# Metal Structures

# Lecture XXI

# Other joints

Contents

# Introduction

Many various types of joints are applied in metal structures. Most of them were presented on lectures #18 - 20.

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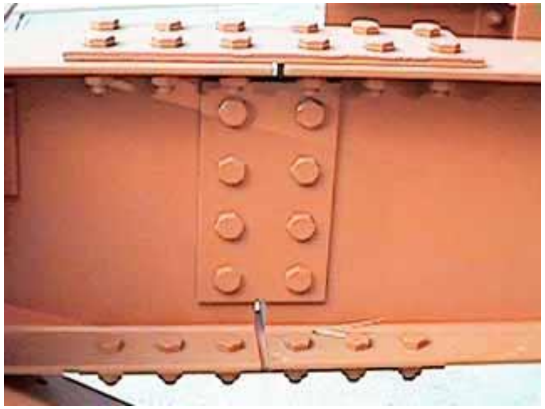

Photo: amsd.co.uk

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The last three groups of joints are:

- stiffeners;
- truss joints;
- cleats and gusset plated for purlins, girts and bracings.

Photo: chinhdaisteel.com

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Photo: Author

## Stiffeners

Reasons for using stiffeners in steel structures:

- Support for slender web / flange (prevention of web local instability);
- Increasing of web resistance under shear forces;
- Support for transverse beams (connection between primary and secondary beams).

Photo: lmsteelfab.com

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Support for slender web (prevention of web local instability);

Support for slender flange (prevention of flange local instability):

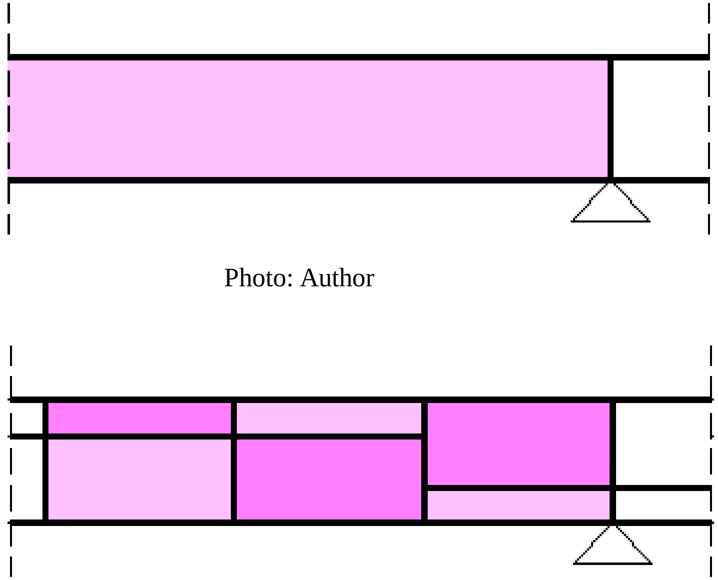

Photo: Author

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Threat of instability for compressed flange and / or compressed part of web is much lower for small sub-panels than for one big panel.

# Increasing of web resistance under shear forces.

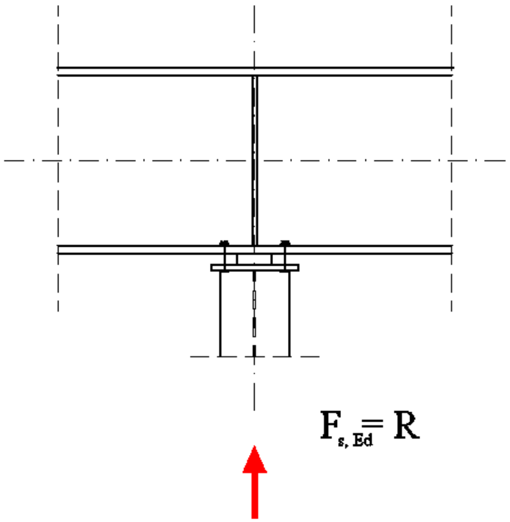


Photo: steelconstruction.info

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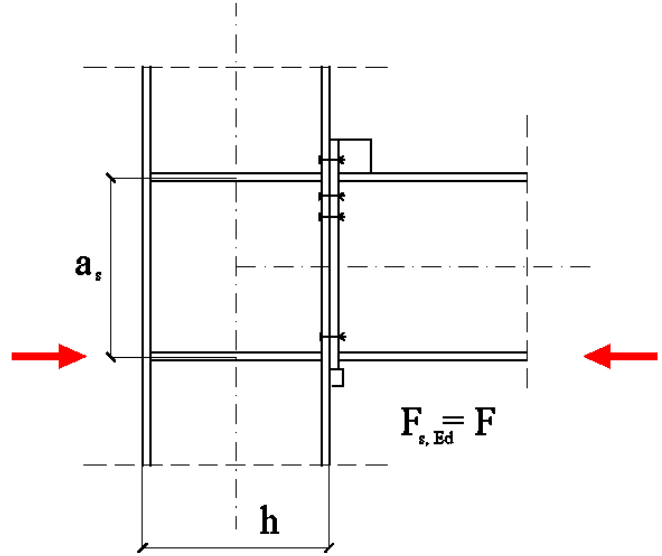


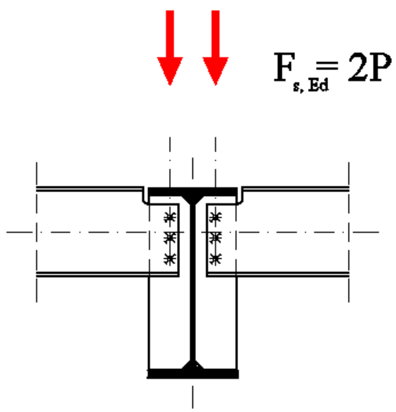


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## Support for transverse beams (connection between primary and secondary beams)

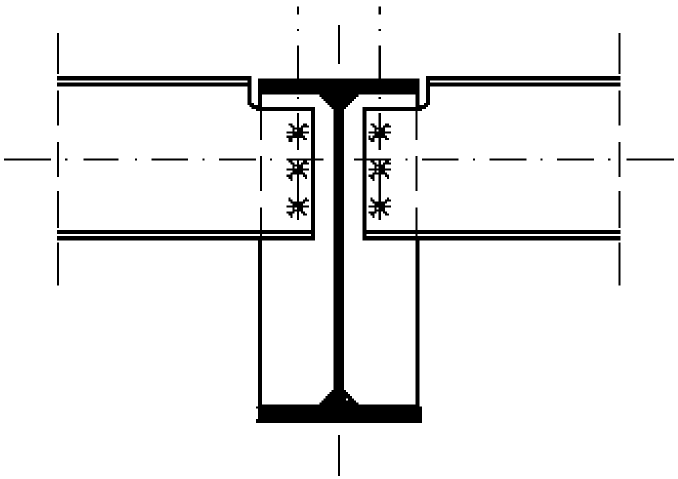
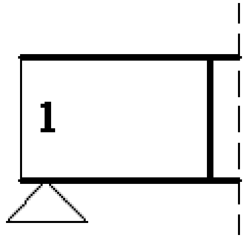


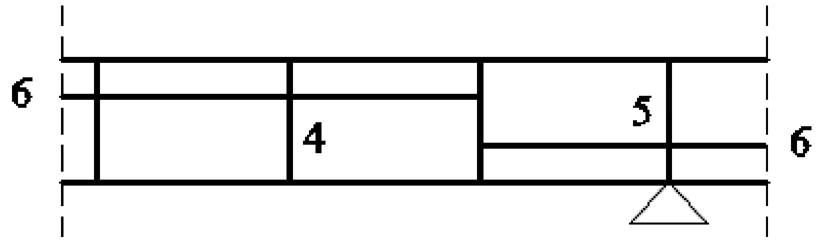
Photo: Author

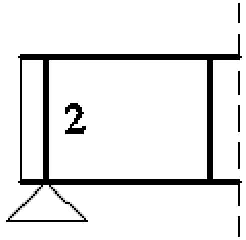
Photo: mscsteel.com

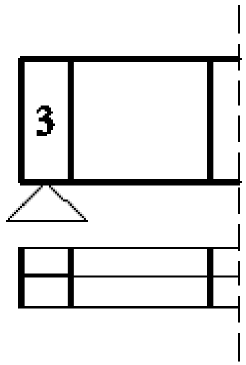
7 types of stiffeners are presented in Eurocode. The last one is presented in literature only.

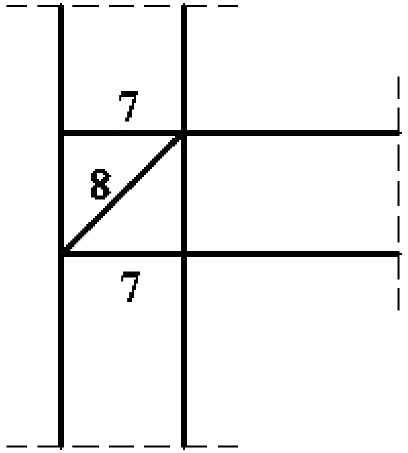
1. No end post
2. Non-rigid end post
3. Rigid end post
4. Transverse stiffener
5. Intermediate support stiffener
6. Longitudinal stiffener
7. Column transverse stiffener
8. Diagonal stiffener

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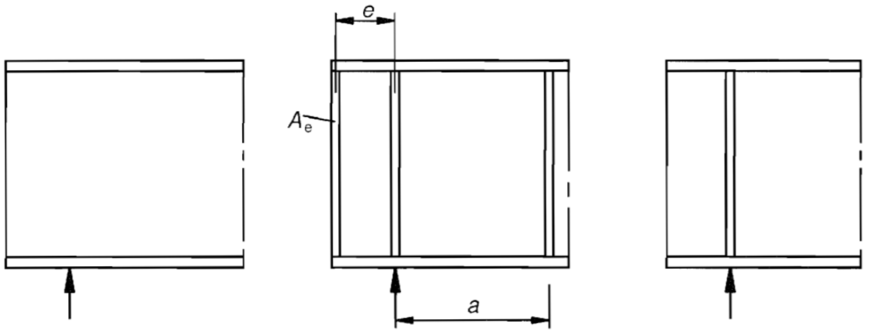



Photo: EN 1993-1-5 fig. 5.1

„No end post” menas of course „no stiffeners”. No additional phenomena to analisis, no specific algotithm for calculation of stiffener.
This solution is not accepted for beams of IV$^{th}$ class of cross-section

Eurocode does not explain when rigid or non-rigid end post should be applied. Information in literature is not entirely clear and sometimes contradictory. The most often recommendation in literature for using of rigid end post:

- for big value of loads;
- for slenderness of web $\bar{\lambda}_w \geq 1{,}08$ ($\rightarrow$ #t / 27).

Unfortunately, in Eurocode can be found many inconsequences. The most often case is:

General situation is divided into sub-cases

A – full information about way of calculation

B – no information about way of calculation

The most part of such situation concern various phenomenon in rigid bolted joints (for example: resistance for netto area around hols for bolts, stiffness of shear joints, impact of axial force for bending moment resistance).

Rare case is contradictions between different points in Eurocode. An examples are calculation of concrete base resistance under hinged support of columns or calculation of built-up columns.

→ #3 / 95

All these problems will be mentioned in future lectures.

Inconsequence in analysed case:

- EN 1993-1-5 5.1.(2) presesented distinction: unstiffened web ↔ stiffened web;
- EN 1993-1-8 fig. 5.1 accepted welded I-beam without stiffeners over supports (unstiffened web);
- EN 1993-1-8 5.3.(1) presented information only for I-beams with stiffners at least over supports;
- EN 1993-1-8 no presented information about calculation of I-beams completely without stiffeners;
- Therefore, it is not known how to understand the condition at point EN 1993-1-5 5.1.(2): where web is unstiffened and stiffened;
- According to literature, beam with stiffeners over supports only could be treated as beam with unstiffened web;
- No information about calculation for case „beam completely without stiffneners" means, that such solution is not recommended;
- Welded I-beam must have at least stiffenenr over supports.

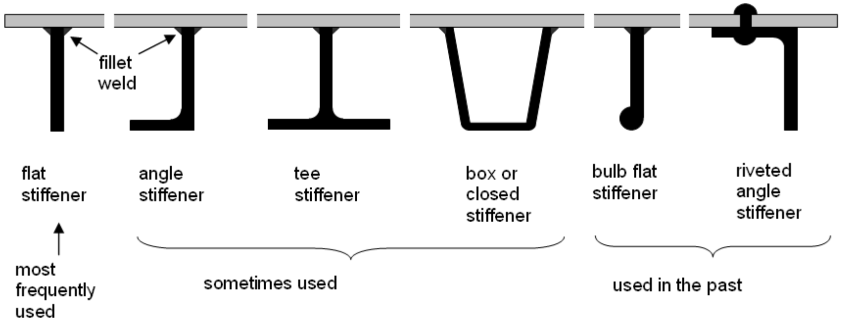
→ Lab #2 / 61

Enigmatic term "big value of loads " means, in practice, first of all bridge beams and crane beams.

Photo: steelconstruction.info

Photo: ellsenoverheadbridgecrane.com

In such cases, also intermediate (non-supporting) stiffeners tend to be more massive.

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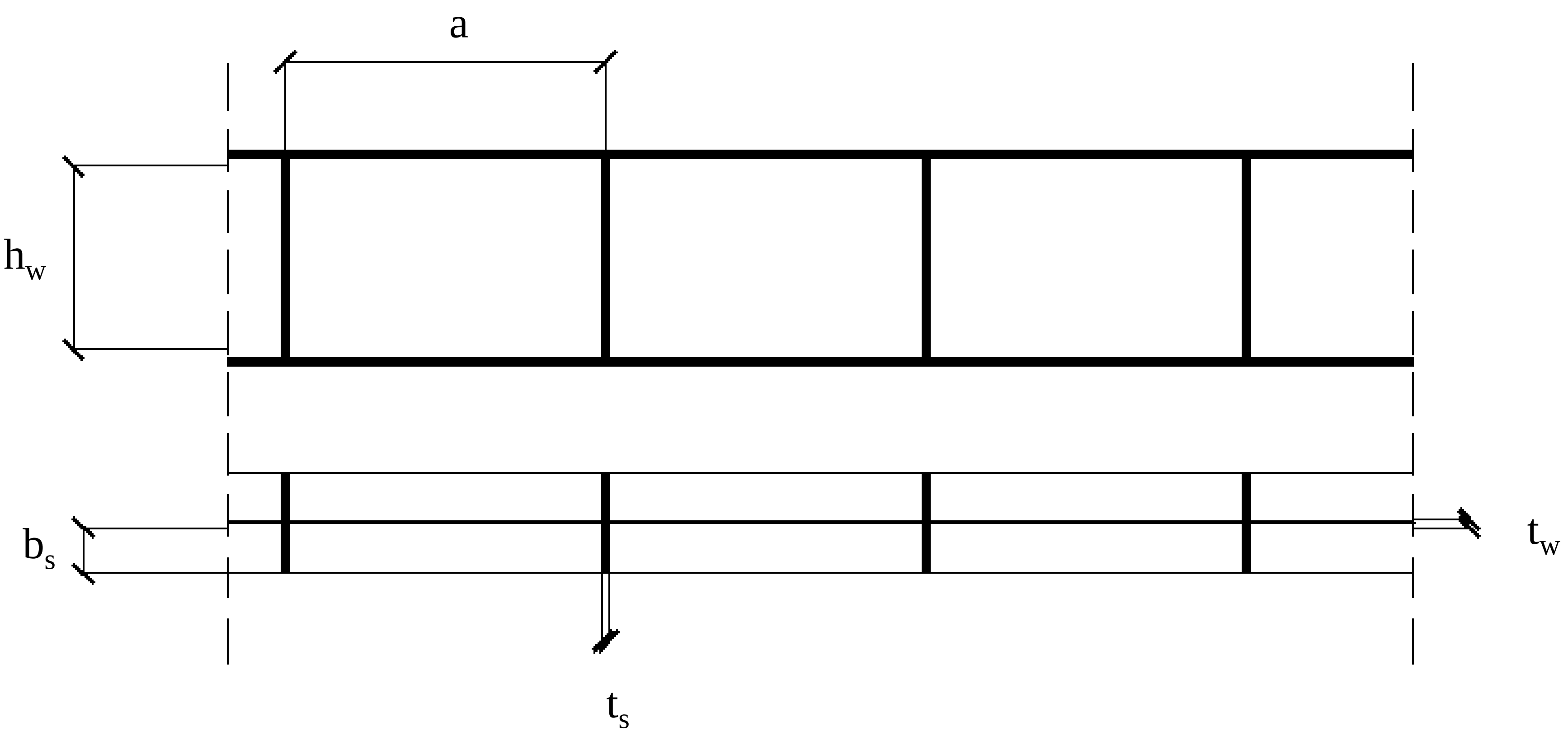
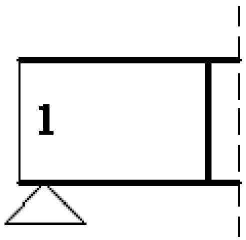
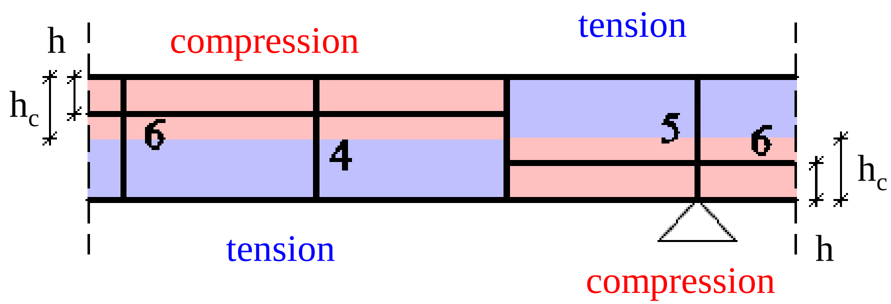
# Geometry of stiffeners:

Photo: Author

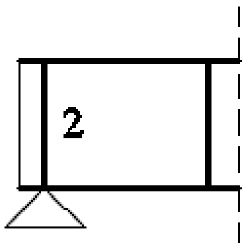


# Location of stiffeners

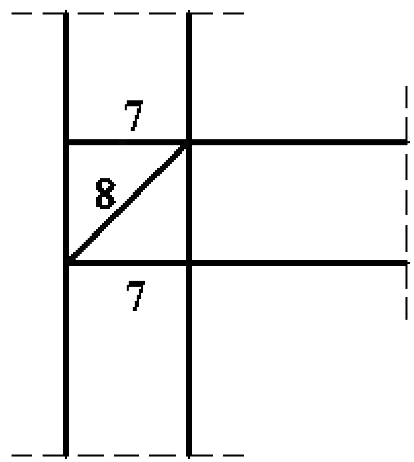



Vertical (2, 3, 4, 5): over supports, in connections between primary and secondary beam and under big transversal loads;




Longitudinal (6): $h / h_c = 1/3 - 1/2$; $h_c$ – height of compressed part



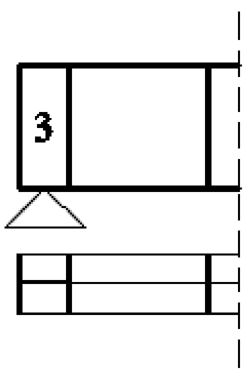




Photo: Author

Transversal (7): on axes of flanges;

Diagonal (8): on joints beam-columns.

Conditions to checking:

| Independent of the load | | | Load-dependent | |
|---|---|---|---|---|
| Conditions: | Stiffeners: | | Stiffeners: | Conditions: |
| Thickness of adjacent elements (#t / 17) | 2, 3, 4, 5, 7 | | 2, 4, 5, 7 | Contact stress (#t / 22) |
| Class of the cross-section (#t / 18) | 2, 3, 4, 5, 6, 7, 8 | | 2, 4, 5, 7 | Axial compression (#t / 23 - 30) |
| Prevent torsional buckling of stiffener (#t / 19) | 2, 4, 5, 7 | | 6 | Longitudinal stiffeners (#t / 31-32) |
| Rigid supports for web panel (#t / 20) | 2, 4, 5, 7 | | 8 | Diagonal stiffener (#t / 33-35) |
| Rigid end post (#t / 21) | 3 | | 2, 3, 4, 5, 6, 7, 8 | Welds (#t / 36) |

## Thickness of adjacent elements:

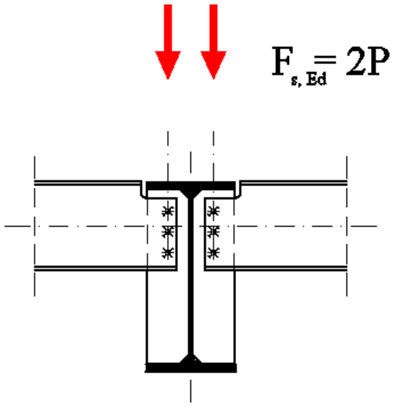


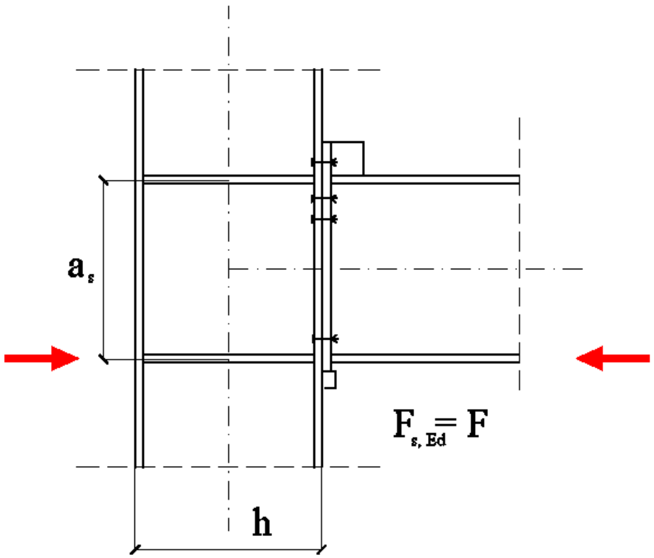

Photo: Author

$t_s \geq$ thickness of secondary beam web

or

$t_s \geq$ thickness of beam flange

Photo: Author




Vertical stiffener - recommendation avoids complicated calculation of block tearing of stiffener (web of beam – tinner element – is more important for this calculation).

Horizontal stiffener - recommendation presented in Eurocode.

**Class of the cross-section**:

There is no special requirements in EN 1993, but each formula for stiffeners is as for cross-section of no-VI$^{th}$ class of cross-section. Because of this stiffeners should have I$^{st}$, II$^{nd}$ or III$^{rd}$ class of cross-section. Additionally, according to old Polish Standard, stiffeners should be no-VI$^{th}$ class of cross-section.

$$b_s / t_s \leq 14\,\varepsilon$$

$b_s$ , $t_s \rightarrow$ #t / 14

## Prevent torsional buckling of stiffener

EN 1993-1-5 (9.3)

$$J_T / J_p \geq 5{,}3\ f_y / E$$

$$J_T = b_s\ t_s^3 / 3$$

$$J_p = b_s^3\ t_s / 3 + b_s\ t_s^3 / 12$$

$b_s$ , $t_s \rightarrow$ #t / 14

## Rigid supports for web panel

EN 1993-1-5 9.3.3 (9.6)

$$a / h_w \geq \sqrt{2} \rightarrow J_{st} \geq 1{,}50\ h_w^3\ t_w^3 / a^2$$

$$a / h_w < \sqrt{2} \rightarrow J_{st} \geq 0{,}75\ h_w\ t_w^3$$

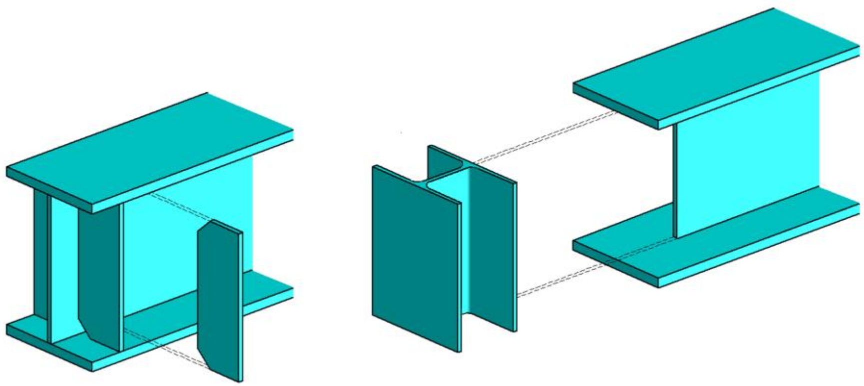
$$J_{st} = 2\ [\ b_s^3\ t_s / 12 + b_s\ t_s\ (b_s + t_w)^2 / 4\ ]$$

$a$, $h_w$ , $t_w$ , $b_s$ , $t_s$ $\rightarrow$ #t / 14

# Rigid end post

EN 1993-1-5 9.3.1 (3)

Photo: Author



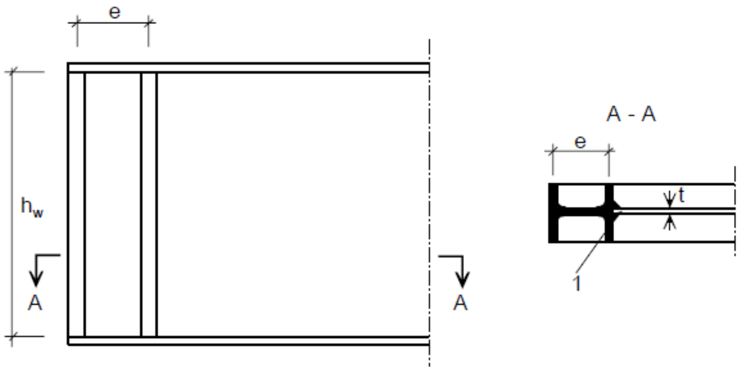



Photo: EN 1993-1-5 fig. 9.6

Rigid: two couples of stiffeners: 2 $A_s$ ; $W_{s,x}$

$e \geq 0{,}1\ h_w$

$A_s \geq 4\ h_w\ t_w^2 / e$ (two couples of plates)

$W_{s,x} \geq 4\ h_w\ t_w^2$ (vertical hot rolled I-beam as end post)

$h_w$ , $t_w \rightarrow$ #t / 14

## Contact stresses

EN 1993-1-5 9.4 (2)

$$F_{s,\,Ed} / (2\, c_s\, t_s\, f_y\, \gamma_{M0}) \leq 1{,}0$$

$b_s$ , $t_s \rightarrow$ #t / 14

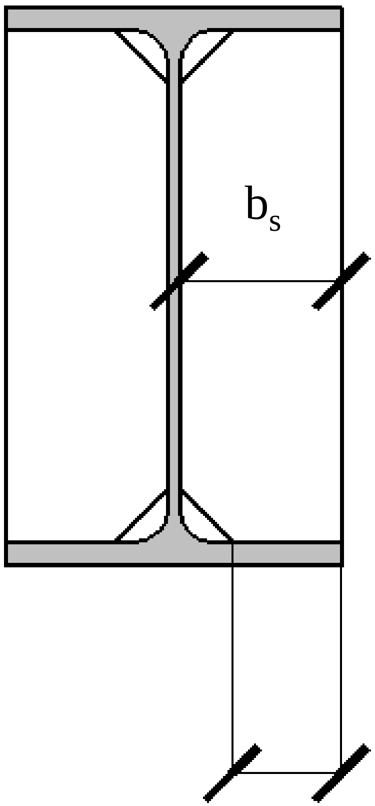



Photo: Author

## Axial compression

EN 1993-1-5 9.2.1

EN 1993-1-5 9.4 (2)

The most complicated part of calculation of stiffener. Imperfections of stiffeners and web must be taken into consideration. Effects of these imperfections are additional vertical force $\Delta N_{st,\,Ed}$ and additional continouds action q.

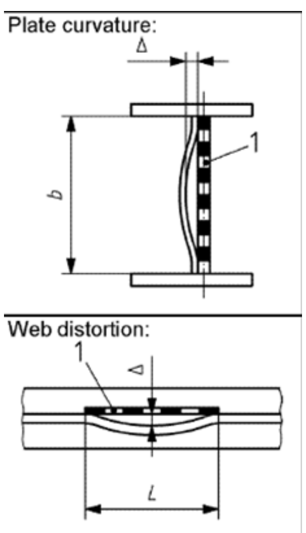



Photo: EN 1090-2 tab. D.1.1

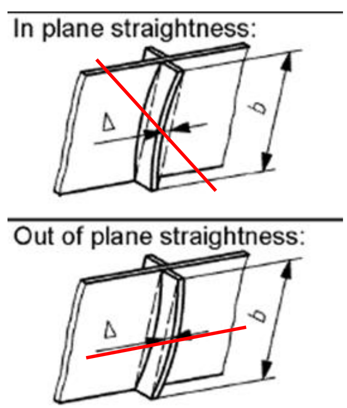



Photo: EN 1090-2 tab. D.1.5

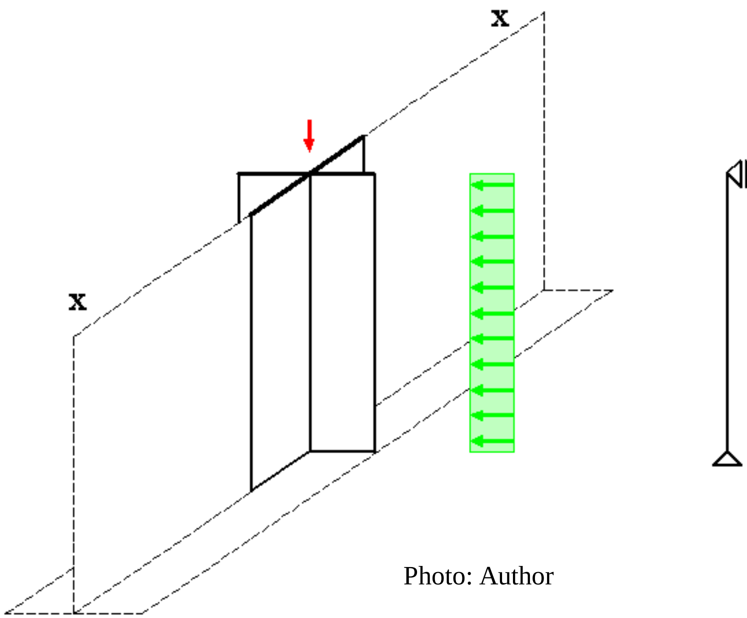



Photo: Author

Photo: lmsteelfab.com

- Stiffeners are treated as bar, compressed by axial force $N_{s,\,Ed}$;
- We analyse flexural buckling about x-x axis;
- We take into consideration cross-section of stiffeners and cooperating part of web (┼ cross-section);
- Axial force $N_{s,\,Ed}$ contains imperfections of stiffeners;
- Additionally we must analyse imperfections of web - represented by additional load $q$;
- We must analyse interaction between axial force $N_{s,\,Ed}$, buckling about x-x and bending moment $M_{s,\,Ed}$ ($q$).

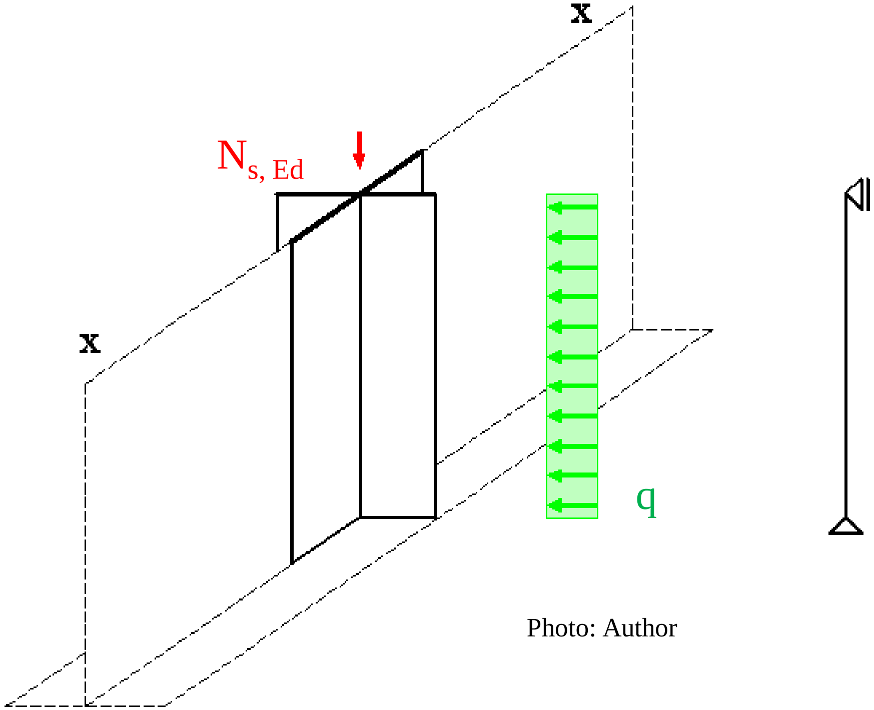



Photo: Author

$N_{s, Ed}$ = max ($F_{s, Ed} + \Delta N_{st}$ ; $V^{*}_{Ed} + \Delta N_{st}$ )

$F_{s, Ed}$ - transverse force, acts on stiffeners (from secondary beam, from supports...); there is possible that $F_{s, Ed} = 0$ (for stiffeners used as support for slender web or flange only → #t / 26 );

$\Delta N_{st} = \sigma_m b^2 / \pi^2$ - from imperfections of stiffeners ($b = h_w$ for analysis of stiffeners);

$V^{*}_{Ed}$ = max [$V_{Ed} - f_{yw} h_w t_w / (\bar{\lambda}_w \gamma_{M1} \sqrt{3})$ ; 0] - part of shear force over web resistance of shearing;

$V_{Ed}$ - shear force at the distance 0,5 $h_w$ from the edge of the panel with the largest shear force;

$\gamma_{M1} = 1,0$;

$\bar{\lambda}_w$ → #t / 27;

$\sigma_m$ → #t / 28;

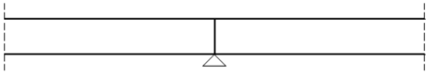
| $N_{s,\,Ed} = \Delta N_{st}$ (for stiffeners used as support for slender web or flange only) | $N_{s,\,Ed} > \Delta N_{st}$ |
|---|---|
| Requirements from #t / 25 may be assumed to be satisfied provided when:<br><br>$J_{st} \geq (1 + 300\, w_0\, u / b)\, (\sigma_m\, b^4) / (E\, \pi^4)$<br>$J_{st} = 2\, [\, b_s^3\, t_s / 12 + b_s\, t_s\, (b_s + t_w)^2 / 4\, ]$ | Total procedure of calculations is necessary (#t / 23 - 30):<br><br>$q = \pi\, \sigma_m\, (w_0 + w_{el}) / 4$ |

$\sigma_m \rightarrow$ #t / 28;

$w_0$, $u$, $w_{el} \rightarrow$ #t / 29;

$b = h_w$

# Slenderness factor for web resistance:



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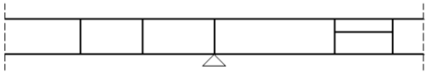


$$\bar{\lambda}_w = h_w / (86,4\ t_w\ \varepsilon)$$

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

EN 1993-1-5 A.3

$\alpha = a / h_w$

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

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

$J_{st}$ - about axis z (vertical) for longitudinal stiffeners;

If no longitudinal stiffeners, $k_{zts} = 0$

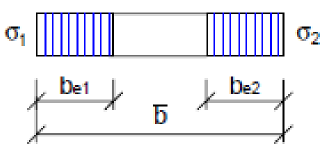
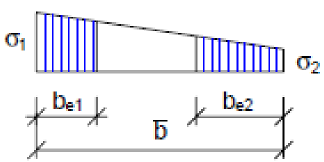
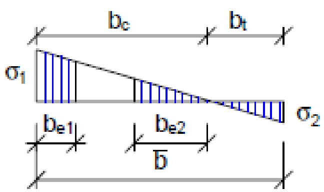
$$\sigma_m = (\sigma_{cr,\,c} / \sigma_{cr,\,p})\,(1 / a_1 + 1 / a_2)\,N_{eq\text{-}\,axial} / b$$

$$\sigma_{cr,\,c} = \pi^2\,E\,t_w^{\,2} / [\,12\,(1 - \nu^2)\,a^2\,] \approx 190\,000\,(t_w / a)^2\ [MPa]$$

$$\sigma_{cr,\,p} = k_\sigma\,(28{,}4\,\varepsilon)^2\,f_y\,(t_w / b)^2 \approx 190\,000\,k_\sigma\,(t_w / b)^2\ [MPa]$$

$$a = (a_1 + a_2) / 2\quad (\text{usually } a = a_1 = a_2)$$

$$b = h_w$$

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

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

$k_\sigma$ - for web, according to EN 1993-1-5 tab. 4.1

$N_{eq\text{-}\,axial} \rightarrow$ #t / 29;

$w_0 = s / 300$

$s = \min (a_1 \;;\; a_2 \;;\; b)$

$w_{el} = b / 300$

$u = \max [\, 1{,}0 \;;\; \pi^2 E\, e_{max}\, \gamma_{M1} / (f_y\, 300\, b) \,]$

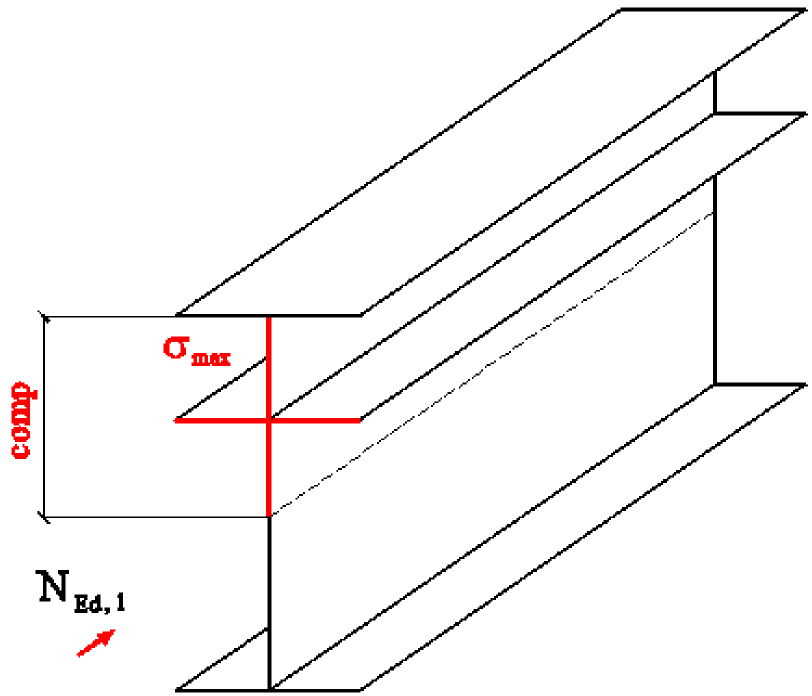
$e_{max} = b_s / 2$

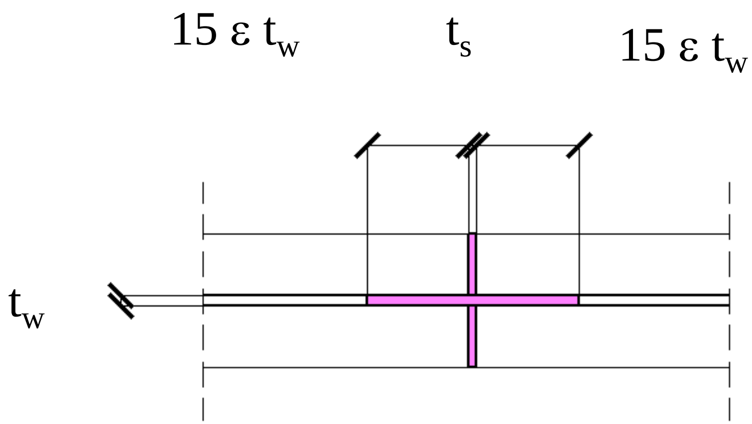
$b = h_w$

$N_{eq\text{- }axial} = \max (N_{Ed,\,1} \;;\; \sigma_{max} A / 2 )$

$\sigma_{max}$ is analised when $M_{Ed}$ or $M_{Ed} + N_{Ed,1}$ are applied to beam.

Photo: Author






$M_{s,\ Ed} = q\ h_w^{\ 2} / 8$

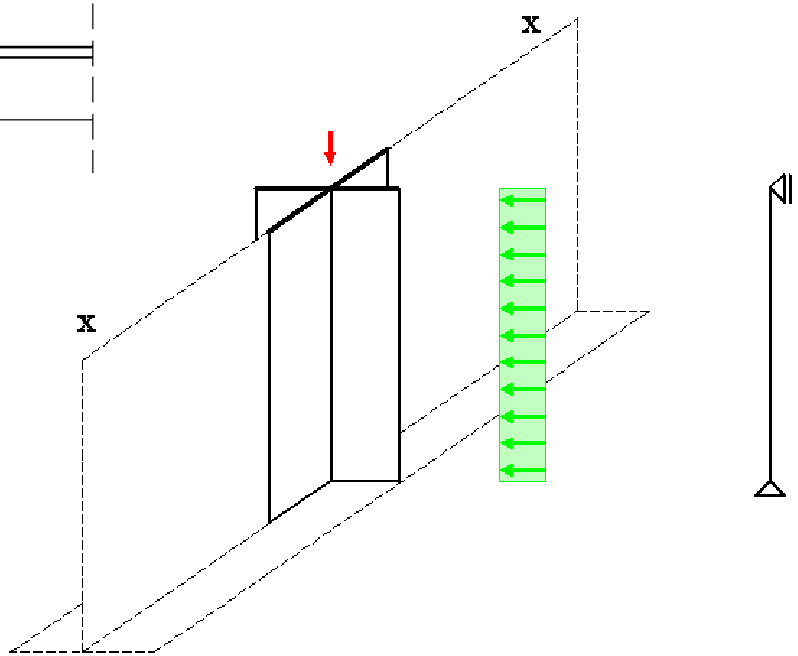
$\chi_x = \chi_x\ (c, +, l_{cr})$ (according to lecture #5)

$l_{cr} = 0{,}75\ h_w$

$N_{Rd} = A_{+}\ f_y / \gamma_{M0}$

$M_{Rd} = W_{+,\ x,\ el}\ f_y / \gamma_{M0}$

Photo: Author




$N_{s,\ Ed} / (\chi_x\ N_{Rd}) + M_{s,\ Ed} / M_{Rd} \leq 1{,}0 - \Delta_{0,\ x}$

| Class of cross-section $+$ | 1 or 2 | 3 or 4 |
|---|---|---|
| $\Delta_{0,\ x}$ | $0{,}1 + 0{,}2\ [\ (W_{+,\ x,\ pl} / W_{+,\ x,\ el}) - 1]$ | 0,1 |

EN 1993-1-1 NA.20

# Longitudinal stiffeners

EN 1993-1-5  9.3.4

Photo: hebsteelstructure.en.made-in-china.com

There are two possibilities of calculations:

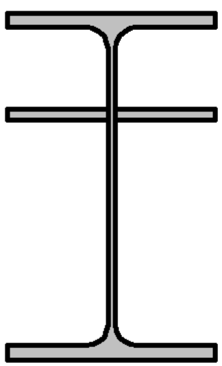
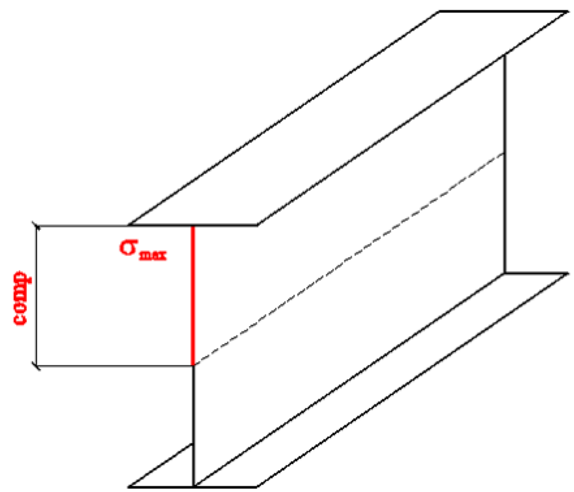


Photo: Author




## $1^{st}$ way

Longitudinal stiffeners are treated as a part of cross-section; their geometry is added to geometry of beam;

## $2^{nd}$ way

$N_{Ed,\ eq} = \sigma_{comp,\ max}\ A_{comp} / 2$

$N_{Rd} = 2\ b_{h\text{-}s}\ t_{h\text{-}s}\ f_y$

$N_{Ed,eq} / N_{Rd} \leq 1,0$

# Longitudinal stiffeners according to old Polish Standard PN-B 3200

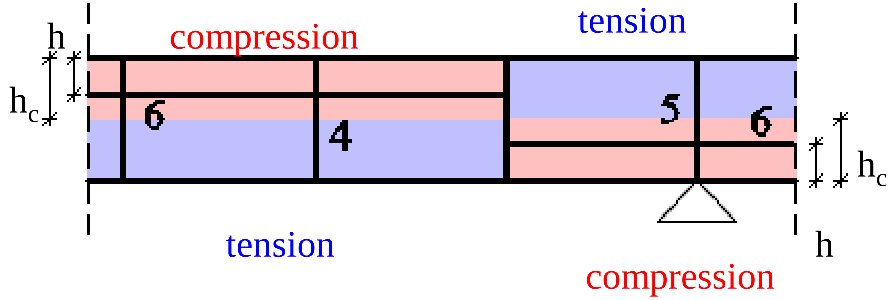



Photo: Author

$$J_{st} \geq k\, h_w\, t_w^3$$

$J_{st}$ - about axis z (vertical) for longitudinal stiffener;

$0{,}25 \leq h / h_c \leq 0{,}33$ and $0{,}5 \leq a / h_w \leq 2{,}0$ :

$$k = 4 \sqrt{[(a / h_w)^3\, \delta]}$$

$$0{,}05 \leq \delta = (b_s\, t_s) / (t_w\, h_w) \leq 0{,}20$$

$a,\ h_w\ ,\ t_w\ ,\ b_s\ ,\ t_s \longrightarrow$ #t / 14

$h / h_c = 0{,}5$ and $a \geq h_w$

$$k = \max \{3{,}0 \sqrt{[(a / h_w)^3\, \delta]}\ \ ;\ \ 0{,}7 \sqrt{(a / h_w)^3}]$$

## Diagonal stiffeners

Photo: microstran.com.au

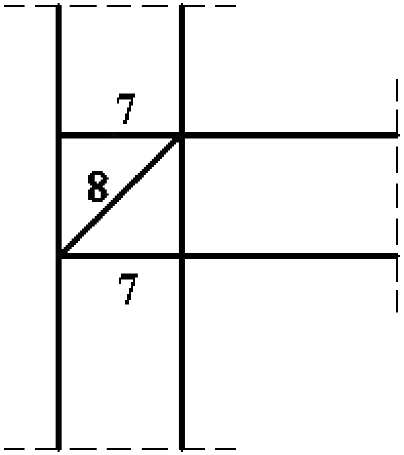



Photo: Author

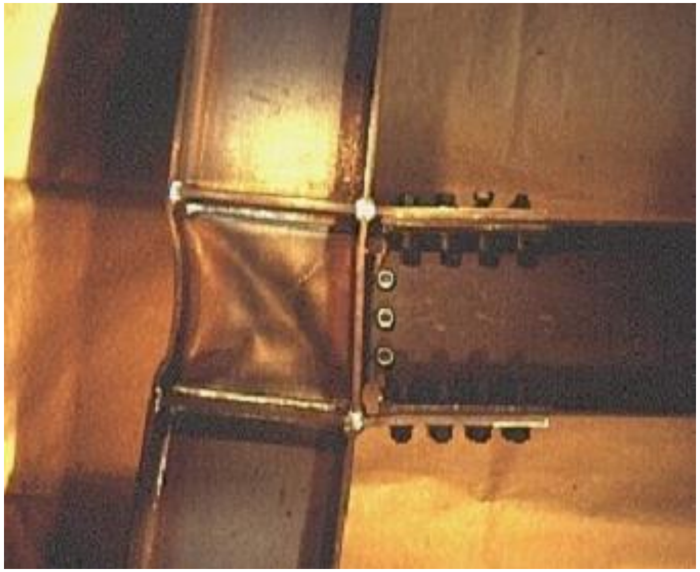


Photo: fgg.uni-lj.si

- Possibility of increasing resistance and stiffness of column web in shear;
- Rarely use, first of all in exterior columns of frames;
- No information in Eurocode;
- No clear guidance in the literature;

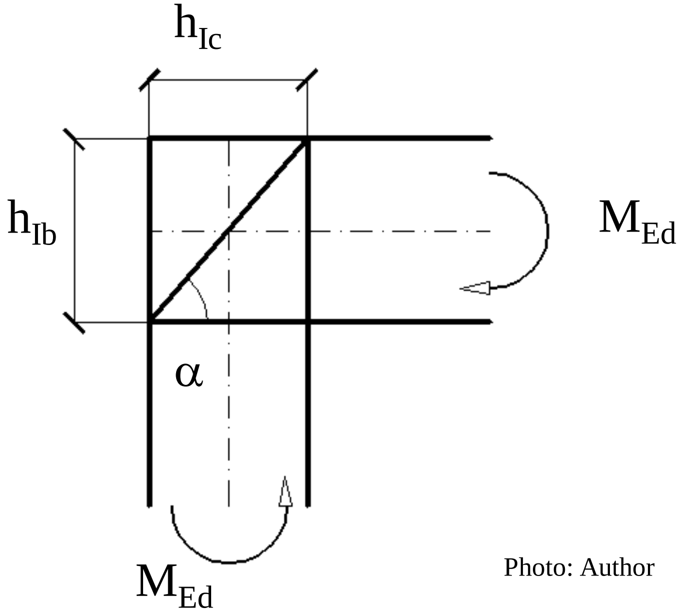
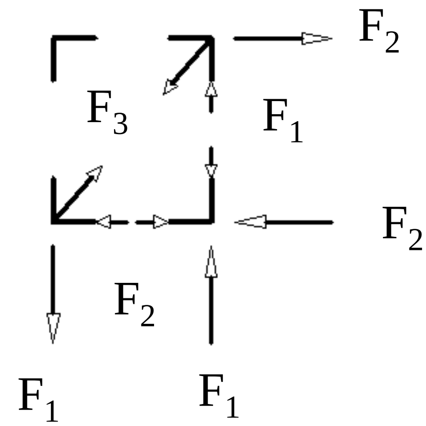



Photo: Author

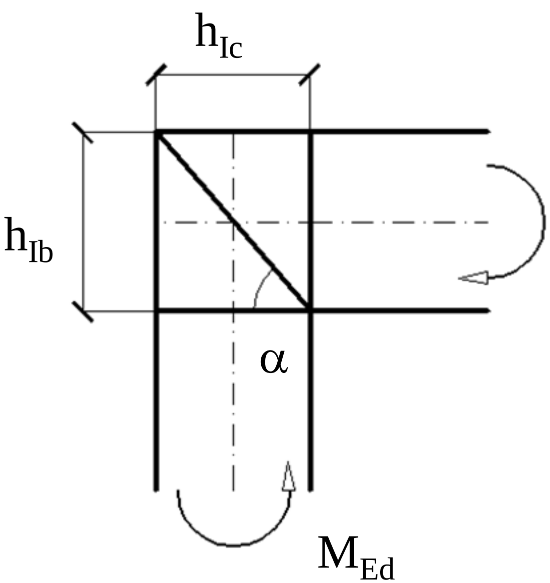


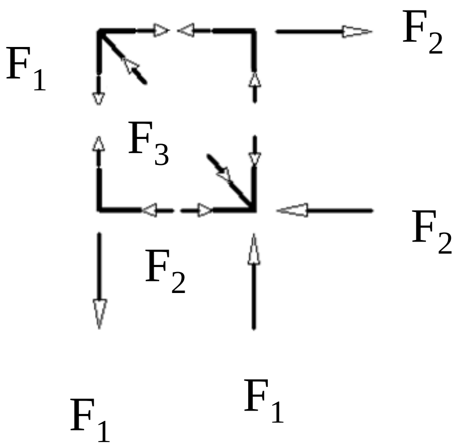

Diagonal stiffener under tension

$$F_1 = M_{Ed} / h_{Ic}$$

$$F_2 = M_{Ed} / h_{Ib}$$

$$F_3 = \sqrt{(F_1^2 + F_2^2)}$$




Diagonal stiffener under compression (more often used)

Photo: microstran.com.au

Photo: skcthailand.com

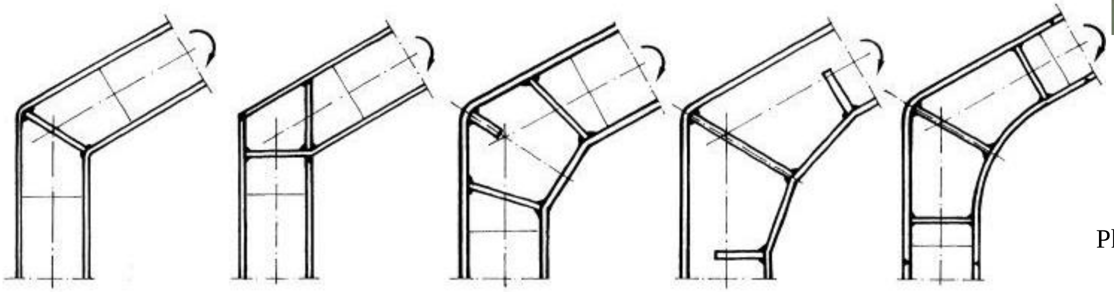


Photo: chodor-projekt.net

According to supposition in literature, if diagonal stiffness and its welds have enough resistance to bear tension $F_3$:

- resistance of column web in shear $V_{wp,\,Rd} \rightarrow \infty$ ($\rightarrow$ lec #19);
- stiffness of column web in shear $k_2 \rightarrow \infty$ ($\rightarrow$ lec #15);

**Welds**

Vertical and transversal stiffeners:

Lecture #17 example 1, $F_V = (F_{s,\,Ed} + \Delta N_{st}\ ;\ V^*_{Ed} + \Delta N_{st}) \rightarrow$ #t / 25;

Longitudinal stiffeners:

Lecture #17 example 7a, $\sigma_1$, $\tau_1$ according to max values on analysed member;

Diagolnal stiffeners:

Lecture #17 example 1, $F_V = F_3 \rightarrow$ #t / 34;

# Truss nodes

For steel memebers used as a part of truss, requirements for resistance and stability must be satisfied (→ lec #9). Situation is much more complicated for truss nodes. Many additional requiremenst must be satisfied. Two aspects must be taken into consideration:

- real behavior of analysed truss;
- resistance of joints.

Truss behaves as ideal truss only if many condition are satisfied. In opposite situation, truss behaves rather as frame. Satisfaction / dissatisfaction of these additional requirements changes behavior  of truss and change its statical model (→ #t / 38 - 50).

Additionally, many various types of local instabilities and local concentration of stresses are very important for resistance of joints (→ #t / 51 - 56).

# Definition

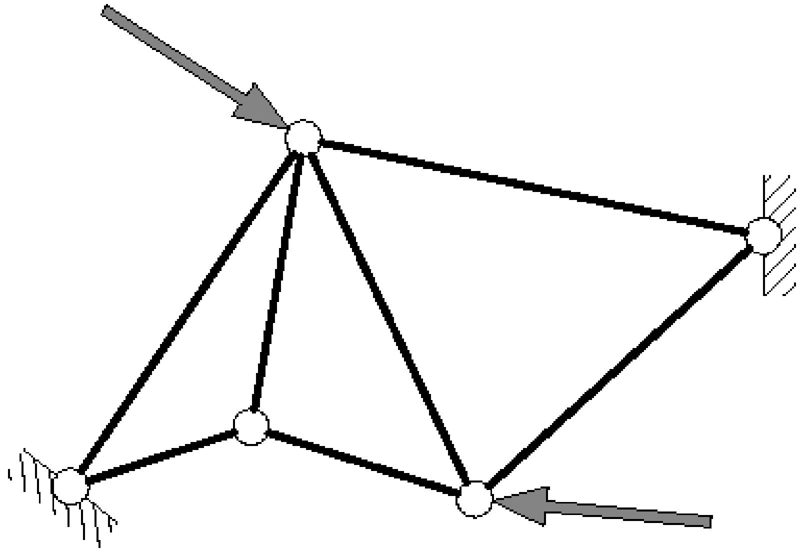


Photo: Author

Truss – theory (idealization):

- Straight bars only;
- Hinge joints;
- Forces in joints only;

Schwedler-Żurawski formula for straight bar:

$$d\,M(x) / dx = Q(x)$$
$$d\,Q(x) / dx = q(x)$$

Forces in joints only → no loads along bar ($q(x) = 0$):

$$q(x) = 0 \rightarrow Q(x) = \text{const} = C \rightarrow M(x) = C\,x + A$$

Hinges:

$$M(0) = 0 \rightarrow A = 0 \quad ; \quad M(L) = 0 \rightarrow C = 0$$

$$M(x) = 0 \quad ; \quad Q(x) = 0$$

→ Des #1 / 10

There are axial forces only

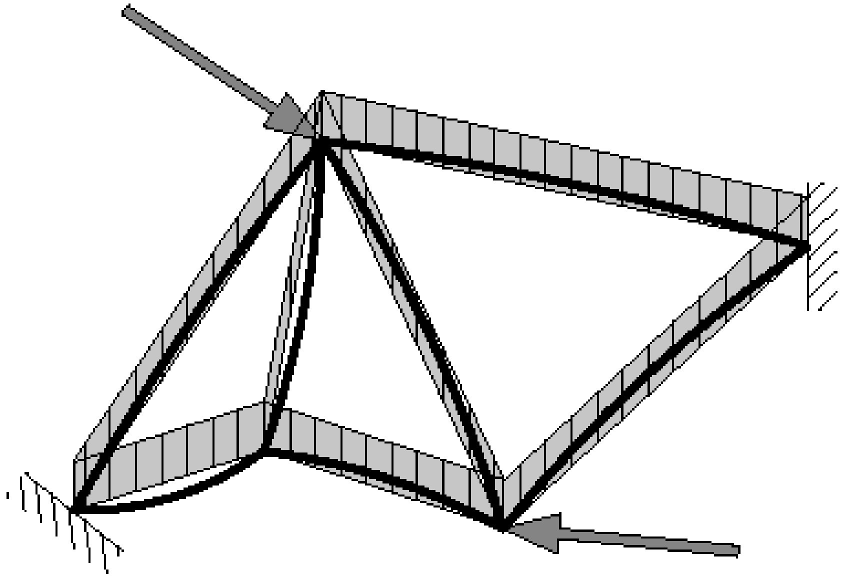


Photo: Author

Truss – real:

- Bars with imperfections;
- No ideal hinge joints;
- Gravity along bars;

It’s rather frame

→ #9 / 4

Additional requirements during analysis of truss / frame behavior can be divided into three groups (EN 1993-1-8 5.1.5):

→ #9 / 46

- General and additional requirements, permissible shapes of joints (→ #9 / 47 - 50);
- Loads according to definition of ideal truss, in nodes only (→ #9 / 50);
- Acceptable values of eccentricities (→ #9 / 47, 51-53).

Consequences of satisfaction / dissatisfaction (partial satisfaction) are various, in dependence of group of requirements. Generally, 5 various static models of truss / frame is taken into consideration.

The more conditions are dissatisfied, the static model of the truss is getting closer to the frame with rigid joints.

→ #9 / 47

# Additional requirements for truss nodes

# (EN 1993-1-8 7.1):

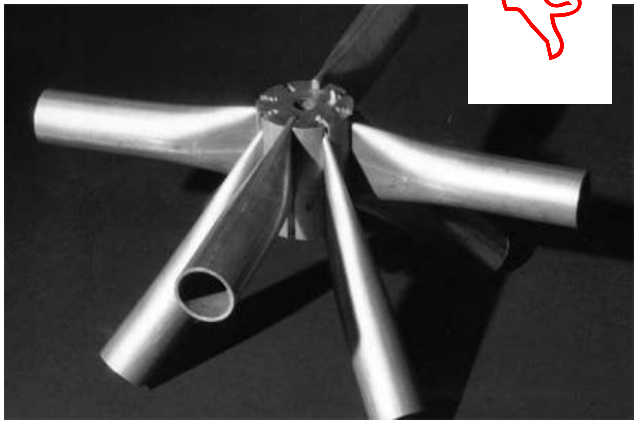


Photo: tatasteelconstruction.com

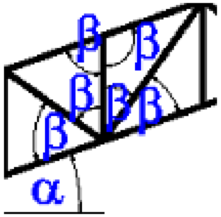
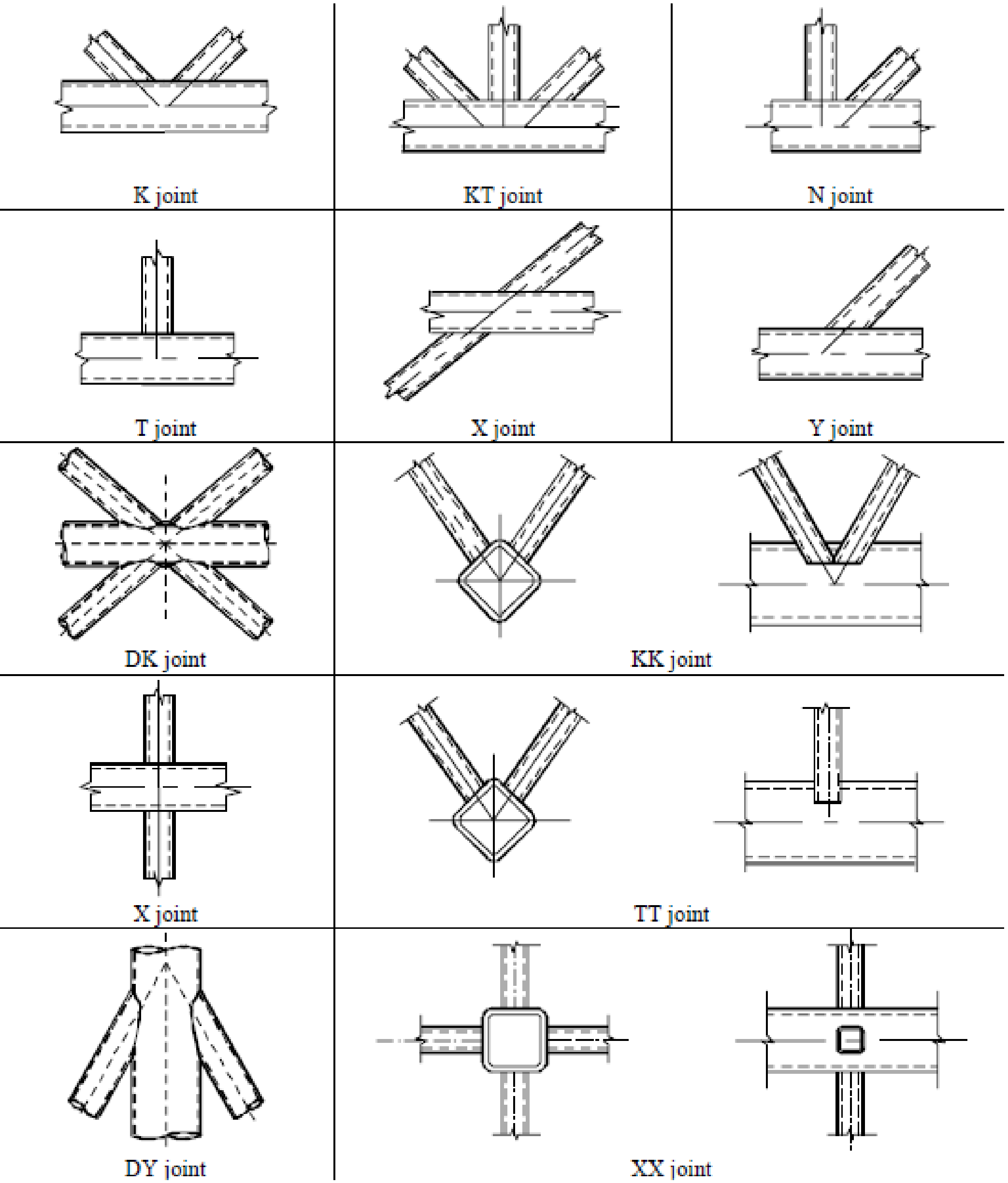



Photo: Author

- Chords → ▯ □ ○ I ;
- Web members → ▯ □ ○ ;
- Deformation ends of element are not accepted;
- $f_y$ (▯ □ ○) ≤ 460 MPa;
- $f_y$ (▯ □ ○) > 355 MPa → $f_{y,\ design}$ = 0,9 $f_y$;
- t (▯ □ ○) ≥ 2,5 mm;
- $t_{chord}$ (▯ □ ○) ≤ 25 mm;
- Compressed chords and web memebers → I$^{st}$ or II$^{nd}$ class of cross-section;
- $\beta_i \geq 30^o$;
- Distances between web members (eccentricities) must be respected (→ #9 / 51 - 53);
- Shape of joints must be respected (EN 1993-1-8 fig. 7.1), (→ #9 / 48);
- (Length of member) / (depth of member) > 6 (EN 1993-1-8 5.1.5.(3));




# Types of permissible joints

Photo: EN 1993-1-8 fig. 7.1

→ #9 / 48

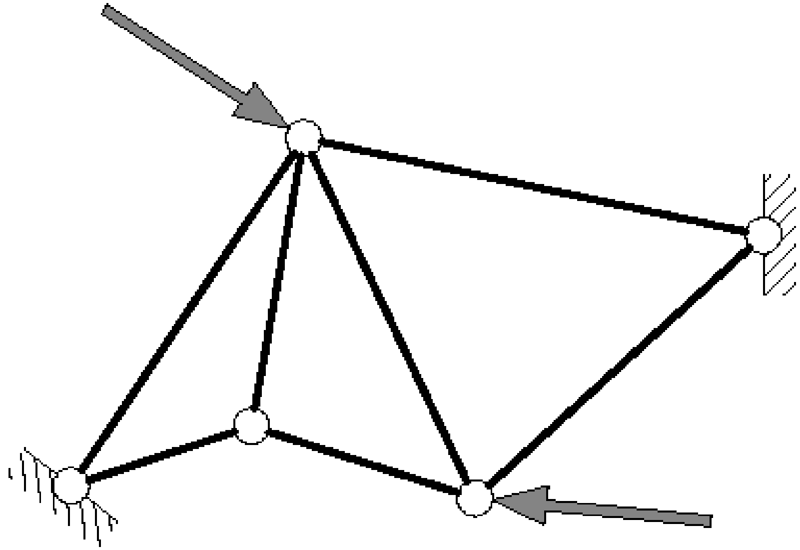
For each type of joint, many additional requirements must be satisfied. These requirements are presented in few tables in EN 1993-1-8; symbols are explained in EN 1993-1-8  1.5.(4), (5), (6).

| Joint | | Table | Comments |
|---|---|---|---|
| Chord | Web members | | |
| CHS | CHS | 7.1 | - |
| RHS | CHS, RHS | 7.8, 7.9 | - |
| I-beam | CHS, RHS | 7.20 | - |
| C-section | CHS, RHS | 7.21 | C-section for chord is acceptet, but in this situation local bending moments must be taken into consideration (this means, this structure is not ideal truss). |

Geneally, requirements presentes in tables, are as follow:

→ #9 / 49

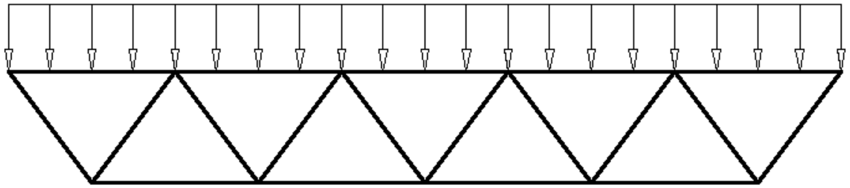
min ≤ (depth of HS) / (thickness of its wall) ≤ max



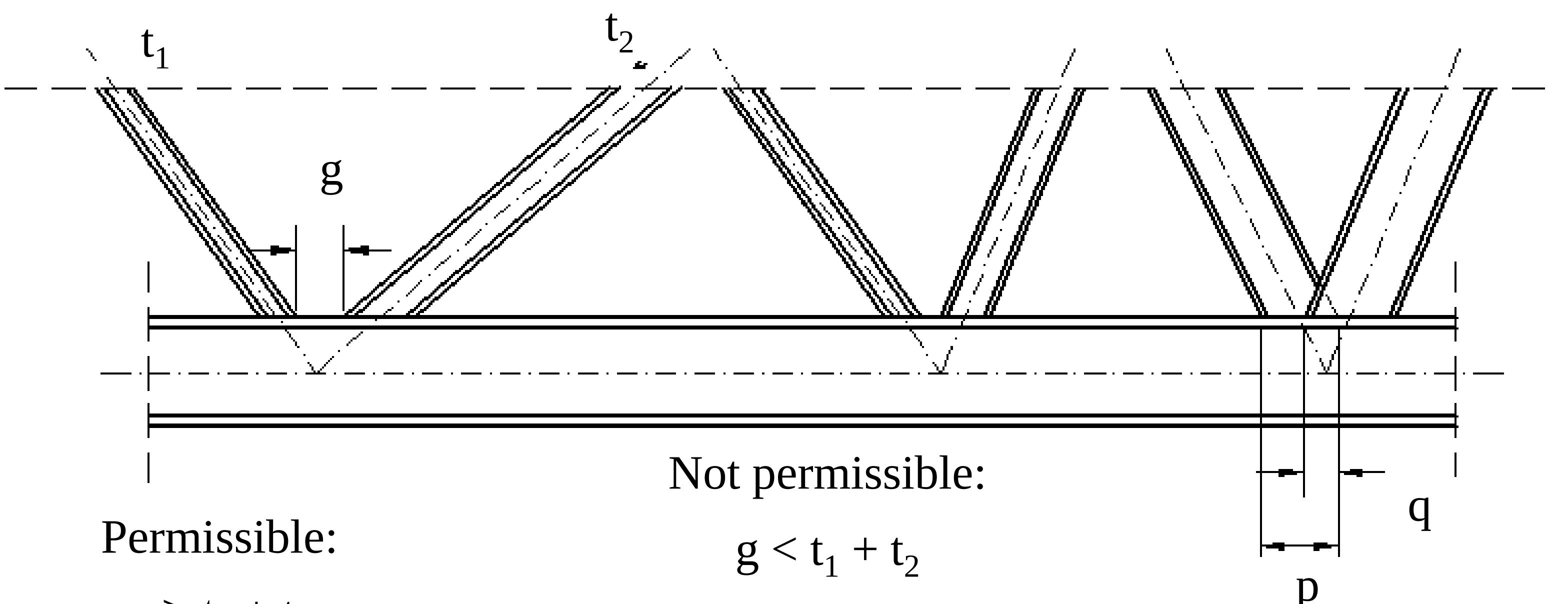
Truss – theory (idealization):

- Straight bars only;
- Hinge joints;
- Forces in joints only;

Photo: Author



Truss purlin not satisfy requirement „forces in joints only”. The same in case of force applied out of joint.




Not permissible:

$g < t_1 + t_2$

or

$q / p < 0,25$

Permissible:

$g \geq t_1 + t_2$

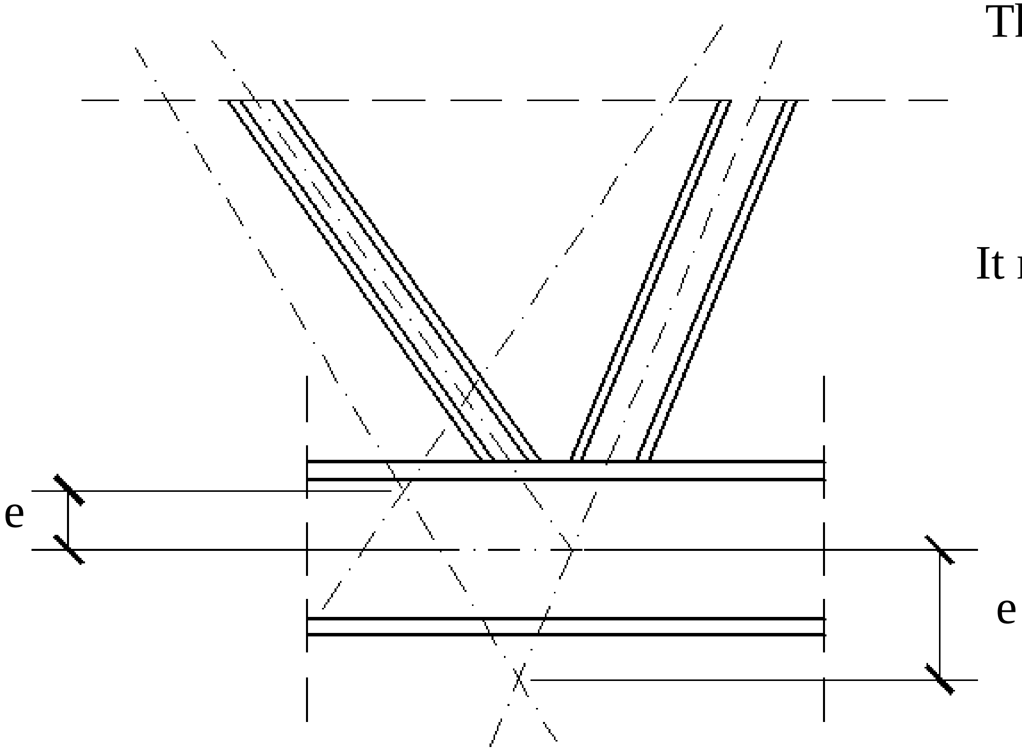
Permissible:

$q / p \geq 0,25$

EN 1993-1-8 7.1

Photo: Author

Results:



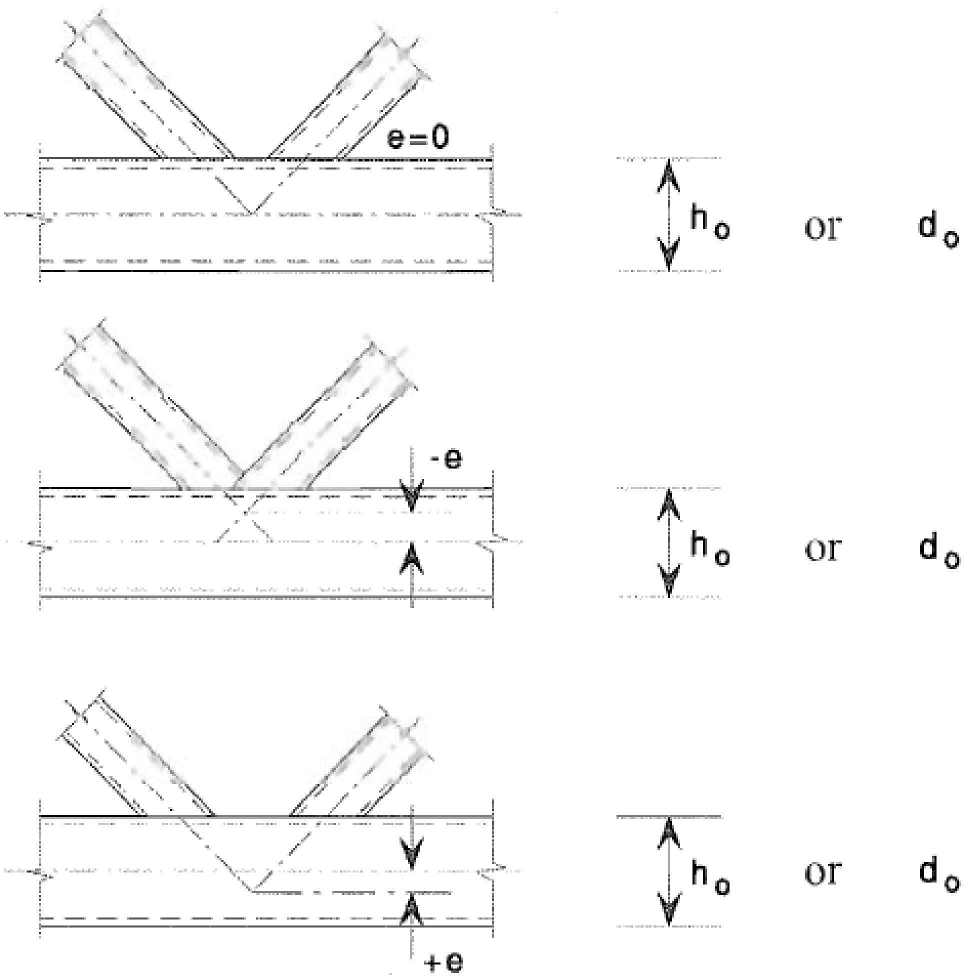

There is possible, that we must trace other axis of member to satisfy requiremen for

$$g \geq t_1 + t_2 \text{ or } q / p \geq 0{,}25$$

It makes eccentricities. Eccentricities make non-zero values of bending moment.

Photo: Author

Photo: EN 1993-1-8 fig. 5.3




Acceptable limits for eccentricities:

$$-0{,}55\ a_0 \leq e \leq +0{,}25\ a_0$$

$$a_0 = h_0 \text{ or } d_0$$

EN 1993-1-8 (5.1a), (5.1b)

Three possibilities must be taken into accout:

- no eccenticities;
- eccentricities inside limits;
- eccentricities outside limits,

## Stiffness

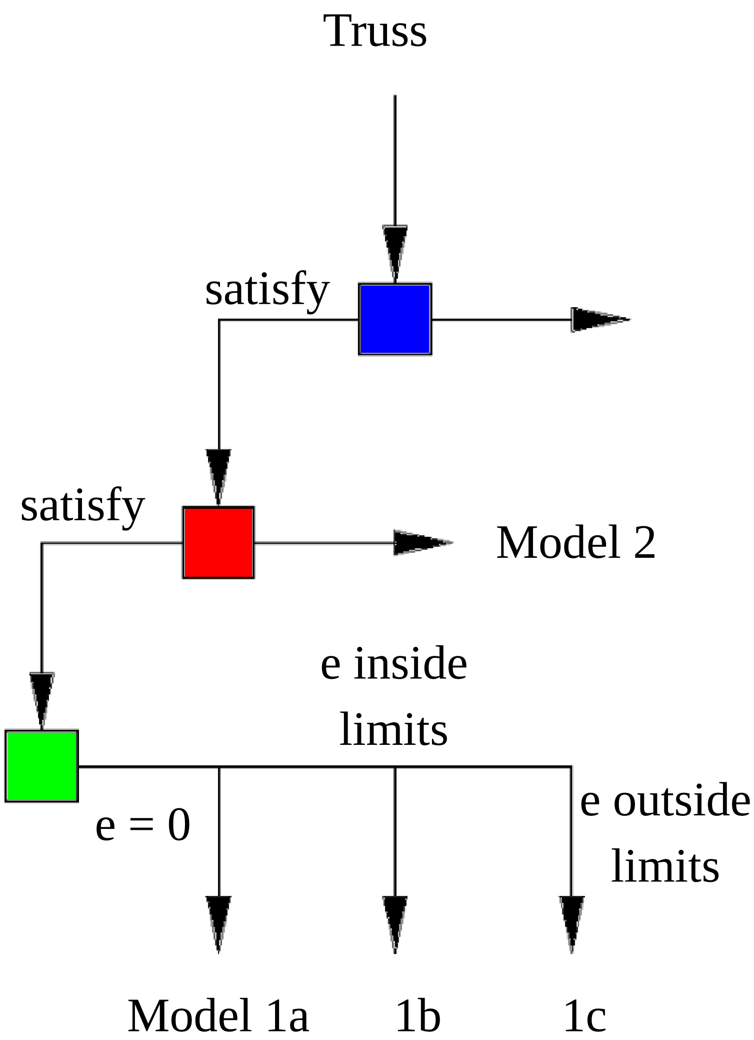



Photo: Author

→ Des #1/ 76

Satisfaction or dissatisfaction of three groups of conditions qualifies truss nodes as:

- all rigid;
- rigid and pinned;
- all pinned with secondary bending moments;
- all perfectly pinned.

Third group of condition („green” condition) will be analysed in Your range of project only.
More information will be presented on lecture #9.

5 various models of truss behavior:

→ #9 / 55

Photo: Author

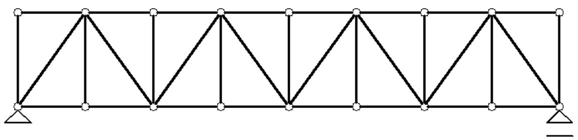
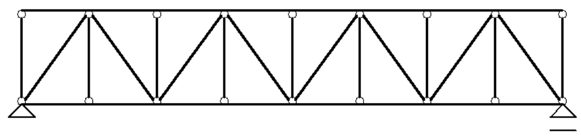
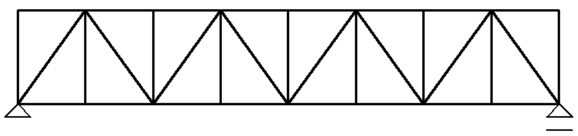
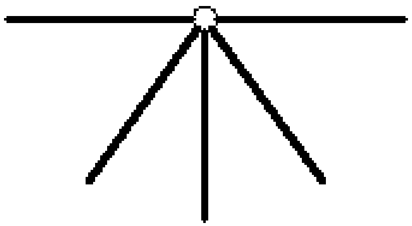
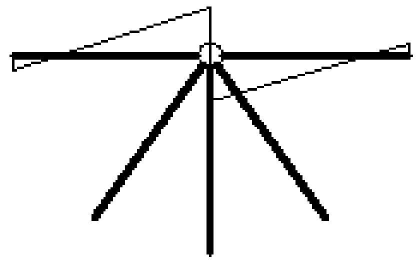
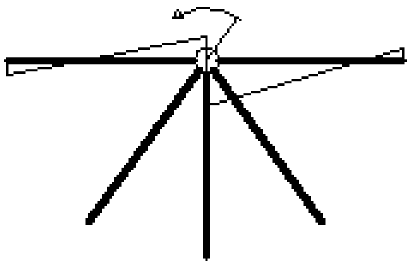
|  |  |  |
|---|---|---|
| 1. Ideal truss. | 2. Continous top / bottom chord. | 3. Frame. |
| Subtypes: 1a, 1b, 1c. | No subtypes. | No subtypes. |
| Hinge joints only. | Hinge and rigid joints. | Rigid joints only. |
| Axial forces for each element; bending moments and shear forces for part of joints and elements in dependence of subtypes. | Axial forces for each element; bending moments and shear forces for part of joints and elements in dependence of type of neighboring joints (hinge / rigid) | Axial forces, bending moments and shear forces for each element and joint |

Photo: Author

3 subtypes of ideal truss (model 1):

| | | |
|---|---|---|
|  |  |  |
| 1a. No bending moments. | 1b. Bending moments (axial force · eccentricity) act on chord only. | 1c. Bending moments (axial force · eccentricity) act on joint and chord. |
| Ideal truss: axial forces only for elements and joints.<br><br>→ #9 / 56 | Axial forces only for most part of elements;<br><br>Axial forces, bending moments and shear forces for part of chords;<br><br>Axial forces for joints. | Axial forces only for most part of elements;<br><br>Axial forces, bending moments and shear forces for part of chords;<br><br>Axial forces and bending moments for joints. |

There are six ways of local destrucion for truss nodes. According to EN 1993-1-8, there are 6 modes of failure, presented on fig. 7.3, 7.4. :

Photo: tatasteel.com

**1. Chord face failure**; phenomenon not occurs for I-chords

**2. Chord wall / web failure**

Photo: R. Feng, Y. Liu, J. Zhu, Tests of CHS-to-SHS tubular connections in stainless steel, Engineering Structures, 199/ 2019

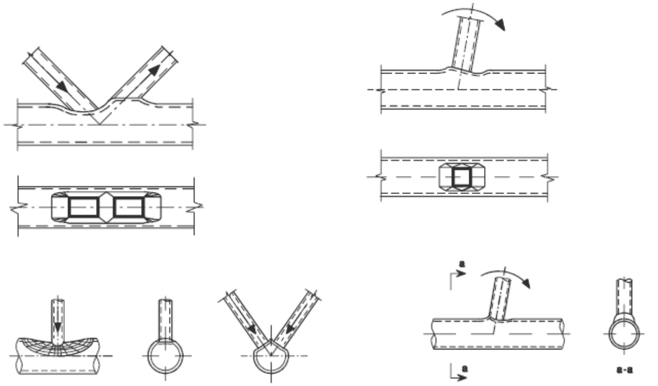
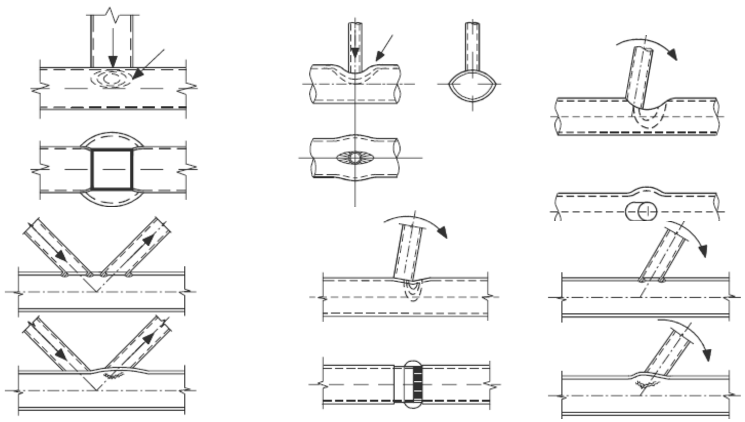
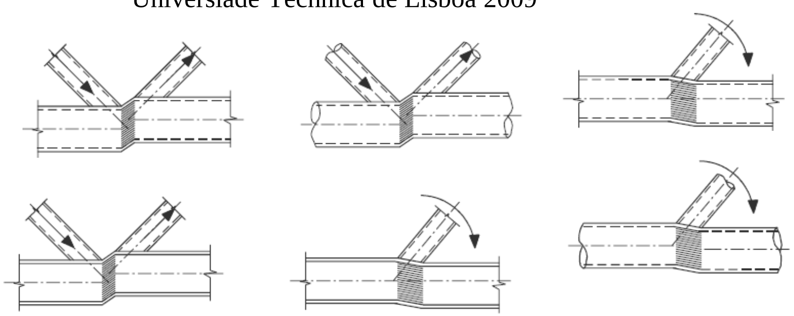


Photo: EN 1993-1-8 fig 7.3, 7.4



## 3. Chord shear failure

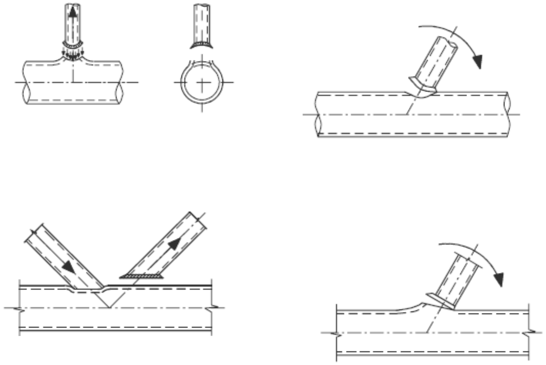
Photo: O. P. Videira, Composite Truss Bridge Decks,
Universiade Technica de Lisboa 2009



## 4. **Punching shear**; phenomenon not occurs for I-chords

Photo: semanticscholar.org


Photo: EN 1993-1-8 fig 7.3, 7.4



# 5. Brace failure

# 6. Local buckling

Photo: eqclearinghouse.org

Photo: J. Packer, theoretical analysis of welded steel joints in rectangular hollow sections. PhD thesis, University of Nottingham. , 1978

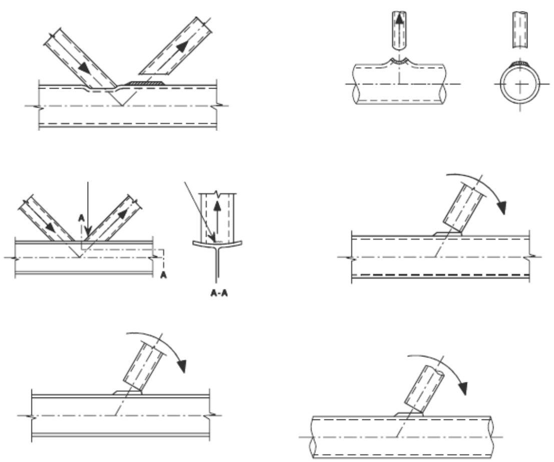
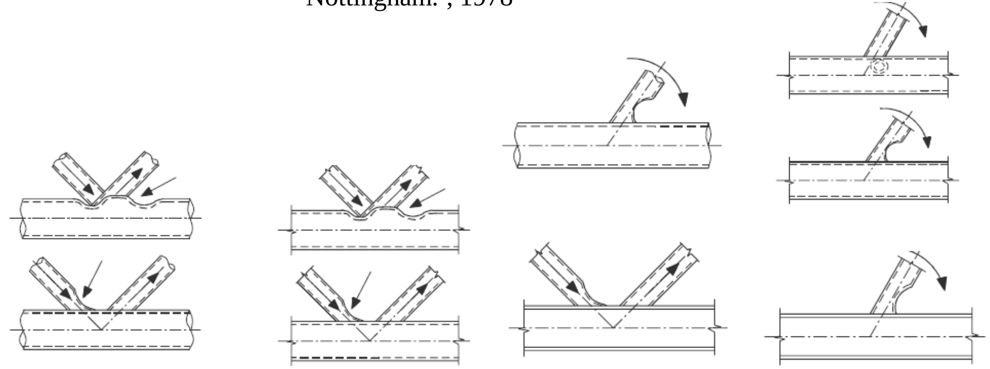




Photo: EN 1993-1-8 fig 7.3, 7.4



For various shapes of nodes and various cross-sections of members (CHS, RHS, I-beam), various modes of failure are the most dangerous. Resistance of node is defined in dependence of the most dangerous modes.

Photo: waldenstructures.com

Nodes are loaded by axial forces ($N_{i,\,Ed}$) from web members, first of all. In many cases, local bending moments must be taken into consideration. For flat truss we analysed only bending moments in plane (ip) of truss ($M_{ip,\,i,\,Ed}$). Additionally, for multi-chords truss, bending moments out of plane (op) of truss ($M_{op,\,i,\,Ed}$) must be taken into consideration.

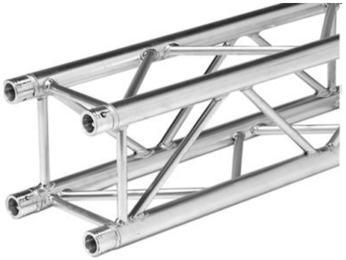


Photo: conference-truss-hire.co.uk

There are different formulas for resistance of truss nodes ($N_{i,\,Rd}$, $M_{ip,\,i,\,Rd}$, $M_{op,\,i,\,Rd}$) for different shapes of nodes and different modes of failue. These formulas are presented on EN 1993-1-8, tab. 7.2-7.7, 7.10-7.19, 7.21, 7.22, 7.24. Way of check of resistance depends on type of cross-sections of members (CHS, RHS, I-beam).

| Web memebers - chord | Chapter | Tables | General formula | Comment |
|---|---|---|---|---|
| CHS - CHS | 7.4 | 7.2, 7.3, 7.4, 7.5, 7.6, 7.7 | $N_{i, Ed} / N_{i, Rd} + (M_{ip, i, Ed} / M_{ip, i, Rd})^2 + M_{op, i, Ed} / M_{op, i, Rd} \leq 1,0$ | ◆ Gusset plates are possible<br>◆ Tab. 7.4 - I/H is not a chord |
| C/R HS - RHS | 7.5 | 7.10, 7.11, 7.12, 7.13, 7.14, 7.15, 7.16, 7.17, 7.18, 7.19, | $N_{i, Ed} / N_{i, Rd} + M_{ip, i, Ed} / M_{ip, i, Rd} + M_{op, i, Ed} / M_{op, i, Rd} \leq 1,0$ | ◆ Gusset plates are possible |
| C/R HS - I / H | 7.6 | 7.21, 7.22 | $N_{i, Ed} / N_{i, Rd} + M_{ip, i, Ed} / M_{ip, i, Rd} \leq 1,0$ | ◆ I / H chord is not recommended for multi-chords trusses |
| C/R HS - C | 7.7 | 7.24 | Different formulas | ◆ Secondary moments should be taken into account |

Modern truss: for joints must be checked:

Stiffness (#t / 39-49)

Resistance (#t / 50-55)

Connecting element (welds; #17 example 4)

Old type truss (designed „under rule” of old Polish Standard) - only connecting elements (welds or bolted joints) were calculated; gusset plates were drawed; ideal hinge of joints was accepted without checking.

Old type:

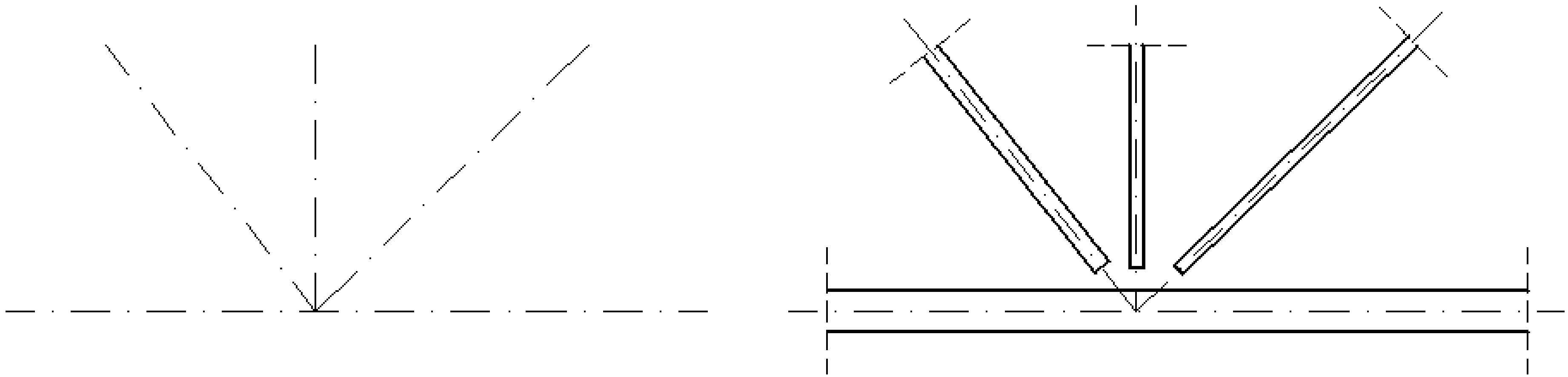
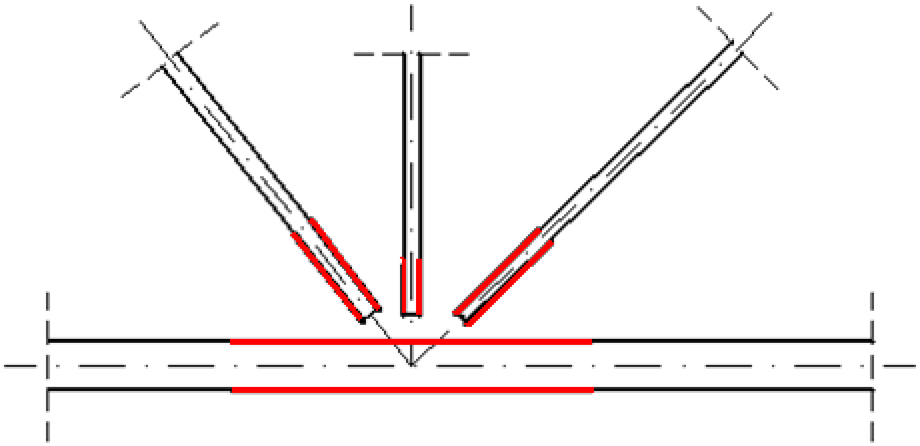


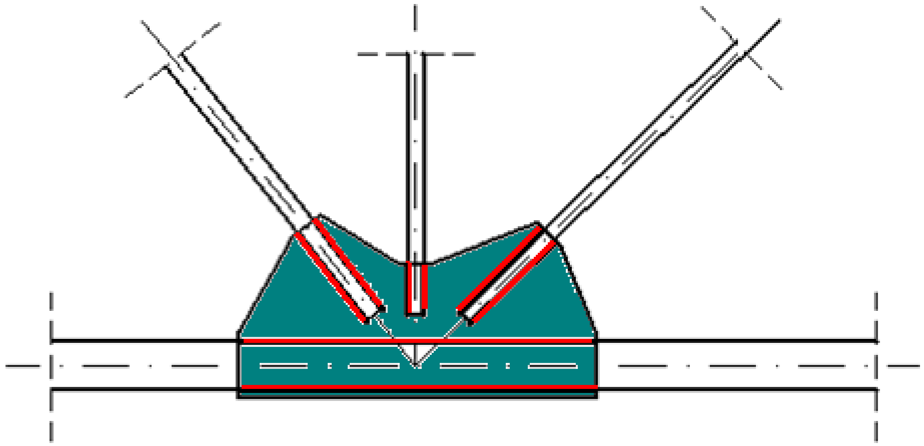
Photo: Author

Axes must intersect in one point;

Ends of members must be as close as possible each other (10-15 mm place for welds);

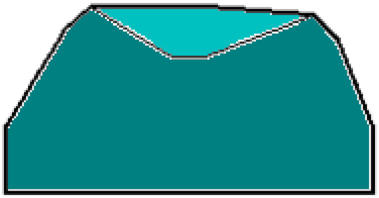
Photo: Author





There must be marked length of connecting elements (length of weld / space for bolts) along members;

Outline of gusset plate = ends of connecting elements;



Reflex angles are not accepted.

# Old type and modern trusses cam be designed with gusset plates or without gusset plates

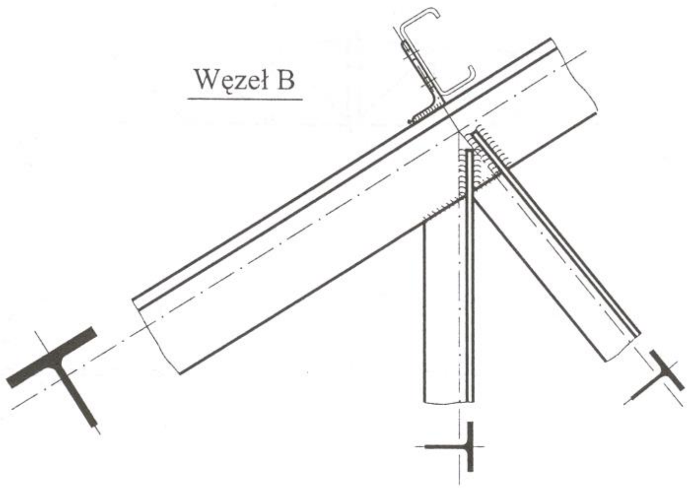
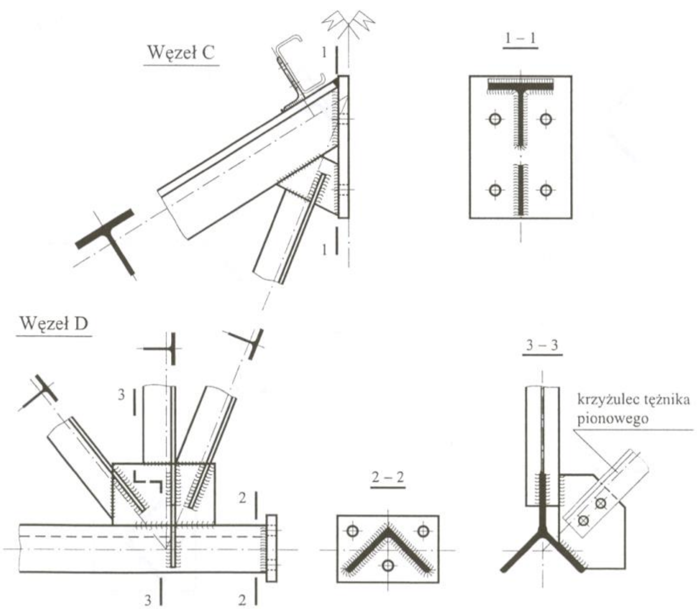
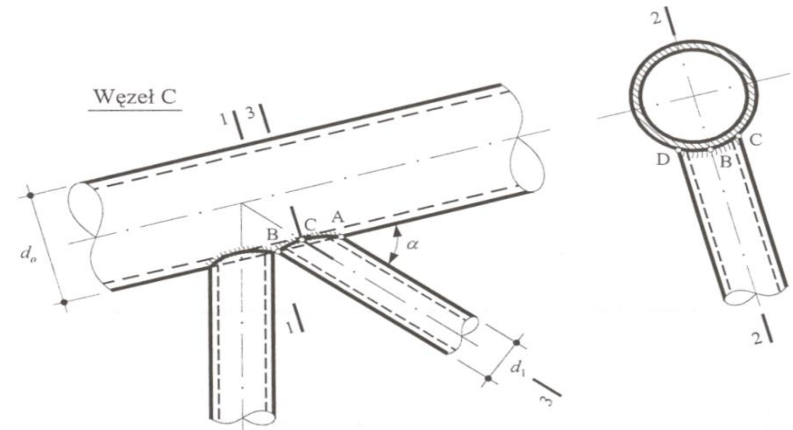
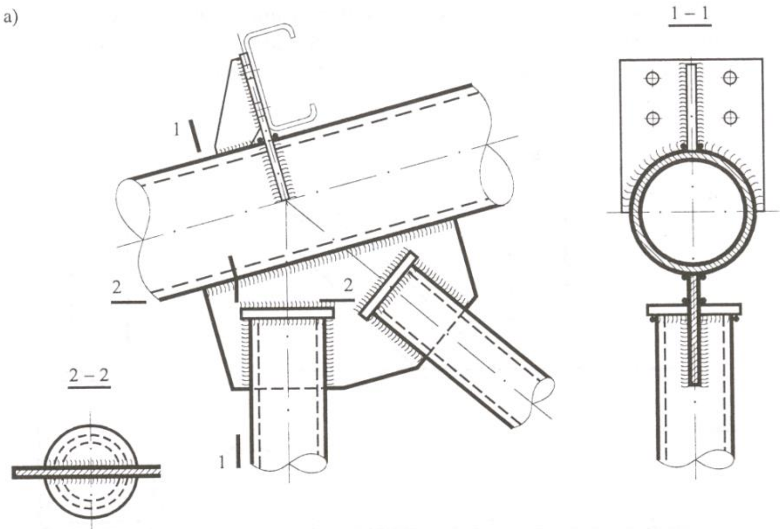



Photo: Konstrukcje stalowe, K. Rykaluk,
Dolnośląskie Wydawnictwo Edukacyjne
Wrocław 2001




a)



There is need additional flat surface ("mounting table") for support of purlins when we use CHS

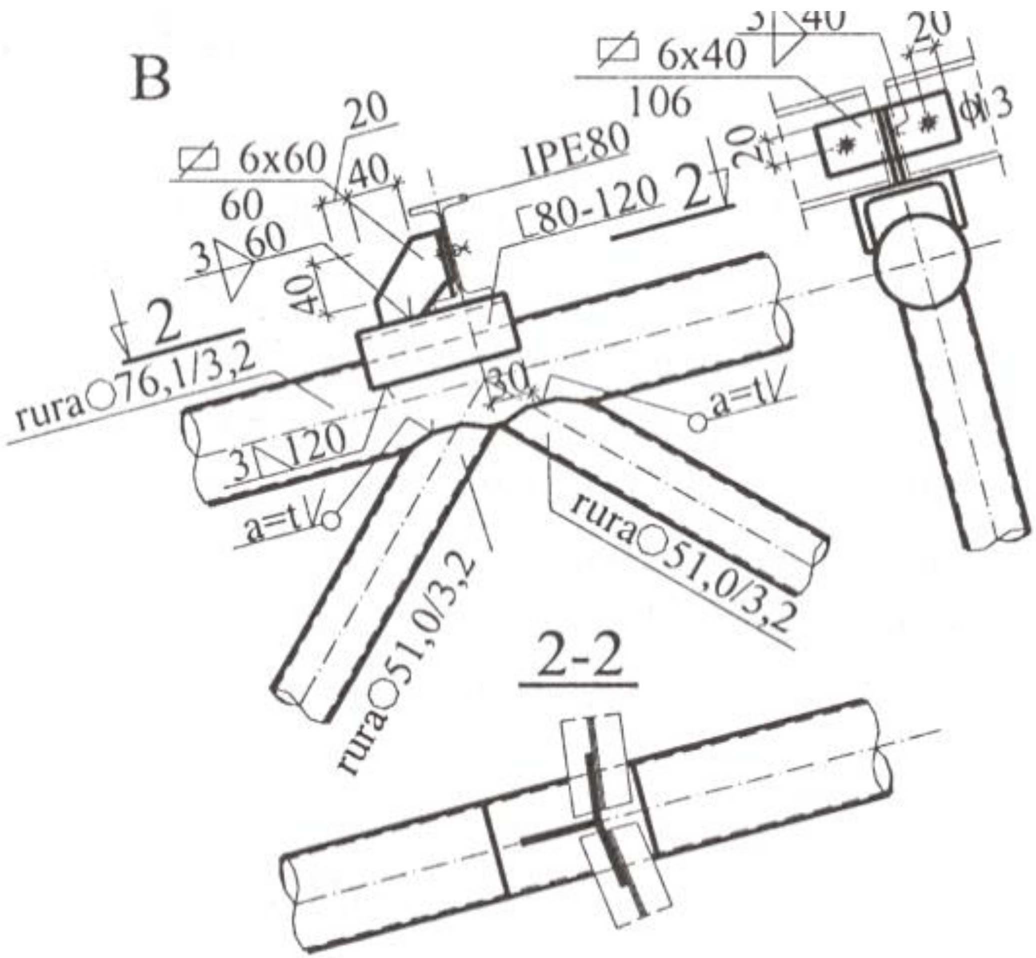
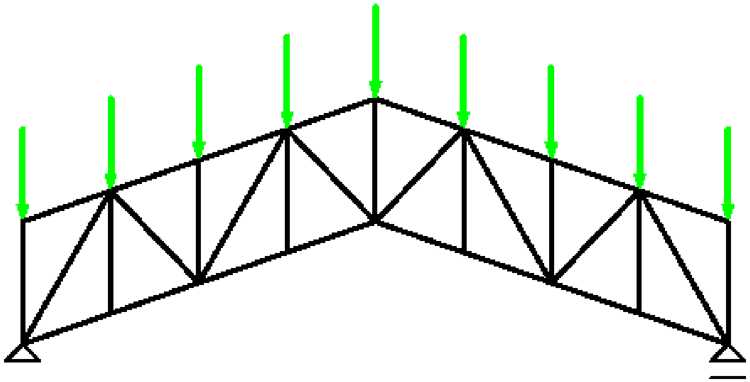
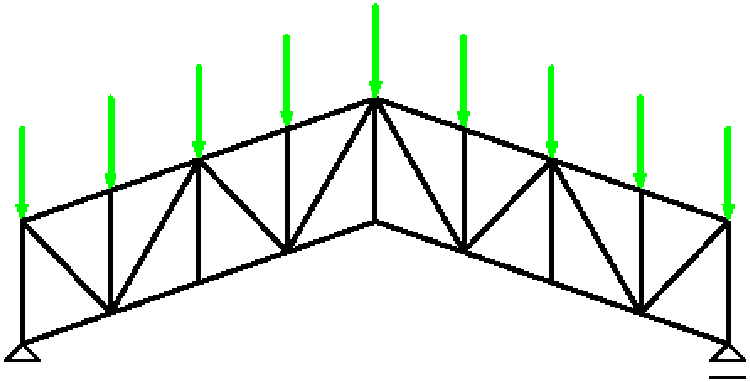



Photo: Konstrukcje stalowe, K. Rykaluk, Dolnośląskie Wydawnictwo Edukacyjne Wrocław 2001

# Difference between two directions of web members

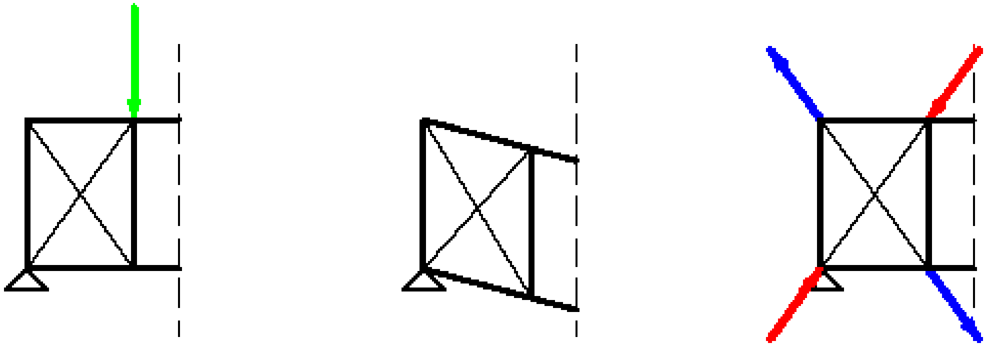




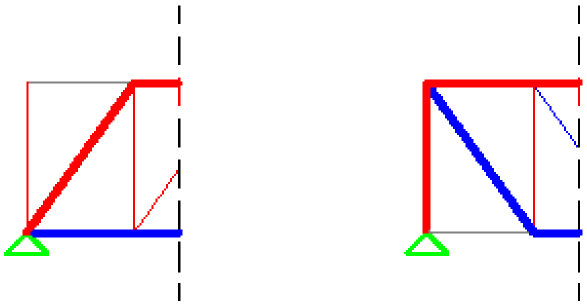
→ #9 / 95

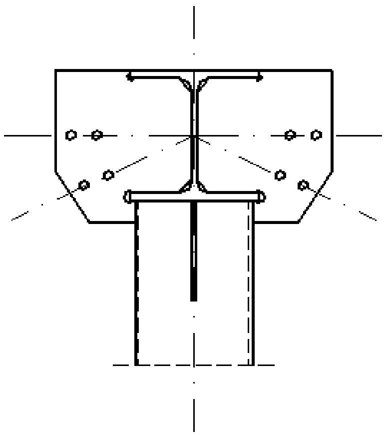
Deformations: elongation (tensile force) and abridgement (compressive force):

Photo: Author

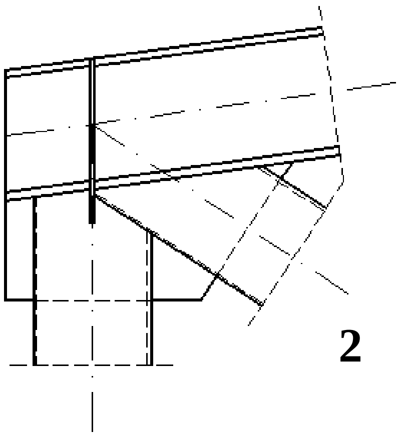


Transmission of forces form chords to supports and zero force members:





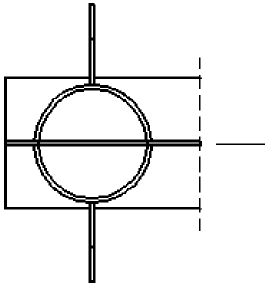
1

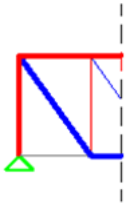


2

Members with big value of forces should goes to nodes as close as posible. But, first of all, we keep continuity of chords.

3





Forces : |1| ≈ |2| > |3| > |4|

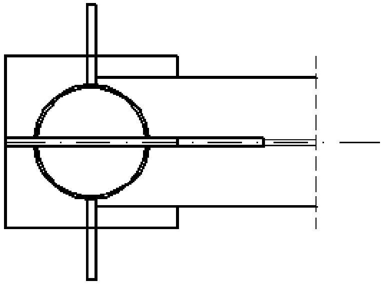
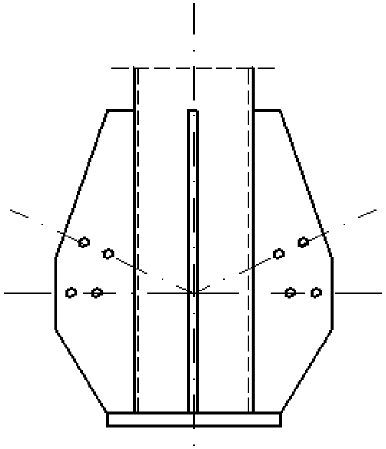
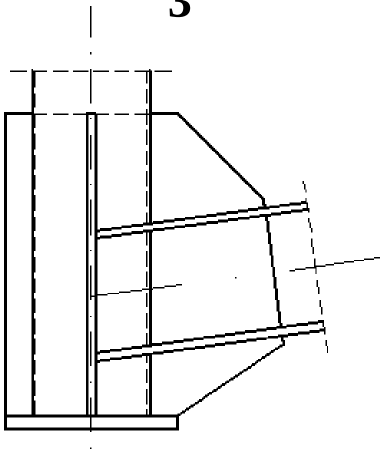


Photo: Author

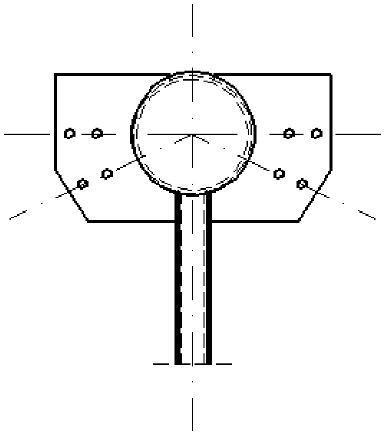
3

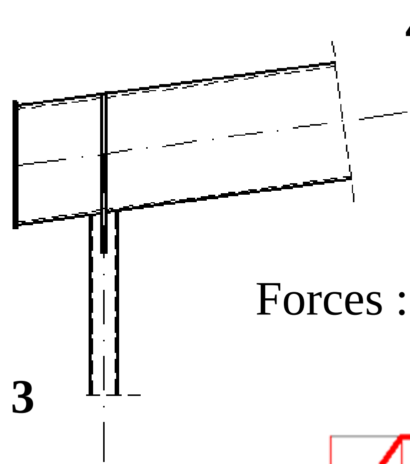




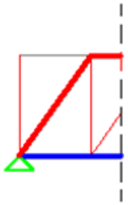
4

Example: web members CHS, chords HEB






Forces : |1| ≈ |2| > |3| > |4|



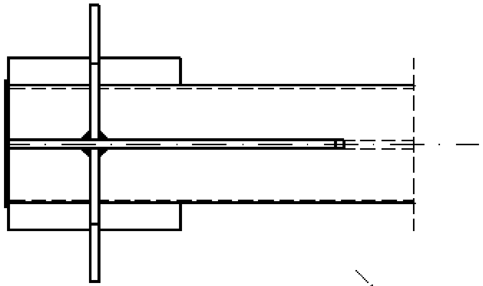
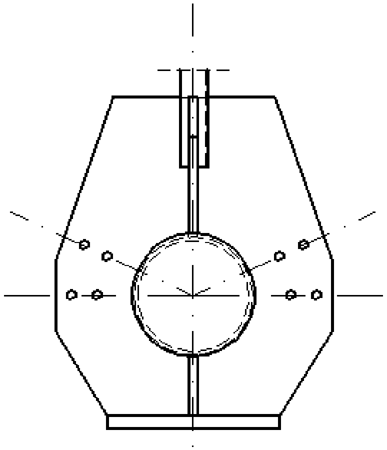
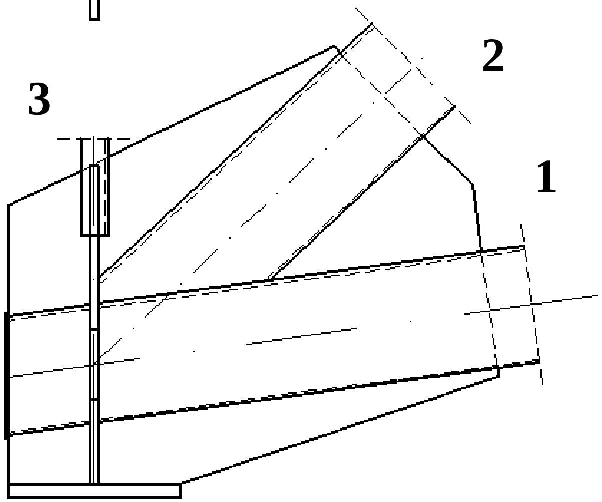


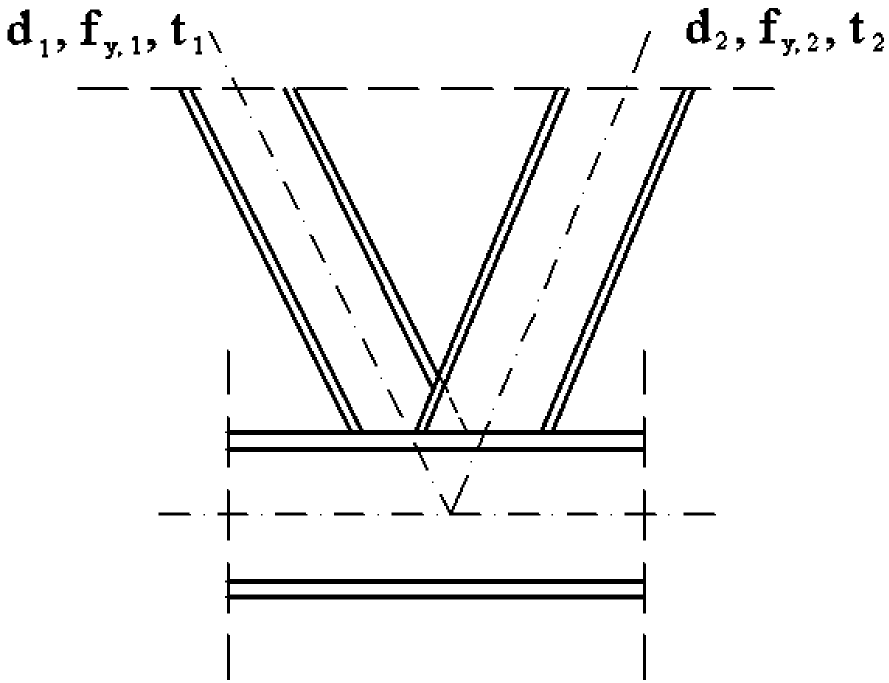
Photo: Author






Example: web members CHS, chords CHS

When it is necessary to undercut bars in joint, the "weaker" truss bar is undercut:




$d_1 \leq d_2$

and

$f_{y,1}\ t_1 \leq f_{y,2}\ t_2$

Photo: Author

EN 1993-1-8 7.1

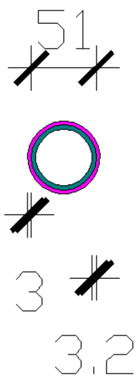



Photo: Author

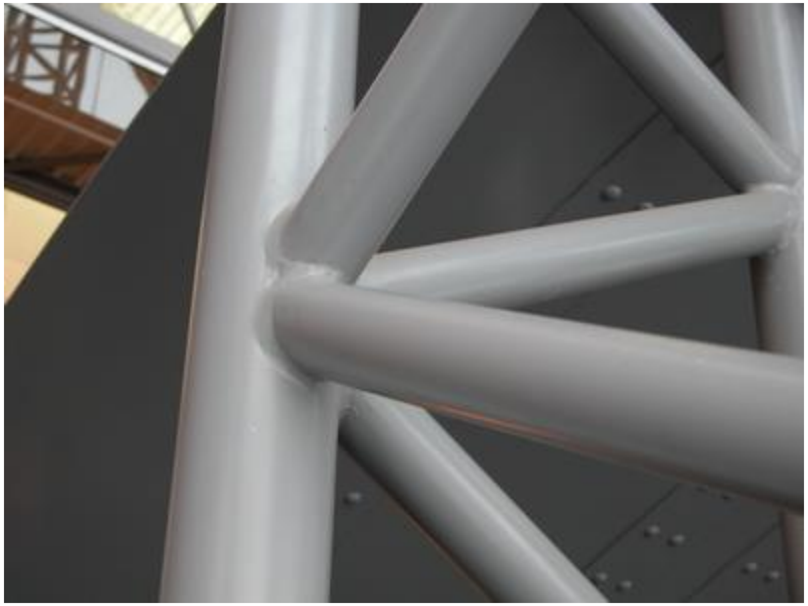


Photo: tboake.com

Welds: example 4 Lec #17

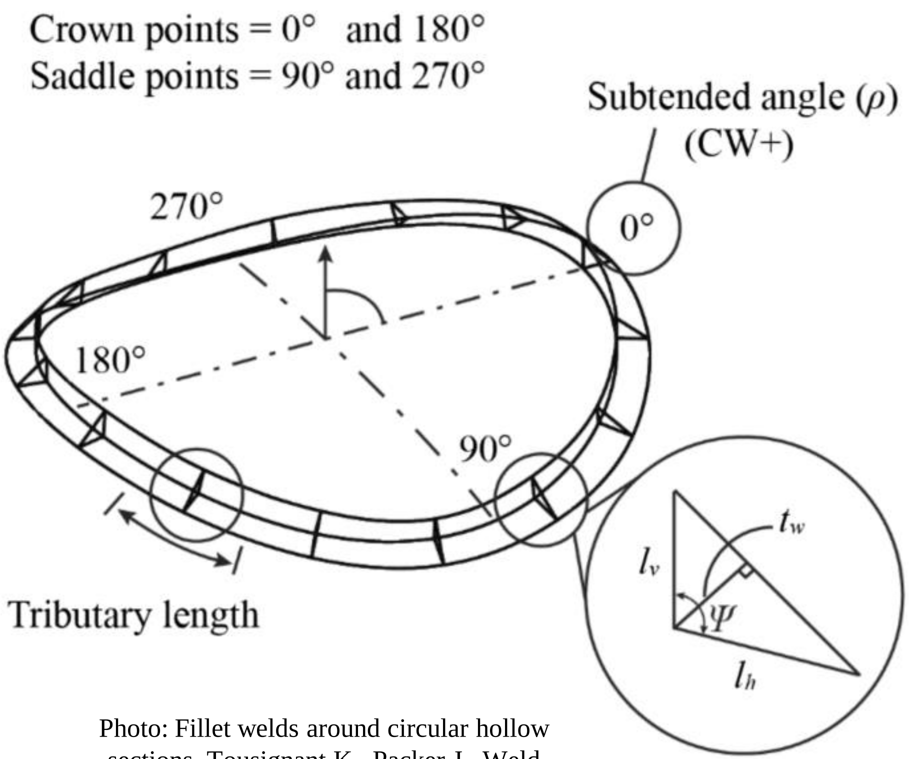



Photo: Fillet welds around circular hollow sections, Tousignant K., Packer J., Weld World 63, 421–433 (2019)

Angle Ψ between two CHS varies along weld. It is possible for ranges to occur simultaneously in which

- $\Psi \geq 120^{o}$ (fillet welds tested experimentally);
- $120^{o} > \Psi > 60^{o}$ ("normal" fillet welds);
- $\Psi \leq 60^{o}$ (fillet welds counted as butt welds with partial penetration).

→ #17 / 56

Therefore, technical requirements specify need to use both butt and fillet welds, depending on angle value.

Standard EN 1090-2 presents technical requirements for making of welds CHS-CHS. Technical requirements for butt welds are presented on fig. E2.

→ #17 / 57

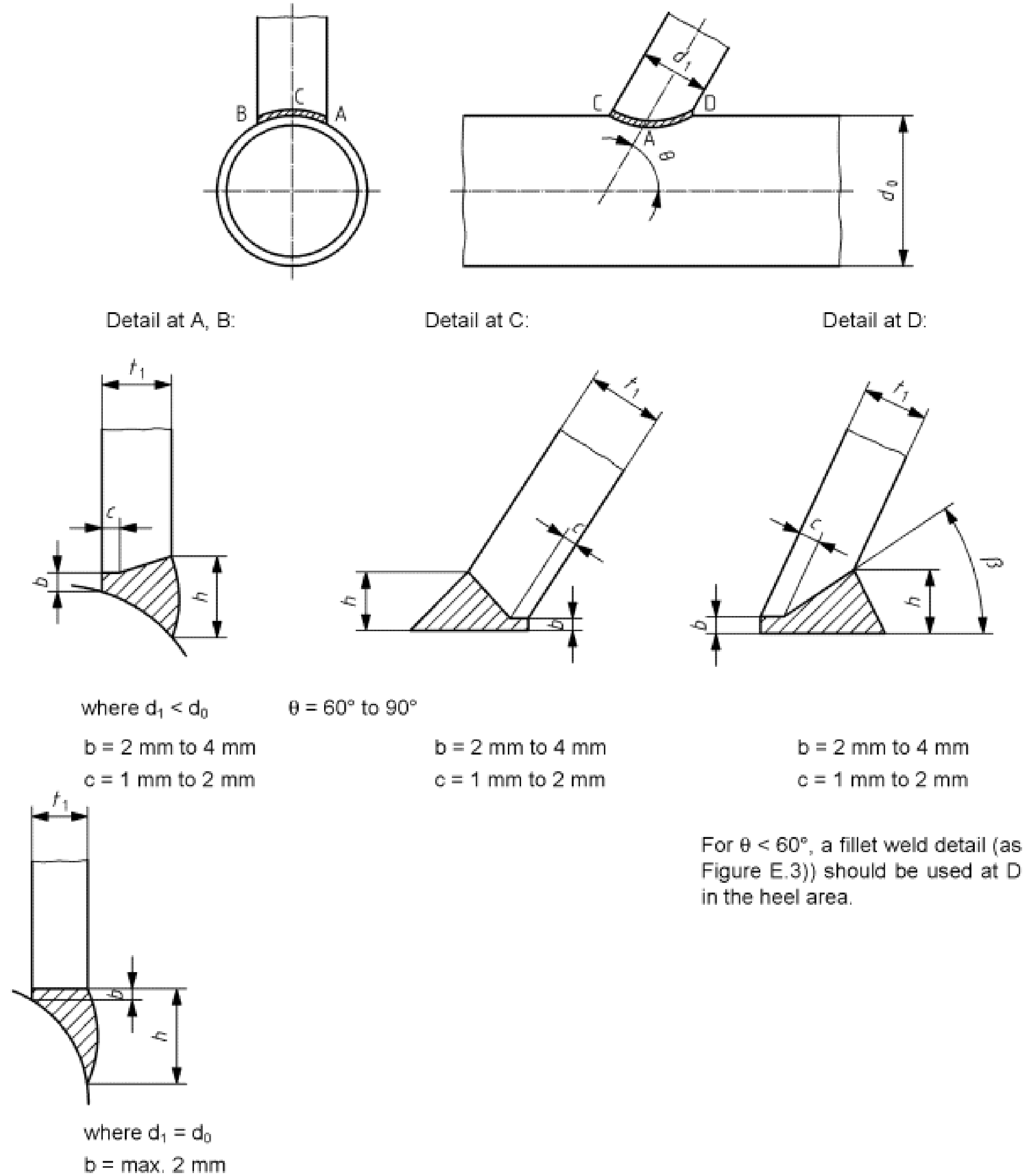



Photo: EN 1090-2 fig. E2

Technical requirements for fillet welds are presented on fig. E2.

→ #17 / 58

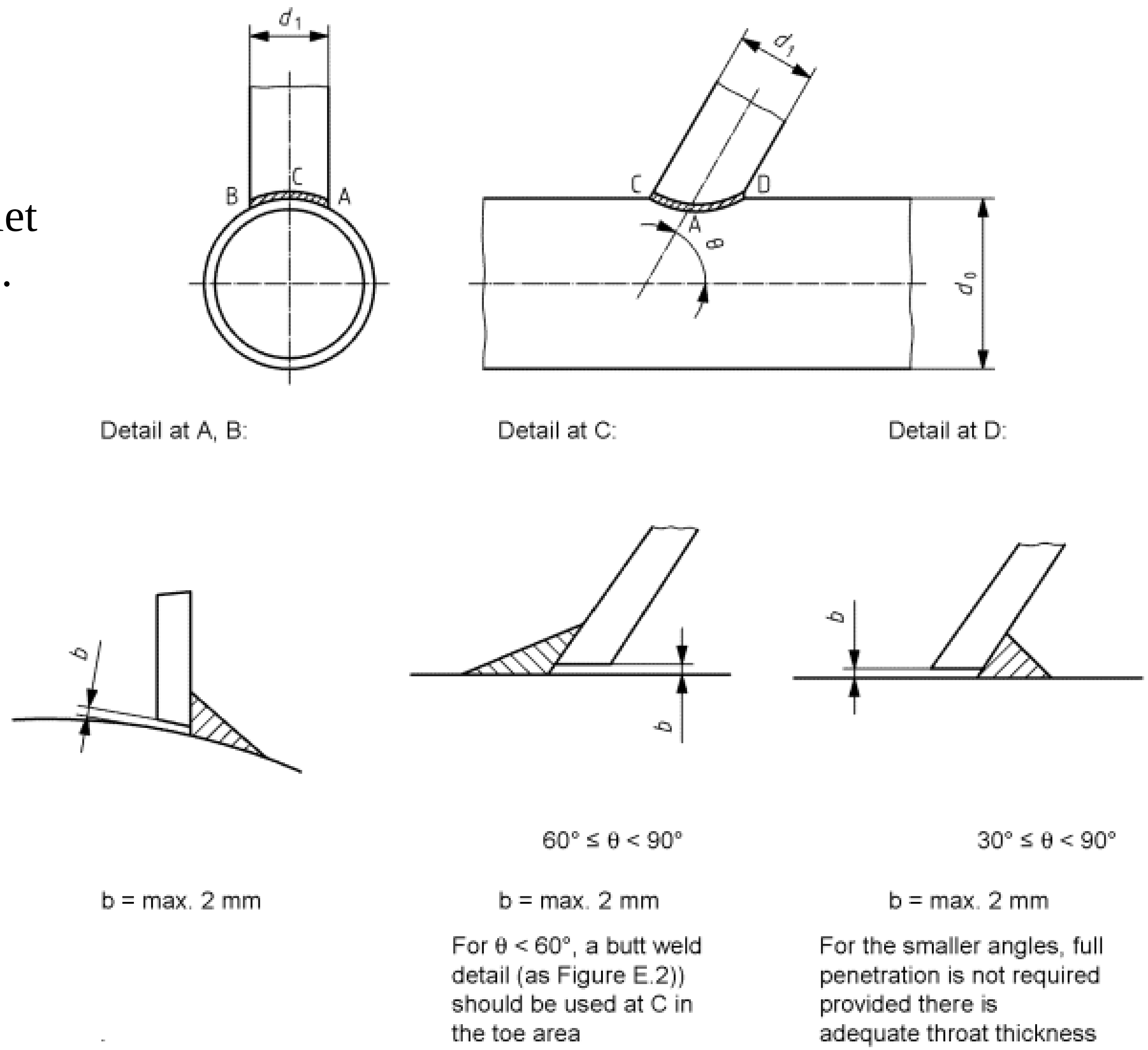



Photo: EN 1090-2 fig. E3

## Conclusions

Accurate weld analysis in the case of CHS joints is extremely complicated:

- weld forms a curved line in three dimensions;
- opening angle between the CHS walls varies along the joint;
- it is recommended to use both butt and fillet welds in one node;
- cooperation of these two types of welds is little described in literature.

→ #17 / 62

# Splice joints of hollow sections

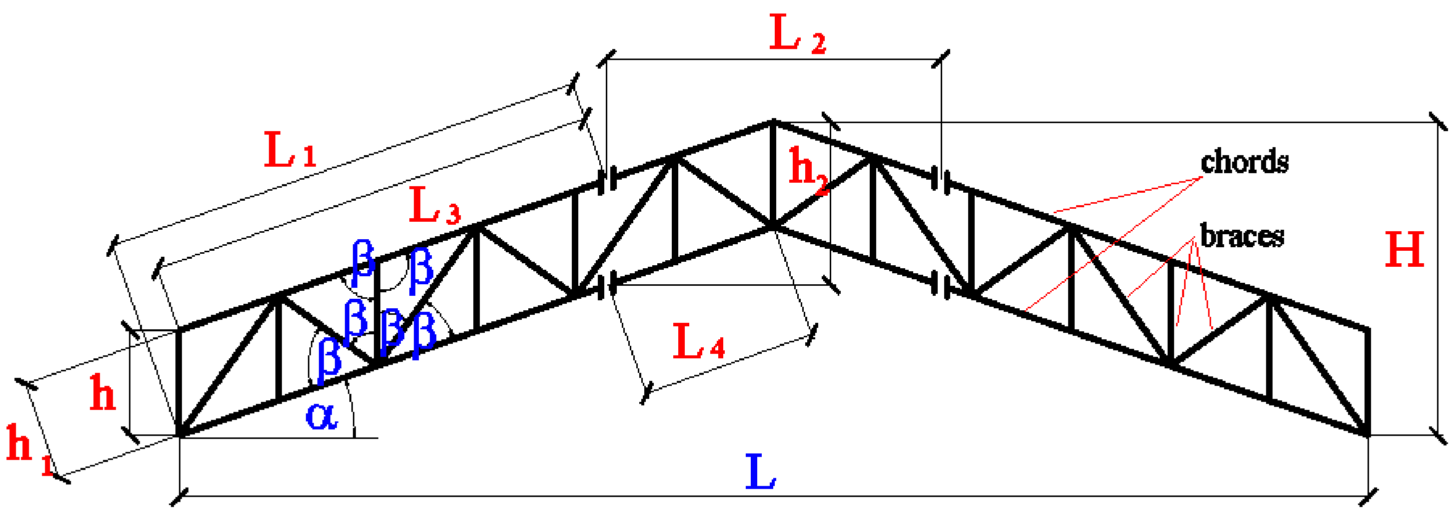



Photo: Author

Because of transport loading gauge, long structre should be divided into transport members. There is good idea, that max length of membes ($L_1$ or $L_2$) should not be greated than 12,00 m. Members are connected each other by splice joints on construction site.

For I-beam chords, there can be adopted shear joint (→ Des #2).

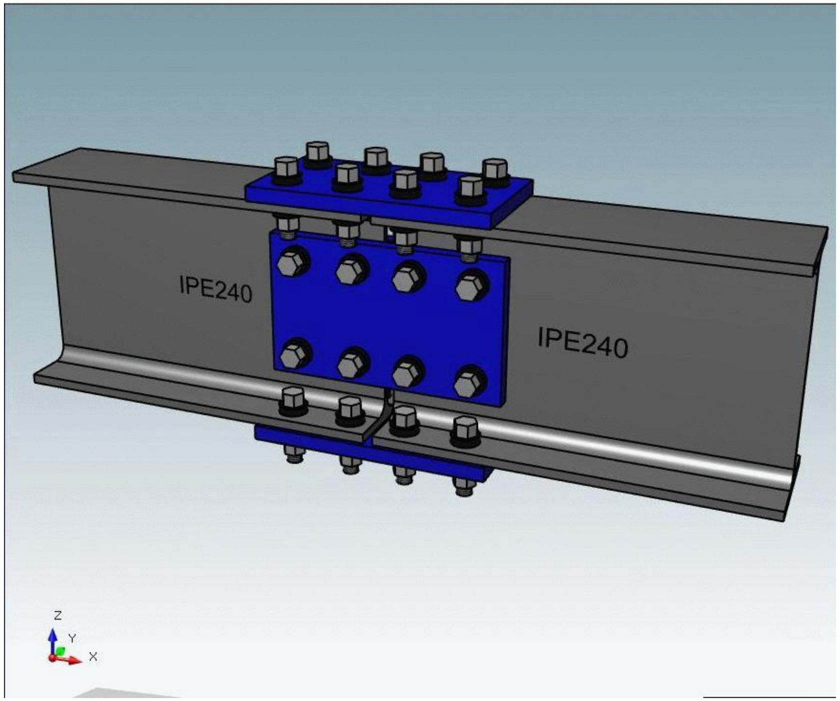



Photo: gsi-eng.eu

Photo: encrypted-tbn0.gstatic.com

For hollow sections, there is used specific tension joint.

Photo: zs4-sanok.pl

The problem is, that in EN 1993-1-8 tension joint is presented for I-beams only. The same in old Polish Standard PN B 03200.

Such type of joint is mentioned in EN 1993-3-1. Information is incomplete:

- In case ot tensile force, preloaded bolts are recommended;
- Diameter of bolts should be bigger than 12 mm;
- Tensile force in one bolt $N_b = N_{Ed}\ k_p / n$

  n – number of bolts, $k_p$ = 1,2 for preloaded bolts; 1,8 for „normal” bolts
- For thickness of flange, shear resistance of flange along of connected circular leg section sould be calculated (but no information about way of calculation resistance of flange for shera force);
- The same, bending moment $M = N_{Ed}\ (D_b – D_i) / 2$ must be calculated (but no information about way of calculation resistance of flange for bending moment).

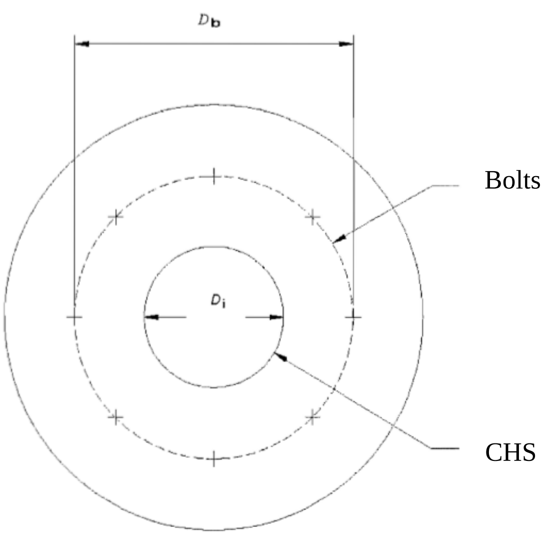



Photo: EN 1993-3-1 fig. 6.1

Generalisations of calculation of tension joint for hollot section for old Polish Standard and Eurocode are presented on literature.

| Generalisation of | Literature | Comments | Page |
|---|---|---|---|
| PN B 03200 | J. Bródka, M. Broniewicz, Konstrukcje stalowe z rur, Arkady 2001 | ◆ Separated procedures for CHS and RHS;<br>◆ No clear influence of longitudinal stiffeners for resistance;<br>◆ Similar methods, only few small differeces; | |
| EN 1993-1-8 | Access Steel SN044a-EU Design models for splices in structural hollow, Internet edition | | #t / 74 - 76 |

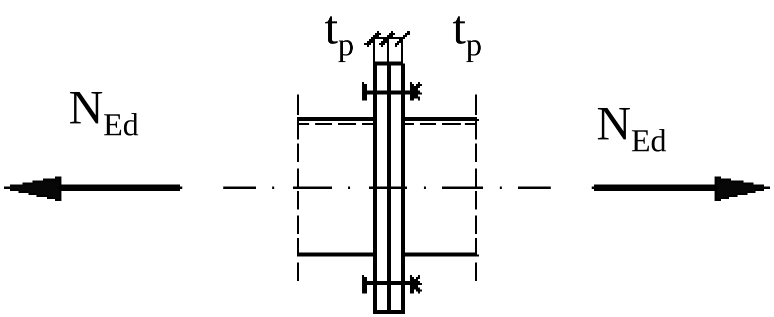
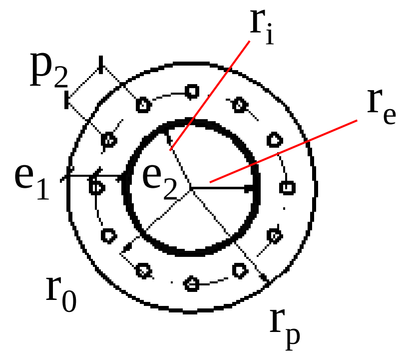



Photo: Author




$r_e = r_i + t_{CHS}$

$r_0 = r_e + e_2$

$r_p = r_0 + e_1$

$d_0$ – hole of bolt diameter

n – numer of bolts

$r_p = 2\, r_0 - r_i$

$\mathbf{N_{Rd} = min\,(N_{Rd1}\ \ ;\ \ N_{Rd2})}$

$\mathbf{N_{Rd1} = t_p^{\,2}\, f_{yp}\, \pi\, k \,/\, (2\, \gamma_{M0})}$

$k = [k_2 + \sqrt{(k_2^{\,2} - 4k_1^{\,2})}] \,/\, (2\, k_1)$

$k_1 = \ln\,(r_0 / r_i)$

$k_2 = k_1 + 2$

$\mathbf{N_{Ed} / N_{Rd} \leq 1{,}0}$

$\mathbf{N_{Rd2} = n\, F_{t,\, Rd} \,/\, k_3}$

$F_{t,\, Rd} \rightarrow$ #18 / 50

$k_3 = 1 - 1 / k + 1 / (k\, k_5)$

$k_5 = \ln\,(r_{eff} / r_0)$

$r_{eff} = r_e + e_2 + e_{eff}$

$e_{eff} = \min\,(e_2\ \ ;\ \ 1{,}25\, e_1)$

$2{,}2\, d_0 \leq p_2 \leq \min\,(14\, t_p\ ;\ 200\ \text{mm})$

$d_0 = d + 2\ \text{mm}\ \ (d \leq 24\ \text{mm})$

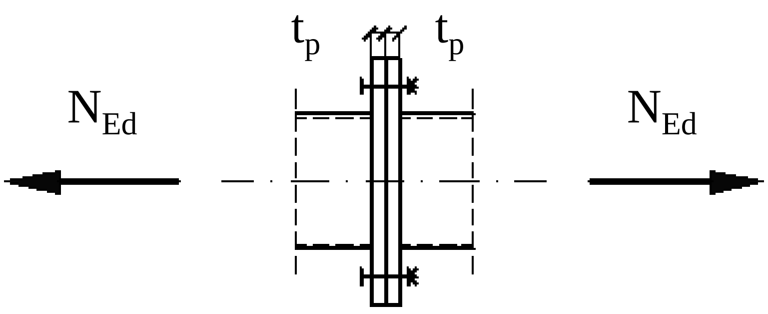
$d_0 = d + 3\ \text{mm}\ \ (d > 24\ \text{mm})$

$1{,}2\, d_0 \leq e_2\ \leq (1{,}5 - 2{,}0)\, d$

$1{,}2\, d_0 \leq e_1$

Additionally, welds should be calculated according to Lec #17 example 4.

Part of bolt's location is not recommended



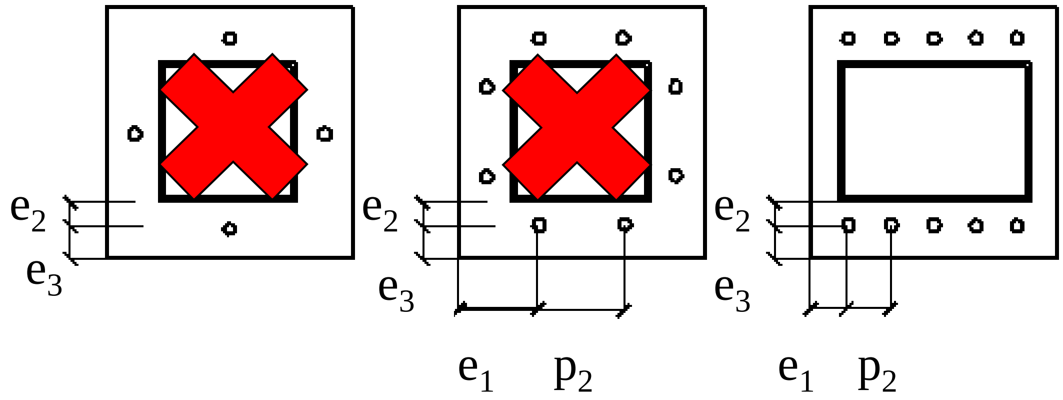




Photo: Author

$12\ \text{mm} \leq t_p \leq 26\ \text{mm}$

$4 \leq n \leq 2 + 2\ h_{RHS} / p_2$

$d_0 = d + 2\ \text{mm}\ \ (d \leq 24\ \text{mm})$

$d_0 = d + 3\ \text{mm}\ \ (d > 24\ \text{mm})$

$b_{red} = e_2 + t_{RHS} - d / 2$

$1{,}2\ d_0 \leq e_2 \leq (1{,}5 - 2{,}0)\ d$

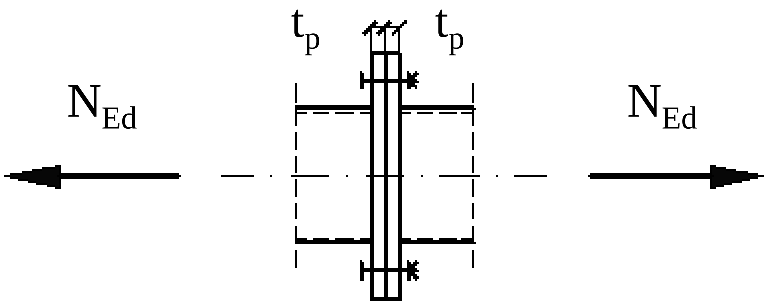
n – number of bolts

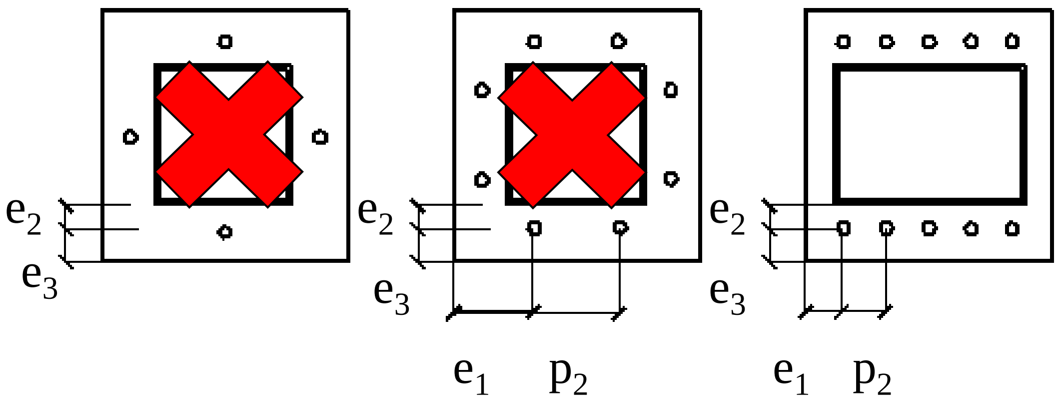
$1{,}2\ d_0 \leq e_1$

$2{,}2\ d_0 \leq p_2 \leq \min (5{,}0\ d\ \ ;\ \ 14\ t_p\ \ ;\ \ 200\ \text{mm})$

$e_3 = 1{,}25\ e_2$

Photo: Author




$\sqrt{\{K\ N_{Ed} / [n\ (1+\delta)]\}} \leq t_p \leq \sqrt{(K\ N_{Ed} / n)}$

$K = 4\ b_{red} / (0{,}9\ f_{yp}\ p_2 / \gamma_{M0})$

$\delta = 1 - d_0 / p_2$

$\mathbf{N_{Rd} = min\ (n\ F_{t,\ Rd}\ \ ;\ \ n\ B_{p,Rd}\ \ ;\ \ N_{1,\ Rd})}$

$\mathbf{F_{t,\ Rd}} \rightarrow$ **#18 / 50**

$\mathbf{B_{p,Rd}} \rightarrow$ **#18 / 95**

$\mathbf{N_{1,\ Rd} = t_p^{\ 2}\ (1 + \delta\ a_1)\ n / (K\ \gamma_{M2})}$

$a_1 =$

$= [(K\ S_{Rt} / t_p^{\ 2}) - 1]\ (e_3 + 0{,}5\ d) / [\delta\ (e_3 + e_2 + t_{RHS})]$

$S_{Rt} = A_s \cdot \min\ (0{,}65 f_{ub}\ \ ;\ \ 0{,}85 f_{yb})$

$\mathbf{N_{Ed} / N_{Rd} \leq 1{,}0}$

Additionally, welds should be calculated according to Lec #17 example 4.

In above procedures, it is assumed that sufficient resistance of joint also means full stiffness. There is no closer analysis of influence of stiffners on resistance and stiffness. Of course, their use should increase both resistance and stiffness. On the other hand, Eurocode prefers cheaper structures, i.e. without stiffeners (less materials and labor).

Photo: encrypted-tbn0.gstatic.com

Photo: en.wikiarquitectura.com

Shape and position of welds between hollow section and flange are very important in case of fatigue influence. Fatigue resistance coud be decresased nearly 2 times (from 71 MPa to 40 MPa) in case of wrong solution of welds.

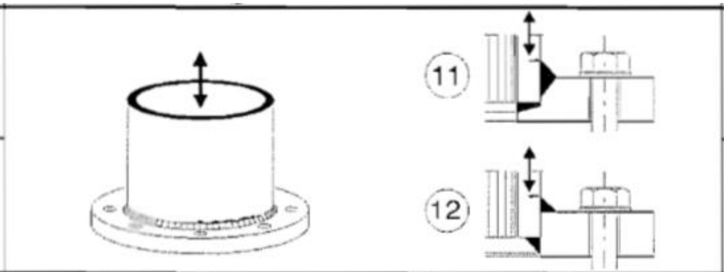
| | | | |
|---|---|---|---|
| 71 |  | 11) Tube socket joint with 80% full penetration butt welds. | 11) Weld toe ground. Δσ computed in tube. |
| 40 | | 12) Tube socket joint with fillet welds. | 12) Δσ computed in tube. |

Photo: EN 1993-1-9 tab. 8.5

Application of stiffeners is way to reduce risk of fatigue. Formulas for checking for FAT LS (EN 1993-1-9 (8.1), (8.2), (8.3)):

Photo: encrypted-tbn0.gstatic.com

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

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

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

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

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

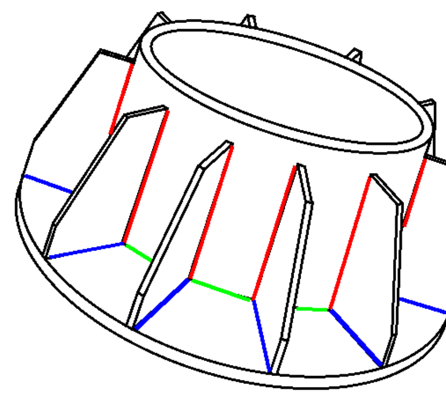
$\Delta\sigma_E$ , $\Delta\tau_E$ – amplitudes of stresses between two opposite most extreme loads (max tension – max compression);

$f_y$ – yield strength of steel;

$\Delta\sigma_C$ , $\Delta\tau_C$ – fatigue resistances;

Photo: Author

Photo: Author



Stiffeners → longer wleds → bigger areas of welds → smaller stresses (#17 / example 9) → smaller aplitude of stresses

Example of incorrect solution of splice joint in truss. Resistance of gusset plates is the same as resistance of chord, but stiffness is much lower than stiffness of chord.

Photo: construsoftbimawardscom

Photo: construsoftbimawardscom

This solution causes unforeseen hinges to appear in the truss and turns it into a mechanism. Joints must not only have adequate resistance but also stiffness.

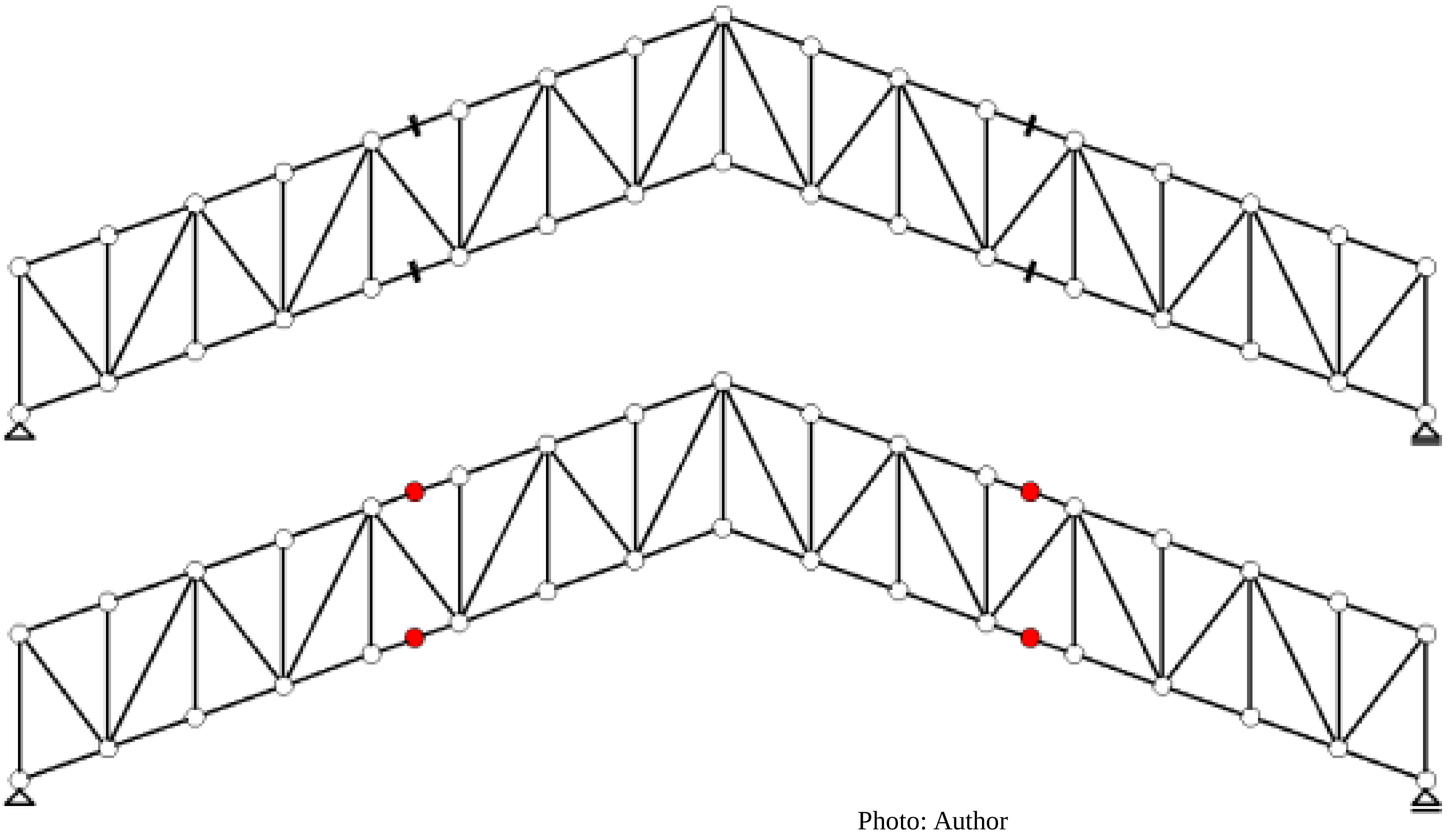


Photo: Author

# Cleats and gusset plates

Cleats and gusset plates are elements of lesser importance. They are often not checked computationally because load values are very small. Systemic solutions are used often for cold-formed purlins and wall girts.

These elements are important for:

- Support of purlins;
- Support of wall girts;
- Joints for tie-beams;
- Joints for bracings;
- Joints for hangers.

Photo: kingspan.com

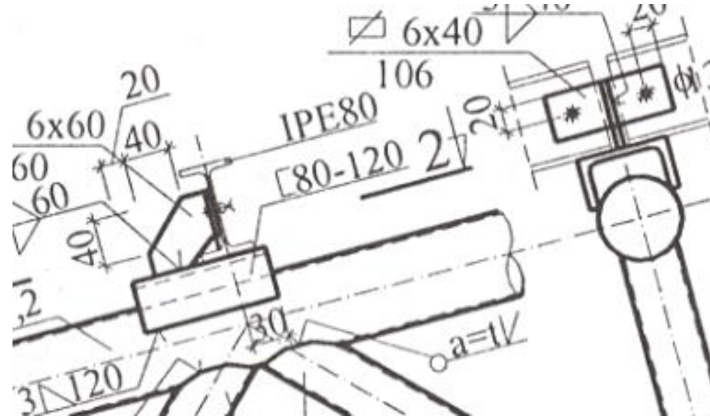
Photo: voestalpine.com

Photo: zed-purlins.co.uk

# Support for purlins

Photo: Author

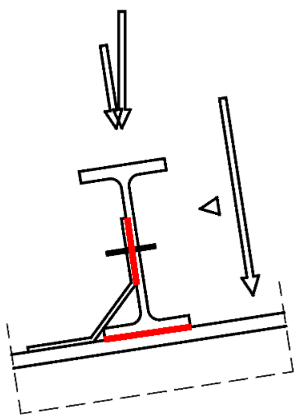
## Hot-rolled

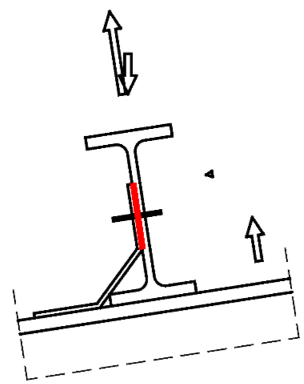



→ #8 / 39

Photo: M. Gwóźdź, M. Maślak, Przykłady projektowania wybranych stalowych konstrukcji prętowych, Politechnika Krakowska 2003

Snow + dead-weight + wind pressure:
by contact between bottom flange and girder; web and support plate





Wind suction bigger than dead-weight:
by contact between web and support plate; shear of bolt

Photo: Author

→ #6 / 68

Cold-formed purlins are fixed above top surface of girder, with a small gap. This avoids local deformation of purlins when pressed against girder. Cold-formed purlin is thin-walled section. Even small deformations could significantly change its cross-section and reduce bending resistance.

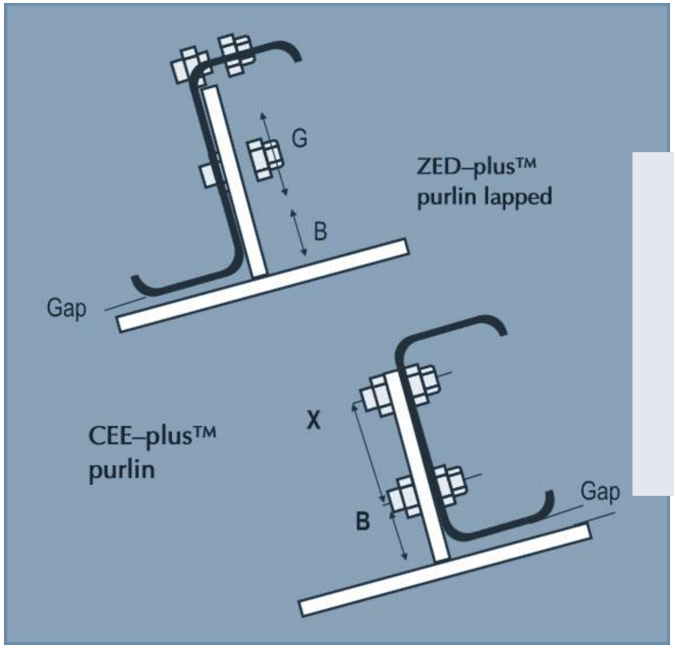



Photo: gscpl.net

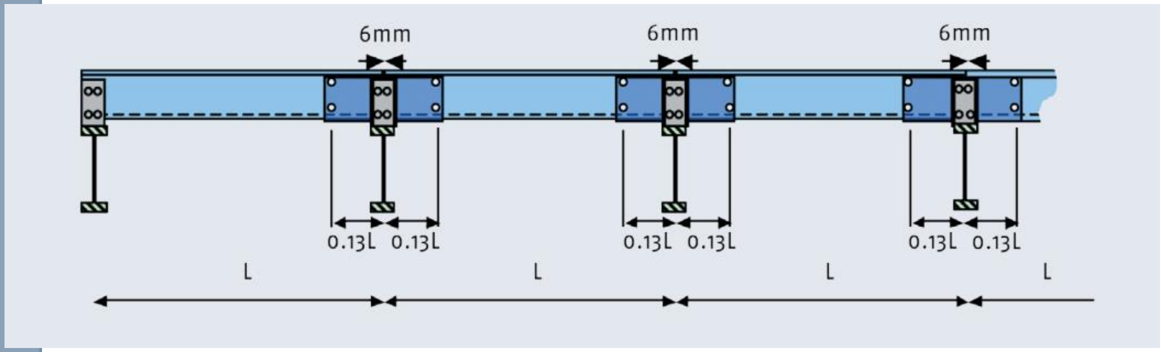



Photo: ruuki.com

Photo: steel.ie

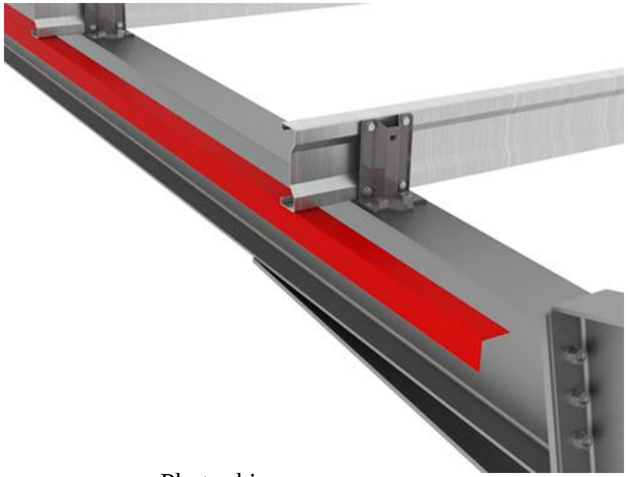


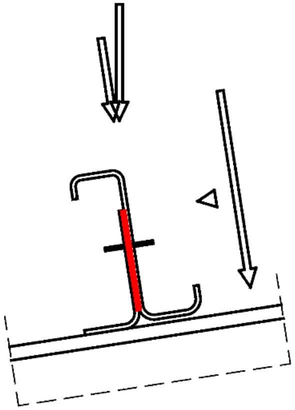
Photo: kingspan.com

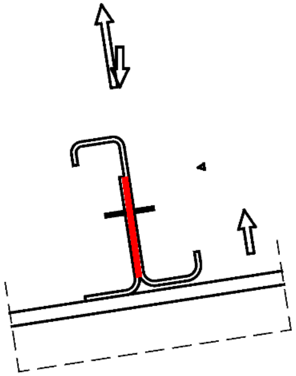
Support for purlins

Cold-formed

→ #8 / 41

Snow + dead-weight + wind pressure:
by contact between bottom flange and girder; web and support plate





Wind suction bigger than dead-weight:
by contact between web and support plate; shear of bolt

Photo: Author

Purlin cleat can be made as short part of L-section, system solution or welded plates.

Photo: Author

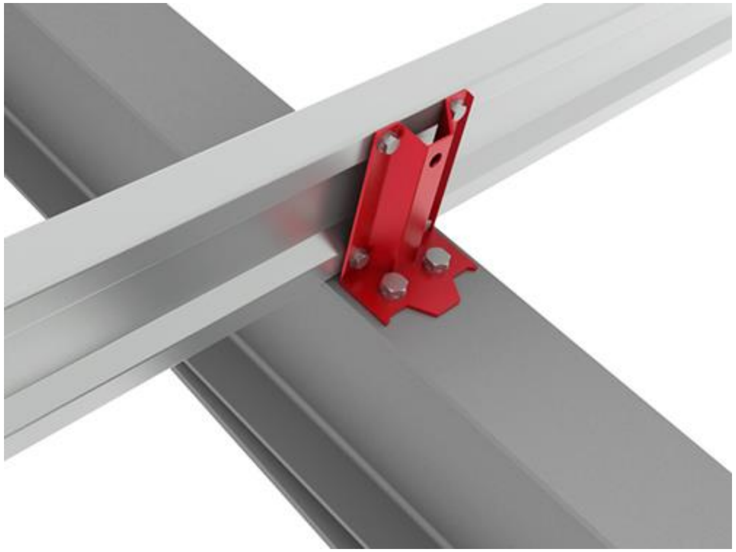


Photo: steel.ie


Photo: builderbill-diy-help.com

Photo: indiamart.com

Support for wall girts depends on type of girt's crosss-section: cold-formed, hot rolled C, hot rolled RHS.

PhotoAuthor

Photo: devisdepro.com

PhotoAuthor

Wall girts cleat can be made according to similar ways: as short part of L-section, system solution or gusset plate. RHS and cold-formed cross-sections can be fixed by self-tapping bolts.

Photo: magnabuild.co.uk

Photo: indooroutdoors.co.uk

Photo: indiamart.com

Photo: kobexstal.pl

Photo: tues.ru

Rigid bracings and rigid tie-beams are, mostly, joined by gusset plates.

Photo: exp.ncree.org

Photo: atlastube.com

Photo: designboom.com

Non -rigid bracings and hangers are supported by gusset plates or systemic sloution elements.

Photo: Author

Photo: panelsandprofiles.co.uk

Photo: zed-purlins.co.uk

Each of these joints are taken into consideration as hinge joints.

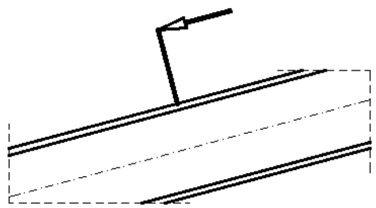


Photo: Author

Photo: Author

Supports for purlins are bending in direction parallell to roof surface, but value of this force is very small. Checking of resistance is non obligatory for such smal value of action.


Photo: builderbill-diy-help.com

In longitudinal direction, support is connected with purlin by two (nearly-hinge) or sometimes one bolt (ideal hinge connection). According to information #t / 84, calculation of bolted joint must be made for big value of wind suction only.

Calculation of supports for wall girts could be more complicated, than for purlins. Vertical force comes from dead-weight of housing and girts; it could be significant value. Bending moments applicated to gusset plates or short cantilevers should be checked. The same, resistance of bolted joint, calculated as category A.

PhotoAuthor

Photo: devisdepro.com

PhotoAuthor

In case of bracings and hangers, force is applicated axially to gusset plate. This is classical bolted joint category A: sheaf of bolt, bearing resistance, block tearing and netto area for plate.

Photo: designboom.com

Photo: exp.ncree.org

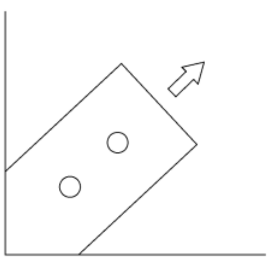


Photo: Author

Welded joint are not recommended on construction site, but sometimes applicated. Calculation is presented on Lec #17, example #5.

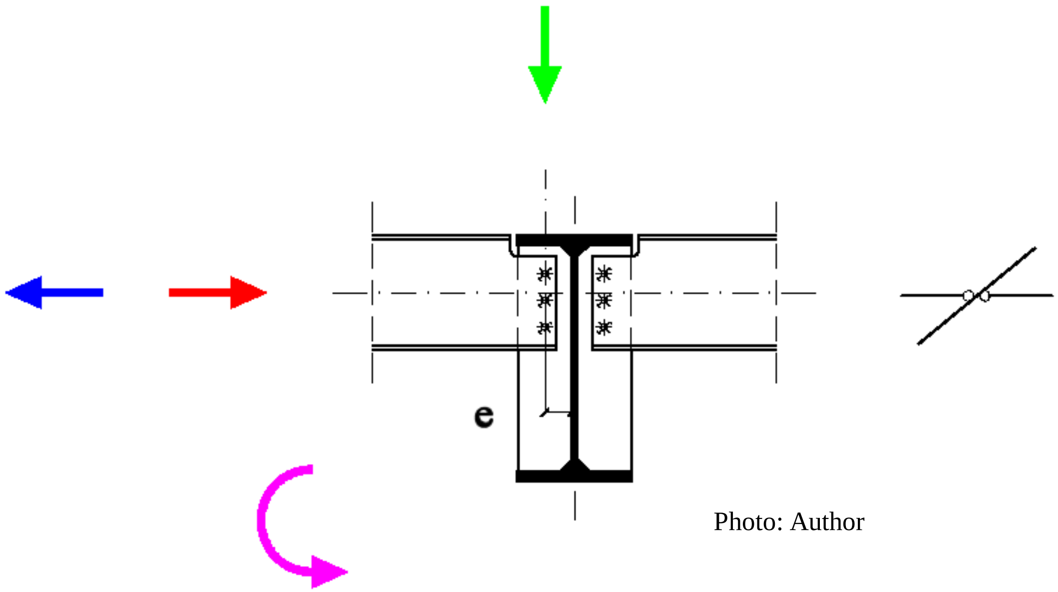



Photo: Author

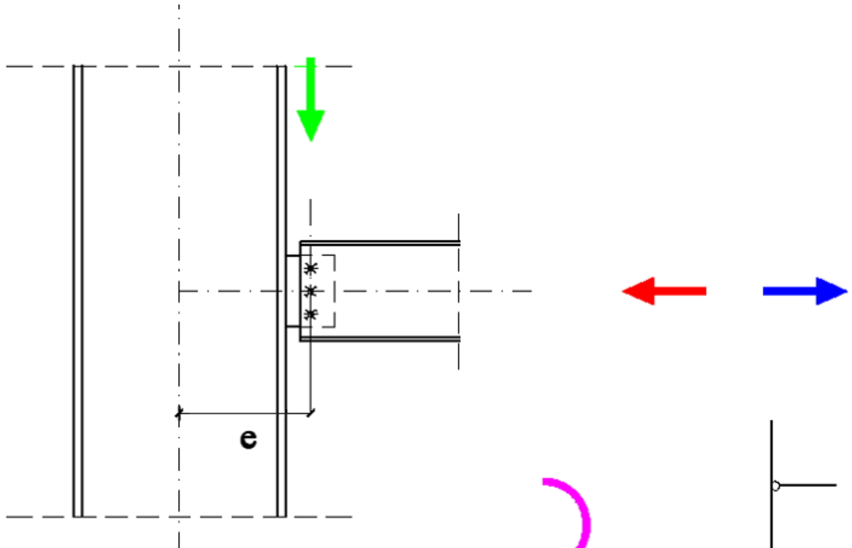
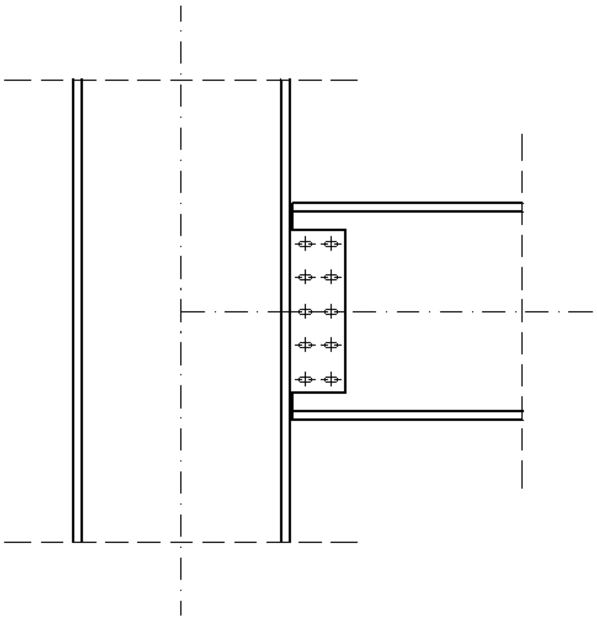



Photo: Author

There are eccentricities between theoretical (axis of column, axis of beam) and real (axis of bolts) supports of elements (secondary beam, wall girts). Impact of these eccentricities is not cleary defined. Part of literature posiotions rcecommended ignore ecentricities, part recommneded taking into consideration secondary bending moment from eccentricities and vertical force.

Additional recommendation is application of slotted holes in case of hinge support of secondary beams or wall girts.



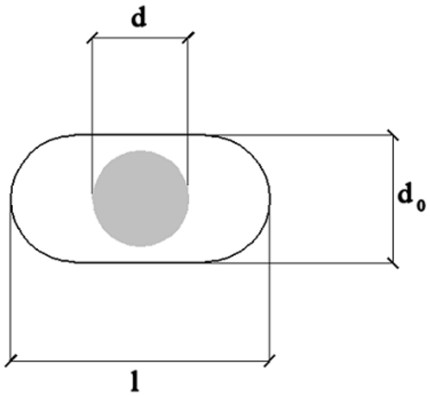



Photo: Author

Photo: tekla-detailed-structural-fabrication.com

# Examination issues

Types of static models of truss

Modes of failure for truss nodes

The role and placement of horizontal and vertical stiffeners

Calculation of vertical stiffeners

Whether-in-plane - obciążenie przęsłowe w prętach kratownicy
Out-of-plane - obciążenie prostopadłe do płaszczyzny kratownicy
Chord size failure - zniszczenie przystykowe pasa
Chord size wall / web failure - zniszczenie boków / środnika pasa
Chord shear failure - ścięcie pasa
Punching shear - przebicie
Brace failure - zniszczenie elementu skratowania
Local buckling - wyboczenie miejscowe
Gussed plate - blacha węzłowa
Splice joint - styk montażowy
Stiffener - żebro
Contact strength - docisk
Cleat – nakładka kątowa

Thank you for attention



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