$$\gamma_{Ff}\ \Delta\sigma \leq \Delta\sigma_R/\gamma_{Mf} \qquad (9.5)$$

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:

$$D_d \leq 1 \qquad \text{where} \quad D_d = \Sigma \frac{n_i}{N_i} \qquad (9.6)$$

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* $\Delta\sigma_D/\gamma_{Mf} > \gamma_{Ff}\ \Delta\sigma_i\ \geq \Delta\sigma_L/\gamma_{Mf}$:

$$N_i = 5 \times 10^6 \left[\frac{\Delta\sigma_D/\gamma_{Mf}}{\gamma_{Ff}\Delta\sigma_i} \right]^5 \tag{9.8}$$

- γ_{Ff} $\Delta\sigma_i \circ \Delta\sigma_L/\gamma_{Mf}$:

$$N_i = \infty \tag{9.9}$$

(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* $\gamma_{Ff}\ \Delta\tau_i\ \Pi \Delta\tau_L/\gamma_{Mf}$:

$$N_i = \infty \tag{9.13}$$

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:

$$D_{d.\sigma} + D_{d.\tau} \leq 1 \qquad (9.14)$$

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 $\Delta\tau_i$

(5) When using equivalent constant amplitude stress ranges, this criterion generally becomes:

$$\left[\frac{\gamma_{Ff}\ \Delta\sigma_E}{\Delta\sigma_R/\gamma_{Mf}}\right]^3 + \left[\frac{\gamma_{Ff}\ \Delta\tau_E}{\Delta\tau_R/\gamma_{Mf}}\right]^5 \leq 1 \qquad (9.15)$$

(5) *Alternatively, an equivalent constant amplitude fatigue assessment may be made using the specific criterion:*

$$\left[\frac{\gamma_{Ff}\ \Delta\sigma_{E.2}}{\Delta\sigma_C/\gamma_{Mf}}\right]^3 + \left[\frac{\gamma_{Ff}\ \Delta\tau_{E.2}}{\Delta\tau_C/\gamma_{Mf}}\right]^5 \leq 1 \qquad (9.16)$$

(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:

$$D_{d.\sigma} + D_{d.\tau} \leq 1 \qquad (9.17)$$

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:

$$\log N = \log a - m \log \Delta\sigma_R \qquad (9.18)$$

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**.

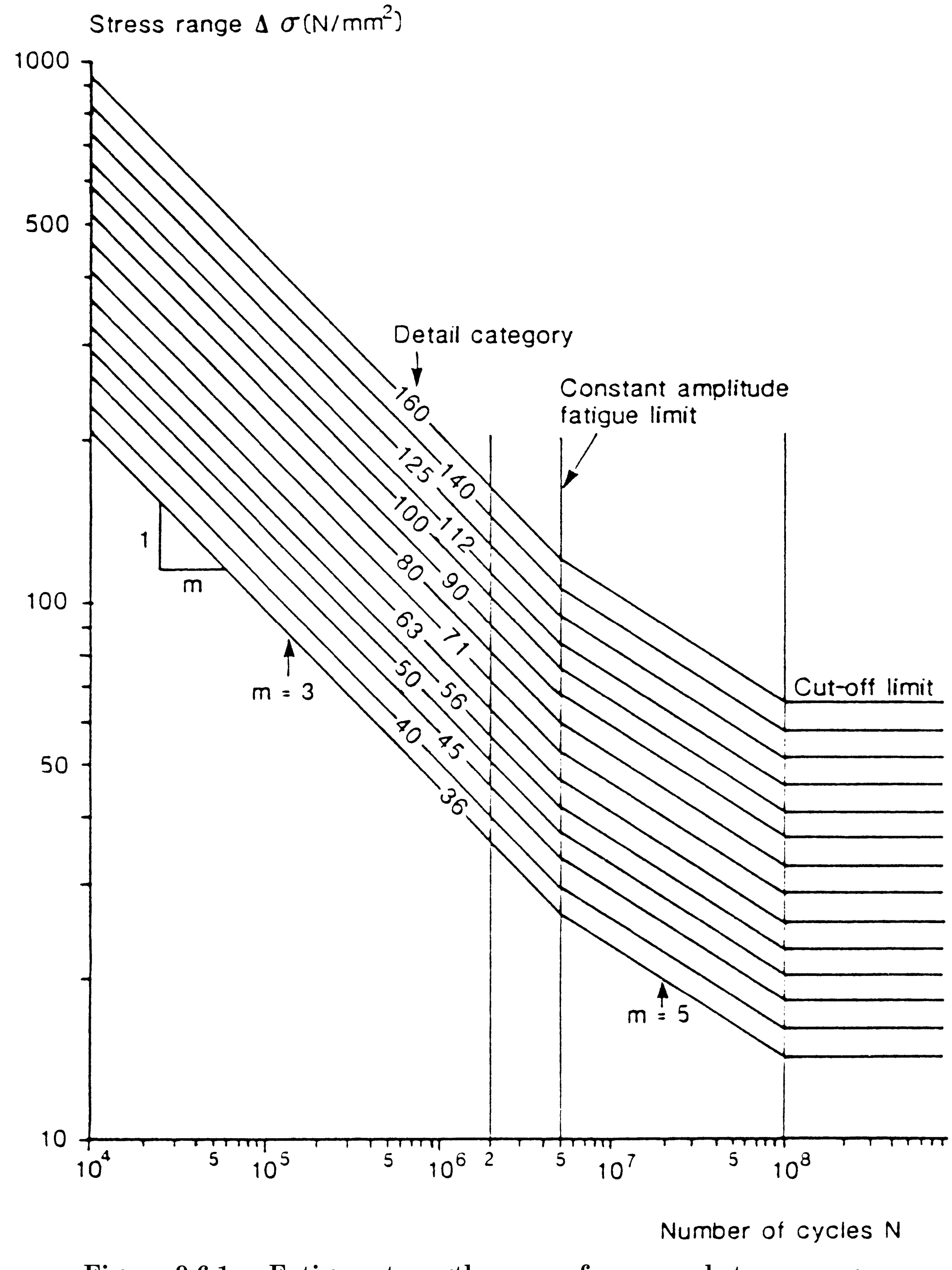


Figure 9.6.1 — Fatigue strength curves for normal stress ranges

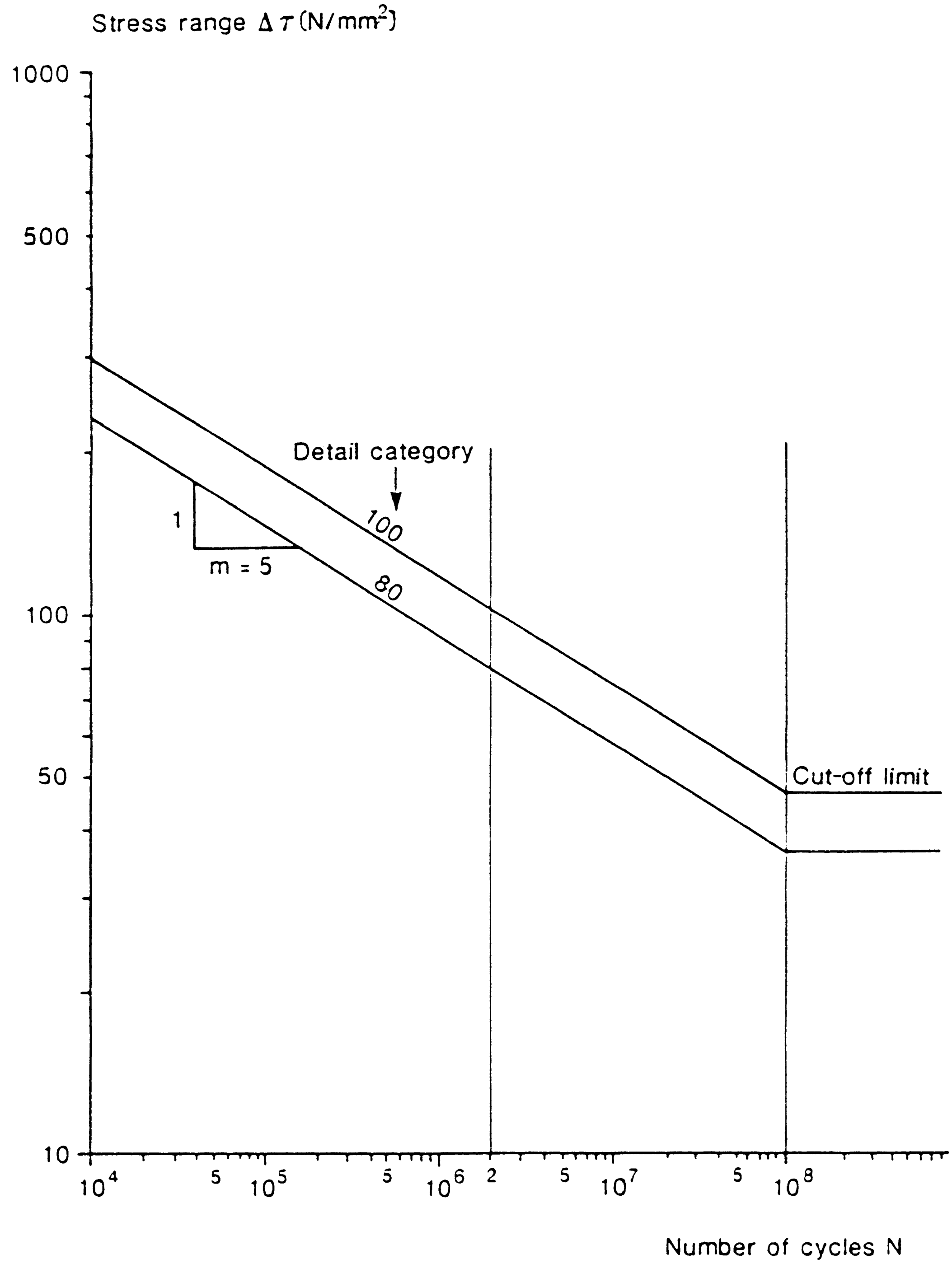


Figure 9.6.2 — Fatigue strength curves for shear stress ranges

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 $N < 10^8$		Stress range at constant amplitude fatigue limit	Stress range at cut-off limit
	$N \le 5 \times 10^6$	$N \ge 5 \times 10^6$	$(N = 5 \times 10^6)$	$(N = 10^8)$
$\Delta\sigma_C$ (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 $N < 10^8$ (m = 5)	Stress range at cut-off limit $(N = 10^8)$ $\Delta\tau_L$ (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.

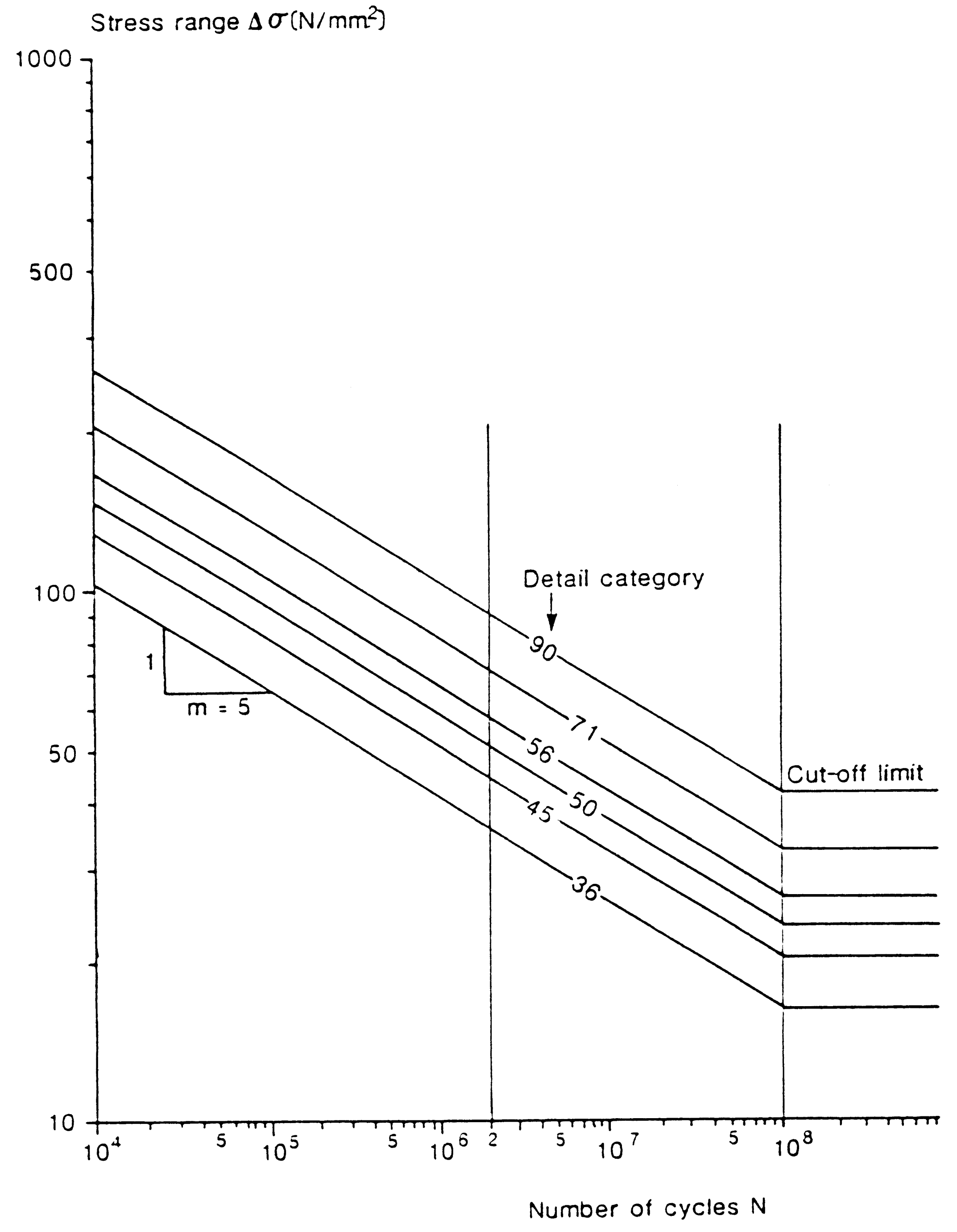


Figure 9.6.3 — Fatigue strength curves for joints in tubular lattice girders

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)	$N \le 10^7$ (m = 3)	$N \ge 10^7$ (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.

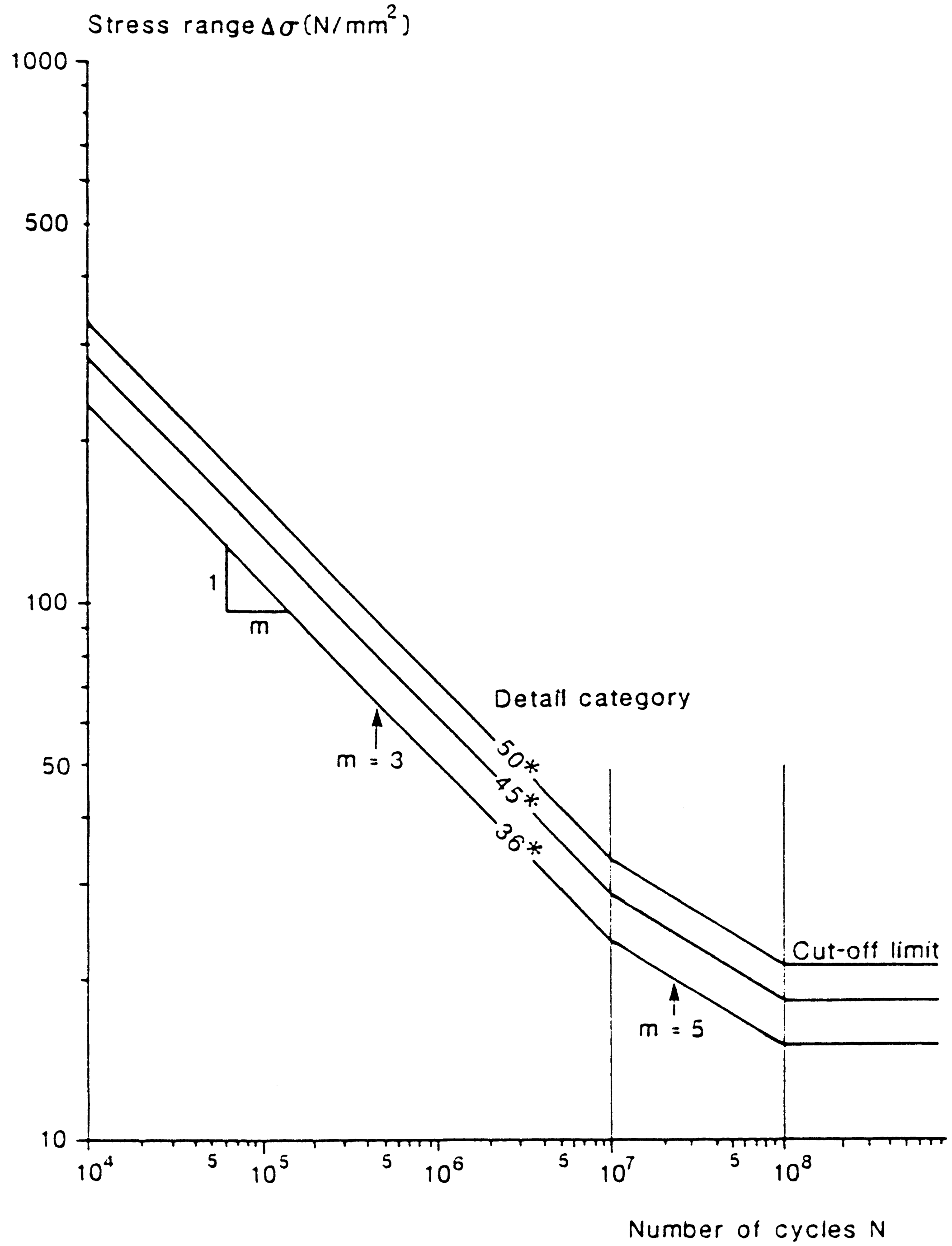


Figure 9.7.1 — Modified fatigue strength curve

Table 9.8.1 — Non-welded details — sheet 1 of 2

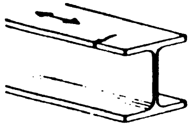
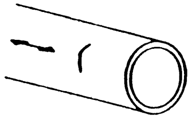
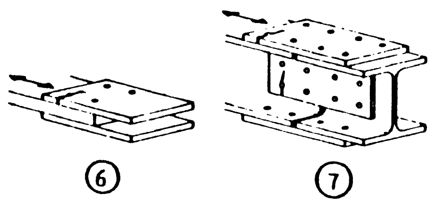
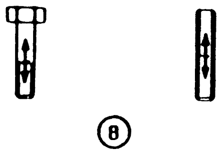
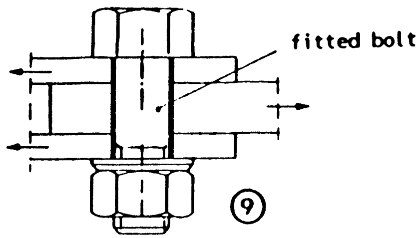
Detail category	Constructional details	Description	Requirement
160	 ①  ②  ③	Rolled and extruded products ① Plates and flats. ② Rolled sections. ③ Seamless hollow sections (see Table 9.8.6 and Table 9.8.7.	① to ③: Sharp edges, surface and rolling flaws to be improved by grinding.
140	 ④	Sheared or gas 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	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

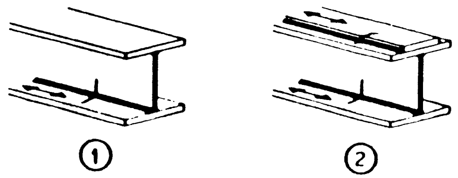
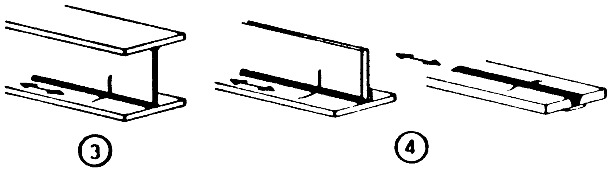
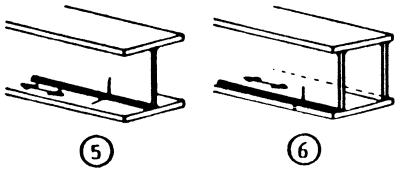
Detail category	Constructional details	Description	Requirement
125		Continuous longitudinal welds ① Automatic butt welds carried out from both sides. If a specialist inspection demonstrates that longitudinal welds are free from significant flaws, category 140 may be used.	① and ② — No stop/start position is permitted except when the repair is performed by a specialist and inspection carried out to verify the proper execution of the repair.
112		② Automatic fillet welds. Cover plate ends shall be verified using detail ⑤ in Table 9.8.5. 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

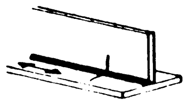
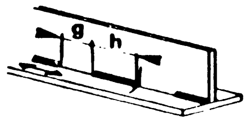
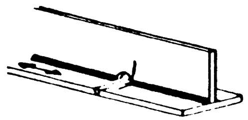
Detail category	Constructional details	Description	Requirement
100	 ⑦	⑦ Repaired automatic or manual fillet or butt welds.	⑦ — Improvement methods which are adequately verified may restore the original category.
80	 ⑧	Intermittent longitudinal welds ⑧ Stitch or tack welds not subsequently covered by a continuous weld.	⑧ — Intermittent fillet weld with gap ratio g/h $\leqslant$ 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

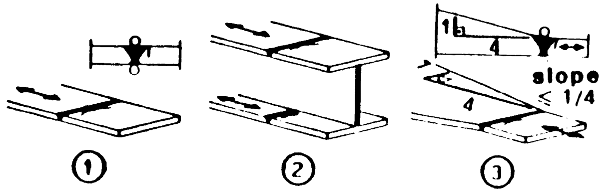
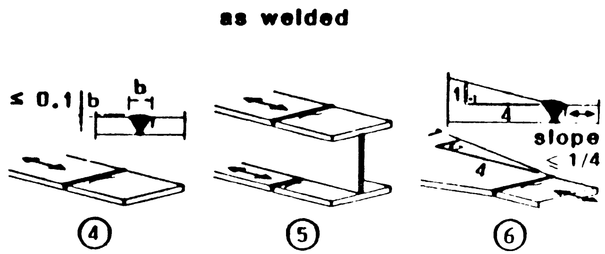
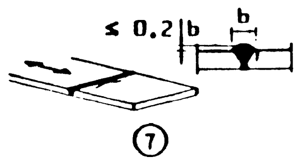
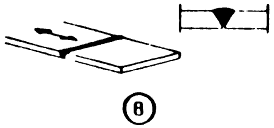
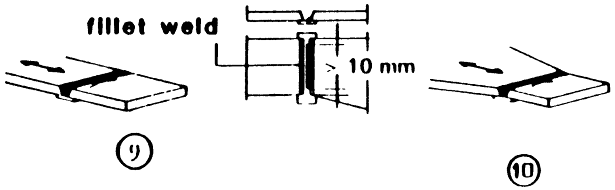
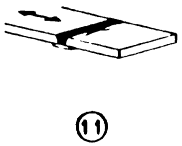
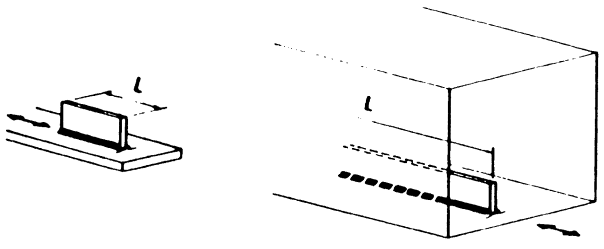
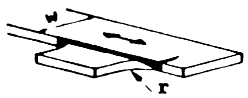
Detail category	Constructional details	Description	Requirement
112		Without backing bar ① Transverse splices in plates, flats and rolled sections. ② Flange splices in plate girders before assembly. ③ Transverse splices in plates or flats tapered in width or in thickness where the slope is not greater 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 Butt welds made from one side only.	⑧ — Without backing strip.
71		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. ⑪ Transverse butt weld on a permanent backing strip.	⑨ and ⑩ — The fillet weld which attaches the backing strip to terminate more then 10 mm from the edges of the stressed plate. ⑪ — When backing strip fillet welds are terminated closer than 10 mm to the plate edge, or when good fit cannot be guaranteed.
50			

[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 ⩽ 50 mm	 ①	<u>Longitudinal attachments</u> ① The Detail Category varies according to the length of the attachment, L.	
71	50 < L ⩽ 100 mm			
50[a]	L > 100 mm			
90	$\frac{1}{3} \leqslant \frac{r}{w}$ $r > 150mm$	 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

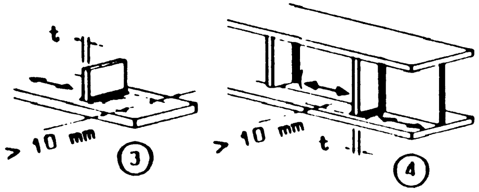
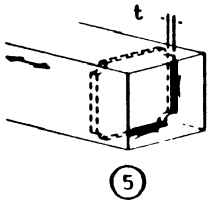
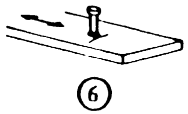
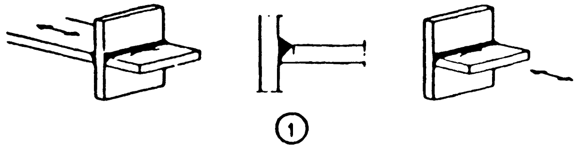
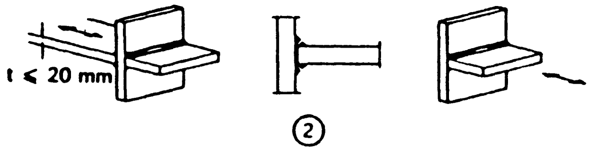
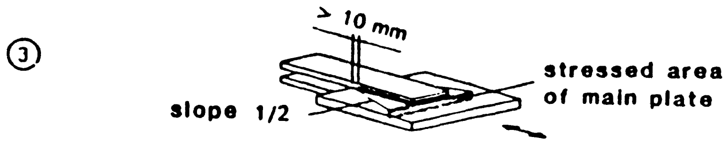
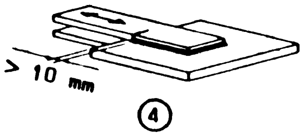
Detail category	Constructional details		Description	Requirement
80	t ⩽ 12 mm	 	<u>Transverse attachments</u> ③ Welds which terminate more than 10 mm from the edge of the plate. ④ Vertical stiffeners welded to a beam or a plate girder. ⑤ Diaphragms of box girders welded to the flange or web.	④ — The stress range should be calculated using principal stresses if the stiffener terminates in the web.
71	t > 12 mm			
80			⑥ The effect of welded shear connectors on base material.	

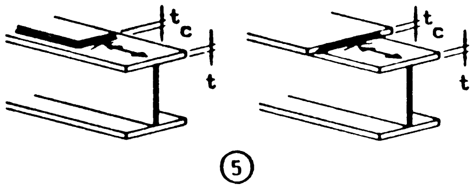
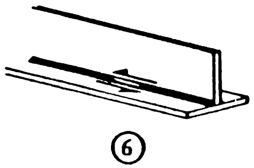
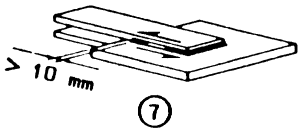
Table 9.8.5 — Welded joints with load-carrying welds — sheet 1 of 3

Detail category	Constructional details	Description	Requirement
71		<u>Cruciform joints</u> 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		<u>Overlapped welded joints</u> 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"></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"></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"></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

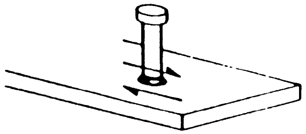
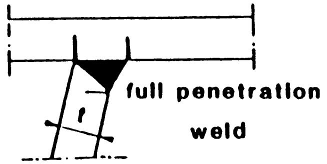
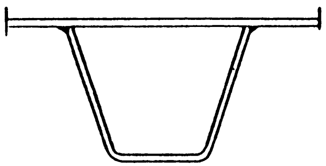
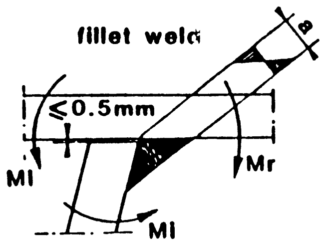
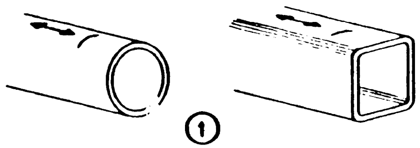
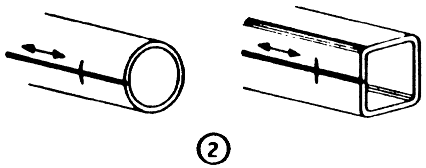
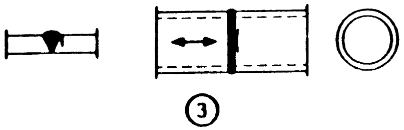
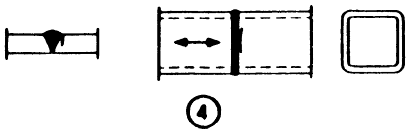
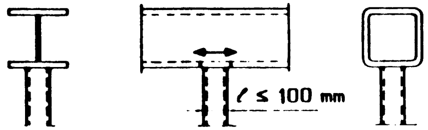
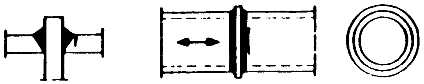
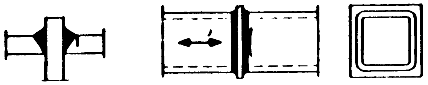
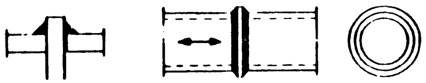
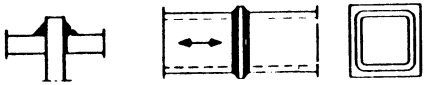
Detail category	Constructional details		Description	Requirement
80 m = 5	 ⑧		<u>Welds in shear</u> ⑧ Stud connectors (failure in the weld or heat effected zone).	⑧ — The shear stress to be calculated on the nominal cross-section of the stud.
71	 	 ⑨	<u>Trapezoidal stiffener to deck plate welds</u> ⑨ 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.	

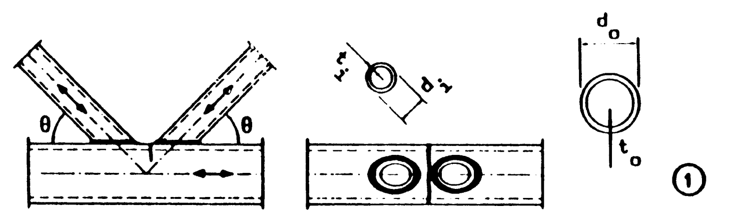
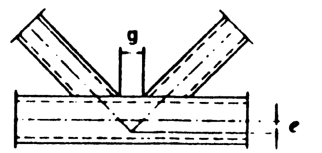
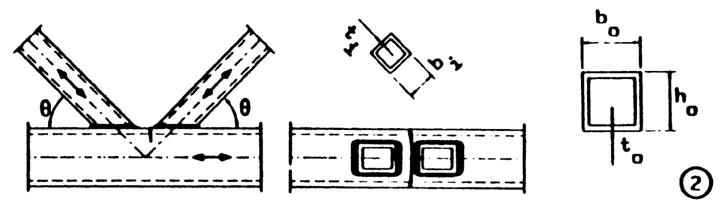
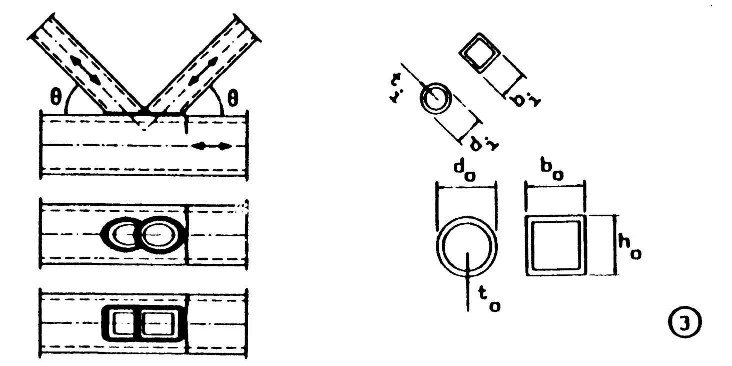
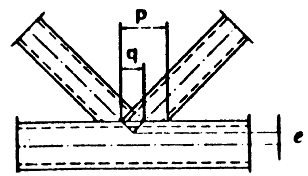
[a] (t ⩽ 12.5 mm)

Table 9.8.6 — Hollow sections — sheet 2 of 2([a])

Detail category	Constructional details	Description	Requirement
71	 ⑤	<u>Welded attachments</u> 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	 ⑥	— <u>Welded splices</u> 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	⑦ Rectangular hollow sections, butt welded end-to-end with an intermediate plate.	
40	 ⑧	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	⑨ Rectangular hollow sections, fillet welded end-to-end with an intermediate plate.	

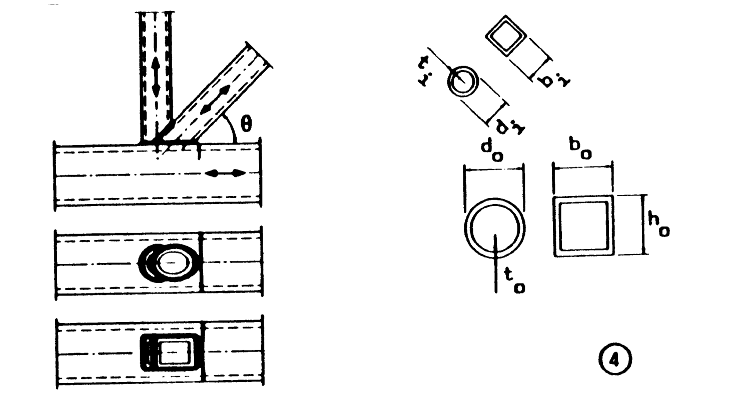
[a] (t ⩽ 12.5 mm)

Table 9.8.7 — Lattice girder joints — sheet 1 of 2 (m = 5)

Detail category	Constructional details		Description	Requirement
90	$\frac{t_o}{t_i} \geqslant 2.0$		<u>Joints with gap</u> (*) ① Circular hollow sections, K and N joints.	
45	$\frac{t_o}{t_i} = 1.0$			
71	$\frac{t_o}{t_i} \geqslant 2.0$		② Rectangular hollow sections, K and N joints.	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
36	$\frac{t_o}{t_i} = 1.0$			
71	$\frac{t_o}{t_i} \geqslant 1.4$		<u>Joints with overlap</u> (*) ③ K-joint.	③ and ④ — Overlap between 30 % and 100 %.  $\frac{q}{p} \cdot 100$: overlap e: negative eccentricity
56	$\frac{t_o}{t_i} = 1.0$			

— 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.

Table 9.8.7 — Lattice girder joints — sheet 2 of 2 (m = 5)

Detail category	Constructional details		Description	Requirement
71	$\frac{t_o}{t_i} \geqslant 1.4$		Joints with overlap (*) 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 $f_{y\,T}$, reduces with thickness and may be obtained from:

$$f_{y\,T} = f_{yo} - 0{,}25\ (t/t_1)\ (f_{yo}/235) \qquad \text{(C.1)}$$

where f_{yo} is the base value of $f_{y\,T}$ (in N/mm²)

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/mm²)	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:

$$K_{1C} = [\gamma_c \alpha]^{0,55} f_y \ell t^{0,5}/1,226 \qquad (C.2)$$

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:

$$T_{min} = 1,4\, T_{cv} + 25 + \beta + (83 - 0,08\, f_y \ell)\, [k_d]^{0,17} \qquad (C.3)$$

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:

$$\eta_1 = K_c/(K_c + K_{11} + K_{12}) \qquad (E.1)$$

$$\eta_2 = K_c/(K_c + K_{21} + K_{22}) \qquad (E.2)$$

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.

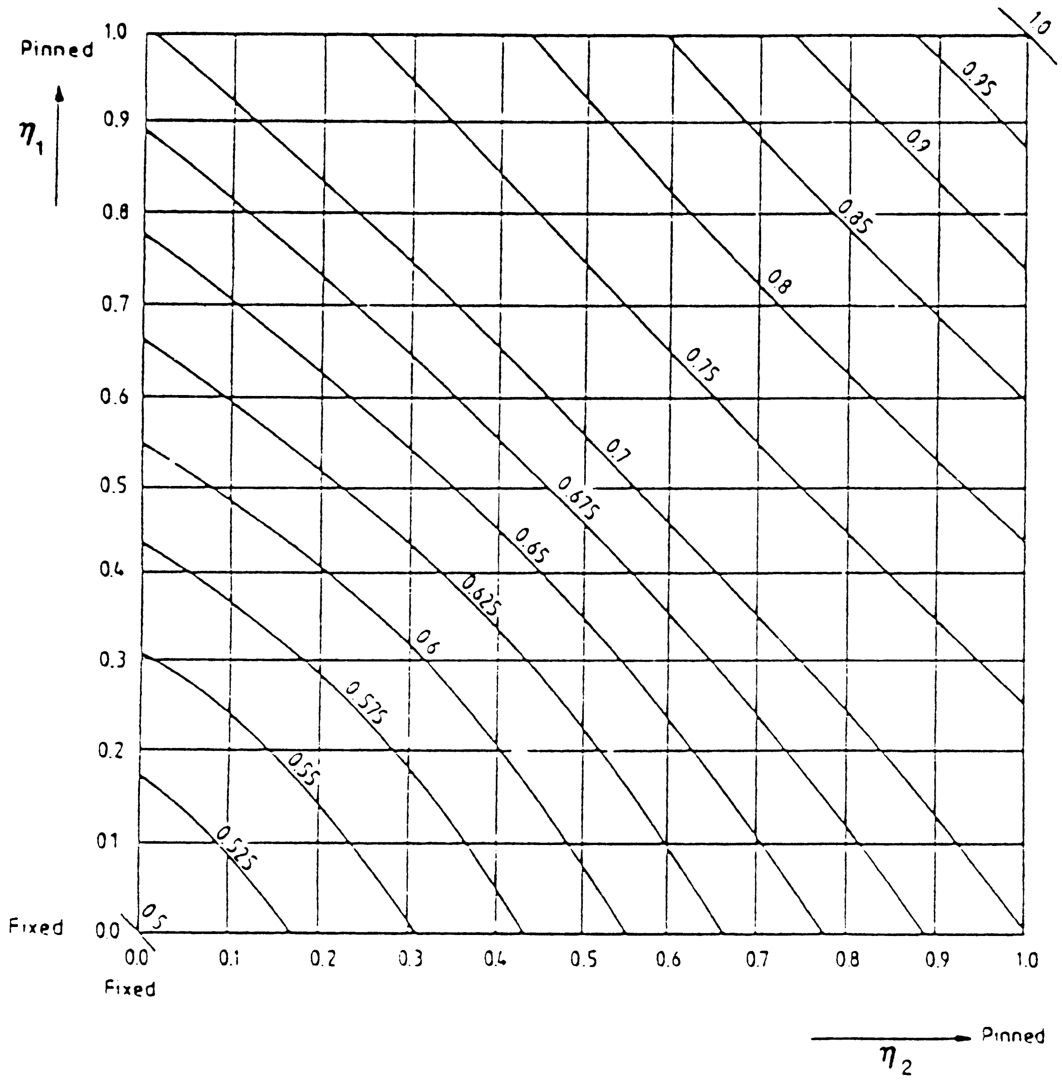


Figure E.2.1 — Buckling length ratio ℓ/L for a column in a non-sway mode

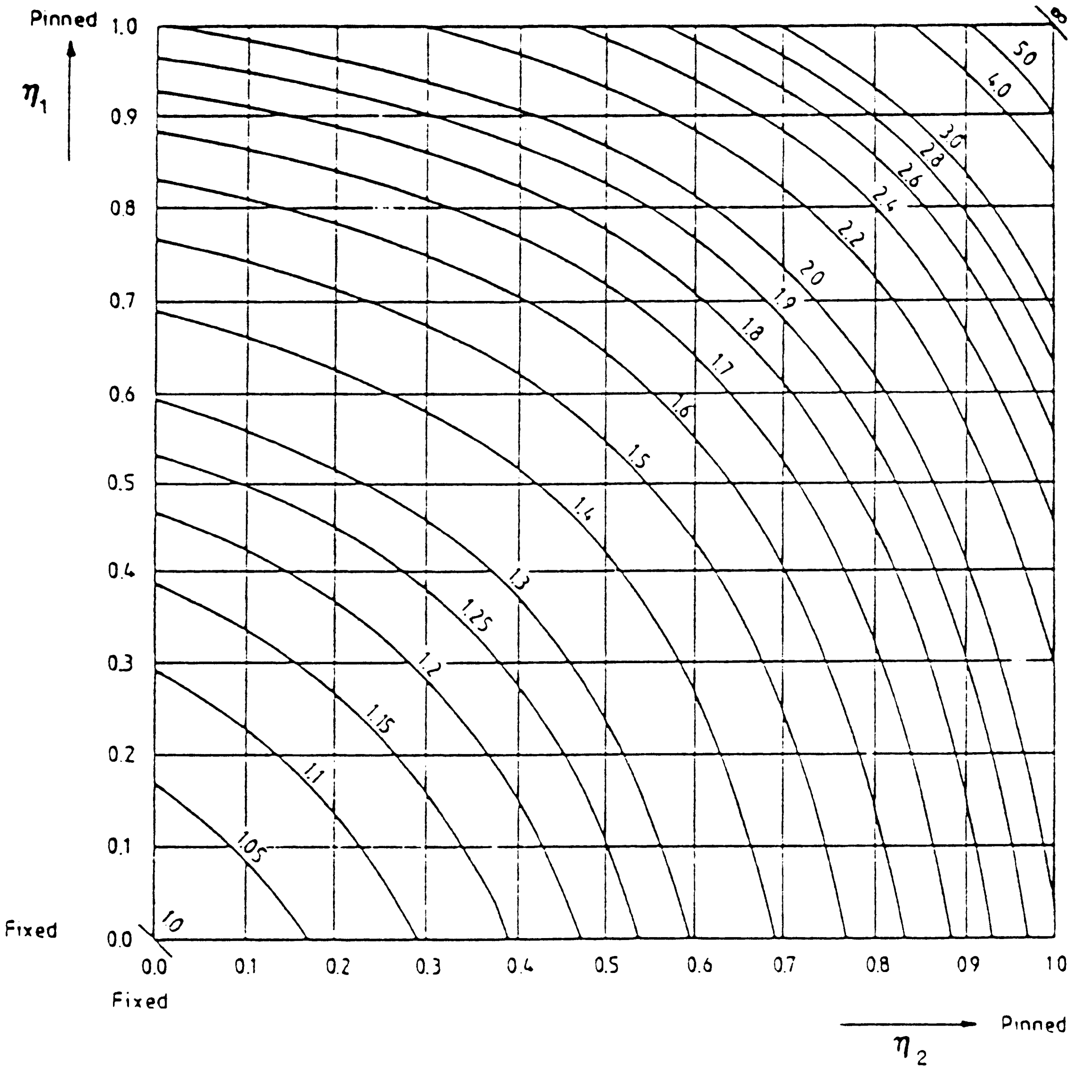


Figure E.2.2 — Buckling length ratio ℓ/L for a column in a sway mode

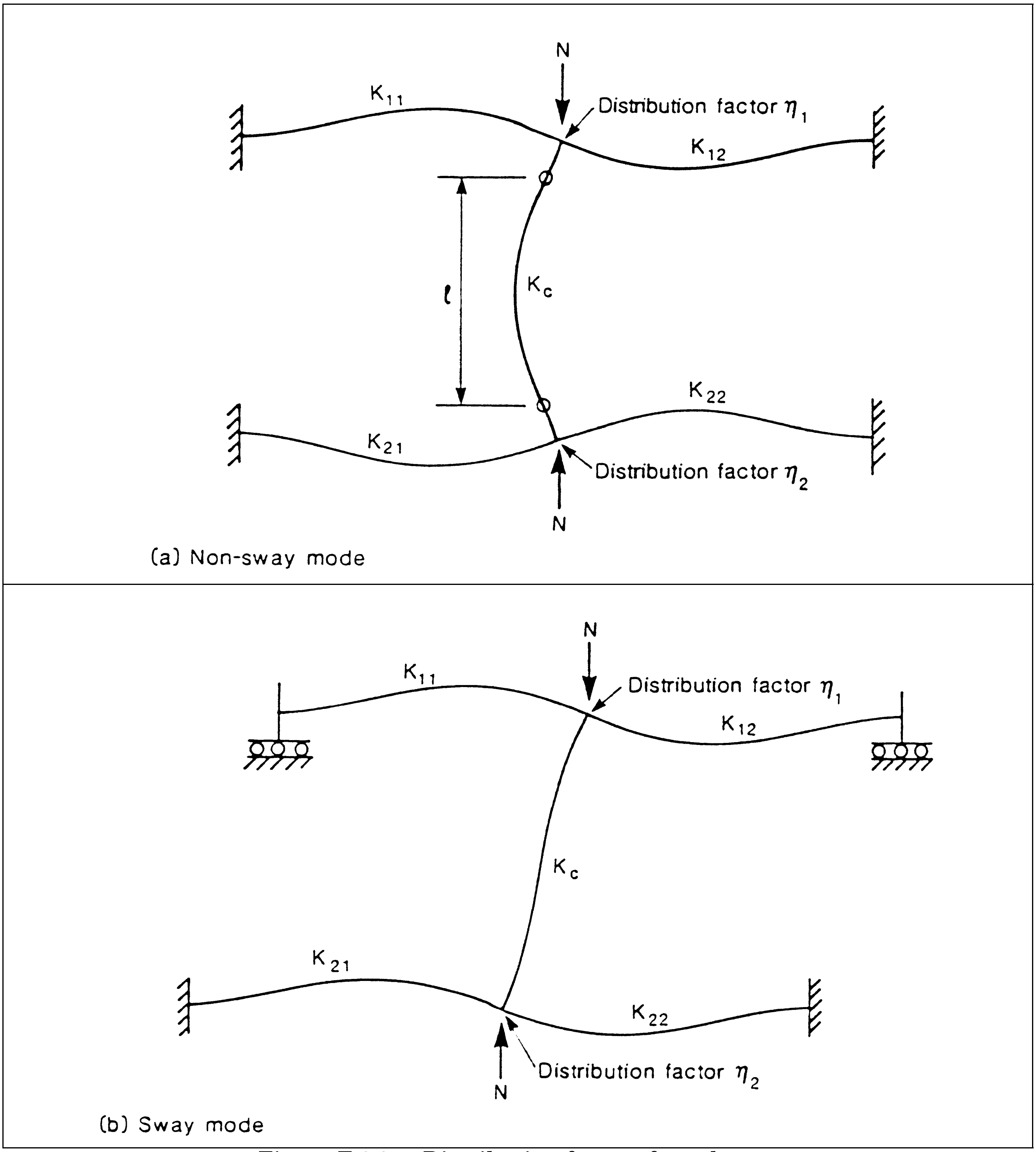


Figure E.2.3 — Distribution factors for columns

(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

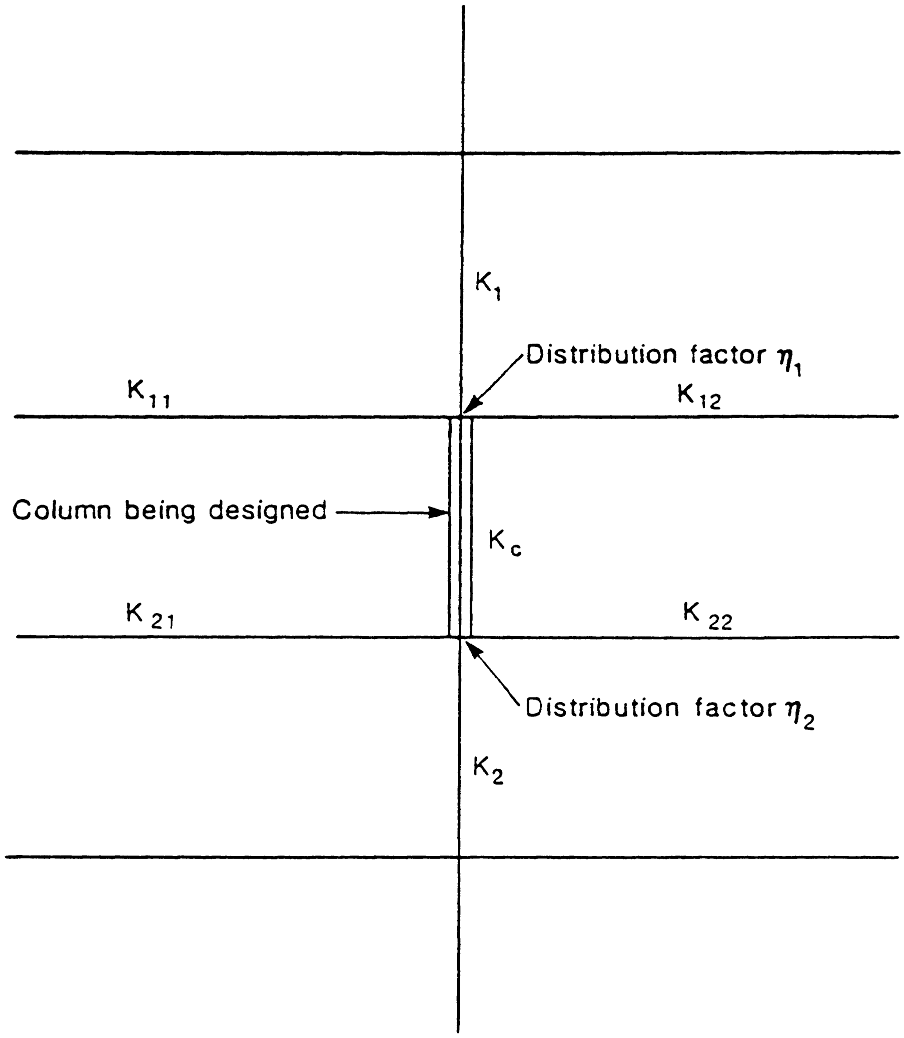
Loading conditions for the beam	Non-sway mode	Sway mode
Beams directly supporting concrete floor slabs	1,0 I/L	1,0 I/L
Other beams with direct loads	0,75 I/L	1,0 I/L
Beams with end moments only	0,5 I/L	1,5 I/L

(8) *Where, for the same load case, the design moment in any of the beams exceeds $W_{e\ell}\, f_y/\gamma_{M0}$, 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 $G = \dfrac{E}{2(1+v)}$

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:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \left\{ \left[\left(\frac{k}{k_w} \right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} + [C_2 z_g - C_3 z_j]^2 \right]^{0,5} - [C_2 z_g - C_3 z_j] \right\} \qquad \text{(F.2)}$$

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:*

$$C_1 = 1,88 - 1,40\,\psi + 0,52\psi^2 \text{ but } C_1 \leq 2,70 \qquad \text{(F.3)}$$

(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:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \left\{ \left[\left(\frac{k}{k_w} \right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} + [C_2 z_g]^2 \right]^{0,5} - C_2 z_g \right\} \qquad \text{(F.4)}$$

(2) For end-moment loading $C_2 = 0$ and for transverse loads applied at the shear centre $z_g = 0$. For these cases:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \left[\left(\frac{k}{k_w} \right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} \right]^{0,5} \qquad \text{(F.5)}$$

(3) When $k = k_w = 1,0$ (no end fixity):

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \left[\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z} \right]^{0,5} \quad \text{(F.6)}$$

Table F.1.1 — Values of factors C_1, C_2 and C_3 corresponding to values of factor k: End moment loading

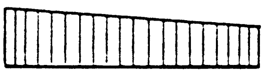
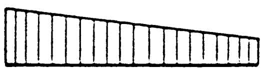
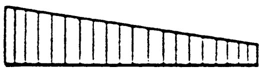
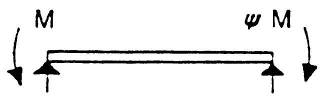
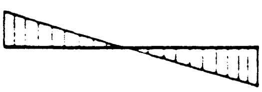
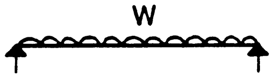
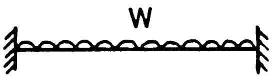
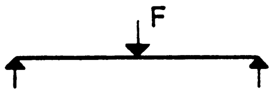
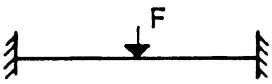
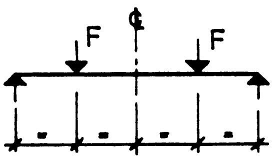
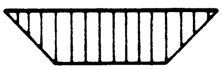
Loading and support conditions	Bending moment diagram	Value of k	Values of factors		
			C_1	C_2	C_3
	$\psi = +1$ 	1,0 0,7 0,5	1,000 1,000 1,000	—	1,000 1,113 1,144
	$\psi = +3/4$ 	1,0 0,7 0,5	1,141 1,270 1,305	—	0,998 1,565 2,283
	$\psi = +1/2$ 	1,0 0,7 0,5	1,323 1,473 1,514	—	0,992 1,556 2,271
	$\psi = +1/4$ 	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

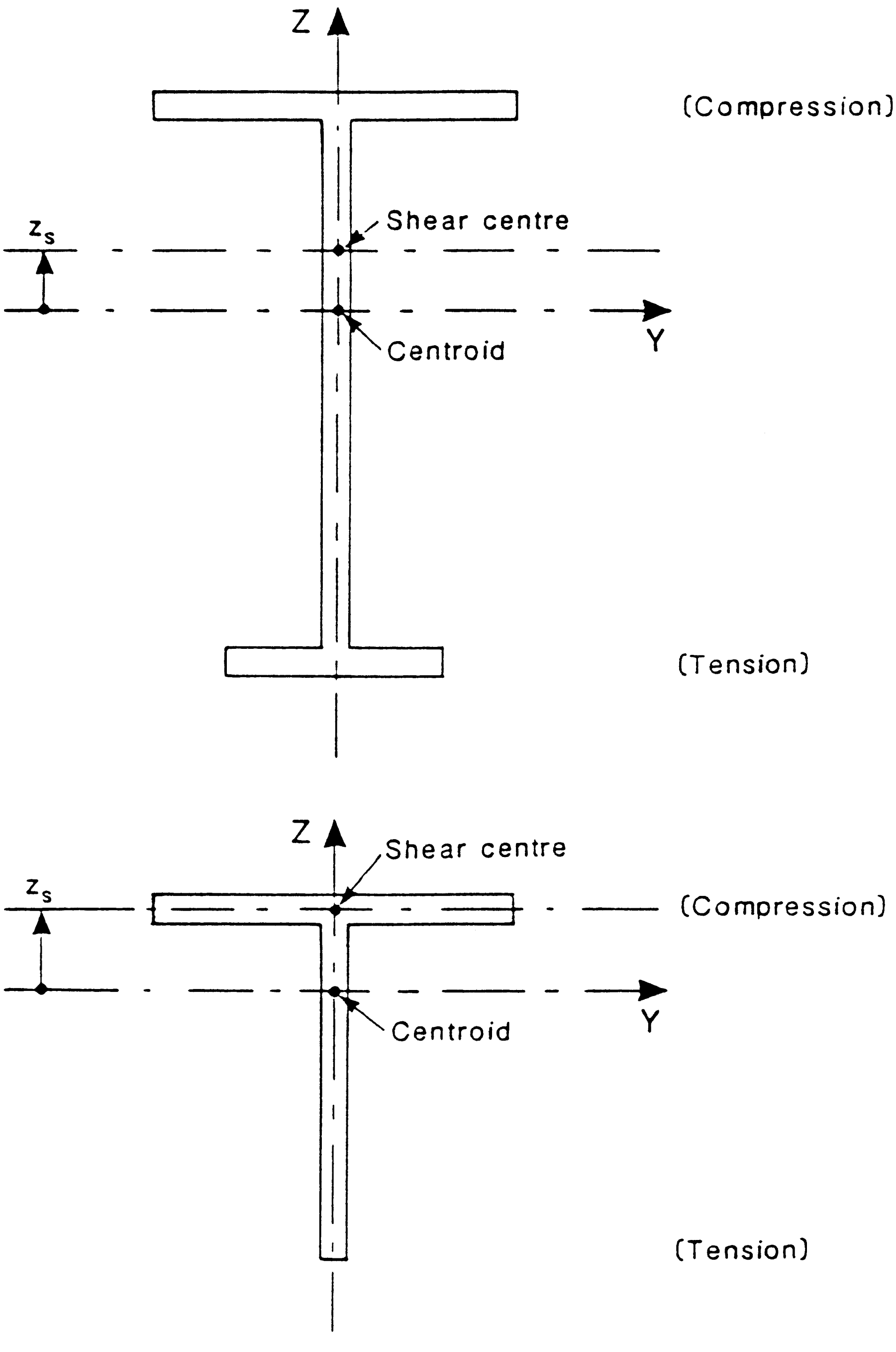


Figure F.1.1 — Sign convention for determining z_j

F.1.4 *Beams with uniform monosymmetric cross-sections with unequal flanges*

(1) For an I-section with unequal flanges:

$$I_w = \beta_f\,(1 - \beta_f)\,I_z\,h_s^2 \qquad \text{(F.7)}$$

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 $\beta_f > 0{,}5$:

$$z_j = 0{,}8\,(2\beta_f\ \Pi\ 1)\,h_s/2 \qquad \text{(F.8)}$$

when $\beta_f < 0{,}5$:

$$z_j = 1{,}0\,(2\beta_f\ \Pi\ 1)\,h_s/2 \qquad \text{(F.9)}$$

for sections with a lipped compression flange:

$$z_j = 0{,}8\,(2\beta_f\ \Pi\ 1)\,(1 + h_L/h)\,h_s/2 \text{ when } \beta_f > 0{,}5 \qquad \text{(F.10)}$$

$$z_j = 1{,}0\,(2\beta_f\ \Pi\ 1)\,(1 + h_L/h)\,h_s/2 \text{ when } \beta_f < 0{,}5 \qquad \text{(F.11)}$$

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:

$$\bar{\lambda}_{LT} = [\lambda_{LT} / \lambda_1]\,[\beta_w]^{0,5} \qquad \text{(F.12)}$$

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:

$$\lambda_{LT} = [\pi^2\,EW_{p\ell.y}/M_{cr}]^{0,5} \qquad \text{(F.13)}$$

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:

$$\lambda_{LT} = \frac{L\left[\dfrac{W_{p\ell.y}^2}{I_z\,I_w}\right]^{0,25}}{(C_1)^{0,5}\left[1 + \dfrac{L^2 G I_t}{\pi^2 E I_w}\right]^{0,25}} \qquad \text{(F.14)}$$

which can also be written:

$$\lambda_{LT} = \frac{L/i_{LT}}{(C_1)^{0,5}\left[1 + \dfrac{(L/a_{LT})^2}{25{,}66}\right]^{0,25}} \qquad \text{(F.15)}$$

where $a_{LT} = (I_w/I_t)^{0.5}$

(2) For a plain I or H section (without lips):

$I_w = I_z\, h_s^2/4$ (F.16)

where $h_s = h - t_f$

(3) *For a doubly symmetric cross-section, the value of i_{LT} is given by:*

$i_{LT} = [I_z\, I_w / W_{p\ell.y}^2]^{0,25}$ *(F.17)*

or with a slight approximation by:

$i_{LT} = [I_z / (A - 0,5\, t_w\, h_s)]^{0,5}$ *(F.18)*

(4) *For rolled I or H sections conforming with Reference Standard 2, the following conservative approximations can be used:*

$$\lambda_{LT} = \frac{L/i_{LT}}{(C_1)^{0,5}\left[1 + \frac{1}{20}\left[\frac{L/i_{LT}}{h/t_f}\right]^2\right]^{0,25}} \qquad \text{(F.19)}$$

or

$$\lambda_{LT} = \frac{0,9\; L/i_z}{(C_1)^{0,5}\left[1 + \frac{1}{20}\left[\frac{L/i_z}{h/t_f}\right]^2\right]^{0,25}} \qquad \text{(F.20)}$$

(5) *For any plain I or H section with equal flanges, the following approximation is conservative:*

$$\lambda_{LT} = \frac{L/i_z}{(C_1)^{0,5}\left[1 + \frac{1}{20}\left[\frac{L/i_z}{h/t_f}\right]^2\right]^{0,25}} \qquad \text{(F.21)}$$

(6) *Cases with $k < 1,0$ and/or $k_w < 1,0$ can be included by using:*

$$\lambda_{LT} = \frac{kL\left[\frac{W_{p\ell.y}^2}{I_z\, I_w}\right]^{0,25}}{(C_1)^{0,5}\left[\left[\frac{k}{k_w}\right]^2 + \frac{(kL)^2\, G\, I_t}{\pi^2\, E\, I_w}\right]^{0,25}} \qquad \text{(F.22)}$$

or

$$\lambda_{LT} = \frac{kL/i_{LT}}{(C_1)^{0,5}\left[\left[\frac{k}{k_w}\right]^2 + \frac{(kL/a_{LT})^2}{25,66}\right]^{0,25}} \qquad \text{(F.23)}$$

or for standard rolled I or H sections:

$$\lambda_{LT} = \frac{kL/i_{LT}}{(C_1)^{0,5}\left[\left[\frac{k}{k_w}\right]^2 + \frac{1}{20}\left[\frac{kL/i_{LT}}{h/t_f}\right]^2\right]^{0,25}} \qquad \text{(F.24)}$$

or $$\lambda_{LT} = \frac{0{,}9\ kL/i_z}{(C_1)^{0,5}\left[\left[\frac{k}{k_w}\right]^2 + \frac{1}{20}\left[\frac{kL/i_z}{h/t_f}\right]^2\right]^{0,25}} \qquad \text{(F.25)}$$

or for any plain I or H section with equal flanges:

$$\lambda_{LT} = \frac{kL/i_z}{(C_1)^{0,5}\left[\left[\frac{k}{k_w}\right]^2 + \frac{1}{20}\left[\frac{kL/i_z}{h/t_f}\right]^2\right]^{0,25}} \qquad \text{(F.26)}$$

(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:*

$$\lambda_{LT} = \frac{kL\left[\frac{W_{pl.y}^2}{I_z\, I_w}\right]^{0,25}}{(C_1)^{0,5}\left\{\left[\left[\frac{k}{k_w}\right]^2 + \frac{(kL)^2\, GI_t}{\pi^2\, EI_w} + (C_2\, z_g)^2\,\frac{I_z}{I_w}\right]^{0,5} - C_2\, z_g\left[\frac{I_z}{I_w}\right]^{0,5}\right\}^{0,5}} \qquad \text{(F.27)}$$

or alternatively:

$$\lambda_{LT} = \frac{kL/i_{LT}}{(C_1)^{0,5}\left\{\left[\left[\frac{k}{k_w}\right]^2 + \frac{(kL/a_{LT})^2}{25{,}66} + \left[\frac{2C_2\, z_g}{h_s}\right]^2\right]^{0,5} - \frac{2C_2\, z_g}{h_s}\right\}^{0,5}} \qquad \text{(F.28)}$$

or for standard rolled I or H sections:

$$\lambda_{LT} = \frac{kL/i_{LT}}{(C_1)^{0,5}\left\{\left[\left[\frac{k}{k_w}\right]^2 + \frac{1}{20}\left[\frac{kL/i_{LT}}{h/t_f}\right]^2 + \left[\frac{2C_2\, z_g}{h_s}\right]^2\right]^{0,5} - \frac{2C_2\, z_g}{h_s}\right\}^{0,5}} \qquad \text{(F.29)}$$

or alternatively:

$$\lambda_{LT} = \frac{0{,}9\ kL/i_z}{(C_1)^{0,5}\left\{\left[\left[\frac{k}{k_w}\right]^2 + \frac{1}{20}\left[\frac{kL/i_z}{h/t_f}\right]^2 + \left[\frac{2C_2\, z_g}{h_s}\right]^2\right]^{0,5} - \frac{2C_2\, z_g}{h_s}\right\}^{0,5}} \qquad \text{(F.30)}$$

or for any plain I or H section with equal flanges:

$$\lambda_{LT} = \frac{kL/i_z}{(C_1)^{0,5}\left\{\left[\left[\frac{k}{k_w}\right]^2 + \frac{1}{20}\left[\frac{kL/i_z}{h/t_f}\right]^2 + \left[\frac{2C_2\, z_g}{h_s}\right]^2\right]^{0,5} - \frac{2C_2\, z_g}{h_s}\right\}^{0,5}} \tag{F.31}$$

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.*

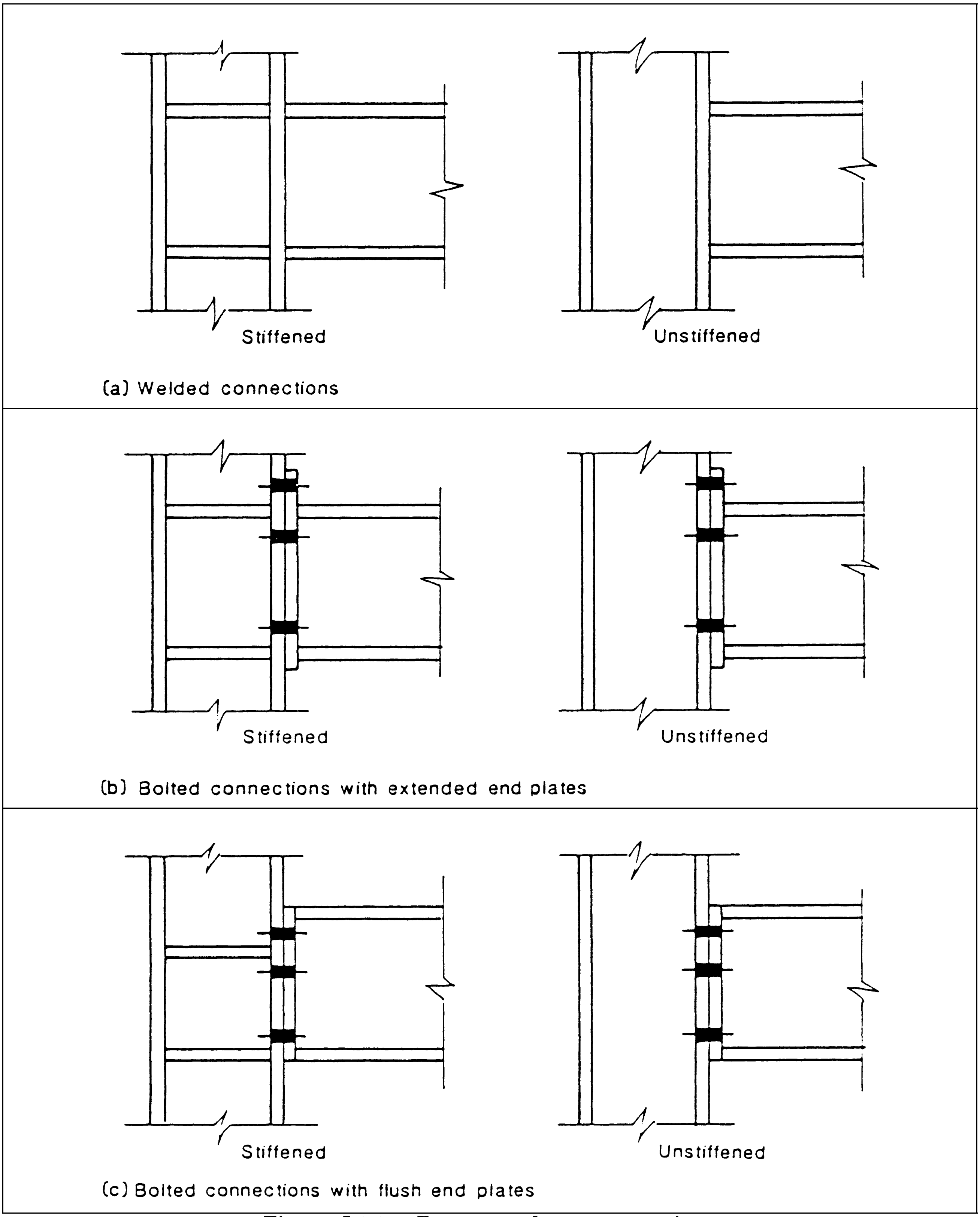


Figure J.1.1 — Beam-to-column connections

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:*

$$a \geq t_s/\sqrt{2} \qquad (J.3)$$

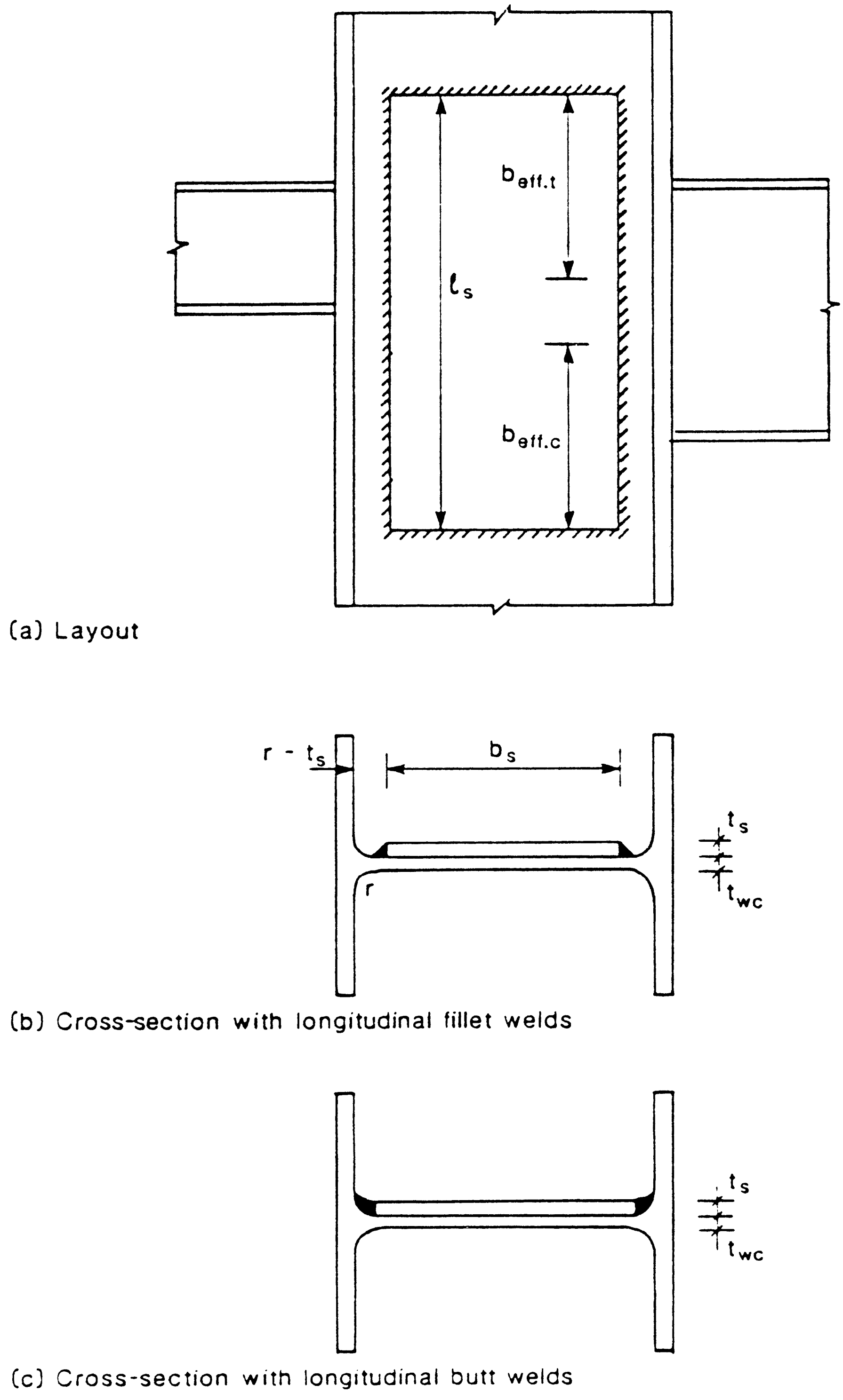


Figure J.2.1 — Supplementary web plate

(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:*

$$F_{t.Rd} = [f_{yb}\, t_{fb}\, (t_{wc} + 2r_c) + 7\, f_{yc}\, t_{fc}^2]/\gamma_{M0} \qquad (J.4)$$

but $$F_{t.Rd} \leq f_{yb}\, t_{fb}\, [t_{wc} + 2r_c + 7\, t_{fc}]/\gamma_{M0} \qquad (J.5)$$

- *for a welded I or H section column:*

$$F_{t.Rd} = [f_{yb}\, t_{fb}\, (t_{wc} + 2\sqrt{2}\, a_c) + 7\, f_{yc}\, t_{fc}^2]/\gamma_{M0} \qquad (J.6)$$

but $$F_{t.Rd} \leq f_{yb}\, t_{fb}\, [t_{wc} + 2\sqrt{2}\, a_c + 7\, t_{fc}]/\gamma_{M0} \qquad (J.7)$$

(2) *If the design resistance $F_{t.Rd}$ obtained from (1) does not satisfy the following condition, the joint should be stiffened:*

$$F_{t.Rd} \geq 0{,}7\, f_{yb}\, t_{fb}\, b_{fb}/\gamma_{M0} \qquad (J.8)$$

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 $f_{yb}\, t_{fb}\, b_{fb}/\gamma_{M0}$.*

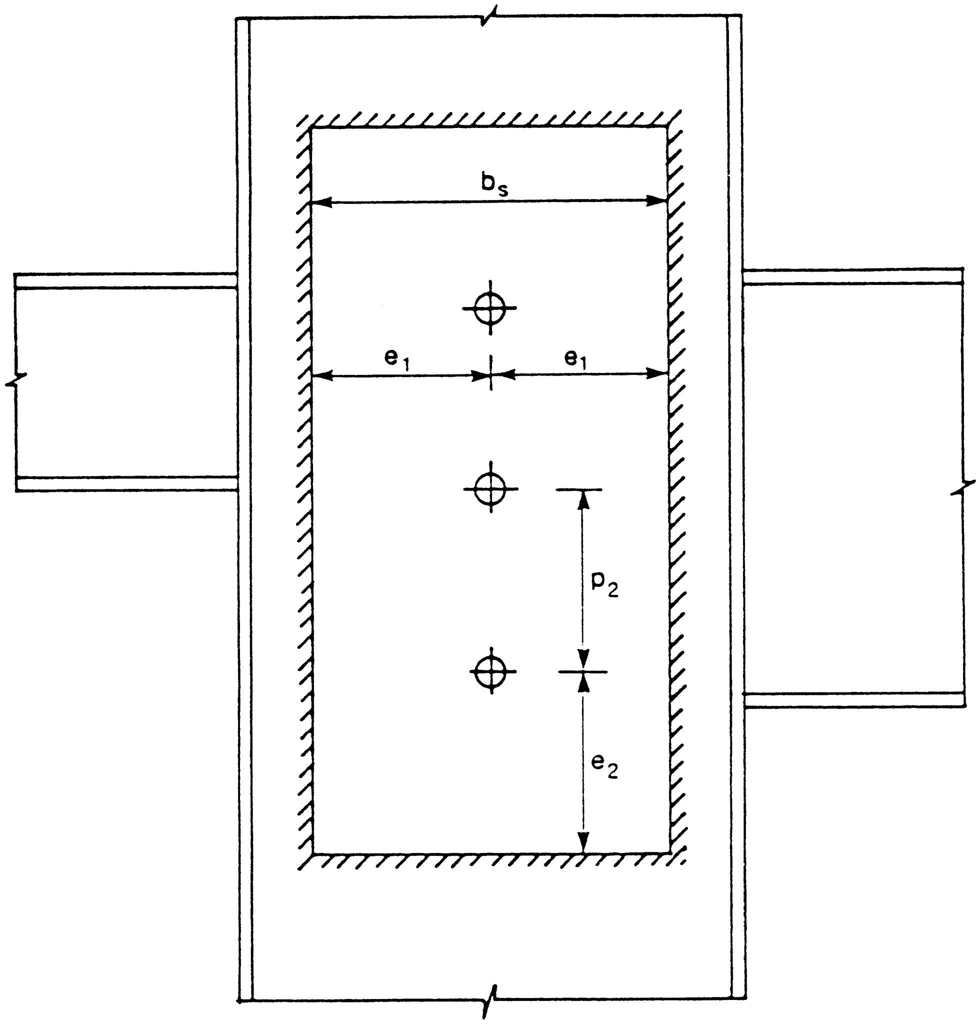


Figure J.2.2 — Spacing of plug welds or bolts for supplementary web plate

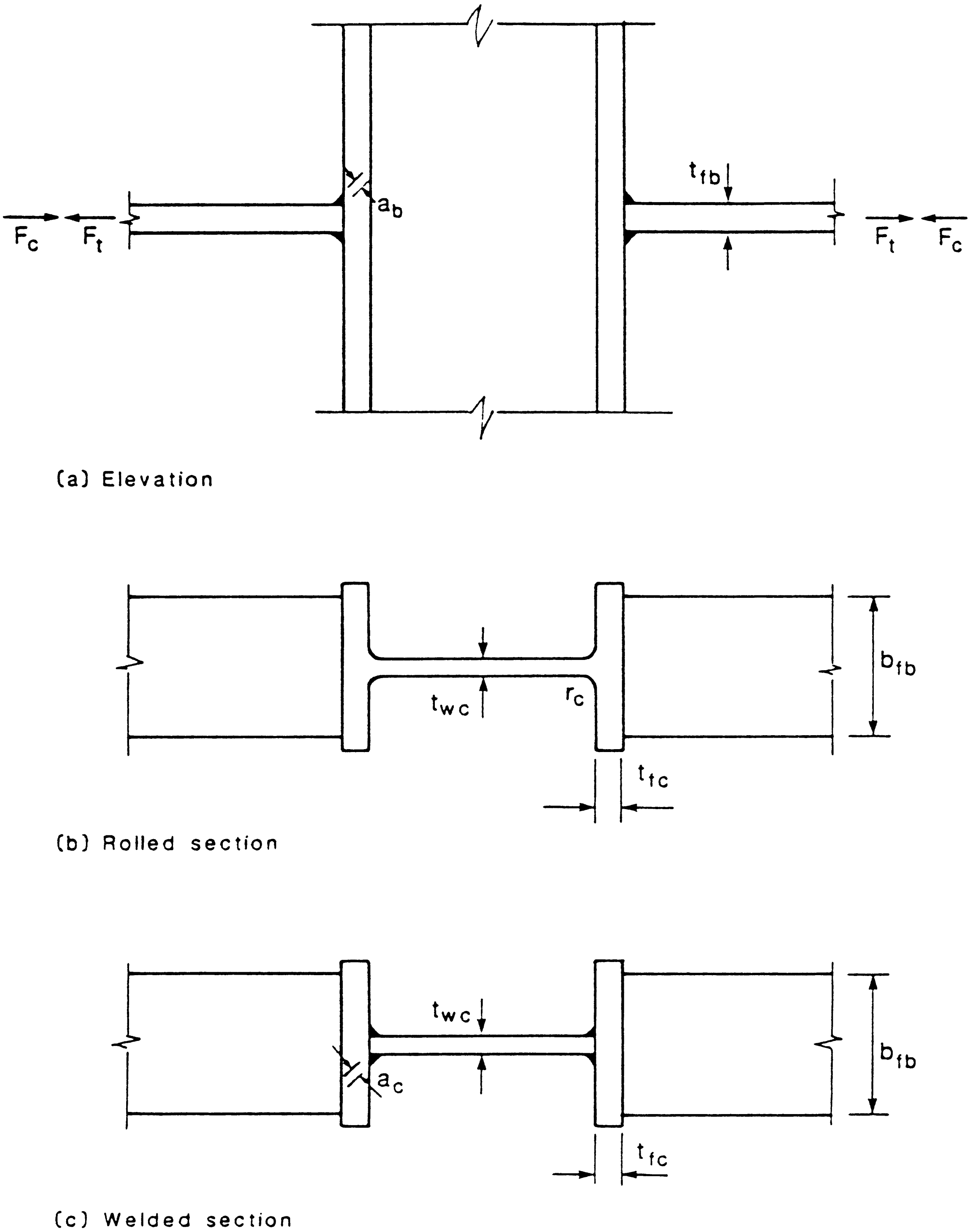


Figure J.2.3 — Transverse force on an unstiffened column

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****.*

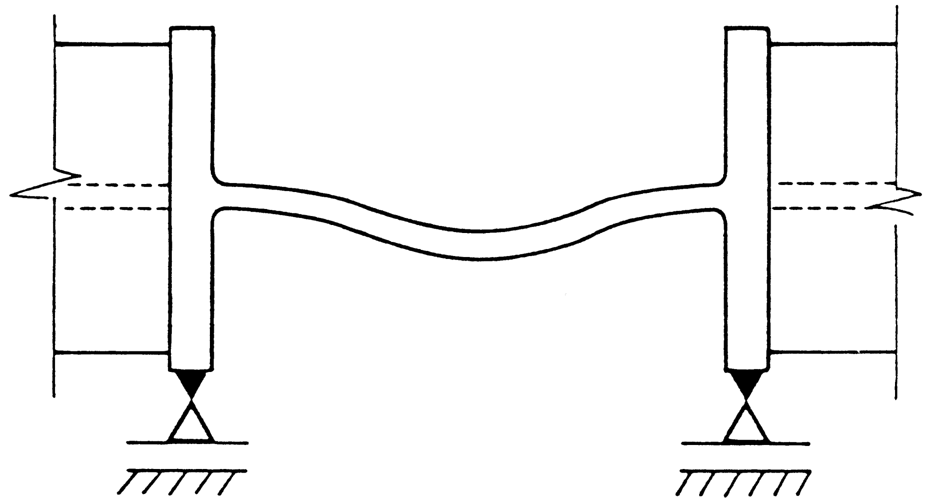
(4) *The sway mode shown in Figure J.2.4(b) should normally be prevented by constructional restraints.*

(5) *An unstiffened column web may be strengthened by adding a supplementary web plate conforming with* ***J.2.2****.*

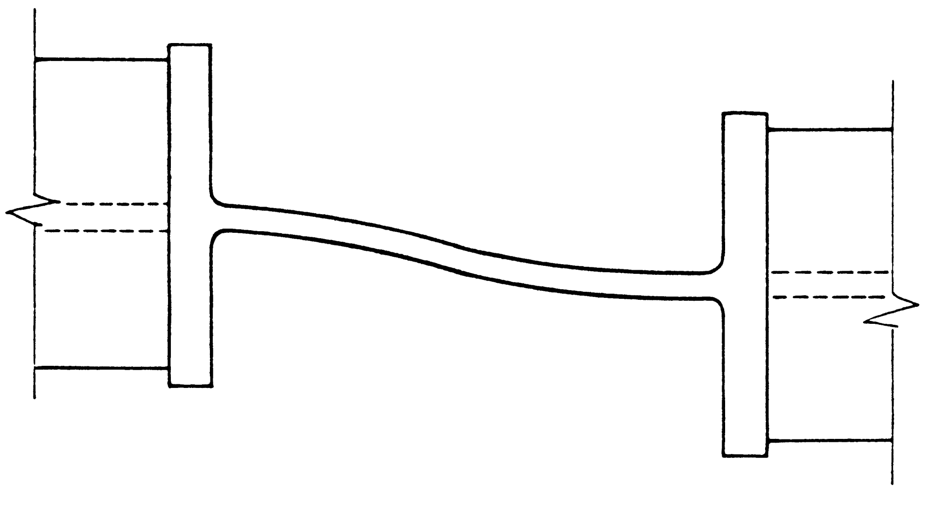
(6) *In calculating the design crushing resistance of a column web with a supplementary web plate, the effective thickness of the web may be taken as* $1{,}5t_{wc}$ *when one supplementary web plate is added or* $2{,}0t_{wc}$ *when supplementary web plates are added both sides of the web.*

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)***.



(a) Non-sway mode



(b) Sway mode

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.*

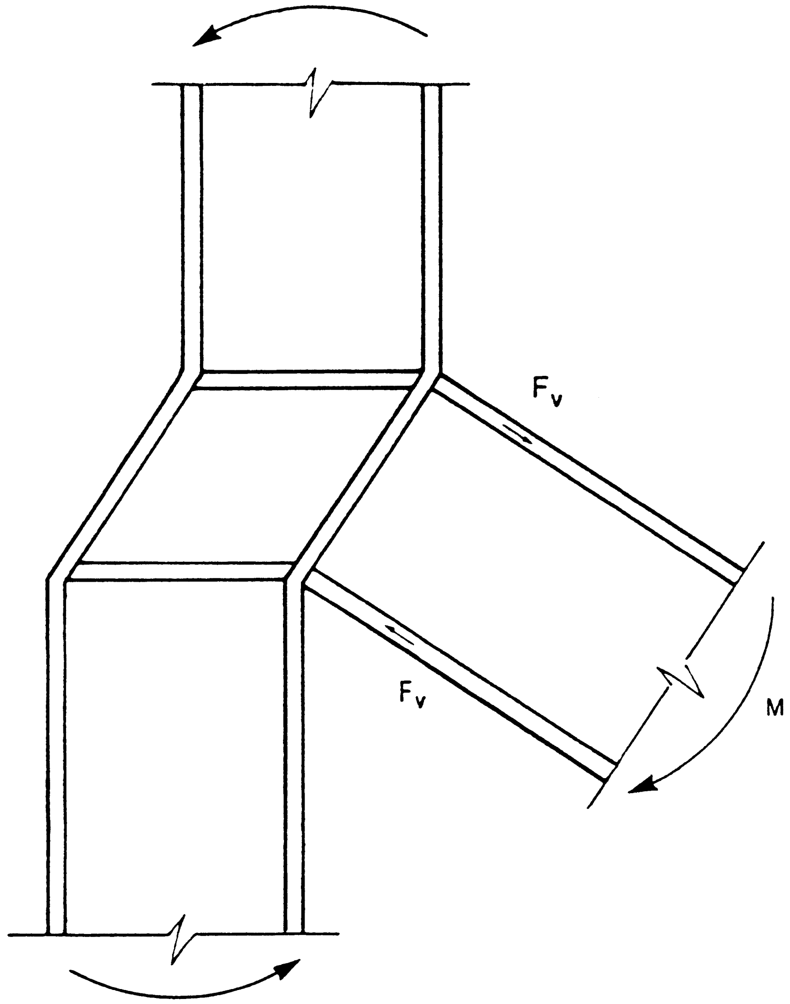


Figure J.2.5 — Unstiffened column web panel subject to a shear force

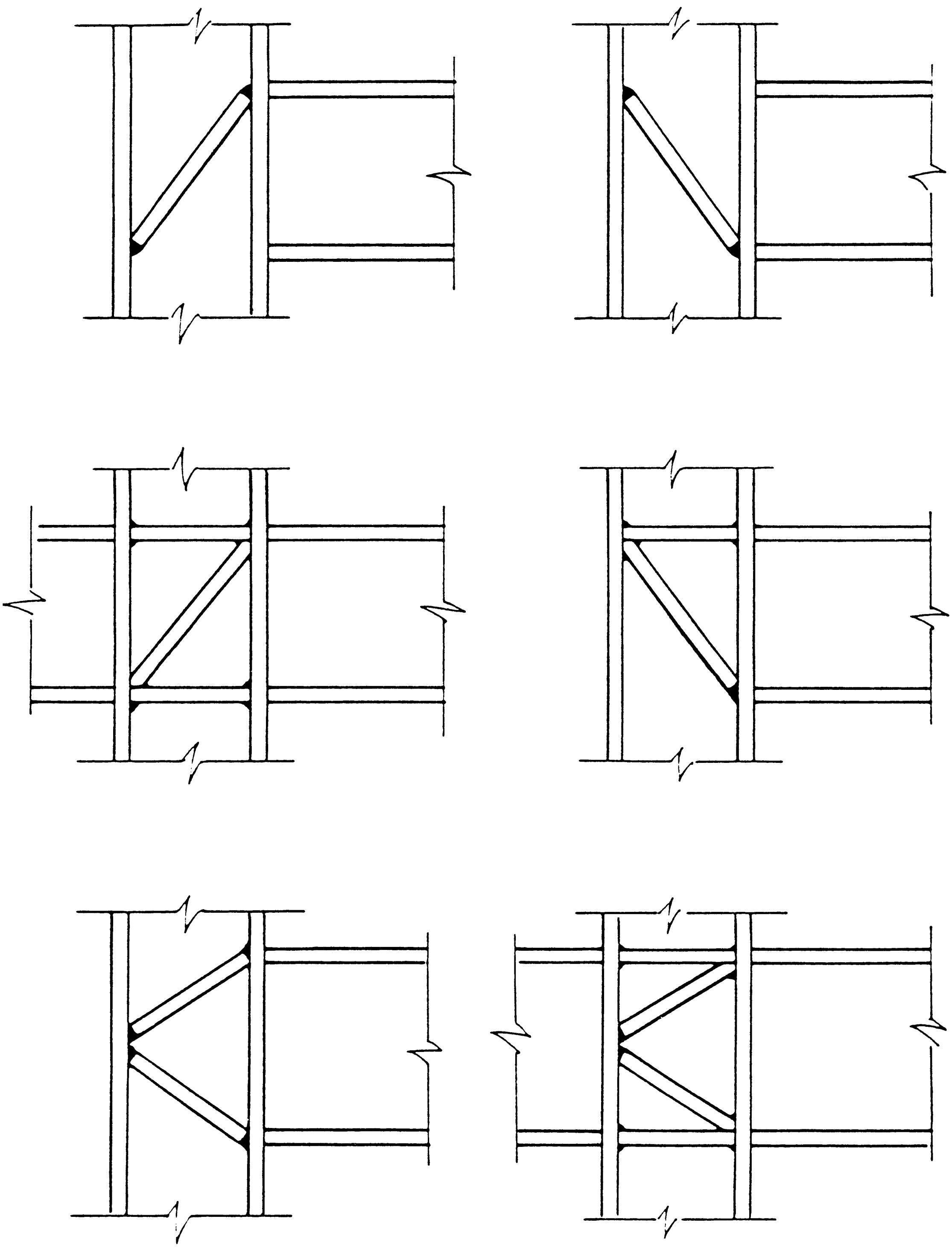


Figure J.2.6 — Column web panels with diagonal stiffeners

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 $2\ \pi m$, 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 $2\ \pi m$.

(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 $\lambda = n/m$

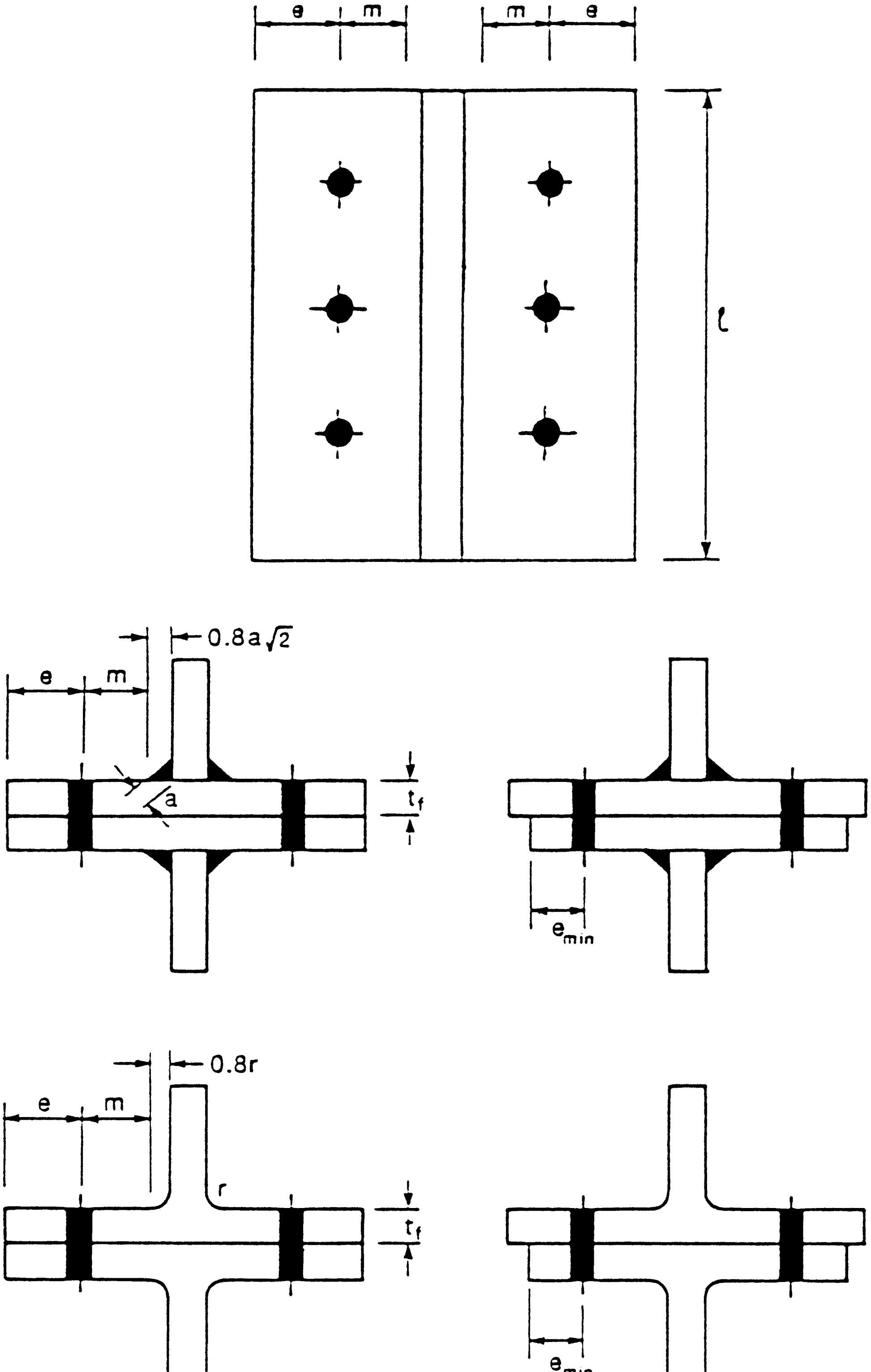


Figure J.3.1 — Dimensions of a T-stub

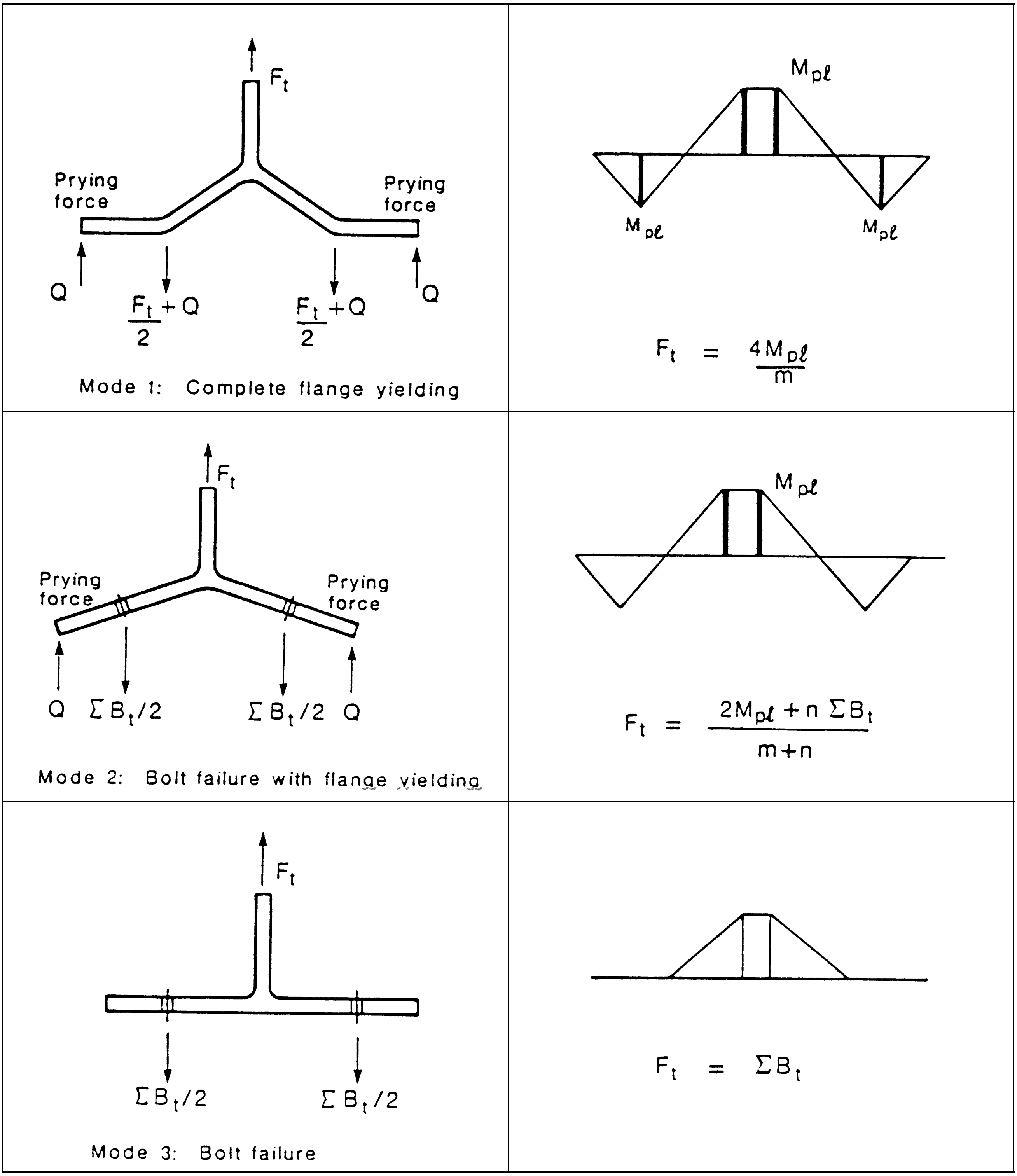


Figure J.3.2 — Failure modes of a T-stub flange

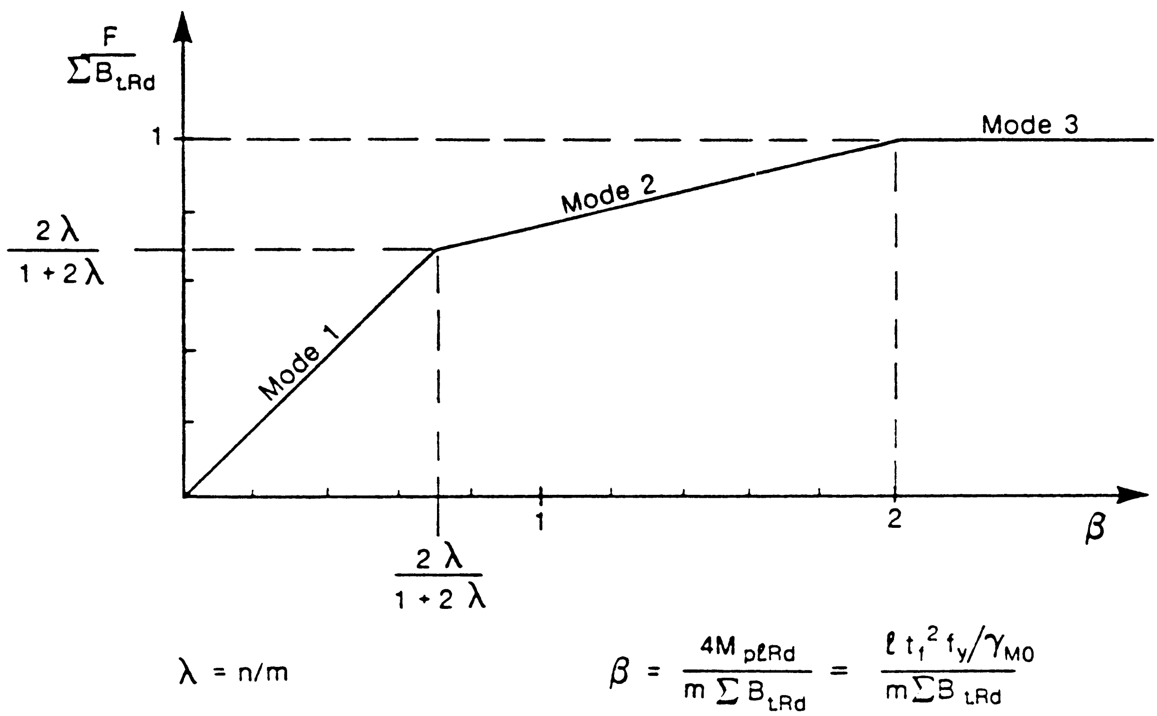


Figure J.3.3 — Effect of connection geometry on failure mode of a T-stub flange

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:*

$$F_{t.Rd} = \frac{4M_{pl.Rd} + 2M_{bp.Rd}}{m} \qquad (J.33)$$

Mode 2: *Bolt failure with yielding of flange only:*

$$F_{t.Rd} = \frac{2M_{pl.Rd} + n\Sigma B_{t.Rd}}{m + n} \qquad (J.23)$$

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:*

$$M_{bp.Rd} = 0,25\, \ell_{eff}\, t_{bp}^2\, f_{y.bp}/\gamma_{M0} \qquad (J.25)$$

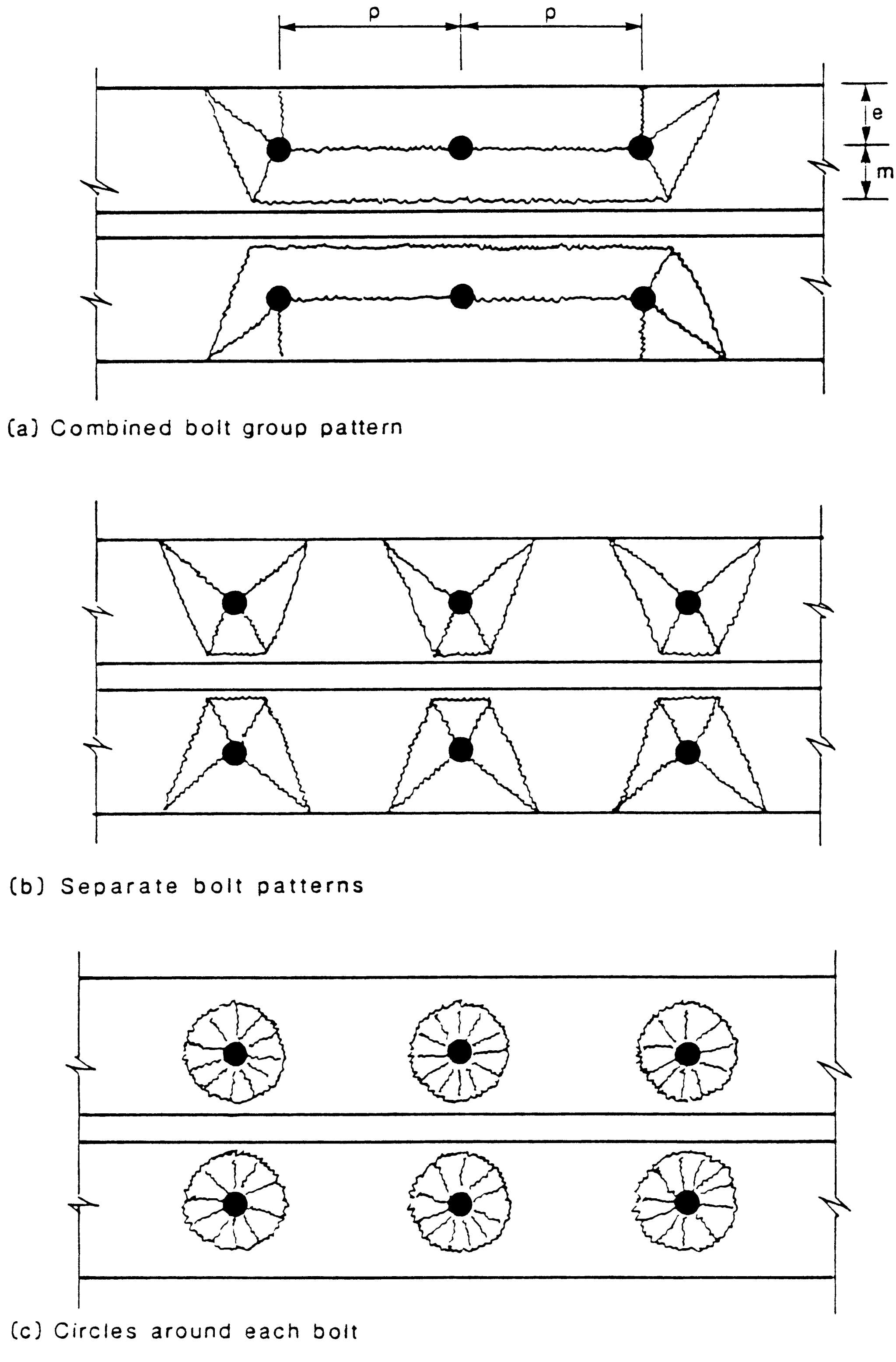


Figure J.3.4 — Yield line patterns for an unstiffened flange

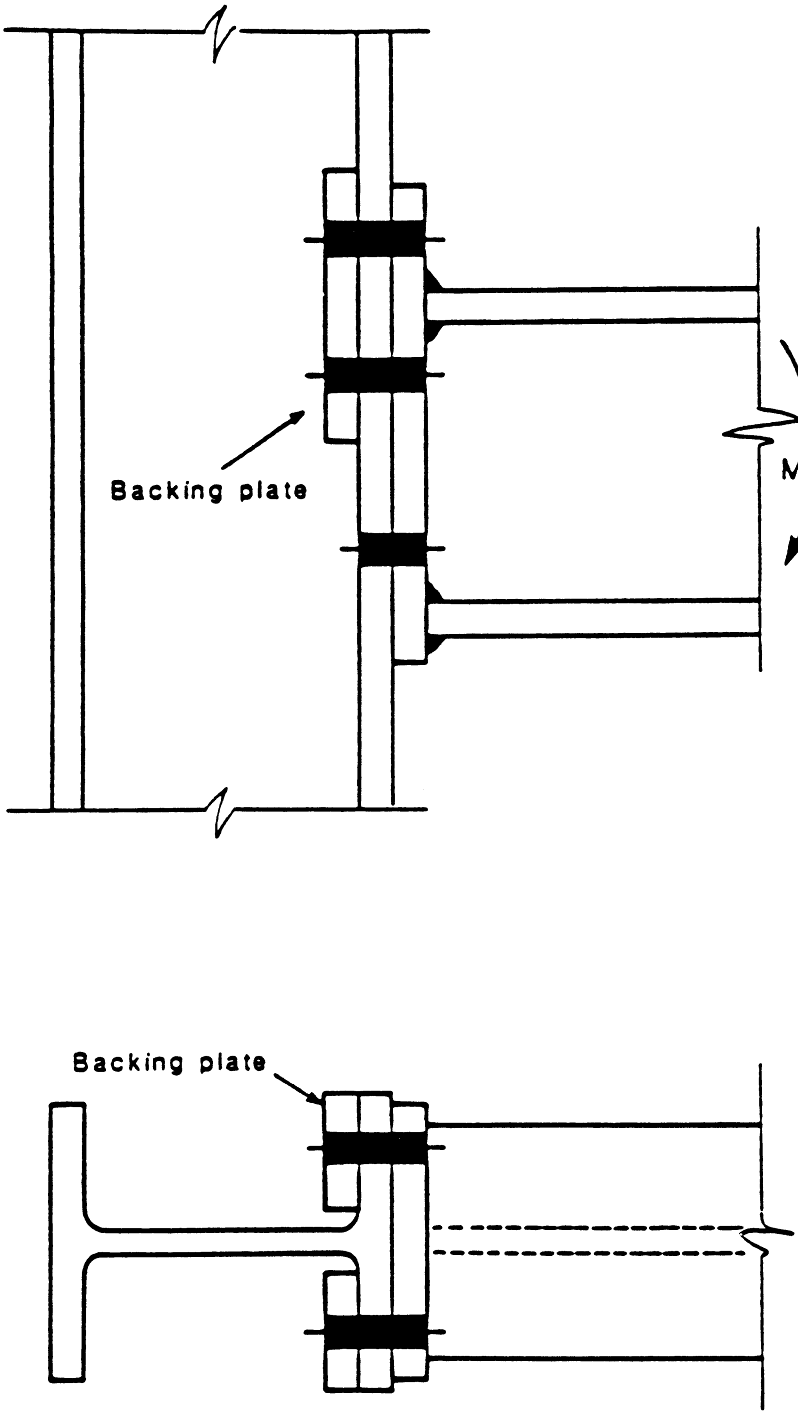


Figure J.3.5 — Column flange with backing plates in the tension zone

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/mm² 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).*

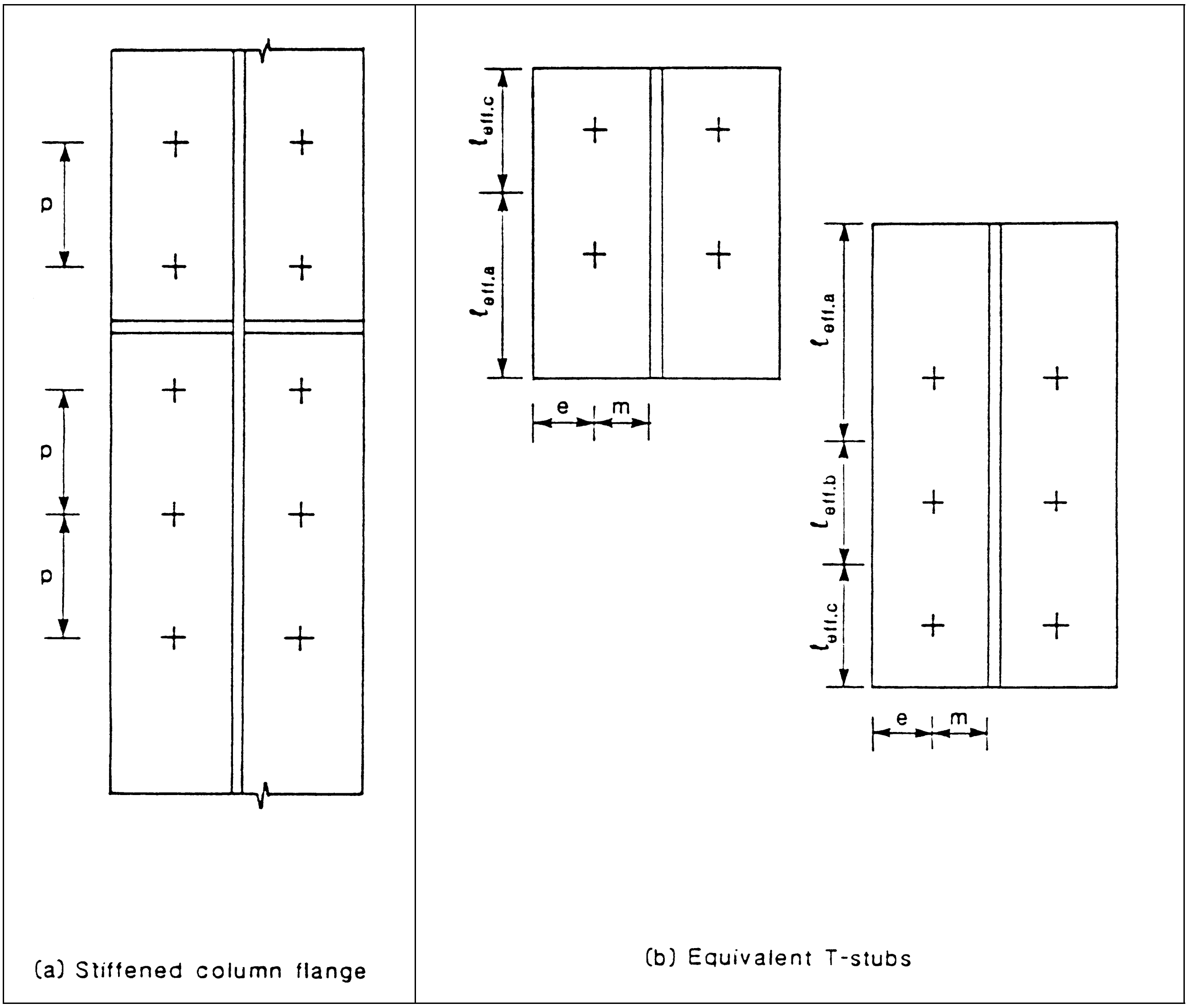
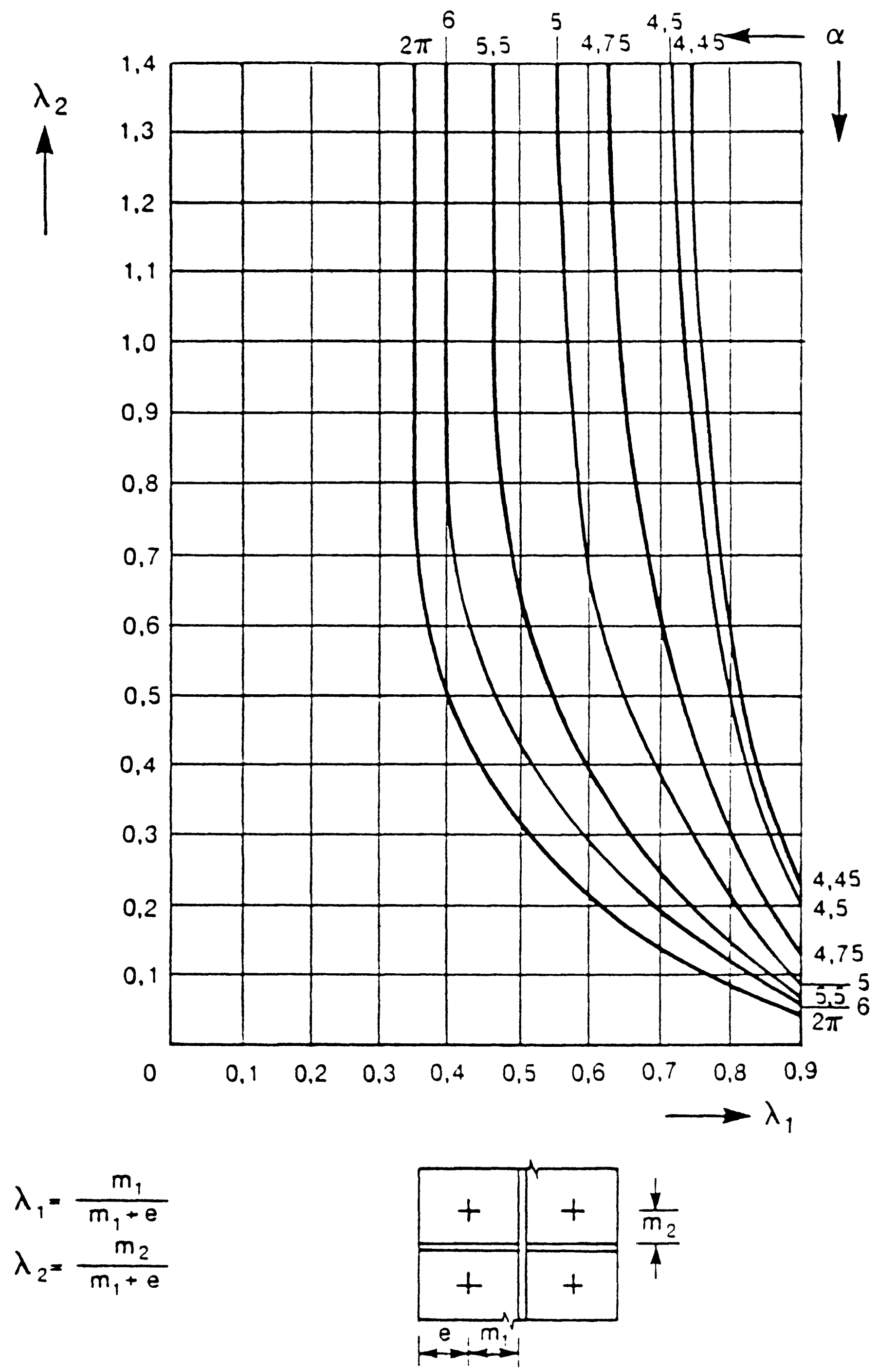


Figure J.3.6 — Effective lengths of equivalent T-stub flanges representing a stiffened column flange



$$\lambda_1 = \frac{m_1}{m_1 + e}$$

$$\lambda_2 = \frac{m_2}{m_1 + e}$$

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:***

$$\ell_{eff.a} = 0{,}5b_p \quad (J.40)$$

$$\ell_{eff.a} = 0{,}5w + 2m_x + 0{,}625e_x \quad (J.41)$$

$$\ell_{eff.a} = 4m_x + 1{,}25e_x \quad (J.42)$$

$$\ell_{eff.a} = 2\pi m \quad (J.43)$$

b) ***for first row of bolts below tension flange of beam:***

$$\ell_{eff.b} = \alpha m \quad (J.44)$$

$$\ell_{eff.b} = 2\pi m \quad (J.31)$$

c) ***for other inner bolts:***

$$\ell_{eff.c} = p \quad (J.45)$$

$$\ell_{eff.c} = 4m + 1{,}25e \quad (J.38)$$

$$\ell_{eff.c} = 2\pi m \quad (J.39)$$

d) ***for other end bolts:***

$$\ell_{eff.d} = 0{,}5p + 2m + 0{,}625e \quad (J.46)$$

$$\ell_{eff.d} = 4m + 1{,}25e \quad (J.47)$$

$$\ell_{eff.d} = 2\pi m \quad (J.48)$$

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.*

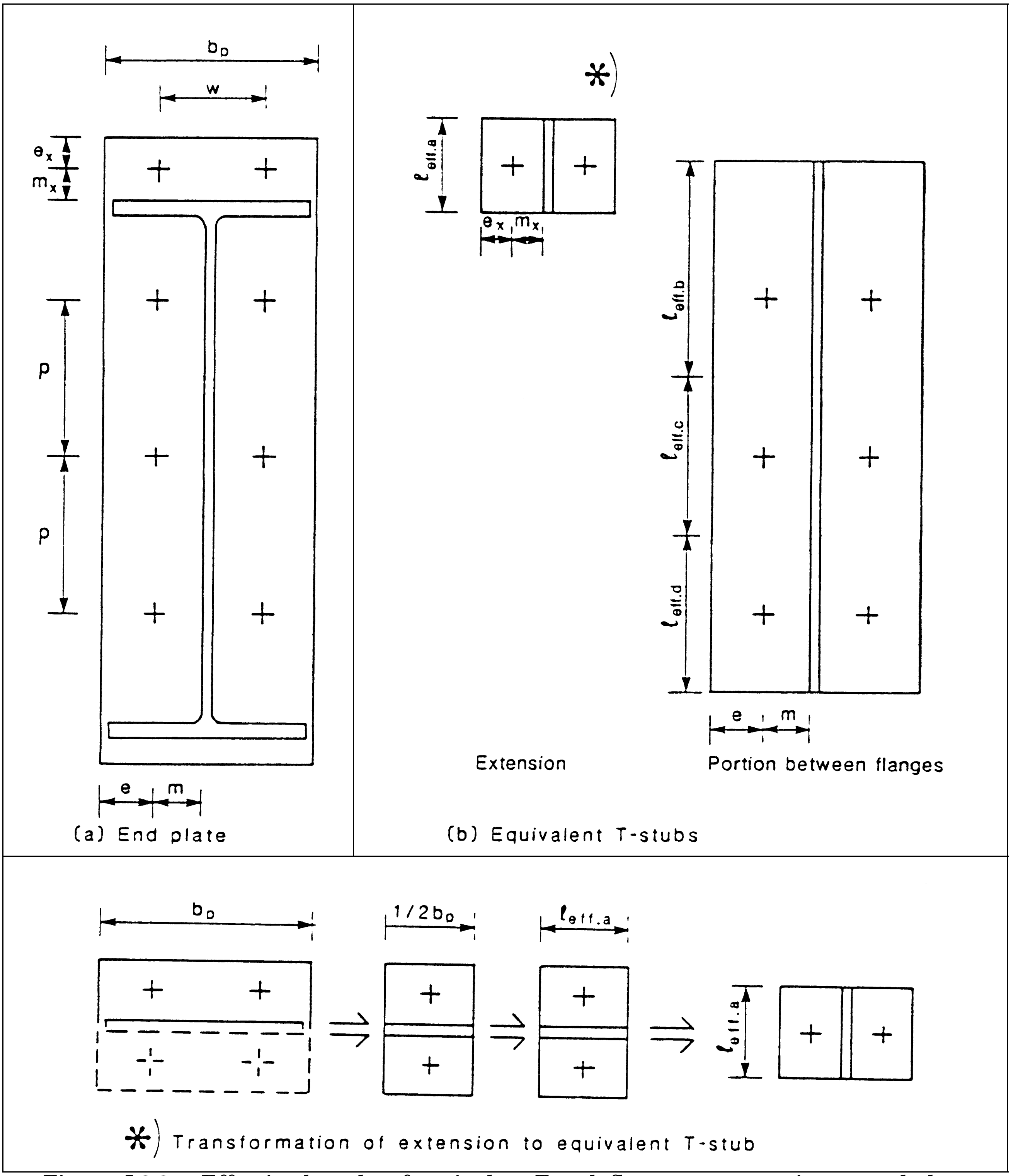


Figure J.3.8 — Effective lengths of equivalent T-stub flanges representing an end plate

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:*

$$F_{t.Rd} = f_{yc}\, t_{wc}\, b_{eff}/\gamma_{M0} \qquad (J.9)$$

(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 2πm, 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 $$F_{c.Rd} \le f_{Yc}\, t_{wc}\, b_{eff}/\gamma_{M0} \qquad (J.16)$$

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 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* ($M \leq M_{Rd}$)
- 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

$$k_6 \geq \frac{t_e^3}{4m^2 \, t_{wc}}$$

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:*

$$k_4 = \frac{t_{fc}^3}{12\lambda_2 \, m^2 \, t_{wc}} \quad \text{but} \quad k_4 \geq \frac{t_{fc}^3}{4m^2 \, t_{wc}}$$

(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:*

$$\beta \leq \frac{2\lambda}{1 + 2\lambda} \tag{J.54}$$

*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/mm²*.

(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 $(q/p) \times 100$ % as shown in Figure K.1(b).*

(4) *The symbols used in the tables in this Annex are defined in **K.9**.*

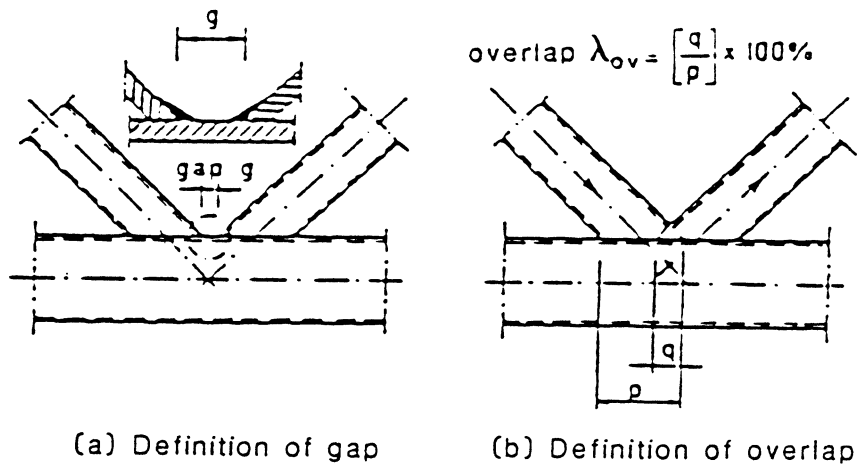


Figure K.1 — Gap and overlap of joints

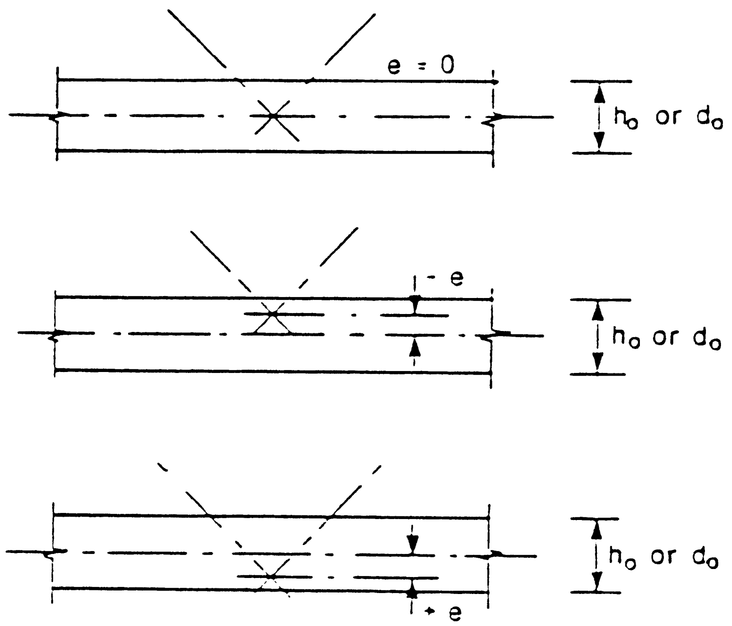


Figure K.2 — Eccentricity of joints

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:*

$$\alpha = \frac{1{,}1}{\gamma_{Mj}} \times \frac{\gamma_{Mw}}{1{,}25} \qquad \text{(K.3)}$$

(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

$0{,}2 \leq \dfrac{d_i}{d_o} \leq 1{,}0$
$5 \leq \dfrac{d_i}{2t_i} \leq 25$
$5 \leq \dfrac{d_o}{2t_o} \leq 25$ $5 \leq \dfrac{d_o}{2t_o} \leq 20$ 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.*

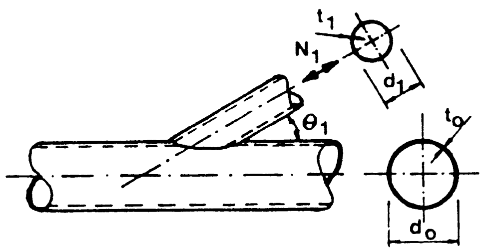
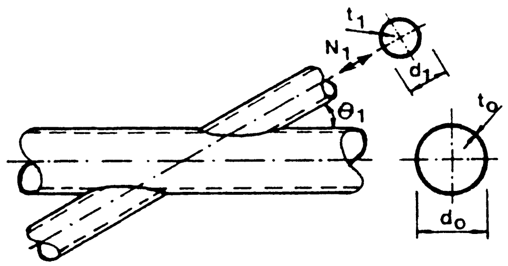
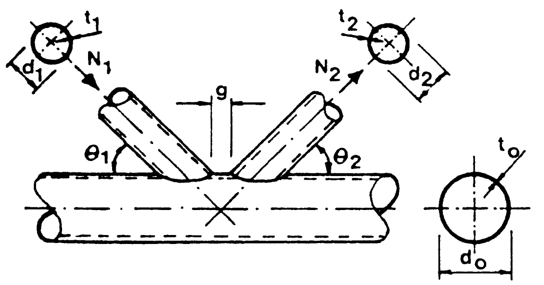
(2) *The design values of the internal axial forces in the brace members at the ultimate limit state should also not exceed the design resistances of the joints.*

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 $d_i \le d_o - 2t_o$	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 $k_p \le 1{,}0$	for $n_p \le 0$ (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)

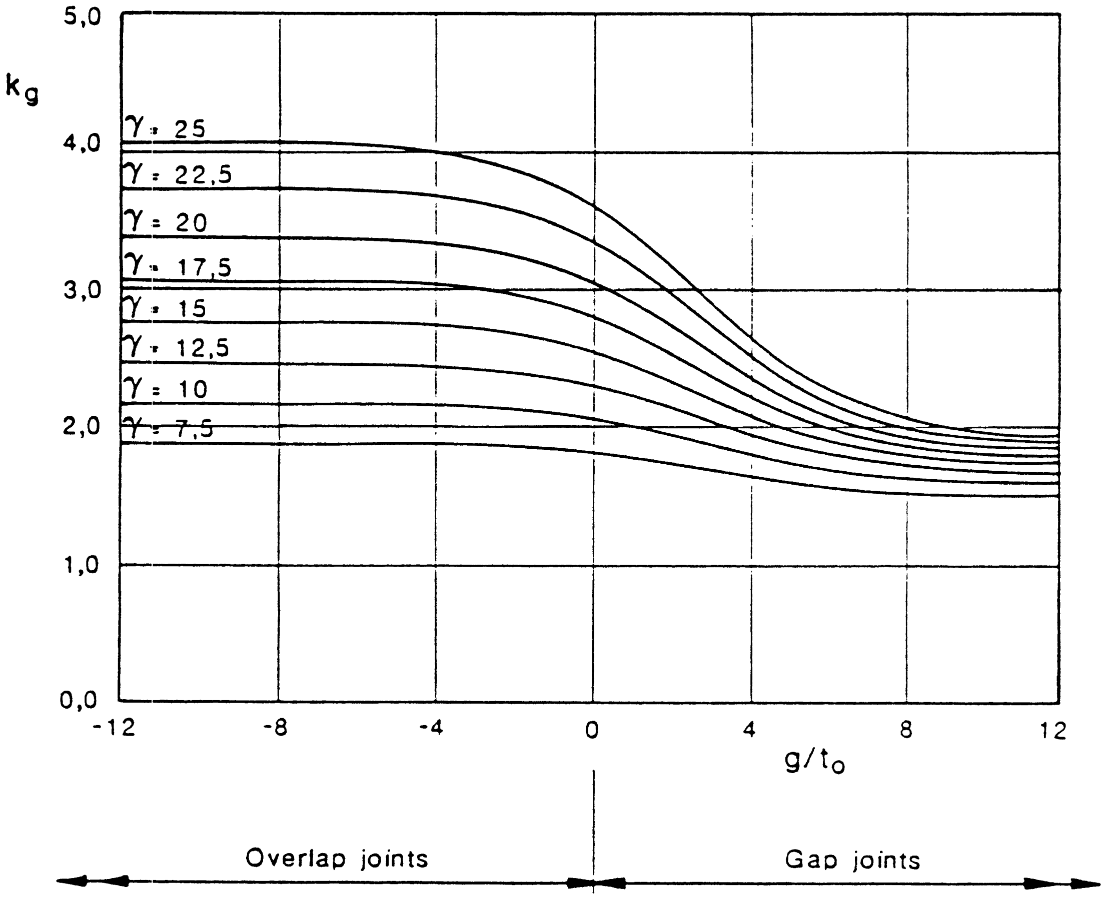


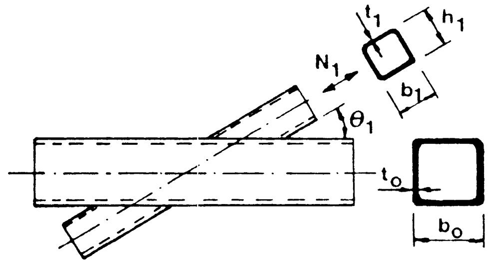
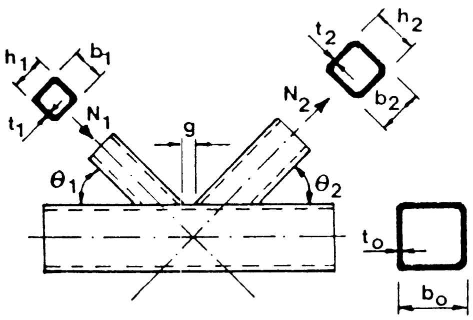
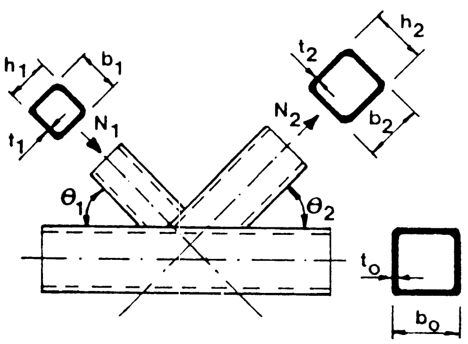
Figure K.3 — Values of factor k_g

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

<table>
<tr><th>Type of joint</th><th>Design resistance (i = 1 or 2, j = overlapped brace)</th></tr>
<tr><td>T, Y and X joints</td><td>Chord face yielding $\beta \leq 0,85$</td></tr>
<tr><td>
</td><td>$$N_{1.Rd} = \frac{f_{yo}\, t_o^2}{(1-\beta)\sin\theta_1}\left[\frac{2\beta}{\sin\theta_1} + 4\,(1-\beta)^{0,5}\right] k_n \left[\frac{1,1}{\gamma_{Mj}}\right]$$</td></tr>
<tr><td>K and N gap joints</td><td>Chord face yielding $\beta \leq 1,0$</td></tr>
<tr><td>
</td><td>$$N_{i.Rd} = \frac{8,9\, f_{yo}\, t_o^2}{\sin\theta_i}\left[\frac{b_1 + b_2}{2b_o}\right]\gamma^{0,5}\, k_n \left[\frac{1,1}{\gamma_{Mj}}\right]$$</td></tr>
<tr><td>K and N overlap joints[a]</td><td>Effective width $25\ \% \leq \lambda_{ov} < 50\ \%$</td></tr>
<tr><td rowspan="5">
</td><td>$$N_{i.Rd} = f_{yi}\, t_i \left[\frac{\lambda_{ov}}{50}(2h_i - 4t_i) + b_{eff} + b_{e.ov}\right]\left[\frac{1,1}{\gamma_{Mj}}\right]$$</td></tr>
<tr><td>Effective width $50\ \% \leq \lambda_{ov} \leq 80\ \%$</td></tr>
<tr><td>$$N_{i.Rd} = f_{yi}\, t_i\, [2h_i - 4t_i + b_{eff} + b_{e.ov}]\left[\frac{1,1}{\gamma_{Mj}}\right]$$</td></tr>
<tr><td>Effective width $\lambda_{ov} \geq 80\ \%$</td></tr>
<tr><td>$$N_{i.Rd} = f_{yi}\, t_i\, [2h_i - 4t_i + b_i + b_{e.ov}]\left[\frac{1,1}{\gamma_{Mj}}\right]$$</td></tr>
<tr><td>Circular braces</td><td>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.</td></tr>
<tr><td colspan="2">Functions</td></tr>
<tr><td>for $n \leq 0$ (tension): $k_n = 1,0$</td><td>for $n \geq 0$ (compression): $k_n = 1,3 - \frac{0,4n}{\beta}$ but $k_n \leq 1,0$</td></tr>
<tr><td>$b_{eff} = \frac{10}{b_o/t_o}\frac{f_{yo}\, t_o}{f_{yi}\, t_i}\, b_i$ but $b_{eff} \leq b_i$</td><td>$b_{e.ov} = \frac{10}{b_j/t_j}\frac{f_{yj}\, t_j}{f_{yi}\, t_i}\, b_i$ but $b_{e.ov} \leq b_i$</td></tr>
<tr><td colspan="2">[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.</td></tr>
</table>

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* $V_{Sd}/V_{p\ell.Rd} \leq 1,0$:

$$N_{o.Rd} = f_{yo} [A_o - A_v (2V_{Sd}/V_{p\ell.Rd} - 1)^2]/\gamma_{M0} \quad \text{(K.5)}$$

(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.*

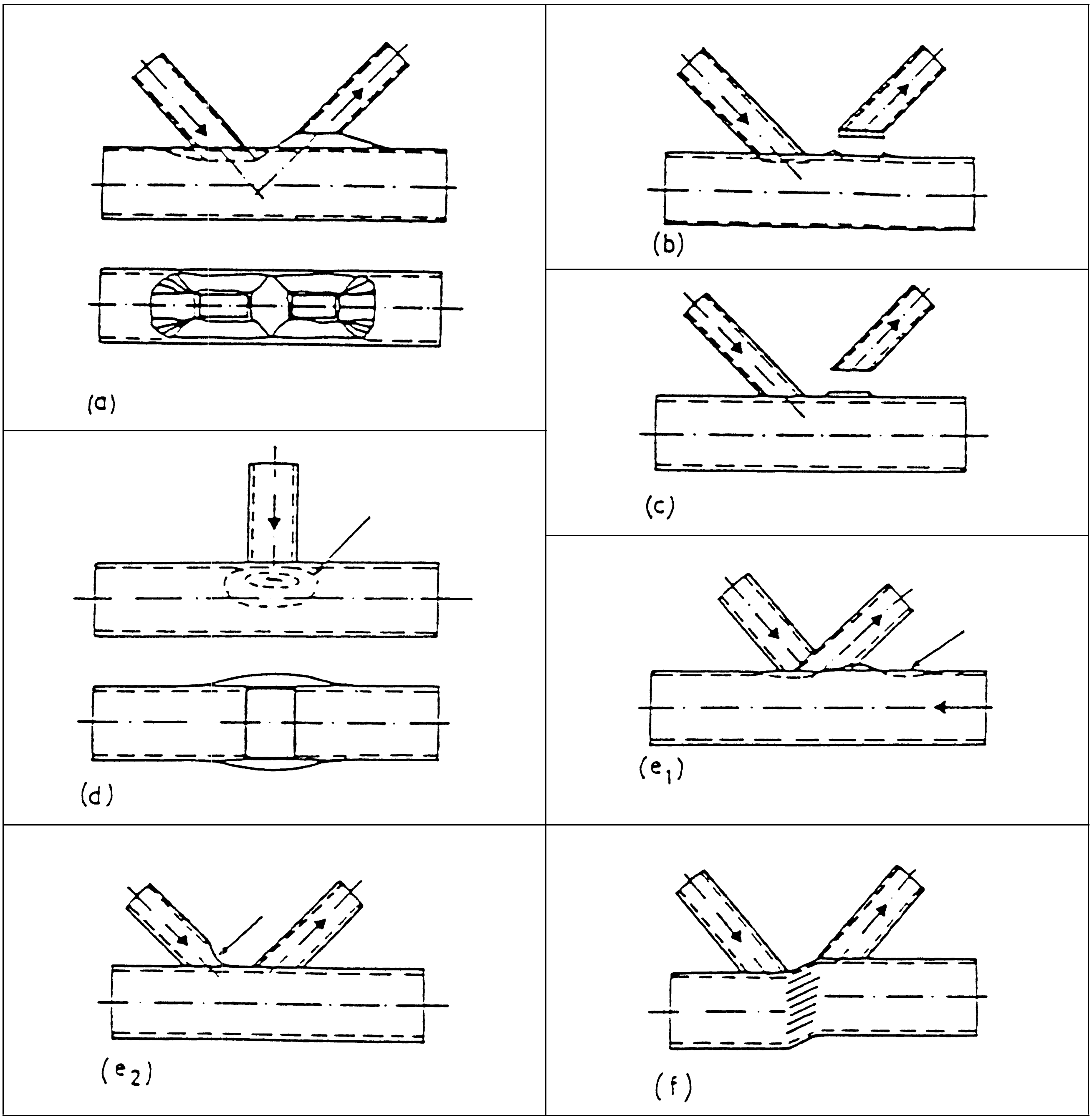
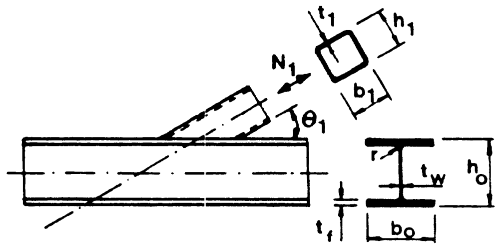
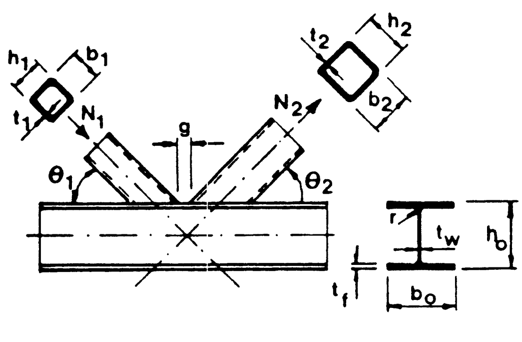
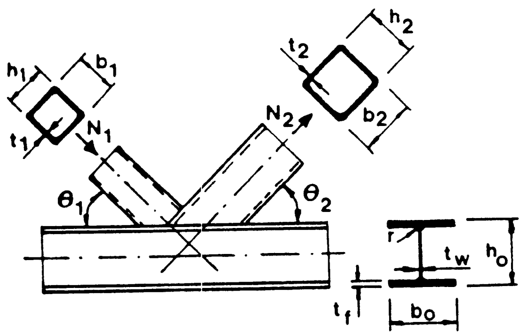


Figure K.4 — Modes of failure — rectangular sections

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	$\frac{b_i}{t_i}, \frac{h_i}{t_i}, \frac{d_i}{t_i}$ 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

Type of joint	Design resistance (i = 1 or 2, j = overlapped brace)
T, Y and X joints	Chord web yielding
	$N_{1.Rd} = \frac{f_{yo}\, t_w\, b_w}{\sin\theta_1}\left[\frac{1,1}{\gamma_{Mj}}\right]$
	Effective width
	$N_{1.Rd} = 2 f_{y1}\, t_1\, b_{eff}\left[\frac{1,1}{\gamma_{Mj}}\right]$
K and N gap joints	Chord web stability
	$N_{i.Rd} = \frac{f_{yo}\, t_w\, b_w}{\sin\theta_i}\left[\frac{1,1}{\gamma_{Mj}}\right]$
	Effective width
	$N_{i.Rd} = 2 f_{yi}\, t_i\, b_{eff}\left[\frac{1,1}{\gamma_{Mj}}\right]$
	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
	Chord shear
	$N_{i.Rd} = \frac{f_{yo}\, A_v}{\sqrt{3}\,\sin\theta_i}\left[\frac{1,1}{\gamma_{Mj}}\right]$
K and N overlap joints[a]	Effective width $25\ \% \leq \lambda_{ov} < 50\ \%$
	$N_{i.Rd} = f_{yi}\, t_i\left[\frac{\lambda_{ov}}{50}(2h_i - 4t_i) + b_{eff} + b_{e.ov}\right]\left[\frac{1,1}{\gamma_{Mj}}\right]$
	Effective width $50\ \% \leq \lambda_{ov} < 80\ \%$
	$N_{i.Rd} = f_{yi}\, t_i\, [2h_i - 4t_i + b_{eff} + b_{e.ov}]\left[\frac{1,1}{\gamma_{Mj}}\right]$
	Effective width $\lambda_{ov} \geq 80\ \%$
	$N_{i.Rd} = f_{yi}\, t_i\, [2h_i - 4t_i + b_i + b_{e.ov}]\left[\frac{1,1}{\gamma_{Mj}}\right]$
Functions	
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 $\begin{cases} b_w = \frac{d_i}{\sin\theta_i} + 5(t_f + r) \\ b_w \leq 2t_i + 10(t_f + r) \end{cases}$	$A_v = A_o - (2 - \alpha)b_o\, t_f + (t_w + 2r)t_f$ for RHS $\alpha = \left[\frac{1}{1 + \frac{4g^2}{3t_f^2}}\right]^{0,5}$ for CHS brace $\alpha = 0$
$b_{eff} = t_w + 2r + 7\frac{f_{yo}}{f_{yi}}t_f$ but $b_{eff} \leq b_i$	$b_{e.ov} = \frac{10}{b_j/t_j}\,\frac{f_{yj}\, t_j}{f_{yi}\, t_i}\, b_i$ but $b_{e.ov} \leq b_i$

[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.

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* $\left(\frac{d_1}{d_o}, \frac{d_1 + d_2}{2\, d_o}, \frac{b_1}{b_o} \text{ or } \frac{b_1 + b_2}{2\, b_o}\right)$
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.

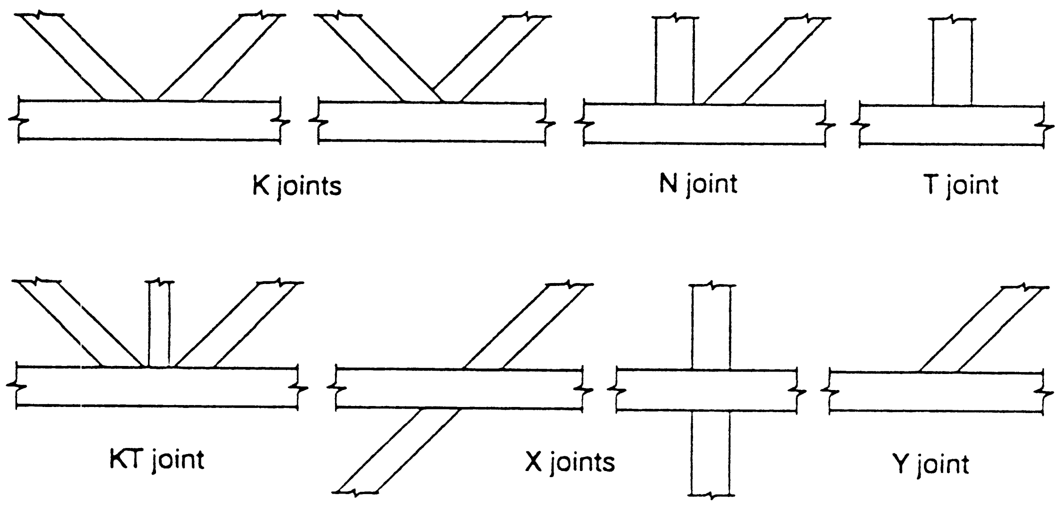


Figure K.5 — Types of joints

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:*

$$f_j = \beta_j \, k_j \, f_{cd} \qquad (L.3)$$

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:

$$k_j = \left[\frac{a_1 \, b_1}{ab}\right]^{0,5} \qquad (L.4)$$

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.

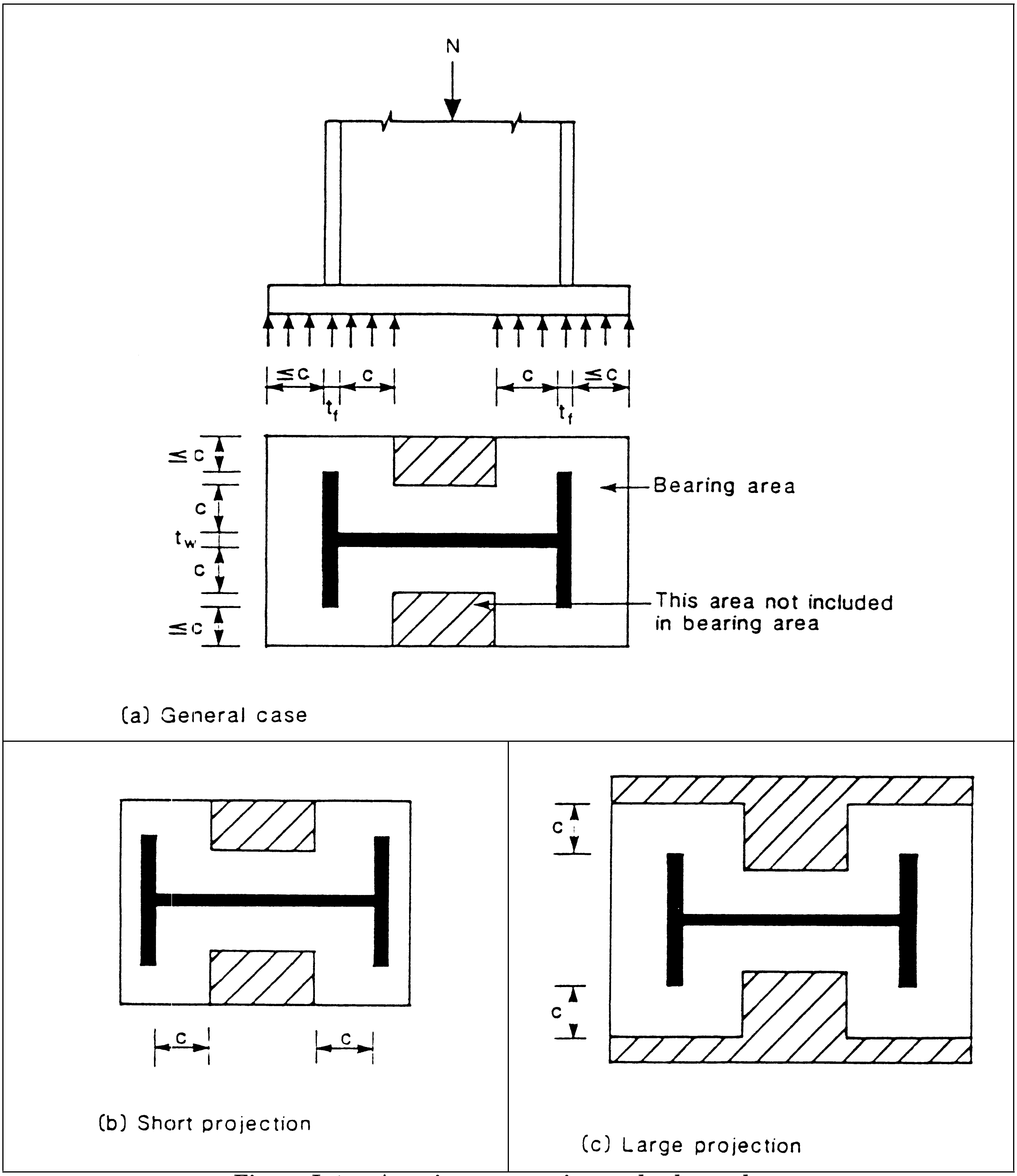


Figure L.1 — Area in compression under base plate

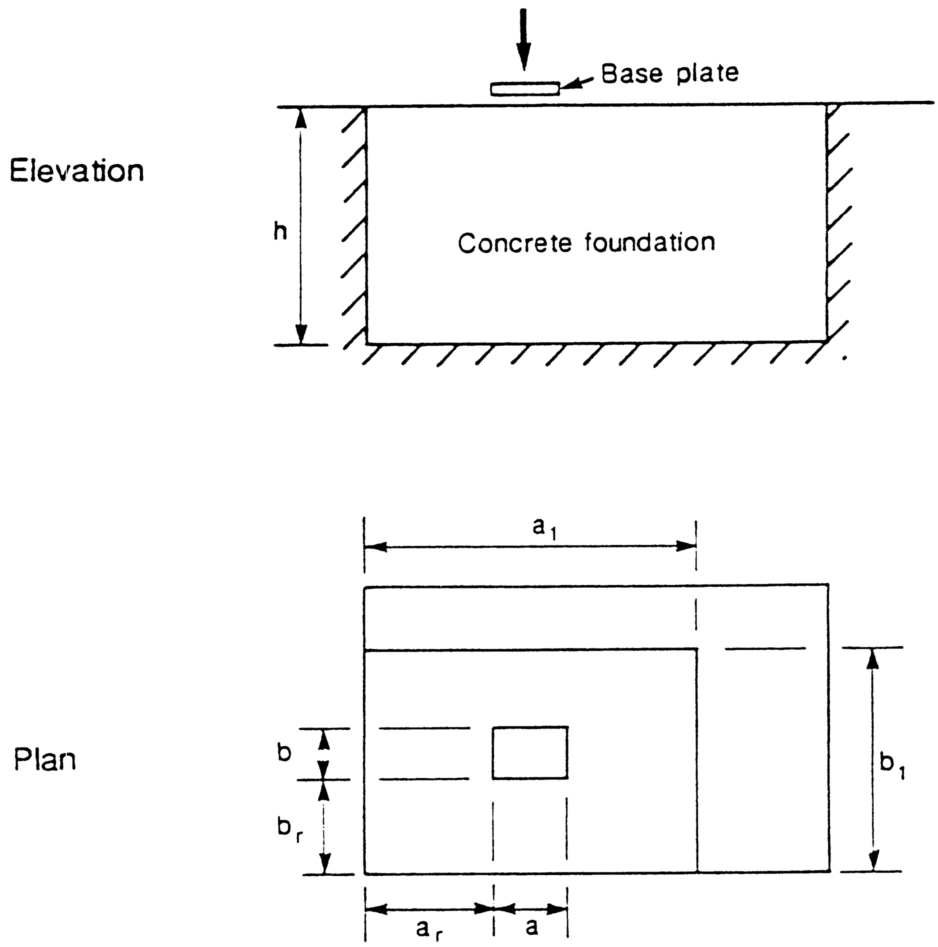


Figure L.2 — Column base

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/mm²*.

(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.*

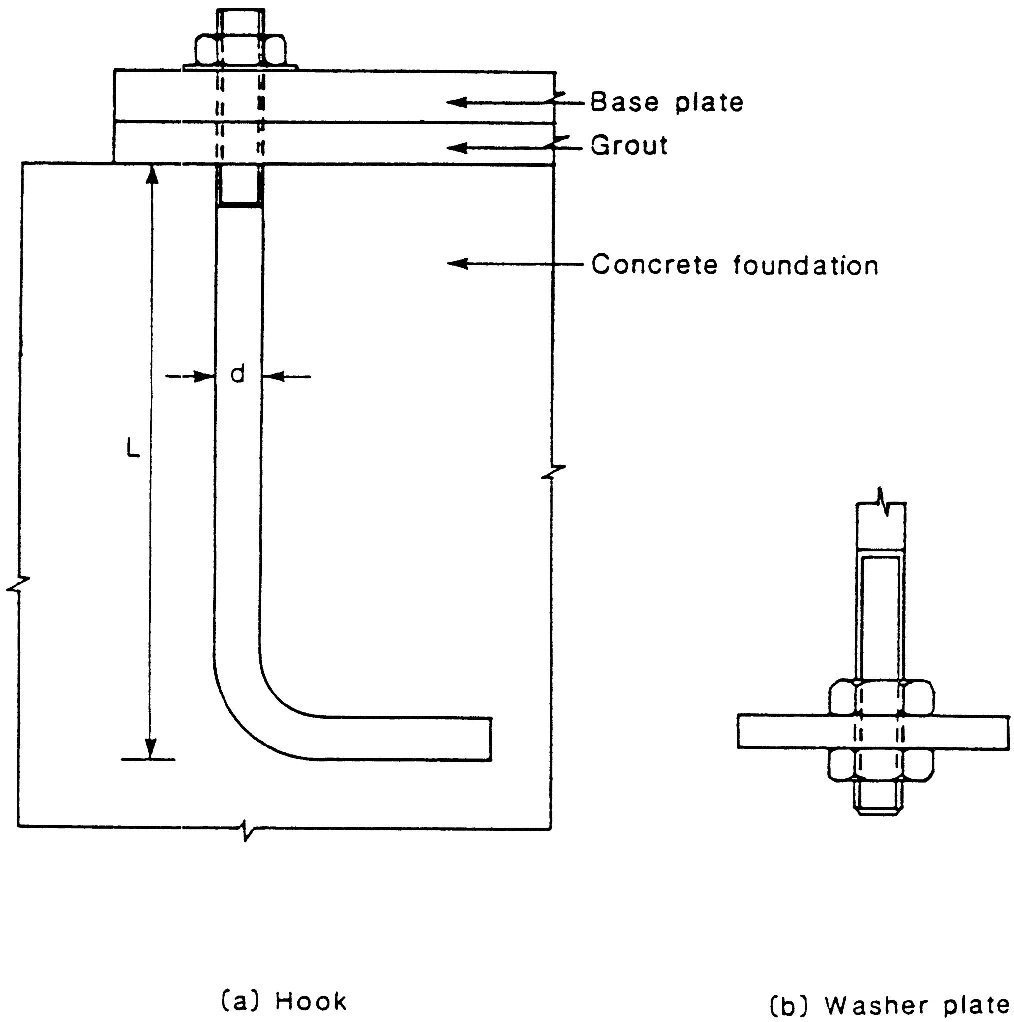


Figure L.3 — Anchorage of holding down bolts

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*

$\sigma_{||}$ *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*

$\tau_{||}$ *is the shear stress (in the plane of the throat) parallel to the axis of the weld.*

(4) *The normal stress $\sigma_{||}$ 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:*

$$[\sigma_{\perp}^2 + 3\,(\tau_{\perp}^2 + \tau_{||}^2)]^{0,5} \le f_u/(\beta_w\ \gamma_{Mw}) \qquad \text{(M1)}$$

$$\text{and} \quad \sigma_{\perp} \le f_u/\gamma_{Mw}$$

*where f_u and β_w are as defined in **6.6.5.3**.*

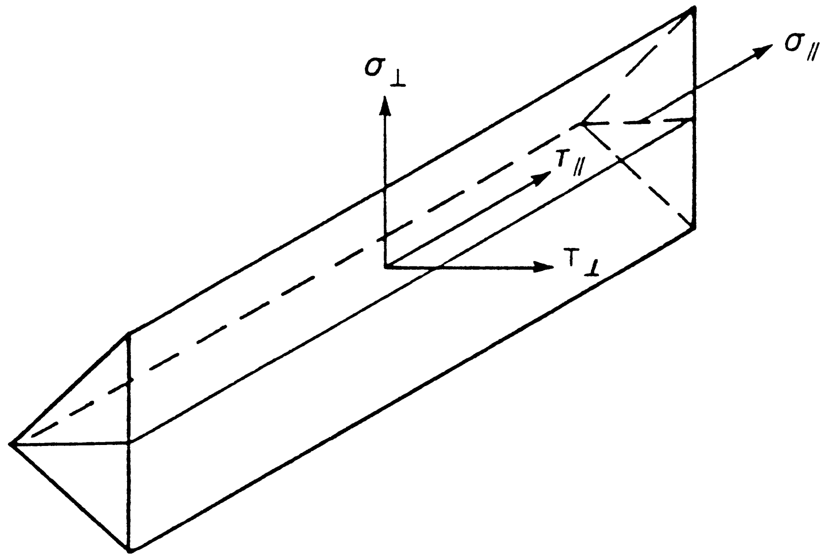


Figure M.1 — Stresses on the throat section of a fillet weld

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:

$$F_{Rd} = 0{,}9F_{test.R.min}\,(f_y/f_{ym})/\gamma_{M1} \qquad \text{(Y.2)}$$

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:

$$F_{Rd} = 0{,}9F_{test.R.min}(f_y/f_{um})/\gamma_{M1} \qquad \text{(Y.3)}$$

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:

$$F_{Rd} = 0{,}75F_{test.R.min}(f_y/f_{ym})/\gamma_{M1} \qquad \text{(Y.4)}$$

(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:

$$F_{Rd} = 0{,}9F_{test.R.min}[(\chi\, f_y)/(\chi_m\, f_{ym})]/\gamma_{M1} \qquad \text{(Y.5)}$$

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
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