

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

NO COPYING WITHOUT BSI PERMISSION EXCEPT AS PERMITTED BY COPYRIGHT LAW



Document Sharing Hub

Cooperating organizations

The European Committee for Standardization (CEN), under whose supervision this European Standard was prepared, comprises the national standards organizations of the following countries:

Austria	Oesterreichisches Normungsinstitut
Belgium	Institut belge de normalisation
Denmark	Dansk Standardiseringsraad
Finland	Suomen Standardisoimisliitto, r.y.
France	Association française de normalisation
Germany	Deutsches Institut für Normung e.V.
Greece	Hellenic Organization for Standardization
Iceland	Technological Institute of Iceland
Ireland	National Standards Authority of Ireland
Italy	Ente Nazionale Italiano di Unificazione
Luxembourg	Inspection du Travail et des Mines
Netherlands	Nederlands Normalisatie-instituut
Norway	Norges Standardiseringsforbund
Portugal	Instituto Português da Qualidade
Spain	Asociación Española de Normalización y Certificación
Sweden	Standardiseringskommissionen i Sverige
Switzerland	Association suisse de normalisation
United Kingdom	British Standards Institution

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

© BSI 04-2000

The following BSI reference relates to the work on this Draft for Development:
Committee reference B/525/31

ISBN 0 580 21226 2

Amendments issued since publication

Amd. No.	Date	Comments

Contents

	Page
Cooperating organizations	Inside front cover
National foreword	ii
Text of National Application Document	v
Foreword	2
Text of ENV 1993-1-1	13
National annex NA (informative) Committees responsible	Inside back cover

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 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

	Page
Introduction	v
1 Scope	v
2 References	v
3 Partial safety factors, combination factors and other values	v
4 Loading codes	vii
5 Reference standards	viii
6 Additional recommendations	xi
Annex A (normative) General recommendations for structural integrity	xvi
Annex B (normative) Application rules for columns in simple framing	xvii
Table 1 — Partial safety factors (γ factors)	v
Table 2 — Partial safety factors for fatigue strength	vi
Table 3 — Combination factors (ψ factors)	vii
Table 4 — Combination factors for accidental loads	vii
Table 5 — Reference standard 1. Weldable structural steel	viii
Table 6 — Reference standard 2. Dimensions of sections and plates	viii
Table 7 — Reference standard 2. Dimensions of sections and plates: tolerances	ix
Table 8 — Reference standard 3. Bolts, nuts and washers: non-pre-loaded bolts	x
Table 9 — Reference standard 3. Bolts, nuts and washers: pre-loaded bolts	x
Table 10 — Reference standard 4. Welding consumables	x
Table 11 — Reference standard 5. Rivets	xi
Table 12 — Reference standards 6 to 9. Execution standards	xi
Table 13 — Reference standard 10. Corrosion protection	xi
Table 14 — Directly referenced supporting standards	xi
Table 15 — Maximum thickness for statically loaded structural elements	xiii
List of references	xix

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	$\gamma_{G, \text{inf}}$	Favourable	1.00	1.00
		$\gamma_{G, \text{sup}}$	Unfavourable	1.35	1.35
2.3.3.1(1)	Partial safety factors for variable action	$\gamma_{Q, \text{inf}}$	Favourable	0.00	0.00
		$\gamma_{Q, \text{sup}}$	Unfavourable	1.50	1.50
		$\gamma_{Q, \text{sup}}$	2 or more combined	1.50	1.50
2.3.3.1(3)	Partial safety factors for permanent action	$\gamma_{G, \text{inf}}$	Favourable part	1.10	1.10
		$\gamma_{G, \text{sup}}$	Unfavourable part	1.35	1.35
		$\gamma_{G, \text{inf}}$	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	$\gamma_{Ms,ult}$	Ultimate limit state	1.25	1.20
		$\gamma_{Ms,ser}$	Serviceability limit state	1.10	1.35
		$\gamma_{Ms,ult}$	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
				Fe 510 or Fe E 355	1.50
				All other grades	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.

- 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.
- 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 $+35^{\circ}\text{C}$. 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. Fatiguea) *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 basesa) *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_b , 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/m² 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: $0.5w_f s_t L_a$ for any internal ties and $0.25w_f s_t L_a$ 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 m² 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 m² 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/m². 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.*

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

© 1992 Copyright reserved to CEN members

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	<i>Basis of design and actions on structures</i>
EN 1992	Eurocode 2	<i>Design of concrete structures</i>
EN 1993	Eurocode 3	<i>Design of steel structures</i>
EN 1994	Eurocode 4	<i>Design of composite steel and concrete structures</i>
EN 1995	Eurocode 5	<i>Design of timber structures</i>

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 ☐. 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

	Page	Page
Foreword	2	
1 Introduction	13	
1.1 Scope	13	
1.1.1 Scope of Eurocode 3	13	
1.1.2 Scope of Part 1.1 of Eurocode 3	13	
1.1.3 Further Parts of Eurocode 3	14	
1.2 Distinction between principles and application rules	14	
1.3 Assumptions	15	
1.4 Definitions	15	
1.4.1 Terms common to all Structural Eurocodes	15	
1.4.2 Special terms used in this Part 1.1 of Eurocode 3	16	
1.5 S.I. units	16	
1.6 Symbols used in Part 1.1 of Eurocode 3	18	
1.6.1 Latin upper case letters	18	
1.6.2 Greek upper case letters	18	
1.6.3 Latin lower case letters	18	
1.6.4 Greek lower case letters	19	
1.6.5 Subscripts	19	
1.6.6 Use of subscripts in Part 1.1 of Eurocode 3	21	
1.6.7 Conventions for member axes	22	
2 Basis of design	24	
2.1 Fundamental requirements	24	
2.2 Definitions and classifications	24	
2.2.1 Limit states and design situations	24	
2.2.2 Actions	25	
2.2.3 Material properties	26	
2.2.4 Geometrical data	27	
2.2.5 Load arrangements and load cases	27	
2.3 Design requirements	27	
2.3.1 General	27	
2.3.2 Ultimate limit states	27	
2.3.3 Partial safety factors for ultimate limit states	29	
2.3.4 Serviceability limit states	30	
2.4 Durability	31	
2.5 Fire resistance	32	
3 Materials	32	
3.1 General	32	
3.2 Structural steel	32	
3.2.1 Scope	32	
		3.2.2 Material properties for hot rolled steel 32
		3.2.3 Material properties for cold formed steel 33
		3.2.4 Dimensions, mass and tolerances 33
		3.2.5 Design values of material coefficients 33
		3.3 Connecting devices 34
		3.3.1 General 34
		3.3.2 Bolts, nuts and washers 35
		3.3.3 Other types of preloaded fasteners 35
		3.3.4 Rivets 35
		3.3.5 Welding consumables 35
		4 Serviceability limit states 35
		4.1 Basis 35
		4.2 Deflections 36
		4.2.1 Requirements 36
		4.2.2 Limiting values 36
		4.2.3 Ponding 36
		4.3 Dynamic effects 37
		4.3.1 Requirements 37
		4.3.2 Structures open to the public 38
		4.3.3 Wind-excited oscillations 38
		5 Ultimate limit states 38
		5.1 Basis 38
		5.1.1 General 38
		5.1.2 Frame design 38
		5.1.3 Tension members 39
		5.1.4 Compression members 39
		5.1.5 Beams 39
		5.1.6 Members with combined axial force and moment 39
		5.1.7 Joints and connections 39
		5.1.8 Fatigue 39
		5.2 Calculation of internal forces and moments 40
		5.2.1 Global analysis 40
		5.2.2 Design assumptions 42
		5.2.3 Structural systems 43
		5.2.4 Allowance for imperfections 44
		5.2.5 Sway stability 46
		5.2.6 Frame stability 50
		5.2.7 Column requirements for plastic analysis 51
		5.3 Classification of cross-sections 52
		5.3.1 Basis 52
		5.3.2 Classification 52
		5.3.3 Cross-section requirements for plastic global analysis 53

		Page			Page
5.3.4	Cross-section requirements when elastic global analysis is used	53	5.9.3	Battened compression members	100
5.3.5	Effective cross-section properties of Class 4 cross-sections	57	5.9.4	Closely spaced built-up members	102
5.3.6	Effects of transverse forces on webs	62	5.9.5	Star-battened angle members	102
5.4	Resistance of cross-sections	62	6	Connections subject to static loading	104
5.4.1	General	62	6.1	Basis	104
5.4.2	Section properties	63	6.1.1	Introduction	104
5.4.3	Tension	65	6.1.2	Applied forces and moments	105
5.4.4	Compression	65	6.1.3	Resistance of connections	105
5.4.5	Bending moment	65	6.1.4	Design assumptions	105
5.4.6	Shear	66	6.1.5	Fabrication and erection	105
5.4.7	Bending and shear	67	6.2	Intersections	105
5.4.8	Bending and axial force	67	6.3	Joints loaded in shear subject to vibration and/or load reversal	106
5.4.9	Bending, shear and axial force	70	6.4	Classification of connections	106
5.4.10	Transverse forces on webs	70	6.4.1	General	106
5.5	Buckling resistance of members	72	6.4.2	Classification by rigidity	106
5.5.1	Compression members	72	6.4.3	Classification by strength	106
5.5.2	Lateral-torsional buckling of beams	77	6.5	Connections made with bolts, rivets or pins	107
5.5.3	Bending and axial tension	78	6.5.1	Positioning of holes for bolts and rivets	107
5.5.4	Bending and axial compression	78	6.5.2	Deductions for fastener holes	109
5.6	Shear buckling resistance	80	6.5.3	Categories of bolted connections	113
5.6.1	Basis	80	6.5.4	Distribution of forces between fasteners	114
5.6.2	Design methods	81	6.5.5	Design resistance of bolts	116
5.6.3	Simple post-critical method	81	6.5.6	Design resistance of rivets	117
5.6.4	Tension field method	82	6.5.7	Countersunk bolts and rivets	118
5.6.5	Intermediate transverse stiffeners	84	6.5.8	High strength bolts in slip-resistant connections	119
5.6.6	Welds	85	6.5.9	Prying forces	120
5.6.7	Interaction between shear force, bending moment and axial force	86	6.5.10	Long joints	120
5.7	Resistance of webs to transverse forces	88	6.5.11	Single lap joints with one bolt	122
5.7.1	Basis	88	6.5.12	Fasteners through packings	122
5.7.2	Length of stiff bearing	89	6.5.13	Pin connections	123
5.7.3	Crushing resistance	91	6.6	Welded connections	126
5.7.4	Crippling resistance	91	6.6.1	General	126
5.7.5	Buckling resistance	91	6.6.2	Geometry and dimensions	126
5.7.6	Transverse stiffeners	93	6.6.3	Lamellar tearing	132
5.7.7	Flange induced buckling	93	6.6.4	Distribution of forces	133
5.8	Triangulated structures	94	6.6.5	Design resistance of a fillet weld	133
5.8.1	General	94	6.6.6	Design resistance of butt welds	135
5.8.2	Buckling length of members	94	6.6.7	Design resistance of plug welds	136
5.8.3	Angles as web members in compression	94	6.6.8	Joints to unstiffened flanges	137
5.9	Built-up compression members	94	6.6.9	Long joints	137
5.9.1	Basis	94	6.6.10	Angles connected by one leg	138
5.9.2	Laced compression members	95	6.7	Hybrid connections	138

	Page		Page
6.8 Splices	139	8 Design assisted by testing	162
6.8.1 General	139	8.1 Basis	162
6.8.2 Splices in compression members	139	8.2 Planning of tests	163
6.8.3 Splices in tension members	139	8.3 Execution of tests	163
6.9 Beam-to-column connections	139	8.4 Test evaluation	163
6.9.1 Basis	139	8.5 Documentation	164
6.9.2 Moment-rotation characteristic	139	9 Fatigue	164
6.9.3 Moment resistance	141	9.1 General	164
6.9.4 Rotational stiffness	141	9.1.1 Basis	164
6.9.5 Rotation capacity	141	9.1.2 Scope	164
6.9.6 Classification of beam-to-column connections	147	9.1.3 Limitations	164
6.9.7 Calculated properties	151	9.1.4 Necessity for fatigue assessment	164
6.9.8 Application rules	151	9.1.5 Definitions	165
6.10 Hollow section lattice girder joints	152	9.1.6 Symbols	168
6.10.1 Design resistance	152	9.2 Fatigue loading	168
6.10.2 Application rules	153	9.3 Partial safety factors	169
6.11 Column bases	153	9.3.1 General	169
6.11.1 Base plates	153	9.3.2 Partial safety factors for fatigue loading	169
6.11.2 Holding down bolts	153	9.3.3 Partial safety factors for fatigue strength	169
6.11.3 Application rules	153	9.3.4 Recommended values for Y_{Mf}	169
7 Fabrication and erection	153	9.4 Fatigue stress spectra	170
7.1 General	153	9.4.1 Calculation of stresses	170
7.1.1 Scope	153	9.4.2 Stress range in parent material	170
7.1.2 Requirements	154	9.4.3 Stress range for welds	170
7.2 Project specification	154	9.4.4 Design stress range spectrum	170
7.3 Fabrication restrictions	154	9.5 Fatigue assessment procedures	171
7.4 Preparation of material	155	9.5.1 General	171
7.5 Bolted connections	155	9.5.2 Fatigue assessment based on nominal stress ranges	171
7.5.1 Holes	155	9.5.3 Fatigue assessments based on geometric stress ranges	173
7.5.2 Clearance in holes for fasteners	155	9.6 Fatigue strength	173
7.5.3 Bolts	156	9.6.1 General	173
7.5.4 Nuts	156	9.6.2 Fatigue strength curves for classified details	176
7.5.5 Washers	156	9.6.3 Fatigue strength curves for non-classified details	179
7.5.6 Tightening of bolts	157	9.7 Fatigue strength modifications	180
7.5.7 Slip resistant contact surfaces	157	9.7.1 Stress range in non-welded or stress relieved details	180
7.5.8 Fit of contact surfaces	157	9.7.2 Influence of thickness	180
7.6 Welded connections	157	9.7.3 Modified fatigue strength curves	180
7.7 Tolerances	158	9.8 Classification tables	180
7.7.1 Types of tolerances	158	Annex B (normative) Reference standards	197
7.7.2 Application of tolerances	158	B.1 Scope	197
7.7.3 Normal erection tolerances	158		
7.7.4 Fabrication tolerances	161		
7.7.5 Position of holding down bolts	162		
7.8 Inspection and Testing	162		

	Page		Page
B.2 Definitions	197	J.2 Welded beam-to-column connections	217
B.2.1 Reference standard 1: "Weldable structural steel"	197	J.2.1 Moment resistance	217
B.2.2 Reference standard 2: "Dimensions of sections and plates"	197	J.2.2 Supplementary web plates	217
B.2.3 Tolerances	198	J.2.3 Resistance of tension zone	219
B.2.4 Reference standard 3: "Bolts, nuts and washers"	198	J.2.4 Resistance of compression zone	222
B.2.5 Reference standard 4: "Welding consumables"	198	J.2.5 Resistance of shear zone	224
B.2.6 Reference standard 5: "Rivets"	198	J.2.6 Rotational stiffness	227
B.2.7 Reference standards 6 to 9: "Process standards"	199	J.2.7 Rotation capacity	227
B.2.8 Reference standard 10: "Corrosion protection"	199	J.3 Bolted beam-to-column connections	227
Annex C (informative) Design against brittle fracture	199	J.3.1 Limitations	227
C.1 Resistance to brittle fracture	199	J.3.2 Moment resistance	228
C.2 Calculation procedure	199	J.3.3 Equivalent T-stub	232
C.2.1 Service conditions	199	J.3.4 Resistance of tension zone	235
C.2.2 Loading rate	199	J.3.5 Resistance of compression zone	245
C.2.3 Consequences of failure	200	J.3.6 Resistance of shear zone	245
C.2.4 Nominal yield strength	200	J.3.7 Rotational stiffness	246
C.2.5 Parameters	200	J.3.8 Rotation capacity	247
C.2.6 Calculations	201	Annex K (normative) Hollow section lattice girder connections	248
Annex E (informative) Buckling length of a compression member	201	K.1 General	248
E.1 Basis	201	K.2 Definitions	248
E.2 Columns in building frames	201	K.3 Field of application	249
Annex F (informative) Lateral-torsional buckling	207	K.4 Analysis	250
F.1 Elastic critical moment	207	K.5 Welds	250
F.1.1 Basis	207	K.6 Welded joints between circular hollow sections	250
F.1.2 General formula for cross-sections symmetrical about the minor axis	208	K.7 Welded joints between hollow section brace members and square or rectangular hollow section chords	251
F.1.3 Beams with uniform doubly symmetric cross-sections	208	K.7.1 General	251
F.1.4 Beams with uniform monosymmetric cross-sections with unequal flanges	212	K.7.2 Square or circular brace members and square chords	251
F.2 Slenderness	212	K.7.3 Rectangular sections	256
F.2.1 General	212	K.8 Welded joints between hollow section brace members and an I or H section chord	256
F.2.2 Beams with uniform doubly symmetric cross-sections	212	K.9 Symbols used in tables	260
Annex J (normative) Beam-to-column connections	215	Annex L (normative) Column bases	261
J.1 Scope	215	L.1 Base plates	261
J.1.1 Types of connections covered	215	L.2 Holding down bolts	264
J.1.2 Other types of connections	215	Annex M (normative) Alternative method for fillet welds	265
		Annex Y (informative) Guidelines for loading tests	266
		Y.1 General	266
		Y.2 Test conditions	267
		Y.3 General test procedures	267

	Page		Page
Y.4 Specific test procedures	267	Figure 5.7.3 — Effective breadth for web buckling resistance	92
Y.4.1 Acceptance test	267	Figure 5.7.4 — Effective cross-section of stiffeners	93
Y.4.2 Strength test	268	Figure 5.9.1 — Single lacing systems on opposite sides of main components	96
Y.4.3 Test to failure	268	Figure 5.9.2 — Lacing systems combined with other components perpendicular to the longitudinal axis of the member	97
Y.4.4 Check tests	269	Figure 5.9.3 — Laced compression member	98
Y.4.5 Testing to determine strength functions and model factors	269	Figure 5.9.4 — Buckling lengths of angle chords in laced members	99
Y.4.6 Other test procedures	270	Figure 5.9.5 — Battened compression member	103
Figure 1.1 — Dimensions and axes of sections	23	Figure 5.9.6 — Closely spaced built-up members	103
Figure 4.1 — Vertical deflections to be considered	37	Figure 5.9.7 — Star-battened angle members	104
Figure 5.2.1 — Bi-linear stress-strain relationship	41	Figure 6.5.1 — Symbols for spacing of fasteners	108
Figure 5.2.2 — Alternative bi-linear stress-strain relationship (for use in elasto-plastic analysis only)	41	Figure 6.5.2 — Staggered spacing — compression	108
Figure 5.2.3 — Replacement of initial sway imperfections by equivalent horizontal forces	46	Figure 6.5.3 — Spacing in tension members	109
Figure 5.2.4 — Equivalent horizontal forces	46	Figure 6.5.4 — End and edge distances for slotted holes	109
Figure 5.2.5 — Equivalent stabilizing force	47	Figure 6.5.5 — Block shear — effective shear area	111
Figure 5.2.6 — Bracing forces at splices in compression members	48	Figure 6.5.6 — Connections of angles	113
Figure 5.2.7 — Building frame with beams connecting each column at each storey level	50	Figure 6.5.7 — Distribution of loads between fasteners	115
Figure 5.2.8 — Sway mechanism involving plastic hinges in columns at bases only	52	Figure 6.5.8 — Prying forces	121
Figure 5.3.1 — Class 4 cross-sections — axial force	61	Figure 6.5.9 — Effect of details on prying forces	121
Figure 5.3.2 — Class 4 cross-sections — bending moment	62	Figure 6.5.10 — Long joints	122
Figure 5.4.1 — Staggered holes	64	Figure 6.5.11 — Single lap joint with one bolt	122
Figure 5.4.2 — Angles with holes in both legs	64	Figure 6.5.12 — Bending moment in a pin	125
Figure 5.4.3 — Stresses in web panel due to bending moment, axial force and transverse force	71	Figure 6.6.1 — Intermittent of fillet welds	129
Figure 5.5.1 — Design values of equivalent initial bow imperfection $e_{o,d}$	75	Figure 6.6.2 — Single fillet welds and single-sided partial penetration butt welds	130
Figure 5.5.2 — Average yield strength f_{ya} of cold formed structural hollow sections	77	Figure 6.6.3 — Effective throat of flare groove welds in rectangular structural hollow sections	131
Figure 5.5.3 — Equivalent uniform moment factors	80	Figure 6.6.4 — Effective throat of flare groove welds in solid sections	132
Figure 5.6.1 — Tension field geometry	83	Figure 6.6.5 — Avoiding lamellar tearing	132
Figure 5.6.2 — End panel geometry	83	Figure 6.6.6 — Throat thickness of a fillet weld	134
Figure 5.6.3 — Tension field stresses	85	Figure 6.6.7 — Throat thickness of a deep penetration fillet weld	134
Figure 5.6.4 — Interaction of shear buckling resistance and moment resistance	87	Figure 6.6.8 — Partial penetration butt welds	136
Figure 5.7.1 — Forces applied through a flange	89	Figure 6.6.9 — Tee-butt welds	136
Figure 5.7.2 — Length of stiff bearing	90		

	Page		Page
Figure 6.6.10 — Effective breadth of an unstiffened tee-joint	137	Figure J.2.1 — Supplementary web plate	218
Figure 6.7.1 — Hybrid connections	138	Figure J.2.2 — Spacing of plug welds or bolts for supplementary web plate	220
Figure 6.9.1 — Modelling a connection as a rotational spring	140	Figure J.2.3 — Transverse force on an unstiffened column	221
Figure 6.9.2 — Derivation of approximate moment-rotation characteristics	142	Figure J.2.4 — Column buckling modes of an unstiffened web	223
Figure 6.9.3 — Properties of a design moment-rotation characteristic	143	Figure J.2.5 — Unstiffened column web panel subject to a shear force	225
Figure 6.9.4 — Moment-rotation characteristic with an initial hinge rotation	144	Figure J.2.6 — Column web panels with diagonal stiffeners	226
Figure 6.9.5 — Rotational stiffness S_j	145	Figure J.3.1 — Dimensions of a T-stub	233
Figure 6.9.6 — Variation of rotational stiffness with applied moment	146	Figure J.3.2 — Failure modes of a T-stub flange	234
Figure 6.9.7 — Rotation capacity ϕ_{Cd}	147	Figure J.3.3 — Effect of connection geometry on failure mode of a T-stub flange	235
Figure 6.9.8 — Recommended classification boundaries for rigid beam-to-column connections	149	Figure J.3.4 — Yield line patterns for an unstiffened flange	237
Figure 6.9.9 — Examples of classification of moment-rotation characteristics for beam-to-column connections	150	Figure J.3.5 — Column flange with backing plates in the tension zone	238
Figure 6.9.10 — Critical zones in beam-to-column connections	152	Figure J.3.6 — Effective lengths of equivalent T-stub flanges representing a stiffened column flange	240
Figure 7.1 — Maximum step between adjacent surfaces	157	Figure J.3.7 — Values of α for stiffened column flanges	241
Figure 7.2.1 — Normal tolerances after erection — Part 1	160	Figure J.3.8 — Effective lengths of equivalent T-stub flanges representing an end plate	243
Figure 7.2.2 — Normal tolerances after erection — Part 2	161	Figure K.1 — Gap and overlap of joints	248
Figure 9.1.1 — Design spectrum	166	Figure K.2 — Eccentricity of joints	249
Figure 9.1.2 — Fatigue strength curve	167	Figure K.3 — Values of factor k_g	253
Figure 9.6.1 — Fatigue strength curves for normal stress ranges	175	Figure K.4 — Modes of failure — rectangular sections	257
Figure 9.6.2 — Fatigue strength curves for shear stress ranges	176	Figure K.5 — Types of joints	261
Figure 9.6.3 — Fatigue strength curves for joints in tubular lattice girders	178	Figure L.1 — Area in compression under base plate	263
Figure 9.7.1 — Modified fatigue strength curve	181	Figure L.2 — Column base	264
Figure E.2.1 — Buckling length ratio l/L for a column in a non-sway mode	202	Figure L.3 — Anchorage of holding down bolts	265
Figure E.2.2 — Buckling length ratio l/L for a column in a sway mode	203	Figure M.1 — Stresses on the throat section of a fillet weld	266
Figure E.2.3 — Distribution factors for columns	204	Procedure J.3.1 Moment resistance of a bolted beam-to-column connection — plastic distribution of bolt forces	228
Figure E.2.4 — Distribution factors for continuous columns	206	Procedure J.3.2 Moment resistance of a bolted beam-to-column connection — distribution of bolt forces proportional to distance from centre of compression	230
Figure F.1.1 — Sign convention for determining z_j	211	Procedure J.3.3 Effective design resistance for bolt-rows	244
Figure J.1.1 — Beam-to-column connections	216		

	Page		Page
Table 1.1 — List of equivalent terms in various languages	17	Table 9.6.4 — Coefficients to account for secondary bending moments in joints of lattice girders made from circular hollow sections	179
Table 2.1 — Design values for actions for use in the combination of actions	28	Table 9.6.5 — Coefficients to account for secondary bending moments in joints of lattice girders made from rectangular hollow sections	179
Table 2.2 — Partial safety factors for actions on building structures for persistent and transient design situations	30	Table 9.7.1 — Numerical values for modified fatigue strength curves for normal stress ranges	180
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	32	Table 9.8.1 — Non-welded details (sheets 1 and 2)	182
Table 3.2 — Maximum thickness for statically loaded structural elements without reference to Annex C	34	Table 9.8.2 — Welded built-up sections (sheets 1 and 2)	184
Table 3.3 — Nominal values of yield strength f_{yb} and ultimate tensile strength f_{ub} for bolts	35	Table 9.8.3 — Transverse butt welds (sheets 1 and 2)	186
Table 4.1 — Recommended limiting values for vertical deflections	37	Table 9.8.4 — Welded attachments with non-load carrying welds (sheets 1 and 2)	188
Table 5.2.1 — Design assumptions	43	Table 9.8.5 — Welded joints with load-carrying welds (sheets 1 to 3)	190
Table 5.3.1 — Maximum width-to-thickness ratios for compression elements (sheets 1 to 4)	54	Table 9.8.6 — Hollow sections (sheets 1 and 2)	193
Table 5.3.2 — Internal compression elements	59	Table 9.8.7 — Lattice girder joints (sheets 1 and 2)	195
Table 5.3.3 — Outstand compression elements	60	Table C.1 — Base value of the mean lower yield strength	200
Table 5.5.1 — Imperfection factors	73	Table C.2 — Values of constants	200
Table 5.5.2 — Reduction factors χ	73	Table C.3 — Charpy V-notch test temperature	201
Table 5.5.3 — Selection of buckling curve for a cross-section	76	Table E.1 — Effective stiffness coefficient for a beam	205
Table 6.5.1 — Reduction factors β_2 and β_3	112	Table E.2 — Effective stiffness coefficient for a beam in a building frame with concrete floor slabs	205
Table 6.5.2 — Categories of bolted connections	114	Table E.3 — Approximate formulae for reduced beam stiffness coefficients due to axial compression	207
Table 6.5.3 — Design resistances for bolts	117	Table F.1.1 — Values of factors C_1 , C_2 and C_3 corresponding to values of factor k : End moment loading	209
Table 6.5.4 — Design bearing resistance — based on bolt diameter	117	Table F.1.2 — Values of factors C_1 , C_2 and C_3 corresponding to values of factor k : Transverse loading cases	210
Table 6.5.5 — Design resistances for rivets	118	Table K.6.1 — Range of validity for welded joints between circular hollow sections	251
Table 6.5.6 — Geometrical conditions for plates in pin connections	124	Table K.6.2 — Design resistances of welded joints between circular hollow sections	252
Table 6.5.7 — Design resistances for pin connections	123	Table K.7.1 — Range of validity for welded joints between square or circular hollow section brace members and square hollow section chords	254
Table 6.6.1 — Common types of welded joints	127		
Table 7.1 — Normal tolerances after erection	159		
Table 7.2 — Straightness tolerances incorporated in design rules	162		
Table 9.3.1 — Partial safety factor for fatigue strength Y_{Mf}	170		
Table 9.6.1 — Numerical values for fatigue strength curves for normal stress ranges	177		
Table 9.6.2 — Numerical values for fatigue strength curves for shear stress ranges	177		
Table 9.6.3 — Numerical values for fatigue strength curves for hollow sections	179		

	Page
Table K.7.2 — Design resistances of welded joints between square or circular hollow section brace members and square hollow section chords	255
Table K.8.1 — Range of validity for welded joints between hollow sections brace members and I or H section chords	258
Table K.8.2 — Design resistances of welded joints between hollow section brace members and I or H section chords	259

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) Execution¹⁾ 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 works¹⁾ and methods of construction¹⁾.

(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 construction¹⁾.

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 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 operations³⁾. 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 rigidity⁴⁾. 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 (I/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. p^l , 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.*

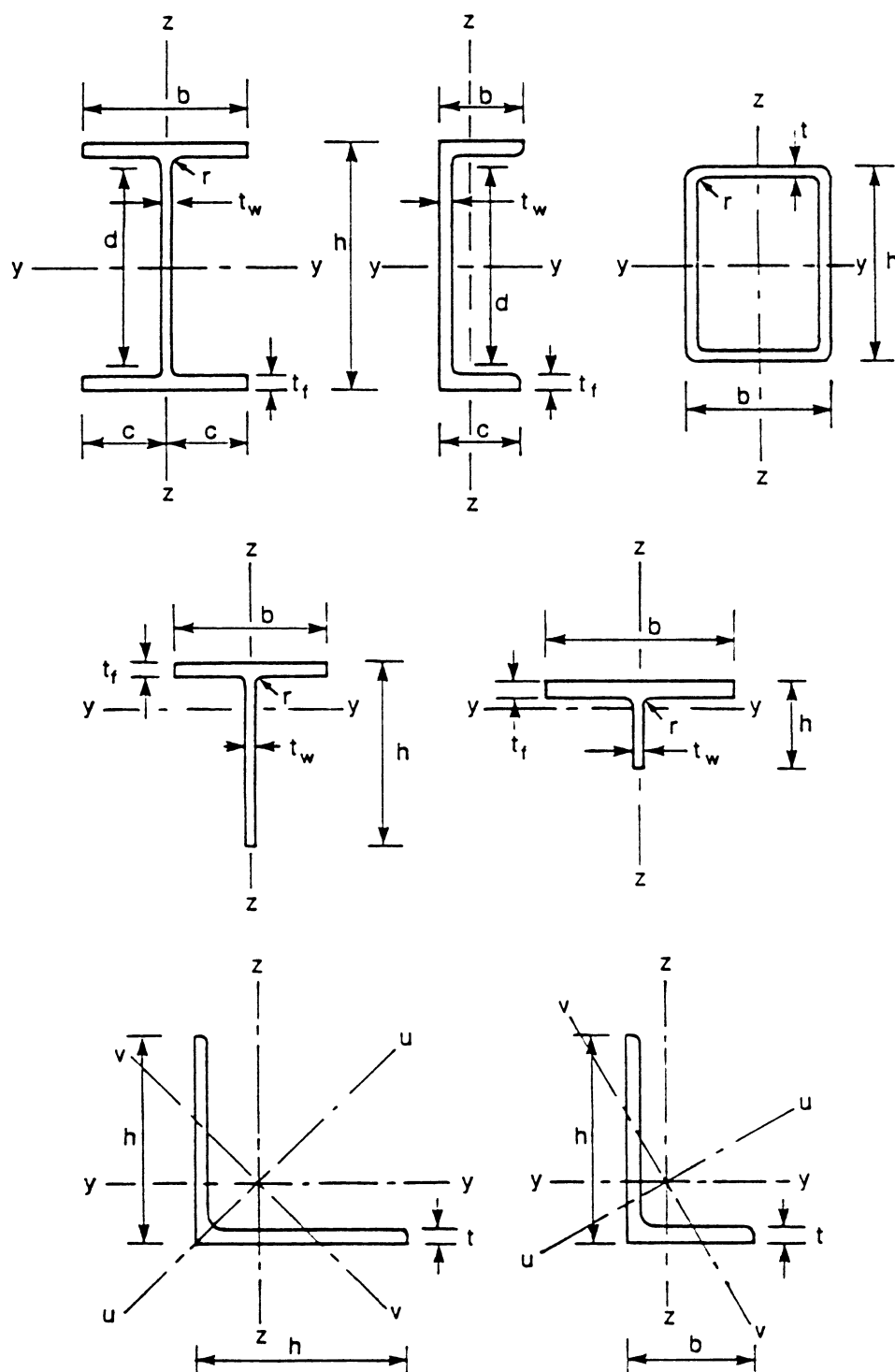


Figure 1.1 — Dimensions and axes of sections

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 classification⁵⁾

(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, ancillaries 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 actions⁶⁾

(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:

$$\begin{aligned} G_d &= \gamma_G G_k \\ Q_d &= \gamma_Q Q_k \text{ or } \gamma_Q \psi_1 Q_k \\ A_d &= \gamma_A A_k \text{ (if } A_d \text{ is not directly specified)} \end{aligned}$$

(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:

$$\begin{aligned} G_{d,\text{sup}} &= \gamma_{G,\text{sup}} G_k \\ G_{d,\text{inf}} &= \gamma_{G,\text{inf}} G_k \end{aligned}$$

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

$$\begin{aligned} G_{d,\text{sup}} &= \gamma_{G,\text{sup}} G_{k,\text{sup}} \\ G_{d,\text{inf}} &= \gamma_{G,\text{inf}} G_{k,\text{inf}} \end{aligned}$$

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

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

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

$\gamma_{G,\text{sup}}$ 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:

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

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:

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