Cold-Formed (CF) Structures
Eurocode 9 - Part 1.4

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Cold-Formed Structures (Part 1.4)

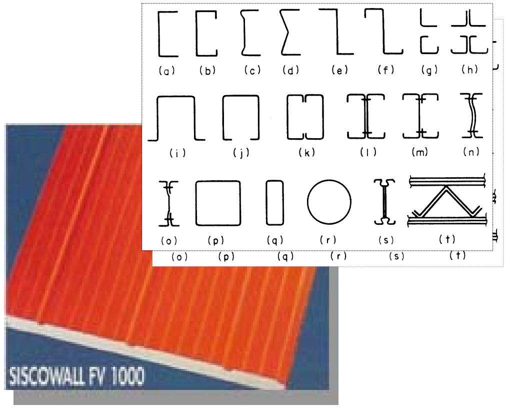
Part 1. Introduction

1.1 TYPES AND SHAPES

1.2 COLD–FORMING TECHNIQUES

1.3 BEHAVIOURAL FEATURES

1.4 TYPES OF LIGHT-WEIGHT STRUCTURES



Part 2. Design of aluminium CF structures according to EC9

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PART 1.1 (EN 1999-1-1)

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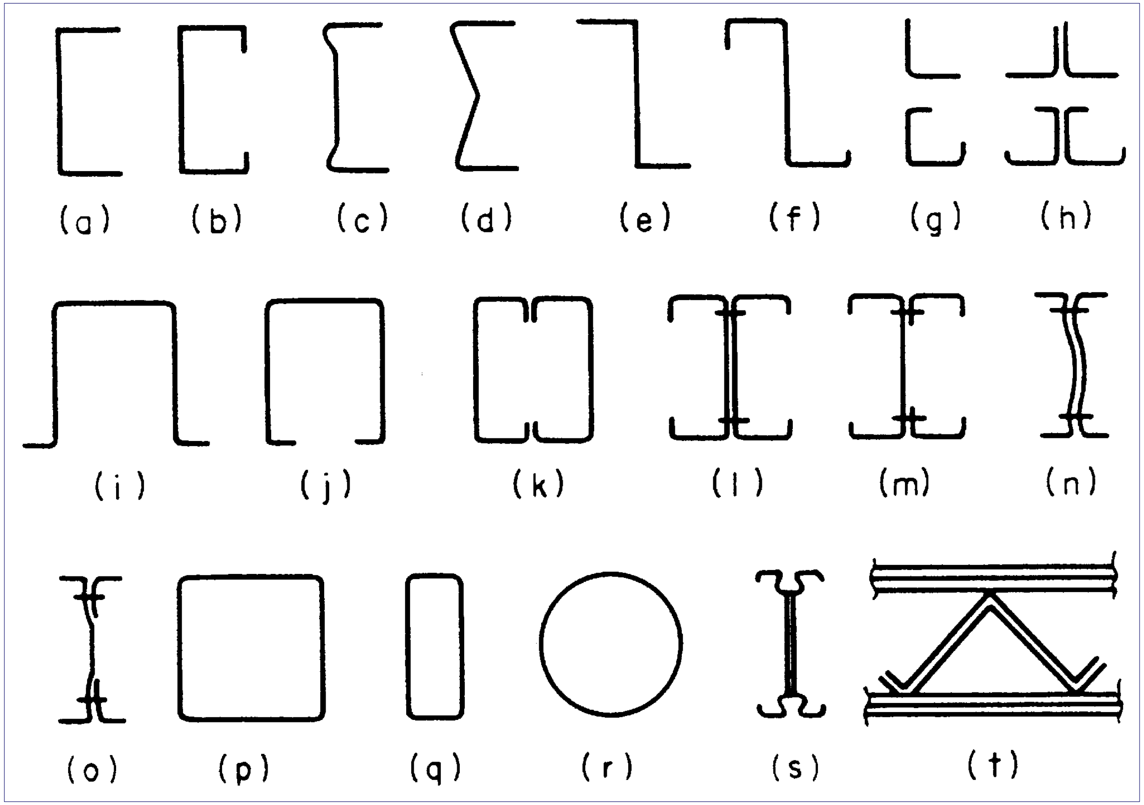
Part 1. Introduction

1.1 TYPES AND SHAPES

Cold-formed (CF) structural products can be classified into three main typologies:

- **members**
- sheeting
- sandwich panels



Structural members are mainly used in the higher range of thickness, as beams for comparatively low loads on small spans (purlins and rails), as columns and vertical supports, and as bars in trusses.

The depth of CF members ranges from 50 to 300 mm and the thickness of material ranges from 1.0 to 8.0 mm, although depth and thickness outside these ranges also are used

1.1 TYPES AND SHAPES

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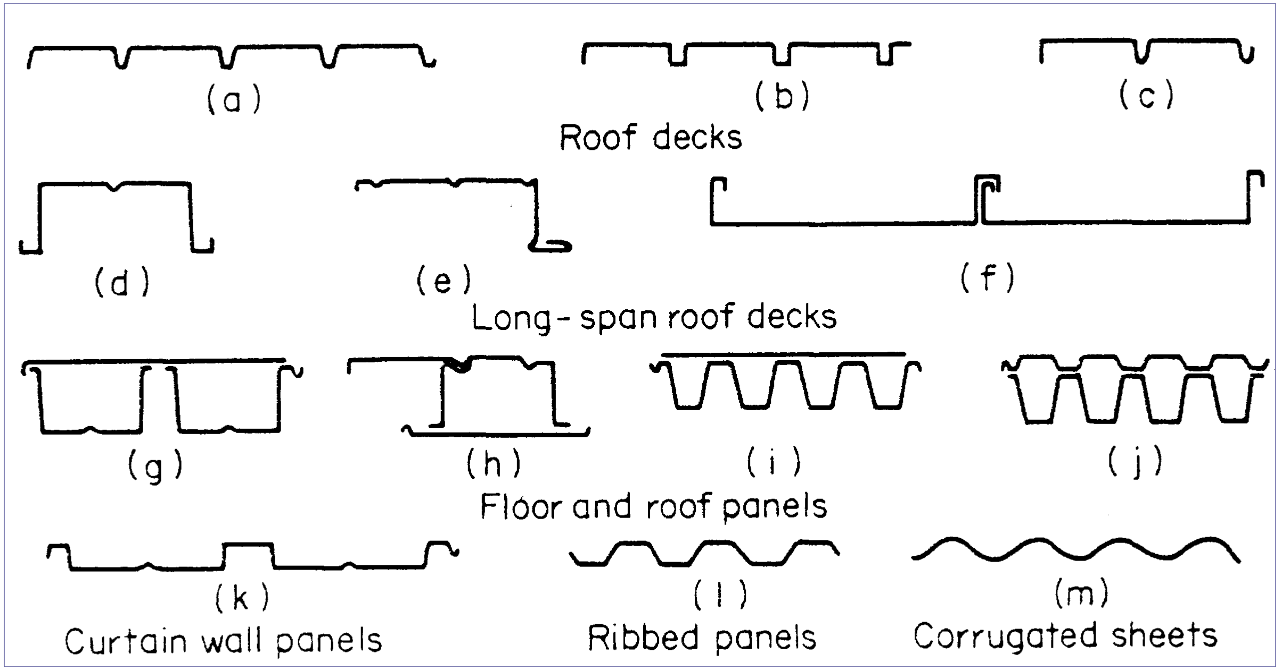


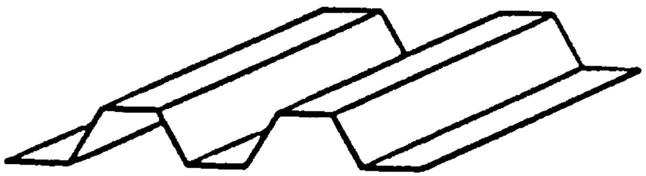
Sheeting are plane load bearing members in the lower range of thickness, generally used when a space covering function under moderate distributed loading is needed, e.g. roof decks, floor decks, wall panels.
The depth of panels generally ranges from 40 to 200 mm and the thickness of material ranges from 0.5 to 2.0 mm.

1.1 TYPES AND SHAPES

Three generations of sheeting

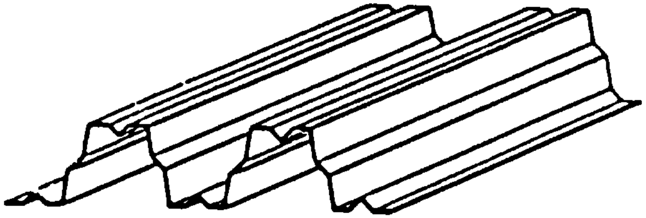
1st generation

The first generation includes plane trapezoidal profiles without stiffeners, allowing spans between secondary members of no more than 3 m.



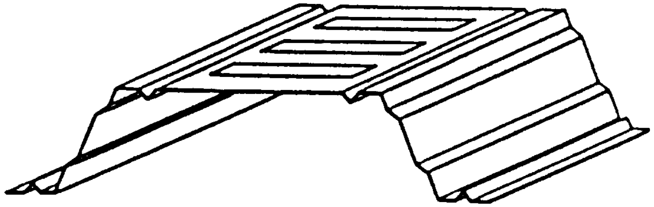
2nd generation

In the second generation the trapezoidal sheets are stiffened in longitudinal direction by appropriate folding and may span up to 6 - 7 m.



3rd generation

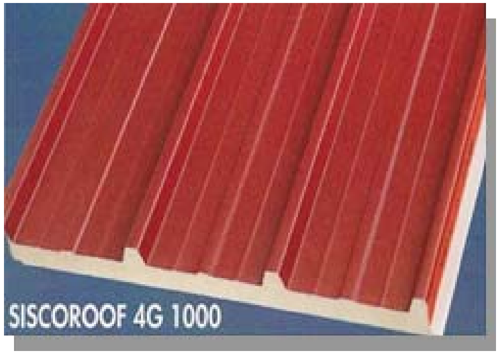
The third generation profiles are trapezoidal units with both longitudinal and transversal stiffeners, which provide suitable solution for spans up to 12 m without purlins.



1.1 TYPES AND SHAPES

Cold-formed (CF) structural products can be classified into two main typologies:

- members
- sheeting
- **sandwich panels**



The prefabricated sandwich panels are particularly suitable because they provide thermal insulation at the same time as the basic weather shield

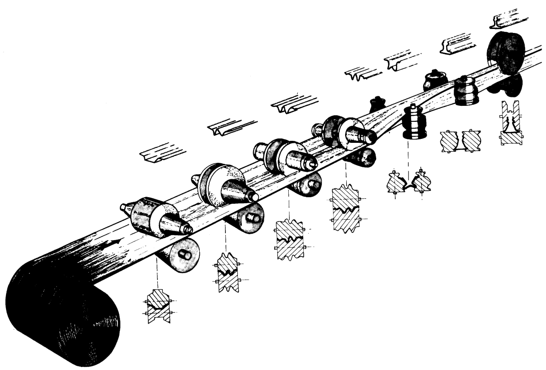
It consists of two metal faces bonded to an internal layer of rigid foam

Such panels may be installed very quickly thus saving time on site

1.2 COLD – FORMING TECHNIQUES

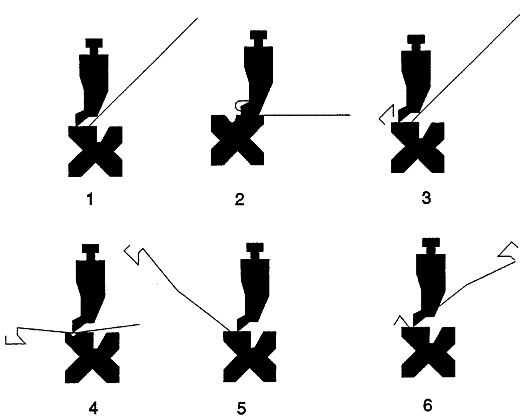
CF sections can be generally obtained through two manufacturing methods:

1. continuous process: cold - rolling

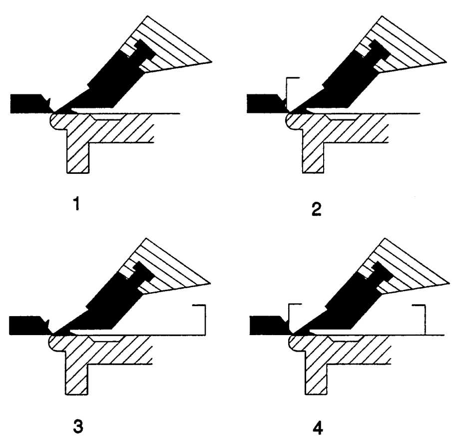


The process of cold-rolling is widely used for the production of individual structural members and corrugated sheeting. The final required shape is obtained from a strip which is formed gradually, by feeding it continuously through successive pairs of rolls which act as male and female dies.

2. discontinuous process: press braking or folding



Press braking



Folding

In these processes, short lengths of strip are fed into the brake and bent or pressed round shaped dies to form the final shape. Usually each bend is formed separately and the complexity of shape is limited to that into which the die can fit.

1.3 BEHAVIOURAL FEATURES

If compared with conventional metallic member, thin-walled CF elements are mainly characterised by:

1. the constant thickness of the formed section
2. the relatively high width-to-thickness ratio of the elements
3. the variety of cross-sectional shapes

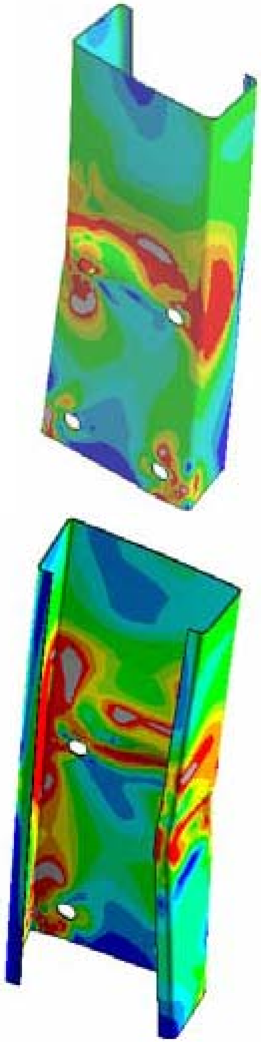
The feature 2. gives rise to local buckling phenomena, which penalise the load-bearing capacity.

As a consequence, structural analysis and design of thin-walled CF elements is generally complicated by the effects arising from the above features, which do not affect the structural response of more simple and compact sections.

1.3 BEHAVIOURAL FEATURES

The main aspects that influence the structural behaviour of thin-walled sections are:

- local buckling of the compression parts
- interaction between local and overall buckling modes
- shear-lag and curling effects
- effects of cold-forming process



Besides, since CF sections are generally thin-walled and of open cross-section, torsional-flexural buckling may be the critical phenomenon influencing the design

1.4 TYPES OF LIGHT-WEIGHT STRUCTURES

Early applications of CF thin-walled aluminium sections were restricted to situations where weight saving was important. With the advance in the raw material itself and the manufacturing processes, the range of actual and potential use is virtually unlimited.

The main structural typologies are:

- **Industrial building**
- **Housing**
- **Temporary structures**

1.4 TYPES OF LIGHT-WEIGHT STRUCTURES

Industrial building

Trusses made of CF members may be found in industrial and storage buildings.
The main chords are usually channel sections joined back to back.
The web members are normally single channels.

1.4 TYPES OF LIGHT-WEIGHT STRUCTURES

Industrial building

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Industrial building

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Industrial building

Aluminium extruded products

1.4 TYPES OF LIGHT-WEIGHT STRUCTURES

Temporary structures

Modular unit for houses, offices, construction site accommodation, etc., may conveniently be produced using CF sections and flat products.v

Motorized roofing for concert stage (Europoint s.n.c.)

1.4 TYPES OF LIGHT-WEIGHT STRUCTURES

Temporary structures

Motor show Bologna, Italy - (Europoint s.n.c.)

1.4 TYPES OF LIGHT-WEIGHT STRUCTURES

Temporary structures

Prefabricated industrial hangar - (CoverTech)

1.4 TYPES OF LIGHT-WEIGHT STRUCTURES

Housing

With regards to housing in CF members, this development is being led by the USA, but interesting applications are coming up also in Europe. The primary framing elements for this construction system are cold-formed metallic wall studs and floor joists.

1.4 TYPES OF LIGHT-WEIGHT STRUCTURES

Housing

Part 2.
Design of aluminium
CF structures according to EC9

2.1 General information

2.1.1 FOREWORD

The European code for the design of aluminium structures, Eurocode 9, provides in **Part 1.1** (*EN 1999-1-1*) general rules for local buckling resistance. In addition, **Part 1.4** (pr*EN 1993-1-4*) provides supplementary rules for CF sheeting.

EUROPEAN STANDARD
PRÉNORME EUROPÉENNE
EUROPÄISCHE VORNORM

EN 1999-1-1

February 2007

UDC

Descriptors:

English version

Eurocode 9:-Design of aluminium structures

Part 1-1 : General structural rules

Calcul des structures en aluminium

Partie 1-1 : Règles générales

Bemessung und Konstruktion von Aluminium-tragverke

Teil 1-1 : Allgemeine Bemessungsregeln

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EUROPÄISCHE VORNORM

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Eurocode 9 : Design of aluminium structures

Part 1-4 : General structural rules

Supplementary rules for cold-formed sheeting

Eurocode 9: Calcul des structures en aluminium - Partie 1-4: Règles générales - Règles supplémentaires pour les plaques formés à froid

Eurocode 9: Bemessung und Konstruktion von Aluminiumbauten - Teil 1-4: Allgemeine Regels - Ergänzende Regeln fur kaltgeformte Bleche

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2.1.2 CONTENT

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3 *Materials*
4 *Durability*
5 *Structural analysis*
6 *Ultimate limit states for members*
7 *Serviceability limit states*
8 *Design of joints*
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ANNEX B [normative]–*Equivalent t-stub in tension*
ANNEX C [informative]–*Materials selection*
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ANNEX E [informative]–*Analytical models for stress strain relationship*
ANNEX F [informative]–*Behaviour of cross-sections beyond the elastic limit*
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6.1.5
Local buckling resistance

Part 1.4 *EN 1999 1-4*

1 *Introduction*
2 *Basis of design*
3 *Materials*
4 *Durability*
5 *Structural analysis*
6 *Ultimate limit states for members*
7 *Serviceability limit states*
8 *Joint with mechanical fasteners*
ANNEX A [normative]–*Testing procedures*
ANNEX B [informative]–*Durability of fasteners*

2.2 General rules for local buckling resistance *Part 1.1* (EN 1999-1-1)

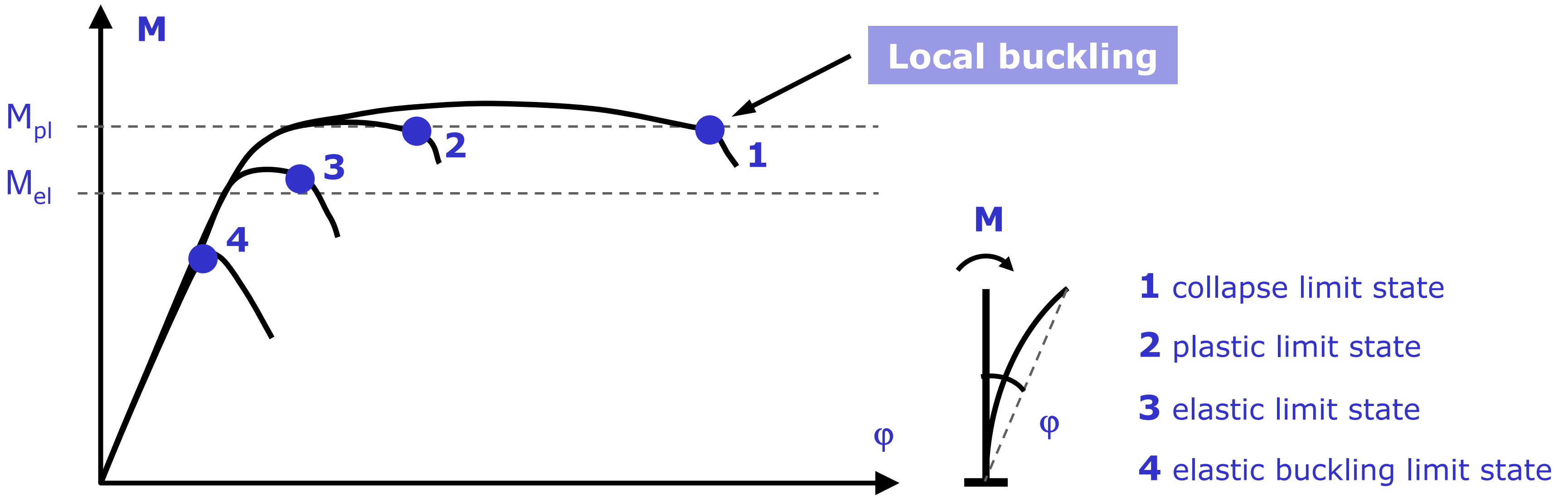
2.2.1 BASIC ASSUMPTION

The behaviour of a cross-section and the corresponding idealisation to be used in structural analysis is related to the capability to reach a given limit state, which corresponds to a particular assumption on the state of stress acting on the section.

Referring to the global behaviour of a cross-section, regardless of the internal action considered (axial load, bending moment or shear), the following limit states can be defined:

1. **Collapse limit state**
2. **Plastic limit state**
3. **Elastic limit state**
4. **Elastic buckling limit state**

2.2.2 CLASSIFICATION OF CROSS- SECTIONS



Bending moment vs rotation

Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance

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

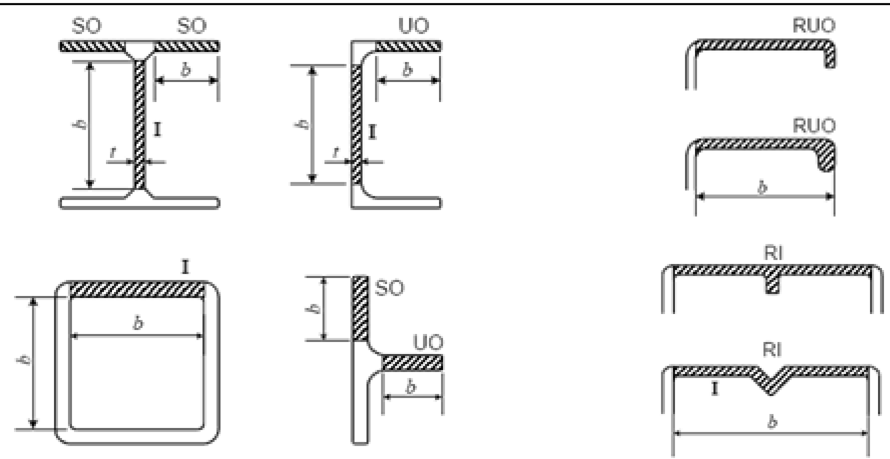
Class 3 cross-sections are those in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance

Class 4 cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section

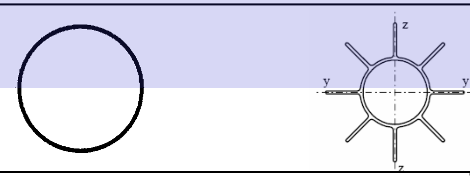
2.2.3 ELEMENT TYPES OF THIN-WALLED ELEMENTS

The following basic types of thin-walled elements are identified in this classification:

	Unreinforced	Reinforced
1. *flat outstand element*	**SO** (Symmetrical Outstand) **UO** (Unsymmetrical Outstand)	**RUO** (Reiforced Unsymmetrical Outstand)
2. *flat internal element*	**I** (Internal cross section part)	**RI** (Reiforced Internal)



3. *curved internal element*



2.2.4 SLENDERNESS OF UNREINFORCED FLAT ELEMENTS

The susceptibility of an *unreinforced flat part* to local buckling is defined by the *parameter* β, which has the following values:

- flat internal parts with no stress gradient or flat outstands with no stress gradient or peak compression at toe → $\beta = b/t$
- internal parts with a stress gradient that results in a neutral axis at the center → $\beta = 0{,}40\ b/t$
- internal parts with stress gradient and outstands with peak compression at root → $\beta = \eta \cdot b/t$

in which:

b is the width of an element;

t is the thickness of a cross-section

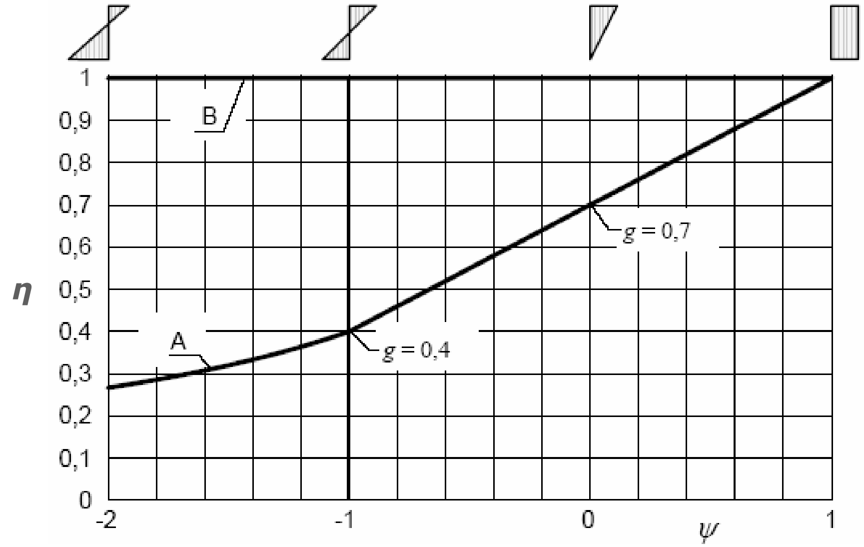
η is the stress gradient factor given by the following expressions

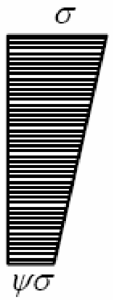
2.2.4 SLENDERNESS OF UNREINFORCED FLAT ELEMENTS

Relationship defining the *stress gradient coefficient* η :

η = 0.70 + 0.30 ψ $(1 \geq \psi \geq -1)$
η = 0.80 / $(1+\psi)$ $(\psi < -1)$

Where ψ is the ratio of the stresses at the edges of the plate under consideration related to the maximum compressive stress.





stress gradient coefficient η

vs

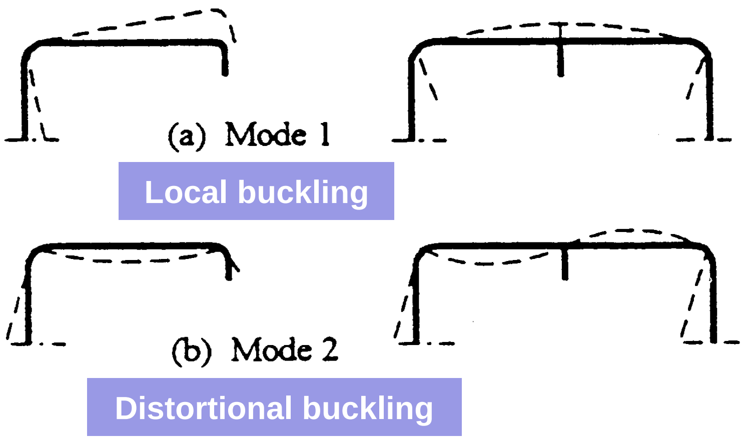
ψ coefficient

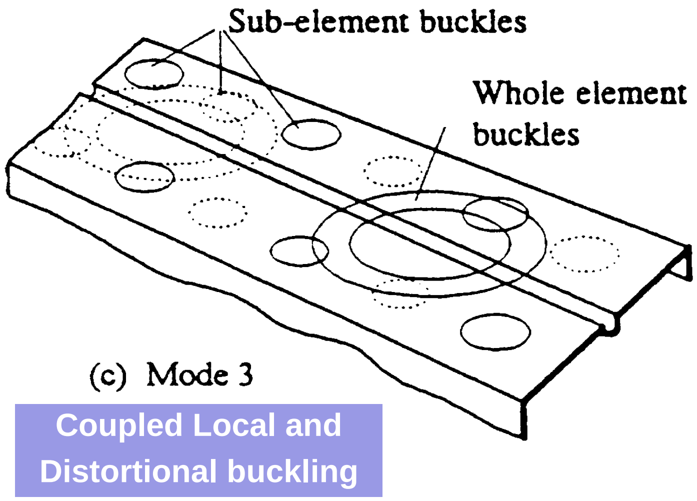
Flat internal parts under stress gradient, values of η
For internal parts or outstands (peak compression at root) use curve A
For outstands (peak compression at toe) use line B

2.2.5 SLENDERNESS OF REINFORCED FLAT ELEMENTS

In the case of **plane stiffened elements**, more complex formulations are provided in order to take into account three possible buckling modes:

- **mode 1**: the stiffened element buckles as a unit, so that the stiffener buckles with the same curvature as the element (**a**)
- **mode 2**: the sub-elements and the stiffener buckle as individual elements with the junction between them remaining straight (**b**)
- **mode 3**: this is a combination of modes 1 and 2, in which both sub-elements and whole element buckle (**c**)





2.2.5 SLENDERNESS OF REINFORCED FLAT ELEMENTS

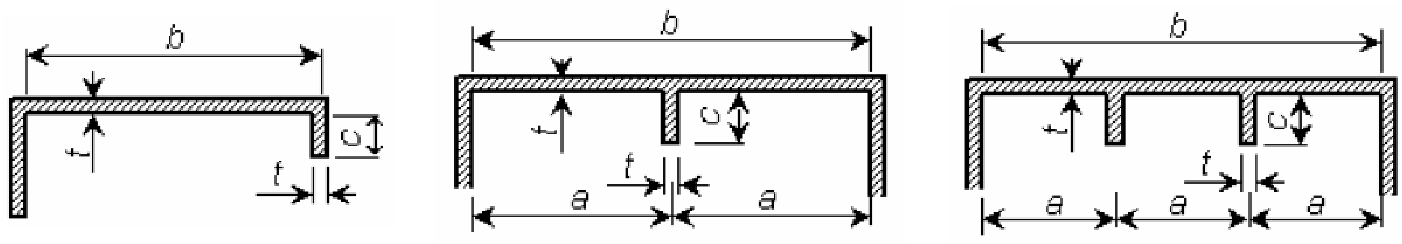
In the case of plane stiffened elements, β is related to:

- type of buckling mode (*mode 1*, *mode 2*)
- stress distribution (uniform compression, stress gradient)
- reinforcement type (standard, non-standard, complex)

Mode1

a) Uniform compression, *standard reinforcement*

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



where:

η depends on *b/t* and *c/t* rations (*c* is the lip depth or rib depth)

b) Uniform compression, *non-standard reinforcement*

The reinforcement is replaced by an equivalent rib or lip equal in thickness to the part. The value of *c* for the equivalent rib or lip is chosen so that the second moment of area of the reinforcement about the mid-plane of the plate is equal to that of the non-standard reinforcement about the same plane.

2.2.5 SLENDERNESS OF REINFORCED FLAT ELEMENTS

c) Uniform compression, *complex reinforcement*

$$\beta = \left(\frac{\sigma_{cr0}}{\sigma_{cr}}\right)^{0.4} \frac{b}{t}$$

where

$\boldsymbol{\sigma_{cr}}$ is the elastic critical stress for the reinforced part assuming simply supported edges

$\boldsymbol{\sigma_{cr0}}$ is the elastic critical stress for the unreinforced part assuming simply supported edges.

d) Stress gradient

In the case of stress gradient σ_{cr} and σ_{cr0} are relate to the stress at the more heavily compressed edge of the part

Mode 2

$\beta = b/t$ is calculated separately for each sub-part

2.2.6 CROSS-SECTION CLASSIFICATION PARTS

Element classification as a function of:

- **β value**
- **Member type**
  - *beam*
  - *strut*

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

Limit parameters β_1, β_2 and β_3 as function of:

- **Element type**
  - *Outstand*
  - *Internal*
- **Alloy type**
  - *Buckling class (Class A, Class B)*
  - *Welded*
  - *Unwelded*

Material classification according to Table 3.2	Internal part			Outstand part		
	β_1/ε	β_2/ε	β_3/ε	β_1/ε	β_2/ε	β_3/ε
Class A, without welds	11	16	22	3	4,5	6
Class A, with welds	9	13	18	2,5	4	5
Class B, without welds	13	16,5	18	3,5	4,5	5
Class B, with welds	10	13,5	15	3	3,5	4

$$\varepsilon = \sqrt{250 / f_0}$$

***f*o**: 0.2% proof strength in MPa

2.2.7 BASIC ASSUMPTIONS

CF thin-gauge metal sections: Class 4 cross-sections

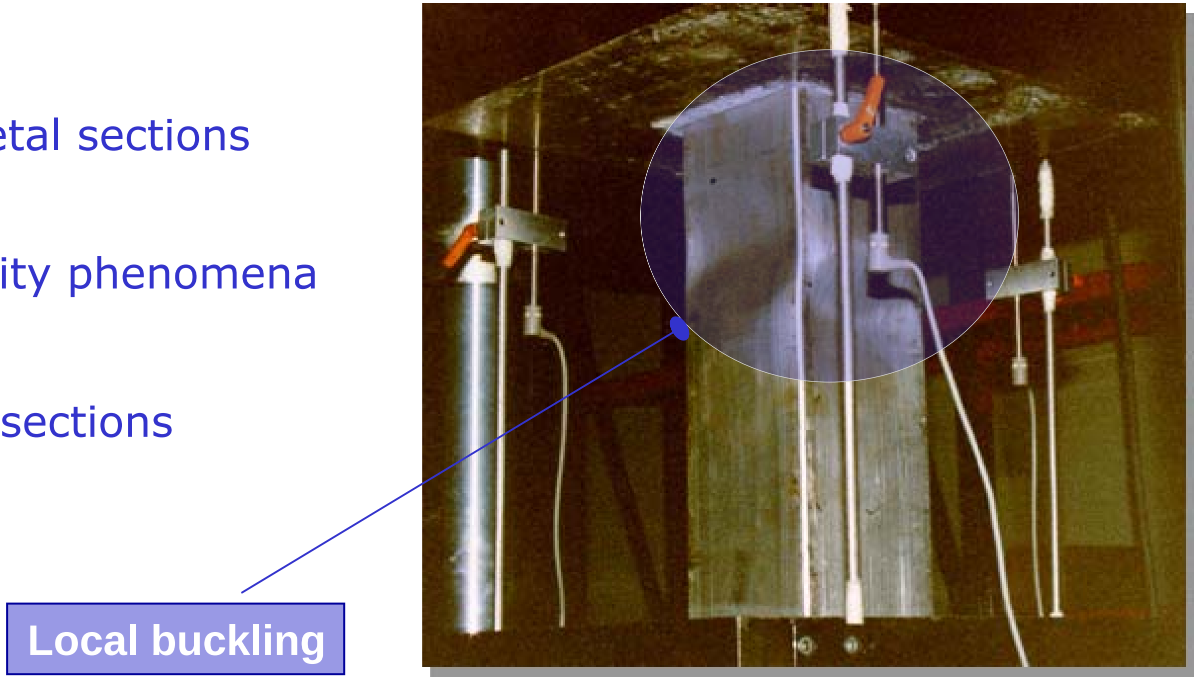
CF thin-gauge metal sections

↓

Local elastic instability phenomena

↓

Class 4 cross-sections

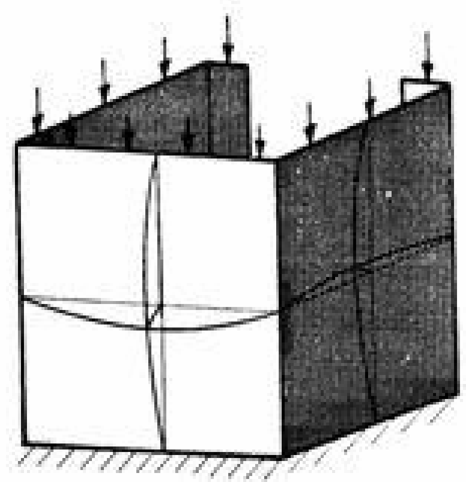


The response of CF thin-gauge metal sections is strongly affected by *local instability phenomena*, which arise in the compressed parts, and the determinant limit state is, of course, the elastic buckling one

2.2.7 BASIC ASSUMPTION

Element model approach

The exact analysis of a thin-walled member requires to treat it as a **continuous folded plate**, but the mathematical complexities of such an analysis are very cumbersome. Most analyses, therefore, consider the member as being made up of an **assembly of individual plates**, with proper boundary and loading conditions, such that the behaviour of the individual plates defines the behaviour of the whole section.



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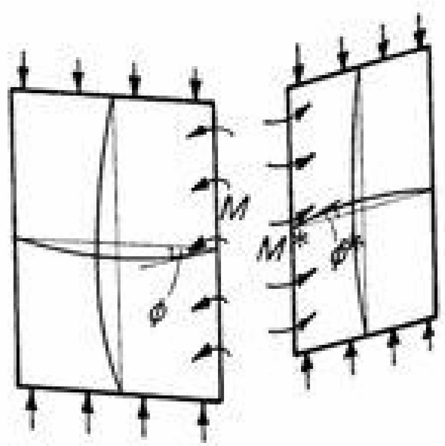


Continuous folded plate **Assembly of individual plates**

2.2.8 INSTABILITY OF PLATES

The analysis of the buckling behaviour of flat plates loaded by forces acting in their middle plane is rather complex, being substantially affected by two kinds of non-linearity: geometrical and mechanical.

The analysis of the stability of plate elements can be performed following two different levels:

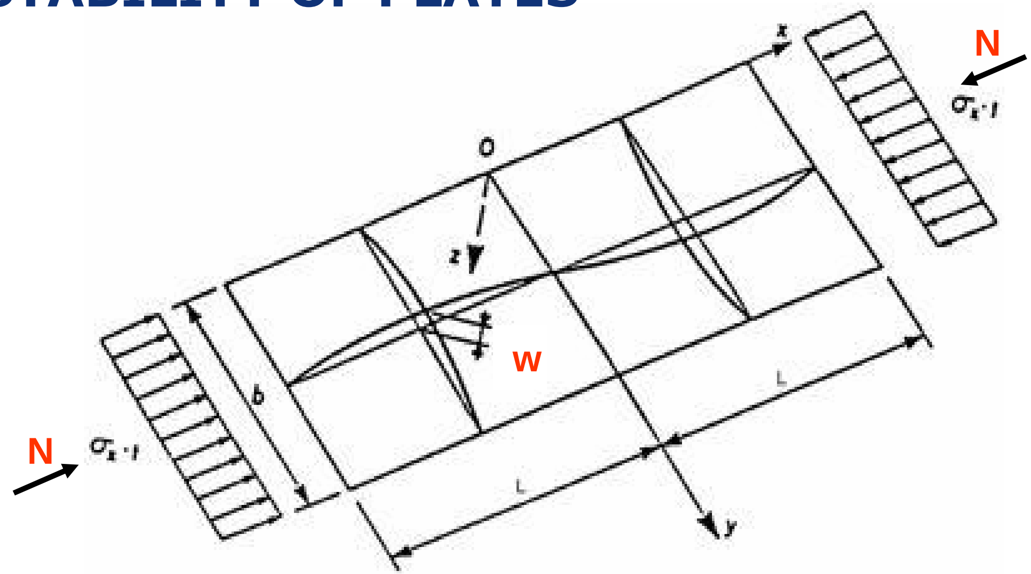
1. **Linear theory**
2. **Nonlinear theory**

Cold-Formed Structures (Part 1.4)

2.2.8 INSTABILITY OF PLATES

Rectangular flat element with:

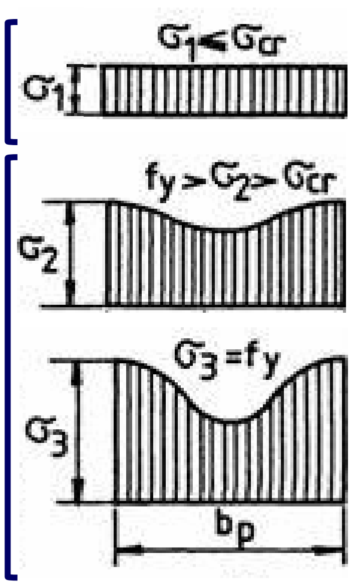
- length **L**
- width **b**
- uniform thickness **t**



The stress distribution:

Linear behaviour

Nonlinear behaviour



1 Before reaching the elastic buckling is uniform in the element

2 After the elastic buckling a non-uniform stress distribution results and a portion of load from the mid strip transfers to the edge parts of the element.

3 The process continues until the maximum stress (along the plate edges) reaches the yield point of the material and then the element begins to fail.

2.2.8 INSTABILITY OF PLATES

1 According to the **linear theory**, the behaviour of a perfectly elastic material in the field of small deformations is examined

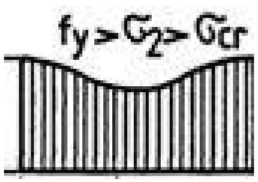
2 According to the **non-linear theory**, the behaviour of plates in post-buckling range is analysed, taking into account both geometrical and mechanical nonlinearities, together with the presence of geometrical and mechanical imperfections

1. Linear theory → linear behaviour

$N \leq N_{cr}$

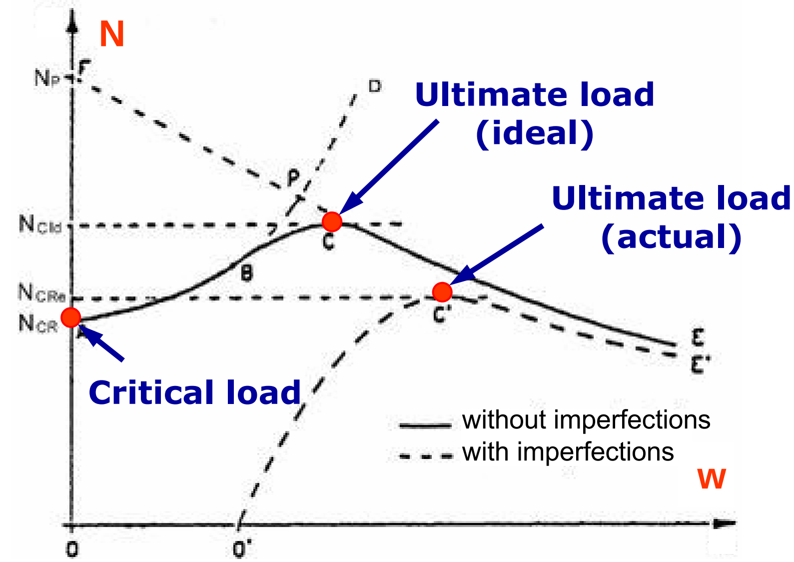
Critical load (N_{cr}) evaluation

2. Nonlinear theory → nonlinear behaviour



$N_u \geq N > N_{cr}$

Ultimate load (N_u) evaluation



Axial in-plane load vs. out-of-plane displacement

The methods based on the linear theory lead to the evaluation of the critical load (Euler load), but they are not valid for a correct estimate of the ultimate load, which can be calculated exclusively by means of a nonlinear analysis

2.2.8 INSTABILITY OF PLATES

1. Elastic buckling behaviour : **Linear theory**

The study of the elastic buckling behaviour of plates according to the **linear theory** leads to the following expression (Euler formula) of the critical stress σ_{cr}:

$$\sigma_{cr} = \frac{k_\sigma \pi^2 E}{12(1-\nu^2)\left(\frac{b}{t}\right)^2} = \frac{\pi^2 E}{\lambda_p^2} \qquad \lambda_p = \sqrt{\frac{12(1-\nu^2)}{k_\sigma}} \cdot \frac{b}{t}$$

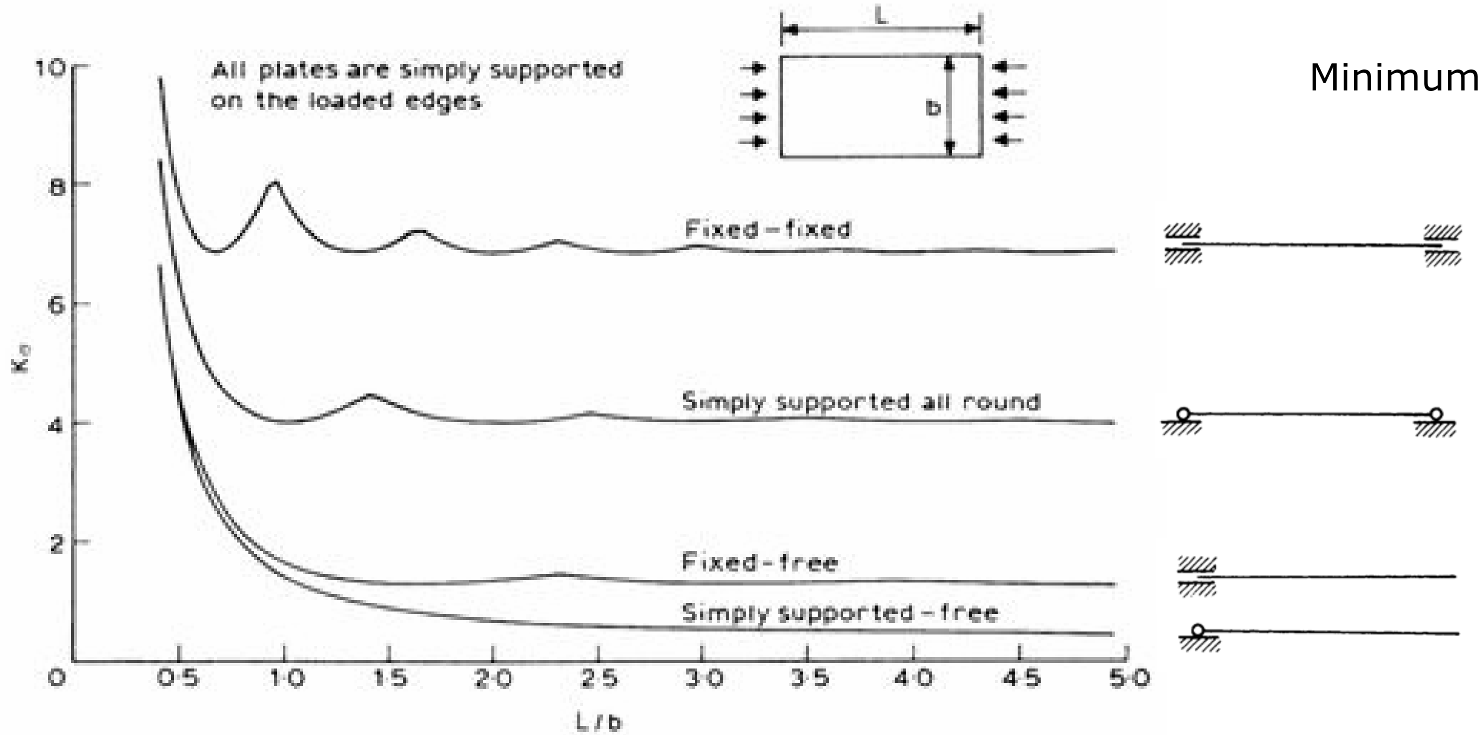
where:

- λ_p is the plate slenderness
- **E** is the Young's modulus
- ν is the Poisson's ratio
- k_σ is the local buckling coefficient which depends on:
- - distribution of axial stress
- - restraint conditions of the unloaded edges
- - geometrical dimensions (L/b)
- **L** is the plate length
- **b** is the plate width
- **t** is the plate thickness

2.2.8 INSTABILITY OF PLATES

1. Elastic buckling behaviour : **Linear theory**

For plates subjected to uniform stress distributions along the length, the variation of k_σ in relation to the length over width ratio L/b depends on the restraint conditions along the unloaded edges.



Minimum k_σ coefficient

k_σ= 6.97

k_σ= 4.00

k_σ= 1.28

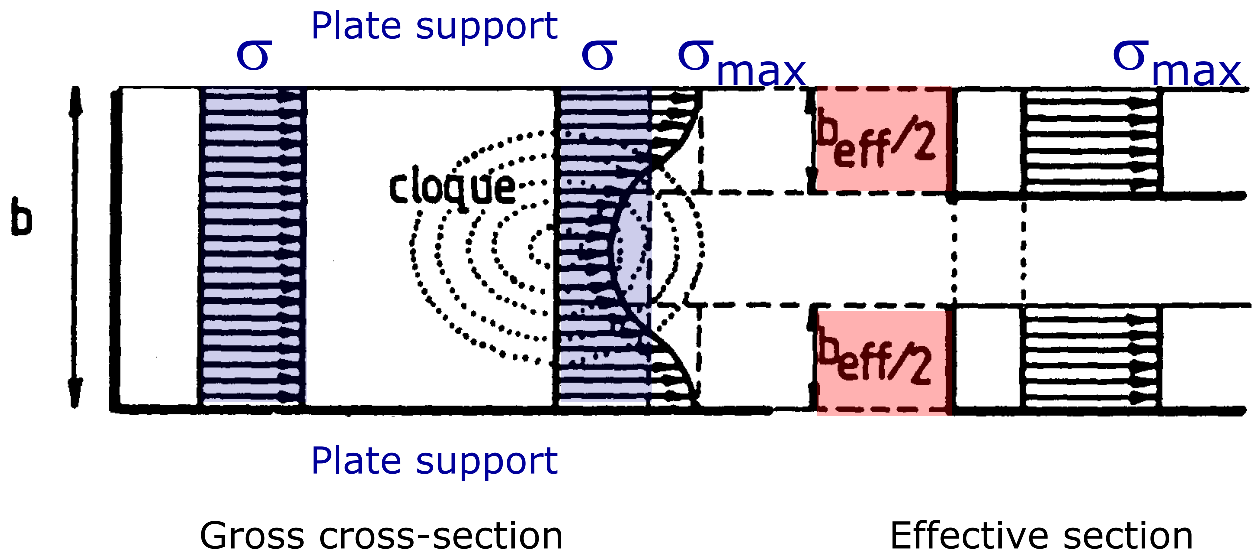
k_σ= 0.43

Design codes generally suggest to use the k_σ coefficients corresponding to simple support or to free conditions

2.2.8 INSTABILITY OF PLATES

2. Post buckling behaviour : **Nonlinear theory**

According to the Von Karman's semi-empirical approach, the non-uniform distribution of stresses, arising during the post-buckling range, can be replaced by an equivalent uniform stress distribution $\sigma=\sigma_{max}$ acting on an "**effective width**" of the plate (b_{eff}), being σ_{max} the actual stress along the unloaded edges



"EFFECTIVE WIDTH" METHOD

2.2.9 INSTABILITY OF STEEL PLATES

2. Post buckling behaviour : **Nonlinear theory**

Following the Von Karman's theory, b_{eff} is the width of a plate for which σ_{max} is equal to the elastic critical stress ($\sigma_{cr,beff}$), so that:

$$\sigma_{cr,beff} = \frac{k_\sigma \pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b_{eff}}\right)^2 = \sigma_{max}$$

As a consequence, the normalised ultimate strength of a slender plate N_u/N_y, without imperfection, may be easily obtained by considering above equation and substituting $\sigma_{max}=f_y$:

$$f_y = \frac{k_\sigma \pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b_{eff}}\right)^2 = \frac{\pi^2 E}{\lambda_1^2} \qquad \text{with} \qquad \lambda_1 = \pi\sqrt{E/f_y}$$

and considering the **Euler formula**:

$$\sigma_{cr} = \frac{k_\sigma \pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2 = \frac{\pi^2 E}{\lambda_p^2} \qquad \text{with} \qquad \lambda_p = \sqrt{\frac{12(1-\nu^2)}{k_\sigma}}\cdot\frac{b}{t}$$

the **Von Karman's equation** can be obtained:

$$\frac{b_{eff}}{b} = \frac{N_u}{N_y} = \sqrt{\frac{\sigma_{cr}}{f_y}} = \frac{\lambda_1}{\lambda_p} = \frac{1}{\overline{\lambda}_p}$$

2.2.9 INSTABILITY OF STEEL PLATES

2. Post buckling behaviour : **Nonlinear theory**

The Von Karman's equation can be easily compared with the Euler formula

$$\frac{b_{eff}}{b}=\frac{N_u}{N_y}=\sqrt{\frac{\sigma_{cr}}{f_y}}=\frac{\lambda_1}{\lambda_p}=\frac{1}{\bar{\lambda}_p}$$

Von Karman's equation

$$\frac{b_{eff}}{b}=\frac{N_{cr}}{N_y}=\frac{\sigma_{cr}}{f_y}=\left(\frac{\lambda_1}{\lambda_p}\right)^2=\frac{1}{\bar{\lambda}_p^2}$$

Euler formula

Winter modified the equation obtained by Von Karman for taking into account **geometrical** and **mechanical imperfections** on the base of a large series of tests on CF steel beams:

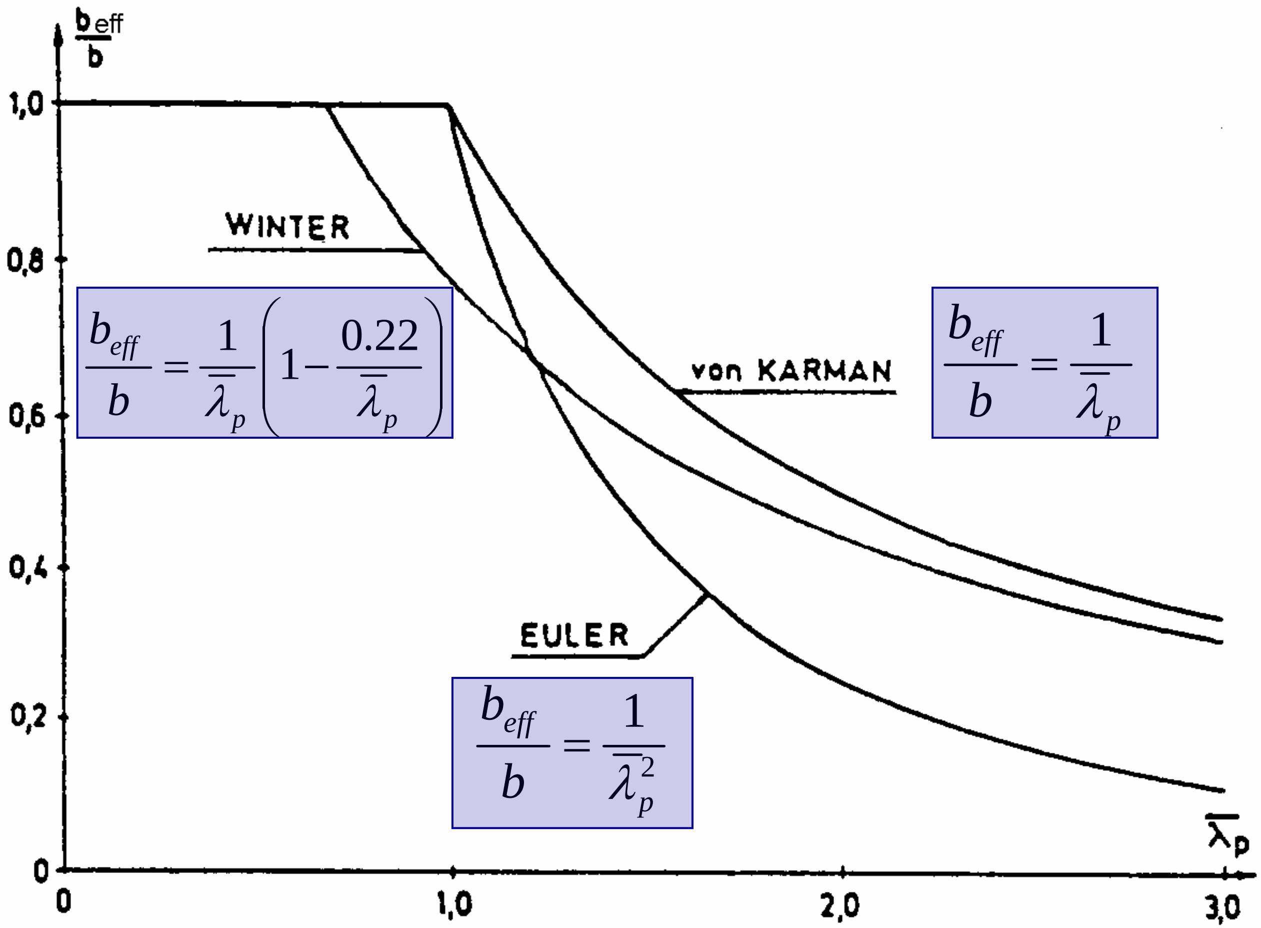
Winter's equation

$$\frac{b_{eff}}{b}=\frac{1}{\bar{\lambda}_p}\left(1-\frac{0.22}{\bar{\lambda}_p}\right)$$

The Winter's expression is currently used in the EC3-Part 1.3, in the AISI Specification and in other national Codes for the design of CF thin-walled steel members

2.2.9 INSTABILITY OF STEEL PLATES

2. Post buckling behaviour : **Nonlinear theory**



$$\frac{b_{eff}}{b} = \frac{1}{\bar{\lambda}_p}\left(1 - \frac{0.22}{\bar{\lambda}_p}\right)$$

$$\frac{b_{eff}}{b} = \frac{1}{\bar{\lambda}_p}$$

$$\frac{b_{eff}}{b} = \frac{1}{\bar{\lambda}_p^2}$$

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}}$$

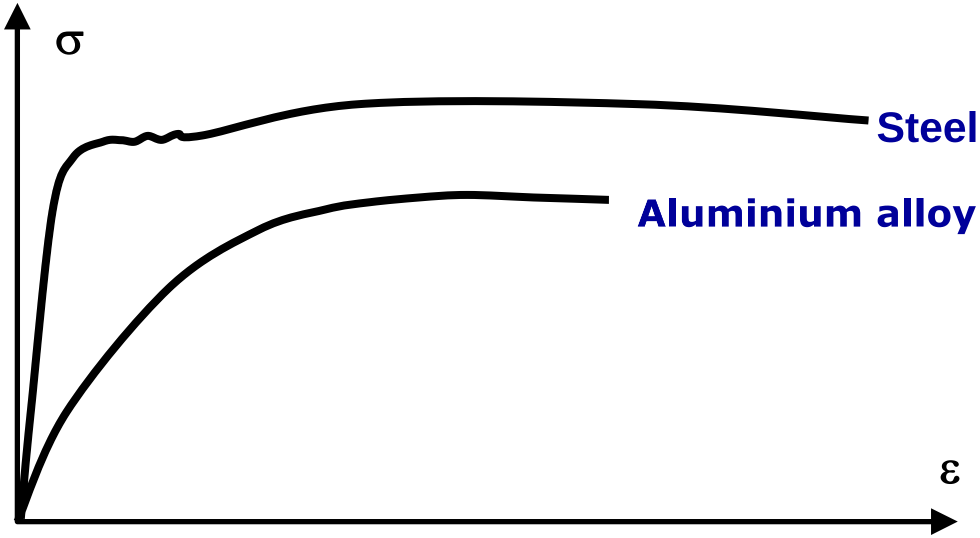
Comparison between equations of Winter, of Von Karman and the critical curve of Euler

2.2.10 INSTABILITY OF ALUMINIUM PLATES

The begin and the evolution of local instability phenomena are strictly connected to the mechanical behaviour of material, which may be characterised by:

- **Elastic-plastic stress-strain law** (like in steel)
- **Nonlinear stress-strain law** (like in aluminium alloys)

In addition, the particular hardening features of the aluminium alloys can play a significant role, mainly in the post-critical behaviour of plate elements which the section is made of.



2.2.10 INSTABILITY OF ALUMINIUM PLATES

Several models have been proposed in the technical literature for modelling the stress-strain relationship of aluminium alloys:

- *discontinuous relationships*, where different formulations are used for each portion of the diagram;

- *continuous relationships*, such as that proposed by Ramberg and Osgood, which is the most used one.

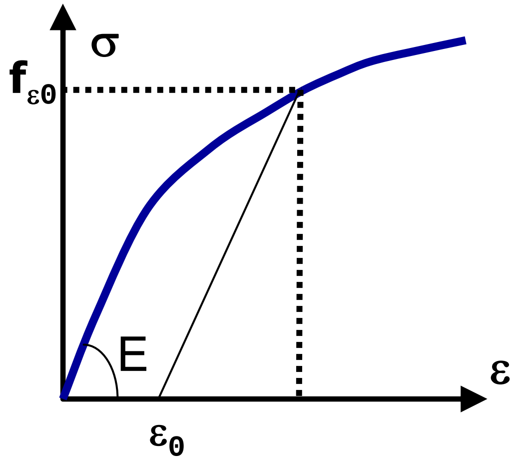
Ramberg and Osgood law

$$\varepsilon = \frac{\sigma}{E} + \varepsilon_0 \cdot \left(\frac{\sigma}{f_{\varepsilon_0}} \right)^n$$

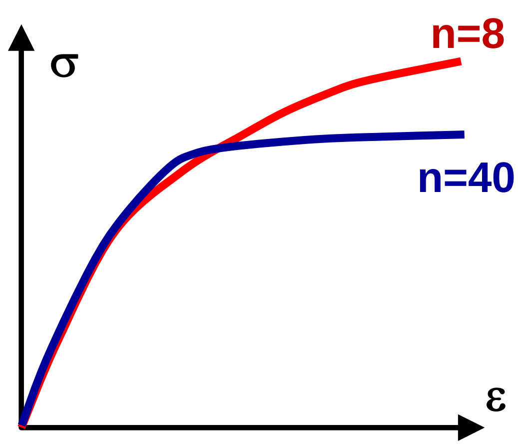
where:

- **E** is the initial elastic modulus
- **n** represents the hardening parameter of the material
- $\mathbf{f_{e0}}$ is the conventional elastic limit stress (usually assumed as that one related to the 0.2% offset proof stress)
- ε_0 is the residual deformation corresponding to the conventional elastic limit stress

2.2.10 INSTABILITY OF ALUMINIUM PLATES



$f_{\varepsilon 0} = f_{0.2}$



Exponent n of Ramberg-Osgood law

$$\varepsilon = \frac{\sigma}{E} + 0.002 \cdot \left(\frac{\sigma}{f_{0.2}} \right)^n \qquad n = \frac{\log 0.5}{\log \frac{f_{0.1}}{f_{0.2}}} \cong \frac{f_{0.2}}{10}$$

The exponent n of Ramberg-Osgood law may be assumed as a material characteristic parameter.
As regards aluminium alloys, the hardening amount depends on several factors:
- the chemical composition of the alloy
- the fabrication process
- the type of heat treatment

In particular, the type of heat treatment is the most influencing one, since it generally produces both a strength increase and a hardening decrease

n ranges from:
- **8 to 15 for non-heat-treated alloy**
- **20 to 40 for heat-treated alloy**

2.2.10 INSTABILITY OF ALUMINIUM PLATES

Critical load in inelastic range

$$\overline{\sigma}_{cr} = \eta \cdot \overline{\sigma}_{cr,e}$$

where:

$\overline{\sigma}_{cr}$ is the normalised inelastic buckling stress

$\overline{\sigma}_{cr,e}$ is the normalise elastic buckling stress

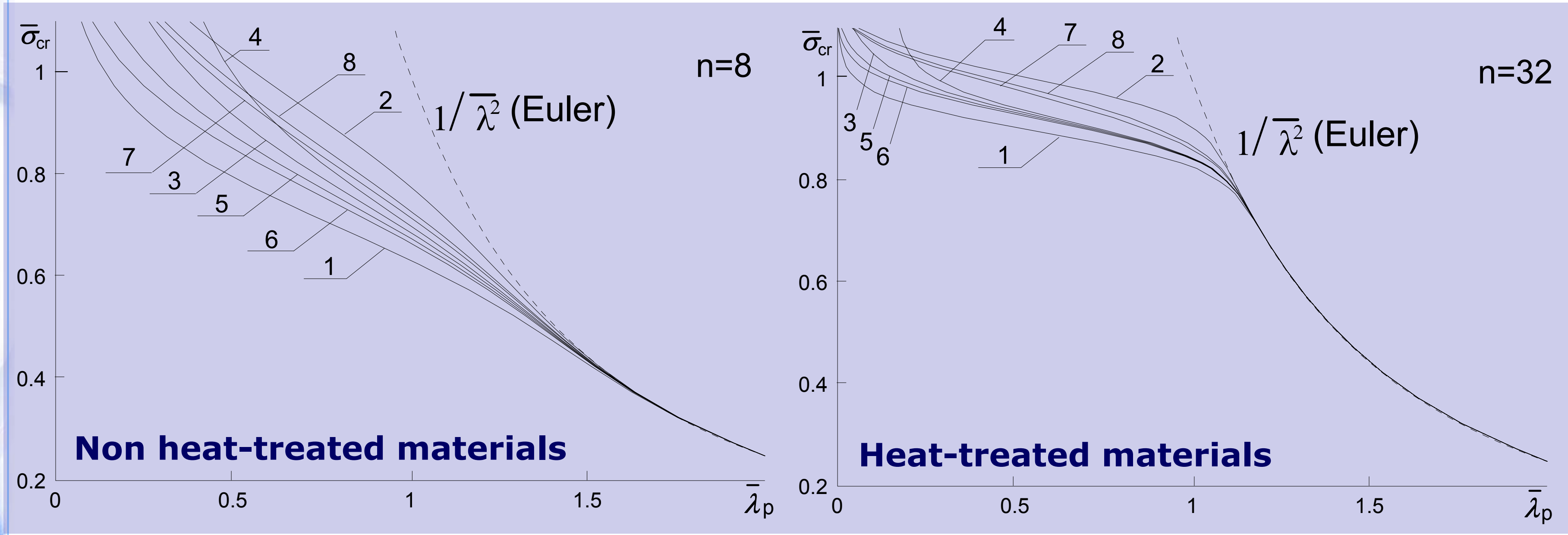
η is the plasticity factor

Model	η factor	References (see Ghersi and Landolfo, 1996)
1	E_t / E	Tangent modulus buckling curve
2	E_s / E	Stowell[1948], Bijlaard[1949], Vol'Mir[1965], Gerard[1957]
3	$\sqrt{E_t / E}$	Bleich[1952], Vol'Mir[1965], Pearson[1950]
4	$\sqrt{E_t / E_s}$	Radhakrishnan[1956]
5	$\frac{\sqrt{E_t E_s}}{E}$	Gerard[1962]
6	$\frac{E_s}{E}\sqrt{\frac{E_t}{E}}$	Weingarten et al. [1960]
7	$\frac{E_s}{E}\left(0.33 + 0.67\sqrt{0.25 + 0.75\frac{E_t}{E_s}}\right)$	Stowell[1948], Bijlaard[1949]
8	$\frac{E_s}{E}\left(0.5 + 0.5\sqrt{0.25 + 0.75\frac{E_t}{E_s}}\right)$	Stowell[1948], Gerard & Becker[1957]

2.2.10 INSTABILITY OF ALUMINIUM PLATES

Critical load in inelastic range: the effect of material hardening and buckling models

Non-dimensional elastic buckling curves corresponding to the above-mentioned buckling models (expression of η), evaluated by using a Ramberg-Osgood type law for material with: **$f_{0.2}$=180 MPa** and **E=70000 MPa**



It is possible to observe that:

- the differences related to the different formulations of η are more evident in case of non-heat-treated materials
- for both n=8 and n=32 all curves converge, when normalised slenderness ratio increases

2.2.10 INSTABILITY OF ALUMINIUM PLATES

Ultimate load: a simulation mode for aluminium plate

In order to extend the Von Karman’s approach to the case of round-house type materials a comprehensive study has been carried out by Ghersi and Landolfo through a simulation model based on the effective width approach which follows step-by-step the increase of strain and stress.

For each given value of strain, the stress is obtained by Ramberg-Osgood law and the consequent effective width is evaluated. The ultimate strength is defined as the value that corresponds to a maximum or, if the strength is always increasing, to a limit value of strain (usually the one corresponding to $f_{0.2}$).

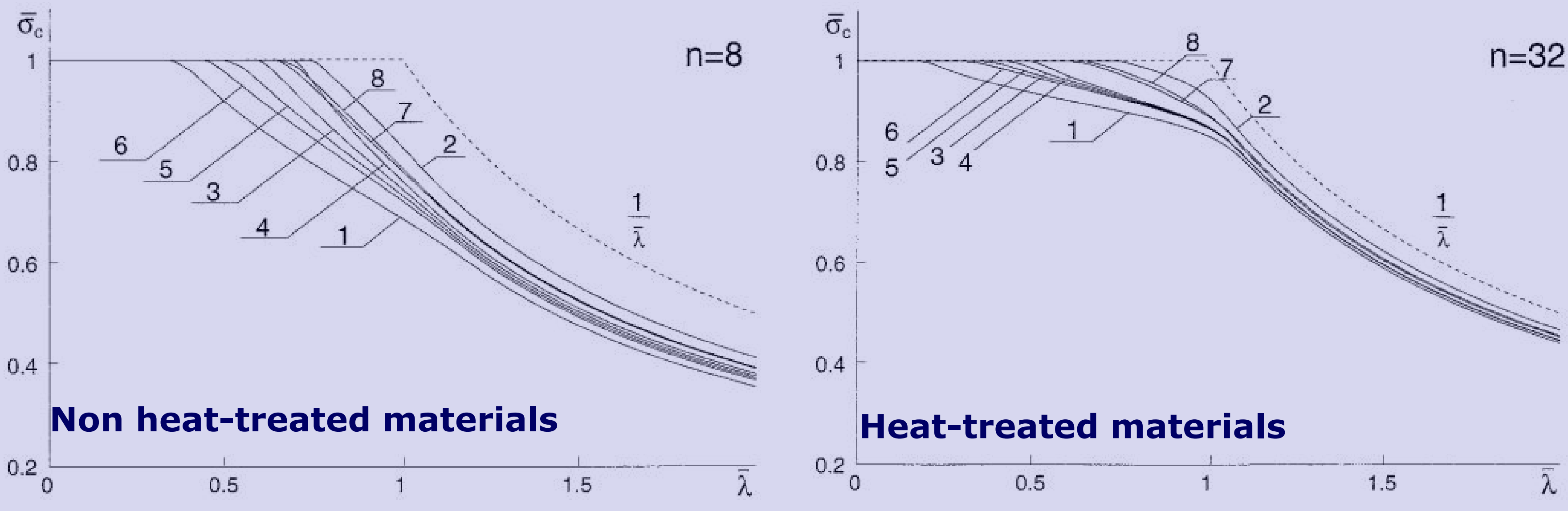
The results obtained in this way depend on

- material properties (ultimate strength, material hardening)
- formulation adopted for the η factor (buckling model)
- geometrical and mechanical imperfections

2.2.10 INSTABILITY OF ALUMINIUM PLATES

Ultimate load: the effect of material hardening and buckling models

Non-dimensional inelastic buckling curves corresponding to the above-mentioned buckling models (expression of η), evaluated by using a Ramberg-Osgood type law for material with: **$f_{0.2}$=180 MPa** and **E=70000 MPa**



It is possible to observe that:

- in case of non-heat-treated materials the differences related to the different formulations of η are remarkable for all values of
- in case of heat-treated materials the results are very close for $\bar{\lambda} > 1$, while the scattering is greater when $\bar{\lambda} < 1$

2.2.10 INSTABILITY OF ALUMINIUM PLATES

Ultimate load: comparison with test results

The procedure previously described has been applied for predicting the strength of plates tested by Dwight and Mofflin :
76 tests on individual aluminium plates loaded in uniaxial compression

Research objectives

To investigate the effect of:

- material hardening (hardening parameter n);
- buckling models (plasticity factor η);
- geometrical and mechanical imperfections (imperfection parameter α);

on the structural response of a aluminium thin-walled plate.

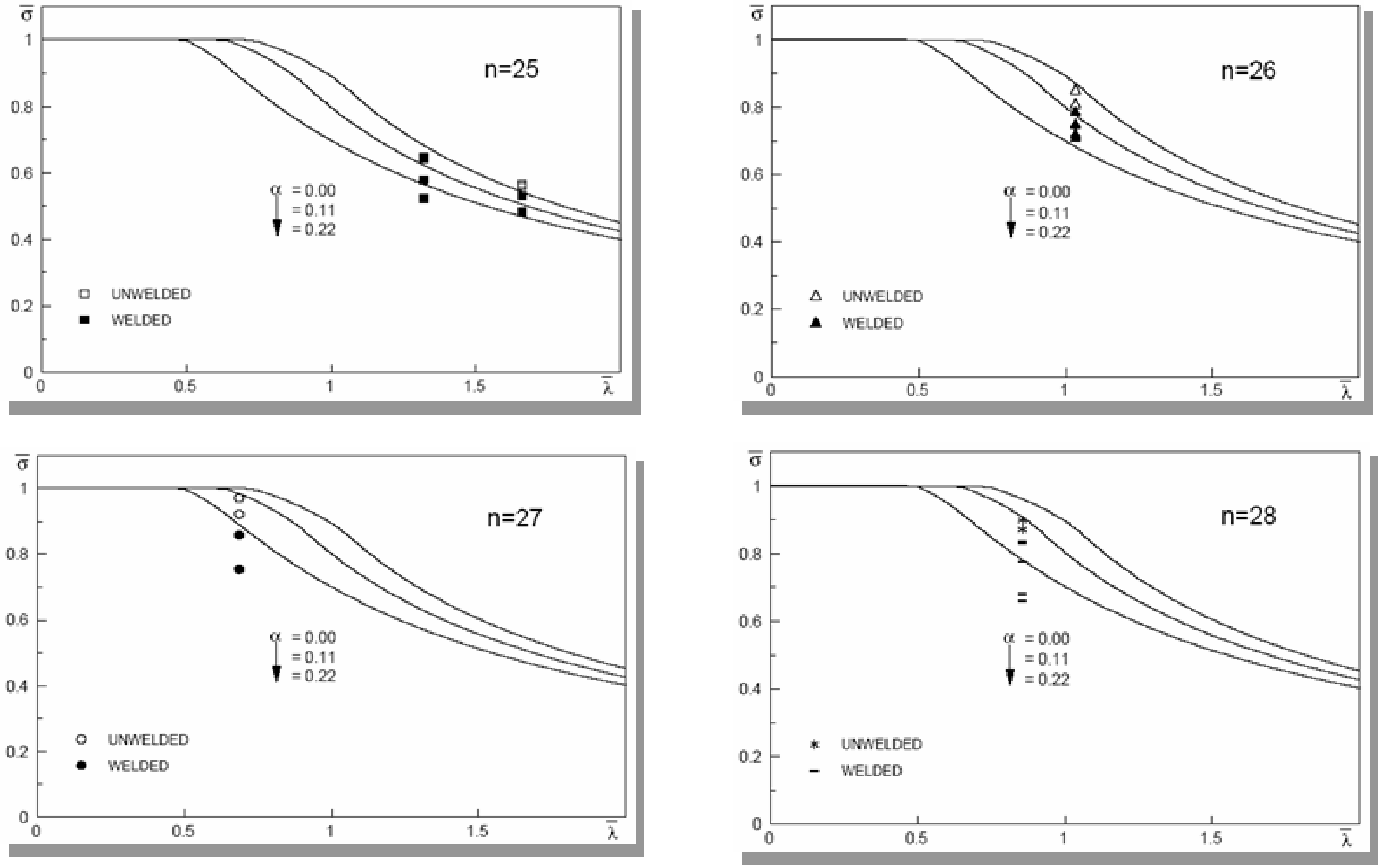
Assumptions

- material behaviour described through a Ramberg-Osgood law;
- $f_{0.2}$ equal to the experimental values;
- $n=25 \div 28$ (heat-treated material);
  $n=8 \div 18$ (non-heat-treated material):
- $\eta=\sqrt{(Et/Es)}$ (according to model 4)

$\alpha=0$ (no imperfections)
$\alpha=0.11$ (mean imperfections)
$\alpha=0.22$ (high imperfections)

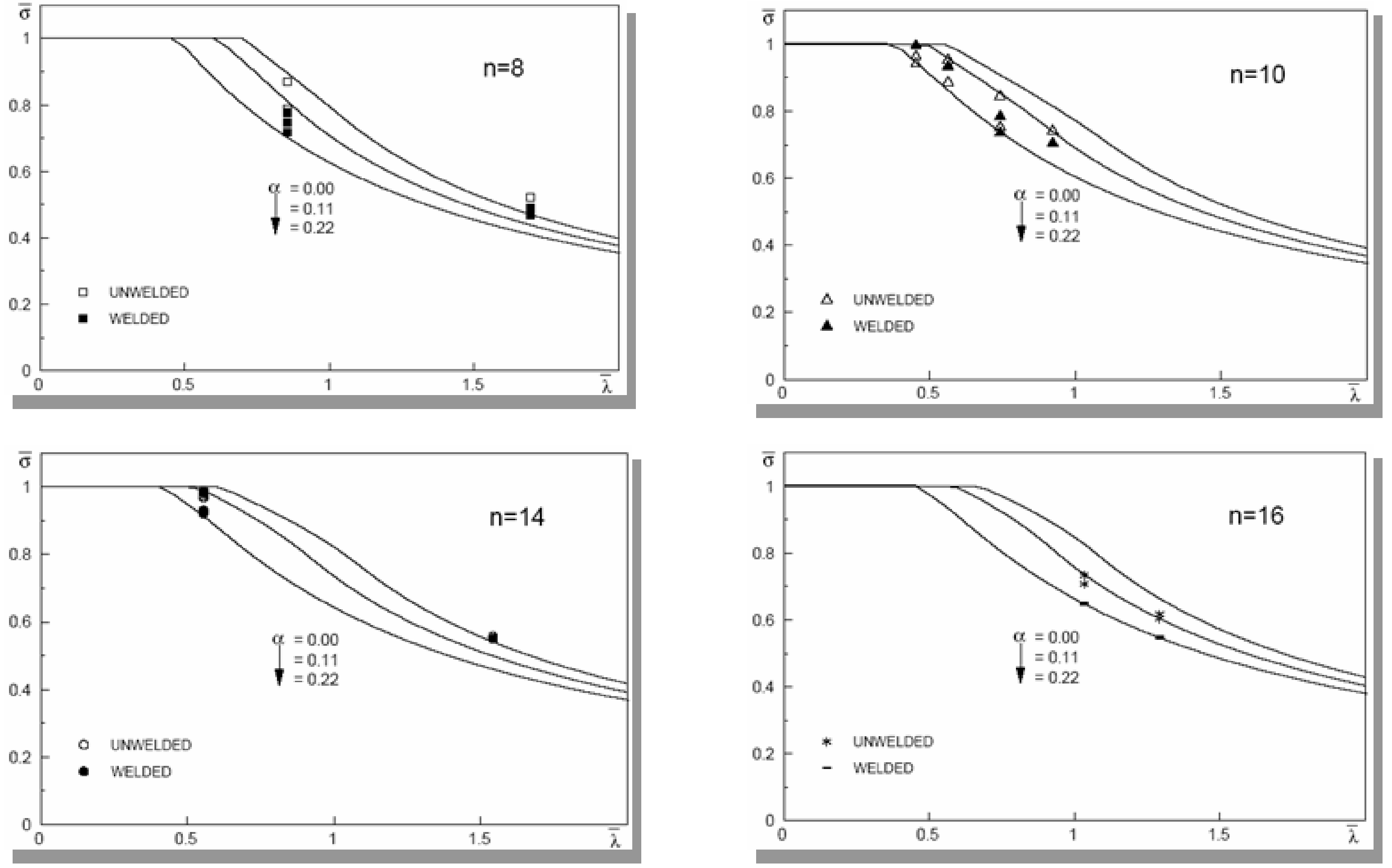
2.2.10 INSTABILITY OF ALUMINIUM PLATES

Ultimate load: comparison with test results - Heat-treated material



2.2.10 INSTABILITY OF ALUMINIUM PLATES

Ultimate load: comparison with test results –Non heat-treated material



2.2.10 INSTABILITY OF ALUMINIUM PLATES

Ultimate load: comparison with test results

Results:

- the numerical procedure allows a good prediction of plate strength in all examined cases;
- the tests performed on unwelded plates are in agreement with the numerical results corresponding to an α factor equal to 0.11;
- in case of welded plates the curve with $\alpha=0.22$ appear to be more adequate.

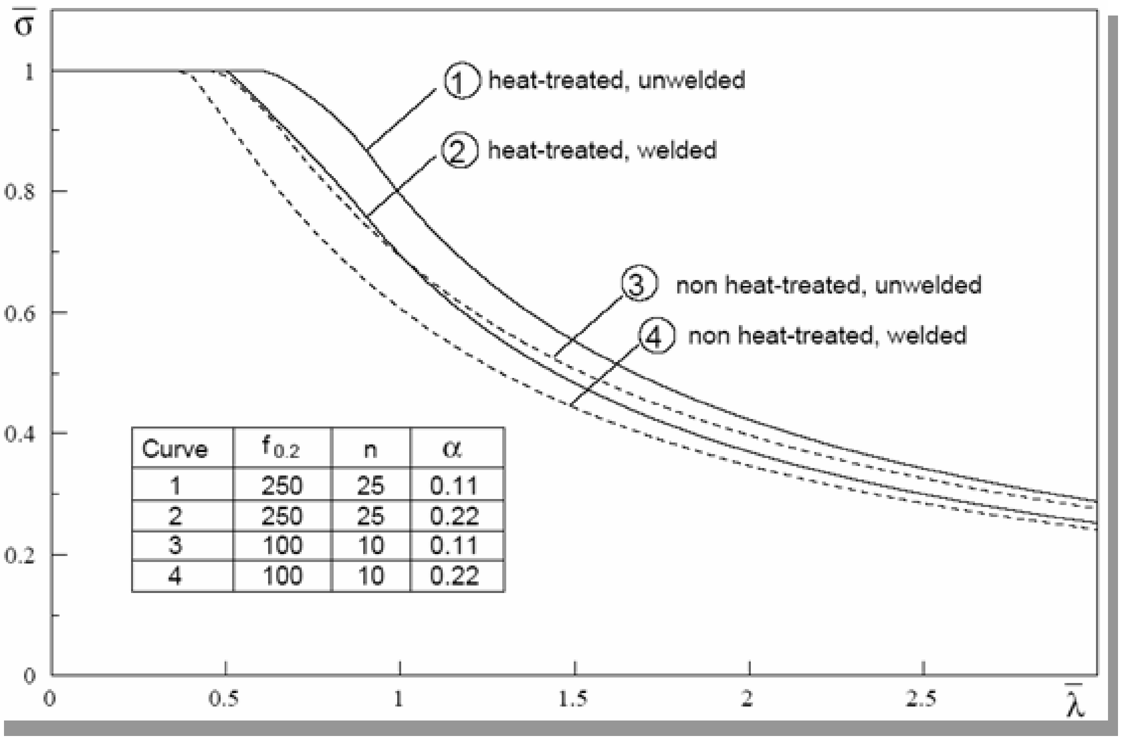
According to the above results, four theoretical strength curves for aluminium plate in compression can be defined, all corresponding to simple supported edge conditions.

They cover:

- both unwelded ($\alpha=0.11$) and welded ($\alpha=0.22$) plates
- made of non-heat-treated ($n=10$, $f_{0.2}=100$ MPa) and heat-treated ($n=25$, $f_{0.2}=250$ MPa) alloys

2.2.10 INSTABILITY OF ALUMINIUM PLATES

Theoretical buckling curves

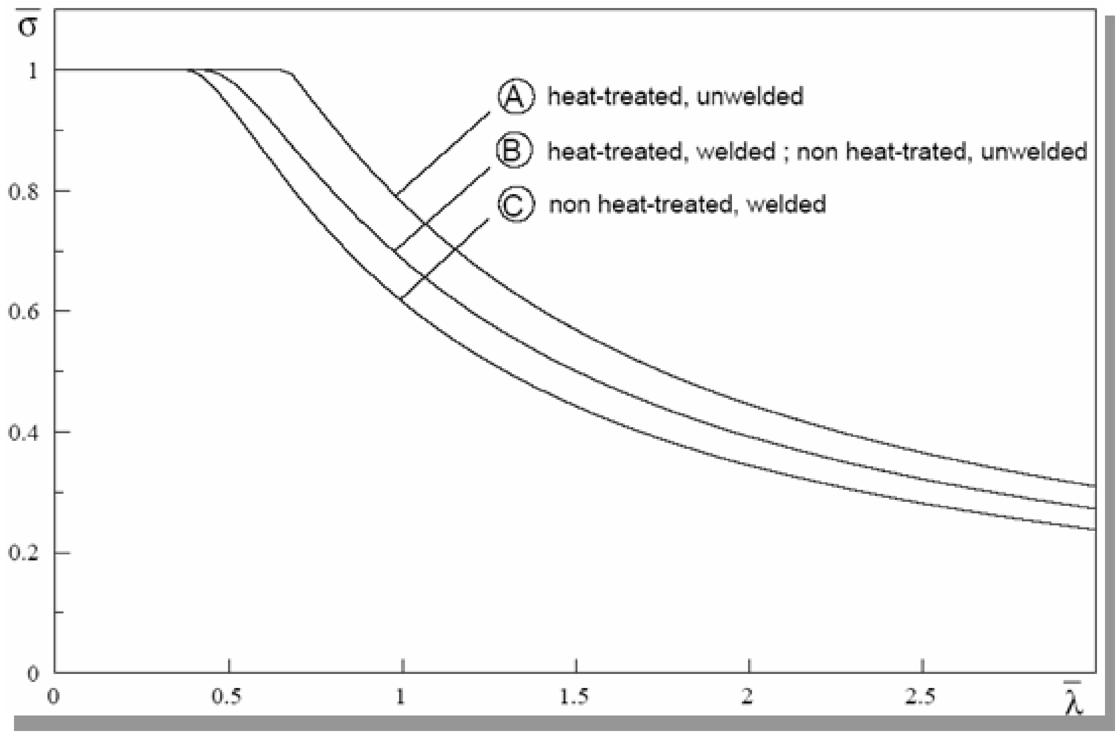


Curve	$f_{0.2}$	n	α
1	250	25	0.11
2	250	25	0.22
3	100	10	0.11
4	100	10	0.22

The comparison among the theoretical curves (1, 2, 3 and 4) shows that the curve for welded plates in heat-treated alloys (curve 2) and the one corresponding to unwelded plate in non heat-treated material (curve 3) are very similar and quite coincident.

2.2.10 INSTABILITY OF ALUMINIUM PLATES

Design buckling curves (*Landolfo and Mazzolani*)



On the basis of this evidence, it is possible to conclude that only three design curves are enough to characterise the buckling behaviour of aluminium plates in compression.

- **Curve A**: unwelded plates in heat-treated alloy (n>10)
- **Curve B**: welded plates in heat-treated alloy (n>10); unwelded plates in non-heat-treated alloy (n≤10)
- **Curve C**: welded plates in non-heat-treated alloy (n≤10)

2.2.10 INSTABILITY OF ALUMINIUM PLATES

Design buckling curves

These design buckling curves can be expressed in a non-dimensional form by the following equation:

$$\bar{\sigma} = (\omega_1 / \bar{\lambda}) \cdot (1 - \omega_2 / \bar{\lambda})$$

where:

ω_1 and ω_2 are numerical coefficients given together the limit value of the normalised slenderness $\bar{\lambda}_0$ which corresponds to $\bar{\sigma} = 1$

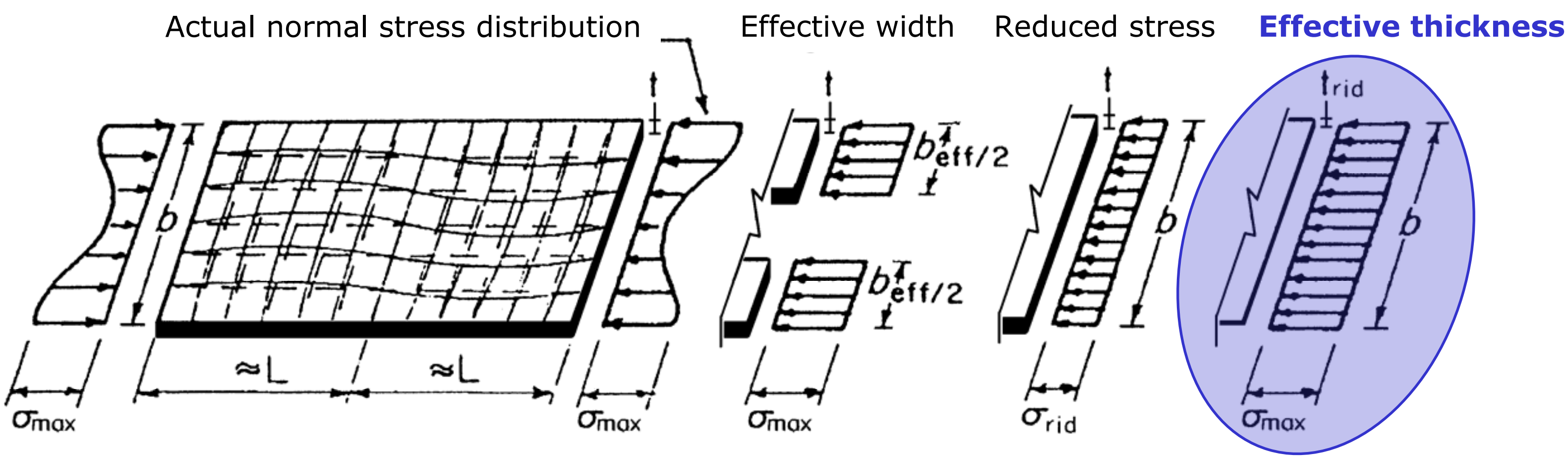
Curve	ω_1	ω_2	$\bar{\lambda}_0$
A	1.00	0.22	0.673
B	0.88	0.22	0.440
C	0.76	0.19	0.380

For the first curve, such relationship is coincident with the Winter formulation, which is assumed in the American and European codes on cold-formed steel sections for determining the effective width ratio.
For the other ones, a similar structure is kept, practically by assuming appropriate equivalent reduction factors in the Winter formulation. **This approach has been used as a basis for checking slender sections in the final version of Eurocode 9**.

2.2.10 INSTABILITY OF ALUMINIUM PLATES

General

The effect of local buckling on each compression element of the cross-section shall be conventionally accounted by replacing the non-uniform distribution of stress, occurring in the post-buckling range, with a uniform distribution of the *maximum stress* (σ_{max}) acting on a reduced portion of the element, having the same *width* (***b***) but a *reduced thickness* (effective thickness, t_{eff}).



2.2.11 LOCAL AND DISTORTIONAL BUCKLING

Local and distortional buckling - Eurocode 9 Part 1.1

Part 1.1 of Eurocode 9 uses the above-mentioned approach for class 4 compression elements.

For sake of simplicity, it modifies the formulations by explicitly introducing the $\beta = b/t$ ratio and rounding the subsequent coefficients so as to obtain integers.

Part 1.1 of Eurocode 9 prescribes to use the same formulations also for stiffened elements and to apply the factor ρ to the area of the stiffener as well as to the basic plate thickness.

Local and distortional buckling - Eurocode 9 Part 1.4

Part 1.4 of Eurocode 9 gives a more specific and detailed approach for CF thin-walled aluminium sheeting, although it is easily extensible to aluminium CF.

2.2.12 LOCAL AND DISTORTIONAL BUCKLING – PART 1.1

The effective section is obtained by using a local buckling coefficient ρ_c that factor down the thickness of any slender element which is wholly or partly in compression.

$$\rho_c = 1.0 \quad \text{if} \quad \beta \le \beta_3$$

$$\rho_c = \frac{C_1}{\beta/\varepsilon} - \frac{C_2}{(\beta/\varepsilon)^2} \quad \text{if} \quad \beta > \beta_3$$

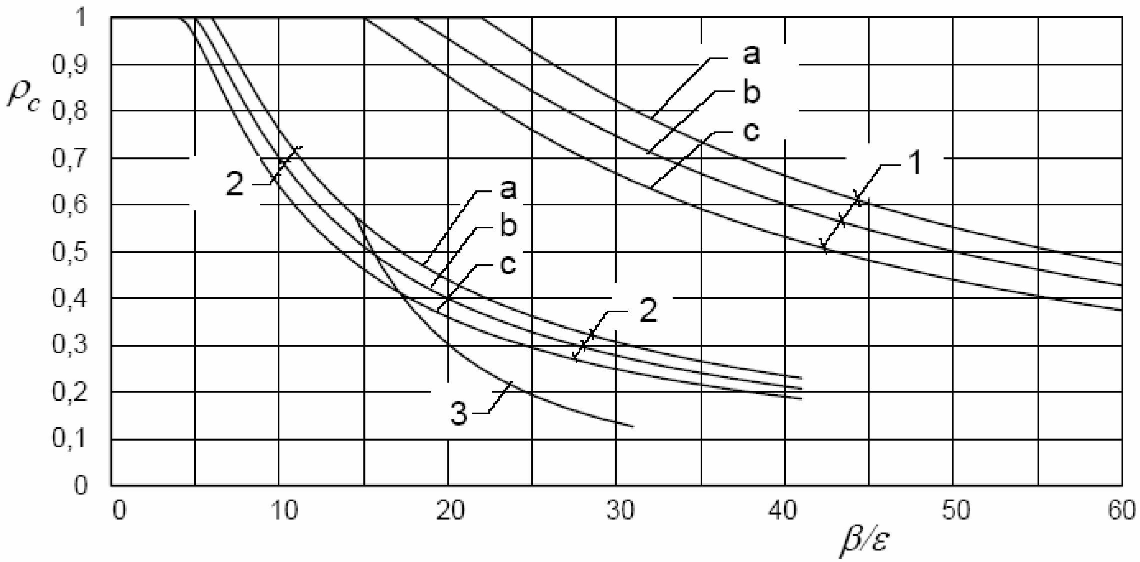
Material classification according to Table 3.2	Internal part		Outstand part	
	C_1	C_2	C_1	C_2
Class A, without welds	32	220	10	24
Class A, with welds	29	198	9	20
Class B, without welds	29	198	9	20
Class B, with welds	25	150	8	16

Constants C_1 and C_2 in expressions for ρ_c

Class A: *Heat-treated*
Class B: *Non heat-treated*

2.2.12 LOCAL AND DISTORTIONAL BUCKLING – PART 1.1

Design buckling curves



1 Internal parts and round tubes, 2 Symmetrical outstands, 3 Un-symmetrical outstands
a) class A, without welds,
b) class A, with welds or class B, without welds
c) class B, with welds

Relationship between ρ_c and β/ε for outstands, internal parts and round tubes

Class A: *Heat-treated*
Class B: *Non heat-treated*

2.3 General rules for cold-formed sheeting *Part 1.4* (EN 1999-1-4)

2.3.1 BASES OF DESIGN

For the design of structures made of cold-formed sheeting a distinction Structural Classes dependent on its function in the structure defined as follows:

Structural Class I	Construction where cold-formed sheeting is designed to contribute to the overall strength and stability of the structure
Structural Class II	Construction where cold-formed sheeting is designed to contribute to the strength and stability of individual structural components
Structural Class III	Construction where cold-formed sheeting is used as a component that only transfers loads to the structure

2.3.2 MATERIAL

Properties

Designation numerical EN AW-	Designation chemical EN AW-	Durability rating 5)	Temper 1), 2), 3)	Thickness up to mm	f_u R_m N/mm^2	f_o $R_{p0,2}$ 1) N/mm^2	A_{50} % 4)
3003	AlMn1Cu	A	H18	3,0	190	170	2
			H48	3,0	180	165	2
3004	AlMn1Mg1	A	H14 | H24/H34	6 | 3	220	180 | 170	2-3 | 4
			H16 | H26/H36	4 | 3	240	200 | 190	1-2 | 3
			H18 | H28/H38	3 | 1,5	260	230 | 220	1-2 | 3
			H44	3	210	180	4
			H46	3	230	200	3
			H48	3	260	220	3
3005	AlMn1Mg0,5	A	H16	4	195	175	2
			H18 | H28	3	220	200 | 190	2 | 2-3
			H48	3	210	180	2
3103	AlMn1	A	H18	3	185	165	2
3105	AlMn0,5Mg0,5	A	H18 | H28	3 | 1,5	195	180 | 170	1 | 2
			H48	3	195	170	2
5005	AlMg1(B)	A	H18	3	185	165	2
5052	AlMg2,5	A	H14	6	230	180	3-4
			H16 | H26/H36	6	250	210 | 180	3 | 4-6
			H18 | H28/H38	3	270	240 | 210	2 | 3-4
			H46	3	250	180	4-5
			H48	3	270	210	3-4
5251	AlMg2	A	H14	6	210	170	2-4
			H16 | H26/H36	4	230	200 | 170	2-3 | 4-7
			H18 | H28/H38	3	255	230 | 200	2 | 3
			H46	3	210	165	4-5
			H48	3	250	215	3

1) The values for temper H1x, H2x, H3x according to EN 485-2:1994-11

2) The values for temper H4x (coil coated sheet and strip) according to EN 1396:1997-2

3) If two (three) tempers are specified in one line, tempers separated by "|" have different technological values, but separated by "/" have same values.

4) A50 may be depending on the thickness of material in the listed range, therefore sometimes also a A50- range is given.

5) Durability rating, see EN 1999-1-1

Characteristic values of 0,2% proof strength fo, ultimate tensile strength, fu, elongation A50, for sheet and strip for tempers with fo > 165 N/mm2 and thickness between 0,5 and 6 mm

2.3.3 SECTION PROPERTIES

Thickness and geometrical tolerances

The provisions for design by calculation given in this *EN 1999-1-4* may be used for alloy within the following ranges of *nominal thickness* t_{nom} of the sheeting exclusive of organic coatings:

$$t_{nom} \geq 0{,}5 \text{ mm}$$

- The *nominal thickness* t_{nom} should be used as design thickness t if a negative deviation is less than 5 %.

- Otherwise

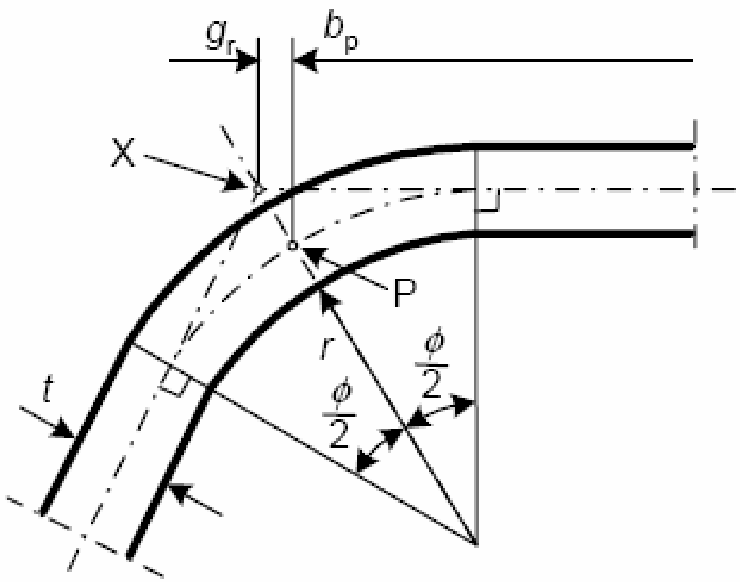
$$t = t_{nom} \, (100 - \text{dev}) \, / \, 95 \quad (3.1)$$

where dev is the negative deviation in %.

2.3.3 SECTION PROPERTIES

Influence of rounded corners

As in the Eurocode 3, also *Eurocode 9 – Part 1.4* takes into account the presence of rounded corners by referring to the notational flat width b_p of each plane element, measured from the midpoints of adjacent corner elements.



(a) midpoint of corner or bend

X is intersection of midlines

P is midpoint of corner

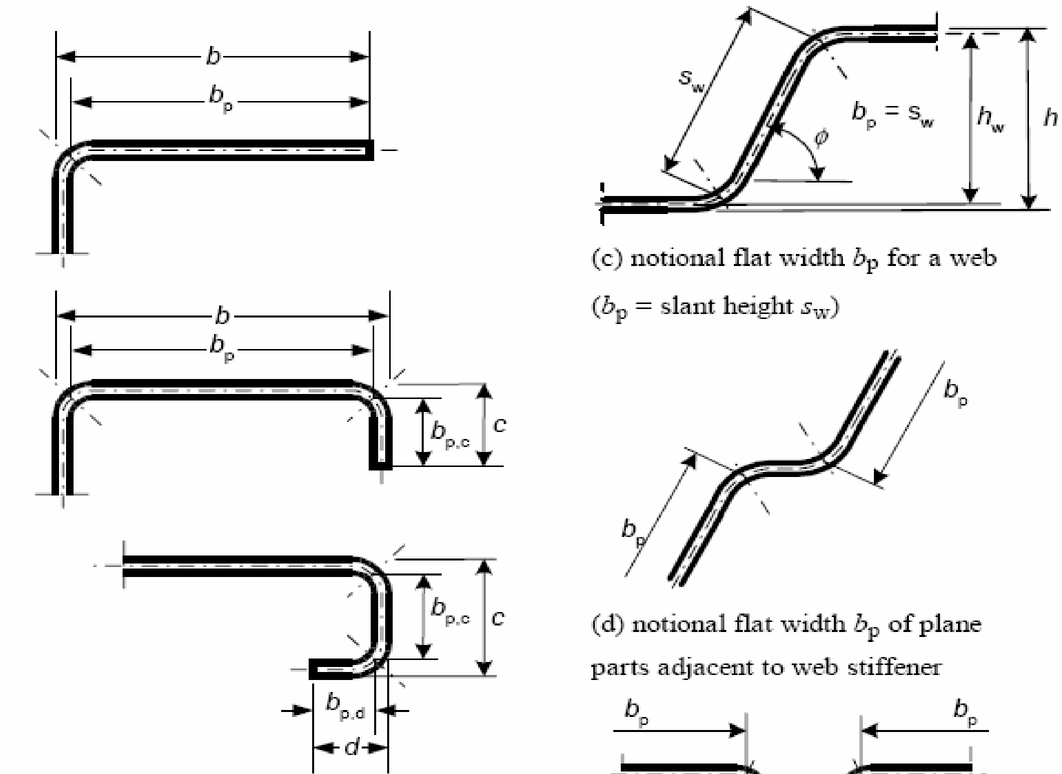
$$r_m = r + t/2$$

$$g_r = r_m \left(\tan\left(\frac{\phi}{2}\right) - \sin\left(\frac{\phi}{2}\right) \right)$$

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

2.3.3 SECTION PROPERTIES

Influence of rounded corners



(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

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

2.3.3 SECTION PROPERTIES

Influence of rounded corners

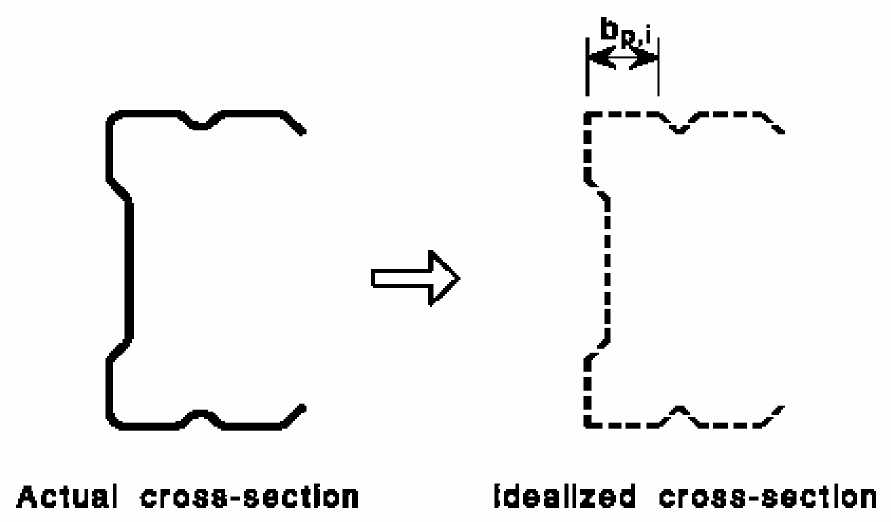
According to the code provisions, the influence of rounded corners with internal radius

$r \leq 10\ t$

And

$r \leq 0.15\ b_p$

on section properties might be neglected, and the cross-section might be assumed to consist of plane elements with sharp corners



Approximate allowance for rounded corners

2.3.3 SECTION PROPERTIES

Geometrical proportions

The provisions of *Eurocode 9 – Part 1.4* may be applied only to cross-sections within the range of width-to-thickness ratios for which sufficient experience and verification by testing is available:

- **$b/t \leq 300$ for compressed flanges**
- **$b/t \leq E/f_0$ for webs**

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

2.3.4 LOCAL AND DISTORTIONAL BUCKLING – PART 1.4

Unreinforced (without stiffeners) plane elements

The effective thickness t_{eff} of a compression element is evaluated as:

$$t_{eff} = \rho\, t$$

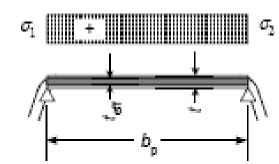
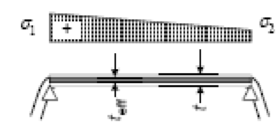
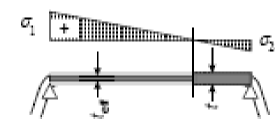
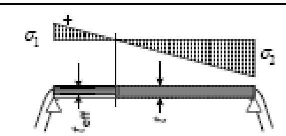
where

ρ is a reduction factor based on the largest compressive stress $\sigma_{com,Ed}$ acting in the element when the resistance of the cross-section is reached.

When $\sigma_{com,Ed} = f_0/\gamma_{M1}$, *Part 1.4 of Eurocode 9* suggests to evaluate the reduction factor ρ by means of the following expressions:

$$\rho = 1.0 \quad \text{if} \quad \bar{\lambda}_p \le \bar{\lambda}_{\lim} \qquad\qquad \rho = \frac{\alpha \cdot (1 - 0.22/\bar{\lambda}_p)}{\bar{\lambda}_p}$$

$$\bar{\lambda}_p = \sqrt{\frac{f_0}{\sigma_{cr}}} = \frac{b_p}{t}\sqrt{\frac{12\,(1-\nu^2)\; f_0}{\pi^2\; E\, k_\sigma}} \cong 1.052\, \frac{b_p}{t}\sqrt{\frac{f_0}{E\, k_\sigma}}$$

Cross-section part (+ = compression)	$\psi = \sigma_2/\sigma_1$	Buckling factor k_σ
	$\psi = +1$	$k_\sigma = 4{,}0$
	$+1 > \psi \ge 0$	$k_\sigma = \dfrac{8{,}2}{1{,}05 + \psi}$
	$0 > \psi \ge -1$	$k_\sigma = 7{,}81 - 6{,}26\psi + 9{,}78\psi^2$
	$-1 > \psi \ge -3$	$k_\sigma = 5{,}98(1-\psi)^2$

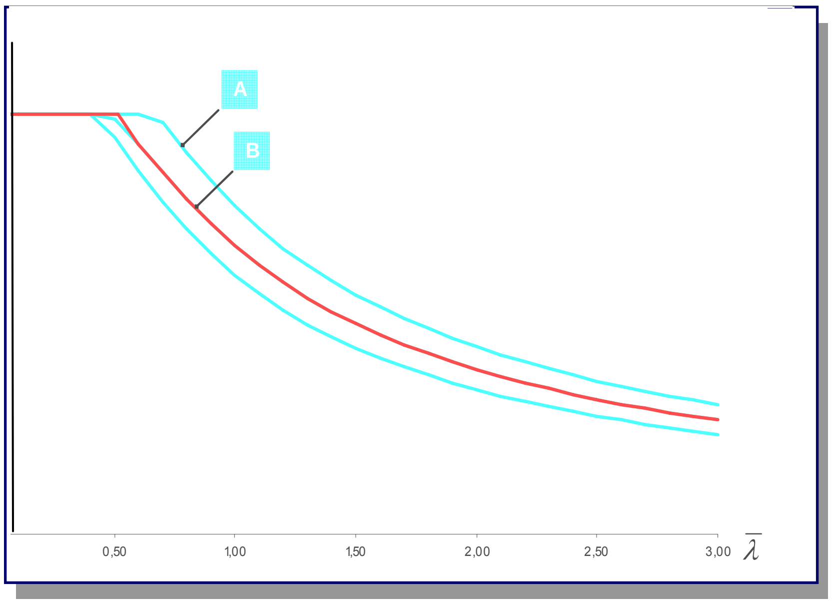
Buckling factor k_σ for internal compression elements

$\bar{\lambda}_{\lim}$	α
0,517	0,90

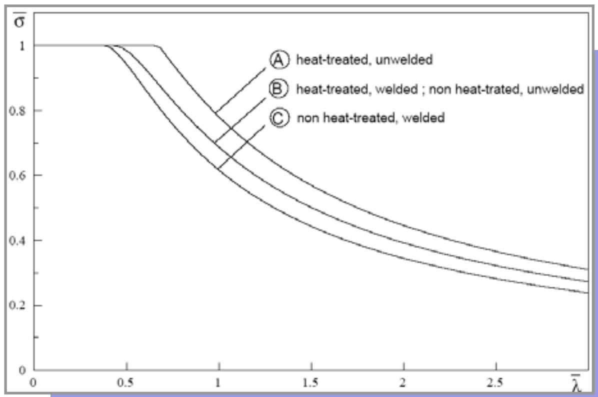
Parameters λ_{lim} and α

2.3.4 LOCAL AND DISTORTIONAL BUCKLING – PART 1.4

Comparison buckling curves

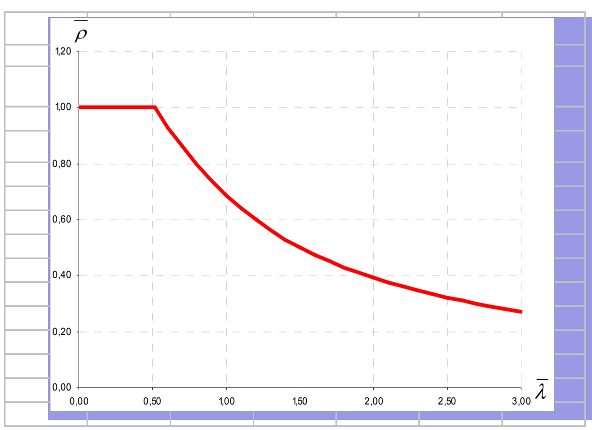


Comparison buckling curves



ALUMINIUM PLATES

Design buckling curves (Landolfo and Mazzolani)



ALUMINIUM SHEETING

- **A** heat-treated, unwelded plates
- **B** heat-treated, welded plates; non heat-treated unwelded plates
- **C** non heat-treated, welded
- **D** aluminium sheeting curve

2.3.4 LOCAL AND DISTORTIONAL BUCKLING – PART 1.4

Unreinforced (without stiffeners) plane elements

If $\sigma_{com,Ed} < f_0 / \gamma_{M1}$, *Part 1.4 of Eurocode 9* suggests to evaluate the reduction factor ρ by above presented expressions:

$$\rho = 1.0 \quad \text{if} \quad \bar{\lambda}_{p,red} \leq \bar{\lambda}_{\lim}$$

$$\frac{\alpha \cdot (1 - 0.22 / \bar{\lambda}_{p,red})}{\bar{\lambda}_{p,red}} \quad \text{if} \quad \bar{\lambda}_{p,red} >$$

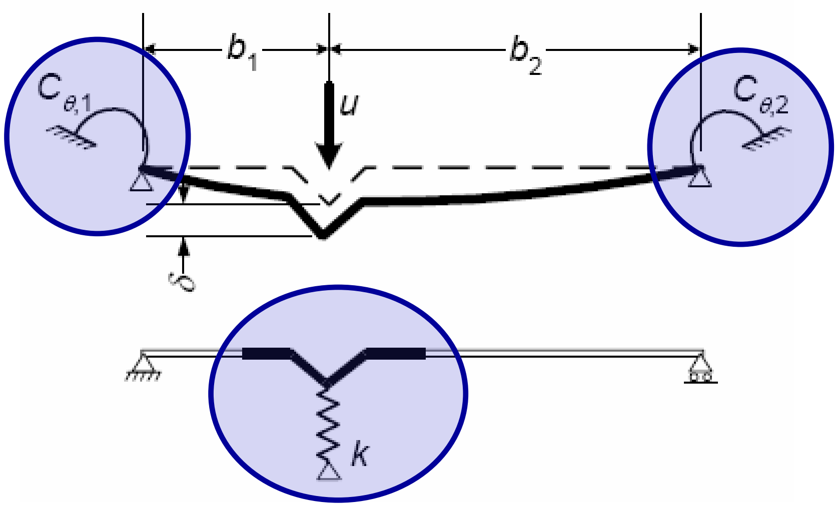
but *replace the plate slenderness* by the reduced plate slenderness:

$$\bar{\lambda}_{p.red} = \bar{\lambda}_p \sqrt{\frac{\sigma_{com,Ed}}{f_0 / \gamma_{M1}}}$$

2.3.4 LOCAL AND DISTORTIONAL BUCKLING – PART 1.4

Plane cross-section parts with intermediate stiffeners – General method

The *effectiveness of the restraint* provided by the stiffeners is analysed assuming that they behave as compression members with continuous elastic restrain, having spring stiffness dependent on the flexural stiffness of the adjacent elements. The approach is analogous to the one followed by Eurocode 3, with some modifications necessary for taking into account the peculiarities of the aluminium plates' buckling.



Model for determination of spring stiffness

u is unit length
k is the spring stiffness per unit length may be determined from:

$$k = u / \delta$$

δ is the deflection of a transverse plate strip due to the unit load u acting at the centroid (b1) of the effective part of the stiffener

$C_{\theta,1}$ and $C_{\theta,2}$ are the values of the rotational spring stiffness from the geometry of the cross-section.

2.3.4 LOCAL AND DISTORTIONAL BUCKLING – PART 1.4

Design procedure – Iterative method

The design procedure should be carried out in steps as follows:

- **STEP 1**

  Obtain an initial effective cross-section for the stiffener to calculate the cross-section area A_s using effective thickness determined by assuming that the stiffener is longitudinally supported and that

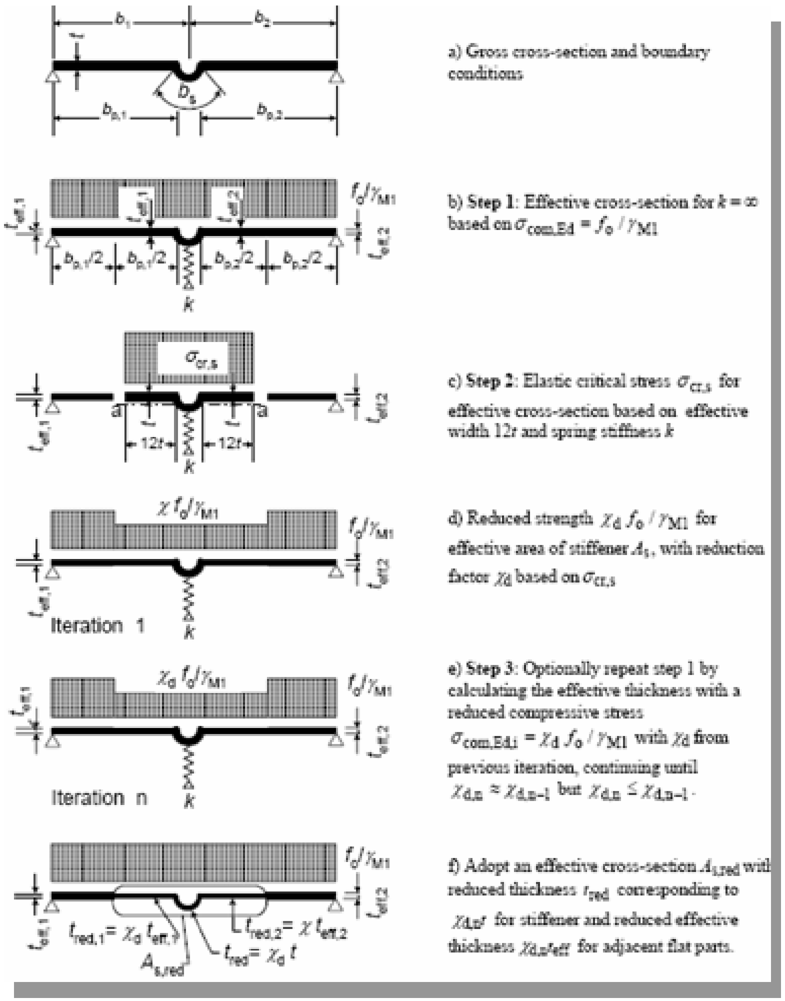
  $$\sigma_{com,Ed} = f_o/\gamma_{M1}$$

- **STEP 2**

  Use another effective cross-section of the stiffener to calculate the effective second moment of inertia in order to determine the reduction factor for distortional buckling, allowing for the effects of the continuous spring restraint

- **STEP 3**

  Optionally iterate to refine the value of the reduction factor for buckling of the stiffener



Model for calculation of compression resistance of a flange with intermediate stiffener

2.3.4 LOCAL AND DISTORTIONAL BUCKLING – PART 1.4

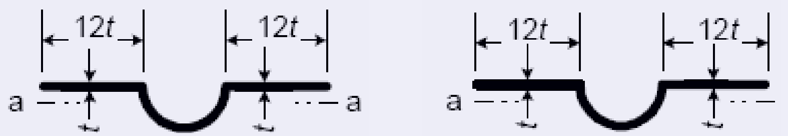
Trapezoidal sheeting profiles with intermediate stiffeners

This sub-clause should be used for flanges with intermediate flange stiffeners and for webs with intermediate stiffeners.

Flanges with intermediate stiffeners

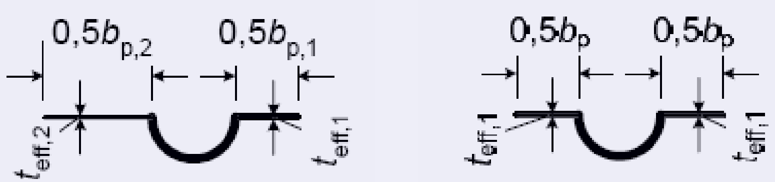
Cross – section for I_s

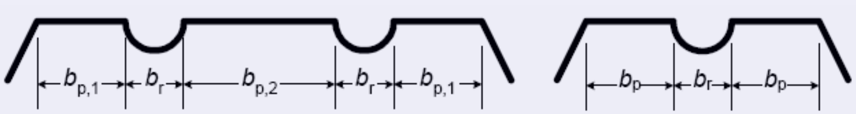
(effective second moment of area of the stiffener)



Cross – section for A_s

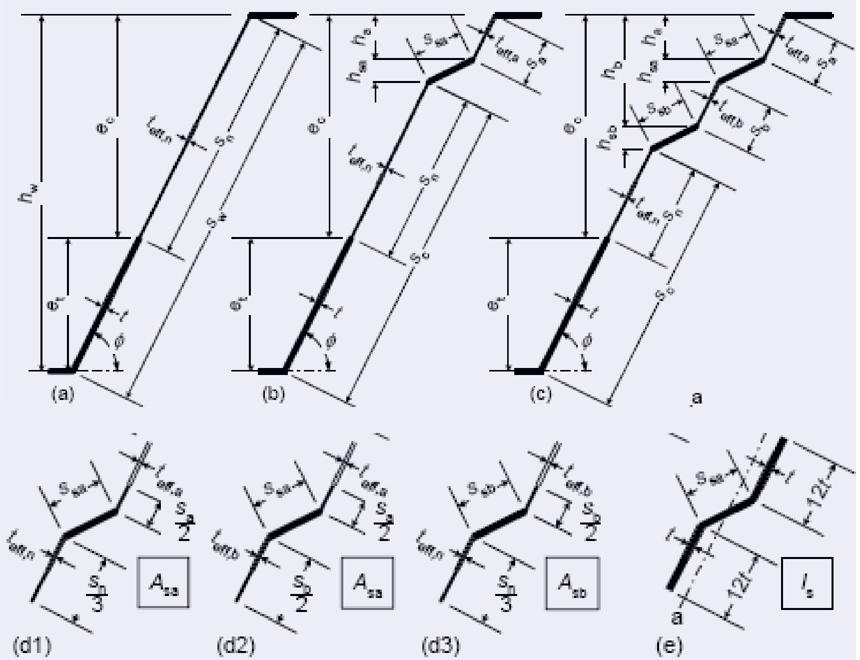
(effective cross-sectional area)





Effective cross section for calculation of I_s and A_s for compression flange with two or one stiffener

Webs with up to two intermediate stiffeners under stress gradient



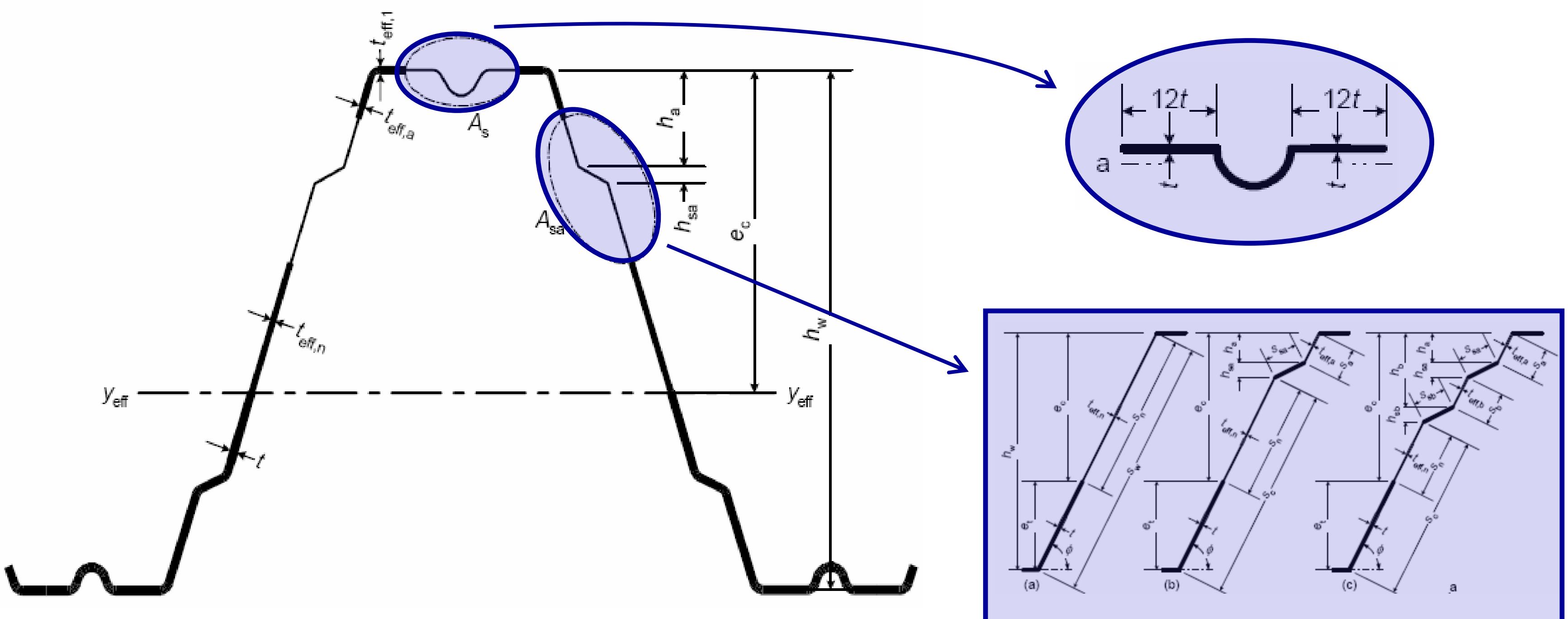
Effective cross-sections of webs of cold-formed profiled sheets

Prof. Raffaele Landolfo - University of Naples “Federico II”

2.3.4 LOCAL AND DISTORTIONAL BUCKLING – PART 1.4

Trapezoidal sheeting profiles with intermediate stiffeners

In the case of sheeting with intermediate stiffeners in the flanges and in the webs interaction between the distortional 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.



Effective cross section of cold-formed profiled sheeting with flange stiffeners and web stiffeners

2.3.5 RESISTANCE OF CROSS-SECTIONS – PART 1.4

Resistance under axial tension

The design tension resistance of a cross-section $N_{t,Rd}$ shall be determined by assuming that it is subjected to a uniform tensile stress equal to f_0/γ_{M1}:

$$N_{t,Rd} = \frac{A_g\ f_0}{\gamma_{M1}} \quad \text{but} \quad N_{t,Rd} \leq F_{net,Rd}$$

where:

A_g is the gross area of the cross section
f_0 is the 0,2% proof strength
$F_{net,Rd}$ is the net-section resistance for the appropriate type of mechanical

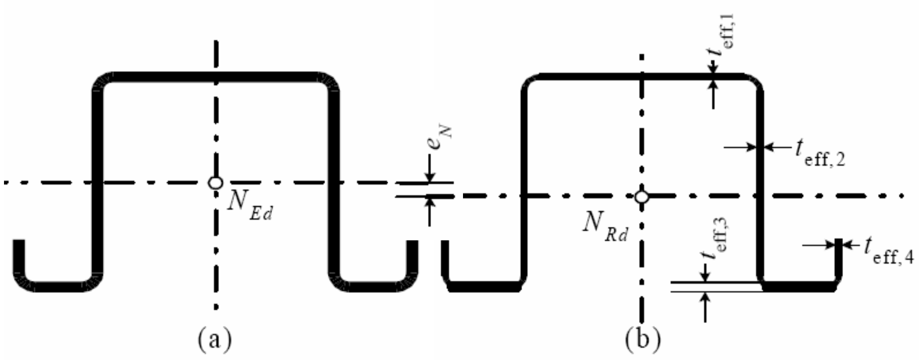
2.3.5 RESISTANCE OF CROSS-SECTIONS – PART 1.4

Resistance under axial compression

Effective area A_{eff} is less than the gross area A_g section with reduction due to local and/or distortional buckling.

$$A_{eff} \leq A_g$$

The design compression resistance of a cross-section $N_{c,Rd}$ shall be determined by considering the effective area A_{eff} of the cross-section subject to a uniform compressive stress $\sigma_{com,Ed}$ equal to f_0/γ_{M1}:



Effective cross-section under compression

$$N_{c,Rd} = \frac{A_{eff}\, f_0}{\gamma_{M1}}$$

where:

- **A_{eff}** is the effective area obtained by assuming a uniform distribution of stress equal to $\sigma_{com,Ed}$.
- **f_0** is the 0.2% proof strength

If the centroid of the effective cross-section does not coincide with the centroid of the gross cross-section, the shift e_N of the centroidal axes shall be taken into account, considering the effect of combined compression and bending.

2.3.5 RESISTANCE OF CROSS-SECTIONS – PART 1.4

Resistance under axial compression

Effective area A_{eff} is equal to the gross area A_g section with no reduction due to local and/or distortional buckling

$$A_{eff} = A_g$$

The design compression resistance shall be determined by considering the following equation

$$N_{c,Rd} = \frac{A_g \ f_0}{\gamma_{M1}}$$

where:

- A_g is the gross area
- f_0 is the 0.2% proof strength

2.3.5 RESISTANCE OF CROSS-SECTIONS – PART 1.4

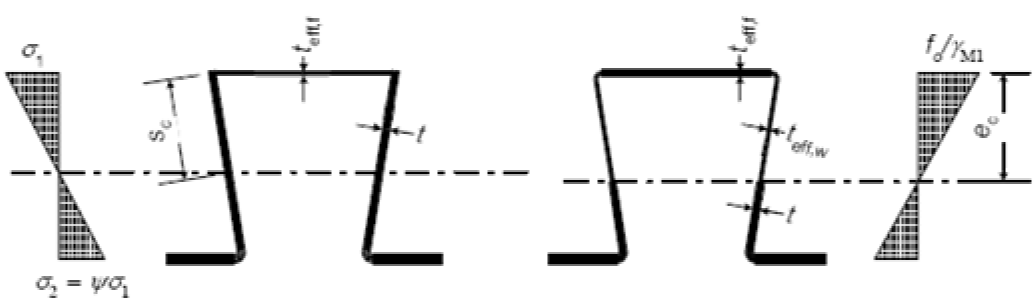
Resistance under bending moment

Elastic and elastic-plastic resistance with yielding at the compressed flange

If the effective section modulus W_{eff} is less than the gross elastic section modulus W_{el}

$$\mathbf{W_{eff} \leq W_{el}}$$

The design moment resistance of a cross-section for bending about a principal axis $M_{c,Rd}$ shall be determined by considering the effective area of the cross-section subjected to a linear stress distribution, with a maximum compressive stress $\sigma_{max,Ed}$ equal to f_0/γ_{M1}



Effective cross-section for resistance to bending moments

$$M_{c,Rd} = \frac{W_{eff}\ f_0}{\gamma_{M1}}$$

where:

- $\mathbf{W_{eff}}$ is the effective section modulus
- $\mathbf{f_0}$ is the 0.2% proof strength

2.3.5 RESISTANCE OF CROSS-SECTIONS – PART 1.4

Resistance under bending moment

Elastic and elastic-plastic resistance with yielding at the compressed flange

If the effective section modulus W_{eff} is equal to the gross elastic section modulus W_{el}

$$W_{eff} = W_{el}$$

The design moment resistance of a cross-section for bending about a principal axis $M_{c,Rd}$ shall be determined by considering the following equation:

$$M_{c,Rd} = \frac{W_{el}\ f_0}{\gamma_{M1}}$$

where:

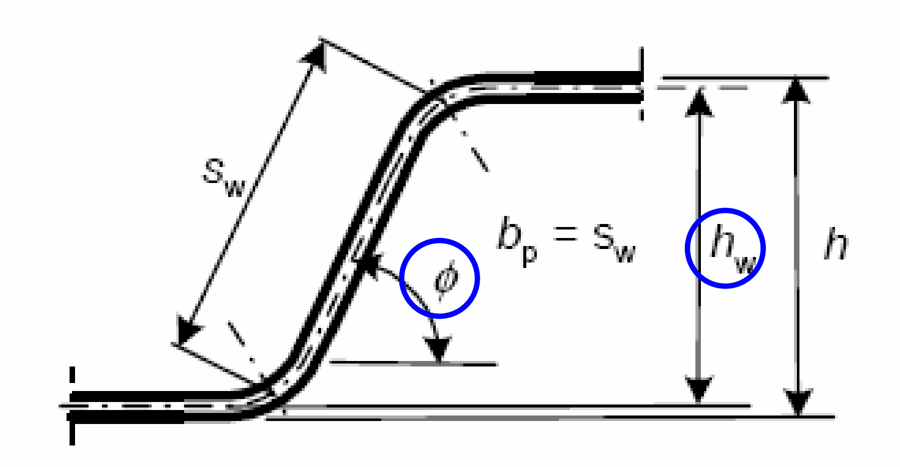
- W_{el} is the elastic section modulus
- f_0 is the 0.2% proof strength

2.3.5 RESISTANCE OF CROSS-SECTIONS – PART 1.4

Resistance under shear

The shear resistance $V_{b,Rd}$ should be determined from:

$$V_{b,Rd} = \frac{\frac{h_w}{\sin\phi} t \cdot f_{bV}}{\gamma_{M1}}$$



where:

- f_{bv} is the shear strenght considering buckling
- h_w is the web height between the midlines of the flanges
- ϕ is the slope of the web relative to the flanges

2.3.5 RESISTANCE OF CROSS-SECTIONS – PART 1.4

Resistance under shear

The shear buckling strength f_{bv} is given as function of:

- relative web slenderness λ_w
- web stiffening

Table 6.1 - Shear buckling strength f_{bv} in relation to web slenderness parameter $\bar{\lambda}_w$

Web slenderness parameter	Web without stiffening at the support	Web with stiffening at the support [1)]
$\bar{\lambda}_w \geq 0{,}83$	$0{,}58\, f_o$	$0{,}58\, f_o$
$0{,}83 < \bar{\lambda}_w \leq 1{,}40$	$0{,}48\, f_o / \bar{\lambda}_w$	$0{,}48\, f_o / \bar{\lambda}_w$
$\bar{\lambda}_w \geq 1{,}40$	$0{,}67\, f_o / \bar{\lambda}_w^2$	$0{,}48\, f_o / \bar{\lambda}_w$
1) Stiffening at the support, such as cleats, arranged to prevent distortion of the web and designed to resist the support reaction.		

Shear buckling strength

2.3.5 RESISTANCE OF CROSS-SECTIONS – PART 1.4

Resistance under shear – Web slenderness

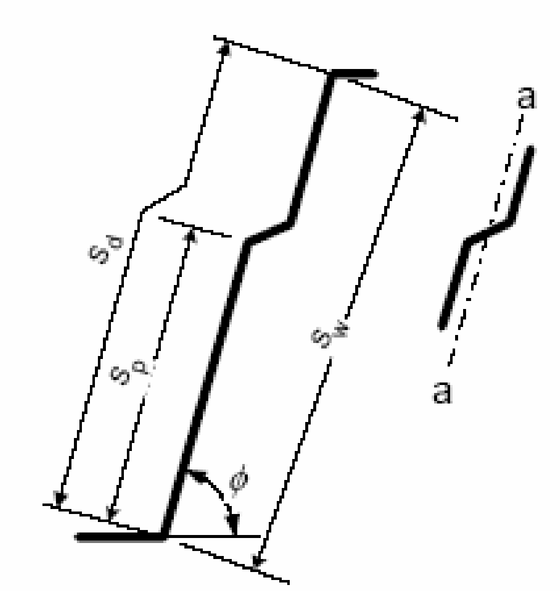
For webs without longitudinal stiffeners:

$$\overline{\lambda}_w = 0.346 \frac{s_w}{t} \sqrt{f_0 / E}$$

For webs with longitudinal stiffeners:

$$\overline{\lambda}_w = 0.346 \frac{s_d}{t} \sqrt{5.34 f_0 / k_\tau E} \geq 0.346 \frac{s_p}{t} \sqrt{f_0 / E}$$

$$k_\tau = 5.34 + \frac{2.10}{t} \left(\frac{\Sigma I_s}{s_d} \right)^{1/3}$$



Longitudinally stiffened web

where:

- **I_s** is the second moment of area of the individual longitudinal stiffener about the axis a–a
- **s_d** is the total developed slant height of the web
- **s_p** is the slant height of the largest plane element in the web
- **s_w** is the slant height of the web between the midpoints of the corners

2.3.5 RESISTANCE OF CROSS-SECTIONS – PART 1.4

Combined axial tension force and bending

According to Part 1.4 of Eurocode 9, a cross-section subject to combined axial tension force N_{Ed} and bending moment $M_{y,Ed}$ shall satisfy the criterion:

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

where:

- **$N_{t,Rd}$** is the design resistance of a cross-section for uniform tension;
- **$M_{cy,Rd,ten}$** is the design moment resistance of a cross-section for maximum tensile stress if subject only to moment about the y - y axes.

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

$$\frac{M_{y,Ed}}{M_{cy,Rd,com}} - \frac{N_{Ed}}{N_{t,Rd}} \leq 1$$

where:

- **$M_{cy,Rd,com}$** is the moment resistance of the maximum compressive stress in a cross-section that is subject to moment only

2.3.5 RESISTANCE OF CROSS-SECTIONS – PART 1.4

Combined axial compression force and bending

According to Part 1.4 of Eurocode 9, a cross-section subject to combined axial compression force N_{Ed} and bending moment $M_{y,Ed}$ shall satisfy the criterion:

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

where:

- **N_{cRd}** is the design resistance of a cross-section for uniform compression;
- **$M_{cy,Rd,ten}$** is the moment resistance maximum compressive stress in a cross-section that is subject to moment only

The additional moments due to the shifts of the centroidal axes shall be taken into account.

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

$$\frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{cy,Rd,ten}} - \frac{N_{Ed}}{N_{c,Rd}} \leq 1$$

where:

- **$M_{cy,Rd,ten}$** is the design moment resistance of a cross-section for maximum tensile stress if subject only to moment about the y - y axes.

2.3.5 RESISTANCE OF CROSS-SECTIONS – PART 1.4

Combined shear force, axial force and bending moment

Cross-sections subject to the combined action of an axial force N_{Ed}, a bending moment M_{Ed} and a shear force V_{Ed} following equation should be satisfied:

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

where:

- **N_{Rd}** is the design resistance of a cross-section for uniform tension or compression
- **$M_{y,Rd}$** is the design moment resistance of the cross-section
- **$V_{w,Rd}$** is the design shear resistance of the web given
- **$M_{f,Rd}$** is the design plastic moment resistance of a cross-section consisting only of flanges
- **$M_{pl,Rd}$** is the plastic moment resistance of the cross-section

For members and sheeting with more than one web $V_{w,Rd}$ is the sum of the resistances of the webs

2.3.6 BUCKLING RESISTANCE– PART 1.4

Flexural buckling

The effects of local buckling are taken into account by using effective section properties. The design buckling resistance for axial compression $N_{b,Rd}$ shall therefore be obtained from:

$$N_{b,Rd} = \frac{\chi \, A_{eff} \, f_0}{\gamma_{M1}}$$

where:

- **A_{eff}** is the effective area obtained by assuming a uniform distribution of stress $\sigma_{com,Ed}$ equal to f_0 / γ_{M1}
- **f_0** is the 0.2% proof strength
- χ is the appropriate value of the reduction factor for buckling resistance, obtained in function of the relative slenderness for the relevant buckling mode and of the imperfection factors α and:

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 + \bar{\lambda}^2}} \quad \text{but} \quad \chi \leq 1 \qquad \phi = 0.5\,[1 + \alpha\,(\bar{\lambda} - \bar{\lambda}_0) + \bar{\lambda}^2]$$

2.3.6 BUCKLING RESISTANCE– PART 1.4

Flexural buckling

$\bar{\lambda}$ is the relative slenderness for flexural buckling about a given axis, determined as

$$\bar{\lambda} = \frac{\lambda}{\lambda_1}\sqrt{\frac{A_{eff}}{A_g}}$$

in which:

$$\lambda = L / i \qquad \lambda_1 = \pi\sqrt{E / f_{0.2}}$$

with:

- **L** buckling length for flexural buckling about the relevant axis;
- **i** radius of gyration about the corresponding axis, based on the properties of the gross section.

2.3.6 BUCKLING RESISTANCE– PART 1.4

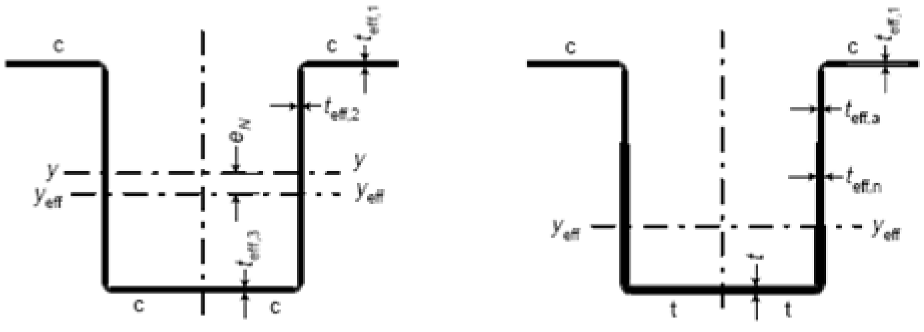
Bending and axial compression

According to Part 1.1 of Eurocode 9, all members subject to combined bending and axial compression shall satisfy the criterion:

$$\frac{N_{Ed}}{\chi_y f_0 \omega_x A_{eff} / \gamma_{M1}} + \frac{M_{y,Ed} + \Delta M_{y,Ed}}{f_0 W_{eff,y,com} / \gamma_{M1}} \leq 1$$

where:

- A_{eff} is the effective area of an effective cross-section that is subject only to axial compression;
- $W_{eff,y,com}$ is the effective section modulus for the maximum compressive stress in an effective cross-section that is subject only to moment about the y-y axis
- $\Delta M_{y,Ed}$ is the additional moment due to possible shift of the centroidal axis in the y direction
- χ_y is the reduction factor from for buckling about the y-y axis;
- ω_z is an interaction expression



(a) Axial compression (b) moment about y – y axis.

Model for calculation of effective section properties

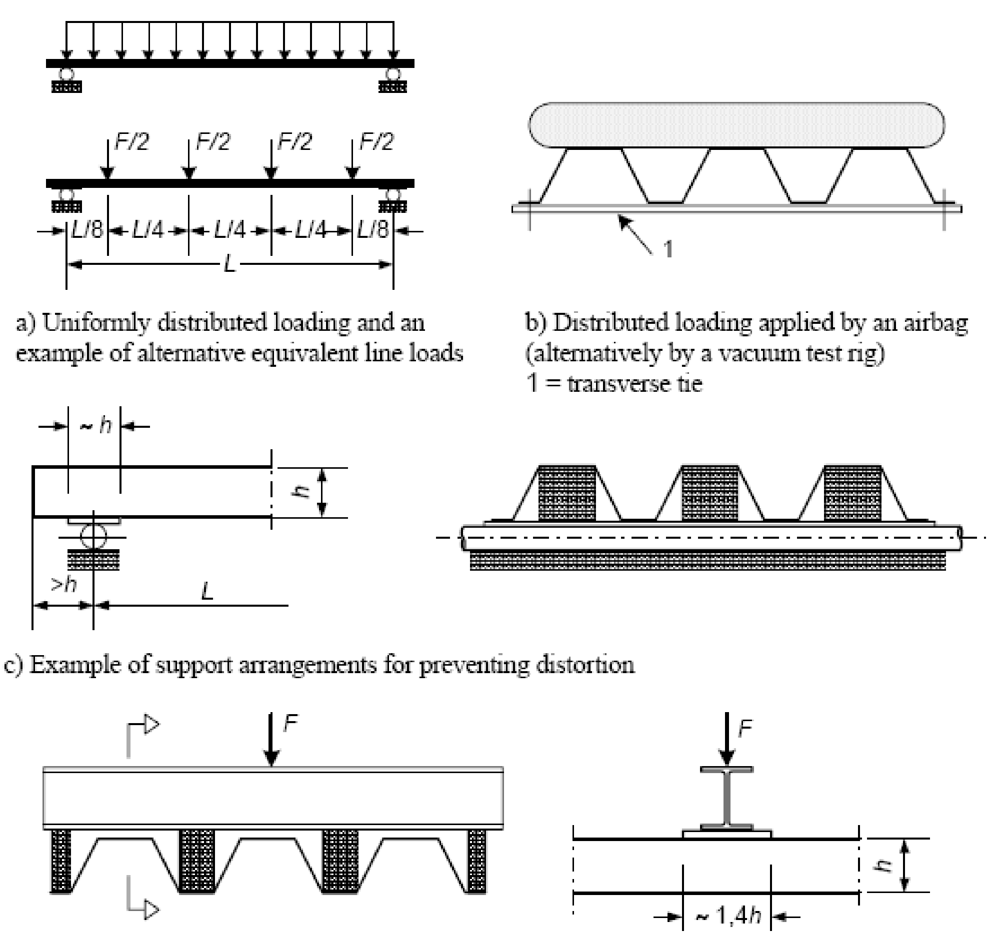
$$\varpi_x = \frac{1}{\chi_y + (1-\chi_y)\sin\pi \cdot x_s / l_c}$$

- X_s is the distance from the studied section to a hinged support or a point of contra-flexure of the deflection curve for elastic buckling of an axial force only
- $l_c = K_L$ is the buckling length

NOTE: For simplification ω_x = 1 may be used

2.3.7 DESIGN ASSISTED BY TESTING

Annex A [normative] – Testing procedures



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)
1 = transverse tie

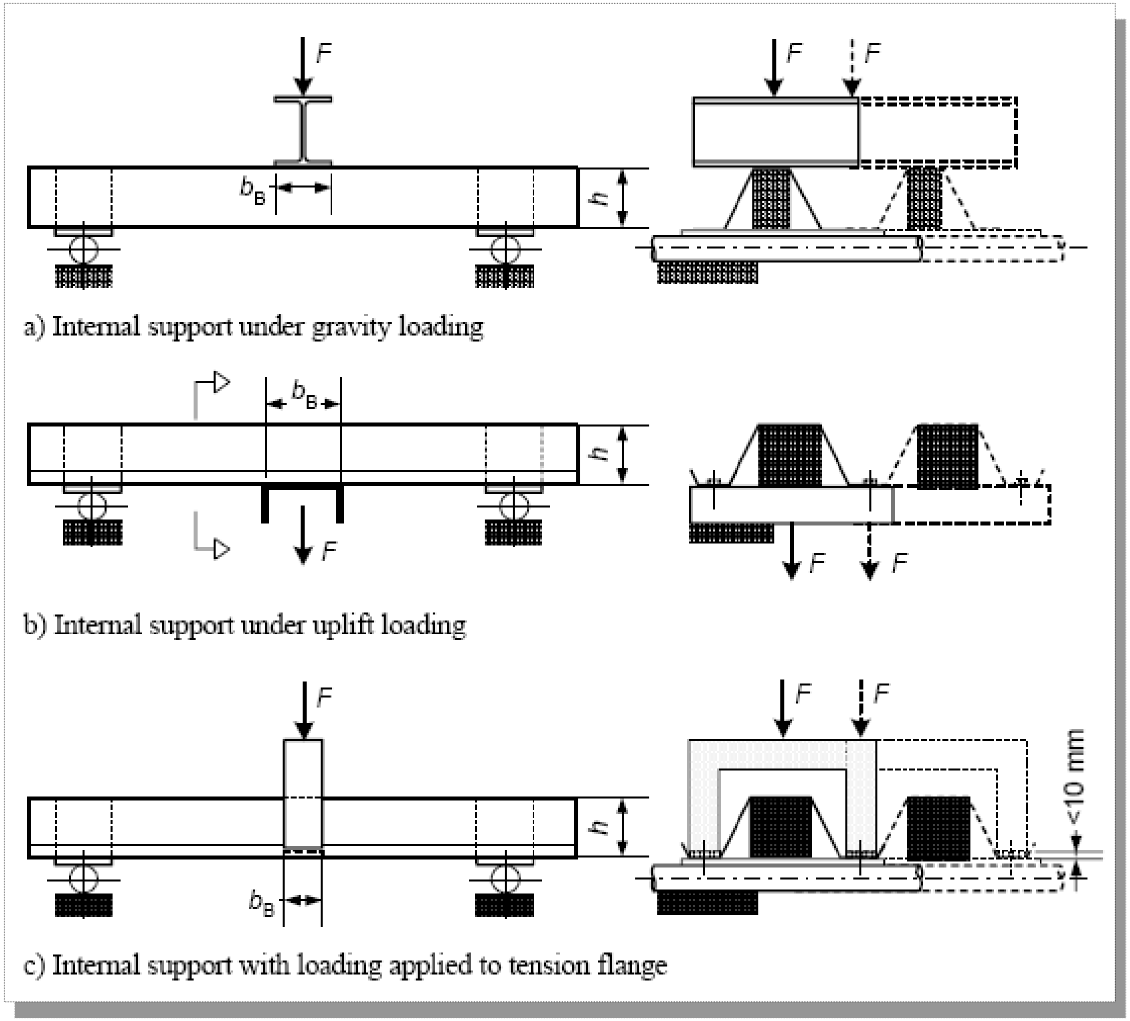
c) Example of support arrangements for preventing distortion

d) Example of method of applying a line load

Test set-up for single span test

2.3.7 DESIGN ASSISTED BY TESTING

Annex A [normative] – Testing procedures



a) Internal support under gravity loading

b) Internal support under uplift loading

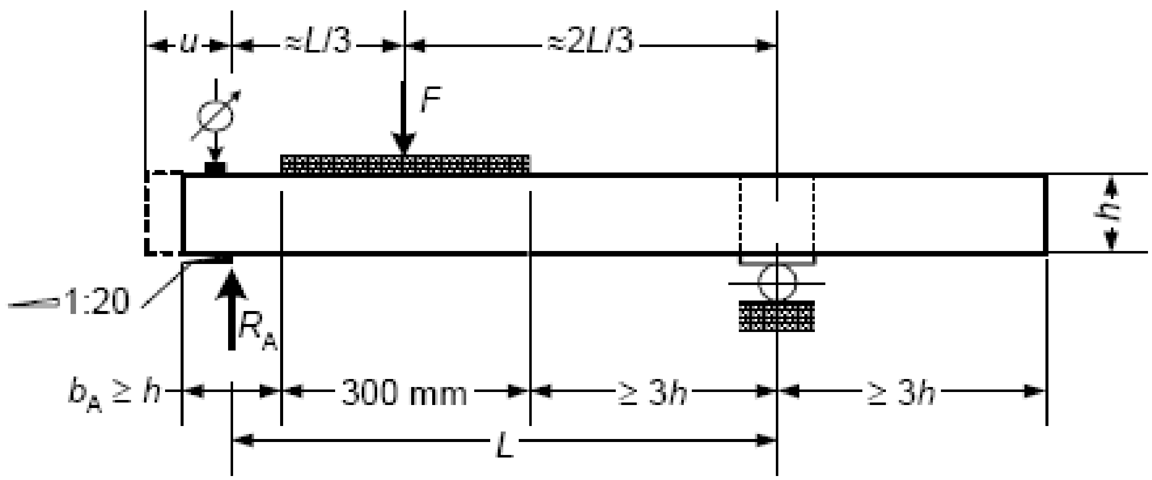
c) Internal support with loading applied to tension flange

Test set-up for internal support test

Cold-Formed Structures (Part 1.4)

2.3.7 DESIGN ASSISTED BY TESTING

Annex A [normative] – Testing procedures



Key:
b_A = support length
u = length from internal edge of end support to end of sheet

Test set-up for end support test

CONCLUSIONS

- In the Eurocode 9, Part 1.1 (EN 1999-1-1) provides all the calculation methods dealing with slender section (class 4), which cover thin-walled aluminium sections. More specific provisions for cold-formed thin-walled sheeting are given in Part 1.4 (EN 1999-1-4).

- The framing of the Eurocode 9 Part 1.4 is similar to that of the Eurocode 3 Part 1.1. and some specific issues are treated in a similar way (i.e. influence of rounded corners, effectiveness of the restraint provided by the stiffeners).

NEW

INNOVATIVE ISSUES

- The local buckling effect in the CF thin-gauge members is taken into account by means of a calculation method based on the effective thickness concept.

- Three specific buckling curves proposed by Landolfo and Mazzolani for aluminium slender sections are given in Part 1.1.

Thanks for attention