DRAFT FOR DEVELOPMENT

DD ENV 1993-1-3:2001

Incorporating Corrigendum No. 1

Eurocode 3: Design of steel structures —

Part 1.3: General rules — Supplementary rules for cold formed thin gauge members and sheeting

(together with United Kingdom National Application Document)

ICS 91.010.30; 91.080.10

Committees responsible for this Draft for Development

The preparation of this Draft for Development was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/31, Structural use of steel, upon which the following bodies were represented:

British Constructional Steelwork Association

Cold Rolled Sections Association

Confederation of British Forgers

Department of the Environment, Transport and the Regions

Department of the Environment, Transport and the Regions — Construction Directorate

Department of the Environment, Transport and the Regions — Highways Agency

Health and Safety Executive

Institution of Civil Engineers

Institution of Structural Engineers

Steel Construction Institute

UK Steel Association

Welding Institute

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National foreword

This publication has been prepared by Subcommittee B/525/31 and is the English language version of ENV 1993-1-3:1996, *Eurocode 3: Design of steel structures — Part 1.3: General rules — Supplementary rules for cold formed thin gauge members and sheeting* incorporating its corrigendum of October 1997, as published by the European Committee for Standardization (CEN). This Draft for Development also includes the United Kingdom (UK) National Application Document (NAD) to be used with the ENV in the design of buildings to be constructed in the UK.

ENV 1993-1-3:1996 results from a programme of work sponsored by the European Commission to make available a common set of rules for the design of building and civil engineering works.

This publication should not be regarded as a British Standard.

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

The value for certain parameters in the ENV Eurocodes may be set by CEN members so as to meet the requirements of national regulations. These parameters are designated by □ (boxed values) in the ENV.

During the ENV period of validity, reference should be made to the supporting documents listed in the NAD.

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

The Building Regulations 1991, Approved Document A 1992 (published December 1991)[1)], draws designers' attention to the potential use of ENV Eurocodes as an alternative approach to Building Regulation compliance. ENV 1993-1-3:1996 has been thoroughly examined over a period of several years and is considered to offer such an alternative approach, when used in conjunction with this NAD.

Compliance with DD ENV 1993-1-3:2001 does not of itself confer immunity from legal obligations.

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

Comments should be sent in writing to BSI, 389 Chiswick High Road, London W4 4AL, quoting the document reference, the relevant clause and, where possible, proposed revised wording.

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Summary of pages

This document comprises a front cover, an inside front cover, pages i to xvi, the ENV title page, pages 2 to 128 and a back cover.

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1) Available from The Stationery Office, PO Box 29, St Crispins House, Duke Street, Norwich NR3 1GN.

National Application Document

for use in the UK with ENV 1993-1-3:1996

Contents of National Application Document

Introduction

This National Application Document (NAD) has been prepared by Subcommittee B/525/31. It has been developed from:

a) a textual examination of ENV 1993-1-3:1996;

b) calibration against UK practice, supporting standards and test data.

NOTE *Design of cold formed steel sections and sheeting to Eurocode 3:Part 1.3* [1] gives a series of worked examples based on ENV 1993-1-3:1996 and this NAD.

It should be noted that this NAD, in common with ENV 1993-1-3 and supporting CEN standards, uses a comma (,) where a decimal point (.) would be traditionally used in the UK.

1 Scope

This NAD provides information required to enable ENV 1993-1-3:1996 to be used for the fire resistant design of buildings to be constructed in the UK.

2 Normative references

The following normative documents contain provisions, which, through reference in this text, constitute provisions of this NAD. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. For undated references, the latest edition of the publication referred to applies.

BS 648:1964 (all parts), *Schedule of weights of building materials.*

BS 6399-1:1996, *Loadings for buildings — Part 1: Code of practice for dead and imposed loads.*

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

CP 3:Chapter V:Part 2:1972, *Code of basic data for the design of buildings — Loading — Wind loads.*

3 Partial safety factors and other factors

3.1 Material factors

The values for the partial safety factor γ_M for use with ENV 1993-1-3:1996 should be as given in Table 1 of this NAD.

The values for the load factors for acceptance tests are given in **6.10** of this NAD.

Table 1 — Partial safety factors (γ_M)

Reference in ENV 1993-1-3	Definition	Symbol	Condition	Value	
				Boxed ENV value	Value for UK use
2.2(3)P	Partial safety factor for verification at the ultimate limit state.	γ_{M0}	Resistance of cross-section where failure is caused by yielding.	1,10	1,05
		γ_{M1}	Resistance of members and sheeting where failure is caused by buckling.	1,10	1,05
		γ_{M2}	Resistance of net section of bolt holes.	1,25	1,20
2.3(3)P	Partial safety factor for verifications at serviceability limit state.	$\gamma_{M,ser}$	—	1,00	1,00
8.4(6)P	Partial safety factor for calculating the design resistance of mechanical fasteners.	γ_{M2}	Bolts	1,25	1,35
			Rivets	1,25	1,35
			Pins	1,25	1,35
			Spot welds	1,25	1,35
			Lap welds	1,25	1,35
10.2.2.1(1)	Partial safety factor for steel liner trays restrained by sheeting.	γ_{M2}	Wide flange in compression	1,25	1,20
10.2.2.2(1)	Partial safety factor for steel liner trays restrained by sheeting.	γ_{M2}	Wide flange in tension	1,25	1,20
A.6.4	Partial factor for difference in behaviour under test conditions and service conditions.	γ_{sys}	—	1,0	1,0

4 Loading codes

The loading codes to be used are:

BS 648:1964 (all parts), *Schedule of weights of building materials*.

BS 6399-1:1996, *Loadings for buildings — Code of practice for dead and imposed loads*.

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

CP3:Chapter V:Part 2:1972, *Code of basic data for the design of buildings — Loading — Wind loads*.

In using these documents with ENV 1993-1-3:1996, the following modifications should be noted.

a) The imposed floor loads of a building should be treated as one variable action to which the reduction factors given in clause **5** of BS 6399-1:1984 are applicable.

b) The wind loading should be taken as 90 % of the value obtained from clause **4.3** of CP3:Chapter V:Part 2:1972.

NOTE Although it is intended that BS 6399-2 will eventually replace CP3:Chapter V:Part 2, wind loads for structures designed in accordance with ENV 1993-1-3:1996 should continue to be determined in accordance with CP3:Chapter V:Part 2 rather than in accordance with BS 6399-2 until such time as CP3:Chapter V:Part 2 is withdrawn. In such cases, local wind pressure and suction need not be considered in the design of purlins and sheeting rails.

c) The design for structural integrity should follow the provisions in **6.2**a) of this NAD.

d) Reference should be made to clause **12** of BS 6399-1:1996 for the determination of accidental loads.

5 Reference standards

Where ENs are directly referred to by ENV 1993-1-3:1996, the appropriate BS ENs should be used. The remaining supporting standards to be used for construction with cold formed thin gauge members and sheeting designed in accordance with ENV 1993-1-3:1996 are given in Table 2 of this NAD.

Table 2 — Directly referenced supporting standards in ENV 1993-1-3

ENV 1993-1-3:1996 calls up	UK supporting standard
BS EN 10149-2	BS EN 10149-2
BS EN 10149-3	BS EN 10149-3
ENV 1090-2	DD ENV 1090-2[a] BS 5950-7
ENV 1991-1	BS 6399, CP3:Chapter V:Part 2[a]
ENV 1993-1-1	DD ENV 1993-1-1:1992
ISO 4997	ISO 4997
ISO 1000	ISO 1000 (BS 5555)

[a] Currently in preparation.

[a] See the note in clause **4** of this NAD.

6 Additional recommendations

6.1 Chapter 1 General

a) *1.1 Scope*

Cold formed thin gauge members may be either open or closed and should be made up of flat elements bounded either by free edges or by bends with included angles not exceeding 135° and internal radii not exceeding 5*t* where *t* is the material thickness. ENV 1993-1-3:1995 does not apply to cold formed structural hollow sections complying with EN 10219, for which reference should be made to ENV 1993-1-1:1992.

The designer responsible for the overall stability of the structure should be clearly identified. This designer should ensure the compatibility of the structural design and detailing between all those structural parts and components that are needed for overall stability, even if some or all of the structural design and detailing of those structural parts and components is carried out by another designer.

b) *1.1(3)*

The detailed design of stressed-skin constructions should be in accordance with BS 5950-9.

c) *1.1(5)*

The limitations that do not apply to design assisted by testing are:

1) *b*/*t* ratio;

2) thickness;

3) material properties.

d) *1.5(3)*

System lines of flanges means mid-lines of flanges.

e) *1.7.4(2)*

To simplify the design rules for torsional and torsional-flexural buckling in **6.2.3** of ENV 1993-1-3:1996, the convention for member axes differs from that used in ENV 1993-1-1:1992 and it may also change depending on the design situation.

6.2 Chapter 2 Basis of design

a) *2.1 General*

1) Structures constructed using cold formed thin gauge members and sheeting should be designed to fulfil, with due regard to economy, their intended function and should sustain the design loads for their intended life. The design should also facilitate fabrication, erection and future maintenance.

2) A structure should also be designed so that it should not be damaged by events, explosions, impact or the consequences of human error, to an extend disproportionate to the original cause. Design rules to provide structural integrity by limiting the effects of accidental damage are given in Annex A of the NAD for ENV 1993-1-1:1992.

In construction where vertical loads are resisted by an assembly of closely spaced elements, (e.g. cold formed steel framing) the tying members should be distributed to ensure that the entire assembly is effectively tied. In such cases the forces for anchoring the vertical elements at the periphery should be based on the spacing of the elements or taken as 1 % of the factored vertical load in the element without applying the minimum value of 75 kN or 40 kN to the individual elements, provided that each tying member and its connections are designed to resist the appropriate loading.

NOTE 1 The above recommendations should be met by the choice of suitable materials, by appropriate design and detailing and by specifying control procedures for production, construction and use as relevant for the particular project

NOTE 2 Further guidance on methods of reducing the sensitivity of buildings to disproportionate collapse in the event of an accident are given in Approved Document A of the Building Regulations [2].

b) *2.1(4)P*

The values of partial factors given in this NAD should be adopted for Construction Clauses I, II and III.

c) *2.2(1)P*

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

2) When designing for the accidental situation in Table 2.1 of ENV 1993-1-1:1992, the values of ψ_1 and ψ_2 should be determined from Table 4 of the NAD for ENV 1993-1-1:1992. For the determination of the accidental load (A_k), reference should be made to BS 6399-1 where appropriate.

The accidental total A_k should be multiplied by a γ_A factor of 1,05 and the γ_{GA} factor should be taken as 1,05, except where the dead load is considered to consist of unfavourable and favourable parts. In this case, the favourable part should be multiplied by a γ_{GA} factor of 0,9 and the unfavourable part should be multiplied by a factor of 1,05.

6.3 Chapter 3 Properties of materials and cross-sections

a) *3.1.1(7)P*

Although the real value for the modulus of elasticity for cold formed steel is less, the value of 210 000 N/m² should be used because the formulae have been developed and calibrated using this value.

b) *2.1.2(3)P*

For cross-sections which are not fully effective, the increase in yield strength due to cold forming may be calculated using the recommendations given in BS 5950-5.

c) *3.1.2(7)P*

The increase in yield strength due to cold working should not be utilized for members which undergo welding, annealing, galvanizing or any other heat treatment after forming that may produce softening.

d) *3.1.3(1)P*

For steel sheeting, the upper limit for the nominal core thickness (t_{cor}) may be taken as 8.0 mm. No lower limit is needed for the sheeting providing that it can be demonstrated to have adequate resistance to denting from construction and maintenance traffic. ENV 1993-1-1:1992 may be used for steel with a nominal core thickness t_{cor} exclusive of zinc or organic coating greater than 5 mm. No lower limit is necessary for the nominal core thickness of members.

e) *313(3)P amd 313(4)P*

The design thickness should be obtained from the following expression:

$t = k \times t_{cor}$

where

k is obtained from Table 3 of this NAD.

f) *3.1.3(5)*

The nominal core thickness (t_{cor}) should be calculated using the following expression:

$t_{cor} = t_{nom} - t_{coating}$

where

t_{nom} is the nominal thickness;

$t_{coating}$ is the thickness of the coating (i.e. zinc, paint, etc.)

Table 3 — Values of factor *k*

Nominal thickness t_{cor}	Members		Sheeting	
	Normal tolerances	Special tolerances	Normal tolerances	Special tolerances
$1{,}0 \leq t_{nom}$	1,0	—[a]	1,0	—[a]
$0{,}6 \leq t_{nom} < 1{,}0$	0,95	1,0	1,0	—[a]
$0{,}4 \leq t_{nom} < 0{,}6$	0,9	0,95	0,95	1,0

[a] Not applicable.

6.4 Chapter 4 local buckling

a) *4.2(5)*

Alternative 1 should be used and Alternative 2 should be ignored.

NOTE Alternative 1 is based on the ultimate limit state.

b) *4.2(6)*

Alternative 2 should be used and Alternative 1 should be ignored.

NOTE Alternative 2 is based on the serviceability limit state.

c) *4.2(9)*

Values of effective section properties obtained without iteration may be used. Alternatively, the iteration process may be used if it gives a larger value.

NOTE Under some circumstances iteration may give a smaller value. If this is the case, the initial value may be used.

d) *4.3.2.1(1)*

Provided that the conditions given in **4.3.2.1**(2)P of ENV 1993-1-3:1996 and modified by the conditions given in **6.4**c) of this NAD are met, the effectiveness of a multiple edge fold stiffener may be determined from the procedures given in **4.3.2.1**(3) of ENV 1993-1-3:1996.

e) *4.3.2.1(2)P*

It is not necessary to limit c such that:

$c \leq 0{,}2b_p$

f) *Figure 4.7*

For the figure showing one stiffener, $S_{eff,4}$ should read $S_{eff,n}$, and for the figure showing two stiffeners, $S_{eff,6}$ should read $S_{eff,n}$.

g) *4.3.2.2(8) and 4.3.2.2(9)*

The value of χ may be obtained without iteration. Alternatively, the iteration process may be used if it gives a higher value.

h) *4.3.2.3(3)*

The values of 0,5 and 1,0 for χ should be changed to 0,45 and 0,9 respectively.

i) 4.3.3.1

The general procedure in **4.3.3.2** of ENV 1993-1-3:1996 and the simplified procedure in **4.3.3.3** of ENV 1993-1-3:1996 do not include the effects of flange curling and give inaccurate results for members subject to bending actions. These procedures should not be used for beams where the ratio of b/t is greater than 300.

j) *4.3.4.2 Flanges with intermediate stiffeners*

The procedure in **4.3.4.2** of ENV 1993-1-3:1996 does not include the effects of flange curling and gives inaccurate results for sheeting subjected to bending actions. This procedure should not be used for beams where the ratio of b/t is greater than 300.

k) *4.3.4.2(3)*

The expression for b_e is incorrect and should be replaced with the following correct expression:

$$b_e = 2b_{p,1} + b_{p,2} + 2b_s$$

l) *Figure 4.3*

When considering this figure, it should be noted that c_{eff} is not necessarily equal to c.

6.5 Chapter 5 Resistance of cross-sections

a) *5.1 General*

Attention is drawn to the fact that the following four different cross-sectional properties are used in ENV 1993-1-3:1996:

1) gross cross-sectional properties;
2) effective area;
3) effective section modulus about the major axis;
4) effective section modulus about the minor axis.

b) *Figure 5.1*

In Figure 5.1, the applied axial force N_{Sd} is assumed to act at the centroid of the gross cross-section, whereas the resistance to axial force N_{Rd} is assumed to act at the centroid of the effective cross-section. Thus the force N_{Sd} should be treated as being applied at an eccentricity equal to the shift e_N of the centroid.

c) *5.4.2 Partial plastic resistance*

If the section conforms to the requirements for a class 1 cross-section given in clause **5.3** of ENV 1993-1-1:1992, the method given in **5.2.3.1** of BS 5950-5:1998 for calculating the plastic bending category may be used.

d) *5.4.2(2)*

A bilinear distribution is shown in the right hand diagram in Figure 5.3.

e) *5.4.2(4)*

Alternatively, the provisions given in **7.2** may be demonstrated by calculation in appropriate cases.

f) *5.4.3(1)P*

The effects of shear lag should be taken into account if the length L_m is less than $20b_0$ for simply-supported beams with a uniformly-distributed load, and less than $50b_0$ for all other cases.

g) *Table 5.1 Reduction factors β_i for shear lag*

It should be noted that L_m is the distance between points of contraflexure.

h) 5.6(2)P

The definitions for e_{Ny} and e_{Nz} are incorrect — e_{Ny} and e_{Nz} are shifts of the y-y and z-z centroid axes respectively, under axial loading.

i) *5.8(5)P*

When using Table 5.2, the value of f_{bv} should be calculated using the formulae for webs without stiffening at the support, irrespective of whether stiffeners are present or not.

j) *5.8(6)*

The correct interpretation for k_r is:

$$k_r = 5,34 + \frac{2,10}{t}\left[\frac{I_s}{s_d}\right]^{1/3}$$

k) *5.9.2*

For I-beams with restraint against web rotation, the method given in **5.3** of BS 5950-5:1998 may be used.

6.6 Chapter 6 Buckling resistance

a) *Expression 6.4a*

Expression 6.4a is incorrect and should be replaced by the following expression

$\bar{\lambda} = (f_{yb}/\sigma_{cr})^{0,5}[\beta_A]^{0,5}$

b) *6.2.3 Torsional buckling and torsional-flexural buckling*

All sections should be checked for torsional, torsional-flexural and flexural buckling.

c) *6.2.3(1)P*

An example of a point-symmetric open cross-section is a Z-section with similar flanges.

d) *6.2.3(7)P*

For non-symmetrical cross-sections, the maximum stress should be determined by second-order analysis.

e) *6.3(1)P*

Alternatively, M_{cr} can be obtained from **6.6.2.2** of BS 5950-5:1998, its value being that of M_E.

f) *6.4 Distortional buckling*

For distortional buckling, reference should be made to AS/NZS 4600:1996 or a geometrical non-linear analysis could be carried out using suitable initial imperfections.

g) *6.5 Bending and axial compression*

All members subject to combined bending and axial compression should be designed in accordance with the recommendations given in **6.4** of BS 5950-5:1998.

6.7 Chapter 7 Serviceability limit states

a) *7.3 Deflections*

The designs for deflections should follow the provisions in clause **4**, in particular, Table 4.1 and **4.2.3** of ENV 1993-1-1:1992.

b) *7.3(3)*

There is no limit to the deflection of purlins, provided the provisions in **4.2.3** of ENV 1993-1-1:1992 are complied with in respect of the supporting structure.

c) *7.4(2)*

Serviceability limits for sheeting should be obtained from BS 5427-1 and BS 5950-6.

6.8 Chapter 8 Joints and connections

NOTE Design guidance for butt and V-flared welds is given in **6.6.2**(6) of ENV 1993-1-1:1992,

a) *Table 8.4 Bearing resistance*

The bearing resistance given in Table 8.4 can only be used if washers are used under both the head and nut of the bolt.

b) *8.4(10)*

If both $F_{t,Rd}$ and $F_{v,Rd}$ are obtained by calculation, the following equation may be used:

$$\frac{F_{t,Sd}}{F_{t,Rd}} + \frac{F_{v,Sd}}{1,4 F_{v,Rd}} \leq 1$$

where

$F_{v,Sd} \leq F_{v,Rd}$.

If either $F_{v,Sd}$ or $F_{v,Rd}$ is obtained by testing, the linear equation given by expression 8.2 should be used.

Alternatively, combined tension and shear testing may be carried out.

c) *8.5(7)P*

The thickness t of the test specimen should be the same as the thickness of the specimens used in practice.

d) *8.6.2*

The values of $L_{w,e}$ and $L_{w,s}$ used in expressions 8.4a and 8.4b respectively for calculating the design resistance $F_{w,Rd}$ of a fillet weld, should not exceed the width of the connected part or sheet, b.

e) *8.6.3(5)P*

1) The minimum distance measured parallel to the direction of force transfer, from the centreline of an arc spot weld to the nearest edge of an adjacent weld or to the end of the connected part towards which the force is directed, should not be less than the value of e_{min} given by the following.

If $\frac{f_u}{f_y} \geq 1,15$

$$e_{min} = 1,8 \times \frac{F_{wSd}}{t f_u / \gamma_{M2}}$$

If $\frac{f_u}{f_y} \leq 1,15$

$$e_{min} = 2,1 \times \frac{F_{wSd}}{t f_u / \gamma_{M2}}$$

2) The minimum distance from the centreline of a circular arc spot weld to the end or edge of the connected sheet should not be less than $1,5d_w$ where d_w is the visible diameter of the arc spot weld.

3) The minimum clear distance between an elongated arc spot weld and the end of the sheet and between the weld and the edge of the sheet should not be less than $1,0d_w$.

f) *8.6.3(6)*

This clause should be replaced by the following.

The design shear resistance $F_{W,Rd}$ of a circular arc spot weld should be determined using the following expression:

$$F_{w,Rd} = \frac{\pi}{4} d_s^2 \times 0{,}625 \frac{f_{uw}}{\gamma_{M2}}$$

where

f_{uw} is the minimum ultimate tensile strength of the welding electrodes.

$F_{W,Rd}$ should not be taken as more than the peripheral resistance given by the following expression:

$$\text{If } \leq 18 \times \left(\frac{420}{f_u}\right)^{0,5}$$

$$F_{w,Rd} = 1{,}5 d_p \Sigma t \, f_u / \gamma_{m2}$$

The minimum distance from the centreline of a circular arc spot weld to the end or edge of the connected sheet should be not less than $1{,}5d_w$, where d_w is the visible diameter of the arc spot weld.

The minimum clear distance between an elongated arc spot weld and the end of the sheet and between the weld and the edge of the sheet should be not less than $1{,}0d_w$.

$$\text{If } 18 \times \left(\frac{420}{f_u}\right)^{0,5} < \frac{d_p}{\Sigma t} < 30 \times \left(\frac{420}{f_u}\right)^{0,5}$$

$$F_{W,Rd} = 27(420/f_u)^{0,5} (\Sigma t)^2 f_u / \gamma_{M2}$$

$$\text{If } \frac{d_p}{\Sigma t} \geq 30 \times \left(\frac{420}{f_u}\right)^{0,5}$$

$$F_w = 0{,}9 \, d_p \Sigma \mathrm{t} \, f_u / \gamma_{M2}$$

g) *8.6.3(7)*

This clause should be replaced by the following.

The interface diameter d_s of an arc spot weld (see Figure 8.6), should be obtained from the following expression:

$$d_s = 0{,}7d_w - 1{,}5\Sigma t$$

where

d_s $\leq 0{,}55d_w$;

d_w is the visible diameter of the arc spot weld (see Figure 8.6).

h) *8.6.3(9)*

This clause should be replaced by the following.

The design shear resistance $F_{W,Rd}$ of an elongation arc spot weld should be determined from the following expression:

$$F_{W,Rd} = \left[\left(\frac{\pi}{4}\right) \times d_s^2 + L_w \times d_s\right] \times \left[0{,}625 \times \frac{f_{uw}}{\gamma_{M2}}\right]$$

where

$F_{W,Rd}$ should not be taken as more than the peripheral resistance given by:

$F_{W,Rd} = (0,5\ L_w + 1,67\ d_p)\Sigma t\ f_u/\gamma_{M2}$

where

L_w is the length of the elongated arc spot weld, measured as shown in Figure 8.7.

6.9 Particular applications

a) *10.1.3.4(2)*

Alternatively, the characteristics may be determined by calculation.

b) *Figure 10.7*

The orientation of the members is not dependent on the number of spans (i.e. the left hand diagram is not related to the right hand diagram).

c) *10.3 Stressed skin design*

The detailed design of stressed-skin construction should also be in accordance with BS 5950-9.

6.10 Annex A Testing procedures

a) *Table A.1 Number of tests*

When the general shape of the buckling curve is obtained from prior knowledge, a smaller number of tests may be carried out, providing it contains a significant number of tests at $\bar{\lambda} = 1,0$.

b) *A.3.4(2)*

Lateral means in any direction at a right angle to the longitudinal axis.

c) *A.4.1 Acceptance tests*

The values for the load factors for acceptance tests for use with ENV 1993-1-3:1996 should be taken as equal to the sum of:

1) 1,0 × the actual self-weight present during the test;

2) one of the following as appropriate:

i) 1,25 × (the imposed load) + 1,15 × (the remainder of the permanent load);

ii) 1,15 × (the remainder of the permanent load) + 1,25 × (the wind load);

iii) 1,25 × (the wind uplift) – 1,0 × (the remainder of the permanent load);

iv) 1,15 × (the remainder of the permanent load) + 1,0 × (the imposed load and the wind load).

d) *A.4.1(6)*

On the attainment of the acceptance test load, it should be maintained at a near constant value to allow repeat measurements for the detection of possible creep. The loads and deflections should be measured at regular checking intervals of at least 5 min. The loading should be adjusted to remain constant until there is no significant increase in deflection during at least three checking intervals subsequent to the attainment of the acceptance test load.

e) *A.5.2.3 Interpretation of test results*

The factor of 0,9 need not be applied if the procedure in **A.6** of ENV 1993-1-3:1996 is followed. Figure A.9 and A.10 are for illustration only. The procedure can be applied to sheeting.

f) *A.6.2(6) Resistance adjustment coefficient*

If $f_{yb,obs} > f_{yb}$,

$a = 1,0$ should be used in all cases.

g) *A.6.3 Characteristic values*

Replace expression A.11 with the following expression:

$R_k = 1{,}1 \times (R_m - k_s)$

where

$R_k \leq R_m$

h) *A.6.3.3 Characteristic values based on a small number of tests*

For calculation of the characteristic value of resistance (R_k) the value of η_k should be taken as 0,9 for all modes of failure

If two or three tests are performed, the characteristic value of resistance should be obtained from the following expression:

$R_k = \eta_k \times R_{min}$

Bibliography

BS 5427-1, *Code of practice for the use of profiled sheet for roof and wall cladding on buildings — Part 1: Design*.

BS 5950-6, *Structural use of steelwork in building — Part 6: Code of practice for design of light gauge profiled steel sheeting*.

BS 5950-7, *Structural use of steelwork in building — Part 7: Specification for materials and workmanship: cold formed sections*.

BS EN 10149-2, *Specification for hot-rolled products made of high yield strength steels for cold forming — Part 2: Delivery conditions for thermomechanically rolled steels*.

BS EN 10149-3, *Specification for hot-rolled products made of high yield strength steels for cold forming — Part 3: Delivery conditions for normalized or normalized rolled steels*.

BS EN 10219, *Cold formed welded structural sections of non-alloy and fine grain steels*.

ISO 1000, *SI units and recommendations for the use of their multiples and of certain other units*.

ISO 4997, *Cold-reduced steel sheet of structural quality*.

[1] COUCHMAN, G.H. *Design of cold formed steel sections and sheeting to Eurocode 3:Part 1.3:1999*[2]. ISBN 1 85942 086 9.

[2] GREAT BRITAIN. The Building Regulations 1991, Approved Document A 1992. London: The Stationery Office[3].

[2] Available from The Steel Construction Institute, Silwood Park, Ascot, Berkshire SL5 7QN.
[3] Available from the Stationery Office, PO Box 29, St Crispins House, Duke Street, Norwich NR3 1GN.

EUROPEAN PRESTANDARD
PRÉNORME EUROPÉENNE
EUROPÄISCHE VORNORM

ENV 1993-1-3:1996/AC

October 1997
Octobre 1997
Oktober 1997

English version
Version Française
Deutsche Fassung

Eurocode 3: Design of steel structures - Part 1-3: General rules - Supplementary rules for cold formed thin gauge members and sheeting

Eurocode 3: Calcul des structures en acier - Partie 1-3: Règles générales - Règles supplémentaires pour les éléments minces formés à froid - Produits longs et produits plats

Eurocode 3: Bemessung und Konstruktion von Stahlbauten - Teil 1-3: Allgemeine Regeln - Ergänzende Regeln für kaltgeformte dünnwandige Bauteile und Bleche

This corrigendum becomes effective on 2 October 1997 for incorporation in the official English version of the ENV.

Ce corrigendum prendra effet le 2 octobre 1997 pour incorporation dans la version anglaise officielle de la ENV.

Die Berichtigung tritt am 2.Oktober 1997 in Kraft zur Einarbeitung der offiziellen Englischen Fassung der ENV einzufügen.

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

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

Ref. No. ENV 1993-1-3:1996/AC:1997 E

The following editorial corrections should be made to ENV 1993-1-3.

Page 24, clause 3.5, below 3.5(1) and above table 3.3, insert paragraph (2) reading:

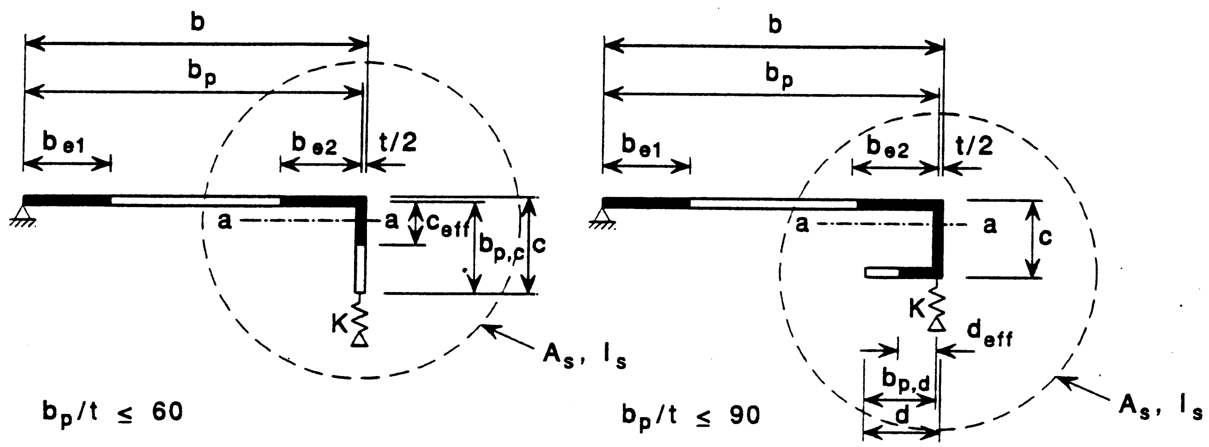
(2) The mutual influence of multiple stiffeners should be taken into account.

Page 27, table 4.2, row 4, column 4, replace 1 by −1.

Page 27, table 4.2, row 5, last column should read:

$$0{,}57 - 0{,}21\psi + 0{,}07\psi^2$$

Page 30, figure 4.2 should be replaced by:



a) single edge fold

b) double edge fold

Figure 4.2: Edge stiffeners

Page 31, paragraph 4.3.2.2(4), expression (4.10c) should read:

$$k_\sigma = 0{,}5 + 0{,}83 \times \sqrt[3]{(b_{p,c}/b_p - 0{,}35)^2} \quad \ldots (4.10c)$$

Page 33, paragraph 4.3.2.3(2), line 4 should read:

ρ should be obtained from 4.2(5) with $\sigma_{com,Ed}$ equal to $\chi f_{yb}/\gamma_{M1}$, so that:

Page 33, paragraph 4.3.2.2(10), replace expression (4.14) by two lines reading:

$$A_{s,red} = \chi A_s \left[\frac{f_{yb}/\gamma_{M1}}{\sigma_{com,Ed}} \right] \quad \text{but} \quad A_{s,red} \leq A_s \qquad \ldots (4.14)$$

in which $\sigma_{com,Ed}$ is the calculated stress at the centreline of the stiffener.

Page 33, paragraph 4.3.2.2(11), modify line 2 to read:

by using a reduced thickness $t_{red} = tA_{s,red}/A_s$ for all the elements included in A_s.

Page 33, paragraph 4.3.2.3(4), replace expression (4.19) by two lines reading:

$$A_{s,red} = \chi A_s \left[\frac{f_{yb}/\gamma_{M1}}{\sigma_{com,Ed}} \right] \quad \text{but} \quad A_{s,red} \leq A_s \qquad \ldots (4.19)$$

in which $\sigma_{com,Ed}$ is the calculated stress at the centreline of the stiffener.

Page 33, paragraph 4.3.2.3(5), modify line 2 to read:

by using a reduced thickness $t_{red} = tA_{s,red}/A_s$ for all the elements included in A_s.

Page 35, paragraph 4.3.3.2(9), replace expression (4.23) by two lines reading:

$$A_{s,red} = \chi A_s \left[\frac{f_{yb}/\gamma_{M1}}{\sigma_{com,Ed}} \right] \quad \text{but} \quad A_{s,red} \leq A_s \qquad \ldots (4.23)$$

in which $\sigma_{com,Ed}$ is the calculated stress at the centreline of the stiffener.

Page 35, paragraph 4.3.3.2(10), modify line 2 to read:

by using a reduced thickness $t_{red} = tA_{s,red}/A_s$ for all the elements included in A_s.

Page 35, paragraph 4.3.3.3(3), line 1 should start:

The effective widths $b_{1,e2}$ and $b_{2,e1}$ should .....

and line 3 should read:

to $\chi f_{yb}/\gamma_{M1}$, so that:

Page 37, paragraph 4.3.3.3(5), replace expression (4.28) by two lines reading:

$$A_{s,red} = \chi A_s \left[\frac{f_{yb}/\gamma_{M1}}{\sigma_{com,Ed}} \right] \quad \text{but} \quad A_{s,red} \le A_s \qquad \text{... (4.28)}$$

in which $\sigma_{com,Ed}$ is the calculated stress at the centreline of the stiffener.

Page 37, paragraph 4.3.3.3(6), modify line 2 to read:

by using a reduced thickness $t_{red} = tA_{s,red}/A_s$ for all the elements included in A_s.

Page 45, paragraph 5.4.1(1)P, expression (5.3b) should read:

$$M_{c,Rd} = f_{ya} W_{et} / \gamma_{M0} \qquad \text{... (5.3b)}$$

Page 53, paragraph 5.8(6)P, expression (5.15a) should read:

$$\bar{\lambda}_w = \sqrt{\frac{f_{yb}/\sqrt{3}}{\tau_{cr}}} \equiv \frac{s_w}{t} \sqrt{\frac{12(1-\nu^2) f_{yb}}{\sqrt{3}\,\pi^2 E k_\tau}} \qquad \text{... (5.15a)}$$

Page 57, paragraph 5.9.3(2), delete line 6 reading:

s_s is the actual length of stiff bearing;

Page 58, paragraph 5.9.3(4), insert after line 8:

where:

s_s is the actual length of stiff bearing;

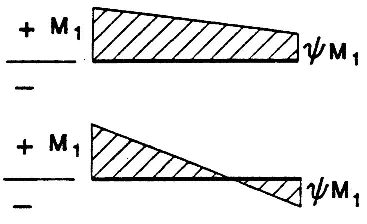
Page 60, paragraph 5.9.4(2), expression (5.24) should read:

$$\kappa_{a,s} = 1{,}45 - 0{,}05\, e_{max}/t \quad \text{but} \quad \kappa_{a,s} \le 0{,}95 + 35\,000 t^2 e_{min} / (b_d^2 s_p) \qquad \text{... (5.24)}$$

Page 68, paragraph 6.3(1)P, line 4 should read:

in which χ_{LT} is obtained from the following:

Page 71, table 6.4, column 1, row 2, replace the second and third sketches by:



Page 77, figure 8.2 should be replaced by:

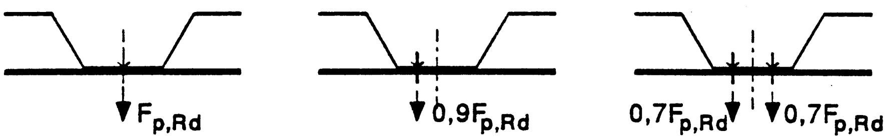


Figure 8.2: Reduction of tension resistance due to the position of fasteners

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Page 81, table 8.4, row 2, line 2 should read:

$F_{b,Rd} \;=\; 2{,}5\,d\,t\,f_u / \gamma_{M2}$ but $F_{b,Rd} \;\leq\; (e_1 t / 1{,}2)(f_u / \gamma_{M2})$

Page 82, table 8.5, row 2, lines 3 to 5 should read:

$F_{tb,Rd} \;=\; 2{,}7\sqrt{t}\; d_s f_u / \gamma_{M2}$ [with t in mm]

- if $t_1 > 2{,}5t$:

$F_{tb,Rd} \;=\; 2{,}7\sqrt{t}\; d_s f_u / \gamma_{M2}$ but $F_{tb,Rd} \;\leq\; 0{,}7\, d_s^2 f_u / \gamma_{M2}$ and $F_{tb,Rd} \;\leq\; 3{,}1\, t\, d_s f_u / \gamma_{M2}$

Page 84, paragraph 8.6.3(3)P should read:

Arc spot welds shall have an interface diameter d_s of not less than 10 mm.

Page 101, paragraph 10.1.5.2(10), expression (10.18) should read:

$$C_{D,A} \;=\; \frac{h^2}{\left(1/K_A + 1/K_B\right) - 4\left(1 - \nu^2\right) h^2 \left(h_d + e\right) / \left(E t^3\right)} \qquad \text{... (10.18)}$$

Page 105, paragraph 10.2.2.2(1), in step 2 the last line should read:

I_a is the second moment of area of the wide flange, about its own centroid, see figure 10.8.

Page 105, paragraph 10.2.2.2(2), the first sentence should read:

The effects of shear lag need not be considered if $L / b_{u,eff} \geq 20$.

Page 109, paragraph 10.3.5(4), lines 2 to 4 should read:

$$T_{v,Rd} = 6E \sqrt[4]{I_a (t/b_u)^9} \qquad \text{... (10.22)}$$

where:

I_a is the second moment of area of the wide flange, about its own centroid, see figure 10.8;

EUROPEAN PRESTANDARD

PRÉNORME EUROPÉENNE

EUROPÄISCHE VORNORM

ENV 1993-1-3

April 1996

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English version

Eurocode 3: Design of steel structures - Part 1-3: General rules - Supplementary rules for cold formed thin gauge members and sheeting

Eurocode 3: Calcul des structures en acier - Partie 1-3: Règles générales - Règles supplémentaires pour les éléments minces formés à froid - Produits longs et produits plats

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S

This European Prestandard (ENV) was approved by CEN on 1993-06-04 as a prospective standard for provisional application. The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into an European Standard (EN).

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

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

CEN

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

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

Ref. No. ENV 1993-1-3:1996 E

Contents

Foreword

Objectives of the Eurocodes

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

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

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

Background to the Eurocode programme

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

(5) In 1990, after consulting their respective member states, the CEC transferred the work of further development, issue and updating of the Structural Eurocodes to CEN, and the EFTA Secretariat agreed to support the CEN work.

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

Eurocode programme

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

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

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

(9) This Part 1.3 of Eurocode 3 is published by CEN as a European Prestandard (ENV) with an initial life of three years.

(10) This Prestandard is intended for experimental application and for the submission of comments.

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

(12) Meanwhile feedback and comments on this Prestandard should be sent to the secretariat of CEN/TC 250/SC 3 at the following address:

BSI Standards
British Standards House
389 Chiswick High Road
London W4 4AL
England

or to your national standards organization.

National Application Documents (NAD's)

(13) In view of the responsibilities of the authorities in member countries for safety, health and other matters covered by the essential requirements of the Construction Products Directive (CPD), certain safety elements in this ENV have been assigned indicative values which are identified by $\boxed{}$ ("boxed values"). The authorities in each member country are expected to review the "boxed values" and <u>may</u> substitute alternative definitive values for these safety elements for use in national application.

(14) Some of the supporting European or International Standards might not be available by the time this Prestandard is issued. It is therefore anticipated that a National Application Document (NAD) giving any substitute definitive values for safety elements, referencing compatible supporting standards and providing guidance on the national application of this Prestandard, will be issued by each member country or its Standards Organization.

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

Matters specific to this Prestandard

(16) The Parts of ENV 1993 that are currently envisaged are:

ENV 1993-1-1	General rules and rules for buildings;
ENV 1993-1-2	Supplementary rules for structural fire design;
ENV 1993-1-3	Supplementary rules for cold formed thin gauge members and sheeting;
ENV 1993-1-4	Supplementary rules for stainless steels;
ENV 1993-2	Bridges and plated structures;
ENV 1993-3	Towers, masts and chimneys;
ENV 1993-4	Silos, tanks and pipelines;
ENV 1993-5	Piling;
ENV 1993-6	Crane supporting structures;
ENV 1993-7	Marine and maritime structures;
ENV 1993-8	Agricultural structures.

(17) Work on this Part 1.3 of Eurocode 3 was initiated by the Commission of the European Communities. The work was carried out in collaboration with a working group of the European Convention for Constructional Steelwork (ECCS) and a draft was issued in 1990 as a "Draft Eurocode 3 : Annex A".

(18) With the transfer of work on Structural Eurocodes to CEN, the responsibility for completing this document passed to CEN Technical Committee CEN/TC 250, sub-committee CEN/TC 250/SC 3.

(19) In this Part 1.3 of Eurocode 3, a distinction is made between three construction classes using cold formed thin gauge members and sheeting. The boxed values of the partial factors given in Part 1.3 are recommended values for Construction Class I and Construction Class II.

1 General

1.1 Scope

(1)P This Part 1.3 of ENV 1993 deals with the design of steel structures comprising cold formed thin gauge members and sheeting. It gives supplementary provisions for structural applications using cold formed steel products made from coated or uncoated thin gauge hot or cold rolled sheet or strip. It is intended to be used for the design of buildings and civil engineering works in conjunction with ENV 1993-1-1.

NOTE: The execution of steel structures comprised of cold formed thin gauge members and sheeting is covered in ENV 1090-2.

(2)P Methods are given for determining the load bearing capacity and serviceability of elements and connections under predominantly static loads. These design methods apply to steel members and profiled steel sheets that have been cold formed by such processes as cold-rolled forming or press-braking. They are also applicable for the design of profiled steel sheeting for composite steel and concrete slabs at the construction stage, see ENV 1994-1-1.

(3)P Methods are also given for stressed-skin design using steel sheeting as a structural diaphragm.

(4)P This Part 1.3 of ENV 1993 gives methods for design by calculation and for design assisted by testing.

NOTE: In the field of cold formed members and sheeting, many standard products are commonly used for which design by calculation might not lead to economical solutions, so it is frequently desirable to use design assisted by testing. Appropriate test methods are given in annex A.

(5) The methods for design by calculation apply only within stated ranges of material properties and geometrical proportions for which sufficient experience and test evidence is available. These limitations do not apply to design assisted by testing.

(6) This Part does not apply to cold formed structural hollow sections, for which reference should be made to ENV 1993-1-1.

1.2 Distinction between principles and application rules

(1)P Depending on the character of the individual paragraphs, a distinction is made in this Part between principles and application rules.

(2)P The principles comprise:

- general or definitive statements for which there is no alternative;
- requirements and analytical models for which no alternative is permitted unless specifically stated.

(3) The principles are identified by the letter P following the paragraph number.

(4)P The application rules are generally recognized rules that follow the principles and satisfy their requirements. Alternative design rules different from the application rules given in the Eurocode may be used, provided that it is shown that the alternative rule accords with the relevant principles and has at least the same reliability.

(5) In this Part the application rules are identified by a number in brackets, as in this paragraph.

1.3 Normative references

This European Prestandard incorporates, by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Prestandard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies.

EN 10002 *Metallic materials - Tensile testing:*

Part 1: *Method of test (at ambient temperature)*;

EN 10025 *Hot rolled products of non-alloy structural steels - Technical delivery conditions*;

EN 10113 *Hot rolled products in weldable fine grain structural steels:*

Part 2: *Delivery conditions for normalized/normalized rolled steels*;

Part 3: *Delivery conditions for thermomechanical rolled steels*;

EN 10143 *Continuously hot-dip metal coated steel sheet and strip - Tolerances on dimensions and shape*;

EN 10147 *Specification for continuously hot-dip zinc coated structural steel sheet - Technical delivery conditions*;

prEN 10149 *Hot rolled flat products made of high yield strength steels for cold forming:*

Part 2: *Delivery conditions for normalized/normalized rolled steels*;

Part 3: *Delivery conditions for thermomechanical rolled steels*;

EN 10155 *Structural steels with improved atmospheric corrosion resistance - Technical delivery conditions*;

ENV 1090 *Execution of steel structures:*

Part 2: *Rules for cold formed thin gauge members and sheeting*;

ENV 1991 *Eurocode 1: Basis of design and actions on structures:*

Part 1: *Basis of design*;

ENV 1993 *Eurocode 3: Design of steel structures:*

Part 1.1: *General rules : General rules and rules for buildings*;

ENV 1994 *Eurocode 4: Design of composite steel and concrete structures:*

Part 1.1: *General rules : General rules and rules for buildings*;

ISO 1000 *SI units*;

ISO 4997 *Cold reduced steel sheet of structural quality*.

1.4 Definitions

Supplementary to ENV 1993-1-1, for the purposes of this Part 1.3 of ENV 1993, the following definitions apply:

1.4.1 basic material: The flat sheet steel material out of which cold formed sections and profiled sheets are made by cold forming.

1.4.2 basic yield strength: The tensile yield strength of the basic material.

1.4.3 diaphragm action: Structural behaviour involving in-plane shear in the sheeting.

1.4.4 liner tray: Profiled sheet with large lipped edge stiffeners, suitable for interlocking with adjacent liner trays to form a plane of ribbed sheeting that is capable of supporting a parallel plane of profiled sheeting spanning perpendicular to the span of the liner trays.

1.4.5 partial restraint: Restriction of the lateral or rotational movement, or the torsional or warping deformation, of a member or plane element, that increases its buckling resistance in a similar way to a spring support, but to a lesser extent than a rigid support.

1.4.6 relative slenderness: A normalized slenderness ratio.

NOTE: In ENV 1993-1-1 relative slenderness is termed "non-dimensional slenderness".

1.4.7 restraint: Restriction of the lateral or rotational movement, or the torsional or warping deformation, of a member or plane element, that increases its buckling resistance to the same extent as a rigid support.

1.4.8 stressed-skin design: A design method that allows for the contribution made by diaphragm action in the sheeting to the stiffness and strength of a structure.

1.4.9 support: A location at which a member is able to transfer forces or moments to a foundation, or to another member or other structural component.

1.5 Symbols

(1) In addition to those given in ENV 1993-1-1, the following main symbols are used:

C Rotational spring stiffness;

K Linear spring stiffness;

θ Rotation.

(2) In addition to those given in ENV 1993-1-1, the following subscripts are used:

d Developed;

red Reduced;

spn Span;

sup Support;

TF Torsional-flexural.

(3) In addition to those used in ENV 1993-1-1, the following major symbols are used:

b_p Notional flat width of plane element;

h_w Web height, measured between system lines of flanges;

s_w Slant height of a web, measured between midpoints of corners.

(4) Further symbols are defined where they first occur.

1.6 Units

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

(2) The following units are recommended for use in calculations:

- forces and loads: kN, kN/m, kN/m^2;
- unit mass: kg/m^3;
- unit weight: kN/m^3;
- stresses and strengths: N/mm^2 ($\equiv$ MN/m^2 or MPa);
- bending moments: kNm;
- torsional moments: kNm.

1.7 Terminology

1.7.1 Form of sections

(1) Cold formed members and profiled sheets are steel products made from coated or uncoated hot rolled or cold rolled flat products. Within the permitted tolerances, they have a constant thickness over their entire length and may have either a constant or a variable cross-section.

NOTE: These products are obtained solely by cold forming, for example profiled by a rolling machine or formed by a press or press brake.

(2) The cross-sections of cold formed members and profiled sheets essentially comprise a number of curved elements joined by plane elements.

(3) Typical forms of sections for cold formed members include:

- single open sections as shown in figure 1.1(a);
- open built-up sections as shown in figure 1.1(b);
- closed built-up sections as shown in figure 1.1(c).

(4) Examples of cross-sections for cold formed members and sheets are illustrated as follows:

- compression members and tension members, in figure 1.2(a);
- beams and other members subject to bending, in figure 1.2(b);
- profiled sheets and liner trays, in figure 1.2(c).

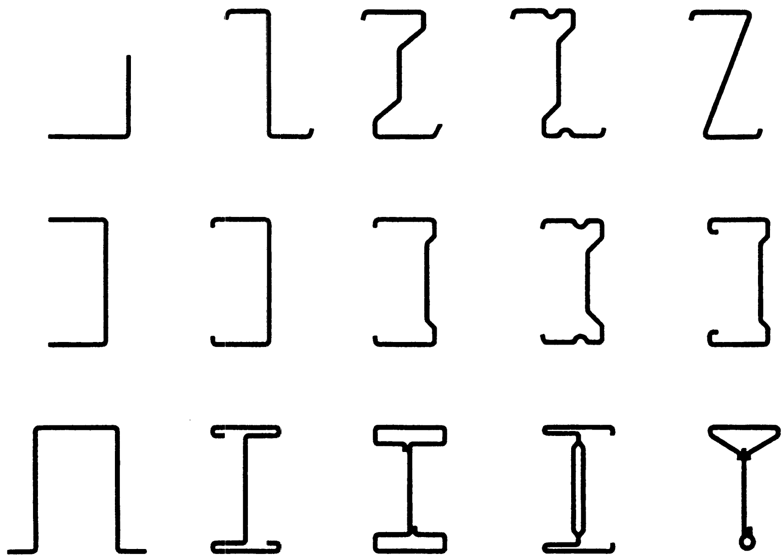
(5) Cross-sections of cold formed members and sheets can either be unstiffened or incorporate longitudinal stiffeners in their webs or flanges, or in both.

1.7.2 Form of stiffeners

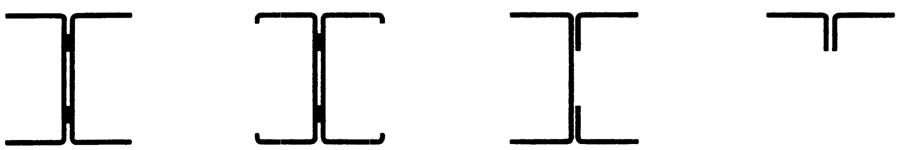
(1) Typical forms of stiffeners for cold formed members and sheets include:

- folds and bends, see figure 1.3(a);
- folded or curved grooves, see figure 1.3(b);
- other sections, bolted, rivetted or welded on, see figure 1.3(c).

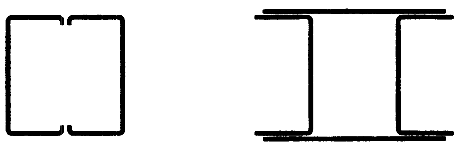
(2) Longitudinal flange stiffeners can be either edge stiffeners or intermediate stiffeners.



a) Single open sections



b) Open built-up sections



c) Closed built-up sections

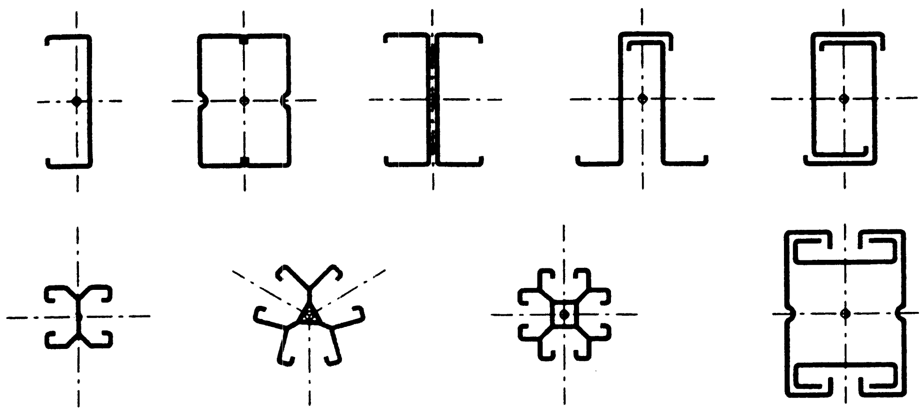
Figure 1.1: Typical forms of sections for cold formed members

(3) Typical edge stiffeners include:

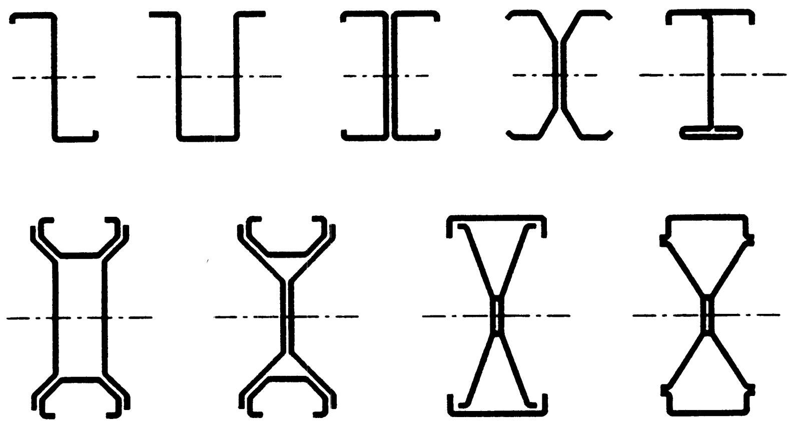
- single edge fold stiffeners or lips, see figure 1.4(a);
- double edge fold stiffeners, see figure 1.4(b).

(4) Typical intermediate longitudinal stiffeners are illustrated as follows:

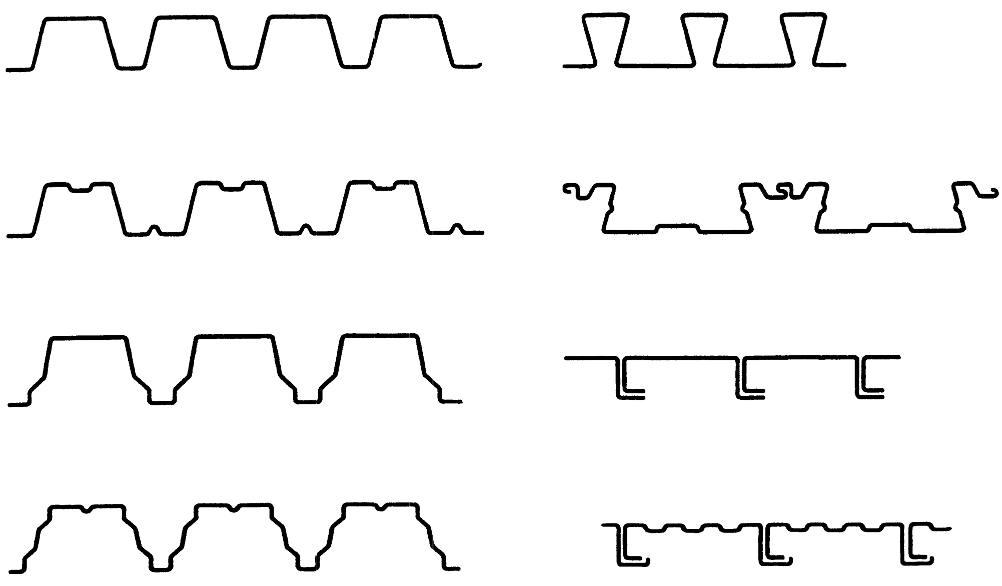
- for flanges, in figure 1.5(a);
- for webs in figure 1.5(b).



a) Compression members and tension members

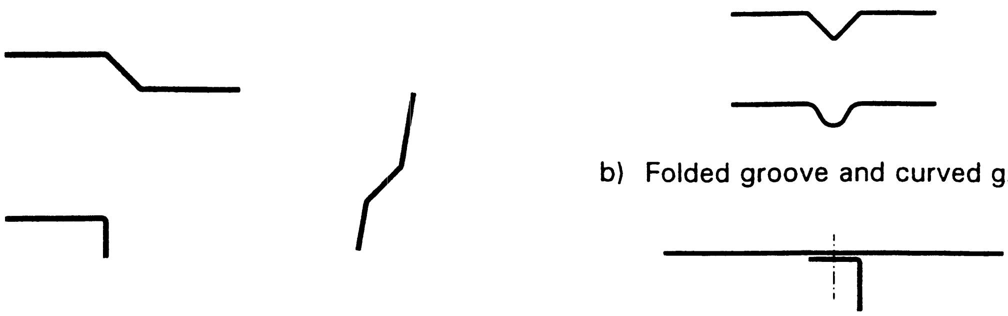


b) Beams and other members subject to bending



c) Profiled sheets and liner trays

Figure 1.2: Examples of cold formed members and profiled sheets



b) Folded groove and curved groove

a) Folds and bends

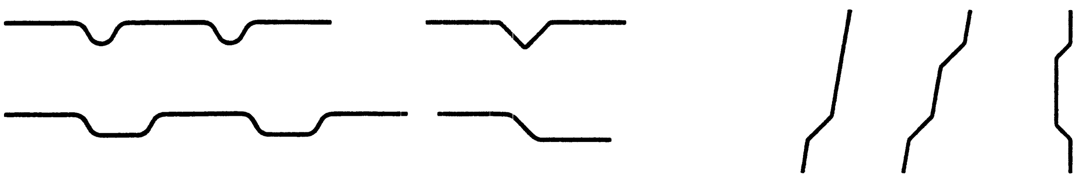
c) Bolted angle stiffener

Figure 1.3: Typical forms of stiffeners for cold formed members and sheeting

a) Single edge fold stiffeners

b) Double edge fold stiffeners

Figure 1.4: Typical edge stiffeners



a) Intermediate flange stiffeners

b) Intermediate web stiffeners

Figure 1.5: Typical intermediate longitudinal stiffeners

1.7.3 Cross-section dimensions

(1) Overall dimensions of cold formed thin gauge members and sheeting, including overall width b, overall height h, internal bend radius r and other external dimensions denoted by symbols without subscripts, such as a, c or d, are measured to the face of the material, unless stated otherwise.

(2) Unless stated otherwise, the other cross-sectional dimensions of cold formed thin gauge members and sheeting, denoted by symbols with subscripts, such as b_d, h_w or s_w, are measured either to the midline of the material, the system line of the element or the midpoint of the corner.

(3) In the case of sloping elements, such as webs of trapezoidal profiled sheets, the slant height s is measured parallel to the slope.

(4) The developed height of a web is measured along its midline, including any web stiffeners.

(5) The developed width of a flange is measured along its midline, including any intermediate stiffeners.

1.7.4 Convention for member axes

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

- x - x along the member;
- y - y axis of the cross-section;
- z - z axis of the cross-section.

(2) For cold formed steel members the following axis convention is used in this Part 1.3 of ENV 1993:

- for monosymmetric cross-sections:
  - y - y the axis of symmetry of the cross-section;
  - z - z the other principal axis of the cross-section;
- otherwise:
  - y - y major axis;
  - z - z minor axis.
- where necessary:
  - u - u axis perpendicular to the height (if this does not coincide with y - y or z - z);
  - v - v axis parallel to the height (if this does not coincide with y - y or z - z).

NOTE: This differs from the axis convention used in ENV 1993-1-1 in order to give rules for torsional-flexural buckling that can be applied consistently to all cross-sections.

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(3) The use of u - u and v - v axes is illustrated in figure 1.6.

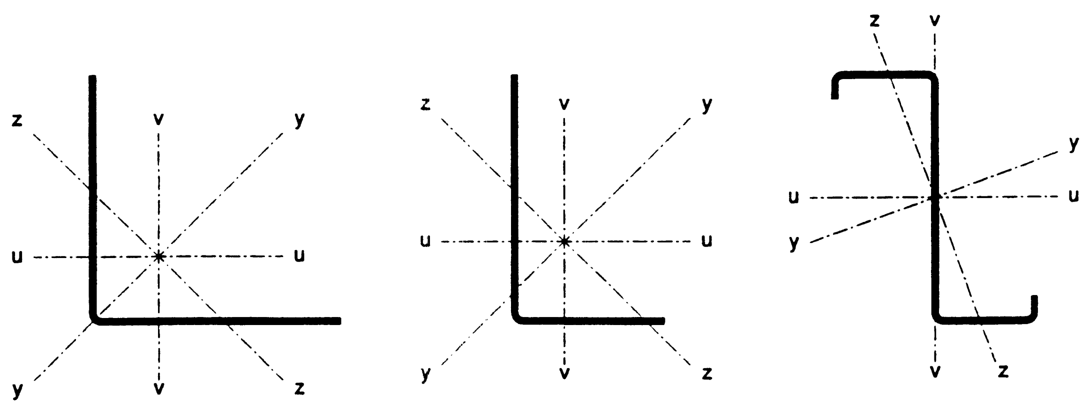

Figure 1.6: Axis convention

(4) For profiled sheets and liner trays the following axis convention is used in this Part 1.3 of ENV 1993:

- y - y axis parallel to the plane of sheeting;
- z - z axis perpendicular to the plane of sheeting.

2 Basis of design

2.1 General

(1)P For the purpose of differentiating levels of reliability, a distinction may be made between three "construction classes" defined as follows:

- **Construction Class I**: Construction where cold formed thin gauge members and sheeting are designed to contribute to the overall strength and stability of a structure;
- **Construction Class II**: Construction where cold formed thin gauge members and sheeting are designed to contribute to the strength and stability of individual structural elements;
- **Construction Class III**: Construction where cold formed sheeting is used as an element that only transfers loads to the structure.

(2)P The methods for design by calculation and for design assisted by testing given in this Part 1.3 of ENV 1993 may be adopted for all construction classes.

(3)P Appropriate partial factors shall be adopted for ultimate limit states and serviceability limit states.

(4)P The values of the partial factors given in this Part 1.3 of ENV 1993 shall be adopted for Construction Class I and Construction Class II.

2.2 Ultimate limit states

(1)P The principles for ultimate limit states given in Sections 2 and 5 in Part 1.1 of ENV 1993 shall also be applied to cold formed thin gauge members and sheeting.

(2) The application rules for ultimate limit states given in Sections 2 and 4 of Part 1.1 of ENV 1993 should also be applied, except where different application rules are given in this Part 1.3.

(3)P For verifications by calculation at ultimate limit states the partial factor γ_M shall be taken as follows:

- resistance of cross-sections where failure is caused by yielding:

$$\gamma_{M0} = \boxed{1{,}1}$$

- resistance of members and sheeting where failure is caused by buckling:

$$\gamma_{M1} = \boxed{1{,}1}$$

- resistance of net sections at bolt holes:

$$\gamma_{M2} = \boxed{1{,}25}$$

(4)P For values of γ_M for resistances of connections, see Section 8 of this Part 1.3.

2.3 Serviceability limit states

(1)P The principles for serviceability limit states given in Sections 2 and 4 of Part 1.1 of ENV 1993 shall also be applied to cold formed thin gauge members and sheeting, see 7.1(1)P in this Part 1.3.

(2) The application rules for serviceability limit states given in Sections 2 and 4 of Part 1.1 of ENV 1993 should also be applied, except where different application rules are given in Section 7 of this Part 1.3.

(3)P For verifications at serviceability limit states the partial factor $\gamma_{M,ser}$ shall be taken as follows:

$$\gamma_{M,ser} = \boxed{1,0}$$

2.4 Design assisted by testing

(1)P Verifications for ultimate limit states or serviceability limit states that rely on the results of testing shall be in accordance with Section 9.

(2) Test specimens for sheet profiles should normally comprise at least two complete corrugations, but a test specimen may comprise just one complete corrugation, provided that the stiffness of the corrugations is sufficient.

2.5 Durability

(1)P To ensure adequate durability of cold formed components under conditions relevant to both their intended use and their intended life, the following inter-related factors shall be taken into account at the design stage:

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

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

(3) Special attention should be given to cases in which different materials are intended to act compositely, if these materials are such that electrochemical phenomena might produce conditions leading to corrosion.

(4) The environmental conditions prevailing from the time of manufacture, including those during transport and storage on site, should be taken into account.

3 Properties of materials and cross-sections

3.1 Structural steel

3.1.1 General

(1)P All steels used for cold formed members and profiled sheets shall be suitable for cold forming and welding. Steels used for members and sheets to be galvanized shall also be suitable for galvanizing.

(2)P The methods for design by calculation given in this Part 1.3 of ENV 1993 may be used for structural steels conforming with the European Standards and International Standards listed in table 3.1.

Table 3.1: Nominal values of basic yield strength f_{yb} and ultimate tensile strength f_u

Type of steel	Standard	Grade	f_{yb} N/mm^2	f_u N/mm^2
Hot rolled steel sheet of structural quality	EN 10025	S 235 S 275 S 355	235 275 355	360 430 510
Hot rolled steel sheet of high yield strength of structural quality	EN 10113: Part 2	S 275 N S 355 N S 420 N S 460 N	275 355 420 460	370 470 520 550
	EN 10113: Part 3	S 275 M S 355 M S 420 M S 460 M	275 355 420 460	360 450 500 530
Cold reduced steel sheet of structural quality	ISO 4997	CR 220 CR 250 CR 320	220 250 320	300 330 400
Continuous hot dip zinc coated carbon steel sheet of structural quality	EN 10147	Fe E 220 G Fe E 250 G Fe E 280 G Fe E 320 G Fe E 350 G	220 250 280 320 350	300 330 360 390 420
High yield strength steels for cold forming	prEN 10149: Part 2	S 315 MC S 355 MC S 420 MC S 460 MC S 500 MC S 550 MC	315 355 420 460 500 550	390 430 480 520 550 600
	prEN 10149: Part 3	S 260 NC S 315 NC S 355 NC S 420 NC	260 315 355 420	370 430 470 530

(3)P These design methods may also be applied to other structural steels with similar strength and toughness properties, provided that all of the following conditions are satisfied:

a) the steel satisfies the requirements for chemical analysis, mechanical tests and other control procedures to the extent and in the manner prescribed in the standards that are listed in table 3.1;

b) the ratio of the specified minimum ultimate tensile strength f_u to the specified minimum basic yield strength f_{yb} is not less than 1,2;

c) the steel is supplied either:

- to another recognized standard for structural steel sheet;
- with mechanical properties and chemical composition at least equivalent to one of the steel grades that are listed in table 3.1.

(4)P The nominal values of the basic yield strength f_{yb} and ultimate tensile strength f_u given in table 3.1 shall be adopted as characteristic values in design calculations. For other steels the characteristic values shall be based on the results of tensile tests carried out in accordance with EN 10002-1.

(5) It should be assumed that the properties of steel in compression are the same as those in tension.

(6)P Where the yield strength is specified using the symbol f_y the average yield strength f_{ya} may be used, if the conditions given in 3.1.2 are satisfied, otherwise the basic yield strength f_{yb} shall be used. Where the yield strength is specified using the symbol f_{yb} the basic yield strength f_{yb} shall be used.

(7)P For the steels covered by this Part 1.3 of ENV 1993, the other material properties to be used in design shall be taken as follows:

- modulus of elasticity: $E = 210\,000\ \text{N/mm}^2$;
- shear modulus: $G = E/2(1+\nu)\ \text{N/mm}^2$;
- Poisson's ratio: $\nu = 0{,}3$;
- coefficient of linear thermal elongation: $\alpha = 12 \times 10^{-6}\ 1/\text{K}$;
- unit mass $\rho = 7850\ \text{kg/m}^3$.

3.1.2 Average yield strength

(1)P The average yield strength f_{ya} of a cross-section due to cold working may be determined from the results of full size tests in accordance with Section 9.

(2)P Alternatively the increased average yield strength f_{ya} may be determined by calculation using:

$$f_{ya} = f_{yb} + (f_u - f_{yb})knt^2/A_g \quad \text{but} \quad f_{ya} \leq (f_u + f_{yb})/2 \qquad \ldots (3.1)$$

where:

A_g is the gross cross-sectional area;

k is a numerical coefficient that depends on the type of forming as follows:

- $k = 7$ for cold rolling;
- $k = 5$ for other methods of forming;

n is the number of 90° bends in the cross-section with an internal radius $r \leq 5t$ (fractions of 90° bends should be counted as fractions of n);

t is the nominal core thickness t_{cor} of the steel material before cold forming, exclusive of zinc or organic coatings, see 3.1.3.

(3)P The increased yield strength due to cold forming shall be taken into account only as follows:

- in axially loaded members in which the effective cross-sectional area A_{eff} equals the gross area A_g;
- in other cases in which it can be shown that the effects of cold forming lead to an increase in the load carrying capacity.

(4) In determining A_{eff} the yield strength f_y should be taken as f_{yb}.

(5) The average yield strength f_{ya} may be utilized in determining:

- the cross-section resistance of an axially loaded tension member;
- the cross-section resistance and the buckling resistance of an axially loaded compression member with a fully effective cross-section;
- the moment resistance of a cross-section with fully effective flanges.

(6) To determine the moment resistance of a cross-section with fully effective flanges, the cross-section may be subdivided into m nominal plane elements, such as flanges. Expression (3.1) may then be used to obtain values of increased yield strength $f_{y,i}$ separately for each nominal plane element i, provided that:

$$\sum_{i=1}^{m} A_{g,i} f_{y,i} / \sum_{i=1}^{m} A_{g,i} \leq f_{ya} \qquad \text{... (3.2)}$$

where:

$A_{g,i}$ is the gross cross-sectional area of nominal plane element i.

(7)P The increase in yield strength due to cold forming shall not be utilized for members that are subjected to heat treatment after forming at more than 520°C for more than one hour.

(8) Special attention should be paid to the fact that some heat treatments (especially annealing) might induce a reduced yield strength lower than the basic yield strength f_{yb}.

3.1.3 Thickness

(1)P The provisions for design by calculation given in this Part 1.3 of ENV 1993 may be used only for steel within the following ranges of nominal core thickness t_{cor} exclusive of zinc or organic coatings:

- for sheeting: 0,5 mm $\leq t_{cor} \leq$ 4,0 mm;
- for members: 1,0 mm $\leq t_{cor} \leq$ 8,0 mm.

(2)P Thicker or thinner material may also be used, provided that the load bearing capacity is determined by design assisted by testing in accordance with Section 9.

(3)P Because the design provisions for cold formed members and sheeting given in this Part 1.3 have been developed on the basis of thickness tolerances that are approximately half the tolerance values specified as "normal tolerances" in EN 10143, if larger tolerances are used the nominal values of thickness t_{nom} shall be adjusted to maintain the equivalent reliability.

(4) For continuously hot-dip metal coated members and sheeting of nominal thickness $t_{nom} <$ 1,5 mm supplied with negative tolerances equal to the "special tolerances (S)" given in EN 10143, the design thickness t may be taken as equal to the nominal core thickness t_{cor}.

(5) In the case of continuously hot-dip metal coated steel sheet and strip conforming with EN 10147, the nominal core thickness t_{cor} may be taken as $t_{nom} - t_{zin}$ where t_{nom} is the nominal sheet thickness and t_{zin} is the total thickness of zinc coating, including both surfaces.

NOTE: For the usual Z 275 zinc coating, t_{zin} = 0,04 mm.

3.2 Connecting devices

3.2.1 Bolt assemblies

(1)P Bolts, nuts and washers shall conform to the requirements given in ENV 1993-1-1.

3.2.2 Other types of mechanical fastener

(1)P The following additional types of mechanical fasteners may be used:

- self-tapping screws;
- cartridge-fired pins;
- blind rivets.

(2)P Self tapping screws may be:

- thread-forming self-tapping screws;
- thread-cutting self-tapping screws;
- self-drilling self-tapping screws.

(3) For details concerning suitable self-tapping screws, cartridge-fired pins and blind rivets reference should be made to ENV 1090: Part 2.*)

3.2.3 Welding consumables

(1)P Welding consumables shall conform to the requirements given in ENV 1993-1-1.

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3.3 Section properties

3.3.1 General

(1)P Section properties shall be calculated according to normal good practice, taking due account of the sensitivity of the properties of the overall cross-section to any approximations used, see 3.3.4, and their influence on the predicted resistance of the member.

(2)P The effects of local buckling shall be taken into account by using effective cross-sections as specified in Section 4.

3.3.2 Gross cross-section

(1)P The properties of the gross cross-section shall be determined using the specified nominal dimensions. In calculating gross cross-sectional properties, holes for fasteners need not be deducted but allowance shall be made for large openings. Plates that are used solely in splices or as battens shall not be included.

3.3.3 Net area

(1)P The net area of a member cross-section, or of an element of a cross-section, shall be taken as its gross area minus appropriate deductions for all fastener holes and other openings.

(2)P In deducting holes for fasteners, the nominal hole diameter shall be used, not the fastener diameter.

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*) This document is in preparation.

(3) For countersunk holes, the area to be deducted should be the gross cross-sectional area of the hole, including the countersunk portion, in the plane of its axis.

(4)P Provided that the fastener holes are not staggered, the area to be deducted from the gross cross-sectional area shall be the maximum sum of the sectional areas of the fastener holes in any cross-section perpendicular to the direction of direct stress in the member.

(5)P Where the fastener holes are staggered, the area to be deducted shall be the greater of:

a) The deduction for non-staggered holes given in (4)P;

b) The sum of the sectional areas of all holes in any diagonal or zigzag line extending progressively across the member or element, see figure 3.1, minus an allowance for each gauge space p in the chain of holes. This allowance shall be taken as $0{,}25 s^2 t/p$ but not more than $0{,}6 s t$, where:

- p is the gauge space, i.e. the distance measured perpendicular to the direction of load transfer, between the centres of two consecutive holes in the chain;
- s is the staggered pitch, i.e. the distance, measured parallel to the direction of load transfer, between the centres of the same two holes;
- t is the thickness of the holed material.

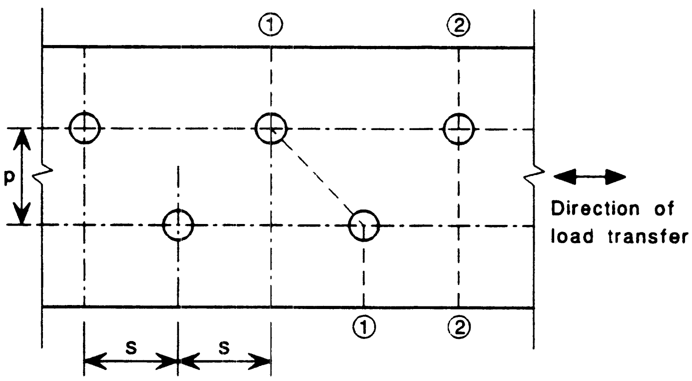


Figure 3.1: Staggered holes and appropriate sections

(6)P For cross-sections such as angles with holes in more than one plane, the spacing p shall be measured along the centre of thickness of the material, see figure 3.2.

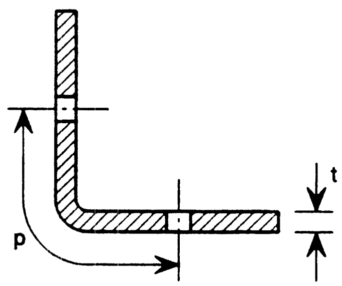


Figure 3.2: Angles with holes in both legs

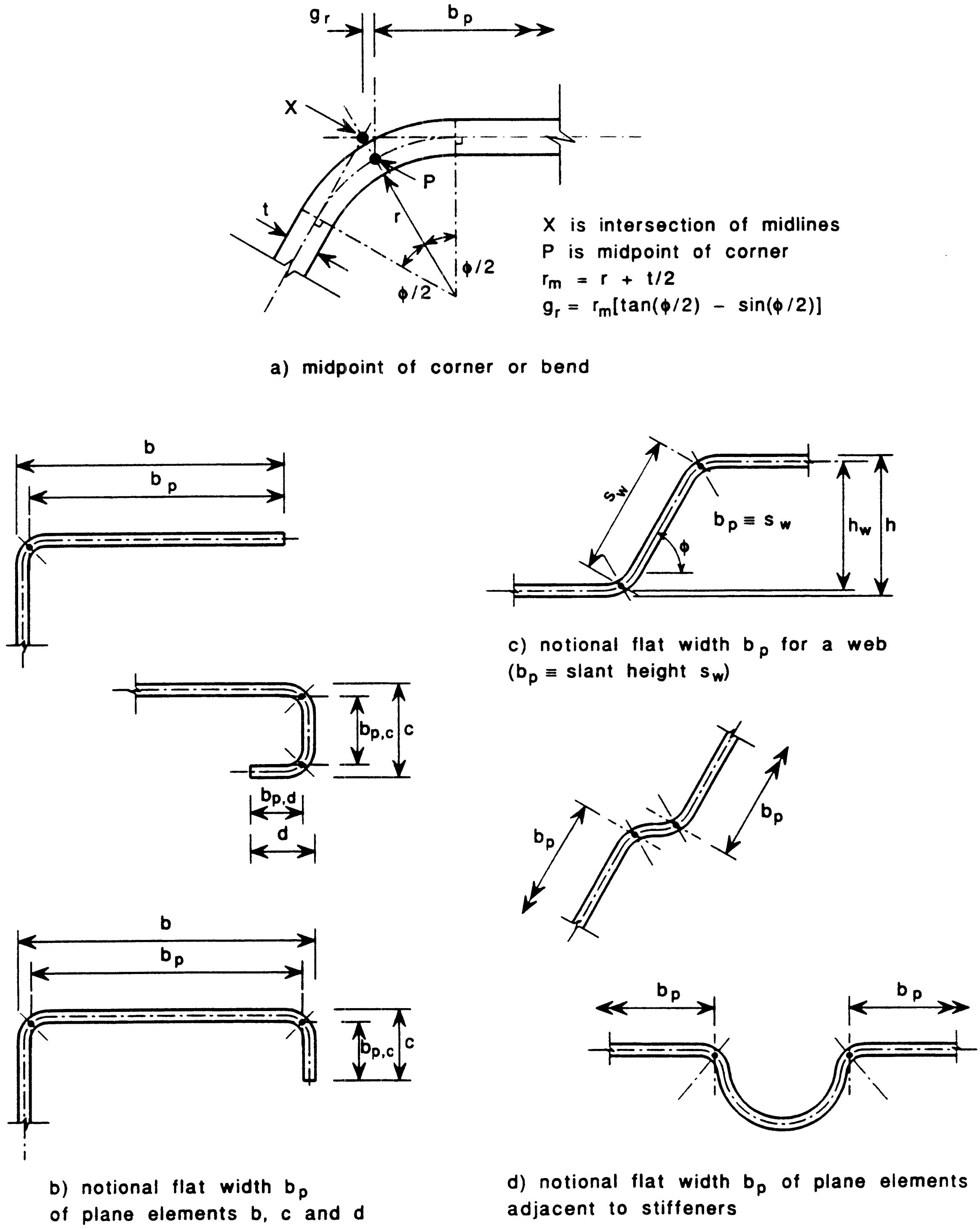
(7)P In a built-up member where the critical chains of holes in each component part do not correspond with the critical chain of holes for the member as a whole, the resistances of any fasteners joining the parts between such chains of holes shall be taken into account in determining the resistance of the member.

NOTE: No general rules can be given for continuously perforated members because the resistance is influenced by the form and pattern of the perforations.

3.3.4 Influence of rounded corners

(1)P In cross-sections with rounded corners, the notional flat widths b_p of the plane elements shall be measured from the midpoints of the adjacent corner elements as indicated in figure 3.3.

(2)P In cross-sections with rounded corners, the calculation of section properties shall be based upon the actual geometry of the cross-section.



a) midpoint of corner or bend

b) notional flat width b_p of plane elements b, c and d

c) notional flat width b_p for a web ($b_p \equiv$ slant height s_w)

d) notional flat width b_p of plane elements adjacent to stiffeners

Figure 3.3: Notional widths of plane elements b_p allowing for corner radii

(3) The influence of rounded corners with internal radius $r \leq 5t$ and $r \leq 0{,}15b_p$ on section properties may be neglected, and the cross-section may be assumed to consist of plane elements with sharp corners.

(4) If the internal radius r exceeds the limits given in (3) the influence of rounded corners on section properties should be allowed for. This may be done with sufficient accuracy by reducing the properties calculated for an otherwise similar cross-section with sharp corners, see figure 3.4, using the following approximations:

$$A_g \approx A_{g,sh}(1 - \delta) \qquad \text{... (3.3a)}$$

$$I_g \approx I_{g,sh}(1 - 2\delta) \qquad \text{... (3.3b)}$$

$$I_w \approx I_{w,sh}(1 - 4\delta) \qquad \text{... (3.3c)}$$

with:

$$\delta = 0{,}43 \sum_{j=1}^{n} r_j \Big/ \sum_{i=1}^{m} b_{p,i} \qquad \text{... (3.3d)}$$

where:

A_g is the area of the gross cross-section;

$A_{g,sh}$ is the value of A_g for a cross-section with sharp corners;

$b_{p,i}$ is the notional flat width of plane element i for a cross-section with sharp corners;

I_g is the second moment of area of the gross cross-section;

$I_{g,sh}$ is the value of I_g for a cross-section with sharp corners;

I_w is the warping constant of the gross cross-section;

$I_{w,sh}$ is the value of I_w for a cross-section with sharp corners;

m is the number of plane elements;

n is the number of curved elements;

r_j is the internal radius of curved element j.

(5) The reductions given by expression (3.3) may also be applied in calculating the effective section properties A_{eff}, $I_{y,eff}$, $I_{z,eff}$ and $I_{w,eff}$, provided that the notional flat widths of the plane elements are measured to the points of intersection of their midlines.

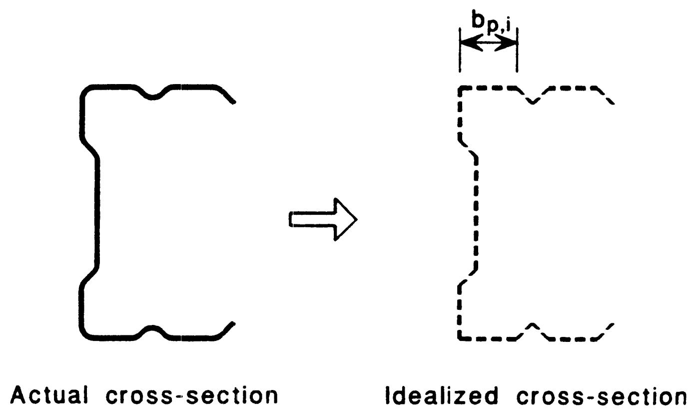


Figure 3.4: Approximate allowance for rounded corners

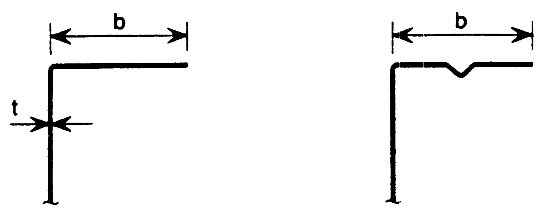
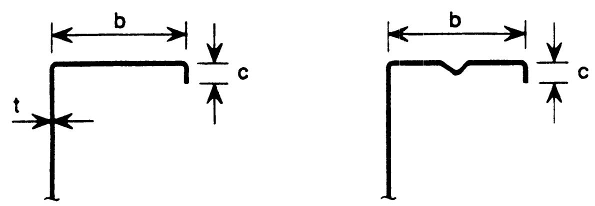
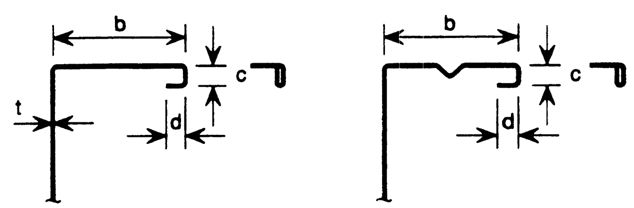
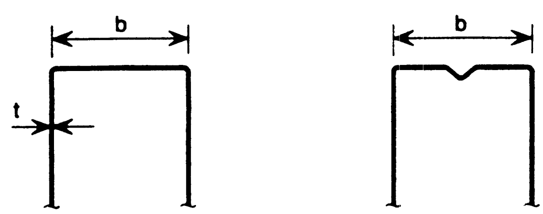
3.4 Geometrical proportions

(1)P The provisions for design by calculation given in this Part 1.3 of ENV 1993 shall not be applied to cross-sections outside the range of width-to-thickness ratios for which sufficient experience and verification by testing is available.

(2) The maximum width-to-thickness ratios b/t and h/t given in table 3.2 may be assumed to represent the field for which sufficient experience and verification by testing is already available.

(3) Cross-sections with larger width-to-thickness ratios may also be used, provided that their resistance at ultimate limit states and their behaviour at serviceability limit states are verified by testing in accordance with Section 9.

Table 3.2: Maximum width-to-thickness ratios

Element of cross-section	Maximum value
	$b/t \leq 50$
	$b/t \leq 60$
	$b/t \leq 90$
	$b/t \leq 500$
	$45° \leq \phi \leq 90°$ $h/t \leq 500 \sin \phi$

(4) In order to provide sufficient stiffness and to avoid primary buckling of the stiffener itself, the sizes of stiffeners should be within the following ranges:

$$0{,}2 \leq c/b \leq 0{,}6 \qquad \text{... (3.4a)}$$

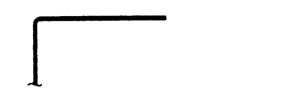
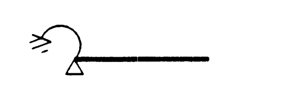
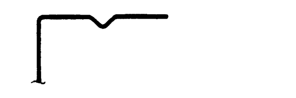
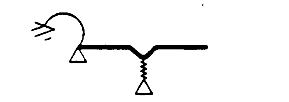
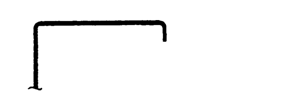
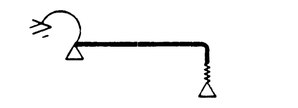
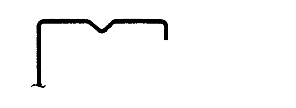
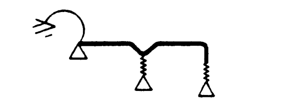
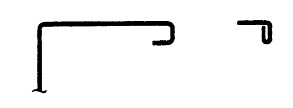
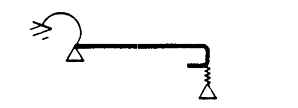
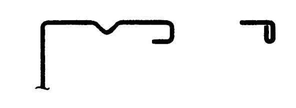
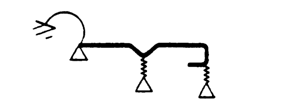
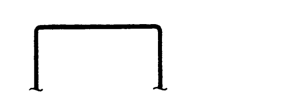
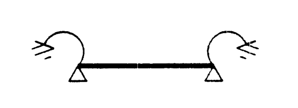
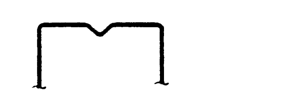
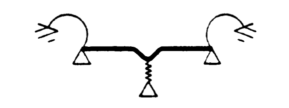
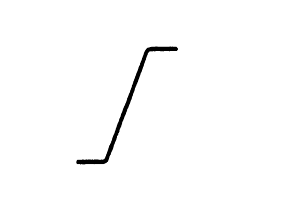
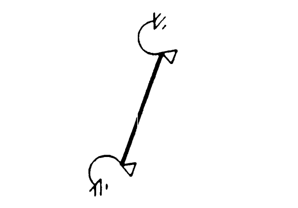
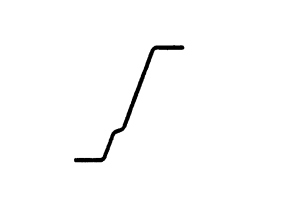
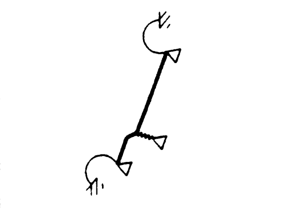
$$0{,}1 \leq d/b \leq 0{,}3 \qquad \text{... (3.4b)}$$

in which the dimensions b, c and d are as indicated in table 3.2.

3.5 Modelling for cross-section analysis

(1) The elements of a cross-section may be modelled for analysis as indicated in table 3.3.

Table 3.3: Modelling of elements of a cross-section

Type of element	Model	Type of element	Model
			
			
			
			
			

4 Local buckling

4.1 General

(1)P The effects of local buckling shall be taken into account in determining the resistance and stiffness of cold formed members and sheeting.

(2)P This may be done by using effective cross-sectional properties, calculated on the basis of the effective widths of those elements that are prone to local buckling.

(3)P The possible shift of the centroidal axis of the effective cross-section relative to the centroidal axis of the gross cross-section shall be taken into account.

(4) In determining resistance to local buckling, the yield strength f_y should be taken as f_{yb}.

(5) In determining the resistance of a cross-section, the effective width of a compression element should be based on the compressive stress $\sigma_{com.Ed}$ in the element when the cross-section resistance is reached.

(6) For serviceability verifications, the effective width of a compression element should be based on the compressive stress $\sigma_{com,Ed,ser}$ in the element under the serviceability limit state loading.

4.2 Plane elements without stiffeners

(1)P The effective widths of compression elements shall be obtained from table 4.1 for doubly supported compression elements or table 4.2 for outstand compression elements.

(2)P The notional flat width b_p of a plane element shall be determined as specified in 3.3.4. In the case of plane elements in a sloping web, the appropriate slant height shall be used.

NOTE: In ENV 1993-1-1 the symbol $\bar{b}$ is used for the notional flat width of a plane element.

(3)P The reduction factor ρ used in tables 4.1 and 4.2 to determine b_{eff} shall be based on the largest compressive stress $\sigma_{com,Ed}$ in the relevant element (calculated on the basis of the effective cross-section and taking account of possible second order effects), when the resistance of the cross-section is reached.

(4) If $\sigma_{com,Ed} = f_{yb}/\gamma_{M1}$ the reduction factor ρ should be obtained from the following:

- if $\bar{\lambda}_p \leq 0{,}673$: $\rho = 1{,}0$ ... (4.1a)
- if $\bar{\lambda}_p > 0{,}673$: $\rho = (1{,}0 - 0{,}22/\bar{\lambda}_p)/\bar{\lambda}_p$ ... (4.1b)

in which the plate slenderness $\bar{\lambda}_p$ is given by:

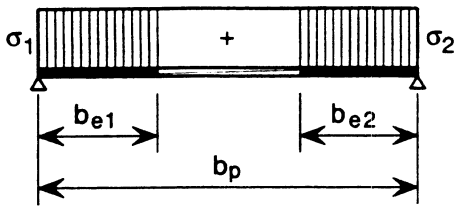
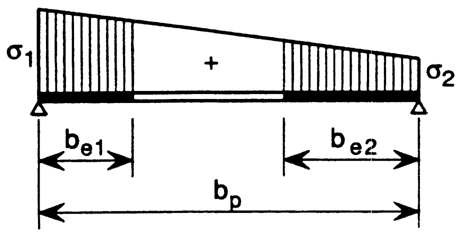
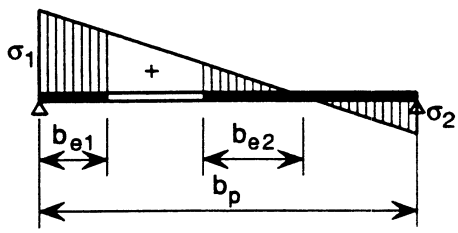
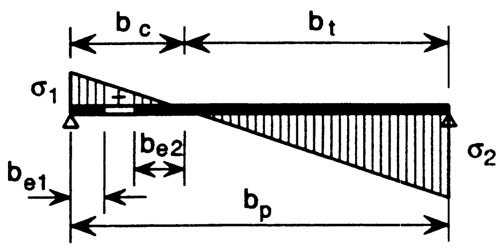
$$\bar{\lambda}_p = \sqrt{\frac{f_{yb}}{\sigma_{cr}}} \equiv \frac{b_p}{t}\sqrt{\frac{12(1-\nu^2)f_{yb}}{\pi^2 E k_\sigma}} \cong 1{,}052\,\frac{b_p}{t}\sqrt{\frac{f_{yb}}{E k_\sigma}} \cong \frac{b_p/t}{28{,}4\,\varepsilon\sqrt{k_\sigma}} \quad \text{... (4.2)}$$

where:

k_σ is the relevant buckling factor from table 4.1 or 4.2;

ε is the ratio $\sqrt{235/f_{yb}}$ with f_{yb} in N/mm^2.

Table 4.1: Doubly supported compression elements

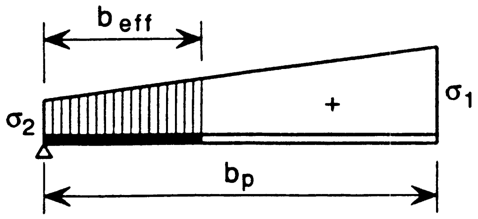
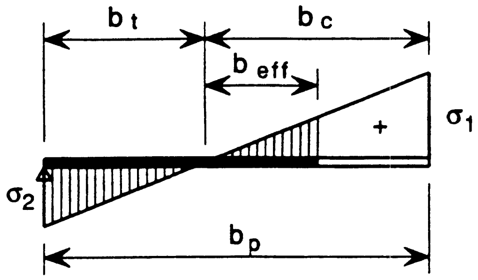
Stress distribution [compression positive]	Effective width b_{eff}
	**$\psi = +1$:** $b_{eff} = \rho b_p$ $b_{e1} = 0{,}5 b_{eff}$ $b_{e2} = 0{,}5 b_{eff}$
	**$+1 > \psi \geq 0$:** $b_{eff} = \rho b_p$ $b_{e1} = \dfrac{2 b_{eff}}{5 - \psi}$ $b_{e2} = b_{eff} - b_{e1}$
	**$0 > \psi \geq -1$:** $b_{eff} = \rho b_c$ $b_{e1} = 0{,}4 b_{eff}$ $b_{e2} = 0{,}6 b_{eff}$
	**$\psi < -1$:** $b_{eff} = \rho b_c$ $b_{e1} = 0{,}4 b_{eff}$ $b_{e2} = 0{,}6 b_{eff}$

$\psi = \sigma_2 / \sigma_1$	+1	$+1 > \psi > 0$	0	$0 > \psi > -1$	-1	$-1 > \psi > -3$
Buckling factor k_σ	4,0	$\dfrac{8{,}2}{1{,}05 + \psi}$	7,81	$7{,}81 - 6{,}29\psi + 9{,}78\psi^2$	23,9	$5{,}98(1 - \psi)^2$

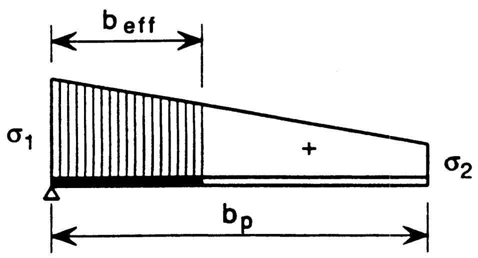
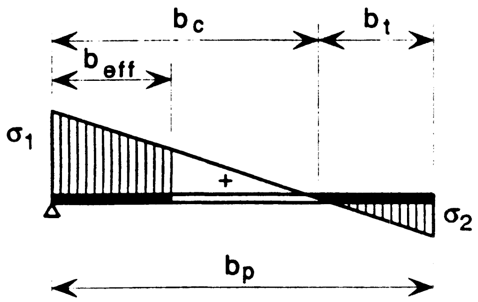
Alternatively, for $+1 \geq \psi \geq -1$:

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

Table 4.2: Outstand compression elements

Stress distribution [compression positive]	Effective width b_{eff}
	**$+1 > \psi \geq 0$:** $b_{eff} = \rho b_p$
	**$\psi < 0$:** $b_{eff} = \rho b_c$

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

Stress distribution [compression positive]	Effective width b_{eff}
	**$+1 > \psi \geq 0$:** $b_{eff} = \rho b_p$
	**$\psi < 0$:** $b_{eff} = \rho b_c$

$\psi = \sigma_2 / \sigma_1$	+1	$+1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$\dfrac{0{,}578}{\psi + 0{,}34}$	1,70	$1{,}70 - 5\psi + 17{,}1\psi^2$	23,8

(5) If $\sigma_{com,Ed} < f_{yb}/\gamma_{M1}$ the reduction factor ρ should be determined as follows:

- **Alternative 1**: Use expressions (4.1a) and (4.1b) but replace the plate slenderness $\bar{\lambda}_p$ by the reduced plate slenderness $\bar{\lambda}_{p,red}$ given by:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\frac{\sigma_{com,Ed}}{f_{yb}/\gamma_{M1}}} \quad \ldots (4.3)$$

- **Alternative 2**: Replace expressions (4.1a) and (4.1b) by expressions (4.4a) and (4.4b) as follows:
  - if $\bar{\lambda}_{p,red} \leq 0{,}673$: $\rho = 1{,}0$ ... (4.4a)
  - if $\bar{\lambda}_{p,red} > 0{,}673$:

$$\rho = \frac{1 - 0{,}22/\bar{\lambda}_{p,red}}{\bar{\lambda}_{p,red}} + 0{,}18\frac{\bar{\lambda}_p - \bar{\lambda}_{p,red}}{\bar{\lambda}_p - 0{,}6} \quad \text{but} \quad \rho \leq 1{,}0 \quad \ldots (4.4b)$$

(6) For effective widths at serviceability limit states ρ should be determined as follows:

- **Alternative 1**: Use expressions (4.1a) and (4.1b) but replace the ultimate limit states plate slenderness $\bar{\lambda}_p$ by the serviceability limit states plate slenderness $\bar{\lambda}_{p,ser}$ given by:

$$\bar{\lambda}_{p,ser} = \bar{\lambda}_p \sqrt{\frac{\sigma_{com,Ed,ser}}{f_{yb}}} \quad \ldots (4.5)$$

where:

$\sigma_{com,Ed,ser}$ is the largest compressive stress in the relevant element (calculated on the basis of the effective cross-section) under the serviceability limit state loading.

- **Alternative 2**: Use expressions (4.4a) and (4.4b) but replace the reduced plate slenderness $\bar{\lambda}_{p,red}$ by the serviceability limit states plate slenderness $\bar{\lambda}_{p,ser}$ from expression (4.5).

(7) In determining the effective width of a flange element subject to stress gradient, the stress ratio ψ used in tables 4.1 and 4.2 may be based on the properties of the gross cross-section.

(8) In determining the effective width of a web element the stress ratio ψ used in table 4.1 may be obtained using the effective area of the compression flange but the gross area of the web.

(9) Optionally the effective section properties may be refined by repeating (7) and (8) iteratively, but using the effective cross-section already found in place of the gross cross-section.

(10) In the case of webs of trapezoidal profiled sheets under stress gradient, the simplified method given in 4.3.4 may be used.

4.3 Plane elements with edge or intermediate stiffeners

4.3.1 General

(1)P The design of compression elements with edge or intermediate stiffeners shall be based on the assumption that the stiffener behaves as a compression member with continuous partial restraint, with a spring stiffness that depends on the boundary conditions and the flexural stiffness of the adjacent plane elements.

(2) The spring stiffness of a stiffener should be determined by applying a unit load per unit length u as illustrated in figure 4.1. The spring stiffness K per unit length may be determined from:

$$K = u/\delta \qquad \dots (4.6)$$

where:

δ is the deflection of the stiffener due to the unit load u.

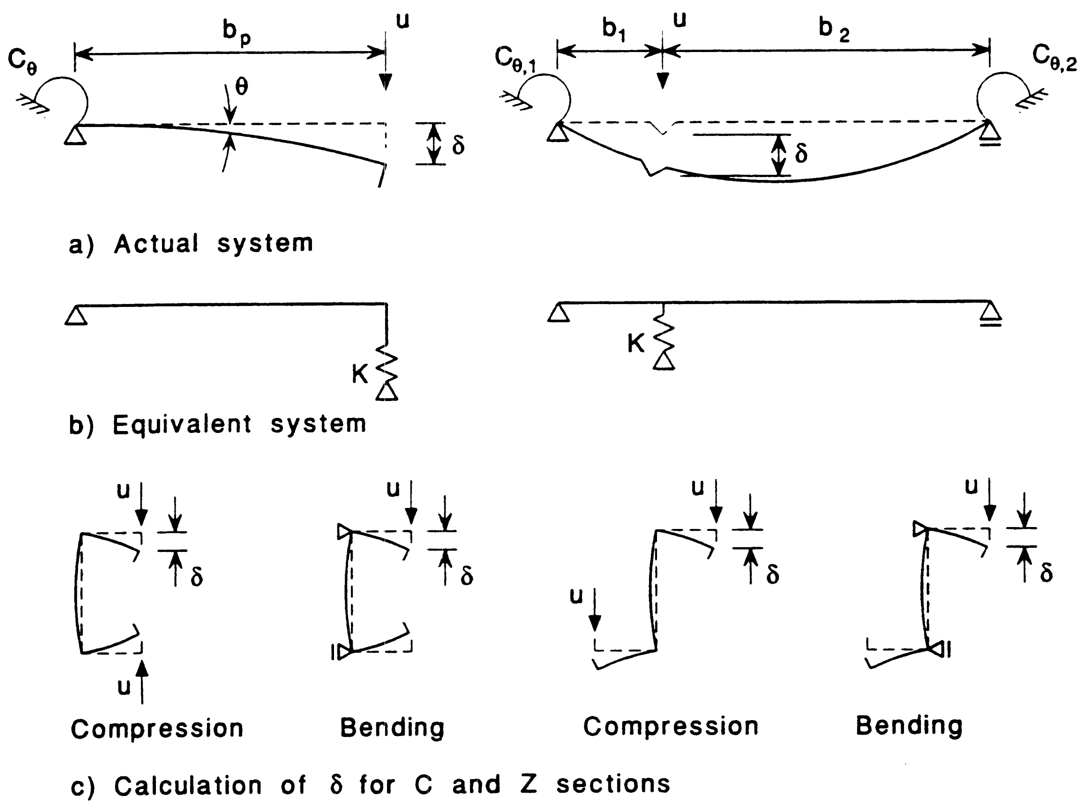


Figure 4.1: Determination of spring stiffness

(3) In determining the values of the rotational spring stiffnesses C_θ, $C_{\theta,1}$ and $C_{\theta,2}$ from the geometry of the cross-section, account should be taken of the possible effects of other stiffeners that exist on the same element, or on any other element of the cross-section that is subject to compression.

(4) For an edge stiffener, the deflection δ should be obtained from:

$$\delta = \theta b_p + \frac{u b_p^3}{3} \times \frac{12(1-\nu^2)}{E t^3} \qquad \dots (4.7)$$

with:

$$\theta = u b_p / C_\theta$$

(5) In the case of the edge stiffeners of lipped C-sections and lipped Z-sections, C_θ should be determined with the unit loads u applied as shown in figure 4.1(c).

(6) For an intermediate stiffener, as a conservative alternative the values of the rotational spring stiffnesses $C_{\theta,1}$ and $C_{\theta,2}$ may be taken as equal to zero, and the deflection δ may be obtained from:

$$\delta = \frac{u\, b_1^2 b_2^2}{3(b_1 + b_2)} \times \frac{12(1-\nu^2)}{E t^3} \qquad \text{... (4.8)}$$

(7) The reduction factor χ for the flexural buckling resistance of a stiffener should be obtained from 6.2.1(2)P using buckling curve a_0 (imperfection factor $\alpha = 0{,}13$) for the relative slenderness $\bar{\lambda}$ from:

$$\bar{\lambda} = \sqrt{f_{yb} / \sigma_{cr,s}} \qquad \text{... (4.9)}$$

where:

$\sigma_{cr,s}$ is the elastic critical stress for the stiffener from 4.3.2, 4.3.3 or 4.3.4.

4.3.2 Plane elements with edge stiffeners

4.3.2.1 Conditions

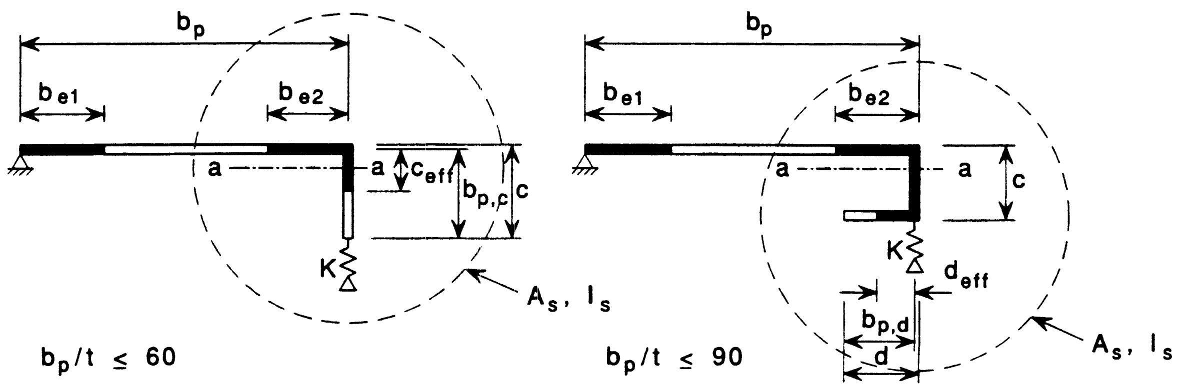
(1) An edge stiffener may be either a single edge fold or a double edge fold as illustrated in figure 4.2.

(2)P An edge stiffener shall not be taken into account in determining the resistance of the plane element to which it is attached unless the following conditions are met:

- the angle between the stiffener and the plane element is not less than 45° and not more than 135°;
- the outstand c is not less than $0{,}2b_p$, where b_p and c are as shown in figure 4.2;
- the ratio b_p/t is not more than 60 for a single edge fold stiffener, or 90 for a double edge fold stiffener.

(3) If the criteria in (1) and (2)P are met, the effectiveness of the stiffener may be determined from either:

- the general procedure given in 4.3.2.2;
- the simplified procedure given in 4.3.2.3.



a) single edge fold **b) double edge fold**

Figure 4.2: Edge stiffeners

4.3.2.2 General procedure

(1) The cross-section of an edge stiffener should be taken as comprising the effective portions of the stiffener, element c or elements c and d as shown in figure 4.2, plus the adjacent effective portion of the plane element b_p.

(2) The procedure, which is illustrated in figure 4.3, should be carried out in steps as follows:

- **Step 1**: Obtain an initial effective cross-section for the stiffener using effective widths determined by assuming that the stiffener gives full restraint and that $\sigma_{com,Ed} = f_{yb}/\gamma_{M1}$, see (3) to (5);
- **Step 2**: Use the initial effective cross-section of the stiffener to determine the reduction factor for flexural buckling, allowing for the effects of the continuous spring restraint, see (6) and (7);
- **Step 3**: Iterate to refine the value of the reduction factor for buckling of the stiffener, see (8) and (9).

(3) Initial values of the effective widths b_{e1} and b_{e2} shown in figure 4.2 should be determined from clause 4.2 by assuming that the plane element b_p is doubly supported, see table 4.1.

(4) Initial values of the effective widths c_{eff} and d_{eff} shown in figure 4.2 should be obtained as follows:

a) for a single edge fold stiffener:

$$c_{eff} = \rho b_{p,c} \quad \text{... (4.10a)}$$

with ρ obtained from 4.2(4), except using a value of the buckling factor k_σ given by the following:

- if $b_{p,c}/b_p \leq 0{,}35$:

$$k_\sigma = 0{,}5 \quad \text{... (4.10b)}$$

- if $0{,}35 < b_{p,c}/b_p \leq 0{,}6$:

$$k_\sigma = 0{,}5 - 0{,}83 \times \sqrt[3]{(b_{p,c}/b_p - 0{,}35)^2} \quad \text{... (4.10c)}$$

b) for a double edge fold stiffener:

$$c_{eff} = \rho b_{p,c} \quad \text{... (4.10d)}$$

with ρ obtained from 4.2(4) with a buckling factor k_σ for a doubly supported element from table 4.1;

$$d_{eff} = \rho b_{p,d} \quad \text{... (4.10e)}$$

with ρ obtained from 4.2(4) with a buckling factor k_σ for an outstand element from table 4.2.

(5) The effective cross-sectional area of the edge stiffener A_s should be obtained from:

$$A_s = t(b_{e2} + c_{eff} + d_{eff}) \quad \text{... (4.11)}$$

(6) The elastic critical buckling stress $\sigma_{cr,s}$ for an edge stiffener should be obtained from:

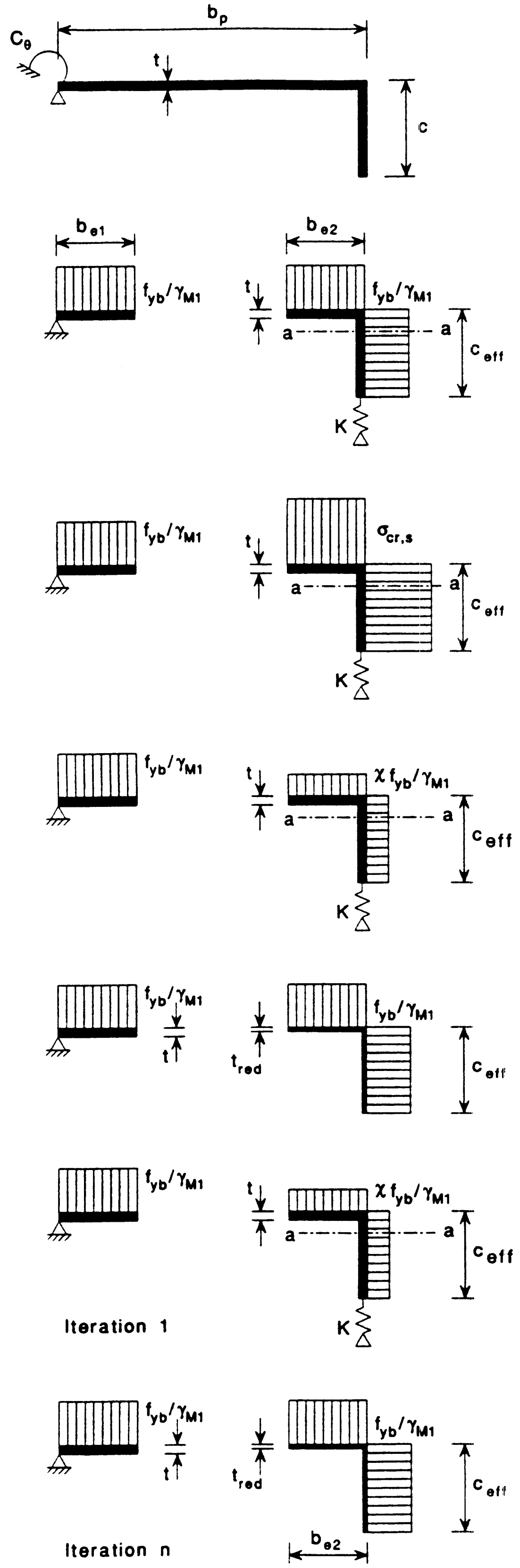
$$\sigma_{cr,s} = \frac{2\sqrt{KEI_s}}{A_s} \quad \text{... (4.12)}$$

where:

K is the spring stiffness per unit length, see 4.3.1(2).

I_s is the effective second moment of area of the stiffener, taken as that of its effective area A_s about the centroidal axis a - a of its effective cross-section, see figure 4.2.

(7) The reduction factor χ for the flexural buckling resistance of an edge stiffener should be obtained from the value of $\sigma_{cr,s}$ using the method given in 4.3.1(7).



a) Gross cross-section and boundary conditions

b) **Step 1**: Effective cross-section for $K = \infty$ based on $\sigma_{com,Ed} = f_{yb}/\gamma_{M1}$

c) **Step 2**: Elastic critical stress $\sigma_{cr,s}$ for effective area of stiffener A_s from step 1

d) Reduced strength $\chi f_{yb}/\gamma_{M1}$ for effective area of stiffener A_s, with reduction factor χ based on $\sigma_{cr,s}$

e) Reduced thickness t_{red} for effective width of stiffener, with reduction factor χ based on $\sigma_{cr,s}$

f) **Step 3**: Repeat step 2 using reduced effective area of stiffener $A_{s,red}$ from previous iteration, continuing until $\chi_n \approx \chi_{(n-1)}$ but $\chi_n \leq \chi_{(n-1)}$

g) Adopt an effective cross-section with b_{e2}, c_{eff} and reduced thickness t_{red} corresponding to χ_n

Figure 4.3: Compression resistance of a flange with an edge stiffener

(8) If $\chi < 1$ it may optionally be refined iteratively, starting the iteration with modified values of ρ obtained using 4.2(5) with $\sigma_{com,Ed}$ equal to $\chi f_{yb} / \gamma_{M1}$, so that:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi} \qquad \text{... (4.13)}$$

(9) If iteration is carried out, it should be continued until the current value of χ is approximately equal to, but not more than, the previous value.

(10) The reduced effective area of the stiffener $A_{s,red}$ allowing for flexural buckling should be taken as:

$$A_{s,red} = \chi A_s \qquad \text{... (4.14)}$$

(11) In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = \chi t$ for all the elements included in A_s.

(12) The effective section properties at serviceability limit states should be based on the design thickness t.

4.3.2.3 Simplified procedure

(1) As an alternative to the general procedure given in 4.3.2.2, the following simplified procedure may be used to determine the reduced effective area $A_{s,red}$ of an edge stiffener as shown in figure 4.2.

(2) The effective cross-sectional area of the edge stiffener A_s should be obtained from:

$$A_s = t(b_{e2} + c_{eff} + d_{eff}) \qquad \text{... (4.15)}$$

in which the effective widths b_{e2}, c_{eff} and d_{eff} should be obtained as in 4.3.2.2(3) and (4), except that ρ should be obtained from 4.2(5) with $\sigma_{com,Ed}$ equal to $\chi f_y / \gamma_{M1}$, so that:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi} \qquad \text{... (4.16)}$$

(3) The reduction factor χ may be taken as equal to 0,5 if:

$$I_s \geq 0{,}31(1{,}5 + h/b_p)(f_{yb}/E)^2(b_p/t)^3 A_s^{\,2} \qquad \text{... (4.17)}$$

otherwise the reduction factor χ may be taken as approximately equal to 1,0 if:

$$I_s \geq 4{,}86(1{,}5 + h/b_p)(f_{yb}/E)^2(b_p/t)^3 A_s^{\,2} \qquad \text{... (4.18)}$$

where:

- b_p is the notional flat width of the plane outstand element, see figure 4.2;
- h is the overall depth of the adjacent web;
- I_s is the effective second moment of area of the edge stiffener, taken as that of its effective area A_s about the centroidal axis a - a of its effective cross-section, see figure 4.2.

(4) The reduced effective area of the stiffener $A_{s,red}$ allowing for flexural buckling should be taken as:

$$A_{s,red} = \chi A_s \qquad \text{... (4.19)}$$

(5) In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = \chi t$ for all the elements included in A_s.

(6) The effective section properties at serviceability limit states should be based on the design thickness t for all values of I_s.

4.3.3 Plane elements with intermediate stiffeners

4.3.3.1 Conditions

(1) Intermediate stiffeners may be formed by grooves or bends.

(2) The stiffeners should be equally shaped and not more than two in number.

(3) If the criteria in (1) and (2) are met the effectiveness of the stiffener may be determined from either:

- the general procedure given in 4.3.3.2;
- the simplified procedure given in 4.3.3.3.

4.3.3.2 General procedure

(1) The cross-section of an intermediate stiffener should be taken as comprising the stiffener itself plus the adjacent effective portions of the adjacent plane elements $b_{p,1}$ and $b_{p,2}$ shown in figure 4.4.

(2) The procedure, which is illustrated in figure 4.5, should be carried out in steps as follows:

- **Step 1**: Obtain an initial effective cross-section for the stiffener using effective widths determined by assuming that the stiffener gives full restraint and that $\sigma_{com,Ed} = f_{yb}/\gamma_{M1}$, see (3) and (4);
- **Step 2**: Use the initial effective cross-section of the stiffener to determine the reduction factor for flexural buckling, allowing for the effects of the continuous spring restraint, see (5) and (6);
- **Step 3**: Iterate to refine the value of the reduction factor for buckling of the stiffener, see (7) and (8).

(3) Initial values of the effective widths $b_{1,e2}$ and $b_{2,e1}$ shown in figure 4.4 should be determined from 4.2 by assuming that the plane elements $b_{p,1}$ and $b_{p,2}$ are doubly supported, see table 4.1.

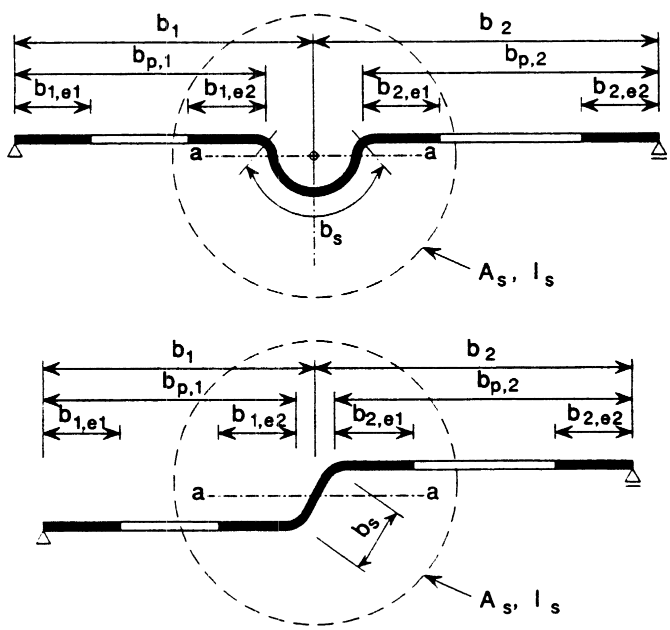


Figure 4.4: Intermediate stiffeners

(4) The effective cross-sectional area of an intermediate stiffener A_s should be obtained from:

$$A_s = t(b_{1,e2} + b_{2,e1} + b_s) \quad \text{... (4.20)}$$

in which the stiffener width b_s is as shown in figure 4.4.

(5) The critical buckling stress $\sigma_{cr,s}$ for an intermediate stiffener should be obtained from:

$$\sigma_{cr,s} = \frac{2\sqrt{KEI_s}}{A_s} \quad \text{... (4.21)}$$

where:

K is the spring stiffness per unit length, see 4.3.1(2).

I_s is the effective second moment of area of the stiffener, taken as that of its effective area A_s about the centroidal axis a - a of its effective cross-section, see figure 4.4.

(6) The reduction factor χ for the flexural buckling resistance of an intermediate stiffener should be obtained from the value of $\sigma_{cr,s}$ using the method given in 4.3.1(7).

(7) If $\chi < 1$ it may optionally be refined iteratively, starting the iteration with modified values of ρ obtained using 4.2(5) with $\sigma_{com,Ed}$ equal to $\chi f_{yb}/\gamma_{M1}$, so that:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi} \quad \text{... (4.22)}$$

(8) If iteration is carried out, it should be continued until the current value of χ is approximately equal to, but not more than, the previous value.

(9) The reduced effective area of the stiffener $A_{s,red}$ allowing for flexural buckling should be taken as:

$$A_{s,red} = \chi A_s \quad \text{... (4.23)}$$

(10) In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = \chi t$ for all the elements included in A_s.

(11) The effective section properties at serviceability limit states should be based on the design thickness t.

4.3.3.3 Simplified procedure

(1) As an alternative to the general procedure given in 4.3.3.2, the following simplified procedure may be used to determine the reduced effective area $A_{s,red}$ of an intermediate stiffener as shown in figure 4.4.

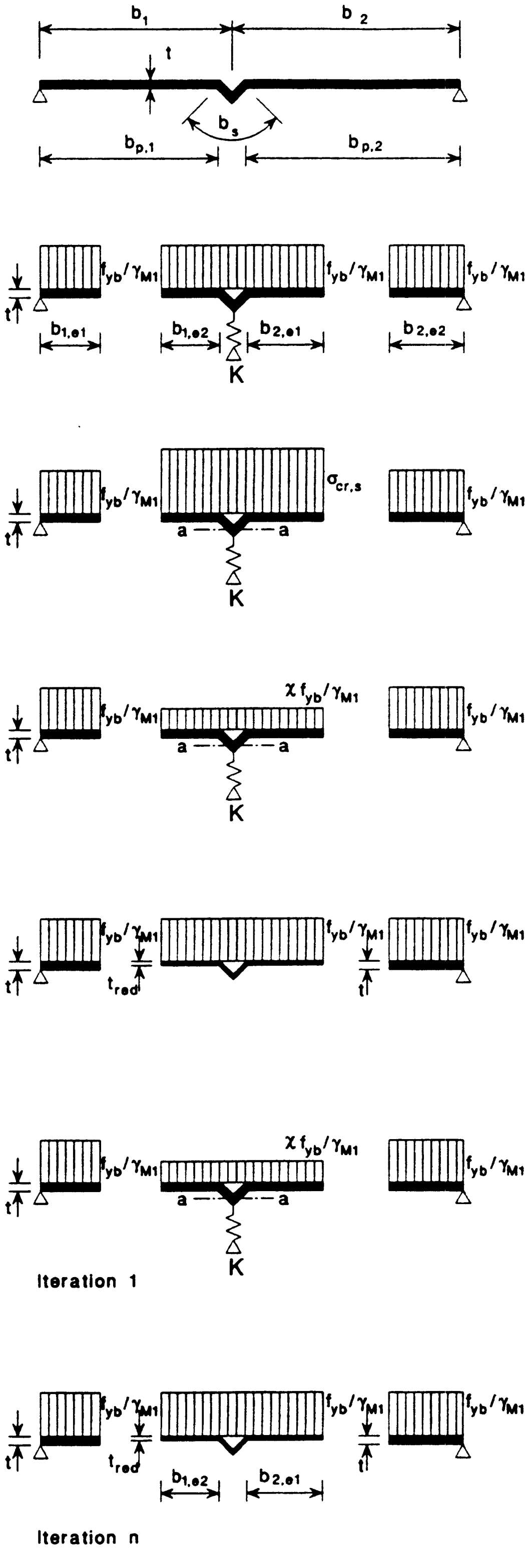
(2) The effective cross-sectional area of an intermediate stiffener A_s should be obtained from:

$$A_s = t(b_{1,e2} + b_{2,e1} + b_s) \quad \text{... (4.24)}$$

in which the effective widths $b_{1,e2}$ and $b_{2,e1}$ and the stiffener width b_s are as shown in figure 4.4.

(3) The effective widths $b_{1,e2}$ and $b_{1,e1}$ should be determined from 4.2 with a buckling factor k_σ for a doubly supported element from table 4.1, using a value of ρ obtained from 4.2(5) with $\sigma_{com,Ed}$ equal to $\chi f_y/\gamma_{M1}$, so that:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi} \quad \text{... (4.25)}$$



a) Gross cross-section and boundary conditions

b) **Step 1**: Effective cross-section for $K = \infty$ based on $\sigma_{com,Ed} = f_{yb}/\gamma_{M1}$

c) **Step 2**: Elastic critical stress $\sigma_{cr,s}$ for effective area of stiffener A_s from step 1

d) Reduced strength $\chi f_{yb}/\gamma_{M1}$ for effective area of stiffener A_s, with reduction factor χ based on $\sigma_{cr,s}$

e) Reduced thickness t_{red} for effective width of stiffener, with reduction factor χ based on $\sigma_{cr,s}$

f) **Step 3**: Repeat step 2 using reduced effective area of stiffener $A_{s,red}$ from previous iteration, continuing until $\chi_n \approx \chi_{(n-1)}$ but $\chi_n \leq \chi_{(n-1)}$

g) Adopt an effective cross-section with $b_{1,e2}$, $b_{2,e1}$ and reduced thickness t_{red} corresponding to χ_n

Figure 4.5: Compression resistance of a flange with an intermediate stiffener

(4) The reduction factor χ may be taken as equal to 0,5 if:

$$I_s \geq 0{,}016\,(f_{yb}/E)^2\,(b_o/t)^3 A_s^2 \qquad \dots (4.26)$$

otherwise the reduction factor χ may be taken as approximately equal to 1,0 if:

$$I_s \geq 0{,}24\,(f_{yb}/E)^2\,(b_o/t)^3 A_s^2 \qquad \dots (4.27)$$

where:

b_o = $b_1 + b_2$ (see figure 4.4);

I_s is the effective second moment of area of the stiffener, taken as that of its effective area A_s about the centroidal axis a - a of its effective cross-section, see figure 4.4.

(5) The reduced effective area of the stiffener $A_{s,red}$ allowing for flexural buckling should be taken as:

$$A_{s,red} = \chi A_s \qquad \dots (4.28)$$

(6) In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = \chi t$ for all the elements included in A_s.

(7) The effective section properties at serviceability limit states should be based on the design thickness t for all values of I_s.

4.3.4 Trapezoidal sheeting profiles with intermediate stiffeners

4.3.4.1 General

(1) This sub-clause 4.3.4 should be used for trapezoidal profiled sheets, in association with 4.3.2 for flanges with intermediate flange stiffeners and 4.3.3 for webs with intermediate stiffeners.

(2) Interaction between the buckling of intermediate flange stiffeners and intermediate web stiffeners should also be taken into account using the method given in 4.3.4.4.

4.3.4.2 Flanges with intermediate stiffeners

(1) If it is subject to uniform compression, the effective cross-section of a flange with intermediate stiffeners should be assumed to consist of the reduced effective areas $A_{s,red}$ of up to two intermediate stiffeners and two strips of width $0{,}5b_{eff}$ adjacent to the edges supported by webs, see figure 4.6.

(2) For one central flange stiffener, the elastic critical buckling stress $\sigma_{cr,s}$ should be obtained from:

$$\sigma_{cr,s} = \frac{4{,}2\,k_w E}{A_s}\sqrt{\frac{I_s t^3}{4 b_p^2 (2 b_p + 3 b_s)}} \qquad \dots (4.29)$$

where:

b_p is the notional flat width of plane element shown in figure 4.6;

b_s is the stiffener width, measured around the perimeter of the stiffener, see figure 4.6;

k_w is a coefficient that allows for partial rotational restraint of the stiffened flange by the webs or other adjacent elements, see (5) and (6);

and A_s and I_s are as defined in 4.3.3.2.

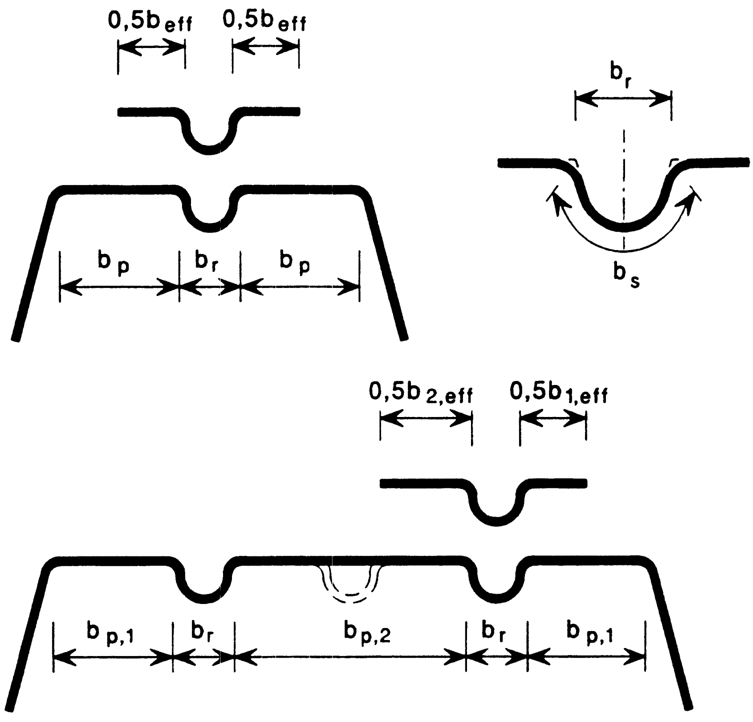


Figure 4.6: Compression flange with one, two or three stiffeners

(3) For two symmetrically placed flange stiffeners, the elastic critical buckling stress $\sigma_{cr,s}$ should be obtained from:

$$\sigma_{cr,s} = \frac{4{,}2\, k_w E}{A_s} \sqrt{\frac{I_s t^3}{8 b_1^2 (3 b_e - 4 b_1)}} \qquad \dots (4.30)$$

with:

$$b_e = 2b_{p,1} + b_{p,2} - 2b_s$$

$$b_1 = b_{p,1} + 0{,}5\, b_r$$

where:

$b_{p,1}$ is the notional flat width of an outer plane element, as shown in figure 4.6;

$b_{p,2}$ is the notional flat width of the central plane element, as shown in figure 4.6;

b_r is the overall width of a stiffener, see figure 4.6.

(4) If there are three stiffeners, the one in the middle should be assumed to be ineffective.

(5) The value of k_w may be calculated from the compression flange buckling wavelength ℓ_b as follows:

- if $\ell_b / s_w \geq 2$:

$$k_w = k_{wo} \qquad \dots (4.31a)$$

- if $\ell_b / s_w < 2$:

$$k_w = k_{wo} - (k_{wo} - 1)[2\ell_b / s_w - (\ell_b / s_w)^2] \qquad \dots (4.31b)$$

where:

s_w is the slant height of the web, see figure 3.3(c).

(6) Alternatively, the rotational restraint coefficient k_w may conservatively be taken as equal to 1,0 corresponding to a pin-jointed condition.

(7) The values of ℓ_b and k_{wo} may be determined from the following:

- for a compression flange with one intermediate stiffener:

$$\ell_b = 3{,}07 \sqrt[4]{I_s b_p^2 (2b_p + 3b_s)/t^3} \quad \text{... (4.32)}$$

$$k_{wo} = \sqrt{\frac{s_w + 2b_d}{s_w + 0{,}5b_d}} \quad \text{... (4.33)}$$

with:

$$b_d = 2b_p + b_s$$

- for a compression flange with two or three intermediate stiffeners:

$$\ell_b = 3{,}65 \sqrt[4]{I_s b_1^2 (3b_e - 4b_1)/t^3} \quad \text{... (4.34)}$$

$$k_{wo} = \sqrt{\frac{(2b_e + s_w)(3b_e - 4b_1)}{b_1(4b_e - 6b_1) + s_w(3b_e - 4b_1)}} \quad \text{... (4.35)}$$

(8) The reduced effective area of the stiffener $A_{s,red}$ allowing for flexural buckling should be taken as:

$$A_{s,red} = \chi A_s \quad \text{... (4.36)}$$

(9) If the webs are unstiffened, the reduction factor χ should be obtained directly from $\sigma_{cr,s}$ using the method given in 4.3.1(7).

(10) If the webs are also stiffened, the reduction factor χ should be obtained using the method given in 4.3.1(7), but with the modified elastic critical stress $\sigma_{cr,mod}$ given in 4.3.4.4.

(11) In determining effective section properties, the reduced effective area $A_{s,red}$ should be represented by using a reduced thickness $t_{red} = \chi t$ for all the elements included in A_s.

(12) The effective section properties at serviceability limit states should be based on the design thickness t.

4.3.4.3 Webs with up to two intermediate stiffeners

(1) The effective cross-section of the compression zone of a web (or other element of a cross-section that is subject to stress gradient) should be assumed to consist of the reduced effective areas $A_{s,red}$ of up to two intermediate stiffeners, a strip adjacent to the compression flange and a strip adjacent to the centroidal axis of the effective cross-section, see figure 4.7.

(2) The effective cross-section of a web as shown in figure 4.7 should be taken to include:

a) a strip of width $s_{eff,1}$ adjacent to the compression flange;

b) the reduced effective area $A_{s,red}$ of each web stiffener, up to a maximum of two;

c) a strip of width $s_{eff,n}$ adjacent to the effective centroidal axis;

d) the part of the web in tension.

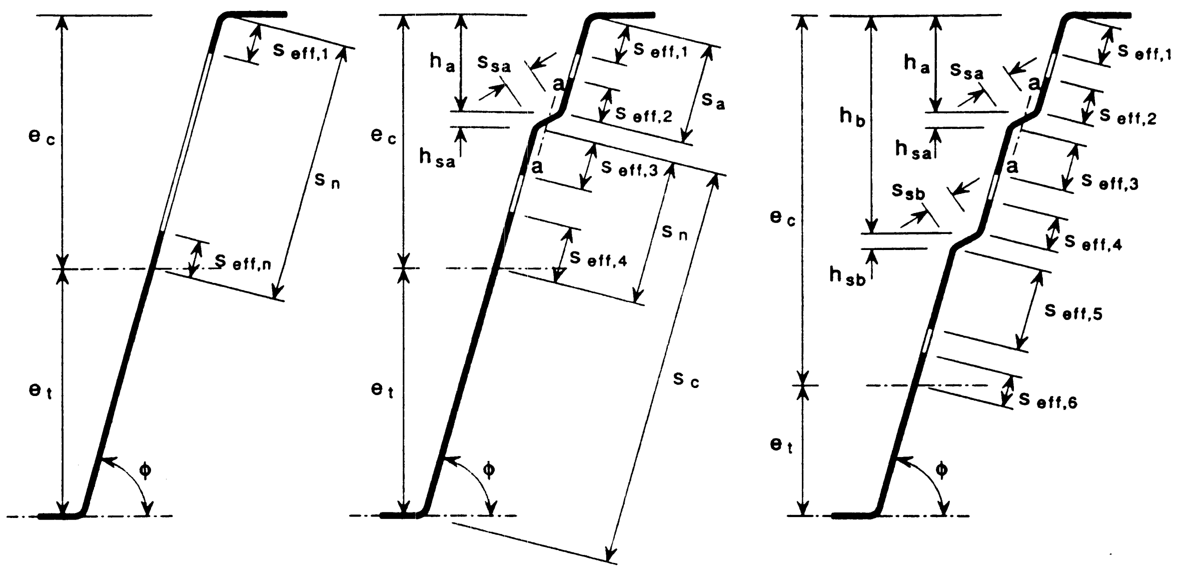


Figure 4.7: Effective cross-sections of webs of trapezoidal profiled sheets

(3) The effective areas of the stiffeners should be obtained from the following:

- for a single stiffener, or for the stiffener closer to the compression flange:

$$A_{sa} = t(s_{eff,2} + s_{eff,3} + s_{sa}) \qquad \text{... (4.37)}$$

- for a second stiffener:

$$A_{sb} = t(s_{eff,4} + s_{eff,5} + s_{sb}) \qquad \text{... (4.38)}$$

in which the dimensions $s_{eff,1}$ to $s_{eff,n}$ and s_{sa} and s_{sb} are as shown in figure 4.7.

(4) Initially the location of the effective centroidal axis should be based on the effective cross-sections of the flanges but the gross cross-sections of the webs. In this case the basic effective width $s_{eff,0}$ should be obtained from:

$$s_{eff,0} = 0{,}76t\sqrt{E/(\gamma_{M1}\sigma_{com,Ed})} \qquad \text{... (4.39)}$$

where:

$\sigma_{com,Ed}$ is the stress in the compression flange when the cross-section resistance is reached.

(5) If the web is not fully effective, the dimensions $s_{eff,1}$ to $s_{eff,n}$ should be determined as follows:

$$s_{eff,1} = s_{eff,0} \qquad \text{... (4.40a)}$$

$$s_{eff,2} = (1 + 0{,}5h_a/e_c)s_{eff,0} \qquad \text{... (4.40b)}$$

$$s_{eff,3} = [1 + 0{,}5(h_a + h_{sa})/e_c]s_{eff,0} \qquad \text{... (4.40c)}$$

$$s_{eff,4} = (1 + 0{,}5h_b/e_c)s_{eff,0} \qquad \text{... (4.40d)}$$

$$s_{eff,5} = [1 + 0{,}5(h_b + h_{sb})/e_c]s_{eff,0} \qquad \text{... (4.40e)}$$

$$s_{eff,n} = 1{,}5s_{eff,0} \qquad \text{... (4.40f)}$$

where:

e_c is the distance from the effective centroidal axis to the system line of the compression flange, see figure 4.7;

and the dimensions h_a, h_b, h_{sa} and h_{sb} are as shown in figure 4.7.

(6) The dimensions $s_{eff,1}$ to $s_{eff,n}$ should initially be determined from (5) and then revised if the relevant plane element is fully effective, using the following:

- in an unstiffened web, if $s_{eff,1} + s_{eff,n} \geq s_n$ the entire web is effective, so revise as follows:

$$s_{eff,1} = 0{,}4 s_n \quad \text{... (4.41a)}$$

$$s_{eff,n} = 0{,}6 s_n \quad \text{... (4.41b)}$$

- in stiffened web, if $s_{eff,1} + s_{eff,2} \geq s_a$ the whole of s_a is effective, so revise as follows:

$$s_{eff,1} = s_a / (2 + 0{,}5 h_a / e_c) \quad \text{... (4.42a)}$$

$$s_{eff,2} = s_a (1 + 0{,}5 h_a / e_c) / (2 + 0{,}5 h_a / e_c) \quad \text{... (4.42b)}$$

- in a web with one stiffener, if $s_{eff,3} + s_{eff,n} \geq s_n$ the whole of s_n is effective, so revise as follows:

$$s_{eff,3} = s_n [1 + 0{,}5(h_a + h_{sa}) / e_c] / [2{,}5 + 0{,}5(h_a + h_{sa}) / e_c] \quad \text{... (4.43a)}$$

$$s_{eff,n} = 1{,}5 s_n / [2{,}5 + 0{,}5(h_a + h_{sa}) / e_c] \quad \text{... (4.43b)}$$

- in a web with two stiffeners:
  - if $s_{eff,3} + s_{eff,4} \geq s_b$ the whole of s_b is effective, so revise as follows:

$$s_{eff,3} = s_b [1 + 0{,}5(h_a + h_{sa}) / e_c] / [2 + 0{,}5(h_a + h_{sa} + h_b) / e_c] \quad \text{... (4.44a)}$$

$$s_{eff,4} = s_b (1 + 0{,}5 h_b / e_c) / [2 + 0{,}5(h_a + h_{sa} + h_b) / e_c] \quad \text{... (4.44b)}$$

  - if $s_{eff,5} + s_{eff,n} \geq s_n$ the whole of s_n is effective, so revise as follows:

$$s_{eff,5} = s_n [1 + 0{,}5(h_b + h_{sb}) / e_c] / [2{,}5 + 0{,}5(h_b + h_{sb}) / e_c] \quad \text{... (4.45a)}$$

$$s_{eff,n} = 1{,}5 s_n / [2{,}5 + 0{,}5(h_b + h_{sb}) / e_c] \quad \text{... (4.45b)}$$

(7) For a single stiffener, or for the stiffener closer to the compression flange in webs with two stiffeners, the elastic critical buckling stress $\sigma_{cr,sa}$ should be determined using:

$$\sigma_{cr,sa} = \frac{1{,}05 k_f E \sqrt{I_s t^3 s_1}}{A_{sa} s_2 (s_1 - s_2)} \quad \text{... (4.46a)}$$

in which s_1 is given by the following:

- for a single stiffener:

$$s_1 = 0{,}9 (s_a + s_{sa} + s_c) \quad \text{... (4.46b)}$$

- for the stiffener closer to the compression flange, in webs with two stiffeners:

$$s_1 = s_a + s_{sa} + s_b + 0{,}5(s_{sb} + s_c) \quad \text{... (4.46c)}$$

with:

$$s_2 = s_1 - s_a - 0{,}5 s_{sa} \quad \text{... (4.46d)}$$

where:

k_f is a coefficient that allows for partial rotational restraint of the stiffened web by the flanges;

I_s is the second moment of area of a stiffener cross-section comprising the fold width s_{sa} and two adjacent strips, each of width $s_{eff,1}$, about its own centroidal axis parallel to the plane web elements, see figure 4.8. In calculating I_s the possible difference in slope between the plane web elements on either side of the stiffener may be neglected.

(8) In the absence of a more detailed investigation, the rotational restraint coefficient k_f may conservatively be taken as equal to 1,0 corresponding to a pin-jointed condition.

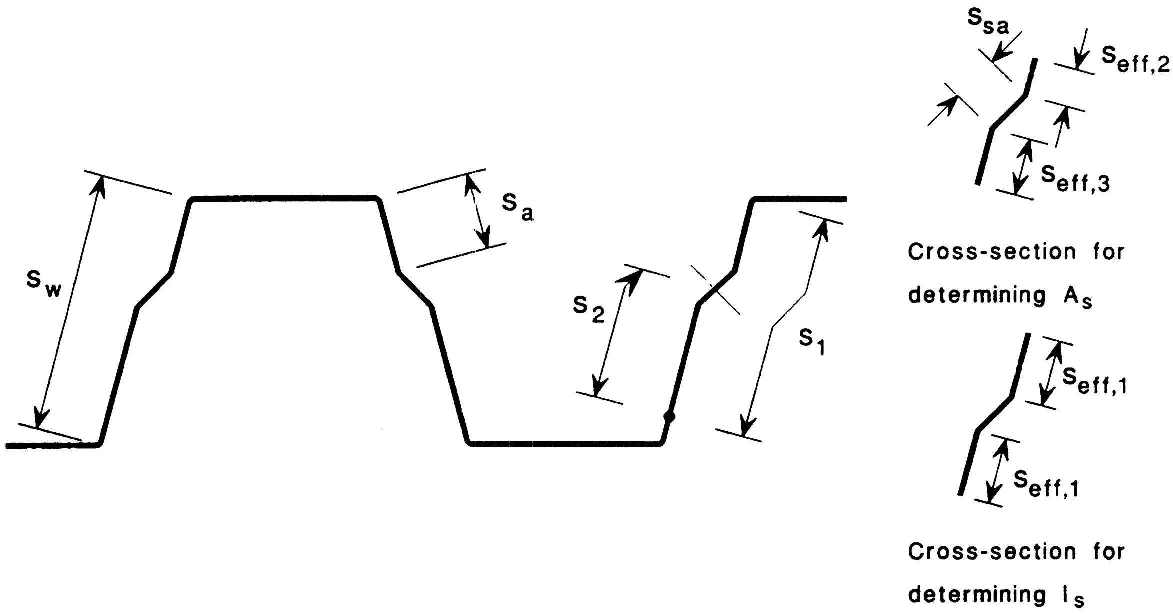


Figure 4.8: Web stiffeners for trapezoidal profiled sheeting

(9) For a single stiffener in compression, or for the stiffener closer to the compression flange in webs with two stiffeners, the reduced effective area $A_{sa,red}$ should be determined from:

$$A_{sa,red} = \chi A_{sa} / [1 - (h_a + 0,5 h_{sa}) / e_c] \quad \text{but} \quad A_{sa,red} \leq A_{sa} \qquad \ldots (4.47)$$

(10) If the flanges are unstiffened, the reduction factor χ should be obtained directly from $\sigma_{cr,sa}$ using the method given in 4.3.1(7).

(11) If the flanges are also stiffened, the reduction factor χ should be obtained using the method given in 4.3.1(7), but with the modified elastic critical stress $\sigma_{cr,mod}$ given in 4.3.4.4.

(12) For a single stiffener in tension, the reduced effective area $A_{sa,red}$ should be taken as equal to A_{sa}.

(13) For webs with two stiffeners, the reduced effective area $A_{sb,red}$ for the second stiffener, should be taken as equal to A_{sb}.

(14) In determining effective section properties, the reduced effective area $A_{sa,red}$ should be represented by using a reduced thickness $t_{red} = \chi t$ for all the elements included in A_{sa}.

(15) The effective section properties at serviceability limit states should be based on the design thickness t.

(16) Optionally, the effective section properties may be refined iteratively by basing the location of the effective centroidal axis on the effective cross-sections of the webs determined by the previous iteration and the effective cross-sections of the flanges determined using the reduced thickness t_{red} for all the elements included in the flange stiffener areas A_s. This iteration should be based on an increased basic effective width $s_{eff,0}$ obtained from:

$$s_{eff,0} = 0,95 t \sqrt{E / (\gamma_{M1} \sigma_{com,Ed})} \qquad \ldots (4.48)$$

4.3.4.4 Sheeting with flange stiffeners and web stiffeners

(1) In the case of sheeting with intermediate stiffeners in the flanges and in the webs, see figure 4.9, interaction between the flexural buckling of the flange stiffeners and the web stiffeners should be allowed for by using a modified elastic critical stress $\sigma_{cr,mod}$ for both types of stiffeners, obtained from:

$$\sigma_{cr,mod} = \frac{\sigma_{cr,sa}}{\left[\left(\sigma_{cr,sa}/\sigma_{cr,s}\right)^4 + \left(1 - (h_a + 0{,}5h_{sa})/e_c\right)^4\right]^{0,25}} \quad \text{... (4.49)}$$

where:

$\sigma_{cr,s}$ is the elastic critical stress for an intermediate flange stiffener, see 4.3.4.2(2) for a flange with a single stiffener or 4.3.4.2(3) for a flange with two stiffeners;

$\sigma_{cr,sa}$ is the elastic critical stress for a single web stiffener, or the stiffener closer to the compression flange in webs with two stiffeners, see 4.3.4.3(7).

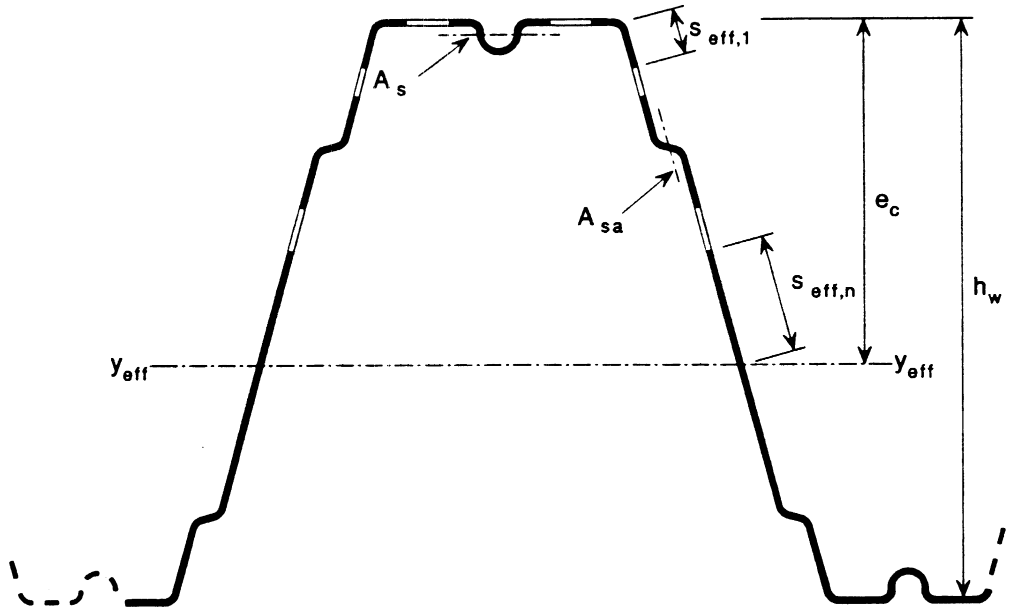


Figure 4.9: Trapezoidal profiled sheeting with flange stiffeners and web stiffeners

5 Resistance of cross-sections

5.1 General

(1)P The design values of the internal forces and moments at each cross-section shall not exceed the design values of the corresponding resistances.

(2)P The design resistance of a cross-section shall be determined either by calculation, using the methods given in this Section 5, or by design assisted by testing, in accordance with Section 9.

(3)P For design by calculation, the resistance of the cross-section shall be determined for:

- axial tension, as given in 5.2;
- axial compression, as given in 5.3;
- bending moment, as given in 5.4;
- combined bending and axial tension, as given in 5.5;
- combined bending and axial compression, as given in 5.6;
- torsional moment, as given in 5.7;
- shear force, as given in 5.8;
- local transverse forces, as given in 5.9;
- combined bending moment and shear force, as given in 5.10;
- combined bending moment and local transverse force, as given in 5.11.

(4) Design assisted by testing may be used instead of design by calculation for any of these resistances.

NOTE: Design assisted by testing is particularly likely to be beneficial for cross-sections with relatively high b_p / t ratios, e.g. in relation to inelastic behaviour, web crippling or shear lag.

(5)P For design by calculation, the effects of local buckling shall be taken into account by using effective section properties determined as specified in Section 4.

(6)P The buckling resistance of members shall be verified as specified in Section 6.

(7)P The effects of frame instability shall be taken into account as specified in ENV 1993-1-1.

(8)P In members with cross-sections that are susceptible to cross-sectional distortion, account shall be taken of possible lateral buckling of compression flanges and lateral bending of flanges generally, see 6.4.

5.2 Axial tension

(1)P The design tension resistance of a cross-section $N_{t,Rd}$ shall be determined from:

$$N_{t,Rd} = f_{ya} A_g / \gamma_{M0} \quad \text{but} \quad N_{t,Rd} \leq F_{n,Rd} \qquad \dots (5.1)$$

where:

A_g is the gross area of the cross-section;

$F_{n,Rd}$ is the net-section resistance from 8.4 for the appropriate type of mechanical fastener;

f_{ya} is the average yield strength, see 3.1.2.

(2) The tension resistance of an angle connected through one leg, or other types of section connected through outstands, should be determined as specified in ENV 1993-1-1.

5.3 Axial compression

(1)P The design compression resistance of a cross-section $N_{c,Rd}$ shall be determined from the following:

- if its effective area A_{eff} is less than its gross area A_g:

$$N_{c,Rd} = f_{yb} A_{eff} / \gamma_{M1} \qquad \text{... (5.2a)}$$

- if its effective area A_{eff} is equal to its gross area A_g:

$$N_{c,Rd} = f_{ya} A_g / \gamma_{M0} \qquad \text{... (5.2b)}$$

where:

A_{eff} is the effective area of the cross-section, obtained from Section 4 by assuming a uniform compressive stress $\sigma_{com,Ed}$ equal to f_{yb}/γ_{M1};

f_{ya} is the average yield strength, see 3.1.2.

f_{yb} is the basic yield strength.

(2)P The internal axial force in a member shall be taken as acting at the centroid of its gross cross-section.

(3)P The resistance of a cross-section to axial compression shall be assumed to act at the centroid of its effective cross-section. If this does not coincide with the centroid of its gross cross-section, the shift e_N of the centroidal axes (see figure 5.1) shall be taken into account, using the method given in 5.6.

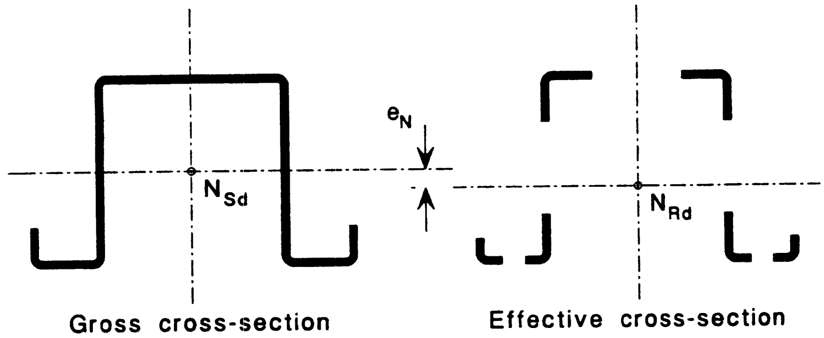


Figure 5.1: Effective cross-section under compression

5.4 Bending moment

5.4.1 General

(1)P The moment resistance of a cross-section for bending about a principal axis shall be obtained from the following:

- if the effective section modulus W_{eff} is less than the gross elastic section modulus W_{el}:

$$M_{c,Rd} = f_y W_{eff} / \gamma_{M1} \qquad \text{... (5.3a)}$$

- if the effective section modulus W_{eff} is equal to the gross elastic section modulus W_{el}:

$$N_{c,Rd} = f_{ya} W_{el} / \gamma_{M0} \qquad \text{... (5.3b)}$$

where:

f_y is the yield strength as defined in 3.1.1(6)P.

(2)P The effective section modulus W_{eff} shall be based on an effective cross-section that is subject only to bending moment about the relevant principal axis, with a maximum stress $\sigma_{max,Ed}$ equal to f_{yb} / γ_{M1}, allowing for the effects of local buckling as specified in Section 4. Where shear lag is relevant, allowance shall also be made for its effects as specified in 5.4.3.

(3) The stress ratio $\psi = \sigma_2 / \sigma_1$ used to determine the effective portions of the web may be obtained by using the effective area of the compression flange but the gross area of the web, see figure 5.2.

(4)P If yielding occurs first at the compression edge of the cross-section, unless the conditions given in 5.4.2(5)P are met the value of W_{eff} shall be based on a linear distribution of stress across the cross-section.

(5)P For biaxial bending the following criterion shall be satisfied:

$$M_{y,Sd} / M_{cy,Rd} + M_{z,Sd} / M_{cz,Rd} \leq 1 \qquad \text{... (5.4)}$$

where:

- $M_{y,Sd}$ is the applied bending moment about the major axis;
- $M_{z,Sd}$ is the applied bending moment about the minor axis;
- $M_{cy,Rd}$ is the resistance of the cross-section if subject only to moment about the y - y axis;
- $M_{cz,Rd}$ is the resistance of the cross-section if subject only to moment about the z - z axis.

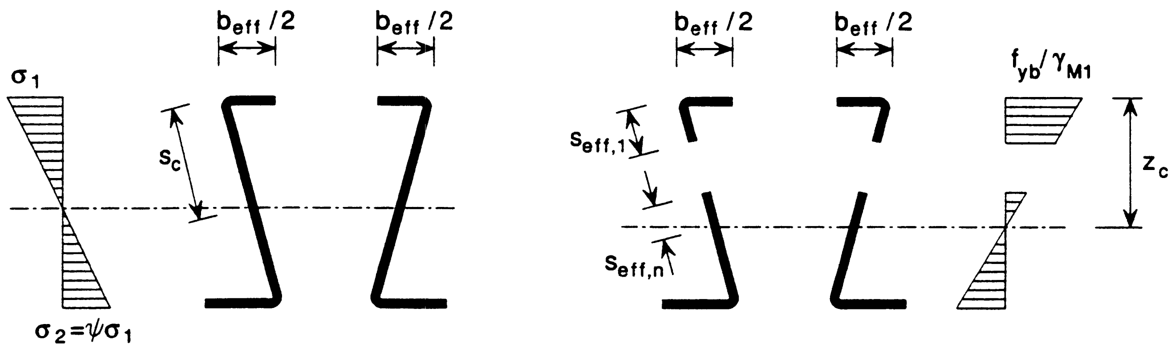


Figure 5.2: Effective cross-section for resistance to bending moments

5.4.2 Partially plastic resistance

(1)P Provided that bending moment is applied only about one principal axis of the cross-section, and provided that yielding occurs first at the tension edge, plastic reserves in the tension zone may be utilized without any strain limit until the maximum compressive stress $\sigma_{com,Ed}$ reaches f_{yb} / γ_{M1}.

(2) In this case, the effective partially plastic section modulus $W_{pp,eff}$ should be based on a stress distribution that is bilinear in the tension zone but linear in the compression zone.

(3) In the absence of a more detailed analysis, the effective width b_{eff} of an element subject to stress gradient may be obtained using 4.2 by basing b_c on the bilinear stress distribution, but ignoring the shape of the stress distribution in determining ψ.

(4) If redistribution of bending moments is assumed in the global analysis, it should be demonstrated from the results of tests in accordance with Section 9 that the provisions given in 7.2 are satisfied.

(5)P Plastic reserves may also be utilized in the compression zone, up to the strain specified in (6)P, provided that all the following conditions are satisfied:

a) Bending moment is applied only about one principal axes of the cross-section;

b) The member is not subject to torsion, or to torsional, torsional-flexural or lateral-torsional buckling;

c) Distortion of compressed parts of the cross-section is prevented;

d) The angle between any web and the vertical does not exceed 30 °;

e) The slant height s_c of the compressed portion of the web satisfies:

$$s_c / t \quad \leq \quad 1{,}11\sqrt{E/f_{yb}} \qquad [\cong \quad 33{,}18\varepsilon] \qquad \ldots (5.5)$$

(6)P The compressive strain $\varepsilon_{com,Ed}$ shall not exceed $C_y \varepsilon_y / \gamma_{M1}$, where $\varepsilon_y = f_{yb}/E$, see figure 5.3, and the factor C_y is obtained from the following:

- for doubly supported compression elements without intermediate stiffeners:

  - if $s_c/t \leq 1{,}11\sqrt{E/f_{yb}}$ $[\cong \ 33{,}18\varepsilon]$:

    $$C_y = 3$$

  - if $s_c/t \leq 1{,}29\sqrt{E/f_{yb}}$ $[\cong \ 38{,}56\varepsilon]$:

    $$C_y = 1$$

  - if $1{,}11\sqrt{E/f_{yb}} < s_c/t < 1{,}29\sqrt{E/f_{yb}}$ [i.e. if $33{,}18\varepsilon < s_c/t < 38{,}56\varepsilon$]:

    $$C_y = 3 - \frac{\left((s_c/t)\sqrt{f_{yb}/E} - 1{,}11\right)}{0{,}09} \qquad \left[\cong \ 3 - \frac{\left(s_c/t - 33{,}18\varepsilon\right)}{2{,}69\varepsilon}\right]$$

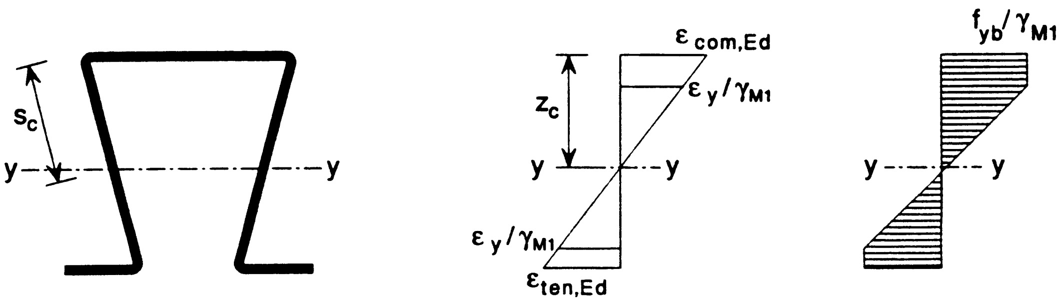
- for outstand elements:

  $$C_y = 1$$

- for elements with edge or intermediate stiffeners:

  $$C_y = 1$$

(7) In this case, the partially plastic section modulus W_{pp} should be based on a stress distribution that is bilinear in both the tension zone and the compression zone, as indicated in figure 5.3.



Effective cross-section for bending about the y-y axis

Design strain distribution $\varepsilon_{com,Ed} \leq C_y \varepsilon_y / \gamma_{M1}$

Design stress distribution

Figure 5.3: Partially plastic moment resistance

5.4.3 Effects of shear lag

(1)P The effects of shear lag shall be taken into account in flanges of flexural members if the length L_m between points of zero moment is less than $20b_o$, where b_o is the width of flange contributing to shear lag, as shown in figure 5.4.

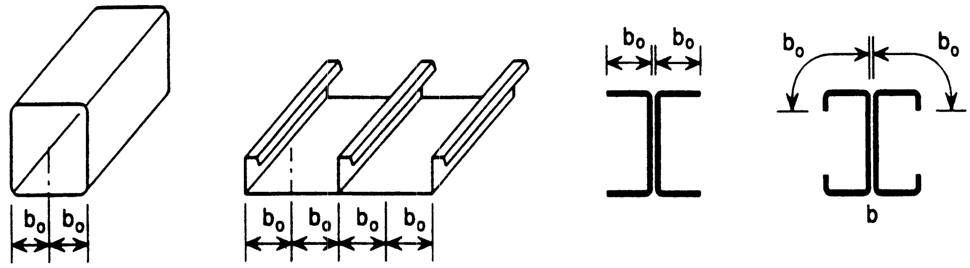


Figure 5.4: Width b_o contributing to shear lag

(2) In the absence of better information the following method may be adopted:

- for tension elements, replace b_o by b_{eff} given by:

$$b_{eff} = \beta_i b_o \quad \text{... (5.6)}$$

- for compression elements, replace the reduction factor ρ for local buckling (see 4.2(4)) by:

$$\rho_L = \beta_i^{\eta} \rho \quad \text{... (5.7a)}$$

in which:

- for stiffened flanges:

$$\eta = b_o / L_m \quad \text{... (5.7b)}$$

- for unstiffened flanges:

$$\eta = (b_o / L_m) / \delta \quad \text{... (5.7c)}$$

with:

$$\delta = \frac{2b_o}{t}\sqrt{\frac{f_y}{E}} \cong \frac{b_o/t}{14{,}95\varepsilon} \quad \text{but} \quad \delta \geq 1{,}0 \quad \text{... (5.8)}$$

where:

β_i is the appropriate value of the reduction factor for shear lag given in table 5.1.

NOTE: Further information is given in ENV 1993-2.*)

(3) For flanges with intermediate stiffeners b_o should be taken as half of the developed width b_d of the stiffened element, see figure 5.5.

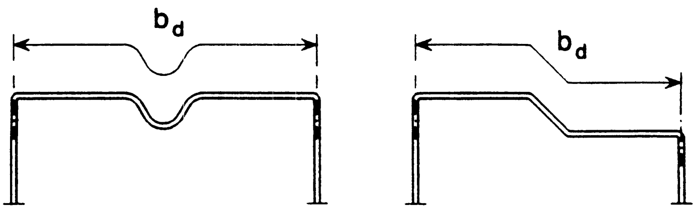
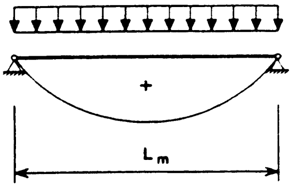
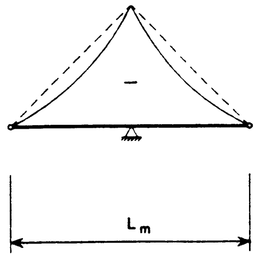
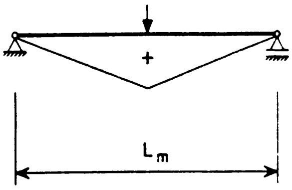
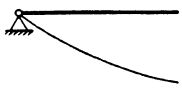
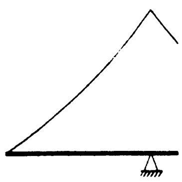


Figure 5.5: Developed width b_d of flanges with intermediate stiffeners

*) This document is in preparation.

Table 5.1: Reduction factors β_i for shear lag

Case and moment diagram	Reduction factor β_i
Span moment in simple or continuous beam - with uniformly distributed load 	for $b_o/L_m \geq 1/20$: $\beta_1 = \dfrac{1}{1 + 6{,}4(b_o/L_m)^2}$ for $b_o/L_m < 1/20$: $\beta_1 = 1{,}0$
Internal support of continuous beam or cantilever 	for $b_o/L_m \geq 1/20$: $\beta_2 = \dfrac{1}{1 + 6{,}0(b_o/L_m) + 1{,}6(b_o/L_m)^2}$ for $b_o/L_m < 1/50$: $\beta_2 = 1{,}0$ for $1/50 \leq b_o/L_m \leq 1/20$: $\beta_2 = 1{,}155 - 7{,}76(b_o/L_m)$
Span moment in simple or continuous beam - with central point load 	for $b_o/L_m \geq 1/20$: $\beta_3 = \dfrac{1}{1 + 4{,}0(b_o/L_m) + 3{,}2(b_o/L_m)^2}$ for $b_o/L_m < 1/50$: $\beta_3 = 1{,}0$ for $1/50 \leq b_o/L_m \leq 1/20$: $\beta_3 = 1{,}115 - 5{,}74(b_o/L_m)$
End support of beam 	$\beta_0 = (0{,}55 + 0{,}025 L_m/b_o)\beta_1$ but $\beta_0 \leq \beta_1$
Cantilever 	$\beta_0 = 1{,}0$

(4) As a simplification, in continuous beams the lengths L_m between points of zero moment may be replaced by the effective lengths L_e shown in figure 5.6, provided that no span is longer than 1,5 times an adjacent span, and no cantilever is longer than half the adjacent span.

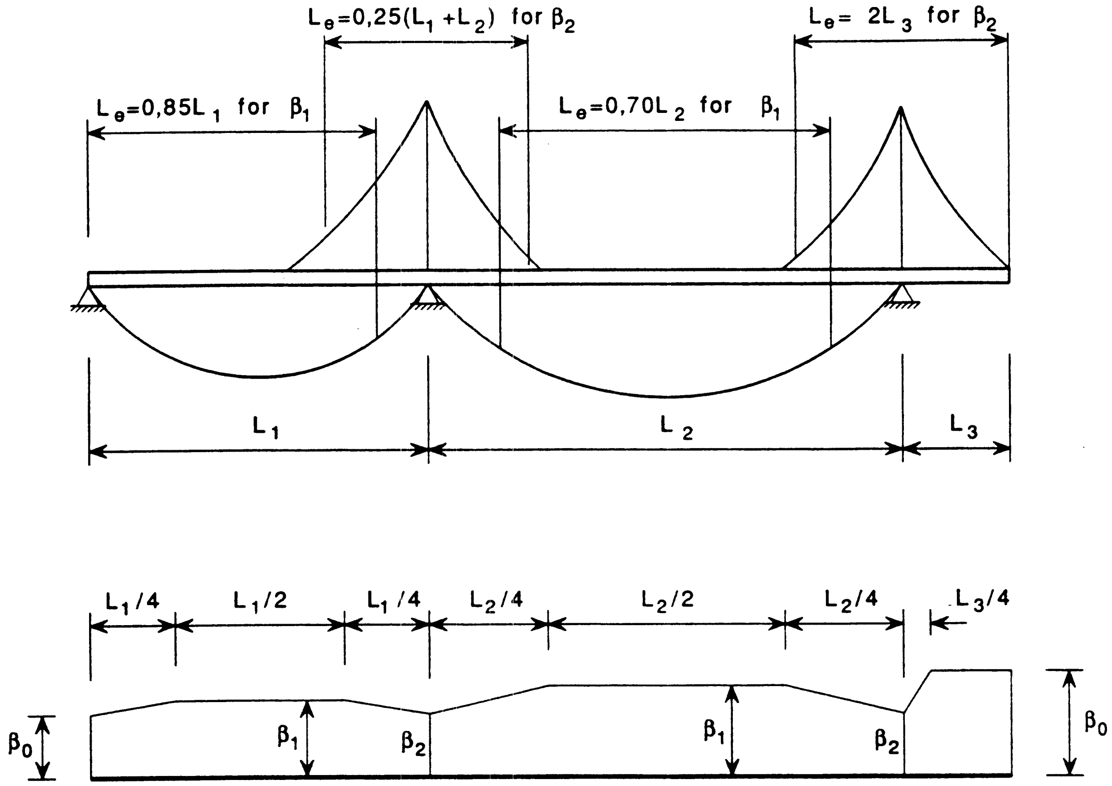


Figure 5.6: Simplified assumptions for continuous beams

5.5 Combined tension and bending

(1)P Cross-sections subject to combined axial tension N_{Sd} and bending moments $M_{y,Sd}$ and $M_{z,Sd}$ shall satisfy the criterion:

$$\frac{N_{Sd}}{f_y A_g / \gamma_M} + \frac{M_{y,Sd}}{f_y W_{eff,y,ten} / \gamma_M} + \frac{M_{z,Sd}}{f_y W_{eff,z,ten} / \gamma_M} \leq 1 \qquad \text{... (5.9a)}$$

where:

$W_{eff,y,ten}$ is the effective section modulus for maximum tensile stress if subject only to moment about the y - y axis;

$W_{eff,z,ten}$ is the effective section modulus for maximum tensile stress if subject only to moment about the z - z axis;

and $\gamma_M = \gamma_{M0}$ if $W_{eff} = W_{el}$ for each axis about which a bending moment acts, otherwise $\gamma_M = \gamma_{M1}$.

(2)P If $W_{eff,y,ten} \geq W_{eff,y,com}$ or $W_{eff,z,ten} \geq W_{eff,z,com}$ (where $W_{eff,y,com}$ and $W_{eff,z,com}$ are the effective section moduli for the maximum compressive stress in a effective cross-section that is subject only to moment about the relevant axis), the following criterion shall also be satisfied:

$$\frac{M_{y,Sd}}{f_y W_{eff,y,com} / \gamma_M} + \frac{M_{z,Sd}}{f_y W_{eff,z,com} / \gamma_M} - \frac{\psi_{vec} N_{Sd}}{f_y A_g / \gamma_M} \leq 1 \qquad \text{... (5.9b)}$$

in which ψ_{vec} is the factor for vectorial effects defined in ENV 1993-1-1.

5.6 Combined compression and bending

(1)P Cross-sections subject to combined axial compression N_{Sd} and bending moments $M_{y,Sd}$ and $M_{z,Sd}$ shall satisfy the criterion:

$$\frac{N_{Sd}}{f_y A_{eff} / \gamma_M} + \frac{M_{y,Sd} + \Delta M_{y,Sd}}{f_y W_{eff,y,com} / \gamma_M} + \frac{M_{z,Sd} + \Delta M_{z,Sd}}{f_y W_{eff,z,com} / \gamma_M} \leq 1 \qquad \text{... (5.10a)}$$

in which A_{eff} is as defined in 5.3, $W_{eff,y,com}$ and $W_{eff,z,com}$ are as defined in 5.5 and $\gamma_M = \gamma_{M0}$ if $A_{eff} = A_g$, otherwise $\gamma_M = \gamma_{M1}$.

(2)P The additional moments $\Delta M_{y,Sd}$ and $\Delta M_{z,Sd}$ due to shifts of the centroidal axes shall be taken as:

$$\Delta M_{y,Sd} = N_{Sd} e_{Ny}$$

$$\Delta M_{z,Sd} = N_{Sd} e_{Nz}$$

in which e_{Ny} and e_{Nz} are the shifts of the centroidal axes in the y and z directions, see 5.3(3)P.

(3)P If $W_{eff,y,com} \geq W_{eff,y,ten}$ or $W_{eff,z,com} \geq W_{eff,z,ten}$ the following criterion shall also be satisfied:

$$\frac{M_{y,Sd} + \Delta M_{y,Sd}}{f_y W_{eff,y,ten} / \gamma_M} + \frac{M_{z,Sd} + \Delta M_{z,Sd}}{f_y W_{eff,z,ten} / \gamma_M} - \frac{\psi_{vec} N_{Sd}}{f_y A_g / \gamma_M} \leq 1 \qquad \text{... (5.10b)}$$

in which $W_{eff,y,ten}$, $W_{eff,z,ten}$ and ψ_{vec} are as defined in 5.5.

5.7 Torsional moment

(1)P Where loads are applied eccentric to the shear centre of the cross-section, the effects of torsion shall be taken into account.

NOTE: As far as practicable, torsional moments are best avoided or reduced by restraints, because they substantially reduce the load bearing capacity, especially with open sections.

(2) The centroidal axis and shear centre to be used in determining the effects of the torsional moment, should be taken as those of the effective cross-section for the bending moment due to the relevant load.

(3) The direct stresses due to the axial force N_{Sd} and the bending moments $M_{y,Sd}$ and $M_{z,Sd}$ should be based on the respective effective cross-sections used in 5.2 to 5.4. The shear stresses due to transverse shear forces, the shear stress due to uniform (St. Venant) torsion and the direct stresses and shear stresses due to warping, should all be based on the properties of the gross cross-section.

(4) In cross-sections subject to torsion, the following conditions should be satisfied:

$$\sigma_{tot,Ed} \leq f_y / \gamma_M \qquad \text{... (5.11a)}$$

$$\tau_{tot,Ed} \leq \left(f_y / \sqrt{3}\right) / \gamma_{M0} \qquad \text{... (5.11b)}$$

$$\sqrt{\sigma_{tot,Ed}^2 + 3\tau_{tot,Ed}^2} \leq 1{,}1 f_y / \gamma_M \qquad \text{... (5.11c)}$$

where:

$\sigma_{tot,Ed}$ is the total direct stress, calculated on the relevant effective cross-section;

$\tau_{tot,Ed}$ is the total shear stress, calculated on the gross cross-section.

and $\gamma_M = \gamma_{M0}$ if $W_{eff} = W_{e\ell}$ for each axis about which a bending moment acts, otherwise $\gamma_M = \gamma_{M1}$.

(5) The total direct stress $\sigma_{tot,Ed}$ and the total shear stress $\tau_{tot,Ed}$ should by obtained from:

$$\sigma_{tot,Ed} = \sigma_{N,Ed} + \sigma_{My,Ed} + \sigma_{Mz,Ed} + \sigma_{w,Ed} \qquad \text{... (5.12a)}$$

$$\tau_{tot,Ed} = \tau_{Vy,Ed} + \tau_{Vz,Ed} + \tau_{t,Ed} + \tau_{w,Ed} \qquad \text{... (5.12b)}$$

where:

- $\sigma_{My,Ed}$ is the direct stress due to the bending moment $M_{y,Sd}$;
- $\sigma_{Mz,Ed}$ is the direct stress due to the bending moment $M_{z,Sd}$;
- $\sigma_{N,Ed}$ is the direct stress due to the axial force N_{Sd};
- $\sigma_{w,Ed}$ is the direct stress due to warping;
- $\tau_{Vy,Ed}$ is the shear stress due to the transverse shear force $V_{y,Sd}$;
- $\tau_{Vz,Ed}$ is the shear stress due to the transverse shear force $V_{z,Sd}$;
- $\tau_{t,Ed}$ is the shear stress due to uniform (St. Venant) torsion;
- $\tau_{w,Ed}$ is the shear stress due to warping.

5.8 Shear force

(1)P The shear resistance of the web $V_{w,Rd}$ shall be taken as the lesser of the shear buckling resistance $V_{b,Rd}$ and the plastic shear resistance $V_{p\ell,Rd}$.

(2) The plastic shear resistance $V_{p\ell,Rd}$ should be checked in the case of a web without longitudinal stiffeners if $s_w/t \leq 72\varepsilon(f_{yb}/f_y)(\gamma_{M0}/\gamma_{M1})$ or generally if $\bar{\lambda}_w \leq 0{,}83(f_{yb}/f_y)(\gamma_{M0}/\gamma_{M1})$.

(3)P The shear buckling resistance $V_{b,Rd}$ shall be determined from:

$$V_{b,Rd} = (h_w/\sin\phi)\, t f_{bv}/\gamma_{M1} \qquad \text{... (5.13)}$$

where:

- f_{bv} is the shear buckling strength;
- h_w is the web height between the midlines of the flanges, see figure 3.3(c);
- ϕ is the slope of the web relative to the flanges.

(4)P The plastic shear resistance $V_{p\ell,Rd}$ shall be determined from:

$$V_{p\ell,Rd} = (h_w/\sin\phi)\, t\,(f_y/\sqrt{3})/\gamma_{M0} \qquad \text{... (5.14)}$$

(5)P The shear buckling strength f_{bv} for the appropriate value of the relative web slenderness $\bar{\lambda}_w$ shall be obtained from table 5.2.

Table 5.2: Shear buckling strength f_{bv}

Relative web slenderness	Web without stiffening at the support	Web with stiffening at the support [1)
$\bar{\lambda}_w < 1{,}40$	$0{,}48 f_{yb}/\bar{\lambda}_w$	$0{,}48 f_{yb}/\bar{\lambda}_w$
$\bar{\lambda}_w \geq 1{,}40$	$0{,}67 f_{yb}/\bar{\lambda}_w^2$	$0{,}48 f_{yb}/\bar{\lambda}_w$

1) Stiffening at the support, such as cleats, arranged to prevent distortion of the web and designed to resist the support reaction.

(6)P The relative web slenderness $\bar{\lambda}_w$ shall be obtained from the following:

$$\bar{\lambda}_w = \sqrt{\frac{f_{yb}/\sqrt{3}}{\tau_{cr}}} \equiv \frac{b_p}{t}\sqrt{\frac{12(1-\nu^2)f_{yb}}{\sqrt{3}\,\pi^2 E k_\tau}} \qquad \text{... (5.15a)}$$

- for webs without longitudinal stiffeners:

$$\bar{\lambda}_w = 0{,}346\,\frac{s_w}{t}\sqrt{\frac{f_{yb}}{E}} \cong \frac{s_w/t}{86{,}4\,\varepsilon} \qquad \text{... (5.15b)}$$

- for webs with longitudinal stiffeners, see figure 5.7:

$$\left.\begin{aligned} \bar{\lambda}_w &= 0{,}346\,\frac{s_d}{t}\sqrt{\frac{5{,}34}{k_\tau}\frac{f_{yb}}{E}} \cong \frac{s_d/t}{86{,}4\varepsilon}\sqrt{\frac{5{,}34}{k_\tau}} \\ \text{but } \bar{\lambda}_w &\geq 0{,}346\,\frac{s_p}{t}\sqrt{\frac{f_{yb}}{E}} \cong \frac{s_p/t}{86{,}4\varepsilon} \end{aligned}\right\} \qquad \text{... (5.15c)}$$

with:

$$k_\tau = 5{,}34 + \frac{2{,}10}{t}\sqrt[3]{\frac{I_s}{s_d}}$$

where:

I_s is the second moment of area of the longitudinal stiffener as defined in 4.3.4.3(7), about the axis a - a as indicated in figure 5.7;

s_d is the total developed slant height of the web, as indicated in figure 5.7;

s_p is the slant height of the largest plane element in the web, see figure 5.7;

s_w is the slant height of the web, as shown in figure 5.7, between the midpoints of the corners, see figure 3.3(c).

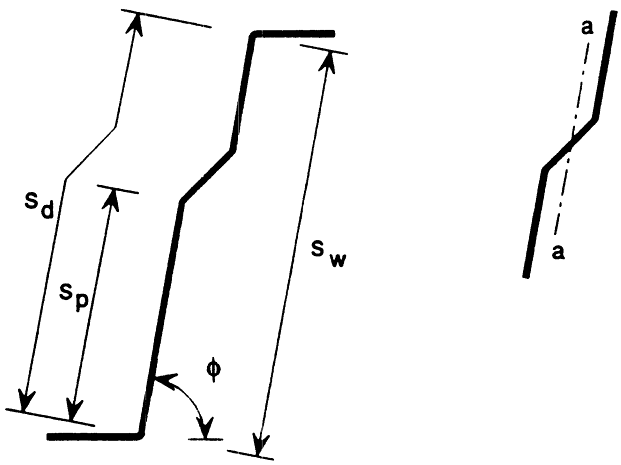


Figure 5.7: Longitudinally stiffened web

5.9 Local transverse forces

5.9.1 General

(1)P To avoid crushing, crippling or buckling in a web subject to a support reaction or other local transverse force applied through the flange, the transverse force F_{Sd} shall satisfy:

$$F_{Sd} \leq R_{w,Rd} \qquad \text{... (5.16)}$$

where:

$R_{w,Rd}$ is the local transverse resistance of the web.

(2)P The local transverse resistance of a web $R_{w,Rd}$ shall be obtained as follows:

a) for an unstiffened web:

- for a cross-section with a single web: from 5.9.2;
- for any other case, including sheeting: from 5.9.3;

b) for a stiffened web: from 5.9.4.

(3) Where the local load or support reaction is applied through a cleat that is arranged to prevent distortion of the web and is designed to resist the local transverse force, the local resistance of the web to the transverse force need not be considered.

(4) In beams with I-shaped cross-sections built up from two channels, or with similar cross-sections in which two components are interconnected through their webs, the connections between the webs should be located as close as practicable to the flanges of the beam.

5.9.2 Cross-sections with a single unstiffened web

(1) For a cross-section with a single unstiffened web, see figure 5.8, the local transverse resistance of the web may be determined as specified in (2), provided that the cross-section satisfies the following criteria:

$$h_w/t \leq 200 \qquad \text{... (5.17a)}$$

$$r/t \leq 6 \qquad \text{... (5.17b)}$$

$$45° \leq \phi \leq 90° \qquad \text{... (5.17c)}$$

where:

h_w is the web height between the midlines of the flanges;

r is the internal radius of the corners;

ϕ is the slope of the web relative to the flanges [degrees].

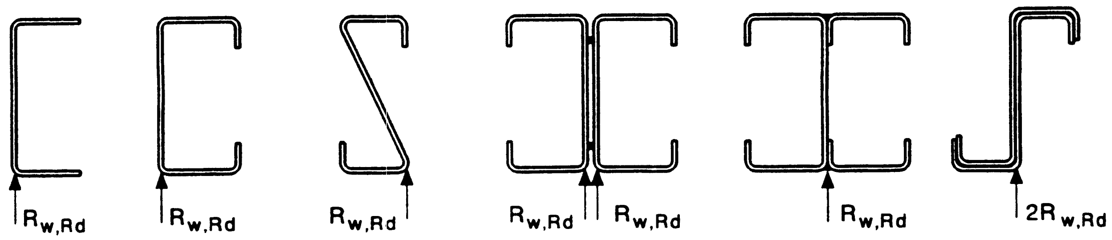


Figure 5.8: Examples of cross-sections with a single web

(2) For cross-sections that satisfy the criteria specified in (1), the local transverse resistance of a web $R_{w,Rd}$ may be determined as follows:

a) for a single local load or support reaction, see figure 5.9(a):

i) $c \leq 1{,}5\, h_w$ clear from a free end:

- for a cross-section with stiffened flanges:

$$R_{w,Rd} = k_1 k_2 k_3 \, [9{,}04 - (h_w/t)/60] \, [1 + 0{,}01\,(s_s/t)] \, t^2 f_{yb}/\gamma_{M1} \qquad \ldots (5.18a)$$

- for a cross-section with unstiffened flanges:

- if $s_s/t \leq 60$:

$$R_{w,Rd} = k_1 k_2 k_3 \, [5{,}92 - (h_w/t)/132] \, [1 + 0{,}01\,(s_s/t)] \, t^2 f_{yb}/\gamma_{M1} \qquad \ldots (5.18b)$$

- if $s_s/t > 60$:

$$R_{w,Rd} = k_1 k_2 k_3 \, [5{,}92 - (h_w/t)/132] \, [0{,}71 + 0{,}015\,(s_s/t)] \, t^2 f_{yb}/\gamma_{M1} \qquad \ldots (5.18c)$$

ii) $c > 1{,}5\, h_w$ clear from a free end:

- if $s_s/t \leq 60$:

$$R_{w,Rd} = k_3 k_4 k_5 \, [14{,}7 - (h_w/t)/49{,}5] \, [1 + 0{,}007\,(s_s/t)] \, t^2 f_{yb}/\gamma_{M1} \qquad \ldots (5.18d)$$

- if $s_s/t > 60$:

$$R_{w,Rd} = k_3 k_4 k_5 \, [14{,}7 - (h_w/t)/49{,}5] \, [0{,}75 + 0{,}011\,(s_s/t)] \, t^2 f_{yb}/\gamma_{M1} \qquad \ldots (5.18e)$$

b) for two opposing local transverse forces closer together than $1{,}5\, h_w$, see figure 5.9(b):

i) $c \leq 1{,}5\, h_w$ clear from a free end:

$$R_{w,Rd} = k_1 k_2 k_3 \, [6{,}66 - (h_w/t)/64] \, [1 + 0{,}01\,(s_s/t)] \, t^2 f_{yb}/\gamma_{M1} \qquad \ldots (5.18f)$$

ii) $c > 1{,}5\, h_w$ clear from a free end:

$$R_{w,Rd} = k_3 k_4 k_5 \, [21{,}0 - (h_w/t)/16{,}3] \, [1 + 0{,}0013\,(s_s/t)] \, t^2 f_{yb}/\gamma_{M1} \qquad \ldots (5.18g)$$

(3) The values of the constants k_1 to k_5 should be determined as follows:

$k_1 = (1{,}33 - 0{,}33\,k)$

$k_2 = (1{,}15 - 0{,}15\,r/t)$ but $k_2 \geq 0{,}50$ and $k_2 \leq 1{,}0$

$k_3 = 0{,}7 + 0{,}3\,(\phi/90)^2$

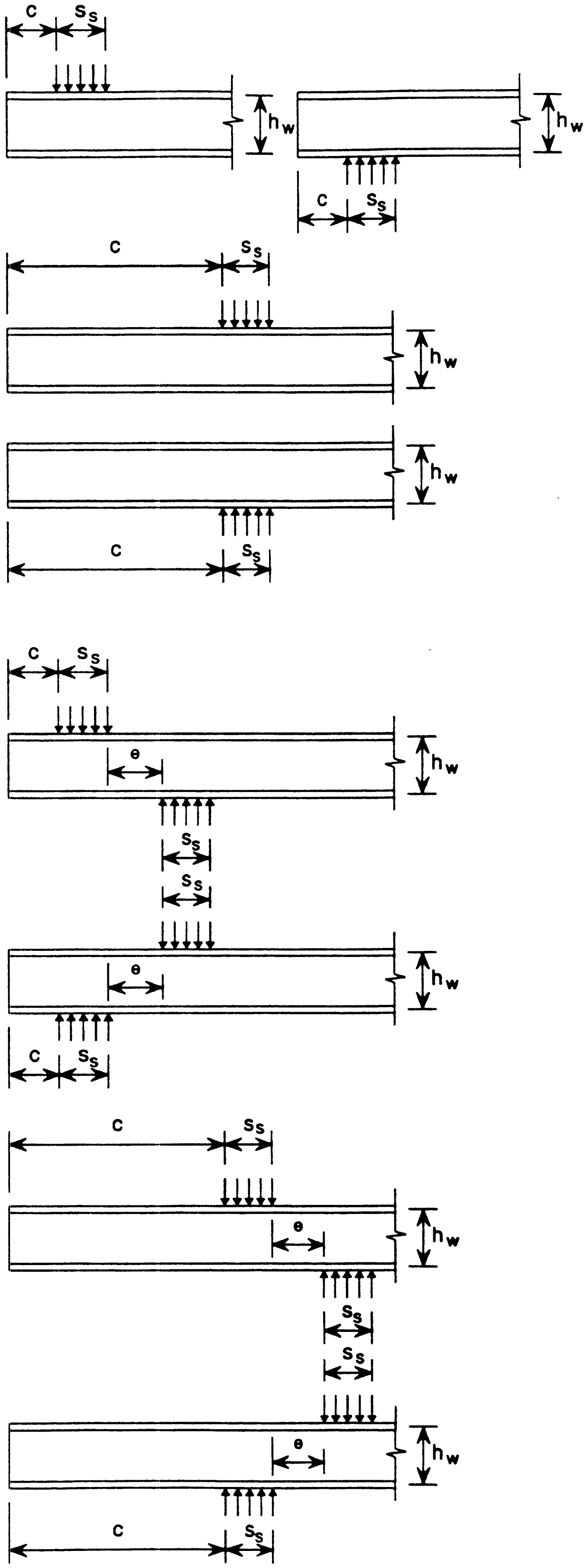
$k_4 = (1{,}22 - 0{,}22\,k)$

$k_5 = (1{,}06 - 0{,}06\,r/t)$ but $k_5 \leq 1{,}0$

where:

$k = f_{yb}/228$ [with f_{yb} in N/mm^2];

s_s is the actual length of stiff bearing. In the case of two equal and opposite local transverse forces distributed over unequal bearing lengths, the smaller value of s_s should be used.



(i) $c \leq 1{,}5h_w$

(ii) $c > 1{,}5h_w$

(a) Single load or support reaction

(i) $c \leq 1{,}5h_w$

(ii) $c > 1{,}5h_w$

(b) Opposing forces with $e < 1{,}5h_w$

Figure 5.9: Local loads and supports — cross-sections with a single web

5.9.3 Cross-sections with two or more unstiffened webs

(1) In cross-sections with two or more webs, including sheeting, see figure 5.10, the local transverse resistance of an unstiffened web should be determined as specified in (2), provided that both of the following conditions are satisfied:

- the clear distance c from the actual bearing length for the support reaction or local load to a free end, see figure 5.11, is at least 40 mm;
- the cross-section satisfies the following criteria:

$$r/t \leq 10 \quad \text{... (5.19a)}$$

$$h_w/t \leq 200 \sin \phi \quad \text{... (5.19b)}$$

$$45° \leq \phi \leq 90° \quad \text{... (5.19c)}$$

where:

h_w	is	the web height between the midlines of the flanges;
r	is	the internal radius of the corners;
ϕ	is	the slope of the web relative to the flanges [degrees].

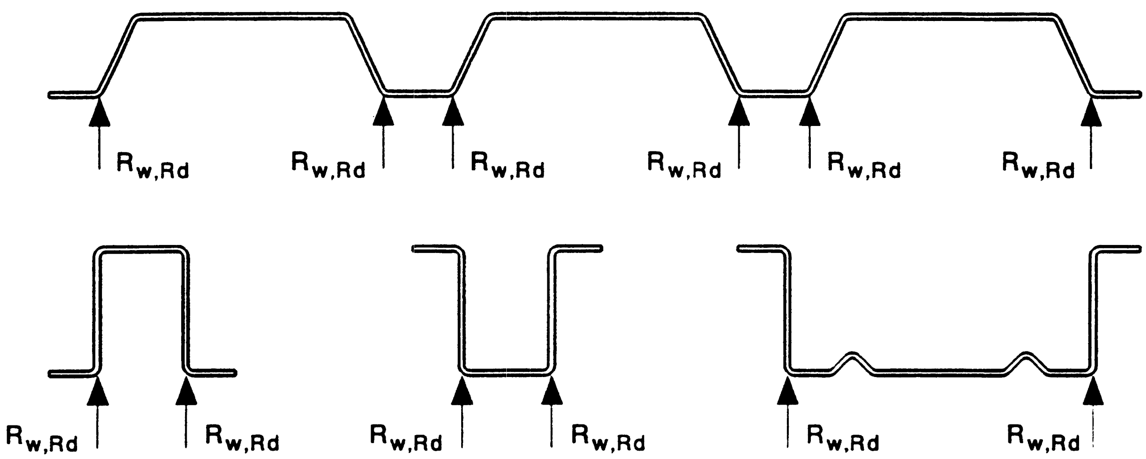


Figure 5.10: Examples of cross-sections with two or more webs

(2) Where both of the conditions specified in (1) are satisfied, the local transverse resistance $R_{w,Rd}$ per web of the cross-section should be determined from:

$$R_{w,Rd} = \alpha t^2 \sqrt{f_{yb} E} \left(1 - 0,1\sqrt{r/t}\right)\left[0,5 + \sqrt{0,02\,\ell_a/t}\right]\left(2,4 + (\phi/90)^2\right)/\gamma_{M1} \quad \text{... (5.20)}$$

where:

ℓ_a	is	the effective bearing length for the relevant category, see (3);
s_s	is	the actual length of stiff bearing;
α	is	the coefficient for the relevant category, see (3).

(3) The values of ℓ_a and α should be obtained from (4) and (5) respectively. The relevant category (1 or 2) should be based on the clear distance e between the local load and the nearest support, or the clear distance c from the support reaction or local load to a free end, see figure 5.11, as follows:

a) Category 1, see figure 5.11(a):

- local load applied with $e \leq 1{,}5\, h_w$ clear from the nearest support;
- local load applied with $c \leq 1{,}5\, h_w$ clear from a free end;
- reaction at end support with $c \leq 1{,}5\, h_w$ clear from a free end.

b) Category 2, see figure 5.11(b):

- local load applied with $e > 1{,}5\, h_w$ clear from the nearest support;
- local load applied with $c > 1{,}5\, h_w$ clear from a free end;
- reaction at end support with $c > 1{,}5\, h_w$ clear from a free end;
- reaction at internal support.

(4) The value of the effective bearing length ℓ_a should be obtained from the following:

a) for Category 1: $\ell_a = 10\,\text{mm}$ ... (5.21a)

b) for Category 2:

- $\beta_V \leq 0{,}2$: $\ell_a = s_s$ ... (5.21b)
- $\beta_V \geq 0{,}3$: $\ell_a = 10\,\text{mm}$ ... (5.21c)
- $0{,}2 < \beta_V < 0{,}3$: Interpolate linearly between the values of ℓ_a for 0,2 and 0,3.

with:

$$\beta_V = \frac{|V_{Sd,1}| - |V_{Sd,2}|}{|V_{Sd,1}| + |V_{Sd,2}|}$$

in which $|V_{Sd,1}|$ and $|V_{Sd,2}|$ are the absolute values of the transverse shear forces on each side of the local load or support reaction, and $|V_{Sd,1}| \geq |V_{Sd,2}|$.

(5) The value of the coefficent α should be obtained from the following:

a) for Category 1:

- for sheeting profiles: $\alpha = 0{,}075$ ... (5.22a)
- for liner trays and hat sections: $\alpha = 0{,}057$ ... (5.22b)

b) for Category 2:

- for sheeting profiles: $\alpha = 0{,}15$ ... (5.22c)
- for liner trays and hat sections: $\alpha = 0{,}115$ ... (5.22d)

(i) $e \leq 1{,}5h_w$

(ii) $c \leq 1{,}5h_w$

(a) Category 1

(i) $e > 1{,}5h_w$

(ii) $c > 1{,}5h_w$

(iii) Intermediate support

(b) Category 2

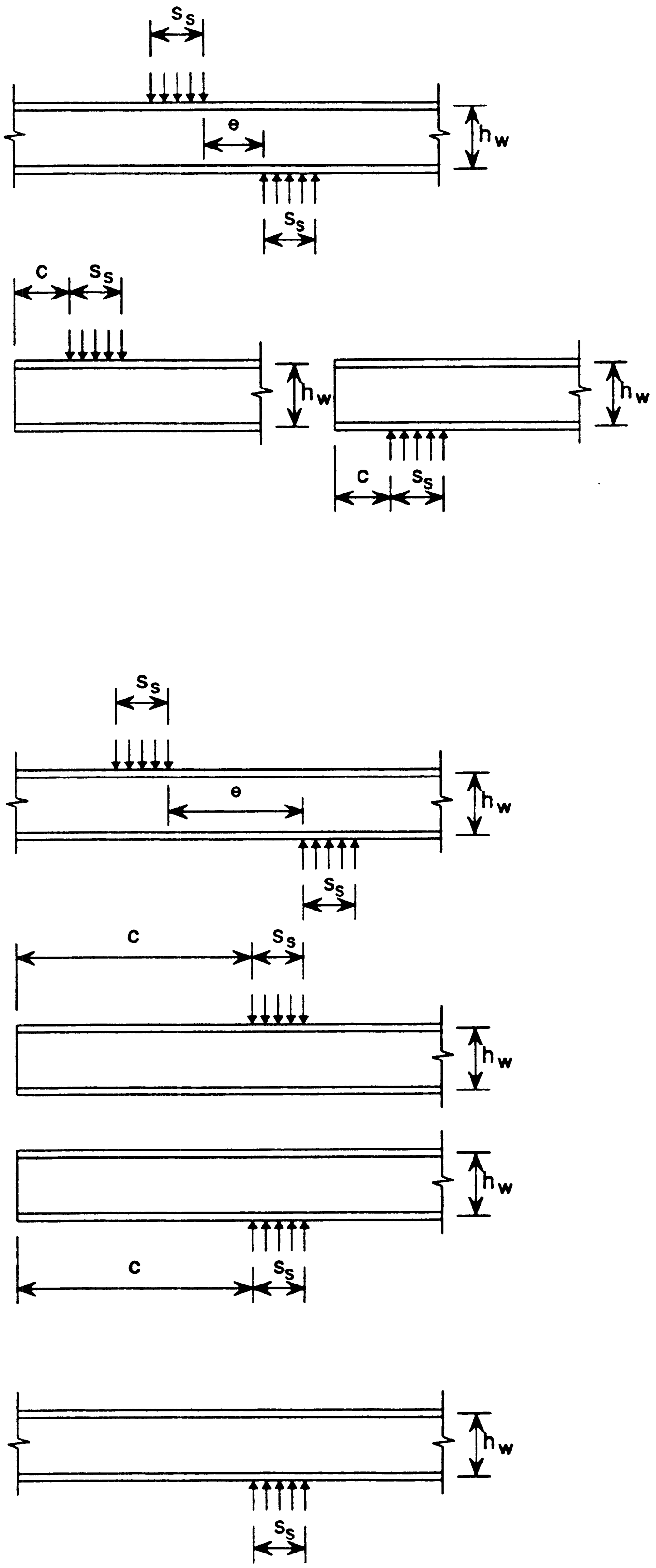


Figure 5.11: Local loads and supports — cross-sections with two or more webs

5.9.4 Stiffened webs

(1) The local transverse resistance of a stiffened web may be determined as specified in (2) for cross-sections with longitudinal web stiffeners folded in such a way that the two folds in the web are on opposite sides of the system line of the web joining the points of intersection of the midline of the web with the midlines of the flanges, see figure 5.12, that satisfy the condition:

$$2 \quad < \quad e_{max}/t \quad < \quad 12 \qquad \text{... (5.23)}$$

where:

e_{max} is the larger eccentricity of the folds relative to the system line of the web.

(2) For cross-sections with stiffened webs satisfying the conditions specified in (1), the local transverse resistance of a stiffened web may be determined by multiplying the corresponding value for a similar unstiffened web, obtained from 5.9.2 or 5.9.3 as appropriate, by the factor $\kappa_{a,s}$ given by:

$$\kappa_{a,s} = 1{,}45 - 0{,}05\, e_{max}/t \qquad \text{but} \quad \kappa_{a,s} \leq 0{,}95 + 35\,000\, t^2\, e_{min}/(b_l^{\,2}\, h_p) \qquad \text{... (5.24)}$$

where:

b_d is the developed width of the loaded flange, see figure 5.12;

e_{min} is the smaller eccentricity of the folds relative to the system line of the web;

s_p is the slant height of the plane web element nearest to the loaded flange, see figure 5.12.

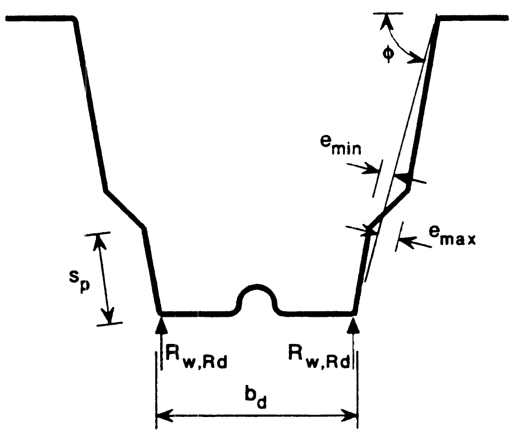


Figure 5.12: Stiffened webs

5.10 Combined shear force and bending moment

(1)P Cross-sections subject to the combined action of a bending moment M_{Sd} and a shear force V_{Sd} shall satisfy:

$$\left[\frac{M_{Sd}}{M_{c,Rd}}\right]^2 + \left[\frac{V_{Sd}}{V_{w,Rd}}\right]^2 \leq 1 \qquad \text{... (5.25)}$$

where:

$M_{c,Rd}$ is the moment resistance of the cross-section given in 5.4.1(1)P;

$V_{w,Rd}$ is the shear resistance of the web given in 5.8(1)P.

5.11 Combined bending moment and local load or support reaction

(1)P Cross-sections subject to the combined action of a bending moment M_{Sd} and a transverse force due to a local load or support reaction F_{Sd} shall satisfy the following:

$$M_{Sd}/M_{c,Rd} \leq 1 \qquad \text{... (5.26a)}$$

$$F_{Sd}/R_{w,Rd} \leq 1 \qquad \text{... (5.26b)}$$

$$\frac{M_{Sd}}{M_{c,Rd}} + \frac{F_{Sd}}{R_{w,Rd}} \leq 1{,}25 \qquad \text{... (5.26c)}$$

where:

$M_{c,Rd}$ is the moment resistance of the cross-section given in 5.4.1(1)P;

$R_{w,Rd}$ is the appropriate value of the local transverse resistance of the web from 5.9.

6 Buckling resistance

6.1 General

(1)P The design values of the internal forces and moments in each member shall not exceed its design buckling resistance to:

- axial compression, as given in 6.2;
- bending moments, as given in 6.3;
- combined bending and axial compression, as given in 6.5.

(2)P In members with cross-sections that are susceptible to cross-sectional distortion, account shall be taken of possible lateral buckling of compression flanges and lateral bending of flanges generally, see 6.4.

(3)P The effects of local buckling shall be taken into account by using effective section properties determined as specified in Section 4.

(4)P The internal axial force in a member shall be taken as acting at the centroid of its gross cross-section.

(5) The resistance of a member to axial compression should be assumed to act at the centroid of its effective cross-section. If this does not coincide with the centroid of its gross cross-section, moments corresponding to the shift of the centroidal axes (see figure 6.1) should be taken into account, using the method given in 6.5.

(6)P Frame instability shall be taken into account as specified in ENV 1993-1-1.

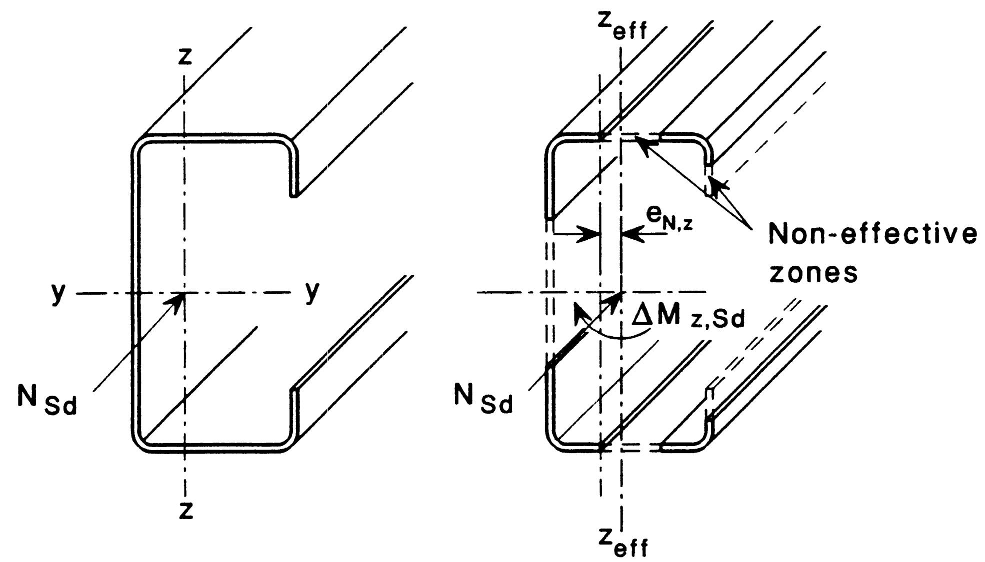


Figure 6.1: Shift of centroidal axis

6.2 Axial compression

6.2.1 Design buckling resistance

(1)P Unless determined by a second-order analysis of the member, see 6.2.2(6)P, the design buckling resistance for axial compression $N_{b,Rd}$ shall be obtained from:

$$N_{b,Rd} = \chi A_{eff} f_y / \gamma_{M1} \cong \chi \beta_A A_g f_y / \gamma_{M1} \qquad \text{... (6.1)}$$

where:

A_{eff} is the effective area of the cross-section, obtained from Section 4 by assuming a uniform compressive stress $\sigma_{com,Ed}$ equal to f_{yb}/γ_{M1} ;

A_g is the area of the gross cross-section;

χ is the appropriate value of the reduction factor for buckling resistance.

in which the reduction factor β_A is given by:

$$\beta_A = A_{eff}/A_g$$

(2)P The reduction factor χ for buckling resistance shall be determined from:

$$\chi = \frac{1}{\phi + [\phi^2 - \bar{\lambda}^2]^{0,5}} \quad \text{but} \quad \chi \leq 1,0 \qquad \text{... (6.2a)}$$

with:

$$\phi = 0,5[1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2] \qquad \text{... (6.2b)}$$

where:

α is an imperfection factor, depending on the appropriate buckling curve;

$\bar{\lambda}$ is the relative slenderness for the relevant buckling mode.

(3)P The lowest value of χ for flexural buckling of the member about any relevant axis, or for torsional or torsional-flexural buckling, shall be used.

(4)P The imperfection factor α corresponding to the appropriate buckling curve shall be obtained from table 6.1.

Table 6.1: Imperfection factor α

Buckling curve	a_0	a	b	c
α	0,13	0,21	0,34	0,49

6.2.2 Flexural buckling

(1)P The design buckling resistance $N_{b,Rd}$ for flexural buckling shall be obtained from 6.2.1 using the appropriate buckling curve from table 6.2 according to the type of cross-section and axis of buckling.

(2)P The buckling curve for a cross-section not included in table 6.2 may be obtained by analogy.

(3)P The buckling resistance of a closed built-up cross-section shall be determined using either:

- buckling curve b in association with the basic yield strength f_{yb} of the flat sheet material out of which the member is made by cold forming;
- buckling curve c in association with the average yield strength f_{ya} of the member after cold forming, determined as specified in 3.1.2, provided that $\beta_A = 1,0$.

(4)P The relative slenderness $\bar{\lambda}$ for flexural buckling about a given axis ($\bar{\lambda}_y$ or $\bar{\lambda}_z$) shall be determined from the following:

$$\bar{\lambda} = (\lambda / \lambda_1)[\beta_A]^{0,5} \qquad \text{... (6.3a)}$$

with:

$$\lambda = \ell / i \qquad \text{... (6.3b)}$$

$$\lambda_1 = \pi [E/f_y]^{0,5} \qquad \text{... (6.3c)}$$

where:

ℓ is the buckling length for flexural buckling about the relevant axis (ℓ_y or ℓ_z);

i is the radius of gyration about the corresponding axis (i_y or i_z), based on the properties of the gross cross-section.

(5) Reference should be made to ENV 1993-1-1 for information on determining the buckling length ℓ for flexural buckling of a compression member, from its system length L.

(6)P As an alternative to (1)P, the design buckling resistance $N_{b,Rd}$ for flexural buckling may be obtained from a second-order analysis of the member as specified in ENV 1993-1-1, based on the properties of the effective cross-section obtained from Section 4.

6.2.3 Torsional buckling and torsional-flexural buckling

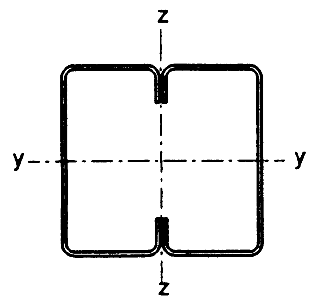
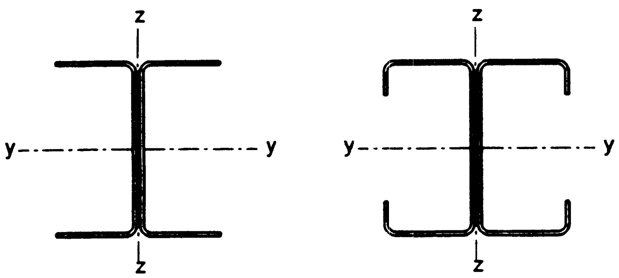
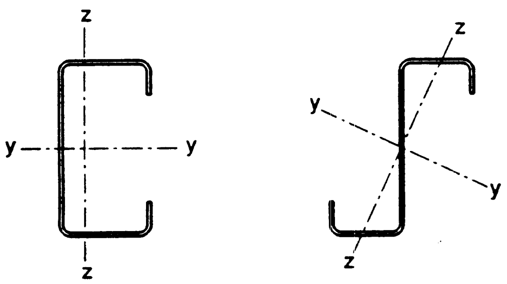
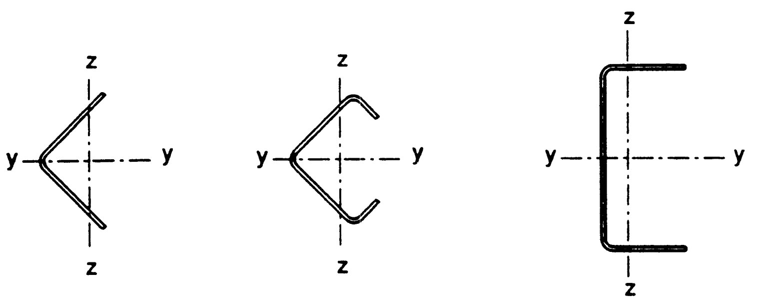
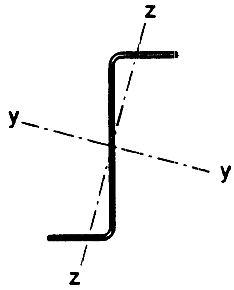
(1)P For members with point-symmetric open cross-sections, account shall be taken of the possibility that the resistance of the member to torsional buckling might be less than its resistance to flexural buckling.

(2)P For members with mono-symmetric open cross-sections, see figure 6.2, account shall be taken of the possibility that the resistance of the member to torsional-flexural buckling might be less than its resistance to flexural buckling.

(3)P For members with non-symmetric open cross-sections, account shall be taken of the possibility that the resistance of the member to either torsional or torsional-flexural buckling might be less than its resistance to flexural buckling.

(4)P The design buckling resistance $N_{b,Rd}$ for torsional or torsional-flexural buckling shall be obtained from 6.2.1 using buckling curve b.

Table 6.2: Appropriate buckling curve for various types of cross-section

<table>
<tr><th colspan="2">Type of cross-section</th><th>Buckling about axis</th><th>Buckling curve</th></tr>
<tr><td rowspan="2"></td><td>if f_{yb} is used</td><td>any</td><td>b</td></tr>
<tr><td>if f_{ya} is used *)</td><td>any</td><td>c</td></tr>
<tr><td colspan="2" rowspan="2"></td><td>y - y</td><td>a</td></tr>
<tr><td>z - z</td><td>b</td></tr>
<tr><td colspan="2"></td><td>any</td><td>b</td></tr>
<tr><td colspan="2">

or other cross-section</td><td>any</td><td>c</td></tr>
<tr><td colspan="4">*) The average yield strength f_{ya} should not be used unless $A_{eff} = A_g$</td></tr>
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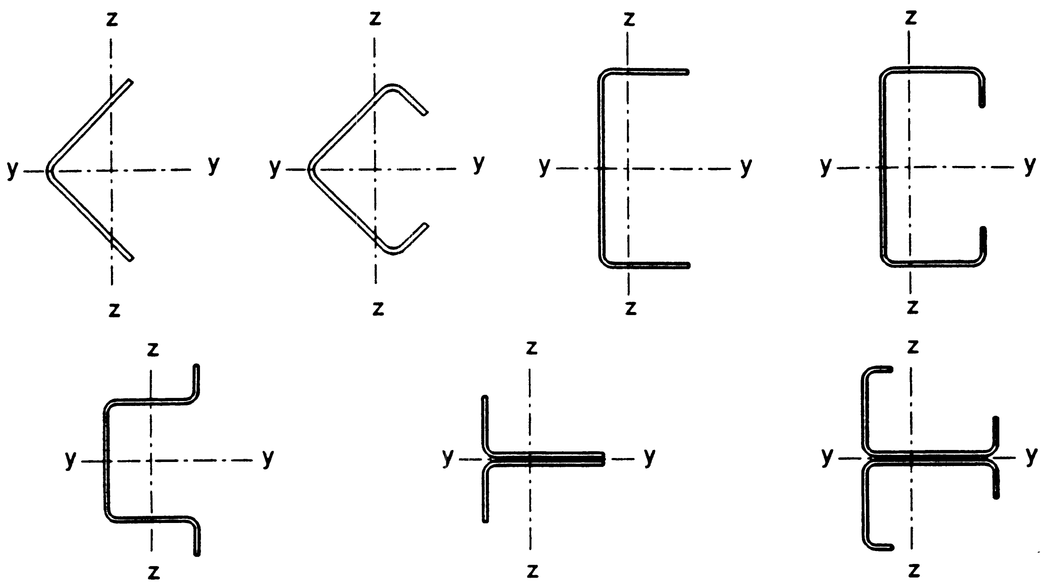


Figure 6.2: Cross-sections susceptible to torsional-flexural buckling

(5)P The relative slenderness $\bar{\lambda}$ for torsional or torsional-flexural buckling shall be obtained from:

$$\bar{\lambda} = (f_{yb} / \sigma_{cr})[\beta_A]^{1/2} \qquad \text{... (6.4a)}$$

with:

$$\sigma_{cr} = \sigma_{cr,TF} \quad \text{but} \quad \sigma_{cr} \leq \sigma_{cr,T} \qquad \text{... (6.4b)}$$

where:

- $\sigma_{cr,T}$ is the elastic critical stress for torsional buckling, see (6)P;
- $\sigma_{cr,TF}$ is the elastic critical stress for torsional-flexural buckling, see (7)P.

(6)P The elastic critical stress $\sigma_{cr,T}$ for torsional buckling shall be determined from:

$$\sigma_{cr,T} = \frac{1}{A_g i_o^2}\left[GI_t + \frac{\pi^2 EI_w}{\ell_T^2}\right] \qquad \text{... (6.5a)}$$

with:

$$i_o^2 = i_y^2 + i_z^2 + y_o^2 \qquad \text{... (6.5b)}$$

where:

- G is the shear modulus;
- I_t is the torsion constant of the gross cross-section;
- I_w is the warping constant of the gross cross-section;
- i_y is the radius of gyration of the gross cross-section about the y - y axis;
- i_z is the radius of gyration of the gross cross-section about the z - z axis;
- ℓ_T is the buckling length of the member for torsional buckling;
- y_0 is the distance from the shear centre to the centroid of the gross cross-section.

(7)P For cross-sections that are symmetrical about the y - y axis, the elastic critical stress $\sigma_{cr,TF}$ for torsional-flexural buckling shall be determined from:

$$\sigma_{cr,TF} = \frac{1}{2\beta}\left[(\sigma_{cr,y} + \sigma_{cr,T}) - \sqrt{(\sigma_{cr,y} + \sigma_{cr,T})^2 - 4\beta\sigma_{cr,y}\sigma_{cr,T}}\right] \quad \text{... (6.6)}$$

with:

$$\sigma_{cr,y} = \pi^2 E/(\ell_y/i_y)^2$$

$$\beta = 1 - (y_o/i_o)^2$$

where:

ℓ_y is the buckling length for flexural buckling about the y - y axis.

(8)P The buckling length ℓ_T for torsional or torsional-flexural buckling shall be determined taking into account the degree of torsional and warping restraint at each end of the system length L_T.

(9) Normal practical connections should not be assumed to provide full torsional or warping restraint and therefore the theoretical values of ℓ_T/L_T (1,0 for "torsion fixed, warping free" or 0,5 for "torsion fixed, warping fixed") should not normally be used directly in design.

(10) For practical connections at each end, the value of ℓ_T/L_T may be taken as follows:

- 1,0 for connections that provide partial restraint against torsion and warping, see figure 6.3(a);
- 0,7 for connections that provide significant restraint against torsion and warping, see figure 6.3(b).

(11) Improved values of ℓ_T/L_T may be used where this is justified by tests in accordance with Section 9.

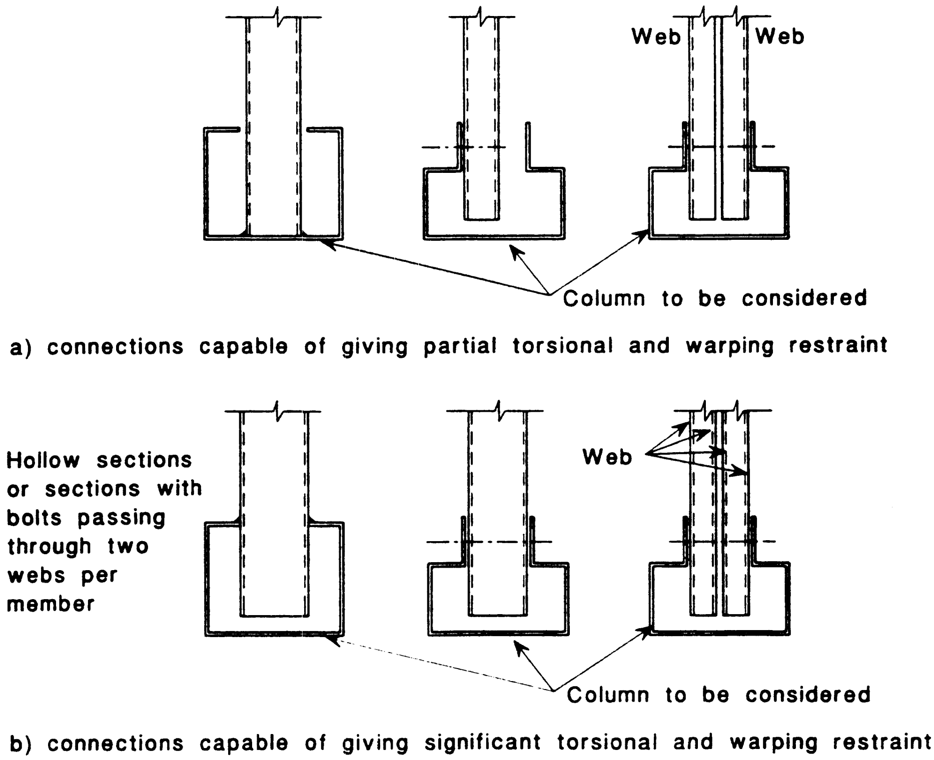


Figure 6.3: Torsional and warping restraint from practical connections

6.3 Lateral-torsional buckling of members subject to bending

(1)P The design buckling resistance moment of a member that is susceptible to lateral-torsional buckling shall be determined from:

$$M_{b,Rd} = \chi_{LT} W_{eff} f_{yb} / \gamma_{M1} \qquad \text{... (6.7)}$$

in which $\bar{\lambda}_{LT}$ is obtained from the following:

- if $\bar{\lambda}_{LT} \leq 0,4$:

$$\chi_{LT} = 1,0 \qquad \text{... (6.8a)}$$

- if $\bar{\lambda}_{LT} > 0,4$:

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \bar{\lambda}_{LT}^2]^{0,5}} \qquad \text{... (6.8b)}$$

with:

$$\phi_{LT} = 0,5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0,2\right) + \bar{\lambda}_{LT}^2\right] \qquad \text{... (6.9a)}$$

$$\bar{\lambda}_{LT} = [f_y W_{eff} / M_{cr}]^{0,5} \qquad \text{... (6.9b)}$$

α_{LT} = 0,21 [buckling curve a in table 6.1]

where:

M_{cr} is the elastic critical moment of the gross cross-section, for lateral-torsional buckling about the relevant axis;

W_{eff} is the section modulus of the effective cross-section, if subject only to moment about the relevant axis.

NOTE: Information for the calculation of M_{cr} is given in annex F of ENV 1993-1-1.

(2) This method should not be used for U-sections and similar sections that have a significant angle between the principal axes of the effective cross-section, compared to those of the gross cross-section.

6.4 Distortional buckling

(1)P Distortional buckling shall be taken into account where it constitutes the critical failure mode.

(2) The effects of distortional buckling should be allowed for in cases such as those indicated in figures 6.4(a), (b) and (c), if the lowest elastic critical stress for a distortional buckling mode, evaluated by examining the various possible deformation modes, is lower than the elastic critical stresses for local and overall buckling, as indicated in figure 6.5.

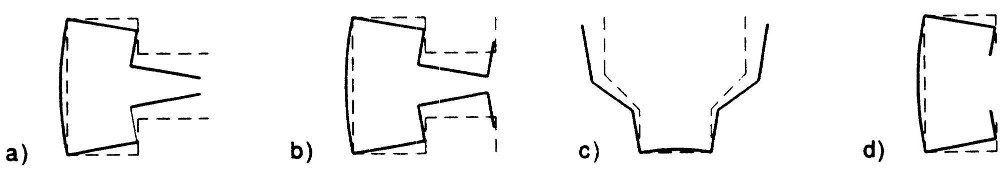


Figure 6.4: Examples of distortional buckling modes

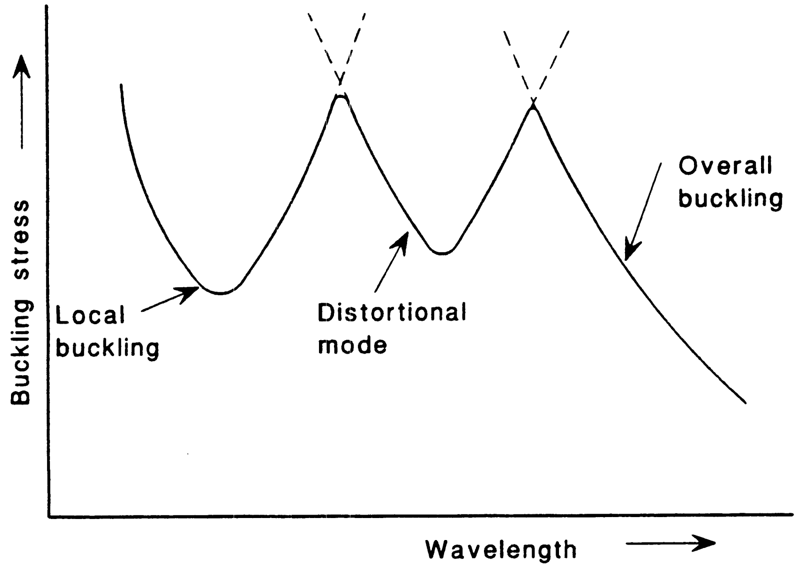


Figure 6.5: Elastic critical buckling stresses for various failure modes

(3) For elements with edge or intermediate stiffeners as indicated in figure 6.4(d), no further allowance need be made for distortional buckling if the effective area of the stiffener is reduced as specified in 4.3.

6.5 Bending and axial compression

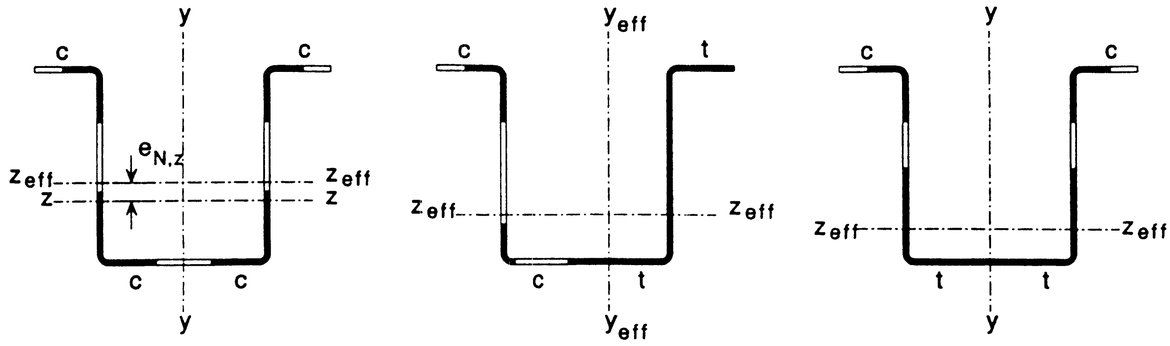
6.5.1 General

(1)P All members subject to combined bending and axial compression shall satisfy the criterion:

$$\frac{N_{Sd}}{\chi_{min} f_{yb} A_{eff} / \gamma_{M1}} + \frac{\kappa_y (M_{y,Sd} + \Delta M_{y,Sd})}{f_{yb} W_{eff,y,com} / \gamma_{M1}} + \frac{\kappa_z (M_{z,Sd} + \Delta M_{z,Sd})}{f_{yb} W_{eff,z,com} / \gamma_{M1}} \leq 1 \quad \text{... (6.10)}$$

where:

A_{eff}	is	the effective area of an effective cross-section that is subject only to axial compression, see figure 6.6(a);
$W_{eff,y,com}$	is	the effective section modulus for the maximum compressive stress in an effective cross-section that is subject only to moment about the y - y axis, see figure 6.6(b);
$W_{eff,z,com}$	is	the effective section modulus for the maximum compressive stress in an effective cross-section that is subject only to moment about the z - z axis, see figure 6.6(c);
$\Delta M_{y,Sd}$	is	the additional moment due to possible shift of the centroidal axis in the y direction, see 5.6(2)P;
$\Delta M_{z,Sd}$	is	the additional moment due to possible shift of the centroidal axis in the z direction, see 5.6(2)P;
χ_y	is	the reduction factor from 6.2 for buckling about the y - y axis;
χ_z	is	the reduction factor from 6.2 for buckling about the z - z axis;
χ_{min}	is	the lesser of χ_y and χ_z.



a) Axial compression b) Moment about y-y axis c) Moment about z-z axis

c = compression t = tension

Figure 6.6: Calculation of effective section properties

(2)P The factors κ_y and κ_z in expression (6.10) shall be obtained from:

$$\kappa_y = 1 - \frac{\mu_y N_{Sd}}{\chi_y f_{yb} A_{eff}} \quad \text{but } \kappa_y \leq 1{,}50 \qquad \ldots (6.11a)$$

$$\kappa_z = 1 - \frac{\mu_z N_{Sd}}{\chi_z f_{yb} A_{eff}} \quad \text{but } \kappa_z \leq 1{,}50 \qquad \ldots (6.11b)$$

with:

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

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

where:

$\beta_{M,y}$ is the equivalent uniform moment factor for buckling about the y - y axis;

$\beta_{M,z}$ is the equivalent uniform moment factor for buckling about the z - z axis;

NOTE: The expressions for μ_y and μ_z can result in negative values.

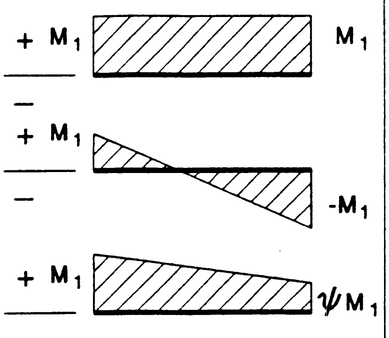
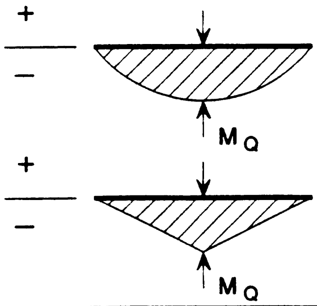
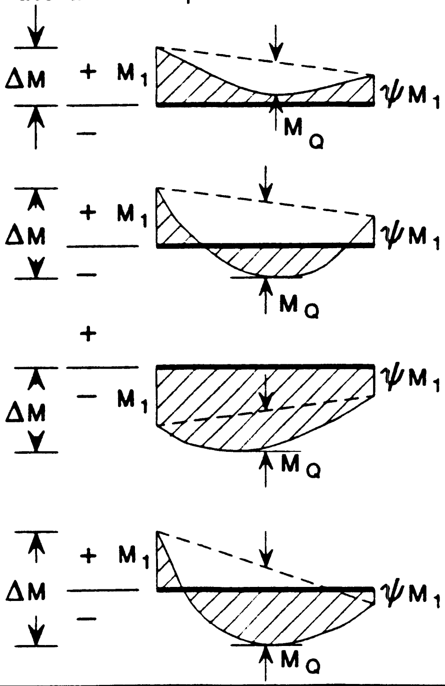
(3)P The equivalent uniform moment factors $\beta_{M,y}$ and $\beta_{M,z}$ shall be based on the shape of the bending moment diagram about the relevant axis, between points that are braced in the relevant direction, as given in table 6.3. The bending moments taken into account shall include the additional moments $\Delta M_{y,Sd}$ and $\Delta M_{z,Sd}$ due to possible shift of the centroidal axes.

(4)P The equivalent uniform moment factors $\beta_{M,y}$ and $\beta_{M,z}$ shall be determined from table 6.4.

Table 6.3: Relevant axes for determining β_M factors

Factor	Diagram of bending moments applied about axis:	Lateral buckling about axis:	System length taken between points braced in direction:
$\beta_{M,y}$	y - y	y - y	z - z
$\beta_{M,z}$	z - z	z - z	y - y
$\beta_{M,LT}$	y - y	z - z	y - y

Table 6.4: Equivalent uniform moment factors

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

6.5.2 Bending and axial compression with lateral-torsional buckling

(1)P Members that are susceptible to lateral-torsional buckling shall also satisfy the criterion:

$$\frac{N_{Sd}}{\chi_{lat} f_{yb} A_{eff} / \gamma_{M1}} + \frac{\kappa_{LT}(M_{y,Sd} + \Delta M_{y,Sd})}{\chi_{LT} f_{yb} W_{eff,y,com} / \gamma_{M1}} + \frac{\kappa_z (M_{z,Sd} + \Delta M_{z,Sd})}{f_{yb} W_{eff,z,com} / \gamma_{M1}} \leq 1 \qquad \text{... (6.12)}$$

where:

χ_{LT} is the reduction factor for lateral-torsional buckling, see 6.3.

(2)P Generally the reduction factor χ_{lat} shall be taken as equal to χ_z. However if torsional-flexural buckling (see 6.2.3) or distortional buckling (see 6.3) are potential failure modes, χ_{lat} shall be taken as equal to the smallest of χ_z and the values of χ for torsional-flexural or distortional buckling.

(3)P The factor κ_{LT} in expression (6.12) shall be obtained from:

$$\kappa_{LT} = 1 - \frac{\mu_{LT} N_{Sd}}{\chi_z f_{yb} A_{eff}} \quad \text{but} \quad \kappa_{LT} \leq 1{,}0 \qquad \text{... (6.13a)}$$

with:

$$\mu_{LT} = 0{,}15 \bar{\lambda}_{lat} \beta_{M,LT} - 0{,}15 \quad \text{but} \quad \mu_{LT} \leq 0{,}90 \qquad \text{... (6.13b)}$$

(4)P The equivalent uniform moment factor for lateral-torsional buckling $\beta_{M,LT}$ shall be based on the shape of the bending moment diagram about the y - y axis, between points that are braced in the y - y direction, as also given in table 6.3. The bending moments taken into account shall include the additional moment $\Delta M_{y,Sd}$ due to possible shift of the centroidal axis.

(5)P The equivalent uniform moment factor $\beta_{M,LT}$ shall be determined from table 6.4.

7 Serviceability limit states

7.1 General

(1)P The principles for serviceability limit states given in Section 4 of ENV 1993-1-1 shall also be applied to cold formed thin gauge members and sheeting.

(2) The application rules given in Section 4 of ENV 1993-1-1 should also be applied to cold formed thin gauge members and sheeting, except as modified by the supplementary application rules in this Section 7.

(3) The design values for the characteristic (rare) load combination, see ENV 1991-1, should be used in serviceability limit states verifications for plastic deformation and for deflection calculations.

NOTE: In ENV 1993-1-1 the characteristic (rare) combination is termed the "rare combination".

(4) The properties of the effective cross-section for serviceability limit states obtained from Section 4 should be used in all serviceability limit state calculations for cold formed thin-gauge members and sheeting.

(5) The effective second moment of area I_{eff} may be taken as variable along the span. Alternatively a uniform value may be used, based on the maximum span moment due to serviceability loading.

7.2 Plastic deformation

(1) In order to avoid excessive plastic deformation under service conditions, if redistribution of internal moments and forces is used in the global analysis for ultimate limit states, it should be ensured that no significant plastic deformations are liable to appear under serviceability loading.

(2) In such cases, the combination of support moment and support reaction at an internal support should not exceed 0,9 times the combined design resistance, determined using $\gamma_{M,ser}$.

(3) The combined design resistance may be determined from 5.11, but using the effective cross-section for serviceability limit states and $\gamma_{M,ser}$. Alternatively the design resistance may be determined from tests in accordance with Section 9, by dividing the characteristic resistance R_k by $\gamma_{M,ser}$.

NOTE: Appropriate testing procedures are given in annex A.

7.3 Deflections

(1) Deflections should be limited to values that would not adversely affect the appearance or effective use of the structure, or damage finishes or non-structural elements.

(2) The deflections may be calculated assuming elastic behaviour.

(3) The calculated deflection of a purlin in the direction perpendicular to the roof surface, due to variable gravity loads, should not exceed $L / 180$, where L is the span of the purlin.

7.4 Sheeting

(1) The provisions given in 7.2 should also be adopted for the design of sheeting.

(2) The provisions given in 7.3(1) and (2) should also be adopted for the design of sheeting, but the deflection limit given in 7.3(3) does not apply to sheeting.

8 Joints and connections

8.1 General

8.1.1 Design assumptions

(1)P Joints shall be designed on the basis of a realistic assumption of the distribution of internal forces and moments, having regard to relative stiffnesses within the joint. This distribution shall correspond with direct load paths through the elements of the joint. It shall be ensured that equilibrium is maintained with the applied external forces and moments.

(2)P Allowance may be made for the ductility of steel in facilitating the redistribution of internal forces generated within a joint. Accordingly, residual stresses and stresses due to tightening of fasteners and normal accuracy of fit-up need not be considered.

(3)P Ease of fabrication and erection shall be taken into account in the design of the details of connections and splices. Attention shall be paid to the clearances necessary for tightening of fasteners, the requirements of welding procedures, and the need for subsequent inspection, surface treatment and maintenance.

8.1.2 Intersections

(1)P Members meeting at a joint shall normally be arranged so that their centroidal axes intersect at a point.

(2)P If there is eccentricity at intersections, the members and connections shall be designed to accommodate the moments that result.

(3) In the case of bolted framing of angles and tees, the setting out lines for the bolts may be used instead of the centroidal axes for the purpose of intersection at the joint.

8.1.3 Connections subject to impact, vibration or load reversal

(1)P Where a connection is subject to impact or vibration, either preloaded bolts, bolts with locking devices or welding shall be used.

(2)P Where a connection that is loaded in shear is subject to reversal of stress (unless such stress is due solely to wind) or where for some special reason slipping of bolts is not acceptable, either preloaded bolts, fitted bolts or welding shall be used.

8.2 Requirements for joints

8.2.1 Joints in simple framing

(1)P In simple framing, the joints between the members shall have nominally pinned connections that:

- are capable of transmitting the forces calculated in the global analysis;
- are able to sustain the resulting rotations;
- do not develop significant moments adversely affecting members of the structure.

8.2.2 Joints in continuous framing

(1)P In continuous framing, the joints between the members shall be capable of transmitting the forces and moments calculated in the global analysis.

(2) If elastic global analysis is used, the rigidity of a moment-resisting joint should not be less than that of the connected member.

(3) In the case of plastic global analysis, the moment resistance of a moment-resisting joint that is located at, or adjacent to, a plastic hinge location, should not be less than the moment resistance of the cross-section of the connected member. In addition, the joint should have sufficient rotation capacity.

8.2.3 Joints in semi-continuous framing

(1)P In semi-continuous framing, the joints between the members shall be capable of providing a predictable degree of interaction. The moment-resisting joints shall be capable of resisting the internal moments developed by the joints themselves, in addition to the other internal forces and moments at the joints.

(2) The moment-resisting joints should have sufficient rigidity to develop the moments calculated in the global analysis, but sufficient flexibility to avoid developing larger moments than they can resist.

(3) If the design value of the moment resistance of a joint is less than that of the connected member, it should be demonstrated that the rotation capacity of the joint is sufficient to allow the necessary redistribution of internal moments and forces to take place.

8.3 Splices and end connections of members subject to compression

(1)P Splices and end connections in members that are subject to compression, shall either have at least the same resistance as the cross-section of the member, or be designed to resist an additional bending moment due to the second-order effects within the member, in addition to the internal compressive force N_{Sd} and the internal moments $M_{y,Sd}$ and $M_{z,Sd}$ obtained from the global analysis.

(2) In the absence of a second-order analysis of the member, this additional moment ΔM_{Sd} should be taken as acting about the cross-sectional axis that gives the smallest value of the reduction factor χ for flexural buckling, see 6.2.1(2)P, with a value determined from:

$$\Delta M_{Sd} = N_{Sd} \left[\frac{1}{\chi} - 1 \right] \frac{W_{eff}}{A_{eff}} \sin \frac{\pi a}{\ell} \qquad \text{... (8.1)}$$

where:

- A_{eff} is the effective area of the cross-section;
- a is the distance from the splice or end connection to the nearer point of contraflexure;
- ℓ is the buckling length of the member between points of contraflexure, for buckling about the relevant axis;
- W_{eff} is the section modulus of the effective cross-section for bending about the relevant axis.

(3) Splices and end connections should be designed in such a way that load can be transmitted to the effective portions of the cross-section.

(4) If the constructional details at the ends of a member are such that the line of action of the internal axial force cannot be clearly identified, a suitable eccentricity should be assumed and the resulting moments should be taken into account in the design of the member, the end connections and the splice, if there is one.

8.4 Connections with mechanical fasteners

(1)P Connections with mechanical fasteners shall be compact in shape. The positions of the fasteners shall be arranged to provide sufficient room for satisfactory assembly and maintenance.

(2)P The shear forces on individual mechanical fasteners in a connection may be assumed to be equal, provided that:

- the fasteners have sufficient ductility;
- shear is not the critical failure mode.

(3)P The fasteners to be used shall have known and documented resistances.

(4)P The resistances of individual mechanical fasteners shall be determined either by calculation or from the results of tests in accordance with Section 9. For design by calculation, the resistances of mechanical fasteners subject to static loads shall be determined from:

- table 8.1 for blind rivets;
- table 8.2 for self-tapping screws;
- table 8.3 for cartridge fired pins;
- table 8.4 for bolts.

(5)P In tables 8.1 to 8.4 the meanings of the symbols shall be taken as follows:

A	is	the gross cross-sectional area of a bolt;
A_s	is	the tensile stress area of a bolt;
A_{net}	is	the net cross-sectional area of the connected part;
d	is	the nominal diameter of the fastener;
d_o	is	the nominal diameter of the hole;
d_w	is	the diameter of the washer or the head of the fastener;
e_1	is	the end distance from the centre of the fastener to the adjacent end of the connected part, in the direction of load transfer, see figure 8.1;
e_2	is	the edge distance from the centre of the fastener to the adjacent edge of the connected part, in the direction perpendicular to the direction of load transfer, see figure 8.1;
f_{ub}	is	the ultimate tensile strength of the bolt material;
$f_{u,sup}$	is	the ultimate tensile strength of the supporting member into which a screw is fixed;
n	is	the number of sheets that are fixed to the supporting member by the same screw or pin;
p_1	is	the spacing centre-to-centre of fasteners in the direction of load transfer, see figure 8.1;
p_2	is	the spacing centre-to-centre of fasteners in the direction perpendicular to the direction of load transfer, see figure 8.1;
t	is	the thickness of the thinner connected part or sheet;
t_1	is	the thickness of the thicker connected part or sheet;
t_{sup}	is	the thickness of the supporting member into which a screw or a pin is fixed.

(6)P The partial factor γ_M for calculating the design resistances of mechanical fasteners shall be taken as:

$$\gamma_{M2} = \boxed{1{,}25}$$

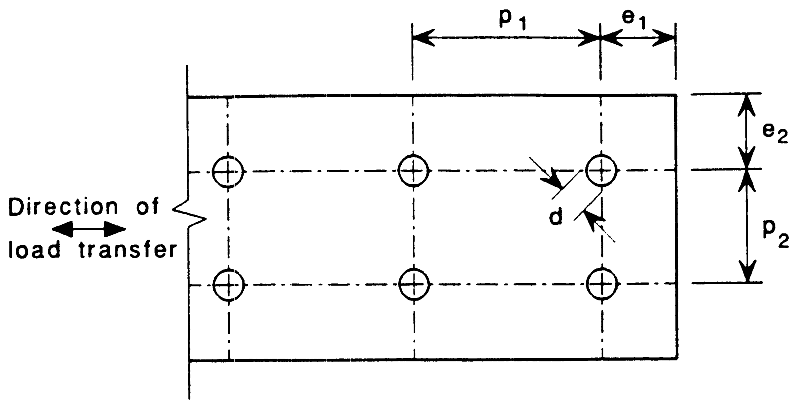


Figure 8.1: End distance, edge distance and spacings for fasteners and spot welds

(7)P The resistance of a connection shall preferably be based on tests in accordance with Section 9. Alternatively, the resistance of a connection subject to static loads may be determined from tables 8.1 to 8.4, provided that the limits on dimensions of fasteners and thicknesses of sheets stated in the tables are satisfied.

(8) If the pull-out resistance $F_{o,Rd}$ of a fastener is smaller than its pull-through resistance $F_{p,Rd}$ the deformation capacity should be determined from the results of tests in accordance with Section 9.

(9) The pull-through resistances given in tables 8.2 and 8.3 for self-tapping screws and cartridge fired pins should be reduced if the fasteners are not located centrally in the troughs of the sheeting. If attachment is at a quarter point, the design resistance should be reduced to $0{,}9F_{p,Rd}$ and if there are fasteners at both quarter points, the resistance should be taken as $0{,}7F_{p,Rd}$ per fastener, see figure 8.2.

(10) For a fastener loaded in combined shear and tension, if either the shear resistance $F_{v,Rd}$ or the tension resistance $F_{t,Rd}$ have been determined by testing, the resistance to combined shear and tension should also be verified on the basis of tests in accordance with Section 9. Provided that both $F_{t,Rd}$ and $F_{v,Rd}$ are determined by calculation on the basis of tables 8.1 to 8.4, the resistance of the fastener to combined shear and tension may be verified using:

$$F_{t,Sd}/F_{t,Rd} + F_{v,Sd}/F_{v,Rd} \leq 1 \qquad \ldots (8.2)$$

(11) The limit state of gross distortion may be assumed to be satisfied if the design resistance is obtained from tables 8.1 to 8.4, provided that the fastening is through a flange not more than 150 mm wide.

(12) The diameter of pre-drilled holes for screws should be in accordance with the manufacturer's guidelines. These guidelines should be based on following criteria:

- the applied torque should be just higher than the threading torque;
- the applied torque should be lower than the thread stripping torque or head-shearing torque;
- the threading torque should be smaller than 2/3 of the head-shearing torque.

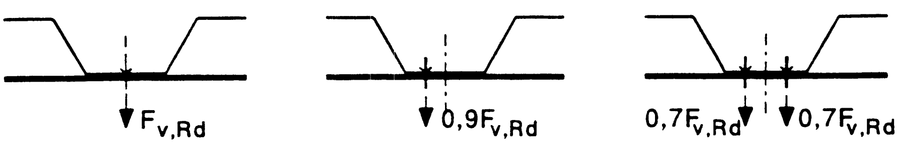


Figure 8.2: Reduction of tension resistance due to the position of fasteners

Table 8.1: Design resistances for blind rivets [1)]

Rivets loaded in shear:

Bearing resistance:

$$F_{b,Rd} = \alpha f_u d t / \gamma_{M2}$$

in which α is given by the following:

- if $t = t_1$: $\alpha = 3,2\sqrt{t/d}$ but $\alpha \leq 2,1$
- if $t_1 \geq 2,5t$: $\alpha = 2,1$
- if $t < t_1 \leq 2,5t$: obtain α by linear interpolation.

Net-section resistance:

$$F_{n,Rd} = A_{net} f_u / \gamma_{M2}$$

Shear resistance:

Shear resistance $F_{v,Rd}$ to be determined by testing.

Conditions:

$F_{v,Rd} \geq 1,2 F_{b,Rd}$ and $F_{v,Rd} \geq 1,2 F_{n,Rd}$

Rivets loaded in tension: [2)]

Pull-through resistance:

Pull-through resistance $F_{p,Rd}$ to be determined by testing.

Pull-out resistance:

Not relevant for rivets.

Tension resistance:

Tension resistance $F_{t,Rd}$ to be determined by testing.

Conditions:

$F_{t,Rd} \geq n F_{p,Rd}$

Range of validity: [3)]

$e_1 \geq 3d$ $p_1 \geq 3d$ $2,6\,\mathrm{mm} \leq d \leq 6,4\,\mathrm{mm}$

$e_2 \geq 1,5d$ $p_2 \geq 3d$

1) In this table it is assumed that the thinnest sheet is next to the preformed head of the blind rivet.

2) Blind rivets are not usually used in tension.

3) Blind rivets may be used beyond this range of validity if the resistance is determined from the results of tests in accordance with Section 9.

Table 8.2: Design resistances for self-tapping screws [1)]

Screws loaded in shear:

Bearing resistance:

$$F_{b,Rd} = \alpha f_u d t / \gamma_{M2}$$

in which α is given by the following:

- if $t = t_1$: $\alpha = 3{,}2\sqrt{t/d}$ but $\alpha \leq 2{,}1$
- if $t_1 \geq 2{,}5t$: $\alpha = 2{,}1$
- if $t < t_1 \leq 2{,}5t$: obtain α by linear interpolation.

Net-section resistance:

$$F_{n,Rd} = A_{net} f_u / \gamma_{M2}$$

Shear resistance:

Shear resistance $F_{v,Rd}$ to be determined by testing.

Conditions:

$F_{v,Rd} \geq 1{,}2 F_{b,Rd}$ and $F_{v,Rd} \geq 1{,}2 F_{n,Rd}$

Screws loaded in tension:

Pull-through resistance: [2)]

- for static loads: $F_{p,Rd} = d_w t f_u / \gamma_{M2}$
- for screws subject to repeated wind loads: $F_{pr,Rd} = 0{,}5 d_w t f_u / \gamma_{M2}$

Pull-out resistance:

$$F_{o,Rd} = 0{,}65 d t_{sup} f_{u,sup} / \gamma_{M2}$$

Tension resistance:

Tension resistance $F_{t,Rd}$ to be determined by testing.

Conditions:

$F_{t,Rd} \geq n F_{p,Rd}$ and $F_{t,Rd} \geq F_{o,Rd}$

Range of validity: [3)]

Generally:	$e_1 \geq 3d$	$p_1 \geq 3d$	$3{,}0\,\text{mm} \leq d \leq 8{,}0\,\text{mm}$
	$e_2 \geq 1{,}5d$	$p_2 \geq 3d$	
For tension:	$0{,}5\,\text{mm} \leq t \leq 1{,}5\,\text{mm}$	and	$t_1 \geq 0{,}9\,\text{mm}$

1) In this table it is assumed that the thinnest sheet is next to the head of the screw.

2) These values assume that the washer has sufficient rigidity to prevent it from being deformed appreciably or pulled over the head of the fastener.

3) Self-tapping screws may be used beyond this range of validity if the resistance is determined from the results of tests in accordance with Section 9.

Table 8.3: Design resistances for cartridge fired pins

Pins loaded in shear:

Bearing resistance:

$F_{b,Rd} = 3{,}2 f_u d t / \gamma_{M2}$

Net-section resistance:

$F_{n,Rd} = A_{net} f_u / \gamma_{M2}$

Shear resistance:

Shear resistance $F_{v,Rd}$ to be determined by testing.

Conditions:

$F_{v,Rd} \geq 1{,}5 F_{b,Rd}$ and $F_{v,Rd} \geq 1{,}5 F_{n,Rd}$

Pins loaded in tension:

Pull-through resistance: [1]

- for static loads: $F_{p,Rd} = d_w t f_u / \gamma_{M2}$
- for repeated wind loads: $F_{pr,Rd} = 0{,}5 d_w t f_u / \gamma_{M2}$

Pull-out resistance:

Pull-out resistance $F_{o,Rd}$ to be determined by testing.

Tension resistance:

Tension resistance $F_{t,Rd}$ to be determined by testing.

Conditions:

$F_{o,Rd} \geq n F_{p,Rd}$

$F_{t,Rd} \geq F_{o,Rd}$

Range of validity: [2]

Generally:	$e_1 \geq 4{,}5 d$	$3{,}7\,\text{mm} \leq d \leq 6{,}0\,\text{mm}$
	$e_2 \geq 4{,}5 d$	for $d = 3{,}7\,\text{mm}$: $t_{sup} \geq 4{,}0\,\text{mm}$
	$p_1 \geq 4{,}5 d$	for $d = 4{,}5\,\text{mm}$: $t_{sup} \geq 6{,}0\,\text{mm}$
	$p_2 \geq 4{,}5 d$	for $d = 5{,}2\,\text{mm}$: $t_{sup} \geq 8{,}0\,\text{mm}$
For tension:	$0{,}5\,\text{mm} \leq t \leq 1{,}5\,\text{mm}$	$t_{sup} \geq 6{,}0\,\text{mm}$

1) These values assume that the washer has sufficient rigidity to prevent it from being deformed appreciably or pulled over the head of the fastener.

2) Cartridge fired pins may be used beyond this range of validity if the resistance is determined from the results of tests in accordance with Section 9.

Table 8.4: Design resistances for bolts

Bolts loaded in shear:
Bearing resistance: $F_{b,Rd} = 2{,}5 f_u d t / \gamma_{M2}$ but $F_{b,Rd} \leq f_u e_1 t / 1{,}2 / \gamma_{M2}$ Net-section resistance: $F_{n,Rd} = (1 + 3r(d_o/u - 0{,}3)) A_{net} f_u / \gamma_{M2}$ but $F_{n,Rd} \leq A_{net} f_u / \gamma_{M2}$ with: r = [number of bolts at the cross-section] / [total number of bolts in the connection] $u = 2e_2$ but $u \leq p_2$ Shear resistance: - for strength grades 4.6, 5.6 and 8.8: $F_{v,Rd} = 0{,}6 f_{ub} A_s / \gamma_{M2}$ - for strength grades 4.8, 5.8, 6.8 and 10.9: $F_{v,Rd} = 0{,}5 f_{ub} A_s / \gamma_{M2}$
Conditions: $F_{v,Rd} \geq 1{,}2 F_{b,Rd}$ and $F_{v,Rd} \geq 1{,}2 F_{n,Rd}$
Bolts loaded in tension:
Pull-through resistance: Pull-through resistance $F_{p,Rd}$ to be determined by testing. Pull-out resistance: Not relevant for bolts. Tension resistance: $F_{t,Rd} = 0{,}9 f_{ub} A_s / \gamma_{M2}$
Conditions: $F_{t,Rd} \geq n F_{p,Rd}$
Range of validity: [1]
$e_1 \geq 1{,}5d$ $p_1 \geq 3d$ $t \geq 1{,}25$ mm Minimum bolt size: M6 $e_2 \geq 1{,}5d$ $p_2 \geq 3d$ Strength grades: 4.6 - 10.9
[1] Bolts may be used beyond this range of validity if the resistance is determined from the results of tests in accordance with Section 9.

8.5 Spot welds

(1)P Spot welds may be used with as-rolled or galvanized parent material up to 4,0 mm thick, provided that the thinner connected part is not more than 3,0 mm thick.

(2) Spot welds may be either resistance welded or fusion welded.

(3)P The design resistance $F_{v,Rd}$ of a spot weld loaded in shear shall be determined using table 8.5.

(4)P In table 8.5 the meanings of the symbols shall be taken as follows:

A_{net} is the net cross-sectional area of the connected part;

t is the thickness of the thinner connected part or sheet [mm];

t_1 is the thickness of the thicker connected part or sheet;

and the end and edge distances e_1 and e_2 and the spacings p_1 and p_2 are as defined in 8.4(4)P.

(5)P The partial factor γ_M for calculating the design resistances of spot welds shall be taken as:

$\gamma_{M2} = \boxed{1,25}$

Table 8.5: Design resistances for spot welds

Spot welds loaded in shear:
Tearing and bearing resistance: - if $t \leq t_1 \leq 2,5t$: $F_{tb,Rd} = 2,7\sqrt{3}\,t\,d_s f_u / \gamma_{M2}$ - if $t_1 > 2,5t$: $F_{tb,Rd} = 2,7\sqrt{3}\,t\,d_s f_u / \gamma_{M2}$ but $F_{tb,Rd} \leq 0,7 d_s^2 f_u / \gamma_{M2}$ and $F_{tb,Rd} \leq 3,1\,t\,d_s f_u / \gamma_{M2}$ End resistance: $F_{e,Rd} = 1,4\,t\,e_1 f_u / \gamma_{M2}$ Net section resistance: $F_{n,Rd} = A_{net} f_u / \gamma_{M2}$ Shear resistance: $F_{v,Rd} = \frac{\pi}{4} d_s^2 f_u / \gamma_{M2}$
Conditions: $F_{v,Rd} \geq 1,25 F_{tb,Rd}$ and $F_{v,Rd} \geq 1,25 F_{e,Rd}$ and $F_{v,Rd} \geq 1,25 F_{n,Rd}$
Range of validity:
$2d_s \leq e_1 \leq 6d_s$ $3d_s \leq p_1 \leq 8d_s$ $e_2 \leq 4d_s$ $3d_s \leq p_2 \leq 6d_s$

(6)P The interface diameter d_s of a spot weld shall be determined from the following:

- for fusion welding: $d_s = 0{,}5t + 5\,\text{mm}$ ... (8.3a)
- for resistance welding: $d_s = 5\sqrt{t}$ [with t in mm] ... (8.3b)

(7)P The value of d_s actually produced by the welding procedure shall be verified by shear tests in accordance with Section 9, using single-lap test specimens as shown in figure 8.3.

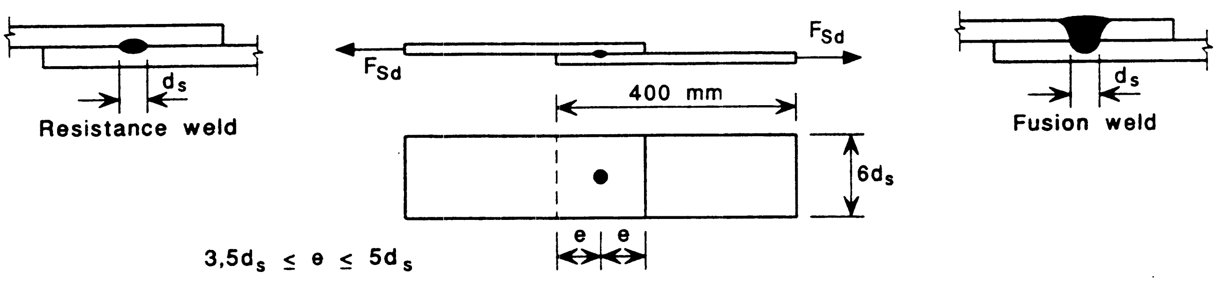


Figure 8.3: Test specimen for shear tests of spot welds

8.6 Lap welds

8.6.1 General

(1)P This clause 8.6 shall be used for the design of arc-welded lap welds where the parent material is 4,0 mm thick or less. For thicker parent material, lap welds shall be designed using ENV 1993-1-1.

(2)P The weld size shall be chosen such that the resistance of the connection is governed by the thickness of the connected part or sheet, rather than the weld.

(3) The requirement in (2)P may be assumed to be satisfied if the throat size of the weld is at least equal to the thickness of the connected part or sheet.

(4)P The partial factor γ_M for calculating the design resistances of lap welds shall be taken as:

$$\gamma_{M2} = \boxed{1{,}25}$$

8.6.2 Fillet welds

(1)P The design resistance $F_{w.Rd}$ of a fillet-welded connection shall be determined from the following:

- for a side fillet that comprises one of a pair of side fillets:

$$F_{w,Rd} = tL_{w,s}(0{,}9 - 0{,}45L_{w,s}/b)f_u/\gamma_{M2} \quad \text{... (8.4a)}$$

- for an end fillet:

$$F_{w,Rd} = tL_{w,e}(1 - 0{,}3L_{w,e}/b)f_u/\gamma_{M2} \quad \text{[for one weld]} \quad \text{... (8.4b)}$$

where:

b	is	the width of the connected part or sheet, see figure 8.4;
$L_{w,e}$	is	the effective length of the end fillet weld, see figure 8.4;
$L_{w,s}$	is	the effective length of a side fillet weld, see figure 8.4.

S

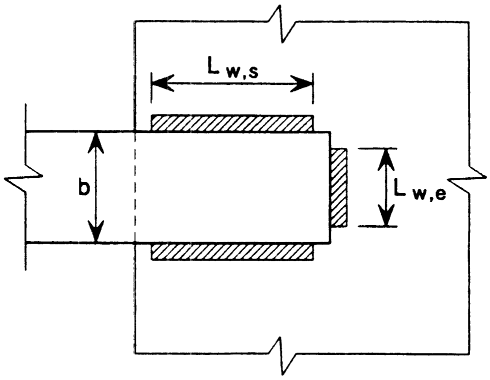


Figure 8.4: Fillet welded lap connection

(2)P If a combination of end fillets and side fillets is used in the same connection, its total resistance shall be taken as equal to the sum of the resistances of the end fillets and the side fillets.

(3)P The effective length L_w of a fillet weld shall be taken as the overall length of the full-size fillet, including end returns. Provided that the weld is full size throughout this length, no reduction in effective length need be made for either the start or termination of the weld.

(4) Fillet welds with effective lengths less than 8 times the thickness of the thinner connected part should not be designed to transmit any forces.

8.6.3 Arc spot welds

(1)P Arc spot welds shall not be designed to transmit any forces other than in shear.

(2)P Arc spot welds shall not be used through connected parts or sheets with a total thickness Σt of more than 4 mm, or where the thinnest connected part or sheet is more than 4 mm thick.

(3)P Arc spot welds shall have an effective diameter d_{eff} of not less than 10 mm.

(4)P If the connected part or sheet is less than 0,7 mm thick, a weld washer shall be used, see figure 8.5.

(5)P Arc spot welds shall have adequate end and edge distances.

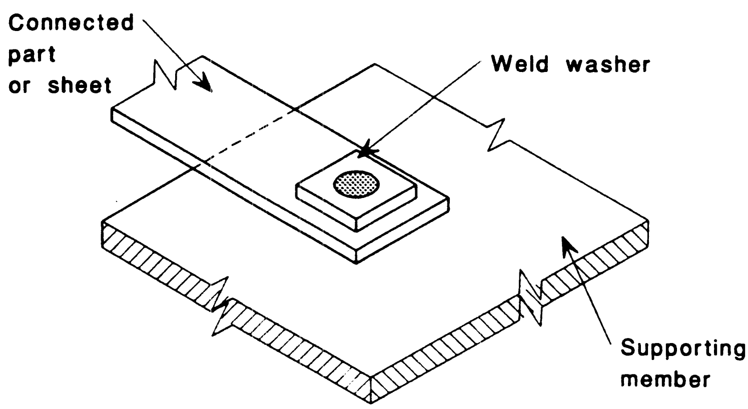


Figure 8.5: Arc spot weld with weld washer

(6) The design shear resistance $F_{w,Rd}$ of a circular arc spot weld should be determined as follows:

$$F_{w,Rd} = (\pi/4) d_s^2 \times 0{,}5 f_{uw} / \gamma_{M2} \qquad \text{... (8.5a)}$$

where:

f_{uw} is the ultimate tensile strength of the welding electrodes;

but $F_{w,Rd}$ should not be taken as more than the peripheral resistance given by the following:

- if $d_p / \Sigma t \leq 24\varepsilon$:

$$F_{w,Rd} = 1{,}33 d_p \Sigma t f_u / \gamma_{M2} \qquad \text{... (8.5b)}$$

- if $24\varepsilon < d_p/\Sigma t < 41{,}5\varepsilon$:

$$F_{w,Rd} = 0{,}17 \left(d_p + 164 \varepsilon \Sigma t \right) \Sigma t f_u / \gamma_{M2} \qquad \text{... (8.5c)}$$

- if $d_p/\Sigma t \geq 41{,}5\varepsilon$:

$$F_{w,Rd} = 0{,}84 d_p \Sigma t f_u / \gamma_{M2} \qquad \text{... (8.5d)}$$

(7) The interface diameter d_s of an arc spot weld, see figure 8.6, should be obtained from:

$$d_s = 0{,}7 d_w - 1{,}5 \Sigma t \qquad \text{... (8.6)}$$

where:

d_w is the visible diameter of the arc spot weld, see figure 8.6.

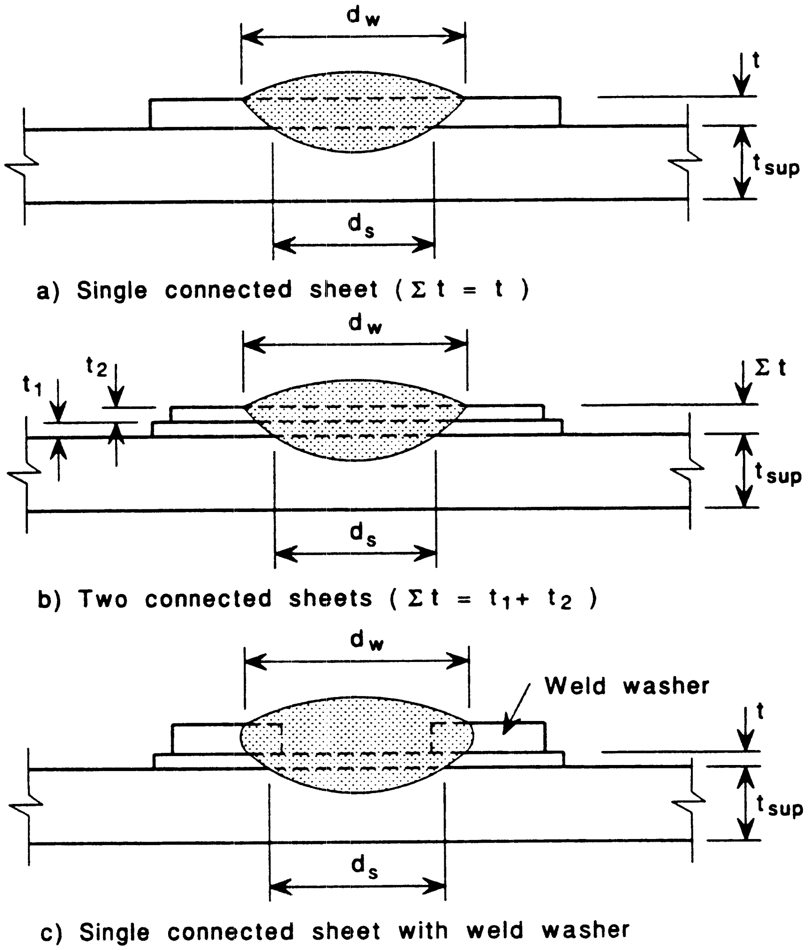


Figure 8.6: Arc spot welds

(8) The effective peripheral diameter d_p of an arc spot weld should be obtained as follows:

- for a single connected sheet or part of thickness t:

$$d_p = d_w - t \qquad \text{... (8.7a)}$$

- for multiple connected sheets or parts of total thickness Σt:

$$d_p = d_w - 2\Sigma t \qquad \text{... (8.7b)}$$

(9) The design shear resistance $F_{w,Rd}$ of an elongated arc spot weld should be determined from:

$$F_{w,Rd} = [(\pi/4)d_s^2 + L_w d_s] \times 0{,}5 f_{uw}/\gamma_{M2} \qquad \text{... (8.8a)}$$

but $F_{w,Rd}$ should not be taken as more than the peripheral resistance given by:

$$F_{w,Rd} = (0{,}4L_w + 1{,}33 d_p)\Sigma t f_u/\gamma_{M2} \qquad \text{... (8.8b)}$$

where:

L_w is the length of the elongated arc spot weld, measured as shown in figure 8.7.

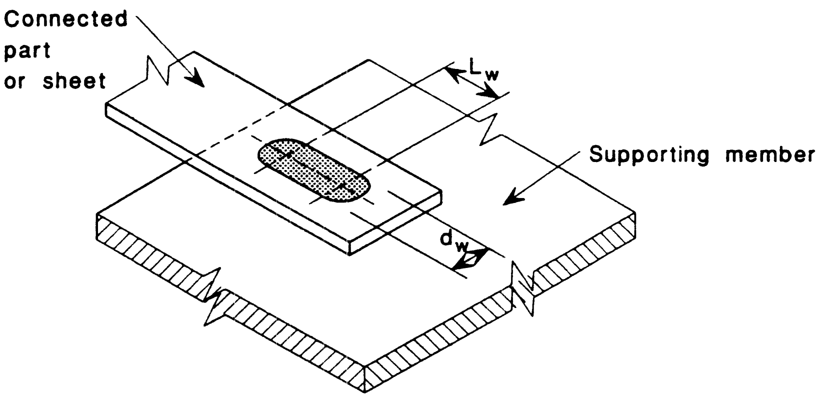


Figure 8.7: Elongated arc spot weld

9 Design assisted by testing

9.1 Basis

(1)P This Section 9 shall be used to apply the principles for design assisted by testing given in Section 8 of ENV 1993-1-1, to the specific requirements of cold formed thin gauge members and sheeting.

(2) Testing may be undertaken under any of the following circumstances:

a) if the properties of the steel are unknown;

b) if it is desired to take account of the actual properties of the cold formed member or sheet;

c) if adequate analytical procedures are not available for designing a component by calculation alone;

d) if realistic data for design cannot otherwise be obtained;

e) if it is desired to check the performance of an existing structure or structural component;

f) if it is desired to build a number of similar structures or components on the basis of a prototype;

g) if confirmation of consistency of production is required;

h) if it is desired to determine the effects of interaction with other structural components;

i) if it is desired to determine the effects of the lateral or torsional restraint supplied by other components;

j) if it is desired to prove the validity and adequacy of an analytical procedure;

k) if it is desired to produce resistance tables based on tests, or on a combination of testing and analysis;

l) if it is desired to take into account practical factors that might alter the performance of a structure, but are not addressed by the relevant analysis method for design by calculation.

(3) Testing as a basis for tables of load carrying capacity should be in accordance with 9.3.

NOTE: Information is given in annex A on procedures for:

- tests on profiled sheets and liner trays;
- tests on cold formed members;
- tests on structures and portions of structures;
- tests on beams torsionally restrained by sheeting;
- evaluation of test results to determine design values.

(4) Tensile testing of steel should be carried out in accordance with EN 10002-1. Testing of other steel properties should be carried out in accordance with the relevant European Standards.

(5) Testing of fasteners and connections should be carried out in accordance with the relevant European Standard or International Standard if there is one.

NOTE: Pending availability of an appropriate European or International Standard, guidance on testing procedures for fasteners can be obtained from:

ECCS Publication No. 21 (1983): *European recommendations for steel construction: the design and testing of connections in steel sheeting and sections*;

ECCS Publication No. 42 (1983): *European recommendations for steel construction: mechanical fasteners for use in steel sheeting and sections.*

9.2 Conditions

(1)P The planning, execution, evaluation and documentation of tests shall be in accordance with the minimum requirements specified in this Section 9.

(2)P The performance of experimental assessments shall be entrusted only to organisations where the staff is sufficiently knowledgeable and experienced in the planning, execution and evaluation of tests.

(3)P The testing laboratory shall be adequately equipped and the testing organisation shall ensure careful management and documentation of all tests.

(4)P The application of any test result shall be consistent with the particular conditions used in the test.

(5)P Tests shall simulate the behaviour of the member, sheeting or assembly under practical conditions, and the loading, support and constraint conditions used in the test shall model those that apply in practice.

NOTE: Good practice guides define conventional practical conditions for specific applications.

(6) The rate of load application should be such that the behaviour can be considered to be quasi-static.

(7) A comprehensive record of load-deformation behaviour should be made, containing an adequate number of readings for each variable monitored.

(8) Testing may be carried out using any of the following methods:

- incremental loading;
- continuously variable and continuously monitoring machines;
- a test rig with continuously variable applicators, such as air bags or vacuum boxes.

(9) For incremental loading, the increments should be determined from the expected load-deformation behaviour and their number should be sufficient to give a full record of the behaviour of the test specimen. The deformations at critical points should be measured at each increment of the loading.

(10) During load application, up to attainment of the service load, the load may be removed and then reapplied. For this purpose the service load may be taken as equal to the serviceability limit state design load for the characteristic (rare) combination as defined in ENV 1991-1. Above the service load, the loading should be held constant at each increment until any time-dependent deformations due to plastic behaviour have become negligible.

NOTE: In ENV 1993-1-1 the characteristic (rare) combination is termed the "rare combination".

(11)P A test report shall be prepared giving the following information:

a) a specification of the test;

b) a diagram indicating the geometry of the structure or component;

c) a diagram indicating the positions of the loading points and locations of the measuring devices;

d) details of the loading method and procedure;

e) the actual dimensional measurements of the structure or component;

f) the deflections and strains measured in the test, with the corresponding stage of loading or unloading;

g) a record of all other observations from the test.

(12) The test report should be supplemented by an evaluation of the resistance of the structure or component.

9.3 Load tables based on testing

9.3.1 General

(1) Load tables giving the load carrying capacity of specific structural components may be either based completely on the results of tests, or based on a combination of testing and rational analysis.

(2) Such load tables may represent the performance of a member when used within a specific structural system in which its behaviour is influenced by interaction with cladding and other structural components.

(3) If the performance of a system relies on the stabilizing effect of associated materials, such as sheeting on roof purlin systems, load tables based on testing should clearly state the necessary conditions of validity in terms of the associated materials and the methods of fixing them.

(4) In preparing load tables, account should be taken of the possibility that relevant serviceability limit state criteria, rather than the ultimate limit state design resistance, might govern the load carrying capacity.

(5) The tests should verify that, under the serviceability limit state characteristic (rare) combination, see ENV 1991-1, the member has no significant local deformation and no significant permanent deformation.

NOTE: In ENV 1993-1-1 the characteristic (rare) combination is termed the "rare combination".

9.3.2 Tables based completely on testing

(1) If the load tables are based completely on tests, these tests should adequately cover the whole range of geometries and loading conditions to be included in the load tables, and the support conditions and connections used in the tests should correspond with those stated in the load tables.

(2) Extrapolation should generally be avoided. However, limited extrapolation may be used where this can be justified on the basis of a specific and appropriate analysis of the test results, provided that it can be demonstrated that this extrapolation does not lead to conditions in which a different failure mode is likely.

9.3.3 Tables based on combined testing and analysis

(1) As an alternative to 9.3.2, load tables may be based on a rational analysis assisted by testing. The mathematical model of the resistance should take account of all failure modes that are possible within the range of the load tables. This mathematical model should be validated by testing.

(2) The validation of the mathematical model may be by means of full scale tests on a completely representative portion of a structure, comprising the structural components and connections, together with the associated materials and the methods of fixing them to be used in service.

(3) Alternatively the mathematical model may be validated by carrying out separate tests on all members, connections and other structural components to determine their strength and stiffness, and the rotational restraint given to members by the cladding. This analysis should also take account of all failure modes that are possible within the range of the load tables. If this is in doubt, sufficient full scale tests as described in (2) should be carried out to remove the doubt.

(4) In comparing the results of a test with those of the mathematical model, the actual thickness and yield strength of the critical component should be used.

(5)P Appropriate safety factors shall be applied. The mathematical model may be adjusted to achieve compliance with this requirement.

NOTE: Information on appropriate procedures is given in annex A.

10 Particular applications

10.1 Beams restrained by sheeting

10.1.1 General

(1) The provisions given in this clause 10.1 may be applied to purlins of Z, C, Σ or similar shaped cross-section with continuous full lateral restraint to one flange. The purlins may be designed by calculation, by testing in accordance with Section 9, or by a combination of calculation and testing.

(2) These provisions may also be applied to cold formed members used as side rails, floor beams and other similar types of beam that are similarly restrained by sheeting.

(3) Side rails may be designed on the basis that wind pressure has a similar effect on them to gravity loading on purlins, and that wind suction acts on them in a similar way to uplift loading on purlins.

(4) Full continuous lateral restraint may be supplied by trapezoidal steel sheeting or other profiled steel sheeting with sufficient stiffness, continuously connected to the top flange of the purlin through the troughs of the sheets. In other cases (for example, fastening through the crests of the sheets) the degree of restraint should either be validated by experience, or determined from tests in accordance with Section 9.

(5) Unless alternative support arrangements can be justified from the results of tests in accordance with Section 9, the purlin should have support details, such as cleats, that prevent rotation and lateral displacement at its supports. The effects of forces in the plane of the sheeting, that are transmitted to the supports of the purlin, should be taken into account in the design of the support details.

(6) The behaviour of a laterally restrained purlin should be modelled as outlined in figure 10.1. The connection of the purlin to the sheeting may be assumed to partially restrain the twisting of the purlin. This partial torsional restraint may be represented by a rotational spring with a spring stiffness C_D. The stresses in the free flange, not directly connected to the sheeting, should then be calculated by superposing the effects of in-plane bending and the effects of torsion, including lateral bending due to cross-sectional distortion.

(7) Where the free flange of a single span purlin is in compression under uplift loading, allowance should also be made for the amplification of the stresses due to torsion and distortion.

10.1.2 Calculation methods

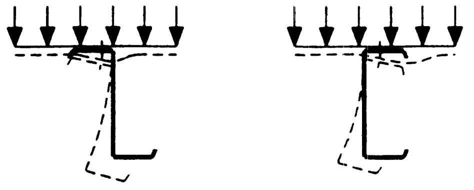
(1) Unless a second order analysis is carried out, the method given in 10.1.3 and 10.1.4 should be used to allow for the tendency of the free flange to move laterally (thus inducing additional stresses) by treating it as a beam subject to a lateral load $q_{h,Fd}$, see figure 10.1.

(2) For use in this method, the rotational spring should be replaced by an equivalent lateral linear spring of stiffness K. In determining K the effects of cross-sectional distortion should also be allowed for. For this purpose, the free flange may be treated as a compression member subject to a non-uniform axial force, with a continuous lateral spring support of stiffness K.

(3) If the free flange of a purlin is in compression due to in-plane bending (for example, due to uplift loading in a single span purlin), the resistance of the free flange to lateral buckling should also be verified.

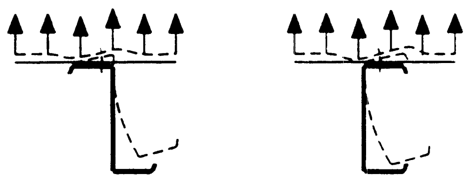
(4) For a more precise calculation, a second order analysis should be carried out, using values of the rotational spring stiffness C_D obtained from 10.1.5.2. Allowance should be made for the effects of an initial bow imperfection of $L/500$ in the free flange, where L is the span.

(5) A second order analysis using the rotational spring stiffness C_D obtained from 10.1.5.2 may also be used if lateral restraint is not supplied or if its effectiveness cannot be proved.



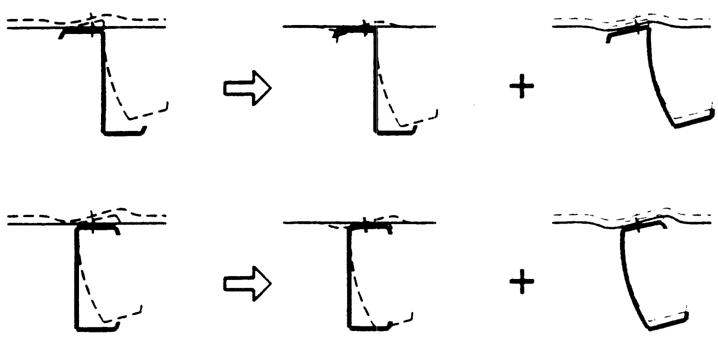
Z or C section purlin with its upper flange connected to sheeting:

- gravity loading.



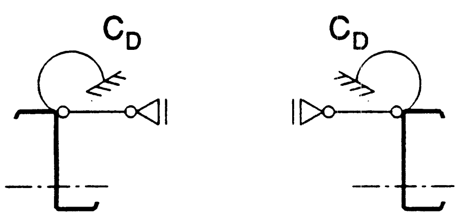
Z or C section purlin with its upper flange connected to sheeting:

- uplift loading.



Total deformation split into two parts:

- torsion and lateral bending;
- in-plane bending.

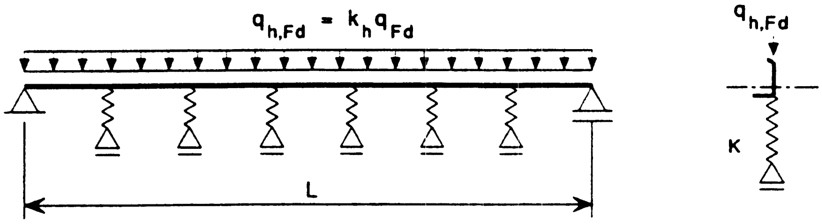


Model purlin as laterally braced with rotational spring restraint C_D from the sheeting.

As a simplification, replace the rotational spring C_D by a lateral spring of stiffness K.

Simplified calculation model used in 10.1.4.

Free flange of purlin modelled as beam on elastic foundation.



Model representing effects of torsion and lateral bending (including cross-section distortion) on single span purlin with uplift loading.

Figure 10.1: Modelling laterally braced purlins rotationally restrained by sheeting

10.1.3 Design criteria

10.1.3.1 Single span purlins

(1) For gravity loading, a single span purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1. If it is subject to axial compression, it should also satisfy the criteria for stability of the free flange given in 10.1.4.2.

(2) For uplift loading, a single span purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1 and the criteria for stability of the free flange given in 10.1.4.2.

10.1.3.2 Purlins continuous over two spans

(1) The moments due to gravity loading in a purlin that is physically continuous over two spans without overlaps or sleeves, may be either be obtained by calculation or based on the results of tests.

(2) If the moments are calculated they should be determined using elastic global analysis. The purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1. For the moment at the internal support, the criteria for stability of the free flange given in 10.1.4.2 should also be satisfied.

(3) Alternatively the moments may be determined using the results of tests in accordance with Section 9 on the moment-rotation behaviour of the purlin over the internal support.

NOTE: Appropriate testing procedures are given in annex A.

(4) The design value of the resistance moment at the supports $M_{sup,Rd}$ for a given value of the load per unit length q_{Fd}, should be obtained from the intersection of two curves representing the design values of:

- the moment-rotation characteristic at the support, obtained by testing in accordance with Section 9;
- the theoretical relationship between the support moment $M_{sup,Sd}$ and the corresponding plastic hinge rotation ϕ_{Ed} in the purlin over the support.

(5) The span moments should then be calculated from the value of the support moment.

(6) The following expressions may be used for a purlin with two equal spans:

$$\phi_{Ed} = \frac{L}{12\,EI_{eff}}\left[q_{Fd}L^2 - 8M_{sup,Sd}\right] \qquad \text{... (10.1)}$$

$$M_{spn,Sd} = \frac{\left(q_{Fd}L^2 - 2M_{sup,Rd}\right)^2}{8q_{Fd}L^2} \qquad \text{... (10.2)}$$

where:

I_{eff} is the effective second moment of area for the moment $M_{spn,Sd}$;

L is the span;

$M_{spn,Sd}$ is the maximum moment in the span.

(7) The expressions for a purlin with two unequal spans should be obtained by analysis.

(8) The maximum span moment $M_{spn,Sd}$ in the purlin should satisfy the criteria for cross-section resistance given in 10.1.4.1. Alternatively the resistance moment in the span may be determined by testing in accordance with Section 9, using single span tests with a span comparable to the distance between the points of contraflexure in the span.

10.1.3.3 Two-span continuous purlins with uplift loading

(1) The moments due to uplift loading in a purlin that is physically continuous over two spans without overlaps or sleeves, should be determined using elastic global analysis.

(2) The moment over the internal support should satisfy the criteria for cross-section resistance given in 10.1.4.1. Because the support reaction is a tensile force, no account need be taken of its interaction with the support moment.

(3) The moments in the spans should satisfy the criteria for stability of the free flange given in 10.1.4.2.

10.1.3.4 Purlins with continuity given by overlaps or sleeves

(1) The moments in purlins in which continuity over two or more spans is given by overlaps or sleeves at internal supports, should be determined taking into account the effective section properties of the cross-section and the effects of the overlaps or sleeves.

(2) Tests in accordance with Section 9 should be carried out on the support details to determine:

- the flexural stiffness of the overlapped or sleeved part;
- the moment-rotation characteristic for the overlapped or sleeved part;
- the resistance of the overlapped or sleeved part to combined support reaction and moment;
- the resistance of the non-overlapped unsleeved part to combined shear force and bending moment.

(3) For gravity loading, the purlin should satisfy the following criteria:

- at internal supports, the resistance to combined support reaction and moment determined by testing;
- near supports, the resistance to combined shear force and bending moment determined by testing;
- in the spans, the criteria for cross-section resistance given in 10.1.4.1;
- if the purlin is subject to axial compression, the criteria for stability of the free flange given in 10.1.4.2.

(4) For uplift loading, the purlin should satisfy the following criteria:

- at internal supports, the resistance to combined support reaction and moment determined by testing, taking into account the fact that the support reaction is a tensile force in this case;
- near supports, the resistance to combined shear force and bending moment determined by testing;
- in the spans, the criteria for stability of the free flange given in 10.1.4.2.

10.1.3.5 Serviceability criteria

(1) The serviceability criteria relevant to purlins given in Section 7 should also be satisfied.

10.1.4 Design resistance

10.1.4.1 Resistance of cross-sections

(1) For a purlin subject to axial force and transverse load the resistance of the cross-section should be verified as indicated in figure 10.2 by superposing the stresses due to:

- the in-plane bending moment $M_{y,Sd}$;
- the axial force N_{Sd};
- a lateral load $q_{h,Fd}$ acting on the free flange, due to torsion and lateral bending, see (3).

(2) The maximum stresses in the cross-section should satisfy the following:

- restrained flange:

$$\sigma_{max,Ed} = \frac{M_{y,Sd}}{W_{eff,y}} + \frac{N_{Sd}}{A_{eff}} \leq f_y/\gamma_M \qquad \text{... (10.3a)}$$

- free flange:

$$\sigma_{max,Ed} = \frac{M_{y,Sd}}{W_{eff,y}} + \frac{N_{Sd}}{A_{eff}} + \frac{M_{fz,Sd}}{W_{fz}} \leq f_y/\gamma_M \qquad \text{... (10.3b)}$$

where:

A_{eff}	is	the effective area of the cross-section for uniform compression;
f_y	is	the yield strength as defined in 3.1.1(6)P;
$M_{fz,Sd}$	is	the bending moment in the free flange due to the lateral load $q_{h,Fd}$;
$W_{eff,y}$	is	the effective section modulus of the cross-section for bending about the y - y axis;
W_{fz}	is	the gross elastic section modulus of the free flange plus 1/6 of the web height, for bending about the z - z axis;

and $\gamma_M = \gamma_{M0}$ if $A_{eff} = A_{e\ell}$ or if $W_{eff,y} = W_{e\ell,y}$ and $N_{Sd} = 0$, otherwise $\gamma_M = \gamma_{M1}$.

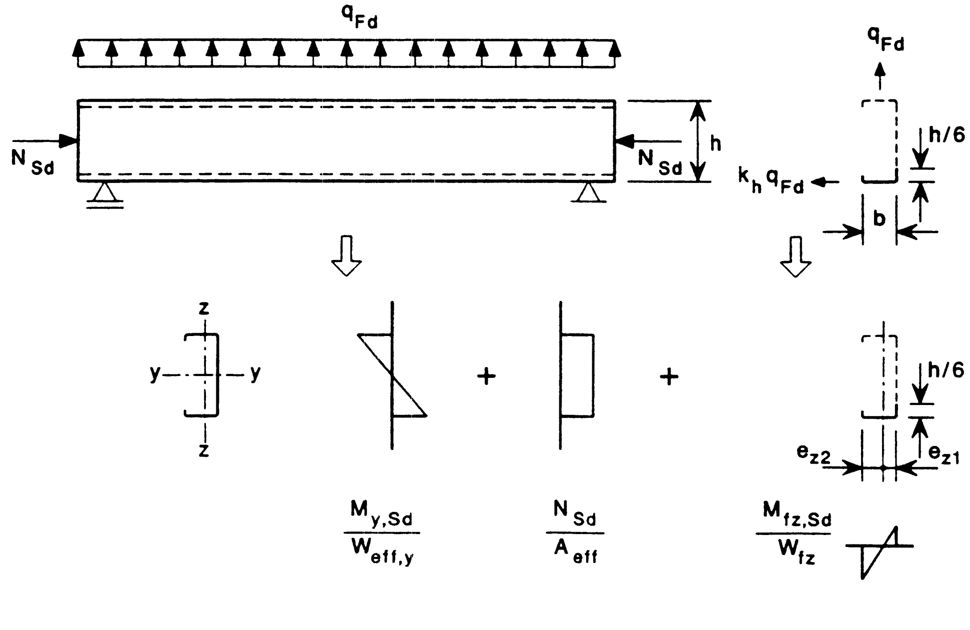


Figure 10.2: Superposition of stresses

(3) The lateral load $q_{h,Fd}$ acting on the free flange, due to torsion and lateral bending, should be obtained from:

$$q_{h,Fd} = k_h q_{Fd} \quad \text{... (10.4)}$$

(4) The coefficient k_h should be obtained as indicated in figure 10.3 for common types of cross-section.

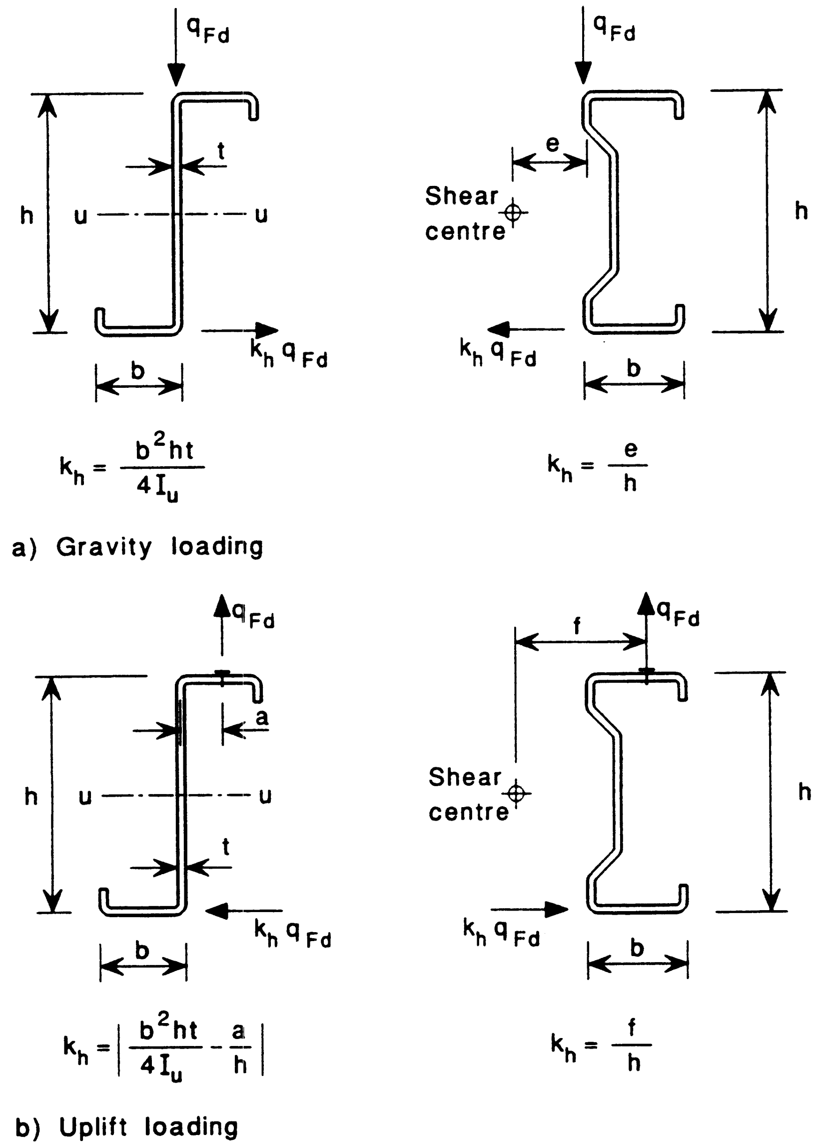


Figure 10.3: Conversion of torsion into lateral bending of the free flange

(5) The lateral bending moment $M_{fz,Sd}$ should be taken as equal to zero if the free flange is in tension, otherwise $M_{fz,Sd}$ should be determined from:

$$M_{fz,Sd} = \beta_R M_{0,fz,Sd} \quad \text{... (10.5)}$$

where:

$M_{0,fz,Sd}$ is the initial lateral bending moment in the free flange without any spring support;

β_R is a correction factor for the effective spring support.

(6) The initial lateral bending moment in the free flange $M_{0,fz,Sd}$ should be determined from table 10.1 for the critical locations in the span, at supports, at anti-sag bars and between anti-sag bars.

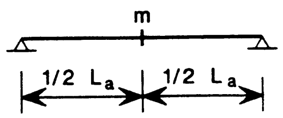
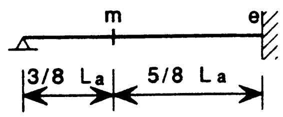
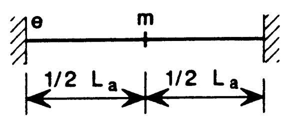
(7) The correction factor β_R for the relevant location and boundary conditions, should be determined from table 10.1, using the value of the coefficient R of the spring support given by:

$$R = \frac{K L_a^4}{\pi^4 E I_{fz}} \qquad \text{... (10.6)}$$

where:

- I_{fz} is the second moment of area of the gross cross-section of the free flange plus 1/6 of the web height, for bending about the z - z axis;
- K is the lateral spring stiffness per unit length from 10.1.5.1;
- L_a is the distance between anti-sag bars, or if none are present, the span L of the purlin.

Table 10.1: Values of initial moment $M_{0,fz,Sd}$ and correction factor β_R

System	Location	$M_{0,fz,Sd}$	β_R
	m	$\frac{1}{8} q_{h,Fd} L_a^2$	$\beta_R = \frac{1 - 0{,}0225R}{1 + 1{,}013R}$
	m	$\frac{9}{128} q_{h,Fd} L_a^2$	$\beta_R = \frac{1 - 0{,}0141R}{1 + 0{,}416R}$
	e	$-\frac{1}{8} q_{h,Fd} L_a^2$	$\beta_R = \frac{1 + 0{,}0314R}{1 + 0{,}396R}$
	m	$\frac{1}{24} q_{h,Fd} L_a^2$	$\beta_R = \frac{1 - 0{,}0125R}{1 + 0{,}198R}$
	e	$-\frac{1}{12} q_{h,Fd} L_a^2$	$\beta_R = \frac{1 + 0{,}0178R}{1 + 0{,}191R}$

10.1.4.2 Buckling resistance of free flange

(1) If the free flange is in compression, its buckling resistance should be verified using:

$$\frac{1}{\chi}\left[\frac{M_{y,Sd}}{W_{eff,y}} + \frac{N_{Sd}}{A_{eff}}\right] + \frac{M_{fz,Sd}}{W_{fz}} \leq f_{yb}/\gamma_{M1} \qquad \text{... (10.7)}$$

in which χ is the reduction factor for flexural buckling of the free flange, obtained from 6.2.1(2)P using buckling curve a (imperfection factor α = 0,21) for the relative slenderness λ_{fz} given in (2).

(2) The relative slenderness $\bar{\lambda}_{fz}$ for flexural buckling of the free flange should be determined from:

$$\bar{\lambda}_{fz} = \frac{\ell_{fz}/i_{fz}}{\lambda_1} \qquad \text{... (10.8)}$$

with:

$$\lambda_1 = \pi\left[E/f_{yb}\right]^{0,5}$$

where:

ℓ_{fz} is the buckling length for the free flange from (3) to (7);

i_{fz} is the radius of gyration of the gross cross-section of the free flange plus 1/6 of the web height, about the z - z axis.

(3) For gravity loading, provided that $0 \leq R \leq 200$, the buckling length of the free flange for a variation of the compressive stress over the length L as shown in figure 10.4 may be obtained from:

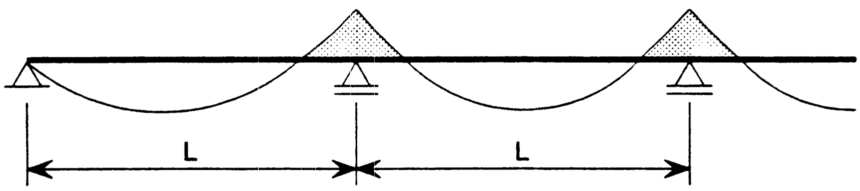
$$\ell_{fz} = \eta_1 L_a\left(1 + \eta_2 R^{\eta_3}\right)^{\eta_4} \qquad \text{... (10.9)}$$

where:

L_a is the distance between anti-sag bars, or if none are present, the span L of the purlin;

R is as given in 10.1.4.1(7);

and η_1 to η_4 are coefficients that depend on the number of anti-sag bars, as given in table 10.2.



[Dotted areas show regions in compression]

Figure 10.4: Varying compressive stress in free flange for gravity load cases

Table 10.2: Coefficients η_i

Number of anti-sag bars per span	η_1	η_2	η_3	η_4
0	0,526	22,8	2,12	- 0,108
1	0,622	66,7	2,68	- 0,084
2 or 3	0,713	62,7	2,75	- 0,084
more than 3	1,000	30,4	2,28	- 0,108

(4) For gravity loading, if there are more than three equally spaced anti-sag bars, the buckling length need not be taken as greater than the value for two anti-sag bars, with $L_a = L/3$.

(5) If the compressive stress over the length L is almost constant, due to the application of a relatively large axial force, the buckling length should be determined using the values of η_i from table 10.2 for the case shown as more than three anti-sag bars per span, but the actual spacing L_a.

(6) For uplift loading, provided that $0 \leq R_0 \leq 200$, the buckling length of the free flange for variations of the compressive stress over the length L_0 as shown in figure 10.5, may be obtained from:

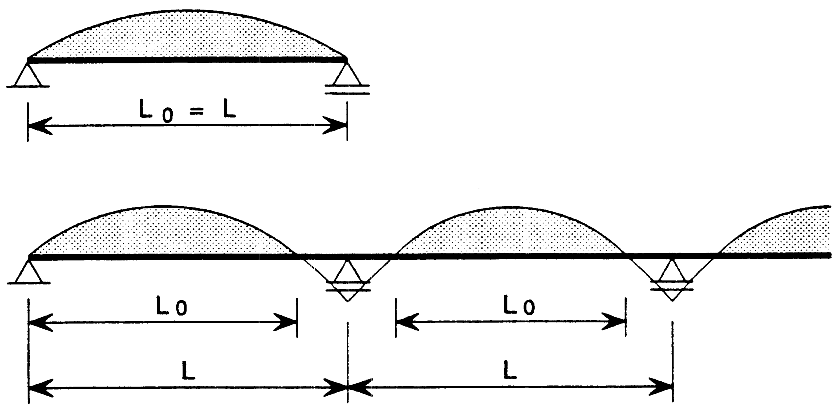
$$\ell_{fz} = 0{,}7L_0(1 + 13{,}1\,R_0^{1{,}6})^{-0{,}125} \qquad \dots (10.10a)$$

with:

$$R_0 = \frac{K\,L_0^4}{\pi^4\,E\,I_{fz}} \qquad \dots (10.10b)$$

in which I_{fz} and K are as defined in 10.1.4.1(7).

(7) For uplift loading, if the free flange is effectively held in position laterally at intervals by anti-sag bars, the buckling length may conservatively be taken as that for a uniform moment, determined as in (4).



[Dotted areas show regions in compression]

Figure 10.5: Varying compressive stress in free flange for uplift cases

10.1.5 Rotational restraint given by the sheeting

10.1.5.1 Lateral spring stiffness

(1) The lateral spring support given to the free flange of the purlin by the sheeting should be modelled as a lateral spring acting at the free flange, see figure 10.1. The total lateral spring stiffness K per unit length should be determined from:

$$\frac{1}{K} = \frac{1}{K_A} + \frac{1}{K_B} + \frac{1}{K_C} \qquad \dots (10.11)$$

where:

K_A is the lateral stiffness corresponding to the rotational stiffness of the connection between the sheeting and the purlin;

K_B is the lateral stiffness due to distortion of the cross-section of the purlin;

K_C is the lateral stiffness due to the flexural stiffness of the sheeting.

(2) Normally it may be assumed to be safe as well as realistic to neglect $1/K_C$ because K_C is very large compared to K_A and K_B. The value of K should then be obtained from:

$$K = \frac{1}{\left(1/K_A + 1/K_B\right)} \qquad \dots (10.12)$$

(3) The value of $\left(1/K_A + 1/K_B\right)$ may be obtained either by testing in accordance with Section 9, or by calculation.

NOTE: Appropriate testing procedures are given in annex A.

(4) The lateral spring stiffness K per unit length may be determined by calculation using:

$$\frac{1}{K} = \frac{4(1 - \nu^2)\, h^2\, (h_d + e)}{E\, t^3} + \frac{h^2}{C_D} \qquad \dots (10.13)$$

in which the dimension e is determined as follows:

- for cases bringing the purlin into contact with the sheeting at the purlin web:

  $e = a$

- for cases bringing the purlin into contact with the sheeting at the tip of the purlin flange:

  $e = 2a + b$

where:

a	is	the distance from the sheet-to-purlin fastener to the purlin web, see figure 10.6;
b	is	the width of the purlin flange connected to the sheeting, see figure 10.6;
C_D	is	the total rotational spring stiffness from 10.1.5.2;
h	is	the overall height of the purlin;
h_d	is	the developed height of the purlin web, see figure 10.6.

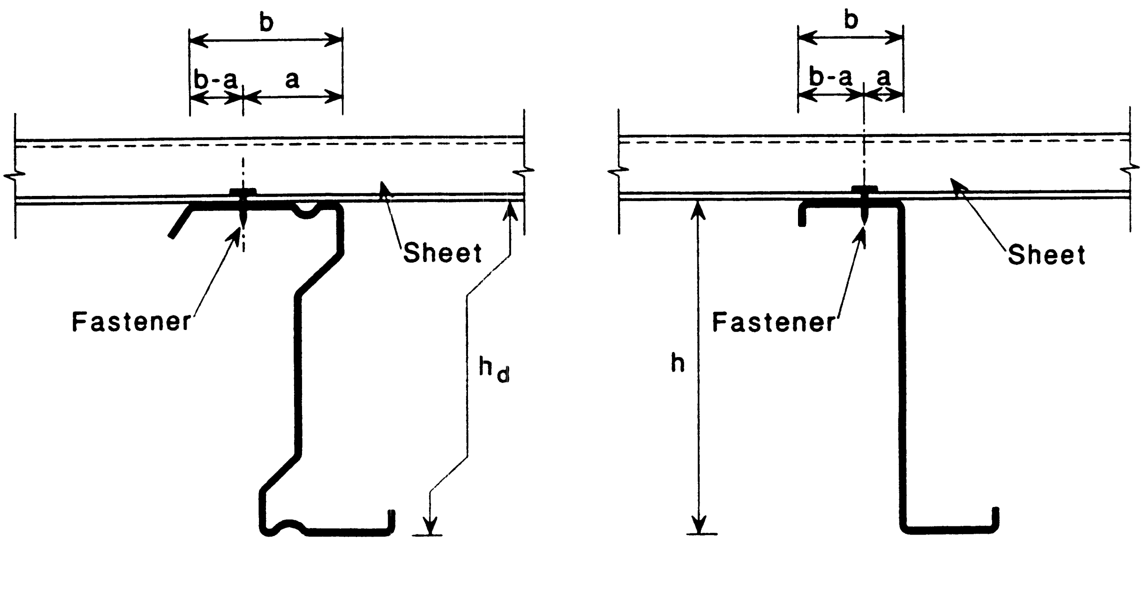


Figure 10.6: Purlin and attached sheeting

10.1.5.2 Rotational spring stiffness

(1) The rotational restraint given to the purlin by the sheeting that is connected to its top flange, should be modelled as a rotational spring acting at the top flange of the purlin, see figure 10.1. The total rotational spring stiffness C_D should be determined from:

$$C_D = \frac{1}{\left(1/C_{D,A} + 1/C_{D,C}\right)} \quad \text{... (10.14)}$$

where:

$C_{D,A}$ is the rotational stiffness of the connection between the sheeting and the purlin;

$C_{D,C}$ is the rotational stiffness corresponding to the flexural stiffness of the sheeting.

(2) Generally $C_{D,A}$ may be calculated as given in (5) and (7). Alternatively $C_{D,A}$ may be obtained by testing, see (9).

(3) The value of $C_{D,C}$ may be taken as the minimum value obtained from calculational models of the type shown in figure 10.7, taking account of the rotations of the adjacent purlins and the degree of continuity of the sheeting, using:

$$C_{D,C} = m / \theta \quad \text{... (10.15)}$$

where:

I_{eff} is the effective second moment of area per unit width of the sheeting;

m is the applied moment per unit width of sheeting, applied as indicated in figure 10.7;

θ is the resulting rotation, measured as indicated in figure 10.7 [radians].

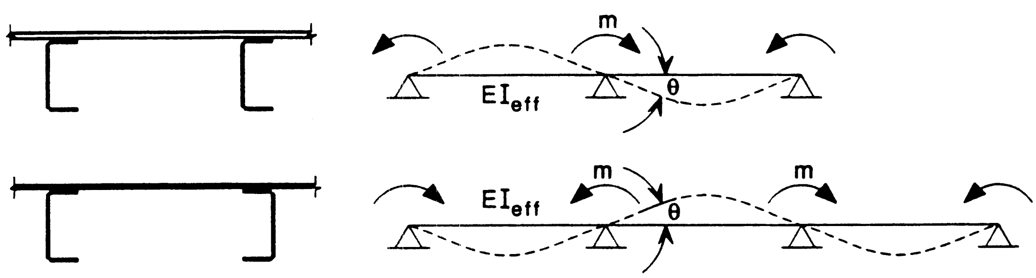


Figure 10.7: Model for calculating $C_{D,C}$

(4) Alternatively a conservative value of $C_{D,C}$ may be obtained from:

$$C_{D,C} = \frac{k E I_{eff}}{s} \quad \text{... (10.16)}$$

in which k is a numerical coefficient, with values as follows:

- for single span sheets: $k = 2$
- for sheets that are continuous over two or more spans: $k = 4$

where:

s is the spacing of the purlins.

(5) Provided that the sheet-to-purlin fasteners are positioned centrally on the flange of the purlin, the value of $C_{D,A}$ for trapezoidal sheeting connected to the top flange of the purlin may be determined as follows:

- if $b_a \leq 125$:

$$C_{D,A} = C_{100} \left[\frac{b_a}{100} \right]^2 \qquad \ldots (10.17a)$$

- if $125 \leq b_a \leq 200$:

$$C_{D,A} = 1{,}25\, C_{100} \left[\frac{b_a}{100} \right] \qquad \ldots (10.17b)$$

where:

b_a is the width of the purlin flange [in mm];

C_{100} is a rotation coefficient, representing the value of $C_{D,A}$ if $b_a = 100\,\text{mm}$.

(6) Provided that there is no insulation between the sheeting and the purlins, the value of the rotation coefficient C_{100} may be obtained from table 10.3.

(7) Alternatively $C_{D,A}$ may be taken as equal to $130p$ [Nm/m/radian], where p is the number of sheet-to-purlin fasteners per metre length of purlin (but not more than one per rib of sheeting), provided that:

- the flange width b of the sheeting through which it is fastened does not exceed 120 mm;
- the nominal core thickness t of the sheeting is at least 0,66 mm;
- the distance a or $b - a$ between the centreline of the fastener and the centre of rotation of the purlin (depending on the direction of rotation), as shown in figure 10.6, is at least 25 mm.

(8) If the effects of cross-section distortion (which is not included in the rotational spring stiffness C_D) have to be taken into account, see 10.1.5.1, it may be assumed to be realistic to neglect $C_{D,C}$, because the spring stiffness is mainly influenced by the value of $C_{D,A}$ and the cross-section distortion.

(9) Alternatively, values of $C_{D,A}$ may be obtained from a combination of testing and calculation.

(10) If the value of $(1/K_A + 1/K_B)$ is obtained by testing, the values of $C_{D,A}$ for gravity loading and for uplift loading should be determined from:

$$C_{D,A} = \frac{h^2}{(1/K_A + 1/K_B) - 4(1 - \nu^2)\, h^2 (h_d + e)/E t^3} \qquad \ldots (10.18)$$

in which e, h and h_d are as defined in 10.1.5.1(4).

Table 10.3: Rotation coefficient C_{100} for trapezoidal steel sheeting

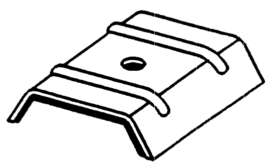
Positioning of sheeting		Sheet fastened through		Pitch of fasteners		Washer diameter	C_{100}	$b_{T,max}$
Positive	Negative	Trough	Crest	$e = b_R$	$e = 2b_R$	[mm]	[kNm/m]	[mm]
For gravity loading:								
×		×		×		22	5,2	40
×		×			×	22	3,1	40
	×		×	×		K_a	10,0	40
	×		×		×	K_a	5,2	40
	×	×		×		22	3,1	120
	×	×			×	22	2,0	120
For uplift loading:								
×		×		×		16	2,6	40
×		×			×	16	1,7	40

Key:

b_R is the corrugation width [185 mm maximum];

b_T is the width of the sheeting flange through which it is fastened to the purlin.

K_a indicates a steel saddle washer as shown below with $t \geq 0,75$ mm

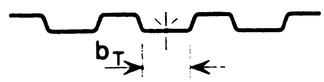


The values in this table are valid for:

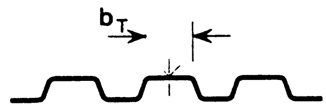
- sheet fastener screws of diameter: $\phi = 6,3$ mm;
- steel washers of thickness: $t_w \geq 1,0$ mm;
- sheeting of nominal core thickness: $t \geq 0,66$ mm.

Sheet fastened:

- through the trough:



- through the crest:



10.2 Liner trays restrained by sheeting

10.2.1 General

(1) Liner trays should be large channel-type sections, with two narrow flanges, two webs and one wide flange, generally as shown in figure 10.8. The two narrow flanges should be laterally restrained by attached profiled steel sheeting.

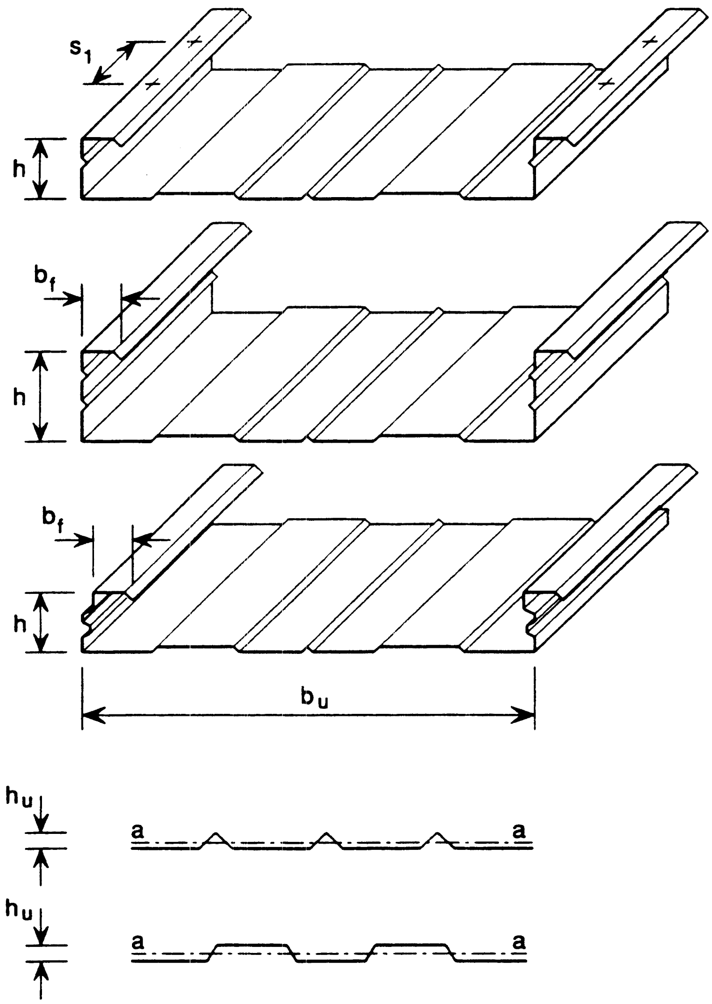


Figure 10.8: Typical geometry for liner trays

(2) The resistance of the webs of liner trays to shear forces and to local transverse forces should be obtained using 5.8 to 5.11, but using the value of $M_{c,Rd}$ given by (3) or (4).

(3) The moment resistance $M_{c,Rd}$ of a liner tray may be obtained using 10.2.2 provided that:

- the geometrical properties are within the range given in table 10.4;
- the depth h_u of the corrugations of the wide flange does not exceed $h/8$, where h is the overall depth of the liner tray.

(4) Alternatively the moment resistance of a liner tray may be determined by testing in accordance with Section 9, provided that it is ensured that the local behaviour of the liner tray is not affected by the testing equipment.

NOTE: Appropriate testing procedures are given in annex A.

Table 10.4: Range of validity of 10.2.2

0,75 mm	$\leq$	t_{nom}	$\leq$	1,5 mm
30 mm	$\leq$	b_f	$\leq$	60 mm
60 mm	$\leq$	h	$\leq$	200 mm
300 mm	$\leq$	b_u	$\leq$	600 mm
		I_a / b_u	$\leq$	10 mm^4 / mm
		s_1	$\leq$	1000 mm

10.2.2 Moment resistance

10.2.2.1 Wide flange in compression

(1) The moment resistance of a liner tray with its wide flange in compression should be determined using the step-by-step procedure outlined in figure 10.9 as follows:

- **Step 1:** Determine the effective areas of all compression elements of the cross-section, based on values of the stress ratio $\psi = \sigma_2 / \sigma_1$ obtained using the effective widths of the compression flanges but the gross areas of the webs;

- **Step 2:** Find the centroid of the effective cross-section, then obtain the moment resistance $M_{c,Rd}$ from:

$$M_{c,Rd} = W_{eff,min} f_{yb} / \gamma_{M2} \qquad \text{... (10.19)}$$

with:

$$W_{eff,min} = I_{y,eff} / z_c \quad \text{but} \quad W_{eff,min} \leq I_{y,eff} / z_t$$

$$\gamma_{M2} = \boxed{1,25}$$

where z_c and z_t are as indicated in figure 10.9.

NOTE: The use of γ_{M2} in this expression rather than γ_{M0} is necessary for calibration.

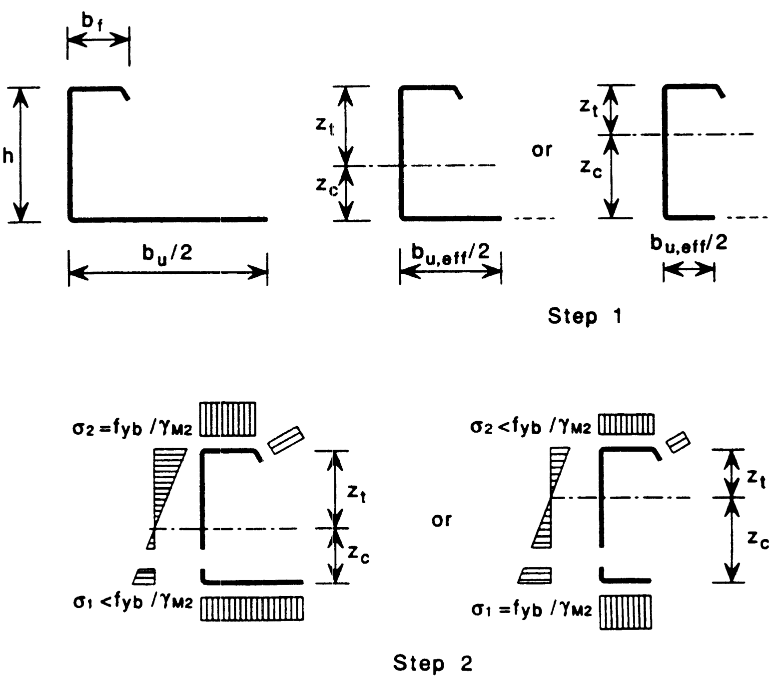


Figure 10.9: Determination of moment resistance — wide flange in compression

10.2.2.2 Wide flange in tension

(1) The moment resistance of a liner tray with its wide flange in tension should be determined using the step-by-step procedure outlined in figure 10.10 as follows:

- **Step 1:** Locate the centroid of the gross cross-section;
- **Step 2:** Obtain the effective width of the wide flange $b_{u,eff}$, allowing for possible flange curling, from:

$$b_{u,eff} = \frac{53{,}3 \times 10^{10} e_o^2 t^3 t_{eq}}{h L b_u^3} \quad \text{... (10.20)}$$

where:

b_u is the overall width of the wide flange;

e_o is the distance from the centroidal axis of the gross cross-section to the centroidal axis of the narrow flanges;

h is the overall depth of the liner tray;

L is the span of the liner tray;

t_{eq} is the equivalent thickness of the wide flange, given by:

$$t_{eq} = (12 I_a / b_u)^{1/3}$$

I_a is the second moment of area of the wide flange, about its own centroid.

- **Step 3:** Determine the effective areas of all the compression elements, based on values of the stress ratio $\psi = \sigma_2 / \sigma_1$ obtained using the effective widths of the flanges but the gross areas of the webs;

- **Step 4:** Find the centroid of the effective cross-section, then obtain the buckling resistance moment $M_{b,Rd}$ using:

$$M_{b,Rd} = \beta_b W_{eff,com} f_{yb} / \gamma_{M2} \quad \text{but} \quad M_{b,Rd} \leq W_{eff,t} f_{yb} / \gamma_{M2} \quad \text{... (10.21)}$$

with:

$$W_{eff,com} = I_{y,eff} / z_c$$

$$W_{eff,t} = I_{y,eff} / z_t$$

in which the correlation factor β_b is given by the following:

- if $s_1 \leq 300\,\text{mm}$:

$$\beta_b = 1{,}0$$

- if $300\,\text{mm} \leq s_1 \leq 1000\,\text{mm}$:

$$\beta_b = 1{,}15 - s_1 / 2000$$

where:

s_1 is the longitudinal spacing of fasteners supplying lateral restraint to the narrow flanges, see figure 10.8;

$\gamma_{M2} = \boxed{1{,}25}$

NOTE: The use of γ_{M2} in this expression rather than γ_{M0} and γ_{M1} is necessary for calibration.

(2) The effects of shear lag need not be considered if $L / b_{u,eff} \leq 20$. Otherwise a reduced value of ρ should be determined as specified in 5.4.3.

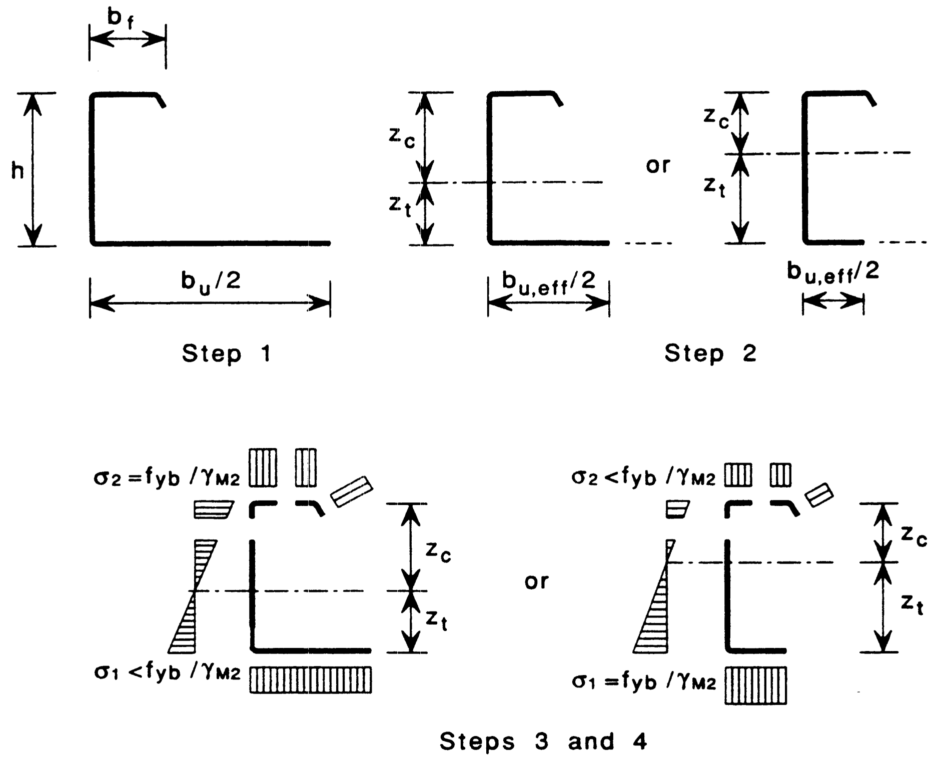


Figure 10.10: Determination of moment resistance — wide flange in tension

(3) Flange curling need not be taken into account in determining deflections at serviceability limit states.

(4) As a simplified alternative, the moment resistance of a liner tray with an unstiffened wide flange may be approximated by taking the same effective area for the wide flange in tension as for the two narrow flanges in compression combined.

10.3 Stressed skin design

10.3.1 General

(1)P The interaction between structural members and sheeting panels that are designed to act together as parts of a combined structural system, may be allowed for as described in this clause 10.3.

(2)P The provisions given in this clause shall be applied only to sheet diaphragms that are made of steel.

(3)P Diaphragms may be formed from profiled sheeting used as roof or wall cladding or for floors. They may also be formed from wall or roof structures based upon liner trays.

NOTE: Guidance on the verification of such diaphragms can be obtained from:

ECCS Publication No. 88 (1995): *European recommendations for the application of metal sheeting acting as a diaphragm.*

10.3.2 Diaphragm action

(1)P In stressed skin design, advantage may be taken of the contribution that diaphragms of sheeting used as roofing, flooring or wall cladding make to the overall stiffness and strength of the structural frame, by means of their stiffness and strength in shear.

(2) Roofs and floors may be treated as deep plate girders extending throughout the length of a building, resisting transverse in-plane loads and transmitting them to end gables, or to intermediate stiffened frames. The panel of sheeting may be treated as a web that resists in-plane transverse loads in shear, with the edge members acting as flanges that resist axial tension and compression forces, see figures 10.11 and 10.12.

(3) Similarly, rectangular wall panels may be treated as bracing systems that act as shear diaphragms to resist in-plane forces.

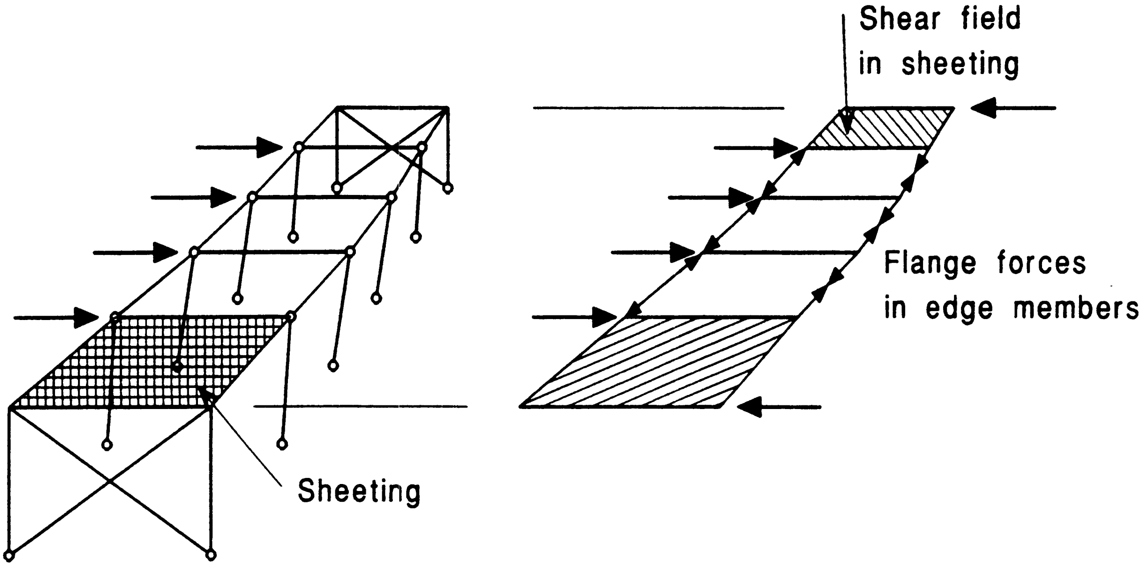


Figure 10.11: Stressed skin action in a flat-roof building

10.3.3 Necessary conditions

(1)P Methods of stressed skin design that utilize sheeting as an integral part of a structure, may be used only under the following conditions:

- the use made of the sheeting, in addition to its primary purpose, is limited to the formation of shear diaphragms to resist structural displacement in the plane of that sheeting;
- the diaphragms have longitudinal edge members to carry flange forces arising from diaphragm action;
- the diaphragm forces in the plane of a roof or floor are transmitted to the foundations by means of braced frames, further stressed-skin diaphragms, or other methods of sway resistance;
- suitable structural connections are used to transmit diaphragm forces to the main steel framework and to join the edge members acting as flanges;
- the sheeting is treated as a structural component that cannot be removed without proper consideration;
- the project specification, including the calculations and drawings, draws attention to the fact that the building is designed to utilize stressed skin action.

(2)P Stressed skin design shall be used predominantly in low-rise buildings, or in the floors and facades of high-rise buildings.

(3)P Stressed skin diaphragms shall be used predominantly to resist wind loads, snow loads and other loads that are applied through the sheeting itself. They may also be used to resist small transient loads, such as surge from light overhead cranes or hoists on runway beams, but may not be used to resist permanent external loads, such as those from plant.

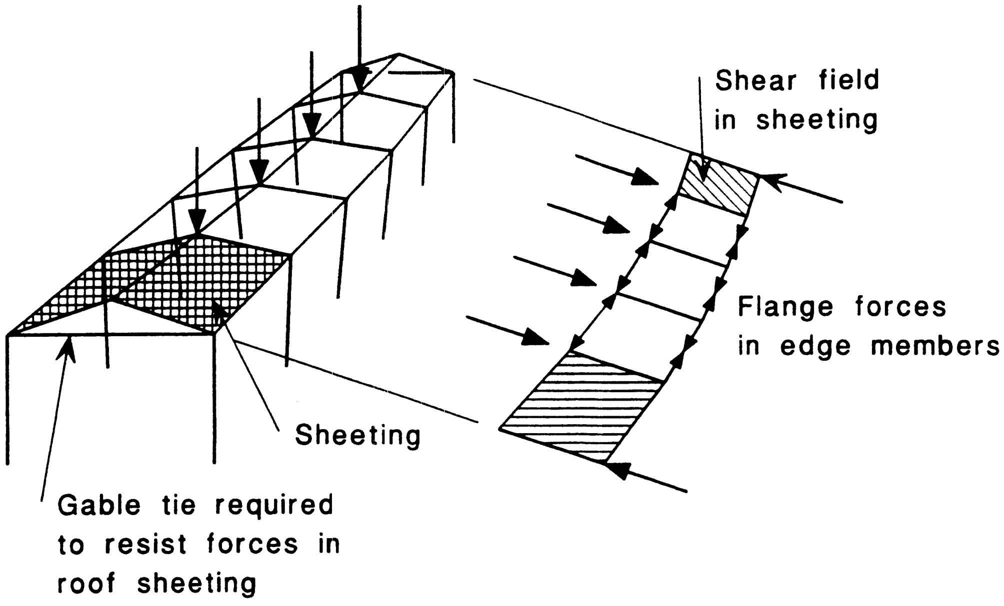


Figure 10.12: Stressed skin action in a pitched roof building

10.3.4 Profiled steel sheet diaphragms

(1)P In a profiled steel sheet diaphragm, see figure 10.13, both ends of the sheets shall be attached to the supporting members by means of self-tapping screws, cartridge fired pins, welding, bolts or other fasteners of a type that will not work loose in service, pull out, or fail in shear before causing tearing of the sheeting. All such fasteners shall be fixed directly through the sheeting into the supporting member, for example through the troughs of profiled sheets, unless special measures are taken to ensure that the connections effectively transmit the forces assumed in the design.

(2)P The seams between adjacent sheets shall be fastened by rivets, self-drilling screws, welds, or other fasteners of a type that will not work loose in service, pull out, or fail in shear before causing tearing of the sheeting. The spacing of such fasteners shall not exceed 500 mm.

(3)P The distances from all fasteners to the edges and ends of the sheets shall be adequate to prevent premature tearing of the sheets.

(4) Small randomly arranged openings, up to 3% of the relevant area, may be introduced without special calculation, provided that the total number of fasteners is not reduced. Openings up to 15% of the relevant area may be introduced if justified by detailed calculations. Areas that contain larger openings should be split into smaller areas, each with full diaphragm action.

(5)P All sheeting that also forms part of a stressed-skin diaphragm shall first be designed for its primary purpose in bending. To ensure that any deterioration of the sheeting would be apparent in bending before the resistance to stressed skin action is affected, it shall then be verified that the shear stress due to diaphragm action does not exceed $0{,}25 f_{yb}/\gamma_{M1}$.

(6)P The shear resistance of a stressed-skin diaphragm shall be based on the least tearing strength of the seam fasteners or the sheet-to-member fasteners parallel to the corrugations or, for diaphragms fastened only to longitudinal edge members, the end sheet-to-member fasteners. The calculated shear resistance for any other type of failure shall exceed this minimum value by at least the following:

- for failure of the sheet-to-purlin fasteners under combined shear and wind uplift, by at least 40%;
- for any other type of failure, by at least 25%.

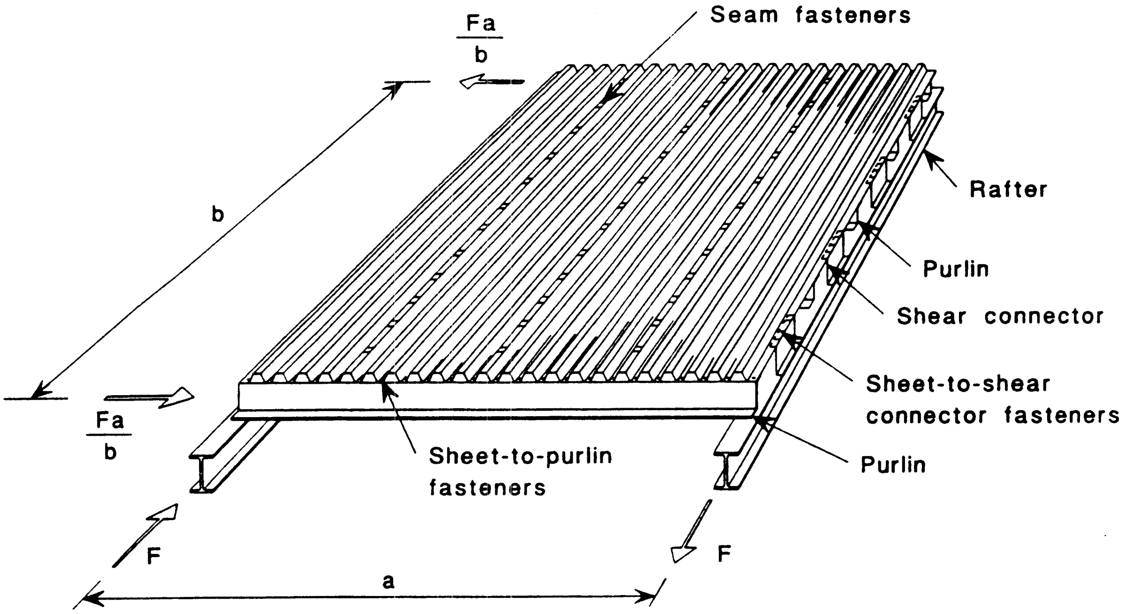


Figure 10.13: Arrangement of an individual panel

10.3.5 Steel liner tray diaphragms

(1) Liner trays used to form shear diaphragms should have stiffened wide flanges.

(2) Liner trays in shear diaphragms should be inter-connected by seam fasteners through the web at a spacing e_s of not more than 300 mm by seam fasteners (normally blind rivets) located at a distance e_u from the wide flange of not more than 30 mm, all as shown in figure 10.14.

(3) An accurate evaluation of deflections due to fasteners may be made using a similar procedure to that for trapezoidal profiled sheeting.

(4) The shear flow $T_{v,Sd}$ due to ultimate limit states design loads should not exceed $T_{v,Rd}$ given by:

$$T_{v,Rd} = 0{,}2\,E\,\sqrt[4]{I_1\,(t/b_u)^9} \qquad \text{... (10.22)}$$

where:

I_1 is the second moment of area of the wide flange;

b_u is the overall width of the wide flange.

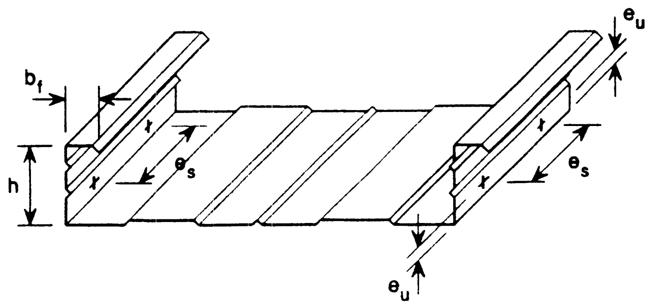


Figure 10.14: Location of seam fasteners

(5) The shear flow $T_{v,ser}$ due to serviceability design loads should not exceed $T_{v,Cd}$ given by:

$$T_{v,Cd} = S_v / 375 \qquad \text{... (10.23)}$$

where:

S_v is the shear stiffness of the diaphragm, per unit length of the span of the liner trays.

(6) The shear stiffness S_v per unit length may be obtained from:

$$S_v = \frac{\alpha L b_u}{e_s (b - b_u)} \qquad \text{... (10.24)}$$

where:

L is the overall length of the shear diaphragm (in the direction of the span of the liner trays);

b is the overall width of the shear diaphragm ($b = \Sigma b_u$);

α is the stiffness factor.

(7) The stiffness factor α may be derived from tests in accordance with Section 9. Alternatively, in the absence of test results, α may conservatively be taken as equal to 2000 N/mm.

10.4 Perforated sheeting

(1) Perforated sheeting may be designed by calculation, provided that the rules for non-perforated sheeting are modified by introducing the effective thicknesses given below.

NOTE: These calculation rules tend to give rather conservative values. More economical solutions might be obtained from design assisted by testing, see Section 9.

(2) Provided that $0{,}2 \leq d/a \leq 0{,}8$ gross section properties may be calculated using 3.3.2, but replacing t by $t_{a,eff}$ obtained from:

$$t_{a,eff} = 1{,}18t\left(1 - 0{,}9d/a\right) \qquad \text{... (10.25)}$$

where:

d is the diameter of the perforations;

a is the spacing between the centres of the perforations.

(3) Provided that $0{,}2 \leq d/a \leq 1{,}0$ effective section properties may be calculated using Section 4, but replacing t by $t_{b,eff}$ obtained from:

$$t_{b,eff} = t\sqrt[3]{1{,}18\left(1 - d/a\right)} \qquad \text{... (10.26)}$$

(4) The resistance of a single unstiffened web to local transverse forces may be calculated using 5.9, but replacing t by $t_{c,eff}$ obtained from:

$$t_{c,eff} = t\left[1 - \left(d/a\right)^2 s_{per} / s_w\right]^{3/2} \qquad \text{... (10.27)}$$

where:

s_{per} is the slant height of the perforated portion of the web;

s_w is the total slant height of the web.

Annex A [informative]

Testing procedures

A.1 General

(1) This annex A gives appropriate standardized testing and evaluation procedures for a number of tests that are commonly required in practice, as a basis for harmonization of future testing.

(2) It is, however, recognized that most existing test data have been obtained on the basis of tests that differ to some extent from these procedures.

(3) So that the existing data can continue to be used, and to allow sufficient time for transition to harmonized procedures after adequate trial implementation, these testing procedures are presented as an informative annex, covering:

- tests on profiled sheets and liner trays, see A.2;
- tests on cold formed members, see A.3;
- tests on structures and portions of structures, see A.4;
- tests on beams torsionally restrained by sheeting, see A.5;
- evaluation of test results to determine design values, see A.6.

A.2 Tests on profiled sheets and liner trays

A.2.1 General

(1) Although these test procedures are presented in terms of profiled sheets, similar test procedures based on the same principles may also be used for liner trays.

(2) Loading may be applied through air bags or in a vacuum chamber or by steel or timber cross beams arranged to simulate uniformly distributed loading.

(3) To prevent spreading of corrugations, transverse ties or other appropriate test accessories such as timber blocks may be applied to the test specimen. Some examples are given in figure A.1.

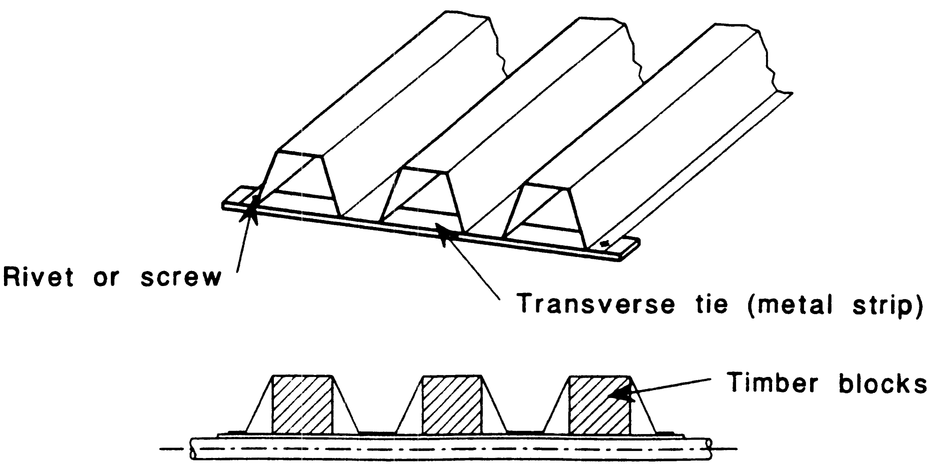


Figure A.1: Examples of appropriate test accessories

(4) For uplift tests, the test set-up should realistically simulate the behaviour of the sheeting under practical conditions. The type of connections between the sheet and the supports should be the same as in the connections to be used in practice.

(5) To give the results a wide range of applicability, hinged and roller supports should preferably be used, to avoid any influence of torsional restraint at the supports on the test results,

(6) It should be ensured that the direction of the loading remains perpendicular to the initial plane of the sheet throughout the test procedure.

(7) To eliminate the deformations of the supports, the deflections at both ends of the test specimen should also be measured.

(8) The test result should be taken as the maximum value of the loading applied to the specimen either coincident with failure or immediately prior to failure as appropriate.

A.2.2 Single span test

(1) A test set-up equivalent to that shown in figure A.2 may be used to determine the midspan moment resistance (in the absence of shear force) and the effective flexural stiffness.

(2) The span should be chosen such that the test results represent the moment resistance of the sheet.

(3) The moment resistance should be determined from the test result.

(4) The flexural stiffness should be determined from a plot of the load-deflection behaviour.

A.2.3 Double span test

(1) The test set-up shown in figure A.3 may be used to determine the resistance of a sheet that is continuous over two or more spans to combinations of moment and shear at internal supports, and its resistance to combined moment and support reaction for a given support width.

(2) The loading should preferably be uniformly distributed (applied using an air bag or a vacuum chamber, for example).

(3) Alternatively any number of line loads (transverse to the span) may be used, arranged to produce internal moments and forces that are appropriate to represent the effects of uniformly distributed loading. Some examples of suitable arrangements are shown in figure A.4.

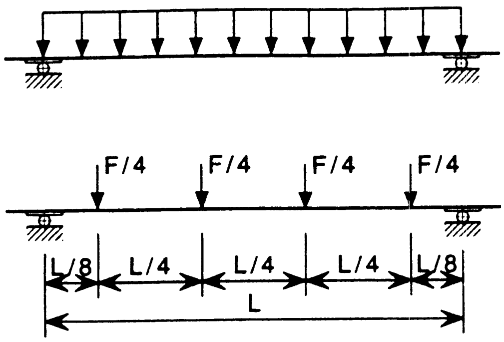
A.2.4 Internal support test

(1) As an alternative to A.2.3, the test set-up shown in figure A.5 may be used to determine the resistance of a sheet that is continuous over two or more spans to combinations of moment and shear at internal supports, and its resistance to combined moment and support reaction for a given support width.

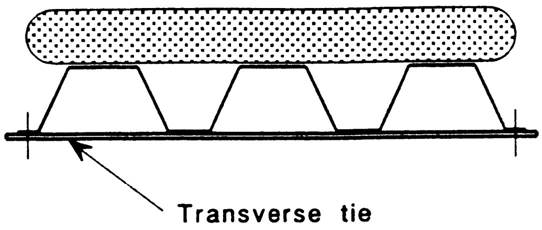
(3) The test span s used to represent the portion of the sheet between the points of contraflexure each side of the internal support, in a sheet continuous over two equal spans L may be obtained from:

$$s = 0{,}4L \qquad \text{... (A.1)}$$

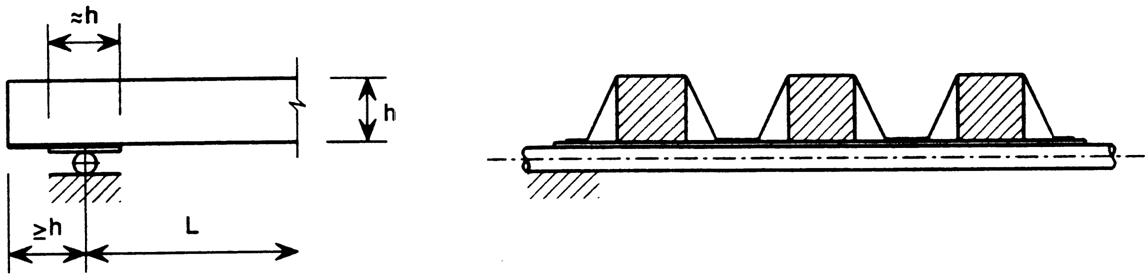
(3) If plastic redistribution of the support moment is expected, the test span s should be reduced to represent the appropriate ratio of support moment to shear force.



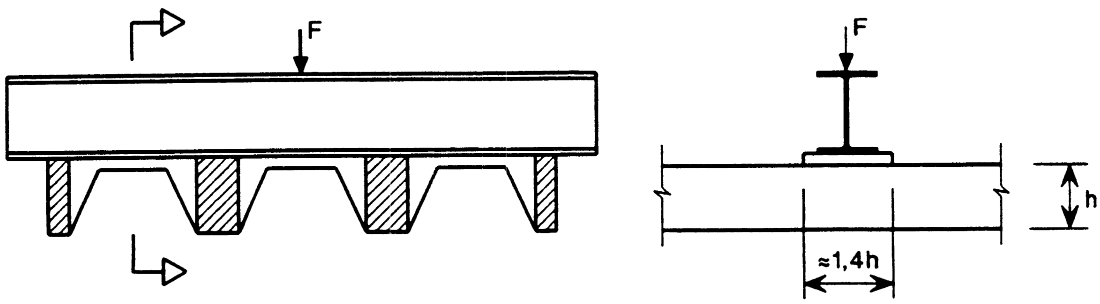
a) Uniformly distributed loading and an example of alternative equivalent line loads



b) Distributed loading applied by an airbag (alternatively by a vacuum test rig)



c) Example of support arrangements for preventing distortion



d) Example of method of applying a line load

Figure A.2: Test set-up for single span tests

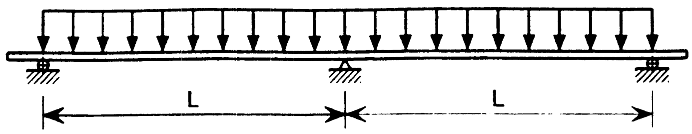


Figure A.3: Test setup for double span tests

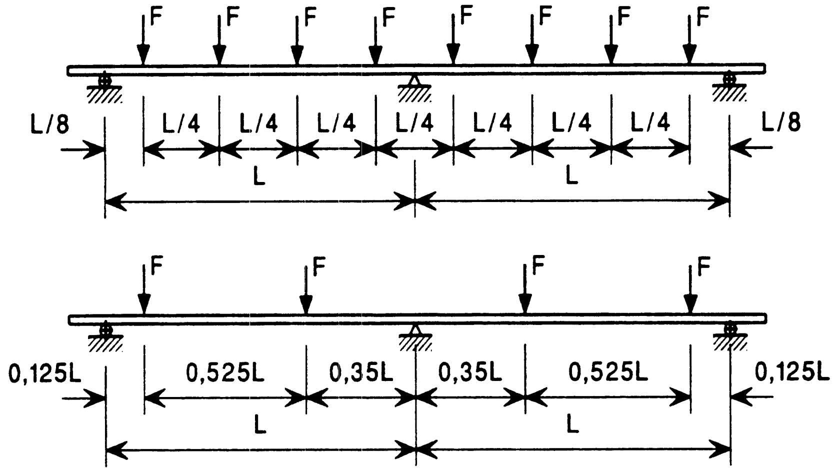


Figure A.4: Examples of suitable arrangements of alternative line loads

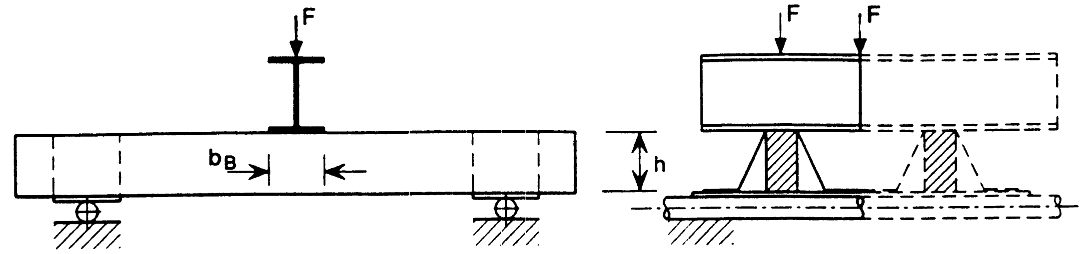
(4) The width b_B of the beam used to apply the test load should be selected to represent the actual support width to be used in practice.

(5) Each test result may be used to represent the resistance to combined bending moment and support reaction (or shear force) for a given span and a given support width. To obtain information about the interaction of bending moment and support reaction, tests should be carried out for several different spans.

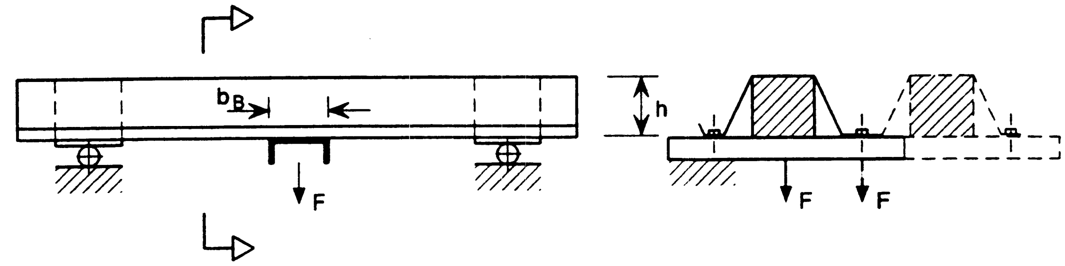
A.2.5 End support test

(1) The test set-up shown in figure A.6 may be used to determine the shear resistance of a sheet at an end support.

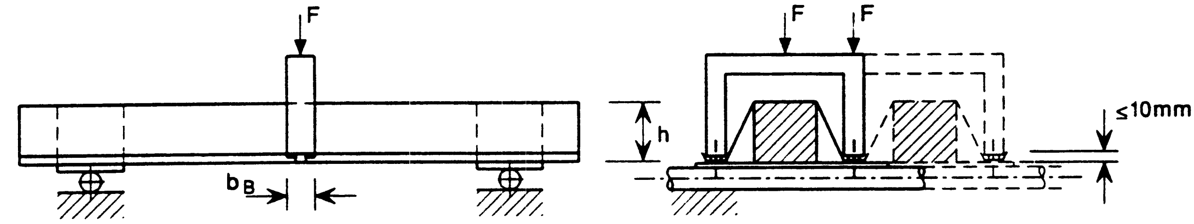
(2) Separate tests should be carried out to determine the shear resistance of the sheet for different lengths u from the contact point at the inner edge of the end support, to the actual end of the sheet, see figure A.6.



a) Internal support under gravity loading

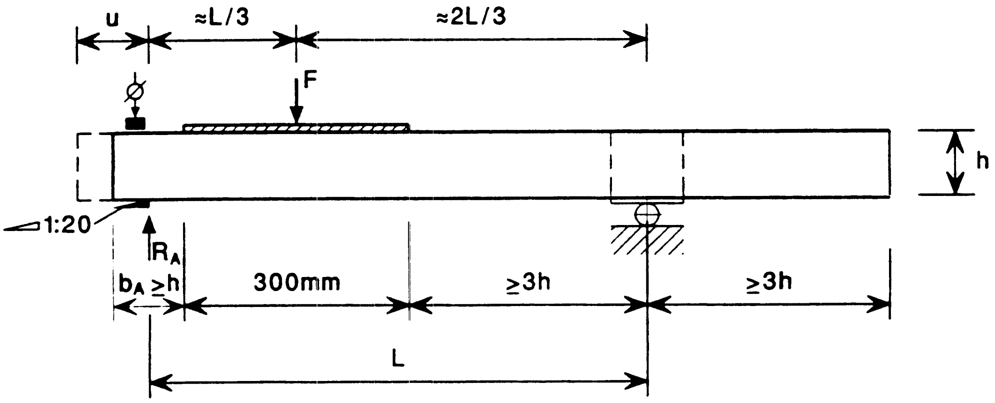


b) Internal support under simulated uplift loading



c) Internal support with loading applied to tension flange

Figure A.5: Test set-up for internal support tests



Key:

b_A = support width

u = length from internal edge of end support to end of sheet

Figure A.6: Test set-up for end support tests

A.3 Tests on cold formed members

A.3.1 General

(1) Each test specimen should be similar in all respects to the component or structure that it represents.

(2) The supporting devices used for tests should preferably provide end conditions that closely reproduce those supplied by the connections to be used in service. Where this cannot be achieved, less favourable end conditions that decrease the load carrying capacity or increase the flexibility should be used, as relevant.

(3) The devices used to apply the test loads should reproduce the way that the loads would be applied in service. It should be ensured that they do not offer more resistance to transverse deformations of the cross-section than would be available in the event of an overload in service. It should also be ensured that they do not localize the applied forces onto the lines of greatest resistance.

(4) If the given load combination includes forces on more than one line of action, each increment of the test loading should be applied proportionately to each of these forces.

(5) At each stage of the loading, the displacements or strains should be measured at one or more principal locations on the structure. Readings of displacements or strains should not be taken until the structure has completely stabilized after a load increment.

(6) Failure of a test specimen should be considered to have occurred in any of the following cases:

- at collapse or fracture;
- if a crack begins to spread in a vital part of the specimen;
- if the displacement is excessive.

(7) The test result should be taken as the maximum value of the loading applied to the specimen either coincident with failure or immediately prior to failure as appropriate.

(8) The accuracy of all measurements should be compatible with the magnitude of the measurement concerned and should in any case not exceed $\pm 1\%$ of the value to be determined.

(9) The measurements of the cross-sectional geometry of the test specimen should include:

- the overall dimensions (width, depth and length) to an accuracy of $\pm 1{,}0$ mm;
- widths of plane elements of the cross-section to an accuracy of $\pm 1{,}0$ mm;
- radii of bends to an accuracy of $\pm 1{,}0$ mm;
- inclinations of plane elements to an accuracy of $\pm 2{,}0°$;
- angles between flat surfaces to an accuracy of $\pm 2{,}0°$;
- locations and dimensions of intermediate stiffeners to an accuracy of $\pm 1{,}0$ mm;
- the thickness of the material to an accuracy of $\pm 0{,}01$ mm.

(10) All other relevant parameters should also be measured, such as:

- locations of components relative to each other;
- locations of fasteners;
- the values of torques etc. used to tighten fasteners.

A.3.2 Full cross-section compression tests

A.3.2.1 Stub column test

(1) Stub column tests may be used to allow for the effects of local buckling in thin gauge cross-sections, by determining the value of the ratio $\beta_A = A_{eff}/A_g$ and the location of the effective centroidal axis.

(2) If local buckling of the plane elements governs the resistance of the cross-section, the specimen should have a length of at least 3 times the width of the widest plate element.

(3) The lengths of specimens with perforated cross-sections should include at least 5 pitches of the perforations, and should be such that the specimen is cut to length midway between two perforations.

(4) In the case of a cross-section with edge or intermediate stiffeners, it should be ensured that the length of the specimen is not less than the expected buckling lengths of the stiffeners.

(5) If the overall length of the specimen exceeds 20 times the least radius of gyration of its gross cross-section i_{min}, intermediate lateral restraints should be supplied at a spacing of not more than $20\,i_{min}$.

(6) Before testing, the tolerances of the cross-sectional dimensions of the specimen should be checked to ensure that they are within the permitted deviations.

(7) The cut ends of the specimen should be flat, and should be perpendicular to its longitudinal axis.

(8) An axial compressive force should be applied to each end of the specimen through pressure pads at least 30 mm thick, that protrude at least 10 mm beyond the perimeter of the cross-section.

(9) The test specimen should be placed in the testing machine with a ball bearing at each end. There should be small drilled indentations in the pressure pads to receive the ball bearings. The ball bearings should be located in line with the centroid of the calculated effective cross-section. If the calculated location of this effective centroid proves not to be correct, it may be adjusted within the test series.

(10) In the case of open cross-sections, possible spring-back may be corrected.

(11) Stub column tests may be used to determine the compression resistance of a cross-section. In interpreting the test results, the following parameters should be treated as variables:

- the thickness;
- the ratio b_p/t;
- the ratio f_u/f_{yb};
- the location of the centroid of the effective cross-section;
- imperfections in the shape of the elements of the cross-section;
- the method of cold rolling (for example increasing the yield strength by introducing a deformation that is subsequently removed).

A.3.2.2 Member buckling test

(1) Member buckling tests may be used to determine the resistance of compression members with thin gauge cross-sections to overall buckling (including flexural buckling, torsional buckling and torsional-flexural buckling) and the interaction between local buckling and overall buckling.

(2) The method of carrying out the test should be generally as given for stub column tests in A.3.2.1.

(3) A series of tests on axially loaded specimens may be used to determine the appropriate buckling curve for a given type of cross-section and a given grade of steel, produced by a specific process. The values of relative slenderness $\bar{\lambda}$ to be tested and the minimum number of tests n at each value, should be as given in table A.1.

Table A.1: Relative slenderness values and numbers of tests

$\bar{\lambda}$	0,2	0,5	0,7	1,0	1,3	1,6	2,0	3,0
n	3	5	5	5	5	5	5	5

(4) Similar tests may also be used to determine the effect of introducing intermediate restraints on the torsional buckling resistance of a member.

(5) For the interpretation of the test results the following parameters should be taken into account:

- the parameters listed for stub column tests in A.3.2.1(11);
- overall lack of straightness imperfections;
- type of end or intermediate restraint (flexural, torsional or both).

A.3.3 Full cross-section tension test

(1) This test may be used to determine the average yield strength f_{ya} of the cross-section.

(2) The specimen should have a length of at least 5 times the width of the widest plane element in the cross-section.

(3) The load should be applied through end supports that ensure a uniform stress distribution across the cross-section.

(4) The failure zone should occur at a distance from the end supports of not less than the width of the widest plane element in the cross-section.

A.3.4 Full cross-section bending test

(1) This test may be used to determine the moment resistance and rotation capacity of a cross-section.

(2) The specimen should have a length of at least 15 times its greatest lateral dimension. The spacing of lateral restraints to the compression flange should not be less than the spacing to be used in service.

(3) A pair of point loads should be applied to the specimen to produce a length under uniform bending moment at midspan of at least 0,2 × (span) but not more than 0,33 × (span). These loads should be applied through the shear centre of the cross-section. If necessary, local buckling of the specimen should be prevented at the points of load application, to ensure that failure occurs within the central portion of the span. The deflection should be measured at the load positions, at midspan and at the ends of the specimen.

(4) In interpreting the test results, the following parameters should be treated as variables:

- the thickness;
- the ratio b_p / t;
- the ratio f_u / f_y;
- differences between restraints used in the test and those available in service;
- the support conditions.

A.4 Tests on structures and portions of structures

A.4.1 Acceptance test

(1) This acceptance test may be used as a non-destructive test to confirm the structural performance of a structure or portion of a structure.

(2) The test load for an acceptance test should be taken as equal to the sum of:

- $\boxed{1,0}$ × (the actual self-weight present during the test);
- $\boxed{1,15}$ × (the remainder of the permanent load);
- $\boxed{1,25}$ × (the variable loads).

but need not be taken as more than the mean of the total ultimate limit state design load and the total serviceability limit state design load for the characteristic (rare) load combination.

(3) Before carrying out the acceptance test, preliminary bedding down loading (not exceeding the characteristic values of the loads) may optionally be applied, and then removed.

(4) The structure should first be loaded up to a load equal to the total characteristic load. Under this load it should demonstrate substantially elastic behaviour. On removal of this load the residual deflection should not exceed 20% of the maximum recorded. If these criteria are not satisfied this part of the test procedure should be repeat. In this repeat load cycle, the structure should demonstrate substantially linear behaviour up to the characteristic load and the residual deflection should not exceed 10% of the maximum recorded.

(5) During the acceptance test, the loads should be applied in a number of regular increments at regular time intervals and the principal deflections should be measured at each stage. When the deflections show significant non-linearity, the load increments should be reduced.

(6) On the attainment of the acceptance test load, the load should be maintained for at least one hour and deflection measurements should be taken to establish whether the structure is subject to any time-dependent deformations, such as deformations of fasteners or deformations arising from creep in the zinc layer.

(7) Unloading should be completed in regular decrements, with deflection readings taken at each stage.

(8) The structure should prove capable of sustaining the acceptance test load, and there should be no significant local distortion or defects likely to render the structure unserviceable after the test.

A.4.2 Strength test

(1) This strength test may be used to confirm the calculated load carrying capacity of a structure or portion of a structure. Where a number of similar items are to be constructed to a common design, and one or more prototypes have been submitted to and met all the requirements of this strength test, the others may be accepted without further testing provided that they are similar in all relevant respects to the prototypes.

(2) Before carrying out a strength test the specimen should first pass the acceptance test detailed in A.4.1.

(3) The load should then be increased in increments up to the strength test load and the principal deflections should be measured at each stage. The strength test load should be maintained for at least one hour and deflection measurements should be taken to establish whether the structure is subject to creep.

(4) Unloading should be completed in regular decrements with deflection readings taken at each stage.

(5) The total test load (including self-weight) for a strength test F_{str} should be determined from the total design load F_{Sd} specified for ultimate limit state verifications by calculation, using:

$$F_{str} = \mu_F F_{Sd} \quad \text{... (A.2)}$$

in which μ_F is the load adjustment coefficient.

(6) The load adjustment coefficient μ_F should take account of variations in the load carrying capacity of the structure, or portion of a structure, due to the effects of variation in the material yield strength, local buckling, overall buckling and any other relevant parameters or considerations.

(7) Where a realistic assessment of the load carrying capacity of the structure, or portion of a structure, can be made using the provisions of this Part 1.3 of ENV 1993 for design by calculation, or another proven method of analysis that takes account of all buckling effects, the load adjustment coefficient μ_F may be taken as equal to the ratio of (the value of the assessed load carrying capacity based on the averaged basic yield strength f_{ym}) compared to (the corresponding value based on the nominal basic yield strength f_{yb}).

(8) The value of f_{ym} should be determined from the measured basic strength $f_{yb,obs}$ of the various components of the structure, or portion of a structure, with due regard to their relative importance.

(9) If realistic theoretical assessments of the load carrying capacity cannot be made, the load adjustment coefficient μ_F should be taken as equal to the resistance adjustment coefficient μ_R defined in A.6.2.

(10) Under the test load there should be no failure by buckling or rupture in any part of the specimen.

(11) On removal of the test load, the deflection should be reduced by at least 20%.

A.4.3 Prototype failure test

(1) A test to failure may be used to determine the real mode of failure and the true load carrying capacity of a structure or assembly. If the prototype is not required for use, it may optionally be used to obtain this additional information after completing the strength test described in A.4.2.

(2) Alternatively a test to failure may be carried out to determine the true design load carrying capacity from the ultimate test load. As the acceptance and strength test procedures should preferably be carried out first, an estimate should be made of the anticipated design load carrying capacity as a basis for such tests.

(3) Before carrying out a test to failure, the specimen should first pass the strength test described in A.4.2. Its estimated design load carrying capacity may then be adjusted based on its behaviour in the strength test.

(4) During a test to failure, the loading should first be applied in increments up to the strength test load. Subsequent load increments should then be based on an examination of the plot of the principal deflections.

(5) The ultimate load carrying capacity should be taken as the value of the test load at that point at which the structure or assembly is unable to sustain any further increase in load.

> **NOTE:** At this point gross permanent distortion is likely to have occurred. In some cases gross deformation might define the test limit.

A.4.4 Identification test

(1) An identification test may be used to:

- verify load bearing behaviour relative to analytical design models;
- quantify parameters derived from design models, such as strength or stiffness of members or joints.

A.5 Tests on beams torsionally restrained by sheeting

A.5.1 General

(1) These test procedures may be used for beams that are partially restrained against torsional displacement, by means of trapezoidal profiled steel sheeting or other suitable cladding.

(2) These procedures may be used for purlins, side rails, floor beams and other similar types of beams that have relevant restraint conditions.

A.5.2 Internal support test

A.5.2.1 Test set-up

(1) The test set-up shown in figure A.7 may be used to determine the resistance of a beam that is continuous over two or more spans, to combinations of bending moment and shear force at internal supports.

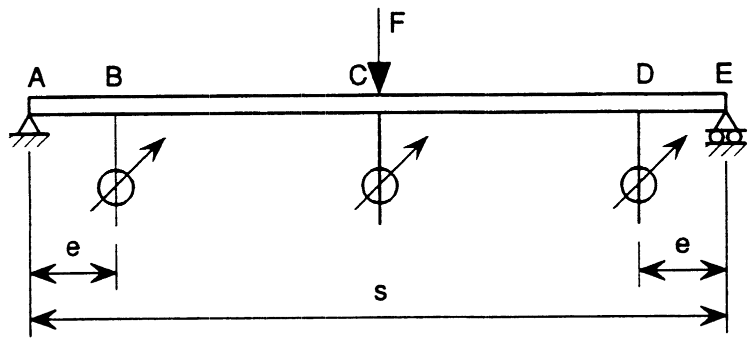


Figure A.7: Test set-up for internal support tests

(2) The supports at **A** and **E** should be hinged and roller supports respectively. At these supports, rotation about the longitudinal axis of the beam may be prevented, for example by means of cleats.

(3) The method of applying the load at **C** should correspond with the method to be used in service.

NOTE: In many cases this will mean that lateral displacement of both flanges is prevented at **C**.

(4) In order to eliminate the displacements of the supports, the vertical deflections should be measured at the point of load application **C** and at two points **B** and **D** located at a distance e from each support.

(5) The test span s should be chosen to produce combinations of bending moment and shear force that represent those expected to occur in practical application under the design load for the relevant limit state.

(6) For double span beams of span L subject to uniformly distributed loads, the test span s should normally be taken as equal to $0,4L$. However, if plastic redistribution of the support moment is expected, the test span s should be reduced to represent the appropriate ratio of support moment to shear force.

A.5.2.2 Execution of tests

(1) In addition to the general rules for testing, the following specific aspects should be taken into account.

(2) Testing should continue beyond the peak load and the recording of the deflections should be continued either until the applied load has reduced to between 10% and 15% of its peak value or until the deflection has reached a value 6 times the maximum elastic displacement.

A.5.2.3 Interpretation of test results

(1) The actual measured test results $R_{obs,i}$ should be adjusted as specified in A.6.2 to obtain adjusted values $R_{adj,i}$ related to the nominal basic yield strength f_{yb} and design thickness t of the steel, see 3.1.3.

(2) For each value of the test span s the support reaction R should be taken as the mean of the adjusted values of the peak load F_{max} for that value of s. The corresponding value of the support moment M should then be determined from:

$$M = \frac{sR}{4} \qquad \text{... (A.3)}$$

(3) The pairs of values of M and R for each value of s should be plotted as shown in figure A.8. Pairs of values for intermediate combinations of M and R may then be determined by linear interpolation.

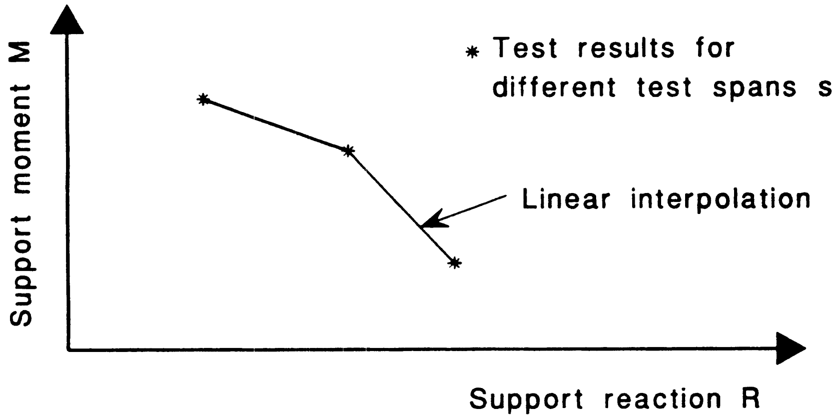


Figure A.8: Relation between support moment and support reaction

(4) The net deflection at the point of load application **C** in figure A.7 should be obtained from the gross measured values by deducting the mean of the corresponding deflections measured at the points **B** and **D** located at a distance e from the support points **A** and **E**, see figure A.7.

(5) For each test the applied load should be plotted against the corresponding net deflection, see figure A.9. From this plot, the rotation θ should be obtained for a range of values of the applied load using:

$$\theta = \frac{2(\delta_{p\ell} - \delta_{e\ell})}{0{,}5\,s - e} \qquad \text{... (A.4)}$$

where:

$\delta_{e\ell}$ is the net deflection for a given load on the rising part of the curve, before F_{max};

$\delta_{p\ell}$ is the net deflection for the same load on the falling part of the curve, after F_{max};

s is the test span;

e is the distance between a deflection measurement point and a support, see figure A.7.

(6) The relationship between M and θ should then be plotted for each test at a given test span s corresponding to a given value of beam span L as shown in figure A.10. The design M - θ characteristic for the moment resistance of the beam over an internal support should then be taken as equal to 0,9 times the mean value of M for all the tests corresponding to that value of the beam span L.

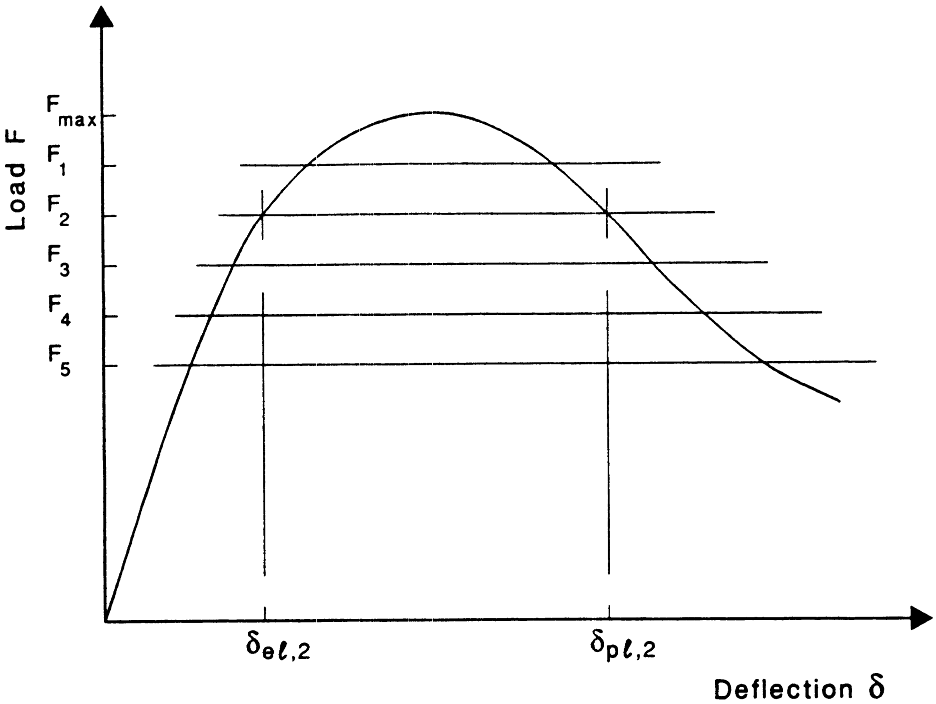


Figure A.9: Relationship between load and net deflection

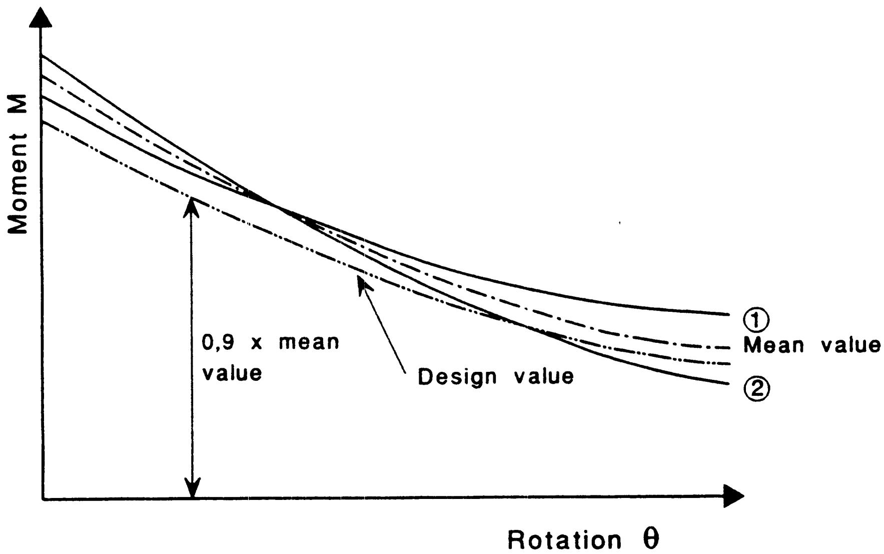


Figure A.10: Derivation of the design moment-rotation characteristic

A.5.3 Determination of torsional restraint

(1) The test set-up shown in figure A.11 may be used to determine the amount of torsional restraint given by adequately fastened sheeting or by another member perpendicular to the span of the beam.

(2) This test set-up covers two different contributions to the total amount of restraint as follows:

a) The lateral stiffness K_A per unit length corresponding to the rotational stiffness of the connection between the sheeting and the beam;

b) The lateral stiffness K_B per unit length due to distortion of the cross-section of the purlin.

(3) The combined restraint per unit length may be determined from:

$$\left(1/K_A + 1/K_B\right) = \delta/F \qquad \text{... (A.5)}$$

where:

F is the load per unit length of the test specimen to produce a lateral deflection of $h/10$;

h is the overall depth of the specimen;

δ is the lateral displacement of the top flange in the direction of the load F.

(4) In interpreting the test results, the following parameters should be treated as variables:

- the number of fasteners per unit length of the specimen;
- the type of fasteners;
- the flexural stiffness of the beam, relative to its thickness;
- the flexural stiffness of the bottom flange of the sheeting, relative to its thickness;
- the positions of the fasteners in the flange of the sheeting;
- the distance from the fasteners to the centre of rotation of the beam;
- the overall depth of the purlin;
- the possible presence of insulation between the beam and the sheeting.

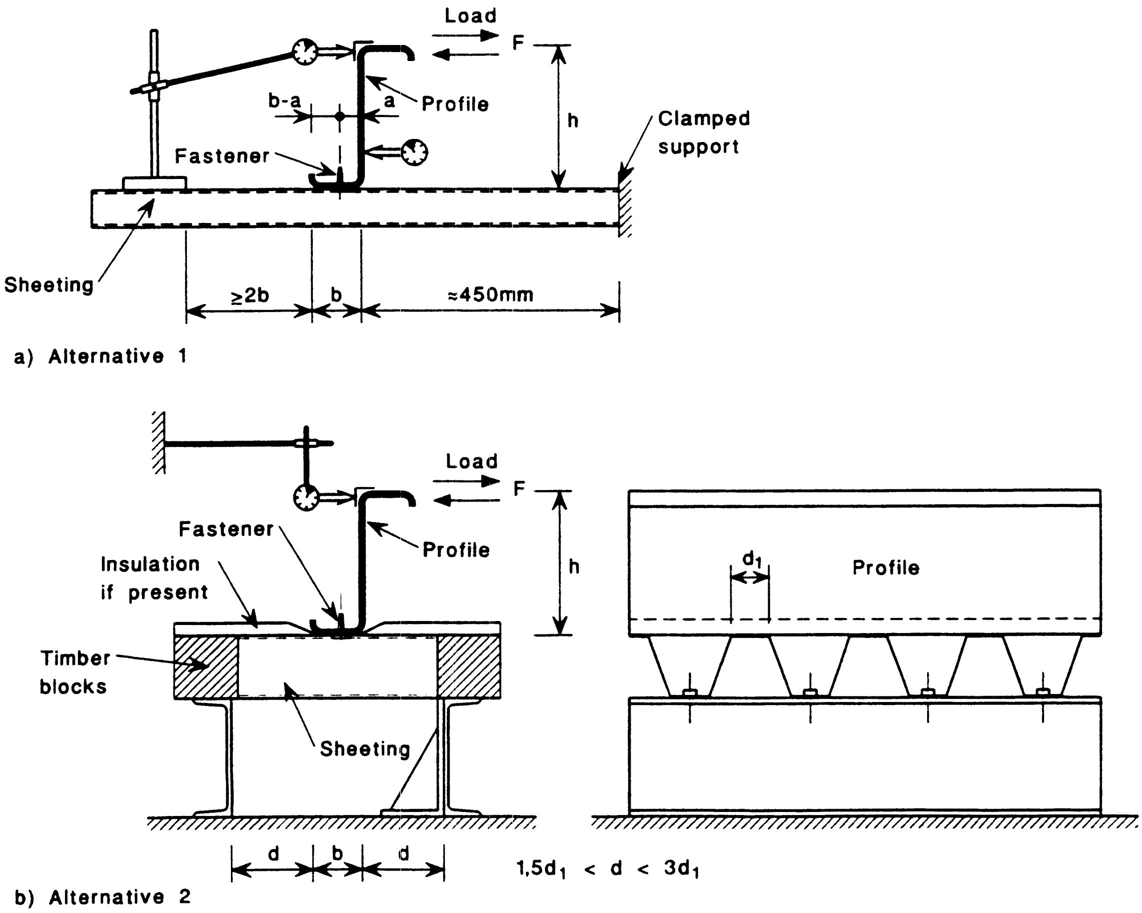


Figure A.11: Experimental determination of spring stiffnesses K_A and K_B

A.6 Evaluation of test results

A.6.1 General

(1) A specimen under test should be regarded as having failed if the applied test loads reach their maximum values, or if the gross deformations exceed specified limits.

(2) The gross deformations of members should generally satisfy:

$$\delta \leq L/50 \quad \text{... (A.6)}$$

$$\phi \leq 1/50 \quad \text{... (A.7)}$$

where:

δ is the maximum deflection of a beam of span L;

ϕ is the sway angle of a structure.

(3) In the testing of connections, or of components in which the examination of large deformations is necessary for accurate assessment (for example, in evaluating the moment-rotation characteristics of sleeves), no limit need be placed on the gross deformation during the test.

(4) An appropriate margin of safety should be available between a ductile failure mode and possible brittle failure modes. As brittle failure modes do not usually appear in large scale tests, additional detail tests should be carried out where necessary.

NOTE: This is often the case for connections.

A.6.2 Adjustment of test results

(1) Test results should be appropriately adjusted to allow for variations between the actual measured properties of the test specimens and their nominal values.

(2) The actual measured basic yield strength $f_{yb,obs}$ should not deviate by more than $\pm 25\%$ from the nominal basic yield strength f_{yb}.

(3) The actual measured material thickness t_{obs} should not exceed the design thickness t based on the nominal material thickness t_{nom} (see 3.1.3) by more than 12%.

(4) Adjustments should be made in respect of the actual measured values of the material thickness t_{obs} and the basic yield strength $f_{yb,obs}$ for all tests, except where the design expression that uses the test results also uses the actual measured value of the thickness or yield strength of the material, as appropriate.

(5) The adjusted value $R_{adj,i}$ of the test result for test i should be determined from the actual measured test result $R_{obs,i}$ using:

$$R_{adj,i} = R_{obs,i} / \mu_R \quad \text{... (A.8)}$$

in which μ_R is the resistance adjustment coefficient given by:

$$\mu_R = \left(\frac{f_{yb,obs}}{f_{yb}} \right)^{\alpha} \left(\frac{t_{obs}}{t} \right)^{\beta} \quad \text{... (A.9)}$$

(6) The exponent α for use in expression (A.9) should be obtained as follows:

- if $f_{yb,obs} \leq f_{yb}$: $\alpha = 0$
- if $f_{yb,obs} > f_{yb}$:
  - generally: $\alpha = 1$
  - for profiled sheets (or liner trays) in which compression elements have such large b_p/t ratios that local buckling is clearly the failure mode: $\alpha = 0{,}5$

(7) The exponent β for use in expression (A.9) should be obtained as follows:

- if $t_{obs} \leq t$: $\beta = 1$
- if $t_{obs} > t$:
  - for tests on profiled sheets or liner trays: $\beta = 2$
  - for tests on members, structures or portions of structures:
    - if $b_p/t \leq (b_p/t)_{lim}$: $\beta = 1$
    - if $b_p/t > 1{,}5(b_p/t)_{lim}$: $\beta = 2$
    - if $(b_p/t)_{lim} < b_p/t < 1{,}5(b_p/t)_{lim}$: obtain β by linear interpolation.

in which the limiting width-to thickness ratio $(b_p/t)_{lim}$ given by:

$$(b_p/t)_{lim} = 0{,}64\sqrt{\frac{E k_\sigma}{f_{yb}}} \times \sqrt{\frac{f_{yb}/\gamma_{M1}}{\sigma_{com,Ed}}} \cong 19{,}1\,\varepsilon\sqrt{k_\sigma} \times \sqrt{\frac{f_{yb}/\gamma_{M1}}{\sigma_{com,Ed}}} \qquad \text{... (A.10)}$$

where:

b_p	is	the notional flat width of a plane element;
k_σ	is	the relevant buckling factor from table 4.1 or 4.2;
$\sigma_{com,Ed}$	is	the largest calculated compressive stress in that element, when the resistance of the cross-section is reached.

A.6.3 Characteristic values

A.6.3.1 General

(1) Characteristic values may be determined statistically, provided that there are at least 4 test results.

NOTE: A larger number is generally preferable, particularly if the scatter is relatively wide.

(2) If the number of test results available is 3 or less, the method given in A.6.3.3 may be used.

(3) The characteristic minimum value should be determined using the following provisions. If the characteristic maximum value or the characteristic mean value is required, it should be determined by using appropriate adaptations of the provisions given for the characteristic minimum value.

(4) The characteristic value of a resistance R_k determined on the basis of at least 4 tests may be obtained from:

$$R_k = R_m - ks \qquad \text{... (A.11)}$$

where:

s is the standard deviation;

k is the appropriate coefficient from table A.2;

R_m is the mean value of the adjusted test results R_{adj}.

(5) The standard deviation s may be determined using:

$$s = \sqrt{\frac{\sum_{i=1}^{n}\left(R_{adj,i} - R_m\right)^2}{n-1}} \equiv \sqrt{\frac{\sum_{i=1}^{n}\left(R_{adj,i}\right)^2 - \frac{1}{n}\left(\sum_{i=1}^{n} R_{adj,i}\right)^2}{n-1}} \qquad \text{... (A.12)}$$

where:

$R_{adj,i}$ is the adjusted test result for test i;

n is the number of tests.

Table A.2: Values of the coefficient k

n	4	5	6	8	10	20	30	∞
k	2,63	2,33	2,18	2,00	1,92	1,76	1,73	1,64

A.6.3.2 Characteristic values for families of tests

(1) A series of tests carried out on a number of otherwise similar structures, portions of structures, members, sheets or other structural components, in which one or more parameters is varied, may be treated as a single family of tests, provided that they all have the same failure mode. The parameters that are varied may include cross-sectional dimensions, spans, thicknesses and material strengths.

(2) The characteristic resistances of the members of a family may be determined on the basis of a suitable design expression that relates the test results to all the relevant parameters. This design expression may either be based on the appropriate equations of structural mechanics, or determined on an empirical basis.

(3) The design expression should be modified to predict the mean measured resistance as accurately as practicable, by adjusting the coefficients to optimize the correlation.

NOTE: Information on this process is given in annex Z*) of ENV 1993-1-1.

(4) In order to calculate the standard deviation s each test result should first be normalized by dividing it by the corresponding value predicted by the design expression. If the design expression has been modified as specified in (3), the mean value of the normalized test results will be unity. The number of tests n should be taken as equal to the total number of tests in the family.

(5) For a family of at least four tests, the characteristic resistance R_k should then be obtained from expression (A.11) by taking R_m as equal to the value predicted by the design expression, and using the value of k from table A.2 corresponding to a value of n equal to the total number of tests in the family.

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*) This annex is in preparation.

A.6.3.3 Characteristic values based on a small number of tests

(1) If only one test is carried out, then the characteristic resistance R_k corresponding to this test should be obtained from the adjusted test result R_{adj} using:

$$R_k = 0,9\,\eta_k R_{adj} \qquad \text{... (A.13)}$$

in which η_k should be taken as follows, depending on the failure mode:

- yielding failure: $\eta_k = 0,9$;
- gross deformation: $\eta_k = 0,9$;
- local buckling: $\eta_k = 0,8$;
- overall instability: $\eta_k = 0,7$.

(2) For a family of two or three tests, provided that each adjusted test result $R_{adj,i}$ is within $\pm 10\%$ of the mean value R_m of the adjusted test results, the characteristic resistance R_k should be obtained using:

$$R_k = \eta_k R_m \qquad \text{... (A.14)}$$

(3) The characteristic values of stiffness properties (such as flexural or rotational stiffness) may be taken as the mean value of at least two tests, provided that each test result is within $\pm 10\%$ of the mean value.

A.6.4 Design values

(1) The design value of a resistance R_d should be derived from the corresponding characteristic value R_k determined by testing, using:

$$R_d = R_k / \gamma_M / \gamma_{sys} \qquad \text{... (A.15)}$$

where:

γ_M is the partial factor for resistance;

γ_{sys} is a partial factor for differences in behaviour under test conditions and service conditions.

(2) For a family of at least four tests, the value of γ_M may be determined using statistical methods.

NOTE: Information on an appropriate method is given in annex Z*) of ENV 1993-1-1.

(3) Alternatively γ_M may be taken as equal to the appropriate value of γ_M for design by calculation given in Section 2 or Section 8 of this Part 1.3.

(4) The appropriate value for γ_{sys} should be agreed between the client, the designer, the testing organisation and the competent authority.

(5) For sheeting and for other well defined standard testing procedures (including A.3.2.1 stub column tests, A.3.3 tension tests and A.3.4 bending tests) γ_{sys} may be taken as equal to 1,0.

(6) For other types of tests in which possible instability phenomena, or modes of behaviour, of structures or structural components might not be covered sufficiently by the tests, the value of γ_{sys} should be assessed taking into account the actual testing conditions, in order to achieve the necessary reliability.

A.6.5 Serviceability

(1) The provisions given in Section 7 should be satisfied.

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*) This annex is in preparation.

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