DRAFT FOR DEVELOPMENT

DD ENV 1999-1-1:2000

Eurocode 9: Design of aluminium structures —

Part 1-1: General rules — General rules and rules for buildings

ICS 91.010.30; 91.080.10

National foreword

This Draft for Development is the official English version of ENV 1999-1-1:1998.

This publication is not to be regarded as a British Standard.

It is being issued in the Draft for Development series of publications and is of a provisional nature. It should be applied on this provisional basis, so that information and experience of its practical application may be obtained.

Comments arising from the use of this Draft for Development are requested so that UK experience can be reported to the European organization responsible for its conversion into a European Standard. A review of this publication will be initiated 2 years after its publication by the European organization so that a decision can be taken on its status at the end of its three-year life. The commencement of the review period will be notified by an announcement in *Update Standards*.

According to the replies received by the end of the review period, the responsible BSI Committee will decide whether to support the conversion into a European standard, to extend the life of the prestandard or to withdraw it. Comments should be sent in writing to the Secretary of BSI Subcommittee B/525/9, Structural use of aluminium, at 389 Chiswick High Road, London W4 4AL, giving the document reference and clause number and proposing, where possible, an appropriate revision of the text.

A list of organizations represented on this committee can be obtained on request to its secretary.

Cross-references

The British Standards which implement international or European publications referred to in this document may be found in the BSI Standards Catalogue under the section entitled "International Standards Correspondence Index", or by using the "Find" facility of the BSI Standards Electronic Catalogue.

Summary of pages

This document comprises a front cover, an inside front cover, the National Application Document (14 pages), the ENV title page, pages 2 to 212, an inside back cover and a back cover.

The BSI copyright notice displayed in this document indicates when the document was last issued.

This Draft for Development, having been prepared under the direction of the Sector Committee for Building and Civil Engineering, was published under the authority of the Standards Committee and comes into effect on 15 December 2000

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Amendments issued since publication

Amd. No.	Date	Comments

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National Application Document

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National Application Document

Introduction

This National Application Document (NAD) has been prepared under the direction ofv Technical Committee B525, Building & Civil Engineering structures, (by sub-committeev B525/9). It has been developed from:v

a) a textural examination of ENV 1999-1-1:1998;v

b) a parametric calibration against BS 8118, supporting standards and test data;v

c) trial calculationsv

It should be noted that partial safety factors in ENV 1999-1-1 are in general lower thanv those recommended in BS 8118, but that the load factors in DD ENV 1991-1-1 arev higher than those given in BS 8118.v

1 Scope

This NAD provides information to enable ENV 1999-1-1:1998 (Eurocode 9 Part 1.1) tov be used for the design of buildings and civil and structural engineering works in the UK.v

2 References

2.1 Normative References

This vNAD vincorporates, vby vreference, vprovisions vfrom vspecific veditions vof votherv publications. These normative references are cited at the appropriate points in the text.v Subsequent amendments to, or revision of, any of these publications apply to this NADv only when incorporated in it by updating or revision.v

v

2.2 Informative References

This NAD refers to other publications that provide information or guidance. Editions ofv these publications current at the time of issue of this standard are listed, but referencev should be made to the latest editions.v

3 Partial safety factors, combination factors and other values

a) The values for partial load factors (γ) should be those given in DD ENV 1991-1-1:v 1996v
(Eurocode 1 together with its NAD).v

b) The values for combination factors (ψ) should be those given in DD ENV 1991-1-v 1: 1996.v c)v The values for partial safety factors (γ_m) should be those given in ENV 1999-1-1v except as listed in table 1 of this NADv

TABLE 1 Partial Safety Factors (γ_m)v

Reference In EC9 : Part 1.1	Definition	Symbol	Condition	Value	
				Boxedv In EC9v	UKv
				1.65v	1.6v
6.6.1(4)v	Resistance ofv weldedv connectionsv	(γ_m)v	Specified quality lower than given inv this standardv		

d)v The values for other boxed values should be those in ENV 1999-1-1.v

4 Loading Codes

BS 648: 1964 "Schedule of weights of building materials"v
BS 2573: Part 1: 1983 "Rules for the design of cranes"v
BS 2573: Part 2: 1980 "Rules for the design of cranes"v
BS 5400: Part 2: 1978 "Steel, Concrete and Composite Bridges"v
BS 6399: Part 1: 1996 "Code of practice for dead and imposed loads"v
BS 6399: Part 2: 1997 "Code of practice for wind loads"v
BS 6399: Part 3: 1988 "Code of practice for imposed roof loads"v
BS 8100: Part 1: 1986 "Lattice towers and masts"v

5 Reference Standards

The supporting standards to be used are listed in Tables 3 to 13.v

6 Additional Recommendations

6.1 Guidance on Eurocode 9 : Part 1.1

Note. **6.1.1** to **6.1.4** should be followed when designing in accordance with Eurocode 9 :v Part 1.1.v

6.1.1 *Chapter 2 Basis of Design*

a)v Clause 2.3.4 Serviceability limit states.v
Re-number as Clause 2.3.3.v

b)v Clause 2.5 Fire Resistancev
Reference to av ENV 1999-1-2 should be to D.D. ENV 1999-1-2.v

6.1.2 *Chapter 3 Materials*

a)v *Clause 3.2.1*
Range of materials. Use of aluminium alloys listed in the British Standards givenv in Table 2 is also permitted, subject to the use of the values for 0.2% Proof stressv and minimum tensile strength given in British Standard BS 8118 for the alloy inv question for calculation of f_o, f_a and fv in accordance with clause 5.3.5(2). Itv should also be noted that these values may differ from alloys with the samev numerical designation produced in accordance with an EN standard.v

NOTE 1: This provision is to enable the use of materials held in stock which were produced prior to thev adoption of the EN material standards.v

NOTE 2: BS 8118 uses the terms "minimum 0.2% tensile proof stress" and "minimum tensile strength"v whereas the corresponding terms in ENV 1999 are "0.2% proof strength" and "ultimate tensilev strength"v

b) *Clause 3.2.2.1(2)* insert pr before reference to ENV 1999-1-2 in final line.v

c) *Tables 3.2a to 3.2d incl. Values for strength*
It should be noted that properties vary between different forms of product (withv the v same v numerical v designation) v that v a v fabricator v may v consider v asv interchangeable e.g. strip v. extruded plate. The designer should ensure that thev product form is precisely specified or else use the lower bound figure for design.v

d) *Clause 3.3.4(1)* reference to clause 1.3.3 should be to 1.3.2.v

e) *Clause 3.3.4(2)* replace “Table B.5 and B.6” with “Table B.3”.v

f) *Table 3.5 and Table 3.6* replace “table B.5” with “Table B.2” in the NOTE.v

g) *Clause 3.4.3.2 Metal to metal contacts*
Contact between aluminium and copper (or alloys containing copper) is notv covered by this clause and is not normally recommended. If used, the protectionv should be in accordance with clause 7.7.3.2 Procedure Y.v

6.1.3 *Chapter 5. Ultimate Limit States (Members)*

a) *Clause 5.1.9(3)*

Replace the whole sub-clause with "Fatigue design should be verified using DDv ENV 1999 Part 2".v

b)v *Clause 5.3.3(3)*
Replace the first three lines of sub-section b) with:-v
b) a deduction taken as the greater ofv
$\sum Ah - s^2t/(4p)$v
orv
$\sum Ah - 0.65st$v

Where $\sum Ah$ is the sum of the hole areas on the diagonal or zig-zag sectionv considered.v

c)v *Clause 5.3.3(6)*
Add "which satisfies this limit" at the end of the sentence.v

d)v *Clause 5.5.2(3)*
Replace the first sentence by the following:v
"The severity of softening can be taken into account either by reducing the valuev of the characteristic strength in the H.A.Z. or by reducing the assumed crossv sectional area over which the stresses act"v

e)v *Clause 5.5.3 and Table 5.2*
The designer should specify the weld process to be used, or alternatively use thev lower values.v

f)v *Clause 5.8.4.1(1)*
Insert "A_1" between "or" and "in" in the second line of the note.v

g)v *Table 5.9*
Change the heading from "figure 5.9" to "Table 5.9" on Page 78.v
Change the value 0.2D/B to 0.02D/B in the expression for Y for section type 17.v

h)v *Clause 5.9.3.1(3)*
Change the $N_{t, Ed}$ to $N_{t, sd}$ in the text.v

i)v *Clause 5.9.3.2(1)*
The values for η_0, γ_0 and ε_0 shall be taken as 1.0 and not the alternative higherv figures.v

The values for W_y and W_z should be based on elastic gross section.v

j)v *Clause 5.9.3.2(2)*
Replace "5.9.4(5) or (6) with "5.9.4.5 or 5.9.4.6" in the definition for ωo.v

k)v *Clause 5.9.4.5(1)*
Change the reference to clause "5.4.2" for P_{HAZ} to clause "5.5.2".v

l)v *Clause 5.10.5(3) a).*
Insert the equationv
$V_{RD} = A_{net}\ fv/_{\gamma m1}$ below the heading "clause 1, 2 or 3 cross sections".v

m)v *Clause 5.11.5(1)*
Add the following conditionsv

c)v The pitch W of the stiffeners or corrugations does not exceed 0.3L (seev Figure 5.13).v

d)v Any outstand element is classified as compact in terms of axial resistance.v

e)v Any internal element is classified as compact in terms of shear resistance.v

n)v *Clause 5.11.5(3)*
Change the cross reference in the term for I_z from "5.12.7(3)" to 5.12.10.3(3)"v

6.1.4 Chapter 6 Connections Subject to Static Loading

a)v *Clause 6.5.4(3)*
Change reference to "6.5.10" to "6.5.11".v

b)v *Clause 6.5.5(7)*
Change reference to "7.5.2(1)" to "7.3.6".v

c)v *Clause 6.5.14.2(1)*
Add figure 1.0 below.v

d)v *Clause 6.6.3.1 and Table 6.*8v
The designer should be wary of specifying filler metal purely on the grounds ofv increased strength. (Refer to pr EN 1011-4 for further guidance on suitable fillerv metal selection.)v

If in doubt use the lower figure.v

e)v *Clause 6.6.3.2.1(1)*
Change "shall be applied" to "are preferable".v

f)v *Clause 6.6.3.3(3)*
Change "100 times the throat thickness"v
to "10 times the throat thickness"v

g) *Clause 6.6.3.3(6)*
Change the formula for $L_{W,eff}$ to $L_{W,eff}$ = v (1.125 – 1.25L_W/100a) L_w)v

h) *Clause 6.6.3.3(11)*
Change equations (6.38) and (6.39) tov

$$\sigma_c \leq \frac{0.85\, f_{wv}}{\gamma_{Mwv}}$$

$$\sigma_\perp \leq \frac{0.85\, f_w}{\gamma_{Mw}}$$

j) *Clause 6.6.3.3. (12)*
Change equations (6.40) and (6.42) tov

$$a > \frac{0.7}{0.85} \quad \frac{\sigma_t}{f_w/\gamma_{Mw}}$$

$$a > \frac{\tau\ tv}{f_w / \gamma_{Mwv}}$$

TABLE 2. Permitted British Standards for Alloys

BS 1470:1987 Wrought aluminium and aluminium alloys for general engineeringv purposes: plate, sheet and strip.v

BS 1471:1972 Specification for wrought aluminium and aluminium alloys- Drawn tube.v

BS 1472:1972 Specification for wrought aluminium and aluminium alloys- Forgingv stock and forgingsv

BS 1474:1987 Specification for wrought aluminium and aluminium alloys for generalv engineering purposes: bars, extruded round tubes and sectionsv

BS 1475:1972 Specification for wrought aluminium and aluminium alloys- Wirev

BS 1490:1988 Specification for aluminium and aluminium alloy ingots and castings forv general engineering purposesv

BS 4300-1:1967 Specification (supplementary series) for wrought aluminium andv aluminium alloys Part 1: Aluminium alloy longitudinally welded tubev

BS 4300-4:1973 Specification (supplementary series) for wrought aluminium andv aluminium alloys for general engineering purposes- Part 4:6463 Solid extruded bars andv sections suitable for bright trim/reflector applicationsv

BS 4300-5:1973 Specification (supplementary series) for wrought aluminium andv aluminium alloys for general engineering purposes- Part 5:2011 Free cutting bar and wirev

BS 4300-12:1969 Specification (supplementary series) for wrought aluminium andv aluminium alloys- Part 12:5454 Bar, extruded round tube and sectionsv

BS 4300-15:1973 Specification (supplementary series) for wrought aluminium andv aluminium alloys Part 15:7020 Bar, extruded round tube and sectionsv

Table 3 Reference standards common to different product forms	
BS EN 515:1993 v	Aluminium and aluminium alloys- Wrought products- Temperv designations.v
BS EN 573-3:1995 v	Aluminium and aluminium alloys- Chemical composition andv form of wrought products- Part 3: Chemical composition.v
BS EN 573-4:1995	Aluminium and aluminium alloys- Chemical composition andv form of wrought products- Part 4: Forms of product.v

Table 4: Reference Standards for Aluminium and Aluminium alloy Plate, Sheet and Strip (Replaces BS 1470:1987)	
BS EN 485-1:1995	Aluminium and aluminium alloys- Sheet, strip and plate- Part 1: Technical conditions for inspection and delivery.
BS EN 485-2:1995	Aluminium and aluminium alloys- Sheet, strip and plate- Part 2: Mechanical properties.
BS EN 485-3:1994	Aluminium and aluminium alloys- Sheet, strip and plate- Part 3: Tolerances on shape and dimensions for hot rolled products.
BS EN 485-4:1994	Aluminium and aluminium alloys- Sheet, strip and plate- Part 4: Tolerances on shape and dimensions for cold rolled products.

Table 5: Reference Standards for Aluminium and Aluminium Alloy Drawn Tube. (Replaces BS 1471: 1972)	
BS EN 754-1:1997	Aluminium and aluminium alloys- Cold drawn rod/bar and tube- Part 1: Technical conditions for inspection and delivery.
BS EN 754-2:1997	Aluminium and aluminium alloys- Cold drawn rod/bar and tube- Part 2: Mechanical properties.
BS EN 754-7:1998	Aluminium and aluminium alloys- Cold drawn rod/bar and tube- Part 7: Seamless tubes, tolerances on dimensions and form.
BS EN 754-8:1998	Aluminium and aluminium alloys- Cold drawn rod/bar and tube- Part 8: Porthole tubes, tolerances on dimensions and form.

Table 6: Reference Standards for Aluminium an Aluminium Alloy Forging Stock and Forgings (Replaces BS 1472: 1972	
BS EN 586-1: 1998	Aluminium and aluminium alloys- Forgings- Part 1: Technical conditions for inspection and delivery.
BS EN 586-2: 1994	Aluminium and aluminium alloys- Forgings- Part 2: Mechanical properties and additional property requirements.
BS EN 603-1: 1997	Aluminium and aluminium alloys- Wrought forging stock- Part 1: Technical conditions for inspection and delivery.
BSEN 603-2: 1997	Aluminium and aluminium alloys- Wrought forging stock- Part 2: Mechanical properties.
BS EN 604-1: 1997	Aluminium and aluminium alloys- Cast forging stock- Part 1: Technical conditions for inspection and delivery.
BS EN 604-2: 1997	Aluminium and aluminium alloys- Cast forging stock- Part 2: Tolerances on dimensions and form

Table 7: Reference Standards for Aluminium and Aluminium Alloy Bars, Extruded Round Tubes and Sections. (Replaces BS 1474: 1987, BS4300-12; and BS 4300-15:1973)	
BS EN 755-1: 1997	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 1: Technical conditions for inspection and delivery
BS EN 755-2: 1997	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 2: Mechanical properties.
BS EN 755-3: 1996	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 3: Round bars, tolerances on dimensions and form.
BS EN 755-4: 1996	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 4: Square bars, tolerances on dimensions and form.
BS EN 755-5: 1996	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 5: Rectangular bars, tolerances on dimensions and form.
BS EN 755-6: 1996	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 6: Hexagonal bars, tolerances on dimensions and form.
BS EN 755-7: 1998	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 7: Seamless tubes, tolerances on dimensions and form.
BS EN 755-8: 1999	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 8: Porthole tubes, tolerances on dimensions and form.
EN 755-9:	Aluminium and aluminium alloys- Extruded rod/bar, tubes and profiles- Part 9: Profiles, tolerances on dimensions and form.
EN 12020-1	Aluminium and aluminium alloys- Extruded precision profiles in alloys EN AW 6060/EN AW 6063- Part 1: Technical conditions for inspection and delivery.
EN 12020-2	Aluminium and aluminium alloys- Extruded precision profiles in alloys EN AW 6060/EN AW 6063- Part 2: Tolerances on dimensions and form.

Table 8: Reference Standards for Aluminium and Aluminium Alloy Wire (Replaces BS 1475: 1972)	
BS EN 1301-1: 1997	Aluminium and aluminium alloys- Drawn wire- Part 1: Technical conditions for inspection and delivery.
BS EN 1301-2: 1997	Aluminium and aluminium alloys- Drawn wire- Part 2: Mechanical properties.
BS EN 1301-3: 1997	Aluminium and aluminium alloys- Drawn wire-Part 3: Tolerances on dimensions.

Table 9: Reference Standards for Aluminium and Aluminium Alloy Ingots and Castings for General Engineering Purposes. (Replaces BS 1490: 1988)	
BS EN 1559-1: 1997	Founding- Technical conditions of delivery- Part 1: General.
EN 1559-4:	Founding- Technical conditions of delivery- Part 4: Additional requirements for aluminium castings (under preparation).
BS EN 1676: 1997	Aluminium and aluminium alloys- Alloyed ingots for remelting- Specifications.
BS EN 1706: 1998	Aluminium and aluminium alloys- Castings- Chemical composition and mechanical properties.

Table 10: Reference Standards for Aluminium and Aluminium Alloy Longitudinally Welded Tube. (Replaces BS 4300-1: 1967)	
BS EN 1592-1: 1998	Aluminium and aluminium alloys- HF seamwelded tubes- Part 1: Technical conditions for inspection and delivery.
BS EN 1592-2: 1998	Aluminium and aluminium alloys- HF seamwelded tubes- Part 2: Mechanical properties.
BS EN 1592-3: 1998	Aluminium and aluminium alloys- HF seamwelded tubes- Part 3: Tolerances on dimensions and form of circular tubes.
BS EN 1592-4: 1998	Aluminium and aluminium alloys- HF seam welded tubes- Part 4: Tolerances on dimensions and form for square, rectangular and shaped tubes.

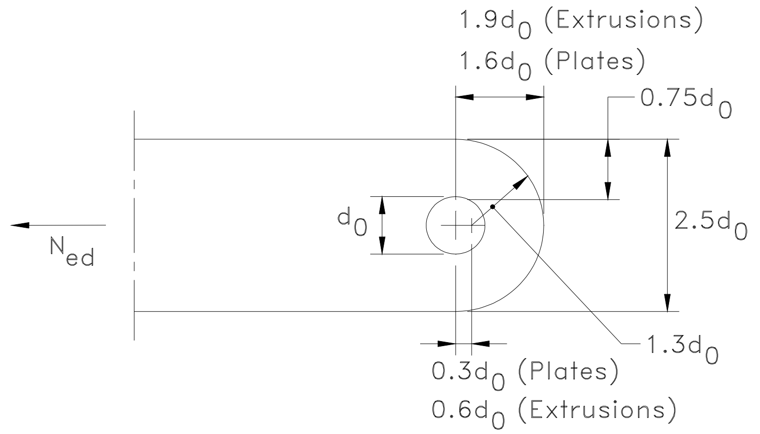
Table 11: Reference Standards for Aluminium and Aluminium Alloy Solid Extruded Bars and Sections suitable for bright trim/reflector applications. (Replaces BS 4300-4: 1973)	
BS EN 755-1: 1997	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 1: Technical conditions for inspection and delivery.
BS EN 755-2: 1997	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 2: Mechanical properties.

Table 11: Reference Standards for Aluminium and Aluminium Alloy Solid Extruded Bars and Sections suitable for bright trim/reflector applications. (Replaces BS 4300-4: 1973) (Cont'd)	
BSEN 755-3: 1996	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 3: Round bars, tolerances on dimensions and form.
BS EN 755-4: 1996	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 4: Square bars, tolerances on dimensions and form.
BS EN 755-5: 1996	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 5: Rectangular bars, tolerances on dimensions and form.
BS EN 755-6: 1996	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 6: Hexagonal bars, tolerances on dimensions and form.
EN 755-9	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 9: Profile, tolerances on dimensions and form.

Table 12: Reference Standards for Aluminium and Aluminium Alloy Free Cutting Bar and Wire. (Replaces BS 4300-5: 1973).	
BS EN 754-1: 1997	Aluminium and aluminium alloys- Cold drawn rod/bar and tube- Part 1: Technical conditions for inspection and delivery.
BS EN 754-2: 1997	Aluminium and aluminium alloys- Cold drawn rod/bar and tube- Part 2: Mechanical properties.
BS EN 754-3: 1996	Aluminium and aluminium alloys- Cold drawn rod/bar and tube- Part 3: Round bars, tolerances on dimensions and form.
BS EN 754-6: 1996	Aluminium and aluminium alloys- Cold drawn rod/bar and tube- Part 6: Hexagonal bars, tolerances on dimensions and form.
BS EN 755-1: 1997	Aluminium and aluminium alloys- Extruded rod/bar, tubes and profiles- Part 1: Technical conditions for inspection and delivery.
BS EN 755-2: 1997	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 2: Mechanical properties.
BS EN 755-3: 1996	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 3: Round bars, tolerances on dimensions and form.
BS EN 755-6: 1996	Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles- Part 6: Hexagonal bars, tolerances on dimensions and form.

FIGURE 1.0 Geometrical conditions in pin connections

Given geometry



$$t \geq 0.7 \left[\frac{N_{ed}\;\gamma_{mp}}{f_u}\right]^{1/2} \quad : \quad d_0 \leq 2.5t$$

EUROPEAN PRESTANDARD
PRÉNORME EUROPÉENNE
EUROPÄISCHE VORNORM

ENV 1999-1-1

May 1998

ICS 91.010.30; 91.080.10

Descriptors: civil engineering, steel construction, aluminium, design, building codes, computation, generalities

English version

Eurocode 9: Design of aluminium structures - Part 1-1: General rules - General rules and rules for buildings

Eurocode 9: Conception et dimensionnement des structures en aluminium - Partie 1-1: Règles générales - Règles générales et règles pour les bâtiments

Eurocode 9: Bemessung und Konstruktion von Aluminiumbauten - Teil 1-1: Allgemeine Regeln - Allgemeine Bemessungsregeln und Bemessungsregeln für den Hochbau

This European Prestandard (ENV) was approved by CEN on 26 October 1997 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.

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, Czech Republic, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.

EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

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

Ref. No. ENV 1999-1-1:1998 E

Contents

Foreword

Objectives of the Eurocodes

The Structural Eurocodes comprise a group of standards for the structural and geo-technical design of buildings and civil engineering works.

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

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

- As a framework for drawing up harmonized technical specifications for construction products.

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.

Until the necessary set of harmonized technical specifications for products and for methods of testing their performance is available, some of the Structural Eurocodes cover some of these aspects in informative Annexes.

Background to the Eurocode Programme

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".

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.

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

Eurocode programme

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

EN 1991	Eurocode 1	Basis of design and actions on structures
EN 1992	Eurocode 2	Design of concrete structures
EN 1993	Eurocode 3	Design of steel structures
EN 1994	Eurocode 4	Design of composite steel and concrete structures
EN 1995	Eurocode 5	Design of timber structures
EN 1996	Eurocode 6	Design of masonry structures
EN 1997	Eurocode 7	Geo-technical design
EN 1998	Eurocode 8	Design of structures for earthquake resistance
EN 1999	Eurocode 9	Design of aluminium structures

Separate sub-committees have been formed by CEN/TC 250 for the various Eurocodes listed above.

This part of the Structural Eurocode for Design of Aluminium Structures, is being issued by CEN as a European Prestandard (ENV) with an initial life of three years.

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.

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

Meanwhile feedback and comments on this Prestandard should be sent to Secretariat of sub-committee CEN/TC 250/SC 9 at the following address:

Secretariat of CEN/TC 250/SC 9
c/o Norwegian Council for Building Standardization
Postboks 129 Blindern
N - 0314 OSLO

or to your national standards organization.

National Application Documents

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.

Many of the harmonized supporting standards 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 Organization.

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.

Matters specific to this Prestandard

General

The scope of Eurocode 9 is defined in 1.1.1 and the scope of this Part of Eurocode 9 is defined in 1.1.2.

In using this Prestandard in practice, particular regard should be paid to the underlying assumptions and conditions given in 1.4.

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

Use of Annexes

The eight sections of this Prestandard are complemented by a number of Annexes, some normative and some informative.

The normative Annexes have the same status as the sections 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.

Concept of Reference Standards

When 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.

This Prestandard mentions certain "Reference 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 section 3 will be used for buildings and civil engineering works designed to this European Prestandard.

Partial safety factors

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

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.

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

- γ_{M1} is to be applied to resistance related to the 0,2 % proof strength $f_{0,2}$ (e.g. for all instability phenomena)
- γ_{M2} is to be applied to resistance related to the ultimate tensile stress f_u (e.g. net section strength in tension or bolt and weld resistances).

Fabrication and execution

Section 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.

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.

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

Design assisted by testing

Section 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.

1 General

1.1 Scope

1.1.1 Scope of ENV 1999 Eurocode 9

(1) ENV 1999 Eurocode 9 applies to the design of buildings, civil and structural engineering works in aluminium. It is subdivided into various separate parts, see 1.1.2.

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

(3) Execution is covered to the extent that is necessary to indicate the quality of the construction material 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 and structural engineering works and methods of construction.

(4) ENV 1999 Eurocode 9 does not cover the special requirements of seismic design.

(5) Numerical values of the actions on buildings and civil and structural engineering works to be taken into account in the design are not given in ENV 1999 Eurocode 9. They are provided in ENV 1991 Eurocode 1 "Basis of design and actions on structures" which is applicable to all types of construction.

1.1.2 Scope of Part 1.1 of ENV 1999 Eurocode 9

(1) This European Prestandard gives a general basis for the design of buildings and civil and structural engineering works in aluminium alloy.

(2) The following subjects are dealt with in this initial version of this European Prestandard.

- Section 1: General
- Section 2: Basis of design
- Section 3: Materials
- Section 4: Serviceability limit states
- Section 5: Ultimate limit states (members)
- Section 6: Connections subject to static loading
- Section 7: Fabrication and execution
- Section 8: Design assisted by testing

(3) Most of the contents of Section 1 and Section 2 are common to all Structural Eurocodes, with the exception of some additional clauses which are specific to individual Eurocodes.

(4) This European Prestandard does not cover:

- resistance to fire;
- cases where special measures may be necessary to limit the consequences of accidents;
- fatigue.

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 identified by the letter P following the paragraph number.

(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) In this Eurocode the Application Rules are identified by a number in brackets, as in this paragraph.

1.3 Normative references

(1)P This European prestandard incorporates, by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed below. For undated references the latest edition of the publication referred to applies.

1.3.1 References on aluminium alloys

1.3.1.1 Chemical composition, form and temper definition of wrought products

EN 573-1:1994	Aluminium and aluminium alloys - Chemical composition and form of wrought products - Part 1: Numerical designation system.
EN 573-2:1994	Aluminium and aluminium alloys - Chemical composition and form of wrought products - Part 2: Chemical symbol based designation system
EN 573-3:1994	Aluminium and aluminium alloys - Chemical composition and form of wrought products - Part 3: Chemical compositions
EN 573-4:1994	Aluminium and aluminium alloys - Chemical composition and form of wrought products - Part 4: Forms of products
EN 515:1993	Aluminium and aluminium alloys - Wrought products - Temper designations

1.3.1.2 Technical delivery conditions

EN 485-1:1993	Aluminium and aluminium alloys - Sheet, strip and plate - Part 1: Technical conditions for inspection and delivery
prEN 586-1:1996	Aluminium and aluminium alloys - Forgings - Part 1: Technical conditions for inspection and delivery
prEN 754-1:1996	Aluminium and aluminium alloys - Cold drawn rod/bar and tube - Part 1: Technical conditions for inspection and delivery
prEN 755-1:1996	Aluminium and aluminium alloys - Extruded rod/bar, tube and profiles - Part 1: Technical conditions for inspection and delivery
prEN 1592-1:1996	Aluminium and aluminium alloys - HF seam welded tubes - Part 1: Technical conditions for inspection and delivery

prEN 12020-1:1995	Aluminium and aluminium alloys - Extruded precision profiles in alloys EN AW-6060 and EN AW-6063- Part 1: Technical conditions for inspection and delivery

1.3.1.3 Dimensions and mechanical properties

EN 485-2:1994	Aluminium and aluminium alloys - Sheet, strip and plate - Part 2: Mechanical properties
EN 485-3:1993	Aluminium and aluminium alloys - Sheet, strip and plate - Part 3: Tolerances on shape and dimensions for hot-rolled products
EN 485-4:1993	Aluminium and aluminium alloys - Sheet, strip and plate - Part 4: Tolerances on shape and dimensions for cold-rolled products
prEN 508-2:1996	Roofing products from metal sheet - Specifications for self supporting products of steel, aluminium or stainless steel - Part 2: Aluminium
EN 586-2:1994	Aluminium and aluminium alloys - Forgings - Part 2: Mechanical properties and additional property requirements
prEN 586-3:1996	Aluminium and aluminium alloys - Forgings - Part 3: Tolerances on dimension and form
prEN 754-2:1996	Aluminium and aluminium alloys - Cold drawn rod/bar and tube - Part 2: Mechanical properties
EN 754-3:1995	Aluminium and aluminium alloys - Cold drawn rod/bar and tube - Part 3: Round bars, tolerances on dimension and form
EN 754-4:1995	Aluminium and aluminium alloys - Cold drawn rod/bar and tube - Part 4: Square bars, tolerances on dimension and form
EN 754-5:1995	Aluminium and aluminium alloys - Cold drawn rod/bar and tube - Part 5: Rectangular bars, tolerances on dimension and form
EN 754-6:1995	Aluminium and aluminium alloys - Cold drawn rod/bar and tube - Part 6: Hexagonal bars, tolerances on dimension and form
prEN 754-7:1995	Aluminium and aluminium alloys - Cold drawn rod/bar and tube - Part 7: Seamless tubes, tolerances on dimension and form
prEN 754-8:1995	Aluminium and aluminium alloys - Cold drawn rod/bar and tube - Part 8: Porthole tubes, tolerances on dimension and form
prEN 755-2:1996	Aluminium and aluminium alloys - Extruded rod/bar, tube and profiles - Part 2: Mechanical properties
EN 755-3:1995	Aluminium and aluminium alloys - Extruded rod/bar, tube and profiles- Part 3: Round bars, tolerances on dimension and form
EN 755-4:1995	Aluminium and aluminium alloys - Extruded rod/bar, tube and profiles- Part 4: Square bars, tolerances on dimension and form
EN 755-5:1995	Aluminium and aluminium alloys - Extruded rod/bar, tube and profiles- Part 5: Rectangular bars, tolerances on dimension and form

EN 755-6:1995	Aluminium and aluminium alloys - Extruded rod/bar, tube and profiles- Part 6: Hexagonal bars, tolerances on dimension and form
prEN 755-7:1995	Aluminium and aluminium alloys - Extruded rod/bar, tube and profiles- Part 7: Seamless tubes, tolerances on dimension and form
prEN 755-8:1995	Aluminium and aluminium alloys - Extruded rod/bar, tube and profiles- Part 8: Porthole tubes, tolerances on dimension and form
prEN 755-9:1995	Aluminium and aluminium alloys - Extruded rod/bar, tube and profiles- Part 9: Profiles, tolerances on dimension and form
prEN 12020-2:1995	Aluminium and aluminium alloys - Extruded precision profiles in alloys EN AW-6060 and EN AW-6063- Part 2: Tolerances on dimensions and form
prEN 1592-2:1994	Aluminium and aluminium alloys - HF seam welded tubes - Part 2: - Mechanical properties
prEN 1592-3:1994	Aluminium and aluminium alloys - HF seam welded tubes - Part 3: - Tolerance on dimensions and shape of circular tubes
prEN 1592-4:1994	Aluminium and aluminium alloys - HF seam welded tubes - Part 4: - Tolerance on dimensions and form for square, rectangular and shaped tubes

1.3.1.4 Aluminium alloy castings

prEN 1559-1	Founding - Technical conditions of delivery - Part 1: General
prEN 1559-2	Founding - Technical conditions of delivery - Part 4: Additional requirements for aluminium alloy castings
prEN 1706:1993	Aluminium and aluminium alloys - Castings - Chemical composition and mechanical properties
prEN190/120	Castings - System of dimensional tolerances and machining allowances

1.3.2 References on welding

EN 287-2:1992	Approval testing of welders - Fusion welding - Part 2: Aluminium and aluminium alloys
EN 288-1:1992	Specification and approval of welding procedures for metallic materials - Part 1: General rules for fusion welding
EN 288-4:1992	Specification and approval of welding procedures for metallic materials - Part 4; Welding procedure tests for the arc welding of aluminium and its alloys
prEN 288-13	Specification and approval of welding procedures for metallic materials - Part 13 Welding procedure test for the arc welding of cast aluminium and combinations of cast to wrought materials
EN 439:1994	Welding consumables - Shielding gases for arc welding and cutting.
prEN 970	Non destructive examination of welds - Visual examination
prEN 1011-1	Welding - Fusion welding of metallic materials - Part 1: General

prEN 1011-4	Requirements for fusion welding of metallic materials - Part 4: Aluminium and aluminium alloys
prEN 1418	Welding personnel. Approval testing of welding operators for fusion welding and resistance weld setters for fully mechanised and automatic welding of metallic materials
EN 30042:1994	Arc-welded joints in aluminium and its weldable alloys - Guidance on quality levels for imperfections
prEN (WI 121127)	Welding consumables - Wire electrodes, wires and rods for arc welding of aluminium and aluminium alloys. Classification
prEN (WI 121214)	Welding consumables - Covered electrodes for manual metal arc welding of aluminium and aluminium alloys. Classification

1.3.3 Other references

ENV 1991-1: 1994	Eurocode 1 - Basis of design and actions on structures - Part 1: Basis of design
ENV 1991-2-1: 1995	Eurocode 1 - Basis of design and actions on structures - Part 2.1: Actions on structures - Densities, self-weight and imposed loads
ENV 1991-2-2: 1994	Eurocode 1 - Basis of design and actions on structures - Part 2.2: Actions on structures imposed to fire
ENV 1991-2-3: 1995	Eurocode 1 - Basis of design and actions on structures - Part 2.3: Actions on structures - Snow loads
ENV 1991-2-4: 1995	Eurocode 1 - Basis of design and actions on structures - Part 2.4: Actions on structures - Wind actions
ENV 1993-1-1: 1992	Eurocode 3: Design of steel structures - Part 1.1: General rules and rules for buildings
ENV 1999-1-2	Eurocode 9: Design of aluminium structures - Part 1.2: Structural fire design
ENV 1999-2	Eurocode 9: Design of aluminium structures Part 2: Structures susceptible to fatigue
ISO 1000:1981	SI units and recommendations for the use of their multiples and of certain other units
ISO 8930:1987	General principles on reliability for structures - List of equivalent terms
ISO 468:1982	Surface roughness. Parameters, their values and general rules for specifying requirements
ISO 11003-1:1993	Adhesives Determination of shear behaviour of structural bonds Part 1: Torsion test method using butt-bonded hollow cylinders
ISO 11003-2:1993	Adhesives Determination of shear behaviour of structural bonds Part 2: Thick-adherend tensile-test method
ISO 468:1982	Surface roughness .Parameters, their values and general rules for specifying requirement
ISO 1302:1992	Technical drawings. Method of indicating surface texture

1.4 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 (see clause 7.7).
- 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 Section 7 are also complied with.

(3) Numerical values identified by ☐ are given as indications. Other values may be specified by Member Countries.

1.5 Definitions

1.5.1 Terms common to all Structural Eurocodes

(1) Unless otherwise in the following, the terminology used in normative reference ISO 8930: 1987 applies.

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

- **construction works:** Everything that is constructed or results from construction operations. This term covers both building, civil and structural engineering works. It refers to the complete construction comprising both structural and non-structural elements.

- **execution:** The activity of creating a building or civil or structural 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:** Organised combination of connected parts designed to provide some measure of rigidity and strength. This term refers to load carrying parts.

- **type of building or civil and structural engineering works:** Type of "construction works" designating its intended purpose, e.g. dwelling house, industrial building, road bridge, railway carriage, road vehicle, waterborne structure, mast or tower.

  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, tubular structure, arch.

- **construction material:** A material used in construction work, e.g. concrete, steel, timber, masonry, aluminium.

- **type of construction:** Indication of principal structural material, e.g. reinforced concrete construction, steel construction, timber construction, masonry construction, aluminium construction.

- **method of construction:** Manner in which the construction will be carried out, e.g. site welded, prefabricated, cantilevered.

- **structural system:** The load bearing elements of a building or civil or structural engineering work 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.

Table 1.1: List of equivalent terms in various languages

English	Francais	Deutsch
Construction works	Construction	Bauwerk
Execution	Exécution	(Bau)-Ausführung
Structure	Structure	Tragwerk
Type of building or civil engineering works	Nature de construction	Art des Bauwerks
Form of structure	Type de structure	Art des Tragwerks
Construction material	Matériau de construction	Baustoff; Werkstoff*) (* nur im Stahlbau)
Type of construction	Mode de construction	Bauweise
Method of construction	Procédé d'exécution	Bauverfahren
Structural system	Système structural	Tragsystem

Table 1.1: List of equivalent terms in various languages (continued)

Italiano	Dutch	Espanol
Costruzione	Bouwwerk	Construcción
Esecuzione	Uitvoering	Ejecución
Struttura	Draagconstructie	Estructura
Tipo di Costruzione	Type Bouwwerk	Natureleza de la construcción
Tipo di struttura	Type draagconstructie	Tipo de estructura
Materiale da costruzione	Constructie materiaal	Material de construcción
Sistema costruttivo	Bouwwijze	Modo de construcción
Procedimento esecutivo	Bouwmethode	Procedimiento de ejecución
Sistema strutturale	constructief systeem	Sistema estructural

Table 1.1: List of equivalent terms in various languages (continued)

Portuguese	Swedish	Norwegian
Construção	Byggnadsverk	Byggverk
Execução	Utförande	Utførelse
Estrutura	Bärverk	Bærende konstruksjon
Tipo de edificio ou de obras de engenharia civil	Typ av byggnadsverk	Type byggverk
Tipo de estrutura	Typ av bärverk	Konstruksjonsform
Material de construção	Byggmaterial	Byggemateriale
Tipo de construção	Typ av konstruktion med avseende på material	Konstruksjonstype (etter hovedmateriale)
Processo construtivo	Byggnadssätt	Utførelsesmetode
Sistema estrutural	Bärande system	Bærende system

Table 1.1: List of equivalent terms in various languages (continued)

Suomi	Danish	Ελλζνικα
Rakennuskohde	Bygge- og anlægsarbejde	Κατασκειές
Työnsuoritus	Udførelse	Εκτελεση έργου
Rakenne	Bærende konstruktion	Κατασκευή
Rakennuksen tai maa- ja vesirakennuskohteen tyyppi	Arten af bygge- och anlægsarbejde	Ειδος κατασκευής
Rakenteen muoto	Konstruktionsprincip	Ειδος ξορέα
Rakennusmateriaali	Konstruktionsmateriale	Υλικο κατασκευής
Rakenteen materiaali	Konstruktionstype	Τροηος εκτελεσης
Rakentamistapa	Udførelsesmetode	Μεδοδος εκτέλεσης
Rakennejärjestelmä	Bærende system	Φορέασs

1.6 S.I. units

(1) S.I. units shall be used in accordance with ISO 1000: 1981.

(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.

1.7 Symbols used in this European Prestandard

1.7.1 Latin upper case letters

A Accidental action; Area
B Bolt force
C Capacity; Fixed value, Factor
D Damage (fatigue assessment)
E Modulus of elasticity
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

1.7.2 Greek upper case letters

Δ Difference in ... (precedes main symbol)

1.7.3 Latin lower case letters

a Distance; Geometrical data
a Throat thickness of a weld; 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; Stress gradient coefficient

h Height
i Radius of gyration; Integer
k Coefficient; Factor
l(or *ℓ* or *L*) Length; Span; System length
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
u, v, w Components of deflection
uu Major axis
vv Minor axis
xx, yy, zz Rectangular axes

1.7.4 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
ε (epsilon)	Strain;
η (eta)	Coefficient
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

1.7.5 Subscripts

A	Accidental; Area
a	Local capacity of a net section in tension or compression
a,b...	First, second ... alternative
b	Bearing, Buckling
b	Bolt; Beam; Batten
C	Capacity; Consequences
c	Cross section
c	Concrete; column
com	Compression
cr	Critical
d	Design; Diagonal
dst	Destabilizing
E	Effect of actions (with *d* or *k*)
E	Euler
eff	Effective
e	Effective (with further subscript)
el	Elastic
ext	External

f	Flange; Fastener
fic	Fictitious
g (or *gr*)	Gross
G	Permanent action
h	Height; Higher; Horizontal
haz	Heat Affected Zone
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	Overall yielding in tension and compression
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); overall buckling stability
s	Slip; Storey;
st	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	Axis of cross-section
z	Axis of cross-section
σ	Normal stress

τ	Shear stress
$\perp$	Perpendicular
//	Parallel
0,2	0,2 % proof strength

1.7.6 Use of subscripts in this European Prestandard

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

$f_{0,2}$	proof strength, simplified notation: f_0
f_{haz}	heat affected zone strength
f_u	ultimate strength
E	modulus of elasticity

(2) To avoid ambiguity, subscripts are given in full in this European Prestandard, 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: e.g. M, N, β
- variant type: e.g. *pl*, *eff*, *b*, *c*
- sense: e.g. t, v
- axis: e.g. y, z
- location: e.g. 1, 2, 3
- nature: e.g. R, E
- level: e.g. d, k
- index: e.g. 1, 2, 3

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

- Subscripts with more than one character are not sub-divided.
- Example: 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:

e.g. M, ψ

1.7.7 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 aluminium alloy I 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) A selection of aluminium alloy extruded sections is given in figure 1.1.

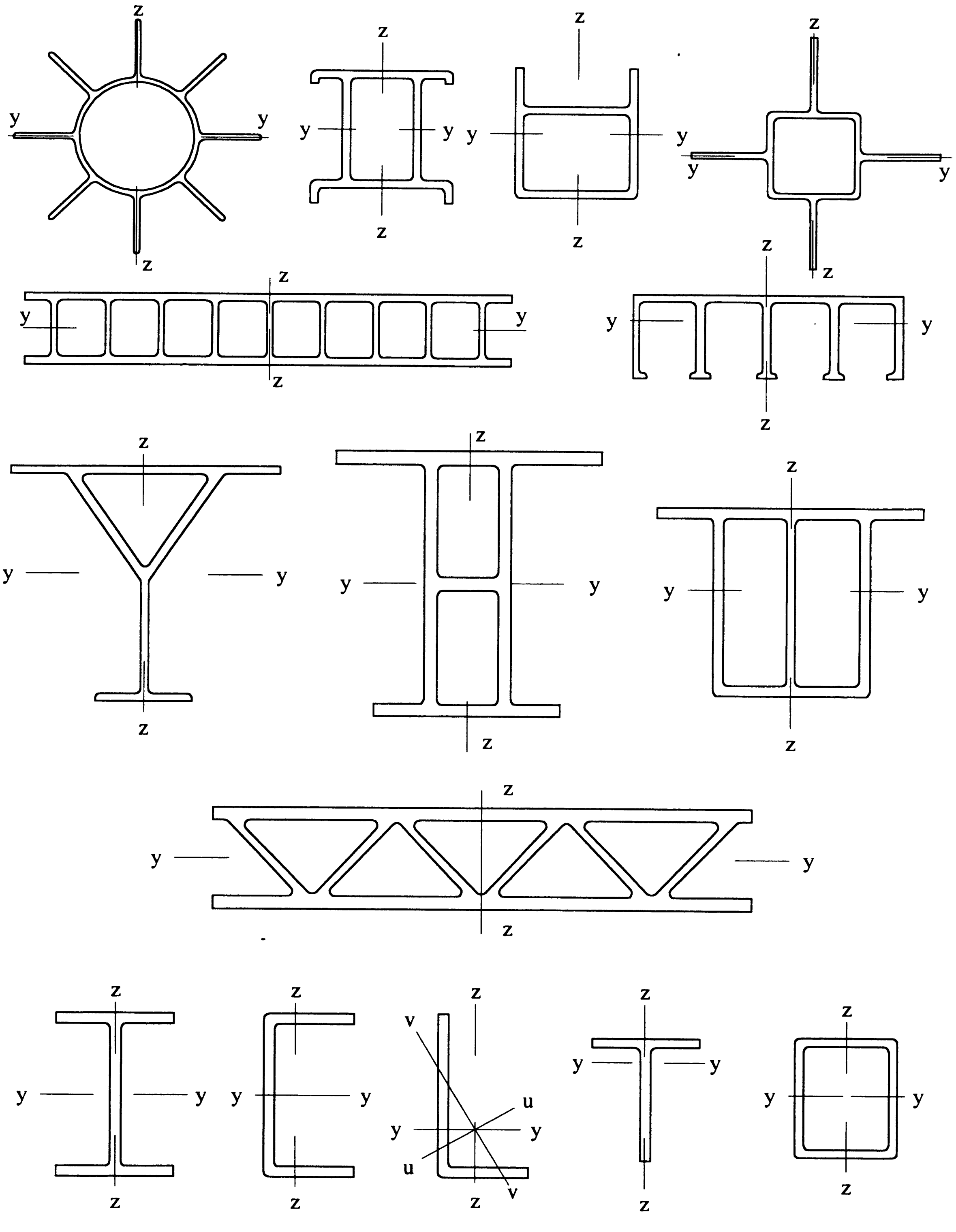


Figure 1.1: Aluminium alloy sections

(4) The convention used for subscripts which indicate axes for moments is:
"Use the axis about which the moment acts."

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

2 Basis of Design

2.1 Fundamental requirements

(1)P 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)P 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)P The potential damage shall 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
- tying the structure together
- selecting a structural form and design that can survive adequately the accidental removal of an individual element.

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

(3)P 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)P Ultimate limit states which shall 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. Accidental situations include exceptional situations that may not be the result of an accident.

2.2.2 Actions

2.2.2.1 Definitions and principal classifications[1]

(1)P 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)P Actions are classified:

a) 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, vehicle loads, wind loads, snow loads, wave loads
- accidental actions (A), e.g. explosions, impact from flying masses, impact from collisions.

b) by their spatial variation:

- fixed actions, eg 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, eg movable imposed loads, wind loads, snow loads, wave 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)P Characteristic values F_k are specified:

- in Eurocode 1 Part 2 (ENV 1991-2-1, ENV 1991-2-2, ENV 1991-2-3, ENV 1991-2-4) 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.

(2)P 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.

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[1] Fuller definitions of the classification of actions will be found in ENV 1991-1.

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

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

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

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

2.2.2.3 Representative values of variable actions[2]

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

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

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

(4)P 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.

2.2.2.4 Design values of actions

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

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

where

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

(2) Specific examples of the use of γ_F are given in ENV 1991-1.

[2] Fuller definitions of representative values will be found in ENV 1991-1.

2.2.2.5 Design values of the effects of actions

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

$$E_d = E(F_d, a_d, \ldots) \tag{2.2}$$

where

a_d is defined in 2.2.4.

2.2.3 Material properties

2.2.3.1 Characteristic values

(1)P 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 aluminium structures are generally represented by nominal values used as characteristic values.

NOTE: A guaranteed minimum value can be defined as nominal value, see 3.1. (1).

(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 0,2% proof strength, for example, should be considered in special cases where over strength effects may produce a reduction in safety.

2.2.3.2 Design values

(1)P 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)P For aluminium alloy structures, the design resistance R_d is generally determined directly from the characteristic values of the material properties and geometrical data:

$$R_d = R(X_k, a_K, \ldots) / \gamma_M \tag{2.3}$$

where

γ_M is the partial safety factor for the resistance.

(3) The characteristic value R_k may be determined from tests. Guidance is given in Section 8.

2.2.4 Geometrical data

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

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

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

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

The values of Δa are given in the appropriate clauses.

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

2.2.5 Load arrangements and load cases[3]

(1)P A load arrangement identifies the position, magnitude and direction of a free action, see ENV 1991-1.

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

2.3 Design requirements

2.3.1 General

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

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

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

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

2.3.2 Ultimate limit states

2.3.2.1 Verification conditions

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

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

where:

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

[3] Detailed rules on load arrangements and load cases are given in ENV 1991-1.

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

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

where:

E_d is the design value of an internal force or moment (or of a respective vector of several internal forces or moments)
R_d is the corresponding design resistance

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

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

(5)P When considering a limit state of rupture induced by fatigue, it shall be verified that the design value of the damage indicator D_d does not exceed unity, see ENV 1999-2.

2.3.2.2 Combinations of actions

(1)P For each load case, design values E_d for the effects of actions shall be determined from combination rules involving the design values of actions given in ENV 1991-1.

2.3.2.3 Partial safety factors for resistances (Ultimate Limit State)

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

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

(3) For fatigue verifications see prENV 1999-2.

2.3.4 Serviceability limit states

(1)P It shall be verified that

$$E_d \leq C_d \text{ or } E_d \leq R_d \tag{2.8}$$

where:

C_d is a nominal value or a function of certain design properties of materials related to the design effect of actions considered, and
E_d is the design effect of actions, determined on the basis of one of the combinations defined below.

The required combination is identified in the particular clause for each serviceability verification.

(2)P Combinations of actions for serviceability limit states are defined in ENV 1991-1.

(3) Values of γ_M should be taken as $\boxed{1{,}0}$ for all serviceability limit states, except where stated otherwise in particular clauses.

2.4 Durability

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

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

(2) The internal and external environmental conditions should be estimated at the design stage to assess their significance in relation to durability and to enable adequate provisions to be made for protection of the materials (see clause 3.4).

2.5 Fire resistance

(1) For fire resistance, refer to avENV 1999-1-2.

3 Materials

3.1 General

(1) The material properties given in this section are minimum guaranteed values as nominal values to be adopted as characteristic values in design calculations (see 5.3.5).

(2) Other material properties are given in the ENs listed in 1.3.1.3 and 1.3.1.4 and in prENs and ISO Standards.

3.2 Structural aluminium

3.2.1 Range of materials

(1) This European prestandard covers the design of structures fabricated from aluminium alloy material listed in table 3.1a for wrought alloys conforming to the ENs listed in 1.3.1.1, and for the use in structures of cast alloys in table 3.1b conforming to the ENs listed in 1.3.1.4.

Table 3.1a: Wrought aluminium alloys for structures

Alloy designation		Form of product	Durability
Numerical	Chemical symbols		
EN AW-3103	EN AW-Al Mn1	SH,ST,PL,ET	A
EN AW-5083	EN AW-Al Mg4,5Mn0,7	SH,ST,PL,ET,SEP,ER/B,DT, FO	A
EN AW-5052	EN AW-Al Mg2,5	SH,ST,PL	A
EN AW-5454	EN AW-Al Mg3Mn	SH,ST,PL	A
EN AW-5754	EN AW-Al Mg3	SH,ST,PL,FO	A
EN AW-6060	EN AW-Al MgSi	ET,EP,ER/B,DT	B
EN AW-6061	EN AW-Al Mg1SiCu	SH, ST, PL, ET,EP,ER/B,DT	B
EN AW-6063	EN AW-Al Mg0,7Si	ET,EP,ER/B,DT	B
EN AW-6005	EN AW-Al SiMg(A)	EP	B
EN AW-6082	EN AW-Al Si1MgMn	SH,ST,PL,ET,EP,ER/B,DT,FO	B
EN AW-7020	EN AW-Al Zn4,5MgCu	SH,ST,PL,ET,SEP,ER/B,DT	C

Key:
- SH - Sheet
- ST - Strip
- PL - Plate
- ET - Extruded Tube
- EP - Extruded Profiles
- SEP - Simple Extruded Profiles
- ER/B - Extruded Rod and Bar
- DT - Drawn Tube
- FO - Forgings

Table 3.1b: Cast aluminium alloys for structures

Alloy designation		Durability
Numerical	Chemical symbols	
EN AC-42100	EN AC-Al Si7Mg0,3	B
EN AC-42200	EN AC-Al Si7Mg0,6	B
EN AC-43200	EN AC-Al Si10Mg(Cu)	C
EN AC-44100	EN AC-Al Si12(b)	B
EN AC-51300	EN AC-Al Mg5	A

(2) It may also be used for other structural aluminium alloys listed in EN or ISO Standards, provided that adequate data exist to justify the application of the relevant design and fabrication rules.

(3)P If these adequate data have to be found by tests the test procedures and test evaluation shall conform with 6.5.9.5 and 6.8.3 of this European prestandard and the test requirements shall align with those in the ENs and prENs listed in 1.3.1.

(4) For advice on the selection of aluminium alloys see annex B.

3.2.2 Material properties for wrought aluminium alloys

3.2.2.1 Minimum guaranteed values

(1) The minimum limiting values of the 0,2% proof strength $f_{0,2}$ and the ultimate tensile strength f_u for wrought aluminium alloys for a range of tempers and thicknesses are given in table 3.2a for sheet, strip and plate products; table 3.2b for extruded rod/bar, extruded tube and extruded profiles and drawn tube, table 3.2c for electrically welded tube and table 3.2d for forgings.

(2) The minimum limiting values in table 3.2a to 3.2d may be adopted as characteristic values in calculations for structures subject to service temperatures below 100°C (see 5.3.5). For structures subject to elevated temperatures associated with fire see ENV 1999-1-2:1997.

Table 3.2a: Minimum guaranteed values of 0,2% proof strength $f_{0,2}$ and ultimate tensile strength f_u for wrought aluminium alloys - Sheet, strip and plate

Alloy	Temper	Thickness mm		$f_{0,2}$ 0,2% Proof strength N/mm^2	f_u Ultimate strength N/mm^2	A_{50} Minimum elongation %
		over	up to			
EN AW-3103	H14	0,2	25	120	140	2
	H16	0,2	4	145	160	1
EN AW-5052	H12	0,2	4	160	210	4
	H14	0,2	2	180	230	3
EN AW-5454	O/H111	0,2	8	85	215	12
	H24/H34	0,2	25	200	270	4
EN AW-5754	O/H111	0,2	100	80	190	12
	H24/H34	0,2	25	160	240	6
EN AW-5083	O/H111	0,2	50	125	275	11
		50	80	115	270	14
	H24/H34	0,2	25	250	340	4
EN AW-6061	T4	0,4	12,5	110	205	12
	T6	0,4	12	240	290	6
EN AW-6082	T4	0,4	12	110	205	12
	T6	0,4	6	260	310	6
		6	12,5	255	300	9
	T651	12	100	240	295	8
EN AW-7020	T6	0,4	12,5	280	350	7
	T651	12,5	40			9[1)]

[1)] Based on A not A_{50}

NOTE: The minimum elongation values do not apply across the whole range of thickness given, but generally to the thinner materials. Higher values of elongation usually apply to thicker material. For actual minimum values see the ENs and prENs listed in 1.3.1.3.

Table 3.2b: Minimum guaranteed values of 0,2% proof strength $f_{0,2}$ and ultimate tensile strength f_u for wrought aluminium alloys - Extruded profiles, extruded tube, extruded rod/bar and drawn tube

Alloy	Product form	Temper	Dimension t wall thickness or thickness mm	$f_{0,2}$ 0,2% Proof strength N/mm²	f_u Ultimate strength N/mm²	A Minimum elongation %
EN AW-5083	ET, EP,ER/B	F,H112	**$t \leq 200$**	110	**270**	12
	DT	H12, H22 H32	$t \leq 10$	200	280	6
		H14, H24 H34	$t \leq 5$	235	300	4
EN AW-6060	EP,ET,ER/B	T5	$t \leq 5$	**120**	**160**	8
	EP		$5 < t \leq 25$	100	140	8
	ET,EP,ER/B	T6	**$t \leq 15$**	**140**	**170**	8
	DT		$t \leq 20$	160	215	12
EN AW-6061	ET,EP,ER/B, DT	T6	**$t \leq 20$**	240	**260**	**8**
EN AW-6063	EP,ET,ER/B	T5	$t \leq 3$	**130**	**175**	8
	EP		$3 < t \leq 25$	110	160	**7**
	ET,EP,ER/B	T6	**$t \leq 10$**	170	215	**8**
	DT		$t \leq 20$	190	220	10
EN W-6005A	EP/O	T6	$t \leq 5$	225	270	8
			$5 < t \leq 10$	215	260	8
			$10 < t \leq 25$	200	250	8
	EP/H	T6	$t \leq 5$	215	255	8
			$5 < t \leq 15$	200	250	8
EN AW-6082	EP,ET,ER/B	T4	**$t \leq 25$**	110	205	14
	EP/O, EP/H	T5	$t \leq 5$	230	270	8
	EP/O,EP/H	T6	$t \leq 5$	250	290	8
	ET		$5 < t \leq 25$	260	310	10
	ER/B	T6	$t \leq 20$	250	295	8
			$20 < t \leq 150$	260	310	8
	DT	T6	$t \leq 5$	255	310	8
			$5 < t \leq 20$	240	310	10
EN AW-7020	EP/ER/B,DT, ET	T6	**$t \leq 15$**	**280**	350	10

Table 3.2b Continued

Key: EP	- Extruded profiles	EP/O	- Extruded open profiles
EP/H	- Extruded hollow profiles	ET	- Extruded tube
ER/B	- Extruded rod and bar	DT	- Drawn tube

NOTE 1: Where values are quoted in **bold** greater thicknesses and/or higher mechanical properties may be permitted in some forms see ENs and prENs listed in 1.3.1.3.

NOTE 2: Where minimum elongation values are given in **bold**, higher minimum values may be given for some forms or thicknesses.

NOTE 3: Where an extruded product employs thicknesses across a thickness range given above, the highest value given may be used provided the manufacturer can support the value by an appropriate quality assurance certificate.

Table 3.2c: Minimum guaranteed values of 0,2% proof strength $f_{0,2}$ and ultimate tensile strength f_u for wrought aluminium alloys - Electrically welded tube

Alloy	Temper	$f_{0,2}$ 0,2% Proof strength N/mm^2	f_u Ultimate strength N/mm^2	A Minimum elongation %
EN AW-3103	Hx65	150	170	3
	Hx85	170	190	2

Table 3.2d: Minimum guaranteed values of 0,2% proof strength $f_{0,2}$ and ultimate tensile strength f_u for wrought aluminium alloys - Forgings (L: long)

Alloy	Temper	Thickness up to mm	Direction	$f_{0,2}$ 0,2% Proof strength N/mm^2	f_u Ultimate strength N/mm^2	A Minimum elongation %
EN AW-5754	H112	150	Longitudinal (L)	80	180	15
EN AW-5083	H112	150	Longitudinal (L)	120	270	12
			Transverse (T)	110	260	10
EN AW-6082	T6	100	Longitudinal (L)	260	310	6
			Transverse (T)	250	290	5

(3) As an alternative, the limiting minimum values specified in ENs and prENs listed in 1.3.1.3 a large range of tempers and thicknesses in addition to those covered in tables 3.2a to 3.2d may be used as characteristic values.

(4) The minimum values of elongation given in table 3.2a to 3.2d are for information only.

3.2.3 Material properties for cast aluminium alloys

3.2.3.1 Minimum guaranteed values

(1) The minimum guaranteed values of the 0,2% proof strength $f_{0,2}$ and the ultimate tensile strength f_u for sand and permanent mould cast aluminium alloys test bars for a range of alloys and tempers are given in table 3.3.

Table 3.3: Minimum guaranteed values of 0,2% proof strength $f_{0,2}$ and ultimate tensile strength f_u for cast aluminium alloys - Castings

Alloy	Casting process	Temper	$f_{0,2}$ 0.2% Proof strength N/mm^2	f_u Ultimate strength N/mm^2	A_{50} Minimum elongation %
EN AC-42100	Sand cast	T6	190	230	2
	Permanent mould	T6	210	290	4
EN AC-42200	Sand cast	T6	210	250	1
	Permanent mould	T6	240	320	3
EN AC-43200	Sand cast	F	80	160	1
	Sand cast	T6	180	220	1
	Permanent mould	F	90	180	1
	Permanent mould	T6	200	240	1
EN AC-44100	Sand cast	F	70	150	4
	Permanent mould		80	170	5
EN AC-51300	Sand cast	F	90	160	3
	Permanent mould		100	180	4
NOTE: Minimum mechanical properties quoted are for separately cast test bars, and not for castings.					

(2) As an alternative, the values specified in the ENs and prENs listed in 1.3.1.4 for a range of casting processes and of other tempers may be used.

(3)P The design rules in this European prestandard shall not be applied to castings. The casting alloys given in table 3.3 should only be used in load bearing structures provided that their suitability and resistance can be determined by testing, see 8.1(2). In addition quality control procedures for the production of the castings shall be to the satisfaction of the engineer.

3.2.4 Dimensions, mass and tolerances

(1)P The dimensions and tolerances of structural extruded products, sheet and plate products, drawn tube, electrically welded tube, wire and forgings, shall conform with the ENs and prENs listed in 1.3.1.3.

(2)P The dimensions and tolerances of structural cast products shall conform with the ENs and prENs listed in 1.3.1.4.

3.2.5 Design values of material constants

(1)P The material constants to be adopted in calculations for the aluminium alloys covered by this European prestandard shall be taken as follows:

- modulus of elasticity E = 70 000 N/mm^2;
- shear modulus G = 27 000 N/mm^2;

- Poisson's ratio $\nu = 0{,}3$;
- coefficient of linear thermal expansion $\alpha = 23 \times 10^{-6}$ per °C;

- unit mass $\rho = 2\ 700$ kg/m³.

(2) For material properties in structures subject to elevated temperatures associated with fire see prEN 1999-1-2:1997.

3.3 Connecting devices

3.3.1 General

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

(2) Suitable connecting devices include bolts, friction grip fasteners, solid and hollow rivets, special fasteners, welds and adhesives.

3.3.2 Bolts, nuts and washers

3.3.2.1 General

(1)P Bolts, nuts and washers shall conform with existing ENs, prENs and ISO Standards.

(2) The minimum guaranteed values of the 0,2% proof strength $f_{0,2}$ and the ultimate strength f_u to be adopted as characteristic values in calculations, are given in table 3.4.

Table 3.4: Minimum guaranteed values of 0,2 % proof strength $f_{0,2}$ and ultimate strength f_u for Bolts, solid and hollow rivets

Material	Type of fastener	Alloy grade	Temper	$f_{0,2}$ 0,2% Proof strength N/mm²	f_u Ultimate strength N/mm²
Aluminium alloy	Solid Rivets	5056A	O	145	270
		5086	O	100	240
		6082	T4[1)]	-	200
			T6[1)]	-	295
	Hollow Rivets	5154A	O or F	-	215
	Bolts	6082	T6	260	310
		6061	T6	245	310
		2017A	T4	250	380
		7075	T6	440	510
Steel	Bolts	4.6		240	400
		5.6		300	500
		6.8		480	600
		8.8		640	800
		10.9		900	1000
Stainless Steel	Bolts	A4	A4-50	210	500
		A4	A4-70	450	700
		A4	A4-80	600	800
[1)] Cold driven					

3.3.2.2 Preloaded bolts

(1) High strength bolts may be used as preloaded bolts with controlled tightening, provided they conform with the requirements for preloaded bolts in existing ENs, prENs and ISO Standards.

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

3.3.3 Rivets

(1)P The material properties, dimensions and tolerances of aluminium alloy solid and hollow rivets shall conform with ENs, prENs or ISO Standards (if and when they are available).

(2) The minimum guaranteed values of the 0,2% proof strength $f_{0,2}$ and the ultimate strength f_u to be adopted as characteristic values in calculations, are given in table 3.4.

3.3.4 Welding consumables

(1)P All welding consumables shall conform with ENs, prENs or ISO Standards (if available) listed in 1.3.3.

Note: prEN (WI 121 127 and WI 121 214) are in preparation.

(2)P The selection of welding filler metal for the combination of alloys being joined shall be made from prEN 1011-4 table B.5 and B.6 in conjunction with the design requirements for the joint, see 6.6.3.1. Guidance on the selection of filler metal for the range of parent metals given in this document is given in tables 3.5 and 3.6.

Table 3.5: Alloy grouping used in table 3.6

Filler metal grouping	Alloys
Type 3	3103
Type 4	4043A, 4047A[1)]
Type 5	5056A, 5356, 5556A, 5183
[1)] 4047A is specifically used to prevent weld metal cracking in joints involving high dilution and high restraint. In most other cases, 4043A is preferable. NOTE: See prEN 1011-4 table B:5 for a wider range of filler metals and their characteristics.	

Table 3.6: Selection of filler metals (see table 3.5 for alloy types)

Parent metal combination [1]							
1st Part	2nd Part						
	Al-Si castings	Al-Mg castings	3000 series alloys	Other 5000 series alloys	5083	6000 series alloys	7020
7020	NR[2]	Type 5 Type 5 Type 5	Type 5 Type 5 Type 4	Type 5 Type 5 Type 5	5556A Type 5 5556A	Type 5 Type 5 Type 4	5556A Type 5 Type 4[4]
6000 series alloys	Type 4 Type 4 Type 4	Type 5 Type 5 Type 5	Type 4 Type 4 Type 4	Type 5 Type 5 Type 5	Type 5 Type 5 Type 5	Type 5 Type 4 Type 4	
5083	NR[2]	Type 5 Type 5 Type 5	Type 5 Type 5 Type 5	Type 5 Type 5 Type 5	5556A Type 5 Type 5		
other 5000 series alloys	NR[2]	Type 5 Type 5 Type 5	Type 5 Type 5 Type 5	Type 5 [3] Type 5			
3000 series alloys	Type 4 Type 4 Type 4	Type 5 Type 5 Type 5	Type 3 Type 3 Type 3				
Al-Mg castings	NR[2]	Type 5 Type 5 Type 5					
Al-Si castings	Type 4 Type 4 Type 4						

[1] Filler metals for parent metal combination to be welded are shown in one box, which is located at the intersection of the relevant parent metal row and column. In each box, the filler metal for the maximum weld strength is shown in the top line; in the case of 6000 series and 7020 alloys, this will be below the fully heat treated parent metal strength. The filler metal for maximum resistance to corrosion is shown in the middle line. The filler metal for freedom from persistant weld cracking is shown on the bottom line.

NR[2] = Not recommended. The welding of alloys containing approximately 2% or more of Mg with Al-Si filler metal, or vice-versa is not recommended because sufficient Mg_2Si precipitate is formed at the fusion boundaries to embrittle the weld. Where unavoidable see prEN 1011-4.

[3] The corrosion behaviour of weld metal is likely to be better if its alloy content is close to that of the parent metal and not markedly higher. Thus for service in potentially corrosive environments it is preferable to weld 5454 with 5454 filler metal. However, in some cases this may only be possible at the expense of weld soundness, so that a compromise will be necessary.

[4] Only in special cases due to the lower strength of the weld and elongation of the joint.

NOTE: See prEN 1011-4 table B.5 for a wider range of parent metals, filler metals and more details on their selection.

3.3.5 Adhesives

(1) The recommended families of adhesives for the assembly of aluminium structures are: single and two part modified epoxies, modified acrylics, one or two part polyurethanes; anaerobic adhesives can also be used in the case of pin- and collar-assemblies.

(2) On receipt of the adhesive, its freshness can be checked before curing by the following methods:
- chemical analysis;
- thermal analysis;
- measurements of the viscosity and of the dry extract in confirmity with existing ENs, prENs and ISO Standards related to adhesives.

(3) The strength of an adhesive joint depends on the following factors:

a) the specific strength of the adhesive itself, that can be measured by standardized tests (see ISO 11003-2);

b) the alloy, and especially its proof stress when the yield stress of the metal is exceeded before the adhesive fails;

c) the surface pre-treatment: chemical conversion and anodizing generally give better long term results than degreasing and mechanical abrasion; the use of primers is possible provided that one makes sure that the primer, the alloy and the adhesive are compatible by using bonding tests;

d) the environment and the ageing; the presence of water or damp atmosphere or aggressive environment can drastically lower the long term performance of the joint (especially in the case of poor surface pre-treatments);

e) the configuration of the joint and the related stress distribution, i.e. the ratio of the maximum shear stress τmax to the mean one (τ_{max}/τ_{mean}) and the ratio of the maximum peel stress σmax to the mean shear one (σ_{max}/τ_{mean}), both maxima occuring at the end of the joint; the stress concentrations should be reduced as much as possible; they depend on the stiffness of the assembly (thickness and Young's modulus of the adherent) and on the overlap length of the joint.

(4)P Knowledge of the specific strength of the adhesive is not sufficient to evaluate the strength of the joint, one must evaluate it by laboratory tests taking into account the whole assembly, i.e. the combinations of alloy/pre-treatment/adhesive, and the ageing or environment (see 6.8.3 and 8).

(5)P The strength obtained on specimens at the laboratory should be used as guidelines; one must check the joint performances in real conditions: the use of prototypes is recommended (see 6.8.3).

3.4 Durability and corrosion protection

3.4.1 General

(1) In many instances the standard materials listed in tables 3.1a and 3.1b can be used in the mill-finish, as extruded or as welded or as cast condition without the need for surface protection.

(2) The good corrosion resistance of aluminium and its alloys is attributable to the protective oxide film which forms on the surface of the metal immediately on exposure to air. This film is normally invisible, relatively inert and as it forms naturally on exposure to air or oxygen, and in many complex environments containing oxygen; the protective film is thus self sealing.

(3) In mild environments an aluminium surface will retain its original appearance for years, and no protection is needed for most alloys. In moderate industrial conditions there will be a darkening and roughening of the surface. As the atmosphere becomes more aggressive such as in certain strongly acidic or strongly alkaline environments, the surface discoloration and roughening will be worse with visible white powdery surface oxides and the oxide film may itself be soluble. The metal ceases to be fully protected and added protection is necessary. These conditions may also occur in crevices due to high local acid or alkaline conditions, but agents having this extreme effect are relatively few in number.

(4) In coastal and marine environments the surface will roughen and acquire a grey, stone-like, appearance, and protection of some alloys is necessary. Where aluminium is immersed in water special precautions may be necessary.

(5) Where surface attack does occur corrosion time curves for aluminium and aluminium alloys usually follow an exponential form, with an initial loss of reflectivity after slight weathering. After this there is very little further change over very extensive periods. On atmospheric exposure, the initial stage may be a few months or two to three years, followed by little, if any, further change over periods of twenty, thirty or even eighty years. Such behaviour is consistent for all external freely exposed conditions and for all internal or shielded conditions, except where extremes of acidity or alkalinity can develop. Tropical environments are in general no more harmful to aluminium than temperate environments, although certain 5000-alloys are affected by long exposure to high ambient temperatures, particularly when in marine environment.

3.4.2 Durability

(1) The aluminium alloys listed in tables 3.1a and 3.1b are categorised into three durability ratings; A (excellent) , B and C in descending order of durability. These ratings are used to determine the need and degree of protection required. In constructions employing more than one alloy, including filler metals in welded constructions, the protection should be in accordance with the lowest of their durability ratings.

(2)P Where other structural aluminium alloys listed in the standards listed in 1.3.1 are used, adequate data shall be sought to give the alloy a durability category so as to justify the application.

(3) For advice on the durability of aluminium alloys see annex B

3.4.3 Corrosion protection

3.4.3.1 Overall corrosion protection

(1) The need to provide overall corrosion protection to structures constructed from the alloys or combination of alloys listed in tables 3.1a and 3.1b when exposed to different environments is given in table 3.7. The methods of providing corrosion protection in these environments are detailed in 7.7. For the protection of sheet used in roofing and siding see prEN 508-2:1996.

Table 3.7: General corrosion protection of aluminium structures

Alloy durability rating	Material thickness mm	Protection according to the environment							
		Atmospheric						Immersed	
		Rural	Industrial/urban		Marine			Fresh - water	Sea water
			Moderate	Severe	Non-industrial	Moderate	Severe		
A	All	0	0	P	0	0	P	0	(P)
B	< 3	0	(P)	P	(P)	(P)	P	P	P
	≥ 3	0	0	P	0	(P)	P	(P)	P
C	All	0	(P)[2]	P	(P)[2]	(P)[2]	P	(P)[1]	NR

0 Normally no protection necessary
P Protection normally required except in special cases, see 3.4.3.1, to be decided by the designer
(P) The need for protection depends on the special conditions of the structure, see 3.4.3.1, to be decided by the designer
NR Immersion in sea water is not recommended

[1] For 7020, protection only required in Heat Affected Zone (HAZ) if heat treatment not applied after welding

[2] When heat treatment of 7020 after welding is not applied, the need to protect the HAZ shall be checked with respect to conditions, see 3.4.3.1

NOTE: For the protection of sheet used in roofing and siding see prEN 508-2:1996.

(2) In selecting the appropriate column of table 3.7 for an atmospheric environment it must be remembered that there may be localities within a region that have 'microclimates' significantly different from the environmental characteristics of the region as a whole. A region designated 'rural' may have local environments more closely resembling an industrial atmosphere at sites close to and down wind of factories. Similarly, a site near the sea but close to shore installations may, with the appropriate prevailing winds, have the characteristics of an industrial, rather than marine, atmosphere. The environment is not necessarily the same for a structure inside a building as for one outside.

(3) The occurrence of corrosion depends not only on the susceptibility of the material and the global conditions, but in practice more on the period of time during which moisture may be present in conjunction with entrapped dirt and corrosive agents. Areas of members, or structural details, where dirt is trapped or retained are more critical than areas where rain, and wind driven rain, cleans the surface and drying occurs quickly.

(4) In assessing the need and level of protection required the design life history of the structure should be considered. For short life structures less stringent measures or no protection may be acceptable. Where planned inspection and maintenance will reveal the onset of corrosion at an early stage, so allowing remedial action to be taken, the initial level of protection provided may be permitted to be relaxed. Whereas, where inspection is impractical and evidence of corrosion attack will not be revealed, the initial level of protection must be higher. Therefore the need for protection in those cases marked (P) on table 3.7 should be established in conjunction with the engineer, manufacturer and if necessary a corrosion specialist.

(5) Because of these factors, localised conditions of increased severity may result. It is advisable to study the precise conditions prevailing at the actual site before deciding on the appropriate environment column of table 3.7.

(6)P Where hollow sections are employed consideration shall be given to the need to protect the internal void to prevent corrosion arising from the ingress of corrosive agents. Because of the difficulty of painting such sections, chemical conversion coatings may be of benefit. Where the internal void is sealed effectively, internal protection is not necessary.

3.4.3.2 Metal to metal contacts including joints

(1)P Consideration shall also be given to contacting surfaces in crevices and contact with certain metals or washings from certain metals which may cause electrochemical attack of aluminium. Such conditions can occur within a structure at joints. Contact surfaces and joints of aluminium to aluminium or to other metals and contact surfaces in bolted, riveted, welded and high strength fiction grip bolted joints should be given additional protection to that required by table 3.7 as defined in table 3.8. Details of the corrosion protection procedure required are given in 7.7.3. For the protection of metal to metal contacts including joints for sheet used in roofing and siding see prEN 508-2:1996.

3.4.3.3 Contact with other non-metallic materials

(1) Contact with concrete, masonry or plaster.
Aluminium in contact with dense compact concrete, masonry or plaster in a dry unpolluted or mild environment should be coated in the contacting surface with a coat of bituminous paint, or a coating providing the same protection. In an industrial or marine environment the contacting surface of the aluminium should be coated with at least two coats of heavy duty bituminous paint; the surface of the contacting material should preferably be similarly painted. Submerged contact between aluminium and such materials is not recommended, but if unavoidable, separation of the materials is recommended by the use of a suitable mastic or a heavy duty damp course layer.

Lightweight concrete and similar products require additional consideration when water or rising damp can extract a steady supply of aggressive alkali from the cement. The alkali water can then attack aluminium surfaces other than the direct contact surfaces.

(2) Embedment in concrete
The aluminium surfaces before embedment should be protected with at least two coats of bituminous paint or hot bitumen, and the coats should extend at least 75 mm above the concrete surface after embedment.

Where the concrete contains chlorides (e.g. as additives or due to the use of sea-dredged aggregate), at least two coats of plasticised coal-tar pitch should be applied in accordance with the manufacturer's instructions and the finished assembly should be over-painted locally with the same material, after the concrete has fully set, to seal the surface. Care should be taken where metallic contact occurs between the embedded aluminium parts and any steel reinforcement.

(3) Contact with timber
In an industrial, damp or marine environment the timber should be primed and painted in accordance with good practice.

Some wood preservatives may be harmful to aluminium. The following preservatives are generally accepted as be safe for use with aluminium without special precautions:

- Coal tar creosote; coal tar oil; chlorinated napthalenes; zinc napthanates; pentachorophenol; organo-tin oxides; orthophenylphenol; fluro-chrome-arsenate-dinitrophenol.

The following preservatives should only be used in dry situations and where the aluminium surface in contact with the treated timber has a substantial application of sealant.

- Copper napthenate; copper-chrome; copper-chrome-arsenate; borax-boric acid.

The following preservatives should not be used in association with aluminium:

- zinc chloride; mercury salts; copper sulphate

Oak, chestnut and western red cedar, unless well seasoned, are likely to be harmful to aluminium, particularly where these are through fastenings.

(4) Contact with soils
The surface of the metal should be protected with at least two coats of bituminous paint, hot bitumen, or plasticised coal tar pitch. Additional wrapping-tapes may be used to prevent mechanical damage to the coating.

(5) Immersion in water
Where aluminium parts are immersed in fresh or sea water including contaminated water, the aluminium should preferably be of durability rating A, with fastenings of aluminium or corrosion-resisting steel or fastened by welding. Tables 3.7 and 3.8 give the protection requirements for fresh water and sea water immersion.

In addition the engineer should obtain competent advice on the oxygen content, pH number, chemical or metallic, particularly copper, content and the amount of movement of the water as these factors may affect the degree of protection required.

(6) Contact with chemicals used in the building industry
Fungicides and mould repellents may contain metal compounds based on copper, mercury, tin and lead which, under wet or damp conditions could cause corrosion of the aluminium. The harmful effects may be countered by protecting the contacting surfaces which may be subject to washing or seepage from the chemicals.

Some cleaning materials can affect the surface of the aluminium. Where such chemicals are used to clean aluminium or other materials in the structure, care should be taken to ensure that the effects will not be detrimental to the aluminium. Often quick and adequate water rinsing will suffice, while in other situations temporary measures may be necessary to protect the aluminium from contact with the cleaners.

(7) Contact with insulating materials used in the building industry
Products such as glass fibre, polyurethane and various insulation products may contain corrosive agents which can be extracted under moist conditions to the detriment of the aluminium. Insulating materials should be tested for compatibility with aluminium under damp and saline conditions. Where there is doubt a sealant should be applied to the associated aluminium surfaces.

3.5 Selection criteria for aluminium alloys

(1) The choice of an aluminium alloy or alloys for any structure is determined by a combination of a number of factors: strength, durability, physical properties, weldability, formability and availability in both the particular form and alloy required. The materials listed in tables 3.1a and 3.1b are described in annex B in terms of the above factors.

Table 3.8: Additional protection at metal-to-metal contacts to combat crevice and galvanic effects

Metal to be joined to aluminium	Bolt or rivet material	Protection according to environment																	
		Atmospheric								Marine						Immersed			
		Rural				Industrial urban				Non industrial		Industrial				Fresh water		Sea water	
		Dry, Un-polluted		Mild		Moderate		Severe				Moderate		Severe					
(M)	(B/R)	M	B/R	M	B/R	M	B/R	M	B/R	M	B/R	M	B/R	M	B/R	M	B/R	M	B/R
	Aluminium		0		0		1		1 3		1 (3)		1 (3)		1 3		2 3		2 3
Aluminium	Stainless steel	O	0	O	0	O/X	0	X	3	O/X	(3)	O/X	3	X	3	X	3	X	3 4
	Zinc-coated steel		0		(4)		(3) (4)	a/g	3 (4)		(3) (4)	a	(3) (4)	a/z/g	3 (4)		3 4	(g)	3 4
Zinc-coated steel	Aluminium		0		0		1		1 3		1 (3)		1 (3)		1 3		2 3		2 3 4
	Stainless steel	O	0	O	0	O/X	0	X	3	O/X	0	O/X	(3)	X	3	X	3 (4)	Y (Z)	3 4
Painted steel	Zinc-coated steel		0		(4)	a	(4)	a/z/g	3 (4)	a	(3) (4)	a	3 (4)	a/z/g	3 (4)	z	3 4	z/(g)	3 4
Stainless steel	Aluminium		0		0		1		1 3		1 (3)		1 (3)		1 3		2 3 4		2 3 4
	Stainless steel	O	0	O	0	O/X	0	X	3	O/X	0	O/X	(3)	X	3	Y (X) (Z)	3	Y (Z)	3 4
	Zinc-coated steel		0		(4)	a	(4)	a/z/g	3 (4)	a	(3) (4)	a	(3) (4)	a/g	3 (4)		3 4	(g)	3 4

NOTE 1: Details of the corrosion protection procedures indicated by O, X, Y, Z, 0, 1, 2, 3, 4, and a, z, g above, are given in 7.7.3
NOTE 2: Where more than one procedure is given all shall apply as appropriate
NOTE 3: For the proctection of sheet used in roofing or siding see prEN 508-2:1996
NOTE 4: Values in ( ) to be decided by the designer
NOTE 5: For stainless steels see also Eurocode 3 Part 1.4.

4 Serviceability Limit States

4.1 Basis

(1) The Serviceability Limit States for structural aluminium are:

- deformations or deflections that adversely affect the use of the structure (including the proper functioning of machines or services)

- deformations or deflections that cause damage to finishes or non-structural elements

- deformations or deflections that adversely affect the appearance of the structure

- shape distortion due to the gradual build-up of dimensional changes in structures that have to be assembled and disassembled frequently

- vibrations that cause damage to finishes or non-structural elements

- vibrations that cause discomfort to the users of structures or damage to equipment supported by the structure.

4.2 Deflections

4.2.1 Limiting values of deflection

(1) The limiting values of deflection should be agreed between the designer, the client and the competent authority.

(2) In the absence of a special agreement between designer and client a structure may be deemed to be acceptable in terms of deformation if the limits given hereafter are satisfied.

4.2.2 Irrecoverable deflections

(1) It may be noted that components whose static strength has been calculated in accordance with section 5 of this European prestandard will not suffer significant permanent deformation under the action of rare load combination. This applies to all alloy groups. Attention should be given to hybrid girders.

4.2.3 Recoverable elastic deflections

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

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

where:

δ_{max} is the sagging in the final state relative to the straight line joining the supports
δ_0 is the pre-camber (hogging) of the beam in the unloaded state, (state 0)
δ_1 is the variation of the deflection of the beam due to the permanent loads immediately after loading, (state1)
δ_2 is the variation of the deflection of the beam due to the variable loading plus any time dependent deformations due to the permanent load, (state 2).

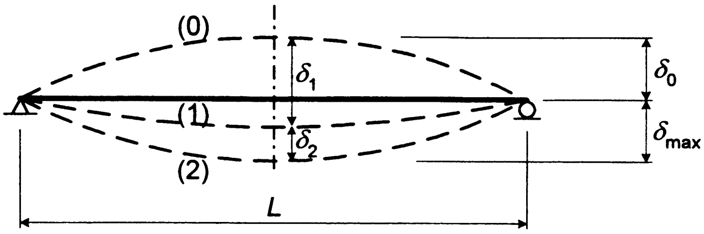


Figure 4.1: Vertical deflections to be considered

(2) For building structures the following values should not be exceeded. The elastic deflections should be determined for frequent load combinations:

(3) Cantilevers carrying floors: $\boxed{L/180}$

Beams carrying plaster or other brittle finish: $\boxed{L/360}$

Purlins and sheeting rails:
a) under dead load only: $\boxed{L/200}$
b) under the worst combination of dead, imposed, wind and snow loads: $\boxed{L/100}$

Curtain wall mullions and transoms: $\boxed{L/250}$ or $\boxed{15\ \text{mm}}$
whichever is less. Design shall ignore increase of stiffness by glass products.

NOTE: In order to prevent that the durability of glass products and their performances will be affected negatively, no higher values of deflection should be taken.

Tops of columns: horizontal deflection $\boxed{L/300}$

(where L is the length between supports).

4.2.4 Calculation of elastic deflection

(1) If the real stiffness is not taken into account iteratively in calculating deflections the following simplified procedure can be used. The calculation of elastic deflection should generally be based on the properties of the gross cross section of the member. However, for slender sections it may be necessary to take reduced section properties to allow for local buckling (see section 5.4.5). Due allowance of effects of partitioning and other stiffening effects, second order effects and changes in geometry should also be made.

(2) Advantage may be taken from reduced stress levels for class 4 section to calculate the effective thickness, using the following fictitious second moment of area I_{fic}, constant along the beam

$$I_{fic} = I_{gr} - \frac{\sigma_{gr}}{f_o}(I_{gr} - I_{eff}) \tag{4.2}$$

where:

I_{gr} is the second moment of area of the gross cross section
I_{eff} is the second moment of area of the effective cross section in the ultimate limit state, with allowance for local buckling, see 5.4.5

σ_{gr} is the maximum compressive bending stress in the serviceability limit state, based on the gross cross section (positive in the formula).
f_o is the characteristic strength for bending and overall yielding (section 5.3.5).

(3) Deflections should be calculated making also due allowance for the rotational stiffness of any semi-rigid joints, and the possible recurrence of local plastic deformation at the serviceability limit state.

4.3 Vibrations

4.3.1 Resonance

(1)P The natural frequencies of structures or structural components shall be sufficiently different from those of the excitation source to avoid resonance. In checking for the incompatibility of vibration amplitudes nominal loads shall be used. If vibration is thought to be a potential problem, particularly gust and vortex induced vibration due to wind, the possibility of fatigue failure shall be examined.

4.3.2 Damping

(1) When the effects of vibrations are assessed, the damping characteristics of structural forms and materials should be taken into account. The need to provide artificial damping should be examined, and tests on prototype components may be necessary.

4.3.3 User discomfort

(1) The vibration of structures at low natural frequencies can cause discomfort to users and should be considered in the design.

(2) The lowest natural frequency of supporting structures over which the general public walks (e.g. floors, footbridge decks, gangways) should not be lower than $\boxed{3}$ cycles/second. This limitation may be relaxed if the structure is highly damped.

(3) The lowest natural frequency of supporting structures on which people dance or jump about rhythmically should not be less than $\boxed{5}$ cycles/second.

4.4 Dynamic Effects

4.4.1 Dynamic loading

(1) The serviceability limit states for deformation or deflections apply to dynamic as well as statically applied loading. Forces from dynamic effects are treated as imposed loads when choosing load factors. If a 'dynamic magnification factor' is used the designer should be aware that this procedure is not recommended if it does not take the response of the structure into account.

5 Ultimate Limit States (Members)

5.1 Basis

5.1.1 General

(1)P Aluminium structures and components shall be proportioned so that the basic design requirements for the ultimate limit state given in Section 2 are satisfied. The basic design requirements for fatigue are given in Part 2. The design recommendations are for structures subjected to normal atmospheric conditions.

(2)P The partial safety factor γ_M shall be taken as follows for welded, riveted, bolted or bonded members:

- resistance of class 1 cross sections:[1] $\gamma_{M1} = \boxed{1,10}$

- resistance of class 2 or 3 cross sections:[1] $\gamma_{M1} = \boxed{1,10}$

- resistance of class 4 cross sections:[1] $\gamma_{M1} = \boxed{1,10}$

- resistance of member to buckling: $\gamma_{M1} = \boxed{1,10}$

- resistance of net section at bolt holes: $\gamma_{M2} = \boxed{1,25}$

The values of γ_M for riveted, bolted, welded or bonded joints are given in Section 6 (connections subject to static loading).

(3) The rules given for the design of members presume that the member forces and moments have previously been obtained from an appropriate form of overall structural analysis. Guidance on methods of analysis is provided in Section 5.2.

5.1.2 Tension members

(1)P Tension members shall be checked for the resistance of the cross section (see 5.7).

5.1.3 Compression members

(1)P Compression members shall be checked for the resistance of the cross section and the resistance to buckling (see 5.8).

5.1.4 Beams

(1)P Members subjected to bending shall be checked for:

- resistance to bending (5.6.2)
- resistance to shear (5.6.3)
- resistance to combined bending and shear (5.6.4)
- resistance to web bearing (5.6.5)
- resistance to lateral-torsional buckling (5.6.6)

5.1.5 Members with combined axial force, moment and shear

(1)P Members subjected to combined axial force, moment and shear shall be checked for the resistance of

[1] For classification of cross sections see 5.4

cross sections to combined actions and the resistance to buckling under combined actions (see 5.9).

5.1.6 Frameworks

(1)P Frameworks shall be checked for:

- resistance of members
- resistance of connections between members (Section 6)
- resistance to overall buckling.

5.1.7 Plating

(1)P Unstiffened and stiffened plating not forming part of plate girders shall be checked for:

- resistance of cross sections under uniform compression (5.10.2 and 5.11.2)
- resistance under in-plane moments or longitudinal stress gradient (5.10.3, 5.10.4, 5.11.3, 5.11.4)
- resistance under shear (5.10.5 and 5.11.5)
- resistance of cross sections to combined actions and the resistance to buckling under combined actions (5.10.6).

(2)P Plate girders shall be checked for:

- resistance of longitudinally and/or transversely stiffened plate girders to in-plane bending, shear and combined effects of bending, shear and axial forces (5.12.2, 5.12.3, 5.12.4 and 5.12.8)
- resistance of web stiffeners (5.12.5)
- resistance of webs to transverse forces applied through the flange (5.12.6)
- resistance of corrugated or closely-stiffened webs (5.12.7)
- resistance of girders with stiffeners at the supports (5.12.3).

5.1.8 Connections (see Section 6)

(1)P Connections between structural members, or between the elements of a 'built-up' member, or between local details and structural members, shall be designed to meet the ultimate limit states of static strength and fatigue. Connections shall be checked for:

- resistance of fasteners, rivets and bolts (in non-welded designs) in shear, axial tension, combined shear and tension, and in bearing
- resistance of high strength friction-grip bolts (when used) in terms of friction capacity, prestress and slip factor
- resistance of pins (in pinned joints) in shear or bending
- resistance of welds (in welded designs), in shear or under normal stresses for butt welds and fillet welds
- resistance of heat affected zones (HAZ) adjacent to welds
- resistance of bonded joints when bonding is used as the joining method.

5.1.9 Fatigue (see Part 2)

(1)P Structures subjected to fluctuating service loads are susceptible to fatigue, and their resistance to fatigue shall be checked. Fatigue performance is affected particularly by the following conditions:

- a high ratio of dynamic to static loads
- welded joints between members and between local attachments and members
- complexity of joint detail
- low natural frequencies in structural members
- poor thermal and chemical environment.

(2)P Wherever possible, aluminium structures shall be designed on the basis of providing an acceptable level of safety throughout its life. Fatigue assessment methods should be planned to ensure that the probability of fatigue failure during the life of a structure is comparable with that for other ultimate limit states load of failure.

(3) Fatigue design should be based on real nominal (average) stresses or real local stresses calculated by elastic global analysis and with an effective cross section that has been reduced in area to account for local buckling but with heat affected zones disregarded. In a web in shear a nominal shear stress based on an effective thickness 1,7 $\rho_c t_w$ (but not more than t_w) should be used (see 5.12.3 and 5.12.4).

5.1.10 Vibration

(1)P Resistance to vibration shall be assessed by a fatigue check and by a check on the damping characteristics of the structure and its material. This applies particularly to:

- structures supporting lifting appliances or rolling loads
- structures supporting vibrating machinery
- structures subjected to wind-induced oscillations
- structures subjected to crowd-induced oscillations.

5.1.11 Hybrid sections

(1)P The capacity of a hybrid section, containing aluminium alloys of different strengths, shall be found by considering the resistance of the various parts, taking due account of the classes of the parts.

(2) If hybrid sections of aluminium and steel are used, yielding behavior should be checked and thermal strains considered.

5.2 Calculation of internal forces and moments

5.2.1 Global analysis

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

(2)P When calculating the internal forces and moments in a statically indeterminate structure, due account shall be taken of the deformations that result from elastic and plastic deformation or local buckling. Also from the effects of shear lag where this is important.

(3) The internal forces and moments should be determined using either:

a) Elastic global analysis (linear or non linear)
b) Plastic global analysis (without or with hardening).

The different analysis methods are described in Annex C (informative).

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

(5)P Plastic global analysis shall be used only where the member cross-sections satisfy the requirements specified for Class 1 in 5.4. Cross-sections of Class 2, 3 and 4 are not allowed. For Class 1 sections it is always requested to check the deformation capacity in relation to the ductility demand of the structural scheme (see Annex D (informative)).

(6) For more details on the global analysis methods, see Annex D (informative).

(7) The design assumption for the connections should agree with those given in Annex C (informative).

5.3 Resistance of cross sections

5.3.1 General

(1) All members should satisfy the requirements in the ultimate limit states and serviceability limit states. Members are usually formed of extrusions, plate, sheet, tubes, forgings, or a combination of these. Designers wishing to employ castings should do so in close consultation with the manufacturers thereof.

(2) Where reference is made to design information in the form of mathematical expressions, it is permissible for the designers to use instead graphs or tables derived from the expressions. Members can be safely designed using the recommendations of section 5, but appendices are provided that allow fuller treatment of certain aspects of member behavior. Their use may lead to more economic and lighter structures.

5.3.2 Design resistance

(1) The expressions given for the design resistance of a member (which may not be less than the design action-effect on the member arising from factored loading) contain characteristic strengths (f_o, f_a, f_v, f_s) related to material properties. These are defined in 5.3.5. The expressions also contain the partial safety factor γ_M, for resistance see 5.1.1.

(2) The resistance of member cross sections may be limited by:

- The plastic resistance of cross section
- The resistance of net section at holes for fasteners
- Local stability verification of elements of the cross section.

(3) Where appropriate, overall stability should also be verified.

(4) The resistance of members with slender elements may be reduced as a result of premature local buckling. The proposed design of a member in uniform compression or in compression as the result of bending is checked by classifying the section in terms of its susceptibility to local buckling. Section classification is discussed in 5.4.

5.3.3 Section properties

(1)P The properties of the gross cross section shall be found by using the specified dimensions. Holes for fasteners need not be deducted, but allowance shall be made for larger openings. Splice materials and battens shall not be included.

(2) The net area of a member (A_{net}) or element cross section should be the gross area less appropriate deductions for holes and other openings. When calculating net section properties, the deduction for a single fastener hole should be the gross cross-sectional area of the hole in the plane of its axis. For countersunk holes the appropriate allowance should be made for the countersunk portion. Provided the fasteners holes are not staggered, the total area to be deducted for the fastener holes should be the maximum sum of the sectional areas of the holes in any cross section perpendicular to the member axis.

(3) When the fastener holes are staggered, the total area to be deducted for fastener holes should be the greater of:

a) the deduction for non-staggered holes given in (2)

b) the sum of the sectional areas of all holes in any diagonal or zig-zag line extending progressively across the member or part of the member, less

$$s^2t/(4p) \text{ and } 0{,}65st \qquad (5.1)$$

for each gauge space in the chain of holes, where:

s is staggered pitch (longitudinal pitch)
p is the spacing of the centers of the same two holes measured perpendicular to the member axis (transverse pitch)
t is the thickness (or effective thickness in a member containing HAZ material).

In an angle or other member with holes in more than one plane, the spacing p should be measured along the center of thickness of the material.

(4) In the design of connections in compression members or compressed parts of members, no deduction for fastener holes is normally required except for slotted holes. If in connections in compressed members or compressed parts of members no plastic deformation of the net section is acceptable, deduction of holes should be taken into account.

(5) In the design of connections in other types of member the provisions given in 5.7.3 apply for tension.

(6) Fastener holes in the tension flange need not be allowed for provided that for the tension flange:

$$0{,}9\,\frac{A_{net}}{A_g} \geq \frac{f_o/\gamma_{M1}}{f_a/\gamma_{M2}} \tag{5.2}$$

When A_{net}/A_g is less than this limit, a reduced flange area may be assumed.

(7) Fastener holes in the tension zone of the web need not to be allowed for, provided that the limit given in the expression above is satisfied for the complete tension zone comprising the tension flange plus the tension zone of the web.

(8) Fastener holes need not be allowed for in shear verifications provided that:

$$\frac{A_{v,net}}{A_v} \geq \frac{f_o/\gamma_{M1}}{f_a/\gamma_{M2}} \tag{5.3}$$

When $A_{v,net}/A_v$ is less than this limit, an effective shear area of $(f_a/\gamma_{M2})/(f_o/\gamma_{M1})\,A_{v,net}$ may be assumed. The block shear criterion given in section 6 should be verified at the ends of member.

5.3.4 Heat affected zones (HAZs)

(1)P Structural aluminium material of many alloys and conditions is weakened in the heat affected zones (HAZs) adjacent to welds, and this shall be taken into account in design calculations. Exceptions to this rule, where there is no weakening adjacent to welds, occur in alloys in the O or T4 condition; or when the material is in the F condition and design strength is based on O-condition properties.

(2) Rules for estimating the severity and extent of HAZ softening are given in 5.5.

(3) It is important for designers to realize that a small weld to connect a small attachment to a main member may considerably reduce the resistance of the member due to the presence of a HAZ. In beam design it is often beneficial to locate welds and attachments in low stress areas, i.e. near the neutral axis or away from regions of high bending moment.

5.3.5 Characteristic strength

(1) Resistance calculations for members are made using characteristic strength as follows:

f_o is the characteristic strength for bending and overall yielding in tension and compression
f_a is the characteristic strength for the local capacity of a net section in tension or compression
f_v is the characteristic strength in shear
f_s is the characteristic strength for overall buckling stability.

(2) Values of f_o, f_a and f_v depend on material properties and are defined as follows:

a) $f_o = f_{0,2}$, see 3.2 (5.4)

b) $f_a = f_u$, see 3.2 (5.5)

c) $$f_v = \frac{f_o}{\sqrt{3}} \tag{5.6}$$

5.4 Classification of cross sections

5.4.1 General

(1)P Section classification is necessary when considering the resistance of members to bending moments and to combined bending, shear and axial forces. It is also necessary when considering the possibility that members in bending or in axial compression may have a reduced resistance due to the local buckling of slender elements.

5.4.2 Classification

(1) Four classes of cross sections are defined, as follows:

- Class 1 cross sections are those which can form a plastic hinge with the rotation capacity required for plastic analysis. Further information on class 1 cross sections is given in Annex G (informative).

- Class 2 cross sections are those which can develop their plastic moment resistance, but have limited rotation capacity.

- Class 3 cross sections are those in which the calculated stress in the extreme compression fibre of the member can reach its proof strength, but local buckling is liable to prevent development of the full plastic moment resistance.

- Class 4 cross sections are those in which it is necessary to make explicit allowances for the effects of local buckling when determining their moment resistance or compression resistance.

(2) The classification of a section depends on the proportions of each of its compression elements.

(3) The compression elements include every element of a cross section that is either totally or partially in compression, due to axial force or bending moment, under the load combination considered.

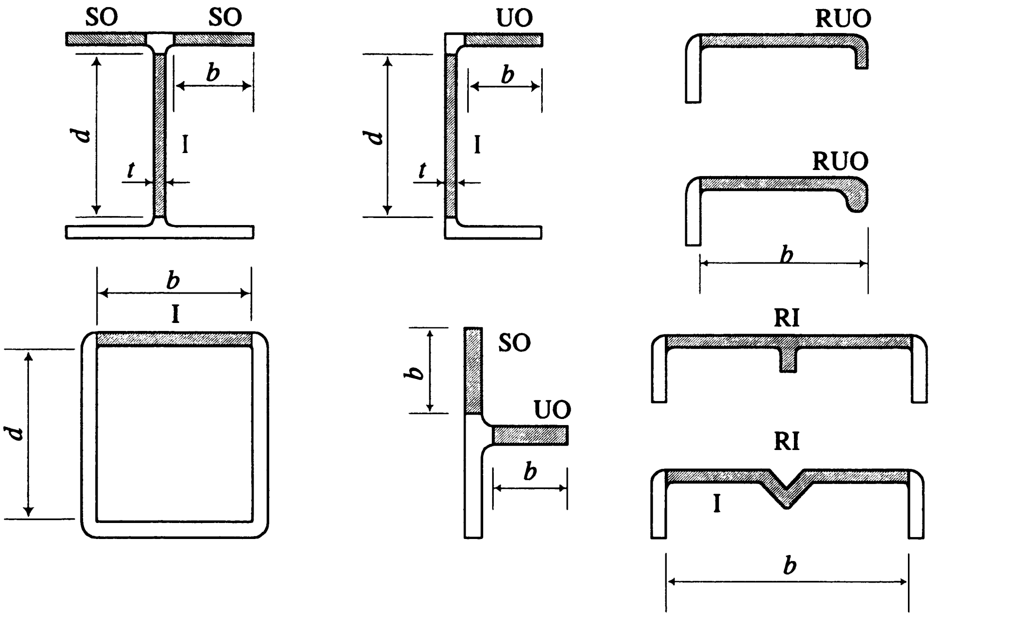
(4) The various compression elements in a cross section (such as web or a flange) can, in general, be in different classes.

(5) Cross sections should be classified accounting for the slenderness parameters of the elements which compose the section (see Annex G). In the absence of a more precise classification method, a cross section can be classified by quoting the least favourable class of its compression elements.

(6) The following basic types of thin-walled element are identified in the classification process:

a) flat outstand element;
b) flat internal element;
c) curved internal element.

These elements can be unreinforced, or reinforced by longitudinal stiffening ribs or edge lips or bulbs (see figure 5.1).



(a) ---------------- Unreinforced ---------------- (b) Reinforced

Key SO: symmetric outstand
UO: unsymmetric outstand
I: internal

RI: reinforced, internal
RUO: reinforced unsymmertical outstand

Figure 5.1: Types of element

5.4.3 Slenderness parameters

(1) The susceptibility of an unreinforced flat element to local buckling is defined by the parameter β, which has the following values:

a) flat outstand or internal elements with no stress gradient $\beta = b/t$

b) internal element with a stress gradient that results in a neutral axis at the center $\beta = 0{,}40\ b/t$, or $\beta = 0{,}40\ d/t$

c) for any other stress gradients $\beta = g\ b/t$ or $\beta = g\ d/t$

where:

b is the width of an element
t is the element thickness
d is the depth of a web element in a beam
g is the stress gradient coefficient.

g is given by the expressions:

$g = 0{,}70 + 0{,}30\ \psi$ $\quad (1 > \psi > -1)$, (5.7)
$g = 0{,}80/(1 - \psi)$ $\quad (\psi \leq -1)$, see figure 5.4 (5.8)

where

ψ is the ratio of the stresses at the edges of the plate under consideration related to the maximum compressive stress. In general the neutral axis should be the elastic neutral axis, but in checking whether a section is class 2 it is permissible to use the plastic neutral axis.

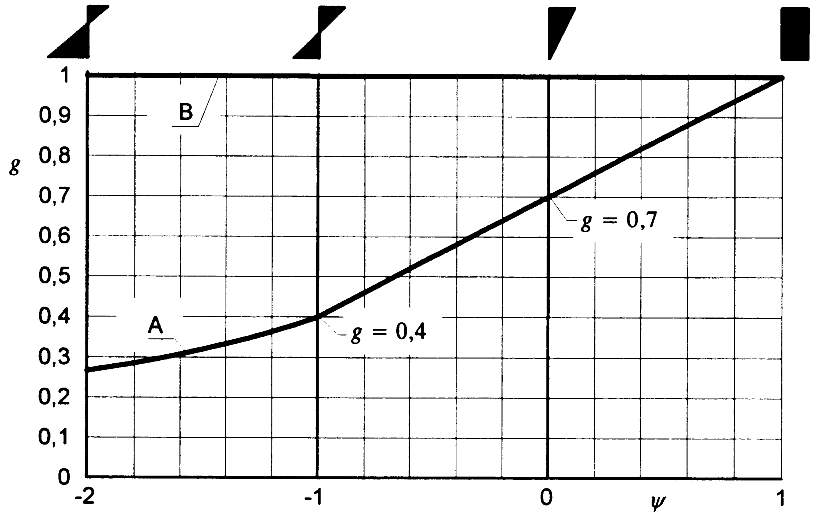


Figure 5.2: Flat internal elements under stress gradient, values of g. For internal elements or outstands (peak compression at root) use curve A. For outstands (peak compression at toe) use line B.

(2) When considering the susceptibility of a reinforced flat element to local buckling, three possible buckling modes should be considered, as shown in figure 5.3. Separate values of β should be found for each mode. The modes are:

a) Mode 1: the reinforced element buckles as a unit, so that the reinforcement buckles with the same curvature as the element.

b) Mode 2: the sub-elements and the reinforcement buckle as individual elements with the junction between them remaining straight.

c) Mode 3: this is a combination of Modes 1 and 2 in which sub-element buckles are superimposed on the buckles of the whole element. This is indicated in figure 5.3(c).

Values of β are found as follows:

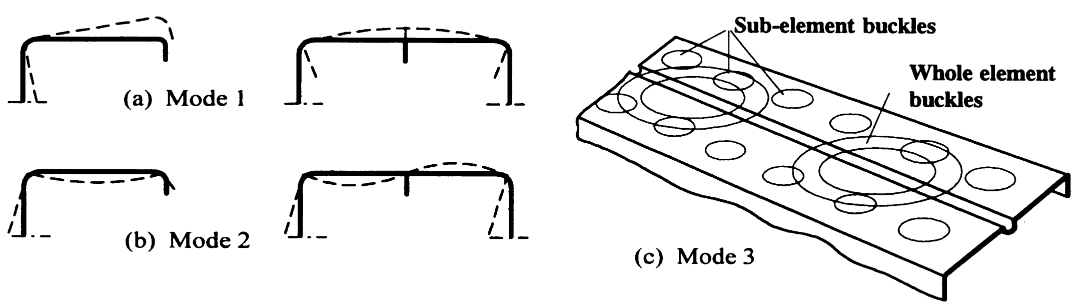


Figure 5.3: Buckling modes for flat reinforced elements

1) Mode 1, uniform compression, standard reinforcement:
When the reinforcement is a single-sided rib or lip of thickness equal to the element thickness t,

$$\beta = \eta \frac{b}{t} \tag{5.9}$$

where η is given in expressions 5.9a, b or c, or is read from figure 5.4(a), (b) or (c). In this figure the depth c of the rib or lip is measured to the inner surface of the plate element.

$$\eta = \frac{1}{\sqrt{1 + 0{,}1\,(c/t - 1)^2}} \qquad \text{(Figure 5.4a)} \tag{5.9a}$$

$$\eta = \frac{1}{\sqrt{1 + 2,5 \dfrac{(c/t - 1)^2}{b/t}}} \qquad \text{(Figure 5.4b)} \qquad (5.9b)$$

$$\eta = \frac{1}{\sqrt{1 + 4,5 \dfrac{(c/t - 1)^2}{b/t}}} \qquad \text{(Figure 5.4c)} \qquad (5.9c)$$

2) Mode 1, uniform compression, non-standard reinforcement:
With any other single shape of reinforcement, the reinforcement is replaced by an equivalent rib or lip equal in thickness to the element (t). The value of c for the equivalent rib or lip is chosen so that the second moment of area of the reinforcement about the mid-plane of the plate element is equal to that of the non-standard reinforcement about the same plane.

3) Mode 1, uniform compression, complex reinforcement:
For unusual shapes of reinforcement not amenable to the analysis described above,

$$\beta = \frac{b}{t}\left(\frac{\sigma_{cr0}}{\sigma_{cr}}\right)^{0,4} \qquad (5.10)$$

σ_{cr} is the elastic critical stress for the reinforced element assuming simply supported edges
σ_{cr0} is the elastic critical stress for the unreinforced element assuming simply supported edges.

4) Mode 1, stress gradient:
The value of β is found from the expression in 3) above, where σ_{cr} and σ_{cr0} now relate to the stress at the more heavily compressed edge of the element.

5) Mode 2:
The value of β is found separately for each sub-element in accordance with 5.4.3(1)a).

(3) The susceptibility of a uniformly compressed shallow curved unreinforced internal element to local buckling is defined by β, where

$$\beta = \frac{b}{t} \frac{1}{\sqrt{1 + 0,006 \dfrac{b^4}{R^2 t^2}}} \qquad (5.11)$$

R = radius of curvature to the mid-material
b = developed width of the element at mid-material
t = thickness.

The above treatment is valid if $R/b > 0,1\ b/t$. Sections containing more deeply curved elements require special study or acceptance by testing.

(4) The susceptibility of a thin walled round tube to local buckling, whether in uniform compression or in bending is defined by β, where :

$$\beta = 3\sqrt{\frac{D}{t}} \qquad (5.12)$$

D = diameter to mid-material.

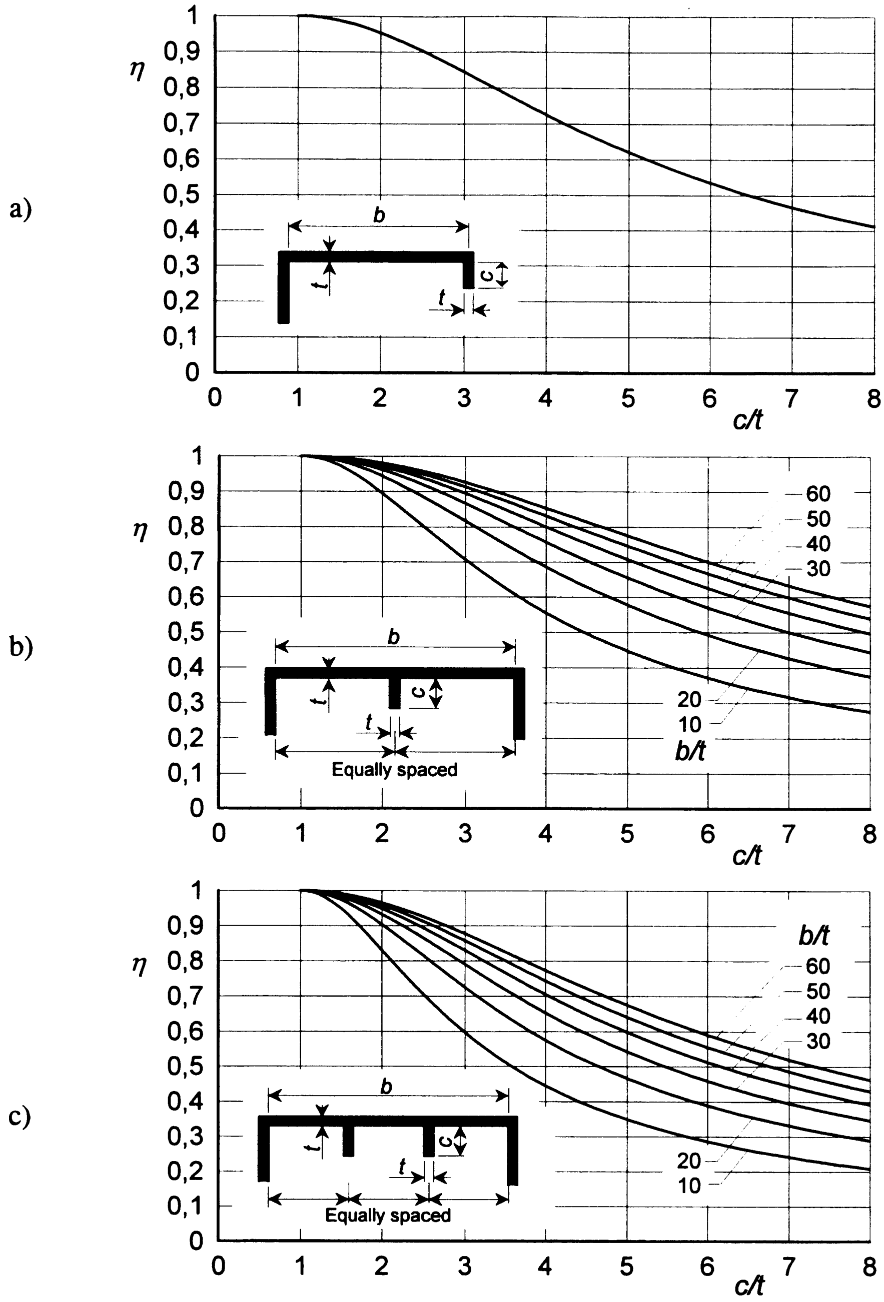


Figure 5.4: Values of η for reinforced elements

5.4.4 Element classification

(1) The classification of elements in cross sections is linked to the values of the slenderness parameter β as follows:

Elements in beams		Elements in struts	
$\beta \leq \beta_1$	: class 1	$\beta \leq \beta_2$	: class 1 or 2
$\beta_1 < \beta \leq \beta_2$	: class 2	$\beta_2 < \beta \leq \beta_3$	: class 3
$\beta_2 < \beta \leq \beta_3$	: class 3	$\beta_3 < \beta$	: class 4
$\beta_3 < \beta$	: class 4		

(2) Values of β_1, β_2 and β_3 are given in table 5.1.

Table 5.1: Slenderness parameters β_1, β_2 and β_3

Elements	β_1			β_2			β_3		
	Heat treated, unwelded	**Heat treated, welded or non heat treated, unwelded**	**Non heat treated, welded**	**Heat treated, unwelded**	**Heat treated, welded or non heat treated, unwelded**	**Non heat treated, welded**	**Heat treated, unwelded**	**Heat treated, welded or non heat treated, unwelded**	**Non heat treated, welded**
Outstand	$3\ \varepsilon$	$2,5\ \varepsilon$	$2\ \varepsilon$	$4,5\ \varepsilon$	$4\ \varepsilon$	$3\ \varepsilon$	$6\ \varepsilon$	$5\ \varepsilon$	$4\ \varepsilon$
Internal	$11\ \varepsilon$	$9\ \varepsilon$	$7\ \varepsilon$	$16\ \varepsilon$	$13\ \varepsilon$	$11\ \varepsilon$	$22\ \varepsilon$	$18\ \varepsilon$	$15\ \varepsilon$

$\varepsilon = \sqrt{250/f_o}$ where f_o is in N/mm^2

(3) In the table, an element is considered welded if it contains welding at an edge or at any point within its width. However, cross sections of a member that do not contain welding may be considered as unwelded even if the member is welded elsewhere along its length.

(4) Note that in a welded element the classification is independent of the extent of the HAZ.

(5) It is permissible to use a modified expression, $\varepsilon = \sqrt{\dfrac{250 z_1}{f_o z_2}}$ when classifying flange elements in members under bending, if the elements are less highly stressed than the most severely stressed fibres in the section. In this expression, z_1 is the distance from the elastic neutral axis of the effective section to the most severely stressed fibres, and z_2 is the distance from the elastic neutral axis of the effective section to the element under consideration. z_1 and z_2 should be evaluated on the effective section by means of an iterative procedure (minimum two steps).

5.4.5 Local buckling

(1) Local buckling in class 4 members is generally allowed for by replacing the true section by an effective section. The effective section is obtained by employing a local buckling coefficient ρ_c to factor down the thickness. ρ_c is applied to any uniform thickness class 4 element that is wholly or partly in compression. Elements that are not uniform in thickness require special study by the designer.

(2) The coefficient ρ_c is found separately for different elements of the section, in terms of the ratio β/ε, where β is found as in 5.4.3 and ε is defined in 5.4.4.

(3) Values of ρ_c are as follows:

a) Flat outstand elements in symmetrical cross sections (figure 5.1):

- heat treated, unwelded:

$\rho_c = 1,0$ when $\beta/\varepsilon \leq 6$,
$\rho_c = 10/(\beta/\varepsilon) - 24/(\beta/\varepsilon)^2$ when $\beta/\varepsilon > 6$.

- heat treated, welded and non heat treated, unwelded:

$\rho_c = 1,0$ when $\beta/\varepsilon \leq 5$,
$\rho_c = 9/(\beta/\varepsilon) - 20/(\beta/\varepsilon)^2$ when $\beta/\varepsilon > 5$.

- non heat treated, welded:

$\rho_c = 1,0$ when $\beta/\varepsilon \leq 4$,
$\rho_c = 8/(\beta/\varepsilon) - 16/(\beta/\varepsilon)^2$ when $\beta/\varepsilon > 4$.

b) Flat outstand elements in unsymmetrical cross sections (figure 5.1), ρ_c is given by the above expressions for flat outstand in symmetrical sections, but not more than $120/(\beta/\varepsilon)^2$.

c) Flat internal elements or round tubes:

- heat treated, unwelded:

$\rho_c = 1{,}0$ when $\beta/\varepsilon \leq 22$,
$\rho_c = 32/(\beta/\varepsilon) - 220/(\beta/\varepsilon)^2$ when $\beta/\varepsilon > 22$.

- heat treated, welded or non heat treated, unwelded:

$\rho_c = 1{,}0$ when $\beta/\varepsilon \leq 18$,
$\rho_c = 29/(\beta/\varepsilon) - 198/(\beta/\varepsilon)^2$ when $\beta/\varepsilon > 18$.

- non heat treated, welded:

$\rho_c = 1{,}0$ when $\beta/\varepsilon \leq 15$,
$\rho_c = 25/(\beta/\varepsilon) - 150/(\beta/\varepsilon)^2$ when $\beta/\varepsilon > 15$.

d) Reinforced elements: Consider all possible modes of buckling, and take the lower value of ρ_c. In the case of mode 1 buckling the factor ρ_c should be applied to the area of the reinforcement as well as to the basic plate thickness.

(4) The relationships between ρ_c and (β/ε) are summarised in figure 5.5.

(5) For the determination of ρ_c in sections required to carry biaxial bending or combined buckling and axial load, see section 5.9.

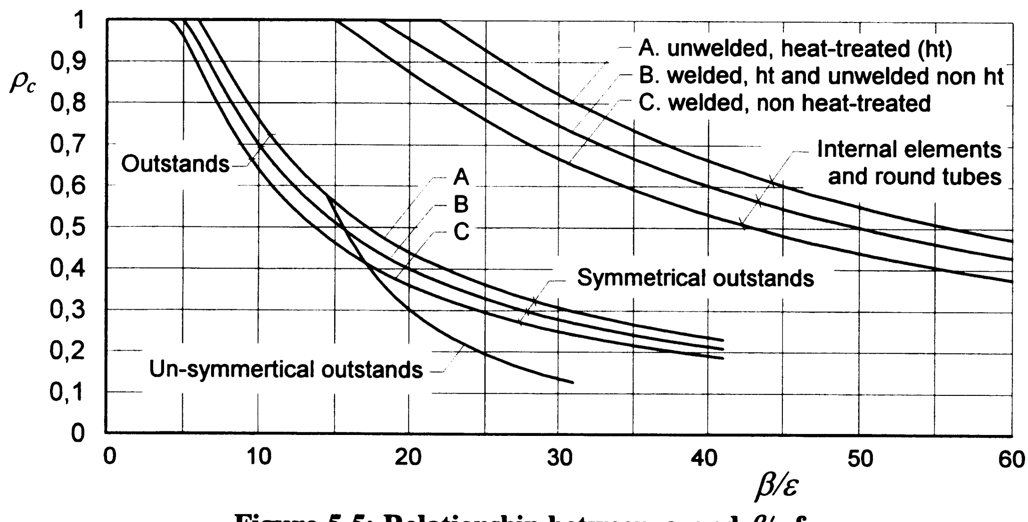


Figure 5.5: Relationship between ρ_c and β/ε for internal elements, outstand elements and round tubes

5.5 HAZ softening adjacent to welds

5.5.1 General

(1)P In the design of welded structures using high strength structural alloys it is necessary to allow for the reduction in strength properties that occurs in the vicinity of welds. The reduction affects the 0,2% proof stress of the material more severely than the ultimate tensile strength.The affected region extends immediately around the weld, beyond which the strength properties rapidly recover to their full unwelded values.

(2) For design purposes it is assumed that throughout the heat affected zone (HAZ) the strength properties are reduced by a constant factor, ρ_{haz}. The severity of softening, as defined by ρ_{haz}, is discussed in 5.5.2.

The extent of the HAZ, defined by a distance b_{haz} from the edge of the weld, is considered in 5.5.3.

(3) It is sometimes possible to mitigate the effects of HAZ softening by means of artificial ageing applied after welding.

5.5.2 Severity of softening

(1) The characteristic strengths f_o, f_a and f_v in the HAZ are calculated in a similar way to that given in 5.3.5, but are then multiplied by the appropriate value of ρ_{haz}, taken from table 5.2. These values refer to a range of typical structural alloys.

In 7xxx material, values of ρ_{haz} are influenced by the nature of the stresses on the HAZ. In table 5.2 two values are given:

a) apply when a tensile stress acts transversely to the axis on a butt or a fillet weld;
b) apply for all other conditions, i.e. a longitudinal stress, a transverse compressive stress or a shear stress.

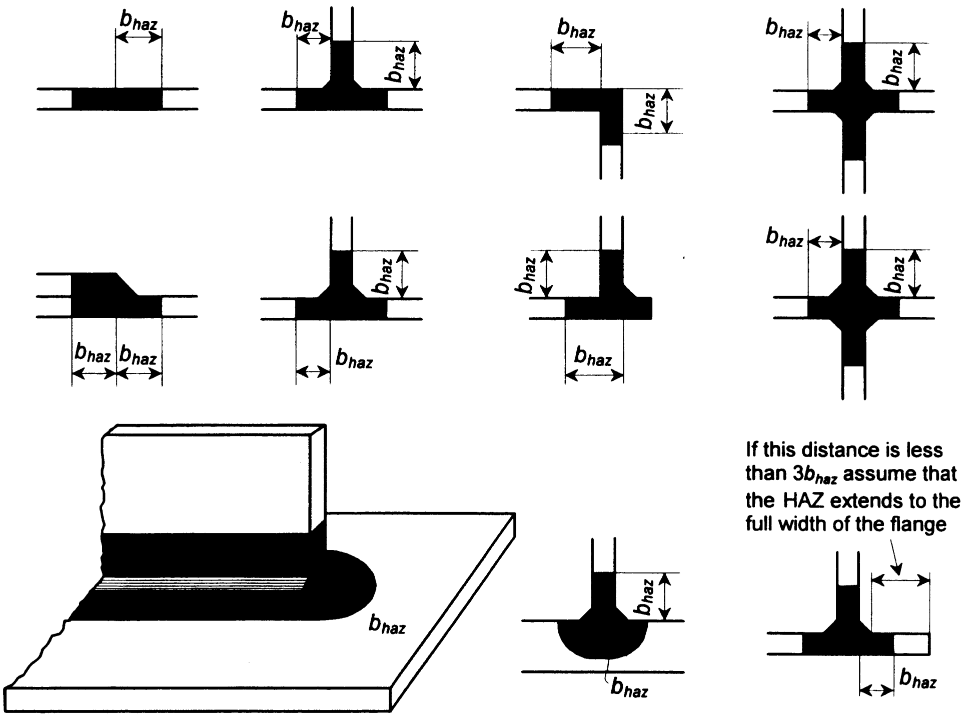


Figure 5.6: The extent of heat-affected zones (HAZ)

(2) The values in table 5.2 are valid from the following times after welding, providing the material has been held at a temperature not less than 10° C:

6xxx series alloys	3 days
7xxx series alloys	30 days.

If the material is held at a temperature below 10° C after welding, the recovery time will be prolonged. Advice should be sought from manufacturers.

(3) The severity of softening can be taken into account by the characteristic strength in the HAZ as for the parent metal, or reducing the area over which the stresses act. Thus the characteristic resistance of a simple rectangular section affected by HAZ softening can be expressed as $(f_a\rho_{haz})A$ or $f_a(A\rho_{haz})$.

5.5.3 Extent of HAZ

(1) The HAZ is assumed to extend a distance b_{haz} in any direction from a weld, measured as follows (see figure 5.6).

a) transversely from the centre line of an in-line butt weld
b) transversely from the point of intersection of the welded surfaces at fillet welds
c) transversely from the point of intersection of the welded surfaces at butt welds used in corner, tee or cruciform joints.
d) in any radial direction from the end of a weld.

(2) The HAZ boundaries should generally be taken as straight lines normal to the metal surface, particularly when welding thin material. However, when surface welding is applied to thick material it is permissible to assume a curved boundary of radius b_{haz}, as shown in figure 5.6).

Table 5.2: HAZ Softening Factor (ρ_{haz})

For all alloys supplied as extrusions, sheet, plate, drawn tubes and foreging in the O and F condition, ρ_{haz} = 1,0.			
Extrusions, sheet, plate, drawn tube and forging in 6xxx and 7xxx alloys in the T4, T5 and T6 condition:			
Alloy Series	Condition	ρ_{haz} (MIG welding)	ρ_{haz} (TIG welding)
6xxx	T4 T5 T6	1,0 0,65 0,65	-- 0,60 0,50
7xxx	T6	0,80 [a)] 1,0 [b)]	0,60 [a)] 0,80 [b)]
Sheet, plate or forging in 5xxx, 3xxx and 1xxx alloys (in the work hardened (H) condition):			
Alloy Series	Condition	ρ_{haz} (MIG welding)	ρ_{haz} (TIG welding)
5xxx	H22 H24	0,86 0,80	0,86 0,80
3xxx	H14, 16, 18	0,60	0,60
1xxx	H14	0,60	0,60

[a, b)] For the definition of *a)* and *b)* see 5.5.2 (1)

(3) For a MIG weld laid on unheated material, and with interpass cooling to 60°C or less when multi-pass welds are laid, values of b_{haz} are as follows:

$0 < t \leq 6$ mm: $b_{haz} = 20$ mm

$6 < t \leq 12$ mm: $b_{haz} = 30$ mm

$12 < t \leq 25$ mm: $b_{haz} = 35$ mm

$t > 25$ mm: $b_{haz} = 40$ mm

(4) For thickness > 12 mm there may be a temperature effect, because interpass cooling may exceed 60°C unless there is strict quality control. This will increase the width of the Heat Affected Zone.

(5) The above figures apply to in-line butt welds (two valid heat paths) or to fillet welds at T-junctions (three valid heat paths) in 6xxx or 7xxx series alloys, or 5xxx series alloys in the work-hardened condition.

(6) For a TIG weld the extent of the HAZ is greater because the heat input is greater than for a MIG weld. TIG welds for in-line butt or fillet welds in 6xxx, 7xxx or work-hardened 5xxx series alloys, have a value of b_{haz} given by (see 6.6.1(2)):

$0 < t \leq 6$ mm: $\quad b_{haz} = 30$ mm

(7) When two or more welds are close to each other, their HAZ boundaries overlap. A single HAZ then exists for the entire group of welds. When a weld is located too close to the free edge of an outstand the dispersal of heat is less effective. This applies when the distance from the edge of the weld to the free edge is less than $3b_{haz}$. In these circumstances assume that the entire width of the outstand is subject to the factor ρ_{haz}.

(8) Other factors that affect the value of b_{haz} are as follows:

a) Influence of temperatures above 60°C

When multi-pass welds are being laid down, there could be a build-up of temperature between passes. This results in an increase in the extent of the HAZ. If the interpass temperature = T_1(°C), which should be somewhere between 60°C and 120°C, it is conservative to assume that for 6xxx, 7xxx or work-hardened 5xxx series alloys b_{haz} will be multiplied by a factor α_2, as follows:

6xxx alloys: $\quad \alpha_2 = 1 + (T_1 - 60)/120$,
7xxx alloys: $\quad \alpha_2 = 1 + 1{,}5(T_1 - 60)/120$.

If a less conservative value of α_2 is desired, hardness tests on test specimens will indicate the true extent of the HAZ. A temperature of 120°C is the maximum recommended temperature for welding aluminium alloys.

b) Variations in element thickness

If the elements to be joined by welds do not have a common thickness t, it is conservative to assume in all the above expressions that t is the average thickness of all elements. This applies as long as the average thickness does not exceed 1,5 × the smallest thickness. For greater variations of thickness, the extent of the HAZ should be determined from hardness tests on specimens.

c) Variations in numbers of heat paths

If the junctions between elements are fillet welded, but have different numbers of heat paths (N) from the three designated at (5) above, multiply the value of b_{haz} by $3/N$.

5.6 Resistance of beams

5.6.1 General

(1)P The following resistances shall normally be checked:

a) Bending (see 5.6.2), including, where appropriate, allowance for coincident shear (see 5.6.4)
b) Shear (see 5.6.3)
c) Web bearing (see 5.6.5)
d) Lateral torsional buckling (see 5.6.6)

(2)P Due account shall be taken of the class of cross-section (see 5.4), the presence of any heat affected zones (see 5.5) and the need to allow for the presence of holes (see 5.3).

(3)P For members required to resist bending combined with axial load reference shall be made to 5.9.

(4) Biaxial bending is covered under 5.6.7 or if combined with axial load under 5.9.

5.6.2 Uniaxial bending resistance

5.6.2.1 Basis

(1) In the absence of shear force, the design value of the bending moment resistance M_{Rd} should be the lesser of $M_{a,Rd}$ and $M_{c,Rd}$ where

$$M_{a,Rd} = f_a W_{net} / \gamma_{M2} \tag{5.13}$$

in a net section and

$$M_{c,Rd} = f_o \alpha W_{el} / \gamma_{M1} \tag{5.14}$$

at each cross-section where:

α is the shape factor, see table 5.3
W_{el} is the elastic modulus of the gross section (see 5.6.2.2)
W_{net} is the elastic modulus of the net section allowing for holes and HAZ softening, if welded.

Table 5.3: Values of shape factor α

Class	Unwelded	Welded
1	W_{pl}/W_{el}, but see Annex G	W_{ple}/W_{el}, but see Annex G
2	W_{pl}/W_{el}	W_{ple}/W_{el}
3	$\alpha_{3,u}$	$\alpha_{3,w}$
4	W_{eff}/W_{el}	W_{effe}/W_{el}

In table 5.3 the various section moduli W and $\alpha_{3,u}$, $\alpha_{3,w}$ are defined as:

W_{pl} plastic modulus of gross section
W_{eff} effective elastic section modulus, obtained using a reduced thickness t_{eff} for the class 4 elements (see 5.6.2.2)
W_{ele} effective elastic modulus of the gross section, obtained using a reduced thickness $\rho_{haz}t$ for the HAZ material (see 5.6.2.2)
W_{ple} effective plastic modulus of the gross section, obtained using a reduced thickness $\rho_{haz}t$ for the HAZ material (see 5.6.2.2)
W_{effe} effective elastic section modulus, obtained using a reduced thickness t_{eff} for the class 4 elements or a reduced thickness $\rho_{haz}t$ for the HAZ material, whichever is the smaller (see 5.6.2.2)
$\alpha_{3,u} = 1$ or may alternatively be taken as:

$$\alpha_{3,u} = \left[1 + \left(\frac{\beta_3 - \beta}{\beta_3 - \beta_2} \right) \left(\frac{W_{pl}}{W_{el}} - 1 \right) \right] \tag{5.15}$$

$\alpha_{3,w} = W_{ele}/W_{el}$ or may alternatively be taken as:

$$\alpha_{3,w} = \left[\frac{W_{ele}}{W_{el}} + \left(\frac{\beta_3 - \beta}{\beta_3 - \beta_2} \right) \left(\frac{W_{ple} - W_{ele}}{W_{el}} \right) \right] \tag{5.16}$$

where:

β is the slenderness parameter for the most critical element in the section
β_2 and β_3 are the limiting values for that same element according to table 5.1.

(2) Refer to 5.6.4 for combination of bending moment and shear force.

(3)P In addition, the resistance of the member to lateral-torsional buckling shall also be verified, see 5.6.6.

5.6.2.2 Assumed section

(1) The terminology used in 5.6.2.1 is as follows:

a) net section includes the deduction for holes and includes the allowance for reduced strength material taken in the vicinity of the welds to allow for HAZ softening, if welded;

b) effective section includes the allowance for HAZ softening and local buckling, but with no reduction for holes.

(2) In items a) and b) above the allowance for reductions in material strength should generally be taken as follows for different elements in the section.

a) Class 4 element free of HAZ effects. A value $t_{eff} = \rho_c t$ is taken for the compressed part of element, where ρ_c is found as in 5.4.5.

b) Class 1, 2 or 3 elements subject to HAZ effects. A value $\rho_{haz} t$ is taken in the softened parts of the element, where ρ_{haz} and the extent of the softening are as given in 5.5.2 and 5.5.3.

c) Class 4 element with HAZ effects. The allowance is taken as the lesser of that corresponding to the reduced thickness t_{eff} and that corresponding to the reduced thickness in the softened part, $\rho_{haz} t$ and as t_{eff} in the rest of the element.

d) In the case of reinforced elements ρ_c should be applied to the area of the reinforcement as well as to the basic plate thickness.

e) For a welded element in a Class 3 or 4 section a more favourable assumed thickness may be taken as follows:

1) HAZ softening is ignored in any material less than $\rho_{haz} y_1$ from the elastic neutral axis of the gross section, where y_1 is the distance from there to the furthest extreme fibres of the section.

2) For HAZ material, at a distance y ($> \rho_{haz} y_1$) from the neutral axis, ρ_{haz} may be replaced by a value k_{zy} determined as follows:

$$k_{zy} = \rho_{haz} + 1 - y/y_1$$

5.6.3 Shear force resistance

(1) The design value of the shear force V_{Ed} at each cross-section should satisfy:

$$V_{Ed} \leq V_{c,Rd} \tag{5.17}$$

where $V_{c,Rd}$ is the design shear resistance of the cross-section that depends upon section classification for shear (see 5.10.5) and is taken as:

(2) Class 1, 2 or 3 sections

$$V_{c,Rd} = A_v f_v / \gamma_{M1}$$

where A_v is the shear area, taken as:

a) For sections containing shear webs

$$A_v = \sum_1^N \left[0{,}8\,D(t_w)_i - (1 - \rho_{haz})d_z(t_w)_i\right] \tag{5.18}$$

where:

d_z is the total depth of HAZ material occurring between the clear depth of the web between flanges. For unwelded sections, $\rho_{haz} = 1$.
D is the overall depth of section measured to the outer surfaces of the flanges
t_w is the web thickness
N is the number of webs.

b) For a solid bar and a round tube

$$A_v = \eta_v A_e \tag{5.19}$$

where:

$\eta_v = 0{,}8$ for a solid bar
$\eta_v = 0{,}6$ for a round tube

A_e is the section area of an unwelded section, and the effective section area obtained by taking a reduced thickness $\rho_{haz}t$ for the HAZ material of a welded section.

(3) Class 4 sections

Class 4 sections are covered in 5.12.4 - 5.12.6.

5.6.4 Combined bending and shear

(1)P The theoretical resistance moment of a cross-section is reduced by the presence of shear. For small values of the shear force this reduction is so small that it may be neglected. However, when the shear force exceeds half the shear resistance, allowance shall be made for its effect on the resistance moment.

(2) Provided that the design value of the shear force V_{Ed} does not exceed 50% of the design shear resistance $V_{pl,Rd}$ no reduction need be made in the resistance moment given by 5.6.2.1.

(3) When V_{Ed} exceeds 50% of $V_{pl,Rd}$ the design resistance moment of the cross-section should be reduced to $M_{v,Rd}$ the reduced design plastic resistance moment allowing for the shear force, obtained as follows:

a) For the shear area the contribution to the cross-section's moment capacity is based on a reduced value of the material strength f_{ow}, given by:

$$f_{ow} = f_0\sqrt{1 - (V_{Ed}/V_{pl,Rd})^2} \tag{5.20}$$

b) In the case of an equal-flanged I-section classified as class 1, 2 or 3 in bending, the resulting value of $M_{v,Rd}$ is:

$$M_{v,Rd} = t_f b_f (h_s - t_f) \frac{f_0}{\gamma_{M1}} + \frac{t_w h_w^2}{4} \frac{f_{ow}}{\gamma_{M1}} \tag{5.21}$$

(4) For sections classified as class 4 in bending or affected by HAZ softening, α should be based on the assumed section used in the determination of $M_{c,Rd}$ (see 5.6.2.2).

5.6.5 Web bearing

(1) This clause concerns the design of webs subjected to localised forces caused by concentrated loads or reactions applied to a beam. For unstiffened web this subject is covered in Section 5.12.

(2) For transversely stiffened web a bearing stiffener, if fitted, should be of class 2 section or better. It may be conservatively designed on the assumption that it resists the entire bearing force, unaided by the web, the stiffener being checked as a strut (see 5.8) for out-of-plane column buckling and local squashing, with bending effects allowed for if necessary (see 5.9). Alternatively, a more economical stiffener may be designed by referring to the plate girder stiffener clauses (see 5.12).

5.6.6 Lateral torsional buckling

5.6.6.1 General

(1)P All beams, apart from those allowed exceptions in 5.6.6.2, shall be checked against possible failure by lateral torsional buckling.

(2) All unbraced lengths between points of adequate lateral support should satisfy the condition:

$$M_{ed} \leq M_{b,Rd} \tag{5.22}$$

where the design buckling resistance moment $M_{b,Rd} = f_s \alpha W_{el,y} / \gamma_{M1}$

$W_{el,y}$ is the elastic section modulus of the gross section, without reduction for HAZ softening, local buckling or holes. α is taken from table 5.3 subject to the limitation $\alpha \leq W_{pl}/W_{e,y}$. f_s is the lateral torsional buckling stress (see 5.6.6.3).

5.6.6.2 Exceptions

(1) Lateral torsional buckling need not be checked in any of the following circumstances:

a) Bending takes place about the minor principal axis.

b) The beam is fully restrained against lateral movement throughout its length.

c) The non-dimensional slenderness $\bar{\lambda}_{LT}$ (see 5.6.6.3) between points of effective lateral restraint is less than 0,4.

5.6.6.3 Buckling stress

(1) The lateral torsional buckling stress f_s for the appropriate non-dimensional slenderness $\bar{\lambda}_{LT}$ may be obtained from:

$$f_s = \chi_{LT} f_0 \tag{5.23}$$

in which χ_{LT} is taken from figure 5.7 or from :

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \tag{5.24}$$

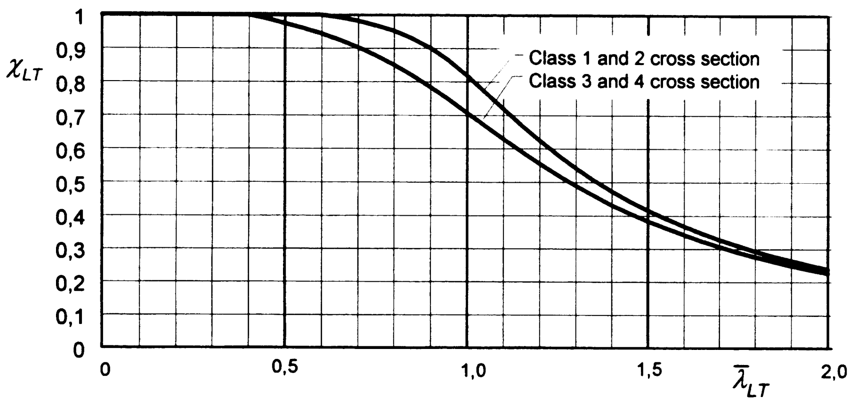


Figure 5.7: Reduction factor χ_{LT} for lateral-torsional buckling

$$\phi_{LT} = 0,5\left[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{0,LT}) + \bar{\lambda}_{LT}^2\right] \tag{5.25}$$

(2) The value of α_{LT} and $\bar{\lambda}_{0,LT}$ should be taken as:

α_{LT} = 0,10 and $\bar{\lambda}_{0,LT}$ = 0,6 for class 1 and 2 cross sections α_{LT} = 0,20 and $\bar{\lambda}_{0,LT}$ = 0,4 for class 3 and 4 cross sections.

(3) The value of $\bar{\lambda}_{LT}$ may be determined from $\bar{\lambda}_{LT} = \sqrt{\dfrac{\alpha W_{el,y} f_o}{M_{cr}}}$

where:

α is taken from table 5.3 subject to the limitation $\alpha \leq W_{pl} / W_{el,y}$
M_{cr} is the elastic critical moment for lateral-torsional buckling, (see Annex H).

(4) Conservatively the value of $\bar{\lambda}_{LT}$ may be obtained from:

$$\bar{\lambda}_{LT} = \lambda_{LT} \frac{1}{\pi} \sqrt{\frac{f_o}{E}} \tag{5.26}$$

where:

$\lambda_{LT} = l/i_z$
l is the effective length for lateral torsional buckling
i_z is the minor axis radius of gyration of the gross section.

(5) For I-sections and channels covered by table 5.4 the value of λ_{LT} may be obtained from:

$$\lambda_{LT} = \frac{X\,L/i_z}{\left[1 + Y\left(\dfrac{L/i_z}{D/t_2}\right)\right]^{\frac{1}{4}}} \tag{5.27}$$

where:

D is the overall section depth
t_2 is the flange thickness
X and Y are coefficients obtained from table 5.4. It is conservative to take $X = 1{,}0$ and $Y = 0{,}05$.

When the flange reinforcement to an I-section or channel is not of the precise form shown in table 5.4 (simple lips), it is still permissable to obtain λ_{LT} using the above expression, providing X and Y are taken as for an equivalent simple lip having the same internal depth C, while i_z is calculated for the section with its actual reinforcement.

(6) For the following cases reference should be made to Annex H to find the appropriate value of λ_{LT} or M_{cr}.

a) Cantilever beams.
b) Beams with cross-sections that are symmetrical about the minor axis only.
c) Beams subject to vertical loads
d) Beams subject to unequal end moments.

For all other cases l may be taken as the distance between points of effective lateral restraint.

5.6.6.4 Effective Lateral Restraints

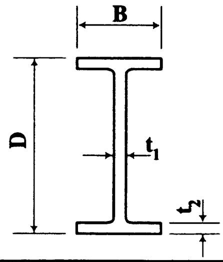
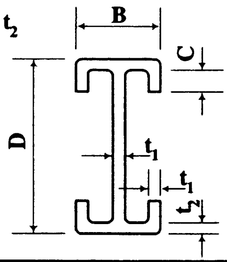
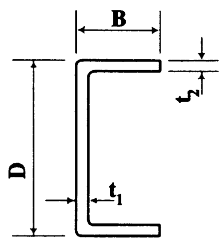
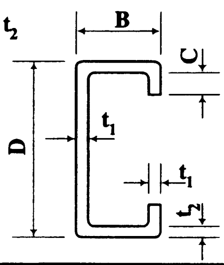
(1) Bracing systems providing lateral restraint should be designed on the assumption that the total lateral force exerted by a compression flange, under factored loading, shared between the points of restraint in any one span, is 3% of the compression in that flange.

(2) Where a series of two or more parallel beams require lateral restraint, it is not adequate merely to tie the compression flanges together so that they become mutually dependent. Adequate restraint will be provided only by anchoring the ties to an independent robust support, or by providing a triangulated bracing system. If the number of parallel beams exceeds three, it is sufficient for the restraint system to be designed to resist the sum of the lateral forces derived from the three largest compressive forces only.

5.6.7 Biaxial Bending

(1) Members subject to bending about both principal axes may be designed using the provisious of 5.9.4 but noting that the term in N_{Ed} should be deleted.

Table 5.4: Lateral torsional buckling of beams, coefficients *X* and *Y*[1)]

	$X = 0{,}90 - 0{,}03\,D/B + 0{,}04\,t_2/t_1$ $Y = 0{,}05 - 0{,}010\,\sqrt{D/B(t_2/t_1 - 1)}$
$t_1 = t_2$ 	$X = 0{,}94 - D/B(0{,}03 - 0{,}07\,C/B) - 0{,}3\,C/B$ $Y = 0{,}05 - 0{,}06\,C/D$
	$X = 0{,}95 - 0{,}03\,D/B + 0{,}06\,t_2/t_1$ $Y = 0{,}07 - 0{,}014\,\sqrt{D/B(t_2/t_1 - 1)}$
$t_1 = t_2$ 	$X = 1{,}01 - D/B(0{,}03 - 0{,}06\,C/B) - 0{,}3\,C/B$ $Y = 0{,}07 - 0{,}10\,C/D$

1) The expressions for X and Y are valid for $1{,}5 \leq D/B \leq 4{,}5$, $1 \leq t_2/t_1 \leq 2$ and $0 \leq C/B \leq 0{,}5$

5.7 Resistance of tension members

5.7.1 General

(1)P For members in axial tension, the design value of the tensile force N_{Sd} at each cross section shall satisfy:

$$N_{Ed} \leq N_{t,Rd} \tag{5.28}$$

where $N_{t,Rd}$ is the design tension resistance of the cross section, taken as the smaller of two values corresponding respectively to:

a) general yielding along the member (see 5.7.2);

b) local failure at a critical section (see 5.7.3).

(2) For angles connected through one leg see section 6: Connections. Similar consideration should also be given to other types of sections connected through outstands such as T-sections and channels.

(3) For staggered holes, see 5.3.3(3).

5.7.2 General yielding

(1) The value $N_{t,Rd}$ is based on the general cross section of the member along its length, ignoring the effect of end connections, occasional holes or localized HAZ regions as follows:

$$N_{t,Rd} = A_g f_o / \gamma_{M1} \tag{5.29}$$

where:

A_g is either the gross section or a reduced cross section to allow for HAZ softening. In the latter case A_g is found by taking a reduced area equal to ρ_{haz} times the total area of the HAZ, see section 5.5.2

f_o is the characteristic strength (see 5.3.5(2))

γ_{M1} is the partial safety factor for the material (see 5.1.1(2)).

5.7.3 Local failure

(1) The value of $N_{t,Rd}$ is based on the most critical section as follows:

$$N_{t,Rd} = A_{net} f_a / \gamma_{M2} \tag{5.30}$$

where:

f_a is the characteristic strength (see 5.3.5(2))

A_{net} is the net section area, with deduction for holes; and a deduction when required to allow for the effect of HAZ softening. The latter deduction is based on the reduced thickness of $\rho_{haz}\, t$

γ_{M2} is the material factor (see 5.1.1(2)).

5.8 Resistance of compression members

5.8.1 General

(1)P Members subject to axial compression may fail in one of three ways:

a) Flexural (see 5.8.4)
b) Torsional (see 5.8.5)

c) Local squashing (see 5.8.6)

Check a) shall always be made. Check b) is generally necessary but may be waived in some cases. Check c) is only necessary for struts of low slenderness that are significantly weakened locally by holes or welding.

(2) For members subject to combined compression and bending refer to 5.9.

5.8.2 Section Classification for Axial Compression

(1) Before making any of the checks given in 5.8.1 it is first necessary to classify the cross-section as class 1, 2 or 3 or class 4. The classification is based on that of the least favourable of its component elements, in accordance with 5.4.4.

5.8.3 Overall Buckling Resistance

(1) Both checks in 5.8.1(1) a) and b) should satisfy the condition

$$N_{Ed} \leq N_{b,Rd} \tag{5.31}$$

where:

$N_{b,Rd}$ is the design buckling resistance and is equal to $f_s A / \gamma_{M1}$
A is the gross area, without reduction for HAZ softening, local buckling or holes
f_s is the buckling stress for flexural buckling (5.8.4) or torsional buckling (5.8.5)

(2) When determining f_s for flexural buckling, failure about both principal axes should be considered and the lower value taken.

5.8.4 Flexural buckling resistance

5.8.4.1 Buckling stress

(1) The value of f_s for the appropriate non-dimensional slenderness may be determined from

$$f_s = \chi \eta k_1 k_2 f_o \tag{5.32}$$

where χ is taken from figure 5.8 or from:

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \tag{5.33}$$

$\phi = 0{,}5\,[1 + \alpha(\bar{\lambda} - \bar{\lambda}_o) + \bar{\lambda}^2]$
α is an imperfection factor, see table 5.6
$\bar{\lambda}_o$ is the limit of the horizontal plateau, see table 5.6

$$\bar{\lambda} = \sqrt{\frac{A\,\eta f_o}{N_{cr}}} = \frac{\lambda}{\lambda_1}$$

λ = is the slenderness for the relevant axis (see 5.8.4.2)

$$\lambda_1 = \pi \sqrt{\frac{E}{\eta f_o}}$$

N_{cr} is the elastic critical load for the relevant axis
η is the factor to allow for any loss of effectiveness due to the presence of class 4 elements, given by:
$\eta = 1$ for class 1, 2 or 3 cross-section
$\eta = A_e/A$ for class 4 cross-section with $A_e = A - A_c(1 - \rho_c)$, A_c = area of class 4 element,

$\rho_c = t_{ef}/t$ for each class 4 element.

k_1 = is a factor to allow for the asymmetry of the cross-section, given in table 5.5

k_2 = is a factor to allow for the weakening effects of welding, given in table 5.5.

NOTE For class 4 cross-sections containing HAZ material A_c should be taken as the lesser of values given previously or in table 5.5. For cross-sections containing more than one class 4 element or more than one HAZ affected element the reduction may need to take account of different values of ρ_{haz} or ρ_c for different plate elements.

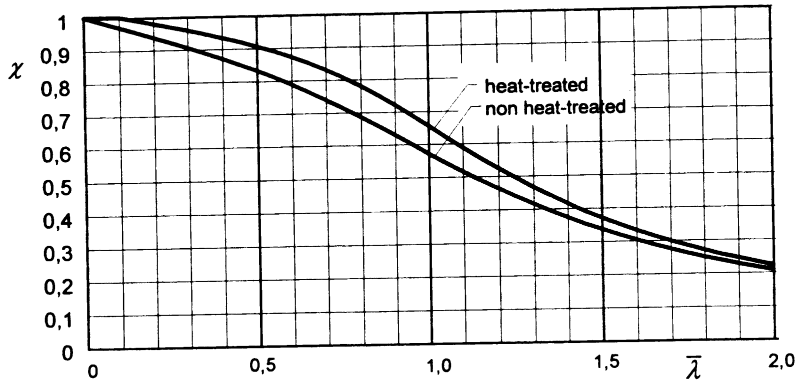


Figure 5.8: Reduction factor χ for flexural buckling

(2) The value of α and $\bar{\lambda}_o$ should be selected in accordance with table 5.6.

Table 5.5: Values of k_1 and k_2 factors

		Heat-Treated alloys	Non Heat-Treated alloys
k_1	symmetric cross section	$k_1 = 1$	$k_1 = 1$
	asymmetric cross section	$k_1 = 1 - 2{,}4\,\psi^2 \dfrac{\bar{\lambda}^2}{(1+\bar{\lambda}^2)(1+\bar{\lambda})^2}$	$k_1 = 1 - 3{,}2\,\psi^2 \dfrac{\bar{\lambda}^2}{(1+\bar{\lambda}^2)(1+\bar{\lambda})^2}$
		$\psi = \dfrac{\gamma_{max} - \gamma_{min}}{h}$ where γ_{max} and γ_{min} are the distances between the section edges and the centroid in the plane of buckling and h is the cross-section depth	
k_2	longitudinal welds	$k_2 = 1 - \left(1 - \dfrac{A_1}{A}\right)10^{-\bar{\lambda}} - \left(0{,}05 + 0{,}1\dfrac{A_1}{A}\right)\bar{\lambda}^{1{,}3(1-\bar{\lambda})}$ with $A_1 = A - A_{haz}(1-\rho_{haz})$ in which A_{haz} = area of HAZ	$k_2 = 1 + 0{,}04(4\bar{\lambda})^{(0{,}5-\bar{\lambda})} - 0{,}22\bar{\lambda}^{1{,}4(1-\bar{\lambda})}$
	transversal welds	$k_2 = \rho_{haz}$ or ω_x according to 5.9.4.5	$k_2 = \rho_{haz}$ or ω_x according to 5.9.4.5

Table 5.6: Values of imperfection factor α and $\bar{\lambda}_o$

Alloy	α	$\bar{\lambda}_o$
Heat-Treated	0,20	0,10
Non Heat-Treated	0,32	0,00

5.8.4.2 Slenderness parameter

(1) The column buckling slenderness parameter λ is defined as follows

$$\lambda = l/i \tag{5.34}$$

where:

l is the effective length
i is the radius gyration

both appropriate to the direction of buckling considered.

The effective length l should be taken as KL, where L is the length between points of lateral support; for a cantilever strut, L is its length. The value of K, the effective length factor for struts, should be assessed from a knowledge of the end conditions; table 5.7 gives guidance.

The value of i should be based on the gross section for all members.

NOTE: When the cross section is wholly or substantially affected by HAZ softening at a directionally restrained end of a member, such restraint should be ignored in arriving at a suitable value for K. Thus for case 1 in table 5.7 K should be taken as 1,0 if the section is fully softened at each end.

Table 5.7: Effective length factor *K* for struts

End conditions	***K***
1. Held in position and restrained in direction at both ends	0,7
2. Held in position at both ends and restrained in direction at one end	0,85
3. Held in position at both ends, but not restrained in direction	1,0
4. Held in position at one end, and restrained in direction at both ends	1,25
5. Held in position and restrained in direction at one end, and partially restrained in direction but not held in position at the other end	1,5
6. Held in position and restrained in direction at one end, but not held in position or restrained at the other end	2,0

5.8.5 Torsional buckling

5.8.5.1 Exceptions

(1) The possibility of torsional buckling may be ignored for the following:

a) closed hollow sections
b) doubly symmetrical I-sections
c) sections composed entirely of radiating outstands, e.g. angles, tees, cruciforms, that are classified as class 1 in accordance with 5.4.4

5.8.5.2 Slenderness parameter

(1) The torsional buckling slenderness parameter λ may be obtained using either a) or b) below, or else by referring to Annex J. It should always be based on the gross area of the section as follows.

a) General expression

$$\lambda = \pi \sqrt{\frac{EA}{N_{cr}}} \tag{5.35}$$

where:

A is the gross section area, without reduction for local buckling, HAZ softening or holes
E is the modulus of elasticity
N_{cr} is the elastic critical load for torsional buckling, allowing for interaction with column buckling when necessary

b) Sections as given in table 5.9

$$\lambda = k\lambda_t \tag{5.36}$$

where k is read from figure 5.9 or given by the expression:

$$k = \sqrt{\frac{2Xs^2}{1 + s^2 - \sqrt{(1 + s^2)^2 - 4Xs^2}}} \tag{5.36a}$$

in which X and s are found in Table 5.9.

λ_t is found as follows:

1) for angles, tees, cruciforms $\quad \lambda_t = \lambda_o$ (5.36b)

2) for channels, top-hats $\quad \lambda_t = \dfrac{\lambda_o}{\sqrt{1 + Y\lambda_o^2/\lambda_y^2}}$ (5.36c)

Table 5.9 contains expressions for λ_o and Y and also for s and X (needed in expression 5.36b and for figure 5.9).

In 2) the quantity λ_y should be taken as the effective slenderness for column buckling about axis y-y (as defined in table 5.9).

5.8.5.3 Buckling stress

(1) The value of f_s for torsional buckling should be obtained from the expression given in 5.8.4.1 (1) with ϕ determined from

$$\phi = 0{,}5\,[1 + \alpha(\bar{\lambda} - \bar{\lambda}_l) + \bar{\lambda}^2] \tag{5.37}$$

and using a value for α and $\bar{\lambda}_l$ selected in accordance with table 5.8.

Table 5.8 : Values of α and $\bar{\lambda}_1$ for torsional buckling

Cross-section	Value of α	Value of $\bar{\lambda}_1$
general composed entirely of radiating outstands (see 5.8.5.4)	0,35 0,20	0,4 0,6

5.8.5.4 Sections composed of radiating outstands

(1) For sections such as angles, tees and cruciforms, composed entirely of radiating outstands, local and torsional buckling are closely related.

(2) When considering the torsional buckling of sections containing only unreinforced outstands, allowance should be made, where appropriate, for the presence of HAZ material when determining A_e but no reduction should be made for local buckling i.e. $\rho_c = 1$.

(3) For sections containing reinforced outstands such that mode 1 would be critical in terms of local buckling (see 5.4.3), the member should be regarded as "general" in table 5.8 and A_e determined allowing for either or both local buckling and HAZ material.

5.8.6 Local squashing

(1) The axial thrust N_{Ed} should not exceed $N_{a,Rd}$ for the most unfavourable section along the length of the strut determined as follows:

Class 1, 2 or 3 sections, free from HAZ effects $\quad N_{a,Rd} = f_o A_n / \gamma_{M1}$
Other sections, generally $\quad N_{a,Rd} = f_o A_{ne} / \gamma_{M1}$

in which:

A_n is the net section area, with deductions for unfilled holes
A_{ne} is the net effective section area

The area A_{ne} should be taken as A_e less deduction for unfilled holes, where A_e is the effective area used in the consideration of overall buckling (flexural or torsional). For holes located in reduced thickness regions the deduction may be based on the reduced thickness, instead of the full thickness.

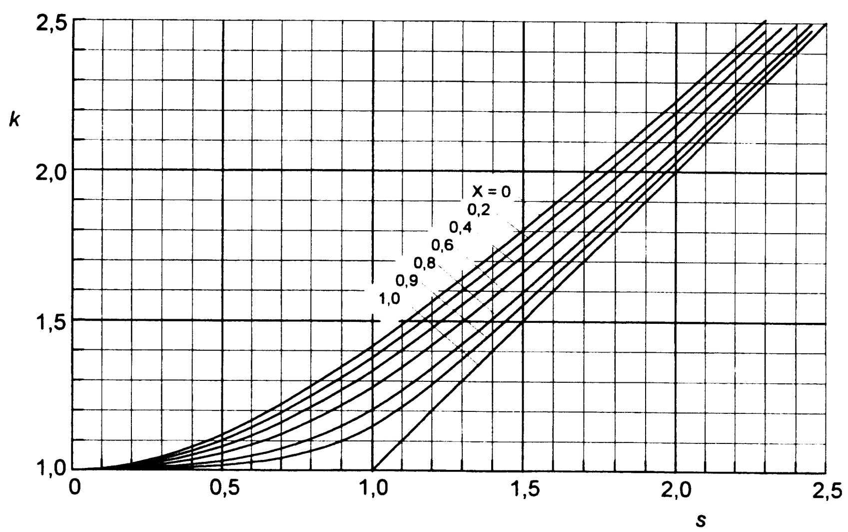


Figure 5.9 : Torsional buckling of struts, interaction factor k
For the definition of s, see table 5.9

Table 5.9 Torsional buckling parameters for struts

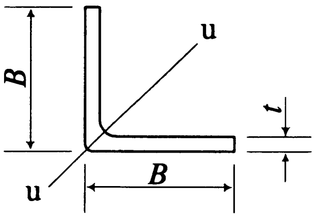
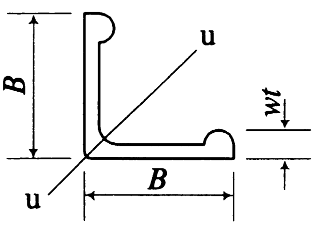
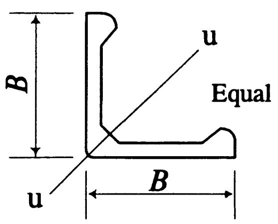
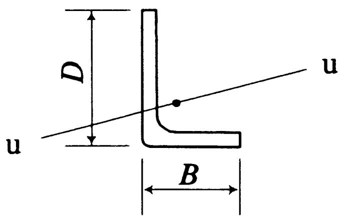
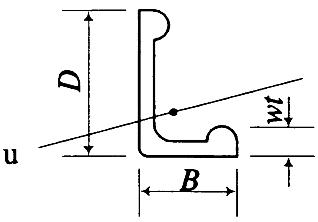
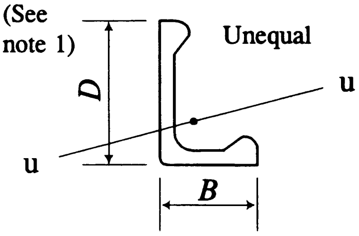
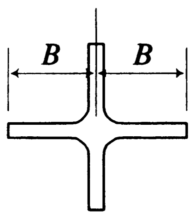
1		$\rho \leq 5$	$\lambda_o = \lambda_1 = 5B/t - 0{,}6\rho^{1{,}5}(B/t)^{0{,}5}$ $s = \lambda_u/\lambda_o$ $X = 0{,}6$
2		$\rho \leq 5$ $1 \leq w \leq 2{,}5$	$\lambda_o = \lambda_1 - (w - 1)[2(w - 1)^2 - 1{,}5\rho]$ $s = \lambda_u/\lambda_o$ $X = 0{,}6$
3	(See note 1) 		$\lambda_o = 66$ $s = \lambda_u/\lambda_o$ $X = 0{,}61$
4		$\rho \leq 5$ $0{,}5 \leq B/D \leq 1$	$\lambda_o = (D/t)[4{,}2 + 0{,}8(B/D)^2] - 0{,}6\rho^{1{,}5}(D/t)^{0{,}5}$ $s = s_4 = \{1 + 6(1 - B/D)^2\}(\lambda_u/\lambda_o)$ $X = X_4 = 0{,}6 - 0{,}4(1 - B/D)^2$
5		$\rho \leq 5$ $0{,}5 \leq B/D \leq 1$ $1 \leq w \leq 2{,}5$	$\lambda_o = \lambda_4 + 1{,}5\rho(w - 1) - 2(w - 1)^3$ $s = s_4$ $X = X_4$
6	(See note 1) 		$\lambda_o = 57$ $s = 1{,}4\lambda_u/\lambda_o$ $X = 0{,}6$
7		$\rho \leq 3{,}5$	$\lambda_o = 5{,}1B/t - \rho^{1{,}5}(B/t)^{0{,}5}$ $X = 1$

Table 5.9 Torsional buckling parameters for struts (continued)

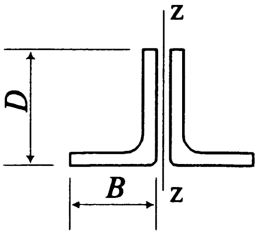
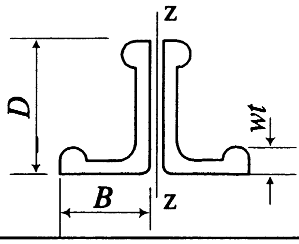
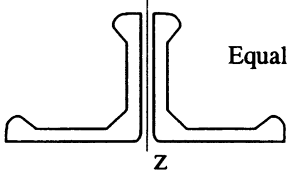
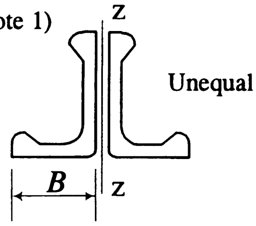
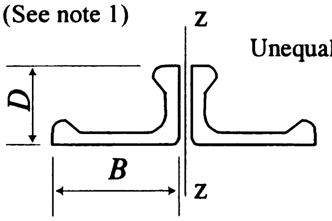
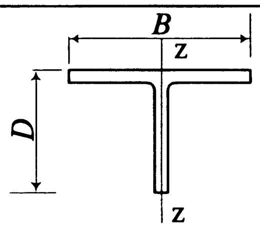
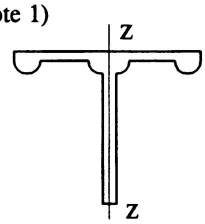
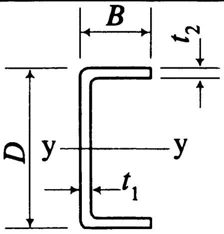
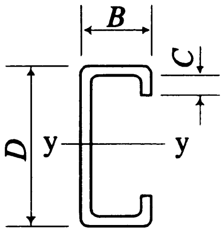
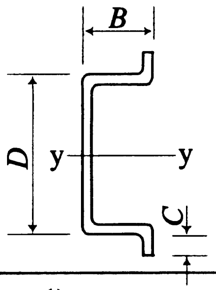
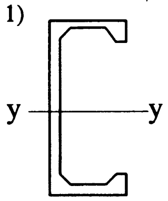
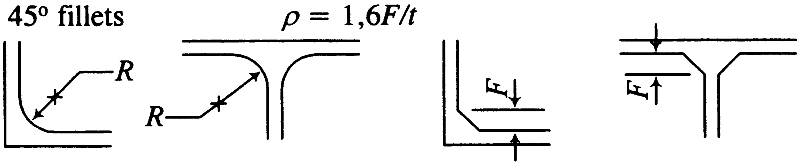
8		$\rho \leq 5$ $0,5 \leq D/B \leq 2$	$\lambda_o = \lambda_8 = (B/t)[4,4 + 1,1(B/D)^2]$ - $- 0,7\rho^{1,5}(B/t)^{0,5}$ $s = \lambda_z/\lambda_o$ $X = X_8 = 1,1 - 0,3D/B$
9		$\rho \leq 5$ $0,5 \leq D/B \leq 2$ $1 \leq w \leq 2,5$	$\lambda_o = \lambda_8 + 1,5\rho(w - 1) - 2(w - 1)^3$ $s = \lambda_z/\lambda_o$ $X = X_8$
10	(See note 1) 		$\lambda_o = 70$ $s = \lambda_z/\lambda_o$ $X = 0,83$
11	(See note 1) 		$\lambda_o = 60$ $s = \lambda_z\lambda_o$ $X = 0,76$
12	(See note 1) 		$\lambda_o = 63$ $s = \lambda_z/\lambda_o$ $X = 0,89$
13		$\rho \leq 3,5$ $0,5 \leq D/B \leq 2$	$\lambda_o = (D/t)[1,4 + 1,5B/D + 1,1D/B]$ - $- \rho^{1,5}(D/t)^{0,5}$ $s = \lambda_z/\lambda_o$ $X = 1,3 - 0,8D/B + 0,2(D/B)^2$
14	(See note 1) 		$\lambda_o = 65$ $s = \lambda_z/\lambda_o$ $X = 0,78$

Figure 5.9 Torsional buckling parameters for struts (continued)

15		$1 \le D/B \le 3$ $1 \le t_2/t_1 \le 2$	$\lambda_o = (B/t_2)[7 + 1{,}5(D/B)(t_2/t_1)]$ $s = \lambda_y/\lambda_t$ $X = 0{,}38D/B - 0{,}04(D/B)^2$ $Y = 0{,}14 - 0{,}02D/B - 0{,}02t_2/t_1$
16		$1 \le D/B \le 3$ $C/B \le 0{,}4$	$\lambda_o = (B/t)(7 + 1{,}5D/B + 5C/B)$ $s = \lambda_y/\lambda_t$ $X = 0{,}38D/B - 0{,}04(D/B)^2 - 0{,}25C/B$ $Y = 0{,}12 - 0{,}02D/B + \dfrac{0{,}6(C/B)^2}{D/B - 0{,}5}$
17		$1 \le D/B \le 3$ $C/B \le 0{,}4$	$\lambda_o = (B/t)(7 + 1{,}5D/B + 5C/B)$ $s = \lambda_y/\lambda_t$ $X = 0{,}38D/B - 0{,}04(D/B)^2$ $Y = 0{,}12 - 0{,}2D/B - \dfrac{0{,}05C/B}{D/B - 0{,}5}$
18	(See note 1) 		$\lambda_o = 126$ $s = \lambda_y/\lambda_t$ $X = 0{,}59$ $Y = 0{,}104$

1) The sections are generally of uniform thickness t, except cases 14 and 15.
2) λ_u, λ_y or λ_z is the slenderness parameter (l/r) for flexural buckling about the u, y, or z axis.
3) ρ is a factor depending on the amount of material at the root of the section as follows:

Radiused fillets $\rho = R/t$
45° fillets $\rho = 1{,}6F/t$



4) The values given for λ_o, X and Y are only valid within the limits shown. In the case of back-to-back angles (cases 8 to 12) the expressions ceases to apply if the gap between the angles exceeds $2t$.

5.8.7 Eccentrically connected struts

5.8.7.1 Single - bay struts

(1) Providing the end attachment prevents rotation in the plane of the connected element and no deliberate bending is applied, the following types of eccentrically connected strut may be designed using a simplified approach. This represents an alternative to the general method for combined bending and compression of 5.9.

a) single angle connected through one leg only
b) back to back angles connected to one side of a gusset plate
c) single channel connected by its web only
d) single tee connected by its flanges only

(2) When checking flexural buckling using 5.8.4 out of the plane of the attached element(s) eccentricity of loading should be ignored and the value of f_s should be taken as 40% of the value for centroidal loading.

(3) The value for a) should be that about the axis parallel to the connected element(s). For torsional buckling no change to the method of 5.8.5 is necessary.

5.8.7.2 Struts composed of two back to back components

(1) Struts comprising pairs of angles, channels or tees, connected either side of end gusset plates may be designed as a single compound member providing:

a) the two components are securely fastened at their ends
b) the two components are also interconnected at their third points, using spacers equal to the gusset thickness

5.8.8 Battened struts

(1) Battened struts should generally be designed by first determining the forces to which each component will be subjected and then proportioning each component to resist these forces. However, providing the arrangement satisfies the following 7 conditions it may be designed as a single compound member:

a) It should be axially loaded.

b) It should comprise two main components joined by equally spaced battens, the cross section being symmetrical about an axis normal to the battens.

c) Battens should generally be in pairs. However, if the main components are toe-to-toe tees or angles, single battens are allowed.

d) $\lambda_2 \leq 0{,}8\ \lambda_1$

where

λ_1 and λ_2 are the slenderness parameters for column buckling of the complete member about axes parallel to and normal to the battens, respectively.

e) $\lambda_3 \leq 0{,}7\ \lambda_2$

where

λ_3 is the slenderness parameter for buckling of one main component between battens, based on column or torsional buckling which ever is the more critical.

f) The batten system should be designed to resist a total shear force V in the plane of the batten, taken as 2,5 % of the axial force in the whole member under factored loading.

g) The connection of each batten to each main component should be designed to transmit the following simultaneous actions under factored loading:

1) longitudinal shear of V_{Ed}/N

2) moment of $V_{Ed}/2N$ acting in the plane of the batten

where:

d is the longitudinal spacing between centres of battens;
a is the spacing of main components measured of the centroids of the connections to each batten;
N is the number of battens at each position (1 or 2).

In designing the battens it is important to consider the possible weakening effects of local buckling and HAZ softening (if welded).

5.9 Resistance of members with axial force and biaxial bending

5.9.1 General

(1) This clause gives interaction expressions for checking members subjected to a combination of axial force and major axis and/or minor axis bending.

(2) Two checks are in general needed

- flexural buckling
- lateral-torsional buckling.

(3) Section check is included in the check of flexural and lateral-torsional buckling if the methods in 5.9.3 and 5.9.4 is used.

(4) When calculating the resistance N_{Rd}, $M_{y,Rd}$ and $M_{z,Rd}$ due account of the presence of HAZ-softening from longitudinal welds should be taken. (See 5.6.2 and 5.8.7). The presence of localized HAZ-softening from transverse welds and the presence of holes shall be taken care of according to 5.9.4(5) and (6) respectively.

5.9.2 Section classification and local buckling under combined actions

(1)P Classification of cross sections for members with combined bending and axial forces is made for the loading components separately according to 5.4. No classification is made for the combined state of stress.

(2) A cross section can belong to different classes for axial force, major axis bending and minor axis bending. The combined state of stress is taken care of in the interaction expressions in 5.9.3 and 5.9.4. These interaction expressions can be used for all classes of cross section. The influence of local buckling and yielding on the resistance for combined loading is taken care of by the capacities in the denominators and the exponents which are functions of the slenderness of the cross section.

5.9.3 Bending and axial force. Section check

5.9.3.1 Bending and axial tension

(1)P Members subject to combined bending and axial tension shall be checked for resistance to lateral-torsional buckling, treating the axial force and bending moment as a vectorial effect.

(2) Where the axial force and bending moment can vary independently, the design value of the axial tension should be multiplied by a reduction factor for vectorial effects:

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

(3) The net calculated stress $\sigma_{com,Ed}$ (which can exceed f_o/γ_{M1}) in the extreme compression fibre due to the vectorial effects should be determined from:

$$\sigma_{com,Ed} = M_{Sd} / W_{com} - \psi_{vec} N_{t,Sd} / A \qquad (5.38)$$

where W_{com} is the elastic section modulus for the extreme compression fibre and $N_{t,Ed}$ is the design value of the axial tension.

(4) The verification should be carried out using an effective design internal moment $M_{eff,Ed}$ obtained from:

$$M_{eff,Ed} = W_{com}\, \sigma_{com,Ed} \qquad (5.39)$$

(5) The design buckling resistance moment $M_{b,Rd}$ should be obtained using 5.6.6.

(6) Members subjected to combined bending and axial tension and members subjected to combined bending and axial compression where there is no risk of flexural or lateral-torsional buckling shall satisfy expressions 5.40 and 5.41 or 5.43. They also apply to other members subjected to bending and axial compression under the condition that $M_{y,Ed}$ and $M_{z,Ed}$ is determined with regard to second order theory and with respect to the increased deflections that occur due to residual stresses and actual stress-strain curve.

5.9.3.2 I-beams

(1) The following two expressions should be checked:

$$\left(\frac{N_{Ed}}{\omega_0 N_{Rd}}\right)^{\xi_0} + \frac{M_{y,Ed}}{\omega_0 M_{y,Rd}} \leq 1{,}00 \qquad (5.40)$$

and

$$\left(\frac{N_{Ed}}{\omega_0 N_{Rd}}\right)^{\eta_0} + \left(\frac{M_{y,Ed}}{\omega_0 M_{y,Rd}}\right)^{\gamma_0} + \left(\frac{M_{z,Ed}}{\omega_0 M_{z,Rd}}\right)^{\xi_0} \leq 1{,}00 \qquad (5.41)$$

where the exponents η_0, γ_0 and ξ_0 are:

$$\eta_0 = 1{,}0 \text{ or may alternatively be taken as } \alpha_z^2 \alpha_y^2 \text{ but } \eta_0 \geq 1 \text{ and } \eta_0 \leq 2 \qquad (5.42a)$$

$$\gamma_0 = 1{,}0 \text{ or may alternatively be taken as } \alpha_z^2 \text{ but } \gamma_0 \geq 1 \text{ and } \gamma_0 \leq 1{,}56 \qquad (5.42b)$$

$$\xi_0 = 1{,}0 \text{ or may alternatively be taken as } \alpha_y^2 \text{ but } \xi_0 \geq 1 \qquad (5.42c)$$

(2) The notations in 5.9.3.2 - 5.9.3.5 are:

N_{Ed} = axial compression or tension force
$M_{y,Ed}$ = bending moment about the y-axis
$M_{z,Ed}$ = bending moment about the z-axis
N_{Rd} = Af_o/γ_{M1} or $A_{ef}f_o/\gamma_{M1}$ for class 4 cross sections
$M_{y,Rd}$ = $\alpha_y W_y f_o / \gamma_{M1}$

$M_{z,Rd}$ $= \alpha_z W f_o / \gamma_{M1}$
ω_0 = 1 for beam-columns without localized welds or holes. Otherwise, see 5.9.4(5) or (6)
α_y, α_z = shape factor for bending about the y and z axis, based on the gross sections with allowance for local buckling and HAZ softening from longitudinal welds, see 5.6.2 α_z should not be taken larger than 1,25.

5.9.3.3 Solid cross sections and hollow sections

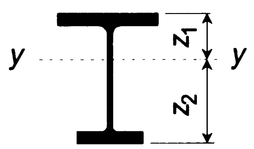
(1) The following expression should be checked:

$$\left(\frac{N_{Ed}}{\omega_0 N_{Rd}}\right)^{\psi} + \left[\left(\frac{M_{y,Ed}}{\omega_0 M_{y,Rd}}\right)^{1,7} + \left(\frac{M_{z,Ed}}{\omega_0 M_{z,Rd}}\right)^{1,7}\right]^{0,6} \leq 1,00 \qquad (5.43)$$

where ψ = 2 for solid cross sections and 1,3 for hollow sections. Alternatively ψ may be taken as $\alpha_y \alpha_z$ but $\psi \leq 2$.

5.9.3.4 Other cross sections

(1) Expression 5.41 may be used with $\eta_0 = \alpha_0^2$ (but $\eta_0 \leq 2,0$ and $\eta_0 > 1$) and $\gamma_0 = \xi_0 = 1$ where α_0 is the largest of α_{y1} and α_{y2} for the two extreme fibres. See figure 5.10. For thin walled cross sections, see 5.9.4.4.



$$\alpha_{y1} = \frac{W_{pl,y}\, z_1}{I_y}$$

$$\alpha_{y2} = \frac{W_{pl,y}\, z_2}{I_y}$$

Figure 5.10 : Shape factors for an unsymmetrical class 1 or 2 cross section

5.9.3.5 Members containing localized welds

(1) In a section affected by HAZ softening the value of f_o shall be taken as the characteristic strength for *overall yielding* of the reduced strength material. This includes HAZ effects due to the welding of temporary attachments.

(2) However when such HAZ softening has a specified location along the length and if the softening does not extend longitudinally a distance greater than the least width of the member, then the design strength f_o/γ_{M1} shall be taken as the design *ultimate strength* f_a/γ_{M2} of the reduced strength material.

5.9.4 Bending and axial compression

5.9.4.1 General

(1)P For members subjected to axial compression and bending appropriate interaction expressions for

a) flexural buckling and
b) lateral-torsional buckling shall be satisfied on any unsupported length liable to buckle.

(2) All quantities in the interaction expressions should be taken as positive.

5.9.4.2 Flexural buckling

The following expressions shall be checked:

(1) y-axis bending of an I-beam

$$\left(\frac{N_{Ed}}{\chi_y\,\omega_x N_{Rd}}\right)^{\xi_{yc}} + \frac{M_{y,Ed}}{\omega_0 M_{y,Rd}} \leq 1,00 \tag{5.44}$$

(2) z-axis bending of an I-beam

$$\left(\frac{N_{Ed}}{\chi_z\,\omega_x N_{Rd}}\right)^{\eta_c} + \left(\frac{M_{z,Ed}}{\omega_0 M_{z,Rd}}\right)^{\xi_{zc}} \leq 1,00 \tag{5.45}$$

In the expressions 5.44 and 5.45 all the exponents can be taken as 0,8 or alternatively

$\eta_c = \eta_0\chi_z$ but $\eta_c \geq 0,8$
$\xi_{yc} = \xi_0\chi_y$ but $\xi_{yc} \geq 0,8$
$\xi_{zc} = \xi_0\chi_z$ but $\xi_{zc} \geq 0,8$

where:

η_0 and ξ_0 are according to 5.9.3.2(1)

$\omega_x = \omega_0 = 1$ for beam-columns without localized welds. Otherwise, see 5.9.4.5.

(3) Solid cross sections

Expression 5.45 may be used with the exponents taken as 0,8 or

$\eta_c = 2\chi$ but $\eta_c \geq 0,8$
$\xi_c = 1,56\chi$ but $\xi_c \geq 0,8$

(4) Hollow cross sections and pipes

The expression 5.46 may be used with $\psi_c = 0,8$ or alternatively taken as $\chi_y\psi$ or $\chi_z\psi$ depending on direction of buckling, but $\psi_c \geq 0,8$. ψ is according to expression 5.43.

$$\left(\frac{N_{Ed}}{\chi_{min}\,\omega_x N_{Rd}}\right)^{\psi_c} + \frac{1}{\omega_0}\left[\left(\frac{M_{y,Ed}}{M_{y,Rd}}\right)^{1,7} + \left(\frac{M_{z,Ed}}{M_{z,Rd}}\right)^{1,7}\right]^{0,6} \leq 1,00 \tag{5.46}$$

(5) Other cross sections

Expression 5.44 may be used for symmetrical and bi-symmetrical cross sections bending about either axis, for z-axis bending replacing ξ_{yc}, $M_{y,Ed}$, $M_{y,Rd}$ and χ_y by ξ_{zc}, $M_{z,Ed}$, $M_{z,Rd}$ and χ_z.

(6) The notations in expressions 5.44, 5.45 and expression 5.46 are:

N_{Ed} = axial compressive force
N_{Rd} = Af_o/γ_{M1} or $A_{ef}f_o/\gamma_{M1}$ for class 4 cross sections
χ_y = reduction factor for buckling in the z-x plane
χ_z = reduction factor for buckling in the y-x plane
$M_{y,Ed}$, $M_{z,Ed}$ = bending moment about the y and z axis. The moments are calculated according to first order theory

$M_{y,Rd}$ = $\alpha_y W_y f_o / \gamma_{M1}$ bending moment capacity about the y-axis
$M_{z,Rd}$ = $\alpha_z W_z f_o / \gamma_{M1}$ bending moment capacity about the z-axis
α_y, α_z = shape factor but α_z should not be taken larger than 1,25
ω_x, ω_0 = HAZ-softening factors according to 5.9.4.5. If there are no welds, $\omega_x = \omega_0 = 1$.

5.9.4.3 Lateral-torsional buckling

(1) The following expression should be checked for beam-columns with I shaped and similar cross sections.

$$\left(\frac{N_{Ed}}{\chi_z \omega_x N_{Rd}}\right)^{\eta_c} + \left(\frac{M_{y,Ed}}{\chi_{LT}\omega_{xLT} M_{y,Rd}}\right)^{\gamma_c} + \left(\frac{M_{z,Ed}}{\omega_0 M_{z,Rd}}\right)^{\xi_{zc}} \leq 1,00 \tag{5.47}$$

where:

N_{Ed} = axial force
$M_{y,Ed}$ = bending moments about the y-axis. In the case of beam-columns with hinged ends and in the case of members in non-sway frames, $M_{y,Ed}$ is moment of the *first order*. For members in frames free to sway, $M_{y,Ed}$ is bending moment according to *second order* theory.
$M_{z,Ed}$ = bending moments about the z-axis. $M_{z,Ed}$ is bending moment according to *first order* theory
N_{Rd} = Af_o/γ_{M1} or $A_{ef} f_o/\gamma_{M1}$ for class 4 cross sections.
χ_z = reduction factor for buckling when one or both flanges deflects laterally (buckling in the y-z plane or lateral-torsional buckling)
$M_{y,Rd}$ = $\alpha_y W_y f_o/\gamma_{M1}$ = the bending moment capacity for y-axis bending
χ_{LT} = reduction factor for lateral-torsional buckling
$M_{z,Rd}$ = $\alpha_z W_z f_o/\gamma_{M1}$ = the bending moment capacity for z-axis bending
η_c = 0,8 or alternatively $\eta_0 \chi_z$ but $\eta_c \geq 0,8$
γ_c = γ_0
ξ_{zc} = 0,8 or alternatively $\xi_0 \chi_z$ but $\xi_{zc} \geq 0,8$
ω_x, ω_0 and ω_{xLT} = HAZ-softening factors, see 5.9.4.5

η_0, γ_0 and ξ_0 are defined according to the expression in 5.9.3.2.

The expression for flexural buckling, see 5.9.4(2), must also be checked.

5.9.4.4 Thin walled cross sections

(1) The interaction expressions according to 5.40 and 5.41 should be used. Local buckling is taken into account by the shape factors α_y and α_z. The coefficients η_0, γ_0 and ξ_0 should not be less than 1.

5.9.4.5 Members containing localized welds

(1) The value of ω_0, ω_x and ω_{xLT} for a beam-column, subject to HAZ softening, should generally be based on the ultimate strength of the HAZ softened material. On the safe side it could be referred to the most unfavourable section in the bay considered, even when such softening occurs only locally along the length. Then ω_0, ω_x and ω_{xLT} in the expressions in 5.44, 5.45 and 5.46 are

$$\omega_0 = \omega_x = \omega_{xLT} = \frac{\rho_{haz} f_a/\gamma_{M2}}{f_o/\gamma_{M1}} \quad \text{but} \leq 1,00 \tag{5.48}$$

where

ρ_{haz} is the reduction factor for the heat affected material according to 5.4.2.

(2) However, when HAZ softening occurs close to the ends of the bay, or close to points of contra flexure only, ω_x and ω_{xLT} may be increased in considering flexural and lateral-torsional buckling, provided that such softening does not extend a distance along the member greater than the least width of the section.

$$\omega_x = \frac{\omega_o}{\chi + (1-\chi)\sin\dfrac{\pi x_s}{l_c}} \tag{5.49}$$

$$\omega_{xLT} = \frac{\omega_o}{\chi_{LT} + (1-\chi_{LT})\sin\dfrac{\pi x_s}{l_c}} \tag{5.50}$$

$$\omega_o = \frac{\rho_{haz} f_a / \gamma_{M2}}{f_o / \gamma_{M1}} \quad \text{but } \omega_0 \leq 1{,}00 \tag{5.51}$$

where:

χ = χ_y or χ_z depending on buckling direction
χ_{LT} = reduction factor for lateral-torsional buckling of the beam-column in bending only
x_s = distance from the localized weld to a support or point of contra flexure for the deflection curve for elastic buckling of axial force only, compare figure 5.11
l_c = buckling length.

(3) Calculation of χ and χ_{LT} should be based on the yield strength of the parent material.

(4) When the length of the softening region is larger than the least width of the section, then the design strength f_a/γ_{M2} for local failure in the expressions for ω_0, ω_x and ω_{xLT} shall be replaced by the design strength f_o/γ_{M1} for overall yielding.

(5) When the localized softening region covers a part of the cross section (e.g. one flange) then the whole cross section is supposed to be softened.

5.9.4.6 Members containing localized reduction of cross section

(1) Members containing localized reduction of cross section, e.g. bolt holes or flange cut-outs, can be checked according to 5.9.4(5) by replacing ρ_{haz} in ω_0, ω_x, ω_{xLT} with A_{net}/A_g where:

A_{net} = net section area, with reduction of holes
A_g = gross section area

5.9.4.7 Unequal end moments and/or transverse loads

(1) For members subjected to combined axial force and unequal end moments and/or transverse loads, different sections along the beam-column are checked. The actual bending moment in the studied section is used in the interaction expressions. ω_x and ω_{xLT} are (compare 5.9.4(5))

$$\omega_x = \frac{1}{\chi + (1-\chi)\sin\dfrac{\pi x_s}{l_c}} \tag{5.52}$$

$$\omega_{xLT} = \frac{1}{\chi_{LT} + (1-\chi_{LT})\sin\dfrac{\pi x_s}{l_c}} \tag{5.53}$$

(2) x_s is the distance from the studied section to a simple support or point of contra flexure of the deflection curve for elastic buckling of axial force only, see figure 5.11.

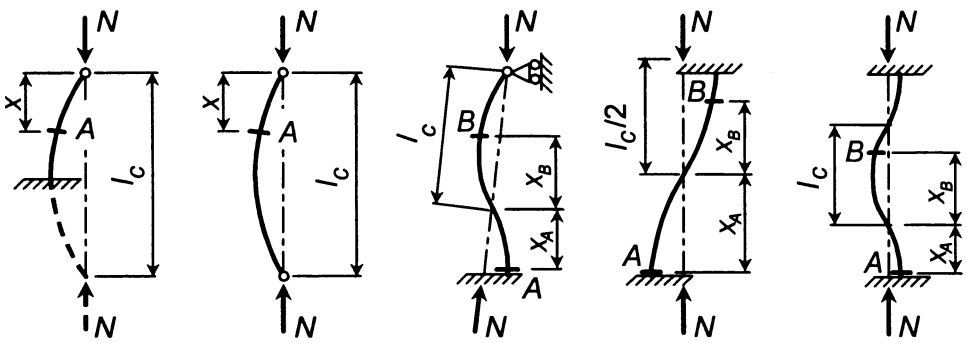


Figure 5.11: Buckling length l_c and definition of x

A and B in figure 5.11 are examples of studied sections marked by transverse lines.
See table 5.7 for value of buckling length $l_c = KL$.

5.10 Resistance of unstiffened plates under inplane loading

5.10.1 General

(1) In certain types of structure unstiffened plates can exist as separate components under direct stress, shear stress, or a combination of the two. The plates are attached to the supporting structure by welding, riveting, bolting or bonding, and the form of attachment can affect the boundary conditions. Thin plates must be checked for the ultimate limit states of bending under lateral loading, buckling under edge stresses in the plane of the plate, and for combinations of bending and buckling. The design rules given in this section only refer to rectangular plates.

5.10.2 Resistance under uniform compression

(1) A rectangular plate under uniform end compression is shown in figure 5.12. The length of the plate in the direction of compression = a, and the width across the plate = b. The thickness is assumed to be uniform, and equal to t. The plate can be supported on all four edges, where the support conditions are hinged, elastically restrained or fixed, or it can be free along one longitudinal edge.

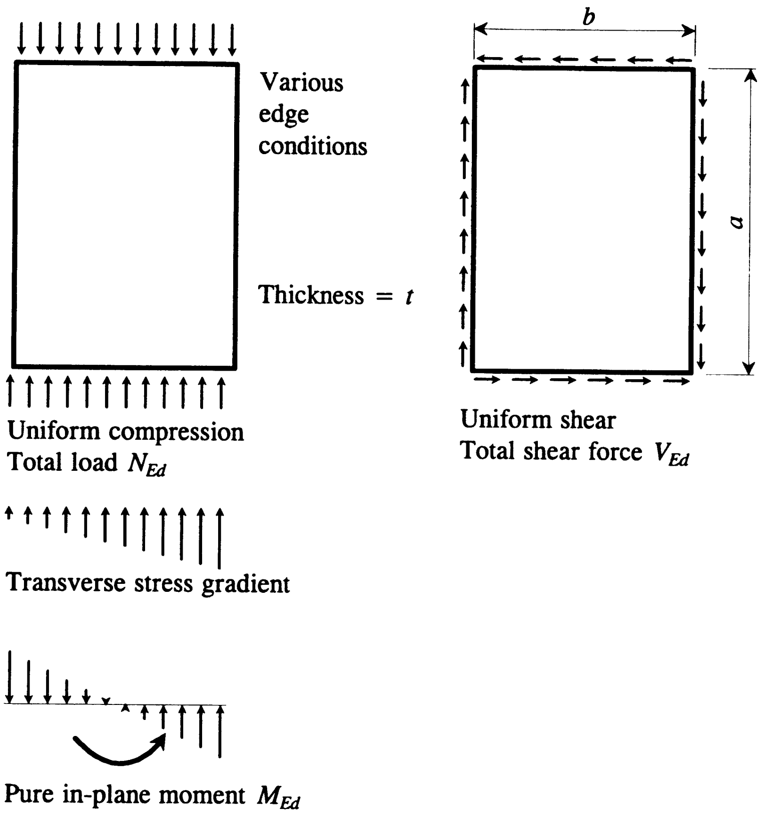


Figure 5.12: Unstiffened plates

(2) The susceptibility of the unstiffened plate to buckling is defined by the parameter β, where $\beta = b/t$. The classification of the cross section is carried out in the same way as that described in section 5.4.4, where plates with longitudinal edges simply supported, elastically restrained, or completely fixed are taken to correspond to "internal elements", and plates with one longitudinal edge free correspond to "outstands". Thus

$\beta \leq \beta_2$	class 1 or 2
$\beta_2 < \beta \leq \beta_3$	class 3
$\beta_3 < \beta$	class 4

where values of β_2 and β_3 are given in table 5.1.

(3)P The design value of the compression force N_{Ed} shall satisfy

$$N_{Ed} \leq N_{Rd} \tag{5.54}$$

where N_{Rd} is the lesser of

$$N_{o,Rd} = A_{ef} f_o / \gamma_{M1} \quad \text{(overall yielding and local buckling)} \tag{5.55}$$

and

$$N_{a,Rd} = A_{net} f_a / \gamma_{M2} \quad \text{(local failure)} \tag{5.56}$$

where:

A_{ef} is the effective area of the cross section taking account of local buckling for class 4 cross section and HAZ softening of longitudinal welds

A_{net} is the area of the least favourable cross section taking account of unfilled holes and HAZ softening of transverse or longitudinal welds if necessary

f_o is the characteristic strength for overall yielding (see section 5.3.5)

f_a is the characteristic strength for local failure (see section 5.3.5)

(4) A_{ef} for class 4 cross section is obtained by taking a reduced thickness to allow for buckling as well as for HAZ softening, but with the presence of holes ignored. A_{ef} is generally based on the least favourable cross section, taking a thickness equal to the lesser of $\rho_c t$ and $\rho_{haz} t$ in HAZ regions, and $\rho_c t$ elsewhere. In this check HAZ softening due to welds at the loaded edges may be ignored.

The factor ρ_c is found from the more favourable of the following treatments:

a) Calculate ρ_c from 5.4.5 (3) or read from figure 5.5, using the internal element expressions for plates that are simply supported, elastically restrained, or fixed along longitudinal edges, and the outstand element expressions for plates with one longitudinal free edge.

b) Take $\rho_c = \chi$, where χ is the column buckling reduction factor from section (5.8.3). In calculating χ take a slenderness parameter λ equal to 3,5 *a/t*, which corresponds to simple support at the loaded edges. For restrained loaded edges a lower value of λ can be used at the discretion of the designer.

5.10.3 Resistance under in-plane moment

(1) If a pure in-plane moment acts on the ends (width = b) of a rectangular unstiffened plate (see figure 5.12) the susceptibility to buckling is defined by the parameter β, where $\beta = 0{,}40\ b/t$. The classification of the cross section is carried out in the same way as described in section 5.10.2.

(2) The design value of the bending moment M_{Ed} should satisfy

$$M_{Ed} \leq M_{Rd} \tag{5.57}$$

where the design bending moment resistance M_{Rd} is the lesser of $M_{o,Rd}$ and $M_{a,Rd}$ according to 5.10.3 (3) and (4).

(3) The design bending moment resistance $M_{o,Rd}$ for overall yielding and local buckling is as follows:

Class 1 and 2 cross section

$$M_{o,Rd} = W_{pl} f_o / \gamma_{M1} \tag{5.58}$$

Class 3 cross section

$$M_{o,Rd} = \left[W_{el} + \frac{\beta_3 - \beta}{\beta_3 - \beta_2} \left(W_{pl} - W_{el} \right) \right] f_o / \gamma_{M1} \tag{5.59}$$

Class 4 cross section

$$M_{o,Rd} = W_{ef} f_o / \gamma_{M1} \tag{5.60}$$

where:

W_{pl} and W_{el} are the plastic and elastic moduli for the gross cross section or a reduced cross section to allow for HAZ softening from longitudinal welds, but with the presence of holes ignored

W_{ef} is the elastic modulus for the effective cross section obtained by taking a reduced thickness to allow for buckling as well as HAZ softening from longitudinal welds when required, but with the presence of holes ignored

β is the slenderness factor for the most critical element in the section

β_2 and β_3 are the class 2 and class 3 limiting values of β for that element
f_o is the characteristic strength for overall yielding

(4) The design bending moment resistance $M_{a,Rd}$ for local failure at sections with holes or transverse welds is:

$$M_{a,Rd} = W_{net} f_a / \gamma_{M2} \tag{5.61}$$

where

W_{net} is the plastic modulus allowing for holes and taking a reduced thickness $\rho_{haz}t$ in any region affected by HAZ softening.

5.10.4 Resistance under transverse or longitudinal stress gradient

(1) When the applied actions at the end of a rectangular plate result in a transverse stress gradient, the susceptibility to buckling is defined by $\beta = gt$, where g is found from section 5.4.3. Having calculated β proceed as in 5.10.2 above.

(2) When the applied compression or in-plane bending moment varies longitudinally along the plate (i.e. in the direction of the dimension a), the design moment resistance for class 1, 2 or 3 cross sections at any cross section should not be less than the action arising at that section under factored loading. For class 4 cross sections the yielding check may be satisfied at every cross section, but for the buckling check it is permissible to compare the design compressive or moment resistance with the action arising at a distance from the more heavily loaded end of the plate equal to 0,4 times the elastic plate buckling half wavelength.

5.10.5 Resistance under shear

(1) A rectangular plate under uniform shear forces is shown in figure 5.12. The thickness is assumed to be uniform and the support conditions along all four edges are either simply supported, elastically restrained or fixed.

(2) The susceptibility to shear buckling is defined by the parameter β, where $\beta = b/t$ and b is the shorter of the side dimensions. For all edge conditions the classification of the cross section is as follows:

$\beta \leq 49\varepsilon$ classes 1, 2 and 3

$\beta > 49\varepsilon$ class 4

where:

$\varepsilon = \sqrt{150/f_v}$
f_v = the characteristic strength in shear in N/mm^2.

(3) The design value of the shear force V_{Ed} at each cross section should satisfy

$$V_{Ed} \leq V_{Rd} \tag{5.62}$$

where V_{Rd} is the design shear resistance of the cross section based on the least favourable cross section as follows.

a) Class 1, 2 or 3 cross sections: (5.63)

where f_v is the characteristic strength in shear (see 5.3.5), and A_{net} is the net effective area allowing for holes, and taking a reduced thickness $\rho_{haz}t$ in any area affected by HAZ softening. If the HAZ extends around the entire perimeter of the plate the reduced thickness is assumed to extend over the entire cross section. In allowing for holes, the presence of small holes may be ignored if their total cross sectional area is less than 20% of the total cross sectional area bt.

b) Class 4 cross sections:

values of V_{Rd} for both yielding and buckling should be checked. For the yielding check use a) above for class 1, 2 or 3 cross sections. For the buckling check:

$$V_{Rd} = v_1 bt f_v / \gamma_{M1} \tag{5.64}$$

where:

$$v_1 = \left[5{,}35 + 4(b/a)^2\right] \frac{430\, t^2 \varepsilon^2}{b^2} \qquad \text{for } 0{,}40 \le b/a < 1{,}0$$

$$v_1 = \left[5{,}35\,(b/a)^2 + 4\right] \frac{430\, t^2 \varepsilon^2}{b^2} \qquad \text{for } 1{,}0 \le b/a$$

These expressions do not take advantage of tension field action, but if it is known that the edge supports for the plate are capable of sustaining a tension field, the treatment given in 5.12.3 can be employed.

5.10.6 Resistance under combined action

(1) A plate subjected to combined axial force and in-plane moment under factored loading should be given a separate classification for the separate actions in accordance with 5.10.2. In so doing, the value of β should be based on the pattern of edge stress produced when the force (N_{Ed}) and the moment (M_{Ed}) act separately.

(2) When the plate is class 4, each individual resistance, $N_{o,Rd}$ and $M_{o,Rd}$ should be based on the specific type of action considered (see 5.9).

(3) If the combined action is axial force and in plane moment, the following conditions should be satisfied:

$$(N_{Ed} / N_{c,Rd}) + (M_{Ed} / M_{c,Rd}) \le 1{,}0 \tag{5.65}$$

(4) If the combined action includes the effect of a coincident shear force, V_{Ed}, then V_{Ed} may be ignored if it does not exceed 0,5 V_{Rd} (see 5.9.5).

If $V_{Ed} > 0{,}5\ V_{Rd}$, the values of $N_{c,Rd}$ and $M_{c,Rd}$ may each be reduced by the following factor:

Shear reduction factor = $1{,}6 - 1{,}2\ V_{ed}/V_{Rd}$.

5.11 Resistance of stiffened plates under inplane loading

5.11.1 General

(1) The following rules concern plates, supported on all four edges and reinforced with one or two, central or eccentric longitudinal stiffeners, or three or more equally spaced longitudinal stiffeners or corrugations (see figure 5.13). Also general rules for orthotropic plating (figure 5.13(c), (d) and 5.11.6) are given. Rules for extruded profiles with one or two open stiffeners are given in 5.3.5.

(2) The stiffeners may be unsupported on their whole length or else be continuous over intermediate transverse stiffeners. The dimension L should be taken as the spacing of the supports when fitted. An essential feature of the design is that the longitudinal reinforcement, but not transverse stiffening, is "sub-critical", i.e. it can deform with the plating in an overall buckling mode.

(3) The resistance of such plating to longitudinal direct stress in the direction of the reinforcement is given in 5.11.2 to 5.11.4, and the resistance in shear is given in 5.11.5. Interaction between different effects may be allowed for in the same way as for unstiffened plates (see 5.10.6). The treatments are valid also if the cross section contains elements that are classified as slender.

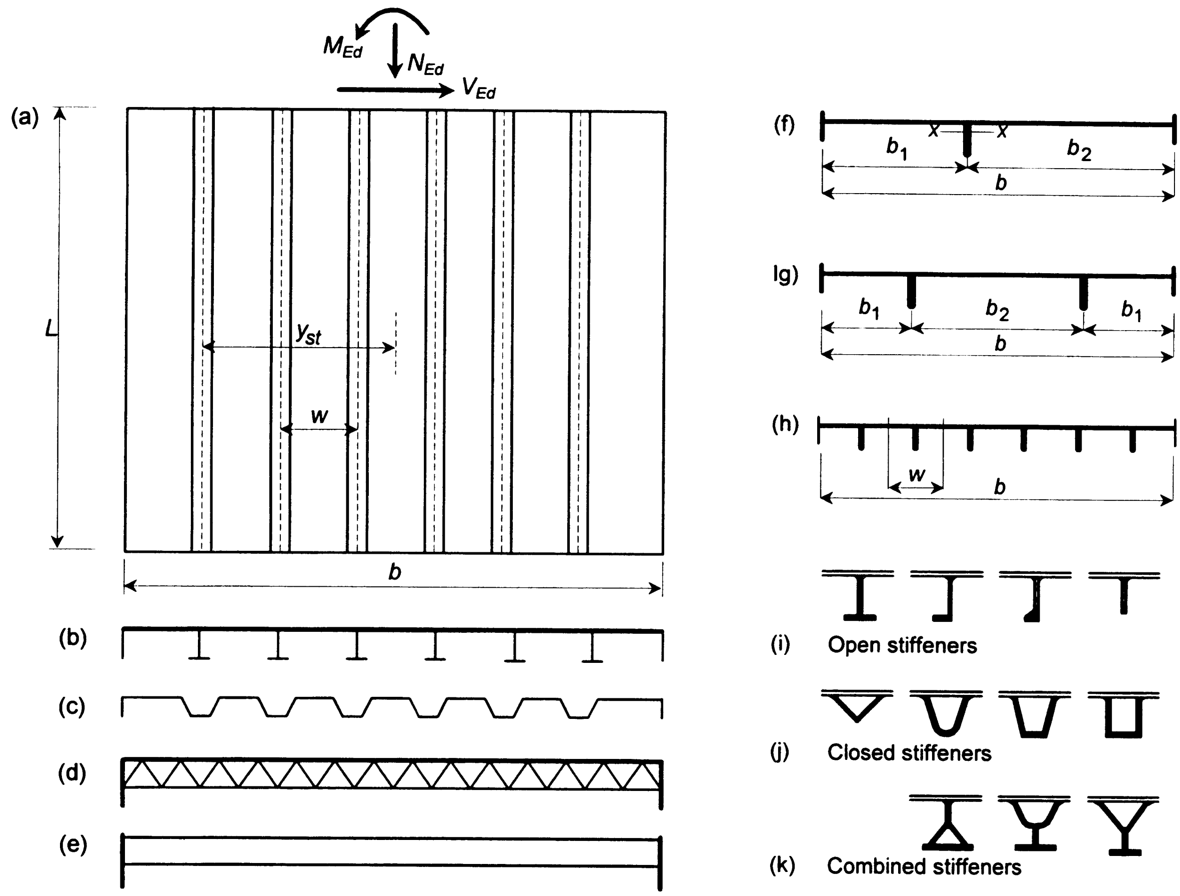


Figure 5.13: Stiffened plate and types of stiffeners

(4) When the construction consists of flat plating with applied stiffeners, the resistance to transverse direct stress may be taken the same as for an unstiffened plate. With corrugated construction it is negligible. Orthotropic plating and sandwich panels may have considerable resistance to transverse load.

5.11.2 Stiffened plates under uniform compression

(1)P General

The cross section shall be classified as compact or slender in accordance with 5.4, considering all the component elements before carrying out either check.

The design value of the compression force N_{Ed} shall satisfy

$$N_{Ed} \leq N_{Rd} \tag{5.66}$$

where N_{Rd} is the lesser of $N_{a,Rd}$ and $N_{o,Rd}$ according to 5.11.2(2) and (3).

(2) Yielding check

The entire section should be checked for local squashing in the same way as for a strut (see 5.8). The design resistance $N_{a,Rd}$ should be based on the net section area A_{net} for the least favourable cross section, taking account of local buckling and HAZ softening if necessary, and also any unfilled holes.

$$N_{a,Rd} = A_{net} f_a / \gamma_{M2} \qquad (5.67)$$

where f_a is the characteristic strength for local failure.

(3) Column check

The plating is regarded as an assemblage of identical column sub-units, each containing one centrally located stiffener or corrugation and with a width equal to the pitch w. The design axial resistance $N_{c,Rd}$ is then taken as:

$$N_{c,Rd} = A_{ef} \chi_o f_o / \gamma_{M1} \qquad (5.68)$$

where:

- χ_c is the reduction factor for column buckling
- A_{ef} is the effective area of the cross section of the plating. HAZ softening due to welds at the loaded edges or at transverse stiffeners may be ignored in finding A_{ef}. Also unfilled holes may be ignored.
- f_o characteristic strength for overall yielding of plate material

The reduction factor χ_c should be obtained from the appropriate column curve relevant to column buckling of the sub-unit as a simple strut out of the plane of the plating.

(4) The slenderness parameter λ_c in calculating χ_c is

$$\lambda_c = \sqrt{\frac{A_{ef} f_o}{N_{cr}}} \qquad (5.69)$$

where

$N_{cr} =$ the elastic orthotropic buckling load based on the gross cross section except for slender outstand elements where effective thickness should be used.

(5) For a plate with *open stiffeners*:

$$N_{cr} = \frac{\pi^2 E I_x}{L^2} + \frac{L^2 c}{\pi^2} \qquad \text{when } L < \pi \sqrt[4]{\frac{EI_x}{c}} \qquad (5.70)$$

$$N_{cr} = 2\sqrt{c E I_x} \qquad \text{when } L \geq \pi \sqrt[4]{\frac{EI_x}{c}} \qquad (5.71)$$

where c is the elastic support from the plate according to expressions (5.72), (5.73) or (5.74) and I_x is the second moment of area of all stiffeners within the plate width b.

(6) For an element with *one central or eccentric stiffener* (figure 5.13(f)):

$$c = \frac{0{,}27 E t^3 b}{b_1^2 b_2^2} \qquad (5.72)$$

where t is the thickness of the plate, b is the overall width of the plate and b_1 and b_2 are the width of plate elements on both sides of the stiffener.

(7) For an element with *two symmetrical stiffeners* located a distance b_1 from the longitudinal supports (figure 5.13(g)):

$$c = \frac{1{,}1\ Et^3}{b_1^2(3b - 4b_1)} \tag{5.73}$$

(8) For a *multi-stiffened plate with open stiffeners* (figure 5.13(h) and (i)) with small torsional stiffness

$$c = \frac{8{,}9\ Et^3}{b^3} \tag{5.74}$$

(9) For a *multi-stiffened plate with closed or partly closed stiffeners* (figure 5.13(j))

N_{cr} is the elastic orthotropic buckling load. See 5.11.6.

(10) The half-wavelength in elastic buckling is

$$l_w = \pi \sqrt[4]{\frac{EI_x}{c}} \tag{5.75}$$

This half-wave length is used when the applied action varies in the direction of the stiffener or corrugations. See 5.11.4(3).

5.11.3 Stiffened plates under in-plane moment

(1)P General

Two checks shall be performed, a yielding check (see 5.11.3(3)) and a column check (see 5.11.3(4)).

(2) Section classification and local buckling

The cross section should be classified as compact, semi-compact or slender (see 5.3) when carrying out either check. For the purpose of classifying individual elements, and also when determining effective thicknesses for slender elements, it may generally be assumed that each element is under uniform compression taking $g = 1$ in 5.3.3. However, in the case of the yielding check only, it is permissible to base g on the actual stress pattern in elements comprising the outermost region of the plating, and to repeat this value for the corresponding elements further in. This may be favourable when the number of stiffeners or corrugations is small.

(3) Yielding check

The entire cross section of the plating should be treated as a beam under in-plane bending (see 5.5). The design moment resistance M_{Rd} should be based on the least favourable cross section, taking account of local buckling and HAZ softening if necessary, and also any holes.

(4) Column check

The plating is regarded as an assemblage of column sub-units in the same general way as for axial compression (see 5.11.2(3)), the design moment resistance $M_{c,Rd}$ being taken as follows

$$M_{c,Rd} = \frac{\chi_c I_{ef} f_o}{y_{st} \gamma_{M1}} \tag{5.76}$$

where:

χ_c is the reduction factor for column buckling of sub-unit
I_{ef} is the second moment of area of the effective cross section of the plating for in-plane bending
y_{st} is the distance from centre of plating to centre of outermost stiffener
f_o is the characteristic strength for overall yielding of plate material

The reduction factor χ_c should be determined in the same way as for uniform compression (see 5.11.2(3)).

5.11.4 Longitudinal stress gradient on multi-stiffened plates

(1) General

Cases where the applied action N_{Ed} or M_{Ed} on a multi-stiffened plate varies in the direction of the stiffeners or corrugations are described in 5.11.4(2) and 5.11.4(3).

(2) Yielding check

The design resistance at any cross section should be not less than the design action effect arising at that section.

(3) Column check

For the column check it is sufficient to compare the design resistance with the design action effect arising at a distance x from the more heavily loaded end of a panel, where x is 0,4 times the half wavelength l_w in elastic buckling according to 5.11.2(10).

5.11.5 Multi-stiffened plating in shear

(1)P General

A yielding check (see 5.11.5(2)) and a buckling check (see 5.11.5(3)) shall be performed. The methods given in 5.11.5(2) and (3) are valid provided the stiffeners or corrugations, as well as the actual plating, are as follows:

a) effectively connected to the transverse framing at either end;
b) continuous at any transverse stiffener position.

(2) Yielding check

The design shear force resistance V_{Rd} is taken as the same as that for a flat unstiffened plate of the same overall aspect ($L \times b$) and the same general thickness t, found in accordance with 5.10.5(2).

(3) Buckling check

The design shear force resistance is found from 5.12.10.

In order to calculate the resistance the following values should be used:

I_z = $t^3/10{,}9$ for a flat plate with stiffeners, otherwise according to 5.12.7(3) with $t_w = t$
I_x is the second moment of area per unit width of plating and stiffener about a centroidal axis parallel to the plane of the plating
h_w is the effective length l which may be safely taken as the unsupported length L (see figure 5.13). When L greatly exceeds b, a more favourable result may be obtained by putting $V_{o,cr}$ equal to the elastic orthotropic shear buckling force. No allowance for HAZ softening needs to be made in performing the buckling check.

5.11.6 Orthotropic plates

(1) For an orthotropic plate under *uniform compression* the procedure in 5.11.2 could be used. The elastic orthotropic buckling load N_{cr} for a simply supported orthotropic plate is given by

$$N_{cr} = \frac{\pi^2}{b}\left[\frac{B_x}{(L/b)^2} + 2H + B_y(L/b)^2\right] \qquad \text{when } \frac{L}{b} < \sqrt[4]{\frac{B_x}{B_y}} \tag{5.77}$$

$$N_{cr} = \frac{2\pi^2}{b}\left[\sqrt{B_x B_y} + H\right] \qquad \text{when } \frac{L}{b} \geq \sqrt[4]{\frac{B_x}{B_y}} \tag{5.78}$$

Expressions for B_x, B_y and H for different cross sections are given in table 5.10 where Eq.1, Eq.2, Eq.3, and Eq.4 are given below.

Eq.1:

$$B_y = \frac{2Ba}{2a_4 + \dfrac{2a_1 a_3 t_1^3(4a_2 t_3^3 - a_3 t_2^3)}{a_3 t_1^3(4a_2 t_3^3 - a_3 t_2^3) + a_1 t_3^3(12a_2 t_3^3 - 4a_3 t_2^3)}} \tag{5.79a}$$

where $B = \dfrac{Et_1^3}{12(1-\nu^2)}$

Eq.2:

$$H = 2B + \frac{\dfrac{GI_T}{2a}}{1 + \dfrac{1{,}6GI_T a_4^2}{L^2 aB}\left[1 + \dfrac{1}{10C_1/L^4 + C_2}\right]} \tag{5.79b}$$

where

$$C_1 = 4(1-\nu^2)(a_2 + a_3)a_1^2 a_4^2 h^2 t_2/(3at_1^3) \tag{5.79c}$$

$$B = \frac{Et_1^3}{12(1-\nu^2)} \tag{5.79d}$$

$$C_2 = \frac{4(a_1 + a_2)^2 a_1 a_4 (1 + a_1/a_2 + a_2/a_1 + a^2/(a_1 a_3))}{a_2^3(3a_3 + 4a_2)}\left(\frac{t_2}{t_1}\right)^3 \tag{5.79e}$$

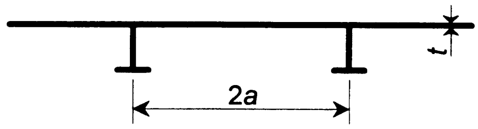
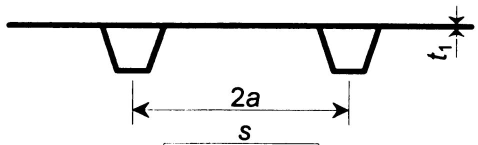
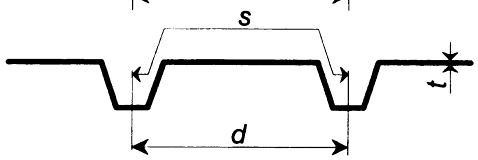
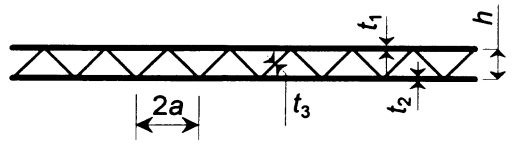
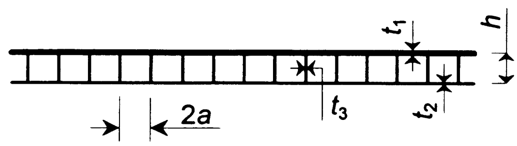
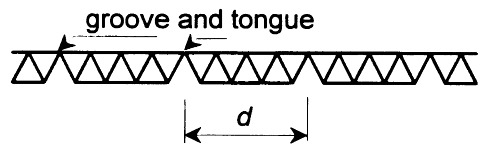
Eq.3:

$$B_y = \frac{Et_1^3}{12(1-\nu^2)}\;\frac{10b^2}{32a^2}\;\frac{at_3^3 + at_2^3 t_3^3/t_1^3 + 6ht_2^3}{at_3^3 + 2h(t_1^3 + t_2^3) + 3h^2 t_1^3 t_2^3/(at_3^3)} \tag{5.80a}$$

Eq.4:

$$H = \frac{2E}{3\left(1 - \dfrac{t_3}{2a}\right)^3} \left[\frac{t_1^3}{1 + \dfrac{6t_1}{2a - t_3}} + \frac{t_2^3}{1 + \dfrac{6t_2}{2a - t_3}} \right] \tag{5.80b}$$

Table 5.10: Flexural and torsional rigidity of orthotropic plates

Cross section	B_x	B_y	H
	$\frac{EI_L}{2a}$	$\frac{Et^3}{12(1-\nu^2)}$	$\frac{Gt^3}{6}$
	$\frac{EI_L}{2a}$	Eq.1	Eq.2
	$\frac{EI_L}{d}$	$\frac{d}{s}\frac{Et^3}{12(1-\nu^2)}$	$\frac{d}{s}\frac{Gt^3}{6}$
	$\frac{EI_L}{2a}$	$\frac{Et_1t_2h^2}{t_1+t_2}$	$\frac{GI_T}{2a}$
	$\frac{EI_L}{2a}$	Eq.3	Eq.4
	$\frac{EI_L}{d}$	0	$\frac{GI_T}{d}$

I_L is the second moment of area of one stiffener and adjacent plating (within $2a$ or d) in the longitudinal direction

I_T is the torsional stiffness of the same cross section.

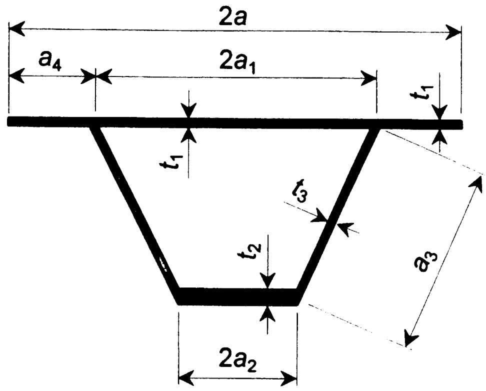


Figure 5.14: Cross section notations of stiffeners

(2) The *shear force* resistance for an orthotropic plate with respect to global buckling can be calculated according to 5.12.7 where:

$$V_{o,cr} = \frac{k_\tau \pi^2}{b} \sqrt[4]{B_x B_y^3} \tag{5.81}$$

$$k_\tau = 3{,}25 - 0{,}567\phi + 1{,}92\phi^2 + (1{,}95 + 0{,}1\phi + 2{,}75\phi^2)\,\eta \tag{5.82}$$

$$\phi = \frac{L}{b}\sqrt[4]{\frac{B_y}{B_x}} \quad \text{(Valid for } 0 < \phi < 1{,}2\text{)} \tag{5.83}$$

$$\eta = \frac{H}{\sqrt{B_x B_y}} \quad \text{(Valid for } 0 < \eta < 1{,}5\text{)} \tag{5.84}$$

B_y, B_x and H are given in table 5.10.

(3) For an orthotropic plate with a free edge forming a part of a channel structure as in figure 5.15, overall buckling should be based on cross section properties given in table 5.10.

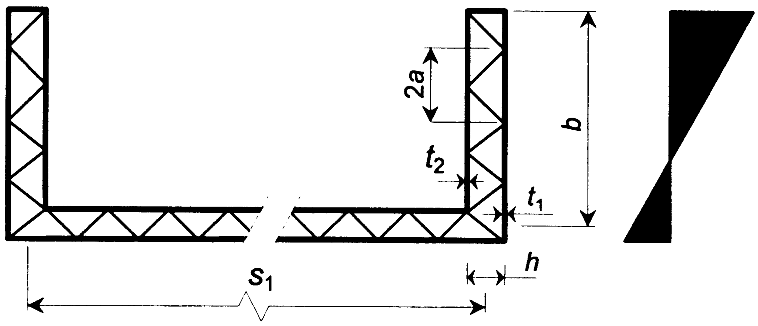


Figure 5.15: Orthotropic plate with a free edge

5.12 Resistance of plate girders

5.12.1 General

(1) A plate girder is a fabricated deep beam with a tension flange, compression flange, and a web plate. The web is usually of slender proportions and may be reinforced transversely with bearing and intermediate stiffeners. It can also be further reinforced by longitudinal stiffeners as indicated in figure 5.16.

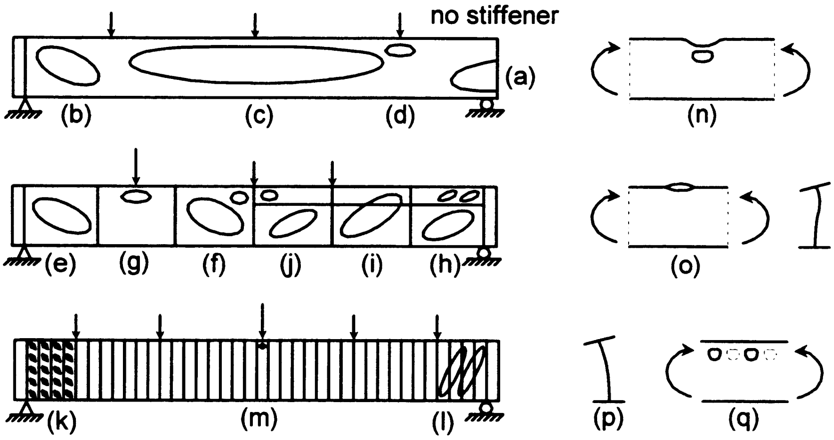


Figure 5.16 (a)-(n) and (q): Web buckling modes and (o)-(p) flange buckling modes

(2) Webs buckle in shear at relatively low applied loads, but considerable amount of post-buckled strength can be mobilized due to tension field action. Plate girders are sometimes constructed with transverse web reinforcement in the form of corrugations or closely-spaced transverse stiffeners.

(3) Plate girders can be subjected to combinations of moment, shear and axial loading, and to local loading on the flanges. Because of their slender proportions they may be subjected to lateral torsional buckling, figure 5.16(p), unless properly supported along their length.

(4) The treatment of plate girders given here is also generally applicable to the side members of box girders.

(5) The resistance of plate girder webs depends on depth-to-thickness ratio b_w/t_w and web stiffener arrangements.

Failure modes and references to clauses with resistance expressions are given in table 5.11.

Table 5.11: Buckling modes (figure 5.16) and corresponding clause with resistance expressions

Buckling mode	Figure 5.16	Clause
Web buckling by compressive stresses	q	5.12.2 and 5.12.3
Shear buckling	b, d, e, h, k, l	5.12.4, 5.12.5 and 5.12.10
Interaction between shear force and bending moment	f, j	5.12.7
Buckling of web because of local loading on flanges	d, g, m	5.12.8
Curvature induced web buckling	c, n	5.12.9
Torsional buckling of flange	o	5.4.5
Lateral torsional buckling	p	5.6.6

5.12.2 Resistance of girders under in-plane bending

(1) A yielding check and a buckling check should be made, and for webs with continuous longitudinal welds the effect of the HAZ should be investigated. The HAZ effect caused by the welding of transverse stiffeners may be neglected and small holes in the web may be ignored provided they do not occupy more than 20 % of the cross sectional area of the web. The web depth between flanges is b_w.

(2)P For the yielding check, the design value of the moment, M_{Ed} at each cross section shall satisfy

$$M_{Ed} \leq M_{o,Rd} \tag{5.85}$$

where $M_{o,Rd}$ is the design moment resistance of the cross section that would apply if the section were designated Class 3. Thus,

$$M_{o,Rd} = W_{el} f_o / \gamma_{M1} \tag{5.86}$$

where W_{el} is the elastic modulus allowing for holes and taking a reduced thickness $\rho_{haz} t$ in regions adjacent to the flanges which might be affected by HAZ softening (see 5.5.2).

(3) In applying the buckling check it is assumed that transverse stiffeners comply with the requirements of the effective stiffener section given in 5.12.6. It is also assumed that the spacing between adjacent transverse stiffeners is greater than half the clear depth of the web between flange plates. If this is not the case, refer to 5.12.10 for corrugated or closely stiffened webs.

(4) For each bay of the girder of length a between transverse stiffeners, the moment arising under design load at a distance 0,4 a from the more heavily stressed end should not exceed the design moment resistance, $M_{o,Rd}$ for that bay, where

$$M_{o,Rd} = W_{ef} f_o / \gamma_{M1}$$

W_{ef} is the effective elastic modulus obtained by taking a reduced thickness to allow for buckling as well as HAZ softening, but with the presence of holes ignored. The reduced thickness is equal to the lesser of ρ_{haz} t and $\rho_c t$ in HAZ regions, and $\rho_c t$ elsewhere.

(5) The thickness is reduced in the compressed part b_c of the web only.

The stress ratio ψ used in 5.4.3 and corresponding width b_c may be obtained using the effective area of the compression flange and the gross area of the web, see figure 5.17.

(6) If the compression edge of the web is nearer to the neutral axis of the girder than in the tension flange, see figure 5.17(c) the plate slenderness λ_p of an element may be replaced by

$$\lambda_{p,red} = \lambda_p \sqrt{\sigma_{com,Ed} / f_{yd}} \tag{5.87}$$

where

$\sigma_{com,Ed}$ is the maximum design compressive stress in the element using effective areas of all the compression elements.

This procedure generally requires an iterative calculation in which ψ is determined again at each step from the stresses calculated on the effective cross-section defined at the end of the previous step.

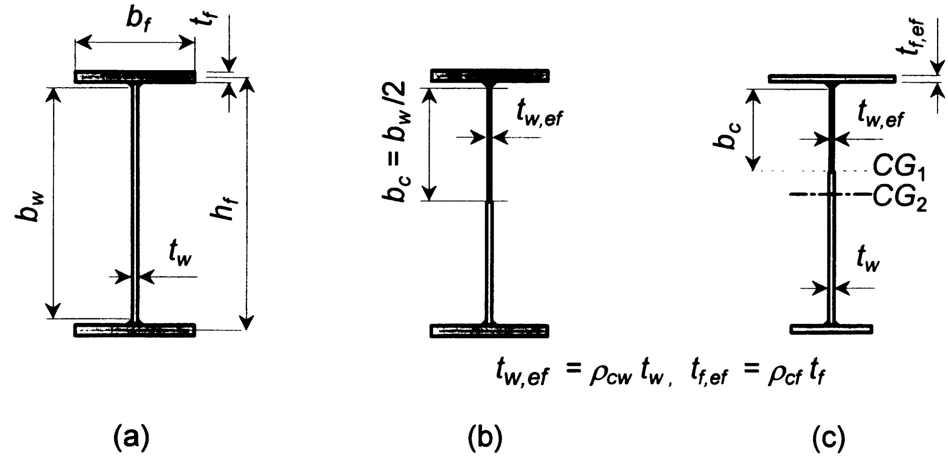


Figure 5.17: (a) Cross section notations. (b) Effective cross section for a symmetric plate girder with class 1, 2 or 3 flanges. (c) Effective cross section for a plate girder with smaller tension flange and class 4 compression flange.

5.12.3 Resistance of girders with longitudinal web stiffeners

(1) Plate buckling due to longitudinal compressive stresses may be taken into account by the use of an effective cross-section applicable to class 4 cross-sections.

(2) The effective cross-section properties should be based on the effective areas of the compression elements and their locations within the effective cross-section.

(3) In a first step the effective areas of flat compression sub panels between stiffeners should be obtained using effective thicknesses according to 5.4.5. See figure 5.18.

(4) Overall plate buckling, including buckling of the stiffeners, is considered as column buckling of a fictitious column consisting of the stiffeners and half the adjacent part of the web. If the stresses change from compression to tension within the sub panel, one third of the compressed part is taken as part of the fictitious column. See figure 5.18(c).

(5) The effective thicknesses of the different parts of the fictitious column section are further reduced in a second step with a reduction factor χ_c obtained from the appropriate column curve relevant for column buckling of the fictitious column as a simple strut out of the plane of the web.

(6) The non-dimensional slenderness parameter λ_c in calculating χ_c is

$$\lambda_c = \sqrt{\frac{A_{st,ef} f_o}{N_{cr}}} \tag{5.88}$$

where

$A_{st,ef}$ is the effective area of the fictitious column from the first step. N_{cr} is the elastic buckling load given by the following expression:

$$N_{cr} = 1{,}05\, E \frac{\sqrt{I_{st}\, t_w^3\, b_w}}{b_1\, b_2} \qquad \text{if} \quad a > a_c \tag{5.89}$$

$$N_{cr} = \frac{\pi^2 E I_{st}}{a^2} + \frac{E t_w^3 b_w a^2}{4\pi^2 (1 - \nu^2)\, b_1^2\, b_2^2} \qquad \text{if} \quad a \leq a_c \tag{5.90}$$

$$a_c = 4{,}33 \sqrt[4]{\frac{I_{st}\, b_1^2\, b_2^2}{t_w^3\, b_w}} \tag{5.91}$$

where:

I_{st} is second moment area of the gross cross section of the fictitious column (see 5.12.3(7)) about an axis through its centroid and parallel to the plane of the web

b_1 and b_2 are distances from longitudinal edges to the stiffener ($b_1 + b_2 = b_w$).

(7) When calculating I_{st} the column consists of the actual stiffener together with an effective width 15 t_w of the web plate on both sides of the stiffener. See figure 5.18(d1) and (d2).

(8) In case of two longitudinal stiffeners, both in compression, the two stiffeners are considered as lumped together, with an effective area and a second moment of area equal to the sum of those of the individual stiffeners. The location of the lumped stiffener is the position of the resultant of the axial forces in the stiffeners. If one of the stiffeners is in tension the procedure will be conservative.

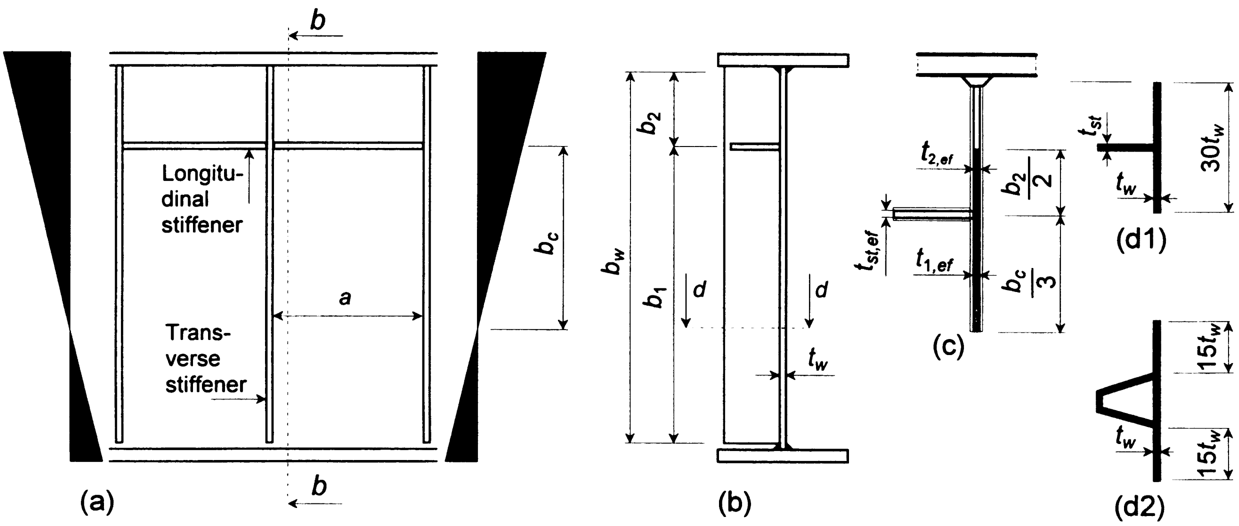


Figure 5.18 (a) Stiffened web (b) cross section (c) effective area of fictitious column (d) fictitious column cross section for calculation of I_{st} (d1) open stiffener (d2) closed stiffener

5.12.4 Shear resistance of plate girders with stiffeners at supports only

(1) For webs with transverse stiffeners at supports only, the design shear buckling resistance V_{Rd} should be obtained from

$$V_{Rd} = \rho_v\, t_w\, b_w f_{ow} / \gamma_{M1} \tag{5.92}$$

where ρ_v is a reduction factor for shear buckling according to table 5.12 and figure 5.20.

Table 5.12: Reduction factor ρ_v for shear buckling

λ_w	Rigid end post	Non-rigid end post
$\lambda_w \leq 0{,}48/\eta$	η	η
$0{,}48/\eta < \lambda_w < 0{,}949$	$0{,}48/\lambda_w$	$0{,}48/\lambda_w$
$0{,}949 \leq \lambda_w$	$1{,}32/(1{,}66+\lambda_w)$	$0{,}48/\lambda_w$

$\eta = 0{,}4 + 0{,}2 f_{uw}/f_{ow}$ but not more than 0,7 where
f_{ow} is the strength for overall yielding and f_{uw} is the ultimate strength of the web material.

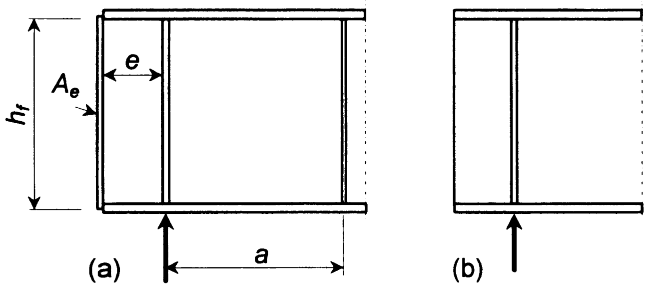


Figure 5.19: (a) Rigid and (b) non-rigid end post

(2) A distinction should be made between:

a) rigid end posts according to 5.12.5(6). This case is also applicable for panels not at the end of the girder and at an intermediate support of a continuous girder.

b) non rigid end posts according to 5.12.5(7).

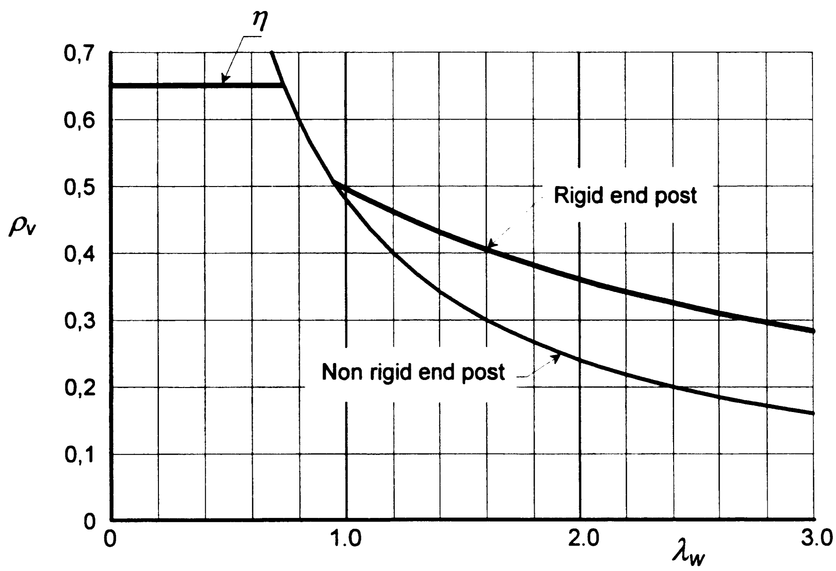


Figure 5.20: Reduction factor ρ_v for shear buckling

(3) The slenderness parameter λ_w in table 5.12 and figure 5.20 is

$$\lambda_w = 0{,}35 \frac{b_w}{t_w} \sqrt{\frac{f_o}{E}} \tag{5.93}$$

5.12.5 Shear resistance of webs with intermediate stiffeners

(1) For beams with transverse and longitudinal stiffeners the design shear buckling resistance V_{Rd} is the sum of the contribution $V_{w,Rd}$ of the web and $V_{f,Rd}$ of the flanges.

$$V_{Rd} = V_{w,Rd} + V_{f,Rd} \tag{5.94}$$

(2) $V_{w,Rd}$ includes partial tension field action in the web according to 5.12.5(3). $V_{f,Rd}$ is an increase of the tension field caused by local bending resistance of the flanges. See 5.12.5(8).

(3) The shear design resistance of the web is

$$V_{w,Rd} = \rho_v t_w b_w f_{ow} / \gamma_{M1} \tag{5.95}$$

where ρ_v is the effective thickness factor for shear buckling according to table 5.12 and figure 5.20.

(4) The slenderness parameter λ_w is

$$\lambda_w = \frac{0{,}81}{\sqrt{k_\tau}} \frac{b_w}{t_w} \sqrt{\frac{f_o}{E}} \tag{5.96}$$

(5) The shear buckling coefficient k_τ in (4) is:

$$k_\tau = 5{,}34 + 4{,}00(b_w/a)^2 + k_{\tau st} \quad \text{when} \quad a/b_w \geq 1 \tag{5.97}$$

$$k_\tau = 4{,}00 + 5{,}34(b_w/a)^2 + k_{\tau st} \quad \text{when} \quad a/b_w < 1 \tag{5.98}$$

where

$$k_{\tau st} = 9\left(\frac{b_w}{a}\right)^2 \left(\frac{I_{st}}{t_w^3 b_w}\right)^{\frac{3}{4}} \text{ but not less than } \frac{2{,}1}{t_w}\left(\frac{I_{st}}{b_w}\right)^{\frac{1}{3}} \tag{5.99}$$

a is the distance between transverse stiffeners. See figure 5.21.

I_{st} is the second moment of area of the longitudinal stiffener with regard to the z-axis. See figure 5.21(b). For webs with two or more equal stiffeners, not necessarily equally spaced, I_{st} is the sum of the stiffness for the individual stiffeners.

(6) For webs with longitudinal stiffeners the slenderness parameter λ_w should be taken not less than

$$\lambda_w = \frac{0{,}81}{\sqrt{k_{\tau 1}}} \frac{b_{w1}}{t_w} \sqrt{\frac{f_o}{E}} \tag{5.100}$$

where the shear buckling coefficient $k_{\tau 1}$ refers to the largest sub-panel with depth b_{w1} and length a, see figure 5.21. Expression in 5.12.5(5) may be used with $k_{\tau st} = 0$.

(7) If the flanges are not completely utilized by bending moment ($M_{Ed} < M_{f,Rd}$) there is a shear resistance contribution $V_{f,Rd}$ of the flanges obtained from

$$V_{f,Rd} = \frac{b_f t_f^2 f_{of}}{c\,\gamma_{M1}} \left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right) \tag{5.101}$$

where

$$c = \left(0,08 + \frac{4,4\, b_f t_f^2 f_{of}}{t_w b_w^2 f_{ow}} \right) a$$

b_f, t_f are taken for the smaller flange.

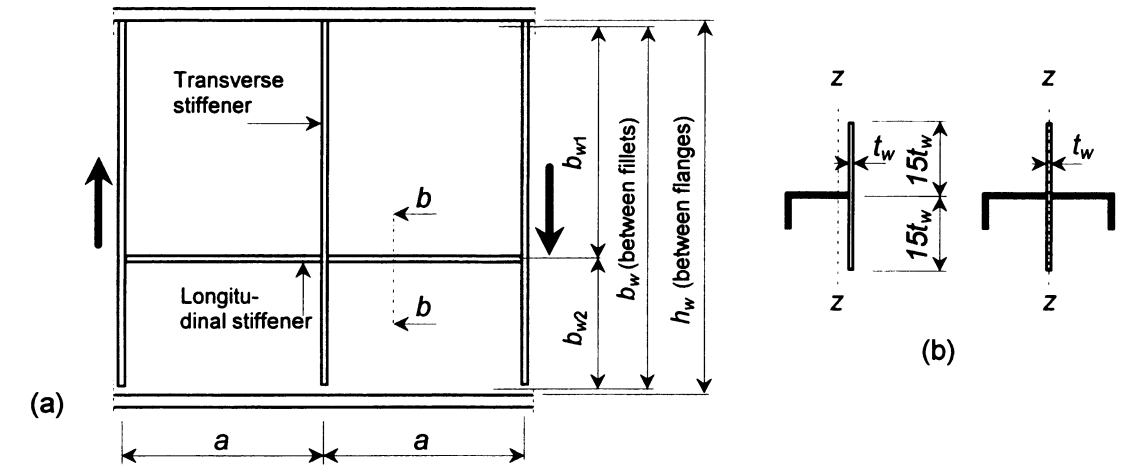


Figure 5.21: Web with transverse and longitudinal stiffeners

(8) When an axial force N_{Sd} is also applied, the value of $M_{f,Rd}$ should be reduced by a factor

$$\left(1 - \frac{N_{Sd}}{(A_{f1} + A_{f2}) f_{of} / \gamma_{M1}} \right) \qquad (5.102)$$

f_{of} is the characteristic strength for overall yielding of the flange material and A_{f1} and A_{f2} are the areas of the flanges.

(9) When $M_{Sd} \geq M_{f,Rd}$ then $V_{f,Rd} = 0$

5.12.6 Web stiffeners

5.12.6.1 Rigid end post

(1) The rigid end post should act as a bearing stiffener resisting the reaction at the girder support, and as a short beam resisting the longitudinal membrane stresses in the plane of the web.

(2) A rigid end post may comprise of two double-sided transverse stiffeners that form the flanges of a short beam of length h_f, see figure 5.19(a). The strip of web plate between the stiffeners forms the web of the short beam. Alternatively, an end post may be in the form of an inserted section, connected to the end of the web plate.

(3) Each stiffener should have a cross sectional area of at least $4\, h_f t_w^2 / e$ where e is the distance between the stiffeners, $e > 0,1\, h_f$, see figure 5.19(a).

(4) If an end post is the only means of providing resistance against twist at the end of a girder, the second moment of area of the end-post section about the center-line of the web (I_{ep}) should satisfy:

$$I_{ep} \geq b_w^3 t_f R_{Ed} / 250 W_{Ed} \qquad (5.103)$$

where:

t_f is the maximum value of flange thickness along the girder
R_{Ed} is the reaction at the end of the girder under design loading
W_{Ed} is the total design loading on the adjacent span.

5.12.6.2 Non-rigid end post

(1) A non-rigid end post may be a single stiffener as shown in figure 5.19(b). It may be assumed to act as a bearing stiffener resisting the reaction at the girder support.

5.12.6.3 Intermediate transverse stiffeners

(1) Intermediate stiffeners acting as rigid supports of interior panels of the web should be checked for resistance and stiffness.

(2) Other intermediate transverse stiffeners may be considered flexible, their stiffness being considered in the calculation of k_τ in 5.12.5(4).

(3) Intermediate transverse stiffeners acting as rigid supports for the web panel should have a second moment of area fulfilling the following:

$$I_{st} \geq 1{,}5\, h_w^3 t_w^3 / a^2 \qquad \text{if} \qquad a/h_w < \sqrt{2}\ : \tag{5.104}$$

$$I_{st} \geq 0{,}75\, h_w t_w^3 \qquad \text{if} \qquad a/h_w \geq \sqrt{2}\ : \tag{5.105}$$

The strength of intermediate rigid stiffeners should be checked for an axial force equal to V_{Ed} minus $\rho_v b_w t_w f_o$ of the web with the considered stiffener removed.

5.12.6.4 Longitudinal stiffeners

(1) Longitudinal stiffeners may be either rigid or flexible. In both cases their stiffness should be taken into account when determining the slenderness λ_w in 5.12.5.

(2) If the value of λ_w is governed by the sub-panel then the stiffener may be considered as rigid.

(3) The strength should be checked for direct stresses if the stiffeners are taken into account for resisting direct stress.

5.12.7 Interaction between shear force, bending moment and axial force

(1) Provided that the flanges can resist the whole of the design value of the bending moment and axial force in the member, the design shear resistance of the web need not be reduced to allow for the moment and axial force in the member, except as given in 5.12.5(8).

(2) When $M_{Ed} > M_{f,Rd}$ the following two expressions should be satisfied:

$$\frac{\dfrac{M_{Ed}}{M_{pl,Rd}} + 2\dfrac{V_{Ed}}{V_{w,Rd}}\left(1 - \dfrac{M_{f,Rd}}{M_{pl,Rd}}\right)}{2 - \dfrac{M_{f,Rd}}{M_{pl,Rd}}} \leq 1{,}00 \tag{5.106}$$

$$M_{Ed} \leq M_{ef,Rd}$$

where:

$M_{ef,Rd}$ is the design bending moment resistance according to 5.12.3.
$M_{f,Rd}$ is the design bending moment resistance of the flanges only, see 5.12.5(7). Effective thickness is used for class 4 compression flange. Effective thickness is used for class 4 compression flange.

(3) If an axial force N_{Ed} is also applied, then $M_{pl,Rd}$ should be replaced by the reduced plastic moment resistance $M_{N,Rd}$ given by

$$M_{N,Rd} = M_{pl,Rd}\left(1 - \left(\frac{N_{Ed}}{(A_{f1} + A_{f2})f_o / \gamma_{M1}}\right)^2\right) \tag{5.107}$$

where

A_{f1}, A_{f2} are the area of the flanges.

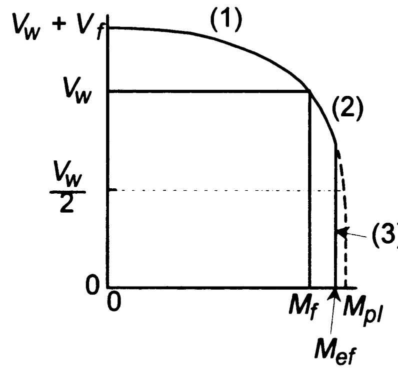


Figure 5.22: Interaction of shear resistance and bending moment resistance

5.12.8 Resistance of webs to transverse forces

(1) The resistance of an unstiffened web to transverse forces applied through a flange, is governed by one of the following modes of failure

- crushing of the web close to the flange, accompanied by plastic deformation of the flange

- crippling of the web in form of localized buckling and crushing of the web close to the flange, accompanied by plastic deformation of the flange

- buckling of the web over most of the depth of the member

- overall buckling of the web over a large part of the length of the member. This failure mode is most likely to take place when there are a number of transverse forces or a distributed load along the member.

(2) A distinction is made between three types of load application, as follows:

a) Forces applied through one flange and resisted by shear forces in the web. See figure 5.23(a).

b) Forces applied to one flange and transferred through the web directly to the other flange, see figure 5.23(b).

c) Forces applied through one flange close to an unstiffened end, see figure 5.23(c).

(3) For box girders with inclined webs the resistance should be checked for web and flange. The load effects are the components of the external load in the plane of the web and flange respectively.

(4) In addition the effect of the transverse force on the moment resistance of the member should be considered.

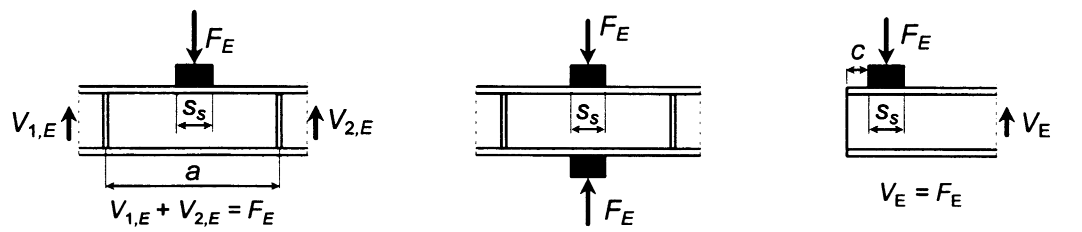
(5) The resistance of a longitudinally stiffened web is increased due to the presence of the stiffeners but no rules about this are given here.

(6) The design resistance F_{Rd} for a transverse force (figure 5.23(a), (b) and (c)) is obtained from

$$F_{Rd} = 0{,}57\, t_w^2 \sqrt{\frac{k_F\, l_y f_{ow} E}{b_w}}\ \frac{1}{\gamma_{M1}} \quad \text{but not more than} \quad t_w l_y \frac{f_{ow}}{\gamma_{M1}} \tag{5.108}$$

where f_{ow} is the characteristic strength of the web material and
where k_F is given in figure 5.23.

The effective loading length l_y is depending on length of stiff bearing s_s and cross section dimensions.



application a) application b) application c)

$$k_F = 6 + 2\left(\frac{b_w}{a}\right)^2 \qquad k_F = 3.5 + 2\left(\frac{b_w}{a}\right)^2 \qquad k_F = 2 + 6\,\frac{s_s + c}{b_w} \le 6$$

Figure 5.23: Load applications including buckling coefficients

(7) The length of stiff bearing, s_s, on the flange is the distance over which the applied force is effectively distributed and it may be determined by dispersion of load through solid material at a slope of 1:1, see figure 5.24. s_s should not be taken as more than b_w.

(8) If several concentrated loads are closely spaced, the resistance should be checked for each individual load as well as for the total load. In the latter case s_s, should be taken as the center distance between the outer loads.

(9) The effective loaded length l_y, is calculated using the dimensionless parameters

$$m_1 = \frac{f_{of} b_f}{f_{ow} t_w} \tag{5.109}$$

$$m_2 = 0{,}02\left(\frac{b_w}{t_f}\right)^2 \quad \text{if} \quad \frac{(s_s + 4t_f) b_w f_{ow}}{k_F E t_w^2} > 0{,}2 \quad \text{else} \quad m_2 = 0 \tag{5.110}$$

For box girders, b_f in expression (5.109) is limited to $25t_f$ on each side of the web.

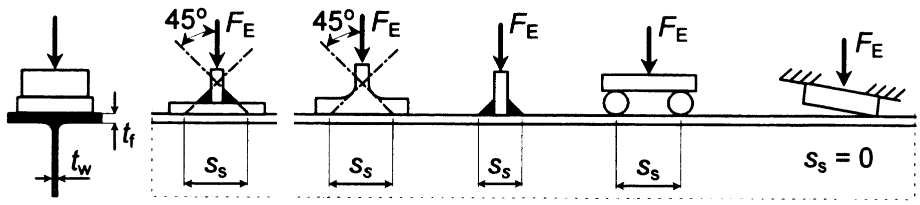


Figure 5.24: Length of stiff bearing

(10) For load application a) and b) in figure 5.23 l_y is given by

$$l_y = s_s + 2t_f\left(1 + \sqrt{m_1 + m_2}\right) \tag{5.111}$$

(11) For load application c) in figure 5.23, l_y is given by the smaller of expressions (5.111), (5.113) and (5.114). Note that $s_s = 0$ if the loading device does not follow the change in slope of the girder end.

$$l_{ef} = \frac{k_F E t_w^2}{2 f_{ow} b_w} \leq s_s + c \tag{5.112}$$

$$l_y = l_{ef} + t_f \sqrt{\frac{m_1}{2} + \left(\frac{l_{ef}}{t_f}\right)^2 + m_2} \tag{5.113}$$

$$l_y = l_{ef} + t_f \sqrt{m_1 + m_2} \tag{5.114}$$

5.12.9 Flange induced buckling

(1) To prevent the possibility of the compression flange buckling in the plane of the web, the ratio b_w/t_w of the web shall satisfy the following expression

$$\frac{b_w}{t_w} \leq \frac{kE}{f_{of}} \sqrt{\frac{A_w}{A_{fc}}} \tag{5.115}$$

where:

A_w is the area of the web
A_{fc} is the area of the compression flange

The value of the factor k should be taken as follows:

Plastic rotation utilized	0,3
Plastic moment resistance utilized	0,4
Elastic moment resistance utilized	0,55

(2) When the girder is curved in elevation, with the compression flange on the concave face, the following expression should be checked in addition to (5.115)

$$\frac{b_w}{t_w} \leq \frac{kE}{f_{of}} \sqrt{\frac{A_w}{A_{fc}}} \frac{1}{\sqrt{1 + \dfrac{b_w E}{3 r f_{of}}}} \tag{5.116}$$

where

r is the radius of curvature of the compression flange.

When the girder has transverse web stiffeners, the limiting value of b_w/t_w may be increased by the factor $1 + (b_w/a)^2$.

5.12.10 Corrugated or closely stiffened webs

(1) In plate girders with transverse reinforcement in the form of corrugations or closely spaced transverse stiffeners ($a/b_w < 0{,}3$) the flat parts between stiffeners can buckle locally, see figure 5.16(k), and the transverse reinforcement may deform with the web in an overall buckling mode, see figure 5.16(l).

a) If the web is a flat plate with multi stiffeners, the moment resistance and shear force resistance should be found according to section 5.11.

b) If the web is a corrugated plate it can be assumed that the moment resistance of the girder is provided solely by the flanges, and the web contribution is zero.

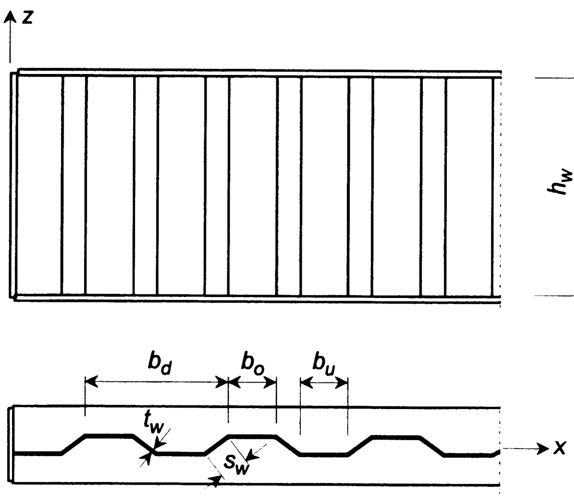


Figure 5.25: Corrugated web

(2) The shear force resistance with respect to local shear buckling of flat parts is

$$V_{w,Rd} = 0{,}7\, \rho_v t_w h_w f_{ow} / \gamma_{M1} \tag{5.117}$$

where:

ρ_v is found in table 5.12 for $\lambda_w = 0{,}35 \dfrac{b_m}{t_w} \sqrt{\dfrac{f_o}{E}}$

b_m is equal to the greatest width of the corrugated web plate panels, b_o, b_u or s_w.

(3) The shear force resistance with respect to global shear buckling is determined according to

$$V_{o,Rd} = \chi_o h_w t_w \frac{f_o}{\gamma_{M1}} \tag{5.118}$$

where:

$$\chi_o = \frac{0{,}60}{0{,}8 + \lambda_{ow}^2} \quad \text{but not more than } 1{,}0 \tag{5.119}$$

$$\lambda_{ow} = \sqrt{\frac{h_w t_w f_o}{V_{o,cr}}} \tag{5.120}$$

$$V_{o,cr} = \frac{60E}{h_w} \sqrt[4]{I_z \cdot I_x^3} \tag{5.121}$$

$$I_z = \frac{b_d}{b_u + b_o + 2s_w} \frac{t_w^3}{10{,}9} \tag{5.122}$$

I_x is second moment of area of corrugated web per unit width, see figure 5.25.

5.12.11 Tongue plates

(1) A tongue plate comprises material extending inwards from a flange to form a thickened outer section of the girder web. To be effective its cross sectional dimensions should be Class 1 or Class 2 when the tongue is considered as a plain outstand in axial compression (see section 5.4.4).

(2) When a tongue is of two or three ply construction, comprising the web plate connected to elements integral with the flange, the thickness t required for checking its classification may be taken as the total thickness. However, in riveted or bolted construction it is necessary to check that any outstand beyond the last line of fasteners is Class 1 or Class 2.

6 Connections subject to static loading

6.1 Basis for bolted, riveted and welded connections

6.1.1 Introduction

(1)P All connections shall have a design resistance such that the structure remains effective and is capable of satisfying all the basic design requirements given in section 2.

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

- resistance of bolted connections: $\gamma_{Mb} = \boxed{1{,}25}$
- resistance of riveted connections: $\gamma_{Mr} = \boxed{1{,}25}$
- resistance of pin connections: $\gamma_{Mp} = \boxed{1{,}25}$
- resistance of welded connections: $\gamma_{Mw} = \boxed{1{,}25}$
- slip resistance connections: γ_{Ms} see 6.5.9.3
- adhesive bonded connections: $\gamma_{Ma} \geq \boxed{3{,}0}$
- resistance of members and cross-sections: γ_{M1} and γ_{M2} see 5.1.1

(3)P Connections subject to fatigue shall also satisfy the requirements given in ENV 1999-2.

6.1.2 Applied forces and moments

(1)P The forces and moments applied to connections at the ultimate limit state shall be determined by global analysis conforming with 5.

(2)P These applied forces and moments shall include:

- second order effects;
- the effects of imperfections (see Annex C.4);
- the effects of connection flexibility (see 6.4).

6.1.3 Resistance of connections

(1)P The resistance of a connection shall be determined on the basis of the resistances of the individual fasteners or welds.

(2)P Linear-elastic analysis shall generally be used in the design of the connection. Alternatively non-linear analysis of the connection may be employed provided that it takes account of the load deformation characteristics of all the components of the connection.

(3)P If the design model is based on yield lines such as block shear e.g., the adequacy of the model shall be demonstrated on the basis of physical tests.

6.1.4 Design assumptions

(1) Connections may be designed by distributing the internal forces and moments in whatever rational way is best, provided that:

(a) the assumed internal forces and moments are in equilibrium with the applied forces and moments;

(b) each element in the connection is capable of resisting the forces or stresses assumed in the analysis;

(c) the deformations implied by this distribution are within the deformation capacity of the fasteners or welds and of the connected parts, and

(d) the deformations assumed in any design model based on yield lines are based on rigid body rotations (and in-plane deformations) which are physically possible.

(2)P In addition, the assumed distribution of internal forces shall be realistic with regard to relative stiffnesses within the joint. The internal forces will seek to follow the path with the greatest rigidity. This path shall be clearly identified and consistently followed throughout the design of the connection.

(3) Residual stresses and stresses due to tightening of fasteners and due to ordinary accuracy of fit-up need not normally be allowed for.

6.1.5 Fabrication and execution

(1)P Ease of fabrication and execution shall be considered in the design of all joints and splices.

(2) Attention should be paid to:

- the clearances necessary for safe execution;
- the clearances needed for tightening fasteners;
- the need for access for welding;
- the requirements of welding procedures, and
- the effects of angular and length tolerances on fit-up.

(3) Attention should also be paid to the requirements for:

- subsequent inspection;
- surface treatment, and
- maintenance.

For detailed rules on fabrication and execution see section 7.

6.2 Intersections for bolted, riveted and welded connections

(1)P Members meeting at a joint shall normally be arranged with their centroidal axes intersecting at a point.

(2)P Any kind of eccentricity in the nodes shall be taken into account, except in the case of particular types of structures where it has been demonstrated that it is not necessary.

6.3 Joints loaded in shear subject to vibration and/or load reversal

(1)P Where a joint loaded in shear is subject to impact or significant vibration, either welding or else bolts with locking devices, preloaded bolts, injection bolts or other types of bolts which effectively prevent movement shall be used.

(2)P Where slipping is not acceptable in a joint because it is subject to reversal of shear load (or for any other reason), either preloaded bolts in a slip-resistant connection (category B or C as appropriate, see 6.5.3), fitted bolts or welding shall be used.

(3) For wind and/or stability bracings, bolts in bearing type connections (category A in 6.5.3) may normally be used.

6.4 Classification of connections

6.4.1 General

(1) Connection is defined as the system which mechanically fastens a given member to the remaining part of the structure. It should be distinguished from the term "joint", which usually means the system composed by the connection itself plus the corresponding interaction zone between the connected members (see figure 6.1).

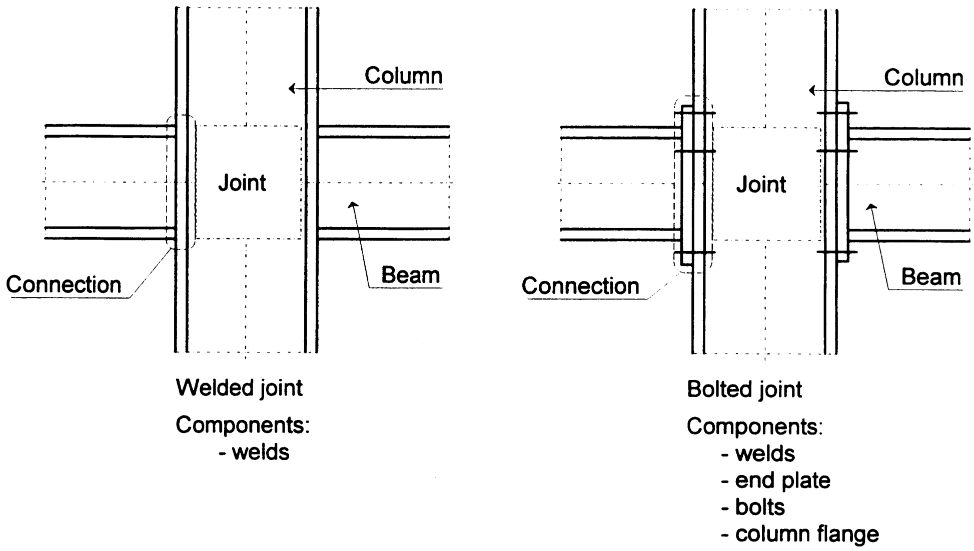


Figure 6.1: Definition of "connection" and "joint"

(2)P The structural properties of all connections shall be such as to achieve the assumptions made in the analysis of the structure and in the design of the members.

(3) In the following the symbols "F" and "D" refer to a generalized force (axial load, shear load or bending moment) and to the corresponding generalized deformation (elongation, distorsion or rotation), respectively. The subscripts "e" and "u" refer to the elastic and ultimate limit state, respectively.

(4) Connections may be classified according to their capability to restore the behavioural properties (rigidity, strength and ductility) of the connected member. With respect to the global behaviour of the connected member, two main classes are defined (figure. 6.2):

- Fully restoring connections;
- Partially restoring connection;

(5) With respect to the single behavioural property of the connected member, connections may be classified according to (figures 6.2 b)-d)):
- Rigidity;
- Strength;
- Ductility.

(6) The types of connection should conform with the member design assumptions and the method of global analysis.

6.4.2 Fully restoring connections

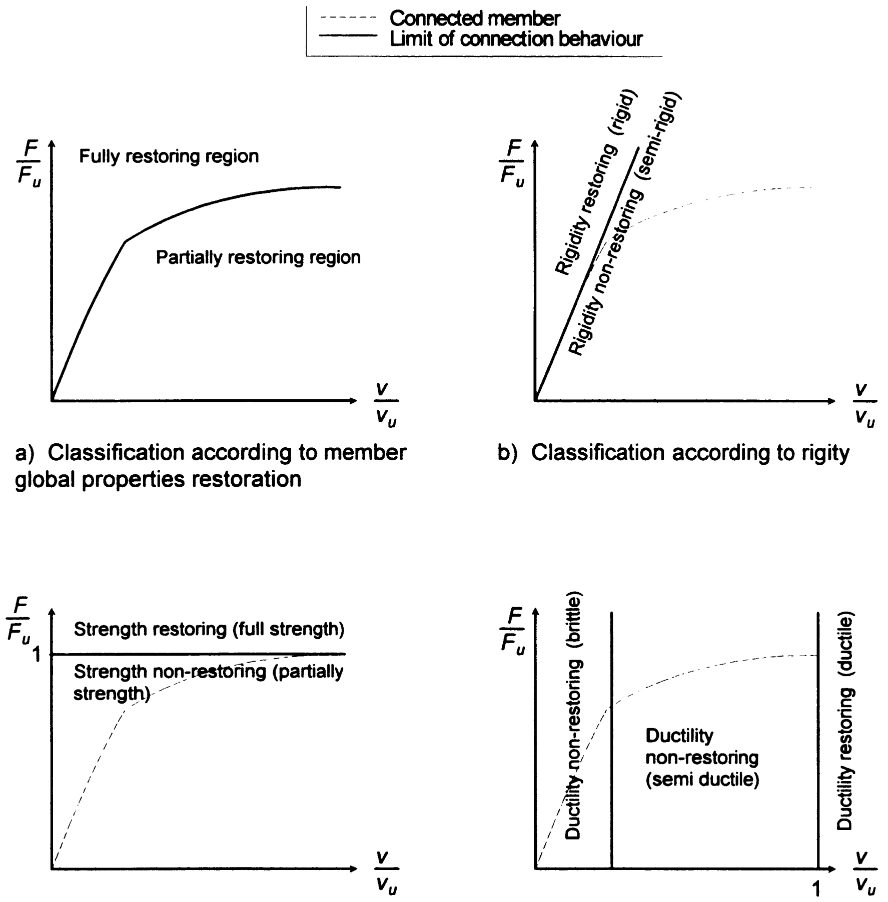
(1) They are designed in such a way to have behavioural properties always equal to or higher than those of the connected member, in terms of elastic rigidity, ultimate strength and ductility. The generalized force-displacement curve of the connection always lies above the one of the connected member.

(2) The existence of the connection may be ignored in the structural analysis.

6.4.3 Partially restoring connections

(1) The behavioural properties of the connection do not reach those of the connected member, due to its lack of capability to restore either elastic rigidity, ultimate strength or ductility of the connected member. The generalized force-displacement curve may in some part fall below the one of the connected member.

(2)P The existence of such connections must be considered in the structural analysis.



a) Classification according to member global properties restoration

b) Classification according to rigity

c) Classification according to strength

d) Classification according to ductility

Figure 6.2 a) - d): Classification of connections

6.4.4 Classification according to rigidity

(1) With respect to rigidity, connections can be classified as (figure 6.2b):

- Rigidity restoring (rigid) connections (R1);
- Rigidity non-restoring connections (semi-rigid) connections (R2),

depending on whether the initial stiffness of the connected member is restored or not, regardless of strength and ductility.

6.4.5 Classification according to strength

(1) With respect to strength, connections can be classified as (figure 6.2c):

- Strength restoring (full strength) connections;
- Strength non-restoring connections (partial strength) connections,

depending on whether the ultimate strength of the connected member is restored or not, regardless of rigidity and ductility.

6.4.6 Classification according to ductility

(1) With respect to ductility, connections can be classified as (fig. 6.2d):

- Ductility restoring (ductile) connections;
- Ductility non-restoring (semi-ductile or brittle) connections,

depending on whether the ductility of the connection is higher or lower than that of the connected member, regardless of strength and rigidity.

(2) Ductile connections have a ductility equal or higher than that of the connected member; elongation or rotation limitations may be ignored in structural analysis.

(3)P Semi-ductile connections have a ductility less than the one of the connected member, but higher than its elastic limit deformation; elongation or rotation limitations must be considered in inelastic analysis.

(4)P Brittle connections have a ductility less than the elastic limit deformation of the connected member; elongation or rotation limitations must be considered in both elastic and inelastic analysis.

6.4.7 General design requirements for connections

(1) The relevant combinations of the main behavioural properties (rigidity, strength and ductility) of connections give rise to several cases (figure 6.3).

In table 6.1 they are shown with reference to the corresponding requirements for methods of global analysis (see 5.2.1).

6.4.8 Requirements for framing connections

6.4.8.1 General

(1) With respect to the moment-curvature relationship, the connection types adopted in frame structures can be divided into:

- Nominally pinned connections;
- Built-in connections.

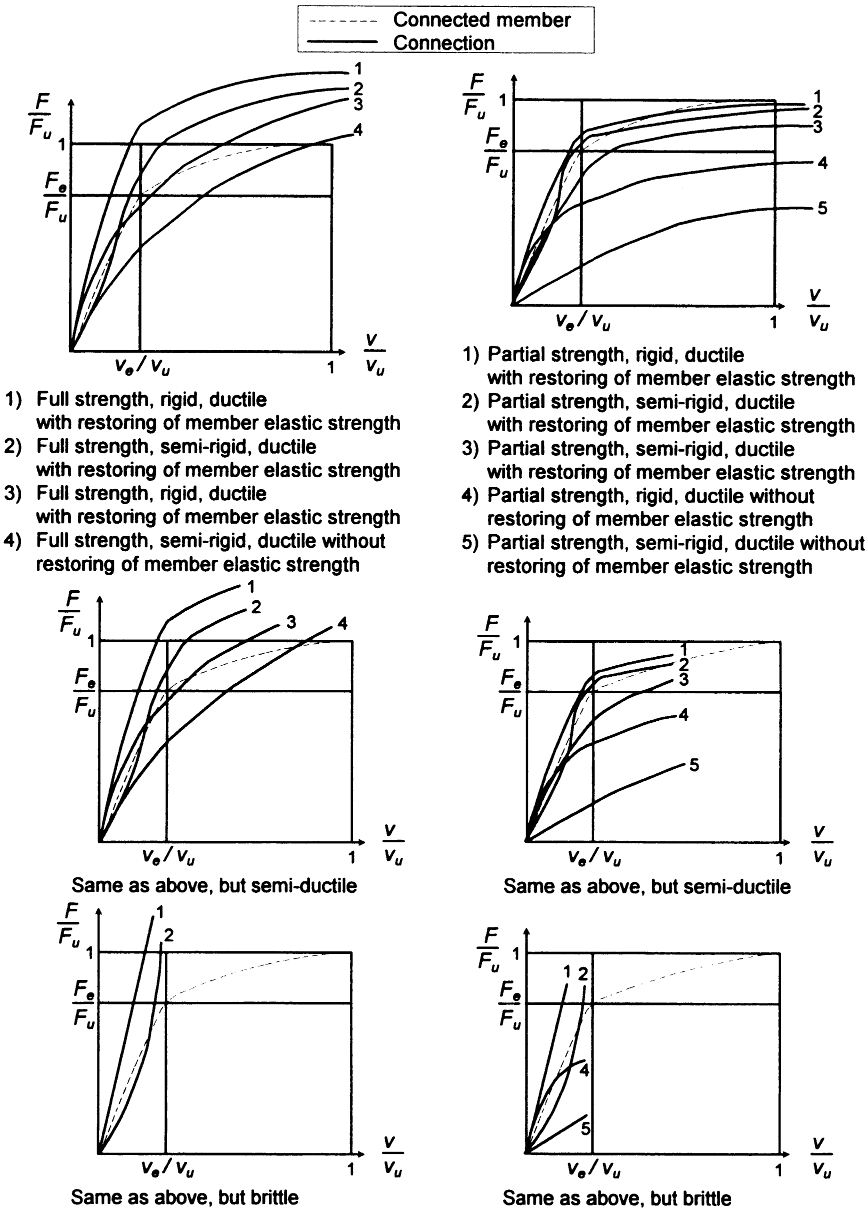


Figure 6.3: Main connection types

(2) The types of connections should conform with table 6.1 in accordance with the method of global analysis (see 5.2.1) and the member design assumptions.(Annex C).

6.4.8.2 Nominally pinned connections

(1)P A nominally pinned connection shall be designed in such a way to transmit the design axial and shear forces without developing significant moments which might adversely affect members of the structure.

(2) Nominally pinned connections should be capable of transmitting the forces calculated in design and should be capable of accepting the resulting rotations.

(3) The rotation capacity of a nominally pinned connection should be sufficient to enable all the necessary plastic hinges to develop under the design loads.

Table 6.1: General design requirements

Method of global analysis (see 5.2.1)	Type of connection which must be accounted for	Type of connection which may be ignored
ELASTIC	Semi-rigid connections (full or partial strength, ductile or non-ductile with or without restoring of member elastic strength) Partial strength connections (rigid or semi-rigid, ductile or non-ductile) without restoring of member elastic strength	Fully restoring connections Rigid connections (full or partial strength, ductile or non-ductile) with restoring of member elastic strength Partial strength connections (rigid, ductile or non-ductile) with restoring of member elastic strength
PLASTIC (rigid-plastic elastic-plastic ineline-plastic)	Partial strength connections (rigid or semi-rigid ductile or non-ductile) without restoring of member elastic strength	Fully restoring connections Partial strength, ductile connections (rigid or semi-rigid) with restoring of member elastic strength Full strength connections
HARDENING (rigid-hardening elastic-hardening generically inelastic)	Partially restoring connections	Fully restoring connections

6.4.8.3 Built-in connections

(1) Built-in connections allow for the transmission of bending moment between connected members, together with axial and shear forces. They can be classified according to rigidity and strength as follows (see 6.4.4 and 6.4.5):

- Rigid connections;
- Semi-rigid connections;
- Full strength connections;
- Partial strength connections.

(2)P A rigid connection shall be designed in such a way that its deformation has a negligible influence on the distribution of internal forces and moments in the structure, nor on its overall deformation.

(3) The deformations of rigid connections should be such that they do not reduce the resistance of the structure by more than 5%.

(4) Semi-rigid connections should provide a predictable degree of interaction between members, based on the design moment-rotation characteristics of the joints.

(5) Rigid and semi-rigid connections should be capable of transmitting the forces and moments calculated in design.

(6) The rigidity of full-strength and partial-strength connections should be such that, under the design loads, the rotations at the necessary plastic hinges do not exceed their rotation capacities.

(7)P The rotation capacity of a partial-strength connection which occurs at a plastic hinge location shall be not less than that needed to enable all the necessary plastic hinges to develop under the design loads.

(8) The rotation capacity of a connection may be demonstrated by experimental evidence. Experimental demonstration is not required when using details which experience has proved have adequate properties in relation with the structural scheme.

6.5 Connections made with bolts, rivets or pins

6.5.1 Positioning of holes for bolts and rivets

6.5.1.1 Basis

(1)P The positioning of holes for bolts and rivets shall be such as to prevent corrosion and local buckling and to facilitate the installation of the bolts or rivets.

(2)P The positioning of the holes shall also be in conformity with the limits of validity of the rules used to determine the design resistances of the bolts and rivets.

6.5.1.2 End distance

(1) The end distance e_1 from the centre of a fastener hole to the adjacent end of any part, measured in the direction of load transfer (see figure 6.4), should normally be $2{,}0d_0$. In extreme situations it should be not less than $1{,}2d_0$, provided that the bearing stress is reduced accordingly, see 6.5.5 and 6.5.6. d_0 is the hole diameter, see 7.3.6.

(2) The end distance should be increased if necessary to provide adequate bearing resistance, see 6.5.5 and 6.5.6, a larger end distance of $3{,}0d_0$ has no effect for the bearing capacity.

6.5.1.3 Minimum edge distance

(1) The edge distance e_2 from the centre of a fastener hole to the adjacent edge of any part, measured at right angles to the direction of load transfer (see figure 6.4), should normally be not less than $1{,}5d_0$.

(2) In extreme situations the edge distance may be reduced to not less than $1{,}2d_0$ provided that the design bearing resistance is reduced accordingly, see 6.5.5 and 6.5.6.

6.5.1.4 Maximum end and edge distances

(1) Where the members are exposed to the weather or other corrosive influences, the maximum end or edge distance should not exceed 40 mm + $4t$, where t is the thickness of the thinner outer connected part.

(2) In other cases the end or edge distance should not exceed $12t$ or 150 mm, whichever is the larger.

(3) The edge distance should also not exceed the maximum to satisfy local buckling requirements for an outstand element. This requirement does not apply to fasteners interconnecting the components of tension members. The end distance is not affected by this requirement.

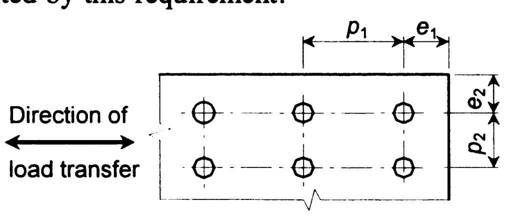


Figure 6.4 Symbols for spacing of fasteners

6.5.1.5 Minimum spacing

(1) The spacing p_1 between centres of fasteners in the direction of load transfer (see figure 6.4), should normally be $2{,}5d_0$, in extreme situations not less than $2{,}2d_0$, provided that the bearing stress is reduced accordingly, see 6.5.5 and 6.5.6. The spacing should be increased if necessary to provide adequate bearing resistance, see 6.5.5 and 6.5.6.

(2) The spacing p_2 between rows of fasteners, measured perpendicular to the direction of load transfer (see figure 6.4), should normally be $3{,}0d_0$. This spacing may be reduced to $2{,}4d_0$ provided that the design bearing resistance is reduced accordingly, see 6.5.5 and 6.5.6.

6.5.1.6 Maximum spacing in compression members

(1) The spacing p_1 of the fasteners in each row and the spacing p_2 between rows of fasteners, should not exceed the lesser of $14t$ or 200 mm. Adjacent rows of fasteners may be symmetrically staggered, see figure 6.5.

(2) The centre-to-centre spacing of fasteners should also not exceed the maximum width which satisfies local buckling requirements for an internal element, see 5.4.5.

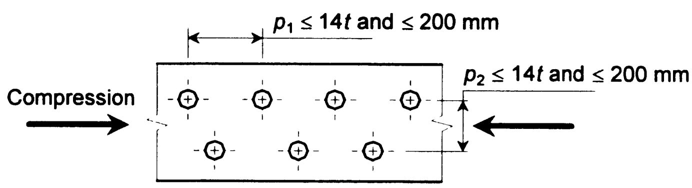


Figure 6.5: Staggered spacing - compression

6.5.1.7 Maximum spacing in tension members

(1) In tension members the centre-to-centre spacing $p_{1,i}$ of fasteners in inner rows may be twice that given in 6.5.1.6 (1) for compression members, provided that the spacing $p_{1,o}$ in the outer row along each edge does not exceed that given in 6.5.1.6 (1), see figure 6.6.

(2) Both of these values may be multiplied by 1,5 in members not exposed to corrosive influences.

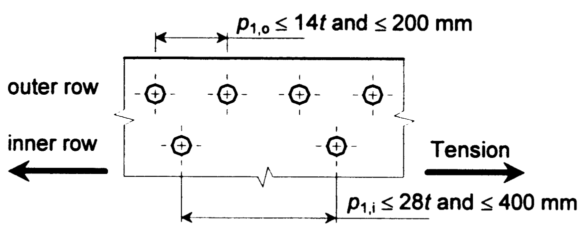


Figure 6.6: Spacing in tension member

6.5.1.8 Slotted Holes

(1) Slotted holes are not recommended.

6.5.2 Deductions for fastener holes

6.5.2.1 General

For detailed rules for the design of members with holes see 5.6.2.2

6.5.2.2 Design shear rupture resistance

(1)P "Block shear" failure at a group of fastener holes near the end of a beam web or a bracket or connections with similar behaviour, see figure 6.7, shall be prevented by using appropriate hole spacing. This mode of failure generally consists of tensile rupture along the horizontal line of fastener holes on the tension face of the hole group, accompanied by gross section yielding in shear at the vertical row of fastener holes along the shear face of the hole group, see figure 6.7.

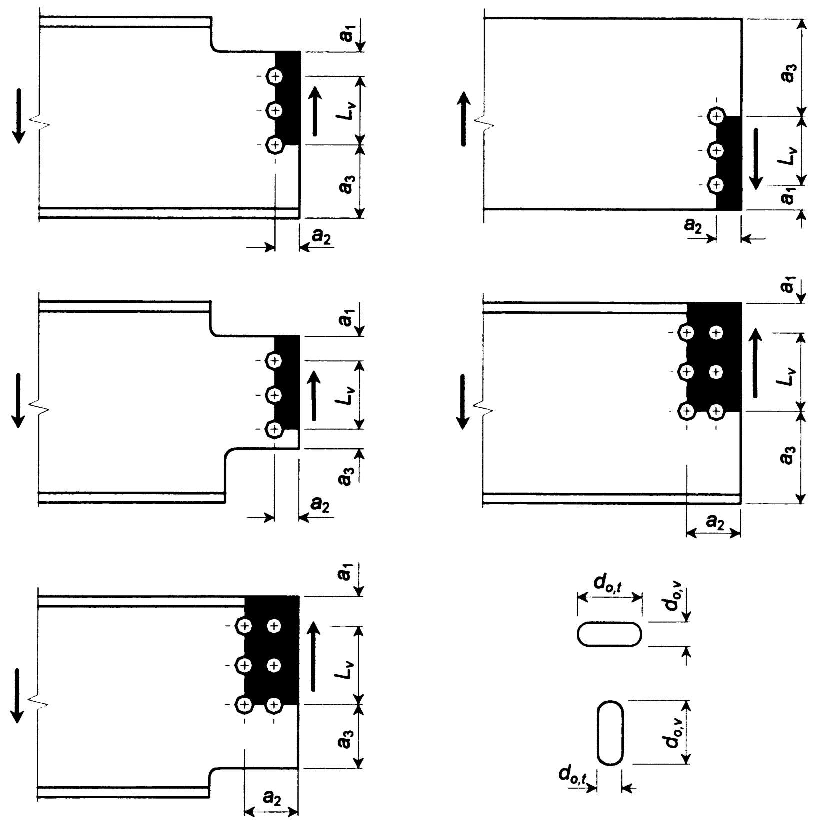


Figure 6.7: Block shear - effective shear area

(2) The design value of the effective resistance to block shear $V_{eff,Rd}$ or $N_{eff,Rd}$ should be determined from:

$$V_{eff,Rd} = \left(f_o/\sqrt{3}\right)A_{v,eff}/\gamma_{M1} \qquad (6.1)$$

where $A_{v,eff}$ is the effective shear area.

(3) The effective shear area $A_{v,eff}$ should be determined as follows:

$$A_{v,eff} = t\,L_{v,eff} \qquad (6.2)$$

where

$$L_{v,eff} = L_v + L_1 + L_2 \text{ but } L_{v,eff} \leq L_3 \qquad (6.3)$$

in which

$$L_1 = a_1 \text{ but } L_1 \leq 5\,d \qquad (6.4)$$

$$L_2 = (a_2 - k\,d_{0,t})\,(f_u/f_o) \qquad (6.5)$$

and

$$L_3 = L_v + a_1 + a_3 \text{ but } L_3 \leq (L_v + a_1 + a_3 - nd_{0,v})\,(f_u/f_0) \tag{6.6}$$

where

a_1, a_2, a_3 and L_v are as indicated in figure 6.7

d is the nominal diameter of the fasteners,
$d_{0,t}$ is the hole size for the tension face, generally the hole diameter,
$d_{0,v}$ is the hole size for the shear face, generally the hole diameter,
n is the number of fastener holes in the shear face,
t is the thickness of the web or bracket, and
k is a coefficient with values as follows:
- for a single row of bolts: $k = 0,5$,
- for two rows of bolts: $k = 2,5$.

6.5.2.3 Angles and angles with bulbs

(1)P In the case of unsymmetrical or unsymmetrically connected members such as e.g. angles or angles with bulbs, the eccentricity of fasteners in end connections and the effects of the spacing and edge distances of the bolts shall be taken into account when determining the design resistances.

(2) Angles and angles with bulbs connected by a single row of bolts, see figure 6.8, may be treated as concentrically loaded and the design ultimate resistance of the net section determined as follows:

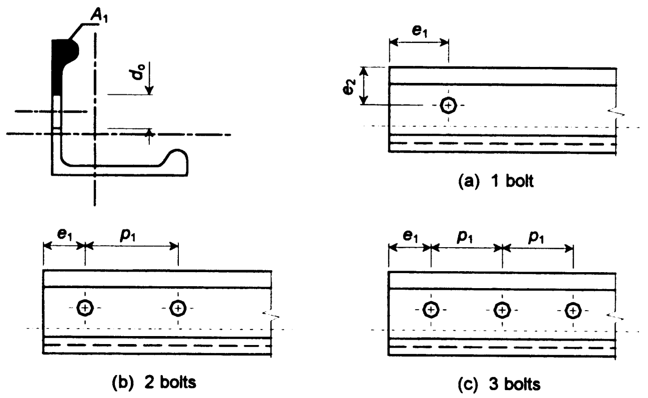


Figure 6.8: Connections of angles with bulbs (covers also angles without bulbs)

$$\text{with 1 bolt: } N_{u,Rd} = \frac{2 A_1 f_u}{\gamma_{M2}} \tag{6.7}$$

$$\text{with 2 bolts: } N_{u,Rd} = \frac{\beta_2 A_{Net} f_u}{\gamma_{M2}} \tag{6.8}$$

$$\text{with 3 bolts: } N_{u,Rd} = \frac{\beta_3 A_{Net} f_u}{\gamma_{M2}} \tag{6.9}$$

where

β_2 and β_3 are reduction factors dependent on the pitch p_1 as given in table 6.2. For intermediate values of p_1 the values of β may be determined by linear interpolation.

A_{net} is the net area of the angle. For an unequal-leg angle connected by its smaller leg, A_{net} should be taken as equal to the net section area of an equivalent equal-leg angle of leg size equal to that of the smaller leg.

(3) The design buckling resistance of a compression member, see 5.8, should be based on the gross cross sectional area, but should not be taken as more than the design resistance of the cross section given in (2).

Table 6.2: Reduction factors β_2 and β_3

Pitch p_1	$\leq 2{,}5\ d_0$	$\geq 5{,}0\ d_0$
β_2 for 2 bolts	0,4	0,7
β_3 for 3 bolts or more	0,5	0,7

6.5.3 Categories of bolted connections

6.5.3.1 Shear connection

(1)P The design of a bolted connection loaded in shear shall conform with one of the following categories, see table 6.3.

Table 6.3: Categories of bolted connection

Shear connections		
Category	Criteria	Remarks
A bearing type	$F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	No preloading required. All grades from 4.6 to 10.9.
B slip resistant at serviceability	$F_{v,Ed,ser} \leq F_{s,Rd,ser}$ $F_{v,Ed} \leq F_{s,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Preloaded high strength bolts. No slip at the serviceability limit state.
C slip resistant at ultimate	$F_{v,Ed} \leq F_{s,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Preloaded high strength bolts. No slip at the ultimate limit state.
Tension connections		
Category	Criterion	Remarks
D non-preloaded	$F_{t,Ed} \leq F_{t,Rd}$	No preloading required. All grades from 4.6 to 10.9.
E preloaded	$F_{t,Ed} \leq F_{t,Rd}$	Preloaded high strength bolts.

Key: $F_{v,Ed}$ design shear force per bolt for the ultimate limit state
$F_{v,Rd}$ design shear resistance per bolt
$F_{s,Rd}$ design slip resistance per bolt at the ultimate limit state
$F_{v,Ed,ser}$ design shear force per bolt for the serviceability limit state
$F_{s,Rd,ser}$ design slip resistance per bolt at the serviceability limit state
$F_{b,Rd}$ design bearing resistance per bolt
$F_{t,Ed}$ design tensile force per bolt for the ultimate limit state
$F_{t,Rd}$ design tension resistance per bolt

(2)P Category A: Bearing type
In this category protected steel bolts (ordinary or high strength type) or stainless steel bolts or aluminium bolts or aluminium rivets shall be used. No preloading and special provisions for contact surfaces are required. The design ultimate shear load shall not exceed the design shear resistance nor the design bearing resistance, obtained from 6.5.5.

(3)P Category B: Slip-resistant at serviceability limit state
In this category preloaded high strength bolts with controlled tightening in conformity with Reference Standard 8 of ENV 1993-1-1:1992 shall be used. Slip shall not occur at the serviceability limit state. The combination of actions to be considered shall be selected from 2.3.4 depending on the load cases where resistance to slip is required. The design serviceability shear load should not exceed the design slip resistance, obtained from 6.5.9. The design ultimate shear load shall not exceed the design shear resistance nor the design bearing resistance, obtained from 6.5.5.

(4)P Category C: Slip resistant at ultimate limit state
In this category preloaded high strength bolts with controlled tightening in conformity with Reference Standard 8 of ENV 1993-1-1:1992 shall be used. Slip shall not occur at the ultimate limit state. The design ultimate shear load shall not exceed the design slip resistance obtained from 6.5.9 nor the design bearing resistance obtained from 6.5.5.

In addition, at the ultimate limit state the design plastic resistance of the net section at bolt holes $N_{net,Rd}$ (see 5.7.3) shall be taken as:

$$N_{net,Rd} = A_{net} f_o / \gamma_{M1} \tag{6.10}$$

6.5.3.2 Tension connections

(1)P The design of a bolted connection loaded in tension shall conform with one of the following categories, see table 6.3.

(2)P Category D: Connections with non-preloaded bolts
In this category ordinary bolts grade 4.6 and 5.6 (manufactured from low carbon steel) or high strength bolts grade 8.8 and 10.9 or aluminium bolts or stainless steel bolts shall be used. No preloading is required. This category shall not be used where the connections are frequently subjected to variations of tensile loading. However, they may be used in connections designed to resist normal wind loads.

(3)P Category E: Connections with preloaded high strength bolts
In this category preloaded high strength bolts with controlled tightening in conformity with Reference Standard 8 of ENV 1993-1-1:1992 shall be used. Such preloading improves fatigue resistance. However, the extent of the improvement depends on detailing and tolerances.

(4) For tension connections of both categories D and E no special treatment of contact surfaces is necessary, except where connections of category E are subject to both tension and shear (combination E-B or E-C).

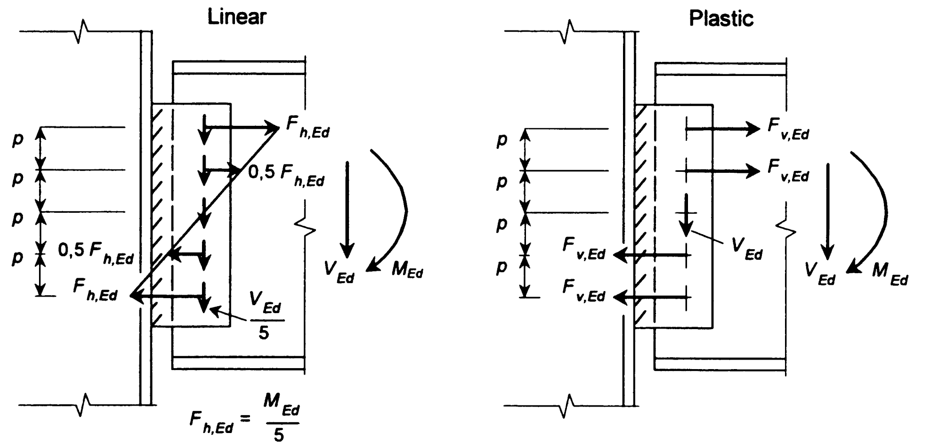
6.5.4 Distribution of forces between fasteners

(1)P The distribution of internal forces between fasteners due to the bending moment at the ultimate limit state shall be proportional to the distance from the centre of rotation and the distribution of the shear force shall be equal, see figure 6.9(a), in the following cases:

- Category C slip-resistant connections;
- Other shear connections where the design shear resistance $F_{v,Rd}$ of a fastener is less than the design bearing resistance $F_{b,Rd}$.

(2) In other cases the distribution of internal forces between fasteners due to the bending moment at the ultimate limit state may be assumed plastic and the distribution of the shear force may be assumed equal, see figure 6.9(b).

(3) In a lap joint, the same bearing resistance in any particular direction should be assumed for each fastener up to a maximium length of max $L = 15\ d$, where d is the nominal diameter of the bolt or rivet. For $L > 15\ d$ see 6.5.10.



(a) Distribution proportional to distance from centre of rotation

(b) Possible plastic distribution with one fastener resisting V_{Ed} and 4 resisting M_E

$$F_{v,Ed} = \sqrt{\left(\frac{M_{Ed}}{5p}\right) + \left(\frac{V_{Ed}}{5}\right)} \quad (6.11)$$

$$F_{v,Ed} = \frac{M_{Ed}}{6p} \quad (6.12)$$

Figure 6.9: Distribution of loads between fasteners
a) elastic load distribution
b) plastic load distribution

6.5.5 Design resistances of bolts

(1)P The design resistances given in this clause apply to standard manufactured bolts of strength grades 4.6, 5.6, 8.8 and 10.9 or aluminium bolts or stainless steel bolts, which conform with relevant prENs or ENs, see annex B of ENV 1993-1-1:1992. Nuts and washers shall also conform with relevant prENs or ENs and shall have the corresponding specified strengths.

(2)P At the ultimate limit state the design shear force $F_{v,Ed}$ on a bolt shall not exceed the lesser of:

- the design shear resistance $F_{v,Rd}$;
- the design bearing resistance $F_{b,Rd}$,

both as given in table 6.4.

(3)P At the ultimate limit state the design tensile force $F_{t,Ed}$, inclusive of any force due to prying action, shall not exceed the design tension resistance $B_{t,Rd}$ of the bolt-plate assembly.

(4)P The design tension resistance of the bolt-plate assembly $B_{t,Rd}$ shall be taken as the smaller of the design tension resistance $F_{t,Rd}$ of the bolt given in table 6.4 and the design punching shear resistance of the bolt head and the nut, $B_{p,Rd}$ obtained from:

$$B_{p,Rd} = 0{,}6\ \pi\ d_m\ t_p f_0 / \gamma_{Mb} \quad (6.19)$$

where:

p is the center to center distance between bolt holes;

t_p is the thickness of the plate under the bolt head or the nut;

d_m is the mean of the across points and across flats dimensions of the bolt head or the nut, whichever is smaller;
f_o characteristic strength of the member material.

Table 6.4: Design resistance for bolts

Shear resistance per shear plane: - for strength grades lower than 10.9 $F_{v,Rd} = \dfrac{0,6\, f_{ub}\, A}{\gamma_{Mb}}$ (6.13) - for strength grades 10.9, stainless steel bolts and aluminium bolts $F_{v,Rd} = \dfrac{0,5\, f_{ub}\, A}{\gamma_{Mb}}$ (6.14) $A = A_S$, if the shear plane passes through the threaded portion of the bolt $A = A$, if the shear plane passes through the unthreaded portion of the bolt f_{ub} = characteristic ultimate tensile strength of the bolt material
Bearing resistance: $F_{b,Rd} = \dfrac{2,5\, \alpha f_u\, d\, t}{\gamma_{Mb}}$ (6.15) where α is the smallest of: $\dfrac{e_1}{3d_0};\ \dfrac{p_1}{3d_0} - \dfrac{1}{4};\ \dfrac{f_{ub}}{f_u}$ *or* 1,0. (6.16) f_u is the characteristic ultimate strenght of the material of the connected parts
Tension resistance $F_{t,Rd} = \dfrac{0,9\, f_{ub}\, A_S}{\gamma_{Mb}}$ for steel bolts (6.17) $F_{t,Rd} = \dfrac{0,6\, f_{ub}\, A_S}{\gamma_{Mb}}$ for aluminium bolts (6.18)
A is the shank cross sectional area of bolt A_S is the stress area of bolt d is the bolt diameter d_0 is the hole diameter e_1, p_1 see figure 6.4

(5)P Bolts subject to both shear force and tensile force shall in addition satisfy the following requirement:

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4\ F_{t,Rd}} \leq 1{,}0 \qquad (6.20)$$

(6)P The design resistances for tension and for shear through the threaded portion given in table 6.4 are restricted to bolts manufactured in conformity with relevant prENs or ENs, see normative annex B of ENV 1993-1-1:1992. For other items with cut threads, such as holding-down bolts or tie rods fabricated from round steel bars where the threads are cut by the steelwork fabricator and not by a specialist bolt manufacturer, the relevant values from table 6.4 such as the stress area A_S shall be reduced by multiplying them by a factor of 0,85.

(7) The values for design shear resistance $F_{v,Rd}$ given in table 6.4 apply only where the bolts are used in holes with nominal clearances not exceeding those for standard holes as specified in 7.5.2(1).

(8) M12 and M14 bolts may also be used in 2 mm clearance holes provided that:

- for bolts of strength grade 10.9 the design shear resistance $F_{v,Rd}$ is taken as 0,85 times the value given in table 6.4.

- the design shear resistance $F_{v,Rd}$ (reduced as above if applicable) is not less than the design bearing resistance $F_{b,Rd}$.

(9)P The values for the design bearing resistance in table 6.4 shall only be applied where the edge distance e_2 is not less than 1,5 d_0 and the spacing p_2 measured transverse to the load direction is at least 3,0 d_0.

(10)P If e_2 is reduced to 1,2 d_0 and/or p_2 is reduced to 2,4 d_0, then the bearing resistance $F_{b,Rd}$ shall be reduced tc 2/3 of the value given in table 6.4. For intermediate values 1,2 $d_0 \leq e_2 < 1{,}5\ d_0$ and/or 2,4 $d_0 \leq p_2 \leq 3\ d_0$ the value of $F_{b,Rd}$ may be determined by linear interpolation.

(11) For bolts in standard clearance holes (see 7.3.6) conservative values of the design bearing resistance $F_{b,Rd}$ based on the bolt diameter d, may be obtained from table 6.4.

6.5.6 Design resistance of rivets

(1)P At the ultimate limit state the design shear force $F_{v,Ed}$ on a rivet shall not exceed the lesser of:

- the design shear resistance $F_{v,Rd}$
- the design bearing resistance $F_{b,Rd}$

both as given in table 6.5.

(2)P Riveted connections shall be designed to transfer forces in shear and bearing. Tension in aluminium rivets is not recommended.

(3)P Rivets subject to both shear and tensile forces shall in addition satisfy the following:

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4\ F_{t,Rd}} \leq 1{,}0 \qquad (6.21)$$

(4)P The values for the design bearing resistance $F_{b,Rd}$ in table 6.5 shall only be applied where the edge distance e_2 is not less than $1{,}5d_0$ and the spacing p_2 measured transverse to the load direction is at least $3{,}0d_0$.

(5)P For smaller values of e_2 and/or p_2 the same reduction of $F_{b,Rd}$ shall be applied as given in 6.5.5(10) for bolts.

(6) As a general rule, the grip length of a rivet should not exceed $4{,}5d$ for hammer riveting and $6{,}5d$ for press riveting.

Table 6.5 Design resistance for aluminium rivets

Shear resistance per shear plane: $F_{v,Rd} = \dfrac{0{,}6 f_{ur} A}{\gamma_{Mr}}$	(6.22)
Bearing resistance: $F_{b,Rd} = \dfrac{2{,}5\, \alpha f_u d_0 t}{\gamma_{Mr}}$	(6.23)
where α is the smallest of: $\dfrac{e_1}{3d_0}$; $\dfrac{p_1}{3d_0} - \dfrac{1}{4}$; $\dfrac{f_{ur}}{f_u}$ or 1,0	(6.24)
f_u is characteristic ultimate strengths of the material of the connected parts	
Tension resistance: Not recommended.	
A is the area of the rivet hole d_0 is the diameter of the rivet hole f_{ur} is the specified ultimate strength of the rivet e_1, p_1 see figure 6.4	

6.5.7 Countersunk bolts and rivets

(1)P The design tension resistance $F_{t,Rd}$ of a countersunk bolt shall be taken as 0,7 times the design tension resistance given in table 6.4 or table 6.5 respectively.

(2)P The angle and depth of countersinking shall conform with the head of the actual countersunk bolt, otherwise the tension resistance shall be adjusted accordingly.

(3)P The design bearing resistance $F_{b,Rd}$ of a countersunk bolt or rivet shall be calculated as specified in 6.5.5 or 6.5.6 respectively, with half the depth of the countersink deducted from the thickness t of the relevant part jointed.

6.5.8 Hollow rivets and rivets with mandrills

(1)P The design strength of hollow rivets and rivets with mandrills shall be determined by tests.

6.5.9 High strength bolts in slip-resistant connections

6.5.9.1 General

(1) Design may be based on calculations for joints where the proof strength of the material of the connected parts is higher than 200 N/mm². Otherwise the strength of joints using general grade high-strength bolts should be proved by testing. In aluminium structures the relaxation of bolt preload due to tension in the joined material cannot be ignored.

(2) The effect of extreme temperature changes and/or long grip lengths which may cause a reduction or increase of the friction capacity due to the differential thermal expansion between aluminium and bolt steel cannot be ignored.

6.5.9.2 Ultimate limit state

(1)P It is possible to take the slip resistance as ultimate or serviceability limit state, see 6.5.3.1, but, besides, at the ultimate limit state the design shear force, $F_{v,Ed}$ on a high strength bolt shall not exceed the lesser of:

- the design shear resistance $F_{v,Rd}$
- the design bearing resistance $F_{b,Rd}$
- the tensile or compression or bending resistance of the member in the net section and in the gross cross section.

6.5.9.3 Slip resistance/Shear resistance

(1)P The design slip resistance of a preloaded high-strength bolt shall be taken as:

$$F_{s,Rd} = \frac{n\mu}{\gamma_{Ms}} F_{p,Cd} \tag{6.25}$$

where:

$F_{p,Cd}$ is the design preloading force, given in 6.5.9.4.
μ is the slip factor, see 6.5.9.5 and
n is the number of friction interfaces.

(2)P For bolts in standard nominal clearance holes, the partial safety factor for slip resistance γ_{Ms} shall be taken as:

$\gamma_{Ms,ult} = \boxed{1,25}$ for the ultimate limit state,

$\gamma_{Ms,ser} = \boxed{1,10}$ for the serviceability limit state.

If the slip factor μ is found by tests in conformity with Annex A the partial safety factor for the ultimate limit state may be reduced by 0,1.

(3) Slotted or oversized holes are not usual in aluminium structures and are not covered by these clauses.

6.5.9.4 Preloading

(1)P For high strength bolts conforming with relevant prENs or ENs, with controlled tightening in conformity with section 7, the design preloading force $F_{p,Cd}$, to be used in the design calculations, shall be taken as:

$$F_{p,Cd} = 0{,}65\, f_{ub}\, A_S \text{ for 8.8 bolts} \tag{6.26a}$$

$$F_{p,Cd} = 0{,}7\, f_{ub}\, A_S \text{ for 10.9 bolts} \tag{6.26b}$$

(2)P Where other types of preloaded bolts or other types of preloaded fasteners are used, the design preloading force $F_{p,Cd}$ shall be agreed between the client, the designer and the competent authority.

6.5.9.5 Slip factor

(1) The design value of the slip factor μ is dependent on the specified class of surface treatment. The value of μ for the lightly shot blasting standard treatment, N10a, see ISO 468/1302, without surface protection treatments, should be taken from table 6.6.

Table 6.6: Slip factor of treated friction surfaces

Total joint thickness mm	Slip factor μ
$12 \le \Sigma t < 18$	0,27
$18 \le \Sigma t < 24$	0,33
$24 \le \Sigma t < 30$	0,37
$30 \le \Sigma t$	0,40

Experience show that surface protection treatments applied before shot blasting lead to lower slip factors.

(2)P The calculations for any other surface treatment or the use of higher slip factors shall be based on specimens representative of the surfaces used in the structure using the procedure set out in Annex A.

6.5.9.6 Combined tension and shear

(1)P If a slip-resistant connection is subjected to an applied tensile force F_t in addition to the shear force F_v tending to produce slip, the slip resistance per bolt shall be taken as follows:

Category B: Slip-resistant at serviceability limit state

$$F_{s,Rd,ser} = \frac{n\,\mu\,(F_{p,Cd} - 0{,}8\; F_{t,Ed,ser})}{\gamma_{Ms,ser}} \tag{6.27}$$

Category C: Slip-resistant at ultimate limit state

$$F_{s,Rd} = \frac{n\,\mu\,(F_{p,Cd} - 0{,}8\; F_{t,Ed})}{\gamma_{Ms,ult}} \tag{6.28}$$

6.5.10 Prying forces

(1)P Where fasteners are required to carry an applied tensile force, they shall be proportioned to also resist the additional force due to prying action, where this can occur, see figure 6.10.

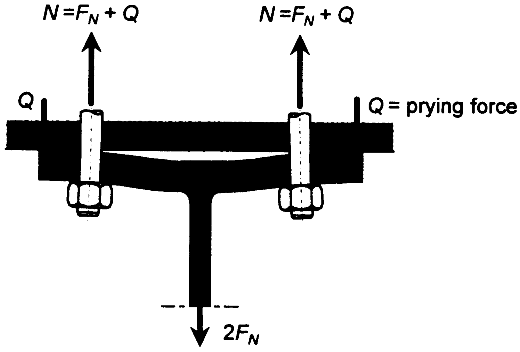


Figure 6.10: Prying forces

(2) The prying forces depend on the relative stiffness and geometrical proportions of the parts of the connection, see figure 6.11.

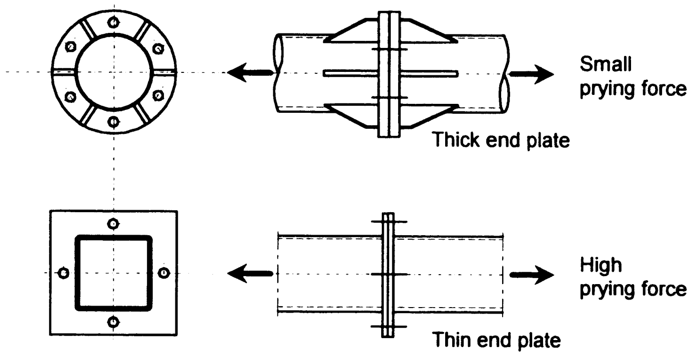


Figure 6.11: Effect of details on prying forces

(3) If the effect of the prying force is taken advantage of in the design of the parts, then the prying force should be determined by a suitable analysis.

6.5.11 Long joints

(1)P Where the distance L_j between the centres of the end fasteners in a joint, measured in the direction of the transfer of force (see figure 6.12), is more than 15 d, where d is the nominal diameter of the bolts or rivets, the design shear resistance $F_{v,Rd}$ of all the fasteners calculated as specified in 6.5.5 or 6.5.6 as appropriate shall be reduced by multiplying it by a reduction factor β_{Lf}, given by:

$$\beta_{Lf} = 1 - \frac{L_j - 15\,d}{200\,d} \tag{6.29}$$

but $0{,}75 \leq \beta_{Lf} \leq 1{,}0$.

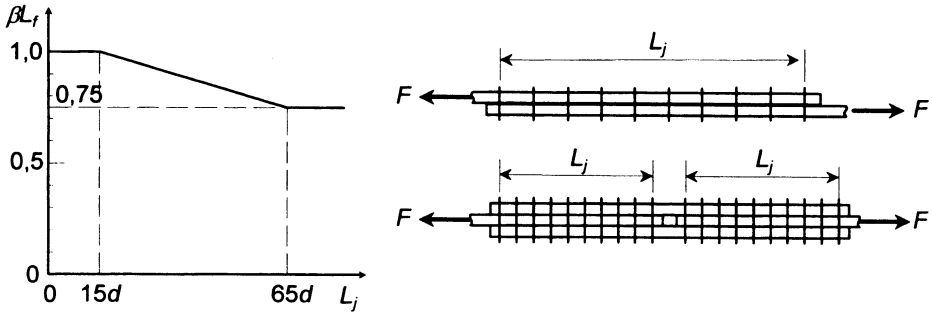


Figure 6.12: Long joints

(2) This provision does not apply where there is a uniform distribution of force transfer over the length of the joint, e.g. the transfer of shear force from the web of a section to the flange.

6.5.12 Single lap joints with one fastener

(1)P In single lap joints of flats with only one fastener, see figure 6.13, the bolt shall be provided with washers under both the head and the nut to avoid pull-out failure. Single rivets should not be used in single lap joints.

(2)P The bearing resistance $F_{b,Rd}$ determined in accordance with 6.5.5 shall be limited to:

$$F_{b,Rd} \leq 1{,}5\, f_u\, d\, t\, / \gamma_{Mb} \tag{6.30}$$

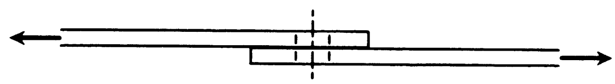


Figure 6.13: Single lap joint with one bolt

(3) In the case of high strength bolts, grades 8.8 or 10.9, appropriate washers should be used for single lap joints of flats with only one bolt, even where the bolts are not preloaded.

6.5.13 Fasteners through packings

(1)P Where bolts or rivets transmitting load in shear and bearing pass through packings of total thickness t_p greater than one-third of the nominal diameter d, the design shear resistance $F_{v.Rd}$ calculated as specified in 6.5.5 or 6.5.6 as appropriate, shall be reduced by multiplying it by a reduction factor β_p given by:

$$\beta_p = \frac{9d}{8d + 3\,t_p} \quad but\ \beta_p \leq 1{,}0 \tag{6.31}$$

(2 For double shear connections with packings on both sides of the splice, t_p should be taken as the thickness of the thicker packing.

(3) Any additional fasteners required due to the application of the reduction factor β_p may optionally be placed in an extension of the packing.

6.5.14 Pin connections

6.5.14.1 Scope

(1) This clause applies to pin connections where free rotation is required. Pin connections in which no rotation is required may be designed as single bolted connections, see 6.5.5 and 6.5.9.

Pins may not be loaded in single shear, so one of the members to be joined should have a fork end, or clevis. The pin retaining system, e.g. spring clip, should be designed to withstand a lateral load equal to 10% of the total shear load of the pin.

6.5.14.2 Pin holes and pin plates

(1)P The geometry of plates in pin connections shall be in accordance with the dimensional requirements.

(2)P At the ultimate limit state the design force N_{Rd} in the plate shall not exceed the design bearing resistance given in table 6.7.

(3)P Pin plates provided to increase the net area of a member or to increase the bearing resistance of a pin shall be of sufficient size to transfer the design force from the pin into the member and shall be arranged to avoid eccentricity.

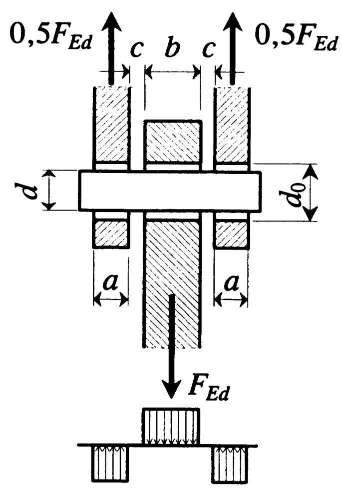
6.5.14.3 Design of pins

(1) The bending moments in a pin should be calculated as indicated in figure 6.14.

(2)P At the ultimate limit state the design forces and moments in a pin shall not exceed the relevant design resistances given in table 6.7.

Table 6.7: Design resistances for pin connections

Criterion	Resistance
Shear of the pin	$F_{v,Rd} = 0{,}6\, A\, f_{up}/\gamma_{Mp}$
Bending of the pin	$M_{Rd} = 0{,}8\, W_{el}\, f_{up}/\gamma_{Mp}$
Combined shear and bending of the pin	$[M_{Ed}/M_{Rd}]^2 + [F_{v,Ed}/F_{v,Rd}]^2 \leq 1{,}0$
Bearing of the plate and the pin	$F_{b,Rd} = 1{,}5\, t\, d\, f_0/\gamma_{Mp}$



$$M_{Ed} = \frac{F_{Ed}}{8}(2a + 4c + b) \tag{6.32}$$

Figure 6.14: Bending moment in a pin

6.6 Welded connections

6.6.1 General

(1) In the design of welded joints consideration should be given both to the strength of the welds and to the strength of the HAZ.

(2)P The design guidance given here shall be applied to:

- The welding process MIG for all thicknesses and TIG only for material thicknesses up to $t = 6$ mm and for repair;

- the welder and the welding procedure are approved in accordance with qualification requirements as specified, i.e. normal quality level, see 7.5;

- Combinations of parent and filler metal as given in 3.3.4;

- Structures loaded with predominantly static loads.

(3)P If - in case of strength members - the above conditions are not fulfilled special test pieces have to be welded and tested, see 7.5.

(4)P If for partial or non strength members a lower quality level has been specified by the designer lower design strength values $\gamma_M = \boxed{1,65}$ instead of $\gamma_M = \boxed{1,25}$ shall be used, see also 6.1.1.

(5)P In order to ensure the welding quality qualification specimens have to be welded according to a written welding procedure specification. This enables to approve the welder and the welding method as well as to determine the welding parameters and other relevant data which can be added to the welding procedure specification. And, if necessary, they can be subjected to mechanical testing to prove the design and procedure.

6.6.2 Heat-affected zone (HAZ)

(1)P For the following classes of alloys a heat-affected zone has to be taken into account (see also 5.3.4):

- Heat-treatable alloys in any heat-treated condition above T4 (6xxx and 7xxx series);
- Non-heat-treatable alloys in any work-hardened condition (3xxx and 5xxx series).

(2)P The severity and extent (dimensions) of HAZ softening given in 5.5 shall be taken into account. Both severity and extent are different for TIG and MIG welding. For TIG welding a higher extent (larger HAZ area) and more severe softening due to the higher heat-input shall be applied.

(3)P The characteristic strengths $f_{a,haz}$ and $f_{v,haz}$ for the material in the HAZ are given in 5.5.2. HAZ softening factors shall be taken from the table 5.2.

6.6.3 Design of welded connections

(1)P For the design of welded connections the following has to be verified:

- the design of the welds, see 6.6.3.2 and 6.6.3.3;
- the design strength of the HAZ adjacent to a weld, see 6.6.3.4;
- the design of connections with combined welds, see 6.6.3.5.

(2) The deformation capacity of a welded joint can be improved when the design strength of the welds is greater than that of the material in the HAZ.

6.6.3.1 Characteristic strength of weld metal

(1) For the characteristic strength of weld metal (f_w) the values according to table 6.8 may be used, provided that the combinations of parent metal and filler metal as given in 3.3.4, are applied.

(2) With the design of welded connections in aluminium alloy structures it should be noted that - except for the strength reduction in the HAZ - also the strength of the weld metal usually appears to be lower than the strength of the parent metal.

Table 6.8: Characteristic strength values of weld metal f_w

Characteristic strength	Filler metal	Alloy								
		3103	5052	5083	5454	6060	6005A	6061	6082	7020
f_w [N/mm^2]	5356	-	170	240	220	160	180	190	210	260
	4043A	95	-	-	-	150	160	170	190	210[1)]

NOTE 1: For extruded profiles and material thickness $5 < t \leq 25$ mm in alloy 6060-T5 the above values have to be reduced to 140 N/mm^2 (see table 3.2b).
NOTE 2: For alloy 5754 the values of alloy 5454 and for alloy 6063 the values of alloy 6060 can be used.
NOTE 3: If filler metals 5056A, 5556A, or 5183 are used then the values for 5356 have to be applied.
NOTE 4: If filler metals 4047A or 3103 are used then the values of 4043A have to be applied.
NOTE 5: For different combinations of alloys the lowest characteristic strength of the weld metal has to be used.
[1)] Only in special cases due to the low strength and elongation of the joint

(3)P The characteristic strength of weld metal shall be distinguished according to the filler metal used. The choice of filler metal has a significant influence on the strength of the weld metal.

6.6.3.2 Design of butt welds

6.6.3.2.1 Full Penetration Butt Welds

(1)P Full penetration butt welds shall be applied for strength members.

(2) The effective thickness of a full penetration butt weld shall be taken as the thickness of the connected members provided well-exercised welding. With different member thicknesses the smallest member thickness shall be taken into account as weld thickness.

(3) Reinforcement or undercut of the weld within the limits as specified should be neglected for the design.

(4)P The effective length shall be taken as equal to the total weld length when run-on and run-off plates are used. Otherwise the total length shall be reduced by twice the thickness t.

6.6.3.2.2 Partial Penetration Butt Welds

(1)P Partial penetration butt welds shall only be used for strength members when verified by testing that no serious weld defects are apparent.

In other cases partial penetration butt welds shall be only applied with a higher γ_{Mw} value because of the high susceptibility for weld defects of partial penetration butt welds. For partial penetration butt welds an effective throat section has to be applied (see figure 6.22).

6.6.3.2.3 Design Formulae for Butt Welds

(1)P For the design stresses the following shall be applied:

- normal stress, tension or compression, perpendicular to the weld axis, see figure 6.15:

$$\sigma_{\perp} \leq \frac{f_w}{\gamma_{Mw}} \tag{6.33}$$

- shear stress, see figure 6.16:

$$\tau \leq 0{,}6\ \frac{f_w}{\gamma_{Mw}} \tag{6.34}$$

- combined normal and shear stresses:

$$\sqrt{\sigma_{\perp}^2 + 3\ \tau^2} \leq \frac{f_w}{\gamma_{Mw}} \tag{6.35}$$

where:

f_w characteristic strength weld metal according to table 6.8;
$\sigma_{\perp}$ normal stress, perpendicular to the weld axis;
τ shear stress, parallel to the weld axis;
γ_{Mw} partial safety factor for welded joints, see 6.1.1.

NOTE: Normal stresses parallel to the weld axis do not have to be considered, see figure 6.15.

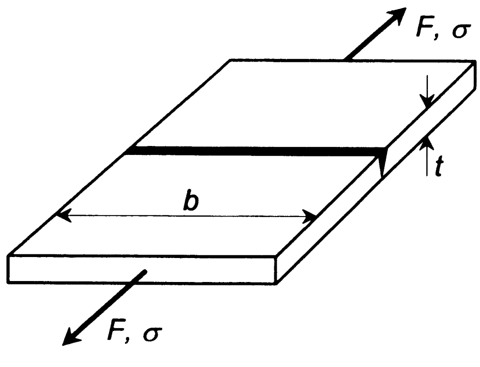


Figure 6.15: Butt weld, normal stresses

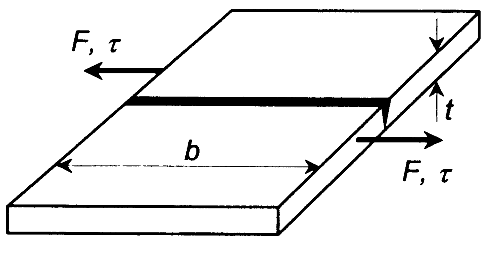


Figure 6.16: Butt weld, shear stresses

6.6.3.3 Design of fillet welds

(1)P For the design of fillet welds the throat section shall be taken as the governing section, since the actual strength of a fillet weld is well approximated by considering the throat section and the forces acting on that section.

(2)P The throat section shall be determined by the effective length and the effective throat thickness of a fillet weld.

(3)P The effective length shall be taken as the total length of a fillet weld when:

- the length of the fillet weld is at least 8 times the throat thickness, and
- the length of the fillet weld does not exceed 100 times the throat thickness with non-uniform stress
- the stress distribution along the length of the weld is constant for instance in case of lap joints as shown in figure 6.17).

NOTE 1: With uniform stress distributions no restriction for the length of a fillet weld is necessary, see figure 6.17.

(4)P If the above listed requirements are not fulfilled the effective weld length of longitudinal fillet welds shall be taken as given below.

(5)P If the stiffness of the connected members differs considerably from each other reduction of the effective weld length has to be taken into account.

(6)P If the length of longitudinal fillet welds has to be reduced, the following shall be applied:

$$L_{w,eff} = (1{,}2 - 0{,}2\, L_w/100\, a)\, L_w \text{ with } L_w \leq 100\, a \qquad (6.36)$$

where:

$L_{w,eff}$ = effective length of longitudinal fillet welds
L_w = total length longitudinal fillet welds
a = effective throat thickness, see figure 6.18.

(7) With non-uniform stress distributions and thin, long welds the deformation capacity at the ends may be exhausted before the middle part of the weld yields; thus the connection fails by a kind of zipper-effect.

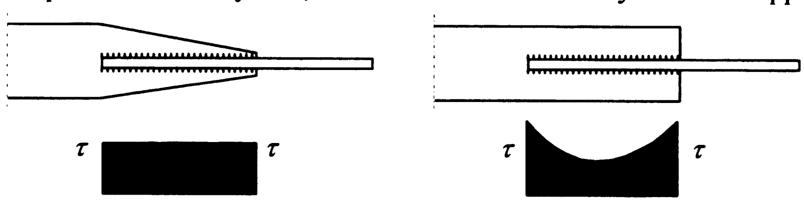


Figure 6.17: Stress Distributions in Lap Joints with Fillet Welds
a) Example of a uniform stress distribution
b) Example of a non uniform stress distribution

(8)P The effective throat thickness a has to be determined as indicated in figure 6.18 (a the height of the largest triangle which can be inscribed within the weld).

(9) When the qualification specimens show a consistent, positive root penetration, for design purposes the following may be assumed:

- The throat thickness may be increased by 20% or 2 mm whichever is smaller, under the condition that a qualification procedure has been prepared. Thus: a = 1,2 a or $a = a + 2$ mm.
- With deep penetration fillet welds the additional throat thickness may be taken into account provided that consistent penetration has been proved by test. Thus: $a = a + a_{pen}$, see figure 6.18.

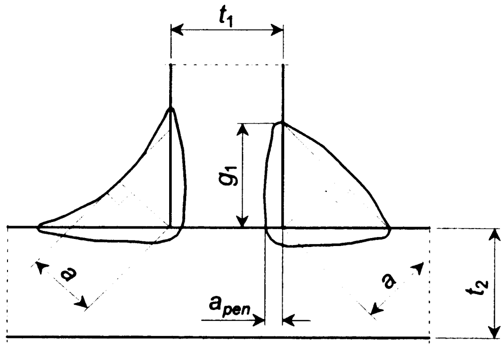


Figure 6.18: Effective throat thickness *a*; positive root penetration a_{pen}

(10) The forces acting on a fillet weld can be resolved into stress components with respect to the throat section, see figure 6.19. These components are:

- a normal stress $\sigma_\perp$, perpendicular to the throat section;
- a shear stress $\tau_\perp$, acting on the throat section perpendicular to the weld axis;
- a shear stress $\tau_{||}$, acting on the throat section parallel to the weld axis.

NOTE: A normal stress $\sigma_{||}$ acting along the weld axis does not have to be considered, see figure 6.19.

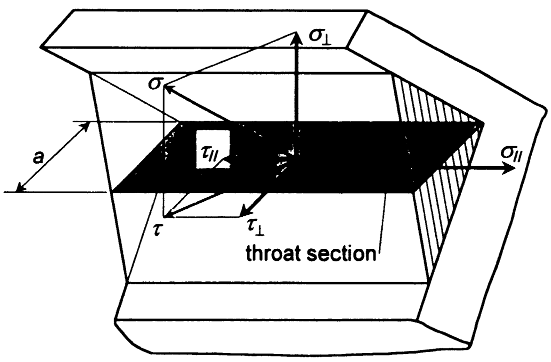


Figure 6.19: Stresses $\sigma_\perp$, $\tau_\perp$ and $\tau_{||}$, acting on the throat section of a fillet weld.

(11)P These stress components shall be combined to a comparison stress σ_c as follows:

$$\sigma_c = \sqrt{\sigma_\perp^2 + 3\left(\tau_\perp^2 + \tau_{II}^2\right)} \tag{6.37}$$

where

For the design stresses the following shall be applied:

$$\sigma_c \leq \frac{f_w}{\gamma_{Mw}} \tag{6.38}$$

$$\sigma_\perp \leq \frac{f_w}{\gamma_{Mw}} \tag{6.39}$$

where:

f_w is the characteristic strength of weld metal according to table 6.8;

γ_{Mw} is the partial safety factor for welded joints, see 6.1.1.

(12)P For two frequently occurring cases the following design formulas derived from the β-formula, shall be applied:

- Double fillet welded joint, loaded perpendicularly to the weld axis (see figure 6.20). For the throat thickness a holds:

$$a > 0{,}7 \frac{\sigma t}{f_w / \gamma_{Mw}} \tag{6.40}$$

where:

$$\sigma = \frac{F}{t\,b} \text{ normal stress in the connected member;} \tag{6.41}$$

F design load in the connected member;

f_w characteristic strength of weld metal according to table 6.8;

t thickness of the connected member, see figure 6.20;

b width of the connected member.

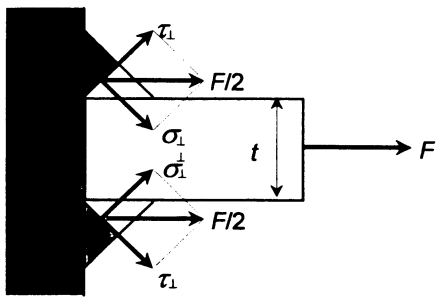


Figure 6.20: Double fillet welded joint loaded perpendicularly to the weld axis

- Double fillet welded joint, loaded parallel to the weld axis (see figure 6.21). For the throat thickness a shall be applied:

$$a > 0{,}85 \frac{\tau \ t}{f_w / \gamma_{Mw}} \tag{6.42}$$

where:

$\tau = \dfrac{F}{t\ h}$ shear stress in the connected member; (6.43)

F load in the connected member;

f_w characteristic strength of weld metal according to table 6.8;

t thickness of the connected member, see figure 6.21;

h height of the connected member, see figure 6.21.

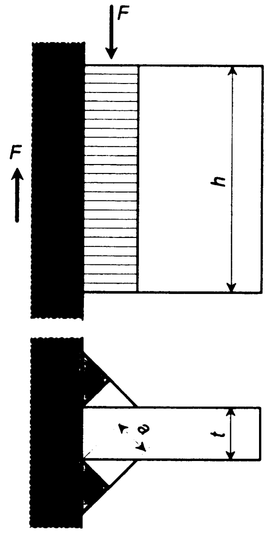


Figure 6.21: Double fillet welded joint loaded parallel to the weld axis

6.6.3.4 Design strength HAZ

(1)P The design strength of a HAZ adjacent to a weld shall be taken as follows:

a) Tensile force perpendicular to the failure plane (see figure 6.22):
HAZ butt welds:

$\sigma_{haz} \leq \dfrac{f_{a,haz}}{\gamma_{Mw}}$ at the toe of the weld (full cross section); (6.44)

HAZ partial penetration butt welds:

$\sigma_{haz} \leq \dfrac{t_e\ f_{a,haz}}{t\ \gamma_{Mw}}$ at the fusion boundary (partial penetration ($t_e < t$); (6.45)

HAZ fillet welds:

$\sigma_{haz} \leq \dfrac{f_{a,haz}}{\gamma_{Mw}}$ at the toe of the weld (full cross section); (6.46)

$\sigma_{haz} \leq \dfrac{g_1\ f_{a,haz}}{t\ \gamma_{Mw}}$ at the fusion boundary, see figures 6.18 and 6.22. (6.47)

where:

σ_{haz} design normal stress perpendicular to the weld axis;

t thickness connected member;

t_e effective throat thickness partial penetration butt weld;

g_1 leg length fillet weld (see figure 6.18);

$f_{a,haz}$ characteristic strength HAZ, see 6.6.2;

γ_{Mw} material factor for welded joints, see 6.1.1.

b) Shear force in failure plane:

HAZ butt welds:

$$\tau_{haz} \leq \frac{f_{v,haz}}{\gamma_{Mw}} \text{ at the toe of the weld;} \tag{6.48}$$

$$\tau_{haz} \leq \frac{t_e}{t} \frac{f_{v,haz}}{\gamma_{Mw}} \text{ at the fusion boundary.} \tag{6.49}$$

HAZ fillet welds:

$$\tau_{haz} \leq \frac{f_{v,haz}}{\gamma_{Mw}} \text{ at the toe of the weld;} \tag{6.50}$$

$$\tau_{haz} \leq \frac{g_1}{t} \frac{f_{v,haz}}{\gamma_{Mw}} \text{ at the fusion boundary.} \tag{6.51}$$

where:

τ_{haz} shear stress parallel to the weld axis;

$f_{v,haz}$ characteristic shear strength HAZ, see 6.6.2;

γ_{Mw} material factor for welded joints, see 6.1.1;.

Other symbols: see 6.6.3.4a).

c) Combined shear and tension:

HAZ butt welds:

$$\sqrt{\sigma^2 + 3\,\tau^2} \leq \frac{f_{v,haz}}{\gamma_{Mw}} \text{ at toe of the weld;} \tag{6.52}$$

$$\sqrt{\sigma^2 + 3\,\tau^2} \leq \frac{t_e}{t} \frac{f_{v,haz}}{\gamma_{Mw}} \text{ at fusion boundary.} \tag{6.53}$$

HAZ fillet welds:

$$\sqrt{\sigma^2 + 3\,\tau^2} \leq \frac{f_{a,haz}}{\gamma_{Mw}} \text{ at the toe of the weld;} \tag{6.54}$$

$$\sqrt{\sigma^2 + 3\,\tau^2} \leq \frac{g_1}{t} \frac{f_{a,haz}}{\gamma_{Mw}} \text{ at fusion boundary.} \tag{6.55}$$

Symbols see 6.6.3.4a) and b).

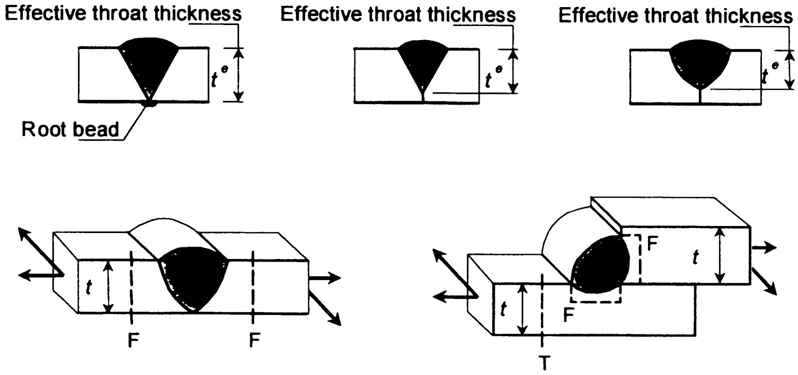


Figure 6.22: Failure planes HAZ adjacent to a weld; F = HAZ, fusion boundary; T = HAZ, toe of the weld, full cross section

(2) The above design guidance about HAZ is dealing with welded connections as such. In 5.3 and 5.5 design guidance is given for the effect of HAZ on the structural behaviour of members.

6.6.3.5 Design of connections with combined welds

(1)P For the design of connections with combined welds one of the two following methods shall be applied:

Method 1: The loads acting on the joint are distributed to the respective welds which are most suited to carry them.
Method 2: The welds are designed for the stresses occurring in the parent metal of the different parts of the joint.

(2) Applying one of the above methods the design of connections with combined welds is reduced to the design of the constituent welds.

NOTE 1: With method 1 it has to be checked whether the weld possesses sufficient deformation capacity to allow for such a simplified load distribution. Besides, the assumed loads in the welds should not give rise to overloading of the connected members.
NOTE 2: With method 2 the above problems do not exist, but sometimes it may be difficult to determine the stresses in the parent metal of the different parts of the joint.
NOTE 3: Assuming a simplified load distribution, like described as method 1, is the most commonly applied method. Since the actual distribution of loads between the welds is highly indeterminate, such assumptions have been found to be an acceptable and satisfactory design practice. However, these assumptions rely on the demonstrated ability of welds to redistribute loads by yielding.

(3) Residual stresses and other stresses not participating in the transfer load need not be considered for the design. For instance, stresses due to minor eccentricities in the joint need not be considered.

6.7 Hybrid connections

(1)P When different forms of fasteners are used to carry a shear load or when welding and fasteners are used in combination, the designer shall verify that they act together.

(2) In general the degree of collaboration may be evaluated through a consideration of the load-displacement curves of the particular connection with individual kind of joining, or also by adequate tests of the complete hybrid connection.

(3)P In particular normal bolts with hole clearance shall not collaborate with welding.

(4) Preloaded high-strength bolts in connections designed as slip-resistant at the ultimate limit state (Category C in 6.5.3.1) may be assumed to share load with welds, provided that the final tightening of the bolts is carried out after the welding is complete. The total design load should be given by the appropriate design load of each fastener with its corresponding γ_M-value.

6.8 Adhesive bonded connections

6.8.1 General

(1) Structural joints in aluminium may be made by bonding with adhesive. Bonding needs an expert technique and should be used with great care.

(2)P The design guidance given here shall only be applied under the condition that:

- the joint design is such that only shear forces have to be transmitted (see 6.8.2.1);

- appropriate adhesives are applied (see 6.8.2.2);

- the surface preparation procedures before bonding do meet the specifications as required by the application (see 6.8.2.2(3)).

(3)P The use of adhesive for main structural joints shall not be contemplated unless considerable testing has established its validity, including environmental testing and fatigue testing if relevant.

(4) Adhesive jointing can be suitably applied for instance for plate/stiffener combinations and other secondary stressed conditions.

(5) Loads should be carried over as large an area as possible. Increasing the width of joints usually increases the strength pro rata. Increasing the length is beneficial only for very short overlaps. Longer overlaps do result in more severe stress concentrations in particular at the ends of the laps.

6.8.2 Design of adhesive bonded connections

6.8.2.1 Joint design

(1) With the design of adhesive bonded joints shear forces should be looked after; tensile forces - in particular peeling or other forces tending to open the joint - should be avoided or should be transmitted by complementary structural means. Furthermore uniform distribution of stresses and sufficient deformation capacity to enable a ductile type of failure of the component are to be strived for.

For example tensile forces on a joint can be transmitted by extruded parts of a joint while shear forces are taken by adhesive bonding, see figure 6.23.

Sufficient deformation capacity is arrived at in case the design strength of the joint is greater than the yield strength of the connected member.

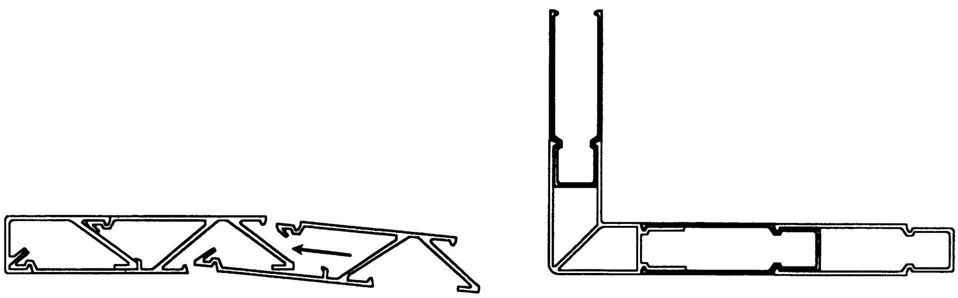


Figure 6.23: Extruded members; in-plane tensile forces transmitted by snapping parts; shear loading transmitted by adhesive bonding

6.8.2.2 Characteristic strength of adhesives

(1) As far as the mechanical properties are concerned high strength adhesives should be used for structural applications (see table 6.9). However, also the toughness should be sufficient to overcome stress/strain concentrations and to enable a ductile type of failure. The influence of the adhesives E-modulus on the strength and stiffness of the joint is not significant. But, low E-modulus adhesives are more sensitive to creep. Concerning other adhesives properties it is noted that in the temperature range -20°C up to +60°C the adhesive properties do not vary too much as long as the glass transition temperature is not exceeded.

(2)P Pre-treatments of the surfaces to be bonded have to be chosen such that the bonded joint meets the design requirements during service life of the structure. Sometimes, simply degreasing will suffice, but often additional mechanical (e.g. brushing) or chemical pre-treatments (e.g. etching, anodizing, chromate conversion of the surface) should be considered with joints in highly stressed components.

(3) For the characteristic shear strength of adhesives $f_{v,adh}$ for structural applications the values of table 6.9 may be used.

Table 6.9: Characteristic shear strength values of adhesives

Adhesive types	$f_{v,adh}$ N/mm²
1- component, heat cured, modified epoxide	35
2- components, cold cured, modified epoxide	25
2- components, cold cured, modified acrylic	20

(4) The adhesive types as mentioned in table 6.9 may be used in structural applications under the conditions as given earlier in 6.8.2.1 resp. 6.8.2.2. The values given in table 6.9 are based on results of extensive research. However, it is allowed to use higher shear strength values than the ones given in table 6.9 provided that adequate tests are carried out, see 6.8.3.

6.8.2.3 Design shear stress

(1)P The design shear stress shall be taken as

$$\tau \leq \frac{f_{v,adh}}{\gamma_{Ma}} \qquad (6.56)$$

where:

τ shear stress in the adhesive layer;

$f_{v,adh}$ characteristic shear strength value adhesive, see 6.8.2.2;

γ_{Ma} =3,0; material factor for adhesive bonded joints, see 6.1.1.

The above high value of γ_{Ma} has to be used since:

- the design of the joint is based on ultimate shear strength of the adhesive;
- the scatter in adhesive strength can be considerable;
- the experience with adhesive bonded joints is small.

6.8.3 Tests

(1) Higher characteristic shear strength values of adhesives than given in table 6.9 may be used when thick adherend shear tests are carried out, see figure 6.24. The results of these tests have to be evaluated according to section 8.4 to arrive at a reliable shear strength value of the applied adhesive.

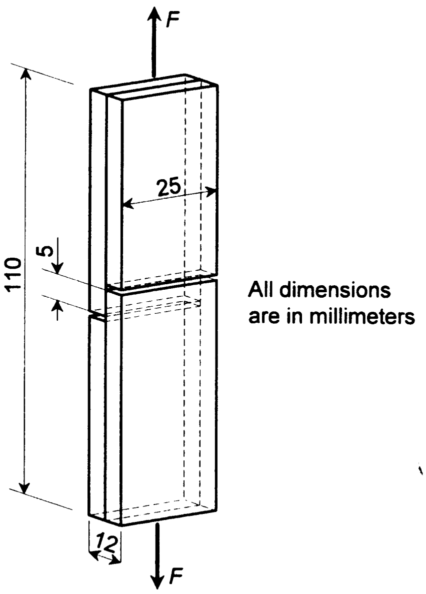


Figure 6.24: Thick adherend shear test specimen

(2) The strength of adhesive bonded joints or members may also be determined by testing according to section 8. Sample joints should be made at full scale, using the same manufacturing procedure as for production joints. These sample joints should be tested with similar joint construction and loading to that occurring in the actual structure.

7 Fabrication and Execution

7.1 General

7.1.1 Scope

(1) This chapter identifies those aspects of workmanship which need to be specified for fabrication and execution to ensure that the design assumptions of this Eurocode are satisfied and hence that the intended level of structural safety can be attained.

(2)P Any supplementary requirement specific to particular structures shall be stated in the Project Specification.

7.1.2 Requirements

(1)P All structural aluminium alloy materials, fasteners and welding consumables shall conform with the requirements specified in section 3.

(2)P If any alternative or additional material is used, the requirements specified in (1) shall be supplemented as necessary to ensure a similar level of safety and serviceability (durability).

7.2 Project specification

(1)P The designer/specifier shall provide, or adopt, a project specification containing details of all the requirements for materials, fabrication, protection and erection necessary to ensure compliance with the design assumptions relevant to the particular structure.

(2)P The Project Specifications shall refer to the requirements of 7.1.2 but may be supplemented by any special requirement for:

- fabrication;
- execution;
- protection;
- inspection;
- acceptance.

(3)P The Project Specification may supplement the requirements of the Reference Standards but it shall not relax their technological requirements and it shall not supersede the minimum requirements specified in this chapter.

(4)P The Project Specification shall specify which options listed in the documentation in 7.1.2 shall apply to the project.

(5) The Project Specification may include drawings in addition to text.

(6)P Once approved, the Project Specification shall not be altered without the agreement of the specifier/designer.

(7) As far as possible, the requirements in the Project Specification should not be altered from 7.1.2.

7.3 Preparation of material

7.3.1 General

(1)P Fabrication and assembly operations for aluminium shall take into account the low weight of the structures and assemblies, the great flexibility of members, the dimensional changes due to temperature and the ease with which aluminium can be machined.

(2)P During erection the structure shall be securely bolted or fastened. Temporary bracing shall be used if necessary to ensure stability under all erection loads and conditions, including those due to erection equipment and its operation.

7.3.2 Storage and transportation

(1)P Aluminium shall be stored in dry places, clear of the ground. Contact with other metals and with materials such as cement and damp timber shall be avoided.

NOTE: This is to avoid the possibility of superficial corrosion which may cause unsightly staining or marking. Sheet and plate are particularly susceptible.

(2)P Care shall be taken of material for architectural use, particularly if the material is anodized; surfaces shall be protected with strippable tapes, waxes of lacquers while danger of damage exists.

(3)P Sheet and plate shall be stacked, if possible on end in racks, to avoid distortion.

(4)P Aluminium construction parts shall be packed to avoid mechanical damage, abrasion and contact by agents liable to cause surface corrosion and staining during transport.

7.3.3 Fabrication, erection and assembly tolerances

(1)P Tolerances on the fabrication, the site erection and the assembly of structures and components shall be as specified by the designer.

7.3.4 Marking out

(1)P Fine scribing-lines shall not be used on critically stressed areas of thin material, and where subsequent welding in involved, paint, chalk, graphite or other marking materials likely to contaminate shall not be used.

(2)P Due attention shall be given to the effects of the relatively high coefficient of expansion of aluminium in measuring, marking out and assembly, particularly when temperature variations are large.

7.3.5 Cutting

(1)P Cutting shall be machining, shearing or arc-cutting. Bandsaws and circular saws shall have a tooth form and pitch to suit the thickness of the material to be cut. Cut edges shall be smooth and free from burrs, distortions and other irregularities. Care shall be taken to avoid the use of tools contaminated by other metals, particularly copper or brass. Shearing shall normally be limited to material 6 mm thick or less. Arc-cutting shall be applied by a process shown by test, to the satisfaction of the designer, to have no deleterious effect on the material. Flame-cutting shall not be used, but it is possible to use water cutting.

(2)P Sheared or arc-cut edges shall be subsequently machined or filed smooth if used as edge preparations for welds in strength members. For 7xxx series alloys, sheared edges shall be machined back to remove areas of residual stress.

7.3.6 Drilling, punching and reaming

(1)P Holes shall be made by either drilling, or drilling followed by reaming. Punching can be used up to thicknesses of about 20 mm except for 7xxx series alloys. In the case of 7xxx series alloys, undersize punching could be performed provided that all burrs, edge defects, local distortions and areas of residual stresses are removed by subsequent reaming.

(2)P Holes for bolts and rivets, unless otherwise specified by the designer, shall be of the sizes specified in table 7.1. Where holes for bolts and rivets in certain members are required by the designer to be drilled with parts assembled and tightly clamped together, the parts shall be subsequently separated to remove burrs.

Table 7.1: Hole clearance for bolts, rivets and special fasteners

Type	Material	Diameter mm	Clearance on diameter mm	
Bolts close fitting	-	any	≤ 0,15[1)]	
Bolts not close fitting	-	 < 13 ≥ 13	reduced clearance ≤ 0,5[1)] ≤ 1[1)]	normal clearance 1 2
Solid rivets	Aluminium	< 13 ≥ 13	≤ 0,4 ≤ 0,8	
	Steel or corrosion resisting steels	< 13 ≥ 13	≤ 0,8 ≤ 1,6	
HS bolts (all plies up to three, or outer two plies or greater than three plies)	Steel or corrosion resisting steels	≤ 24 > 24	≤ 2 ≤ 3	
HS bolts (above three plies, inner plies)	Steel or corrosion resisting steels	any	≤ 3	
Special rivets and fasteners	As recommended by the manufacturer			

[1)] from measurements of actual bolt and hole diameters.
NOTE 1: The clearance should not be increased on account of irregular or excess zinc coating on the bolts.
NOTE 2: For metal-sprayed parts the clearance before spraying may at the discretion of the designer, be increased by 0,15 mm except where the hole is deep and spraying is non-uniform

(3)P The depth of countersinking shall not exceed the thickness of the countersunk part less 4 mm unless specified by the designer. In no case shall the depth exceed the thickness of the countersunk part less 1 mm.

7.3.7 Bending and forming

(1)P Where bending or forming is required the fabricator shall consult the manufacturer's recommendations regarding the bend radius, alloy and temper appropriate to the operation and any subsequent heat treatment required. When doubt exists about the formability of a member, tests shall be carried out to the approval of the designer before fabrication commences.

(2)P Heat treatment and hot forming shall be carried out only under competent metallurgical direction and supervision. Without the agreement of the manufacturer, the 7xxx series alloys shall not be bent or cold formed in the fully heat-treated condition because of the risk of stress corrosion cracking. However, minor corrections to the shape of profiles or to weld-distortion may by performed.

(3)P Any piece of component that cracks or fractures because of forming shall be rejected.

7.4 Bolted connection

7.4.1 Bolting

(1)P Where either the full area of the shank of the bolt, or the full bearing area of the shank of the bolt, is to be developed, the threaded portion of the bolt shall not extend within the thickness of the connected parts. In addition, the length of the bolt shall be such that at least one clear thread shows above the nut after tightening, and at least one thread plus the thread runout is clear between the nut and the un-threaded shank of the bolt.

(2)P Washers shall be provided under all the bolt heads and nuts. Galvanized steel washers shall be used with steel bolts. Washers of pure aluminium, or of the same material as the bolt or the member, shall be used with corrosion resisting steel bolts.

(3)P Nuts shall be fully, but not excessively, tightened. Locking devices shall be used as required.

(4)P The threads of aluminium and stainless steel bolts shall be lubricated before assembly, if the joint will subsequently be dismantled.

NOTE: Lanolin sealing is recommended for the threads of anodized bolts.

7.4.2 Friction grip bolting

7.4.2.1 Surfaces in contact

(1)P For joints between aluminium members, at the time of assembly, the contact surfaces shall be clean, free from burrs and defects which would prevent solid seating of the parts, and free from substances that would interfere with the development of friction between them.

(2)P When the aluminium surfaces are treated by grit blasting, (for slip coefficients, refer to (6.5.9.5)), the blasting treatment shall be sufficient to give a visually uniform coverage of the surfaces.

NOTE: When it is required to ensure consistent friction properties to conform with test results, careful quality control of the process is required by such methods as the use of test strips.

(3)P Grit quality shall be controlled, particularly where grit is re-used.

7.4.2.2 Holes in members

(1)P Holes in members shall comply with 7.3.6.

7.4.2.3 Assembly

(1)P When other bolts than those complying with EN, prEN or ISO Standards related to bolts are used, the procedure for achieving the required shank tension shall be agreed with the designer.

(2)P If a joint has been dismantled it shall not be re-assembled unless the surface treatment of the interface has been re-applied.

7.4.3 Riveting

(1)P Riveted joints shall be tightly drawn together before and during riveting. Rivets shall be driven so as to completely fill the holes, including any countersunk holes. Heads shall be concentric with their shanks and in close contact with the riveted surfaces.

(2)P Tubular and other special rivets shall be cold formed using the tools and procedure recommended by the supplier.

(3)P Loose or defective rivets shall be removed, preferably by drilling or machining away the head and punching the shank through. Then new rivets shall be driven.

7.5 Welding

7.5.1 General

(1)P Site work shall be avoided if possible. It shall be done only where there is complete protection which simulates shop conditions.

(2)P Welding other than the specified on the drawing shall not be allowed without the prior written agreement of the designer. Temporary welded attachment shall not be included unless specifically agreed with the designer.

(3)P Dimensions shall have allowance for the effects of weld shrinkage. Welding sequences and heat input shall be balanced to avoid warping and distortion. In the case of complex structures the design engineer should consult a welding engineer at an early stage.

(4)P Where permanent backing bars are used they shall be of material compatible with the members joined.

7.5.2 Welding consumables

(1)P The filler wire shall be selected in accordance with 3.3.4 or prEN 1011-4:1995 taking into account the specific requirements of the joint, or it shall be as otherwise specified by the designer.

7.5.3 Welding processes

(1)P Strength members shall be welded by either the tungsten inert-gas welding (TIG) or the metal inert-gas welding (MIG) processes in accordance with EN 288-1:1992 and EN 288-4:1992, taking account of special procedure requirements, see 7.5.6.

(2) New and high effective methods (laser, high energy MIG, friction stir welding…) can be advantageously used after approval tests.

7.5.4 Approval of welding procedures

(1)P Unless otherwise specified by the designer, the precise course of action to be followed for each type of joint shall be documented as a welding procedure and approved by the designer in accordance with EN 288-1:1992, EN 288-4:1992 and prEN 288-13.

(2)P Where the production joint design or application is such that none of the test pieces can be regarded as representative, e.g. attachment welds to thin pipes, then a special test piece shall be made which simulates the production joint in all essential features, e.g. dimensions, restraint, access, heat-sink effects. The type of special test piece to be welded and the tests to be carried out shall be agreed by the contracting parties at the time of placing an order.

NOTE: Procedure need not be re-approved if the fabricator satisfies the designer that similar procedures have been approved previously.

(3) If procedures of this section are followed, a value of $\gamma_M = \boxed{1,25}$ may be used. If not, $\gamma_M = \boxed{1,6}$ should be taken (see 6.6.1).

7.5.5 Approval of welders

(1)P Unless otherwise specified by the designer all welders shall be approved for each type of joint to EN 287-2 (See also prEN 1418 for fully mechanized automatic welding).

7.5.6 Weld quality and inspection

7.5.6.1 General

(1) The main requirements for control of weld quality are:

(a) procedure approval (see 7.5.4);
(b) welder approval (see 7.5.5);
(c) non-destructive examination of production welds immediately before and after welding; and testing of production control test pieces after welding (see 7.5.6.2).

7.5.6.2 Methods of inspection and testing of production welds

(1)P All production welds both immediately before and after welding shall be subject to approval according to project specifications.

(2)P Visual inspection shall be by carried out by experienced personnel.

NOTE: Guidance on visual inspection, including weld-size gauges and magnifying glasses, is found in prEN 970. See also prEN 1011-4.

7.5.6.3 Quality levels

(1)P The extent of the inspection and inspection methods (see 7.5.6.4) and the magnitude and level of acceptable imperfections (see 7.5.6.5.) are both dependent on the quality level required for the weld. The quality level specified for each weld depends on the stressing requirements and shall be one of the following.

(a) Minimum quality (Level D, EN 30042) may only be used where the designer has indicated on the drawings those parts to be considered as requiring minimum quality.

NOTE 1: The minimum quality level will normally apply where the actions under factored loading do not exceed one third of the factored resistance of the member or joint, e.g. stiffness may dictate design. This will apply to both static and fatigue resistance.

(b) Normal quality (Level C, EN 30042) shall be applied where the drawings do not specify any other quality level requirement. Absence of indication of quality level shall be taken to be normal quality level.

NOTE 2: The normal quality level will normally apply where the actions under factored loading exceed one-third of the factored resistance of the member or joint, and where the required class for fatigue does not exceed 20.

c) Fatigue quality (Level B, EN 30042) shall be applied where the designer has indicated on the drawings the detail or details requiring an appropriate fatigue quality level by means of „Fat" level numbers and an arrow indicating the direction of stress fluctuation (see prENV 1999-2, 6.2.).

NOTE 3:The fatigue quality level will normally apply where the required level for fatigue exceeds 20 (see prENV 1999-2, 6.2.). There are five possible fatigue qualities, depending on the type of joint and the degree of stress fluctuation (required level). They are referred to as Fat 25, 31, 39, 49, 62. The joint type and stress directions to which they can apply are limited (see tables 5.2a, 5.2b and 5.3 of prENV 1999-2). For non-welded material the surface finish is defined in table 5.1 of prENV 1999-2.

(2)P Where however there is doubt as to the quality level for a particular welded joint or detail the engineer shall be consulted.

7.5.6.4 Extent of inspection

(1)P The extent of visual and dimensional inspection immediately before welding and the visual, penetrant and ultrasonic and/or radiographic inspection required after welding for each weld quality level and type of connection shall be specified by the designer.

(2)P For welds of fatigue quality class Fat 39 and above and for welds in difficult conditions or where specified by the designer, production control test pieces or run-off test plates which simulate the production joint in all essential features, e.g. dimension, restraint, access, heat-sink effects, shall be clamped in line with the joint and welded using the production welding procedure. These shall be inspected and approved to the same inspection levels as equivalent production weld.

NOTE 1: Mechanical test may be needed on production control test pieces including run-off test plates where specified by the designer.

NOTE 2: Examination of such production control test pieces of run-off test plates may be used additionally as an aid to the interpretation of production non-destructive testing. Special inspection procedures such as other non-destructive tests, shall be employed if specified.

(3)P Where a welded structure or component is to be post-weld heat treated the inspection after welding shall be performed after the heat treatment.

7.5.6.5 Inspection acceptance levels

(1)P The acceptance level for imperfection in welded joint both immediately before and after welding shall be as specified by the designer.

(2)P Where any inadequacy in joint preparation, detected by inspection immediately before welding, exceeds that specified, the work shall be re-prepared, tacked or jigged as appropriate and re-inspected.

(3)P Where any imperfection detected after welding exceeds that specified, or where other unacceptable imperfections are revealed by any special inspection procedure, the weld shall be rejected and repaired in accordance with 7.5.7.

7.5.7 Repair or replacement

(1)P All procedures for repair or replacement of any weld shall be approved by the designer.

(2)P The extent of any defective weld shall be determined by the appropriate inspection procedure and clearly marked on the joint.

(3)P The repaired weld shall be re-inspected in accordance with 7.5.6.2. No weld shall be remade, nor any part of a weld replaced, more than twice without reference to the Engineer.

7.6. Structural adhesives

(1)P The method of making bonded joints shall be documented so that the process is repeatable after the procedure has been properly established. The joining procedure shall be proved by adequate tests and shall receive the approval of the designer before it is used in actual fabrication. Approval tests shall be specified by the designer, bearing in mind the service conditions of the structure, and the specimens shall be representative of the size and type of joint to be fabricated.

NOTE 1: Procedures need not be re-approved if the fabricator satisfies the designer that similar procedures have been approved previously.

(2) The use of adhesive for main structural joints shall not be contemplated unless considerable testing has established its validity, including environmental testing and fatigue testing is relevant.

NOTE 2: However, adhesive joining can be considered for plate/stiffener combinations and other secondary stressed conditions.

7.7 Protection

7.7.1 General

(1)P Where aluminium requires added protection, the protection system applied to parts or all of the structure shall be as specified by the designer.

NOTE: Where structural work has a direct aesthetic content, the aluminium surface may be given decorative finishes such as by painting, anodizing or certain chemical conversion processes. These processes also impart varying degrees of protection to the aluminium.

7.7.2 Painting

7.7.2.1 General

(1)P Where painting is specified by the designer the following procedure shall be followed.

(2)P Painting shall be preceded by appropriate pre-treatment with the operations in 7.7.7.2 and 7.7.7.3, carried out in sequence without intermediate delays. Surface shall be thoroughly dry, and coatings applied when the ambient temperature is above 4° C unless agreed otherwise by the designer. Contact surfaces shall be painted as specified by the designer.

7.7.2.2 Cleaning

(1)P The surface shall be cleaned, dried and thoroughly degreased by an appropriate organic solvent. Flame cleaning shall not be performed.

7.7.2.3 Pre-treatment

(1)P The clean degreased surfaces shall be treated to ensure paint adhesion.

NOTE 1: The treatment may be carried out by mechanical roughening with abrasive paper or adhesive-impregnated nylon pads or by abrasive-blasting, provided that, in each case, the abrasive is either alumina or other non-metallic and copper-free grit. Mechanical roughening may also be carried out with corrosion-resisting steel wire brushes.

(2)P Copper-plated-steel wool or wire brushes shall not be used for mechanical roughening.

NOTE 2:Alternatively, paint adhesion may be obtained by the use of an appropriate conversion coating or an etch-primer or wash primer, provided that the metal surface is clean and free from thick or irregular oxide coatings.

7.7.2.4 Non-bituminous system

(1)P The pre-treated (see 7.7.2.3) or metal-sprayed (see 7.7.3.1) surfaces shall receive a priming coat with an appropriate inhibiting pigment compatible with the aluminium substrate and any subsequent coats.

(2)P The priming coat shall not contain any copper, mercury, tin or lead compounds, graphite or carbonaceous materials.

(3)P The primed surface shall be painted with one or more coats of paints of a type recommended by the paint manufacturer to be compatible with the priming coat, and any subsequent coats, and chosen to suit the environment and provide the durability required. Such subsequent coat shall not contain any copper, mercury or tin compound, graphite or carbonaceous materials and preferably shall not contain lead compounds. An aluminium paint system shall consist of a non-leafing undercoat and leafing finishing coat.

7.7.2.5 Bituminous paint systems

(1)P Bituminous paint or dip-applied hot bitumen shall be used.

NOTE: Bituminous paint is preferred.

(2)P The materials shall be applied direct to surfaces cleaned in accordance with 7.7.2.2 and roughened if necessary by mechanical means (see 7.7.2.3); they shall not be applied over painted or etch primed surfaces.

7.7.2.6 Pre-coated material and factory applied finishes

(1)P The fabricator shall ensure to the satisfaction of the designer that the use of pre-coated material or factory applied finishes affords the protection appropriate to the assembly and to the environment.

NOTE 1: Aluminium materials may be supplied ready painted, using either a solvent based liquid or a dry (powder coating) process. The finish can be applied prior to fabrication, to partly fabricated material, or following fabrication. Liquid coatings are usually cured by stoving at an elevated temperature. Powder coatings are always cured by stoving. In both cases the stoved coatings will have improved properties with regard to hardness and durability compared to an air dried coating. Such finishes are suitable for application strip, coil or extruded sections. The coating may be ductile enough to permit slight deformation without damage, as by press-braking or by roll forming. The use of factory applied finishes and pre-coated materials, generally provides an acceptable degree of protection.

NOTE 2: Additional protection at joints (see 7.7.3) may be necessary. The fabricator shall ensure that the aluminium alloy is not significantly weakened by the stoving process.

NOTE 3: Most factory applied finishes are cured by heating to about 200° C for a short time, which normally has only a very slight effect on the mechanical properties. It is however possible for significant weakening to occur, depending on the alloy, its condition and also on the temperature profile of the stoving process. Routine thermal monitoring is normal practice and is particularly important with thick sections, where there is more risk of under or over curing. A report on the thermal history of the metal from a suitable cure monitoring system is recommended.

7.7.3 Protection of metal-to-metal contact surfaces and at bolted and riveted joints

7.7.3.1 General

(1)P At metal-to-metal contact surfaces additional protection to that applied in 7.7.2 shall be applied as indicated in table 3.8 and as specified by the designer. The protection system procedures indicated in table 3.8 shall be obtained by following the procedures described in 7.7.3.2 to 7.7.3.4 using the procedure notation used on table 3.8. Where metal-to-metal contacts occur which are not covered by table 3.8 specialist advice shall be sought.

(2) The procedures as determined from table 3.8 apply to the contact areas of the structural parts, see 7.7.3.2; to the treatment applied to the bolt and rivets, see 7.7.3.3; and to additional treatments see 7.7.3.4.

(3)P Sprayed metal, if specified by the designer to coat steelwork which is in contact with aluminium as a means of protecting the steel and the contacting aluminium, shall comply with grade 1050A.

(4)P Aluminium sheet roofing and siding shall be protected in accordance with prEN 508-2:1996.

7.7.3.2 Treatment applied to the contact areas of structural members

(1) Procedure O
No treatment shall be applied.

(2) Procedure O/X
Treatment depend on structural conditions. Small contact areas and areas which dry quickly may be assembled without sealing (see X).

(3) Procedure X
Both contact surfaces must be assembled so that no crevices exist where water can penetrate. Both contact surfaces, including bolt and rivet holes shall, before assembly, be cleaned, pre-treated and receive one priming coat, see 7.7.2.4, or sealing compound, extending beyond the contact area. The surfaces should be brought together while the priming coat is still wet. When assembling pre-painted or protected components sealing of the contact surfaces may not be necessary dependant on the composition of the paint or protection system employed, the expected life and the environment. The need for additional sealing shall be established by agreement with the designer.

(4) Procedure Y
Full electrical insulation between the two metals and all fixings shall be ensured by the insertion of non-absorbent, non-conducting tapes, gaskets and washers to prevent metallic and electrical contact between the different metals of the joint. Care must be taken to ensure that there are no crevices between the insulation materials and the metal. The use of additional coating or sealants may be necessary.

(5) Procedure Z
Where procedure Y is required and the load transfer through the joint precludes the use of insolating materials, the joint should be assembled without the use of insolating elements, with the whole joint assembly completely sealed externally to prevent moisture ingress to elements of the joint. Procedures shall be established by agreement with the designer.

7.7.3.3 Treatment applied to bolts and rivets

(1)P Procedure 0
No additional treatment shall be applied.

(2) Procedure 1
Aluminium bolts in 7075 or 2017A should be used unprotected only in dry or mild climates, otherwise they must be adequately protected.

(3) Procedure 2
Aluminium bolts in 7075 or 2017A should not be used even if protected.

(4)P Procedure 3
Inert washers or jointing compound shall be applied between the bolt heads, nuts, washers and connected materials to seal the joint and to prevent moisture entering the interface between components and fixings. Care should be employed to ensure that load transfer through the joint is not adversely affected by the washers or jointing compounds.

(5)P Procedure 4
Where the joint is not painted or coated for other reasons, the heads of bolts, nuts and rivets and the surrounding areas as noted below, shall be protected with at least one priming coat (see 7.7.2.4), care being taken to seal all crevices.

Where zinc-coated bolts are used, the protection on the aluminium side of the joint is not necessary.

Where aluminium bolts or rivets are used, the protection on the aluminium side of the joint is not necessary.

Where stainless steel bolts are used in combination with aluminium and zinc-coated steel parts, the surrounding zinc-coated area of the joint shall be similarly protected.

7.7.3.4 Further treatments

(1) Procedure a
If not painted for other reasons it may be necessary to protect the adjacent metallic parts of the contact area by a suitable paint coating in cases where dirt may be entrapped or where moisture retained.

(2) Procedure z
Additional protection of zinc-coated structural parts as a whole may be necessary.

(3) Procedure g
The protection of all aluminium parts is generally necessary see table 3.7.

7.7.4 High strength slip resistant joints

(1)P Where additional protection is specified by the designer it shall comply with 7.7.3 as far as is permitted by the mechanical requirements for the contact surfaces of the joint. Where epoxy resin has been used further protection shall be unnecessary, but where a dry joint is used the joint edges shall be sealed to prevent the ingress of moisture (see 7.7.3.3).

7.7.5 Pinned joints

(1)P Where additional protection is specified by the designer it shall follow 7.7.3 as far as is permitted by the requirements and configuration of the joint.

NOTE: Further protection may be provided by coating the contact surfaces, holes for the pin, and the pin with a high melting point reversible grease.

7.7.6 Welded joints

(1)P The protection system specified by the designer shall be applied in accordance with the procedures in 7.7.3.

7.7.7 Bonded joints

(1)P The protection system specified by the designer shall be applied. The advice of the adhesive manufacturer shall be sought to ensure there shall be no interaction between the adhesive and the protective system, e.g. solvent or heat effects.

7.7.8 Contact between aluminium and non-metallic materials

(1) The method of protection specified by the designer to avoid contacts between aluminium and non-metallic materials such as used in the building industry shall be applied in accordance with the requirements for the relevant paint or protection procedure.

7.8 Inspection and safety

(1)P The designer or such representatives of independent inspection and testing agencies as may be appointed shall have access to all places where work both on site and at the contractor's premises is being carried out. Facilities for inspection and testing of the work shall be provided in accordance with the procedure agreed between the fabricator and designer.

NOTE: Attention is drawn to the appropriate statutory requirements which affect operations involved in manufacture, fabrication, erection and service.

8 Design Assisted by Testing

8.1 Basis

(1) The provisions in this section give guidance to designers who may become involved with experimental assessments.

(2)P When the calculation models available are not sufficient for a particular structure or structural component, experimental assessment shall be undertaken in place of design by calculation or to supplement design by calculation.

(3) Experimental verification may also be undertaken where the rules for design by calculation given in this Eurocode would lead to uneconomic results. However, the conservative assumptions in the specified calculation models (which are intended to account for unfavourable calculation influences not explicitly considered in the specified calculation models) shall not be by-passed.

(4)P The planning, execution, evaluation and documentation of tests shall be in accordance with the minimum requirements stated in this section.

(5) Because circumstances and test facilities vary greatly, the test procedures should be agreed in advance by all concerned.

8.2 Planning of tests

(1)P The experimental assessment shall be based on tentative calculation models, which may be incomplete, but which relate one or several relevant variables to the structural behaviour under consideration, such that basic tendencies are adequately predicted. The experimental assessment shall then be confined to the evaluation of correction terms in the tentative calculation model.

(2)P If the prediction of the relevant calculation models or of the failure mode to be expected in the tests is extremely doubtful, the test plan shall be developed on the basis of accompanying pilot tests.

(3)P Prior to the execution of tests, a test plan shall be drawn up by the designer and the testing organisation. This shall contain the objective of the tests and all the instructions and other specifications necessary for the selection or production of the test specimens, the execution of the tests and the test evaluation.

(4) Reference should be made to Reference Standards 14 and 15 (see annex A) and to annex J for guidance in preparing the test plan.

(5)P The test plan shall deal with the following items:

(a) Scope of information required from the tests (e.g. required parameters and range of validity).

(b) Description of all properties of the members considered which may influence the behaviour at a limit state, (e.g. form of the member, stiffness, aluminium grade and quality and relevant material properties, geometrical and structural parameters and their tolerances, parameters influenced by fabrication and erection procedures.

(c) Specifications on the properties of the test specimen (e.g. sampling procedures, specification for dimensions, material and fabrication of prototypes, number of specimens, number of subsets, restraints).

(d) Description of the actions to which the members are required to react and demonstrate the properties referred to in (b), (e.g. load arrangements, load cases, load combinations).

(e) Specifications on the loading and environmental conditions in the test (e.g. loading points, loading methods, loading path in time and space, temperatures).

(f) Modes of failure and tentative calculation models with the corresponding relevant variables, see (8.2 (1).

(g) Testing arrangements (including measures to ensure sufficient strength and stiffness of the loading and supporting rigs and clearance for deflections etc.).

(h) Determination of the monitoring points and methods for observation and recording (e.g. time histories of strains, forces, deflections).

(i) Determination of the type and control of load application (stress-controlled, strain-controlled etc.).

(6)P All details on the sampling or manufacturing of the specimens shall be reported and measurements shall be carried out on these test specimens before the execution of tests starts, in order to demonstrate that the test plan has been fulfilled, otherwise it shall be revised.

8.3 Execution of Tests

(1)P The performance of experimental assessments shall be entrusted only to organisations where the staff is sufficiently knowledgeable and experienced in the planning, execution and evaluation of tests.

(2)P The testing laboratory shall be adequately equipped and the testing organisation shall ensure careful management and documentation of all tests.

8.4 Test evaluation

(1)P The test evaluation shall take account of the random character of all data.

(2) This test evaluation should be carried out using the method given in annex Z of ENV 1993-1-1:1992.

8.5 Documentation

(1)P The following documentation shall be provided in the test report

- the test plan (including any revisions);
- description and specifications of all test specimens;
- details of the testing arrangements;
- details of the execution of the tests, and

- the test results which are necessary for the test evaluation.

***Annex A** (normative) to be replaced by a European Standard „Test on Slip Factors for High Strength Friction Grip Connections in Aluminium“ (when available)*

Annex A (normative) Test of Slip Factor

A.1 The purpose of testing

(1) The purpose of this testing procedure is to determine the slip factor of a friction surface after a particular treatment, generally involving a surface coating.

(2) The procedure is intended to ensure that adequate account is taken of the possibility of creep deformation of the connection.

A.2 Significant variables

(1)P The validity of the test results for coated surfaces is limited to cases where all significant variables are similar to those of the test specimens.

(2)P The following variables shall be taken as significant:

a) the composition of the coating, see A.2 (3);
b) the maximum thickness of the coating, see A.3 (5);
c) the minimum time interval between application of the coating and application of load to the connection, see A.2 (4);
d) the curing procedure, see A.2 (5);
e) the property class of the hexagon bolt, see A.6 (4).

(3)P The composition of the coating shall be taken to also include the method of manufacture and any thinners used.

(4)P The curing procedure shall be documented, either by reference to published recommendations or by detailing the actual procedure.

(5)P The time interval (in hours) between coating and testing shall be recorded.

A.3 Test specimens

(1)P The test specimens shall conform to the dimensional details shown in figure A.1 a) or b). The aluminium material shall conform to EN 573, EN 515, EN 485, EN 586, EN 755, EN 52.1, prEN 132/100, prEN 190/110.

(2)P In order to ensure that the two inner plates have the same thickness, they shall be produced by cutting them consecutively from the same piece of material and assembled in their original relative positions.

(3)P The plates shall not have thermally cut edges. They shall be sufficiently flat to permit the prepared surfaces to be in full contact when the bolts have been preloaded.

(4)P The preload in the bolts shall be measured and shall be equal to the specified preload for the size and property class of the bolt used.

(5)P The specified surface treatment and coating shall be applied to the contact surfaces of the test specimens in a manner consistent with the intended structural application. The mean coating thickness on the contact surface of the test specimens shall be at least 0,05 mm greater than the maximum mean thickness specified for use in the structure.

(6)P The specimens shall be assembled such that the bolts are bearing in the opposite direction as the applied tension.

A.4 Test procedure

(1)P Five test specimens shall be tested. Four tests shall be loaded at normal speed (duration of test approximately 10 to 15 minutes). The fifth test specimen shall be used for a creep test.

(2)P The specimens shall be tested in a tension loading machine. The load-slip relationship shall be recorded.

(3)P The slip shall be taken as the relative displacement between adjacent points on an inner plate and a cover plate, in the direction of the applied load. It shall be measured for each end of the specimen separately. For each end, the slip shall be taken as the mean of the displacements on both sides of the specimen.

(4) The slip load is defined as the load at which a slip of 0,15 mm occurs.

(5)P The fifth test specimen shall be loaded with a specific load of 90 % of the mean slip load from the first four specimens (i.e. the mean of eight values).

(6)P If, following a time period between five minutes and three hours after the application of the load, the displacements at each end of the fifth test specimen have not increased by more than 0,002 mm, the slip loads for the fifth test specimen shall be determined as for the first four. If larger displacements occur extended creep tests shall be carried out, see A.5.

(7)P If the standard deviation of the ten values (obtained from the five test specimens) exceeds 8 % of the mean value, additional specimens shall be tested. The total number of test specimens (including the first five) shall be determined from:

$$n \geq (\delta/3{,}5)^2 \qquad \text{(A.1)}$$

where:

n is the number of test specimens;
δ is the standard deviation for the first five specimens (ten values) as a percentage of the mean value.

A.5 Extended creep test

(1)P Where it is necessary to carry out extended creep tests, see A.5 (6), at least three test specimens (six connections) shall be tested.

(2)P A specific load shall be applied, determined using the slip factor proposed for use in the structural application.

(3)P A „displacement - log time curve" shall be plotted (see figure A.2) to demonstrate that the load determined using the proposed slip factor will not cause displacements greater than 0,3 mm during the life of the structure, taken as 50 years unless specified otherwise. The „displacement - log time curve" may be extrapolated linearly as soon as the tangent can be determined with sufficient accuracy.

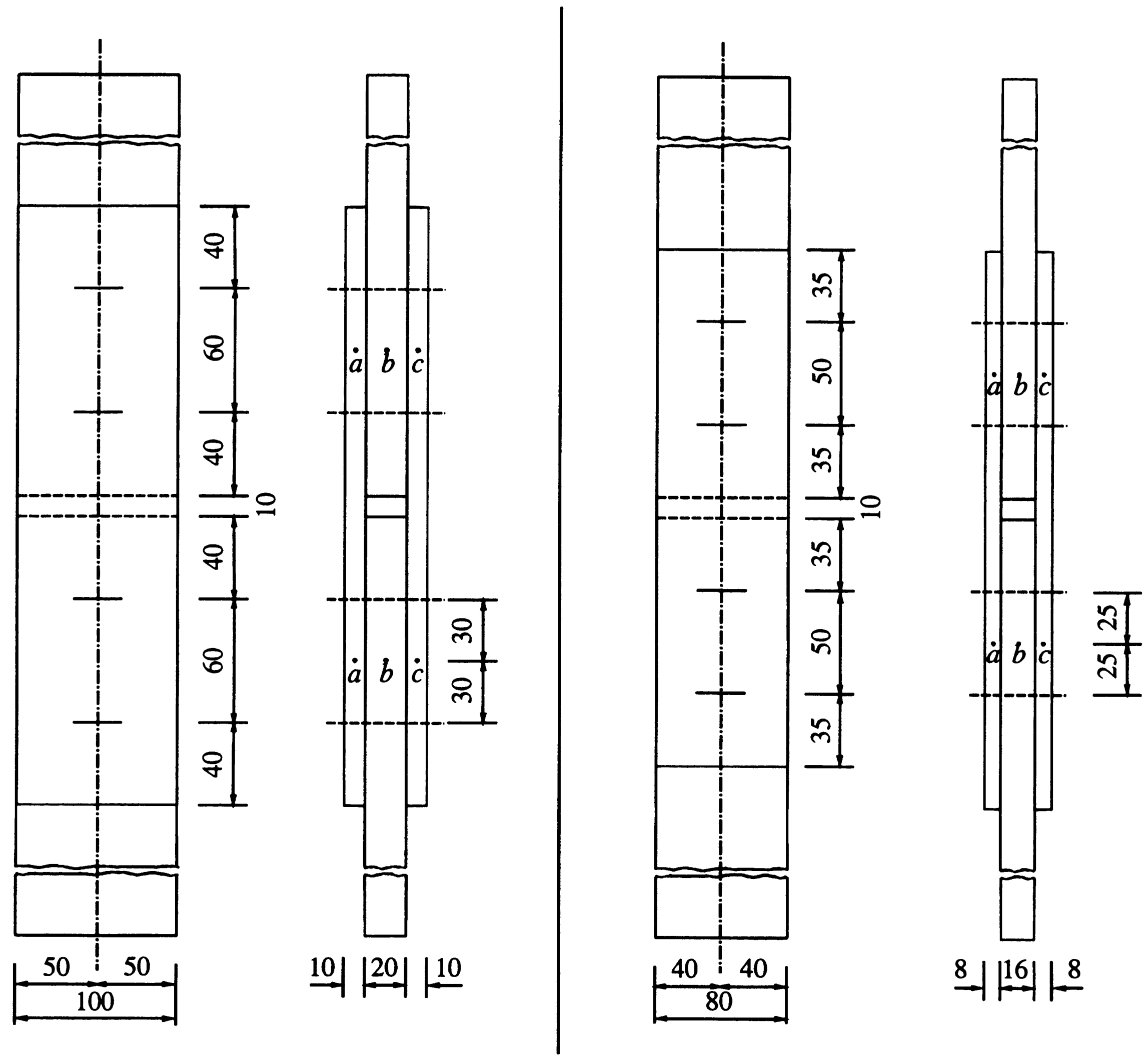
A.6 Test results

(1)P The characteristic slip factor resulting from testing shall be taken as the value statistically attainable in 95 % of all test specimens.

(2) For ten values, from five specimens, the characteristic value may be taken as the mean value minus 1,83 times the standard deviation.

(3) For extended creep test, the characteristic slip factor may be taken as the value demonstrated to satisfy the specified creep limit, see A.5 (3).

(4) Slip factor determined using bolts property class 10.9 may also be used for bolts property class 8.8. Alternatively separate tests may be carried out for bolts property class 8.8. Slip factors determined using bolts property class 8.8 shall not be assumed valid for bolts property class 10.9.



a) M20 bolts in 22 mm dia. holes

b) M16 bolts in 18 mm dia. holes

Figure A.1: Standard test specimens for slip factor test

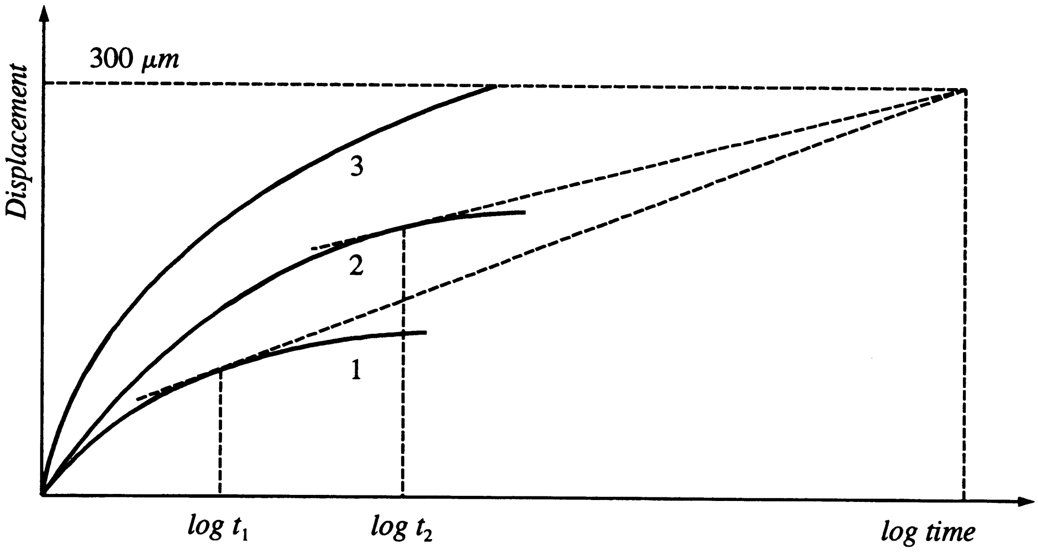


Figure A.2: Use of the displacement - log time curve for extended creep test

Annex B [informative]

Materials selection

B.1 General

(1) The choice of a suitable aluminium or aluminium alloy material for any application in the structural field is determined by a combination of factors; strength, durability, physical properties, weldability, formability and availability both in the alloy and particular form required. The wrought and cast alloys are described below subdivided into heat treatable and non-heat treatable alloys.

(2) The properties and characteristics of these alloys may be compared in general terms in table B.1 for wrought aluminium alloys and table B.2 for casting alloys. Properties and characteristics may vary with temper of the alloy.

(3) When connections are to be made to other metals, specialist advice should be sought on the protective measures necessary to avoid galvanic corrosion.

B.2 Wrought products

B.2.1 Wrought heat treatable alloys

(1) Within the 6xxx series alloys, the alloys EN AW-6082, EN AW-6061, EN AW-6005A, EN AW-6060, EN and EN AW-6063 are suitable for structural applications. The alloys EN AW-6082, EN AW-6061, EN AW-6005A, EN AW-6060 and EN AW-6063 all have durability rating B. Within the 7xxx series alloys, the alloy EN AW-7020 is suitable for general structural applications and has durability rating C.

B.2.1.1 Alloys EN AW-6082 and EN AW-6061

(1) EN AW-6082 is one of the most widely used heat treatable alloy and often the principal structural alloy in many countries for welded and non-welded applications. It is a high strength alloy available in most forms; solid and hollow extrusions, plate, sheet, tube and forging, and finds increasing use in components exposed to the marine environment. EN AW-6061 is also a widely used heat treatable alloy for welded and non-welded applications available in solid and hollow extrusions and tube. Both alloys are used normally in the fully heat treated condition EN AW-6082-T6 and EN AW-6061-T6.

(2) The choice of these alloys as a structural material is based on a favourable combination of properties; high strength after heat treatment, good corrosion resistance, good weldability by both the MIG and TIG processes, good formability in the T4 temper and good machining properties. Care must be taken to account for the loss of strength in the heat affected zone (HAZ) of welded joints. Strength can be recovered to a limited degree by post weld natural ageing. When used in extrusions it is generally restricted to thicker less intricate shapes than with the other 6xxx series alloys. AW-6082 is a common alloy for extrusions, plate and sheet from stock. The alloy may be rivetted using alloys EN AW-6082, EN AW-5056A or EN AW-5086 in O or harder tempers, filler metals for welding are specified in prEN 1011-4.

B.2.1.2 Alloys EN AW-6005A

(1) EN AW-6005A alloy which is also recommended for structural applications, is available in extruded forms only and combines medium strength with the ability to be extruded into shapes more complex than those obtainable with EN AW-6082 or EN AW-6061. This is particularly true for thin-walled hollow shapes. Like EN AW-6082 and EN AW-6061, the alloys are readily welded by the TIG and MIG processes and have similar loss of strength in the HAZ in welded joints. Filler metals for welding these alloys are specified in prEN 1011-4.

(2) The corrosion resistance of welded and unwelded components is similar or better than EN AW-6082. The machining properties are similar to those of EN AW-6082.

Table B.1: Comparison of general characteristics and other properties for structural alloys

Alloy CEN No	Form and temper availability							Strength	Resistance to corrosion	Formability	Machinability	Weldability	Decorative of hard anodising
	Sheet, strip & plate	Extruded products			Cold drawn products	Forging	Electrically welded tube						
		Bar/ rod	Tube	Pro- file	Tube								
EN AW-3103	0	-	-	-	-	-	0	II/IV	I	I/II	III/IV	I	II
EN AW-5083	0	0	0	x	0	0	-	I/II	I	II/IV	II/III	I	I/II
EN AW-5052	0	-	-	-	-	-	-	II/III	I	I/III	II/IV	I	I/II
EN AW-5454	0	-	-	-	-	-	-	II/III	I	I/III	II/IV	I	I/II
EN AW-5754	0	-	-	-	-	0	-	II/III	I	II/IV	II/III	I	I/II
EN AW-6060	-	0	0	0	0	-	-	II/III	II	II/III	II/III	I	I
EN AW-6061	-	0	0	0	0	-	-	II/III	II	II/III	II	I	I/II
EN AW-6063	-	0	0	0	0	-	-	II/III	II	II/III	II/III	I	I/II
EN AW-6005A	-	-	-	0	-	-	-	II	II	III	II	I	I
EN AW-6082	0	0	0	0	0	0	-	I/II	II	II/III	II	I	II
EN AW-7020	0	0	0	x	0	-	-	I	III	III	II	I	II

Key: 0 Available in a range of tempers, properties apply through this range
- Not normally available
x Simple sections only
I Excellent
II Good
III Fair
IV Poor

NOTE: These indications are for guidance only and each ranking is only applicable in the column concerned and may vary with temper

Table B.2: Comparison of casting characteristics and other general properties

Casting alloy	Form of casting			Castability				Machinability				
Designation	Sand	Chill or permanent mould	Pressure die	Fluidity	Resistance to hot tearing	Pressure tightness	Strength	as cast	after HT	Resistance to corrosion	Decorative anodising	Weldability
EN AC-42100	●	●		II	I	II	II	-	II	II	IV	II
EN AC-42200	●	●		II	I	II	II	-	II	II	IV	II
EN AC-43200	●	●		I	I	II	III	III	II	III	V	II
EN AC-44100	●	●		I	I	I	IV	IV	-	II/III	V	I
EN AC-51300	●	●		III	IV	IV	IV	I	-	I	I	II

Key: I Excellent
II Good
III Fair
IV Poor
V Not recommended
● Indicates the casting method most commonly used for each alloy.

NOTE 1: These indications are for guidance only and each ranking is only applicable in the column concerned.

NOTE 2: The properties will vary with the condition of the casting.

B.2.1.3 Alloys EN AW-6060 and EN AW-6063

(1) EN AW-6060 and EN AW-6063 alloys are recommended for structural applications and are available in extruded and cold drawn products only. They are used when strength is not of paramount importance and has to be compromised with appearance where they offer good durability and surface finish and the ability to be extruded into thin walled and intricate shapes. The alloys are particularly suited to anodising and similar finishing processes. Like other 6xxx series alloys they are readily weldable by both MIG and TIG processes and lose strength in welded joints in the HAZ. Filler metals for welding these alloys are specified in prEN 1011-4.

B.2.1.4 Alloys EN AW-7020

(1) EN AW-7020 alloys are recommended for structural applications for welded and non-welded applications. It is a high strength alloy available in solid and hollow extrusions; plate and sheet and tube. This alloy is not as easy to produce in complicated extrusions as 6xxx series alloys and is not readily available. It is used normally in the fully heat treated condition EN AW-7020-T6. It has better post weld strength than the 6xxx series due to its natural ageing property. This alloy and others in the 7xxx series of alloys are however sensitive to environmental conditions and its satisfactory performance is as dependant on correct methods of manufacture and fabrication as on control of composition and tensile properties.Due to the susceptibility of exfoliation corrosion, material in T4 temper should only be used in the fabrication stage provided the struture can be artificially aged after completion. When heat treatment is not applied after welding, the necessity of protection of the HAZ must be checked with the condition defined in 3.4.3.1. If a material in the T6 condition is subjected to any operations which induce cold work such as bending, shearing or punching etc., the alloy may be made susceptible to stress corrosion cracking. It is essential therefore that there be direct collaboration between the engineer and the manufacturer on the intended use and the likely service conditions.

B.2.2 Wrought non-heat treatable alloys

(1) Within the 5xxx series alloys, the alloys EN AW-5052 EN AW-5454 and EN AW-5754 and EN AW-5083 are recommended for structural applications all have durability rating I. Another non-heat treatable alloy considered for less stressed structural applications is EN AW-3103 again with durability rating I.

B.2.2.1 EN AW-5052, EN AW-5454 and EN AW-5754

(1) EN AW-5052, EN AW-5454 and EN AW-5754 are suitable for welded or mechanically joined structural parts subjected to moderate stress. The alloys are ductile in the annealed condition, but loose ductility rapidly with cold forming. They are readily welded by MIG and TIG processes using filler metals specified in prEN 1011-4. and offer very good resistance to corrosive attack, especially in a marine atmosphere. Available principally as rolled products their reduced magnesium content also allows only simple extruded solid shapes.

(2) The alloys can be easily machined in the harder tempers. EN AW-5754 is the strongest 5xxx series alloy offering practical immunity to intergranular corrosion and stress corrosion.

B 2.2.2 EN AW-5083

(1) EN AW-5083 is the strongest structural non-heat treatable alloy in general commercial use, including marine applications, possessing good as welded properties and very good corrosion resistance.

resistance in the marine environment. It is ductile in the soft condition with good forming properties but looses ductility with cold forming, and can become hard with low ductility.

(2) However the alloy may in all tempers, especially in H32 and H34 (HX) tempers, be susceptible to intergranular corrosion, which under certain circumstances, may develop into stress corrosion cracking under sustained loading. Special tempers such as H116 have been developed to minimise this effect. Nevertheless the use of this alloy is not recommended where the material is to be subjected to further heavy cold working and/or where the service temperature is expected to be above 65° C. In such cases the alloy EN AW-5754 should be selected instead.

(3) But where there is a particular requirement for the selection of an alloy/temper combination which may be susceptible to stress corrosion cracking due to its service conditions, when agreed between producer and purchaser and stated on the order, this material should be submitted to a stress corrosion test according to procedures to be defined.

(4) The alloy EN AW-5083 is easily welded by both MIG and TIG processes using the filler metals specified in prEN 1011-4, but when strain hardened materials are welded the properties in the HAZ will revert to the annealed value. The alloy is available in plate, sheet, drawn tube and forging. Due to the high magnesium content it is particularly hard to extrude in hollow or complex sections so is limited to simple full extruded sections. It has good machining qualities in all tempers. For rivetting alloy EN AW-5754-0 or H32 is suggested.

B.2.2.3 EN AW-3103

(1) EN AW-3103 is available in sheet and plate forms only. It is slightly stronger and harder than „commercially pure“ aluminium with high ductility, weldability and good corrosion resistance.

B.3 Cast products

(1) Five foundry alloys are recommended for structural applications, three heat treatable alloys EN AC-42100, EN AC-42200 and EN AC-43200 plus two non-heat treatable alloys, EN AC-44100 and EN AC-51400. These alloys are described below. All may be welded using the filler metals specified in prEN 1011-4, but some alloy combinations of cast and wrought products are not recommended because of the formation of brittle intermetallics.

(2) The design rules in Part 1.1 do not apply to castings. The casting alloys given below should only be used in load bearing structures after both adequate testing and the setting up of quality control procedures for the production of the castings has been performed to the approval of the engineer.

B.3.1 Heat treatable casting alloys EN AC-42100, EN AC-42200 and EN AC-43200

(1) EN AC-42100, EN AC-42200 and EN AC-43200 are all alloys in the Al-Si-Mg system and are responsive to heat treatment. All are suitable for sand and chill or permanent mould castings but are not normally used for pressure die castings except by using advanced casting methods. The highest strength is achieved with EN AC-42200-T6 but with a lower ductility than EN AC-42100.

(2) EN AC-43200 exhibits the best foundry castability with fair resistance to corrosion, good machinability and weldability. Foundry castability of alloys EN AC-42100 and EN AC-42200 is good, with good resistance to corrosion, machinability and weldability.

B.3.2 Non-heat treatable casting alloys EN AC-44100 and EN AC-51300

(1) EN AC-44100 and EN AC-51300 alloys are suitable for sand and chill or permanent mould castings but not recommended for pressure die castings. Alloy EN AC-44100 possesses excellent foundry castability, but EN AC-51300 has fair castability and is only suitable for more simple shapes. EN AC-51300 has the highest strength, has excellent resistance to corrosion and is machinable. EN AC-44100 has better weldability than EN AC-51300. The EN AC-51300 alloy may be decoratively anodised.

Annex C (informative)

Calculation of internal forces and moments

C.1 Global analysis

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

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

a) Elastic global analysis (linear or non linear)

b) Plastic global analysis (without or with hardening)

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

(4) Plastic global analysis may be used only where the member cross-sections satisfy the requirements specified for Class 1 in 5.4. Cross-sections of Class 2, 3 and 4 are not allowed. For Class 1 sections it is always requested to check the deformation capacity in relation to the ductility demand of the structural scheme (see Annex D).

(5) For more details on the global analysis methods, see Annex D.

(6) The design assumption for the connections should satisfy the requirements specified in C.2.

C.1.2 Elastic global analysis (linear)

(1) Elastic global analysis (in the linear range) should be based on the assumption that the stress-strain relationship of the material is linear, whatever the stress level is (figure C.1a).

(2) This assumption may be maintained for both first order and second order elastic analysis (see C.1.6), even where the resistance of a cross section is based on its resistance beyond the elastic limit.

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

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

b) all the members in which the moments are reduced have Class 1 or Class 2 cross-sections (see 5.4).

C.1.3 Elastic global analysis (non linear)

(1) Elastic global analysis (in the non linear range) should be based on the assumption that the stress-strain relationship of the material is non linear. The value of the instantaneous tangent modulus depends on the stress level (figure C.1b).

(2) This assumption may be maintained for both first order and second order elastic analysis (see C.1.6), even where the resistance of a cross section is based on its resistance beyond the elastic limit.

(3) The characterization of the stress-strain law of the material should take into account the actual non linear behaviour of the alloy. This may be described on the basis of the models given in Annex E.

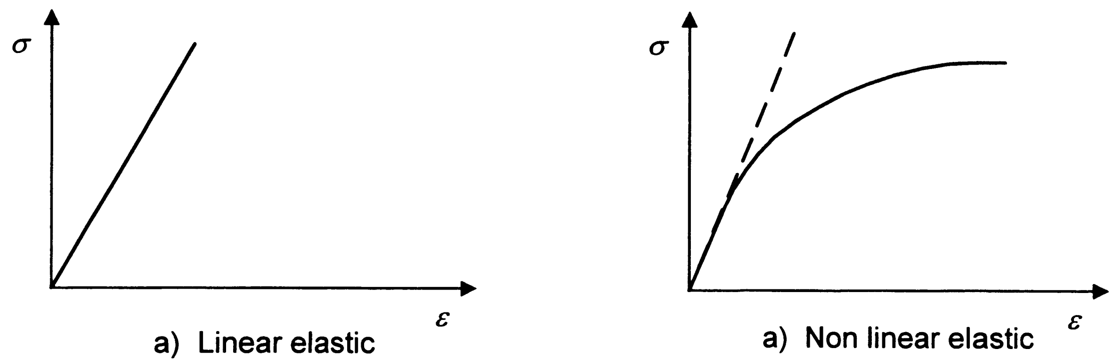


Figure C.1: Material behavioural models for elastic global analysis

C.1.4 Plastic global analysis (without hardening)

(1) Plastic global analysis (neglecting the hardening effect) may be carried out by assuming for the material the following behavioural models:

- Rigid-Perfectly plastic (figure C.2a)
- Elastic-Perfectly plastic (figure C.2b)
- Inelastic-Perfectly plastic (figure C.2c)

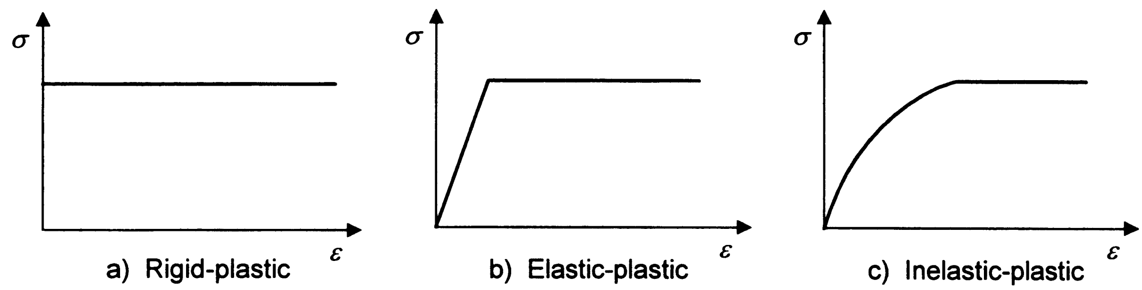


Figure C.2: Material behavioural models for perfectly plastic global analysis

(2) In case the "Rigid-Perfectly plastic" model is used for the material, elastic deformations of cross sections, members and foundations may be neglected. Plastic deformations are assumed to be concentrated at plastic hinge locations.

(3) In case the "Elastic-Perfectly plastic" model is used for the material, the behavior of the cross-sections remains perfectly elastic until the elastic limit stress in the most highly stressed fibres is reached. The transition to the plastic range will be more or less gradual depending on both load condition and section shape. Plastic deformations are assumed to be concentrated at plastic hinge locations.

(4) In case the "Inelastic-Plastic" model is used for the material, the actual non linear elastic behavior of both material and cross-section is considered in the evaluation of the deformations occurring in a given member before the formation of the plastic hinge. The cross-sections remain fully elastic until the stress in the most highly stressed fibres reaches the elastic limit. As the internal actions continue to increase, the section gradually enters the inelastic range, until the fully plastic resistance is reached.

C.1.5 Plastic global analysis (with hardening)

(1) Plastic global analysis allowing for the strain-hardening effect may be carried out by assuming for the material the following behavioural models:

- Rigid-Hardening (figure C.3a)

- Elastic-Hardening (figure C.3b)

- Generically inelastic (figure C.3c)

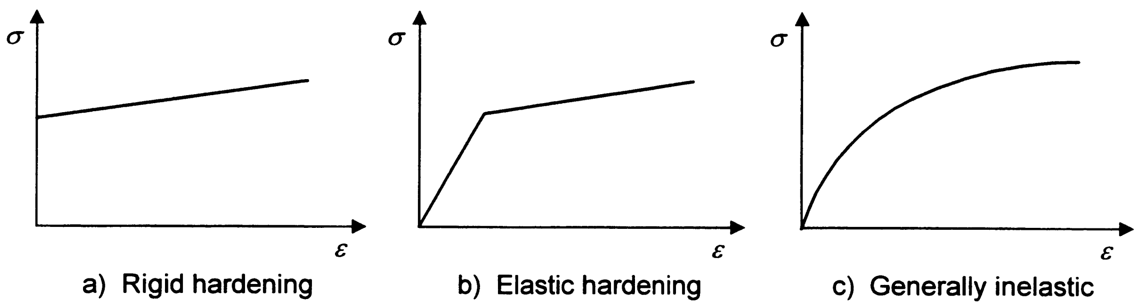


Figure C.3: Material behavioural models for plastic-hardening global analysis

(2) In case the "Rigid-Hardening" model is used for the material, elastic deformations of members, cross-sections and foundations may be neglected. Plastic deformations are assumed to be concentrated at plastic hinge locations. The strength in plastic hinges continues to increase, after the plastic resistance has been attained. The analysis stops when a given limit value of strength or deformation is reached.

(3) In case the "Elastic-Hardening" model is used for the material, the behaviour of the cross-sections remains elastic until the elastic limit stress in the most highly stressed fibres is reached. The strength in plastic hinges continues to increase, after the elastic limit resistance has been attained, without reaching an ultimate plastic strength. The transition to the plastic range will be more or less gradual depending on both load conditions and section shape. The analysis stops when a limit value of strength or deformation is attained. Plastic deformations are assumed to be concentrated at plastic hinge locations.

(4) In case the "Generically inelastic" model is used, both material and sections are described according to their actual stress-strain and generalized force-displacement relationships, respectively. The transition from the elastic to the plastic range is gradual and the achievement of the ultimate limit state is defined by a given limit value of strength or deformation.

C.1.6 Effect of deformations

(1) The internal actions may generally be determined using either:

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

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

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

a) non-sway structures

b) design methods which make indirect allowances for second order effects.

(3) Second order theory may be used for the global analysis in all cases except when analysis based on rigid-perfectly plastic or rigid-hardening idealizations for cross-sections is used.

C.1.7 Evaluation of generalized force-displacement relationship for cross sections

(1) The choice of the section generalized force-displacement relationship to use in the analysis should be consistent with the assumptions made for the material behaviour (see Annex D).

(2) If a perfectly plastic behaviour is assumed for the material, the following generalized force (F) - displacement (D) relationships may be assumed for the cross-sections, according to both shape and mechanical properties of the section:

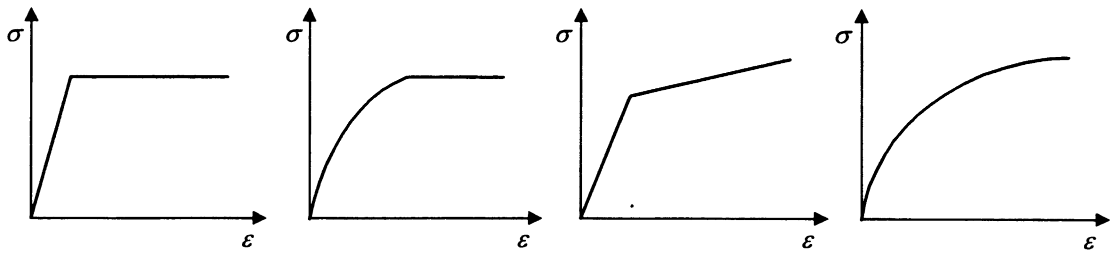
a) Elastic-Perfectly plastic (figure C.4a)

b) Inelastic-Perfectly plastic (figure C.4b)

(3) If a hardening behaviour is assumed for the material, the following generalized force-displacement relationships may be assumed for the cross-sections, according to both shape and mechanical properties of the section:

c) Elastic-Hardening (figure C.4c)

d) Generically inelastic (figure C.4d)



a) Elastic-plastic b) Inelastic-plastic c) Elastic hardening d) Generally inelastic

Figure C.4: Generalized force-displacement relationship for cross sections

C.2 Design assumptions

C.2.1 Basis

(1) The assumptions made in the global analysis of the structure should be consistent with the type of behaviour of the connections (see 6.4).

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

(3) The requirements for the various types of connections are given in 6.4.

(4) Table 6.4.1 shows the type of connection required for the different types of framing, depending on the method of global analysis used.

(5) When calculating the elastic critical load for failure of a frame in a sway mode, account should be taken of the effects of the actual rigidity of connections, irrespectively of whether elastic analysis or plastic analysis is used for the global analysis of the frame.

(6) When calculating the elastic critical loads or buckling lengths of a member, the actual initial value of connection rigidity may be accounted for.

C.2.2 Simple framing

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

(2) The connections should satisfy the requirements for nominally pinned connections (see 6.4.4.1).

C.2.3 Continuous framing

(1) Elastic analysis (linear or non linear) should be based on the assumption of full continuity, with rigid connections which satisfy the requirements given in 6.4.4.

(2) Rigid-Plastic analysis (without or with hardening) should be based on the assumption of full continuity, with full strength connections which satisfy the requirements given in 6.4.4.

(3) Elastic-Plastic analysis (without or with hardening) should be based on the assumption of full continuity, with rigid full strength connections which satisfy the requirements given in 6.4.4.

(4) Inelastic analysis (with or without plateau) should be based on the assumption of full continuity, with rigid full strength connections which satisfy the requirements given in 6.4.4.

C.2.4 Semi-continuous framing

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

(2) Rigid-Plastic analysis (without or with hardening) should be based on the design moment resistance of connections which have been demonstrated to have sufficient rotation capacity, see Annex D.

(3) Elastic-Plastic analysis (without or with hardening) should be based on the design moment-rotation or force-displacement relationship of the connections, see 6.4.4.

(4) Inelastic analysis (with or without plateau) should be based on the design moment-rotation or force-displacement relationship of the connections, see 6.4.4.

C.3 Structural system

C.3.1 Structures

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

a) Simple structural elements:

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

b) Continuous beams and non-sway frames:

Continuous beams and frames in which sway effects are negligible, or are eliminated by suitable means (see C.5), should be analyzed under appropriate arrangements of the variable loads to determine those combinations of internal actions which are critical for verifying the resistance of the individual members and of the connections.

c) Sway-frames

Sway frames (see C.5) should be analyzed under those arrangements of the variable loads which are critical for failure in a sway mode. In addition, sway frames should also be analyzed for the non sway mode as described in (b).

(2) The initial sway imperfections specified in C.4.3 - and member imperfections where necessary, see C.4.2(4) - should be included in the global analysis of all frames.

C.3.2 Sub-frames

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

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

C.3.3 Stiffness of bases

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

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

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

C.4 Allowance for imperfections

C.4.1 Basis

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

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

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

a) Global analysis

b) Analysis of bracing system

c) Member design

C.4.2 Method of application

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

(2) The effects of the frame imperfections given in C.4.3 should be included in the global analysis of the structure. The resulting forces and moments should be used for member design.

(3) The effects of the imperfections given in C.4.4 should be included in the analysis of bracing system. The resulting forces should be used for member design.

(4) The effects of member imperfections (see C.4.5) may be neglected when carrying out the global analysis of frames, except in sway frames (see C.5.2) in the case of members which are subject to axial compression, which have moment-resisting connections and in which:

$$\bar{\lambda} > 0,5 \sqrt{\frac{A_{fo}}{N_{Ed}}} \tag{C.1}$$

where:

N_{Ed} is the design value of the compressive force
$\bar{\lambda}$ is the in-plane non-dimensional slenderness (see 5.8.4) calculated using a buckling length equal to the system length.

C.4.3 Structural imperfections for global analysis

(1) The effects of imperfection should be allowed for in frame analysis by means of an equivalent geometric imperfections in the form of an initial sway imperfection Φ determined from (see figure C.5):

$$\Phi = k_c\, k_s\, \Phi_0 \tag{C.2}$$

where:

$\Phi_0 = 1/200$
$k_c = \sqrt{0,5 + 1/n_c}$ but $k_c \leq 1,0$
$k_s = \sqrt{0,5 + 1/n_s}$ but $k_s \leq 1,0$

where:

n_c is the number of columns per plane
n_s is the number of storeys.

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

(3) Columns which do not extend through all the storeys included in n_s should not be included in n_c. Those floor levels and roof levels which are not connected to all the columns included in n_c should not be included when determining n_s. Where more than one combination of n_c and n_s satisfies these conditions, any such combination can safely be used.

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

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

(6) The initial sway imperfections may be replaced by a closed system of equivalent horizontal forces, see figure C.5.

(7) In beam-and-column frames, these equivalent horizontal forces should be applied at each floor and roof level and should be proportionate to the vertical loads applied to the structure at that level, see figure C.6.

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

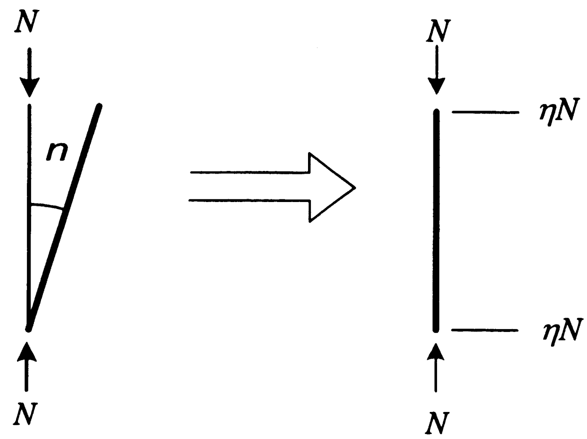


Figure C.5: Replacement of initial sway imperfections by equivalent horizontal forces

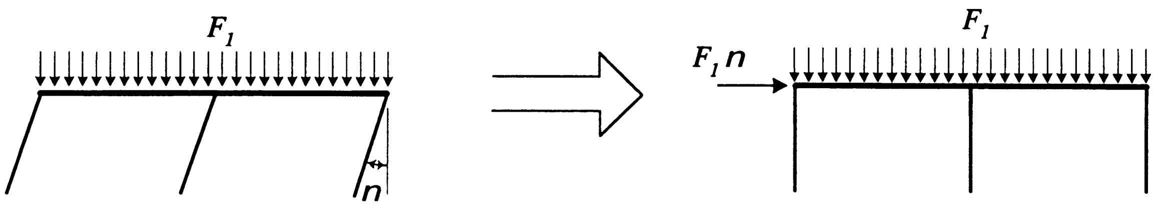


Figure C.6: Equivalent horizontal forces

C.4.4 Imperfections for analysis of bracing systems

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

$$e_0 = \frac{k_r L}{500} \tag{C.3}$$

where:

L is the span of the bracing system
$k_r = \sqrt{0,2 + 1/n_r}$ but $k_r \leq 1,0$

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

(2) For convenience, the initial bow imperfections of the members to be restrained by a bracing system, may be replaced by the equivalent stabilizing force shown in figure C.7.

(3) Where the bracing system is required to stabilize a beam, the force N in figure C.7 should be obtained from

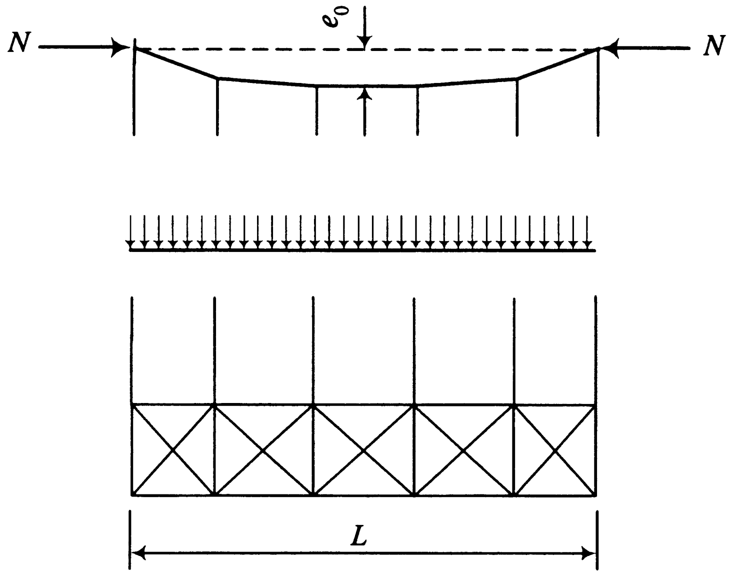
$$N = M/h \tag{C.4}$$

where:

M is the maximum moment in the beam
h is the overall depth of the beam.

(4) At points where beams or compression members are spliced, it should also be verified that the bracing system is able to resist a local force equal to $k_r N/100$ applied to it by each beam or compression member which is spliced at that point, and to transmit this force to the adjacent points at which that beam or compression member is restrained, see figure C.8.

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



e_0 = imperfection

q = equivalent stabilizing force per unit length

Bracing system

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

For a single restrained member:

$$\text{for} \quad \rho_q \le \frac{L}{2500} : \quad q = \frac{N}{50L}$$

$$\text{for} \quad \rho_q > \frac{L}{2500} : \quad q = \frac{N}{60L}(1+\sigma)$$

where:

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

σ $= 500\,\rho_q r$ but $\sigma \ge 0{,}2$

For multiple restrained members:

$$\text{for} \quad \rho_q \le \frac{L}{2500} : \quad q = \frac{\Sigma N}{60L}(k_r + 0{,}21)$$

$$\text{for} \quad \rho_q > \frac{L}{2500} : \quad q = \frac{\Sigma N}{60L}(k_r + \sigma)$$

Figure C.7: Equivalent stabilizing force

C.4.5 Member imperfections

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

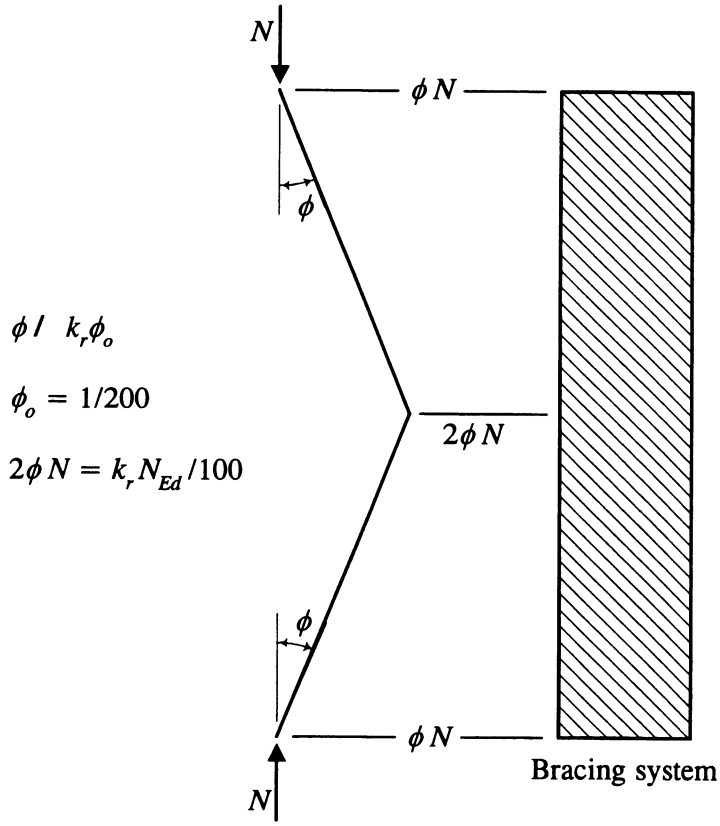


Figure C.8: Bracing forces at splices in compression elements

C.5 Sway stability

C.5.1 Sway stiffness

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

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

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

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

- triangulation
- the stiffness of the connections
- cantilever columns

(2) Semi-rigid connections may be used, provided that they can be demonstrated to provide sufficient reliable rotational stiffness to satisfy the requirements for sway-mode frame stability, see 6.4.

(3) The calculation methods for sway frames are given in Annex F.

C.5.2 Classification as sway or non-sway

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

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

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

$$\frac{V_{Ed}}{V_{cr}} \leq 0{,}1 \tag{C.5}$$

where:

V_{Ed} is the design value of the total vertical load
V_{cr} is its elastic critical value for failure in a sway mode.

(4) Beam-and-column type plane frames in building structures with beams connecting each column at each storey level (see figure C.9) may be classified as non-sway for a given load case if the following criterion is satisfied. When first order theory is used, the horizontal displacements in each storey due to the design load (both horizontal and vertical), plus the initial sway imperfection (see C.4.3) applied in the form of equivalent horizontal forces, should satisfy the criterion:

$$\frac{\delta}{h}\frac{V}{H} \leq 0{,}1 \tag{C.6}$$

where:

δ is the horizontal displacement at the top of the storey, relative to the bottom of the storey
h is the storey height
H is the total horizontal reaction at the bottom of the storey
V is the total vertical reaction at the bottom of the storey.

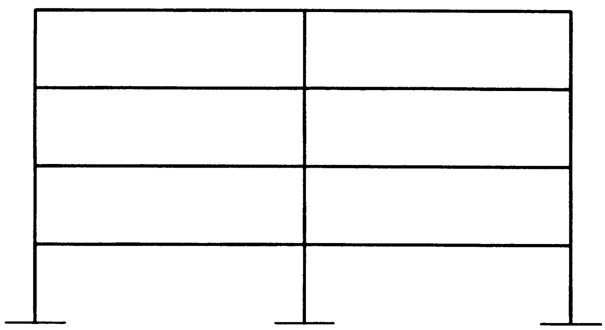


Figure C.9: Building frame with beams connecting each column at each storey level

C.5.3 Classification as braced or unbraced

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

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

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

(4) The effects of the initial sway imperfections (see C.4.3) in the braced frame should be taken into account in the design of the bracing system.

(5) The initial sway imperfections (or the equivalent horizontal forces, see C.4.3) plus any horizontal loads applied to a braced frame, may be treated as affecting only the bracing system.

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

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

(7) Where the bracing system is a frame or sub-frame, it may itself be either sway or non-sway, see C.5.2.

(8) When applying the criterion given in C.5.2(3) to a frame or sub-frame acting as a bracing system, the total vertical load acting on all the frames which it braces should also be included.

(9) When applying the criterion given in C.5.2(4) to a frame or sub-frame acting as a bracing system, the total horizontal and vertical load acting on all the frames which it braces should also be included, plus the initial sway imperfection applied in the form of the equivalent horizontal forces from the bracing system itself and from all the frames which it braces.

Annex D (informative)

Methods of global analysis

D.1 General

(1) For the general classification of calculation methods for structures, refer to section 5.2.1.

(2) Depending on whether the structural behaviour of the member is known or not, the methods of analysis are divided into:

a) Methods which operate on the structure considered as an assemblage of simple structural members (beams, columns, plates, etc.) whose individual structural behaviour is fully known.

b) Methods which require the structure to be discretized into finite elements, whose individual structural behaviour is defined by means of suitable numerical idealization.

(3) The methods of global analysis which fall into the previous point a) are (see 5.2.1):

- Linear elastic analysis

- Rigid-perfectly plastic analysis

- Elastic-perfectly plastic analysis

- Rigid-hardening analysis

- Elastic-hardening analysis

(4) All the methods of analysis previously listed are based on the assemblage of simple elements for which it is possible to express the nodal stiffness or deformability through closed-form relationships.

(5) Possible plastic strains are considered as concentrated in single sections (end sections, loaded sections, changes of cross sections, etc.) in the form of plastic hinge. Within two of these sections, the behaviour remain perfectly elastic. For this reason, all the methods of analysis previously listed, except for the linear elastic analysis, are referred to in the following as "plastic hinge methods". For more details on the application of such methods see section D.3.

(6) The methods of global analysis which fall into the previous point b) are (see 5.2.1):

- Non linear elastic analysis

- Inelastic-perfectly plastic analysis

- Generically inelastic

(7) These methods allow for the actual inelastic behaviour of the structure to be taken into account, with a degree of accuracy which increases as far as the degree of discretization increases. In particular, it may be assumed that the generically inelastic approach provides a reliable representation of the structural behaviour with regard to the evaluation of both load bearing capacity and ductility demand (see section D.2).

(8) Whatever the method of analysis used, the assumptions on the generalized force-displacement relationship for the cross-sections shall be consistent with the assumption on the stress-strain law of the material. The possible combinations are shown in table D.1, for sections subjected to axial load and bending:

Table D.1: Relationship between material and section behavioural models

Material law (see section 5.2.1)	Generalized force-displacement relationship	
	Sections in axial load (N-ε)	Sections in bending (M-χ)
Linear elastic	Linear elastic	Linear elastic
Non linear elastic	Non linear elastic	Non linear elastic
Rigid-plastic	Rigid-plastic	Rigid-plastic
Rigid-hardening	Rigid-hardening	Rigid-hardening
Elastic-plastic	Elastic-plastic	Inelastic-plastic Elastic-plastic (for $\alpha_0 < 1{,}2$)
Elastic-hardening	Elastic-hardening	Generically inelastic Elastic-hardening (for $\alpha_0 < 1{,}2$)
Inelastic-plastic	Inelastic-plastic	Inelastic-plastic
Generically inelastic	Generically inelastic	Generically inelastic

D.2 Assessment of ductility demand

(1) Rules supplied in this section apply only to structures made of class 1 cross-sections (see Annex H), i.e. for structures for which collapse occurs due to the attainment of ultimate deformation in a certain number of sections. Structures with cross-sections belonging to class 2, 3, or 4 are not covered by the following rules, unless specific account of the effect of local buckling phenomena is taken for the evaluation of both load bearing capacity and available ductility.

(2) The evaluation of the ductility demand of a structural scheme under the design actions can be either:

a) Rigorous

b) Conventional

depending on whether a system of displacements or forces is applied to the structure.

(3) If a system of displacement is applied to the structure, regardless of its strength capability, the ductility demand may be rigorously evaluated as the maximum value of a deformation parameter which the structure must be able to reach under a given load process in which a generical displacement parameter is assumed as independent variable.

(4) If the structure is loaded through the application of a forces system increasing up to collapse, the ductility demand would be nominally infinite. Therefore, a conventional definition must be set; for a generical truss-or beam-made structure, this can be done according to the following criteria (see figure D.1):

a) The ductility demand is defined as the required rotation in the most developed plastic hinge when the collapse mechanism is attained. The structure is assumed to exhibit concentrated plasticity and is solved by means of one of the plastic hinge methods listed in the previous section. If a convenient length for the plastic hinge is assumed, then the maximum required strain can be evaluated.

b) The ductility demand is defined as the rotation required in the most developed plastic hinge when the plastic hinge idealization provides the same load bearing capacity as predicted by a more refined generically inelastic method of analysis operating on a discretized model. The structure is assumed to exhibit concentrated plasticity and is solved with one of the plastic hinge methods listed in the previous section.

c) The ductility demand is defined a priori as a function of the maximum elastic strain of the alloy. The corresponding load bearing capacity can be evaluated through the plastic hinge analysis by using a modified value of the conventional yield stress in order to take into account the actual behaviour of the alloy in terms of ductility and hardening (see section D.3).

(5) Ductility requirements mentioned in (4) shall comply with the alloy deformation features shown in Annex A.

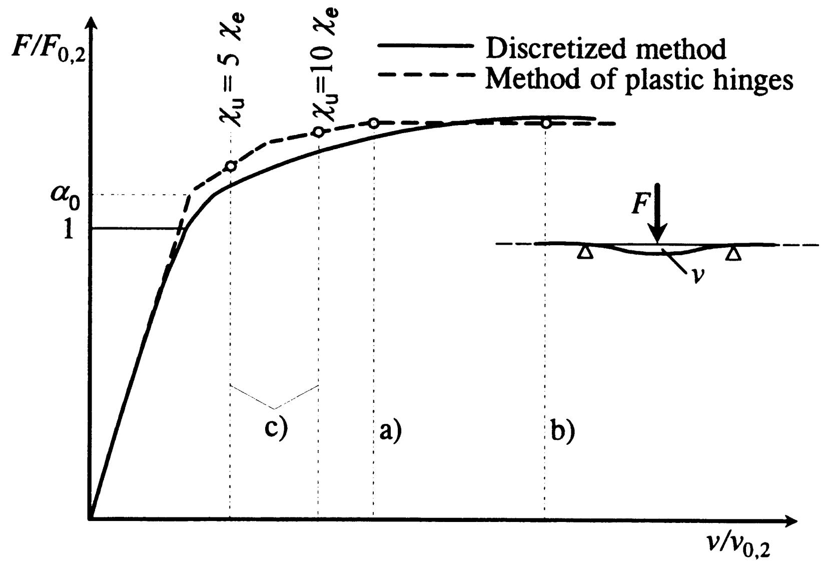


Figure D.1: Conventional evaluation of ductility demand

D.3 Application of plastic hinge methods

(1) Plastic hinge methods may be applied provided that the structural ductility is sufficient to enable the development of the full plastic mechanism.

(2) As a general rule, when the plastic hinge method is applied by considering the elastic-perfectly plastic behaviour of material, the ductility demand of the structural scheme shall be evaluated according to one of the criteria supplied in D.2(4) a) and b), whichever is the most severe.

(3) When the application of the above criteria leads to ductility values which are incompatible with the alloy deformation capability, as well as when the alloy hardening behaviour has to be taken into account, then plastic hinge methods may be applied by assuming the definition of ductility demand of point (4) c) of section D.2. In this case the value of the conventional yield stress f_y to be used in the analysis shall be corrected. In general, f_y is put in the form:

$$f_y = \eta f_{0,2} \quad \text{if} \quad \eta f_{0,2} \leq f_t/\gamma_M$$

$$f_y \leq f_t/\gamma_M \ \text{if} \quad \eta f_{0,2} > f_t/\gamma_M$$

where:

η is a numerical parameter depending on the geometrical shape factor α_0 as well as on the conventional available ductility of the material

γ_M is the material partial safety factor.

(4) If elastic-or-rigid-perfectly plastic behaviour is assumed for the material, the plastic hinge method shall be applied by assuming for the generic section an ultimate moment given by

$$M_u = \alpha_0 f_y W = \alpha_0 \eta f_{0,2} W$$

η being the previously defined correction factor.

(5) If elastic-or-rigid-hardening behaviour is assumed for the material, the plastic hinge method shall be applied by assuming for the generic section a conventional yield moment, corresponding to the beginning of strain-hardening, given by

$$M_y = \alpha_0 f_y W = \alpha_0 \eta f_{0,2} W$$

η being the correction factor previously defined.

The ultimate moment shall be calculated via the expression

$$M_u = \alpha_\xi f_y W = \alpha_\xi \eta f_{0,2} W$$

ξ being equal to 5 or 10 depending on the ductility feature of the alloy (for the definition of α_5 and α_{10} refer to Annex H).

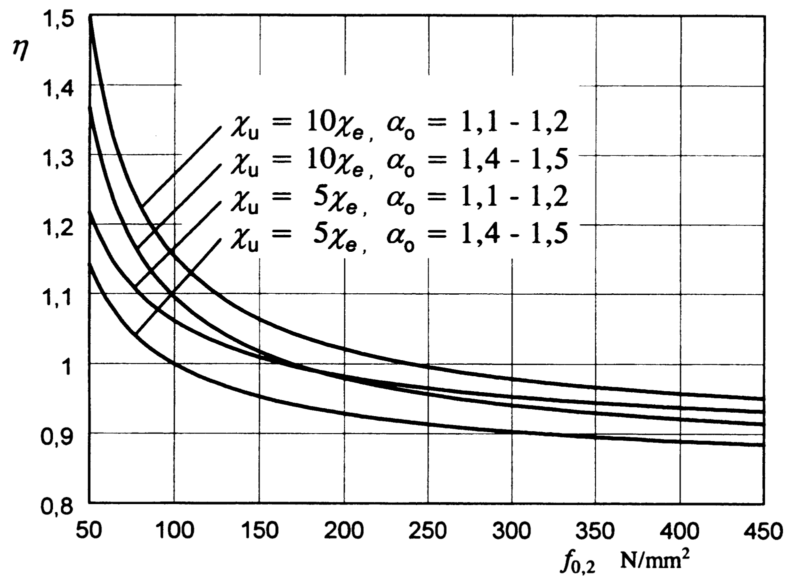


Figure D.2: Value of the correction coefficient η

(6) The correction coefficient η is fitted in such a way that the plastic hinge analysis provides the actual load bearing capacity of the structure, according to the available ductility of the alloy. In general, η is expressed by

$$\eta = \frac{1}{a + b f_{0.2}^{c}} \quad (f_{0,2} \text{ in N/mm}^2)$$

For structures made of beams in bending, the coefficients a, b and c of the previous equation are provided in table D.2.

(7) The conventional ductility demand (see D.2(4) c) of section D.2) may be defined through the conventional ultimate curvature χ_u = 5 or 10 χ_{el} (see Annex H). The conventional ultimate curvature shall be chosen according to the ductility properties of the alloy. This may be done according to tab. D.3 (see also figure D.3).

Table D.2: Values of coefficients *a*, *b* and *c*

Coefficients of the law $\eta = 1/[a+b\, f_{0.2}^{c}]$	(α_0 = 1,4 - 1,5)		(α_0 = 1,1 - 1,2)	
	$\chi_u = 5\ \chi_e$	$\chi_u = 10\ \chi_e$	$\chi_u = 5\ \chi_e$	$\chi_u = 10\ \chi_e$
a	1,2	1,18	1,15	1,13
b	-5	-8,4	-4,4	-11
c	-0,7	-0,75	-0,66	-0,81

(8) From the ductility point of view, two groups of alloys are defined, depending on whether the above-mentioned conventional curvature limits are reached or not:

- Brittle alloys,
if the ultimate tensile deformation is sufficient to develop an ultimate bending curvature equal to $\chi_u = 5\ \chi_e$.

- Ductile alloys,
if the ultimate tensile deformation is sufficient to develop an ultimate bending curvature equal or higher than $\chi_u = 10\ \chi_e$.

The deformation values corresponding to χ_u equal to 5 χ_e and 10 χ_e are shown in table D.3 as a function of the conventional yield stress $f_{0,2}$. If intermediate values of the ultimate curvature are assumed, linear interpolation applies.

(9) The global safety factor evaluated through plastic hinge methods applied with $\eta < 1$ shall be not higher than that evaluated through a linear elastic analysis. If this occurs the results of elastic analysis must be considered.

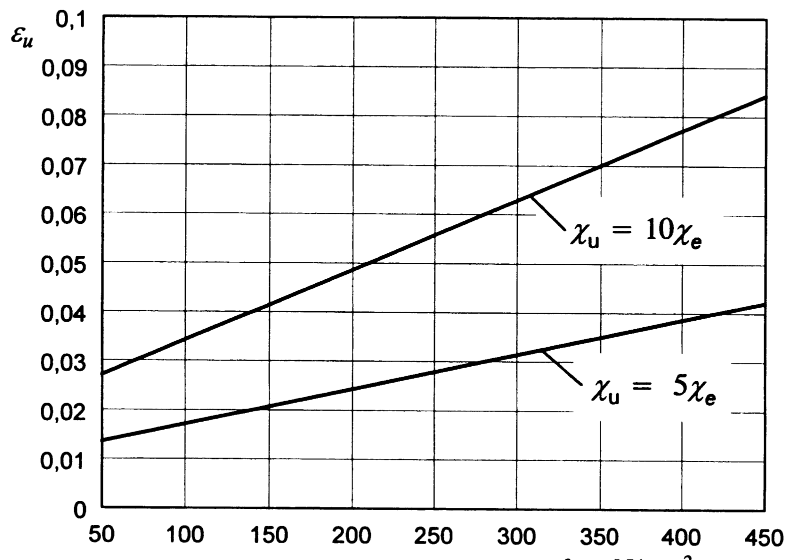


Figure D.3: Value of ultimate strain ϵ_u

Table D.2: Values of the correction coefficient η

$f_{0,2}$ (N/mm^2)	$\chi_u = 5\chi_e$ $\alpha_0 = 1{,}4 - 1{,}5$	$\chi_u = 10\chi_e$ $\alpha_0 = 1{,}4 - 1{,}5$	$\chi_u = 5\chi_e$ $\alpha_0 = 1{,}1 - 1{,}2$	$\chi_u = 10\chi_e$ $\alpha_0 = 1{,}1 - 1{,}2$
50	1,14172	1,36725	1,21714	1,49615
60	1,09325	1,26786	1,16437	1,36657
70	1,05945	1,20272	1,12740	1,28451
80	1,03440	1,15652	1,09989	1,22769
90	1,01502	1,12193	1,07852	1,18590
100	0,99953	1,09498	1,06139	1,15380
110	0,98682	1,07335	1,04730	1,12833
120	0,97620	1,05558	1,03549	1,10760
130	0,96717	1,04069	1,02542	1,09038
140	0,95938	1,02801	1,01672	1,07583
150	0,95258	1,01709	1,00912	1,06337
160	0,94660	1,00756	1,00241	1,05256
170	0,94128	0,99917	0,99644	1,04309
180	0,93653	0,99172	0,99109	1,03473
190	0,93224	0,98505	0,98626	1,02728
200	0,92835	0,97905	0,98187	1,02060
210	0,92481	0,97362	0,97787	1,01457
220	0,92157	0,96868	0,97420	1,00911
230	0,91859	0,96415	0,97082	1,00412
240	0,91584	0,96000	0,96770	0,99956
250	0,91329	0,95617	0,96481	0,99537
260	0,91093	0,95263	0,96211	0,99150
270	0,90872	0,94934	0,95960	0,98792
280	0,90666	0,94628	0,95725	0,98459
290	0,90473	0,94342	0,95505	0,98150
300	0,90291	0,94075	0,95297	0,97861
310	0,90121	0,93824	0,95102	0,97590
320	0,89959	0,93588	0,94918	0,97336
330	0,89807	0,93366	0,94743	0,97097
340	0,89663	0,93156	0,94578	0,96872
350	0,89527	0,92958	0,94421	0,96660
360	0,89397	0,92770	0,94272	0,96459
370	0,89274	0,92591	0,94130	0,96269
380	0,89156	0,92422	0,93995	0,96089
390	0,89044	0,92261	0,93866	0,95917
400	0,88937	0,92107	0,93742	0,95754
410	0,88834	0,91961	0,93624	0,95599
420	0,88736	0,91821	0,93511	0,95451
430	0,88643	0,91687	0,93403	0,95309
440	0,88553	0,91558	0,93298	0,95174
450	0,88466	0,91436	0,93198	0,95045

Table D.3: Values of ultimate strain ε_u

$f_{0,2}$ (N/mm^2)	$\chi_u = 5\chi_e$ $\alpha_0 = 1,4 - 1,5$	$\chi_u = 10\chi_e$ $\alpha_0 = 1,4 - 1,5$
50	0,01357	0,02714
60	0,01429	0,02857
70	0,01500	0,03000
80	0,01571	0,03143
90	0,01643	0,03286
100	0,01714	0,03429
110	0,01786	0,03571
120	0,01857	0,03714
130	0,01929	0,03857
140	0,02000	0,04000
150	0,02071	0,04143
160	0,02143	0,04286
170	0,02214	0,04429
180	0,02286	0,04571
190	0,02357	0,04714
200	0,02429	0,04857
210	0,02500	0,05000
220	0,02571	0,05143
230	0,02643	0,05286
240	0,02714	0,05429
250	0,02786	0,05571
260	0,02857	0,05714
270	0,02929	0,05857
280	0,03000	0,06000
290	0,03071	0,06143
300	0,03143	0,06286
310	0,03214	0,06429
320	0,03286	0,06571
330	0,03357	0,06714
340	0,03429	0,06857
350	0,03500	0,07000
360	0,03571	0,07143
370	0,03643	0,07286
380	0,03714	0,07429
390	0,03786	0,07571
400	0,03857	0,07714
410	0,03929	0,07857
420	0,04000	0,08000
430	0,04071	0,08143
440	0,04143	0,08286
450	0,04214	0,08429

Annex E (informative)

Analytical models for stress strain relationship

E.1 Scope

(1) This Annex provides the models for the idealization of the stress-strain relationship of aluminium alloys. These models are conceived in order to account for the actual elastic-hardening behaviour of such materials.

(2) The proposed models have different levels of complexity according to the accuracy required for calculation.

E.2 Analytical models

(1) The analytical characterization of the stress (σ) - strain (ε) relationship of an aluminium alloy can be done by means of one of the following models:

- Piecewise models

- Continuous models

(2) The numerical parameters which define each model have to be calibrated on the basis of the actual mechanical properties of the material. These should be obtained through appropriate tensile test or, as an alternative, on the bases of the nominal values given, for each alloy, in Section 3.

E.2.1 Piecewise models

(1) These models are based on the assumption that material σ-ε law is described by means of a multi linear curve, each branch of it representing the elastic, inelastic and plastic, with or without hardening, region respectively.

(2) According to this assumption, the characterization of the stress-strain relationship may generally be performed using either:

- bi-linear model with and without hardening (figure E.1)

- three-linear model with and without hardening (figure E.2)

E.2.1.1 Bi-linear model

(1) When a bi-linear model with hardening is used (figure E.1a), the following relationships may be assumed:

$$\sigma = E\varepsilon \qquad \text{for} \qquad 0 < \varepsilon \leq \varepsilon_p$$

$$\sigma = f_p + E_1 (\varepsilon - \varepsilon_p) \quad \text{for} \quad \varepsilon_p < \varepsilon \leq \varepsilon_{max}$$

where:

f_p = conventional elastic limit of proportionality
ε_p = strain corresponding to the stress f_p
ε_{max} = strain corresponding to the stress f_{max}
E = elastic modulus
E_1 = hardening modulus

(2) In case the "Elastic-Perfectly plastic" model is assumed (figure E.1b), the material remains perfectly

elastic until the elastic limit stress f_p. Plastic deformations without hardening (E_1 = 0) should be considered up to ε_{max}.

(3) In the absence of more accurate evaluation of the above parameters the following values may be assumed for both models of figures E.1a) and b):

f_p = nominal value of $f_{0,2}$ (see Section 3)
f_{max} = nominal value of f_u (see Section 3)
ε_{max} = 0,5 ε_u
ε_u = nominal value of ultimate strain (see Section 3)
ε_p = $f_{0,2}/E$
E_1 = $(f_u - f_{0,2})/(0,5\ \varepsilon_u - \varepsilon_p)$

E.2.1.2 Three-linear model

(1) When three-linear model with hardening is used (figure E.2a), the following relationships may be assumed:

$$\sigma = E\varepsilon \qquad \text{for} \quad 0 < \varepsilon \leq \varepsilon_p$$

$$\sigma = f_p + E_1 (\varepsilon - \varepsilon_p) \quad \text{for} \quad \varepsilon_p < \varepsilon \leq \varepsilon_e$$

$$\sigma = f_e + E_2 (\varepsilon - \varepsilon_e) \quad \text{for} \quad \varepsilon_e < \varepsilon \leq \varepsilon_{max}$$

where:

f_p = limit of proportionality
f_e = limit of elasticity
ε_p = strain corresponding to the stress f_p
ε_e = strain corresponding to the stress f_e
ε_{max} = strain corresponding to the stress f_{max}
E = elastic modulus
E_1 = first hardening modulus
E_2 = second hardening moduls

(2) In case the "Perfectly plastic" model is assumed (figure E.2b), plastic deformations without hardening (E_2 = 0) should be considered for strain ranges from ε_e to ε_{max}.

(3) In the absence of more accurate evalutation of the above parameters the following values may be assumed for both models of figures E.2a) and b):

f_e = reduced limit of elasticity (table E.1)
f_p = μf_e
f_{max} = nominal value of f_u (see Section 3)
ε_{max} = 0,5 ε_u
ε_u = nominal value of ultimate strain (see Section 3)
E = reduced elastic modulus E_r (table E.1)
E_1 = E_r/m
E_2 = $(f_{max} - f_e) / (\varepsilon_{max} - \varepsilon_e)$
ε_p = f_p/E_r
ε_e = $\varepsilon_p + (f_e - f_p)/E_1$

with f_e, E_r, μ and m given in table E.1.

Table E.1: Value of f_e, E_r, μ and m in the three-linear models

Aluminium Alloys	f_e N/mm^2	E_r N/mm^2	μ	m
A1ZnMg 1 F36	290	68000	0,85	4,0
A1MgSi 1 F32	270	68000	0,85	4,0
A1MgSi 1 F28	210	65000	0,80	4,0
A1MgSi 0,5 F22	170	65000	0,85	4,5
EN AW 5083	230	65000	0,80	5,0
A1Mg4,5 Mn w/F28 tubes + profiles	150	65000	0,85	5,0
A1MgMnF23 EN AW 5454	170	65000	0,85	4,5
A1MgMnF20	110	60000	0,80	5,0
A1MgMn w/F18 EN AW 5754	80	55000	0,75	5,0

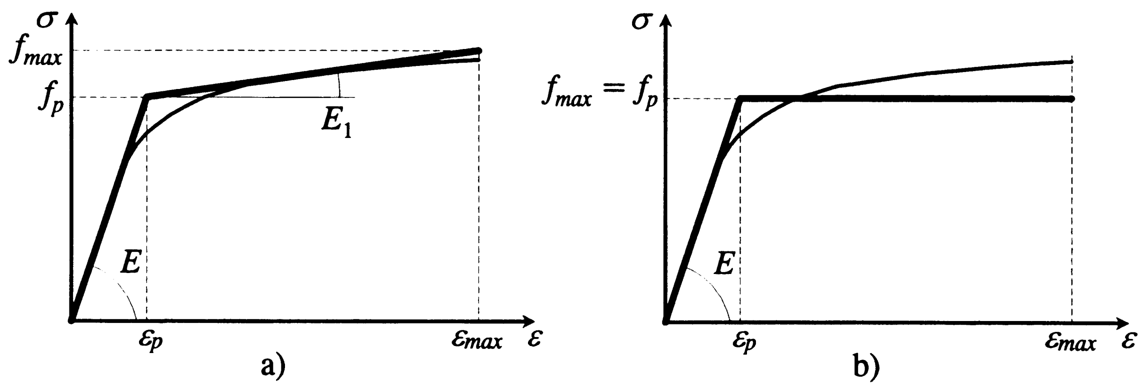


Figure E.1: Bi-linear models

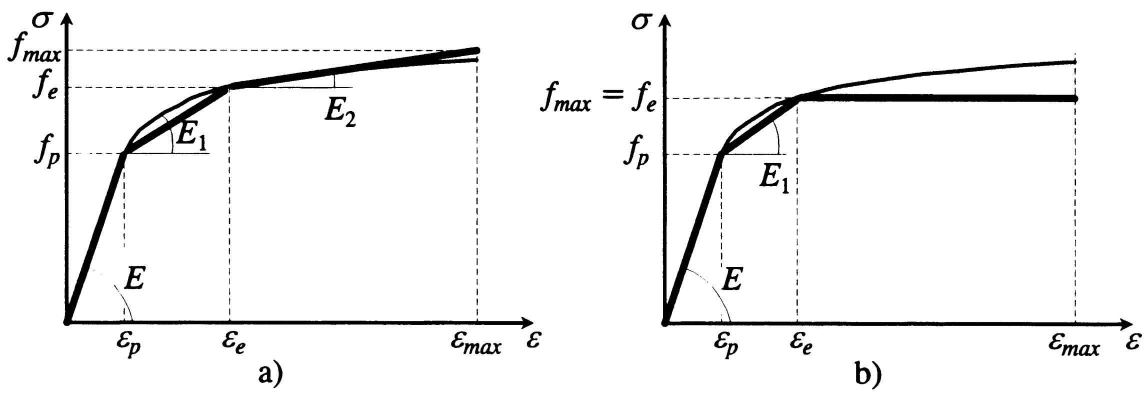


Figure E.2:Three-linear models

E.2.2 Continuous models

(1) These models are based on the assumption that the material σ-ε law is described by means of a continuous relationship representing the elastic, inelastic and plastic, with or without hardening, region respectively.

(2) According to this assumption, the characterization of the stress-strain relationship may generally be performed using either:

- Continuous models in the form $\sigma = \sigma(\varepsilon)$

- Continuous models in the form $\varepsilon = \varepsilon(\sigma)$

E.2.2.1 Continuous models in the form $\sigma = \sigma(\varepsilon)$

(1) When a $\sigma = \sigma(\varepsilon)$ law is assumed, it is convenient to identify three separate regions which can be defined in the following way (see figure E.3a):

- Region 1 elastic behaviour

- Region 2 inelastic behaviour

- Region 3 strain-hardening behaviour

(2) In each region the behaviour of the material is represented by means of different stress versus strain relationships which have to ensure continuity at their limit points. According to this assumption, the characterization of the stress-strain relationship may be expressed as follows (figures E.3b):

Region 1 for $0 < \varepsilon \le \varepsilon_p$ with $\varepsilon_p = 0{,}5\,\bar{\varepsilon}_l$ and $\bar{\varepsilon}_l = f_l/E$

$$\sigma = E\ \varepsilon$$

Region 2 for $\varepsilon_p < \varepsilon \le \varepsilon_l$ with $\varepsilon_l = 1{,}5\ \bar{\varepsilon}_l$ and $\bar{\varepsilon}_l = f_e/E$

$$\sigma = f_e\left[-0{,}2 + 1{,}85\frac{\varepsilon}{\varepsilon_l} - \left(\frac{\varepsilon}{\varepsilon_l}\right)^2 + 0{,}2\left(\frac{\varepsilon}{\varepsilon_l}\right)^3\right]$$

Region 3 for $\varepsilon_l < \varepsilon \le \varepsilon_{max}$

$$\sigma = f_e\left[\frac{f_{\max}}{f_e} - 1{,}5\left(\frac{f_{\max}}{f_e} - 1\right)\frac{\varepsilon_e}{\varepsilon}\right]$$

where:

f_e = conventional elastic limit
f_{max} = tensile strength at the top point of σ - ε curve
ε_e = strain corresponding to the stress f_e
ε_{max} = strain corresponding to the stress f_{max}
E = elastic modulus

(3) In the absence of more accurate evaluation of the above parameters the following values may be assumed:

f_e = nominal value of $f_{0,2}$ (see Section 3)

f_{max} = nominal value of f_u (see Section 3)
ε_{max} = 0,5 ε_u
ε_u = nominal value of ultimate strain (see Section 3)
E = nominal value of elastic modulus (see Section 3)

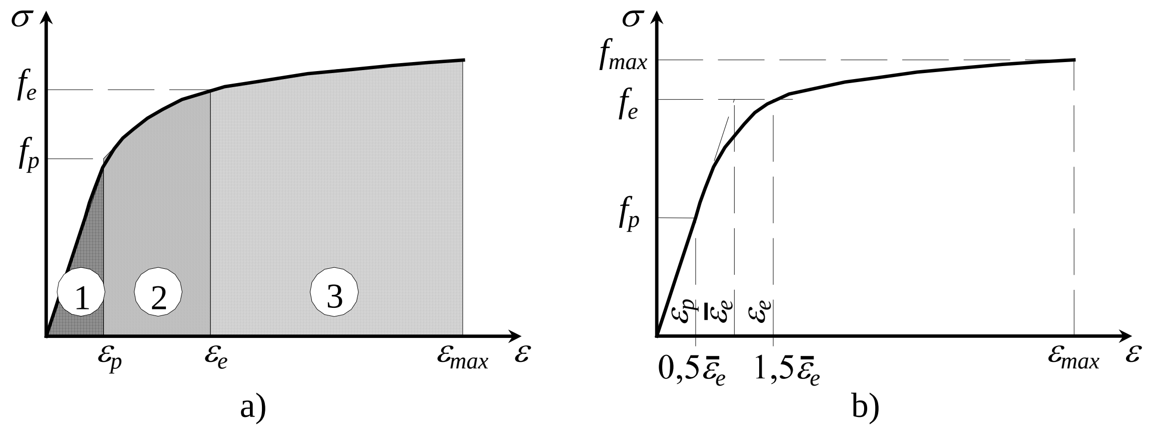


Figure E.3: Continuous models in the form $\sigma = \sigma(\varepsilon)$

E.2.2.2 Continuous models in the form $\varepsilon = \varepsilon(\sigma)$

(1) For materials of round-house type, as aluminium alloys, the Ramberg-Osgood model may be applied to describe the stress versus strain relationship in the form $\varepsilon = \varepsilon(\sigma)$. Such model may be given in a general form as follows (see figure E.4a):

$$\varepsilon = \frac{\sigma}{E} + \varepsilon_{o,e}\left(\frac{\sigma}{f_e}\right)^n$$

where:

f_e = conventional elastic limit
$\varepsilon_{o,e}$ = residual strain corresponding to the stress f_e
n = exponent characterizing the degree of hardening of the curve

(2) In order to evaluate the n exponent, the choice of a second reference stress f_x, in addition to the conventional limit of elasticity f_e, is required. Assuming (figure E.4b):

f_x = second reference stress
$\varepsilon_{o,x}$ = residual strain corresponding to the stress f_x

the exponent n is expressed by:

$$n = \frac{\log\,(\varepsilon_{o,e}/\varepsilon_{o,x})}{\log\,(f_e/f_x)}$$

(3) As conventional elastic limit, the proof stress evaluated by means of 0,2% offset method may be assumed, i.e.:

$f_e = f_{0,2}$
$\varepsilon_{o,e} = 0{,}002$

and the model equation become:

$$\varepsilon = \frac{\sigma}{E} + 0{,}002\left(\frac{\sigma}{f_{0,2}}\right)^{n} \quad \text{and} \quad n = \frac{\log\,(0{,}002/\varepsilon_{o,x})}{\log\,(f_{0,2}/f_x)}$$

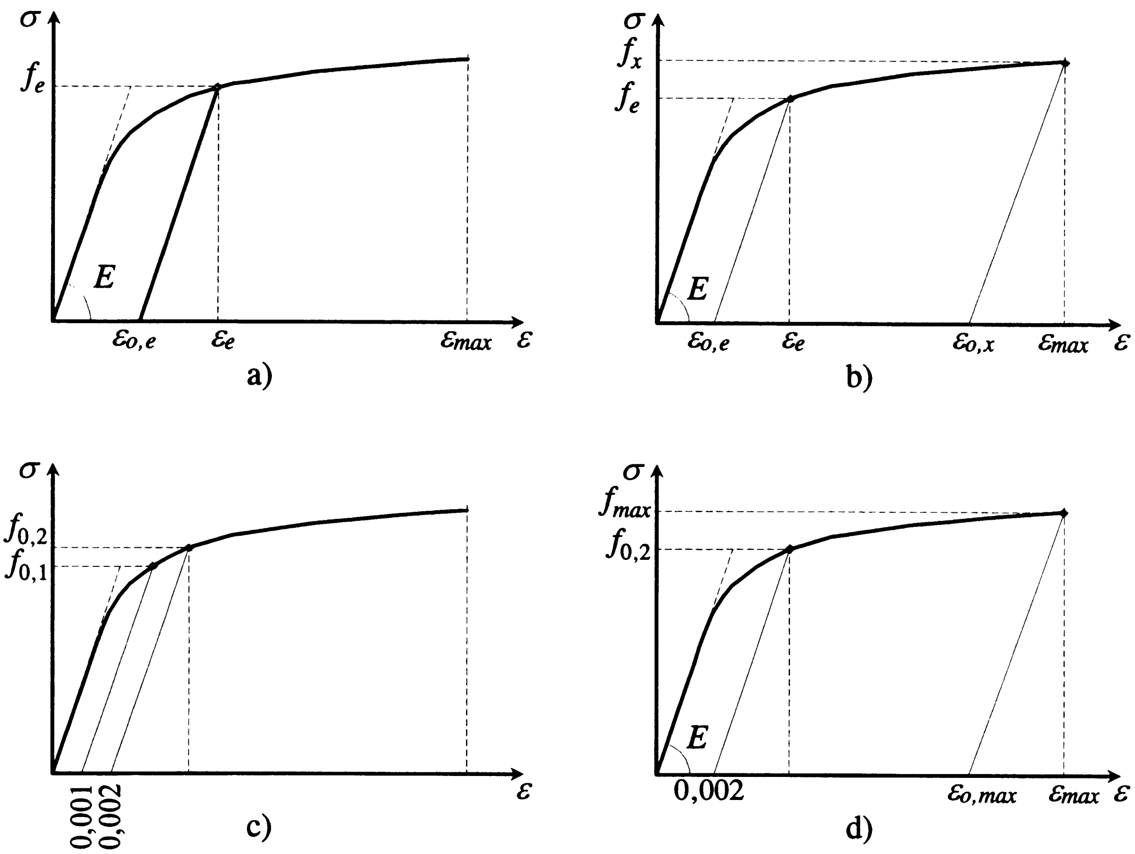


Figure E.4: Continuous models in the form $\varepsilon = \varepsilon\,(\sigma)$

(4) The choice of the second reference point (f_x - $\varepsilon_{o,x}$) should be based on the strain range corresponding to the phenomenon under investigation. The following limit cases may be considered:

a) if the analysis concerns the range of elastic deformations, the proof stress evaluated by means of 0,1 % offset method may be assumed as the second reference point (see figure E.4c), giving:

$$f_x = f_{0,1}$$
$$\varepsilon_{o,x} = 0{,}001$$

and, therefore,

$$n = \frac{\log 2}{\log f_{0,2}/f_{0,1}}$$

b) if the analysis concerns the range of plastic deformations, the tensile stress at the top point of the σ-ε curve may be assumed as the second reference point (see figure E.4d), giving:

$$f_x = f_{max}$$
$\varepsilon_{o,x} = \varepsilon_{o,max}$ = residual strain corresponding to the stress f_{max}

and, therefore,

$$n = \frac{\log\,(0{,}002/\varepsilon_{o,max})}{\log\,(f_{0,2}/f_{max})}$$

(5) In the absence of more accurate evalutation of the above parameters, the following values may be assumed:

a) elastic range ($f_x = f_{o,1}$)

$$n = \frac{\log 2}{\log (1 + k\chi)}$$

with:

$k = 0{,}28$ (mm²/N)

$$\chi = \frac{(f_u - f_{0,2})}{10\,\varepsilon_u}\,\frac{F_u}{f_{0,2}} \quad (\text{N/mm}^2)$$

$f_{0,2}$ = nominal value of proof stress (see Section 3)
f_u = nominal value of ultimate strength (see Section 3)
ε_u = nominal value of ultimate strain (see Section 3)
E = nominal value of elastic modulus (see Section 3)

b) plastic range ($f_x = f_{max}$)

$$n = \frac{\log (0{,}002/\varepsilon_{o,max})}{\log (f_{0,2}/f_{\max})}$$

with:

$f_{0,2}$ = nominal value of proof stress (see Section 3)
f_{max} = nominal value of f_u (see Section 3)
$\varepsilon_{o,max}$ = $0{,}5\varepsilon_u - f_u/E$
ε_t = nominal value of ultimate strain (see Section 3)
E = nominal value of elastic modulus (see Section 3)

Annex F (informative)

Frame stability

F.1 General

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

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

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

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

(5) The use of plastic global analysis with plastic hinge locations in the columns shall be limited to cases where it can be demonstrated that the columns are able to form hinges with sufficient rotation capacity, see F.4.

F.2 Elastic analysis of sway frames

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

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

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

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

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

$$\frac{1}{1 - V_{Ed}/V_{er}}$$

where:

V_{Ed} is the design value of the total vertical load
V_{cr} is its elastic critical value for failure in a sway mode.

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

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

(6) As an alternative for determining V_{Ed}/V_{cr} directly, the following approximation may be used in beam-and-column type frames as described in 5.2.5.2(4):

$$\frac{V_{Ed}}{V_{cr}} = \frac{\delta}{h}\frac{V}{H}$$

where:

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

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

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

F.3 Plastic analysis of sway frames

(1) When plastic global analysis (without or with hardening) is used, allowance shall be made for the second order effects in the sway mode.

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

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

a) Frames of one or two storeys
- no plastic hinge locations occur in the columns, or
- the columns satisfy the requirements given in F.4.

b) Frames with fixed bases, in which the sway failure mode involves plastic hinge locations in the columns at the fixed bases only and the design is based on a mechanism in which the columns are designed to remain elastic.

(4) In the cases given in (3), V_{Ed}/V_{cr} should not exceed 0,20 and all the internal actions should be amplified by the ratio given in F.2 (3).

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

F.4 Column requirements for plastic analysis (without or with hardening)

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

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

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

- in braced frames:

$$\bar{\lambda} \leq 0{,}40\sqrt{\frac{Af_{0,2}}{N_{Ed}}}$$

- in unbraced frames:

$$\bar{\lambda} \leq 0{,}32 \sqrt{\frac{Af_{0,2}}{N_{Ed}}}$$

where

$\bar{\lambda}$ is the in-plane non-dimensional slenderness calculated using buckling length equal to the system length.

(4) In frames designed using first order rigid-plastic global analysis, columns containing plastic hinge locations should also be checked for resistance to in-plane buckling, using buckling lengths equal to their system lengths.

(5) Except for the method outlined in F.3 (3) b), first order rigid-plastic global analysis should not be used for unbraced frames with more than two storeys.

Annex G (informative)

Behaviour of cross-sections beyond elastic limit

G.1 General

(1) This Annex provides the specifications for estimating the post-elastic behaviour of cross-sections according to the mechanical properties of material and the geometrical features of section.

(2) The actual behaviour of cross-sections beyond elastic limit shall be considered in whichever type of inelastic analysis, including the simple elastic analysis when redistributions of internal actions are allowed for (see Section 5.2.1). In addition, suitable limitation to the elastic strength shall be considered also in elastic analysis when slender sections are used.

(3) The choice of the generalized force-displacement relationship for the cross-sections should be consistent with the assumptions on the material law and with the geometrical features of the section itself (see point G.5).

(4) The reliability of the assumptions on behaviour of cross-sections can be checked on the basis of direct experimentation.

G.2 Definition of cross-section limit states

(1) The behaviour of cross-sections and the corresponding idealization to be used in structural analysis shall be related to the capability to reach the limit states listed below, each of them corresponding to a particular assumption on the state of stress acting on the section.

(2) Referring to the global behaviour of a cross-section, regardless of the internal action considered (axial load, bending moment or shear), the following limit states can be defined:

- elastic buckling limit state
- elastic limit state
- plastic limit state
- collapse limit state

(3) Elastic buckling limit state is related to the strength corresponding to the onset of local elastic instability phenomena in the compressed parts of the section.

(4) Elastic limit state is related to the strength corresponding to the attainment of the conventional elastic limit $f_{0,2}$ of material in the most stressed parts of the section.

(5) Plastic limit state is related to the strength of the section, evaluated by assuming a perfectly plastic behaviour for material with a limit value equal to the conventional elastic limit $f_{0,2}$, without considering the effect of hardening.

(6) Collapse limit state is related to the actual ultimate strength of the section, evaluated by assuming a distribution of internal stresses accounting for the actual hardening behaviour of material. Since, under this hypothesis, the generalized force-displacement curve is generally increasing, the collapse strength must be referred to a given limit of the generalized displacement (see point G.5).

G.3 Classification of cross-sections according to limit states

(1) Cross-sections can be classified according to their capability to reach the above defined limit states. Such a classification is complementary to that presented at point 5.3.2 and can be adopted when the section capabilities to get into plastic range must be specified. In such a sense, referring to a generalized force F versus displacement D relationship, cross-sections can be divided as follows (see figure G.1):

- Ductile sections (Class 1)
- Compact sections (Class 2)
- Semi-compact sections (Class 3)
- Slender sections (Class 4)

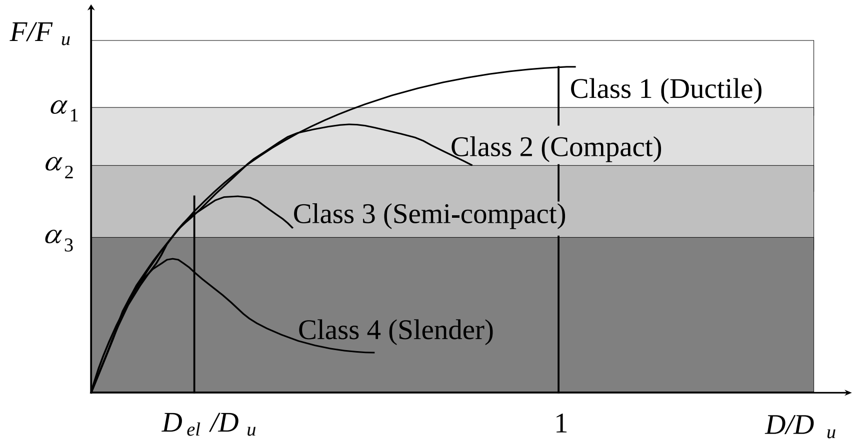


Figure G.1: Classification of cross-sections

(2) Ductile sections (Class 1) develop the collapse resistance as defined in clause (6) of section G.2 without any problem of local instability. The full exploitation of the hardening properties of material is allowed until the ultimate value of deformation, depending on the type of alloy, is reached.

(3) Compact sections (Class 2) are capable of developing the plastic limit resistance as defined in clause (5) of section G.2. The full exploitation of the hardening properties of material is prevented by the onset of plastic instability phenomena.

(4) Semi-compact sections (Class 3) are capable of developing the elastic limit resistance only, as defined in clause (4) of section G.2, without getting into inelastic range owing to instability phenomena. Only small plastic deformations are allowed within the section, whose behaviour remains substantially brittle.

(5) Both serviceability and ultimate behaviour of slender sections (Class 4) are governed by the occurring of local buckling phenomena, which cause the ultimate strength of the cross-section to be determined by the elastic buckling limit state, as defined in clause (3) of section G.2. No plastic deformations are allowed within the section, whose behaviour is remarkably brittle.

G.4 Evaluation of ultimate axial load

(1) The load-bearing capacity of cross-sections under axial load can be evaluated with reference to the above mentioned limit states, by means of the following practical rules.

(2) The value of axial load for a given limit state can be expressed by the generalized formula:

$$N = \alpha_{N,j} A f_d$$

where:

f_d the design value of strength
A the net cross sectional area
$\alpha_{N,j}$ a correction factor, given in table G.1, depending on the assumed limit state.

Table G.1: Ultimate Axial Load

Axial load	Limit State	Section class	Correction factor
N_u	Collapse	Class 1	$\alpha_{N,1} = f_t/f_d$
N_{pl}	Plastic	Class 2	$\alpha_{N,2} = 1$
N_{el}	Elastic	Class 3	$\alpha_{N,3} = 1$
N_{red}	Elastic buckling	Class 4	$\alpha_{N,4} = A_{eff}/A$

where A_{eff} is the effective cross sectional area, evaluated accounting for local buckling phenomena (see section 5.4).

(3) The ultimate load bearing capacity of a section under axial load, evaluated according to the above procedure, does not include the overall buckling phenomena, which must be evaluated according to section....

(4) When welded sections are involved, a reduced value A_{red} of the net cross sectional area shall be used, evaluated by accounting for HAZ.

G.5 Evaluation of ultimate bending moment

(1) The load-bearing capacity of cross-sections under bending moment can be evaluated with reference to the above mentioned limit states, by means of the following practical rules.

(2) The value of bending moment for a given limit state can be expressed by the generalized formula:

$$M = \alpha_{M,j} W f_d$$

where:

f_d the design value of strength
W the section modulus
$\alpha_{M,j}$ a correction factor, given in table G.2, depending on the assumed limit state.

Table G.2: Ultimate Bending Moment

Bending moment	Limit state	Section class	Correction factor
M_u	Collapse	Class 1	$\alpha_{M,1} = \alpha_5 = 5 - (3{,}89 + 0{,}00190n)/\alpha_0^{[0{,}270 + 0{,}0014n)}$ $\alpha_{M,1} = \alpha_{10} = \alpha_0^{[0{,}21 \log(1000n)]} 10^{[7{,}96\times10^{-2} - 8{,}09\times10^{-2} \log(n/10)]}$ (depending on the alloy - see section G.6)
M_{pl}	Plastic	Class 2	$\alpha_{M,2} = \alpha_0 = Z/W$
M_{el}	Elastic	Class 3	$\alpha_{M,3} = 1$
M_{red}	Elastic buckling	Class 4	$\alpha_{M,4} = W_{eff}/W$ (see section 5.3.5)

where:

$n = f_{0,2}$ (in daNmm^{-2}) is the exponent of Ramberg-Osgood law representing the material behaviour (see Annex E)

α_5 and α_{10}, are the section generalized shape factors corresponding respectively to ultimate curvature values $\chi_u = 5\chi_{el}$ and $10\chi_{el}$, χ_{el} being the elastic limit curvature
α_0 is the geometrical shape factor
Z is the section plastic modulus
W_{red} is the section resistance modulus evaluated accounting for local buckling phenomena (see section 5.4).

(3) When welded sections are involved, reduced values W_{red} and Z_{red} of section resistance and plastic modulus shall be used, evaluated by accounting for HAZ.

(4) The evaluation of the correction factor $\alpha_{M,j}$ for a welded section of class 1 can be done by means of the following formula:

$$\alpha_{M,red} = \psi\left(\frac{Z_{red}}{W_{red}}\right)$$

where:

$\psi = \alpha_{M,1}/\alpha_{M,2}$, $\alpha_{M,1}$ and $\alpha_{M,2}$ being the correction factors for unwelded sections of class 1 and 2, respectively.

G.6 Prediction of rotation capacity

(1) Provisions supplied hereafter apply to ductile sections (Class 1) only, in order to define their nominal ultimate load-bearing capacity. However, they may be considered valid also for the evaluation of ultimate strength of compact and semi-compact sections, provided that no premature buckling occurs.

(2) When no reliance can be placed on the ductility properties or when no specific test can be performed on the material, the ultimate values of M_u should be referred to a conventional ultimate bending curvature given by:

$$\chi_u = \xi\chi_{el}$$

where

ξ is a ductility factor depending on the type of alloy and χ_{el} is conventionally assumed equal to $\chi_{0,2}$, which corresponds to the attainment of the proof stress $f_{0,2}$ in the most stressed fibres.

(3) From the ductility point of view the common alloys can be subdivided into two groups (see also Annex D):
- brittle alloys, having $4\ \% \leq \varepsilon_u \leq 8\ \%$, for which it can be assumed $\xi = 5$;
- ductile alloys, having $\varepsilon_u \geq 8\ \%$, for which it can be assumed $\xi = 10$.

(4) The evaluation of elastic and post-elastic behaviour of the cross-section may be done through the moment-curvature relationship, written in the Ramberg-Osgood form:

$$\frac{\chi}{\chi_{0,2}} = \frac{M}{M_{0,2}} + k\left[\frac{M}{M_{0,2}}\right]^m$$

where:

- $M_{0,2}$ and $\chi_{0,2}$ are the conventional elastic limit values corresponding to the attainment of the proof stress $f_{0,2}$
- m and k are numerical parameters which for sections in pure bending are given by:

$$m = \frac{\log\left[(10 - \alpha_{10})/(5 - \alpha_5)\right]}{\log\left(\alpha_{10}/\alpha_5\right)}$$

$$k = \frac{5 - \alpha_5}{\alpha_5 m} = \frac{10 - \alpha_{10}}{\alpha_{10} m}$$

- α_5 and α_{10} being the generalized shape factors corresponding to curvature values equal to 5 and 10 times the elastic curvature, respectively.

(5) The stable part of the rotation capacity R is defined as the ratio between the plastic rotation at the collapse limit state $\Theta_p = \Theta_u - \Theta_{el}$ to the limit elastic rotation Θ_{el} (figure G.2):

$$R = \frac{\Theta_p}{\Theta_{el}} = \frac{\Theta_u - \Theta_{el}}{\Theta_{el}} = \frac{\Theta_u}{\Theta_{el}} - 1$$

where

Θ_u is the maximum plastic rotation corresponding to the ultimate curvature χ_u.

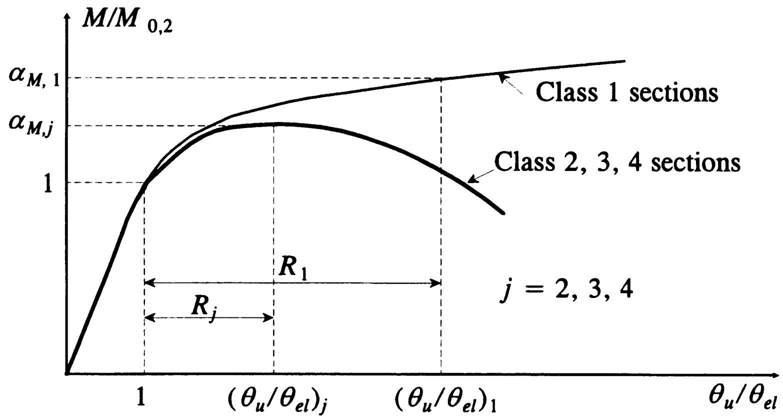


Figure G.2: Definition of rotation capacity

(6) The rotation capacity R may be calculated through the approximate formula:

$$R = \alpha_{M,j}\left(1 + 2\,\frac{k\alpha_{M,j}^{m-1}}{m+1}\right) - 1$$

with m and k defined before.
The value of $\alpha_{M,j}$ is given in table G.2 for the different behavioural classes.

(7) When the material exponent n is known (see Annex E), an approximate evaluation of α_5 and α_{10} can be done through the formulas:

$$\alpha_5 = 5 - (3{,}89 + 0{,}00190n)/\alpha_0^{(0{,}270 + 0{,}0014n)}$$

$$\alpha_{10} = \alpha_0^{[0{,}21 \log(1000n)]} \times 10^{[7{,}96 \times 10^{-2} - 8{,}09 \times 10^{-2} \log(n/10)]}$$

$\alpha_0 = Z/W$ being the geometrical shape factor.

In the absence of more refined evaluations, the value $n = f_{0,2}$ (in daNmm-2) may be assumed.

Annex H (informative)

Lateral torsional buckling

H.1 Elastic critical moment and Slenderness

H.1.1 Basis

(1) The elastic critical moment for lateral-torsional buckling of a beam of uniform symmetrical cross-section with equal flanges, under standard conditions of restraint at each end, loaded through its shear centre and subject to uniform moment is given by:

$$M_{cr} = \frac{\pi^2 EI_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z}}$$

where:

$$G = \frac{E}{2(1+\nu)}$$

I_t is the torsion constant
I_w is the warping constant
I_z is the second moment of area about the minor axis
L is the length of the beam between points which have lateral restraint.

(2) The standard conditions of restraint at each end are:

- restrained against lateral movement

- restrained against rotation about the longitudinal axis

- free to rotate in plan

H.1.2 General formula for cross-sections symmetrical about the minor axis

(1) In the case of a beam of uniform cross-section which is symmetrical about the minor axis, for bending about the major axis the elastic critical moment for lateral-torsional buckling is given by the general formula:

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

where:

C_1, C_2 and C_3 are factors depending on the loading and end restraint conditions
k and k_w are effective length factors
$z_g = z_a - z_s$
$z_j = z_s - 0{,}5 \int_A (y^2 + z^2)\, z\, dA / I_y$
z_a is the coordinate of the point of load application
z_s is the coordinate of the shear centre

Note: See H.1.2 (7) and (8) for sign conventions and H.1.4 (2) for approximations for z_j

(2) The effective length factors k and k_w vary from 0,5 for full restraint to 1,0 for no restraint, with 0,7 for

one end fixed and one end free.

(3) The factor k refers to end rotation on plan. It is analogous to the ratio ℓ/L for a compression member.

(4) The factor k_w refers to end warping. Unless special provision for warping fixity is made, k_w should be taken as 1,0.

(5) Values of C_1, C_2 and C_3 are given in tables H.1.1 and H.1.2 for various load cases, as indicated by the shape of the bending moment diagram over the length L between lateral restraints. Values are given corresponding to various values of k.

(6) For cases with $k = 1{,}0$ the value of C_1 for any ratio of end moment loading as indicated in table H.1.1, is given approximately by:

$$C_1 = 1{,}88 - 1{,}40\,\psi + 0{,}52\,\psi^2 \quad \text{but } C_1 \leq 2{,}70$$

(7) The sign convention for determining z_j, see figure 1.1, is:

- z is positive for the compression flange

- z_j is positive when the flange with the larger value of I_z is in compression at the point of largest moment.

(8) The sign convention for determining z_g is:

- for gravity loads z_g is positive for loads applied above the shear centre

- in the general case z_g is positive for loads acting towards the shear centre from their point of application

H.1.3 Beams with uniform doubly symmetric cross-sections

(1) For doubly symmetric cross-sections $z_j = 0$, thus

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{(kL)^2} \left\{ \left[\left(\frac{k}{k_w} \right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 G I_t}{\pi^2 E I_z} + [C_2 z_g]^2 \right]^{0,5} - C_2 z_g \right\}$$

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

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{(kL)^2} \left[\left(\frac{k}{k_w} \right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 G I_t}{\pi^2 E I_z} \right]^{0,5}$$

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

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{L^2} \left[\frac{I_w}{I_z} + \frac{L^2 G I_t}{\pi^2 E I_z} \right]^{0.5}$$

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

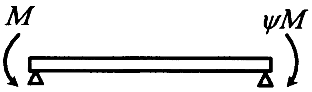
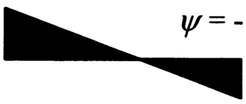
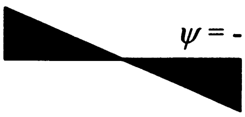
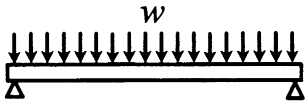
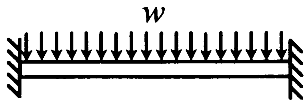
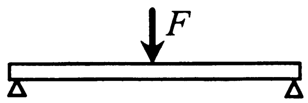
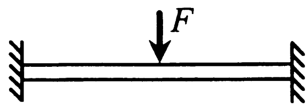
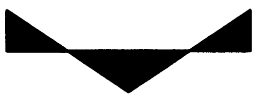
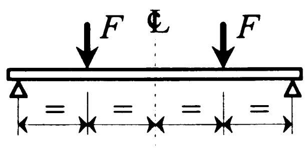
Loading and support conditions	Bending moment diagram	Values of k	Values of factors C_1	C_2	C_3
	$\psi = +1$ 	1,0 0,7 0,5	1,000 1,000 1,000	-	1,000 1,113 1,144
	$\psi = +3/4$ 	1,0 0,7 0,5	1,141 1,270 1,305	-	0,998 1,565 2,283
	$\psi = +1/2$ 	1,0 0,7 0,5	1,323 1,473 1,514	-	0,992 1,556 2,271
	$\psi = +1/4$ 	1,0 0,7 0,5	1,563 1,739 1,788	-	0,977 1,531 2,235
	$\psi = 0$ 	1,0 0,7 0,5	1,879 2,092 2,150	-	0,939 1,473 2,150
	$\psi = -1/4$ 	1,0 0,7 0,5	2,281 2,538 2,609	-	0,855 1,340 1,957
	$\psi = -1/2$ 	1,0 0,7 0,5	2,704 3,009 3,093	-	0,676 1,059 1,546
	$\psi = -3/4$ 	1,0 0,7 0,5	2,927 3,009 3,093	-	0,366 0,575 0,837
	$\psi = -1$ 	1,0 0,7 0,5	2,752 3,063 3,149	-	0,000 0,000 0,000

Table H.1.2: Values of factors C_1, C_2 and C_3 corresponding to values of factor k: Transverse loading cases

Loading and support conditions	Bending moment diagram	Values of k	Values of factors		
			C_1	C_2	C_3
		1,0 0,5	1,132 0,972	0,459 0,304	0,525 0,980
		1,0 0,5	1,285 0,712	1,562 0,652	0,753 1,070
		1,0 0,5	1,365 1,070	0,553 0,432	1,730 3,050
		1,0 0,5	1,565 0,938	1,267 0,715	2,640 4,800
		1,0 0,5	1,046 1,010	0,430 0,410	1,120 1,890

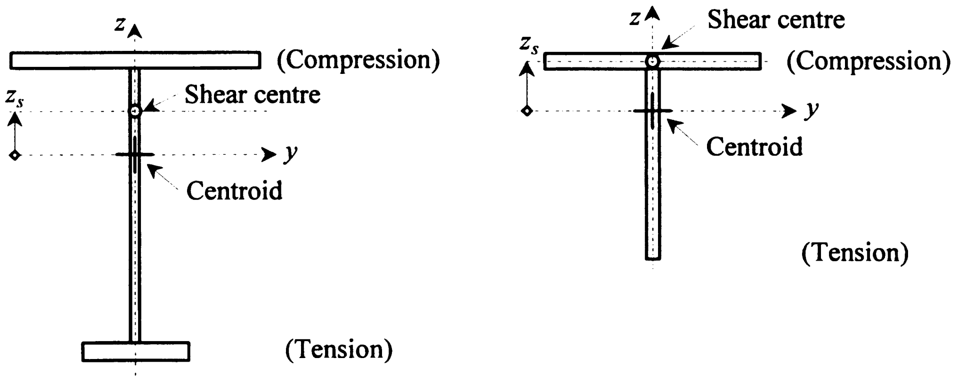


Figure H.1.1: Sign convention for determining z_j

H.1.4 Beams with uniform mono-symmetric cross-sections with unequal flanges

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

$$I_w = \beta_f (1 - \beta_f)\, I_z h_s^2$$

where:

$$\beta_f = \frac{I_{fc}}{I_{fc} + I_{ft}}$$

I_{fc} is the second moment of area of the compression flange about the minor axis of the section
I_{ft} is the second moment of area of the tension flange about the minor axis of the section
h_s is the distance between the shear centres of the flanges.

(2) The following approximations for z_j can be used:

when

$\beta_f > 0{,}5$:

$$z_j = 0{,}8\ (2\beta_f - 1)\ h_s/2$$

when

$\beta_f \leq 0{,}5$:

$$z_j = 1{,}0\ (2\beta_f - 1)\ h_s/2$$

for sections with a lipped compression flange:

$z_j = 0{,}8\ (2\beta_f - 1)\ (1 + h_L/h)\ h_s/2$ when $\beta_f > 0{,}5$
$z_j = 1{,}0\ (2\beta_f - 1)\ (1 + h_L/h)\ h_s/2$ when $\beta_f \leq 0{,}5$

where

h_L is the depth of the lip

H.2 Slenderness

H.2.1 General

(1) The slenderness ratio λ_{LT} for lateral-torsional buckling is given by:

$$\bar{\lambda}_{LT} = \frac{\lambda_{LT}}{\lambda_1} \sqrt{\alpha}$$

where:

$$\lambda_1 = \pi \sqrt{\frac{E}{f_o}} = 52{,}6\ \varepsilon$$

$$\varepsilon = \sqrt{\frac{250}{f_o}} \quad (f_o \text{ in N/mm}^2)$$

α is the shape factor taken from 5.3, but $\alpha \leq W_{pl,y} / W_{el,y}$

(2) The geometrical slenderness ratio λ_{LT} for lateral-torsional buckling is given for all classes of cross-section, by:

$$\lambda_{LT} = \pi \sqrt{\frac{E\,W_{el,y}}{M_{cr}}}$$

H.2.2 Beams with uniform doubly symmetric cross-sections

(1) For cases with $z_g = 0$ (end-moment loading or transverse loads applied at the shear centre) and $k = k_w = 1{,}0$ (no end fixity), the value of λ_{LT} can be obtained from:

$$\lambda_{LT} = \frac{L\left[\dfrac{W_{pl,y}^2}{I_z I_w}\right]^{0,25}}{C_1^{0,5}\left[1 + \dfrac{L^2 G I_t}{\pi^2 E I_w}\right]^{0,25}}$$

which can also be written:

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

where

$$a_{LT} = \sqrt{\frac{I_w}{I_t}}$$

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

$$I_w = I_z h_s^2/4$$

where

$$h_s = h - t_f$$

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

$$i_{LT} = \left(\frac{I_z I_w}{W_{pl,y}^2}\right)^{0,25}$$

or with a slight approximation by:

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

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

$$\lambda_{LT} = \frac{L/i_{LT}}{C_1^{0,5}\left[1 + \dfrac{1}{20}\left[\dfrac{L/i_{LT}}{h/t_f}\right]^2\right]^{0,25}}$$

or $$\lambda_{LT} = \frac{0{,}9\ L/i_z}{C_1^{0,5}\left[1 + \frac{1}{20}\left[\frac{L/i_z}{h/t_f}\right]^2\right]^{0,25}}$$

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

$$\lambda_{LT} = \frac{L/i_z}{C_1^{0,5}\left[1 + \frac{1}{20}\left[\frac{L/i_z}{h/t_f}\right]^2\right]^{0,25}}$$

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

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

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

or for standard rolled I or H sections:

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

or $$\lambda_{LT} = \frac{0.9\ kL/i_z}{C_1^{0,5}\left[\left[\frac{k}{k_w}\right]^2 + \frac{1}{20}\left[\frac{kL/i_z}{h/t_f}\right]^2\right]^{0,25}}$$

for any plain I or H section with equal flanges:

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

(7) Unless special provision for warping fixity is made, k_w should be taken as 1,0.

(8) Cases with transverse loading applied above the shear centre ($z_g > 0{,}0$) or below the shear centre ($z_g < 0{,}0$) can be included by using:

$$\lambda_{LT} = \frac{kL\left[\dfrac{W^2_{p\ell.y}}{I_z I_w}\right]^{0,25}}{C_1^{0,5}\left\{\left[\left[\dfrac{k}{k_w}\right]^2 + \dfrac{(kL)^2\,GI_t}{\pi^2\,EI_w} + (C_2\,z_g)^2\,\dfrac{I_z}{I_w}\right]^{0,5} - C_2\,z_g\left[\dfrac{I_z}{I_w}\right]^{0,5}\right\}^{0,5}}$$

or alternatively:

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

or for standard rolled I or H sections:

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

or alternatively:

$$\lambda_{LT} = \frac{0,9\;kL/i_z}{C_1^{0,5}\left\{\left[\left[\dfrac{k}{k_w}\right]^2 + \dfrac{1}{20}\left[\dfrac{kL/i_z}{h/t_f}\right]^2 + \left[\dfrac{2C_2\,z_g}{h_s}\right]^2\right]^{0,5} - \dfrac{2C_2\,z_g}{h_s}\right\}^{0,5}}$$

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

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

Annex J (informative)

Torsional properties of thin-walled sections

Certain design procedures in section 5.6 and 5.8 require the use of particular cross-sectional properties.

J.1 Torsion constant

(1) For a thin-walled section composed solely of flat plate elements, each of uniform thickness, and reinforced with fillets and/or bulbs, the value of the torsion constant I_t is given by

$$\sum (p + qN)^4 t^4 + \sum \frac{b t^3}{3}$$

in which:

t	=	thickness of adjacent flat material
N	=	fillet or bulb dimension, see figure J.1
p, q	=	coefficient to be read from figure J.1
b	=	width of flat element, measured to the edge of the shaded area in figure J.1 in the case of a flat element abutting a fillet or bolt.

J.2 Position of shear centre

(1) Figure J.2 gives the position of the shear centre for a number of cross-sections.

J.3 Warping constant

(1) Values of the warping constant I_w for certain types of cross-section may be found as follows:

a) for sections composed entirely of radiating outstands e.g. angles, tees, cruciforms, I_w may conservatively be taken as zero.

b) for the specific types of section illustrated in figure J.2 values of I_w may be calculated using the expression given there.

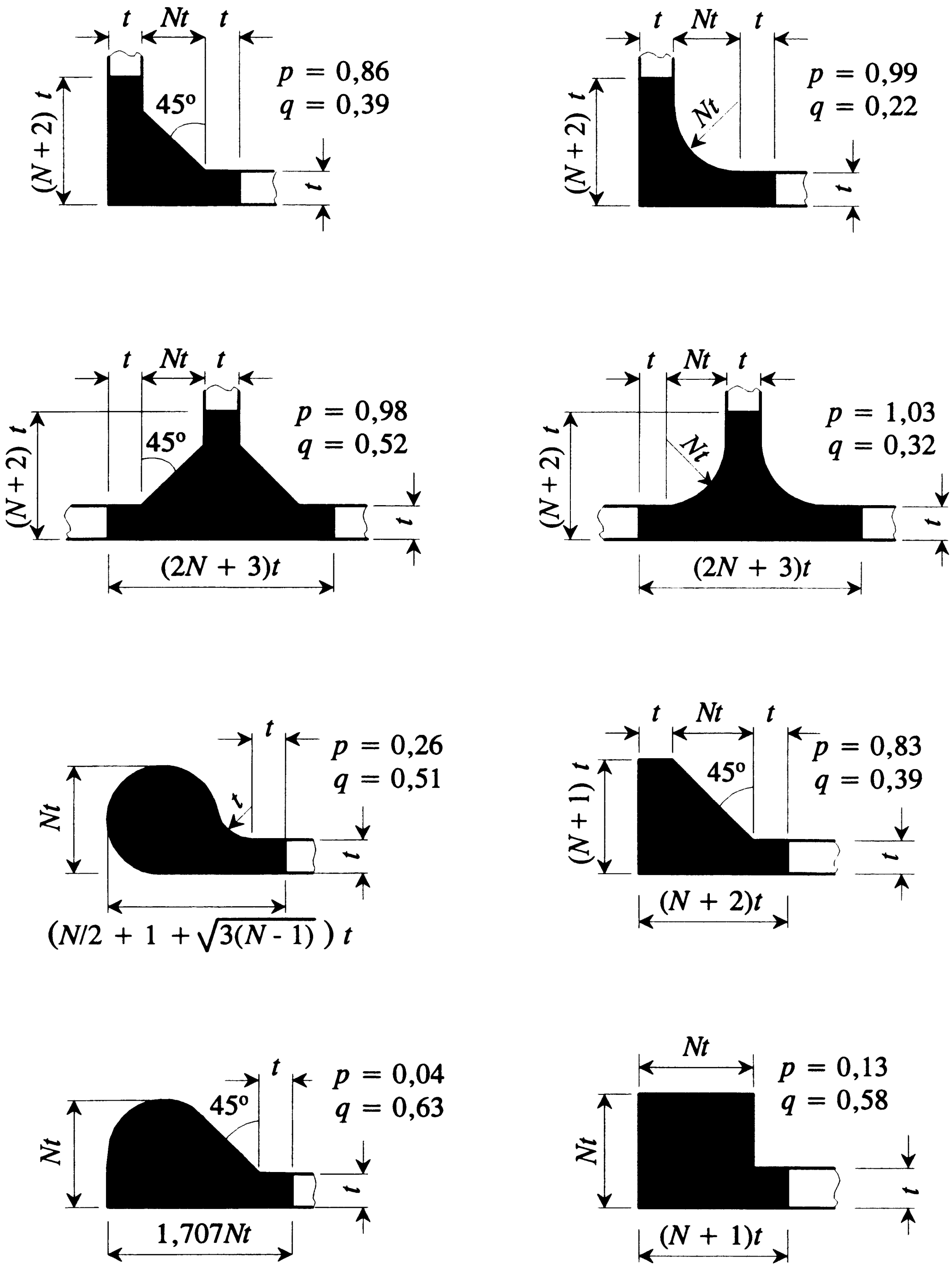
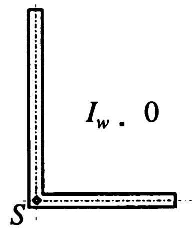
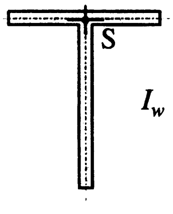


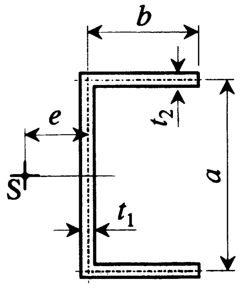
Figure J.1: Torsion constant coefficients for certain fillets and bulbs



I_w . 0



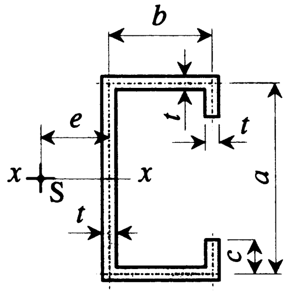
I_w . 0



$$e = \frac{3b}{F + 6}$$

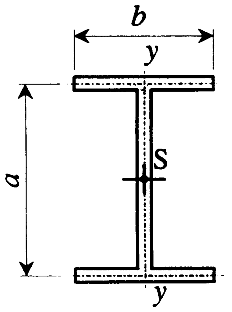
$$I_w = \frac{a^2 b^3 t_2}{12} \times \frac{2F + 3}{F + 6}$$

where $F = \dfrac{a t_1}{b t_2}$

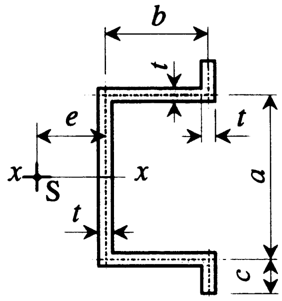


$$e = \frac{a^2 b^2 t}{I_x}\left(\frac{1}{4} + \frac{c}{2b} - \frac{2c^3}{3a^2 b}\right)$$

$$I_w = \frac{b^2 t}{6}\,(4c^3 + 6ac^2 + 3a^2 c + a^2 b) - e^2 I_x$$

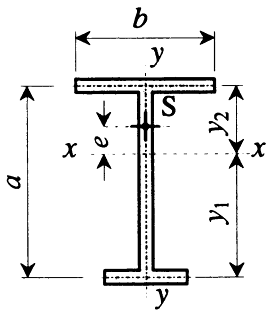


$$I_w = \frac{a^2 I_y}{4}$$



$$e = \frac{a^2 b^2 t}{I_x}\left(\frac{1}{4} + \frac{c}{2b} - \frac{2c^3}{3a^2 b}\right)$$

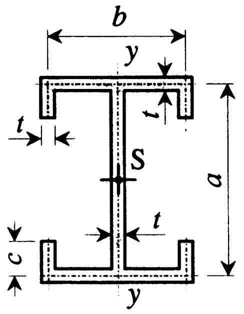
$$I_w = \frac{b^2 t}{6}\,(4c^3 - 6ac^2 + 3a^2 c + a^2 b) - e^2 I_x$$



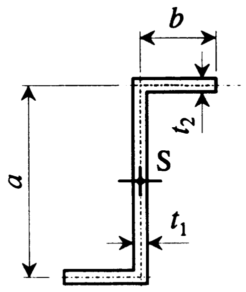
$$e = \frac{y_1 I_1 - y_2 I_2}{I_y}$$

$$I_w = \frac{a^2 I_1 I_2}{I_y}$$

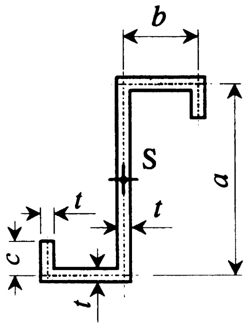
where I_1 and I_2 are the respective second moments of area of the flanges about the *y*-*y* axis



$$I_w = \frac{a^2 I_y}{4} + \frac{c^2 b^2 t}{6}\,(3a + 2c)$$



$$I_w = \frac{a^2 b^3 t_2}{12} \times \frac{2a + b}{a + 2b}$$



$$I_w = \frac{b^2 t}{12(2b + a + 2c)} \times$$

$$\times\,(a^2(b^2 + 2ba + 4bc + 6ac) + 4c^2(3ba + 3a^2 + 4bc + 2ac + c^2))$$

Figure J.2: Shear-centre position (S) and warping factor (*H*) for certain thin-walled sections

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