

Metal Structures

Lecture X

Bracing systems

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Introduction

Plate is stiff member of big resistance (in plane); it can bears even big loads. But without additional support in perpendicular direction is completely unstable (out of plane).



Photo: Author

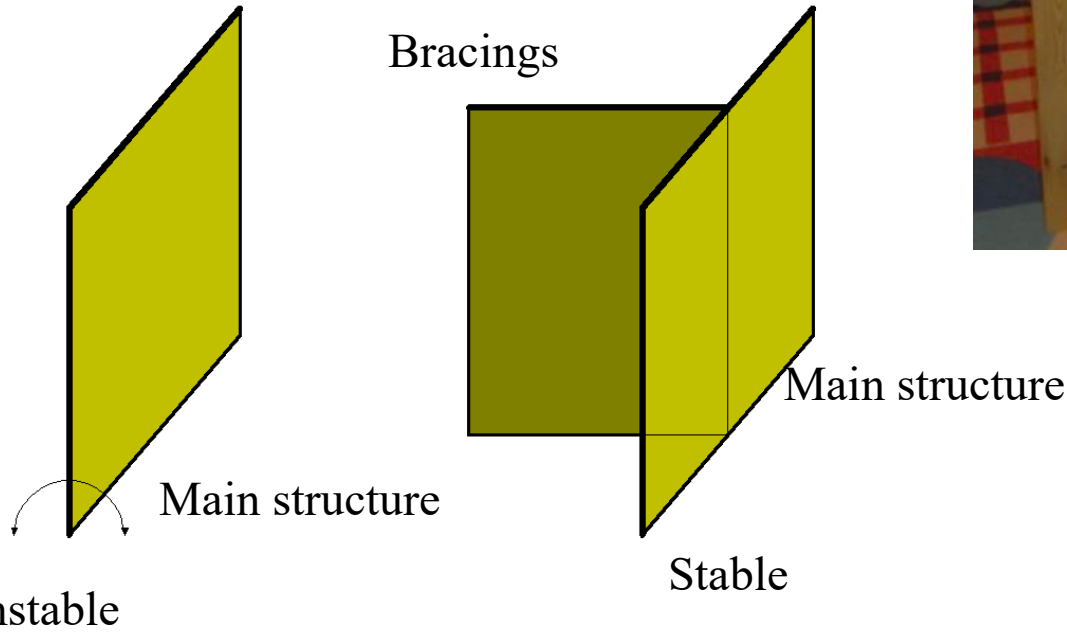




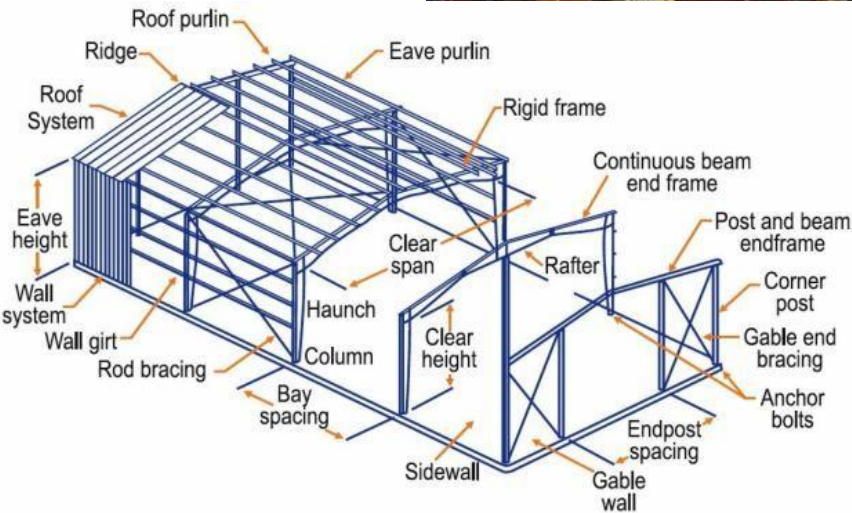
Photo: setrometalgroup.com



Photo: traskostal.pl.

Generally steel structures consist on flat frame (of big stiffness and big resistance in plane), but unstable in perpendicular direction. Additional sub-structures is needed: various types of bracings (in roof, in wall...).

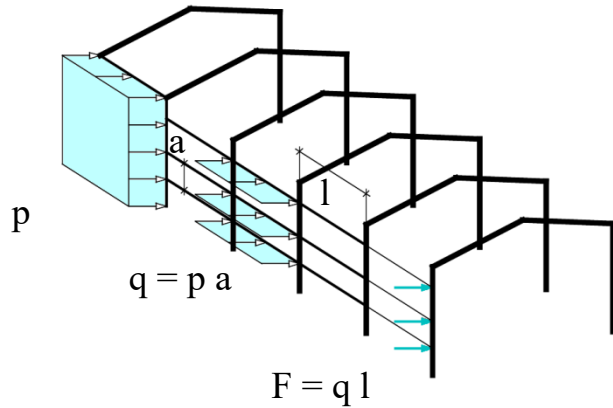
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Wall girts, purlins, roof bracings and side wall bracings make specific system for wind action. Front wall housing columns must be connected with purlins and roof bracings at one point. The same, girts on front and side walls.

Photo: greenterrahomes.com

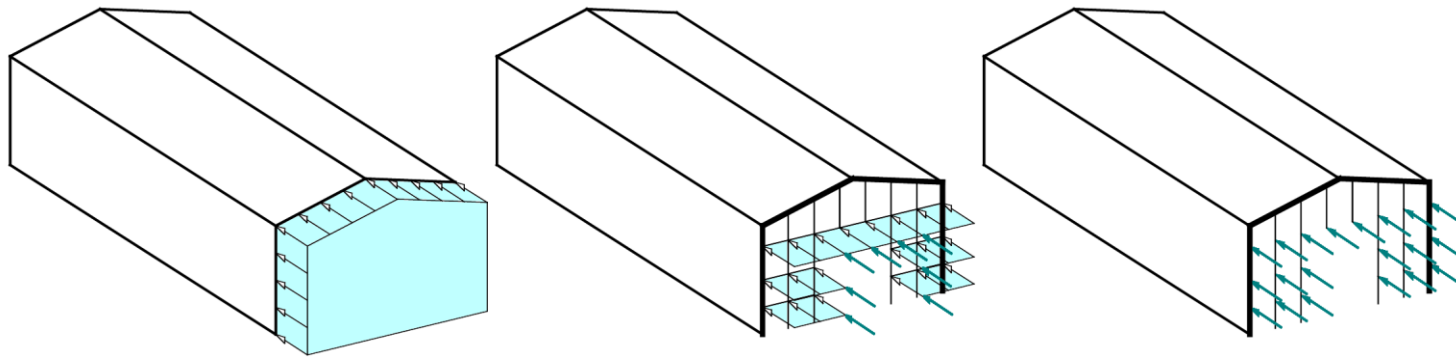
Photo: Author



Wind acts on housing (p , [kN / m^2]). Housing is support on wall girts; loads from housing act on girts as continuous loads (q , kN / m). Girts are under bending (mono- or bi-axial). Loads from girts act on main frames as forces, applied in points of connection girts - main columns.

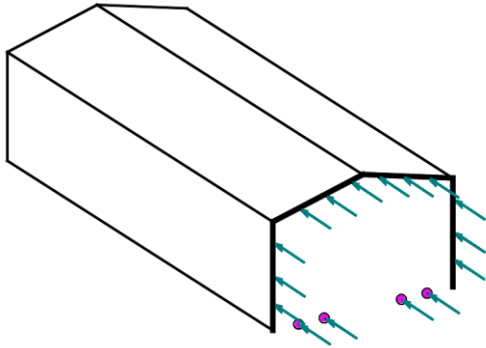
→ #8 / 45

Photo: Author

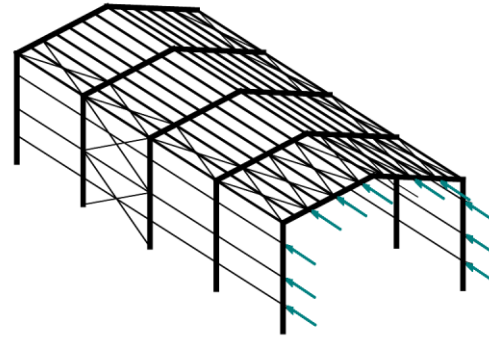


Wind acts on front wall: the same way of recalculation $p \rightarrow q \rightarrow F$. Forces are applied to main frames (perpendicular to theirs plane) and to housing columns. In case of doors (in front / side wall), wind action from door is applied to girts and housing columns around doors.

Photo: Author

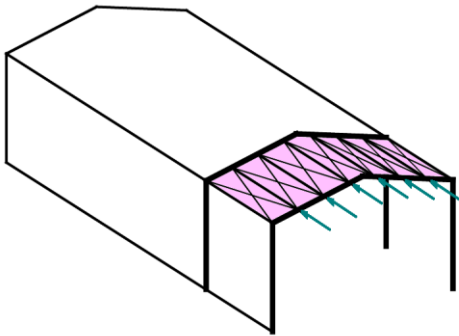


Loads from housing columns finally act on bases of housing column and main frames (main columns, roof girders), perpendicular to their planes. It potentially makes bi-axial bending in main frames.



Main frames are supported in perpendicular direction by bracings, purlins and side wall girts. It prevents bi-axial bending.

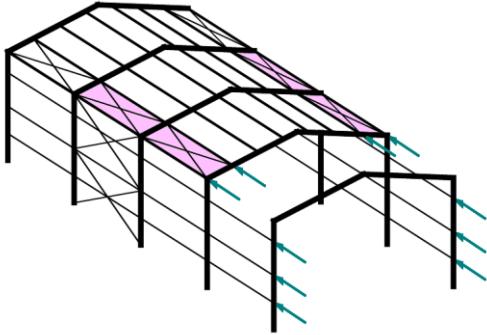
Photo: Author



Roof bracings and purlins make a horizontal truss. Roof girders are chords of the truss. The effect is that loads perpendicular to main frames make additional axial forces in roof girders. Additionally, there are axial forces in purlins.

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Photo: Author



Roof: loads are transported through longitudinal broof bracings.

Wall: loads are transported by side wall girts.

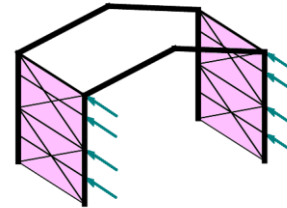


Photo: Author

Finally, loads act on vertical bracings on side walls: vertical trusses. Main columns are chords of truss.

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Reasons of application

The most important rule of bracings is to support structure in „perpendicular direction” to its „main plane”. In case of structure based on flat frames, „main plane” and „perpendicular direction” have intuiital character. In case of skeleton structure, bracings could be applied in both directions.

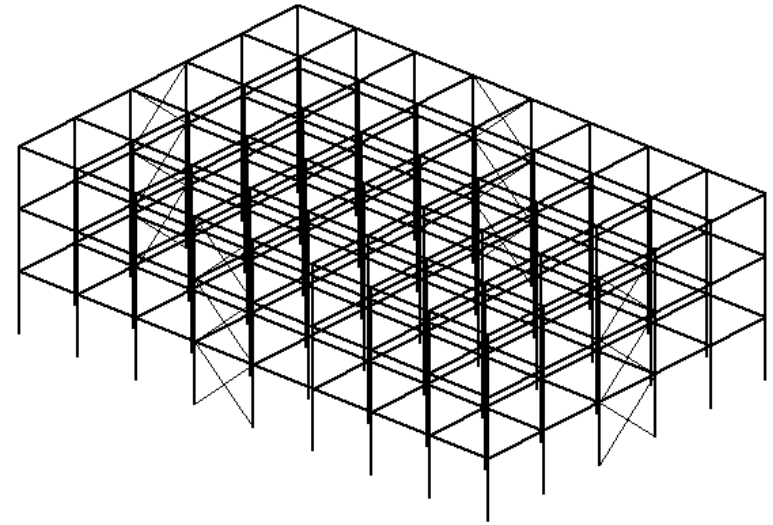
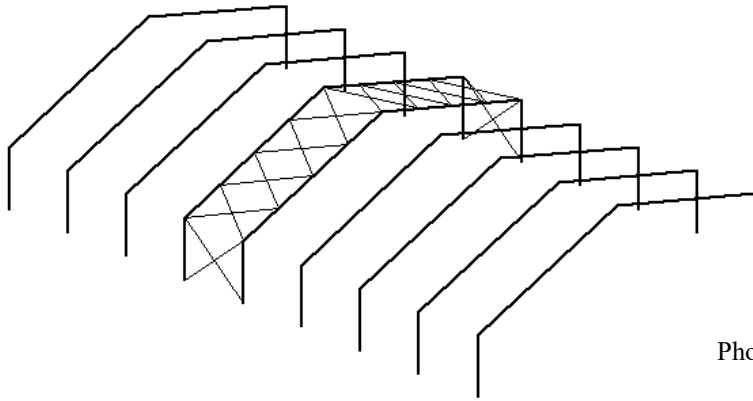


Photo: Author

„Support structure” have two meanings:

- creation supports for memebers in analysis of their lost stability (\rightarrow #t / 10 – 14);
- takeover loads applicated in perpendicular direction (\rightarrow #t / 15 – 17);

Example 1

C 300p

S235 $\rightarrow f_y = 235$ MPa

$L = 3,00$ m

$E = 210$ GPa

$G = 81$ GPa

$A = 52,5$ cm²

$J_y = 7640$ cm⁴

$J_z = 473$ cm⁴

$J_w = 66\,500$ cm⁶

$J_T = 33,9$ cm⁴

$a = 3,12$ cm

$e = 2,89$ cm

$i_y = 12,1$ cm

$i_z = 3,01$ cm

$y_s = a + e = 6,01$ cm

In this case:

$z_s = y_s = 6,01$ cm

$N_{Ed} = 650$ kN

I class of cross-section

\rightarrow #5 / 43

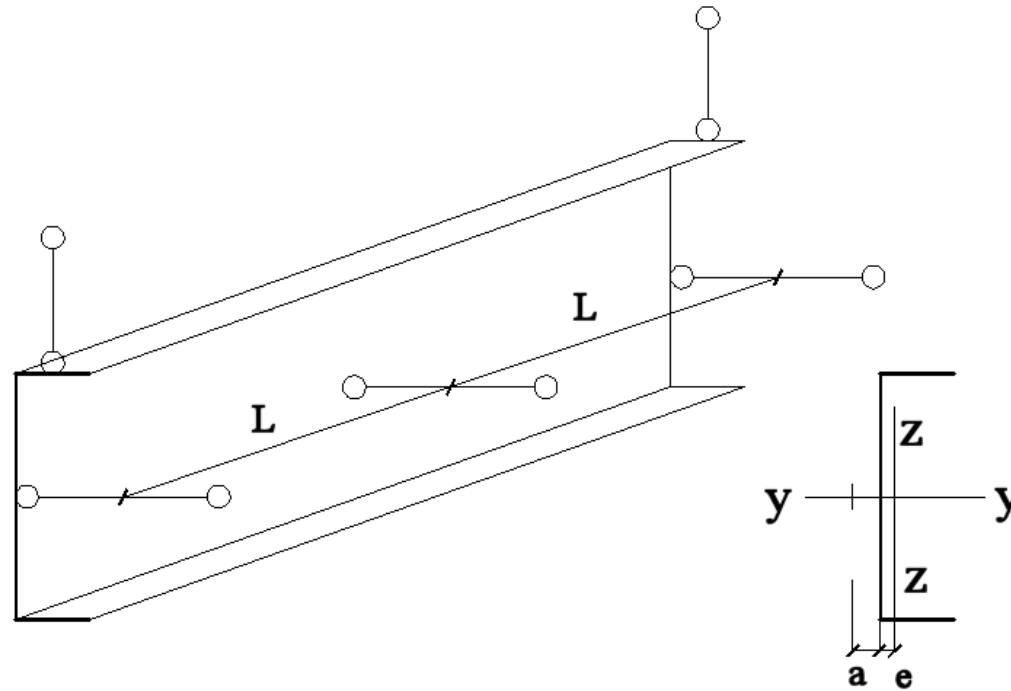
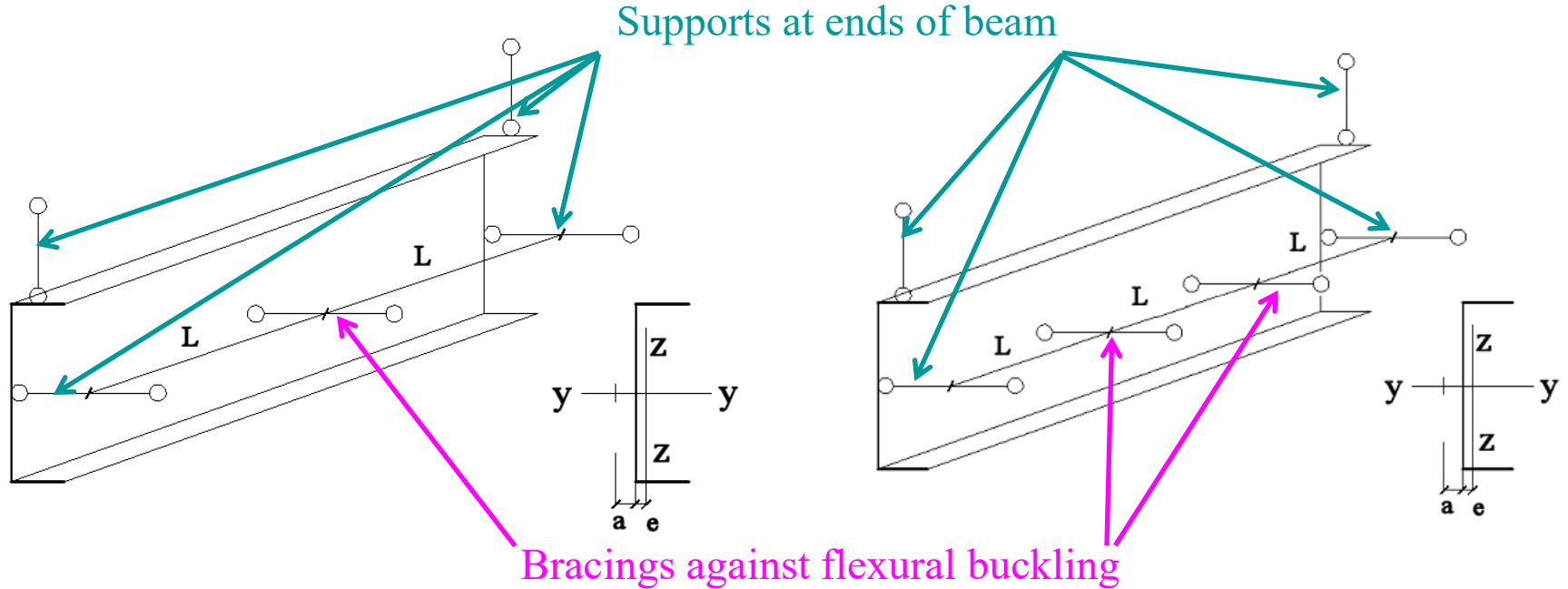


Photo: Author

Example presented in Lec #5 shows difference between two types of support for beam



Critical length for z-buckling:

$$\mu_z L_{0z} = 3,00 \text{ m}$$

$$\mu_z L_{0z} = 2,00 \text{ m}$$

Effort:

$$N_{Ed} / \chi A f_y = 1,186$$

$$N_{Ed} / \chi A f_y = 0,944$$

Flexural buckling of chords:

→ Des #1 / 54

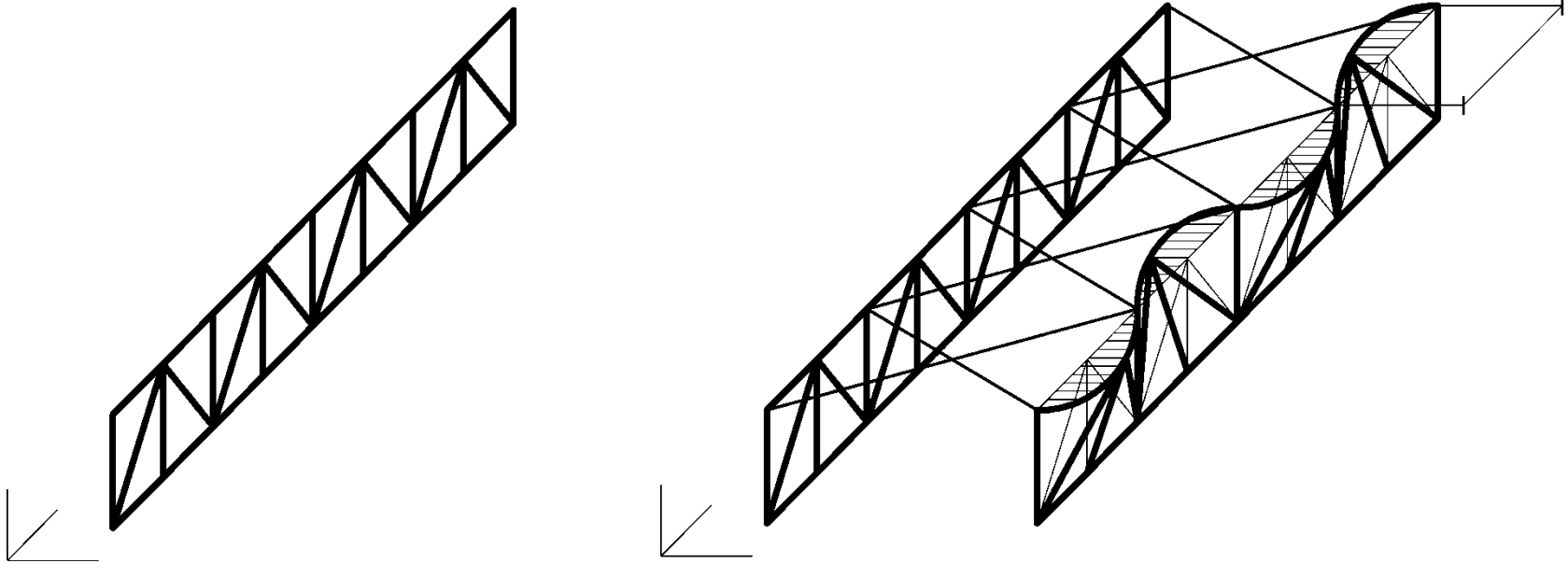


Photo: Author

Top chords in compression; buckling out of plane: distance between supports = distance between horizontal bracings

I-beam roof girder

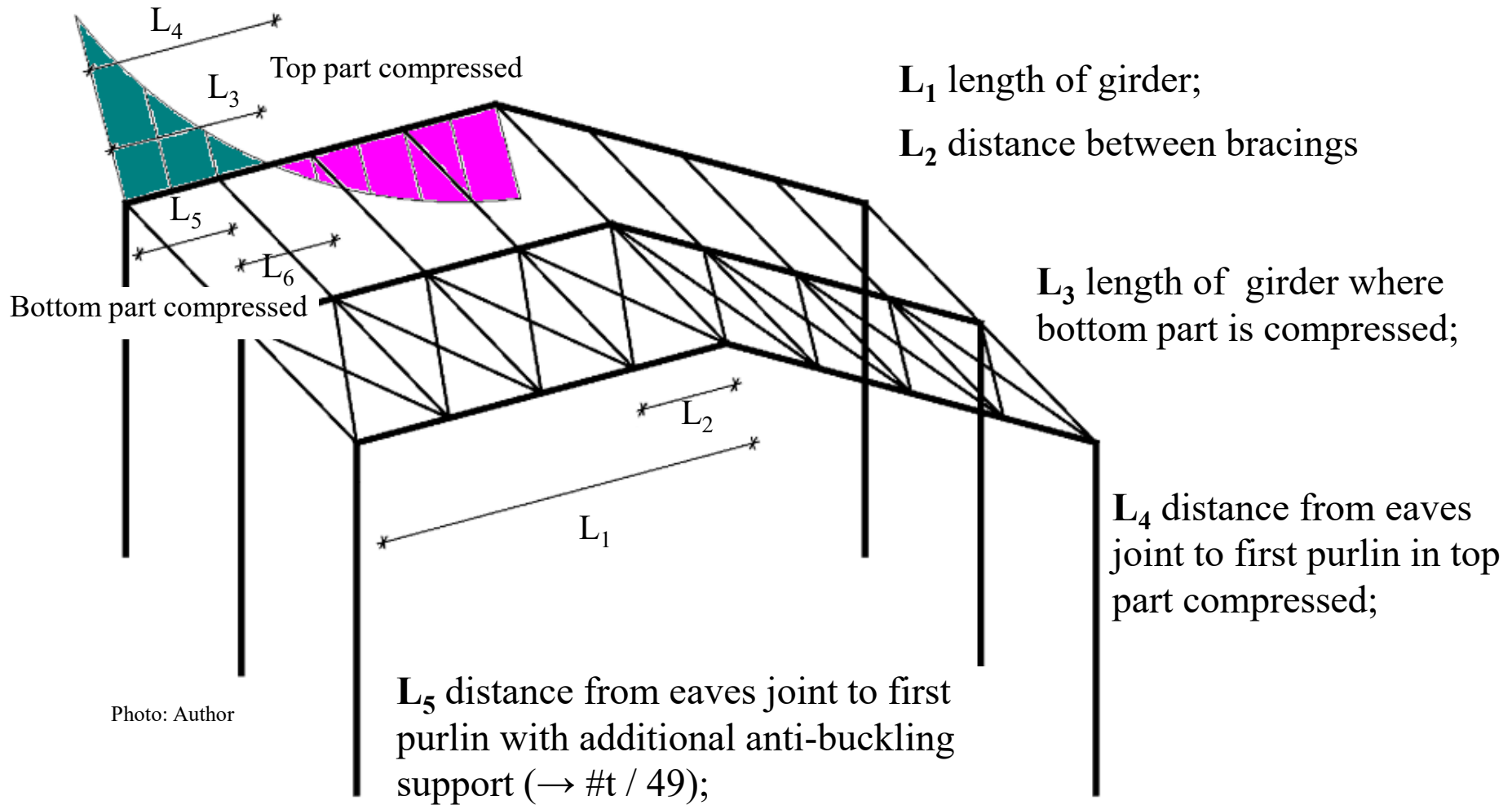
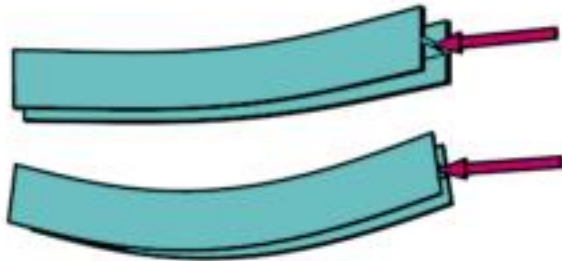


Photo: Author

L_6 distance between the closest purlins with additional anti-buckling supports ($\rightarrow \#t / 49$);

Case of instability		Distance between supports
Flexural buckling	in plane of frame	L_1
	out of plane of frame	L_2
Lateral buckling	if additional anti-buckling supports are applied ($\rightarrow \#t / 49$)	$\max (L_5 ; L_6)$
	if additional anti-buckling supports are not applied ($\rightarrow \#t / 49$)	$\max (L_3 ; L_4)$

Flexural buckling



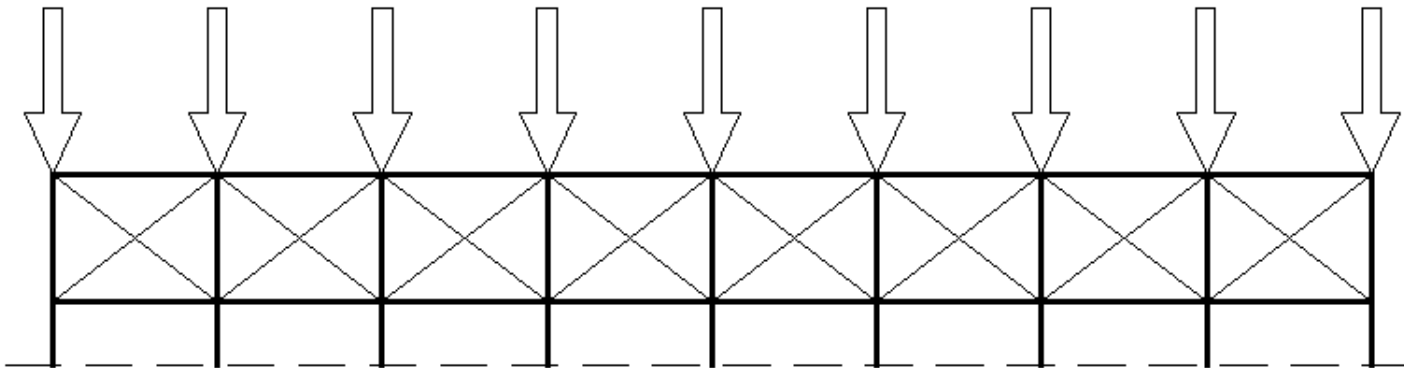
Lateral buckling



Photo: calcresource.com

Three various types of actions must be analysed.

Photo: Author



1. Wind action (pressure on windward wall + suction on leeward wall + wind friction).
Load depends on area of front wall and wind pressure.

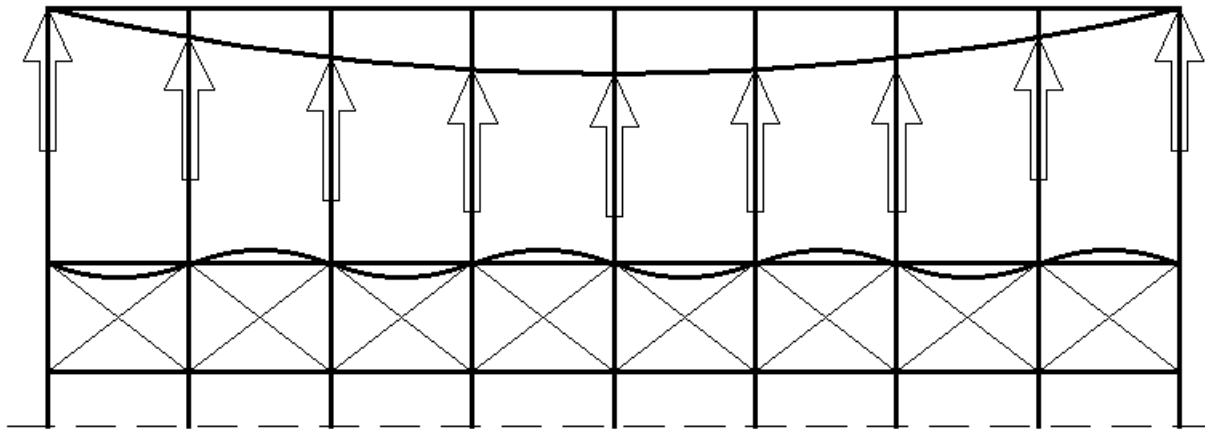
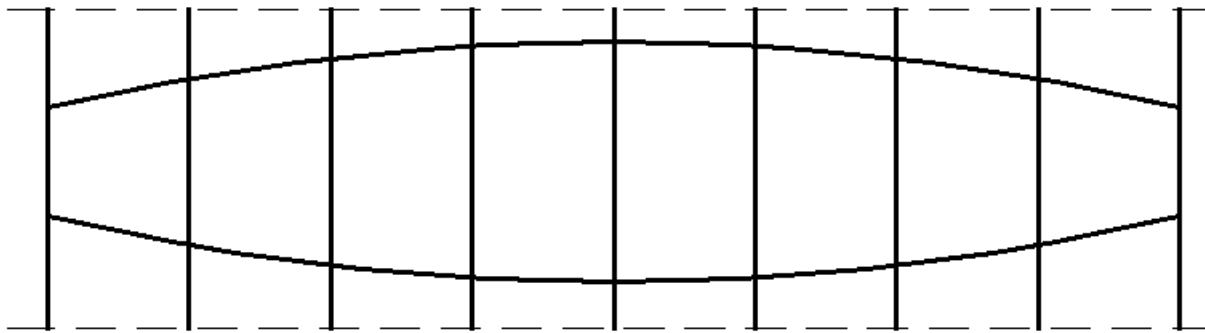


Photo: Author



→ Des #1 / 61

2. Instability of roof girders.

Bracing prevent roof girder from instability out of truss plane.

Without them, buckling of girder occurs on total length. Bracings act as complex of forces, make girder more straight. These forces – as reaction – must be applied to bracings. Value of these forces is **equivalent load**, in proportion to compressive force in girder.

Girders lose stability randomly, in left or right direction. Total effect for entire roof is not:
 (single effect) · (number of girders m)
 but: (single effect) · (reduction factor α_m).

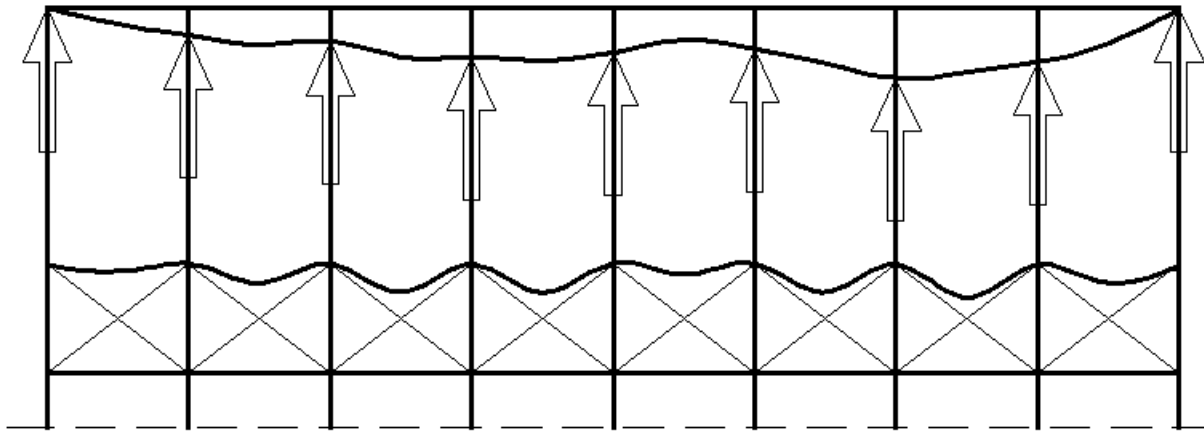
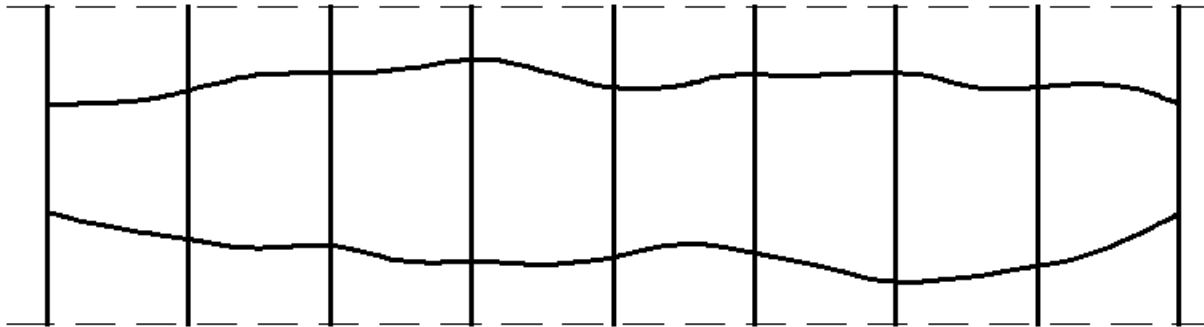


Photo: Author

3. Imperfections of roof girders.
 Roof girders are not perfectly straight. Bracings act as complex of forces, make girder more straight. These forces – as reaction – must be applied to bracings. Value of these forces is **equivalent load**, depends on initial bow imperfection of girder and compressive force in girder.

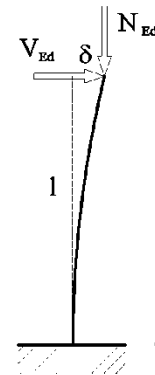
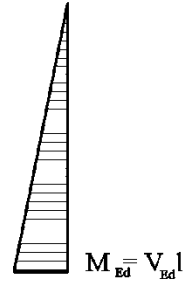
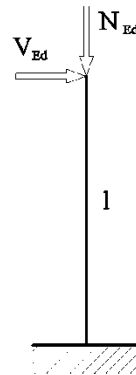
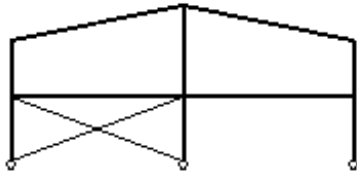


Girders are imperfected randolmly, in left or right. Total effect for entire roof is not:
 (single effect) · (number of girders m)
 but: (single effect) · (reduction factor α_m).

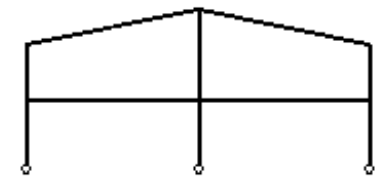
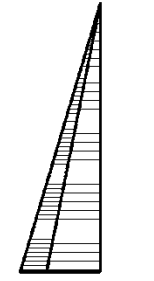
→ Des #1 / 62

Rare reason for using bracing is need to increase rigidity of frame structure in its plane. This increases sway stiffness of frame and prevents II order effects. More information will be given in lecture #13.

Photo: Author



$$\Delta M_{Ed} = \delta N_{Ed}$$



Sometimes massive wall girts are named „wind bracings”. But there is only common name, these members are not bracings. Their rule is only support housing under wind action.

Photo: Author

Types of bracings

Plate systems:

corrugated sheets (\rightarrow #t / 20- 21)

concrete plates (\rightarrow #t / 22)

Bar systems (\rightarrow #t / 23-31)



Photo: nexus.globalquakemodel.org



Photo: lekkaobudowa.pl

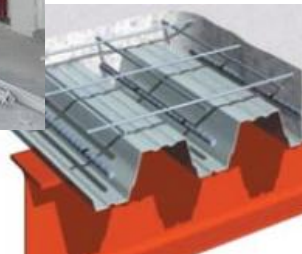


Photo: tatasteelconstruction.com



Photo: nexus.globalquakemodel.org

Corrugated sheet is taken into consideration as protection for purlins and wall girts against flexural and lateral buckling.



Photo: taxonomy.openquake.org



Photo: taxonomy.openquake.org

Complex structure: corrugated sheet & purlins has big stiffness in its plane. Thank to this, we no need to use bracings along structures (for structures without cranes only).

→ #8 / 21

Thermal isolation

Factory-made connecting latch

Anti-buckling protection for purlins according to EN

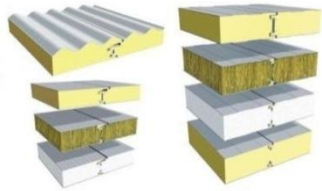


Photo: steelprofil.pl

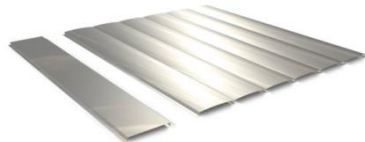


Photo: pruszynski.com.pl

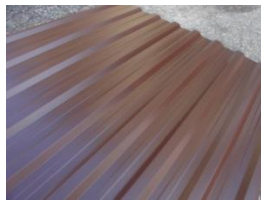


Photo: amarodachy.pl



(per 5 - 10 years from erection)



Photo: building.co.uk

Separated problems (generally: responsible of manufacturer)

The most often case is concrete plate or concrete plate on corrugated sheet as lost formwork. Only stiffness of concrete plate is taken into consideration; participation of corrugated sheet is in this case neglected.

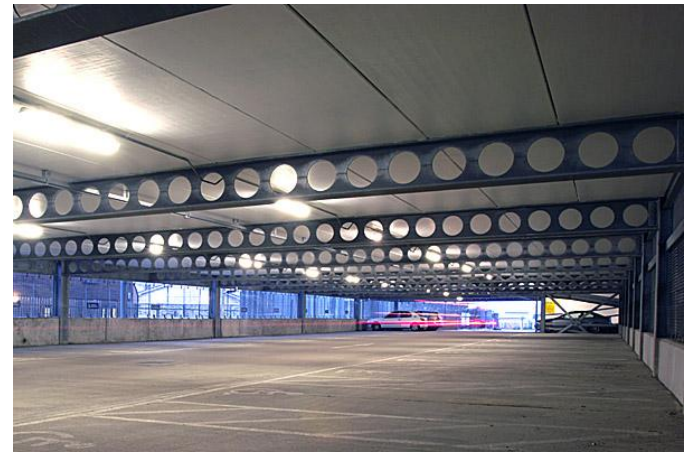


Photo: steelconstruction.info



Photo: steelrite.com

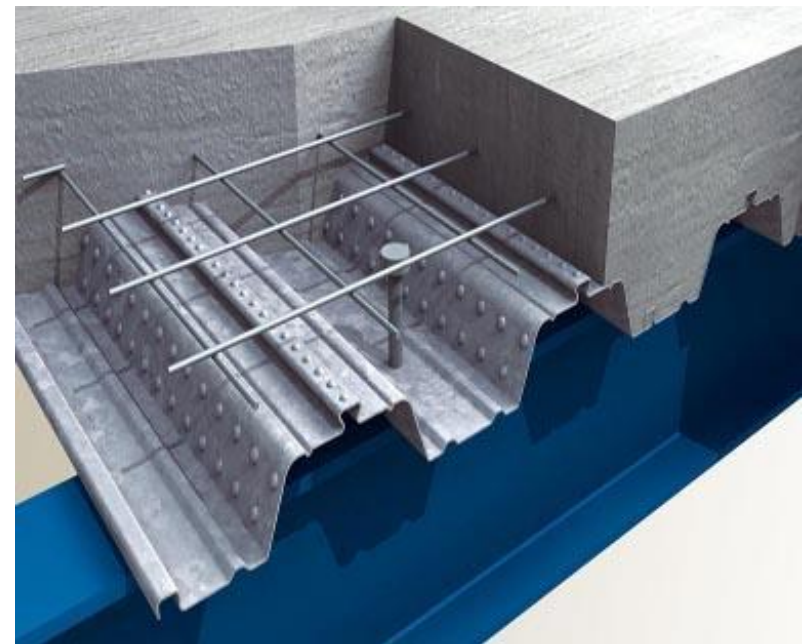


Photo: tatasteelconstruction.com

The most popular scheme of bar bracing is X-bracing. Bracing members can be connected each other in half of span, or can pass off in two various planes.

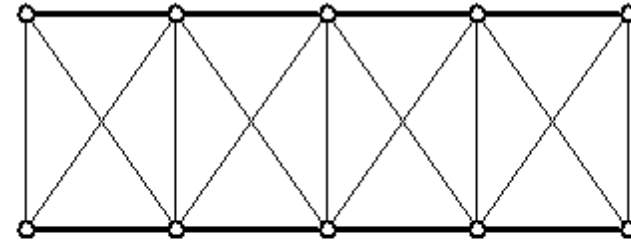
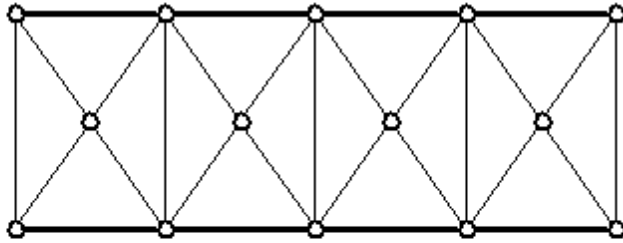


Photo: Author

Photo: halfen.com

Photo: steelconstruction.info



Photo: spantec.com.au

Bracings - recommended cross-sections (→ Lec # 10)



Photo: calgor.com.pl



Photo: rafstal-inox.pl



Photo: rafstal-inox.pl

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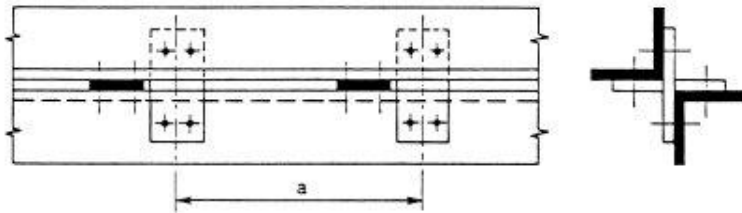
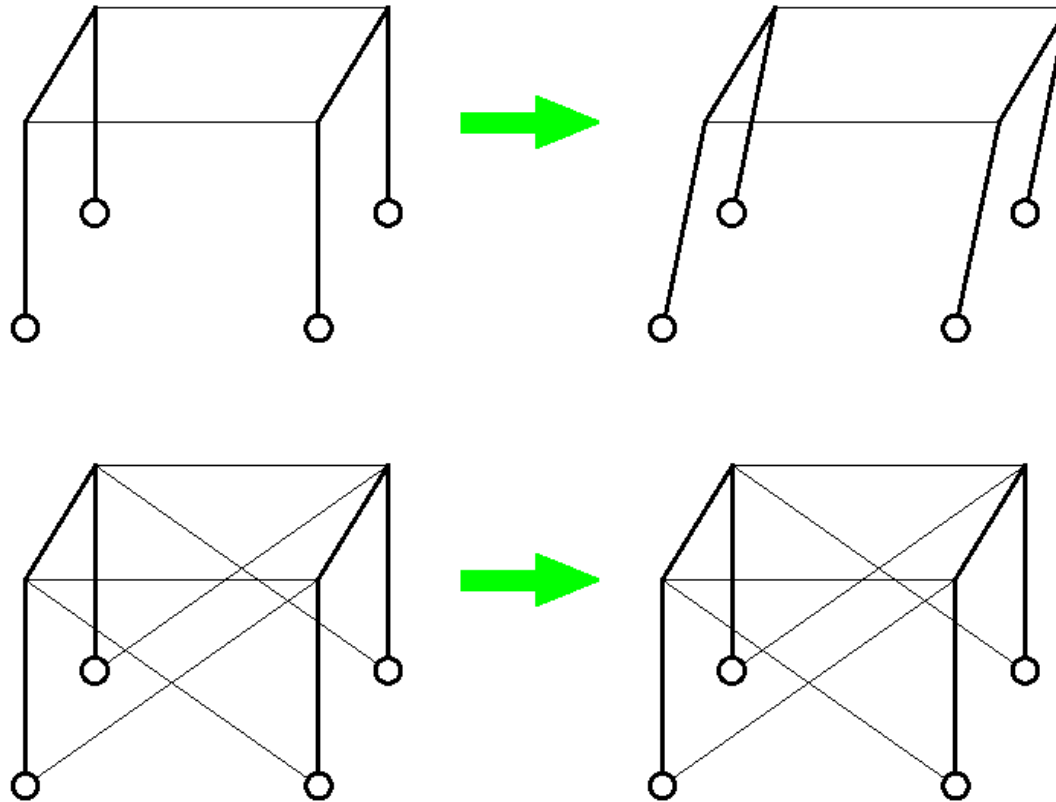


Photo: EN 1993-1-1 fig. 6.13



Photo: stalhart.pl



Very important is specific shape of system bracings-frames. Rectangular shape is not geometrically unchanged and not prevent instability. Only triangles bar bracings are geometrically unchanged.

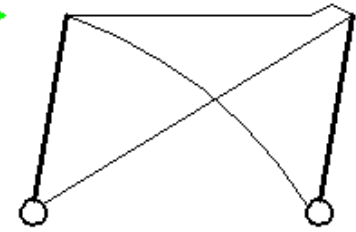
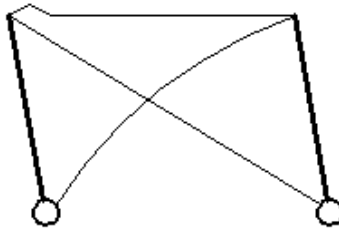
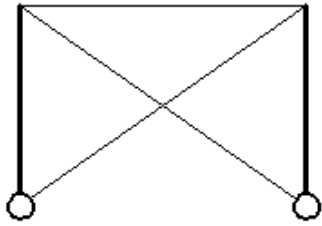


Photo: Author

During its life, structure works under various loads and actions. Bar bracings are subject to alternating buckling. The result is increasing of plastic deformations of bracings.

For corrugated sheets important is ovalisation of holes of bolts and corrosion around holes of bolts. Cooperation housing-structure decreases after few years from erection of structure and anti-buckling protection by housing decreases.



Photo: encrypted-tbn3.gstatic.com



Photo: previews.123rf.com

Photo: Author



Because of buckling, we should mount rigging screws (or other type of system to regulate) for reduction of deformations in case of non-rigid bar bracings.



Photo: dreamstime.com

We should avoid vertical bracings inside structure, to ensure open spaces inside buildings. Bracings in plane and out of plane of frames can be applied only in outer walls of structure.



Photo: lekkaobudowa.pl



Photo: i.wnp.pl

In situation when bracing inside building are needed, should be applied rather portal bracing than X bracing. Portal bracing didn't block internal communication.

General requirements for bar bracings:

- Additional forces in purlins and girders, indicated by co-operation bracing \leftrightarrow main structure, must be taken into account;
- Horizontal distance between both ends of bracing $\leq 6,00$ m \rightarrow dead weight of bracing can be neglected (otherwise – bending moments from dead weight);



Photo: rafstal-inox.pl



Photo: stalhart.pl



Photo: calgor.com.pl

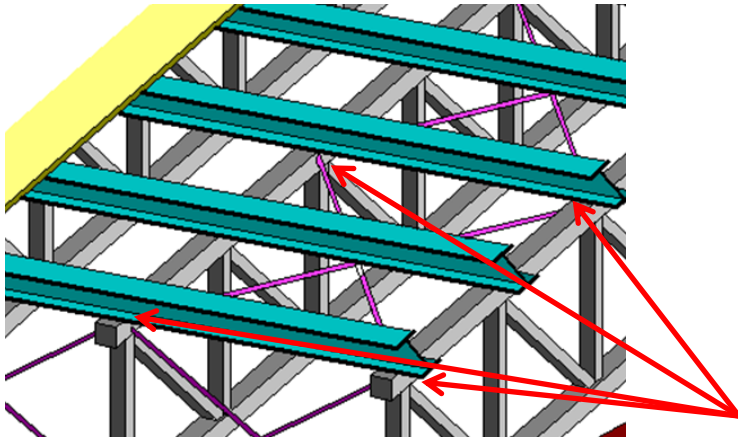
Bar bracings can be divided into two groups:

- non-rigid (which lost stability under even very very small compressive force);
- rigid (resist under compressive force)



Photo: rafstal-inox.pl

Static model 2D (→ #t / 46)



Bracings bars are installed between top chords of vertical truss. Purlins are over top chords of vertical truss. Cooperation between bracing bars and purlins is possible for only these purlins, which have common points with bracing bars.

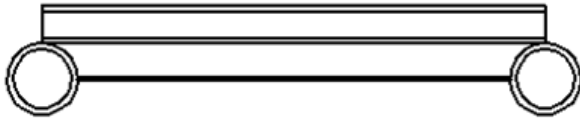
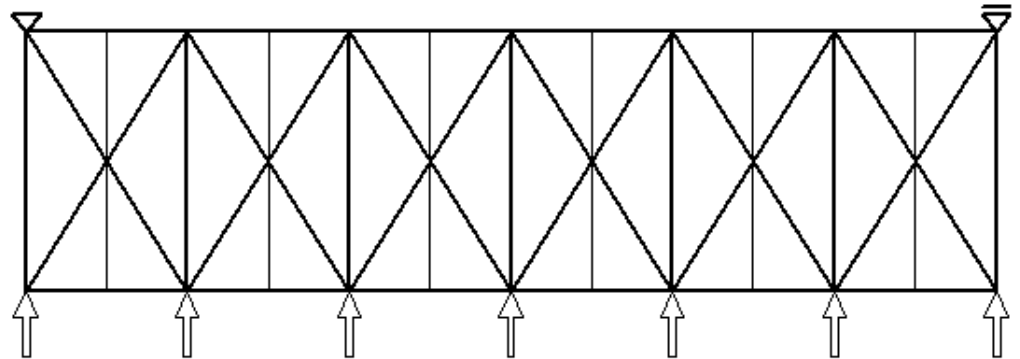


Photo: Author

Purlins not cooperated with bracings are not taken into consideration in static model.



Rigid bracings: both branches (compressed and tensed) are taken into consideration in static model

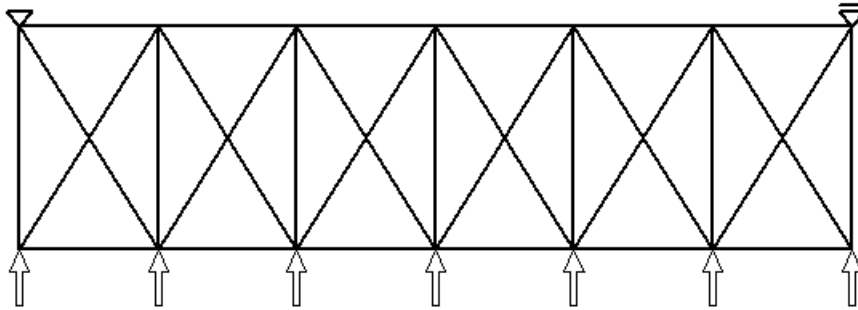
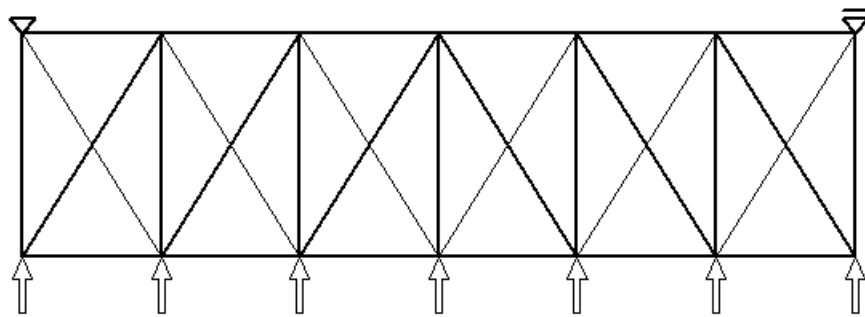
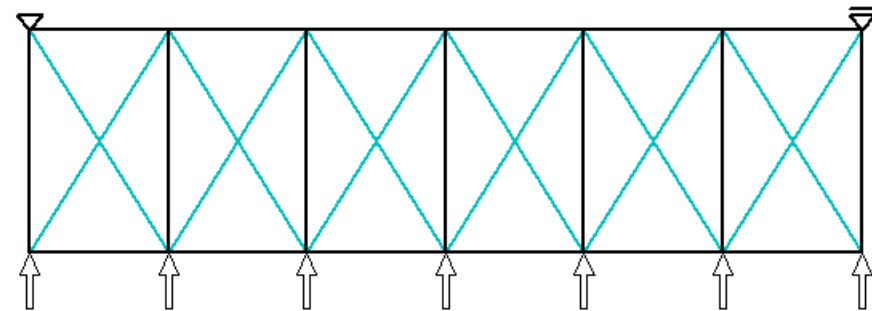


Photo: Author



Non-rigid bracings (first proposal): tensed branches only are taken into consideration in static model.

Non-rigid bracings (second proposal): both branches (compressed and tensed) are taken into consideration in static model with application of material non-linearity in calculation; material which no work for compressive forces.



Not recommended type of longitudinal bracings. Bracing members are connected to purlin. It completely change static scheme of purlin (two-span beam, not one-span). Additionally, on bracing members act loads from purlins.

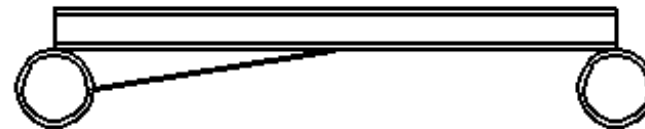
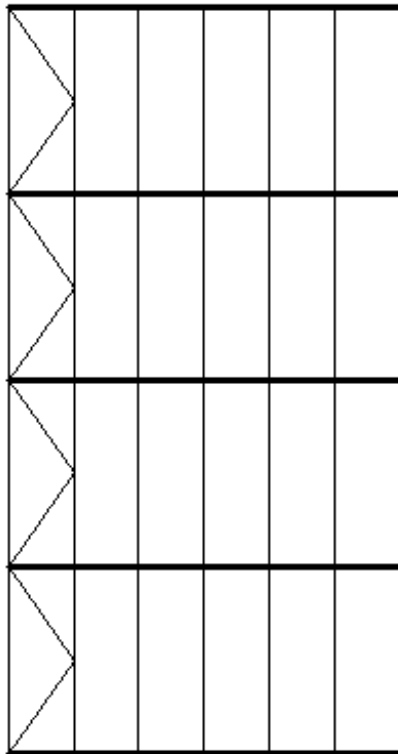
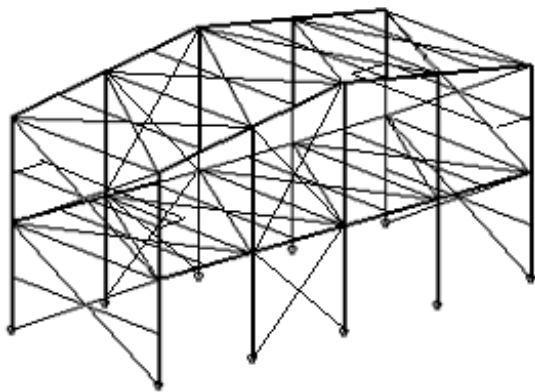


Photo: Author

Location inside structure

Crane bracing (IInd step of study)



Wall in-plane bracing
(very rare)

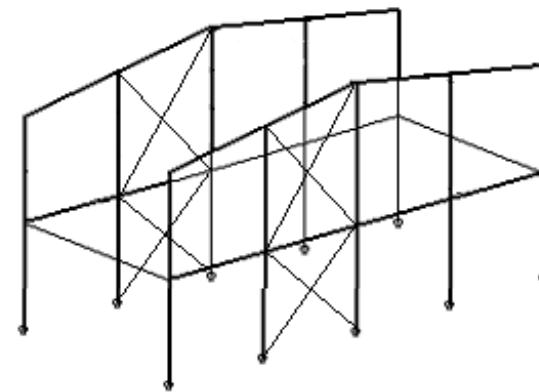
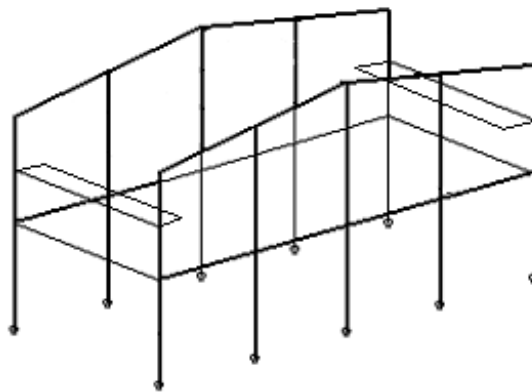
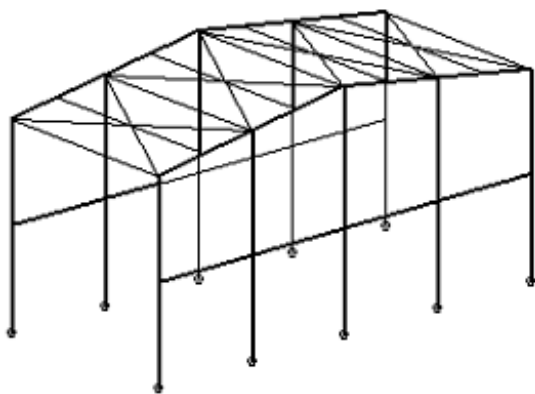
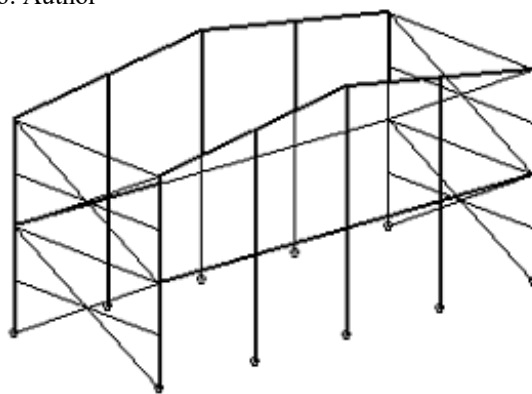


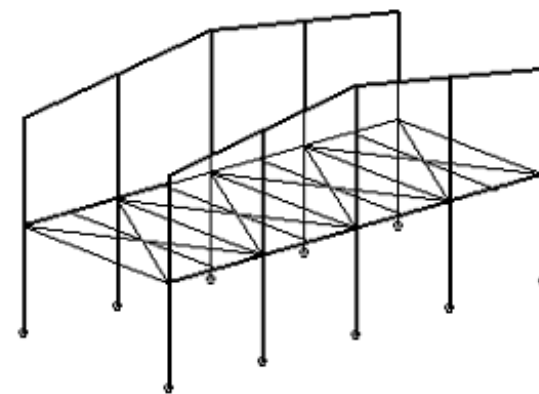
Photo: Author



Roof bracings



Wall out-of-plane bracing



Floor diaphragms

Horizontal crane bracings (surge girder)

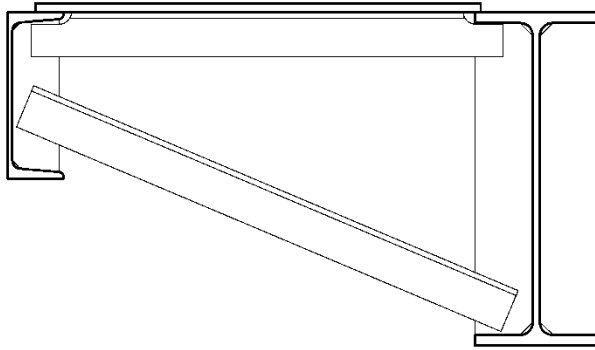


Photo: Author



Photo: konar.eu

→ Metal Structures II, IInd step of study;

Wall in-plane bracings

This type is applied, first of all, for high buildings. This type of bracings will be presented on IInd step of study.
Rare case is increasing sway stiffness for frames (\rightarrow #t / 18)



Photo:propertyeu.info



Photo:wikipedia



Photo: de51gn.com

Roof horizontal upper longitudinal bracings;

For truss and I-beam girders

- Next to eaves and valley
- Loads from horizontal upper transversal bracings

Can be omitted in halls without gantries

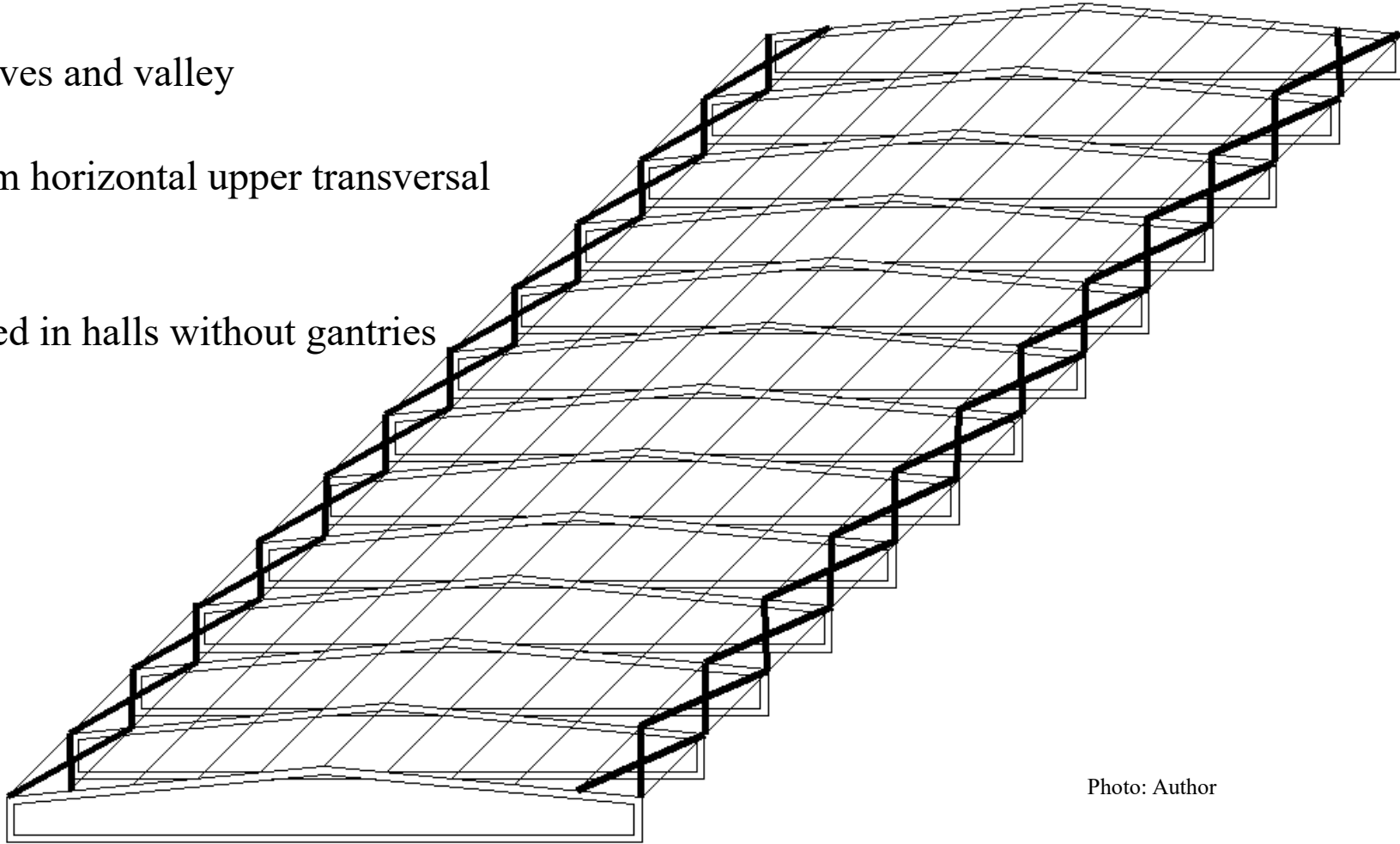


Photo: Author

Roof horizontal upper transversal bracings;

For truss and I-beam girders;

- Every eight band or in the distance of 60,0 m each other;
- At the ends of structure;
- Next to dilatations;
- Loads perpendicular to plane of structure;
- Equivalent effects of imperfections;
- Equivalent effects of instability prevention;

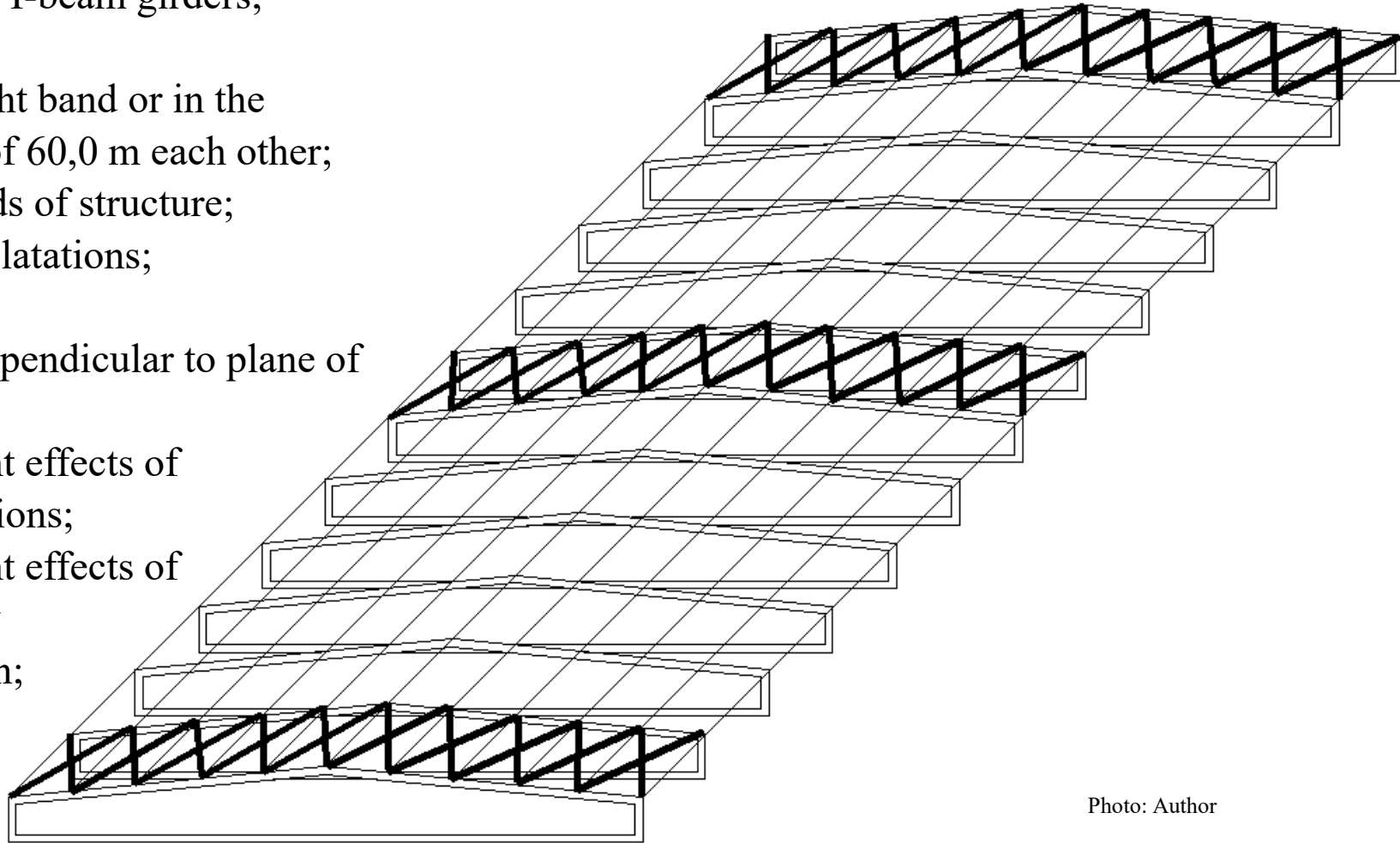


Photo: Author

Roof vertical longitudinal bracings;

For truss girders;

- Next to eaves, ridge and valley, under skylights or in the distance on 15,0 m each other;
- Loads perpendicular to plane of structure in montage stage, protection of bottom chord from buckling

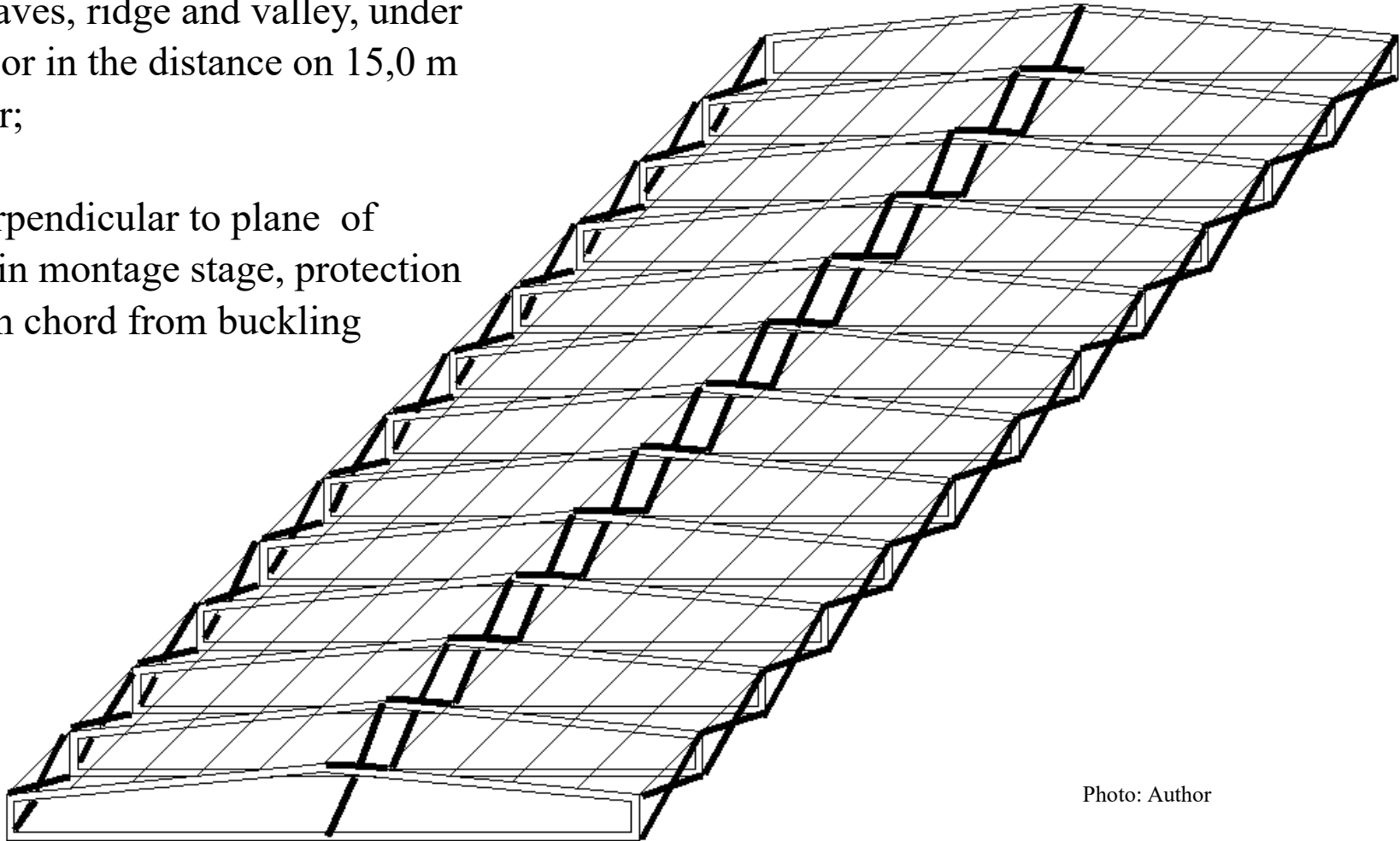


Photo: Author

Roof horizontal lower transversal bracings;

For truss girders;

- Every eighth band or in the distance of 60,0 m each other;
- At both ends of structure;
- Next to dilatations;

For structure with gantries or for big value of wind suction only;

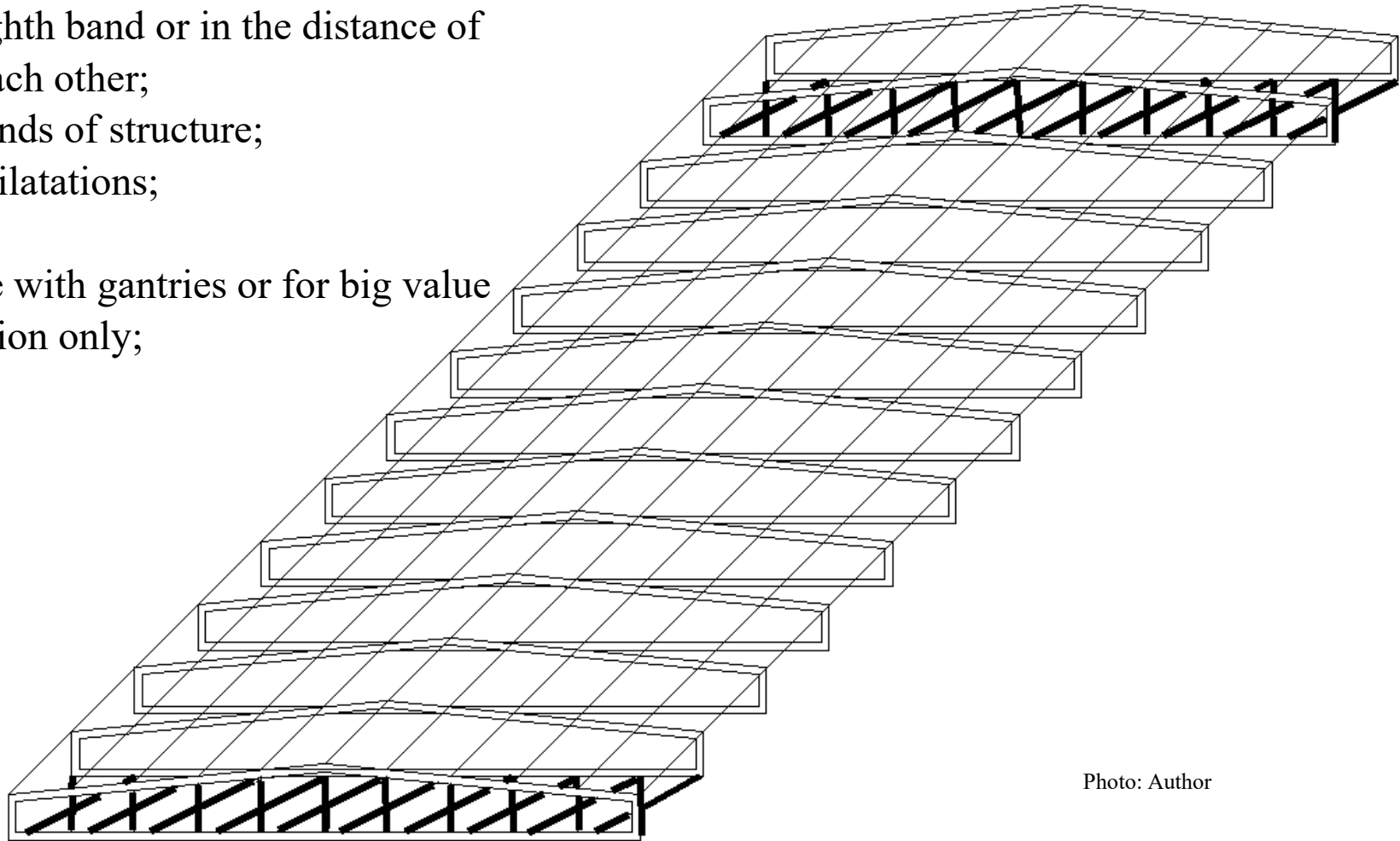


Photo: Author

Roof horizontal lower longitudinal bracings;

For truss girders;

- Next to eaves and valley;

For structure with gantries or for big value of wind suction only;

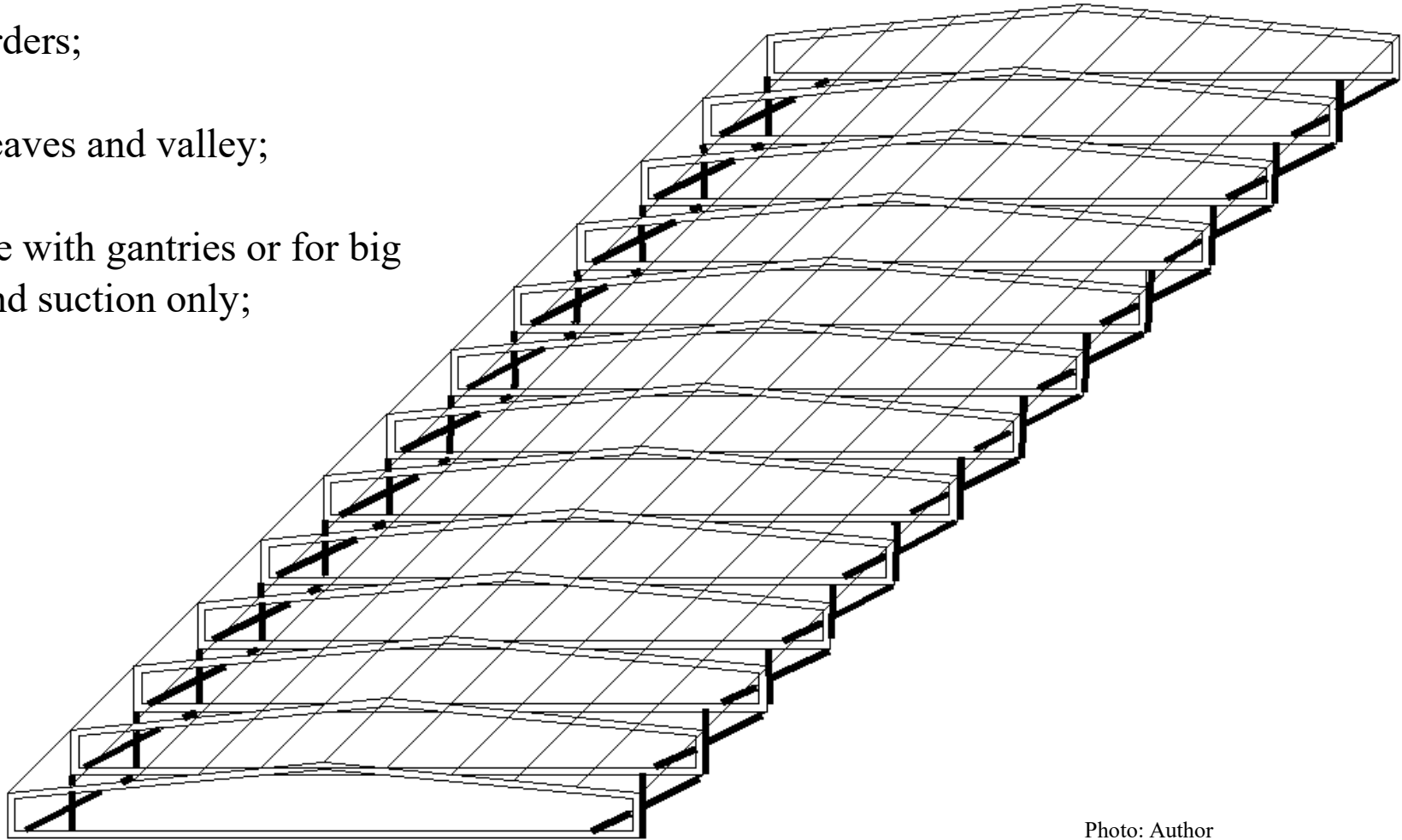


Photo: Author

Wall out-of-plane bracings

- Under transverse roof bracings in half of hall / central part of hall between expansion joints;

Transmission of loads from wall (wind on front wall, prevention of girder buckling, imperfections of columns) to bases;

Loads: perpendicular to plane of structure and sway imperfection of columns.

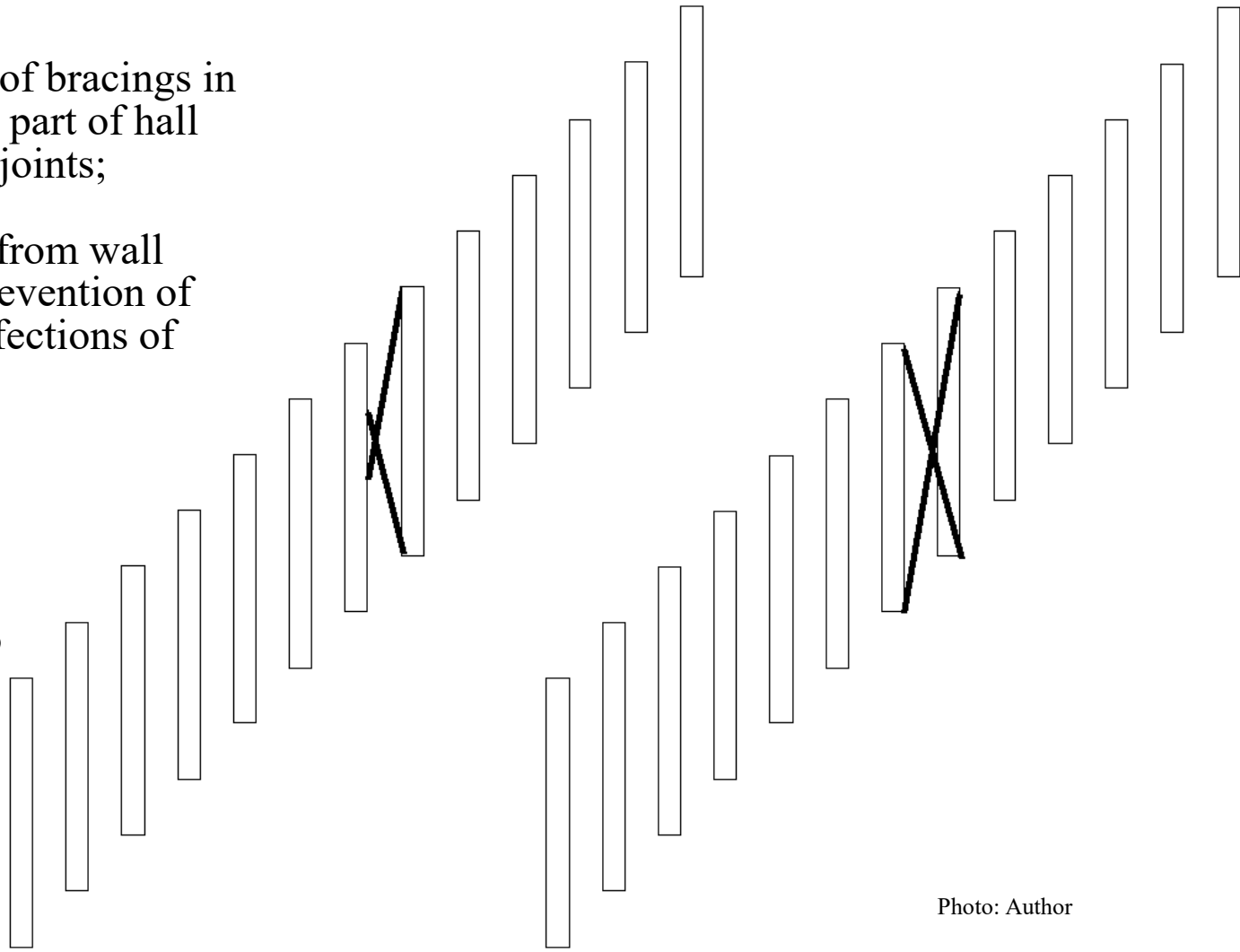


Photo: Author

Floor diafrgm

Generally, floor diafrgms are concrete plates – floors / ceilings between levels.

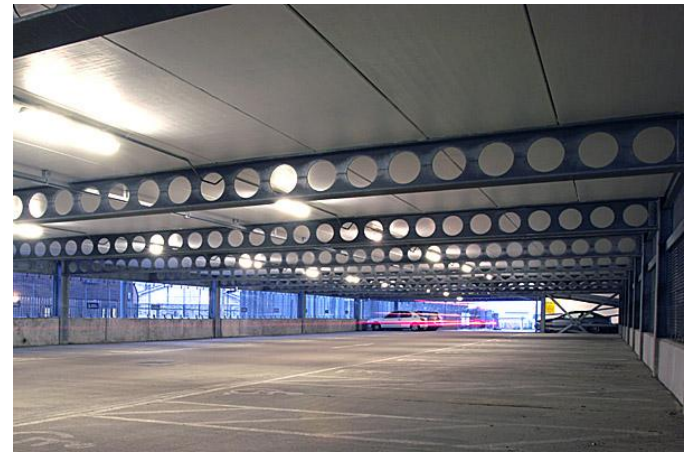


Photo: steelconstruction.info



Photo: steelrite.com

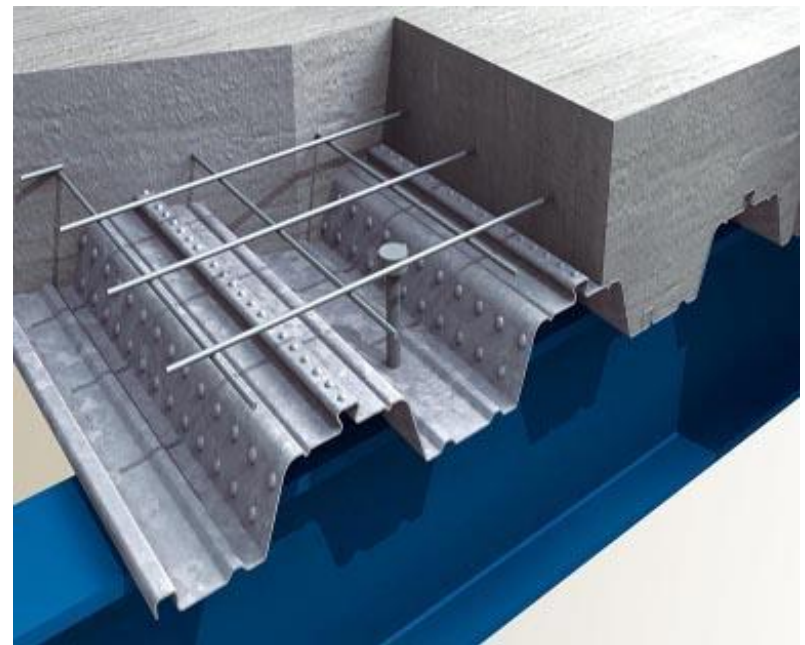


Photo: tatasteelconstruction.com

Calculations

There are many various methods of calculations for bracing systems. It depends on:

- type of bracing system (concrete plate, corrugated sheet, bar system);
- type of instability (flexural buckling, lateral buckling);
- position in structure (various types of roof bracings, wall bracings, crane bracings, floor diaphragm).

Generally, four elements must be calculated:

- equivalent forces (which are result of anti-buckling protection) act on bracings;
- resistance of bracing under loads;
- behaviour of protected member: no instability (sufficient efficiency of bracings), few types of instability only (partial efficiency of bracings), each types of instability (insufficient efficiency of bracings);
- additional forces applied to protected members, as effect of theirs cooperation with bracings.

Sometimes part of this points could be neglected.

Concrete plate:

- calculation of equivalent forces from steel part for concrete plates;
- calculation of resistance concrete plate under these forces are need (→ Concrete Structures);
- there is assumption of total efficiency of protection against each form of instability for beams;
- no additional forces are applied to protected members.



Photo: nexus.globalquakemodel.org

Floor diaphragms: forces from sway imperfection of columns act on plate.

Photo: EN 1993-1-1 fig 5.7

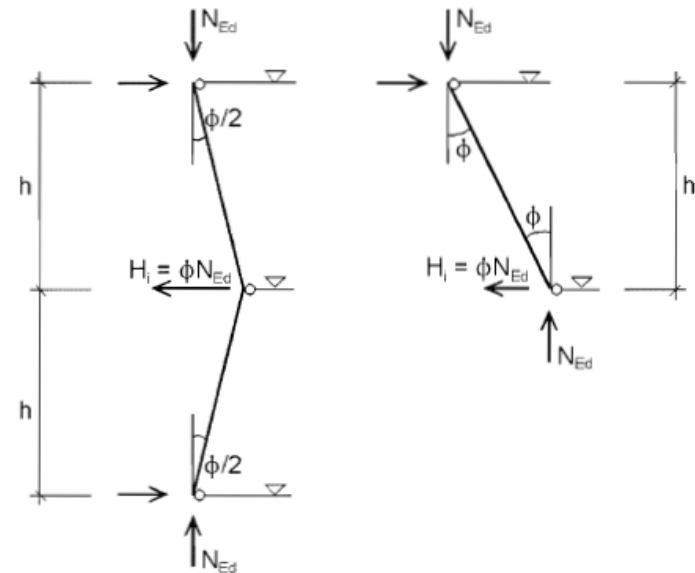




Photo: i.warosu.org

Corrugated sheet:

- no calculation of equivalent forces from protected members for corrugated sheet;
- no calculation of resistance of corrugated sheet;
- two separated ways of calculation for protected members (purlin, girt): for flexural buckling and for lateral buckling; this calculation bases on geometry of beam and corrugated sheet;
- no additional forces are applicated to protected members.

Bar system:

- various technical solutions for protection of beam against flexural and lateral buckling;
- various ways of calculation for various position in structure (various types of roof bracings, wall bracings).
- equivalent forces must be calculated (various way for flexural buckling and lateral buckling protection);
- resistance of bracing under loads must be analysed;
- behaviour of protected member: no instability (sufficient efficiency of bracings), few types of instability only (partial efficiency of bracings), each types of instability (insufficient efficiency of bracings);
- there are additional loads act on structure (not only equivalent forces), as a effect of cooperation of bar bracing systems and rest part of structure;



Photo: steelconstruction.info

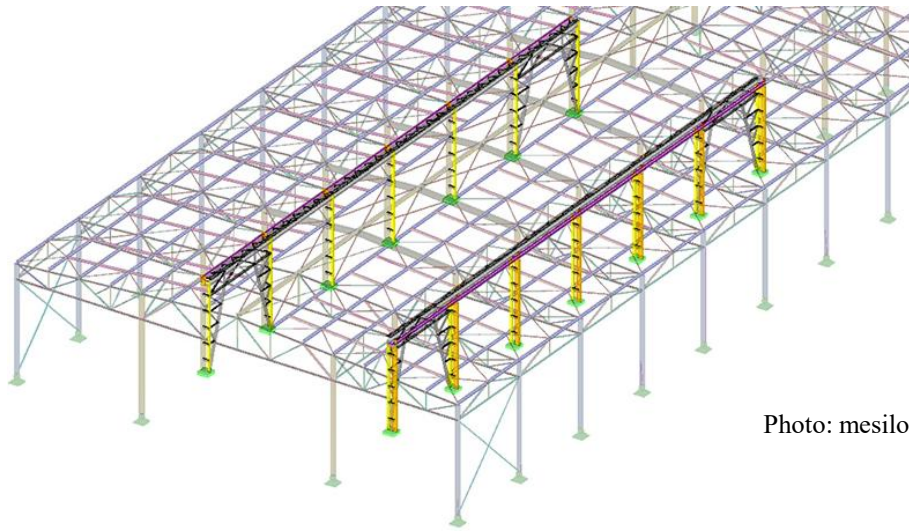


Photo: mesilo.pl

→ Des #1 / 68

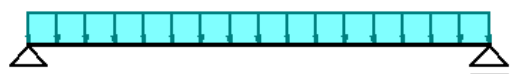
3D model in FEM calculations: we have full information about cooperation between trusses, purlins and bracing bars right away. Calculation is made in two steps:

- initial calculation: dead weight, climatic actions, live loads etc.
- equivalent forces: from imperfections of trusses and from prevention of instability of trusses. Values of these forces are calculated based on forces in trusses after initial calculations.

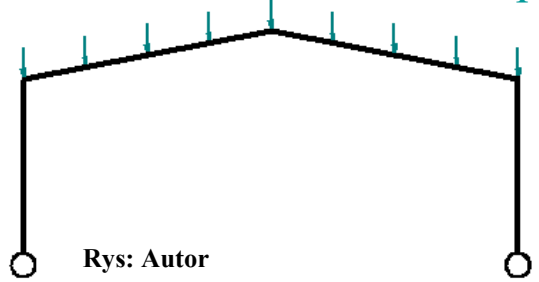
Final effect: forces in chords from initial calculations and equivalent forces is final result of static calculations.

2D model (method of equivalent flat frames) is much more complicated:

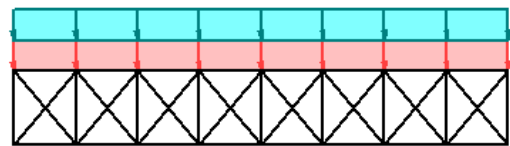
Purlin: **bi-axial bending**



Flat frame: **forces from purlins**

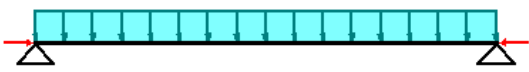


Roof bracings:
horizontal truss

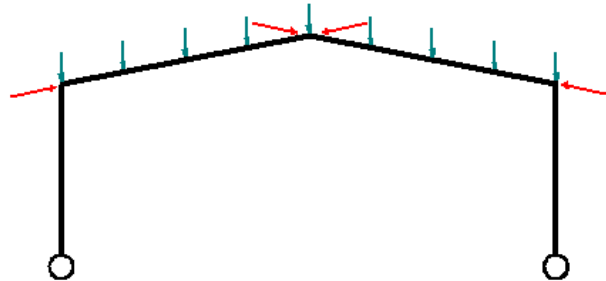


Wind action
Equivalent forces from
instability and imperfection

Cooperation frame-bracings:
additional forces in purlins,
additional forces in frame.



Purlin: **bi-axial bending** +
compressive axial force
(recalculation)



Frame: **additional axial forces**
from cooperation with
bracings (recalculation)

→ Des #1 / 69

Technical solutions

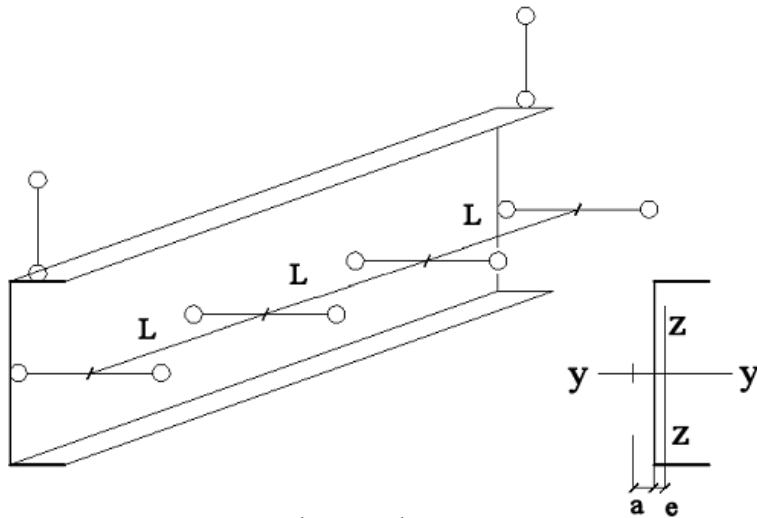


Photo: Author

Bracings against flexural buckling should be applied to centre of gravity of cross-section, perpendicullary to weak axis of inertia.

#t / 51

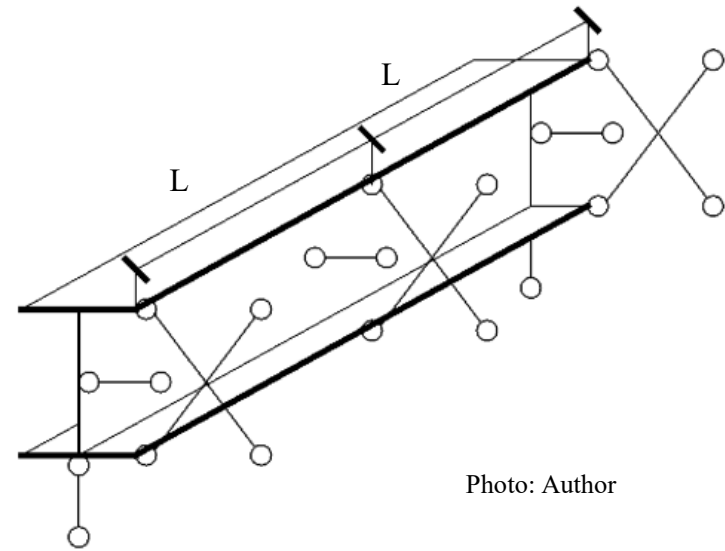


Photo: Author

Bracings against torsional, flexural-torsional and lateral buckling should prevent for torsional deformation of cross-section

#t / 49-51

Bar bracing against lateral buckling of roof girders and of primary beams

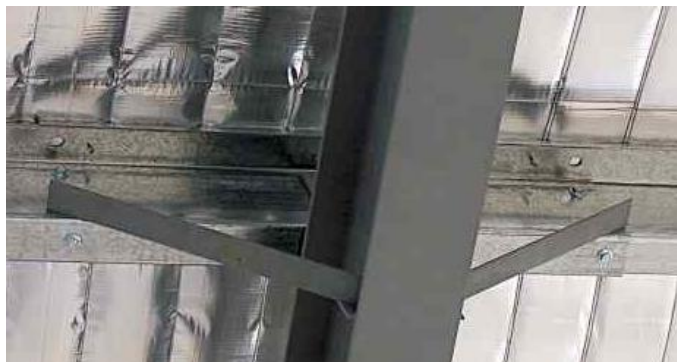


Photo: builderbill-diy-help.com

In case, when top flange of roof girder / primary beam is compressed, the protection against lateral buckling is a purlin-bracing system (system connected to upper flange).

In case, when bottom flange is compressed, it is necessary to provide additional anti-buckling supports connecting bottom flange to purlin-bracing system.

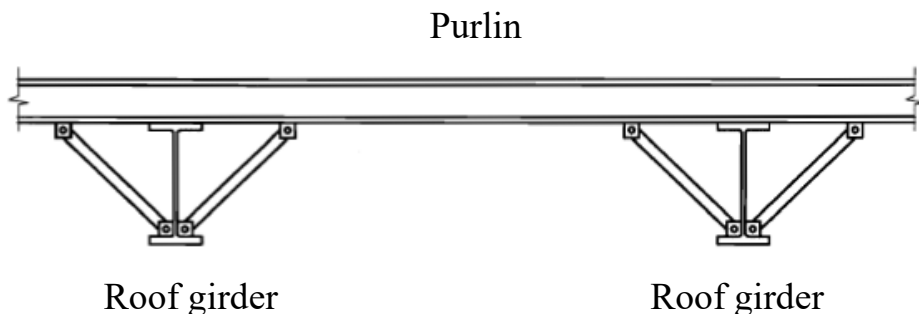


Photo: EN 1993-1-1 fig 6.5



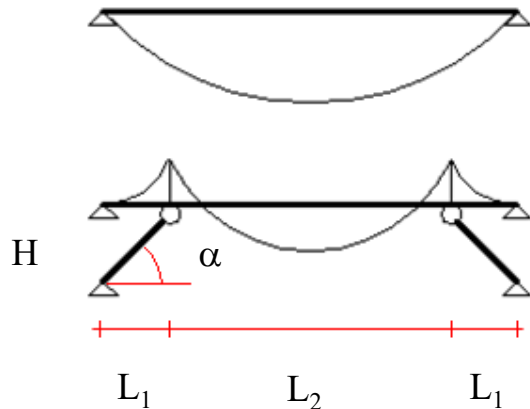
Photo: steadmans.co.uk

For such case, important is analysis of purlin's static scheme.



Photo: builderbill-diy-help.com

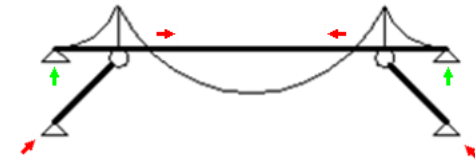
Photo: Author



Angle α should be equal $45^\circ - 60^\circ$. H is depth of roof girder.

If $L_2 \geq 8 L_1$ then purlin could be calculated in first static scheme.


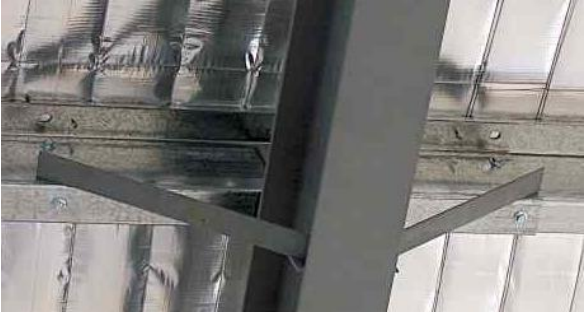
Photo: Author



But for very high cross-section of roof girder is possible, than distance L_1 is big. In such case, second static scheme (purlin-bracings) must be analysed.

Big compressive forces can occur in bracings as the effect of reaction. Additionally, in central part of purlin occurs the same big compression, as effect of co-operation purlin-bracings.

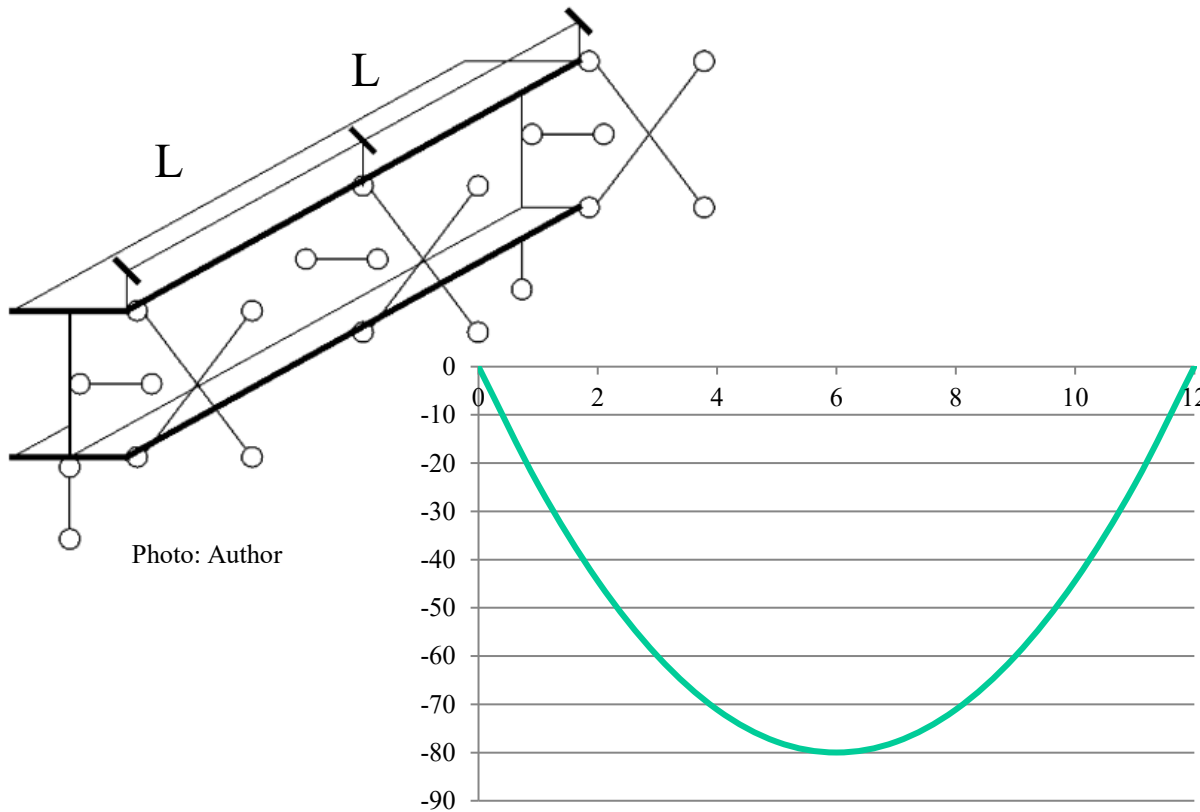
Equivalent forces, applied to bracings from protected members

<p>Effects</p>	 <p>Photo: steelbuildingstructure.com</p>	 <p>Photo: builderbill-diy-help.com</p>
<p>Protection from lost of stability</p>	<p><u>Roof:</u> $F_{Ed, bracing} = 1,0 \alpha_m N_{Ed}^* / 100$ EN 1993-1-1 5.3.3.(4)</p> <p><u>Wall:</u> No analysed</p>	$F_{Ed, bracing} = 1,5 \alpha_m N_{Ed}^* / 100$ EN 1993-1-1 6.3.5.2
<p>Imperfections of protected members</p>	<p><u>Roof:</u> $q_d = \Sigma [8 N_{Ed}^* (e_0 + \delta_q) / L^2]$ (iteration) EN 1993-1-1 (5.13)</p> <p><u>Wall:</u> #6 / 83</p>	<p>No analysed</p>

Example 1

Corrugated sheet, prevention for buckling of purlin.

This example is analysed based on Lec #5 example #2



IPE 300

S235 $\rightarrow f_y = 235 \text{ MPa}$

$L = 6,00 \text{ m}$

$E = 210 \text{ GPa}$

$G = 81 \text{ GPa}$

$J_y = 8\,356 \text{ cm}^4$

$J_z = 603,8 \text{ cm}^4$

$W_y = 557,1 \text{ cm}^3$

$W_{pl,y} = 628,4 \text{ cm}^3$

$J_w = 125\,900 \text{ cm}^6$

$J_T = 20,12 \text{ cm}^4$

$i_y = 12,46 \text{ cm}$

$i_z = 3,35 \text{ cm}$

$y_s = 0,0 \text{ cm}$

$M_{Ed} = 80 \text{ kNm}$

Example 1a

Corrugated sheet T 18

Corrugated sheet, prevention of flexural buckling of purlin

$$t = 0,88 \text{ mm}$$

$$h = 10 \text{ mm}$$

Purlins: IPE 300

$$h = 300 \text{ mm}$$

$$b = 150 \text{ mm}$$

$$t_f = 10,7 \text{ mm}$$

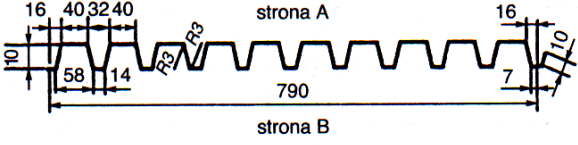
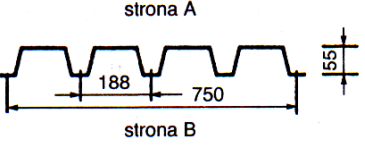
$$t_w = 7,1 \text{ mm}$$

$$J_{z, el} = 604 \text{ cm}^4$$

$$J_w = 125\,900 \text{ cm}^6$$

$$J_t = 20,7 \text{ cm}^4$$

S 235

Podstawowe wymiary blach		Grubość mm	Masa kg/m ²	
Typ	T 18		0,55	5,7
			0,75	7,9
			0,88	9,2
Typ	T 55		0,75	9,1
			0,88	10,7
			1,00	12,1
			1,25	15,1

One span, $l = 6,0 \text{ m}$

Distance between purlins $s = 2,0 \text{ m} = 2\,000 \text{ mm}$

Width of roof $b_{\text{roof}} = 14,0 \text{ m} = 14\,000 \text{ mm}$

Photo: W. Bogucki, M. Żybertowicz, „Tablice do projektowania konstrukcji metalowych”, Arkady 1996

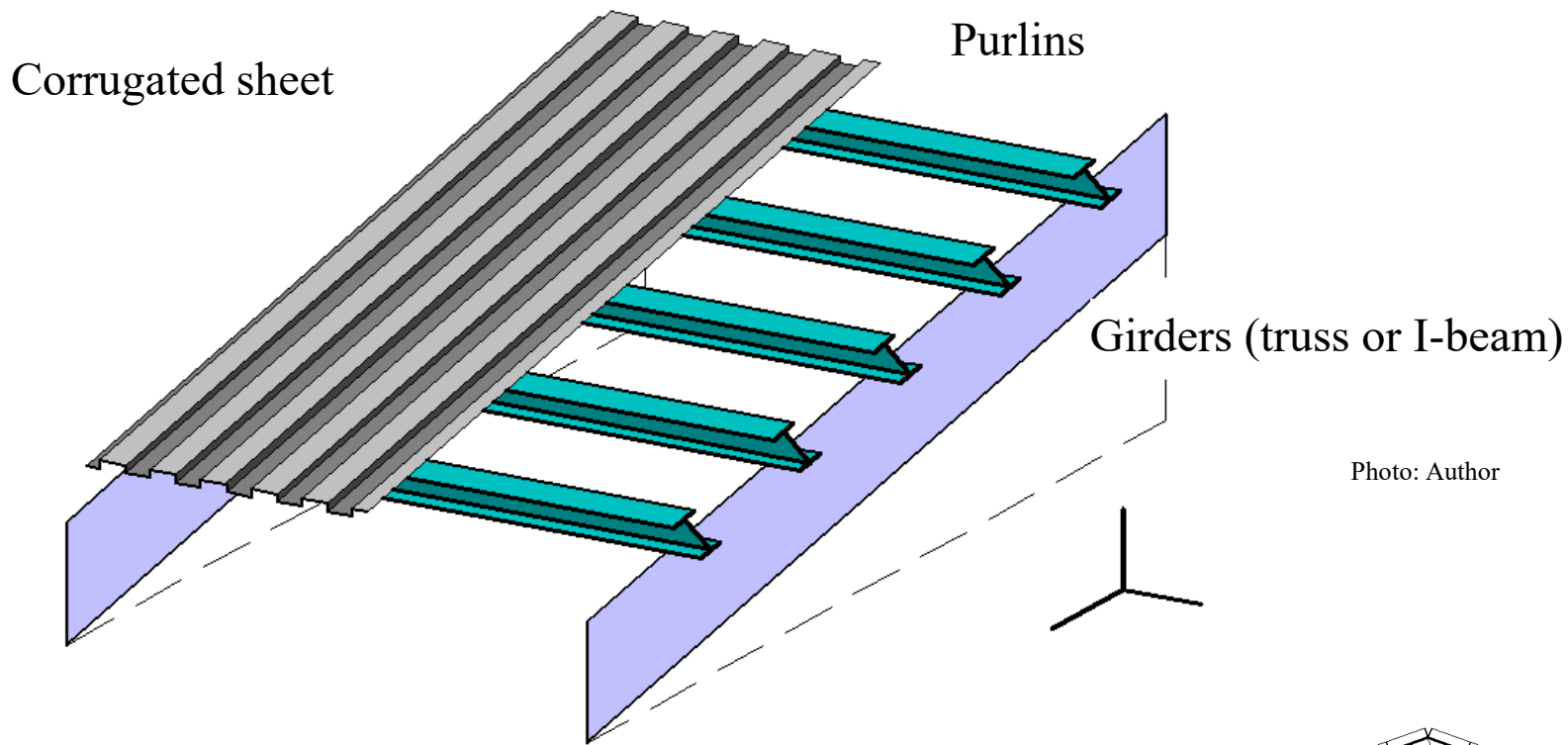
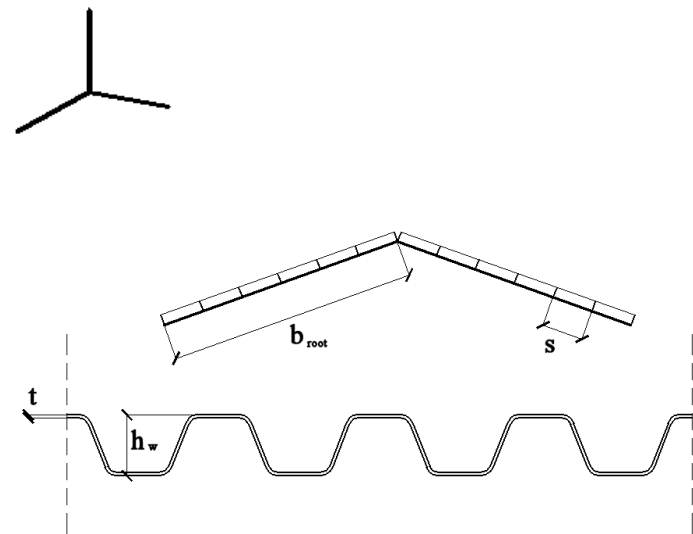


Photo: Author



$$S_{cs} \geq 70 (E J_w \pi^2 / l^2 + G J_t + 0,25 E J_z h \pi^2 / l^2) / h^2$$

$$[N] S_{cs} = 1000 \sqrt{(t^3) [50 + 10 \sqrt[3]{(b_{root})}]} s / h_w [mm]$$

EN 1993-1-3 (10-1a, 10.1b)

Requirement:

$$S_{cs} \geq 70 (E J_w \pi^2 / l^2 + G J_t + 0,25 E J_z h \pi^2 / l^2) / h^2$$
$$[N] S_{cs} = 1000 \sqrt{(t^3)} [50 + 10 \sqrt[3]{(b_{root})}] s / h_w [mm]$$

$$70 (E J_w \pi^2 / l^2 + G J_t + 0,25 E J_z h_1 \pi^2 / l^2) / h_1^2 = 3 451 \text{ kN}$$

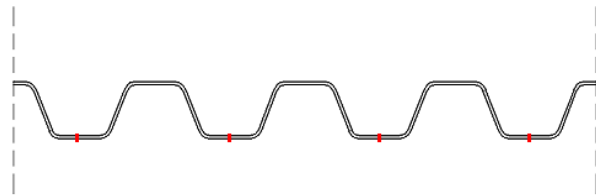
$$\begin{aligned} [N] S_{cs} &= 1000 \sqrt{(t^3)} [50 + 10 \sqrt[3]{(b_{root})}] s / h_w [mm] = \\ &= 1000 \sqrt{(0,88^3)} [50 + 10 \sqrt[3]{(14 000)}] 2 000 / 10 = \\ &= 1000 \cdot 0,826 (50 + 10 \cdot 24,101) 200 = \\ &= 48 074 852 [N] = 48 075 \text{ kN} \end{aligned}$$

$$48 075 \text{ kN} > 3 451 \text{ kN}$$

OK., purlin are protected from flexural buckling

Example 1b

Corrugated sheet, prevention of flexural buckling of purlin



Previous result is true only if the sheeting is connected to beam at every rib.

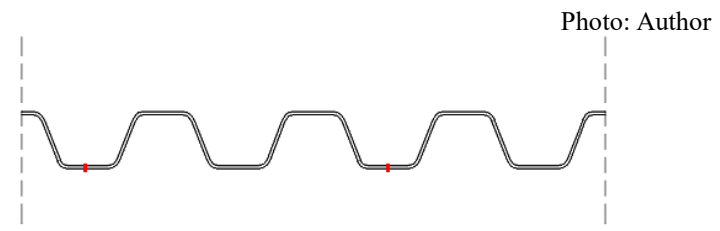


Photo: Author

If the sheeting is connected to beam at every second rib only (effect of several years of exploitation), we must take into consideration $0,20 S_{cs}$

$$70 (E J_w \pi^2 / l^2 + G J_t + 0,25 E J_z h_l \pi^2 / l^2) / h_l^2 = 3\,451 \text{ kN}$$

$$\begin{aligned} [\text{N}] \, 0,20 S_{cs} &= 0,20 \cdot 1000 \sqrt{(t^3)} [50 + 10 \sqrt[3]{(b_{\text{root}})}] s / h_w [\text{mm}] = \\ &= 0,20 \cdot 48\,074\,852 [\text{N}] = 9\,614,970 \text{ kN} \end{aligned}$$

$$9\,614,970 \text{ kN} > 3\,451 \text{ kN}$$

OK., purlin are still protected from flexural buckling even if sheeting is connected to beam at every second rib only.

Example 1c

Corrugated sheet, prevention of lateral buckling of purlin

Corrugated sheet T 18

$t = 0,88 \text{ mm}$

$h = 100 \text{ mm}$

Purlins: IPE 300

$W_{y, pl} = 628,4 \text{ cm}^3$

$J_{z, el} = 604 \text{ cm}^4$

S 235

		Podstawowe wymiary blach	Grubość mm	Masa kg/m ²
Typ	T 18		0,55	5,7
			0,75	7,9
			0,88	9,2
Typ	T 55		0,75	9,1
			0,88	10,7
			1,00	12,1
			1,25	15,1

One span, $l = 6,0 \text{ m}$

Distance between purlins $s = 2,0 \text{ m} = 2\ 000 \text{ mm}$

Photo: W. Bogucki, M. Żybartowicz, „Tablice do projektowania konstrukcji metalowych”, Arkady 1996

Flexural stiffness, influence on protection against lateral buckling

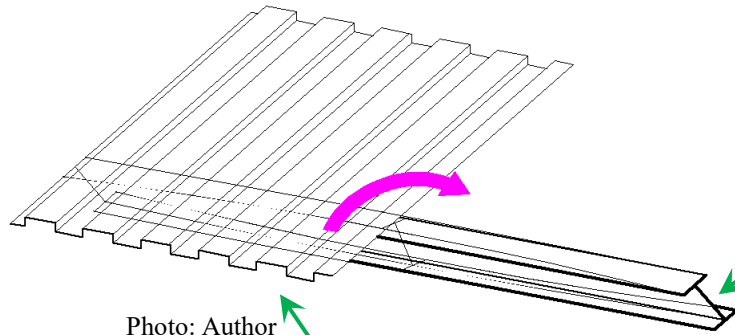


Photo: Author

EN 1993-1-1 (BB.3):

$$S_{\text{member}} = M_{\text{pl}}^2 K_D K_U / E J_z$$

$K_U = 0,35$ (elastic analysis)

$K_U = 1,00$ (plastic analysis)

$K_D \rightarrow \#t / 59$

EN 1993-1-1 (BB.3):

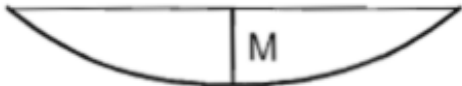




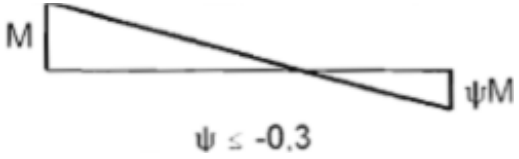
$$C_{\theta, k} \geq S_{\text{member}}$$

EN 1993-1-1 (BB.4), EN 1993-1-3 (10.11):

$$1 / C_{\theta, k} = 1 / C_{\theta R, k} + 1 / C_{\theta C, k} + 1 / C_{\theta D, k}$$

$C_{\theta R, k}$ - rotational stiffness of sheeting; $C_{\theta C, k}$ - rotational stiffness of connectors (bolts); $C_{\theta D, k}$ - distortional stiffness of member.

EN 1991-1-1 tab BB.1

Case	Moment distribution	K_D	
		Without translational resistant	With translational resistant
1		4,0	0,0
2a		3,5	0,12
2b			0,23
3		2,8	0,0
4		1,6	1,0
5		1,0	0,7

Eurocode recommendations for checking torsional stiffness contain numerous inconsistencies:

- EN 1993-1-1: stiffness of member is calculated as for cross-section Ist or IInd class (plastic bending moment);
- EN 1993-1-3: by default it assumes the same stiffness, although Eurocode applies to cold-formed purlins;
- EN 1993-1-1 focuses on issue of securing purlin cross-section against torsion (stiffness has the unit [N / rad]);
- EN 1993-1-3 focuses on support of purlin chords (stiffness has the unit of [N/m²]);
- It is not entirely clear whether, in methodology of these formulas, full protection against lateral buckling requires simultaneous protection against bending and torsion, or whether protection against bending alone is sufficient;
- Assumption that lack of torsion protection does not prevent lateral buckling is assumption on „safe side”.

EN 1993-1-3 (10.14):

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

EN 1993-1-3 (10.16):

$$C_{D,C} = k E J_{\text{eff}} / s$$

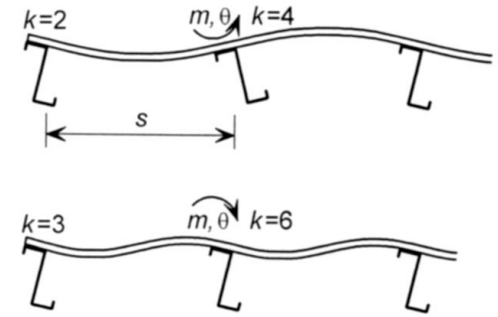


Photo: EN 1993-1-3 fig. 10.7

EN 1993-1-3 10.1.5.2.(4):

$C_{D,A} = 130 p$ [Nm / m / rad]; p - number of sheet-to-purlin fasteners per metre length of purlin

Symbols:

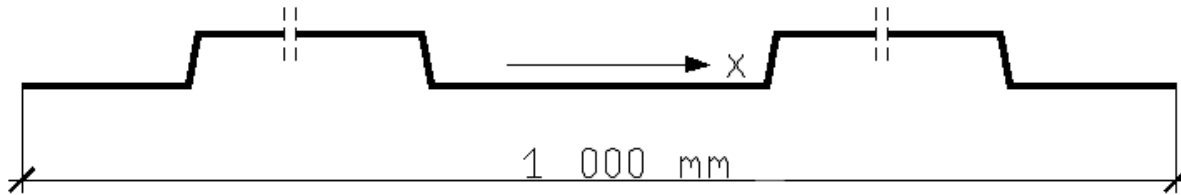
$C_{\theta, k}$ in EN 1993-1-1 (BB.3) = C_D in EN 1993-1-3 (10.14)

Photo: Author

$$J_{x,roofing} = 3,7 \text{ cm}^4$$

$$J_{eff} = J_{x,roofing} / 1 \text{ m} = 0,037 \text{ cm}^3$$

$$C_{D,C} = k E J_{eff} / s = 23\,310 \text{ [N]} \quad (k = \max = 6)$$



$$C_{D,A} = 130 p \text{ [Nm / m]}; \quad p: 2 \cdot 1000 / 72 = 28 \text{ bolts / 1 m of purlin's length}$$

$$C_{D,A} = 3\,640 \text{ [N]}$$

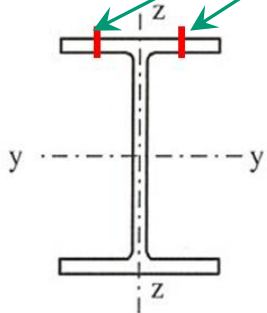
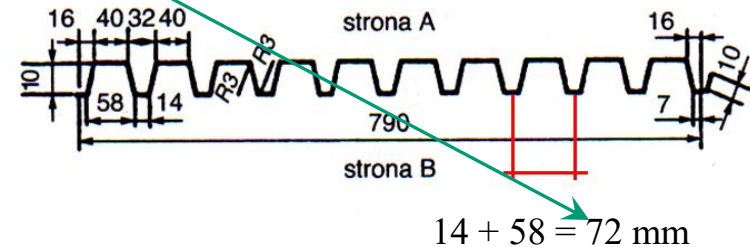


Photo: Author



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

$$C_D = 3\,148 \text{ [N]}$$

Photo: W. Bogucki, M. Żybertowicz, „Tablice do projektowania konstrukcji metalowych”, Arkady 1996

Purlin:

$$M_{pl} = f_y W_{y, pl} = 147,674 \text{ kNm}$$

$$K_U = 0,35 \text{ (elastic analysis)}$$

$$K_D = 3,5 \text{ (as for single-span beam)}$$

$$M_{pl}^2 K_D K_U / E J_z = 21\,072 \text{ [N]}$$

$$C_D = 3\,148 < M_{pl}^2 K_D K_U / E J_z = 21\,072$$

Wrong, purlins are not protected.

According to adopted estimate, protection against bending and lack of protection against twisting do not protect against lateral buckling.

Example 1d

Corrugated sheet, increasing stiffness of purlin

Corrugated sheet is protection for:		Conclusion for protected beam
bending	torsion	
Yes ($\chi_y = 1,0$)	Yes ($\chi_{LT} = 1,0$)	Member is totally protected.
No ($\chi_y < 1,0$)	Yes ($\chi_{LT} = 1,0$)	Such result is very very unlikely, there is a high probability of mistake in calculations.
Yes ($\chi_y = 1,0$)	No ($\chi_{LT} < 1,0$)	Member is partially protected; there is need to calculate lateral buckling under conditions of forced rotation axis
No ($\chi_y < 1,0$)	No ($\chi_{LT} < 1,0$)	Member is not protected, interaction between flexural buckling and lateral buckling must be calculated (\rightarrow Lec. #13)

„Not protected” means buckling factor $\chi < 1,0$. But is possible, than despite $\chi < 1,0$ corrugated sheet increase stiffness sufficiency to prevent instability.

Buckling factor $\chi = 1,0$; critical resistance of member under buckling the same as **resistance of cross-section**

Generally „not protected” ($\chi < 1,0$) but **presence of corrugated sheet increases stiffness of beam** and its critical resistance over level of external action – in real it means „protected”.

Generally „not protected” ($\chi < 1,0$) and increasing of stiffness of beam is not sufficient

Critical resistance of member under buckling **without anti-buckling protection (bar or plate)** – own stiffness of member only, which comes from its length, types of supports and moments of inertia of cross-section.

Photo: Author

Distinction between „red not protected” and „green not protected” will be presented on the calculation example from Lec #5, example #2. General conclusion was, that supports at both ends of beam and one bracing in half of length not protected against lateral buckling; two bracings every each third part of beam were enough.

$$M_{Ed} / \chi_{LT, mod} W_{pl, y} f_y = 1,217 \quad \text{Wrong}$$

$$M_{Ed} / \chi_{LT, mod} W_{pl, y} f_y = 0,845 \quad \text{OK.}$$

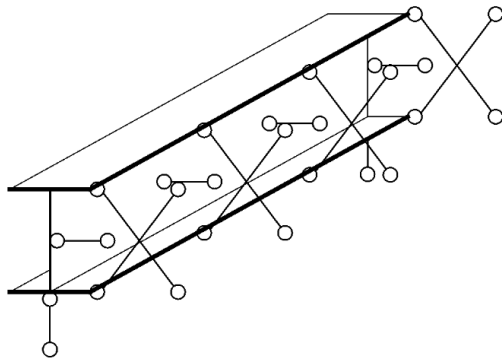
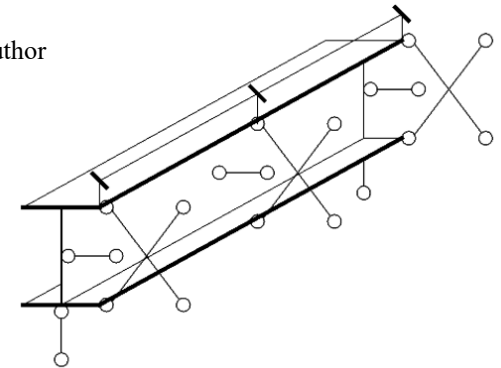


Photo: Author

Photo: Author



What about co-operation between beam and corrugated sheet?

Existence of corrugated sheet changes behavior of beam. Lateral buckling should be calculated as for forced rotation axis, formulas d, #Lec 5 / 67:

$$M_{cr} = (i_s^2 N_{cr, T} + c_y^2 N_{cr, z}) / [C_1 (c_y - b_y) + C_2 (c_y - z_g)]$$

$$N_{cr, z} = 347,603 \text{ kN}$$

$$i_s = 12,90 \text{ cm}$$

$$N_{cr, T} = 1\,463,001 \text{ kN}$$

Geometry (#5 / 65):

y_s - position of shear centre, for I-beam = 0

z_g - distance between shear centre and point of applying load,

for this situation = $h / 2 = 150 \text{ mm}$

$$r_x \text{ (#5 / 65, I-beam #5 / 37)} = 0$$

$$\text{Bending parameter: } b_y = y_s - r_x / 2 = 0 - 0 / 2 = 0$$

c_y - distance between centre of gravity and position of bracings,

for this situation = $h / 2 = 150 \text{ mm}$

$$C_1 \quad C_2 \text{ (#5 / 68)}$$

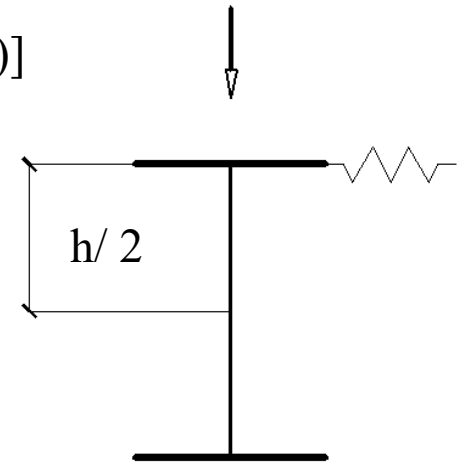


Photo: Author

Parameters calculated for beam with supports and bracings:

$$C_1 = 0,93$$

$$C_2 = 0,81$$

Photo: PN B 3200

(approximation: case for continuous load)

Results for such situation are as follows:

$$M_{cr} = 230,587 \text{ kNm}$$

$$W_{y, pl} f_y = 147,674 \text{ kNm}$$

$$\lambda_{LT} = 0,800$$

$$\Phi_{LT} = 1,429 \text{ (general formula, \#5 / 85)}$$

$$\chi_{LT} = 0,883$$

$$\chi_{LT, mod} = 0,852 \text{ (\#5 / 90)}$$

$$\chi_{LT, mod} W_{pl, y} f_y = 125,818 \text{ kNm}$$

$$M_{Ed} / \chi_{LT, mod} W_{pl, y} f_y = 0,636 \text{ OK}$$

Obciążenie belki (w płaszczyźnie symetrii przekroju YZ)	Warunki podparcia ¹⁾				Współczynniki				
	w płaszczyźnie		μ_y	μ_{z0}	A_1	A_2	B	C_1	C_2
	YZ	XZ							
Obciążenie równomiernie rozłożone	P	P	1	1	0,61	0,53	1,14	0,93	0,81
	P	P	1	0,5	1,23	0,52	1,31	-	-
	P	U	0,5	0,5	0,68	0,29	0,97	1,43	0,61
	U	U	0,5	0,5	0,27	1,61	1,88	0,15	0,91

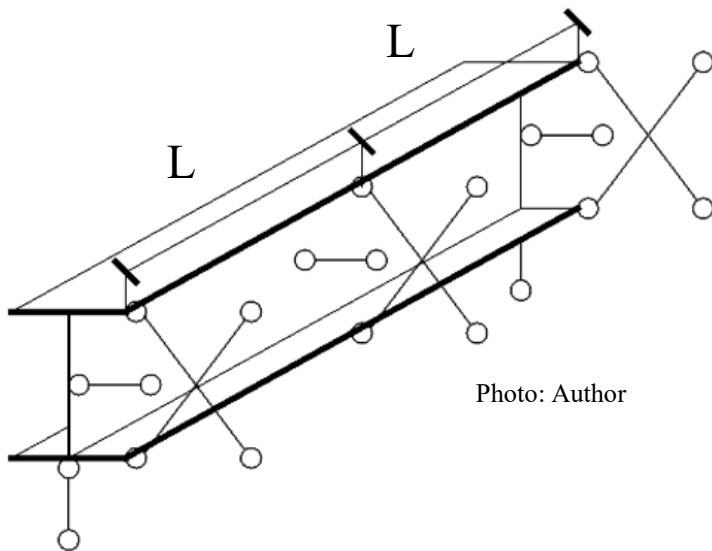
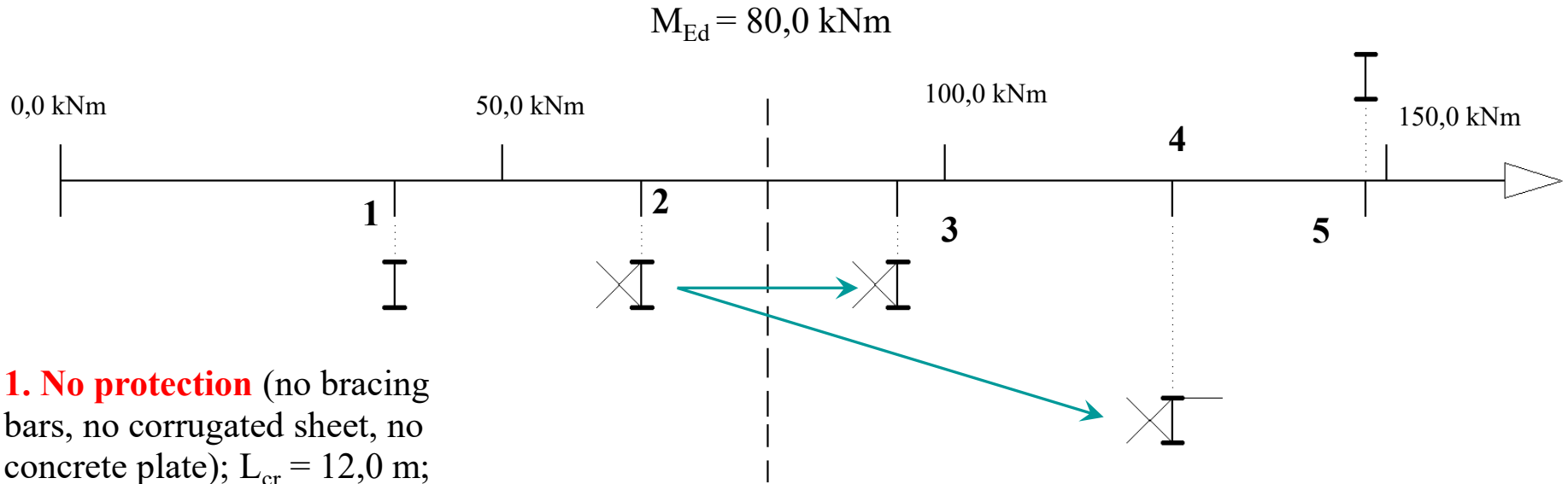


Photo: Author

Conclusions: influence of various types of bracings on loadbearing. Even partial protection from corrugated sheet could be enough in case of lateral buckling.

5. Full protection, $\chi_{LT, mod} = 1,0$
(concrete plate or very small distance between bracing bars or massive corrugated sheet);

$$M_{Rd} = 147,674 \text{ kNm}$$



$$M_{Ed} = 80,0 \text{ kNm}$$

1. No protection (no bracing bars, no corrugated sheet, no concrete plate); $L_{cr} = 12,0 \text{ m}$;

$$\chi_{LT, mod} M_{Rd} = 37,786 \text{ kNm}$$

2. Partial protection, #5 example 2, bracing bars $2 \times 6,00 \text{ m}$;

$$\chi_{LT, mod} M_{Rd} = 65,734 \text{ kNm}$$

3. Partial protection, #5 example 2, bracing bars $3 \times 4,00 \text{ m}$;

$$\chi_{LT, mod} M_{Rd} = 94,711 \text{ kNm}$$

4. Partial protection, #t example 1d, corrugated sheet, partial protection, forced rotation axis, cooperation with bracing bars $2 \times 6,00 \text{ m}$;

$$\chi_{LT, mod} M_{Rd} = 125,818 \text{ kNm}$$

Example 2

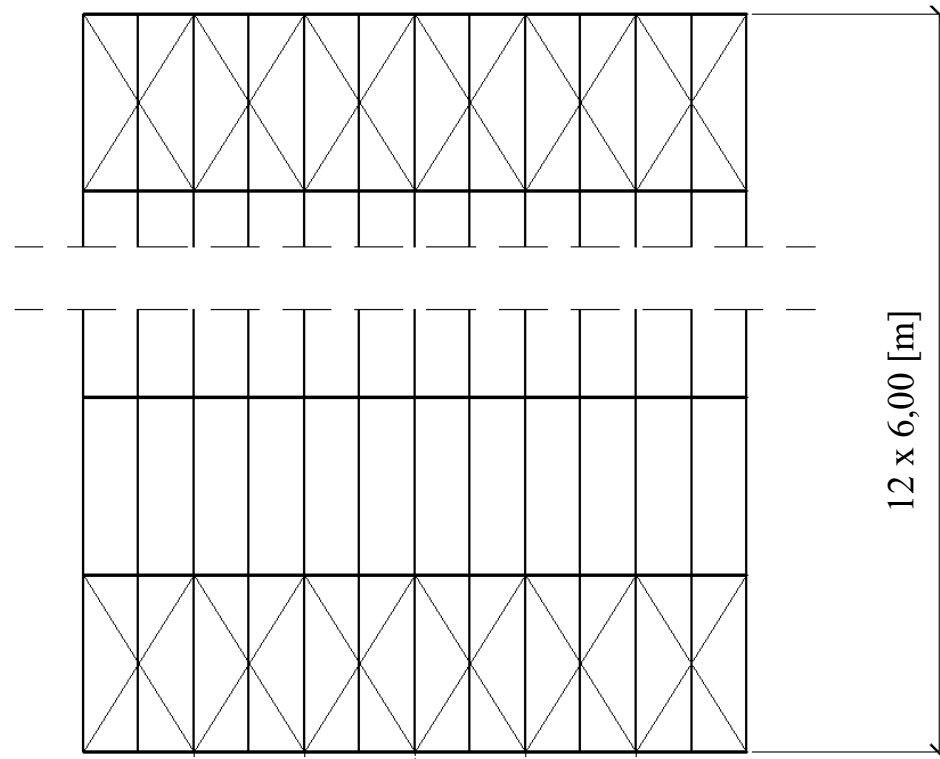
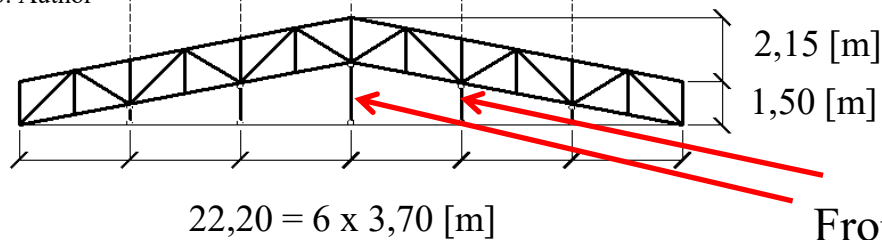


Photo: Author



Front wall columns (on first and last truss only)

Example 2a

Chord	Max tension [kN]	Max compression [kN]	Cross-section
Top	-55,112	302,157	CHS 108x10
Bottom	-304,853	52,789	

Purlins IPE 270 → Des Ex#1 / 45

S 235 → $f_y = 235$ MPa

Wind load:

- winward wall 0,8 kPa
- roof slope 0,7 kPa
- side walls 0,6 kPa
- leeward wall 0,5 kPa

Example is made for 2D model.

$$S\ 235 \rightarrow f_y = 235\ \text{MPa}$$

$$\varepsilon = \sqrt{(235 / f_y)} = 1,0$$

S 235 + CHS hot rolled →
buckling curve a → $\alpha = 0,21$
(EN 1993-1-1 tab. 6.1, 6.2)

Hot-rolled CHS 108x10 (108 mm diameter, 10 mm thickness):

$$A = 30,8\ \text{cm}^2$$

$$J_y = J_z = 373,0\ \text{cm}^4$$

$$i_y = i_z = 3,48\ \text{cm}$$

Wind pressure and wind suction of both front walls are beared by roof bracings.

Roof bracings prevent trusses chords from buckling → buckling equivalent forces from chords act on bracings.

Trusses chords are not ideal straight (imperfection) → imperfection equivalent forces act on roof bracings.

Wind action is sum of:

- wind pressure on on winward wall = wind load · part area of front wall;
- wind friction on roof slope = wind action · friction factor · part area of roof slope +
+ wind action · friction factor · part area of side walls;
- wind suction on leeward wall = wind load · part area of front wall;

→ Des Ex#1 / 56

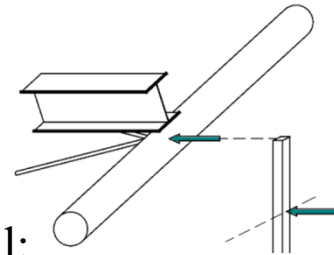
side walls:

- „dead part”, turbulent wind, no friction;
- top part, load applicated to roof bracings – „friction part”;
- bottom part, load applicated to truss bases;

roof slope:

- „dead part”, turbulent wind, no friction;
- „friction part”;

Photo: Author



winward wall:

- top part, load applicated to roof bracings;
- bottom part, load applicated to front wall column bases;

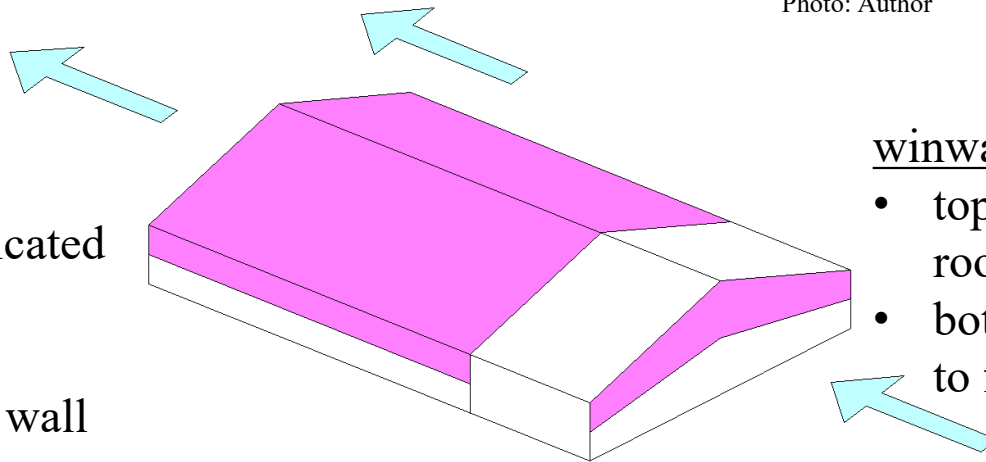
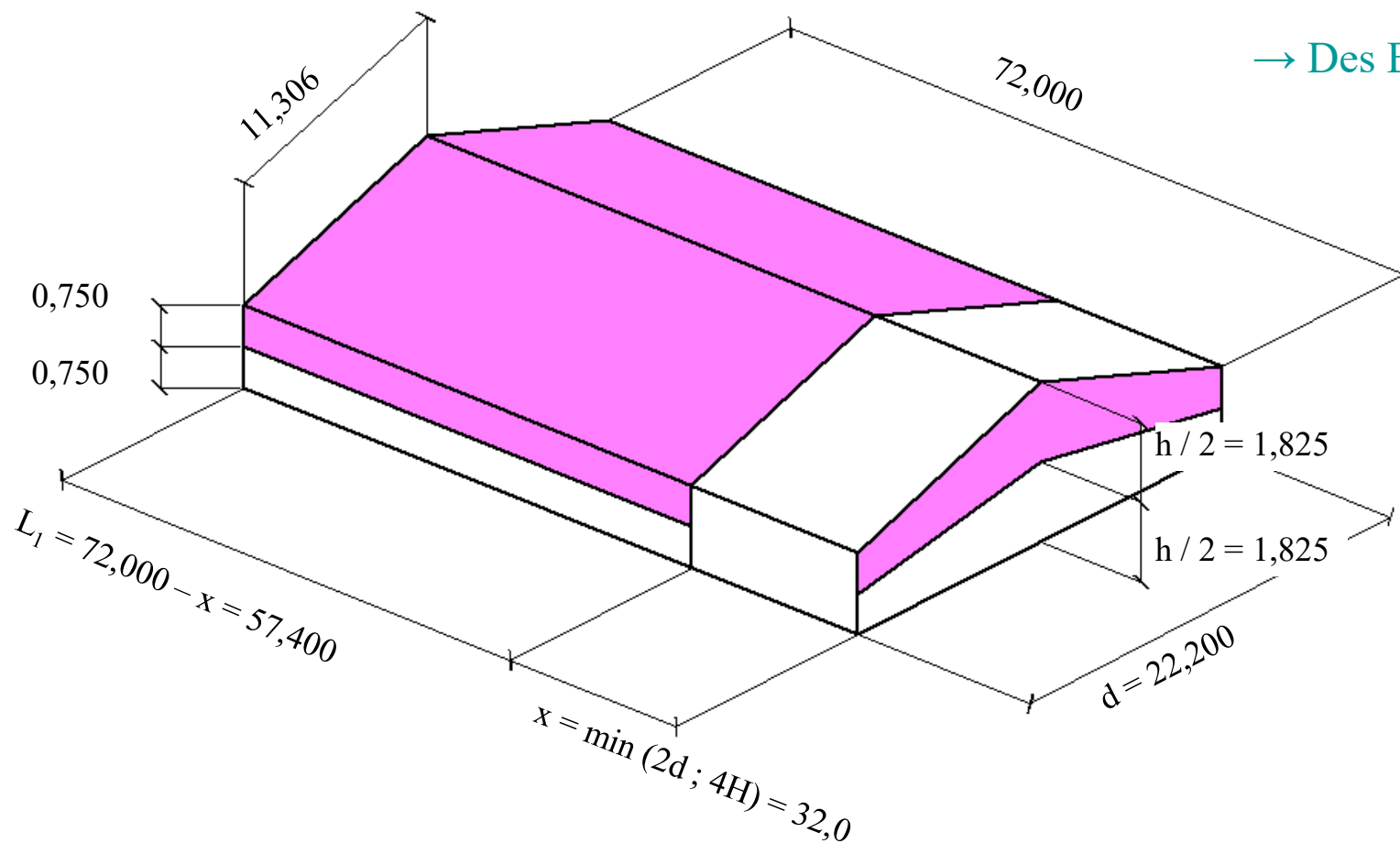


Photo: Author

leeward wall:

- top part, load applicated to roof bracings;
- bottom part, load applicated to front wall column bases;



Geometry of turbulent area (length x) according to EN 1991-1-4 7.5 (h – height of truss = 3,65 m, H – total height of building: wall + truss = 8,0 m)

Surface	c_{fr}
steel, smooth concrete	0,01
rough concrete, tar-boards	0,02
ripples, ribs, folds	0,04

Friction coefficients: EN 1991-1-4 tab 7.10

Winward front wall **active part**: approximation, half of total area of front wall = 28,583 m²

Roof slope **active part**: 1 297,929 m²

→ Des Ex#1 / 58

Side walls **active part**: 60,000 m²

Leeward front wall **active part**: approximation, half of total area of front wall = 28,583 m²

$$\begin{aligned} \text{Wind total} &= A_{\text{wiward}} \cdot q_{w, \text{wiward}} + A_{\text{slope}} \cdot q_{w, \text{slope}} \cdot c_{fr} + A_{\text{side}} \cdot q_{w, \text{side}} \cdot c_{fr} + A_{\text{leeward}} \cdot q_{w, \text{leeward}} = \\ &= 28,583 \cdot 0,8 + 1\,297,929 \cdot 0,7 \cdot 0,01 + 60,000 \cdot 0,6 \cdot 0,01 + 28,583 \cdot 0,5 = 46,761 \text{ kN} \end{aligned}$$

Total force is divided between nodes of horizontal truss; simplification: even division.

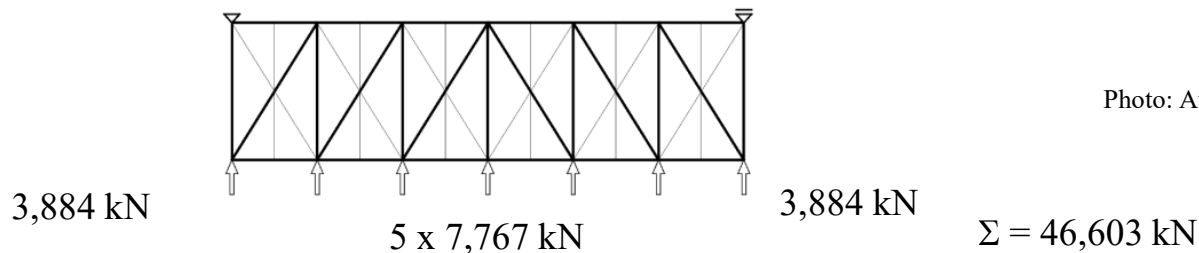
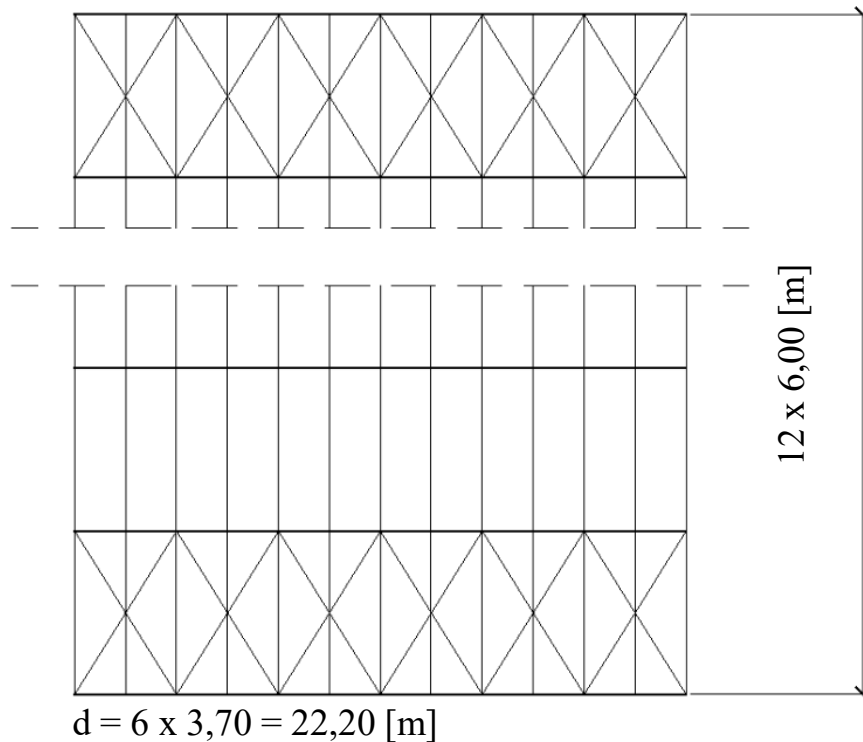


Photo: Author

12 bands between trusses, 13 trusses. 2 braced bands in roof $\rightarrow m = 13 / 2 = 6,5$ trusses for one braced band.



Reduction factor for effects from instability and imperfections (EN 1993-1-1 (5.12)):

$$\alpha_m = \sqrt{0,5 (1 + 1 / m)}$$

$$m = 6,5$$

$$\alpha_m = 0,760$$

\rightarrow Des Ex#1 / 61

Max compressive force in top chord of vertical truss: $N_{Ed, c} = 302, 157 \text{ kN}$ (\rightarrow #Des Ex#1 / 45)

Buckling equivalent force from truss chord to bracings (EN 1993-1-1 5.3.3.(4)):

$$F_{buck} = \alpha_m N_{Ed, c} / 100 = 2,296 \text{ kN}$$

Photo: Author

Initial horizontal bow imperfection for truss chord (EN 1993-1-1 (5.12)):

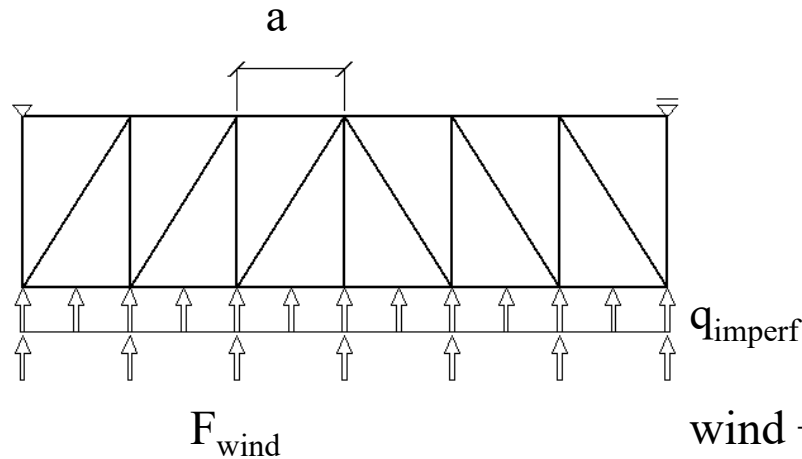
$$e_0 = \alpha_m d / 500 = 33,744 \text{ mm.}$$

Initial bow imperfection is recalculated to equivalent forces according to specific algorithm.

Two alternative schemes of action are taken into consideration:

- wind and equivalent effects from buckling;
- wind and equivalent effects from imperfections.

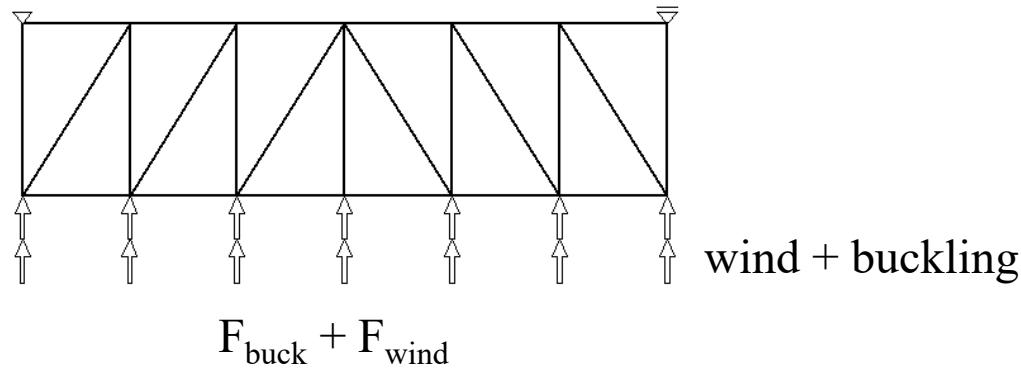
For final analysis more important will be bigger value of both above.



q_{imperf} is of course applied to structure as:

$$F_{imperf} = a \cdot q_{imperf}$$

wind + imperfection



→ Des Ex#1 / 62

Photo: Author

Calculation of equivalent loads from imperfection q_{imperf} for roof bracings is complicated. According to EN 1993-1-1 (5.13), total equivalent load (from wind and imperfection) is calculated as:

$$q_{\text{imperf}} = q_d = \Sigma [8 N_{\text{Ed}, c} (e_0 + \delta_q) / L^2]$$

d_q – deformation comes from load q_d

Total load q_d depends on d_q , but d_q is the effect of q_d . Such problem can be solved by iteration procedure:

$$\delta_q^0 = \delta(q_q) = \delta_q = \text{deflection from wind only} = \delta_w$$

$$q_d^{(0)} = q_d^{(0)}(e_0 + \delta_q) = \Sigma [8 N_{\text{Ed}, c} (e_0 + \delta_q^{(0)}) / L^2]$$

→ Des Ex#1 / 63

$$\delta_q^{(1)} = \delta_q^{(1)}(q_d^{(0)}) \text{ static calculation: deflection}$$

$$q_d^{(1)} = q_d^{(1)}(e_0 + \delta_q^{(1)} + \delta_w) = \Sigma [8 N_{\text{Ed}, c} (e_0 + \delta_q^{(1)} + \delta_w) / L^2]$$

$$\delta_q^{(2)} = \delta_q^{(2)}(q_d^{(1)}) \text{ static calculation: deflection}$$

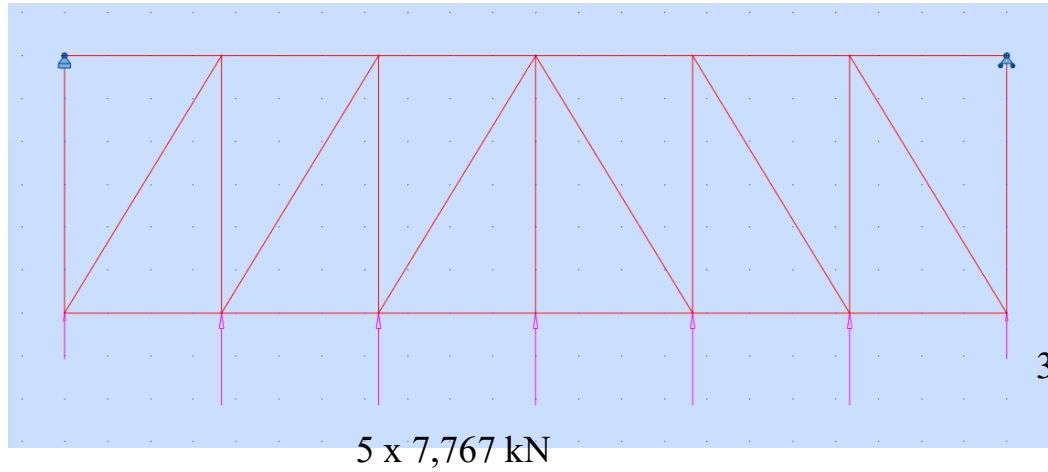
$$q_d^{(2)} = q_d^{(2)}(e_0 + \delta_q^{(2)} + \delta_w) = \dots$$

...

q_d is of course applied to structure as:

$$F_d = a \cdot q_d$$

Iteration: static calculations of bracings must be made few times for iteration procedure.



Step 0: deformation for wind only

Photo: Author

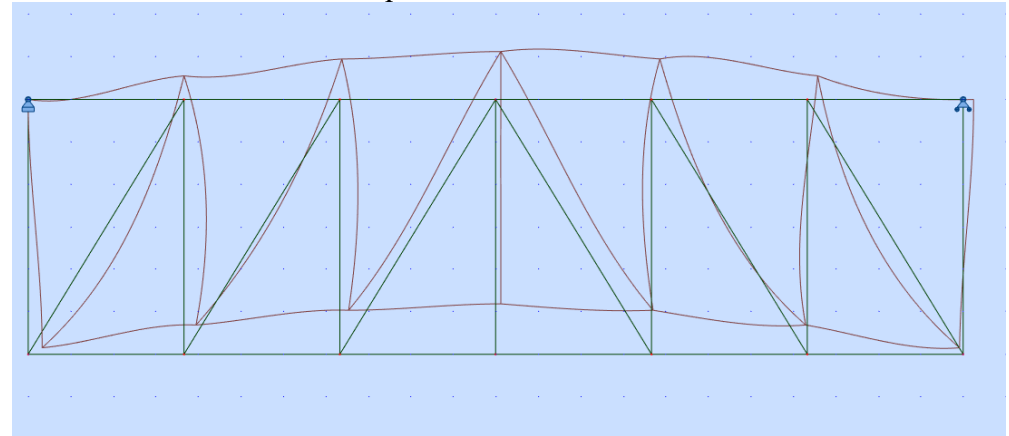
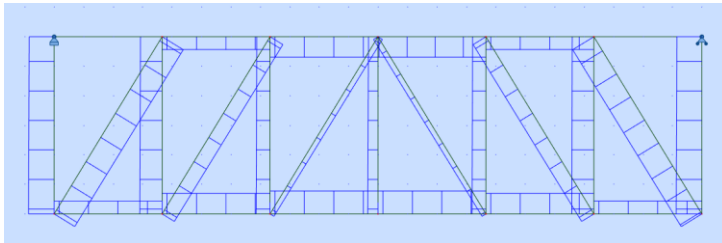
→ Des Ex#1 / 64

3,884 kN

3,884 kN

$5 \times 7,767 \text{ kN}$

$$d_q^0 = d(q_{\text{wind}}) = 2 \text{ mm}$$



Steps of iteration:

$$\delta_q^0 = \delta(q_{wind}) = 4 \text{ mm}$$

$$q_d^{(0)} = \Sigma [8 N_{Ed, c} (e_0 + \delta_{wind}) / L^2] =$$

$$= 6 \cdot 8 \cdot 302,157 \text{ [kN]} \cdot (33,744 \text{ [mm]} + 2 \text{ [mm]}) / (22,2 \text{ [m]})^2 = 1,052 \text{ kN / m}$$

→ force = 3,70 · 1,052 = 3,892 kN

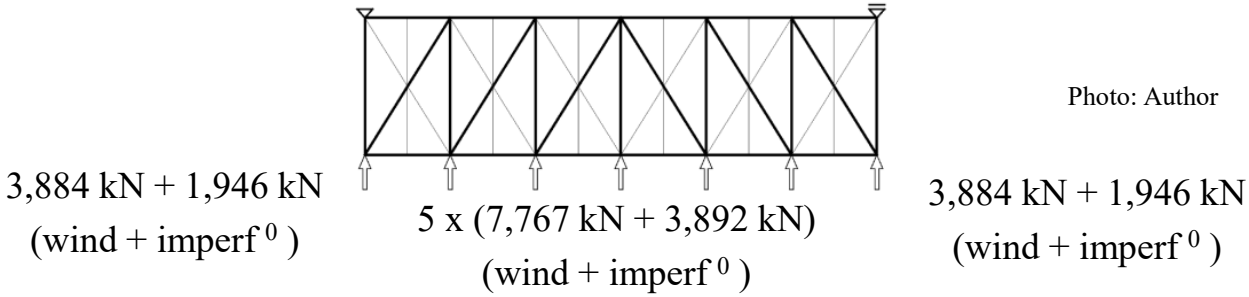


Photo: Author

Static calculation of deformation

$$\delta_q^{(1)} = \delta_q^{(1)}(q_d^{(0)}) = 3 \text{ mm}$$

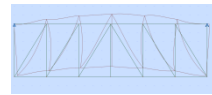


Photo: Author

$$q_d^{(1)} = \Sigma [8 N_{Ed, c} (e_0 + \delta_q^{(1)} + \delta_{wind}) / L^2] =$$

$$= 6 \cdot 8 \cdot 302,157 \text{ [kN]} \cdot (33,744 \text{ [mm]} + 4 \text{ [mm]} + 3 \text{ [mm]}) / (22,2 \text{ [m]})^2 = 1,199 \text{ kN / m}$$

→ force = 3,70 · 1,199 = 4,436 kN

Steps of iteration:

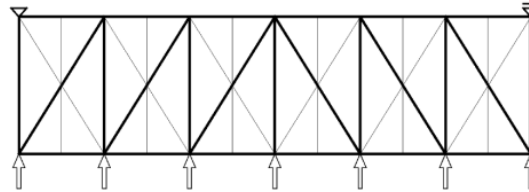


Photo: Author

$$3,884 \text{ kN} + 2,218 \text{ kN}$$

(wind + imperf¹)

$$5 \times (7,767 \text{ kN} + 4,436 \text{ kN})$$

(wind + imperf¹)

$$3,884 \text{ kN} + 2,218 \text{ kN}$$

(wind + imperf¹)

Static callculation of deformation

$$\delta_q^{(2)} = \delta_q^{(2)}(q_d^{(1)}) = 3 \text{ mm}$$

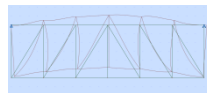
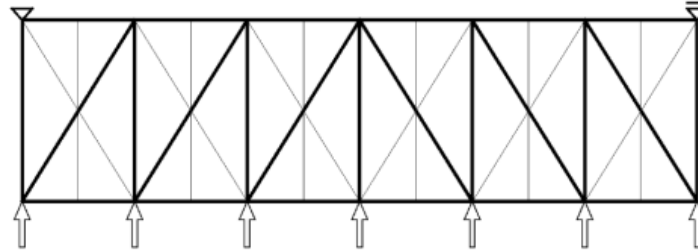


Photo: Author

No difference, end of iteration.

Two combinations must be taken into consideration: wind + equivalent effects of buckling:

Photo: Author



$$3,884 \text{ kN} + 1,149 \text{ kN} = \underline{5,033 \text{ kN}}$$

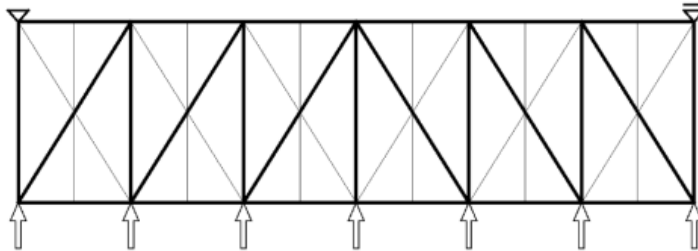
$$3,884 \text{ kN} + 1,149 \text{ kN} = \underline{5,033 \text{ kN}}$$

$$5 \times (7,767 \text{ kN} + 2,296 \text{ kN}) = 5 \times \underline{10,063 \text{ kN}}$$

→ Des Ex#1 / 67

Wind + equivalent effects of imperfections:

Photo: Author



$$3,884 \text{ kN} + 1,946 \text{ kN} = \underline{5,830 \text{ kN}}$$

$$3,897 \text{ kN} + 1,946 \text{ kN} = \underline{5,830 \text{ kN}}$$

$$5 \times (7,767 \text{ kN} + 3,892 \text{ kN}) = 5 \times \underline{11,659 \text{ kN}}$$

Forces for the second one are bigger → the second is important for calculations of horizontal bracing.

Results:

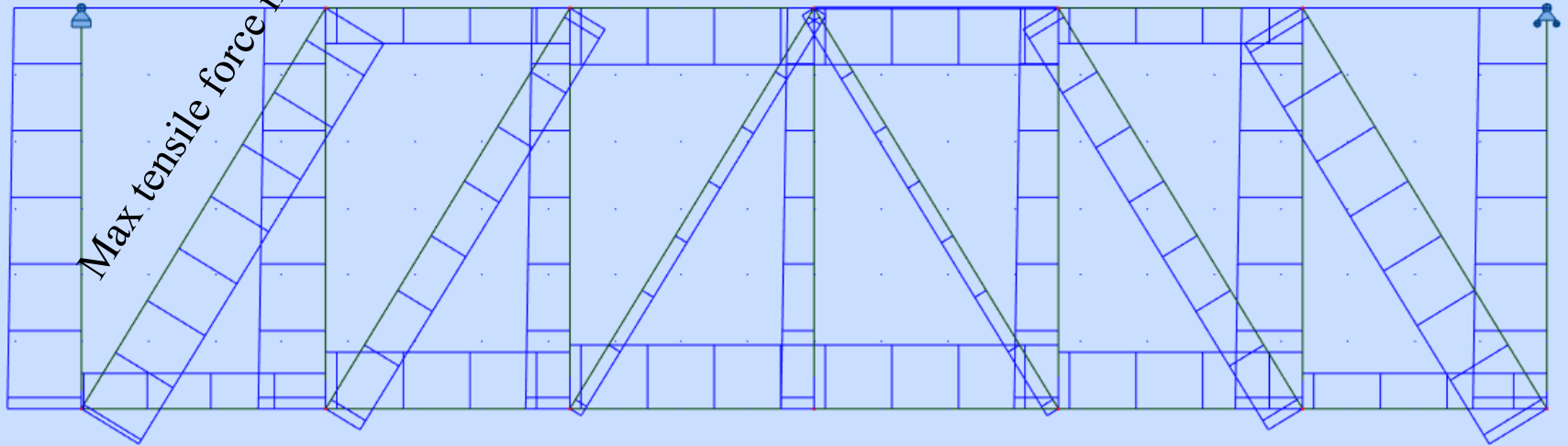
Max compressive force in purlin: $N_{Ed, c, p} = 24,17 \text{ kN}$

Max tensile force in bracing: $N_{Ed, t, b} = 22,02 \text{ kN}$

Photo: Author

Max tensile force in chord: $N_{Ed, c, ch} = 18,46 \text{ kN}$

Max compressive force in chord: $N_{Ed, t, ch} = 20,77 \text{ kN}$



Checking of resistance: round bar (tensile force only) ϕ 25

$$N_{Rd, bb} = f_y A / \gamma_{M0} = 115,355 \text{ kN}$$

$$N_{Ed, c, bb} / N_{Ed, bb} = 0,191 \quad \text{👍}$$

→ Des Ex#1 / 69

Checking of resistance: purlin

Purlin was calculated as bi-axial bending element (→ Des Ex #1 / 18-26). At now, should be recalculated to case „bi-axial bending and compressive force”. In Your range of project it is enough to ensure requirement $E / R \leq 0,8$ (→ Des Ex #1 / 28).

Chords of roof bracings.

Both chords of roof bracings (horizontal truss) are simultaneously top chords of main trusses (vertical trusses). New values of forces in trusses must be taken into consideration as additional forces applied to trusses.

→ Des Ex#1 / 70

Simplification for new values of forces:

Max compressive force from bracings (20,77 kN) together with combination of loads which gives max compression in top chord (302,157 kN).

Max tensile force from bracings (18,46 kN) together with combination of loads which gives max tension (or min compression) in top chord (55,112 kN).

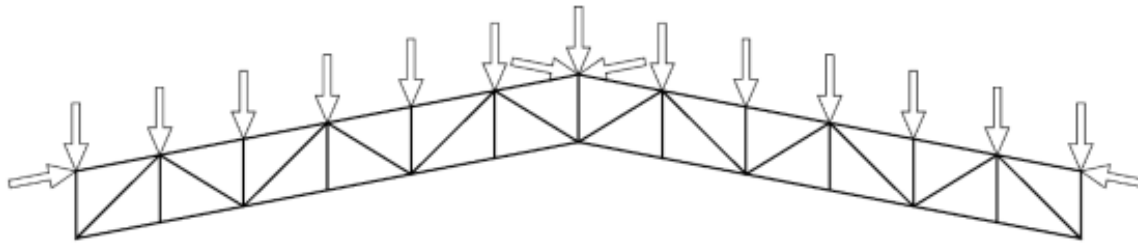
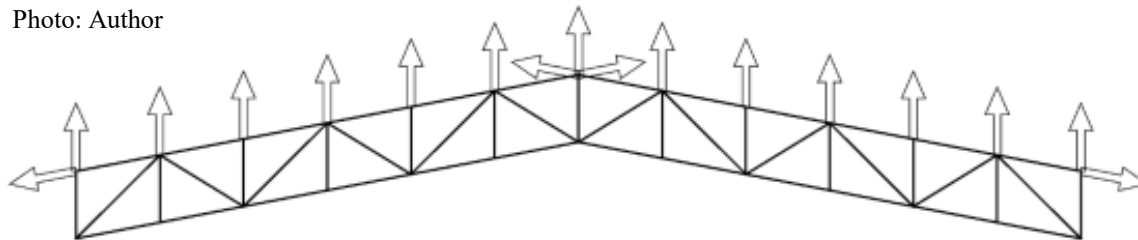


Photo: Author



Simplification: in fact, changes will be smaller, because bottom chord cooperates with top chord and will take over some of new forces.

Initial situation:

Chord	Max tension [kN]	Max compression [kN]	Cross-section
Top	-55,112	302,157	Hot-rolled CHS 108x10
Bottom	-304,853	52,789	

Change as the effect of cooperation with bracigs:

Max compressive force in chord from bracings: $N_{Ed, c, ch} = 20,77$ kNMax tensile force in chord from bracings : $N_{Ed, t, ch} = -18,46$ kN

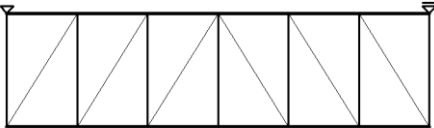
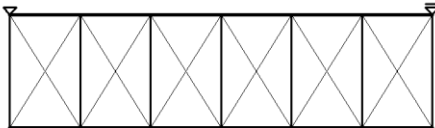
New values of forces:

Chord	Max tension [kN]	Max compression [kN]	Cross-section
Top	-73,572	322,927	Hot-rolled CHS 108x10
Bottom	-304,853	52,789	

Example 2b

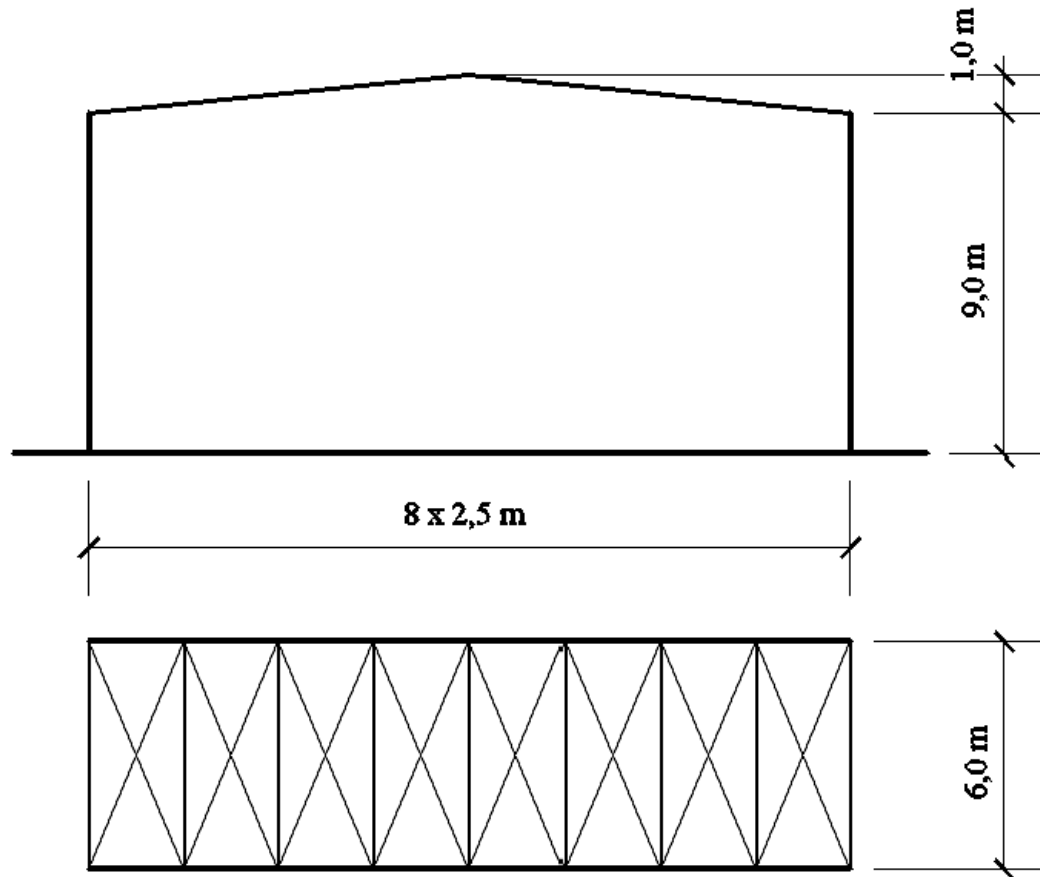
Comparison between non-rigid bracings and rigid bracings.

Type of bracings is initial assumption in calculation. Each next step of calculation must be consistent with this assumption. Appropriate model of bracings must be applied on each step of calculation.

Bracings	Non-rigid, round bar ϕ 25	Rigid CHS 60,3x5
Static model		
Max force in bracings	Tens: 22,02 kN	Tens: 8,89 kN Comp: 11,12 kN
Max compressive force in purlin	24,17 kN	11,66 kN
Change of forces in chord of vertical truss	Tens: 18,46 kN Comp: 20,77 kN	Tens: 17,91 kN Comp: 18,20 kN

Example 2c

Horizontal upper transversal bracings I-beam girders



Purlin: one-span simple-supported
IPE 210

$$M_{Ed, y} = 26,865 \text{ kNm}$$

$$M_{Ed, z} = 2,687 \text{ kNm}$$

Roof girder HEA 550,

$$h_{HEA 500} = 0,54 \text{ m}$$

Wind action on front wall (wind pressure at first front wall + wind suction at the opposite front wall):

$$q_w = 0,8 \text{ kPa}$$

Photo: Author

Differences between example 1a (truss) and 1c (I-girder):

- other value of $F_{\text{buck}} = \alpha_m N_{\text{Ed, c, equ}} / 100$;
- height h according $t\# / 72$ concerns total height of structure, not only roof;
- other cross-section for chords of horizontal truss;
- anti-lateral buckling bracing are necessary;

Rest of calculation is the same.

$$N_{\text{Ed, c, equ}} = \max (N_{\text{Ed, comp}} ; M_{\text{Ed}} / h ; N_{\text{Ed, comp}} / 2 + M_{\text{Ed}} / h) =$$

$$= \max (140,0 ; 932,2 / 0,54 ; 140,0 / 2 + 932,2 / 0,54) =$$

$$= 1796,3 \text{ kN}$$

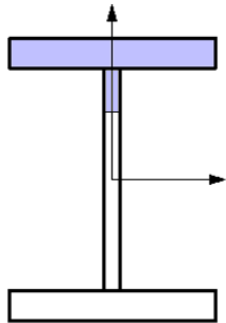
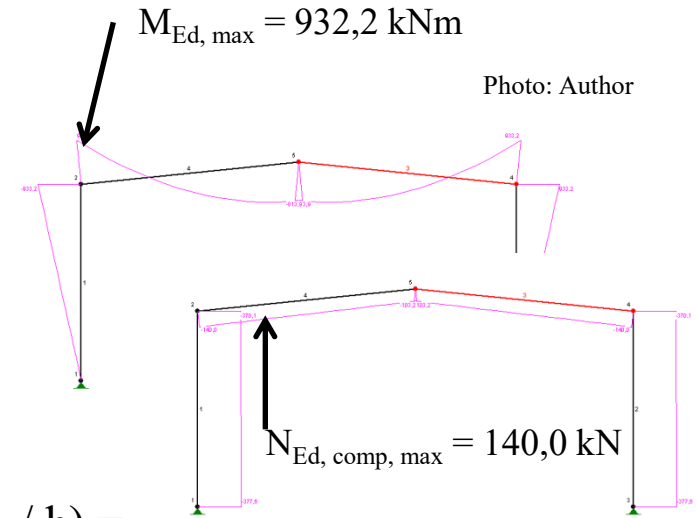


Photo: Author

Top flange of I-girder + 1/5 height of web =
= chord of horizontal truss

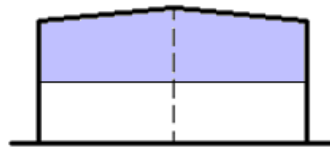


Photo: Author

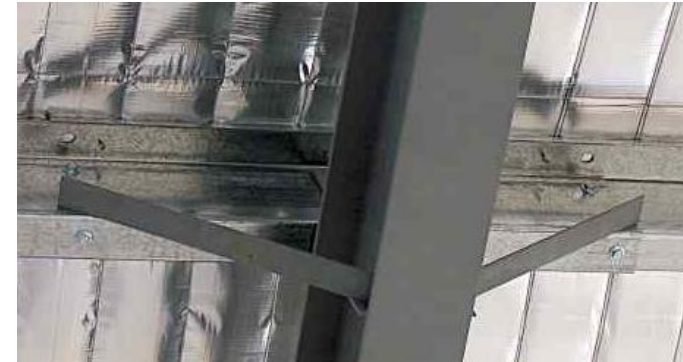


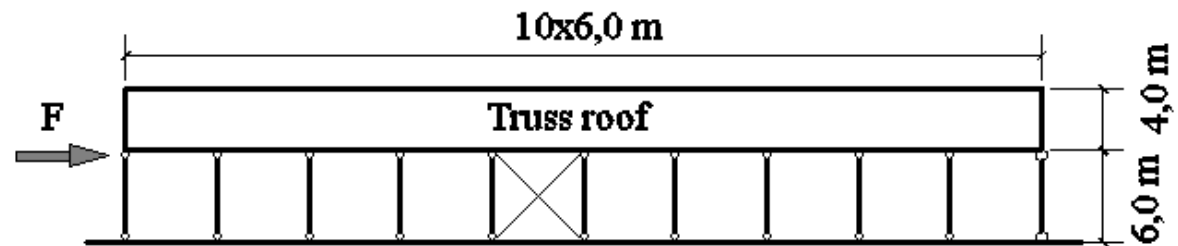
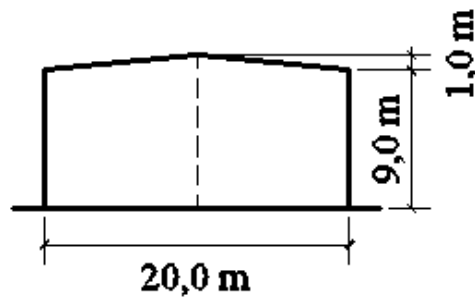
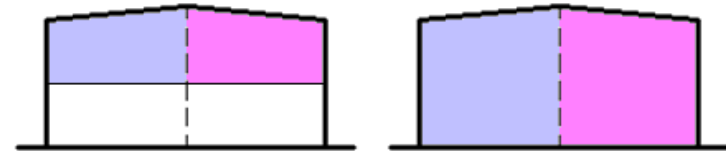
Photo: builderbill-diy-help.com

Example 3

Vertical wall bracings

Generally, wind action from bottom half of wall acts on foundations, from top half acts on purlins and vertical bracing system (\rightarrow #t / 72, 87). But there is assumption, that during calculation of side wall bracings, total wind action acts on bracing system.

Wind from this part acts on left wall roof bracings



Wind from this part acts on right wall roof bracings

$$F = F_{\text{wind}} + F_{\text{column-imperf}}$$

Photo: Author

Loads:

Imperfections:

Axial force in column $N_{Ed} = 160 \text{ kN}$

Number of columns in one wall $m = 11$

High of column $h = 6,0 \text{ m}$

$$F_{\text{column-imperf}} = N_{Ed} \Phi_0 \alpha_h \alpha_m$$

$$\Phi_0 = 1 / 200$$

$$\alpha_h = \max\{ 2 / 3 ; \min[(2 / \sqrt{h}) ; 1,0] \} = 0,814$$

h – heigh of column [m]

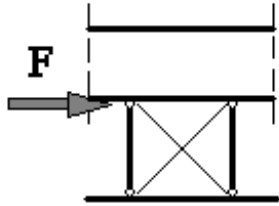
$$\alpha_m = \sqrt{[0,5 (1 + 1 / m)]} = 0,739$$

$$F_{\text{column-imperf}} = 0,481 \text{ kN}$$

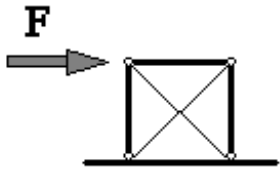
Wind action (2 wall bracings, on left and right side walls):

$$F_{\text{wind}} = (\text{pressure} + \text{suction of both front walls} + \text{total wind friction}) / 2 = 82,184 \text{ kN}$$

$$F = F_{\text{wind}} + F_{\text{column-imperf}} = 82,665 \text{ kN}$$



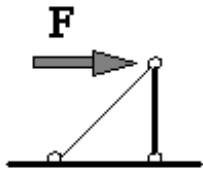
Horizontal force is transported to bases by bracings only - we analyse only one field between columns; columns-bracing system.



When we assume rigid bracings (tensile and compressive forces allow), we must analyse this type of truss.

Compressive force in bracing is equal 65,5 kN. Distance between ends of bracing bar is not greater than 6,0 m; we do not need analyse bending moment from dead-weight of bracings.

Additional compressive force in column 42,1 kN



When we assume non-rigid bracings (tensile force allows only), we must analyse this type of truss.

Tensile force in bracing is equal 116,91 kN. Distance between ends of bracing bar is not greater than 6,0 m; we do not need analyse bending moment from dead-weight of bracings.

Additional compressive force in column 82,67 kN

Photo: Author

Additional notices

Horizontal lower transversal bracings

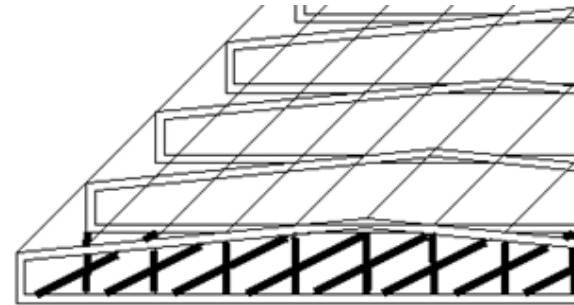


Photo: Author

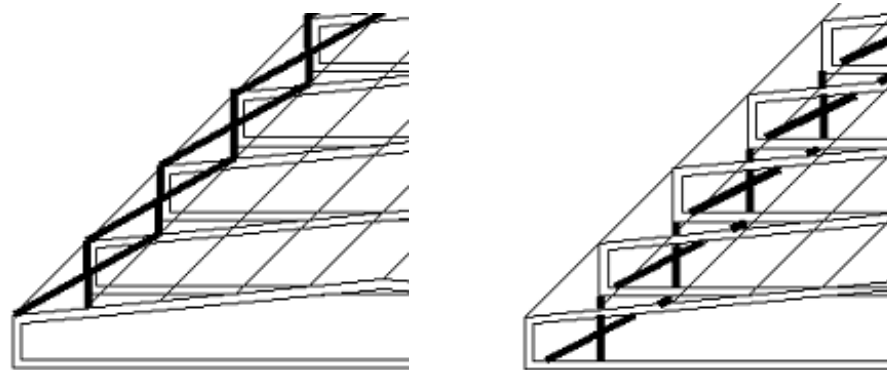
The same rules of calculation, as for horizontal upper transversal bracings. Equivalent forces – from instability of chord and from imperfections of chord – are calculated for characteristics of bottom chord. It could be replaced by inclined bracings.



Photo: builderbill-diy-help.com

Horizontal longitudinal bracings

Photo: Author



In modern structures, this type of bracing is dispensed with in halls without cranes. Without detailed calculations, it is assumed that their role is played by stiffness of roofing.



Photo: taxonomy.openquake.org

These bracings are essential in halls with cranes; then they must be counted in detail.

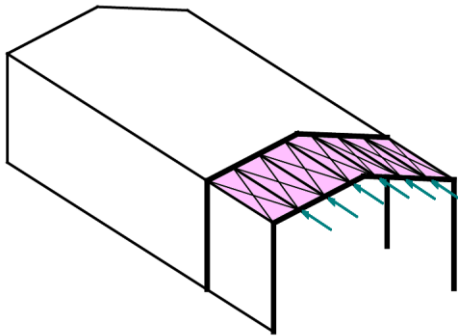
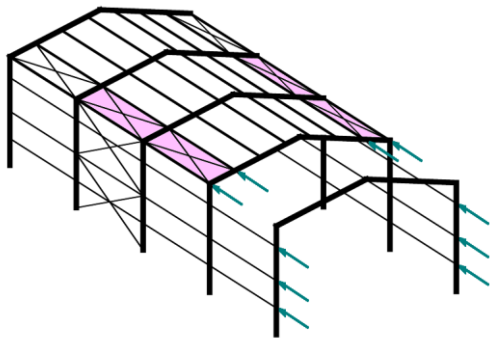


Photo: Author



Photo: U. Pawlak, M. Szczecina, The eigenvalue problem in the dynamics of the steel industrial hall with internal handling system, Structure and Environment, 2016 / 8, 160-167



It works under compression, first of all. So, this kind of bracings can be rigid only. Forces applied to them are equal full from front wall divided between two braced fields.

Vertical longitudinal bracings

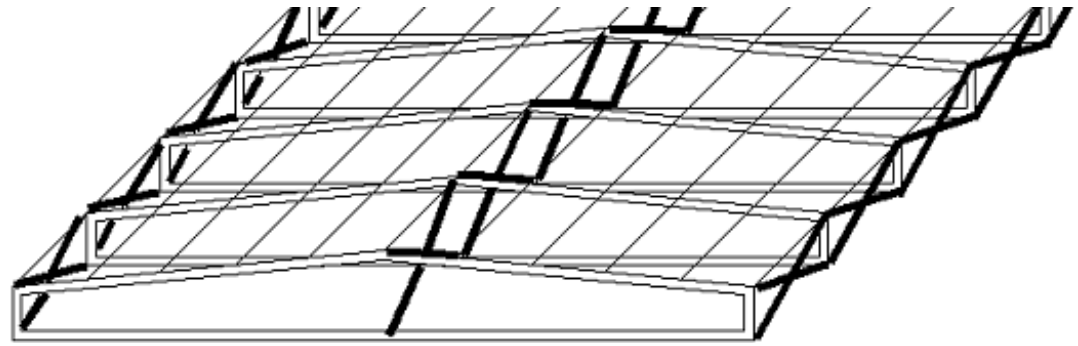


Photo: Author

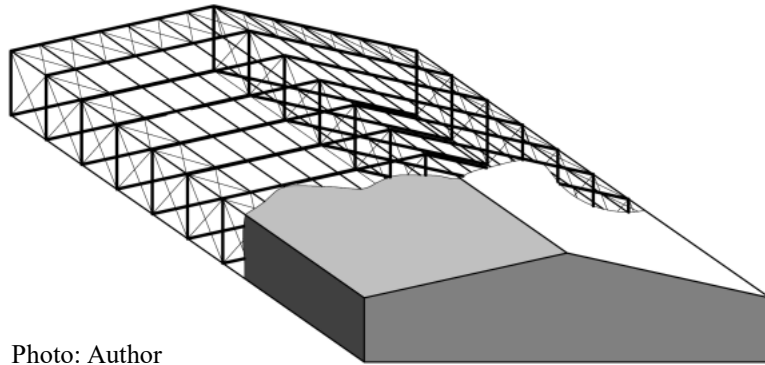
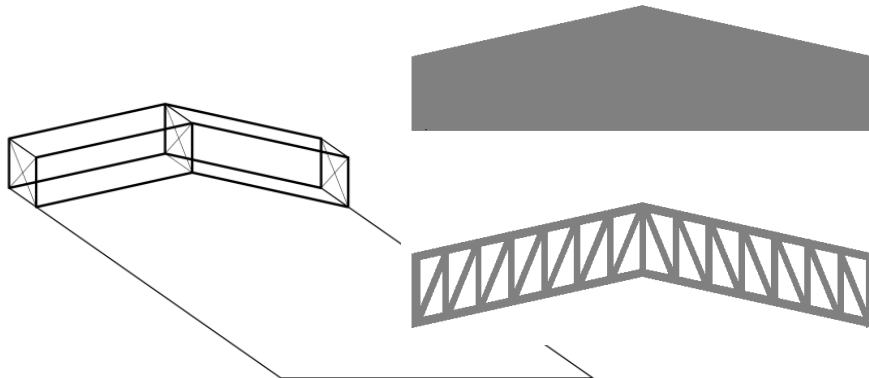


Photo: Author



This kind of bracings the same are under compression, first of all. So, this kind of bracings the same can be rigid only.

Forces applicated to this bracings are equivalent of forces from instability of bottom chord (calcuated by the same formulas, as for top chord), and from wind action on fron wall during mountage stage (area for wind action is much more smaller, than for exploitation stage).

Simplified method, elastic analysis

EN 1993-1-1 6.3.2.4

Beam or girder with discrete lateral bracings to the compression flange are not susceptible to lateral buckling, if distance between bracing L_C satisfies formula as follow:

$$L_C k_c / (i_{f,z} \lambda_1) \leq \lambda_{c0} M_{c,Rd} / M_{y,Ed}$$

$M_{y,Ed}$ - the maximum value of bending moment within the restraint spacing

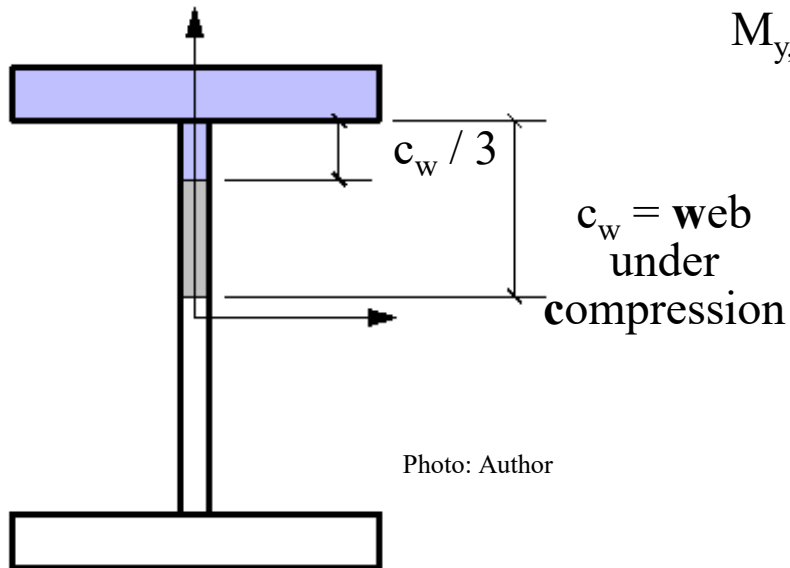
$$M_{c,Rd} = W_{y,c,f} f_y / \gamma_{M1}$$

k_c according to #5 / 63

$$\lambda_1 = 93,9 \varepsilon$$

$$\lambda_{c0} = 0,5$$

$$i_{f,z} = \sqrt{[J_{\text{eff},f,z} / (A_{\text{eff},f} + A_{\text{eff},w})]}$$



Examination issues

Types of bracings

Role and position various roof bracings

Bracings anti lateral buckling and anti flexural buckling - similarities and differences

Critical length of roof girder for various forms of instability

Way of calculations for corrugated sheet

Way of calculation for bar bracings

Bracing system - stężenia

Tract - połąć

Ridge - kalenica

Hood - okap

Rigging screw - śruba rzymska

Valley - kosz

Skylight - świetlik

Thank you for attention

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