European Commission

Eurocodes

Background and Applications

Design of **Steel Buildings**

with worked examples

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Brussels, Belgium

Bolts, welds, column base

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Organised and supported by

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

Joint Research Centre

Motivation

To present

- Content/principles
- Selected particularities
- Questions

To offer

- Worked examples

for design according to **EN1993-1-8: 2005** of

Bolts

Welds

Column bases

Scope of the lecture

Bolts
- General
- Design resistance of individual fasteners
  - Non-preloading bolts
  - Slotted holes
- Design for block tearing
- Worked example
- Summary

Welds
- General
- Fillet weld
  - Design model
  - Design example
  - Welding to flexible plate
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Column bases
- Basis of design
- Components
  - Base plate and concrete in compression
  - Base plate in bending and bolt in tension
  - Anchor bolt in shear
- Assembly
  - Resistance
  - Stiffness
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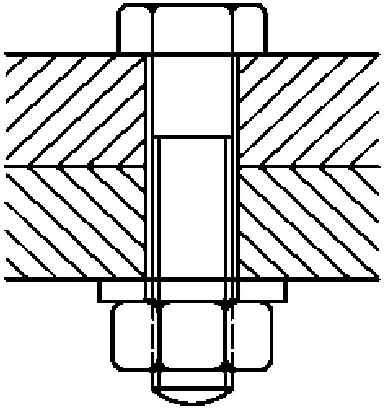

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Connections made with bolts, rivets or pins in EN1993-1-8: 2005

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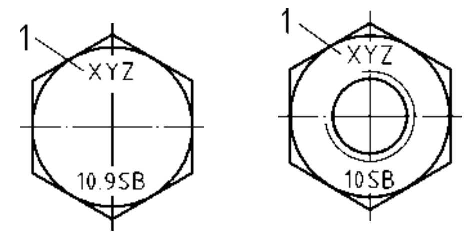
Bolt standards



EN 15048

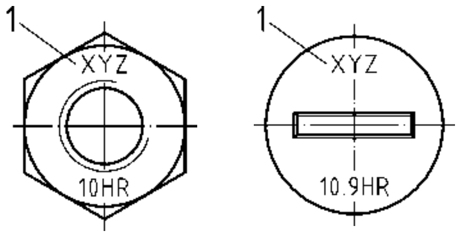
Non-preloaded structural bolting assemblies

- Part 1: General requirements
- Part 2: Suitability test

EN 14399

High-strength structural bolting for preloading

High-strength structural bolting for preloading



EN 14399

Part 1: General requirements

Part 2: Suitability test for preloading

Part 3: System HR — Hexagon bolt and nut assemblies

Part 4: System HV — Hexagon bolt and nut assemblies

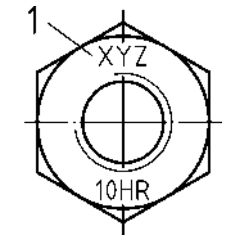
Part 5: Plain washers

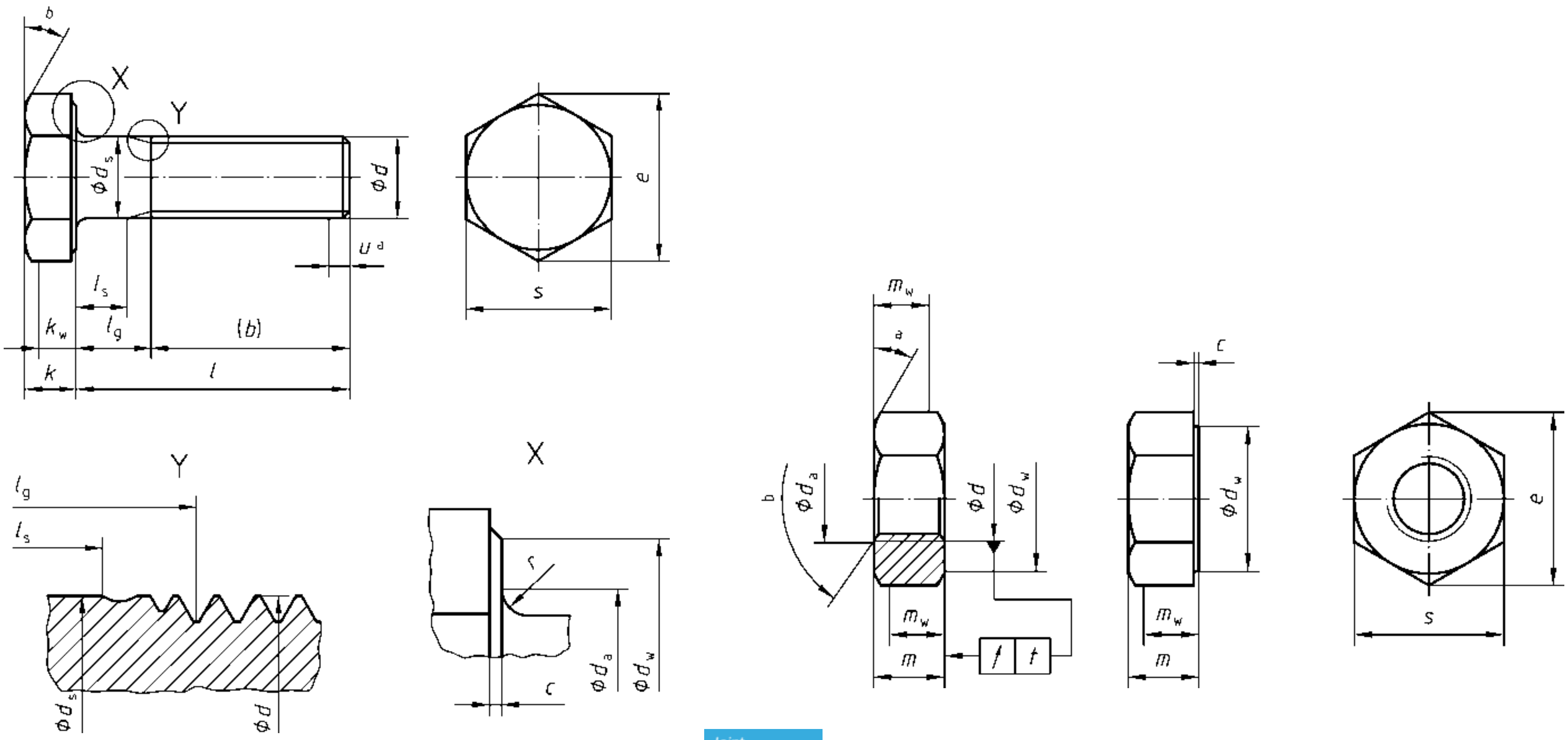
Part 6: Plain chamfered washers

Part 7: System HR – Countersunk head bolt and nut assemblies

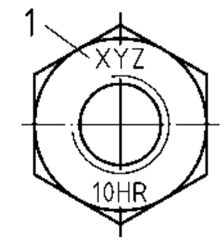
Part 8 : System HV – Hexagon fit bolt and nut assemblies

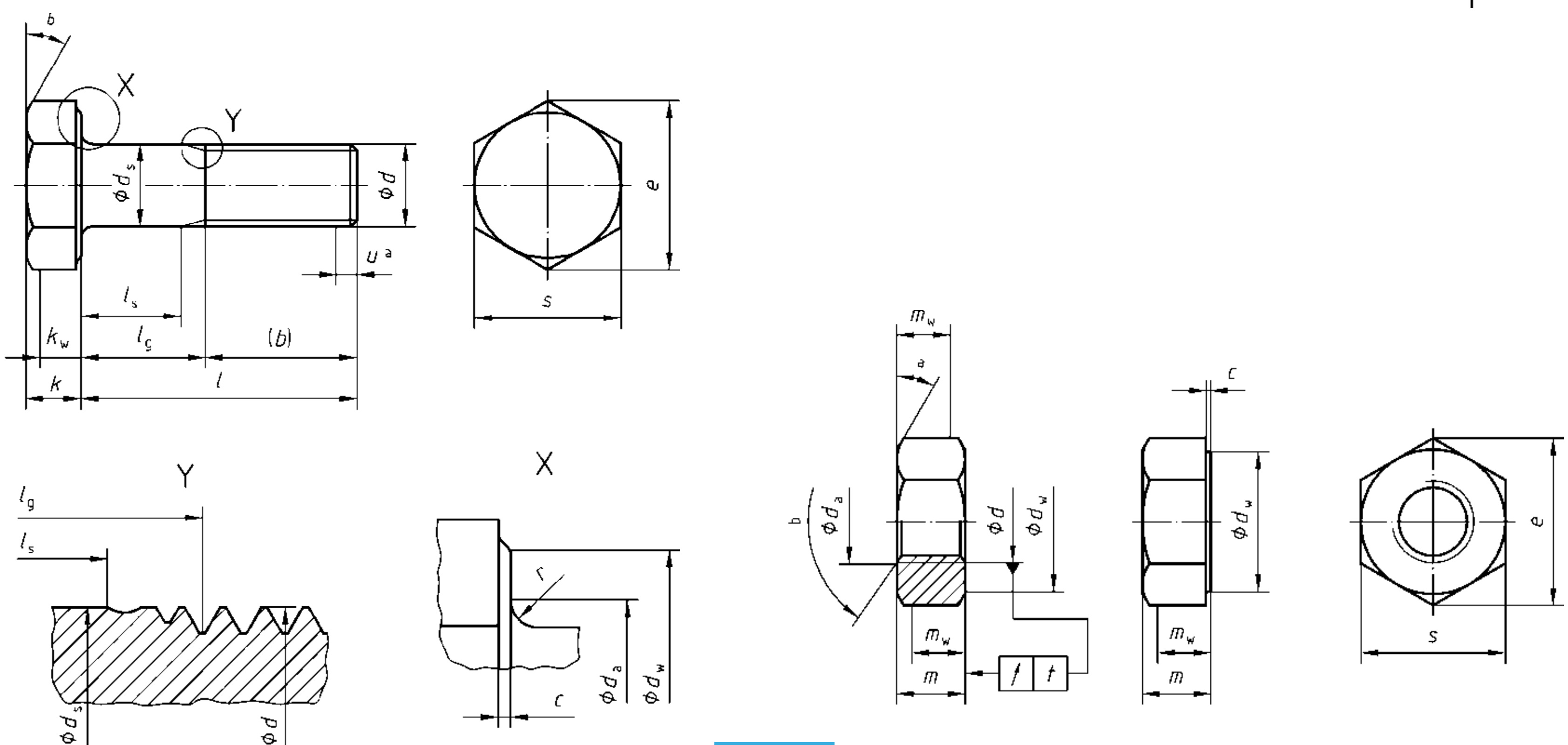
Part 3: System HR —
Hexagon bolt and nut assemblies





Part 4: System HV — Hexagon bolt and nut assemblies





Mechanical properties

EN ISO 898-1, Mechanical properties of fasteners made of carbon steel and alloy steel — Part 1: Bolts, screws and studs **(ISO 898-1:1999)**

EN 20898-2, Mechanical properties of fasteners — Part 2: Nuts with specified proof load values — Coarse thread **(ISO 898-2:1992)**

EN ISO 3506-1, Mechanical properties of corrosion-resistant stainless-steel fasteners — Part 1: Bolts, screws and studs **(ISO 3506-1:1997)**

EN ISO 3506-2, Mechanical properties of corrosion-resistant stainless-steel fasteners — Part 2: Nut **(ISO 3506-2:1997)**

Material

Bolts made of carbon steel and alloy steel:
4.6, 4.8, 5.6, 5.8, 6.8, 8.8, 10.9

Nuts made of carbon steel and alloy steel:
4, 5, 6, 8, 10, 12

Bolts made of austenitic stainless steel:
50, 70, 80

Nuts made of austenitic stainless steel:
50, 70, 80

Washers (if appropriate) according to
hardness class HV 100 or HV 200

EN 1993-1-8
Material Properties

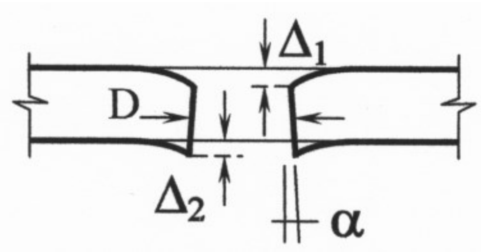
Nominal values for bolts

f_{yb} is yield strength

f_{ub} is ultimate tensile strength

Bolt class	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1000

Holes



- Normal

  +1 mm for M 12

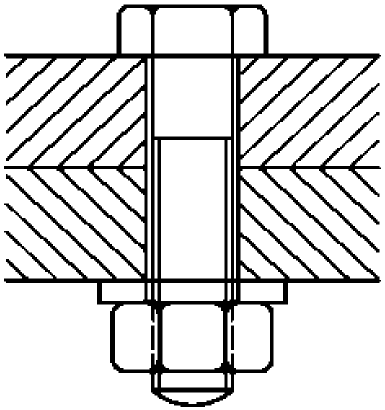
  +2 mm for M 16 up M 24

  +3 mm for M 27 and bigger
- Oversized with 3 mm (M12) up 8 mm (M27)
- Slotted (elongated)
- Close fitting – flushed bolts

  for bolt M20 must be the clearance Δd < 0,3 mm

EN 1090-2 Execution of steel structures and aluminium structures – Part 2: Technical requirements for steel structures

Scope of the lecture



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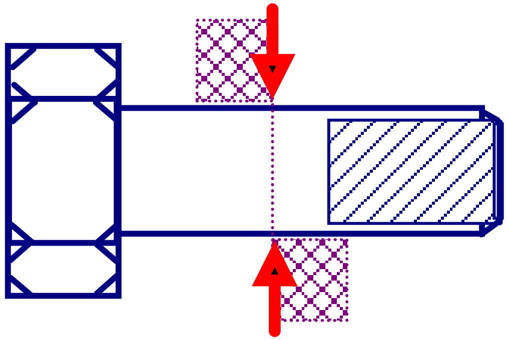
Resistance in shear in one shear plane



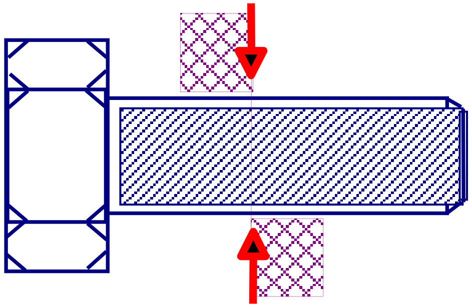
$$F_{v,Rd} = \frac{\alpha_v \; A \, f_{ub}}{\gamma_{M2}}$$

where the shear plane
passes through the **unthreaded portion of the bolt**

$\alpha_v = 0,6$

A is the gross cross section of the bolt
f_{ub} is ultimate tensile strength for bolt
γ_{M2} s partial safety factors for resistance of bolts

Resistance in shear in one shear plane



$$F_{v,Rd} = \frac{\alpha_v \, A \, f_{ub}}{\gamma_{M2}}$$

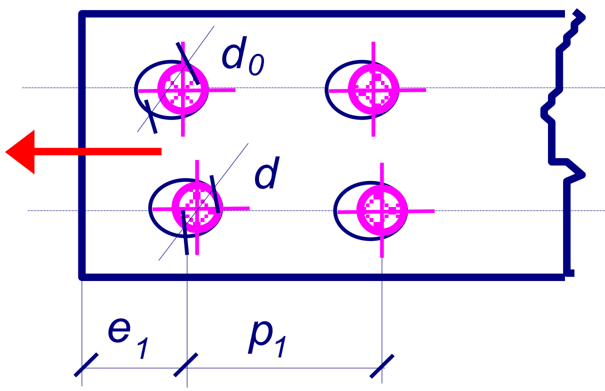
where the shear plane
passes through the **threaded portion** of the bolt

- for classes 4.6, 5.6 and 8.8:
$\alpha_v = 0{,}6$

- for classes 4.8, 5.8, 6.8 and 10.9
$\alpha_v = 0{,}5$

A	is the tensile stress area of the bolt A_s
f_{ub}	is ultimate tensile strength for bolt
γ_{M2}	s partial safety factors for resistance of bolts

Resistance in bearing

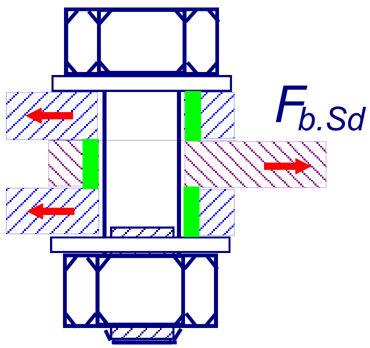


$$F_{b,Rd} = \frac{k_1 \, a_b \, d \, t \, f_u}{\gamma_{M2}}$$

where α_b is the smallest of α_d, $\frac{f_{ub}}{f_u}$ or 1,0

In the direction of load transfer

- for end bolts: $\alpha_d = \frac{e_1}{3\, d_0}$, for inner bolts $\alpha_d = \frac{p_1}{3\, d_0} - \frac{1}{4}$



Perpendicular to the direction of load transfer

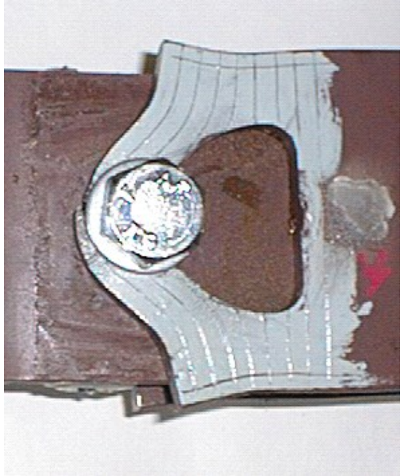
- for edge bolts k_1 is the smallest of $2{,}8 \frac{e_2}{d_0} - 1{,}7$ or 2,5

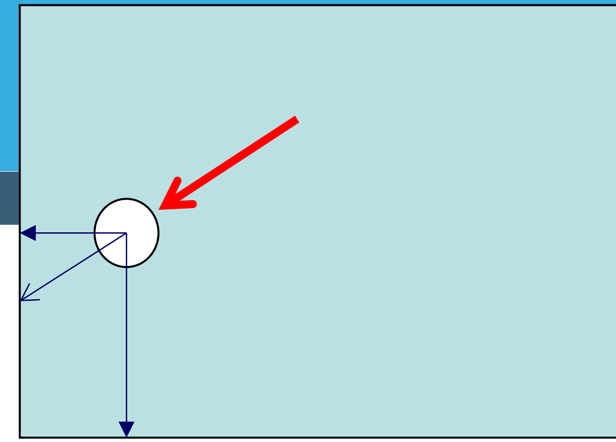
- for inner bolts k_1 is the smallest of $1{,}4 \frac{e_2}{d_0} - 1{,}7$ or 2,5

Bearing of Plate and Bolt

Inner bolt ←

Outer bolt ←



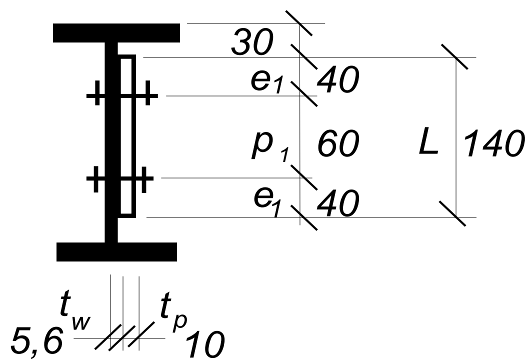


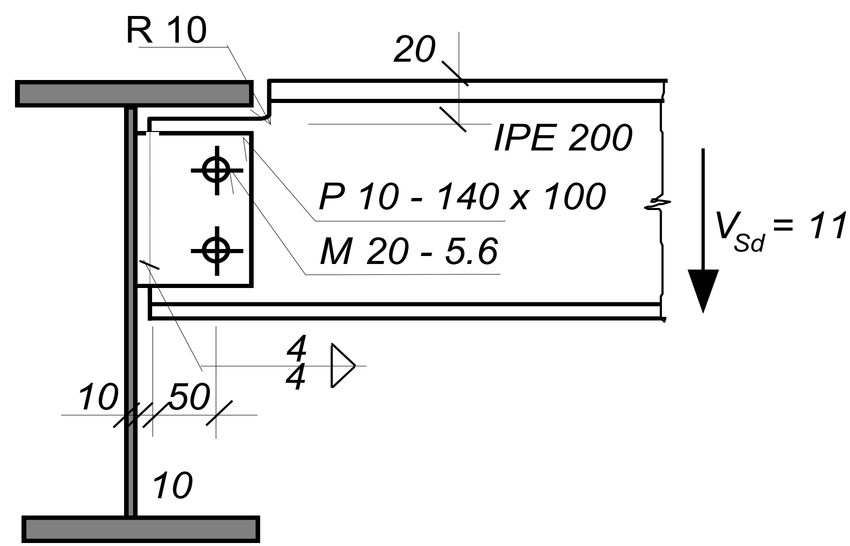
Resistance in Bearing

Load on a bolt not parallel to the edge

the bearing resistance may be verified separately

for the bolt load components parallel and normal to the edge





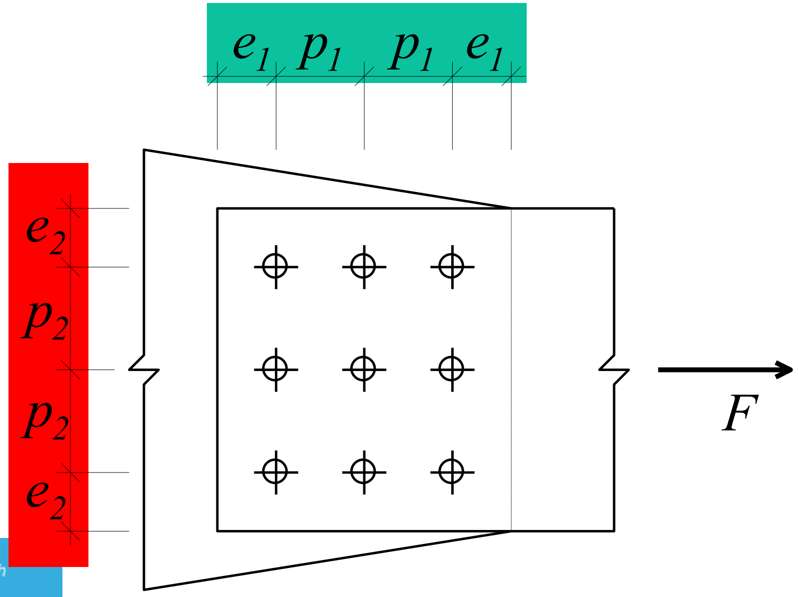
in oversized holes reduce bearing by 0,8

Influence of distances to force

$$F_{b,Rd} = \frac{k_1 \alpha_b \, d \, t \, f_u}{\gamma_{M2}}$$

Prallel to acting force

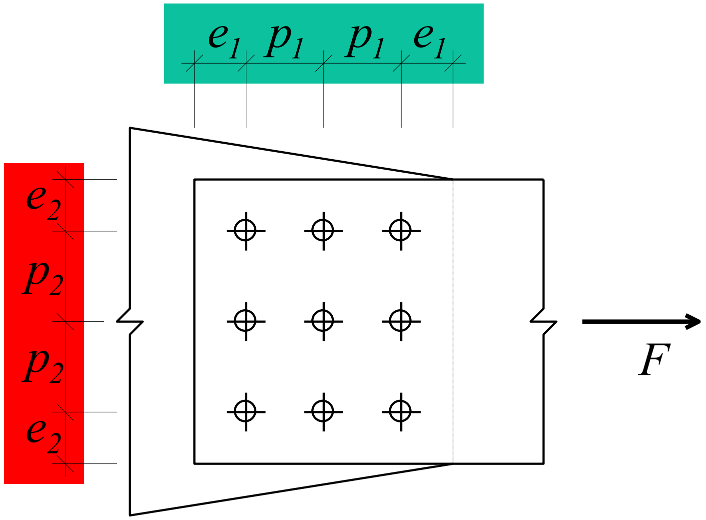
$$\alpha_b = \min \begin{Bmatrix} \dfrac{e_1}{3\,d_0} \\ \dfrac{p_1}{3\,d_0} - 0{,}25 \\ \dfrac{f_{ub}}{f_u} \\ 1 \end{Bmatrix}$$



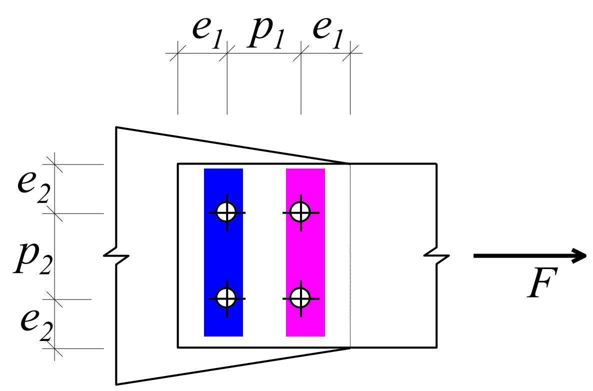
Perpendicular to acting force

$$F_{b,Rd} = \frac{k_1 \alpha_b d\ t\ f_u}{\gamma_{M2}}$$

$$k_1 = min \begin{Bmatrix} 2,8 \dfrac{e_2}{d_0} - 1,7 \\ 1,4 \dfrac{p_2}{d_0} - 1,7 \\ 2,5 \end{Bmatrix}$$



Influence of distances



Nomimal transversal distances:

$e_1 = 1,2\ d_0$

$p_1 = 2,2\ d_0$

$e_2 = 1,2\ d_0$

$p_2 = 2,4\ d_0$

$$k_1 = \min\left\{\begin{matrix} 2,8\frac{e_2}{d_0} - 1,7 \\ 1,4\frac{p_2}{d_0} - 1,7 \\ 2,5 \end{matrix}\right\} = \min\left\{\begin{matrix} 2,8\frac{1,2\ d_0}{d_0} - 1,7 \\ 1,4\frac{2,4\ d_0}{d_0} - 1,7 \\ 2,5 \end{matrix}\right\} = \min\left\{\begin{matrix} 1,66 \\ 1,66 \\ 2,5 \end{matrix}\right\} = 1,66$$

End bolts

$$\alpha_b = \frac{e_1}{3\ d_0} = \frac{1,2\ d_0}{3\ d_0} = 0,400$$

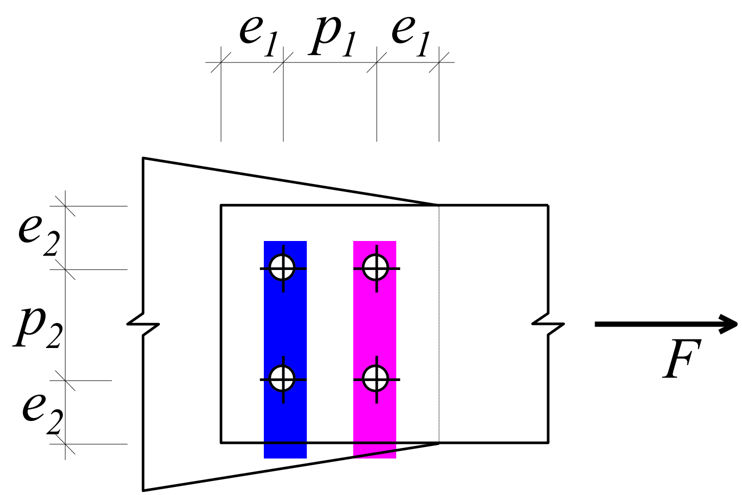
$$F_{b,Rd} = \frac{k_1\ a_b\ d\ t\ f_u}{\gamma_{M2}} = \frac{1,66 \cdot 0,400\ d\ t\ f_u}{\gamma_{M2}} = 0,664\frac{d\ t\ f_u}{\gamma_{M2}}$$

Internal bolts

$$\alpha_b = \frac{p_1}{3\ d_0} - 0,25 = \frac{2,2\ d_0}{3\ d_0} - 0,25 = 0,483$$

$$F_{b,Rd} = \frac{k_1\ a_b\ d\ t\ f_u}{\gamma_{M2}} = \frac{1,66 \cdot 0,483\ d\ t\ f_u}{\gamma_{M2}} = 0,802\frac{d\ t\ f_u}{\gamma_{M2}}$$

Sum



$$e_1 = 1{,}2\ d_0$$
$$p_1 = 3{,}0\ d_0$$
$$e_2 = 1{,}5\ d_0$$
$$p_2 = 3{,}0\ d_0$$

Sum of resistances

$$F_{\mathrm{b,Rd}} = 2 \cdot 1{,}875 \frac{d\,t\,f_{\mathrm{u}}}{\gamma_{\mathrm{M2}}} + 2 \cdot 1{,}00 \frac{d\,t\,f_{\mathrm{u}}}{\gamma_{\mathrm{M2}}} = 5{,}75 \frac{d\,t\,f_{\mathrm{u}}}{\gamma_{\mathrm{M2}}}$$

Minimal resistance

$$F_{\mathrm{b,Rd}} = 4 \cdot 1{,}0 \frac{d\,t\,f_{\mathrm{u}}}{\gamma_{\mathrm{M2}}} = 4{,}0 \frac{d\,t\,f_{\mathrm{u}}}{\gamma_{\mathrm{M2}}}$$

Pitch distances

Min

$p_1 = 2,2\ d_0$

$p_2 = 2,4\ d_0$

optimum e_1 from $1,2\ d_0$ to $1,45\ d_0$

$e_2 = 1,2\ d_0$

$$F_{b,Rd} = \frac{k_1\ a_b\ d\ t\ f_u}{\gamma_{M2}} = \frac{1,66 \cdot 0,483\ d\ t\ f_u}{\gamma_{M2}} = 0,802\ \frac{d\ t\ f_u}{\gamma_{M2}}$$

Large

$p_1 = 3,75\ d_0$

$p_2 = 3,0\ d_0$

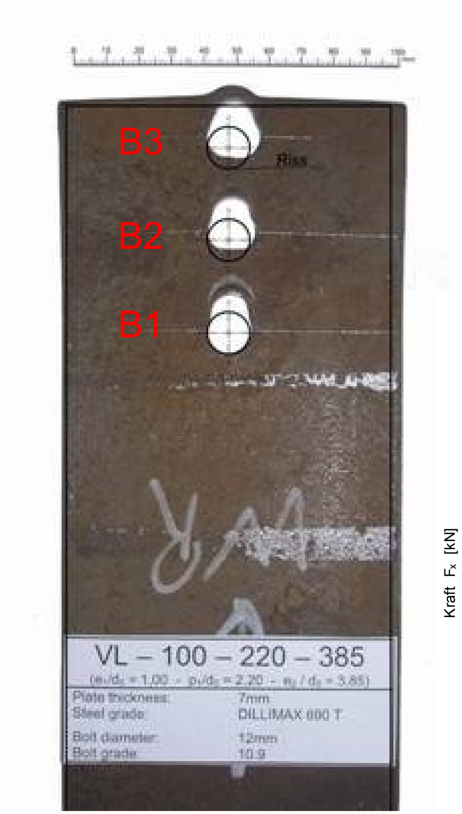
$e_1 = 3,0\ d_0$

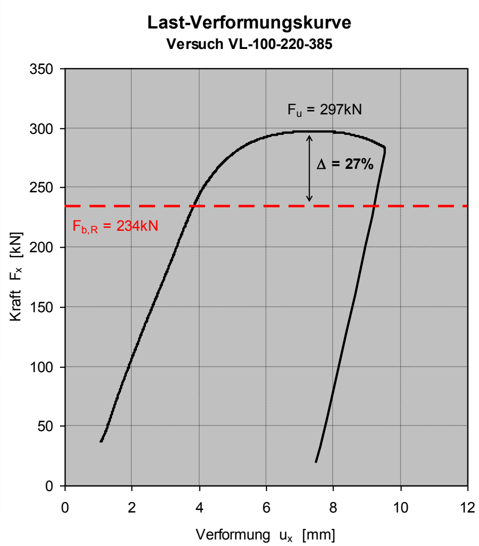
$e_2 = 1,5\ d_0$

$$F_{b,Rd} = \frac{k_1\ a_b\ d\ t\ f_u}{\gamma_{M2}} = \frac{2,5 \cdot 1,0\ d\ t\ f_u}{\gamma_{M2}} = 2,5\ \frac{d\ t\ f_u}{\gamma_{M2}}$$

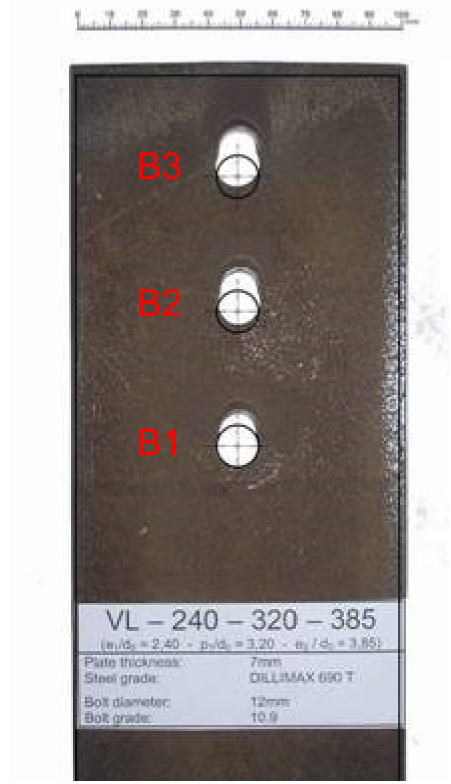
Steel S690

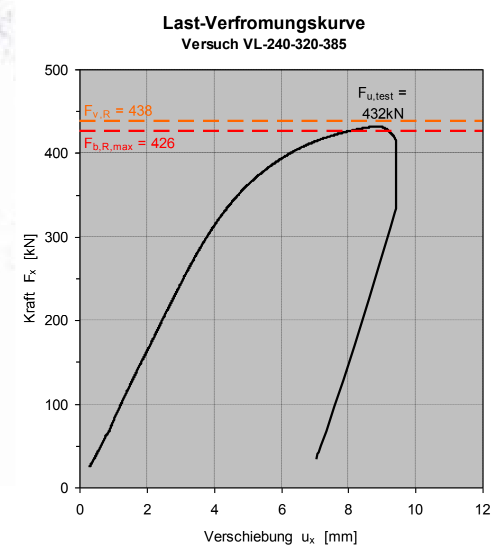
Experiment bolts M12 10.9 minimal and maximal pitch





$e_1 = 1{,}0\ d_0 <$ min. $e_1 = 1{,}2\ d_0$



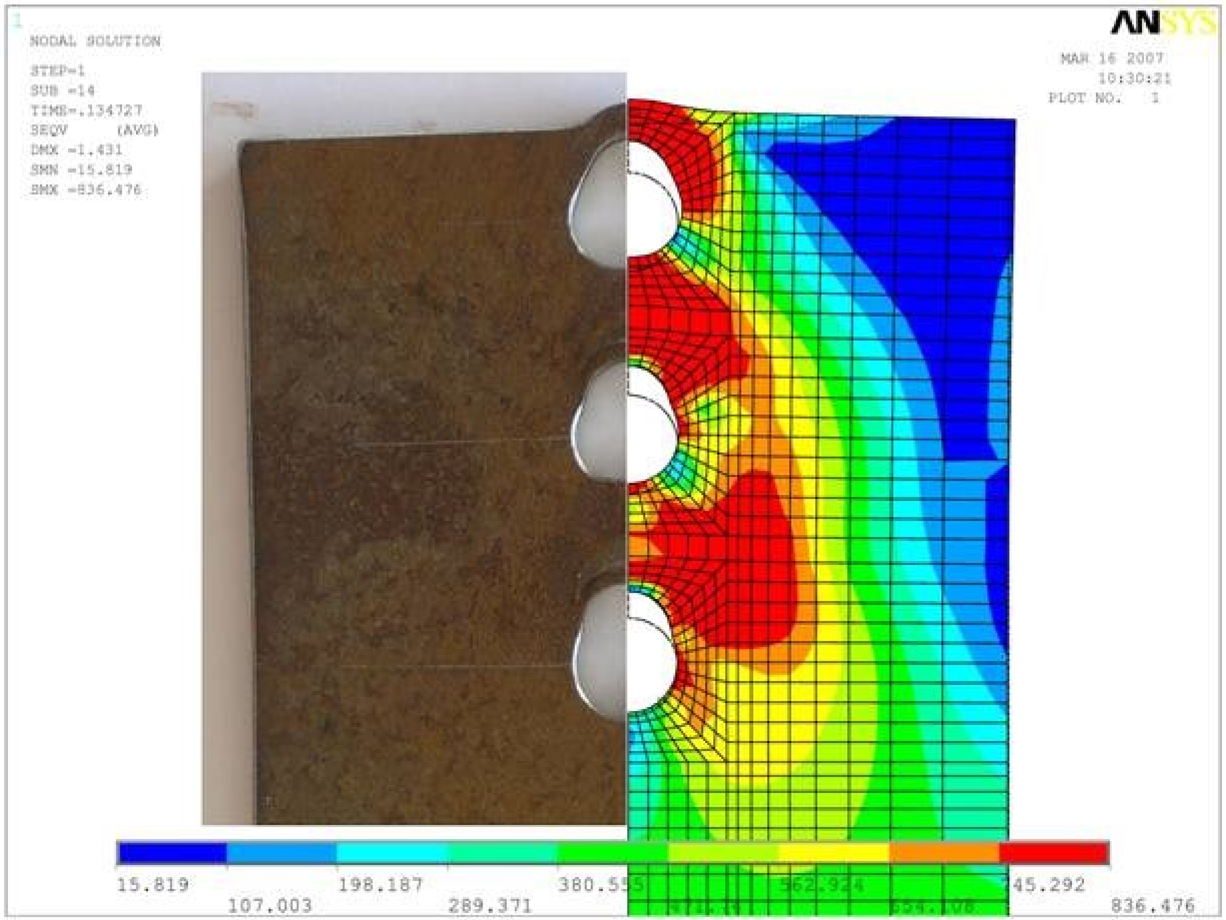


$e_1 = 2{,}4 d_0$ and $p_1 = 3{,}2\ d_0$

Comparison to EN

e_1 and p_1 min.	F_u/F_{EN3} **= 1,27**
e_1 and p_1 max.	$F_u/F_{EN3} \cong$ **1,02**

University of Dortmund, Institute of Steel Structures



Comparison of simulation VL-100-220-385

Numerical model

- **Good agreement**
- **Diffference D=5,5%**

University of Dortmund,
Institute of Steel Structures

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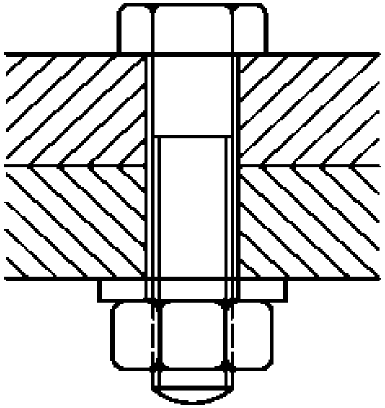

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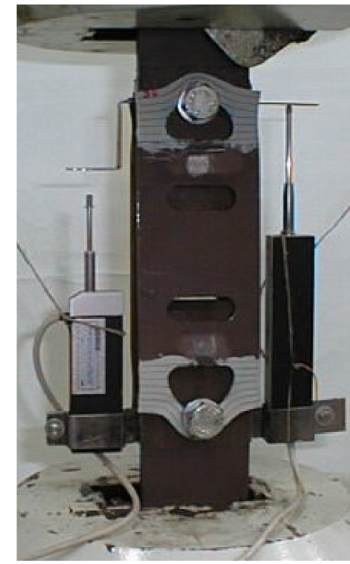
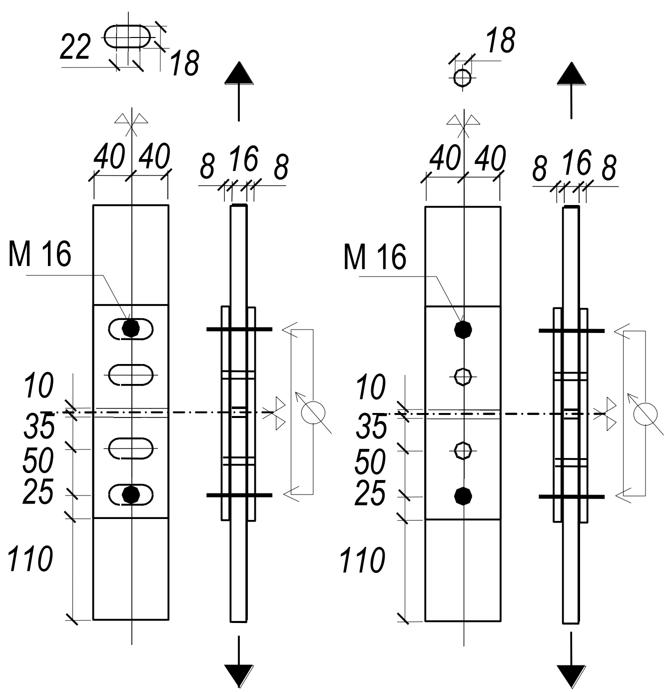
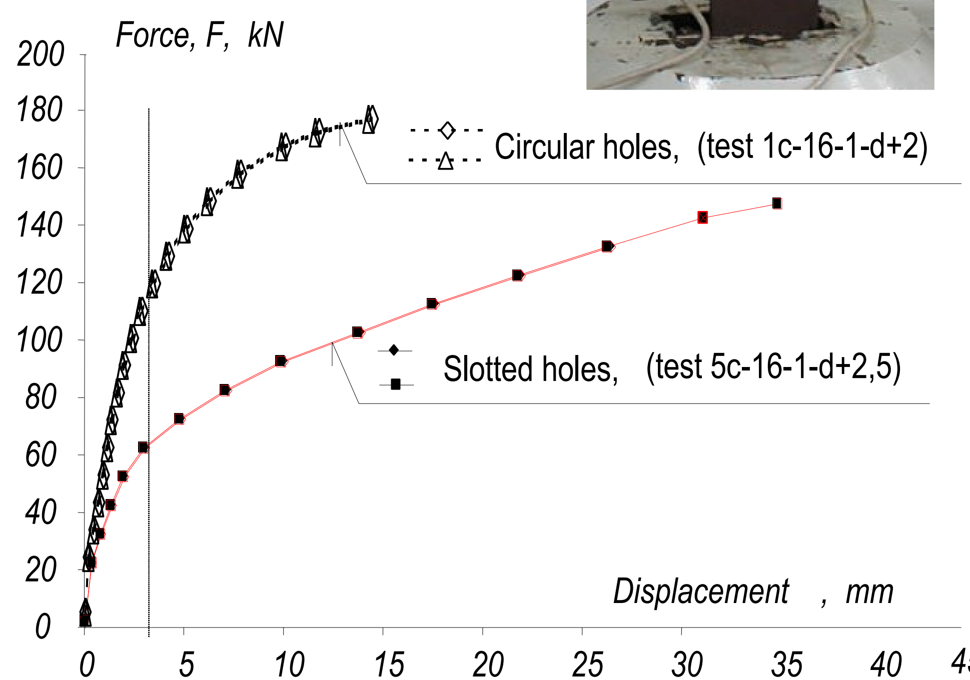
Bearing resistance in slotted holes

Loaded perpendicular to the long direction of the slot

60% of resistance in circular holes







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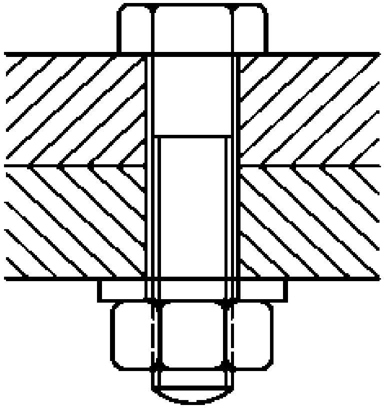

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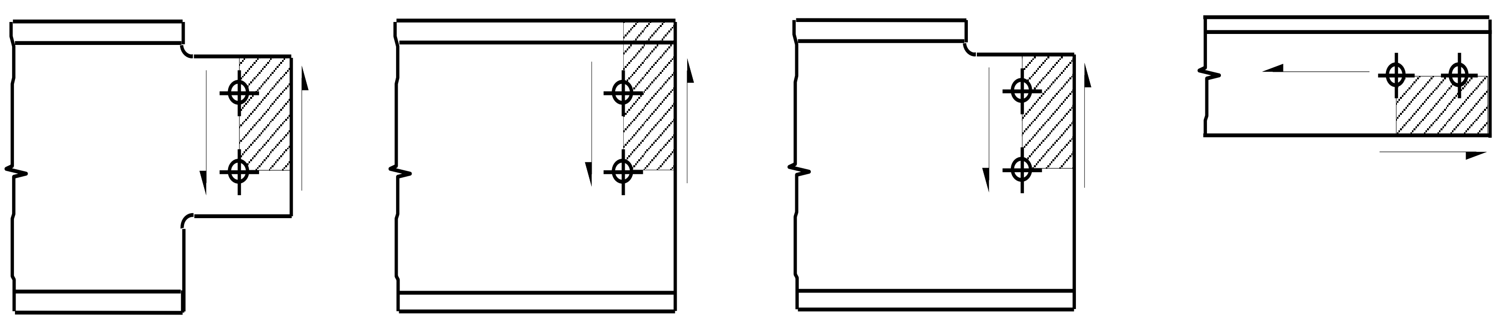
Block tearing

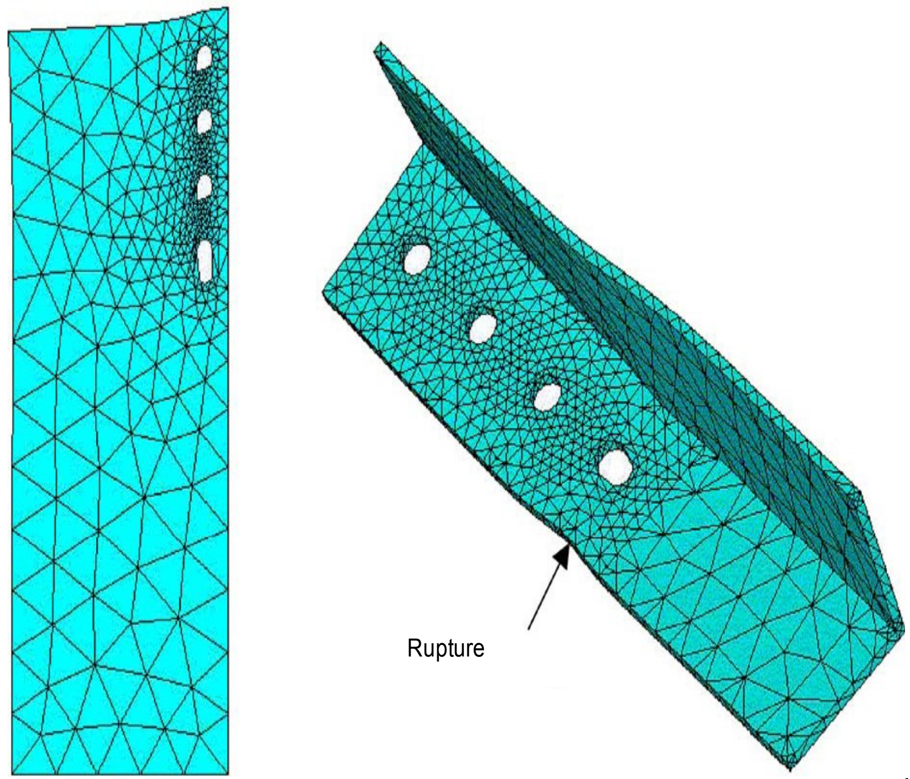
Failure in shear at the row of bolts
along the shear face of the hole group
accompanied by
Tensile rupture
along the line of bolt holes
on the tension face of the bolt group



Test

Orbison J.G., Wagner M. E., Fritz W.P. 1999, Tension plane behavior in single-row bolted connections subject to block shear, Journal of Constructional Steel Research, 49, 225 – 239

FE model



Topkaya C., 2004, A finite element parametric study on block shear failure of steel tension members, Journal of Constructional Steel Research, 60, 1615 – 1635, ISSN 0143-974X.

Design model

Symmetric bolt group subject to concentric loading

$$V_{eff,1,Rd} = f_u A_{nt} / \gamma_{M2} + (1/\sqrt{3}) f_y A_{nv} / \gamma_{M0}$$

A_{nt} net area subjected to tension

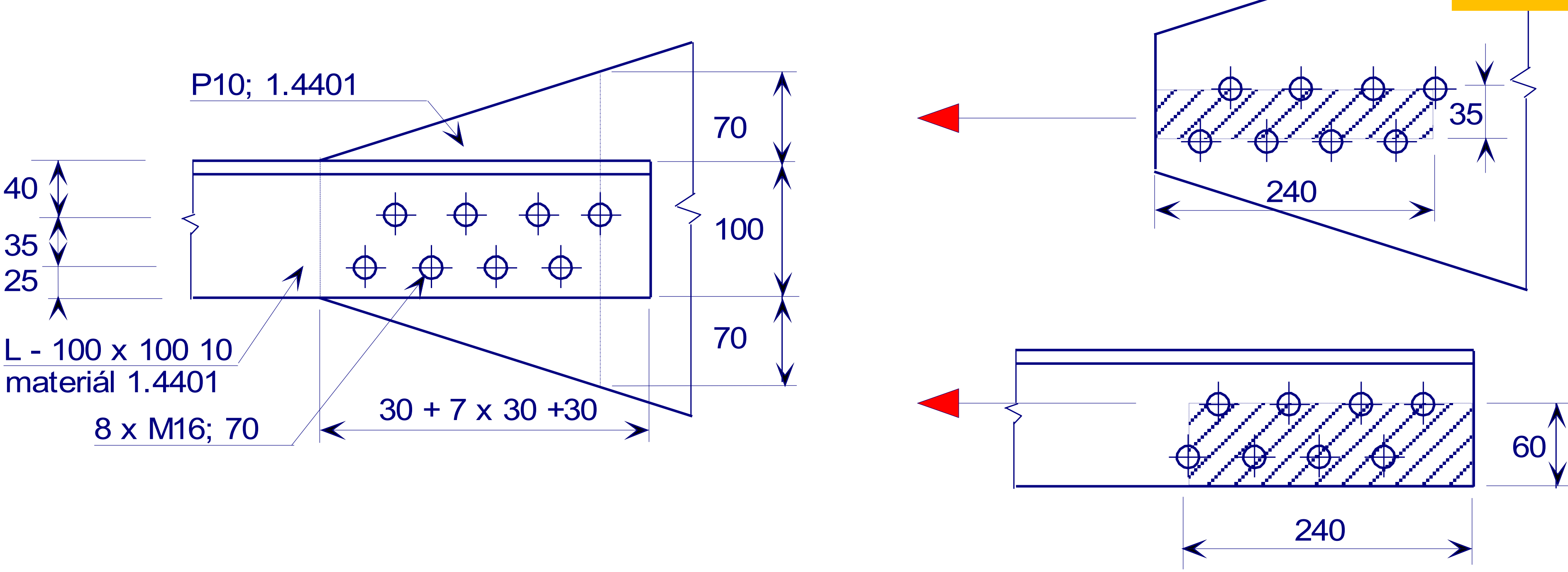
A_{nv} net area subjected to shear

Eccentric loading

$$V_{eff,2,Rd} = 0{,}5 f_u A_{nt} / \gamma_{M2} + (1/\sqrt{3}) f_y A_{nv} / \gamma_{M0}$$

Worked Example – **Angle connected by one leg**

WE



In plate (staggered rows)

$$V_{eff,1,Rd} = \frac{f_u\, A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}}\; f_y\, \frac{A_{nv}}{\gamma_{M0}}$$

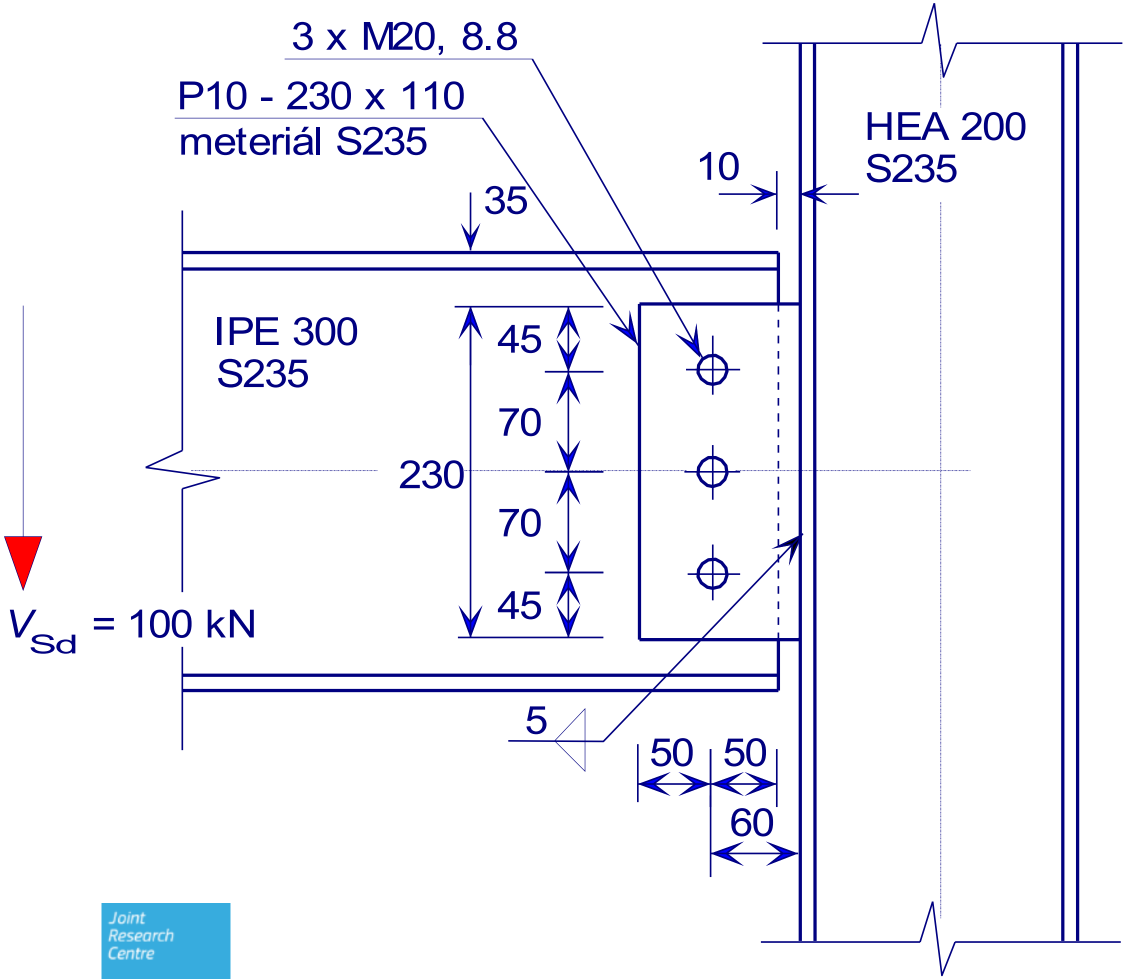
$$= \frac{0{,}5 \cdot 530 \cdot (35 - 2 \cdot 9) \cdot 10}{1{,}25 \cdot 10^3} + \frac{1}{\sqrt{3}} \cdot 220 \cdot \frac{(2 \cdot 240 - 7 \cdot 18 - 1 \cdot 9) \cdot 10}{1{,}0 \cdot 10^3} = 36 + 362 = 397\,\text{kN}$$

In angle (staggered rows)

$$V_{eff,2,Rd} = \frac{0{,}5\, f_{u,p}\, A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}}\; f_{y,p}\, \frac{A_{nv}}{\gamma_{M0}}$$

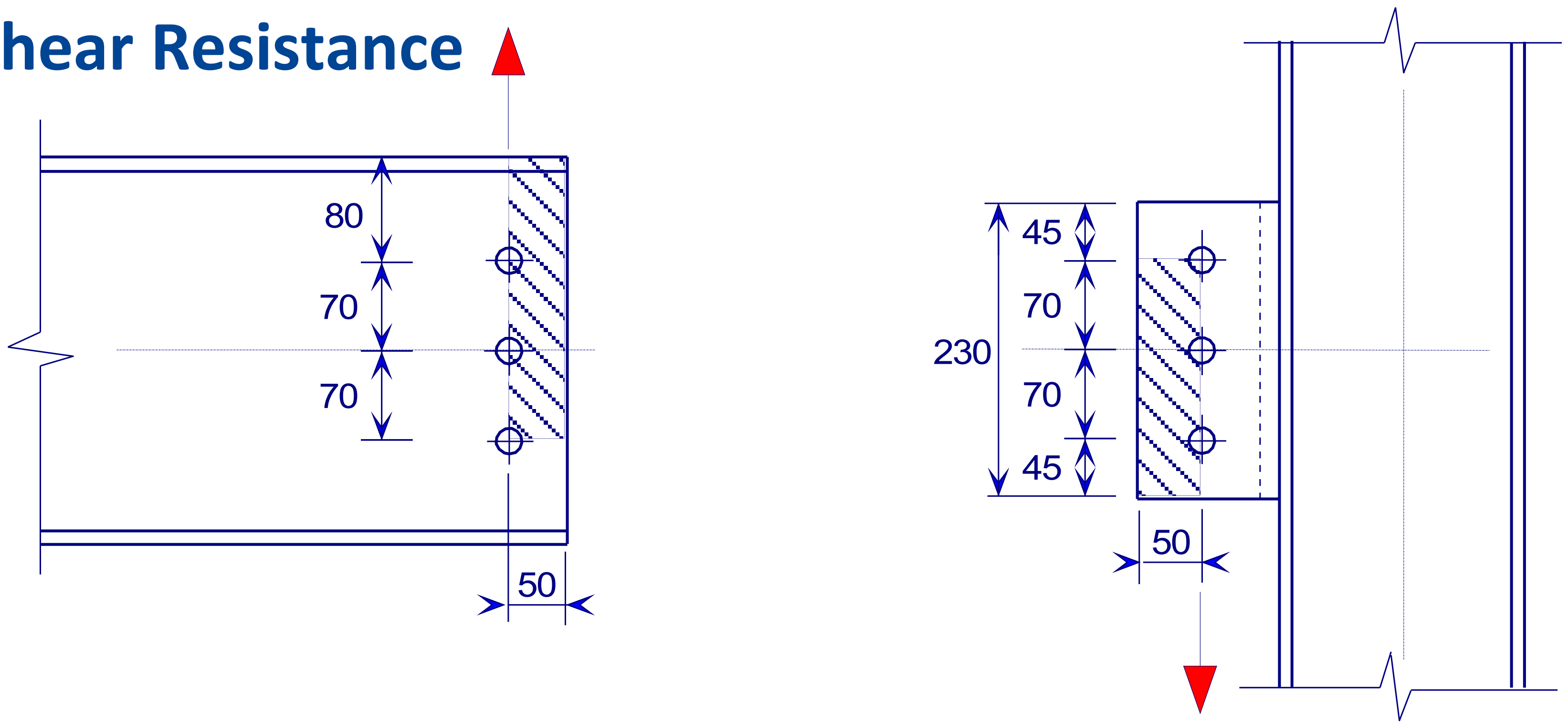
$$= \frac{0{,}5 \cdot 530 \cdot (60 - (18 + 9)) \cdot 10}{1{,}25 \cdot 10^3} + \frac{1}{\sqrt{3}} \cdot 220 \cdot \frac{(240 - 4 \cdot 18) \cdot 10}{1{,}0 \cdot 10^3} = 70 + 175 = 245\,\text{kN}$$

Worked example –
Fin plate connection



Worked Example – Fin Plate connection

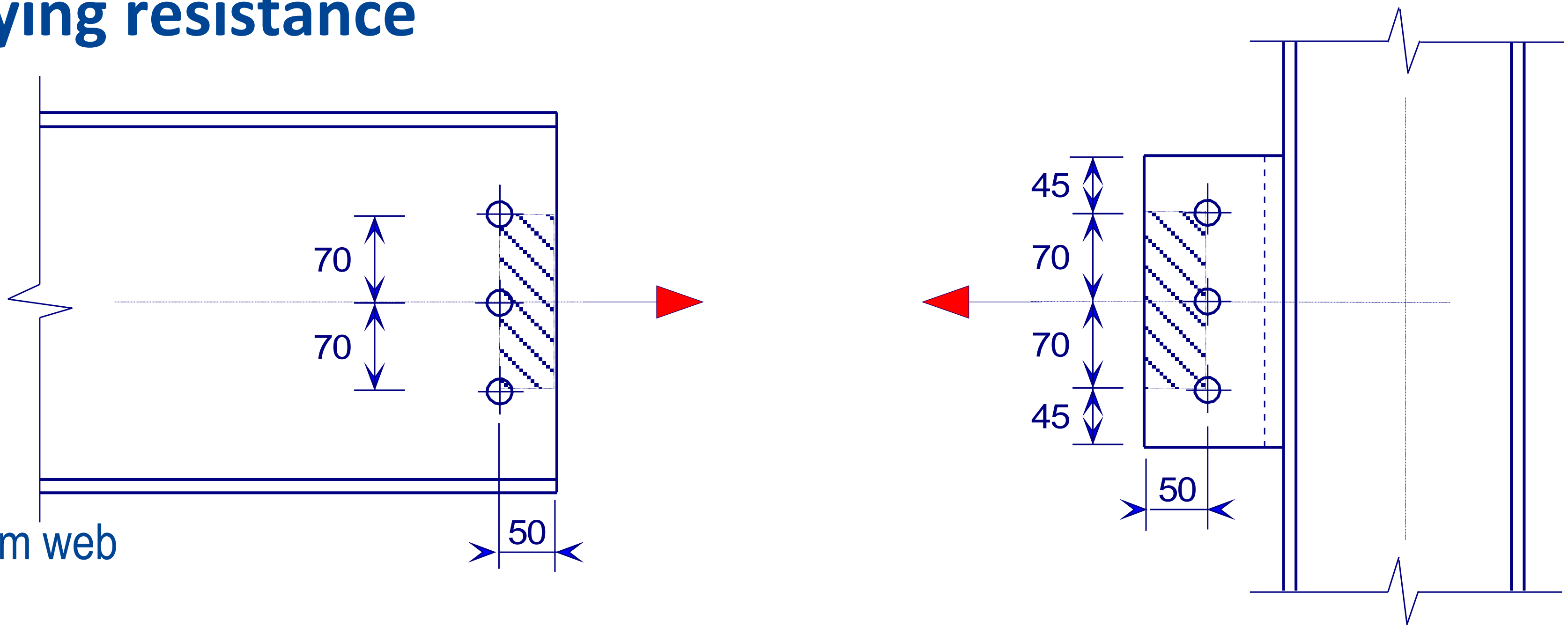
Shear Resistance



$$V_{Rd,11} = \frac{0,5\, f_{u,b1}\, A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}}\, f_{y,b1}\, \frac{A_{nv}}{\gamma_{M0}} = \frac{0,5 \cdot 360 \cdot (50-11) \cdot 7,1}{1,25 \cdot 10^3} + \frac{1}{\sqrt{3}} \cdot 235 \cdot \frac{(220 - 2 \times 22 - 11) \cdot 7,1}{1,0 \cdot 10^3}$$

$$= 39,9 + 159\,\text{kN} = 199\,\text{kN}$$

Worked example – Fin plate,
Tying resistance



In beam web

$$N_{Rd,u,6} = \frac{f_{u,b1} A_{nt}}{\gamma_{M,u}} + \frac{1}{\sqrt{3}} f_{y,b1} \frac{A_{nv}}{\gamma_{M0}} = \frac{360 \times 7{,}1(140 - 2 \times 22)}{1{,}1} + \frac{1}{\sqrt{3}} \times 235 \times \frac{2 \times 7{,}1(50 - 11)}{1{,}0}$$

$$= 223 + 75.1 = 298\,\text{kN}$$

Scope of the lecture

Bolts

- General
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  - Non-preloading bolts
  - Slotted holes
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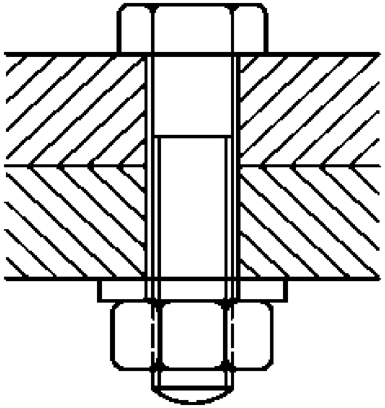
- Summary



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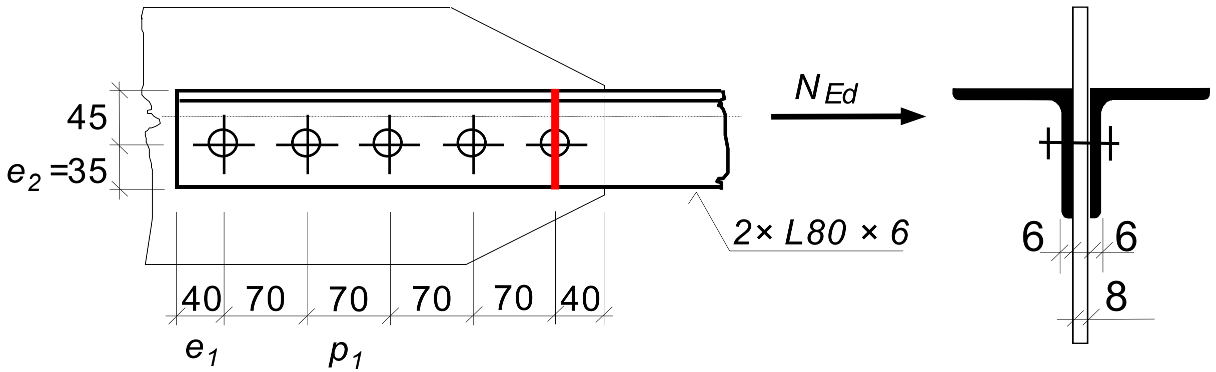
Bolted connection of double angle bar



Bolts M20 class 5.6 fully treated

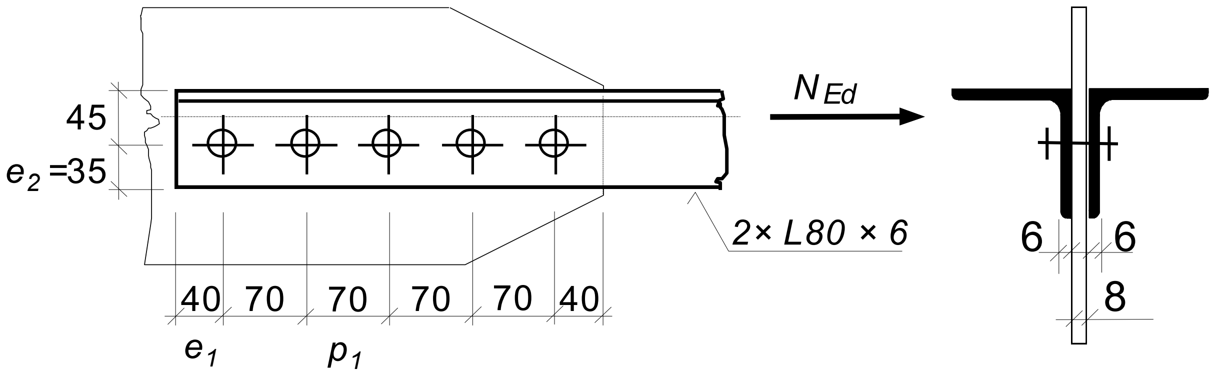
Loading N_{Ed} = 400 kN

Angle net section

$$N_{u,Rd} = \frac{0{,}9\, A_{net}\, f_u}{\gamma_{M2}} = \frac{0{,}9 \cdot 2 \cdot (935 - 22 \cdot 6) \cdot 360}{1{,}25} = 416{,}3\,\text{kN} > N_{Ed} = 400\,\text{kN}$$

Satisfactory

Bolted connection of double angle bar



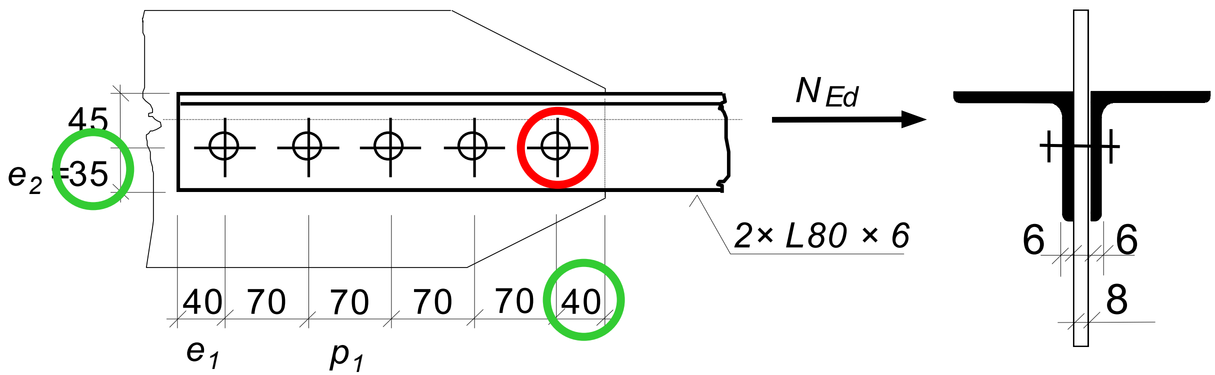
Bolts in shear

Two shear planes
Shear in bolt thread

Resistance for one bolt

$$F_{v.Rd} = 2\frac{\alpha_v A_s f_{ub}}{\gamma_{M2}} = 2\cdot\frac{0{,}6\cdot 245\cdot 500}{1{,}25} = 117{,}6\,\mathrm{kN}$$

Bolted connection of double angle bar



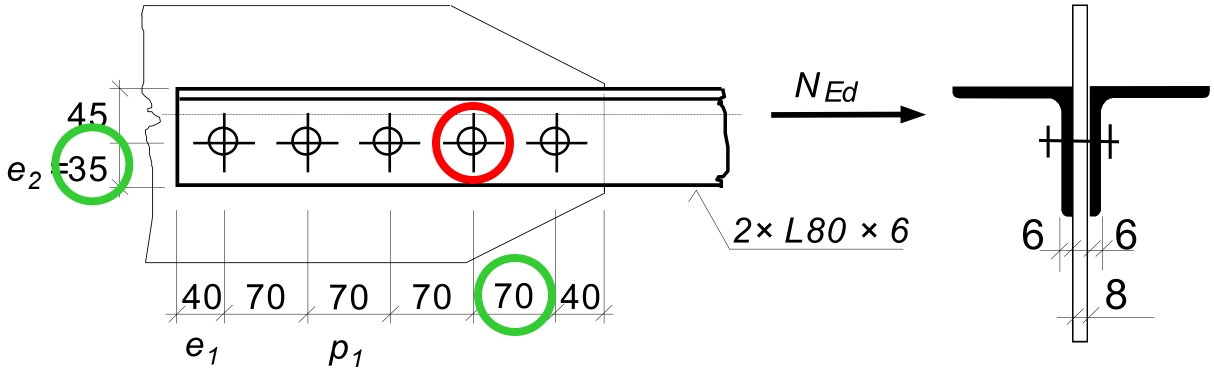
Bearing of end bolt

$$k_1 = \min\left(2{,}8\frac{e_2}{d_0} - 1{,}7;\ 2{,}5\right) = \min\left(2{,}8\cdot\frac{35}{22} - 1{,}7;\ 2{,}5\right) = \min(2{,}75;\ 2{,}5) \quad\rightarrow\quad k_1 = 2{,}5$$

$$\alpha_b = \min\left\{\begin{matrix}\dfrac{e_1}{3\,d_0}\\[2ex] \dfrac{f_{ub}}{f_u}\\[2ex] 1{,}0\end{matrix}\right\} = \min\left\{\begin{matrix}\dfrac{40}{3\cdot 22}\\[2ex] \dfrac{500}{360}\\[2ex] 1{,}0\end{matrix}\right\} = \min\left\{\begin{matrix}0{,}606\\[2ex] 1{,}389\\[2ex] 1{,}0\end{matrix}\right\} = 0{,}606$$

$$F_{b,Rd} = \frac{k_1\ \alpha\ d\ t\ f_u}{\gamma_{M2}} = \frac{2{,}5\cdot 0{,}606\cdot 20\cdot 8\cdot 360}{1{,}25} = 87{,}3\ \text{kN}$$

Bolted connection of double angle bar



Bearing of internal bolt

$$k_1 = \min\left(2{,}8\,\frac{e_2}{d_0} - 1{,}7;\ 2{,}5\right) = \min\left(2{,}8 \cdot \frac{35}{22} - 1{,}7;\ 2{,}5\right) = \min(2{,}75;\ 2{,}5) \rightarrow k_1 = 2{,}5$$

$$\alpha_b = \min\begin{Bmatrix} \dfrac{p_1}{3\,d_0} - \dfrac{1}{4} \\ \dfrac{f_{ub}}{f_u} \\ 1{,}0 \end{Bmatrix} = \min\begin{Bmatrix} \dfrac{70}{3 \cdot 22} - \dfrac{1}{4} \\ \dfrac{500}{360} \\ 1{,}0 \end{Bmatrix} = \min\begin{Bmatrix} 0{,}811 \\ 1{,}389 \\ 1{,}0 \end{Bmatrix} = 0{,}811$$

$$F_{b,Rd} = \frac{2{,}5 \cdot 0{,}811 \cdot 20 \cdot 8 \cdot 360}{1{,}25} = 93{,}4\ \text{kN}$$

Bolted connection of double angle bar

Check of bolts

Shear resistance	117,6 kN
Bearing resistance – end bolt	87,3 kN
Bearing resistance – internal bolt	93,4 kN

Shear is not guiding the resistance, e.g. bearing as sum
For connection with five bolts

$$87{,}3 + 3 \cdot 94{,}3 + 87{,}3 = 457{,}5\,\text{kN} > 400\,\text{kN} = N_{\text{Ed}}$$

Satisfactory

Conservatively elastic (minimal) resistance

Lower resistance from bearings

$$5 \cdot 87{,}3 = 436{,}5\,\text{kN} > 400\,\text{kN} = N_{\text{Ed}}$$

Unsatisfactory

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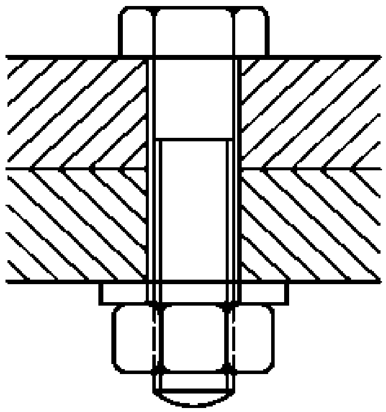

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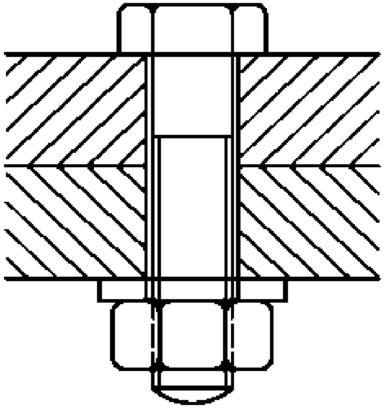
Summary for bolted connections

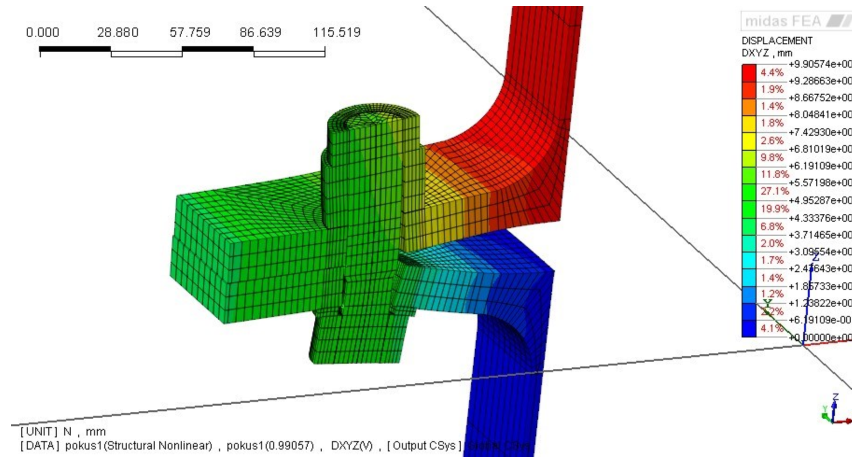


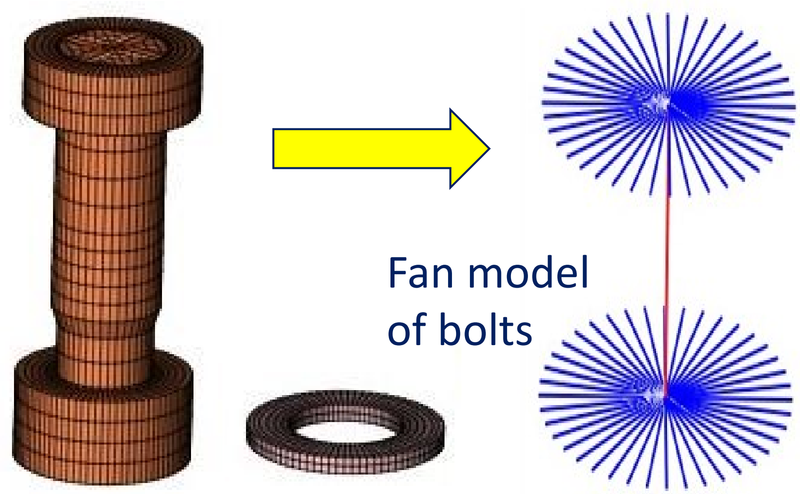
- **Connections made with bolts, rivets or pins** in Chapter 3 of EN 1993-1-8
  - **Non-preloaded bolts**
  - **Preloaded bolts**
    preload (0,7 f_{ub})
  - **Injection bolts**
    replacement of rivets; bolts 8.8 and 10.9
  - **Pins**
    including serviceability

Future?
Advanced models
of bolted connections

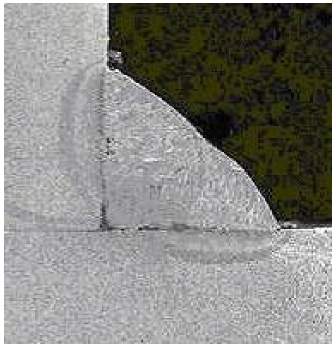
- **FEM research** models
  - Validated on experiments

- **FEM design** models
  - Verified against analytical and numerical models
  - Bars and springs model
    - In tension – stiffness, resistance
    - In shear - contact





Scope of the lecture



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Welded connections in EN1993-1-8: 2005

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4.14 Welding in cold-formed zones 49

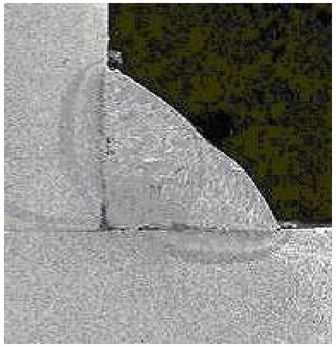
Electrode classification

EN 499 classification of carbon and low alloy steel electrodes

- Covered electrode / manual metal arc welding
- Strength & elongation
- Impact properties (47J at 0°C)
- Type of electrode covering (rutile-cellulosic)
- Recovery and type of current
- Welding position (all positions)

E 38 0 RC 1 1

Scope of the lecture



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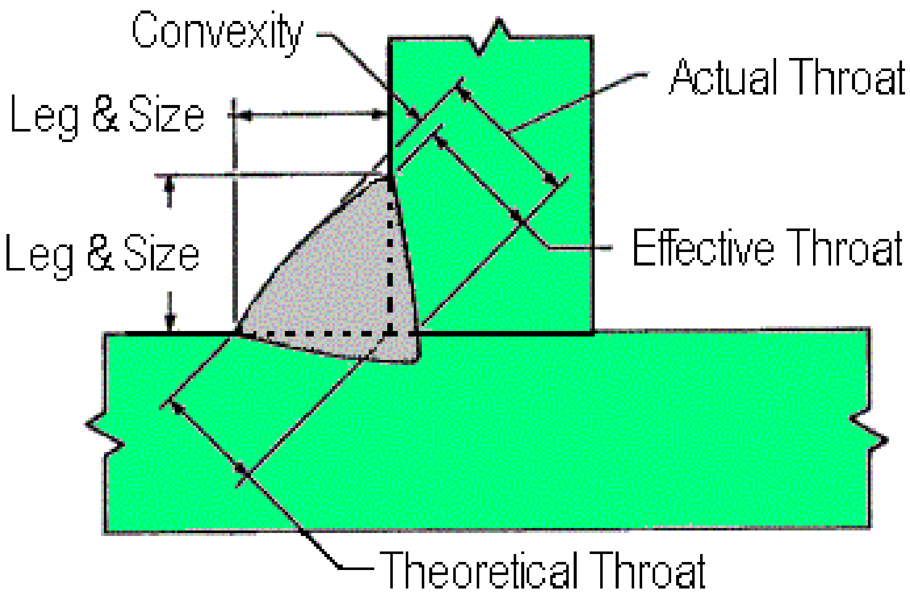
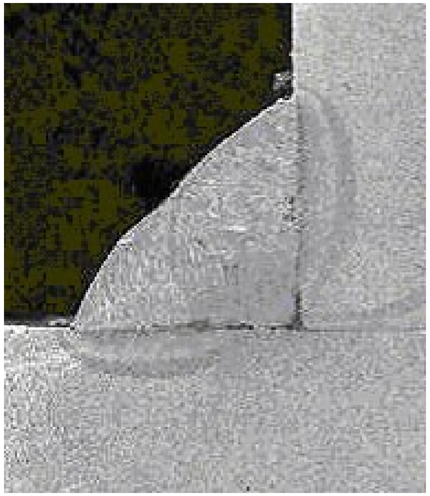
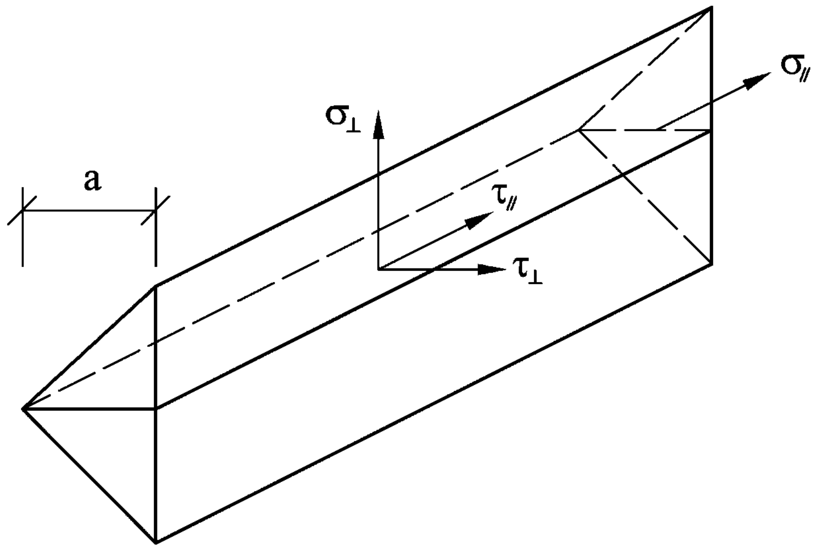
Fillet welds –
Definition of effective throat thickness *a*

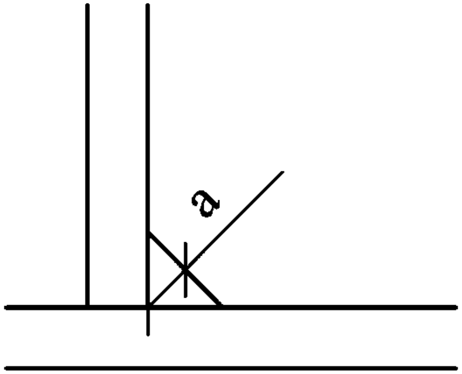


The effective throat thickness of a fillet weld should not be less than 3 mm



Design Model of Fillet Welds





a effective throat thickness of the fillet weld

$\sigma_{\perp}$ normal stresses perpendicular to the throat

$\sigma_{\parallel}$ normal stresses parallel to the axis of weld (omitted)

$\tau_{\perp}$ shear stresses perpendicular to the axis of weld

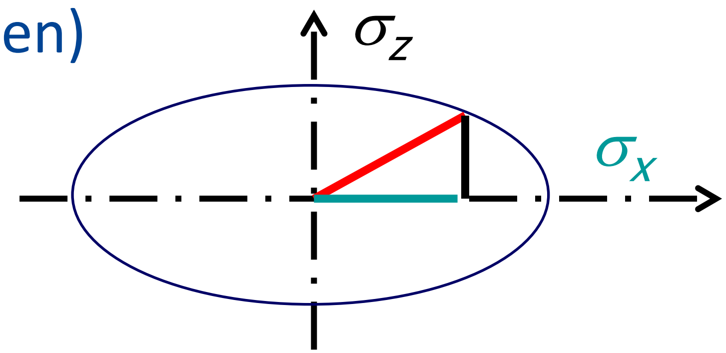
$\tau_{\parallel}$ shear stresses parallel to the axis of weld

Plane Stresses

Huber-Mises-Henckey condition of plasticity (HMH)

- Triaxial state of stress (needed exceptionally only)
- Plane state of stress (needed very often)

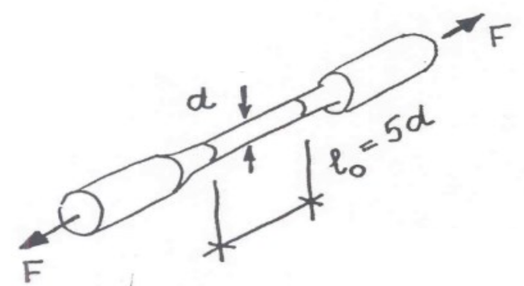
$$\sigma_x^2 + \sigma_z^2 - \sigma_x^2 \sigma_z^2 + 3\tau^2 \leq (f_y / \gamma_M)^2$$



- Uniaxial state of stress (from the material tests)

$$\sigma \leq f_y / \gamma_{M0}$$

$$\tau \leq f_y / (\gamma_{M0} \sqrt{3})$$



Design Resistance of Filet Weld

$$\sqrt{\sigma_{\perp}^2 + 3\left(\tau_{\perp}^2 + \tau_{II}^2\right)} \leq f_u / (\beta_w \gamma_{Mw})$$

$$\sigma_{\perp} \leq f_u / \gamma_{Mw}$$

f_u Ultimate tensile strength of connected material

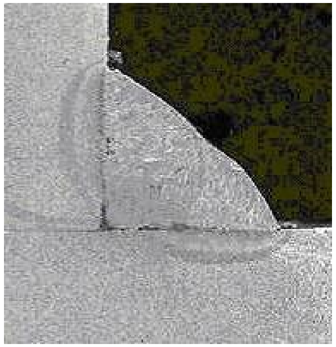
β_w Correlation factor

γ_{Mw} Partial safety factor for material of welds

Correlation factor β_w for fillet welds

Standard and steel grade			Correlation factor β_w
EN 10025	**EN 10210**	**EN 10219**	
S 235 S 235 W	S 235 H	S 235 H	0,80
S 275 S 275 N/NL S 275 M/ML	S 275 H S 275 NH/NLH	S 275 H S 275 NH/NLH S 275 MH/MLH	0,85
S 355 S 355 N/NL S 355 M/ML S 355 W	S 355 H S 355 NH/NLH	S 355 H S 355 NH/NLH S 355 MH/MLH	0,90
S 420 N/NL S 420 M/ML		S 420 MH/MLH	1,00
S 460 N/NL S 460 M/ML S 460 Q/QL/QL1	S 460 NH/NLH	S 460 NH/NLH S 460 MH/MLH	1,00

Scope of the lecture



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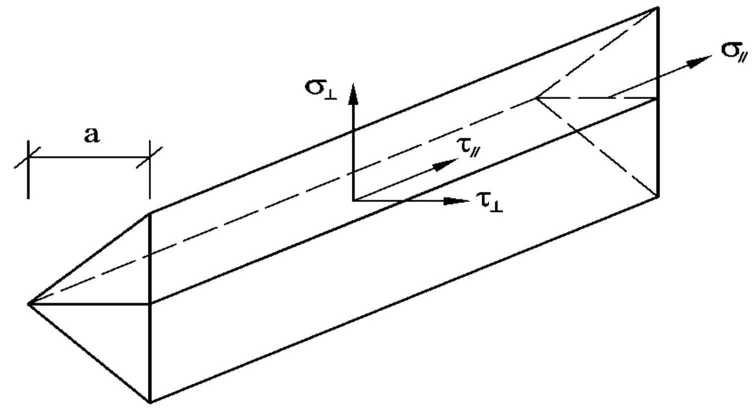
Two fillet welds in parallel shear

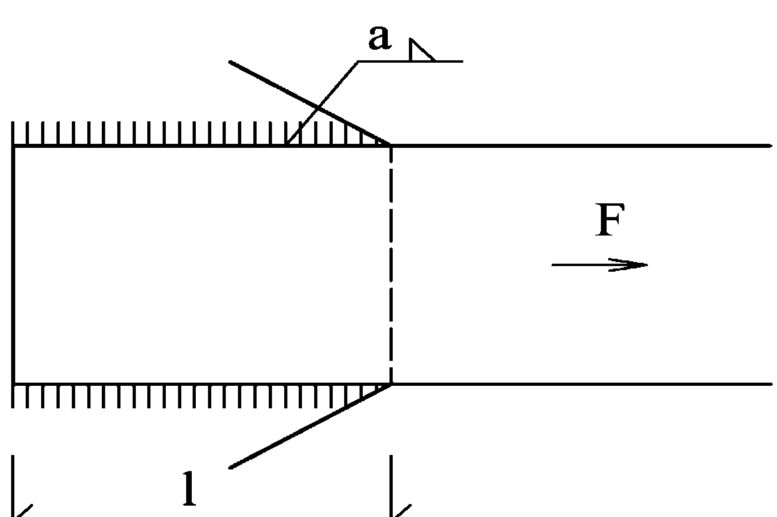


$$\tau_{\|} = F/2a\,\ell_{\|}$$

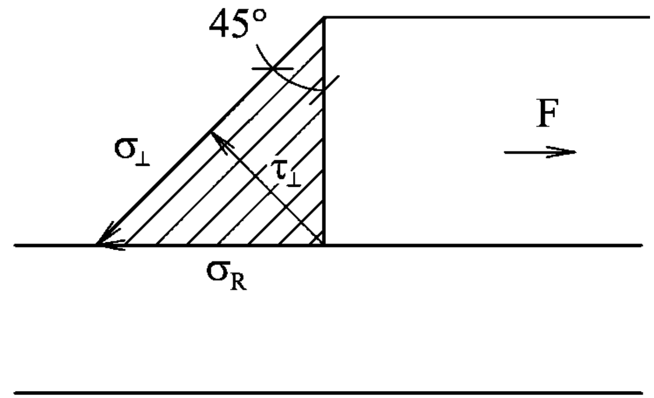
From plane stress analysis is

$$F/2a\,\ell_{\|} \leq f_u/\left(\beta_w \gamma_{Mw} \sqrt{3}\right)$$

Throat thickness, not leg length



Fillet weld in normal shear

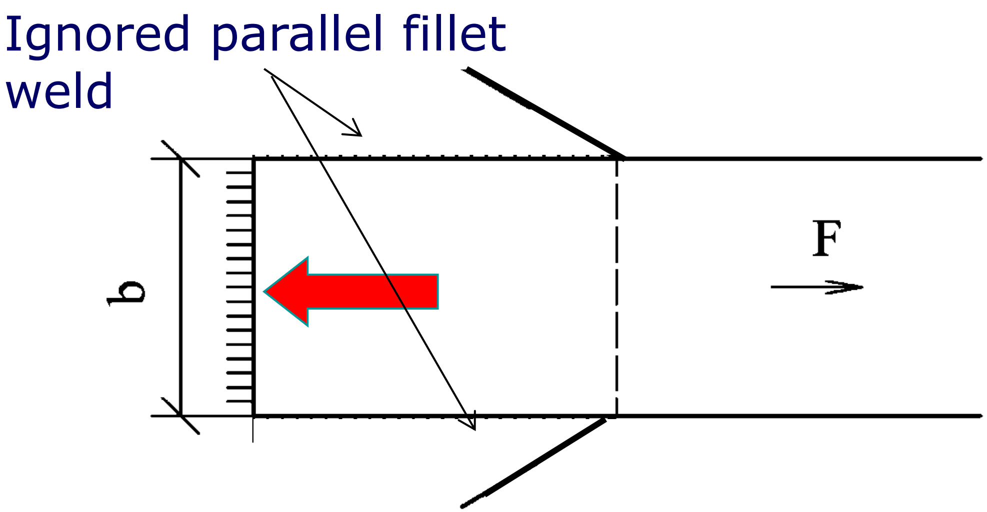


$$\tau_{II} = 0$$

$$\sigma_{\perp} = \tau_{\perp} = \sigma_R / \sqrt{2}$$

Has to be satisfied

$$\sqrt{\sigma_{\perp}^2 + 3\tau_{\perp}^2} \leq f_u / (\beta_w \gamma_{Mw})$$



After substitution

$$\sqrt{\left(\sigma_R/\sqrt{2}\right)^2 + 3\left(\sigma_R/\sqrt{2}\right)^2} = \sqrt{2\sigma_R^2} \leq f_u / (\beta_w \gamma_{Mw})$$

$$\sigma_R \leq f_u / \left(\beta_w \gamma_{Mw} \sqrt{2}\right)$$

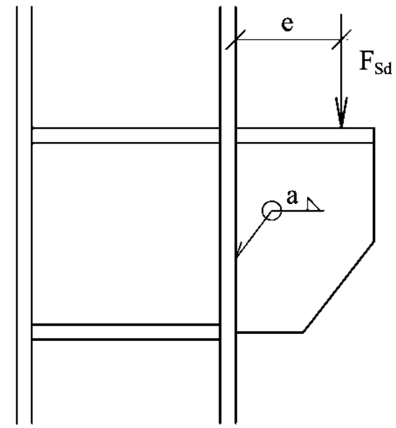
Joint Research Centre

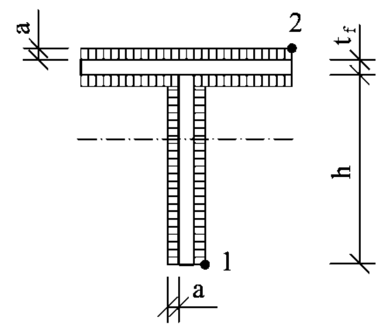
Connection of cantilever

Shear force $V_{Sd} = F_{Sd}.$

Transferred by web fillets $\tau_{II} = F_{Sd} / 2\,a\,h$

Bending moment $M_{Sd} = F_{Sd}\,e$





Transferred by the shape of weld

Centre of gravity, I_{we} and cross section modulus W_{we}

For weld at lower flange cross section modulus $W_{we,1}$ and stress is

$$\sigma_{\perp 1} = \tau_{\perp 1} = \left(M_{Sd} / \sqrt{2}\right) / W_{we,1}$$

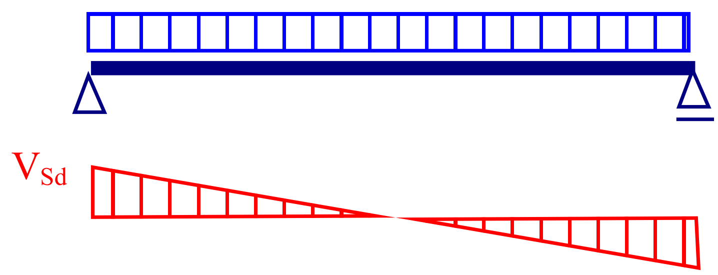
For upper weld on flange is

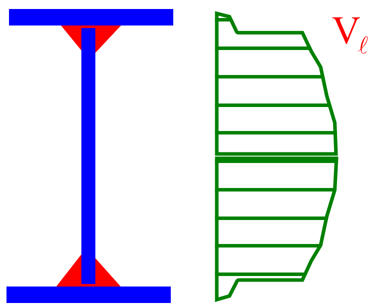
$$\sigma_{\perp 2} = \tau_{\perp 2} = \left(M_{Sd} / \sqrt{2}\right) / W_{we,2}$$

Flange - web weld

Welds are loaded by longitudinal shear force

$$V_l = V_{Sd} S / I$$





where V_{Sd} shear force

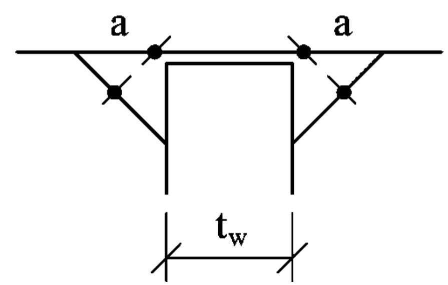
S static moment of flange to neutral axis

I moment of inertia

This longitudinal force is carried by two welds effective thickness a

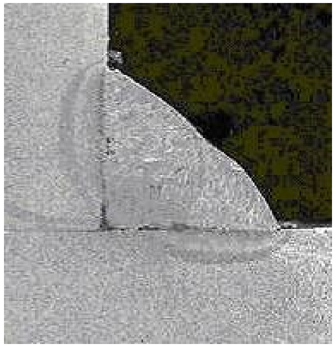
Shear stress

$$\tau_{II} = V_l / 2a \leq f_u / \beta_w \gamma_{Mw} \sqrt{3}$$



Maximum stress is at the point of maximum shear force

Scope of the lecture



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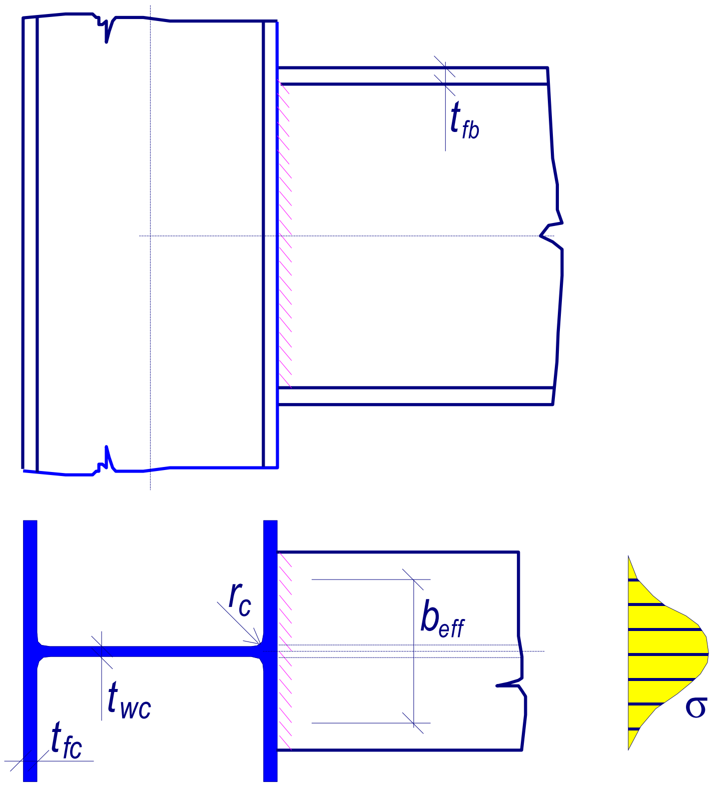
Welds to flexible plate

Effective width of unstiffened column flanges

EN 1993-1-8 Chapter 4.10

$$b_{\mathrm{eff}} = t_{\mathrm{wc}} + 2s + 7t_{\mathrm{fc}}$$

$$b_{\mathrm{eff}} = t_{\mathrm{wc}} + 2s + 7\left(\frac{t_{\mathrm{fc}}^{2}}{t_{\mathrm{fb}}}\right)\left(\frac{f_{\mathrm{yc}}}{f_{\mathrm{yb}}}\right)$$



t_{wc} thickness of column web
t_{fc} thickness of column flange
t_{fb} thickness of beam flange
s equal to fillet radius r_{c} for hot rolled column sections

Effective Width

Unstiffened column flanges
In EN1993-1-8 Clause 6.2.4.4

$$F_{t,fc,Rd} = \left(t_{wc} + 2s + 7k\,t_{fc}\right)\frac{t_{fb}\,f_{yb}}{\gamma_{M0}}$$

$$k = \min\left(\frac{f_{yc}\,t_{fc}}{f_{yb}\,t_{fb}};1\right)$$

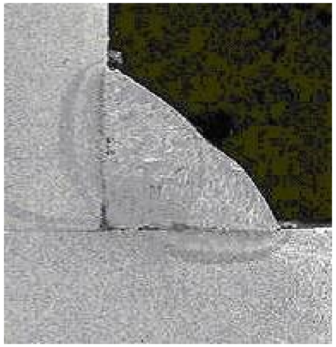
t_{wc} is thickness of column web

t_{fc} thickness of column flange

t_{fb} thickness of beam flange

s is equal to fillet radius r_c for hot rolled column sections

Scope of the lecture



Bolts

- General
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  - Resistance
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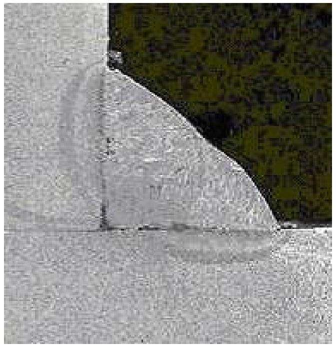
Summary



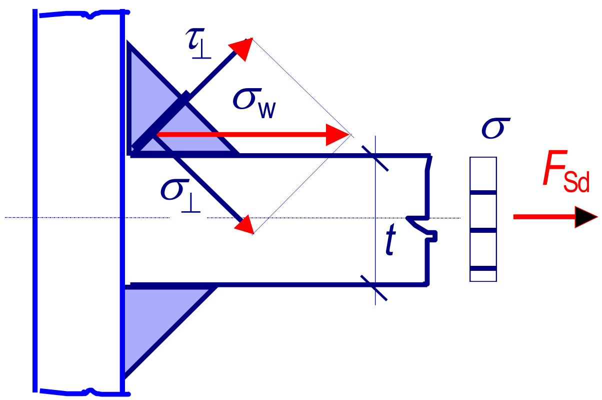
- Chapter 4 in EN1993-1-8: 2005
  - Rules for connection of open sections
- Chapter 7 in EN1993-1-8: 2005
  - Rules for connection of hollow sections
- New rules for high strength steels
- Size of welds
  - Weld design for full resistance

Weld design for full resistance

Loading by normal by connecting member

Not directly in code

$$a > 0{,}7\,\frac{\sigma\, t}{f_u / \gamma_{Mw}}$$



$\sigma = F_{Sd} / (t\ h)$

F_{Sd} is the acting design force

f_u is plate design strength

t is the thinness of connecting plate

b is width of connecting plate

Full capacity of a plate the thickness for steel S235

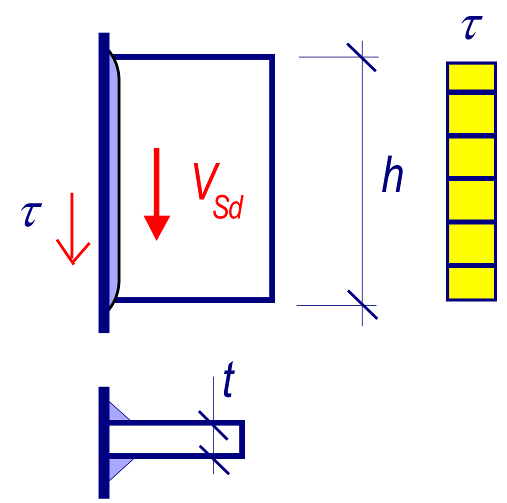
$$a > 0{,}7\,\frac{(f_y / \gamma_{M0})\, t}{f_u / \gamma_{Mw}} = 0{,}7\,\frac{(235/1{,}0)\, t}{360/1{,}25} = 0{,}47\, t \approx 0{,}5\, t$$

Weld design for full resistance

Loading by shear force by connecting member

$\tau = V_{Sd} / (t\, h)$

V_{Sd} is the design shear force in weld



For full capacity of a plate thickness S235

$$a > 0{,}85 \frac{\tau\, t}{f_w / \gamma_{Mw}} \approx 0{,}85 \frac{f_y / (\sqrt{3}\, \gamma_{M0})\, t}{f_u / \gamma_{Mw}} = 0{,}85 \frac{235/(1{,}0\sqrt{3})\, t}{360/1{,}25} = 0{,}33\, t \cong 0{,}3\, t$$

Weld Design
or Full Resistance of Connecting Members

Loading by normal force ~ 0,5 *t*

Loading by shear force ~ 0,3 *t*

Compare to AISC is less economical design

AISC – LRFD – matching with SMAW			EN1993-1-8: 2005		
f_y (N/mm²)	f_{EXX} (ksi – N/mm²)	a	Steel grade	$f_{w.u.side}$ (N/mm²)	a
235	60 – 414 70 – 483	0,37 *t* 0,33 *t*	S235	208	0,37 *t*
355	70 – 483	0,49 *t*	S355	262	0,45 *t*
420	80 – 552	0,51 *t*	S420 M S420 N	230 240	0,61 *t* 0,58 t
485	90 – 621	**0,52 *t***	S460 M S460 N	245 254	**0,63 *t*** **0,61 *t***

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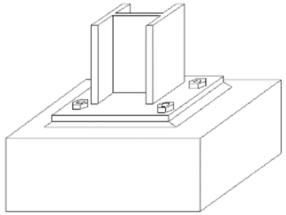

Column bases

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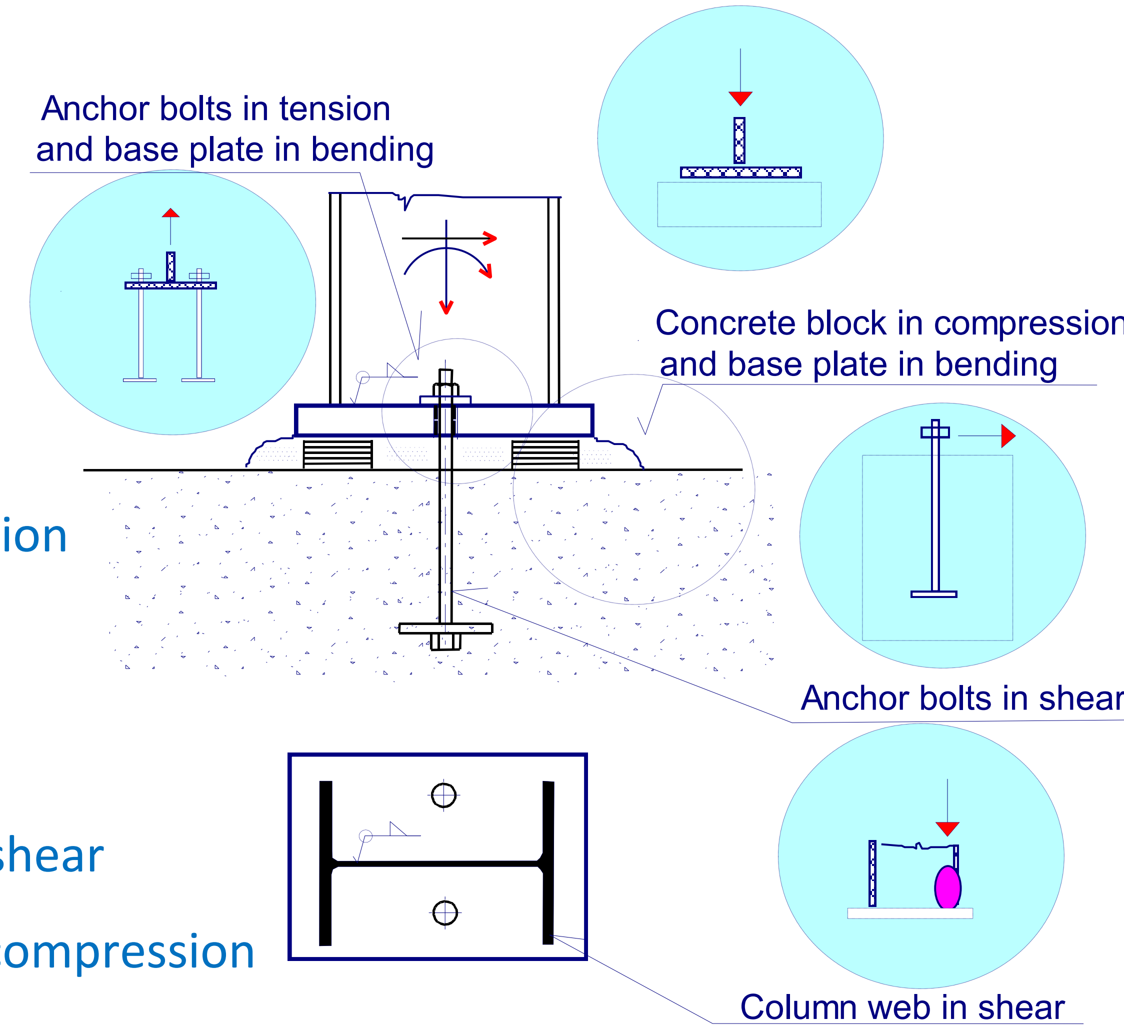
Column bases in chap. 6 of EN 1993-1-8: 2005

Unfortunately in 6 clauses

- Resistance in **cl. 6.2.8, cl. 6.2.5(7)**
- Stiffness in **cl. 6.3.1(4)**
- Classification in **cl. 5.2.2.5**

- Component concrete block in compression and base plate in bending in **cl. 6.7(2)**
- Component anchor bolt in tension and base plate in bendingin **cl. 6.2.6.11(2)**
- Component anchor bolt in shear **cl. 6.2.2(6)**

Component method
cl. 6.2.8



Components

- Concrete block in compression and base plate in bending
- Anchor bolt in tension and base plate in bending
- Component anchor bolt in shear
- Column web and flange in compression

Background materials

Wald F., Sokol Z., Steenhuis M. and Jaspart, J.P.,
Component Method for Steel Column Bases,
Heron. 2008, vol. 53, no. 1/2, 3-20, ISSN 0046-7316

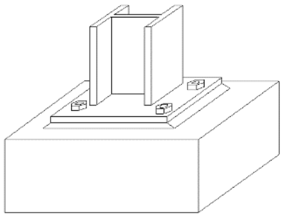
Steenhuis M., Wald F., Sokol Z. and Stark J.W.B.,
Concrete in Compression and Base Plate in Bending,
Heron. 2008, vol. 53, no. 1/2, 51-68, ISSN 0046-7316.

Wald F., Sokol Z. and Jaspart J.P.,
Base Plate in Bending and Anchor Bolts in Tension,
Heron, 2008, vol. 53, no. 1/2, 21-50, ISSN 0046-7316.

Gresnight N., Romeijn A., Wald F., and Steenhuis M.,
Column Bases in Shear and Normal Force,
Heron, 2008, vol. 53, no. 1/2, 87-108, ISSN 0046-7316.

EN 1992-4 Eurocode 2:
Design of concrete structures — Part 4: Design of Fastenings for Use in Concrete

Scope of the lecture



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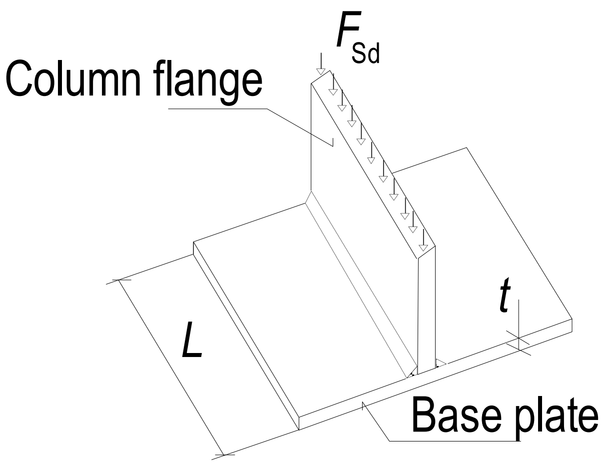
Component
Concrete in compression and base plate in bending

Design principles

- **3D behaviour od concrete** D
  - Design bearing strength of the joint f_{jd}
- **Flexible base plate on concrete block**
  - Effective rigid area under the flexible plate A_{eff}
- Deformation of concrete block
  - Stiffness coefficient for concrete deformation under the flexible plate k_c



Wald F., Sokol Z., Steenhuis M. and Jaspart, J.P.,
Component Method for Steel Column Bases, *Heron* 53 (2008) 3-20.

3D behaviour

Concentrated force for at concrete resistance $F_{Rd,u}$

- Design bearing strength of joint cl. 6.2.5(7)
  - Area of crushing of the concrete A_{c0}
  - According to **cl. 6.7(2) in EN 1992-1-1**

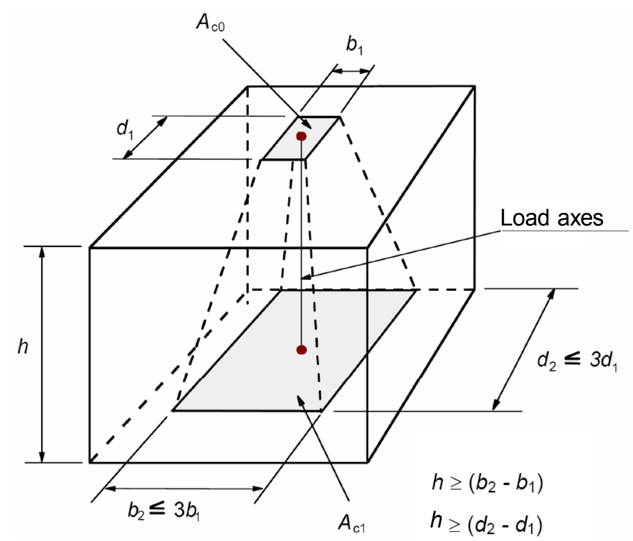
$$F_{Rd,u} = A_{c0}\, f_{cd} \sqrt{\frac{A_{c1}}{A_{c0}}} \leq 3{,}0\, A_{c0}\, f_{cd}$$

$A_{c0} = b_1\, d_1$

$A_{c1} = b_2\, d_2$

$h \geq b_2 - b_1;\ h \geq d_2 - d_1$

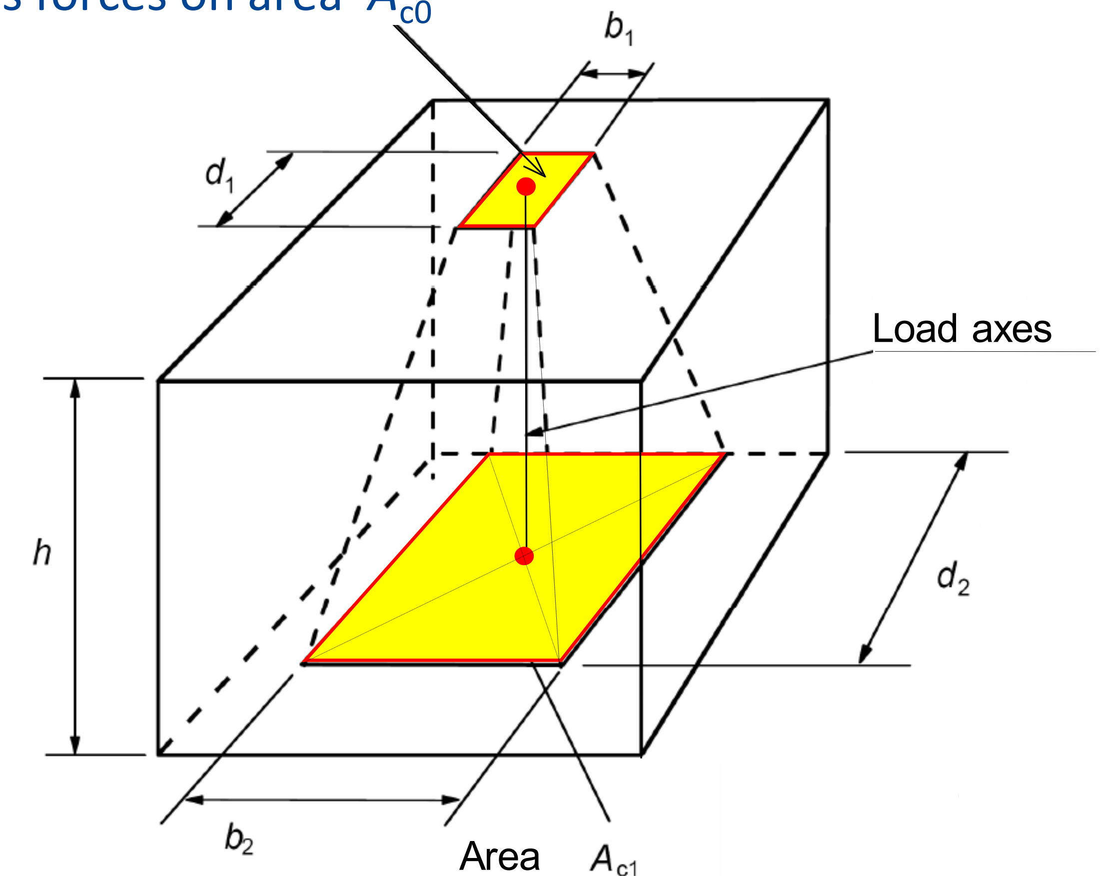
$3 \cdot b_1 \geq b_2$ a $3 \cdot d_1 \geq d_2$



Cursing of the concrete

cl. 6.7(2) in EN 1992-1-1

Homogenous forces on area A_{c0}

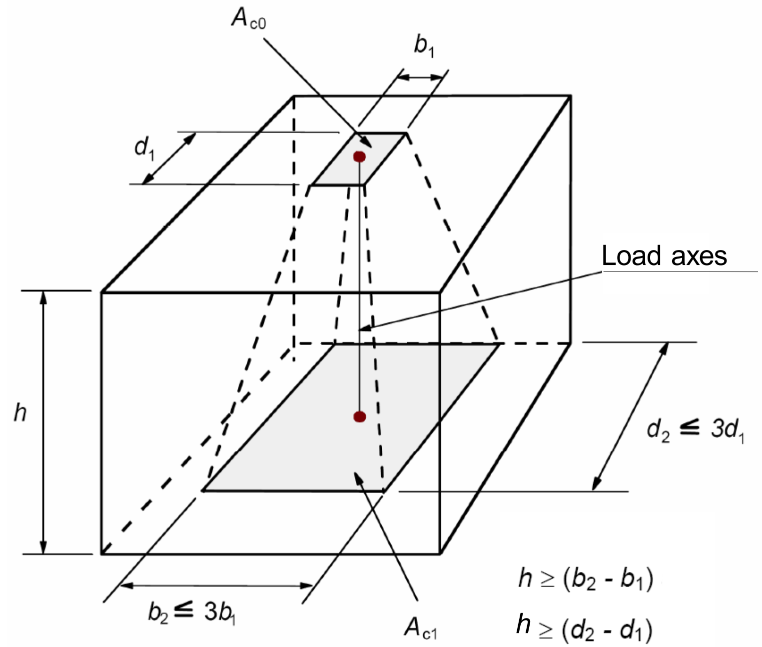


Concrete design strength in joint

$$f_{jd} = \frac{\beta_j\ F_{Rdu}}{b_{ef}\ l_{ef}} = \frac{\beta_j\ A_{c0}\ f_{cd}\sqrt{\dfrac{A_{c1}}{A_{c0}}}}{A_{c0}} = \beta_j\ f_{cd}\sqrt{\frac{A_{c1}}{A_{c0}}} \le \frac{3{,}0\ A_{c0}\ f_{cd}}{A_{c0}} = 3{,}0\ f_{cd}$$

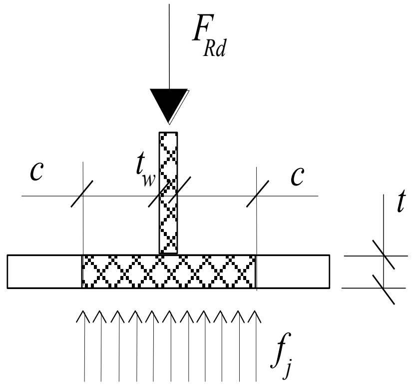
β_j = 2/3 is joint coefficient

F_{cd} is concrete compressive strength



Flexible base plate on concrete block

- Effective rigid plate
- Elastic deformation of plate only



Elastic bending moment of the base plate per unit length is

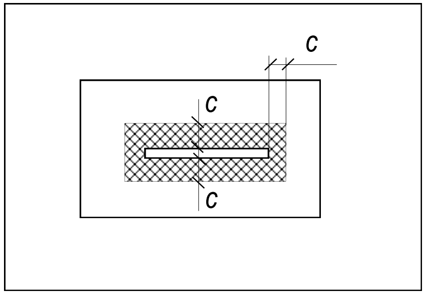
$$M' = \frac{1}{6} t^2 \frac{f_y}{\gamma_{M0}}$$

and the bending moment per unit length on the base plate of span c and loaded by distributed load is

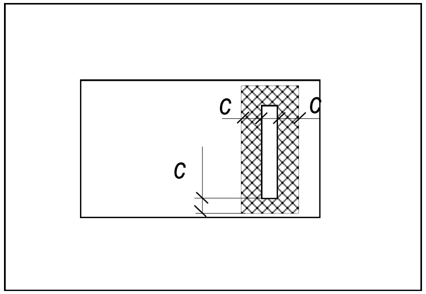
$$M' = \frac{1}{2} f_j \, c^2$$

where f_j is concrete bearing strength

Effective width of flexible plate *c*



Effective width $$c = t\sqrt{\frac{f_y}{3\, f_{jd}\, \gamma_{M0}}}$$



where

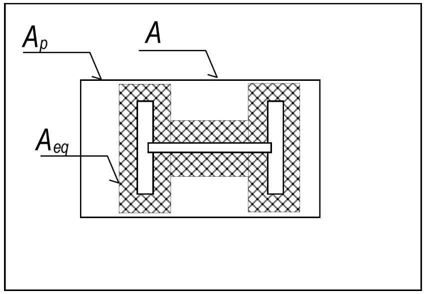
t is the plate thickness

f_y is the base plate yield strength

f_{jd} is the design bearing strength of the joint

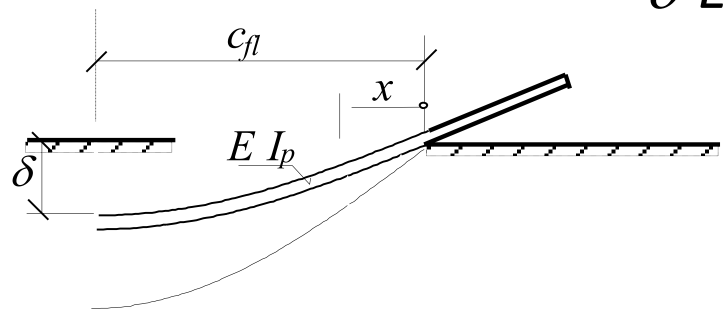
γ_{M0} is the partial safety factor for concrete

Effective area



Stiffness

Deformation of an elastic hemisphere $$\delta_r = \frac{F\,\alpha\,a_r}{\delta\,E} = \frac{0{,}85\,F}{E_c\sqrt{a_{rl}\,L}} =$$



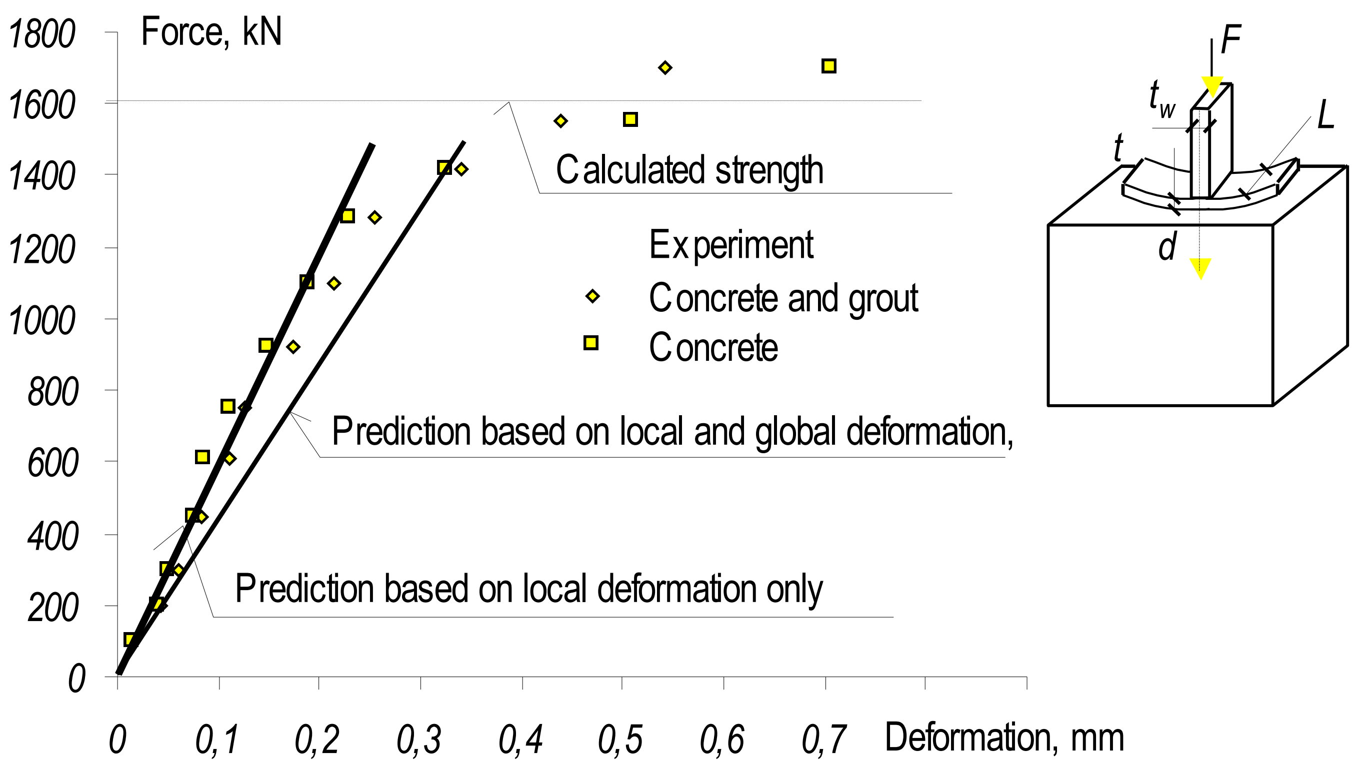
Stiffness coefficient $$k_c = \frac{F}{\delta\,E} = \frac{E_c\,\sqrt{a_{eq,el}\,L}}{1{,}5\cdot 0{,}85\,E} = \frac{E_c\,\sqrt{a_{eq,el}\,L}}{1{,}275\,E}$$

Width of effective T stub a_r in elastic stage

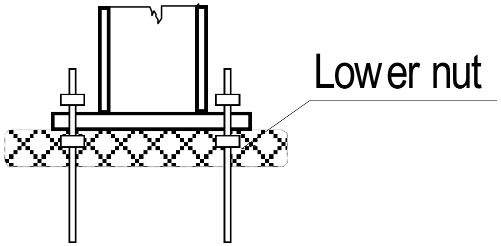
$$a_{eq,el} = t_w + 2{,}5\,t \approx a_{eq,st} =$$

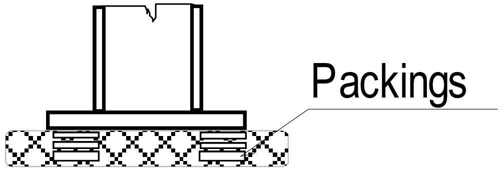
$$= t_w + 2\,c = t_w + 2\,t\sqrt{\frac{f_y}{3\,f_{jd}\,\gamma_{M0}}}$$

Validation to experiments



Influence of grout





- Grout **higher quality** than concrete block

  $\beta_1 = 2/3 \approx 1{,}0$

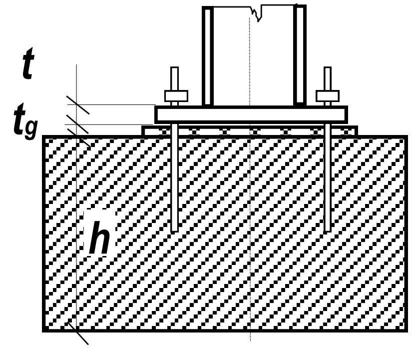
- Grout **lower quality** than concrete block
  - Model as plate on liquid

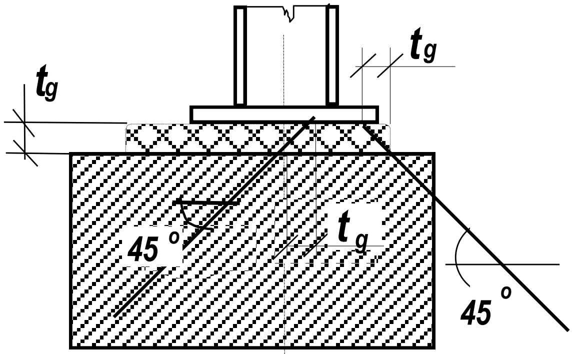
$$\beta_j = 2/3$$

$$f_{c.g} \geq 0{,}2\ f_c$$

$$t_g \leq 0{,}2\ \min\ (a\ ;\ b)$$

$$t_g \geq 0{,}2\ \min\ (a\ ;\ b)$$





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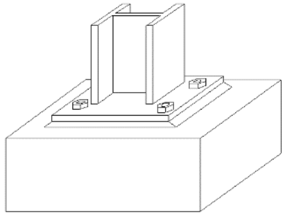

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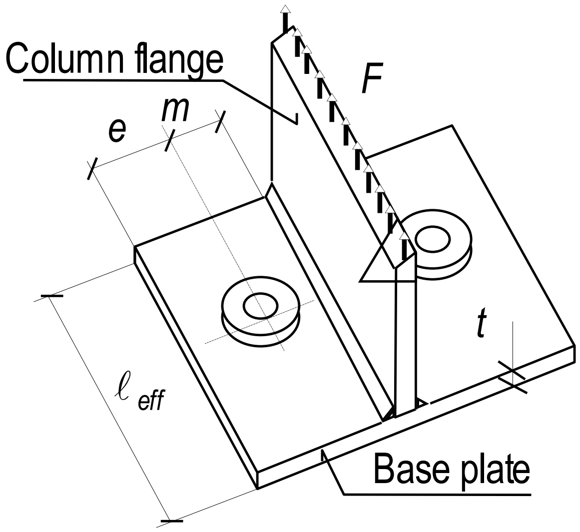
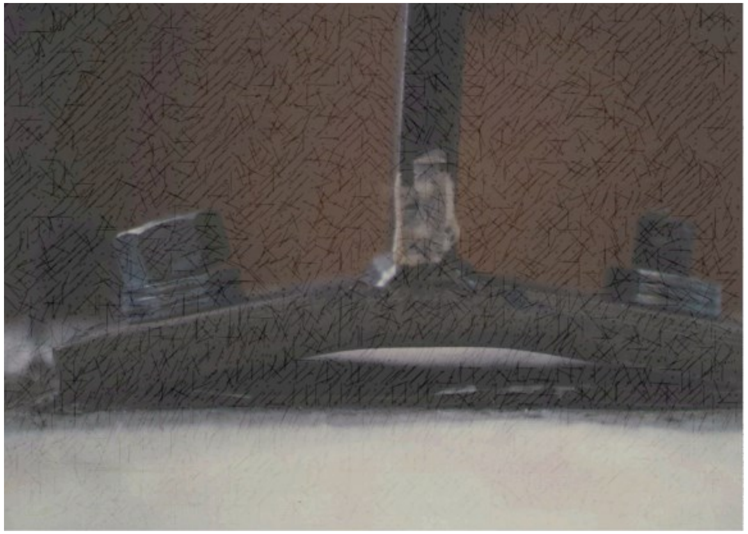
Component
Anchor bolts in tension and base plate in bending

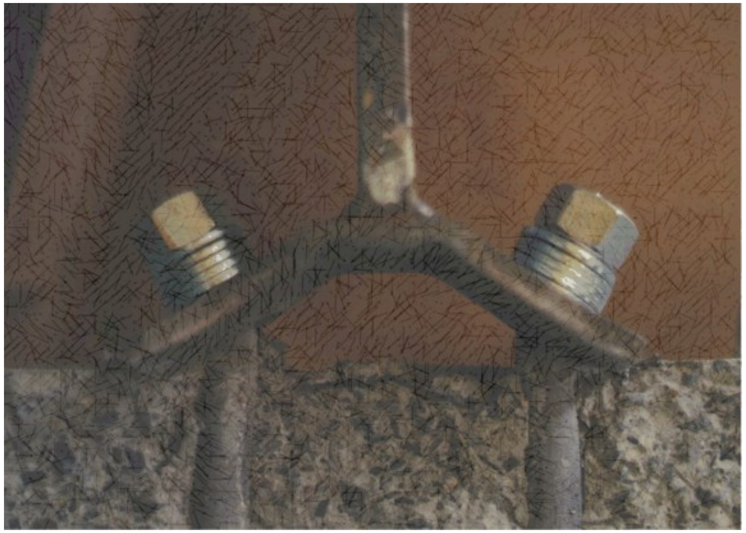
- Base plate compare to end plate
- Base plate is thicker
- Anchor bolt is longer
- In most cases no prying forces

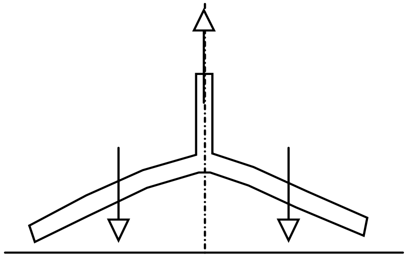


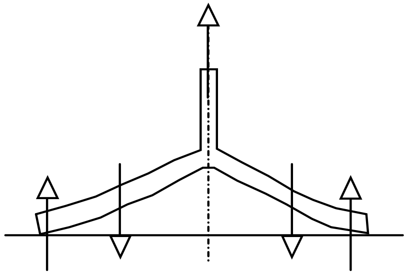
Contact of edge of T stub

on experiment



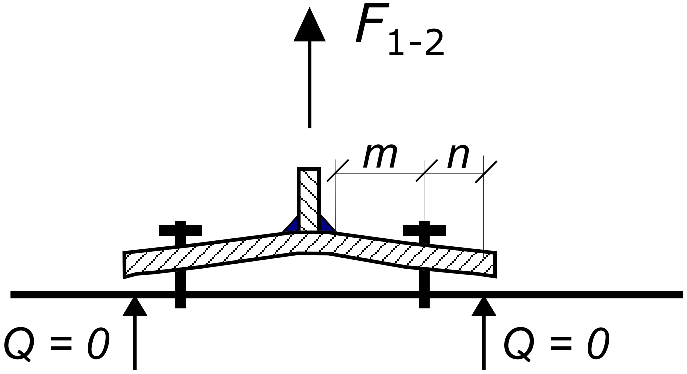


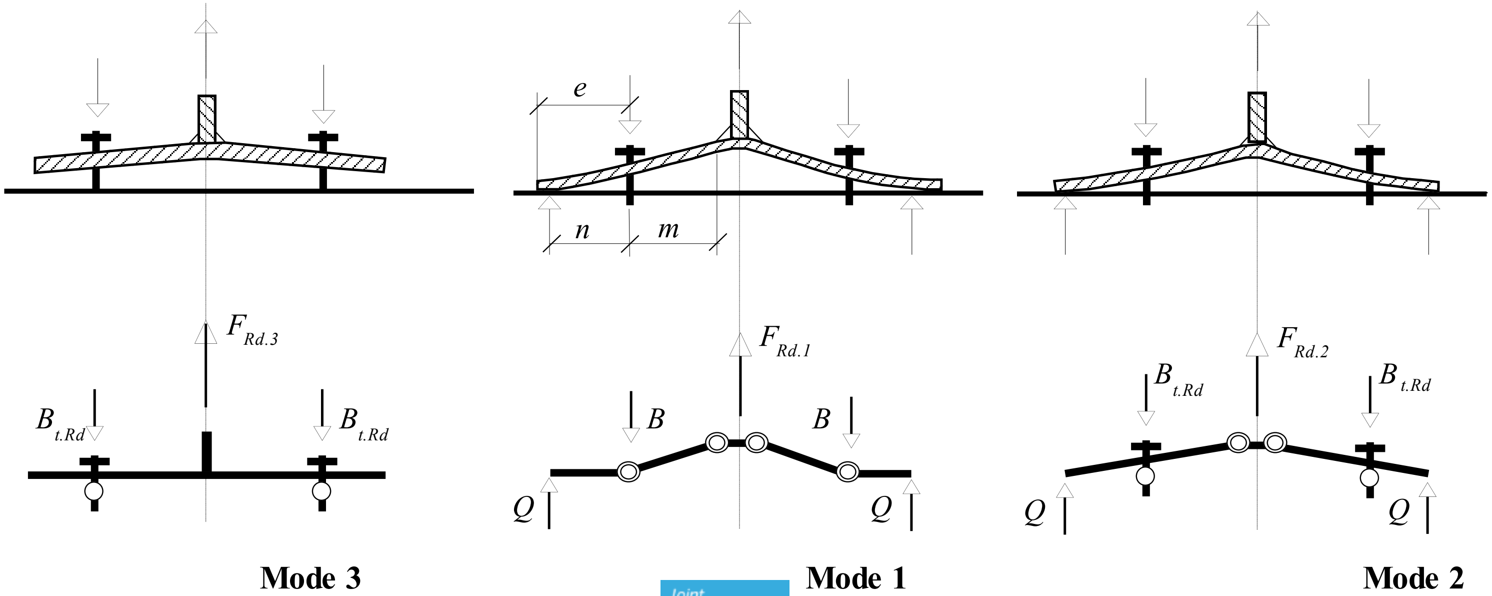




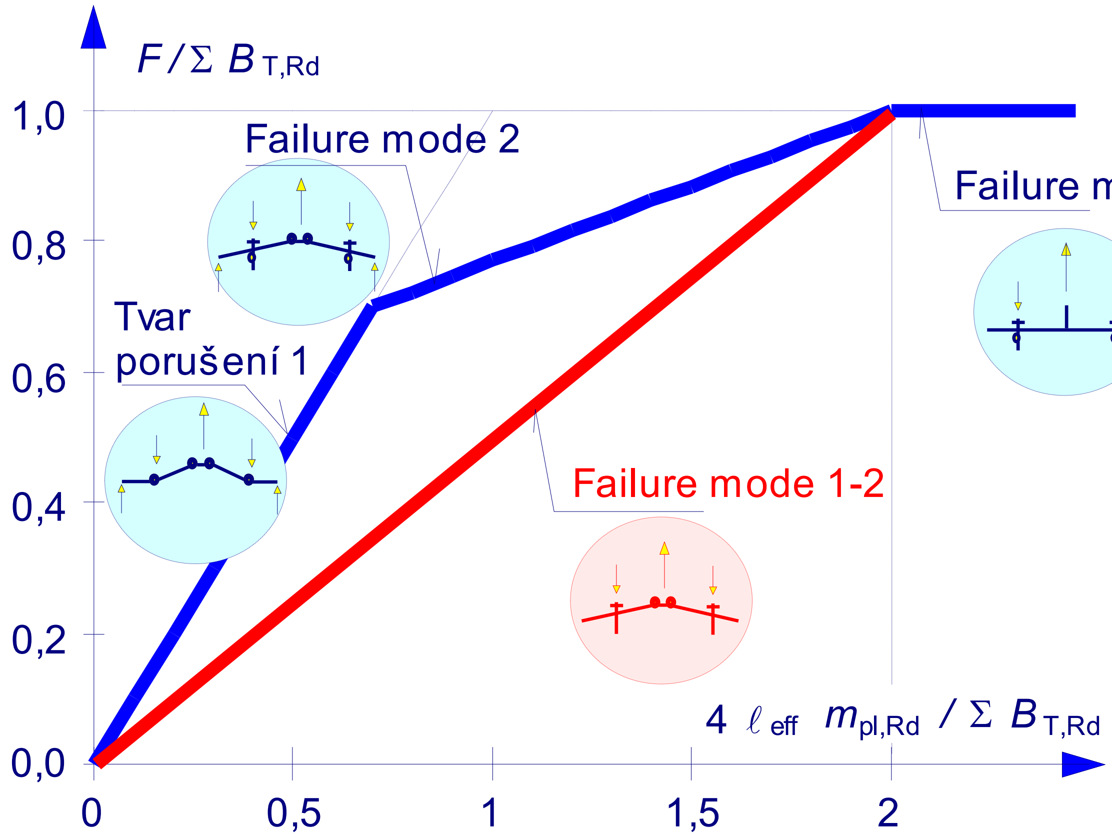
Question of contact

- End plate – contact or no contact
- Base plate – no contact





Failure Mode 1-2

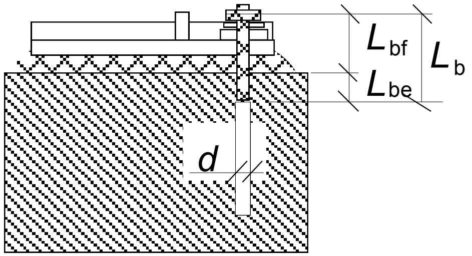


$B_{t.Rd}$ is bolt tensile resistance

$M_{pl,Rd}$ is base plate bending resistance of unique length

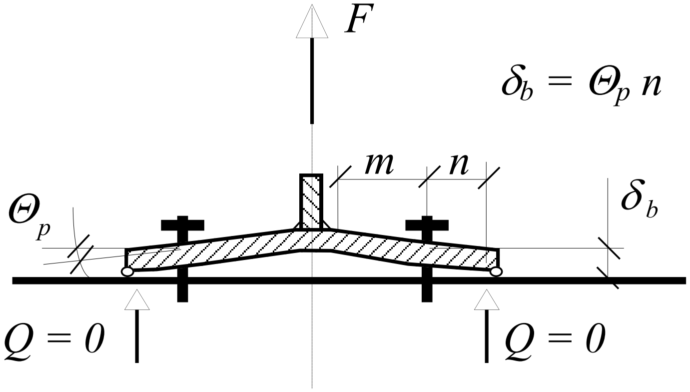
Free length of anchor bolt embedded in concrete

- Bolt effective free length $L_{be} = 8\,d$



- No prying force
  - For $Q = 0$
  - Limiting bolt length

$$L_{b.lim} = \frac{8{,}82 m^3 A_s}{L_{eff}\, t^3} \quad \lessgtr \quad L_b$$



Wald F., Sokol Z., Jaspart J.P.,
Base Plate in Bending and Anchor Bolts in Tension, Heron 53 (2008) 21-50.

Failure mode 1- 2

Design resistance in collapse 1 -2

$$F_{T,1-2,Rd} = \frac{2\, M_{pl,1,Rd}}{m}$$

where m is the lever arm of the anchor bolt

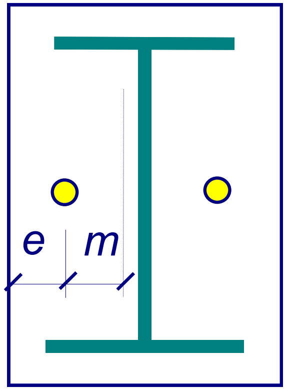
Plastic moment resistance

$$M_{pl,1,Rd} = 0{,}25\, \ell_{eff}\, t_f^{\,2} / \gamma_{M0}$$

For the effective length ℓ_{eff} the yield line method may be used.

Length of effective T stub

Bolts inside the flanges



For bolts inside the section

$$\ell_1 = 2\,\alpha\,m - (4\,m + 1{,}25\,e)$$

$$\ell_2 = 4\,\pi\,m$$

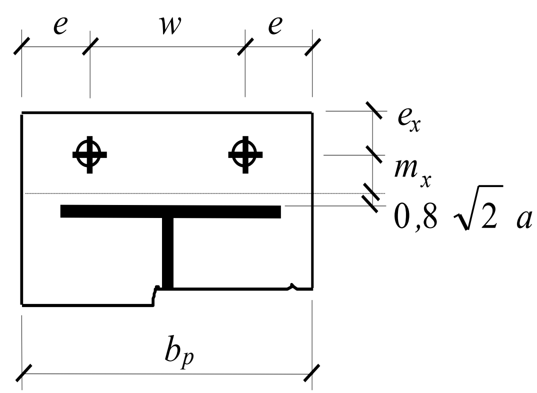
In case of prying

$$\ell_1 = 2\,\alpha\,m - (4\,m + 1{,}25\,e)$$

$$\ell_2 = 2\,\pi\,m$$

Length of effective T stub

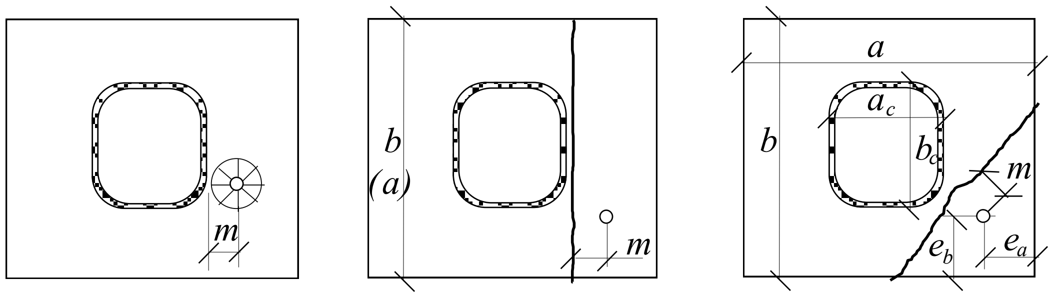
Bolts outside the flanges



ℓ

Prying case	No prying case
$\ell_1 = 4\,\alpha\, m_x + 1{,}25\, e_x$	$\ell_1 = 4\,\alpha\, m_x + 1.25\, e_x$
$\ell_2 = 2\,\pi\, m_x$	$\ell_2 = 2\,\pi\, m_x$
$\ell_3 = 0.5\, b_p$	$\ell_3 = 0.5\, b_p$
$\ell_4 = 0.5\, w + 2\, m_x + 0.625\, e_x$	$\ell_4 = 0.5\, w + 2\, m_x + 0.625\, e_x$
$\ell_5 = e + 2\, m_x + 0.625\, e_x$	$l_5 = e + 2\, m_x + 0.625\, e_x$
$\ell_6 = \pi\, m_x + 2\, e$	$\ell_6 = 2\,\pi\, m_x + 4\, e$
$\ell_7 = \pi\, m_x + w$	$\ell_7 = 2\,(\pi\, m_x + w)$
$\ell_{eff,1} = \min(\ell_1 ; \ell_2 ; \ell_3 ; \ell_4 ; \ell_5 ; \ell_6 ; \ell_7)$	$\ell_{eff,1} = \min(\ell_1 ; \ell_2 ; \ell_3 ; \ell_4 ; \ell_5 ; \ell_6 ; \ell_7)$
$\ell_{eff,2} = \min(\ell_1 ; \ell_2 ; \ell_3 ; \ell_4 ; \ell_5)$	$\ell_{eff,2} = \min(\ell_1 ; \ell_2 ; \ell_3 ; \ell_4 ; \ell_5)$

Length of effective T stub for hollow sections



$$L_{eff.1} = \pi m$$

$$L_{eff.5} = \pi m$$

$$L_{eff.2} = \frac{b}{2}$$

$$L_{eff.4} = \frac{a}{2}$$

$$m = \frac{\sqrt{(a-a_c)^2 + (b-b_c)^2}}{2} - \sqrt{{e_a}^2 + {e_b}^2}$$

$$L_{eff.3} = \frac{\sqrt{{e_a}^2 + {e_b}^2}}{8\, e_a\, e_b}\, m \sqrt{(a-a_c)^2 + (b-b_c)^2}$$

$$L_{eff} = min(L_{eff.1}; L_{eff.2}; L_{eff.3}; L_{eff.4}; L_{eff.5})$$

Wald F., Bouguin V., Sokol Z., Muzeau J.P., *Component Method for Base Plate of RHS*, Proceedings of the Conference Connections in Steel Structures IV: Steel Connections in the New Millenium, Roanoke 2000, IV/8- IV/816.

Heinisuo M., Perttola H., Ronni H., Joints between circular tubes, *Steel Construction*, 5(2) (2012) 101-107.

Stiffness coefficients

of component anchor bolts in tension and base plate in bending

For base plate of thickness *t*

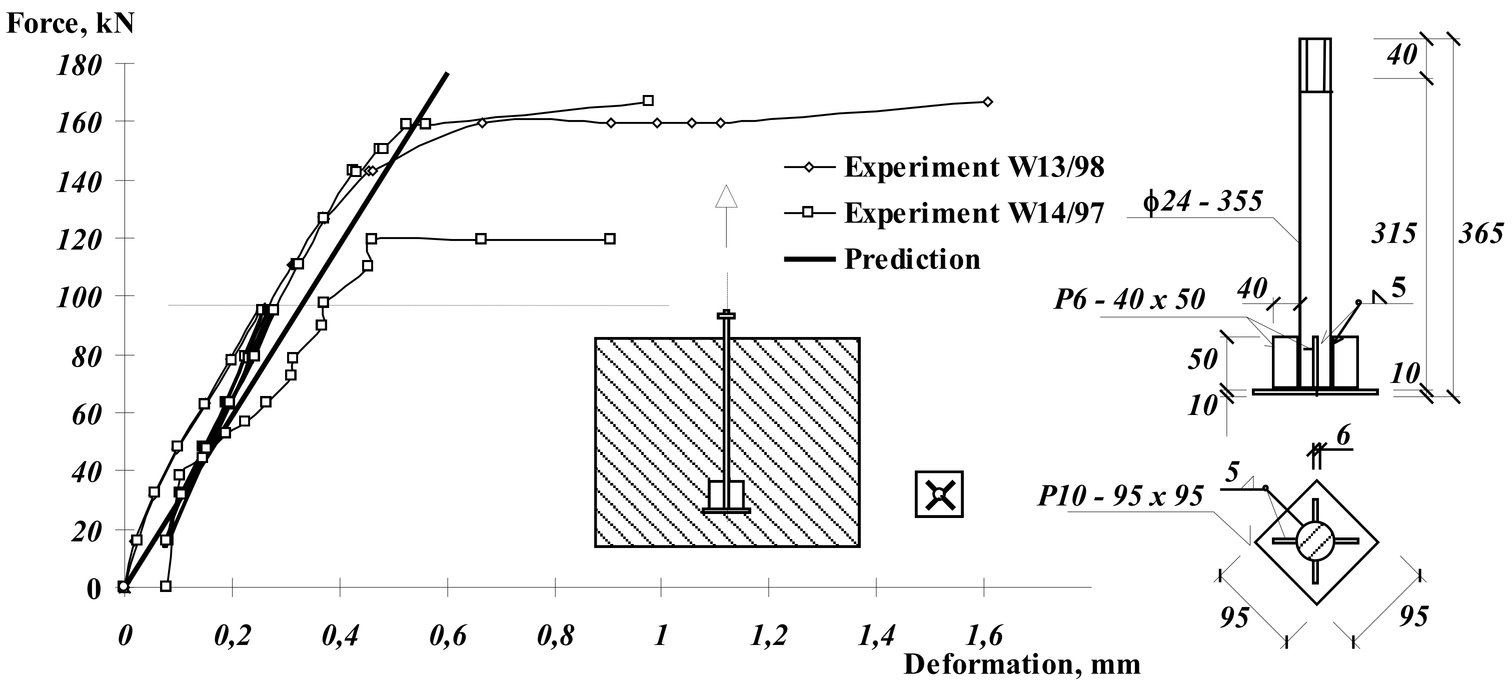
$$k_p = \frac{0{,}425\,\ell_{eff}\,t^3}{m^3}$$

For bolt

$$k_b = 2{,}0\,\frac{A_s}{L_b}$$

Comparison to experiments

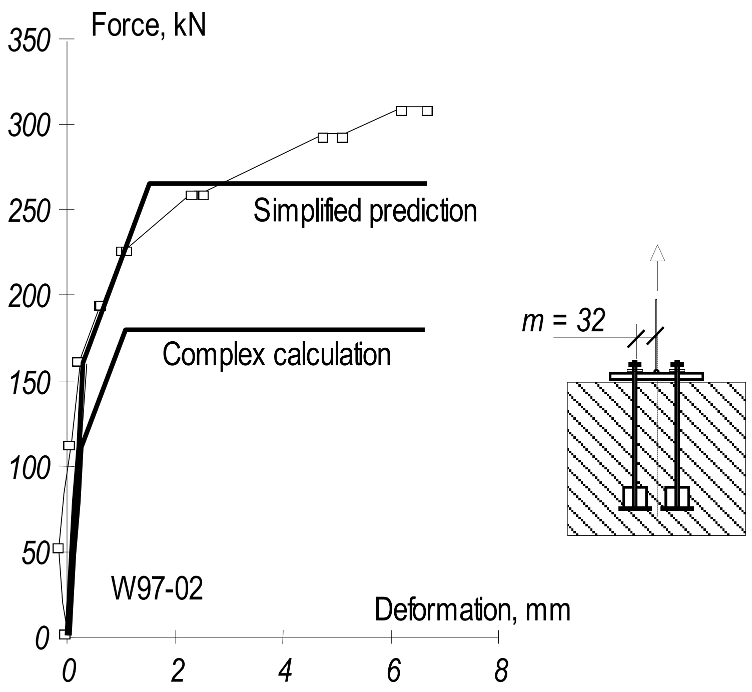
Stiffens and resistance of anchor bolt with header plate

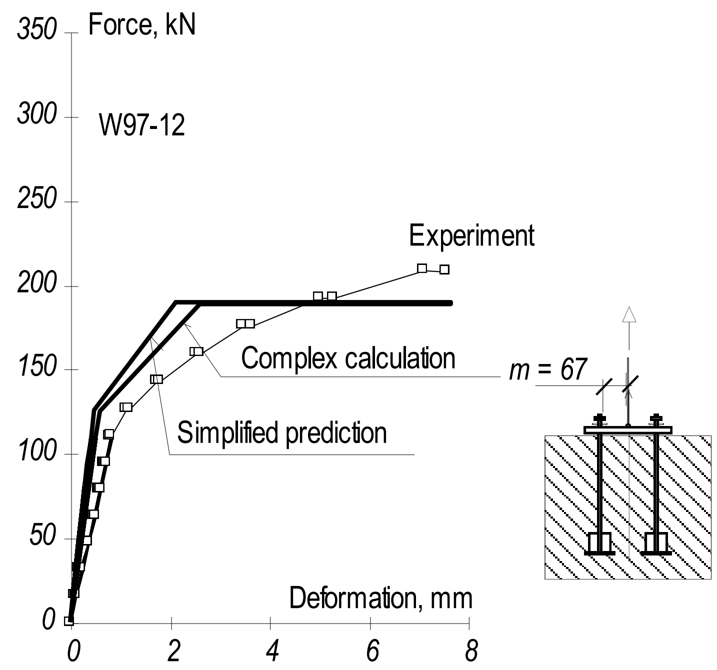


Model anchor bolts for resistence in CEB documents

Eligehausen R., Mallée R., Silva J. F., **Anchorage in Concrete Construction**, Ernst and Sohn Verlag, Darmstadt, 2006, ISBN 978-433-01143-0

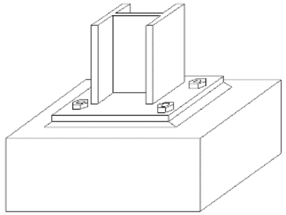
Comparison to experiments





Wald F., Sokol Z., Jaspart J.P.,
Base Plate in Bending and Anchor Bolts in Tension, *Heron* 53 (2008) 21-50.

Scope of the lecture



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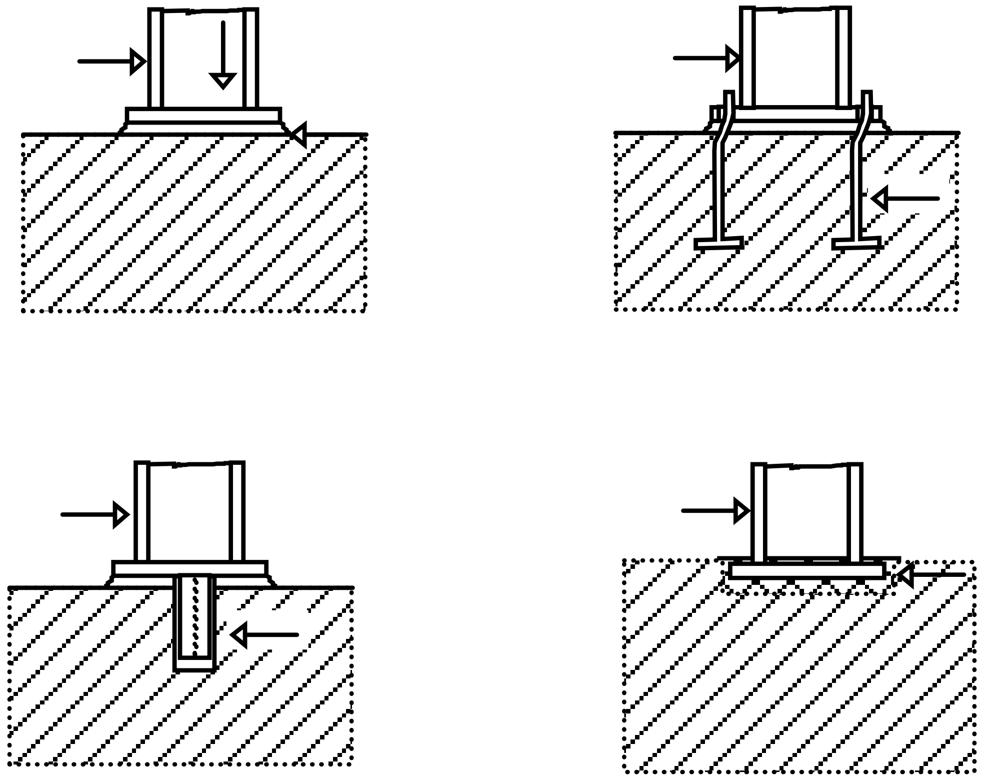
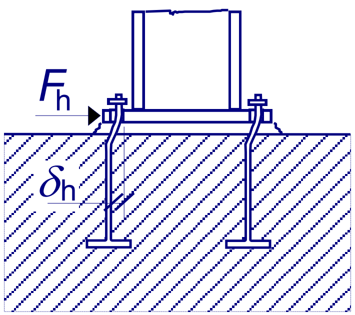
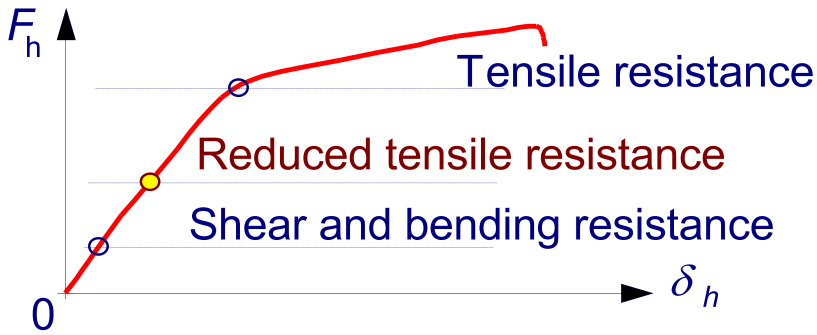
Components in Shear



In EN1993-1-8 cl. 6.2.2(6) - Anchor Bolt in Shear





Simplification by limiting of shear

$$F_{2,vb,Rd} = \frac{\alpha_b \, f_{ub} \, A_s}{\gamma_{Mb}}$$

where f_{ub} is bolt design strength (in range 640 MPa $\geq f_{ub} \geq$ 235 N/mm^2)

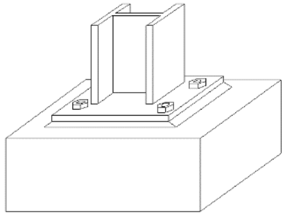
$\alpha_b = 0{,}44 - 0{,}0003\, f_{yb}$

γ_{Mb} is safety factor for bolts

Gresnigt N., Romeijn A., Wald F., Steenhuis M.,

Column Bases in Shear and Normal Force, *Heron* (2008) 87-108.

Scope of the lecture



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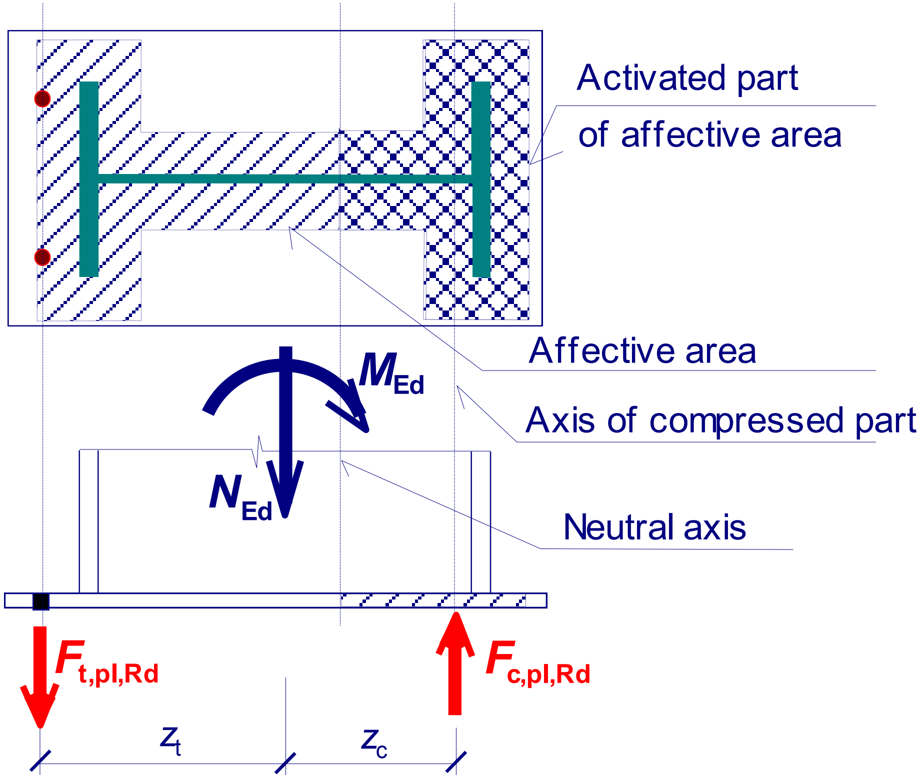
Assembling of components for bending resistance

- Plastic design
- Force equilibrium

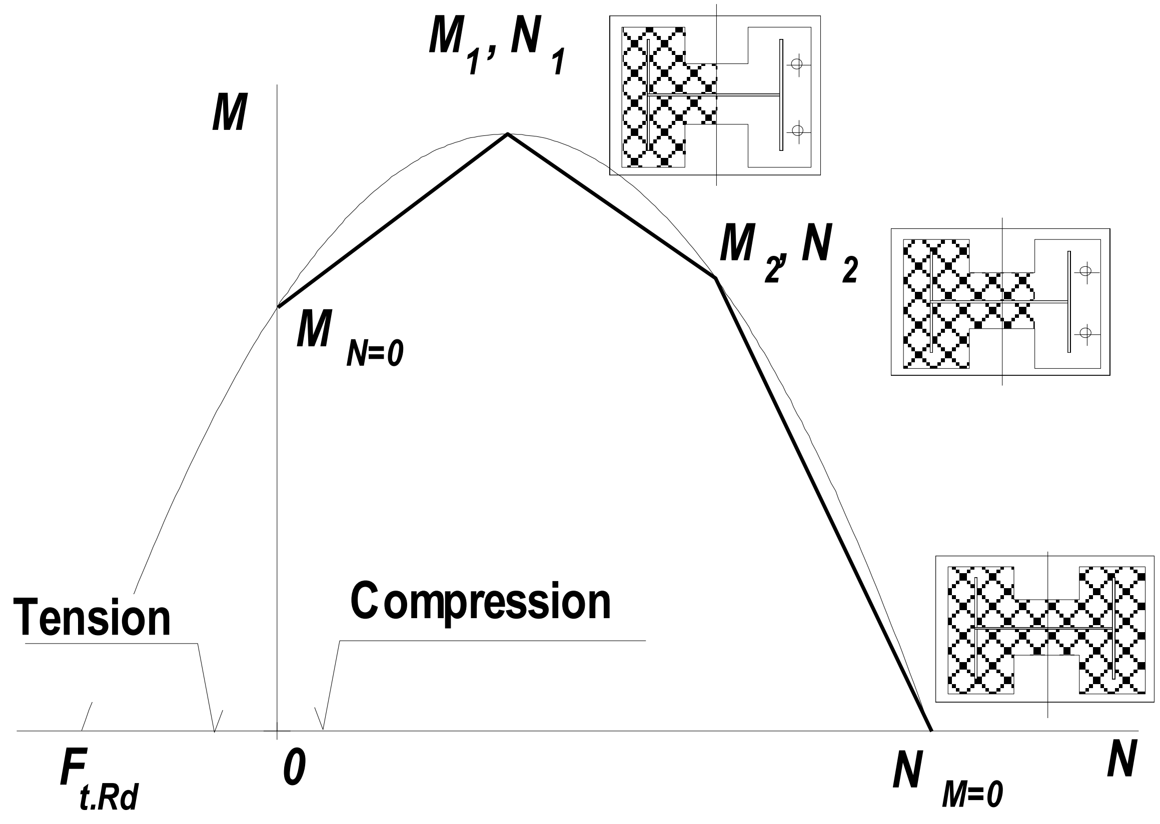
$$\frac{M_{Ed}}{z} - \frac{N_{Ed}\, z_{c,r}}{z} = F_{t,l,Rd}$$

$$\frac{M_{Ed}}{z} + \frac{N_{Ed}\, z_{T,1}}{z} = F_{C,R,Rd}$$



Wald F., Sokol Z., Steenhuis M. and Jaspart J.P.,
Component Method for Steel Column Bases, Heron 53 (2008) 3-20.

M - N interaction diagram



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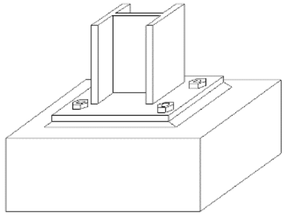

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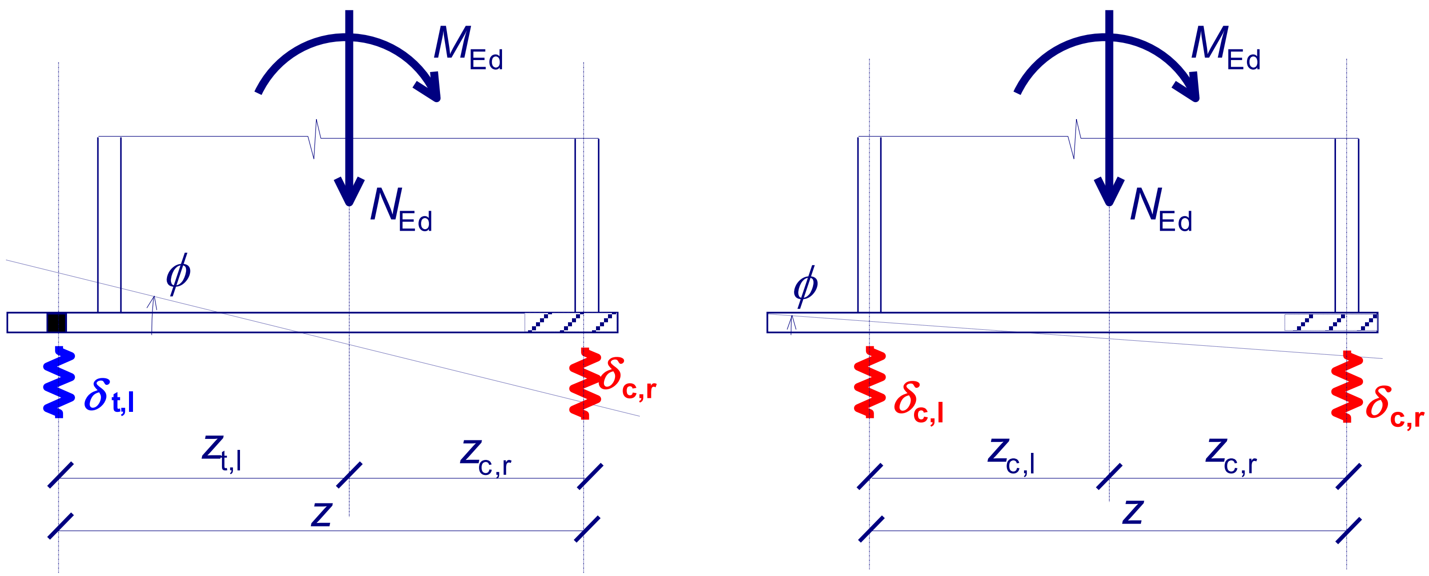
Assembling of components for bending stiffness

From the component deformation stiffness for two cases

- Bolts activated
- Bolts not activated



Wald F., Sokol Z., Steenhuis M. and Jaspart, J.P.,
Component Method for Steel Column Bases, Heron 53 (2008) 3-20.

Stiffness

Simplified contact area round the axes of compressed flange

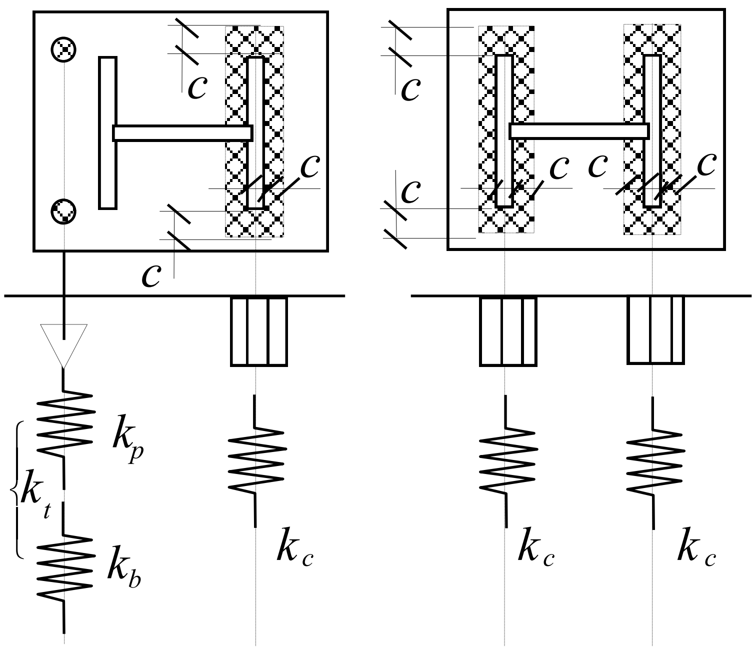
$$M_{Sd} / N_{Sd} = konst. \qquad x_c < N_{Sd} / M_{Sd} < \infty$$

$$S_j = \frac{M_{Sd} / N_{Sd}}{M_{Sd} / N_{Sd} - \alpha} \frac{E\, z^2}{\mu \sum \frac{1}{k_i}}$$

$$\alpha = \frac{z_c\ k_c - z_t\ k_t}{k_c + k_t}$$

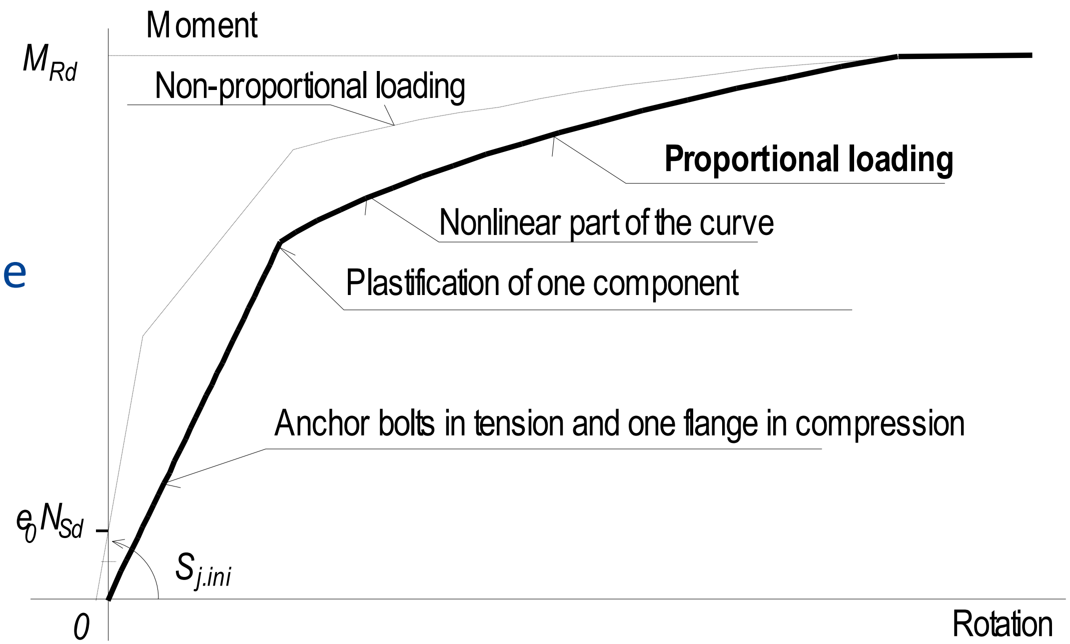
$$\mu = \left(1{,}5\ \gamma\right)^{2{,}7}$$

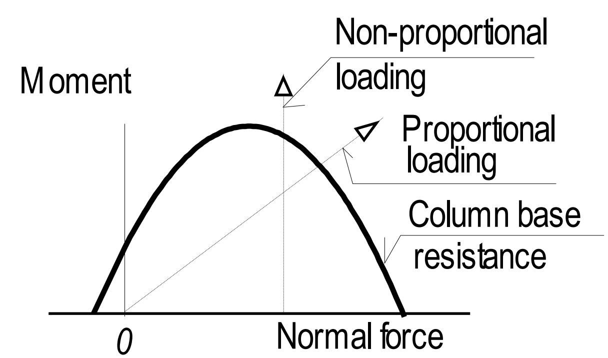
$$\gamma = \frac{1 + \frac{r/2}{M_{Sd} / N_{Sd}}}{M_{Rd} / N_{Sd} + \frac{r/2}{M_{Sd} / N_{Sd}}}$$



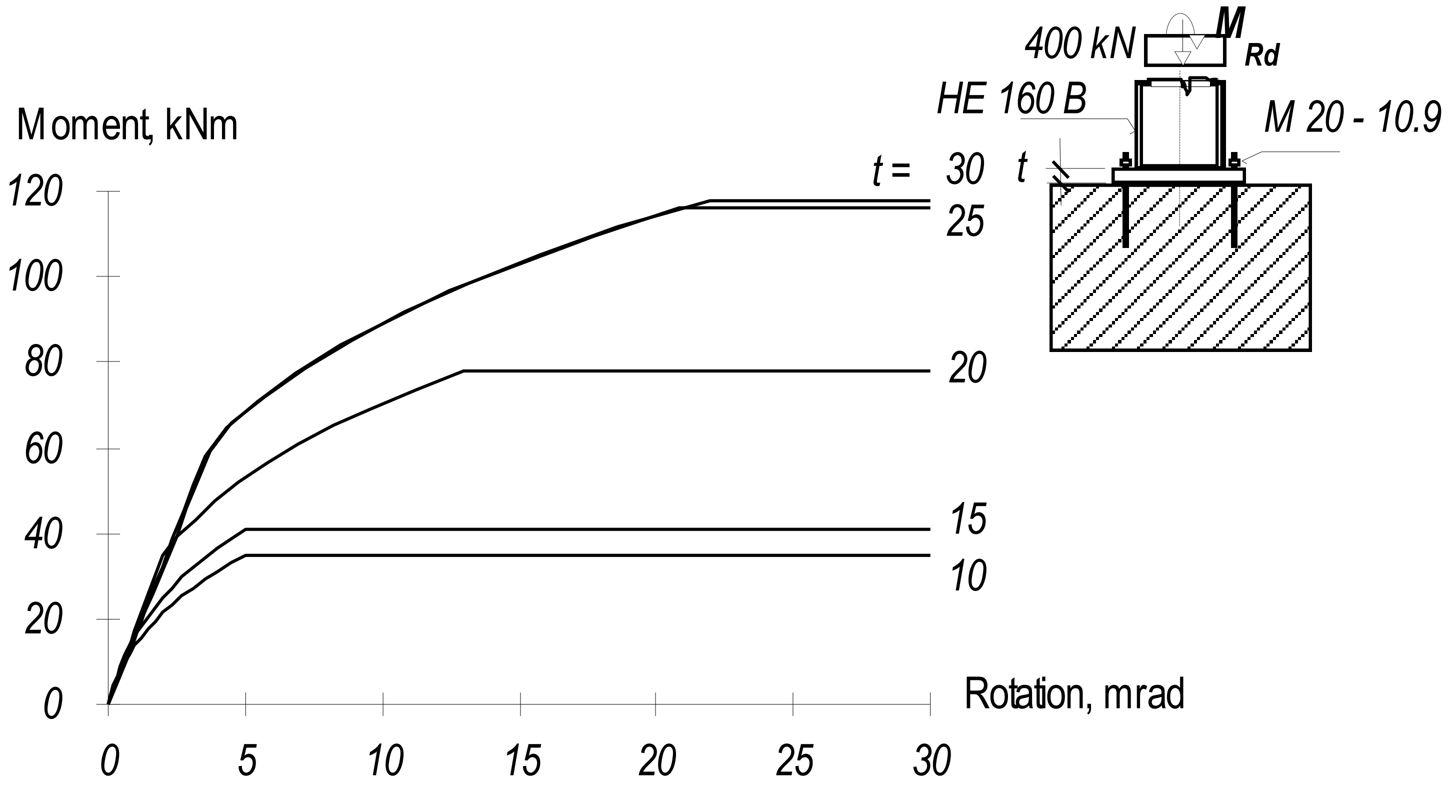
History of loading

- Influence to stiffness
- No influence to resistance

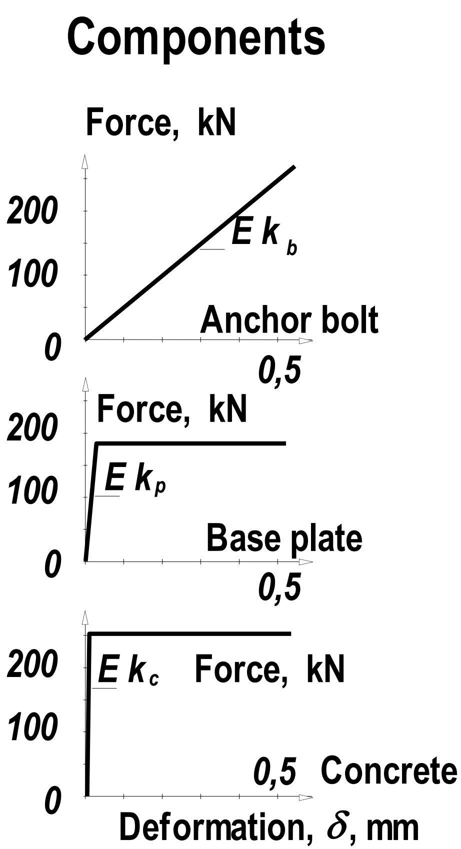


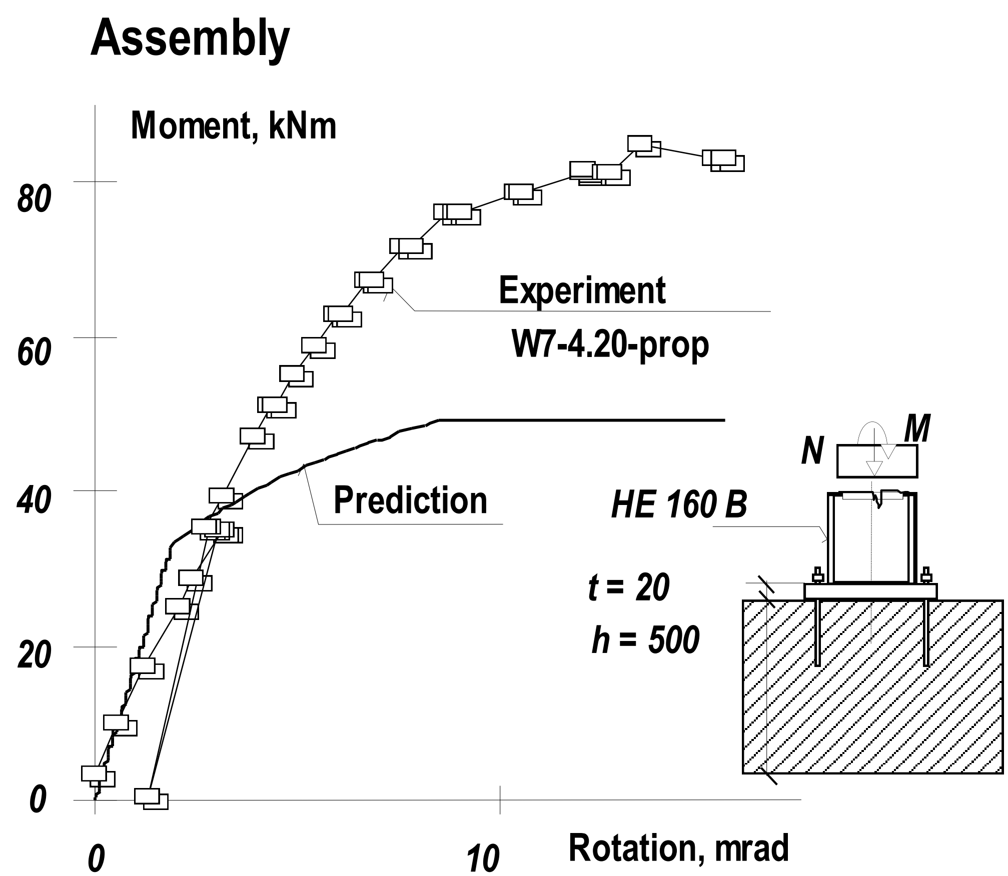


Sensitivity study of **base plate thickness**



Validation on experiment

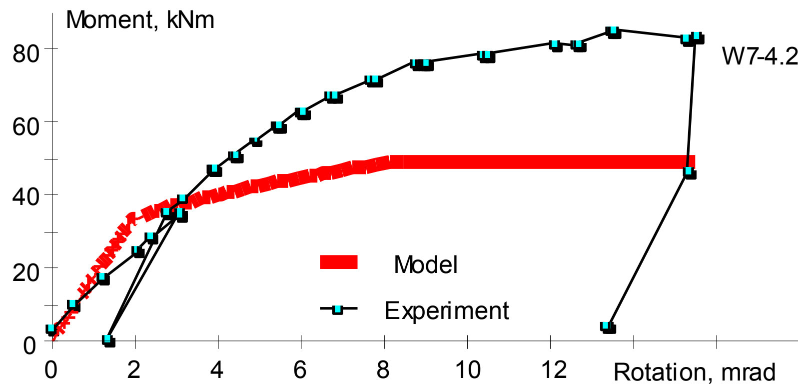


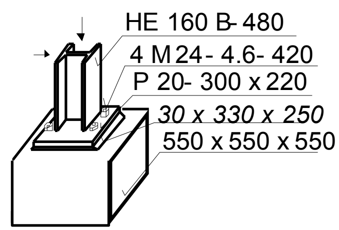


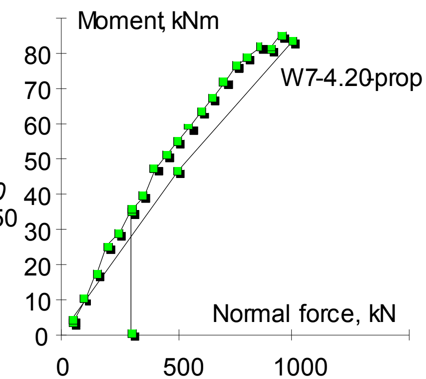
Wald F., Sokol Z., Jaspart J.P.,
Base Plate in Bending and Anchor Bolts in Tension, *Heron* 53 (2008) 21-50.

Validation on experiment

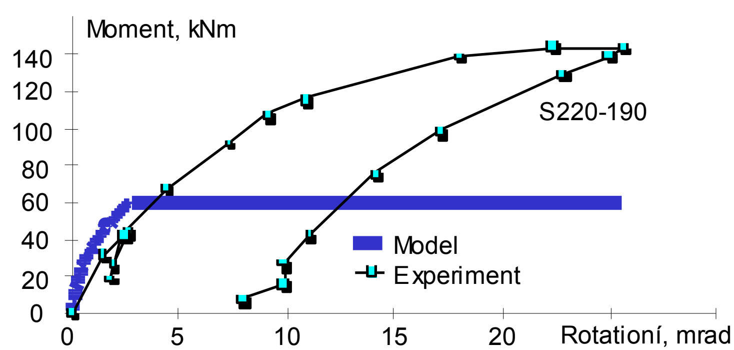
Proportional loading, bolt failure

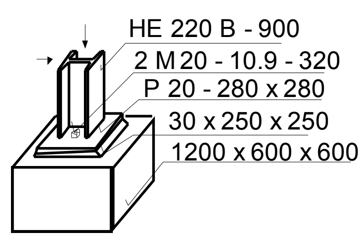


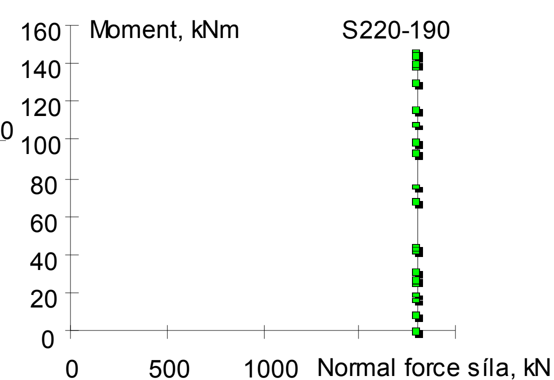




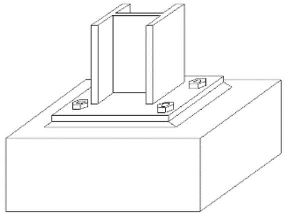
Nonproportional loading, concrete failure







Scope of the lecture



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Classification

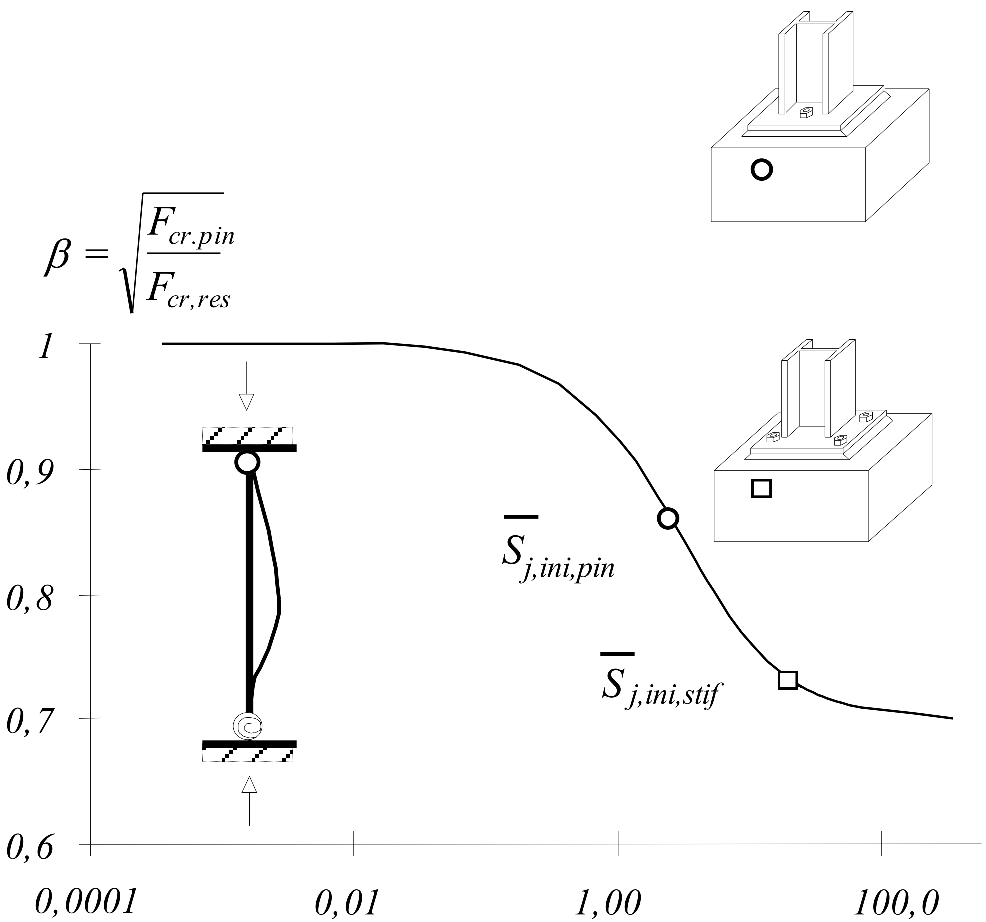
According to stiffness

Asked accuracy of design
5% in resistance
10% in serviceability

Similar to beam-to-column joints

Wald F., Sokol Z., Steenhuis M. and Jaspart, J.P.,
Component Method for Steel Column Bases, Heron 53 (2008) 3-20.

Non-sway frames by resistance



$t = 12\ mm$

$a_1 = b_1 = 280\ mm$

$a = b = 500\ mm$

$h = 1000\ mm$

$M\ 24\ \text{-}420$

$S_{j,ini,pin} = 7\ 100\ kNm\ /\ rad$

$t = 40\ mm$

$a_1 = b_1 = 420\ mm$

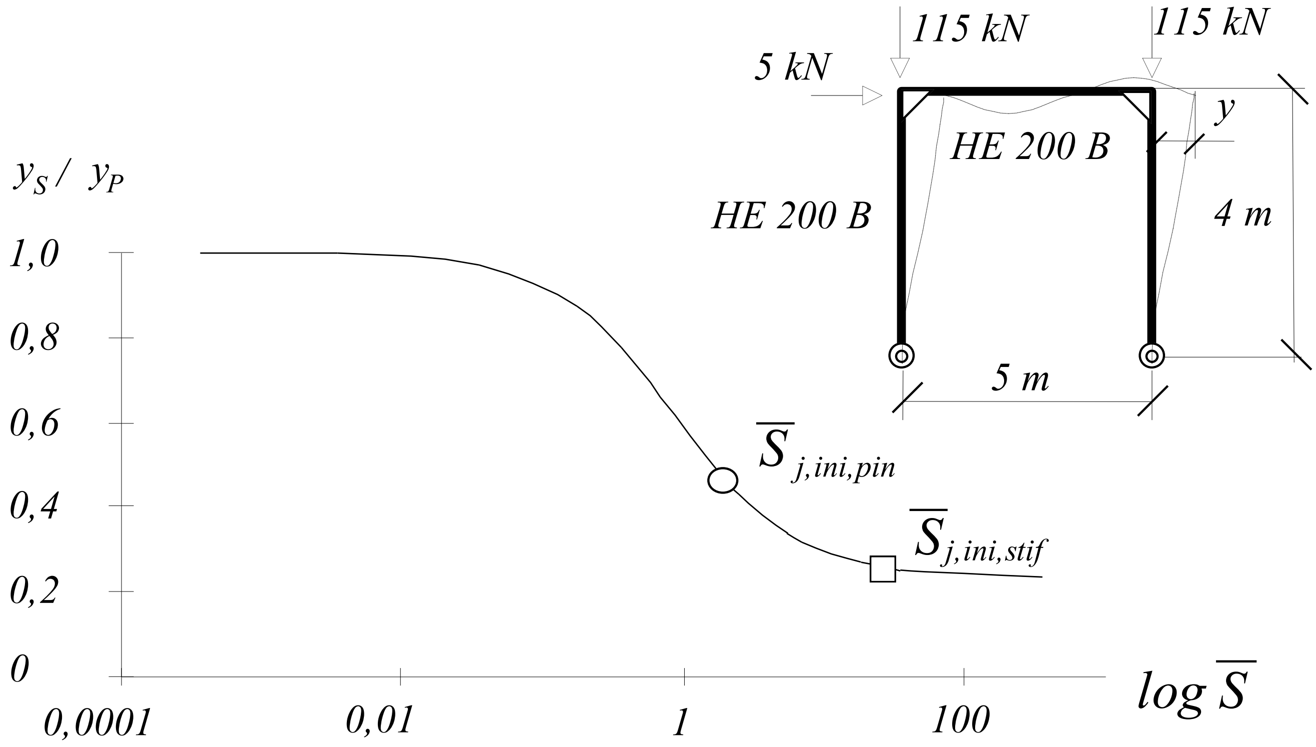
$a = b = 500\ mm$

$h = 1000\ mm$

$M\ 24\ \text{-}420$

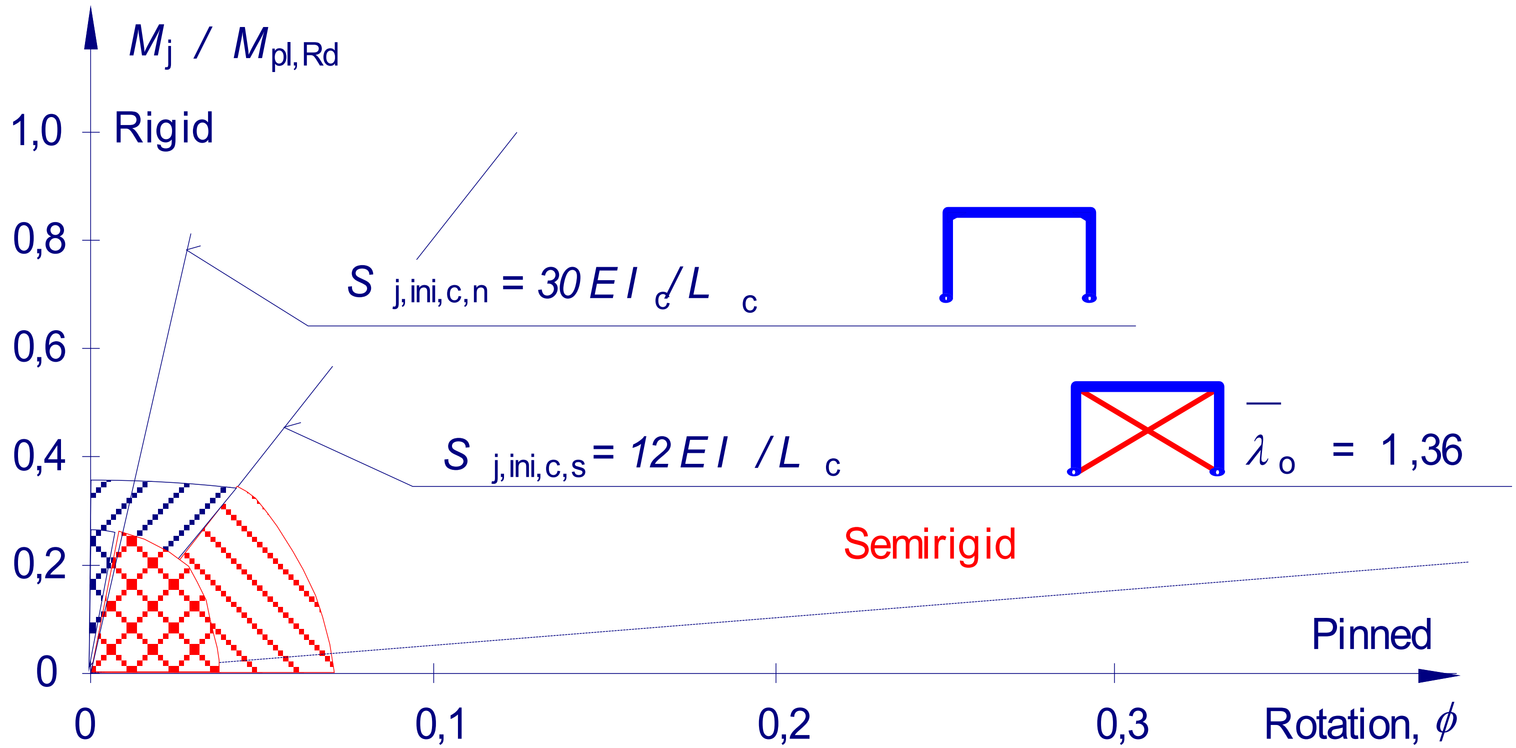
$S_{j,ini,stif} = 74\ 800\ kNm\ /\ rad$

Sway frames for serviceability



Classification based on stiffness

Accuracy - 5% for resistance and 20% for serviceability



Non-sway frames by resistance

For $\bar{\lambda}_o \leq 0{,}5$ $S_{j,ini} \geq 0$

For $0{,}5 < \bar{\lambda}_o < 3{,}93$ $S_{j,ini} \geq 7\,(2\ \ - 1)\,E\,I_c\,/\,L_c$

and for $\bar{\lambda}_o \geq 3{,}93$ $S_{j,ini} \geq 48\,E\,I_c\,/\,L_c$

where is relative stiffness for simple supported column at both ends

For limited stiffness $12\,E\,I_c\,/\,L_c$

For sway frames $S_{j,ini} \geq 30\,E\,I_c\,/\,L_c$

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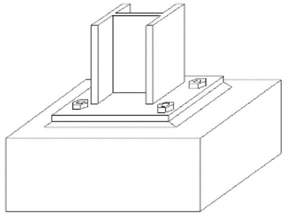

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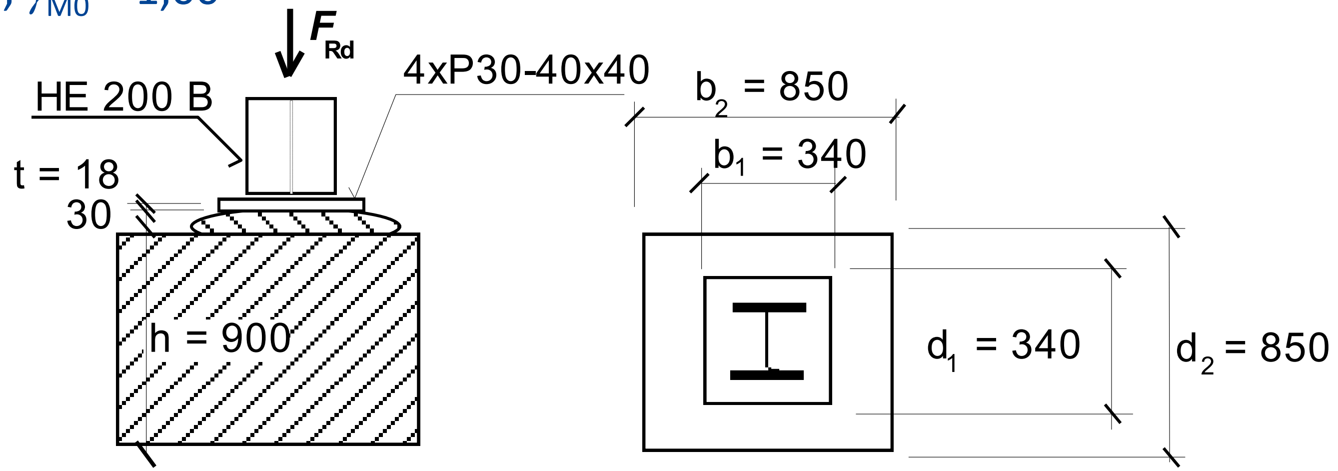
Worked example – **Simple base plate**

Base plate resistance?

- Column HE 200 B
- Concrete block 850 x 850 x 900 mm concrete C 12/15
- Base plate 18 mm, steel S 235
- $\gamma_c = 1{,}50$, $\gamma_{M0} = 1{,}00$



Concrete strength in joint

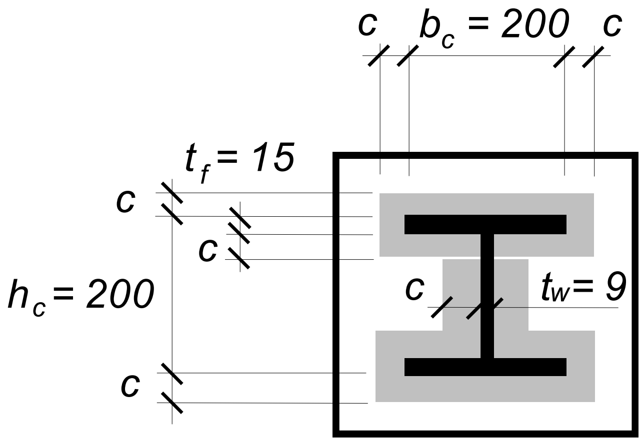
Under the base plate

$$f_{\mathrm{jd}} = \beta_{\mathrm{j}}\, f_{\mathrm{cd}} \sqrt{\frac{A_{\mathrm{c1}}}{A_{\mathrm{c0}}}} = \beta_{\mathrm{j}} \cdot f_{\mathrm{cd}} \sqrt{\frac{b_2\ d_2}{b_1\ d_1}} = \frac{2}{3}\, 12/1{,}5 \sqrt{\frac{850 \cdot 850}{340 \cdot 340}} =$$

$$f_{\mathrm{jd}} = \ = 13{,}3\,\mathrm{MPa} \le 3{,}0 \cdot f_{\mathrm{cd}} = 3{,}0 \cdot 12/1{,}5 = 24\,\mathrm{MPa}$$

Plate effective width

$$c = t\sqrt{\frac{f_y}{3\,f_j\,\gamma_{M0}}} = 18\cdot\sqrt{\frac{235}{3\cdot 13{,}4\cdot 1{,}00}} = 43{,}7\ \text{mm}$$

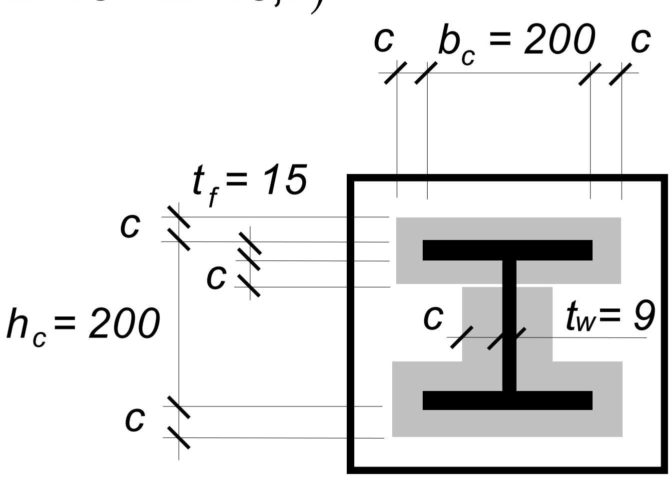


Base plate compression resistance

Effective area under I cross section

$$A_{eff} = \min\left(b; b_c + 2c\right) \cdot \min\left(a; h_f + 2c\right) - \max\left[\min\left(b; b_c + 2c\right) - t_f - 2c; 0\right] \cdot \max\left(h_c - 2t_f - 2c; 0\right)$$

$$A_{eff} = \left(200 + 2 \cdot 43{,}7\right) \cdot \left(200 + 2 \cdot 43{,}7\right) - \left(200 + 2 \cdot 43{,}7 - 9 - 2 \cdot 43{,}7\right) \cdot \left(200 - 2 \cdot 15 - 2 \cdot 43{,}7\right) = 82\,599 - 15\,777 = 66\,722 \text{ mm}^2$$



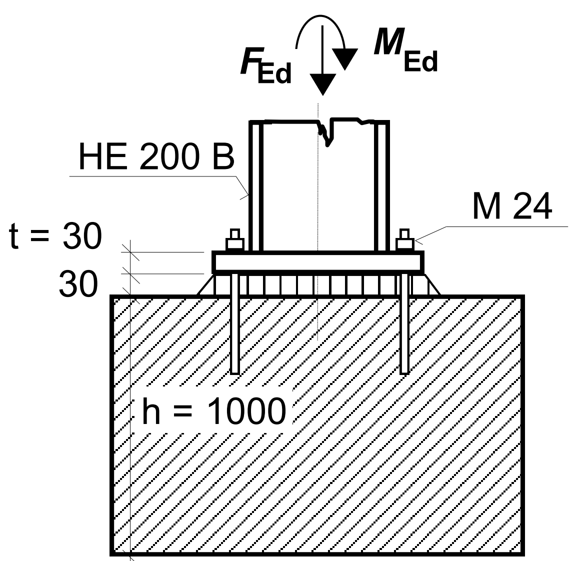
The base plate resistance

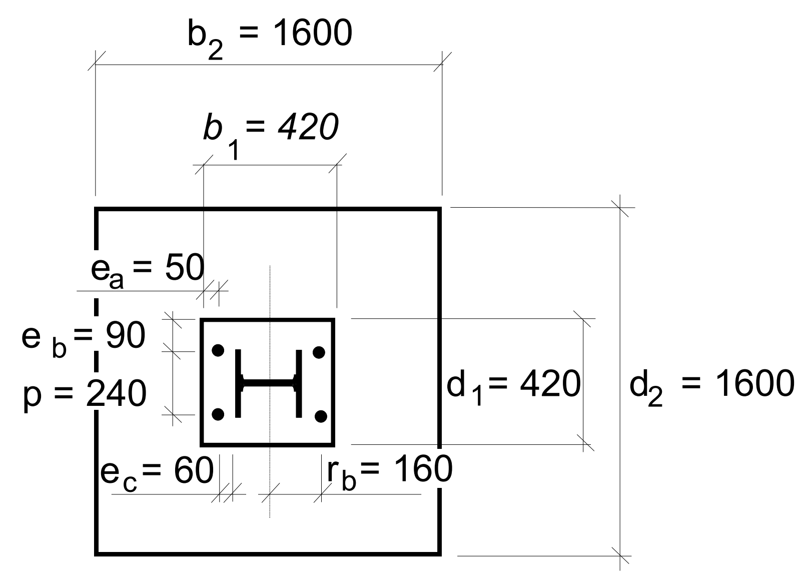
$$N_{Rd} = A_{eff}\, f_{jd} = 66\,722 \cdot 13{,}3 = 887 \cdot 10^3 \text{ N}$$

Work example – rigid base plate

Bending resistance of base plate?

- Column HE 200 B loaded by F_{Ed} = 500 kN
- Concrete block C16/20 of 1 600 x 1 600 x 1000 mm
- Base plate 30 mm from steel S235
- γ_c = 1,50; γ_{M0} = 1,00 and γ_{Mb} = 1,25





Component
Anchor bolt and plate in bending

Bolt resistance

for M 24; A_s = 253 mm^2

$$F_{T,3,Rd} = 2\,B_{t,Rd} = 2 \cdot \frac{0{,}9\,f_{ub}\,A_s}{\gamma_{mb}} =$$

$$= 2 \cdot \frac{0{,}9 \cdot 360 \cdot 353}{1{,}25} = 183{,}0 \cdot 10^3 \text{ N}$$

Component anchor bolt and plate in bending

Bolt distance for weld 6 mm

$$m = 60 - 0{,}8 \cdot a_{wf} \cdot \sqrt{2} = 60 - 0{,}8 \cdot 6 \cdot \sqrt{2} = 53{,}2 \text{ mm}$$

Length of effective T stub

$$\ell_{eff,1} = \min \left\{ \begin{array}{l} 4\, m + 1{,}25\, e_a = 4 \cdot 53{,}2 + 1{,}25 \cdot 50 = 275{,}3 \\ 4\, \pi\, m = 4\, \pi\, 53{,}2 = 668{,}6 \\ 0{,}5\, b = 0{,}5 \cdot 420 = \underline{210} \\ 2\, m + 0{,}625\, e_b + 0{,}5 p = 2 \cdot 53{,}2 + 0{,}625 \cdot 90 + 0{,}5 \cdot 240 = 282{,}7 \\ 2\, m + 0{,}625\, e_b + e_a = 2 \cdot 53{,}2 + 0{,}625 \cdot 90 + 50 = 212{,}7 \\ 2\, \pi\, m + 4\, e_b = 2\, \pi\, 53{,}2 + 4 \cdot 90 = 694{,}2 \\ 2\, \pi\, m + 2\, p = 2\, \pi\, 53{,}2 + 2 \cdot 240 = 814{,}2 \end{array} \right\} =$$

$$\ell_{eff,1} = 210 \text{ mm}$$

T stub resistance

$$F_{T,1\text{-}2,Rd} = \frac{2\, L_{eff,1}\, t^2 f_y}{4\, m\, \gamma_{M0}} = \frac{2 \cdot 210 \cdot 30^2 \cdot 235}{4 \cdot 60 \cdot 1{,}00} = 370{,}0 \cdot 10^3 \text{N}$$

Component
Concrete block and plate in bending

Concrete crushing resistance

$$f_{jd} = \beta_j \, f_{cd} \sqrt{\frac{A_{c1}}{A_{c0}}} = \beta_j \cdot f_{cd} \sqrt{\frac{b_2 \; d_2}{b_1 \; d_1}} = \frac{2}{3} \; 16/1,5 \sqrt{\frac{1420 \cdot 1420}{420 \cdot 420}} =$$

$$= 24,0 \, \text{MPa} \leq 3,0 \cdot f_{cd} = 3,0 \cdot 16/1,5 = 32 \, \text{MPa}$$

Force equilibrium

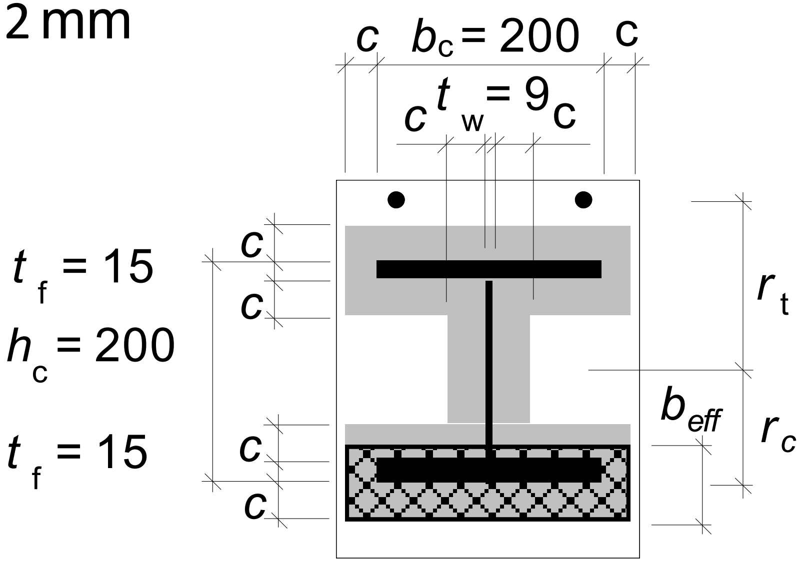
$$F_{Ed} = A_{eff} \; f_j - F_{T,Rd}$$

Contact force for full bolt resistance

$$A_{eff} = \frac{F_{Ed} + F_{Rd,3}}{f_{jd}} = \frac{500 \cdot 10^3 + 183,0 \cdot 10^3}{24,0} = 28\;458 \, \text{mm}^2$$

Effective width of base plate

$$c = t\sqrt{\frac{f_y}{3\, f_{jd}\, \gamma_{M0}}} = 30 \cdot \sqrt{\frac{235}{3 \cdot 24{,}0 \cdot 1{,}00}} = 54{,}2\ \text{mm}$$



The effective width of the area in contact

$$b_{eff} = \frac{A_{eff}}{b_c + 2\,c} = \frac{28\ 458}{200 + 2 \cdot 54{,}2} = 92{,}3\ \text{mm} < t_f + 2\,c = 15 + 2 \cdot 54{,}2 = 123{,}4\ \text{mm}$$

Bending moment resistance

for acting normal force

Lever arm

$$r_c = \frac{h_c}{2} + c - \frac{b_{eff}}{2} = \frac{200}{2} + 54{,}2 - \frac{92{,}3}{2} = 108{,}1\,\text{mm}$$

Bending resistance
for normal force F_{Ed} = 500 kN

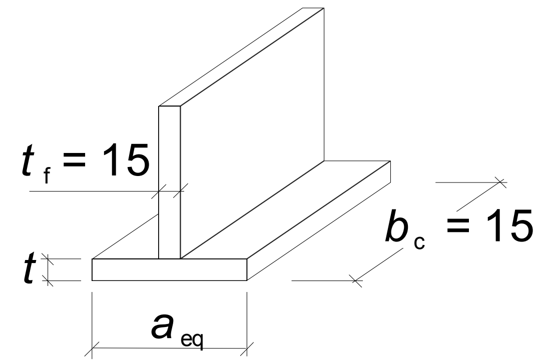
$$M_{Rd} = F_{T,3,Rd}\; r_b + A_{eff}\; f_{jd}\; r_c$$

$$= 183{,}0 \cdot 10^3 \cdot 160 + 28\,458 \cdot 24{,}0 \cdot 108{,}1 =$$

$$= 103{,}1 \cdot 10^6\,\text{Nmm} = 103{,}1\ \text{kNm}$$

Bending stiffness

Stiffness coefficient



- Anchor bolt

$$k_b = 2{,}0\,\frac{A_s}{L_b} = 2{,}0\,\frac{353}{261{,}5} = 2{,}7\ \text{mm}$$

- Base plate

$$k_b = \frac{0{,}425\ L_{b,eff}\ t^3}{m^3} = \frac{0{,}425 \cdot 210 \cdot 30^3}{53{,}2^3} = 16{,}0\ \text{mm}$$

Bending stiffness

Stiffness coefficient

$$k_c = \frac{E_c}{1{,}275\ E_s}\sqrt{a_{eq}\ b_c} = \frac{27\,500}{1{,}275 \cdot 210\,000}\sqrt{90 \cdot 200} = 13{,}8\ \text{mm}$$

Lever arm in tension z_t and in compression z_c to column neutral axes

$$r_t = \frac{h_c}{2} + e_c = \frac{200}{2} + 60 = 160\ \text{mm}$$

For part in tension

$$z_c = \frac{h_c}{2} - \frac{t_f}{2} = \frac{200}{2} - \frac{15}{2} = 92{,}5\ \text{mm}$$

$$k_t = \frac{1}{\dfrac{1}{k_b} + \dfrac{1}{k_p}} = \frac{1}{\dfrac{1}{2{,}7} + \dfrac{1}{16{,}0}} = 2{,}310\ \text{mm}$$

Bending stiffness

Lever arm

$$r = r_t + r_c = 160 + 92{,}5 = 252{,}5\,\text{mm}$$

$$a = \frac{k_c\, r_c - k_t\, r_t}{k_c + k_t} = \frac{13{,}8 \cdot 92{,}5 - 2{,}3 \cdot 160}{13{,}8 + 2{,}3} = 56{,}4\,\text{mm}$$

Eccentricity

$$e = \frac{M_{Rd}}{F_{Ed}} = \frac{103{,}1 \cdot 10^6}{500 \cdot 10^3} = 206{,}2\,\text{mm}$$

Bending stiffness

$$S_{j,ini} = \frac{e}{e+a}\,\frac{E_s\, r^2}{\mu \sum_i \frac{1}{k_i}} = \frac{206{,}2}{206{,}2 + 56{,}4} \cdot \frac{210\,000 \cdot 252{,}5^2}{1 \cdot \left(\frac{1}{2{,}31} + \frac{1}{13{,}78}\right)} =$$

$$= 20{,}799\,\text{Nmm/rad} = 20\,799\,\text{kNm/rad}$$

Classification

Bending stiffness for column HE 200 B of length $L_c = 4{,}0$ m

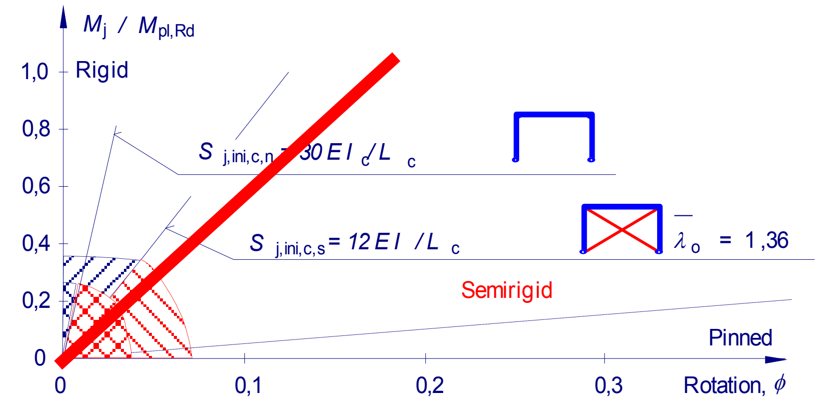
$$\bar{S}_{j,ini} = S_{j,ini} \frac{L_c}{E_s I_c} = 20{,}799 \cdot 10^9 \frac{4\,000}{210\,000 \cdot 56{,}96 \cdot 10^6} = 6{,}96$$

the base plate is **semi-rigid** for braced frames

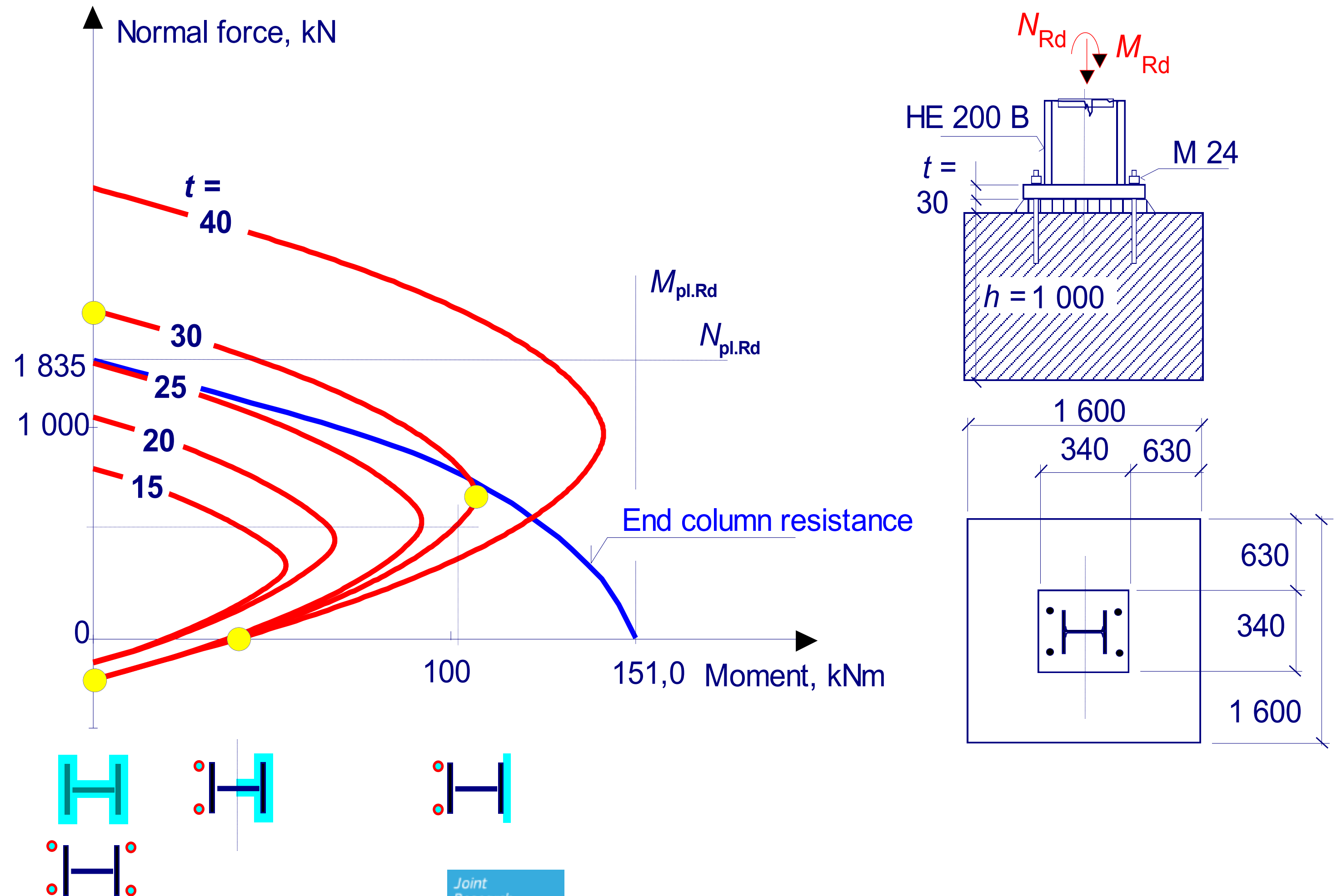
$$\bar{S}_{j,ini} = 6{,}96 < 12 = \bar{S}_{j,ini,EC3,n}$$

and also for unbraced frames

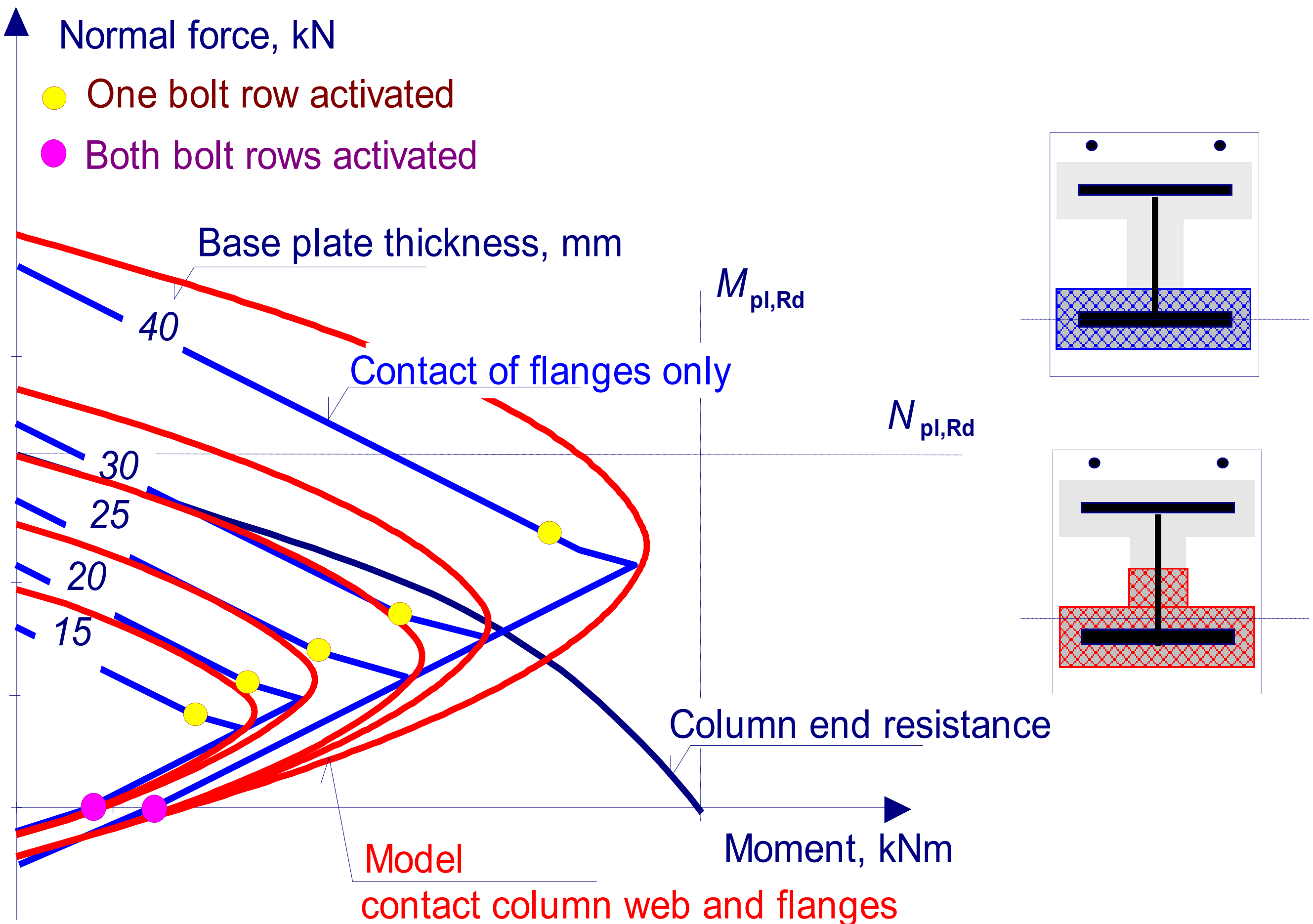
$$\bar{S}_{j,ini} = 6{,}96 < 30 = \bar{S}_{j,ini,EC3,s}$$



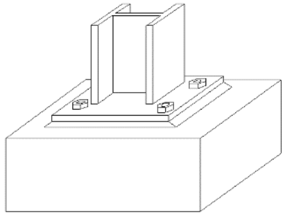
Base plate M-N interaction diagram



Influence of contact in column web



Scope of the lecture



Bolts

- General
- Design resistance of individual fasteners
  - Non-preloading bolts
  - Slotted holes
- Design for block tearing
- Worked example
- Summary

Welds

- General
- Fillet weld
  - Design model
  - Design example
  - Welding to flexible plate
- Summary

Column bases

- Basis of design
- Components
  - Base plate and concrete in compression
  - Base plate in bending and bolt in tension
  - Anchor bolt in shear
- Assembly
  - Resistance
  - Stiffness
- Classification
- Worked examples
- Summary

Summary

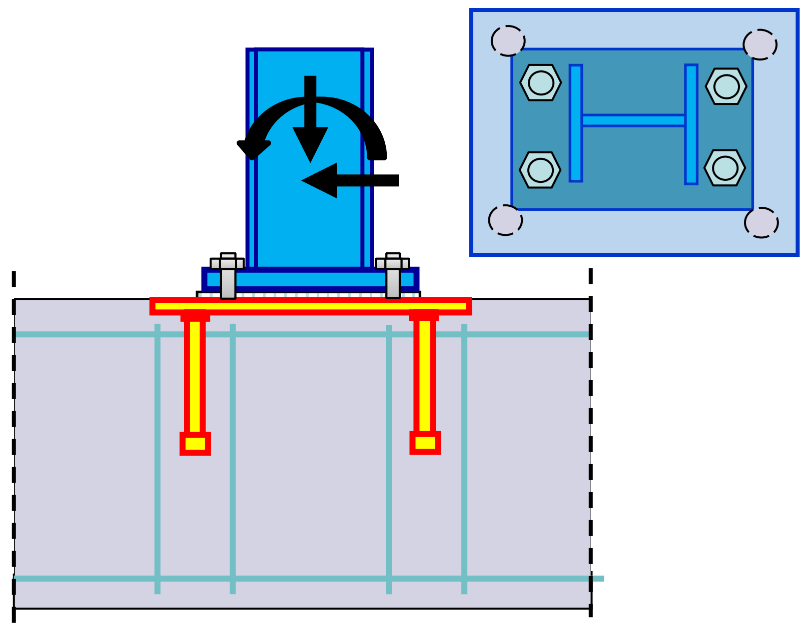
- Component method
  - Good accuracy of prediction

- In EN 1993-1-8 in 6 closures

- Plastic distribution of forces for resistance
  - Concrete 3D strength
  - Effective rigid area under flexible base plate

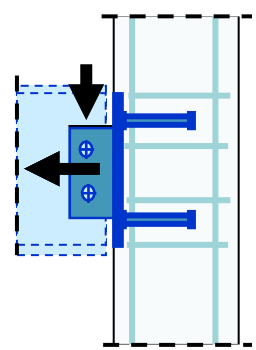
- Questions for next edition
  - Embedded columns?
  - Column bases with base plate and anchor plate?
  - Advanced models?

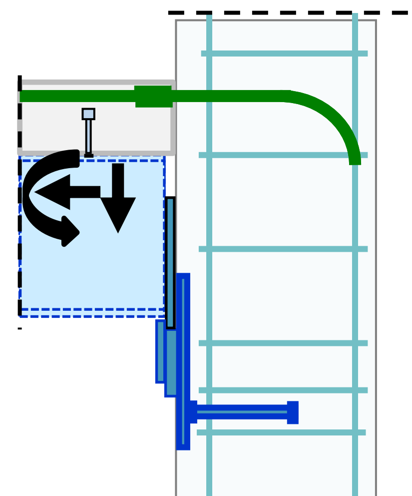
Component based model of column base with base plate and **anchor plate**



RFCS project INFASO+

- Design Manual I
- Design Manual II
- Software toll





http://www.steelconstruct.com/site

http://steel.fsv.cvut.cz/infaso

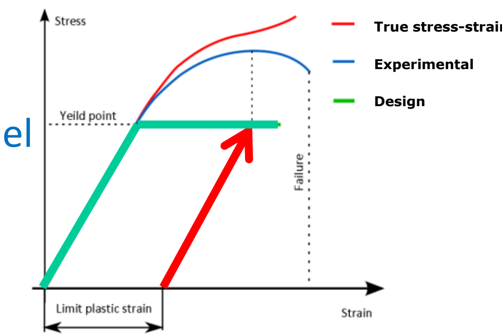
Component based FEM

- Advanced design model based on FEM
- Components integrated FEM
  - Welds
  - Bolts
  - Compressed plates

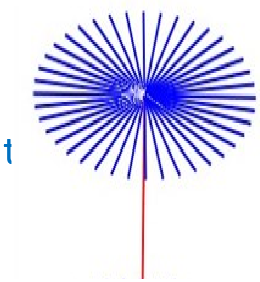
Allows for column bases

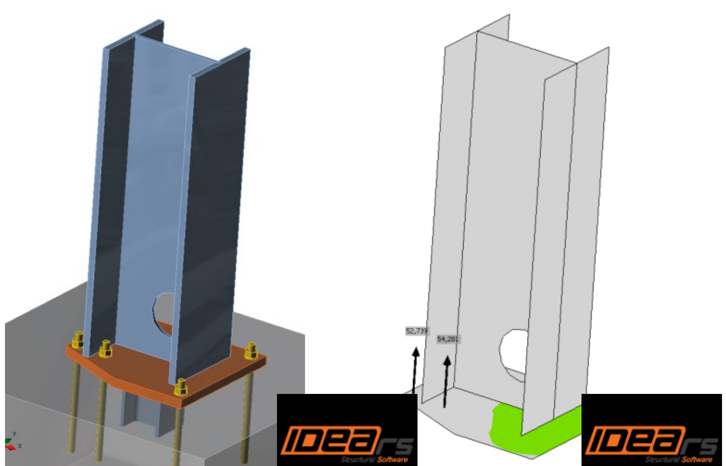
- Generally loaded by *N* and M_y, M_z
- Irregular shape of base plate
- Arbitrary positions of anchors
- Opening in column web

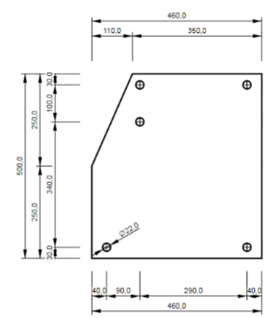
Material nonlinear design model of plate



Fan model of anchor bolt







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European Commission

Eurocodes
Background and Applications

Design of **Steel Buildings**
with worked examples

16-17 October 2014
Brussels, Belgium

Thank you
for your attention

František Wald
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