CEN/TC 250

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Secretariat: BSI

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

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

Eurocode 3 – Calcul des structures en acier - Partie 1-3: Règles générales – Règles supplémentaires pour les profiles et plaques à parois minces formés à froid

1st Draft prepared by the Project Team SC3.T3

See note from the Project Team in the following page

APRIL 2018

ICS:

CCMC will prepare and attach the official title page.

prEN 1993-1-3 – 1st draft

NOTE FROM THE PROJECT TEAM

1) The main modifications are highlighted in this 1st draft with respect to EN 1993-1-3: 2006.

2) Comments are expected on the changes of this 1st draft.

3) No comment on the current standard EN 1993-1-3: 2006 is espexted as it is supposed to have been done by the national standardization bodies during the systematic review.

4) Although the document prEN 1993-1-3 will be checked again by the Project Team, any editorial remark is welcome to improve the quality.

5) The project team CEN/TC250 SC3.T3 decided to incooperate the following amendments and modifications into the 1st draft of revised EN 1993-1-3. All changes compared with EN 1993-1-3 (2010) are highlighted as follows:

- Amendments prepared by the corresponding Working Group CEN/TC250 SC3.WG3 (confirmed by CEN/TC250 SC3), see doc. CEN/TC250 SC3.T3 N056 – highlighted in yellow
- Amendments prepared by the corresponding Working Group CEN/TC250 SC3.WG3 (to be confirmed by CEN/TC250 SC3), see doc. CEN/TC250 SC3.T3 N056 – highlighted in yellow (font colour: red)
- Comments of the systematic review (classified by 1 and 5), see doc. CEN/TC250 SC3.T3 N057 - highlighted in yellow
- Revised structure EN 1993-1-3 according to the common structure of revised EN Eurocode Parts (see CEN/TC250 N1250) - highlighted in green.
- Joint WG3/PT approach of reducing the number of NDPs (confirmed by SC3)
- language improvements in parts by the PT (blue font). This work will continue.
- Major modifications of the PT with respect to clarification and requirements acc. to the drafting principles CEN/TC250 N1250 (purple font) – see also doc. CEN/TC250 SC3.T3 N055

6) This concerns the following main technical amendments (see also doc. CEN/TC250 SC3.T3 N056):

- Materials (old Clause 3.1 – new Clause 5.1)
- Flange curling (old 5.4 – new 7.4)
- Plane elements with edge or intermediate stiffener (old 5.5.3– new 7.5.3)
- Elastic-plastic resistance of members in bending (old 6.1.4 – new 8.1.4)
- Buckling resistance of members in bending and compression (old 6.2.5 – new 8.2.5)
- Connections with mechanical fasteners (old 8.3 – new 10.3)
- Testing (old 9 – new 12, Annex A,)

- Beams restrained by sheeting and the use of sandwich panels for stabilization (old 10 – new 11)
- Restructuring of new Clause 11 – old Clause 10 - (incl. two new Subclauses 11.4 (sheeting with overlaps acc. to the EU mandate M515) and new Subclause 11.5 which has been introduced to summarize all design provisions related to "Lateral and torsional restraints provided by the sheeting". Subclause 11.5 contains paragraphs of the old clause 10.1 and 10.1.5 as well as new paragraphs (AMDs) for sandwich panels. The new Clause 11.5 aims to provide fundamental design rules which can be referred to whenever purlins have to be designed acc. to ECs)
- Old Annex D – new Annex C

7) **A detailed specification of all amendments and modifications in the 1st draft of the revised EN 1993-1-3 related to the WG3 amendments, the Systematic Review and the Task Specification of mandate M515 is given in documents N055 to N058 CEN TC250 SC3.T3, which are enclosed to the 1st draft of the revised EN 1993-1-3.**

Contents

European Foreword

This European Standard EN 1993-1-3, Eurocode 3: Design of steel structures: Part 1-3 General rules – Supplementary rules for cold-formed members and sheeting, has been prepared by Technical Committee CEN/TC250 « Structural Eurocodes », the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

According to the CEN-CENELEC Internal Regulations, the National Standard Organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Romania, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

National Annex for EN 1993-1-3

This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1993-1-3 should have a National Annex containing all Nationally Determined Parameters to be used for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-1-3 through the following Clauses:

- 4(3)P
- 4(4)
- 5.2.4(1)
- 10.3(3), Table 10.2
- 10.3(3), Table 10.3
- 10.3(3), Table 10.4
- 10.3(3), Table 10.5
- A.1(1), NOTE 1
- A.1(1), NOTE 2
- A.8.4(4)

~~5.1(3) Note 1 and Note 2~~

~~7.3(4)~~ ~~10.3(5)~~

~~10.4(5)~~ ~~10.5.1(4)~~

~~12(2)~~

11~~.1.1(1)~~

~~11.1.4.2(1)~~

~~E(1)~~

1 Scope

1.1 Scope of EN 1993

PT-comment: Text to be filled in later

1.2 Scope of EN 1993-1-3

(1) EN 1993-1-3 gives design requirements for cold-formed members and sheeting. It applies to cold-formed steel products made from coated or uncoated hot- or cold-rolled sheet or strip, that have been cold-formed by such processes as cold-rolled forming or press-braking. It is also applicable for the design of profiled steel sheeting for composite steel and concrete slabs at the construction stage, see EN 1994. The execution of steel structures made of cold-formed members and sheeting is covered in EN 1090-4.

NOTE: The rules in EN 1993-1-3 complement the rules in other parts of EN 1993-1.

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

(3) EN 1993-1-3 does not apply to cold-formed circular and rectangular structural hollow sections supplied to EN 10219, for which reference is made to EN 1993-1-1 and EN 1993-1-8.

(4) EN 1993-1-3 gives methods for design by calculation and for design assisted by testing. 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.

1.3 Assumptions

(1)P EN 1993 shall be used in conjunction with:

- EN 1990 “Basis of structural design”

- EN 1991 “Actions on structures”

- The parts of EN 1992 to EN 1999 where steel structures or steel components are referred to within those documents

- EN 1090 “Execution of steel structures and aluminium structures”

- ENs, EADs and ETAs for construction products relevant to steel structures

(2) The design rules of EN 1993-1-3 are only applicable when the execution is carried out according to EN 1090-4.

2 Normative references

2.1 General

(1) The following documents, in whole or in part, are normatively referenced in this document and are indispensable for its application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

2.2 General reference standards

EN 1090-1: *Execution of steel structures – Technical requirements, Part 1: Requirements for conformity assessment of structural components;*

EN 1090-2: *Execution of steel structures and aluminium structures, Part 2: Technical requirements for steel structures:*

EN 1090-4: *Execution of steel structures and aluminium structures, Part 4: Technical requirements for cold-formed structural steel elements and cold-formed structures of roof, ceiling, floor and wall applications;*

EN 1990: *Eurocode 0* - *Basis of structural design;*

EN 1993: *Eurocode 3 – Design of steel structures* - *Part 1-2, Part 1-4, Part 1-6, Part 1-7, Part 1-9 to Part 1-14;*

EN 1993-1-1: 2018, *Eurocode 3 – Design of steel structures – Part 1-1: General rules and rules for building;*

EN 1993-1-5: 2020, *Eurocode 3 – Design of steel structures – Part 1-5: Plated structural elements;*

EN 1993-1-8: 2018, *Eurocode 3 – Design of steel structures – Part 1-8: Design of joints;*

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

EN 14509-2: *Double skin metal faced insulating panels - Factory made products - Specifications - Part 2: Structural applications - Fixings and potential uses of stabilization of individual structural elements;*

EN 15512: *Steel static storage systems - Adjustable pallet racking systems - Principles for structural design*;

EN ISO 12944-2: *Paints and vanishes. Corrosion protection of steel structures by protective paint systems. Part 2: Classification of environments*

2.3 Structural steel reference standards

EN 508-1: *Roofing products from metal sheet - Specification for self-supporting products of steel, aluminium or stainless steel sheet - Part 1: Steel;*

EN 10025-1: *Hot-rolled products of structural steels - Part 1: General delivery conditions;*

EN 10025-2: *Hot-rolled products of structural steels - Part 2: Technical delivery conditions for non-alloy structural steels;*

EN 10025-3: *Hot-rolled products of structural steels - Part 3: Technical delivery conditions for normalized / normalized rolled weldable fine grain structural steels;*

EN 10025-4: *Hot-rolled products of structural steels - Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels;*

EN 10025-5: *Hot-rolled products of structural steels - Part 5: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance;*

EN 10088: *Stainless steels - Part 1 to Part 5;*

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

EN 10149-2: *Hot rolled flat products made of high yield strength steels for cold-forming - Part 2: Delivery conditions for thermomechanical rolled steels*;

EN 10149-3: *Hot rolled flat products made of high yield strength steels for cold-forming - Part 3: Delivery conditions for normalized and normalized rolled steels*;

EN 10204: *Metallic products. Types of inspection documents;*

EN 10268: *Cold-rolled flat products made of high yield strength micro-alloyed steels for cold-forming - General delivery conditions;*

EN 10346: *Continuously hot-dip coated steel flat products for cold-forming - Technical delivery conditions*

EN 14509-1: *Self-supporting double skin metal faced insulating panels - Factory made products - Specifications;*

EN 14782: *Self-supporting metal sheet for roofing, external cladding and internal lining. Product specification and requirements;*

EN ISO 6892-1: *Metallic materials - Tensile testing- Part 1: Method of test at room temperature;*

EN ISO 7438: *Metallic materials - Bend test;*

ISO 4997: *Cold reduced steel sheet of structural quality*;

2.4 Reference standards for fasteners

EN ISO 1478: *Tapping screws thread;*

EN ISO 1479: *Hexagon head tapping screws;*

EN ISO 2702: *Heat-treated steel tapping screws - Mechanical properties;*

EN ISO 7049: *Cross recessed pan head tapping screws;*

EN ISO 10684: *Fasteners – hot dip galvanized coatings*

3 Terms, definitions and symbols

3.1 Terms and definitions

PT-comment: Will be supplemented later

Supplementary to EN 1993-1-1, for the purposes of EN 1993-1-3, the following terms and definitions apply:

3.1.1
basic material
flat sheet steel material used for cold-forming sections and profiled sheeting

3.1.2
basic yield strength
tensile yield strength of the basic material

3.1.3
diaphragm action
structural behaviour involving in-plane shear in the sheeting

3.1.4
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

3.1.5
partial restraint
restriction of the lateral or rotational movement, or the torsional or warping deformation, of a member or element, that increases its buckling resistance in a similar way to a spring support, but to a lesser extent than a rigid support

3.1.6
relative slenderness
normalized non-dimensional slenderness ratio

3.1.7
restraint
restriction of the lateral or rotational movement, or the torsional or warping deformation, of a member or element, that increases its buckling resistance to the same extent as a rigid support

3.1.8
stressed-skin design
design method that allows for the contribution made by diaphragm action in the sheeting to the stiffness and strength of a structure

3.1.9
support
location at which a member is able to transfer forces or moments to a foundation, or to another member or other structural component

3.1.10
nominal thickness
target average thickness inclusive zinc and other metallic coating layers when present rolled and defined by the steel supplier (t_{nom} , not including organic coatings)

3.1.11
steel core thickness
nominal thickness minus zinc and other metallic coating layers (t_{cor})

3.1.12
design thickness
steel thickness used in design by calculation according to 3.3.3(6) and 5.2.4

3.1.13
fastener
connection element

3.1.14
fastening
fastener interaction with surrounding material

3.1.15
connection
set of fastenings used to transfer forces and moments between two or more members at the location at which two or more elements meet

3.1.16
joint
zone where two or more members are interconnected

3.1.17
component I
component of fastening that is facing the head of the fastener (the swage head in the case of blind rivets)

3.1.18
component II
second component of fastening (usually the supporting structure)

3.2 Symbols and abbreviations

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

PT-comment: All relevant symbols used in EN 1993-1-3 will be incorporated later

f_y yield strength

f_{ya} average yield strength, accounting for work hardening due to cold-forming

f_{yb} basic yield strength

t design thickness of steel material before cold-forming, exclusive of metal and organic coating

t_{nom} nominal sheet thickness after cold-forming inclusive of zinc and other metallic coating not including organic coating

t_{cor} nominal thickness minus zinc and other metallic coating

K spring stiffness for displacement

C spring stiffness for rotation

3.3 Terminology and conventions

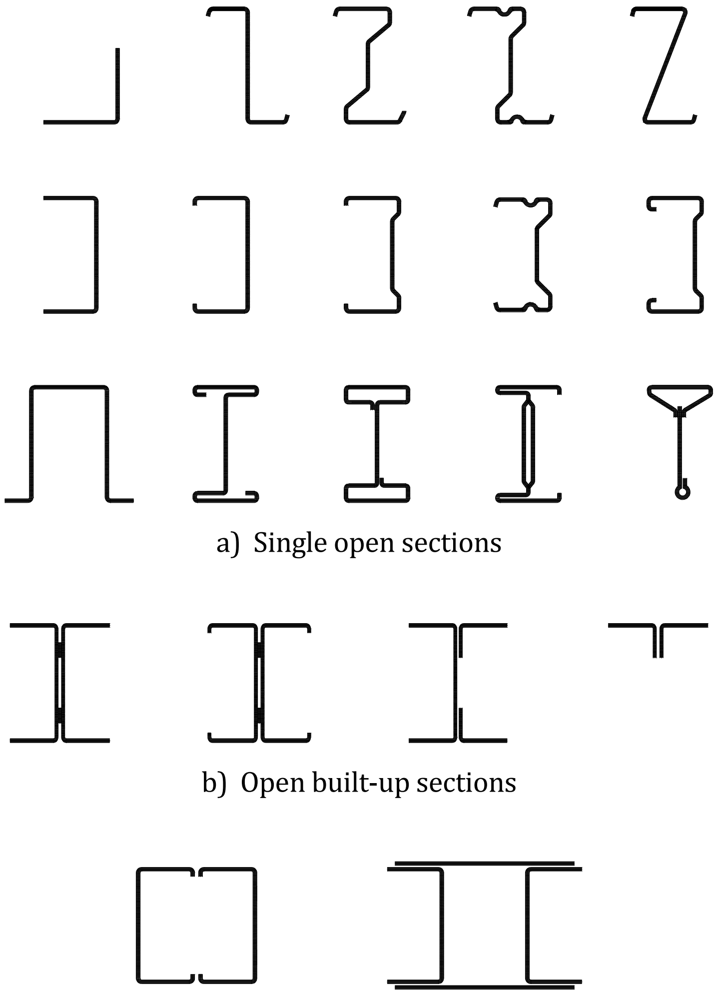
3.3.1 Cross-sectional shapes

(1) Cold-formed members and profiled sheets within the permitted tolerances have a constant nominal thickness over their entire length and may have either a uniform cross section or a tapering cross section along their length.

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

(3) Typical forms of sections for cold-formed members are shown in Figure 3.1.

~~NOTE: The calculation methods of this Part 1-3 of EN 1993 does not cover all the cases shown in figures 1.1-1.2.~~



a) Single open sections

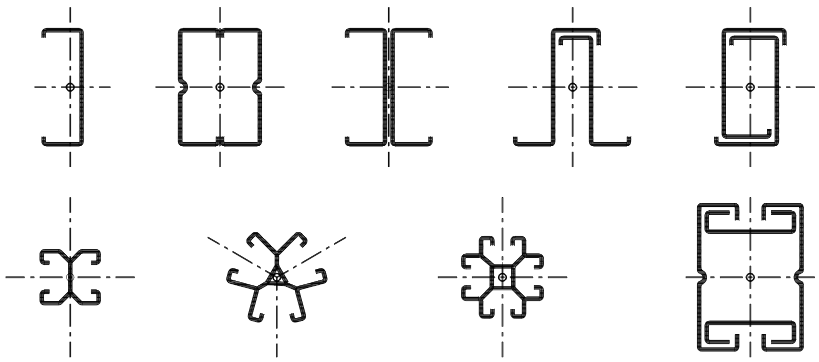
b) Open built-up sections

c) Closed built-up sections

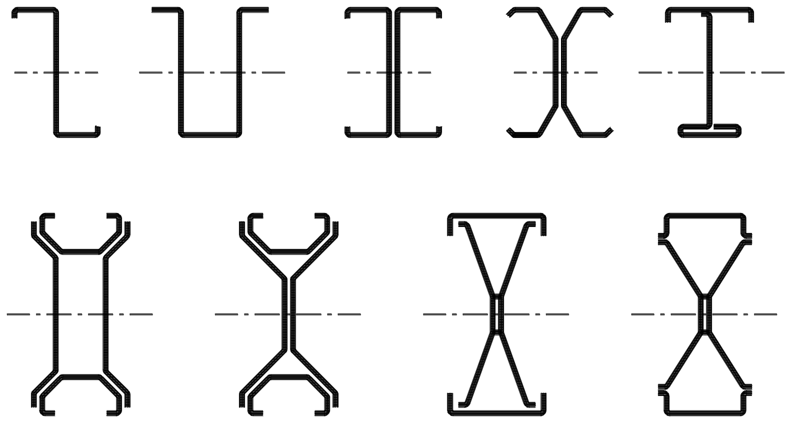
Figure 3.1 - Typical forms of sections for cold-formed members

(4) Examples of cross-sections for cold-formed members and sheeting are illustrated in Figure 3.2.

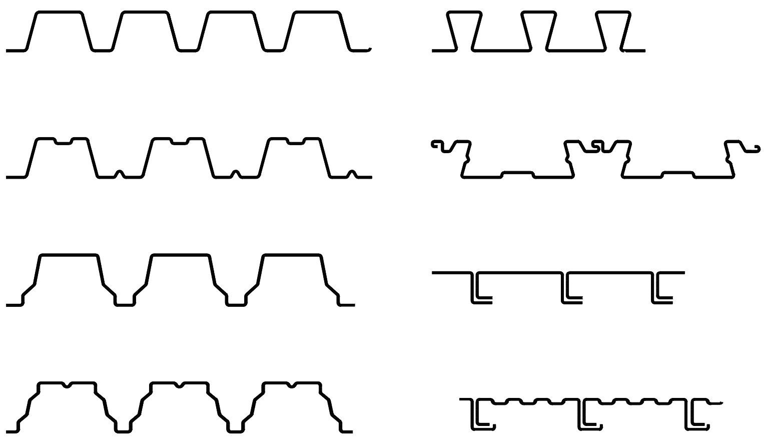
NOTE The design rules **of** EN 1993-1-3 relate to the main axis properties, which are defined by the main axes y - y and z - z for symmetrical sections and u - u and v - v for unsymmetrical sections as e.g. angles and Z-sections. In some cases the bending axis is imposed by connected structural elements whether the cross-section is symmetric or not.



a) Compression members and tension members



b) Beams and other members subject to bending



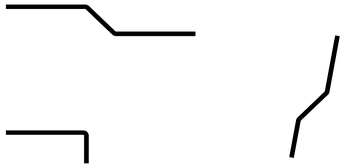
c) Profiled sheeting and liner trays

Figure 3.2 - Examples of cold-formed members and profiled sheets

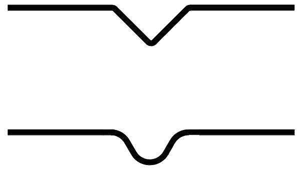
(5) Cross-sections of cold-formed members and sheeting may either be unstiffened or incorporate longitudinal stiffeners in their webs or flanges, or in both.

3.3.2 Stiffener shape

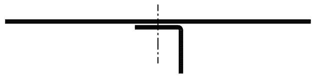
(1) Typical stiffener configurations for cold-formed members and sheeting are shown in Figure 3.3.



a) Folds and bends



b) Folded groove and curved groove



c) Bolted angle stiffener

Figure 3.3 - Typical forms of stiffeners for cold-formed members and sheeting

(2) Longitudinal stiffeners may either be edge stiffeners or intermediate stiffeners.

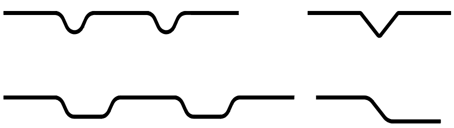
(3) Typical edge stiffeners are shown in Figure 3.4.

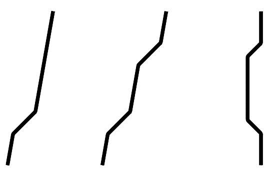
a) Single-fold edge stiffeners

b) Double-fold edge stiffeners

Figure 3.4 - Typical edge stiffeners

(4) Typical intermediate longitudinal stiffeners are illustrated in Figure 3.5.





b) Intermediate web stiffeners

a) Intermediate flange stiffeners

Figure 3.5 - Typical intermediate longitudinal stiffeners

3.3.3 Cross-sectional dimensions

(1) Overall dimensions of cold-formed members and sheeting, including the overall width *b*, the overall height *h*, the internal bend radius *r* and other dimensions denoted by symbols without subscripts, such as *a*, *c* or *d*, are measured to the face of the material, unless stated otherwise, as illustrated in Figure 3.6.

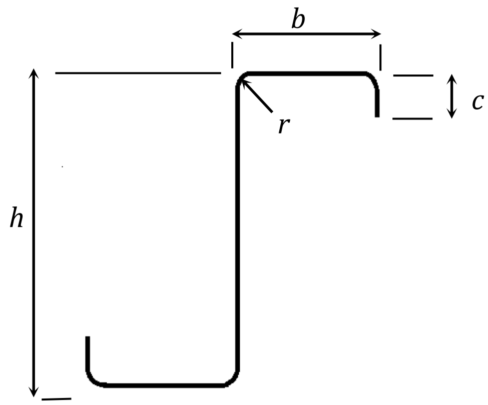


Figure 3.6 - Dimensions of typical cross-section

(2) Unless stated otherwise, the other cross-sectional dimensions of cold-formed 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 or the midpoint of the corner.

(3) In the case of sloping elements, such as webs of trapezoidal sheets, the slant height *s* is measured parallel to the slope. The slope is straight line between intersection points of flanges and web.

(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.

(6) The thickness *t* is a steel design thickness as specified in Clause 5.2.4(3), if not otherwise stated.

3.3.4 Convention for member axes

(1) In general the conventions for members is as used in EN 1993-1-1, see Figure 3.7.

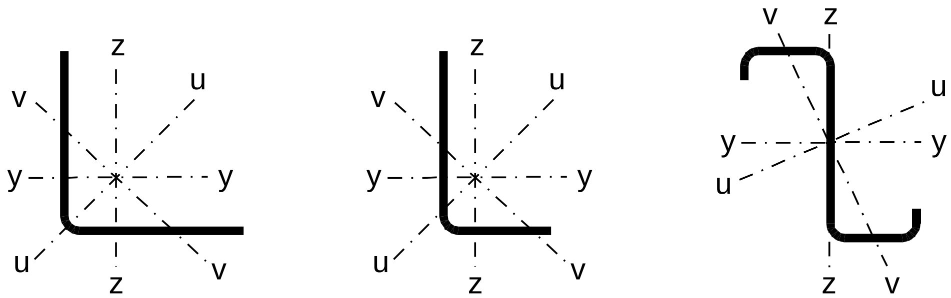


Figure 3.7 - Axis convention

(2) For profiled sheeting and liner trays the following axis convention is used:

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

4 Basis of design

(1) The design of cold-formed members and sheeting shall be in accordance with the general rules given in EN 1990 and EN 1993-1-1. For a general approach based on FE-methods (or others) see EN 1993-1-14.

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

(3)P In ultimate limit state calculations the partial factor γ_M shall be taken as follows:

- when calculating the resistance of cross-sections against yielding and cross-sectional instability including local and distortional buckling: γ_{M0}
- when calculating the resistance of members and sheeting against global buckling: γ_{M1}
- when calculating the resistance of net sections at fastener holes and connections: γ_{M2}

NOTE: The partial factors γ_{Mi} for buildings are given below unless the National Annex gives different values for use in a country:

$\gamma_{M0} = 1{,}00$;

$\gamma_{M1} = 1{,}00$;

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

~~(4) For values of γ_M for resistance of connections, see Section 10.~~

(4)P In serviceability limit state calculations the partial factor $\gamma_{M,ser}$ shall be used.

NOTE: The partial factors $\gamma_{M,ser}$ for buildings are given below unless the National Annex gives different values for use in a country:

$\gamma_{M,ser} = 1{,}00$.

(5) In the design of structures a distinction shall be made between various ‘structural classes’, based on the level of contribution of cold-formed members, sheeting or sandwich panels to the strength and stability of the overall structure or individual structural elements. These structural classes are associated with different requirements in the applicable product and execution standards for cold-formed members, sheeting and sandwich panels, and shall be determined as follows:

- Applications in which cold-formed members, sheeting or sandwich panels are designed to contribute to the overall strength and stability of a structure shall be classified as **Structural Class I**;
- Applications in which cold-formed members, sheeting or sandwich panels are designed to contribute to the strength and stability of individual structural elements shall be classified as **Structural Class II**;

  Sheeting or sandwich panels providing stabilizing support to purlins designed as part of the bracing system of the main structure (e.g. compression members, struts, tie beams) may be classified as Structural Class II.
- Applications in which cold-formed members, sheeting or sandwich panels are used as an element that only transfers loads to the structure shall be classified as **Structural Class III**. Sheeting and sandwich panels in Structural Class III can be further differentiated into \`structural´ or \`non-structural´ applications.

NOTE: Permitted applications of non-structural cold-formed sheeting or non-structural sandwich panels in Structural Class III can be set by the National Annex for use in a country.

EN 1090-1 and EN 1090-4 cover the requirements for execution of structural sheeting and cold-formed members. EN 14782 covers non-structural cold-formed sheeting in Structural Class III. EN 14509-1 covers non-structural sandwich panels in Structural Class III and EN 14509-2 covers structural sandwich panels in Structural Class II.

(6) The classification into Structural Classes is particularly relevant for sheeting and sandwich panels to establish their relation with the overall structure as well as with their supporting members.

NOTE: Structural Class I: The designer of the structures assumes bracing of the structure by the sheeting.

Structural Class II: The designer of the members directly supporting the sheeting or sandwich panels assumes that the latter provide restraint with regard to global buckling or bending in the plane of the sheeting or sandwich panel.

Structural Class III: The designer of the members directly supporting the sheeting or sandwich panels assumes that the latter provides restraint with regard to global buckling or bending in the plane of the sheeting or sandwich panel.

It is the responsibility of the designer of the members supporting sheeting and sandwich panels to establish adequate communication with the the designer of the sheeting or sandwich panels with respect to the assumed structural class and the corresponding implications according to the product standard.

(7) Additional requirements in Structural Classes I and II are given in EN 1090-4, e.g. with respect to:

- Communicating the structural class on layout drawings as well as clearly posting it on the actual structure
- Providing a minimum number of fastenings between sheeting or sandwich panels and the supporting member, as well as between the different sheets or sandwich panels.
- Specifying the Structural Class in the design brief as well as in the operations and maintenance manual

5 Materials

5.1 General

(1) All steels used for cold-formed members and profiled sheeting should be suitable for cold-forming and, if relevant, for welding. Steels used for members and sheets to be galvanized should also be suitable for galvanizing.

Steels specified in Tables 5.1a and 5.1b with properties and chemical composition in compliance with the relevant standards fulfil these requirements.

For other steels the suitability for cold-forming shall be demonstrated by a bend test in accordance with EN ISO 7438 or by an equivalent test.

(2) The nominal values of material properties given in this Clause should be adopted as characteristic values in design calculations.

(3) EN 1993-1-3 covers the design of cold-formed members and profiled sheets fabricated from steel material conforming to the requirements given in Clause 5.2.1.

Table 5.1a - Nominal values of basic yield strength f_{yb} and ultimate tensile strength f_u

Type of steel	Standard	Grade	f_{yb} [N/mm²]	f_u [N/mm²]
Hot rolled products of non-alloy structural steels. Part 2: Technical delivery conditions for non alloy structural steels	EN 10025-2	S 235 S 275 S 355	235 275 355	360 430 510
Hot-rolled products of structural steels. Part 3: Technical delivery conditions for normalized/ normalized rolled weldable fine grain structural steels	EN 10025-3	S 275 N S 355 N S 420 N S 460 N S 275 NL S 355 NL S 420 NL S 460 NL	275 355 420 460 275 355 420 460	370 470 520 550 370 470 520 550
Hot-rolled products of structural steels. Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels	EN 10025-4	S 275 M S 355 M S 420 M S 460 M S 275 ML S 355 ML S 420 ML S 460 ML	275 355 420 460 275 355 420 460	360 450 500 530 360 450 500 530

~~**NOTE 1:** For steel strip less than 3 mm thick conforming to EN 10025, if the width of the original strip is greater than or equal to 600 mm, the characteristic values may be given in the National Annex. Values equal to 0,9 times those given in Table 3.1a are recommended.~~

~~**NOTE 2:** For other steel materials and products see the National Annex. Examples for steel grades that may conform to the requirements of this standard are given in Table 3.1b.~~

Table 5.1b - Nominal values of basic yield strength f_{yb} and ultimate tensile strength f_u

Type of steel	Standard	Grade	f_{yb} [N/mm²]	f_u [N/mm²]
Cold reduced steel sheet of structural quality	ISO 4997	CR 220	220	300
		CR 250	250	330
		CR 320	320	400
Hot-rolled flat products made of high yield strength steels for cold-forming - Part 2: Delivery conditions for thermomechanically rolled steels	EN 10149-2	S 315 MC	315	390
		S 355 MC	355	430
		S 420 MC	420	480
		S 460 MC	460	520
		S 500 MC	500	550
		S 550 MC	550	600
		S 600 MC	600	650
		S 650 MC	650 [b]	700
		S 700 MC	700 [b]	750
Hot-rolled flat products made of high yield strength steels for cold-forming – Part 3: Delivery conditions for normalized or normalized rolled steels	EN 10149-3	S 260 NC	240 [a]	350 [a]
		S 315 NC	295 [a]	410 [a]
		S 355 NC	355 [a]	450 [a]
		S 420 NC	400 [a]	510 [a]
Cold-rolled flat products made of high yield strength micro-alloyed steels for cold-forming	EN 10268	HC240LA	240	340
		HC280LA	280	370
		HC320LA	320	400
		HC360LA	360	430
		HC400LA	400	460
Continuously hot-dip coated steel flat products for construction	EN 10346	S220GD+Z, +ZF, +ZA, +ZM, +AZ	220	300
		S250GD+ Z, +ZF, +ZA, +ZM, +AZ, +AS	250	330
		S280GD+ Z, +ZF, +ZA, +ZM, +AZ, +AS	280	360
		S320GD+ Z, +ZF, +ZA, +ZM, +AZ, +AS	320	390
		S350GD+ Z, +ZF, +ZA, +ZM, +AZ, +AS	350	420
		S390GD+Z, +ZF, +ZA, +ZM, +AZ	390	460
		S420GD+ Z, +ZF, +ZA, +ZM, +AZ	420	480
		S450GD+Z, +ZF, +ZA, +ZM, +AZ	450	510
Continuously hot-dip coated steel flat products of steel with high proof stress for cold-forming	EN 10346	HX260LAD+Z, +ZF, +ZA, +ZM, +AZ, +AS	240 [a]	330 [a]
		HX300LAD+Z, +ZF, +ZA, +ZM, +AZ, +AS	280 [a]	360 [a]
		HX340LAD+Z, +ZF, +ZA, +ZM, +AZ, +AS	320 [a]	390 [a]
		HX380LAD+Z, +ZF, +ZA, +ZM, +AZ, +AS	360 [a]	420 [a]
		HX420LAD+Z, +ZF, +ZA, +ZM, +AZ, +AS	400 [a]	450 [a]
		HX460LAD+Z, +ZF, +ZA, +ZM, +AZ, +AS	435 [a]	475 [a]
		HX500LAD+Z, +ZF, +ZA, +ZM, +AZ, +AS	470 [a]	500 [a c]
Continuously hot-dip coated steel flat products of low carbon steels for cold-forming	EN 10346	DX51D+Z, +ZF, +ZA, +ZM, +AZ, +AS	120 [a c]	250 [a c]
		DX52D+Z, +ZF, +ZA, +ZM, +AZ, +AS	120 [a c]	250 [a c]
		DX53D+Z, +ZF, +ZA, +ZM, +AZ, +AS	120 [a c]	250 [a c]

[a] The nominal values given in the respective product standard correspond to the transverse direction of the strip or sheet. The values given in the Table above correspond to tension in the longitudinal (rolling) direction.
[b] For thicknesses > 8 mm the given yield stress shall be reduced by 20 N/mm².
[c] The product standard does not specify minimum values of the yield stress and ultimate tensile strength. For all steel grades a minimum yield stress of 120 N/mm² and a minimum ultimate tensile strength of 250 N/mm² may be assumed.

NOTE: The materials in Table 5.1a conform to harmonized product standards, while the materials in Table 5.1b conform to EN or ISO product standards. EN1993-1-3 is applicable to the materials listed in both Tables.

5.2 Structural steel

5.2.1 Properties of base material

(1) The nominal values of the basic yield stress f_{yb} or ultimate tensile strength f_u should be obtained:

a) by adopting the values $f_{yb} = R_{eh}$ or $f_{yb} = R_{p0,2}$ and $f_u = R_m$ directly from product standards if they apply to the longitudinal direction or

b) by using the values given in Table 5.1a and 5.1b or

c) from appropriate tests.

(2) Where the characteristic values are determined from tests, such tests should be carried out in accordance with EN ISO 6892-1. The number of test coupons should be at least 5 and should be taken in the following way:

1. Coils:

a) Coils from the same origin (one pot of melted steel): at least one coupon per coil taken from 30% of the coils;

b) Coils from different origin: at least one coupon per coil;

2. Strips:

a) at least one coupon per 2000 kg of steel of the same origin;

b) at least one coupon for each different origin.

The coupons should be taken at random from the concerned lot of steel and the orientation should be in the longitudinal direction of the structural element. The characteristic values should be determined on the basis of a statistical evaluation in accordance with EN 1990, Annex D.

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

NOTE: Information wether the materials given in Table 5.1a and Table 5.1b can be used in a global plastic analysis can be set by the National Annex for use in a country.

(4) The material properties at elevated temperatures are given in EN 1993-1-2.

5.2.2 Material properties of cold-formed sections and sheeting

Comment PT: Restructuring of 5.2.2 for clarification

(1) Where the yield strength is specified using the symbol f_y the average yield strength f_{ya} may be used if (5) to (8) apply. In other cases the basic yield strength f_{yb} should be used. ~~see (4). Where the yield strength is specified using the symbol f_{yb} the basic yield strength f_{yb} should be used.~~

(2) The average yield strength f_{ya} of a cross-section due to cold-forming may be determined from the results of full-scale tests.

(3) Alternatively the average yield strength f_{ya} may be determined as follows:

$$f_{ya} = f_{yb} + (f_u - f_{yb})\frac{knt^2}{A} \qquad \text{but} \qquad f_{ya} \leq \frac{(f_u + f_{yb})}{2} \tag{5.1}$$

where:

A is the area of the gross cross-section;

k is a coefficient which depends on the forming process as follows:

- $k = 7$ for roll-forming;
- $k = 5$ for other forming methods;

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 design ~~core~~ thickness of the steel material as specified in 5.2.4(3).

(4) The basic yield strength f_{yb} may be taken into account as follows:

- when determining the area A_{eff} of the effective cold-formed sections due to local buckling and distortional buckling, see Clause 5.5;
- where the yield strength is specified using the symbol f_{yb}.

~~- in axially loaded members in which the effective cross-sectional area Aeff equals the gross area A_g in determining Aeff the yield strength f_y should be taken as f_{yb}.~~

(5) The average yield strength f_{ya} due to cold-forming may be utilised in determining:

- the resistance of a cross-section in uniform tension
- the resistance of the cross-section in uniform compression according to Clause 8.1.3
- the resistance of the cross-section with fully effective flanges in bending according to Clause 8.1.4
- and the global buckling resistance of an axially loaded compression member according to 8.2.2;

(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 webs or flanges. Formula (5.1) may then be used to obtain values of the increased yield strength $f_{y,i}$ separately for each nominal plane element i *(i=1...m)*, provided that:

$$\frac{\sum_{i=1}^{m} A_i f_{y,i}}{\sum_{i=1}^{m} A_i} \leq f_{ya} \tag{5.2}$$

where:

A_i is the area of the gross cross-section of nominal plane element *i (i=1...m)*,

and when calculating the increased yield strength $f_{y,i}$ using Formula (5.1) half of the bends adjacent to element *i* may be considered to be part of the area A_i.

(7) The increase in yield strength due to cold-forming should not be taken into account in members which have been subjected to a heat treatment after forming consisting of temperatures above 580°C for more than one hour.

NOTE: For further information see EN 1090-2.

(8) Special attention should be paid to the fact that some heat treatments (particularly annealing) might reduce the basic yield strength f_{yb}.

(9) For welding in cold-formed areas see also EN 1993-1-8.

5.2.3 Fracture toughness

(1) See EN 1993-1-1 and EN 1993-1-10.

5.2.4 Thickness and thickness tolerances

(1) The provisions for design by calculation given in EN 1993-1-3 are applicable to members and sheeting with $t_{cor} \geq 0{,}35$ mm.

(2) Thinner material may also be used, provided that the load carrying resistance is determined by design assisted by testing.

(3) See EN 1090-4 for minimum nominal steel thicknesses for different applications.

(4) A design thickness t based on the steel core thickness t_{cor} shall be used (rather than the nominal thickness t_{nom}) and should be determined as follows:

$$t = t_{cor} \qquad \text{if} \qquad tol \leq 5\% \tag{5.3}$$

$$t = t_{cor}\frac{100 - tol}{95} \qquad \text{if} \qquad tol > 5\% \tag{5.4}$$

where:

$$t_{cor} = t_{nom} - t_{\text{metallic coatings}} \tag{5.5}$$

tol is the minus tolerance of the thickness in %, as specified by the product standard or the production specification document;

t_{nom} is the nominal sheet thickness of the original sheet, inclusive of zinc and other metallic coatings, but not including any organic coatings;

t_{cor} is the nominal thickness minus the thickness of zinc and other metallic coatings;

$t_{\text{metallic coatings}}$ is the thickness of the metallic coatings.

For Z 275, the combined thickness of the coating on both sides of the plate may be taken as $t_{\text{metallic coating}} = 0{,}04$ mm.

(4) For continuously hot-dip metal coated members and sheeting supplied with negative tolerances less or equal to the “special tolerances (S)” given in EN 10143, the design thickness according to (5.3) may be used. If the negative tolerance is beyond "special tolerance (S)" given in EN 10143 then the design thickness according to (5.4) may be used.

(5) t_{nom} is the nominal sheet thickness after cold-forming. It may be taken as the value t_{nom} of the original sheet, if the calculative cross-sectional areas before and after cold-forming do not differ more than 2%; otherwise the notional dimensions should be changed.

5.3 Connecting devices

5.3.1 Mechanical fasteners

5.3.1.1 General

The definitions for mechanical fasteners, fastenings, connections and joints for the purposes of cold-formed steel structures are specified in Clause 3.1.

5.3.1.2 Bolt assemblies

(1) Bolts, nuts and washers should conform to the requirements given in EN 1993-1-8.

5.3.1.3 Other types of mechanical fastener

(1) Other types of mechanical fasteners such as

- self-tapping screws as thread forming self-tapping screws, thread cutting self-tapping screws or self-drilling self-tapping screws,
- cartridge-fired pins,
- blind rivets
- clinching
- adhesive bonding
- self-piercing gas-fired pins

may be used if they comply with the relevant European Product Standard or ETA.

(2) The characteristic shear resistance $F_{v,Rk}$ and the characteristic minimum tension resistance $F_{t,Rk}$ of the mechanical fasteners may be taken from the EN Product Standard ~~or ETAG~~ or ETA.

5.3.2 Welding consumables

(1) Welding consumables should conform to the requirements given in EN 1993-1-8.

6 Durability

(1) For basic durability requirements see EN 1993-1-1.

(2) EN 1090-2 lists the factors affecting execution that shall be specified during design.

(3) Special attention should be given to galvanic corrosion in cases where different metals are in contact with each other.

(4) For the corrosion resistance of products, see EN 1990-4. For the corrosion resistance of fasteners and the environmental class following EN ISO 12944-2, see Annex B. For hot dip galvanized fasteners, see EN ISO 10684.

7 Structural analysis

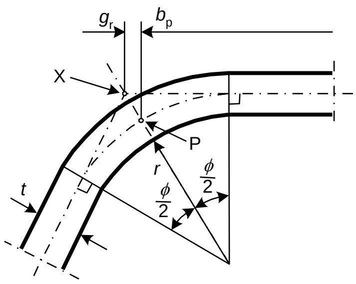
7.1 Influence of rounded corners

(1) In cross-sections with rounded corners, the notional flat widths b_p of the plane elements should be measured from the midpoints of the adjacent corner elements as indicated in Table 7.1.

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

Table 7.1 - Notional widths of plane cross section parts b_p allowing for corner radii

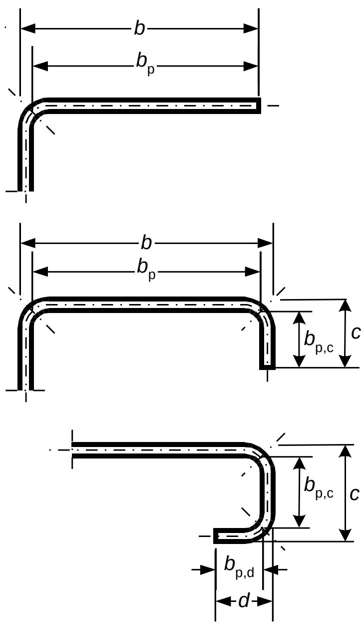
(a) midpoint of corner or bend



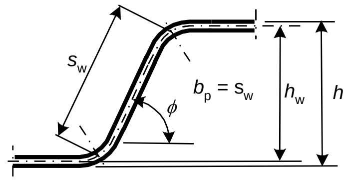
where:

- X is the intersection of midlines
- P is the midpoint of the corner
- $r_m = r + t/2$
- $g_r = r_m \left(tan\frac{\varphi}{2} - sin\frac{\varphi}{2} \right)$

(b) notional flat width b_p of plane parts of flanges

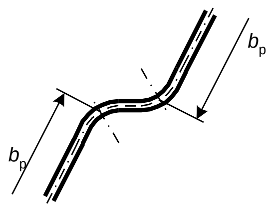


(c) notional flat width b_p for a web

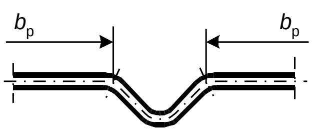


(b_p = slant height s_w)

(d) notional flat width b_p of plane parts adjacent to web stiffener



(e) notional flat width b_p of flat parts adjacent to flange stiffener



(3) Unless more appropriate methods are used to determine the section properties the following approximate procedure may be used. The influence of rounded corners on cross-section resistance may be neglected if the internal radius $r \leq 5\ t$ and $r \leq 0{,}10\ b_p$ and the cross-section may be assumed to consist of plane elements with sharp corners (according to Figure 7.1, note b_p for all flat plane elements, inclusive plane elements in tension). For cross-section stiffness properties the influence of rounded corners should always be taken into account.

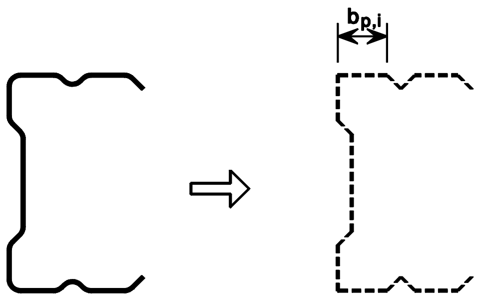


Figure 7.1 - Approximate allowance for rounded corners

(4) The influence of rounded corners on section properties may be taken into account by reducing the properties calculated for an otherwise similar cross-section with sharp corners, see Figure 7.1, using the following approximations:

$$A \sim A_{sh}(1-\delta) \tag{7.1}$$

$$I \sim I_{sh}(1-2\delta) \tag{7.2}$$

$$I_{w} \sim I_{w,sh}(1-4\delta) \tag{7.3}$$

with:

$$\delta = 0{,}43 \frac{\sum_{j=1}^{n} r_j \frac{\phi_j}{90^\circ}}{\sum_{i=1}^{m} b_{p,i}} \tag{7.4}$$

where:

A	is the area of the gross cross-section;
A_{sh}	is the value of A 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	is the second moment of area of the gross cross-section;
I_{sh}	is the value of I 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;
ϕ	is the angle between two plane elements;
m	is the number of plane elements;
n	is the number of curved elements;
r_{j}	is the internal radius of curved element j.

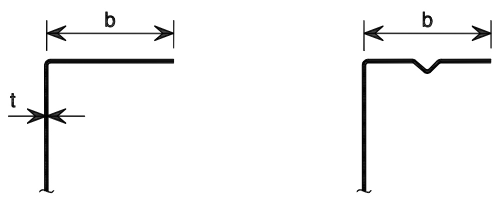
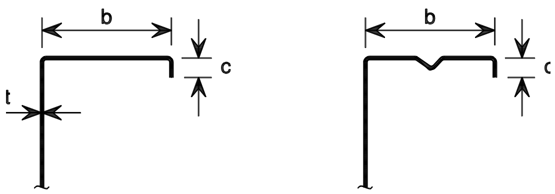
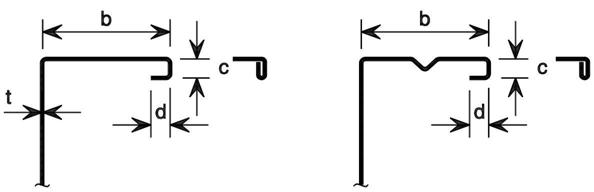
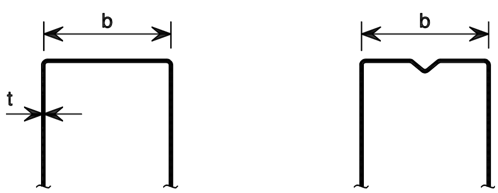
(5) The reductions given by Formulae (7.1) to (7.3) may also be applied in calculating the section properties A_{eff}, $I_{y,eff}$, $I_{z,eff}$ and $I_{w,eff}$ of the effective cross-section, provided that the notional flat widths of the plane elements are measured to the points of intersection of their midlines.

(6) Where the internal radius $r > 0{,}04\, t\, E / f_{y}$, then the resistance of the cross-section should be determined by tests.

7.2 Geometrical proportions

(1) The design rules according to EN 1993-1-3 should not be applied to cross-sections outside the range of width-to-thickness ratios b/t, h/t, c/t and d/t given in Table 7.2. These limits represent the field for which sufficient experience and verification by testing is already available. 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 and/or by calculations, where the results are confirmed by an appropriate number of tests.

Table 7.2 - Maximum width-to-thickness ratios

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

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

$$0,2 \leq c/b \leq 0,6 \quad (7.5)$$

$$0,1 \leq d/b \leq 0,3 \quad (7.6)$$

in which the dimensions b, c and d are as indicated in Table 7.2.

If $c/b <$ 0,2 or $d/b <$ 0,1 the lip should be ignored (c = 0 or d = 0).

NOTE 1: Where section properties of the effective cross-section are determined by testing and by calculations, these limits do not apply.

NOTE 2: For FE-methods see EN 1993-1-14.

(3) Where the lip is not perpendicular to the flange, see Figure 7.2, the equivalent size c_{eq} of the lip should be calculated as follows:

$$c_{eq} = \frac{c}{(cos\varphi)^{1/3}} \qquad \text{if } d=0 \qquad \text{(single-fold lip)} \qquad (7.7)$$

$$c_{eq} = c\left[\frac{c + 4d\,cos\varphi}{cos\varphi\,(c + d\,cos\varphi)}\right]^{1/3} \qquad \text{if } d>0 \qquad \text{(double-fold lip)} \qquad (7.8)$$

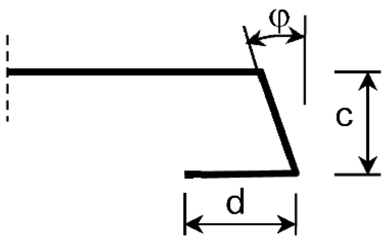


Figure 7.2 - Lips not perpendicular to the flange

7.3 Structural modelling for analysis

(1) Unless more appropriate models are used according to EN 1993-1-5 the elements of a cross-section may be modelled for analysis as indicated in Table 7.3.

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

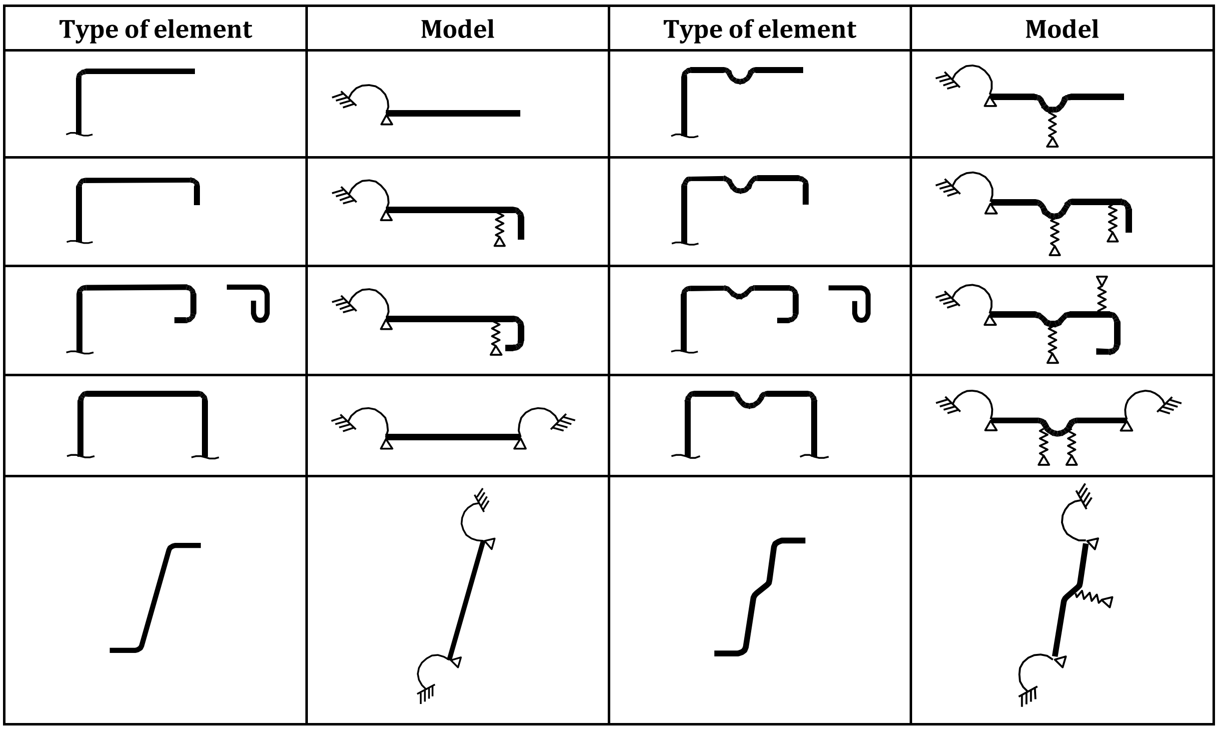
(3) Imperfections related to flexural buckling and torsional flexural buckling should be taken from Table 7.1 of EN 1993-1-1: 2018.

NOTE: See also Clause 7.3.5 of EN 1993-1-1: 2018.

(4) For imperfections related to lateral torsional buckling an initial bow imperfections e_0 of the weak axis of the profile may be assumed without taking account at the same time an initial twist. See EN 1993-1-1: 2018, 7.3.3.2, for magnitude of imperfections.

~~**NOTE:** The magnitude of the imperfection may be taken from the National Annex. The values e_0/L = 1/600 for elastic analysis and e_0/L = 1/500 for plastic analysis are recommended for sections assigned to LTB buckling curve a taken from EN 1993-1-1, section 6.3.2.2.~~

Table 7.3 - Modelling of elements of a cross-section



(5) Unless otherwise justified, the edges designed for lateral overlapping of sheeting should not be considered in the determination of the resistance and the stiffness of the sheeting. The edges designed for lateral overlapping are shown as dotted lines in Figure 7.3.

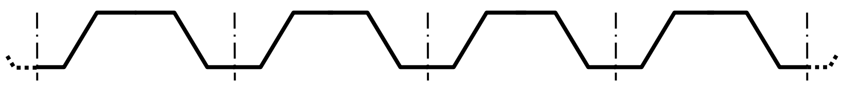


Figure 7.3 - Modelling of sheeting for the determination of resistance and stiffness

7.4 Flange curling

(1) The effect on the loadbearing resistance of curling (i.e. inward curvature towards the neutral plane) of a very wide flange in a profile subjected to flexure, or of a flange in an arched profile subjected to flexure in which the concave side is in compression, should be taken into account unless such curling is less than 5% of the depth of the profile cross-section. If curling is larger, then the reduction in loadbearing resistance, for instance due to a decrease in the length of the lever arm for parts of the wide flanges, and to the possible effect of the bending of the webs should be taken into account.

For profiled sheeting and members the effect of flange curling on the resistance may be ignored, provided that $b/t \leq 250\, h/b$ (see Figure 7.4).

NOTE: For liner trays the effect of flange curling is taken into account in 11.2.2.2.

(2) Calculation of the curling may be carried out according to Formulae (7.9) and (7.10). The Formulae apply to both compression and tensile flanges, both with and without stiffeners, but without closely spaced transversal stiffeners at flanges.

For a profile which is straight prior to application of loading (see Figure 7.4).

$$u = 2\frac{\sigma_a^2}{E^2}\frac{b_s^4}{t^2 z} \tag{7.9}$$

For an arched beam:

$$u = 2\frac{\sigma_a}{E}\frac{b_s^4}{t^2 r} \tag{7.10}$$

where:

u is the bending of the flange towards the neutral axis (curling), see Figure 7.4;

b_s is one half the distance between webs in box and hat sections, or the width of the portion of flange projecting from the web, see Figure 7.4;

t is the flange thickness;

z is the distance of flange under consideration from neutral axis see Figure 7.4;

r is the radius of curvature of arched beam;

σ_a is the mean stress in the flanges calculated with area of the gross cross-section. If the stress has been calculated over the effective cross-section, the mean stress is obtained by multiplying the stress for the effective cross-section by the ratio of the area of the effective flange to the area of the gross cross-section of the flange.

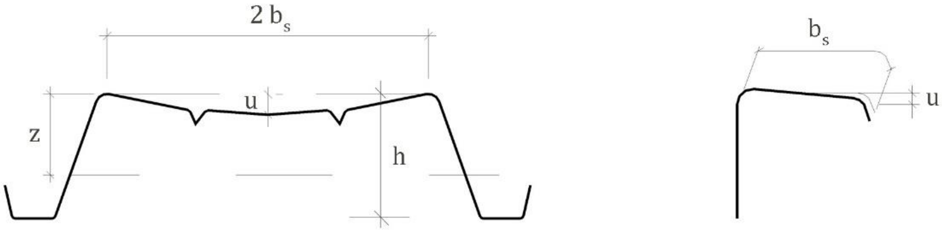


Figure 7.4 - Flange curling

7.5 Local and distortional buckling

7.5.1 General

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

(2) Local buckling effects may be accounted for by using the section properties of the effective cross-section, calculated on the basis of the effective widths, see EN 1993-1-5.

(3) In determining resistance to local buckling, the yield strength f_y should be taken as f_{yb} when calculating effective widths of compressed elements in EN 1993-1-5.

NOTE: For resistance see 8.1.

(4) 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.

(5) The distortional buckling for elements with edge or intermediate stiffeners as indicated in Figure 7.5(d) is considered in 7.5.3.

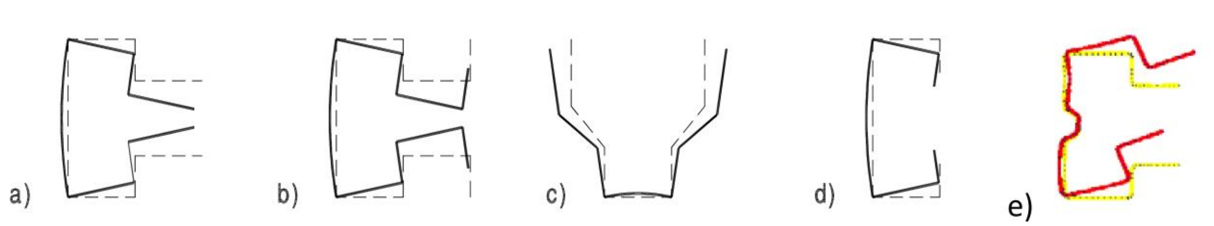


Figure 7.5 - Examples of distortional buckling modes

(6) The effects of distortional buckling should be allowed for in cases such as those indicated in Figures 7.5(a), (b), (c) and e). In these cases the effects of distortional buckling should be determined performing linear (see 7.5.1(7)) or non-linear buckling analysis (see EN 1993-1-5) using numerical methods or column stub tests.

(7) Unless the simplified procedure in 7.5.3 is used and where the elastic buckling stress is obtained from linear buckling analysis the following procedure may be applied:

1) For the wavelength up to the nominal member length, calculate the elastic buckling stress and identify the corresponding buckling modes, see Figure 7.6.

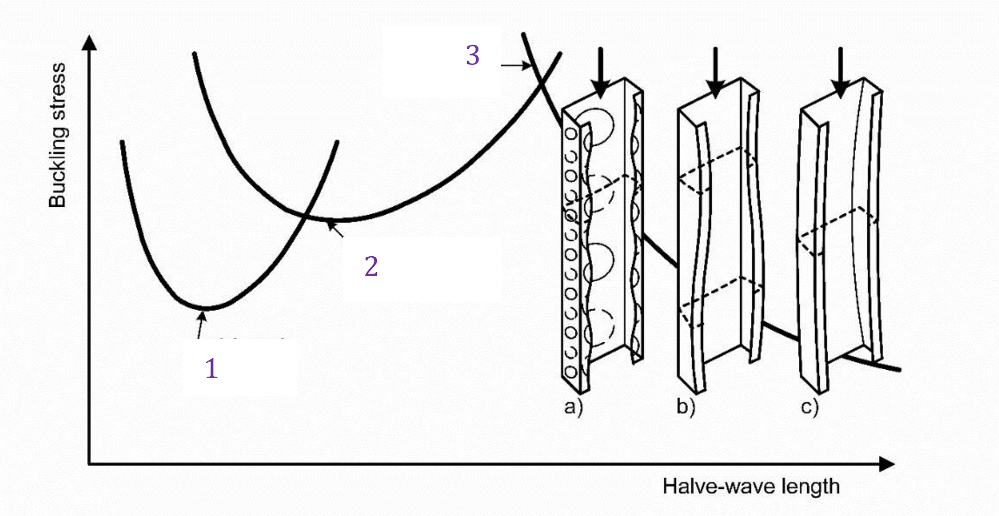
2) Calculate the effective width(s) according to 7.5.2 for locally buckled cross-section parts based on the minimum local buckling stress, see Figure 7.7.

3) Calculate the reduced thickness (see 7.5.3.2 or 7.5.3.3) of edge and intermediate stiffeners or other cross-section parts undergoing distortional buckling based on the minimum distortional buckling stress, see Figure 7.7.

4) Calculate overall buckling resistance according to 8.2 (flexural, torsional or lateral-torsional buckling depending on buckling mode) for nominal member length and based on the effective cross-section from 2) and 3).

(8) Numerical methods e.g. Finite Element Methods (FEM) or Finite Strip Methods (FSM) based on linear elastic bifurcation analysis (LBA) may be used for determining the elastic buckling stress referred to in 7.5.1(7).

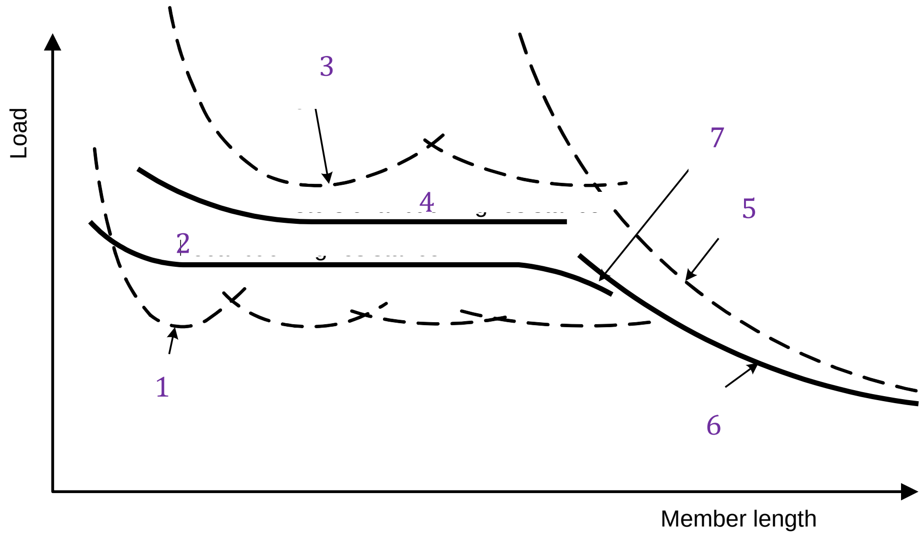
NOTE: For FE-analysis of steel structures, see EN 1993-1-14.



Key

Half-wave length l_{crit}
Buckling stress σ_{cr}
1 a) local buckling
2 b) distortional buckling
3 c) overall buckling

Figure 7.6 - Examples of elastic critical stress for various buckling modes as function of *halve-wave length* and examples of buckling modes.



Key

Member length L
Load N
1 Elastic local buckling (one wave, two waves, three waves, ...)
2 Local buckling resistance
3 Elastic distotional buckling
4 Distortional buckling resistance
5 Elastic overall buckling
6 Overall buckling resistance
7 Possible interaction local-global buckling

Figure 7.7 - Examples of elastic buckling load and buckling resistance as a function of member length

7.5.2 Plane elements without stiffeners

(1) The effective widths of unstiffened elements should be obtained from EN 1993-1-5 using the notional flat width b_{p} for $\bar{b}$ by determining the reduction factors for plate buckling based on the plate slenderness $\bar{\lambda}_p$.

(2) The notional flat width b_{p} of a plane element should be determined as specified in Table 7.1. In the case of plane elements in a sloping webs, the appropriate slant height should be used.

(3) For outstand elements an alternative method for plate buckling analysis is the mixed effective width/effective thickness method which shall be carried out as specified in Annex C.

(4) When applying the method prescribed in EN 1993-1-5 the following procedure may be used:

- The effective area of the flanges in a cross-section subject to a stress gradient may be determined from Tables 6.1 and 6.2 of EN 1993-1-5: 2020, using a stress ratio ψ based on the properties of the gross cross-section.
- The effective area of the web may be determined from Tables 6.1 and 6.2 of EN 1993-1-5: 2020, using a stress ratio ψ based on the area of the effective cross-section of the flange and the properties of the gross cross-section of the web.
- The section properties of the effective cross-section may be refined by recalculating the stress ratio ψ based on the effective cross-section already obtained in the previous step. When applying this option the minimum number of iterations is two.
- The simplified method given in 7.5.3.4 may be used in the case of webs of trapezoidal sheeting under a stress gradient.

7.5.3 Plane elements with edge or intermediate stiffeners

7.5.3.1 General

(1) The design of compression elements with edge or intermediate stiffeners should 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 an unit load u (per unit length) as illustrated in Table 7.4. The spring stiffness K per unit length may be determined as follows:

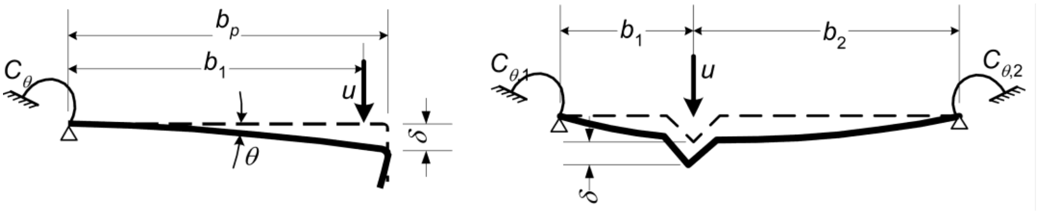
$$K = u/\delta \tag{7.11}$$

where:

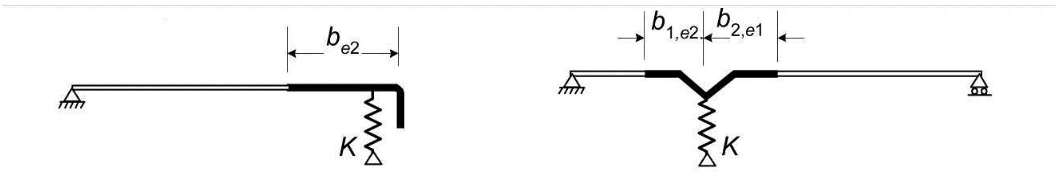
δ is the deflection of the stiffener due to the unit load u acting in the centroid (b_1) of the effective part of the cross-section, see Table 7.4.

Table 7.4 - Determination of spring stiffness

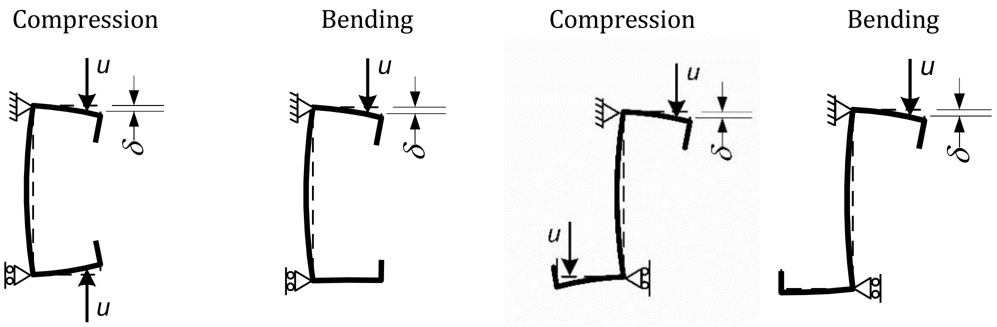
(a) Actual system



(b) Equivalent system



(c) Calculation of δ for C- and Z-sections



(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, see Table 7.4 (a), 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 determined as follows:

$$\delta = \theta b_p + \frac{u b_p^3}{3} \cdot \frac{12(1-v^2)}{E t^3} \tag{7.12}$$

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 Table 7.4 (c). This results in Formula (7.13) for calculating the spring stiffness K_1 for the flange 1:

$$K_1 = \frac{Et^3}{4(1-\nu^2)} \cdot \frac{1}{b_1^2 h_w + b_1^3 + 0{,}5 b_1 b_2 h_w k_f} \tag{7.13}$$

where:

b_1 is the distance from the web-to-flange junction to the centre of gravity of the effective cross-section of the edge stiffener (including effective part b_{e2} of the flange) of flange 1, see Table 7.4 (a);

b_2 is the distance from the web-to-flange junction to the centre of gravity of the effective cross-section of the edge stiffener (including effective part of the flange) of flange 2;

h_w is the web depth;

k_f
- $k_f = 0$ if flange 2 is in tension (e.g. for beam in bending about the y-y axis);
- $k_f = A_{s2}/A_{s1}$ if flange 2 is also in compression (e.g. for a beam in axial compression);
- $k_f = 1$ for a symmetric section in compression.

A_{s1} , A_{s2} is the effective area of the edge stiffener (including effective part b_{e2} of the flange, see Table 7.4 (b) of flange 1 and flange 2 respectively.

(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:

$$\delta = \frac{u b_1^2 b_2^2}{3(b_1 + b_2)} \cdot \frac{12(1-\nu^2)}{Et^3} \tag{7.14}$$

(7) The reduction factor χ_d for the distortional buckling resistance (flexural buckling of a stiffener) should be obtained from the relative slenderness $\bar{\lambda}_d$ as follows:

$$\chi_d = 1{,}0 \quad \text{if} \quad \bar{\lambda}_p \leq 0{,}65 \tag{7.15}$$

$$\chi_d = 1{,}47 - 0{,}723\,\bar{\lambda}_p \quad \text{if} \quad 0{,}65 \leq \bar{\lambda}_p < 1{,}38 \tag{7.16}$$

$$\chi_d = \frac{0{,}66}{\bar{\lambda}_p} \quad \text{if} \quad \bar{\lambda}_p \geq 1{,}38 \tag{7.17}$$

with:

$$\bar{\lambda}_p = \sqrt{f_{yb}/\sigma_{cr,s}} \tag{7.18}$$

where:

$\sigma_{cr,s}$ is the elastic critical stress for the stiffener(s) from 7.5.3.2, 7.5.3.3 or 7.5.3.4.

(8) Alternatively, the elastic critical buckling stress $\sigma_{cr,s}$ may be obtained from elastic first order buckling analysis using numerical methods (see 7.5.1(7)(8)).

(9) In the case of a plane element with an edge and intermediate stiffener(s) in the absence of a more accurate method the effect of the intermediate stiffener(s) may be neglected.

7.5.3.2 Plane elements with edge stiffeners

(1) The following procedure is applicable to an edge stiffener if the requirements in 7.2 are met and the angle between the stiffener and the plane element is between 45° and 135°.

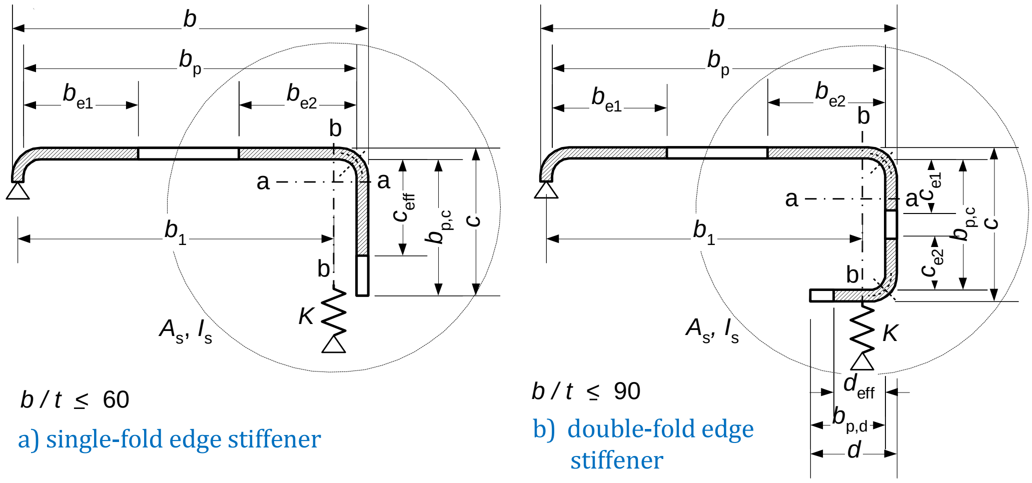


Figure 7.8 - Edge stiffeners

(2) 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 7.8, plus the adjacent effective portion of the plane element b_p.

(3) The procedure, which is illustrated in Table 7.5, should be carried out in steps as follows:

Step 1: **Local buckling of plate elements**

Obtain an initial effective cross-section for the stiffener using effective widths determined by assuming that the stiffener gives full restraint, see (4) to (5);

Step 2: **Distortional buckling (flexural buckling of the edge stiffener)**

Use the initial effective cross-section of the stiffener to determine the reduction factor for distortional buckling (flexural buckling of a stiffener), allowing for the effects of the continuous spring restraint, see (6), (7), (8), (9);

Step 3: **Refine the effective cross-section of the edge stiffener (optionally)**

This step is optional and may be omitted. Iterate to refine the value of the reduction factor for buckling of the stiffener, see (10);

Step 4: **Adopt an effective cross-section of the edge stiffener,** see (11) and (12)

(4) Initial values of the effective widths b_{e1} and b_{e2} shown in Figure 7.8 should be determined from Clause 7.5.2 by assuming that the plane element b_p is doubly supported, see Table 6.1 in EN 1993-1-5: 2020.

(5) Initial values of the effective widths c_{eff} and d_{eff} shown in Figure 7.8 should be obtained as follows:

a) for a single-fold edge stiffener:

$$c_{eff} = \rho \, b_{p,c} \tag{7.19}$$

with:

ρ obtained from 7.5.2, except using a value of the buckling factor k_σ given by the following:

$$k_\sigma = 0{,}5 \qquad \text{if } b_{\text{p,c}} / b_{\text{p}} \leq 0{,}35 \tag{7.20}$$

$$k_\sigma = 0{,}5 + 0{,}83 \sqrt[3]{\left(b_{p,c}/b_p - 0{,}35\right)^2} \qquad \text{if } 0{,}35 < b_{\text{p,c}} / b_{\text{p}} \leq 0{,}6 \tag{7.21}$$

b) for a double-fold edge stiffener:

$$c_{eff} = \rho \, b_{p,c} \tag{7.22}$$

with:

ρ obtained from 7.5.2 with a buckling factor k_σ for a doubly supported element from Table 6.1 in EN 1993-1-5: 2020;

and:

$$d_{eff} = \rho \, b_{p,d} \tag{7.23}$$

with:

ρ obtained from 7.5.2 with a buckling factor k_σ for an outstand element from Table 6.2 in EN 1993-1-5: 2020.

(6) In the calculation of the area A_{s} and the second moment of area I_{s} of the effective cross-section of the edge stiffener, the rounded corners shall be considered using the same principle as used for the calculation of the effective cross-section, see Figure 7.8.

(7) The elastic critical buckling stress $\sigma_{\text{cr,s}}$ for an edge stiffener should be determined as follows:

$$\sigma_{cr,s} = \frac{2\sqrt{K\,E\,I_s}}{A_s} \tag{7.24}$$

where:

K is the spring stiffness per unit length, see 7.5.3.1(2).

I_{s} is the second moment of area of the effective cross-section of the stiffener ~~, taken as that of the area A_{s}~~ about the centroidal axis a – a ~~of its effective cross-section~~, see Figure 7.8.

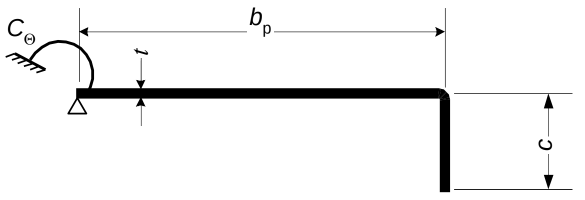
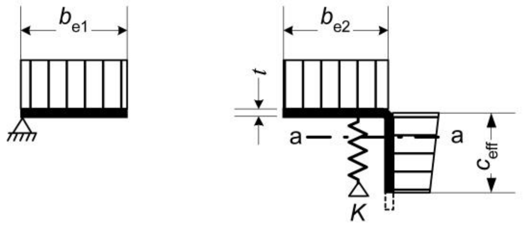
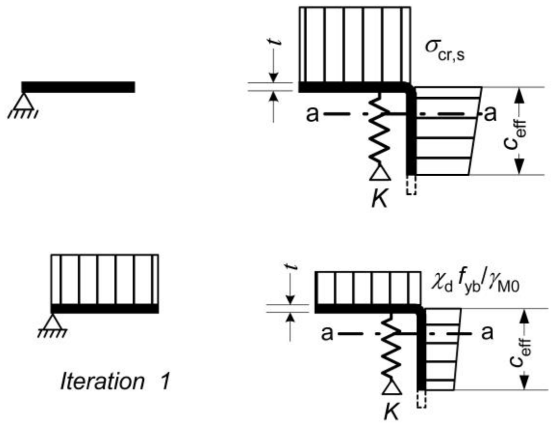
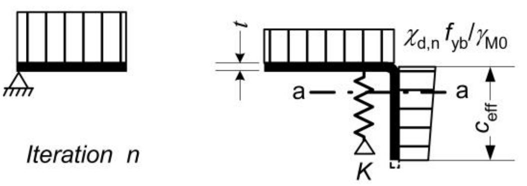
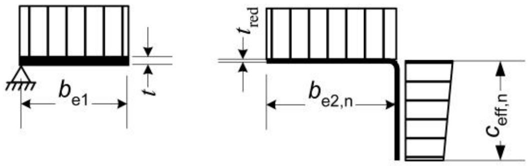
(8) Alternatively, the elastic critical buckling stress $\sigma_{\text{cr,s}}$ may be obtained from elastic first order buckling analyses using numerical methods, see 7.5.1(7)(8).

(9) The reduction factor χ_{d} for the distortional buckling (flexural buckling of a stiffener) resistance of an edge stiffener should be obtained from the value of $\sigma_{\text{cr,s}}$ using the method given in 7.5.3.1(7).

(10) If $\chi_{\text{d}} < 1$ it may be refined iteratively, starting the iteration with modified values of ρ obtained using 7.5.2 with $\sigma_{\text{com,Ed,red,i}}$ equal to $\chi_{\text{d}} f_{\text{yb}} / \gamma_{\text{M0}}$, so that:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi_d} \tag{7.25}$$

Table 7.5 - Compression resistance of a flange with an edge stiffener according to (3)

	**Gross cross-section and boundary conditions according to (1) and (2)**
	**Step 1: Local buckling of plate elements** Calculate the effective cross-section (b_{e1}, b_{e2}, c_{eff}) according to (4) and (5) for $K = \infty$
	**Step 2: Distortional buckling (flexural buckling of the edge stiffener)** a) Calculate the elastic critical stress $\sigma_{cr,s}$ for the area A_s of the effective cross-section of the edge stiffener from step 1 according to (6),(7) or (8) b) Calculate the reduction factor χ_d for the distortional buckling resistance of the edge stiffener according to (9) based on $\sigma_{cr,s}$
	**Step 3: Refine the effective cross-section of the edge stiffener (optionally)** If $\chi_d < 1.0$, repeat step 1 by calculating the effective width of the edge stiffener with a reduced compressive stress $\sigma_{com,Ed,red,i} = \chi_d f_{yb} / \gamma_{M0}$ according to (10) with χ_d from previous iteration, continuing until $\chi_{d,n} \approx \chi_{d,(n-1)}$, but $\chi_{d,n} \leq \chi_{d,(n-1)}$.
	**Step 4: Adopt an effective cross-section of the edge stiffener** Calculate a reduced thickness t_{red} of the edge stiffener (b_{e2}, c_{eff}) corresponding to $\chi_{d,n}$ according to (11) or (12)

(11) The reduced thickness t_{red} of all parts of the edge stiffener ($b_{e2,n}$,$c_{eff,n}$) covering the distortional buckling resistance of the edge stiffener should be taken as:

$$t_{red} = \chi_d \cdot t \qquad (7.26)$$

(12) If the effective widths of the plane elements according to 7.5.2 are determined based on the maximum design compressive stress $\sigma_{com.Ed} < f_{yb}/\gamma_{M0}$, the reduced stiffness t_{red} of the edge stiffener covering the distortional buckling resistance of the edge stiffener should be taken as

$$t_{red} = \chi_d \cdot t \, \frac{f_{yb}/\gamma_{M0}}{\sigma_{com,Ed}} \qquad \text{but } t_{red} \leq t \qquad (7.27)$$

where:

$\sigma_{com,Ed}$ is the initial compressive stress at the centre of gravity of the stiffener calculated on the basis of the effective cross-section according to (4), see step 1 in Table 7.5.

7.5.3.3 Plane elements with intermediate stiffeners

(1) The following procedure is applicable to one or two equal intermediate stiffeners formed by grooves or bends provided that all plane elements are calculated according to 7.5.2.

(2) 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 7.9.

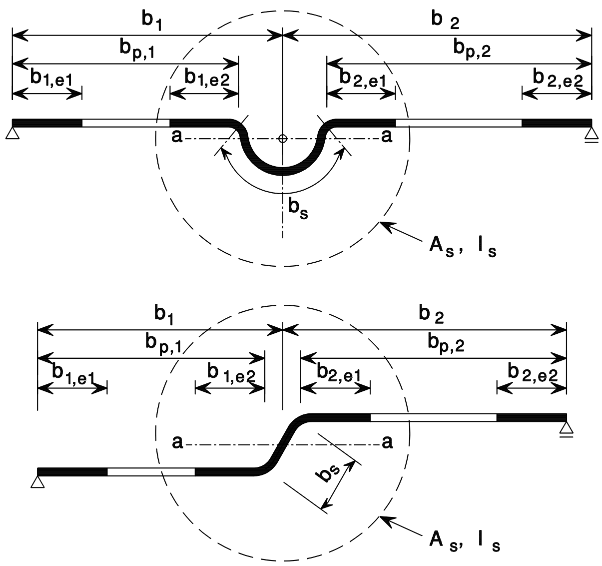


Figure 7.9 - Intermediate stiffeners

(3) The procedure, which is illustrated in Table 7.6, should be carried out in steps as follows:

Step 1: Local buckling of plate elements

Obtain an initial effective cross-section for the stiffener using effective widths determined by assuming that the stiffener gives full restraint ~~and that $\sigma_{com,Ed} = f_{yb}/\gamma_{M0}$~~, see (4) ~~and (5)~~

Step 2: Distortional buckling (flexural buckling of the intermediate stiffener)

Use the initial effective cross-section of the stiffener to determine the reduction factor for distortional buckling (flexural buckling of a stiffener), allowing for the effects of the continuous spring restraint, see (5), (6), (7), (8);

Step 3: Refine the effective cross-section of the intermediate stiffener (optionally)

This step is optional and may be omitted. Iterate to refine the value of the reduction factor for buckling of the stiffener, see (9) ~~and (10);~~

Step 4: Adopt an effective cross-section of the intermediate stiffener, see (10) and (11)

(4) Initial values of the effective widths $b_{1,e2}$ and $b_{2,e1}$ shown in Figure 7.9 should be determined from 7.5.2 by assuming that the plane elements $b_{p,1}$ and $b_{p,2}$ are doubly supported, see Table 6.1 in EN 1993-1-5: 2020.

(5) In the calculation of the cross-sectional area A_s and the second moment of area I_s of the effective cross-section of the intermediate stiffener, the rounded corners shall be considered using the same principle as used for the calculation of the effective cross-section, see Figure 7.9.

(6) The critical buckling stress $\sigma_{cr,s}$ for an intermediate stiffener should be determined as follows:

$$\sigma_{cr,s} = \frac{2\sqrt{K\,E\,I_s}}{A_s} \tag{7.28}$$

where:

K is the spring stiffness per unit length, see 7.5.3.1(2).

I_s is the second moment of area of the effective cross-section 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 7.9.

(7) Alternatively, the elastic critical buckling stress $\sigma_{cr,s}$ may be obtained from elastic first order buckling analyses using numerical methods, see 7.5.1(7)(8).

(8) The reduction factor χ_d for the distortional buckling resistance (flexural buckling of an intermediate stiffener) should be obtained from the value of $\sigma_{cr,s}$ using the method given in 7.5.3.1(7).

(9) If $\chi_d < 1$ it may optionally be refined iteratively, starting the iteration with modified values of ρ obtained using 7.5.2 with $\sigma_{com,Ed,red,i}$ equal to $\chi_d f_{yb}/\gamma_{M0}$, so that:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\chi_d} \tag{7.29}$$

(10) The reduced thickness t_{red} of all parts of the intermediate stiffener ($b_{1,e2.n}$, $b_{2,e1,n}$) covering the distortional buckling resistance of the intermediate stiffener should be taken as

$$t_{red} = \chi_d \cdot t \tag{7.30}$$

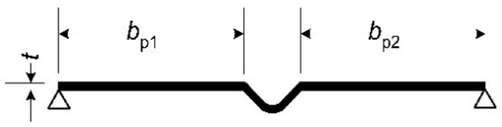
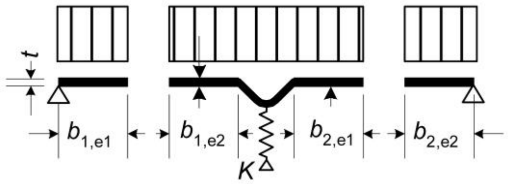
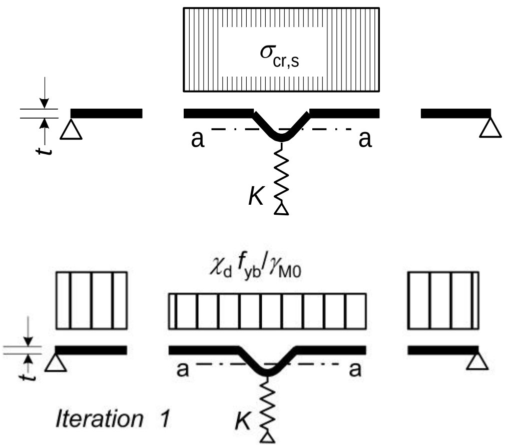
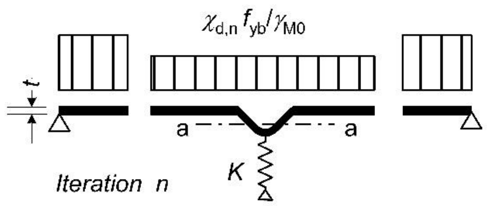
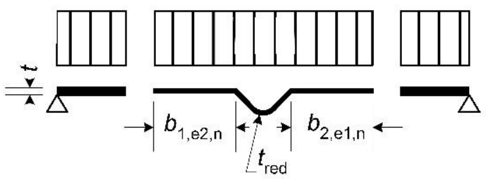
(11) If the effective widths of the plane elements according to 7.5.2 are determined based on the maximum design compressive stress $\sigma_{com.Ed} < f_{yb}/\gamma_{M0}$, the reduced stiffness t_{red} of the intermediate stiffener covering the distortional buckling resistance of the intermediate stiffener should be taken as:

$$t_{red} = \chi_d \cdot t\,\frac{f_{yb}/\gamma_{M0}}{\sigma_{com,Ed}} \quad \text{but } t_{red} \le t \tag{7.31}$$

where:

$\sigma_{com,Ed}$ is the initial compressive stress at the centreline of the stiffener calculated on the basis of the effective cross-section according to (4), see step 1 in Table 7.6.

Table 7.6 - Compression resistance of a flange with an intermediate stiffener according to (3)

	**Gross cross-section and boundary conditions according to (1) and (2)**
	**Step 1: Local buckling of plate elements** Calculate the effective cross-section (b_{1e1}, b_{1e2}, b_{2e1}, b_{2e2}) according to (4) for $K = \infty$
	**Step 2: Distortional buckling (flexural buckling of the intermediate stiffener)** a) Calculate the elastic critical stress $\sigma_{cr,s}$ for the area A_s of the effective cross-section of the intermediate stiffener from step 1 according to (5), (6), (7) b) Calculate the reduction factor χ_d for the distortional buckling resistance of the intermediate stiffener according to (8) based on $\sigma_{cr,s}$
	**Step 3: Refine the effective cross-section of the intermediate stiffener (optionally)** If $\chi_d < 1.0$, repeat step 1 by calculating the effective width of the intermediate stiffener with a reduced compressive stress $\sigma_{com,Ed,red,i} = \chi_d\ f_{yb} / \gamma_{M0}$ according to (9) with χ_d from previous iteration, continuing until $\chi_{d,n} \approx \chi_{d,(n-1)}$, but $\chi_{d,n} \leq \chi_{d,(n-1)}$.
	**Step 4: Adopt an effective cross-section of the intermediate stiffener** Calculate a reduced thickness t_{red} of the intermediate stiffener (b_{1e2}, b_{2e1}) corresponding to $\chi_{d,n}$ according to (10) or (11)

7.5.3.4 Trapezoidal sheeting

7.5.3.4.1 General

(1) This Subclause 7.5.3.4 should be used for trapezoidal sheeting, in association with the general procedure 7.5.3.3 for flanges 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 7.5.3.4.4.

7.5.3.4.2 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 cross-section including two strips of width $0{,}5b_{eff}$ (for calculating A_s) or 20t (for calculating I_s) adjacent to the stiffener, see Table 7.7.

Where the internal elements of a large u-shaped stiffener is not fully effective, the stiffener should be considered as one stiffener with reduced widths acc. to Table 7.7.

NOTE: The widths of the stiffener to calculate the second moment of area I_s according to Table 7.7 can be wider than the flange widths.

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

$$\sigma_{cr,s} = \frac{4{,}2\, k_w E}{A_s} \sqrt{\frac{I_s\, t^3}{4b_p^2\left(2b_p + 3b_s\right)}} \tag{7.32}$$

where:

- b_p is the notional flat width of plane element shown in Table 7.7;
- b_s is the stiffener width, measured around the perimeter of the stiffener, see Table 7.7;
- A_s, I_s are the cross-section area and the second moment of area of the stiffener cross-section according to Table 7.7;
- 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). For the calculation of the effective cross-section in axial compression the value $k_w = 1{,}0$.

The Formula (7.32) may be used for wide grooves provided that flat part of the stiffener is reduced due to local buckling and b_p in Formula (7.32) is replaced by the larger of b_p and $0{,}25(3b_p+b_r)$, see Table 7.7. Similar method is valid for flange with two or more wide grooves.

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

$$\sigma_{cr,s} = \frac{4{,}2\, k_w E}{A_s} \sqrt{\frac{I_s\, t^3}{8b_1^2\left(3b_e - 4b_1\right)}} \tag{7.33}$$

with:

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

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

where:

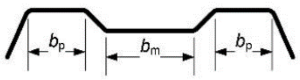
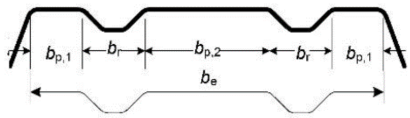
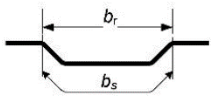
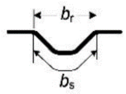
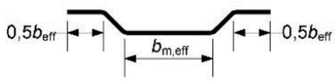
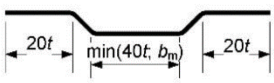
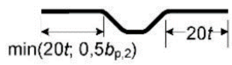
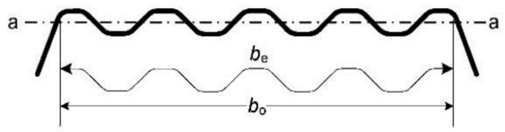
$b_{p,1}$ is the notional flat width of an outer plane element, as shown in Table 7.7;

$b_{p,2}$ is the notional flat width of the central plane element, as shown in Table 7.7;

b_r is the overall width of a stiffener, see Table 7.7;

A_s, I_s are the cross-section area and the second moment of area of the stiffener cross-section according to Table 7.7.

Table 7.7 - Compression flange with one, two or multiple stiffeners

Compression flange with one or two stiffeners		
	one stiffener	two stiffeneners
(a) geometry		
(b) stiffener width b_s		
(c) cross-section for calculating the effective area A_s		
(d) cross-section for calculating the effective second moment of area I_s		
Compression flange with multiple stiffeners		
(a) geometry and centroidal axis a-a (b) developed flange width b_e		

(4) For a multiple stiffened flange (three or more equal stiffeners) the effective area of the *entire flange* is

$$A_{eff} = \rho \, b_e \, t \tag{7.34}$$

where:

ρ is the reduction factor according to EN 1993-1-5: 2020, 6.4.1 (1)-(5) for the slenderness $\bar{\lambda}_p$ based on the elastic buckling stress $\sigma_{cr,s}$

$$\sigma_{cr,s} = 1{,}8 \, E \sqrt{\frac{I_s \, t}{b_o^2 \, b_e^3}} + 3{,}6 \frac{E t^2}{b_o^2} \tag{7.35}$$

where:

I_s is the sum of the second moment of area of the stiffeners about the centroidal axis a-a, neglecting the thickness terms $b \, t^3 /12$;

b_o is the width of the flange as shown in Table 7.7;

b_e is the developed width of the flange as shown in Table 7.7.

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

$$k_w = k_{wo} \qquad \text{if } l_b / s_w \geq 2 \qquad (7.36)$$

$$k_w = k_{wo} - (k_{wo} - 1)\left[\frac{2\,l_b}{s_w} - \left(\frac{l_b}{s_w}\right)^2\right] \qquad \text{if } l_b / s_w < 2: \qquad (7.37)$$

where:

s_w is the slant height of the web, see Table 7.1(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 as follows:

a) for a compression flange with one intermediate stiffener, see Table 7.7:

$$l_b = 3{,}07\sqrt[4]{\frac{I_s\,b_p^2\,(2\,b_p + 3\,b_s)}{t^3}} \qquad (7.38)$$

$$k_{wo} = \sqrt{\frac{s_w + 2b_d}{s_w + 0{,}5b_d}} \qquad (7.39)$$

with:

$b_d = 2b_p + b_s$

b) for a compression flange with two intermediate stiffeners, see Table 7.7:

$$l_b = 3{,}65\sqrt[4]{\frac{I_s\,b_1^2\,(3\,b_e - 4\,b_1)}{t^3}} \qquad (7.40)$$

$$k_{wo} = \sqrt{\frac{(2b_e + s_w)(3b_e - 4b_1)}{b_1(4b_e - 6b_1) + s_w(3b_e - 4b_1)}} \qquad (7.41)$$

(8) If the webs are unstiffened, the reduction factor χ_d should be obtained directly from $\sigma_{cr,s}$ using the method given in 7.5.3.1(7).

(9) If the webs are also stiffened, the reduction factor χ_d should be obtained using the method given in 7.5.3.1(7), but with the modified elastic critical stress $\sigma_{cr,mod}$ given in 7.5.3.4.4.

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

$$A_{s,red} = \chi_d \cdot A_s \frac{f_{yb}/\gamma_{M0}}{\sigma_{com,Ed}} \qquad \text{but } A_{s,red} \leq A_s \qquad (7.42)$$

where:

$\sigma_{com,Ed}$ is the compressive stress at the centreline of the stiffener calculated based on the effective cross-section

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

(12) The section properties of the cross-section of the stiffeners at serviceability limit states should be based on the design thickness t.

7.5.3.4.3 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 7.10.

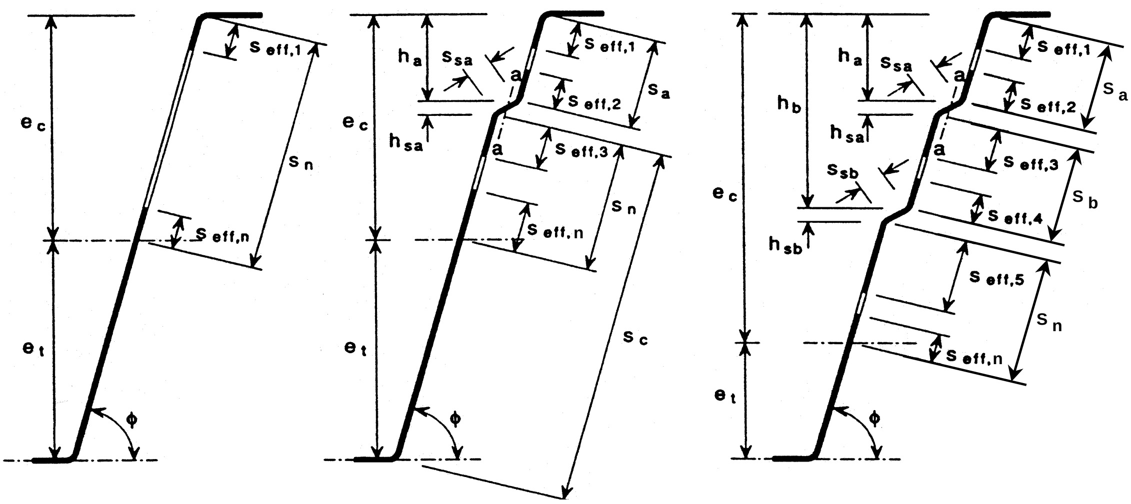
(2) The effective cross-section of a web as shown in Figure 7.10 should be taken to include:

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.



(a) without stiffener (b) one stiffener (c) two stiffeners

Figure 7.10 - Effective cross-sections of webs of trapezoidal sheeting

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

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

$$A_{sa} = t\left(s_{eff,2} + s_{eff,3} + s_{sa}\right) \quad (7.43)$$

b) for a second stiffener:

$$A_{sa} = t\left(s_{eff,4} + s_{eff,5} + s_{sb}\right) \quad (7.44)$$

where:

$s_{eff,1}$ to $s_{eff,n}$ and s_{sa} and s_{sb} are the dimensions shown in Figure 7.10.

(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:

$$s_{eff,0} = 0{,}76\, t \sqrt{\frac{E}{\sigma_{com,Ed} \cdot \gamma_{M0}}} \quad (7.45)$$

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 $s_{\mathrm{eff,1}}$ to $s_{\mathrm{eff,n}}$ should be determined as follows:

$$s_{eff,1} = s_{eff,0} \tag{7.46}$$

$$s_{eff,2} = s_{eff,0} \left(1 + 0{,}5 \frac{h_a}{e_c}\right) \tag{7.47}$$

$$s_{eff,3} = s_{eff,0} \left(1 + 0{,}5 \frac{(h_a + h_{sa})}{e_c}\right) \tag{7.48}$$

$$s_{eff,4} = s_{eff,0} \left(1 + 0{,}5 \frac{h_b}{e_c}\right) \tag{7.49}$$

$$s_{eff,5} = s_{eff,0} \left(1 + 0{,}5 \frac{(h_b + h_{sb})}{e_c}\right) \tag{7.50}$$

$$s_{eff,1} = 1{,}5\, s_{eff,0} \tag{7.51}$$

where:

e_c is the distance from the effective centroidal axis to the system line of the compression flange, see Figure 7.10;

h_a, h_b, h_{sa} and h_{sb} are the dimensions shown in Figure 7.10.

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

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

$$s_{eff,1} = 0{,}4\, s_n \tag{7.52}$$

$$s_{eff,n} = 0{,}6\, s_n \tag{7.53}$$

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

$$s_{eff,1} = s_a \frac{1}{\left(2 + 0{,}5 \frac{h_a}{e_c}\right)} \tag{7.54}$$

$$s_{eff,2} = s_a \frac{\left(1 + 0{,}5 \frac{h_a}{e_c}\right)}{\left(2 + 0{,}5 \frac{h_a}{e_c}\right)} \tag{7.55}$$

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

$$s_{eff,3} = s_n \frac{\left(1 + 0{,}5 \frac{(h_a + h_{sa})}{e_c}\right)}{\left(2{,}5 + 0{,}5 \frac{(h_a + h_{sa})}{e_c}\right)} \tag{7.56}$$

$$s_{eff,n} = s_n \frac{1{,}5}{\left(2{,}5 + 0{,}5 \frac{(h_a + h_{sa})}{e_c}\right)} \tag{7.57}$$

d) 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:

$$s_{eff,3} = s_b \frac{\left(1 + 0{,}5\,\dfrac{(h_a + h_{sa})}{e_c}\right)}{\left(2 + 0{,}5\,\dfrac{(h_a + h_{sa} + h_b)}{e_c}\right)} \tag{7.58}$$

$$s_{eff,4} = s_b \frac{\left(1 + 0{,}5\,\dfrac{h_b}{e_c}\right)}{\left(2 + 0{,}5\,\dfrac{(h_a + h_{sa} + h_b)}{e_c}\right)} \tag{7.59}$$

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

$$s_{eff,5} = s_n \frac{\left(1 + 0{,}5\,\dfrac{(h_b + h_{sb})}{e_c}\right)}{\left(2{,}5 + 0{,}5\,\dfrac{(h_b + h_{sb})}{e_c}\right)} \tag{7.60}$$

$$s_{eff,n} = s_n \frac{1{,}5}{(2{,}5 + 0{,}5\,(h_b + h_{sb})/e_c)} \tag{7.61}$$

(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:

$$\sigma_{cr,sa} = \frac{1{,}05\,k_f\,E}{A_{sa}\,s_2(s_1 - s_2)}\sqrt{I_s\,t^3\,s_1} \tag{7.62}$$

with:

s_1 is given as follows:

- for a single stiffener:

$$s_1 = 0{,}9\,(s_a + s_{sa} + s_c) \tag{7.63}$$

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

$$s_1 = s_a + s_{sa} + s_b + 0{,}5\,(s_{sb} + s_c) \tag{7.64}$$

$$s_2 = s_1 - s_a - 0{,}5\,s_{sa} \tag{7.65}$$

where:

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

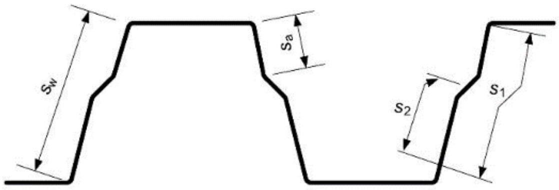
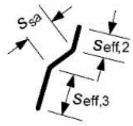
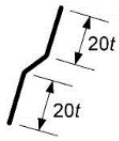
I_s is the second moment of area of a stiffener cross-section comprising the fold width s_{sa} and two adjacent strips, each of width $20t$, about its own centroidal axis parallel to the plane web elements, see Table 7.8. In calculating I_s the possible difference in slope between the plane web elements on either side of the stiffener may be neglected;

s_c as defined in Figure 7.10.

NOTE: The widths of the stiffener to calculate the second moment of area I_s according to Table 7.8 can be wider than the web height.

(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.

Table 7.8 – Web stiffeners for trapezoidal sheeting

(a) geomentry	
(b) cross-section for calculating the effective area A_s	
(c) cross-section for calculating the effective second moment of area I_s	

(9) If the flanges are unstiffened, the reduction factor χ_d should be obtained directly from $\sigma_{cr,sa}$ using the method given in 7.5.3.1(7).

(10) If the flanges are also stiffened, the reduction factor χ_d should be obtained using the method given in 7.5.3.1(7), but with the modified elastic critical stress $\sigma_{cr,mod}$ given in 7.5.3.4.4.

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

$$A_{sa,red} = \frac{\chi_d \, A_{sa}}{1-(h_a+0{,}5\,h_{sa})/e_c} \qquad \text{but} \qquad A_{sa,red} \le A_{sa} \tag{7.66}$$

(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 $t_{red} = \chi_d \, t$ for all the elements included in A_{sa}.

(15) The effective section properties of the stiffeners 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:

$$s_{eff,0} = 0{,}95\, t \sqrt{\frac{E}{\sigma_{com,ser} \cdot \gamma_{M0}}} \tag{7.67}$$

where:

$\sigma_{com,ser}$ is the compressive stress at the centre of gravity of the stiffener calculated based on the effective cross-section in serviceability limit state, simplifying $\sigma_{com,ser} = f_{yb}/1{,}5$ can be used

7.5.3.4.4 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 7.11, 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:

$$\sigma_{cr,mod} = \frac{\sigma_{cr,s}}{\sqrt[4]{1 + \left(\beta_s \frac{\sigma_{cr,s}}{\sigma_{cr,sa}}\right)^4}} \tag{7.68}$$

where:

$\sigma_{cr,s}$ is the elastic critical stress for an intermediate flange stiffener, see 7.5.3.4.2(2) for a flange with a single stiffener or 7.5.3.4.2(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 7.5.3.4.3(7);

A_s is the area of the effective cross-section of an intermediate flange stiffener;

A_{sa} is the area of the effective cross-section of an intermediate web stiffener;

β_s $= 1 - (h_a + 0{,}5\, h_{sa}) / e_c$ for a profile in bending; for dimensions see Figure 7.10

β_s $= 1$ for a profile in axial compression.

(2) The section properties of the effective cross-section of the stiffeners at serviceability limit state should be based on the design thickness t.

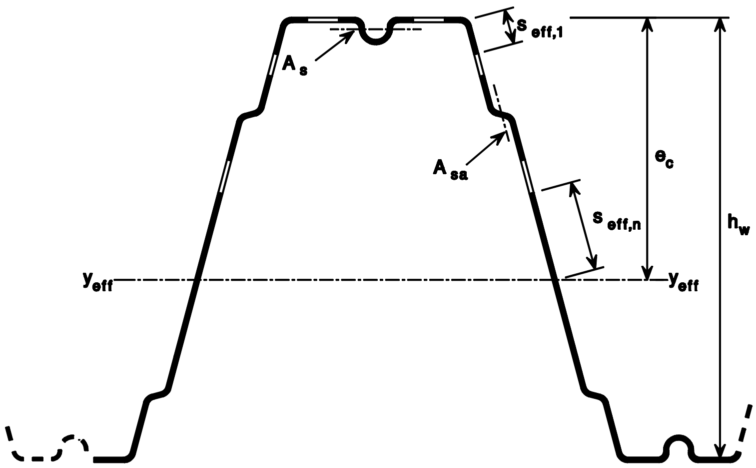


Figure 7.11 - Trapezoidal sheeting with flange stiffeners and web stiffeners

7.6 Plate buckling between fasteners

(1) Plate buckling between fasteners should be checked for elements composed of plates and mechanical fasteners, see Table 5.3 of EN 1993-1-8: 2018.

8 Ultimate limit states

8.1 Resistance of cross-sections

8.1.1 General

(1) 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.

(2) For design by calculation, the effects of local buckling should be taken into account by using the section properties of the effective cross-section determined as specified in Clause 7.5.

(3) The buckling resistance of members should be verified as specified in Clause 8.2.

(4) In members with cross-sections that are susceptible to distortion of the cross-section, account should be taken of possible lateral buckling of compression flanges and lateral bending of flanges generally, see 7.5, and 11.1.

8.1.2 Axial tension

(1) The design resistance of a cross-section for uniform tension $N_{t,Rd}$ should be calculated as follows:

$$N_{t,Rd} = \frac{A\, f_{ya}}{\gamma_{M0}} \qquad \text{but} \qquad N_{t,Rd} \leq F_{n,Rd} \tag{8.1}$$

where:

A	is the area of the gross cross-section
$F_{n,Rd}$	is the net-section resistance, determined according to 10.3 for the relevant type of mechanical fastener;
f_{ya}	is the average yield strength, see 5.2.2.

(2) The design resistance of an angle in uniform tension connected through one leg, or other types of section connected through outstands, should be determined as specified in EN 1993-1-8: 2018, 3.11.3.

8.1.3 Axial compression

(1) The design resistance of a cross-section for compression $N_{c,Rd}$ should be determined as follows:

- if $A_{eff} \leq A$:

$$N_{c,Rd} = \frac{A_{eff}\, f_{yb}}{\gamma_{M0}} \tag{8.2}$$

- if $A_{eff} = A$:

$$N_{c,Rd} = \frac{A\left(f_{yb} + 4\left(f_{ya} - f_{yb}\right)\left(1 - \bar{\lambda}_{e,max}/\bar{\lambda}_{eo}\right)\right)}{\gamma_{M0}} \leq \frac{A\, f_{ya}}{\gamma_{M0}} \tag{8.3}$$

where:

A_{eff} is the area of the effective cross-section accounting for local and distortional buckling in accordance with Clause 7.5 while assuming a uniform compressive stress equal to f_{yb};

A is the area of the gross cross-section

f_{ya} is the average yield strength,, see 5.2.2;

f_{yb} is the basic yield strength, see 5.2.2.;

$\bar{\lambda}_{e,max}$ is the largest value of the relative slenderness over all plate elements comprising the cross-section. The slenderness values $\bar{\lambda}_e$ and $\bar{\lambda}_{e0}$ should be determined as follows:

- for internal compression elements: $\bar{\lambda}_e = \bar{\lambda}_p$ and $\bar{\lambda}_{e0} = 0{,}673$, see 7.5.2;
- for outstand compression elements: $\bar{\lambda}_e = \bar{\lambda}_p$ and $\bar{\lambda}_{e0} = 0{,}748$, see 7.5.2;
- for stiffened elements: $\bar{\lambda}_e = \bar{\lambda}_d$ and $\bar{\lambda}_{e0} = 0{,}65$, see 7.5.3.

(2) The axial force in a member may be assumed to act at the centroid of its gross cross-section. This is a conservative assumption, but may be used without further analysis. Further analysis may give a more accurate distribution of the internal forces.

(3) The design compressive resistance of a cross-section, as calculated using 8.1.3(1), constitutes the resistance against an axial load acting at the centroid of the effective cross-section. If the centroid of the effective cross-section does not coincide with the centroid of the gross cross-section, the additional bending moment caused by the shifts e_{Ny} and e_{Nz} of the centroidal axes (see Figure 8.1) should be taken into account, using the interaction formulae given in Clause 8.1.9. When a shift of the neutral axis has a favourable effect on the cross-sectional resistance, this shift should be neglected, but only if the shift has been calculated at the yield stress and not with the actual compressive stresses.

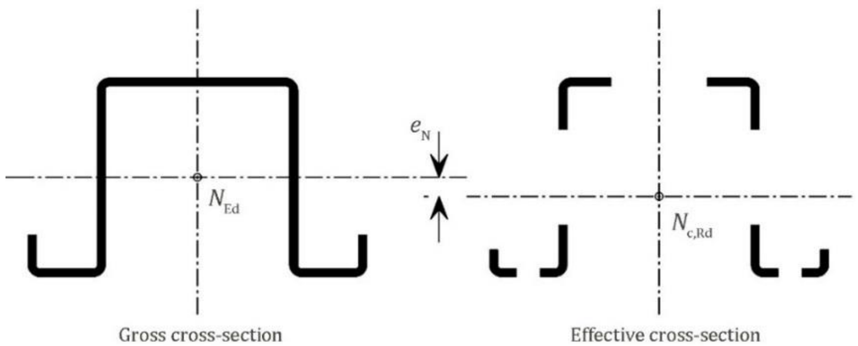


Figure 8.1 - Effective cross-section in compression

8.1.4 Bending moment

8.1.4.1 Elastic and elastic-plastic resistance with first yielding at the compression flange

(1) The design moment resistance of a cross-section in bending about one principal axis $M_{c,Rd}$ should be determined as follows (see Figure 8.2):

- if $W_{eff} \leq W_{el}$

$$M_{c,Rd} = \frac{W_{eff}\, f_{yb}}{\gamma_{M0}} \tag{8.4}$$

- if $W_{eff} = W_{el}$ and the conditions listed in (2) are satisfied:

$$M_{c,Rd} = \frac{\left(W_{el}\, f_{yb} + 3\,(W_{pl} f_{ya} - W_{el} f_{yb})(1 - \bar{\lambda}_{e,max}/\bar{\lambda}_{eo})\right)}{\gamma_{M0}} \leq \frac{W_{pl}\, f_{ya}}{\gamma_{M0}} \tag{8.5}$$

where:

W_{eff} is the elastic section modulus of the effective cross-section accounting for local and distortional buckling in accordance with Clause 7.5, see also (4)

W_{el} is the elastic section modulus of the gross cross-section

W_{pl} is the plastic section modulus of the gross cross-section

f_{ya} is the average yield strength, see 5.2.2;

f_{yb} is the basic yield strength, see 5.2.2.;

$\bar{\lambda}_{e,max}$ is the largest value of the relative slenderness $\bar{\lambda}_e/\bar{\lambda}_{e0}$ over all plate elements comprising the cross-section. The relative slendernesses $\bar{\lambda}_e$ and $\bar{\lambda}_{e0}$ should be determined as follows:

a) For plane elements with or without stiffeners according to 7.5 with the exception of trapezoidal sheeting webs:

- for internal compression elements:

$$\bar{\lambda}_e = \bar{\lambda}_p \quad \text{and} \quad \bar{\lambda}_{e0} = 0{,}5 + \sqrt{0{,}25 - 0{,}055\,(3 + \psi)}$$

where:

ψ is the stress ratio, see 7.5.2;

- for outstand compression elements:

$\bar{\lambda}_e = \bar{\lambda}_p$ and $\bar{\lambda}_{e0} = 0{,}748$, see 7.5.2;

- for stiffened elements:

$\bar{\lambda}_e = \bar{\lambda}_d$ and $\bar{\lambda}_{e0} = 0{,}65$, see 7.5.2;

b) For trapezoidal sheeting webs according to 7.5.3.4.3:

- for trapezoidal sheeting webs without any stiffeners in the compression zone:

$$\frac{\bar{\lambda}_e}{\bar{\lambda}_{e0}} = \frac{s_n}{s_{eff,1} + s_{eff,n}}$$

- for trapezoidal sheeting webs with one or two intermediate stiffeners in the compression zone:

$\bar{\lambda}_e = \bar{\lambda}_d$ and $\bar{\lambda}_{e0} = 0{,}65$, see 7.5.2;

with:

$$\frac{\bar{\lambda}_e}{\bar{\lambda}_{e0}} = \frac{s_n}{s_{eff,1} + s_{eff,2}}$$ for the portion of the web comprised between the compression flange and the first stiffener

$$\frac{\bar{\lambda}_e}{\bar{\lambda}_{e0}} = \frac{s_n}{s_{eff,3} + s_{eff,n}}$$ for the portion of the web comprised between the neutral axis and the stiffener in a web with one stiffener in the compression zone

$$\frac{\bar{\lambda}_e}{\bar{\lambda}_{e0}} = \frac{s_n}{s_{eff,3} + s_{eff,4}}$$ for the portion of the web comprised between two stiffeners

$$\frac{\bar{\lambda}_e}{\bar{\lambda}_{e0}} = \frac{s_n}{s_{eff,5} + s_{eff,n}}$$ for portion of the web nearest to the neutral axis in a web with two stiffeners in the compression zone

where:

s_n is the length indicated in Figure 7.10;

$s_{eff,1}$ to $s_{eff,n}$ are given by Formulae (7.46) to (7.51) and illustrated in Figure 7.10.

The resulting bending moment resistance resulting from Fomula (8.5) is plotted as a function of the relative slenderness in Figure 8.2.

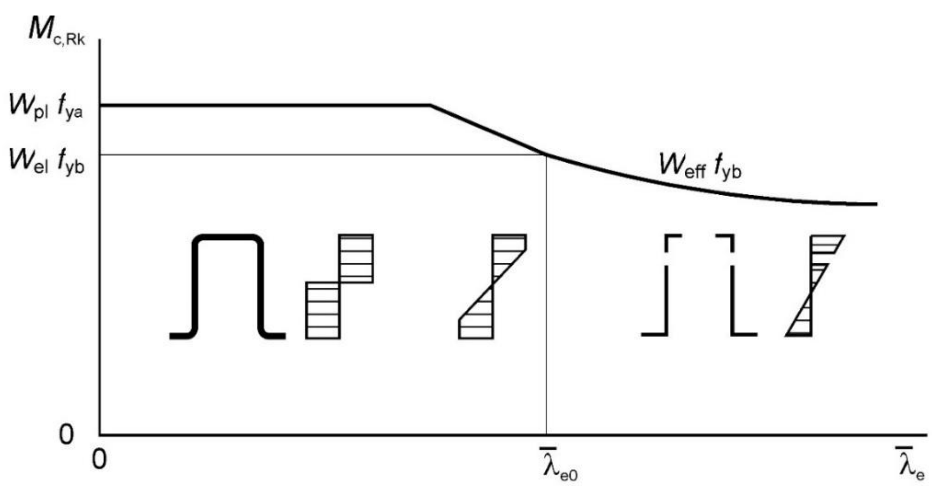


Figure 8.2 - Bending moment resistance as a function of slenderness

(2) Formula (8.5) is applicable provided that the following conditions are simultaneously satisfied:

a) The bending moment is applied about only one principal axis of the cross-section;

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

c) The angle ϕ between the web (see Figure 8.5) and the flange is larger than 60°.

(3) If the conditions in (2) are not fulfilled the design moment resistance of a cross-section in bending about one principal axis should be determined as follows:

$$M_{c,Rd} = \frac{W_{el}\, f_{ya}}{\gamma_{M0}} \tag{8.6}$$

(4) The section modulus of the effective cross-section W_{eff} should be based on an effective cross-section that is subject only to a bending moment about the relevant principal axis, with a maximum stress $\sigma_{\mathrm{max,Ed}}$ equal to $f_{\mathrm{yb}} / \gamma_{\mathrm{M0}}$, allowing for the effects of local and distortional buckling as specified in Clause 7.5. Where shear lag is relevant, allowance should also be made for its effects.

(5) The stress ratio $\psi = \sigma_2 / \sigma_1$ used to determine the effective portions of the web may be obtained by considering the effective area of the compression flange but the area of the gross cross-section of the web, see Figure 8.3.

(6) If yielding occurs first at the compressive edge of the cross-section, unless the conditions given in 8.1.4.2 are met, the value of W_{eff} should be based on a linear stress distribution across the cross-section.

(7) For biaxial bending the following criterion may be used:

$$\frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}} \leq 1 \tag{8.7}$$

where:

$M_{\mathrm{y,Ed}}$ is the bending moment about the major main axis;

$M_{\mathrm{z,Ed}}$ is the bending moment about the minor main axis;

$M_{\mathrm{cy,Rd}}$ is the resistance of the cross-section if subject only to moment about the main y – y axis;

$M_{\mathrm{cz,Rd}}$ is the resistance of the cross-section if subject only to moment about the main z – z axis.

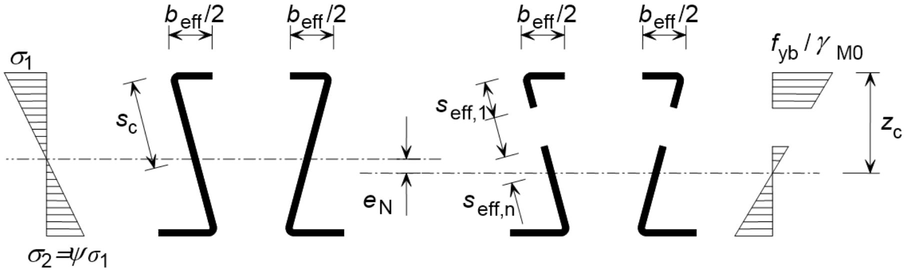


Figure 8.3 - Effective cross-section for resistance to bending moments

(8) Redistribution of the bending moments within the member due to plastification at internal supports may be accounted for in the global analysis, provided the following conditions are met:

- Any residual moment greater than zero has to be justified by the results from tests according to Clause 12.
- The serviceability limit state provisions given in Clause 9.2 have to be satisfied in any case.

8.1.4.2 Elastic and elastic-plastic resistance with first yielding at the tension flange

(1) Provided that the bending moment is applied about only one principal axis of the cross-section, and provided that yielding occurs first at the tension edge, the plastic reserve capacity in the tension zone may be utilised without any strain limit until the maximum compressive stress $\sigma_{com,Ed}$ reaches f_{yb} / γ_{M0}. In this Clause only the bending case is considered. This Clause does not apply to combined axial load and bending, for which Clauses 8.1.8 or 8.1.9 should be used.

(2) In this case, the partially plastic section modulus of the effective cross-section $W_{pp,eff}$ 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 a stress gradient may be obtained according to 7.5.2 with b_c determined from the bilinear stress distribution as indicated in Figure 8.4 and while assuming $\psi = -1$.

(4) Redistribution of the bending moments within the member due to plastification at internal supports may be accounted for in the global analysis, provided the following conditions are met:

- Any residual moment greater than zero has to be justified by testing according to Clause 12.
- The serviceability limit state provisions given in 9.2 have to be satisfied in any case.

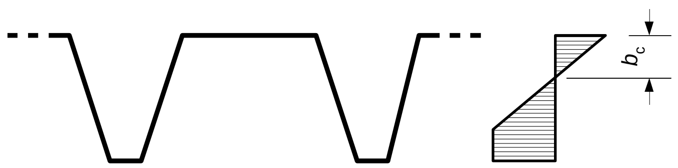


Figure 8.4 - Depth of the compression zone b_c for the determination of the effective width

8.1.4.3 Effects of shear lag

(1) The effects of shear lag should be taken into account according to EN 1993-1-5.

(2) The effect of shear lag for liner trays has been taken into account in 11.2.2.2.

(3) The effect of shear lag may be neglected for sheeting and members having dimensions indicated in 7.2.

8.1.5 Shear force

(1) The design shear resistance $V_{b,Rd}$ in the plane of the web should be determined as follows:

$$V_{b,Rd} = \frac{\frac{h_w}{\sin\phi}\, t\, f_{bv}}{\gamma_{M0}} \tag{8.8}$$

where:

f_{bv} is the shear strength accounting for shear buckling according to Table 8.1;

h_w is the web height between the midlines of the flanges, see Table 7.1(c);

ϕ is the slope of the web relative to the flanges, see Figure 8.5.

The design shear resistance perpendicular to the flanges should be determined as follows:

$$V_{w,Rd} = V_{b,Rd} \sin\phi \tag{8.9}$$

Table 8.1 - Shear buckling strength f_{bv}

Relative web slenderness	Web without stiffening at the support	Web with stiffening at the support [1)]
$\bar{\lambda}_w \leq 0{,}83$	$0{,}58\ f_{yb}$	$0{,}58\ f_{yb}$
$0{,}83 < \bar{\lambda}_w < 1{,}4$	$0{,}48\ f_{yb}/\ \bar{\lambda}_w$	$0{,}48\ f_{yb}/\ \bar{\lambda}_w$
$\bar{\lambda}_w \geq 1{,}4$	$0{,}48\ f_{yb}/\ \bar{\lambda}_w^2$	$0{,}48\ f_{yb}/\ \bar{\lambda}_w$
1) Stiffening at the support, such as cleats, arranged to prevent distortion of the web and designed to resist the support reaction.		

(2) The relative web slenderness $\bar{\lambda}_w$ should be determined as follows:

- for webs without longitudinal stiffeners:

$$\bar{\lambda}_w = 0{,}346 \frac{s_w}{t} \sqrt{\frac{f_{yb}}{E}} \tag{8.10}$$

- for webs with longitudinal stiffeners, see Figure 8.5:

$$\bar{\lambda}_w = 0{,}346 \frac{s_d}{t} \sqrt{\frac{5{,}34\ f_{yb}}{k_\tau\ E}} \quad \text{but} \quad \bar{\lambda}_w \geq 0{,}346 \frac{s_p}{t} \sqrt{\frac{f_{yb}}{E}} \tag{8.11}$$

with:

$$k_\tau = 5{,}34 + \frac{2{,}10}{t} \left(\frac{\sum I_s}{s_d} \right)^{1/3}$$

where:

I_s is the second moment of area of the individual longitudinal stiffener as defined in 7.5.3.4.3(7), about the axis a – a as indicated in Figure 8.5;

s_d is the total developed slant height of the web, as indicated in Figure 8.5;

s_p is the slant height of the widest plane element in the web, see Figure 8.5;

s_w is the slant height of the web, as shown in Figure 8.5, defined by the distance between the midpoints of the corners, see Table 7.1(c).

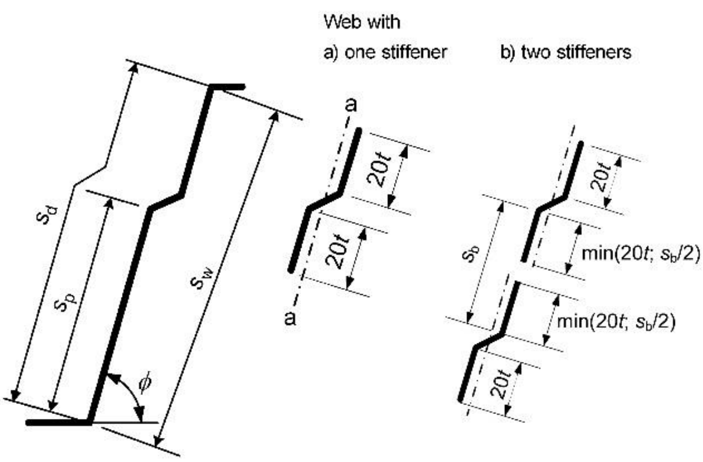


Figure 8.5 - Longitudinally stiffened web

8.1.6 Torsional moment

(1) Where loads are applied eccentrically to the shear centre of the cross-section, the effects of torsion should be taken into account.

(2) The centroidal axis and shear centre and imposed rotation centre to be used in determining the effects of the torsional moment, should be taken as those of the gross cross-section.

(3) The longitudinal stresses due to the axial force N_{Ed} and the bending moments $M_{y,Ed}$ and $M_{z,Ed}$ should be based on the respective effective cross-sections, calculated as specified in 8.1.2 to 8.1.4. The shear stresses due to transverse shear forces, the shear stress due to uniform (St. Venant) torsion and the longitudinal 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 (based on the average yield strength f_{ya}, see 5.2.2):

$$\sigma_{tot,Ed} \leq \frac{f_{ya}}{\gamma_{M0}} \tag{8.12}$$

$$\tau_{tot,Ed} \leq \frac{f_{ya}/\sqrt{3}}{\gamma_{M0}} \tag{8.13}$$

$$\sqrt{\sigma_{tot,Ed}^2 + 3\,\tau_{tot,Ed}^2} \leq 1{,}1\frac{f_{ya}}{\gamma_{M0}} \tag{8.14}$$

where:

$\sigma_{tot,Ed}$ is the total longitudinal-stress resulting from the design values of the actions, see (3)(5)

$\tau_{tot,Ed}$ is the total shear stress resulting from the design values of the actions and calculated on the basis of the gross cross-section, see (5).

(5) The total-longitudinal stress $\sigma_{tot,Ed}$ and the total shear stress $\tau_{tot,Ed}$ should be determined as follows:

$$\sigma_{tot,Ed} = \sigma_{N,Ed} + \sigma_{My,Ed} + \sigma_{Mz,Ed} + \sigma_{w,Ed} \tag{8.15}$$

$$\tau_{tot,Ed} = \tau_{Vy,Ed} + \tau_{Vz,Ed} + \tau_{t,Ed} + \tau_{w,Ed} \tag{8.16}$$

where:

$\sigma_{My,Ed}$ is the longitudinal stress due to the bending moment $M_{y,Ed}$ (using the effective cross-section, see 8.1.4);

$\sigma_{Mz,Ed}$ is the longitudinal stress due to the bending moment $M_{z,Ed}$ (using the effective cross-section, see 8.1.4);

$\sigma_{N,Ed}$ is the longitudinal stress due to the axial force N_{Ed} (using the effective cross-section, see 8.1.2 and 8.1.3);

$\sigma_{w,Ed}$ is the longitudinal stress due to warping (using the gross cross-section);

$\tau_{Vy,Ed}$ is the shear stress due to the transverse shear force $V_{y,Ed}$ (using the gross cross-section);

$\tau_{Vz,Ed}$ is the shear stress due to the transverse shear force $V_{z,Ed}$ (using the gross cross-section);

$\tau_{t,Ed}$ is the shear stress due to uniform (St. Venant) torsion (using the gross cross-section);

$\tau_{w,Ed}$ is the shear stress due to warping (using the gross cross-section).

8.1.7 Resistance to transverse forces

8.1.7.1 General

(1)P To avoid crushing, crippling or buckling in a web subject to a support reaction or other transverse force applied through the flange, the transverse force F_{Ed} shall satisfy:

$$F_{Ed} \leq R_{w,Rd} \tag{8.17}$$

where:

$R_{w,Rd}$ is the resistance to transverse forces of the web.

(2) The resistance to forces of a web $R_{w,Rd}$ should be obtained as follows:

a) for an unstiffened web:

- for a cross-section with a single web: see Clause 8.1.7.2;
- for any other case, including sheeting: see Clause 8.1.7.3;

b) for a stiffened web: see Clause 8.1.7.4.

(3) Where the load or support reaction is applied through a cleat which prevents distortion of the web and is designed to resist the transverse force, the resistance of the web to the transverse force does not need to 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 connected through their webs, the connections should be located as close to the flanges of the beam as practical.

8.1.7.2 Cross-sections with a single unstiffened web

(1) For a cross-section with a single unstiffened web, as shown in the examples provided in Figure 8.6, the resistance to transverse forces of the web may be determined as specified in (2), provided that the cross-section satisfies the following criteria:

$$\frac{h_w}{t} \leq 200 \tag{8.18}$$

$$\frac{r}{t} \leq 6 \tag{8.19}$$

$$45° \leq \phi \leq 90° \tag{8.20}$$

where:

h_w is the web height between the midlines of the flanges;

r is the internal radius of the corners;

ϕ is the angle of the web relative to the flanges [degrees].

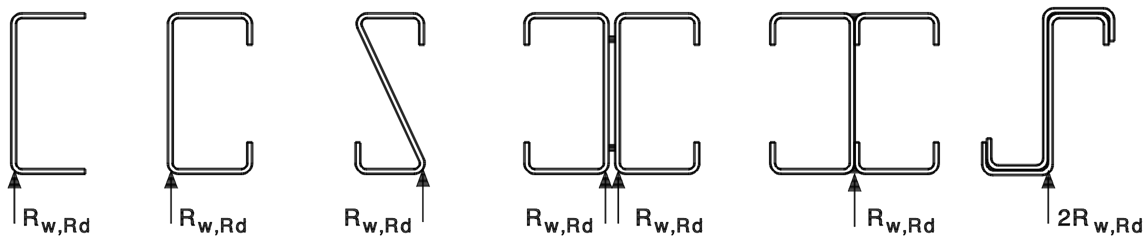
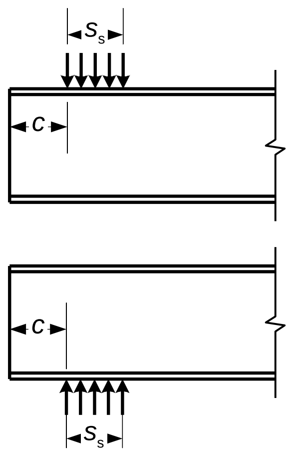


Figure 8.6 - Examples of cross-sections with a single web

(2) For cross-sections which satisfy the criteria specified in (1), the resistance to transverse forces of a web $R_{w,Rd}$ may be determined as shown in Table 8.2 and Table 8.3.

Table 8.2 - Loads and support reactions — cross-sections with a single web – Single transverse force or support reaction

<table>
<tr>
<td>
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(a) the distance to a free end $c \leq 1{,}5\, h_w$

- for a cross-section with stiffened flanges:

$$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[9{,}04 - \frac{h_w/t}{60}\right]\left[1 + 0{,}01\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \quad (8.21)$$

- for a cross-section with unstiffened flanges:

- if $s_s/t \leq 60$:

$$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[5{,}92 - \frac{h_w/t}{132}\right]\left[1 + 0{,}01\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \quad (8.22)$$

- if $s_s/t > 60$:

$$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[5{,}92 - \frac{h_w/t}{132}\right]\left[0{,}71 + 0{,}015\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \quad (8.23)$$

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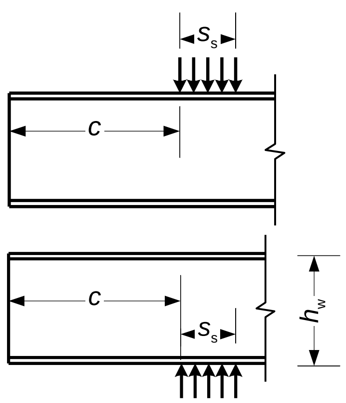
<tr>
<td>
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(b) the distance to a free end $c > 1{,}5\, h_w$

- if $s_s/t \leq 60$:

$$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[14{,}7 - \frac{h_w/t}{49{,}5}\right]\left[1 + 0{,}007\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \quad (8.24)$$

- if $s_s/t > 60$:

$$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[14{,}7 - \frac{h_w/t}{49{,}5}\right]\left[0{,}75 + 0{,}011\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \quad (8.25)$$

</td>
</tr>
</table>

(3) The values of the coefficients k_1 to k_5 should be determined as follows:

$$k_1 = 1{,}33 - 0{,}33\,k \quad (8.26)$$

$$k_2 = 1{,}15 - 0{,}15\,\frac{r}{t} \qquad \text{but} \qquad 0{,}50 \leq k_2 \leq 1{,}0 \quad (8.27)$$

$$k_3 = 0{,}7 + 0{,}3\left(\frac{\phi}{90}\right)^2 \quad (8.28)$$

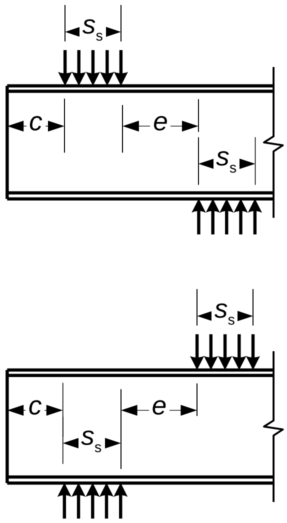
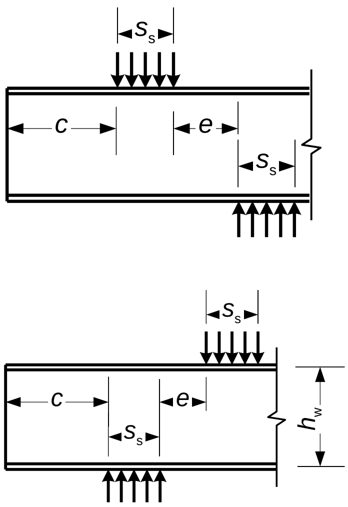
$$k_4 = 1{,}22 - 0{,}22\,k \quad (8.29)$$

$$k_5 = 1{,}06 - 0{,}06\,\frac{r}{t} \qquad \text{but} \qquad k_5 \leq 1{,}0 \quad (8.30)$$

where:

$k = f_{yb} / 228$ with: f_{yb} in N/mm²

Table 8.3 - Loads and support reactions — cross-sections with a single web – Two transverse forces on opposite sides of the web at a spacing $e \leq 1{,}5\, h_w$

	**(a) the distance to a free end $c \leq 1{,}5\, h_w$** $$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[6{,}66 - \frac{h_w/t}{64}\right]\left[1 + 0{,}01\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \quad (8.31)$$
	**(b) the distance to a free end $c > 1{,}5\, h_w$** $$R_{w,Rd} = \frac{k_3 k_4 k_5 \left[21{,}0 - \frac{h_w/t}{16{,}3}\right]\left[1 + 0{,}0013\frac{s_s}{t}\right] t^2 f_{yb}}{\gamma_{M1}} \quad (8.32)$$

(4) If the rotation of the web along its longitudinal edges is prevented either by suitable restraint or because of the section geometry (e.g. in I-beams, as shown in the fourth and fifth example from the left in the Figure 8.6) then the resistance to transverse forces of a web $R_{w,Rd}$ of a cross-section with stiffened or unstiffened flanges may be determined as follows:

- for a single transverse force or support reaction:

  a) $c < 1{,}5\ h_w$ (near or at a free end)

$$R_{w,Rd} = \frac{k_7 \left[8{,}8 + 1{,}1\sqrt{\frac{S_s}{t}}\right] t^2 f_{yb}}{\gamma_{M1}} \tag{8.33}$$

  b) $c > 1{,}5\ h_w$ (far from a free end)

$$R_{w,Rd} = \frac{k_5^*\ k_6 \left[13{,}2 + 2{,}87\sqrt{\frac{S_s}{t}}\right] t^2 f_{yb}}{\gamma_{M1}} \tag{8.34}$$

- for transverse forces or reactions on opposite sides of the web

  a) $c < 1{,}5\ h_w$ (near or at a free end)

$$R_{w,Rd} = \frac{k_{10} k_{11} \left[8{,}8 + 1{,}1\sqrt{\frac{S_s}{t}}\right] t^2 f_{yb}}{\gamma_{M1}} \tag{8.35}$$

  b) $c > 1{,}5\ h_w$ (loads or reactions far from a free end)

$$R_{w,Rd} = \frac{k_8 k_9 \left[13{,}2 + 2{,}87\sqrt{\frac{S_s}{t}}\right] t^2 f_{yb}}{\gamma_{M1}} \tag{8.36}$$

where the values of coefficients k_5^* to k_{11} should be determined as follows:

$$k_5^* = 1{,}49 - 0{,}53k \quad \text{but:} \quad k_5^* \geq 0{,}6 \tag{8.37}$$

$$k_6 = 0{,}88 + \frac{0{,}12t}{1{,}9} \quad \text{with:} \quad t \text{ in [mm]} \tag{8.38}$$

$$k_7 = 1 + \frac{h_w}{(750 - t)} \quad \text{if} \quad \frac{h_w}{t} \leq 150 \tag{8.39}$$

$$k_7 = 1{,}2 \quad \text{if} \quad \frac{h_w}{t} > 150 \tag{8.40}$$

$$k_8 = \frac{1}{k} \quad \text{if} \quad \frac{h_w}{t} \leq 66{,}5 \tag{8.41}$$

$$k_8 = \frac{\left(1{,}1 - \frac{h_w}{665\ t}\right)}{k} \quad \text{if} \quad \frac{h_w}{t} > 66{,}5 \tag{8.42}$$

$$k_9 = 0{,}82 + \frac{0{,}15\ t}{1{,}9} \tag{8.43}$$

$$k_{10} = \frac{\left(0{,}98 - \frac{h_w}{865\ t}\right)}{k} \tag{8.44}$$

$$k_{11} = 0{,}64 + \frac{0{,}31\ t}{1{,}9} \tag{8.45}$$

with:

k $= f_{yb} / 228$ with f_{yb} in [N/mm²];

s_s is the nominal length of stiff bearing. If two transverse forces equal in magnitude act on opposite sides of the web, but are distributed over unequal bearing lengths, the smaller value of s_s should be used.

8.1.7.3 Cross-sections with two or more unstiffened webs

(1) In cross-sections containing two or more webs, including sheeting, see Figure 8.7, the resistance to transverse forces 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 end of the bearing length to a free end is at least 40 mm, see Table 8.4;
- the cross-section satisfies the following criteria:

$$r/t \leq 10 \tag{8.46}$$

$$h_w/t \leq 200 \sin \phi \tag{8.47}$$

$$45° \leq \phi \leq 90° \tag{8.48}$$

where:

h_w is the web height between the midlines of the flanges;

r is the internal radius of the corners;

ϕ is the angle of the web relative to the flanges [degrees].

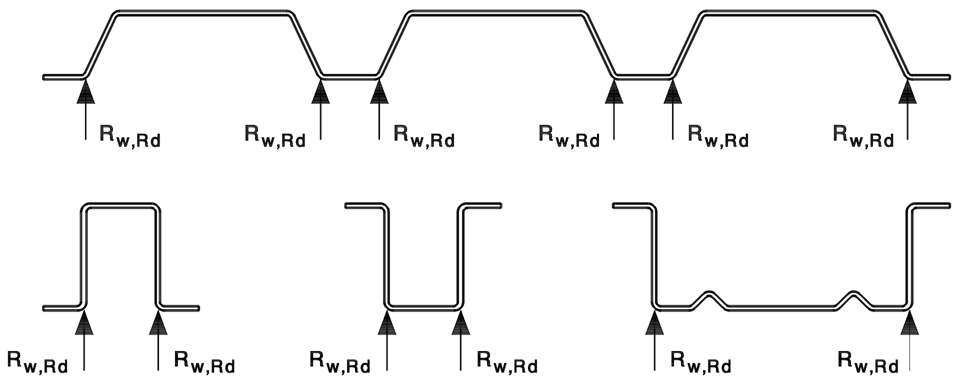


Figure 8.7 - Examples of cross-sections with two or more webs

(2) If both conditions specified in (1) are satisfied, then the resistance to transverse forces $R_{w,Rd}$ of each web of the cross-section should be determined as follows:

$$R_{w,Rd} = \alpha t^2 \sqrt{f_{yb} E} \frac{\left[1 - 0{,}1\sqrt{\frac{r}{t}}\right]\left[0{,}5 + \sqrt{\frac{0{,}02\, l_a}{t}}\right]\left[2{,}4 + \left(\frac{\Phi}{90}\right)^2\right]}{\gamma_{M1}} \tag{8.49}$$

where:

l_a is the effective bearing length dependent on the relevant category as specified in (3);

α is the coefficient dependent on the relevant category as specified in (3).

(3) The values of l_a and α should be obtained from (4) and (5) respectively. The maximum design value for l_a = 200 mm.

- When the support consists of a single cold-formed steel member with a cross-section with one web, and unless more sophisticated analysis or tests are carried out, the value of l_a may be obtained as follows:

$$l_a = min\,(\,17.5\,t_s - 6\text{ mm};\ 50\text{ mm};\ s_s\,) \tag{8.50}$$

where:

t_s is the design thickness of the flange of the support;

s_s is the width of the top flange of the supporting member;

- When the supporting member consists of two cold-formed steel sections with one web each, back-to-back or a nested configuration, l_a may be taken as two times the value of a single section.
- When the support consists of (a) a round tube or (b) a cold-formed steel section with one web subject to torsion, a value of 10 mm should be used for l_a .

The section may be considered subject-to torsion, when its shear centre is outside the area under the top flange or the area above support (for the cases where the loads acts at the top or bottom of the member, respectively) and the torsion is not restrained by other structural elements. For instance, a C-section without anti-sag bars and a span of more than 2 meters or with anti-sag bars and a distance of more than 3 m between the supports and/or the anti-sag bars should be considered to be torsionally unrestrained.

A distinction should be made between two categories (1 or 2) 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 Table 8.4. Category 1 covers cases where the load acts in close proximity to a support or free edge, while Category 2 covers the complementary cases.

(4) The value of the effective bearing length l_a for Category 1 should be taken as:

$$l_a = 10\text{ mm} \tag{8.51}$$

The value of the effective bearing length l_a for Category 2 should be taken as:

$$l_a = s_s \tag{8.52}$$

(5) The value of the coefficient α should be determined as follows:

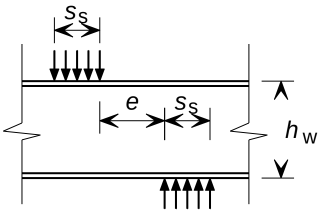
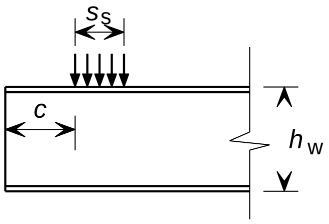
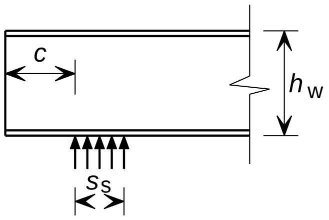
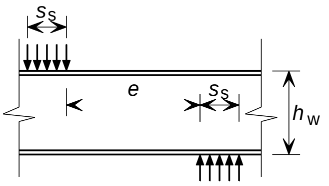
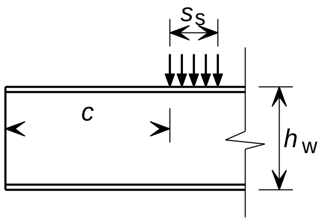
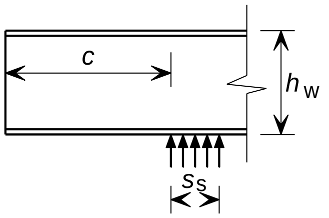
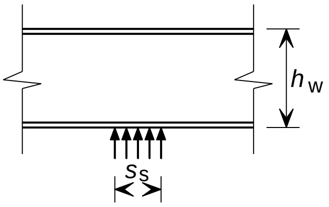
a) for Category 1:

- for sheeting: α = 0,075 (8.53)
- for liner trays and hat sections: α = 0,057 (8.54)

b) for Category 2:

- for sheeting: α = 0,15 (8.55)
- for liner trays and hat sections: α = 0,115 (8.56)

Table 8.4 - Loads and support reactions — categories for cross-sections with two or more webs

	Category 1: • load applied within a distance $e \leq 1{,}5\ h_w$ clear from the nearest support;
	Category 1: • load applied within a distance $c \leq 1{,}5\ h_w$ clear from a free end;
	Category 1: • end support reaction within a distance $c \leq 1{,}5\ h_w$ clear from a free end.
	Category 2: • load applied within a distance $e > 1{,}5\ h_w$ clear from the nearest support;
	Category 2: • load applied within a distance $c > 1{,}5\ h_w$ clear from a free end;
	Category 2: • end support reaction at end support within a distance $c > 1{,}5\ h_w$ clear from a free end;
	Category 2: • internal support reaction with no localized loading applied within a distance of $1{,}5\ h_w$.

8.1.7.4 Stiffened webs

(1) The resistance to transverse forces of a stiffened web may be determined as specified in (2) for cross-sections with longitudinal web stiffeners which are folded in such a way that the two web folds are on opposite sides of the line connecting the end points of the web. These end points are determined as the points of intersection of the midline of the web with the midlines of the flanges, as shown in Figure 8.8. In addition, for (2) to be valid the following condition should be satisfied:

$$2 \leq \frac{e_{max}}{t} < 12 \tag{8.57}$$

where:

e_{max} is the larger eccentricity of the folds relative to the line of the web connecting the end points of the web.

(2) For cross-sections satisfying the conditions specified in (1), the resistance to transverse forces of a stiffened web may be determined by multiplying the resistance of a corresponding unstiffened web, obtained from 8.1.7.2 or 8.1.7.3 as appropriate, by the factor $\kappa_{a,s}$ given by:

$$k_{a,5} = 1{,}45 - 0{,}05\,\frac{e_{max}}{t} \qquad \text{but} \qquad k_{a,5} \leq 0{,}95 + 35000\,t^2\,\frac{e_{min}}{(b_a^2\,s_p)} \tag{8.58}$$

where:

b_d is the developed width of the loaded flange, see Figure 8.8;

e_{min} is the smaller eccentricity of the folds relative to the line connecting the end points of the web;

s_p is the slant height of the plane web element nearest to the loaded flange, see Figure 8.8.

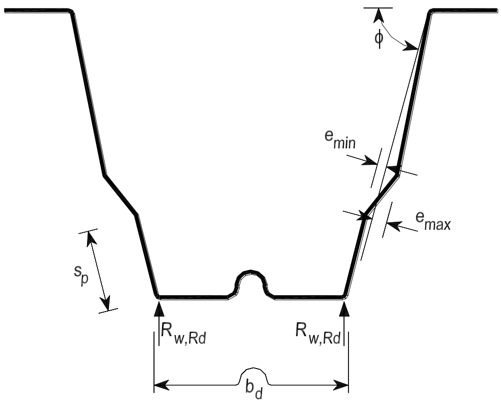


Figure 8.8 - Stiffened webs

8.1.7.5 Concentrated line load and point load

8.1.7.5.1 General

(1) The load distribution (here being understood as the distribution of load at right angles to the direction of span) shall be checked in the case of concentrated loads and line loads, a distinction being made between the direct loading of one or two adjacent ribs (direct load distribution) and indirect load distribution (i.e. loading via load-bearing intermediate systems).

(2) In addition, web-crippling resistance has to be checked.

8.1.7.5.2 Direct load distribution without intermediate systems

(1) Unless a more precise analysis has been carried out, concentrated loads that are applied to one or two adjacent ribs of a profiled sheet at a distance x ≤ L/2 from the support may be assumed to be distributed as specified in Table 8.5 and Figure 8.9 if the load is applied via at least two webs and the load width parallel to the direction of span is at least 50 mm.

Table 8.5 - Load dispersal without intermediate load-distributing systems

Distribution of concentrated load	**Loaded rib C_1 [%]**	**Adjacent rib C_2 [%]**
Both sides	$(352-0{,}8\,b_R)\left(\frac{x}{l}-0{,}5\right)^2+(12+0{,}2\,b_R)$	$(44-0{,}1b_R)\left(1-4\left(\frac{x}{l}-0{,}5\right)^2\right)$
One side (edge rib)	$(240-0{,}6\,b_R)\left(\frac{x}{l}-0{,}5\right)^2+(40+0{,}15\,b_R)$	$(60-0{,}15b_R)\left(1-4\left(\frac{x}{l}-0{,}5\right)^2\right)$
where: l is the span of the profiled sheet, in [m] x is the distance of the concentrated load from the adjacent support, in [m] b_R is the rib width, in [mm]		

(2) The proportion of the load per rib should be taken as:

$$F_{Ri} \leq \frac{C_i}{100} \cdot F \qquad (8.59)$$

where:

C_i is the load dispersal factor according to Table 8.5.

F is the load introduced by direct load distribution, see Figure 8.9

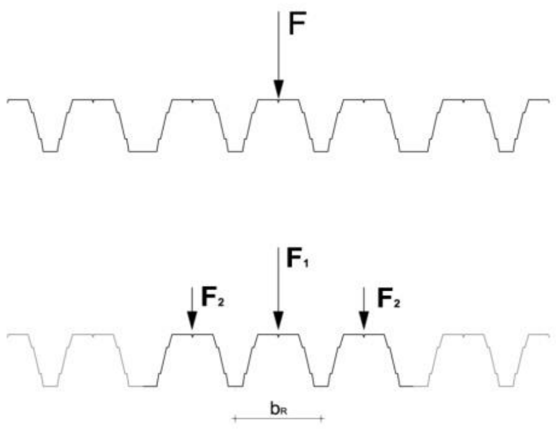


Figure 8.9 - Load distribution without intermediate load-distributing systems

8.1.7.5.3 Direct load distribution with intermediate systems

(1) If distribution is done by hot-rolled or cold-formed steel sections, timber or concrete, the distribution systems have to be designed to show their effectiveness.

8.1.8 Combined tension and bending

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

$$\frac{N_{Ed}}{N_{t,Rd}} + \frac{M_{y,Ed}}{M_{cy,Rd,ten}} + \frac{M_{z,Ed}}{M_{cz,Rd,ten}} \leq 1 \tag{8.60}$$

where:

$N_{t,Rd}$ is the design resistance of a cross-section in uniform tension, see 8.1.2;

$M_{cy,Rd,ten}$ is the design moment resistance of a cross-section using the section modulus on the tension side, if subject only to moment about the y - y axis, see 8.1.4;

$M_{cz,Rd,ten}$ is the design moment resistance of a cross-section using the section modulus on the tension side if subject only to moment about the z - z axis, see 8.1.4.

(2) If $M_{cy,Rd,com} \leq M_{cy,Rd,ten}$ or $M_{cz,Rd,com} \leq M_{cz,Rd,ten}$, the following criterion should also be satisfied:

$$\frac{M_{y,Ed}}{M_{cy,Rd,com}} + \frac{M_{z,Ed}}{M_{cz,Rd,com}} - \frac{N_{Ed}}{N_{t,Rd}} \leq 1 \tag{8.61}$$

where:

$M_{cy,Rd,com}$ is the design moment resistances of a cross-section using the section modulus on the compression side if subject only to a moment about the y - y axis, see 8.1.4;

$M_{cz,Rd,com}$ is the design moment resistances of a cross-section using the section modulus on the compression side if subject only to a moment about the z - z axis, see 8.1.4;

8.1.9 Combined compression and bending

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

$$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{cy,Rd,com}} + \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{cz,Rd,com}} \leq 1 \tag{8.62}$$

where:

$N_{c,Rd}$ is the design resistance of a cross-section in uniform compression, see 8.1.3;

$M_{cy,Rd,com}$ is the design moment resistance of a cross-section using the section modulus on the compression side, if subject only to a moment about the y - y axis, see 8.1.8(2);

$M_{cz,Rd,com}$ is the design moment resistance of a cross-section using the section modulus on the compression side, if subject only to a moment about the z - z axis, see 8.1.8(2).

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

$$\Delta M_{y,Ed} = N_{Ed} \cdot e_{Ny} \tag{8.63}$$

$$\Delta M_{z,Ed} = N_{Ed} \cdot e_{Nz} \tag{8.64}$$

where:

e_{Ny} and e_{Nz} are the shifts of the y-y and z-z centroidal axis in uniform compression, see 8.1.3(3).

(3) If $M_{cy,Rd,ten} \leq M_{cy,Rd,com}$ or $M_{cz,Rd,ten} \leq M_{cz,Rd,com}$ the following criterion should also be satisfied:

$$\frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{cy,Rd,ten}} + \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{cz,Rd,ten}} - \frac{N_{Ed}}{N_{t,Rd}} \leq 1 \tag{8.65}$$

where:

$M_{cy,Rd,ten}$, $M_{cz,Rd,ten}$ are the design moment resistances defined in 8.1.8.

(4) Alternatively, Clause 8.2.5 and Formulae (8.76) and (8.77) may be used as relevant with $\chi_y = \chi_z = \chi_{LT} =$ 1,0 and $C_{x,y} = C_{x,z} = C_{x,LT} = 1{,}0$.

8.1.10 Combined shear force, axial force and bending moment

(1) For cross-sections subject to the combined action of an axial force N_{Ed}, a bending moment M_{Ed} and a shear force V_{Ed} the resistance to account for the presence of the shear force need not to be reduced if $V_{Ed} \leq 0{,}5\ V_{w,Rd}$.

If $V_{Ed} > 0{,}5\ V_{w,Rd}$ then the following equations should be satisfied:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)\left(\frac{2\,V_{Ed}}{V_{w,Rd}} - 1\right)^2 \leq 1 \tag{8.66}$$

where:

N_{Rd} is the design resistance of a cross-section in uniform tension or compression, see 8.1.2 or 8.1.3;

$M_{y,Rd}$ is the design moment resistance of the cross-section, see 8.1.4;

$V_{w,Rd}$ is the design shear resistance of the web, see 8.1.5(1);

$M_{f,Rd}$ is the moment resistance of a cross-section consisting of the effective area of the flanges only, see EN 1993-1-5;

$M_{pl,Rd}$ is the plastic moment ~~of~~ resistance of the cross-section, see EN 1993-1-5.

For members and sheeting with more than one web $V_{w,Rd}$ is the sum of the resistances of the webs, see EN 1993-1-5.

8.1.11 Combined bending moment and localized transverse load or support reaction

(1) Cross-sections subject to the combined action of a bending moment M_{Ed} and a localized transverse force or support reaction F_{Ed} should satisfy the following formulae, unless otherwise justified:

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1 \tag{8.67}$$

$$\frac{F_{Ed}}{R_{w,Rd}} \leq 1 \tag{8.68}$$

$$\frac{M_{Ed}}{M_0} + \frac{F_{Ed}}{R_0} \leq 1 \tag{8.69}$$

with:

$$M_0 = 1{,}25\, M_{c,Rd} \tag{8.70}$$

$$R_0 = 1{,}25\, R_{w,Rd} \tag{8.71}$$

where:

$M_{c,Rd}$ is the moment resistance of the cross-section, see 8.1.4.1(1);

$R_{w,Rd}$ is the resistance to transverse forces of the web, see 8.1.7.

The relations of Formulae (8.67) to (8.71) are shown in Figure 8.10.

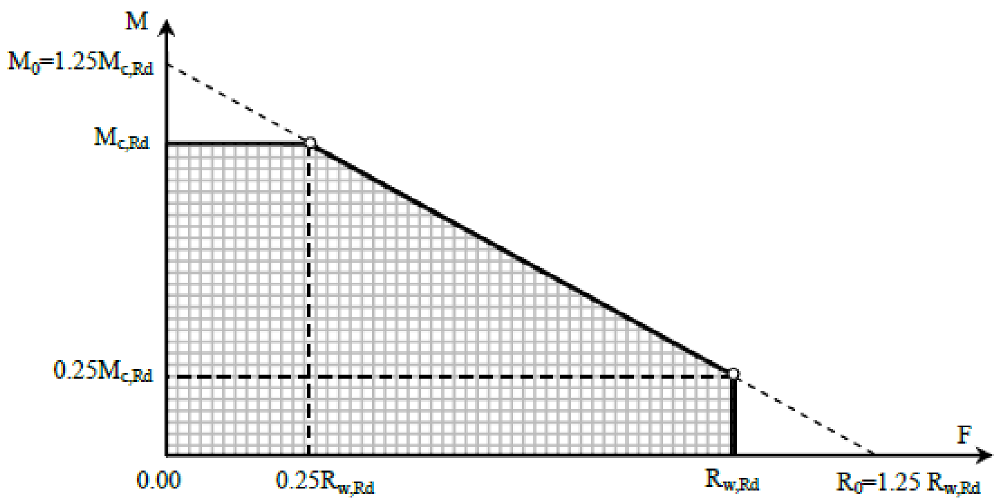


Figure 8.10 - Resistance to combined bending moment and localized transverse load or support reaction

(2) For members and sheeting with more than one web, $R_{w,Rd}$ is the sum of the resistances to transverse forces of the individual webs.

8.2 Buckling resistance

8.2.1 General

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

(2) The effects of local and distortional buckling should be taken into account as specified in Clause 7.5.

8.2.2 Flexural buckling

(1) The design buckling resistance $N_{b,Rd}$ for flexural buckling should be obtained from EN 1993-1-1 using the appropriate buckling curve from Table 8.6 according to the type of cross-section, axis of buckling and yield strength, see (3).

(2) The buckling curve for a cross-section not included in Table 8.6 may be obtained by analogy.

(3) The buckling resistance of a closed built-up cross-section should be determined using either:

- buckling curve b in association with the basic yield strength f_{yb} of the flat sheet material;
- buckling curve c in association with the average yield strength f_{ya} after cold-forming, determined as specified in 5.2.3, provided that $A_{eff} = A$.

8.2.3 Torsional buckling and torsional-flexural buckling of members subject to compression

(1) For members with point-symmetric open cross-sections (e.g Z-purlin with equal flanges), account should be taken of the possibility that the resistance of the member to torsional buckling might be less than its resistance to flexural buckling.

(2) For members with mono-symmetric open cross-sections, see Figure 8.11, account should be taken of the possibility that the resistance of the member to torsional-flexural buckling might be less than its resistance to flexural buckling.

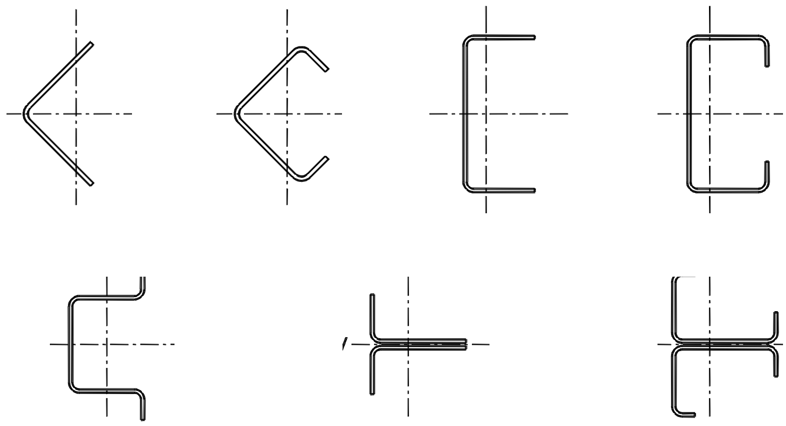
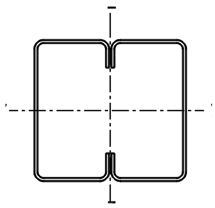
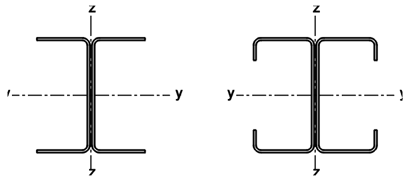
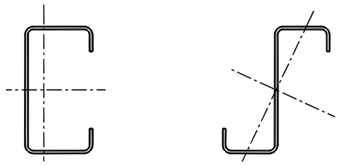
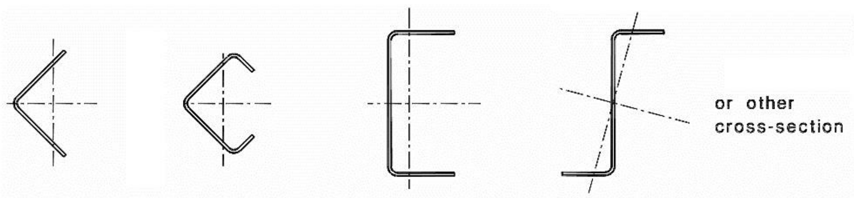


Figure 8.11 - Monosymmetric cross-sections susceptible to torsional-flexural buckling

(3) Members with an asymmetric open cross-section should be designed for torsional-flexural buckling.

(4) The design buckling resistance $N_{b,Rd}$ for torsional or torsional-flexural buckling should be obtained from EN 1993-1-1: 2018, 8.3.1.1 using the relevant buckling curve for buckling about the z-z axis obtained from Table 8.6.

Table 8.6 - Appropriate buckling curve for various types of cross-section

Type of cross-section		Buckling about axis	Buckling curve
	if f_{yb} is used	any	b
	if f_{ya} is used [a]	any	c
		y - y z - z	a b
		any	b
		any	c
[a] The average yield strength f_{ya} should not be used unless $A_{eff} = A$			

(5) The elastic critical buckling load $N_{cr,T}$ for torsional buckling of a member subject to compression should be determined as follows:

$$N_{cr,T} = \frac{1}{i_0^2}\left(GI_T + \frac{\pi^2 EI_w}{l_T^2}\right) \tag{8.72}$$

with:

$$i_0^2 = i_y^2 + i_z^2 + y_0^2 + z_0^2 \tag{8.73}$$

where:

G is the shear modulus;

I_t is the torsional 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_o, z_o are the shear centre co-ordinates with respect to the centroid of the gross cross-section.

(6) For doubly symmetric cross-sections ($y_o = z_o = 0$), the elastic critical buckling load N_{cr} should be determined as follows:

$$N_{cr} = N_{cr,i} \tag{8.74}$$

where:

$N_{cr,i}$ is the minimum of $N_{cr,y}$, $N_{cr,z}$, $N_{cr,T}$.

(7) For cross-sections that are symmetrical about the y - y axis ($z_o = 0$), the elastic critical buckling load $N_{cr,TF}$ for torsional-flexural buckling should be determined from:

$$N_{cr,TF} = \frac{N_{cr,y}}{2\beta}\left(1 + \frac{N_{cr,T}}{N_{cr,y}} - \sqrt{\left(1 - \frac{N_{cr,T}}{N_{cr,y}}\right)^2 + 4\left(\frac{y_0}{i_0}\right)^2 \frac{N_{cr,T}}{N_{cr,y}}}\right) \tag{8.75}$$

with:

$$\beta = 1 - \left(\frac{y_0}{i_0}\right)^2 \tag{8.76}$$

where:

$N_{cr,y}$ is the elastic critical load for flexural buckling about the y - y axis

$N_{cr,T}$ is the elastic critical load for torsional buckling, see (5)

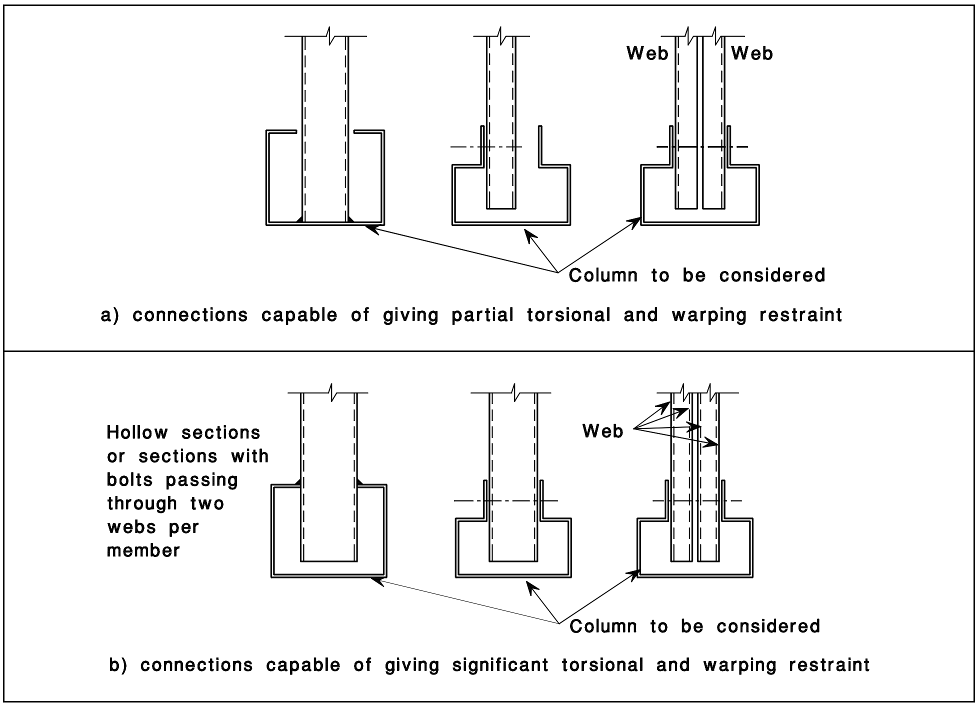
y_o, i_0 are defined in (5)

(8) The buckling length l_T for torsional or flexural-torsional buckling should be determined taking into account the degree of torsional and warping restraint present at each end of the length L_T.

(9) The value of l_T / L_T may be taken as follows:

- 1,0 for connections which provide partial restraint against torsion and warping, see Table 8.7
- 0,7 for connections which provide significant restraint against torsion and warping, see Table 8.7.

Table 8.7 - Torsional and warping restraint provided by connections



8.2.4 Lateral-torsional buckling of members subject to bending

(1) The design buckling resistance of a member in bending which is susceptible to lateral-torsional buckling should be determined according to EN 1993-1-1: 2018, Clause 8.3.2, using buckling curve *b*.

(2) This method should not be used for sections in which the principal axes of the effective cross-section have significantly rotated compared to those of the gross cross-section.

8.2.5 Bending and axial compression

(1) The resistance against combined axial force and bending moment may be obtained from a second-order analysis of the member as specified in EN 1993-1-1, based on the properties of the effective cross-section obtained from 7.5. See also 7.3.

(2) As an alternative to (1) the following criterion should be satisfied in each cross-section along the member:

$$\left(C_{x,y}\frac{N_{Ed}}{\chi_y\, N_{c,Rd}}\right)^{A_y}+\frac{M_{y,Ed}}{M_{y,Rd}}+\left(\frac{M_{z,Ed}}{M_{z,Rd}}\right)^{D_y}\leq 1 \qquad (8.76)$$

$$\left(C_{x,z}\frac{N_{Ed}}{\chi_z\, N_{Rd}}\right)^{A_z}+\left(C_{x,LT}\frac{M_{y,Ed}}{\chi_{LT}\, M_{y,Rd}}\right)^{B_z}+\left(\frac{M_{z,Ed}}{M_{z,Rd}}\right)^{D_z}\leq 1 \qquad (8.77)$$

where:

N_{ed} is the design value of the compressive force;

$M_{y,Ed}$, $M_{z,Ed}$ are the design values of the bending moments about the *y-y*-axis and the *z-z*-axis along the members, including $\Delta M_{ed} = N_{ed}\, e_N$ due to the shift of the centroid of the effective cross-section, if $A_{eff} < A$ (for members with fixed ends: $\Delta M_{ED} = 0$);

- For beam-columns with pinned ends and for members in non-sway frames, M_{Ed} is the first order bending moment.
- For members in frames subject to sway, M_{Ed} is the second order bending moment;

χ_y, χ_z are the reduction factors for flexural buckling according to 8.2.2. In the case of torsional-flexural buckling, χ_y should be replaced with χ_{LT} according to 8.2.3.

χ_{LT} is the reduction factor due to lateral-torsional buckling according to 8.2.4

N_{Rd} is the axial design resistance of the cross-section based on A_{eff}-which are not fully effective cross-sections

$M_{y,Rd}$, $M_{z,Rd}$ are the bending moment resistances of the cross section about the *y-y*-axis and the *z-z* axis, respectively, according to 8.1.4

A_y, A_z, B_z, D_y, D_z are exponents in the interaction formulae, determined according to Table 8.8. They depend on the reduction factors χ and the ratios μ_y and μ_z given in Table 8.8;

$C_{x,y}$, $C_{x,z}$, $C_{x,LT}$ are the interaction factors according to Table 8.9. They depend on the location of the cross-section under consideration.

(3) For asymmetrical sections both the tension and the compression side of bending edges should be checked using the appropriate resistances. In the expressions for the exponents in Table 8.8 the *larger* of μ_y and μ_z for the two sides should be used. If the resistance on the tension side is less than the resistance on the compression side then criteria (8.76) and (8.77) should also be satisfied with:

$M_{y,Rd} = M_{y,Rd,ten}$, see 8.1.8

$M_{z,Rd} = M_{z,Rd,ten}$, see 8.1.8,

$A_y = A_z = 1{,}0$, see Table 8.8

In addition, when determining the interaction factors, $-\chi_y$, $-\chi_z$ and $-\chi_{LT}$ should be used in the first term of the expressions for $C_{x,y}$, $C_{x,z}$ and $C_{x,LT}$, see Table 8.9.

(4) The *y*- and *z*-axis are principal axes.

Table 8.8 - Exponents to be used in the interaction Formulae (8.76) and (8.77)

Exponents	Design resistance of the cross-section in bending about the relevant axis according to 8.1.4.1			Conservative for all cross-sections
	$M_{Rd} \geq W_{el} f_{yb} / \gamma_{M0}$		$M_{Rd} < W_{el} f_{yb} / \gamma_{M0}$	
A_y	$\chi_y \cdot \min(\mu_y^2; 1{,}56)$	but $A_y \geq 1{,}0$	1,0	1,0
A_z	$\chi_z \cdot \min(\mu_y^2 \mu_z^2; 2{,}0)$	but $A_z \geq 1{,}0$	1,0	1,0
B_z	$\min(\mu_z^2; 1{,}56)$		1,0	1,0
D_y	$\chi_y \cdot \min(\mu_y^4; 2{,}0)$	but $D_y \geq 0{,}8$		0,8
D_z	$\chi_z \cdot \min(\mu_y^2; 1{,}56)$	but $D_z \geq 0{,}8$		0,8
where:	$\mu_y = \dfrac{M_{y,Rd}}{W_{el,y} f_{yb} / \gamma_{M0}}$	but $\mu_y \geq 1{,}0$	$\mu_z = \dfrac{M_{z,Rd}}{W_{el,z} f_{yb} / \gamma_{M0}}$	but $\mu_z \geq 1{,}0$

Table 8.9 - Interaction factors $C_{x,y}$, $C_{x,z}$ and $C_{x,LT}$

Factor	For a specific cross-section location along the member	Conservative for all cross-section locations
$C_{x,y}$	$\chi_y + (1 - \chi_y) \sin \dfrac{\pi x_s}{L_{cr,y}}$	1,0
$C_{x,z}$	$\chi_z + (1 - \chi_z) \sin \dfrac{\pi x_s}{L_{cr,z}}$	1,0
$C_{x,LT}$	$\chi_{LT} + (1 - \chi_{LT}) \sin \dfrac{\pi x_s}{L_{cr,LT}}$	1,0

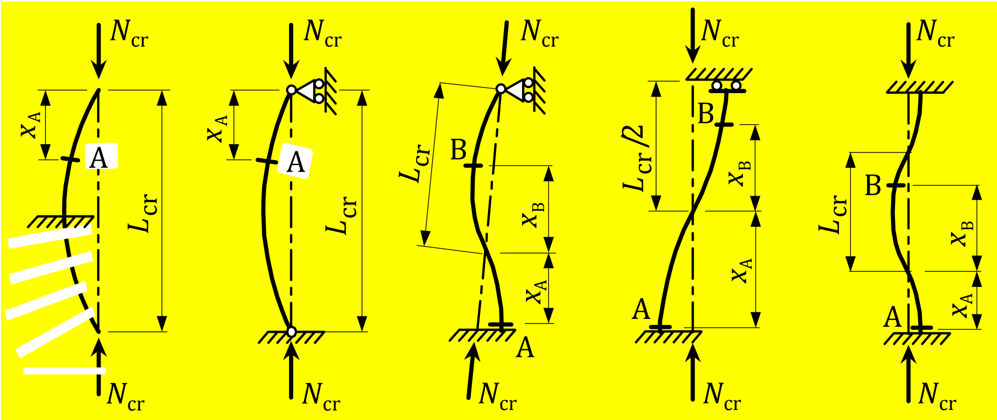
where:

$L_{cr,y}$, $L_{cr,z}$, $L_{cr,LT}$ are the buckling lengths for relevant buckling mode

x_s ($= x_A$ or x_B) is the distance from the cross-section under consideration to a simple support or a point of contraflexure in the elastically buckled shape of the relevant mode, see Figure 8.12 for examples.

For end moments ($M_{Ed,1} > M_{Ed,2}$), the distance x_s to the design section may be obtained from:

$$\cos\left(\frac{\pi x_s}{L_{cr}}\right) = \frac{(M_{Ed,1} - M_{Ed,2})}{M_{Rd}} \cdot \frac{N_{Rd}}{N_{Ed}} \cdot \frac{1}{\pi \left(\dfrac{1}{\chi} - 1\right)} \qquad \text{but } x_s \geq 0$$



NOTE: In these examples cross-sections A and B (marked with transverse line) are the subjects of the design check.

Figure 8.12 - Buckling length L_{cr} and definition of x_s (=x_A or x_B)

8.3 Bending and axial tension

(1) The Formulae for members subject to combined bending and axial compression in 8.2.5 are applicable.

9 Serviceability limit states

9.1 General

(1) The rules for the serviceability limit states given in EN 1993-1-1 also apply to cold-formed members and sheeting.

(2) The properties of the effective cross-section for serviceability limit states obtained from Clause 7.1 should be used in all serviceability limit state calculations for cold-formed members and sheeting.

(3) Alternatively, the second moment of area may be calculated by interpolation between the gross cross-section and the effective cross-section values using the following Formula:

$$I_{fic} = I_{gr} - \frac{\sigma_{gr}}{\sigma}\left(I_{gr} - I(\sigma)_{eff}\right) \tag{9.1}$$

where:

I_{gr} is the second moment of area of the gross cross-section;

σ_{gr} is the maximum compressive bending stress in the serviceability limit state, based on the gross cross-section (positive in Formula);

$I(\sigma)_{eff}$ is the second moment of area of the effective cross-section with allowance for local buckling calculated for a maximum stress $\sigma \geq \sigma_{gr}$, in which the maximum stress is the largest absolute value of stresses within the calculation length considered.

(4) The effective second moment of area I_{eff} (or I_{fic}) may be taken as variable along the span. Alternatively a uniform value may be used, based on the maximum absolute span moment due to serviceability loading.

9.2 Plastic deformation

(1) In case of plastic global analysis the combination of support moment and support reaction at an internal support should not exceed 0,9 times the combined design resistance, determined using $\gamma_{M,ser}$, see Clause 4(5).

(2) The combined design resistance may be determined from 8.1.11, but using the effective cross-section for serviceability limit states and $\gamma_{M,ser}$.

9.3 Deflections

(1) The deflections may be calculated assuming elastic behaviour.

(2) The influence of slip in the connections (for example in the case of continuous beam systems with sleeves and overlaps) should be considered in the calculation of deflections, forces and moments.

9.4 Walkability of trapezoidal sheeting

9.4.1 Walkability during installation

(1) During installation, i.e. not finally fixed, the profiled sheeting may only be walked on in order to install the roof.

(2) If not otherwise specified, the profiled sheeting should only be walked on if load-dispersal measures are adopted (e.g. wooden planks in accordance with strength class C24 with a cross-section of 4 x 24 cm and a length greater than 3,0 m).

(3) If the sheeting span does not exceed the limiting value L_{lim} determined in tests according to Annex A.4.6, then load-dispersal measures may be omitted.

9.4.2 Walkability after installation

(1) After installation, the profiled sheeting may only be walked on by individuals for the purpose of its maintenance and cleaning.

(2) If not otherwise specified, the profiled sheeting should only be walked on if load-dispersal measures are adopted (e.g. wooden planks in accordance with strength class C24 with a cross-section of 4 x 24 cm and a length greater than 3,0 m).

(3) If the sheeting span does not exceed the limiting value L_{lim} determined in tests according to Annex A.4.6, then load-dispersal measures may be omitted.

(4) For profiled sheeting with multi-span supports, load-dispersal measures may be omitted for sheeting spans up to 25% larger than the limiting values determined in tests.

(5) It is advisable to install walkways for access to units requiring regular maintenance or operational elements (e.g. continuous roof lights, chimneys, heating plants and photovoltaic panels).

10 Design of joints

10.1 General

(1) For design assumptions and requirements of joints see EN 1993-1-8.

(2) The following rules apply to core thickness $t_{cor} \leq 4$ mm, not covered by EN 1993-1-8.

10.2 Splices and end connections of members subject to compression

(1) Splices and end connections in members that are subject to compression, should 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_{Ed} and the internal moments $M_{y,Ed}$ and $M_{z,Ed}$ obtained from the global analysis.

(2) In the absence of a second-order analysis of the member, this additional moment ΔM_{Ed} should be taken as acting about the cross-sectional axis that gives the smallest value of the reduction factor χ for flexural buckling, see 8.2.2(1), with a value determined from:

$$\Delta M_{Ed} = N_{Ed}\left(\frac{1}{\chi} - 1\right)\frac{W_{eff}}{A_{eff}}\, sin\frac{\pi a}{l} \tag{10.1}$$

where:

- A_{eff} is the area of the effective 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.

Splices and end connections should be designed to resist an additional internal shear force as follows:

$$\Delta V_{Ed} = \frac{\pi\, N_{Ed}}{l}\left(\frac{1}{\chi} - 1\right)\frac{W_{eff}}{A_{eff}} \tag{10.2}$$

(3) Splices and end connections should be designed in such a way that load may 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.

10.3 Connections with mechanical fasteners

(1) Connections with mechanical fasteners should be compact in shape. The positions of the fasteners should be arranged to provide sufficient room for satisfactory assembly and maintenance.

NOTE: EN 1993-1-8 gives detailed information on these requirements.

(2) 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) For design by calculation the resistances of fastenings subject to predominantly static loads should be determined from:

- Table 10.2 for blind rivets;
- Table 10.3 for self-tapping screws;
- Table 10.4 for cartridge fired pins;
- Table 10.5 for bolts.

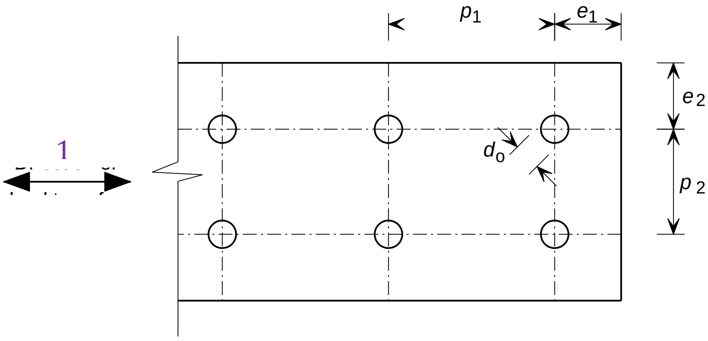
For determining the design resistance of mechanical fasteners by testing see 12 and Annex A.

(5) In Tables 10.2 to 10.5 the meanings of the symbols should be taken as follows:

A_s	the tensile stress area of a fastener;
A_{net}	the net cross-sectional area of the connected part;
β_{Lf}	the reduction factor for long joints according to EN 1993-1-8;
d	the nominal diameter of the fastener;
d_o	the nominal diameter of the hole;
d_w	the diameter of the washer or the head of the fastener;
e_1	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 10.1;
e_2	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 10.1;
f_{ub}	the ultimate tensile strength of the fastener material;
$f_{u,sup}$	the ultimate tensile strength of the supporting member into which a screw is fixed;
n	the number of sheets that are fixed to the supporting member by the same screw or pin;
n_f	the number of mechanical fasteners in one connection;
p_1	the spacing centre-to-centre of fasteners in the direction of load transfer, see Figure 10.1;
p_2	the spacing centre-to-centre of fasteners in the direction perpendicular to the direction of load transfer, see Figure 10.1;
t_{I}	for fastenings with rivets, screws or cartridge fired pins: the thickness of the component dicectly underneath the head of the fastener (the swage head in the case of blind rivets);
t_{II}	for fastenings with rivets, screws or cartridge fired pins: the thickness of the second component of a connection (usually the supporting structure).

(5) The partial factor γ_M for calculating the design resistances of mechanical fasteners should be taken as γ_{M2}, see 4.(3)P:

~~**NOTE:** The value γ_{M2} may be given in the National Annex. The value γ_{M2} = 1,25 is recommended.~~



Key

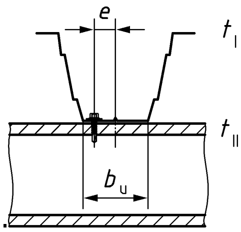
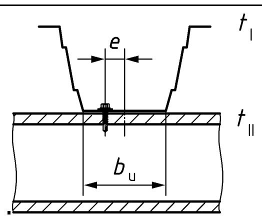
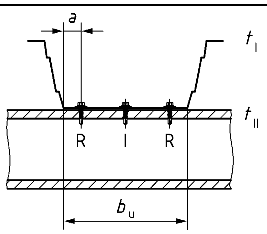
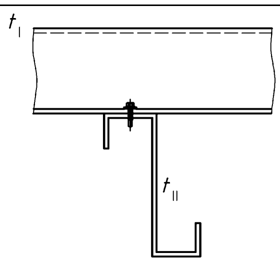
1 Direction of load transfer

Figure 10.1 - End distance, edge distance and spacings for fasteners and spot welds

(6) 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 tests.

(7) The pull-through resistances given in Tables 10.2 to 10.4 for rivets, self-tapping screws and cartridge fired pins should be reduced if the fasteners are not located centrally in the troughs of the sheeting, see Table 10.1.

Table 10.1 - Eccentric attachments

Case	Requirement	Reduction factor for t_I < 1,25 mm
	$e \leq b_U/4$ $b_U \leq 150$ mm	1,0
	$e > b_U/4$ $b_U \leq 150$ mm	0,9
	$0 < e \leq b_U/4$ 150 mm < $b_U \leq 265$ mm	0,7
	$0 < e \leq b_U/2$ 150 mm < $b_U \leq 265$ mm	0,5
	If b_U > 265 mm, at least two fasteners are necessary	for: I 0,0 Ra ≤ 75 mm: 0,7 Ra > 75 mm: 0,35
	Linear profile cross sections such as Z-, C- or Σ-section	see 10.1.6 for calculation of forces in fasteners

(8) For a fastener loaded in combined shear and tension, the resistance of the fastener to combined shear and tension may be verified as follows:

$$\frac{F_{t,Ed}}{\min\left(F_{p,Rd}, F_{o,Rd}\right)} + \frac{F_{v,Ed}}{\min\ \left(F_{b,Rd}, F_{n,Rd}\right)} \leq 1 \qquad (10.3)$$

where:

$F_{\mathrm{p,Rd}}$, $F_{\mathrm{o,Rd}}$, $F_{\mathrm{b,Rd}}$ and $F_{\mathrm{n,Rd}}$ are the design resistances of the fastener calculated using Tables 10.2 to 10.5

(9) The gross section distortion may be neglected if the design resistance is obtained from Tables 10.2 to 10.5, provided that the fastening is through a flange not more than 150 mm wide.

(10) The diameter of 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.

(11) For long joints a reduction factor β_{Lf} should be taken into account according to EN 1993-1-8: 2020, Clause 5.8.

(12) The design rules for blind rivets are valid only if the diameter of the hole is not more than 0,1 mm larger than the diameter of the rivet.

(13) For the bolts M12 and M14 with the hole diameters 2 mm larger than the bolt diameter, reference is made to EN 1993-1-8.

Table 10.2 - Design resistances for blind rivets [a]

Rivets loaded in shear:
Bearing resistance: $F_{b,Rd} = \alpha f_u d t_I / \gamma_{M2}$ but $F_{b,Rd} \leq f_u e_1 t_I / (1{,}2\, \gamma_{M2})$ in which α is given by the following: • if $t_I = t_{II}$: $\alpha = 3{,}6\sqrt{t_1/d}$ but $\alpha \leq 2{,}1$ • if $t_{II} \geq 2{,}5\, t_I$: $\alpha = 2{,}1$ • if $t_I < t_{II} < 2{,}5\, t_I$: 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 *1) and $F_{v,Rd} = F_{v,Rk} / \gamma_{M2}$
Conditions: [d] $F_{v,Rd} \geq 1{,}2\, F_{b,Rd} / (n_f \beta_{Lf})$ or $F_{v,Rd} \geq 1{,}2\, F_{n,Rd}$
Rivets loaded in tension: [b]
Pull-through resistance: Pull-through resistance $F_{p,Rd}$ to be determined by testing *1). Pull-out resistance: Pull-out resistance $F_{o,Rd}$ to be determined by testing *1). Tension resistance: Tension resistance $F_{t,Rd}$ to be determined by testing *1)
Conditions: $F_{t,Rd} \geq \Sigma F_{p,Rd}$
Range of validity: [c]
$e_1 \geq 1{,}5\, d$ $p_1 \geq 3\, d$ $2{,}6 \text{ mm} \leq d \leq 6{,}4 \text{ mm}$ $e_2 \geq 1{,}5\, d$ $p_2 \geq 3\, d$ $f_u \leq 550 \text{ N/mm}^2$
[a] It is assumed that the thinnest sheet is next to the preformed head of the blind rivet, i.e. $t_I \leq t_{II}$. Otherwise $t_I = t_{II}$ shall be assumed for design. [b] Blind rivets are not usually used in tension. [c] Blind rivets may be used beyond this range of validity if the resistance is determined from the results of tests. [d] The required conditions should be fulfilled when deformation capacity of the connection is required. When these conditions are not fulfilled it should be proved that the deformation capacity will be provided by other parts of the structure.

NOTE:*1) Further information about the shear resistance of blind rivets loaded in shear and about the pull-through resistance and tensile resistance of blind rivets loaded in tension can be set by the National Annex for use in a country.

Table 10.3 - Design resistances for self-tapping screws [a]

Screws loaded in shear:

Bearing resistance:

$F_{b,Rd} \quad = \quad \alpha f_u d t_I / \gamma_{M2}$

in which α is given by the following:

- if $t_I = t_{II}$: $\quad \alpha = \alpha_1 = 3{,}2\sqrt{t_1/d} \quad$ but $\alpha \leq 1{,}7$
- if $t_{II} \geq 2{,}5\, t_I$: $\quad \alpha = \alpha_{2,5} = 1{,}7$
- if $t_I \leq t_{II} \leq 2{,}5\, t_I$: $\quad \alpha = \left(\alpha_1 + \left(\alpha_{2,5} - \alpha_1\right)\left(\frac{t_{II}}{t_I} - 1\right)\frac{1}{1{,}5}\right)\frac{t_I + t_{II}}{2} \quad$ but $\alpha \leq \alpha_{2,5} = 1{,}7$

Net-section resistance: $\quad F_{n,Rd} \quad = \quad A_{net} f_u / \gamma_{M2}$

Shear resistance: $\quad$ Shear resistance $F_{v,Rd}$ to be determined by testing *2)

$F_{v,Rd} \quad = \quad F_{v,Rk} / \gamma_{M2}$

Conditions: [d] $\quad F_{v,Rd} \geq 1{,}2\, F_{b,Rd} \quad$ or $\quad \Sigma F_{v,Rd} \geq 1{,}2\, F_{n,Rd}$

Screws loaded in tension:

Pull-through resistance: [b]

- for static loads: $\quad F_{p,Rd} \quad = \quad d_w t_I f_u / \gamma_{M2}$
- for screws subject to wind loads and combination of wind loads and static loads:

$F_{p,Rd} \quad = \quad 0{,}67\, d_w t_I f_u / \gamma_{M2}$

Pull-out resistance:

- if $t_{II} / s < 1$: $\quad F_{o,Rd} \quad = \quad 0{,}45\, d\, t_{II} f_{u,sup} / \gamma_{M2}$ (s is the thread pitch)
- if $t_{II} / s \geq 1$: $\quad F_{o,Rd} \quad = \quad 0{,}65\, d\, t_{II} f_{u,sup} / \gamma_{M2}$

Tension resistance: $\quad$ Tension resistance $F_{t,Rd}$ to be determined by testing *2).

Conditions: [d] $\quad F_{t,Rd} \geq \Sigma F_{p,Rd} \quad$ or $\quad F_{t,Rd} \geq F_{o,Rd}$

Range of validity: [c]

Generally: $\quad e_1 \geq 3\, d \qquad p_1 \geq 3\, d \qquad 3{,}0 \text{ mm} \leq d \leq 8{,}0 \text{ mm}$

$e_2 \geq 1{,}5\, d \qquad p_2 \geq 3\, d$

$f_u \leq 550 \text{ N/mm}^2$

For tension: $\quad 0{,}5 \text{ mm} \leq t_I \leq 1{,}5 \text{ mm} \quad$ and $\quad t_{II} \geq 0{,}9 \text{ mm}$

[a] It is assumed that the thinnest sheet is next to the head of the screw, i.e. $t_I \leq t_{II}$. Otherwise $t_I = t_{II}$ shall be assumed for design.

[b] These values assume that the washer has sufficient rigidity to prevent it from being deformed appreciably and/or pulled over the head of the fastener.

[c] Self-tapping screws may be used beyond this range of validity if the resistance is determined from the results of tests.

[d] The required conditions should be fulfilled when deformation capacity of the connection is required. When these conditions are not fulfilled there should be proved that the deformation capacity will be provided by other parts of the structure.

NOTE:*2) Further information about the shear resistance of self-tapping screws loaded in shear and about the tensile resistance of self-tapping screws loaded in tension can be set by the National Annex for use in a country.

Table 10.4 - 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 *3) $F_{v,Rd} = F_{v,Rk} / \gamma_{M2}$
Conditions: [c] $F_{v,Rd} \geq 1{,}5 \Sigma F_{b,Rd}$ or $\Sigma F_{v,Rd} \geq 1{,}5 F_{n,Rd}$
Pins loaded in tension:
Pull-through resistance: [a] • for static loads: $F_{p,Rd} = d_w t f_u / \gamma_{M2}$ • for wind loads and combination of wind loads and static loads: $F_{p,Rd} = 0{,}5 d_w t f_u / \gamma_{M2}$ Pull-out resistance: Pull-out resistance $F_{o,Rd}$ to be determined by testing *3) Tension resistance: Tension resistance $F_{t,Rd}$ to be determined by testing *3)
Conditions: [c] $F_{o,Rd} \geq \Sigma F_{p,Rd}$ or $F_{t,Rd} \geq F_{o,Rd}$
Range of validity: [b]
Generally: $e_1 \geq 4{,}5 d$ $3{,}7$ mm $\leq d \leq 6{,}0$ mm $e_2 \geq 4{,}5 d$ for $d = 3{,}7$ mm: $t_{II} \geq 4{,}0$ mm $p_1 \geq 4{,}5 d$ for $d = 4{,}5$ mm: $t_{II} \geq 6{,}0$ mm $p_2 \geq 4{,}5 d$ for $d = 5{,}2$ mm: $t_{II} \geq 8{,}0$ mm $f_u \leq 550$ N/mm² For tension: $0{,}5$ mm $\leq t_I \leq 1{,}5$ mm $t_{II} \geq 6{,}0$ mm
[a] These values assume that the washer has sufficient rigidity to prevent it from being deformed appreciably and/or pulled over the head of the fastener. [b] Cartridge fired pins may be used beyond this range of validity if the resistance is determined from the results of tests. [c] The required conditions should be fulfilled when deformation capacity of the connection is required. When these conditions are not fulfilled there should be proved that the deformation capacity will be provided by other parts of the structure.

NOTE:*3) Further information about the shear resistance of cartrige fired pins loaded in shear and about the pull-out resistance and the tensile resistance of cartridge fired pins loaded in tension can be set by the National Annex for use in a country.

Table 10.5 - Design resistances for bolts

Bolts loaded in shear:

Bearing resistance: [b]

$F_{b,Rd} = 2{,}5 \alpha_b \, k_t \, f_u \, d \, t \, / \, \gamma_{M2}$

with:

α_b is the smallest of 1,0 or $e_1 / (3d)$;

$k_t = (0{,}8\,t + 1{,}5) / 2{,}5$ for $0{,}75 \text{ mm} \leq t \leq 1{,}25 \text{ mm}$;

$k_t = 1{,}0$ for $t > 1{,}25 \text{ mm}$

Net-section resistance:

$F_{n,Rd} = (1 + 3\,r\,(d_o / u - 0{,}3))\,A_{net}\,f_u / \gamma_{M2}$ 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]

$u = 2\,e_2$ 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: [c] $F_{v,Rd} \geq 1{,}2 \, \Sigma \, F_{b,Rd}$ or $\Sigma F_{v,Rd} \geq 1{,}2 \, F_{n,Rd}$

Bolts loaded in tension:

Pull-through resistance:	Pull-through resistance $F_{p,Rd}$ to be determined by testing *4).
Pull-out resistance:	Pull-out resistance $F_{o,Rd}$ to be determined by testing *4).
Tension resistance:	$F_{t,Rd} = 0{,}9 f_{ub} A_s / \gamma_{M2}$

Conditions: [c] $F_{t,Rd} \geq \Sigma F_{p,Rd}$

Range of validity: [a]

$e_1 \geq 1{,}0\, d_o$	$p_1 \geq 3\, d_o$	$0{,}75 \text{ mm} \leq t \leq 3 \text{ mm}$	Minimum bolt size:	M 6
$e_2 \geq 1{,}5\, d_o$	$p_2 \geq 3\, d_o$		Strength grades:	4.6 - 10.9
$f_u \leq 550 \text{ N/mm}^2$				

[a] Bolts may be used beyond this range of validity if the resistance is determined from the results of tests.
[b] For thickness larger than or equal to 3 mm the rules for bolts in EN 1993-1-8 should be used.
[c] The required conditions should be fulfilled when deformation capacity of the connection is required. When these conditions are not fulfilled there should be proved that the deformation capacity will be provided by other parts of the structure.

NOTE:*4) Further information about the pull-through resistance of bolts loaded in tension can be set by the National Annex for use in a country.

10.4 Spot welds

Comment PT: All Formulae in Clause 10.4 and Clause 10.5 will be reformatted according to the drafting principles.

(1) 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) The design resistance $F_{v,Rd}$ of a spot weld loaded in shear should be determined using Table 10.6.

(4) In Table 10.6 the meanings of the symbols should be taken as follows:

A_{net} is the net cross-sectional area of the connected part;

n_w is the number of spot welds in one connection;

t is the thickness of the thinner connected part or sheet [mm];

t_{1I} is the thickness of the thicker connected part or sheet [mm];

and the end and edge distances e_1 and e_2 and the spacings p_1 and p_2 are as defined in 10.3(5).

(5) The partial factor γ_M for calculating the design resistances of spot welds should be taken as γ_{M2}, see 4.(3)P.

~~**NOTE:** The National Annex may chose the value of γ_{M2}. The value γ_{M2} = 1,25 is recommended.~~

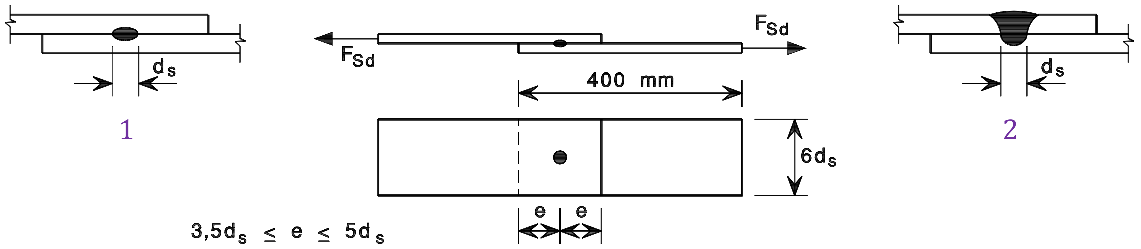
Table 10.6 - Design resistances for spot welds

Spot welds loaded in shear:				
Tearing and bearing resistance: • if $t \leq t_1 \leq 2{,}5\,t$: $F_{tb,Rd} = 2{,}7\sqrt{t}\,d_s f_u / \gamma_{M2}$ with: t in [mm] • if $t_1 > 2{,}5\,t$: $F_{tb,Rd} = 2{,}7\sqrt{t}\,d_s f_u / \gamma_{M2}$ but $F_{tb,Rd} \leq 0{,}7\,d_s^2\,f_u / \gamma_{M2}$ and $F_{tb,Rd} \leq 3{,}1\,t\,d_s\,f_u / \gamma_{M2}$				
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:	$F_{V,Rd} = \pi/4\,d_s^2\,f_u / \gamma_{M2}$			
Conditions:	$F_{v,Rd} \geq 1{,}25\,F_{tb,Rd}$	or $F_{v,Rd} \geq 1{,}25\,F_{e,Rd}$	or	$\Sigma F_{v,Rd} \geq 1{,}25\,F_{n,Rd}$
Range of validity:				
$2\,d_s \leq e_1 \leq 6\,d_s$	$3\,d_s \leq p_1 \leq 8\,d_s$			
$e_2 \leq 4\,d_s$	$3\,d_s \leq p_2 \leq 6\,d_s$			

(6) The interface diameter d_s of a spot weld should be determined from the following:

- for fusion welding: $d_s = 0{,}5\,t + 5$ mm (10.4)
- for resistance welding: $d_s = 5\sqrt{t}$ with t in [mm] (10.5)

(7) The value of d_s actually produced by the welding procedure should be verified by shear tests in accordance with Clause 12, using single-lap test specimens as shown in Figure 10.2. The thickness t of the specimen should be the same as that used in practice.



Key

1 Resistance weld

2 Fusion weld

Figure 10.2 - Test specimen for shear tests of spot welds

10.5 Lap welds

10.5.1 General

(1) This Clause 10.5 should 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 should be designed using EN 1993-1-8.

(2) The weld size should 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) 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) The partial factor γ_M for calculating the design resistances of lap welds should be taken as γ_{M2}, see 4.(3)P.

~~**NOTE:** The National Annex may give a choice of γ_{M2}. The value γ_{M2} = 1,25 is recommended.~~

10.5.2 Fillet welds

(1) The design resistance $F_{w,Rd}$ of a fillet-welded connection should be determined from the following:

- for a side fillet that is one of a pair of side fillets:

$$F_{w,Rd} = t\, L_{w,s}\, (0,9 - 0,45\, L_{w,s} / b)\, f_u / \gamma_{M2} \qquad \text{if } L_{w,s} \leq b \tag{10.6}$$

$$F_{w,Rd} = 0,45\, t\, b\, f_u / \gamma_{M2} \qquad \text{if } L_{w,s} > b \tag{10.7}$$

- for an end fillet:

$$F_{w,Rd} = t\, L_{w,e}\, (1 - 0,3\, L_{w,e} / b)\, f_u / \gamma_{M2} \qquad \text{[for one weld and if } L_{w,s} \leq b\text{]} \tag{10.8}$$

where:

b is the width of the connected part or sheet, see Figure 10.3;

$L_{w,e}$ is the effective length of the end fillet weld, see Figure 10.3;

$L_{w,s}$ is the effective length of a side fillet weld, see Figure 10.3.

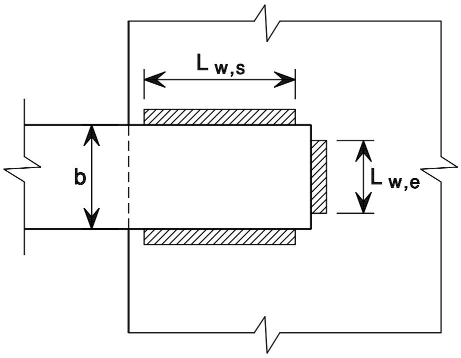


Figure 10.3 - Fillet welded lap connection

(2) If a combination of end fillets and side fillets is used in the same connection, its total resistance should be taken as equal to the sum of the resistances of the end fillets and the side fillets. The position of the centroid and realistic assumption of the distribution of forces should be taken into account.

(3) The effective length L_w of a fillet weld should be taken as the overall length of the full-size fillet, including end returns. Provided that the weld is full size throughout this length, a reduction in effective length need not 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.

10.5.3 Arc spot welds

(1) Arc spot welds should not be designed to transmit any forces other than in shear.

(2) Arc spot welds should not be used through connected parts or sheets with a total thickness Σt of more than 4 mm.

(3) Arc spot welds should have an interface diameter d_s of not less than 10 mm.

(4) If the connected part or sheet is less than 0,7 mm thick, a weld washer should be used, see Figure 10.4.

(5) Arc spot welds should have adequate end and edge distances as given in the following:

(a) 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 $f_u / f_y < 1,15$:

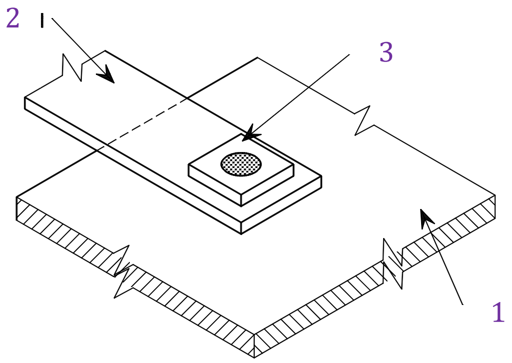
$$e_{min} = 1{,}8 \frac{F_{w,Rd}}{t\ f_u/\gamma_{M2}} \tag{10.9}$$

- if $f_u / f_y \geq 1,15$:

$$e_{min} = 2{,}1 \frac{F_{w,Rd}}{t\ f_u/\gamma_{M2}} \tag{10.10}$$

(b) 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.

(c) 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 $1{,}0\ d_w$.



Key

1 Supporting member
2 Connected part of sheet
3 Weld washer

Figure 10.4 - 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:

$$F_{w,Rd} = (\pi/4)\, d_s^2 \times 0{,}625 f_{uw} / \gamma_{M2} \tag{10.11}$$

where:

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

but $F_{w,Rd}$ should not be taken as more than the resistance given by the following:

- if $d_p / \Sigma t \leq 18\,(420 / f_u)^{0,5}$:

$$F_{w,Rd} = 1{,}5\, d_p \Sigma t\; f_u / \gamma_{M2} \tag{10.12}$$

- if $18\,(420 / f_u)^{0,5} < d_p / \Sigma t < 30\,(420 / f_u)^{0,5}$:

$$F_{w,Rd} = 27\,(420 / f_u)^{0,5}\,(\Sigma t)^2\, f_u / \gamma_{M2} \tag{10.13}$$

- if $d_p / \Sigma t \geq 30\,(420 / f_u)^{0,5}$:

$$F_{w,Rd} = 0{,}9\, d_p \Sigma t\, f_u / \gamma_{M2} \tag{10.14}$$

with d_p according to (8).

(7) The interface diameter d_s of an arc spot weld, see Figure 10.5, should be obtained from:

$$d_s = 0{,}7 d_w - 1{,}5 \Sigma t \quad \text{but} \quad d_s \geq 0{,}55\, d_w \tag{10.15}$$

where:

d_w is the visible diameter of the arc spot weld, see Figure 10.5.

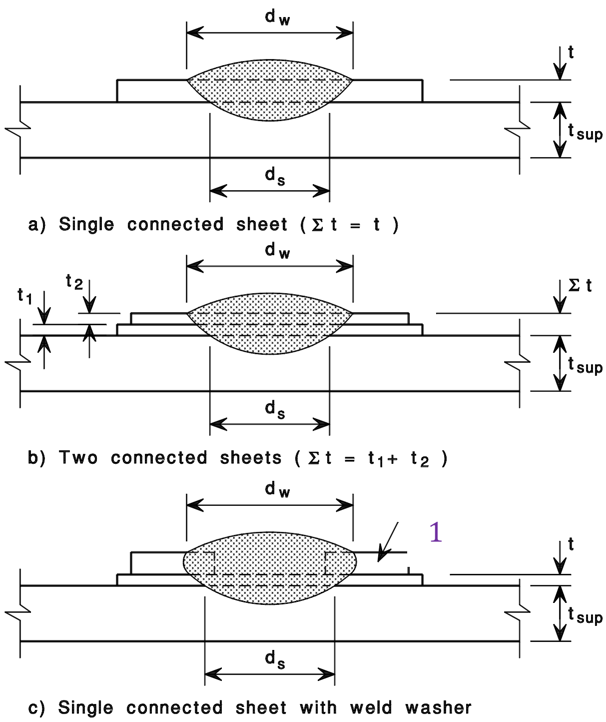
(8) The effective peripheral diameter d_p of an arc spot weld should be obtained as follows:

- for a single connected sheet or part of thickness t:

$$d_p = d_w - t \tag{10.16}$$

- for multiple connected sheets or parts of total thickness Σt:

$$d_p = d_w - 2 \Sigma t \tag{10.17}$$



Key

1 Weld washer

Figure 10.5 - Arc spot welds

(9) The design shear resistance $F_{w,Rd}$ of an elongated arc spot weld should be determined from:

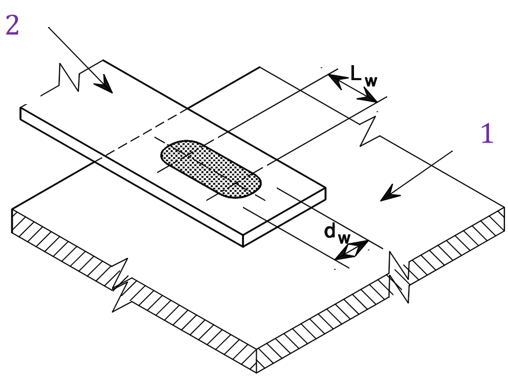
$$F_{w,Rd} = [(\pi/4) d_s^2 + L_w d_s] \times 0{,}625 f_{uw} / \gamma_{M2} \tag{10.18}$$

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

$$F_{w,Rd} = (0{,}5 L_w + 1{,}67 d_p) \Sigma t f_u / \gamma_{M2} \tag{10.19}$$

where:

L_w is the length of the elongated arc spot weld, measured as shown in Figure 10.6.



Key

1 Supporting member

2 Connected part or sheet

Figure 10.6 - Elongated arc spot weld

11 Special considerations for purlins, liner trays and sheetings

11.1 Beams restrained by sheeting or sandwich panels

11.1.1 General

(1) The provisions given in this Clause 11.1 may be applied to beams (called purlins in this Section) of *Z, C, Σ, U* and *Hat* cross-section with $h / t < 240$, $c / t \leq 20$ for single-fold and $d / t \leq 20$ for double-fold edges.

NOTE: Other limits are possible if verified by testing. Standard tests are given in Clause 12 and Annex A unless the National Annex gives information on tests for use in a country.

(2) These provisions may be used for structural systems of purlins with anti-sag bars, continuous, sleeved and overlapped systems.

(3) 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 or sandwich panels.

(4) 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.

(5) Full continuous lateral restraint may be supplied by trapezoidal steel sheeting or other profiled steel sheeting or by sandwich panels with sufficient stiffness, continuously connected to the flange of the purlin through the troughs of the sheets. The purlin at the connection to trapezoidal sheeting or other profiled steel sheeting or to sandwich panels may be regarded as laterally restrained, if the conditions in 11.5.1 are fulfilled. In other cases (e.g. sheeting fixed to the purlin through the crest) the degree of lateral restraint should either be validated by experience, or determined from tests.

NOTE: For tests see Clause 12 and Annex A.

Comment PT: (old paragraphs (6)(10) in new Subclause 11.5.1)

(6) Unless alternative support arrangements may be justified from the results of tests the purlin should have support details, such as cleats, that prevent lateral rotation and displacement at its supports. The effects of forces in the plane of the sheeting or sandwich panels that are transmitted to the supports of the purlin should be taken into account in the design of the support details.
Where cleats are used the lateral local bending of the web due to the lateral load transferred by sheeting may be neglected.
The web crippling of the purlin may be neglected if a gap between the support and the bottom flange of the purlin is conserved. The value of the gap should be sufficient that the bottom flange of the loaded purlin cannot come in contact with the support.

(7) The behaviour of a laterally restrained purlin should be modelled as outlined in Table 11.1. 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. The rotational restraint provided by the sheeting should be determined following 11.5.2.

(8) 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.

11.1.2 Calculation methods

(1) Unless a second order analysis is carried out, the method given in 11.1.3 and 11.1.4 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 $q_{h,Ed}$ equal to $k_h \cdot q_{Ed}$, see Table 11.1.

(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.

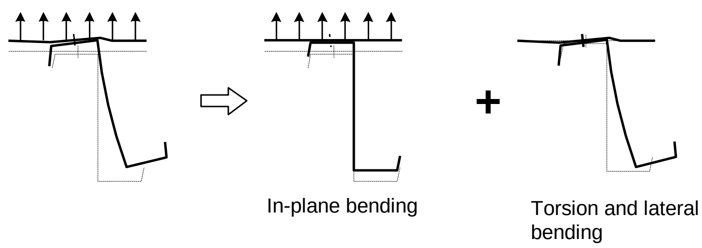
(4) For a more precise calculation, a numerical analysis should be carried out, using values of the rotational spring stiffness C_D obtained from 11.5.2. Allowance should be made for the effects of an initial bow imperfection e_0 in the free flange, defined as in 7.3. The initial imperfection should be compatible with the shape of the relevant buckling mode, determined by the eigen-vectors obtained from the elastic first order buckling analysis.

(5) A numerical analysis using the rotational spring stiffness C_D obtained from 11.5.2 may also be used if lateral restraint is not supplied or if its effectiveness cannot be proved. When the numerical analysis is carried out, it should take into account the bending in two directions, torsional St Venant stiffness and warping stiffness about the imposed rotation axis.

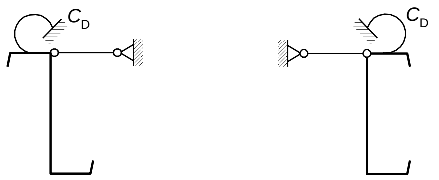
(6) If a 2nd order analysis is carried out, effective sections and stiffness, due to local buckling, should be taken into account. ~~NOTE: For a simplified design of purlins made of C-, Z- and Σ- cross sections see Annex E.~~

Table 11.1 - Modelling laterally braced purlins rotationally restrained by sheeting or sandwich panels

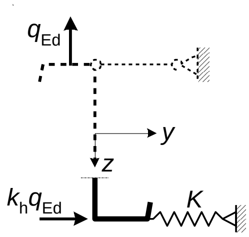
a) Z and C section purlin with upper flange connected to sheeting



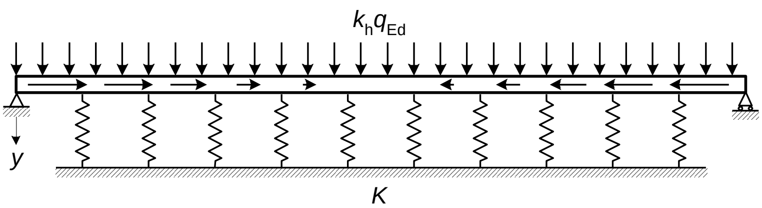
b) Total deformation split into two parts



c) Model purlin as laterally braced with rotationally spring restraint C_D from sheeting



d) As a simplification replace the rotational spring C_D by a lateral spring stiffness K



e) Free flange of purlin modelled as beam on elastic foundation, representing effect of torsion and lateral bending (including cross section distortion) on single span with uplift loading.

11.1.3 Design criteria

11.1.3.1 Single span purlins

(1) The single span purlins or physically continuous over more spans should satisfy the criteria for cross-section resistance given in 11.1.4.1 and the criteria for stability of the free flange given in 11.1.4.2.

11.1.3.2 Two-spans continuous purlins with gravity load

(1) The moments due to gravity loading in a purlin that is physically continuous over two spans without overlaps or sleeves, may 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 11.1.4.1. For the moment at the internal support, the criteria for stability of the free flange given in 11.1.4.2 should also be satisfied.

(3) Alternatively the moments may be determined using the results of tests in accordance with Clause 12 and Annex A.5 on the moment-rotation behaviour of the purlin over the internal support.

NOTE: Appropriate testing procedures are given in Annex A.

(4) The design value of the resistance moment at the supports $M_{\mathrm{sup,Rd}}$ for a given value of the load per unit length q_{Ed}, 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 Clause 12 and Annex A.5;
- the theoretical relationship between the support moment $M_{\mathrm{sup,Ed}}$ and the corresponding plastic hinge rotation ϕ_{Ed} in the purlin over the support.

To determine the final design value of the support moment $M_{\mathrm{sup,Ed}}$ allowance should be made for the effect of the lateral load in the free flange and/or the buckling stability of that free flange around the mid-support, which are not fully taken into account by the internal support test as given in Annex A.5.2. If the free flange is physically continued at the support and if the distance between the support and the nearest anti-sag bar is larger than 0,5 s, the lateral load $q_{\mathrm{h,Ed}}$ according to 11.1.4.1 should be taken into account in verification of the resistance at mid-support. Alternatively, full-scale tests for two or multi-span purlins may be used to determine the effect of the lateral load in the free flange and/or the buckling stability of that free flange around the mid-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:

$$\Phi_{Ed} = \frac{L}{12\, EI_{eff}}\left(q_{Ed}L^2 - 8\, M_{sup,Ed}\right) \tag{11.1}$$

$$\mathrm{M}_{span,Ed} = \frac{\left(q_{Ed}L^2 - 2\, M_{sup,Rd}\right)^2}{8\, q_{Ed}\, L^2} \tag{11.2}$$

where:

I_{eff} is the effective second moment of area for the moment $M_{\mathrm{spn,Ed}}$;

L is the span;

$M_{\mathrm{spn,Ed}}$ is the maximum moment in the span.

(7) For a purlin with two unequal spans, and for non-uniform loading (e.g. snow accumulation), and for other similar cases, the Formulae (11.1) and (11.2) are not valid. Appropriate analysis should be made for these cases.

(8) The maximum span moment $M_{\mathrm{spn,Ed}}$ in the purlin should satisfy the criteria for cross-section resistance given in 11.1.4.1. Alternatively, the resistance moment in the span may be determined by testing. A single span test may be used with a span comparable to the distance between the points of contraflexure in the span.

11.1.3.3 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 11.1.4.1. Because the support reaction is a tensile force, account need not be taken of its interaction with the support moment. The mid-support should be checked also for interaction of bending moment and shear forces.

(3) The moments in the spans should satisfy the criteria for stability of the free flange given in 11.1.4.2.

11.1.3.4 Purlins with semi-continuity provided by overlaps or sleeves

(1) The purlins, in which continuity over two or more spans is provided by overlaps or sleeves at internal supports, should satisfy the criteria for cross-section resistance given in 11.1.4.1 and the criteria for stability of the free flange. In this case the stability criteria given in 11.1.4.2 may be applied.

(2) Tests 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: This is in general non linear, if a linear behaviour is assumed in the calculations, minimum and maximum values should be used (see A.8.3). Only when failure occurs at the support with cleat or similar preventing lateral displacements at the support, then the redistribution of bending moments may be used for sleeved and overlapped systems;
- the resistance of the overlapped or sleeved part to combined support reaction and moment;
- the resistance of the overlapped or sleeved part to combined shear force and bending moment;

Alternatively, the characteristics of the mid-support details may be determined by numerical methods if the design procedure is at least validated by a relevant number of tests.

~~(3) (4) deleted~~

11.1.3.5 Serviceability criteria

(1) The serviceability criteria relevant to purlins should be satisfied.

11.1.4 Design resistance

11.1.4.1 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 11.1 by superposing the stresses due to:

- the in-plane bending moment $M_{y,Ed}$;
- the axial force N_{Ed};
- an equivalent lateral load $q_{h,Ed}$ 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:

$$\sigma_{max,Ed} = \frac{N_{Ed}}{A_{eff}} + \frac{M_{y,Ed}}{W_{eff,y}} \leq \frac{f_y}{\gamma_M} \quad (11.3)$$

- free flange:

$$\sigma_{max,Ed} = \frac{N_{Ed}}{A_{eff}} + \frac{M_{y,Ed}}{W_{eff,y}} + \frac{M_{fz,Ed}}{W_{fz}} \leq \frac{f_y}{\gamma_M} \quad (11.4)$$

where:

A_{eff} is the area of the effective cross-section in uniform compression;

f_y is the yield strength as defined in 5.2.1(5);

$M_{fz,Ed}$ is the bending moment in the free flange due to the lateral load $q_{h,Ed}$, see Formula (11.5);

$W_{eff,y}$ is the section modulus of the effective cross-section in bending about the y - y axis;

W_{fz} is the elastic section modulus of the gross cross-section of the free flange plus the contributing part of the web for bending about the z-z axis; unless a more sophisticated analysis is carried out the contributing part of the web may be taken equal to 1/5 of the web height from the point of web-flange intersection in case of C- and Z-sections and 1/6 of the web height in case of Σ-section, see Figure 11.1;

$\gamma_M = \gamma_{M0}$ if $A_{eff} = A$ or if $W_{eff,y} = W_{el,y}$ and $N_{Ed} = 0$, otherwise $\gamma_M = \gamma_{M1}$.

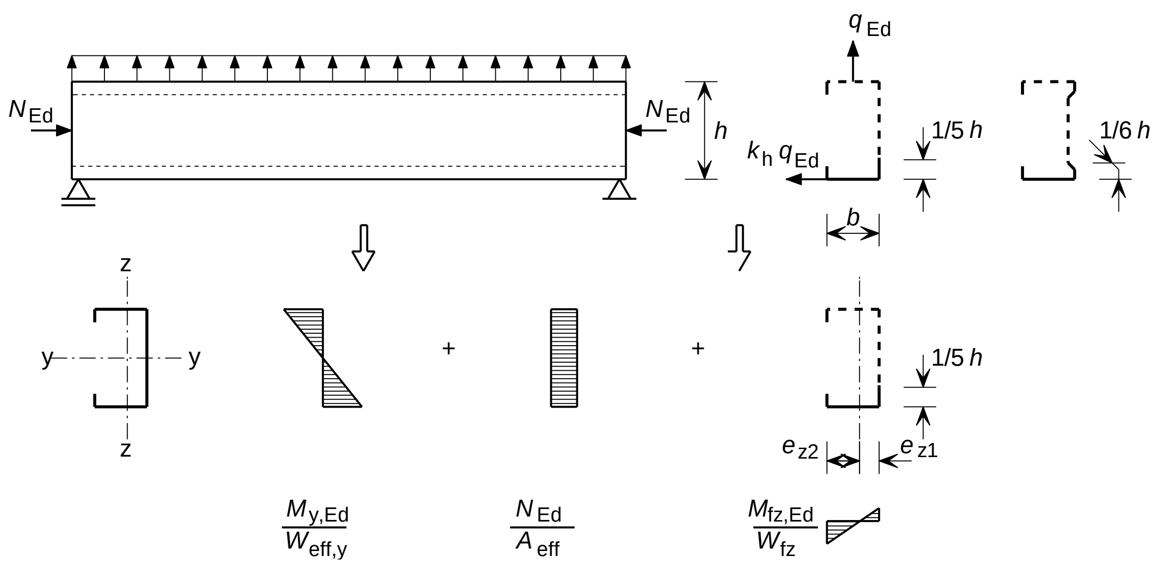


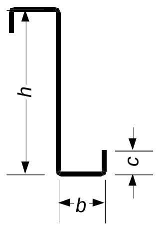
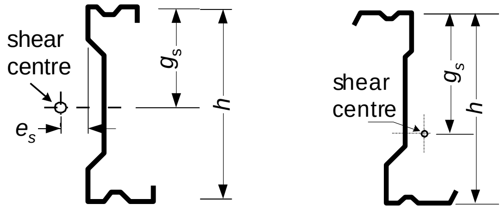
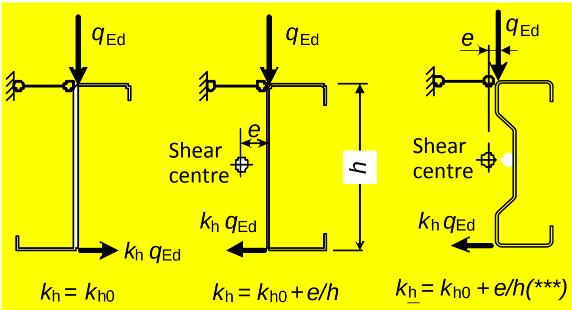
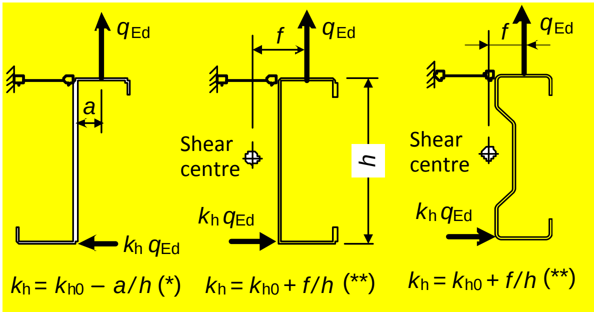
Figure 11.1 - Superposition of stresses

(3) The equivalent lateral load $q_{h,Ed}$ acting on the free flange, due to torsion and lateral bending, should be obtained from:

$$q_{h,Ed} = k_h \, q_{Ed} \tag{11.5}$$

(4) The coefficient k_h should be obtained as indicated in Table 11.2 for common types of cross-section otherwise it should be justified by appropriate analysis..

Table 11.2 - Conversion of torsion and lateral bending into an equivalent lateral load $k_h \, q_{Ed}$

k_{h0} **factor for lateral load on free bottom flange** (k_{h0} corresponds to loading in the shear centre)	
 $k_{h0} = \dfrac{ht(b^2+2cb-2c^2 b/h)}{4I_y}$ (a) Simple symmetrical Z-section	 $k_{h0} = \dfrac{I_{yz}}{I_y}\dfrac{g_s}{h}$ (b) Z-, C- or Σ– sections
Equivalent lateral load factor k_h	
 Z-, C- or Σ– sections (a) Gravity loading	 Z-, C- or Σ– sections (b) Uplift loading
(*) If $a/h > k_{h0}$ then the load $k_h \, q_{Ed}$ is acting in the opposite direction. (**) The value of f is limited to the position of the load q_{Ed} between the edges of the top flange. (***) If the shear centre is at the right hand side of the load q_{Ed} then the equivalent lateral load is acting in the opposite direction.	

(5) The lateral bending moment $M_{fz,Ed}$ may be determined from Formula (11.6) except for a beam with the free flange in tension, where, due to positive influence of flange curling and second order effect moment $M_{fz,Ed}$ may be taken equal to zero:

$$M_{fz,Ed} = \kappa_R \, M_{0,fz,Ed} = \tag{11.6}$$

where:

$M_{0,fz,Ed}$ is the initial lateral bending moment in the free flange without any spring support;

κ_R is a correction factor for the effective spring support.

(6) The initial lateral bending moment in the free flange $M_{0,fz,Ed}$ may be determined from Table 11.3 for the critical locations in the span, at supports, at anti-sag bars and between anti-sag bars. The validity of the Table 11.3 is limited to the range $R \leq 40$.

(7) The correction factor κ_R for the relevant location and boundary conditions, may be determined from Table 11.3 (or using the theory of beams on the elastic Winkler foundation), using the value of the coefficient R of the spring support given by:

$$R = \frac{K\,L_a^4}{\pi^4\,EI_{fz}} \tag{11.7}$$

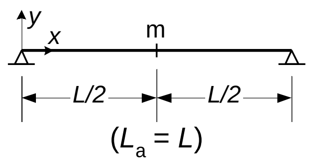
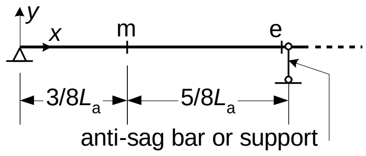
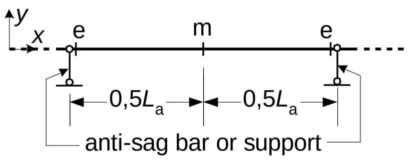
where:

I_{fz} is the second moment of area of the gross cross-section of the free flange plus the contributing part of the web for bending about the z-z axis, see 11.1.4.1(2); when numerical analysis is carried out, see 11.1.2(5);

K is the lateral spring stiffness per unit length from 11.1.5;

L_a is the distance between anti-sag bars, or if none are present, the span L of the purlin.

Table 11.3 - Values of initial moment $M_{0,fz,Ed}$ and correction factor κ_R

System	Location	$M_{0,fz,Ed}$	κ_R
	m	$\frac{1}{8}\,q_{h,Ed}\,L_a^2$	$\kappa_R = \frac{1-0{,}0225\,R}{1+1{,}013\,R}$
	m	$\frac{9}{128}\,q_{h,Ed}\,L_a^2$	$\kappa_R = \frac{1-0{,}0141\,R}{1+0{,}416\,R}$
	e	$-\frac{1}{8}\,q_{h,Ed}\,L_a^2$	$\kappa_R = \frac{1+0{,}0314\,R}{1+0{,}396\,R}$
	m	$\frac{1}{24}\,q_{h,Ed}\,L_a^2$	$\kappa_R = \frac{1-0{,}0125\,R}{1+0{,}198\,R}$
	e	$-\frac{1}{12}\,q_{h,Ed}\,L_a^2$	$\kappa_R = \frac{1+0{,}0178\,R}{1+0{,}191\,R}$

11.1.4.2 Buckling resistance of the free flange

(1) If the free flange is in compression, its buckling resistance should be verified using:

$$\frac{1}{\chi_{LT}}\left(\frac{N_{Ed}}{A_{eff}}+\frac{M_{y,Ed}}{W_{eff,y}}\right)+\frac{M_{fz,Ed}}{W_{fz}}\leq\frac{f_{yb}}{\gamma_{M1}} \qquad (11.8)$$

where:

χ_{LT} is the reduction factor for lateral torsional buckling (flexural buckling of the free flange).

NOTE: The χ_{LT} – value can be set by the National Annex for use in a country. The use of EN 1993-1-1: 2010, Clause 6.3.2.3, using buckling curve b (α_{LT} = 0,34 , β = 0,75 , $\bar{\lambda}_{LT,0}$ = 0,4) is recommended for the relative slenderness $\bar{\lambda}_{fz}$ given in (2).

PT-Comment on the NOTE: Original content of EN 1993-1-3 kept, new AMD-88 specifying the buckling curve to be confirmed by SC3.

In the case of an axial compression force N_{Ed}, when the reduction factor for buckling around the strong axis is smaller than the reduction factor for lateral flange buckling, e.g. in the case of many anti-sag bars, flexural buckling should also be checked following EN 1993-1-1.

(2) The relative slenderness $\bar{\lambda}_{fz}$ for flexural buckling of the free flange should be determined from:

$$\bar{\lambda}_{fz}=\frac{l_{fz}/i_{fz}}{\lambda_1} \qquad (11.9)$$

with:

$$\lambda_1=\pi\sqrt{\frac{E}{f_{yb}}}$$

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 the contributing part of the web for bending about the z-z axis, see 11.1.4.1(2).

(3) Provided that $0 \leq R \leq 200$, the buckling length of the free flange with varying compressive stress over the length L as shown in Figure 11.2 and 11.3 may be obtained from:

$$l_{fz}=\eta_1 L_a(1+\eta_2 R^{\eta_3})^{\eta_4} \qquad (11.10)$$

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 11.1.4.1(7);

η_1 to η_4 are coefficients that depend on the number of anti-sag bars, as given in Table 11.4 for gravity loading and Table 11.5 for uplift loading.

Tables 11.4 and 11.5 are only valid for:

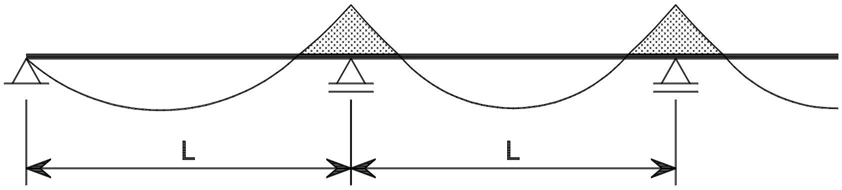
- equal spans;
- equal distances between anti-sag bars;
- uniformly transverse loaded beam systems without overlap or sleeve and without normal load;
- systems with sleeves and overlaps provided that the connection system may be considered as fully continuous with constant second moment of area

- purlins with anti-sag bars that provide lateral rigid support for the free flange.

In other cases the buckling length should be determined by more appropriate calculations or, except cantilevers, the values of the Table 11.4 for the case of 3 anti-sag bars per field may be used:

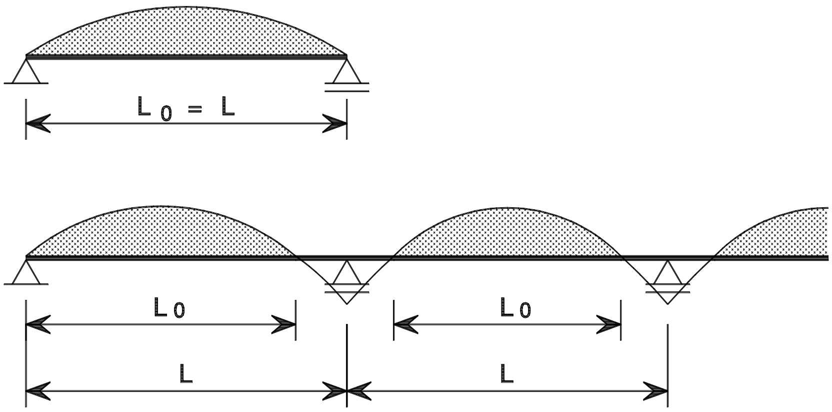
- for gravity loading the values of the Table 10.4 for the case of 3 and 4 anti-sag bars per field may be used, however with η_1 coefficient increased by 0.05 and $\eta_2 = \eta_3 = \eta_4 = 0$;
- for uplift loading the values of the Table 10.5 for the case of 3 and 4 anti-sag bars per field may be used, however with η_1 coefficient increased by 0.05 and $\eta_2 = \eta_3 = \eta_4 = 0$.

NOTE: In some cases, due to rotations in overlap or sleeve connection, the field moment changes resulting in modified buckling lengths. Neglecting the real moment distribution in these cases leads to unsafe design.



NOTE: Dotted areas show regions in compression

Figure 11.2 - Varying compressive stress in the free flange for gravity load cases



NOTE: Dotted areas show regions in compression

Figure 11.3: Varying compressive stress in the free flange for uplift load cases

Table 11.4 - Coefficients η_i for gravity load with 0, 1, 2, 3, 4 anti-sag bars

Situation	**Anti sag-bar Number**	η_1	η_2	η_3	η_4
End span	0	0.414	1.72	1.11	-0.178
Intermediate span		0.657	8.17	2.22	-0.107
End span	1	0.515	1.26	0.868	-0.242
Intermediate span		0.596	2.33	1.15	-0.192
End and intermediate span	2	0.596	2.33	1.15	-0.192
End and intermediate span	3 and 4	0.694	5.45	1.27	-0.168

Table 11.5 - Coefficients η_i for uplift load with 0, 1, 2, 3, 4 anti-sag bars

Situation	Anti sag-bar Number	η_1	η_2	η_3	η_4
Simple span	0	0.694	5.45	1.27	-0.168
End span		0.515	1.26	0.868	-0.242
Intermediate span		0.306	0.232	0.742	-0.279
Simple and end spans	1	0.800	6.75	1.49	-0.155
Intermediate span		0.515	1.26	0.868	-0.242
Simple span	2	0.902	8.55	2.18	-0.111
End and intermediate spans		0.800	6.75	1.49	-0.155
Simple and end spans	3 and 4	0.902	8.55	2.18	-0.111
Intermediate span		0.800	6.75	1.49	-0.155

11.1.5 Lateral spring stiffness provided to the free flange of a purlin

(1) The lateral spring support provided to the free flange of the purlin by the sheeting should be modelled as a lateral spring acting at the free flange, see Table 11.1. The total lateral spring stiffness K per unit length should be determined from:

$$\frac{1}{K} = \frac{1}{K_A} + \frac{1}{K_B} + \frac{1}{K_C} \qquad (11.11)$$

where:

K_A is the lateral stiffness corresponding to the rotational stiffness of the joint 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) -It may be assumed to be safe as well as acceptable to neglect $1/K_C$ because K_C is very large compared to K_A and K_B. ---

(3) The value of K should be obtained form the rotational spring stiffnesses using:

$$\frac{1}{K} = \frac{h^2}{C_D} = \frac{h^2}{C_{D,A}} + \frac{h^2}{C_{D,B}} + \frac{h^2}{C_{D,C}} \qquad (11.12)$$

where:

$C_{D,A}$, $C_{D,B}$, $C_{D,C}$ are the rotational stiffnesses according to Clause 11.5.2.

(4) The rotational stiffness $C_{D,B}$ due to distortion of the cross-section of the purlin may also be determined by calculation using:

$$\frac{h^2}{C_{D,B}} = \frac{4\,h^2(1-\nu^2)(h_d + b_{mod})}{E\,t^3} \qquad (11.13)$$

in which the dimension b_{mod} is determined as follows:

- for cases where the equivalent lateral force $q_{h,Ed}$ bringing the purlin into contact with the sheeting at the purlin web:

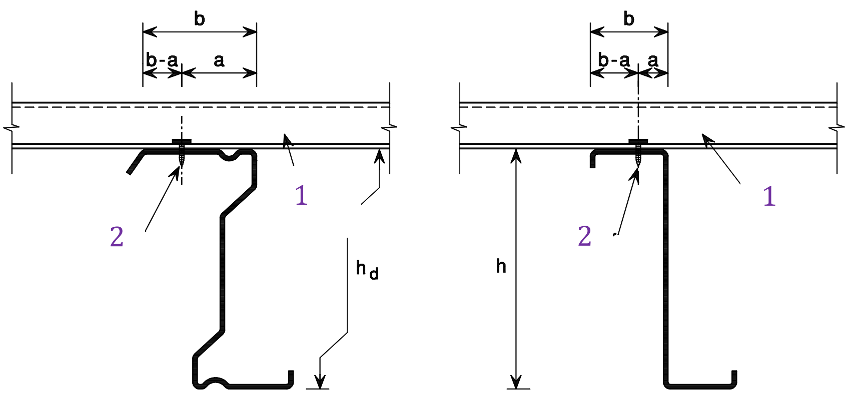
  $b_{\text{mod}} = a$

- for cases where the equivalent lateral force $q_{h,Ed}$ bringing the purlin into contact with the sheeting at the tip of the purlin flange:

  $b_{\text{mod}} = 2a + b$

where:

t	is the thickness of the purlin;
a	is the distance from the sheet-to-purlin fastener to the purlin web, see Figure 11.4;
b	is the width of the purlin flange connected to the sheeting, see Figure 11.4;
h	is the overall height of the purlin;
h_d	is the developed height of the purlin web, see Figure 11.4.



Key

1 Sheet

2 Fastener

Figure 11.4 - Purlin and attached sheeting

PT comment: old 10.1.5.2 transferred to new 11.5.2

11.1.6 Forces in sheet/purlin fasteners and reaction forces

(1) Fasteners fixing the sheeting to the purlin should be checked for a combination of shear force $q_s\,e$, perpendicular to the flange, and tension force $q_t\,e$ where q_s and q_t may be calculated using Table 11.6 and e is the pitch of the fasteners. Shear force due to stabilising effect, see EN1993-1-1, should be added to the shear force. Furthermore, shear force due to diaphragm action, acting parallel to the flange, should be vectorially added to q_s.

Table 11.6 - Shear force and tensile force in fastener along the beam

Beam and loading	Shear force per unite length q_s	Tensile force per unit length q_t
Z-beam, gravity loading	$(1+\xi)\,k_h q_{Ed}$, may be taken as 0	0
Z-beam, uplift loading	$(1+\xi)\,(k_h - a/h)\,q_{Ed}$	$\lvert\,\xi\,k_h\,q_{Ed}\,h/a\,\rvert + q_{Ed} \quad (a \cong b/2)$
C-beam, gravity loading	$(1-\xi)\,k_h q_{Ed}$	$\xi\,k_h\,q_{Ed}\,h/a$
C-beam, uplift loading	$(1-\xi)\,(k_h - a/h)\,q_{Ed}$	$\xi\,k_h\,q_{Ed}\,h/(b-a) + q_{Ed}$

(2) The fasteners fixing the purlins to the supports should be checked for the reaction force R_w in the plane of the web and the transverse reaction forces R_1 and R_2 in the plane of the flanges, see Figure 11.5. Forces R_1 and R_2 may be calculated using Table 11.7. Force R_2 should also include loads parallel to the roof for sloped roofs. If R_1 is positive there is no tension force on the fastener. R_2 should be transferred from the sheeting to the top flange of the purlin and further on to the rafter (main beam) through the purlin/rafter connection (support cleat) or via special shear connectors or directly to the base or similar element. The reaction forces at an inner support of a continuous purlin may be taken as 2,2 times the values given in Table 11.7.

NOTE: For sloped roofs the transversal loads to the purlins are the perpendicular (to the roof plane) components of the vertical loads and parallel components of the vertical loads are acting on the roof plane.

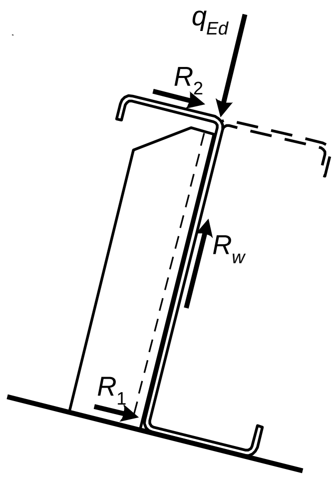


Figure 11.5: Reaction forces at support

(3) The factor ς may be taken as $\varsigma = 1 - \sqrt[3]{\kappa_R^2}$, where κ_R is the correction factor given in Table 11.3, and the factor ξ may be taken as $\xi = 1{,}5\,\varsigma$

Table 11.7 -- Reaction force at support for simply supported beam

Beam and loading	Reaction force on bottom flange R_1	Reaction force on top flange R_2
Z-beam, gravity loading	$(1-\varsigma)k_h q_{Ed} L/2$	$(1+\varsigma)k_h q_{Ed} L/2$
Z-beam, uplift loading	$-(1-\varsigma)k_h q_{Ed} L/2$	$-(1+\varsigma)k_h q_{Ed} L/2$
C-beam, gravity loading	$(1-\varsigma)k_h q_{Ed} L/2$	$-(1-\varsigma)k_h q_{Ed} L/2$
C-beam, uplift loading	$-(1-\varsigma)k_h q_{Ed} L/2$	$(1-\varsigma)k_h q_{Ed} L/2$

11.2 Liner trays restrained by sheeting

11.2.1 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 11.6. The two narrow flanges should be laterally restrained by attached profiled steel sheeting or by steel purlin or by similar component.

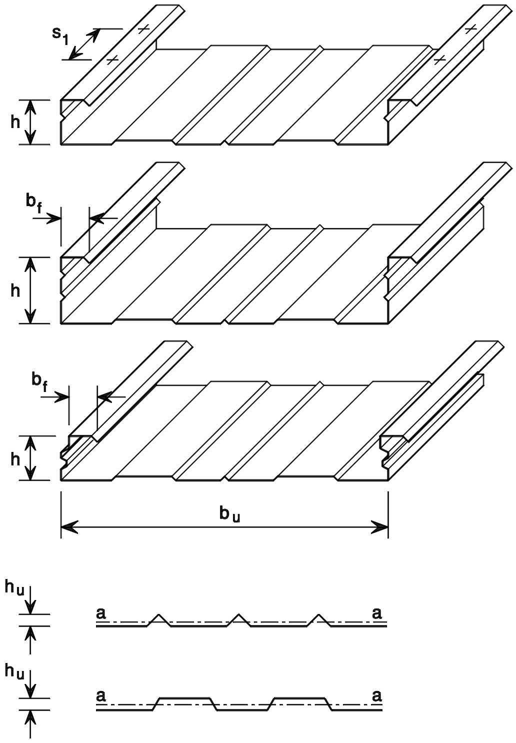


Figure 11.6: Typical geometry for liner trays

(2) The resistance of the webs of liner trays to shear forces and to local transverse forces should be obtained using 8.1.5 to 8.1.11, 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 11.2.2 provided that:

- the geometrical properties are within the range given in Table 11.8;
- 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 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 11.8: Range of validity of Clause 11.2.2

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 mm⁴ / mm
		s_1	$\leq$	1000 mm

11.2.2 Moment resistance

11.2.2.1 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 11.7 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 area of the gross cross-section of the webs;
- **Step 2:** Find the centroid of the effective cross-section, then obtain the moment resistance $M_{c,Rd}$ from:

$$M_{c,Rd} = 0{,}8\, W_{eff,min} \frac{f_{yb}}{\gamma_{M0}} \tag{11.14}$$

with:

$W_{eff,min} = I_{y,eff} / z_c$ but $W_{eff,min} \leq I_{y,eff} / z_t$;

where:

z_c and z_t are as indicated in Figure 11.7.

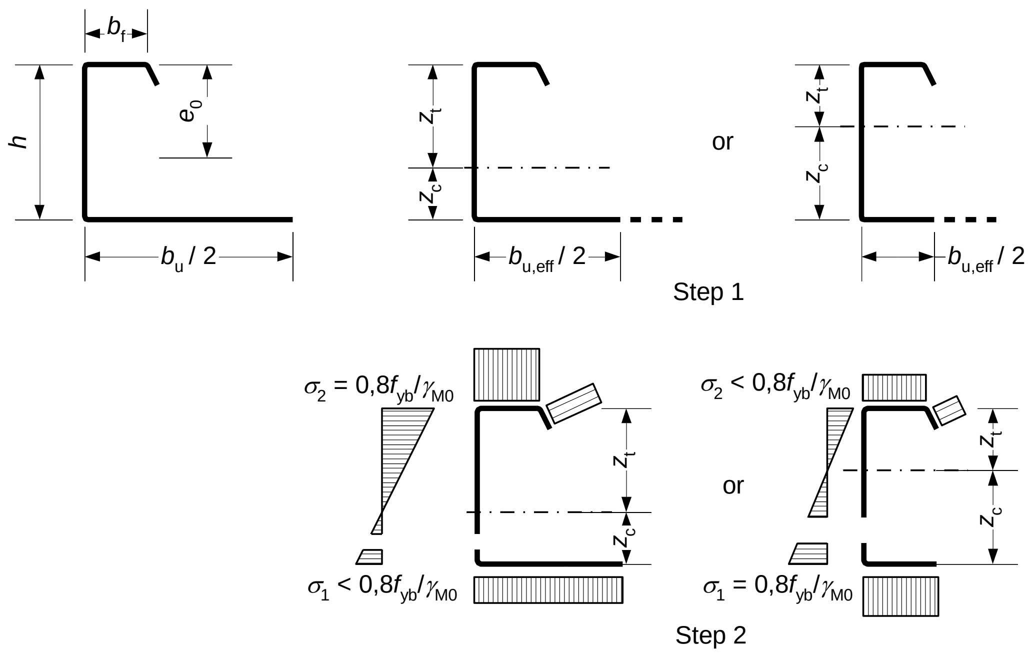


Figure 11.7 - Determination of moment resistance — wide flange in compression

11.2.2.2 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 11.8 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:

$$b_{u,eff} = \frac{53{,}3 \cdot 10^{10}\, e_0^2\, t^3\, t_{eq}}{h\, L\, b_u^3} \tag{11.15}$$

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, see Figure 11.6.

- **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 area of the gross cross-section of the webs;
- **Step 4:** Find the centroid of the effective cross-section, then obtain the buckling resistance moment $M_{b,Rd}$ using:

$$M_{b,Rd} = 0{,}8\, \beta_b\, W_{eff,com} \frac{f_{yb}}{\gamma_{M0}} \quad \text{but} \quad M_{b,Rd} = 0{,}8\, W_{eff,t} \frac{f_{yb}}{\gamma_{M0}} \tag{11.16}$$

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 $s_1 \le 300$ mm:

  $\beta_b = 1{,}0$

- if $300 \text{ mm} \le s_1 \le 1000$ 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 11.6.

(2) The effects of shear lag need not be considered if $L / b_{u,eff} \ge 25$. Otherwise a reduced value of ρ should be determined as specified in 8.1.4.3.

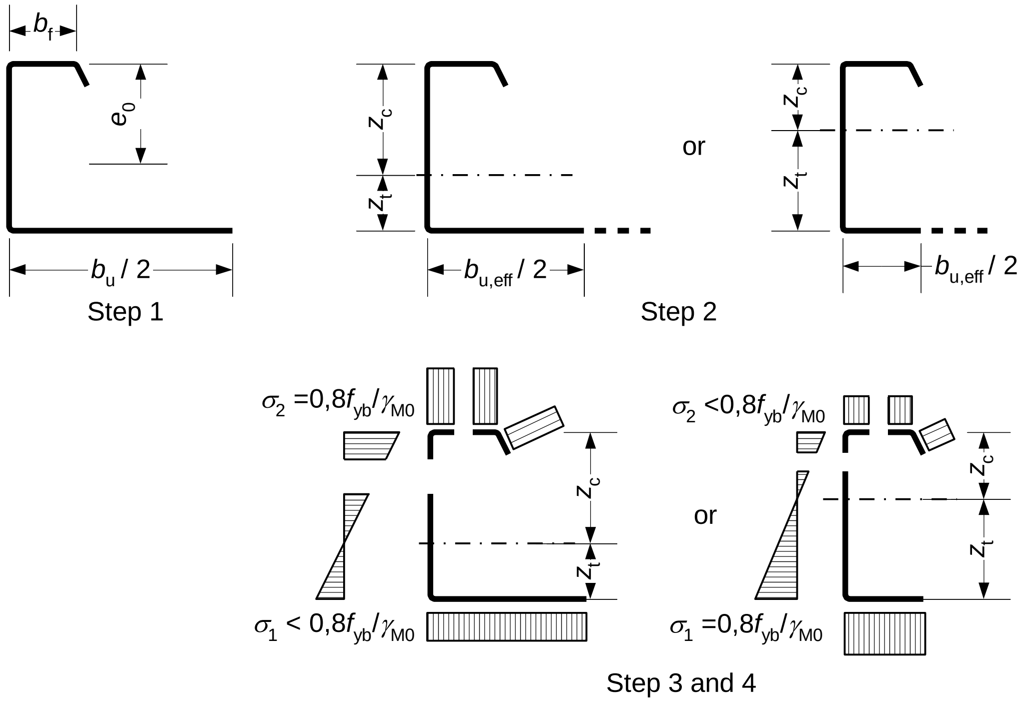


Figure 11.8 - Determination of moment resistance — wide flange in tension

(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.

11.3 Perforated sheeting

(1) Perforated sheeting with the holes arranged in the shape of equilateral triangles 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. For more economical solutions see design assisted by testing in Clause 12 and Annex A.

(2) Provided that $0,2 \leq d/a \leq 0,9$, the section properties of the gross cross-section may be calculated using Clause 7.1, but replacing t by $t_{\mathrm{a,eff}}$ obtained from:

$$t_{a,eff} = 1{,}18\, t \left(1 - \frac{d}{0{,}9\, a}\right) \tag{11.17}$$

where:

d is the diameter of the perforations;

a is the spacing between the centres of the perforations.

(3) Provided that $0,2 \leq d/a \leq 0,9$ effective section properties may be calculated using Clause 7, but replacing t by $t_{\mathrm{b,eff}}$ obtained from:

$$t_{b,eff} = t \sqrt[3]{1{,}18 \left(1 - \frac{d}{a}\right)} \tag{11.18}$$

(4) The resistance of a single web to local transverse forces may be calculated using Clause 8.1.7, but replacing t by $t_{\mathrm{c,eff}}$ obtained from:

$$t_{c,eff} = t \left(1 - \left(\frac{d}{a}\right)^2 \frac{s_{per}}{s_w}\right)^{3/2} \tag{11.19}$$

where:

s_{per} is the slant height of the perforated portion of the web;

s_{w} is the total slant height of the web.

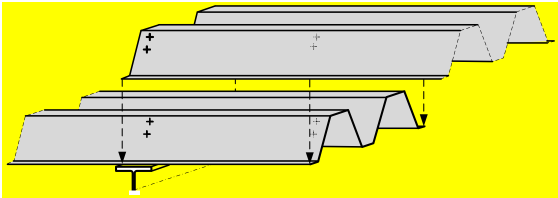
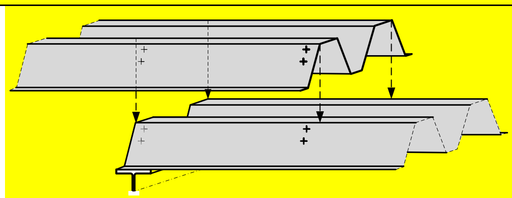
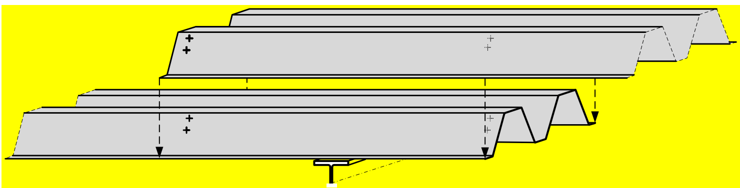
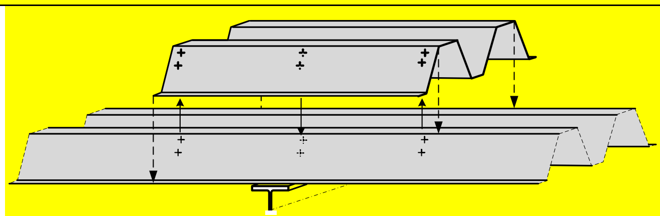
11.4 Trapezoidal sheeting with overlap at support

11.4.1 Moment resisting overlap

11.4.1.1 General

(1) Overlapping of ends of sheets designed to be statically effective should only be used in the area of the inner supports of continuous systems. Different overlapping systems are shown in Table 11.9. They should be dimensioned according to the present Clause and Clause 11.4.2 to 11.4.5, respectively.

Table 11.9 - Static system of the overlapping sheeting with single or double overlap or reinforcement

Clause 11.4.2	Single overlap with cantilevered lower sheeting **SOL-L**	
Clause 11.4.3	Single overlap with cantilevered upper sheeting **SOL-U**	
Clause 11.4.4	Double overlap **DOL**	
Clause 11.4.5	Continuous sheeting reinforced above the support **CR**	

(2) For profiled sheeting thicker than 1,0 mm, depending on the cross section (e.g. steep webs, $\varphi > 75°$, and flat trough, see Figure 11.9) flats sheets may be considered to be inserted in the bottom flange in the area of the support.

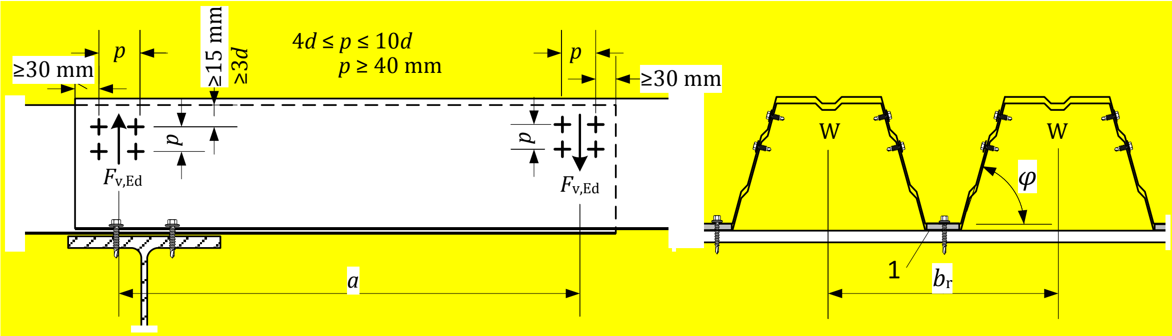
(3) In the case of profiled sheeting partially perforated in the web, the fasteners should be arranged in the unperforated areas of the web.

(4) Overlapping may not be assessed as being statically effective in liner trays.

11.4.1.2 Overlap region considered as continuous sheeting

(1) The overlapping region may be considered as that of continuous sheeting if the overlapping length is at least 0,08L and the fasteners are arranged as in Figure 11.9. The lengths of the spans should be about the same.

(2) For the fasteners, the edge and spacing of Figure 11.9 apply.



Key
1. Shim of flat sheet

Figure 11.9 - Trapezoidal sheeting with single overlap at support

(3) Minimum two fasteners in each vertical row on each side of the free edge of the upper sheeting should be used.

11.4.1.3 Static system considering deformation in the overlaps and the connections

(1) A static system according to Figure 11.10 should be used where the fasteners are modelled as springs with spring constant S_f or S_w according to (5).

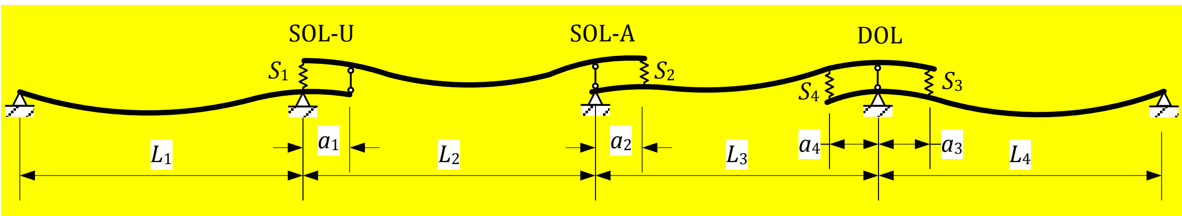


Figure 11.10 – Static system for trapezoidal sheeting with single overlap at supports

(2) The overlapping length should be in the interval 0,065L to 0,11L where L is the larger of the adjacent span but the overlapping length should not be larger than 0,15 times the shortest adjacent span.

(3) End and edge distances and spacing of fasteners should fulfil the conditions in Clause 8.3 and, for fasteners in the crest or trough, in Table 11.9.

(4) The spring stiffness of the connections should be taken as:

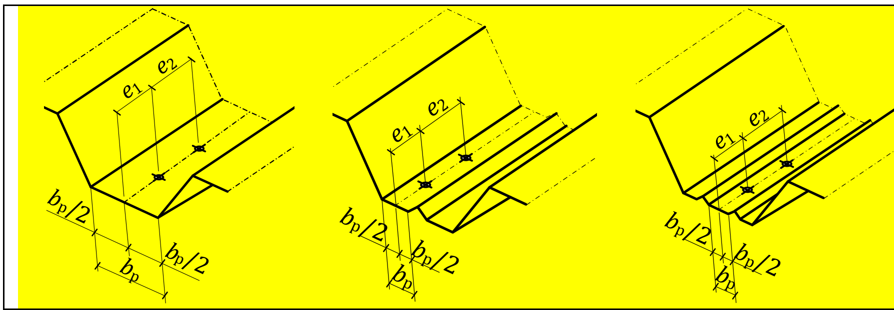
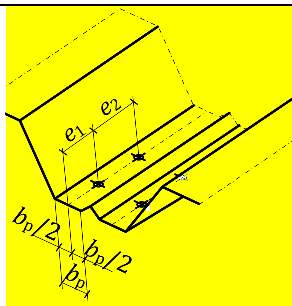
- for fasteners in the trough (bottom flange): $$S_f = 0{,}5\, k_f E \sqrt{\frac{t^3\, d_w}{h_w b_p}} \qquad (11.20)$$
- for fasteners in the crest (top flange) above the support: 2 S_f acc. to (11.22)

- for fasteners in the webs: the deformation may be omitted i.e. $S_w = \infty$

where:

k_f	is a coefficient according to Table 11.10
d	is the diameter of the fastener
d_W	is the diameter of the washer
h_W	is the height of the profile
b_p	is the width of the flat part of the flange according to Table 11.10

Table 11.10 – Arrangement of fasteners at support and coefficient for deformation

	
$e_1 \geq b_p$, $e_2 \geq$ 40 mm k_f = 0,07	$e_1 \geq b_p$, $e_2 \geq$ 40 mm k_f = 0,13

(5) For single overlap with the cantilevered end of the profiled sheets underneath (SOL-L according to Table G.1), the different parts of the overlapping region should be checked for bending moment, shear force and support reaction force in sections A to D in Figure 11.11.

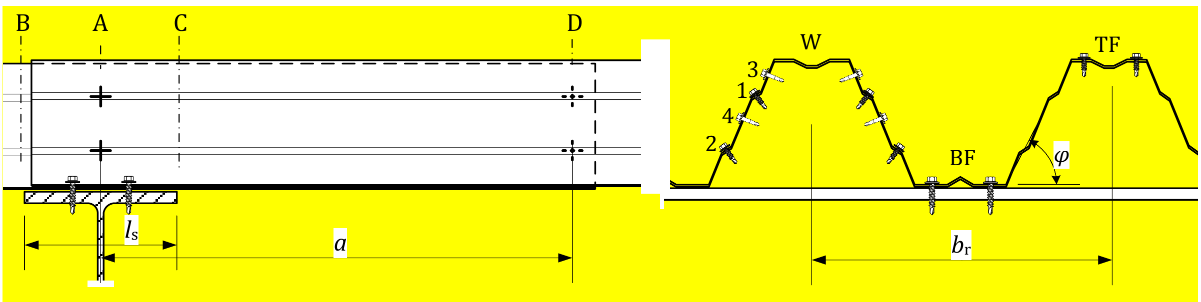
Section A	Bending moment $M_{B,Ed} = M_{S,Ed} - R_{A,Ed}\, l_s / 4$ in the lower sheeting
	Fasteners in tension (TF, BF) or shear (W)
Section B	Alternative W and TF: Bending moment $M_{B,Ed} = M_{S,Ed} - R_{A,Ed}\, l_s / 8$ in the lower sheeting
	Alternative BF: Bending moment $M_{B,Ed} = M_{S,Ed} - R_{A,Ed}\, l_s / 8$ and support reaction $R_{A,Ed}$ in the lower sheeting
Section C	Alternative W: Bending moment in the lower sheeting (no interaction with reaction force)
	Alternative TF: Interaction between bending moment in the lower sheeting and the reduced reaction force $R_{A,Ed} - S_1$, see Figure 11.12
	Alternative BF: Interaction between bending moment in the lower sheeting and the increased reaction force $R_{A,Ed} + S_1$, see Figure 11.12
Section D	Bending moment and shear force in the upper sheeting due to spring force S_1 and loading on the overlap part of the upper sheeting.

(6) The fasteners at the ends of the overlaps should be distributed over the height of the web in the order 1, 2, 3 ... according to Figure 11.11; if needed in two vertical rows.

11.4.2 Single overlap with overlapping lower sheeting (SOL-L)

(1) The fasteners are designed for the force in the springs of the static system according to Figure 11.10 with allowance for the slope of the web.

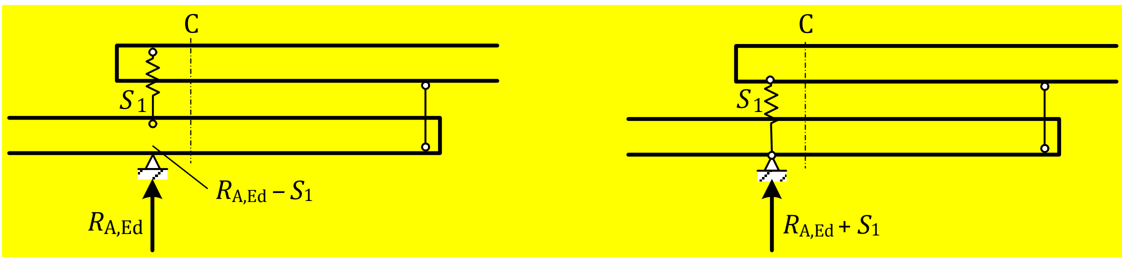
(2) If the gravity load is dominating, the fasteners at the cantilevered end (section D in Figure 11.11) may be omitted in alternative SOL-L. For uplift loading the sheeting should then be considered as simply supported.



(a) Single overlap type SOL-L (b) Alternative positions of fasteners

Figure 11.11 – Trapezoidal sheeting with single overlap at support (SOL-L)

(3) The lower sheeting should be designed according to 11.4.1.3(5) depending on position of fasteners W, FT or BF according to Figure 11.12.



(a) Alternative W or TF
(fasteners in the webs or crests)

(b) Alternative BF
(fasteners in the trough)

Figure 11.12 - Support reaction force on overlapping lower sheeting (SOL-L)

(3) The fasteners should be designed for a shear force per web using:

$$F_{v,Ed} = \frac{|M_{s,Ed}|}{2\, a \sin\varphi}\, b_R \qquad \text{for SOL-L} \qquad (11.21)$$

where:

$M_{s,Ed}$ is the bending moment at the support;

$V_{L,Ed}$ is the shear force to the left of the support;

φ is the slant of the web;

b_R is the pitch of the profile.

11.4.3 Single overlap with overlapping upper sheeting (SOL-U)

(1) In the case of overlapping upper sheeting (SOL-U according to Table 11.9) the support reaction is divided by the two sheeting according to Figure 11.13.

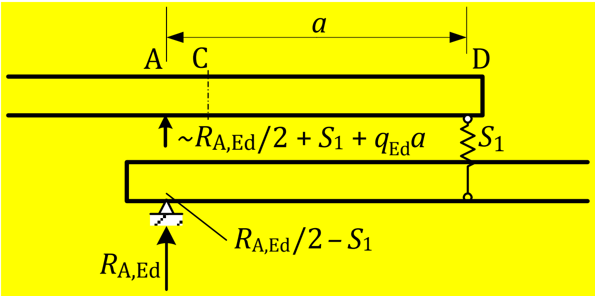


Figure 11.13 - Support reaction force on overlapping upper sheeting (SOL-U)

(2) The fasteners should be designed for a shear force per web using:

$$F_{v,Ed} = \frac{|M_{s,Ed}| + a\,|V_{L,Ed}|}{2\,a\sin\varphi}\,b_R \qquad \text{for SOL-U} \qquad (11.22)$$

where:

$M_{s,Ed}$ is the bending moment at the support;

$V_{L,Ed}$ is the shear force to the left of the support;

φ is the slant of the web;

b_R is the pitch of the profile.

11.4.4 Double overlap (DOL)

(1) The resistance of a double overlap region (interaction of bending moment and support reaction) (DOL according to Table 11.9) should be taken as the sum of the resistance of the bottom sheeting at the support and a bending moment in the upper sheeting taken as the resistance R_{wEd} according to (5) times a, but not more than the bending moment resistance of the sheeting itself, see Figure 11.14. The bending moment and the support reaction should be increased due to the double sheeting but not less than 1,1 times the moment for that of a continuous sheeting.

(2) If the moment is calculated with a model as in Figure 11.10, the moment resistance may be taken as the sum of the resistance of the two sheeting at the support.

(3) The screws at the end of the overlap should be checked for the shear force using:

$$S_1 = \frac{|M_{s,Ed}|/2 + q_{Ed}\,a^2\,/2}{2\,a\sin\varphi}\,b_R \qquad \text{for DOL} \qquad (11.23)$$

where:

$M_{s,Ed}$ is the bending moment at the support;

φ is the slant of the web;

b_R is the pitch of the profile.

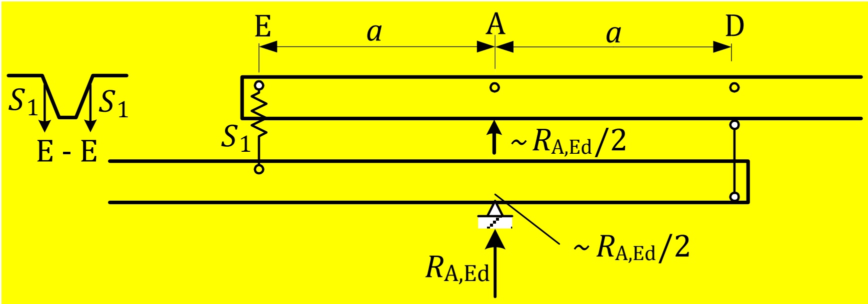


Figure 11.14 - Support reaction force and bending moment in double overlapping sheeting (DOL)

(4) The sheeting outside the ends of the overlap (D and E in Figure 11.14) should be checked for the bending moment at that section without interaction with the reaction force between the two sheeting.

(5) The end of the upper sheeting should be checked for reaction force S_1 and resistance R_{wEd} based on loaded length equal to the vertical distance between the fasteners in section E.

(6) If the gravity load is dominating and there is no resulting upward load, the fasteners at the cantilevered end D in Figure G.6 may be omitted.

11.4.5 Local reinforcement (CR)

(1) The resistance of sheeting reinforced with an extra sheeting above the continuous sheeting (CR according to Table 11.9 and Figure 11.15) may be set to the same as for double overlap in 11.4.3.

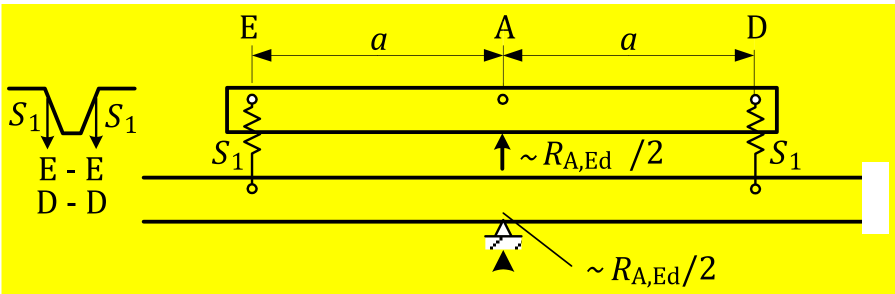


Figure 11.15 - Support reaction force on continuous sheeting with reinforcement (CR)

(2) Fasteners should be mounted in the webs at the cantilevered ends D and E and above the support A.

(3) If the reinforcing sheeting is lying under the continuous sheeting the resistance above the support may be stet to the sum of the resistances of the two sheeting. The sheeting outside the ends of the overlap should be checked according to 11.4.4(4).

11.5 Lateral and torsional restraints provided by sheeting or sandwich panels

11.5.1 Lateral restraints

(1) If sheeting or sandwich panels are connected to a purlin and the condition expressed by the Formula (11.24) is met, the purlin at the connection may be regarded as being laterally restrained in the plane of the sheeting or sandwich panels:

$$S \geq \left(EI_w \frac{\pi^2}{L^2} + GI_T + EI_z \frac{\pi^2}{L^2} 0{,}25\, h^2 \right) \frac{70}{h^2} \qquad (11.24)$$

where:

S	is the portion of the shear stiffness provided by the sheeting for the examined member connected to the sheeting at each rib or by sandwich panels.
I_w	is the warping constant of the purlin;
I_T	is the torsion constant of the purlin;
I_z	is the second moment of area about the minor axis of the cross-section of the purlin;
L	is the span of the purlin;
h	is the height of the purlin.

Formula (11.24) may also be used to determine the lateral stability of member flanges used in combination with other types of cladding than sheeting or sandwich panels, provided that the connections are of suitable design.

(2) The shear stiffness of trapezoidal sheeting connected to the purlin at each rib and connected in every side overlap may be calculated as follows:

$$S = 1000\sqrt{t^3}\left(50 + 10\sqrt[3]{b_{roof}}\right)\frac{s}{h_w} \quad \text{in } [N] \qquad (11.25)$$

where:

t	is the design thickness of sheeting in [mm];
b_{roof}	is the overall length of the steel diaphragm of the roof in [mm];
s	is the distance between the purlins in [mm];
h_w	is the profile depth of sheeting in [mm].

Formula (11.25) only applies, if shear connectors are set between shear sheets and end rafters (but not at internal rafters).

If the sheeting is connected to a purlin only every second rib, then S should be reduced and multiplied by 0,20.

For liner trays the shear stiffness is S_v times distance between purlins, where S_v is calculated according to 11.6.5(6).

NOTE: For additional information about the shear stiffness S for trapezoidal sheeting, see ECCS guidance (NOTE in 11.6.1(3)) or design by testing.

(3) The shear stiffness of sandwich panels connected to the purlin using at minimum one pair and in maximum 4 pairs of fasteners may be calculated as

$$S = \frac{k_v}{2B} \sum_{k=1}^{n_k} c_k^2 \qquad (11.26)$$

where:

k_v is the shear stiffness determined from Table 11.11.

B is the panel width;

c_k is the distance between the two fasteners of a pair k;

n_k is the number of pairs of fasteners;

The sandwich panels should locate perpendicular to the connected purlins.

NOTE: Additional information about the shear stiffness S for sandwich panels (e.g. depending on the edge distance of connectors) see ECCS Publication 135/CIB Publication 379: European Recommendations on the Stabilization of Steel Structures by Sandwich Panels (2014) [2].

Table 11.11 - Shear stiffness k_v [kN/mm] of fasteners :*1)

Steel grade and nominal thickness of inner face sheet	**S 220GD**	**S 280GD**	**≥ S 320GD**
0.40 mm	1.6	1.9	2.0
0.50 mm	2.0	2.3	2.5
0.63 mm	2.4	2.9	3.1
0.75 mm	2.8	3.3	3.6

NOTE:*1) Linear interpolation is allowed with respect to steel grade and nominal thickness of inner face sheet.

The application range of Table 11.11 is given in Table 11.12. The core material has no effect to k_v values.

Table 11.12 -- Application range of Table 11.1

Nominal diameter d of fastener	5.5 mm ≤ d ≤ 8.0 mm
Total panel thickness D	D ≥ 40 mm
Nominal thickness t_{F2} of inner face sheet	0.40 mm ≤ t_{F2} ≤ 1.00 mm
Nominal thickness t_{sup} of supporting structure	1.50 mm ≤ t_{sup}

11.5.2 Rotational restraint

(1) The rotational restraint provided to the purlin by the sheeting or sandwich panels ~~that is~~ connected to its top flange, should be modelled as a rotational spring acting at the top flange of the purlin, see Figure 11.16. The total rotational spring stiffness C_D should be determined from:

$$C_D = \frac{1}{\frac{1}{C_{D,A}} + \frac{1}{C_{D,B}} + \frac{1}{C_{D,C}}} \qquad (11.27)$$

where:

$C_{D,A}$ is the rotational stiffness of the connection between the sheeting or sandwich panel and the purlin;

$C_{D,B}$ is the rotational stiffness due to distortion of the cross-section of the purlins;

$C_{D,C}$ is the rotational stiffness corresponding to the flexural stiffness of the sheeting or sandwich panel.

(2) The values of $C_{D,A}$, $C_{D,B}$, $C_{D,C}$ may be may be obtained either by calculation or testing (see Clause 12 and Annex A.5) or by a combination of testing and calculation.

(3) The value of $C_{D,C}$ may be taken as the minimum value obtained from calculational models of the type shown in Figure 11.16, taking account of the rotations of the adjacent purlins and the degree of continuity of the sheeting or sandwich panels, using:

$$C_{D,C} = m/\theta \qquad (11.28)$$

where:

m is the applied moment per unit width of sheeting or sandwich panels, applied as indicated in Figure 11.16;

θ is the resulting rotation, measured as indicated in Figure 11.16 [radians].

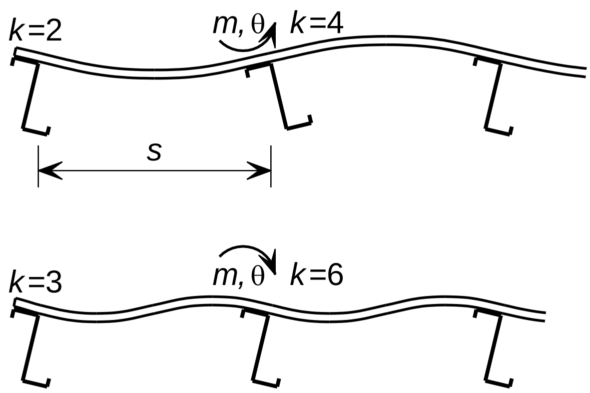


Figure 11.16 - Model for calculating $C_{D,C}$

(4) Alternatively a conservative value of $C_{D,C}$ may be obtained from:

$$C_{D,C} = \frac{k\, EI_{eff}}{s} \qquad (11.29)$$

in which k is a numerical coefficient, with values as follows:

- end, upper case of Figure 11.16 $k = 2$;
- end, lower case of Figure 11.16 $k = 3$;
- mid, upper case of Figure 11.16 $k = 4$;
- mid, lower case of Figure 11.16 $k = 6$;

where:

I_{eff} is the effective second moment of area per unit width of the sheeting or sandwich panel;

s is the spacing of the purlins.

(5) For trapezoidal sheeting connected to the top flange of the purlin, provided that the sheet-to-purlin fasteners are positioned centrally on the flange of the purlin, the value of $C_{D,A}$ may be determined as follows (see Table 11.13):

$$C_{D,A} = C_{100} \cdot k_{ba} \cdot k_t \cdot k_{bR} \cdot k_A \cdot k_{bt} \tag{11.30}$$

where:

$k_{ba} = (b_a / 100)^2$ if $b_a < 125$ mm ;

$k_{ba} = 1,25\ (b_a / 100)$ if $125 \text{ mm} \leq b_a < 200$ mm ;

$k_t = (t_{nom} / 0,75)^{1,1}$ if $t_{nom} \geq 0,75$ mm ; positive position;

$k_t = (t_{nom} / 0,75)^{1,5}$ if $t_{nom} \geq 0,75$ mm ; negative position;

$k_t = (t_{nom} / 0,75)^{1,5}$ if $t_{nom} < 0,75$ mm;

$k_{bR} = 1,0$ if $b_R \leq 185$ mm ;

$k_{bR} = 185 / b_R$ if $b_R > 185$ mm ;

For gravity load:

$k_A = 1,0 + (A\text{-}1,0)\cdot 0,08$ if $t_{nom} = 0,75$ mm ; positive position;

$k_A = 1,0 + (A\text{-}1,0)\cdot 0,16$ if $t_{nom} = 0,75$ mm ; negative position;

$k_A = 1,0 + (A\text{-}1,0)\cdot 0,095$ if $t_{nom} = 1,00$ mm ; positive position;

$k_A = 1,0 + (A\text{-}1,0)\cdot 0,095$ if $t_{nom} = 1,00$ mm ; negative position;

- linear interpolation between $t = 0,75$ and $t = 1,0$ mm is allowed
- for $t < 0.75$ mm the Formula is not valid;
- for $t > 1$ mm, the Formula should be used with $t = 1$ mm

For uplift load:

$k_A = 1,0$;

$$k_{bt} = \sqrt{\frac{b_{T,max}}{b_T}}$$ if $b_T > b_{T,max}$, otherwise $k_{bT} = 1$;

A [kN/m] ≤ 12 kN/m load introduced from sheeting to beam;

where:

b_a is the width of the purlin flange [in mm];

b_R is the corrugation width [in mm];

b_T is the width of the sheeting flange through which it is fastened to the purlin;

$b_{T,max}$ is given in Table 11.13;

C_{100} is a rotation coefficient, representing the value of $C_{D,A}$ if b_a = 100 mm.

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 11.13.

Table 11.13 - Rotation coefficient C_{100} for trapezoidal steel sheeting

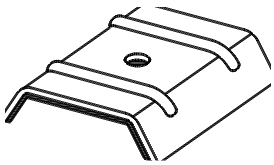
Positioning of sheeting		Sheet fastened through		Pitch of fasteners		Washer diameter	C_{100}	$b_{T,max}$
Positive [a]	Negative [a]	Trough	Crest	$e = b_R$	$e = 2b_R$	[mm]	[kNm/m]	[mm]
For gravity loading:								
×		×		×		22	5,2	40
×		×			×	22	3,1	40
	×		×	×		K_a	10,0	40
	×		×		×	K_a	5,2	40
	×	×		×		22	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;

b_T is the width of the sheeting flange through which it is fastened to the purlin.

K_a indicates a steel saddle washer with t ≥ 0,75 mm, see below

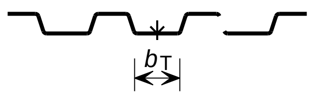


The values in this Table are valid for:

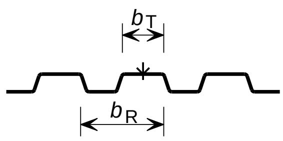
- sheet fastener screws of diameter: Φ = 6,3 mm;
- steel washers of thickness: t_w ≥ 1,0 mm.

Sheet fastened:

- through the trough:



- through the crest:



[a] The position of sheeting in positive when the narrow flange is on the purlin and negative when the wide flange is on the purlin.

(6) For liner trays with t_{nom} ≥ 0,75 mm and b_u ≤ 600 mm, a connection rigidity of $C_{D,A}$ = 1,7 kNm/m may be used if a more precise analysis has not been carried out. The liner tray shall be fixed with at least two fasteners per panel and support, with a distance ≤ 75 mm between the fastener and the web, see EN 1090-4, Figure 5.

NOTE: This also applies to ~~for~~ liner trays with perforation according to Clause 11.3.

(8) For sandwich panels, uplift loading causes an indentation of the fastener and gap between the upper flange of the purlin and the inner face of the sandwich panel, so reduced rotational stiffness $C_{D,A}$ should be used based on testing. For gravity loading the rotational stiffness $C_{D,A}$ is:

- for purlins symmetric about minor axis:

$$C_{D,A} = 0{,}75\ c_1\ E_{C,t,\theta}\ b^2 \tag{11.31}$$

- for ∑-, Z-, U- or C-purlins:

$$C_{D,A} = 0{,}75\ c_2\ E_{C,t,\theta} \tag{11.32}$$

where:

$$E_{C,t,\theta} = \frac{E_C}{1+\varphi_{\theta,t}}\sqrt{k_1^3}$$

$$k_1 = \frac{\sigma_{w,+80°C}}{\sigma_{w,+20°C}}$$ (for defining the factor k_1 see EN 14509:2013, A.5.5.5)

c_1 and c_2 are given in Table 11.14.

Other parameters and the application range is given in Table 11.15.

Table 11.14 - Parameters c_1 and c_2.

Core material	Geometry of outer face (at the head of fastener)	c_1 [-]	c_2 [m²]
PUR/PIR and XPS/EPS	profiled [a]	0.180	6.48×10^{-4}
	slightly profiled/flat	0.142	5.11×10^{-4}
Mineral wool	profiled [a]	0.089	3.20×10^{-4}
	slightly profiled/flat	0.048	1.73×10^{-4}
[a] Depth of profiling ≥ 30 mm.			

Table 11.15 - Application range of Table 11.14

Parameter depending on the duration of loading	$\varphi_{\theta,t}$
Core materials PUR/PIR and XPS/EPS	$\varphi_{\theta,2000} = 1.29$
Core materials PUR/PIR and XPS/EPS	$\varphi_{\theta,100000} = 1.83$ [a]
Core material mineral wool	$\varphi_{\theta,2000} = 1.35$
Core material mineral wool	$\varphi_{\theta,100000} = 2.13$ [a]
Width b [mm] of the flange of the purlin symmetric about minor axis	60 mm ≤ b ≤ 180 mm
Width b [mm] of the flange of ∑-, Z-, U- and C-purlin	60 mm ≤ b ≤ 80 mm
Elastic modulus of the core, mean value E_C of compressive modulus E_{Cc} and tensile modulus E_{Ct}. $E_C = 0.5\ (E_{Cc} + E_{Ct})$ [N/mm²]	2.0 MPa ≤ E_C ≤ 6.0 MPa
Sheet thickness t_{cor} of both faces of panel	0.38 ≤ t_{cor} ≤ 0.71 mm
Characteristic compression strength f_{Cc} of the core material	core materials PUR/PIR and XPS/EPS: f_{Cc} ≥ 0.08 MPa core material mineral wool: f_{Cc} ≥ 0.05 MPa
Characteristic tensile strength f_{Ct} of the core material	f_{Ct} ≥ 0.06 MPa
Diameter of washer d_W	d_W ≥ 16 mm
[a] Values given for a duration loading of t = 100 000 hours may also be applied for longer durations.	
If higher values of parameters b, E_C and t_{cor} occur, the calculation procedure is applicable, but the values should be reduced to the corresponding upper limits of the application range. If lower values occur, tests according EN 14509-2 shall be performed.	

11.6 Stressed skin design

11.6.1 General

(1) 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 11.6.

(2) The provisions given in this Clause should be applied only to sheet diaphragms that are made of steel.

(3) 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.

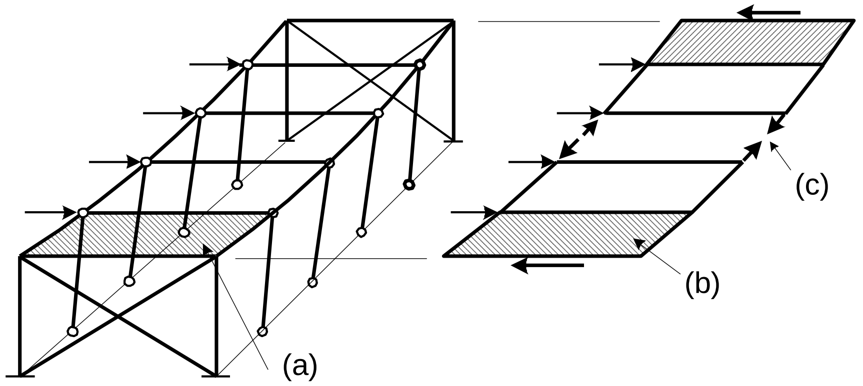
NOTE: Additional information on the verification of such diaphragms are given in ECCS Publication No. 88 (1995): *European recommendations for the application of metal sheeting acting as a diaphragm*. [1]

11.6.2 Diaphragm action

(1) 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 11.17 and 11.18.

(3) Similarly, rectangular wall panels may be treated as bracing systems that act as shear diaphragms to resist in-plane forces.



Key
(a) Sheeting
(b) Shear field in sheeting
(c) Flange forces in edge members

Figure 11.17 - Stressed skin action in a flat-roof building

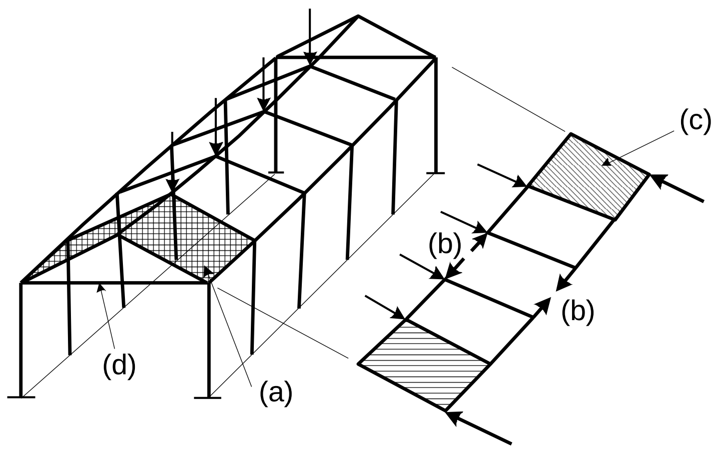
11.6.3 Necessary conditions

(1) 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;
- in sheeting with the corrugation oriented in the longitudinal direction of the roof the flange forces due to diaphragm action may be taken up by the sheeting.

(2) Stressed skin design may be used in the roofs, floors and facades of buildings.

(3) Stressed skin diaphragms may 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.



Key
(a) Sheeting
(b) Flange forces in edge members
(c) Shear field in sheeting
(d) Gable tie required to resist forces in roof sheeting

Figure 11.18 - Stressed skin action in a pitched roof building

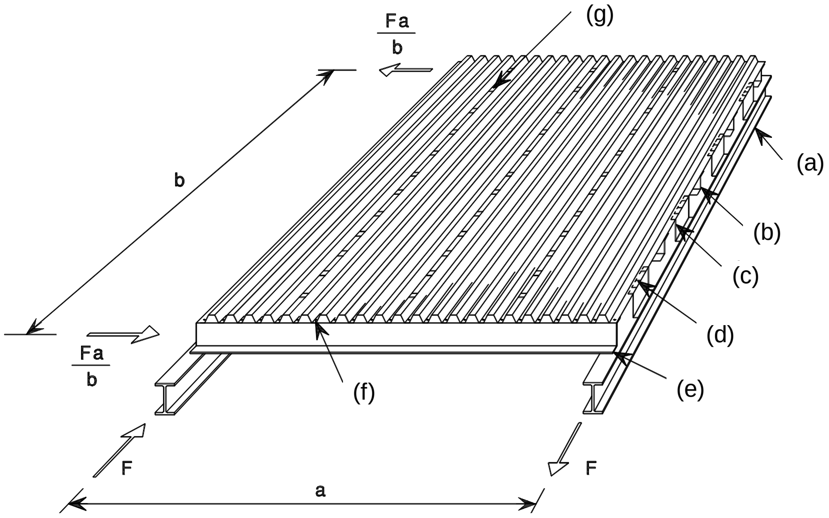
11.6.4 Profiled steel sheet diaphragms

(1) In a profiled steel sheet diaphragm, see Figure 11.19, both ends of the sheets should 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 should 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) The seams between adjacent sheets should 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 should not exceed 500 mm.

(3) The distances from all fasteners to the edges and ends of the sheets should 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 (the area of the surface of the diaphragm taken into account for the calculations) may be introduced if justified by detailed calculations. Areas that contain larger openings should be split into smaller areas, each with full diaphragm action.

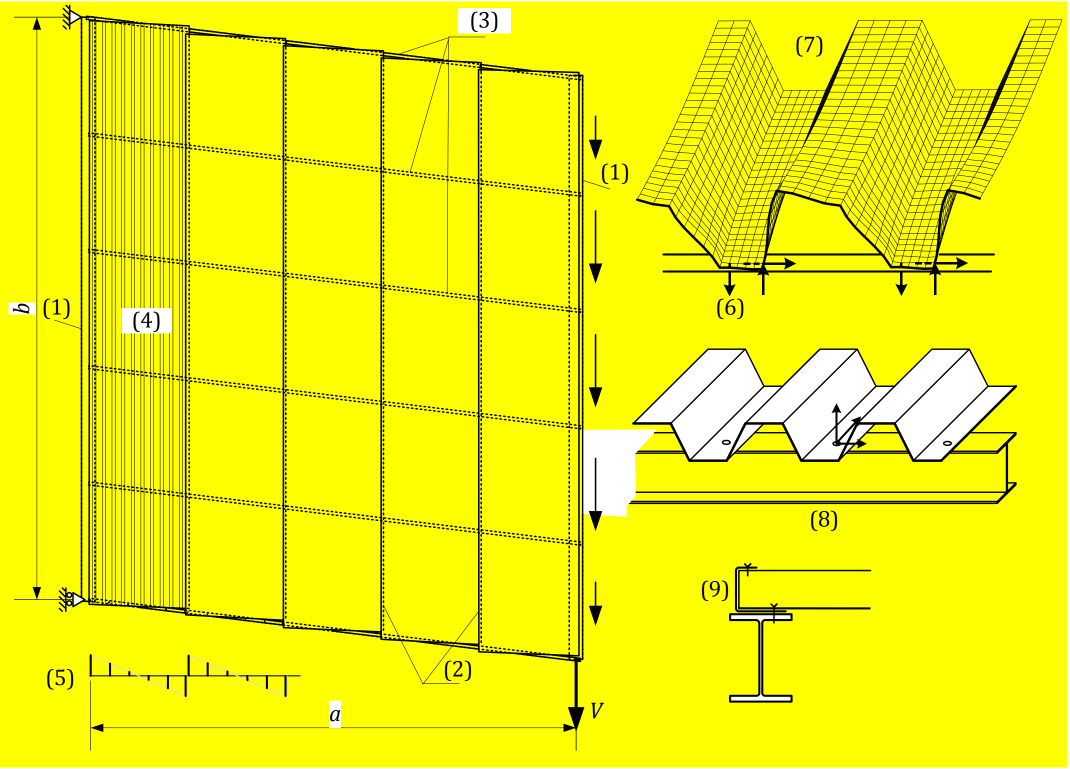


Key
(a) Rafter
(b) Purlin
(c) Shear connector
(d) Sheet-to-shear connector fastener
(e) Purlin
(f) Sheet-to-purlin fastener
(g) Seam fastener

Figure 11.19 - Arrangement of an individual panel

(5) All sheeting that also forms part of a stressed-skin diaphragm should 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 should then be verified that the shear stress due to diaphragm action does not exceed $0{,}25 f_{yb}/\gamma_{M1}$.

(6) The resistance of the seam connections and the sheet-to-member connections parallel to the corrugations should be designed for the acting design load. The shear resistance of the sheet-to-purlin fasteners and sheet-to-rafter fasteners (for sheeting connected directly to main beams) should exceed the acting design load by at least 50% unless the prying action due to profile distortion (see Figure 11.20(6)) and the force perpendicular to the purlin due to slip in seams (see Figure 11.20(5)) are accounted for by appropriate calculation. Prying action due to profile distortion can be avoided by using end reinforcement, see Figure 11.20(9). The combination of shear force and wind uplift should be checked in accordance with Clause 10.3(8). For any other type of failure the resistance should exceed the acting design load by at least 25%.



Key
1 Rafter
2 Seam fasteners
3 Purlins
4 End sheet
5 Shear force due to slip in seam fasteners
6 Shear and prying action effect at end section
7 Mid-section
8 Forces acting on the sheet-to-purlin fasteners
9 End reinforcement preventing the prying action effect at end section

Figure 11.20 - Arrangement of an individual panel

11.6.5 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 11.21.

(3) An accurate evaluation of deflections due to fasteners may be made using a similar procedure to that for trapezoidal sheeting.

(4) The shear flow $T_{v,Ed}$ due to ultimate limit states design loads should not exceed $T_{v,Rd}$ given by:

$$T_{V,Rd} = 8{,}43\, E \sqrt[4]{I_a (t/b_u)^9} \qquad (11.33)$$

where:

I_a is the second moment of area of the wide flange about it own centroid, see Figure 11.6;

b_u is the overall width of the wide flange.

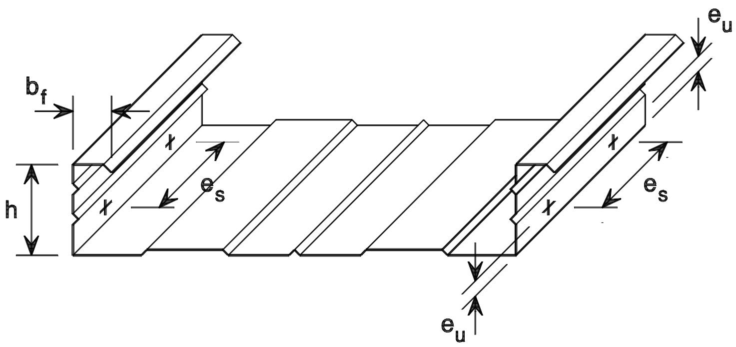


Figure 11.21: - Location of seam fasteners

(5) The shear flow $T_{V,ser}$ due to serviceability design loads should not exceed $T_{V,Cd}$ given by:

$$T_{V,Cd} = \frac{S_V}{375} \qquad (11.34)$$

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:

$$S_V = \frac{\alpha\, L\, b_u}{e_s (b - b_u)} \qquad (11.35)$$

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 conservatively be taken as equal to 2000 N/mm unless more accurate values are derived from tests.

12 Design assisted by testing

(1) Clause 12 should be applied in conjunction with the principles for design assisted by testing given in EN 1990 and EN 1993-1-1. This Clause provides additional requirements specific to cold-formed members and sheeting.

(2) Each test specimen should be similar in all respects to the component or structure which it represents.

(3) In the case of test on an assembly (e.g. an internal support test on an overlap or sleeve system), the test specimens shall be assembled in accordance with the execution specifications.

(4) The supporting devices used ~~for~~ in the tests should preferably provide end conditions which closely ressemble those in the actual structure. If this cannot be achieved, less favourable end conditions which decrease the load carrying capacity or increase the flexibility should be used. If no rotational restraint from the supports can be assumed pin and roller supports or spherical bearings should be used as applicable.

(5) If local buckling governs the resistance of the cross-section, the specimen should have a length of at least three times the width of the widest plate element. In the case of a cross-section with edge or intermediate stiffeners, it should be ensured that the specimen is long enough to accommodate multiple distortional half-waves.

(6) Before testing, the cross-sectional dimensions of the specimen should be checked to ensure that they are within the permitted tolerances specified by the relevant product standard.

(7) 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.

(8) The loads shall either be applied incrementally or continuously. When the load is applied incrementally, the increments shall be chosen so that the behaviour that is under observation is clearly defined. When the load is applied continuously, the rate of loading shall be slow enough to ensure that static conditions prevail.

(9) At each stage of loading, the displacements and/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.

(10) The deflections at supports at both ends of the test specimen should also be considered.

(11) The test result should be taken as corresponding to the maximum value of the loading at the time of failure.

(12) Failure of a test specimen should be considered to have occurred in any of the following cases:

- at collapse or fracture;
- if a crack indicates in a vital part of the specimen;
- if the displacements are excessive.

(13) For each test series, formal documentation shall be prepared providing all the relevant data, so that the test series can be accurately reproduced.

(14) The following test procedures shall be carried out as specified in Annex A:

- tests on single beams and columns, see A.2;
- tests on structures and sub-assemblies , see A.3;
- tests on profiled sheeting and liner trays, see A.4;
- tests on beams restrained by sheeting, see A.5;
- tests on fastenings, see A.6;
- tests on storage equipment, see A.7;
- evaluation of test results to determine design values, see A.8.

Annex A [normative] – Testing procedures

A.1 General

(1) This Annex A gives additional information to Clause 12.

NOTE 1: Further information on testing can be set by the National Annex for use in a country.

NOTE 2: Conversion factors for existing test results to be equivalent to the outcome of standardised tests according to this Annex can be set by the National Annex for use in a country.

(2) This Annex covers:

- tests on single beams and columns, see Clause A.2;
- tests on structures and portions of structures, see Clause A.3;
- tests on profiled sheeting and liner trays, see Clause A.4;
- tests on beams restrained by sheeting, see Clause A.5;
- tests on fastenings, see Clause A.6;
- tests on storage equipment, see Clause A.7;
- evaluation of test results to determine design values, see Clause A.8;

(3) The accuracy of all measurements should be compatible with the magnitude of the measurement concerned and in no case should the error exceed ± 1% of the value to be determined. The following accuracies should also be attained for:

- the overall dimensions (width, depth and length): ± 1,0 mm;
- the widths of plane elements of the cross-section: ± 1,0 mm;
- the radii of bends: ± 1,0 mm;
- the inclinations of plane elements: ± 2,0°;
- the angles between flat surfaces: ± 2,0°;
- the locations and dimensions of intermediate stiffeners: ± 1,0 mm;
- the thickness of the material: ± 0,01 mm;
- all cross-sectional measurements: 0,5 % of the nominal values.

In addition, the locations of fasteners and of all components relative to each other should be recorded.

A.2 Tests on single beams and columns

A.2.1 Full cross-section compression tests

A.2.1.1 Stub column tests

(1) Stub column tests may be used to allow for the effects of local and distortional buckling in thin gauge cross-sections, by determining the value of the ratio $\beta_A = A_{eff} / A$ and the location of the centroidal axis of the effective cross-section.

(2) The lengths of specimens with perforated cross-sections should include at least five pitches of the perforations, and should be such that the specimen is cut to length midway between two perforations.

(3) 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}.

(4) The cut ends of the specimen should be flat, and should be perpendicular to its longitudinal axis.

(5) 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.

(6) 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.

(7) In the case of open cross-sections, possible spring-back may be corrected.

(8) 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 ultimate strength fu and the yield strength 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-forming (for example increasing the yield strength by introducing a deformation that is subsequently removed).

A.2.1.2 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 Clause A.2.1.1.

(3) A series of tests on axially loaded specimens with different member lengths 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 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 ($\bar{\lambda}$) and numbers of tests (N)

$\bar{\lambda}$	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 Clause A.2.1.1(8);
- overall lack of straightness imperfections compared to standard production output, see 12(6)
- type of end or intermediate restraint (flexural, torsional or both).

(6) Overall lack of straightness may be taken into account as follows:

a) Determine the elastic critical compression load of the member by an appropriate analysis with initial bow equal to test sample: $F_{\mathrm{cr,bow,test}}$

b) As a) but with an initial bow equal to the maximum allowed according to the product specification: $F_{\mathrm{cr,bow,max,nom}}$

c) Additional correction factor: $F_{\mathrm{cr,bow,max,nom}} / F_{\mathrm{cr,bow,test}}$

A.2.2 Full cross-section tension tests

(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.2.3 Full cross-section bending tests

(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 transversal 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 s (span) but not more than 0,33 s (span). These loads should be applied through the shear centre of the cross-section. The section should be torsionally restrained at the load points. 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_{yb};
- the ultimate strength f_u and the yield strength f_{yb};
- differences between restraints used in the test and those available in service;
- the support conditions.

A.3 Tests on structures and sub-assemblies

A.3.1 Acceptance test

(1) This acceptance test may be used as a non-destructive test to confirm the structural performance of a structure or sub-assemblies .

(2) The test load for an acceptance test should be taken as equal to the sum of:

- 1,0 × (the actual self-weight present during the test);
- 1,15 × (the remainder of the permanent load);
- 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 repeated. 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 being no changes between a set of adjacent readings 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.3.2 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 strength test the specimen should first pass the acceptance test detailed in Clause A.3.1.

(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_{Ed} specified for ultimate limit state verifications by calculation, using:

$$F_{\mathrm{str}} = \gamma_{\mathrm{Mi}}\ \mu_{\mathrm{F}}\ F_{\mathrm{Ed}} \tag{A.1}$$

where:

μ_F is the load adjustment coefficient;

γ_{Mi} is the partial factor of the ultimate limit state.

(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, may be made using the provisions of this Annex A 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 fym 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 $f_{\mathrm{yb,obs}}$ 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 Clause A.8.2.

(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.3.3 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 Clause A.3.2.

(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 Clause A.3.2. 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 overall permanent distortion is likely to have occurred. In some cases overall deformation defines the test limit.

A.3.4 Calibration test

(1) A calibration 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.

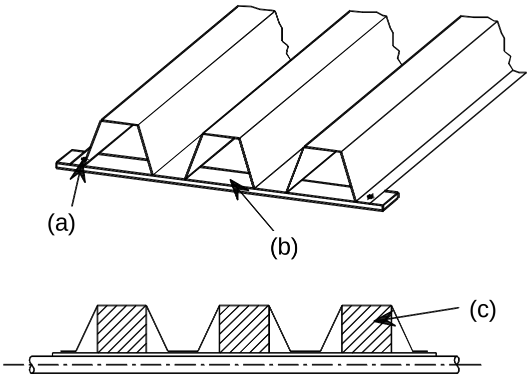
A.4 Tests on profiled sheeting and liner trays

A.4.1 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 and other types of sheeting.

(2) Loading may be applied through air bags or in a vacuum chamber or by steel or timber cross beams arranged to approximate uniformly distributed loading.

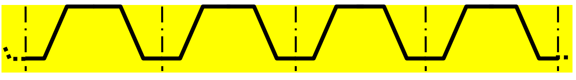
(3) To prevent spreading of corrugations, transverse ties or other appropriate test accessories such as timber blocks may be applied to the test specimen. Some examples are given in Figure A.1.



Key
(a) Rivet or screw
(b) Transverse tie (metal strip)
(c) Timber blocks

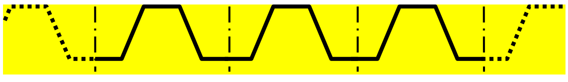
Figure A.1 - Examples of appropriate test accessories

(4) In order to comply with Clause 7.3(5) test specimens comprised of sheeting with more than two ribs should consist of complete ribs, as measured between the centre lines of ribs. If a longitudinal edge of the specimen is in tension, the outstand part of it should be removed, as shown in Figure A.2. If a longitudinal edge rib is in compression, the edge rib should be removed, as shown in Figure A.3.



NOTE: Parts to be removed are drawn in dotted lines

Figure A.2 - Procedure to cut the longitudinal ~~lateral~~ edges in tension



NOTE: Parts to be removed are drawn in dotted lines

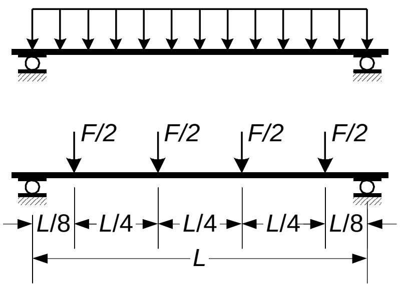
Figure A.3 - Procedure to cut the longitudinal edges in compression

A.4.2 Single span test

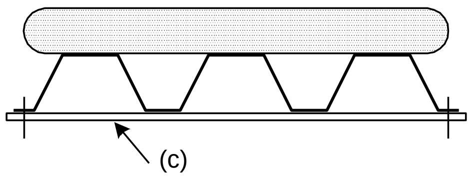
(1) A test set-up equivalent to that shown in Table A.2 may be used to determine the midspan moment resistance (in the absence of shear force) and the effective flexural stiffness.

(2) The flexural stiffness should be determined from the load-deflection diagram as the secant stiffness corresponding to a moment equal to 60% of the ultimate moment.

Table A.2 - Test set-up for single span tests

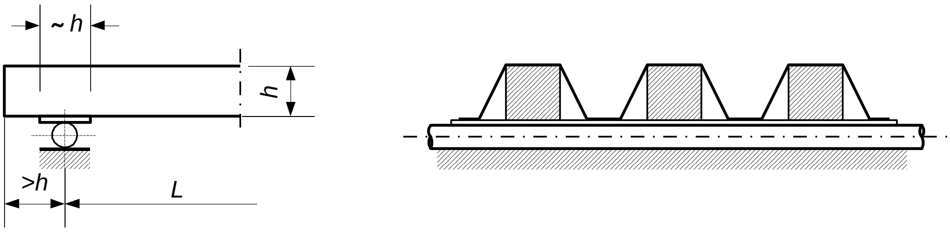


(a) Uniformly distributed loading and an example of alternative equivalent line loads

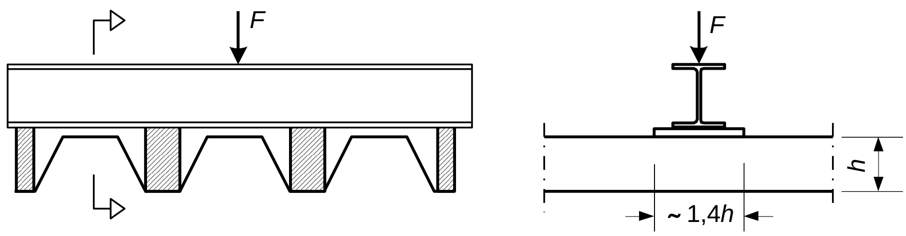


(b) Distributed loading applied by an airbag (alternatively by a vacuum test rig)

Key (c) Transverse tie



(c) Example of support arrangements for preventing distortion



(d) Example of method of applying a line load

(3) For the purposes of assessment (see Clause A.8.3), tests on different specimens may be grouped into families if they consist of specimens which comply with all of the following:

- common nominal yield strength f_{yb}
- common or different testing spans
- common or different nominal thicknesses t_{nom}

(4) For each nominal thickness, at least two specimens shall be tested.

(5) Specimens consisting of perforated sheeting or sheeting with holes should not be grouped into the same family with specimens consisting of unperforated sheeting or sheeting without holes.

A.4.3 Double span test

(1) The test set-ups shown in Figures A.4 and A.5 may be used to determine the global resistance of sheeting or liner trays (for a given support width b_B) which are continuous over two equal spans and subject to a ~~under~~ uniformly distributed load.

NOTE: Clause A.2.1(3) does not apply to these set-ups.

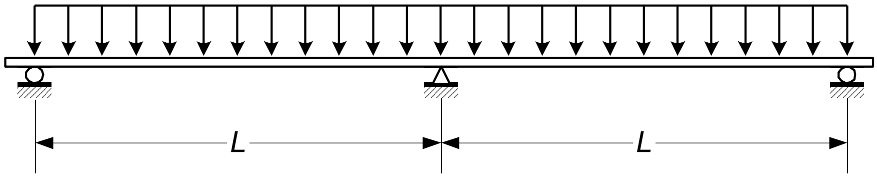


Figure A.4 - Test setup for double span tests

(2) The uniformly distributed load (Figure A.5) may be applied using, for instance, an air bag or a vacuum chamber.

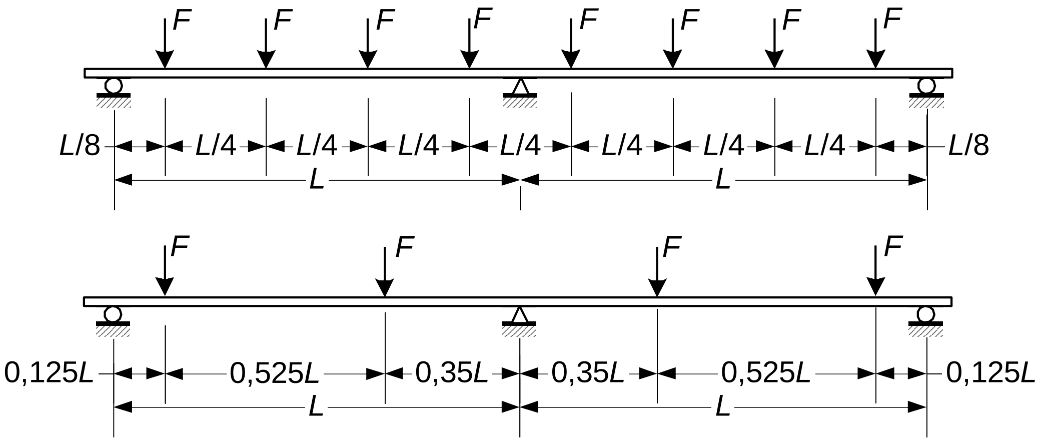


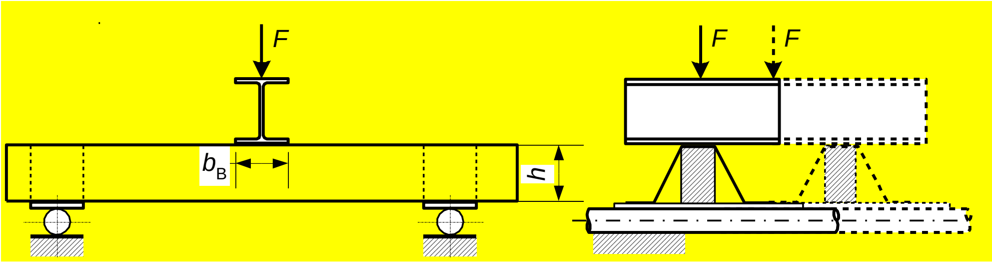
Figure A.5 - Examples of equivalent line loads for two double span tests

(3) These test configurations do not provide direct information about the behaviour under the combined action of a bending moment and an internal support reaction. To determine the moment resistance, as well as the moment rotation behaviour at the support, and to allow for an adequate interpretation of the test results, the support reactions at all supports should be measured.

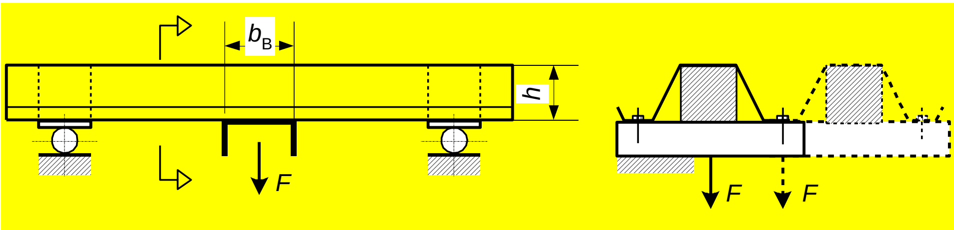
A.4.4 Internal support test

(1) The test set-up shown in Table A.3 is suitable to determine the behaviour of continuous sheeting at internal supports under combined moment and reaction force and for a given support width b_B. This test is applicable to equal or unequal spans under uniformly or not-uniformly distributed load.

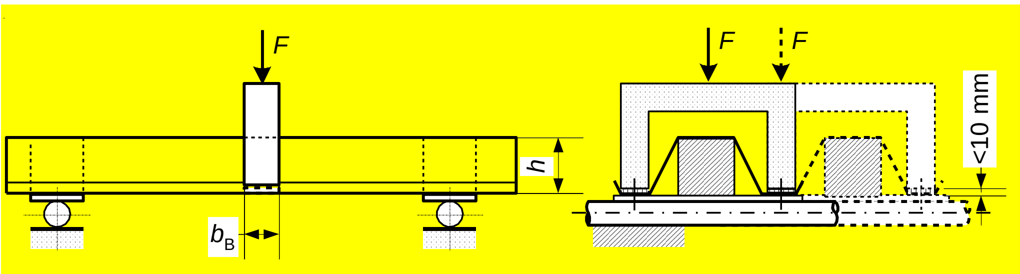
Table A.3 - Test set-up for internal support test



(a) Internal support under gravity loading



(b) Internal support under uplift loading



(c) Internal support with loading applied to tension flange (geometry of contact area equal to geometry of fastener)

(2) In order to obtain the same ratio of the bending moment to the support reaction as in the case of two continuous spans of length L, the test span s may be taken as:

$$s = 0{,}4\,L \tag{A.2}$$

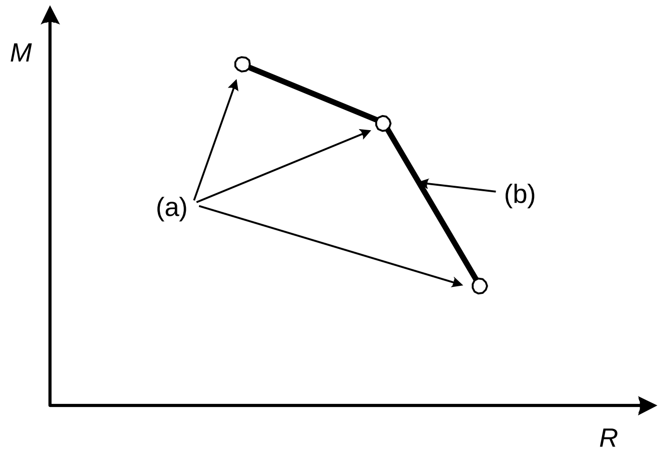
(3) To obtain the behaviour over a range of different spans s, tests should be carried out for at least 3 different spans s, preferably with equal intervals.

(4) 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:

$$M = \frac{s\,R}{4} \tag{A.3}$$

The influence of the dead load should be added when calculating the value of the moment M according to Formula (A.3).

(5) The pairs of values of M and R for each value of s should be plotted as shown in Figure A.6. Pairs of values for intermediate combinations of M and R may then be determined by linear interpolation. Instead of a polygonal line (b), as shown in the Figure A.6, a conservatively chosen straight or curved may be adopted, using appropriate interaction Formulae.



Key
(a) test results for different test spans
(b) linear interpolation

Figure A.6 - Relation between support moment *M* and support reaction *R*

(6) The net deflection at the point of load application C in Figure A.7 should be obtained from the overall measured values by considering 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.

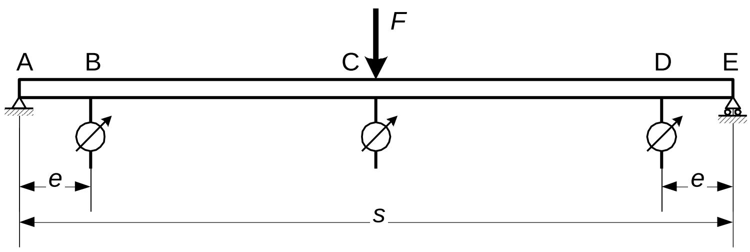


Figure A.7 - Test set-up for internal support tests

(7) For each test the applied load should be plotted against the corresponding net deflection, see Figure A.8. From this plot, the rotation Θ should be obtained for a range of values of the applied load using:

$$\theta = \frac{2\left(\delta_{pl} - \delta_e - \delta_{el}\right)}{0{,}5\, s - e} \tag{A.4}$$

$$\theta = \frac{2\left(\delta_{pl} - \delta_e - \delta_{lin}\right)}{0{,}5\, s - e} \tag{A.5}$$

where:

δ_{el} is the net deflection for a given load on the rising part of the curve, before F_{max};

δ_{pl} is the net deflection for the same load on the falling part of the curve, after F_{max};

δ_{lin} is the fictive net deflection for a given load, that would be obtained with a linear behaviour, see Figure A.8;

δ_e is the average deflection measured at a distance e from the support, see Figure A.7;

s is the test span;

e is the distance between a deflection measurement point and a support, see Figure A.7.

Formula (A.4) is used when analyses are done based on the effective cross-section. Formula (A.5) is used when analyses are done based on the gross cross-section.

If timber blocks according to Clause A.4.1(3) are used, measurement of the deflection δ_e at points B and D is not necessary and δ_e and e may be taken as zero.

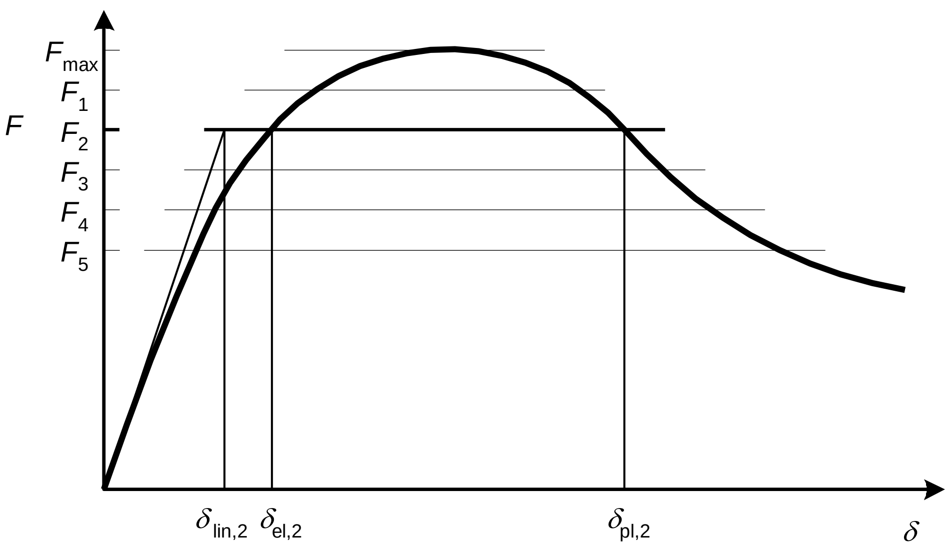
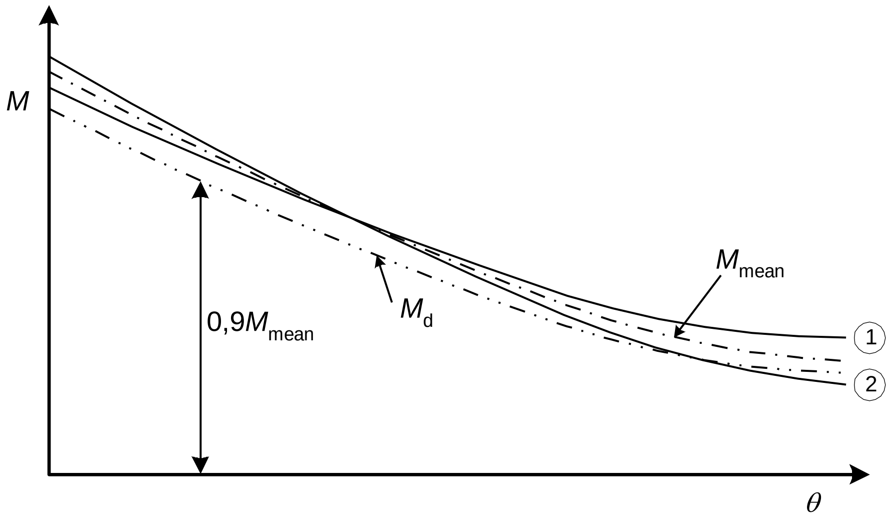


Figure A.8 - Relation between load *F* and deflection *δ*

(8) The relationship between M and θ should then be plotted for each test at a given test span s corresponding to a given actual value of span L as shown in Figure A.9. The design M - θ characteristic for the moment resistance of the sheeting 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 actual span L.



Key
M_{mean} mean value
M_d design value

Figure A.9 - Derivation of moment-rotation (M - θ) relationship

(9) A conservative simplified design may be carried out using a constant value of the support moment $M_{d,lim}$ corresponding to an arbitrary chosen limit rotation θ_{lim} (Figure A.9). As part of the design process it should be verified in design that the rotation in the ultimate limit state does not exceed the limit rotation θ_{lim}.

NOTE: Actual rotation θ is usually less than 0.15 Rad.

(10) For the purposes of assessment (see Clause A.8.3), tests on different specimens may be grouped into families if they consist of specimens which comply with all of the following:

- common nominal yield strength f_{yb}
- common nominal thicknesses t_{nom}
- common support width b_B
- common or different testing spans s

For each span, at least two specimens shall be tested.

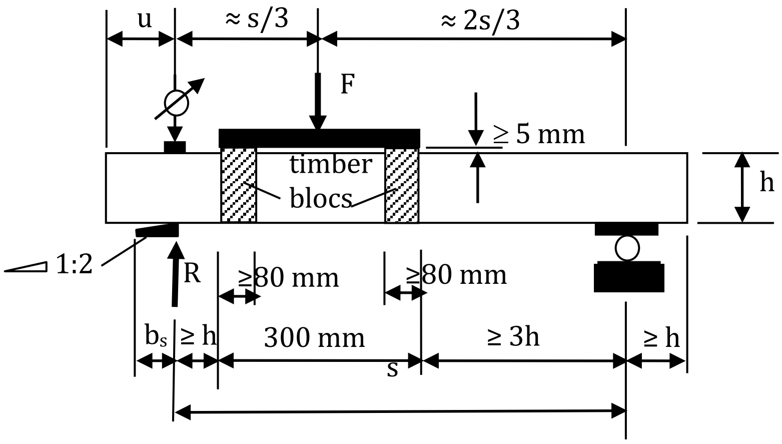
Specimens of perforated sheeting or sheeting with holes (for example sheeting used as decking for slabs of composite beams) should not be grouped into the same family with specimens of un-perforated sheeting or sheeting without holes.

(11) If the characteristics are obtained by tests with different support width b_B, the characteristics for an intermediate support length may be determinate by linear interpolation.

A.4.5 End support test

(1) The test set-up shown in Figure A.10 may be used to determine the shear and web crippling resistance of sheeting at an end support.

NOTE: End support tests performed according to the EN 1993-1-3:2016 lead to more conservative results.



Key
b_s support length
u length from internal edge of end support to end of sheet

Figure A.10 - Test set-up for end support tests

(2) The maximum value of the reaction R corresponding to collapse of the specimen may be considered as the resistance at the end support.

NOTE: Despite load application onto the bottom flange using timber blocks, in some cases failure occurs by bending in the area where the load is introduced, leading to more conservative results.

(3) If the characteristics are obtained through tests with different support lengths b_s, the characteristics for an intermediate support length may be determined by interpolation, assuming linear behaviour.

(4) For the purposes of assessment (see A.8.3), tests on different specimens may be grouped into families if they consist of specimens which comply with all of the following:

- common nominal yield strength f_{yb}
- common or different nominal thicknesses t_{nom}
- common testing spans s

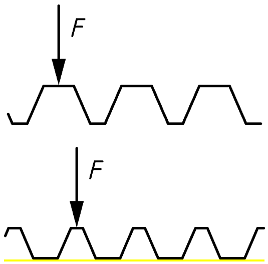
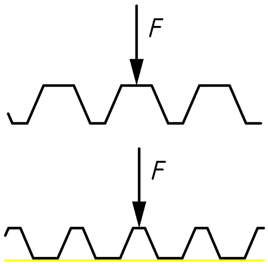
For each nominal thickness, at least two specimens shall be tested.

(5) Specimens consisting of perforated sheeting or sheeting with holes (for example sheeting used as decking for slabs of composite beams) should not be grouped into the same family with specimens consisting of un-perforated sheeting or sheeting without holes.

A.4.6 Walkability

(1) The walkability of sheeting is applicable up to the span for which the assessment criteria in Table A.4 are fulfilled.

Table A.4 - Assessment criteria for walkability

	Loading pattern	Loading F_{min} [kN]	Assessment criterion
Edge loading Outermost completely formed rib in direction of lay		1,2 1,5 2,0	significant permanent deformation failure load sudden failure without significant overall deformation
Central loading		2,0	failure load [a]

[a] After a first sudden drop following the first load peak, membrane effects may lead to a renewed rise in load. Assessment criteria may be applied to the second load peak, provided that the additional criterion $F_{min} \geq 1,5$ kN is satisfied by the first load peak.

(2) The test sheeting should be placed on flat rails not less than 40 mm wide. For edge loading, representative of walkability during installation, Clause A.4.1(3) does not apply.

(3) A concentrated quasi-static loading should be applied in the direction of gravity over a 100 mm x 150 mm area, with the longer side of the area parallel to the direction of the span. In order to prevent any stress concentrations, loading should be applied through a soft layer of about 10 mm thickness, e.g. using a felt pad.

(4) The tests should begin with the largest span envisaged for use in practice. If the assessment criteria given in Table A.4 are not fulfilled by all of the required number of tests, as listed in Table A.5, the span is reduced until the required number of tests satisfy the assessment criteria.

Table A.5 - Minimum number of tests

Number of nominal sheet thicknesses to be tested		Number of tests
for $t_{nom} \geq 0,60$ mm	≥ 3	≥ 2
	2	≥ 3
	1	≥ 4
for $t_{nom} < 0,60$ mm		≥ 4

(5) The maximum span of sheeting L_{lim}, which may be walked on by a single person is the smallest of the spans $L_{lim,test}$ resulting from loading at the edges or central loading.

(6) The test results should be adjusted according to Clause A.8.2, but a statistical evaluation according to clause A.8.3 need not to be done. For $R_{adj,i} < F_{min}$, the test results may be adjusted as follows:

$$L_{min} = L_{min,test} \cdot \min\left\{\frac{R_{obs,min}}{\mu_R\, F_{min}}; 1\right\} \tag{A.6}$$

where:

$L_{min,test}$ is the span used in the tests;

$R_{obs,min}$ is the minimum value obtained from testing over both test series

μ_R is the adjustment coefficient according to Clause A.8.2;

A.5 Tests on torsionally restrained beams

A.5.1 General

(1) Clause A.5 applies to cold-formed beams, such as purlins, side rails, floor beams and other similar types of beams, where one flange is laterally and/or partially torsionally restrained by steel sheeting.

(2) Clause A.5 does not apply to beams restrained by sandwich panels, for which testing procedures and test set-ups are specified in EN 14509-2.

(3) The set-up shown in Figure A.11 should be used for sandwich panels under gravity loading.

NOTE: Additional information on testing procedure for sandwich panels is given in: European Recommendations on the Stabilization of Steel Structures by Sandwich Panels. ECCS Pub. No. 135/CIB Pub. No. 320 (2014) [1].

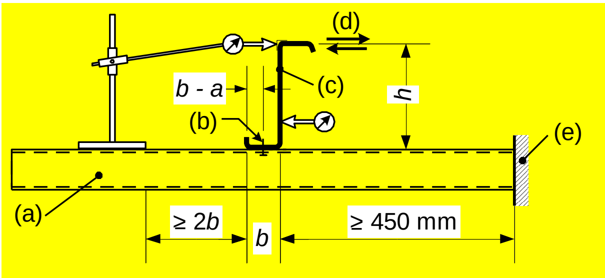
A.5.2 Internal support test

(1) Clause A.2.4.1 applies.

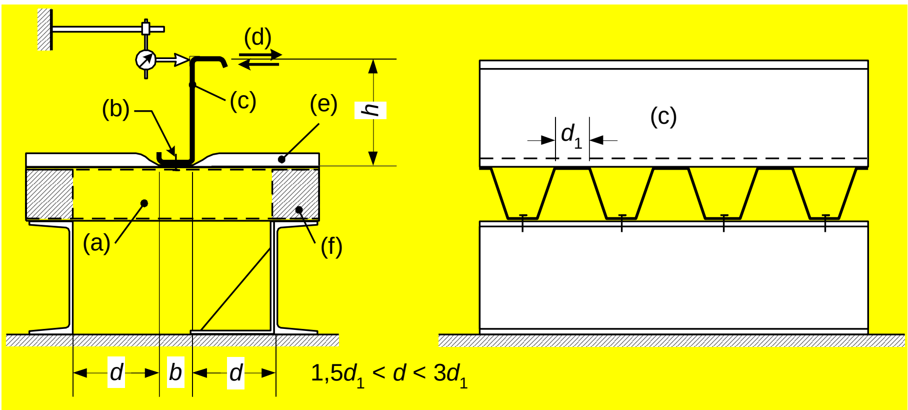
(2) The interpretation of the test results should be carried out according to Clause A.2.4.2. If cleats are used at the supports, the interaction of transverse force and bending moment may be neglected.

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 provided by adequately fastened sheeting or by a discrete member running perpendicularly to the span of the beam.



Key
(a) sheeting, (b) fastener, (c) profile, (d) load, (e) clamped support
a) Alternative 1



Key
(a) sheeting, (b) fastener, (c) profile, (d) load, (e) insulation if available, (f) timber blocks
b) Alternative 2

Figure A.11 - Experimental determination of spring stiffness values K_A and K_B

(2) The test set-up in Figure A.11 measures two combined contributions to the total amount of restraint:

a) The lateral stiffness K_A per unit length resulting from the rotational stiffness of the connection between the sheeting and the beam;

b) The lateral stiffness K_B per unit length resulting from distortion of the cross-section of the purlin.

(3) The evaluation of the test results should be based on:

$$\frac{1}{K_A\, l_A} + \frac{1}{K_B\, l_B} = \frac{\delta}{F} \tag{A.7}$$

where:

l_A is the modular length of the tested sheeting;

l_B is the length of the tested beam;

F is the load which produces a lateral deflection of $h/10$;

δ is the lateral displacement of the top flange in the direction of the load F.

(4) The values of $C_{D,A}$ for gravity loading and for uplift loading should be determined from:

$$C_{D,A} = \frac{\dfrac{h^2}{l_A}}{\dfrac{\delta}{F} - \dfrac{4(1-\upsilon^2)\, h^2\, (h_d + b_{mod})}{E\, t^3\, l_B}} \tag{A.8}$$

with:

b_{mod}, h and h_d are defined in Clause 11.1.5.(4);

l_A and l_B are defined in Clause A.5.3(3).

(5) When interpreting the test results according to Clause A.8, 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 beam;
- the possible presence of insulation between the beam and the sheeting.

A.6 Tests on fastenings

(1) Testing of fasteners is described in EAD 330046-01-0602 Fastening Screws for Metal Members and Sheeting [7].

NOTE: Additional information on testing procedures for fastenings is given in: The Design and Testing of Connections in Steel Sheeting and Sections: ECCS Pub. No. 124 (2009) [3].

(2) An additional-margin of safety should be provided for a brittle failure mode compared to a ductile failure mode.

A.7 Tests on components of storage equipment

(1) Testing of components of adjustable pallet racking is described in EN 15512.

NOTE: For other types of storage equipment no standard is available, however there are a number of industry codes of practice:

- FEM 10.2.06: Shelving [4]
- FEM 10.2.07: Drive-in and drive-through racking [5]
- FEM 10.2.09: Cantilever racking [6].

A.8 Evaluation of test results

A.8.1 General

(1) A test specimen under test should be regarded as having failed if the applied loads have reached their maximum values, a crack has initiated to spread in a vital part of the specimen or the overall deformations have exceeded specified limits.

(2) The overall deformations of members should generally satisfy:

$$\delta \leq L/50 \tag{A.9}$$

$$\phi \leq 1/50 \tag{A.10}$$

where:

δ is the maximum deflection of a beam of span L;

ϕ is the sway angle of a structure.

(3) When testing connections or 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 needs to be placed on the overall deformation during the test.

A.8.2 Adjustment of test results

(1) Test results should be appropriately adjusted to account for differences between the actual measured properties of the test specimens and their nominal values.

(2) The measured basic yield strength $f_{yb,obs}$ should not deviate by more than -25% from the nominal basic yield strength f_{yb}: – $f_{yb,obs} \geq 0{,}75\, f_{yb}$.

(3) The measured thickness t_{obs} should not exceed the nominal material thickness t_{nom} (see Clause 5.2.4) by more than 12%.

(4) Adjustments should be made with respect to the actual measured values of the core material thickness $t_{obs,cor}$ and the basic yield strength $f_{yb,obs}$ for all tests, except if the values measured in tests are used to calibrate a design model. . In the latter case the provisions of Clause A.8.2(5) do not need to be applied.

(5) The adjusted value of resistance $R_{adj,i}$ of the i^{th} test result (i = 1...n) should be determined from the measured test result $R_{obs,i}$ as follows:

$$R_{adj,i} = R_{obs,i} / \mu_R \tag{A.11}$$

where: μ_R is resistance adjustment coefficient given by:

$$\mu_R = \left(\frac{f_{yb,obs}}{f_{yb}}\right)^{\alpha} \left(\frac{t_{obs,cor}}{t_{cor}}\right)^{\beta} \tag{A.12}$$

(6) The exponent α for use in Formula (A.12) should be obtained as follows:

- if $f_{yb,obs} \leq f_{yb}$: $\alpha = 0$
- if $f_{yb,obs} > f_{yb}$: $\alpha = 1$

For profiled sheeting or liner trays in which compression elements have such large b_p/t ratios that local buckling will clearly be the failure mode: $\alpha = 0{,}5$.

(7) The exponent β for use in Formula (A.12) should be obtained as follows:

- if $t_{obs,cor} \leq t_{cor}$: $\beta = 1$
- if $t_{obs,cor} > t_{cor}$:
  - for tests on profiled sheeting or liner trays: $\beta = 2$
  - for tests on members, structures or subassemblies:
    - 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.

  with the limiting width-to thickness ratio $(b_p / t)_{lim}$ given by:

$$\left(\frac{b_p}{t}\right)_{lim} = 0{,}64\sqrt{\frac{E k_\sigma}{f_{yb}}}\sqrt{\frac{f_{yb}/\gamma_{M1}}{\sigma_{com,Ed}}} \cong 19{,}1\,\varepsilon\sqrt{k_\sigma}\sqrt{\frac{f_{yb}/\gamma_{M1}}{\sigma_{com,Ed}}} \tag{A.13}$$

where:

b_p is the notional flat width of a plane element;

k_σ is the relevant buckling factor from Table 6.1 or 6.2 in EN 1993-1-5: 2020;

$\sigma_{com,Ed}$ is the largest calculated compressive stress in the element, at the ultimate limit state.

(8) For adjustment of the second moment of area, where linear behaviour is observed in the serviceability limit state, the exponents in the equation (A.12) should be taken as follows: $\alpha = 0{,}0$ and $\beta = 1{,}0$.

A.8.3 Characteristic values

A.8.3.1 General

(1) Characteristic values of tested properties may be determined statistically, provided that there are at least 3 test results.

(2) Additional rules apply to families of tests, as specified in Clause A.8.3.2, or to cases where only a limited number of test results are available, as specified in Clause A.8.3.3.

(3) The statistical evaluation leading to the determination of the characteristic values should be carried out according EN 1990: 2018, Annex D.

(4) The values of the coefficient k_n should be determined from the Table D1, EN 1990: 2018, according to the number of test results and for an unknown V_x value.

(5) The conversion factor η_d for differences in behaviour under test conditions and service conditions should be determined in dependence of the modelling for testing.

(6) For testing of

- sheeting according to Clause A.4;
- torsionally restrained beams according to Clause A.5;
- fastenings according to Clause A.6;
- components of storage equipment according to Clause A.7;

and for other well defined standard testing procedures η_{d} may be taken as equal to 1.0.

A.8.3.2 Characteristic values for families of tests

(1) Generally, several series of tests carried out on a number of otherwise similar structures, sub-assemblies, members, sheeting or other structural components, in which one or more parameters are 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 the cross-sectional dimensions, the span length, the thickness and the material strengths.

For the single span tests, internal support tests and end support tests the families of tests are defined in Clauses A.4.2, A.4.4 and A.4.5, respectively.

(2) In order to calculate the characteristic value of the resistance $R_{\mathrm{k,j}}$ from a test series j, each test result should first be normalized by dividing it by the mean value of the resistance of the series j, $R_{\mathrm{m,j}}$:

$$x_{i,j} = \frac{R_{adj,i,j}}{R_{m,j}} \qquad i = 1 \dots n_j \tag{A.14}$$

where:

n_j is the number of individual tests in a series j;

$R_{\mathrm{adj,i,j}}$ is the adjusted value of the resistance of specimen i of test series j;

$R_{\mathrm{m,j}}$ is the mean value of the adjusted resistance $R_{adj,i,j}$ of series j;

(3) The value of k_{n} , obtained from Table D1, EN 1990: 2018, should be based on the total number of tests n in the family:

$$n = \sum_j n_j \tag{A.15}$$

where:

n_j is the number of individual tests in the series j.

(4) The standard deviation s_x should be calculated as follows:

$$s_x = \sqrt{\frac{1}{n-1}\sum_j \sum_i \left(x_{i,j} - 1\right)^2} \tag{A.16}$$

(5) The characteristic value of the resistance R_k should be calculated as follows:

$$R_{k,j} = R_{m,j} \cdot (1 \pm k_n s_x) \tag{A.17}$$

where:

s_x is the standard deviation, as specified in (4);

k_n is the appropriate coefficient from Table D.1, EN 1990: 2018;

$R_{m,j}$ is the mean value of the adjusted resistances $R_{adj,i,j}$;

The least favourable sign "+" or "-" should be adopted.

(6) Subclause A.8.3.2 is analogously applicable to determine characteristic values in the serviceability limit state with respect to deformations and rotations and the stiffnesses of members, sheeting and structures.

For characteristic values of rotations, both signs should be considered in Formula (A.17).

A.8.3.3 Characteristic values based on a small number of tests

(1) If only one test is carried out, then the characteristic value of resistance R_k corresponding to this test should be obtained from the adjusted value of the test result R_{adj} using the following Formula:

$$R_k = 0{,}9\, \eta_k R_{adj} \tag{A.18}$$

in which η_k should be taken as follows, depending on the failure mode:

- yielding: $\eta_k = 0{,}9$;
- excessive overall deformation: $\eta_k = 0{,}9$;
- local buckling: $\eta_k = 0{,}8 \,.\, 0{,}9$ depending on effects on global behaviour in tests ;
- overall instability: $\eta_k = 0{,}7$.

(2) For a series of two 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 value of resistance R_k should be obtained using the following Formula:

$$R_k = \eta_k R_m \tag{A.19}$$

(3) If only one test is carried out, the characteristic value of the stiffness should be reduced by 0,95 in case a higher stiffness results in a-favourable effect and increased by 1,05 in case a higher stiffness results in an unfavourable effect.

(4) 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.8.4 Design values

(1) The design value of resistance R_d should be derived from the corresponding characteristic value of resistance R_k determined by testing, using the following Formula:

$$R_d = \eta_{sys} \frac{R_k}{\gamma_M} \tag{A.20}$$

where:

γ_M is the partial factor for the resistance see Clause 4;

η_{sys} is a conversion factor accounting for the differences in behaviour between-test conditions and service conditions.

(2) The appropriate value of η_{sys} should be determined in dependence of the modelling for testing (with the aid of analytical or numerical models).

(3) For testing of

- single beams according to Clause A.2;
- sheeting according to Clause A.4;
- torsionally restrained beams according to Clause A.5;
- fastenings according to Clause A.6;
- components of storage equipment according to Clause A.7;

and for other well defined standard testing procedures, η_{sys} may be taken equal to 1.0.

NOTE: The partial factor γ_M can be set by the National Annex for use in a country. It is recommended to use the γ_M-values given in Clause 4 unless different values result from the use of Annex D of EN 1990.

Annex B [informative] – Durability of fasteners

B.1 Use of this Informative Annex

(1) This Informative Annex provides complementary guidance to that given in Clause 6 for durability, accounting for different materials for sheet and fasteners acting compositely. Table B.1 defines types of material configurations recommended in respect of corrosion.

NOTE: The way in which this Informative Annex can be used in a Country is given in the National Annex. If the National Annex is silent on the use of this informative annex, it can be used.

B.2 Scope and field of application

(1) This Informative Annex covers sheet material and material of fasteners as given in Table B.1.

(2) The environmental classification following EN ISO 12944-2 is presented in Table B.2.

Table B.1 - Fastener material with regard to corrosion environment (and sheeting material only for information). Only the risk of corrosion is considered. Classification of environment according to EN ISO 12944-2.

Classification of environment	Sheet material	Material of fastener					
		Aluminium	Electro galvanized steel. Coat thickness > 7µm	Hot-dip zinc coated steel[b]. Coat thickness > 45 µm	Stainless steel, case hardened. 1.4006 [d]	Stainless steel, 1.4301 [d] 1.4436 [d]	Monel [a]
C1	A, B, C	X	X	X	X	X	X
	D, E, S	X	X	X	X	X	X
C2	A	X	-	X	X	X	X
	C, D, E	X	-	X	X	X	X
	S	X	-	X	X	X	X
C3	A	X	-	X	-	X	X
	C, E	X	-	X	(X)[c]	(X)[c]	-
	D	X	-	X	-	(X)[c]	X
	S	-	-	X	X	X	X
C4	A	X	-	(X)[c]	-	(X)[c]	-
	D	-	-	X	-	(X)[c]	-
	E	X	-	X	-	(X)[c]	-
	S	-	-	X	-	X	X
C5-I	A	X	-	-	-	(X)[c]	-
	D[f]	-	-	X	-	(X)[c]	-
	S	-	-	-	-	X	-
C5-M	A	X	-	-	-	(X)[c]	-
	D[f]	-	-	X	-	(X)[c]	-
	S	-	-	-	-	X	-

NOTE: Fastener of steel without coating can be used in corrosion classification class C1.

Key

A = Aluminium irrespective of surface finish

B = Un-coated steel sheet

C = Hot-dip zinc coated (Z275) or aluzink coated (AZ150) steel sheet

D = Hot-dip zinc coated steel sheet + coating of paint or plastics

E = Aluzink coated (AZ185) steel sheet

S = Stainless steel

X = Type of material recommended from the corrosion standpoint

(X) = Type of material recommended from the corrosion standpoint under the specified condition only

\- = Type of material not recommended from the corrosion standpoint

a Refers to rivets only

b Refers to screws and nuts only

c Insulating washer, of material resistant to ageing, between sheeting and fastener

d Stainless steel EN 10088

e Risk of discoloration.

f Always check with sheet supplier

Table B.2 - Atmospheric-corrosivity categories according to EN ISO 12944-2 and examples of typical environments

Corrosivity category	Corrosivity level	Examples of typical environments in a temperate climate (informative)	
		Exterior	**Interior**
C1	Very low	-	Heated buildings with clean atmospheres, e. g. offices, shops, schools and hotels.
C2	Low	Atmospheres with low level of pollution. Mostly rural areas	Unheated buildings where conden-sation may occur, e. g. depots, sport halls.
C3	Medium	Urban and industrial atmospheres, moderate sulphur dioxide pollution. Coastal areas with low salinity.	Production rooms with high humidity and some air pollution, e. g. food-processing plants, laundries, breweries and dairies.
C4	High	Industrial areas and coastal areas with moderate salinity.	Chemical plants, swimming pools, coastal ship- and boatyards.
C5-I	Very high (industrial)	Industrial areas with high humidity and aggressive atmosphere.	Building or areas with almost per-manent condensation and with high pollution.
C5-M	Very high (marine)	Coastal and offshore areas with high salinity.	Building or areas with almost per-manent condensation and with high pollution.

Annex C [normative] – Mixed effective width/effective thickness method for outstand elements

(1) This Annex may be used for outstand elements in compression and/or bending. The outstand element is divided into a fully effective part of the section and a reduced part of the section due to local plate buckling. The area of the effective cross-section of the outstand element is composed of an effective width b_{e0} times the full element thickness t and an element width $(b_p - b_{e0})$ times the effective thickness t_{eff} reduced due to local plate buckling, see Table C.1.

(2) The reduction factor is given by:

$$\rho = \frac{1}{\bar{\lambda}_p} - \frac{0,188}{\bar{\lambda}_p^2} \leq 1 \qquad \text{but} \geq \frac{0,77}{\sqrt{\bar{\lambda}_p}} \qquad \text{if } \bar{\lambda}_p \geq 0,749 \tag{C.1}$$

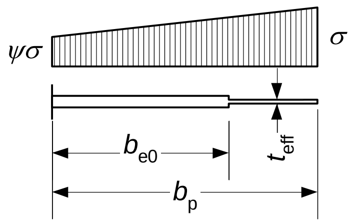
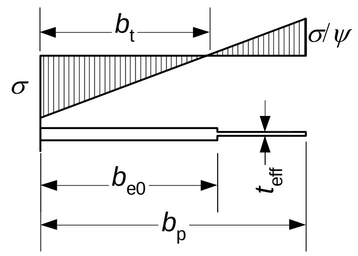
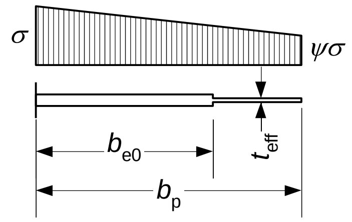
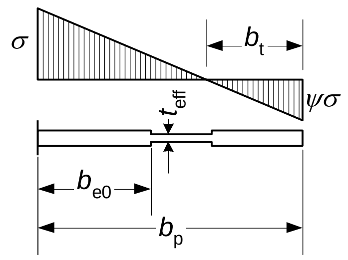
(3) The slenderness parameter $\bar{\lambda}_p$ is given in 7.5.2.

(4) The buckling factor k_σ for different stress distributions may be determined with numerical methods. Conservatively, the values given in EN 1993-1-5: 2020, Table 6.2, may be used.

(5) The stress ratio ψ used in (4) and Table C.1 should be based on the properties of the gross cross-section.

(6) The resistance of the sections should be based on the elastic stress distribution of the effective cross-section.

Table C.1 - Outstand compression elements

<table>
<tr><th colspan="2">Maximum compression at free longitudinal edge</th></tr>
<tr><td>Stress distribution</td><td>Effective width and thickness</td></tr>
<tr><td>
</td><td>$1 \geq \psi \geq 0$
$b_{e0} = 0{,}42\ b_p$
$t_{eff} = (1{,}75\ \rho - 0{,}75)\ t$</td></tr>
<tr><td>
</td><td>$\psi < 0$
$b_{e0} = \dfrac{0{,}42\ b_p}{(1-\psi)} + b_t < b_p$
$b_t = \dfrac{\psi\ b_p}{(\psi - 1)}$
$t_{eff} = (1{,}75\ \rho - 0{,}75 - 0{,}15\ \psi)\ t$</td></tr>
<tr><th colspan="2">Maximum compression at supported longitudinal edge</th></tr>
<tr><td>Stress distribution</td><td>Effective width and thickness</td></tr>
<tr><td>
</td><td>$1 \geq \psi \geq 0$
$b_{e0} = 0{,}42\ b_p$
$t_{eff} = (1{,}75\ \rho - 0{,}75)\ t$</td></tr>
<tr><td>
</td><td>$\psi < 0$
$b_{e0} = \dfrac{0{,}42\ b_p}{(1-\psi)}$
$b_t = \dfrac{\psi\ b_p}{(\psi - 1)}$
$t_{eff} = (1{,}75\ \rho - 0{,}75)\ t$</td></tr>
</table>

Bibliography

[1] ECCS Publication No.088: 1995, *European recommendations for the application of metal sheeting acting as a diaphragm*. ECCS European convention for steel construction. Brussels.
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Available from: http://site.cibworld.nl/dl/publications/pub320.pdf

[3] ECCS Publication No.124; 2009, *The Testing of Connections with Mechanical Fasteners in Steel Sheeting and Sections.* ECCS European convention for steel construction. Brussels.
Available from: http://www.steelconstruct.com

[4] FEM 10.2.06: Shelving. Available from: www.erfed.org

[5] FEM 10.2.07: Drive-in and drive-through racking. Available from: www.erfed.org

[6] FEM 10.2.09: Cantilever racking. Available from: www.erfed.org

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