

Metal Structures II

Lecture IV

Crane supporting structures

Beams, columns, bracings

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Fatigue resistance

EN 1993-1-9

$$\Delta\sigma_E = \sigma_{\max} - \sigma_{\min}$$

$$\sigma_{\max} = \sigma \text{ (dead weight of structure + live load)}$$

$$\sigma_{\min} = \sigma \text{ (dead weight of structure)}$$

$$\Delta\tau_E = \tau_{\max} - \tau_{\min}$$

$$\tau_{\max} = \tau \text{ (dead weight of structure + live load)}$$

$$\tau_{\min} = \tau \text{ (dead weight of structure)}$$

EN 1993-1-9 (8.1), (8.2), (8.3)

$$\Delta\sigma_E / (1,5 f_y) \leq 1,0$$

$$\Delta\tau_E / (1,5 f_y / \sqrt{3}) \leq 1,0$$

$$\gamma_{Ff} \Delta\sigma_E / (\Delta\sigma_C \gamma_{Mf}) \leq 1,0$$

$$\gamma_{Ff} \Delta\tau_E / (\Delta\tau_C \gamma_{Mf}) \leq 1,0$$

$$(\gamma_{Mf} \gamma_{Ff} \Delta\sigma_E / \Delta\sigma_C)^3 + (\gamma_{Mf} \gamma_{Ff} \Delta\tau_E / \Delta\tau_C)^5 \leq 1,0$$

EN 1993-1-9 tab. 3.1

γ_{Mf}

Assessment method	Consequence of failure	
	Low	High
Damage tolerant	1,00	1,15
Safe life	1,15	1,35

$\gamma_{Ff} = 1,0$

Sentence „consequence of failure” concerns not only Consequences Classes. First of all, for this term static scheme of structure is important. Each local damage of simple-span beam (or other statically determinable structure) could cause global destruction.

Consequence	Statically determinable structure	Statically indeterminable structure
High	Main part (for example: beam)	CC3, CC2
Low	Secondary part (for example: bracings)	CC2, CC1

EN 1993-1-9 3(7)

Damage tolerant method

- selecting details, materials and stress level so that in the event of the formation of cracks a low rate of crack propagation and a long critical crack length would result
- provision of multiple load path
- provision of crack-arresting details
- provision of readily inspectable details during regular inspections

Safe-life method

- selecting details and stress levels resulting in a fatigue life sufficient to achieve the β -values to be at least equal to those required for ultimate limit state verifications at the end of the design service life

Method	Consequences	Recommendation
Damage tolerant	Regular inspections and repairs are necessary	When a small local fracture (in time between inspections and repairs) is not dangerous for structure.
Safe-life	No need for regular inspections	When a local fracture could lead to destruction (in time between inspections) element and / or structure.

Damage tolerant method: cheaper structure, additional costs of inspections and repairs

Safe-life method: more expensive structure, no additional costs

EN 1991-3 (2.16)

Stresses are calculated for specific value of live load:

$$\Delta\sigma_E = \sigma_E (\text{dead weight of structure} + Q_e) - \sigma_E (\text{dead weight of structure}) = \sigma_E (Q_e)$$

$$\Delta\tau_E = \tau_E (\text{dead weight of structure} + Q_e) - \tau_E (\text{dead weight of structure}) = \tau_E (Q_e)$$

$Q_e \rightarrow$ lecture #2 / 69 - 78

$\Delta\sigma_C$ $\Delta\tau_C$:

EN 1993-1-9

Tab. 8.1	for plain members and mechanically fastened joints
Tab. 8.2	for welded built-up sections
Tab. 8.3	for transverse butt welds
Tab. 8.4	for weld attachments and stiffeners
Tab. 8.5	for load carrying welded joints
Tab. 8.6	for hollow sections
Tab. 8.7	for lattice girder node joints
Tab. 8.8	for orthotropic decks - closed stringers
Tab. 8.9	for orthotropic decks open stringers
Tab. 8.10	for top flange to web junctions of runway beams

Number of tables which are needed for calculations (tab. 8.1 – cutting of elements):

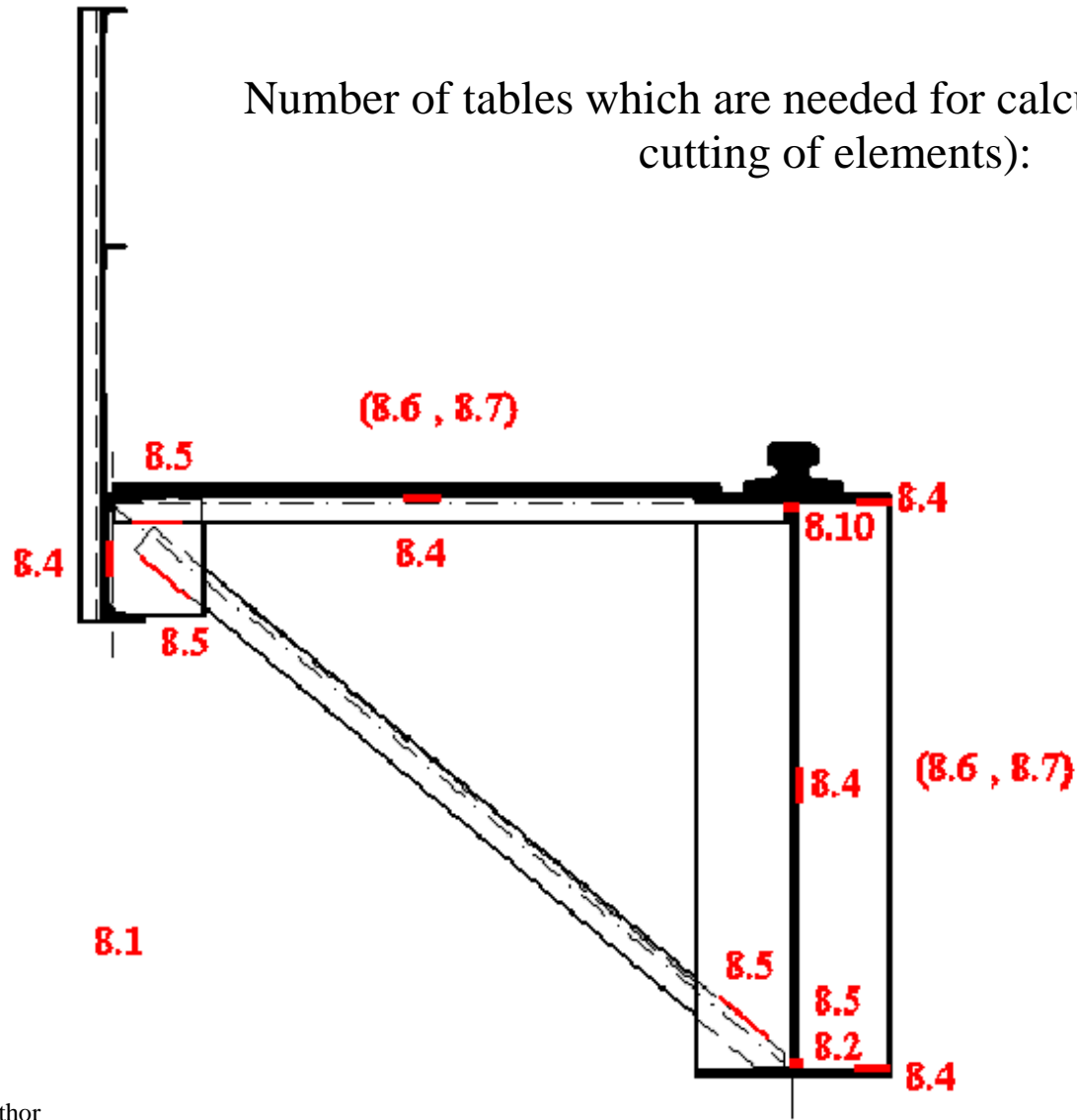






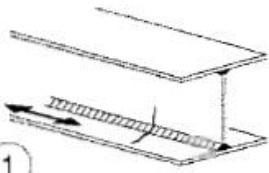
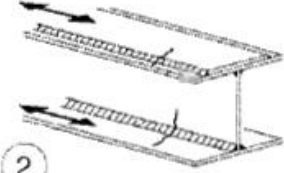
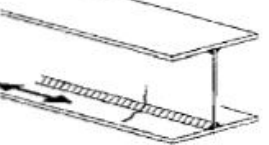
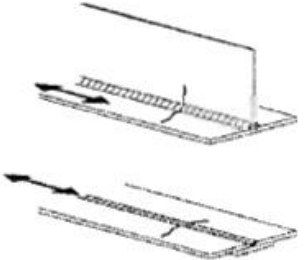
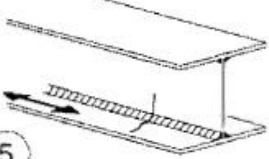
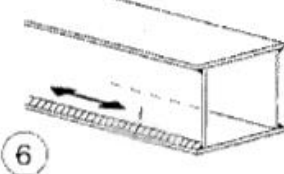
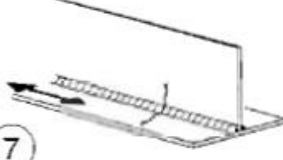
Photo: Author

Tab. 8.1 Cutting of elements

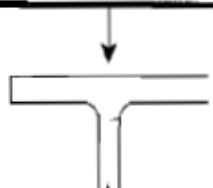

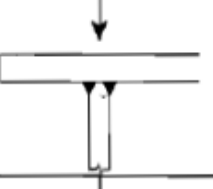
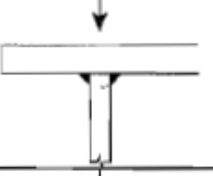
Photo: EN 1993-1-9 tab. 8.1

$\Delta\sigma_C$	160	<p>NOTE The fatigue strength curve associated with category 160 is the highest. No detail can reach a better fatigue strength at any number of cycles.</p> 	<p><u>Rolled or extruded products:</u></p> <ol style="list-style-type: none"> 1) Plates and flats with as rolled edges; 2) Rolled sections with as rolled edges; 3) Seamless hollow sections, either rectangular or circular.
$\Delta\sigma_C$	140		<p><u>Sheared or gas cut plates:</u></p> <ol style="list-style-type: none"> 4) Machine gas cut or sheared material with subsequent dressing. 5) Material with machine gas cut edges having shallow and regular drag lines or manual gas cut material, subsequently dressed to remove all edge discontinuities.
$\Delta\sigma_C$	125		<p>Machine gas cut with cut quality according to EN 1090.</p>
$\Delta\tau_C$	100 m = 5		<p>6) and 7) Rolled or extruded products as in details 1), 2), 3)</p>

Tab. 8.2 Web – flange welds

$\Delta\sigma_C, \Delta\tau_C$	125	 	<p><u>Continuous longitudinal welds:</u></p> <p>1) Automatic or fully mechanized butt welds carried out from both sides.</p> <p>2) Automatic or fully mechanized fillet welds. Cover plate ends to be checked using detail 6) or 7) in Table 8.5.</p>
$\Delta\sigma_C, \Delta\tau_C$	112	 	<p>3) Automatic or fully mechanized fillet or butt weld carried out from both sides but containing stop/start positions.</p> <p>4) Automatic or fully mechanized butt welds made from one side only, with a continuous backing bar, but without start/stop positions.</p>
$\Delta\sigma_C, \Delta\tau_C$	100	 	<p>5) Manual fillet or butt weld.</p> <p>6) Manual or automatic or fully mechanized butt welds carried out from one side only, particularly for box girders</p>
$\Delta\sigma_C, \Delta\tau_C$	100		<p>7) Repaired automatic or fully mechanized or manual fillet or butt welds for categories 1) to 6).</p>

Tab. 8.10 Web – flange welds for top flange

$\Delta\sigma_C, \Delta\tau_C$	160	① 	1) Rolled I- or H-sections
$\Delta\sigma_C, \Delta\tau_C$	71	② 	2) Full penetration tee-butt weld
$\Delta\sigma_C, \Delta\tau_C$	36*	③ 	3) Partial penetration tee-butt welds, or effective full penetration tee-butt weld conforming with EN 1993-1-8
$\Delta\sigma_C, \Delta\tau_C$	36*	④ 	4) Fillet welds

EN 1993-6 8.2

(4) For high fatigue crane classes, transverse web stiffeners or other attachments should not be welded to the top flanges of runway beams.

^{N4)} Odsyłacz krajowy: Błąd w oryginale: powinno być „ ..nie powinny być spawane do dolnych pasów..”. Chodzi o to, by unikać silnego karbu zmęczeniowego, jakim jest spoina poprzeczna względem kierunku rozciągania.

Photo: Author

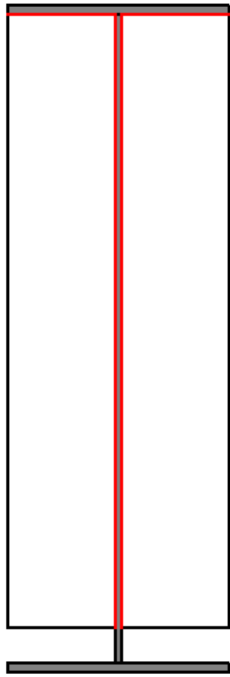
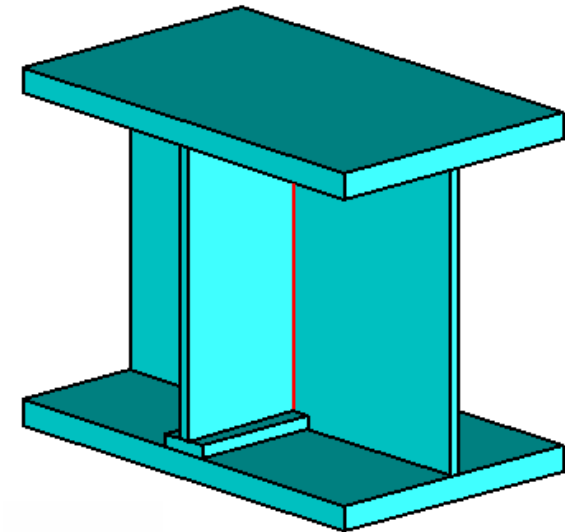
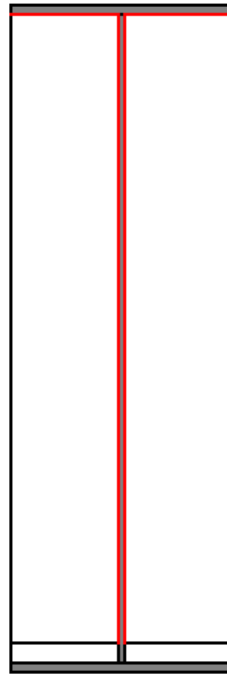


Photo: Author



Stiffeners can't be welded to bottom flange. There are two possibilities of execution:

- Height of stiffener is lower than height of web (no contact between stiffener and bottom flange);
- Between stiffener and bottom flange are inserted additional steel plate by method "push-in" (thickness of plate a little bigger than wide of gap; full contact between stiffener and bottom flange).

One-span beam is recommended structure - for each type of loads top flange is always compressed and bottom flange is always tensed.

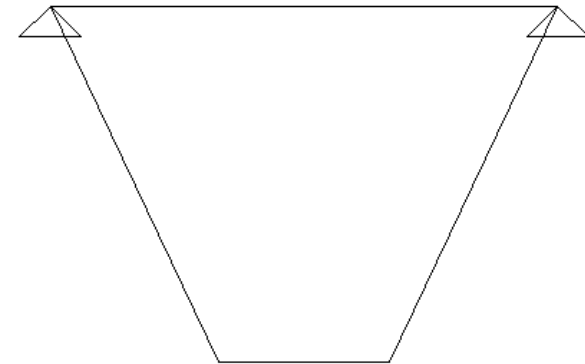


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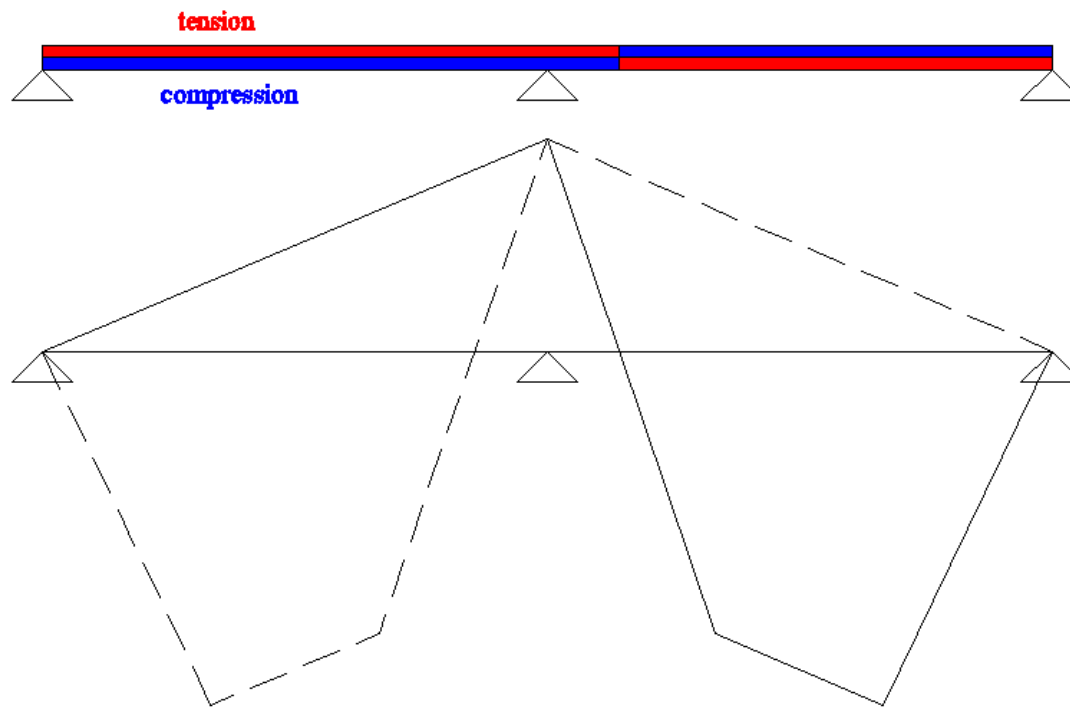
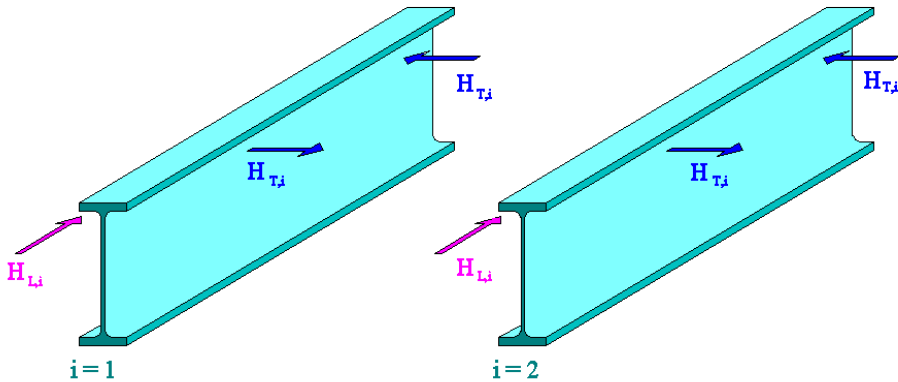


Photo: Author

Multi-span beam is not recommended structure - for different types of loads both flanges are sometimes compressed and sometimes tensed.

Deformations

Photo: Author



There are different values of horizontal and vertical loads, acts on both run-beam. It cases different deformations of beams. We must take into consideration max and min values of these loads = max and min value of deformations.

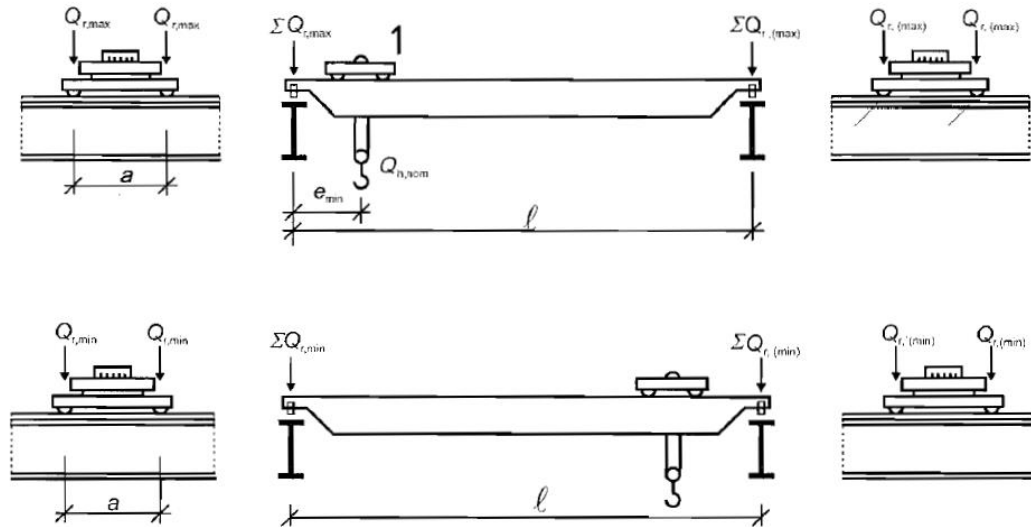


Photo: EN 1991-3 fig. 2.1

There are very important horizontal and vertical displacements for "normal" structures - requirements for beams and columns:

Member	w_{\max} or w_3
Main roof girder (truss or beam)	$L / 250$
Purlin	$L / 200$
Corrugated sheel - roofing	$L / 150$
Floor girder: primary beam	$L / 350$
secondary beam	$L / 250$
Door head or window head	$L / 500$
w_{\max} = netto (total - precamber) w_3 = from variable actions L -length of beam ot 2x length of cantilever	

It is recommended that horizontal displacements do not exceed the following limits:

- $H / 150$ for single-storey structures (without cranes);
- $H / 500$ for multistorey structures;

H - level of considered girder to the top of the foundation

EN 1993-1-1 N.A. 22

Similar conditions exist for run-beams.

Additionally, there must be analysed relative deformations of two run-beams:

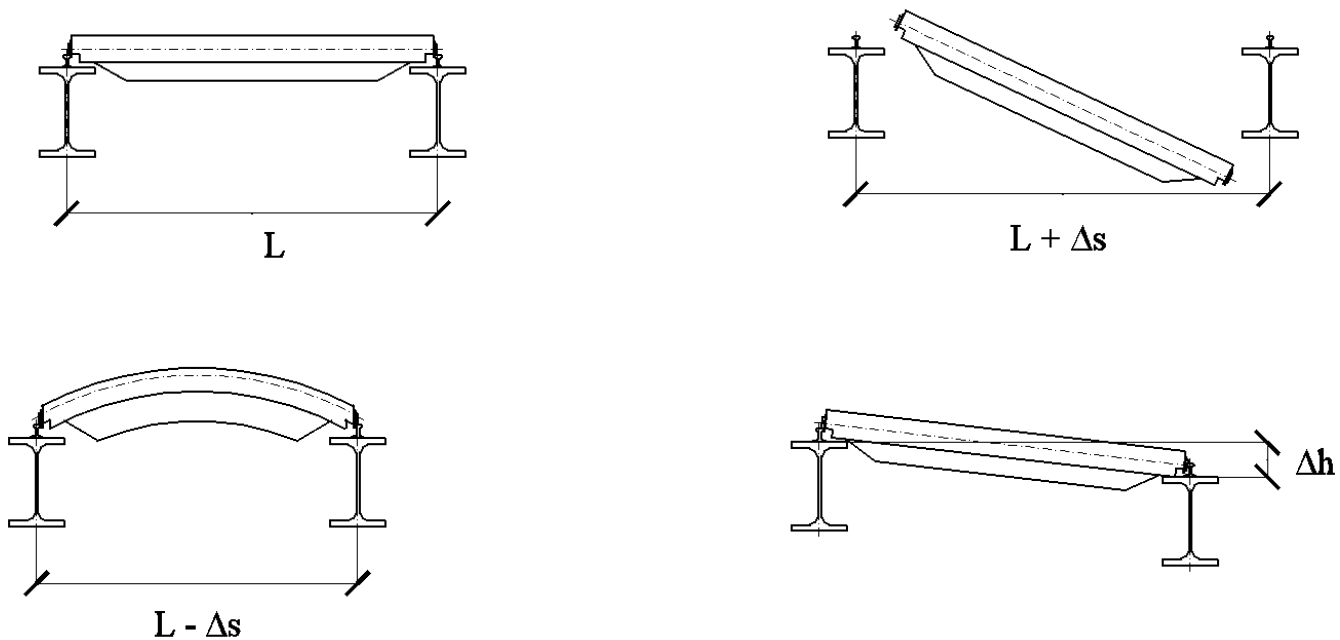


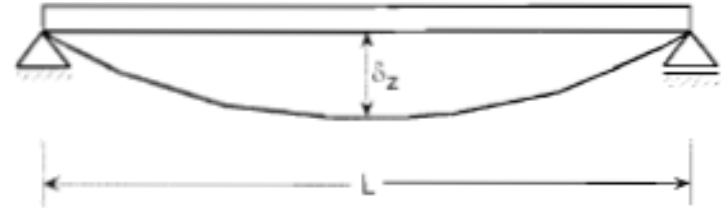
Photo: Author

EN 1993-6 tab. 7.1

Vertical deformation, crane:

$$\delta_z \leq \min(L / 600 ; 25 \text{ mm})$$

Photo: EN 1993-1-9 tab. 7.1

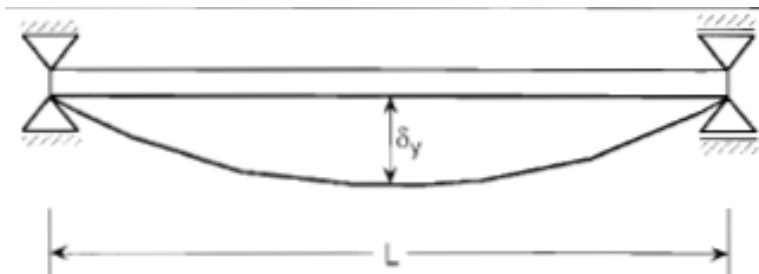


Vertical deformation, monorail hoist block:

$$\delta_{\text{pay}} \leq L / 500$$

Photo: EN 1993-1-9 tab. 7.1

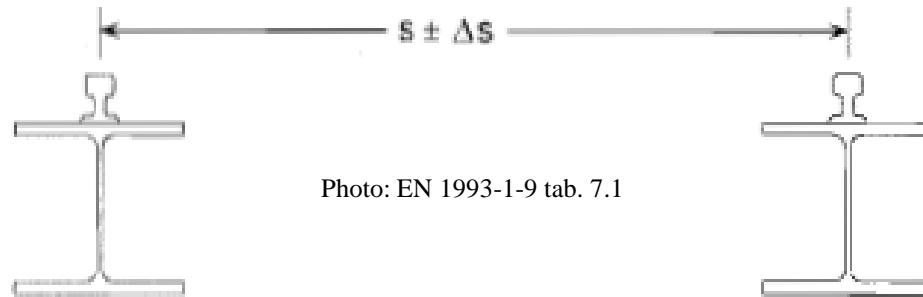
Horizontal deformation:



$$\delta_y \leq L / 600$$

EN 1993-6 tab. 7.1

Change of spacing between centres of crane rail rail, including thermal changes:



$$\Delta s = \delta_{\text{left}} + \delta_{\text{right}} \leq 10 \text{ mm}$$

EN 1993-6 tab. 7.1

Horizontal deflections and deviations of crane runways are considered together in crane design. Acceptable deflections and tolerances depend on the details and clearances in guidance means. Provided that the clearance c between the crane wheel flanges and the crane rail (or between the alternative guidance means and the crane beam) is also sufficient to accommodate the necessary tolerance, larger deflection limits can be specified for each project if agreed with the crane supplier and the client.

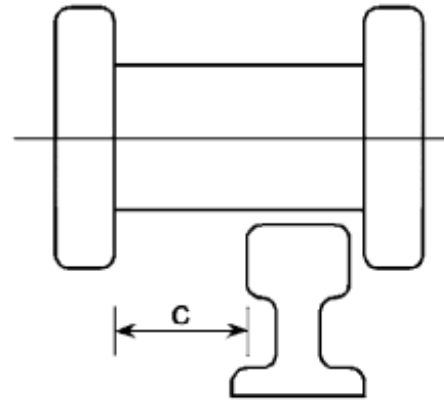
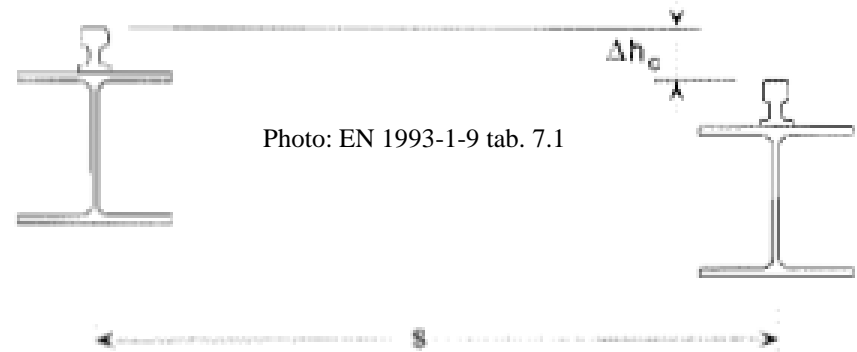


Photo: EN 1993-1-9 tab. 7.1

EN 1993-6 tab. 7.1



Difference between vertical deformations:

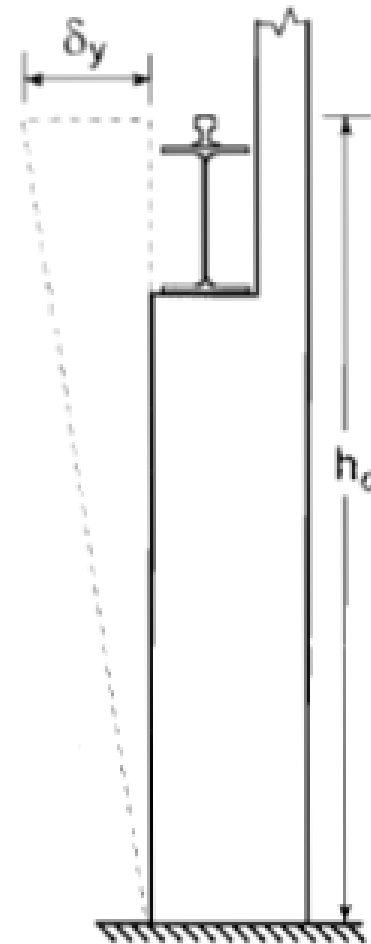
$$h_c \leq s / 600$$

EN 1993-6 tab. 7.1

Photo: EN 1993-1-9 tab. 7.1

Horizontal displacement of frame / column at crane support level:

$$\delta_y \leq h_c / 400$$



Medium wind:

→ #2 / 52

20 m /s

EN 1991-3 A.1 (6)

Strong wind - according to EN 1991-1-4:

	V [m/s]
1	22
2	26
3	22



Photo: EN 1991-1-4 fig. NB.1

EN 1993-6 tab. 7.1

Difference between horizontal displacements of adjaced frames / columns)

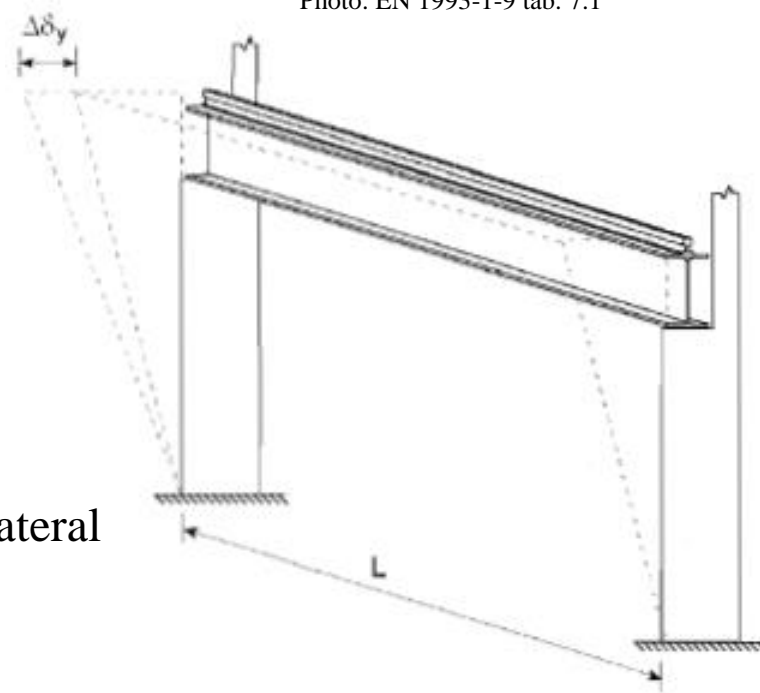
- indoor crane (max wind load);
- outdoor crane (in-service wind load);
- combination: wind (max or in-service, respectively) + lateral crane forces;

$$\Delta\delta_y \leq L / 600$$

- out-off-service wind load (outdoor crane only):

$$\Delta\delta_y \leq L / 400$$

Photo: EN 1993-1-9 tab. 7.1



Connections

Surge connectors attaching the top flange of a runway beam to the supporting structure should be capable of accommodating the rotation movements and vertical movements; should take into account the possible need for lateral and vertical adjustment of the runway beams.

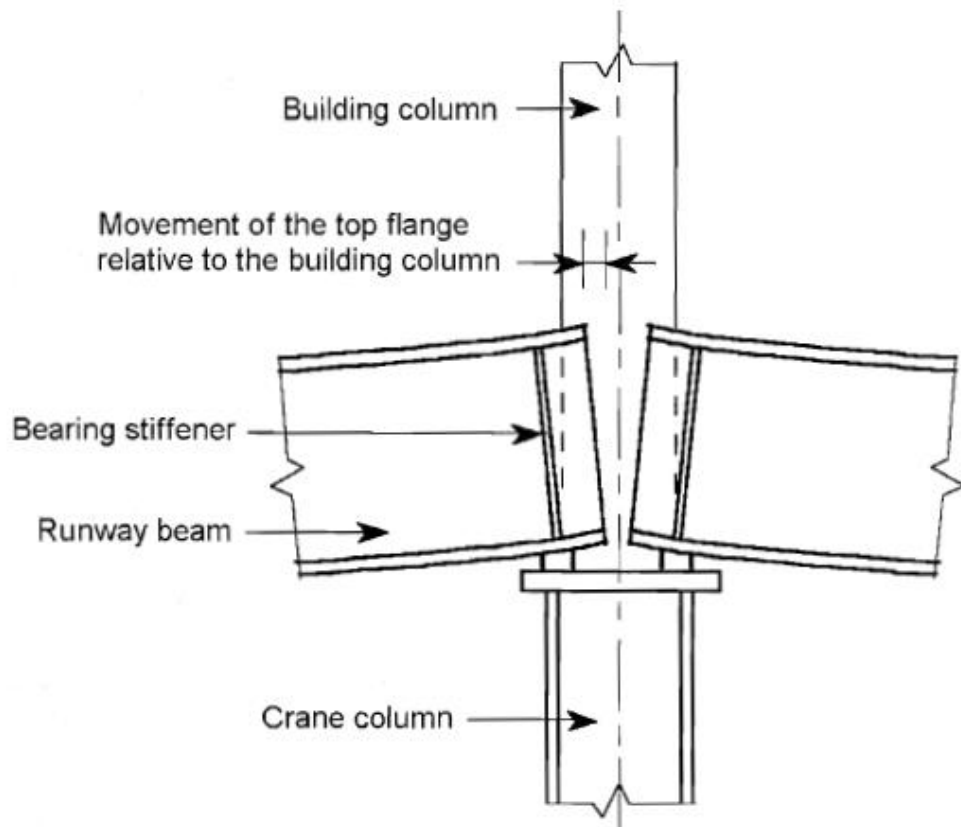
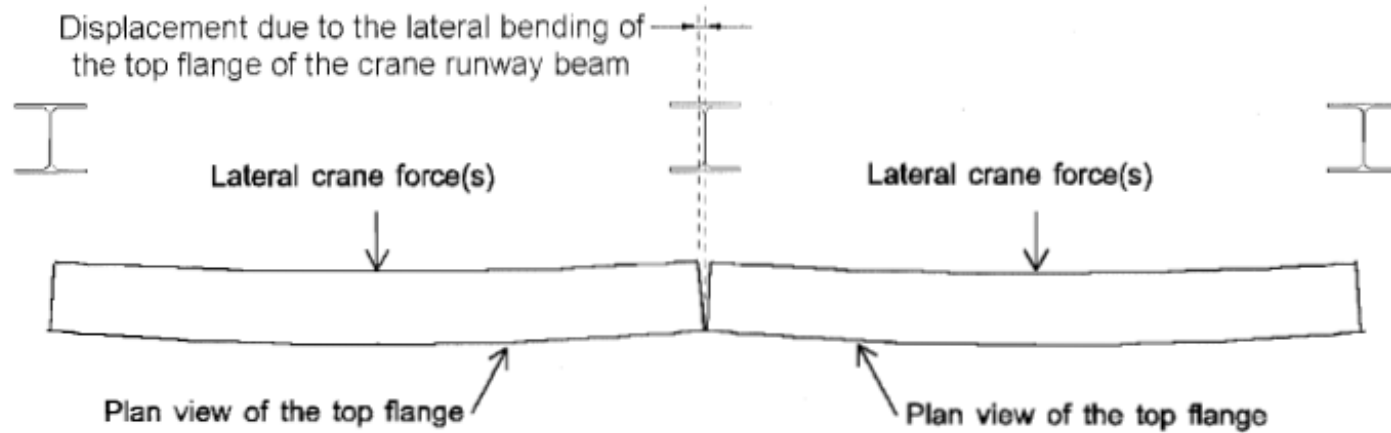


Photo: EN 1993-6 fig. 8.1

Photo: EN 1993-6 fig 8.2



Longitudinal bolted connection in compressed part of cross-section

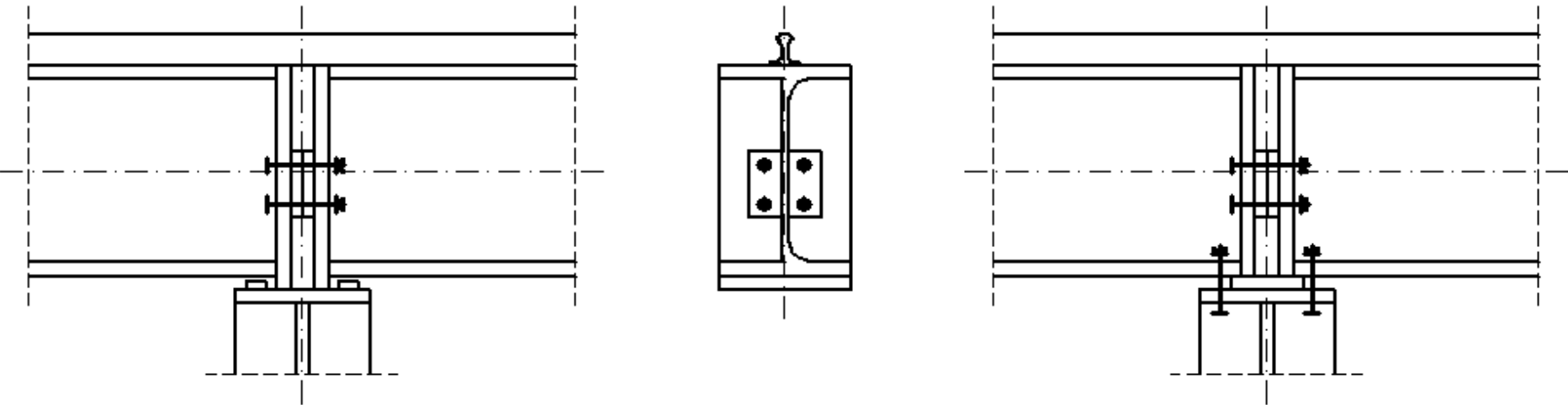


Photo: Author

Rail fixings

Joint of rails oblique and offset from the joint of beams

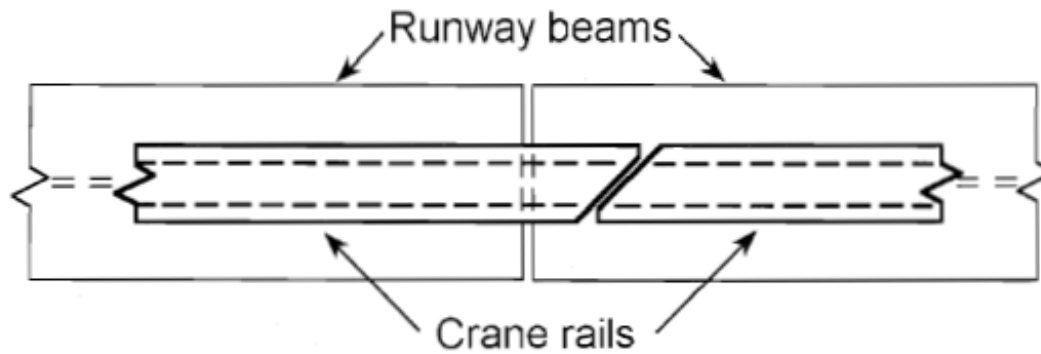


Photo: EN 1993-6 fig. 8.3

Columns

Photo: stabud.eu



Photo: konar.eu

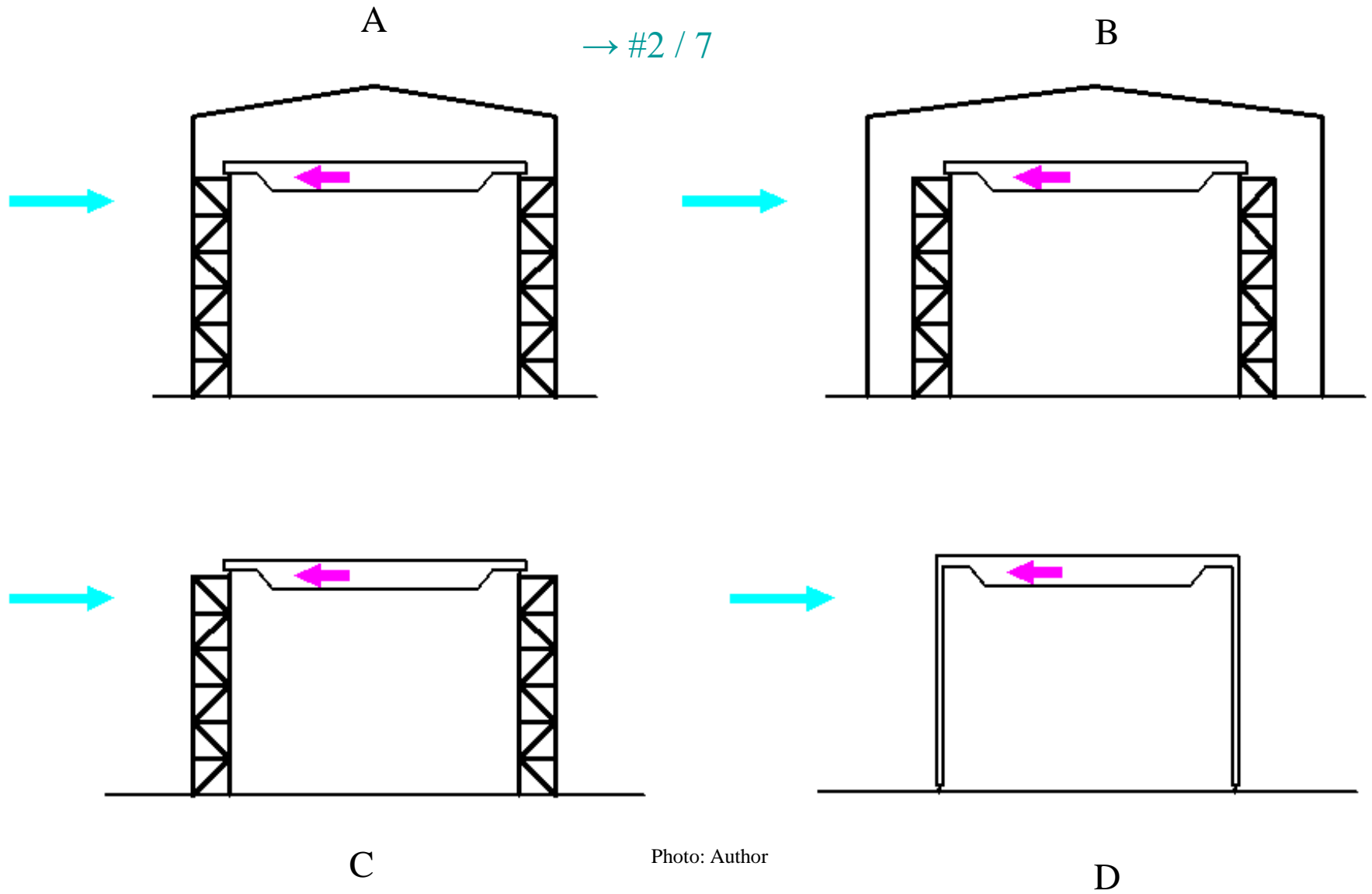


Photo: zksgrzelak.eu

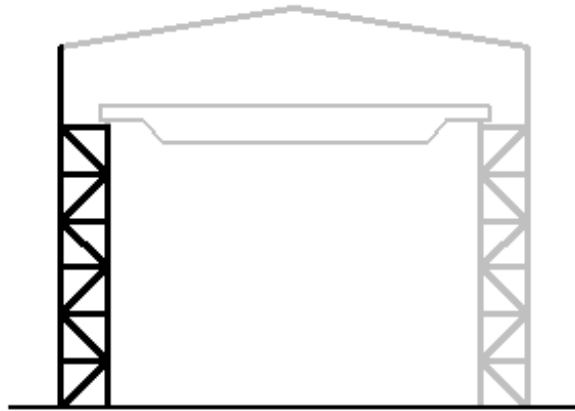
I-column (hot rolled / welded)

Battened column

Laced column



Additional part of column above crane.



No additional part of column.

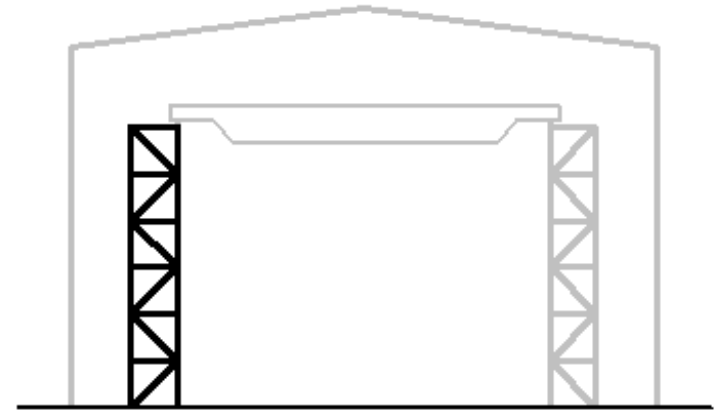


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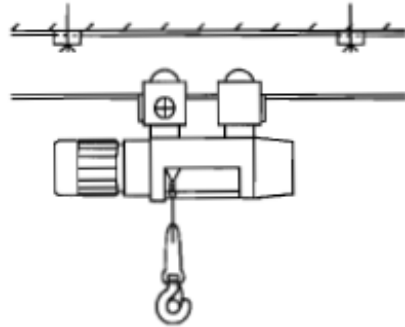
No additional part of column.



No column.

Monorail hoist block

Photo: EN 1991-3 fig.1.2



Overhead underslung crane

Photo: EN 1991-3 fig.1.3

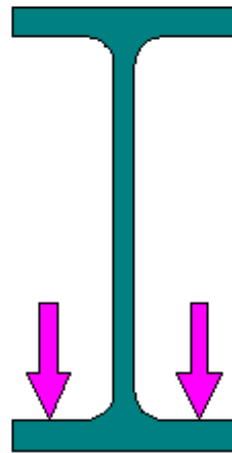
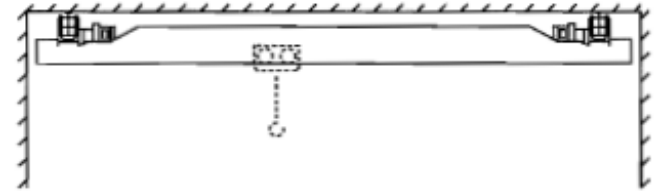


Photo: Author

Run-beams suspended to structure.

Monorail hoist block and underslung crane - the most often solution is suspension run-beams to structure of hall or additional portal frame.



Photo: crscranessystems.com



Photo: abuscranes.pl

Photo: abuscranes.pl



Photo: pelloby.com



Photo: resources.scia.net

Run-beam is connected to hanger by bolts through top flange. Behavior of such joint is similar to rigid bolted joint beam-column.

Only **tensile stresses** exist in joint run-beam – hanger.
Local **shear** region is effect of uneven tension in effective regions of both groups of bolts.

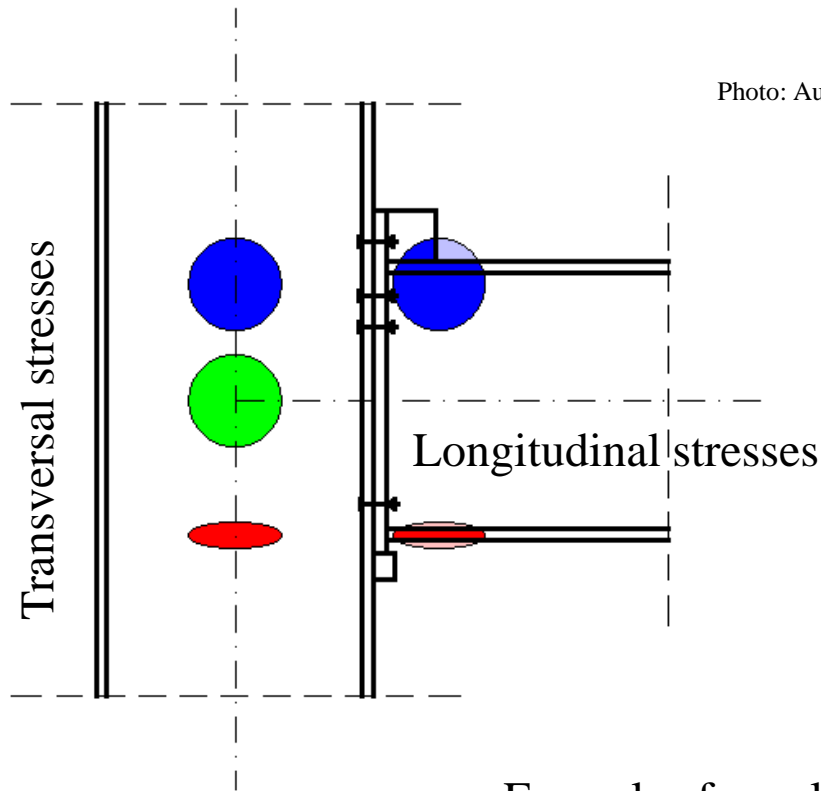
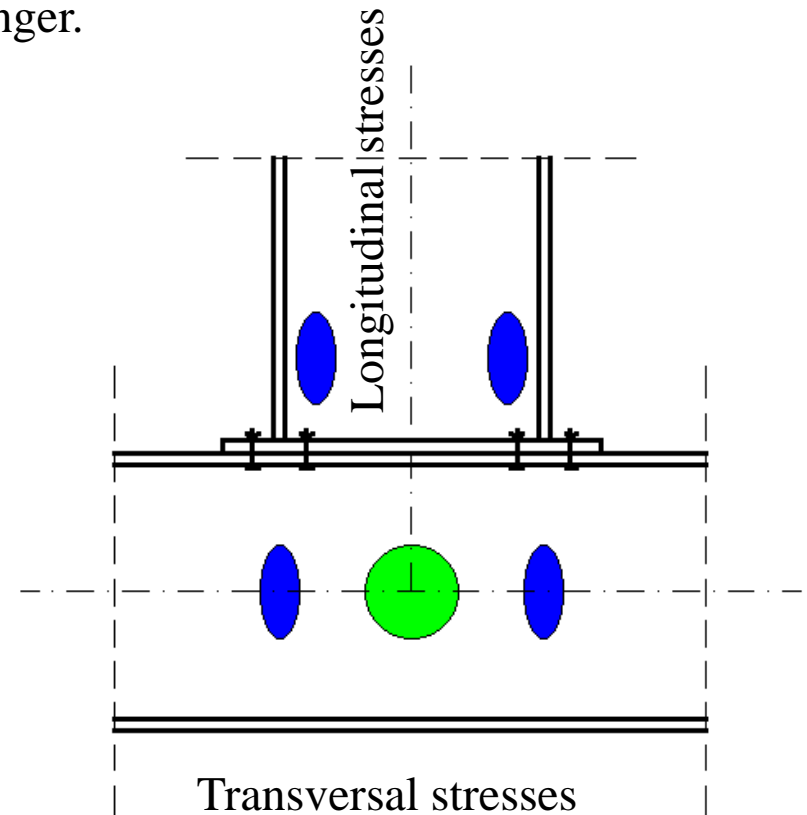


Photo: Author



Formulas for column are used for run-beam – in both types of element stresses are transversal. Longitudinal stresses are in hanger, the same as in frame beam.

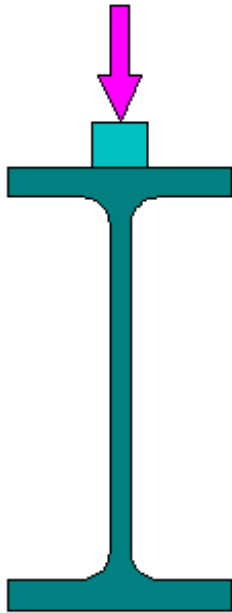
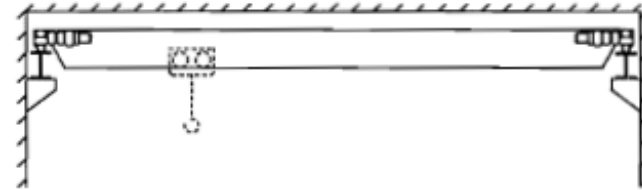


Photo: Author

Overhead top-mounted crane

Photo: EN 1991-3 fig.1.4



Run-beams supported by columns.

Top-mounted crane,
type of run-beam:

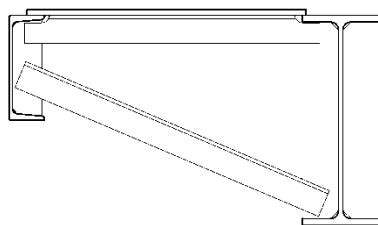
Photo: Author



Hot-rolled I-columns;
Welded I-columns

Hot-rolled I-columns;
Welded I-columns

On cantilever;
Different cross-section for boht part of
column



Welded I-column (C-sections or I-
sections as flanges, plates as web);

Laced columns;
Battened columns;

Welded I-column (C-sections or I-
sections as flanges, plates as web);

Laced columns;
Battened columns;

Different cross-section for boht part of
column

Photo: konar.eu



Cantilever, change of cross-section.

Photo: udhavind.com



Cantilever, no change of cross-section.

Photo: stabud.eu



Welded I-column, change of cross-section.

Photo:hak.com.pl



Batted column, change of cross-section.

Photo: zksgrzelak.eu



Laced column

Photo: konar.eu



Photo: konar.eu

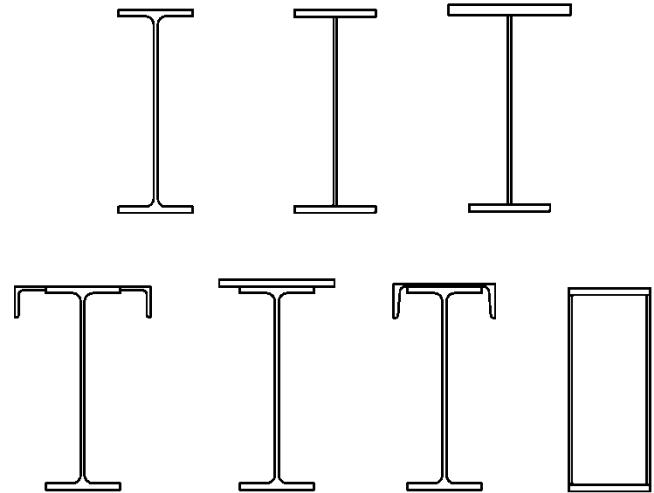


Photo: konar.eu



Short cantilever is one type of beam, for which more important is shear force than bending moment. Load from run-beam is usually very big (big shear force), but short cantilever is too short to develop important value of bending moment.

Beam

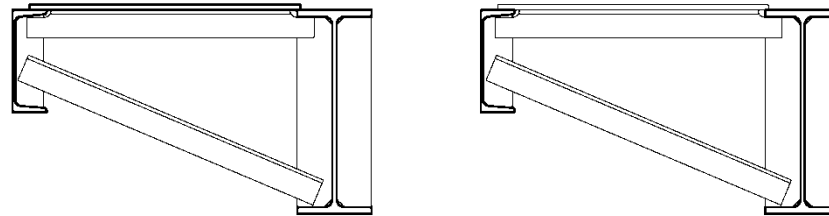


Column

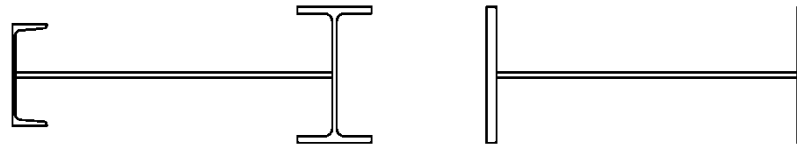
Photo: Author

Small horizontal transversal loads, acts on run-beam → small bending moment in columns
→ no necessary high cross-section of column.

Beam



Column



or battened or laced column

Photo: Author

Big horizontal transversal loads, acts on run-beam \rightarrow big bending moment in columns \rightarrow there is necessary high cross-section of column.

Photo: EN 1993-1-1 fig 6.7

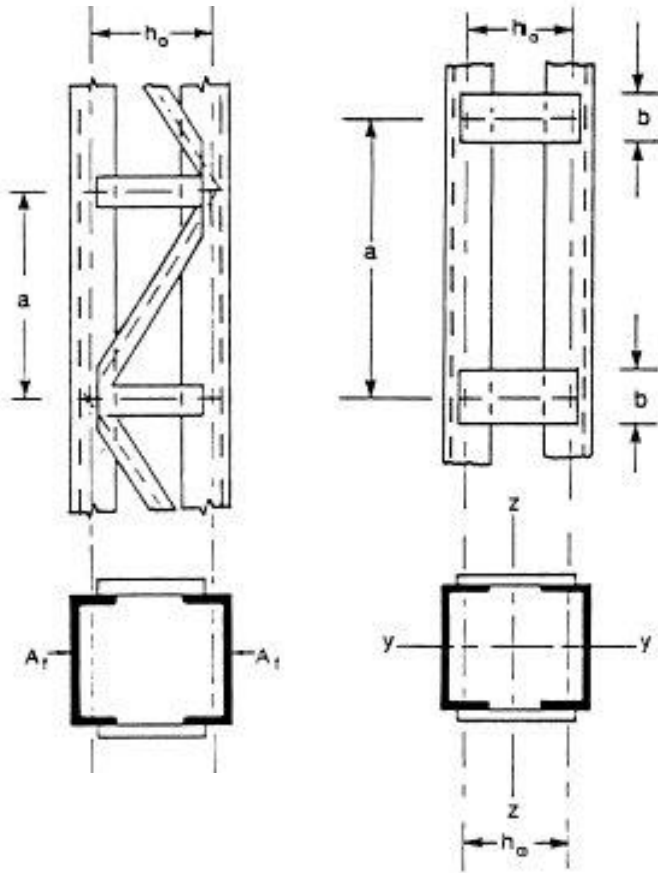
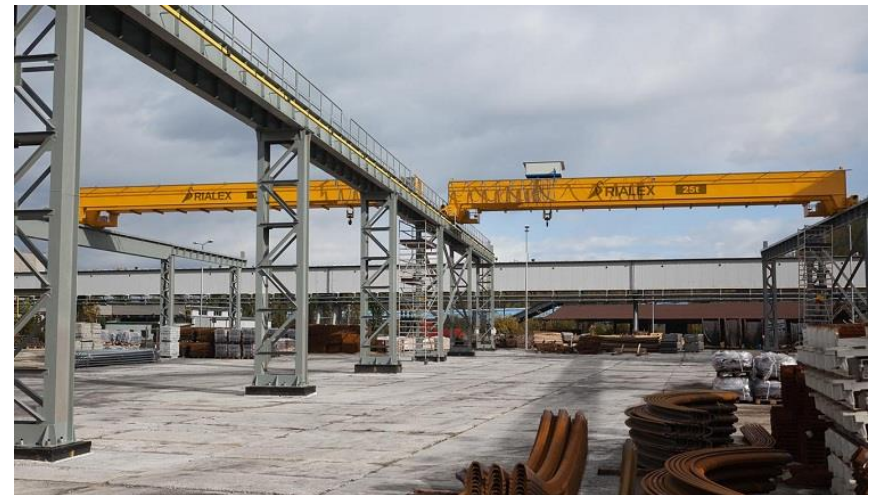


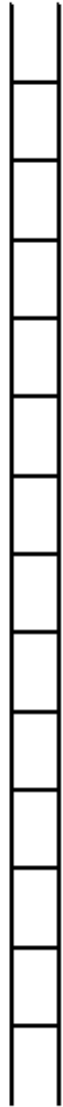
Photo:hak.com.pl



Photo: zksgrzelak.eu

There is special way of calculations for laced columns and battened columns according to Eurocode.

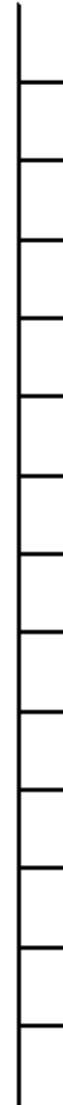




For these three types of members (laced compression members, battened compression members) we should use special way of calculations.

Of course, we can put full geometry (chords and lacing system or batten plates) into computer programm, but each membes consist of many sub-members

$n = 29$



$n = 41$

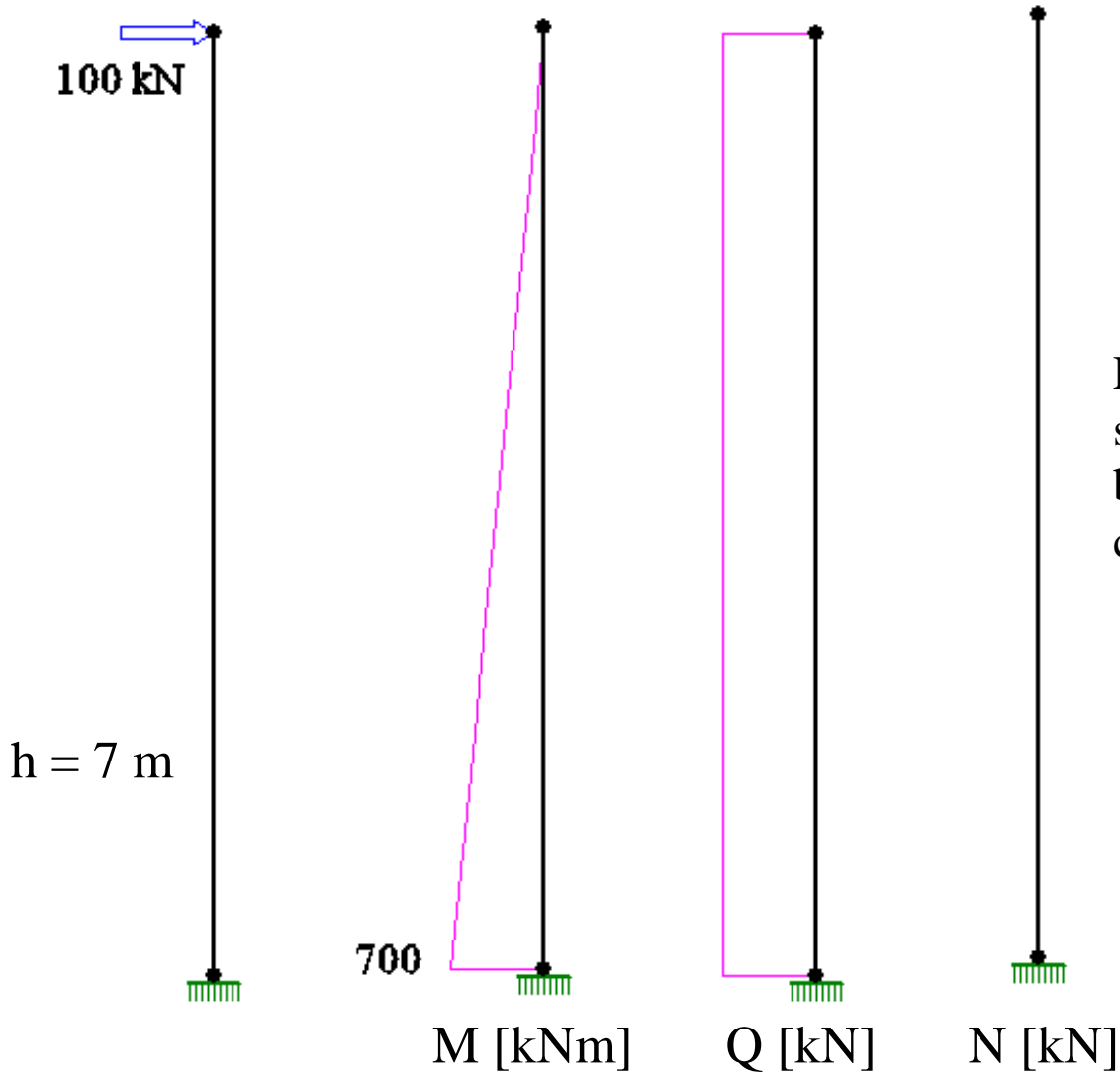
Photo: Author

Photo: Author

There is possible, that in structure we have more than 100 000 sub-members; it means very long term of calculations.



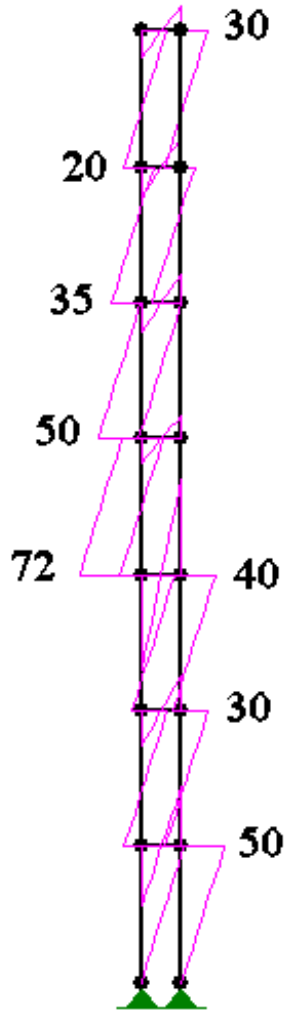
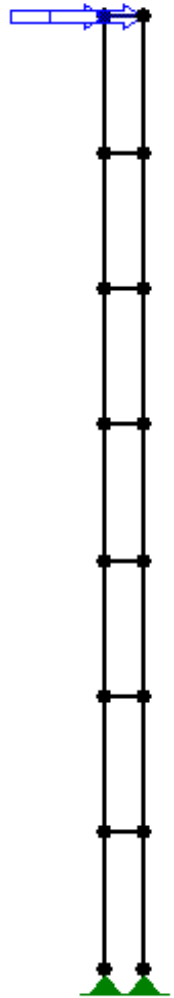
Photo: s9.flog.pl



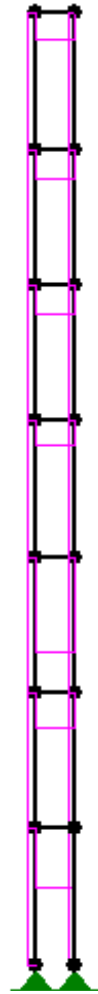
Example – difference between cross-sectional values for normal cantilever, battened cantilever and laced cantilever.

Photo: Author

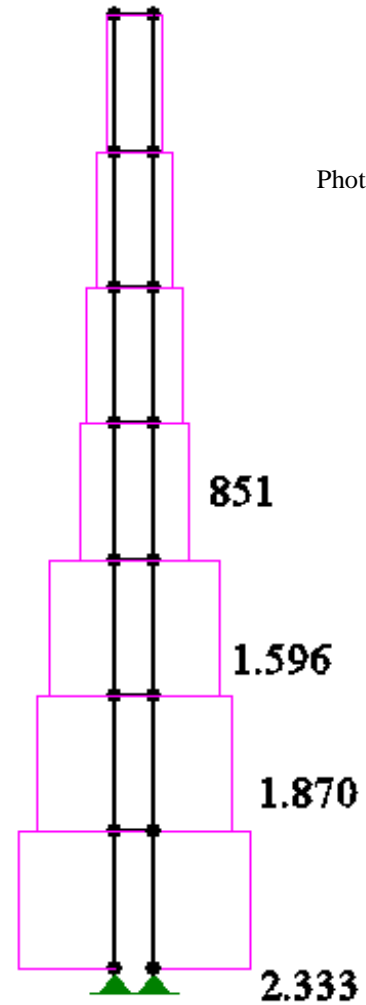
2 x 50 kN



M [kNm]



Q [kN]

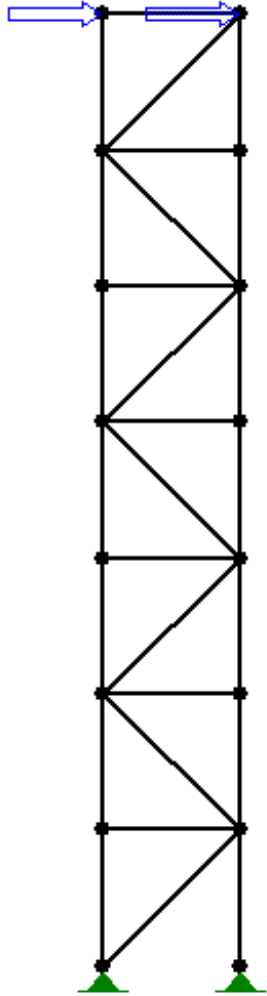


N [kN]

Photo: Author

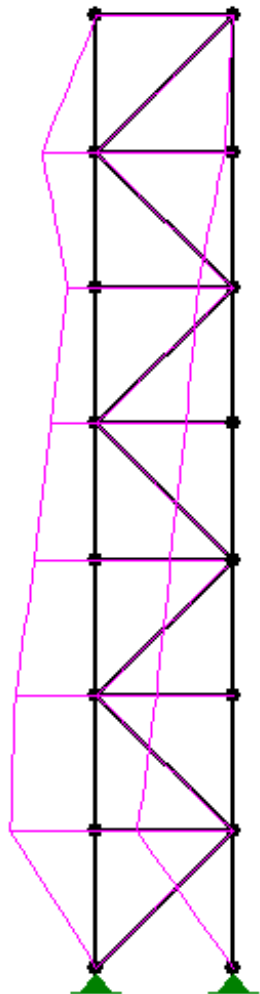
$h_0 = 30 \text{ cm}$

50 kN 50 kN

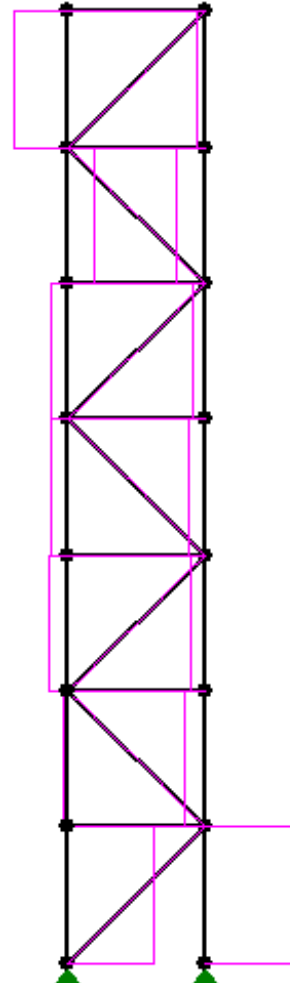


$h_0 = 100$ cm

4

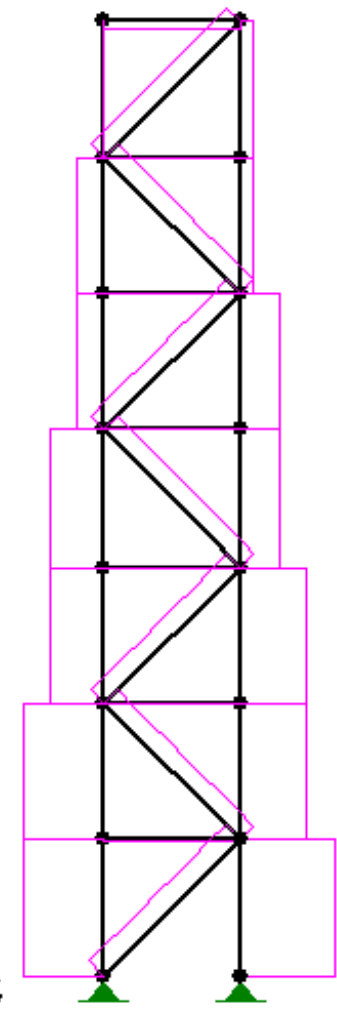


M [kNm]



Q [kN]

592



700

N [kN]

Photo: Author

There are big differences between distance between axes of chords, length of battens and gap between chords. Accurate computer calculations for full geometry are possible only for computer programme with possibility of putting „off-sets”.

Additionally, proportion between length of batten and high of batten is as for plate member, not bar member.

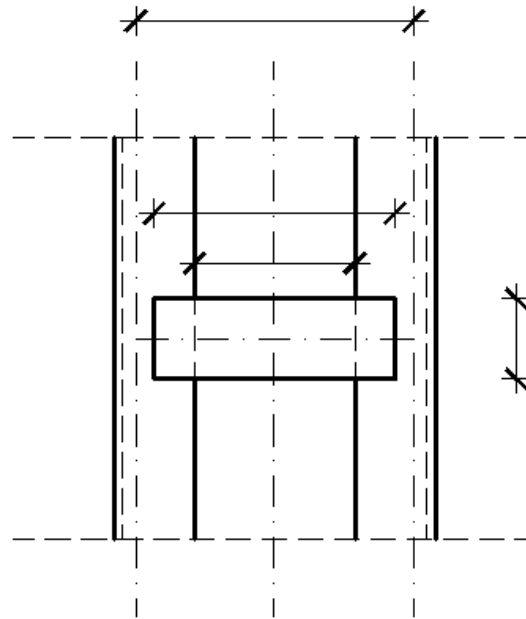
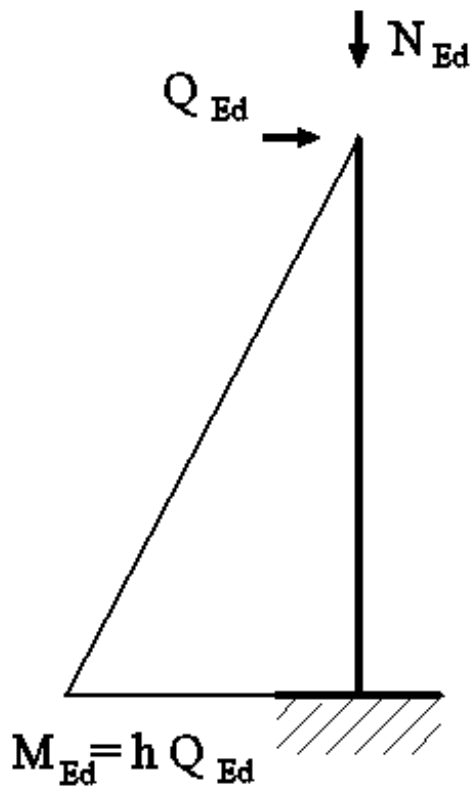


Photo: Author

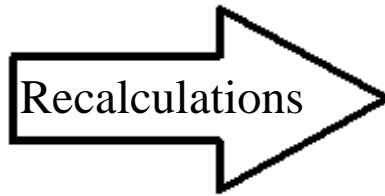
Because of these reasons, we use special type of calculation as follow:

General information about cross-sectional forces:

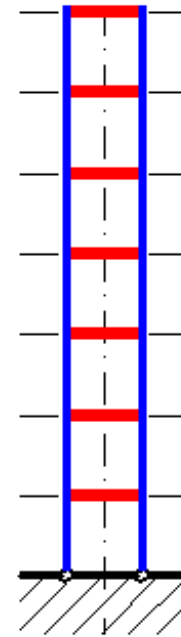


Calculation as for single-bar member; global values of M_{Ed} , V_{Ed} , N_{Ed}

Photo: Author

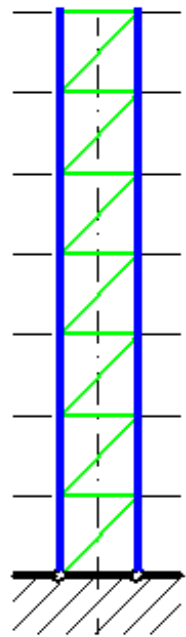


battens:
 $M_{b, Ed}$
 $V_{b, Ed}$



chords:
 $M_{ch, Ed}$
 $N_{ch, Ed}$
 $V_{ch, Ed}$

Laces:
 $N_{l, Ed}$



Local values of $M_{ch, Ed}$, $M_{b, Ed}$, $V_{ch, Ed}$, $V_{b, Ed}$, $N_{ch, Ed}$, $N_{l, Ed}$

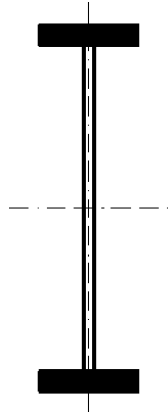
Recalculation is made according to few rules, concern multi-chord members:

- Own stiffness of chord could be neglected for global bending stiffness (#t / 56-57);
- Shear stiffness is important and must be taken into consideration (#t / 58-60);
- Initial imperfections must be analysed (#t / 61);
- Local cross-sectional forces in chords, battens and laces (#t / 62-69);
- Specific form of instability must be taken into consideration (#t / 70-73);
- Battens and laces, resistance and instability (#t / 74-78);
- Interaction of flexural buckling and lateral buckling (#t / 79-81);

Influence of **own stiffness** and **Steiner's theorem** on effective stiffness:

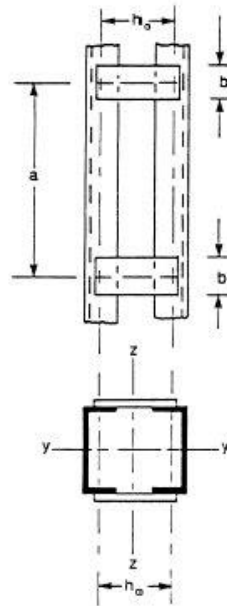
$$J_{\text{eff}} = 2 (h_0 / 2)^2 A_{\text{ch}} + 2 \mu_{\text{eff}} J_{\text{ch}} = 0,5 h_0^2 A_{\text{ch}} + 2 \mu_{\text{eff}} J_{\text{ch}}$$

Photo: Author



$$\mu_{\text{eff}} = 1$$

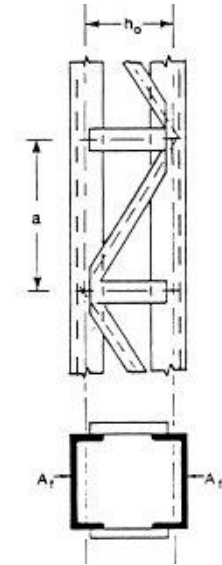
(rigid connection on each point of chord)



$$0 \leq \mu_{\text{eff}} \leq 1$$

(rigid connection not in each point of chord, „average” h_0)

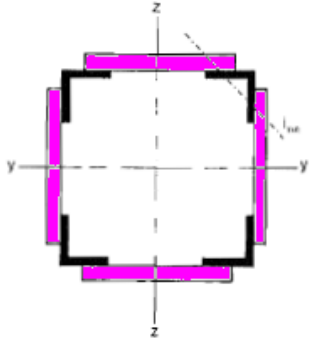
Photo: EN 1993-1-1 fig 6.7



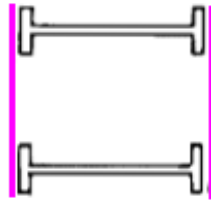
$$\mu_{\text{eff}} = 0$$

(rigid connection not in each point of chord, „large” h_0)

Photo: Author



$n = 4$



$n = 2$

n - number of battens / laces planes

$$J = 0,5 h_0^2 A_{ch} + 2 J_{ch}$$

$$\lambda = \mu L / i_0$$

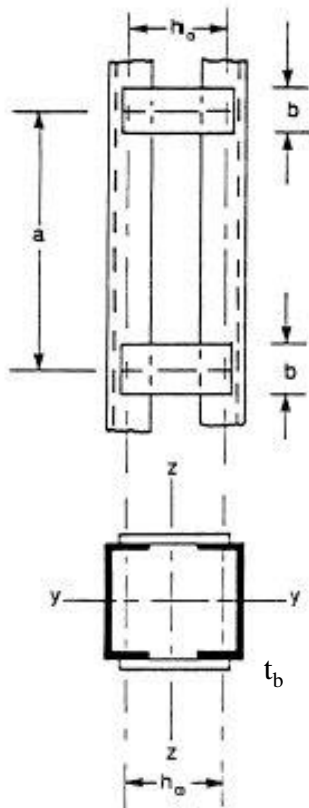
$$i_0 = \sqrt{[J / (2 A_{ch})]}$$

$$J_{eff} = 0,5 h_0^2 A_{ch} + 2 \mu_{eff} J_{ch}$$

λ	μ_{eff}
≥ 150	0
75 - 150	$2 - \lambda / 75$
≤ 75	1,0

EN 1993-1-1 tab. 6.8

Photo: EN 1993-1-1 fig 6.7



Shear stiffness

Battened column:

$$S_V = \min \left\{ 24 \Xi / [1 + 2 J_{ch} h_0 / (n J_b a)] \quad ; \quad 2 \pi \Xi \right\}$$

$$\Xi = E J_{ch} / a^2$$

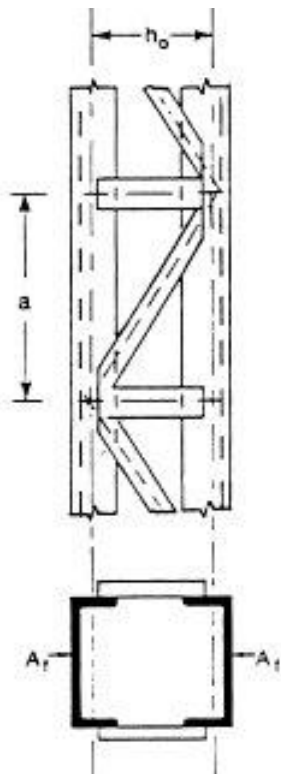
Effective moment of inertia:

$$J_{eff} = 0,5 h_0^2 A_{ch} + 2 \mu_{eff} J_{ch}$$

A_{ch} , J_{ch} – geometrical characteristics for one chord

J_b - moment of inertia for cross-section of batten

$$J_b = b^3 t_b / 12$$



Laced column: $S_V =$

$\Lambda / 2$	Λ	$\Lambda / [1 + A_d h_0^3 / A_v d^3)]$

Photo: EN 1993-1-1 fig 6.9

$$\Lambda = n E A_d a h_0^2 / d^3$$

Effective moment of inertia:

$$J_{\text{eff}} = 0,5 h_0^2 A_{\text{ch}}$$

A_{ch} , J_{ch} – geometrical characteristics for one chord

A_d , A_v – area of cross-sections of laces

Additional information from literature: $S_V =$

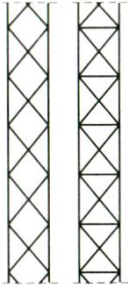
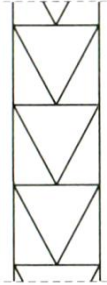
	
2Λ	$\Lambda / 2 [1 + A_d h_0^3 / 8 A_v d^3)]$

Photo: Sz. Pałkowski, Konstrukcje
stalowe, PWN Warszawa 2010

$$\Lambda = n E A_d a h_0^2 / d^3$$

Effective moment of inertia:

$$J_{\text{eff}} = 0,5 h_0^2 A_{\text{ch}}$$

The same symbols as previous

Effect of initial imperfections e_0 (the same for laced columns and battened columns) for axial forces in chords:

$$N_{ch, Ed} = N_{Ed} / 2 + M_{Ed}^{II} z_s A_{ch} / J_{eff}$$

$$M_{Ed}^{II} = (N_{Ed} e_0 + M_{z, Ed}) / [1 - (N_{Ed} / N_{cr}) - (N_{Ed} / S_V)]$$

$$e_0 = L / 500$$

$$N_{cr} = \pi^2 E J_{eff} / (\mu L)^2$$

Number of modules into which the bar is divided by battens ≥ 3

Equal length of modules

Recommended odd number of modules

EN 1993-1-1 6.4.1

There is inconsequence in Eurocode: according EN 1993-1-1 p.6.4.1.(1) - general information - only compressed members can be calculated according to presented procedure. In such situation, bending moment comes only from initial imperfection (M_{Ed}^{II}). On the other hand, in EN 1993-1-1 p.6.4.1.(6), bending moment from external actions is analysed common with bending moment from imperfections.

Influence of initial imperfection is calculated based on model for **one-span simple-supported** member. In Eurocode is no information, what about other type of support of column (for example cantilever as for crane supporting structure). No clear guidance in literature.

Imperfections for one-span simple-supported member make big impact on local cross-sectional forces. Secondary bending moment and secondary shear force, come from imperfection, have sinusoidal or cosinusoidal distribution along column. Effect of imperfection is multiplied by effects of instability (N_{cr}) and shear stiffness (S_V).

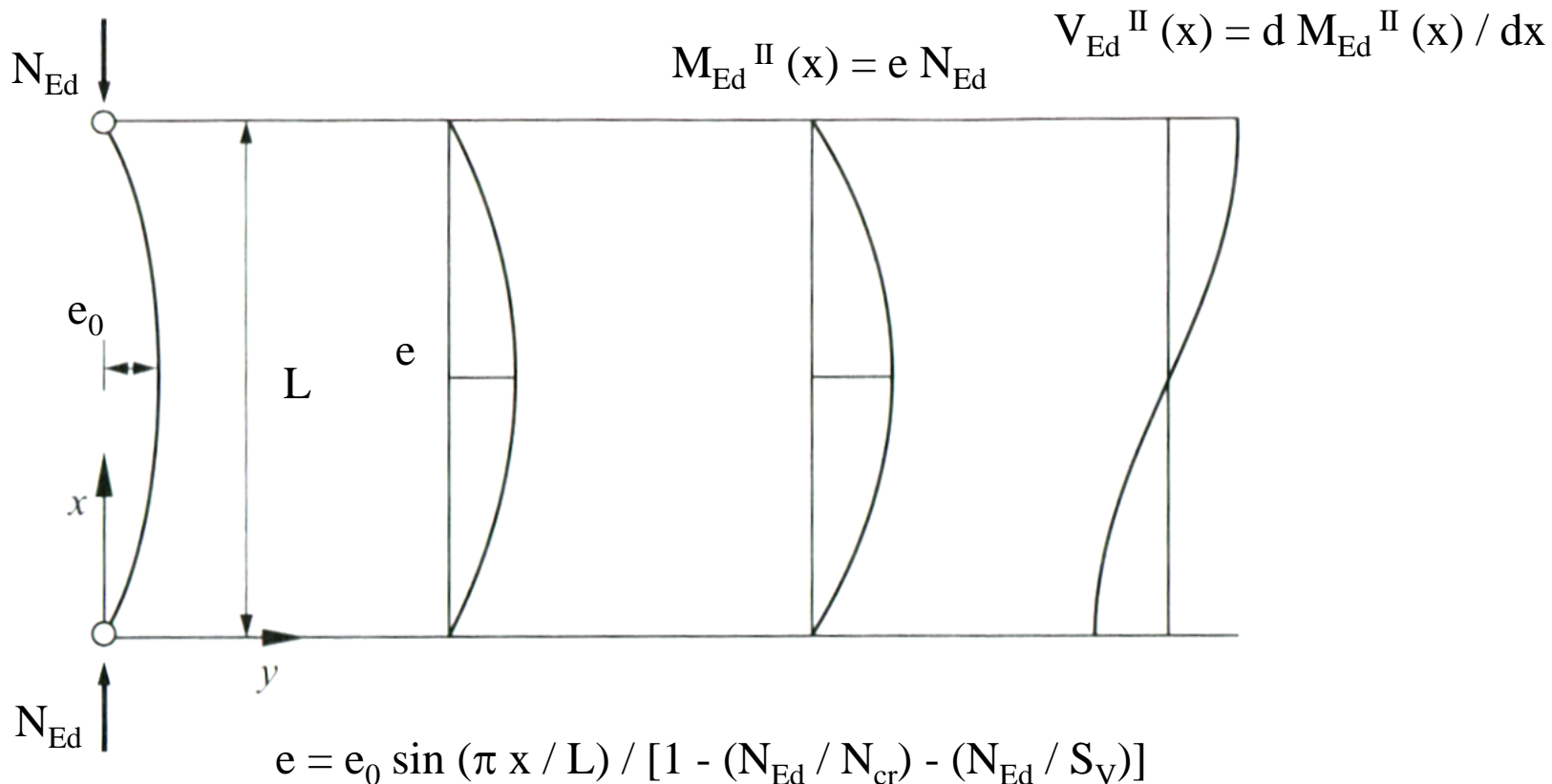
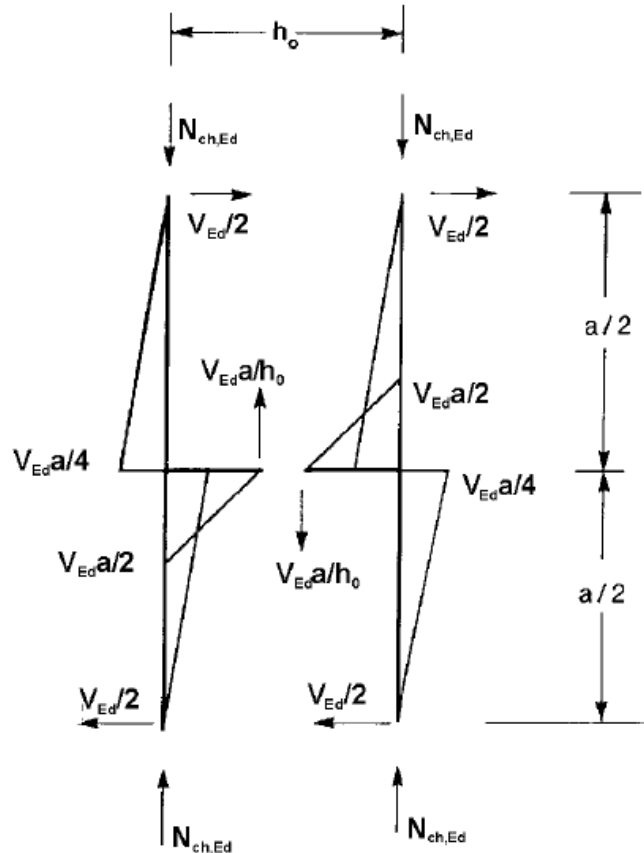


Photo: Sz. Pałkowski, Konstrukcje stalowe, PWN Warszawa 2010

Local cross-sectional forces in chords and battens



$$V_{Ed, \max} = \pi M_{Ed}^{\text{II}} / (n L)$$

$$h_0 = 2 z_s$$

For chord:

$$V_{ch, Ed} = V_{Ed}^{\text{II}} / 2$$

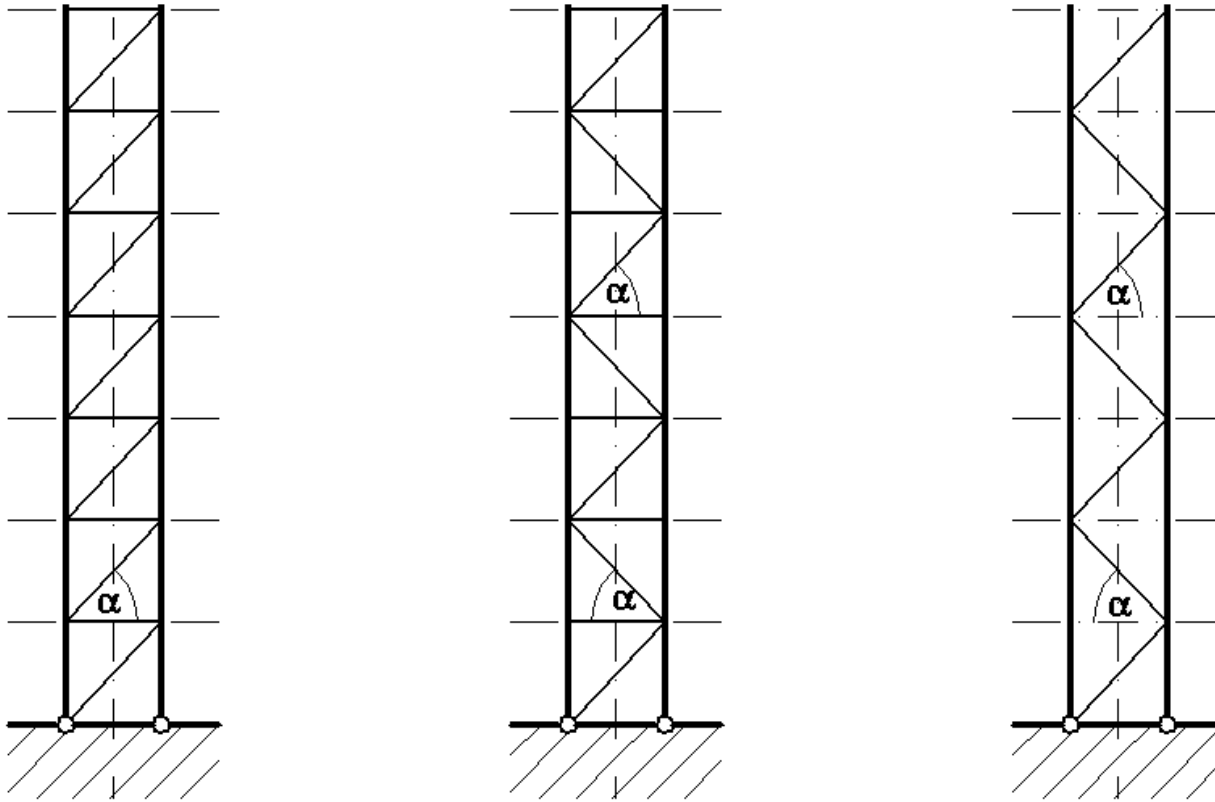
$$M_{ch, Ed} = a V_{Ed}^{\text{II}} / 4$$

For batten:

$$V_{b, Ed} = V_{Ed}^{\text{II}} a / (2 h_0)$$

$$M_{b, Ed} = a V_{Ed}^{\text{II}} / 2$$

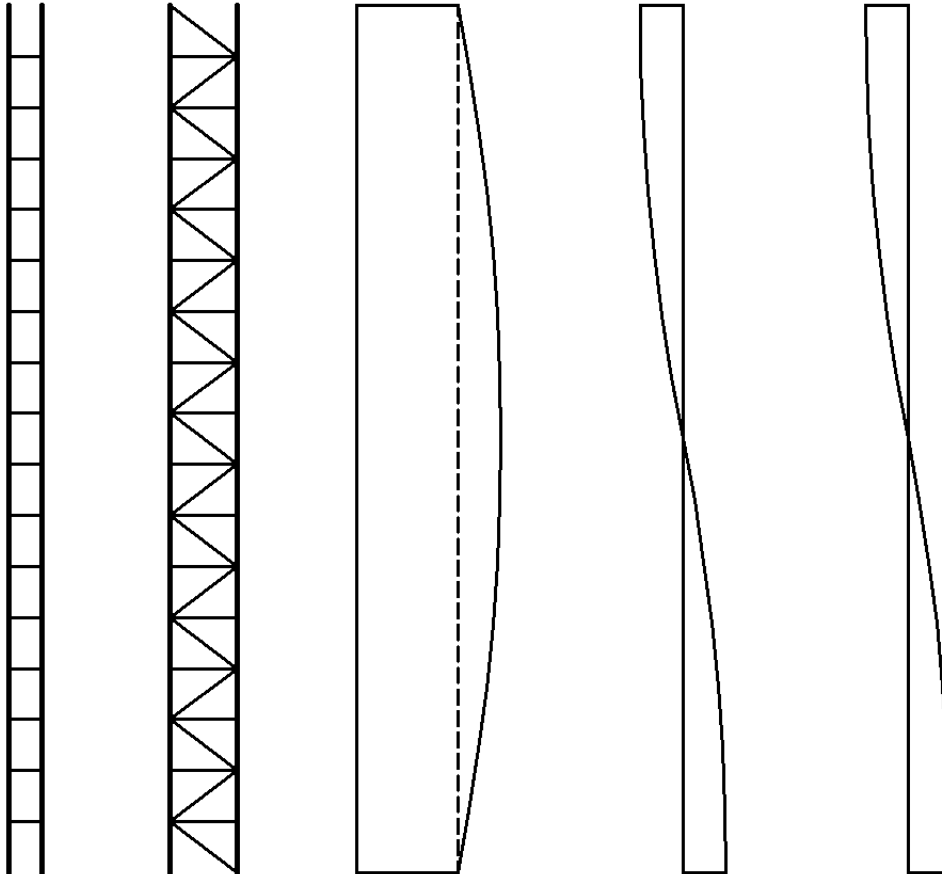
Photo: EN 1993-1-1 fig 6.11



For laces:

$$V_{Ed}^{II} = \pi M_{Ed} / (n L)$$

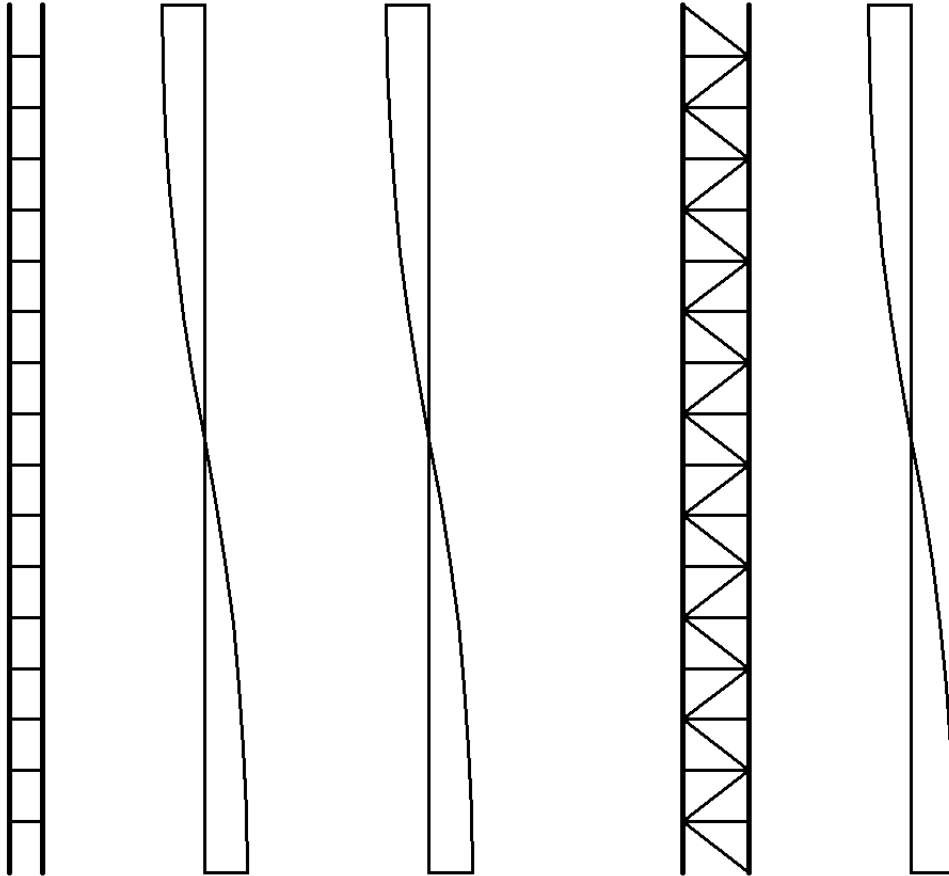
$$N_{l, Ed} = V_{Ed}^{II} / \cos \alpha = \pi M_{Ed} / (n L \cos \alpha)$$

$N_{ch, Ed}$ $M_{ch, Ed}$ $V_{ch, Ed}$ 

In case of chords, two points are important for calculations:

- In half of length: max $N_{ch, Ed}$, 0 for $M_{ch, Ed}$ and $V_{ch, Ed}$
- For top and bottom: max for $M_{ch, Ed}$ and $V_{ch, Ed}$, smaller one value of $N_{ch, Ed}$

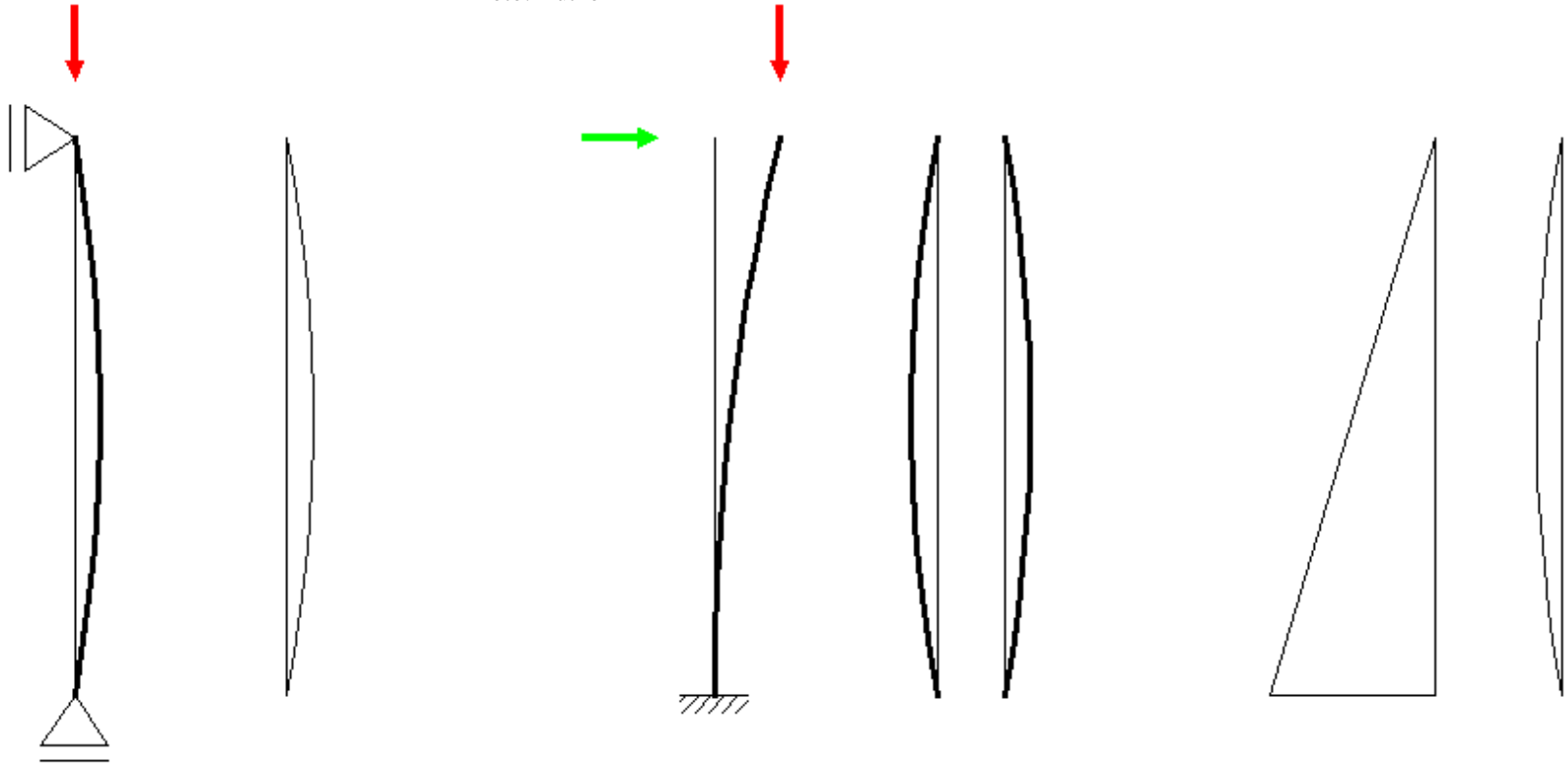
Photo: Author

$M_{b, Ed}$ $V_{b, Ed}$ $N_{l, Ed}$ 

For battens and laces, max values of forces are always on top and bottom of column.

Photo: Author

Photo: Author



Initial imperfection makes bending moment for simple-span column with axial force only.

For cantilever more important is bending moment from **horizontal force**. Bending moment from initial imperfection is much smaller.

In such case, max values of cross-sectional forces are located in bottom end of cantilever.

N_{Ed} – global axial force at top end of column

M_{Ed} – global bending moment at bottom end of column

$$S_1 = 1 - (N_{Ed} / N_{cr}) - (N_{Ed} / S_V)$$

$$N_{ch, Ed, max} = N_{Ed} / 2 + M_{Ed} z_s A_{ch} / (S_1 J_{eff})$$

$$V_{ch, Ed, max} = (\pi N_{Ed} e_0 + M_{Ed}) / (2 L n S_1)$$

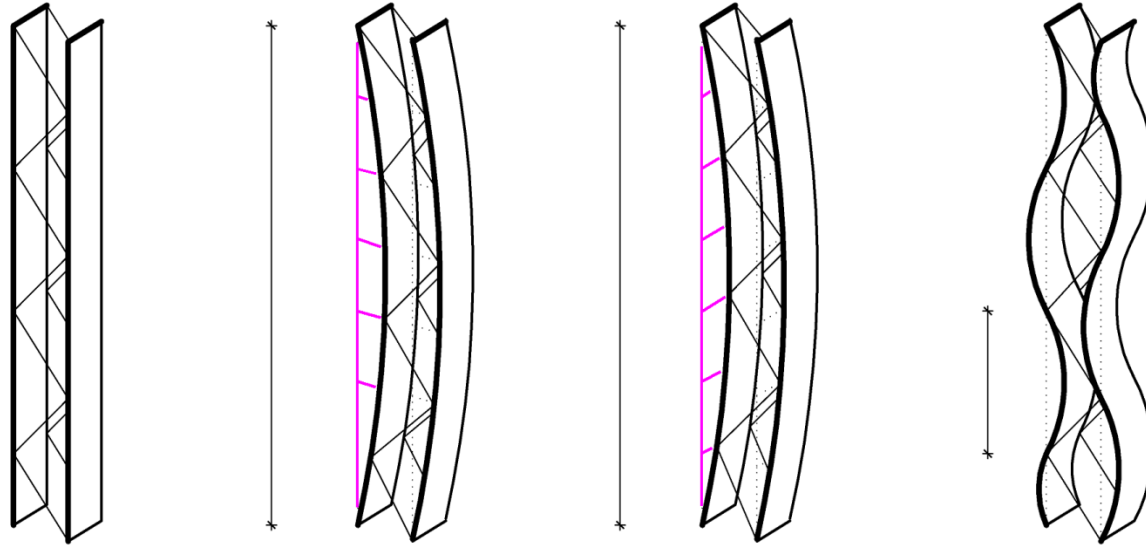
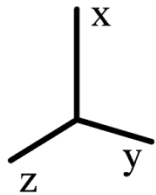
$$M_{ch, Ed, max} = a (\pi N_{Ed} e_0 + M_{Ed}) / (4 L n S_1)$$

$$V_{b, Ed, max} = a (\pi N_{Ed} e_0 + M_{Ed}) / (2 L h_0 n S_1)$$

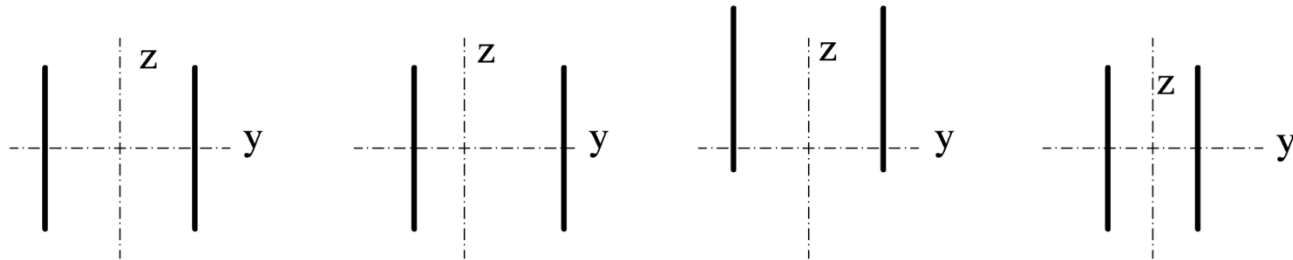
$$M_{b, Ed, max} = a (\pi N_{Ed} e_0 + M_{Ed}) / (2 L n S_1)$$

$$N_{l, Ed} = (\pi N_{Ed} e_0 + M_{Ed}) / (L n S_1 \cos \alpha)$$

Variuos modes of instability must be taken into consideration:



Global about material axis



Global about immaterial axis

Local

Photo: Author

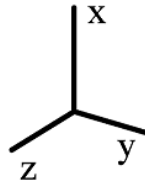
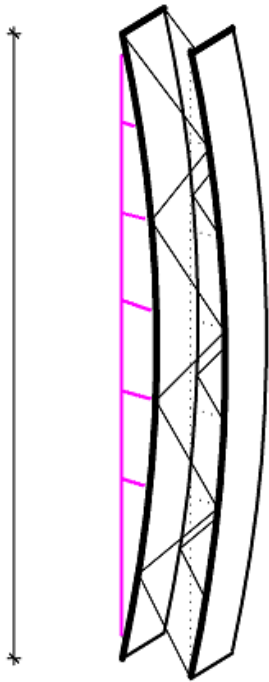
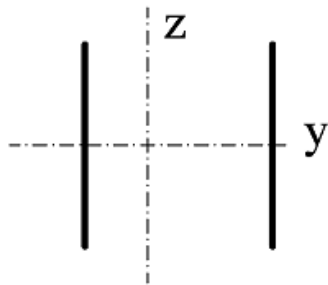


Photo: Author

Flexural buckling about axis z - immaterial (goes out of material of chords):

- No imperfection;
- Global axial force N_{Ed} ;
- Critical length comes from total length of column and type of supports;
- Moment of inertia = effective moment of inertia;



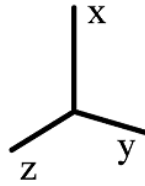
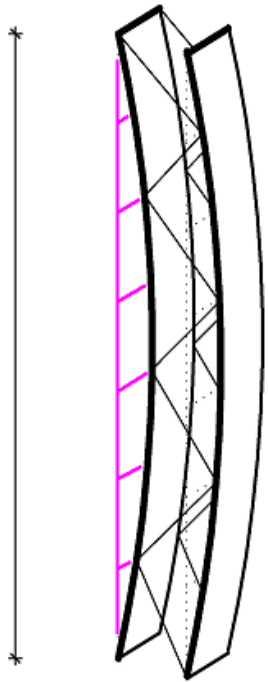
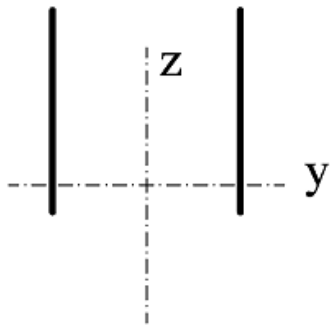


Photo: Author

Flexural buckling about axis y - material (goes through material of chords):

- No imperfection;
- Global axial force N_{Ed} ;
- Critical length comes from total length of column and type of supports;
- Moment of inertia = $2 \cdot$ moment of inertia for one chord;



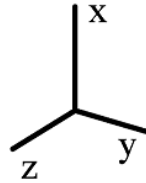
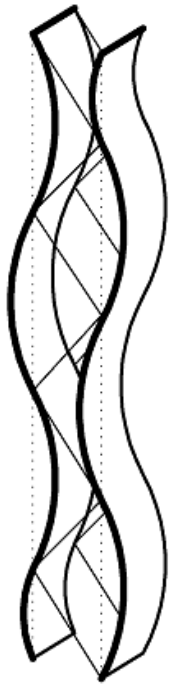
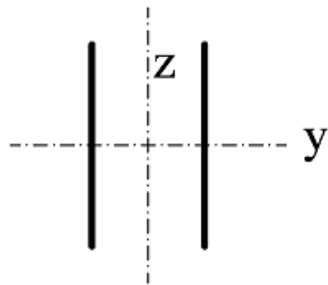


Photo: Author

Flexural buckling about own axis y_1 of one chord:

- Bow imperfection;
- Local axial force in one chord $N_{ch, Ed}$;
- Secondary bending moment and shear force in chord, come from imperfection;
- Critical length = distance between battens or laces;
- Moment of inertia = moment of inertia for one chord;



Battens and laces



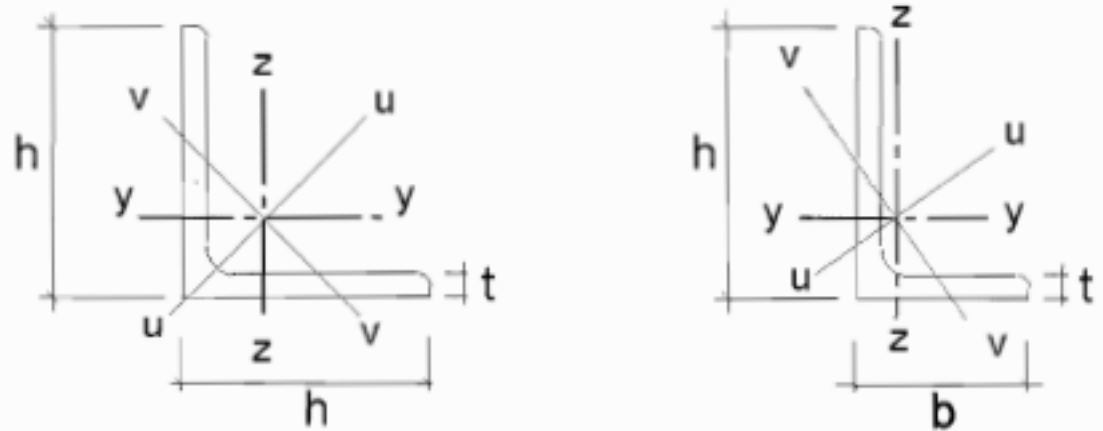
Photo: pks.p.lodz.pl

Bending moment and shear force exist in batten. Rectangular cross-section of the most popular of batten (rectangular plate) is not presented in EN 1993-1-1 tab. 5, this means we don't know, what is class of cross-section and what is resistance for bending moment.

According to literature, calculation for batten should be made in elastic range (elastic sectional modulus as base for $M_{b, Re}$). Interaction for bending moment and shear force should be calculated as effective stress from H-M-H theorem.



Photo: EN 1993-1-1 fig. 1.1



Laces are very often made on L-section. L-section has one axis of symmetry or no axis of symmetry. Four types of instability under axial force must be analysed: flexural buckling about u-u, flexural buckling about v-v, torsional buckling and flexural-torsional buckling.

Photo: Hasheela P., Behaviour of Single Laced Columns versus Double Laced Columns, Faculty of Engineering and Built Environment, University of the Witwatersrand, Johannesburg 2013

$$\chi = \min(\chi_y ; \chi_z ; \chi_T ; \chi_{z,T})$$

Flexural buckling, axis u $N_{cr, u} = \pi^2 E J_u / (\mu_u l_{0u})^2$

Flexural buckling, axis v $N_{cr, v} = \pi^2 E J_v / (\mu_v l_{0v})^2$

Torsional buckling $N_{cr, T} = [\pi^2 E J_w / (\mu_w l_{0w})^2 + G J_t] / i_s^2$

Flexural-torsional buckling $N_{cr, z-T} = \{N_{cr, v} + N_{cr, T} - \sqrt{[(N_{cr, v} + N_{cr, T})^2 - 4 N_{cr, v} N_{cr, T}]}\} / (2 \xi)$

$$\xi = 1 - (\mu z_s^2 / i_s^2)$$

$$\mu = \min [\sqrt{(\mu_z / \mu_T)} ; \sqrt{(\mu_T / \mu_z)}]$$

$$i_0 = \sqrt{(i_u^2 + i_v^2)}$$

$$i_s = \sqrt{(i_0^2 + z_s^2)}$$

z_s - distance between centre of gravity and shear centre ($z_s \geq 0$)

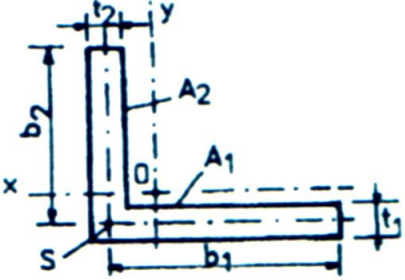
J_u, J_v – moments of inertia

i_u, i_v – radius of inertia

E, G – Young and Kirchhoff modulus

J_t - torsion constant

J_w - warping constant

Przekrój	Cechy geometryczne
	$y_s = e_y - \frac{t}{2}$ $I_\omega = \frac{A_1^3 + A_2^3}{36}$ $I_r = \frac{1}{3} (b_1 I_1^3 + b_2 I_2^3)$
Oznaczenia:	
<p>O — środek ciężkości</p> <p>S — środek ścinania</p> <p>I_1 I_2 (I_3) — momenty bezwładności pól (środnika) względem osi symetrii</p> <p>I_y — moment bezwładności figury względem osi symetrii</p>	

Ptoto: J. Żmuda, „Podstawy projektowania konstrukcji metalowych”, TiT Opole 1992

Axis of L-section doesn't lie on half of width of arm. It is strongly displaced towards connection of both arms. Length of welds must be various to avoid difference between centre of gravity for cross-section and centre of gravity for welds. Such difference – if exists- makes secondary bending moments in joints as effect of eccentricity.

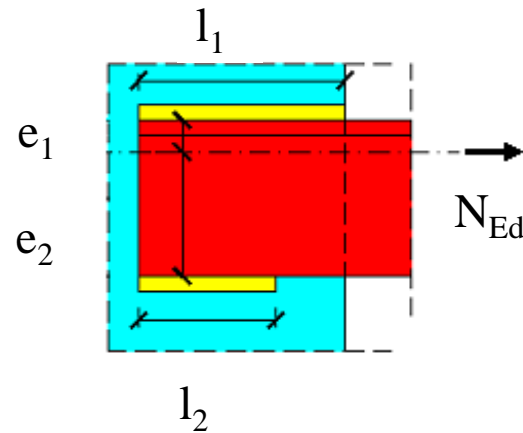


Photo: Author

$$e_1 l_1 = e_2 l_2$$

Interaction of lateral buckling and flexural buckling must be analysed for member under bending moment and compressive axial force. According to literature, multi-chords members can't be calculated, based on EN 1993-1-1 6.3.3.

~~$$N_{Ed} / (\chi_y N_{Rk} / \gamma_{M1}) + k_{yy} (M_{y, Ed} + \Delta M_{y, Ed}) / (\chi_{LT} M_{y, Rk} / \gamma_{M1}) + k_{yz} (M_{z, Ed} + \Delta M_{z, Ed}) / (M_{z, Rk} / \gamma_{M1}) \leq 1,0$$

EN 1993-1-1 (6.61), (6.62)

$$N_{Ed} / (\chi_z N_{Rk} / \gamma_{M1}) + k_{zy} (M_{y, Ed} + \Delta M_{y, Ed}) / (\chi_{LT} M_{y, Rk} / \gamma_{M1}) + k_{zz} (M_{z, Ed} + \Delta M_{z, Ed}) / (M_{z, Rk} / \gamma_{M1}) \leq 1,0$$~~

This point is used rather for mono-chord (single-coherent) members. For lateral buckling important is specific geometrical characteristic: warping constant. Calculation of warping constant is possible for single-coherent cross-section. It could be only roughly evaluated for multi-chord cross-section as mathematical value, with uncertain physical interpretation.

It is suggested, than for such type of member should be calculated rather according to EN 1993-1-1 6.3.4. This point is dedicated to *single members with mono symmetric cross sections, **built-up** or not.*

Formula according EN 1993-1-1 6.3.4. for interaction between flexural buckling and lateral buckling is as follow (EN 1993-1-1 (6.64)):

$$\chi_{op} \alpha_{ult, k} / \gamma_{M1} \geq 1,0 \quad \text{Yes, it's not mistake: } \geq 1,0$$

$\chi_{op} = \chi(\lambda_{op})$ calculated for lateral buckling or flexural buckling

$$\lambda_{op} = \sqrt{(\alpha_{ult, k} / \alpha_{cr, op})}$$

$\alpha_{ult, k}$ - the minimum load amplifier of the design loads to reach the characteristic resistance of the most critical cross section of the structural component considering its in plane behaviour without taking lateral or lateral torsional buckling into account however accounting for all effects due to in plane geometrical deformation and imperfections, global and local, where relevant;

$\alpha_{cr, op}$ - the minimum amplifier for the in plane design loads to reach the elastic critical load of the structural component with regards to lateral or lateral buckling without accounting for in plane flexural buckling;

Values of $\alpha_{ult, k}$ and $\alpha_{cr, op}$ should be taken from numerical analysis.

According to EN 1993-1-1 6.3.4.(4) formula could be simplified to form:

$$1 / \alpha_{ult, k} = N_{Ed} / N_{Rk} + M_{y, Ed} / M_{y, Rk}$$

Value of $\alpha_{ult, k}$ is calculated for each important cross-section. Final value is min from each cross-sections.

According to EN 1993-1-1 (5.1), $\alpha_{cr, op}$ can be presented as:

$$\alpha_{cr, op} = N_{cr} / N_{Ed}$$

In analogy to $\alpha_{ult, k}$, $\alpha_{cr, op}$ is the smallest value for each possible form of instability.

Algorithm of analysis:

- General calculation

$$N_{Ed}, M_{Ed}, V_{Ed}, S_v, J_{eff}, e_0, M_{Ed}^{II}, N_{ch, Ed}, M_{ch, Ed}, V_{ch, Ed}$$

- Resistance for cross-section of both chords at bottom end of column

$$N_{Ed}, M_{Ed}, V_{Ed}, \alpha_{ult, k, both}$$

- Resistance for cross-section of one chord at the bottom of column

$$N_{ch, Ed}, M_{ch, Ed}, V_{ch, Ed}, \alpha_{ult, k, one}$$

- $\alpha_{ult, k} = \min (\alpha_{ult, k, both} ; \alpha_{ult, k, one})$

- Global flexural stability about material axis

cross-section of both chords , J_{material} , $L_{\text{cr}} = H$

$$\alpha_{\text{cr, op, 1}} = N_{\text{cr, mat}} / N_{\text{Ed}}$$

- Global flexural stability about immaterial axis

cross-section of both chords , J_{eff} , $L_{\text{cr}} = H$

$$\alpha_{\text{cr, op, 2}} = N_{\text{cr, immat}} / N_{\text{Ed}}$$

- Local stability of one chord at the end of column

I-section	C-section
cross-section of one chords , J_y , $L_{cr} = a$ $\alpha_{cr, op, 3} = N_{cr, y} / N_{ch, Ed}$	cross-section of one chords , J_y , $L_{cr} = a$ $\alpha_{cr, op, 3} = N_{cr, y} / N_{ch, Ed}$
cross-section of one chords , J_z , $L_{cr} = a$ $\alpha_{cr, op, 4} = N_{cr, z} / N_{ch, Ed}$	cross-section of one chords , J_z , $L_{cr} = a$ $\alpha_{cr, op, 4} = N_{cr, z} / N_{ch, Ed}$
	cross-section of one chords , J_w , J_t , $L_{cr} = a$ $\alpha_{cr, op, 5} = N_{cr, T} / N_{ch, Ed}$
	cross-section of one chords , J_z , J_w , J_t , $L_{cr} = a$ $\alpha_{cr, op, 5} = N_{cr, T-z} / N_{ch, Ed}$
local bending moment acts about weak axis → no lateral buckling	

- $\alpha_{cr, op} = \min (\alpha_{cr, op, 1} ; \alpha_{cr, op, 2} ; \dots)$
- $\lambda_{op} = \sqrt{(\alpha_{ult, k} / \alpha_{cr, op})}$
- $\chi_{op} = \chi(\lambda_{op})$ calculated for flexural buckling (no lateral buckling)
- $\chi_{op} \alpha_{ult, k} / \gamma_{M1} \geq 1,0$

- For battened column: resistance of baten

$$M_{b, Ed} , V_{b, Ed}$$

- For laced column: resistance and stability of lace

$$N_{l, Ed}$$

- Resistance of welds around lace or batten
- Deformation of column

Bases

There are two separated bases of column in case of laced or battened column.



Photo: quatronsteel.com

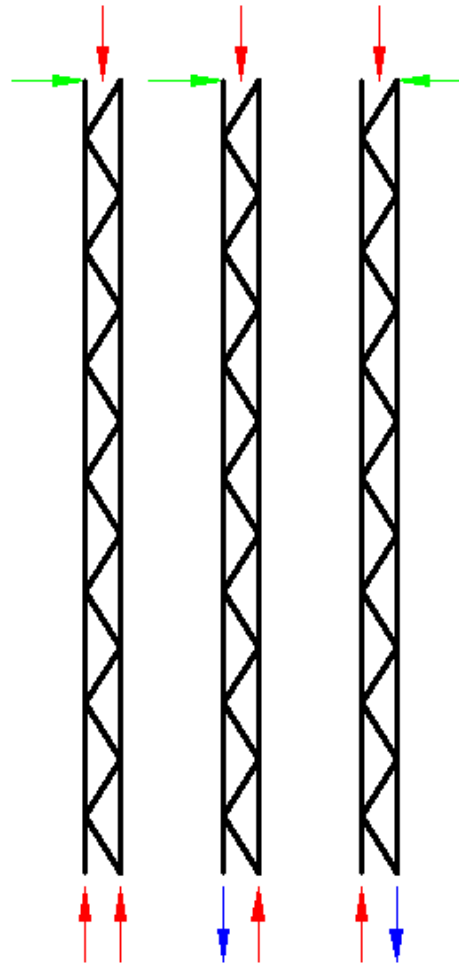


Photo: Author

There are only axial force and shear force in each base of column's chord.

There are different combinations of loads. It is possible, that there are various cases of reactions for various combinations. Resistance of bases should be calculated for two different direction of reaction (**contact steel-concrete** or **tension in anchor bolts and local bending base plate**).



Photo: inzynierbudownictwa.pl

There should be massive anchor system in case of tensile force in column's chord. First of all – base of column's chord must be able to resist of big value of force from anchor bolts.

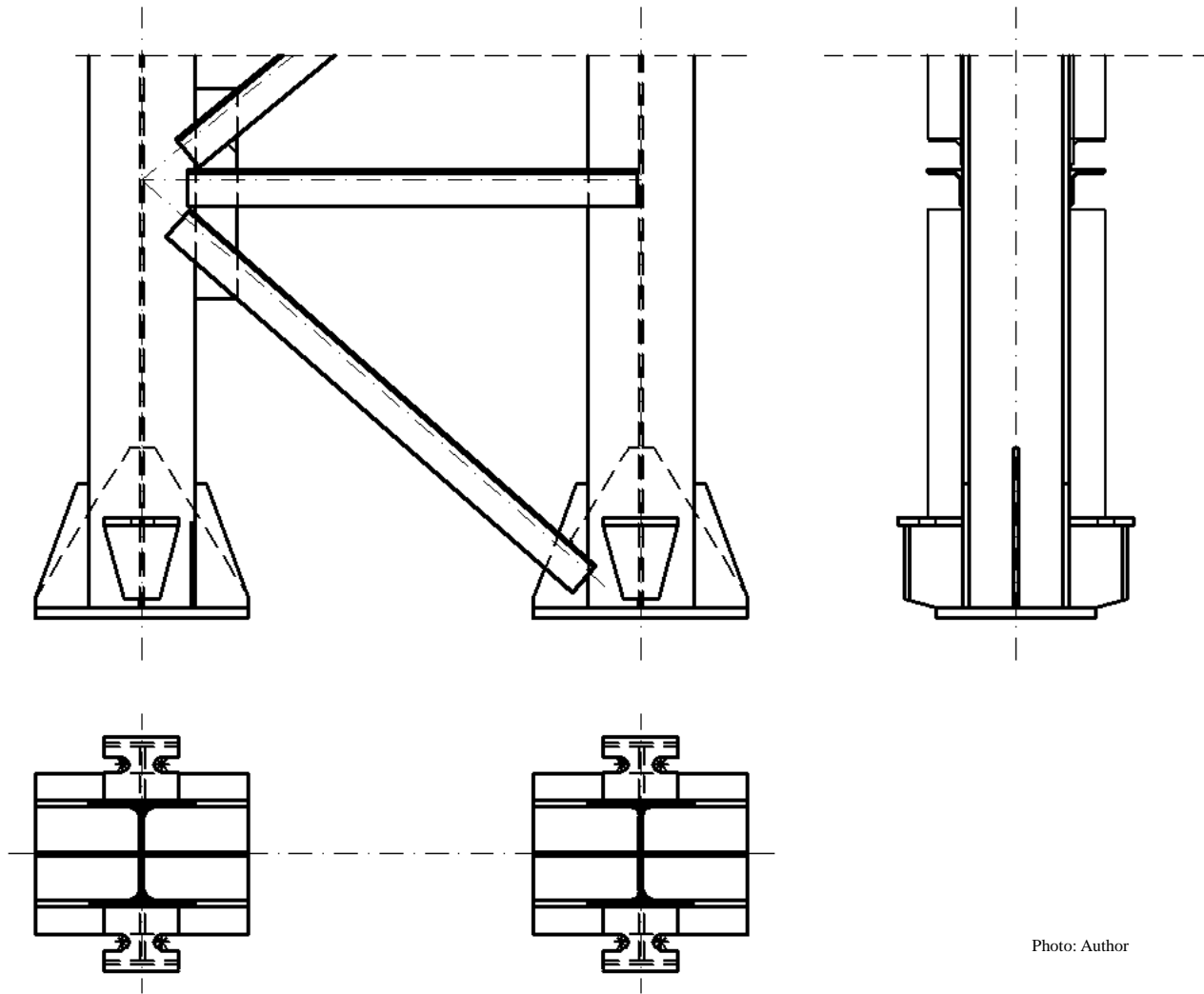


Photo: Author

Anchoring in the concrete foundation is solved in various ways. The easiest - for small forces - is the friction between anchor bolts and concrete and resistance of the extended tip of the bolts.

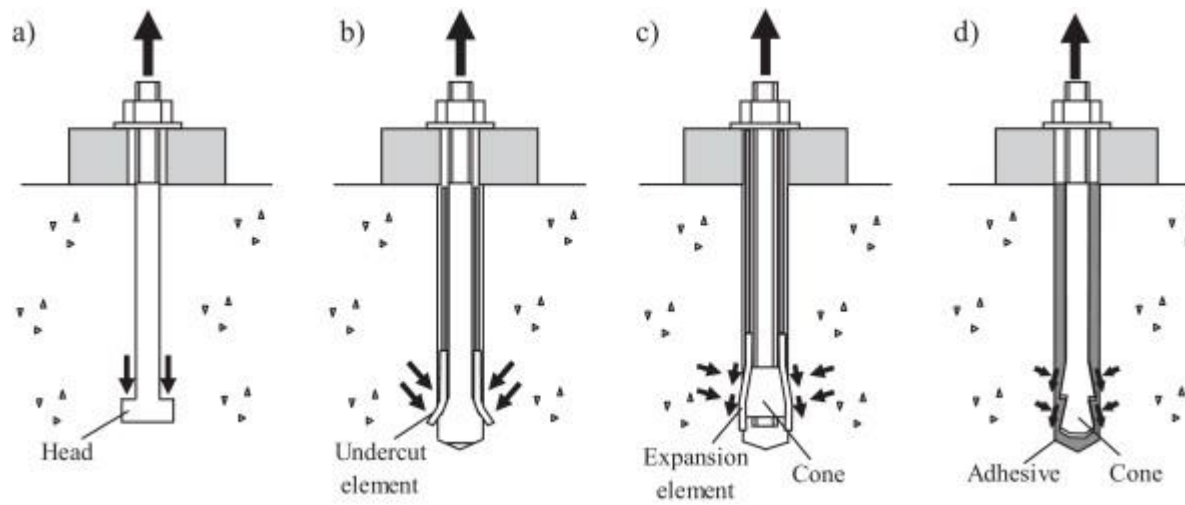


Photo: Post-installed concrete anchors in nuclear power plants: Performance and qualification, Ph. Mahrenholtz, R. Eligehausen Nuclear Engineering and Design 287 / 2015

There are used J-anchor bolts for big value of force. These anchors can be weld to the reinforced steel of base



Photo: civil-engg-world.blogspot.com



Photo: peikko.ca



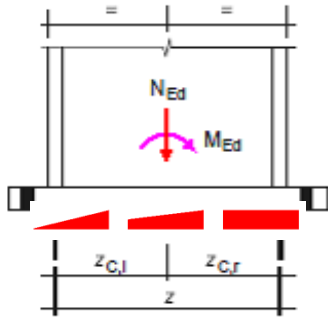
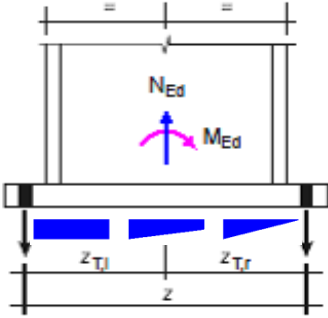
Photo: homemadetools.net



Photo: strongtie.com

The same, there can be used massive anchor assembly (retaining plates).

Resistance of bases are calculate constant value of stresses under base plate.

Loading	Lever arms	Resistance $M_{j, Rd}$	
<p>Left-C, Right-C, exampe:</p> <p>$M_{Ed} > 0$; $N_{Ed} < 0$</p> 	<p>$z = z_{C,l} + z_{C,r}$</p> <p>$e = M_{Ed} / N_{Ed}$</p>	<p>$N_{Ed} \leq 0$ $0 < e < z_{C,l}$</p>	<p>$N_{Ed} \leq 0$ $-z_{C,r} < e \leq 0$</p>
		<p>$\min [-z F_{C,l, Rd} / (1 + z_{C,r} / e)$ $-z F_{C,r, Rd} / (-1 + z_{C,l} / e)]$</p>	<p>$\min [-z F_{C,l, Rd} / (1 + z_{C,r} / e)$ $-z F_{C,r, Rd} / (-1 + z_{C,l} / e)]$</p>
<p>Left-T, Right-T, exampe:</p> <p>$M_{Ed} > 0$; $N_{Ed} > 0$</p> 	<p>$z = z_{T,l} + z_{T,r}$</p> <p>$e = M_{Ed} / N_{Ed}$</p>	<p>$N_{Ed} > 0$ $0 < e < z_{T,l}$</p>	<p>$N_{Ed} > 0$ $-z_{T,r} < e \leq 0$</p>
		<p>$\min [z F_{T,l, Rd} / (1 + z_{T,r} / e)$ $z F_{T,r, Rd} / (-1 + z_{T,l} / e)]$</p>	<p>$\min [z F_{T,l, Rd} / (1 + z_{T,r} / e)$ $z F_{T,r, Rd} / (-1 + z_{T,l} / e)]$</p>

EN 1993-1-8 tab. 6.7

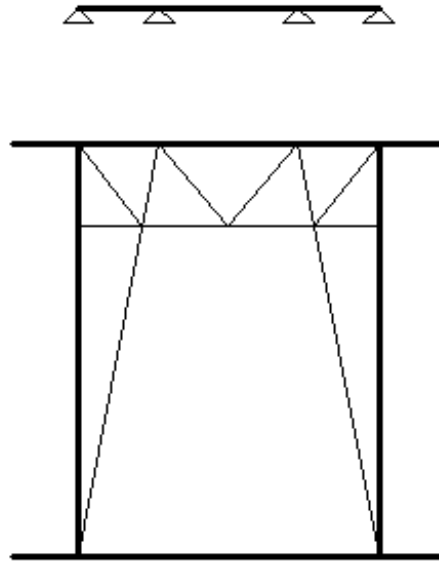
Bracings



Photo: konar.eu

Recommendation for wall / column bracings: no contact between bracings and run-beam.

Contact between bracings and run-beam \rightarrow multi-span run-beam.

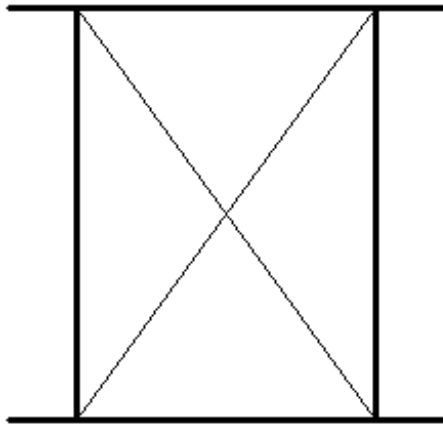


There is one-span run-beam recommended for fatigue calculations (#t / 17 - 18).

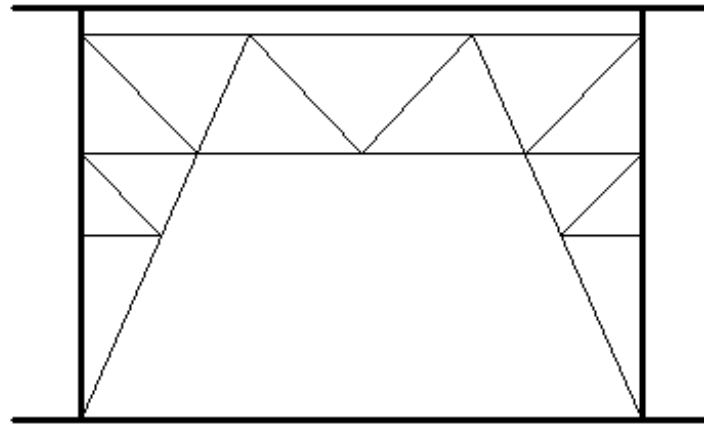
Additionally, on bracings act directly vertical loads from crane.

Photo: Author

Recommended shape of bracings



Distance between columns $\leq 6,0$ m



Distance between columns $> 6,0$ m

Photo: Author

Examination issues

Fatigue calculations for crane runbeams

Displacements and deformations of crane supporting structures

Laced column and battened column – similarities and differences

Surge connectors - podporowe elementy złączne
Butt weld - spoina czołowa
Filled weld - spoina pachwinowa
Notch - karb
Laced members - słup wielogałęziowy skratowany
Battened members - słup wielogałęziowy z przewiązkami
Closely spaced build-up members - pręt wielogałęziowy
Shear stiffness - sztywność postaciowa
Efficiency factor - wskaźnik efektywności

Thank you for attention

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