European Commission

Eurocodes

Background and Applications

Design of **Steel Buildings**

with worked examples

16-17 October 2014
Brussels, Belgium

Cold-formed Steel Design

EUROCODE 3: Design of Steel Structures

PART 1-3 – Design of Cold-formed Steel Structures

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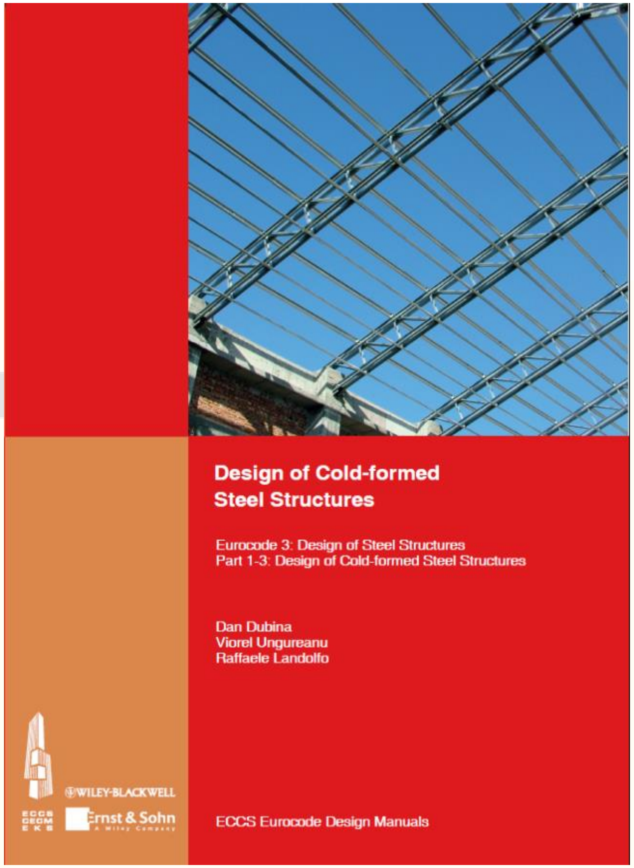
European Committee for Standardization
CEN/TC250/SC3

Joint Research Centre

Cold Formed Steel Design

Contents

- Introduction
- Peculiar characteristics
- Resistance of Sections
- Resistance of Members
- Conceptual Design (case study)



Background and peculiarities

Introduction

General

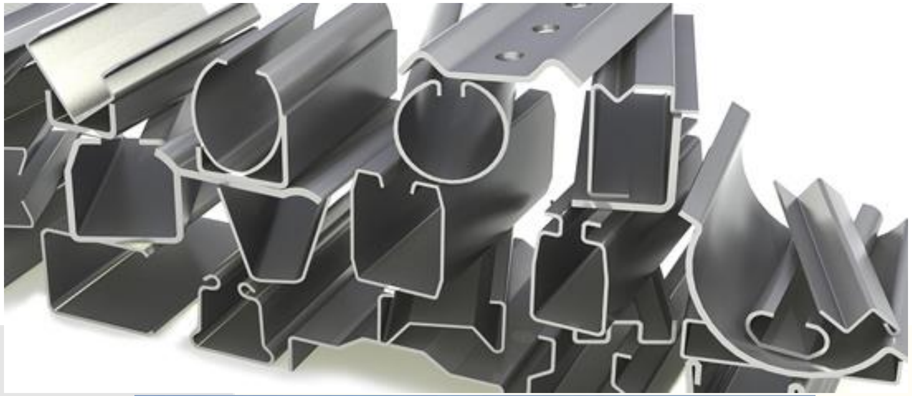
Collection of different cold-formed steel sections shapes (Trebilcock, 1994)

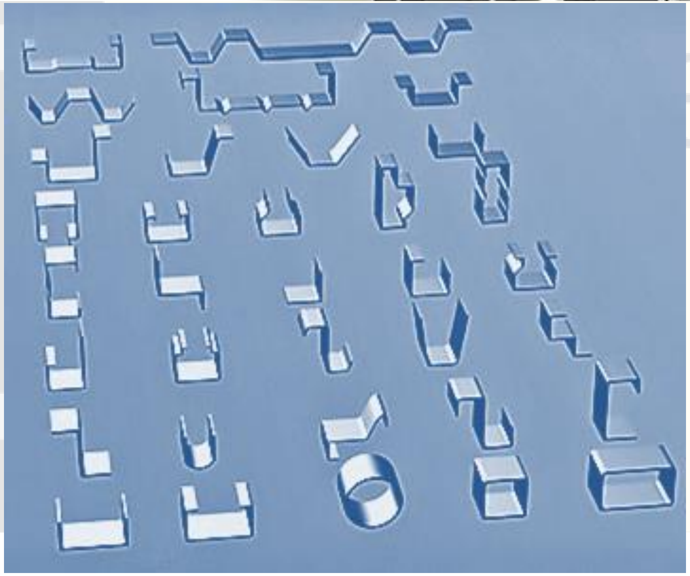
Background and peculiarities

Introduction

General

Collection of different cold-formed steel sections shapes (Trebilcock, 1994)





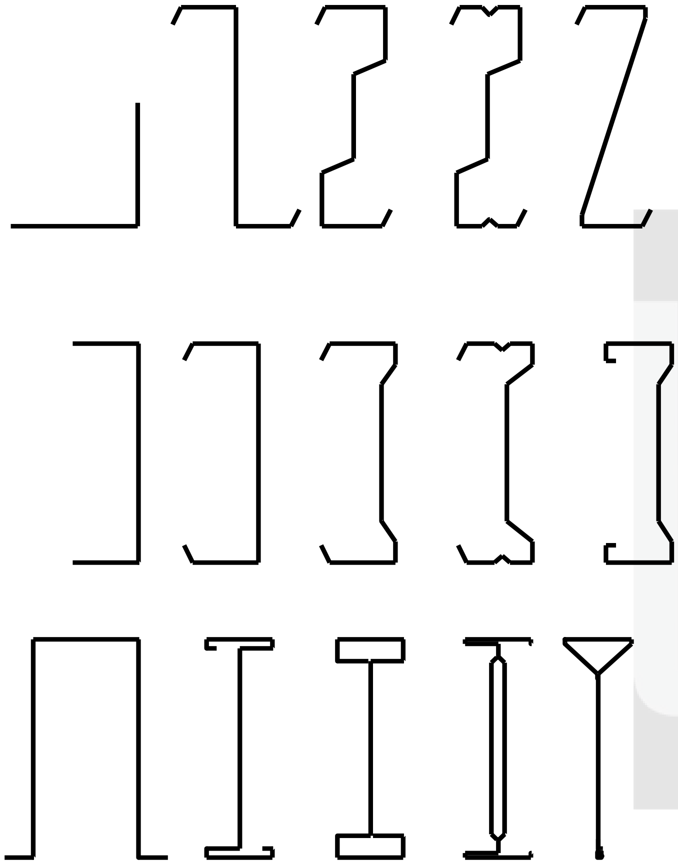
Collection of different cold-formed steel sections shapes

Background and peculiarities

Introduction

Types of cold-formed steel sections

- Typical forms of sections for cold-formed structural members



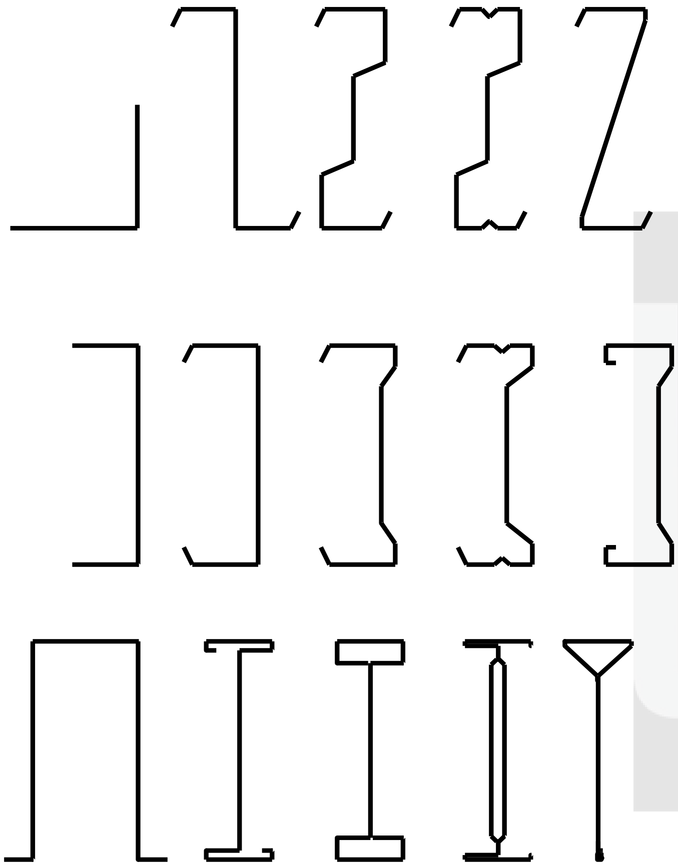
Single open sections

Background and peculiarities

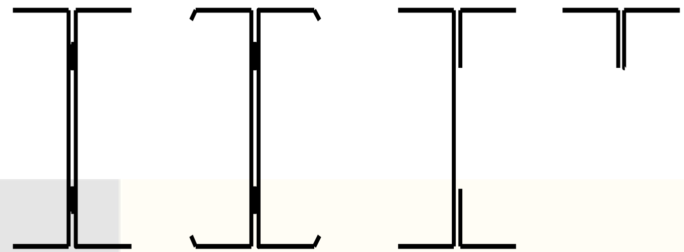
Introduction

Types of cold-formed steel sections

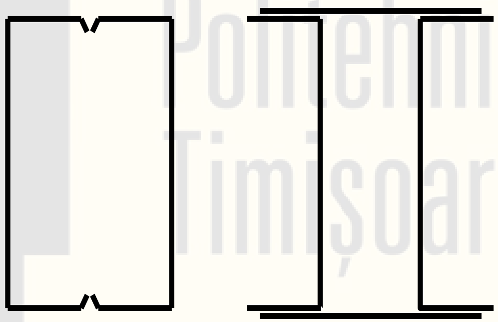
- Typical forms of sections for cold-formed structural members



Single open sections



Open built-up sections



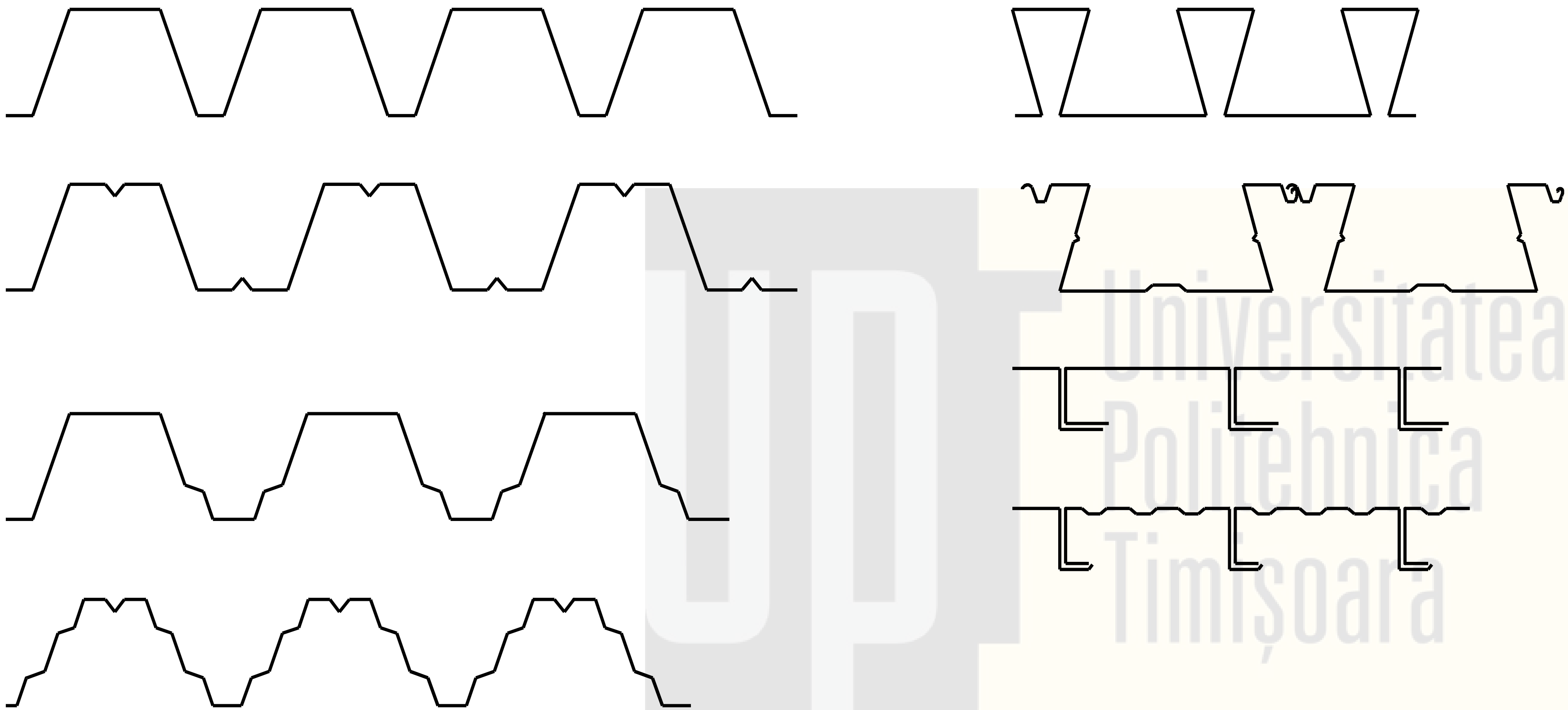
Closed built-up sections

Background and peculiarities

Introduction

Types of cold-formed steel sections

- **Profiled sheets and linear trays sections**



Introduction

Advantages

- **Advantages of Using Cold-Formed Steel Sections**
  - Lightness;
  - High strength and stiffness;
  - Ability to provide long spans;
  - Easy prefabrication and mass production;
  - Fast and easy erection and installation;
  - Substantial elimination of delay due to the weather;
  - More accurate detailing;
  - Non-shrinking and non-creeping at ambient temperatures;
  - Form work unneeded;
  - Termite-proof and rat-proof;
  - Uniform quality;
  - Economy in transportation and handling;
  - Non combustibility;
  - Recyclable material.

Background and peculiarities

Introduction

Manufacturing technologies

- Roll forming;
- Folding;
- Press braking

Background and peculiarities

Introduction

Manufacturing technologies

- Roll forming;
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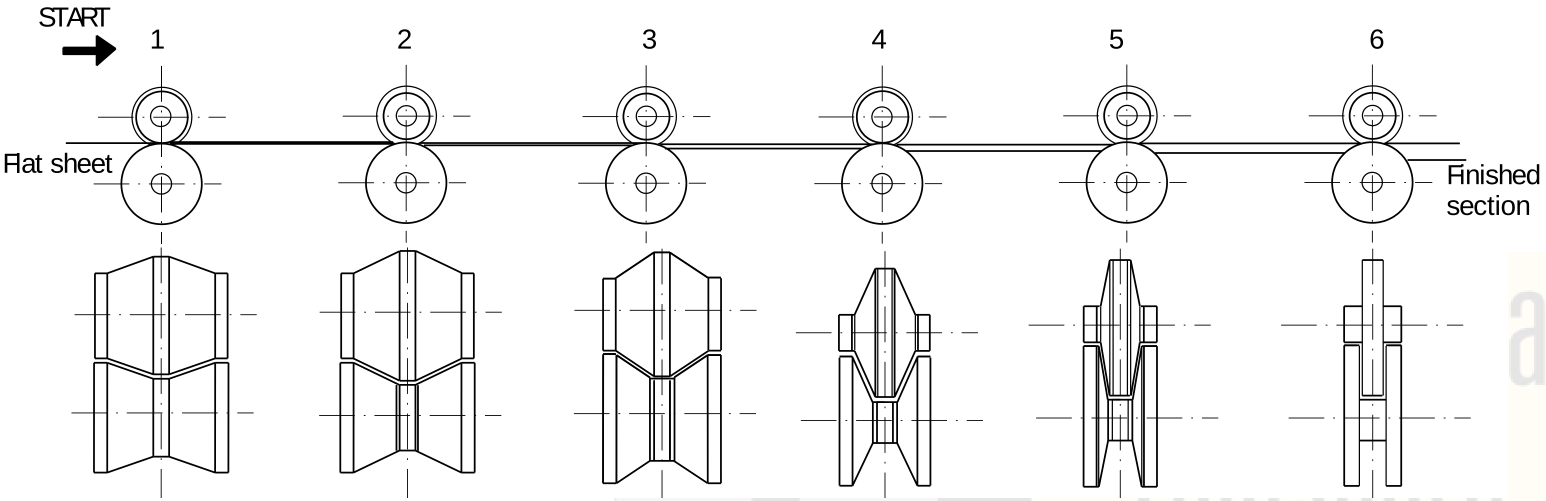

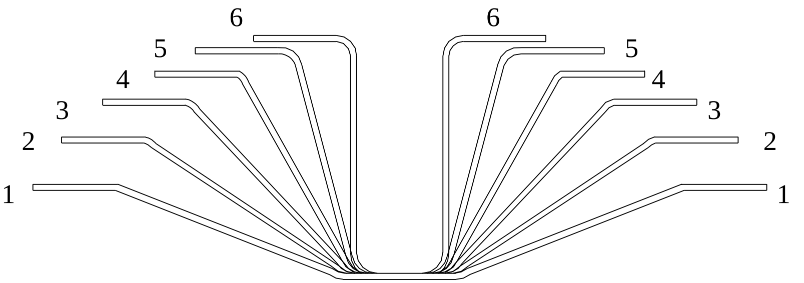

Background and peculiarities

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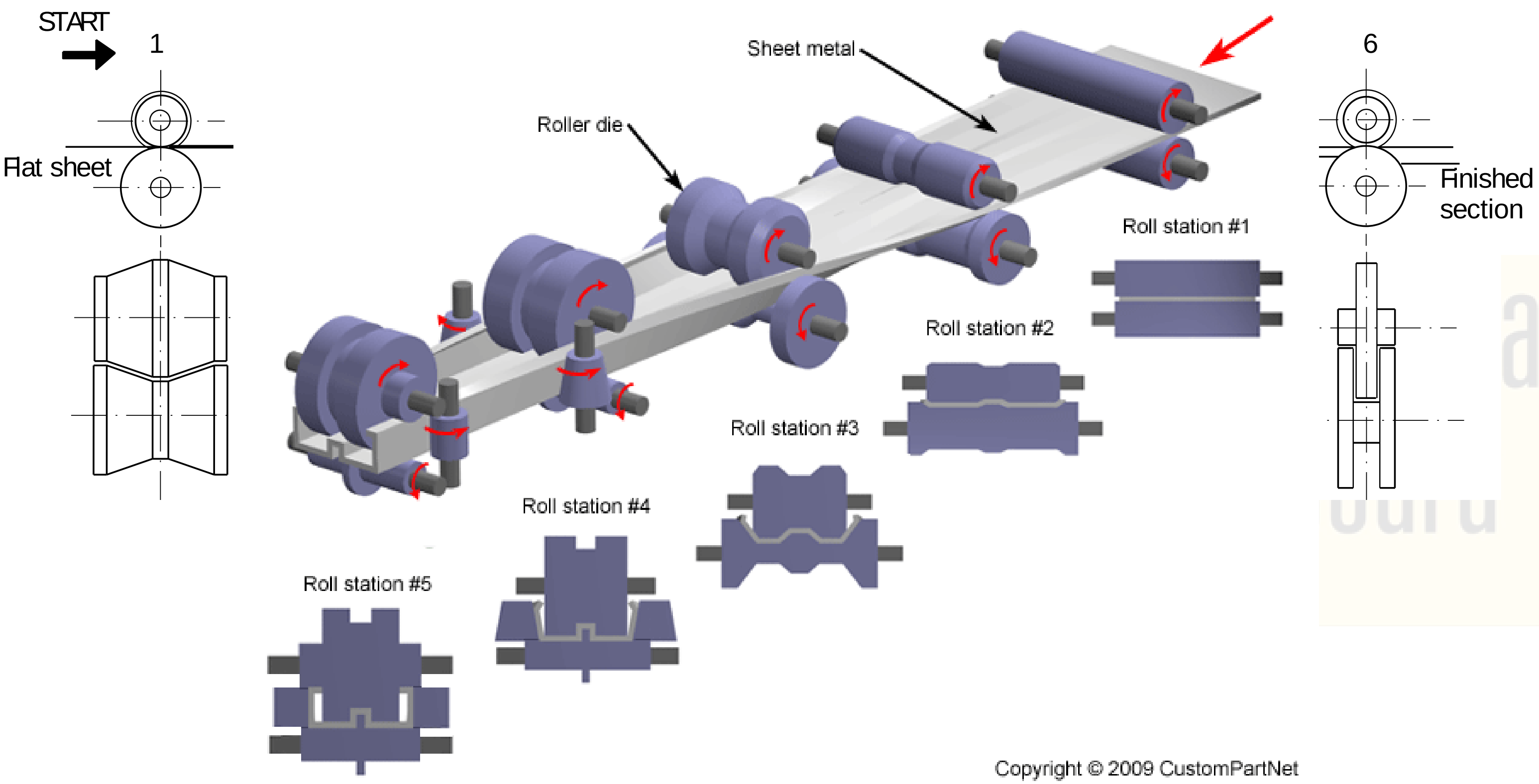

Background and peculiarities

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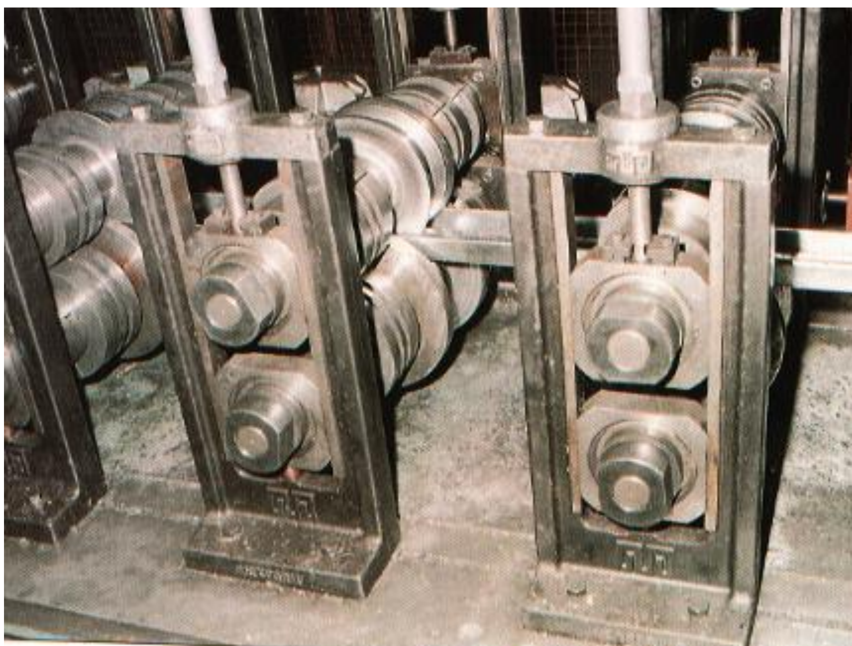

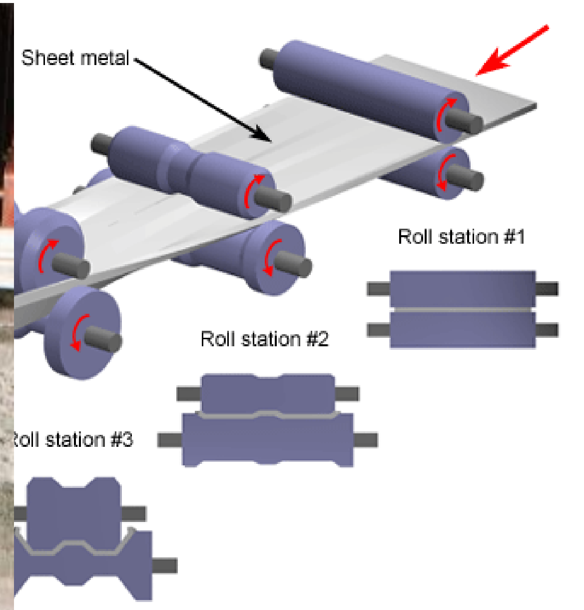

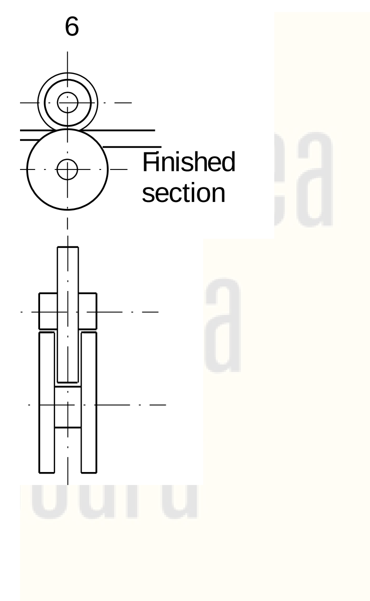

Background and peculiarities

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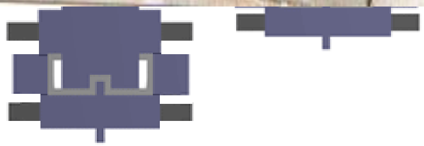

Background and peculiarities

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Manufacturing technologies

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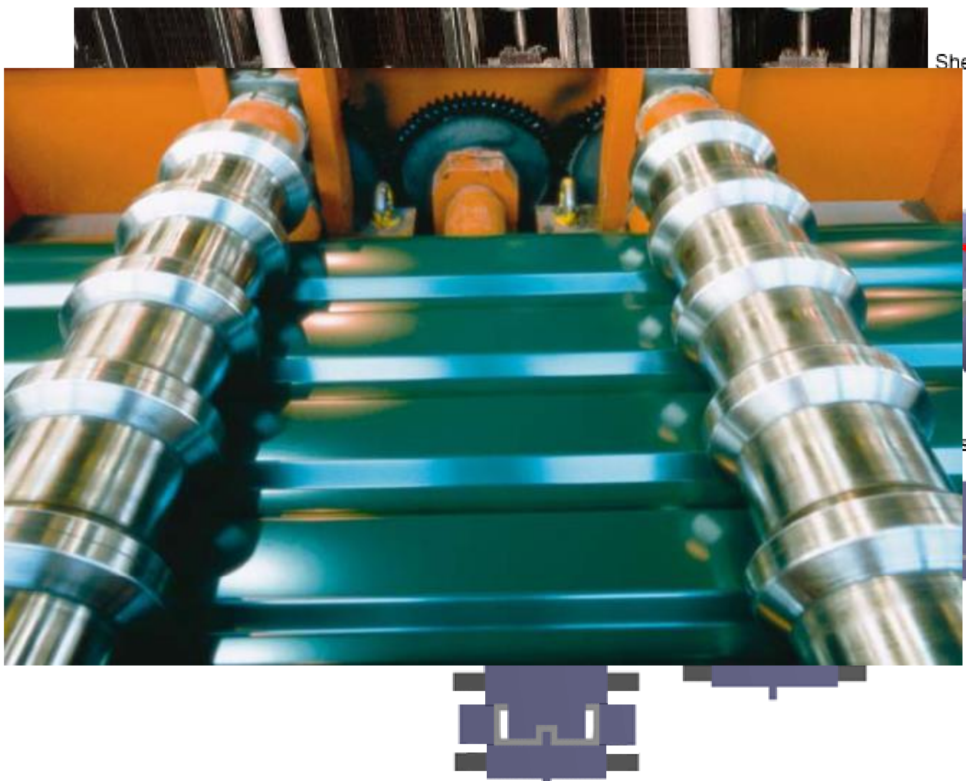

Background and peculiarities

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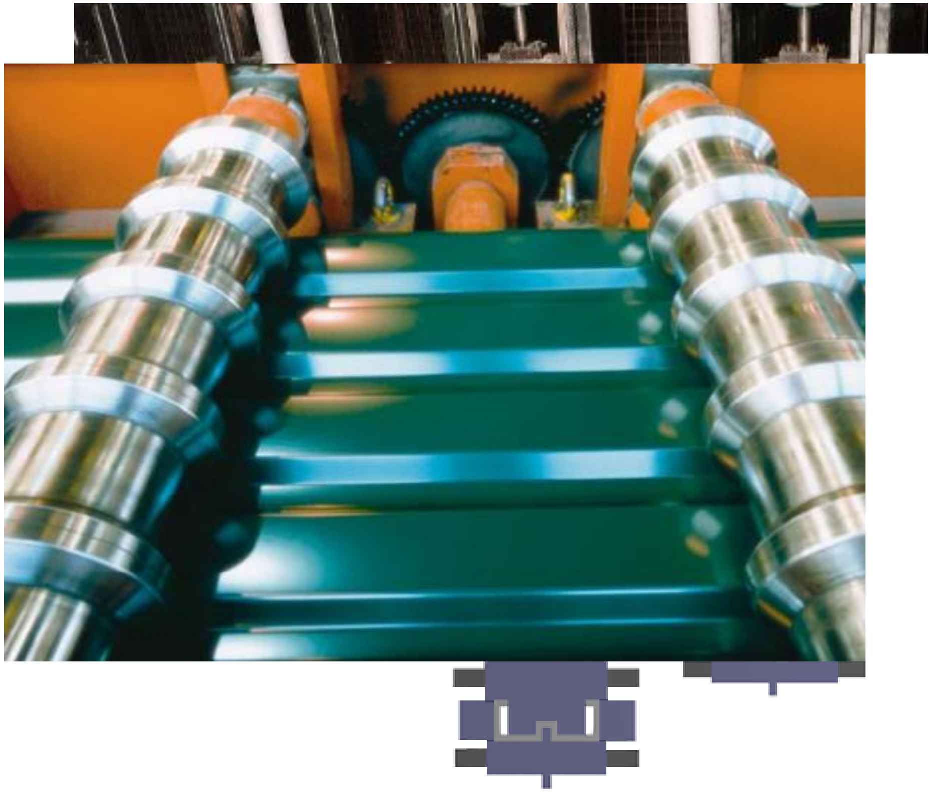

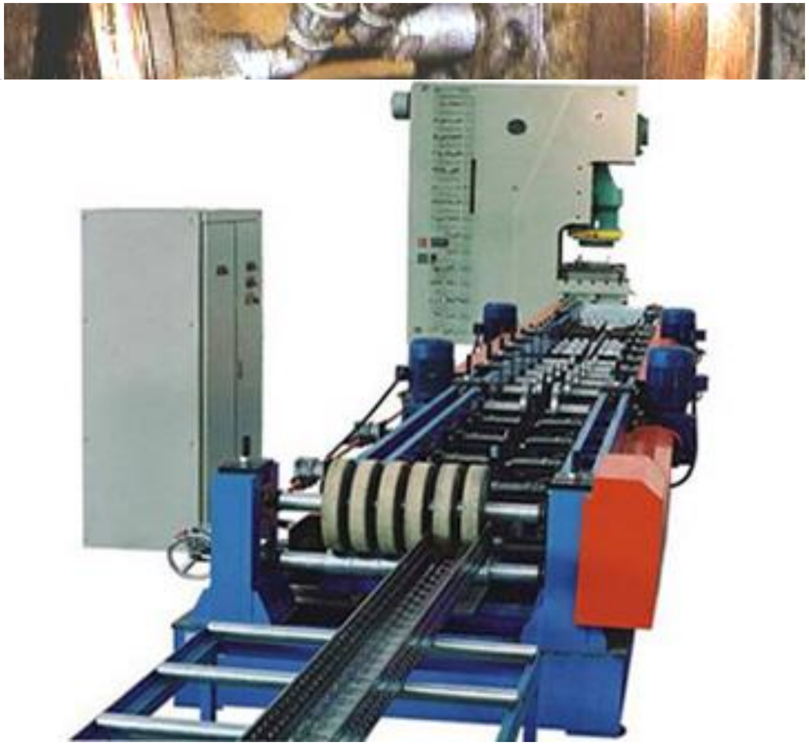

Background and peculiarities

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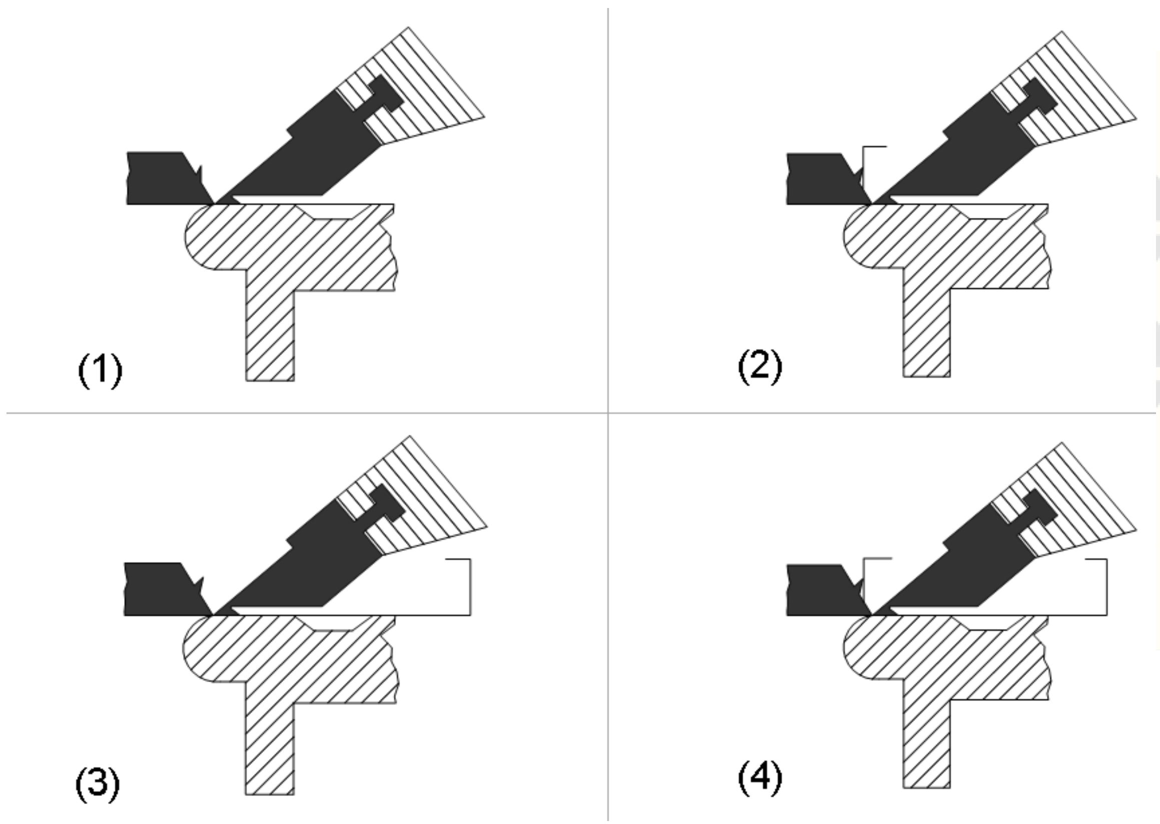

Background and peculiarities

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Manufacturing technologies

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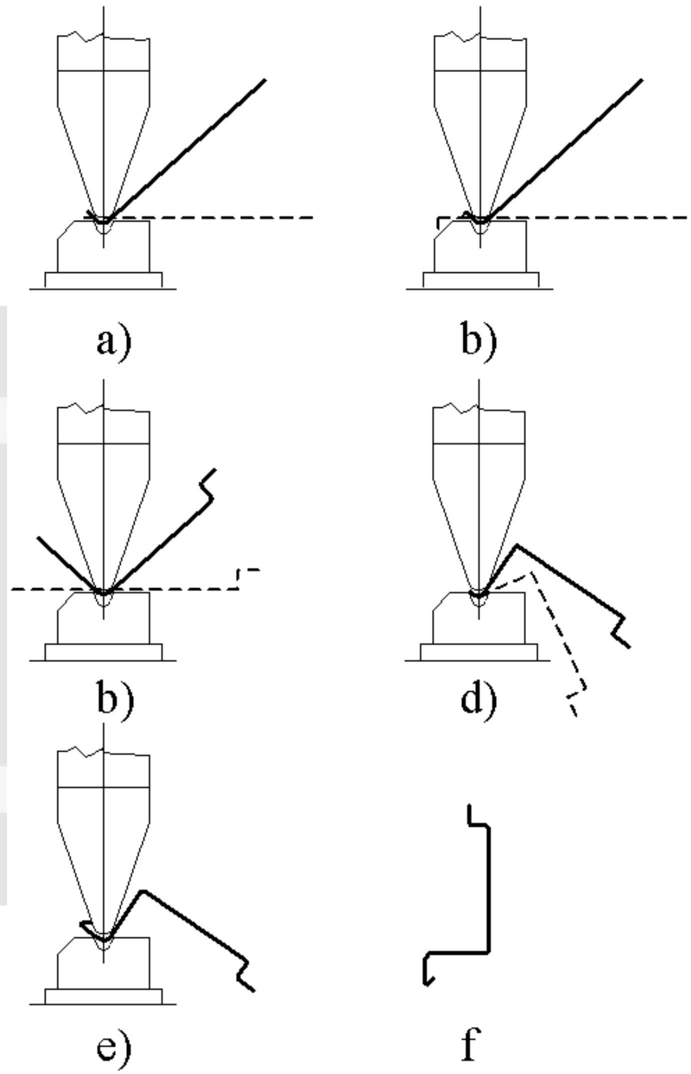


Background and peculiarities

Introduction

Peculiar Characteristics of Cold-Formed Steel Sections

- Imperfections in Thin-Walled Cold-Formed Steel Members

IMPERFECTIONS → mecanical

mecanical → Loading eccentricities

mecanical → Support eccentricities

Background and peculiarities

Introduction

Peculiar Characteristics of Cold-Formed Steel Sections

- Imperfections in Thin-Walled Cold-Formed Steel Members

IMPERFECTIONS

- mecanical
  - Loading eccentricities
  - Support eccentricities
- material
  - Forming process
    - Yielding strength
    - Residual stresses

Background and peculiarities

Introduction

Peculiar Characteristics of Cold-Formed Steel Sections

- Imperfections in Thin-Walled Cold-Formed Steel Members

IMPERFECTIONS

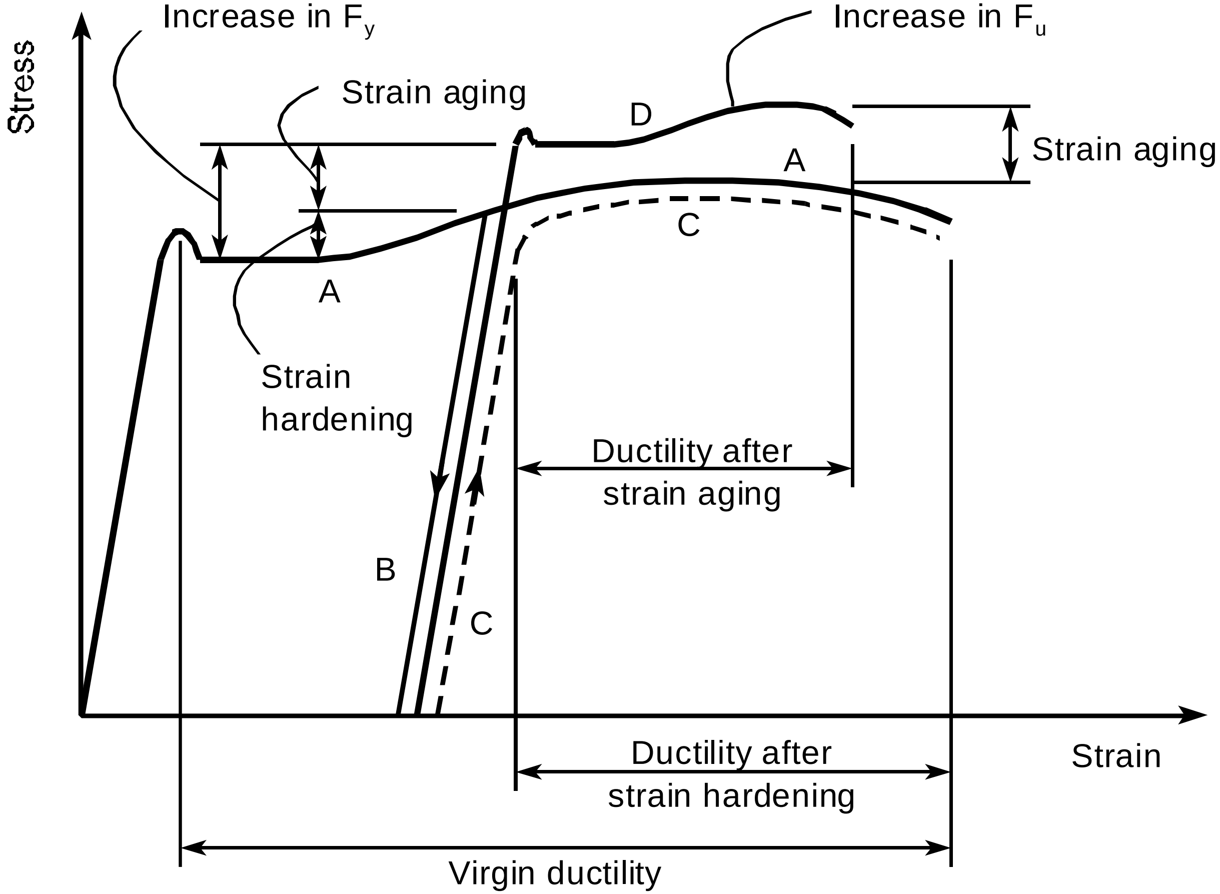
- mecanical
  - Loading eccentricities
  - Support eccentricities
- material
  - Forming process
    - Yielding strength
    - Residual stresses
- geometric
  - Local (sectional)
  - Global (member)

Background and peculiarities

Introduction

Peculiar Characteristics of Cold-Formed Steel Sections

- **Effect of Strain Hardening and Strain Aging on Stress-Strain Characteristics**



Background and peculiarities

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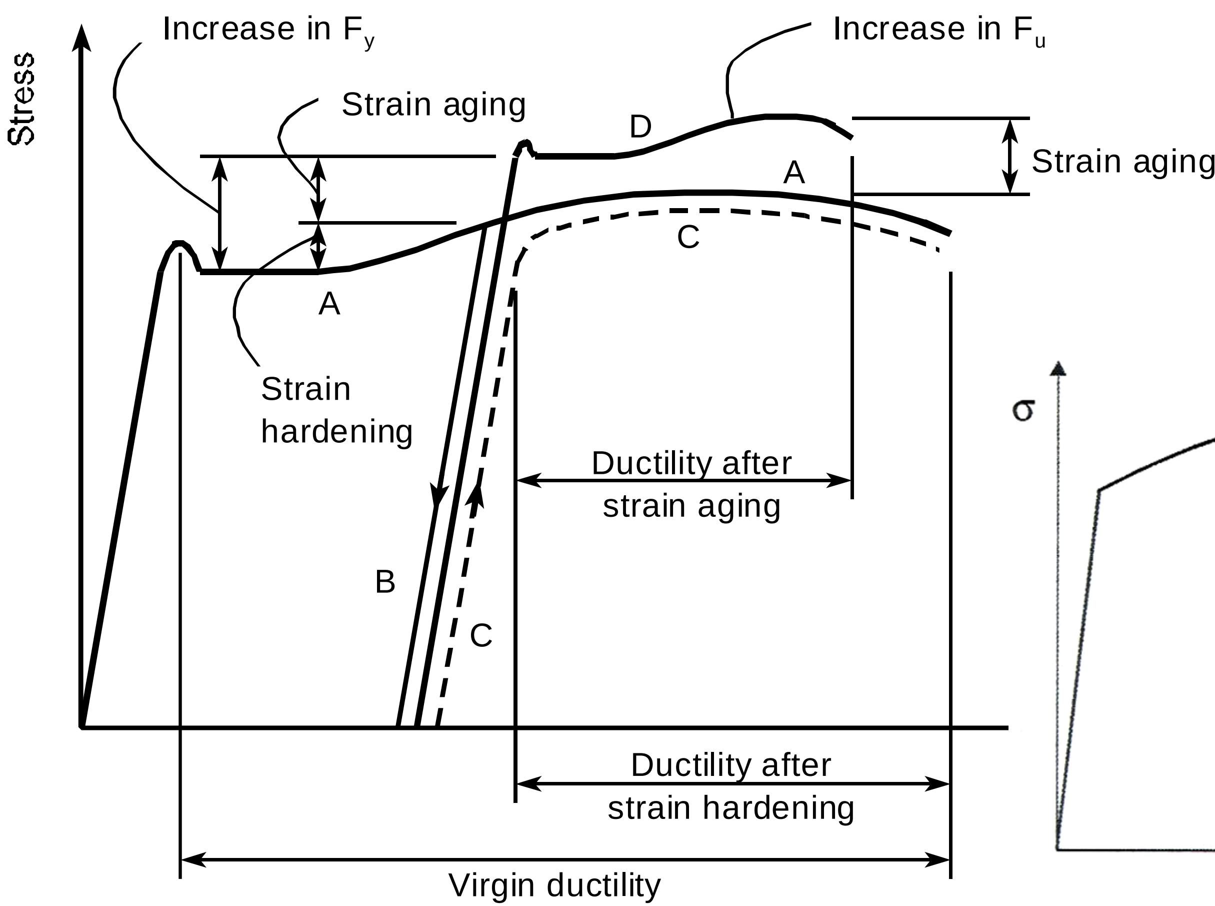
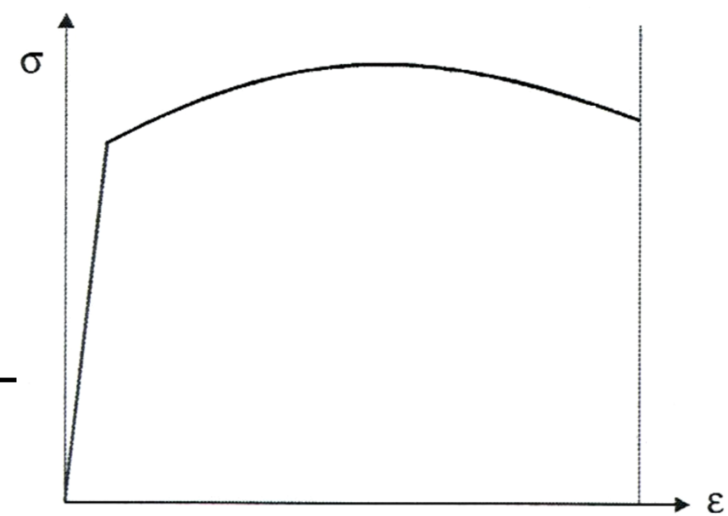
Peculiar Characteristics of Cold-Formed Steel Sections

- **Effect of Strain Hardening and Strain Aging on Stress-Strain Characteristics**

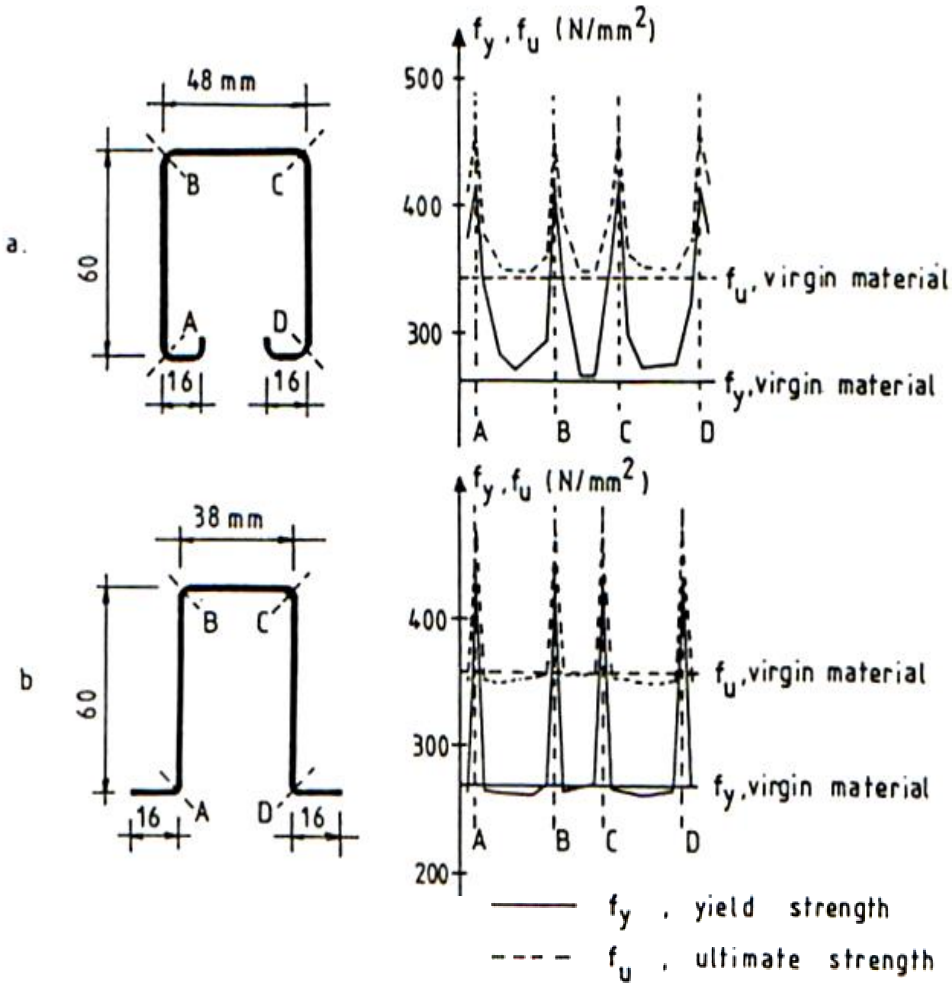




Introduction

Peculiar Characteristics of Cold-Formed Steel Sections

- Increase of the Yield Strength and Ultimate Strength Due to Cold-Forming



Background and peculiarities

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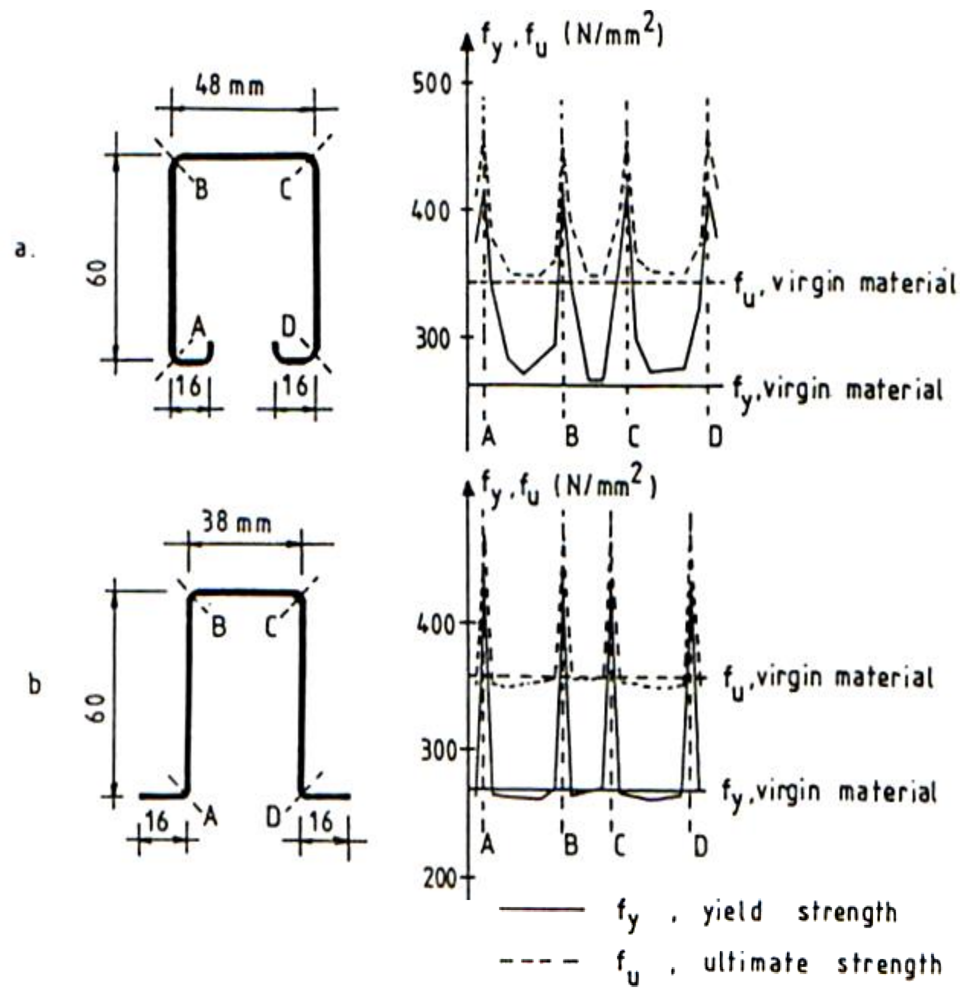
Peculiar Characteristics of Cold-Formed Steel Sections

- Increase of the Yield Strength and Ultimate Strength Due to Cold-Forming



Forming method	Cold rolling	
	Corner	Flange
Yield strength f_y	↑	↗
Ultimate strength f_u	↑	↗

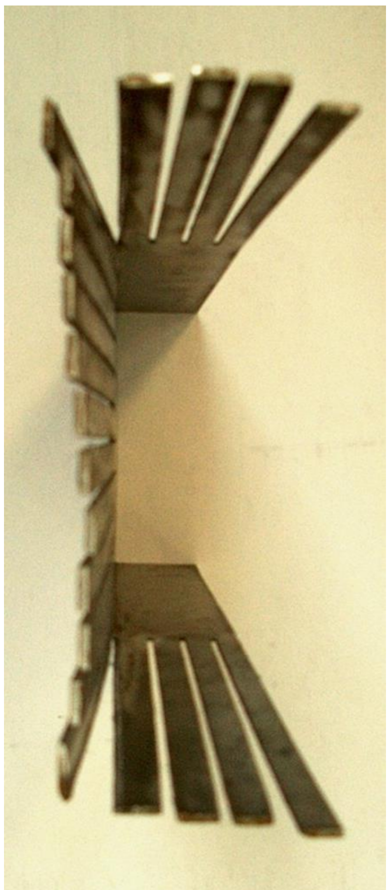
Forming method	Press bracking	
	Corner	Flange
Yield strength f_y	↑	-
Ultimate strength f_u	↑	-

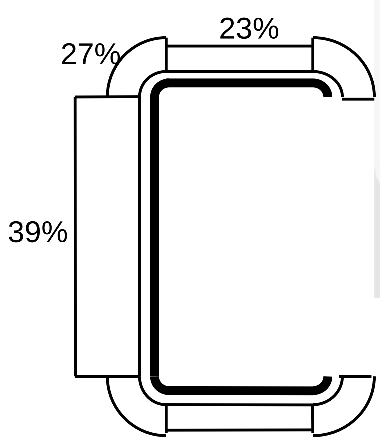
Background and peculiarities

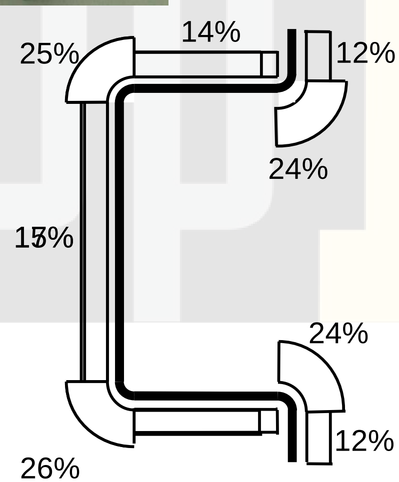
Introduction

Peculiar Characteristics of Cold-Formed Steel Sections

- Flexural Residual Stresses Obtained at the "POLITEHNICA" University of Timisoara







Background and peculiarities

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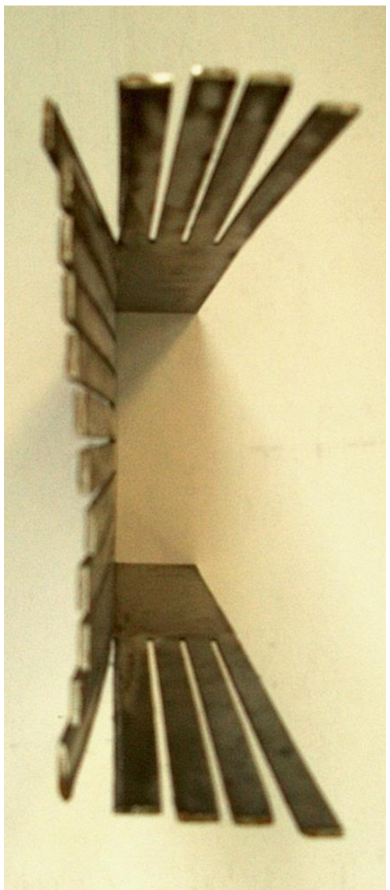
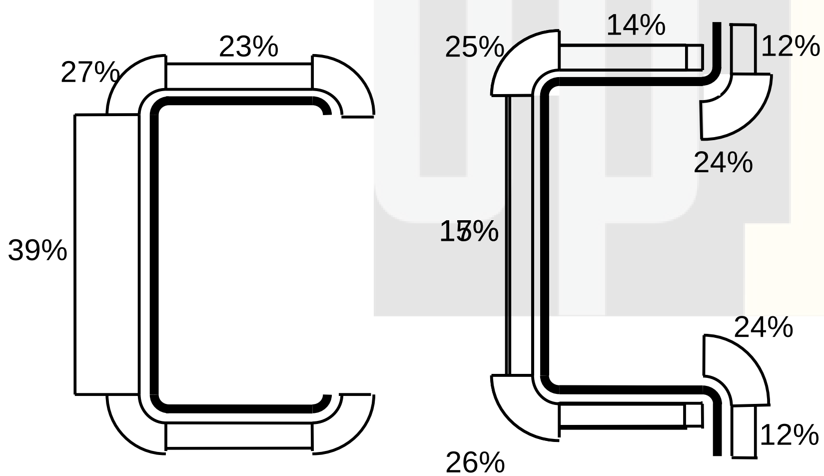
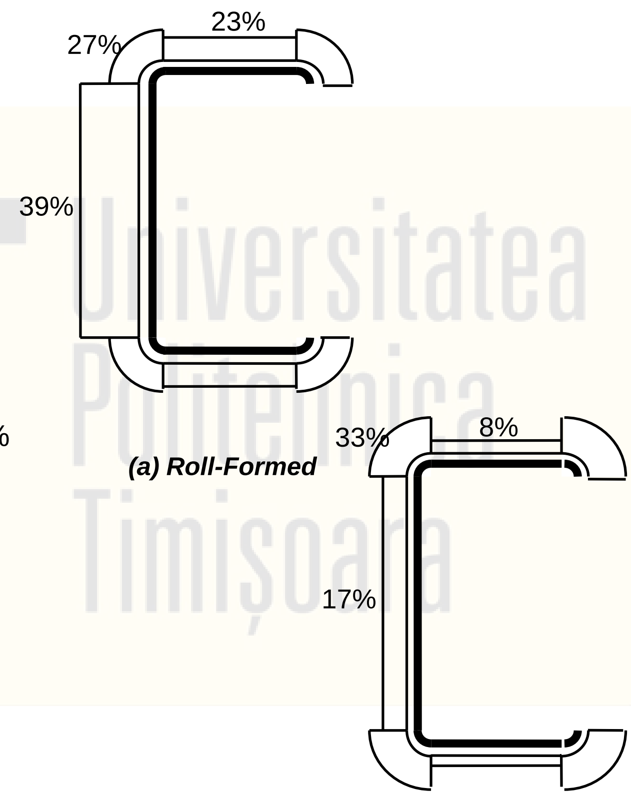




Schafer and Pekoz



(b) Press-Braked

Introduction

Peculiar Characteristics of Cold-Formed Steel Sections

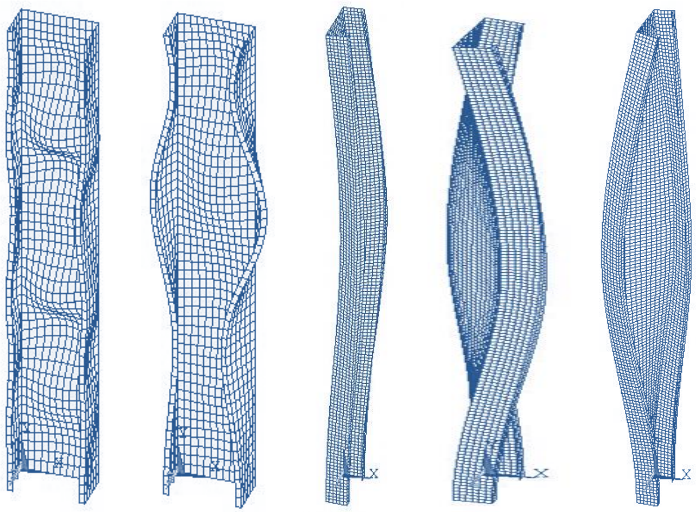
- Type and Magnitude of Residual Stress In Steel Sections

Forming method	Hot rolling	Cold forming	
		Cold rolling	Press braking
Membrane residual stresses (s_{rm})	high	low	low
Flexural residual stresses (s_{rf})	low	high	low

Introduction

Peculiar Problems of Cold-Formed Steel Design

- Buckling modes for a lipped channel in compression



a) b) c) d) e)

Single modes:

(a) local (L);

(b) distortional (D);

(c) flexural (F);

(d) torsional (T);

(e) flexural-torsional (FT).

Coupled (interactive) modes:

(f) L + D;

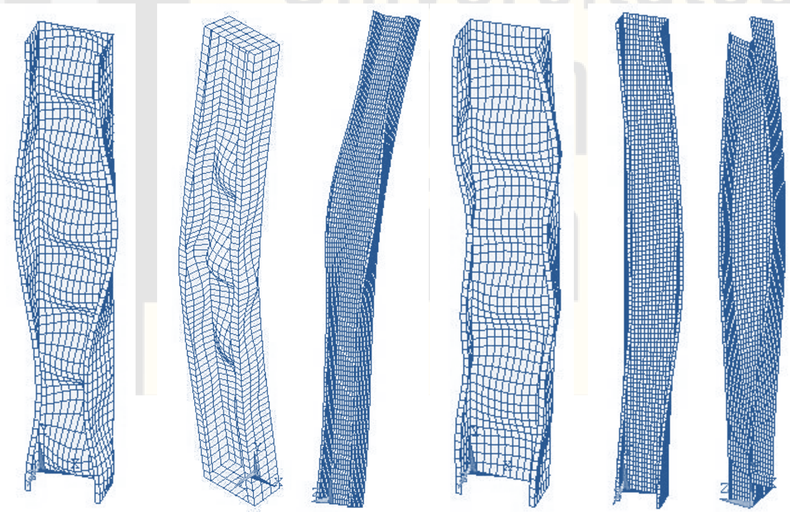
(g) F + L;

(h) F + D;

(i) FT + L;

(j) FT + D;

(k) F + FT



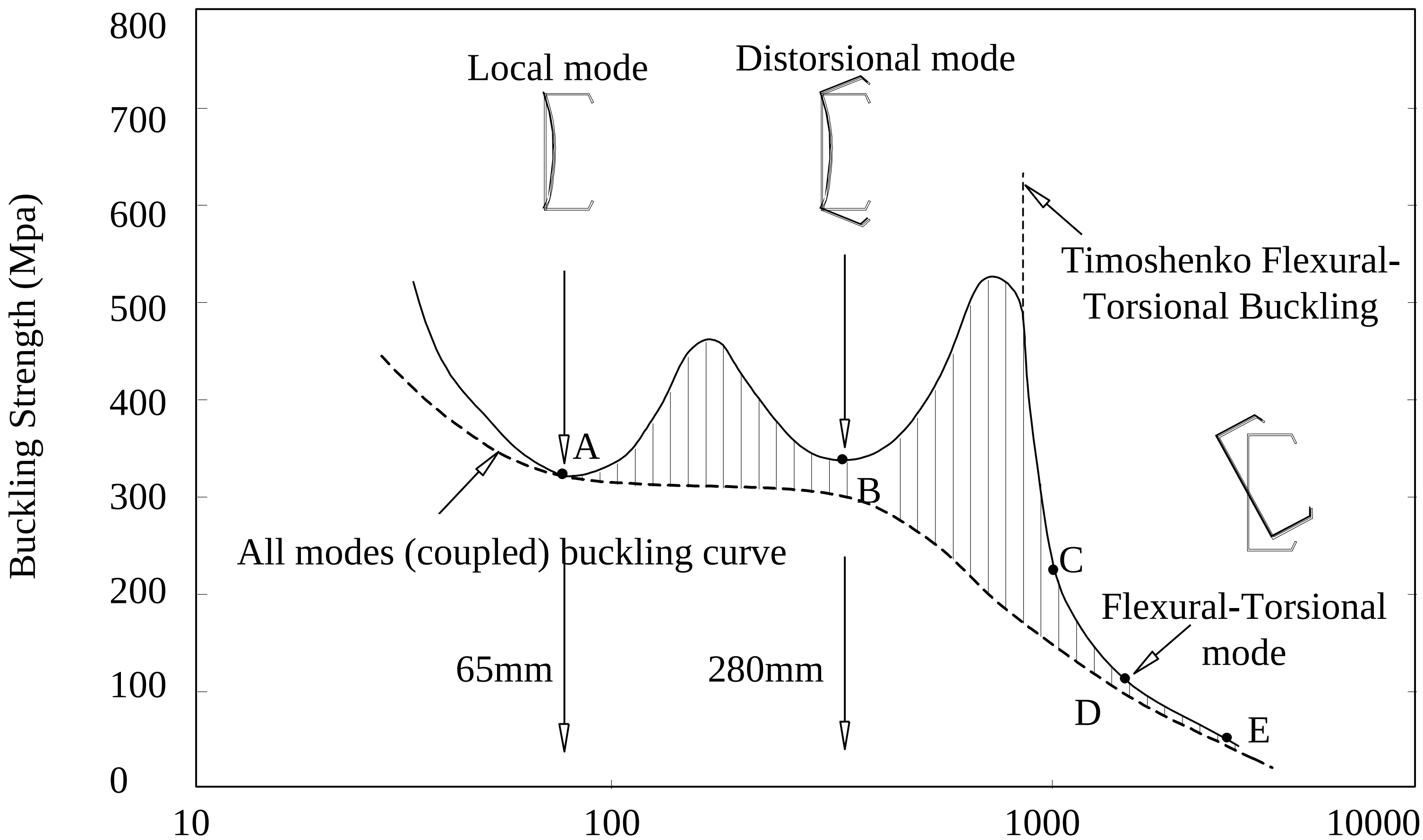
f) g) h) i) j) k)

Background and peculiarities

Introduction

Peculiar Problems of Cold-Formed Steel Design

- Buckling strength versus half-wavelength for a lipped channel in compression (Hancock, 2001)

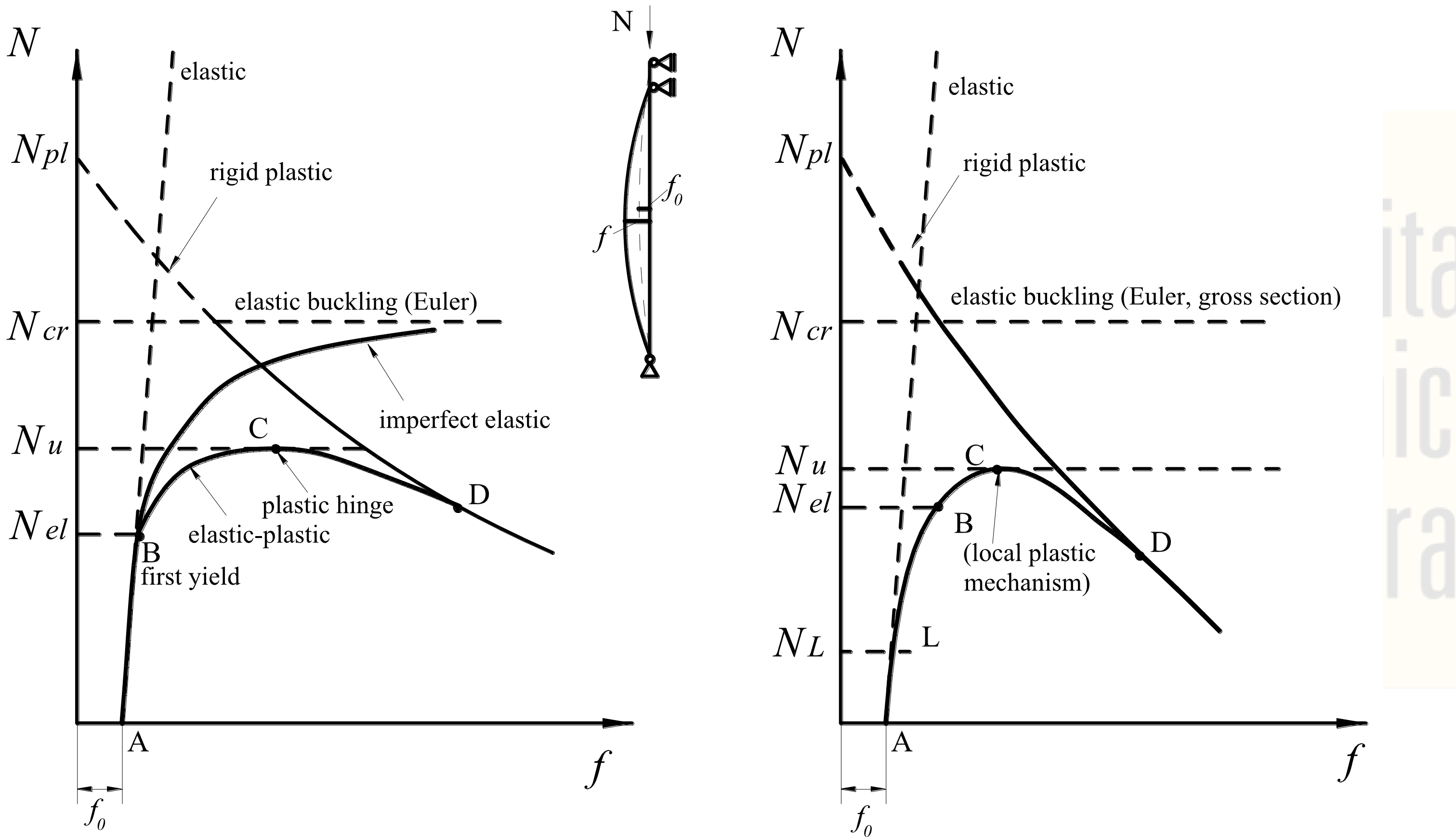


Background and peculiarities

Introduction

Peculiar Problems of Cold-Formed Steel Design

- Behaviour of compression bar
  - (a) slender tick-walled (hot-rolled section)
  - (b) thin-walled (cold-formed section)

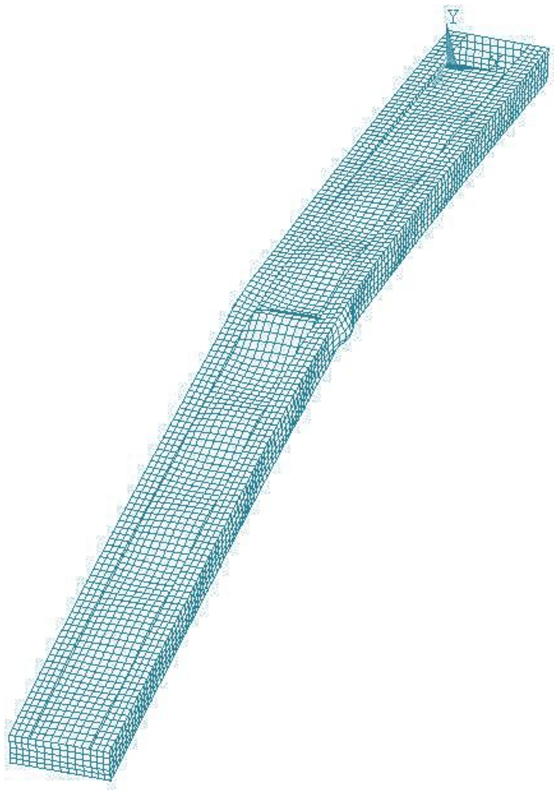


Background and peculiarities

Introduction

Peculiar Problems of Cold-Formed Steel Design

- Failure Mode of a Lipped Channel In Compression

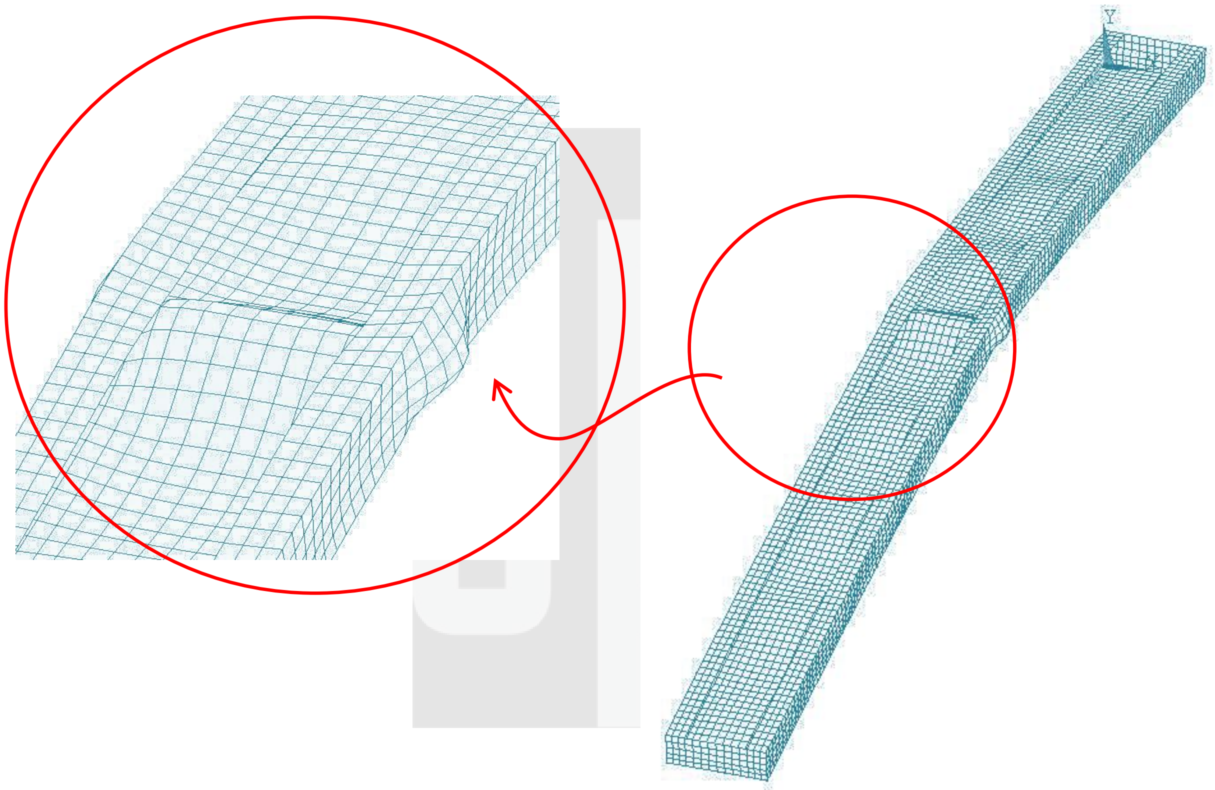


Background and peculiarities

Introduction

Peculiar Problems of Cold-Formed Steel Design

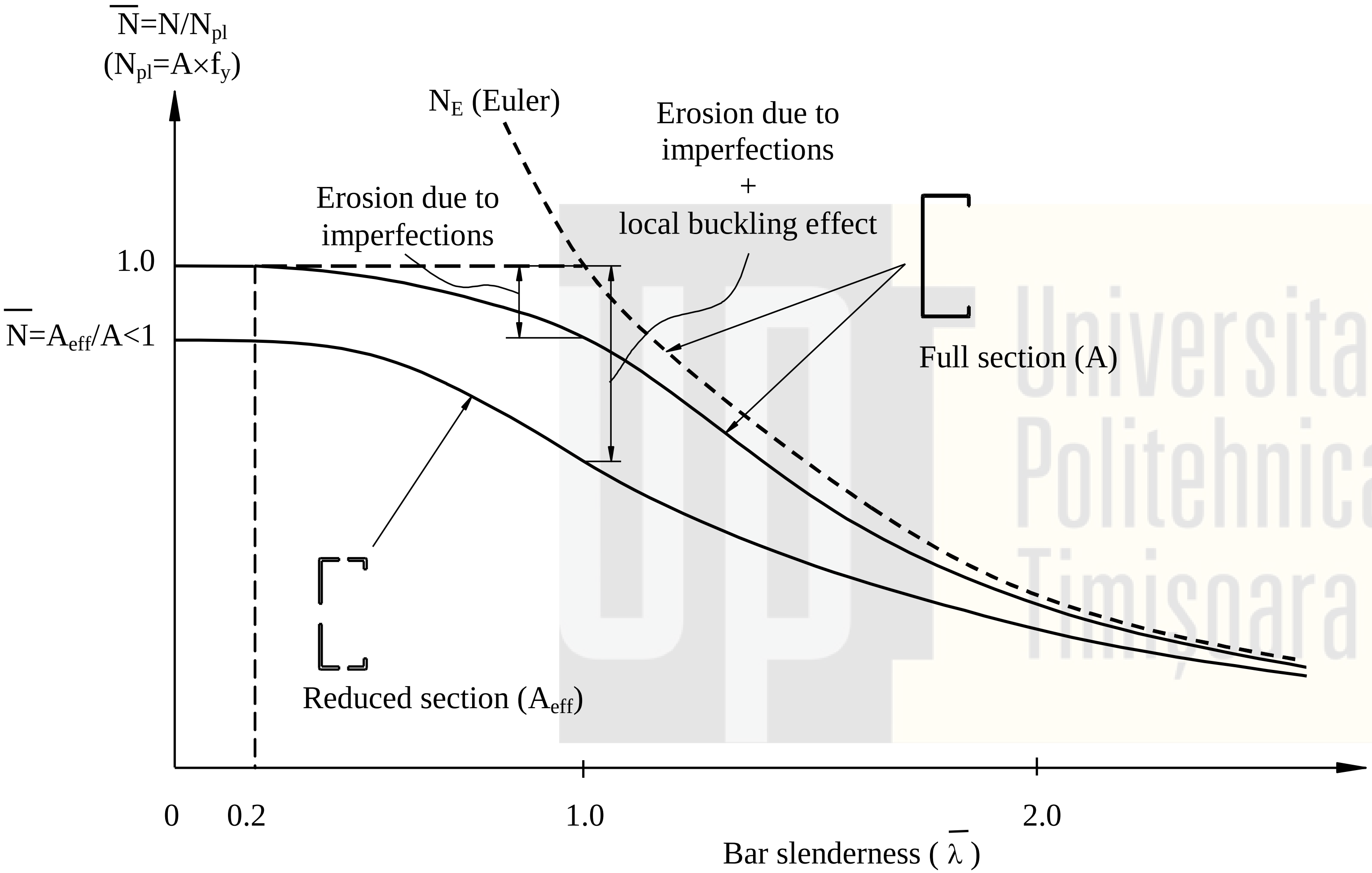
- Failure Mode of a Lipped Channel In Compression



Introduction

Peculiar Problems of Cold-Formed Steel Design

- **Effect of local buckling on the member capacity**

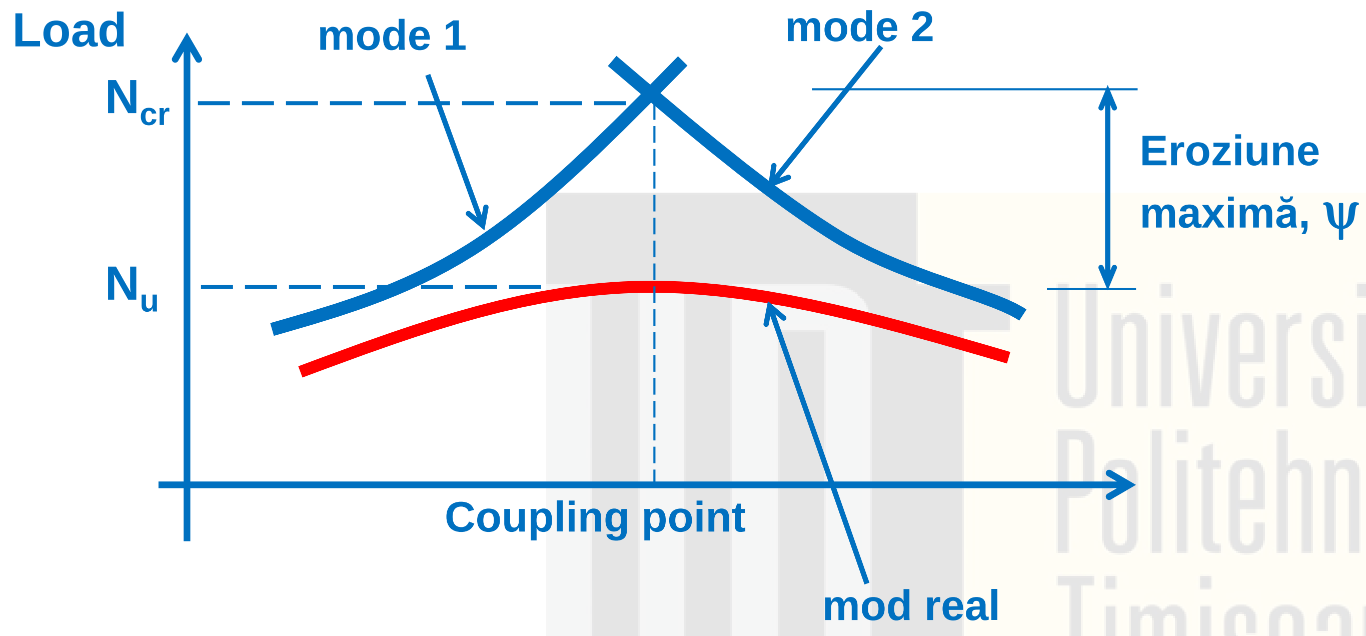


Background and peculiarities

Introduction

Peculiar Problems of Cold-Formed Steel Design

- Erosion Concept – Erosion levels



Background and peculiarities

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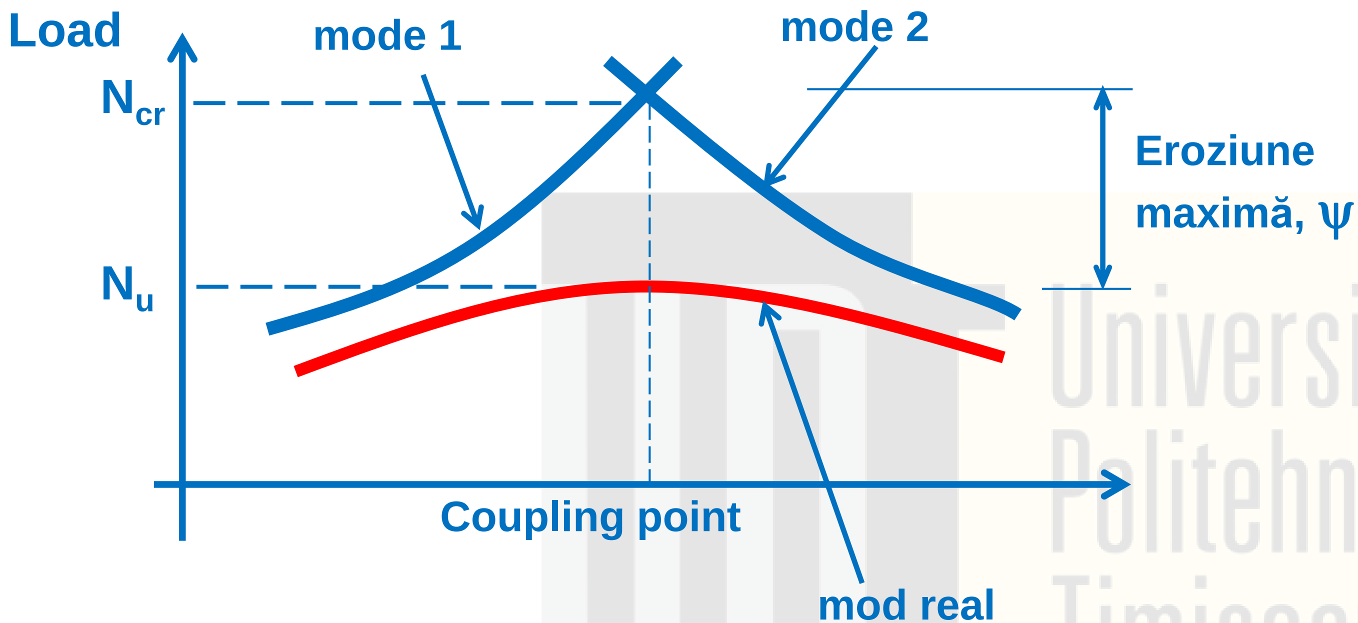
Introduction

Peculiar Problems of Cold-Formed Steel Design

- Erosion Concept – Erosion levels



$N_u = N_{cr} - \psi$

I: Weak interaction (WI), $\psi \leq 0.1$

II: Moderate interaction (MI), $0.1 \leq \psi \leq 0.3$

III: Strong interaction (SI), $0.3 \leq \psi \leq 0.5$

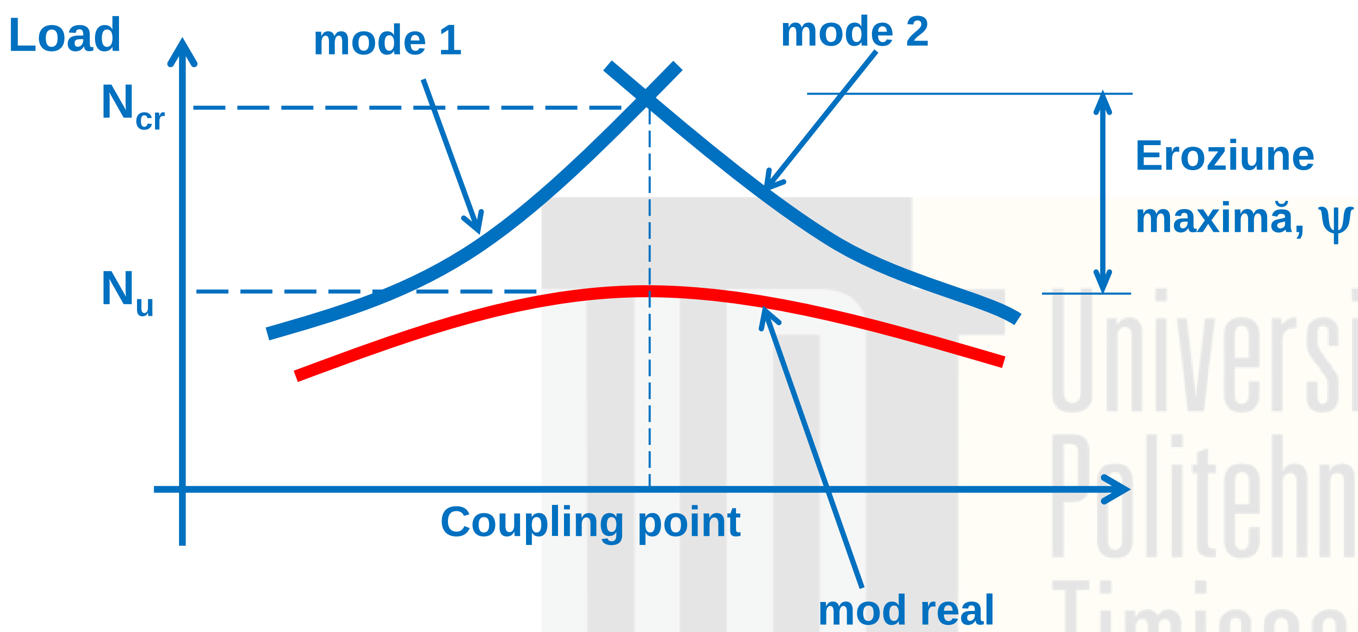
IV: Very Strong interaction (VSI), $\psi > 0.5$

Background and peculiarities

Introduction

Peculiar Problems of Cold-Formed Steel Design

- Erosion Concept – Erosion levels



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IV: Very Strong interaction (VSI), $\psi > 0.5$

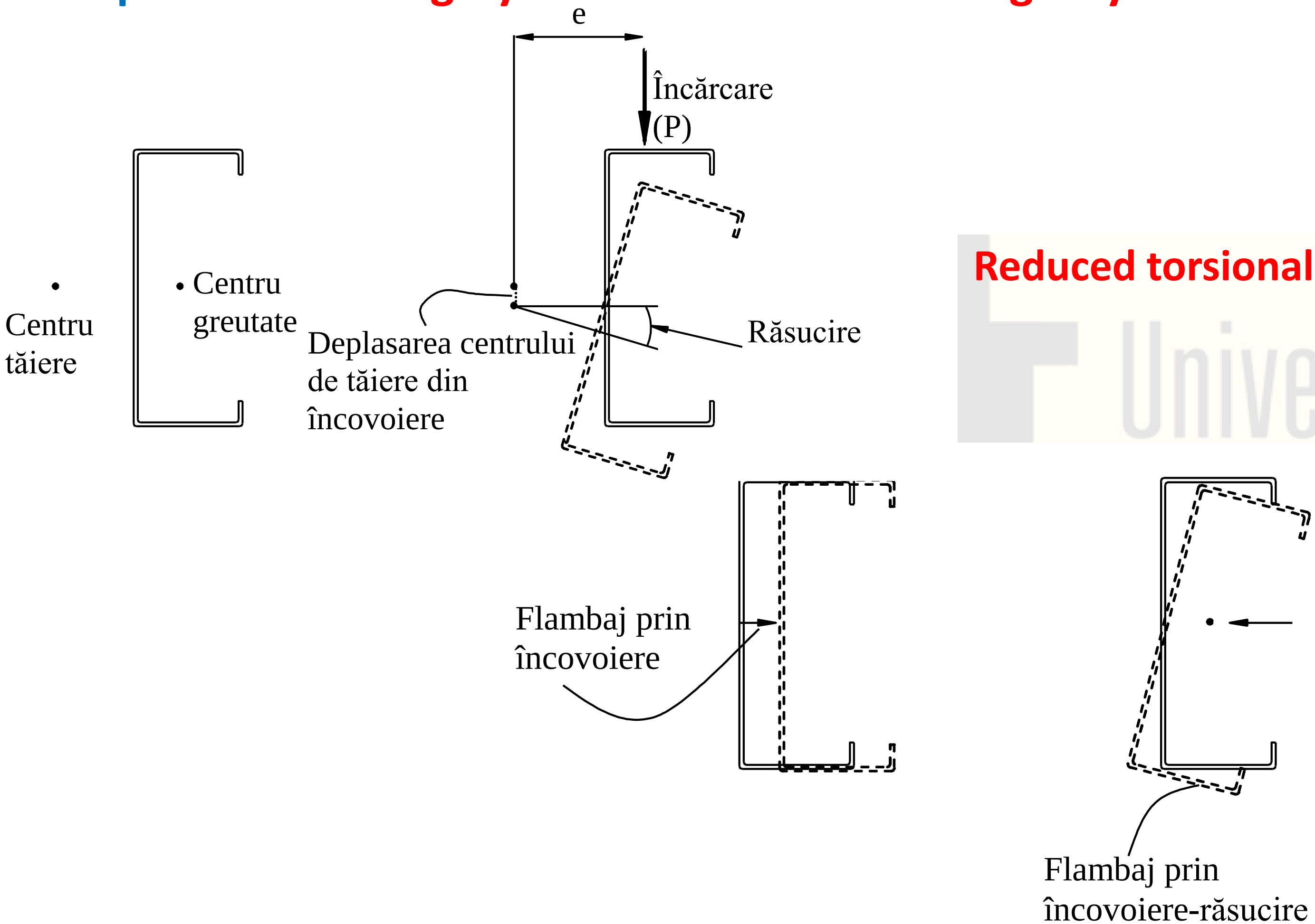
(II and III:) Thin walled members

Background and peculiarities

Introduction

Peculiar Problems of Cold-Formed Steel Design

- Open sections highly sensitive to torsional rigidity



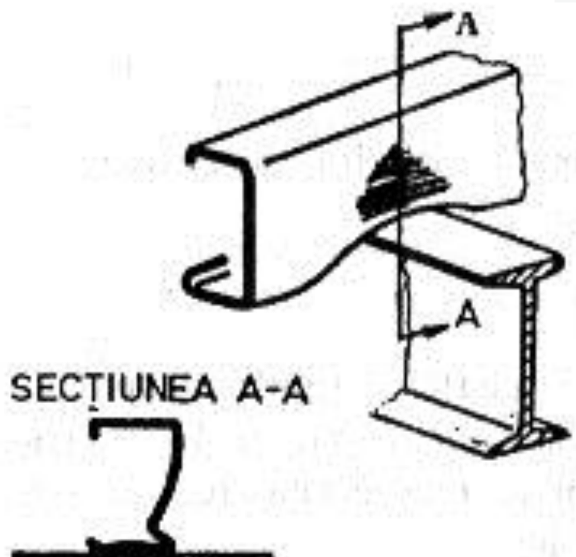
Reduced torsional stiffeness

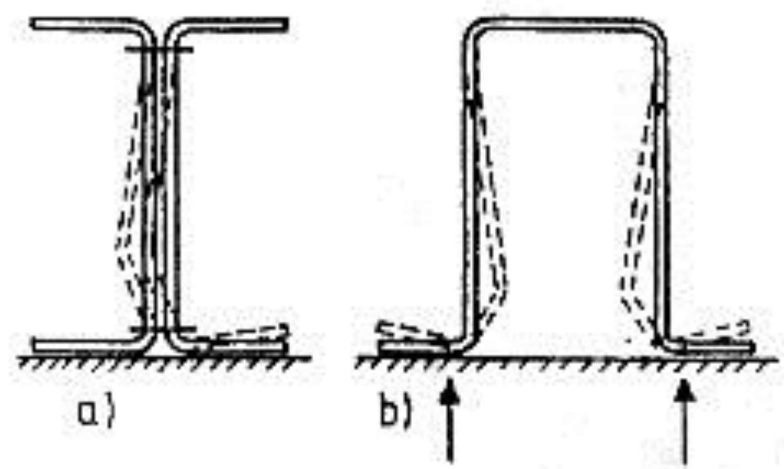
Background and peculiarities

Introduction

Peculiar Problems of Cold-Formed Steel Design

- Web Crippling (Critical Problems)
  - In cold-formed steel design, it is often not practical to provide load bearing and end bearing stiffeners. This is always the case in continuous sheeting and decking spanning several support points.
  - The depth-to-thickness ratios of the webs of cold-formed members are usually larger than hot-rolled structural members.
  - In many cases, the webs are inclined rather than vertical.
  - The intermediate element between the flange, on which the load is applied, d member usually consists of a bend of finite lied





Background and peculiarities

Introduction

Peculiar Problems of Cold-Formed Steel Design

- CONNECTIONS

Thin-to-thin	Thin-to-thick or thin-to-hot rolled	Thick-to-thick or thick-to-hot rolled
– self-drilling, self-tapping screws; – blind rivets; – press-joints; – single-flare V welds; – spot welds; – seam welding; – adhesive bonding.	– self-drilling, self-tapping screws; – fired pins; – bolts; – arc spot puddle welds; – adhesive bonding.	– bolts; – arc welds.

- Special Types of connections

Background and peculiarities

Introduction

Peculiar Problems of Cold-Formed Steel Design

- CONNECTIONS

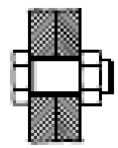
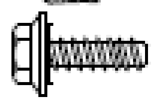
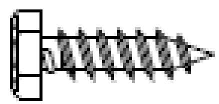
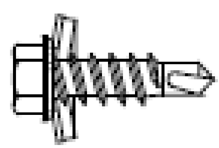
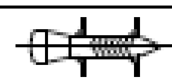
Thin-to-thin	Thin-to-thick or thin-to-hot rolled	Thick-to-thick or thick-to-hot rolled
– self-drilling, self-tapping screws; – blind rivets; – press-joints; – single-flare V welds; – spot welds; – seam welding; – adhesive bonding.	– self-drilling, self-tapping screws; – fired pins; – bolts; – arc spot puddle welds; – adhesive bonding.	– bolts; – arc welds.

- Special Types of connections
- Connections Typology (based on the material)
  - metal-to-metal connections
  - metal-to-sheathing (wood-based and gypsum-based sheathings) connections
  - metal-to-concrete connections.

Background and peculiarities

Introduction

Peculiar Problems of Cold-Formed Steel Design

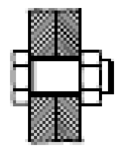
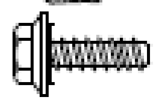
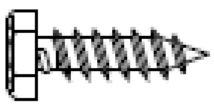
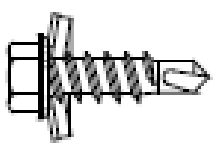
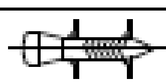
Thin -to- thick	Steel -to- wood	Thin -to-thin	Fasteners	Remark
X		X		Bolts M5-M16
X				Self-tapping screw ϕ6.3 with washer ≥ 16 mm, 1 mm thick with elastomer
	X	X		Hexagon head screw ϕ6.3 or ϕ6.5 with washer ≥ 16 mm, 1 mm thick with elastomer
X		X		Self-drilling screws with diameters: - ϕ4.22 or ϕ4.8 mm - ϕ5.5 mm - ϕ6.3 mm
		X		Blind rivets with diameters: - ϕ4.0 mm - ϕ4.8 mm - ϕ6.4 mm
X				Shot (fired) pins
X				Nuts

Background and peculiarities

Introduction

Peculiar Problems of Cold-Formed Steel Design

Usual mechanical fasters

Thin -to- thick	Steel -to- wood	Thin -to-thin	Fasteners	Remark
X		X		Bolts M5-M16
X				Self-tapping screw ϕ6.3 with washer ≥ 16 mm, 1 mm thick with elastomer
	X	X		Hexagon head screw ϕ6.3 or ϕ6.5 with washer ≥ 16 mm, 1 mm thick with elastomer
X		X		Self-drilling screws with diameters: - ϕ4.22 or ϕ4.8 mm - ϕ5.5 mm - ϕ6.3 mm
		X		Blind rivets with diameters: - ϕ4.0 mm - ϕ4.8 mm - ϕ6.4 mm
X				Shot (fired) pins
X				Nuts

Background and peculiarities

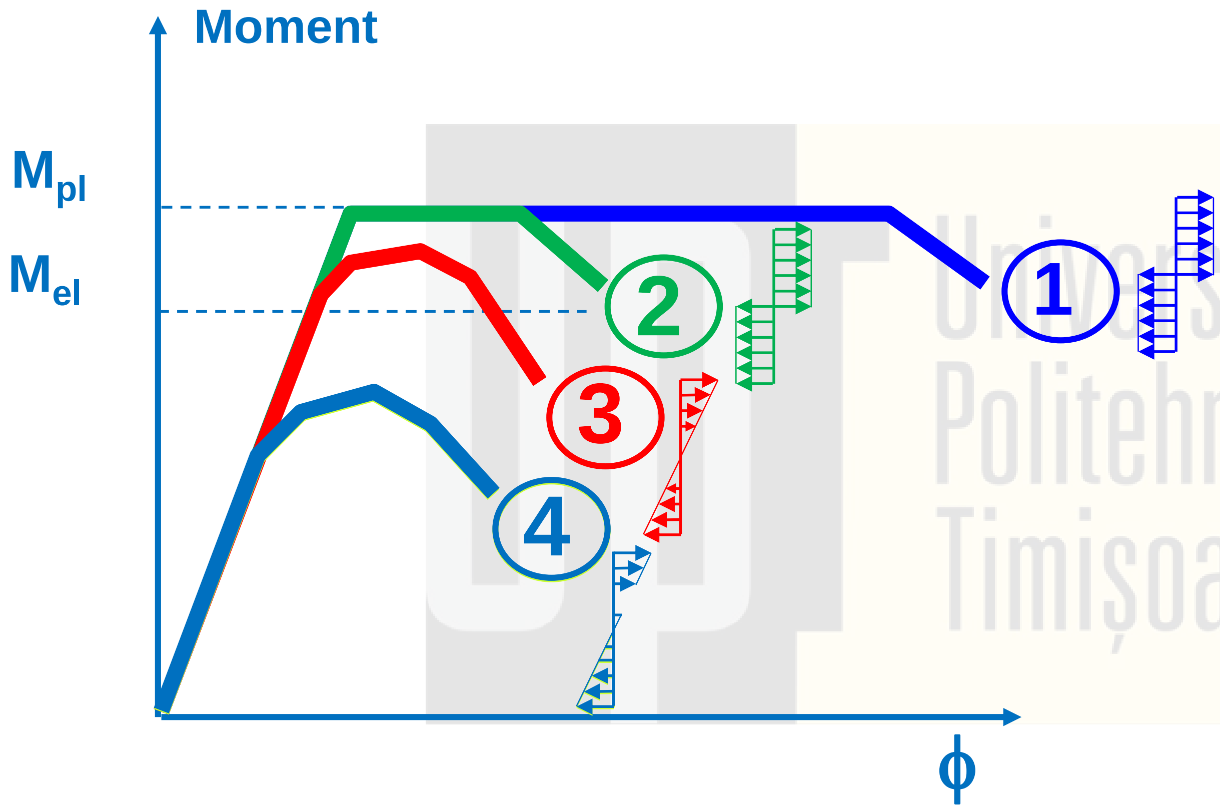
ECCS CECM EKS

JOINT RESEARCH CENTRE

Introduction

Peculiar Problems of Cold-Formed Steel Design

- Ductility and Plastic Design : Cold-Formed Steel Sections are , usually, Class 4 !

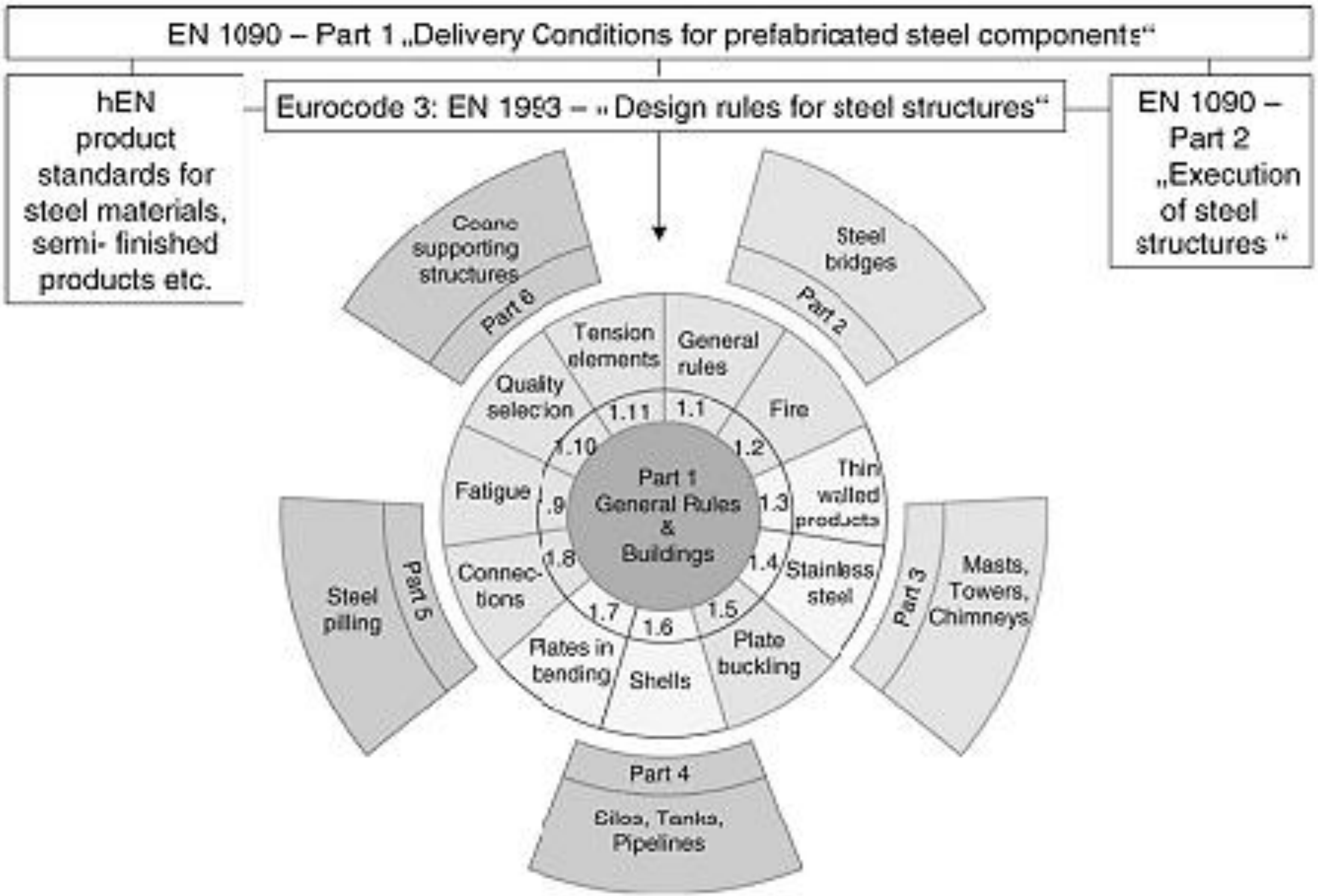


Background and peculiarities

Introduction

Basis of Design

- **Generalities**



Background and peculiarities

Introduction

Basis of Design

- Limit State Design > EN1990 (CEN, 2002a)
  - All the separate conditions that make a structure unfit for use are taken into account. These are the separate limit states;
  - The design is based on the actual behaviour of materials and performance of structures and members in service;
  - Ideally, design should be based on statistical methods with a small probability of the structure reaching a limit state.

ULS – Specific Thin Walled Issues
local instability and strength of sections
interactive instability and influence of specific imperfection
connecting technology and related design procedures
reduced capacity with reference to ductility, plastic design and seismic resistance
fire resistance

Background and peculiarities

Introduction

Basis of Design

- Actions on Structures. Combinations of Actions (EN1991)
  - Verification at the Ultimate Limit State
  - Verification at the Serviceability Limit State

    - safety factors according to EN1990 (as for hot rolled steel)
    - factors:
      - γ_{M0} – resistance of cross sections to excessive yielding including local and distortional buckling
      - γ_{M1} – resistance of members and sheeting where failure is caused by global buckling
      - γ_{M2} – resistance of net sections at fastener holes

Background and peculiarities

Introduction

Basis of Design

- Actions on Structures. Combinations of Actions (EN1991)
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      - γ_{M1} – resistance of members and sheeting where failure is caused by global buckling
      - γ_{M2} – resistance of net sections at fastener holes

Recommended values (EN1993 – 1 – 3)
$\gamma_{M0} = 1.0$
$\gamma_{M1} = 1.0$
$\gamma_{M2} = 1.25$

Background and peculiarities

Introduction

Basis of Design

- **Materials (EN1993–1–3)**

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

Background and peculiarities

Introduction

Basis of Design

- **Materials (EN1993–1–3)**

Type of steel	Standard	Grade	f_{yb} (N/mm^2)	f_u (N/mm^2)
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Hot-rolled products of structural steels. Part 3: Technical delivery conditions for normalized/normali[illegible] weldab[illegible]	[illegible]	S 275 N S 355 N [illegible] [illegible] [illegible] [illegible] [illegible] S 460 NL	275 [illegible] [illegible] [illegible] [illegible] [illegible] 420 460	[illegible] [illegible] [illegible] [illegible] [illegible] 470 520 550
[illegible] products of structural steels. Part 4: Technical delivery conditions for thermo-mechanical rolled weldable fine grain structural steels	EN 10025: Part 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

All steels used for cold-formed members and profiled sheets should be suitable for cold-forming and welding, if needed.

Introduction

Basis of Design

- Materials (EN1993–1–3)
  - yield strength f_{yb} and ultimate tensile strength f_u should be obtained:
    - either by adopting the values $f_y = R_{eh}$ (upper yield strength) or $R_{p0,2}$ (proof strength) and $f_u = R_m$ (tensile strength) direct from product standards
    - by using the values given in Table 2.7a or b of EN1993–1–3
    - by appropriate tests (EN10002-1)

Background and peculiarities

Introduction

Basis of Design

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    - by using the values given in Table 2.7a or b of EN1993–1–3
    - by appropriate tests (EN10002-1)
  - properties of cold-formed sections and sheeting
    - modification of the stress-strain curve of the steel (increase of the yield strength is due to strain hardening, the increase of the ultimate strength is related to strain aging)
    - coiling, uncoiling, cold reducing and the cold-forming process

$$f_{ya} = f_{yb} + \left(f_u - f_{yb}\right)\frac{knt^2}{A_g} \quad \text{but} \quad f_{ya} \leq \frac{\left(f_u + f_{yb}\right)}{2}$$

used for

- cross section resistance of an axially loaded tension member
- buckling resistance of members with fully effective cross section

Background and peculiarities

Introduction

Basis of Design

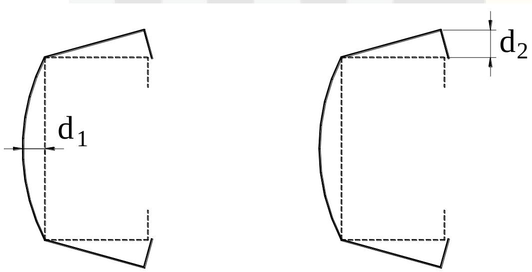
- Methods of Analysis and Design
  - Methods of analysis – Global frame analysis
  - Finite Element Methods (FEM) for analysis and design
  - Design assisted by testing

    - determine the distribution of the internal forces and the corresponding deformations in a structure subjected to a specified loading
    - global analysis of frames is conducted on a model based on many assumptions including those for the structural model, the geometric and material behaviour of the structure, of its sections /members and joints.
      - (1) First-order elastic analysis;
      - (2) Second-order elastic analysis;
      - (3) Elastic-perfectly plastic analysis (Second-order theory);
      - (4) Elasto-plastic analysis (second-order theory);
      - ~~(5) Rigid-plastic analysis (first-order theory)~~

Introduction

Basis of Design

- Methods of Analysis and Design
  - Methods of analysis – Global frame analysis
  - Finite Element Methods (FEM) for analysis and design
  - Design assisted by testing
    - guidance can be found in Annex C of EN1993-1-5
    - The FE-modelling may be carried out either for:
      - (1) the component as a whole
      - (2) a substructure as a part of the whole structure. Also, design of members and details can be assisted by numerical simulations (e.g. numerical testing).
      - Sectional imperfections for Local and Distortional Buckling modes



Introduction

Basis of Design

- **Design assisted by testing – may be undertaken under any of the following circumstances:**
  - if it is desired to prove the validity and adequacy of an analytical procedure;
  - if it is desired to produce resistance tables based on tests, or on a combination of testing and analysis;
  - if it is desired to take into account practical factors that might alter the performance of a structure, but are not addressed by the relevant analysis method for design by calculation.

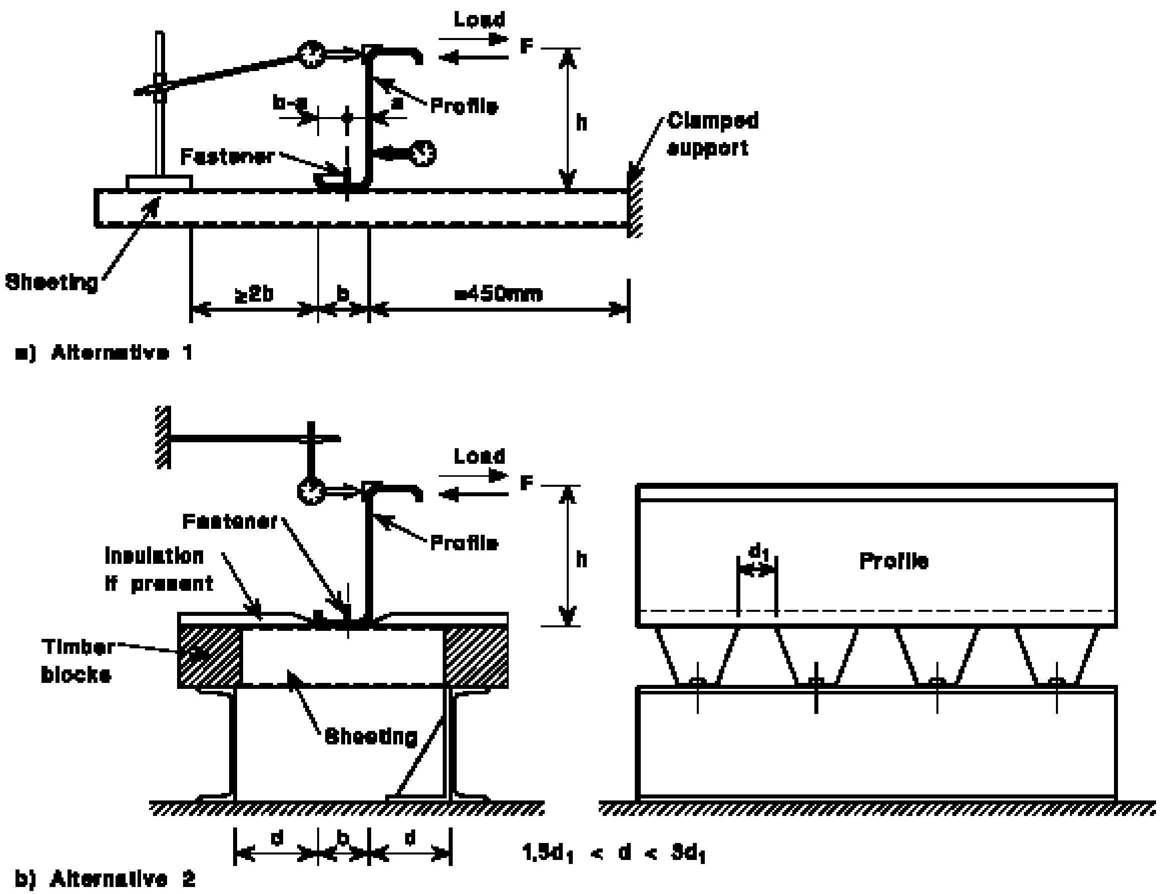
- ECCS No 124 -2008: Testing of connections with mechanical fasteners in sheeting and Sections
- ECCS no. 127-2009 Testing and design of fastenings in Sandwich panels
- En 15129 -2009: Steel static storage systems – Adjustable pallet racking systems
- EN 1990, Annex D : design assisted by testing (reliability and strength)
- EN 1993-1-3, Ch 9, Annex A

Background and peculiarities

Introduction

Basis of Design

- Design assisted by testing – may be undertaken under any of the



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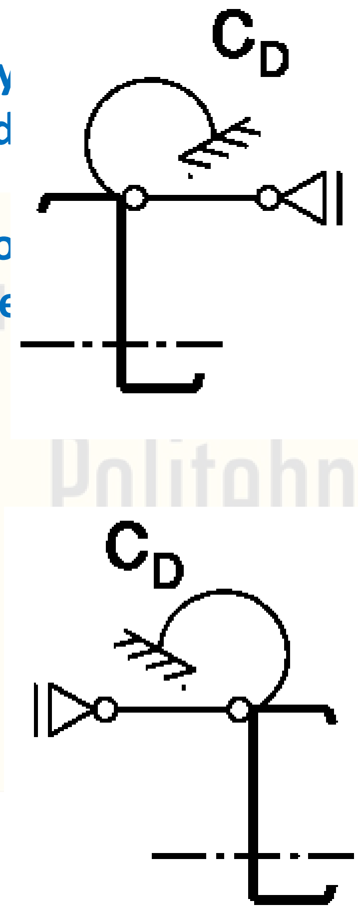
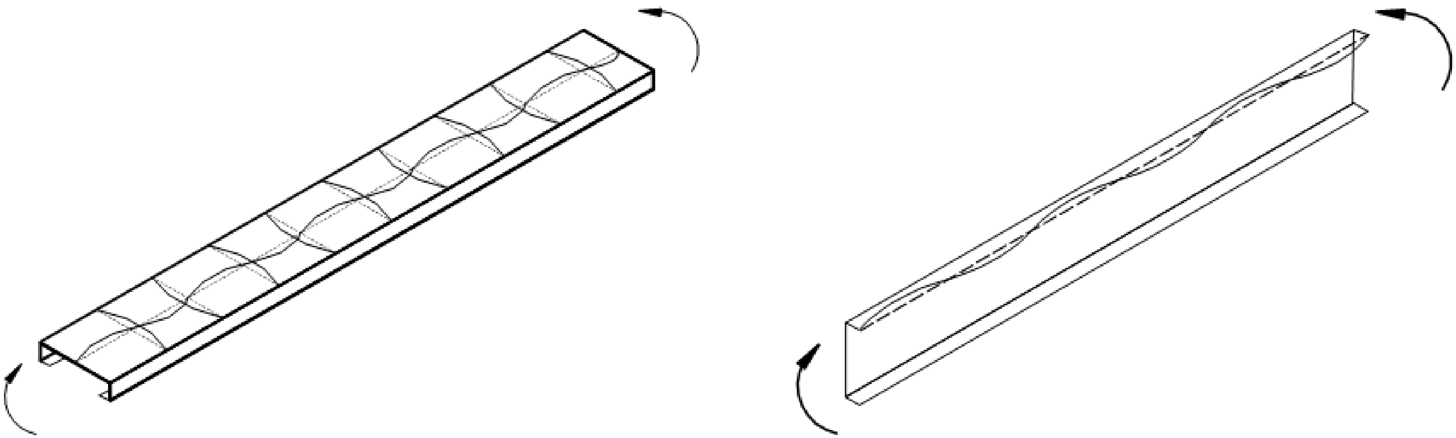
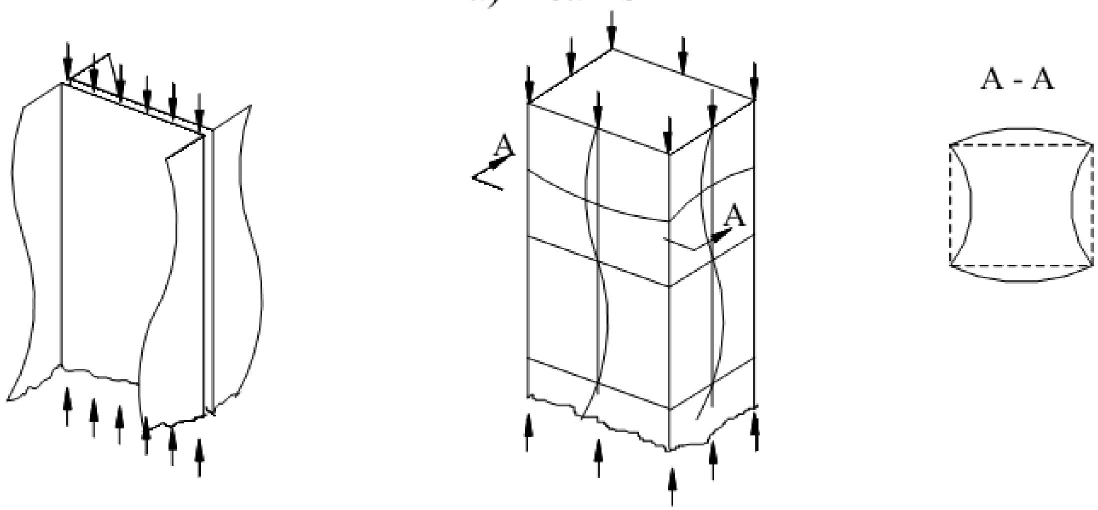


Resistance of sections

General



a) Beams



b) Columns

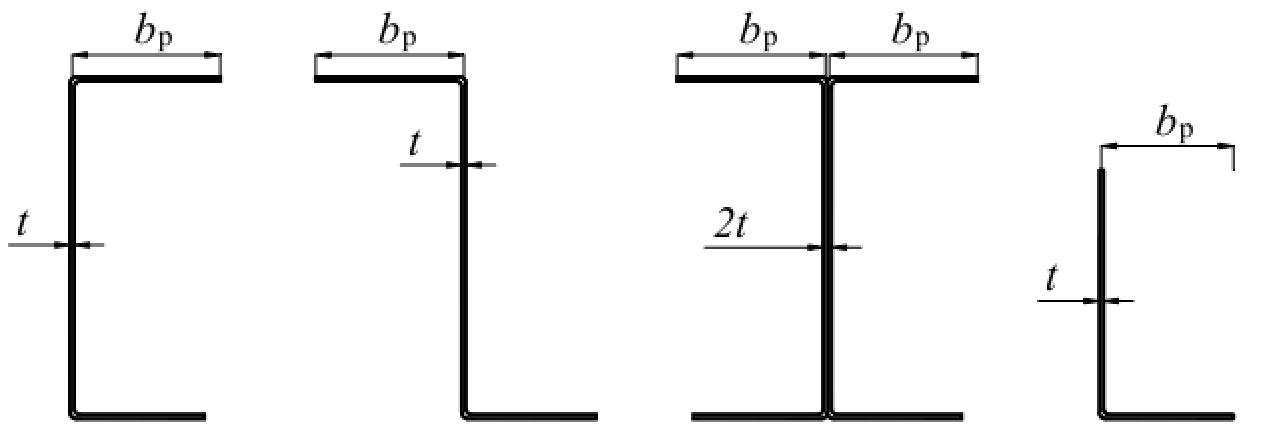
Resistance of sections

Behaviour and Resistance of Cross Section

Properties of gross-cross section

- **Dimensional limits of component walls of CFS**

1. *Unstiffened compression walls.*



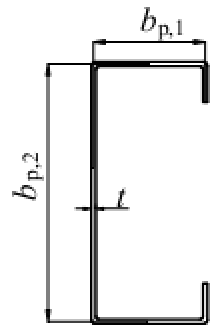
a) Plain channel b) Plain Z-section c) Built-up I-section made of two plain channels back-to-back d) Plain angle

Behaviour and Resistance of Cross Section

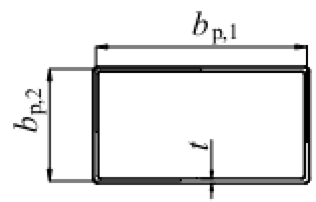
Properties of gross-cross section

- **Dimensional limits of component walls of CFS**

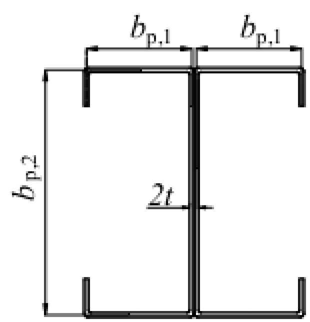
2. *Stiffened or partially stiffened compression walls.*



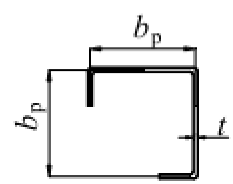
a) Lipped channel



b) Box-section



c) I-section made of two lipped channels back to back



d) Lipped angle with equal stiffened legs

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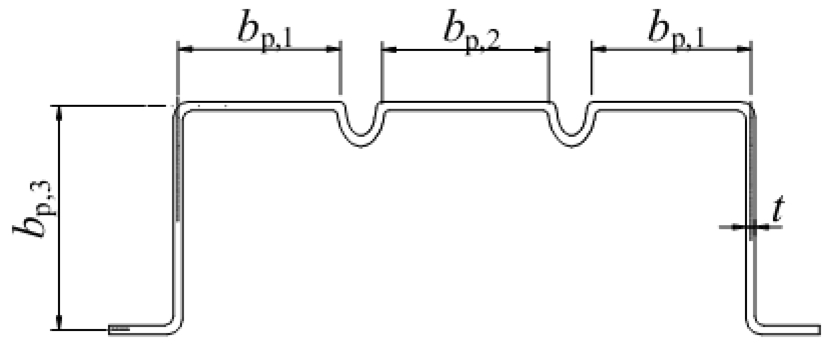
Behaviour and Resistance of Cross Section

Properties of gross-cross section

- **Dimensional limits of component walls of CFS**

3. *Multiple-stiffened walls.*



the sizes of stiffeners should be

$$0.2 \le c / b \le 0.6$$

$$0.1 \le d / b \le 0.3$$

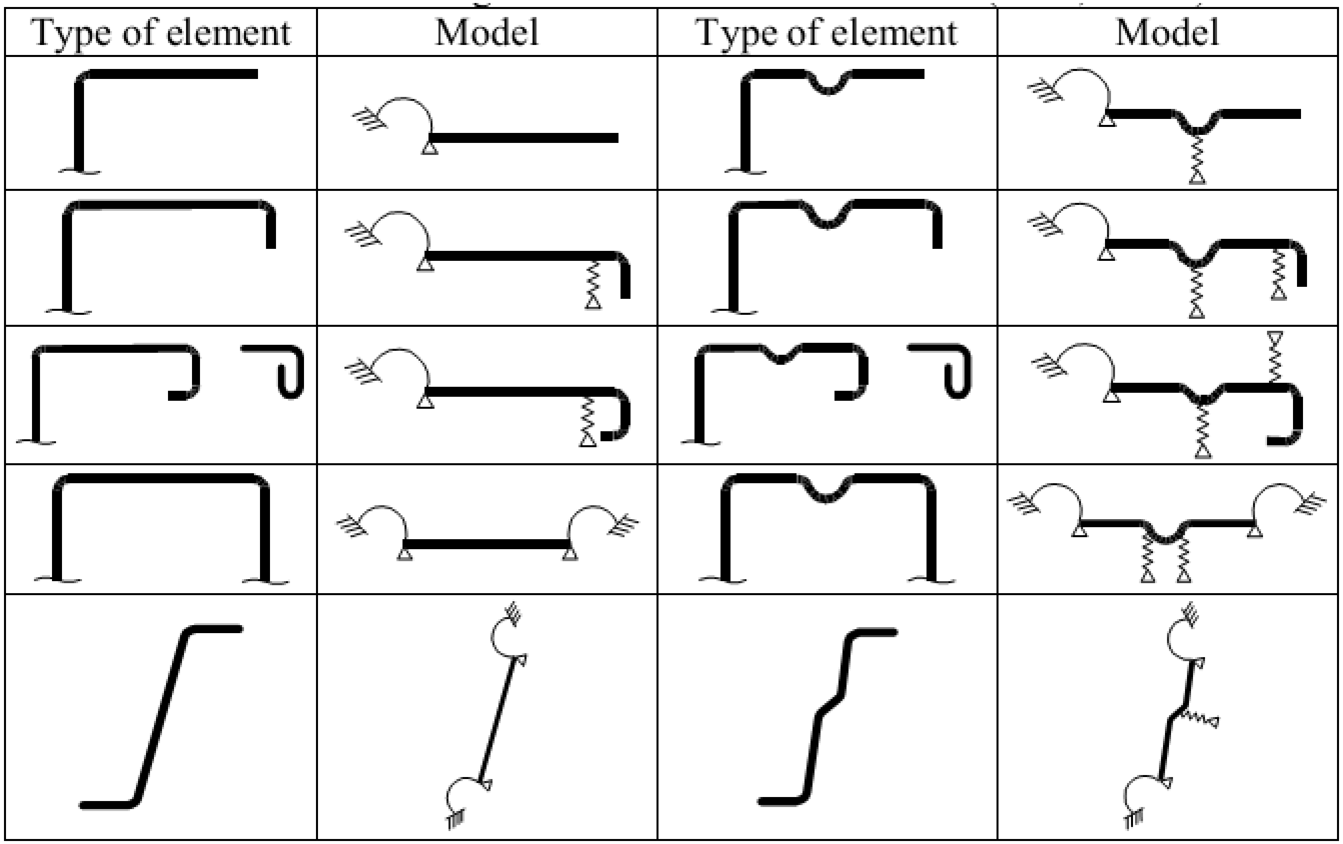
Resistance of sections

Behaviour and Resistance of Cross Section

Properties of gross-cross section

- **Modelling of cross-section for analysis**

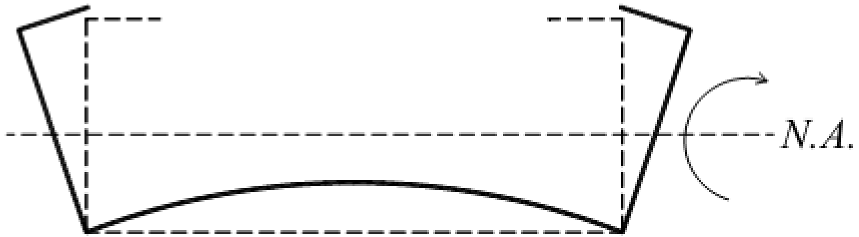
Modelling of elements of a cross section (CEN, 2006a)



Resistance of sections

Behaviour and Resistance of Cross Section

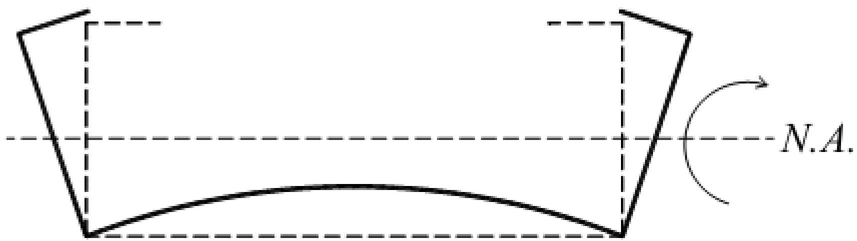
Effect of wall slenderness :Flange curling



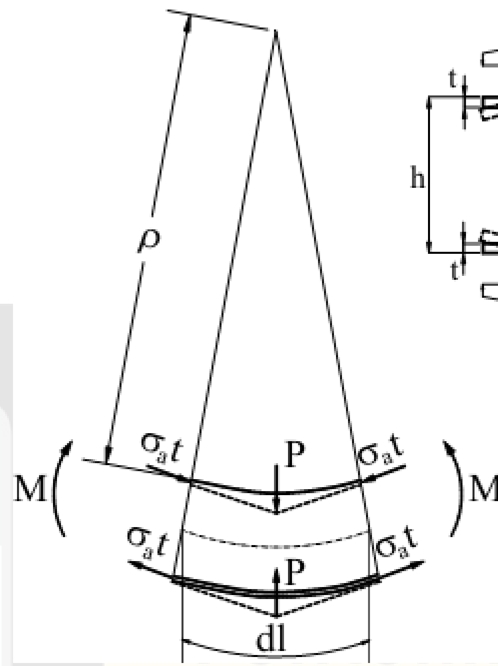
Resistance of sections

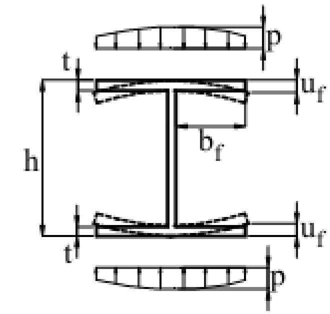
Behaviour and Resistance of Cross Section

Effect of wall slenderness :Flange curling



George Winter (1940)





$$p = \frac{\sigma_a \cdot t \cdot d\phi}{dl} = \frac{\sigma_a \cdot t}{\rho} = \frac{\sigma_a \cdot t}{E \cdot I / M}$$

σ_a is the average (mean) bending stress in flange;

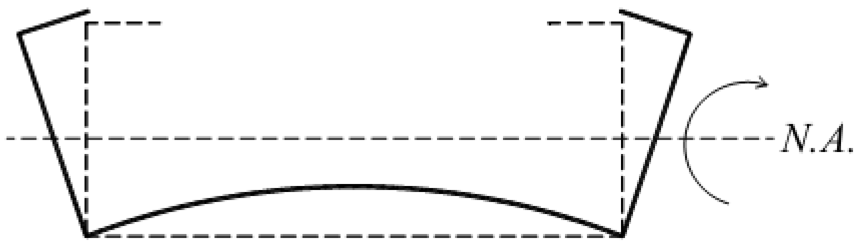
E is the modulus of elasticity;

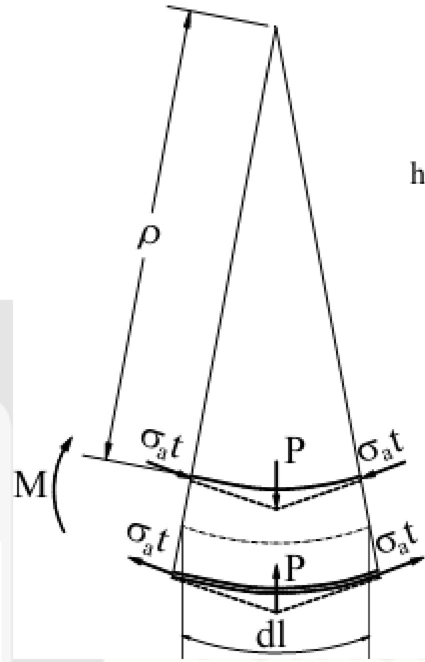
I is the second moment of the beam area;

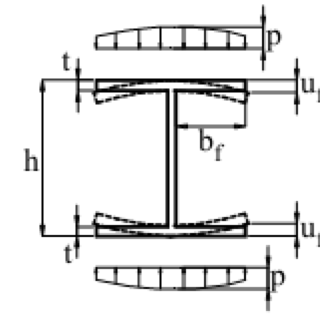
Resistance of sections

Behaviour and Resistance of Cross Section

Effect of wall slenderness :Flange curling







George Winter (1940)

$$p = \frac{\sigma_a \cdot t \cdot d\phi}{dl} = \frac{\sigma_a \cdot t}{\rho} = \frac{\sigma_a \cdot t}{E \cdot I / M}$$

σ_a is the average (mean) bending stress in flange;

E is the modulus of elasticity;

I is the second moment of the beam area;

If p is considered to be uniformly distributed load applied on the flange

$$u_f = \frac{p \cdot b_f^4}{8 \cdot D} = 3 \cdot \left(\frac{\sigma_a}{E}\right)^2 \cdot \left(\frac{b_f^4}{t^2 \cdot d}\right) \cdot (1 - \nu^2)$$

u_f is flange deflection of outer edge;

D is flexural rigidity of plate, $D = E \cdot t^2 / 12 \cdot (1 - \nu^2)$;

ν is Poisson's ratio.

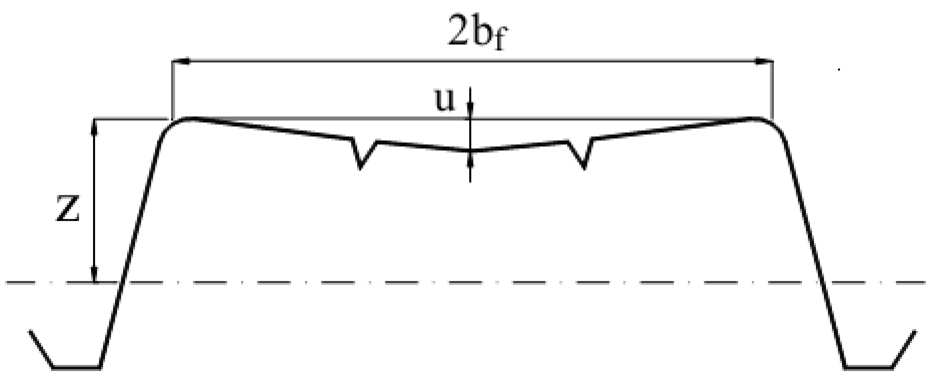
Resistance of sections

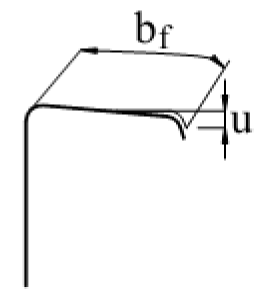
Behaviour and Resistance of Cross Section

Flange curling

The ECCS Recommendations (ECCS, 1987)
both compression and tensile flanges
both with and without stiffeners:

$$u_f = 2 \cdot \frac{\sigma_a^2 \cdot b_f^4}{E^2 \cdot t^2 \cdot z}$$





Resistance of sections

Behaviour and Resistance of Cross Section

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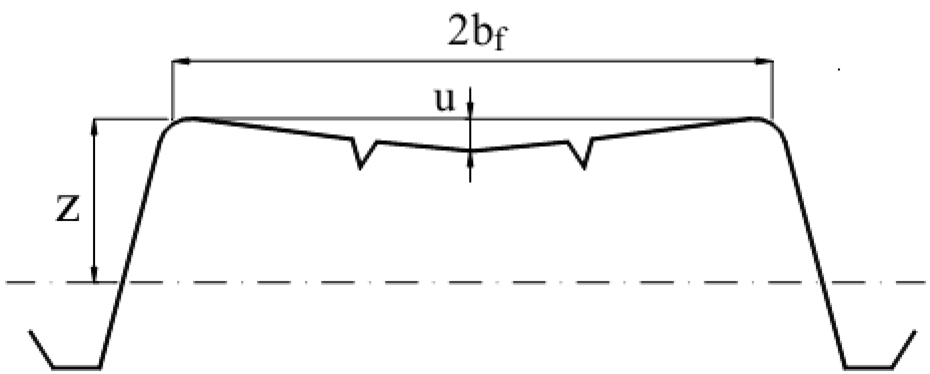

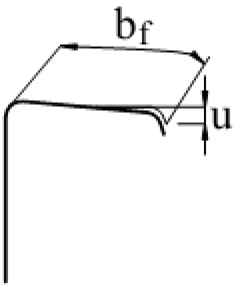

Flange curling can be neglected in calculations if the deflection $u_s < 5\%$

Resistance of sections

Behaviour and Resistance of Cross Section

Flange curling

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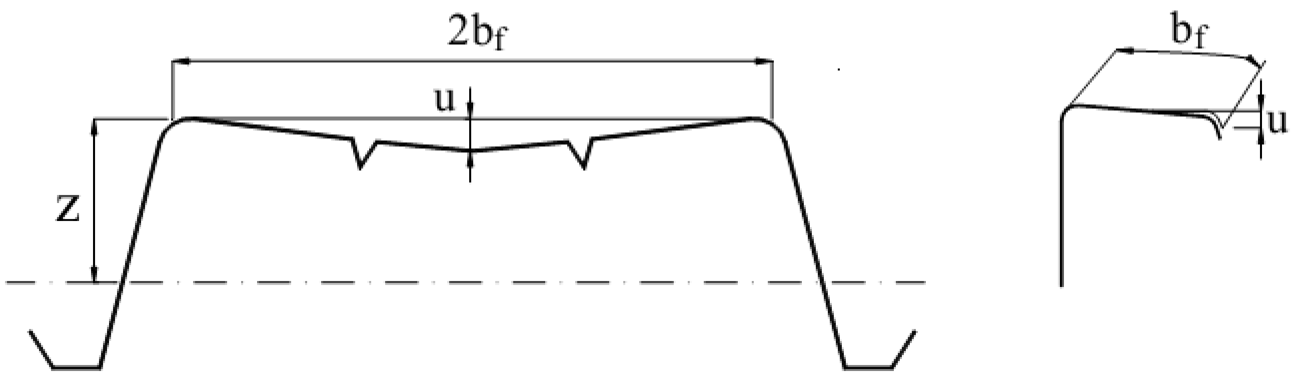


Flange curling can be neglected in calculations if the deflection $u_s < 5\%$

to limit the curling effect

AISI S100-07

AS/NZS-4600:2005

$$b_f \le \sqrt{\frac{0.06t \cdot d \cdot E}{\sigma_a}} \cdot \sqrt[4]{\frac{100 \cdot u_f}{d}}$$

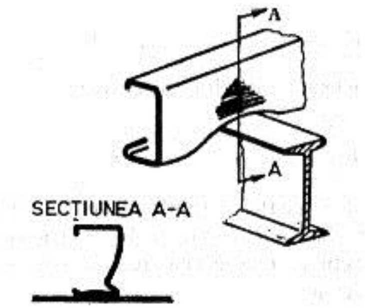
$$u_s < 0.05d$$

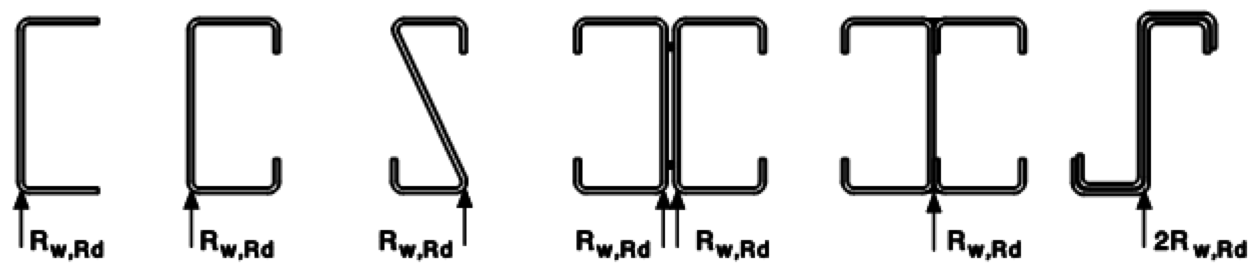
Resistance of sections

Behaviour and Resistance of Cross Section

Web Crippling

- Local Transverse Forces – *Cross sections with a single unstiffened web*



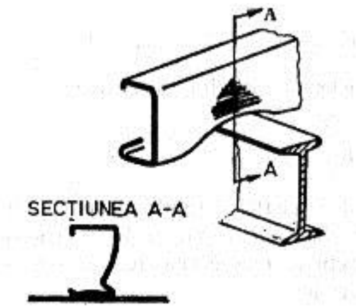


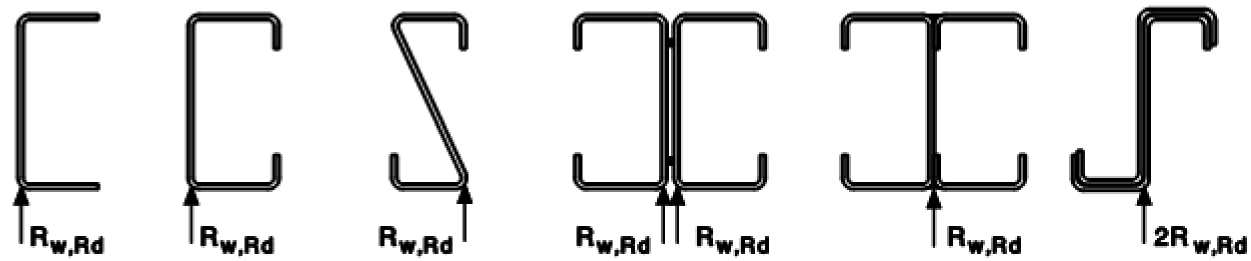
Resistance of sections

Behaviour and Resistance of Cross Section

Web Crippling

- **Local Transverse Forces –** ***Cross sections with a single unstiffened web***



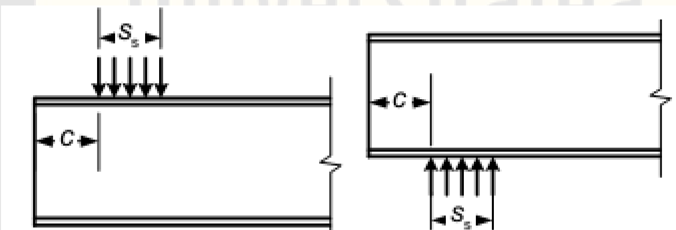


for a single local load or support reaction

- for a cross section with stiffened flanges:

$$R_{w,Rd} = \frac{k_1 k_2 k_3 \left[9.04 - \frac{h_w / t}{60}\right]\left[1 + 0.01\frac{s_s}{t}\right] \cdot t^2 \cdot f_{yb}}{\gamma_{M1}}$$

i) $c \leq 1.5\, h_w$ clear from a free end:

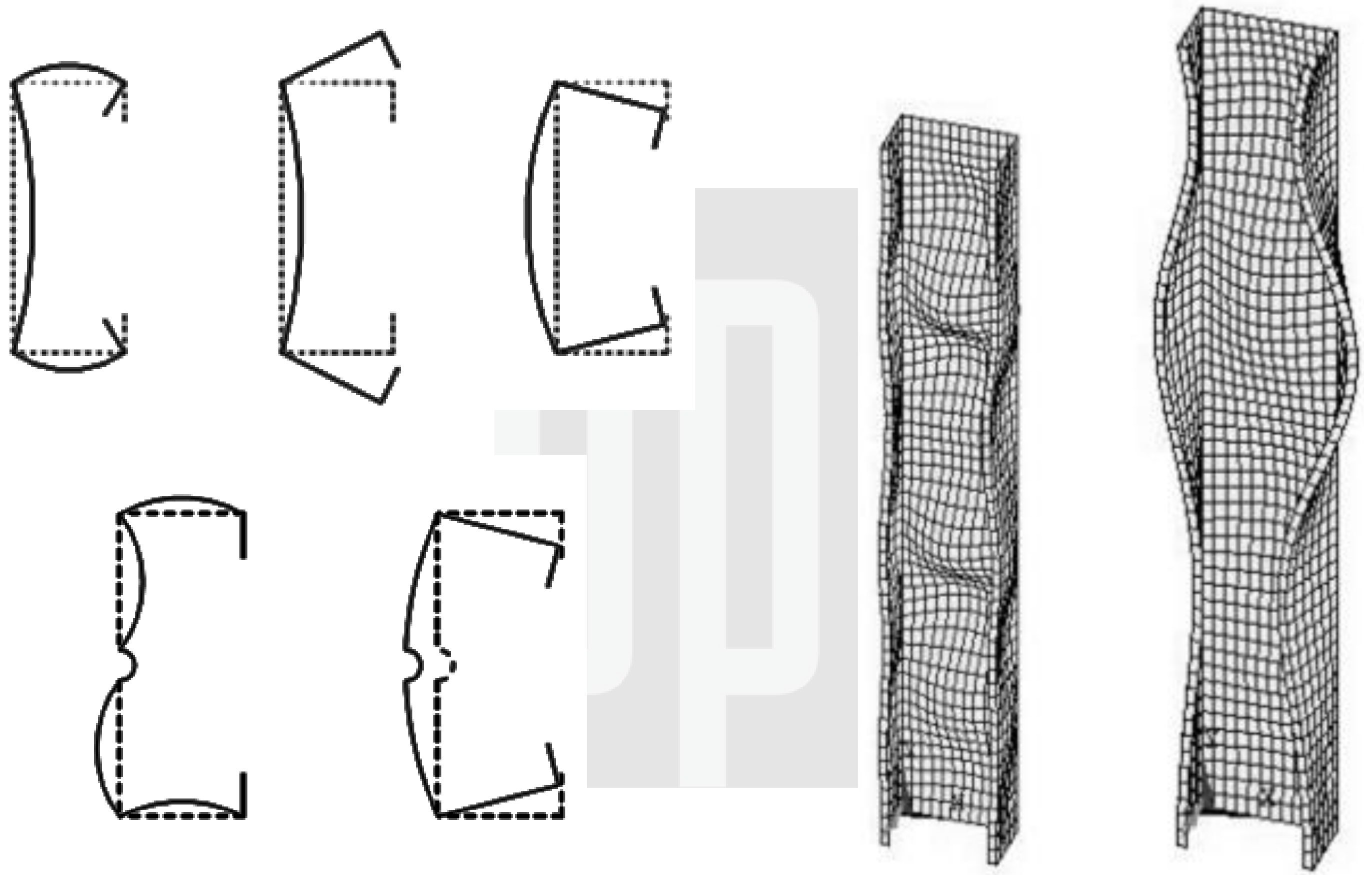


Resistance of sections

Behaviour and Resistance of Cross Section

Local Buckling and Distortional Buckling

- Sectional buckling modes in thin-walled sections

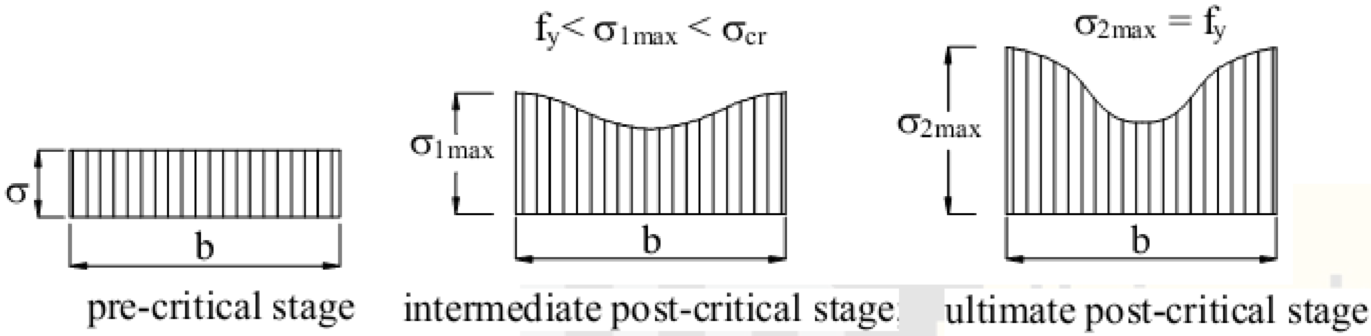


Behaviour and Resistance of Cross Section

Local Buckling

- **Elastic buckling of thin plates**

Consecutive stress distribution in stiffened compression elements



Resistance of sections

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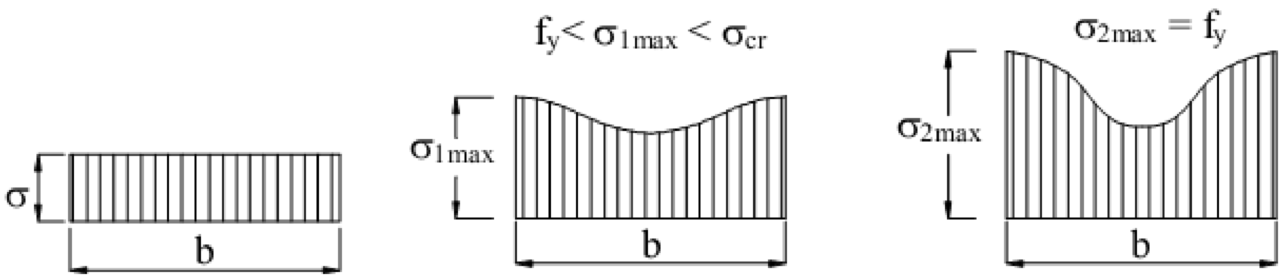
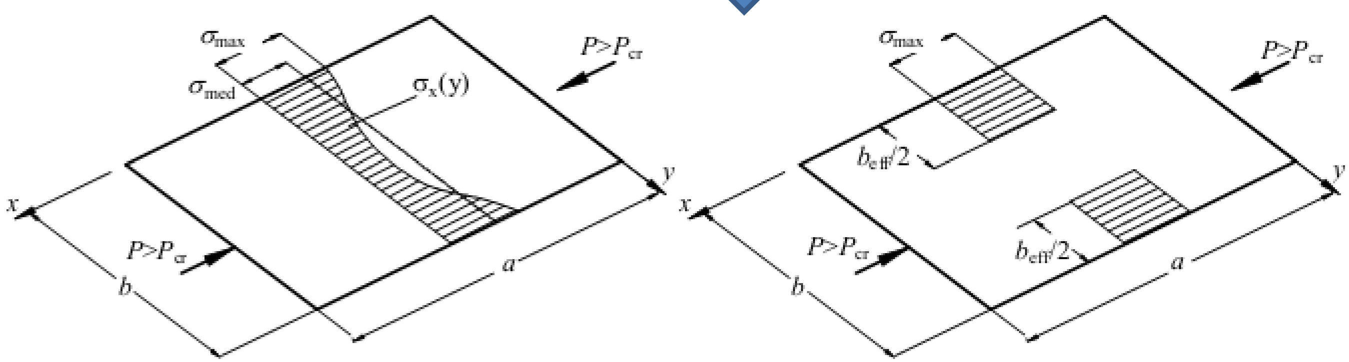


pre-critical stage　intermediate post-critical stage　ultimate post-critical stage

actual stress distribution　　equivalent stress distribution effective width



$$P = \sigma_{med} \cdot b \cdot t = \int_0^b \sigma_x(y) \cdot t \cdot dy = \sigma_{max} \cdot b_{eff} \cdot t$$

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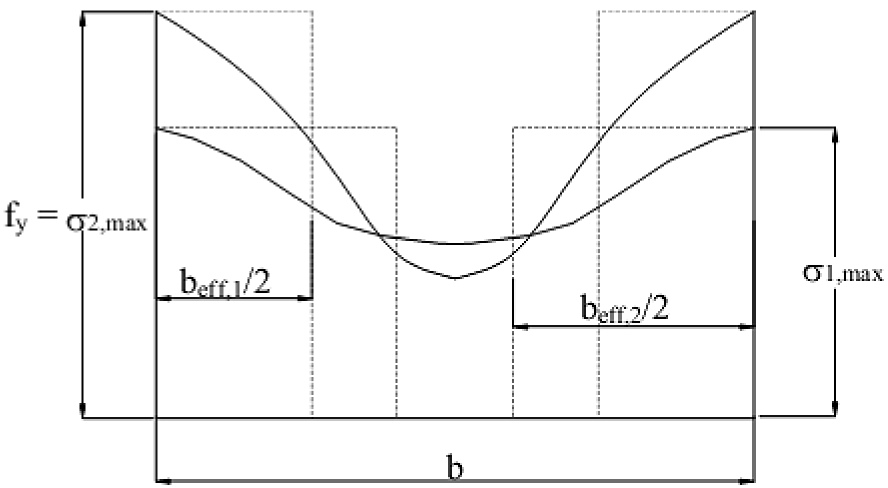
Behaviour and Resistance of Cross Section

Local Buckling

- **Elastic buckling of thin plates**



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

$$b_{eff} = \frac{\sqrt{k_\sigma} \cdot \pi}{\sqrt{12 \cdot (1-\nu^2)}} \cdot t \cdot \sqrt{\frac{E}{f_y}}$$

or

$$b_{eff} = C \cdot t \cdot \sqrt{\frac{E}{f_y}}$$

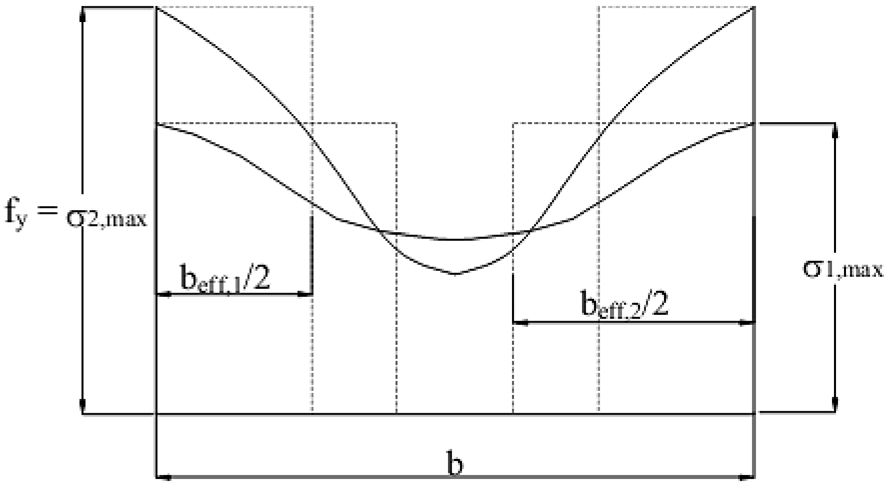
$$C = \sqrt{k_\sigma \cdot \pi^2 / 12 \cdot (1-\nu^2)}$$

Resistance of sections

Behaviour and Resistance of Cross Section

Local Buckling

- **Elastic buckling of thin plates**



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$$b_{eff} = C \cdot t \cdot \sqrt{\frac{E}{f_y}}$$

$$C = \sqrt{k_\sigma \cdot \pi^2 / 12 \cdot (1-\nu^2)}$$

$$\sigma_{cr} = \frac{k_\sigma \cdot \pi^2 \cdot E}{12 \cdot (1-\nu^2) \cdot (b/t)^2}$$

$$\frac{b_{eff}}{b} = \sqrt{\frac{\sigma_{cr}}{f_y}}$$

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{1.052}{\sqrt{k}} \cdot \frac{b}{t} \cdot \sqrt{\frac{f_y}{E}} = \frac{b/t}{28{,}4 \cdot \varepsilon \cdot \sqrt{k}}$$

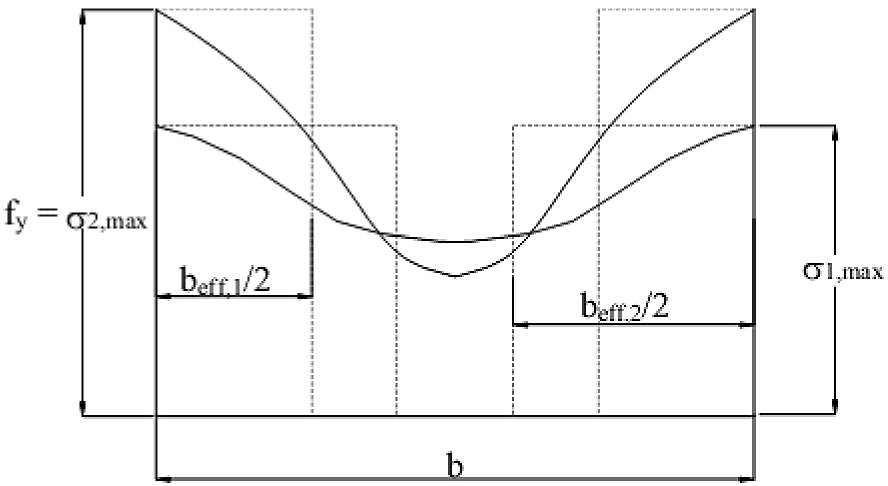
$$\varepsilon = \sqrt{235 / f_y}$$

Resistance of sections

Behaviour and Resistance of Cross Section

Local Buckling

- **Elastic buckling of thin plates**



$$b_{eff} = \rho \cdot b \qquad \rho = \frac{b_{eff}}{b} = \frac{1}{\bar{\lambda}_p} \leq 1$$

$$b_{eff} = C \cdot t \cdot \sqrt{\frac{E}{\sigma_{max}}} \text{ or } \frac{b_{eff}}{b} = \sqrt{\frac{\sigma_{cr}}{\sigma_{max}}} \qquad \bar{\lambda}_p = \sqrt{\frac{\sigma_{max}}{\sigma_{cr}}}$$

$$C = 1.9 \cdot \left[1 - 0.415 \cdot \left(\frac{t}{b} \right) \cdot \sqrt{\frac{E}{f_y}} \right]$$

Resistance of sections

Behaviour and Resistance of Cross Section

Local Buckling

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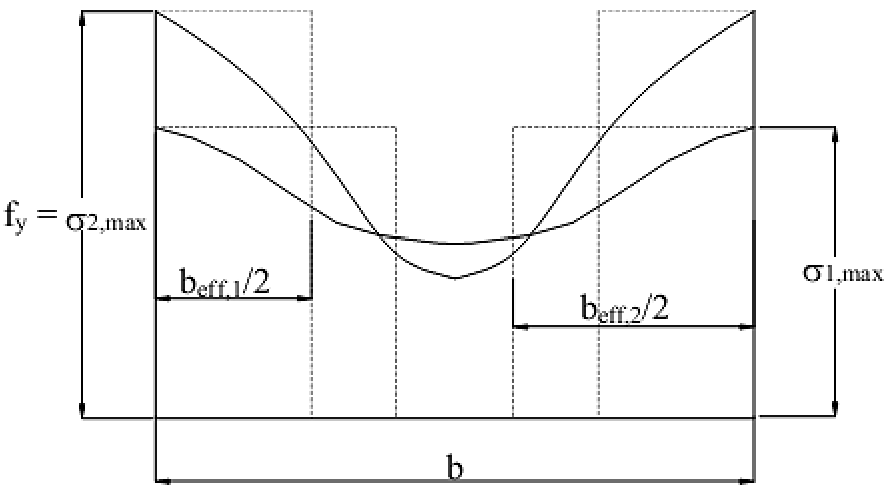

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$$C = 1.9 \cdot \left[1 - 0.415 \cdot \left(\frac{t}{b}\right) \cdot \sqrt{\frac{E}{f_y}}\right]$$

$$\rho = \frac{b_{eff}}{b} = \sqrt{\frac{\sigma_{cr}}{f_y}} \cdot \left(1 - 0.22 \cdot \sqrt{\frac{\sigma_{cr}}{f_y}}\right) \leq 1$$

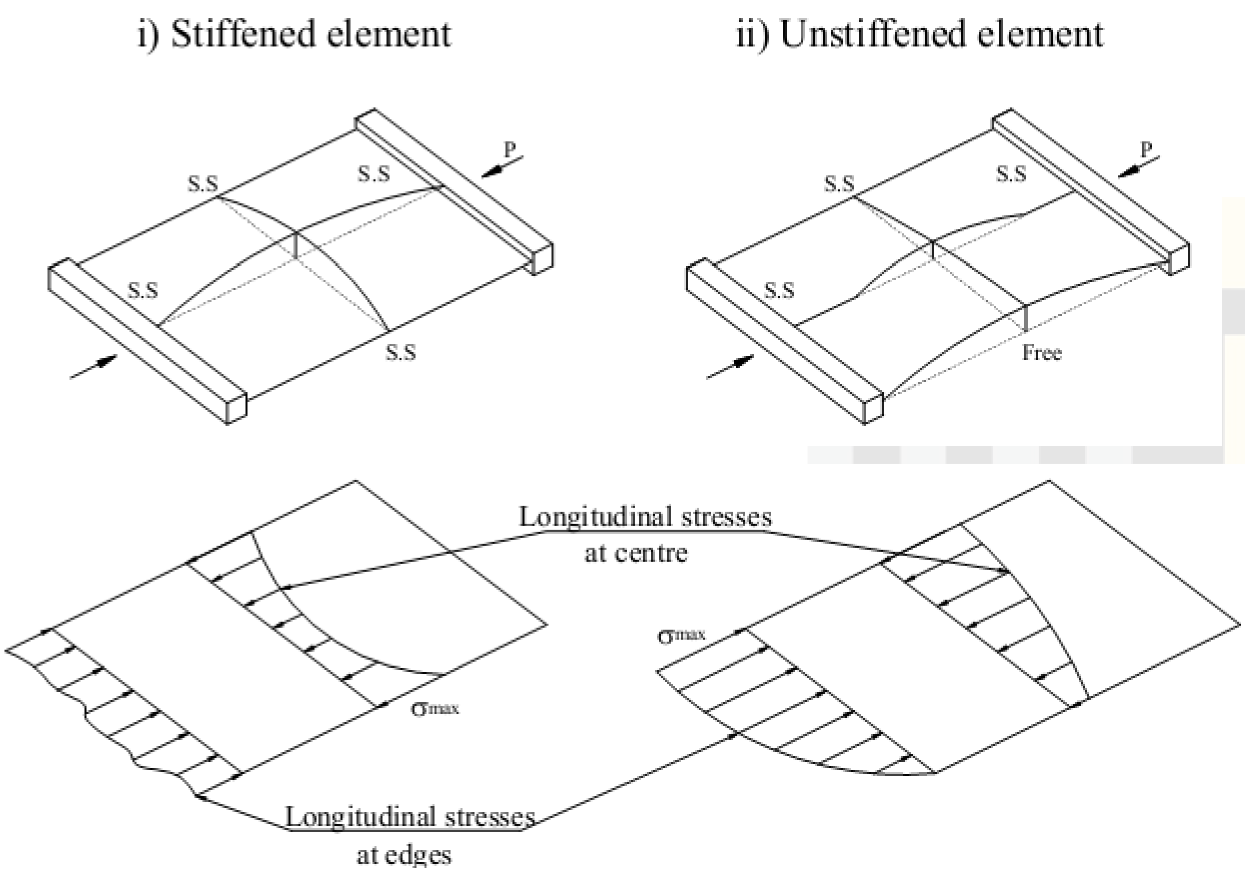
$$\text{or} \quad \rho = \frac{b_{eff}}{b} = \frac{1}{\bar{\lambda}_p} \cdot \left(1 - \frac{0.22}{\bar{\lambda}_p}\right)$$

Resistance of sections

Behaviour and Resistance of Cross Section

Local Buckling

- **Elastic buckling of thin plates**



Hancock, 1998

Resistance of sections

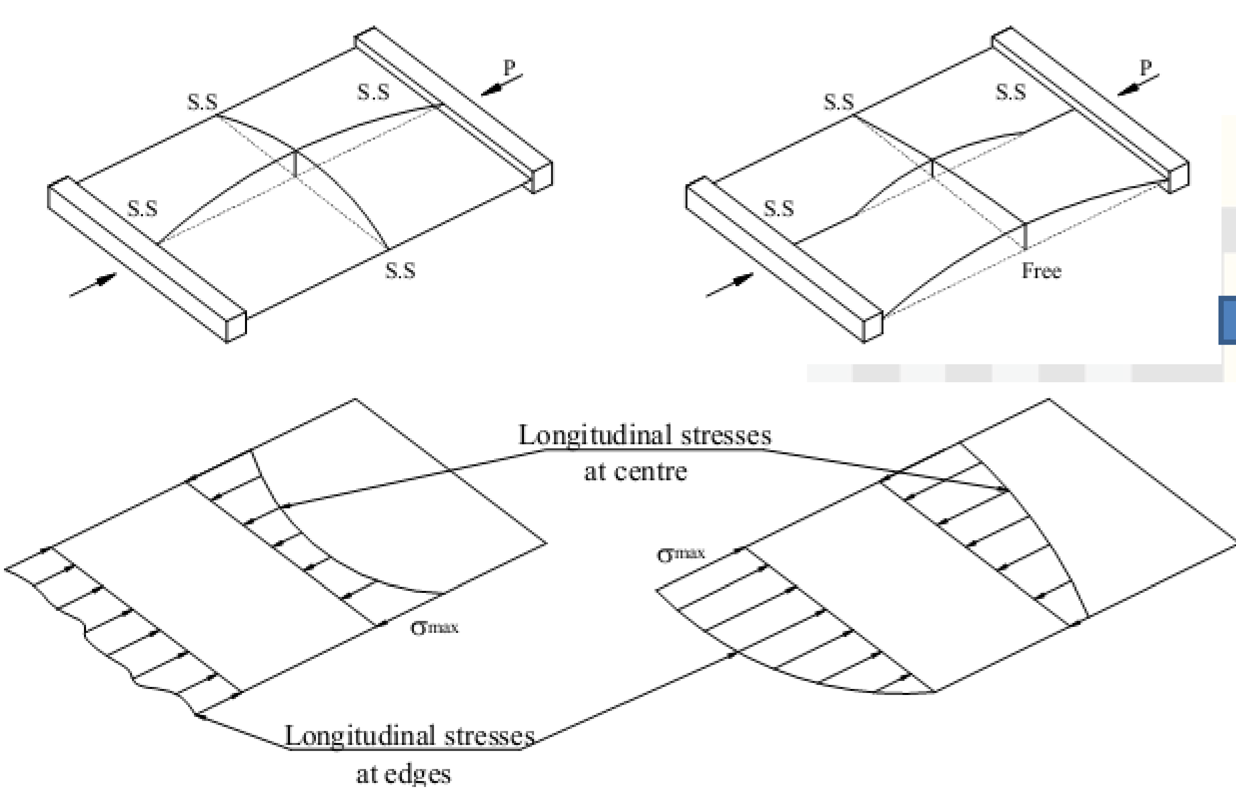
Behaviour and Resistance of Cross Section

Local Buckling

- **Elastic buckling of thin plates**

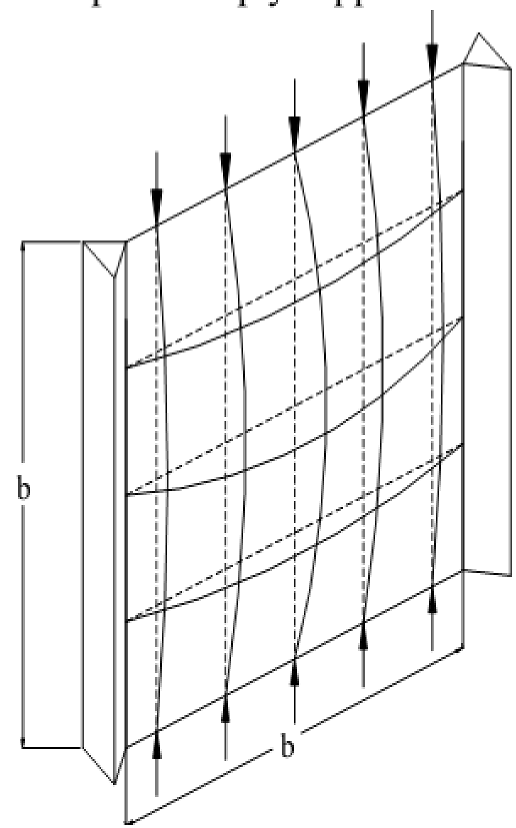
i) Stiffened element

ii) Unstiffened element



Hancock, 1998

"Strut and Tie" grid model of a plate simply supported



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Behaviour and Resistance of Cross Section

Local Buckling

- Elastic buckling of thin plates

$$\frac{b}{t} < \left(\frac{b}{t}\right)_{\lim} = 16.69 \cdot \varepsilon \cdot \sqrt{k_\sigma}$$

Resistance of sections

Behaviour and Resistance of Cross Section

Local Buckling

- **Elastic buckling of thin plates**

$$\frac{b}{t} < \left(\frac{b}{t}\right)_{\lim} = 16.69 \cdot \varepsilon \cdot \sqrt{k_\sigma}$$

$k_\sigma=4$ and $k_\sigma=0.425$

→

web type elements

$$\left(\frac{b}{t}\right)_{\lim} = 38.3 \cdot \varepsilon$$

flange type elements

$$\left(\frac{b}{t}\right)_{\lim} = 12.5 \cdot \varepsilon$$

Resistance of sections

Behaviour and Resistance of Cross Section

Local Buckling

- **Elastic buckling of thin plates**

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flange type elements

$$\left(\frac{b}{t}\right)_{\lim} = 12.5 \cdot \varepsilon$$

$(b/t)_{\lim}$ values for stiffened and unstiffened plate elements

Steel grade	f_y (N/mm^2)	Type of plate element: Stiffened	Type of plate element: Unstiffened
S235	235	38	12.5
S275	275	35	11.5
S355	355	31	10

Behaviour and Resistance of Cross Section

Local Buckling

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$$\frac{b}{t} < \left(\frac{b}{t}\right)_{\lim} = 16.69 \cdot \varepsilon \cdot \sqrt{k_\sigma}$$

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Steel grade	f_y (N/mm^2)	Type of plate element: Stiffened	Unstiffened
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S355	355	31	10

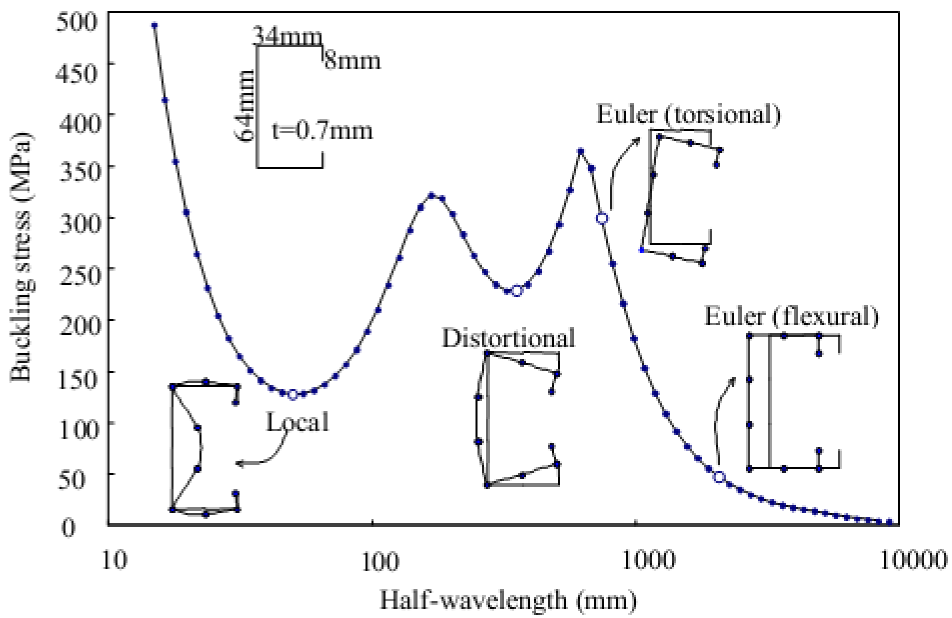
The effective or equivalent width method leads to simple design rules and gives an indication of the behaviour of the plate as the ultimate condition is approached.

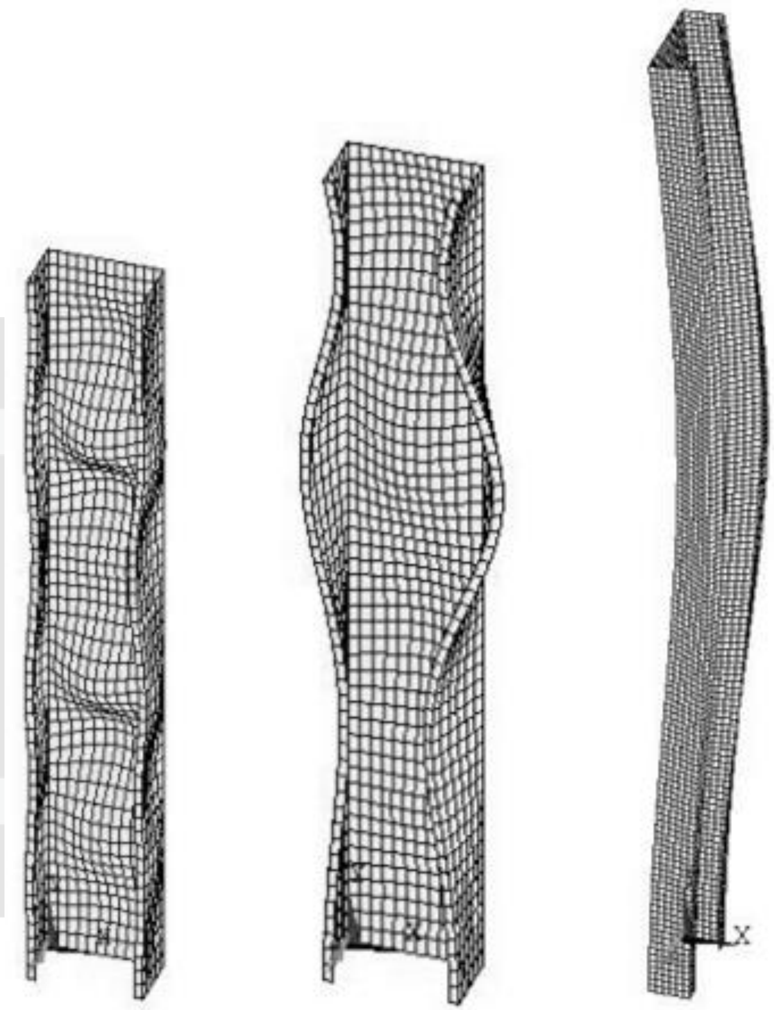
Resistance of sections

Behaviour and Resistance of Cross Section

Distortional buckling: analytical methods

Finite Strip Analysis by Schafer, 2001





Behaviour and Resistance of Cross Section

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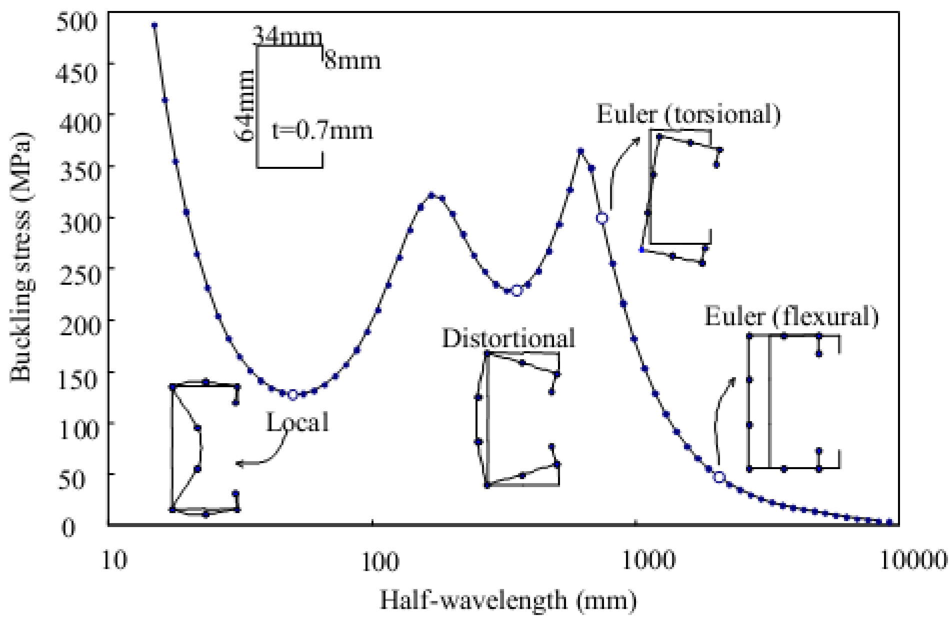

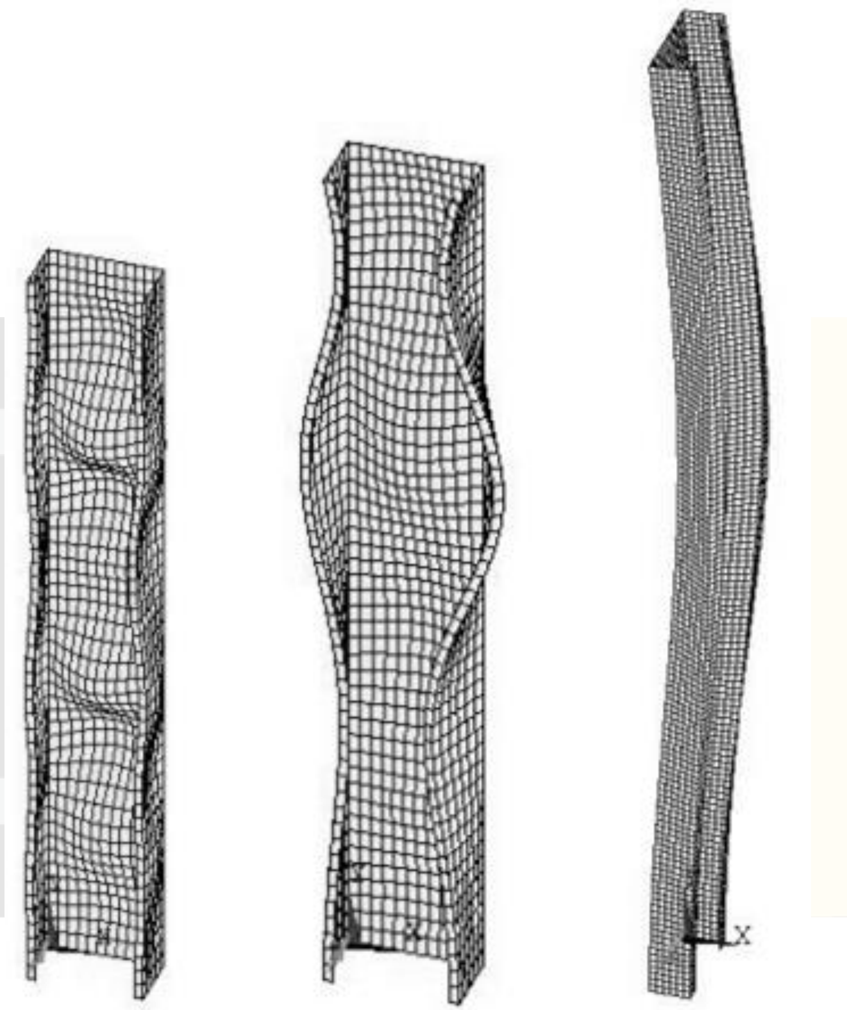

Manual calculation methods

Law & Hancock

Schafer & Peköz

Resistance of sections

Behaviour and Resistance of Cross Section

Distortional buckling: analytical methods

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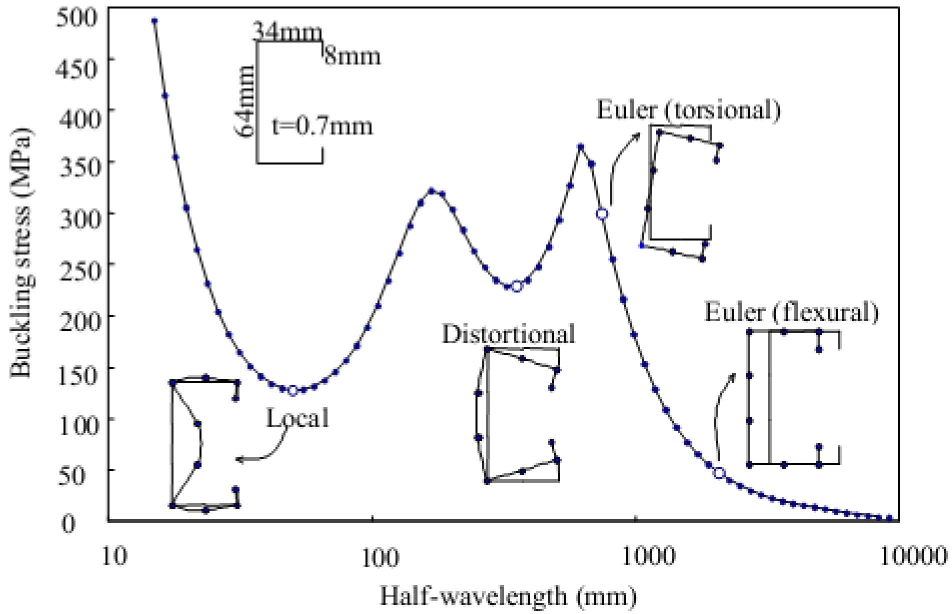
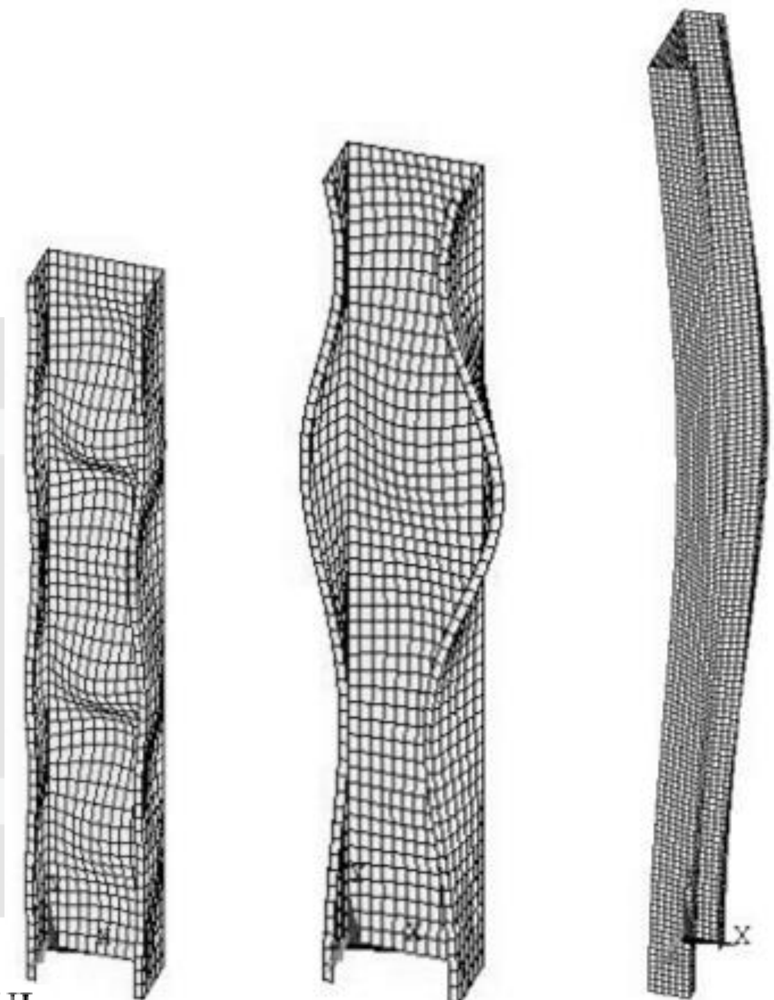




Manual calculation methods

- Law & Hancock
- Schafer & Peköz

Numerical methods

- The finite element method (FEM)
- Generalized Beam Theory (GBT) GBTUL
- The finite strip method (FSM) CUFSM

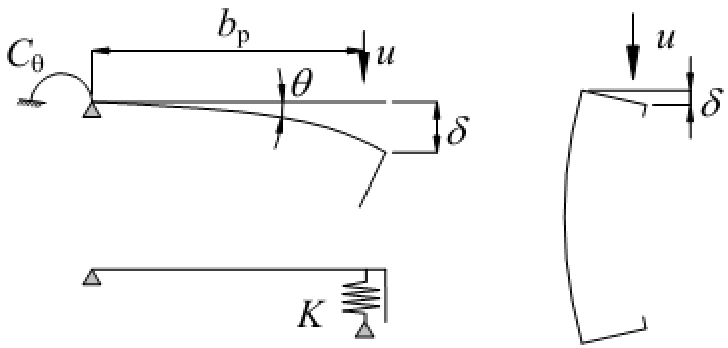
Resistance of sections

Behaviour and Resistance of Cross Section

Distortional buckling: analytical methods

- The method given in EN1993-1-3:2006

$$K = u / \delta$$



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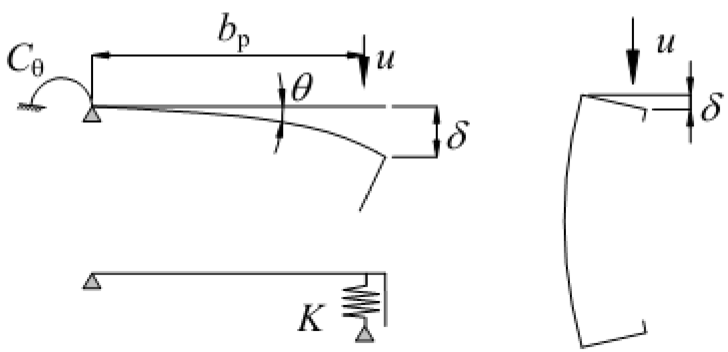
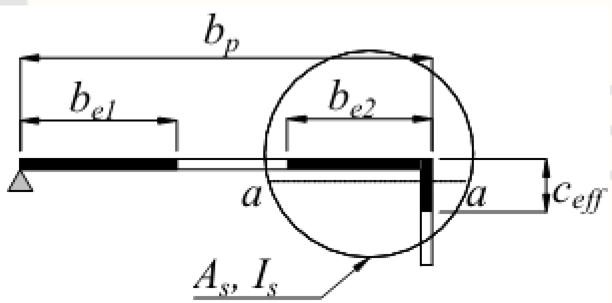


Timoshenko & Gere (1961)

$$\sigma_{cr} = \frac{\pi^2 \cdot E \cdot I_s}{A_s \cdot \lambda^2} + \frac{I}{A_s \cdot \pi^2} K \cdot \lambda^2$$



Behaviour and Resistance of Cross Section

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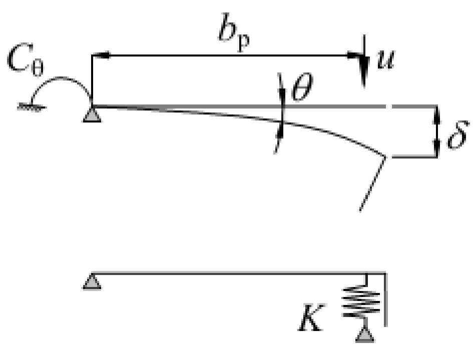
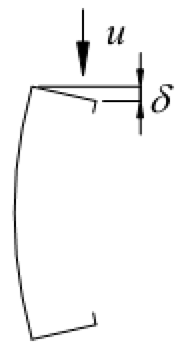
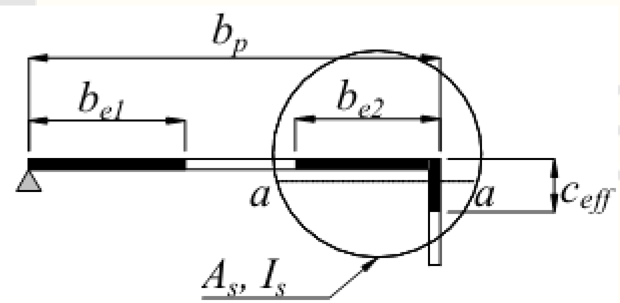




Timoshenko & Gere (1961)

$$\sigma_{cr} = \frac{\pi^2 \cdot E \cdot I_s}{A_s \cdot \lambda^2} + \frac{I}{A_s \cdot \pi^2} K \cdot \lambda^2$$



minimizing the critical stress

$$\lambda_{cr} = \sqrt[4]{\frac{E \cdot I_s}{K}}$$

EN1993-1-3

$$\sigma_{cr} = \frac{2 \cdot \sqrt{K \cdot E \cdot I_s}}{A_s}$$

Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

- **General**

According to EN1993-1-3:

(a) The effects of local and distortional buckling shall be taken into account in determining the resistance and stiffness of cold-formed members and sheeting;

(b) Local buckling effects may be accounted for by using effective cross sectional properties, calculated on the basis of the effective widths of those elements that are prone to local buckling;

(c) The possible shift of the centroidal axis of the effective cross section relative to the centroidal axis of the gross cross section shall be taken into account;

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

Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

- **General**

According to EN1993-1-3:

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

(f) Two cross sections are used in design: gross cross section and effective cross section of which the latter varies as a function of loading (compression, major axis bending etc.);

(g) 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;

(h) Distortional buckling shall be taken into account where it constitutes the critical failure mode.

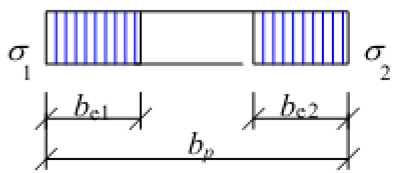
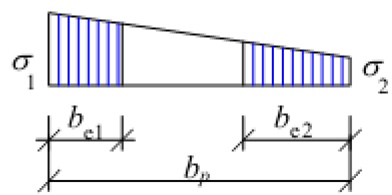
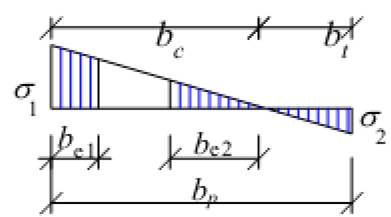
Resistance of sections

Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

- **Plane elements without stiffeners (no interaction)**

The effective widths of compression elements shall be obtained from

Stress distribution (compression positive)	Effective width b_{eff}
	$\psi = 1$ $b_{eff} = \rho \cdot b_p$ $b_{e1} = 0.5 \cdot b_{eff}; b_{e2} = 0.5 \cdot b_{eff}$
	$1 > \psi \geq 0$ $b_{eff} = \rho \cdot b_p$ $b_{e1} = \frac{2}{5-\psi} \cdot b_{eff}; b_{e2} = b_{eff} - b_{e1}$
	$\psi < 0$ $b_{eff} = \rho \cdot b_c = \rho b_p / (1-\psi)$ $b_{e1} = 0.4 \cdot b_{eff}; b_{e2} = 0.6 b_{eff}$

$\psi = \sigma_2 / \sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	$-1 > \psi \geq -3$
Buckling factor k_σ	4.0	$8.2/(1.05+\psi)$	7.81	$7.81 - 6.29\psi + 9.78\psi^2$	23.9	$5.98(1-\psi)^2$

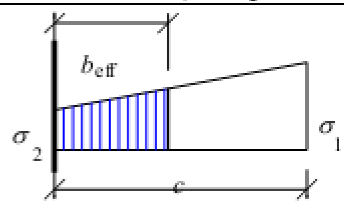
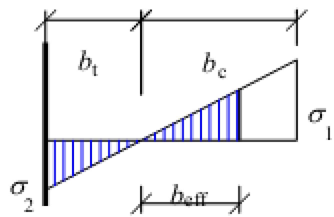
Internal compression elements

Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

- **Plane elements without stiffeners**

The effective widths of compression elements shall be obtained from

<table>
<tr><th colspan="3">Stress distribution (compression positive)</th><th colspan="2">Effective width b_{eff}</th></tr>
<tr><td colspan="3">
</td><td colspan="2">$1 > \psi \geq 0$
$b_{eff} = \rho \cdot c$</td></tr>
<tr><td colspan="3">
</td><td colspan="2">$\psi < 0$
$b_{eff} = \rho \cdot b_c = \rho c / (1 - \psi)$</td></tr>
<tr><td>$\psi = \sigma_2 / \sigma_1$</td><td>1</td><td>0</td><td>-1</td><td>$1 \geq \psi \geq -3$</td></tr>
<tr><td>Buckling factor k_σ</td><td>0.43</td><td>0.57</td><td>0.85</td><td>$0.57 - 0.21\psi + 0.07\psi^2$</td></tr>
</table>

Outstand compression elements

Resistance of sections

Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

- **Plane elements without stiffeners**

The effective widths of compression elements shall be obtained from

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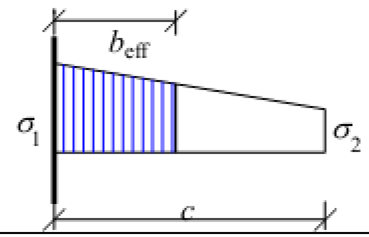
<tr><td colspan="3">
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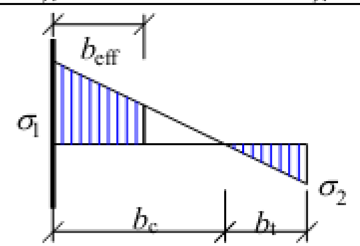
<tr><td colspan="3">
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<tr><td>$\psi = \sigma_2 / \sigma_1$</td><td>1</td><td>$1 > \psi > 0$</td><td>0</td><td>$0 > \psi > -1$</td><td>$-1$</td></tr>
<tr><td>Buckling factor k_σ</td><td>0.43</td><td>$0.578/(\psi + 0.24)$</td><td>1.70</td><td>$1.7 - 5\psi + 17.1\psi^2$</td><td>23.8</td></tr>
</table>

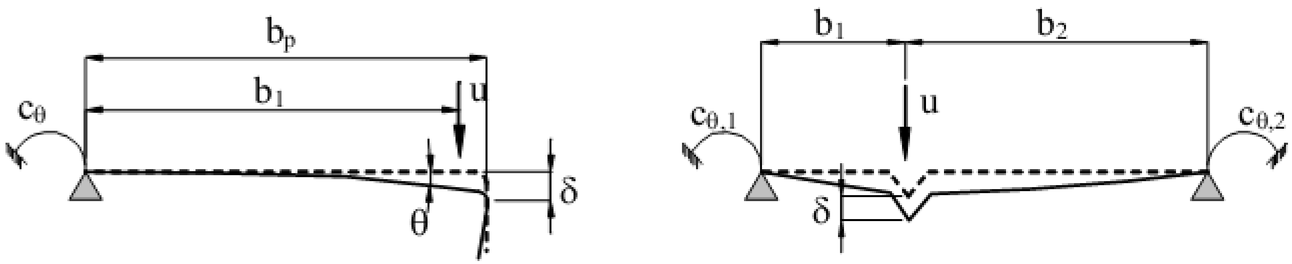
Outstand compression elements

Resistance of sections

Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

Plane elements with edge or intermediate stiffeners (elastic interaction)



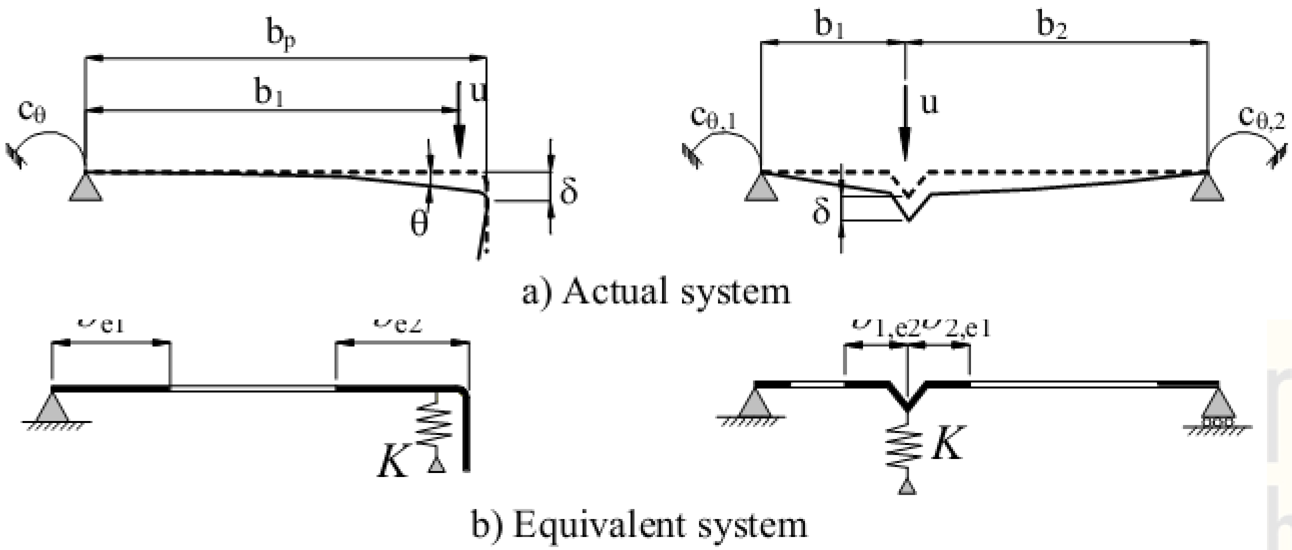
a) Actual system

Resistance of sections

Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

Plane elements with edge or intermediate stiffeners (elastic interaction)



Resistance of sections

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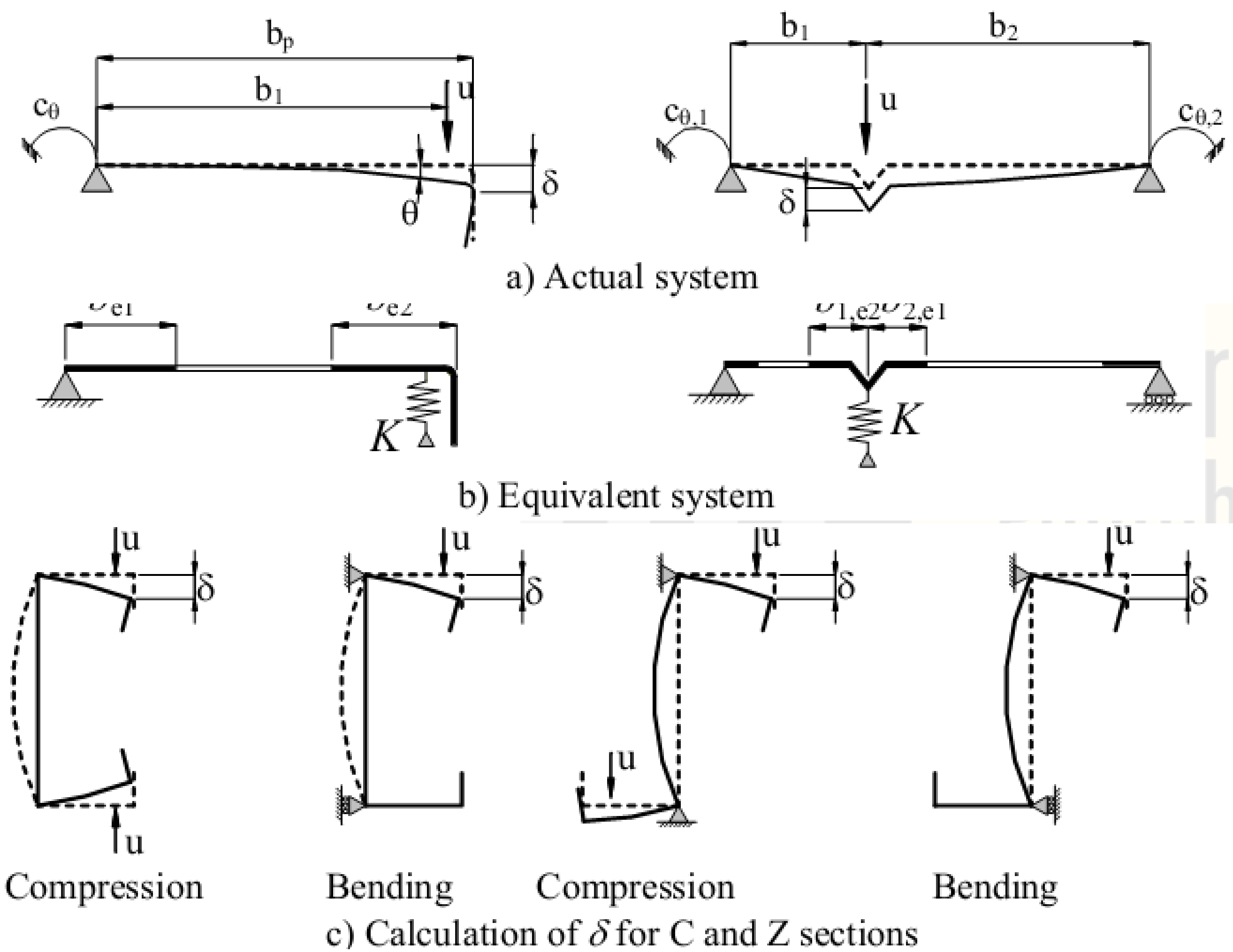

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Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

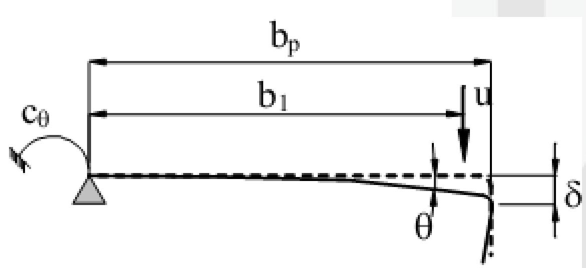
- **Plane elements with edge or intermediate stiffeners**

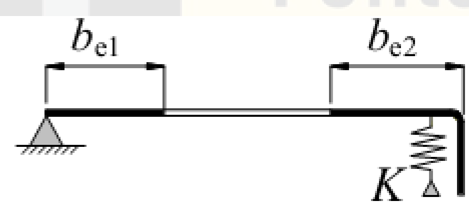
For an edge stiffener, the deflection δ should be obtained from:

$$\delta = \theta \cdot b_p + \frac{u \cdot b_p^3}{3} \cdot \frac{12 \cdot \left(1-\nu^2\right)}{E \cdot t^3} \qquad \text{with:} \quad \theta = u \cdot b_p / C_\theta$$

For the edge stiffeners of lipped C-sections and lipped Z- sections the spring stiffness

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





Resistance of sections

Behaviour and Resistance of Cross Section

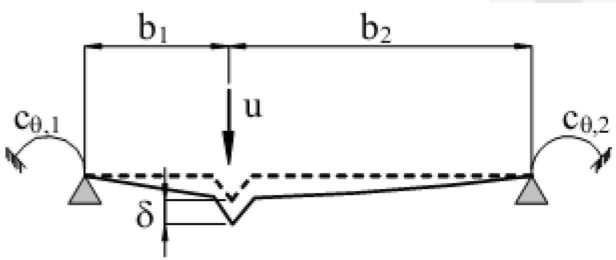
Design Against Local and Distortional Buckling (EN1993-1-3)

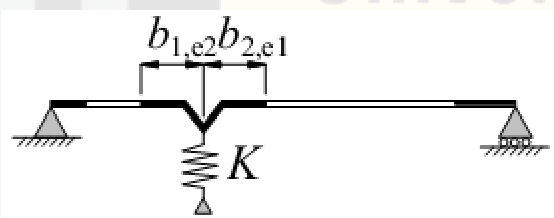
- **Plane elements with edge or intermediate stiffeners**

For an intermediate stiffener the deflection δ should be obtained from

$$\delta = \theta \cdot b_p + \frac{u \cdot b_1^2 \cdot b_2^2}{3 \cdot (b_1 + b_2)} \cdot \frac{12 \cdot (1 - \nu^2)}{E \cdot t^3}$$

$C_{\theta 1}$ and $C_{\theta 2}$ may conservatively be taken as equal to zero





Resistance of sections

Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

- **Plane elements with edge or intermediate stiffeners**

The reduction factor χ_d for the distortional buckling resistance

$\chi_d = 1.0$ if $\bar{\lambda}_d \leq 0.65$

$\chi_d = 1.47 - 0.723 \cdot \bar{\lambda}_d$ if $0.65 < \bar{\lambda}_d \leq 1.38$

$\chi_d = \dfrac{0.66}{\bar{\lambda}_d}$ if $\bar{\lambda}_d \geq 1.38$

$$\bar{\lambda}_d = \sqrt{f_{yb} / \sigma_{cr,s}}$$

$\sigma_{cr,s}$ is the elastic critical stress for the stiffener(s)

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Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

- **Plane elements with edge stiffeners – *conditions***

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

- the angle between the stiffener and the plane element is between 45° and 135°;
- the outstand width c is not less than $0.2b$, where b and c are as shown in Figure 3.35;
- the b/t ratio is not more than 60 for a single edge fold stiffener, or 90 for a double edge fold stiffener.

Resistance of sections

Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

- **Plane elements with edge stiffeners – *conditions***

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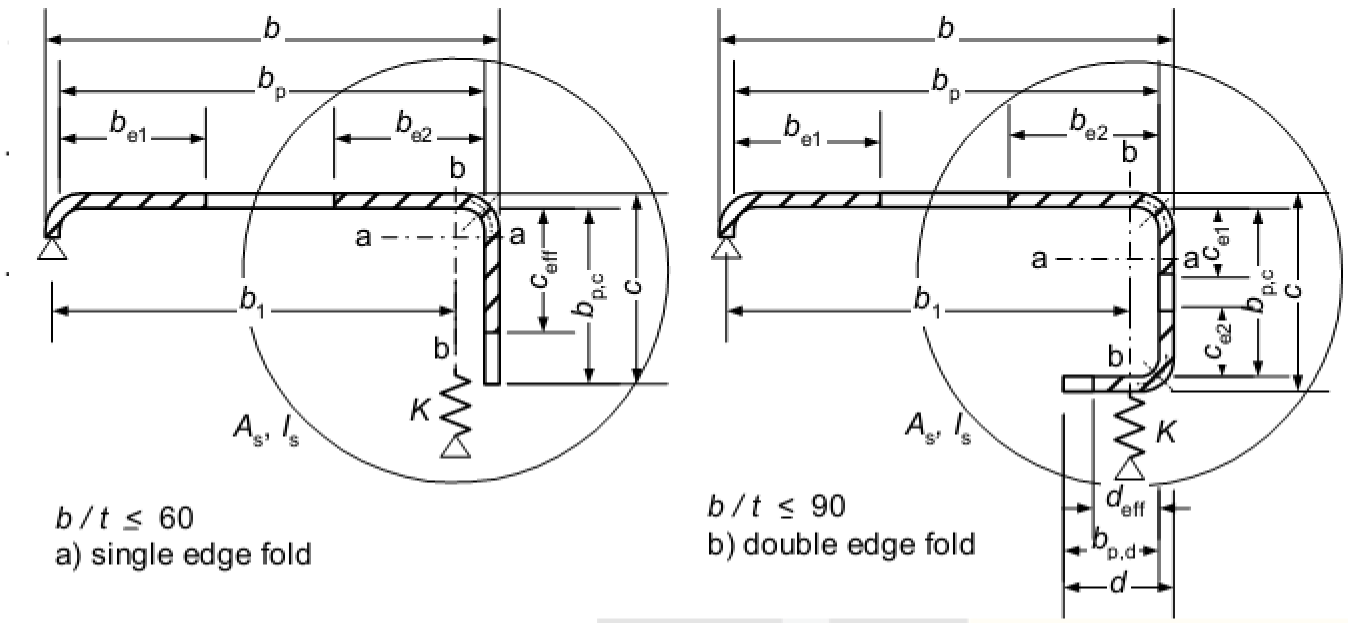

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Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

- **Plane elements with edge stiffeners – *general procedure***

Step 1: Obtain an initial effective cross section for the stiffener using effective widths determined by assuming that the stiffener gives full restraint and that $\sigma_{com,Ed} = f_{yb} / \gamma_{M0}$;

Step 2: Use the initial effective cross section of the stiffener to determine the reduction factor for distortional buckling (flexural buckling of the stiffener), 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.

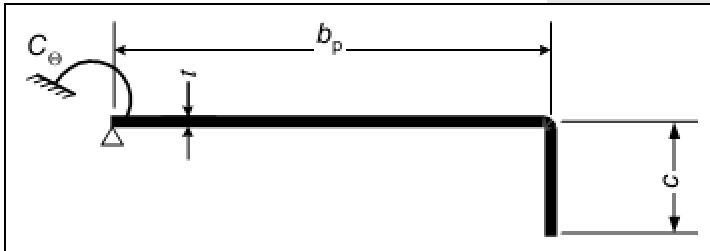
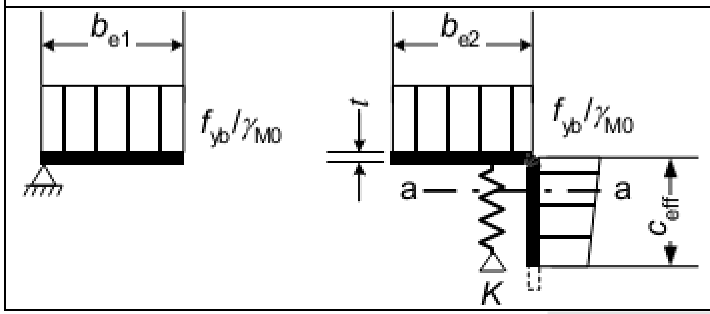
Resistance of sections

Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

- **Plane elements with edge stiffeners –** ***general procedure***

Step 1: Obtain an initial effective cross section for the stiffener using effective widths determined by assuming that the stiffener gives full restraint and that $\sigma_{com,Ed} = f_{yb} / \gamma_{M0}$;

	a) Gross cross section and boundary conditions
	b) **Step 1**: Effective cross section for $K = \infty$ based on $\sigma_{com,Ed} = f_{yb} / \gamma_{M0}$

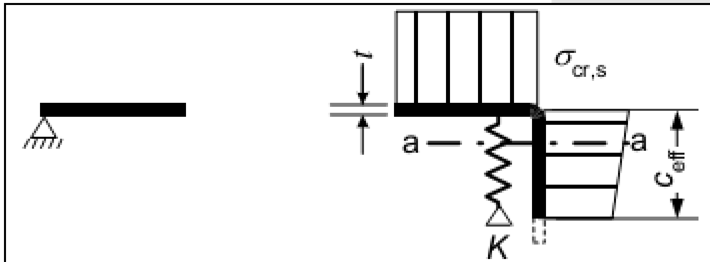
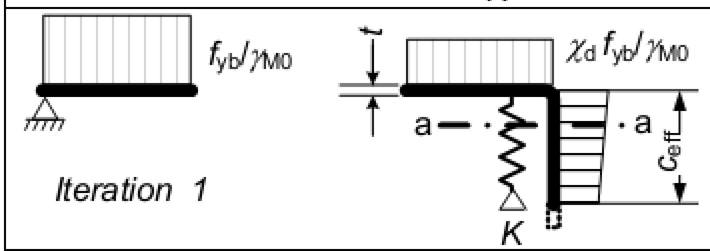
Resistance of sections

Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

- **Plane elements with edge stiffeners – *general procedure***

Step 2: Use the initial effective cross section of the stiffener to determine the reduction factor for distortional buckling (flexural buckling of the stiffener), allowing for the effects of the continuous spring restraint;

	c) **Step 2**: Elastic critical stress $\sigma_{cr,s}$ for effective area of stiffener A_s from Step 1 $\sigma_{cr} = \frac{2 \cdot \sqrt{K \cdot E \cdot I_s}}{A_s}$
	d) Reduced strength $\chi_d f_{yb}/\gamma_{M0}$ for effective area of stiffener A_s, with reduction factor χ_d based on $\sigma_{cr,s}$

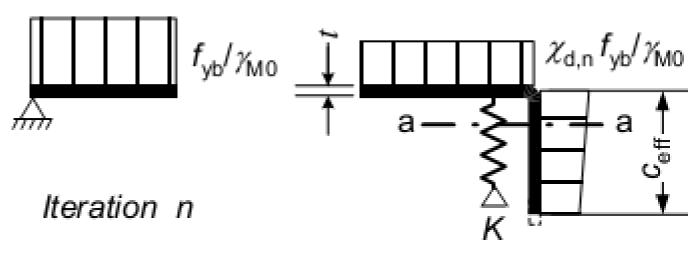
Resistance of sections

Behaviour and Resistance of Cross Section

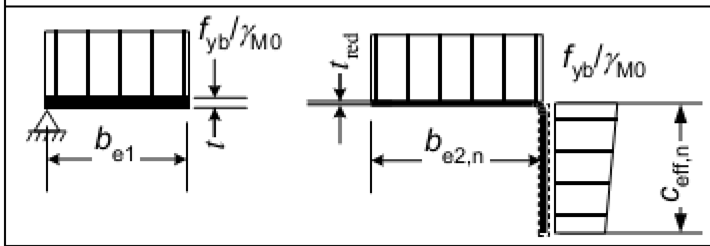
Design Against Local and Distortional Buckling (EN1993-1-3)

- **Plane elements with edge stiffeners – *general procedure***

Step 3: Optionally iterate to refine the value of the reduction factor for buckling of the stiffener.



e) **Step 3**: Optionally repeat Step 1 by calculating the effective width with a reduced compressive stress $\sigma_{com,Ed,i} = \chi_d f_{yb} / \gamma_{M0}$ with χ_d from previous iteration, continuing until $\chi_{d,n} \approx \chi_{d,(n-1)}$ but $\chi_{d,n} \leq \chi_{d,(n-1)}$



f) Adopt an effective cross section with b_{e2}, c_{eff} and reduced thickness t_{red} corresponding to $\chi_{d,n}$

Resistance of sections

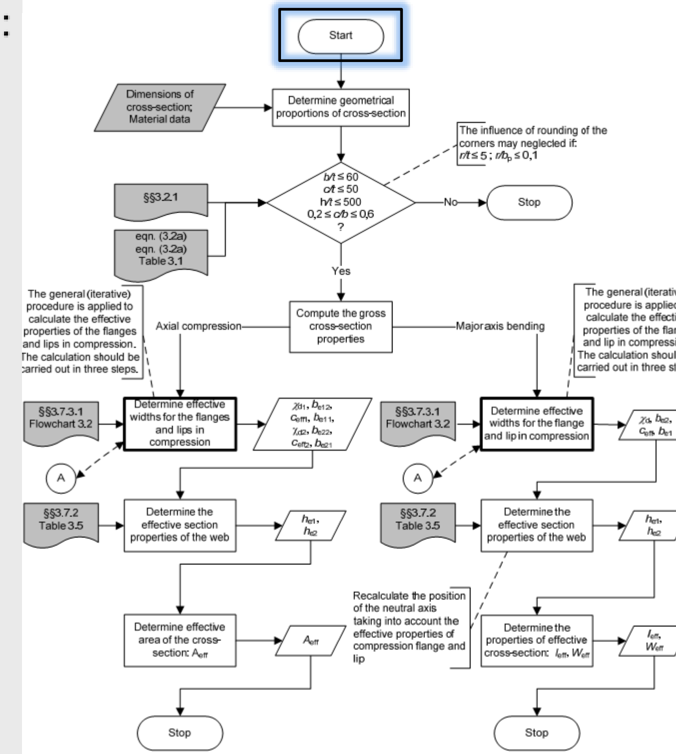
Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

- ***EXAMPLE 2: Calculation of effective section properties for a cold-formed lipped channel section in compression***

The dimensions of the cross section and the material properties are:

Total height	$h = 150$ mm
Total width of flange in compression	$b_1 = 47$ mm
Total width of flange in tension	$b_2 = 41$ mm
Total width of edge fold	$c = 16$ mm
Internal radius	$r = 3$ mm
Nominal thickness	$t_{nom} = 1$ mm
Steel core thickness (§§2.4.2.3)	$t = 0.96$ mm
Basic yield strength	$f_{yb} = 350\ \mathrm{N/mm^2}$
Modulus of elasticity	$E = 210000\ \mathrm{N/mm^2}$
Poisson's ratio	$\nu = 0.3$
Partial factor (§§2.3.1)	$\gamma_{M0} = 1.00$



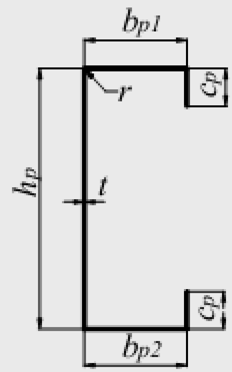
Resistance of sections

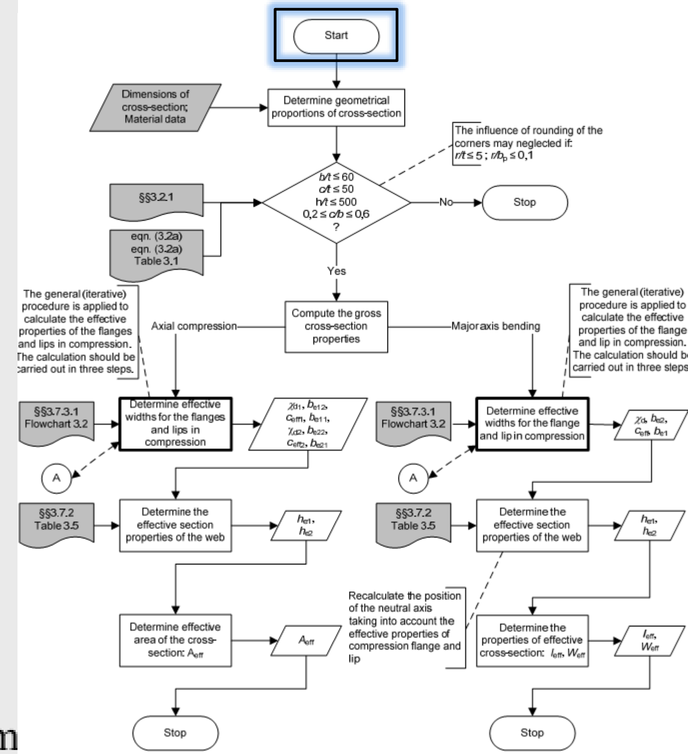
Behaviour and Resistance of Cross Section

EXAMPLE 2: CFS in compression

- ***EXAMPLE 2: Calculation of effective section properties for a cold-formed lipped channel section in compression***

The dimensions of the section centre line are:





Web height	$h_p = h - t_{nom} = 150 - 1 = 149$ mm
Width of flange in compression	$b_{p1} = b_1 - t_{nom} = 47 - 1 = 46$ mm
Width of flange in tension	$b_{p2} = b_2 - t_{nom} = 41 - 1 = 40$ mm
Width of edge fold	$c_p = c - t_{nom}/2 = 16 - 1/2 = 15.5$mm

Behaviour and Resistance of Cross Section

EXAMPLE 2: CFS in compression

- *Effective section properties of the flange and lip in compression*

Final values of effective properties for flange and lip in compression are:

For the upper flange and lip:

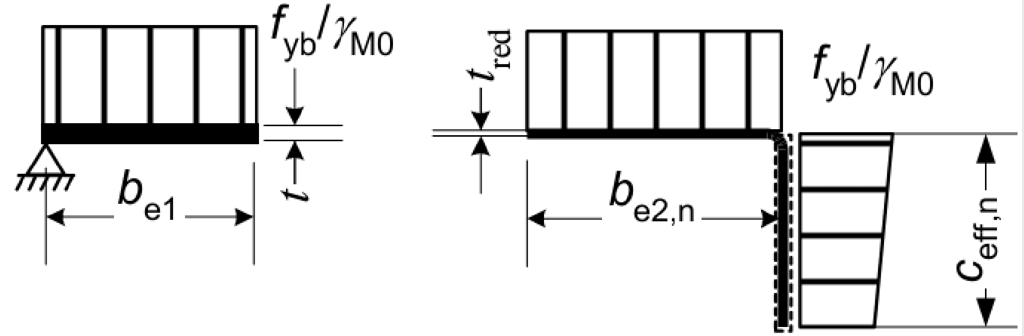
$$\chi_{d1} = 0.622\,,\ b_{e12} = 20.65\text{ mm}\,,\ c_{eff1} = 15.16\text{ mm and } b_{e11} = 17.57\text{ mm}$$

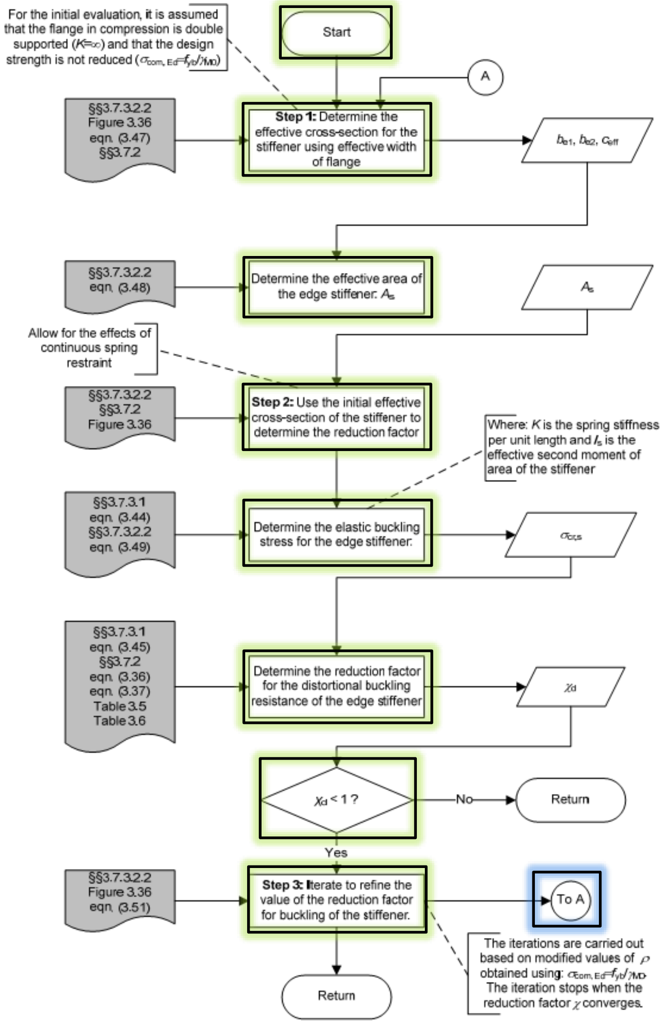
For the bottom flange and lip:

$$\chi_{d2} = 0.693\,,\ b_{e22} = 18.92\text{ mm}\,,\ c_{eff2} = 15.49\text{ mm and } b_{e21} = 16.86\text{ mm}$$

$$t_{red,1} = t\chi_{d1} = 0.96 \times 0.622 = 0.597\text{ mm}$$

$$t_{red,2} = t\chi_{d2} = 0.96 \times 0.693 = 0.665\text{ mm}$$





Resistance of sections

Behaviour and Resistance of Cross Section

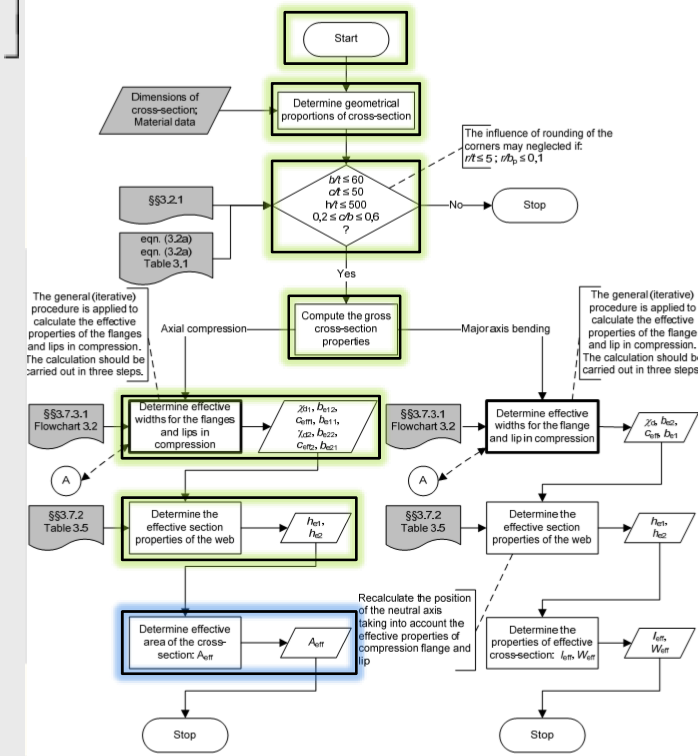
EXAMPLE 2: CFS in compression

- *Effective section properties*

Effective cross section area:

$$A_{eff} = t\left[b_{e11} + b_{e21} + h_{e1} + h_{e2} + \left(b_{e12} + c_{eff1}\right)\chi_{d1} + \left(b_{e22} + c_{eff2}\right)\chi_{d2}\right]$$

$$A_{eff} = 117.37\ \text{mm}^2$$



Resistance of sections

Behaviour and Resistance of Cross Section

EXAMPLE 2: CFS in compression

- ***Effective section properties***

Effective cross section area:

$$A_{eff} = t\left[b_{e11} + b_{e21} + h_{e1} + h_{e2} + \left(b_{e12} + c_{eff1}\right)\chi_{d1} + \left(b_{e22} + c_{eff2}\right)\chi_{d2}\right]$$

$$A_{eff} = 117.37 \text{ mm}^2$$

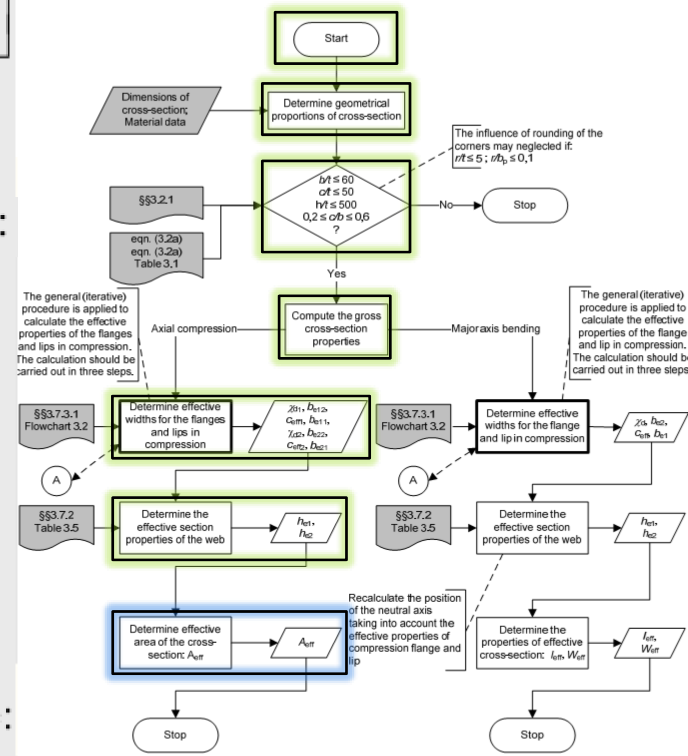
Position of the centroidal axis with regard to the upper flange:

$$z_{G1} = \frac{t\left[c_{eff2}\chi_{d2}\left(h_p - \frac{c_{eff2}}{2}\right) + h_p\left(b_{e22}\chi_{d2} + b_{e21}\right) + \right.}{A_{eff}}$$

$$+ \frac{\left. h_{e2}\left(h_p - \frac{h_{e2}}{2}\right) + \frac{h_{e1}^{\,2}}{2} + \frac{c_{eff1}^{\,2}\chi_{d1}}{2}\right]}{A_{eff}} = 74.92 \text{ mm}$$

Position of the centroidal axis with regard to the bottom flange:

$$z_{G2} = h_p - z_{G1} = 149 - 74.92 = 74.08 \text{ mm}$$



Behaviour and Resistance of Cross Section

Design Against Local and Distortional Buckling (EN1993-1-3)

- ***EXAMPLE 1: Calculation of effective section properties for a cold-formed lipped channel section in bending***

The dimensions of the cross section and the material properties are:

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Total width of flange in compression	$b_1 = 47$ mm
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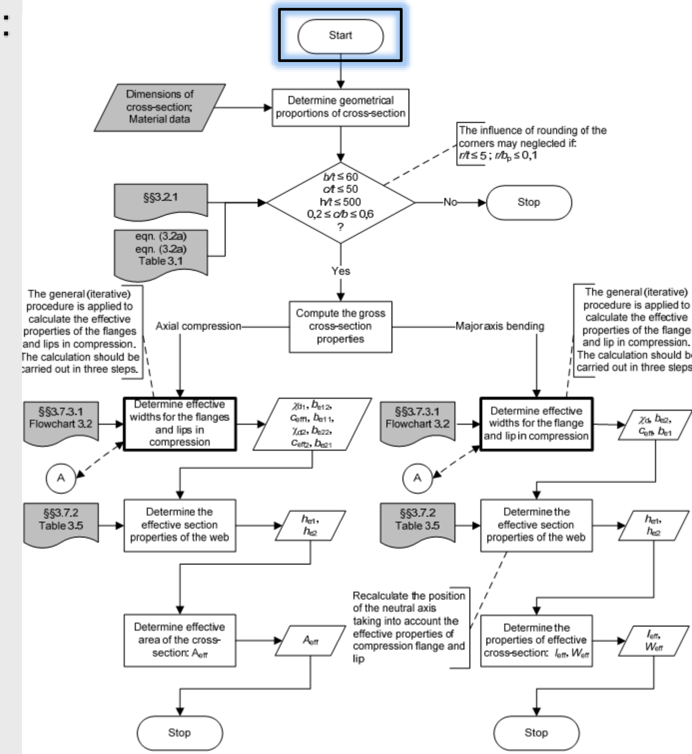
Total width of edge fold	$c = 16$ mm
Internal radius	$r = 3$ mm
Nominal thickness	$t_{nom} = 1$ mm
Steel core thickness (§§2.4.2.3)	$t = 0.96$ mm
Basic yield strength	$f_{yb} = 350\ \text{N/mm}^2$
Modulus of elasticity	$E = 210000\ \text{N/mm}^2$
Poisson's ratio	$\nu = 0.3$
Partial factor (§§2.3.1)	$\gamma_{M0} = 1.00$

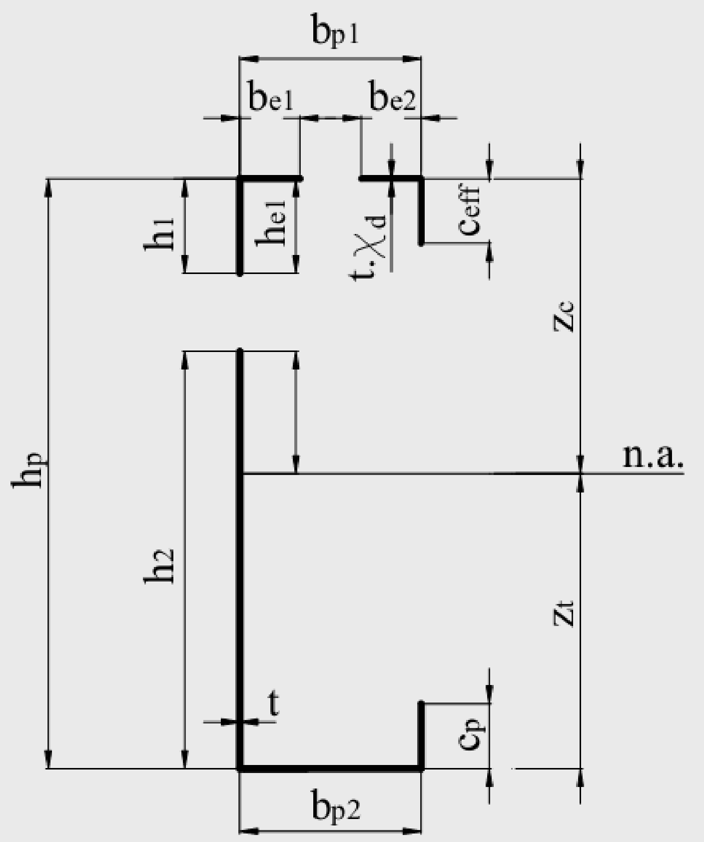


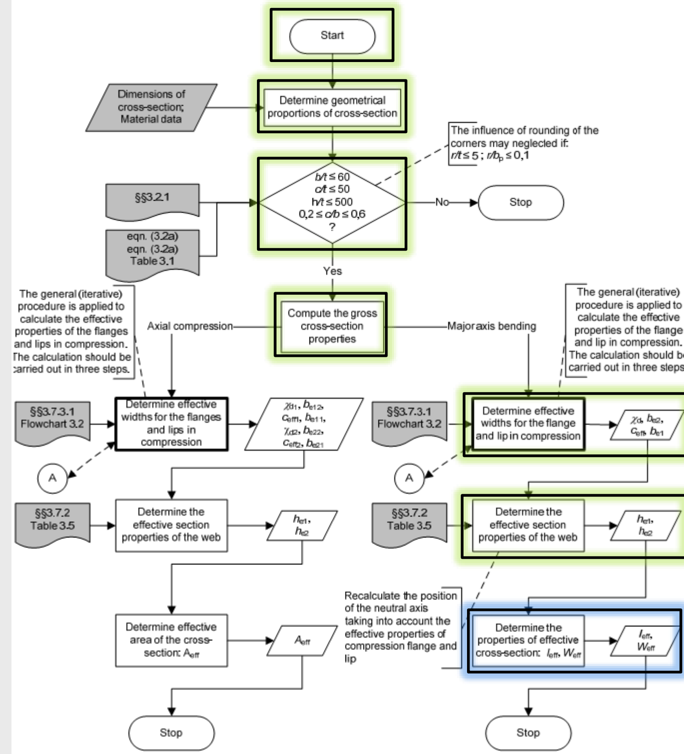
Behaviour and Resistance of Cross Section

EXAMPLE 1: CFS in bending

- *Effective section properties*

Effective cross section area:





Behaviour and Resistance of Cross Section

EXAMPLE 1: CFS in bending

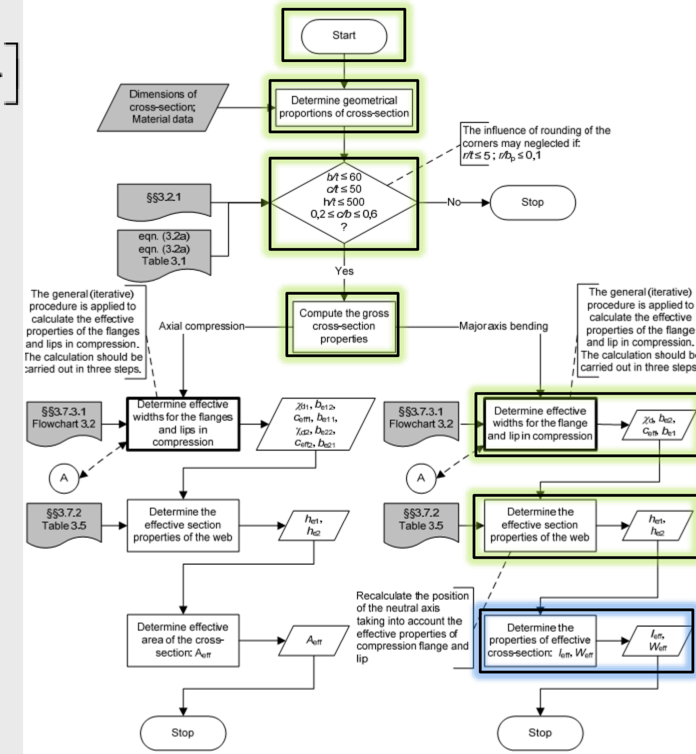
- *Effective section properties*

Effective cross section area:

$$A_{eff} = t[c_p + b_{p2} + h_1 + h_2 + b_{e1} + (b_{e2} + c_{eff})\chi_d]$$

$$A_{eff} = 0.96 \times \left[15.5 + 40 + 20 + 99.5 + 17.57 + (20.736 + 12.77) \times 0.614\right]$$

$$A_{eff} = 204.62\ \text{mm}^2$$



Behaviour and Resistance of Cross Section

EXAMPLE 1: CFS in bending

- *Effective section properties*

Effective cross section area:

$$A_{eff} = t[c_p + b_{p2} + h_1 + h_2 + b_{e1} + (b_{e2} + c_{eff})\chi_d]$$

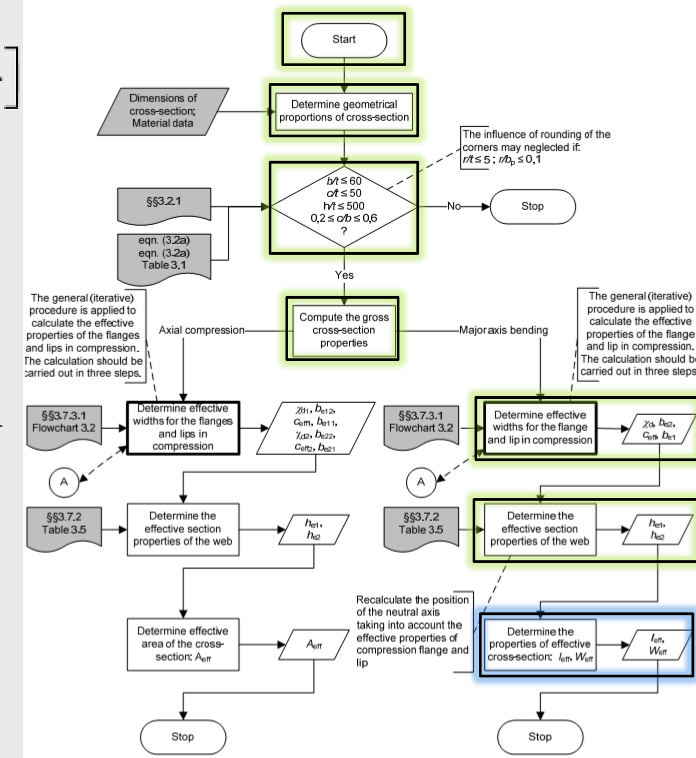
$$A_{eff} = 0.96 \times \left[15.5 + 40 + 20 + 99.5 + 17.57 + (20.736 + 12.77) \times 0.614\right]$$

$$A_{eff} = 204.62 \text{ mm}^2$$

Position of the neutral axis with regard to the flange in compression:

$$z_c = \frac{t\left[c_p\left(h_p - c_p/2\right) + b_{p2}h_p + h_2\left(h_p - h_2/2\right) + h_1^2/2 + c_{eff}^{\ 2}\chi_d/2\right]}{A_{eff}}$$

$$z_c = 85.75 \text{ mm}$$



Behaviour and Resistance of Cross Section

EXAMPLE 1: CFS in bending

- *Effective section properties*

Effective cross section area:

$$A_{eff} = t[c_p + b_{p2} + h_1 + h_2 + b_{e1} + (b_{e2} + c_{eff})\chi_d]$$

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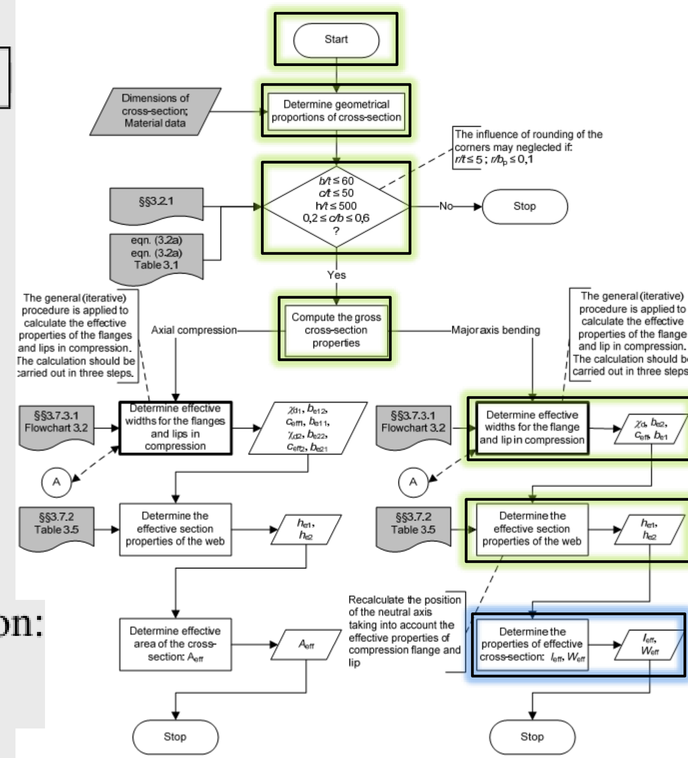
Position of the neutral axis with regard to the flange in compression:

$$z_c = \frac{t\left[c_p\left(h_p - c_p/2\right) + b_{p2}h_p + h_2\left(h_p - h_2/2\right) + h_1^2/2 + c_{eff}{}^2\chi_d/2\right]}{A_{eff}}$$

$$z_c = 85.75 \text{ mm}$$

Position of the neutral axis with regard to the flange in tension:

$$z_t = h_p - z_c = 149 - 85.75 = 63.25 \text{mm}$$



Behaviour and Resistance of Cross Section

EXAMPLE 1: CFS in bending

- *Effective section properties*

Second moment of area:

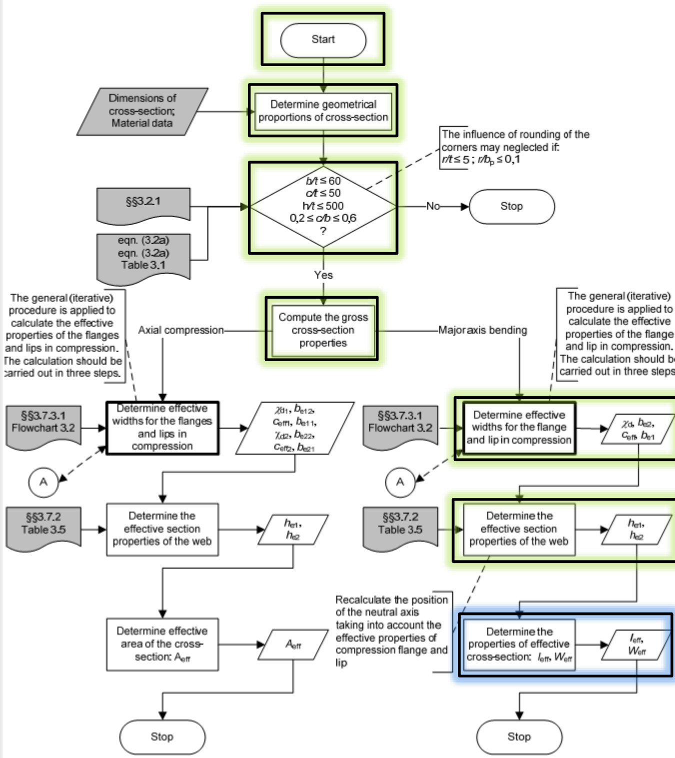
$$I_{eff,y} = \frac{h_1^{\,3}t}{12} + \frac{h_2^{\,3}t}{12} + \frac{b_{p2}t^3}{12} + \frac{c_p^{\,3}t}{12} + \frac{b_{e1}t^3}{12} + \frac{b_{e2}(\chi_d t)^3}{12} +$$

$$+ \frac{c_{eff}^{\,3}(\chi_d t)}{12} + c_p t(z_t - c_p/2)^2 + b_{p2} t z_t^{\,2} + h_2 t(z_t - h_2/2)^2 +$$

$$+ h_1 t(z_c - h_1/2)^2 + b_{e1} t\, z_c^{\,2} + b_{e2}(\chi_d t) z_c^{\,2} +$$

$$+ c_{eff}(\chi_d t)(z_c - c_{eff}/2)^2$$

$$I_{eff,y} = 668103 \text{ mm}^4$$



Behaviour and Resistance of Cross Section

EXAMPLE 1: CFS in bending

- *Effective section properties*

Second moment of area:

$$I_{eff,y} = \frac{h_1{}^3 t}{12} + \frac{h_2{}^3 t}{12} + \frac{b_{p2} t^3}{12} + \frac{c_p{}^3 t}{12} + \frac{b_{e1} t^3}{12} + \frac{b_{e2}(\chi_d t)^3}{12} +$$

$$+ \frac{c_{eff}{}^3(\chi_d t)}{12} + c_p t(z_t - c_p/2)^2 + b_{p2} t z_t{}^2 + h_2 t(z_t - h_2/2)^2 +$$

$$+ h_1 t(z_c - h_1/2)^2 + b_{e1} t\, z_c{}^2 + b_{e2}(\chi_d t) z_c{}^2 +$$

$$+ c_{eff}(\chi_d t)(z_c - c_{eff}/2)^2$$

$$I_{eff,y} = 668103 \text{ mm}^4$$

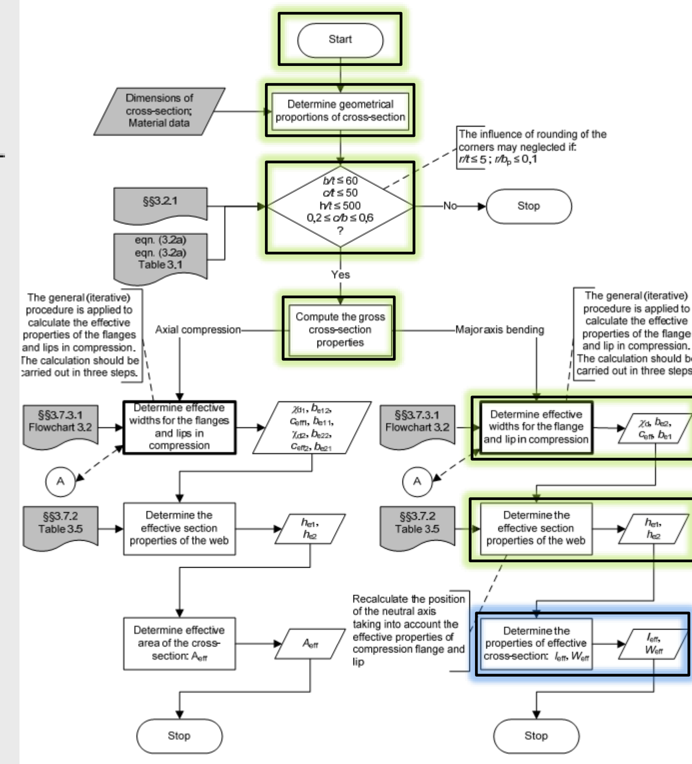
Effective section modulus:

- with regard to the flange in compression

$$W_{eff,y,c} = \frac{I_{eff,y}}{z_c} = \frac{668103}{85.75} = 7791 \text{ mm}^3$$

- with regard to the flange in tension

$$W_{eff,y,t} = \frac{I_{eff,y}}{z_t} = \frac{668103}{63.25} = 10563 \text{ mm}^3$$



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Behaviour and Resistance of Cross Section

Resistance of Cross Sections

- ***Bending Moment - Elastic and elastoplastic resistance with yielding at the tension flange only***

Provided that bending moment is applied only about one principal axis of the cross section, and provided that yielding occurs first at the tension edge, plastic reserves in the tension zone may be utilized without any strain limit until the maximum compressive stress $\sigma_{com,Ed}$ reaches f_{yb}/γ_{M0}. In this paragraph the pure bending case is considered. For combined axial load and

Resistance of sections

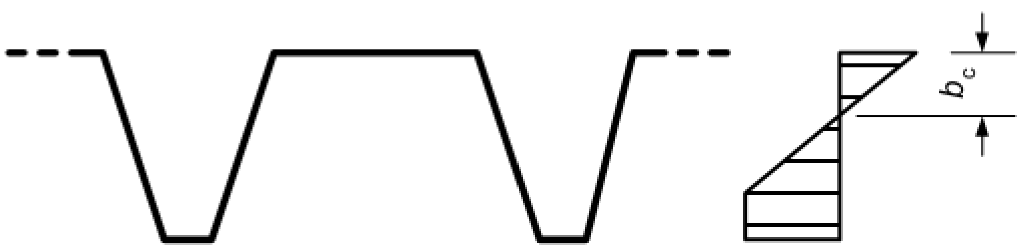
Behaviour and Resistance of Cross Section

Resistance of Cross Sections

- ***Bending Moment - Elastic and elastoplastic resistance with yielding at the tension flange only***

Provided that bending moment is applied only about one principal axis of the cross section, and provided that yielding occurs first at the tension edge, plastic reserves in the tension zone may be utilized without any strain limit until the maximum compressive stress $\sigma_{com,Ed}$ reaches f_{yb}/γ_{M0}. In this paragraph the pure bending case is considered. For combined axial load and

When accounting for plastic reserve capacity, the effective partially plastic section modulus $W_{pp,eff}$ should be based on a stress distribution that is bilinear in the tension zone but linear in the compression zone.



Behaviour and Resistance of Cross Section

Resistance of Cross Sections – *Bending Moment*

- ***Example – Design of a cold-formed steel member in bending***

Basic Data

Span of joist $L = 5.5\ \text{m}$

Spacing between joists $S = 0.6\ \text{m}$

Distributed loads applied to the joist:

self-weight of the beam $q_{G,beam} = 0.06\ \text{kN/m}$

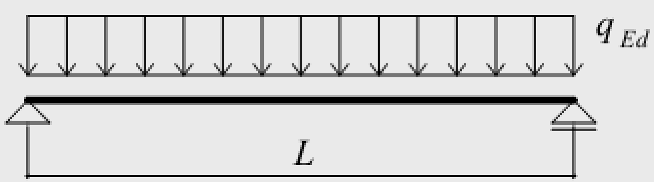
lightweight slab $0.75\ \text{kN/m}^2$

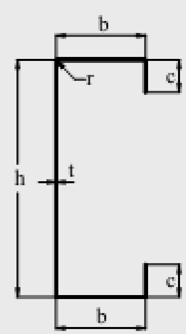
$$q_{G,slab} = 0.75 \times 0.6 = 0.45\ \text{kN/m}$$

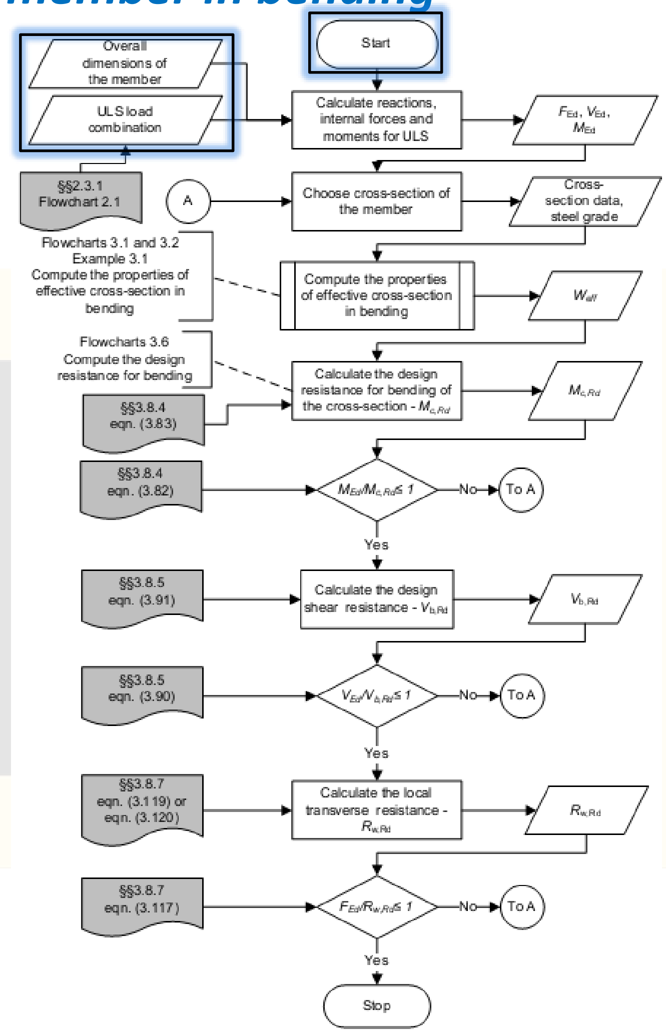
dead load $q_G = q_{G,beam} + q_{G,slab} = 0.51\ \text{kN/m}$

imposed load $2.50\ \text{kN/m}^2$

$$q_Q = 2.50 \times 0.6 = 1.50\ \text{kN/m}$$







Resistance of sections

Behaviour and Resistance of Cross Section

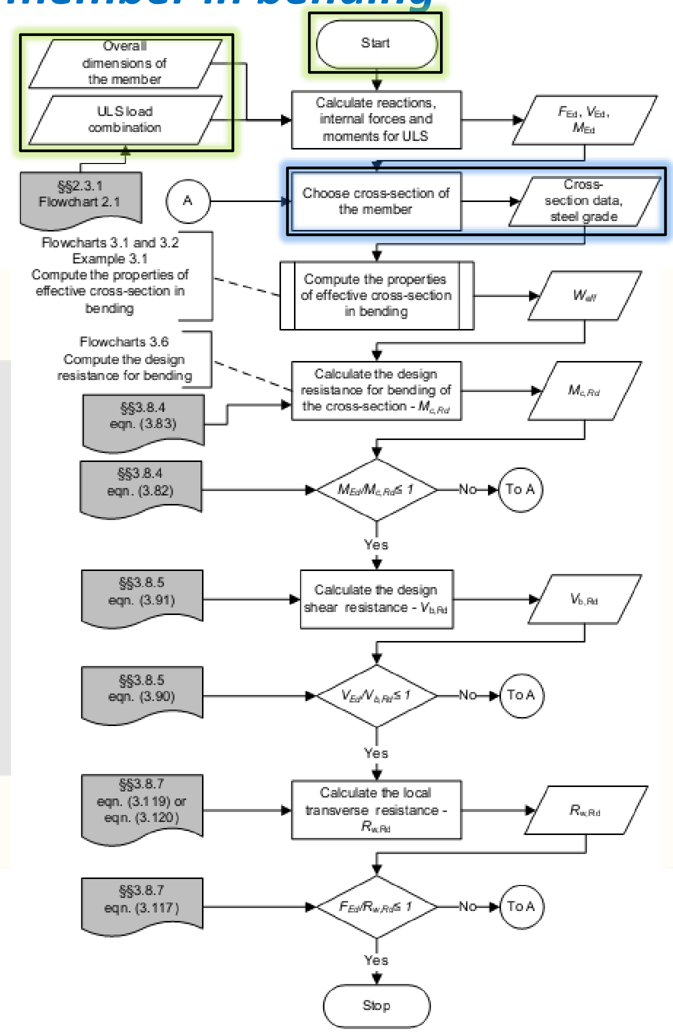
Resistance of Cross Sections – *Bending Moment*

- ***Example – Design of a cold-formed steel member in bending***

Basic Data

The dimensions of the cross section and the material properties are:

Total height	$h = 200$ mm
Total width of flange in compression	$b_1 = 74$ mm
Total width of flange in tension	$b_2 = 66$ mm
Total width of edge fold	$c = 20.8$ mm
Internal radius	$r = 3$ mm
Nominal thickness	$t_{nom} = 2$ mm
Steel core thickness	$t = 1.96$ mm
Basic yield strength	$f_{yb} = 350$ N/mm^2
Modulus of elasticity	$E = 210000$ N/mm^2
Poisson's ratio	$\nu = 0.3$
Partial factors	$\gamma_{M0} = 1.0$
	$\gamma_{M1} = 1.0$
	$\gamma_G = 1.35$ – permanent loads
	$\gamma_Q = 1.50$ – variable loads



Resistance of sections

Behaviour and Resistance of Cross Section

Resistance of Cross Sections

Lipped Channel Beam

- **Calculation of geometrical properties of effective section : $b_{eff,i}$; $A_{eff,i}$ $I_{eff,i}$, $W_{eff,i}$**
- **Cecking for Bending moment, M_{Rd}**
- **Checking for Shear, V_{Rd}**
- **Checking for transverse force (web crippling), R_{Rd}**
- **Checking for interaction M_{Rd} +V_{Rd}**
- **Checking for interaction M_{Rd} +R_{Rd}**

Resistance of sections

Behaviour and Resistance of Cross Section

Resistance of Cross Sections – *Bending Moment*

- ***Example – Design of a cold-formed steel member in bending***

Effective section properties at the ULS

Second moment of area of cold-formed lipped channel section subjected to bending about its major axis:

$$I_{eff,y} = 4139861 \text{ mm}^4$$

Position of the neutral axis:

- from the flange in compression: $z_c = 102.3$ mm
- from the flange in tension: $z_t = 95.7$ mm

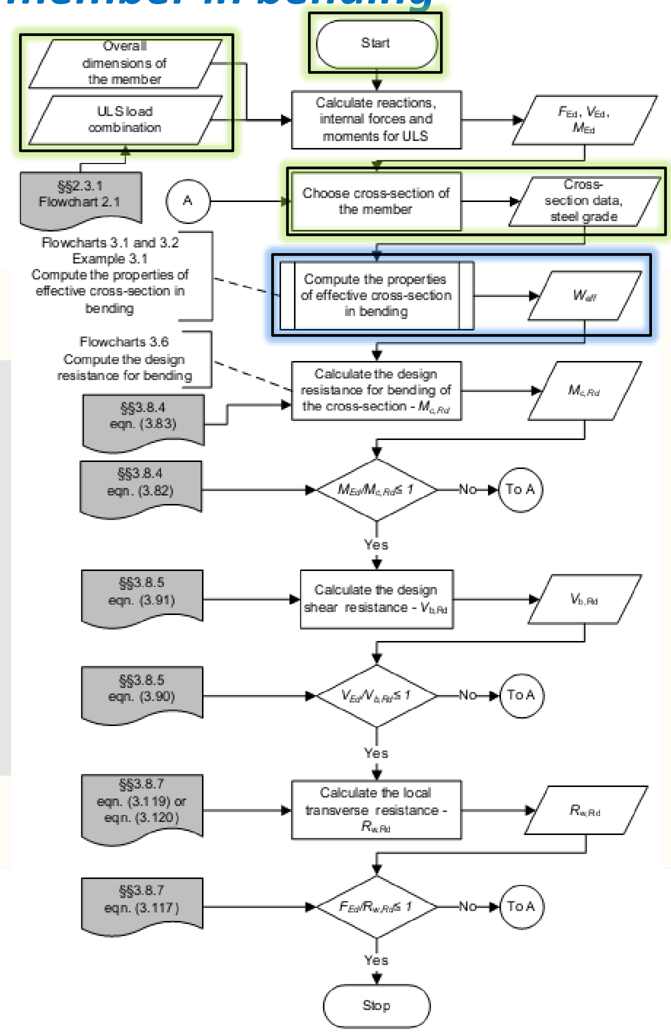
Effective section modulus:

- with respect to the flange in compression:

$$W_{eff,y,c} = \frac{I_{eff,y}}{z_c} = \frac{4139861}{102.3} = 40463 \text{ mm}^3$$

- with respect to the flange in tension:

$$W_{eff,y,t} = \frac{I_{eff,y}}{z_t} = \frac{4139861}{95.7} = 43264 \text{ mm}^3$$



Resistance of sections

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- with respect to the flange in tension:

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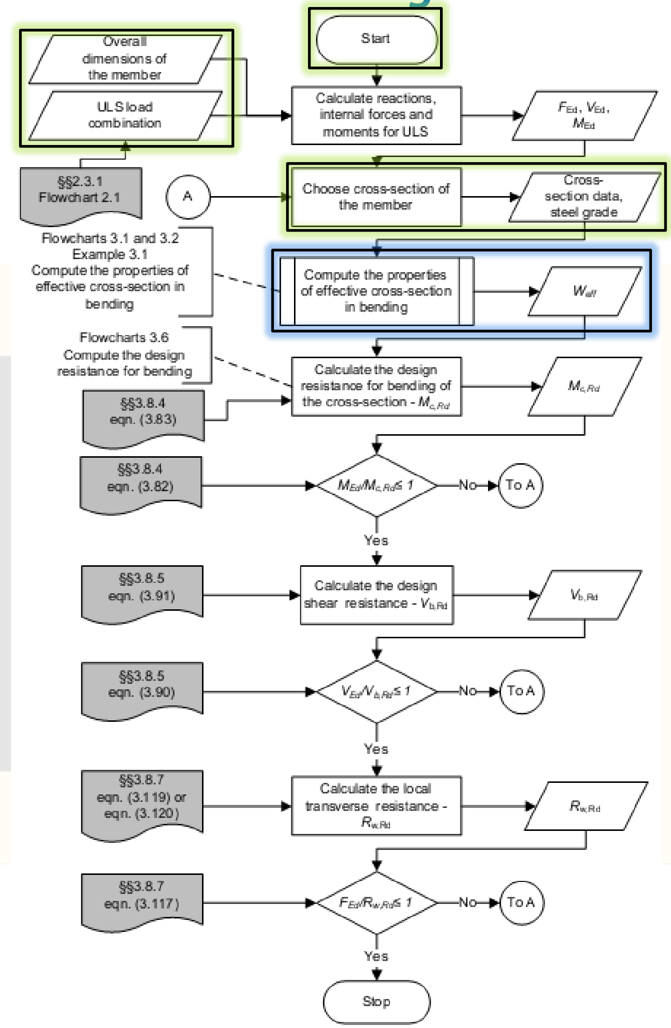
$$W_{eff,y} = \min\left(W_{eff,y,c}, W_{eff,y,t}\right) = 40463 \text{ mm}^3$$



Resistance of sections

Behaviour and Resistance of Cross Section

Resistance of Cross Sections – *Bending Moment*

- ***Example – Design of a cold-formed steel member in bending***

Applied loading on the joist at the ULS

$$q_d = \gamma_G q_G + \gamma_Q q_Q = 1.35 \times 0.51 + 1.50 \times 1.50 = 2.94\ \text{kN/m}$$

Maximum applied bending moment (at mid-span) about the major axis *y-y*:

$$M_{Ed} = q_d L^2 / 8 = 2.94 \times 5.5^2 / 8 = 11.12\ \text{kNm}$$

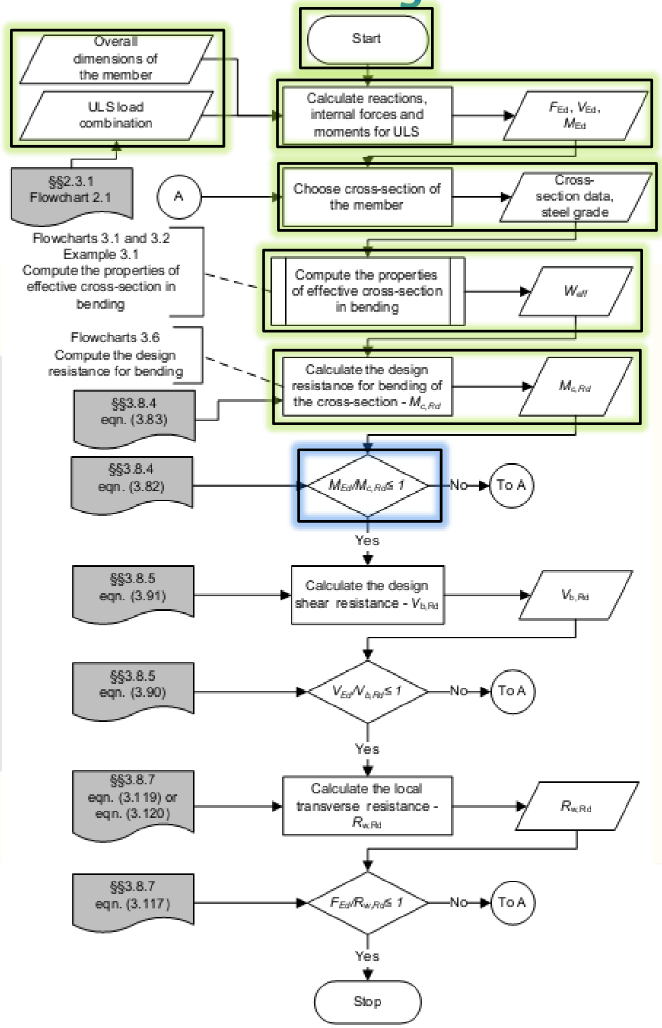
Check of bending resistance at ULS

Design moment resistance of the cross section for bending

$$M_{c,Rd} = W_{eff,y} f_{yb} / \gamma_{M0} = 40463 \times 10^{-9} \times 350 \times 10^3 / 1.0 = 14.16\ \text{kNm}$$

<u>Verification of bending resistance</u>

$$\frac{M_{Ed}}{M_{c,Rd}} = \frac{11.12}{14.16} = 0.785 < 1 \ \text{– OK}$$



Resistance of members

Compression members

- **Buckling resistance of uniform members in compression. Design according to EN1993-1-3**

$$\frac{N_{Ed}}{N_{b.Rd}} \leq 1$$

where

N_{Ed}	is the design value of the compression force;
$N_{b.Rd}$	is the design buckling resistance of the compression member.

Resistance of members

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The design buckling resistance of a compression member with Class 4 cross section should be taken as:

$$N_{b.Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} \quad (4.46)$$

where χ is the reduction factor for the relevant buckling mode.

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EN1993-1-1 + §§6.2.1 of EN1993-1-3

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$$\phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$$

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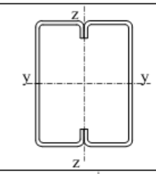
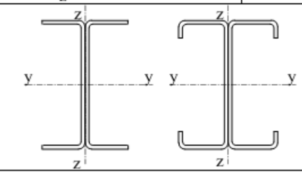
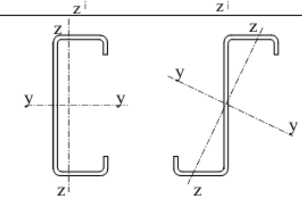
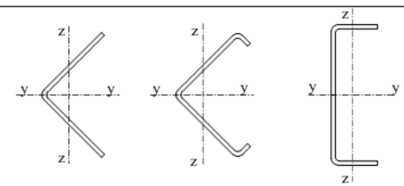
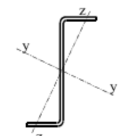
$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} \quad \text{for class 4 cross sections.}$$

Resistance of members

Compression members

- **Buckling resistance of uniform members in compression. Design according to EN1993-1-3**

Flexural buckling

Type of cross section		Buckling about axis	Buckling curve
	if f_{yb} is used	any	b
	if f_{ya} is used *)	any	c
		$y–y$	a
		$z–z$	b
		any	b
 	or other cross section	any	c

*) The average yield strength f_{ya} should not be used unless $A_{eff} = A_g$

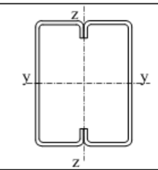
Resistance of members

Compression members

- **Buckling resistance of uniform members in compression. Design according to EN1993-1-3**

Flexural buckling

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} = \frac{L_{cr}}{i} \frac{\sqrt{\frac{A_{eff}}{A}}}{\lambda_1}$$

Type of cross section		Buckling about axis	Buckling curve
	if f_{yb} is used	any	b
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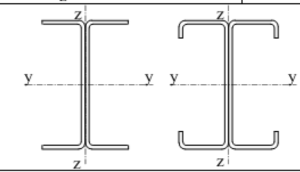
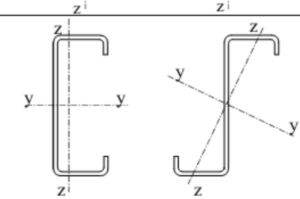
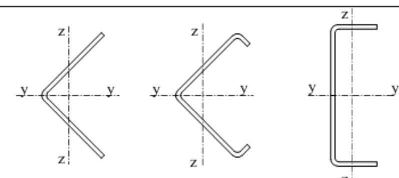
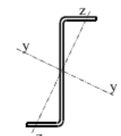
		y–y z–z	a b
		any	b
 	or other cross section	any	c

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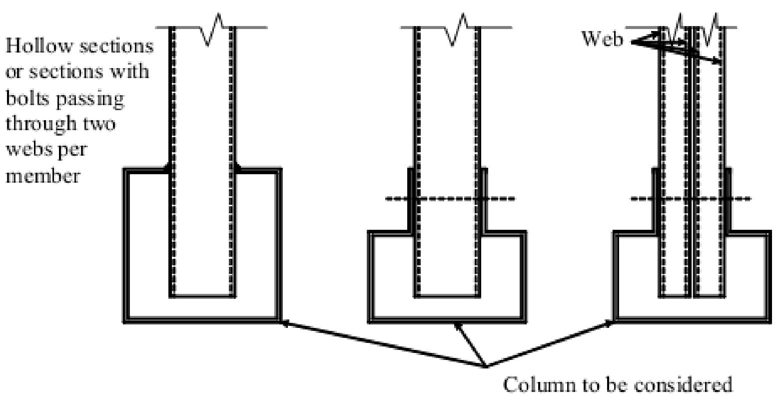
Resistance of members

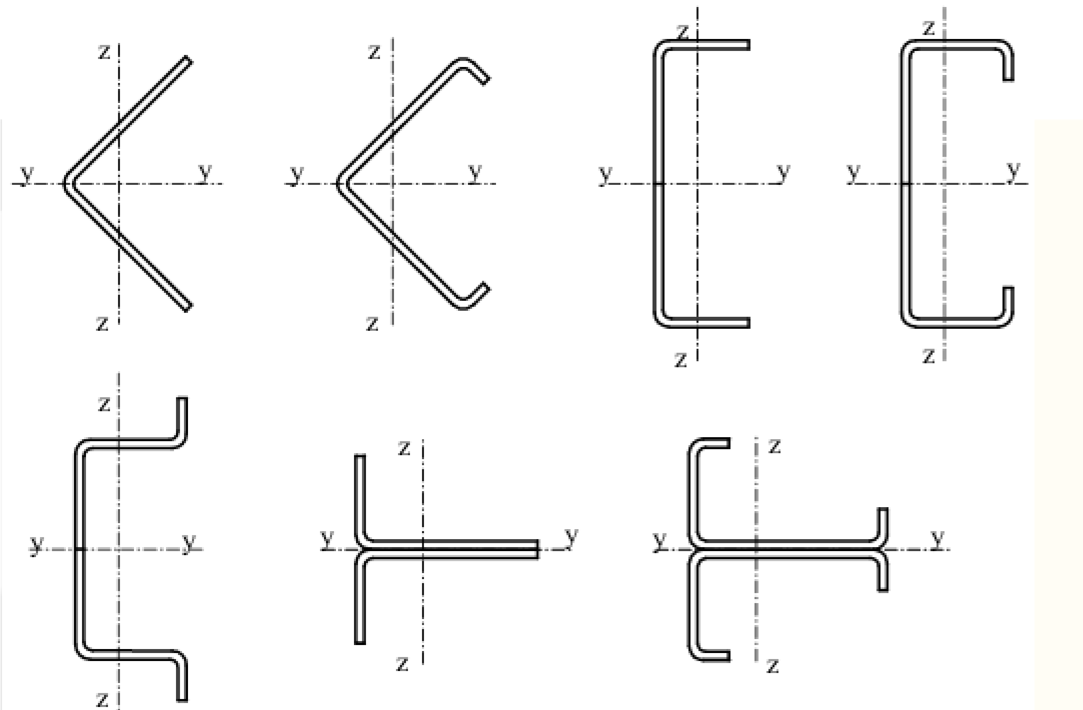
Compression members

- **Buckling resistance of uniform members in compression. Design according to EN1993-1-3**

Torsional and Flexural-Torsional buckling



Connections capable of giving significant torsional and warping restraint



Mono-symmetric cross sections susceptible to torsional-flexural buckling

Resistance of members

Compression members

- **Example – *Design of an internal wall stud in compression***

Basic Data

Height of column $H = 3.00\ \text{m}$

Span of floor $L = 6.00\ \text{m}$

Spacing between floor joists $S = 0.6\ \text{m}$

Distributed loads applied to the floor:

- dead load – lightweight slab: $1.5\ \text{kN/m}^2$

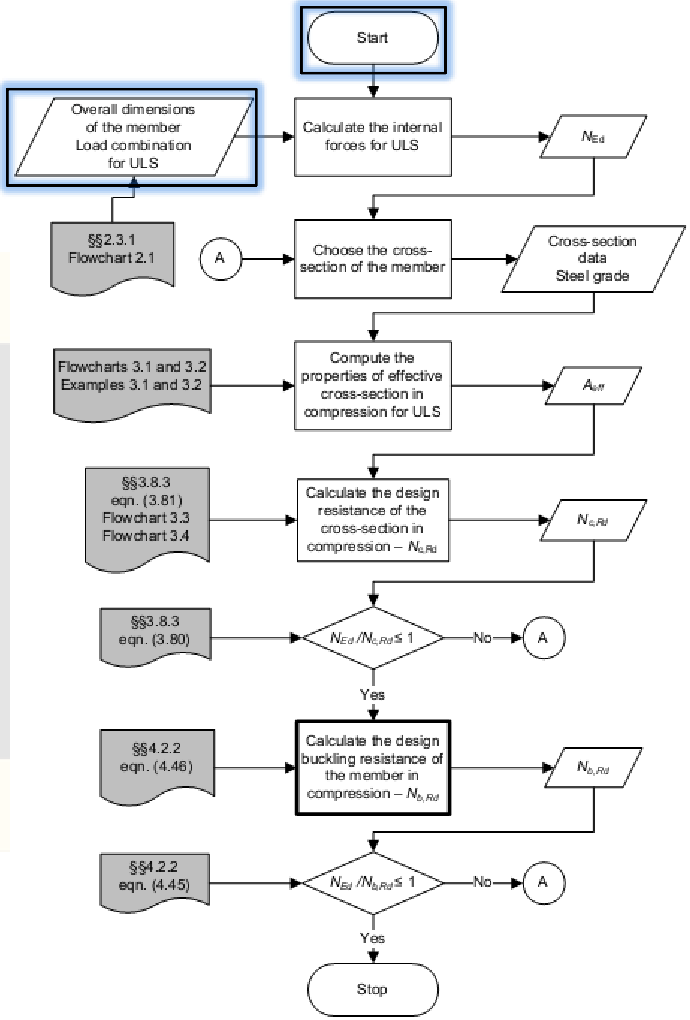
$q_G = 1.5 \times 0.6 = 0.9\ \text{kN/m}$

- imposed load: $3.00\ \text{kN/m}^2$

$q_Q = 3.00 \times 0.6 = 1.80\ \text{kN/m}$

Ultimate Limit State concentrated load

$Q = 7.0\ \text{kN}$



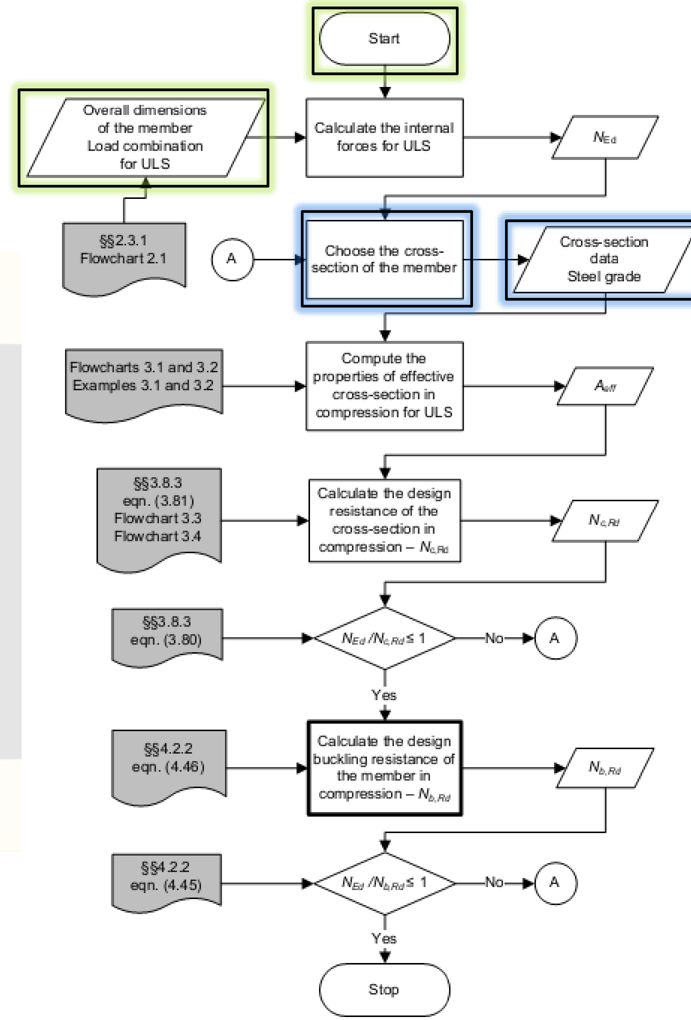
Resistance of members

Compression members

- **Example – *Design of an internal wall stud in compression***

Basic Data

Total height	$h = 150$ mm
Total width of flange	$b = 40$ mm
Total width of edge fold	$c = 15$ mm
Internal radius	$r = 3$ mm
Nominal thickness	$t_{nom} = 1.2$ mm
Steel core thickness (§§2.4.2.3)	$t = 1.16$ mm
Steel grade	S350GD+Z
Basic yield strength	$f_{yb} = 350\ \mathrm{N/mm^2}$
Modulus of elasticity	$E = 210000\ \mathrm{N/mm^2}$
Poisson's ratio	$\nu = 0.3$
Shear modulus	$G = 81000\ \mathrm{N/mm^2}$
Partial factors	$\gamma_{M0} = 1.0$
	$\gamma_{M1} = 1.0$
	$\gamma_G = 1.35$
	$\gamma_Q = 1.50$



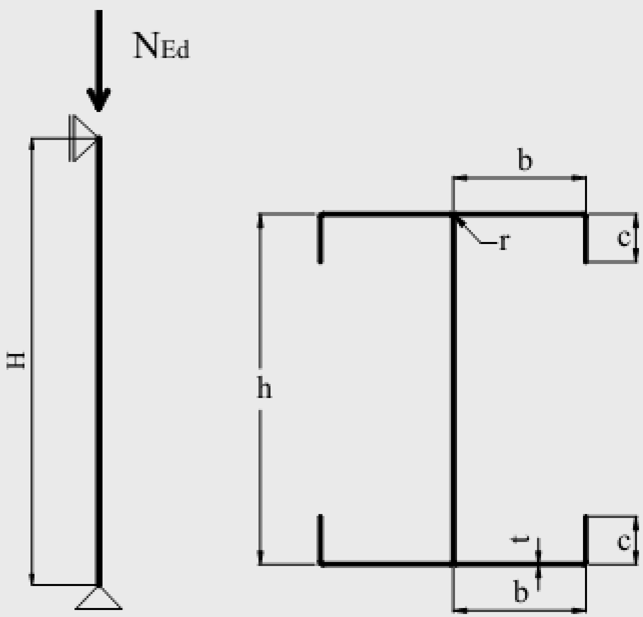
Resistance of members

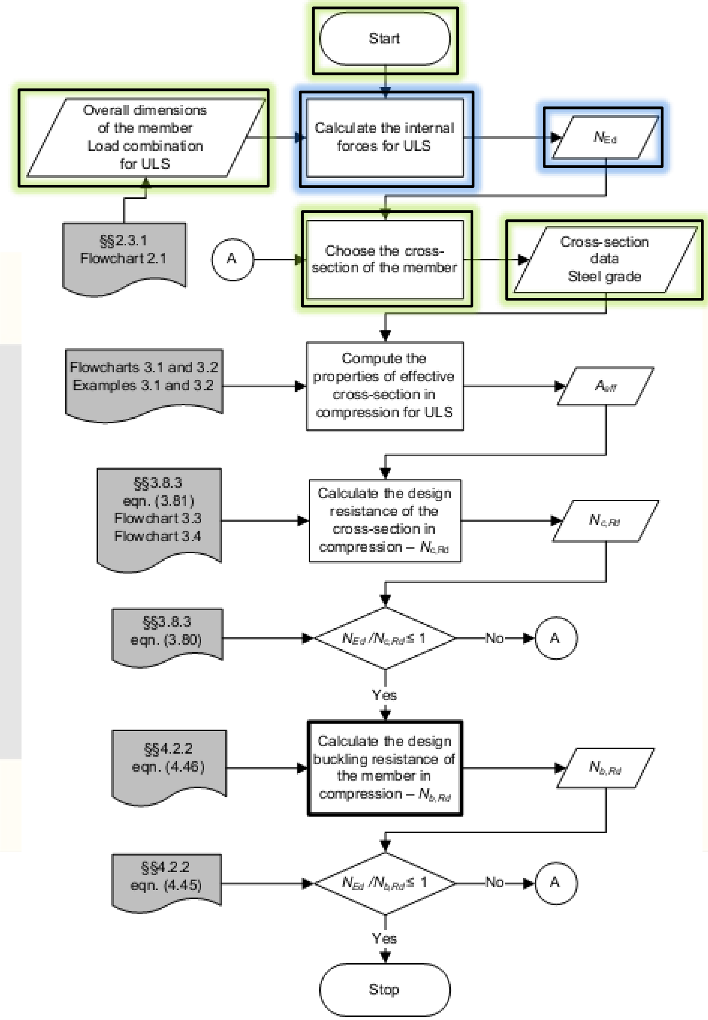
Compression members

- **Example – *Design of an internal wall stud in compression***

Basic Data

$$N_{Ed} = \left(\gamma_G q_G + \gamma_Q q_Q\right) L/2 + Q = 16.79\text{kN}$$





Resistance of members

Compression members

- **Example – *Design of an internal wall stud in compression***

Properties of the gross cross section

Area of gross cross section: $A = 592\ \text{mm}^2$

Radii of gyration: $i_y = 57.2\ \text{mm}\ ;\ i_z = 18\ \text{mm}$

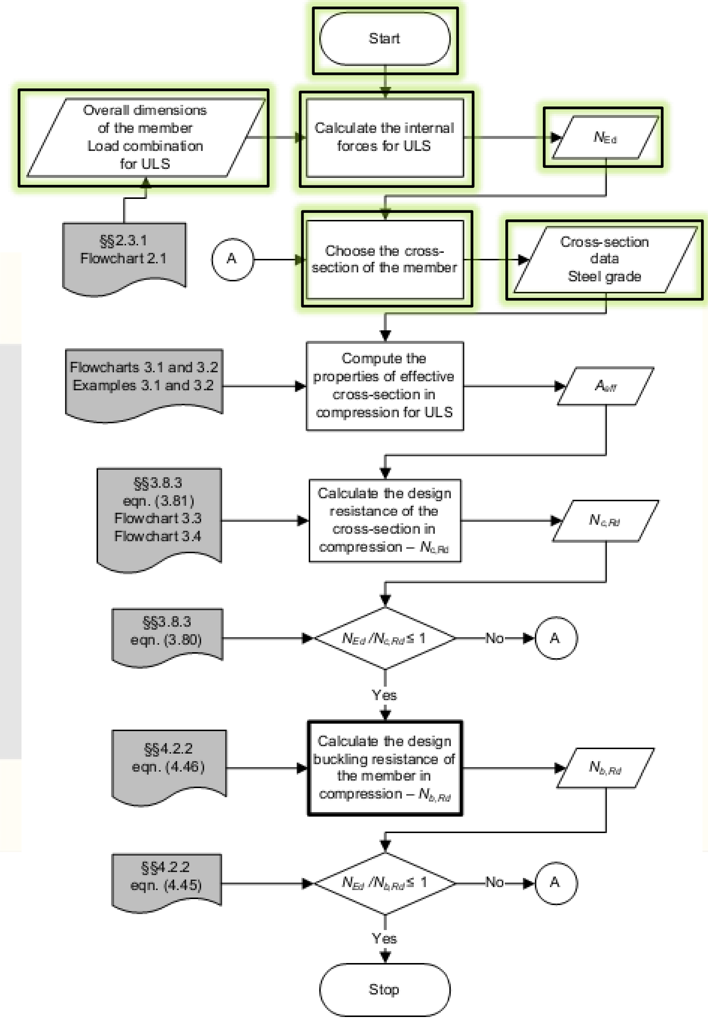
Second moment of area about *y-y*: $I_y = 1.936 \times 10^6\ \text{mm}^4$

Second moment of area about *z-z*: $I_z = 19.13 \times 10^4\ \text{mm}^4$

Warping constant: $I_w = 4.931 \times 10^8\ \text{mm}^6$

Torsion constant: $I_t = 266\ \text{mm}^4$

$$A_{eff} = 322\ \text{mm}^2$$



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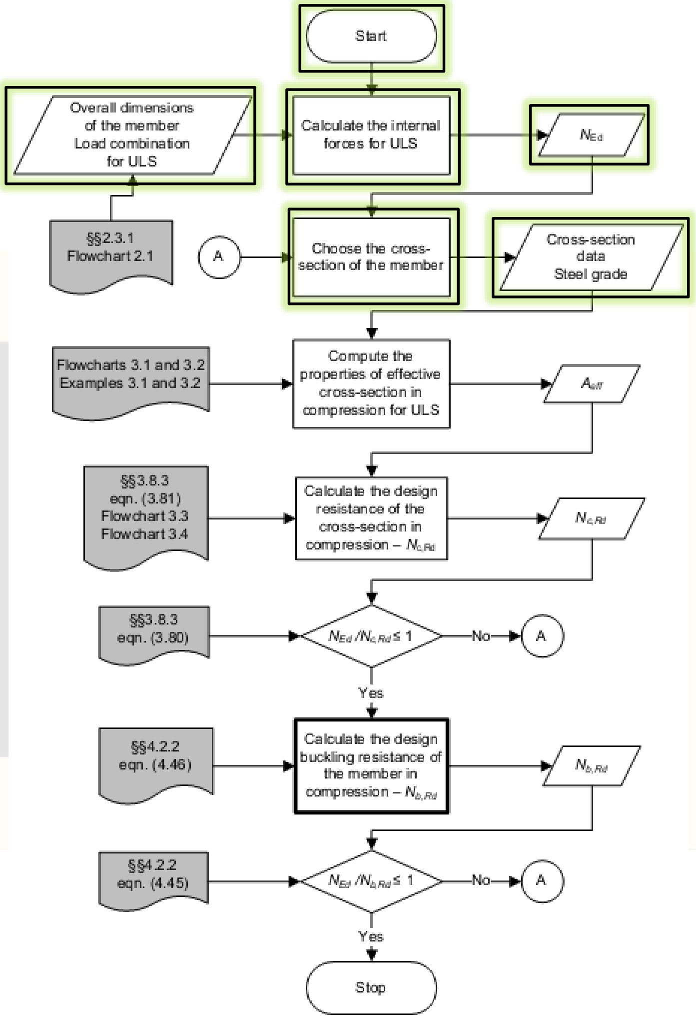
Effective section properties of the cross section

Effective area of the cross section when subjected to compression only:

$$A_{eff} = 322\ \text{mm}^2$$



Resistance of members

Compression members

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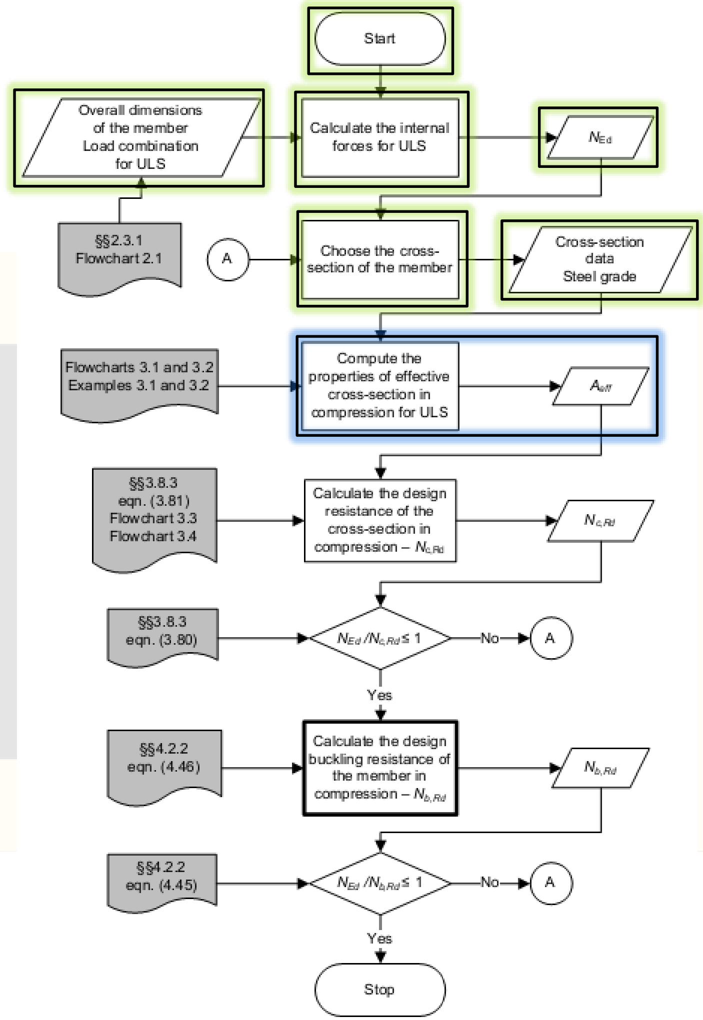


Resistance of members

Compression members

- **Example –** ***Design of an internal wall stud in compression***

Buckling resistance check

Members which are subjected to axial compression should satisfy

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1$$

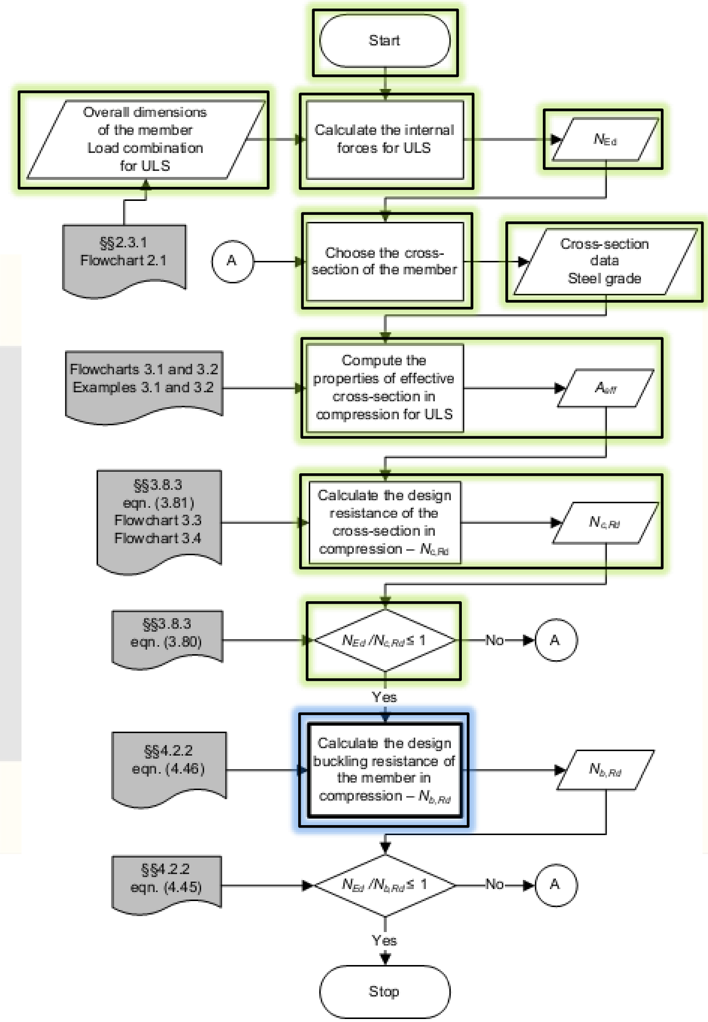
$$N_{b,Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}}$$, where χ is the reduction factor for the relevant buckling mode.

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \quad \text{but} \quad \chi \leq 1.0$$

$$\phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$$

α – imperfection factor

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_{yb}}{N_{cr}}}$$



Resistance of members

Compression members

- Example – *Design of an internal wall stud in compression*

Buckling resistance check

Determination of the reduction factors χ_y, χ_z, χ_T

Flexural buckling

$$\bar{\lambda}_F = \sqrt{\frac{A_{eff} f_{yb}}{N_{cr}}} = \frac{L_{cr}}{i} \frac{\sqrt{A_{eff}/A}}{\lambda_1}$$

The buckling length:

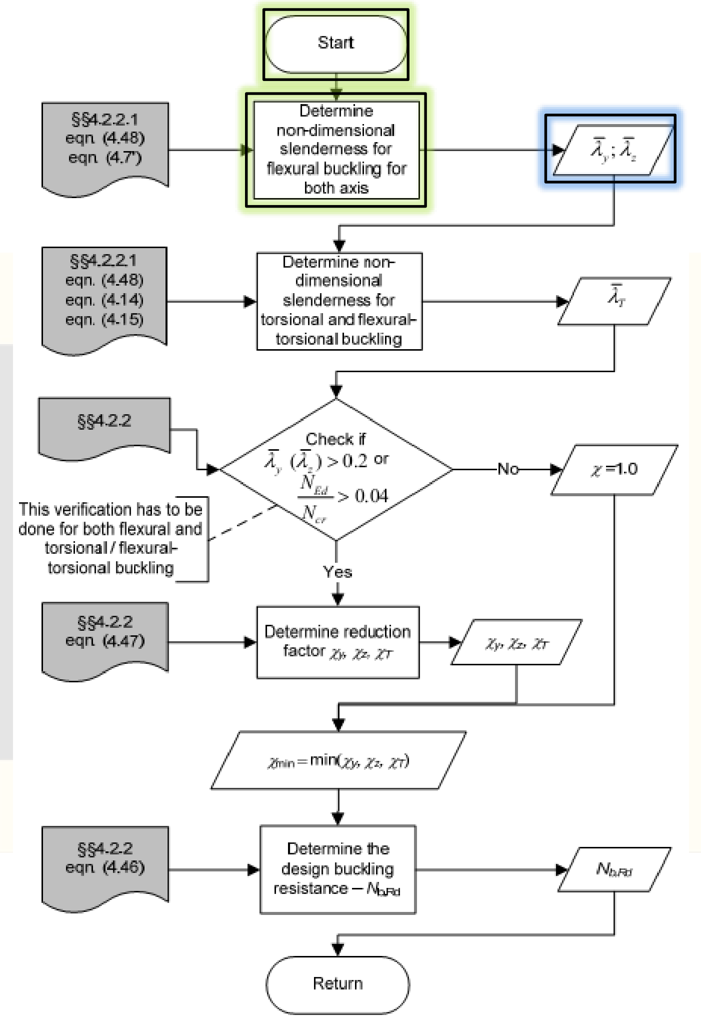
$$L_{cr,y} = L_{cr,z} = H = 3000 \text{ mm}$$

$$\lambda_1 = \pi \sqrt{\frac{E}{f_{yb}}} = \pi \times \sqrt{\frac{210000}{350}} = 76.95$$

Buckling about y–y axis

$$\bar{\lambda}_y = \frac{L_{cr,y}}{i_y} \frac{\sqrt{A_{eff}/A}}{\lambda_1} = \frac{3000}{57.2} \times \frac{\sqrt{322/592}}{76.95} = 0.503$$

$$\alpha_y = 0.21$$



Resistance of members

Compression members

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Buckling resistance check

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Buckling about *y–y* axis

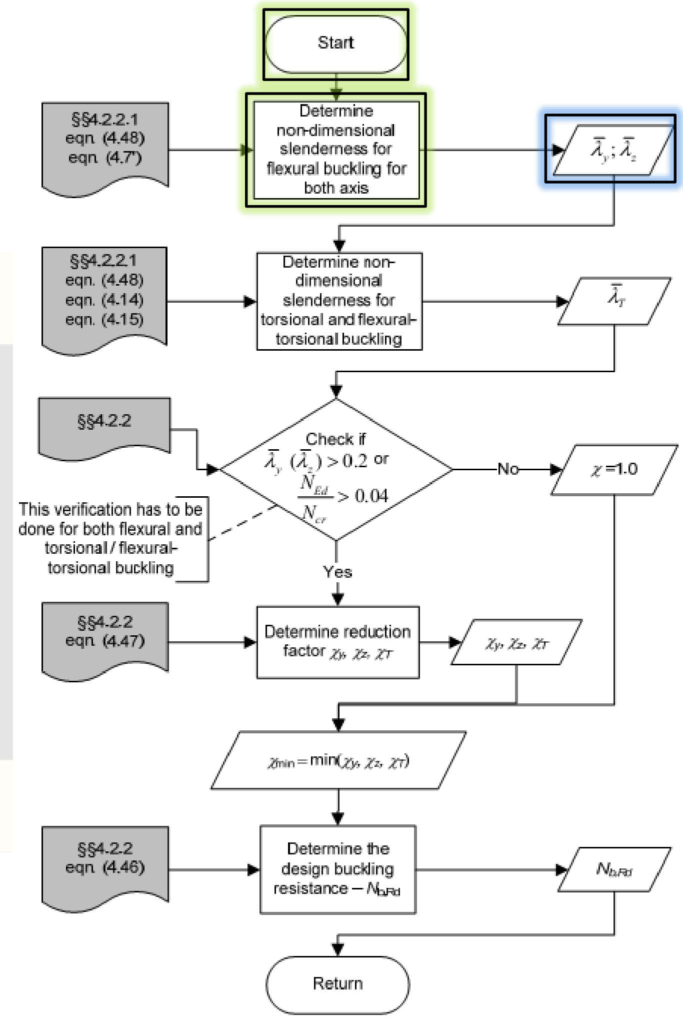
$$\bar{\lambda}_y = \frac{L_{cr,y}}{i_y}\frac{\sqrt{A_{eff}/A}}{\lambda_1} = \frac{3000}{57.2}\times\frac{\sqrt{322/592}}{76.95} = 0.503$$

$$\alpha_y = 0.21$$

$$\phi_y = 0.5\left[1+\alpha_y\left(\bar{\lambda}_y-0.2\right)+\bar{\lambda}_y^{\,2}\right] =$$

$$= 0.5\times\left[1+0.21\times\left(0.503-0.2\right)+0.503^2\right] = 0.658$$

$$\chi_y = \frac{1}{\phi_y+\sqrt{\phi_y^{\,2}-\bar{\lambda}_y^{\,2}}} = \frac{1}{0.658+\sqrt{0.658^2-0.503^2}} = 0.924$$



Resistance of members

Compression members

- **Example – *Design of an internal wall stud in compression***

Buckling resistance check

Determination of the reduction factors χ_y, χ_z, χ_T

Buckling about z–z axis

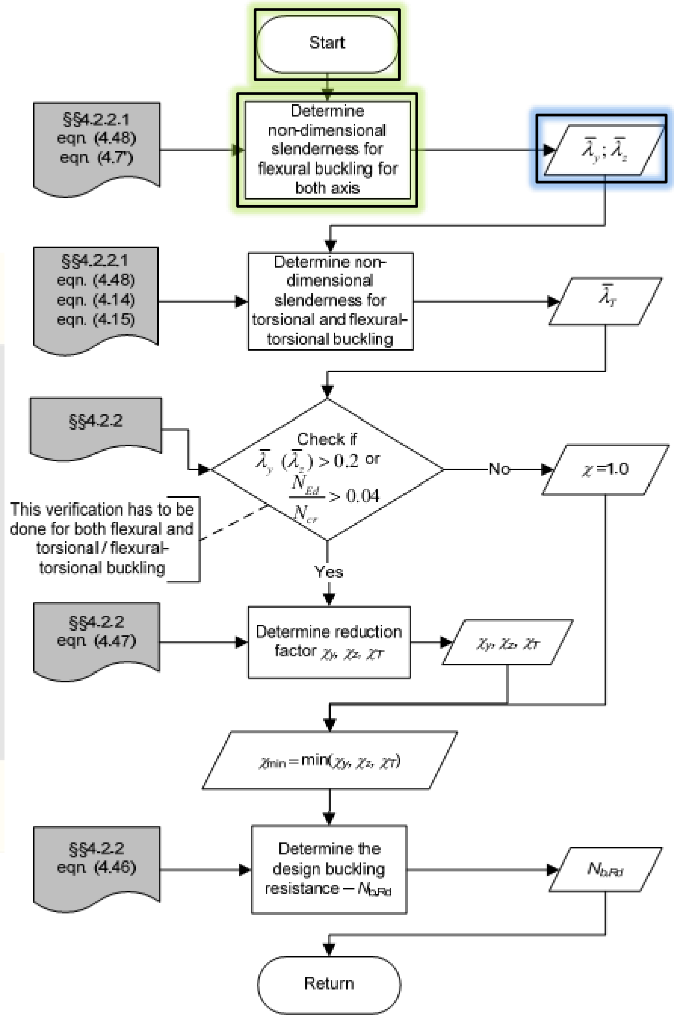
$$\bar{\lambda}_z = \frac{L_{cr,z}}{i_z}\frac{\sqrt{A_{eff}/A}}{\lambda_1} = \frac{3000}{18}\times\frac{\sqrt{322/592}}{76.95} = 1.597$$

$$\alpha_z = 0.34$$

$$\phi_z = 0.5\left[1+\alpha_z\left(\bar{\lambda}_z - 0.2\right)+\bar{\lambda}_z^{\,2}\right] =$$

$$= 0.5\times\left[1+0.34\times\left(1.597-0.2\right)+1.597^2\right] = 2.013$$

$$\chi_z = \frac{1}{\phi_z+\sqrt{\phi_z^{\,2}-\bar{\lambda}_z^{\,2}}} = \frac{1}{2.013+\sqrt{2.013^2-1.597^2}} = 0.309$$



Resistance of members

Compression members

- **Example –** ***Design of an internal wall stud in compression***

Buckling resistance check

Determination of the reduction factors χ_y, χ_z, χ_T

Torsional buckling

$$N_{cr,T} = \frac{1}{i_o^2}\left(GI_t + \frac{\pi^2 EI_w}{l_T^2}\right)$$

where

$$i_o^2 = i_y^2 + i_z^2 + y_o^2 + z_o^2$$

$$y_o = z_o = 0$$

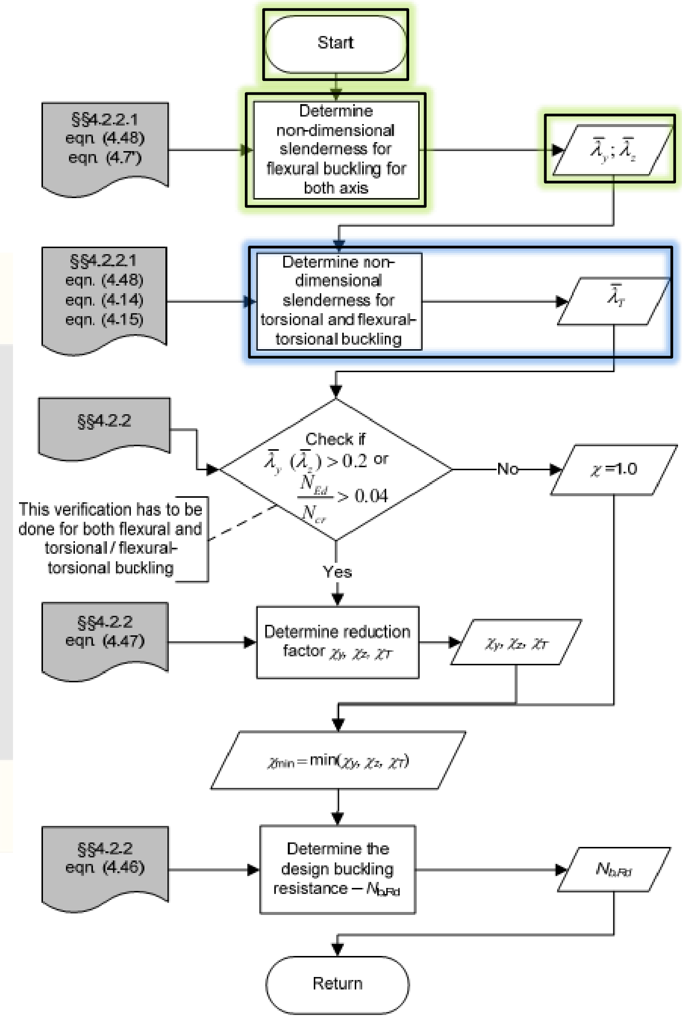
$$i_o^2 = 57.2^2 + 18^2 + 0 + 0 = 3594 \text{ mm}^2$$

$$l_T = H = 3000 \text{ mm}$$

The elastic critical force for torsional buckling is:

$$N_{cr,T} = \frac{1}{3594}\times\left(81000\times 266 + \frac{\pi^2\times 210000\times 4.931\times 10^8}{3000^2}\right) =$$

$$= 37.59\times 10^3 \text{ N}$$



Resistance of members

Compression members

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Buckling resistance check

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$$y_o = z_o = 0$$

$$i_o^2 = 57.2^2 + 18^2 + 0 + 0 = 3594\ \text{mm}^2$$

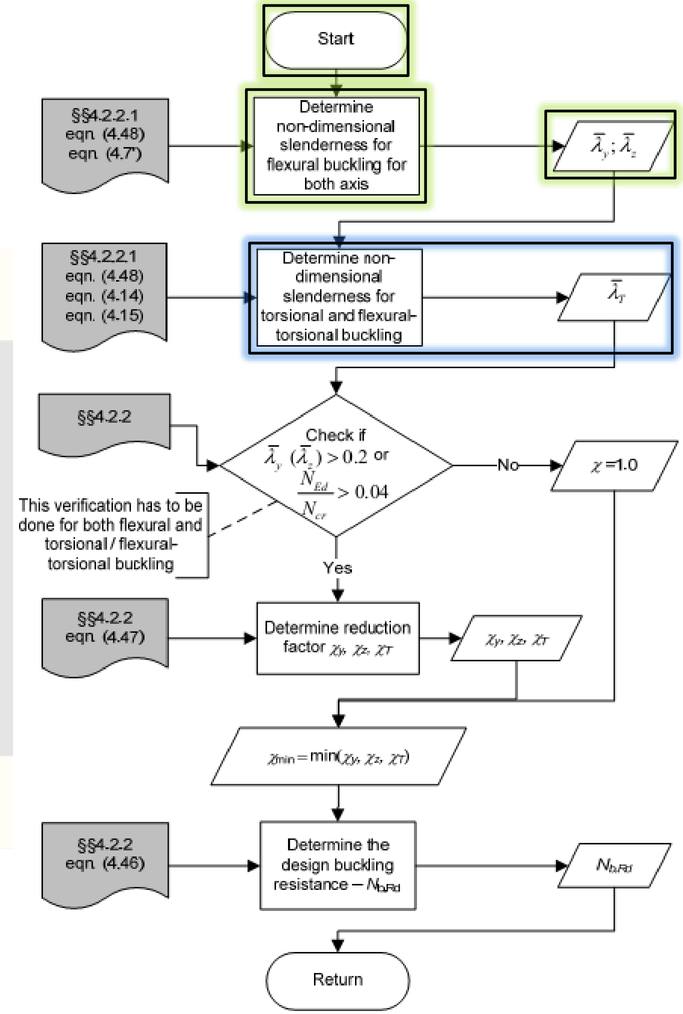
$$l_T = H = 3000\ \text{mm}$$

The elastic critical force for torsional buckling is:

$$N_{cr,T} = \frac{1}{3594} \times \left(81000 \times 266 + \frac{\pi^2 \times 210000 \times 4.931 \times 10^8}{3000^2}\right) =$$

$$= 37.59 \times 10^3\ \text{N}$$

The elastic critical force will be: $N_{cr} = N_{cr,T} = 37.59$ kN



Resistance of members

Compression members

- **Example – *Design of an internal wall stud in compression***

Buckling resistance check

Determination of the reduction factors χ_y, χ_z, χ_T

Torsional buckling

The non-dimensional slenderness is:

$$\bar{\lambda}_T = \sqrt{\frac{A_{eff} f_{yb}}{N_{cr}}} = \sqrt{\frac{322 \times 350}{37.59 \times 10^3}} = 1.731$$

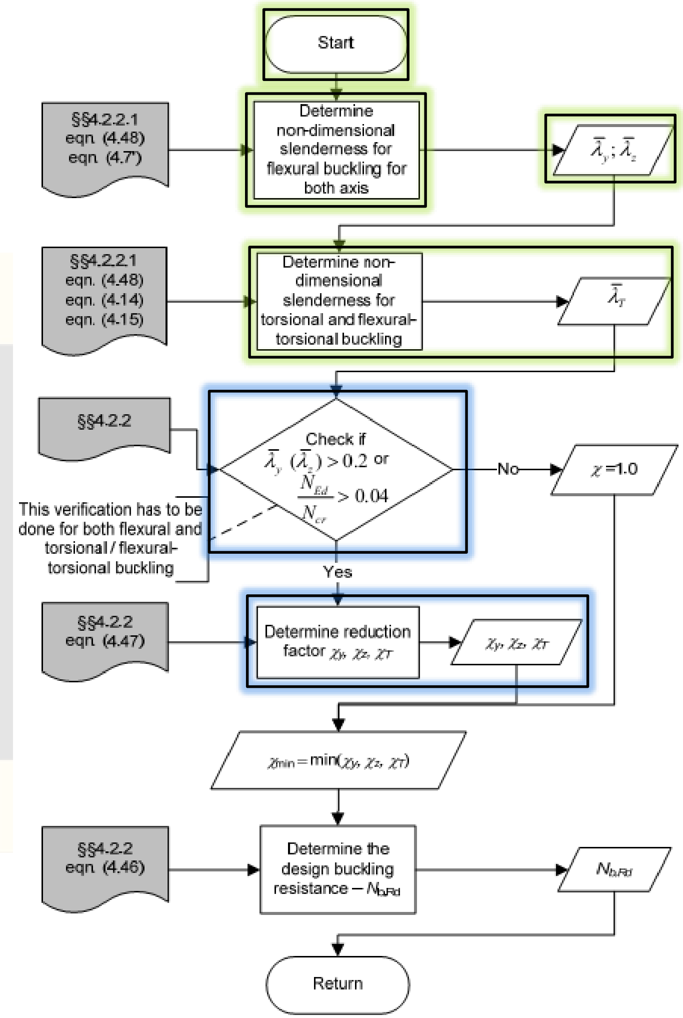
$$\alpha_T = 0.34 \text{ – buckling curve } b$$

$$\phi_T = 0.5\left[1 + \alpha_T\left(\bar{\lambda}_T - 0.2\right) + \bar{\lambda}_T^{\,2}\right] =$$

$$= 0.5 \times \left[1 + 0.34 \times (1.731 - 0.2) + 1.731^2\right] = 2.258$$

The reduction factor for torsional buckling is:

$$\chi_T = \frac{1}{\phi_T + \sqrt{\phi_T^{\,2} - \bar{\lambda}_T^{\,2}}} = \frac{1}{2.258 + \sqrt{2.258^2 - 1.731^2}} = 0.270$$



Resistance of members

Compression members

- **Example – *Design of an internal wall stud in compression***

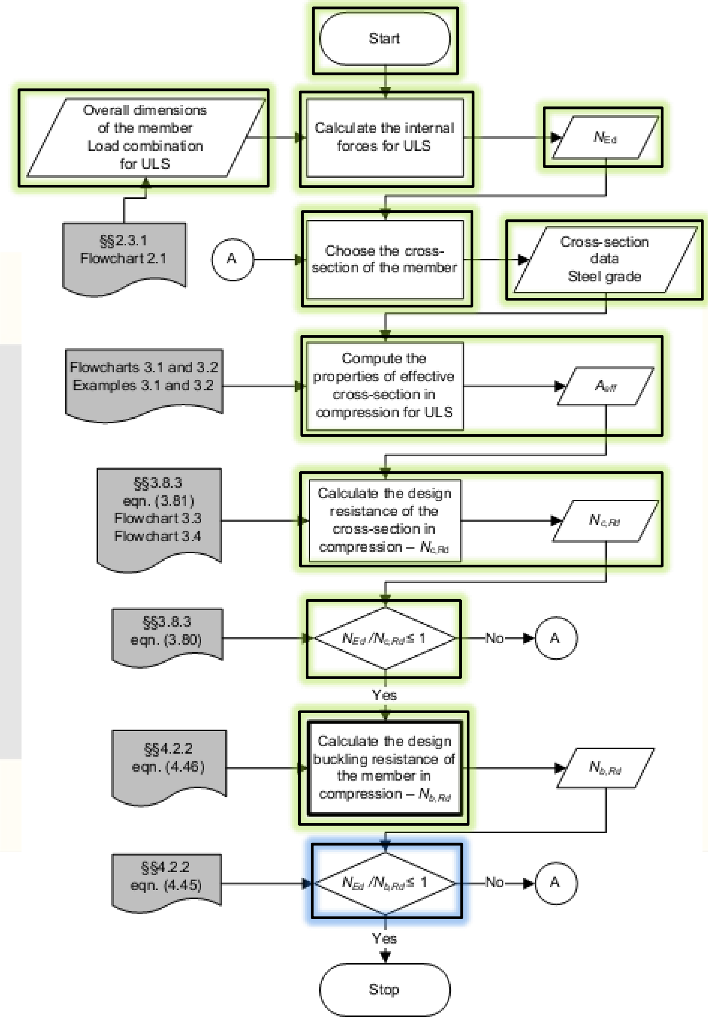
Buckling resistance check

Determination of the reduction factors χ_y, χ_z, χ_T

$$\chi = \min(\chi_y, \chi_z, \chi_T) = \min(0.924, 0.309, 0.270) = 0.270$$

$$N_{b.Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} = \frac{0.270 \times 322 \times 350}{1.00} = 30429\,\text{N} = 30.429\,\text{kN}$$

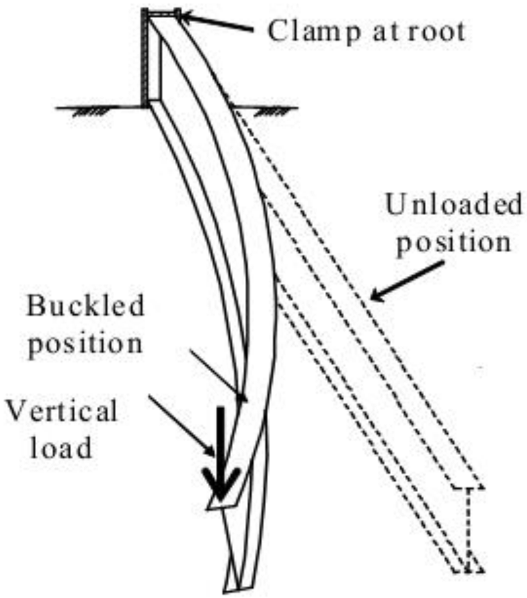
$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{16.79}{30.429} = 0.552 \leq 1 \text{ – OK}$$



Resistance of members

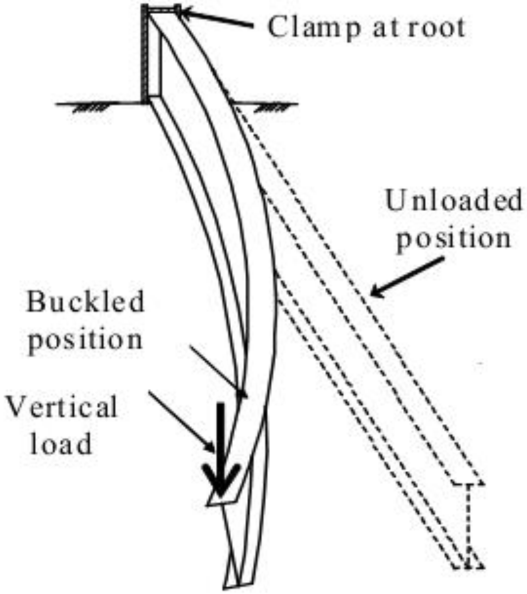
Buckling strength of bending members

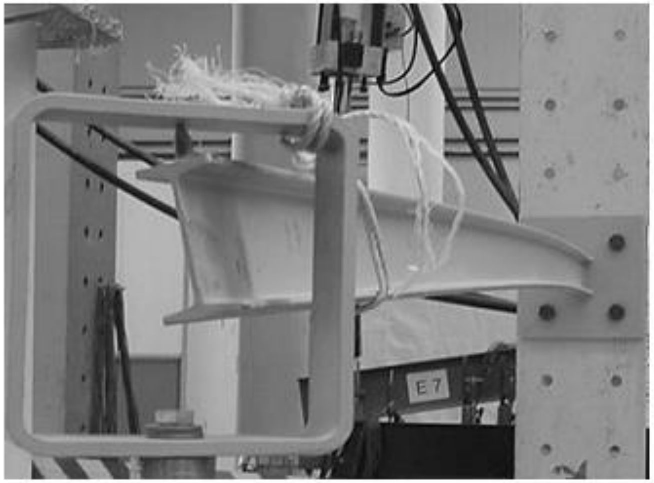
- **Theoretical background**



Buckling strength of bending members

- **Theoretical background**





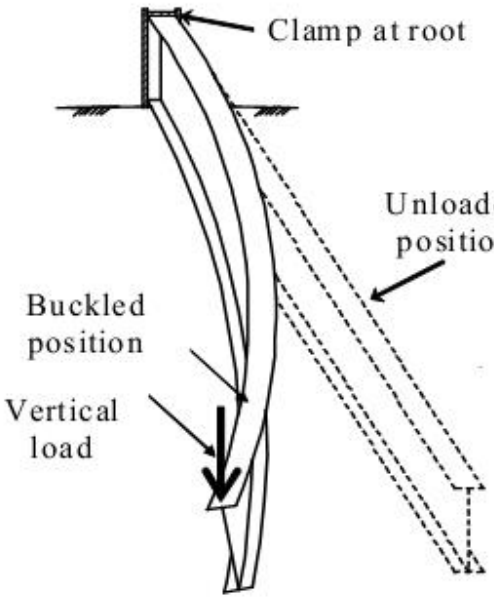
- bending about minor axis, z–z

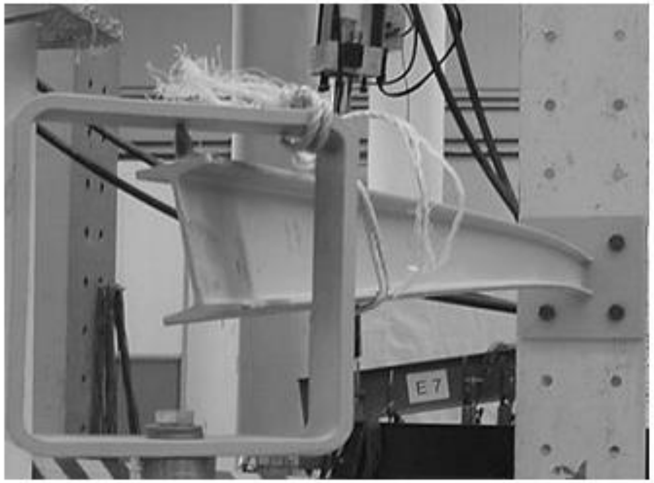
$$EI_z \frac{d^2 v(x)}{dx^2} + \varphi(x) M_y = 0$$

Resistance of members

Buckling strength of bending members

- **Theoretical background**





- bending about minor axis, z–z

$$EI_z \frac{d^2 v(x)}{dx^2} + \varphi(x) M_y = 0$$

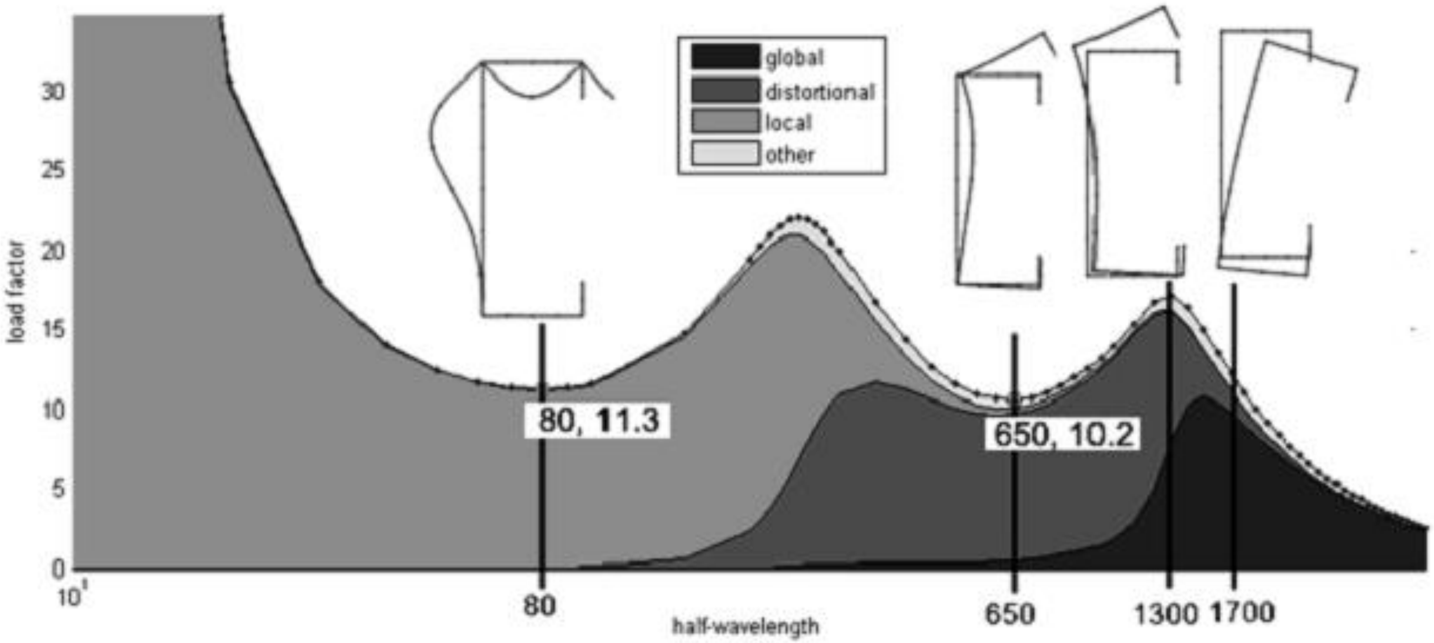
- torsion around x–x axis

$$EI_w \frac{d^3 \varphi(x)}{dx^3} - GI_T \frac{d\varphi(x)}{dx} + M_y \frac{dv(x)}{dx} = 0$$

Resistance of members

Buckling strength of bending members

- Theoretical background



Buckling strength of bending members

- **Design according to EN1993-1-3**

Lateral-torsional buckling of members subject to bending

A laterally unrestrained member subject to major axis bending should be verified against lateral-torsional buckling as follows:

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1.0 \qquad (4.55)$$

where

M_{Ed} is the design value of the moment;

$M_{b.Rd}$ is the design buckling resistance moment.

Resistance of members

Buckling strength of bending members

- **Design according to EN1993-1-3**

Lateral-torsional buckling of members subject to bending

A laterally unrestrained member subject to major axis bending should be verified against lateral-torsional buckling as follows:

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1.0 \qquad (4.55)$$

where

M_{Ed} is the design value of the moment;

$M_{b.Rd}$ is the design buckling resistance moment.

The design buckling resistance moment of a laterally unrestrained beam should be taken as:

$$M_{b,Rd} = \chi_{LT} W_y f_y / \gamma_{M1} \qquad (4.56)$$

where

W_y is the appropriate section modulus as follows:

$W_y = W_{el,y}$ is for class 3 cross section;

$W_y = W_{eff,y}$ is for class 4 cross section;

Buckling strength of bending members

- **Design according to EN1993-1-3**

Lateral-torsional buckling of members subject to bending

In determining W_y, holes for fasteners at the beam ends need not to be taken into account.

χ_{LT} is the reduction factor for lateral-torsional buckling,

$$\chi_{LT} = \frac{1}{\phi_{LT} + \left(\phi_{LT}^{2} - \bar{\lambda}_{LT}^{2}\right)^{0.5}}\text{, but } \chi_{LT} \leq 1$$

with: $\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^{2}\right]$;

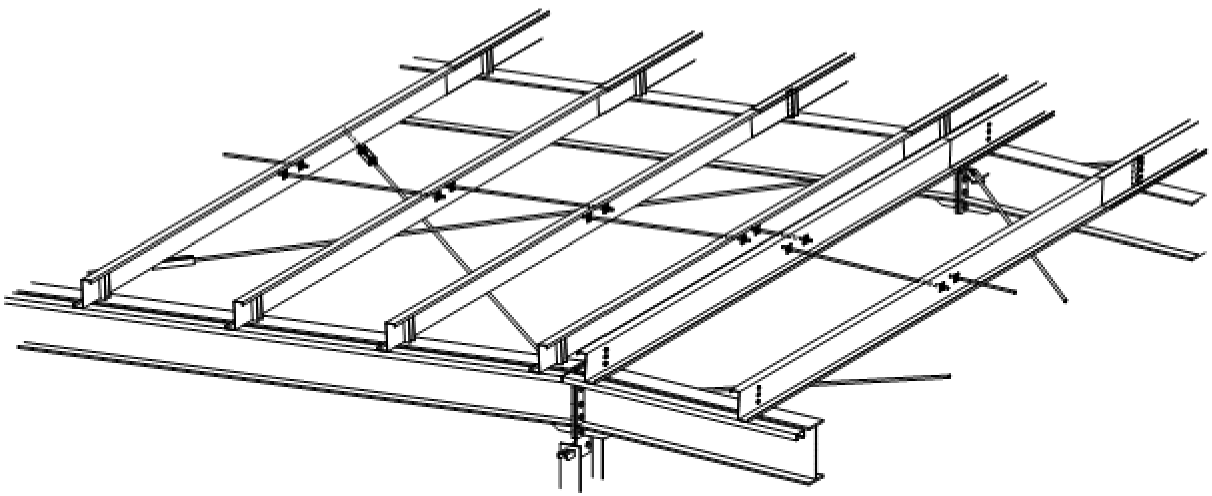
α_{LT} is the imperfection factor corresponding to buckling curve b, $\alpha_{LT} = 0.34$;

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}};$$

Resistance of members

Buckling strength of bending members

Beams with LT restraints in building



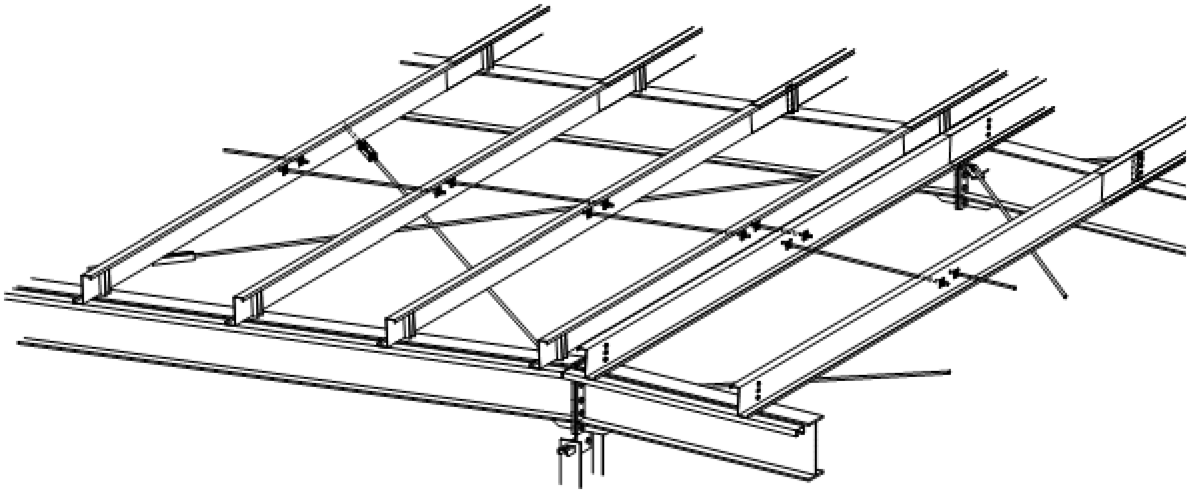
anti-sag bars for Z-purlins and Z-purlins for roof beams

Members with discrete lateral restraint

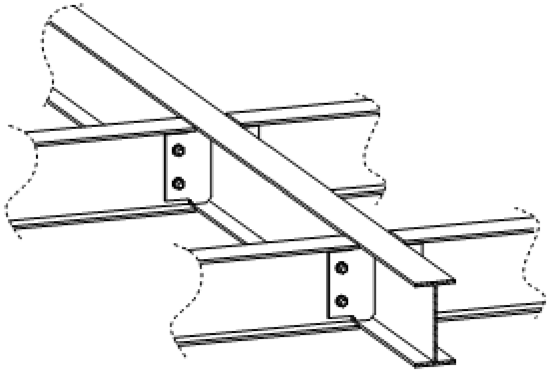
Resistance of members

Buckling strength of bending members

Beams with LT restraints in building



anti-sag bars for Z-purlins and Z-purlins for roof beams



secondary beams

Members with discrete lateral restraint

Resistance of members

Buckling strength of bending members

- ***Example – Design of an cold-formed steel beam in bending***

Basic Data

Span of beam $L = 4.5\text{ m}$

Spacing between beams $S = 3.0\text{ m}$

Distributed loads applied to the joist:

self-weight of the beam $q_{G,beam} = 0.14\text{ kN/m}$

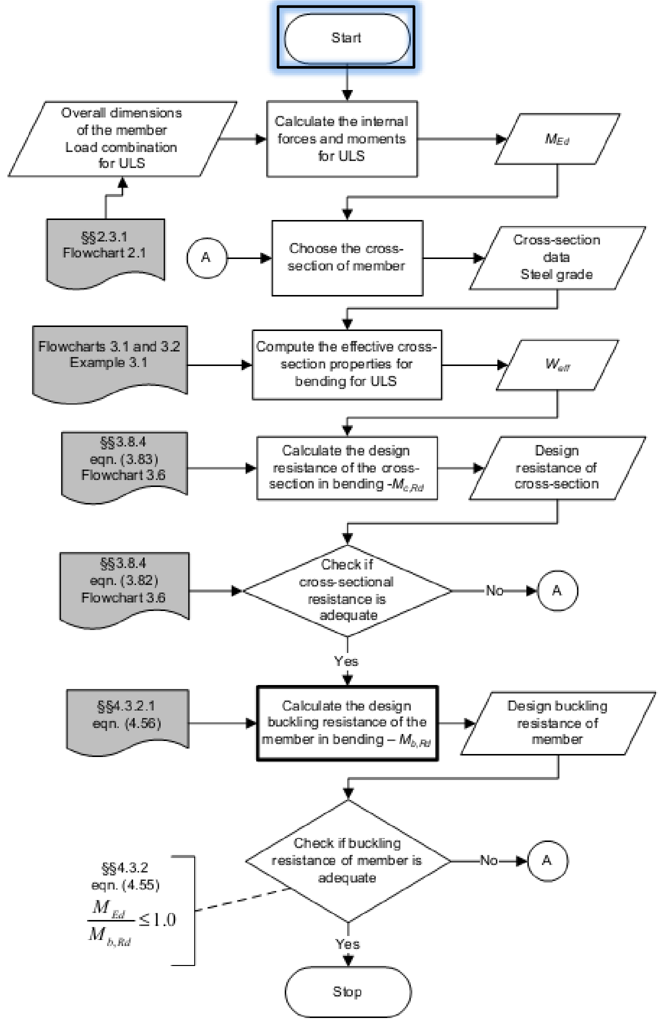
weight of the floor and 0.6 kN/m^2

$$q_{G,slab} = 0.55 \times 3.0 = 1.65\text{ kN/m}$$

total dead load $q_G = q_{G,beam} + q_{G,slab} = 1.79\text{ kN/m}$

imposed load 1.50 kN/m^2

$$q_Q = 1.50 \times 3.0 = 4.50\text{ kN/m}$$

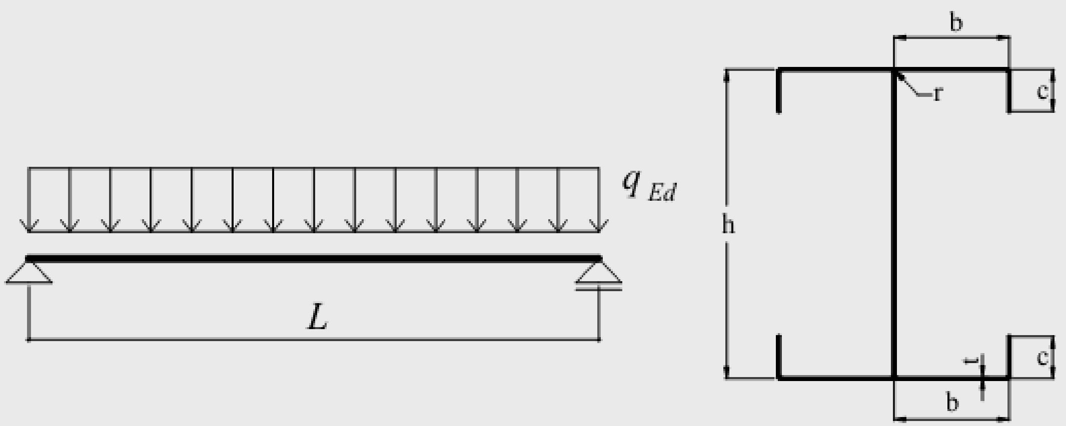


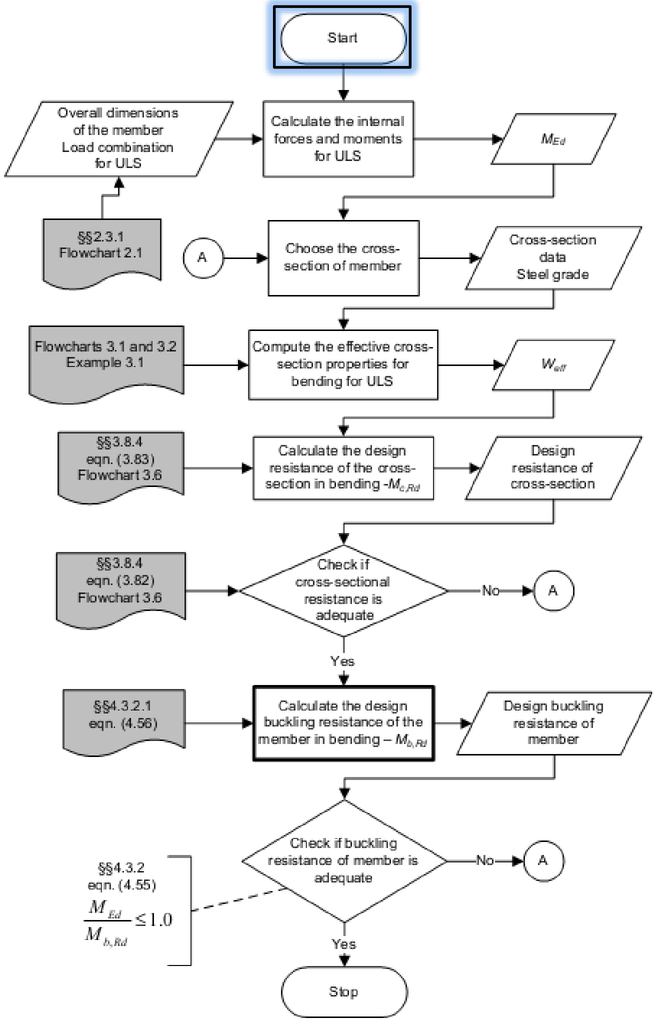
Resistance of members

Buckling strength of bending members

- *Example – Design of an cold-formed steel beam in bending*

Basic Data





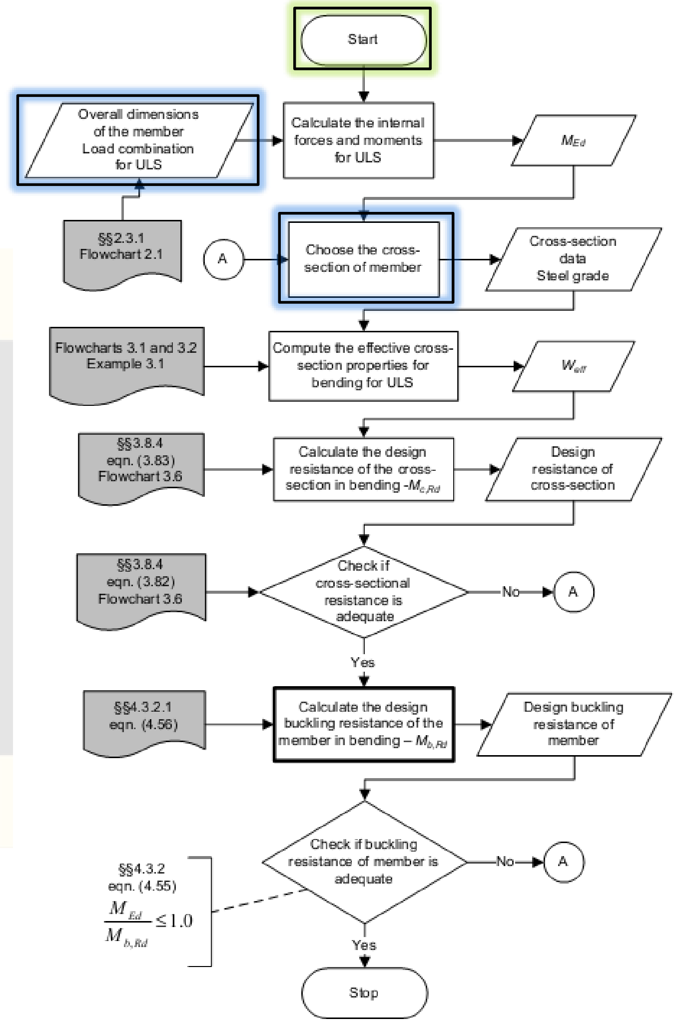
Resistance of members

Behavi Buckling strength of bending members

- *Example – Design of an cold-formed steel beam in bending*

The dimensions of the cross section and the material properties are:

Total height	$h = 250$ mm
Total width of flanges	$b = 70$ mm
Total width of edge fold	$c = 25$ mm
Internal radius	$r = 3$ mm
Nominal thickness	$t_{nom} = 3.0$ mm
Steel core thickness (§§2.4.2.3)	$t = 2.96$ mm
Steel grade	S350GD+Z
Basic yield strength	$f_{yb} = 350\ \mathrm{N/mm^2}$
Modulus of elasticity	$E = 210000\ \mathrm{N/mm^2}$
Poisson's ratio	$\nu = 0.3$
Partial factors	$\gamma_{M0} = 1.0$
	$\gamma_{M1} = 1.0$
	$\gamma_G = 1.35$
	$\gamma_Q = 1.50$



Resistance of members

Buckling strength of bending members

- ***Example – Design of an cold-formed steel beam in bending***

Properties of the gross cross section

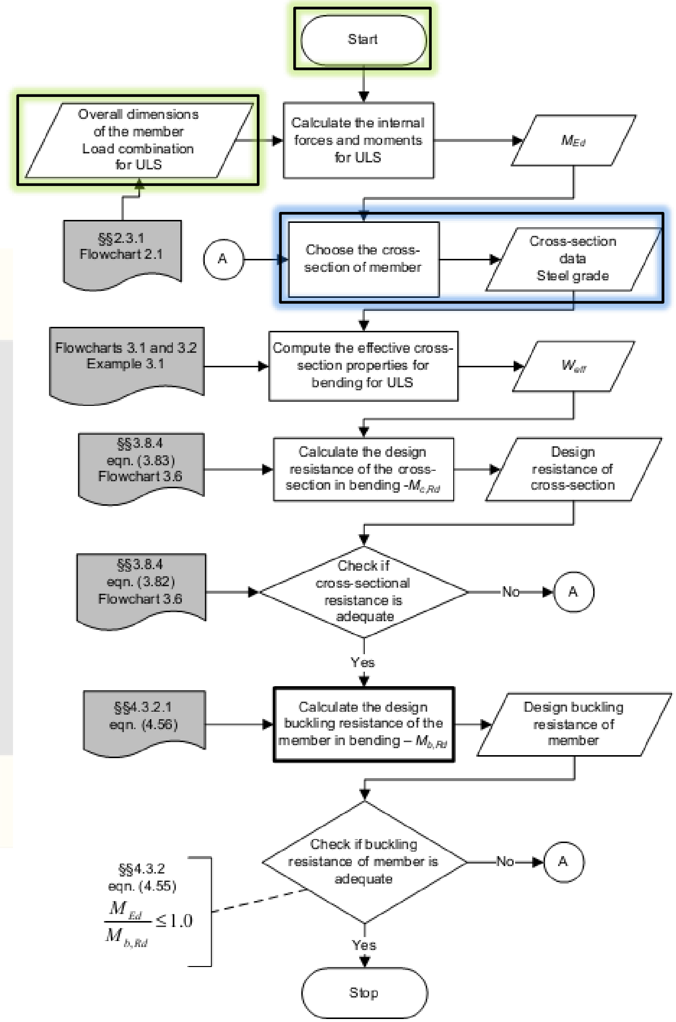
Second moment of area about y–y: $I_y = 2302.15 \times 10^4\ \text{mm}^4$

Second moment of area about z–z: $I_z = 244.24 \times 10^4\ \text{mm}^4$

Radii of gyration: $i_y = 95.3\ \text{mm}$; $i_z = 31\ \text{mm}$

Warping constant: $I_w = 17692.78 \times 10^6\ \text{mm}^6$

Torsion constant: $I_t = 7400\ \text{mm}^4$



Buckling strength of bending members

- *Example – Design of an cold-formed steel beam in bending*

Effective section properties at the ultimate limit state

Second moment of area of cold-formed lipped channel section subjected to bending about its major axis:

$$I_{eff,y} = 22688890 \text{ mm}^4$$

Position of the neutral axis:

- from the flange in compression: $z_c = 124.6$ mm
- from the flange in tension: $z_t = 122.4$ mm

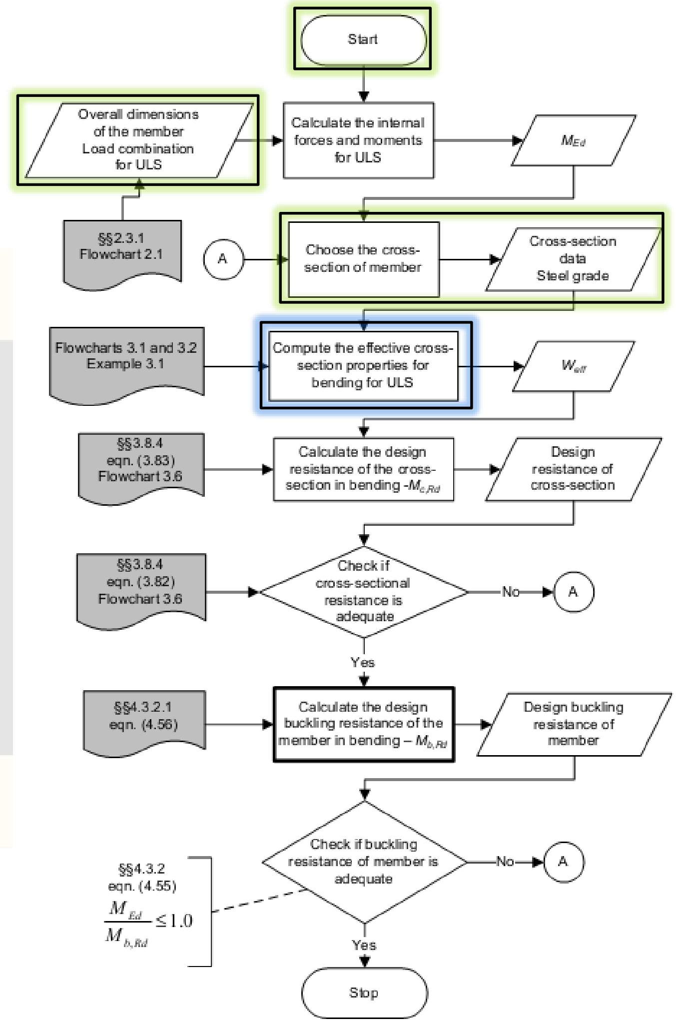
Effective section modulus:

- with respect to the flange in compression:

$$W_{eff,y,c} = \frac{I_{eff,y}}{z_c} = \frac{22688890}{124.6} = 182094 \text{ mm}^3$$

- with respect to the flange in tension:

$$W_{eff,y,t} = \frac{I_{eff,y}}{z_t} = \frac{22688890}{122.4} = 185367 \text{ mm}^3$$



Resistance of members

Buckling strength of bending members

- *Example – Design of an cold-formed steel beam in bending*

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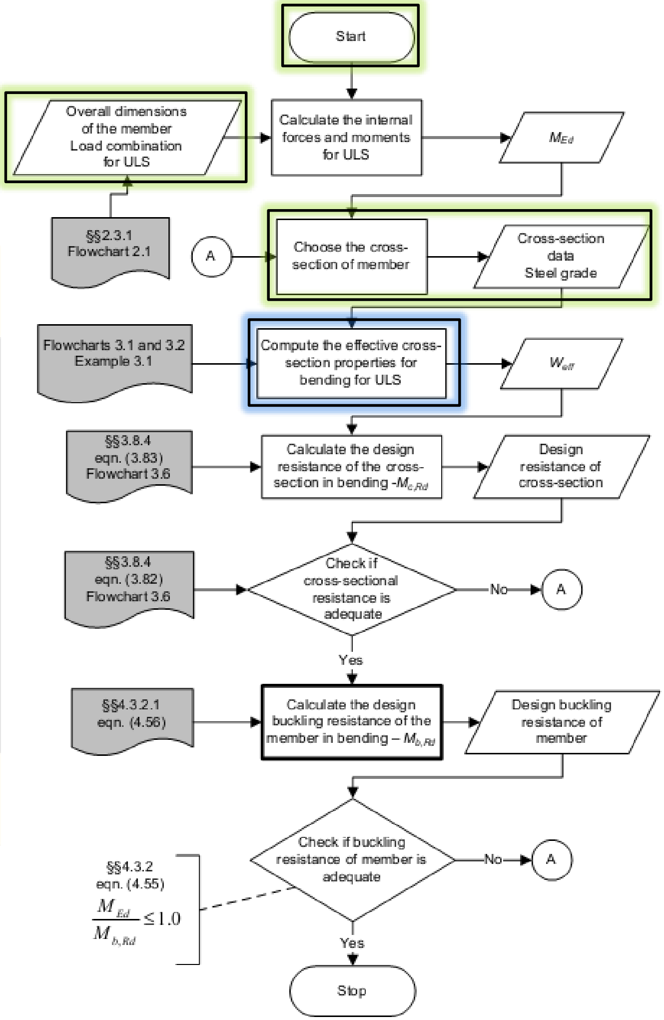

Resistance of members

Buckling strength of bending members

- *Example – Design of an cold-formed steel beam in bending*

Effective section properties at the ultimate limit state

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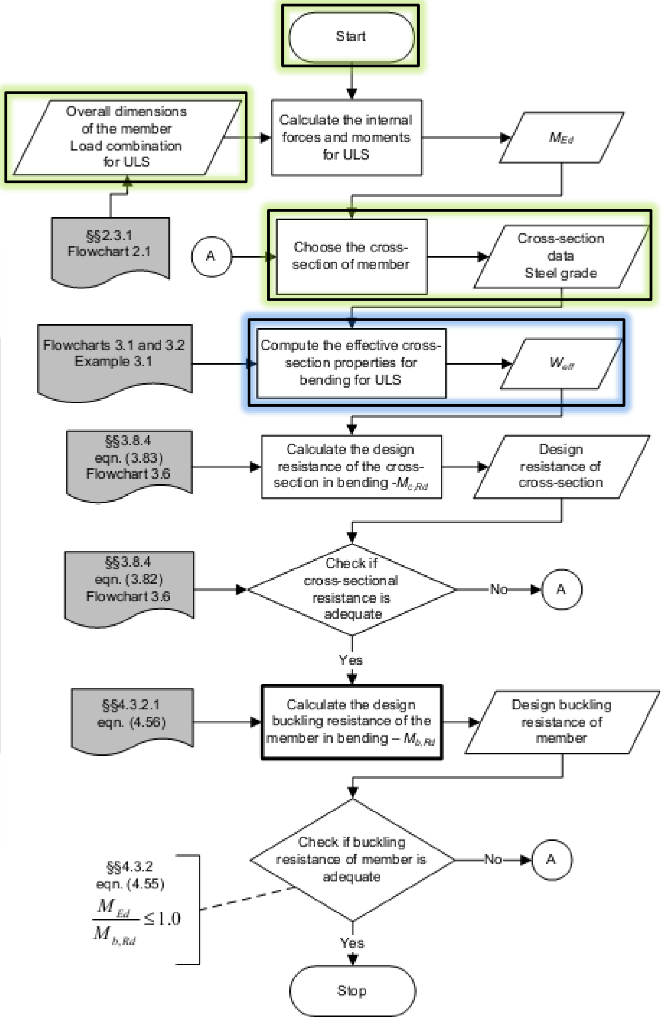
- with respect to the flange in tension:

$$W_{eff,y,t} = \frac{I_{eff,y}}{z_t} = \frac{22688890}{122.4} = 185367 \text{ mm}^3$$

$$W_{eff,y} = \min\left(W_{eff,y,c}, W_{eff,y,t}\right) = 182094 \text{ mm}^3$$



Resistance of members

Buckling strength of bending members

- ***Example – Design of an cold-formed steel beam in bending***

Applied loading on the beam at ULS

$$q_d = \gamma_G q_G + \gamma_Q q_Q = 1.35 \times 1.79 + 1.50 \times 4.5 = 9.17\,\mathrm{kN/m}$$

Maximum applied bending moment about the major axis *y-y*:

$$M_{Ed} = q_d L^2/8 = 9.17 \times 4.5^2/8 = 23.21\,\mathrm{kNm}$$

Check of bending resistance at ULS

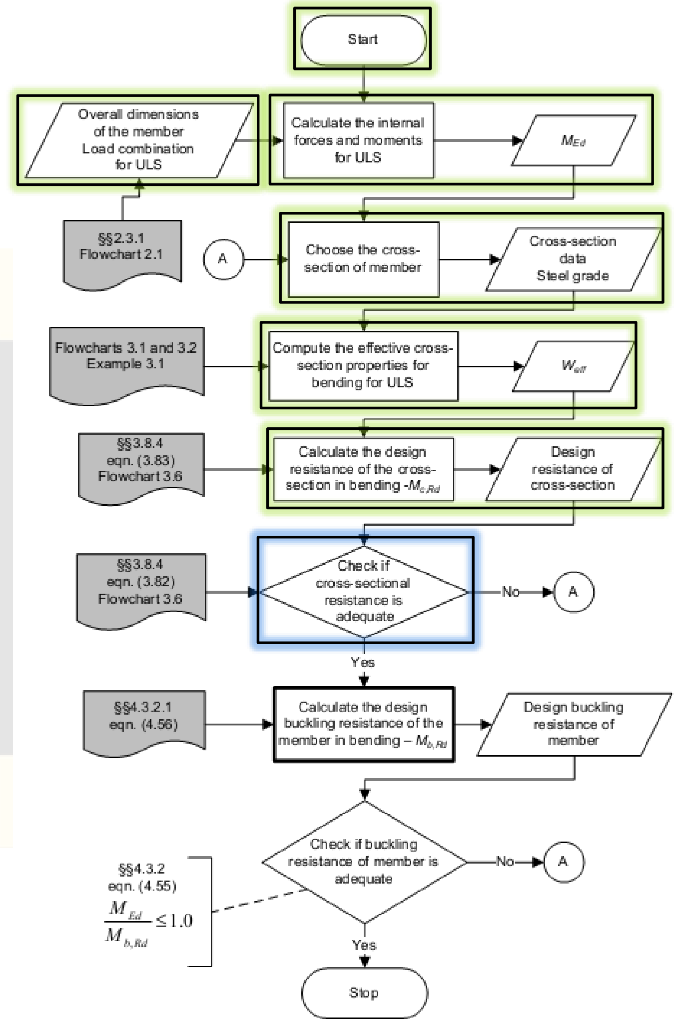
Design moment resistance of the cross section for bending

$$M_{c,Rd} = W_{eff,y} f_{yb}/\gamma_{M0} = 182094 \times 10^{-9} \times 350 \times 10^3/1.0 =$$

$$= 63.73\,\mathrm{kNm}$$

Verification of bending resistance

$$\frac{M_{Ed}}{M_{c,Rd}} = \frac{23.21}{63.73} = 0.364 < 1 \quad – \text{OK}$$



Resistance of members

Buckling strength of bending members

- *Example – Design of an cold-formed steel beam in bending*

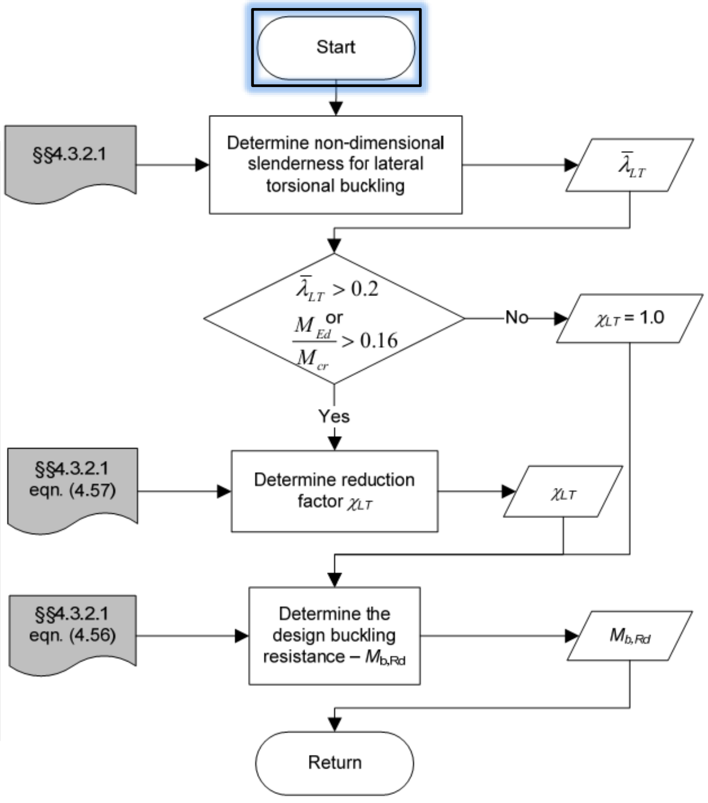
Determination of the reduction factor χ_{LT}

Lateral-torsional buckling

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \text{ but } \chi_{LT} \leq 1.0$$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^2\right]$$

$$\alpha_{LT} = 0.34 \text{ – buckling curve } b$$



Resistance of members

Buckling strength of bending members

- *Example – Design of an cold-formed steel beam in bending*

Determination of the reduction factor χ_{LT}

Lateral-torsional buckling

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \quad \text{but} \quad \chi_{LT} \leq 1.0$$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^2\right]$$

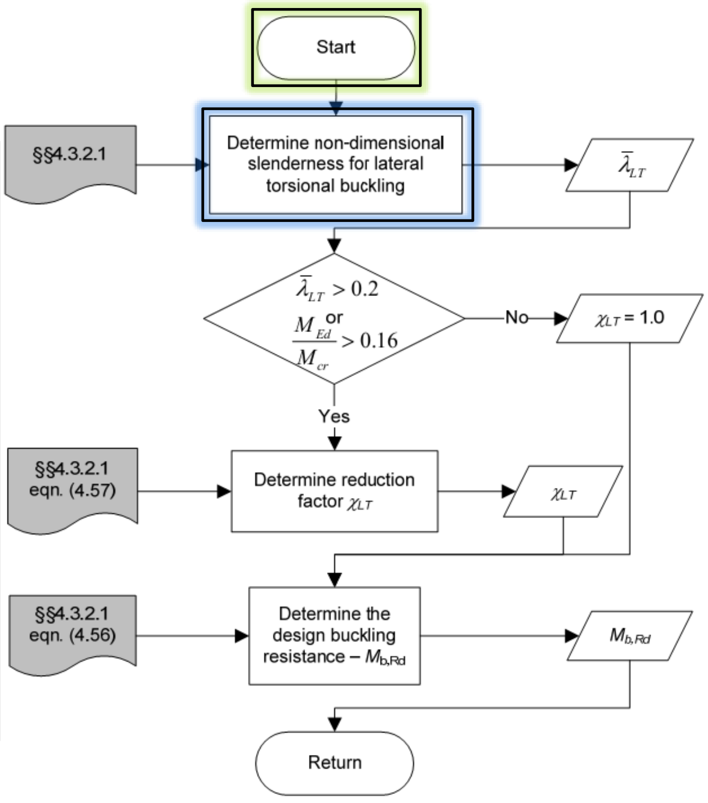
$$\alpha_{LT} = 0.34 \quad \text{– buckling curve } b$$

The non-dimensional slenderness is

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{eff,y,min} f_{yb}}{M_{cr}}}$$

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z}}$$

$C_1 = 1.127$ for a simply supported beam under uniform loading



Resistance of members

Buckling strength of bending members

- *Example – Design of an cold-formed steel beam in bending*

Determination of the reduction factor χ_{LT}

$$M_{cr} = 1.127 \times \frac{\pi^2 \times 210000 \times 244.24 \times 10^4}{4500^2} \times \sqrt{\frac{17692.78 \times 10^6}{244.24 \times 10^4} + \frac{4500^2 \times 81000 \times 7400}{\pi^2 \times 210000 \times 244.24 \times 10^4}}$$

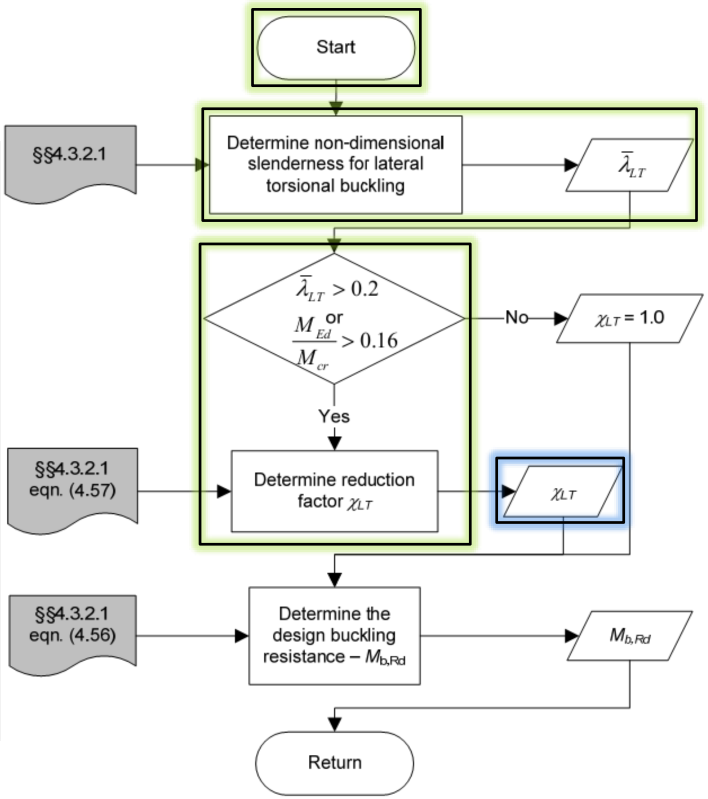
$$M_{cr} = 27.66 \text{ kNm}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{eff,y,min} f_{yb}}{M_{cr}}} = \sqrt{\frac{182094 \times 350}{27.66 \times 10^6}} = 1.518$$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^{\ 2}\right] =$$

$$= 0.5 \times \left[1 + 0.34 \times (1.437 - 0.2) + 1.437^2\right] = 1.743$$

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^{\ 2} - \bar{\lambda}_{LT}^{\ 2}}} = \frac{1}{1.743 + \sqrt{1.734^2 - 1.437^2}} = 0.369$$



Resistance of members

Buckling strength of bending members

- ***Example – Design of an cold-formed steel beam in bending***

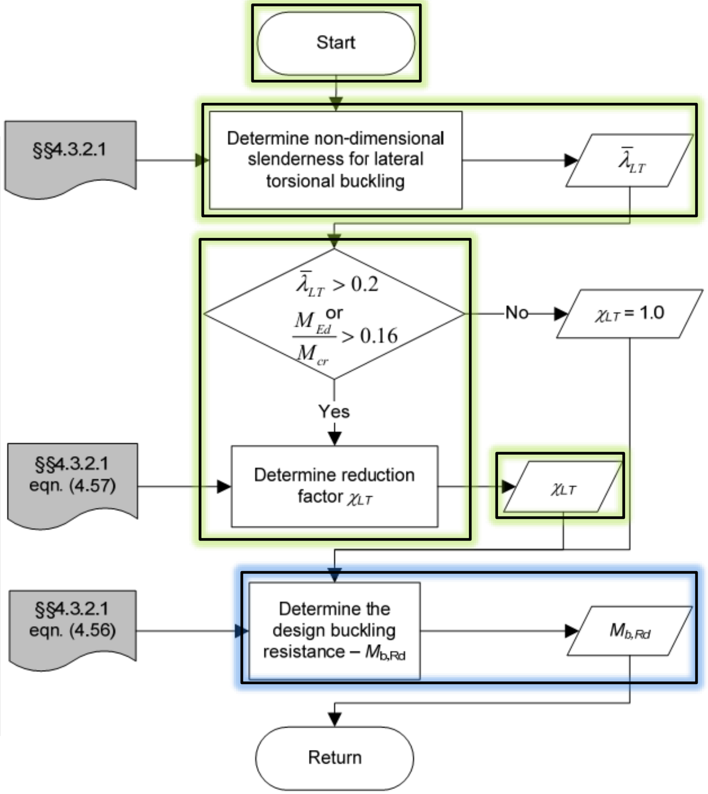
Check of buckling resistance at ULS

Design moment resistance of the cross section for bending

$$M_{b,Rd} = \chi_{LT} W_{eff,y} f_{yb} / \gamma_{M1} =$$

$$= 0.369 \times 182091 \times 10^{-9} \times 350 \times 10^{3} / 1.0 =$$

$$= 23.52 \text{ kNm}$$



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Buckling strength of bending members

- *Example – Design of an cold-formed steel beam in bending*

Check of buckling resistance at ULS

Design moment resistance of the cross section for bending

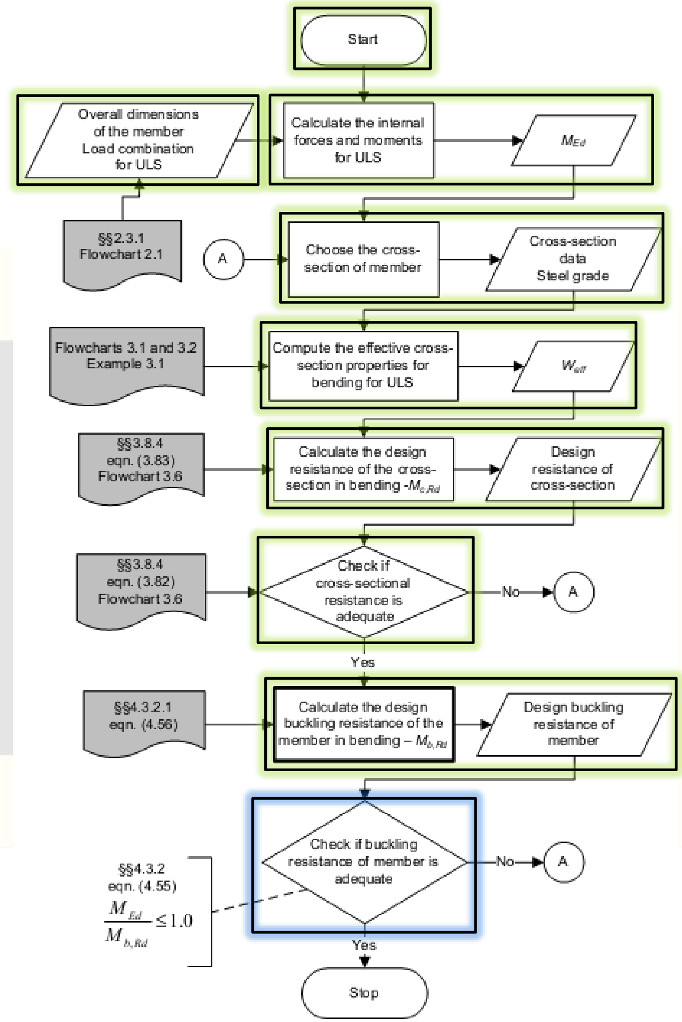
$$M_{b,Rd} = \chi_{LT} W_{eff,y} f_{yb} / \gamma_{M1} =$$

$$= 0.369 \times 182091 \times 10^{-9} \times 350 \times 10^{3} / 1.0 =$$

$$= 23.52 \text{ kNm}$$

<u>Verification of buckling resistance</u>

$$\frac{M_{Ed}}{M_{b,Rd}} = \frac{23.21}{23.52} = 0.987 < 1 \ \text{– OK}$$



Buckling of members in bending and axial compression

- **Design of beam-columns according to EN1993-1-1 and EN1993-1-3**

Two different formats of the interaction formulae

Method 1 (Annex A of EN 1993–1–1) contains a set of formulae that favours transparency and provides a wide range of applicability together with a high level of accuracy and consistency.

Method 2 (Annex B of EN 1993–1–1) is based on the concept of global factors, in which simplicity prevails against transparency. This approach appears to be the more straightforward in terms of a general format.

Resistance of members

Buckling of members in bending and axial compression

- **Design of beam-columns according to EN1993-1-1 and EN1993-1-3**

Members which are subjected to combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}} + k_{yy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} + k_{yz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{M_{z,Rk}/\gamma_{M1}} \leq 1.0$$

$$\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} + k_{zy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} + k_{zz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{M_{z,Rk}/\gamma_{M1}} \leq 1.0$$

Buckling of members in bending and axial compression

- **Design of beam-columns according to EN1993-1-1 and EN1993-1-3**

Members which are subjected to combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}} + k_{yy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} + k_{yz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{M_{z,Rk}/\gamma_{M1}} \le 1.0$$

$$\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} + k_{zy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} + k_{zz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{M_{z,Rk}/\gamma_{M1}} \le 1.0$$

Values for $N_{Rk} = f_y A_i$, $M_{i,Rk} = f_y W_i$ and $\Delta M_{i,Ed}$

Class	1	2	3	4
A_i	A	A	A	A_{eff}
W_y	$W_{pl,y}$	$W_{pl,y}$	$W_{el,y}$	$W_{eff,y}$
W_z	$W_{pl,z}$	$W_{pl,z}$	$W_{el,z}$	$W_{eff,z}$
$\Delta M_{y,Ed}$	0	0	0	$e_{N,y} N_{Ed}$
$\Delta M_{z,Ed}$	0	0	0	$e_{N,z} N_{Ed}$

Resistance of members

Buckling of members in bending and axial compression

- **Design of beam-columns according to EN1993-1-1 and EN1993-1-3**

Members which are subjected to combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}} + k_{yy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} + k_{yz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{M_{z,Rk}/\gamma_{M1}} \leq 1.0$$

$$\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} + k_{zy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} + k_{zz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{M_{z,Rk}/\gamma_{M1}} \leq 1.0$$

The interaction factors k_{yy}, k_{yz}, k_{zy}, k_{zz} depend on the method which is chosen, being derived from two alternative approaches: (1) Alternative method 1 – see Tables 4.7 and 4.8 (Annex A of EN1993-1-1) and (2) Alternative method 2 – see Tables 4.9, 4.10 and 4.11 (Annex B of EN1993-1-1).

Buckling of members in bending and axial compression

- **Design of beam-columns according to EN1993-1-1 and EN1993-1-3**

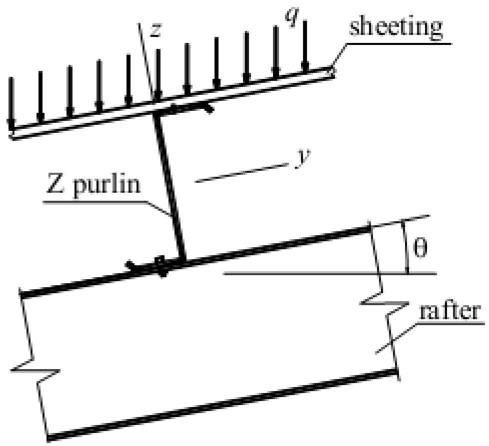
As an alternative, the interaction formula may be used

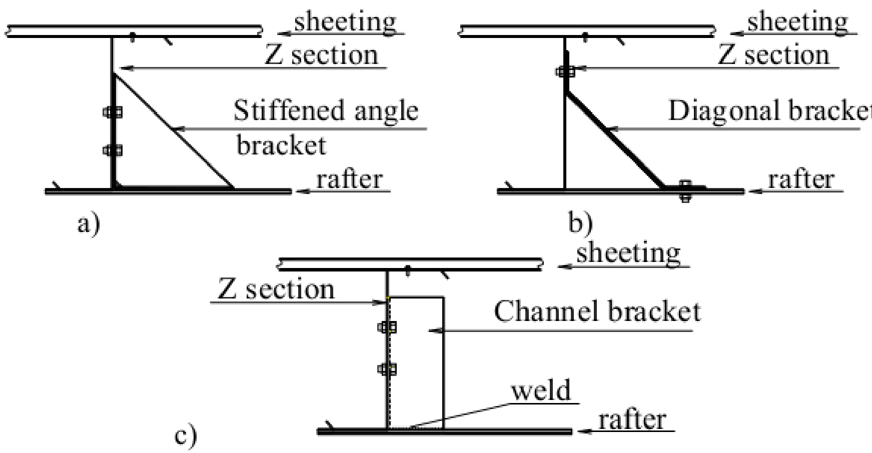
$$\left(\frac{N_{Ed}}{N_{b,Rd}}\right)^{0.8}+\left(\frac{M_{Ed}}{M_{b,Rd}}\right)^{0.8}\leq 1.0$$

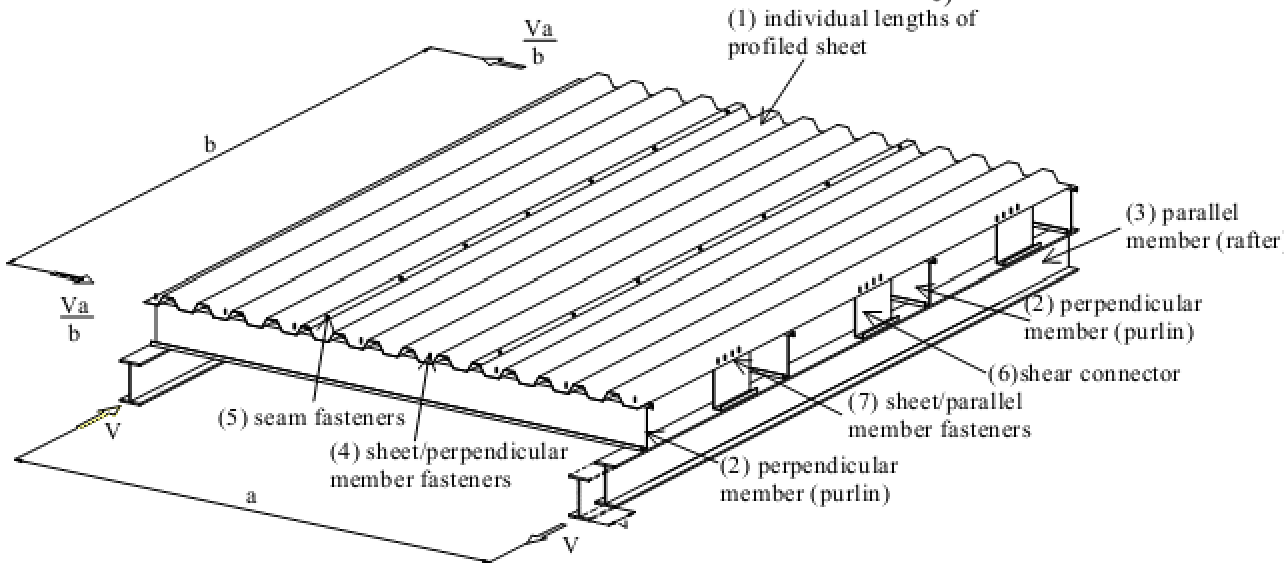
Behaviour and Design Resistance of Bar Members

Beams restrained by sheeting

- **General. Constructional detailing and static system**





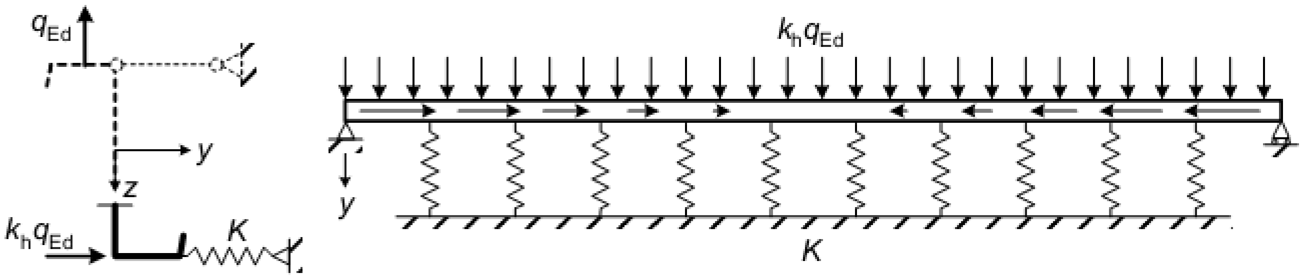


Behaviour and Design Resistance of Bar Members

Beams restrained by sheeting

- **Modeling of beam-sheeting interaction**

To evaluate the restraining effect of sheeting, in EN1993-1-3 the free flange is considered as a beam on an elastic foundation



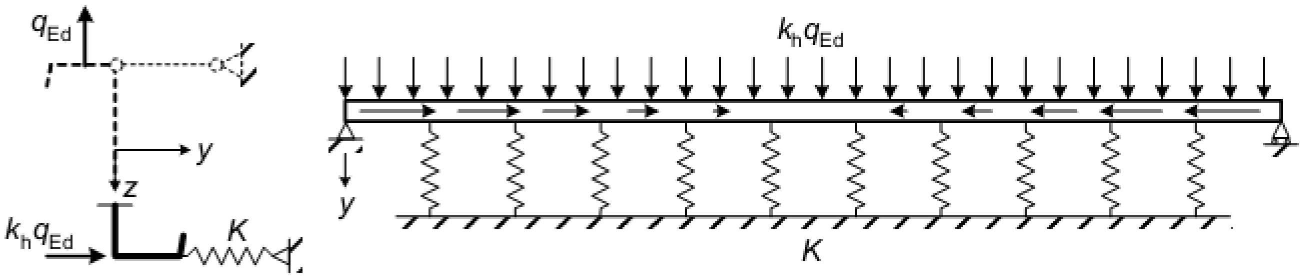
Resistance of members

Behaviour and Design Resistance of Bar Members

Beams restrained by sheeting

- **Modeling of beam-sheeting interaction**

To evaluate the restraining effect of sheeting, in EN1993-1-3 the free flange is considered as a beam on an elastic foundation

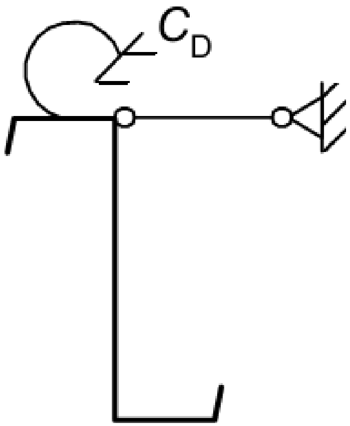


The equivalent lateral spring stiffness for the strength and stability check is obtained by a combination of:

Behaviour and Design Resistance of Bar Members

Beams restrained by sheeting

- **Modeling of beam-sheeting interaction**



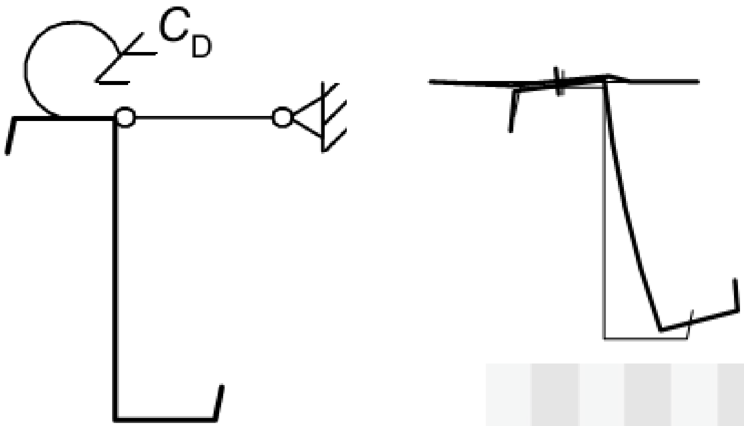
The equivalent lateral spring stiffness for the strength and stability check is obtained by a combination of:

1. Rotational stiffness of the connection between the sheeting and the purlin C_D,

Behaviour and Design Resistance of Bar Members

Beams restrained by sheeting

- Modeling of beam-sheeting interaction



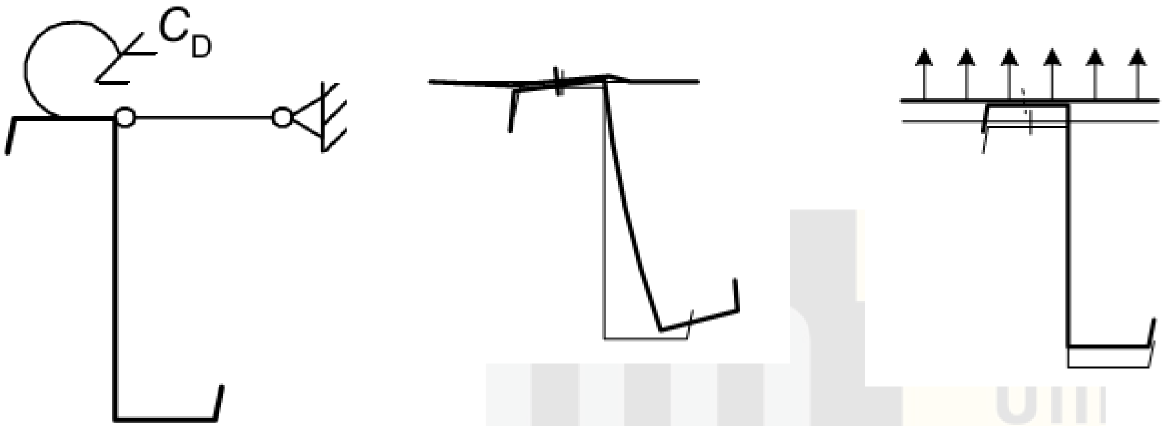
The equivalent lateral spring stiffness for the strength and stability check is obtained by a combination of:

1. Rotational stiffness of the connection between the sheeting and the purlin C_D,
2. Distortion of the cross section of the purlin, K_B,

Behaviour and Design Resistance of Bar Members

Beams restrained by sheeting

- **Modeling of beam-sheeting interaction**



The equivalent lateral spring stiffness for the strength and stability check is obtained by a combination of:

1. Rotational stiffness of the connection between the sheeting and the purlin C_D,
2. Distortion of the cross section of the purlin, K_B,
3. Bending stiffness of the sheeting, $C_{D,C}$, perpendicular to the span of the purlin (see Figure 4.49).

Behaviour and Design Resistance of Bar Members

Beams restrained by sheeting

- **Modeling of beam-sheeting interaction**

According to EN1993-1-3, the partial torsional restraint may be represented by a rotational spring with a spring stiffness C_D, which can be calculated based on the stiffness of the sheeting and the connection between the sheeting and the purlin, as follows,

$$\frac{1}{C_D} = \frac{1}{C_{D,A}} + \frac{1}{C_{D,C}}$$

where

$C_{D,A}$ is the rotational stiffness of the connection between the sheeting and the purlin;

$C_{D,C}$ is the rotational stiffness corresponding to the flexural stiffness of the sheeting.

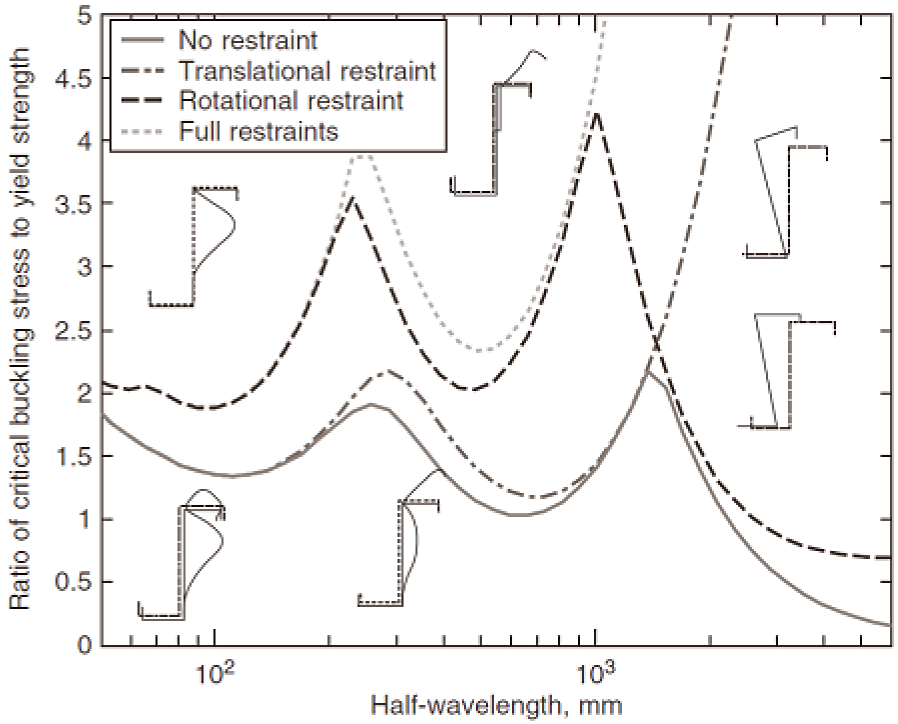
Both $C_{D,A}$ and $C_{D,C}$ are specified in Section 10.1.5 of EN1993-1-3.

Behaviour and Design Resistance of Bar Members

Beams restrained by sheeting

- **Modeling of beam-sheeting interaction**

The restraints of the sheeting to the purlin have important influence on the buckling behaviour of the purlin.



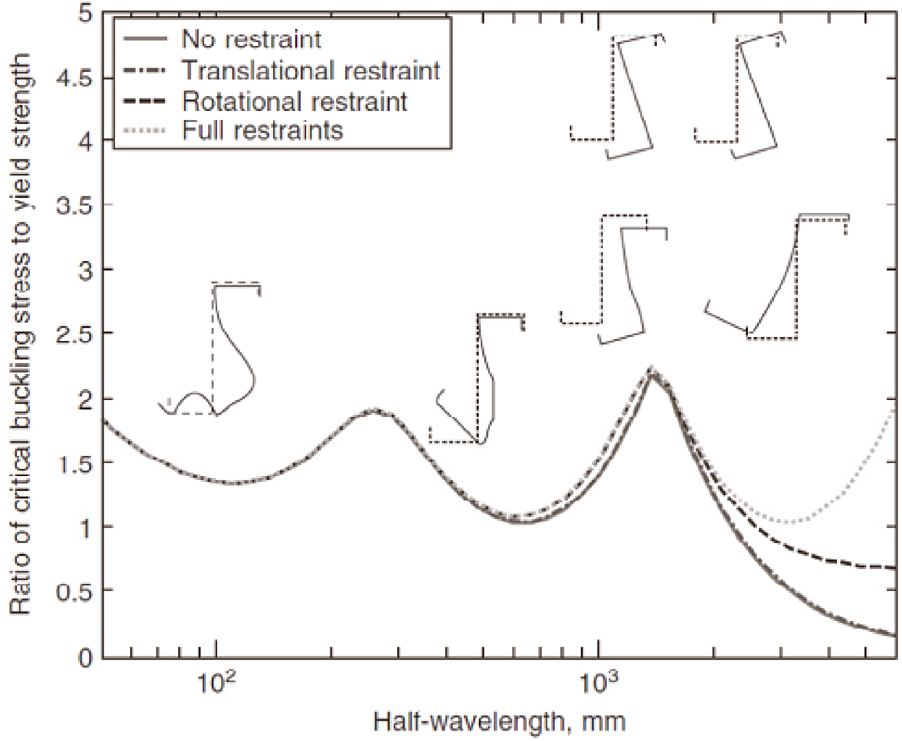
Buckling curves of a simply supported zed section beam with different restraint applied at the junction between web and compression flange subjected to pure bending (h=202 mm, b=75 mm, c=20 mm, t =2.3mm) (Martin & Purkiss, 2008)

Behaviour and Design Resistance of Bar Members

Beams restrained by sheeting

- **Modeling of beam-sheeting interaction**

The restraints of the sheeting to the purlin have important influence on the buckling behaviour of the purlin.



Buckling curves of a simply supported zed section beam with different restraint applied at the junction between web and tension flange subjected to pure bending (h=202 mm, b=75 mm, c=20 mm, t =2.3mm) (Martin & Purkiss, 2008)

Conceptual design

Introduction
Case Studies – Structural Performances

- **Conceptual design principles**

Designing cold-formed steel building structures is based on the conceptual framing of designed conventional steel structures

However, the designer has to manage four specific categories of problems which characterise the behaviour and performance of cold-formed thin-walled structures i.e.

- **Stability and local strength of sections;**
- **Connecting technology and related design procedures;**
- **Reduced ductility with reference to ductility, plastic design and seismic resistance;**
- **Low fire resistance**

If compares with conventional steel construction (EN 1993-1-1), the fact that structural members of a cold formed steel –structures are, in general, of class 4, involves to work with effective cross sections of members in compression - bending.

Even the general rules are similar, the members, stiffness is wakened by local instability increasing the risk of instability of these structures; their sensitivity to imperfections and 2nd order effects must carefully controlled by proper analyses and design.

Conceptual design

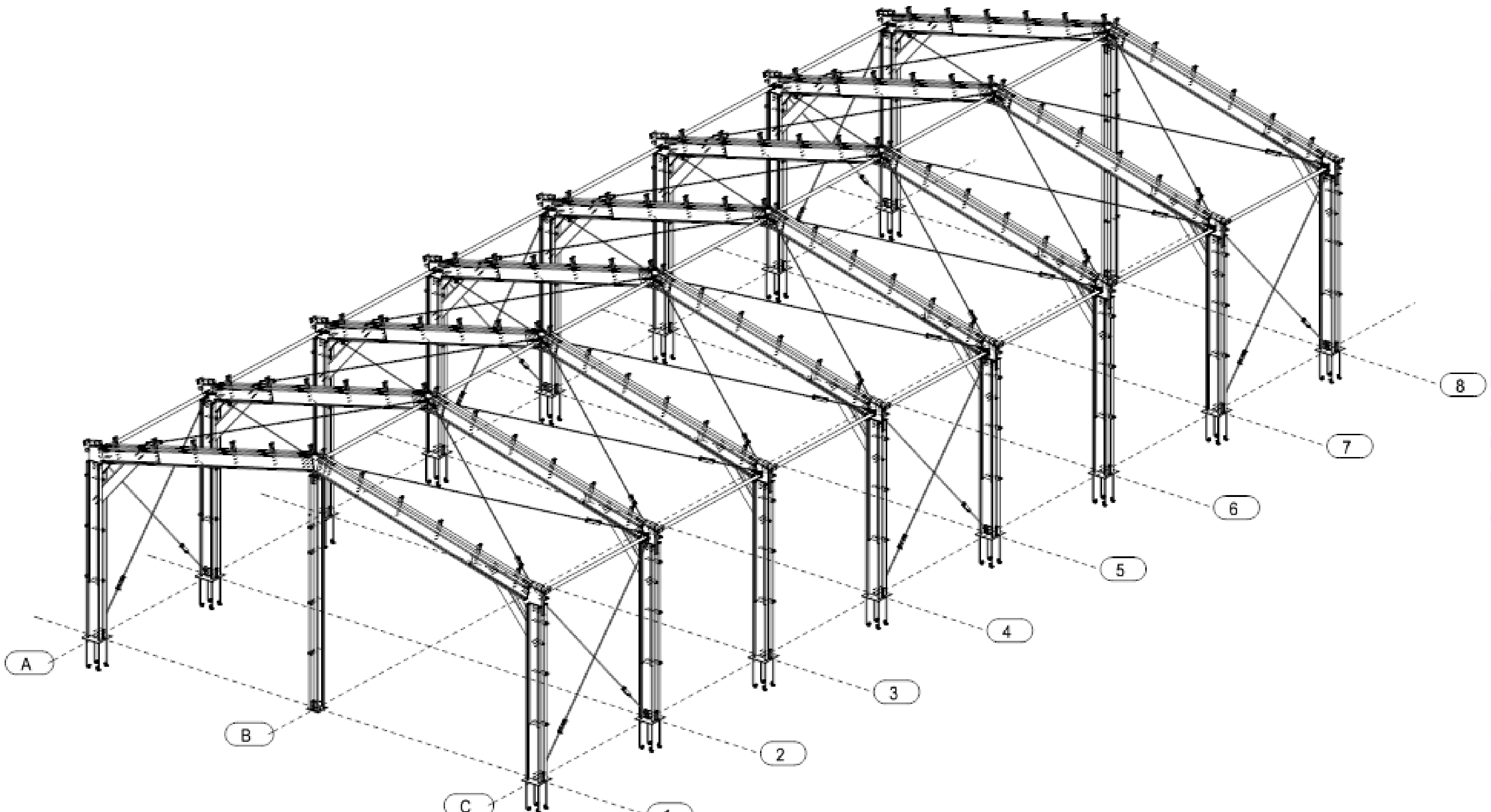
Case Studies

- Hall Type Modular System (HTMS) - POPET Hall

Conceptual design

Case Studies

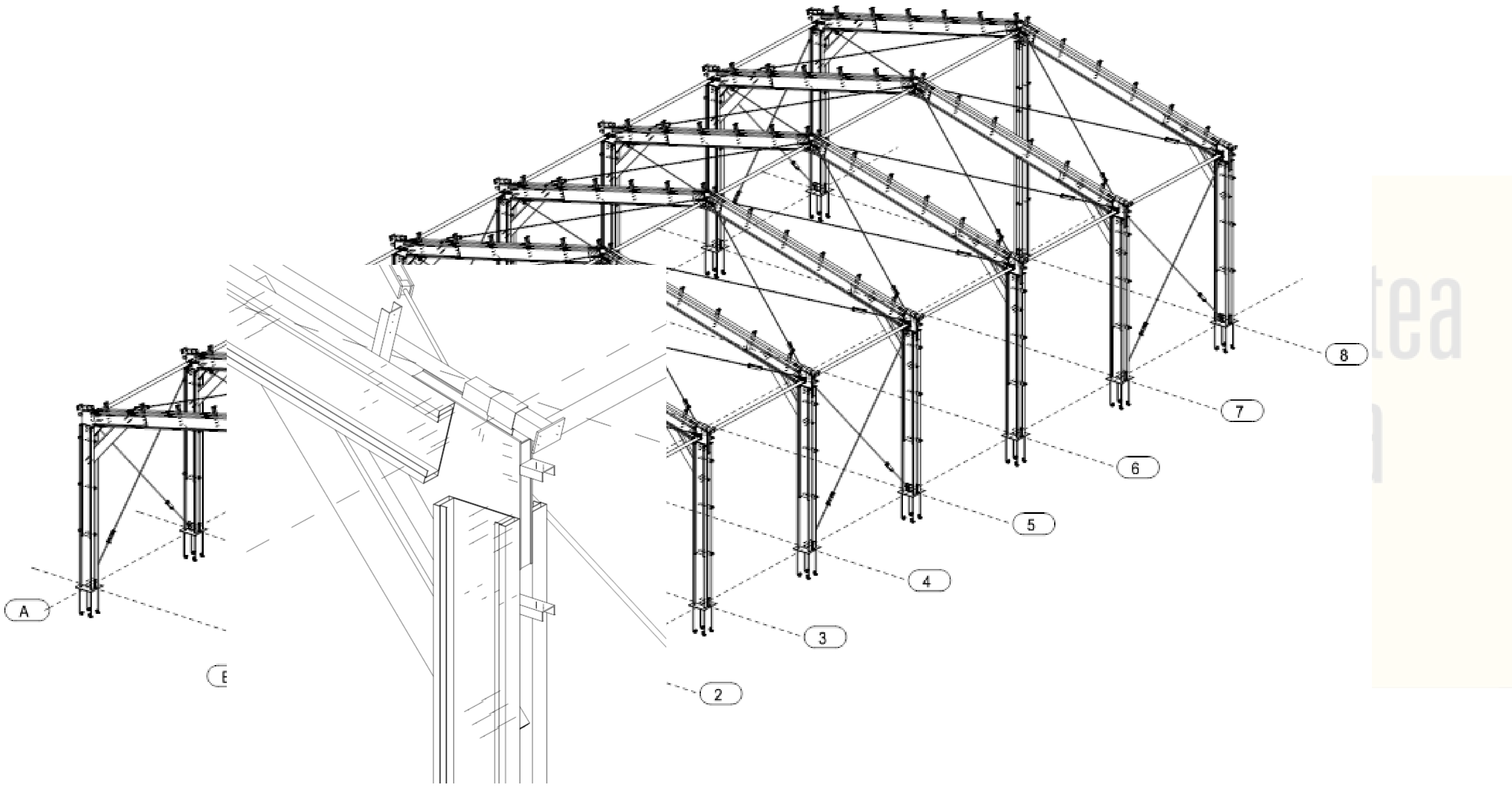
- **Hall Type Modular System (HTMS) - POPET Hall**



Conceptual design

Case Studies

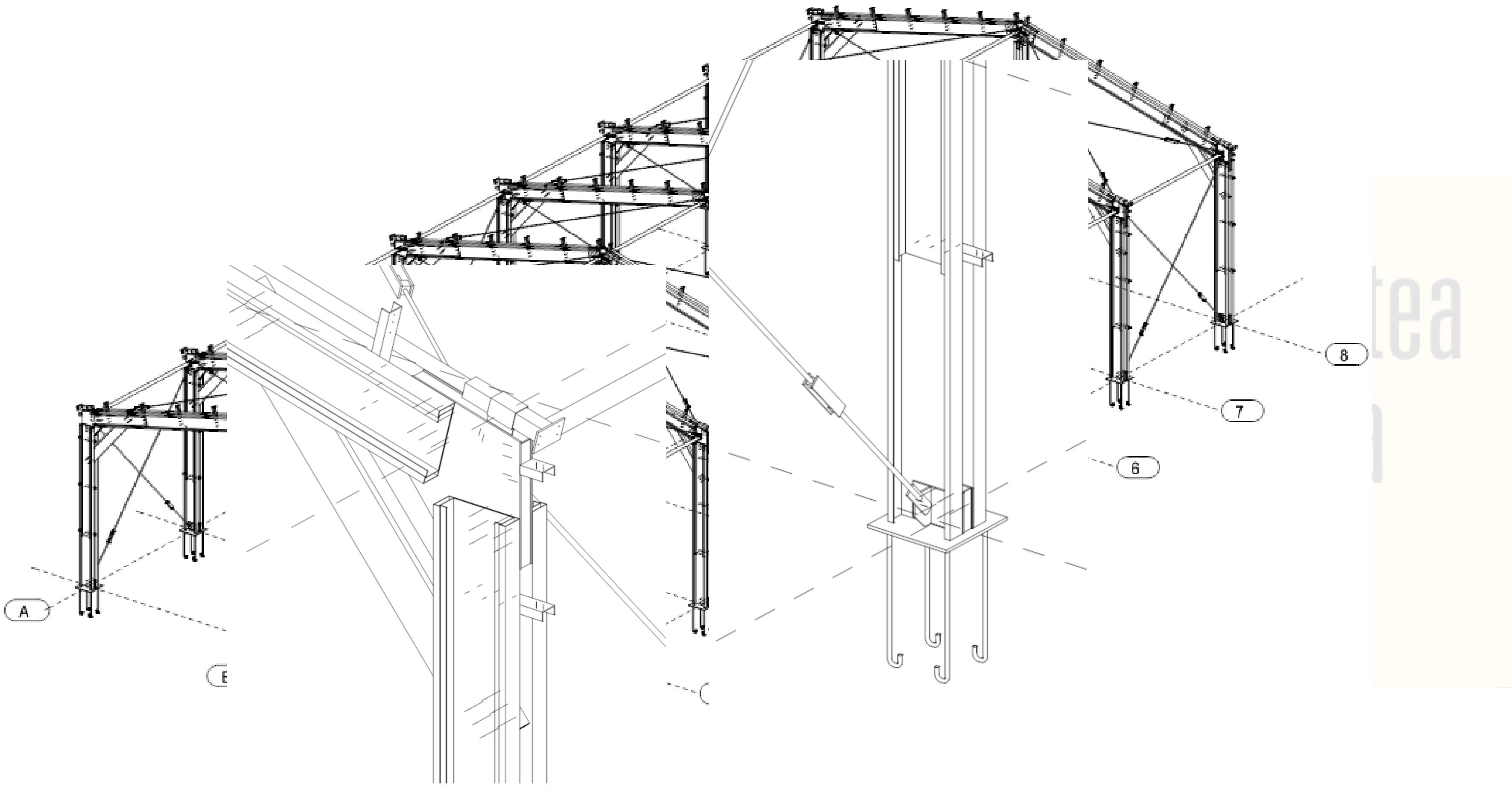
- **Hall Type Modular System (HTMS) - POPET Hall**



Conceptual design

Case Studies

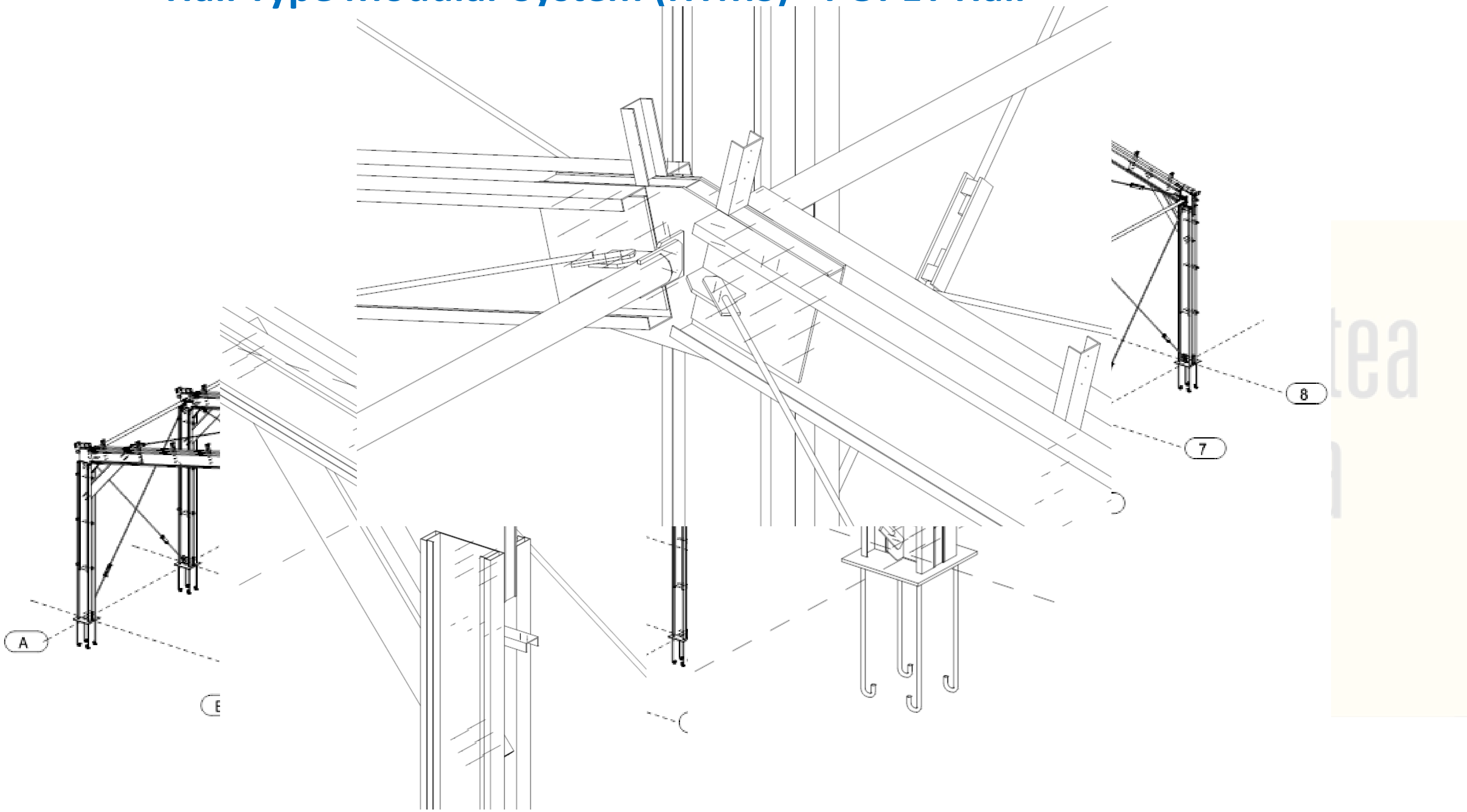
- **Hall Type Modular System (HTMS) - POPET Hall**



Conceptual design

Case Studies

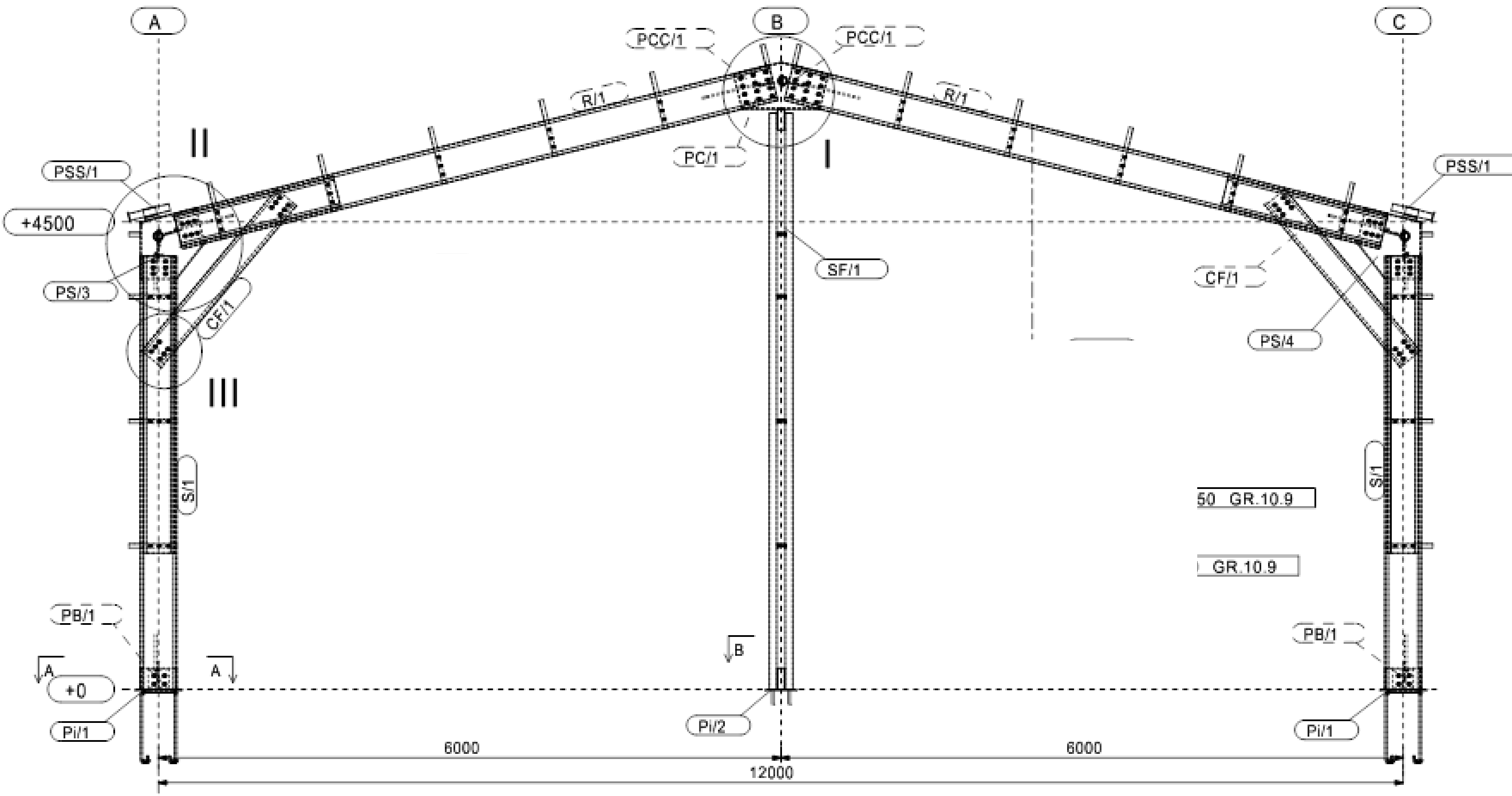
- **Hall Type Modular System (HTMS) - POPET Hall**



Conceptual design

Case Studies

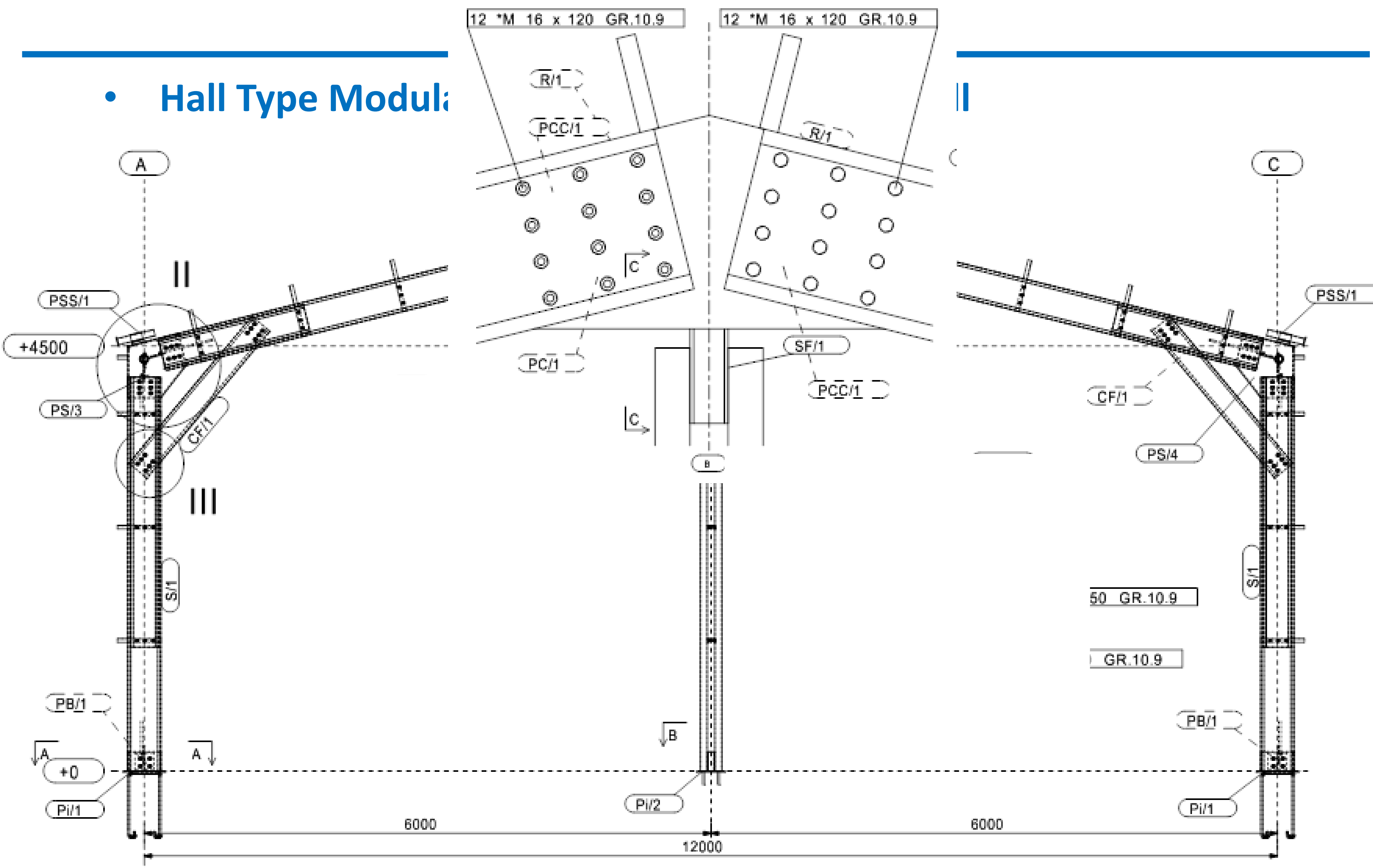
- **Hall Type Modular System (HTMS) - POPET Hall**



Conceptual design

Case Studies

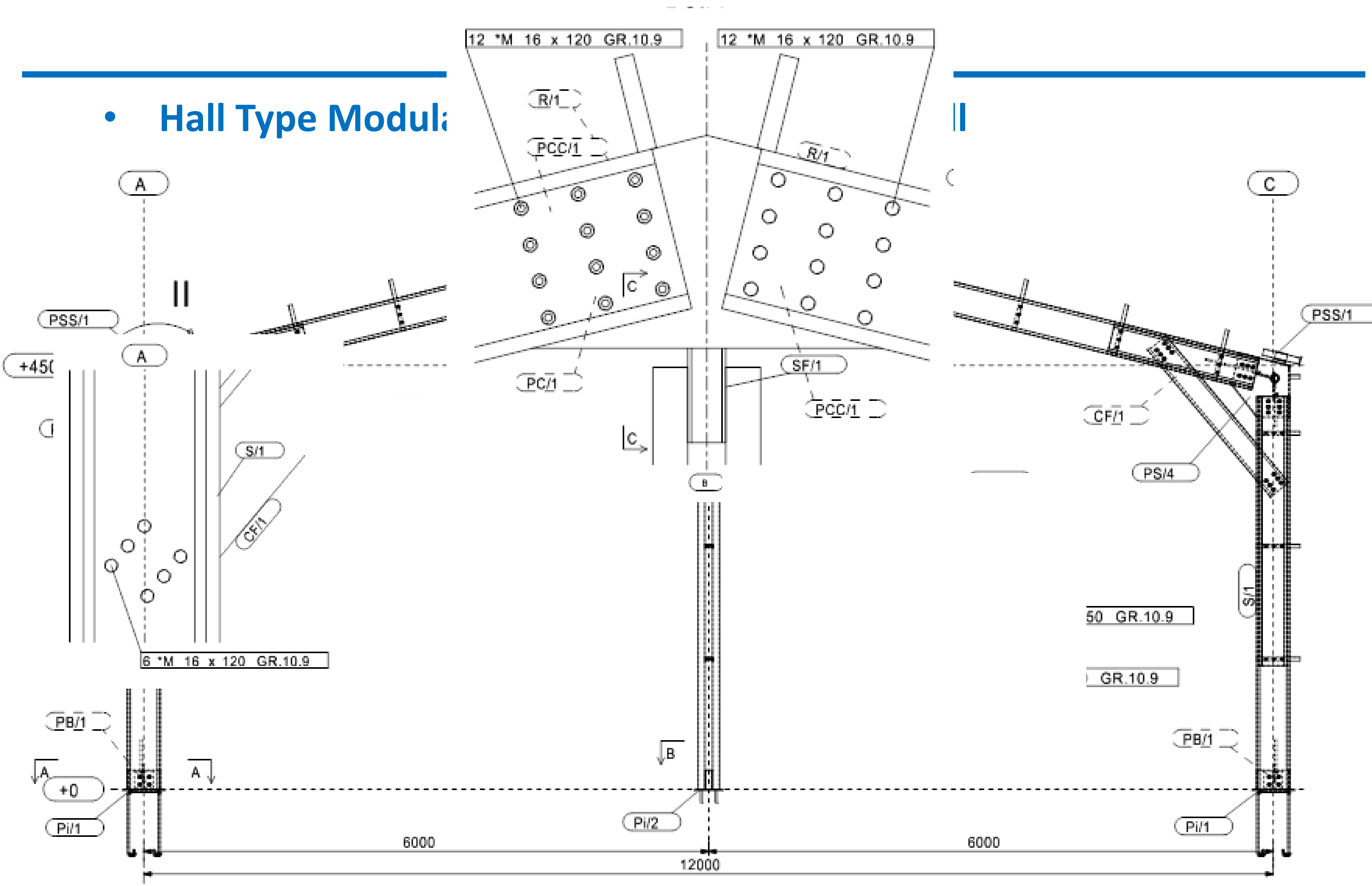
- Hall Type Modul ll



Conceptual design

Case Studies

- Hall Type Modul[...]



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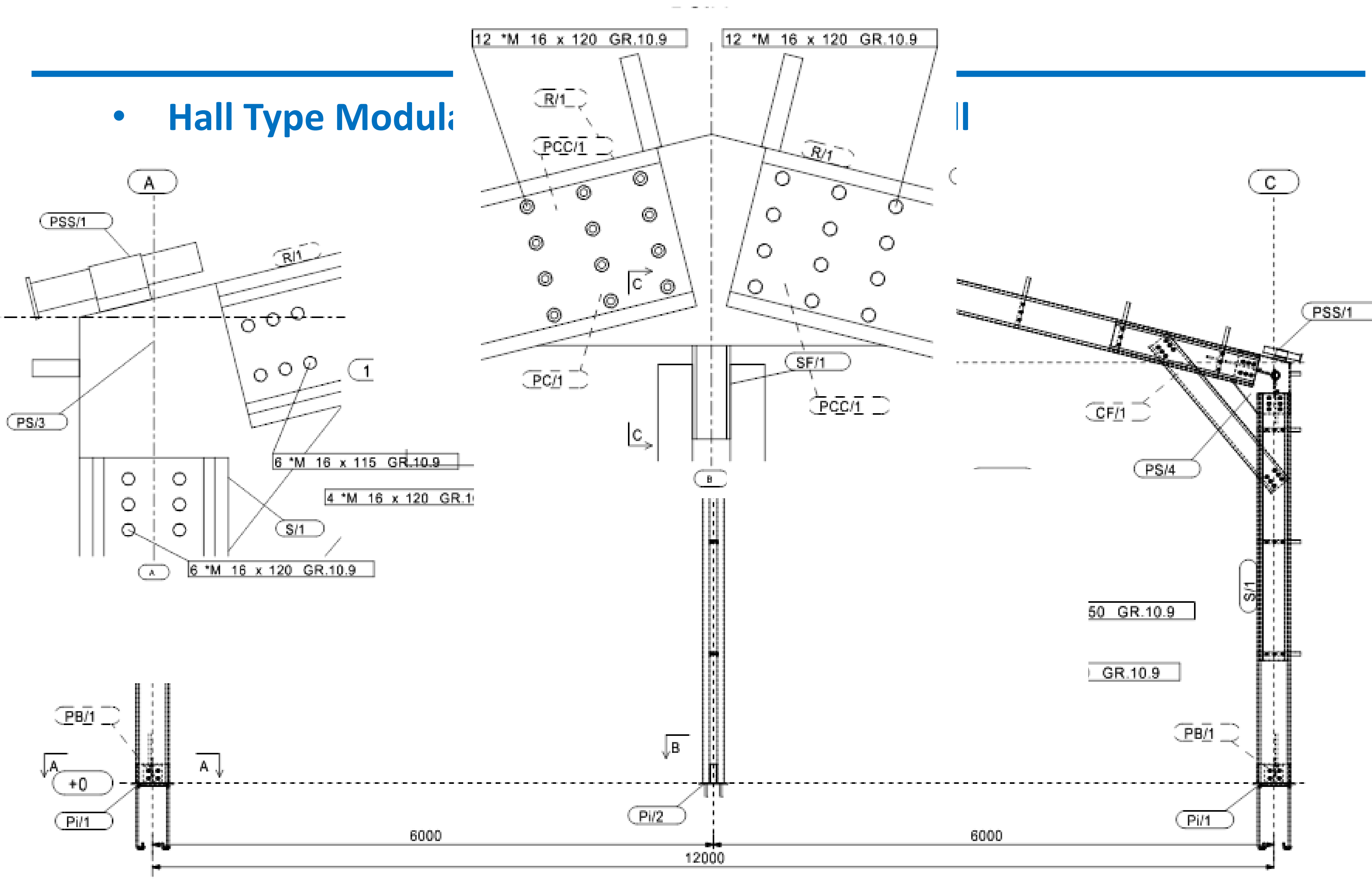
Conceptual design

Case Studies

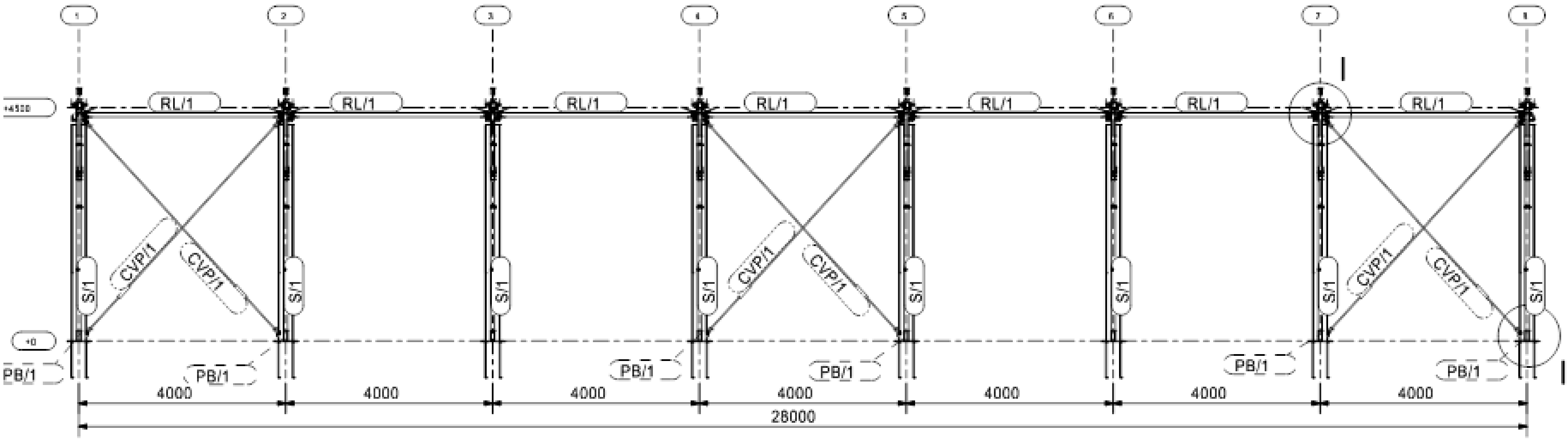
- Hall Type Modul



Conceptual design

Case Studies

- **Hall Type Modular System (HTMS) - POPET Hall**



Conceptual design

Case Studies

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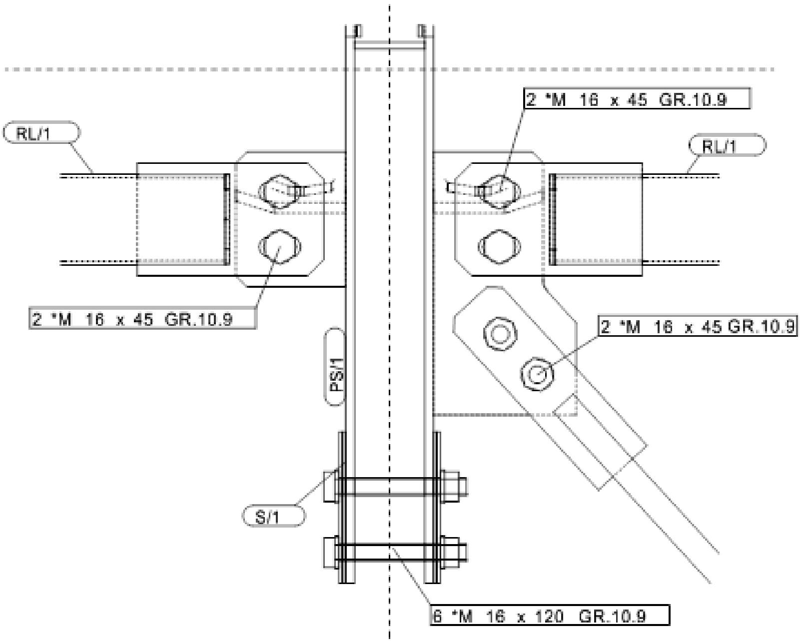

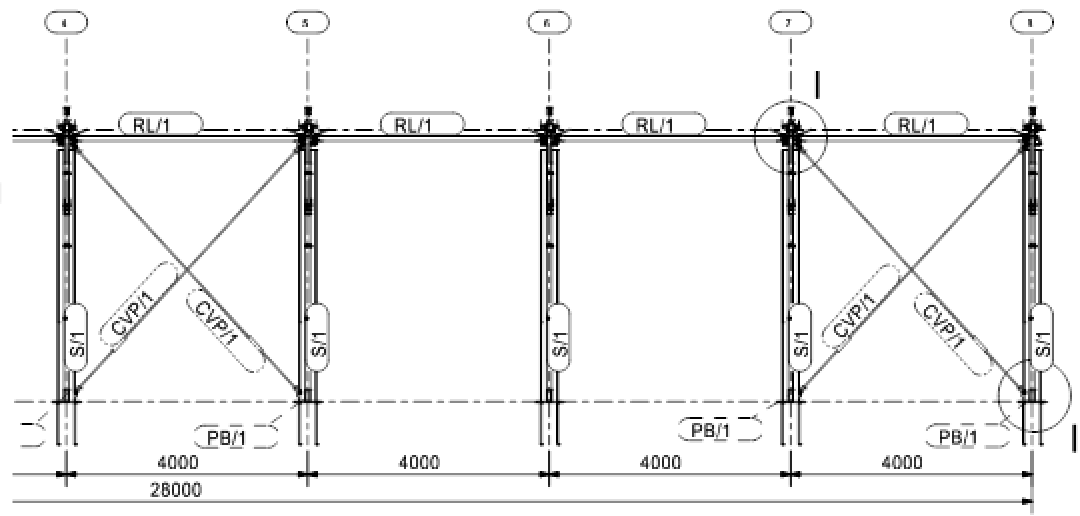

Conceptual design

Case Studies

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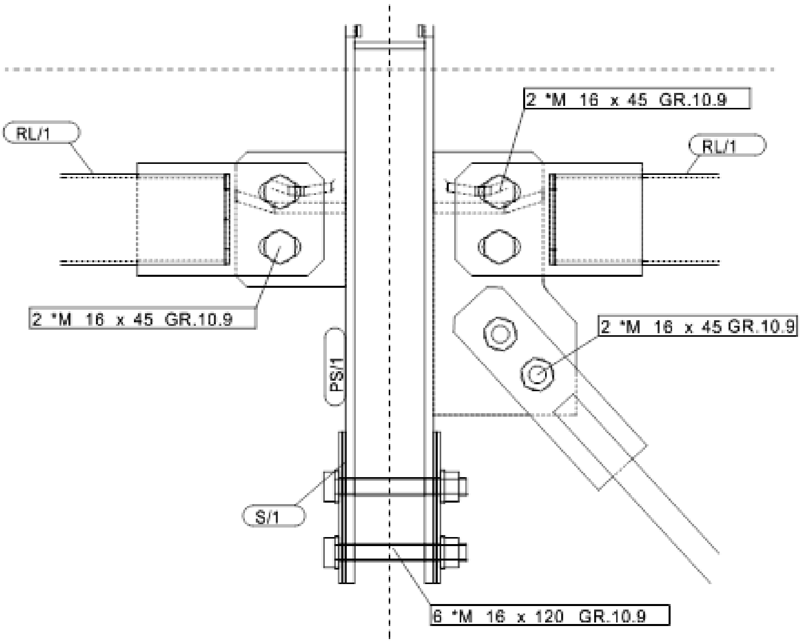
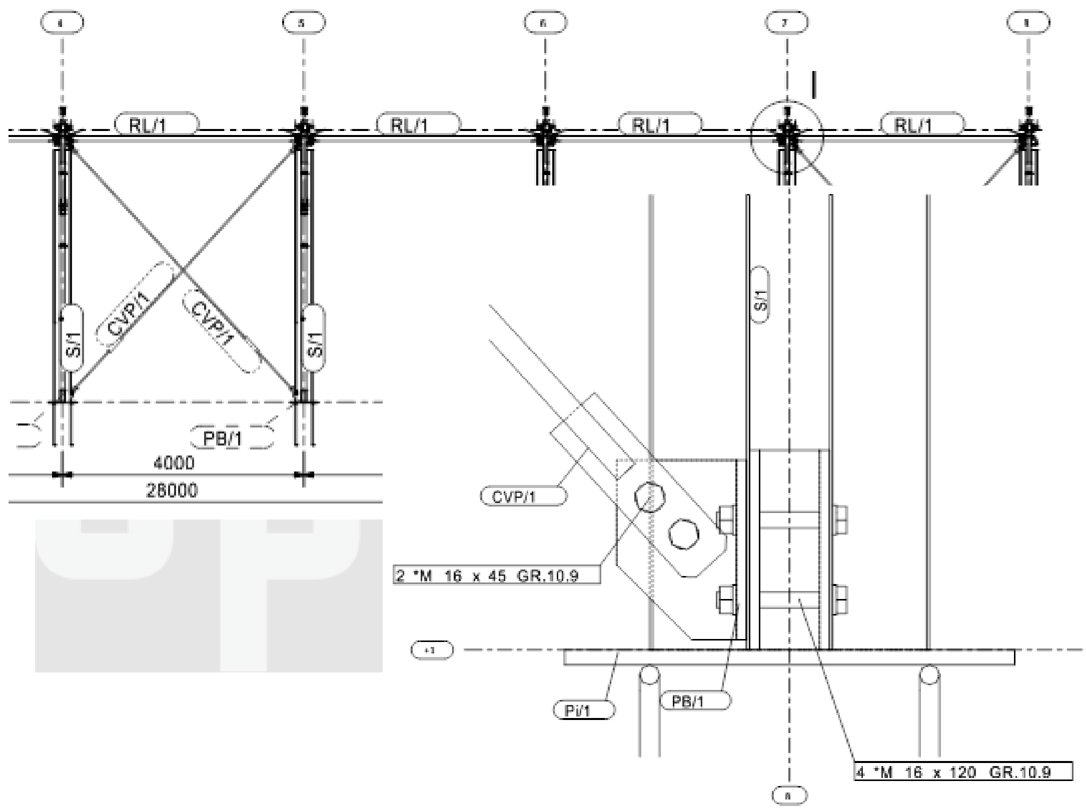
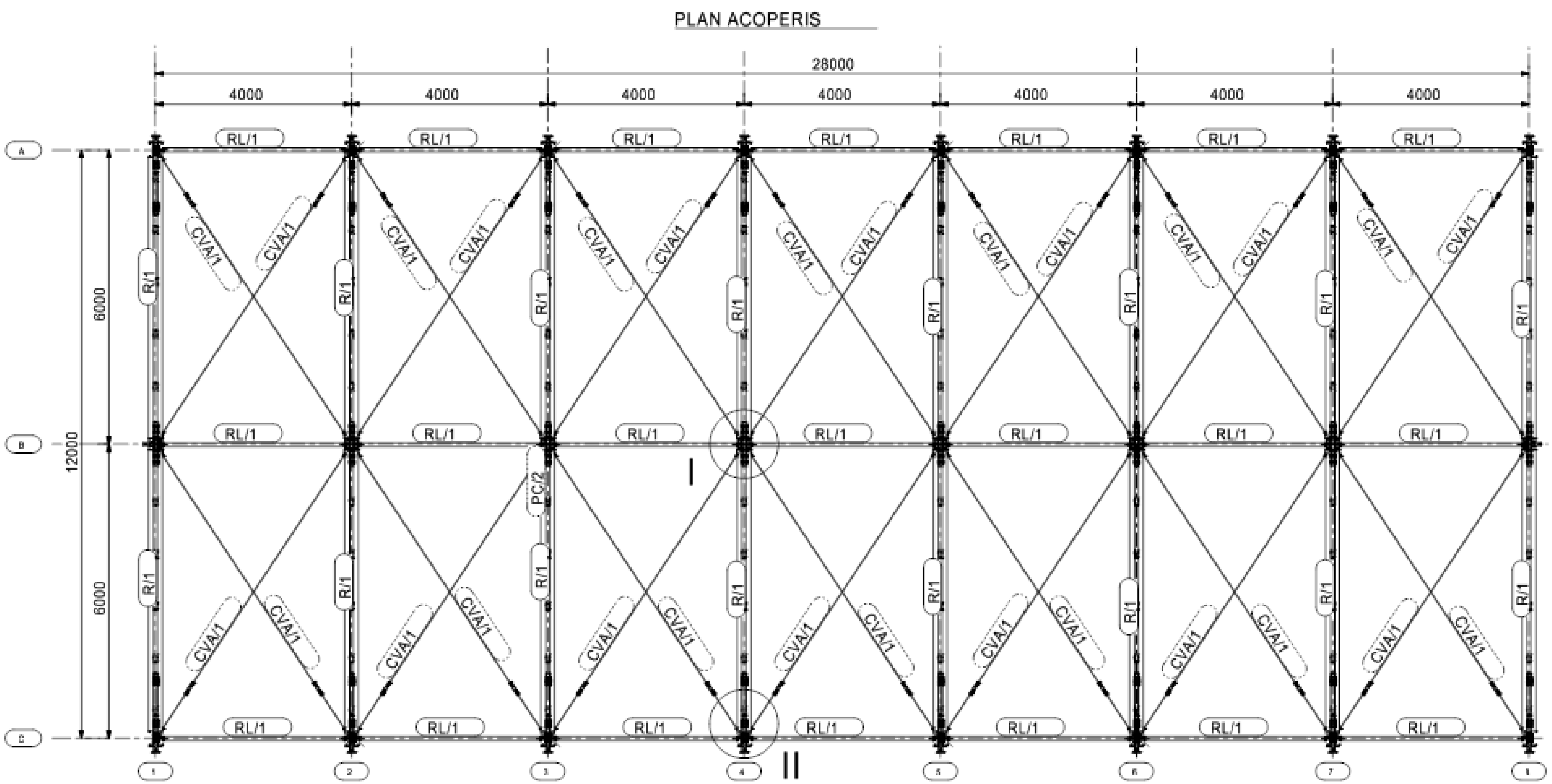




Conceptual design

Case studies

- **Hall Type Modular System (HTMS) - POPET Hall**



Conceptual design

Case studies

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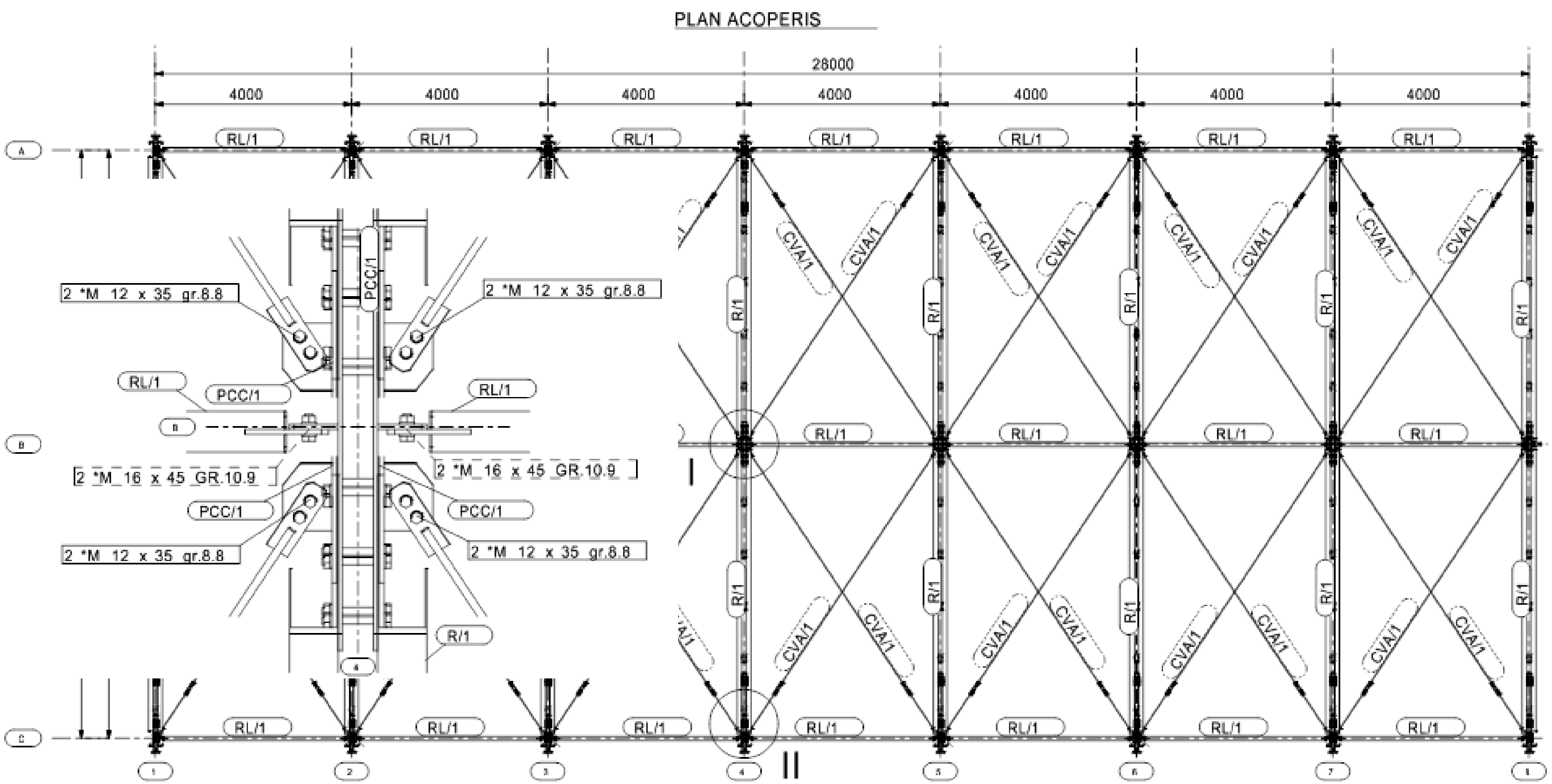

Conceptual design

Case studies

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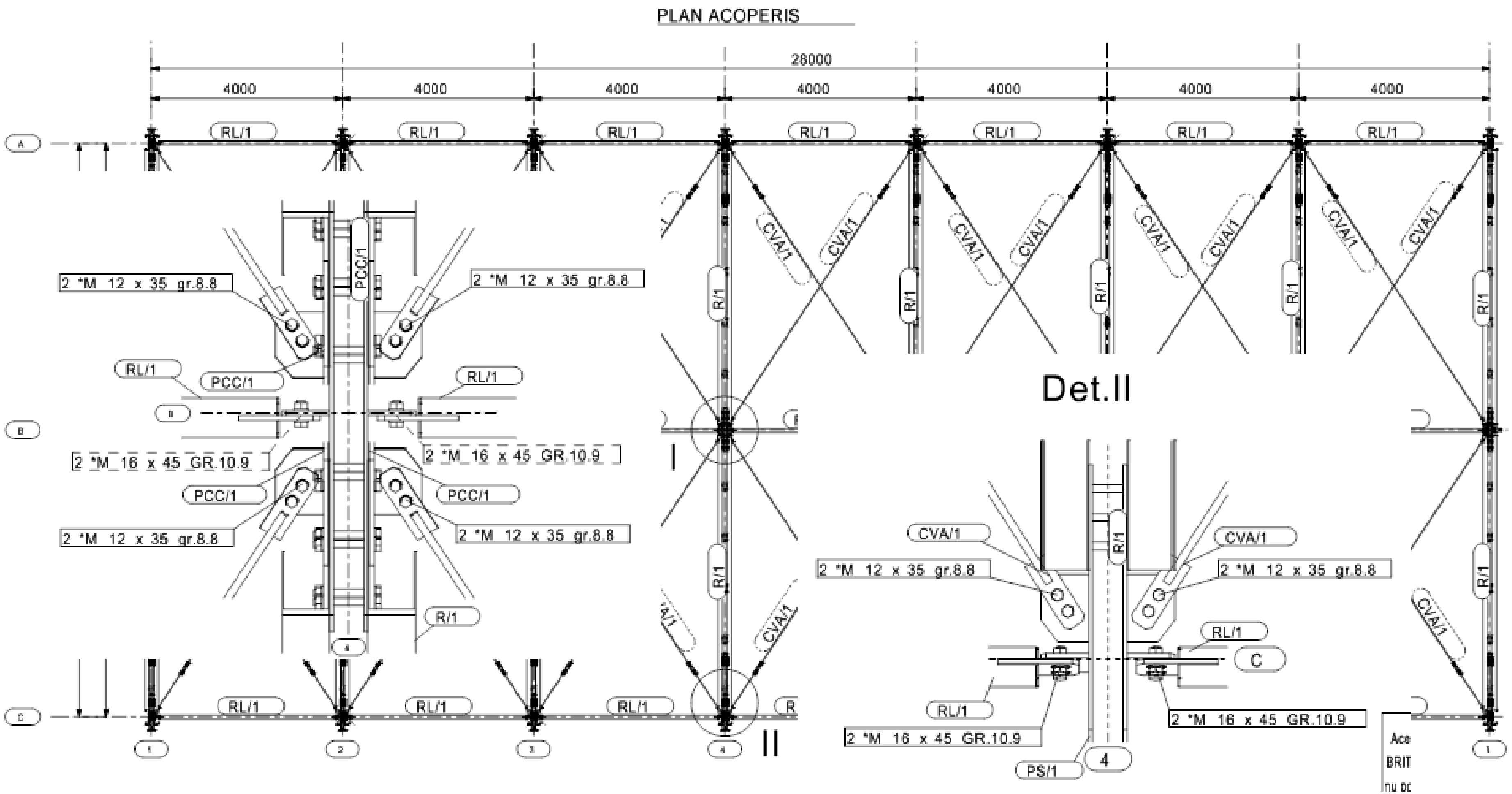
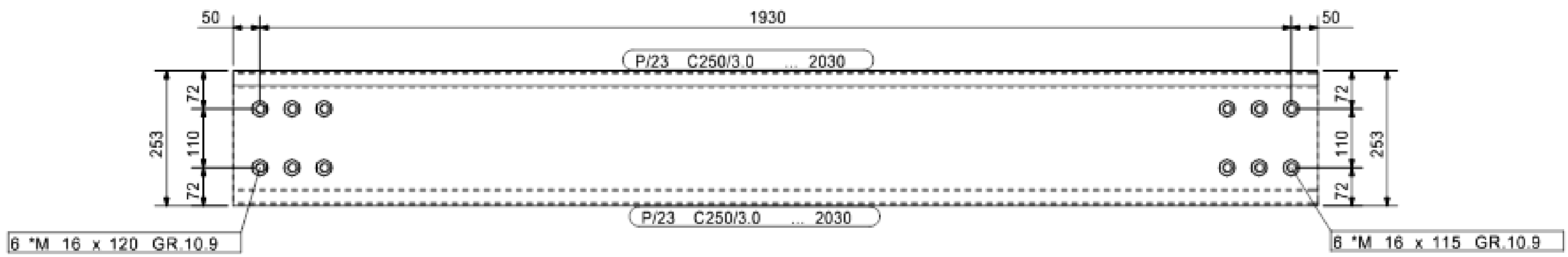


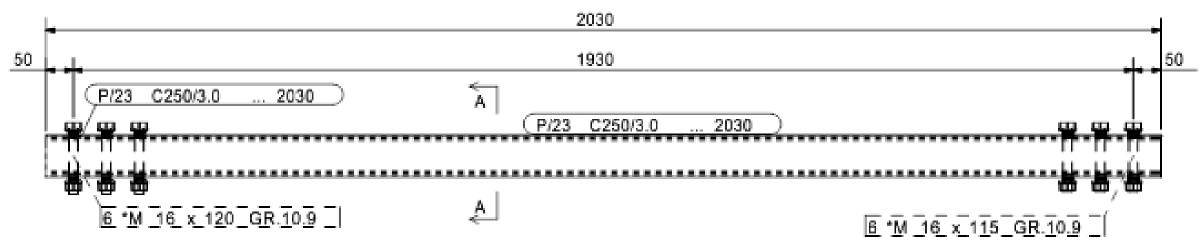
Conceptual design

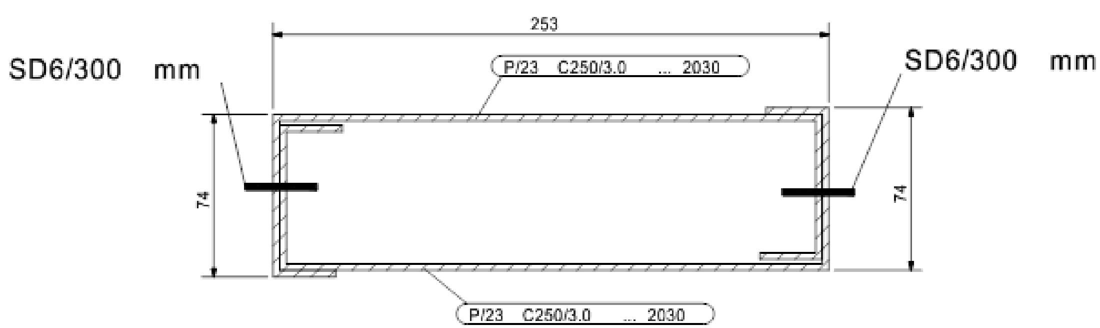
Case Studies

- **Hall Type Modular System (HTMS) - POPET Hall**

A-B/4
A-B/5
A-B/6
A-B/7
A-B/8
B-C/1
B-C/2
B-C/3
B-C/4
B-C/5
B-C/6
B-C/7
B-C/8







Conceptual design

Case Studies

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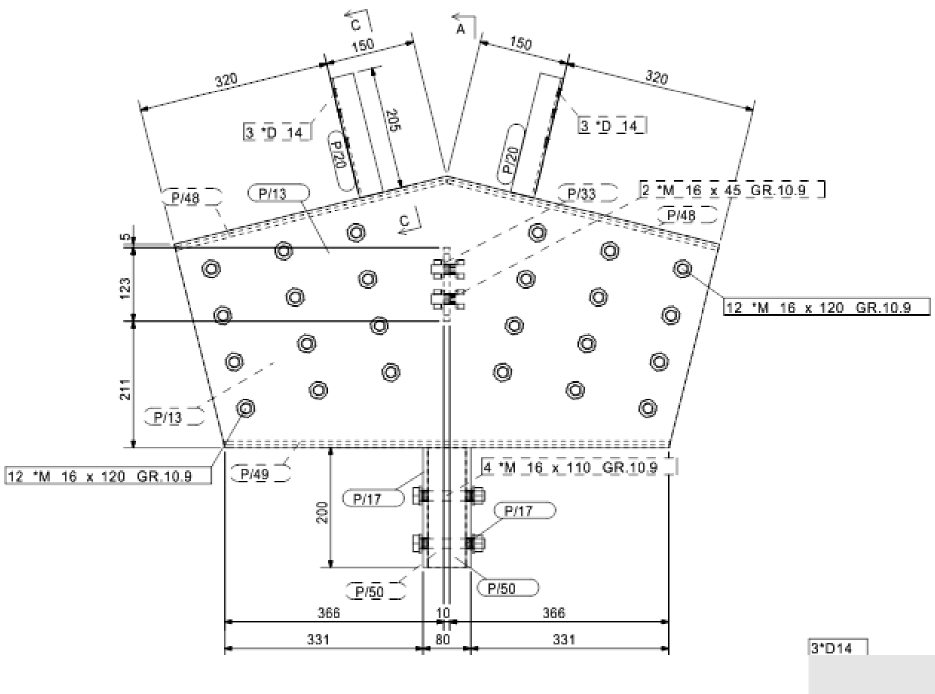
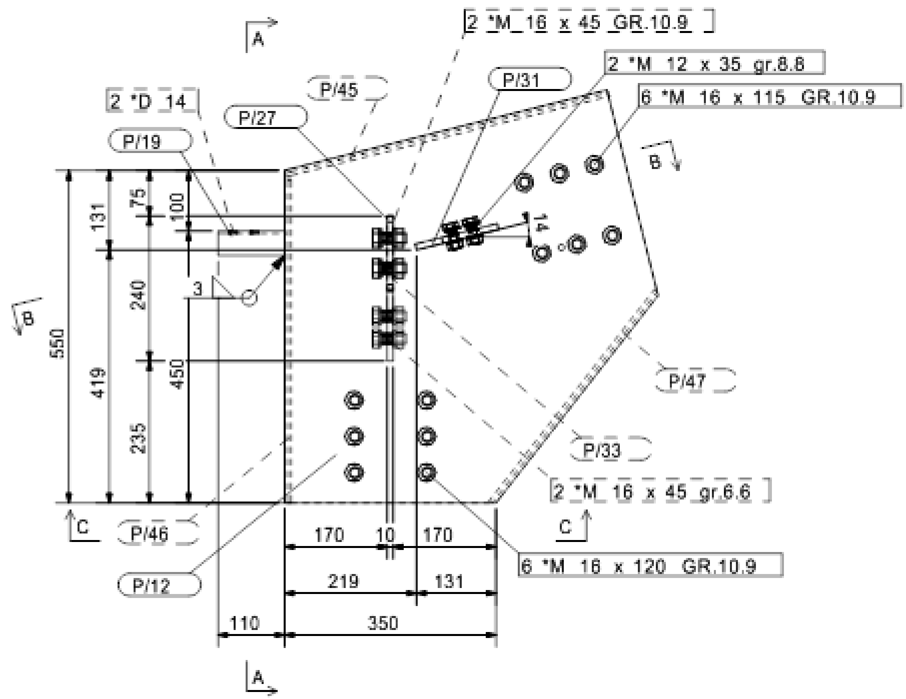
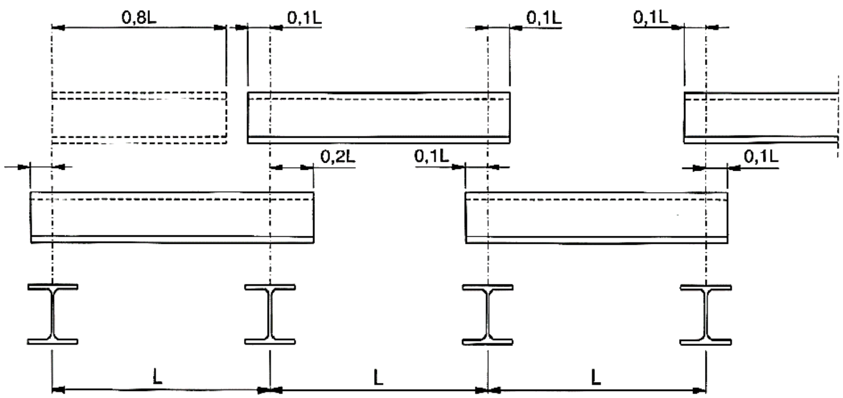




Conceptual design

Case studies : Z roof purlins design

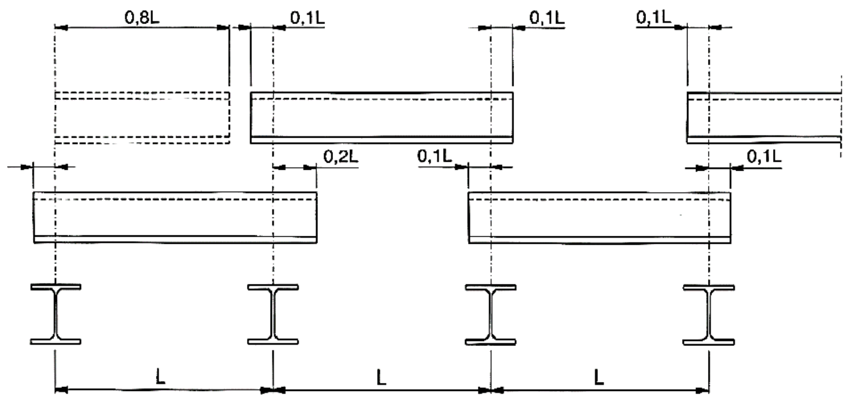
- **Overlap distances**



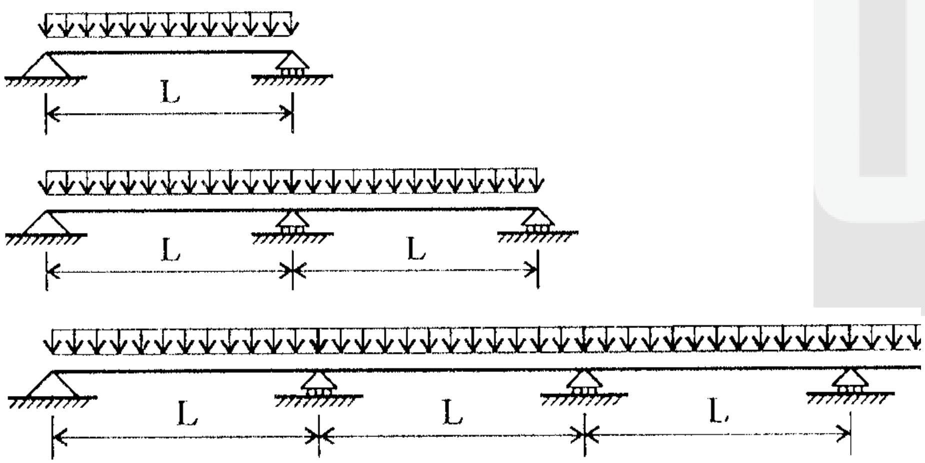
Conceptual design

Case studies : Z roof purlins design

- **Overlap distances**



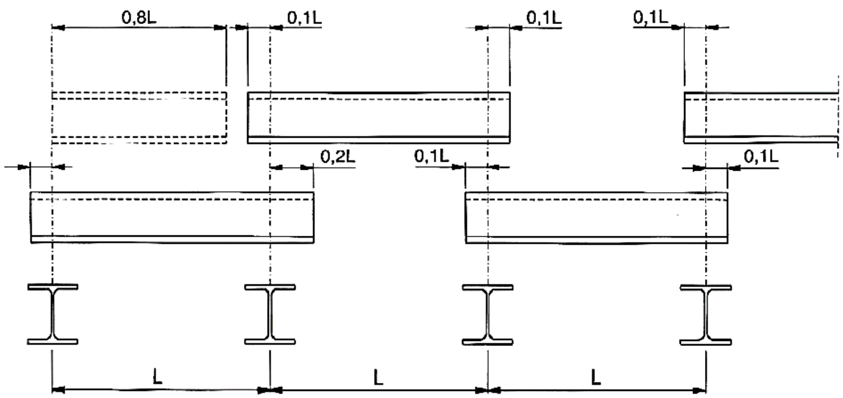
- **Static system**



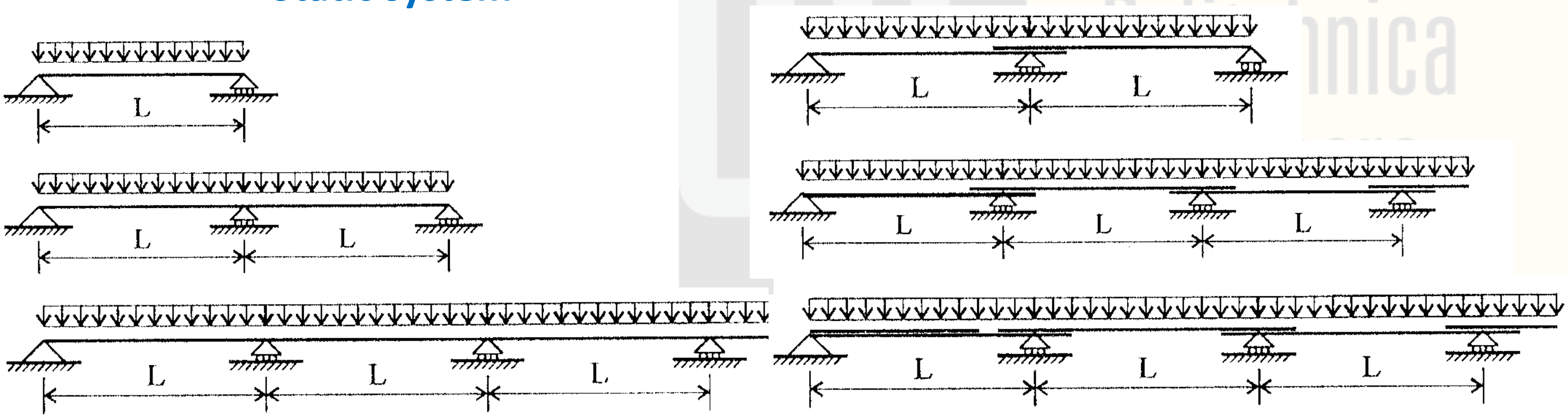
Conceptual design

Case studies : Z roof purlins design

- Overlap distances



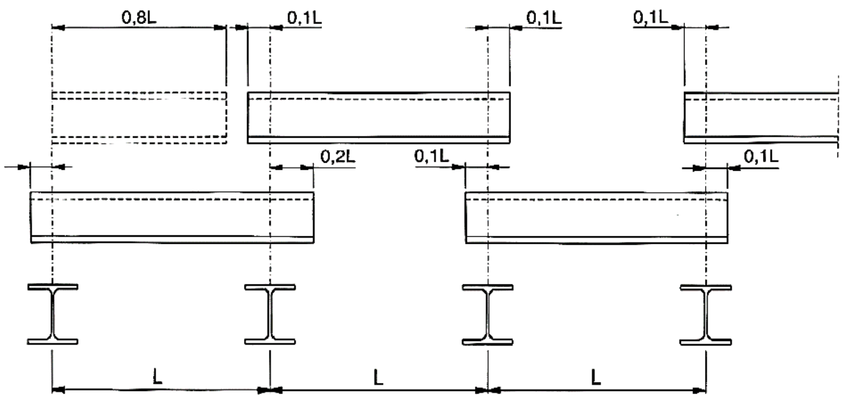
- Static system



Conceptual design

Z roof purlins design

- Overlap distances



- Designed for… (load and deformation)
  1. Corrugated sheet inside - outside (pressure and suction
  2. Corrugated sheet outside (pressure, gravitational)
  3. Corrugated sheet outside (suction)
  4. Allowed deflection L/200
  5. Allowed deflection L/300

Conceptual design

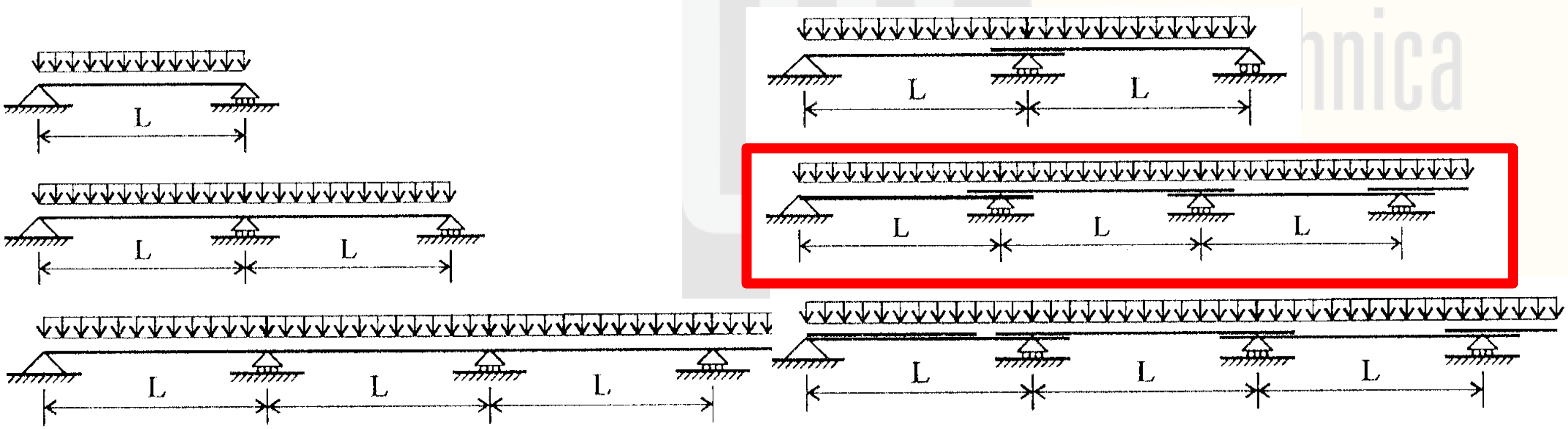
Z roof purlins design

- Initial data
  - Load on purlin 5 kN/m gravitational load
    2 kN/m suction
  - Span 5 m

Conceptual design

Z roof purlins design

- Initial data
  - Load on purlin — 5 kN/m gravitational load; 2 kN/m suction
  - Span — 5 m
  - Structural system — 5



Conceptual design

Z roof purlins design

- Initial data
  - Load on purlin — 5 kN/m gravitational load; 2 kN/m suction
  - Span — 5 m
  - Structural system — 5
  - Analysis type — 2, 3, 5

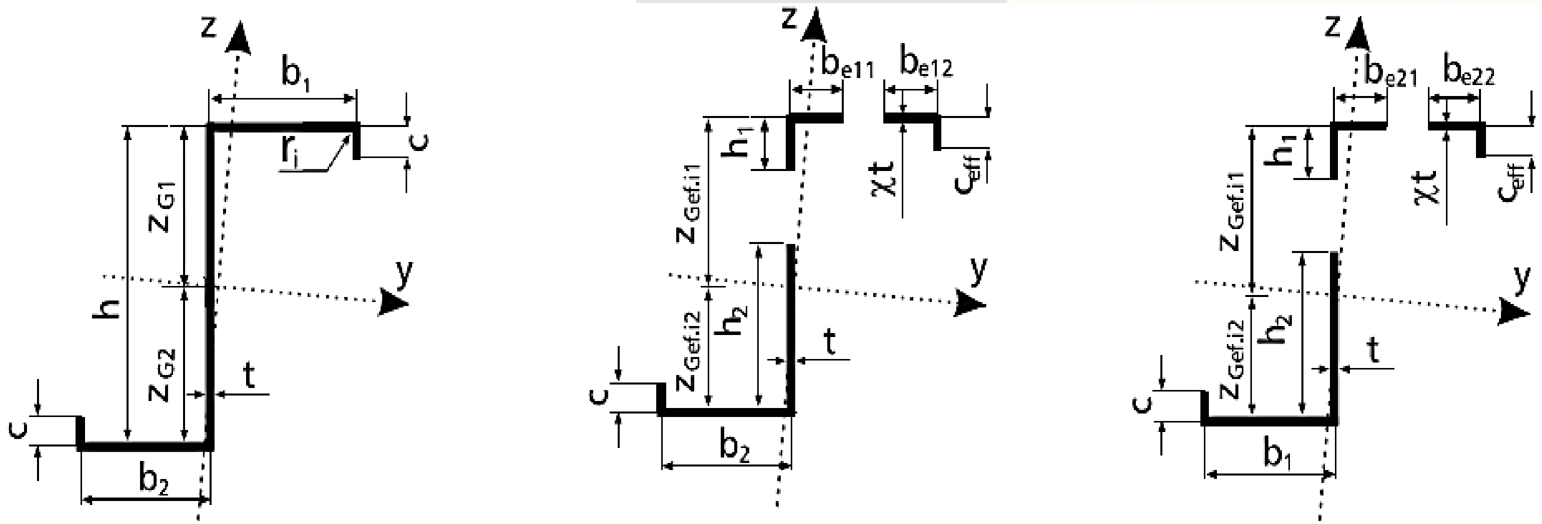
- Designed for… (load and deformation)
  - Corrugated sheet inside - outside (pressure and suction)
  - Corrugated sheet outside (pressure, gravitational)
  - Corrugated sheet outside (suction)

  - Allowed deflection L/200
  - Allowed deflection L/300

Conceptual design

Z roof purlins design

- Initial data
  - Load on purlin — 5 kN/m gravitational load
    2 kN/m suction
  - Span — 5 m
  - Structural system — 5
  - Analysis type — 2, 3, 5



	4	2.10	1.32	0.88	0.62	0.45	0.34	0.26	0.21
	5	1.40	0.88	0.59	0.41	0.30	0.23	0.17	0.14
Z100/2+2		**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**
	1	6.11	4.49	3.44	2.71	2.20	1.82	1.53	1.30
	2	4.49	3.57	2.91	2.41	2.01	1.69	1.44	1.24
	3	3.46	2.59	2.02	1.62	1.33	1.10	0.92	0.78
	4	4.21	2.65	1.78	1.25	0.91	0.68	0.53	0.41
	5	2.81	1.77	1.18	0.83	0.61	0.46	0.35	0.28
Z200/2.5+1.5		**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**
	1	17.85	14.08	11.36	9.33	7.79	6.59	5.64	4.88
	2	12.15	9.08	7.18	5.90	4.99	4.30	3.76	3.31
	3	10.90	7.67	5.63	4.31	3.43	2.81	2.36	2.03
	4	33.85	21.32	14.28	10.03	7.31	5.49	4.23	3.33
	5	22.57	14.21	9.52	6.69	4.87	3.66	2.82	2.22
Z200/2.5+2		**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**
	1	24.99	18.36	14.05	11.10	8.99	7.43	6.25	5.32
	2	13.80	10.28	8.10	6.64	5.61	4.84	4.23	3.73
	3	12.14	8.61	6.41	4.98	4.02	3.34	2.83	2.44
	4	34.89	21.97	14.72	10.34	7.54	5.66	4.36	3.43
	5	23.26	14.65	9.81	6.89	5.02	3.77	2.91	2.29
Z200/2.5+2.5		**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**
	1	25.25	18.55	14.20	11.22	9.09	7.51	6.31	5.38
	2	15.59	11.56	9.08	7.44	6.28	5.41	4.74	4.18
	3	13.61	9.66	7.21	5.63	4.55	3.78	3.20	2.75
	4	36.04	22.69	15.20	10.68	7.78	5.85	4.50	3.54
	5	24.02	15.13	10.14	7.12	5.19	3.90	3.00	2.36
Z250/1.5+1.5		**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**5.50**	**6.00**	**6.50**
	1	13.79	10.13	7.76	6.13	4.96	4.10	3.45	2.94
	2	10.48	7.27	5.25	3.98	3.15	2.58	2.17	1.86

European Commission

Eurocodes

Background and Applications

Design of **Steel Buildings**

with worked examples

16-17 October 2014
Brussels, Belgium

Organised and supported by

European Commission
DG Enterprise and Industry
Joint Research Centre
European Convention for Constructional Steelwork
European Committee for Standardization
CEN/TC250/SC3

Thank you for attention!

Professor Dan DUBINA
Politehnica University Timisoara
Romania

Joint Research Centre