Metal Structures

Lecture XII

Welded beams

Contents

<u>**Welded:**</u>

→ Lab #1 / 10

- Plane web
    - IKS
    - HKS

Photo: weldingweb.com

- Corrugated web

Photo: hxssvic.en.ec21.com

The most often situation:

- VIth class: welded I-beams, cold-formed sections;
- IIIrd class: welded I-beams, hot-rolled I-beams (rare), cold-formed sections;
- IInd clas: hot-rolled I-beams, CHS;
- Ist class: hot-rolled I-beams, CHS.

→ #4 / 87

In addition to the elastic and plastic sectional modulus, it is sometimes necessary to consider the effective sectional modulus. It appears in the situation of very slender cross-sections (thin and high web), for which the local stability of compressed part may be lost.

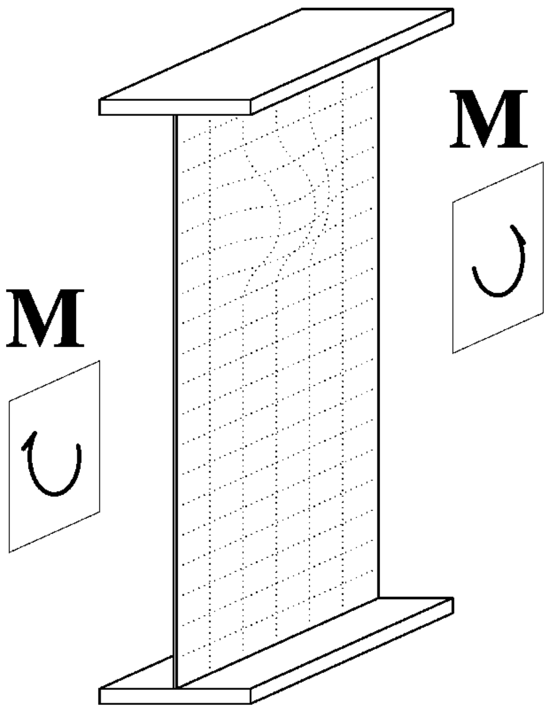


Photo: Autor

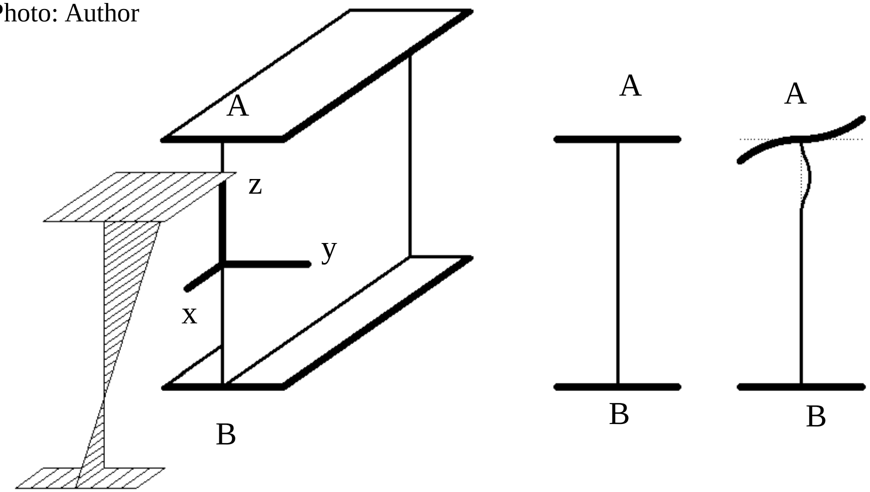
→ Lab #1 / 46

Photo: ijird.com

Instability of flange under compression, instability of web under compression → lab #2

Photo: Saliba, N. Gardner, L. Experimental study of the shear response of lean duplex stainless steel plate girders. Engineering Structures. 1 / 2013

Photo: Author



Reason: axial stresses σ_x (from bending moment and / or compressive axial force). Firts type interaction flange-web: lost of stability for both sub-patrs (web and flange) is independent; behavior of flange doesn’t affected on web and behavior of web doesn’t affected on flange.

Prevention: longitudinal stiffeners → #t / 21

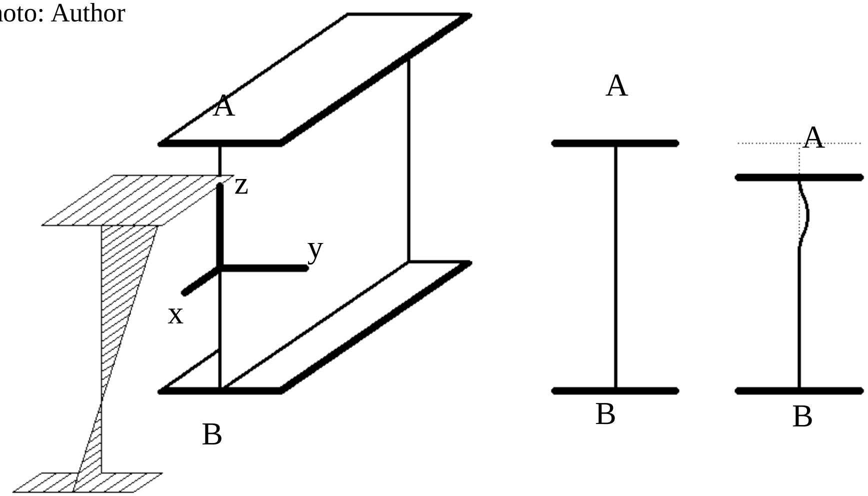
Calculation: effective geometry → lab #2, #t / 15-40

Flange induced buckling

(compression flange buckling in plane of web)

Photo: Web Buckling of High Strength Steel Plate Girders Induced by Bending Curvature, S. Nascimento, J. Pedro, A. Biscaya, Wiley Online Library, 9 III 2020

Photo: Author



Reason: axial stresses σ_x (from bending moment and / or compressive axial force). Second type interaction flage-web: lost of stability for both sub-patrs is dependent each other; behaviors of flange and web are common.

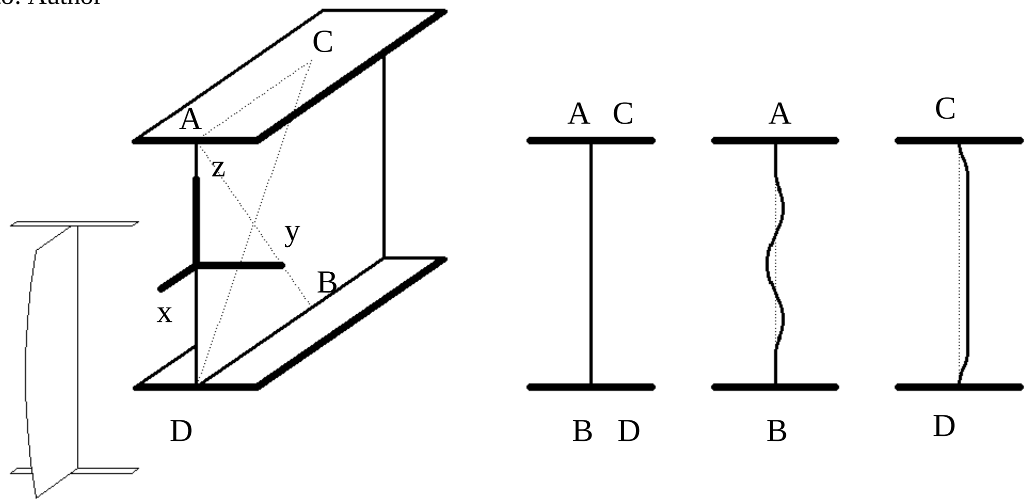
Prevention: good proportion flange to web→ #t / 41

Calculation: checking proportion → #t / 41

Instability of web under shear force

Photo: Saliba, N. Gardner, L. Experimental study of the shear response of lean duplex stainless steel plate girders. Engineering Structures. 1 / 2013

Photo: Author



Reason: shear stresses τ_{xz} (from shear force V_z). Along line A-B local compression, along line C-D local tension.

Prevention: transversal stiffeners → #t / 21

Calculation: checking stability → #t / 42 - 48

Instability of web under transversal force

Photo : Local Web Buckling in Tapered Composite Beams - A Parametric Study, R. Hobbs, P. Vellasco, Journal of the Brazilian Society of Mechanical Sciences 23-4/2001

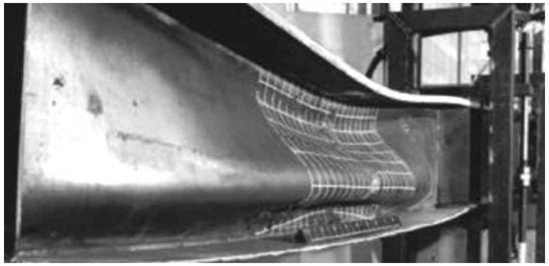
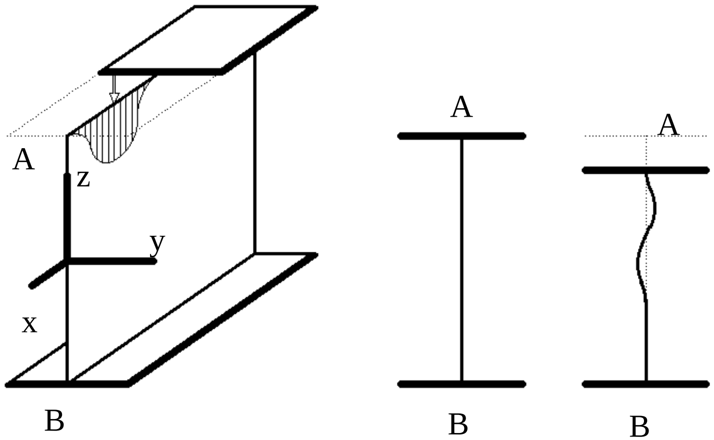


Photo: Author

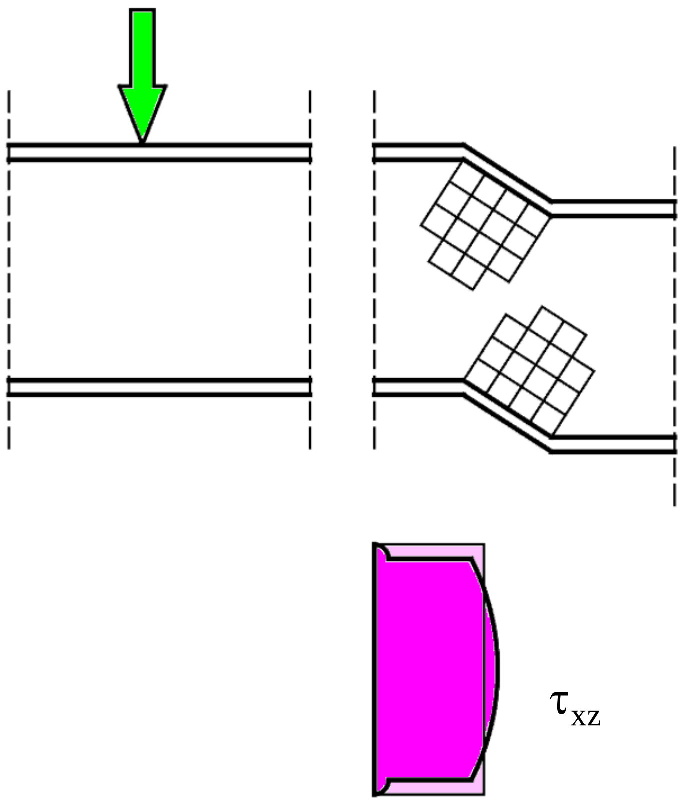


Reason: axial stresses σ_z (from transversal force F_s applicated in point). Axial stresses occur in web at contact with flange.

Prevention: transversal stiffeners → #t / 21

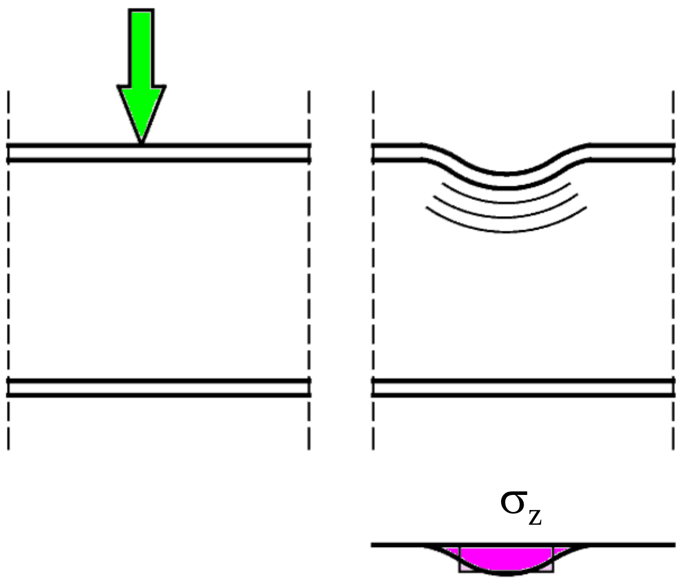
Calculation: checking stability → #t / 49 - 53

Transverse force F_s is force applied in point to beam. Of course, transverse force (type of loads) produces shear force V_{Ed} (type of cross-sectional force).



Shear force: global effect, deformation, instability, shear stresses and its idealisation

Photo: Author



Transverse force: local effect deformation, instability, axial stresses and its idealisation

→ Lab #2 / 68

Both phenomena (shear force, transversal force) are especially dangerous, when force are applied out of vertical stiffeners.

Force	Over vertical stiffener	Out of vertical stiffenet
shear	Lec # 21	#t / 42 – 48
transverse	can be neglected	#t / 49 - 53

Effect of shear force is analysed separately for Ist, IInd, IIIrd class of cross-secont (Lec. #11), and IVth class of cross-section (Lec. #t).

Effect of transversal force is impotrant for IVth class of cross-section; this means generally for welded I-beams.

Designing of welded I-beam:

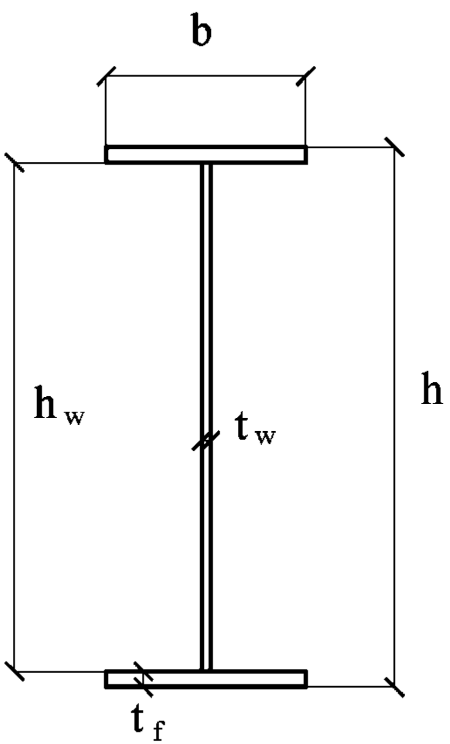


Photo: Author

$h \approx L / 20 \div L / 25$

t_w [mm] ≈ 7 [mm] + 3 h [m]

$b \approx 0{,}2\ h \div 0{,}3\ h$

$t_f \approx 1{,}5\ t_w \div 2{,}0\ t_w$

Example:

L = 25 m

$h \approx 1\,000 \div 1\,250$ [mm] $\rightarrow$ 1 300 [mm] = 1,3 [m]

$t_w \approx 7 + 3 \cdot 1{,}3 = 11$ [mm]

$b \approx 260 \div 390$ [mm] $\rightarrow$ 300 [mm]

$t_f \approx 17 \div 21$ [mm] $\rightarrow$ 20 [mm]

Welds between flange and web

Photo: northernmfg.com

Photo: weldingweb.com

Examples of calculation such type of weld will be presented on Lecture #17

III^{rd} class of cross-section

The most often situation: Welded I-beams are IV^{th} class of cross-section.

Welded beams can be sometimes made as III^{rd} class of cross-section. Information about this class is in Eurocode very short and not complete. Formula for axial force and bending moments is presented as:

$$N_{Ed} / (f_y\, A / \gamma_{M0}) + M_{y,\, Ed} / (f_y\, W_y / \gamma_{M0}) + M_{z,\, Ed} / (f_y\, W_z / \gamma_{M0}) \leq 1{,}0$$

EN 1993-1-1 (6.42)

This formula is true, when shear force doesn't exist.

There is no information, what formula is true in case of shear force not equal 0 or for situaton, when transversal force is applied.

h [mm]	t_w [mm]	b [mm]	t_f [mm]	F_{Ed} [kN]	$N_{Ed,\ comp}$ [kN]	L [m]
1 300	11	300	20	717,082	64,722	25,00

Example:

steel I-beam

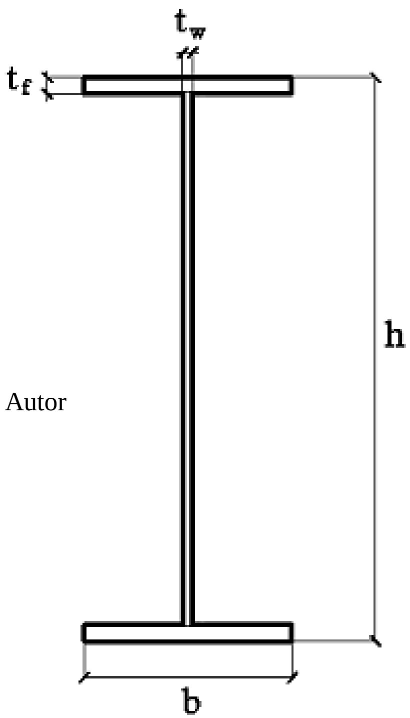


Photo: Autor

Steel S355 → f_y = 355 MPa

Length of beam:

Initial geometry:

A_0 = 258,600 cm^2

$J_{y,\ 0}$ = 674 887,800 cm^4

$W_{y,\ 0}$ = 10 382,889 cm^3

Point in border web-flange:

$W_{y,\ 0,\ wf}$ = 10 712,505 cm^3

Thickness of welds web-flange:

a = 5 mm

F_{Ed} = 717,082 kN

For this statis scheme:

$M_{Ed,\ y} = 6\ F_{Ed}\ L / 32$

$M_{Ed,\ y}$ = 3 361,320 kNm

→ Lab #2 / 22

I^{st} step: shear lag effect

Flange – too wide or not?

L_e = length of the beam = 25 m; → L_e / 50 = 500 mm

b = width of the flange = 300 mm → half of flange $b_0 = b / 2 = 150$ mm

$b_0 < L_e / 50$ 150 < 500 EN 1993-1-5 p.3.1

→ Lab #2 / 23

Flange is not too wide → shear lag in flanges is not danger→ after I^{st} step geometry not must be recalculated

$b_{eff,\,1} = b_{initial}$

$A_{eff\,1} = A_0 = 258{,}600\ cm^2$

$W_{y\ eff\ 1} = W_{y,\,0} = 10\ 382{,}889\ cm^3$

→ Lab #2 / 29

Generally: it is possible that we need to reduce width of flanges after first step due to shear lag effect. This will entail necessity to recalculate effective geometry after first step. We will obtain new values of $A_{eff\,,\,1}$ and $W_{y\ eff,\,1}$. This reduction **applies equally to both flanges**, so initially bi-symmetric cross-section will still remain bi-symmetrical.

II step
Flange under compression

Stress distribution in flange is identical, regardless of whether we take into account compressive axial force or the bending moment: constant value of compression in compressed flange. One common analysis will be made. **Stress distribution is calculated for effective geometry after I step: $A_{eff,1}$, $W_{y,eff,1}$**

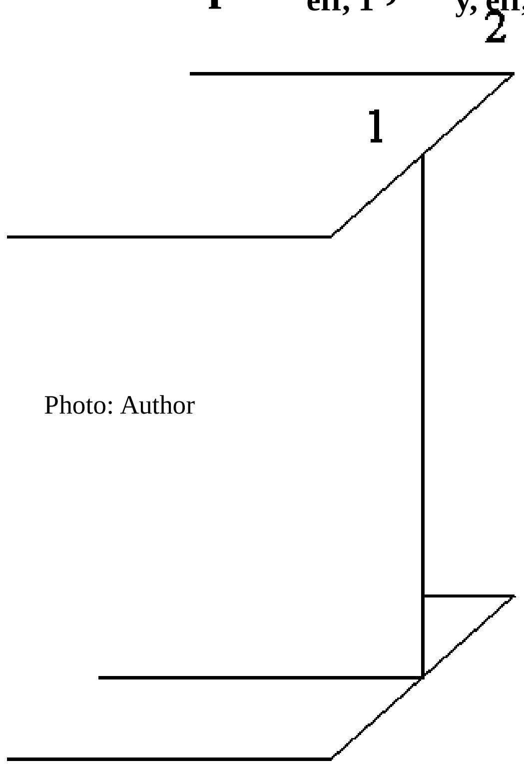
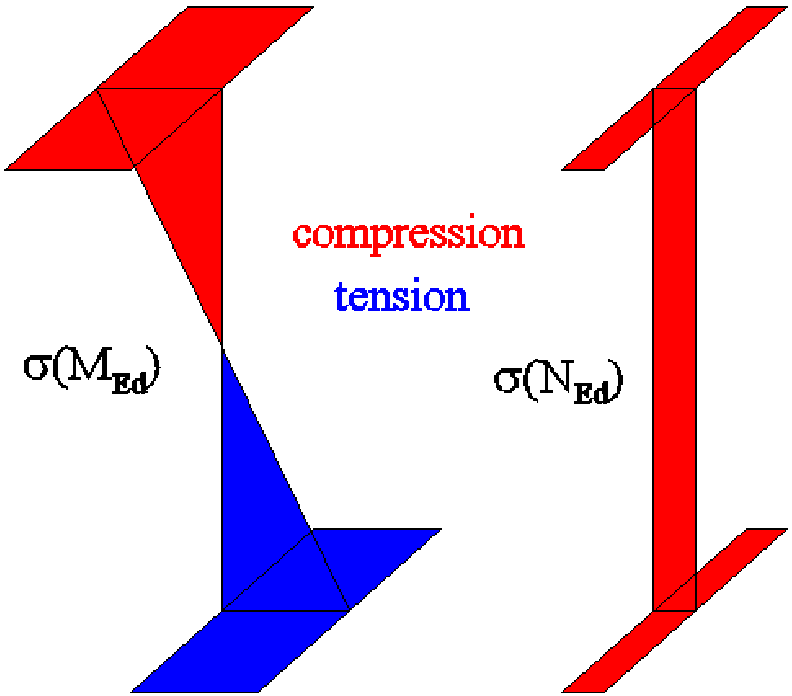
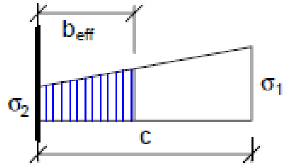
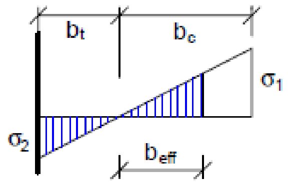


Photo: Author

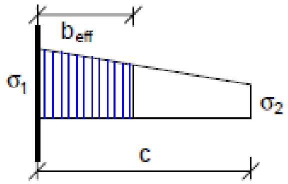
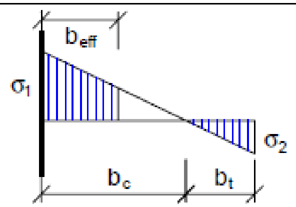


→ Lab #2 / 30

Stresses in flange – „blue part" remains after reduction an effective geometry

Stress distribution (compression positive)	Effective^p width b_{eff}	
	$1 > \psi \geq 0$: $b_{eff} = \rho\, c$	Compression of flange; nex to the web ≤ at the one of flange
	$\psi < 0$: $b_{eff} = \rho\, b_c = \rho\, c\,/\,(1-\psi)$	Compression and tension of flange; tension nex to the web

$\psi = \sigma_2/\sigma_1$	1	0	-1	$1 \geq \psi \geq -3$
Buckling factor k_σ	0,43	0,57	0,85	$0{,}57 - 0{,}21\psi + 0{,}07\psi^2$

Stress distribution (compression positive)	Effective^p width b_{eff}	
	$1 > \psi \geq 0$: $b_{eff} = \rho\, c$	Compression of flange; nex to the web ≥ at the one of flange
	$\psi < 0$: $b_{eff} = \rho\, b_c = \rho\, c\,/\,(1-\psi)$	Compression and tension of flange; tension at the end of flange

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$0{,}578\,/\,(\psi + 0{,}34)$	1,70	$1{,}7 - 5\psi + 17{,}1\psi^2$	23,8

→ Lab #2 / 31

Photo: EN 1993-1-5, tab 4.2

$\psi = \sigma_2 / \sigma_1$ = tension / compression or smaller compression / compression

Flange → σ = const → $\sigma_1 = \sigma_2$ → $\psi = \sigma_2 / \sigma_1 = 1{,}0$

$\psi = 1{,}0$ → table → $k_\sigma = 0{,}43$

Steel S355 → $f_y = 355$ MPa

$\varepsilon = \sqrt{(235 / f_y)}$ → $\varepsilon = \sqrt{(235 / 355)} = 0{,}814$

→ Lab #2 / 32

ρ – reduction factor for compression elements

– internal compression elements:

Internal ≡ web

$\rho = 1{,}0$ for $\bar{\lambda}_p \leq 0{,}673$

$$\rho = \frac{\bar{\lambda}_p - 0{,}055\,(3+\psi)}{\bar{\lambda}_p^2} \leq 1{,}0 \quad \text{for } \bar{\lambda}_p > 0{,}673\text{, where } (3+\psi) \geq 0 \qquad (4.2)$$

– outstand compression elements:

Outstand ≡ flange

$\rho = 1{,}0$ for $\bar{\lambda}_p \leq 0{,}748$

$$\rho = \frac{\bar{\lambda}_p - 0{,}188}{\bar{\lambda}_p^2} \leq 1{,}0 \quad \text{for } \bar{\lambda}_p > 0{,}748 \qquad (4.3)$$

where $\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28{,}4\,\varepsilon\,\sqrt{k_\sigma}}$

ψ is the stress ratio determined in accordance with 4.4(3) and 4.4(4)

$\bar{b}$ is the appropriate width to be taken as follows (for definitions, see Table 5.2 of EN 1993-1-1)

- b_w for webs;
- b for internal flange elements (except RHS);
- $b - 3\,t$ for flanges of RHS;
- c for outstand flanges;
- h for equal-leg angles;
- h for unequal-leg angles;

EN 1993-1-5 p.4.4

b_{eff1} – width of flange; a – thickness of welds; t_w – thickness of web; t_f – thickness of flange;

$c = (b - 2\,a\,\sqrt{2} - t_w\,)\,/\,2 = (300 - 14 - 11)\,/\,2 = 137{,}5$ mm

$\lambda_p = (c\,/\,t_f)\,/\,(28{,}4\,\varepsilon\,\sqrt{k_\sigma}) = (137{,}5\,/\,20)\,/\,(28{,}4 \cdot 0{,}814 \cdot \sqrt{0{,}43}) = 0{,}454 \rightarrow$
$\rightarrow$ EN 1993-1-5 (4.3) $\rightarrow 0{,}454 < 0{,}748 \rightarrow \rho = 1{,}0$

$b_{eff,\,2} = \rho\,b_{eff,\,1} = b_{eff,\,1} = b_{initial}$

No reduction of the compressed flange → after the second step, the geometry is the same as at the beginning:

$A_{eff\,2} = A_0 = 258{,}600\ cm^2$

$W_{y\,eff\,2} = W_{y,\,0} = 10\,382{,}889\ cm^3$

→ Lab #2 / 34

Generally: it is possible that we need to reduce width of flange after second step due to lost of stability under axian compressive stresses. This will entail necessity to recalculate effective geometry after second step. This reduction applies for **flange under compression only** (sometimes top, sometimes bottom, sometimes both), so initially bi-symmetric cross-section will lose symmetry with the horizontal axis. We will obtain new values for unsymmetrical cross-section: $A_{eff\,,\,2}$, $W_{y,\,top,\,eff,\,2}$, $W_{y,\,bottom,\,eff,\,2}$

→ Lab #2 / 35

III step

Web under compression

Stress distribution in web various for effects from bending moment and axial force. Calculation will be made separately, for axial force and for bending moment. **Stress distribution is calculated for effective geometry after II step: $A_{eff,2}$, $W_{y,eff,2}$**

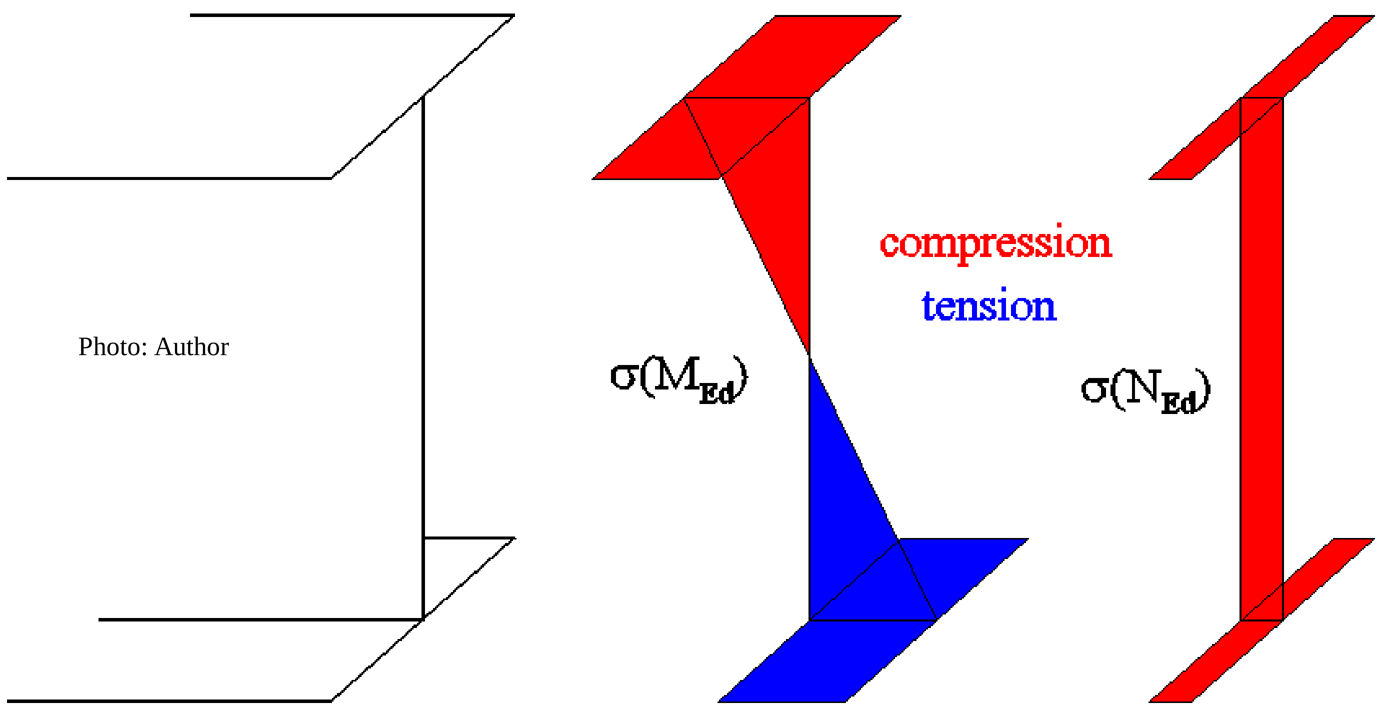


Photo: Author

→ Lab #2 / 36

Axial force:

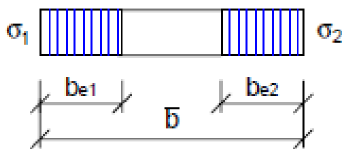
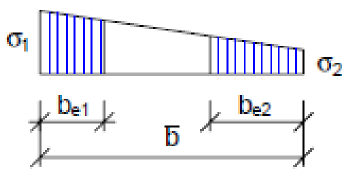
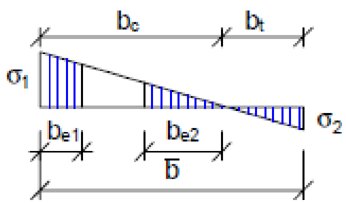
$\sigma_{max} = N_{Ed} / A_{eff,\ 2} = 64{,}722\ kN / 258{,}600\ cm^2 = 2{,}503\ MPa = const$

Bending moment, max value of stress in web (on border between web and flange):

$\sigma_{max} = \pm\ M_{y,\ Ed} / W_{y,\ eff,\ 2,\ wf} = \pm\ 3\ 361{,}320\ kNm / 10\ 712{,}505\ cm^3 =$
$= \pm\ 313{,}775\ MPa$

→ Lab #2 / 37

Stresses in flange – „blue part” remains after reduction an effective geometry

Stress distribution (compression positive)	Effective^p width b_{eff}
	$\underline{\psi = 1}$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = 0{,}5\ b_{eff}$ $\quad b_{e2} = 0{,}5\ b_{eff}$
	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = \frac{2}{5-\psi} b_{eff}$ $\quad b_{e2} = b_{eff} - b_{e1}$
	$\underline{\psi < 0}$: $b_{eff} = \rho\ b_c = \rho\ \bar{b} / (1-\psi)$ $b_{e1} = 0{,}4\ b_{eff}$ $\quad b_{e2} = 0{,}6\ b_{eff}$

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

Axial compression

Eccentric compression – whole web under compressed

Eccentric compression – part of web compressed, part tensed

→ Lab #2 / 38

EN 1993-1-5, tab 4.1

$\psi = \sigma_2 / \sigma_1$ = tension / compression or **smaller compression / compression**

Axial force → σ = const → $\sigma_1 = \sigma_2$ → $\psi = \sigma_2 / \sigma_1 = 1,0$

$\Psi = 1,0$ → table → $k_\sigma = 4,000$

Part of web under compression: total, $h_{wc, 3}$ = 1 260 mm

Bending moment → $\sigma_1 = -\sigma_2$ → $\psi = \sigma_2 / \sigma_1 = -1,0$

$\Psi = -1,0$ → table → $k_\sigma = 23,900$

Part of web under compression: half, $h_{wc, 3}$ = 630 mm

→ Lab #2 / 39

Steel S 355 → f_y = 355 MPa

$\varepsilon = \sqrt{(235 / f_y)}$ → $\varepsilon = \sqrt{(235 / 355)} = 0,814$

Slenderness for axial force:

b – total height of web; a – thickness of weld; t_w – thickness of web;

$c = b - 2 a \sqrt{2} = 1\,260 - 14 = 1\,246$ mm

$\lambda_p = (c / t_w) / (28,4\ \varepsilon \sqrt{k_\sigma}) = (1\,246 / 11) / (28,4 \cdot 0,814 \cdot \sqrt{4,000}) = 2,450$

→ Lab #2 / 40

Slenderness for bending moment:

b – total height of web; a – thickness of weld; t_w – thickness of web;

$c = b - 2 a \sqrt{2} = 1\,260 - 14 = 1\,246$ mm

$\lambda_p = (c / t_w) / (28,4\ \varepsilon \sqrt{k_\sigma}) = (1\,246 / 11) / (28,4 \cdot 0,814 \cdot \sqrt{23,900}) = 1,002$

ρ – reduction factor for compression elements

– internal compression elements:

Internal ≡ web

$\rho = 1{,}0$ for $\bar{\lambda}_p \le 0{,}673$ *

$$\rho = \frac{\bar{\lambda}_p - 0{,}055\,(3+\psi)}{\bar{\lambda}_p^2} \le 1{,}0 \quad \text{for } \bar{\lambda}_p > 0{,}673 \text{ , where } (3+\psi) \ge 0 \tag{4.2}$$ *

– outstand compression elements:

Outstand ≡ flange

$\rho = 1{,}0$ for $\bar{\lambda}_p \le 0{,}748$

$$\rho = \frac{\bar{\lambda}_p - 0{,}188}{\bar{\lambda}_p^2} \le 1{,}0 \quad \text{for } \bar{\lambda}_p > 0{,}748 \tag{4.3}$$

where $\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28{,}4\,\varepsilon\,\sqrt{k_\sigma}}$

Amendment!

ψ is the stress ratio determined in accordance with 4.4(3) and 4.4(4)

$\bar{b}$ is the appropriate width to be taken as follows (for definitions, see Table 5.2 of EN 1993-1-1)

- b_w for webs;
- b for internal flange elements (except RHS);
- $b - 3t$ for flanges of RHS;
- c for outstand flanges;
- h for equal-leg angles;
- h for unequal-leg angles;

→ Lab #2 / 41

EN 1993-1-5, 4.4

Amendments to EN 1993-1-5: limit between various formulas of r is not equal 0,673,

but

$0,5 + \sqrt{(0,085 – 0,055\ \Psi)]}$

So, in analysed cases:

For axial force (ψ = 1,0): $0,5 + \sqrt{(0,085 – 0,055\ \psi)]} = 0,673$

For bending moment (ψ = -1,0): $0,5 + \sqrt{(0,085 – 0,055\ \psi)]} = 0,874$

Slendernesses determined for axial force (2,450) and for bending moment (1,002) are greater than limit values for these loads (0,673 and 0,874, respectively), therefore web will be reduced in both cases.

→ Lab #2 / 42

For axial force:

$\rho = [\lambda_p - 0{,}055\ (3 + \psi)] / \lambda_p^2 = [2{,}450 - 0{,}055(3 + 1)] / 2{,}450^2 = 0{,}372$

Height of compressed part after reduction: $1\ 260 \cdot 0{,}372 = 468$ mm

For bending moment:

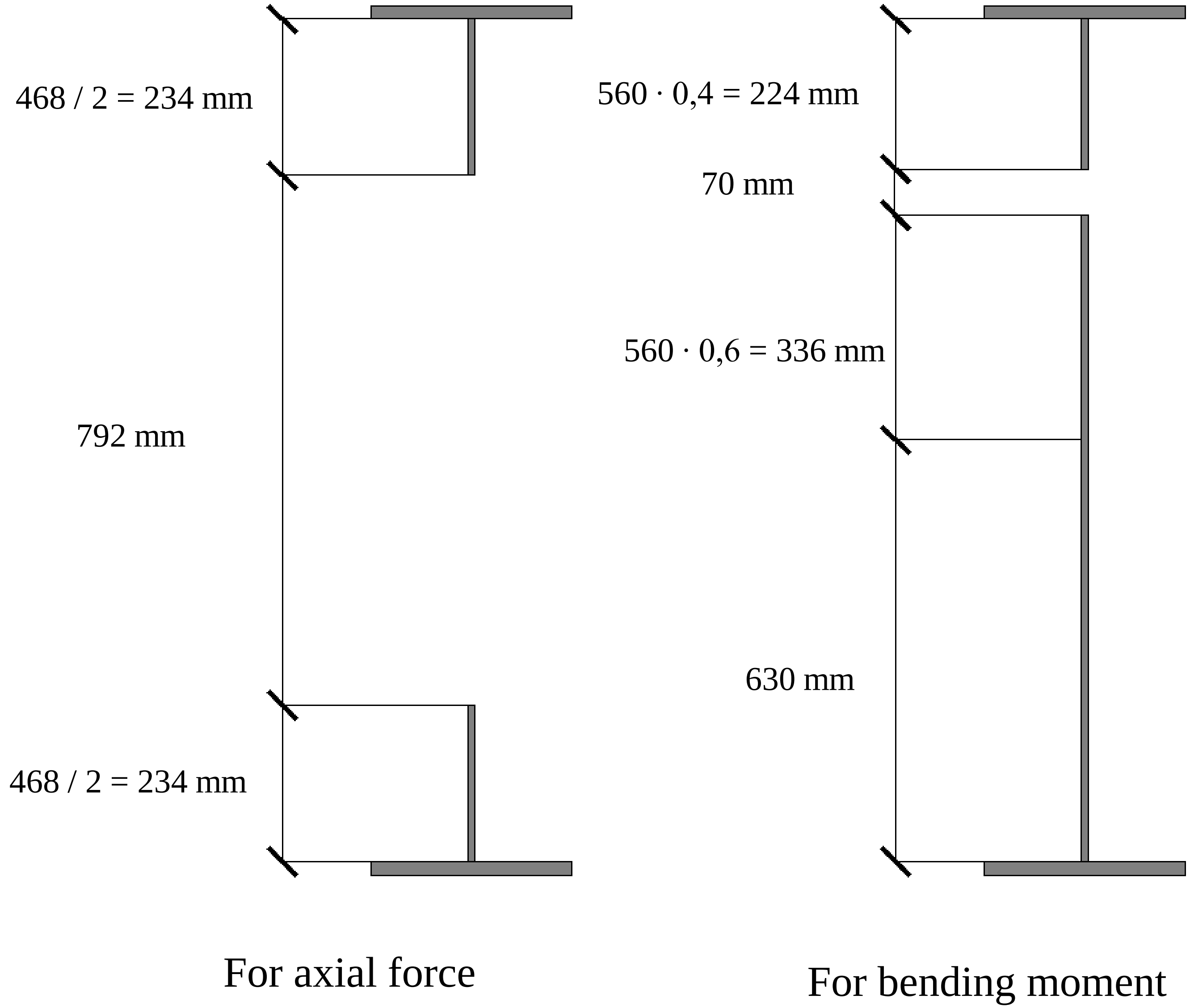
$\rho = [\lambda_p - 0{,}055\ (3 + \psi)] / \lambda_p^2 = [1{,}002 - 0{,}055(3 - 1)] / 1{,}002^2 = 0{,}888$

Height of compressed part after reduction: $630 \cdot 0{,}888 = 560$ mm

→ Lab #2 / 43

Effective geometry after III step:

Photo: Author



→ Lab #2 / 44

Recalculation of geometry

For axial force

→ Lab #2 / 45

$A_{eff, 3, N} = 171{,}480\ cm^2$

For bending moment

$A_{eff, 3, M} = 250{,}900\ cm^2$

S_y (about initial centre of gravity) = $-285{,}670\ cm^3$

$\Delta_y = S_y / A_{eff, 3, M} = -1{,}1\ cm$ (new center of gravity below the initial one)

$z_{top} = 64{,}1\ cm$

$z_{bottom} = 61{,}9\ cm$

$J_{eff, 3, M} = 623\ 222{,}349\ cm^4$

$W_{y, top, eff, 3, M} = J_{eff, 3, M} / z_{top} = 9\ 722{,}658\ cm^3$

$W_{y, bottom, eff, 3, M} = J_{eff, 3, M} / z_{bottom} = 10\ 068{,}212\ cm^3$

On border web-flange:

$W_{y, top, eff, 3, M, w\text{-}f} = 10\ 035{,}787\ cm^3$

$W_{y, bottom, eff, 3, M, w\text{-}f} = 10\ 404{,}380\ cm^3$

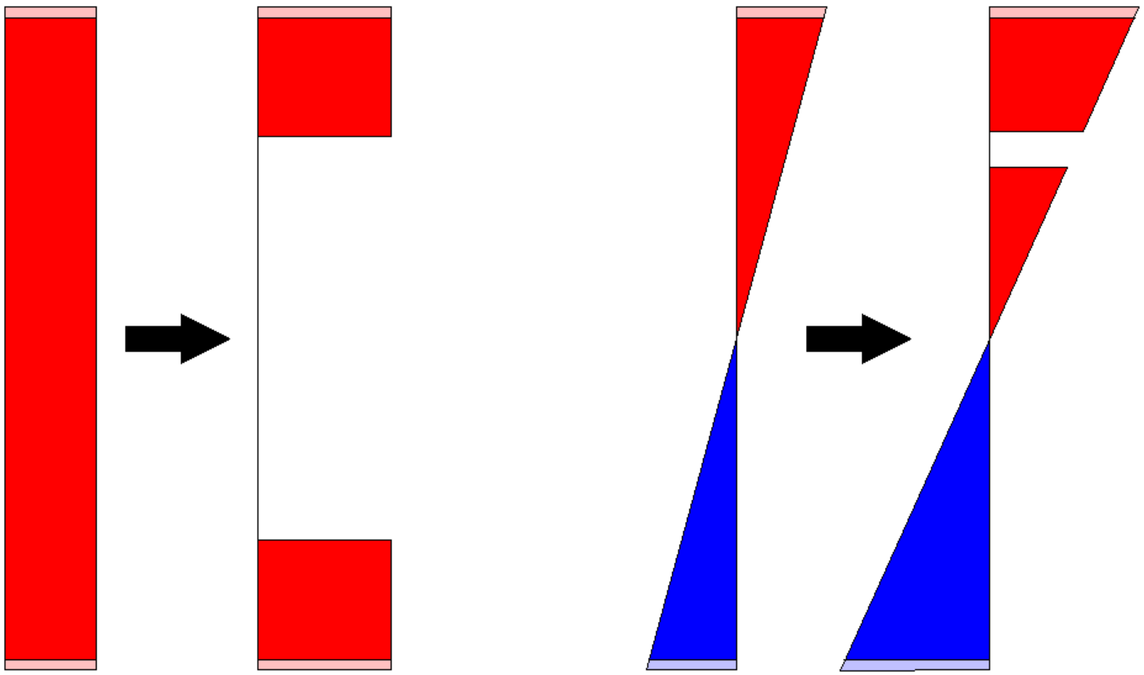
IV step

Web under compression - verification

Stress distribution is calculated for effective geometry after III step: $A_{eff,\,3}$, $W_{y,\,eff,\,3}$.

Stress distribution is totally changed.

→ Lab #2 / 46



Rys: Autor

Axial force

New value of stress in web:

$\sigma_{max} = N_{Ed} / A_{eff,\ 3,\ N} = 64{,}722\ kN / 171{,}480\ cm^2 = 3{,}774\ MPa$ (previous case 2,503 MPa).

So it is still constant distriburion. $\Psi = 1{,}0$ the same as previous. The sane, total height of web (1 260 mm) is under compression. Rest steps of calculation will be made for the same data as previous (Lab #2 / 36-46). Geometry after IV step will be completelu the same as after III step.

→ Lab #2 / 47

New value of stress in web :

$\sigma_{max, top} = M_{y, Ed} / W_{y, top, eff, 3, M, wf}$ = 3 361,320 kNm / 10 035,707 cm^3 =
= 334,936 MPa (compression)

$\sigma_{max, bottom} = -M_{y, Ed} / W_{y, bottom, eff, 3, M, wf}$ = -3 361,320 kNm / 10 404,380 cm^3 =
= -323,068 MPa (tension)

Additionally, due to noew centre of gravity, the height of the compressed part of the web changes to 64.1 cm (previously 63 cm).

$\psi = \sigma_2 / \sigma_1$ = tension / compression or **smaller compression / compression**

ψ = -323,068 / 334,936 = -0,965

Ψ = -0,965 → table → $k_\sigma = 7,81 - 6,29\,\psi + 9,78\,\psi^2 = 22,987$

Steel S 355 → f_y = 355 MPa

$\varepsilon = \sqrt{(235 / f_y)} \rightarrow \varepsilon = \sqrt{(235 / 355)} = 0,814$

→ Lab #2 / 48

Slenderness for bending moment:

b – total height of web; a – thickness of weld; t_w – thickness of web;

$c = b - 2\,a\,\sqrt{2} = 1\,260 - 14 = 1\,246$ mm

$\lambda_p = (c / t_w) / (28{,}4\,\varepsilon\,\sqrt{k_\sigma}) = (1\,246 / 11) / (28{,}4 \cdot 0{,}814 \cdot \sqrt{22{,}987}) = 1{,}022$

Limit for bending moment according to amendments ($\psi = -0{,}965$):

$0{,}5 + \sqrt{(0{,}085 - 0{,}055\,\psi)}] = 0{,}872$

Slenderness is bigger than limit, so there will be reduction of web.

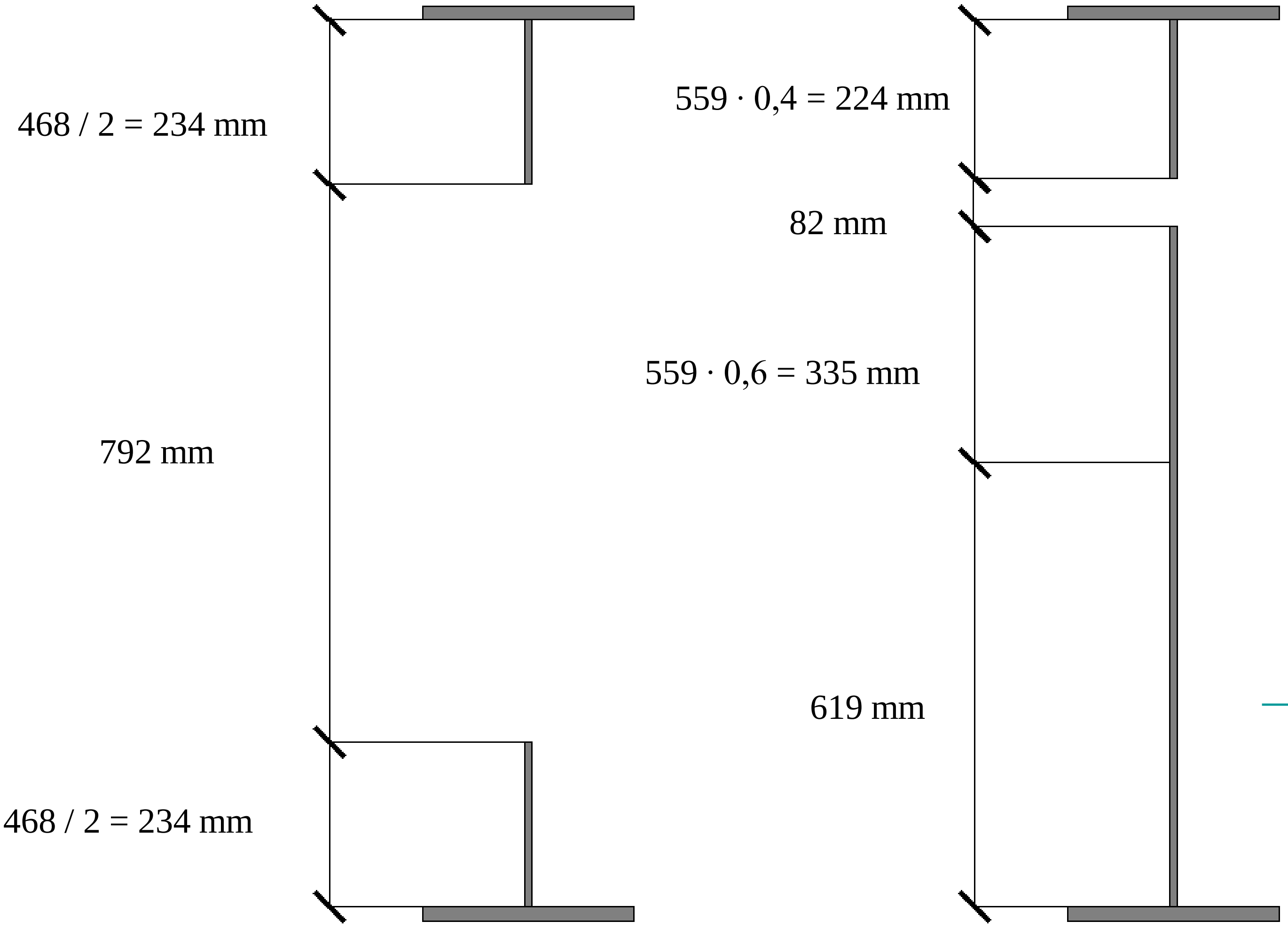
→ Lab #2 / 49

$\rho = [\lambda_p - 0{,}055\,(3 + \psi)] / \lambda_p^2 = [1{,}022 - 0{,}055(3 - 0{,}965)] / 1{,}022^2 = 0{,}871$

Height of compressed part after reduction: $641 \cdot 0{,}871 = 559$ mm

Effective geometry after IV step:

Rys: Autor



For axial force

For bending moment

→ Lab #2 / 50

Recalculation of geometry

For axial force (no changes):

→ Lab #2 / 51

$A_{eff,\,4,\,N} = 171{,}480\ cm^2$

For bending moment:

$A_{eff,\,4,\,M} = 249{,}580\ cm^2$

S_y (about initial centre of gravity) = $-329{,}230\ cm^3$

$\Delta_y = S_y / A_{eff,\,4,\,M} = -1{,}3\ cm$ (new center of gravity below the initial one)

$z_{top} = 64{,}3\ cm$

$z_{bottom} = 61{,}7\ cm$

$J_{eff,\,4,\,M} = 621\ 804{,}844\ cm^4$

$W_{y,\,top,\,eff,\,4,\,M} = J_{eff,\,4,\,M} / z_{top} = 9\ 670{,}371\ cm^3$

$W_{y,\,bottom,\,eff,\,4,\,M} = J_{eff,\,4,\,M} / z_{bottom} = 10\ 077{,}874\ cm^3$

Na granicy półki i środnika:

$W_{y,\,top,\,eff,\,4,\,M,\,wf} = 9\ 980{,}816\ cm^3$

$W_{y,\,bottom,\,eff,\,4,\,M,\,wf} = 10\ 415{,}492\ cm^3$

		A [cm^2]	$W_{y,\ top}$ [cm^2]	$W_{y,\ bottom}$ [cm^2]
0	Start	258,600	10 382,889	10 382,889
I	Shear lag effect	258,600	10 382,889	10 382,889
II	Flange under compression	258,600	10 382,889	10 382,889
III	Web under compression	171,480	9 722,658	10 068,212
IV	Web under compression - verification	171,480	9 670,371	10 077,874
...	...	...	...	...

Difference betwen two last calculation for web			
Difference betwen two last calculation for web	0,000 %	-0,538 %	0,096 %

Completion of iteration procedure is designer's decision. Usually it ends when it is converged < 2%.

$$N_{Rd} = A_{eff,\ 4,\ N}\ f_y / \gamma_{M0} = 6\ 087{,}540\ \mathrm{kN}$$

$$M_{Rd} = W_{y,\ top,\ eff,\ 4,\ M}\ f_y / \gamma_{M0} = 3\ 432{,}982\ \mathrm{kNm}$$

→ Lab #2 / 52

Flange induced buckling

$$h_w / t_w \leq k\,(E / f_{yf})\,[\sqrt{(A_w / A_{fc})}]$$

Class of cross-section	k
1	0,30
2	0,40
3, 4	0,55

EN 1993-1-5 (8.1)

There is no information, what we have to do, if this condition is not fulfills → geometry of I-beam must satisfy this condition.

Web stability and resistance under shear force V_{Ed}

in analogy to stability under axial force:

if buckling can't occurs

$$N_{c,Rd\,(1\text{-}3)} = A\, f_y / \gamma_{M0}$$

if buckling can occurs

$$N_{c,Rd\,(1\text{-}3)} = \chi\, A\, f_y / \gamma_{M0}$$

Difference: buckling factor $\chi \leq 1{,}0$

Both formulas can be presented as:

$$N_{c,Rd\,(1\text{-}3)} = \chi\, A\, f_y / \gamma_{M0}$$

(if buckling not exists, $\chi = 1{,}0$)

General formula for resistance:

EN 1993–1–5 (5.1), (5.2)

$$V_{b,Rb} = \min [\ \chi_w f_{yw} h_w t_w / (\gamma_{M1} \sqrt{3}) + V_{bf,Rd}\ ;\ \eta f_{yw} h_w t_w / (\gamma_{M1} \sqrt{3})]$$

Resistance of web

Resistance of web

Secondary impact of steel grade

Impact of lost of stability

Support from flange

Resistance of web is calculated the same for I[st], II[nd], III[th] and IV[th] class of cross-section

Resistance of web depends on steel grade and geometry of cross-section of web

Impact of lost of stability depends on numer and position of horizontal and vertical stiffeners, geometry of cross-section of web and grade of steel

Support from flange depends on loads (N_{Ed} , M_{Ed}) and resistance of flange

Secondary impact of steel grade depends steel grande

→ Lab #2 / 58

Shear forces - stability of web, local buckling; transverse stiffeners prevent from buckling

Requirement EN 1993-1-5 5.1(2):

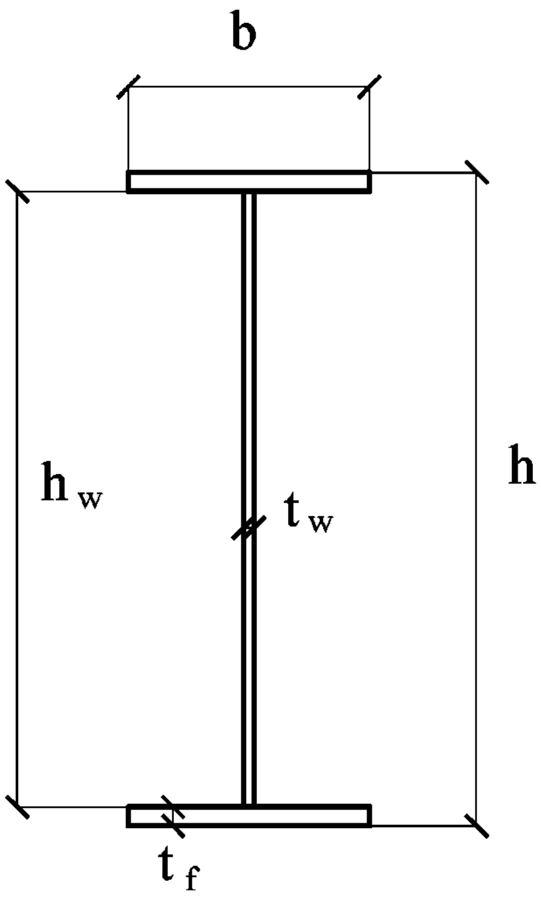
Unstiffened web	Stiffened web
$h_w / t_w \leq 72\ \varepsilon / \eta$	$h_w / t_w \leq 31\ \varepsilon\ \sqrt{(k_\tau)} / \eta$

$k_\tau \rightarrow$ #t / 46

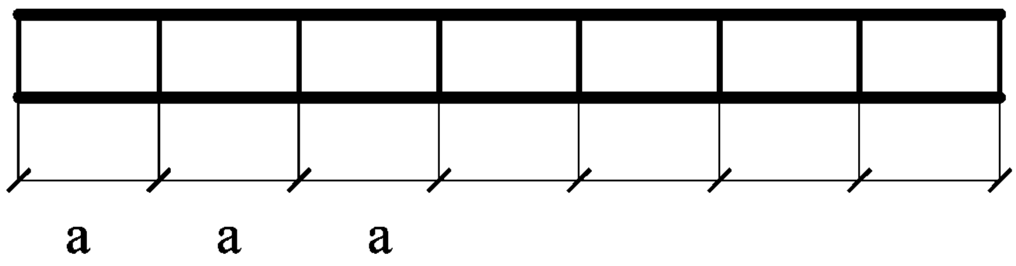
$f_y \leq 460$ MPa	$f_y > 460$ MPa
$\eta = 1,2$	$\eta = 1,0$

Requirement satisfied → buckling can’t occurs→ $\chi_w = 1,0$

Requirement not satisfied → buckling can occurs → $\chi_w \leq 1,0$ → #t / 48



Geometry:



→ Lab #2 / 57

Photo: Author

Number of longitudinal stiffeners	Calculation of k_τ $\alpha = a / h_w$	
0	Procedure A	
1 or 2	$\alpha < 3{,}0$ Procedure B	$\alpha \geq 3{,}0$ Procedure A
3 or more	Procedure A	

Procedure A

	$\alpha < 1{,}0$	$\alpha \geq 1{,}0$
k_τ	$k_{zts} + 4{,}00 + 5{,}34 / \alpha^2$	$k_{zts} + 5{,}35 + 4{,}00 / \alpha^2$

$$k_{zts} = \max \{ [2{,}1 \sqrt[3]{(J_{st} / h_w)}] / t_w \;\; ; \;\; [9\, h_w^2 \sqrt[4]{(J_{st} / (h_w \;\; t^3{}_w))}] / a^2 \}$$

J_{st} – for longitudinal stiffeners (0 if not exist)

EN 1993-1-5 A.3 (1)

Procedure B

$$k_\tau = 4{,}1 + [6{,}3 + 0{,}18\, J_{st} / (t^3\, h_w)\,] / a^2 + 2{,}2 \sqrt[3]{[J_{st} / (t^3\, h_w)\,]}$$

EN 1993-1-5 A.3 (2)

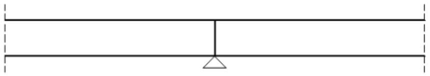
Influence of axial force

$$\rho = 1 - N_{Ed} / [(A_{f,\ top} + A_{f,\ bottom})\ f_{yf} / \gamma_{M0}]$$

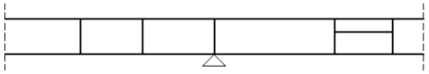
EN 1993-1-5 (5.9)

For χ_w important is slenderness:

Photo: Author



$$\bar{\lambda}_w = h_w / (86{,}4\ t_w\ \varepsilon)$$



$$\bar{\lambda}_w = h_{wi} / (37{,}4\ t_w\ \varepsilon\ \sqrt{k_\tau}\ )$$

$k_\tau \rightarrow$ #t / 46

EN 1993-1-5 (5.5), (5.6)

$\bar{\lambda}_w$	$\chi_w =$	
	Rigid end post	Non-rigid end post
$< 0{,}83 / \eta$	$\min(1 \; ; \; \eta)$	
$0{,}83 / \eta \div 1{,}08$	$0{,}83 / \bar{\lambda}_w$	
$\geq 1{,}08$	$1{,}37 / (0{,}7 + \bar{\lambda}_w)$	$0{,}83 / \bar{\lambda}_w$

EN 1993-1-5 tab. 5.1

$\eta \rightarrow$ #t / 44

Rigid and non-rigid end post → Lec #21

Transverse force

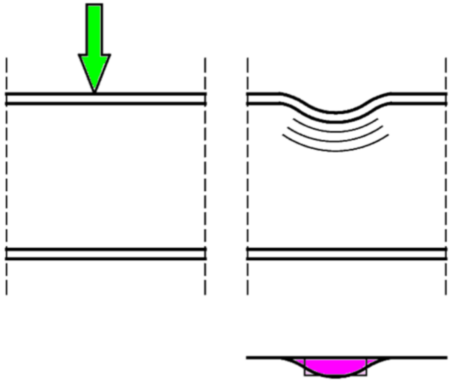


Photo: Author

$$F_s / F_{Rd} \leq 1{,}0$$

$$F_{Rd} = f_{yw} L_{eff} t_w / \gamma_{M1}$$ EN 1993-1-5 (6.1)

$$L_{eff} = \ell_y \overline{\chi}_F$$

$$\overline{\chi}_F = \min (1{,}0 \quad ; \quad 0{,}5 / \lambda_F)$$

$$\overline{\lambda}_F = \sqrt{(\ell_y t_w f_{yw} / F_{cr})}$$ EN 1993-1-5 (6.2) - (6.5)

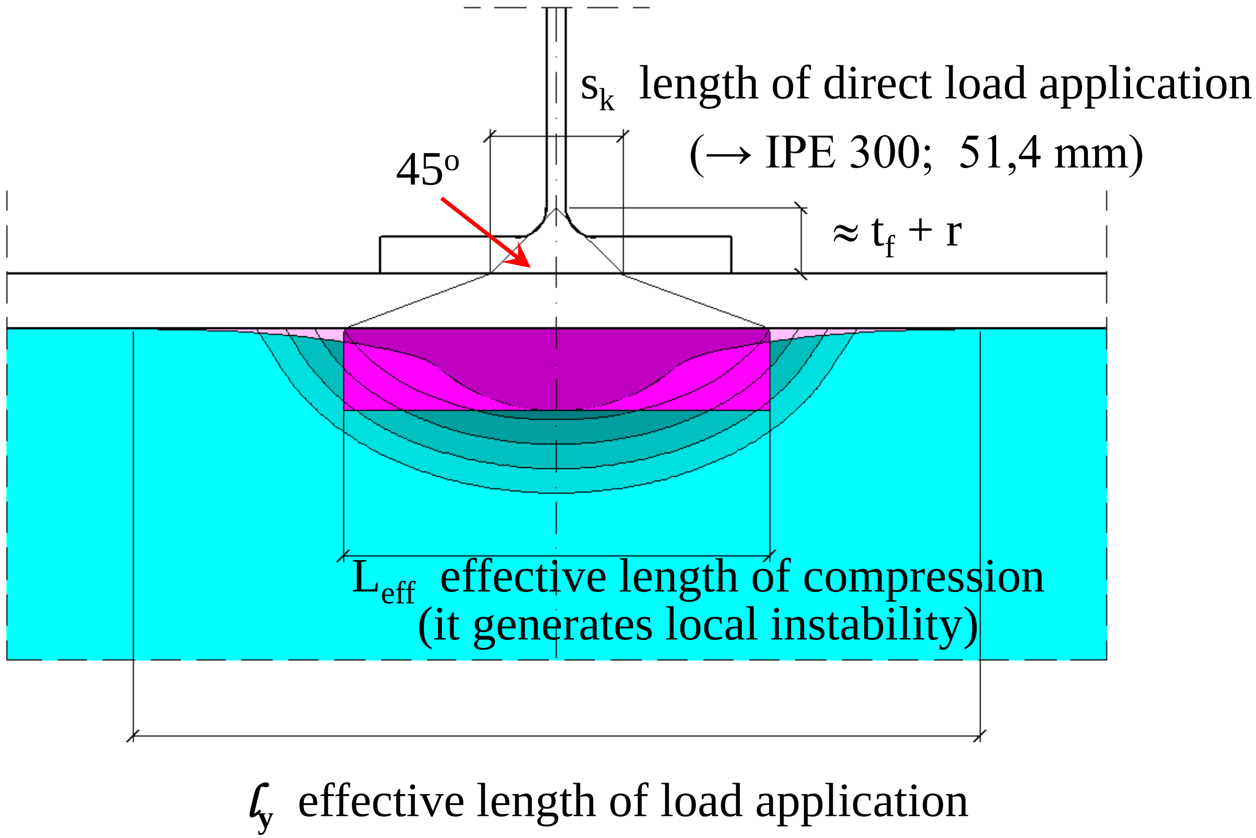
$$F_{cr} = 0{,}9 \, k_F E t_w^3 / h_w$$

$k_F \rightarrow$ #t / 51

$\ell_y \rightarrow$ #t / 50, 52

Important lengths:

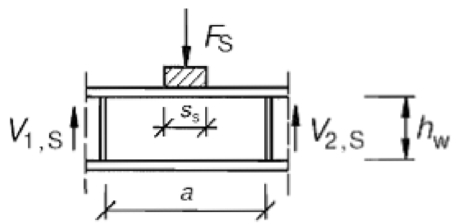
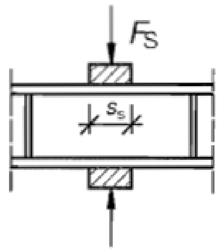
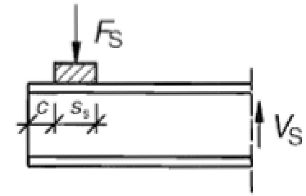
Photo: Author



→ Lab #2 / 69

EN 1993-1-5 fig. 6.1, no longitudinal stiffeners:

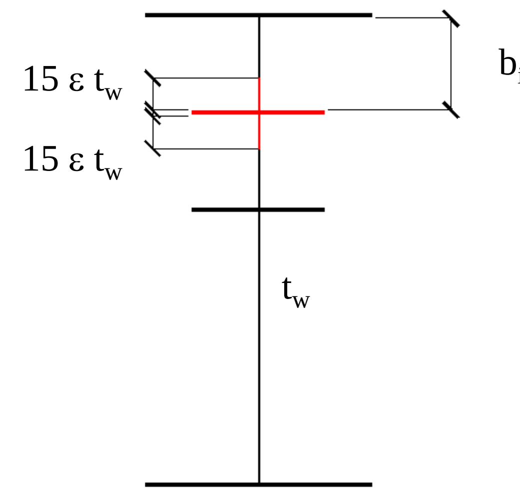
Photo: EN 1993-1-5 fig. 6.1

			
$k_F =$	$6 + 2\,(h_w / a)^2$	$3{,}5 + 2\,(h_w / a)^2$	$\min \{\ 2 + 6\,[\ (s_s + c) / h_w\ ]\ \ ;\ \ 6{,}0\}$

In case of longitudinal stiffeners:

$$k_F = 6 + 2\,(\ h_w / a\ )^2 + (5{,}44\ b_i / a\ - 0{,}21)\ \sqrt{\gamma_s}$$

$$\gamma_s = \min\,[\ 10{,}9\ J_{st,I} / (h_w^{\ 3}\ t_w)\ \ ;\ \ 13\ a / h_w + 210\,(0{,}3 - h_{wi} / a)]$$



EN 1993-1-5 (6.6), (6.7)

Photo: Author

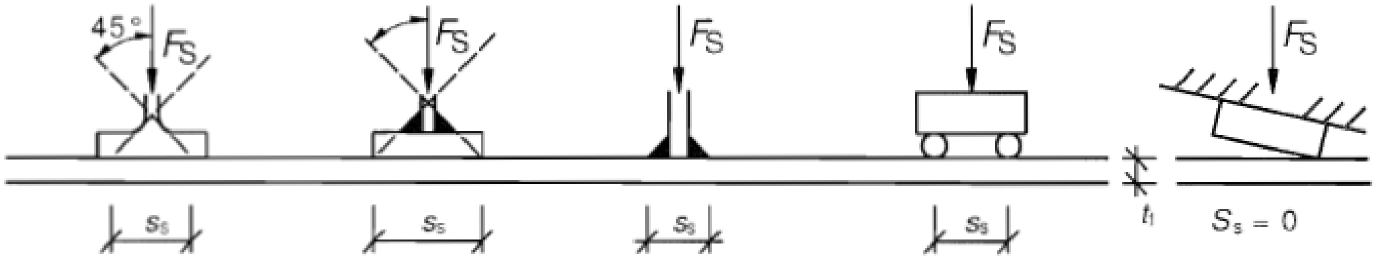
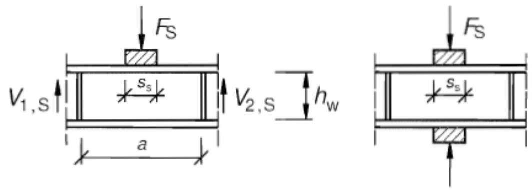
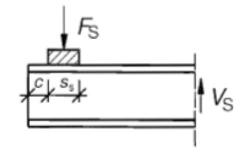


Photo: EN 1993-1-5 fig. 6.2

Photo: EN 1993-1-5 fig. 6.1

EN 1993-1-5 fig. 6.1		
ℓ_y	$\min\{a\;;$ $s_s + 2\,t_f\,[\,1 + \sqrt{(m_1 + m_2)}\,]\,\}$	$\min\{\ell_e + t_f \sqrt{[m_1 / 2 + (\ell_e / t_f)^2 + m_2]}\;;$ $\ell_e + t_f \sqrt{[m_1 + m_2]}\,\}$

$$\ell_e = \min[s_s + c\;;\;\; k_F\,E\,t_w^{\,2} / (2\,f_{yw}\,h_w)\,]$$

EN 1993-1-5 (6.10) - (6.13)

m_1 , m_2 → #t / 53

	$\bar{\lambda}_F \leq 0,5$	$\bar{\lambda}_F > 0,5$
m_1	$(f_{yf}\, b_f \,/\, f_{yw}\, t_w)$	
m_2	0	$0,02\,(h_w \,/\, t_f)^2$

EN 1993-1-5 (6.8), (6.9)

$\bar{\lambda}_F \rightarrow$ #t / 49

Iteration procedure must be applied:

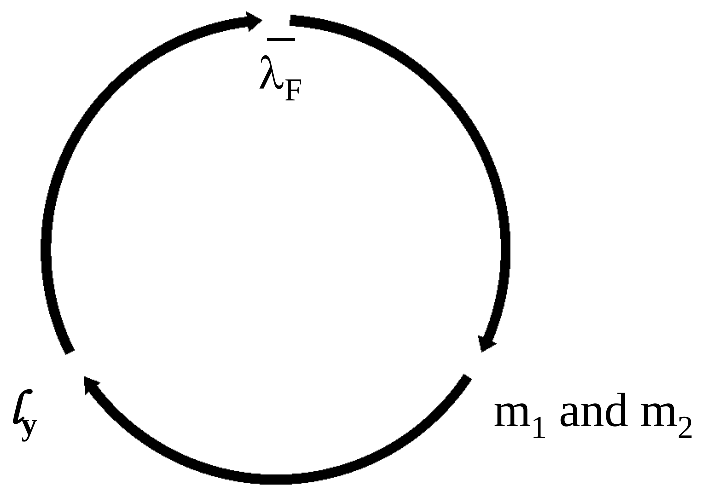


Photo: Author

Interactions between M_{Ed}, V_{Ed}, N_{Ed}, F_s

Example of interaction for hot-rolled I-beam:

between shear force and bending moment

$V_{Ed} / V_{c,Rd} \leq 0,5$

No reduction of bending moment resistance

$0,5 < V_{Ed} / V_{c,Rd} \leq 1,0$

Reduction of bending moment resistance

$$\rho = [\,2\ (V_{Ed} / V_{c,Rd}) - 1]^2$$

$$M_{y,\,V,\,Rd,} = \min\{M_{Rd}\ ;\ [W_{pl} - (\rho\, h_w^2\, t_w / 4)]\, f_y / \gamma_{M0}\}$$

EN 1993-1-1 (6.29), (6.30)

Symbols:

$$\eta_1 = N_{Ed} / (f_y\, A_{eff} / \gamma_{M0}) + (M_{y,\,Ed} + N_{Ed}\, e_{y,N}) / (f_y\, W_{y,\,eff} / \gamma_{M0}) +$$
$$+ (M_{z,\,Ed} + N_{Ed}\, e_{z,N}) / (f_y\, W_{z,\,eff} / \gamma_{M0}) \leq 1{,}0$$

EN 1993-1-5 (4.15)

$$\eta_2 = F_s / (f_{yw}\, L_{eff}\, t_w / \gamma_{M0}) \leq 1{,}0$$

EN 1993-1-5 (6.14)

$L_{eff} \rightarrow$ # t / 49

$$\bar{\eta}_1 = \max\,(M_{f,Rd} / M_{pl,Rd} \;\; ; \;\; M_{Ed} / M_{pl,Rd})$$

EN 1993-1-5 (7.1)

$$\eta_3 = V_{Ed} / V_{b,\,Rd} \leq 1{,}0$$

EN 1993-1-5 (5.10)

$V_{b,\,Rd} \rightarrow$ #t / 43

$$\bar{\eta}_3 = V_{Ed} / V_{bw,Rd}$$

EN 1993-1-5 (7.1)

$$V_{bw,Rd} = \chi_w\, f_{yw}\, h_w\, t_w / (\gamma_{M1} \sqrt{3})$$

Interactions:

If $\bar{\eta}_3 \leq 0,5$:

$\eta_1 \leq 1,0$ and $\eta_2 \leq 1,0$ and $\eta_3 \leq 1,0$ and $\eta_2 + 0,8\,\eta_1 \leq 1,4$

If $0,5 < \bar{\eta}_3 \leq 1,0$:

$$\bar{\eta}_1 + (1 - M_{f,Rd} / M_{pl,Rd})\,(2\,\bar{\eta}_3 - 1)^2 \leq 1,0$$

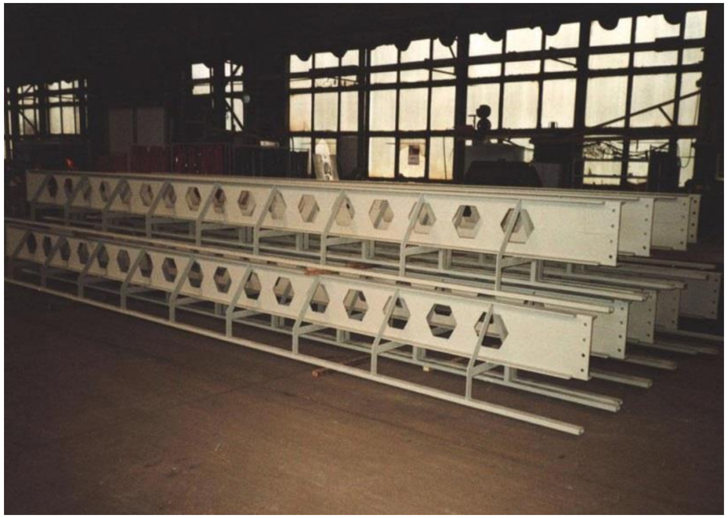
and

$\eta_1 \leq 1,0$ and $\eta_2 \leq 1,0$ and $\eta_3 \leq 1,0$ and $\eta_2 + 0,8\,\eta_1 \leq 1,4$

Castellated beams

Photo: zremb-wojkowice.pl

Photo: gunungsteel.com



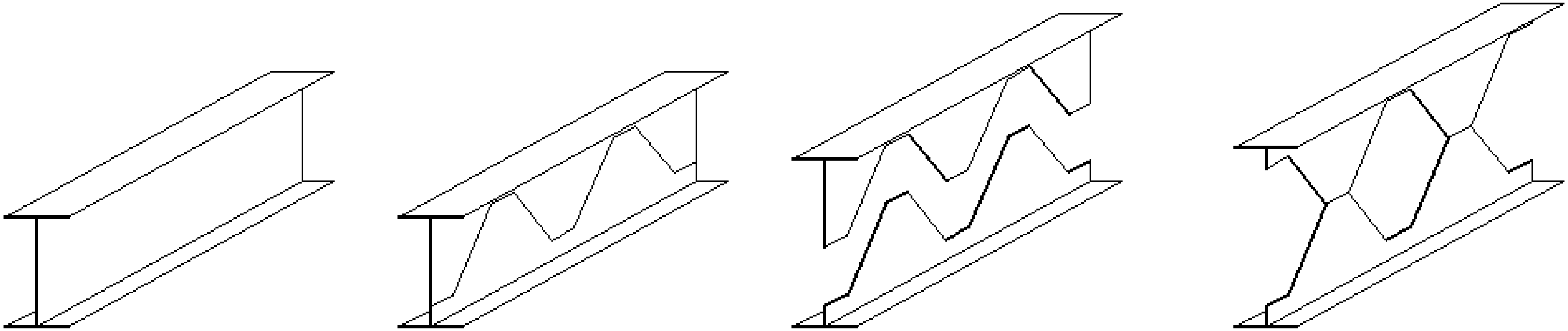


Photo: Author

The same dead weight; much greater moment of inertia and sectional modulus about strong axis; no change about weak axis.

→ #8 / 35

Dynamic loads are not recommended types of loads for this type of member. There should be applied "classical" girders with plane webs in places with big concentration of stresses (over support, joints betwen columns...)

Photo: abmrack.com

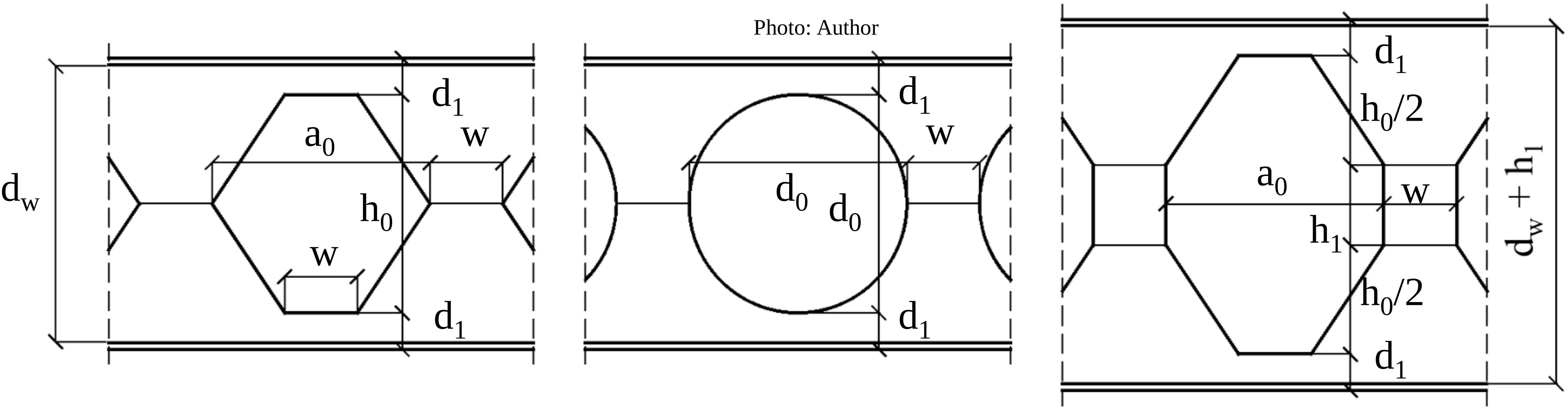
Photo: rfstearns.com

Photo: newsteelconstruction.com

At now way of calculation is not completely clear:

- castellated / cellular beams were presented in appendix to pre-Eurocod ENV 1993-1-1 in 1992 year;
- such part of information is not presented in official version of Eurocode EN 1993-1-1;
- work is currently underway to extend the Eurocode, including the part with working name "EN 1993-1-13", for beams with large holes in the webs; first version of this part was elaborated in 2018 year.

Recommended geometry



$d_1 \geq 0{,}10\ d_w$

$h_0 \leq 0{,}75\ d_w$

$a_0 \leq h_0$

$0{,}25\ a_0 \leq w \leq 0{,}50\ a_0$

$d_1 \geq 0{,}10\ d_w$

$d_0 \leq 0{,}75\ d_w$

$0{,}25\ d_0 \leq w \leq 0{,}50\ d_0$

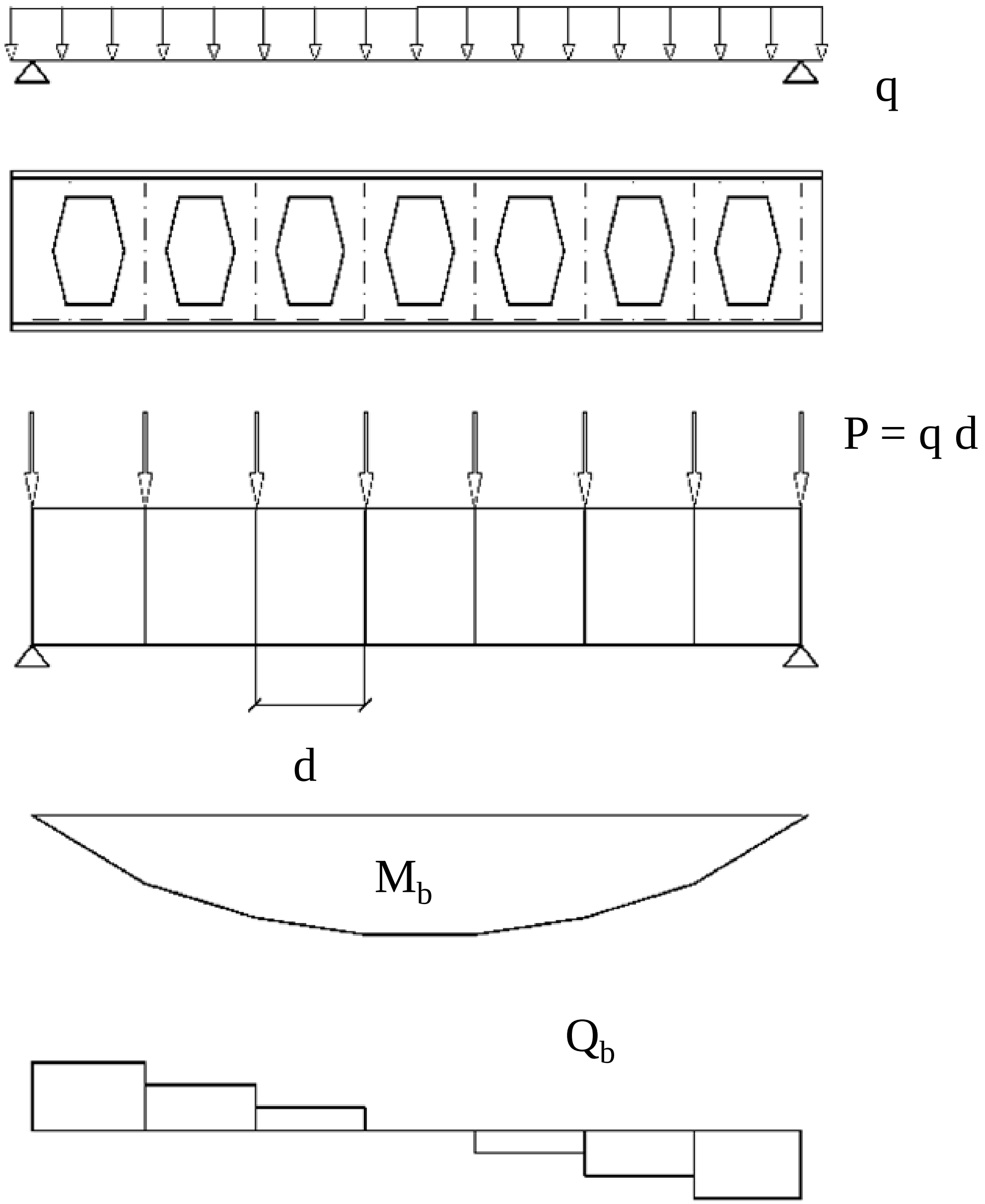
$d_1 \geq 0{,}10\ d_w$

$h_0 \leq 0{,}75\ d_w$

$a_0 \leq h_0$

$0{,}25\ a_0 \leq w \leq 0{,}50\ a_0$

$w \leq h_1 \leq 2{,}00\ w$



Classical method of castellated beams calculation

Photo: Author

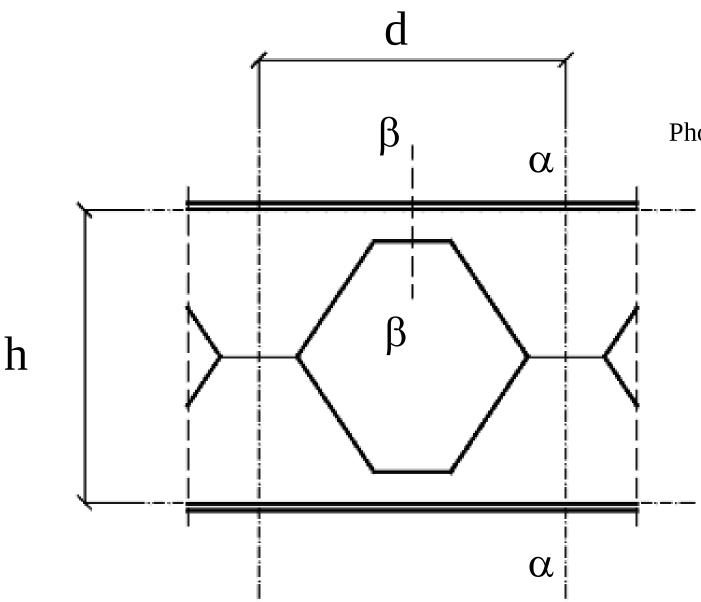


Photo: Author

P – external action (→ #t / 61)

$Q_{b\beta} \rightarrow Q_b$ (→ #t / 61) in cross-section β–β

$M_{b\alpha} \rightarrow M_b$ (→ #t / 61) in cross-section α–α

$M_1 \approx Q_r\, z / 2$; $z = a_0$ or d_0

$M_2 \approx T\,(h - y) / 2$; $y = h_0$ or d_0 or $h_0 + h_1$

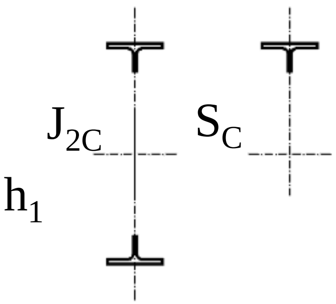


Photo: Author

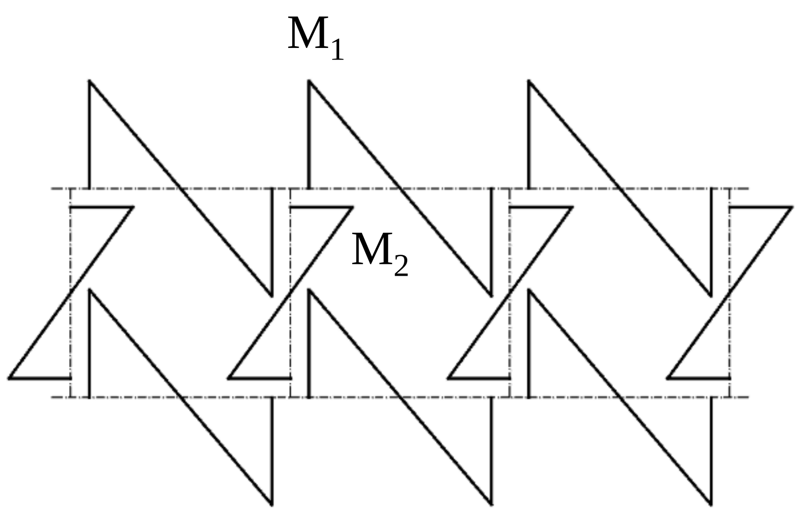
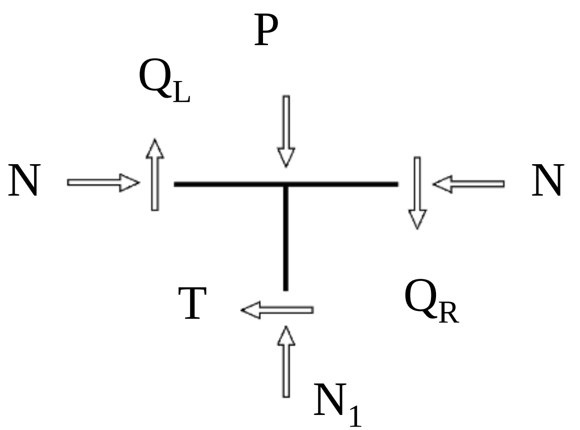


Photo: Author



$Q_L \approx (Q_{b\alpha} + P) / 2$

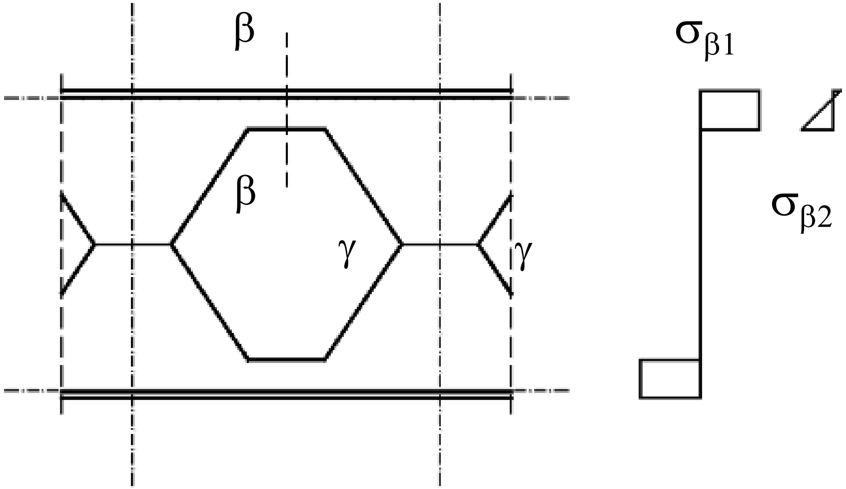
$Q_R \approx Q_{b\alpha} / 2$

$N \approx M_{b\alpha} / h$

$T \approx (2Q_{b\beta} + P)\, d / (2\, h)$

$N_1 \approx P / 2$

Photo: Author

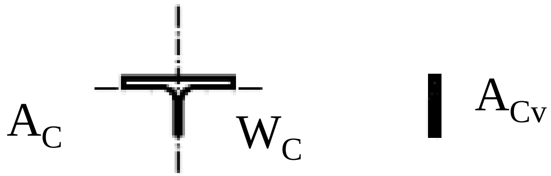


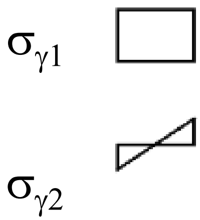
$$\sigma_{\beta 1} = N / A_C$$

$$\sigma_{\beta 2} = M_1 / W_C$$

$$\tau_{\beta} = Q_R / A_{Cv}$$

Photo: Author





$$\sigma_{\gamma 1} = N_1 / A_W$$

$$\sigma_{\gamma 2} = M_2 / W_W$$

$$\tau_{\gamma} = T / A_W$$

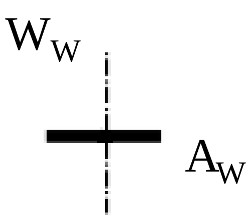


Photo: Author

Requirements for Limit States:

- Resistance in cross-section β–β;
- Local stability in cross-section β–β;
- Resistance in cross-section γ–γ;
- Local stability in cross-section γ–γ;
- Global stability;
- Deflection;

Cross-section β–β:

Resistance:

$\{ \sqrt{[(\sigma_{\beta 1} + \sigma_{\beta 2})^2 + 3 (\tau_\beta)^2]} \} / f_y \leq 1{,}0$

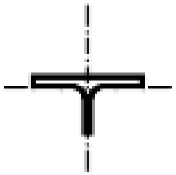
Stability:

$N_{eq} / (\chi N_{Rd}) \leq 1{,}0$

$N_{eq} = (\sigma_{\beta 1} + \sigma_{\beta 1}) A_C$

$N_{Rd} = A_C f_y$

Photo: Validation of an analytical model for curved and tapered cellular beams at normal and fire conditions, S. Durif, O. Vassart, Periodica Polytechnica Civil Engineering 57(1):83 · January 2013



χ for cross-section β–β

and critilal length = d (→ #t / 62)

Photo: Author

Cross-section γ–γ:

Resistance:

$$\{ \sqrt{ [(\sigma_{\gamma 1} + \sigma_{\gamma 1})^2 + 3 (\tau_{\gamma})^2] } \} / f_y \leq 1{,}0$$

Stability:

$$N_{eq} / (\chi \, N_{Rd}) \leq 1{,}0$$

$$N_{eq} = (\sigma_{\gamma 1} + \sigma_{\gamma 1}) \, A_W$$

$$N_{Rd} = A_W \, f_y$$

Photo: Validation of an analytical model for curved and tapered cellular beams at normal and fire conditions, S. Durif, O. Vassart, Periodica Polytechnica Civil Engineering 57(1):83 · January 2013

χ for cross-section γ–γ

and critilal length = h (→ #t / 62)

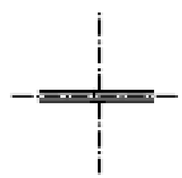


Photo: Author

Analisis of global stability (various types of global buckling) for castellated beams are complicated. Four main inportant geometrical characteristics must be calculated:

Characteristis	Important for
J_y ; moment of inertia about horizontal axis	• Flexural buckling about horizontal axis
J_z ; moment of inertia about vertical axis	• Flexural buckling about vertical axis • Flexural-torsional buckling • Lateral buckling
J_T ; torsional moment of inertia	• Flexural-torsional buckling • Lateral buckling
J_w ; warping constant	• Flexural-torsional buckling • Lateral buckling

General rules of instability analisis were presented on Lec. #5.

The weaknest cross-section – of the bigerst probability of buckling - is cross-section throug hole in web. Geometry consists from two separated areas.

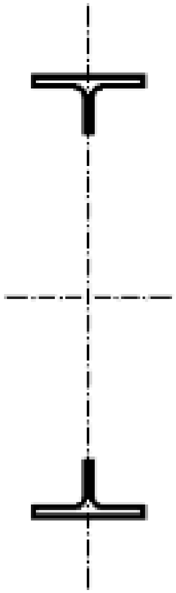


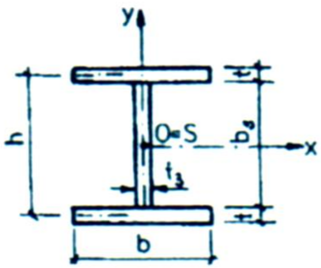
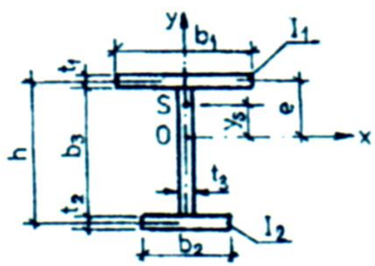
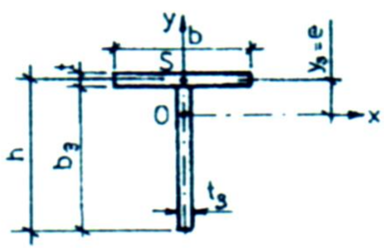
Photo: Author

Way of calculation of J_y and J_z was presented on Lab. #1 (Steiner’s theorem).

Approximation and simplified formulas can be applicated for calculation of J_T (→ #t / 69)

According to classical theory of thin-walled cross-section, J_w can be calculated for homogenous cross-section, not for complex of separated parts. So, there is problem with analysis of lateral buckling.

If we have cross-section, which not exists in tables:

Przekrój	Cechy geometryczne
	$y_s = 0$ $I_\omega = \dfrac{I_y h^2}{4}$ $I_T = \dfrac{1}{3}(2\, b t^3 + b_3\, t_3^3)$ $r_x = 0$ → Lab #1 / 62
	$y_s = \dfrac{1}{I_y}[e\, I_1 - (h - e)\, I_2] = e - \dfrac{I_2}{I_y}\, h$ $I_\omega = \dfrac{I_1\, I_2\, h^2}{I_1 + I_2}$ $I_T = \dfrac{1}{3}(b_1\, t_1^3 + b_2\, t_2^3 + b_3\, t_3^3)$ $r_x = \dfrac{1}{I_x}\left\{y_s\, I_y + b_1\, t_1\, e^3 - b_2\, t_2\,(h - e)^3 + \dfrac{t_3}{4}[e^4 - (h - e)^4]\right\}$
	$y_s = e$ $I_\omega \cong 0$ $I_T = \dfrac{1}{3}(b\, t^3 + b_3\, t_3^3)$ $r_x = \dfrac{1}{I_x}\left\{e\, I_y + b\, t\, e^3 + \dfrac{t_3}{4}[e^4 - (h - e)^4]\right\}$

Castellated beam which cooperates with concrete plate is protected from global instability.

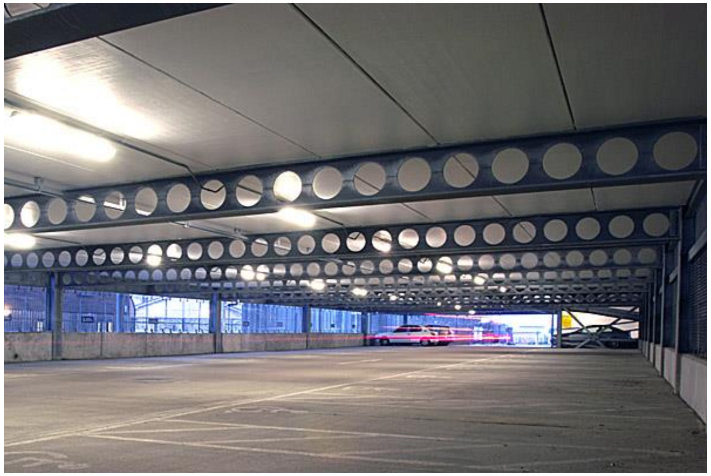


Photo: steelconstruction.info

Photo: newsteelconstruction.com

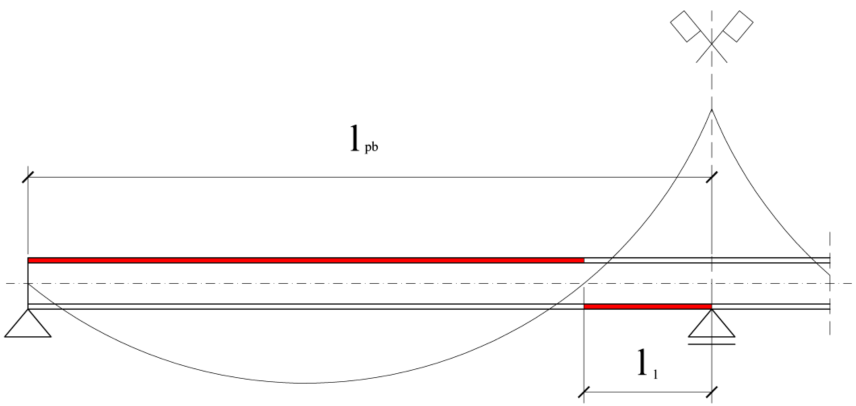
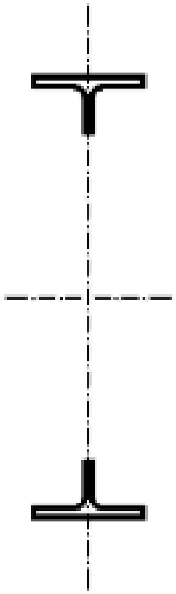


Photo: Author

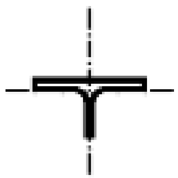
But this protection concerns, first of all, top flange. Around column, bottom flange is under compression and lateral buckling starts from bottom – unprotected – flange.

Support on column should be made for full, not castelltaed cross-section. However, this is not always case.



In case of problem with lateral buckling of castellated beam, simplified method presented in EN 1993-6 6.3 could be adopted. Lateral buckling of castellated cross-section is calculated as flexural buckling of one chord.

Photo: Author



$$M_{Ed} / (\chi_{LT}\, M_{Rd}) \leq 1{,}0 \quad \rightarrow \quad N_{Ed,\, equ} / (\chi_y\, N_{Rd}) \leq 1{,}0$$

N_{Rd} comes from area of T-section

χ_y comes from J_y (about vertical axis) of T-section, buckling curve c and critical length is equal length of flange under compression.

$N_{Ed,\, equ} = (\sigma_{\beta 1} + \sigma_{\beta 2})\, A_C$

$\sigma_{\beta 1}$, $\sigma_{\beta 2}$, $A_C \rightarrow$ #t / 63

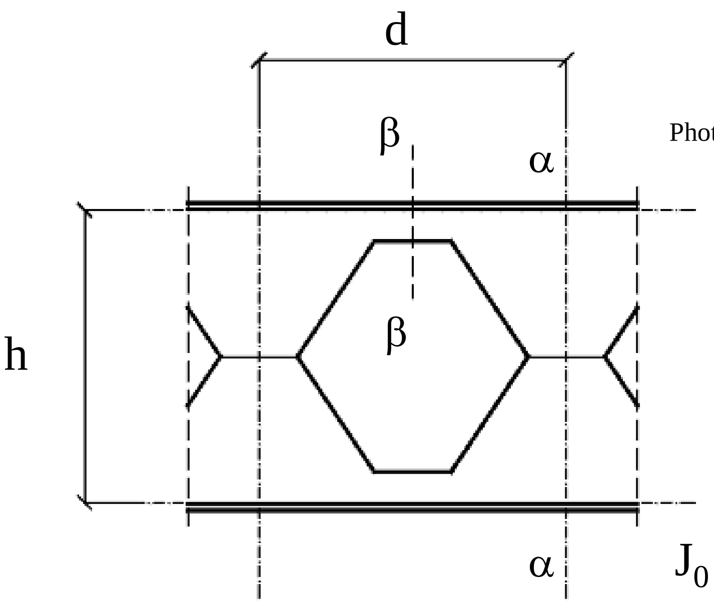
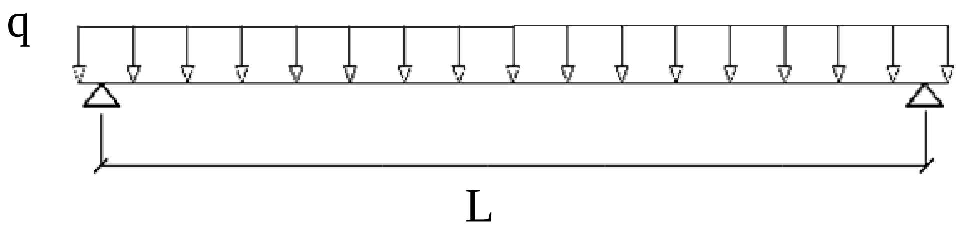


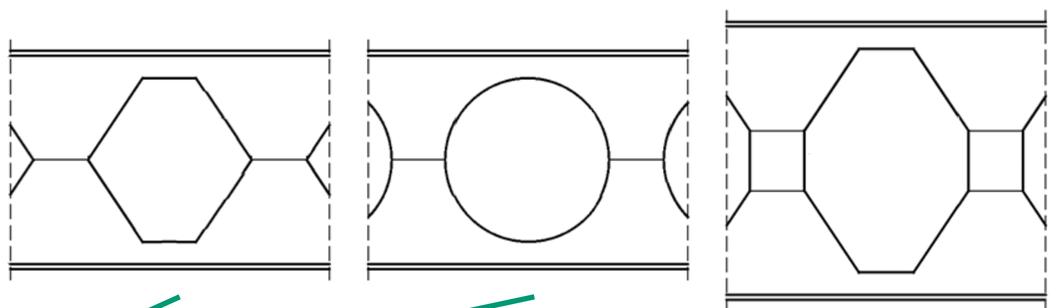
Photo: Author



Deflection colud be calculated as

$$J_0 = (2\,J_\beta + J_\alpha) / 3$$

Photo: Author



$$\Delta = 5\,q\,L^4 / (384\,E\,J_0)$$

$$\Delta = 6\,q\,L^4 / (384\,E\,J_0)$$

Photo: borga.pl

Non uniform members

Photo: largohale.com.pl

1. Generally, EJ = const is the most often case. Differences are negligible, if $\alpha \leq 10^o$; for calculations EJ = min (EJ_1 ; EJ_2). There are different rules for $\alpha > 10^o$; these rules will be presented on lecture #12

Photo: borga.pl

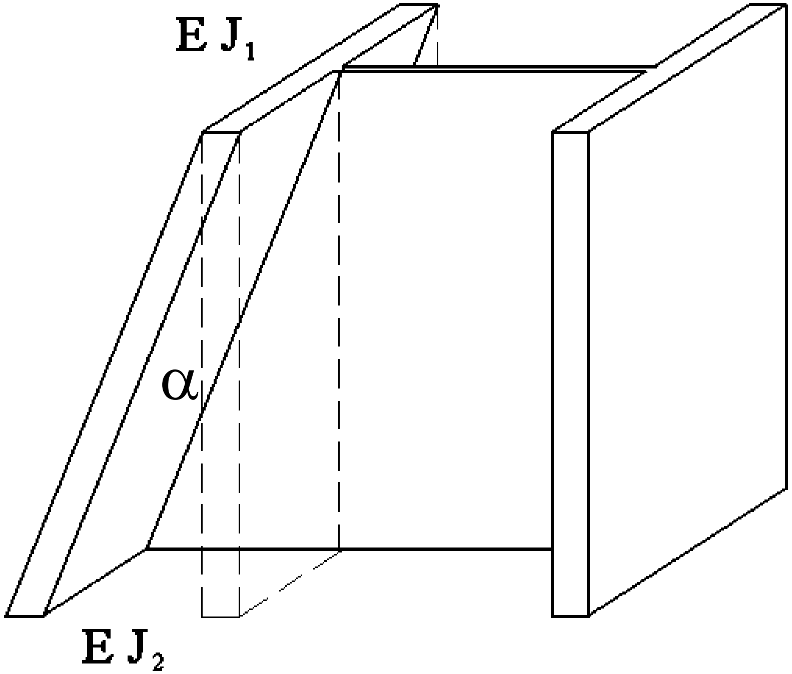
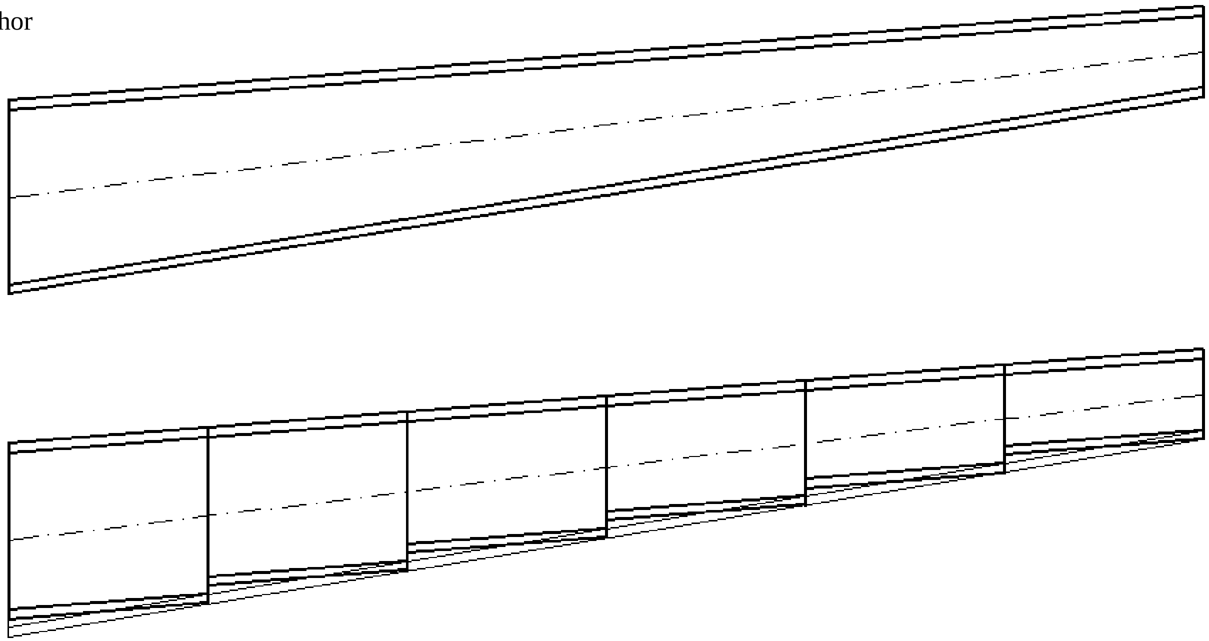


Photo: Author

For resistance calculation (I[st] – III[rd] class of cross-section), we can divide non uniform member into few part. For each of them we calculate resistance;
$EJ = \min (EJ_1 ; EJ_2)$.

Photo: Author



Much more complicated are calculations for stability (flexural buckling, lateral buckling, local buckling of web ↔ effective cross-section for IV[th] class of cross-section).

There are little changes of formulas for calculation of stability.

Phenomenon		Change for	Position
Local stability of web		ρ EN 1993-1-5 4.2 λ EN 1993-1-5 5 e_0 EN 1993-1-1 5.2	EN 1993-1-5 2.5 EN 1993-1-5 B.1
Flexural or lateral buckling	Critical length	α_{cr} EN 1993-1-1 5.2, 6.3.4 $\alpha_{ult,\,k}$ EN 1993-1-1 6.3.4	EN 1993-1-1 6.3.4 EN 1993-1-5 B2
	Stable length of segment	L_{stable}	EN 1993-1-1 BB.3.2 EN 1993-1-1 BB.3.3.3

Plate girders with corrugated web

Photo: hxssvic.en.ec21.com

Axial froce and bi-axial bending are not recommended types of loads for this type of member.

Photo: zemanhdf.pl

Photo: sugamengineers.com

Photo: builtconstructions.in

There should be applied "classical" girders with plane webs in places with big concentration of stresses (over support, joints betwen columns and roof girders, joints between roof girders...)

Resistance for bending moment

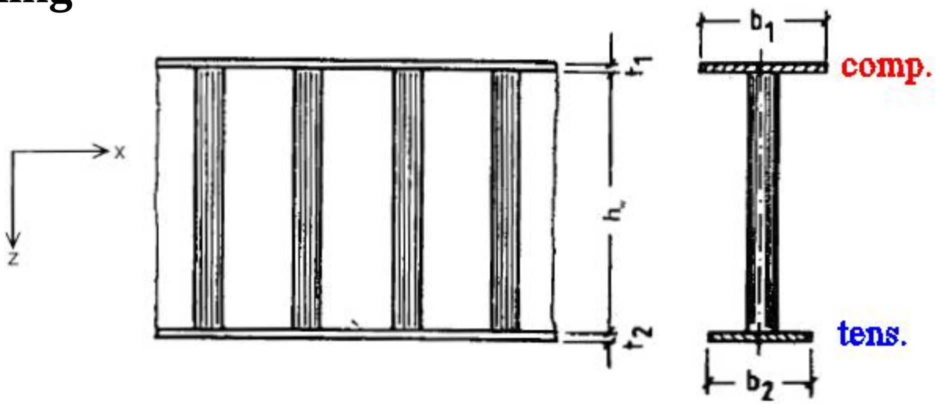
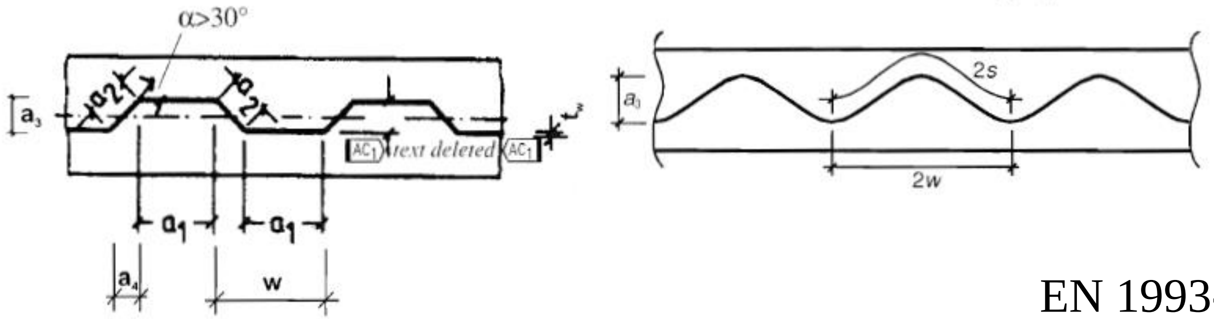


Photo: EN 1993-1-5 fig. D.1



EN 1993-1-5 (D.1)

$$M_{Rd} = \min(b_2\, t_2\, f_{yf,r}\, x / \gamma_{M0} \;\; ; \;\; b_1\, t_1\, f_{yf,r}\, x / \gamma_{M0} \;\; ; \;\; b_1\, t_1\, \chi\, f_{yf}\, x / \gamma_{M1})$$

$$x = h_w + (t_1 + t_2)$$

b_1 , b_2 - effective cross-section

$f_{yf,r} \rightarrow$ #t / 80

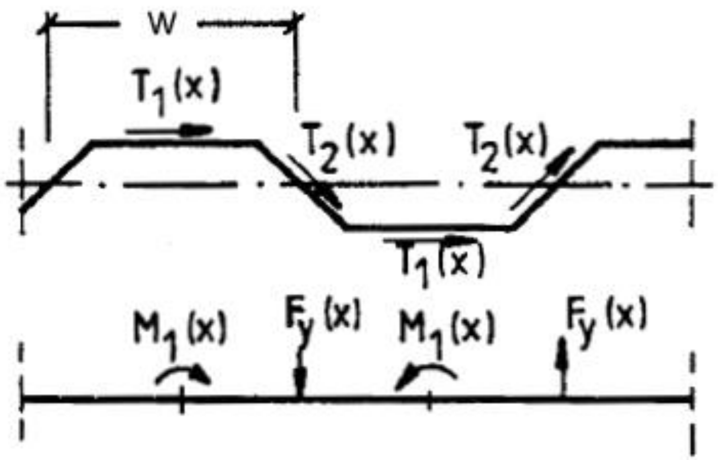
$\chi \rightarrow$ #t / 81

effective cross-section $\rightarrow$ #t / 82

$$f_{yf,r} = f_{yf}\, f_T$$

EN 1993-1-5 (D.1)

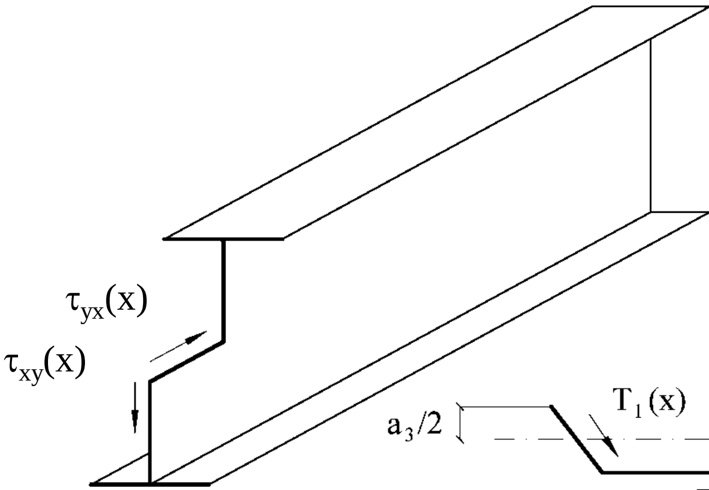
	sin shape	other shape
f_T	1,0	$1 - 0{,}4 \sqrt{\{\sigma_x(M_z) / [f_{yf} / \gamma_{M0}]\}}$



$$M_z = M_1(x)$$

Photo: EN 1993-1-5 fig. D.2

$M_1(x)$, $T_1(x)$ → #t / 81



$$\tau_{xy}(x) = \tau_{yx}(x) \rightarrow T_1(x) = V_{Ed}(x)$$

$$F_y(x) = T_1(x) \sin \alpha$$

$$M_1(x) = T_1(x)\, a_3 / 2$$

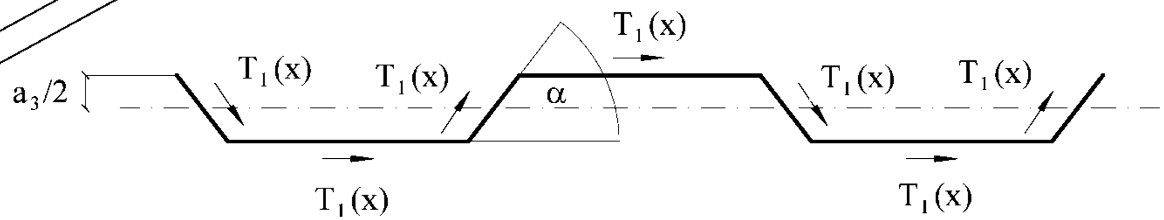
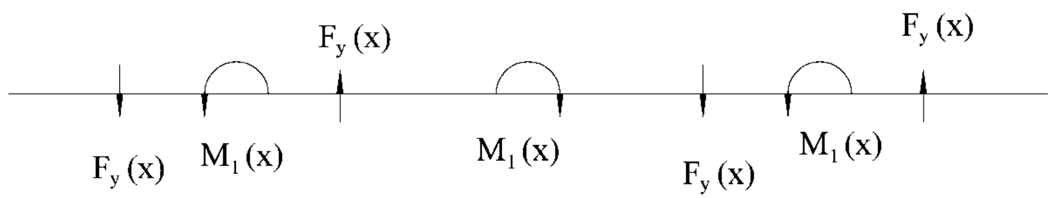
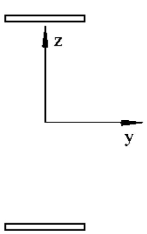


Photo: Author





Flexural buckling factor

$$\chi = \chi_z$$

for flanges only

Effective geometry for welded I-beam:

Plane web	**Corrugated web**
Initial geometry A_0 J_0	Initial geometry A_0 J_0 , **flanges only**
Shear lag in flanges - the same way	
Reduction of compressed flange (#t / 18-23)	Reduction of compressed flange - similar way to I-beam with plane web; other formulas for k_σ : 4.a. $k_\sigma = 0{,}43 + (b_1 / 2a)^2$; $a = a_1 + 2a_4$ or 4.b. $k_\sigma = 0{,}60$ $k_\sigma = \min$ (4.a. ; 4.b)
Reduction of compressed web (#t / 24-33)	-
Second reduction of compressed web (#t / 34-39)	Second reduction of compressed flange
…	…
Comparison results of two last steps	
End of calculation if small difference	

Resistance for shear force

$$V_{Rd} = \chi_c \, f_{yw} \, h_w \, t_w \, / \, (\gamma_{M1} \sqrt{3})$$

$$\chi_c = \min \left(\chi_{c,I} \quad ; \quad \chi_{c,g} \right)$$

$\chi_{c,g} \rightarrow$ #t / 84

EN 1993-1-5 (D.4)

$$\chi_{c,I} = \min \left[1{,}0 \quad ; \quad 1{,}15 \, / \, (0{,}9 + \bar{\lambda}_{c,I}) \right]$$

$$\bar{\lambda}_{c,I} = \sqrt{[f_y \, / \, (\tau_{cr,I} \sqrt{3})]}$$

	sin shape	other shape
$\tau_{cr,I}$	$[\, 5{,}34 + a_3 \, s \, / \, (h_w \, t_w) \,] \, (t_w \, / \, s)^2 \, \{ \pi^2 E \, / \, [12 \, (1-\nu^2) \,] \}$	$4{,}83 \, E \, (t_w \, / \, a_{max})^2$

$$a_{max} = \max \left(a_1 \quad ; \quad a_2 \right)$$

a_1 , a_2 , s $\rightarrow$ #t / 79 (on fig.)

$$\chi_{c,g} = \min [1,0 \quad ; \quad 1,5 / [0,5 + (\bar{\lambda}_{c,g})^2]$$

$$\bar{\lambda}_{c,g} = \sqrt{[f_y / (\tau_{cr, g} \sqrt{3})]}$$

$$\tau_{cr, g} = 32,4 \sqrt[4]{(D_x D_z^3)} / (h_w^2 t_w)$$

$$D_x = t_w^3 E w / [12 s (1-\nu^2)]$$

$$D_z = E J_z / w$$

w, s ⟶ #t / 79 (on fig.)

Box girders

Cross-sections, used, first of all, for various constructions for transport. There are box-complex of flat plates, reinforced by many longitudinal and transversal stiffeners.

Photo: steelconstruction.info

Photo: pixhder.com

Photo: constructionphotography.com

Steel bridges (modern), conveyor galleries (old-fashion)

Photo: newsgd.com

Photo: china.org.cn

Photo: cambelt.com

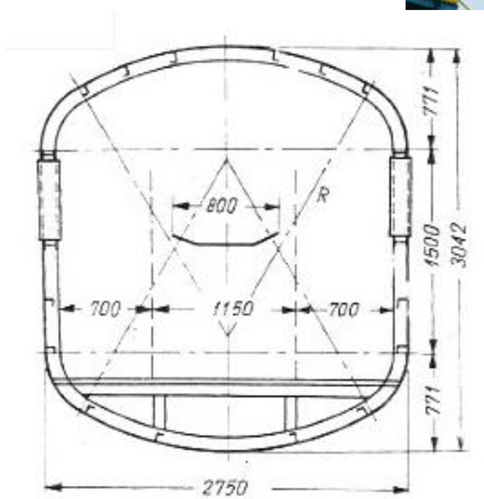


Photo: chodor-projekt.net

There are two ways of calculations of these structures.

First method: full model in FEM.

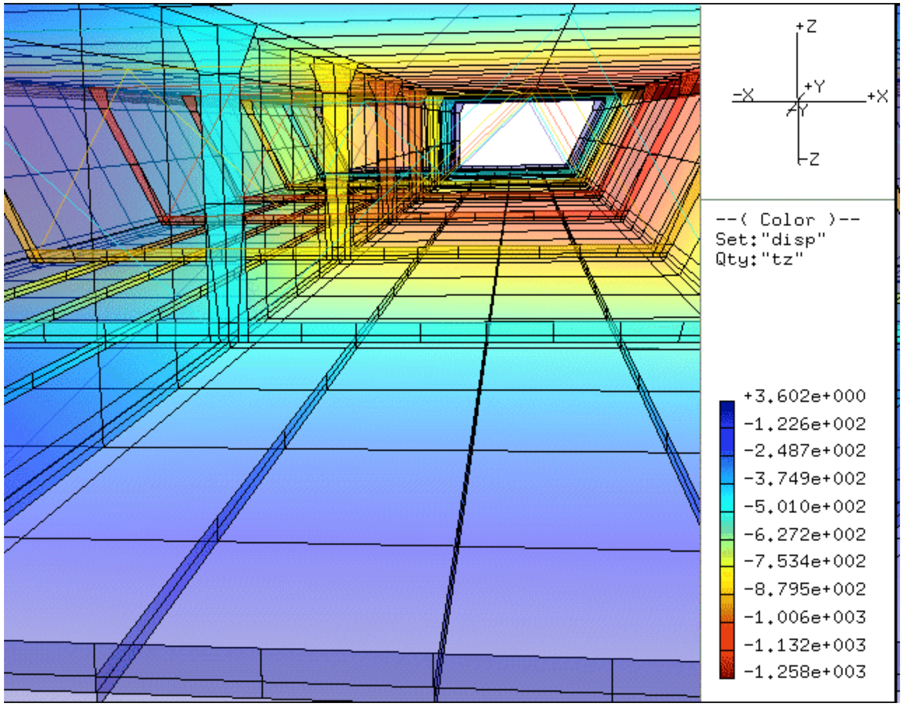


Photo: essie.ufl.edu

Second method: member is treated as a single bar. For this model, cross-sectional forces are calculated. Effective geometry is calculated, based on cross-sectional forces and initial geometry of cross-section. Resistances are calculated for effective geometry.

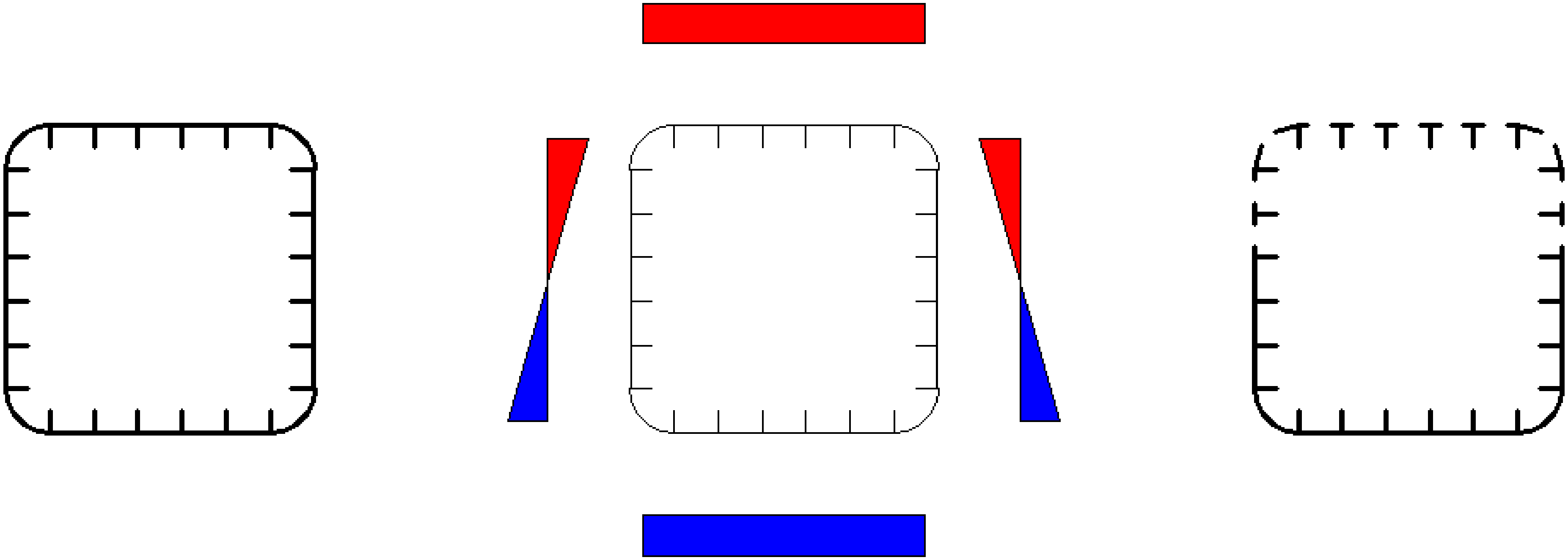


Photo: Author

Calculation of effective geometry for box girder with longitudinal stiffeners - generalisation of calculation for web of I-beam - is presented in EN 1993-1-5 p.4.5. Additional information is presented on EN 1993-1-5 appendix A.

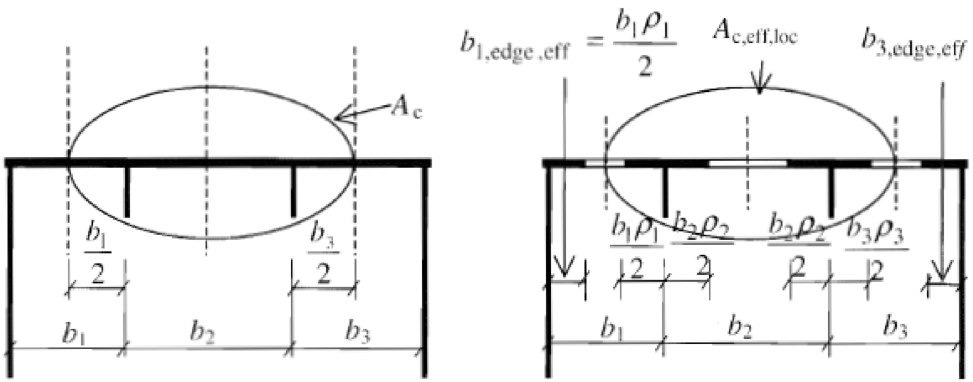


Photo: EN 1993-1-5 fig. 4.4

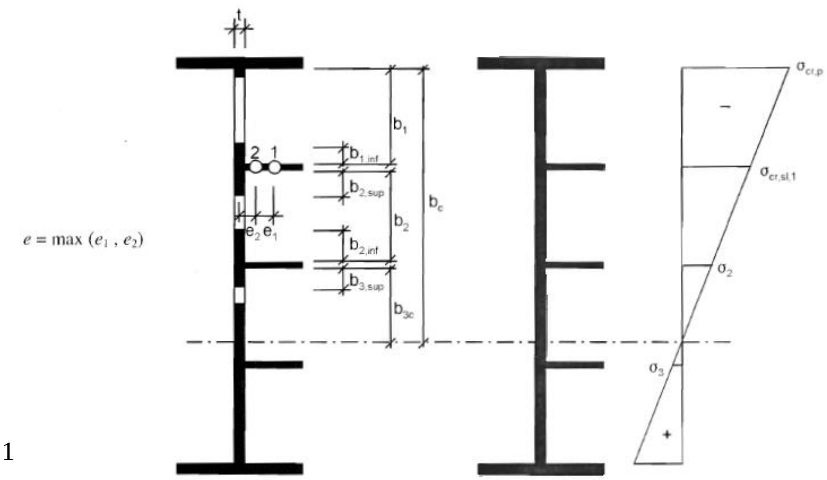


Photo: EN 1993-1-5 fig. A.1

Summary

Issue	Hot rolled I-beam	Welded I-beam
Class of cross-section (the most often)	I, II	III, IV
Analysis	Plastic (I – additionally redistribudion of bending moments)	Elastic (IV – additionally effective geometry)
Shear lag effect	The same calculation	
Resistance	Shear force, bending moment, theirs interactions; (transverse force is not danger for most cross-sections)	Shear force, bending moment, transverse force, theirs interactions
Local instability	Under shear force and flange induced buckling (not danger for most hot-rolled cross-sections)	Under axial stresses (from compressive axial force and / or bending moment) – two possibilities; under shear force; uder transverse force
Global instability	Lateral buckling (total geometry, not effective)	
Deflection	The same limits	
Reasons of application	Ease of use; „factory selection of dimensions” usually protects against local instability	No limit for cross-section depth (theoretically no limit for resistance)

Photo: weldingweb.com

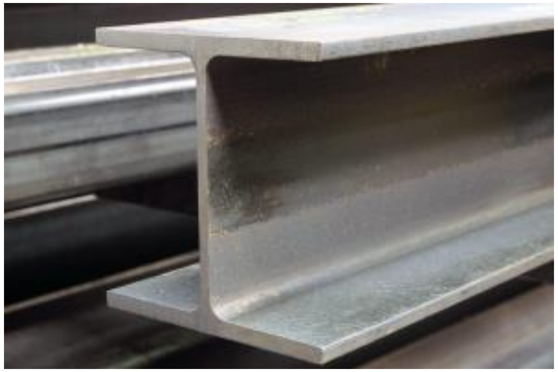


Photo: harrissteels.co.uk

<u>Hot-rolled & welded v.s. castellated</u>:

Lighter structure in case of composite concrete-steel structure →

Easy installation placement →

More colmplicated manufacturing →

Dynamic loads are not recommended →

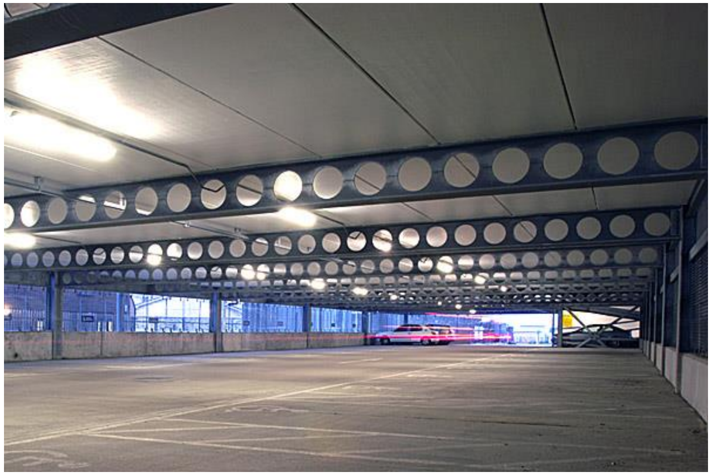


Photo: steelconstruction.info

Photo: c--beams.com

Photo: weldingweb.com

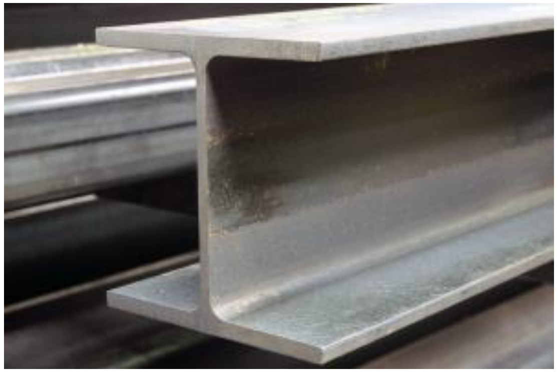


Photo: harrissteels.co.uk

Hot-rolled & welded v.s. non-uniform:

Lighter structure →

Better matching of cross-section geometry to cross-sectional forces →

More complicated manufacturing →

Complex global instability analysis →

Photo: borga.pl

Photo: weldingweb.com

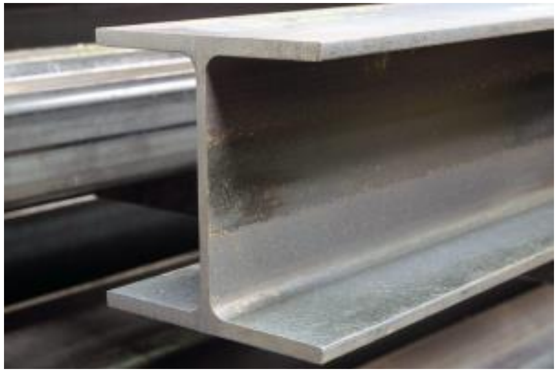


Photo: harrissteels.co.uk

Hot-rolled & welded v.s. corrugated web:

Bigger resistance for shear and transverse force →

Smaller resistance for bending moment →

More complicated manufacturing →

Photo: hxssvic.en.ec21.com

Photo: weldingweb.com

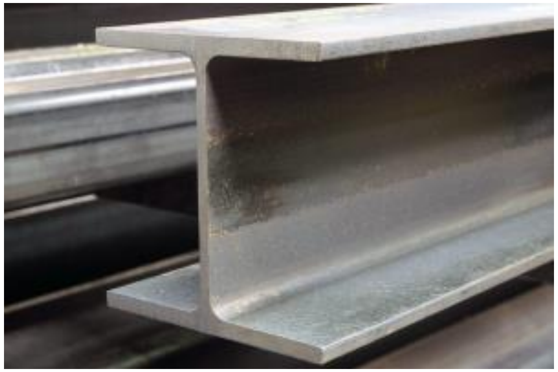


Photo: harrissteels.co.uk

Hot-rolled & welded v.s. box:

Recommended for bridges →

More complicated manufacturing →

Photo: pixhder.com

Examination issues

Types of instability in case of IV class of cross-section

Inital assumptions about plate girders geometry

Algorithm of calculation for effective geometry

Prevention for instability of flange and web

Thank you for attention

tmichal@pk.edu.pl