STEEL BUILDINGS IN EUROPE

Multi-Storey Steel Buildings
Part 5: Joint Design

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Part 5: Joint Design

FOREWORD

This publication is part five of a design guide, *Multi-Storey Steel Buildings*.

The 10 parts in the *Multi-Storey Steel Buildings* guide are:

Part 1: Architect's guide
Part 2: Concept design
Part 3: Actions
Part 4: Detailed design
Part 5: Joint design
Part 6: Fire Engineering
Part 7: Model construction specification
Part 8: Description of member resistance calculator
Part 9: Description of simple connection resistance calculator
Part 10: Guidance to developers of software for the design of composite beams

Multi-Storey Steel Buildings is one of two design guides. The second design guide is *Single Storey Steel Buildings.*

The two design guides have been produced in the framework of the European project "Facilitating the market development for sections in industrial halls and low rise buildings (SECHALO) RFS2-CT-2008-0030".

The design guides have been prepared under the direction of Arcelor Mittal, Peiner Träger and Corus. The technical content has been prepared by CTICM and SCI, collaborating as the Steel Alliance.

Contents

SUMMARY

This design guide gives the design procedure for simple joints in multi-storey buildings according to the Eurocodes.

The guide covers different types of joints:

- Beam-to-beam and beam-to-column joints
  - Partial depth flexible end plate
  - Fin plate
  - Double angle web cleats
- Column splices
- Column bases

Each design procedure is illustrated by a worked example, using the recommended values given in the Eurocodes.

1 INTRODUCTION

1.1 About this design guide

This technical guide is for designing simple joints (nominally pinned) for use in braced multi-storey buildings, designed according to the Eurocodes.

Design procedures are provided for:

- Beam-to-beam and beam-to-column joints
  - Partial depth flexible end plates (also known as header plates)
  - Fin plates
  - Double angle web cleats
- Column splices
- Column bases

The design procedures start with recommended detailing rules (joint geometry) required to ensure ductile behaviour, followed by the checks for each stage of the load transition through the complete joint including welds, plates, bolts and the section webs or flanges as appropriate.

Whilst the Eurocodes establish a common framework for structural calculations across Europe, structural safety remains each country's responsibility. For this reason there are some parameters, called National Determined Parameters (NDP), which each country can decide upon. These are given in the National Annex (NA) documents, which complement the core Eurocodes. However the Eurocode gives some recommendations as to what value each NDP should take. In designing the structure the NDP should be taken from the NA from the country where the structure is to be built.

In this publication the recommended values given in the Eurocode have been adopted in the worked examples.

This publication is complemented by a spreadsheet design tool which allows for NDP for a range of countries. The spreadsheet covers all the joint types included in this publication and can be used in various languages.

1.2 Joint behaviour

Normal practice in simple construction is for beams to be designed as simply supported and for columns to be designed for both the axial compression and, where appropriate, a nominal moment from the beam end connections. In order to ensure that the structure behaves appropriately it is necessary to provide 'simple' connections ('nominally pinned' joints) as defined in EN 1993-1-8, § 5.1.1[1], in which the joint may be assumed not to transfer bending moments. In other words, the joints possess sufficient rotation capacity and sufficient ductility.

Nominally pinned joints have the following characteristics:

1. they are assumed to transfer only the design shear reaction between members
2. they are capable of accepting the resulting rotation
3. they provide the directional restraint to members which has been assumed in the member design
4. they have sufficient robustness to satisfy the structural integrity requirements.

EN 1993-1-8[1] provides two methods to classify joints: stiffness and strength.

- Classification by stiffness: the initial rotational stiffness of the joint, calculated in accordance with Section 6.3.1 of EN 1993-1-8 is compared with the classification boundaries given in Section 5.2 of the same document.
- Classification by strength: the following two requirements must be satisfied in order to classify a joint as pinned:
  - the moment resistance of the joint does not exceed 25% of the moment resistance required for a full-strength joint
  - the joint is capable of accepting the rotation resulting from the design loads.

Alternatively, joints may also be classified based on experimental evidence, experience of previous satisfactory performance in similar cases or by calculations based on test evidence.

Generally, the requirements for nominally pinned behaviour are met by the use of relatively thin plates, combined with full strength welds. Experience and testing have demonstrated that the use of 8 mm or 10 mm end plates, fin plates and angles in S275, with M20 8.8 bolts leads to connections which behave as nominal pins. If details are chosen outside these recommended parameters, the connection should be classified in accordance with EN 1993-1-8.

1.3 Standardised joints

In a typical braced multi-storey frame, the joints may account for less than 5% of the frame weight, but 30% or more of the total cost. Efficient joints will therefore have the lowest detailing, fabrication and erection labour content – they will not necessarily be the lightest.

Use of standardised joints where the fittings, bolts, welds and geometry are fully defined offers the following benefits:

- Reduces buying, storage, and handling time
- Improves availability and leads to a reduction in material costs
- Saves fabrication time and leads to faster erection
- Leads to a better understanding of their performance by all sides of the industry

- Leads to fewer errors.

To take advantage of these benefits, standardised joints are recommended in this publication. A summary of the typical components adopted in this guide is as follows:

- Material of grade S275 for components such as end plates and cleats
- M20 8.8 fully threaded bolts, 60 mm long
- 22 mm holes, punched or drilled
- Fillet welds of 6 mm or 8 mm leg length
- Distance from the top of the beam to the first bolt row of 90 mm
- Vertical bolt spacing (pitch) of 70 mm
- Horizontal bolt spacing (gauge) of 90 or 140 mm
- Top of partial depth end plate, cleat or fin plate is 50 mm below the top of the beam flange.

1.4 Tying resistance

The requirement for sufficient tying resistance is to safeguard the structure against disproportionate collapse. Guidance on the design tying force that a connection must carry is given in EN 1991-1-7 Annex A[2].

EN 1993-1-8 does not give any guidance on how to calculate the tying resistance of joints. Other authoritative sources[3] recommend that the ultimate tensile strength (f_u) should be used for calculating the tying resistance and the partial factor for tying γ_{Mu} should be taken as 1,10. This value applies to the design resistance of all components of the joint: welds, bolts, plate and beam.

1.5 Design guidance in this publication

In this publication, design checks are presented followed in each case by a numerical worked example. The guidance covers:

- partial depth flexible end plates
- fin plates
- double angle web cleats
- column splices
- column bases.

In all worked examples, the section headings correspond to the headings in the design procedure preceeding each workded example.

1.6 Symbols

a is the throat of the fillet weld

b is the breadth of the supported beam

d is the diameter of the bolt

d_0 is the diameter of the hole

$f_{y,b}$ is the yield strength of the supported beam

$f_{u,b}$ is the ultimate tensile strength of the supported beam

$f_{y,p}$ is the yield strength of the plate (end plate, fin plate, flange cover plate, base plate)

$f_{u,p}$ is the ultimate tensile strength of the plate (end plate, fin plate, flange cover plate, base plate)

$f_{y,ac}$ is the yield strength of the angle cleats

$f_{u,ac}$ is the ultimate tensile strength of the angle cleats

f_{ub} is the ultimate tensile strength of the bolt

h_b is the height of the supported beam

h_p is the height of the plate (end plate, fin plate, flange cover plate)

h_{ac} is the height of the angle cleats

n_b is the total number of bolts on supported beam side

n_s is the total number of bolts on supporting beam side

n_1 is the number of horizontal bolt rows

n_2 is the number of vertical bolt rows

t_f is the flange thickness of the supported beam

t_w is the thickness of the supported beam web

t_p is the thickness of the plate (End plate, Fin plate, Flange cover plate, Base plate)

t_{ac} is the thickness of the angle cleats

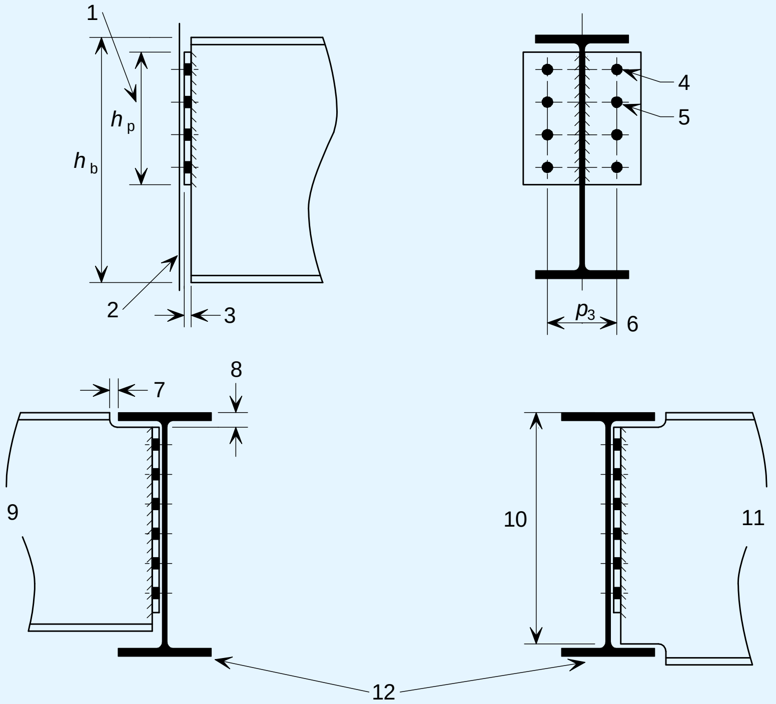
s is the leg length of the fillet weld

γ_{M0} is the partial factor for the resistance of cross section (γ_{M0} = 1,0 is recommended in EN 1993-1-1)

γ_{M1} is the partial factor for the resistance of members to instability assessed by member checks (γ_{M1} = 1,0 is recommended in EN 1993-1-1)

2 PARTIAL DEPTH END PLATE

2.1 Recommended details



1 Length of end plate $h_p \geq 0{,}6h_b$ (see Note 1)
2 Face of beam or column
3 Plate thickness, t_p
t_p = 10 mm or 12 mm (see note 2)
4 Bolt diameter, d
5 Hole diameter, d_0
$d_0 = d + 2$ mm for $d \leq 24$ mm
$d_0 = d + 3$ mm for $d > 24$ mm
6 Gauge; p_3, 90 mm $\leq p_3 \leq$ 140 mm
7 10 mm clearance
8 50 mm but $\geq (t_{f,s} + r_s)$ and $\geq (t_f + r)$
9 Supported beam (single notched)
10 ($h_{b,s}$ – 50 mm) but $\geq (h_b - t_f - r)$
11 Supported beam (double notched)
12 Supporting beam

h_b is the height of the supported beam

$h_{b,s}$ is the height of the supporting beam (if applicable)

t_f is the thickness of the flange of the supported beam

$t_{f,s}$ is the thickness of the flange of the supporting beam (if applicable)

r is the root radius of the supported beam

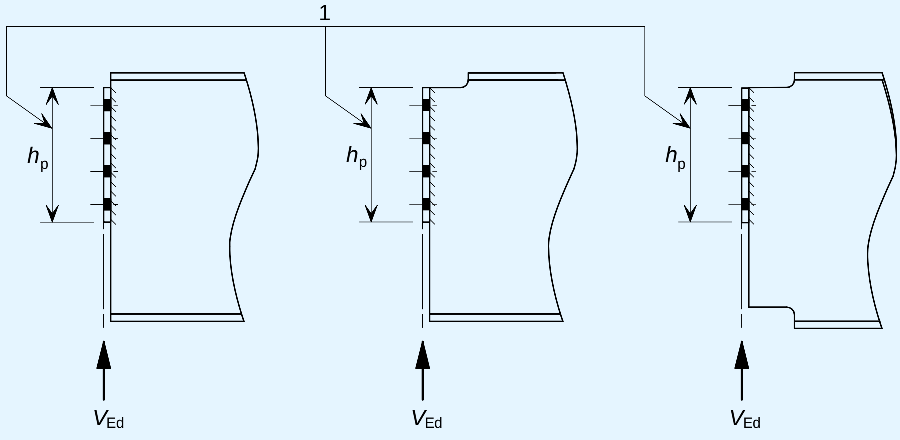
r_s is the root radius of the supporting beam (if applicable)

Notes:

1. The end plate is generally positioned close to the top flange of the beam to provide adequate positional restraint. A plate length of at least $0{,}6h_b$ is usually adopted to give nominal torsional restraint.
2. Although it may be possible to satisfy the design requirements with $t_p < 8$ mm, it is not recommended in practice because of the likelihood of distortion during fabrication and damage during transportation.

2.2 Checks for vertical shear

2.2.1 Shear resistance of the beam web



1 Critical length of web for shear

Shear resistance of the beam web at the end plate

Basic requirement: $V_{Ed} \leq V_{c,Rd}$

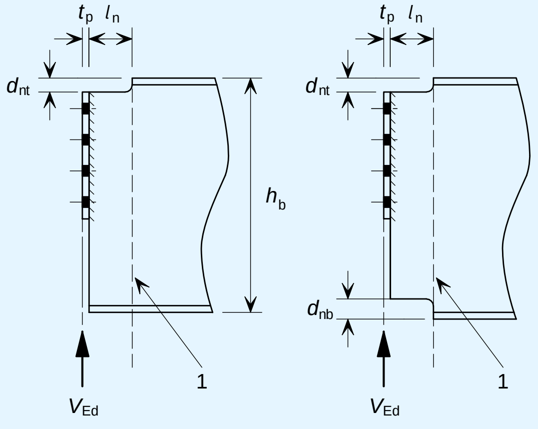
$V_{c,Rd}$ is the design shear resistance of the supported beam connected to the end plate.

$$V_{c,Rd} = V_{pl,Rd} = \frac{A_v \, f_{y,b} / \sqrt{3}}{\gamma_{M0}}$$ [EN 1993-1-1, §6.2.6(1)]

where:

A_v is the shear area, $A_v = h_p t_w$ [Reference 8]

2.2.2 Bending resistance at the notch



1 Critical section

$V_{Ed} \times (t_p + l_n) \leq M_{v,N,Rd}$ or $M_{v,DN,Rd}$

$M_{v,N,Rd}$ isthe moment resistance of a single notched supported beam at the notch in the presence of shear.

$M_{v,DN,Rd}$ is the moment resistance of a double notched supported beam at the notch in the presence of shear.

2.2.2.1 For a single notched beam:

For low shear (i.e. $V_{Ed} \leq 0,5V_{pl,N,Rd}$)

$$M_{v,N,Rd} = \frac{f_{y,b}\, W_{el,N,y}}{\gamma_{M0}}$$ [Reference 4]

For high shear (i.e. $V_{Ed} > 0,5V_{pl,N,Rd}$)

$$M_{v,N,Rd} = \frac{f_{y,b}\, W_{el,N,y}}{\gamma_{M0}}\left[1-\left(\frac{2V_{Ed}}{V_{pl,N,Rd}}-1\right)^2\right]$$ [Reference 4]

2.2.2.2 For double notched beam:

For low shear (i.e. $V_{Ed} \leq 0,5V_{pl,DN,Rd}$)

$$M_{v,DN,Rd} = \frac{f_{y,b}\, t_w}{6\gamma_{M0}}(h_b - d_{nt} - d_{nb})^2$$ [Reference 4]

For high shear (i.e. $V_{Ed} > 0,5V_{pl,DN,Rd}$)

$$M_{v,DN,Rd} = \frac{f_{y,b} t_w}{6\gamma_{M0}}(h_b - d_{nt} - d_{nb})^2\left[1-\left(\frac{2V_{Ed}}{V_{pl,DN,Rd}}-1\right)^2\right]$$ [Reference 4]

$V_{pl,N,Rd}$ is the shear resistance at the notch for single notched beams

$$V_{pl,N,Rd} = \frac{A_{v,N}\, f_{y,b}}{\sqrt{3}\,\gamma_{M0}}$$

$$A_{v,N} = A_{Tee} - bt_f + (t_w + 2r)\frac{t_f}{2}$$

A_{Tee} is the area of the Tee section

$V_{pl,DN,Rd}$ is the shear resistance at the notch for double notched beams

$$V_{pl,DN,Rd} = \frac{A_{v,DN}\, f_{y,b}}{\sqrt{3}\,\gamma_{M0}}$$

$$A_{v,DN} = t_w\,(h_b - d_{nt} - d_{nb})$$

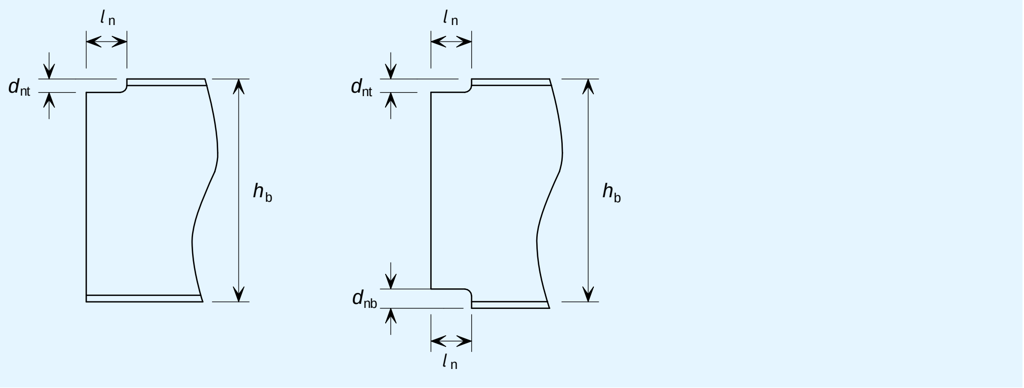
where:

$W_{el,N,y}$ is the elastic modulus of the section at the notch

d_{nt} is the depth of the top notch

d_{nb} is the depth of the bottom notch

2.2.3 Local stability of notched beam



When the beam is restrained against lateral torsional buckling, no account needs be taken of notch stability provided the following conditions are met:

For one flange notched, basic requirement:[5][6]

$d_{nt} \leq h_b / 2$ and:

$l_n \leq h_b$ for $h_b / t_w \leq 54{,}3$ (S275 steel)

$l_n \leq \dfrac{160000\, h_b}{(h_b / t_w)^3}$ for $h_b / t_w > 54{,}3$ (S275 steel)

$l_n \leq h_b$ for $h_b / t_w \leq 48{,}0$ (S355 steel)

$l_n \leq \dfrac{110000\, h_b}{(h_b / t_w)^3}$ for $h_b / t_w > 48{,}0$ (S355 steel)

For both flanges notched, basic requirement:[7]

max $(d_{nt}, d_{nb}) \leq h_b / 5$ and:

$l_n \leq h_b$ for $h_b / t_w \leq 54{,}3$ (S275 steel)

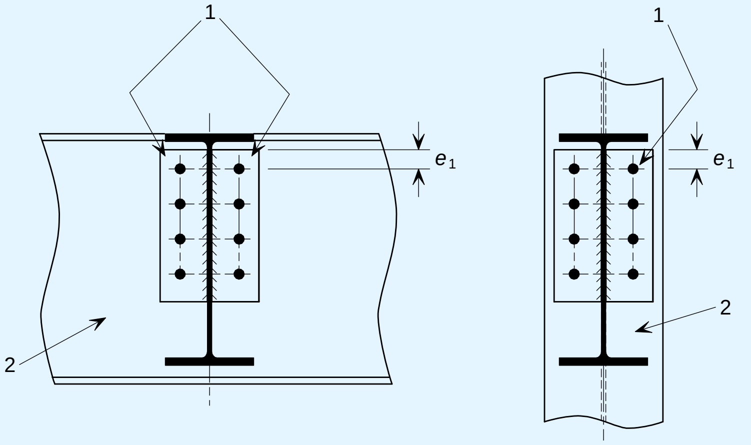
$l_n \leq \dfrac{160000\, h_b}{(h_b / t_w)^3}$ for $h_b / t_w > 54{,}3$ (S275 steel)

$l_n \leq h_b$ for $h_b / t_w \leq 48{,}0$ (S355 steel)

$l_n \leq \dfrac{110000\, h_b}{(h_b / t_w)^3}$ for $h_b / t_w > 48{,}0$ (S355 steel)

Where the notch length l_n exceeds these limits, either suitable stiffening should be provided or the notch should be checked to References 5, 6 and 7.
For S235 and S460 members see References 5, 6 and 7.

2.2.4 Bolt group resistance



1 Check these bolts in shear under concentric load
2 Supporting beam
3 Supporting column

Basic requirement: $V_{Ed} \leq F_{Rd}$

F_{Rd} is the resistance of the bolt group [EN 1993-1-8 ,§3.7(1)]

If $(F_{b,Rd})_{max} \leq F_{v,Rd}$ then $F_{Rd} = \sum F_{b,Rd}$

If $(F_{b,Rd})_{min} \leq F_{v,Rd} \leq (F_{b,Rd})_{max}$ then $F_{Rd} = n_s(F_{b,Rd})_{min}$

If $F_{v,Rd} < (F_{b,Rd})_{min}$ then $F_{Rd} = 0{,}8 n_s F_{v,Rd}$

2.2.4.1 Shear resistance of bolts

$F_{v,Rd}$ is the shear resistance of one bolt

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$ [EN 1993-1-8, Table 3.4]

where:

α_v = 0,6 for 4.6 and 8.8 bolts
= 0,5 for 10.9 bolts

A is the tensile stress area of the bolt, A_s

γ_{M2} is the partial factor for resistance of bolts

2.2.4.2 Bearing resistance

$$F_{b,Rd} = \frac{k_1 \alpha_b f_{u,p} d t_p}{\gamma_{M2}}$$ [EN 1993-1-8 Table 3.4]

where:

γ_{M2} is the partial factor for plate in bearing

– For end bolts (parallel to the direction of load transfer)

$$\alpha_b = \min\left(\frac{e_1}{3d_0};\ \frac{f_{ub}}{f_{u,p}};\ 1{,}0\right)$$

– For inner bolts (parallel to the direction of load transfer)

$$\alpha_b = \min\left(\frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,p}};\ 1{,}0\right)$$

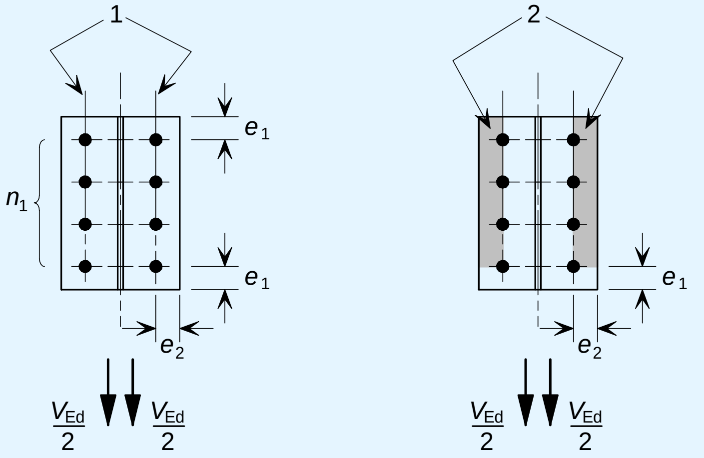
– For edge bolts (perpendicular to the direction of load transfer)

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

– For inner bolts (perpendicular to the direction of load transfer)

$$k_1 = \min\left(1{,}4\frac{p_2}{d_0} - 1{,}7;\ 2{,}5\right)$$

2.2.5 Shear resistance of the end plate



1 Critical section in shear and bearing
2 Block shear – check failure by tearing out of shaded portion

Basic requirement: $V_{Ed} \leq V_{Rd,min}$

$V_{Rd,min} = \min(V_{Rd,g};\ V_{Rd,n};\ V_{Rd,b})$

where:

$V_{Rd,g}$ is the shear resistance of the gross section

$V_{Rd,n}$ is the shear resistance of the net section

$V_{Rd,b}$ is the block tearing resistance

2.2.5.1 Shear resistance of gross section

$$V_{Rd,g} = 2 \times \frac{h_p t_p}{1{,}27} \frac{f_{y,p}}{\sqrt{3}\gamma_{M0}}$$ [Reference 8]

Note: The coefficient 1,27 takes into account the reduction in shear resistance due to the presence of the nominal in-plane bending which produces tension in the bolts[9].

2.2.5.2 Shear resistance of net section

$$V_{\mathrm{Rd,n}} = 2 \times A_{\mathrm{v,net}} \frac{f_{\mathrm{u,p}}}{\sqrt{3}\gamma_{\mathrm{M2}}}$$ [Reference 8]

$$A_{\mathrm{v,net}} = t_{\mathrm{p}} \left(h_{\mathrm{p}} - n_1 d_0 \right)$$

γ_{M2} is the partial factor for the resistance of net sections

2.2.5.3 Block tearing resistance

$$V_{\mathrm{Rd,b}} = 2\left(\frac{f_{\mathrm{u,p}} A_{\mathrm{nt}}}{\gamma_{\mathrm{M2}}} + \frac{f_{\mathrm{y,p}} A_{\mathrm{nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}} \right)$$ [Reference 8]

But if $h_{\mathrm{p}} < 1.36 p_3$ and $n_1 > 1$ then:

$$V_{\mathrm{Rd,b}} = 2\left(\frac{0,5 f_{\mathrm{u,p}} A_{\mathrm{nt}}}{\gamma_{\mathrm{M2}}} + \frac{f_{\mathrm{y,p}} A_{\mathrm{nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}} \right)$$

$$A_{\mathrm{nt}} = t_{\mathrm{p}} \left(e_2 - 0,5 d_0 \right)$$

$$A_{\mathrm{nv}} = t_{\mathrm{p}} \left(h_{\mathrm{p}} - e_1 - (n_1 - 0,5) d_0 \right)$$

where:

p_3 is the gauge (cross centres)

2.2.6 Weld resistance

Full strength symmetrical fillet welds are recommended.

For a full strength weld, the size of each throat should comply with the following requirement[8]:

$a \geq 0,46\, t_{\mathrm{w}}$ for S235 supported beam

$a \geq 0,48\, t_{\mathrm{w}}$ for S275 supported beam

$a \geq 0,55\, t_{\mathrm{w}}$ for S355 supported beam

$a \geq 0,74\, t_{\mathrm{w}}$ for S460 supported beam

where:

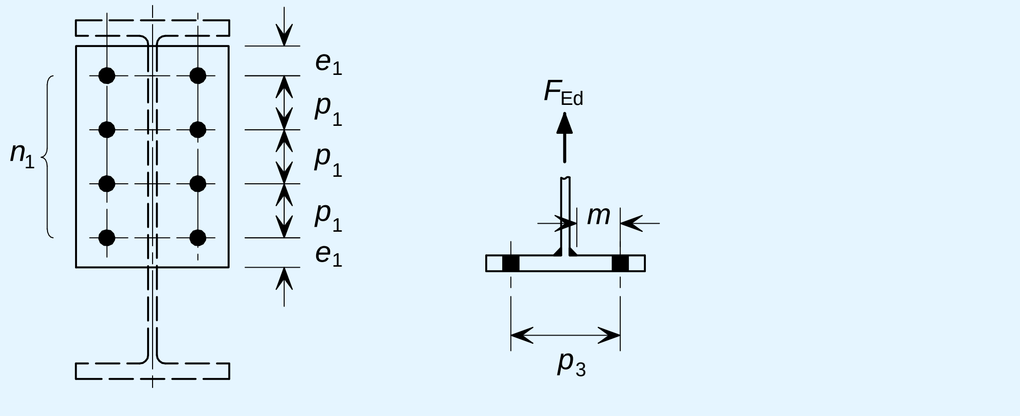
a is the effective weld throat thickness

The leg length is defined as follows: $s = a\sqrt{2}$

2.3 Checks for tying

EN 1993-1-8 does not have a partial factor for structural integrity checks. In this publication γ_{Mu} has been used. A value of $\gamma_{Mu} = 1,1$ is recommended.

2.3.1 Resistance of the end plate in bending



There are three modes of failure for end plates in bending:

Mode 1: complete yielding of the flange

Mode 2: bolt failure with yielding of the flange

Mode 3: bolt failure

Basic requirement: $F_{Ed} \leq \min(F_{Rd,u,1}; F_{Rd,u,2}; F_{Rd,u,3})$

Mode 1 (complete yielding of the end plate)

$$F_{Rd,u,1} = \frac{(8n - 2e_w)M_{pl,1,Rd,u}}{2mn - e_w(m+n)}$$ [EN 1993-1-8, Table 6.2]

Mode 2 (bolt failure with yielding of the end plate)

$$F_{Rd,u,2} = \frac{2M_{pl,2,Rd,u} + n\Sigma F_{t,Rd,u}}{m+n}$$ [EN 1993-1-8, Table 6.2]

Mode 3 (bolt failure)

$$F_{Rd,u,3} = \Sigma F_{t,Rd,u}$$ [EN 1993-1-8, Table 6.2]

$$F_{t,Rd,u} = \frac{k_2 f_{ub} A}{\gamma_{Mu}}$$

where:

$$M_{pl,1,Rd,u} = \frac{0,25\Sigma l_{eff} t_p^2 f_{u,p}}{\gamma_{Mu}}$$

$$M_{pl,2,Rd,u} = M_{pl,1,Rd,u}$$

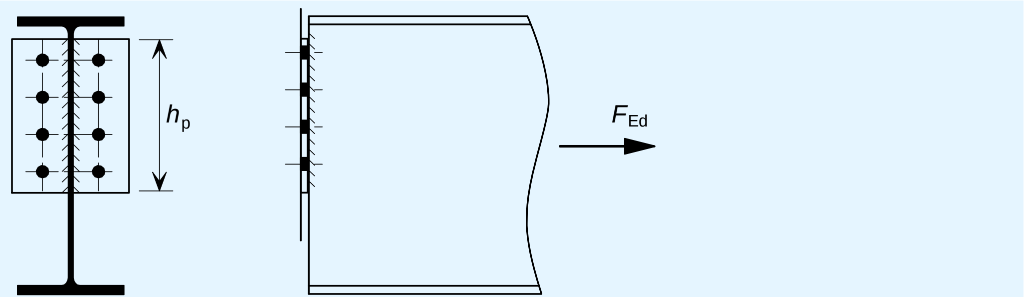
$$m = \frac{p_3 - t_w - 2 \times 0,8 \times a\sqrt{2}}{2}$$

$n = e_{min}$ but $n \leq 1,25m$ where $e_{min} = e_2$

e_w	$= \dfrac{d_w}{4}$
d_w	is the diameter of the washer
k_2	= 0,63 for countersunk bolts = 0,9 otherwise
A	is the tensile stress area of the bolts, A_s
Σl_{eff}	is the effective length of one plastic hinge
Σl_{eff}	$= 2e_{1A} + (n_1 - 1)\, p_{1A}$
e_{1A}	$= e_1$ but $\leq 0{,}5\left(p_3 - t_w - 2a\sqrt{2}\right) + \dfrac{d_0}{2}$
p_{1A}	$= p_1$ but $\leq p_3 - t_w - 2a\sqrt{2} + d_0$

The leg length is defined as follows: $s = a\sqrt{2}$

2.3.2 Beam web resistance



Basic requirement: $F_{Ed} \leq F_{Rd}$

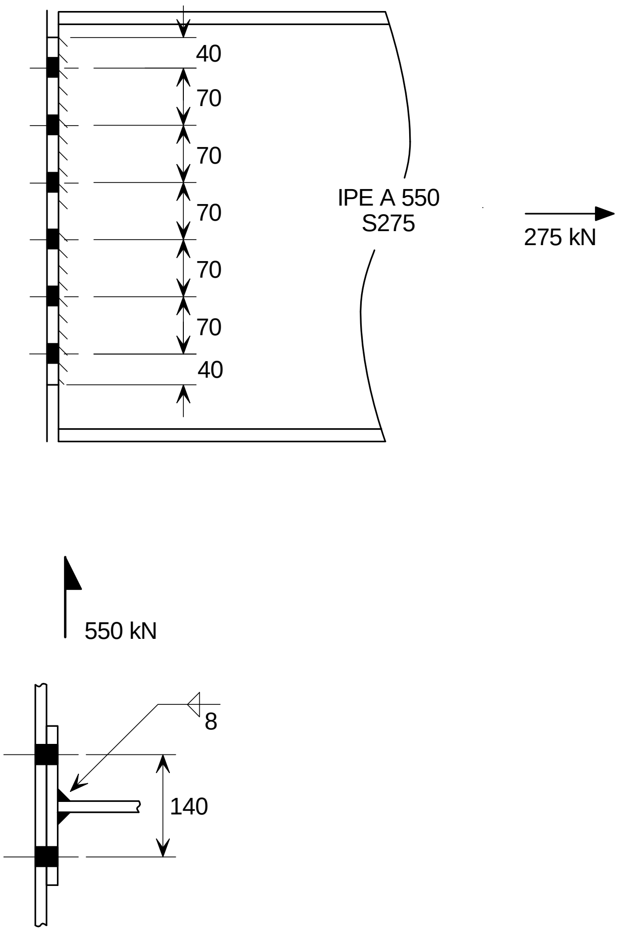
$$F_{Rd} = \frac{t_w h_p f_{u,b}}{\gamma_{Mu}}$$ [Reference 8]

2.3.3 Weld resistance

The weld size specified for shear will be adequate for tying resistance, as it is full strength.

2. Partial depth end plate

Details and data



Beam: IPE A 550 S275

Partial depth flexible end plate: 430 × 200 × 12, S275

Bolts: M20 8.8

Welds: 8 mm fillet (weld throat, a = 5,6 mm)

Summary of full design checks

Design forces

V_{Ed} = 550 kN

F_{Ed} = 275 kN (Tie force)

Shear resistances

Shear resistance of the beam web	614 kN
Bending resistance at the notch	N/A
Local stability of notched beam	N/A
Bolt group resistance	902 kN
Resistance of the end plate	1182 kN
Weld resistance	OK

Tying resistances

Resistance of the end plate in bending	493 kN
Tension resistance of the beam web	1513 kN
Weld resistance	OK

2.1. Recommended details

End plate: 200×12 mm

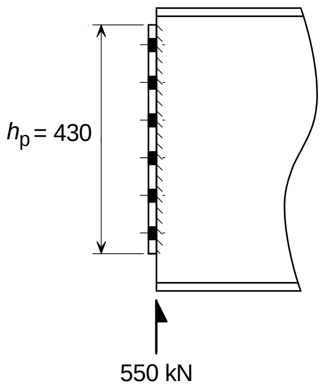
Height of plate: h_p = 430 mm > $0,6h_b$, OK

Bolts: M20, 8.8 at 140 mm gauge

2.2. Checks for vertical shear

2.2.1. Shear resistance of the beam web

Unless noted otherwise, all references are to EN 1993-1-8



Basic requirement: $V_{Ed} \le V_{c,Rd}$

The shear resistance of the beam web, $V_{c,Rd} = \dfrac{A_v\, f_{y,b} / \sqrt{3}}{\gamma_{M0}}$

EN 1993-1-1 § 6.2.6(1)

Shear area of beam web,

$A_v = 430 \times 9 = 3870 \text{ mm}^2$

Shear resistance of beam web,

$$V_{pl,Rd} = \frac{3870 \times 275/\sqrt{3}}{1,0} \times 10^{-3} = 614 \text{ kN}$$

$V_{Ed} = 550 \text{ kN} \leq 614 \text{ kN, OK}$

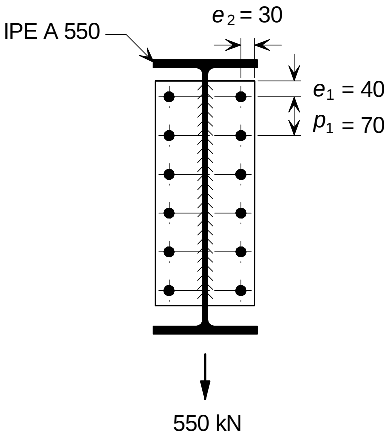
2.2.2. Bending resistance at the notch

Not applicable (No notch)

2.2.3. Local stability of notched beam

Not applicable (No notch)

2.2.4. Bolt group resistance



Basic requirement: $V_{Ed} \leq F_{Rd}$

The design resistance of the bolt group, F_{Rd}: § 3.7

if $(F_{b,Rd})_{max} \leq F_{v,Rd}$ then $F_{Rd} = \Sigma F_{b,Rd}$

if $(F_{b,Rd})_{min} \leq F_{v,Rd} < (F_{b,Rd})_{max}$ then $F_{Rd} = n_s\,(F_{b,Rd})_{min}$

if $F_{v,Rd} < (F_{b,Rd})_{min}$ then $F_{Rd} = 0,8\,n_s F_{v,Rd}$

2.2.4.1. Shear resistance of bolts

The shear resistance of a single bolt, $F_{v,Rd} = \dfrac{\alpha_v f_{ub} A}{\gamma_{M2}}$ Table 3.4

For M20 8.8 bolts, $F_{v,Rd} = \dfrac{0,6 \times 800 \times 245}{1,25} \times 10^{-3} = 94 \text{ kN}$

2.2.4.2. Bearing resistance

Table 3.4

Bearing resistance, $F_{\mathrm{b,Rd}} = \dfrac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,p}} d t_{\mathrm{p}}}{\gamma_{\mathrm{M2}}}$

For edge bolts, $k_1 = \min\left(2{,}8\dfrac{e_2}{d_0} - 1{,}7;\ 2{,}5\right)$

$$= \min\left(2{,}8 \times \frac{30}{22} - 1{,}7;\ 2{,}5\right) = \min(2{,}12;\ 2{,}5) = 2{,}12$$

For end bolts, $\alpha_{\mathrm{b}} = \min\left(\dfrac{e_1}{3d_0};\ \dfrac{f_{\mathrm{ub}}}{f_{\mathrm{u,p}}};\ 1{,}0\right) = \min\left(\dfrac{40}{3 \times 22};\ \dfrac{800}{430};\ 1.0\right)$

$$= \min(0{,}61;\ 1{,}86;\ 1{,}0) = 0{,}61$$

For inner bolts, $\alpha_{\mathrm{b}} = \min\left(\dfrac{p_1}{3d_0} - \dfrac{1}{4};\ \dfrac{f_{\mathrm{ub}}}{f_{\mathrm{u,p}}};\ 1{,}0\right) = \min\left(\dfrac{70}{3 \times 22} - \dfrac{1}{4};\ \dfrac{800}{430};\ 1{,}0\right)$

$$= \min(0{,}81;\ 1{,}86;\ 1{,}0) = 0{,}81$$

End bolts, $F_{\mathrm{b,Rd,end}} = \left(F_{\mathrm{b,Rd}}\right)_{\min} = \dfrac{2{,}12 \times 0{,}61 \times 430 \times 20 \times 12}{1{,}25} \times 10^{-3} = 107\ \mathrm{kN}$

Inner bolts, $F_{\mathrm{b,Rd,inner}} = \left(F_{\mathrm{b,Rd}}\right)_{\max} = \dfrac{2{,}12 \times 0{,}81 \times 430 \times 20 \times 12}{1{,}25} \times 10^{-3} = 142\ \mathrm{kN}$

94 kN < 107 kN thus $F_{\mathrm{v,Rd}} < (F_{\mathrm{b,Rd}})_{\min}$

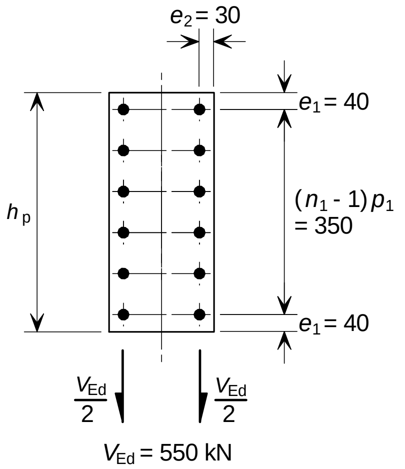
$F_{\mathrm{Rd}} = 0{,}8 n_{\mathrm{s}} (F_{\mathrm{v,Rd}})_{\min} = 0{,}8 \times 12 \times 94 = 902\ \mathrm{kN}$

$V_{\mathrm{Ed}} = 550\ \mathrm{kN} \leq 902\ \mathrm{kN}$, OK

2.2.5. Shear resistance of the end plate

Basic requirement: $V_{\mathrm{Ed}} \leq V_{\mathrm{Rd,min}}$

$V_{\mathrm{Rd,min}} = (V_{\mathrm{Rd,g}};\ V_{\mathrm{Rd,n}};\ V_{\mathrm{Rd,b}})$



2.2.5.1. Shear resistance of gross section

Ref (8)

$$V_{\mathrm{Rd,g}} = \frac{2h_{\mathrm{p}}t_{\mathrm{p}}}{1{,}27}\frac{f_{\mathrm{y,p}}}{\sqrt{3}\gamma_{\mathrm{M0}}} = \frac{2\times430\times12\times275}{1{,}27\times\sqrt{3}\times1{,}0}\times10^{-3} = 1290 \text{ kN}$$

2.2.5.2. Shear resistance of net section

Ref (8)

$$V_{\mathrm{Rd,n}} = 2\times A_{\mathrm{v,net}}\frac{f_{\mathrm{u,p}}}{\sqrt{3}\gamma_{\mathrm{M2}}}$$

Net area, $A_{\mathrm{v,net}} = 12(430-6\times22) = 3576 \text{ mm}^2$

$$V_{\mathrm{Rd,n}} = 2\times3576\times\frac{430}{\sqrt{3}\times1{,}25}\times10^{-3} = 1420 \text{ kN}$$

2.2.5.3. Block tearing resistance

Ref (8)

$h_{\mathrm{p}} = 430$ and $1{,}36\,p_3 = 1{,}36\times140 = 190$ mm

Since $h_{\mathrm{p}} > 1{,}36p_3$ then

$$V_{\mathrm{Rd,b}} = 2\times\left(\frac{f_{\mathrm{u,p}}A_{\mathrm{nt}}}{\gamma_{\mathrm{M2}}} + \frac{f_{\mathrm{y,p}}A_{\mathrm{nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}}\right)$$

Net area subject to tension, $A_{\mathrm{nt}} = t_{\mathrm{p}}(e_2 - 0{,}5d_0)$

$= 12(30-0{,}5\times22) = 228 \text{ mm}^2$

Net area subject to shear, $A_{\mathrm{nv}} = t_{\mathrm{p}}(h_{\mathrm{p}} - e_1 - (n_1 - 0{,}5)d_0)$

$= 12(430-40-(6-0{,}5)22) = 3228 \text{ mm}^2$

$$V_{\mathrm{Rd,b}} = 2\times\left(\frac{430\times228}{1{,}25} + \frac{275\times3228}{\sqrt{3}\times1{,}0}\right)\times10^{-3} = 1182 \text{ kN}$$

$V_{\mathrm{Rd,min}}$ = min(1290; 1420; 1182) = 1182 kN

V_{Ed} = 550 kN $\leq$ 1182 kN, OK

2.2.6. Weld resistance

For a beam in S275 steel

Basic requirement: $a \geq 0{,}48\,t_{\mathrm{w}}$

Ref (8)

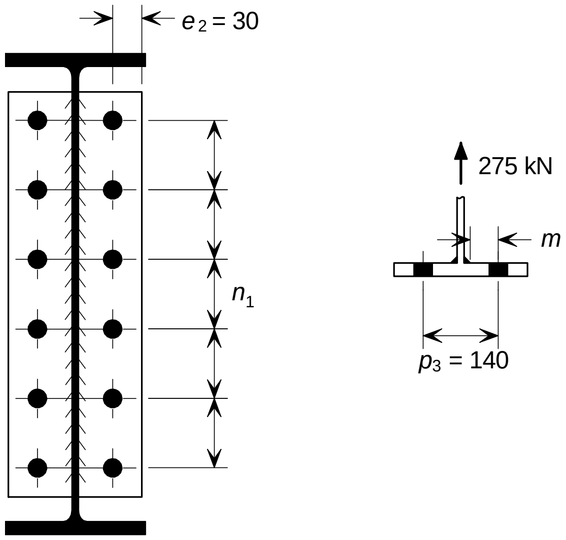
$0{,}48t_{\mathrm{p}} = 0{,}48\times9 = 4{,}32$ mm

$a = 5{,}7 \text{ mm} \geq 0{,}48\,t_{\mathrm{w}}$ OK

2.3. Checks for tying

2.3.1. Resistance of the end plate in bending

Basic requirement: $F_{\rm Ed} \le \min\left(F_{\rm Rd,u,1}, F_{\rm Rd,u,2}, F_{\rm Rd,u,3}\right)$



Mode 1:

Table 6.2

$$F_{\rm Rd,u,1} = \frac{(8n - 2e_{\rm w})M_{\rm pl,1,Rd,u}}{2mn - e_{\rm w}(m+n)}$$

$$\Sigma l_{\rm eff} = 2e_{1\rm A} + (n_1 - 1)\,p_{1\rm A}$$

$$e_{1\rm A} = e_1 \text{ but} \le 0,5(p_3 - t_{\rm w} - 2a\sqrt{2}) + \frac{d_0}{2}$$

$$0,5(140 - 9 - 2\times 5,6\sqrt{2}) + \frac{22}{2} = 69 \text{ mm}$$

$\therefore \quad e_{1\rm A} = 40$

$$p_{1\rm A} = p_1 \text{ but} \le p_3 - t_{\rm w} - 2a\sqrt{2} + d_0$$

$$p_3 - t_{\rm w} - 2a\sqrt{2} + d_0 = 140 - 9 - 2\times 5,6\sqrt{2} + 22 = 137 \text{ mm}$$

$\therefore \quad p_{1\rm A} = 70$

$$\Sigma l_{\rm eff} = 2e_{1\rm A} + (n_1 - 1)\,p_{1\rm A} = 2\times 40 + (6-1)70 = 430 \text{ mm}$$

$$M_{\rm pl,1,Rd,u} = \frac{0,25\Sigma l_{\rm eff,1} t_{\rm p}^{\ 2} f_{\rm u,p}}{\gamma_{\rm Mu}} = \frac{0,25\times 430\times 12^2\times 430}{1,1}\times 10^{-6} = 6,05 \text{ kNm}$$

$$m = \frac{p_3 - t_{\rm w} - 2\times 0,8\times a\sqrt{2}}{2} = \frac{140 - 9 - 2\times 0,8\times 5,6\times\sqrt{2}}{2} = 59 \text{ mm}$$

$$e_{\rm w} = \frac{d_{\rm w}}{4} = \frac{37}{4} = 9,25 \text{ mm}$$

$$n = \min\left(e_2;\ 1,25m\right) = \min\left(30;\ 76\right) = 30 \text{ mm}$$

$$F_{\mathrm{Rd,u,1}} = \frac{(8\times 30 - 2\times 9,25)6,05\times 10^{3}}{2\times 59\times 30 - 9,25(59+30)} = 493 \text{ kN}$$

Mode 2:

Table 6.2

$$F_{\mathrm{Rd,u,2}} = \frac{2M_{\mathrm{pl,2,Rd,u}} + n\Sigma F_{\mathrm{t,Rd,u}}}{m+n}$$

$$M_{\mathrm{pl,2,Rd,u}} = M_{\mathrm{pl,1,Rd,u}} = 6,05 \text{ kNm}$$

$$F_{\mathrm{t,Rd,u}} = \frac{k_2 f_{\mathrm{ub}} A}{\gamma_{\mathrm{Mu}}} = \frac{0,9\times 800\times 245}{1,1}\times 10^{-3} = 160 \text{ kN}$$

$$F_{\mathrm{Rd,u,2}} = \frac{2\times 6,05\times 10^{3} + 30\times 12\times 160}{59+30} = 793 \text{ kN}$$

Mode 3:

Table 6.2

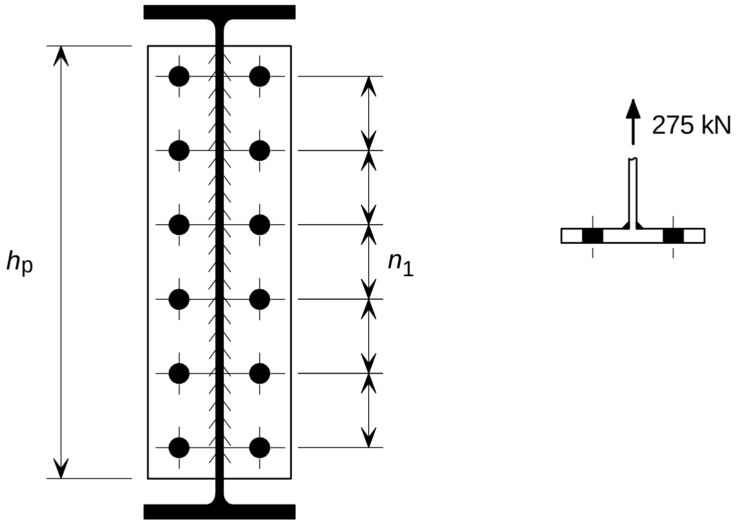
$$F_{\mathrm{Rd,u,3}} = \Sigma F_{\mathrm{t,Rd,u}} = 12\times 160 = 1920 \text{ kN}$$

$$\min\left(F_{\mathrm{Rd,u,1}}, F_{\mathrm{Rd,u,2}}, F_{\mathrm{Rd,u,3}}\right) = \min(493;\ 793;\ 1920) = 493 \text{ kN}$$

F_{Ed} = 275 kN $\leq$ 493 kN, OK

2.3.2. Beam web resistance

Basic requirement: $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd}}$



Ref (8)

$$F_{\mathrm{Rd}} = \frac{t_{\mathrm{w}} h_{\mathrm{p}} f_{\mathrm{u,b}}}{\gamma_{\mathrm{Mu}}} = \frac{9\times 430\times 430}{1,1}\times 10^{-3} = 1513 \text{ kN}$$

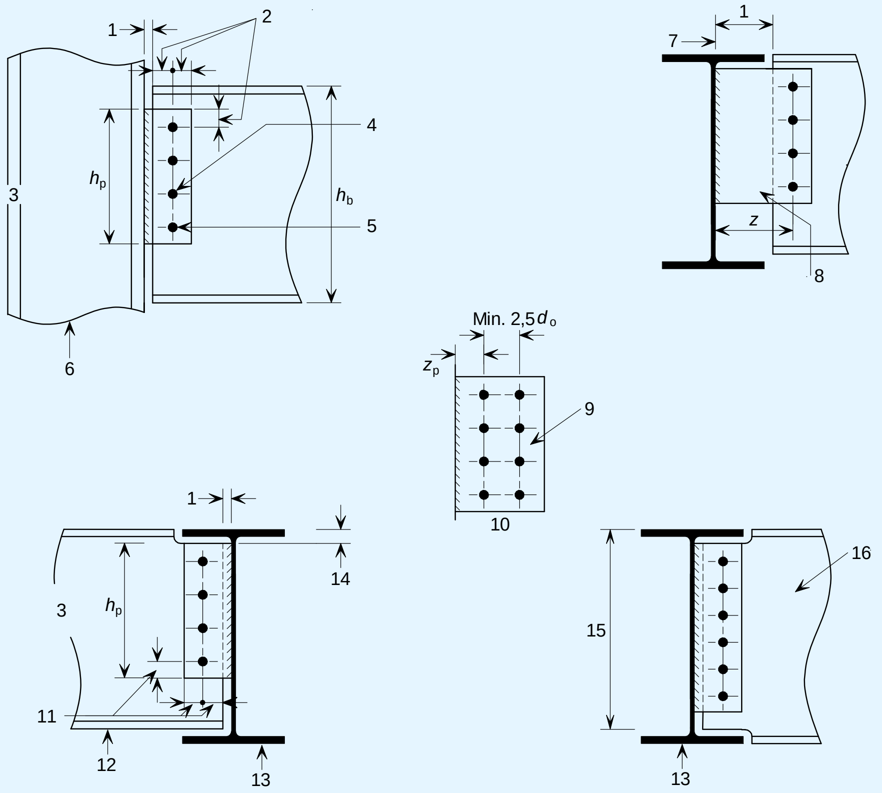
F_{Ed} = 275 kN $\leq$ 1513 kN, OK

2.3.3. Weld resistance

The weld size specified for shear will be adequate for tying resistance, as it is full strength.

3 FIN PLATE

3.1 Recommended details



1 End projection g_h
3 All end and edge distances ≥ $2d$
4 Length of fin plate $h_p \geq 0,6\, h_b$
5 Bolt diameter, d. Only 8.8 bolts to be used, untorqued in clearance holes
6 Hole diameter, d_0. $d_0 = d + 2$ mm for $d \leq 24$ mm; $d_0 = d + 3$ mm for $d > 24$ mm
7 Supporting column
8 Face of web
9 Long fin plate if $z \geq \dfrac{t_p}{0,15}$ t_p = fin plate thickness
10 Fin plate thickness $t_p \leq 0,5d$
11 Double line of bolts
12 All end and edge distances ≥ $2d$
13 Supported beam (Single notched)
14 Supporting beam
15 50 mm but ≥ $(t_f + r)$ and ≥ $(t_{f,s} + r_s)$
16 $(h_{b,s} - 50$ mm$)$ but ≤ $(h_s - t_{f,s} - r_s)$
17 Supported beam (Double notched)

h_b is the height of the supported beam

$h_{b,s}$ is the height of the supporting beam (if applicable)

t_f is the thickness of the flange of the supported beam

$t_{f,s}$ is the thickness of the flange of the supporting beam (if applicable)

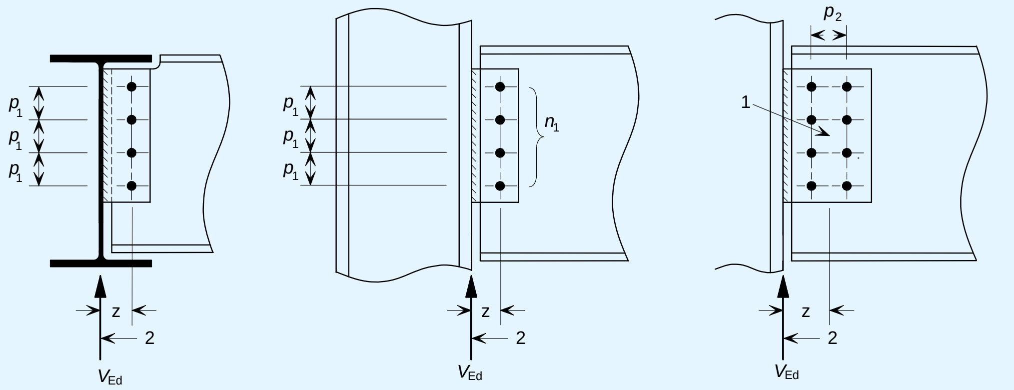
r is the root radius of the supported beam

r_s is the root radius of the supporting beam (if applicable)

3.2 Checks for vertical shear

3.2.1 Bolt group resistance

3.2.1.1 Shear resistance of bolts



1 Centre of bolt group
2 Assumed line of shear transfer

Basic requirement: $V_{Ed} \leq V_{Rd}$

$$V_{Rd} = \frac{n_b F_{v,Rd}}{\sqrt{(1+\alpha n_b)^2 + (\beta n_b)^2}}$$ [Reference 3]

$F_{v,Rd}$ is the shear resistance of one bolt

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

where:

A is the tensile stress area of the bolt, A_s

α_v = 0,6 for 4.6 and 8.8 bolts
= 0,5 for 10.9 bolts

γ_{M2} is the partial factor for resistance of bolts

For a single vertical line of bolts ($n_2 = 1$)

$$\alpha = 0 \text{ and } \beta = \frac{6z}{n_1(n_1+1)p_1}$$

For a double vertical line of bolts ($n_2 = 2$)

$$\alpha = \frac{zp_2}{2I} \text{ and } \beta = \frac{zp_1}{2I}(n_1 - 1)$$

$$I = \frac{n_1}{2} p_2^2 + \frac{1}{6} n_1 \left(n_1^2 - 1\right) p_1^2$$

z is the transverse distance from the face of the supporting element to the centre of the bolt group

3.2.1.2 Bearing resistance of bolts on the fin plate

Basic requirement: $V_{\mathrm{Ed}} \leq V_{\mathrm{Rd}}$

$$V_{\mathrm{Rd}} = \frac{n_{\mathrm{b}}}{\sqrt{\left(\frac{1+\alpha n_{\mathrm{b}}}{F_{\mathrm{b,ver,Rd}}}\right)^2 + \left(\frac{\beta n_{\mathrm{b}}}{F_{\mathrm{b,hor,Rd}}}\right)^2}}$$ [Reference 3]

The bearing resistance of a single bolt is $F_{\mathrm{b,Rd}} = \dfrac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,p}} d t_{\mathrm{p}}}{\gamma_{\mathrm{M2}}}$

The vertical bearing resistance of a single bolt on the fin plate is as follows:

$$F_{\mathrm{b,ver,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,p}} d t_{\mathrm{p}}}{\gamma_{\mathrm{M2}}}$$

The horizontal bearing resistance of a single bolt on the fin plate is as follows:

$$F_{\mathrm{b,hor,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,p}} d t_{\mathrm{p}}}{\gamma_{\mathrm{M2}}}$$

α and β are as defined previously

For $F_{\mathrm{b,ver,Rd}}$:

$$k_1 = \min\left(2{,}8\frac{e_2}{d_0} - 1{,}7;\ 1{,}4\frac{p_2}{d_0} - 1{,}7;\ 2{,}5\right)$$

$$\alpha_{\mathrm{b}} = \min\left(\frac{e_1}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{\mathrm{ub}}}{f_{\mathrm{u,p}}};1{,}0\right)$$

For $F_{\mathrm{b,hor,Rd}}$:

$$k_1 = \min\left(2{,}8\frac{e_1}{d_0} - 1{,}7;\ 1{,}4\frac{p_1}{d_0} - 1{,}7;\ 2{,}5\right)$$

$$\alpha_{\mathrm{b}} = \min\left(\frac{e_2}{3d_0};\ \frac{p_2}{3d_0} - \frac{1}{4};\ \frac{f_{\mathrm{ub}}}{f_{\mathrm{u,p}}};\ 1{,}0\right)$$

3.2.1.3 Bearing resistance of bolts on the beam web

Basic requirement: $V_{\mathrm{Ed}} \leq V_{\mathrm{Rd}}$

$$V_{\mathrm{Rd}} = \frac{n_{\mathrm{b}}}{\sqrt{\left(\frac{1+\alpha n_{\mathrm{b}}}{F_{\mathrm{b,ver,Rd}}}\right)^2 + \left(\frac{\beta n_{\mathrm{b}}}{F_{\mathrm{b,hor,Rd}}}\right)^2}}$$ [Reference 3]

$$F_{\mathrm{b,ver,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,b}} d t_{\mathrm{w}}}{\gamma_{\mathrm{M2}}}$$

$$F_{\mathrm{b,hor,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,b}} d t_{\mathrm{w}}}{\gamma_{\mathrm{M2}}}$$

α and β are as defined previously

γ_{M2} is the partial factor for beam web in bearing

For $F_{b,ver,Rd}$:

$$k_1 = \min\left(2.8\frac{e_{2,b}}{d_0} - 1{,}7;\ 1.4\frac{p_2}{d_0} - 1{,}7;\ 2{,}5\right)$$

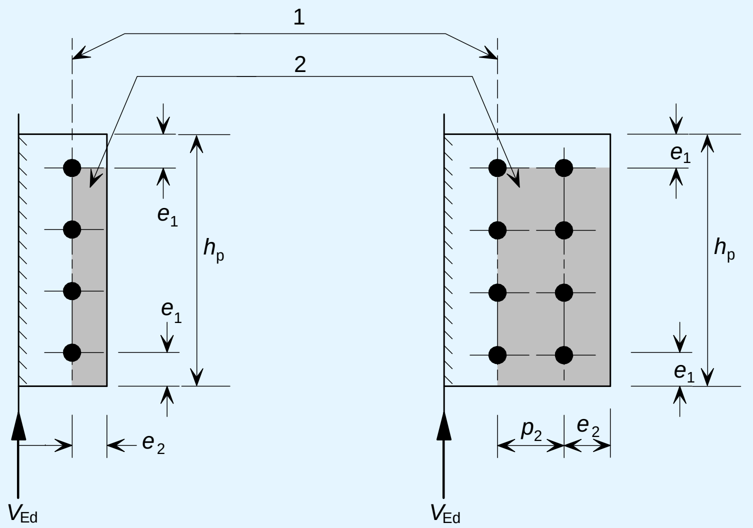
$$\alpha_b = \min\left(\frac{e_{1,b}}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,b}};\ 1{,}0\right)$$

For $F_{b,hor,Rd}$:

$$k_1 = \min\left(2{,}8\frac{e_{1,b}}{d_0} - 1{,}7;\ 1{,}4\frac{p_1}{d_0} - 1{,}7;\ 2{,}5\right)$$

$$\alpha_b = \min\left(\frac{e_{2,b}}{3d_0};\ \frac{p_2}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,b}};\ 1{,}0\right)$$

3.2.2 Shear resistance of the fin plate



1 Critical section in shear and bending
2 Block shear – check failure by tearing out of shaded portion

Basic requirement: $V_{Ed} \leq V_{Rd,min}$

$V_{Rd,min} = \min(V_{Rd,g};\ V_{Rd,n};\ V_{Rd,b})$

3.2.2.1 Shear resistance of gross section

$$V_{Rd,g} = \frac{h_p t_p}{1{,}27}\frac{f_{y,p}}{\sqrt{3}\gamma_{M0}}$$ [Reference 8]

Note: The coefficient 1,27 takes into account the reduction in shear resistance due to the presence of the nominal in-plane bending which produces tension in the bolts[9].

3.2.2.2 Shear resistance of net section

$$V_{\mathrm{Rd,n}} = A_{\mathrm{v,net}} \frac{f_{\mathrm{u,p}}}{\sqrt{3}\gamma_{\mathrm{M2}}}$$ [Reference 8]

$$A_{\mathrm{v,net}} = t_{\mathrm{p}}\left(h_{\mathrm{p}} - n_1 d_0\right)$$

3.2.2.3 Block tearing resistance

$$V_{\mathrm{Rd,b}} = \frac{0,5 f_{\mathrm{u,p}} A_{\mathrm{nt}}}{\gamma_{\mathrm{M2}}} + \frac{f_{\mathrm{y,p}} A_{\mathrm{nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}}$$ [Reference 8]

where:

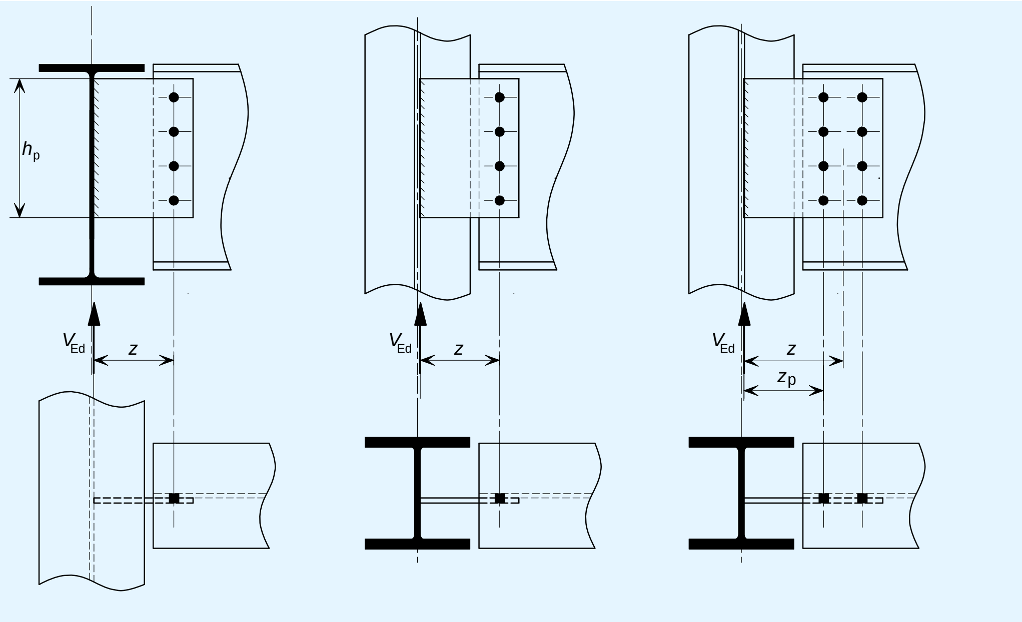
For a single vertical line of bolts, $A_{\mathrm{nt}} = t_{\mathrm{p}}\left(e_2 - 0,5 d_0\right)$

For a double vertical line of bolts, $A_{\mathrm{nt}} = t_{\mathrm{p}}\left(e_2 + p_2 - \frac{3}{2} d_0\right)$

$$A_{\mathrm{nv}} = t_{\mathrm{p}}\left(h_{\mathrm{p}} - e_1 - (n_1 - 0,5) d_0\right)$$

γ_{M2} is the partial factor for the resistance of net sections

3.2.3 Bending resistance of the fin plate



Basic requirement: $V_{\mathrm{Ed}} \leq V_{\mathrm{Rd}}$

If $h_{\mathrm{p}} \geq 2,73 \times z$ then $V_{\mathrm{Rd}} = \infty$ [Reference 8]

Otherwise $V_{\mathrm{Rd}} = \frac{W_{\mathrm{el,p}}}{z} \frac{f_{\mathrm{y,p}}}{\gamma_{\mathrm{M0}}}$

where:

$$W_{\mathrm{el,p}} = \frac{t_{\mathrm{p}} h_{\mathrm{p}}^{2}}{6}$$

3.2.4 Buckling resistance of the fin plate

Lateral-torsional buckling of the fin plate[8].

Basic requirement: $V_{\mathrm{Ed}} \leq V_{\mathrm{Rd}}$

$$\text{If } z > \frac{t_{\mathrm{p}}}{0{,}15} \text{ then } V_{\mathrm{Rd}} = \min\left(\frac{W_{\mathrm{el,p}}}{z}\frac{f_{\mathrm{p,LT}}}{0{,}6\gamma_{\mathrm{M1}}}; \frac{W_{\mathrm{el,p}}}{z}\frac{f_{\mathrm{y,p}}}{\gamma_{\mathrm{M0}}}\right)$$

$$\text{Otherwise } V_{\mathrm{Rd}} = \frac{W_{\mathrm{el,p}}}{z}\frac{f_{\mathrm{y,p}}}{\gamma_{\mathrm{M0}}}$$

where:

$$W_{\mathrm{el,p}} = \frac{t_{\mathrm{p}} h_{\mathrm{p}}^{2}}{6}$$

$f_{\mathrm{p,LT}}$ is the lateral torsional buckling strength of the plate obtained from BS 5950-1 Table 17[10] (See Appendix A) and based on λ_{LT} as follows:

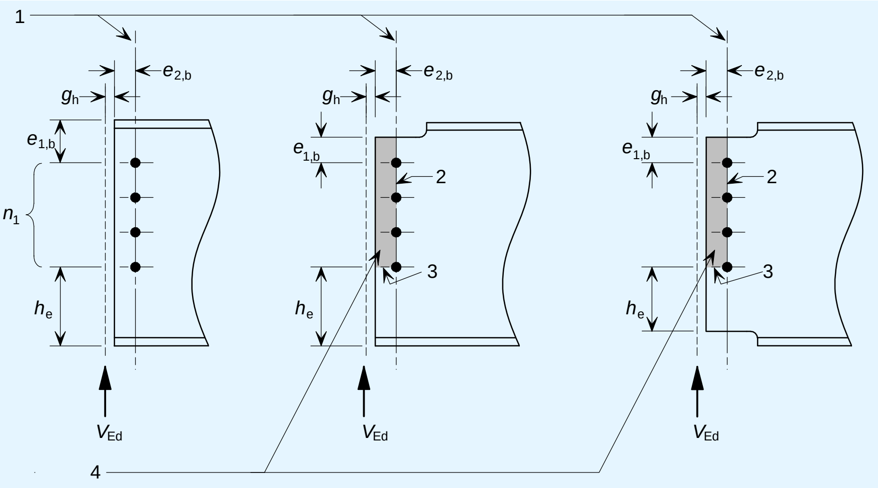
$$\lambda_{\mathrm{LT}} = 2{,}8\left(\frac{z_{\mathrm{p}} h_{\mathrm{p}}}{1{,}5 t_{\mathrm{p}}^{2}}\right)^{1/2}$$

z is the lever arm

z_{p} is the horizontal distance from the supporting web or flange to the first vertical bolt-row

3.2.5 Shear resistance of the beam web

3.2.5.1 Shear and block tearing resistance



1 Critical section in plain shear
2 Shear failure
3 Tension failure
4 Block shear failure tearing out of shaded portion

Basic requirement: $V_{\mathrm{Ed}} \leq V_{\mathrm{Rd,min}}$

$V_{\mathrm{Rd,min}} = \min(V_{\mathrm{Rd,g}}; V_{\mathrm{Rd,n}}; V_{\mathrm{Rd,b}})$

Shear resistance of gross section

$$V_{\mathrm{Rd,g}} = A_{\mathrm{v,wb}} \frac{f_{\mathrm{y,b}}}{\sqrt{3}\gamma_{\mathrm{M0}}}$$ [Reference 8]

where:

$A_{\mathrm{v,wb}} = A - 2bt_{\mathrm{f}} + (t_{\mathrm{w}} + 2r)t_{\mathrm{f}}$ but $\geq \eta\, h_{\mathrm{w}}t_{\mathrm{w}}$ for un-notched beam

$A_{\mathrm{v,wb}} = A_{\mathrm{Tee}} - bt_{\mathrm{f}} + (t_{\mathrm{w}} + 2r)t_{\mathrm{f}}/2$ for single notched beam

$A_{\mathrm{v,wb}} = t_{\mathrm{w}}\,(e_{1,\mathrm{b}} + (n_1 - 1)p_1 + h_{\mathrm{e}})$ for double notched beam

η is a factor from EN 1993-1-5 (it may be conservatively taken as 1,0. National Annex may give an alternative value)

A_{Tee} is the area of the Tee section

d_{nt} is the depth of the top notch

d_{nb} is the depth of the bottom notch

Shear resistance of net section

$$V_{\mathrm{Rd,n}} = A_{\mathrm{v,wb,net}} \frac{f_{\mathrm{u,b}}}{\sqrt{3}\gamma_{\mathrm{M2}}}$$ [Reference 8]

where:

$A_{\mathrm{v,wb,net}} = A_{\mathrm{v,wb}} - n_1 d_0 t_{\mathrm{w}}$

Block tearing resistance

$$V_{\mathrm{Rd,b}} = \frac{0,5\, f_{\mathrm{u,b}}\, A_{\mathrm{nt}}}{\gamma_{\mathrm{M2}}} + \frac{f_{\mathrm{y,b}}\, A_{\mathrm{nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}}$$ [Reference 8]

where:

For a single vertical line of bolts, $A_{\mathrm{nt}} = t_{\mathrm{w}}\left(e_{2,\mathrm{b}} - 0,5 d_0\right)$

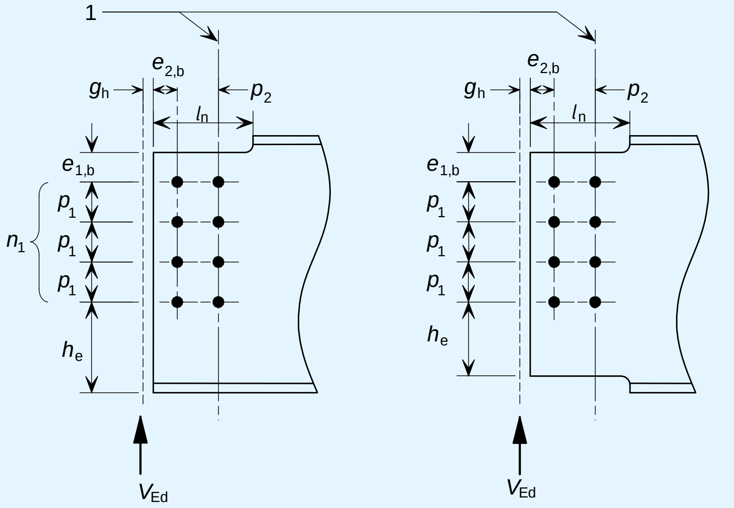
For a double vertical line of bolts, $A_{\mathrm{nt}} = t_{\mathrm{w}}\left(e_{2,\mathrm{b}} + p_2 - \frac{3}{2} d_0\right)$

For a notched beam $A_{\mathrm{nv}} = t_{\mathrm{w}}\left(e_{1,\mathrm{b}} + (n_1 - 1)\,p_1 - (n_1 - 0,5)\,d_0\right)$

For an un-notched beam $A_{\mathrm{nv}} = t_{\mathrm{w}}\left(e_{1,\mathrm{b}} + (n_1 - 1)\,p_1 - (n_1 - 1)d_0\right)$

γ_{M2} is the partial factor for the resistance of net sections.

3.2.5.2 Shear and bending interaction at the 2nd line of bolts, if the notch length $l_n > (e_{2,b} + p_2)$



1 Critical section in plain shear

Basic requirement: $V_{Ed}\,(g_h + e_{2,b} + p_2) \le M_{c,Rd}$

$M_{c,Rd}$ is the moment resistance of the notched beam at the connection in the presence of shear.

For single notched beam

For low shear (i.e. $V_{Ed} \le 0{,}5V_{pl,N,Rd}$)

$$M_{c,Rd} = \frac{f_{y,b}W_{el,N}}{\gamma_{M0}}$$ [Reference 4]

For high shear (i.e. $V_{Ed} > 0{,}5V_{pl,N,Rd}$)

$$M_{c,Rd} = \frac{f_{y,b}W_{el,N}}{\gamma_{M0}}\left[1-\left(\frac{2V_{Ed}}{V_{pl,N,Rd}}-1\right)^2\right]$$ [Reference 4]

$V_{pl,N,Rd} = \min(V_{Rd,g};V_{Rd,b})$

$W_{el,N}$ is the elastic section modulus of the gross Tee section at the notch

For double notched beam

For low shear (i.e. $V_{Ed} \le 0{,}5V_{pl,DN,Rd}$)

$$M_{c,Rd} = \frac{f_{y,b}t_w}{6\gamma_{M0}}\left(e_1 + (n_1-1)p_1 + h_e\right)^2$$ [Reference 4]

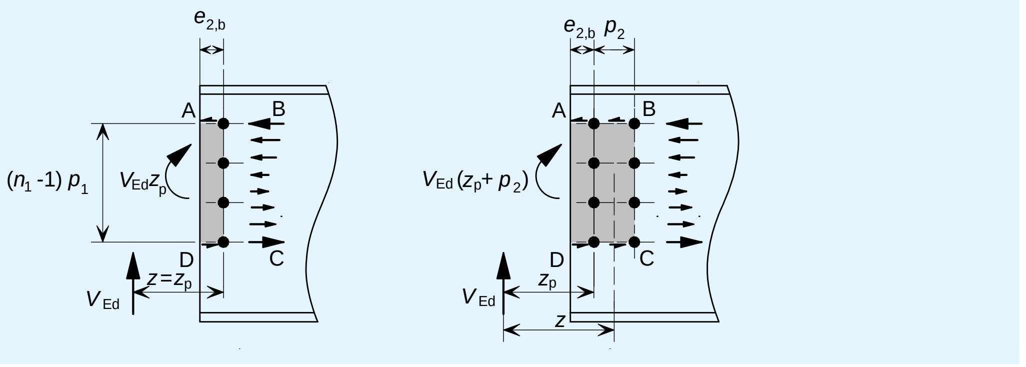
For high shear (i.e. $V_{Ed} > 0{,}5V_{pl,DN,Rd}$)

$$M_{c,Rd} = \frac{f_{y,b}t_w}{6\gamma_{M0}}\left[e_1 + (n_1-1)p_1 + h_e\right]^2\left[1-\left(\frac{2V_{Ed}}{V_{pl,DN,Rd}}-1\right)^2\right]$$ [Reference 4]

$V_{pl,DN,Rd} = \min(V_{Rd,g};V_{Rd,b})$

h_e is the distance between the bottom bolt row and the bottom of the section

3.2.5.3 Shear and bending interaction for un-notched beam



For short fin plates (i.e. $z \le t_p/0{,}15$) the resistance of the web does not need to be checked [4].

For long fin plates (i.e. $z > t_p/0{,}15$) it is necessary to ensure that the section labelled as ABCD in the figure can resist a moment $V_{Ed}z_p$ for a single line of bolts or $V_{Ed}(z_p+p_2)$ for a double line of bolts (AB and CD are in shear and BC is in bending).

Basic requirement:

For a single vertical line of bolts (n_2 = 1)

$V_{Ed}\, z_p \le M_{c,BC,Rd} + F_{pl,AB,Rd}\,(n_1 - 1)\, p_1$ [Reference 4]

For two vertical lines of bolts (n_2 = 2)

$V_{Ed}\,(z_p + p_2/2) \le M_{c,BC,Rd} + F_{pl,AB,Rd}\,(n_1 - 1)p_1$ [Reference 4]

$M_{c,BC,Rd}$ is the moment resistance of the beam web BC

For low shear (i.e. $V_{BC,Ed} \le 0{,}5F_{pl,BC,Rd}$)

$$M_{c,BC,Rd} = \frac{f_{y,b}t_w}{6\gamma_{M0}}\left[(n_1-1)p_1\right]^2$$

For high shear (i.e. $V_{BC,Ed} > 0{,}5F_{pl,BC,Rd}$)

$$M_{c,BC,Rd} = \frac{f_{y,b}t_w}{4\gamma_{M0}}\left[(n_1-1)p_1\right]^2\left[1-\left(\frac{2V_{Ed}}{V_{Rd,min}}-1\right)^2\right]$$

$F_{pl,AB,Rd}$ is the shear resistance of the beam web AB

$F_{pl,BC,Rd}$ is the shear resistance of the beam web BC

where:

For a single vertical line of bolts (n_2 = 1):

$$F_{pl,AB,Rd} = \min\left(\frac{e_{2,b}t_w f_{y,b}}{\sqrt{3}\,\gamma_{M0}};\ \frac{(e_{2,b}-d_0/2)t_w f_{u,b}}{\sqrt{3}\,\gamma_{M2}}\right)$$

$$F_{\mathrm{pl,BC,Rd}} = \min\left(\frac{(n_1-1)p_1 t_{\mathrm{w}} f_{\mathrm{y,b}}}{\sqrt{3}\,\gamma_{\mathrm{M0}}};\ \frac{[(n_1-1)p_1-(n_1-1)d_0]t_{\mathrm{w}} f_{\mathrm{u,b}}}{\sqrt{3}\,\gamma_{\mathrm{M2}}}\right)$$

For two vertical lines of bolts ($n_2 = 2$):

$$F_{\mathrm{pl,AB,Rd}} = \min\left(\frac{(e_{2,\mathrm{b}}+p_2)t_{\mathrm{w}} f_{\mathrm{y,b}}}{\sqrt{3}\,\gamma_{\mathrm{M0}}};\ \frac{(e_{2,\mathrm{b}}+p_2-3d_0/2)t_{\mathrm{w}} f_{\mathrm{u,b}}}{\sqrt{3}\,\gamma_{\mathrm{M2}}}\right)$$

$$F_{\mathrm{pl,BC,Rd}} = \min\left(\frac{(n_1-1)p_1 t_{\mathrm{w}} f_{\mathrm{y,b}}}{\sqrt{3}\,\gamma_{\mathrm{M0}}};\ \frac{[(n_1-1)p_1-(n_1-1)d_0]t_{\mathrm{w}} f_{\mathrm{u,b}}}{\sqrt{3}\,\gamma_{\mathrm{M2}}}\right)$$

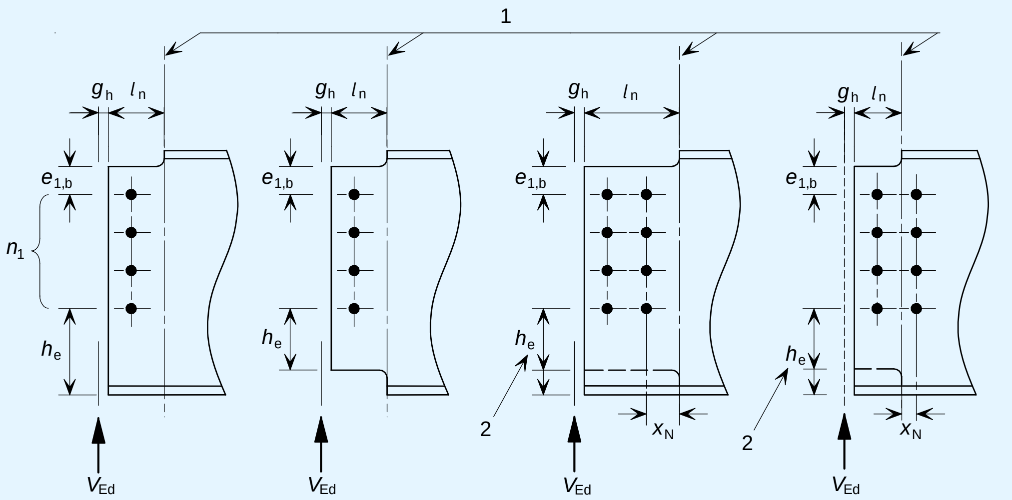
$V_{\mathrm{BC,Ed}}$ is the shear force on the beam web BC
$= V_{\mathrm{Ed}} - (V_{\mathrm{Rd,min}} - F_{\mathrm{pl,BC,Rd}})$ but ≥ 0

$V_{\mathrm{Rd,min}} = \min(V_{\mathrm{Rd,g}};\ V_{\mathrm{Rd,n}})$

z is the transverse distance from face of supporting element to the centre of bolt group.

γ_{M2} is the partial factor for the resistance of net sections.

3.2.6 Bending resistance at the notch



1 Critical sections
2 To notch or beam flange

3.2.6.1 For single bolt line or for double bolt lines, if $x_{\mathrm{N}} \geq 2d$:

$V_{\mathrm{Ed}}\,(g_{\mathrm{h}} + l_{\mathrm{n}}) \leq M_{\mathrm{v,N,Rd}}$ [Reference 4]

$M_{\mathrm{v,N,Rd}}$ is the moment resistance of the beam at the notch in the presence of shear

For single notched beam:

For low shear (i.e. $V_{\mathrm{Ed}} \leq 0{,}5V_{\mathrm{pl,N,Rd}}$)

$$M_{\mathrm{v,N,Rd}} = \frac{f_{\mathrm{y,b}} W_{\mathrm{el,N}}}{\gamma_{\mathrm{M0}}}$$

For high shear (i.e. $V_{Ed} > 0,5V_{pl,N,Rd}$)

$$M_{v,N,Rd} = \frac{f_{y,b} W_{el,N}}{\gamma_{M0}} \left[1 - \left(\frac{2V_{Ed}}{V_{pl,N,Rd}} - 1 \right)^2 \right]$$

For double notched beam:

For low shear (i.e. $V_{Ed} \leq 0,5V_{pl,DN,Rd}$)

$$M_{v,DN,Rd} = \frac{f_{y,b} t_w}{6\gamma_{M0}} \left(e_{1,b} + (n_1 - 1) p_1 + h_e \right)^2$$

For high shear (i.e. $V_{Ed} > 0,5V_{pl,DN,Rd}$)

$$M_{v,DN,Rd} = \frac{f_{y,b} t_w}{4\gamma_{M0}} \left(e_{1,b} + (n_1 - 1) p_1 + h_e \right)^2 \left[1 - \left(\frac{2V_{Ed}}{V_{pl,DN,Rd}} - 1 \right)^2 \right]$$

3.2.6.2 For double bolt lines, if $x_N < 2d$:

max (V_{Ed} ($g_h + l_n$); V_{Ed} ($g_h + e_{2,b} + p_2$)) $\leq M_{v,N,Rd}$ [Reference 4]

$M_{v,N,Rd} = M_{c,Rd}$ from the previous check

where:

$W_{el,N}$ is the elastic section modulus of the gross tee section at the notch

$V_{pl,N,Rd}$ is the shear resistance at the notch for single notched beams

$$= \frac{A_{v,N} f_{y,b}}{\sqrt{3}\,\gamma_{M0}}$$

$$A_{v,N} = A_{Tee} - bt_f + (t_w + 2r)\frac{t_f}{2}$$

$V_{pl,DN,Rd}$ is the shear resistance at the notch for double notched beams

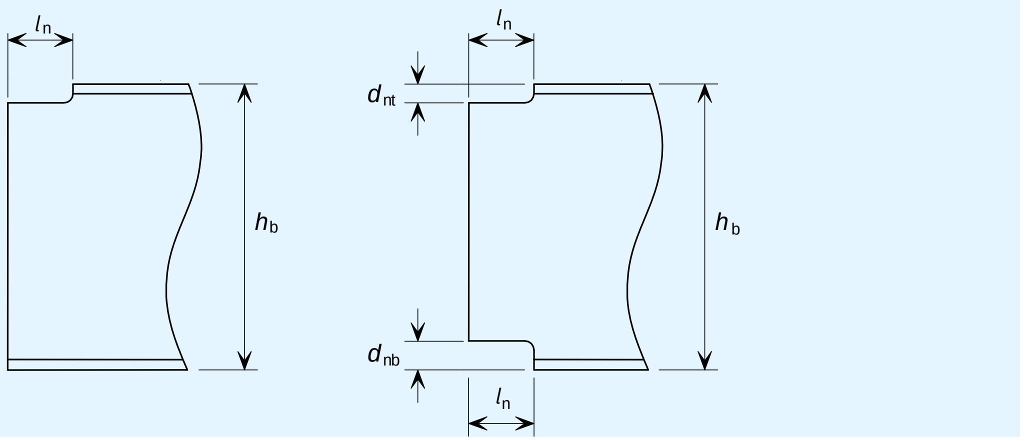
$$= \frac{A_{v,DN} f_{y,b}}{\sqrt{3}\,\gamma_{M0}}$$

$A_{v,DN} = t_w (e_{1,b} + (n_1 - 1) p_1 + h_e)$

h_e is the distance between the bottom bolt row and the bottom of the section

A_{Tee} is the area of the Tee section

3.2.7 Local stability of the notched beam



When the beam is restrained against lateral torsional buckling, no account need be taken of notch stability provided the following conditions are met:

For one flange notched, basic requirement:[5],[6]

d_{nt}	$\leq h_b / 2$	and:			
l_n	$\leq h_b$	for	h_b / t_w	$\leq 54{,}3$	(S275 steel)
l_n	$\leq \dfrac{160000\, h_b}{(h_b / t_w)^3}$	for	h_b / t_w	$> 54{,}3$	(S275 steel)
l_n	$\leq h_b$	for	h_b / t_w	$\leq 48{,}0$	(S355 steel)
l_n	$\leq \dfrac{110000\, h_b}{(h_b / t_w)^3}$	for	h_b / t_w	$> 48{,}0$	(S355 steel)

For both flanges notched, basic requirement:[7]

$\max(d_{nt}, d_{nb}) \leq h_b / 5$		and:			
l_n	$\leq h_b$	for	h_b / t_w	$\leq 54{,}3$	(S275 steel)
l_n	$\leq \dfrac{160000\, h_b}{(h_b / t_w)^3}$	for	h_b / t_w	$> 54{,}3$	(S275 steel)
l_n	$\leq h_b$	for	h_b / t_w	$\leq 48{,}0$	(S355 steel)
l_n	$\leq \dfrac{110000\, h_b}{(h_b / t_w)^3}$	for	h_b / t_w	$> 48{,}0$	(S355 steel)

Where the notch length l_n exceeds these limits, either suitable stiffening should be provided or the notch should be checked to References 5, 6 and 7.
For S235 and S460 members see References 5, 6 and 7.

3.2.8 Weld resistance

Full strength symmetrical fillet welds are recommended.

For a full strength weld, the size of each throat should comply with the following requirement[8]:

$a \geq 0{,}46 t_p$ for S235 fin plate

$a \geq 0{,}48 t_p$ for S275 fin plate

$a \geq 0{,}55 t_p$ for S355 fin plate

$a \geq 0{,}75 t_p$ for S460 fin plate

where:

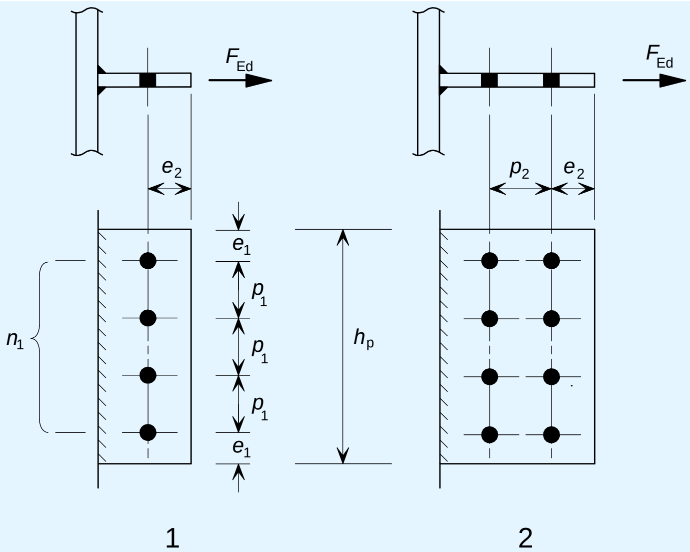
a is the weld throat thickness

The leg length is defined as follows: $s = a\sqrt{2}$

3.3 Checks for tying

EN 1993-1-8 does not have a partial factor for structural integrity checks. In this publication γ_{Mu} has been used. A value of $\gamma_{Mu} = 1{,}1$ is recommended.

3.3.1 Fin plate and bolt group resistance



1 Single line of bolts
2 Double line of bolts

3.3.1.1 Shear resistance of bolts

Basic requirement: $F_{Ed} \leq F_{Rd}$

$F_{Rd} = n_b F_{v,u}$ [Reference 8]

$$F_{v,u} = \frac{\alpha_v f_{ub} A}{\gamma_{Mu}}$$

where:

α_v = 0,6 for 4.6 and 8.8 bolts
= 0,5 for 10.9 bolts

A is the tensile stress area of bolt, A_s

3.3.1.2 Bearing resistance of bolts on the fin plate

Basic requirement: $F_{Ed} \leq F_{Rd}$

$F_{Rd} = n_b F_{b,hor,u,Rd}$

$$F_{b,hor,u,Rd} = \frac{k_1 \alpha_b f_{u,p} d t_p}{\gamma_{Mu}}$$ [Reference 8]

where:

$$k_1 = \min\left(2,8\frac{e_1}{d_0} - 1,7;\ 1,4\frac{p_1}{d_0} - 1,7;\ 2,5\right)$$

$$\alpha_b = \min\left(\frac{e_2}{3d_0};\ \frac{p_2}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,p}};\ 1,0\right)$$

3.3.1.3 Tension resistance of the fin plate

Basic requirement: $F_{Ed} \leq F_{Rd}$

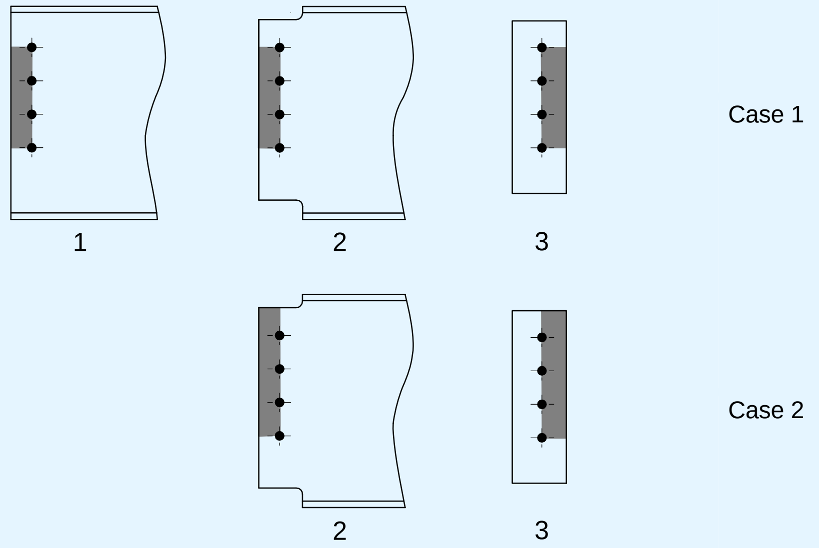
$F_{Rd} = \min(F_{Rd,n}; F_{Rd,b})$

Tension resistance of net section

$$F_{Rd,n} = 0,9 A_{net} \frac{f_{u,p}}{\gamma_{Mu}}$$ [Reference 8]

$$A_{net} = t_p \left(h_p - d_0 n_1\right)$$

Block tearing resistance



1 beam without notch
2 notched beam
3 fin plate

$$F_{\mathrm{Rd,b}} = \frac{f_{\mathrm{u,p}} A_{\mathrm{nt}}}{\gamma_{\mathrm{Mu}}} + \frac{f_{\mathrm{y,p}} A_{\mathrm{nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}}$$ [Reference 8]

Case 1:

$$A_{\mathrm{nt}} = t_{\mathrm{p}}\left((n_1 - 1)p_1 - (n_1 - 1)d_0\right)$$

For a single vertical line of bolts: $A_{\mathrm{nv}} = 2t_{\mathrm{p}}\left(e_2 - 0{,}5d_0\right)$

For a double vertical line of bolts: $A_{\mathrm{nv}} = 2t_{\mathrm{p}}\left(e_2 + p_2 - \frac{3}{2}d_0\right)$

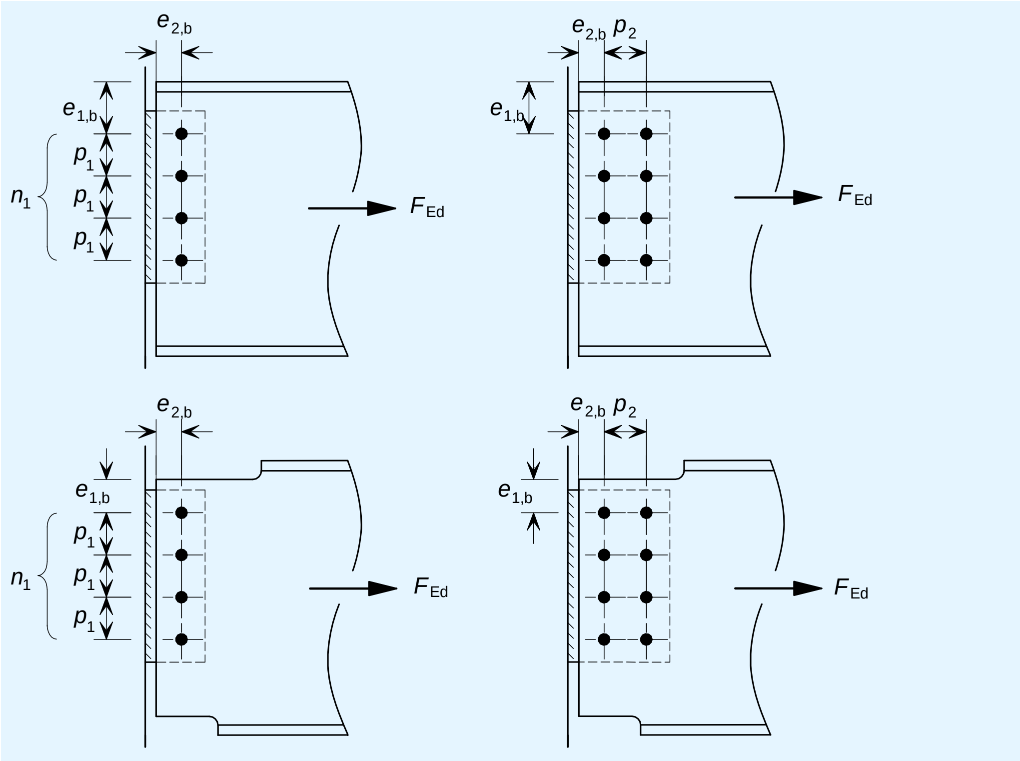
Case 2:

$$A_{\mathrm{nt}} = t_{\mathrm{p}}\left(e_1 + (n_1 - 1)p_1 - (n_1 - 0{,}5)d_0\right)$$

For a single vertical line of bolts, $A_{\mathrm{nv}} = t_{\mathrm{p}}\left(e_2 - 0{,}5d_0\right)$

For a double vertical line of bolts, $A_{\mathrm{nv}} = t_{\mathrm{p}}\left(e_2 + p_2 - \frac{3}{2}d_0\right)$

3.3.2 Beam web resistance



3.3.2.1 Bearing resistance of bolts on the beam web

Basic requirement: $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd}}$

$$F_{\mathrm{Rd}} = n_{\mathrm{b}} F_{\mathrm{b,hor,u,Rd}}$$

$$F_{\mathrm{b,hor,u,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,b}} d t_{\mathrm{w}}}{\gamma_{\mathrm{Mu}}}$$

where:

$$k_1 = \left(2{,}8 \frac{e_{1,\mathrm{b}}}{d_0} - 1{,}7;\ 1{,}4 \frac{p_1}{d_0} - 1{,}7;\ 2{,}5 \right)$$

$$\alpha_{\mathrm{b}} = \left(\frac{e_{2,\mathrm{b}}}{3d_0};\ \frac{p_2}{3d_0} - \frac{1}{4};\ \frac{f_{\mathrm{ub}}}{f_{\mathrm{u,b}}};\ 1{,}0 \right)$$

α_{v} = 0,6 for 4.6 and 8.8 bolts

= 0,5 for 10.9 bolts

3.3.2.2 Tension resistance of the beam web

Basic requirement: $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd}}$

$$F_{\mathrm{Rd}} = \min(F_{\mathrm{Rd,n}};\ F_{\mathrm{Rd,b}})$$

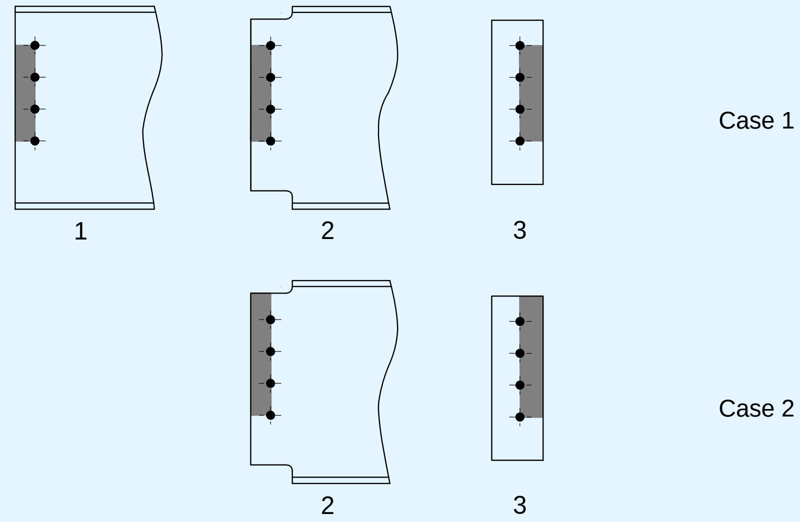
Tension resistance of net section

$$F_{\mathrm{Rd,n}} = 0{,}9\,A_{\mathrm{net,wb}}\frac{f_{\mathrm{u,b}}}{\gamma_{\mathrm{Mu}}}$$

$$A_{\mathrm{net}} = t_{\mathrm{w}} h_{\mathrm{wb}} - d_0 n_1 t_{\mathrm{w}}$$

h_{wb} may be taken as the depth of the fin plate (it is conservative)

Block tearing resistance



1 beam without notch
2 notched beam
3 fin plate

$$F_{\mathrm{Rd,b}} = \frac{f_{\mathrm{u,b}} A_{\mathrm{nt}}}{\gamma_{\mathrm{Mu}}} + \frac{f_{\mathrm{y,b}} A_{\mathrm{nv}} / \sqrt{3}}{\gamma_{\mathrm{M0}}}$$

Case 1:

$$A_{\mathrm{nt}} = t_{\mathrm{w}}\left(\left(n_1 - 1\right)p_1 - \left(n_1 - 1\right)d_0\right)$$

For a single vertical line of bolts, $A_{\mathrm{nv}} = 2t_{\mathrm{w}}\left(e_{2,\mathrm{b}} - 0{,}5d_0\right)$

For a double vertical line of bolts, $A_{\mathrm{nv}} = 2t_{\mathrm{w}}\left(e_{2,\mathrm{b}} + p_2 - \frac{3}{2}d_0\right)$

Case 2 (for notched beam only):

$$A_{\mathrm{nt}} = t_{\mathrm{w}}\left(e_{1,\mathrm{b}} + \left(n_1 - 1\right)p_1 - \left(n_1 - 0{,}5\right)d_0\right)$$

For a single vertical line of bolts, $A_{\mathrm{nv}} = t_{\mathrm{w}}\left(e_{2,\mathrm{b}} - 0{,}5d_0\right)$

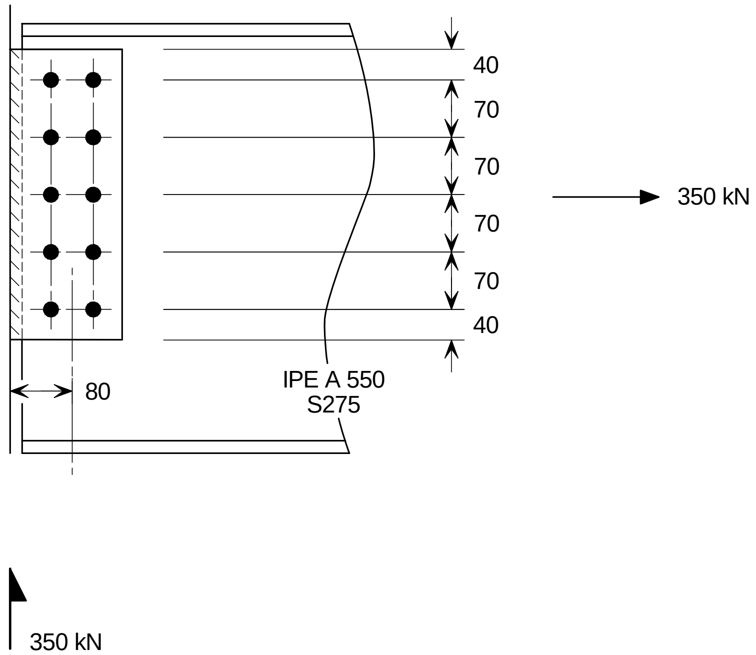
For a double vertical line of bolts, $A_{\mathrm{nv}} = t_{\mathrm{w}}\left(e_{2,\mathrm{b}} + p_2 - \frac{3}{2}d_0\right)$

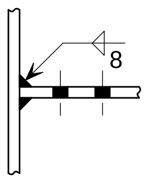
3.3.3 Weld resistance

The weld size specified for shear will be adequate for tying resistance, as it is full strength.

3. Fin plate

Details and data





Beam: IPE A 550 S275

Fin plate: 360 × 160 × 10 S275

Bolts: M20 8.8

Welds: 8 mm fillet (weld throat, a = 5,6 mm)

Summary of full design checks

Design forces

V_{Ed} = 350 kN

F_{Ed} = 350 kN (Tie force)

Shear resistances

Bolt group resistance	
Shear resistance of bolts	584 kN
Bearing resistance of bolts on the fin plate	605 kN
Bearing resistance of bolts on the beam web	624 kN
Shear resistance of the fin plate	450 kN
Bending resistance of the fin plate	∞
Buckling resistance of the fin plate	743 kN
Shear resistance of the beam web	
Shear and block tearing resistance	545 kN
Shear and bending interaction at the 2nd line of bolts	N/A
Shear and bending interaction for un-notched beam	66 kNm
Bending resistance at the notch	N/A
Local stability of the notched beam	N/A
Weld resistance	OK

Tying resistances

Fin plate and bolt group resistance	
Shear resistance of bolts	1070 kN
Bearing resistance of bolts on the fin plate	1290 kN
Tension resistance of the fin plate	880 kN
Beam web resistance	
Bearing resistance of bolts on the beam web	1070 kN
Tension resistance of the beam web	792 kN
Weld resistance	OK

3.1. Recommended details

Unless noted otherwise, all references are to EN 1993-1-8

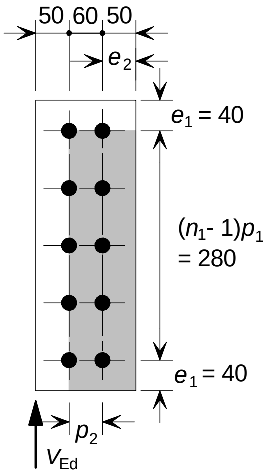
Fin plate thickness: t_p = 10mm $\leq 0{,}5d$

Height of fin plate: h_p = 360 mm $> 0{,}6h_b$

3.2. Checks for vertical shear

3.2.1. Bolt group resistance

3.2.1.1. Shear resistance of bolts



Basic requirement: $V_{Ed} \leq V_{Rd}$

Ref (3)

$$V_{Rd} = \frac{n_b F_{v,Rd}}{\sqrt{(1+\alpha n_b)^2 + (\beta n_b)^2}}$$

Table 3.4

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

For M20 8.8 bolts, $F_{v,Rd} = \dfrac{0.6 \times 800 \times 245}{1{,}25} \times 10^{-3}$ = 94 kN

For a double vertical line of bolts (i.e. n_2 = 2 and n_1 = 5),

$$\alpha = \frac{zp_2}{2I}$$

$$I = \frac{n_1}{2} p_2^2 + \frac{1}{6} n_1 \left(n_1^2 - 1\right) p_1^2 = \frac{5}{2} 60^2 + \frac{1}{6} 5\left(5^2 - 1\right) 70^2 = 107000 \text{ mm}^2$$

$$\alpha = \frac{80 \times 60}{2 \times 107000} = 0{,}022$$

And $\beta = \dfrac{zp_1}{2I}(n_1 - 1) = \dfrac{80 \times 70}{2 \times 107000}(5-1) = 0{,}105$

Thus $V_{Rd} = \dfrac{10 \times 94}{\sqrt{(1 + 0{,}022 \times 10)^2 + (0{,}105 \times 10)^2}}$ = 584 kN

$V_{Ed} = 350 \text{ kN} \leq 584 \text{ kN}$, OK

3.2.1.2. Bearing resistance of bolts on the fin plate

Basic requirement: $V_{Ed} \leq V_{Rd}$

Ref [3]

$$V_{Rd} = \frac{n_b}{\sqrt{\left(\frac{1+\alpha n_b}{F_{b,ver,Rd}}\right)^2 + \left(\frac{\beta n_b}{F_{b,hor,Rd}}\right)^2}}$$

α = 0,022 and β = 0,105, as above

Table 3.4

The vertical bearing resistance of a single bolt, $F_{b,ver,Rd} = \dfrac{k_1 \alpha_b f_{u,p} d t_p}{\gamma_{M2}}$

$$k_1 = \min\left(2{,}8\frac{e_2}{d_0} - 1{,}7;\ 1{,}4\frac{p_2}{d_0} - 1{,}7;\ 2{,}5\right)$$

$$= \min\left(2{,}8\frac{50}{22} - 1{,}7;\ 1{,}4\frac{60}{22} - 1{,}7;\ 2{,}5\right) = \min(4{,}67;\ 2{,}12;\ 2{,}5) = 2{,}12$$

$$\alpha_b = \min\left(\frac{e_1}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,p}};\ 1{,}0\right) = \min\left(\frac{40}{3\times 22};\ \frac{70}{3\times 22} - \frac{1}{4};\ \frac{800}{430};\ 1{,}0\right)$$

$$= \min(0{,}61;\ 0{,}81;\ 1{,}86;\ 1{,}0) = 0{,}61$$

$$F_{b,ver,Rd} = \frac{2{,}12 \times 0{,}61 \times 430 \times 20 \times 10}{1{,}25} \times 10^{-3} = 89 \text{ kN}$$

Table 3.4

The horizontal bearing resistance of a single bolt, $F_{b,hor,Rd} = \dfrac{k_1 \alpha_b f_{u,p} d t_p}{\gamma_{M2}}$

$$k_1 = \min\left(2{,}8\frac{e_1}{d_0} - 1{,}7;\ 1{,}4\frac{p_1}{d_0} - 1{,}7;\ 2{,}5\right)$$

$$= \min\left(2{,}8\frac{40}{22} - 1{,}7;\ 1{,}4\frac{70}{22} - 1{,}7;\ 2{,}5\right) = \min(3{,}39;\ 2{,}75;\ 2{,}5) = 2{,}5$$

$$\alpha_b = \min\left(\frac{e_2}{3d_0};\ \frac{p_2}{3d_0} - 0{,}25;\ \frac{f_{ub}}{f_{u,p}};\ 1{,}0\right)$$

$$= \min\left(\frac{50}{3\times 22};\ \frac{60}{3\times 22} - 0{,}25;\ \frac{800}{430};\ 1{,}0\right) = \min(0{,}75;\ 0{,}66;\ 1{,}0) = 0{,}66$$

$$F_{b,hor,Rd} = \frac{2{,}5 \times 0{,}66 \times 430 \times 20 \times 10}{1{,}25} \times 10^{-3} = 114 \text{ kN}$$

$$V_{Rd} = \frac{10}{\sqrt{\left(\frac{1+0{,}022\times 10}{89}\right)^2 + \left(\frac{0{,}105\times 10}{114}\right)^2}} = 605 \text{ kN}$$

$V_{Ed} = 350$ kN ≤ 605 kN, OK

3.2.1.3. Bearing resistance of bolts on the beam web

Basic requirement: $V_{Ed} \leq V_{Rd}$

Ref (3)

$$V_{Rd} = \frac{n_b}{\sqrt{\left(\frac{1+\alpha n_b}{F_{b,ver,Rd}}\right)^2 + \left(\frac{\beta n_b}{F_{b,hor,Rd}}\right)^2}}$$

α = 0,022 and β = 0,105, as above

The vertical bearing resistance for a single bolt, $F_{b,ver,Rd} = \dfrac{k_1 \alpha_b f_{u,b} d t_w}{\gamma_{M2}}$

Table 3.4

$$k_1 = \min\left(2,8\frac{e_2}{d_0} - 1,7;\ 1,4\frac{p_2}{d_0} - 1,7;\ 2,5\right)$$

$$= \min\left(2,8\frac{40}{22} - 1,7;\ 1,4\frac{60}{22} - 1,7;\ 2,5\right) = \min(3,39;\ 2,12;\ 2,5) = 2,12$$

$$\alpha_b = \min\left(\frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,b}};\ 1,0\right) = \min\left(\frac{70}{3\times 22} - \frac{1}{4};\ \frac{800}{430};\ 1,0\right)$$

$$= \min(0,81;\ 1,86;\ 1,0) = 0,81$$

$$F_{b,ver,Rd} = \frac{2,12 \times 0,81 \times 430 \times 20 \times 9}{1,25} \times 10^{-3} = 106 \text{ kN}$$

The horizontal bearing resistance for a single bolt, $F_{b,hor,Rd} = \dfrac{k_1 \alpha_b f_{u,b} d t_w}{\gamma_{M2}}$

Table 3.4

$$k_1 = \min\left(1,4\frac{p_1}{d_0} - 1,7;\ 2,5\right) = \min\left(1,4\frac{70}{22} - 1,7;\ 2,5\right)$$

$$= \min(2,75;\ 2,5) = 2,5$$

$$\alpha_b = \min\left(\frac{e_{2,b}}{3d_0};\ \frac{p_2}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,b}};\ 1,0\right) = \min\left(\frac{40}{3\times 22};\ \frac{60}{3\times 22} - \frac{1}{4};\ \frac{800}{430};\ 1,0\right)$$

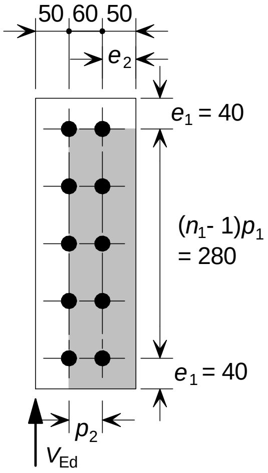
$$= \min(0,61;\ 0,81;\ 1,86;\ 1,0) = 0,61$$

$$F_{b,hor,Rd} = \frac{2,5 \times 0,61 \times 430 \times 20 \times 9}{1,25} \times 10^{-3} = 94 \text{ kN}$$

$$V_{Rd} = \frac{10}{\sqrt{\left(\frac{1+0,022\times 10}{106}\right)^2 + \left(\frac{0,105\times 10}{94}\right)^2}} = 624 \text{ kN}$$

V_{Ed} = 350 kN $\leq$ 624 kN, OK

3.2.2. Shear resistance of the fin plate



Basic requirement: $V_{Ed} \leq V_{Rd,min}$

$V_{Rd,min} = \min(V_{Rd,g}; V_{Rd,n}; V_{Rd,b})$

3.2.2.1. Shear resistance of gross section

Ref (8)

$$V_{Rd,g} = \frac{h_p t_p}{1,27} \frac{f_{y,p}}{\sqrt{3}\gamma_{M0}} = \frac{360 \times 10 \times 275}{1,27 \times \sqrt{3} \times 1,0} \times 10^{-3} = 450 \text{ kN}$$

3.2.2.2. Shear resistance of net section

Ref [8]

$$V_{Rd,n} = A_{v,net} \frac{f_{u,p}}{\sqrt{3}\gamma_{M2}}$$

Net area, $A_{v,net} = t_p \left(h_p - nd_0\right) = = 10\left(360 - 5 \times 22\right) = 2500 \text{ mm}^2$

$$V_{Rd} = 2500 \times \frac{430}{\sqrt{3} \times 1,25} \times 10^{-3} = 497 \text{ kN}$$

3.2.2.3. Block tearing resistance

Ref [8]

$$V_{Rd,b} = \left(\frac{0,5 f_{u,p} A_{nt}}{\gamma_{M2}} + \frac{f_{y,p} A_{nv}}{\sqrt{3}\gamma_{M0}} \right) \times 10^{-3}$$

Net area subject to tension, $A_{nt} = t_p\left(p_2 + e_2 - 1{,}5 d_0\right)$

$= 10(60 + 50 - 1{,}5 \times 22) = 770 \text{ mm}^2$

Net area subject to shear, $A_{nt} = t_p\left(h_p - e_1 - (n_1 - 0{,}5) d_0\right)$

$= 10(360 - 40 - (5 - 0{,}5)22) = 2210 \text{ mm}^2$

$$V_{Rd,b} = \frac{0,5 \times 430 \times 770}{1,25} + \frac{275 \times 2210}{\sqrt{3} \times 1,0} = 483 \text{ kN}$$

$V_{Rd,min} = \min(450; 497; 483) = 450$ kN

$V_{Ed} = 350 \text{ kN} \leq 450 \text{ kN}$, OK

3.2.3. Bending resistance of the fin plate

Basic requirement: $V_{Ed} \leq V_{Rd}$ — Ref [8]

$2,73 \times z = 2,73 \times 80 = 218$ mm

$h_p = 360 \text{ mm} > 218 \text{ mm}$

Then $V_{Rd} = \infty$

$V_{Ed} \leq V_{Rd}$, OK

3.2.4. Buckling resistance of the fin plate

Basic requirement: $V_{Ed} \leq V_{Rd}$

Ref [8]

$$t_p/0,15 = \frac{10}{0,15} = 67 \text{ mm}$$

$z = 80 \text{ mm} > 67 \text{ mm}$

$f_{p,LT}$ from BS5950-1 Table 17 (See Appendix A)

$$V_{Rd} = \min\left(\frac{W_{el,p}}{z}\frac{f_{p,LT}}{0,6\gamma_{M1}}; \frac{W_{el,p}}{z}\frac{f_{y,p}}{\gamma_{M0}}\right)$$

$$W_{el,p} = \frac{t_p h_p^{\ 2}}{6} = \frac{10 \times 360^2}{6} = 216000 \text{ mm}^3$$

$z_p = 80$ mm

$$\lambda_{LT} = 2,8\left(\frac{z_p h_p}{1,5 t_p^{\ 2}}\right)^{1/2} = 2,8\left(\frac{50 \times 360}{1,5 \times 10^2}\right)^{1/2} = 31$$

$f_{p,LT}$ is obtained by interpolation from Appendix A.

$f_{p,LT} = 274 \text{ N/mm}^2$

$$V_{Rd} = \min\left(\frac{216000}{80}\frac{274}{0,6 \times 1,0} \times 10^{-3}; \frac{216000}{80}\frac{275}{1,0} \times 10^{-3}\right)$$

$$= \min(1233; 743) = 743 \text{ kN}$$

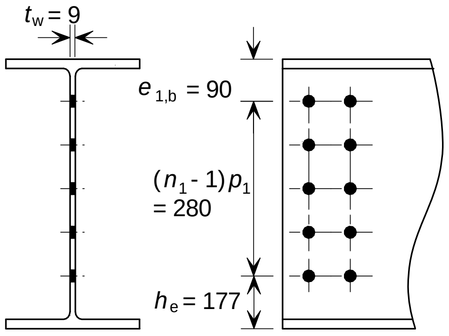
$V_{Ed} = 350 \text{ kN} \leq 743 \text{ kN}$, OK

3.2.5. Shear resistance of the beam web

3.2.5.1. Shear and block tearing resistance

Basic requirement: $V_{Ed} \leq V_{Rd,min}$

$V_{Rd,min} = \min(V_{Rd,g}; V_{Rd,n}; V_{Rd,b})$



Shear resistance of gross section

Ref [8]

$$V_{Rd,g} = A_{v,wb} \frac{f_{y,b}}{\sqrt{3}\gamma_{M0}}$$

Shear area of beam web,

$$A_{v,wb} = A - 2bt_f + (t_w + 2r)t_f$$

$$= 11700 - 2 \times 210 \times 15,7 + (9 + 2 \times 24)15,7 = 6001 \text{ mm}^2$$

$$\eta\, h_w t_w = 1,0 \times 515,6 \times 9 = 4640 \text{ mm}^2$$

$$V_{Rd,g} = \frac{6001 \times 275}{\sqrt{3} \times 1,0} \times 10^{-3} = 953 \text{ kN}$$

Shear resistance of net section

Ref [8]

$$V_{Rd,n} = A_{v,wb,net} \frac{f_{u,b}}{\sqrt{3}\gamma_{M2}}$$

Net area, $A_{v,wb,net} = A - n_1 d_0 t_w = 6001 - 5 \times 22 \times 9 = 5011 \text{ mm}^2$

$$V_{Rd,n} = 5011 \times \frac{430}{\sqrt{3} \times 1,25} \times 10^{-3} = 995 \text{ kN}$$

Block tearing resistance

Ref [8]

$$V_{Rd,b} = \frac{0,5 f_{u,b} A_{nt}}{\gamma_{M2}} + \frac{f_{y,b} A_{nv}}{\sqrt{3}\gamma_{M0}}$$

Net area subject to tension, $A_{nt} = t_p\left(p_2 + e_{2,b} - 1,5d_0\right)$

$$= 9(60 + 40 - 1,5 \times 22) = 603 \text{ mm}^2$$

Net area subject to shear, $A_{nv} = t_p\left(e_{1,b} + (n_1 - 1)p_1 - (n_1 - 0,5)d_0\right)$

$$= 9(90 + (5-1)70 - (5-1)22)$$

$$= 2538 \text{ mm}^2$$

$$V_{Rd,b} = \left(\frac{0,5 \times 430 \times 603}{1,25} + \frac{275 \times 2538}{\sqrt{3} \times 1,0}\right) \times 10^{-3} = 507 \text{ kN}$$

$V_{Rd,min}$ = min(953; 995; 507) = 507 kN

V_{Ed} = 350 kN $\leq$ 507 kN, OK

3.2.5.2. Shear and bending interaction at the 2nd line of bolts

Not applicable

3.2.5.3. Shear and bending interaction of un-notched beam

Shear and bending interaction at the beam web

Ref [4]

$$\frac{t_p}{0{,}15} = \frac{10}{0{,}15} = 67 \text{ mm}$$

z = 80 mm > 67 mm

Therefore this check is required.

Basic requirement: $V_{Ed}\,(z + p_2/2) \leq M_{cBC,Rd} + F_{pl,AB,Rd}\,(n_1 - 1)\,p_1$

$$F_{pl,BC,Rd} = \min\left(\frac{(n_1-1)p_1 t_w f_{y,b}}{\sqrt{3}\,\gamma_{M0}};\ \frac{[(n_1-1)p_1 - (n_1-1)d_0]t_w f_{u,b}}{\sqrt{3}\,\gamma_{M2}}\right)$$

$$F_{pl,BC,Rd} = \min\left(\frac{(5-1)70\times 9\times 275}{\sqrt{3}\times 1{,}0}\times 10^{-3};\ \frac{[(5-1)70-(5-1)22]9\times 430}{\sqrt{3}\times 1{,}25}\times 10^{-3}\right)$$

$= \min(400;343) = 343$ kN

$V_{BC,Ed}$ is the shear force on the beam web BC

$V_{BC,Ed} = V_{Ed} - (V_{Rd,min} - F_{pl,BC,Rd})$ but ≥ 0

$V_{BC,Ed} = 350 - (953 - 343) = -260$ kN

Therefore $V_{BC,Ed} = 0$ kN

As $V_{BC,Ed} \leq 0{,}5 F_{pl,BC,Rd}$ then $M_{c,BC,Rd} = \dfrac{f_{y,b} t_w}{6\gamma_{M0}}[(n_1-1)p_1]^2$

$$M_{c,BC,Rd} = \frac{275\times 9}{6\times 1{,}0}((5-1)70)^2\times 10^{-6} = 32 \text{ kNm}$$

$$F_{pl,AB,Rd} = \min\left(\frac{(e_{2,b}+p_2)t_w f_{y,b}}{\sqrt{3}\,\gamma_{M0}};\ \frac{(e_{2,b}+p_2-3d_0/2)t_w f_{u,b}}{\sqrt{3}\,\gamma_{M2}}\right)$$

$$= \min\left(\frac{(40+60)9\times 275}{\sqrt{3}\times 1{,}0}\times 10^{-3};\ \frac{(40+60-3\times 22/2)9\times 430}{\sqrt{3}\times 1{,}25}\times 10^{-3}\right)$$

$= \min(143;\ 120) = 120$ kN

$M_{cBC,Rd} + F_{pl,AB,Rd}\,(n_1 - 1)\,p_1 = 32 + 120(5 - 1)70\times 10^{-3} = 66$ kNm

$V_{Ed}\,(z + p_2/2) = 350(80 + 60/2)\times 10^{-3} = 38$ kNm

Therefore $V_{Ed}\,(z + p_2/2) \leq M_{cBC,Rd} + F_{pl,AB,Rd}\,(n_1 - 1)\,p_1$ OK

3.2.6. Bending resistance at the notch

Not applicable

3.2.7. Local stability of the notched beam

Not applicable

3.2.8. Weld resistance

For a fin plate in S275 steel

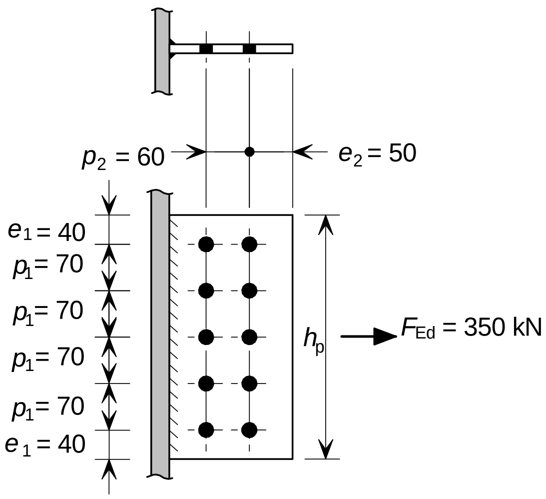
Basic requirement: $a \geq 0{,}48t_p$ — Ref [8]

$0{,}48t_p \quad = 0{,}48 \times 10 \; = 4{,}8$ mm

$a \quad = 5{,}7 \text{ mm} \geq 0{,}48t_p$ OK

3.3. Checks for tying

3.3.1. Fin plate and bolt group resistance



3.3.1.1. Shear resistance of bolts

Basic requirement: $F_{Ed} \leq F_{Rd}$

$F_{Rd} \quad = n_b F_{v,u}$ — Ref [8]

$$F_{v,u} \quad = \frac{\alpha_v f_{ub} A}{\gamma_{Mu}} = \frac{0{,}6 \times 800 \times 245}{1{,}1} \times 10^{-3} = 107 \text{ kN}$$

$F_{Rd} \quad = 10 \times 107 \; = 1070$ kN

$F_{Ed} \quad = 350 \text{ kN} \leq 1070$ kN OK

3.3.1.2. Bearing resistance of bolts on the fin plate

Basic requirement: $F_{Ed} \leq F_{Rd}$

$F_{Rd} \quad = n_b F_{b,hor,u,Rd}$ — Ref [8]

$$F_{b,hor,u,Rd} = \frac{k_1 \alpha_b f_{u,p} d t_p}{\gamma_{Mu}}$$

$$k_1 = \min\left(2,8\frac{e_1}{d_0}-1,7;\ 1,4\frac{p_1}{d_0}-1,7;\ 2,5\right)$$

$$= \min\left(2,8\frac{40}{22}-1,7;\ 1,4\frac{70}{22}-1,7;\ 2,5\right)=\min\left(3,39;2,75;2,5\right) = 2,5$$

$$\alpha_b = \min\left(\frac{e_2}{3d_0};\ \frac{p_2}{3d_0}-\frac{1}{4};\ \frac{f_{ub}}{f_{u,p}};\ 1,0\right)=\min\left(\frac{50}{3\times22};\ \frac{60}{3\times22}-\frac{1}{4};\ \frac{800}{430};\ 1,0\right)$$

$$= \min(0,75;\ 0,66;\ 1,86;\ 1,0) = 0,66$$

$$F_{b,hor,u,Rd} = \frac{2,5\times0,66\times430\times20\times10}{1,1}\times10^{-3} = 129\text{ kN}$$

$$F_{Rd} = 10\times129 = 1290\text{ kN}$$

$F_{Ed} = 350\text{ kN} \leq 1290\text{ kN}$, OK

3.3.1.3. Tension resistance of the fin plate

Basic requirement : $F_{Ed} \leq F_{Rd}$

$$F_{Rd} = \min\left(F_{Rd,b};F_{Rd,n}\right)$$

Tension resistance of net section

Ref [8]

$$F_{Rd,n} = 0,9A_{net}\frac{f_{u,p}}{\gamma_{Mu}}$$

$$A_{net} = t_p\left(h_p - d_0 n_1\right) = 10\left(360-22\times5\right) = 2500\text{ mm}^2$$

$$F_{Rd,n} = 0,9\times2500\frac{430}{1,1}\times10^{-3} = 880\text{ kN}$$

Block tearing resistance

Ref [8]

Case 1

$$F_{Rd,b} = \frac{f_{u,p}A_{nt}}{\gamma_{Mu}}+\frac{f_{y,p}A_{nv}}{\sqrt{3}\gamma_{M0}}$$

$$A_{nt} = t_p[(n_1-1)p_1-(n_1-1)d_0] = 10[(5-1)\times70-(5-1)22] = 1920\text{ mm}^2$$

$$A_{nv} = 2t_p\left(e_2+p_2-\frac{3}{2}d_0\right) = 2\times10\left(50+60-\frac{3}{2}\times22\right) = 1540\text{ mm}^2$$

$$F_{Rd,b} = \left(\frac{430\times1920}{1,1}+\frac{275\times1540}{\sqrt{3}\times1,0}\right)\times10^{-3} = 995\text{ kN}$$

Case 2

$$A_{nt} = t_p\left(e_1+(n_1-1)p_1-(n_1-0,5)d_0\right)$$

$$A_{nt} = 10\left(40+(5-1)\times70-(5-0,5)\times22\right) = 2210\text{ mm}^2$$

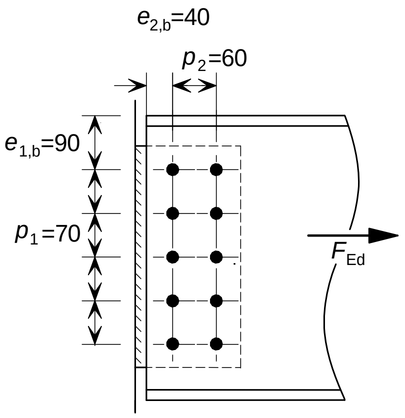
$A_{nv} = t_p\left(e_2 + p_2 - \frac{3}{2}d_0\right) = 10\left(50 + 60 - \frac{3}{2} \times 22\right) = 770 \text{ mm}^2$

$F_{Rd,b} = \left(\frac{430 \times 2210}{1,1} + \frac{275 \times 770}{\sqrt{3} \times 1,0}\right) \times 10^{-3} = 986 \text{ kN}$

$F_{Rd} = \min(880;\ 995;\ 986) = 880 \text{ kN}$

$F_{Ed} = 350 \text{ kN} \leq 880 \text{ kN}$, OK

3.3.2. Beam web resistance



3.3.2.1. Bearing resistance of bolts on the beam web

Basic requirement: $F_{Ed} \leq F_{Rd}$

$F_{Rd} = n_b F_{b,hor,u,Rd}$

$$F_{b,hor,u,Rd} = \frac{k_1 \alpha_b f_{u,b} d t_w}{\gamma_{Mu}}$$

$$k_1 = \min\left(2,8\frac{e_{1,b}}{d_0} - 1,7;\ 1,4\frac{p_1}{d_0} - 1,7;\ 2,5\right)$$

$$= \min\left(2,8\frac{90}{22} - 1,7;\ 1,4\frac{70}{22} - 1,7;\ 2,5\right) = \min(9,8;\ 2,75;\ 2,5) = 2,5$$

$$\alpha_b = \min\left(\frac{e_{2,b}}{3d_0};\ \frac{p_2}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,b}};1,0\right) = \min\left(\frac{40}{3 \times 22};\ \frac{60}{3 \times 22} - \frac{1}{4};\ \frac{800}{430};\ 1,0\right)$$

$$= \min(0,61;\ 0,66;\ 1,86;\ 1,0) = 0,61$$

$$F_{b,hor,u,Rd} = \frac{2,5 \times 0,61 \times 430 \times 20 \times 9}{1,1} \times 10^{-3} = 107 \text{ kN}$$

$F_{Rd} = 10 \times 107 = 1070 \text{ kN}$

$F_{Ed} = 350 \text{ kN} \leq 1070 \text{ kN}$ OK

3.3.2.2. Tension resistance of the beam web

Basic requirement : $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd}}$

$F_{\mathrm{Rd}} = \min\left(F_{\mathrm{Rd,b}}\,; F_{\mathrm{Rd,n}}\right)$

Tension resistance of net section

$$F_{\mathrm{Rd,n}} = 0{,}9\,A_{\mathrm{net,wb}}\frac{f_{\mathrm{u,b}}}{\gamma_{\mathrm{Mu}}}$$

$$A_{\mathrm{net,wb}} = t_{\mathrm{w}}h_{\mathrm{wb}} - d_0 n_1 t_{\mathrm{w}} = 9\times 360 - 22\times 5\times 9 = 2250\ \mathrm{mm}^2$$

$$F_{\mathrm{Rd,n}} = 0{,}9\times 2250\frac{430}{1{,}1}\times 10^{-3} = 792\ \mathrm{kN}$$

Block tearing resistance

$$F_{\mathrm{Rd,b}} = \frac{f_{\mathrm{u,b}}A_{\mathrm{nt}}}{\gamma_{\mathrm{Mu}}} + \frac{f_{\mathrm{y,b}}A_{\mathrm{nv}}/\sqrt{3}}{\gamma_{\mathrm{M0}}}$$

$$A_{\mathrm{nt}} = t_{\mathrm{w}}\left[(n_1-1)p_1 - (n_1-1)d_0\right]$$

$$= 9\left[(5-1)\times 70 - (5-1)22\right] = 1728\ \mathrm{mm}^2$$

$$A_{\mathrm{nv}} = 2t_{\mathrm{w}}\left(e_{2,\mathrm{b}} + p_2 - \frac{3}{2}d_0\right) = 2\times 9\left(40 + 60 - \frac{3}{2}\times 22\right) = 1206\ \mathrm{mm}^2$$

$$F_{\mathrm{Rd,b}} = \left(\frac{430\times 1728}{1{,}1} + \frac{275\times 1206}{\sqrt{3}\times 1{,}0}\right)\times 10^{-3} = 867\ \mathrm{kN}$$

(Case 2 only applies to notched beams)

$F_{\mathrm{Rd}} = \min(792;\ 867) = 792\ \mathrm{kN}$

$F_{\mathrm{Ed}} = 350\ \mathrm{kN} \leq 792\ \mathrm{kN}$, OK

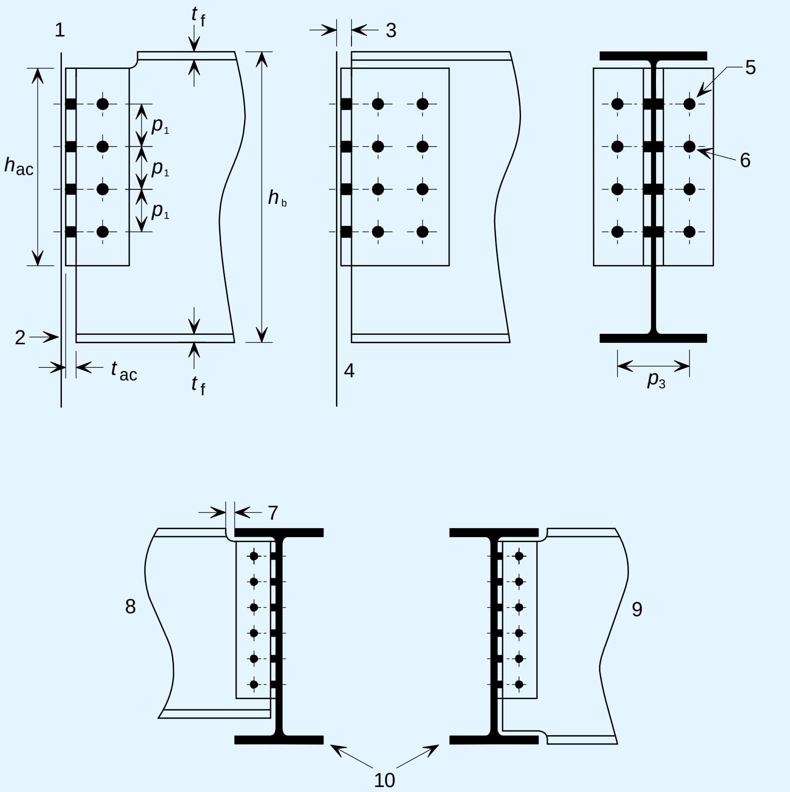
3.3.3. Weld resistance

The weld size specified for shear will be adequate for tying resistance, as it is full strength.

4 DOUBLE ANGLE WEB CLEATS

Unless noted otherwise, the design rules below have been developed from those established for partial depth end plates and fin plates from Reference 8.

4.1 Recommended details



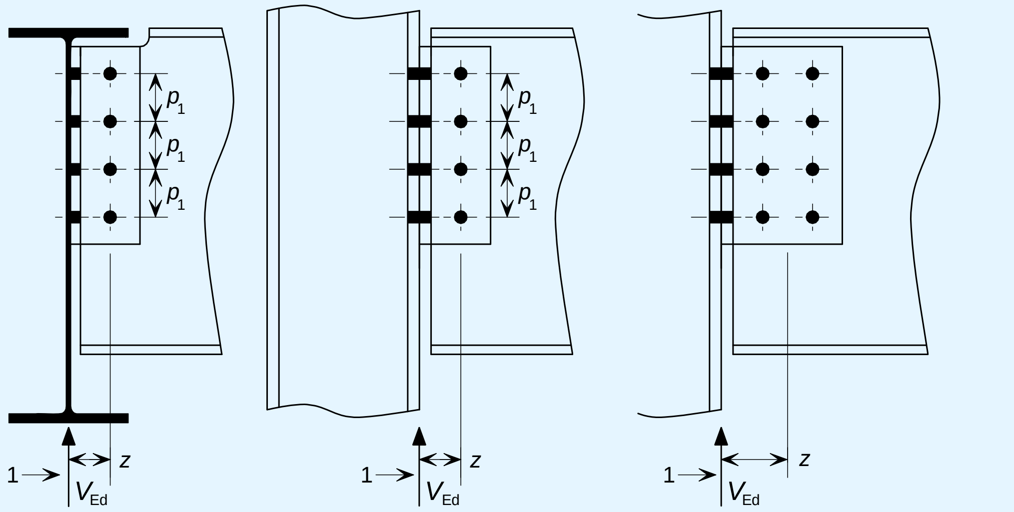
1 Length of cleat $h_{ac} \geq 0{,}6h_b$
2 Face of beam or column
3 End projection, g_h approximately 10 mm
4 Double line of bolts
5 Bolt diameter, d
6 Hold diameters, d_0. $d_0 = d + 2$ mm for $d \leq 24$ mm; $d_0 = d + 3$ mm for $d > 24$ mm
7 10 mm clearance
8 Supported beam (single notched)
9 Supported beam (double notched)
10 Supporting beam

4.2 Checks for vertical shear

4.2.1 Bolt group resistance

4.2.1.1 Supported beam side

Shear resistance of bolts



1 Assumed line of shear transfer

Basic requirement: $V_{Ed} \leq V_{Rd}$

$$V_{Rd} = 2 \times \frac{n_b F_{v,Rd}}{\sqrt{(1 + \alpha n_b)^2 + (\beta n_b)^2}}$$

$F_{v,Rd}$ is the shear resistance of one bolt

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

where:

A is the tensile stress area of the bolt, A_s

α_v = 0,6 for 4.6 and 8.8 bolts
= 0,5 for 10.9 bolts

γ_{M2} is the partial factor for resistance of bolts

For a single vertical line of bolts ($n_2 = 1$)

$$\alpha = 0 \text{ and } \beta = \frac{6z}{n_1(n_1 + 1)p_1}$$

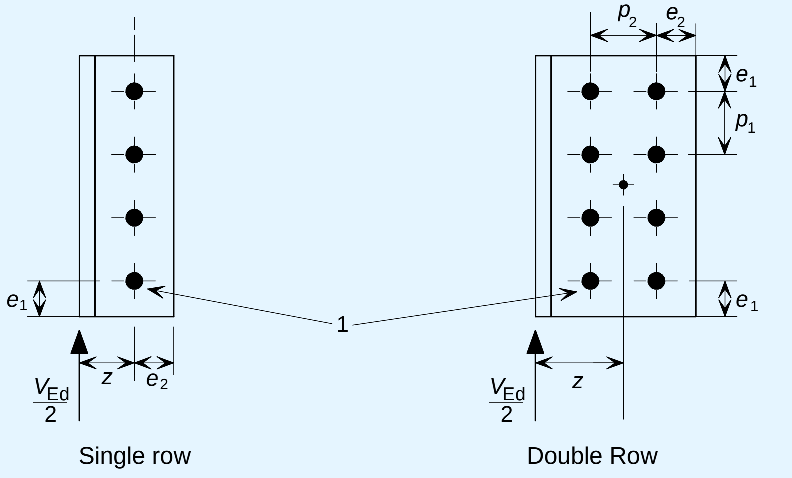
For a double vertical line of bolts ($n_2 = 2$)

$$\alpha = \frac{zp_2}{2I} \text{ and } \beta = \frac{zp_1}{2I}(n_1 - 1)$$

$$I = \frac{n_1}{2} p_2^2 + \frac{1}{6} n_1 \left(n_1^2 - 1\right) p_1^2$$

z is the transverse distance from the face of the supporting element to the centre of the bolt group

Bearing resistance of bolts on the angle cleats



1 Check the bearing strength of cleat under eccentric load

Basic requirement: $V_{\mathrm{Ed}} \leq V_{\mathrm{Rd}}$

$$V_{\mathrm{Rd}} = 2 \times \frac{n_{\mathrm{b}}}{\sqrt{\left(\dfrac{1+\alpha n_{\mathrm{b}}}{F_{\mathrm{b,ver,Rd}}}\right)^2 + \left(\dfrac{\beta n_{\mathrm{b}}}{F_{\mathrm{b,hor,Rd}}}\right)^2}}$$

The bearing resistance of a single bolt is $F_{\mathrm{b,Rd}} = \dfrac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u}} d t}{\gamma_{\mathrm{M2}}}$

The vertical bearing resistance of a single bolt on the angle cleat is as follows:

$$F_{\mathrm{b,ver,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,ac}} d t_{\mathrm{ac}}}{\gamma_{\mathrm{M2}}}$$

The horizontal bearing resistance of a single bolt on the angle cleat is as follows:

$$F_{\mathrm{b,hor,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,ac}} d t_{\mathrm{ac}}}{\gamma_{\mathrm{M2}}}$$

α, β and γ_{M2} are as defined previously.

For $F_{\mathrm{b,ver,Rd}}$:

$$k_1 = \min\left(2{,}8\frac{e_2}{d_0} - 1{,}7;\ 1{,}4\frac{p_2}{d_0} - 1{,}7;\ 2{,}5\right)$$

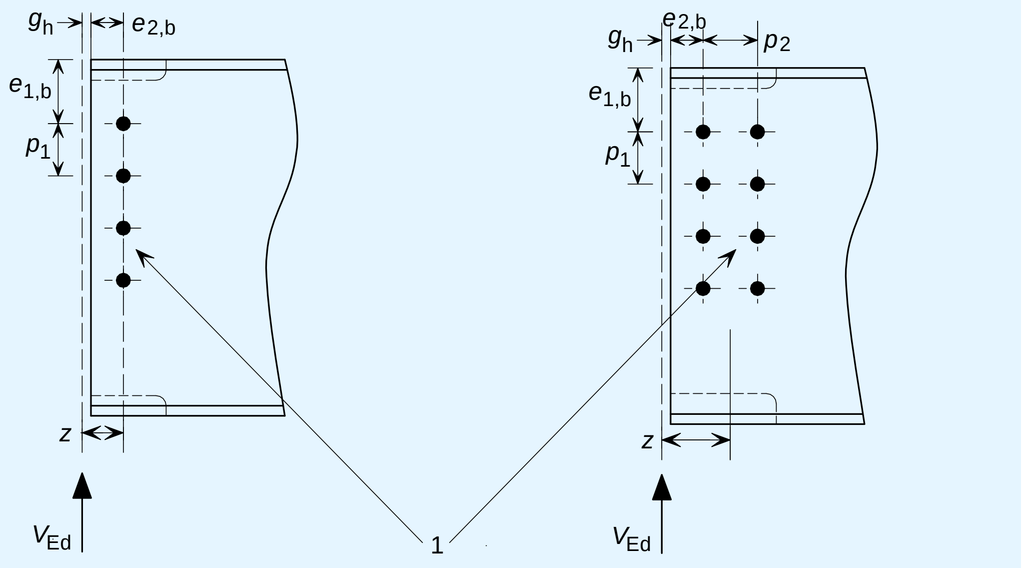
$$\alpha_{\mathrm{b}} = \min\left(\frac{e_1}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{\mathrm{ub}}}{f_{\mathrm{u,ac}}};\ 1{,}0\right)$$

For $F_{\mathrm{b,hor,Rd}}$:

$$k_1 = \min\left(2{,}8\frac{e_1}{d_0}-1{,}7;\ 1{,}4\frac{p_1}{d_0}-1{,}7;\ 2{,}5\right)$$

$$\alpha_{\mathrm{b}} = \min\left(\frac{e_2}{3d_0};\ \frac{p_2}{3d_0}-1{,}4;\ \frac{f_{\mathrm{ub}}}{f_{\mathrm{u,ac}}};\ 1{,}0\right)$$

Bearing resistance of bolts on the beam web



1 Check bolt group in bearing under eccentric load

Basic requirement: $V_{\mathrm{Ed}} \leq V_{\mathrm{Rd}}$

$$V_{\mathrm{Rd}} = \frac{n_{\mathrm{b}}}{\sqrt{\left(\frac{1+\alpha n_{\mathrm{b}}}{F_{\mathrm{b,ver,Rd}}}\right)^2+\left(\frac{\beta n_{\mathrm{b}}}{F_{\mathrm{b,hor,Rd}}}\right)^2}}$$

$$F_{\mathrm{b,ver,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,b}} d t_{\mathrm{w}}}{\gamma_{\mathrm{M2}}}$$

$$F_{\mathrm{b,hor,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,b}} d t_{\mathrm{w}}}{\gamma_{\mathrm{M2}}}$$

α, β and γ_{M2} are as defined previously

For $F_{\mathrm{b,ver,Rd}}$,

$$k_1 = \min\left(2{,}8\frac{e_{2,\mathrm{b}}}{d_0}-1{,}7;\ 1{,}4\frac{p_2}{d_0}-1{,}7;\ 2{,}5\right)$$

$$\alpha_{\mathrm{b}} = \min\left(\frac{e_{1,\mathrm{b}}}{3d_0};\ \frac{p_1}{3d_0}-\frac{1}{4};\ \frac{f_{\mathrm{ub}}}{f_{\mathrm{u,b}}};\ 1{,}0\right)$$

For $F_{b,hor,Rd}$

$$k_1 = \min\left(2{,}8\frac{e_{1,b}}{d_0}-1{,}7;\ 1{,}4\frac{p_1}{d_0}-1{,}7;\ 2{,}5\right)$$

$$\alpha_b = \min\left(\frac{e_{2,b}}{3d_0};\ \frac{p_2}{3d_0}-\frac{1}{4};\ \frac{f_{ub}}{f_{u,b}};\ 1{,}0\right)$$

4.2.1.2 Supporting beam side

Basic requirement:

$V_{Ed} \leq F_{Rd}$

F_{Rd} is the resistance of the bolt group [EN 1993-1-8, §3.7(1)]

If $(F_{b,Rd})_{max} \leq F_{v,Rd}$ then $F_{Rd} = \sum F_{b,Rd}$

If $(F_{b,Rd})_{min} \leq F_{v,Rd} \leq (F_{b,Rd})_{max}$ then $F_{Rd} = n_s(F_{b,Rd})_{min}$

If $F_{v,Rd} \leq (F_{b,Rd})_{min}$ then $F_{Rd} = 0{,}8n_sF_{v,Rd}$

Shear resistance of bolts

$F_{v,Rd}$ is the shear resistance of one bolt

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$ [EN 1993-1-8, Table 3.4]

where:

α_v = 0,6 for 4.6 and 8.8 bolts

= 0,5 for 10.9 bolts

A is the tensile stress area of the bolt, A_s

Bearing resistance of bolts on the angle cleats

$F_{b,Rd}$ is the bearing resistance of a single bolt

$$F_{b,Rd} = \frac{k_1 \alpha_b f_{u,ac} d t_{ac}}{\gamma_{M2}}$$ [EN 1993-1-8, Table 3.4]

where:

γ_{M2} is the partial factor for plates in bearing

– For end bolts (parallel to the direction of load transfer)

$$\alpha_b = \min\left(\frac{e_1}{3d_0};\ \frac{f_{ub}}{f_{u,ac}};\ 1{,}0\right)$$

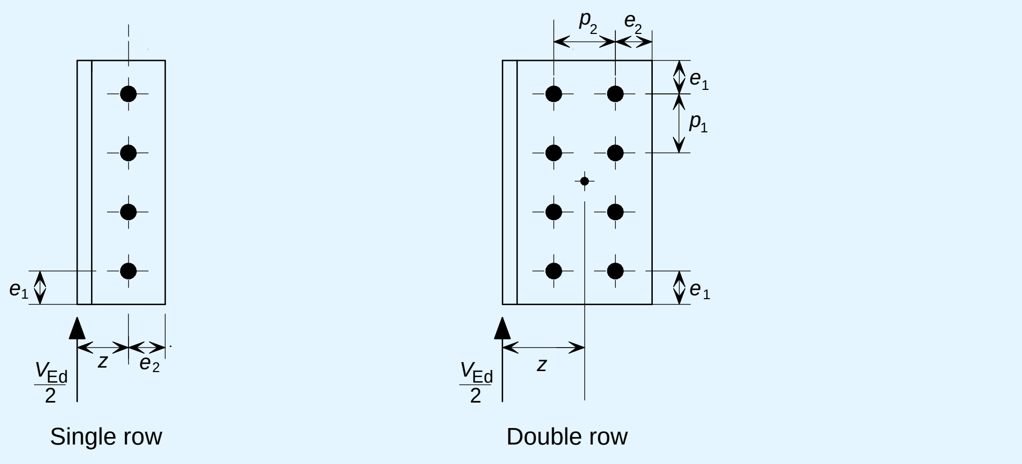
– For inner bolts (parallel to the direction of load transfer)

$$\alpha_b = \min\left(\frac{p_1}{3d_0}-\frac{1}{4};\ \frac{f_{ub}}{f_{u,ac}};\ 1{,}0\right)$$

– For edge bolts (perpendicular to the direction of load transfer)

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

4.2.2 Shear resistance of the angle cleats



4.2.2.1 Supported beam side

Basic requirement: $V_{Ed} \leq V_{Rd,min}$

$V_{Rd,min} = \min(V_{Rd,g};\ V_{Rd,n};\ V_{Rd,b})$

Shear resistance of gross section

$$V_{Rd,g} = 2 \times \frac{h_{ac} t_{ac}}{1{,}27} \frac{f_{y,ac}}{\sqrt{3}\gamma_{M0}}$$

Note: The coefficient 1,27 takes into account the reduction in shear resistance due to the presence of the nominal in-plane bending which produces tension in the bolts.[9]

Shear resistance of net section

$$V_{Rd,n} = 2 \times A_{v,net} \frac{f_{u,ac}}{\sqrt{3}\gamma_{M2}}$$

$$A_{v,net} = t_{ac}\left(h_{ac} - n_1 d_0\right)$$

Block tearing resistance

$$V_{Rd,b} = 2\left(\frac{0{,}5 f_{u,ac} A_{nt}}{\gamma_{M2}} + \frac{f_{y,ac} A_{nv}}{\sqrt{3}\gamma_{M0}}\right)$$

$$A_{nv} = t_{ac}\left(h_{ac} - e_1 - (n_1 - 0{,}5) d_0\right)$$

For a single line of bolts:

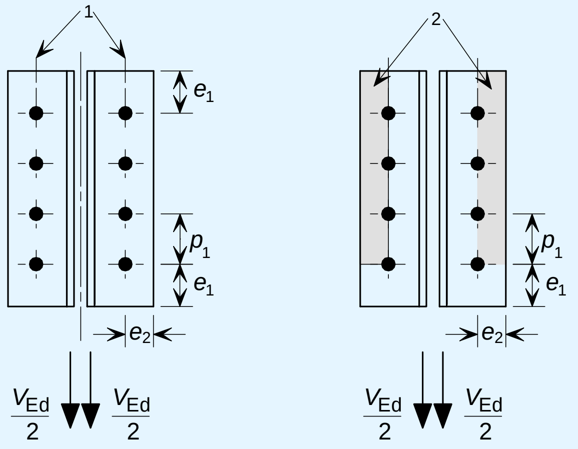
$$A_{nt} = t_{ac}\left(e_2 - 0{,}5 d_0\right)$$

For a double line of bolts:

$$A_{\text{nt}} = t_{\text{ac}}\left(e_2 + p_2 - 1{,}5 d_0\right)$$

γ_{M2} is the partial factor for the resistance of net sections.

4.2.2.2 Supporting beam side



1 Critical section in shear and bearing

2 Block shear – check failure by tearing out of shaded portion

Basic requirement: $V_{\text{Ed}} \leq V_{\text{Rd,min}}$

$V_{\text{Rd,min}} = \min(V_{\text{Rd,g}};\ V_{\text{Rd,n}};\ V_{\text{Rd,b}})$

Shear resistance of gross section

$$V_{\text{Rd,g}} = 2 \times \frac{h_{\text{ac}} t_{\text{ac}}}{1{,}27} \frac{f_{\text{y,ac}}}{\sqrt{3}\gamma_{\text{M0}}}$$

Note: The coefficient 1,27 takes into account the reduction in shear resistance due to the presence of the nominal in-plane bending which produces tension in the bolts[9].

Shear resistance of net section

$$V_{\text{Rd,n}} = 2 \times A_{\text{v,net}} \frac{f_{\text{u,ac}}}{\sqrt{3}\gamma_{\text{M2}}}$$

$$A_{\text{v,net}} = t_{\text{ac}}\left(h_{\text{ac}} - n_1 d_0\right)$$

Block tearing resistance

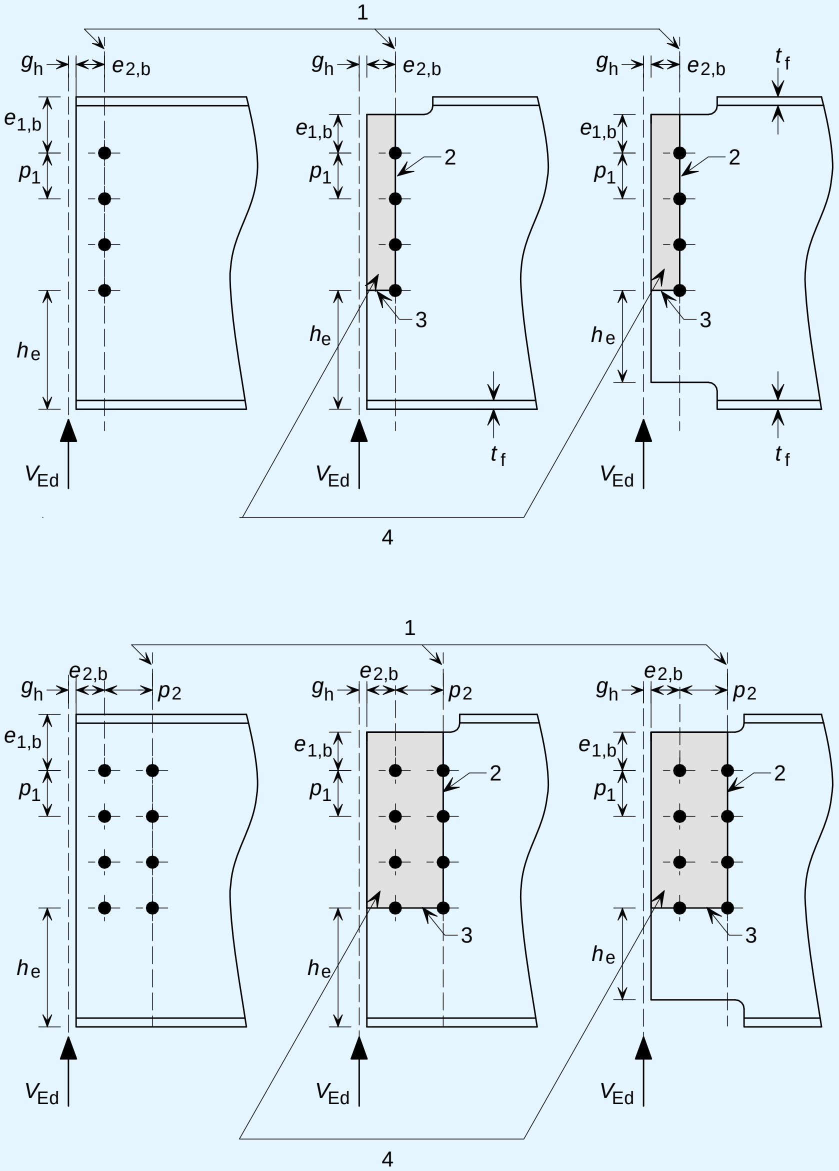
$$V_{\text{Rd,b}} = 2\left(\frac{0{,}5 f_{\text{u,ac}} A_{\text{nt}}}{\gamma_{\text{M2}}} + \frac{f_{\text{y,ac}} A_{\text{nv}}}{\sqrt{3}\gamma_{\text{M0}}}\right)$$

$$A_{\text{nt}} = t_{\text{ac}}\left(e_2 - 0{,}5 d_0\right)$$

$$A_{\text{nv}} = t_{\text{ac}}\left(h_{\text{ac}} - e_1 - (n_1 - 0{,}5) d_0\right)$$

γ_{M2} is the partial factor for the resistance of net sections.

4.2.3 Shear resistance of the beam web



1 Critical section in plain shear
2 Shear failure
3 Tension failure
4 Block shear failure tearing out of shaded portion

4.2.3.1 Shear and block tearing resistance

Basic requirement: $V_{\mathrm{Ed}} \leq V_{\mathrm{Rd,min}}$

$$V_{\mathrm{Rd,min}} = \min(V_{\mathrm{Rd,g}};\ V_{\mathrm{Rd,n}};\ V_{\mathrm{Rd,b}})$$

Shear resistance of gross section

$$V_{\mathrm{Rd,g}} = A_{\mathrm{v,wb}} \frac{f_{\mathrm{y,b}}}{\sqrt{3}\gamma_{\mathrm{M0}}}$$

$A_{\mathrm{v,wb}} = A - 2bt_{\mathrm{f}} + (t_{\mathrm{w}} + 2r)t_{\mathrm{f}}$ but $\geq \eta\ h_{\mathrm{w}}t_{\mathrm{w}}$ for un-notched beam

$A_{\mathrm{v,wb}} = A_{\mathrm{Tee}} - bt_{\mathrm{f}} + (t_{\mathrm{w}} + 2r)t_{\mathrm{f}}/2$ for single notched beam

$A_{v,wb} = t_w (e_{1,b} + (n_1 - 1)p_1 + h_e)$ for double notched beam

η is a factor from EN 1993-1-5 (it may be conservatively taken equal to 1,0)

A_{Tee} is the area of the Tee section

Shear resistance of net section

$$V_{Rd,n} = A_{v,wb,net} \frac{f_{u,b}}{\sqrt{3}\gamma_{M2}}$$

$$A_{v,wb,net} = A_{v,wb} - n_1 d_0 t_w$$

Block tearing resistance

$$V_{Rd,b} = \frac{0,5 f_{u,b} A_{nt}}{\gamma_{M2}} + \frac{f_{y,b} A_{nv}}{\sqrt{3}\gamma_{M0}}$$

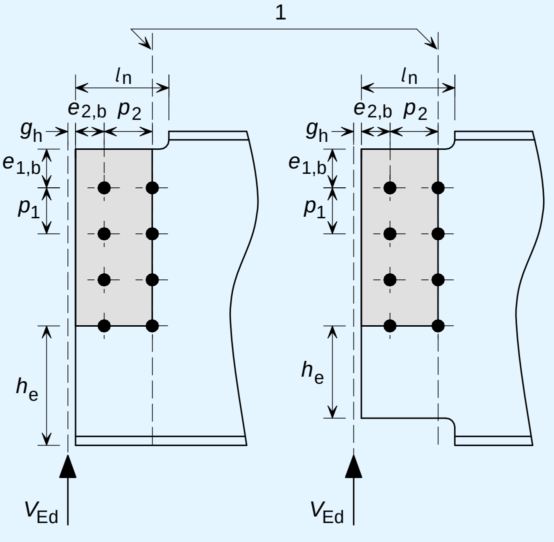
For a single vertical line of bolts, $A_{nt} = t_w \left(e_{2,b} - 0,5 d_0\right)$

For a double vertical line of bolts, $A_{nt} = t_w \left(e_{2,b} + p_2 - \frac{3}{2} d_0\right)$

$$A_{nv} = t_w \left(e_{1,b} + (n_1 - 1) p_1 - (n_1 - 0,5) d_0\right)$$

γ_{M2} is the partial factor for the resistance of net sections.

4.2.3.2 Shear and bending interaction at the 2nd line of bolts, if the notch length $l_n > (e_{2,b} + p_2)$



1 Critical section in plain shear

Basic requirement: $V_{Ed} (g_h + e_{2,b} + p_2) \leq M_{c,Rd}$ [Reference 4]

$M_{c,Rd}$ is the moment resistance of the notched beam at the connection in the presence of shear.

For single notched beam:

For low shear (i.e. $V_{Ed} \leq 0{,}5V_{pl,N,Rd}$)

$$M_{c,Rd} = \frac{f_{y,b} W_{el,N}}{\gamma_{M0}}$$

For high shear (i.e. $V_{Ed} > 0{,}5V_{pl,N,Rd}$)

$$M_{c,Rd} = \frac{f_{y,b} W_{el,N}}{\gamma_{M0}} \left[1 - \left(\frac{2V_{Ed}}{V_{pl,N,Rd}} - 1 \right)^2 \right]$$

$V_{pl,N,Rd} = \min(V_{Rd,g};\ V_{Rd,b})$

$W_{el,N}$ is the elastic section modulus of the gross Tee section at the notch

For double notched beam:

For low shear (i.e. $V_{Ed} \leq 0{,}5V_{pl,DN,Rd}$)

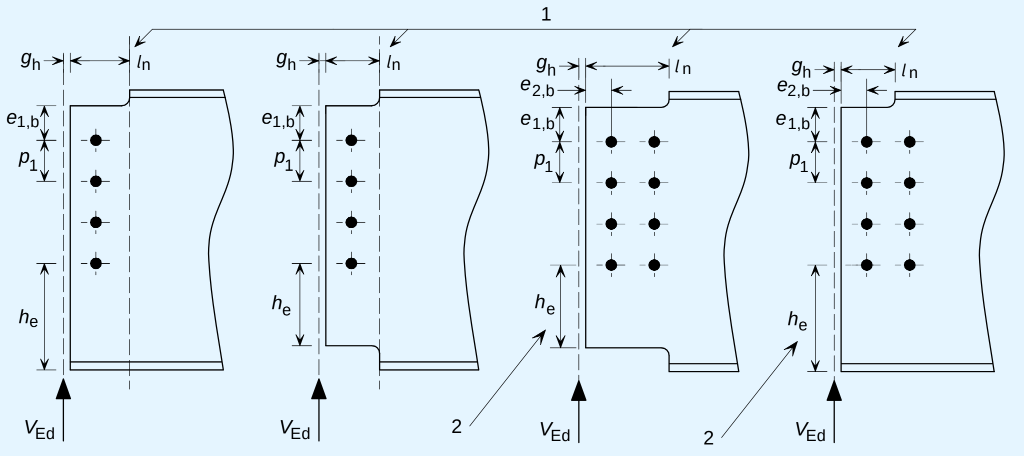
$$M_{c,Rd} = \frac{f_{y,b} t_w}{6\gamma_{M0}} \left(e_1 + (n_1 - 1) p_1 + h_e \right)^2$$

For high shear (i.e. $V_{Ed} > 0{,}5V_{pl,DN,Rd}$)

$$M_{c,Rd} = \frac{f_{y,b} t_w}{6\gamma_{M0}} \left(e_1 + (n_1 - 1) p_1 + h_e \right)^2 \left[1 - \left(\frac{2V_{Ed}}{V_{pl,DN,Rd}} - 1 \right)^2 \right]$$

$V_{pl,DN,Rd} = \min(V_{Rd,g};\ V_{Rd,b})$

4.2.4 Bending resistance at the notch



1 Critical sections
2 To notch or beam flange

Shear and bending interaction at the notch.

4.2.4.1 For single bolt line or for double bolt lines, if $x_N \geq 2d$:

$V_{Ed}\,(g_h + l_n) \leq M_{v,N,Rd}$ [Reference 4]

$M_{v,N,Rd}$ is the moment resistant of the beam at the notch in the presence of shear

For single notched beam

For low shear (i.e. $V_{Ed} \leq 0{,}5V_{pl,N,Rd}$)

$$M_{v,N,Rd} = \frac{f_{y,b} W_{el,N}}{\gamma_{M0}}$$

For high shear (i.e. $V_{Ed} > 0{,}5V_{pl,N,Rd}$)

$$M_{v,N,Rd} = \frac{f_{y,b} W_{el,N}}{\gamma_{M0}} \left[1 - \left(\frac{2V_{Ed}}{V_{pl,N,Rd}} - 1 \right)^2 \right]$$

For double notched beam:

For low shear (i.e. $V_{Ed} \leq 0{,}5V_{pl,DN,Rd}$)

$$M_{v,DN,Rd} = \frac{f_{y,b} t_w}{6\gamma_{M0}} \left(e_{1,b} + (n_1 - 1) p_1 + h_e \right)^2$$

For high shear (i.e. $V_{Ed} > 0{,}5V_{pl,DN,Rd}$)

$$M_{v,DN,Rd} = \frac{f_{y,b} t_w}{4\gamma_{M0}} \left(e_{1,b} + (n_1 - 1) p_1 + h_e \right)^2 \left[1 - \left(\frac{2V_{Ed}}{V_{pl,DN,Rd}} - 1 \right)^2 \right]$$

4.2.4.2 For double bolt lines, if $x_N < 2d$:

max (V_{Ed} ($g_h + l_n$); V_{Ed} ($g_h + e_{2,b} + p_2$)) $\leq$ $M_{v,N,Rd}$ [Reference 4]

$M_{v,N,Rd} = M_{c,Rd}$ from the previous check

where:

$W_{el,N}$ is the elastic section modulus of the gross Tee section at the notch

$V_{pl,N,Rd}$ is the shear resistance at the notch for single notched beams

$$= \frac{A_{v,N}\, f_{y,b}}{\sqrt{3}\, \gamma_{M0}}$$

$A_{v,N}$ $= A_{Tee} - bt_f + (t_w + 2r)\dfrac{t_f}{2}$

$V_{pl,DN,Rd}$ is the shear resistance at the notch for double notched beams

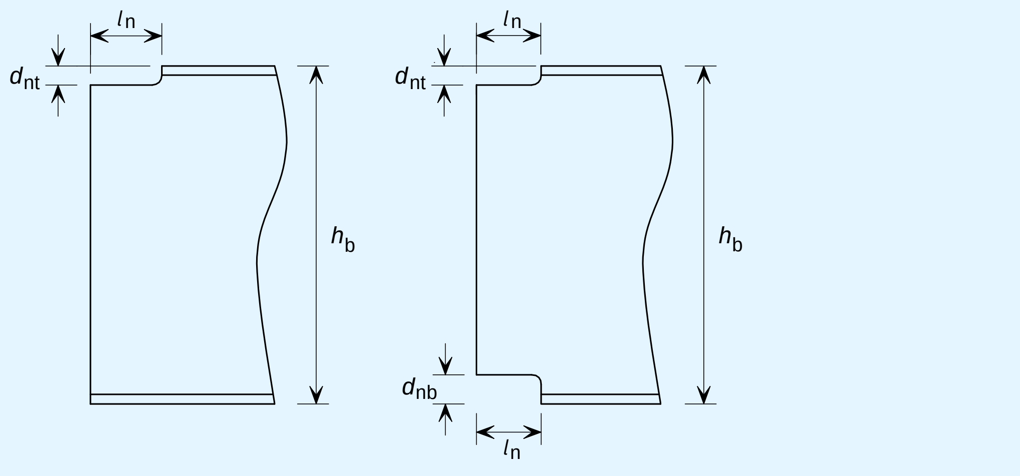
$$= \frac{A_{v,DN}\, f_{y,b}}{\sqrt{3}\, \gamma_{M0}}$$

$A_{v,DN}$ $= t_w\,(e_{1,b} + (n_1 - 1)\, p_1 + h_e)$

where:

A_{Tee} is the area of the Tee section

4.2.5 Local stability of the notched beam



When the beam is restrained against lateral torsional buckling, no account need be taken of notch stability provided the following conditions are met:

For one flange notched, basic requirement: [5],[6]

$d_{nt} \leq h_b / 2$ and:

$l_n \leq h_b$ for $h_b / t_w \leq 54{,}3$ (S275 steel)

$l_n \leq \dfrac{160000\, h_b}{(h_b / t_w)^3}$ for $h_b / t_w > 54{,}3$ (S275 steel)

$l_n \leq h_b$ for $h_b / t_w \leq 48{,}0$ (S355 steel)

$l_n \leq \dfrac{110000\, h_b}{(h_b / t_w)^3}$ for $h_b / t_w > 48{,}0$ (S355 steel)

For both flanges notched, basic requirement: [7]

$\max(d_{nt}; d_{nb}) \leq h_b / 5$ and:

$l_n \leq h_b$ for $h_b / t_w \leq 54{,}3$ (S275 steel)

$l_n \leq \dfrac{160000\, h_b}{(h_b / t_w)^3}$ for $h_b / t_w > 54{,}3$ (S275 steel)

$l_n \leq h_b$ for $h_b / t_w \leq 48{,}0$ (S355 steel)

$l_n \leq \dfrac{110000\, h_b}{(h_b / t_w)^3}$ for $h_b / t_w > 48{,}0$ (S355 steel)

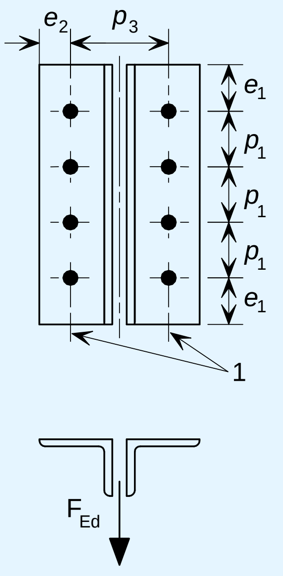
Where the notch length l_n exceeds these limits, either suitable stiffening should be provided or the notch should be checked to References 5, 6 and 7.
For S235 and S460 members see References 5, 6 and 7.

4.3 Checks for tying

EN 1993-1-8 does not have a partial factor for structural integrity checks. In this publication γ_{Mu} has been used. A value of $\gamma_{Mu} = 1{,}1$ is recommended.

4.3.1 Angle cleats and bolt group resistance

4.3.1.1 Resistance of the angle cleats in bending



1 Critical sections

There are three modes of failure for angle cleats in bending:

Mode 1: complete yielding of the plate

Mode 2: bolt failure with yielding of the plate

Mode 3: bolt failure

Basic requirement: $F_{Ed} \leq F_{Rd}$

$F_{Rd} = \min(F_{Rd,u,1}, F_{Rd,u,2}, F_{Rd,u,3})$

Mode 1 (the complete yielding of the angle cleats)

$$F_{Rd,u,1} = \frac{(8n - 2e_w)M_{pl,1,Rd,u}}{2mn - e_w(m+n)}$$ [EN 1993-1-8 Table 6.2]

Mode 2 (bolt failure with yielding of the angle cleats)

$$F_{Rd,u,2} = \frac{2M_{pl,2,Rd,u} + n\Sigma F_{t,Rd,u}}{m+n}$$ [EN 1993-1-8 Table 6.2]

Mode 3 (bolt failure)

$$F_{Rd,u,3} = \Sigma F_{t,Rd,u}$$ [EN 1993-1-8 Table 6.2]

$$F_{t,Rd,u} = \frac{k_2 f_{ub} A}{\gamma_{Mu}}$$

where:

$$M_{\text{pl,1,Rd,u}} = \frac{0,25\Sigma l_{\text{eff}}\, t_{\text{ac}}^{2}\, f_{\text{u,ac}}}{\gamma_{\text{Mu}}}$$

$$M_{\text{pl,2,Rd,u}} = M_{\text{pl,1,Rd,u}}$$

$$m = \frac{p_3 - t_{\text{w}} - 2t_{\text{ac}} - 2\times 0,8\times r}{2}$$

n $= e_{\text{min}}$ but $n \leq 1,25m$ where $e_{\text{min}} = e_2$

$$e_{\text{w}} = \frac{d_{\text{w}}}{4}$$

d_{w} is the diameter of the washer

k_2 = 0,63 for countersunk bolts

= 0,9 otherwise

A is the tensile stress area of the bolt, A_{s}

Σl_{eff} is the effective length of a plastic hinge

$$\Sigma l_{\text{eff}} = 2e_{1\text{A}} + (n_1 - 1)\, p_{1\text{A}}$$

$$e_{1\text{A}} = e_1 \text{ but} \leq 0,5(p_3 - t_{\text{w}} - 2r) + \frac{d_0}{2}$$

$$p_{1\text{A}} = p_1 \text{ but} \leq p_3 - t_{\text{w}} - 2r + d_0$$

4.3.1.2 Shear resistance of bolts

Basic requirement: $F_{\text{Ed}} \leq F_{\text{Rd}}$

$$F_{\text{Rd}} = 2n_{\text{b}}F_{\text{v,u}}$$

$$F_{\text{v,u}} = \frac{\alpha_{\text{v}}\, f_{\text{ub}}\, A}{\gamma_{\text{Mu}}}$$

where:

α_{v} = 0,6 for 4.6 and 8.8 bolts

= 0,5 for 10.9 bolts

A is the tensile stress area of the bolt, A_{s}

4.3.1.3 Bearing resistance of bolts on the angle cleats

Basic requirement: $F_{\text{Ed}} \leq F_{\text{Rd}}$

$$F_{\text{Rd}} = 2n_{\text{b}}F_{\text{b,hor,u,Rd}}$$

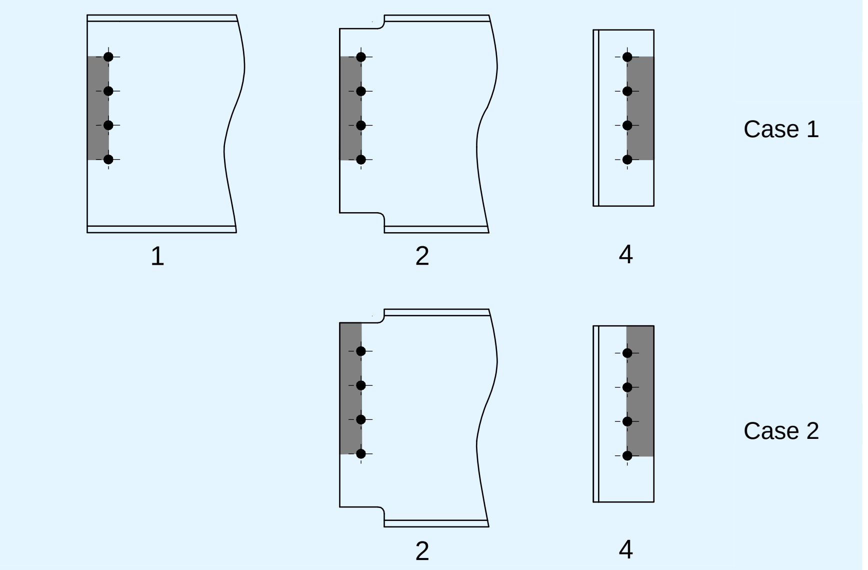
$$F_{\text{b,hor,u,Rd}} = \frac{k_1 \alpha_{\text{b}}\, f_{\text{u,ac}}\, d t_{\text{ac}}}{\gamma_{\text{Mu}}}$$

where:

$$k_1 = \min\left(2{,}8\frac{e_1}{d_0}-1{,}7;1{,}4\frac{p_1}{d_0}-1{,}7;2{,}5\right)$$

$$\alpha_b = \min\left(\frac{e_2}{3d_0};\frac{p_2}{3d_0}-\frac{1}{4};\frac{f_{ub}}{f_{u,ac}};1{,}0\right)$$

4.3.1.4 Block tearing resistance



1 Beam without notch
2 Notched beam
4 Double angle cleat

Basic requirement : $F_{Ed} \leq F_{Rd,b}$

$$F_{Rd,b} = \frac{f_{u,ac}A_{nt}}{\gamma_{Mu}} + \frac{f_{y,ac}A_{nv}}{\sqrt{3}\gamma_{M0}}$$

Case 1:

$$A_{nt} = 2t_{ac}\left[(n_1-1)p_1-(n_1-1)d_0\right]$$

For a single vertical line of bolts: $A_{nv} = 4t_{ac}\left(e_2-0{,}5d_0\right)$

For a double vertical line of bolts: $A_{nv} = 4t_{ac}\left(e_2+p_2-\frac{3}{2}d_0\right)$

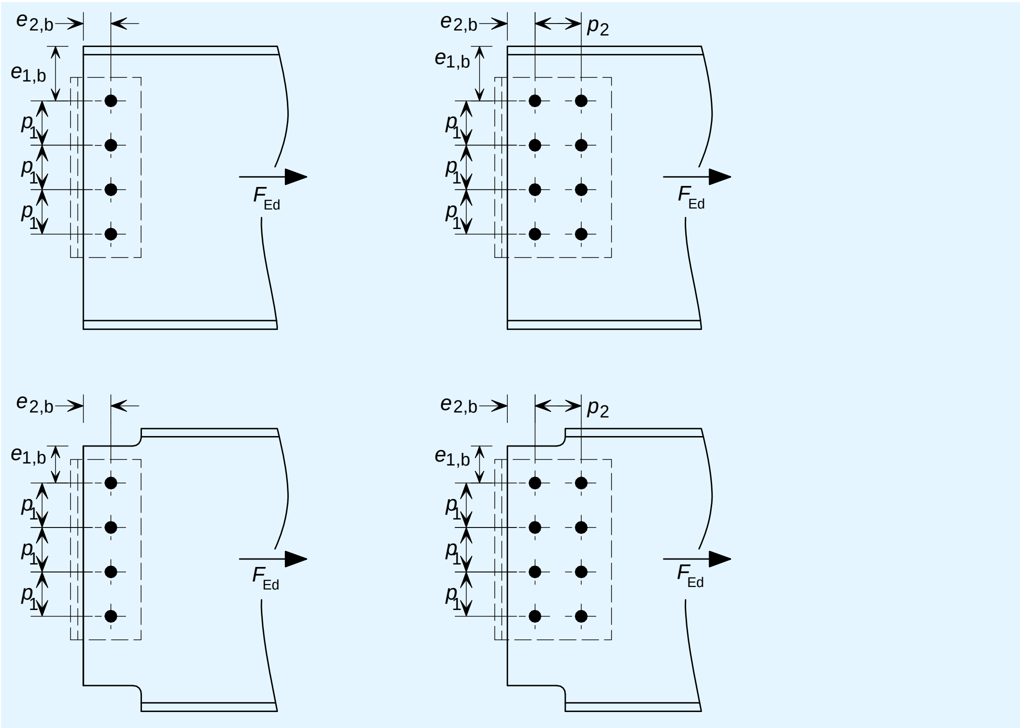
Case 2:

$$A_{nt} = 2t_{ac}\left[e_1+(n_1-1)p_1-(n_1-0{,}5)d_0\right]$$

For a single vertical line of bolts: $A_{nv} = 2t_{ac}\left(e_2-0{,}5d_0\right)$

For a double vertical line of bolts: $A_{nv} = 2t_{ac}\left(e_2+p_2-\frac{3}{2}d_0\right)$

4.3.2 Beam web resistance



4.3.2.1 Bearing resistance of bolts on the beam web

Basic requirement: $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd}}$

$F_{\mathrm{Rd}} = n_{\mathrm{b}} F_{\mathrm{b,hor,u,Rd}}$

$$F_{\mathrm{b,hor,u,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,b}} d t_{\mathrm{w,b}}}{\gamma_{\mathrm{Mu}}}$$

where:

$$k_1 = \left(2{,}8\frac{e_{1,\mathrm{b}}}{d_0} - 1{,}7;\ 1{,}4\frac{p_1}{d_0} - 1{,}7;\ 2{,}5 \right)$$

$$\alpha_{\mathrm{b}} = \left(\frac{e_{2,\mathrm{b}}}{3d_0};\ \frac{p_2}{3d_0} - \frac{1}{4};\ \frac{f_{\mathrm{ub}}}{f_{\mathrm{u,b}}};\ 1{,}0 \right)$$

α_{v} = 0,6 for 4.6 and 8.8 bolts

= 0,5 for 10.9 bolts

4.3.2.2 Tension resistance of the beam web

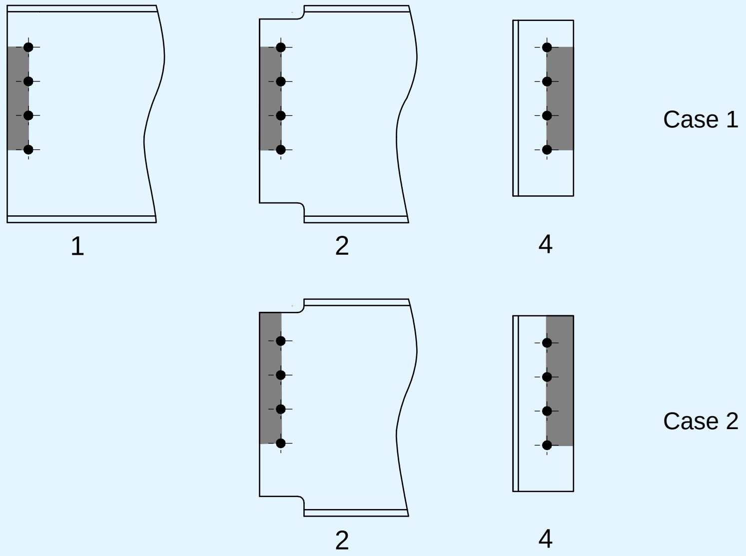
Basic requirement: $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd,n}}$

$$F_{\mathrm{Rd,n}} = 0{,}9 A_{\mathrm{net,wb}} \frac{f_{\mathrm{u,b}}}{\gamma_{\mathrm{Mu}}}$$

where:

$$A_{\mathrm{net,wb}} = t_{\mathrm{w}} h_{\mathrm{ac}} - d_0 n_1 t_{\mathrm{w}}$$

4.3.2.3 Block tearing resistance



1 Beam without notch
2 Notched beam
4 Double angle cleat

Basic requirement: $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd,b}}$

$$F_{\mathrm{Rd,b}} = \frac{f_{\mathrm{u,b}} A_{\mathrm{nt}}}{\gamma_{\mathrm{Mu}}} + \frac{f_{\mathrm{y,b}} A_{\mathrm{nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}}$$

Case 1:

$$A_{\mathrm{nt}} = t_{\mathrm{w}} \left((n_1 - 1) p_1 - (n_1 - 1) d_0 \right)$$

For a single vertical line of bolts, $A_{\mathrm{nv}} = 2 t_{\mathrm{w}} \left(e_{2,\mathrm{b}} - 0{,}5 d_0 \right)$

For double vertical line of bolts, $A_{\mathrm{nv}} = 2 t_{\mathrm{w}} \left(e_{2,\mathrm{b}} + p_2 - \frac{3}{2} d_0 \right)$

Case 2 (for notched beams only):

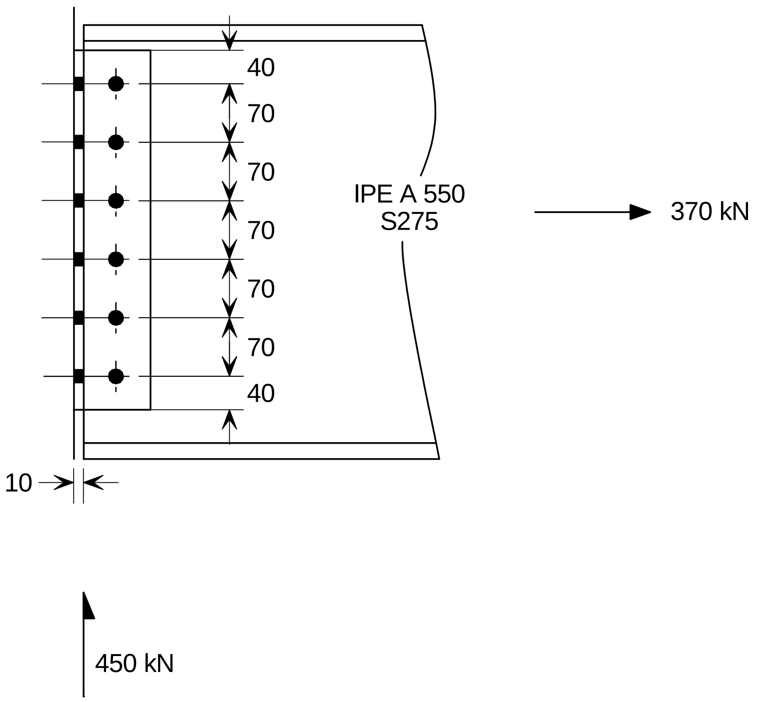
$$A_{\mathrm{nt}} = t_{\mathrm{w}} \left(e_{1,\mathrm{b}} + (n_1 - 1) p_1 - (n_1 - 0{,}5) d_0 \right)$$

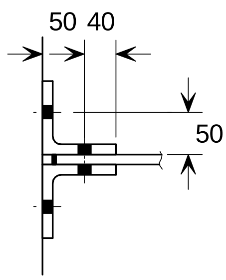
For a single vertical line of bolts, $A_{\mathrm{nv}} = t_{\mathrm{w}} \left(e_{2,\mathrm{b}} - 0{,}5 d_0 \right)$

For double vertical line of bolts, $A_{\mathrm{nv}} = t_{\mathrm{w}} \left(e_{2,\mathrm{b}} + p_2 - \frac{3}{2} d_0 \right)$

4. Angle Web Cleats

Details and data





Beam: IPE A 550 S275

Angle cleats: 2/90 × 90 × 10, S275

Bolts: M20 8.8

Summary of full design checks

Design forces

V_{Ed} = 450 kN

F_{Ed} = 370 kN (Tie force)

Shear resistances

Bolt group resistance	
Supported beam side	
Shear resistance of bolts	962 kN
Bearing resistance of bolts on the angle cleats	1075 kN
Bearing resistance of bolts on the beam web	583 kN
Supporting beam side	
Resistance	902 kN
Shear resistance of the angle cleats	
Supported beam side	
Shear resistance	954 kN
Supporting beam side	
Shear resistance	954 kN
Shear resistance of the beam web	
Shear and block tearing resistance	
Shear resistance	501 kN
Shear and bending interaction at the 2nd line of bolts	N/A
Bending resistance at the notch	N/A
Local stability of the notched beam	N/A

Tying resistances

Angle cleats and bolt group resistance	
Resistance of the angle cleats in bending	696 kN
Shear resistance of bolts	1284 kN
Bearing resistance of bolts on the angle cleats	1428 kN
Block tearing resistance	2060 kN
Beam web resistance	
Bearing resistance of bolts on the beam web	642 kN
Tension resistance of the beam web	944 kN
Block tearing resistance	927 kN

4.1. Recommended details

Unless noted otherwise, all references are to EN 1993-1-8

Cleats 10 mm thick

Length, h_{ac} = 430mm > 0,6h_b, OK

4.2. Checks for vertical shear

4.2.1. Bolt group resistance

4.2.1.1. Supported beam side

Shear resistance of bolts

Basic requirement: $V_{Ed} \leq V_{Rd}$

$$V_{Rd} = 2 \times \frac{n_b F_{v,Rd}}{\sqrt{(1+\alpha n_b)^2 + (\beta n_b)^2}}$$

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

For M20 8.8 bolts, $F_{v,Rd} = \dfrac{0,6 \times 800 \times 245}{1,25} \times 10^{-3}$ = 94 kN

For a single vertical line of bolts (i.e. n_2 = 1 and n_1 = 6), α = 0

$$\beta = \frac{6z}{n_1(n_1+1)p_1} = \frac{6 \times 50}{6(6+1)70} = 0{,}102$$

Thus $V_{Rd} = 2 \times \dfrac{6 \times 94}{\sqrt{(1+0 \times 6)^2 + (0{,}102 \times 6)^2}}$ = 962 kN

V_{Ed} = 450 kN ≤ 962 kN, OK

Bearing resistance of bolts on the angle cleats

Basic requirement: $V_{Ed} \leq V_{b,Rd}$

$$V_{b,Rd} = 2 \times \frac{n_b}{\sqrt{\left(\frac{1+\alpha n_b}{F_{b,ver,Rd}}\right)^2 + \left(\frac{\beta n_b}{F_{b,hor,Rd}}\right)^2}}$$

α = 0 and β = 0,102, as above

The vertical bearing resistance for a single bolt, $F_{b,ver,Rd} = \dfrac{k_1 \alpha_b f_{u,ac} d t_{ac}}{\gamma_{M2}}$

$$k_1 = \min\left(2{,}8\frac{e_2}{d_0} - 1{,}7;\ 2{,}5\right) = \min\left(2{,}8\frac{40}{22} - 1{,}7;\ 2{,}5\right) = \min(3{,}39;\ 2{,}5)$$

$= 2{,}5$

α_b $= \min\left(\frac{e_1}{3d_0};\ \frac{p_1}{3d_0}-\frac{1}{4};\ \frac{f_{ub}}{f_{u,ac}};\ 1{,}0\right)$

$= \min\left(\frac{40}{3\times 22};\ \frac{70}{3\times 22}-0{,}25;\ \frac{800}{430};\ 1{,}0\right) = \min(0{,}61;\ 0{,}81;\ 1{,}86;\ 1{,}0)$

α_b $= 0{,}61$

$F_{b,ver,Rd} = \frac{2{,}5\times 0{,}61\times 430\times 20\times 10}{1{,}25}\times 10^{-3} = 105\ \text{kN}$

The horizontal bearing resistance for a single bolt, $F_{b,hor,Rd} = \frac{k_1 \alpha_b f_{u,ac} d t_{ac}}{\gamma_{M2}}$

k_1 $= \min\left(2{,}8\frac{e_1}{d_0}-1{,}7;\ 1{,}4\frac{p_1}{d_0}-1{,}7;\ 2{,}5\right)$

$= \min\left(2{,}8\frac{40}{22}-1{,}7;\ 1{,}4\frac{70}{22}-1{,}7;\ 2{,}5\right) = \min(3{,}39;\ 2{,}75;\ 2{,}5) = 2{,}5$

α_b $= \min\left(\frac{e_2}{3d_0};\ \frac{f_{ub}}{f_{u,ac}};\ 1{,}0\right) = \min\left(\frac{40}{3\times 22};\ \frac{800}{430};\ 1{,}0\right)$

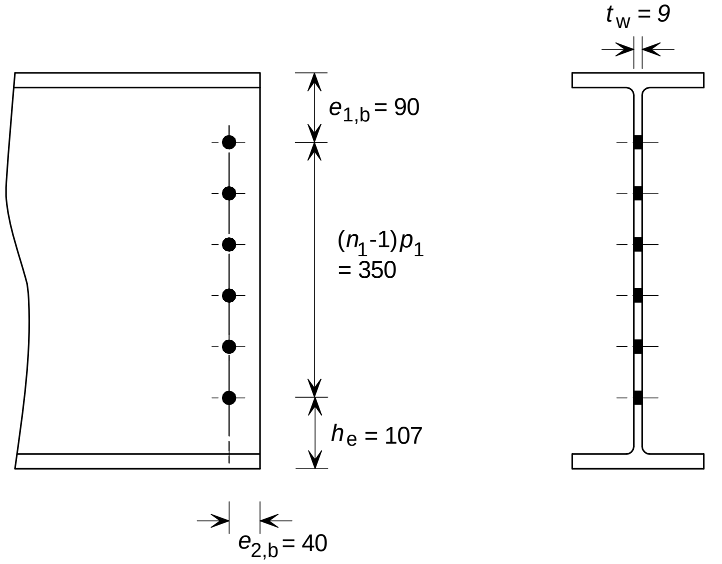
$= \min(0{,}61;\ 1{,}86;\ 1{,}0) = 0{,}61$

$F_{b,hor,Rd} = \frac{2{,}5\times 0{,}61\times 430\times 20\times 10}{1{,}25}\times 10^{-3} = 105\ \text{kN}$

V_{Rd} $= 2\times \frac{6}{\sqrt{\left(\frac{1+0\times 6}{105}\right)^2+\left(\frac{0{,}102\times 6}{105}\right)^2}} = 1075\ \text{kN}$

V_{Ed} $= 450\ \text{kN} \leq 1075\ \text{kN}$, OK

Bearing resistance of bolts on the beam web



Basic requirement: $V_{Ed} \leq V_{Rd}$

$$V_{Rd} = \frac{n_b}{\sqrt{\left(\frac{1+\alpha n_b}{F_{b,ver,Rd}}\right)^2 + \left(\frac{\beta n_b}{F_{b,hor,Rd}}\right)^2}}$$

α = 0 and β = 0,102, as above

The vertical bearing resistance of a single bolt, $F_{b,ver,Rd} = \dfrac{k_1 \alpha_b f_{u,b} d t_w}{\gamma_{M2}}$

$$k_1 = \min\left(2{,}8\frac{e_{2,b}}{d_0} - 1{,}7;\ 2{,}5\right) = \min\left(2{,}8\frac{40}{22} - 1{,}7;\ 2{,}5\right) = \min(3{,}4;\ 2{,}5)$$

$$= 2{,}5$$

$$\alpha_b = \min\left(\frac{e_{1,b}}{3d_0};\ \frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{ub}}{f_{u,b}};\ 1{,}0\right) = \min\left(\frac{90}{3\times 22};\ \frac{70}{3\times 22} - \frac{1}{4};\ \frac{800}{430};\ 1{,}0\right)$$

$$= \min(1{,}36;\ 0{,}81;\ 1{,}86;\ 1{,}0) = 0{,}81$$

$$F_{b,ver,Rd} = \frac{2{,}5 \times 0{,}81 \times 430 \times 20 \times 9}{1{,}25} \times 10^{-3} = 125 \text{ kN}$$

The horizontal bearing resistance of a single bolt,

$$F_{b,hor,Rd} = \frac{k_1 \alpha_b f_{u,b} d t_w}{\gamma_{M2}}$$

$$k_1 = \min\left(2{,}8\frac{e_{1,b}}{d_0} - 1{,}7;\ 1{,}4\frac{p_1}{d_0} - 1{,}7;\ 2{,}5\right)$$

$$= \min\left(2{,}8\frac{90}{22} - 1{,}7;\ 1{,}4\frac{70}{22} - 1{,}7;\ 2{,}5\right) = \min(9{,}75;\ 2{,}75;\ 2{,}5) = 2{,}5$$

$$\alpha_b = \min\left(\frac{e_{2,b}}{3d_0};\ \frac{f_{ub}}{f_{u,b}};\ 1{,}0\right) = \min\left(\frac{40}{3\times 22};\ \frac{800}{430};\ 1{,}0\right)$$

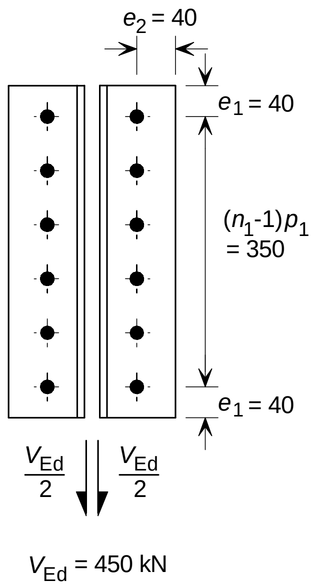
$$= \min(0{,}61;\ 1{,}86;\ 1{,}0) = 0{,}61$$

$$F_{b,hor,Rd} = \frac{2{,}5\times 0{,}61\times 430\times 20\times 9}{1{,}25}\times 10^{-3} = 94\ \text{kN}$$

$$V_{Rd} = \frac{6}{\sqrt{\left(\frac{1+0\times 6}{125}\right)^2+\left(\frac{0{,}102\times 6}{94}\right)^2}} = 583\ \text{kN}$$

V_{Ed} = 450 kN ≤ 583 kN, OK

4.2.1.2. Supporting beam side



Basic requirement: $V_{Ed} \leq F_{Rd}$

The design resistance of the bolt group, F_{Rd} :

If $(F_{b,Rd})_{max} \leq F_{v,Rd}$ then $F_{Rd} = \Sigma F_{b,Rd}$

If $(F_{b,Rd})_{min} \leq F_{v,Rd} < (F_{b,Rd})_{max}$ then $F_{Rd} = n_s\,(F_{b,Rd})_{min}$

If $F_{v,Rd} < (F_{b,Rd})_{min}$ then $F_{Rd} = 0{,}8\,n_s F_{v,Rd}$

Shear resistance of bolts

The shear resistance of a single bolt, $F_{v,Rd} = \dfrac{\alpha_v f_{ub} A}{\gamma_{M2}}$

For M20 8.8 bolts, $F_{v,Rd} = \dfrac{0{,}6\times 800\times 245}{1{,}25}\times 10^{-3}$ = 94 kN

Bearing resistance of bolts on the angle cleats

$$V_{\mathrm{b,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,ac}} d t_{\mathrm{ac}}}{\gamma_{\mathrm{M2}}}$$

For edge bolts, $k_1 = \min(2{,}8\frac{e_2}{d_0} - 1{,}7;\ 2{,}5) = \min(2{,}8 \times \frac{40}{22} - 1{,}7;\ 2{,}5)$

$= \min(3{,}39;\ 2{,}5) = 2{,}5$

For end bolts, $\alpha_{\mathrm{b}} = \min(\frac{e_1}{3d_0};\ \frac{f_{\mathrm{ub}}}{f_{\mathrm{u,ac}}};\ 1{,}0) = \min(\frac{40}{3 \times 22};\ \frac{800}{430};\ 1{,}0)$

$= \min(0{,}61;\ 1{,}86;\ 1{,}0) = 0{,}61$

For inner bolts, $\alpha_{\mathrm{b}} = \min(\frac{p_1}{3d_0} - \frac{1}{4};\ \frac{f_{\mathrm{ub}}}{f_{\mathrm{u,ac}}};\ 1{,}0)$

$= \min(\frac{70}{3 \times 22} - \frac{1}{4};\ \frac{800}{430};\ 1{,}0) = \min(0{,}81;\ 1{,}86;\ 1{,}0)$

$= 0{,}81$

End bolts, $F_{\mathrm{b,Rd,end}} = \left(F_{\mathrm{b,Rd}}\right)_{\min} = \frac{2{,}5 \times 0{,}61 \times 430 \times 20 \times 10}{1{,}25} \times 10^{-3}$

$= 105\ \mathrm{kN}$

Inner bolts, $F_{\mathrm{b,Rd,inner}} = \left(F_{\mathrm{b,Rd}}\right)_{\max} = \frac{2{,}5 \times 0{,}81 \times 430 \times 20 \times 10}{1{,}25} \times 10^{-3}$

$= 139\ \mathrm{kN}$

94 kN < 105 kN thus $F_{\mathrm{v,Rd}} < (F_{\mathrm{b,Rd}})_{\min}$

$F_{\mathrm{Rd}} = 0{,}8 n_{\mathrm{s}} F_{\mathrm{v,Rd}} = 0{,}8 \times 12 \times 94 = 902\ \mathrm{kN}$

$V_{\mathrm{Ed}} = 550\ \mathrm{kN} \leq 902\ \mathrm{kN}$, OK

4.2.2. Shear resistance of the angle cleats

4.2.2.1. Supported beam side

Basic requirement: $V_{\mathrm{Ed}} \leq V_{\mathrm{Rd,min}}$

$V_{\mathrm{Rd,min}} = \min(V_{\mathrm{Rd,g}};\ V_{\mathrm{Rd,n}};\ V_{\mathrm{Rd,b}})$

Shear resistance of gross section

$$V_{\mathrm{Rd,g}} = 2 \times \frac{h_{\mathrm{ac}} t_{\mathrm{ac}}}{1{,}27} \frac{f_{\mathrm{y,ac}}}{\sqrt{3}\gamma_{\mathrm{M0}}} = 2 \times \frac{430 \times 10 \times 275}{1{,}27 \times \sqrt{3} \times 1{,}0} \times 10^{-3} = 1076\ \mathrm{kN}$$

Shear resistance of net section

$$V_{\mathrm{Rd,n}} = 2 \times A_{\mathrm{v,net}} \frac{f_{\mathrm{u,ac}}}{\sqrt{3}\gamma_{\mathrm{M2}}}$$

Net area, $A_{v,net} = t_{ac}\left(h_{ac} - n_1 d_0\right) = 10\left(430 - 6 \times 22\right) = 2980 \text{ mm}^2$

$$V_{Rd,n} = 2 \times 2980 \times \frac{430}{\sqrt{3} \times 1,25} \times 10^{-3} = 1184 \text{ kN}$$

Block tearing resistance

$$V_{Rd,b} = 2\left(\frac{0,5 f_{u,ac} A_{nt}}{\gamma_{M2}} + \frac{f_{y,ac} A_{nv}}{\sqrt{3}\gamma_{M0}}\right)$$

Net area subject to tension, $A_{nt} = t_{ac}\left(e_2 - 0,5 d_0\right)$

$= 10\left(40 - 0,5 \times 22\right) = 290 \text{ mm}^2$

Net area subject to shear, $A_{nv} = t_{ac}\left(h_{ac} - e_1 - (n_1 - 0,5) d_0\right)$

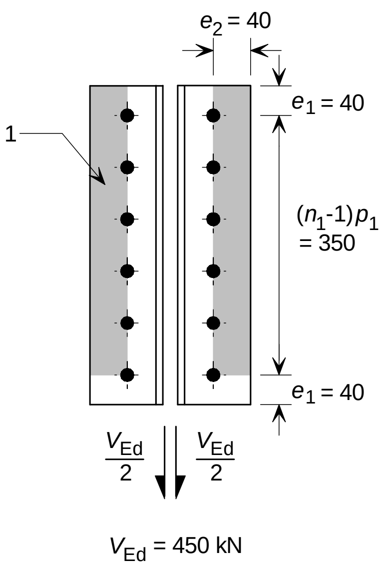
$= 10\left(430 - 40 - (6 - 0,5) 22\right) = 2690 \text{ mm}^2$

$$V_{Rd,b} = 2\left(\frac{0,5 \times 430 \times 290}{1,25} + \frac{275 \times 2690}{\sqrt{3} \times 1,0}\right) \times 10^{-3} = 954 \text{ kN}$$

$V_{Rd,min} = 954$ kN

$V_{Ed} = 450 \text{ kN} \leq 954$ kN, OK

4.2.2.2. Supporting beam side



1 Block shear failure

Basic requirement: $V_{Ed} \leq V_{Rd,min}$

$V_{Rd,min} = \min(V_{Rd,g}; V_{Rd,n}; V_{Rd,b})$

Shear resistance of gross section

$$V_{Rd,g} = 2 \times \frac{h_{ac} t_{ac}}{1,27} \frac{f_{y,ac}}{\sqrt{3}\gamma_{M0}} = 2 \times \frac{430 \times 10 \times 275}{1,27 \times \sqrt{3} \times 1,0} \times 10^{-3} = 1076 \text{ kN}$$

Shear resistance of net section

$$V_{\mathrm{Rd,n}} = 2 \times A_{\mathrm{v,net}} \frac{f_{\mathrm{u,ac}}}{\sqrt{3}\gamma_{\mathrm{M2}}}$$

Net area, $A_{\mathrm{v,net}} = t_{\mathrm{ac}}\left(h_{\mathrm{ac}} - n_1 d_0\right) = 10\left(430 - 6 \times 22\right) = 2980 \text{ mm}^2$

$$V_{\mathrm{Rd,n}} = 2 \times 2980 \times \frac{430}{\sqrt{3} \times 1,25} \times 10^{-3} = 1184 \text{ kN}$$

Block tearing resistance

$$V_{\mathrm{Rd,b}} = 2\left(\frac{0,5 f_{\mathrm{u,ac}} A_{\mathrm{nt}}}{\gamma_{\mathrm{M2}}} + \frac{f_{\mathrm{y,ac}} A_{\mathrm{nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}}\right)$$

Net area subject to tension, $A_{\mathrm{nt}} = t_{\mathrm{ac}}\left(e_2 - 0,5 d_0\right)$

$= 10\left(40 - 0,5 \times 22\right) = 290 \text{ mm}^2$

Net area subject to shear, $A_{\mathrm{nv}} = t_{\mathrm{ac}}\left(h_{\mathrm{ac}} - e_1 - (n_1 - 0,5) d_0\right)$

$= 10\left(430 - 40 - (6 - 0,5)22\right) = 2690 \text{ mm}^2$

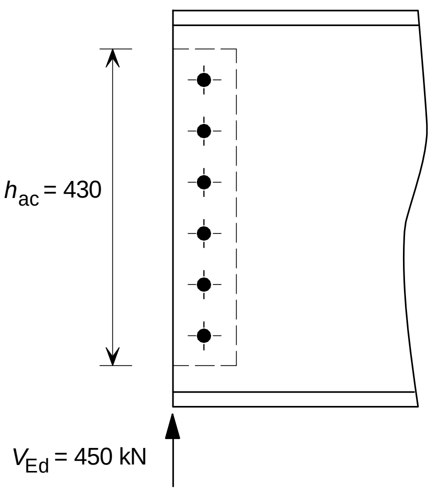
$$V_{\mathrm{Rd,b}} = 2\left(\frac{0,5 \times 430 \times 290}{1,25} + \frac{275 \times 2690}{\sqrt{3} \times 1,0}\right) \times 10^{-3} = 954 \text{ kN}$$

$V_{\mathrm{Rd,min}} = 954$ kN

$V_{\mathrm{Ed}} = 450 \text{ kN} \leq 954 \text{ kN}$, OK

4.2.3. Shear resistance of the beam web

4.2.3.1. Shear and block tearing resistance



Basic requirement: $V_{\mathrm{Ed}} \leq V_{\mathrm{Rd,min}}$

$V_{\mathrm{Rd,min}} = \min(V_{\mathrm{Rd,g}};\ V_{\mathrm{Rd,n}};\ V_{\mathrm{Rd,b}})$

Shear resistance of gross section

$$V_{\mathrm{Rd,g}} = A_{\mathrm{v,wb}} \frac{f_{\mathrm{y,b}}}{\sqrt{3}\gamma_{\mathrm{M0}}}$$

Shear area of beam web,

$A_{\mathrm{v,wb}} = A - 2bt_{\mathrm{f}} + (t_{\mathrm{w}} + 2r)t_{\mathrm{f}} = 11700 - 2\times 210\times 15{,}7 + (9 + 2\times 24)15{,}7$

$A_{\mathrm{v,wb}} = 6001\ \mathrm{mm}^2$

$\eta\, h_{\mathrm{w}}t_{\mathrm{w}} = 1{,}0\times 515{,}6\times 9 = 4640\ \mathrm{mm}^2$

$$V_{\mathrm{Rd,g}} = \frac{6001\times 275}{\sqrt{3}\times 1{,}0}\times 10^{-3} = 953\ \mathrm{kN}$$

Shear resistance of net section

$$V_{\mathrm{Rd,n}} = A_{\mathrm{v,wb,net}} \frac{f_{\mathrm{u,b}}}{\sqrt{3}\gamma_{\mathrm{M2}}}$$

Net area, $A_{\mathrm{v,wb,net}} = A_{\mathrm{v,wb}} - n_1 d_0 t_{\mathrm{w}} = 6001 - 6\times 22\times 9 = 4813\ \mathrm{mm}^2$

$$V_{\mathrm{Rd,n}} = 4813\times \frac{430}{\sqrt{3}\times 1{,}25}\times 10^{-3} = 956\ \mathrm{kN}$$

Block tearing resistance

$$V_{\mathrm{Rd,b}} = \left(\frac{0{,}5 f_{\mathrm{u,b}} A_{\mathrm{nt}}}{\gamma_{\mathrm{M2}}} + \frac{f_{\mathrm{y,b}} A_{\mathrm{nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}}\right)$$

Net area subject to tension, $A_{\mathrm{nt}} = t_{\mathrm{w}}\left(e_{2,\mathrm{b}} - 0{,}5 d_0\right)$

$= 9\left(40 - 0{,}5\times 22\right) = 261\ \mathrm{mm}^2$

Net area subject to shear, $A_{\mathrm{nv}} = t_{\mathrm{w}}\left(e_{1,\mathrm{b}} + (n_1 - 1)p_1 - (n_1 - 0{,}5)d_0\right)$

$= 9\left(90 + (6-1)70 - (6 - 0{,}5)22\right)$

$= 2871\ \mathrm{mm}^2$

$$V_{\mathrm{Rd,b}} = \left(\frac{0{,}5\times 430\times 261}{1{,}25} + \frac{275\times 2871}{\sqrt{3}\times 1{,}0}\right)\times 10^{-3} = 501\ \mathrm{kN}$$

$V_{\mathrm{Rd,min}} = \min(953;\ 956;\ 501) = 501\ \mathrm{kN}$

$V_{\mathrm{Ed}} = 450\ \mathrm{kN} \leq 501\ \mathrm{kN}$, OK

4.2.3.2. Shear and bending interaction at the 2nd line of bolts

Not applicable because it is un-notched

4.2.4. Bending resistance at the notch

Not applicable because it is un-notched

4.2.5. Local stability of the notched beam

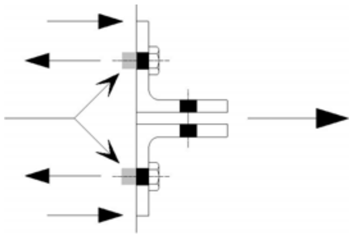
Not applicable because it is un-notched

4.3. Checks for tying

4.3.1. Angle cleats and bolt group resistance

4.3.1.1. Resistance of the angle cleats in bending

Basic requirement: $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd}}$



$$F_{\mathrm{Rd}} = \min\left(F_{\mathrm{Rd,u,1}}, F_{\mathrm{Rd,u,2}}, F_{\mathrm{Rd,u,3}}\right)$$

The tying resistance for Mode 1, $F_{\mathrm{Rd,u,1}}$ is given by:

$$F_{\mathrm{Rd,u,1}} = \frac{(8n - 2e_{\mathrm{w}})M_{\mathrm{pl,1,Rd,u}}}{2mn - e_{\mathrm{w}}(m+n)}$$

$$\Sigma l_{\mathrm{eff}} = 2e_{1\mathrm{A}} + (n_1 - 1)\, p_{1\mathrm{A}}$$

$$e_{1\mathrm{A}} = e_1 \text{ but} \leq 0,5(p_3 - t_{\mathrm{w}} - 2r) + \frac{d_0}{2}$$

$$0,5(109 - 9 - 2\times 11) + \frac{22}{2} = 50 \text{ mm}$$

$$\therefore\ e_{1\mathrm{A}} = 40 \text{ mm}$$

$$p_{1\mathrm{A}} = p_1 \text{ but} \leq p_3 - t_{\mathrm{w}} - 2r + d_0$$

$$p_3 - t_{\mathrm{w}} - 2r + d_0 = 109 - 9 - 2\times 11 + 22 = 100 \text{ mm}$$

$$\therefore\ p_{1\mathrm{A}} = 70 \text{ mm}$$

$$\Sigma l_{\mathrm{eff}} = 2e_{1\mathrm{A}} + (n_1 - 1)\, p_{1\mathrm{A}} = 2\times 40 + (6-1)70 = 430 \text{ mm}$$

$$M_{\mathrm{pl,1,Rd,u}} = \frac{0,25\Sigma l_{\mathrm{eff,1}} t_{\mathrm{f}}^{\,2} f_{\mathrm{u,ac}}}{\gamma_{\mathrm{Mu}}} = \frac{0,25\times 430\times 10^2 \times 430}{1,1}\times 10^{-6} = 4,2\text{kNm}$$

$$m = \frac{p_3 - t_{\mathrm{w}} - 2t_{\mathrm{ac}} - 2\times 0,8\times r}{2} = \frac{109 - 9 - 2\times 10 - 2\times 0,8\times 11}{2} = 31 \text{ mm}$$

$$e_{\mathrm{w}} = \frac{d_{\mathrm{w}}}{4} = \frac{37}{4} = 9,25\,\text{mm}$$

$$n = \min\left(e_2\,; 1,25m\right) = \min\left(40; 39\right) = 39 \text{ mm}$$

$$F_{\mathrm{Rd,u,1}} = \frac{(8\times 39 - 2\times 9,25)4,2\times 10^{3}}{2\times 31\times 39 - 9,25(31+39)} = 696 \text{ kN}$$

The tying resistance for mode 2, $F_{\mathrm{Rd,u,2}}$ is given by:

$$F_{\mathrm{Rd,u,2}} = \frac{2M_{\mathrm{pl,2,Rd,u}} + n\Sigma F_{\mathrm{t,Rd,u}}}{m+n}$$

$M_{\mathrm{pl,2,Rd,u}} = M_{\mathrm{pl,1,Rd,u}} = 4{,}20$ kNm

$$F_{\mathrm{t,Rd,u}} = \frac{k_2 f_{\mathrm{ub}} A}{\gamma_{\mathrm{Mu}}} = \frac{0,9\times 800\times 245}{1,1}\times 10^{-3} = 160 \text{ kN}$$

$$F_{\mathrm{Rd,u,2}} = \frac{2\times 4,2\times 10^{3} + 39\times 12\times 160}{31+39} = 1190 \text{ kN}$$

The tying resistance for mode 3, $F_{\mathrm{Rd,u,3}}$ is given by

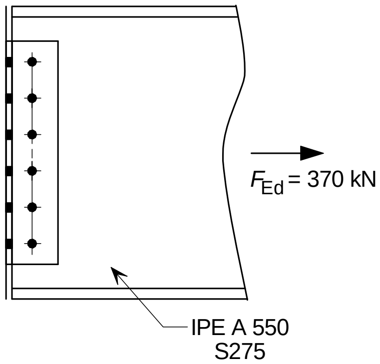
$F_{\mathrm{Rd,u,3}} = \Sigma F_{\mathrm{t,Rd,u}} = 12\times 160 = 1920$ kN

$F_{\mathrm{Rd}} = \min\left(F_{\mathrm{Rd,u,1}}, F_{\mathrm{Rd,u,2}}, F_{\mathrm{Rd,u,3}}\right)$

$F_{\mathrm{Rd}} = \min(696,\ 1190,\ 1920) = 696$ kN

$F_{\mathrm{Ed}} = 370 \text{ kN} \leq 696 \text{ kN}$, OK

4.3.1.2. Shear resistance of bolts



Basic requirement: $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd}}$

$F_{\mathrm{Rd}} = 2n_{\mathrm{b}}F_{\mathrm{v,u}}$

$$F_{\mathrm{v,u}} = \frac{\alpha_{\mathrm{v}} f_{\mathrm{ub}} A}{\gamma_{\mathrm{Mu}}} = \frac{0,6\times 800\times 245}{1,1}\times 10^{-3} = 107 \text{ kN}$$

$F_{\mathrm{Rd}} = 2\times 6\times 107 = 1284$ kN

$F_{\mathrm{Ed}} = 370 \text{ kN} \leq 1284 \text{ kN}$, OK

4.3.1.3. Bearing resistance of bolts on the angle cleats

Basic requirement: $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd}}$

$F_{\mathrm{Rd}} = 2n_{\mathrm{b}}F_{\mathrm{b,hor,u,Rd}}$

$$F_{\mathrm{b,hor,u,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,ac}} d t_{\mathrm{ac}}}{\gamma_{\mathrm{Mu}}}$$

$$k_1 = \min\left(2{,}8\frac{e_1}{d_0} - 1{,}7;\ 1.4\frac{p_1}{d_0} - 1{,}7;\ 2{,}5\right)$$

$$= \min\left(2{,}8\frac{40}{22} - 1{,}7;\ 1{,}4\frac{70}{22} - 1{,}7;\ 2{,}5\right) = \min(3{,}39;\ 2{,}75;\ 2{,}5) = 2{,}5$$

$$\alpha_{\mathrm{b}} = \min\left(\frac{e_2}{3d_0};\ \frac{f_{\mathrm{ub}}}{f_{\mathrm{u,ac}}};\ 1{,}0\right) = \min\left(\frac{40}{3\times 22};\ \frac{800}{430};\ 1{,}0\right)$$

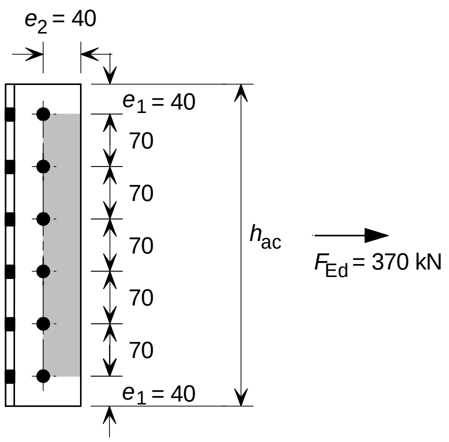
$$= \min(0{,}61;\ 1{,}86;\ 1{,}0) = 0{,}61$$

$$F_{\mathrm{b,hor,u,Rd}} = \frac{2{,}5\times 0{,}61\times 430\times 20\times 10}{1{,}1}\times 10^{-3} = 119\ \mathrm{kN}$$

$$F_{\mathrm{Rd}} = 2\times 6\times 119 = 1428\ \mathrm{kN}$$

$$F_{\mathrm{Ed}} = 350\ \mathrm{kN} \leq 1428\ \mathrm{kN},\quad \mathrm{OK}$$

4.3.1.4. Block tearing resistance



Basic requirement : $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd,b}}$

$$F_{\mathrm{Rd,b}} = \frac{f_{\mathrm{u,ac}} A_{\mathrm{nt}}}{\gamma_{\mathrm{Mu}}} + \frac{f_{\mathrm{y,ac}} A_{\mathrm{nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}}$$

Case 1

$$A_{\mathrm{nt}} = 2t_{\mathrm{ac}}\left[(n_1 - 1)p_1 - (n_1 - 1)d_0\right] = 2\times 10\times\left[(6-1)\times 70 - (6-1)\times 22\right]$$

$$= 4800\ \mathrm{mm}^2$$

$$A_{\mathrm{nv}} = 4t_{\mathrm{ac}}(e_2 - 0.5d_0) = 4\times 10(40 - 0{,}5\times 22) = 1160\ \mathrm{mm}^2$$

$$F_{\mathrm{Rd,b}} = \left(\frac{430\times 4800}{1{,}1} + \frac{275\times 1160}{\sqrt{3}\times 1{,}0}\right)\times 10^{-3} = 2060\ \mathrm{kN}$$

Case 2

$A_{nt} = 2t_{ac}[e_1 + (n_1 - 1)p_1 - (n_1 - 0{,}5)d_0]$

$A_{nt} = 2 \times 10 \times [40 + (6-1) \times 70 - (6 - 0{,}5) \times 22] = 5380 \text{ mm}^2$

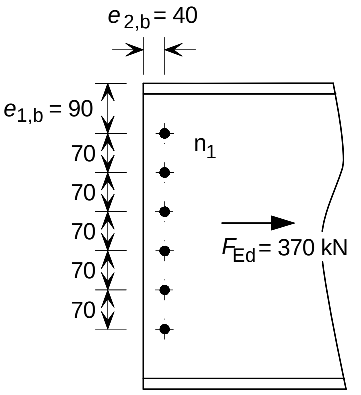
$A_{nv} = 2t_{ac}(e_2 - 0{,}5d_0) = 2 \times 10 \times (40 - 0{,}5 \times 22) = 580 \text{ mm}^2$

$$F_{Rd,b} = \left(\frac{430 \times 5380}{1{,}1} + \frac{275 \times 580}{\sqrt{3} \times 1{,}0}\right) \times 10^{-3} = 2195 \text{ kN}$$

$F_{Ed} = 370 \text{ kN} \leq 2060 \text{ and } 2195 \text{ kN}$, OK

4.3.2. Beam web resistance

4.3.2.1. Bearing resistance of bolts on the beam web



Basic requirement: $F_{Ed} \leq F_{Rd}$

$F_{Rd} = n_b F_{b,hor,u,Rd}$

$$F_{b,hor,u,Rd} = \frac{k_1 \alpha_b f_{u,b} d t_w}{\gamma_{Mu}}$$

$$k_1 = \min\left(1{,}4\frac{p_1}{d_0} - 1{,}7;\ 2{,}5\right) = \min\left(1{,}4\frac{70}{22} - 1{,}7;\ 2{,}5\right) = 2{,}5$$

$$\alpha_b = \min\left(\frac{e_{2,b}}{3d_0};\ \frac{f_{ub}}{f_{u,b}};\ 1{,}0\right) = \min\left(\frac{40}{3 \times 22};\ \frac{800}{430};\ 1{,}0\right) = 0{,}61$$

$$F_{b,hor,u,Rd} = \frac{2{,}5 \times 0{,}61 \times 430 \times 20 \times 9}{1{,}1} \times 10^{-3} = 107 \text{ kN}$$

$F_{Rd} = 6 \times 107 = 642 \text{ kN}$

$F_{Ed} = 370 \text{ kN} \leq 642 \text{ kN}$, OK

4.3.2.2. Tension resistance of the beam web

Basic requirement: $F_{Ed} \leq F_{Rd,n}$

$$F_{Rd,n} = 0{,}9 A_{net,wb} \frac{f_{u,b}}{\gamma_{Mu}}$$

$A_{net,wb} = t_w h_{ac} - d_0 n_1 t_w = 9 \times 430 - 22 \times 6 \times 9 = 2682 \text{ mm}^2$

$F_{\mathrm{Rd,n}} = 0,9 \times 2682 \dfrac{430}{1,1} \times 10^{-3} = 944$ kN

$F_{\mathrm{Ed}} = 370$ kN $\leq$ 944 kN, OK

4.3.2.3. Block tearing resistance

Basic requirement : $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd,b}}$

$$F_{\mathrm{Rd,b}} = \frac{f_{\mathrm{u,b}} A_{\mathrm{nt}}}{\gamma_{\mathrm{Mu}}} + \frac{f_{\mathrm{y,b}} A_{\mathrm{nv}}}{\sqrt{3}\gamma_{\mathrm{M0}}}$$

Case 1

$A_{\mathrm{nt}} = t_{\mathrm{w}}\left[(n_1 - 1)p_1 - (n_1 - 1)d_0\right] = 9\left[(6-1)\times 70 - (6-1)22\right]$

$= 2160 \text{ mm}^2$

$A_{\mathrm{nv}} = 2t_{\mathrm{w}}\left(e_{2,\mathrm{b}} - 0,5d_0\right) = 2 \times 9\left(40 - 0,5 \times 22\right) = 522 \text{ mm}^2$

$$F_{\mathrm{Rd,b}} = \left(\frac{430 \times 2160}{1,1} + \frac{275 \times 522}{\sqrt{3} \times 1,0}\right) \times 10^{-3} = 927 \text{ kN}$$

$F_{\mathrm{Ed}} = 370 \leq 927$ kN, OK

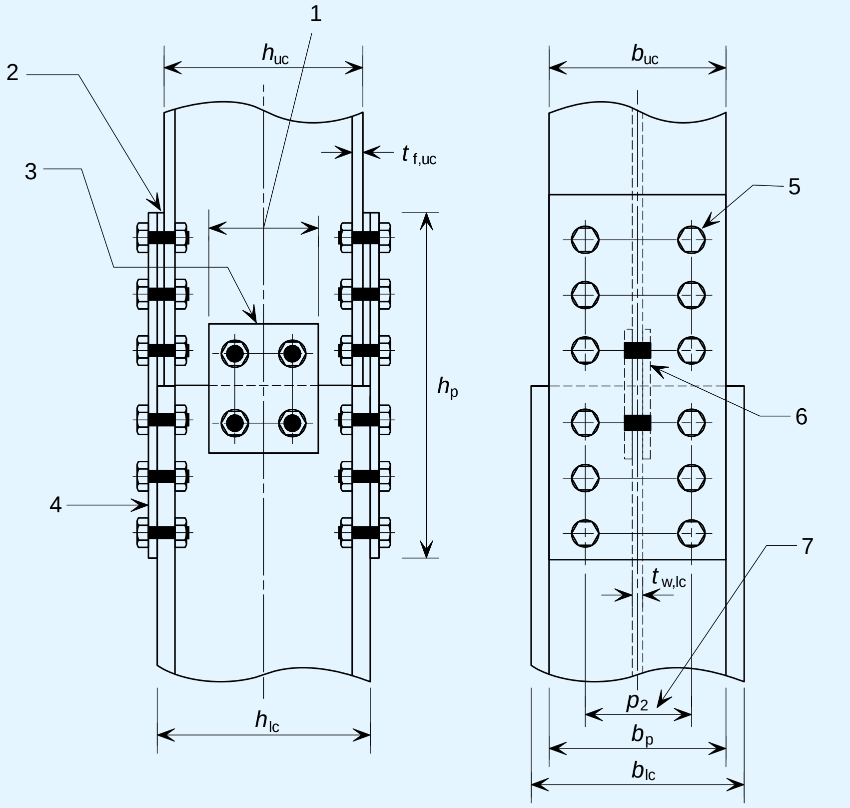
(Case 2 only applies to notched beams)

5 COLUMN SPLICES (BEARING TYPE)

Column splices are designed assuming they must resist both the axial compression and, where appropriate, a nominal moment from the connection to the beams.

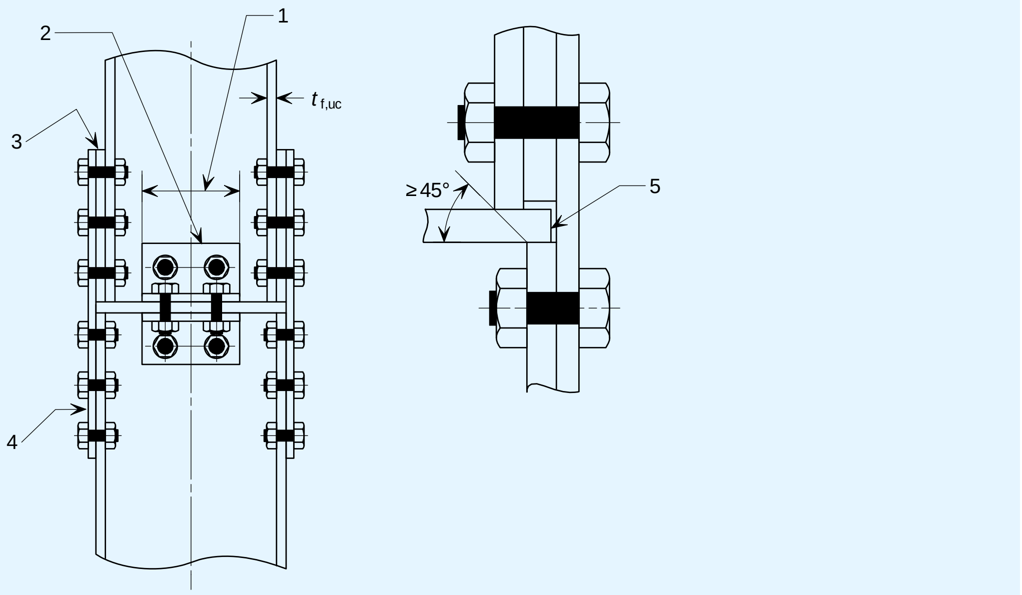
In bearing type splices, the axial load is transferred directly between the ends of the column sections (with a division plate if necessary due to the change of serial size) and not via the splices plates. The plates provide the splice with adequate stiffness and tying resistance.

5.1 Recommended details



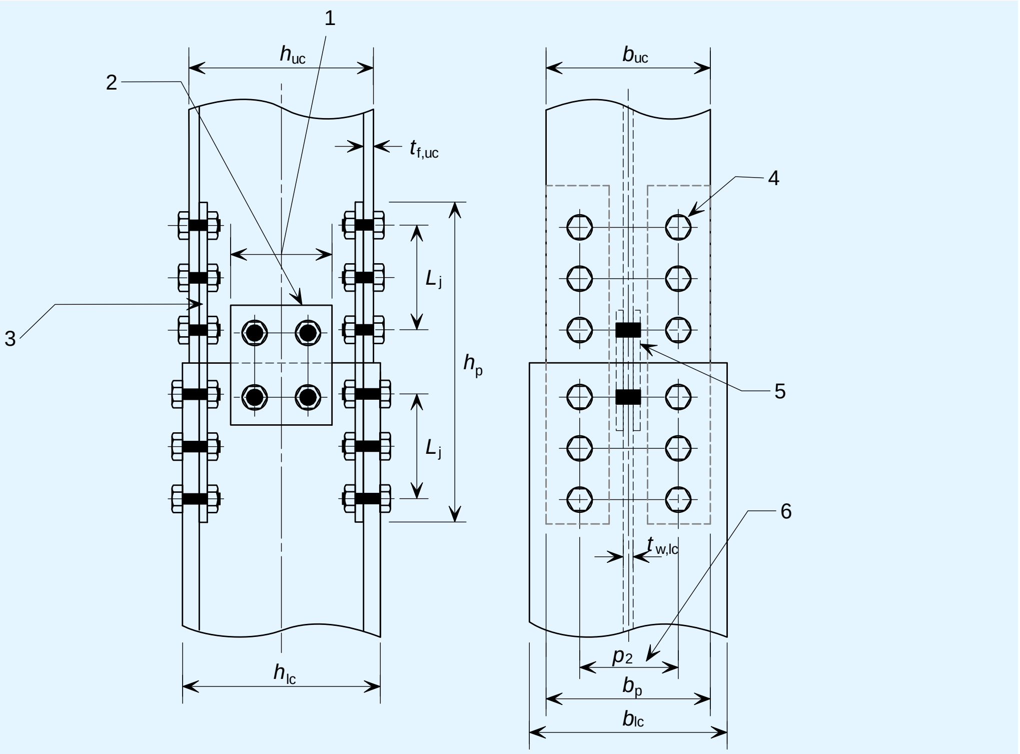
1 Web cover plate width ≥ 0,5h_{uc}
2 Multiple packs thickness t_{pa}
3 Web cover plate at least 4 no. M20, 8.8 bolts – See notes
For double-sided web cover plates, thickness ≥ $t_{w,uc}/2$
For single-sided web cover plates, thickness ≥ $t_{w,uc}$
4 Flange cover plate. Height: $h_p \geq 2b_{uc}$ and 225 mm; Width: $b_p \geq b_{uc}$;
Thickness: $t_p \geq t_{f,uc}/2$ and 10 mm
5 Bolts (normally untorqued in clearance holes) – See notes
6 Packs arranged as necessary
7 Wide bolt spacing for joint rigidity

Ends of column sections in direct bearing



1 Cleat length > 0,5h_{uc}
2 Web angle cleats at least 2 no. M20, 8.8 bolts each side
3 Multiple packs thickness t_{pa}
4 Flange cover plate as above
5 Division plate thickness should be at least, [(h_{lc} – 2$t_{f,lc}$) – (h_{uc} – 2$t_{f,lc}$)]/

Direct bearing onto a division plate



1 Web cover plate width ≥ 0,5h_{uc}
2 Web cover plate at least 4 no. M20, 8.8 bolts
3 Flange cover plate. Height: h_p ≥ 2b_{uc} and 450 mm; Width: b_p ≥ (b_{uc} – $t_{w,lc}$ – 2r_{lc})/2;
Thickness: t_p ≥ $t_{f,uc}$/2 and 10 mm, (r_{lc} = root radius)
4 Bolts (normally untorqued in clearance holes) – See notes
5 Packs arranged as necessary
6 Wide bolt spacing for joint rigidity

Internal flange cover plates

Notes:

1. Bolt spacing and edge distances should comply with the recommendations of EN 1993-1-8:2005
2. The thickness of the flange cover plate should be at least the minimum of 10, $t_{f,uc}/2$ and $p_1/14$.
3. The thickness mentioned in Note 2 is in most cases sufficient to transmit at least 25% of the maximum compressive force in the column, as required by EN 1993-1-8 [§6.2.7.1(14)].
4. The column splices should be located at approximately 600 mm above floor beam level. It is also recommended to use a minimum of four bolts. In a braced frame, columns containing such splices will behave satisfactorily even if the joint effectively behaves as a pin. In practice, typical bearing column splices as given in this guide will provide significant stiffness about both axes, although less than full stiffness.
5. In common practice the width of the flange cover plate would not be greater than the width of the lower column flange. However if the width of the flange cover plate is narrower than the upper column flange then edge and end distances should be checked against the Eurocode (EN 1993-1-8 Table 3.3).

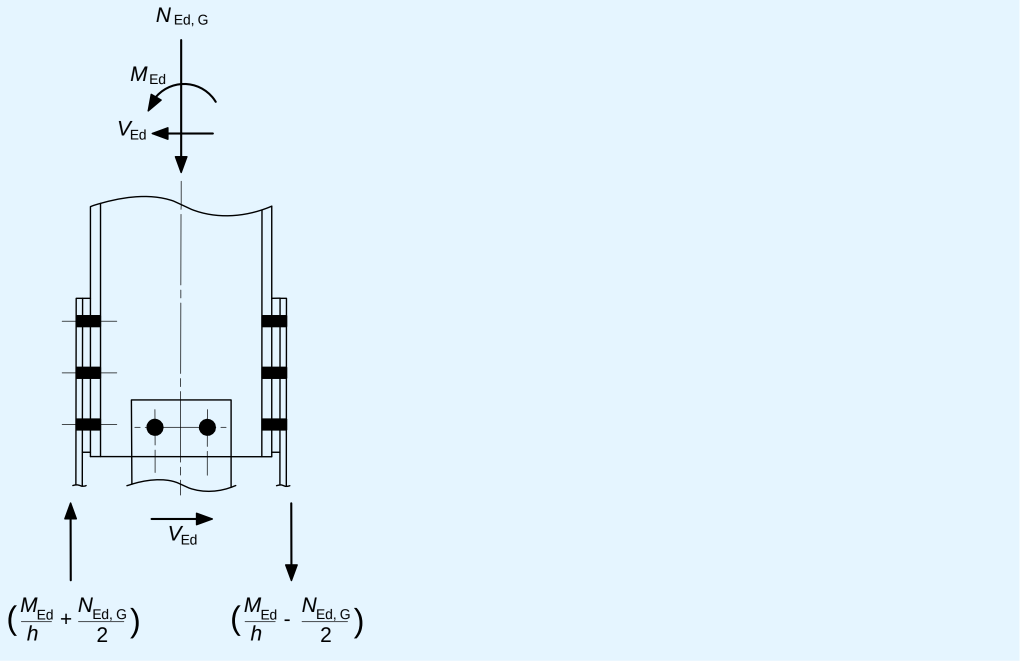
If there is significant net tension then the following notes should be adhered to:

6. Bolt diameters must be at least 75% of packing thickness t_{pa}[11].
7. The number of plies in multiple packs should not exceed four[11].
8. There should not be more than one jump in serial size of column at the splice.
9. If external and internal flange covers are to be provided, the size should be similar to those shown in the figures and the combined thickness of the external and internal cover plates must be at least $t_{f,uc}$ / 2. All cover plates should be at least 10 mm thick.

5.2 Checks for tension

5.2.1 Net tension

5.2.1.1 Net tension effects



The following checks the presence of net tension due to axial load and bending moment[4]:

If $M_{Ed} \leq \frac{N_{Ed,G}\,h}{2}$ then net tension does not occur and so the splice need only be detailed to transmit axial compression by direct bearing.

If $M_{Ed} > \frac{N_{Ed,G}\,h}{2}$ then net tension does occur and the flange cover plates and their fasteners should be checked for tensile force, F_{Ed}.

$$F_{Ed} = \frac{M_{Ed}}{h} - \frac{N_{Ed,G}}{2}$$

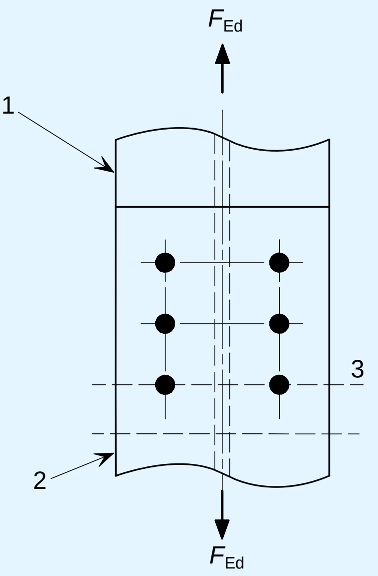
M_{Ed} is the nominal moment due to factored permanent and variable loads (i.e. column design moment) at the floor level immediately below the splice.

$N_{Ed,G}$ is the axial compressive force due to factored permanent loads only.

h is conservatively the overall depth of the smaller column (for external flange cover plates) or the centreline to centreline distance between the flange cover plates (for internal flange cover plates).

Preloaded bolts should be used when net tension induces stress in the upper column flange > 10% of the design strength of that column.

5.2.1.2 Tension resistance of the flange cover plate



1 Column flange
2 Flange cover plate
3 Critical section

Basic requirement: $F_{Ed} \leq N_{t,Rd}$

$N_{t,Rd} = \min(N_{pl,Rd}; N_{u,Rd}; N_{bt,Rd})$

Tension resistance of gross section

$N_{pl,Rd}$ is the tension plastic resistance of the gross section

$$N_{pl,Rd} = \frac{A_{fp}\, f_{y,p}}{\gamma_{M0}}$$ [EN 1993-1-1 §6.2.3(2)]

where:

A_{fp} is the gross area of the flange cover plate(s) attached to one flange

Tension resistance of net section

$N_{u,Rd}$ is the tension ultimate resistance of the net area

$$N_{u,Rd} = \frac{0,9\, A_{fp,net}\, f_{u,p}}{\gamma_{M2}}$$ [EN 1993-1-1 §6.2.3(2)]

where:

$A_{fp,net}$ is the net area of the flange cover plate(s) attached to one flange

$A_{fp,net} = A_{fp} - n_2 d_0 t_p$

Block tearing resistance

$N_{bt,Rd}$ is the block tearing resistance

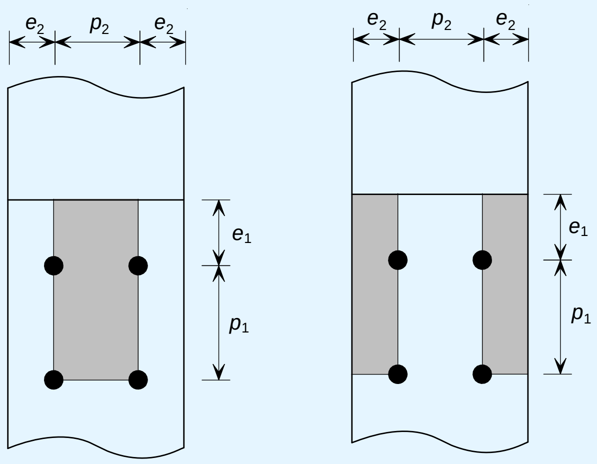


Figure A Figure B

For a concentrically loaded bolt group: $N_{bt,Rd} = V_{eff,1,Rd}$

$$V_{eff,1,Rd} = \frac{f_{u,p} A_{fp,nt}}{\gamma_{M2}} + \frac{f_{y,p} A_{fp,nv}}{\sqrt{3}\gamma_{M0}}$$ [EN 1993-1-8 §3.10.2(2)]

For eccentrically loaded bolt group: $N_{bt,Rd} = V_{eff,2,Rd}$

$$V_{eff,2,Rd} = \frac{0,5 f_{u,p} A_{fp,nt}}{\gamma_{M2}} + \frac{f_{y,p} A_{fp,nv}}{\sqrt{3}\gamma_{M0}}$$ [EN 1993-1-8 §3.10.2(3)]

where:

$f_{y,uc}$ is the yield strength of the upper column

$f_{u,uc}$ is the ultimate tensile strength of the the upper column

$A_{fp,nv}$ is the net area of the flange cover plate subjected to shear

$A_{fp,nv} = 2t_p\,(e_1 + (n_1 - 1)p_1 - (n_1 - 0,5)d_0)$

$A_{fp,nt}$ is the net area of the flange cover plate subjected to tension

If $p_2 \leq 2e_2$ $A_{fp,nt} = t_p(p_2 - d_0)$ (Figure A)

If $p_2 > 2e_2$ $A_{fp,nt} = t_p(2e_2 - d_0)$ (Figure B)

γ_{M2} is the partial factor for the resistance of net sections

Check for significant net tension:

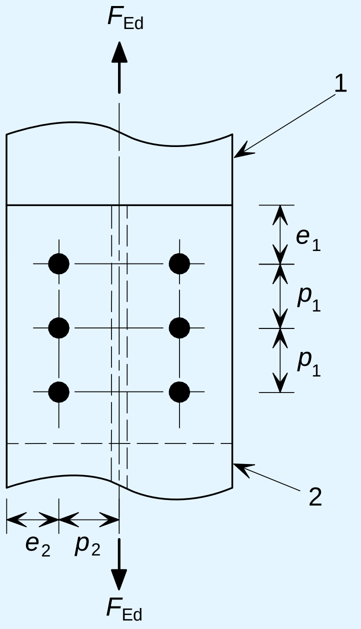
If $\frac{F_{Ed}}{t_{f,uc} b_{f,uc} f_{y,uc}} > 0,1$ then preloaded bolts should be used[4].

where:

$t_{f,uc}$ is the flange thickness of the upper column

$b_{f,uc}$ is the flange width of the upper column

5.2.1.3 Bolt group resistance



1 Column flange
2 Flange cover plate

Basic requirement: $F_{Ed} \leq F_{Rd}$

$F_{Rd,fp}$ is the design resistance of bolt group [EN 1993-1-8, §3.7(1)]

$F_{Rd} = \Sigma F_{b,Rd}$ if $(F_{b,Rd})_{max} \leq F_{v,Rd}$

$F_{Rd} = n_{fp}(F_{b,Rd})_{min}$ if $(F_{b.Rd})_{min} \leq F_{v,Rd} \leq (F_{b,Rd})_{max}$

$F_{Rd} = n_{fp}F_{v,Rd}$ if $F_{v,Rd} \leq (F_{b,Rd})_{min}$

n_{fp} is the number of bolts between one flange cover plate and upper column

Shear resistance of bolts

$F_{v,Rd}$ is the shear resistance of a single bolt

$$F_{v,Rd} = \beta_p \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$ [EN 1993-1-8, Table 3.4]

where:

α_v = 0,6 for 4.6 and 8.8 bolts = 0,5 for 10.9 bolts

A is the tensile stress area of the bolt, A_s

β_p = 1,0 if $t_{pa} \leq d/3$ [EN 1993-1-8 §3.6.1(12)]

$= \dfrac{9d}{8d + 3t_{pa}}$ if $t_{pa} > d/3$

t_{pa} is the total thickness of the packing

γ_{M2} is the partial factor for resistance of bolts

Check for long joint:

L_j is the joint length from EN1993-1-8[1], § 3.8

If $L_j > 15d$ the design shear resistance $F_{v,Rd}$ should be reduced by multiplying it by a reduction factor β_{Lf}.

$$\beta_{Lf} = 1 - \frac{L_j - 15d}{200d}$$

Bearing resistance

$F_{b,Rd}$ is the bearing resistance of a single bolt

$$F_{b,Rd} = \frac{k_1 \alpha_b f_{u,p} d t_p}{\gamma_{M2}}$$ [EN 1993-1-8 Table 3.4]

Note: If the thickness of the column flange is less than the thickness of the flange cover plates, then the bearing resistance of the column flange should also be checked.

For end bolts:

$$\alpha_b = \min\left(\frac{e_1}{3d_0};\quad \frac{f_{ub}}{f_{u,p}};\quad 1,0\right)$$

For inner bolts:

$$\alpha_b = \min\left(\frac{p_1}{3d_0} - \frac{1}{4};\quad \frac{f_{ub}}{f_{u,p}};\quad 1,0\right)$$

For edge bolts:

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

For inner bolts:

$$k_1 = \min\left(1,4\frac{p_2}{d_0} - 1,7;\quad 2,5\right)$$

γ_{M2} is the partial factor for plate in bearing

Resistance of preloaded bolts:

$F_{Ed} \leq F_{s,Rd}$

For joints designed to be non-slip under factored loads.

$F_{s,Rd}$ is the design slip resistance

$$= \frac{k_s n_{fs} \mu}{\gamma_{M3}} F_{p,C}$$ [EN 1993-1-8 §3.9.1(1)]

where:

k_s = 1.0 for fasteners in standard clearance holes (Table 3.6 of EN1993-1-8)[1]

n_{fs} is the number of friction surfaces

μ is the slip factor (Table 18 of EN1090-2 [12])

$F_{p,C} = 0{,}7\, f_{ub}\, A_s$ [EN 1993-1-8 §3.6.1(2)]

A_s is the tensile stress area of the bolt

γ_{M3} is the partial factor for slip resistance at ultimate limit state

5.3 Check for horizontal shear

For a bearing type splice, any horizontal shear V_{Ed} is assumed to be resisted by friction across the splice interface[4].

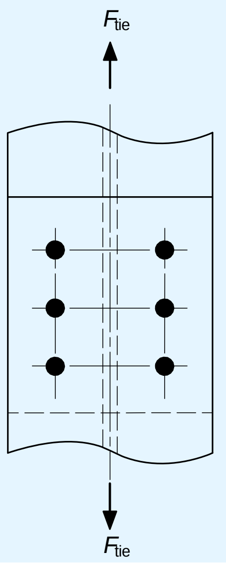
Basic requirement: $V_{Ed} \leq$ shear resistance of splice interface

The coefficient of friction μ_f for a steel interface depends upon the surface condition of the steel and on any coatings provided.

Conservatively, for steel with no surface treatment, with mill scale, the coefficient of friction, μ_f may be taken as 0,2.

Shear resistance of splice interface = Vertical load × Coefficient of friction

5.4 Checks for vertical tying



If it is necessary to comply with structural integrity requirements, then checks 4.2.1.2 and 4.2.1.3 should be carried out[4] with:

$$F_{Ed} = \frac{F_{tie}}{2}$$

The yield strength should be replaced with the ultimate strength.

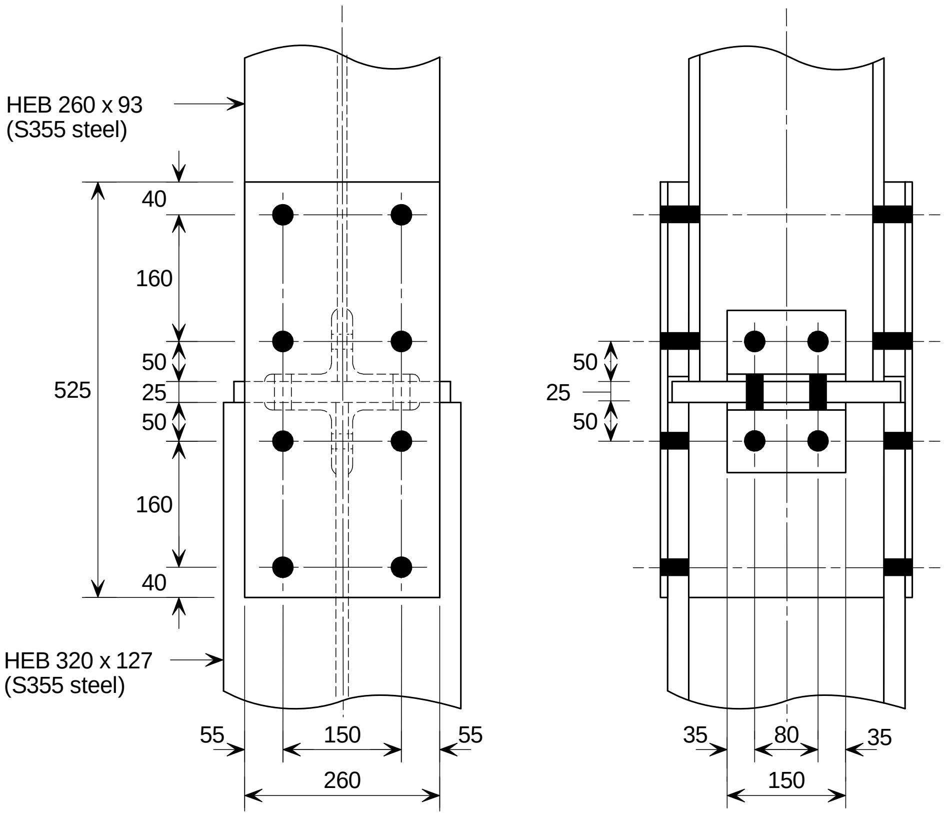
The partial safety factors (γ_{M0}, γ_{M2}) should also be replaced with the partial factor for tying resistance. (γ_{Mu} = 1,1).

Note:

1. The structural integrity checks are based on the conservative assumption that the tie force is resisted by the two flange cover plates.
5. F_{tie} is the tensile force from EN1991-1-7, § A.6.

5. Column Splice

Details and data



Flange cover plates:	2/260 × 12 × 525
Flange packs:	2/260 × 30 × 240
Cleats:	4/90 × 90 × 8 Angles × 150 long
Web Packs:	2/85 × 2 × 150
Division plate:	265 × 25 × 310
Bolts:	M20 8.8
Fittings material:	S275 steel

Summary of full design checks

Unless noted otherwise, all references are to EN 1993-1-8

Design forces

$N_{Ed,G}$ = 760 kN

$N_{Ed,Q}$ = 870 kN

M_{Ed} = 110 kNm (about yy axis of column)

V_{Ed} = 60 kN

Tension resistances

Net tension

Tensile resistance of the flange cover plate	802 kN
Bolt group resistance	272 kN

Horizontal shear resistance	161 kN

Tying resistances

Tensile resistance of the flange cover plate	912 kN
Bolt group resistance	308 kN

5.1. Recommended details

External flange cover plates

Height, $h_p \geq 2b_{uc}$ and 450 mm

Width, $b_p \geq b_{uc}$ = 260 mm Say 260 mm, OK

Maximum vertical bolt spacing, $p_1 = 14t_p$, i.e. minimum thickness is p_1 /14

Thickness, t_p $\geq \dfrac{t_{f,uc}}{2}$ and 10 mm and $\dfrac{p_1}{14}$

$= \dfrac{17,5}{2}$ and 10 mm and $\dfrac{160}{14}$

= 8,75 mm and 10 mm and 11,4 mm

Say 12 mm, OK

Packs, t_{pa} $= \dfrac{h_{lc} - h_{uc}}{2} = \dfrac{320 - 260}{2}$ = 30 mm

Say 30 mm, OK

Division plate

Thickness $\geq \dfrac{\left[\left(h_{lc} - 2t_{f,lc}\right) - \left(h_{uc} - 2t_{f,uc}\right)\right]}{2}$

$= \dfrac{\left[\left(320 - 2 \times 20,5\right) - \left(260 - 2 \times 17,5\right)\right]}{2}$ = 27 mm

Say 25mm, OK

Web cleats

Use 90×90×8 angles to accommodate M20 bolts in opposite positions on adjoining legs.

Length $\geq 0{,}5h_{uc}$ $= 0{,}5 \times 260 = 130$ mm Say 150 mm, OK

Packs, t_{pa} $= \dfrac{t_{w,lc} - t_{w,uc}}{2} = \dfrac{11{,}5 - 10}{2} = 0{,}8$ mm Say 2 mm, OK

5.2. Checks for vertical shear

5.2.1. Net tension

5.2.1.1. Net tension effects

Basic requirement for no net tension: $M_{Ed} \leq \dfrac{N_{Ed,G} \times h}{2}$

$$\frac{N_{Ed,G} \times h}{2} = \frac{760 \times 260}{2} \times 10^{-3} = 99 \text{ kNm}$$

M_{Ed} = 110 kNm > 99 kNm

Net tension does occur and the flange cover plates and their fastenings must be checked for a tensile force F_{Ed}.

$$F_{Ed} = \frac{M_{Ed}}{h} - \frac{N_{Ed,G}}{2} = \frac{110}{260 \times 10^{-3}} - \frac{760}{2} = 43 \text{ kN}$$

5.2.1.2. Tension resistance of the flange cover plate

Basic requirement: $F_{Ed} \leq N_{t,Rd}$

Where $N_{t,Rd} = \min\left(N_{pl,Rd}\,;N_{u,Rd}\,;N_{bt,Rd}\right)$

Tension resistance of gross section

$$N_{pl,Rd} = \frac{A_{fp}\, f_{y,p}}{\gamma_{M0}}$$

EN 1993-1-1 § 6.2.3(2)

Gross area, $A_{fp} = 260 \times 12 = 3120$ mm^2

$$N_{pl,Rd} = \frac{3120 \times 275}{1{,}0} \times 10^{-3} = 858 \text{ kN}$$

Tension resistance of net section

$$N_{u,Rd} = \frac{0{,}9\, A_{fp,net}\, f_{u,p}}{\gamma_{M2}}$$

EN 1993-1-1 § 6.2.3(2)

Net area, $A_{fp,net} = 260 \times 12 - 2 \times 22 \times 12 = 2592$ mm^2

$$N_{u,Rd} = \frac{0{,}9 \times 2592 \times 430}{1{,}25} \times 10^{-3} = 802 \text{ kN}$$

Thus $N_{u,Rd} = 802$ kN

Block tearing resistance

For concentrically loaded bolt group: $N_{\mathrm{bt,Rd}} = V_{\mathrm{eff,1,Rd}}$ — § 3.10.2(3)

$2e_2 \quad = 2\times55 = 110$ mm

$p_2 \quad = 150 \leq 2e_2$

Hence

$A_{\mathrm{fp,nt}} = t_{\mathrm{p}}(\, 2e_2 - d_0\,) = 12\,(2\times55 - 22) = 1056 \text{ mm}^2$

$A_{\mathrm{fp,nv}} = 2t_{\mathrm{p}}\,(\, e_1 + (n_1 - 1)p_1 - (n_1 - 0{,}5)d_0\,)$

$= 2\times12\,(40 + (2 - 1)\times160 - (2 - 0{,}5)\times22) = 4008 \text{ mm}^2$

$$V_{\mathrm{eff,1,Rd}} = \left(\frac{430\times1056}{1{,}25} + \frac{275\times4008}{\sqrt{3}\times1{,}0}\right)\times10^{-3} = 1000 \text{ kN}$$

$N_{\mathrm{bt,Rd}} = 1000$ kN

$N_{\mathrm{t,Rd}} = \min(858;\, 802;\, 1000) = 802$ kN

$F_{\mathrm{Ed}} = 43 \text{ kN} \leq 802 \text{ kN}$, OK

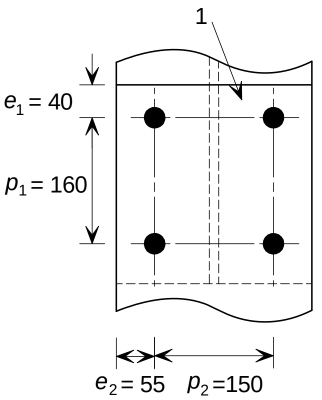
Check for the suitability of ordinary bolts.

(It is sufficiently accurate to base this calculation on the gross area of the flange) — Ref [4]

$$\frac{F_{\mathrm{Ed}}}{t_{\mathrm{f,uc}}\, b_{\mathrm{f,uc}}\, f_{\mathrm{y,uc}}} = \frac{43\times10^{3}}{12{,}5\times260\times355} = 0{,}04 < 0{,}1$$

There is no significant net tension in the column flange and the use of ordinary bolts in clearance holes is satisfactory.

5.2.1.3. Bolt group resistance



1 Flange cover plate

Shear and bearing resistance of the flange cover plate

Basic requirement: $F_{\mathrm{Ed}} \leq F_{\mathrm{Rd}}$

The design resistance of the bolt group, $F_{\mathrm{Rd,fp}}$: § 3.7

$F_{\mathrm{Rd}} = \Sigma F_{\mathrm{b,Rd}}$ if $(F_{\mathrm{b,Rd}})_{\max} \leq F_{\mathrm{v,Rd}}$

$F_{\mathrm{Rd}} = n_{\mathrm{fp}}(F_{\mathrm{b,Rd}})_{\min}$ if $(F_{\mathrm{b,Rd}})_{\min} \leq F_{\mathrm{v,Rd}} < (F_{\mathrm{b,Rd}})_{\max}$

$F_{\mathrm{Rd}} = n_{\mathrm{fp}} F_{\mathrm{v,Rd}}$ if $F_{\mathrm{v,Rd}} < (F_{\mathrm{b,Rd}})_{\min}$

Shear resistance of bolts

The shear resistance of a single bolt, $F_{\mathrm{v,Rd}} = \dfrac{\alpha_{\mathrm{v}} f_{\mathrm{ub}} A}{\gamma_{\mathrm{M2}}}$ Table 3.4

A factor to account for the long joint effect must be introduced if $L_{\mathrm{j}} > 15d$ § 3.8

$15d = 15 \times 20 = 300$ mm

$L_{\mathrm{j}} = 160$ mm, $< 15d$

Therefore there is no long joint effect.

Total thickness of flange pack, $t_{\mathrm{pa}} = 30\text{mm} > \dfrac{d}{3} = 6{,}7$ mm

Therefore $F_{\mathrm{v,Rd}}$ must be multiplied by a reduction factor β_{p}.

$$\beta_{\mathrm{p}} = \frac{9d}{8d + 3t_{\mathrm{pa}}} = \frac{9 \times 20}{8 \times 20 + 3 \times 30} = 0{,}72$$

For M20 8.8 bolts, $F_{\mathrm{v,Rd}} = 0{,}72 \times \dfrac{0{,}6 \times 800 \times 245}{1{,}25} \times 10^{-3} = 68$ kN

Bearing resistance

Bearing resistance, $F_{\mathrm{b,Rd}} = \dfrac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,p}} d t_{\mathrm{p}}}{\gamma_{\mathrm{M2}}}$ Table 3.4

For edge bolts, $k_1 = \min\left(2{,}8\dfrac{e_2}{d_0} - 1{,}7;\ 2{,}5\right) = \min\left(2{,}8\dfrac{55}{22} - 1{,}7;\ 2{,}5\right)$

$= \min(5{,}3;\ 2{,}5) = 2{,}5$

For end bolts $\alpha_{\mathrm{b}} = \min\left(\dfrac{e_1}{3d_0};\ \dfrac{f_{\mathrm{ub}}}{f_{\mathrm{u,p}}};\ 1{,}0\right) = \min\left(\dfrac{40}{3 \times 22};\ \dfrac{800}{430};\ 1{,}0\right)$

$= \min(0{,}61;\ 1{,}86;\ 1{,}0) = 0{,}61$

For inner bolts, $\alpha_{\mathrm{b}} = \min\left(\dfrac{p_1}{3d_0} - 0{,}25;\ \dfrac{f_{\mathrm{ub}}}{f_{\mathrm{u,p}}};\ 1{,}0\right)$

$= \min\left(\dfrac{160}{3 \times 22} - 0{,}25;\ \dfrac{800}{430};\ 1{,}0\right)$

$= \min(2{,}17;\ 1{,}86;\ 1{,}0) = 1{,}0$

End bolts, $F_{b,Rd,end} = \left(F_{b,Rd}\right)_{min} = \dfrac{2,5 \times 0,61 \times 430 \times 20 \times 12}{1,25} \times 10^{-3}$

$= 126$ kN

Inner bolts, $F_{b,Rd,inner} = \left(F_{b,Rd}\right)_{max} = \dfrac{2,5 \times 1,0 \times 430 \times 20 \times 12}{1,25} \times 10^{-3}$

$= 206$ kN

Thus $F_{v,Rd} < \left(F_{b,Rd}\right)_{min}$

$F_{Rd} = n_{fp} \times F_{v,Rd} = 4 \times 68 = 272$ kN

$F_{Ed} = 43$ kN $\leq$ 272 kN, OK

5.2.2. Check for horizontal shear

For a bearing type splice, any horizontal shear V_{Ed} is assumed to be resisted by friction across the splice interface. — Ref [4]

Basic requirement: $V_{Ed} \leq$ shear resistance of splice interface

Vertical load with coexistent shear

$$\frac{M_{Ed}}{h} + \frac{N_{Ed,G}}{2} = \frac{110 \times 10^{3}}{260} + \frac{760}{2} = 803 \text{ kN}$$

Shear resistance of splice interface: 803×0,2 = 161kN

$V_{Ed} = 60$ kN $\leq$ 161 kN, OK

5.3. Structural integrity

5.3.1. Tying force

Check 4.2.1.2 and 4.2.1.3 should be carried out with:

$$F_{Ed} = \frac{F_{tie}}{2}$$

$$F_{tie} = 2 \times F_{Ed} = 2 \times 43 = 86 \text{ kN}$$

5.3.2. Tensile resistance of the flange cover plate

Basic requirement: $F_{tie} \leq N_{t,Rd}$

Where $N_{t,Rd} = \min\left(N_{pl,Rd}\, ; N_{u,Rd}\, ; N_{bt,Rd}\right)$

5.3.2.1. Tension resistance of the gross area

$$N_{pl,Rd} = \frac{A_{fp}\, f_{u,p}}{\gamma_{Mu}}$$

EN 1993-1-1 § 6.2.3(2)

Gross area, $A_{fp} = 260 \times 12 = 3120$ mm^2

$$N_{pl,Rd} = \frac{3120 \times 430}{1,1} \times 10^{-3} = 1220 \text{ kN}$$

5.3.2.2. Tension resistance of the net area

EN 1993-1-1 § 6.2.3(2)

$$N_{u,Rd} = \frac{0,9\,A_{fp,net}\,f_{u,p}}{\gamma_{Mu}}$$

Net area, $A_{fp,net} = 260\times12 - 2\times22\times12 = 2592$ mm^2

$$N_{u,Rd} = \frac{0,9\times2592\times430}{1,1}\times10^{-3} = 912 \text{ kN}$$

Thus $N_{u,Rd} = 912$ kN

5.3.2.3. Block tearing resistance

For concentrically loaded bolt group: $N_{bt,Rd} = V_{eff,1,Rd}$ — Table 3.4

$2e_2 = 2\times55 = 110$ mm

$p_2 = 150 \leq 2e_2$

Hence

$A_{fp,nt} = t_p(\,2e_2 - d_0\,) = 12\,(2\times55 - 22) = 1056$ mm^2

$A_{fp,nv} = 2t_p\,(\,e_1+(n_1-1)p_1-(n_1-0,5)d_0\,)$

$= 2\times12\,[40 + (2-1)\times160 - (2-0,5)\times22] = 4008$ mm^2

$$V_{eff,1,Rd} = \left(\frac{430\times1056}{1,1}+\frac{275\times4008}{\sqrt{3}\times1,0}\right)\times10^{-3} = 1049 \text{ kN}$$

$N_{bt,Rd} = 1049$ kN

$N_{t,Rd} = \min(1220;\ 912;\ 1049) = 802$ kN

$F_{tie} = 86 \text{ kN} \leq 912$ kN, OK

5.3.2.4. Bolt group resistance

Shear and bearing resistance of the flange cover plate

Basic requirement: $F_{tie} \leq F_{Rd}$

The design resistance of the bolt group, $F_{Rd,fp}$: — § 3.7

$F_{Rd} = \Sigma F_{b,Rd}$ if $\left(F_{b,Rd}\right)_{max} \leq F_{v,Rd}$

$F_{Rd} = n_{fp}\,(F_{b,Rd})_{min}$ if $(F_{b,Rd})_{min} \leq F_{v,Rd} < (F_{b,Rd})_{max}$

$F_{Rd} = n_{fp}\,F_{v,Rd}$ if $F_{v,Rd} < \left(F_{b,Rd}\right)_{min}$

Shear resistance of bolts

The shear resistance of a single bolt, $F_{v,Rd} = \dfrac{\alpha_v\,f_{ub}\,A}{\gamma_{Mu}}$ — Table 3.4

A factor to account for the long joint effect must be introduced if $L_j > 15d$

$15d = 15\times20 = 300$ mm

$L_j = 160$ mm, $< 15d$

Therefore there is no long joint effect.

Total thickness of flange pack, t_{pa} = 30mm > $\frac{d}{3} = 6,7$ mm

Therefore $F_{\mathrm{v,Rd}}$ must be multiplied by a reduction factor β_{p}.

$$\beta_{\mathrm{p}} = \frac{9d}{8d + 3t_{\mathrm{pa}}} = \frac{9 \times 20}{8 \times 20 + 3 \times 30} = 0,72$$

For M20 8.8 bolts, $F_{\mathrm{v,Rd}} = 0,72 \times \frac{0,6 \times 800 \times 245}{1,1} \times 10^{-3}$ = 77 kN

Bearing resistance

Bearing resistance, $F_{\mathrm{b,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u,p}} d t_{\mathrm{p}}}{\gamma_{\mathrm{Mu}}}$

Table 3.4

For edge bolts, $k_1 = \min\left(2,8\frac{e_2}{d_0} - 1,7;\ 2,5\right) = \min\left(2,8\frac{55}{22} - 1,7;\ 2,5\right)$

$= \min(5,3;\ 2,5) = 2,5$

For end bolts $\alpha_{\mathrm{b}} = \min\left(\frac{e_1}{3d_0};\ \frac{f_{\mathrm{ub}}}{f_{\mathrm{u,p}}};\ 1,0\right) = \min\left(\frac{40}{3 \times 22};\ \frac{800}{430};\ 1,0\right)$

$= \min(0,61;\ 1,86;\ 1,0) = 0,61$

For inner bolts, $\alpha_{\mathrm{b}} = \min\left(\frac{p_1}{3d_0} - 0,25;\ \frac{f_{\mathrm{ub}}}{f_{\mathrm{u,p}}};\ 1,0\right)$

$= \min\left(\frac{160}{3 \times 22} - 0,25;\ \frac{800}{430};\ 1,0\right)$

$= \min(2,17;\ 1,86;\ 1,0) = 1,0$

End bolts, $F_{\mathrm{b,Rd,end}} = \left(F_{\mathrm{b,Rd}}\right)_{\min} = \frac{2,5 \times 0,61 \times 430 \times 20 \times 12}{1,1} \times 10^{-3}$

= 143kN

Inner bolts, $F_{\mathrm{b,Rd,inner}} = \left(F_{\mathrm{b,Rd}}\right)_{\max} = \frac{2,5 \times 1,0 \times 430 \times 20 \times 12}{1,1} \times 10^{-3}$

= 235 kN

Thus $F_{\mathrm{v,Rd}} < \left(F_{\mathrm{b,Rd}}\right)_{\min}$

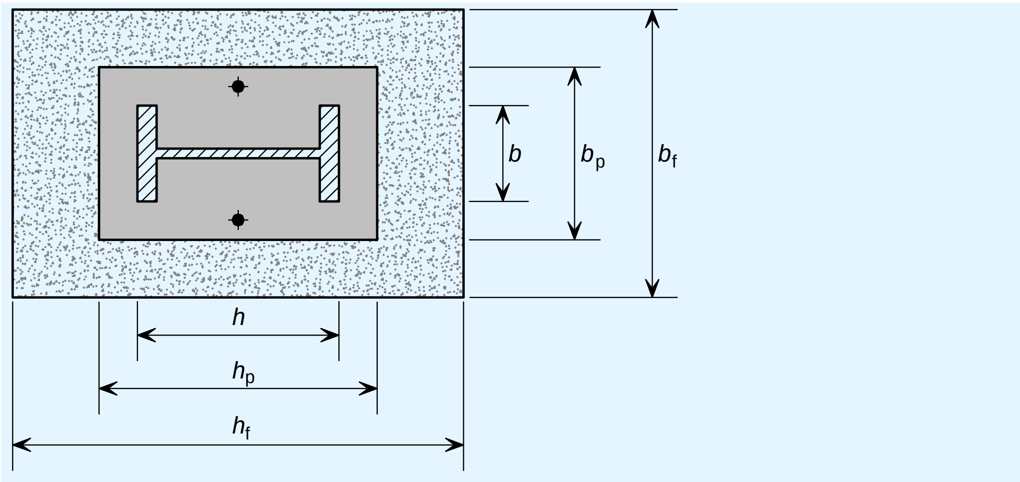
$F_{\mathrm{Rd}} = n_{\mathrm{fp}} \times F_{\mathrm{v,Rd}} = 4 \times 77 = 308$ kN

$F_{\mathrm{tie}} = 86$ kN $\leq$ 308 kN, OK

6 COLUMN BASES

This design method applied to fixed bases of I section columns transmitting an axial compressive force, and a shear force (i.e. a nominally pinned column base). The rectangular base plate is welded to the column section in a symmetrically position so that it has projections beyond the column flange outer edges on all sides.

6.1 Base plate size



Basic requirement: $A_p \geq A_{req}$ [Reference 4]

A_p = area of base plate

= $h_p b_p$ for rectangular plates

A_{req} = required area of base plate

$$= \frac{F_{Ed}}{f_{jd}}$$

$$f_{jd} = \frac{2}{3}\alpha f_{cd}$$

where:

$$\alpha = \min\left[\left(1+\frac{d_f}{\max(h_p, b_p)}\right), \left(1+2\frac{e_h}{h_p}\right), \left(1+2\frac{e_b}{b_p}\right), 3\right]$$ [Reference 3]

If some dimensions are unknown, a value of α = 1,5 is generally appropriate.

h_p is the length of the base plate

b_p is the width of the base plate

d_f is the depth of the concrete foundation

h_f is the length of the concrete foundation

b_f is the width of the concrete foundation

t_f is the flange thickness of the column

e_b is the additional width outside of the base plate
$= (b_f - b - 2t_f)/2$

e_h is the additional depth outside of the base plate
$= (h_f - h - 2t_f)/2$

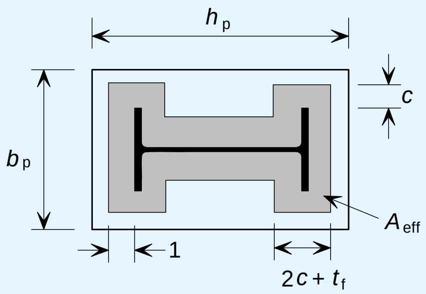
$$f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_c}$$ [EN 1992-1-1, §3.1.6(1)]

α_{cc} is a coefficient that takes into account long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied. [13]

γ_c is the material factor for concrete from EN 1992-1-1, §2.4.2.4[13]

Concrete class	C20/25	C25/30	C30/37	C35/45
Cylinder strength, fck (N/mm2)	20	25	30	35
Cube strength, fck,cube (N/mm2)	25	30	37	45

6.2 Calculation of *c*



Projection, c
t_f = flange thickness
t_w = web thickness

Basic requirement: $A_{req} = A_{eff}$

- If $2c \leq h - 2t_f$, then there is no overlap.

Thus c may be calculated from the following equations for I and H sections:

$$A_{eff} \approx 4c^2 + Per_{col}c + A_{col}$$

where:

A_{col} is the cross sectional area of the column

Per_{col} is the column perimeter

- If $2c > h - 2t_f$, then there is an overlap.

Thus c may be calculated from the following equations for I and H sections:

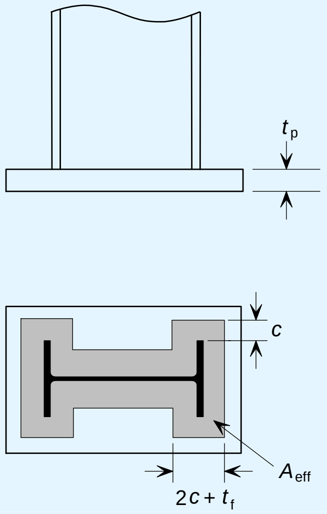
$$A_{eff} \approx 4c^2 + 2(h + b)c + h \times b$$

To ensure that the effective area fits on the base plate:

$h + 2c < h_p$

$b + 2c < b_p$

6.3 Base plate thickness



Basic requirement: $t_p \geq t_{p,min}$

$$t_{p,min} = c\sqrt{\frac{3 f_{jd} \gamma_{M0}}{f_{yp}}}$$ [Reference 3]

where:

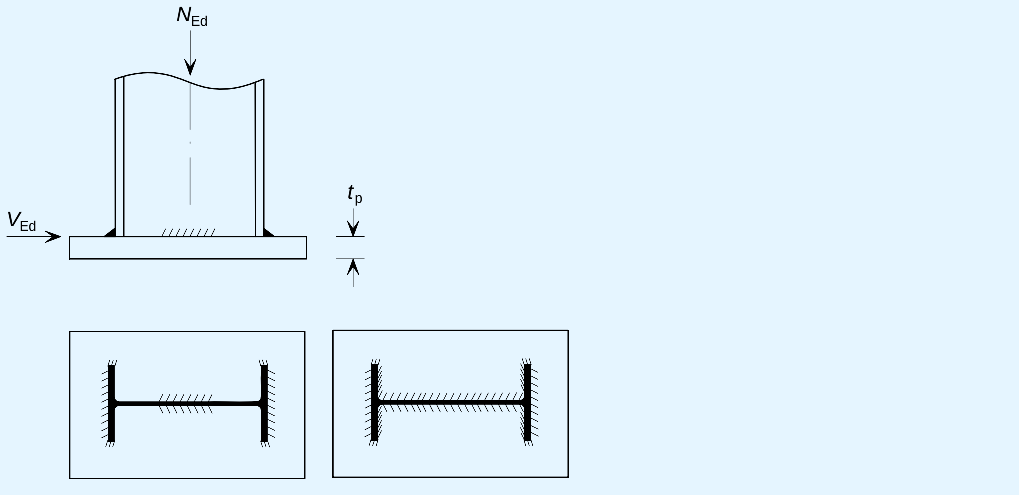
f_{yp} is the yield strength of the base plate

$f_{jd} = \frac{2}{3} \alpha f_{cd}$

$f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_c}$

α, α_{cc}, γ_c, f_{ck}, and c are as defined previously.

6.4 Base plate welds



Basic requirement:

For shear: $V_{\mathrm{Ed}} \leq F_{\mathrm{w,Rd}} \times \ell_{\mathrm{weld,shear}}$ [Reference 4]

For axial load:

This check is only necessary when the contact faces of the column and base plate are not in tight bearing. See Reference [4] for more details.

$F_{\mathrm{Ed}} \quad \leq F_{\mathrm{w,Rd}} \times \ell_{\mathrm{weld,axial}}$

where:

$F_{\mathrm{w,Rd}}$ is the resistance of the fillet weld per unit length = $f_{\mathrm{vw,d}} \times a$

$$f_{\mathrm{vw,d}} = \frac{f_{\mathrm{u}} / \sqrt{3}}{\beta_{\mathrm{w}} \gamma_{\mathrm{M2}}}$$ [EN 1993-1-8 §4.5.3.3(3)]

f_{u} is ultimate tensile strength of the weaker part joined

β_{w} = 0,8 for S235 steel

= 0,85 for S275 steel

= 0,9 for S355 steel

= 1,0 for S460 steel

a is the weld throat

$\ell_{\mathrm{weld,shear}}$ is total effective length of the welds in the direction of shear

$\ell_{\mathrm{weld,shear}} = 2(l - 2s)$ (for IPE, HE, HD sections)

l is the weld length in the direction of shear

$\ell_{\mathrm{weld,axial}}$ is the total effective length of the welds to the column flange for rolled sections

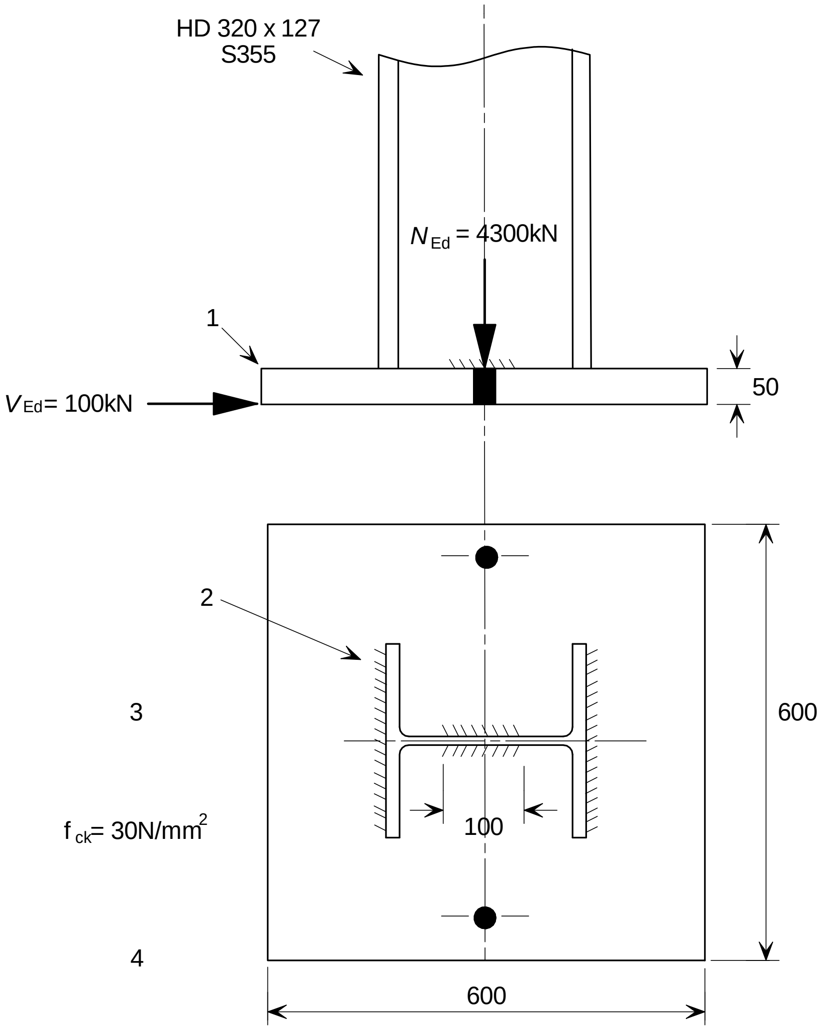
γ_{M2} is the partial factor for welds from EN 1993-1-8

The leg length is defined as follows: $s = a\sqrt{2}$

6. Column base

Details and data

Unless noted otherwise, all references are to EN 1993-1-8



1 600 × 600 × 50 Base plate S275
2 8 mm fillet welds
3 M24 grade 4.6 holding down bolts
4 Contact faces of the column and the base plate are in direct bearing

6.1. Base plate size

Basic requirement: $A_p \geq A_{req}$

Ref [3]

Area of base plate: $A_p = h_p \times b_p = 600 \times 600 = 360000\ \text{mm}^2$

Design strength of the concrete: $f_{cd} = \alpha_{cc} \dfrac{f_{ck}}{\gamma_c} = 1{,}0 \times \dfrac{30}{1{,}5} = 20\ \text{N/mm}^2$

f_{cd} from EN 1992-1-1 §3.1.6(1)

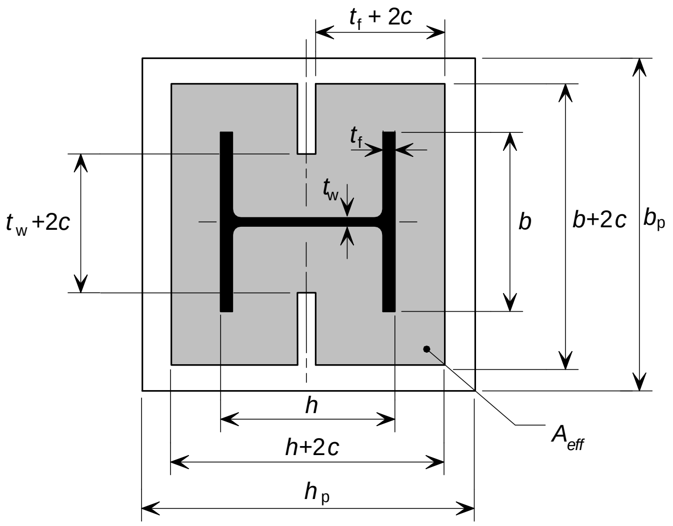
Area required: $A_{req} = \dfrac{N_{Ed}}{f_{jd}} = \dfrac{4300 \times 10^3}{\frac{2}{3} \times 1{,}5 \times 20} = 215000\ \text{mm}^2$

α_{cc} from EN 1992-1-1 §3.1.6(1)

γ_C from EN 1992-1-1 Table 2.1N

$A_p = 360000\ \text{mm}^2 > 215000\ \text{mm}^2$ OK

6.2. Calculation of *c*



Basic requirement: $A_{eff} = A_{req}$

To calculate the effective area, assume first that there is no overlap.

$A_{eff} \approx 4c^2 + Per_{col}c + A_{col}$

Column perimeter $Per_{col} = 1771$ mm

Area of column $A_{col} = 16130\ \text{mm}^2$

$A_{eff} \approx 4c^2 + 1771c + 16130 = 215000 = A_{req}$

$\therefore c = 93$ mm

To ensure that there is no overlap, *c* has to be less than half the depth between flanges:

$$\frac{h - 2t_f}{2} = \frac{320 - 2 \times 20{,}5}{2} = 139{,}5\ \text{mm} > 93\ \text{mm}$$

Therefore the assumption that there is no overlap is correct.

To check that the effective area fits on the base plate:

$h + 2c \;=\; 320 + 2\times 93 \;=\; 506 \text{ mm} \;<\; 600 \text{ mm}$

$b + 2c \;=\; 300 + 2\times 93 \;=\; 486 \text{ mm} \;<\; 600 \text{ mm}$

Therefore the calculated value of c is valid (otherwise recalculate c).

6.3. Base plate thickness

Ref (3)

$$t_{\mathrm{p,min}} = c\sqrt{\frac{3\, f_{\mathrm{jd}}\,\gamma_{\mathrm{M0}}}{f_{\mathrm{y,p}}}}$$

$$f_{\mathrm{jd}} = \frac{2}{3}\alpha\, f_{\mathrm{cd}} = \frac{2}{3}\times 1{,}5\times 20 = 20 \text{ N/mm}^2$$

Yield strength of the 50 mm plate, $f_{\mathrm{y,p}} = 255 \text{ N/mm}^2$

$$t_{\mathrm{p,min}} = 93\sqrt{\frac{3\times 20\times 1{,}0}{255}} = 45 \text{ mm}$$

$t_{\mathrm{p}} = 50 \text{ mm} > 45 \text{ mm}$ OK

6.4. Base plate welds (Shear resistance of column-to-base weld)

Ref [4]

Basic requirement: $V_{\mathrm{Ed}} \le F_{\mathrm{w,Rd}} \times l_{\mathrm{eff,shear}}$

Ultimate tensile strength of the 50 mm plate, $f_{\mathrm{u,p}} = 410 \text{ N/mm}^2$

$F_{\mathrm{w,Rd}}$ from § 4.5.3.3(3)

$$F_{\mathrm{w,Rd}} = f_{\mathrm{vw,d}}\times a = \frac{f_{\mathrm{u}}/\sqrt{3}}{\beta_{\mathrm{w}}\gamma_{\mathrm{M2}}}\times a = \frac{410/\sqrt{3}}{0{,}85\times 1{,}25}\times 0{,}7\times 8 = 1248 \text{ N/mm}$$

$$\ell_{\mathrm{eff,shear}} = 2\,(l - 2s) = 2\left(100 - 2\times 8\right) = 168 \text{ mm}$$

$$F_{\mathrm{w,Rd}}\times\ell_{\mathrm{eff,shear}} = 1248\times 168\times 10^{-3} = 210 \text{ kN}$$

$V_{\mathrm{Ed}} = 100 \text{ kN} \le 210 \text{ kN}$ OK

APPENDIX A Lateral torsional buckling strength

Lateral torsional buckling strength taken from BS 5950-1 Table 17[10]

Lateral torsional buckling strength (N/mm^2)

λ_{LT}	Steel grade									
	S275					S355				
	235	**245**	**255**	**265**	**275**	**315**	**325**	**335**	**345**	**355**
25	235	245	255	265	275	315	325	335	345	355
30	235	245	255	265	275	315	325	335	345	355
35	235	245	255	265	272	300	307	314	321	328
40	224	231	237	244	250	276	282	288	295	301
45	206	212	218	224	230	253	259	265	270	276
50	190	196	201	207	212	233	238	243	248	253
55	175	180	185	190	195	214	219	223	227	232
60	162	167	171	176	180	197	201	205	209	212
65	150	154	158	162	166	183	188	194	199	204
70	139	142	146	150	155	177	182	187	192	196
75	130	135	140	145	151	170	175	179	184	188
80	126	131	136	141	146	163	168	172	176	179
85	122	127	131	136	140	156	160	164	167	171
90	118	123	127	131	135	149	152	156	159	162
95	114	118	122	125	129	142	144	146	148	150
100	110	113	117	120	123	132	134	136	137	139
105	106	109	112	115	117	123	125	126	128	129
110	101	104	106	107	109	115	116	117	119	120
115	96	97	99	101	102	107	108	109	110	111
120	90	91	93	94	96	100	101	102	103	104
125	85	86	87	89	90	94	95	96	96	97
130	80	81	82	83	84	88	89	90	90	91
135	75	76	77	78	79	83	83	84	85	85
140	71	72	73	74	75	78	78	79	80	80
145	67	68	69	70	71	73	74	74	75	75
150	64	64	65	66	67	69	70	70	71	71
155	60	61	62	62	63	65	66	66	67	67
160	57	58	59	59	60	62	62	63	63	63
165	54	55	56	56	57	59	59	59	60	60
170	52	52	53	53	54	56	56	56	57	57
175	49	50	50	51	51	53	53	53	54	54
180	47	47	48	48	49	50	51	51	51	51
185	45	45	46	46	46	48	48	48	49	49
190	43	43	44	44	44	46	46	46	46	47
195	41	41	42	42	42	43	44	44	44	44
200	39	39	40	40	40	42	42	42	42	42
210	36	36	37	37	37	38	38	38	39	39
220	33	33	34	34	34	35	35	35	35	36
230	31	31	31	31	31	32	32	33	33	33
240	28	29	29	29	29	30	30	30	30	30
250	26	27	27	27	27	28	28	28	28	28

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12 EN 1090-2:2008: Execution of steel structures and aluminium structures. Technical requirements for the execution of steel structures

13 EN 1992-1-1:2004: Eurocode 2 Design of concrete structures. General rules and rules for buildings

Flow charts

Flow charts for End plate, Fin plate and Column bases are available on the Access Steel web site (http://www.access-steel.com)

The document references for these joint types are as follows:

Partial depth end plate	SF008a
Fin plate	SF009a
Column bases	SF010a