

## STEEL STRUCTURES - DESIGN OF THE JOINTS

### Deductions for fastener holes

#### General

For member design, the deductions for holes should be made according to EC3-1-1.

For a symmetrically connected member in tension,  $N_{t,Rd}$ , is given by

$$N_{t,Rd} = \min \left\{ N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} ; N_{u,Rd} = 0,9 \frac{A_{net} \cdot f_u}{\gamma_{M2}} \right\}$$

#### Block tearing resistance

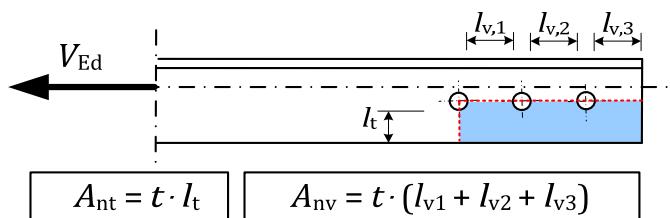
Failure along (i) the shear face(s) and  
(ii) the tension face of the hole group

- For a symmetric bolt group subject to concentric loading,

$$V_{eff,Rd} = 1,0 A_{nt} \cdot \frac{f_u}{\gamma_{M2}} + A_{nv} \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}}$$

- otherwise  $V_{eff,Rd} = 0,5 A_{nt} \cdot \frac{f_u}{\gamma_{M2}} + A_{nv} \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}}$

where  $A_{nt}$  and  $A_{nv}$  are the net areas subject to tension and shear, respectively.



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### Angles in tension bolted in one leg

A single angle in tension connected by a single row of bolts in one leg may be verified assuming that it is concentrically loaded and taking  $N_{u,Rd}$  given by:

- with 1 bolt  $N_{u,Rd} = (2e_2 - d_0) \cdot t \cdot f_u / \gamma_{M2}$

- with 2 or more bolts  $N_{u,Rd} = \beta \cdot A_{net} \cdot f_u / \gamma_{M2}$  ( $\beta$  in Table below)

$p_1/d_0$	$\leq 2,5$	$2,5 \text{ a } 5,0$	$\geq 5,0$	Detail of connection
2 bolts	$\beta = 0,4$			
3 or more bolts	$\beta = 0,5$		$\beta = 0,7$	

linear interpolation

## STEEL STRUCTURES - DESIGN OF THE JOINTS

### Example 3

$$N_{Ed} = 538 \text{ kN}$$

2 angles L 90 × 8 and gusset plate  $t_g = 12\text{mm}$   
– steel S355; 3 bolts M20 (grade 8.8)

$$\begin{aligned} d_0 &= 22 \text{ mm} \rightarrow A_{net} = (1390 - 22 \times 8) = 1214 \text{ mm}^2 \\ \text{With } p_1 &= 80 \text{ mm} \\ p_1/d_0 &= 3,64 \rightarrow \beta = 0,59 \end{aligned}$$

- Verification of the block tearing resistance

$$A_{nt} = 8,0 \times (40 - 22/2) = 232 \text{ mm}^2$$

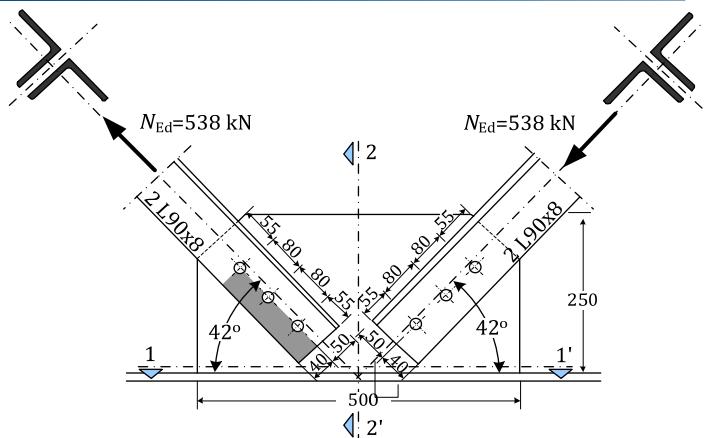
$$A_{nv} = 8,0 \times [(2 \times 80 + 55) - 2,5 \times 22] = 1280 \text{ mm}^2$$

$$V_{eff,Rd,angle} = 0,5 \times 232 \times \frac{0,470}{1,25} + 1280 \times \frac{0,355/\sqrt{3}}{1,0} = 306,0 \text{ kN/angle}$$

$$V_{eff,Rd} = 2 \times 306 = 612 \text{ kN} \geq N_{Ed} = 538 \text{ kN} \quad \checkmark$$

- Tension resistance of the 2 angles L 90 × 8, back-to-back bolted on one edge

$$N_{t,Rd} = \min \left\{ N_{pl,Rd} = \frac{2 \times 13,9 \times 35,5}{1,0} = 987; N_{u,Rd} = 0,59 \times \frac{2 \times 12,14 \times 47}{1,25} = 540 \right\} = 540 \text{ kN} \geq N_{Ed} = 538 \text{ kN} \quad \checkmark$$



## STEEL STRUCTURES - DESIGN OF THE JOINTS

- Verification of "global" (gusset) cross-sectional resistances ( $t_g = 12\text{mm}$ )

### Section 1-1'

$$N_{Ed} = M_{Ed} = 0$$

$$V_{Ed} = 2 \times 538 \times \cos(42^\circ) = 800 \text{ kN}$$

$$\begin{aligned} V_{pl,Rd} &= (500 \times 12) \times \frac{0,355/\sqrt{3}}{1,0} \\ &= 1230 \text{ kN} \geq V_{Ed} = 800 \text{ kN} \quad \checkmark \end{aligned}$$

- Section 2-2' (simplified procedure, without considering any contribution of the top chord and neglecting the eccentricity of the force transmitted by the diagonal)

$$N_{Ed} = 538 \times \cos(42^\circ) = 400 \text{ kN}$$

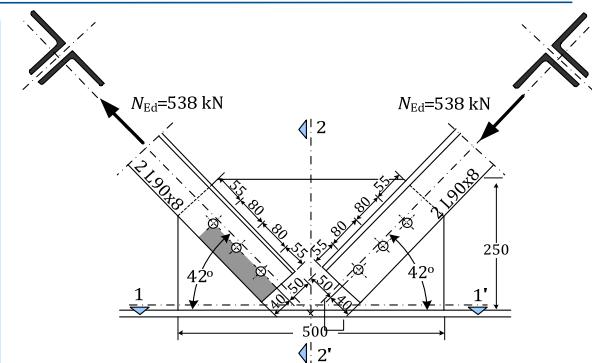
$$\sigma_{Ed} = \frac{400 \times 10^3}{250 \times 12} = 133,3 \text{ MPa}$$

$$V_{Ed} = 538 \times \sin(42^\circ) = 360 \text{ kN}$$

$$\tau_{Ed} = \frac{360 \times 10^3}{250 \times 12} = 120,0 \text{ MPa}$$

von Mises criterium

$$\sigma_{C,Ed} = \sqrt{\sigma_{Ed}^2 + 3 \tau_{Ed}^2} = 247 \text{ MPa} \approx 70\% \frac{f_y}{\gamma_{M0}} \quad \checkmark$$



## STEEL STRUCTURES - DESIGN OF THE JOINTS

- Verification of “local” effective (gusset) cross-sectional resistances

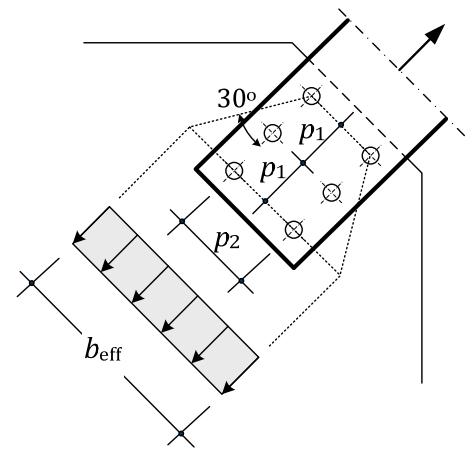
(Whitmore model)

$$b_{\text{eff}} = p_2 + 2(n_b - 1) \cdot p_1 \cdot \tan(30^\circ) = \\ = 0 + 2 \times (3 - 1) \times 80 \times \tan(30^\circ) = 184,8 \text{ mm}$$

$$A = b_{\text{eff}} \cdot t_g = 184,8 \times 12 = 2217 \text{ mm}^2$$

$$A_{\text{net}} = (b_{\text{eff}} - d_0) \cdot t_g = (184,8 - 22) \times 12 = 1953 \text{ mm}^2$$

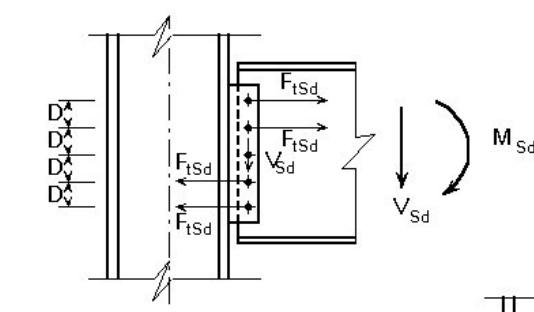
$$N_{t,Rd} = \min \left\{ \begin{array}{l} N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = 787 \text{ kN} \\ N_{u,Rd} = 0,9 \frac{A_{\text{net}} \cdot f_u}{\gamma_{M2}} = 661 \text{ kN} \end{array} \right\} = 661 \text{ kN} \geq N_{Ed} = 538 \text{ kN} \quad \checkmark$$



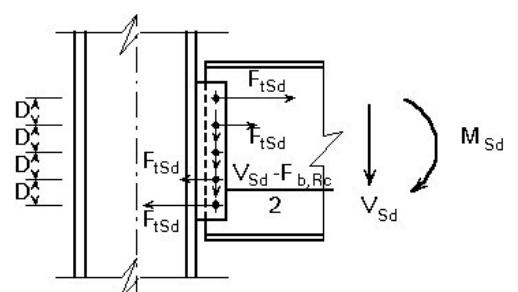
- Design of the **welds** that connect the gusset plate to the beam, with a length of 2x 500 mm long,

$$F_{w,Ed} = 2 \times 538 \times \cos(42^\circ) = 800 \text{ kN} \rightarrow \frac{0,470 / \sqrt{3}}{0,90 \times 1,25} \cdot (2a) \geq \frac{800}{500} \rightarrow a \geq 3,3 \text{ mm} \rightarrow a = 4 \text{ mm}$$

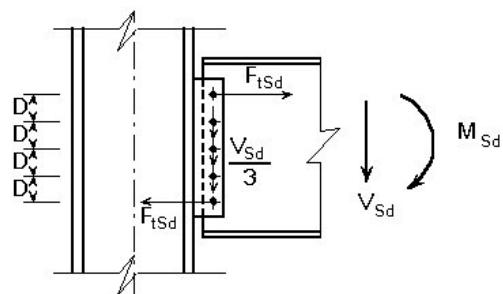
## STEEL STRUCTURES - DESIGN OF THE JOINTS



1 bolt for V: 4 bolts for M



1 bolt for V: 2 bolts for V and M



3 bolts for V: 2 bolts for M

✓ Possible distributions of bolt forces for a beam-column semi-rigid connection

## STEEL STRUCTURES - DESIGN OF THE JOINTS

### Example 4

$$V_{Ed} = 350 \text{ kN} ; M_{Ed} = 12,75 \text{ kN m}$$

Profiles and gusset plate – steel S355

- Design of the bolts

$$\text{Pre-design: } 4 \text{ M20 cl.8.8} \rightarrow F_{Ed} = 350 \text{ kN} < 4 \times 94,1 = 376,4 \text{ kN}$$

$$\text{Using a compact detailing } e_1 = 40 \text{ mm}; e_2 = 30 \text{ mm}; p_1 = (350-80)/3 = 90 \text{ mm} > p_{1,\min} = 57 \text{ mm}$$

Assuming an elastic distribution with all bolts working in shear and bending:

$$V_{Ed} = V_{A,Ed} \times 2 + V_{B,Ed} \times 2 \rightarrow V_{A,Ed} = 87,50 \text{ kN}; V_{B,Ed} = 87,50 \text{ kN}$$

$$M_{Ed} = H_{A,Ed} \times 0,27 + H_{B,Ed} \times 0,09 \rightarrow H_{A,Ed} = 42,50 \text{ kN}; H_{B,Ed} = 14,17 \text{ kN}$$

Thus

$$F_{A,Ed} = 97,28 \text{ kN} > F_{V,Rd} = 94,10 \text{ kN} \times ; V_{B,Ed} = 88,64 \text{ kN} < F_{V,Rd} = 94,10 \text{ kN} \checkmark$$

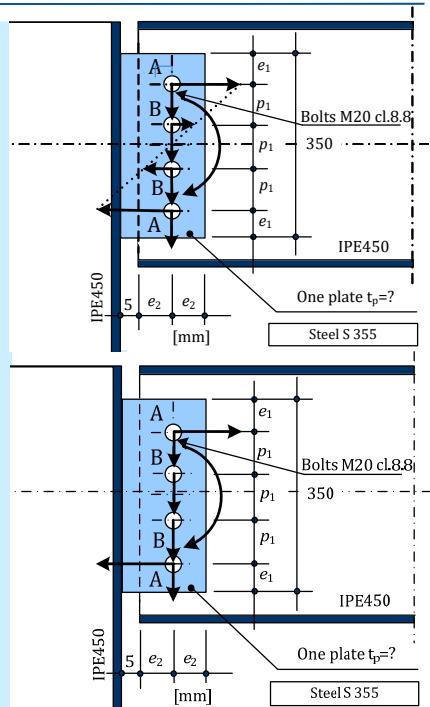
Assuming a plastic distribution with the central bolts only working under shear:

$$V_{Ed} = V_{A,Ed} \times 2 + 94,1 \times 2 \rightarrow V_{A,Ed} = 80,90 \text{ kN}; V_{B,Ed} = 94,10 \text{ kN}$$

$$M_{Ed} = H_{A,Ed} \times 0,27 \rightarrow H_{A,Ed} = 47,20 \text{ kN}; H_{B,Ed} = 0 \text{ kN}$$

Thus

$$F_{A,Ed} = 93,70 \text{ kN} < F_{V,Rd} = 94,10 \text{ kN} \checkmark ; V_{B,Ed} = 94,10 \text{ kN} \leq F_{V,Rd} = 94,10 \text{ kN} \checkmark$$



## STEEL STRUCTURES - DESIGN OF THE JOINTS

### Example 4

$$V_{Ed} = 350 \text{ kN} ; M_{Ed} = 12,75 \text{ kN m}$$

(Profiles and gusset plates – steel S355)

- Design of the connection plate

Pre-design of the thickness:

$$A_{v,IPE\ 450} \cong 450 \times 9,4 = 350 \cdot t_p \rightarrow t_p = 12 \text{ mm}$$

Bloc failure verification:

$$V_{eff,Rd} = 0,5 \times \left( 30 - \frac{22}{2} \right) \cdot t_p \cdot \frac{0,470}{1,25} + (40 + 3 \times 90 - 3,5 \times 22) \cdot t_p \cdot \frac{0,355/\sqrt{3}}{1,0} = 616 \text{ kN} \geq V_{B,Ed} = 350 \text{ kN} \checkmark$$

Verification of the vertical section between holes:

$$V_{eff,Rd} = (2 \times 40 + 3 \times 90 - 4 \times 22) \cdot t_p \cdot \frac{0,355/\sqrt{3}}{1,0} = 644,4 \text{ kN} \geq V_{B,Ed} = 350 \text{ kN}$$

Verification of the bearing plate:

$$V_{b,Rd} = [4 \times 96,5] \cdot \frac{t_p}{10} = 463,2 \text{ kN} \geq V_{B,Ed} = 350 \text{ kN} \checkmark$$

