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EUROPEAN CONVENTION FOR CONSTRUCTIONAL STEELWORK
CONVENTION EUROPEENNE DE LA CONSTRUCTION METALLIQUE
EUROPAISCHE KONVENTION FÜR STAHLBAU

ECCS - Advisory Committee 5
Application of Eurocode 3

Examples to Eurocode 3

FIRST EDITION

1993

N°71

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Introduction

The European Convention for Structural Steelwork (ECCS) has as one of its primary objectives the promotion of the safe and economical use of steel in structures. ECCS believes that the introduction of the harmonized Eurocodes has a great value in achieving this objective; accordingly ECCS has set up an Advisory Committee, AC 5, charged with the task of promoting the introduction and adoption of the Eurocodes.

The AC 5 Committee have considered how this process could be best achieved and concluded that a three stage approach was desirable. The first stage was to produce a concise version of the Eurocodes which can be used for normal every day design; this part has already been issued as ECCS Publication No. 65.

The second stage is the production of this document which gives design examples to EC 3/1 and E-EC 3 and has been prepared as a design aid to facilitate the use of EC 3/1 for the design of steel buildings during the ENV period. These examples concentrate on those aspects which are likely to be needed for daily practical design work.

The third and final stage will be the production of a series of "Design Aids" which will enable the design process to be made more quickly by using tabulated or graphical values for the various design formula contained in EC 3/1.

The combination of these three documents will enable practising engineers to more easily adopt to the use of the new Eurocodes and should have a beneficial help in their speedy introduction.

Scope

These Design Examples to EC 3/1 and E-EC 3 have been prepared by the ECCS - Advisory Committee AC 5 as a design aid in supplement to the complete EC 3/1 to facilitate the use of EC 3/1 for the design of steel buildings in the ENV-period.

The Design Examples only contains examples to EC 3/1 and E-EC3 that are likely to be needed for daily practical design work.

The γ values used in this document are the values recommended in EC 3 main document. These values may deviate from the values recommended in the National Application Documents (NAD) of the member states.

The ECCS - Advisory Committee 5 is at present composed of the following members:

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Grotmann, D.	Germany
Taylor, J.C.	United Kingdom

Also particular thanks are given to the ECCS Technical Committees TC 8 and TC 10 who have contributed to the work.

References

(1) EC 3/1: ENV 1993-1-1 Eurocode 3: Part 1.1
(2) E-EC 3: Essentials of Eurocode 3 - Design Manual for Steel Structures in Buildings, ECCS-Publication No. 65
(3) References to EC 3/1 are given in brackets [...]
(4) References to E-EC 3 are given without brackets

Contents

Part 1

Load combination
Methods of analysis
Frame analysis
Bracing system analysis

1.1 Load combination

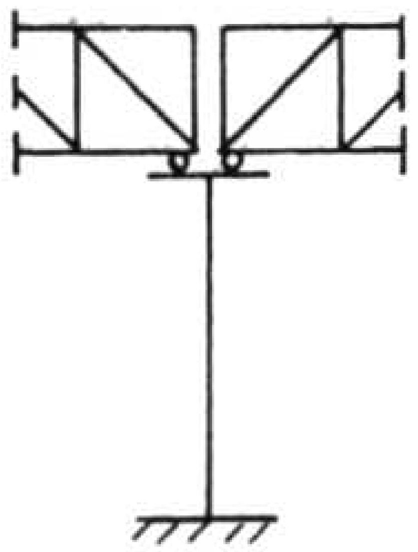
These examples demonstrate how the design values of action effects (N_{Sd}, V_{Sd}, M_{Sd}, etc) are determined from the load assumptions. The further steps of design are not treated in this part. An action is a force (load) applied to the structure or an imposed deformation (e.g. temperature effects or settlements). Characteristic (unfactored) values of these actions are specified in ENV 1991 Eurocode 1 or other relevant loadings codes. These values of actions shall be multiplied by relevant safety factors and combination factors, see chapter "Combinations of actions" in ENV 1993 Eurocode 3 or Table 2.1 in the Essentials of Eurocode 3 to determine the design values of the effects of actions. The following examples show the method of determination of the maximum effects of actions. Not all possible combinations of actions are presented nor are relevant combinations worked out. In practice, one will collect experience to easily find out which load combination is decisive for verification of the structures.

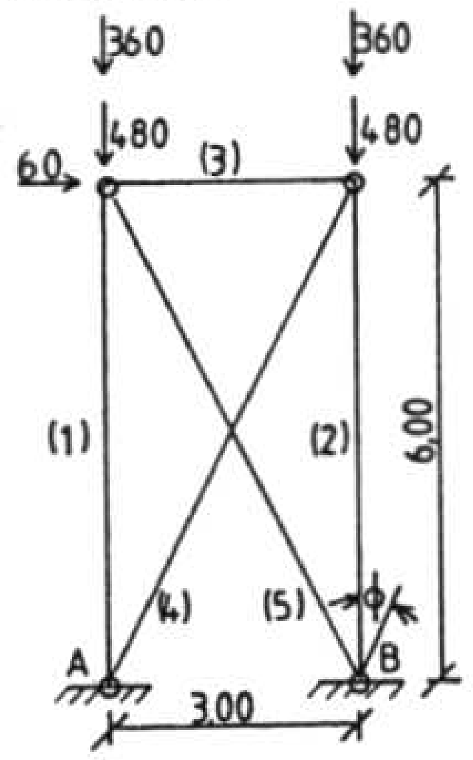
The following examples are included in this chapter:

Example 1.1.1: Braced portal frame
Example 1.1.2: Purlin treated as continuous beam
Example 1.1.3: Single storey frame without crane girder
Example 1.1.4: Crane girder
Example 1.1.5: Single storey frame with a crane girder

Problem, Sketch, Calculation	Reference	Commentary

Example 1.1.1: Braced portal frame





1. System and load configuration

Load assumptions:

G = 480 kN
P = 360 kN
W = 60 kN

Frame imperfections Φ = 1/200

Steel grade: Fe 360

Member (1) and (2): 2 I-profiles with welded flange plates, A = $104 \cdot 10^2$ mm^2

Member (3): IPE 400, A = $84{,}5 \cdot 10^2$ mm^2

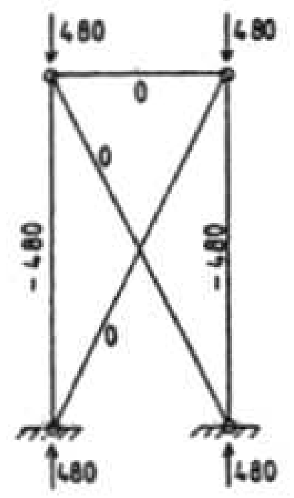
Member (4) and (5): IPE 200, A = $28{,}5 \cdot 10^2$ mm^2

Reference: Table 5.6

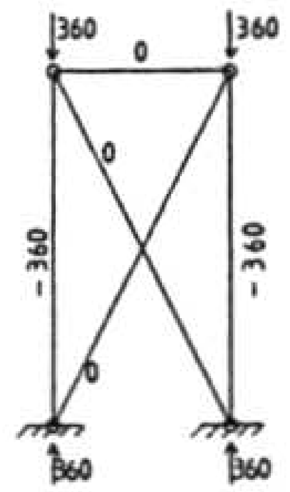
Commentary: Characteristic (unfactored) values of actions are specified in ENV 1991 Eurocode 1 or other relevant loading codes.
The effects of imperfections shall always be taken into account in both first and second order theory.
Here: Frame imperfections are considered

2. Distribution of internal forces

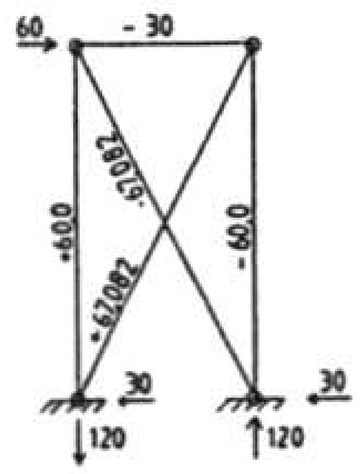
due to G



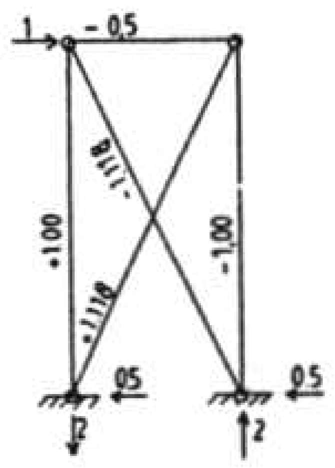
due to P



due to W



due to imperfections
equivalent horizontal unit force F_I = 1



Commentary: Internal forces and moments (effects of actions) are responses of the structure to the loading (actions). Design values of the internal forces and moments are determined from the design (factored) values of loading using elastic or plastic global analysis.
Here: Using elastic global analysis, first order theory with unfactored loading by separating in single actions. This simplifies the decision of which load combination gives the worst effect.

3. Relevant load combinations

3.1 Relevant load combination for max compression force in the column (permanent + imposed load (1,35G + 1,5P))

Imperfection: $F_I = 2 \cdot (1{,}35 \cdot 480 + 1{,}5 \cdot 360)/200 = 11{,}88$ kN

maximum axial force in column (2): $N_{Sd} = 1{,}35 \cdot 480 + 1{,}5 \cdot 360 + 11{,}88 \cdot 1{,}0 = 1199{,}9$ kN

Reference: Table 2.1 [2.3.3.1.(1) and (5)]

Commentary: The effect of loading is oriented in the same direction.

3.2 Relevant load combination for max compression/tension in the diagonals (permanent + imposed load + wind (1,35·(G+P+W))

Imperfection: $F_I = 2 \cdot (1{,}35 \cdot 480 + 1{,}35 \cdot 360)/200 = 11{,}34$ kN

maximum compression force in diagonal (5): $N_{Sd} = 1{,}35 \cdot 67{,}082 + 11{,}34 \cdot 1{,}118 = 103{,}2$ kN

maximum tension force in diagonal (4): $N_{Sd} = 1{,}35 \cdot 67{,}082 + 11{,}34 \cdot 1{,}118 = 103{,}2$ kN

maximum reaction force at (B): $N_{Sd} = 1{,}35 \cdot (480 + 360 + 120) + 11{,}34 \cdot 2{,}0 = 1318{,}7$ kN

Reference: Table 2.1 [2.3.3.1.(1) and (5)]

Commentary: The effect of loading is oriented in the same direction.
The maximum reaction force is needed for base plate design. For foundation design characteristic values are needed.

3.3 Relevant load combination for uplift (permanent - wind (1,0·G - 1,5·W))

uplift reaction force at support (A): $N_{Sd} = 480 - 1{,}5 \cdot 60{,}0 = 390{,}0$ kN

---> no uplift effect

Reference: Table 2.1 [2.3.3.1.(1) and (5)]

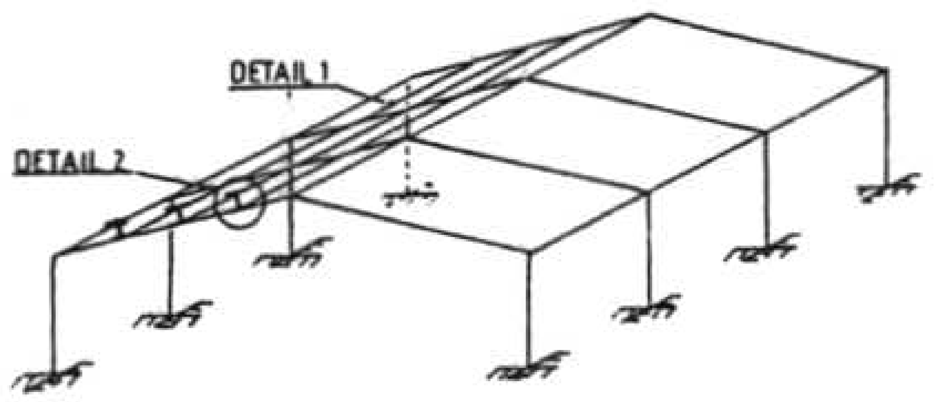
Commentary: The effect of dead load counteracts the effect of wind loading in column (1).

Problem, Sketch, Calculation	Reference	Commentary
3.4 **Relevant load combination for max compression force in the beam (permanent + wind load (1,35G + 1,5W))** Imperfection: $F_I = 2 \cdot (1{,}35 \cdot 480)/200 = 6{,}48$ kN maximum compression force in beam (3): $N_{Sd} = 1{,}5 \cdot 30{,}0 + 6{,}48 \cdot 0{,}5 = 48{,}2$ kN	Table 2.1 [2.3.3.1.(1) and (5)]	**The effect of loading is oriented in the same direction.**

Problem, Sketch, Calculation	Reference	Commentary

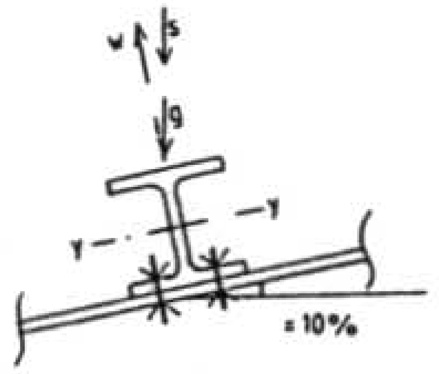
Example 1.1.2: Purlin treated as a continuous beam

1. System



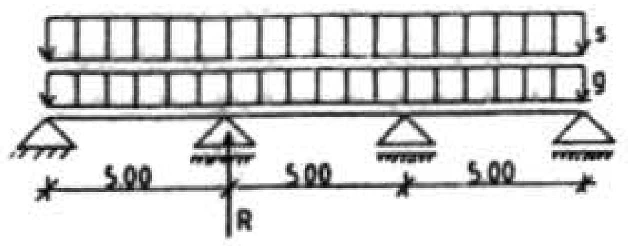
Detail 1: Purlin

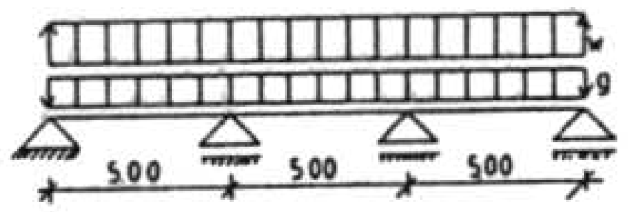
Detail 2: Fixing of purlin



Purlin (3-span-beam): IPE 140
Steel grade: Fe 430
Each span length: L = 5,00 m

2. Load configuration





permanent load: g_k = 1,0 kN/m
snow load: s_k = 1,5 kN/m
wind load (uplift): w_k = - 2,0 kN/m

Commentary: Purlins are laterally restrained by cladding ⟶ sufficient resistance to prevent minor axis buckling

3. Relevant load combinations for ultimate limit state verification of purlins

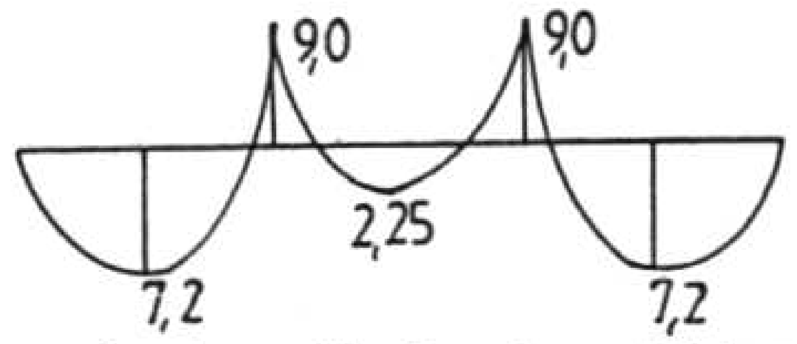
3.1 Relevant load combination for moment distribution (permanent + snow (1,35G + 1,5S))

q = 1,35 g + 1,5 s = 1,35•1,0 + 1,5•1,5 = 3,6 kN/m

Plastic global analysis: $W_{pl} \geq \dfrac{q\,\ell^2\,\gamma_{Mo}}{11{,}66\,f_y} = \dfrac{3{,}6\cdot 5{,}0^2\cdot 1{,}1\cdot 10^6}{11{,}66\cdot 275} = 30{,}9\cdot 10^3\ \text{mm}^3$

⟶ chosen: IPE 140 (W_{pl} = 88,3•10³ mm³ ; $M_{pl.Rd}$ = 22,1 kNm)

Moment distribution due to q = 3,6 kN/m



Reference: Table 2.1 [2.3.3.1.(1) and (5)]

Commentary: No imperfection effects
This load combination leads to the following effects:
- max M_{Sd} (strength, lateral torsional buckling)
- max V_{Sd} (shear)
- max R_{Sd} (reaction)

In view of erection an IPE 140 is chosen.

3.2 **Relevant load combination for uplift (permanent + wind (1,0G - 1,5W))**

q = 1,0 g + 1,5 w = 1,0•1,0 - 1,5•2,0 = -2,0 kN/m

Reference: Table 2.1 [2.3.3.1.(1) and (5)]

Commentary: This load combination leads to the following effects:
- M_{Sd} (lateral torsional buckling)
- R_{Sd} (reaction)

4. Relevant load combinations for serviceability limit state

4.1 Serviceability limit state requirement (example only)

Requirement: $\delta_2 \leq L/200$ (for snow only)

q = 1,0 s = 1,5 kN/m

δ = 18,0 mm = L/280 < L/200

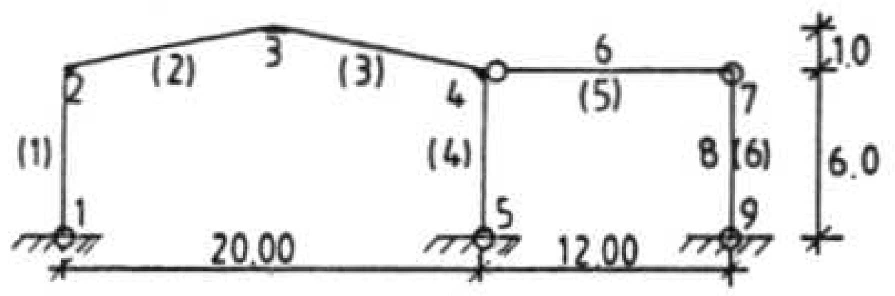
Reference: Table 2.3; 4.2.1.(1); Table 4.2; [2.3.4]

Commentary: For roofs in general less stringent values may be used [4.2.1.(2)].

Problem, Sketch, Calculation	Reference	Commentary

Example 1.1.3: Single storey frame

1. **System**



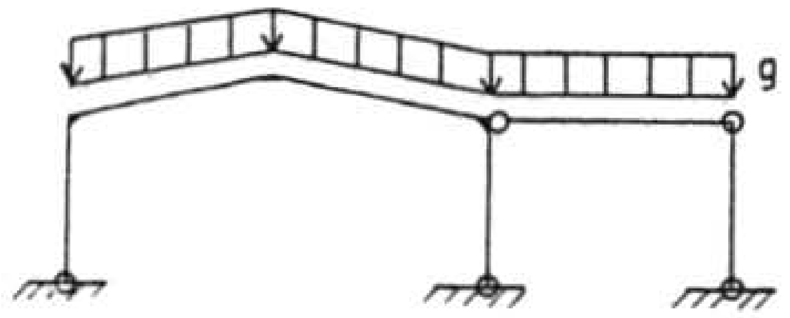
column (1) and (4):	IPE 450
beam (2) and (3):	IPE 400
column (6):	HE 180 A
beam (5):	IPE 400
Steel grade:	Fe 430

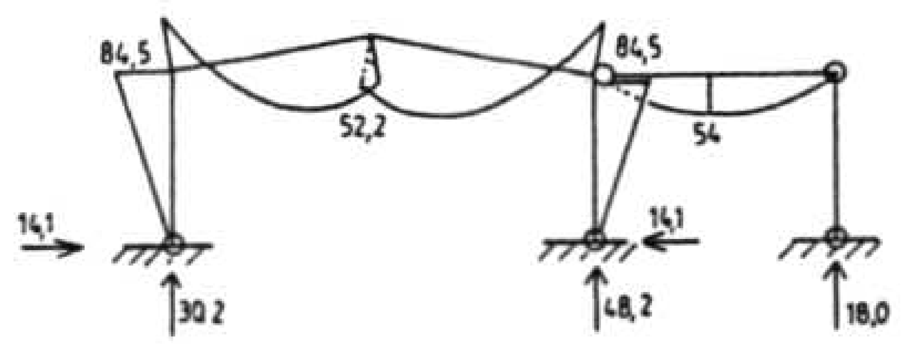
2. Load configuration and distribution of moments

permanent:

$g = 3{,}0$ kN/m

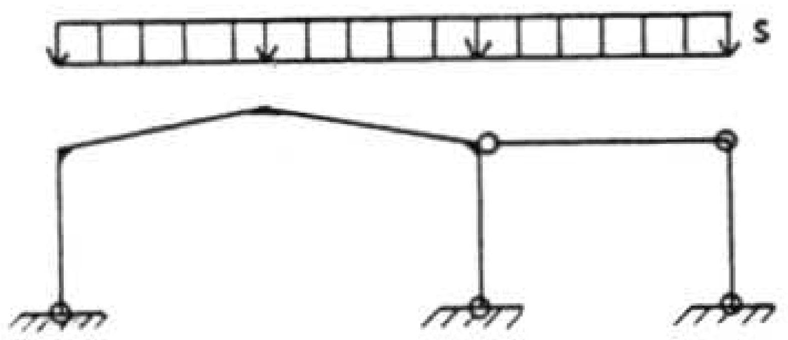
Using first order theory for determining the internal forces and moments with $\gamma_F = 1{,}0$ for finding out the relevant load combination

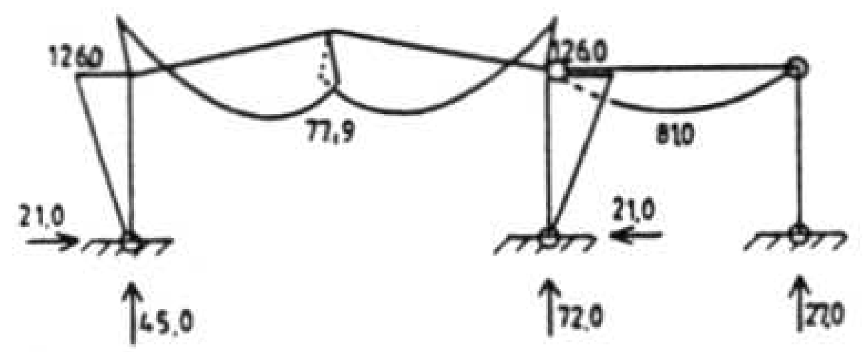




snow:

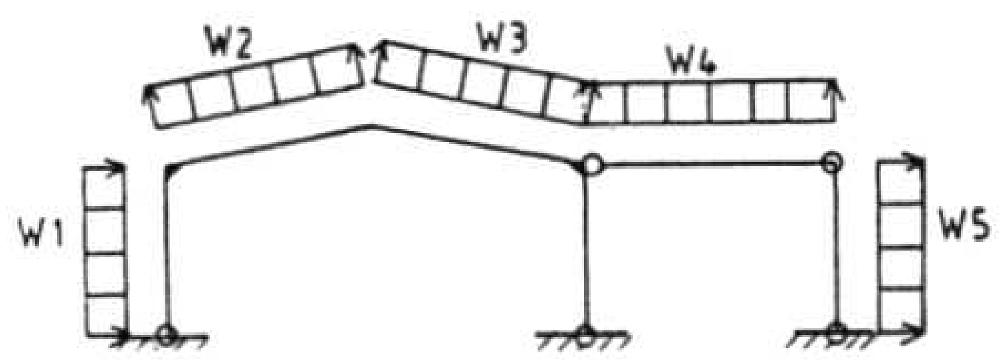
$s = 4{,}5$ kN/m

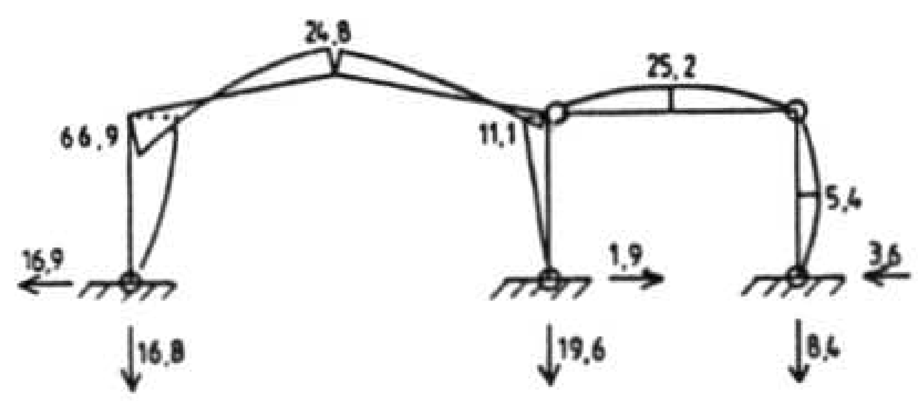


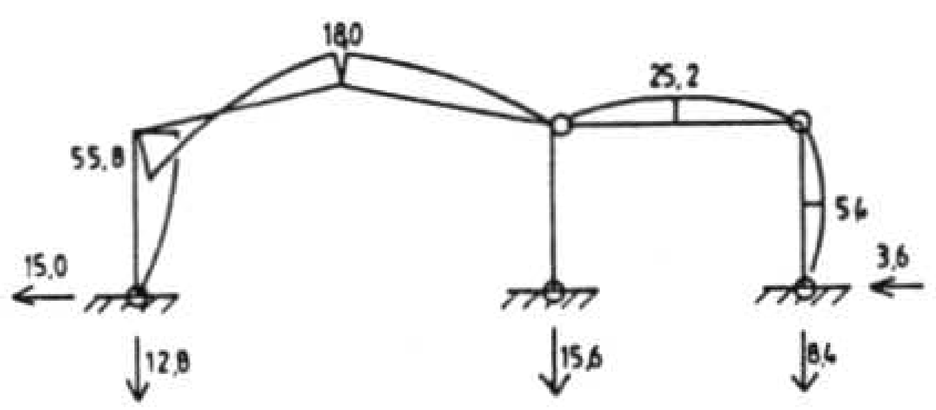


wind from left side

$w_1 = 1{,}9$ kN/m
$w_2 = 1{,}0$ kN/m or $1{,}4$ kN/m
(whichever gives a larger effect)
$w_3 = 1{,}0$ kN/m or $1{,}4$ kN/m
(whichever gives a larger effect)
$w_4 = 1{,}4$ kN/m
$w_5 = 1{,}2$ kN/m

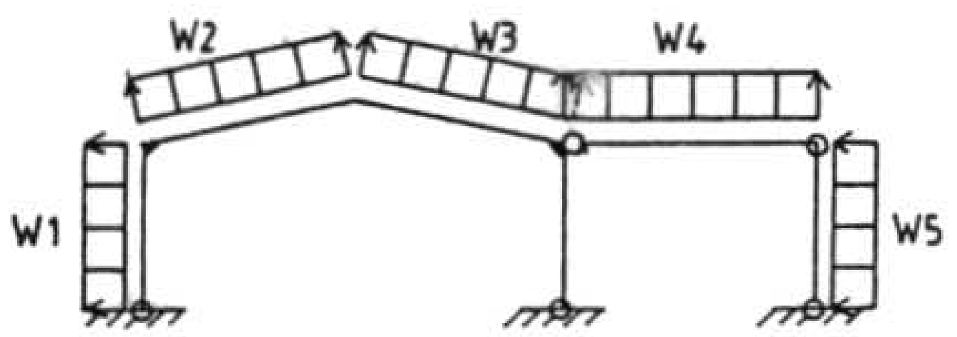


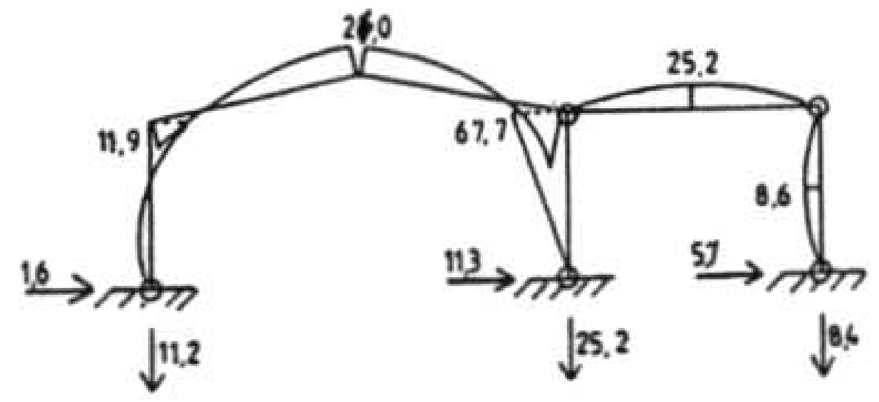


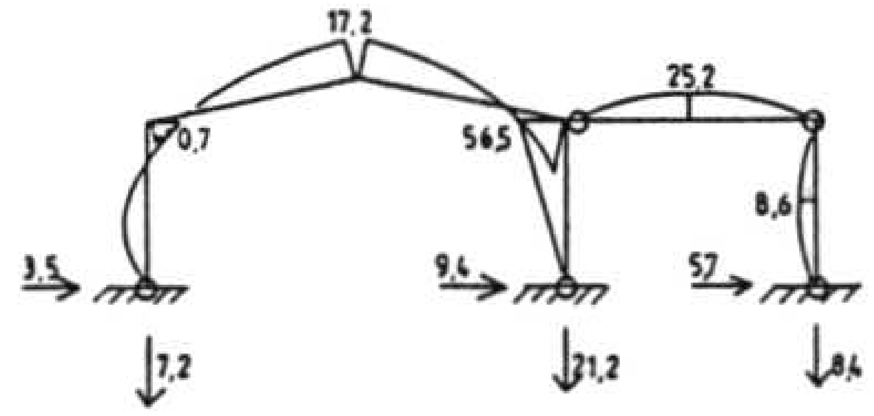


Problem, Sketch, Calculation	Reference	Commentary

wind from right side

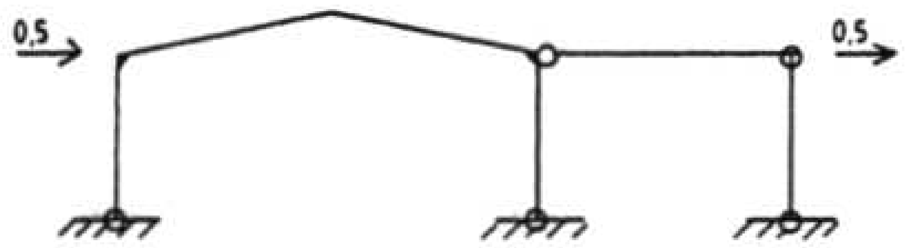


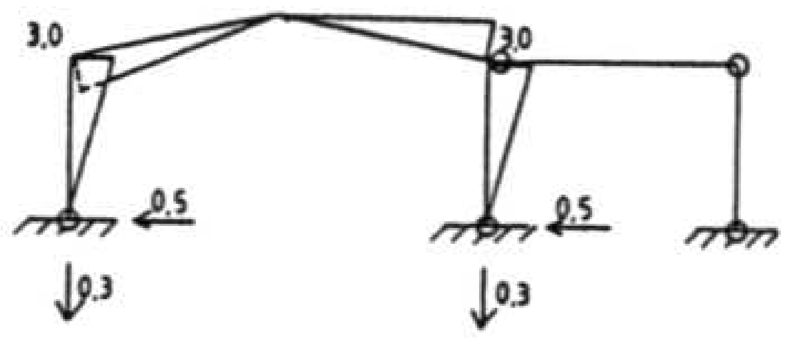




w_1 = 1,9 kN/m
w_2 = 1,0 kN/m or 1,4 kN/m
(whichever gives a larger effect)
w_3 = 1,0 kN/m or 1,4 kN/m
(whichever gives a larger effect)
w_4 = 1,4 kN/m
w_5 = 1,2 kN/m

Imperfections (equivalent horizontal unit force)





3. **Relevant load combinations for ultimate limit state verification**

3.1 **permanent + snow**

1,35 g + 1,5 s

Reference: Table 2.1; [2.3.3.1.(5)]

Commentary: This load combination leads to following effects:
- max N_{Sd} at column (4) and (6)
- max M_{Sd} at 2, 3, 4 and 6
- max $R_{v.Sd}$ at 5 and 9

$R_{v.Sd}$ is needed for base plate design. For foundation design characteristic values are needed.

3.1.1 **Determination of the initial sway imperfections**

Reference: 5.2.1.2; [5.2.1.2]

Column (3) does not contribute to the resistance

$\longrightarrow$ n_c = 2 ; n_s = 1

$\longrightarrow$ $\Phi = k_c k_s \varphi_o$ (see table 5.6 in E-EC3)

Φ = 1/200

Reference: 5.2.3.1.2; Table 5.6 [5.2.4.3]

Equivalent horizontal forces F_i

Reference: Table 5.5; [Figure 5.2.4]

due to g: ΣV_{Sd} = 1,35•[60,6 + 96,6] = 212,2 kN
due to s: ΣV_{Sd} = 1,50•[45,2 + 72,2] = 176,1 kN

F = 212,2 + 176,1 = 388,3 kN

$F_i = \Phi \cdot F$ = 388,3/200 = 1,9 kN

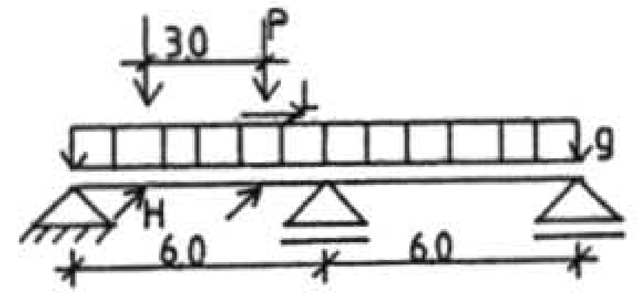
Problem, Sketch, Calculation	Reference	Commentary
3.1.2 Maximum effects due to the load combination permanent + snow max N at column (4): N_{Sd} = 1,35•48,2 + 1,5•72,0 + 0,3•1,9 = 173,6 kN max N at column (6): N_{Sd} = 1,35•18,0 + 1,5•27,0 = 173,6 kN max M at 2 and 4: M_{Sd} = 1,35•84,9 + 1,5•126,0 + 3,0•1,9 = 309,3 kNm max M at 3: M_{Sd} = 1,35•52,2 + 1,5•77,9 = 187,3 kNm max M at 6: M_{Sd} = 1,35•54,0 + 1,5•81,0 = 194,4 kNm		max $R_{v.Sd}$ at 5 and 9 are equal N_{Sd} at column (4) and (6).
3.2 permanent + snow + wind 1,35 (g + s + w) wind is considered from the right side with w_2 and w_3 = 1,0 kN/m	Table 2.1 [2.3.3.1.(5)]	This load combination leads to the following effect: - max $R_{h.Sd}$ (reaction horizontal). $R_{h.Sd}$ is needed for base plate design.
3.2.1 Determination of the initial sway imperfections Column (3) does not contribute to the resistance ---→ Φ = 1/200 (as demonstrated in 3.1.1)	5.2.1.2; [5.2.1.2] 5.2.3.1.2; Table 5.6 [5.2.4.3]	
Equivalent horizontal forces F_i due to g: ΣV_{Sd} = 1,35•(60,6 + 96,6) = 212,2 kN due to s: ΣV_{Sd} = 1,35•(45,2 + 72,2 = 158,5 kN due to w: ΣV_{Sd} = 1,35•(-11,2 - 25,2) = -49,1 kN F = 212,2 + 158,5 - 49,1 = 321,6 kN F_i = Φ•F = 321,6/200 = 1,6 kN	Table 5.5; [Figure 5.2.4]	
3.2.2 Maximum effects due to load combination permanent + snow + wind $R_{h.Sd}$ = 1,35•(14,1 + 21,0 + 3,5) + 0,5•1,6 = 52,9 kN		
3.3 Further commentary Not all possible load combinations are given in this example. It could be that for different elements different load combinations are relevant. E.g. for a beam-column two checks are necessary: (1) max M and associated N (2) max N and associated M Therefore the load combination "permanent + wind" could also be relevant.		

Problem, Sketch, Calculation	Reference	Commentary

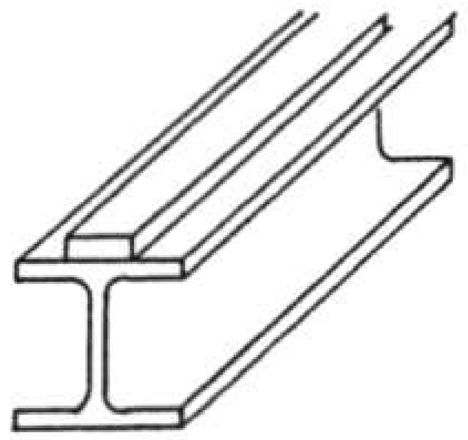
Example 1.1.4: Crane girder

No fatigue is included here

1. System and load configuration



Detail:



Crane girder (two span-beam)

wheel base = 3,00 m

2. Load assumptions

Lateral forces and wheel loads can act together
Braking forces are independent from lateral forces

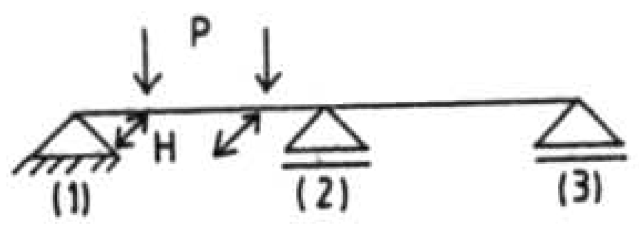
permanent: $g = 2{,}0$ kN/m
wheel load: $P = 130$ kN
wheel loads must be increased by a dynamic coefficient φ; $\varphi = 1{,}2$ (purely for calculation)
Lateral force: $H = 13$ kN (lateral forces due to crabbing, surging, etc)
Braking force: $L = 15$ kN

3. Relevant load combinations for ultimate limit state verification

3.1 Load position at mid-span see sketch

No imperfection effects
For the determination of the internal forces and moments influence lines are used

Relevant load combination for max mid-span moment and max reaction force at support (3)
$(1{,}35g + 1{,}5(P + H))$



g: $1{,}35 \cdot 0{,}0703 \cdot 2{,}0 \cdot 6{,}0^2 = 6{,}8$ kNm
P: $1{,}50 \cdot 0{,}23 \cdot 1{,}2 \cdot 130 \cdot 6{,}0 = 322{,}9$ kNm
$\longrightarrow M_{y.Sd} = 6{,}8 + 322{,}9 = 329{,}7$ kNm
H: $M_{z.Sd} = 1{,}50 \cdot 0{,}23 \cdot 13 \cdot 6{,}0 = 26{,}9$ kNm

Relevant load combination for maximum moment at the inner support
$(1{,}35g + 1{,}5(P + H))$
g: $- 1{,}35 \cdot 0{,}125 \cdot 2{,}0 \cdot 6{,}0^2 = - 12{,}2$ kNm
P: $- 1{,}50 \cdot 0{,}14 \cdot 1{,}2 \cdot 130 \cdot 6{,}0 = - 196{,}6$ kNm
$\longrightarrow M_{y.Sd} = - 12{,}2 - 196{,}6 = - 208{,}8$ kNm
H: $M_{z.Sd} = -1{,}50 \cdot 0{,}14 \cdot 13 \cdot 6{,}0 = 16{,}4$ kNm

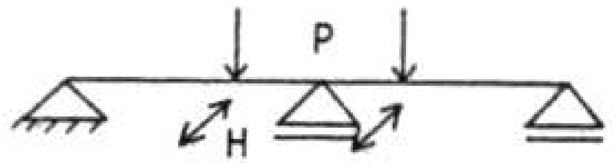
Relevant load combination for maximum uplift at the end support
$(1{,}0g + 1{,}5P)$
g: $1{,}0 \cdot 0{,}375 \cdot 2{,}0 \cdot 6{,}0 = 4{,}5$ kN
P: $- 1{,}50 \cdot 0{,}14 \cdot 1{,}2 \cdot 130 = - 32{,}8$ kN
$\longrightarrow R_{z.Sd} = 4{,}5 - 32{,}8 = - 28{,}3$ kN

Problem, Sketch, Calculation	Reference	Commentary

3.2 **Load position at inner support see sketch**

Relevant load combination for max reaction force at support (2)

(1,35g + 1,5(P + H))



g: 1,35•1,25•2,0•6,0 = 20,3 kN

P: 1,50•1,82•1,2•130 = 425,9 kN

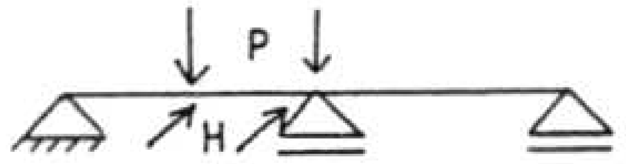
⟶ $R_{z.Sd}$ = 20,3 + 425,9 = 446,2 kN

H: $R_{y.Sd}$ = 1,50•1,82•13 = 35,5 kN

3.3 **Load position at inner support see sketch**

Relevant load combination for max shear force at support (2)

(1,35g + 1,5(P + H))



g: 1,35•0,625•2,0•6,0 = 10,1 kN

P: 1,50•1,60•1,2•130 = 374,4 kN

⟶ $V_{z.Sd}$ = 10,1 + 374,4 = 384,5 kN

H: $V_{y.Sd}$ = 1,50•1,6•13 = 31,2 kN

3.4 **Relevant load combination for determining the horizontal forces which must be applied at the bracing system**

parallel to crane girder

1,35(g + P + H + L)

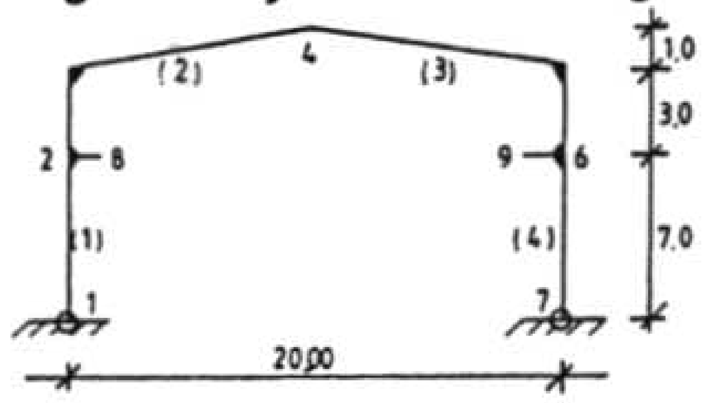
perpendicular to crane girder

1,35g + 1,5 H

Problem, Sketch, Calculation	Reference	Commentary

Example 1.1.5: Single storey frame including a crane girder

1. System



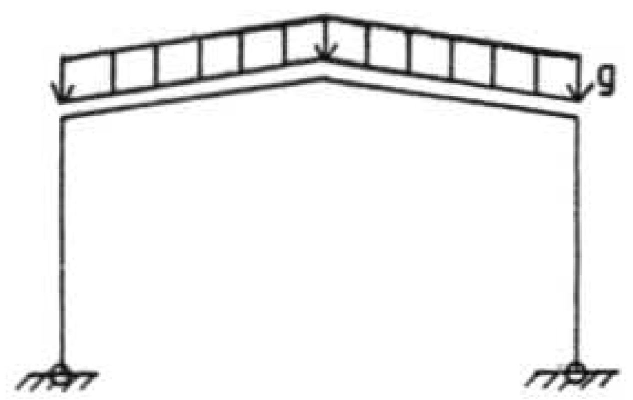
column (1) and (4): HE 400 A
beam (2) and (3): IPE 600
Steel grade: Fe 430

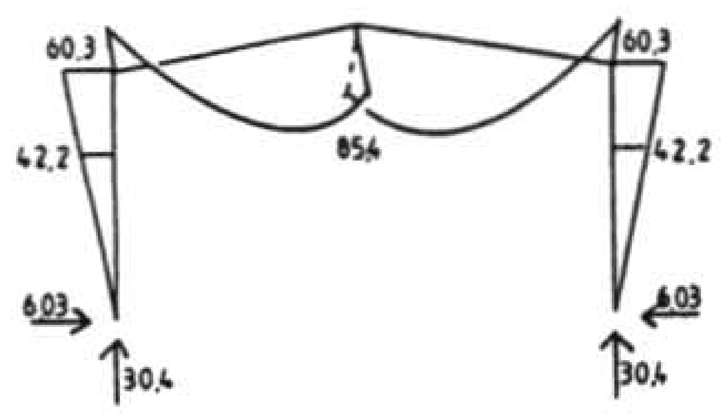
2. Load configuration and distribution of moments

Using first order theory for determining the internal forces and moments with γ_F = 1,0 for finding out the relevant load combination for each element.

permanent:

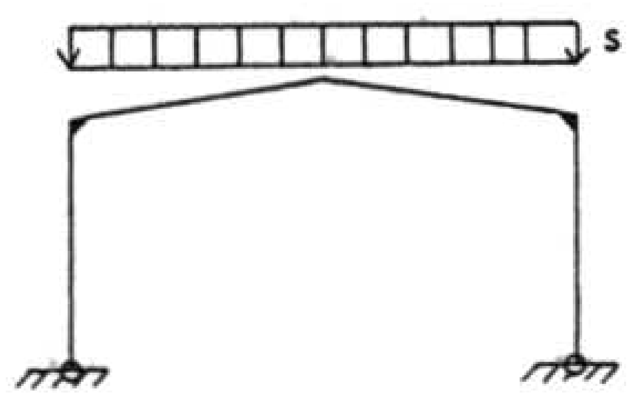
g = 3,0 kN/m

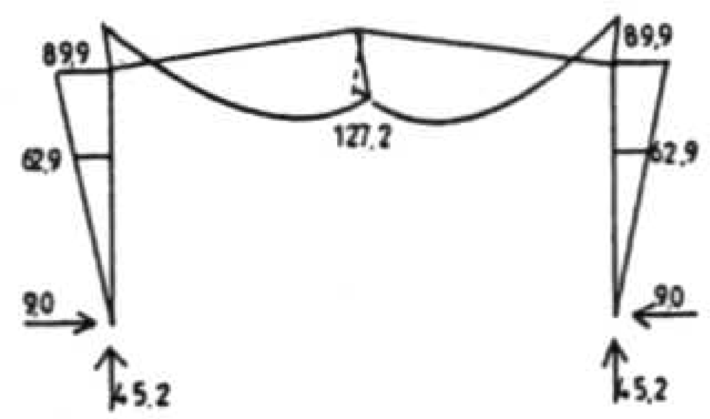




snow:

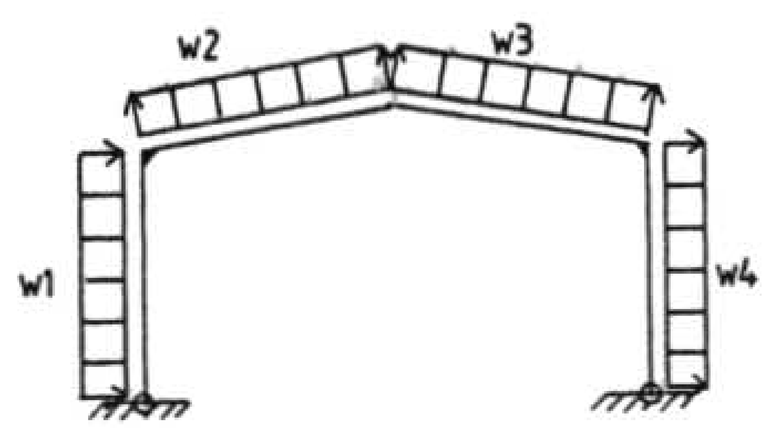
s = 4,5 kN/m

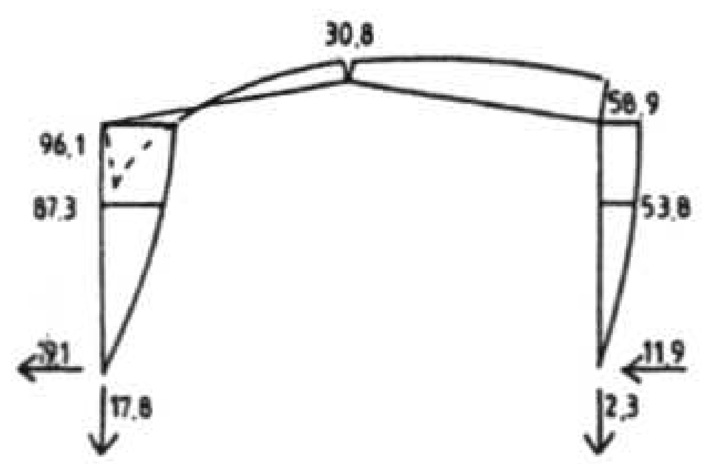




wind from left side

w_1 = 1,9 kN/m
w_2 = 1,0 kN/m
w_3 = 1,0 kN/m
w_4 = 1,2 kN/m

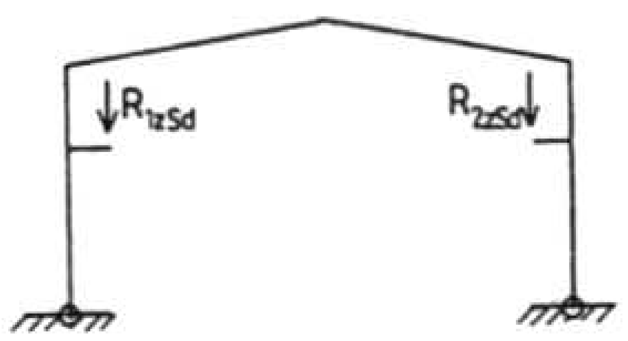


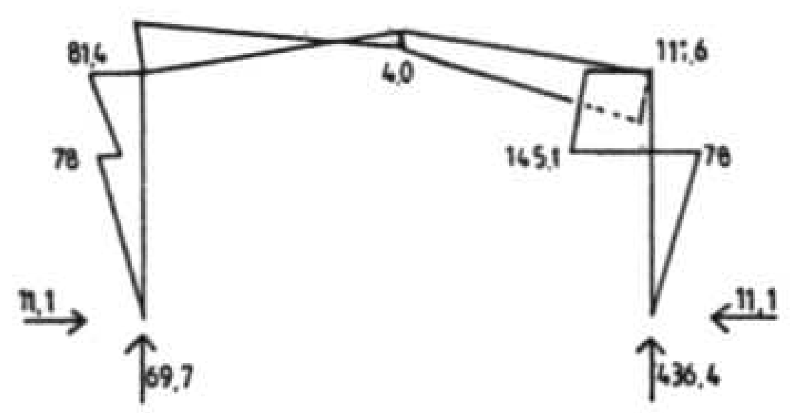


Support load from crane girder (vertical)

$R_{1z.Sd}$ = 60 kN
$R_{2z.Sd}$ = 446 kN

Loads of crane girder are already multiplied with partial safety factors.

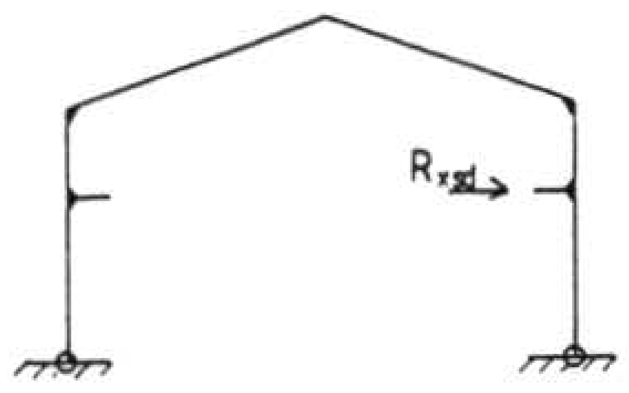


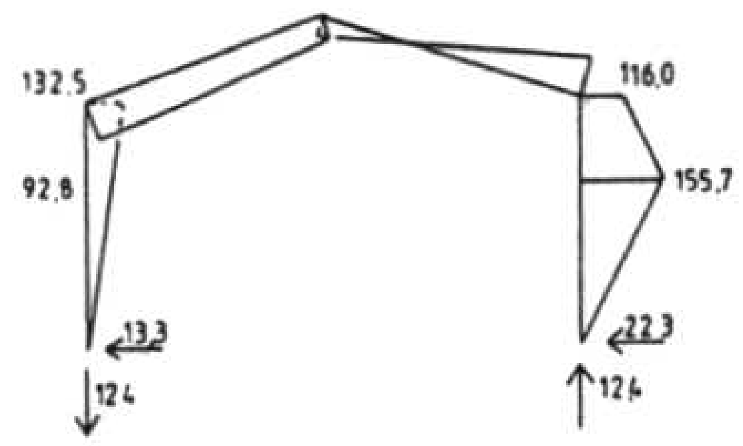


Problem, Sketch, Calculation	Reference	Commentary

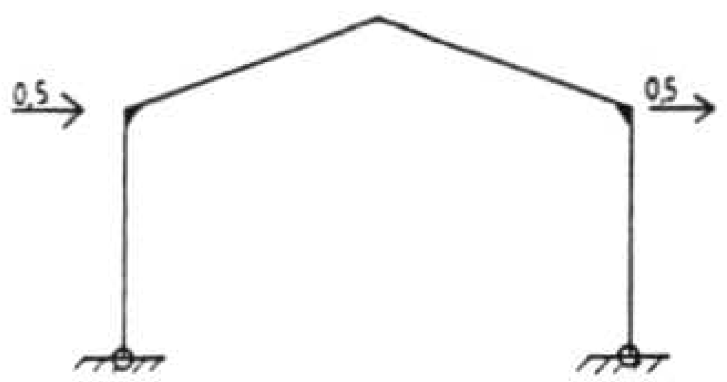
Support load from crane girder (horizontal)

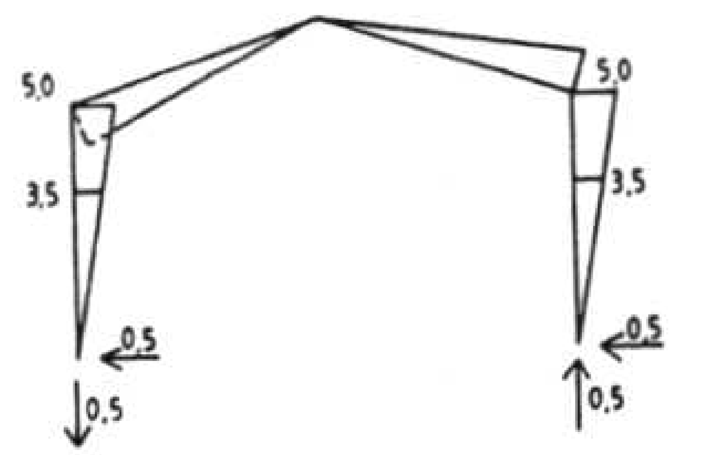
$R_{2x.Sd}$ = 35,5 kN





Imperfections (equivalent horizontal unit force)





3. **Relevant load combinations for ultimate limit state verification**

3.1 **permanent + snow including crane loads**

1,35 (g + s) + $R_{z.Sd}$ + $R_{x.Sd}$

Table 2.1; [2.3.3.1.(5)]

This load combination leads to the following effects:
- **max N_{Sd} at column (4)**
- **max M_{Sd} at 3, 4 and 5**
- **max $R_{v.Sd}$ at 7**

3.1.1 **Determination of the initial sway imperfections**

⟶ n_c = 2 ; n_s = 1

⟶ $\Phi = k_c k_s \varphi_o$ (see table 5.6 in E-EC3)

Φ = 1/200

5.2.1.2; [5.2.1.2]

For the determination of the initial sway imperfections only self-weight and snow is considered.

5.2.3.1.2; Table 5.6 [5.2.4.3]

Equivalent horizontal forces F_I

due to g: ΣV_{Sd} = 1,35•30,4•2 = 82,1 kN

due to s: ΣV_{Sd} = 1,50•45,2•2 = 135,6 kN

F = 82,1 + 135,6 = 217,7 kN

$F_I = \Phi \cdot F$ = 217,7/200 = 1,1 kN

Table 5.5; [Figure 5.2.4]

For the columns it must be checked whether member imperfections may be neglected [5.2.4.2.(4)] with the criterion:

$$\bar{\lambda} > 0{,}5 \sqrt{\frac{A\,f_y}{N_{Sd}}}$$

If this criterion is fulfilled member imperfections shall be considered.

	Problem, Sketch, Calculation	Reference	Commentary
3.1.2	**Effects of load combination permanent and snow** max N_{Sd} in column: N_{Sd} = 1,35•(30,4 + 45,2) + 436,4 + 12,4 + 1,1•0,5 = 551,4 kN max M_{Sd} at 4 M_{Sd} = 1,35•(85,4 + 127,2) + 4,0 + 8,3 = 299,3 kNm max M_{Sd} at 3 and 5 M_{Sd} = 1,35•(60,3 + 89,9) + 81,4 + 116,0 + 1,1•5,0 = 405,7		$R_{V.Sd}$ is equal to N_{Sd} in column.
3.2	**permanent + wind** 1,35 (g + w) + $R_{z.Sd}$ + $R_{x.Sd}$	Table 2.1 [2.3.3.1.(5)]	This load combination leads to the following effect: - max M_{Sd} at 2 and 6
3.2.1	**Determination of the initial sway imperfections** ⟶ $n_c = 2$; $n_s = 1$ ⟶ $\Phi = k_c k_s \varphi_o$ (see table 5.6 in E-EC3) $\Phi = 1/200$	5.2.1.2; [5.2.1.2] 5.2.3.1.2; Table 5.6 [5.2.4.3]	
	Equivalent horizontal forces F_i due to g: ΣV_{Sd} = 1,35•30,4•2 = 82,1 kN $F_i = \Phi \cdot \Sigma V_{Sd}$ = 82,1/200 = 0,4 kN	Table 5.5; [Figure 5.2.4]	
3.2.2	**Effects of load combination permanent and wind** max M_{Sd} at 2: M_{Sd} = 1,35•(42,2 + 53,8) + 78,0 + 155,7 + 0,4•3,5 = 364,7 kNm max M_{Sd} at 6: M_{Sd} = 1,35•(42,2 - 87,3) - 145,1 - 92,8 - 0,4•3,5 = - 300,2 kNm		
3.3	**Further comentary** Not all possible load combinations are given in this example. It could be that for different elements different load combinations are relevant, e.g. "permanent and snow without crane loads" (1,35 g + 1,5 s).		

1.2 Methods of analysis

These examples demonstrate on continuous beams how the design values of action effects are determined using either plastic global analysis or elastic global analysis and plastic or elastic stress distribution. All of the methods of analysis presented may also be applied on frames.
The further steps of design are not treated in this part.

Note: For plastic global analysis special requirements specified in [5.2.7, 5.3.3 and 3.2.2.2] shall be satisfied.

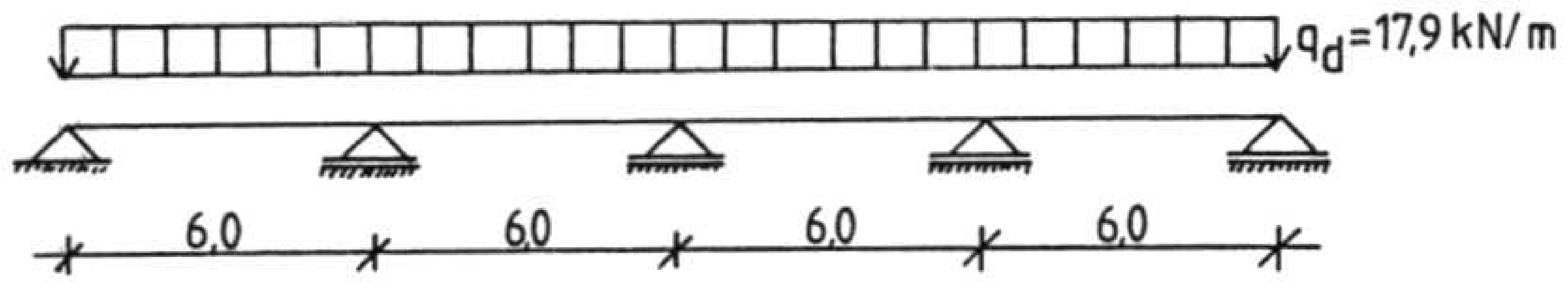
The following examples are included in this chapter:

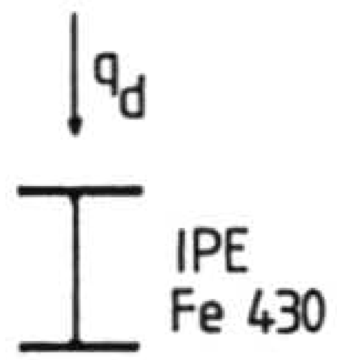
Example 1.2.1:	Continuous beam (plastic - plastic)
Example 1.2.2:	Continuous beam (elastic - plastic) with limited redistribution
Example 1.2.3:	Continuous beam (elastic - elastic) without redistribution
Example 1.2.4:	Continuous beam (elastic - elastic)

Problem, Sketch, Calculation	Reference	Commentary

Example 1.2.1: Continuous beam (plastic - plastic)

1. System





Commentary: The beam is laterally restrained to prevent lateral torsional buckling. Further, lateral restraints have to be provided at all plastic hinge locations, see [5.2.1.4 and 5.3.3]. Fe 430 satisfies the requirements specified in [3.2.2.2]

2. Determination of internal forces and moments

2.1 Plastic global analysis

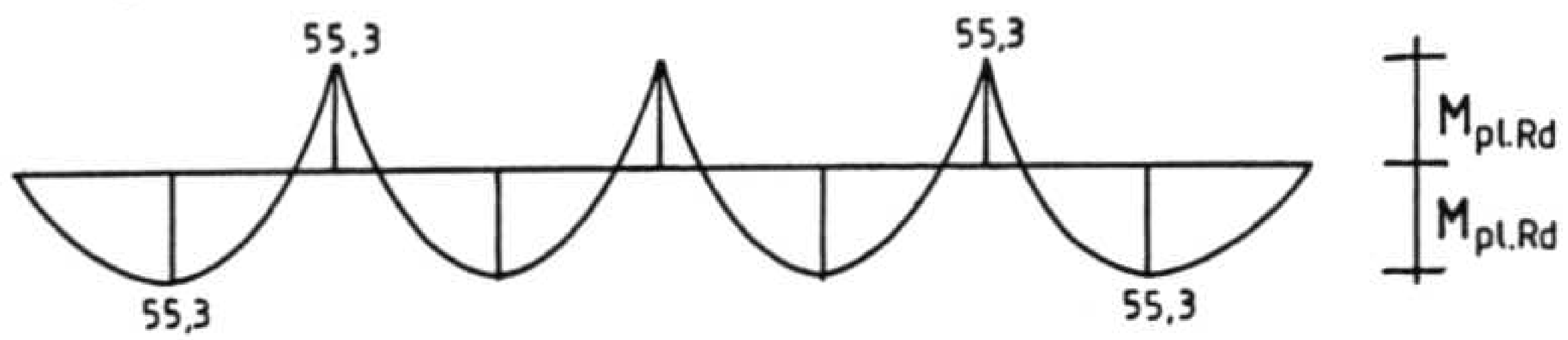
Commentary: The plastic hinges occur at the first inner support and in the outer span at 2,568 m from the outer support.

$$W_{pl} \geq \frac{q_d \cdot \ell^2 \cdot \gamma_{Mo}}{11{,}666 \cdot f_y} = \frac{17{,}9 \cdot 6{,}0^2 \cdot 1{,}1 \cdot 10^6}{11{,}666 \cdot 275} = 220{,}9 \cdot 10^3 \text{ mm}^3$$

$\longrightarrow$ chosen: IPE 200 (W_{pl} = 221•10^3 mm^3)

2.2 Plastic bending moment distribution for IPE 200

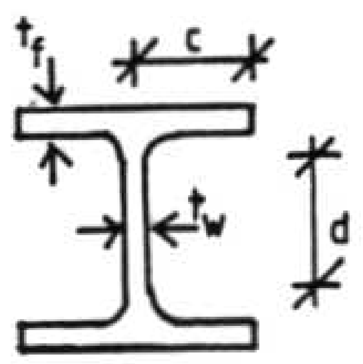
max q_d = 17,91 kN/m



Commentary:

$M_{pl.Rd}$ = f_y•W_{pl}/γ_{Mo} = 275•221•10^3•10^{-6}/1,1
= 55,3 kNm

2.3 Width-to-thickness ratios



Fe 430 $\longrightarrow$ ϵ = 0,92

Flange:

c/t = 50/8,5 = 5,9 < 10ϵ = 9,2 $\longrightarrow$ class 1

Web:

d/t = 159/5,6 = 28,4 < 72ϵ = 66,2 $\longrightarrow$ class 1

$\longrightarrow$ A_{eff} = A ; W_{eff} = W_{pl}

Reference: [5.3.3(4); Table 5.3.1]

Commentary: Plastic global analysis may be used only where the cross-sections satisfy the requirements specified in [5.3.3].

d/t = 28,4 < 69ϵ = 63,5

$\longrightarrow$ no shear buckling verification is required

Reference: 5.3.3; [5.4.6]

Problem, Sketch, Calculation	Reference	Commentary

Example 1.2.2: Continuous beam (elastic - plastic) with limited redistribution of moments

1. System

see example 1

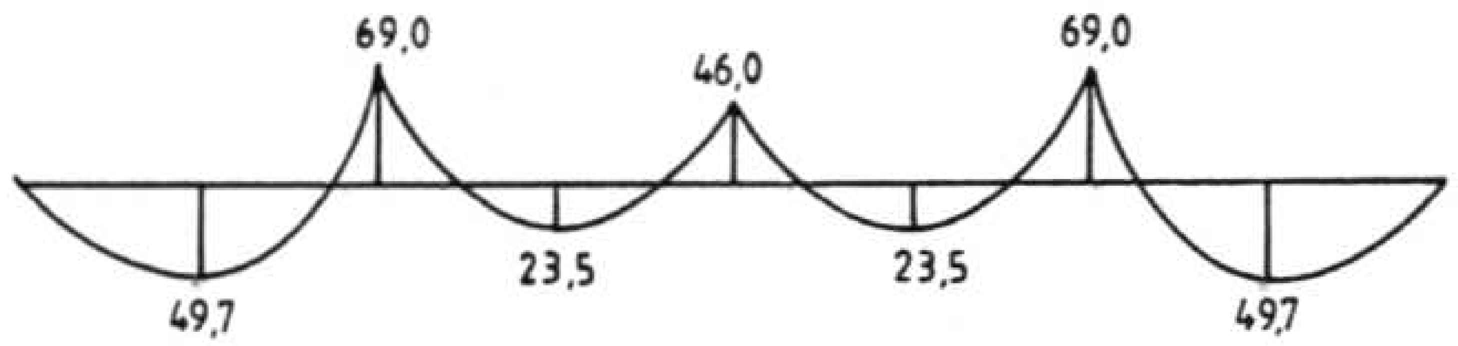
2. Determination of internal forces and moments

2.1 Elastic global analysis

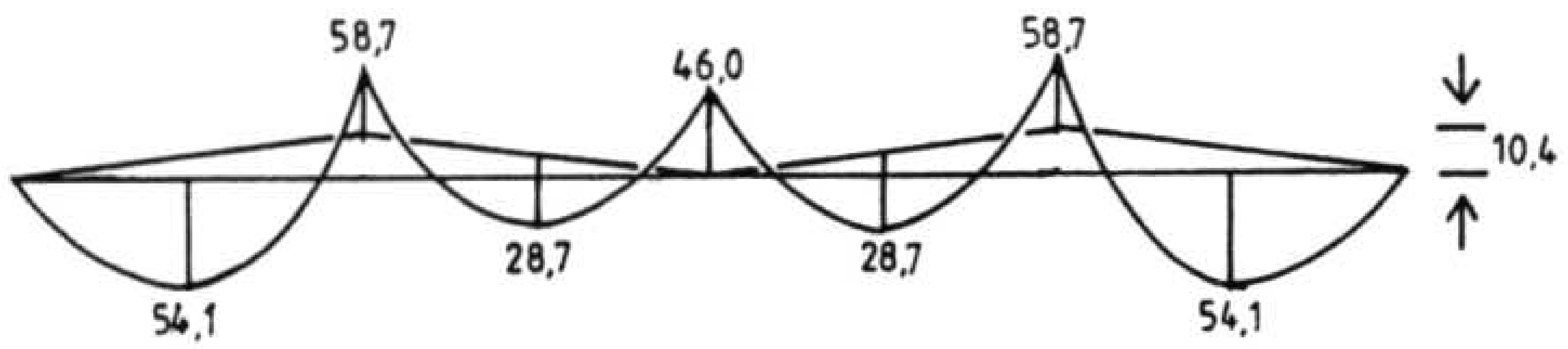
2.1.1 Moment before redistribution

[5.2.1.3.(3)]

The moments calculated using elastic global analysis are modified by redistribution of the peak calculated moment in any member (maximum value equal 15%) and the members have Class 1 or Class 2 cross-secions.



2.1.2 Moment after redistribution



$0{,}15 \cdot 69{,}0 = 10{,}4$ kNm

$49{,}7 + 0{,}42 \cdot 10{,}4 = 54{,}1 \text{ kNm} < 58{,}7 \text{ kNm}$

Approximative estimation of maximum redistribution:

$69 \cdot (1 - x) = 49{,}7 \cdot (1 + x)$

$\longrightarrow x_{max} \leq 16{,}3\%$

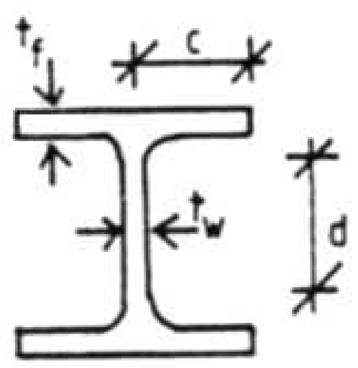
$\longrightarrow$ redistribution of 15% is allowed

$$W_{pl} \geq \frac{M_{Sd}\,\gamma_{Mo}}{f_y} = \frac{58{,}65 \cdot 1{,}1 \cdot 10^6}{275} = 234{,}6 \cdot 10^3 \text{ mm}^3$$

$\longrightarrow$ chosen: IPE 220 ($W_{pl} = 285 \cdot 10^3$ mm^3)

2.2 Width-to-thickness ratios

Table 5.10; [5.3; Table 5.3.1]



Fe 430 $\longrightarrow \epsilon = 0{,}92$

Flange:

$c/t = 55/9{,}2 = 6{,}0 < 10\epsilon = 9{,}2$

Web:

$d/t = 177/5{,}9 = 30{,}0 < 72\epsilon = 66{,}2$

$\longrightarrow A_{eff} = A\,;\; W_{eff} = W_{pl}$

5.3.3; [5.4.6]

$d/t = 30{,}0 < 69\epsilon = 63{,}5$

$\longrightarrow$ no shear buckling verification is required

Problem, Sketch, Calculation	Reference	Commentary

Example 1.2.3: Continuous beam (elastic - plastic) without redistribution of moments

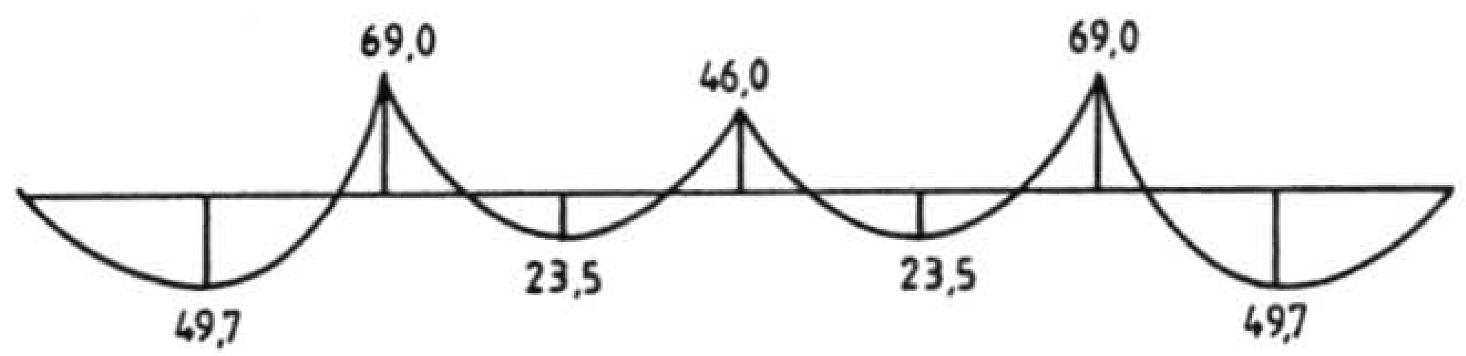
1. System

see example 1

2. Determination of internal forces and moments

2.1 Elastic global analysis

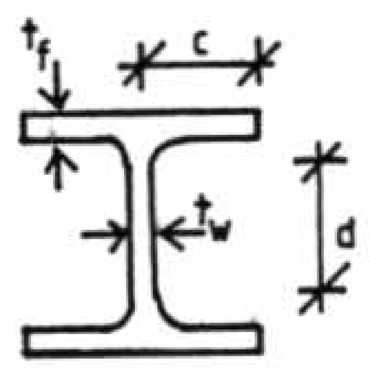
Moment distribution:



$$W_{pl} \geq \frac{M_{Sd}\,\gamma_{Mo}}{f_y} = \frac{69{,}0 \cdot 1{,}1 \cdot 10^6}{275} = 276{,}0 \cdot 10^3 \text{ mm}^3$$

$\longrightarrow$ chosen: IPE 220 ($W_{pl} = 285 \cdot 10^3$ mm^3)

2.2 Width-to-thickness ratios



Fe 430 $\longrightarrow$ $\epsilon = 0{,}92$ — Reference: Table 5.10; [5.3; Table 5.3.1]

Flange:

$c/t = 55/9{,}2 = 6{,}0 < 10\epsilon = 9{,}2$

Web:

$d/t = 177/5{,}9 = 30 < 72\epsilon = 66{,}2$

$\longrightarrow A_{eff} = A$; $W_{eff} = W_{pl}$

$d/t = 30 < 69\epsilon = 63{,}5$ — Reference: 5.3.3; [5.4.6]

$\longrightarrow$ no shear buckling verification is required

Problem, Sketch, Calculation	Reference	Commentary

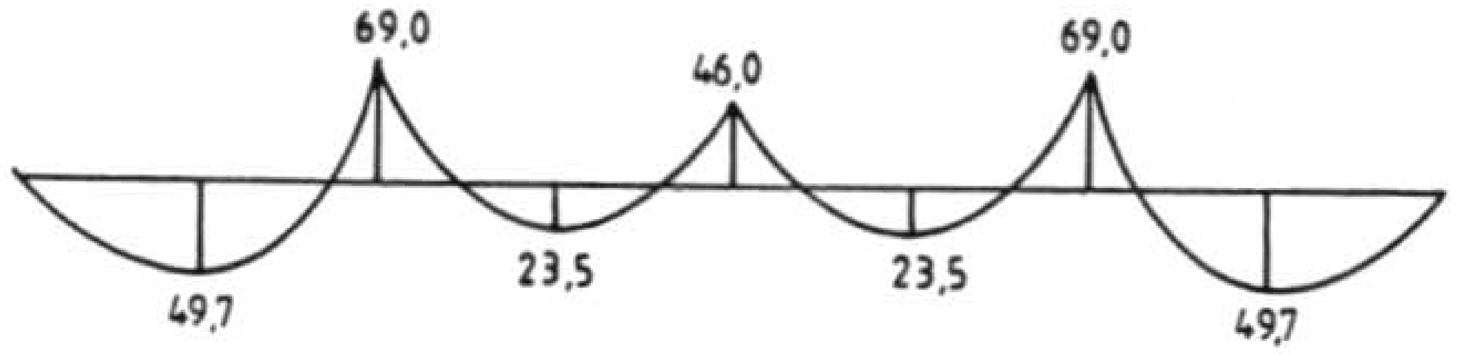
Example 1.2.4: Continuous beam (elastic - elastic)

1. System

see example 1

2. Determination of internal forces and moments

2.1 Elastic global analysis

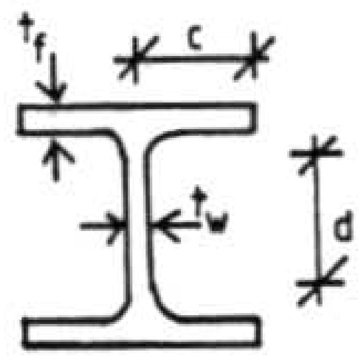


$$W_{el} \geq \frac{M_{Sd}\ \gamma_{Mo}}{f_y} = \frac{69{,}0 \cdot 1{,}1 \cdot 10^6}{275} = 276{,}0 \cdot 10^3\ mm^3$$

$\longrightarrow$ chosen: IPE 240 ($W_{el} = 324 \cdot 10^3\ mm^3$)

2.2 Width-to-thickness ratios

Reference: Table 5.11; [5.3; Table 5.3.1]



Fe 430 $\longrightarrow \epsilon = 0{,}92$

Flange:

$c/t = 60{,}0/9{,}8 = 6{,}1 < 10\epsilon = 9{,}2$

Web:

$d/t = 190/6{,}2 = 30{,}6 < 72\epsilon = 66{,}2$

$\longrightarrow A_{eff} = A\ ;\ W_{eff} = W_{pl}$

$d/t = 30{,}6 < 69\epsilon = 63{,}5$ (Reference: 5.3.3; [5.4.6])

$\longrightarrow$ no shear buckling verification is required

1.3 Frame analysis

The first example demonstrates the application of the criterion "sway - non sway" and how the second order effects in the sway mode are included using first order analysis with amplified sway moments. The second example demonstrate how frame imperfections are determined.
The further steps of design are not treated in this part.

The following examples are included in this chapter:

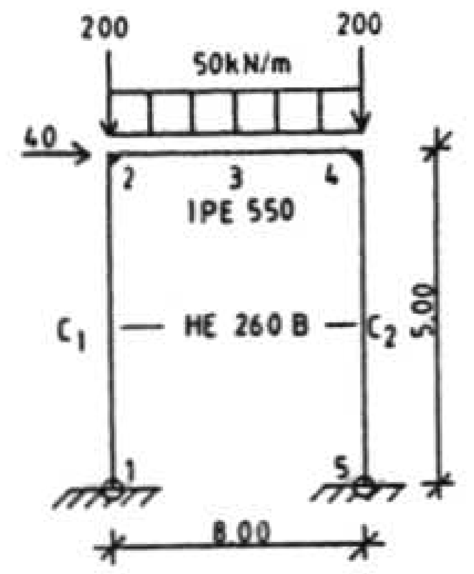
Example 1.3.1: Calculation of a sway frame
Example 1.3.2: Determination of frame imperfections

Problem, Sketch, Calculation	Reference	Commentary

Example 1.3.1: Calculation of a sway frame

1. System and loading

The partial safety factor for actions is included in the loading.



2. Frame imperfections

$F = 2 \cdot 200 + 50 \cdot 8 = 800$ kN

$\Phi = 1/200$

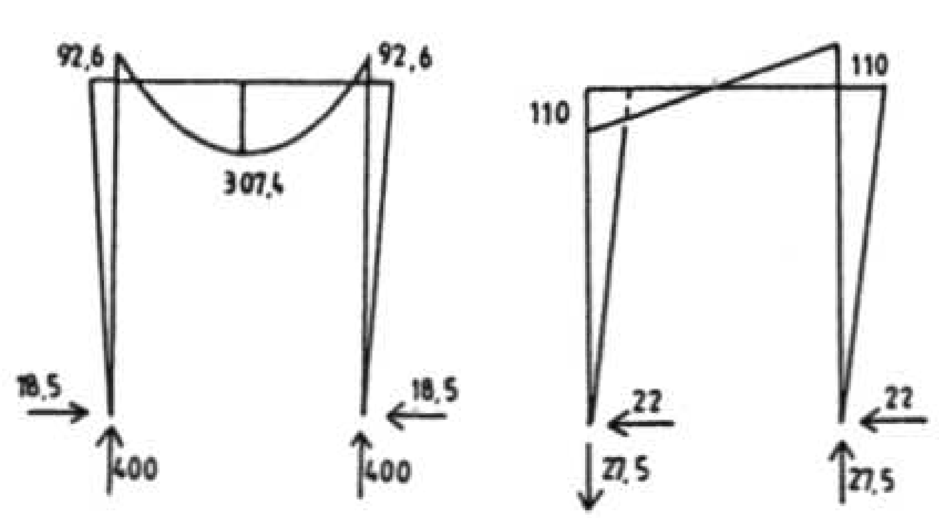
$\longrightarrow F_i = 800/200 = 4{,}0$ kN

Reference: 5.2.3.1.2; Table 5.6 [5.2.4.3]

The internal forces and moments may generally be determined in the global analysis taking into account the effects of imperfections

3. Internal forces and moments due to 1st order

symmetrical loading | antimetrical loading



Maximum axial compression in column (C2):	$N_{Sd} = 427{,}5$ kN
Maximum shear force at point (4) in the beam:	$V_{Sd} = 227{,}5$ kN
Moment distribution:	see sketch
Horizontal deflection of the storey:	$\delta = 34{,}5$ mm
Buckling length for non-sway mode of column:	$\ell = 3{,}75$ m

constant for frame analysis:

$$k = \frac{I_b}{I_c}\,\frac{h_c}{\ell_b} = \frac{67120 \cdot 10^4}{14920 \cdot 10^4}\,\frac{5{,}0}{8{,}0} = 2{,}812$$

In the calculation of the deflection the effects of imperfections are taken into account.

4. Criterion sway - non-sway

$\Sigma V = 800$ kN

$\Sigma H = 44$ kN (including the equivalent imperfection load)

$$\frac{\delta}{h}\,\frac{\sum V}{\sum H} = \frac{34{,}5 \cdot 10^{-3}}{5{,}0}\,\frac{800}{44} = 0{,}125 > 0{,}1$$

$\longrightarrow$ the frame is classified as sway

Reference: Table 5.4 [5.2.5.2.(4)]

This criterion gives an indication when second order theory should be used for structures as frames, bracing systems, etc.

Alternatively the $\Delta M/M$ - method (mentioned in 5.2.5.2(3)) may be used (Normally 2 iteration steps are needed).

$$\frac{\delta}{h}\,\frac{\sum V}{\sum H} = \frac{\Delta M}{M}$$

(Requirement: After each iteration step $\Delta M/M$ = const.)

$M^{II} = M_o + M_1/(1 - M_2/M_1)$

M_o edge moment due to symmetrical loading

M_1 edge moment due to antimetrical loading

M_2 edge moment after first iteration

$M_2 = 2{,}5 \cdot 800 \cdot 34{,}5/5000 = 13{,}8$ kNm

here: $M_2/M_1 = 13{,}8/110 = 0{,}125$

Problem, Sketch, Calculation	Reference	Commentary

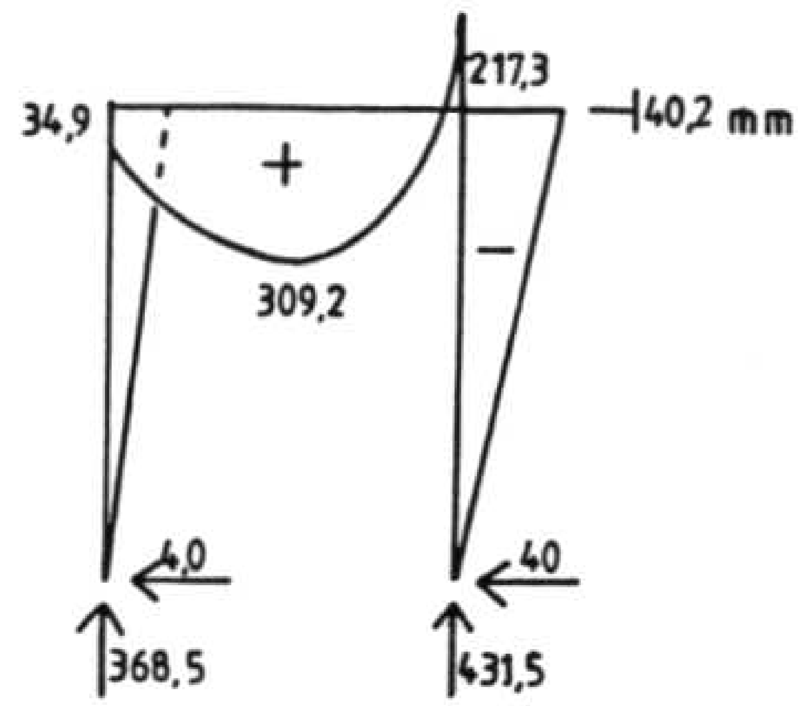
5. Verification using 1st order theory with amplified sway moments
5.1 Internal forces and moments

5.2.3.1.3; [5.2.6.2]

$$\alpha = \frac{1}{1 - \frac{\delta \sum V}{h \sum H}} = \frac{1}{1 - 0{,}125} = 1{,}14$$

$M''_{4.Sd} = 92{,}6 + 1{,}14 \cdot 110 = 218$ kNm
$N''_{c.Sd} = 400{,}0 + 1{,}14 \cdot 27{,}5 = 431{,}4$ kN
$V''_{4b.Sd} = 200{,}0 + 1{,}14 \cdot 27{,}5 = 231{,}4$ kN
$V''_{4c.Sd} = 18{,}5 + 1{,}14 \cdot 22 = 43{,}6$ kN

For comparison: Moment distribution (calculation by computer)



In the amplified sway moment method the sway internal forces and moments found by first order elastic analysis should be increased by multiplying them by the following factor:

$$\frac{1}{1 - \frac{\delta \sum V}{h \sum H}} = \frac{1}{1 - \frac{\Delta M}{M}}$$

5.2 Moment resistance

5.3.2; [5.4.5]

$$M_{pl.Rd} = W_{pl} \frac{f_y}{\gamma_{Mo}} = 1283 \cdot 10^3 \cdot \frac{235}{1{,}1} 10^{-6} = 274{,}1 \text{ kNm}$$

5.3 Shear resistance of the cross-section

5.3.3 [5.4.6]

Beam: $$V_{Rd} = \frac{A_v f_y}{\sqrt{3}\, \gamma_{Mo}} = \frac{61{,}0 \cdot 10^2 \cdot 235}{\sqrt{3} \cdot 1{,}1} 10^{-3} = 752{,}4 \text{ kN}$$

$0{,}5V_{Rd} = 376{,}2$ kN $> 200 + 1{,}14 \cdot 27{,}5 = 231{,}4$ kN
$\longrightarrow$ no interaction bending - shear is required

Column: $$V_{Rd} = \frac{A_v f_y}{\sqrt{3}\, \gamma_{Mo}} = \frac{26{,}0 \cdot 10^2 \cdot 235}{\sqrt{3} \cdot 1{,}1} 10^{-3} = 320{,}7 \text{ kN}$$

$0{,}5V_{Rd} = 160{,}3$ kN $> 18{,}5 + 1{,}14 \cdot 22{,}0 = 43{,}6$ kN
$\longrightarrow$ no interaction bending - shear is required

5.4 Axial compression

5.3.1 [5.4.4]

Column: $$N_{pl.Rd} = \frac{A \cdot f_y}{\gamma_{Mo}} = \frac{118 \cdot 10^2 \cdot 235}{1{,}1} 10^{-3} = 2520{,}9 \text{ kN}$$

$0{,}25N_{Rd} = 630{,}2$ kN
$dtf_y/2\gamma_{Mo} = 177 \cdot 10 \cdot 235/(2 \cdot 1{,}1) \cdot 10^{-3} = 189{,}1$ kN $< 400 + 1{,}14 \cdot 27{,}5 = 431{,}4$ kN
$\longrightarrow$ interaction bending - axial compression is required

Problem, Sketch, Calculation	Reference	Commentary

5.5 Interaction bending - axial compression

Reference: 5.3.4; [5.4.9]

$$M_{N.Rd} = 1{,}11 \cdot \left(1 - \frac{N_{Sd}}{N_{pl.Rd}}\right) \cdot M_{pl.Rd} = 1{,}11 \cdot \left(1 - \frac{427{,}5}{2520{,}9}\right) \cdot 274{,}1 = 252{,}7 \text{ kNm} > 92{,}6 + 110 \cdot 1{,}14 = 218{,}0 \text{ kNm}$$

5.6 Member resistance of the column

Reference: 5.4.2; [5.5.1; 5.2.6.2.(2)]

Commentary: When second order elastic global analysis is used, in-plane buckling lengths for the non-sway mode may be used for member design.

Buckling length of column: ℓ = 3,98 m

From the moment distribution $\longrightarrow$ β_M = 1,8

$$\bar{\lambda} = \frac{\ell}{i}\,\frac{1}{\lambda_1} = \frac{3{,}98 \cdot 10^3}{112} \cdot \frac{1}{93{,}9} = 0{,}38$$

using buckling curve b $\longrightarrow$ χ_b = 0,9339

$$N_{b.Rd} = \chi_b \cdot \frac{A \cdot f_y}{\gamma_{M1}} = 0{,}9339 \cdot \frac{118 \cdot 10^2 \cdot 235}{1{,}1} \, 10^{-3} = 2354{,}3 \text{ kN} > 431{,}0 \text{ kN}$$

$n = N_{Sd}/N_{b.Rd} = 431{,}0/2354{,}3 = 0{,}183$

$\alpha = \underline{W}_{pl}/W_{el} - 1 = 1283 \cdot 10^3/1150 \cdot 10^3 - 1 = 0{,}116$

$\mu = \lambda_2(2\beta_M - 4) + \alpha = 0{,}36\ (2 \cdot 1{,}8 - 4) + 0{,}116 = -0{,}028 < 0{,}9$

$1 - \mu n/\gamma_{M1} = 1 + 0{,}028 \cdot 0{,}183/1{,}1 = 1{,}005 < 1{,}5 \longrightarrow k = 1{,}005$

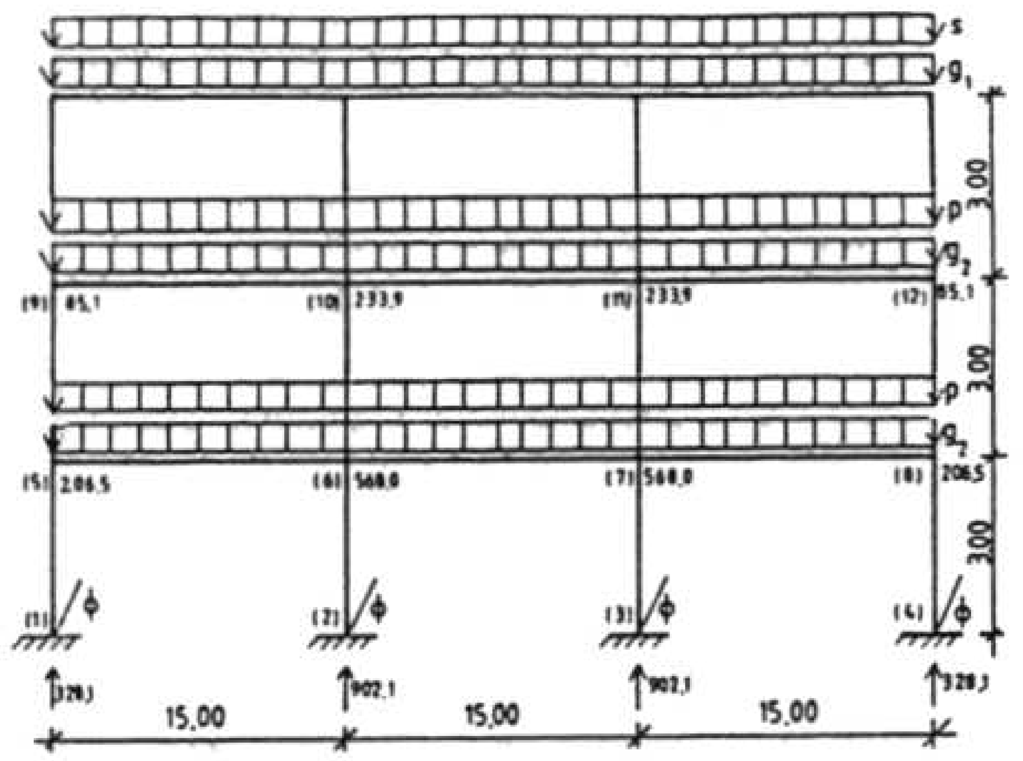
$$n + \frac{k_y\, M_{Sd}}{W_{y.pl}\,\dfrac{f_y}{\gamma_{Mo}}} = 0{,}183 + \frac{1{,}005 \cdot 218{,}0}{1283 \cdot 10^3 \cdot \dfrac{235}{1{,}1}}\, 10^6 = 0{,}98 < 1$$

this check is fulfilled

Problem, Sketch, Calculation	Reference	Commentary

Example 1.3.2: Determination of frame imperfections

1. System and loading



g_1 = 6,0 kN/m
g_2 = 10,0 kN/m
p = 5,0 kN/m
s = 4,5 kN/m

2. Determination of the imperfections for the load case 1,35 (g + p + s)
2.1 Total vertical reaction per storey
2.1.1 Ground floor

Reference: 5.2.3.1.2; [5.2.4.3]

ΣV_{Sd} = 1,35•[6,0 + 4,5 + 2•(10,0 + 5)]•15,0•3,0 = 2460,4 kN

Half of the "mean reaction force" per column:
$0{,}5 \cdot \bar{V}_{Sd} = 0{,}5 \cdot \Sigma V_{Sd}/4 = 0{,}5 \cdot 2460{,}4/4 = 307{,}6$ kN

The axial force of each column at this floor is higher than 307,6 kN, therefore all columns are included in n_c

2.1.2 1st floor

Reference: 5.2.3.1.2; [5.2.4.3]

ΣV_{Sd} = 1,35•[6,0 + 4,5 + 10,0 + 5]•15,0•3,0 = 1549,1 kN

Half of the "mean reaction force" per column:
$0{,}5 \cdot \bar{V}_{Sd} = 0{,}5 \cdot \Sigma V_{Sd}/4 = 0{,}5 \cdot 1549{,}1/4 = 193{,}6$ kN

The axial force of each column at this floor is higher than 193,6 kN, therefore all columns are included in n_c

Problem, Sketch, Calculation	Reference	Commentary
2.1.3 2[nd] floor	5.2.3.1.2; [5.2.4.3]	

ΣV_{Sd} = 1,35•[6,0 + 4,5]•15,0•3,0 = 637,9 kN

Half of the "mean reaction force" per column:
$0{,}5 \cdot \bar{V}_{Sd} = 0{,}5 \cdot \Sigma V_{Sd}/4 = 0{,}5 \cdot 637{,}9/4 = 79{,}7$ kN

The axial force of each column at this floor is higher than 79,7 kN, therefore all columns are included in n_c

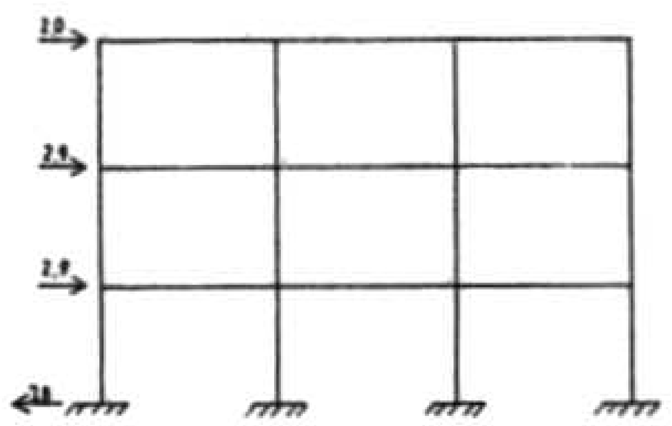
Problem, Sketch, Calculation	Reference	Commentary
2.2 Determination of the initial sway imperfection	5.2.3.1.2; [5.2.4.3]	

All columns extend through all storeys and therefore are included in n_s

⟶ $n_c = 4$; $n_s = 3$

⟶ $\Phi = k_c k_s \varphi_o$ (see table 5.6 in E-EC3)

$\Phi = 1/315$



$F_{I,2nd}$ = 637,9/315 = 2,0 kN
$F_{I,1st}$ = 911,3/315 = 2,9 kN
$F_{I,Gf}$ = 911,3/315 = 2,9 kN

1.4 Bracing system analysis

The first three examples demonstrate the application of the criterion "braced - unbraced". The fourth example demonstrates how bracing imperfections are determined.
The further steps of design are not treated in this part.

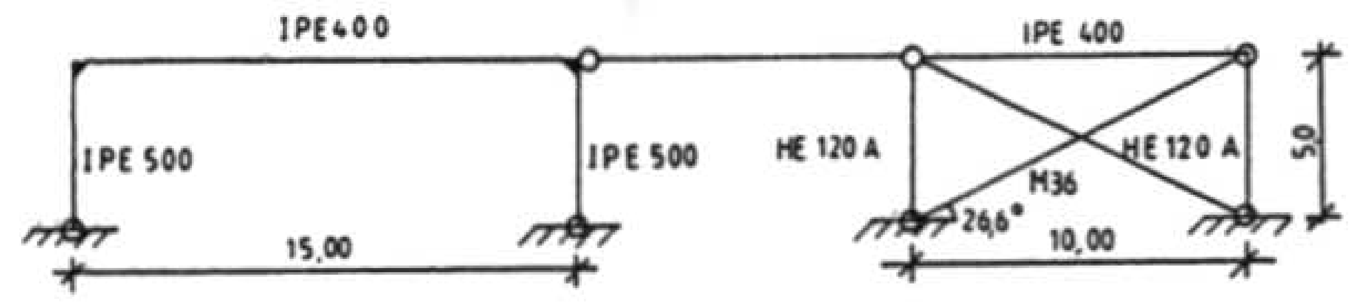
The following examples are included in this chapter:

Example 1.4.1:	A frame is braced by a bracing system
Example 1.4.2:	A frame is braced by a frame
Example 1.4.3:	A frame is braced by a bracing system
Example 1.4.4:	Bracing system imperfections for a wind bracing

Problem, Sketch, Calculation	Reference	Commentary

Example 1.4.1: A frame is braced by a bracing system

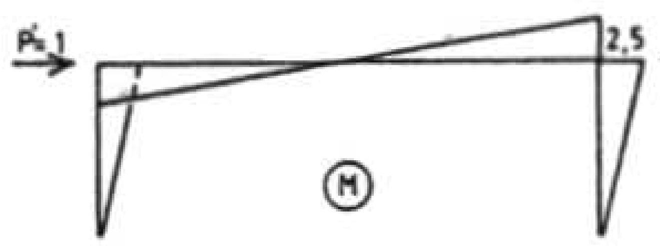
1. System



Frame:	column:	IPE 500
	beam:	IPE 400
Bracing system:	column 1:	HE 120 A
	column 2:	HE 120 A
	beam:	IPE 400
	diagonals:	ϕ 36

The criterion braced - unbraced gives an indication to which part of a structure or sub-structure the horizontal loads are transferred (i.e. any horizontal loads plus the initial sway imperfections applied on a braced frame are treated as affecting only the bracing system). However it is possible that the bracing system itself could be classified as sway.

2. Stiffness of the frame



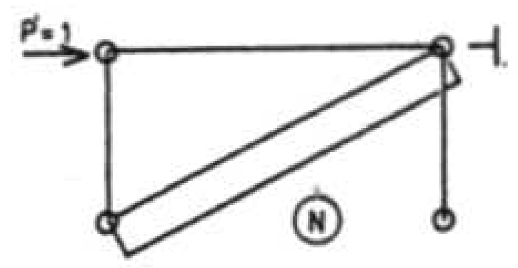
$$\delta = \int \frac{M\,M'}{E\,I}\,dx \qquad \text{Assumption: } EA \approx \infty$$

$$\delta = \frac{2{,}5^2 \cdot 5{,}0}{3 \cdot 210000 \cdot 48200 \cdot 10^4} \cdot 2 \cdot 10^9 + \frac{2{,}5^2 \cdot 7{,}5}{3 \cdot 210000 \cdot 23130 \cdot 10^4} \cdot 2 \cdot 10^9 = 0{,}00085 \text{ m/kN}$$

$$S_{frame} = \frac{1}{\delta} = 1177{,}6 \text{ kN/m}$$

A braced frame may be treated as fully supported laterally.

3. Stiffness of the bracing system



$$\delta = \int \frac{N\,N'}{E\,A}\,dx$$

$$\delta = \frac{1{,}0}{210000 \cdot 10{,}2 \cdot 10^2 \cdot \cos^2 26{,}6°} \cdot \frac{10{,}0}{\cos 26{,}6°} \cdot 10^3 = 0{,}000065 \text{ m/kN}$$

$$S_{brace} = \frac{1}{\delta} = 15336 \text{ kN/m}$$

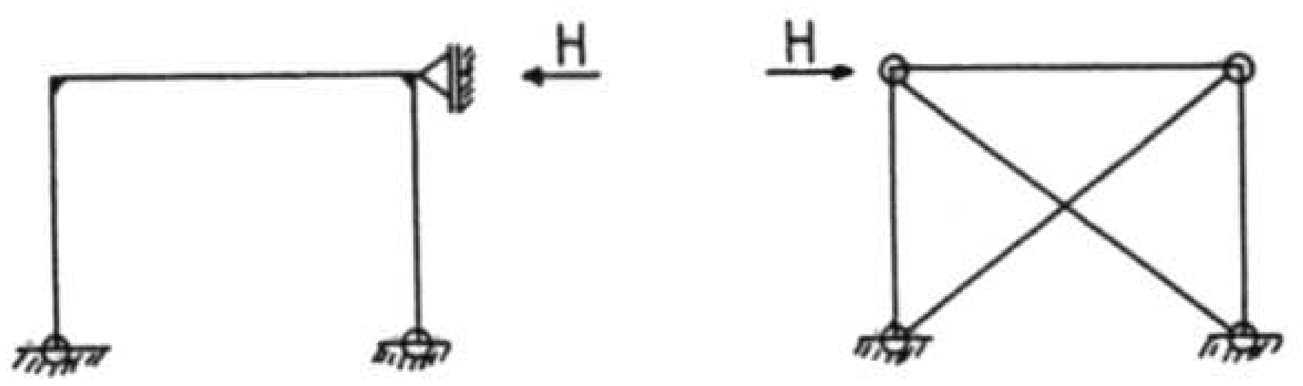
Only the bracing diagonal loaded in tension is used to calculate the system stiffness (since the compression diagonal has insignificant resistance.

4. Criterion

Reference: 5.2.3; Table 5.3 [5.2.5.3]

$S_{brace}/S_{frame} = 15336/1177{,}6 = 13{,}02 > 5$

----→ the continued system can be calculated as a unique system or separated into sub-systems where the frame may be considered as braced and the bracing system is subjected with the horizontal forces



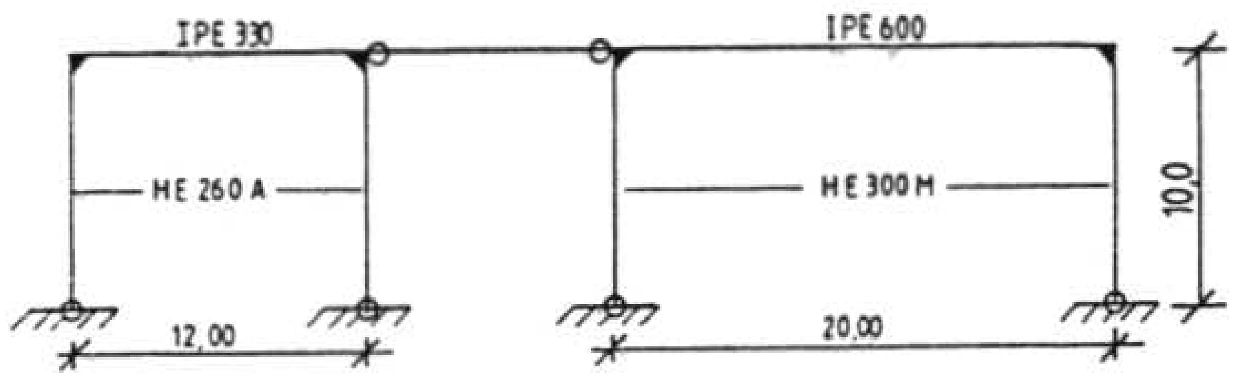
A frame may be classified as braced if the stiffness of the bracing system is at least 5 times greater than the stiffness of the frame.

Problem, Sketch, Calculation	Reference	Commentary

Example 1.4.2: A frame is braced by a frame

Commentary: A frame may be assumed as a bracing system when it is sufficiently stiff to resist all horizontal loads.

1. System

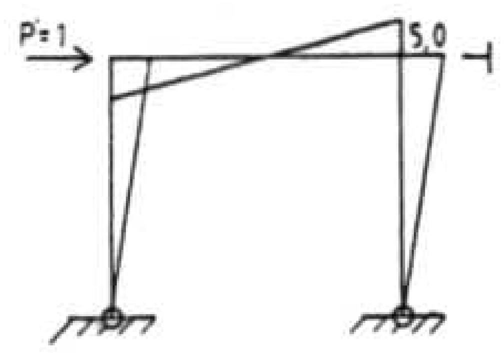


frame.1 frame.2

Frame 1: column: HE 260 A
beam: IPE 330

Frame 2: column: HE 300 M
beam: IPE 600

2. Stiffness of the frame.1



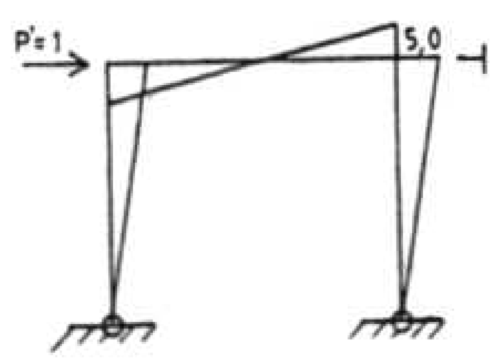
$$\delta = \int \frac{M\,M'}{E\,I}\,dx$$

Assumption: $EA \approx \infty$

$$\delta = \frac{5{,}0^2 \cdot 10{,}0}{3 \cdot 210000 \cdot 10450 \cdot 10^4} \cdot 2 \cdot 10^9 + \frac{5{,}0^2 \cdot 6{,}0}{3 \cdot 210000 \cdot 11770 \cdot 10^4} \cdot 2 \cdot 10^9 = 0{,}0116 \text{ m/kN}$$

$$S_{frame.1} = \frac{1}{\delta} = 85{,}9 \text{ kN/m}$$

3. Stiffness of the frame.2



$$\delta = \int \frac{M\,M'}{E\,I}\,dx$$

Assumption: $EA \approx \infty$

$$\delta = \frac{5{,}0^2 \cdot 10{,}0}{3 \cdot 210000 \cdot 59200 \cdot 10^4} \cdot 2 \cdot 10^9 + \frac{5{,}0^2 \cdot 10{,}0}{3 \cdot 210000 \cdot 92080 \cdot 10^4} \cdot 2 \cdot 10^9 = 0{,}0022 \text{ m/kN}$$

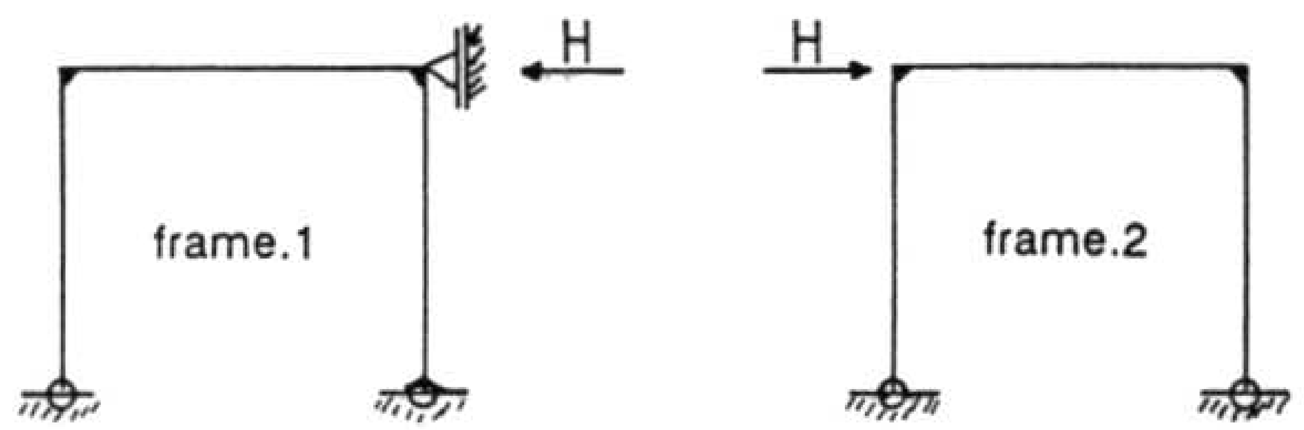
$$S_{frame.2} = \frac{1}{\delta} = 454{,}0 \text{ kN/m}$$

4. Criterion

Reference: 5.2.3; Table 5.3
[5.2.5.3]

$S_{frame.2}/S_{frame.1} = 454/85{,}9 = 5{,}29 > 5 \bullet S_{frame.1}$

⟶ frame.1 may be considered as braced

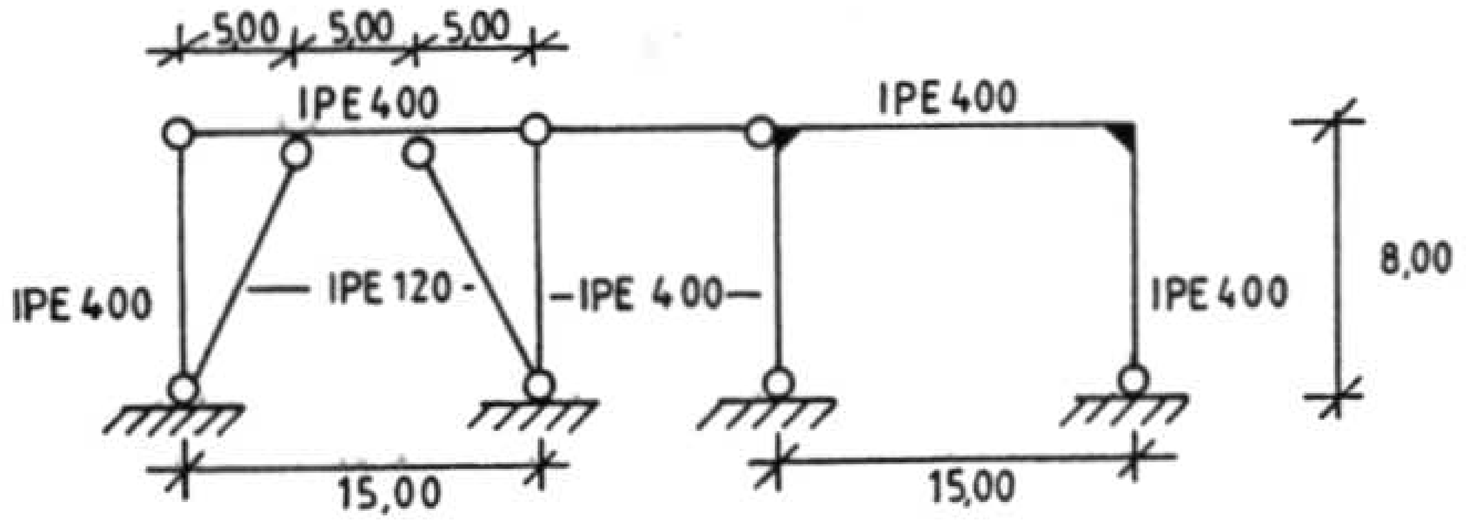


Commentary: Frame.1 may be classified as braced if the stiffness of frame.2 is 5 times greater than the stiffness of the frame.1.
The continued system may be separated into subframes where the frame.1 may be considered as braced and the frame.2 which is designed to resist the horizontal forces.

Problem, Sketch, Calculation	Reference	Commentary

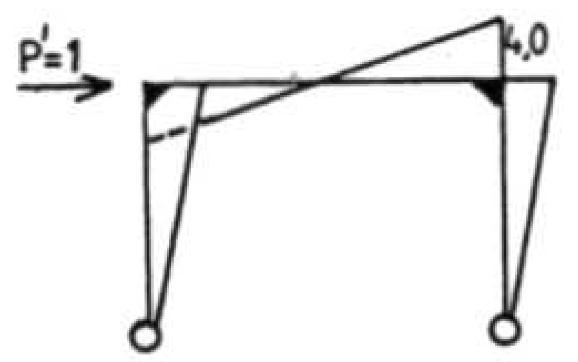
Example 1.4.3: A frame is braced by a bracing system

1. System



Frame:	column:	IPE 400
	beam:	IPE 400
Bracing system:	column:	IPE 400
	beam:	IPE 400
	Diagonals:	IPE 120

2. Stiffness of the frame



$$\delta = \int \frac{N\,N'}{E\,A}\,dx + \int \frac{M\,M'}{E\,I}\,dx$$

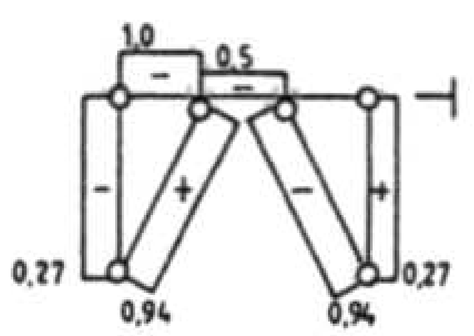
Assumption: $EA \approx \infty$

$$\delta = \frac{4{,}0^2 \cdot 8{,}0}{3 \cdot 210000 \cdot 23130 \cdot 10^4} \cdot 2 \cdot 10^9 + \frac{4{,}0^2 \cdot 7{,}5}{3 \cdot 210000 \cdot 23130 \cdot 10^4} \cdot 2 \cdot 10^9 = 0{,}0034 \text{ m/kN}$$

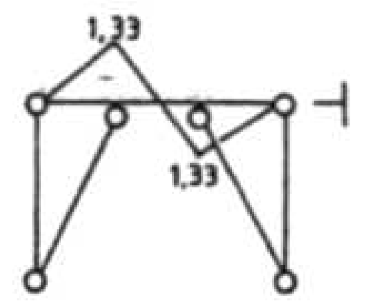
$$S_{frame.1} = \frac{1}{\delta} = 293{,}8 \text{ kN/m}$$

3. Stiffness of the bracing system

For the calculation of the system - stiffness both diagonals are considered (The connections of the diagonals must be formed such that the compression forces can be transferred).



$$\delta = \int \frac{N\,N'}{E\,A}\,dx + \int \frac{M\,M'}{E\,I}\,dx$$



$$\delta = \frac{0{,}27^2 \cdot 8{,}0 \cdot 2 \cdot 10^3}{210000 \cdot 84{,}5 \cdot 10^2} + \frac{0{,}94^2 \cdot 9{,}434 \cdot 2 \cdot 10^3}{210000 \cdot 13{,}2 \cdot 10^2} + \frac{5{,}0 \cdot 10^3}{210000 \cdot 84{,}5 \cdot 10^2} + \frac{0{,}5^2 \cdot 5{,}0 \cdot 10^3}{210000 \cdot 84{,}5 \cdot 10^2} +$$

$$\frac{1{,}33^2 \cdot 5{,}0 \cdot 2 \cdot 10^9}{3 \cdot 210000 \cdot 23130 \cdot 10^4} + \frac{1{,}33^2 \cdot 2{,}5 \cdot 2 \cdot 10^9}{3 \cdot 210000 \cdot 23130 \cdot 10^4} = 0{,}00024641 \text{ m/kN}$$

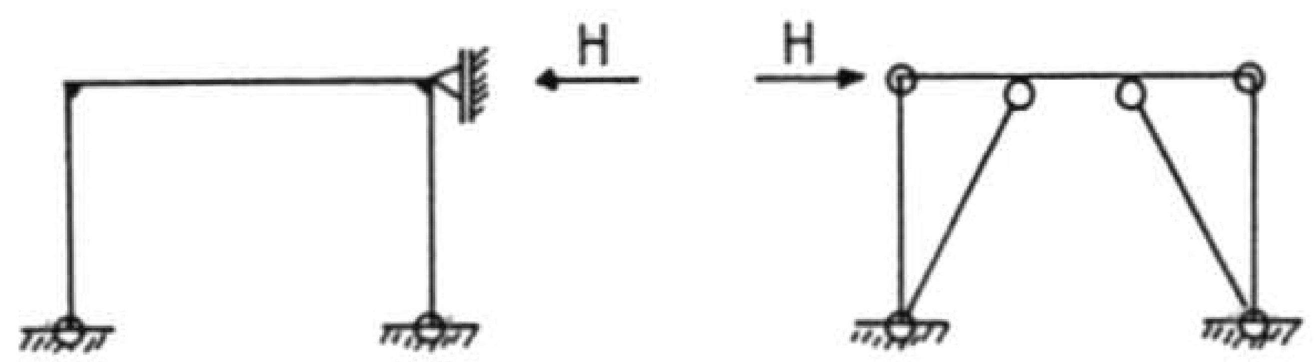
$$S_{frame.2} = \frac{1}{\delta} = 4058{,}3 \text{ kN/m}$$

4. Criterion

Reference: 5.2.3; Table 5.3 [5.2.5.3]

$S_{brace}/S_{frame} = 4058{,}3/293{,}8 = 13{,}8 > 5$

$\longrightarrow$ the frame could be considered as braced

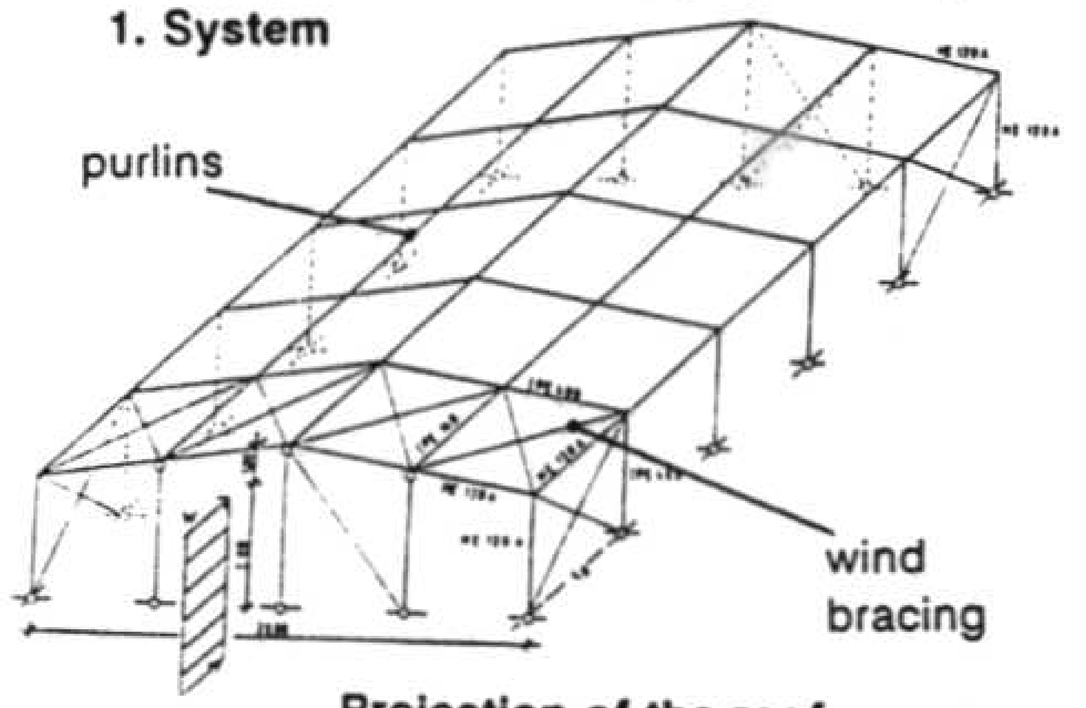


The frame may be classified as braced if the stiffness of the bracing system is at least 5 times greater than the stiffness of the frame.
The frame may be considered as braced and the bracing system is designed to resist the horizontal forces.

Problem, Sketch, Calculation	Reference	Commentary

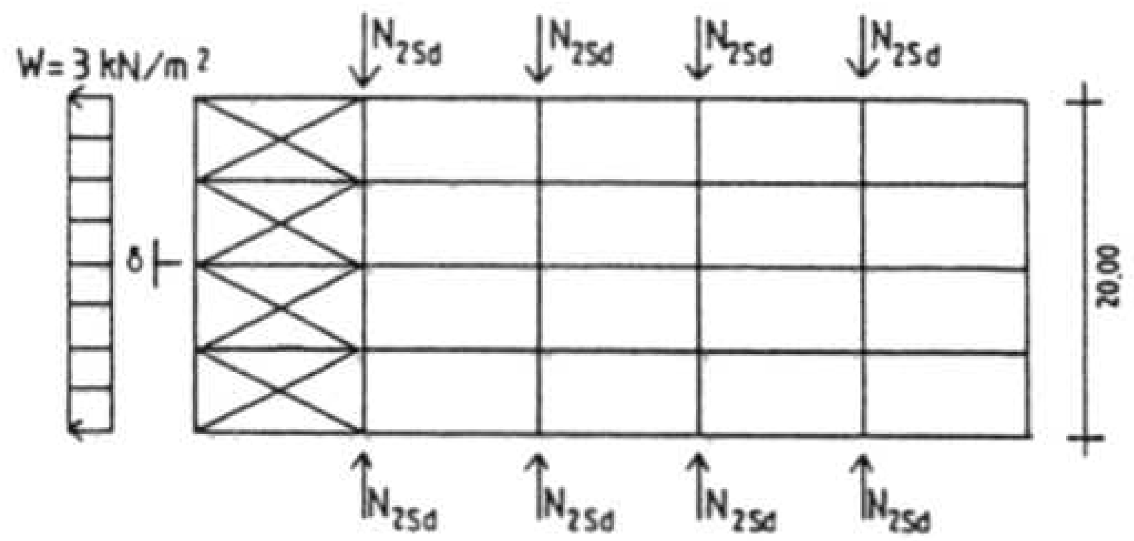
Example 1.4.4: Bracing system imperfections for a wind bracing

1. System



Frames: column: IPE 500
beams: IPE 400
Bracing: Diagonals: L 60x40x5
Chord 1: HE 120 A
Chord 2: IPE 400

Projection of the roof



2. Loading

$\gamma_F = 1{,}5$

$w = 1{,}5 \cdot 2{,}0 = 3{,}0$ kN/m

h - depth of the beam

Mid - span moment of beam:

$M_{Sd} = 310$ kNm

Axial force applied on the bracing system:

$N_{Sd} = M_{Sd}/h = 310/0{,}4 = 775$ kN

Reference: Table 5.8 [5.2.4.4.(3)]

3. Determination of the bracing system imperfections

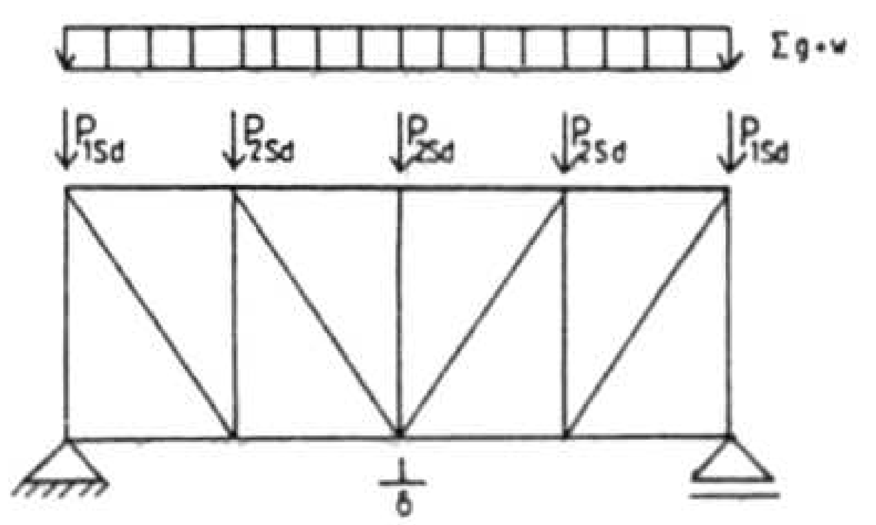
Assumption: the total deflection δ is lower than L/2000

---→ with $n_r = 4$ (number of members to be restrained)

$n_p = 4$ (number of panels) ---→ $\zeta = 1{,}0$

$\Sigma N_{Sd} = 4 \cdot 775 = 3100$ kN

Reference: 5.2.3.2.1; Table 5.7; Table 5.8 [5.2.4.4]



---→ Equivalent stabilizing force Σq

$$\sum q = \frac{1{,}0}{67{,}9}\;\frac{3100{,}0}{20{,}0} = 2{,}28 \text{ kN/m}$$

Reference: Table 5.8; [Figure 5.2.5]

4. Determination of the deformation δ of the bracing system

The total loads applied on the wind bracing are $\Sigma q + w$

---→ Equivalent panel loads from $\Sigma q + w$

$P_{2\,Sd} = (2{,}28 + 3{,}0) \cdot 20/4 = 26{,}4$ kN

$P_{1\,Sd} = P_{2\,Sd}/2 = 13{,}2$ kN

---→ $\delta = 9{,}5$ mm < 10 mm = L/2000

Reference: Table 5.8 [Figure 5.2.5]

The bracing system has to be designed to resist [5.2.5.3.(6)]:

- any loads applied directly on the bracing system
- any horizontal loads applied to the frames which it braces
- the effects of the initial imperfections

Part 2

Members in compression
Members in bending
Combined loading - Bending and compression
Local stresses

2.1 Members in compression

These examples demonstrate the verification of members in compression assuming design values of action effects (N_{Sd}) which have been calculated by an analysis of the structure and these values already include ψ, γ_F etc.

The results presented in the examples are rounded values. For the purpose of easy re-calculation each formula and each check is calculated with the rounded values.

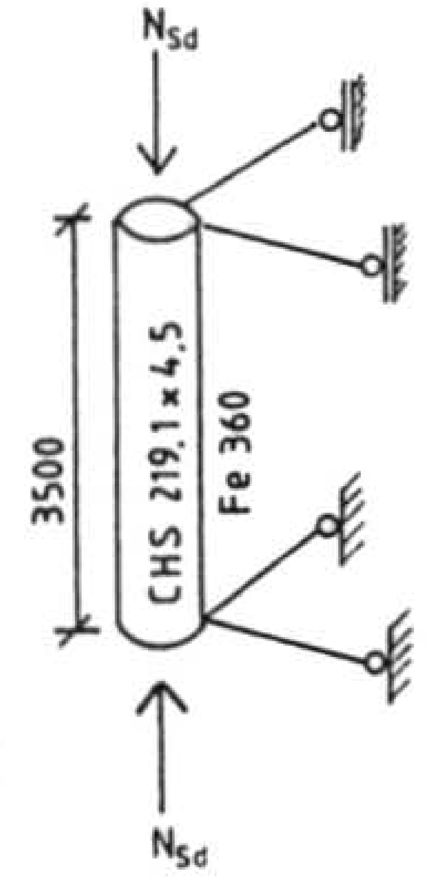
The following examples are included in this chapter:

Example 2.1.1:	Circular hollow section as a column
Example 2.1.2:	HEB profile as column
Example 2.1.3:	Cold formed RHS as a strut of a lattice girder
Example 2.1.4:	Angle as a strut of a lattice girder
Example 2.1.5:	Cold formed RHS (class 4 cross-section)

Problem, Sketch, Calculation	Reference	Commentary

Example 2.1.1: Circular hollow section as a column

1. System



Column: CHS 219,1 x 4,5 hot finished
Steel grade: Fe 360
System length = Buckling length = ℓ = 3,50 m

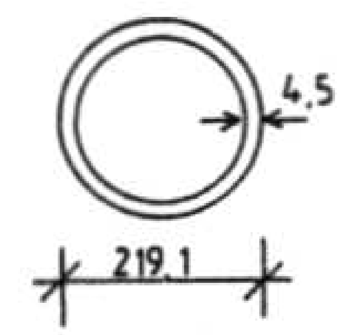
Commentary: The wording in Eurocode 3 used for "hot finished" is "hot rolled"

2. Loading

Compression force: N_{Sd} = 600 kN

3. Determination of the resistance

3.1 Diameter-to-thickness ratio



Fe 360 $\longrightarrow \epsilon = 1,0$
$d/t = 219,1/4,5 = 48,7 < 90\epsilon^2 = 90$
$\longrightarrow A_{eff} = A$

Reference: 5.3.1; Table 5.9; [5.3; Table 5.3.1]

Commentary: The whole cross-section is effective to resist the loadings.

3.2 Member resistance

Buckling length: ℓ = 3,50 m

$$\bar{\lambda} = \frac{\ell}{i}\frac{1}{\lambda_1} = \frac{3,50\cdot 10^3}{75,9}\cdot\frac{1}{93,9} = 0,49$$

using buckling curve a $\longrightarrow \chi_a = 0,9273$

$$N_{b.Rd} = \chi_a\cdot\frac{A\cdot f_y}{\gamma_{M1}} = 0,9273\cdot\frac{30,3\cdot 10^2\cdot 235}{1,1}\,10^{-3} = 600,3\text{ kN} > 600\text{ kN}$$

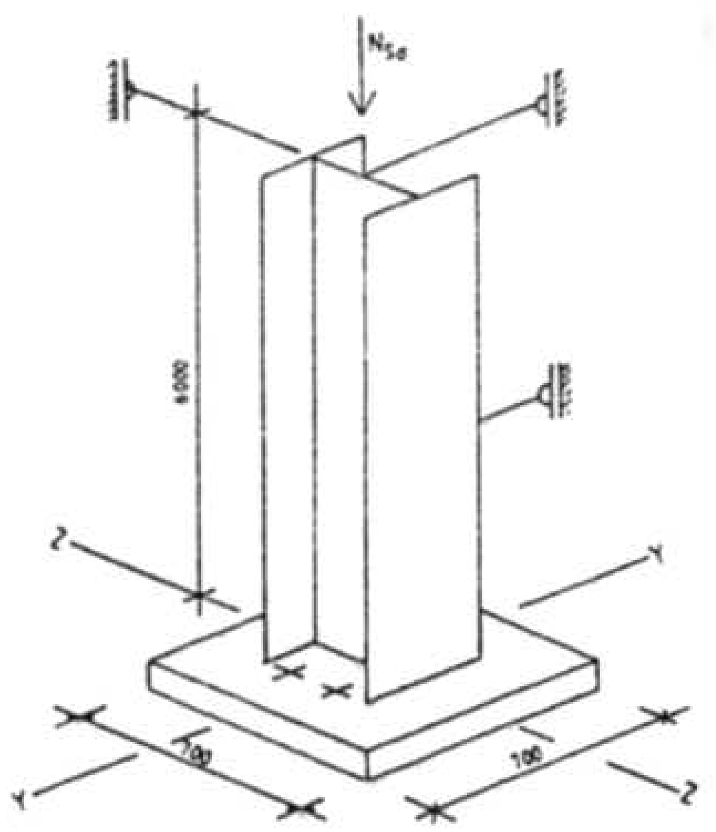
Reference: 5.4.2; Table 5.21 [5.5.1; Table 5.5.3]

Commentary: Determination of χ, see tables in Appendix A

Problem, Sketch, Calculation	Reference	Commentary

Example 2.1.2: HEB profile as column

1. System



Column: HE 300 B

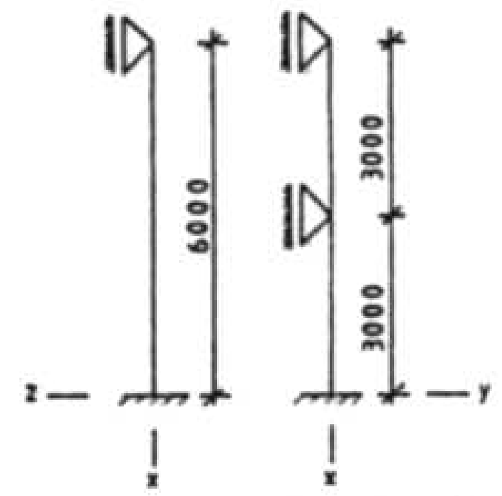
Steel grade: Fe 360

System length: L = 6,0 m

Buckling length: $\ell_y = 0{,}7 \cdot 6{,}0 \text{ m} = 4{,}20 \text{ m}$

$\ell_z = 0{,}88 \cdot 3{,}0 \text{ m} = 2{,}64 \text{ m}$

Commentary: The determination of the buckling length is based on a simple assumption [Annex E.2.1]:



2. Loading

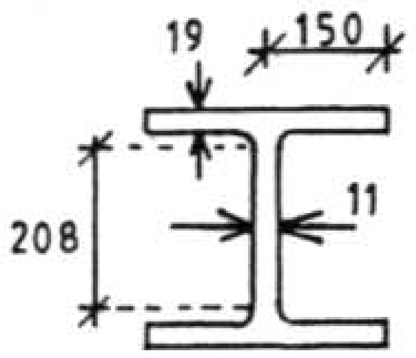
Compression force: $N_{Sd} = 2900 \text{ kN}$

3. Determination of the resistance

3.1 Width-to-thickness ratio

Reference: 5.3.1; Table 5.9; [5.3; Table 5.3.1]

Commentary: The whole cross-section is effective to resist the loadings.



Fe 360 $\longrightarrow \epsilon = 1{,}0$

Flange:

$c/t = 150/19 = 7{,}9 < 15\epsilon = 15$

Web:

$d/t = 208/11 = 18{,}9 < 42\epsilon = 42$

$\longrightarrow A_{eff} = A$

3.2 Member resistance

3.2.1 Buckling about the strong axis

Reference: 5.4.2; Table 5.21 [5.5.1; Table 5.5.3]

Commentary: The stability of compression members needs to be checked according to the two principal axes of the section with appropriate effective lengths. Determination of χ, see tables in Appendix A

Buckling length: $\ell_y = 4{,}20 \text{ m}$

$$\bar{\lambda}_y = \frac{\ell_y}{i_y}\,\frac{1}{\lambda_1} = \frac{4{,}20 \cdot 10^3}{130} \cdot \frac{1}{93{,}9} = 0{,}344 \approx 0{,}35$$

using buckling curve b $\longrightarrow \chi_b = 0{,}9455$

$$N_{b.Rd} = \chi_b \cdot \frac{A \cdot f_y}{\gamma_{M1}} = 0{,}9455 \cdot \frac{149 \cdot 10^2 \cdot 235}{1{,}1}\, 10^{-3} = 3009{,}6 \text{ kN} > 2900 \text{ kN}$$

3.2 Member resistance

3.2.2 Buckling about the weak axis

Reference: 5.4.2; Table 5.21 [5.5.1; Table 5.5.3]

Commentary: Determination of χ, see tables in Appendix A

Buckling length: $\ell_z = 2{,}64 \text{ m}$

$$\bar{\lambda}_z = \frac{\ell_z}{i_z}\,\frac{1}{\lambda_1} = \frac{2{,}64 \cdot 10^3}{75{,}8} \cdot \frac{1}{93{,}9} = 0{,}37$$

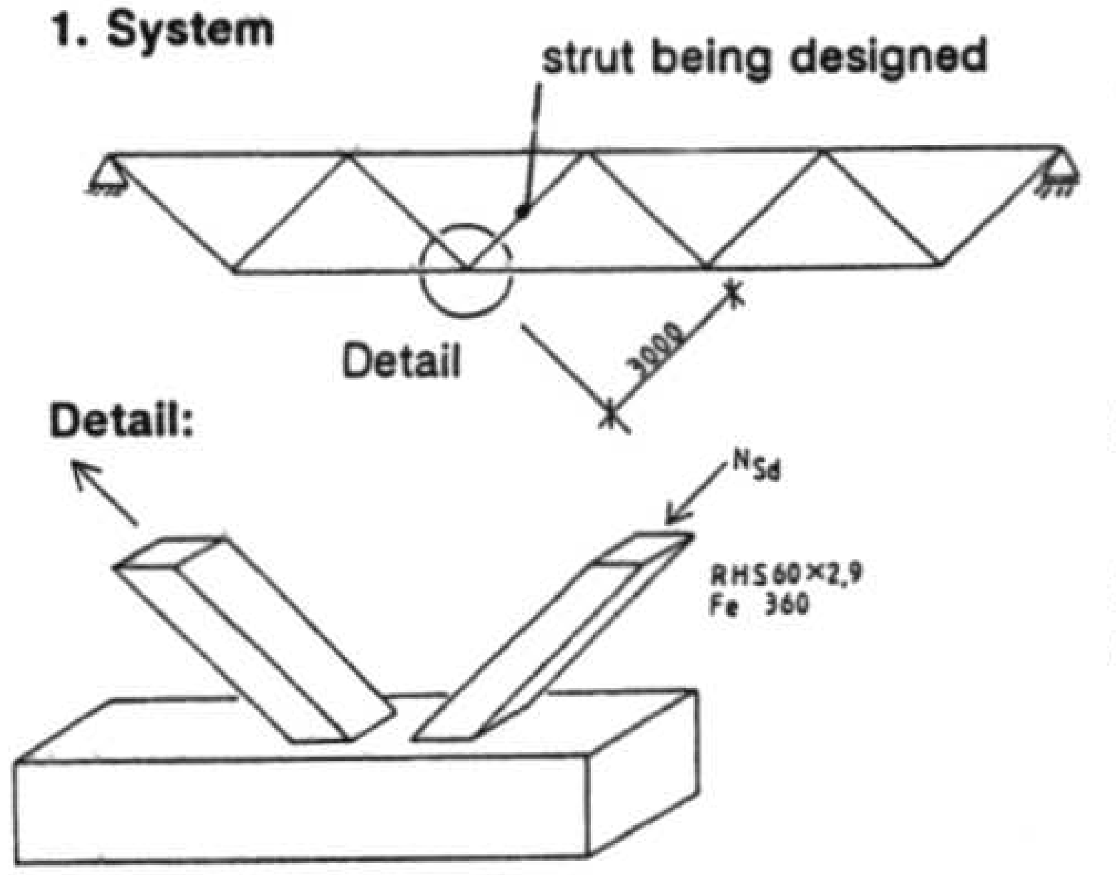
using buckling curve c $\longrightarrow \chi_c = 0{,}9131$

$$N_{b.Rd} = \chi_c \cdot \frac{A \cdot f_y}{\gamma_{M1}} = 0{,}9131 \cdot \frac{149 \cdot 10^2 \cdot 235}{1{,}1}\, 10^{-3} = 2906{,}6 \text{ kN} > 2900 \text{ kN}$$

Problem, Sketch, Calculation	Reference	Commentary

Example 2.1.3: Cold formed RHS as a strut of a lattice girder

1. System



Strut: RHS 60 x 2,9 cold formed
Steel grade: Fe 360
System length: L = 3,0 m
Buckling length: ℓ = 0,75•3,0 m = 2,25 m

Commentary: The buckling length of hollow section bracings welded around the perimeter without cropping and flattening can be taken as 0,75•L for in and out of plane buckling. [Annex KK (in preparation)]
The choice of the profile should be made considering the practical and cost aspects of the connection design.

2. Loading

Compression force: N_{Sd} = 70 kN

3. Determination of the resistance

3.1 Width-to-thickness ratio

Reference: 5.3.1; Table 5.9; [5.3; Table 5.3.1]

Fe 360 ⟶ $\epsilon = 1{,}0$
$d = b - 3t = 60 - 3 \cdot 2{,}9 = 51{,}3$ mm
$d/t = 51{,}3/2{,}9 = 17{,}7 < 38\epsilon = 38$
⟶ $A_{eff} = A$

Commentary: The whole cross-section is effective to resist the loadings. Class 2 is necessary because of redistribution of secondary effects.

3.2 Member resistance

Reference: 5.4.2; Table 5.21 [5.5.1; Table 5.5.3]

Commentary: Determination of χ, see tables in Appendix A

Buckling length: ℓ = 2,25 m

$$\bar{\lambda} = \frac{\ell}{i}\frac{1}{\lambda_1} = \frac{2{,}25 \cdot 10^3}{23{,}1} \cdot \frac{1}{93{,}9} = 1{,}04$$

using buckling curve b ⟶ $\chi_b = 0{,}5719$

$$N_{b.Rd} = \chi_b \cdot \frac{A \cdot f_y}{\gamma_{M1}} = 0{,}5719 \cdot \frac{6{,}41 \cdot 10^2 \cdot 235}{1{,}1}\ 10^{-3} = 78{,}3 \text{ kN} > 70 \text{ kN}$$

Alternative verification by using the average yield strength f_{ya}

Reference: -; [5.5.1.4; Fig. 5.5.2; Table 5.5.3]

Commentary:
f_{ya} = average yield strength
f_{yb} = yield strength of basic material

k = 7 for cold rolling
n = 4 bends of 90°

Determination of χ, see tables in Appendix A

$$f_{ya} = f_{yb} + \frac{k\,n\,t^2}{A_g}(f_{ub} - f_{yb}) = 235 + \frac{7 \cdot 4 \cdot 2{,}9^2}{641} \cdot (360 - 235) = 280{,}9 \text{ N/mm}^2$$

$$< f_{ub} = 360 \text{ N/mm}^2$$

$$< 1{,}2 \cdot f_{yb} = 282 \text{ N/mm}^2$$

$$\lambda_1 = \pi\sqrt{\frac{E}{f_{ya}}} = \pi\sqrt{\frac{210000}{280{,}9}} = 85{,}9 \ \text{--->}\ \bar{\lambda} = \frac{\ell}{i}\frac{1}{\lambda_1} = \frac{2{,}25 \cdot 10^3}{23{,}1} \cdot \frac{1}{85{,}9} = 1{,}13$$

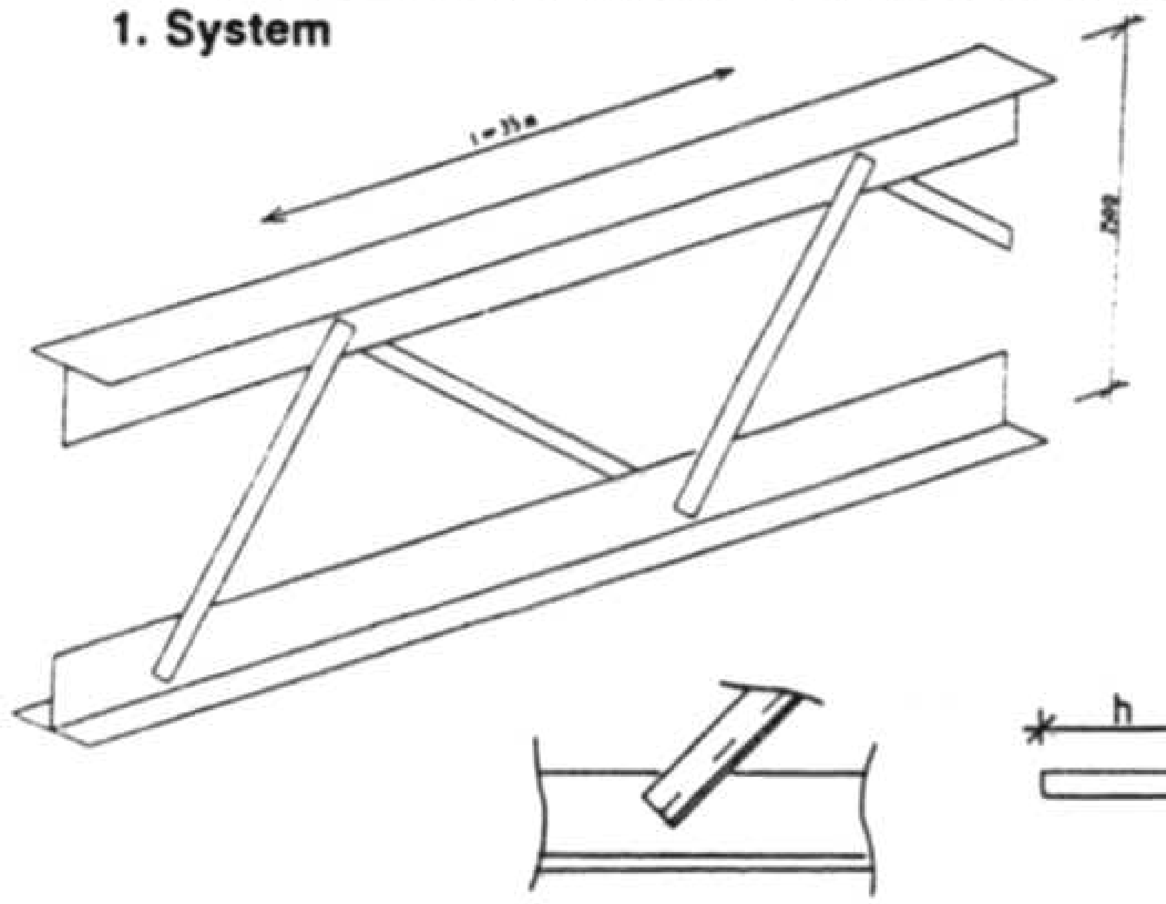
using buckling curve c ⟶ $\chi_c = 0{,}4685$

$$N_{b.Rd} = \chi_c \cdot \frac{A \cdot f_{ya}}{\gamma_{M1}} = 0{,}4685 \cdot \frac{6{,}41 \cdot 10^2 \cdot 28{,}09}{1{,}1}\ 10^{-3} = 76{,}7 \text{ kN} > 70 \text{ kN}$$

Problem, Sketch, Calculation	Reference	Commentary

Example 2.1.4: Angle as a strut of a lattice girder

1. System



Strut: L 100 x 10

Steel grade:	Fe 430
System length:	L = 3,50 m
Buckling length:	ℓ = 3,50 m

Reference: [5.8.3]

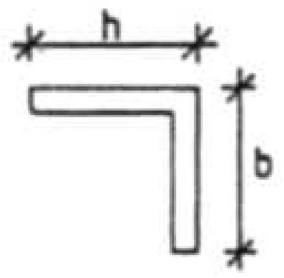
Commentary: If the chords supply appropriate end restraint and the end connections supply appropriate fixity (e.g. at least 2 bolts or welded) the effective length of a web member as an angle is taken as 1,0L. The eccentricities of the connection may be neglected and an effective slenderness ratio used in such cases.

2. Loading

Compression force: N_{Sd} = 100 kN

3. Determination of the resistance

3.1 Width-to-thickness ratio



Fe 430 $\longrightarrow \epsilon = 0{,}92$

$(h+b)/2t = 200/20 = 10 < 11{,}5\epsilon = 10{,}6$

$\longrightarrow A_{eff} = A$

Reference: 5.3.1; Table 5.9; [5.3; Table 5.3.1]

Commentary: $h/t = 100/10 = 10 < 15\epsilon = 13{,}8$
$\rightarrow$ criterion only decisive, when $b < 0{,}53 \cdot h$
The whole cross-section is effective to resist the loadings.

3.2 Member resistance

Buckling length: ℓ = 3,50 m

$$\bar{\lambda}_v = \frac{\ell}{i_v}\cdot\frac{1}{\lambda_1} = \frac{3{,}50\cdot 10^3}{19{,}5}\cdot\frac{1}{86{,}8} = 2{,}07 \quad \longrightarrow \bar{\lambda}_{eff,v} = 0{,}35 + 0{,}7\bar{\lambda}_v = 0{,}35 + 0{,}7\cdot 2{,}07 = 1{,}80$$

using buckling curve c $\longrightarrow \chi_c = 0{,}2345$

$$N_{b,Rd} = \chi_c\cdot\frac{A\cdot f_y}{\gamma_{M1}} = 0{,}2345\cdot\frac{19{,}2\cdot 10^2\cdot 275}{1{,}1}\,10^{-3} = 112{,}6 \text{ kN} > 100 \text{ kN}$$

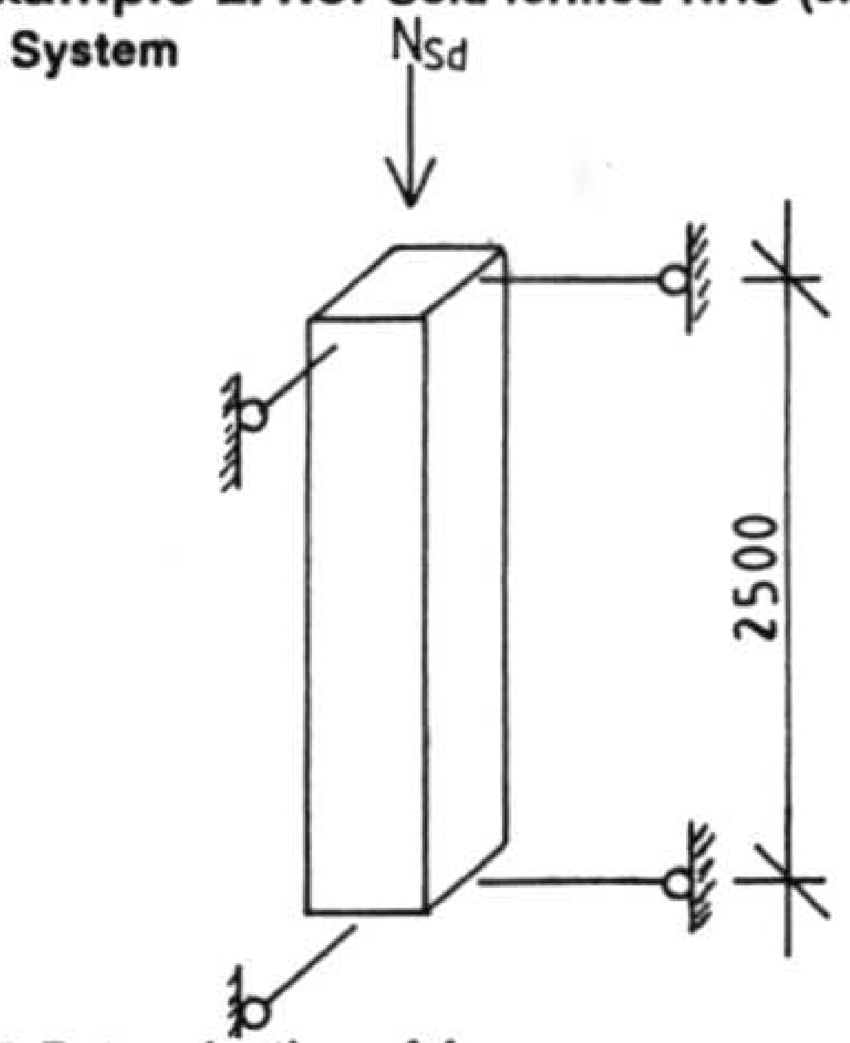
Reference: 5.4.2; Table 5.21 [5.5.1; Table 5.5.3]

Commentary: Buckling about the weak axis v is critical. Determination of χ, see tables in Appendix A.

Problem, Sketch, Calculation	Reference	Commentary

Example 2.1.5: Cold formed RHS (class 4 cross-section)

1. System



compression member: RHS 400 x 8,0 , cold formed
Steel grade: Fe 510
System length: L = 2,50 m
Buckling length: ℓ = 2,50 m

2. Loading

Compression force: N_{Sd} = 3200 kN

3. Determination of the resistance

3.1 Width-to-thickness ratio

Fe 510 $\longrightarrow \epsilon = 0{,}81$

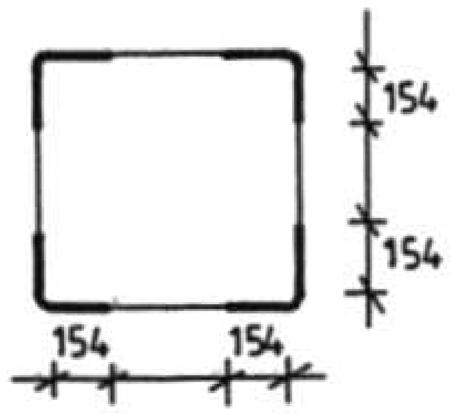
$d = b - 3t = 400 - 3 \cdot 8{,}0 = 376$ mm

$d/t = 376/8{,}0 = 47 > 42\epsilon = 34{,}0$

Reference: 5.3.1; Table 5.9; [5.3; Table 5.3.1]

Commentary: The whole cross-section is not effective to resist the loadings. Therefore the effective cross-section properties have to be determined.

3.2 Determination of A_{eff}



Fe 510 $\longrightarrow 1/\epsilon = 1{,}23$

$$\bar{b} = b - 3t = 376 \text{ mm}; \quad \bar{\lambda}_p = \frac{(b-3t)/t}{28{,}4\,\epsilon\,\sqrt{k_\sigma}} = \frac{b-3t}{t}\,\frac{1}{56{,}8\,\epsilon} = 47 \cdot \frac{1{,}23}{56{,}8} = 1{,}018 > 0{,}673$$

$$\rho = \frac{\bar{\lambda}_p - 0{,}22}{\bar{\lambda}_p^2} = \frac{1{,}018 - 0{,}22}{1{,}018^2} = 0{,}77$$

$d_{eff} = \rho \cdot \bar{b} = 0{,}77 \cdot 400 = 308$ mm

$A_{eff} = A - 4t(\bar{b} - d_{eff}) = 123 \cdot 10^2 - 4 \cdot 8{,}0 \cdot (376 - 308) = 101{,}24 \cdot 10^2$ mm^2

Reference: 5.3.5; Table 5.18 [5.3.5; Table 5.3.2]

Commentary: The effective cross-section properties are calculated on the basis of f_y for compression elements. As there is no shift of the centroidal axis when the effective cross section is subject to uniform compression, then no additional moment needs to be considered.

3.3 Buckling resistance

Buckling length: ℓ = 2,50 m

$$\bar{\lambda} = \frac{\ell}{i}\,\frac{1}{\lambda_1}\sqrt{\frac{A_{eff}}{A}} = \frac{2{,}50 \cdot 10^3}{159} \cdot \frac{1}{76{,}4} \cdot \sqrt{\frac{101{,}24 \cdot 10^2}{123{,}00 \cdot 10^2}} = 0{,}187 < 0{,}2 \quad \longrightarrow \chi = 1$$

$$N_{c.Rd} = \frac{A_{eff} \cdot f_y}{\gamma_{M1}} = \frac{101{,}24 \cdot 10^2 \cdot 355}{1{,}1}\,10^{-3} = 3267{,}3 \text{ kN} > 3200{,}0 \text{ kN}$$

Reference: 5.4.2; [5.5.1]

Commentary: The radius of gyration of the gross section is used to determine the non-dimensional slenderness, but the ratio of A_{eff} to A has to be taken into account.

Because of $\bar{\lambda} < 0{,}2$ no allowance for buckling is necessary, i.e. only cross-section verification is required $\longrightarrow$ use of γ_{M1} because of class 4 section.

2.2 Members in bending

Examples 1, 2 and 3 demonstrate the verification of members in bending assuming design values of action effects (V_{Sd}, M_{Sd}, etc) which are calculated by an analysis of the sub-structure.
Example 4 demonstrates the verification of members in bending assuming design values of action effects (V_{Sd}, M_{Sd}, etc) which have been calculated by an analysis of the structure and these values already include ψ, γ_F etc.
The results presented in the examples are rounded values. For the purpose of easy re-calculation each formula and each check is calculated with the rounded values.

The following examples are included in this chapter:

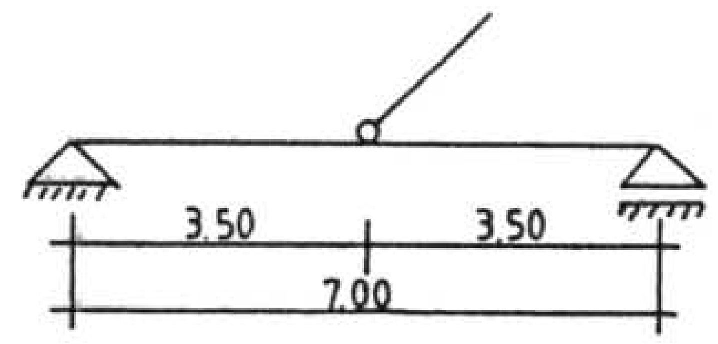
Example 2.2.1:	Single span beam
Example 2.2.2:	Single span beam with lateral torsional buckling
Example 2.2.3:	Single span beam with shear buckling verification
Example 2.2.4:	Class 4 cross-section loaded in bending

Problem, Sketch, Calculation	Reference	Commentary

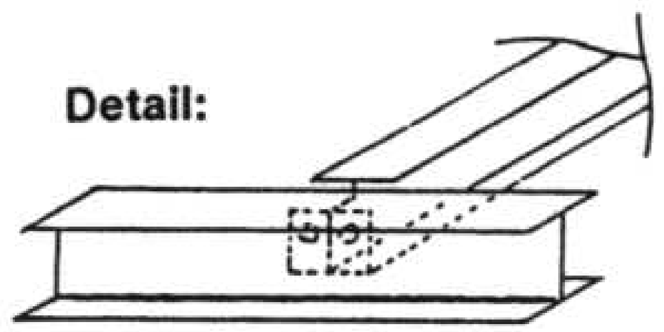
Example 2.2.1: Single span beam

1. System

At mid span a side girder is connected by a nominally pinned connection.
Lateral restraints are assumed at the supports.
For determinig the cross-section properties I_w, I_t, I, W, A, A_v, i etc. dimension tables are used.



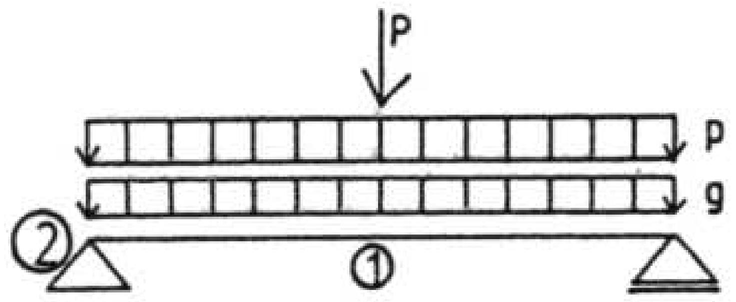
Detail:



Main girder:	IPE 500	
	Steel grade:	Fe 430
	System length:	ℓ = 7,00 m

2. Loading

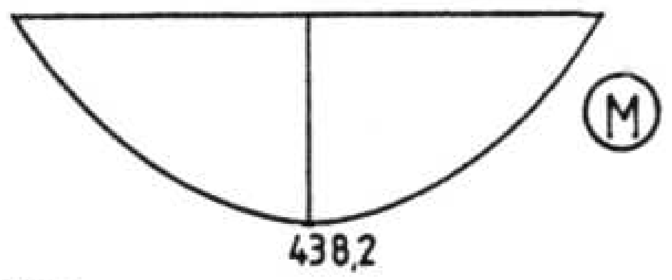
Dead load includes the weight of floor, etc.
Since the imposed loads p and P are not independent from each other they are considered as one variable action.



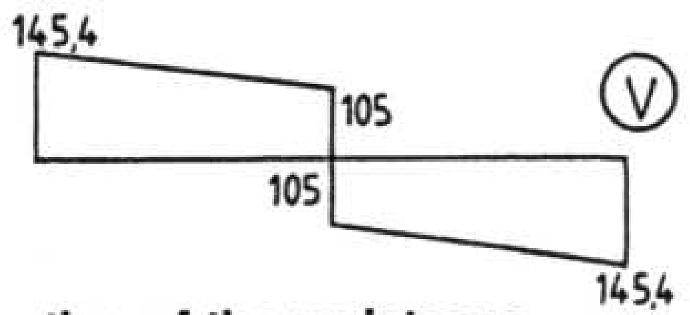
Dead load: g = 3,0 kN/m
Imposed load: p = 5,0 kN/m; P = 140 kN
Relevant load combination of this load case:
1,35•g + 1,5•(p + P)

Reference: 2.3.2.2; Table 2.1 [2.3.3.1.(5)]

3. Internal forces and moments



M_{Sd} = (1,35•3,0 + 1,5•5,0)•7^2/8 + 1,5•140•7/4 = 438,2 kNm



$V_{1.Sd}$ = 1,5•140/2 = 105 kN
$V_{2.Sd}$ = R_{Sd} = 1,5•140/2 + (1,35•3,0 + 1,5•5,0)•7/2 = 145,4 kN

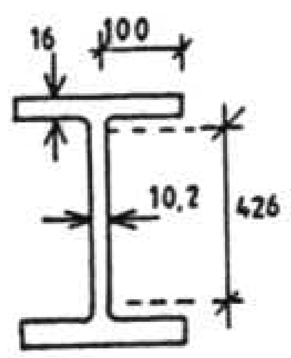
4. Determination of the resistance

4.1 Width-to-thickness ratios

Reference: 5.3.1; Table 5.10 [5.3; Table 5.3.1]

The classification of cross-sections could be made by appropriate dimension tables.

For rolled sections the shear buckling verification need not normally be considered.



Fe 430 ---→ ϵ = 0,92
Flange:
c/t = 100/16 = 6,3 < 11ϵ = 10,1
Web:
d/t = 426/10,2 = 41,8 < 83ϵ = 76,4
---→ A_{eff} = A ; W_{eff} = W_{pl}
d/t = 41,8 < 69ϵ = 63,5
---→ no shear buckling verification is required

Problem, Sketch, Calculation	Reference	Commentary

4.2 Shear resistance verification

4.2.1 At support

Reference: 5.3.3; [5.4.6]

Commentary: $A_v = h \cdot t_w = 500 \cdot 10{,}2 = 5100\ mm^2$
It is allowed to consider larger values by using the formulae given in EC 3 [5.4.6]

$$V_{Rd} = \frac{A_v\, f_y}{\sqrt{3}\, \gamma_{Mo}} = \frac{51{,}0 \cdot 10^2 \cdot 275}{\sqrt{3} \cdot 1{,}1} 10^{-3} = 736{,}1\ kN > 145{,}4\ kN$$

4.2.2 At mid-span

Reference: 5.3.4; [5.4.7]

$$0{,}5\ V_{Rd} = 0{,}5 \cdot \frac{A_v\, f_y}{\sqrt{3}\, \gamma_{Mo}} = 0{,}5 \cdot \frac{51{,}0 \cdot 10^2 \cdot 275}{\sqrt{3} \cdot 1{,}1} 10^{-3} = 368{,}1\ kN > 105{,}0\ kN$$

$\longrightarrow$ no interaction bending - shear is required

4.3 Member resistance verification

4.3.1 Determination of the critical bending moment M_{cr}

Reference: 5.4.3; [Annex F]

Commentary: Due to the side girder which provides a lateral support the buckling length is half the span.

Buckling length $\ell_{LT} = 3{,}50$ m

$k = k_w = 1{,}0$; $\psi = 0 \longrightarrow C_1 = 1{,}88$

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{\ell_{LT}^2} \sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + 0{,}039\, \ell_{LT}^2 \frac{I_t}{I_z}} = 1{,}88 \cdot \frac{\pi^2 \cdot 210000 \cdot 2140 \cdot 10^4}{3{,}50^2 \cdot 10^{12}} \sqrt{\frac{1249000 \cdot 10^6 + 0{,}039 \cdot 3{,}50^2 \cdot 10^6 \cdot 89{,}7 \cdot 10^4}{2140 \cdot 10^4}} = 1905{,}8\ kNm$$

4.3.2 Lateral torsional buckling verification

Reference: 5.4.3; [5.5.2]

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl}\, f_y}{M_{cr}}} = \sqrt{\frac{2194 \cdot 10^3 \cdot 275}{1905{,}8 \cdot} \cdot 10^{-6}} = 0{,}56 > 0{,}4$$

using buckling curve a $\longrightarrow \chi_{LT.a} = 0{,}9045$

$$M_{b.Rd} = \chi_{LT.a} \frac{W_{pl}\, f_y}{\gamma_{M1}} = 0{,}9045 \cdot \frac{2194 \cdot 10^3 \cdot 275}{1{,}1} \cdot 10^{-6} = 496{,}1\ kNm > 438{,}2\ kNm$$

5. Serviceability verification

Reference: 4.2.2; Table 4.1 [4.4.2; Table 4.1]

Relevant load combination: 1,0 g + 1,0 p + 1,0 P

Deflection due to dead loads:

$$\frac{5}{384} \frac{g \cdot \ell^4}{E\, I} = \frac{5}{384} \frac{3{,}0 \cdot 7{,}0^4}{210000 \cdot 48200 \cdot 10^4} \cdot 10^{12} = 0{,}93\ mm$$

Deflection due to imposed loads:

$$\delta_2 = \frac{5}{384} \frac{g \cdot \ell^4}{E\, I} + \frac{1}{48} \frac{P\, \ell^3}{E\, I} = \frac{5}{384} \frac{5{,}0 \cdot 7{,}0^4}{210000 \cdot 48200 \cdot 10^4} \cdot 10^{12} + \frac{1}{48} \frac{140 \cdot 7{,}0^3}{210000 \cdot 48200 \cdot 10^4} \cdot 10^{12}$$

$= 11{,}4\ mm = \ell/614 < \ell/300$

Maximum deflection:

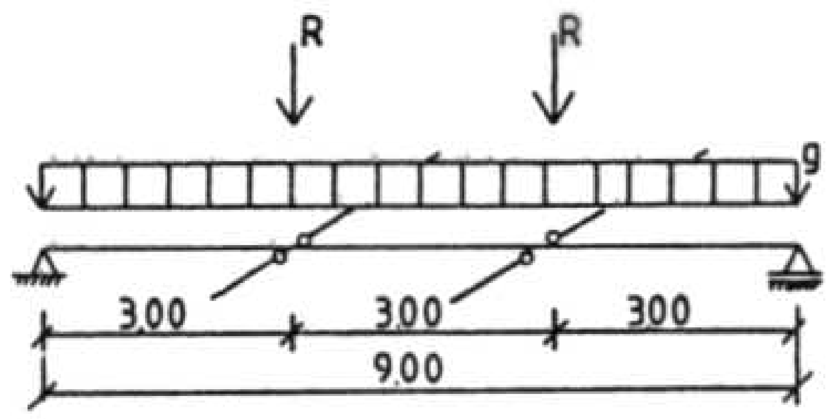
$\delta_{max} = 11{,}4 + 0{,}93 = 12{,}33\ mm = \ell/570 < \ell/250$

$\longrightarrow$ EC 3 recommendations are fulfilled

Problem, Sketch, Calculation	Reference	Commentary

Example 2.2.2: Single span beam with lateral torsional buckling

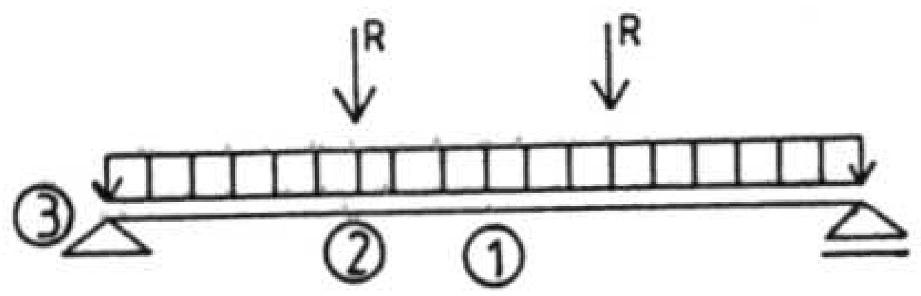
1. System



Main girder: HE 450 A
Steel grade: Fe 360
System length: ℓ = 9,00 m

Commentary: At the third points of the main girder side girders are connected by a nominally pinned connections. Lateral restraints are assumed at the supports. For determinig the cross-section properties I_w, I_t, I, W, A, A_v, i etc. dimension tables are used.

2. Loading

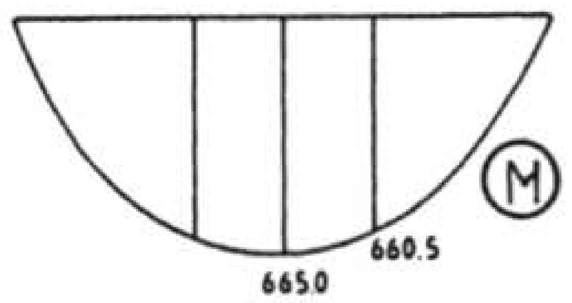


Dead load: g = 3,0 kN/m
Reaction force: R = 208 kN
Relevant load combination of this load case:
1,35•g + R

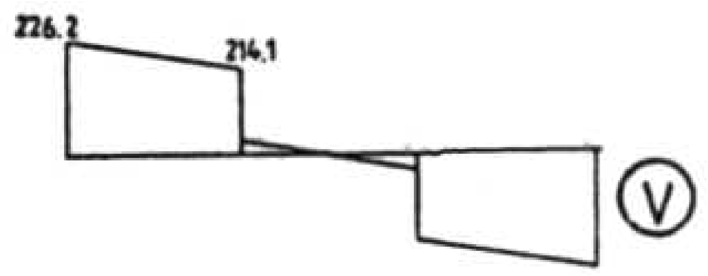
Reference: 2.3.2.2; Table 2.1 [2.3.3.1.(5)]

Commentary: Dead load includes the weight of floor, etc. R is the largest reaction force from the design of the side girders. The reaction force R includes the load factors.

3. Internal forces and moments



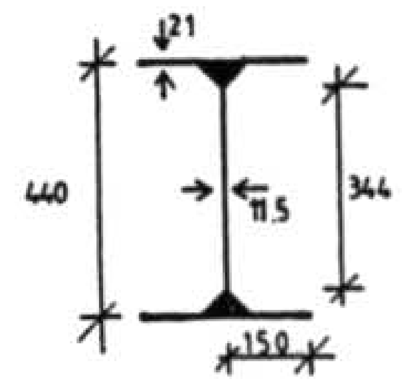
$M_{1.Sd}$ = 1,35•3,0•9^2/8 + 208•3,0 = 665,0 kNm
$M_{2.Sd}$ = 1,35•3,0•3^2 + 208•3,0 = 660,5 kNm



$V_{3.Sd}$ = $R_{3.Sd}$ = 1,35•3,0•9,0/2 + 208 = 226,2 kN
$V_{2.Sd}$ = - 1,35•3,0•3,0/6 + 226,2 = 214,1 kN

4. Determination of the resistance

4.1 Width-to-thickness ratios



Fe 360 ⟶ ϵ = 1,0
Flange:
c/t = 150/21 = 7,1 < 11ϵ = 11
Web:
d/t = 344/11,5 = 29,9 < 83ϵ = 83
⟶ A_{eff} = A ; W_{eff} = W_{pl}
d/t = 29,9 < 69ϵ = 69
⟶ no shear buckling verification is required

Reference: 5.3.1; Table 5.10 [5.3; Table 5.3.1]

Commentary: The classification of cross-sections could be made by appropriate dimension tables.

For rolled sections the shear buckling verification need not normally be considered.

Problem, Sketch, Calculation	Reference	Commentary

4.2 Shear resistance verification

4.2.1 At support

5.3.3; [5.4.6]

$A_v = h \cdot t_w = 440 \cdot 11{,}5 = 5060\ \text{mm}^2$
It is allowed to consider larger values by using the formulae given in EC 3 [5.4.6].

$$V_{Rd} = \frac{A_v\, f_y}{\sqrt{3}\,\gamma_{Mo}} = \frac{50{,}6 \cdot 10^2 \cdot 235}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 624{,}1\ \text{kN} > 226{,}2\ \text{kN}$$

4.2.2 At mid-span

5.3.4; [5.4.7]

$$0{,}5\ V_{Rd} = 0{,}5 \cdot \frac{A_v\, f_y}{\sqrt{3}\,\gamma_{Mo}} = 0{,}5 \cdot \frac{50{,}6 \cdot 10^2 \cdot 235}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 312{,}1\ \text{kN} > 214{,}1\ \text{kN} \longrightarrow \text{no interaction bending - shear is required}$$

4.3 Member resistance verification

4.3.1 Determination of the critical bending moment M_{cr}

5.4.3; [Annex F]

Because of lateral restraints the buckling length is limited to 3,0 m

Buckling length $\ell_{LT} = 3{,}0$ m

$k = k_w = 1{,}0$; $\psi \approx 1{,}0 \longrightarrow C_1 = 1{,}0$

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{\ell_{LT}^2} \sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + 0{,}039\, \ell_{LT}^2 \frac{I_t}{I_z}} = 1{,}0 \cdot \frac{\pi^2 \cdot 210000 \cdot 9470 \cdot 10^4}{3{,}0^2 \cdot 10^{12}} \sqrt{\frac{4146000 \cdot 10^6 + 0{,}039 \cdot 3{,}0^2 \cdot 10^6 \cdot 245 \cdot 10^4}{9470 \cdot 10^4}} = 5014{,}1\ \text{kNm}$$

4.3.2 Lateral torsional buckling verification

5.4.3; [5.5.2]

No lateral torsional buckling to be considered. Only cross-section verification is necessary, i.e. γ_{MO} is used

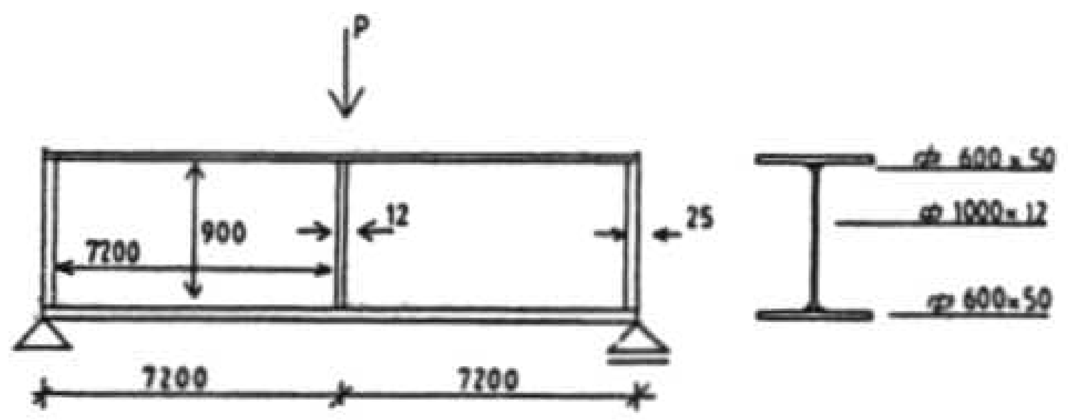
$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl}\, f_y}{M_{cr}}} = \sqrt{\frac{3216 \cdot 10^3 \cdot 235}{5014{,}1} \cdot 10^{-6}} = 0{,}39 < 0{,}4$$

$$M_{b.Rd} = \frac{W_{pl}\, f_y}{\gamma_{MO}} = \frac{3216 \cdot 10^3 \cdot 235}{1{,}1} \cdot 10^{-6} = 687{,}1\ \text{kNm} > 665{,}0\ \text{kNm}$$

Problem, Sketch, Calculation	Reference	Commentary

Example 2.2.3: **Single span girder with shear buckling verification**

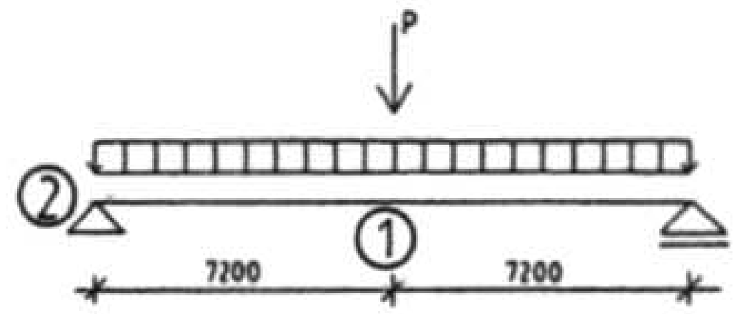
1. System



Beam: Welded section with stiffeners at the supports and at mid span
Dimensions see sketch
Steel grade: Fe 510
System length: ℓ = 14,40 m

Commentary: The beam is continuously laterally restrained.
Cross-section properties
$A = 708 \cdot 10^2$ mm²
$I = 1427900 \cdot 10^4$ mm⁴
throat thickness between flange and web:
a = 7 mm

2. Loading

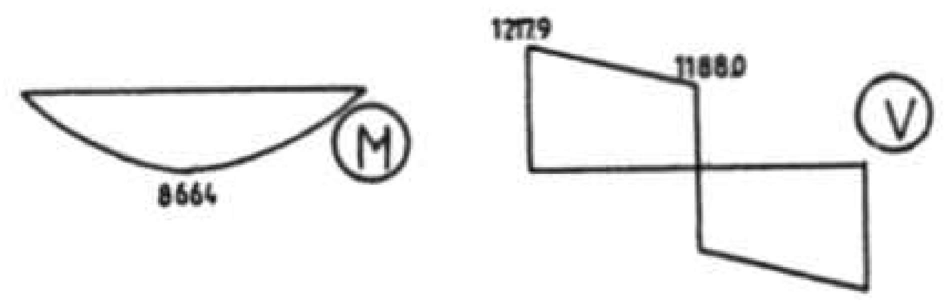


Dead load: g = 3,0 kN/m
Reaction force: P = 1585,0 kN
Relevant load combination of this load case:
$1{,}35 \cdot g + 1{,}5 \cdot P$

Reference: 2.3.2.2; Table 2.1 [2.3.3.1.(5)]

Commentary: Dead load includes the weight of floor, etc.

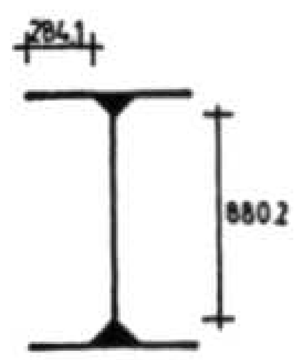
3. Internal forces and moments



$M_{1.Sd} = 1{,}35 \cdot 3{,}0 \cdot 14{,}4^2/8 + 1{,}5 \cdot 1585{,}0 \cdot 14{,}4/4 = 8664{,}0$ kNm

$V_{1.Sd} = 0{,}5 \cdot 1{,}5 \cdot 1585{,}0 = 1188{,}8$ kN
$V_{2.Sd} = R_{2.Sd} = 1{,}35 \cdot 3{,}0 \cdot 14{,}4/2 + 1{,}5 \cdot 1585{,}0/2 = 1217{,}9$ kN

4. Determination of the resistance
4.1 Width-to-thickness ratios



Flange: t_f = 50 mm > 40 mm $\longrightarrow$ f_y = 335 N/mm²
Fe 510 $\longrightarrow$ $\epsilon = \sqrt{(235/335)} = 0{,}84$
$c = 300 - 12/2 - \sqrt{2} \cdot 7 = 284{,}1$ mm
$c/t = 284{,}1/50 = 5{,}7 < 14\epsilon = 11{,}8$
Web:
Fe 510 $\longrightarrow$ $\epsilon = 0{,}81$
$d/t = 900/12 = 75 < 124\epsilon = 100{,}4$
$\longrightarrow$ $A_{eff} = A$; $W_{eff} = W_{el}$

Reference: - ; [5.3; Table 5.3.1]

Commentary: Note that where the plate thickness is greater than 40 mm the yield strength has to be reduced.
In the classification of welded cross-sections the welds are taken into account.

$$a/d = 7200/900 = 8{,}0 \longrightarrow k_\tau = 5{,}34 + \frac{4}{(a/d)^2} = 5{,}34 + \frac{4}{8^2} = 5{,}4$$

$d/t = 75 > 30\epsilon\sqrt{k_\tau} = 56{,}5$
$\longrightarrow$ shear buckling verification is required

Reference: Table 5.13; [5.4.6.(7)]

Problem, Sketch, Calculation	Reference	Commentary

4.2 Flange induced buckling

Reference: 5.3.5.(6); [5.7.7]

$$\frac{d}{t_w} = 73{,}4 < k\,\frac{E}{f_{yf}}\sqrt{\frac{A_w}{A_{fc}}} = 0{,}55\cdot\frac{210000}{335}\sqrt{\frac{900\cdot 12}{600\cdot 50}} = 206{,}9 > 75$$

Commentary: k = 0,55 for class 3 and class 4 section

4.3 Shear buckling verification of web

Reference: 5.4.5; [5.6.3]

$$\bar{\lambda}_w = \frac{d/t_w}{37{,}4\;\varepsilon_w\sqrt{k_\tau}} = \frac{75}{37{,}4\cdot 0{,}81\cdot\sqrt{5{,}4}} = 1{,}065$$

$$\longrightarrow \tau_{ba} = f_{yw}/\sqrt{3}\,(1{,}5 - 0{,}625\,\bar{\lambda}_w) = 35{,}5/\sqrt{3}\,(1{,}5 - 0{,}625\cdot 1{,}065)\cdot 10 = 171\ \text{N/mm}^2$$

$$V_{ba.Rd} = \frac{d\,t_w\,\tau_{ba}}{\gamma_{M1}} = \frac{900\cdot 12\cdot 171}{1{,}1}\cdot 10^{-3} = 1678{,}9\ \text{kN} > 1217{,}9\ \text{kN}$$

$1188{,}0 > 0{,}5V_{ba.Rd} = 839{,}5$ kN

$\longrightarrow$ Interaction bending - shear is required

Reference: [5.6.7.2.(1)]

Commentary: The cross-section may be assumed to be satisfactory, without investigating the effect of shear force on the design resistance moment when both checks are satisfied ($V_{Sd} \leq V_{Rd}$; $M_{Sd} \leq M_{f.Rd}$).
The simple post critical method is used for determining the shear buckling resistance.
$M_{f.Rd}$ The design moment resistance of a cross section consisting of the flanges only.

4.4 Determination of the design moment resistance consisting the flanges only

$M_{f.Rd} = A_f\,(h - t_f)\,f_{yf}/\gamma_{M0} = 600\cdot 50\cdot(1000 - 50)\cdot 335\cdot 10^{-6}/1{,}1 = 8679{,}5$ kNm $> 8664{,}0$ kNm

4.5 Bending moment resistance

Reference: 5.3.2; [5.4.5]

$W_{el} = 28558\cdot 10^3\ \text{mm}^3$

$M_{c.Rd} = W_{el} f_y/\gamma_{M0} = 28558\cdot 10^3\cdot 335\cdot 10^{-6}/1{,}1 = 8697{,}2$ kN $> 8664{,}0$ kNm

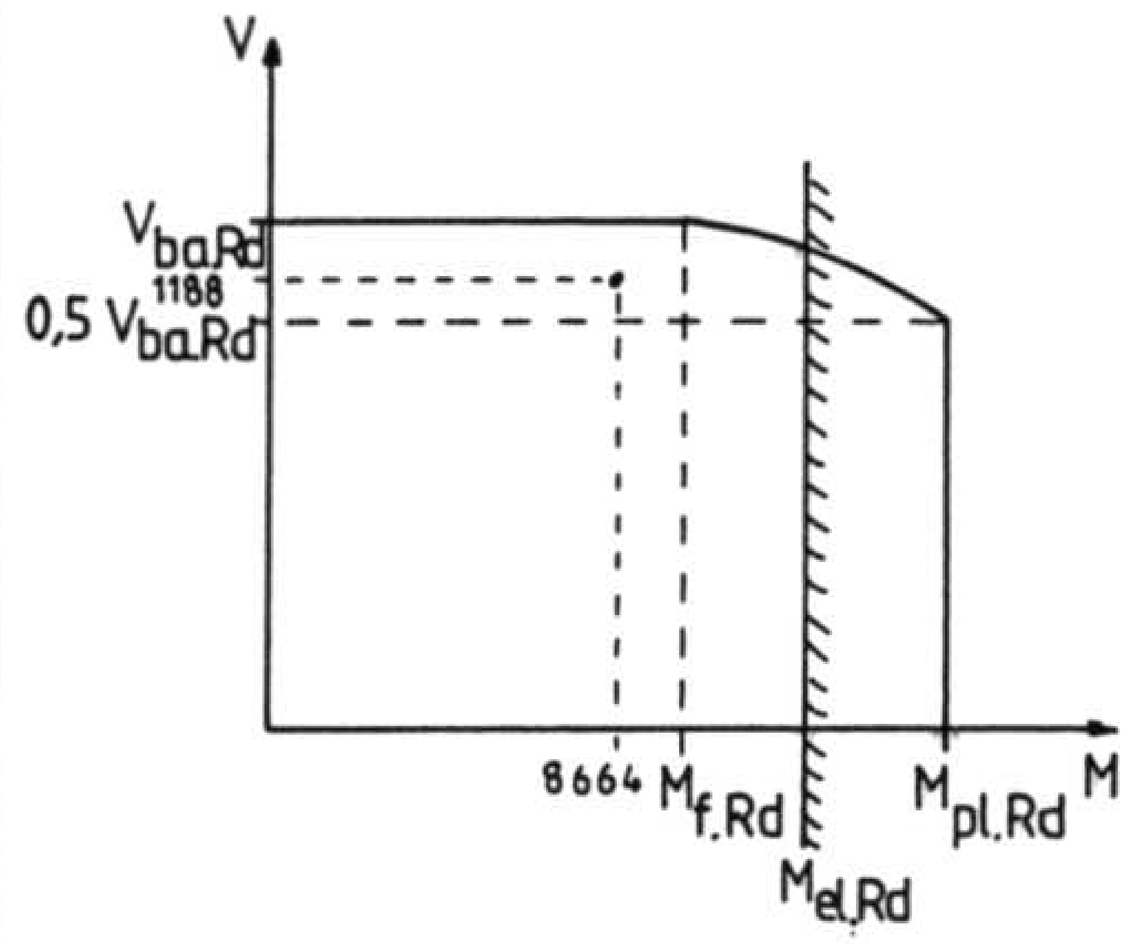
Commentary: $M_{w.Rd}$ the design moment resistance of a cross section consisting of the web only

4.6 Determination of the design plastic moment

Reference: 5.4.5.3 [5.6.7.2]

$M_{w.Rd} = d^2 t_w f_{yw}/4/1{,}1 = 900^2\cdot 12/4\cdot 355\cdot 10^{-6}/1{,}1 = 784{,}2$ kNm

$M_{pl.Rd} = (M_{f.Rd} + M_{w.Rd}) = 8679{,}5 + 784{,}2 = 9463{,}7$ kNm



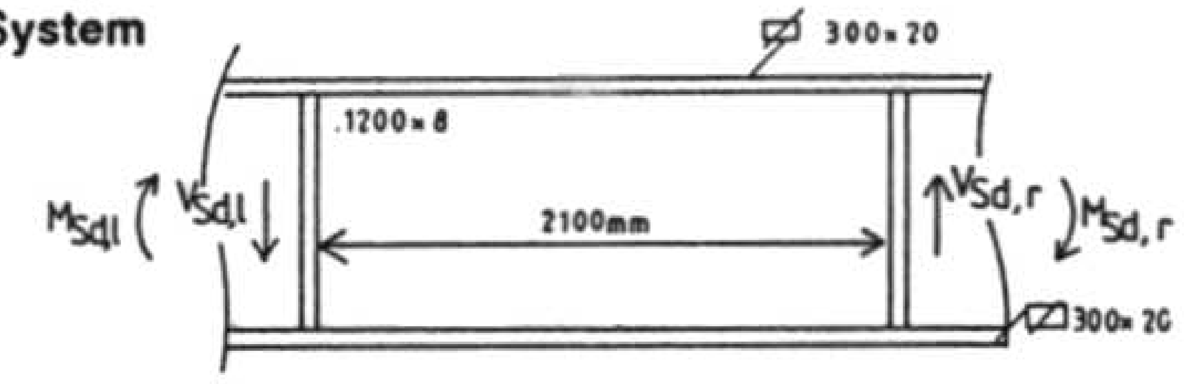
4.7 Interaction bending - shear buckling verification

$\longrightarrow$ not necessary because of $M_{Sd} < M_{f.Rd}$

Problem, Sketch, Calculation	Reference	Commentary

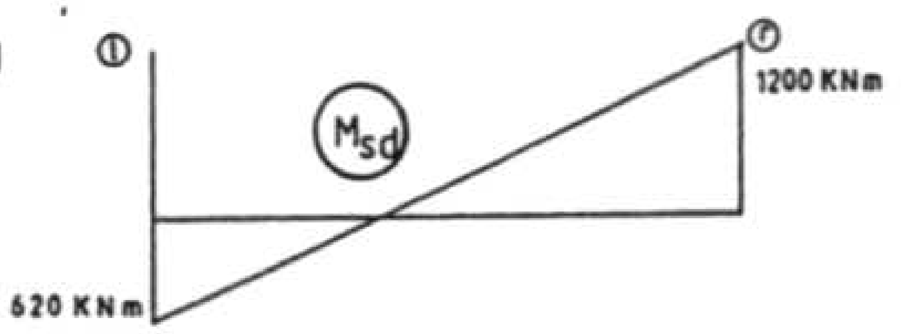
Example 2.2.4: Class 4 cross-section loaded in bending

1. System



Part of a Beam: Welded section
Dimensions see sketch
Steel grade: Fe 360

2. Loading



Bending moment distribution: see sketch
Shear force: V_{Sd} = 700,0 kN

3. Determination of the resistance

3.1 Width-to-thickness ratios

Fe 360 $\longrightarrow \epsilon = 1{,}0$

Flange:

$c = 150 - 8/2 - \sqrt{2} \cdot 5 = 138{,}9$ mm

$c/t = 138{,}9/20 = 6{,}9 < 14\epsilon = 14{,}0$

$\longrightarrow$ Flange is fully effective

Web:

$d/t = 1200/8 = 150{,}0 > 124\epsilon = 124{,}0$

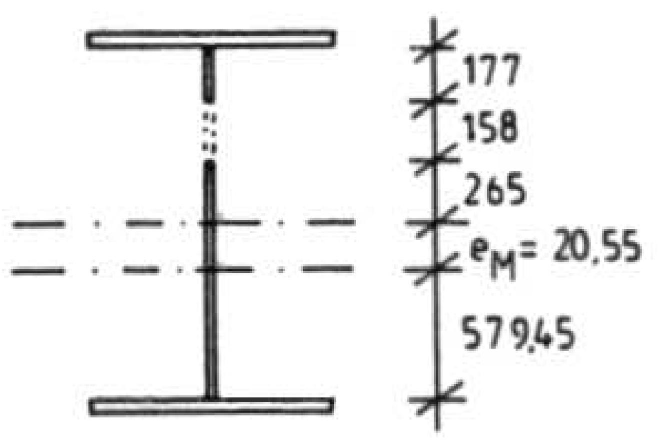
$\longrightarrow$ Determination of W_{eff}

$$a/d = 2100/1200 = 1{,}75 > 1{,}0 \longrightarrow k_\tau = 5{,}34 + \frac{4}{(a/d)^2} = 5{,}34 + \frac{4}{1{,}75^2} = 6{,}646$$

$d/t = 150{,}0 > 30\epsilon\sqrt{k_\tau} = 77{,}3 \longrightarrow$ shear buckling verification is required

3.2 Determination of W_{eff} of the web

Fe 360 $\longrightarrow 1/\epsilon = 1{,}0$



$$\bar{b} = d = 1200 \text{ mm}; \quad \bar{\lambda}_p = \frac{d/t}{28{,}4\,\epsilon\,\sqrt{k_\sigma}} = \frac{d}{t}\,\frac{1}{138{,}8\,\epsilon} = 150{,}0 \cdot \frac{1{,}0}{138{,}8} = 1{,}081 > 0{,}673$$

$$\rho = \frac{\bar{\lambda}_p - 0{,}22}{\bar{\lambda}_p^2} = \frac{1{,}081 - 0{,}22}{1{,}081^2} = 0{,}737$$

$d_{eff\,1} = 0{,}4 \cdot \rho \cdot d/2 = 0{,}4 \cdot 0{,}737 \cdot 1200/2 = 177$ mm

$d_{eff\,2} = 0{,}6 \cdot \rho \cdot d/2 = 0{,}6 \cdot 0{,}743 \cdot 1200/2 = 265$ mm

$A_{eff} = 203{,}9 \cdot 10^2$ mm^2 ; $e_M = 20{,}55$ mm

$I_{eff} = 547148{,}5 \cdot 10^4$ mm^4 ; $W_{eff} = 8542 \cdot 10^3$ mm^3

Reference

- ;
[5.3;
Table 5.3.1]

5.4.5.2; [5.6.3]

5.3.3; [5.4.6]

5.3.5; [5.3.5]

Commentary

Cross-section properties
$A = 216 \cdot 10^2$ mm^2
$I = 561760 \cdot 10^4$ mm^4
throat thickness between flange and web:
a = 5 mm

By the classification of welded cross-sections the welds are taken into account.

Flange is fully effective
$A_{eff} = (265 + 177 + 593 + 2\sqrt{2} \cdot 5) \cdot 8 + 300 \cdot 20 \cdot 2$
$= 203{,}9 \cdot 10^2$ mm^2

The effective width of the web is calculated on the basis of f_y and $\Psi = -1$.

Problem, Sketch, Calculation	Reference	Commentary
3.3 Shear buckling verification of web using "simple post-critical method"	5.4.5; [5.6]	Criterion is not fulfilled, therefore ⟶ use a thicker plate ⟶ or decrease the aspect ratio ⟶ or use a more precise verification

$$\bar{\lambda}_w = \frac{d/t_w}{37{,}4\ \varepsilon_w \sqrt{k_\tau}} = \frac{150}{37{,}4 \cdot \sqrt{6{,}646}} = 1{,}556 > 1{,}2$$

$\longrightarrow \tau_{ba} = f_{yw}/\surd 3 \cdot 0{,}9/\bar{\lambda}_w = 235/\surd 3 \cdot 0{,}9/1{,}556 = 78{,}5\ \mathrm{N/mm^2}$

$$V_{ba.Rd} = \frac{d\ t_w\ \tau_{ba}}{\gamma_{M1}} = \frac{1200 \cdot 8 \cdot 78{,}5}{1{,}1} \cdot 10^{-3} = 685{,}1\ \mathrm{kN} < 700{,}0\ \mathrm{kN}$$

⟶ Criterion is not satisfied

Problem, Sketch, Calculation	Reference	Commentary
3.4 Shear buckling verification using tension field method	- ; [5.6.4]	A more precise verification is presented
Requirement: $1 \leq a/d = 1{,}75 \leq 3$	- ; [5.6.2]	
3.4.1 Initial shear buckling strength	- ; [5.6.4.1.(2)]	

$\bar{\lambda}_w = 1{,}556 > 1{,}25 \longrightarrow \tau_{bb} = f_{yw}/\surd 3 \cdot 1/\bar{\lambda}_w^{\,2} = 235/\surd 3 \cdot 1/1{,}556^2 = 56{,}0\ \mathrm{N/mm^2}$

Problem, Sketch, Calculation	Reference	Commentary
3.4.2 Determination of the reduced plastic moment resistance $M_{Nf.Rk}$:	- ;[5.6.4.1.(4)]	Only the flanges carry the moments whilst the web carries the the shear force. In this example both flange hinges occur in the tension flange

On the right: $N_{r.ft.Sd} = M_{Sd}/(h - t_f) = 1200/(1240 - 20) \cdot 10^3 = 983{,}6\ \mathrm{kN}$

On the left: $N_{l.ft.Sd} = M_{Sd}/(h - t_f) = 620/(1240 - 20) \cdot 10^3 = 508{,}2\ \mathrm{kN}$

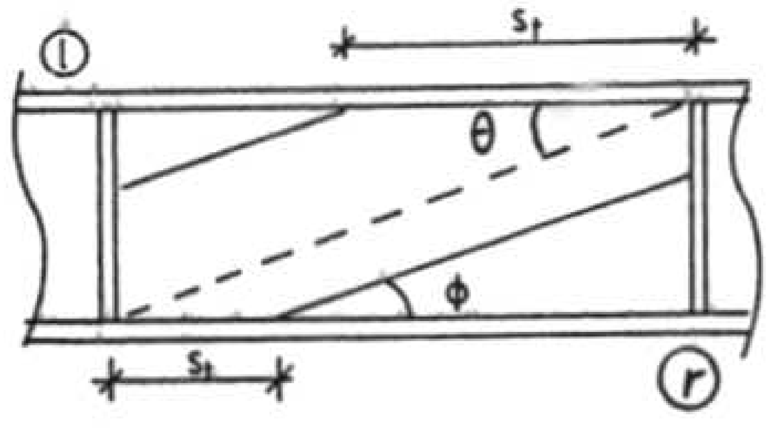
$$M_{r.ft.Rk} = 0{,}25 b t_f^2 f_{yf} \left[1 - \left(\frac{N_{r.ft.Sd}}{b t_f f_{yf}/\gamma_{Mo}}\right)^2\right] = 0{,}25 \cdot 300 \cdot 20^2 \cdot 235 \left[1 - \left(\frac{983{,}6 \cdot 10^3}{300 \cdot 20 \cdot 235/1{,}1}\right)^2\right] \cdot 10^{-6} = 2{,}90\ \mathrm{kNm}$$

$$M_{l.ft.Rk} = 0{,}25 b t_f^2 f_{yf} \left[1 - \left(\frac{N_{r.ft.Sd}}{b t_f f_{yf}/\gamma_{Mo}}\right)^2\right] = 0{,}25 \cdot 300 \cdot 20^2 \cdot 235 \left[1 - \left(\frac{508{,}2 \cdot 10^3}{300 \cdot 20 \cdot 235/1{,}1}\right)^2\right] \cdot 10^{-6} = 5{,}94\ \mathrm{kNm}$$

Problem, Sketch, Calculation	Reference	Commentary
3.4.3 Determination of the strength of the tension field		
$\Theta = \arctan d/a = \arctan 1200/2100 = 29{,}7^\circ$	- ; [5.6.4.2]	
conservative assumption: $\Phi \approx \Theta/1{,}5 = 19{,}8^\circ$		
$\psi = 1{,}5\tau_{bb} \sin(2\Phi) = 1{,}5 \cdot 56{,}0 \cdot \sin(2 \cdot 19{,}8) = 53{,}5\ \mathrm{N/mm^2}$	- ; [5.6.4.1.(1)]	

$$\sigma_{bb} = \sqrt{f_{yw}^2 - 3\tau_{bb}^2 + \psi^2} - \psi = \sqrt{235^2 - 3 \cdot 56{,}0^2 + 53{,}5^2} - 53{,}5 = 167{,}1\ \mathrm{N/mm^2}$$

Problem, Sketch, Calculation	Reference	Commentary
3.4.4 Anchorage lengths of the tension field	- ; [5.6.4.1.(3)]	



$$s_{r,t} = \frac{2}{\sin\Phi}\sqrt{\frac{M_{r.fc.Rk}}{t_w\,\sigma_{bb}}} = \frac{2}{\sin 19{,}8}\sqrt{\frac{2{,}90\cdot10^6}{8\cdot167{,}1}} = 275\ \text{mm}$$

$$s_{l,t} = \frac{2}{\sin\Phi}\sqrt{\frac{M_{l.fc.Rk}}{t_w\,\sigma_{bb}}} = \frac{2}{\sin 19{,}8}\sqrt{\frac{5{,}94\cdot10^6}{8\cdot167{,}1}} = 393{,}6\ \text{mm}$$

3.4.5 Width of the tension field — - ; [5.6.4.1.(3)]

$g = d\cdot\cos\Phi - (a - s_{r,t} - s_{l,t})\cdot\sin\Phi = 1200\cdot\cos 19{,}8 - (2100 - 275 - 393{,}6)\cdot\sin 19{,}8 = 644{,}2$ mm

3.4.6 Shear buckling resistance — - ; [5.6.4.1.(1)]

$$V_{bb.Rd} = \frac{d\,t_w\,\tau_{bb} + 0{,}9\,g\,t_w\,\sigma_{bb}\,\sin\Phi}{\gamma_{M1}} = \frac{1200\cdot8\cdot56{,}0+0{,}9\cdot644{,}2\cdot8\cdot167{,}1\cdot\sin19{,}8}{1{,}1}\cdot10^{-3} = 727{,}4\ \text{kN} > 700\ \text{kN}$$

$700 > 0{,}5V_{bb.Rd} = 359{,}8$ kN

⟶ interaction bending - shear is required

3.4.7 Determination of the design resistance moment considering the flanges only — - ; [5.6.7.3.(1)]

$M_{f.Rd} = A_f\,(d + t_f)\,f_{yf}/\gamma_{Mo} = 300\cdot20\cdot(1200 + 20)\cdot235\cdot10^{-6}/1{,}1 = 1563{,}8$ kNm > 1200,0 kNm

3.4.8 Interaction bending - shear buckling — - ; [5.6.7.3.(1)]

⟶ not necessary because of $M_{Sd} < M_{f.Rd}$

2.3 Combined loading - Bending and compression

These examples demonstrate the verification of members loaded by the combination of bending and compression assuming design values of action effects (N_{Sd}, V_{Sd}, M_{Sd}, etc) which have been calculated by an analysis of the structure and these values already include ψ, γ_F etc. The second order effects are considered by using first order elastic analysis with sway-mode buckling lengths. The results presented in the examples are rounded values. For the purpose of easy re-calculation each formula and each check is calculated with the rounded values.

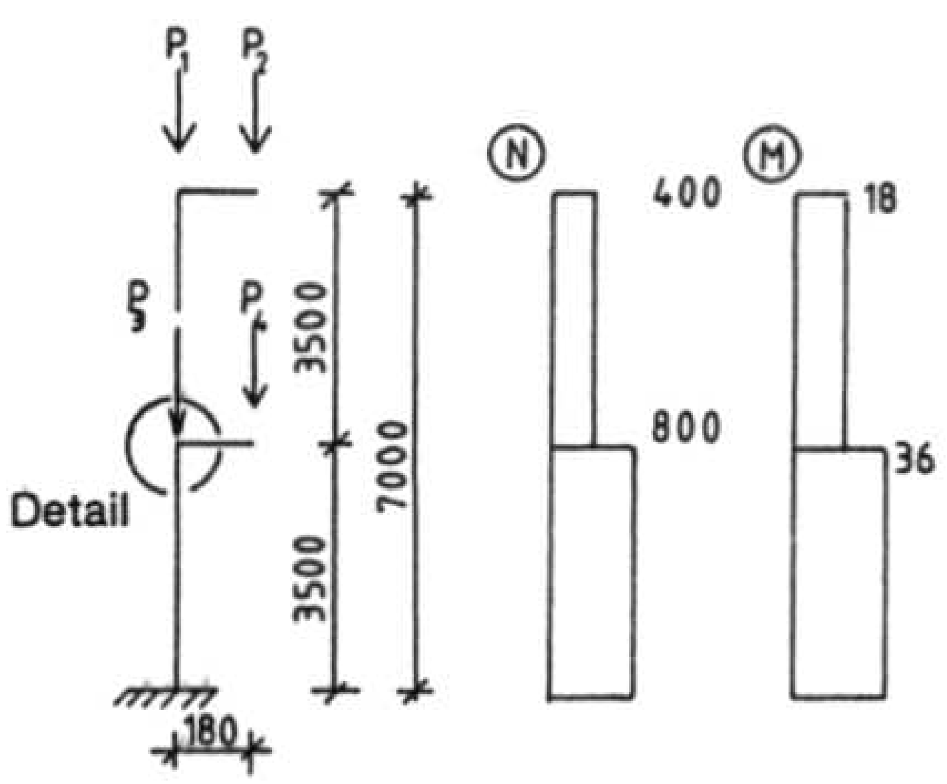
The following examples are included in this chapter:

Example 2.3.1:	RHS column loaded in bending and axial compression
Example 2.3.2:	HEA profile loaded in bending and axial compression
Example 2.3.3:	Column of a frame
Example 2.3.4:	Rafter of a frame

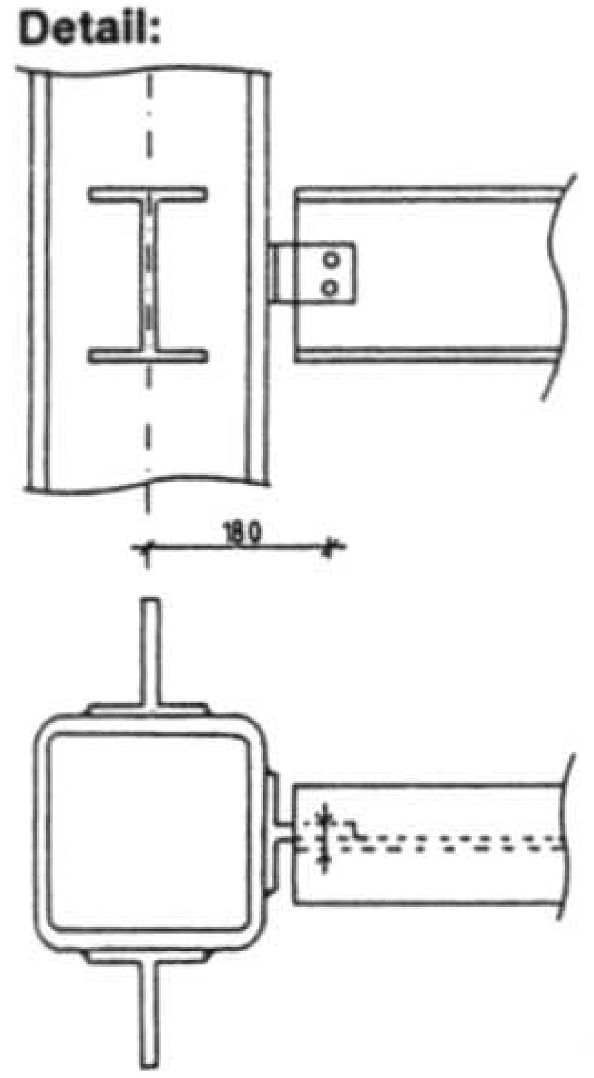
Problem, Sketch, Calculation	Reference	Commentary

Example 2.3.1: RHS profile loaded in bending and axial compression

1. System



Detail:



Column: RHS 250 x 10,0, hot rolled
Steel grade: Fe 430
System length: $L_1 = L_2 = 3{,}50$ m
Buckling lengths: $\ell_1 = 10{,}85$ m
$\ell_2 = 15{,}34$ m

Commentary: No restraints at the load introduction points. For determinig the cross-section properties I_w, I_t, I, W, A, A_v, i etc. dimension tables are used. The buckling lengths are determined by a separate calculation using the literature

2. Loading

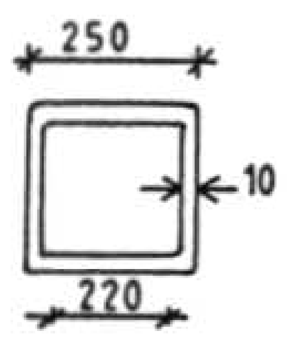
Axial compression and moment distribution see sketch
$P_1 = P_3 = 300$ kN; $P_2 = P_4 = 100$ kN

Commentary: Load factors are included. Global imperfections are included in the global analysis to determine the applied loads.

3. Determination of the resistance

3.1 Width-to-thickness ratios

Reference: 5.3.1; Table 5.10 [5.3; Table 5.3.1]



Fe 430 $\longrightarrow \epsilon = 0{,}92$
Flange:
$d = b\text{-}3t = 250 - 3\cdot10 = 220$ mm
$d/t = 220/10 = 22 < 38\epsilon = 35$
Web:
$d/t = 220/10 = 22 < 83\epsilon = 76{,}4$
$\longrightarrow A_{eff} = A$; $W_{eff} = W_{pl}$
$d/t = 22 < 69\epsilon = 63{,}5$
$\longrightarrow$ no shear buckling verification is required

Reference: 5.3.3; [5.4.6]

Commentary: The classification of cross-sections could be made by appropriate dimension tables.

Firstly a stress distribution of pure bending in the web is considered. When the real stress distribution is known a final check has to be made. This final check is not made.

For rolled sections the shear buckling verification need normally not to be considered.

3.2 Member resistance verification

3.2.1 Axial compression resistance

Reference: 5.4.2; [5.5.1]

Part 1: Buckling length: $\ell = 10{,}85$ m

$$\bar{\lambda} = \frac{\ell}{i}\,\frac{1}{\lambda_1} = \frac{10{,}85\cdot10^3}{96{,}7}\cdot\frac{1}{86{,}8} = 1{,}29$$

using buckling curve a $\longrightarrow \chi_a = 0{,}4760$

$$N_{b.Rd} = \chi_a\cdot\frac{A\cdot f_y}{\gamma_{M1}} = 0{,}4760\cdot\frac{91{,}7\cdot10^2\cdot275}{1{,}1}\cdot10^{-3} = 1091{,}2\text{ kN} > 800{,}0\text{kN}$$

Commentary: The buckling check is made for the two individual segments (Part 1 and Part 2) by using first order analysis with sway-mode buckling lengths. The global column buckling mode is included in the check of the first segment (Part 1).

Problem, Sketch, Calculation	Reference	Commentary

Part 2: Buckling length: $\ell = 15{,}34$ m — Reference: 5.4.2; [5.5.1]

$$\bar{\lambda} = \frac{\ell}{i}\,\frac{1}{\lambda_1} = \frac{15{,}34 \cdot 10^3}{96{,}7} \cdot \frac{1}{86{,}8} = 1{,}83$$

using buckling curve a $\longrightarrow \chi_a = 0{,}2623$

$$N_{b.Rd} = \chi_a \cdot \frac{A \cdot f_y}{\gamma_{M1}} = 0{,}2623 \cdot \frac{91{,}7 \cdot 10^2 \cdot 275}{1{,}1} \cdot 10^{-3} = 601{,}3 \text{ kN} > 400{,}0 \text{kN}$$

3.2.2 Bending moment resistance — Reference: 5.3.2; [5.4.5]

Commentary: RHS $\longrightarrow$ No lateral torsional buckling verification is required.

$$M_{y.Rd} = \frac{W_{pl}\, f_y}{\gamma_{Mo}} = \frac{832{,}8 \cdot 10^3 \cdot 275}{1{,}1} \cdot 10^{-6} = 208{,}2 \text{ kNm} > 40{,}0 \text{ kNm}$$

3.2.3 Interaction Bending - axial compression without lateral torsional buckling phenomena — Reference: 5.4.4; Table 5.26 [5.5.4]

Part 1:

$n = N_{Sd}/N_{b.Rd} = 800{,}0/1091{,}2 = 0{,}73$

$\Psi = 1{,}0 \longrightarrow \beta_M = 1{,}1$

$\alpha = W_{pl}/W_{el} - 1 = 832{,}8 \cdot 10^3/707{,}3 \cdot 10^3 - 1 = 0{,}177$

$\mu = \lambda_1(2\beta_M - 4) + \alpha = 1{,}21\,(2 \cdot 1{,}1 - 4) + 0{,}177 = -2{,}001 < 0{,}9$

$1 - \mu n/\gamma_{M1} = 1 + 2{,}001 \cdot 0{,}73/1{,}1 = 2{,}33 > 1{,}5 \longrightarrow k = 1{,}5$

$$n + \frac{1{,}5\, M_{Sd}}{W_{pl}\,\frac{f_y}{\gamma_{M1}}} = 0{,}73 + \frac{1{,}50 \cdot 36{,}0}{832{,}8 \cdot 10^3 \cdot \frac{275}{1{,}1} \cdot 10^{-6}} = 0{,}99 < 1$$

Part 2: — Reference: 5.4.4; Table 5.26 [5.5.4]

$n = N_{Sd}/N_{b.Rd} = 400{,}0/601{,}3 = 0{,}67$

$\Psi = 1{,}0 \longrightarrow \beta_M = 1{,}1$

$\alpha = W_{pl}/W_{el} - 1 = 832{,}8 \cdot 10^3/707{,}3 \cdot 10^3 - 1 = 0{,}177$

$\mu = \lambda_2(2\beta_M - 4) + \alpha = 1{,}92\,(2 \cdot 1{,}1 - 4) + 0{,}177 = -3{,}279 < 0{,}9$

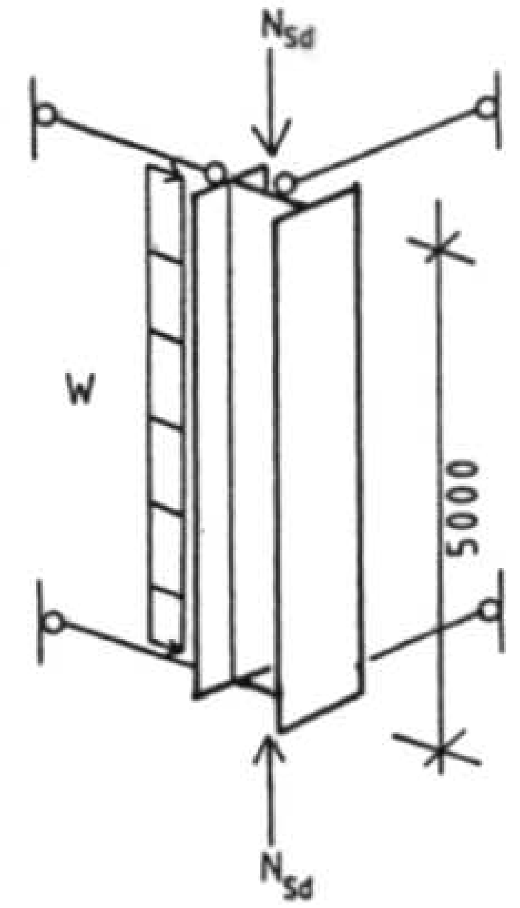
$1 - \mu n/\gamma_{M1} = 1 + 3{,}279 \cdot 0{,}67/1{,}1 = 3{,}0 > 1{,}5 \longrightarrow k = 1{,}5$

$$n + \frac{1{,}5\, M_{Sd}}{W_{pl}\,\frac{f_y}{\gamma_{M1}}} = 0{,}67 + \frac{1{,}50 \cdot 18{,}0}{832{,}8 \cdot 10^3 \cdot \frac{275}{1{,}1} \cdot 10^{-6}} = 0{,}80 < 1$$

Problem, Sketch, Calculation	Reference	Commentary

Example 2.3.2: HEA profile loaded in bending and axial compression

1. System



Column: HE 140 A
Steel grade: Fe 360
System length: L = 5,00 m
Buckling length: ℓ_y = 5,00 m
ℓ_z = 5,00 m

Commentary: For determinig the cross-section properties I_w, I_t, I, W, A, A_v, i etc. dimension tables are used.

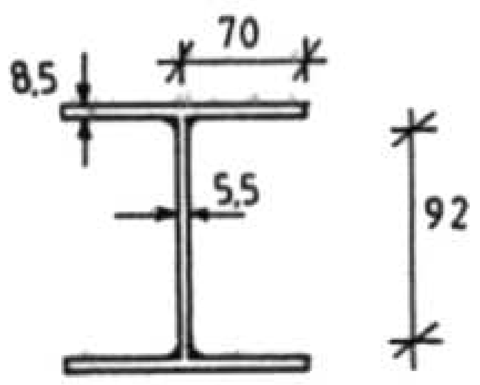
2. Loading

Axial compression:	N_{Sd}	= 120 kN
Uniform distributed load:	w	= 2,67 kN/m
Moment at mid-span:	M_{Sd}	$= 1{,}5 \cdot 2{,}67 \cdot 5^2/8$
		= 12,5 kNm

3. Determination of the resistance

3.1 Width-to-thickness ratios

Reference: 5.3.1; Table 5.10 [5.3; Table 5.3.1]



Fe 360 $\longrightarrow \epsilon = 1{,}00$

Flange:

$c/t = 70/8{,}5 = 8{,}2 < 11\epsilon = 11$

Web:

$d/t = 92/5{,}5 = 16{,}7 < 83\epsilon = 83$

$\longrightarrow A_{eff} = A\ ;\ W_{eff} = W_{pl}$

$d/t = 16{,}7 < 69\epsilon = 69$

$\longrightarrow$ no shear buckling verification is required

Reference: 5.3.3; [5.4.6]

Commentary: The classification of cross-sections could be made by appropriate dimension tables.

Firstly a stress distribution on pure bending in the web is considered. When the real stress distribution is known a final check has to be made. The final check is not made.

For rolled sections the shear buckling verification need normally not to be considered.

3.2 Member resistance verification

3.2.1 Axial compression resistance

Reference: 5.4.2; [5.5.1]

Strong axis: Buckling length: ℓ_y = 5,00 m

$$\bar{\lambda}_y = \frac{\ell_y}{i_y}\,\frac{1}{\lambda_1} = \frac{5{,}00\cdot 10^3}{57{,}3}\cdot\frac{1}{93{,}9} = 0{,}93$$

using buckling curve b $\longrightarrow \chi_b = 0{,}6419$

$$N_{y.b.Rd} = \chi_b \cdot \frac{A \cdot f_y}{\gamma_{M1}} = 0{,}6419 \cdot \frac{31{,}4\cdot 10^2\cdot 235}{1{,}1}\cdot 10^{-3} = 430{,}6\ \text{kN} > 120{,}0\text{kN}$$

Problem, Sketch, Calculation	Reference	Commentary

Weak axis: Buckling length: $\ell_z = 5{,}00$ m

5.4.2; [5.5.1]

$$\bar{\lambda}_z = \frac{\ell_z}{i_z}\,\frac{1}{\lambda_1} = \frac{5{,}00\cdot10^3}{35{,}2}\cdot\frac{1}{93{,}9} = 1{,}51$$

using buckling curve c $\longrightarrow \chi_c = 0{,}3113$

$$N_{z.b.Rd} = \chi_c\cdot\frac{A\cdot f_y}{\gamma_{M1}} = 0{,}3113\cdot\frac{31{,}4\cdot10^2\cdot235}{1{,}1}\cdot10^{-3} = 208{,}8\text{ kN} > 120{,}0\text{kN}$$

3.2.2 Bending moment resistance

3.2.2.1 Determination of the critical bending moment M_{cr}

5.4.3;
[Annex F; 5.5.2]

Buckling length $\ell_{LT} = 5{,}0$ m

$k = k_w = 1{,}0$;

Moment distribution

$\longrightarrow$ $C_1 = 1{,}132$

$$M_{cr} = C_1\,\frac{\pi^2 E I_z}{\ell_{LT}^2}\sqrt{\left(\frac{k}{k_w}\right)^2\frac{I_w}{I_z} + 0{,}039\,\ell_{LT}^2\,\frac{I_t}{I_z}} = 1{,}132\cdot\frac{\pi^2\cdot210000\cdot389\cdot10^4}{5{,}0^2\cdot10^{12}}\sqrt{\frac{15060\cdot10^8 + 0{,}039\cdot5{,}0^2\cdot10^6\cdot8{,}16\cdot10^4}{389\cdot10^4}} = 56{,}9\text{ kNm}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl}\,f_y}{M_{cr}}} = \sqrt{\frac{173\cdot10^3\cdot235}{56{,}9\cdot10^6}} = 0{,}85 > 0{,}4$$

using buckling curve a $\longrightarrow \chi_{LT.a} = 0{,}7659$

3.2.2.2 Bending moment resistance

$$M_{b.Rd} = \chi_{LT}\,\frac{W_{pl}\,f_y}{\gamma_{M1}} = 0{,}7659\cdot\frac{173{,}0\cdot10^3\cdot235}{1{,}1}\cdot10^{-6} = 28{,}3\text{ kNm} > 12{,}5\text{ kNm}$$

3.2.3 Interaction Bending - axial compression without lateral torsional buckling phenomena

5.4.4; Table 5.26
[5.5.4]

For mono-axial bending one should take the relevant buckling curve for buckling in the moment plane.

$n_y = N_{Sd}/N_{y.b.Rd} = 120{,}0/430{,}6 = 0{,}28$

$\beta_M = 1{,}3$

$\alpha = W_{pl}/W_{el} - 1 = 173{,}0\cdot10^3/155{,}0\cdot10^3 - 1 = 0{,}116$

$\mu_y = \bar{\lambda}_y(2\beta_M - 4) + \alpha = 0{,}93\,(2\cdot1{,}3 - 4) + 0{,}116 = -1{,}186 < 0{,}9$

$1 - \mu_y n_y/\gamma_{M1} = 1 + 1{,}186\cdot0{,}28/1{,}1 = 1{,}30 < 1{,}5 \longrightarrow k_y = 1{,}30$

$$n_y + \frac{k_y\,M_{y.Sd}}{W_{pl.y}\,\frac{f_y}{\gamma_{M1}}} = 0{,}28 + \frac{1{,}30\cdot12{,}5\cdot10^6}{173\cdot10^3\cdot\frac{235}{1{,}1}} = 0{,}72 < 1$$

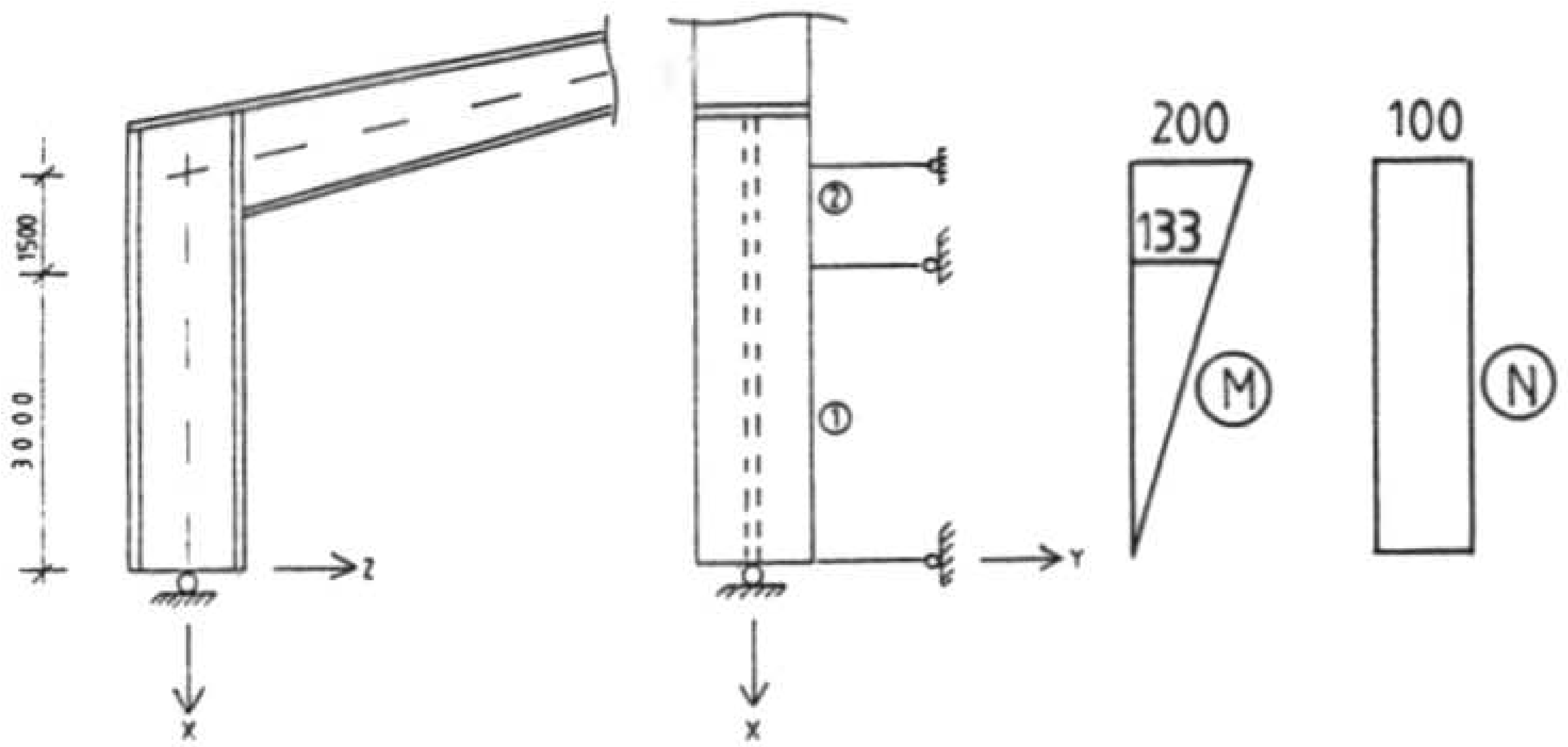
Problem, Sketch, Calculation	Reference	Commentary
3.2.4 Interaction Bending - axial compression with lateral torsional buckling phenomena $n_z = N_{Sd}/N_{z.b.Rd} = 120{,}0/208{,}8 = 0{,}575$ $\beta_{M.LT} = 1{,}3$ $\mu_{LT} = 0{,}15\lambda_z\beta_{M.LT} - 0{,}15 = 0{,}15 \bullet 1{,}51 \bullet 1{,}3 - 0{,}15 = 0{,}144 < 0{,}9$ $1 - \mu_{LT} n_z/\gamma_{M1} = 1 - 0{,}144 \bullet 0{,}575/1{,}1 = 0{,}925 < 1{,}0 \longrightarrow k_{LT} = 0{,}925$ $n_z + \frac{k_{LT}\, M_{y.Sd}}{M_{b.Rd}} = 0{,}575 + \frac{0{,}925 \cdot 12{,}5}{28{,}3} = 0{,}98 < 1$	5.4.4; Table 5.26 [5.5.4]	

Problem, Sketch, Calculation	Reference	Commentary

Example 2.3.3: **Column of a frame**

1. System

For determining the cross-section properties I_w, I_t, I, W, A, A_v, i etc. dimension tables are used.



Column: IPE 360 A
- Steel grade: Fe 510
- System length: L = 4,50 m
- Buckling length: ℓ_z = 18,00 m
  - $\ell_{1,y}$ = 3,00 m
  - $\ell_{2,y}$ = 1,50 m

2. Loading

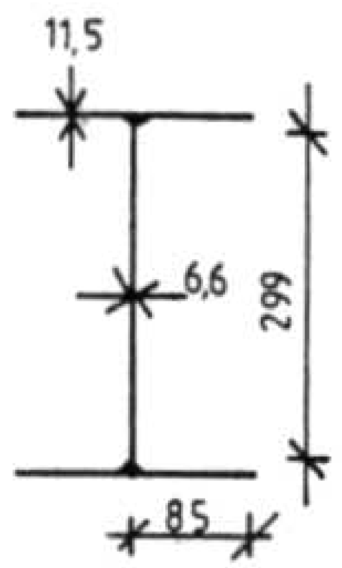
Axial compression: N_{Sd} = 100 kN
Moment distribution see sketch

3. Determination of the resistance

3.1 Width-to-thickness ratios

Reference: 5.3.1; Table 5.10 [5.3; Table 5.3.1]

Commentary: The classification of cross-sections could be made by appropriate dimension tables.



Fe 510 ⟶ ϵ = 0,81

Flange:

$c/t = 85/11{,}5 = 7{,}4 < 11\epsilon = 8{,}9$

Web:

$d/t = 299/6{,}6 = 45{,}3 < 83\epsilon = 67{,}2$

⟶ $A_{eff} = A$; $W_{eff} = W_{pl}$

$d/t = 45{,}3 < 69\epsilon = 55{,}9$

⟶ no shear buckling verification is required

Reference: 5.3.3; [5.4.6]

Check of the b/t ratios after knowing the real stress distribution:

$\sigma_M = 200 \cdot 100/812 = 246\ N/mm^2$; $\sigma_N = 100 \cdot 10/64 = 16\ N/mm^2$

$\alpha = 1/[1 + (246 - 16)/(246 + 16)] = 0{,}533$

$d/t = 45{,}3 < 456\epsilon/(13\alpha - 1) = 62{,}4$; ⟶ $A_{eff} = A$; $W_{eff} = W_{pl}$

Commentary: Firstly a stress distribution due to bending in the web is only considered. When the real stress distribution is known a final check has to be made. The final check is not made.

3.2 Member resistance verification

3.2.1 Axial compression resistance

Reference: 5.4.2; [5.5.1]

Commentary: First order elastic analysis with sway-mode buckling length is used.

Strong axis: Buckling length: ℓ_y = 18,00 m

$$\bar{\lambda}_y = \frac{\ell_y}{i_y}\frac{1}{\lambda_1} = \frac{18{,}00 \cdot 10^3}{151} \cdot \frac{1}{76{,}4} = 1{,}56$$

using buckling curve a ⟶ χ_a = 0,3482

$$N_{y.b.Rd} = \chi_a \cdot \frac{A \cdot f_y}{\gamma_{M1}} = 0{,}3482 \cdot \frac{64{,}0 \cdot 10^2 \cdot 355}{1{,}1} \cdot 10^{-3} = 719{,}2\ kN > 100{,}0 kN$$

Weak axis: (Reference: 5.4.2; [5.5.1])

Section 1: Buckling length: $\ell_{1.z} = 3{,}00$ m

$$\bar{\lambda}_{1.z} = \frac{\ell_{1.z}}{i_z}\,\frac{1}{\lambda_1} = \frac{3{,}00\cdot 10^3}{38{,}4}\cdot\frac{1}{76{,}4} = 1{,}02$$

using buckling curve b $\longrightarrow$ $\chi_b = 0{,}5844$

$$N_{1z.b.Rd} = \chi_b\cdot\frac{A\cdot f_y}{\gamma_{M1}} = 0{,}5844\cdot\frac{64{,}0\cdot 10^2\cdot 355}{1{,}1}\cdot 10^{-3} = 1207{,}1\ \text{kN} > 100{,}0\text{kN}$$

Section 2: Buckling length: $\ell_{2.z} = 1{,}50$ m

$$\bar{\lambda}_{2.z} = \frac{\ell_{2.z}}{i_z}\,\frac{1}{\lambda_1} = \frac{1{,}50\cdot 10^3}{38{,}4}\cdot\frac{1}{76{,}4} = 0{,}51$$

using buckling curve b $\longrightarrow$ $\chi_b = 0{,}8798$

$$N_{2z.b.Rd} = \chi_b\cdot\frac{A\cdot f_y}{\gamma_{M1}} = 0{,}8798\cdot\frac{64{,}0\cdot 10^2\cdot 355}{1{,}1}\cdot 10^{-3} = 1817{,}2\ \text{kN} > 100{,}0\text{kN}$$

3.2.2 Bending moment resistance (Reference: 5.4.3; [5.5.2])

3.2.2.1 Section 1

Buckling length $\ell_{LT} = 3{,}0$ m

$k = k_w = 1{,}0$; $\Psi = 0 \longrightarrow C_1 = 1{,}879$

$$M_{cr} = C_1\,\frac{\pi^2 E I_z}{\ell_{LT}^2}\sqrt{\left(\frac{k}{k_w}\right)^2\frac{I_w}{I_z} + 0{,}039\,\ell_{LT}^2\,\frac{I_t}{I_z}} = 1{,}879\cdot\frac{\pi^2\cdot 210000\cdot 944\cdot 10^4}{3{,}0^2\cdot 10^{12}}\sqrt{\frac{282000\cdot 10^6 + 0{,}039\cdot 3{,}0^2\cdot 10^6\cdot 26{,}5\cdot 10^4}{944\cdot 10^4}} = 814{,}2\ \text{kNm}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl}\,f_y}{M_{cr}}} = \sqrt{\frac{907\cdot 10^3\cdot 355}{814{,}2\cdot 10^6}} = 0{,}63 > 0{,}4$$

using buckling curve a $\longrightarrow$ $\chi_{LT.a} = 0{,}8783$

$$M_{1.b.Rd} = \chi_{LT.a}\,\frac{W_{pl}\,f_y}{\gamma_{M1}} = 0{,}8783\cdot\frac{907{,}0\cdot 10^3\cdot 355}{1{,}1}\cdot 10^{-6} = 257{,}1\ \text{kNm} > 133{,}3\ \text{kNm}$$

Problem, Sketch, Calculation	Reference	Commentary

3.2.2.2 Section 2 — Reference: 5.4.3; [5.5.2]

Buckling length $\ell_{LT} = 1,5$ m

$k = k_w = 1,0$; $\Psi = 133/200 \longrightarrow C_1 = 1,199$

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{\ell_{LT}^2} \sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + 0,039\, \ell_{LT}^2 \frac{I_t}{I_z}} = 1,199 \cdot \frac{\pi^2 \cdot 210000 \cdot 944 \cdot 10^4}{1,5^2 \cdot 10^{12}} \sqrt{\frac{282000 \cdot 10^6 + 0,039 \cdot 1,5^2 \cdot 10^6 \cdot 26,5 \cdot 10^4}{944 \cdot 10^4}} = 1874,9 \text{ kNm}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl} f_y}{M_{cr}}} = \sqrt{\frac{907 \cdot 10^3 \cdot 355}{1874,9 \cdot 10^6}} = 0,41 > 0,4 \quad \text{using buckling curve a} \longrightarrow \chi_{LT.a} = 0,9501$$

$$M_{2.b.Rd} = \chi_{LT.a} \frac{W_{pl} f_y}{\gamma_{M1}} = 0,9501 \cdot \frac{907,0 \cdot 10^3 \cdot 355}{1,1} \cdot 10^{-6} = 278,1 \text{ kNm} > 200 \text{ kNm}$$

3.2.3 Interaction Bending - axial compression without lateral torsional buckling phenomena — Reference: 5.4.4; Table 5.26 [5.5.4]

$n_y = N_{Sd}/N_{y.Rd} = 100,0/719,2 = 0,14$

$\Psi = 0 \longrightarrow \beta_{M.y} = 1,8$

$\alpha = W_{pl}/W_{el} - 1 = 907 \cdot 10^3/812 \cdot 10^3 - 1 = 0,12$

$\mu_y = \lambda_y(2\beta_{M.y} - 4) + \alpha = 1,56\,(2 \cdot 1,8 - 4) + 0,12 = -0,5 < 0,9$

$1 - \mu_y n_y/\gamma_{M1} = 1 + 0,5 \cdot 0,14/1,1 = 1,06 < 1,5 \longrightarrow k_y = 1,06$

$$n_y + \frac{k_y M_{y.Sd}}{W_{pl.y} \frac{f_y}{\gamma_{M1}}} = 0,14 + \frac{1,06 \cdot 200,0 \cdot 10^6}{907 \cdot 10^3 \cdot \frac{355}{1,1}} = 0,86 < 1$$

3.2.4 Interaction Bending - axial compression with lateral torsional buckling phenomena — Reference: 5.4.4; Table 5.26 [5.5.4]

3.2.4.1 Section 1:

$n_z = N_{Sd}/N_{z.Rd} = 100,0/1207,1 = 0,08$

$\Psi = 0 \longrightarrow \beta_{M.y} = 1,8$

$\mu_{LT} = 0,15\lambda_{1.z}\beta_{M.LT} - 0,15 = 0,15 \cdot 1,02 \cdot 1,8 - 0,15 = 0,125 < 0,9$

$1 - \mu_{LT} n_z/\gamma_{M1} = 1 - 0,125 \cdot 0,08/1,1 = 0,991 < 1,0 \longrightarrow k_{LT} = 0,991$

$$n_z + \frac{k_{LT} M_{y.Sd}}{M_{1.b.Rd}} = 0,08 + \frac{0,991 \cdot 133,3}{257,1} = 0,59 < 1$$

3.2.4.2 Section 2: — Reference: 5.4.4; Table 5.26 [5.5.4]

$n_z = N_{Sd}/N_{z.Rd} = 100,0/1817,2 = 0,06$

$\Psi = 133,3/200 = 0,67 \longrightarrow \beta_{M.LT} = 1,8 - 0,7 \cdot 0,67 = 1,331$

$\mu_{LT} = 0,15\lambda_{2.z}\beta_{M.LT} - 0,15 = 0,15 \cdot 0,51 \cdot 1,331 - 0,15 = -0,05 < 0,9$

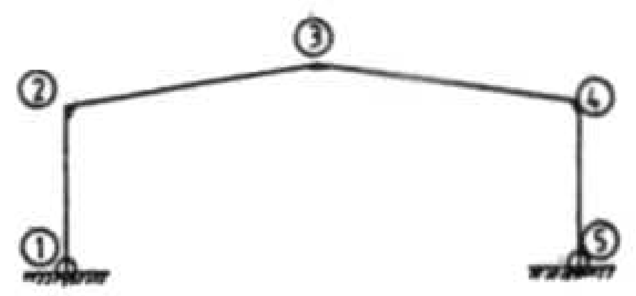
$1 - \mu_{LT} n_z/\gamma_{M1} = 1 + 0,05 \cdot 0,06/1,1 = 1,003 > 1,0 \longrightarrow k_{LT} = 1,0$

$$n_z + \frac{k_{LT} M_{y.Sd}}{M_{2.b.Rd}} = 0,06 + \frac{1,0 \cdot 200,0}{278,1} = 0,78 < 1$$

Problem, Sketch, Calculation	Reference	Commentary

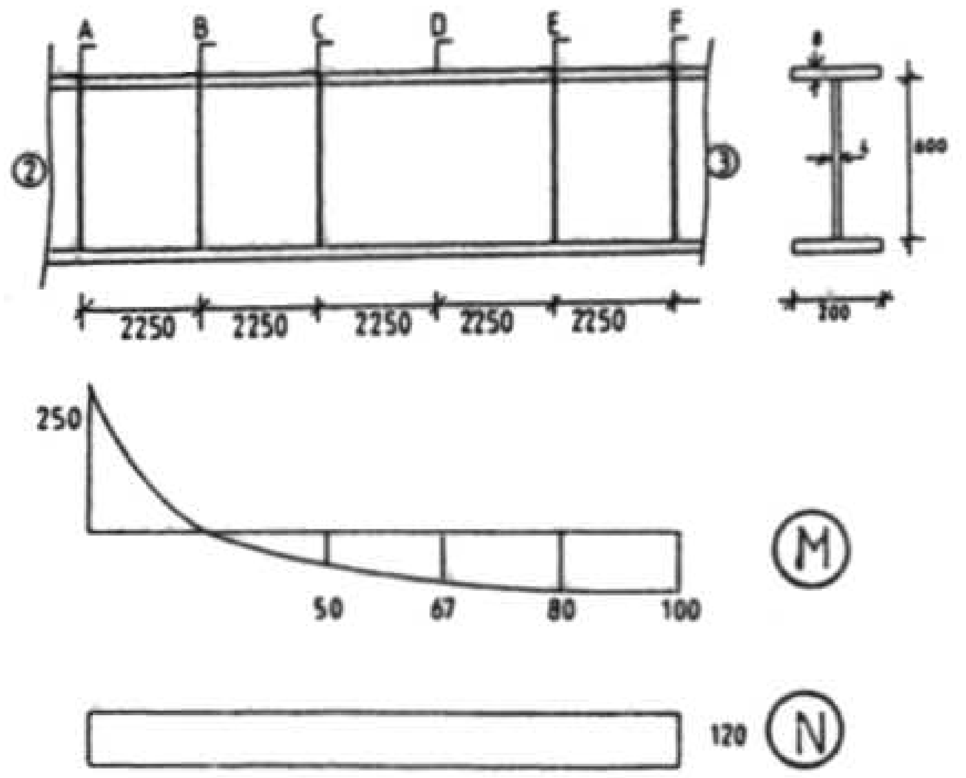
Example 2.3.4: Rafter of a frame

1. System

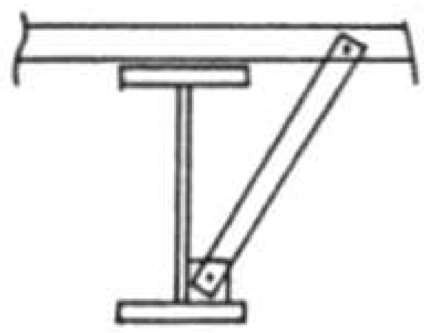


Welded section Steel grade Fe 510
Dimensions: see sketch
Buckling length: ℓ_y = 14,2875 m
Rafter laterally restraint at A, B, C, E, F

Weld between flange and web: a = 3 mm
$A = 56 \cdot 10^2$ mm²
$I_y = 36775 \cdot 10^4$ mm⁴ i_y = 256,3 mm
$I_z = 1067 \cdot 10^4$ mm⁴ i_z = 43,6 mm
The buckling length is calculated separately.



Detail of
lateral restraints



2. Loading

see sketch

3. Determination of the resistance

3.1 Width-to-thickness ratios

Reference: 5.3.1; Table 5.10 [5.3; Table 5.3.1]

Fe 510 ⟶ ϵ = 0,81

Flange: c = 100 - 2 - $\sqrt{2} \cdot 3$ = 94 mm

c/t = 94/8 = 11,8 > 14ϵ = 11,3

Web:

d/t = 600/4 = 150 > 124ϵ = 100,4

⟶ Determination of A_{eff} and W_{eff}

Reference: 5.3.3 [5.4.6]

d/t = 150 > 69ϵ = 55,9

⟶ shear buckling verification is required

The classification with the exact stress distribution leads also to a class 4 section.

The verification of shear buckling is not contained in this example.

3.2 Determination of effective properties of the cross-section

3.2.1 A_{eff} for compression

Reference: 5.3.5; Table 5.18 [5.3.5]

Fe 510 ⟶ 1/ϵ = 1,23

Flange:

$$\bar{b} = c = 94 \text{ mm}; \quad \bar{\lambda}_p = \frac{c/t}{28{,}4\, \epsilon \sqrt{k_\sigma}} = \frac{c}{t}\,\frac{1}{18{,}6\,\epsilon} = 11{,}8 \cdot \frac{1{,}23}{18{,}6} = 0{,}78 > 0{,}673$$

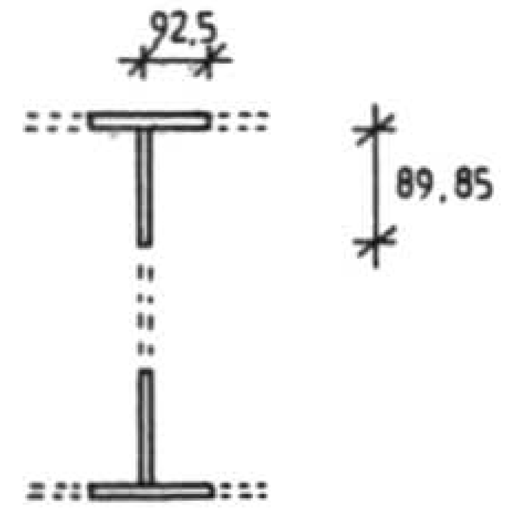
$$\rho = \frac{\bar{\lambda}_p - 0{,}22}{\bar{\lambda}_p^2} = \frac{0{,}78 - 0{,}22}{0{,}78^2} = 0{,}92$$

$c_{eff} = \rho \cdot c = 0{,}92 \cdot 94 = 86{,}5$ mm

Problem, Sketch, Calculation	Reference	Commentary

Web:

$\bar{b} = d = 592$ mm ; $\bar{\lambda}_p = \dfrac{d/t}{28{,}4\ \varepsilon \sqrt{k_\sigma}} = \dfrac{d}{t}\ \dfrac{1}{56{,}8\ \varepsilon} = 148 \cdot \dfrac{1{,}23}{56{,}8} = 3{,}20 > 0{,}673$



$\rho = \dfrac{\bar{\lambda}_p - 0{,}22}{\bar{\lambda}_p^2} = \dfrac{3{,}20 - 0{,}22}{3{,}20^2} = 0{,}29$

$d_{eff} = \rho \cdot d = 0{,}29 \cdot 592 = 171{,}7$ mm

$A_{eff} = 2 \cdot (2 \cdot 86{,}5 + 12) \cdot 8 + (171{,}7 + 8) \cdot 4 = 36{,}8 \cdot 10^2$ mm^2

3.2.2 W_{eff} for bending

5.3.5; Table 5.18
[5.3.5]

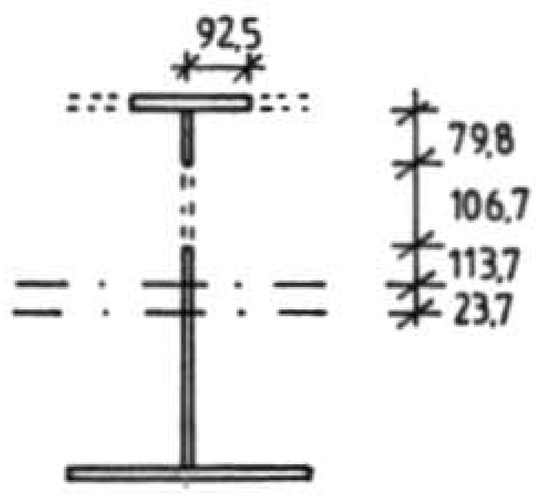
Flange:

The compressed flange is reduced as above

$c_{eff} = \rho \cdot c = 0{,}92 \cdot 94 = 86{,}5$ mm

Web:

$\bar{b} = d = 592$ mm ; $\bar{\lambda}_p = \dfrac{d/t}{28{,}4\ \varepsilon \sqrt{k_\sigma}} = \dfrac{d}{t}\ \dfrac{1}{138{,}8\ \varepsilon} = 148 \cdot \dfrac{1{,}23}{138{,}8} = 1{,}31 > 0{,}673$



$\rho = \dfrac{\bar{\lambda}_p - 0{,}22}{\bar{\lambda}_p^2} = \dfrac{1{,}31 - 0{,}22}{1{,}31^2} = 0{,}64$

$d_{eff.1} = 0{,}4 \cdot \rho \cdot d_c = 0{,}4 \cdot 0{,}64 \cdot 0{,}5 \cdot 592 = 75{,}8$ mm

$d_{eff.2} = 0{,}6 \cdot \rho \cdot d_c = 0{,}6 \cdot 0{,}64 \cdot 0{,}5 \cdot 592 = 113{,}7$ mm

$A_{eff} = (2 \cdot 86{,}5 + 12 + 200) \cdot 8 + (75{,}8 + 113{,}7 + 0{,}5 \cdot 592 + 8) \cdot 4) = 50{,}5 \cdot 10^2$ mm^2

$e = (608 \cdot 185 \cdot 8 + 80 \cdot 4 \cdot 564 + 413{,}9 \cdot 4 \cdot 211)/505 = 280{,}3$ mm

$\Delta e_M = 304 - 280{,}3 = 23{,}70$ mm

$I_{eff.y} = 4 \cdot (80^3 + 413{,}9^3)/12 + 2 \cdot 8^3 \cdot (179 + 200)/12 + 280{,}3^2 \cdot 200 \cdot 8 + 327{,}2^2 \cdot 179 \cdot 8 + 285{,}8^2 \cdot 80 \cdot 4 + 413{,}9 \cdot 4 \cdot 69{,}45^2$

$= 33698 \cdot 10^4$ mm^4

$W_{eff.y} = 33698 \cdot 10^4/331{,}7 = 1016 \cdot 10^3$ mm^3

3.3 Member resistance verification

3.3.1 Axial compression resistance

5.4.2; [5.5.1]

For determining the slenderness λ of a class 4 section the radius of gyration about the relevant axis from the gross cross-section should always be used.

Strong axis: Buckling length: $\ell_y = 14{,}2875$ m

$\bar{\lambda}_y = \dfrac{\ell_y}{i_y}\ \dfrac{1}{\lambda_1} \sqrt{\dfrac{A_{eff}}{A}} = \dfrac{14{,}2875 \cdot 10^3}{256{,}3} \cdot \dfrac{1}{76{,}4} \cdot \sqrt{\dfrac{36{,}8 \cdot 10^2}{56 \cdot 10^2}} = 0{,}59$

using buckling curve b ----→ $\chi_b = 0{,}8420$

$N_{y.b.Rd} = \chi_b \cdot \dfrac{A_{eff} \cdot f_y}{\gamma_{M1}} = 0{,}8420 \cdot \dfrac{36{,}8 \cdot 10^2 \cdot 355}{1{,}1} \cdot 10^{-3} = 1000{,}0$ kN $> 120{,}0$kN

Problem, Sketch, Calculation	Reference	Commentary

Weak axis: between A-B, B-C and E-F: Buckling length: $\ell_z = 2{,}25$ m

$$\bar{\lambda}_z = \frac{\ell_z}{i_z}\,\frac{1}{\lambda_1}\sqrt{\frac{A_{eff}}{A}} = \frac{2{,}25\cdot 10^3}{43{,}6}\cdot\frac{1}{76{,}4}\cdot\sqrt{\frac{36{,}8\cdot 10^2}{56\cdot 10^2}} = 0{,}55$$

using buckling curve c $\longrightarrow \chi_c = 0{,}8146$

$$N_{z.b.Rd} = \chi_c\cdot\frac{A_{eff}\cdot f_y}{\gamma_{M1}} = 0{,}8146\cdot\frac{36{,}8\cdot 10^2\cdot 355}{1{,}1}\cdot 10^{-3} = 967{,}4 \text{ kN} > 120{,}0 \text{kN}$$

Weak axis: between C - E: Buckling length: $\ell_z = 4{,}50$ m

$$\bar{\lambda}_z = \frac{\ell_z}{i_z}\,\frac{1}{\lambda_1}\sqrt{\frac{A_{eff}}{A}} = \frac{4{,}5\cdot 10^3}{43{,}6}\cdot\frac{1}{76{,}4}\cdot\sqrt{\frac{36{,}8\cdot 10^2}{56\cdot 10^2}} = 1{,}10$$

using buckling curve c $\longrightarrow \chi_c = 0{,}4842$

$$N_{z.b.Rd} = \chi_c\cdot\frac{A_{eff}\cdot f_y}{\gamma_{M1}} = 0{,}4842\cdot\frac{36{,}8\cdot 10^2\cdot 355}{1{,}1}\cdot 10^{-3} = 575{,}1 \text{ kN} > 120{,}0 \text{kN}$$

3.3.2 Bending moment resistance

Reference: 5.4.3; [5.5.2]

Commentary: For determining M_{cr} the properties of the gross cross-section should always be used.

Part A - B:

buckling length $\ell_{LT} = 2{,}25$ m

$k = k_w = 1{,}0$; $\Psi \approx 0 \longrightarrow C_1 = 1{,}879$

$$M_{cr} = C_1\,\frac{\pi^2 E I_z}{\ell_{LT}^2}\sqrt{\left(\frac{k}{k_w}\right)^2\frac{I_w}{I_z} + 0{,}039\,\ell_{LT}^2\,\frac{I_t}{I_z}} = 1{,}879\cdot\frac{\pi^2\cdot 210000\cdot 1067\cdot 10^4}{2{,}25^2\cdot 10^{12}}\sqrt{\frac{986000\cdot 10^6 + 0{,}039\cdot 2{,}25^2\cdot 10^6\cdot 8{,}1\cdot 10^4}{1067\cdot 10^4}} = 2515{,}3 \text{ kNm}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{eff}\,f_y}{M_{cr}}} = \sqrt{\frac{1016\cdot 10^3\cdot 355}{2515{,}3\cdot 10^6}} = 0{,}38 < 0{,}4$$

$\longrightarrow$ no lateral torsional buckling verification is required

Part D - E:

buckling length $\ell_{LT} = 2{,}25$ m

$k = k_w = 1{,}0$; $\Psi = 67/80 = 0{,}838 \longrightarrow C_1 = 1{,}091$

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{\ell_{LT}^2}\sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + 0{,}039\, \ell_{LT}^2 \frac{I_t}{I_z}} = 1{,}091\cdot\frac{\pi^2\cdot 210000\cdot 1067\cdot 10^4}{2{,}25^2\cdot 10^{12}}\sqrt{\frac{986000\cdot 10^6 + 0{,}039\cdot 2{,}25^2\cdot 10^6\cdot 8{,}1\cdot 10^4}{1067\cdot 10^4}} = 1460{,}5 \text{ kNm}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{eff}\, f_y}{M_{cr}}} = \sqrt{\frac{1016\cdot 10^3\cdot 355}{1460{,}5\cdot 10^6}} = 0{,}50 > 0{,}4$$

$\longrightarrow$ using buckling curve c $\longrightarrow \chi_{LT.c} = 0{,}8430$

$$M_{b.Rd} = \chi_{LT.c}\,\frac{W_{eff}\, f_y}{\gamma_{M1}} = 0{,}8430\cdot\frac{1016\cdot 10^3\cdot 355}{1{,}1}\cdot 10^{-6} = 276{,}4 \text{ kNm} > 80{,}0 \text{ kNm}$$

Part E - F:

buckling length $\ell_{LT} = 2{,}25$ m

$k = k_w = 1{,}0$; $\Psi = 80/100 = 0{,}8 \longrightarrow C_1 = 1{,}113$

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{\ell_{LT}^2}\sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + 0{,}039\, \ell_{LT}^2 \frac{I_t}{I_z}} = 1{,}113\cdot\frac{\pi^2\cdot 210000\cdot 1067\cdot 10^4}{2{,}25^2\cdot 10^{12}}\sqrt{\frac{986000\cdot 10^6 + 0{,}039\cdot 2{,}25^2\cdot 10^6\cdot 8{,}1\cdot 10^4}{1067\cdot 10^4}} = 1489{,}9 \text{ kNm}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{eff}\, f_y}{M_{cr}}} = \sqrt{\frac{1016\cdot 10^3\cdot 355}{1489{,}9\cdot 10^6}} = 0{,}49 > 0{,}4$$

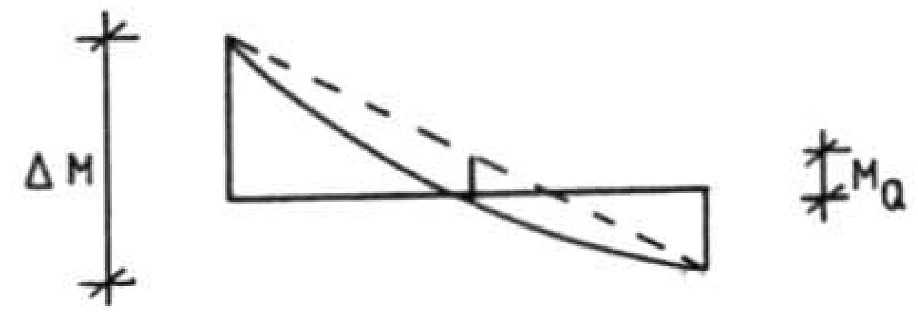
$\longrightarrow$ using buckling curve c $\longrightarrow \chi_{LT.c} = 0{,}8486$

$$M_{b.Rd} = \chi_{LT.c}\,\frac{W_{eff}\, f_y}{\gamma_{M1}} = 0{,}8486\cdot\frac{1016\cdot 10^3\cdot 355}{1{,}1}\cdot 10^{-6} = 278{,}2 \text{ kNm} > 100{,}0 \text{ kNm}$$

3.3.3 Interaction Bending - Axial compression without lateral torsional buckling

Reference: 5.4.4; Table 5.26 [5.5.4]

Commentary: $M_Q = 160$ kNm; $\Delta M = 350$ kNm



Part A-B: $n_y = N_{Sd}/N_{y.b.Rd} = 120{,}0/1000{,}0 = 0{,}12$

$\Psi = -100/250 = -0{,}4$

$\longrightarrow \beta_{M.\psi} \approx 1{,}8 + 0{,}7\bullet 0{,}4 = 2{,}08$

$\beta_{M.y} = \beta_{M.\psi} + M_Q/\Delta M\bullet(\beta_{M.Q} - \beta_{M.\psi}) = 2{,}08 + 160/350\bullet(1{,}3 - 2{,}08) = 1{,}72$

$\mu_y = \lambda_y(2\beta_{M.y} - 4) = 0{,}58\ (2\bullet 1{,}72 - 4) = -0{,}325 < 0{,}9$

$1 - \mu_y n_y/\gamma_{M1} = 1 + 0{,}325\bullet 0{,}12/1{,}1 = 1{,}04 < 1{,}5 \longrightarrow k_y = 1{,}04$

$$n_y + \frac{k_y\, M_{y.Sd}}{W_{eff.y}\,\frac{f_y}{1{,}1}} = 0{,}12 + \frac{1{,}04\cdot 250{,}0\cdot 10^6}{1016\cdot 10^3\cdot\frac{355}{1{,}1}} = 0{,}91 < 1$$

Problem, Sketch, Calculation	Reference	Commentary
3.3.4 Interaction Bending - Axial compression with lateral torsional buckling	5.4.4; Table 5.26 [5.5.4]	Lateral torsional buckling is considered between D - E whereas flexural buckling is considered between C - E.

Part C - E:

$n_z = N_{Sd}/N_{z.b.Rd} = 120{,}0/575{,}1 = 0{,}21$

$\Psi = 67/80 = 0{,}84 \longrightarrow \beta_{M.LT} = 1{,}8 - 0{,}7 \bullet 0{,}84 = 1{,}212$

$\mu_{LT} = 0{,}15 \lambda_{1.z} \beta_{M.LT} - 0{,}15 = 0{,}15 \bullet 1{,}08 \bullet 1{,}212 - 0{,}15 = 0{,}05 < 0{,}9$

$1 - \mu_{LT} n_z / \gamma_{M1} = 1 - 0{,}05 \bullet 0{,}21/1{,}1 = 0{,}99 < 1{,}0 \longrightarrow k_{LT} = 0{,}99$

$$n_z + \frac{k_{LT}\, M_{y.Sd}}{M_{1.b.Rd}} = 0{,}21 + \frac{0{,}99 \cdot 80}{276{,}4} = 0{,}50 < 1$$

Part E - F:

$n_z = N_{Sd}/N_{z.b.Rd} = 120{,}0/967{,}4 = 0{,}12$

$\Psi = 80/100 = 0{,}8 \longrightarrow \beta_{M.LT} = 1{,}8 - 0{,}7 \bullet 0{,}8 = 1{,}24$

$\mu_{LT} = 0{,}15 \lambda_{1.z} \beta_{M.LT} - 0{,}15 = 0{,}15 \bullet 0{,}54 \bullet 1{,}24 - 0{,}15 = -0{,}05 < 0{,}9$

$1 - \mu_{LT} n_z / \gamma_{M1} = 1 + 0{,}05 \bullet 0{,}12/1{,}1 = 1{,}01 > 1{,}0 \longrightarrow k_{LT} = 1{,}0$

$$n_z + \frac{k_{LT}\, M_{y.Sd}}{M_{1.b.Rd}} = 0{,}13 + \frac{1{,}0 \cdot 100}{276{,}4} = 0{,}49 < 1$$

2.4 Local stresses

These examples demonstrate the verification of load introduction problems assuming design values of action effects (N_{Sd}, V_{Sd}, M_{Sd}, etc) which have been calculated by an analysis of the structure and these values already include ψ, γ_F etc.
The results presented in the examples are rounded values. For the purpose of easy re-calculation each formula and each check is calculated with the rounded values.

The following examples are included in this chapter:

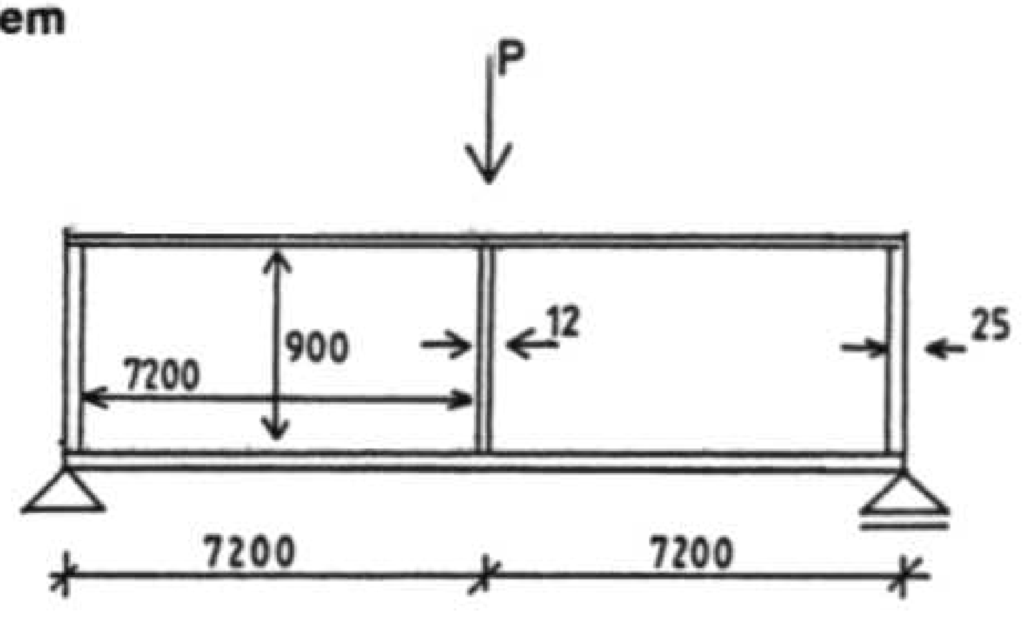
Example 2.4.1:	Design of transverse stiffeners (continuation of example 3 of chapter "Members in Bending")
Example 2.4.2:	Design of intermediate transverse stiffeners (continuation of example 4 of chapter "Members in Bending")
Example 2.4.3:	Axially loaded column supported by a beam
Example 2.4.4:	Load introduction of wheel loads from cranes
Example 2.4.5:	Beam supported by a beam

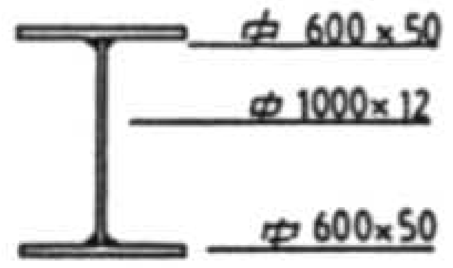
Problem, Sketch, Calculation	Reference	Commentary

Example 2.4.1: Design of transverse stiffeners (continuation of example 2.2.3 of chapter "Members in Bending")

The beam is continuously laterally restrained.
Throat thickness between flange and web:
$a = 7$ mm

1. System





Beam: Welded section with stiffeners at the supports and at mid-span
Dimensions see sketch
Steel grade: Fe 510
System length: $L = 14{,}40$ m

2. Loading

At mid-span: $P_{Sd} = 2377{,}5$ kN
Internal forces and moments at mid span: $M_{Sd} = 8664{,}0$ kNm
$V_{Sd} = 1188{,}8$ kN

3. Determination of the resistance of the transverse stiffener at mid span

3.1 Verification of the stiffness effect of the stiffener

Reference: 5.4.5.4.(6); [5.6.5.(3)]

$a/d = 8{,}0 > \sqrt{2}$

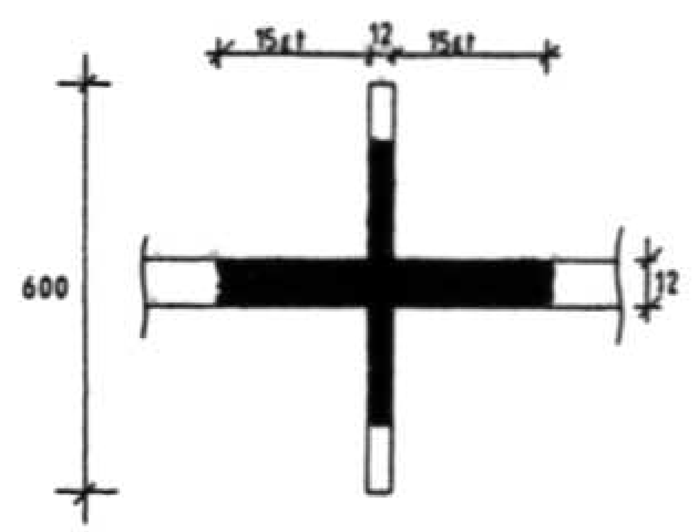
$\longrightarrow \quad I_s = 600^3 \cdot 12/12 = 21600 \cdot 10^4\ \text{mm}^4 > 0{,}75 d t_w^3 = 0{,}75 \cdot 880{,}2 \cdot 12^3 = 114{,}1 \cdot 10^4\ \text{mm}^4$

3.2 Buckling resistance verification (perpendicular to the web-plane)

3.2.1 Effective cross-section of the stiffener

Reference: 5.4.5.4.(1); [5.7.6.(1)]

In determining the effective cross-section the cut off for the welds of the stiffeners need not to be considered.



Fe 510 $\longrightarrow \varepsilon = 0{,}81$

$\bar{b}/t = 294/12 = 24{,}5 > 15\varepsilon = 12{,}2 \longrightarrow$ calculation of effective cross-section of stiffener

Reference: 5.3.5(3)

$$\bar{\lambda}_p = \frac{b/t}{28{,}4\,\varepsilon\sqrt{k_\sigma}} = \frac{24{,}5}{28{,}4 \cdot 0{,}81 \cdot \sqrt{0{,}43}} = 1{,}624 > 0{,}673 \qquad \rho = \frac{\bar{\lambda}_p - 0{,}22}{\bar{\lambda}_p^2} = \frac{1{,}624 - 0{,}22}{1{,}624^2} = 0{,}532$$

$15\varepsilon t_w = 15 \cdot 0{,}81 \cdot 12 = 146$ mm

$A_{eff} = 0{,}532 \cdot 2 \cdot 294 \cdot 12 + (2 \cdot 146 + 12) \cdot 12 = 37{,}5 \cdot 10^2 + 36{,}5 \cdot 10^2 = 74 \cdot 10^2\ \text{mm}^2$

$A = 146 \cdot 12 \cdot 2 + 12 \cdot 600 = 107{,}0 \cdot 10^2\ \text{mm}^2$

$I_z = 600^3 \cdot 12/12 = 21600 \cdot 10^4\ \text{mm}^4$

$i_z = \sqrt{(I_z/A)} = \sqrt{21600 \cdot 10^4 / 107 \cdot 10^2} = 142$ mm

3.2.2 Buckling length

Reference: 5.4.5.4.(2); [5.7.6.(2)]

Assumption: $\ell = 0{,}9$ m $> 0{,}675$ m $= 0{,}75d$

0,75d as a buckling length could be taken if appropriate restraint is provided

Problem, Sketch, Calculation	Reference	Commentary

3.2.4 Buckling verification

Reference: 5.4.2; [5.5.1]

$$\bar{\lambda} = \frac{\ell}{i_z}\,\frac{1}{\lambda_1}\sqrt{\frac{A_{eff}}{A}} = \frac{0{,}9\cdot 10^3}{142}\,\frac{1}{76{,}4}\sqrt{\frac{74\cdot 10^2}{107\cdot 10^2}} = 0{,}07 < 0{,}2$$

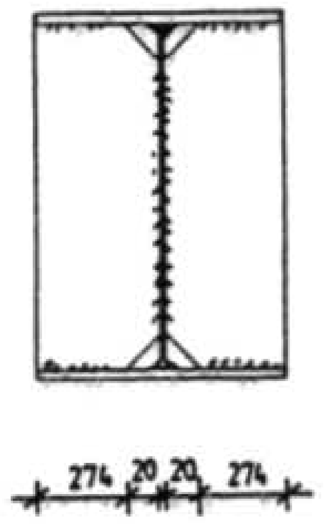
⟶ no reduction due to buckling is required

$$\frac{N_{Sd}}{A_{eff}\,\frac{f_y}{\gamma_{M1}}} = \frac{2377{,}5\cdot 10^3}{74\cdot 10^2\cdot\frac{355}{1{,}1}} = 0{,}996 < 1$$

Commentary: When $\bar{\lambda} < 0{,}2$ no buckling mode is govern. Only cross-section verification is necessary. γ_{M1} is used because of class 4 cross-section.

3.3 Crushing resistance verification of the stiffened web and the stiffener

Reference: 5.6.3 [5.7.6.(6); 5.7.3]



$\sigma_{f.Ed} = M\bullet z/I = 8664{,}0\bullet 100\bullet 500/1427900 = 303\ N/mm^2$

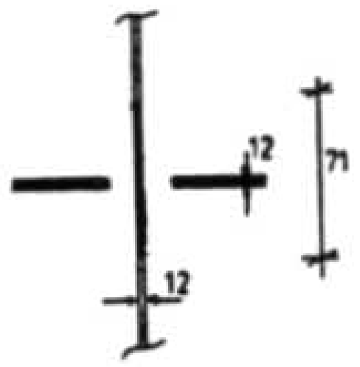
$b_f = 600\ mm < 25t_f = 25\bullet 50 = 1250\ mm$

$$s_y = 2\,t_f\sqrt{\frac{b_f}{t_w}}\sqrt{\frac{f_{yf}}{f_{yw}}}\sqrt{1-\left(\frac{\gamma_{Mo}\sigma_{f.Ed}}{f_{yf}}\right)^2} = 2\cdot 50\sqrt{\frac{600}{12}}\sqrt{1-\left(\frac{1{,}1\cdot 303}{335}\right)^2} = 71\ mm$$

$$< 2\cdot 15\,\varepsilon\,t_w + t_s = 2\cdot 146 + 12 = 304\ mm$$

$f_{yw} = f_{y.stiffener} \longrightarrow A = s_y t_w + t_s(b_f - t_w - 2c) = 71\bullet 12 + 12\bullet(600 - 12 - 2\bullet 20) = 74{,}3\bullet 10^2\ mm^2$

$$R_{y.Rd} = A\,\frac{f_y}{\gamma_{M1}} = 74{,}3\cdot 10^2\cdot\frac{355}{1{,}1}\cdot 10^{-3} = 2397{,}9\ kN > 2377{,}5\ kN$$



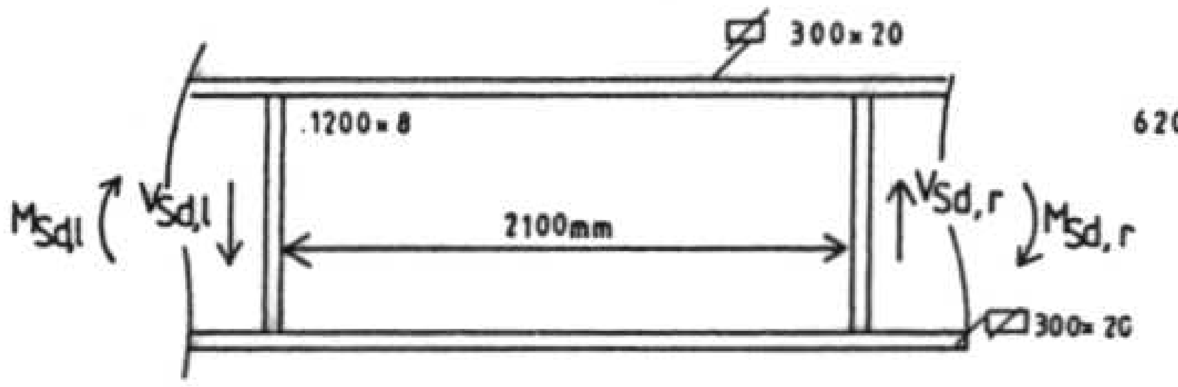
Commentary: Allowance needs to be made for the openings cut in the stiffener to clear the web-flange welds. This check is only necessary for a load bearing stiffener.

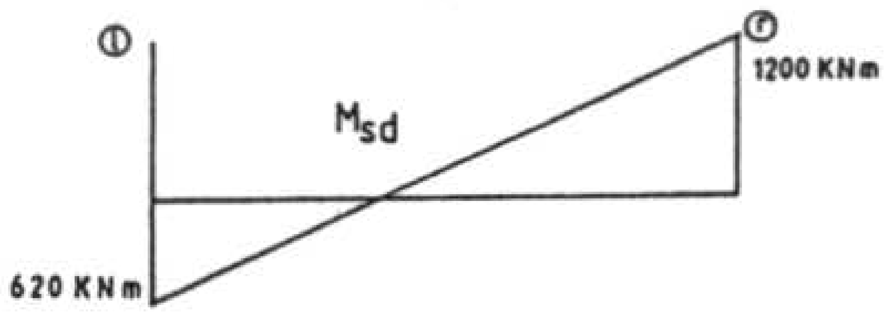
Problem, Sketch, Calculation	Reference	Commentary

Example 2.4.2: Design of intermediate transverse stiffeners (continuation of example 2.2.4 of chapter "Members in Bending")

The beam is continuously laterally restrained.
Throat thickness between flange and web:
a = 7 mm

1. System





Beam: Welded section
Dimensions see sketch
Steel grade: Fe 360

2. Loading

Bending moment distribution: see sketch
Shear force: V_{Sd} = 700,0 kN

3. Determination of the resistance of the intermediate transverse stiffeners

3.1 Verification of the stiffness effect of the stiffener

Reference: 5.4.5.4.(6); [5.6.5.(3)]

$a/d = 1{,}75 > \sqrt{2}$

$\longrightarrow \quad I_s = 300^3 \cdot 6/12 = 1350 \cdot 10^4 \text{ mm}^4 > 0{,}75 d t_w^3 = 0{,}75 \cdot 1482{,}5 \cdot 8^3 = 56{,}9 \cdot 10^4 \text{ mm}^4$

3.2 Buckling resistance verification (perpendicular to the web-plane)

3.2.1 Determination of the compression force in the stiffener

Reference: 5.4.5.4.(1); [5.6.5.(1)]

Commentary: No change of sign in the shear force line

$\underline{V}_{Sd} = 700{,}0$ kN

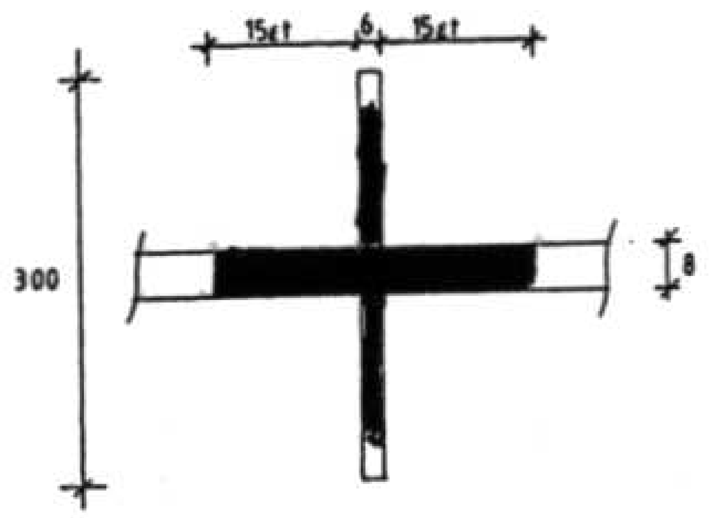
$\bar{\lambda}_w = 1{,}556 \longrightarrow \tau_{bb} = (1/\bar{\lambda}_w^2)\, f_{yw}/\sqrt{3} = 1/1{,}556^2\; 235/\sqrt{3} = 56{,}0 \text{ N/mm}^2$

$N_{Sd} = V_{Sd} - d t_w \tau_{bb}/\gamma_{M1} = 700{,}0 - 1200 \cdot 8 \cdot 56{,}0 \cdot 10^{-3}/1{,}1 = 211{,}3 \text{ kN}$

3.2.2 Effective cross-section of the stiffener

Reference: 5.4.5.4.(1); [5.7.6.(1)]

Commentary: In determining the effective cross-section the cut off, for the welds, of the stiffeners need not to be considered.



Fe 360 $\longrightarrow \varepsilon = 1{,}0$; $k_\sigma = 0{,}43$

$\bar{b}/t = 146/6 = 24{,}3 > 15\varepsilon = 15 \longrightarrow$ calculation of effective cross-section of stiffener

$$\bar{\lambda}_p = \frac{b/t}{28{,}4\, \varepsilon \sqrt{k_\sigma}} = \frac{24{,}3}{28{,}4 \cdot 1{,}0 \cdot \sqrt{0{,}43}} = 1{,}305 > 0{,}673 \qquad \rho = \frac{\bar{\lambda}_p - 0{,}22}{\bar{\lambda}_p^2} = \frac{1{,}305 - 0{,}22}{1{,}305^2} = 0{,}637$$

$15\varepsilon t_w = 15 \cdot 1{,}0 \cdot 8 = 120$ mm

$A_{eff} = 0{,}637 \cdot 2 \cdot 146 \cdot 6 + (2 \cdot 120 + 6) \cdot 8 = 11{,}2 \cdot 10^2 + 19{,}7 \cdot 10^2 = 30{,}9 \cdot 10^2 \text{ mm}^2$

$A = 120 \cdot 8 \cdot 2 + 6 \cdot 300 = 37{,}2 \cdot 10^2 \text{ mm}^2$

$I_z = 300^3 \cdot 6/12 = 1350 \cdot 10^4 \text{ mm}^4$

$i_z = \sqrt{(I_z/A)} = \sqrt{1350 \cdot 10^4 / 37{,}2 \cdot 10^2} = 60{,}2$ mm

3.2.3 Buckling length

Reference: 5.4.5.4.(2); [5.7.6.(2)]

Commentary: 0,75d as a buckling length could be taken if appropriate restraint is provided

Assumption: $\ell = 1{,}20 \text{ m} > 0{,}9 \text{ m} = 0{,}75d$

3.2.4 Buckling verification

Reference: 5.4.2; [5.5.1]

Commentary: When $\bar{\lambda} < 0{,}2$ no buckling mode is govern. Only cross-section verification is necessary. γ_{M1} is used because of class 4 cross-section.
For intermediate transverse stiffeners it is only necessary to check the buckling resistance provided that they are not subject to external loads

$$\bar{\lambda} = \frac{\ell}{i_z} \frac{1}{\lambda_1} \sqrt{\frac{A_{eff}}{A}} = \frac{1{,}20 \cdot 10^3}{60{,}2} \frac{1}{93{,}9} \sqrt{\frac{30{,}9}{37{,}2}} = 0{,}19$$

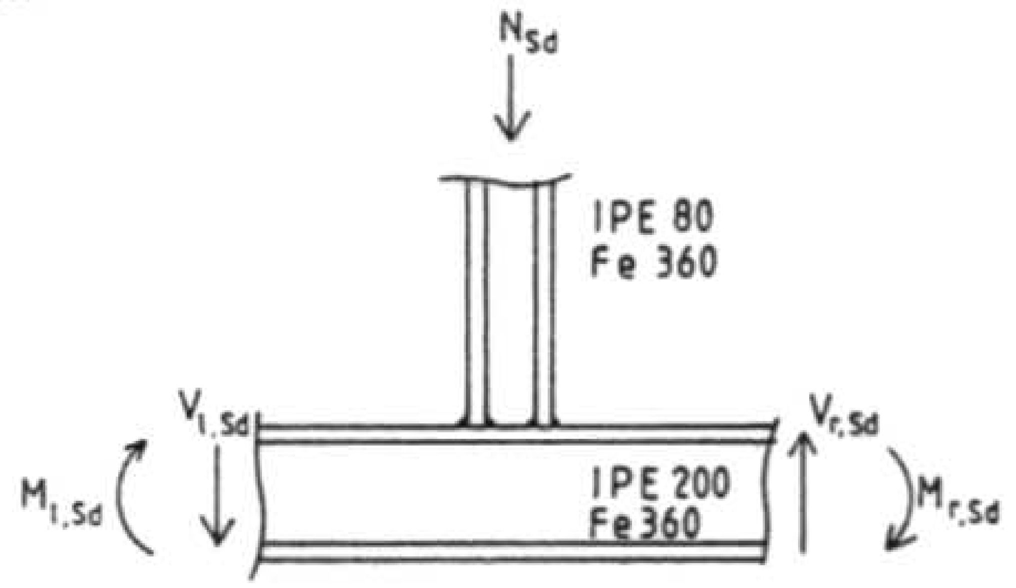
$$N_{b.Rd} = A_{eff} \frac{f_y}{\gamma_{M1}} = 30{,}9 \cdot 10^2 \cdot \frac{235}{1{,}1} \cdot 10^{-3} = 660{,}1 \text{ kN} > 211{,}3 \text{ kN}$$

Problem, Sketch, Calculation	Reference	Commentary

Example 2.4.3: Axial loaded column supported by a beam

Throat thickness of the weld connecting the chord member and the bracing member: a = 3 mm

1. System



Beam:	IPE 200	Steel grade:	Fe 360
Bracing:	IPE 80	Steel grade:	Fe 360

2. Loading

Compression force:	N_{Sd} = 100,0 kN
Bending moment:	$M_{r,Sd}$ = 40,0 kNm
	$M_{l,Sd}$ = 40,0 kNm
Shear force:	$V_{r,Sd}$ = 50,0 kN
	$V_{l,Sd}$ = 50,0 kN

3. Determination of the resistances

3.1 Determination of the reduced effective breadth resistance

Reference: 6.3.6; [6.6.8]

$$b_{eff} = \min \begin{cases} t_w + 2r + 7t_f \\ t_w + 2r + 7\,\dfrac{t_f^2}{t_p}\,\dfrac{f_y}{f_{yp}} \end{cases}$$

$$= \min \begin{cases} 5{,}6 + 2\cdot 12 + 7\cdot 8{,}5 = 89{,}1\ mm \\ 5{,}6 + 2\cdot 12 + 7\cdot \dfrac{8{,}5^2}{5{,}2} = 126{,}9\ mm \end{cases}$$

$\longrightarrow$ b_{eff} = 89,1 mm > b = 80 mm

$\longrightarrow$ the flanges of the bracing are fully effective and no stiffening of the joint is required

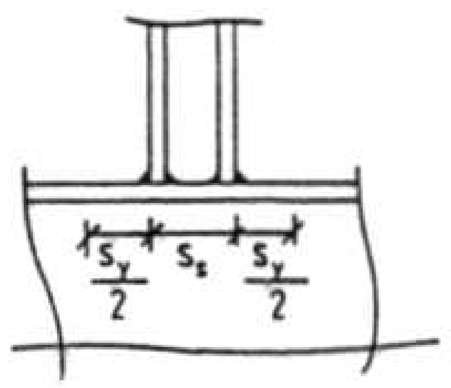
3.2 Crushing resistance

Reference: 5.6.2; [5.7.2]

The length of stiff bearing on the flange is the distance over which the applied force is effectively distributed

3.2.1 Determination of the length of stiff bearing s_s

$s_s = h_{bracing}$ = 80 mm



3.2.2 Determination of s_y

Reference: 5.6.3; [5.7.3]

$\sigma_{f.Ed} = M_{Sd}/W_{el} = 40\cdot 10^6/194\cdot 10^3 = 206\ N/mm^2$

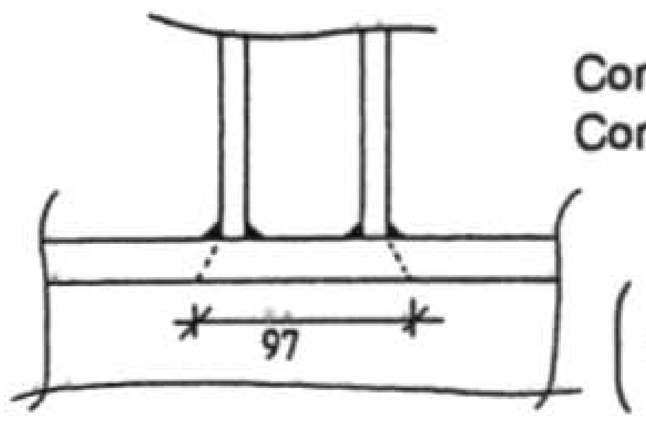
b_f = 100 mm < 212,5 mm = $25t_f$

$$s_y = 2t_f \sqrt{\frac{b_f}{t_w}\,\frac{f_{yf}}{f_{yw}}\left[1 - \left(\frac{\sigma_{f.Ed}\ \gamma_{Mo}}{f_{yf}}\right)^2\right]} = 2\cdot 8{,}5\sqrt{\frac{100}{5{,}6}\left[1 - \left(\frac{206\cdot 1{,}1}{235}\right)^2\right]} = 19{,}0\ mm$$

3.2.3 Verification

Reference: 5.6.3; [5.7.3]

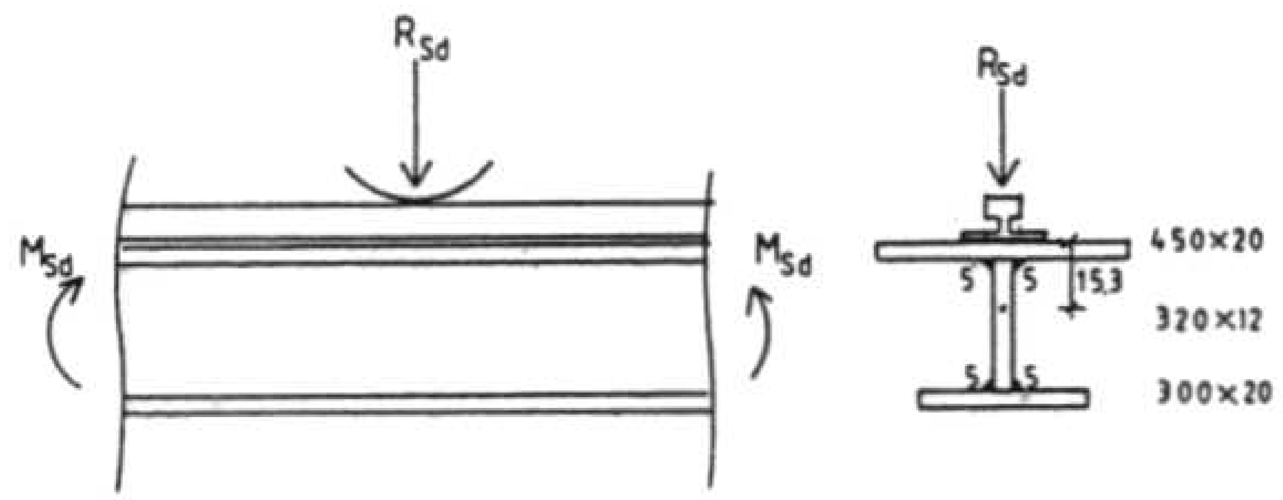
$$R_{y.Rd} = (s_s + s_y)\,t_w\,\frac{f_{yw}}{\gamma_{M1}} = (80 + 19)\cdot 5{,}6\cdot\frac{235}{1{,}1}\cdot 10^{-3} = 118{,}4\ kN > 100\ kN$$

Problem, Sketch, Calculation	Reference	Commentary
3.3 Crippling resistance $s_s/d = 80/159 = 0{,}5 > 0{,}2 \longrightarrow s_s/d = 0{,}2$ $R_{a.Rd} = 0{,}5\, t_w^2 \sqrt{E\, f_{yw}} \left[\sqrt{\frac{t_f}{t_w}} + 3\,\frac{t_w}{t_f}\,\frac{s_s}{d}\right] / \gamma_{M1} = 0{,}5\cdot 5{,}6^2 \sqrt{210000\cdot 235} \left[\sqrt{\frac{8{,}5}{5{,}6}} + 3\,\frac{5{,}6}{8{,}5}\,0{,}2\right]\cdot 10^{-3} / 1{,}1 = 163{,}0 \text{ kN} > 100 \text{ kN}$ $M_{c.Rd} = W_{pl}\cdot f_y/\gamma_{Mo} = 221\cdot 10^3\cdot 235\cdot 10^{-6}/1{,}1 = 47{,}2 \text{ kNm} > 40 \text{ kNm}$	5.6.4.(1); [5.7.4.(1)]	
3.3.1 Interaction $\frac{F_{Sd}}{R_{a.Rd}} + \frac{M_{Sd}}{M_{c.Rd}} = \frac{100{,}0}{163{,}0} + \frac{40{,}0}{47{,}2} = 1{,}46 < 1{,}5$	5.6.4.(2); [5.7.4.(2)]	Where the member is subject to bending moments, the interaction criterion has to be fulfilled
3.4 Buckling resistance **3.4.1 Determination of the effective breadth of the web** $b_{eff} = \sqrt{(h^2 + s_s^2)} = \sqrt{(200^2 + 80^2)} = 215{,}4$ mm	5.6.5; [5.7.5]	Where forces are applied through one flange and resisted by shear forces in the web, the buckling resistance of the web to transverse forces need not be considered.
3.4.2 Buckling resistance verification Assumption: buckling length $\ell = 0{,}159$ m $A = 215{,}4\cdot 5{,}6 = 12{,}1\cdot 10^2$ mm²; $I = 215{,}4\cdot 5{,}6^3/12 = 0{,}315\cdot 10^4$ mm⁴ $i = \sqrt{0{,}315\cdot 10^4/12{,}1\cdot 10^2} = 1{,}6$ mm $\bar{\lambda} = \frac{\ell}{i}\,\frac{1}{\lambda_1} = \frac{0{,}159\cdot 10^3}{1{,}6}\,\frac{1}{93{,}9} = 1{,}06$	5.6.5; [5.7.5]	When the detail is such that there is a doubt as to whether buckling mode governs, it should be considered.
using buckling curve c $\longrightarrow \chi_c = 0{,}5059$ $R_{b.Rd} = \chi_c\, A\, \frac{f_{yw}}{\gamma_{M1}} = 0{,}5059\cdot 12{,}1\cdot 10^2\cdot \frac{235}{1{,}1}\cdot 10^{-3} = 130{,}8 \text{ kN} > 100 \text{ kN}$	5.4.2; [5.5.1]	
3.5 Cross-section resistance verification $V_{pl.Rd} = A_v f_y/\sqrt{3}/\gamma_{Mo} = 11{,}2\cdot 10^2\cdot 235\cdot 10^{-3}/\sqrt{3}/1{,}1 = 138{,}1 \text{ kN} > 50 \text{ kN}$ $0{,}5V_{pl.Rd} = 69{,}1 \text{ kN} > 50 \text{ kN} \longrightarrow$ no interaction bending shear is required  Compression: $\sigma_{x.Ed} = M_{Sd}/W_{el} = 40\cdot 10^6/194\cdot 10^3\cdot 91{,}5/100 = 189 \text{ N/mm}^2 < 214 \text{ N/mm}^2 = 235/1{,}1 = f_y/\gamma_{Mo}$ Compression: $\sigma_{z.Ed} = N_{Sd}/(s_s t_w) = 100\cdot 10^3/(97\cdot 5{,}6) = 184 \text{ N/mm}^2 < 214 \text{ N/mm}^2$ $\left(\frac{\sigma_{x.Ed}}{f_{yd}}\right)^2 + \left(\frac{\sigma_{z.Ed}}{f_{yd}}\right)^2 - \frac{\sigma_{x.Ed}}{f_{yd}}\,\frac{\sigma_{z.Ed}}{f_{yd}} = \left(\frac{189}{214}\right)^2 + \left(\frac{184}{214}\right)^2 - \frac{189}{214}\,\frac{184}{214} = 0{,}78 + 0{,}74 - 0{,}88\cdot 0{,}86 = 0{,}76 < 1$	-; [5.4.10]	$A_v = 11{,}2\cdot 10^2$ mm² Higher values may be taken by using the formula given in [5.4.6(2)] The effect of the transverse force on the bending moment resistance of the member should also be considered.

Problem, Sketch, Calculation	Reference	Commentary

Example 2.4.4: Load introduction of wheel loads from cranes

1. System



Beam: Welded section (see sketch)
Steel grade: Fe 360
Crane rail: A 45

2. Loading

Wheel load: R_{Sd} = 200,0 kN
Bending moment: $M_{r.Sd}$ = 366,0 kNm
$M_{l.Sd}$ = 366,0 kNm

Commentary: $I_y = 45296 \cdot 10^4\ mm^4$
Throat thickness of the weld connecting the flange and the web: a = 5 mm
The crane rail is not welded on the flange.

3. Determination of the resistances
3.1 Crushing resistance
3.1.1 Determination of s_y

Reference: -; 5.7.3.(4)

Commentary: Where the details are such that there is doubt over which mode (crushing, crippling or buckling) governs, all three modes should be considered.
k_R is a constant taken as 3,25 for crane rails mounted directly on the flange

$\sigma_{f.Ed} = M_{Sd}/W_{el} = 366 \cdot 10^6 \cdot 153/45296 \cdot 10^4 = 124\ N/mm^2$
$I_R = 91 \cdot 10^4\ mm^4$
$I_f = 450 \cdot 20^3/12 = 30 \cdot 10^4\ mm^4$

$$s_y = k_r \left(\frac{I_f + I_R}{t_w} \right)^{1/3} \sqrt{1 - \left(\frac{\sigma_{f.Ed}\ \gamma_{Mo}}{f_{yf}} \right)^2} = 3,25 \left(\frac{30 \cdot 10^4 + 91 \cdot 10^4}{12} \right)^{1/3} \sqrt{1 - \left(\frac{124 \cdot 1,1}{235} \right)^2} = 123,2\ mm$$

3.1.2 Verification

Reference: 5.6.3; [5.7.3]

$$R_{y.Rd} = s_y\ t_w\ \frac{f_{yw}}{\gamma_{M1}} = 123,2 \cdot 12 \cdot \frac{235}{1,1} \cdot 10^{-3} = 315,8\ kN > 200\ kN$$

3.2 Crippling resistance

Reference: 5.6.4.(1); [5.7.4.(1)]

$s_s = 0$

$$R_{a.Rd} = 0,5\ t_w^2 \sqrt{E\ f_{yw}} \left[\sqrt{\frac{t_f}{t_w}} + 3\ \frac{t_w}{t_f}\ \frac{s_s}{d} \right] / \gamma_{M1} = 0,5 \cdot 12^2 \sqrt{210000 \cdot 235} \left[\sqrt{\frac{20}{12}} \right] \cdot 10^{-3} / 1,1 = 593,6\ kN > 200\ kN$$

$M_{c.Rd} = W_{el} \cdot f_y/\gamma_{Mo} = 2188 \cdot 10^3 \cdot 235 \cdot 10^{-6}/1,1 = 467,4\ kNm > 366,0\ kNm$

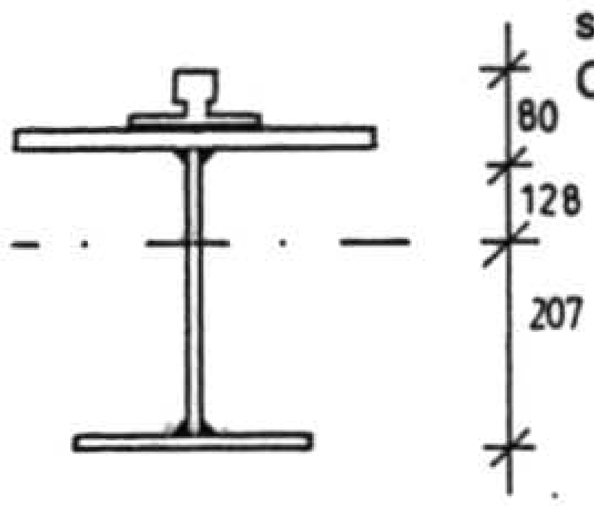
3.2.1 Interaction

Reference: 5.6.4.(2); [5.7.4.(2)]

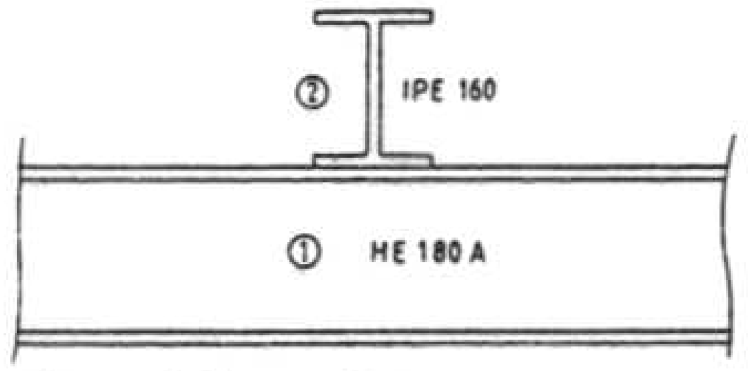
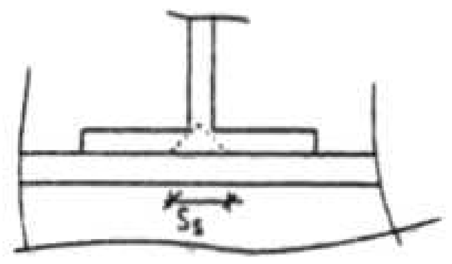
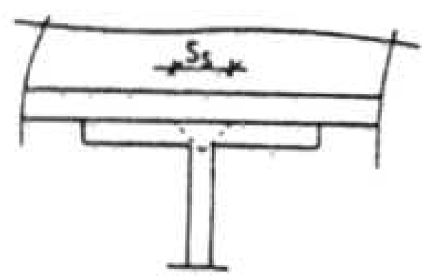
Commentary: Where the member is subject to bending moments, the interaction criterion has to be fulfilled

$$\frac{F_{Sd}}{R_{a.Rd}} + \frac{M_{Sd}}{M_{c.Rd}} = \frac{200,0}{593,6} + \frac{366,0}{467,4} = 1,12 < 1,5$$

Problem, Sketch, Calculation	Reference	Commentary
3.3 Buckling resistance **3.3.1 Determination of the effective breadth of the web** $b_{eff} = h = 360$ mm	5.6.5; [5.7.5]	Where forces are applied through one flange and resisted by shear forces in the web, the buckling resistance of the web to transverse forces need not be considered.
3.3.2 Buckling resistance verification Assumption: buckling length $\ell = 0{,}360$ m $A = 360 \cdot 12 = 43{,}2 \cdot 10^2$ mm^2 ; $I = 360 \cdot 12^3/12 = 5{,}184 \cdot 10^4$ mm^4 $i = \sqrt{5{,}184 \cdot 10^4 / 43{,}2 \cdot 10^2} = 3{,}5$ mm $\bar{\lambda} = \frac{\ell}{i}\frac{1}{\lambda_1} = \frac{0{,}36 \cdot 10^3}{3{,}5}\frac{1}{93{,}9} = 1{,}10$	5.6.5; [5.7.5]	When the detail is such that there is a doubt as to whether buckling mode governs, it should be considered.
using buckling curve c $\longrightarrow \chi_c = 0{,}4842$	5.4.2; [5.5.1]	
$R_{b.Rd} = \chi_c A \frac{f_{yw}}{\gamma_{M1}} = 0{,}4842 \cdot 43{,}2 \cdot 10^2 \cdot \frac{235}{1{,}1} \cdot 10^{-3} = 446{,}9 \text{ kN} > 200 \text{ kN}$		
3.4 Cross-section resistance verification Compression: $\sigma_{x.Ed} = M_{Sd} \cdot z/I = 366 \cdot 10^6 \cdot 128/45296 \cdot 10^4 = 103 \text{ N/mm}^2 < 214 \text{ N/mm}^2 = 235/1{,}1 = f_y/\gamma_{Mo}$ $s_s = 2 \cdot 80 = 160$ mm Compression: $\sigma_{z.Ed} = N_{Sd}/(s_s t_w) = 200 \cdot 10^3/(160 \cdot 12) = 104{,}2 \text{ N/mm}^2$	-; [5.4.10]	The effect of the transverse force on the bending moment resistance of the member should also be considered.



$$\left(\frac{\sigma_{x.Ed}}{f_{yd}}\right)^2 + \left(\frac{\sigma_{z.Ed}}{f_{yd}}\right)^2 - \frac{\sigma_{x.Ed}}{f_{yd}}\frac{\sigma_{z.Ed}}{f_{yd}} = \left(\frac{103}{214}\right)^2 + \left(\frac{104{,}2}{214}\right)^2 - \frac{103}{214}\frac{104{,}2}{214} = 0{,}23 + 0{,}24 - 0{,}48 \cdot 0{,}49 = 0{,}23 < 1$$

Problem, Sketch, Calculation	Reference	Commentary
Example 2.4.5: Beam supported by a beam		
1. System 		
2. Loading Reaction force: R_{sd} = 60 kN Bending moment: Beam "1": 55 kNm Beam "2": 16 kNm		
3. Determination of the resistance **3.1 Crushing resistance** **3.1.1 Beam "1" (HE 180 A)**		For rolled sections: $s_s = t_w + 2t_f + 4r(1 - \sqrt{2}/2)$
3.1.1.1 Bearing stiff length $s_s = 5{,}0 + 2 \cdot 7{,}4 + 4 \cdot 9 \cdot (1 - \sqrt{2}/2) = 30{,}3$ mm	5.6.2; [5.7.2.(3)]	
		
3.1.1.2 Determination of s_y $\sigma_{f.Ed} = M_{Sd}/W_{el} = 55 \cdot 10^6/294 \cdot 10^3 = 187$ N/mm² $b_f = 180$ mm $< 237{,}5$ mm $= 25t_f$	5.6.3; [5.7.3.(1)]	
$s_y = 2t_f \sqrt{\frac{b_f}{t_w}\frac{f_{yf}}{f_{yw}}\left[1 - \left(\frac{\sigma_{f.Ed}\,\gamma_{M0}}{f_{yf}}\right)^2\right]} = 2 \cdot 9{,}5 \sqrt{\frac{180}{6}\left[1 - \left(\frac{187 \cdot 1{,}1}{235}\right)^2\right]} = 50{,}3$ mm		
3.1.1.3 Verification	5.6.3; [5.7.3.(1)]	
$R_{y.Rd} = (s_s + s_y)\, t_w \frac{f_{yw}}{\gamma_{M1}} = (30{,}3 + 50{,}3) \cdot 6 \cdot \frac{235}{1{,}1} \cdot 10^{-3} = 103{,}3$ kN > 60 kN		
3.1.2 Beam "2" (IPE 160)		For rolled sections: $s_s = t_w + 2t_f + 4r(1 - \sqrt{2}/2)$
3.1.2.1 Bearing stiff length $s_s = 6{,}0 + 2 \cdot 9{,}5 + 4 \cdot 15 \cdot (1 - \sqrt{2}/2) = 42{,}6$ mm	5.6.2; [5.7.2.(3)]	
		
3.1.2.2 Determination of s_y $\sigma_{f.Ed} = M_{Sd}/W_{el} = 16 \cdot 10^6/109 \cdot 10^3 = 147$ N/mm² $b_f = 82$ mm < 185 mm $= 25t_f$	5.6.3; [5.7.3.(1)]	
$s_y = 2t_f \sqrt{\frac{b_f}{t_w}\frac{f_{yf}}{f_{yw}}\left[1 - \left(\frac{\sigma_{f.Ed}\,\gamma_{M0}}{f_{yf}}\right)^2\right]} = 2 \cdot 7{,}4 \sqrt{\frac{82}{5}\left[1 - \left(\frac{147 \cdot 1{,}1}{235}\right)^2\right]} = 43{,}5$ mm		
3.1.2.3 Verification	5.6.3; [5.7.3.(1)]	
$R_{y.Rd} = (s_s + s_y)\, t_w \frac{f_{yw}}{\gamma_{M1}} = (42{,}6 + 43{,}5) \cdot 5 \cdot \frac{235}{1{,}1} \cdot 10^{-3} = 92{,}0$ kN > 60 kN		

Problem, Sketch, Calculation	Reference	Commentary

3.2 Crippling resistance

3.2.1 Beam "1" (HE 180 A)

Reference: 5.6.4.(1) [5.7.4.(1)]

$s_s/d = 30{,}3/122 = 0{,}25 > 0{,}2 \longrightarrow s_s/d = 0{,}2$

$$R_{a.Rd} = 0{,}5\, t_w^2 \sqrt{E\, f_{yw}} \left[\sqrt{\frac{t_f}{t_w}} + 3\,\frac{t_w}{t_f}\,\frac{s_s}{d}\right] / \gamma_{M1} = 0{,}5\cdot 6^2 \sqrt{210000\cdot 235}\left[\sqrt{\frac{9{,}5}{6}} + 3\,\frac{6}{9{,}5}\,0{,}2\right]\cdot 10^{-3} / 1{,}1 = 188{,}2 \text{ kN} > 60 \text{ kN}$$

$M_{c.Rd} = W_{pl}\cdot f_y/\gamma_{Mo} = 325\cdot 10^3\cdot 235\cdot 10^{-6}/1{,}1 = 69{,}4 \text{ kNm} > 55 \text{ kNm}$

3.2.1.1 Interaction

Reference: 5.6.4.(2) [5.7.4.(2)]

$$\frac{F_{Sd}}{R_{a.Rd}} + \frac{M_{Sd}}{M_{c.Rd}} = \frac{60{,}0}{188{,}2} + \frac{55{,}0}{69{,}4} = 1{,}11 < 1{,}5$$

3.2.2 Beam "2" (IPE 160)

Reference: 5.6.4.(1) [5.7.4.(1)]

$s_s/d = 42{,}6/127 = 0{,}34 > 0{,}2 \longrightarrow s_s/d = 0{,}2$

$$R_{a.Rd} = 0{,}5\, t_w^2 \sqrt{E\, f_{yw}} \left[\sqrt{\frac{t_f}{t_w}} + 3\,\frac{t_w}{t_f}\,\frac{s_s}{d}\right] / \gamma_{M1} = 0{,}5\cdot 5^2 \sqrt{210000\cdot 235}\left[\sqrt{\frac{7{,}4}{5}} + 3\,\frac{5}{7{,}4}\,0{,}2\right]\cdot 10^{-3} / 1{,}1 = 129{,}5 \text{ kN} > 60 \text{ kN}$$

$M_{c.Rd} = W_{pl}\cdot f_y/\gamma_{Mo} = 124\cdot 10^3\cdot 235\cdot 10^{-6}/1{,}1 = 26{,}5 \text{ kNm} > 16 \text{ kNm}$

3.2.2.1 Interaction

Reference: 5.6.4.(2) [5.7.4.(2)]

Commentary: **Here:** Local buckling does not need to be checked

$$\frac{F_{Sd}}{R_{a.Rd}} + \frac{M_{Sd}}{M_{c.Rd}} = \frac{60{,}0}{129{,}5} + \frac{16{,}0}{26{,}5} = 1{,}07 < 1{,}5$$

3.3 Cross-section verification

3.3.1 Beam "1" (HE 180 A)

Reference: -; [5.4.10]

Assumption: no interaction bending shear is required

Compression: $\sigma_x = M_{Sd}/W_{el} = 55\cdot 10^6/294\cdot 10^3\cdot 76/85{,}5 = 166 \text{ N/mm}^2 < 214 \text{ N/mm}^2 = 235/1{,}1 = f_y/\gamma_{Mo}$

Compression: $\sigma_z = N_{Sd}/(s_s t_w) = 60\cdot 10^3/((30{,}3 + 2\cdot 9{,}5)\cdot 6) = 202{,}8 \text{ N/mm}^2 < 214 \text{ N/mm}^2$

$$\left(\frac{\sigma_{x.Ed}}{f_{yd}}\right)^2 + \left(\frac{\sigma_{z.Ed}}{f_{yd}}\right)^2 - \frac{\sigma_{x.Ed}}{f_{yd}}\,\frac{\sigma_{z.Ed}}{f_{yd}} = \left(\frac{166}{214}\right)^2 + \left(\frac{203}{214}\right)^2 - \frac{166}{214}\,\frac{203}{214} = 0{,}60 + 0{,}90 - 0{,}78\cdot 0{,}95 = 0{,}76 < 1$$

3.3.2 Beam "1" (IPE 160)

Reference: -; [5.4.10]

Assumption: no interaction bending shear is required

Compression: $\sigma_x = M_{Sd}/W_{el} = 16\cdot 10^6/109\cdot 10^3\cdot 72{,}6/80 = 133 \text{ N/mm}^2 < 214 \text{ N/mm}^2 = 235/1{,}1 = f_y/\gamma_{Mo}$

Compression: $\sigma_z = N_{Sd}/(s_s t_w) = 60\cdot 10^3/((42{,}6 + 2\cdot 7{,}4)\cdot 5) = 209 \text{ N/mm}^2 < 214 \text{ N/mm}^2$

$$\left(\frac{\sigma_{x.Ed}}{f_{yd}}\right)^2 + \left(\frac{\sigma_{z.Ed}}{f_{yd}}\right)^2 - \frac{\sigma_{x.Ed}}{f_{yd}}\,\frac{\sigma_{z.Ed}}{f_{yd}} = \left(\frac{133}{214}\right)^2 + \left(\frac{209}{214}\right)^2 - \frac{133}{214}\,\frac{209}{214} = 0{,}39 + 0{,}95 - 0{,}62\cdot 0{,}98 = 0{,}73 < 1$$

Part 3

Bolted connections
Welded connections
Pin connections

3.1 Bolted connections

These examples demonstrate the verification of bolted connections assuming design values of action effects (N_{Sd}, V_{Sd}, M_{Sd}, etc) which have been calculated by an analysis of the structure and these values already include ψ, γ_F etc.
The results presented in the examples are rounded values. For the purpose of easy re-calculation each formula and each check is calculated with the rounded values.

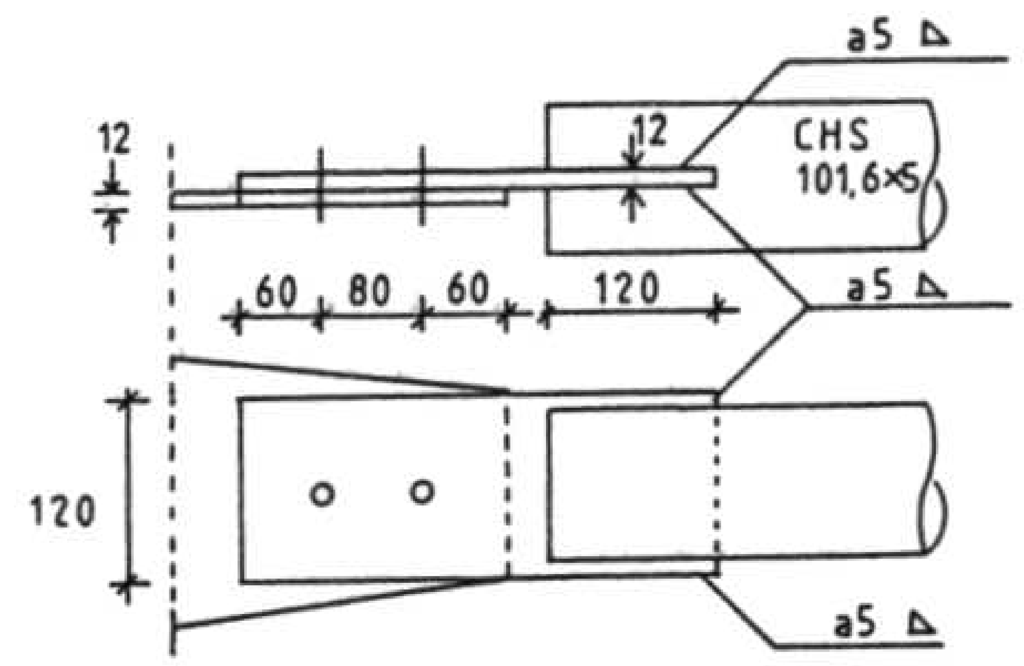
The following examples are included in this chapter:

Example 3.1.1:	Bolted connection of a tension member to a gusset plate
Example 3.1.2:	Erection splice at mid span of a lattice girder
Example 3.1.3:	Angle connected to a gusset plate
Example 3.1.4:	Fin plate connection to H section column
Example 3.1.5:	Fin plate connection to RHS column
Example 3.1.6:	Flexible end plate connection
Example 3.1.7:	Beam to beam connection with cleats
Example 3.1.8:	Splice of an unsymmetrical I-section
Example 3.1.9:	Bolted end plate connection

Problem, Sketch, Calculation	Reference	Commentary

Example 3.1.1: Bolted connection of a tension bar on a gusset plate

1. System



Member:	CHS 101,6 x 5,0	Steel grade:	Fe 360
Plate:	120 x 12	Steel grade:	Fe 360
Gusset plate:	120 x 12	Steel grade:	Fe 360
Bolts:	M 24, 8.8		
	$\longrightarrow d_o = 26$ mm		
weld length:	$\ell = 120$ mm		
throat thickness:a	= 5 mm		

Reference: Table 6.7;[7.5.2]

2. Loading

Tension force: $N_{Sd} = 260$ kN

3. Determination of the resistance of the members

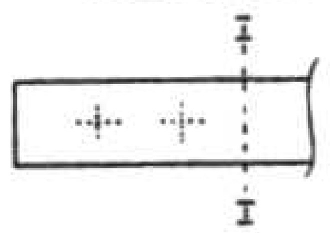
3.1 Gross section (Circular hollow section)

Reference: 5.5.1; [5.4.3]

$$N_{pl.Rd} = \frac{A \cdot f_y}{\gamma_{Mo}} = \frac{15{,}17 \cdot 10^2 \cdot 235}{1{,}1} \cdot 10^{-3} = 324{,}1 \text{ kN} > 260 \text{ kN}$$

3.2 Gross section (Plate and gusset plate)

Reference: 5.5.1; [5.4.3]

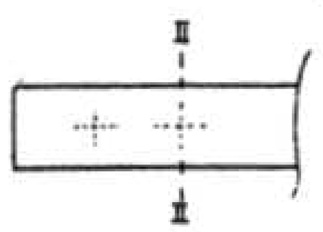


$$N_{pl.Rd} = \frac{A \cdot f_y}{\gamma_{Mo}} = \frac{120 \cdot 12 \cdot 235}{1{,}1} \cdot 10^{-3} = 307{,}6 \text{ kN} > 260 \text{ kN}$$

3.3 Net section (Plate and gusset plate)

Reference: 5.5.1; [5.4.3]

$$A_{net} = (120 - 26) \cdot 12 = 11{,}3 \cdot 10^2 \text{ mm}^2$$



$$N_{u.Rd} = 0{,}9 \frac{A_{net} \cdot f_u}{\gamma_{M2}} = 0{,}9 \frac{11{,}3 \cdot 10^2 \cdot 360}{1{,}25} \cdot 10^{-3} = 292{,}9 \text{ kN} > 260 \text{ kN}$$

4. Determination of the connection resistance

4.1 Bolted connection

Commentary: **Assumption:** Shear in thread

Reference: [6.5.5; Table 6.5.3]

Shear resistance of bolts: $$F_{v.Rd} = \frac{0{,}6\, f_{ub}\, A_s}{\gamma_{Mb}} = \frac{0{,}6 \cdot 800 \cdot 3{,}53 \cdot 10^2}{1{,}25} \cdot 10^{-3} = 135{,}6 \text{ kN} > \frac{260}{2} = 130{,}0 \text{ kN}$$

Reference: 6.2.3.2; Table 6.7

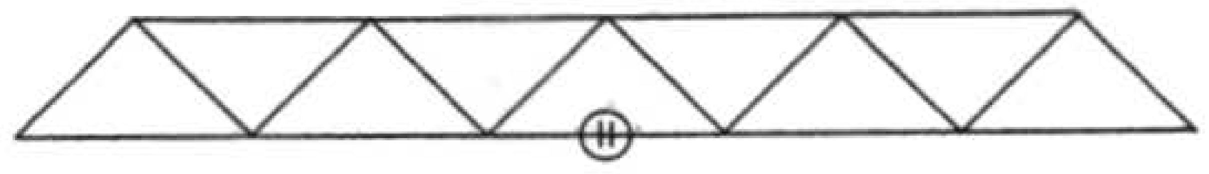
$$F_{v.Rd} = \frac{169{,}4}{1{,}25} = 135{,}5 \text{ kN} > \frac{260}{2} = 130{,}0 \text{ kN}$$

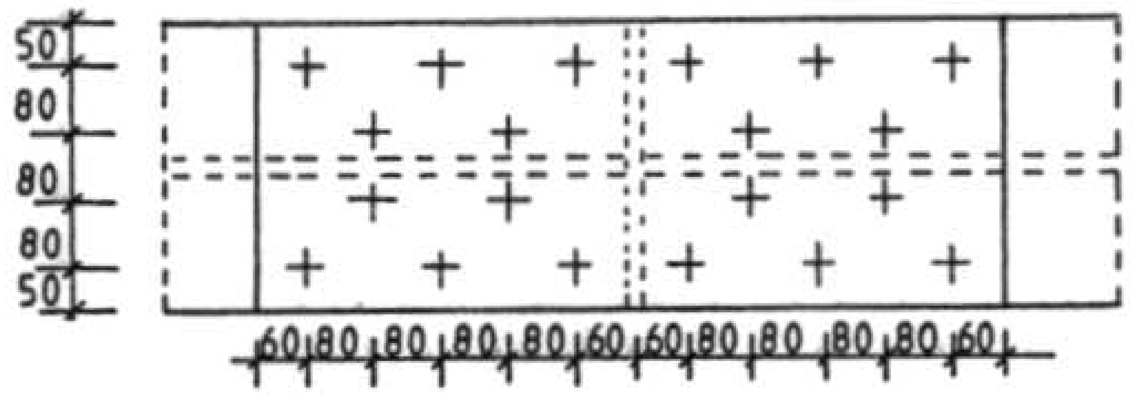
Commentary: The eccentricity Δe is covered by the design resistances for bolts and need not to be taken into account

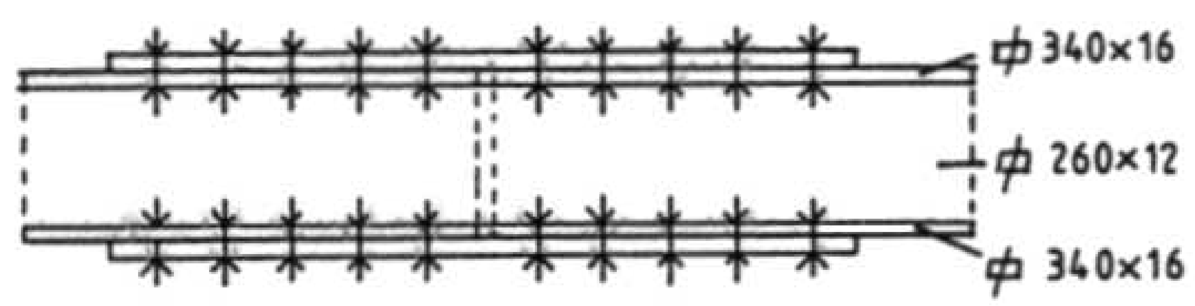
Problem, Sketch, Calculation	Reference	Commentary
Bearing resistance: $e_1/3d_o = 60/(3\cdot 26) = 0,77$; $p_1/3d_o - 1/4 = 80/(3\cdot 26) - 1/4 = 0,77$; $\longrightarrow \alpha = 0,77$		The shear resistance is governing ($F_{v.Rd} < F_{b.Rd}$). If high deformation capacity is needed, the shear resistance must be increased.
$F_{b.Rd} = \dfrac{2,5\,\alpha\, f_u d t}{\gamma_{Mb}} = \dfrac{2,5\cdot 0,77\cdot 360\cdot 24\cdot 12}{1,25}\cdot 10^{-3} = 159,7\ \text{kN} > \dfrac{260}{2} = 130,0\ \text{kN}$	[6.5.5; Table 6.5.3]	
$F_{b.Rd} = \dfrac{166,2}{1,25}\cdot\dfrac{12}{10} = 159,6\ \text{kN} > \dfrac{260}{2} = 130,0\ \text{kN}$	6.2.3.1; Table 6.6	
4.2 Welded connection (Plate - CHS) Resistance of the fillet weld		
$F_{w.Rd} = 4\,\dfrac{f_u}{\sqrt{3}\,\beta_w}\,\dfrac{a\,L}{\gamma_{Mw}} = 4\cdot\dfrac{360}{\sqrt{3}\cdot 0,8}\cdot\dfrac{5\cdot 120}{1,25}\cdot 10^{-3} = 498,8\ \text{kN} > 260\ \text{kN}$	[6.6.5.3]	
$F_{w.Rd} = 4\cdot\dfrac{129,9}{1,25}\cdot\dfrac{120}{100} = 498,8\ \text{kN} > 260\ \text{kN}$	6.3.4.3; Table 6.15	
Local shear resistance for CHS		CHS is critical for local shear failure. The verification for the plate may be neglected.
$V_{pl.Rd} = 4\,t\,L\,\dfrac{f_y}{\sqrt{3}\cdot\gamma_{Mo}} = 4\cdot 5\cdot 120\cdot\dfrac{235}{\sqrt{3}\cdot 1,1}\cdot 10^{-3} = 296,0\ \text{kN} > 260,0\ \text{kN}$	5.3.3; [5.4.6]	

Problem, Sketch, Calculation	Reference	Commentary

Example 3.1.2: Erection splice at mid span of a lattice girder







1. System

Member:	Welded section	
	Steel grade:	Fe 510
	see sketch	
Splice plates:	880 x 340 x 16 mm thick	
	Steel grade:	Fe 510
Bolts:	M 24, 8.8	
	$\longrightarrow d_o = 26$ mm	

Reference: Table 6.7; [7.5.2]

2. Loading

Tension force: $N_{Sd} = 3000$ kN

3. Determination of the resistance of the members

3.1 Gross section I - I

Reference: 5.5.1; [5.4.3]

Welded section:

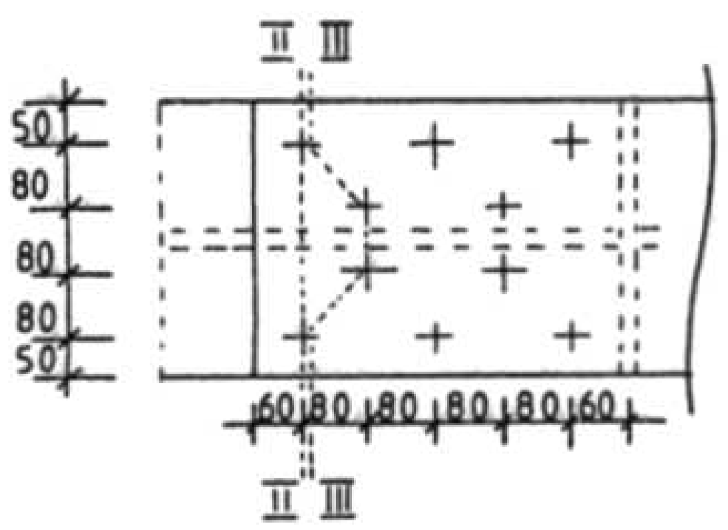
$A = 340 \cdot 16 \cdot 2 + 260 \cdot 12 = 140{,}0 \cdot 10^2\ \text{mm}^2$

Splice plates:

$A = 2 \cdot 340 \cdot 16 = 108{,}8 \cdot 10^2\ \text{mm}^2$

$$N_{pl.Rd} = \frac{A \cdot f_y}{\gamma_{Mo}} = \frac{108{,}8 \cdot 10^2 \cdot 355}{1{,}1} \cdot 10^{-3} = 3511{,}3\ \text{kN} > 3000\ \text{kN}$$

3.2 Net section



$\Delta A_{II} = 4 \cdot 26 \cdot 16 = 16{,}64 \cdot 10^2\ \text{mm}^2$

Reference: - ;[5.4.2.2.(4)]

$$\Delta A_{III} = \sum \Delta A_i - \sum \frac{s_i^2\, t}{4\, p_i} = \left(26 \cdot 16 \cdot 4 - 2 \cdot \frac{80^2 \cdot 16}{4 \cdot 80}\right) \cdot 2 = 20{,}48 \cdot 10^2\ \text{mm}^2$$

$A_{net} = A - \max(\Delta A_{II}\ ;\ \Delta A_{III}) = 108{,}8 \cdot 10^2 - 20{,}48 \cdot 10^2 = 88{,}32 \cdot 10^2\ \text{mm}^2$

$$N_{u.Rd} = 0{,}9\ \frac{A_{net} \cdot f_u}{\gamma_{M2}} = 0{,}9\ \frac{88{,}32 \cdot 10^2 \cdot 510}{1{,}25} \cdot 10^{-3} = 3243{,}1\ \text{kN} > 3000\ \text{kN}$$

Reference: 5.5.1; [5.4.3]

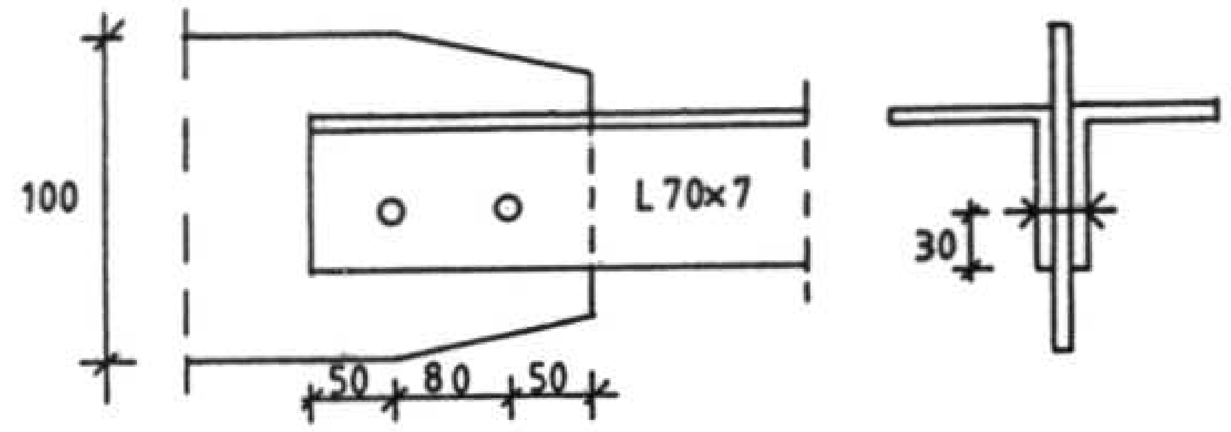
Commentary: If $A_{net} \le 1{,}11\ \frac{f_y\ \gamma_{M2}}{f_u\ \gamma_{M0}}\ A$ then only net section verification is necessary.

Problem, Sketch, Calculation	Reference	Commentary
4. Determination of the connection resistance **4.1 Bolted connection**		**Assumption:** Shear in shank $\longrightarrow A = \pi d^2/4 = 4{,}52 \cdot 10^2\ mm^2$
Shear resistance of bolts: $F_{v.Rd} = \frac{0{,}6\ f_{ub}\ A}{\gamma_{Mb}} = \frac{0{,}6 \cdot 800 \cdot 4{,}52 \cdot 10^2}{1{,}25} \cdot 10^{-3} = 173{,}6\ kN > \frac{3000}{2 \cdot 10} = 150{,}0\ kN$	6.2.3.2 [6.5.5; Table 6.5.3]	
Bearing resistance: $e_1/3d_o = 60/(3 \cdot 26) = 0{,}77$; $p_1/3d_o - 1/4 = 160/(3 \cdot 26) - 1/4 = 1{,}8$; $\longrightarrow \alpha = 0{,}77$ $F_{b.Rd} = \frac{2{,}5 \alpha f_u d t}{\gamma_{Mb}} = \frac{2{,}5 \cdot 0{,}77 \cdot 510 \cdot 24 \cdot 16}{1{,}25} \cdot 10^{-3} = 301{,}6\ kN > 150{,}0\ kN$	[6.5.5; Table 6.5.3]	
$F_{b.Rd} = \frac{235{,}4}{1{,}25} \cdot \frac{16}{10} = 301{,}3\ kN > 150{,}0\ kN$	6.2.3.1; Table 6.6	

Problem, Sketch, Calculation	Reference	Commentary

Example 3.1.3: Angle connected on a gusset plate

In the case of joints with angles connected by at least two bolts, the setting out lines for the bolts in the angles may be substituted for the centroidal axes for the purpose of intersection at the joint.



1. System

Member: 2 L 70 x 7		Steel grade:	Fe 430
Plate:	100 x 15 mm	Steel grade:	Fe 430
Bolts:	M 20, 8.8		
	$\longrightarrow d_o = 22$ mm		

Table 6.7; [7.5.2]

2. Loading

Tension force: $N_{Sd} = 285$ kN

3. Determination of the resistance

3.1 Gross section (angle)

5.5.1; [5.4.3]

$$N_{pl.Rd} = \frac{A \cdot f_y}{\gamma_{Mo}} = 2 \cdot \frac{9{,}4 \cdot 10^2 \cdot 275}{1{,}1} \cdot 10^{-3} = 470{,}0 \text{ kN} > 285 \text{ kN}$$

3.2 Gross section (plate)

5.5.1; [5.4.3]

$$N_{pl.Rd} = \frac{A \cdot f_y}{\gamma_{Mo}} = \frac{100 \cdot 15 \cdot 275}{1{,}1} \cdot 10^{-3} = 375{,}0 \text{ kN} > 285 \text{ kN}$$

3.3 Net section (Angle)

$A_{net} = 9{,}4 \cdot 10^2 - 22 \cdot 7 = 7{,}86 \cdot 10^2$ mm²

Determination of β_2 by linear interpolation:

$$\beta_2 = 0{,}4 + \frac{0{,}3}{5 \cdot 22 - 2{,}5 \cdot 22}(80 - 2{,}5 \cdot 22) = 0{,}536$$

5.5.1; Table 5.33 [5.4.3; 6.5.2.3]

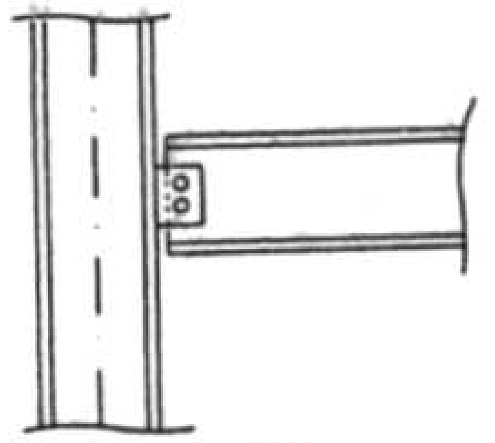
Angles connected by a single row of bolts in one leg may be treated as concentrically loaded if the design ultimate resistance is determined as 5.5.1.(2); [6.5.2.3], i.e. the effect of eccentricities is included in the joint design.

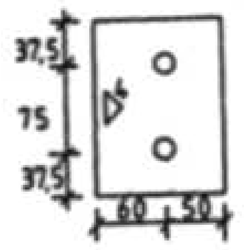
$$N_{u.Rd} = \beta_2 \frac{A_{net} \cdot f_u}{\gamma_{M2}} = 2 \cdot 0{,}536 \frac{7{,}86 \cdot 10^2 \cdot 430}{1{,}25} \cdot 10^{-3} = 289{,}9 \text{ kN} > 285{,}0 \text{ kN}$$

Problem, Sketch, Calculation	Reference	Commentary
4. Determination of the connection resistance **4.1 Bolted connection**		**Here:** Two shear planes **Assumption:** Shear in thread
Shear resistance of bolts: $F_{v.Rd} = 2\,\frac{0{,}6\,f_{ub}\,A_s}{\gamma_{Mb}} = 2\,\frac{0{,}6\cdot 800\cdot 2{,}45\cdot 10^2}{1{,}25}\cdot 10^{-3} = 188{,}2\ \text{kN} > \frac{285}{2} = 142{,}5\ \text{kN}$	[6.5.5; Table 6.5.3]	
or $F_{v.Rd} = 2\,\frac{117{,}6}{1{,}25} = 188{,}2\ \text{kN} > \frac{285}{2} = 142{,}5\ \text{kN}$	6.2.3.2; Table 6.7	
Bearing resistance: Plate: $e_1/3d_o = 50/3\cdot 22 = 0{,}76$; $p_1/3d_o - 1/4 = 80/3\cdot 22 - 1/4 = 0{,}96$; $\longrightarrow \alpha = 0{,}76$		If the end distance is equal then only one calculation is necessary.
$F_{b.Rd} = \frac{2{,}5\,\alpha\, f_u\, d\, t}{\gamma_{Mb}} = \frac{2{,}5\cdot 0{,}76\cdot 430\cdot 20\cdot 15}{1{,}25}\cdot 10^{-3} = 196{,}1\ \text{kN} > \frac{285}{2} = 142{,}5\ \text{kN}$	[6.5.5; Table 6.5.3] 6.2.3.1; Table 6.6	
Angles: $e_1/3d_o = 50/3\cdot 22 = 0{,}76$; $p_1/3d_o - 1/4 = 80/3\cdot 22 - 1/4 = 0{,}96$; $\longrightarrow \alpha = 0{,}76$		The bearing resistance is calculated per member (**here:** per angle).
$e_2 = 30\ \text{mm} = 1{,}36 d_o < 1{,}5 d_o$ $\longrightarrow$ Reduction of the bearing resistance required		
Bearing resistance without reduction:	[6.5.5; Table 6.5.3]	
$F_{b.Rd} = \frac{2{,}5\,\alpha\, f_u\, d\, t}{\gamma_{Mb}} = \frac{2{,}5\cdot 0{,}76\cdot 430\cdot 20\cdot 7}{1{,}25}\cdot 10^{-3} = 91{,}5\ \text{kN}$	6.2.3.1; Table 6.6	
or $F_{b.Rd} = \frac{162{,}9}{1{,}25}\,\frac{7}{10} = 91{,}2\ \text{kN}$		
Reduktion of the bearing resistance:	[6.5.5(10)]	
$F_{b.Rd} = 91{,}5\cdot\left(\frac{2}{3} + \frac{1{,}36 - 1{,}20}{1{,}50 - 1{,}20}\cdot\frac{1}{3}\right) = 77{,}3\ \text{kN} > \frac{285}{2\cdot 2} = 71{,}3\ \text{kN}$		

Problem, Sketch, Calculation	Reference	Commentary

Example 3.1.4: Fin plate connection with supporting member as an I-section

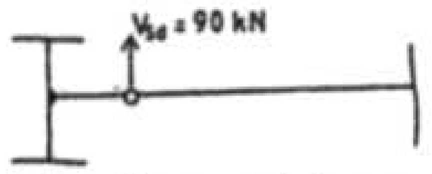




1. System:

Column member:	HE 200 B	Steel grade:	Fe 430
Beam member:	IPE 200	Steel grade:	Fe 510
Fin plate:	150 x 110 x 10	Steel grade:	Fe 510
Bolts:	M 20, 8.8		
	$\longrightarrow d_o = 22$ mm		

Commentary: Fin plate connections are assumed as hinged connections. The location of the hinge is controlled by the stiffness of the supporting member.
Here: The column web is a rigid support of the fin plate. Therefore it may be assumed that the hinge occurs at the bolts.



2. Loading

Shear force: $V_{Sd} = 90$ kN

3. Determination of the resistance of the connection:

3.1 Design resistance of the connection (fin plate - beam)

Commentary: **Assumption:** Shear in thread

Shear resistance of bolts: [6.5.5; Table 6.5.3]

$$F_{v.Rd} = \frac{0{,}6\, f_{ub}\, A_s}{\gamma_{Mb}} = \frac{0{,}6 \cdot 800 \cdot 2{,}45 \cdot 10^2}{1{,}25} \cdot 10^{-3} = 94{,}1 \text{ kN} > \frac{90}{2} = 45 \text{ kN}$$

or 6.2.3.2; Table 6.7

$$F_{v.Rd} = \frac{117{,}6}{1{,}25} = 94{,}1 \text{ kN} > \frac{90}{2} = 45 \text{ kN}$$

Bearing resistance:

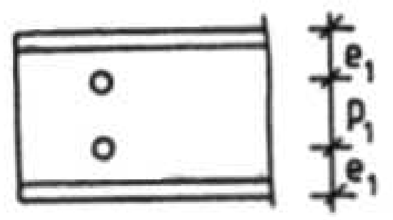
Fin plate: 6.2.3.1; Table 6.6 [6.5.5; Table 6.5.3]

$$e_1/3d_o = 37{,}5/3 \cdot 22 = 0{,}57 \;;\; p_1/3d_o - 1/4 = 75/3 \cdot 22 - 1/4 = 0{,}89 \;;\; \longrightarrow \alpha = 0{,}57$$

$$F_{b.Rd} = \frac{2{,}5\, \alpha\, f_u\, d\, t}{\gamma_{Mb}} = \frac{2{,}5 \cdot 0{,}57 \cdot 510 \cdot 20 \cdot 10}{1{,}25} \cdot 10^{-3} = 116{,}3 \text{ kN} > 45 \text{ kN}$$

Commentary: The edge distance e_1 should be taken as the minimum value of both sides when the acting force effects not parallel to one of both edge distances

Beam web:



$$p_1/3d_o - 1/4 = 75/3 \cdot 22 - 1/4 = 0{,}89 \;;\; \longrightarrow \alpha = 0{,}89$$

$$F_{b.Rd} = \frac{2{,}5\, \alpha\, f_u\, d\, t}{\gamma_{Mb}} = \frac{2{,}5 \cdot 0{,}89 \cdot 510 \cdot 20 \cdot 5{,}6}{1{,}25} \cdot 10^{-3} = 101{,}7 \text{ kN} > 45 \text{ kN}$$

3.2 Shear resistance of the beam

$A_{v.net} = 11{,}2 \cdot 10^2 - 2 \cdot 22 \cdot 5{,}6 = 8{,}7 \cdot 10^2 \text{ mm}^2 \geq A_v \cdot f_y/f_u = 11{,}2 \cdot 10^2 \cdot 355/510 = 7{,}8 \cdot 10^2 \text{ mm}^2$

$\longrightarrow$ fastener holes need not be allowed for in shear verification

Commentary: $A_v = h \cdot t_w = 11{,}2 \cdot 10^2 \text{ mm}^2$
It is allowed to consider larger values by using the formulae given in EC 3 [5.4.6]

5.3.3; [5.4.6]

$$V_{pl.Rd} = A_v \frac{f_y}{\sqrt{3}\,\gamma_{Mo}} = 11{,}2 \cdot 10^2 \cdot \frac{355}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 208{,}7 \text{ kN} > 90{,}0 \text{ kN}$$

Problem, Sketch, Calculation	Reference	Commentary

3.3 Design resistance of fin plate

$A_{v,net} = 150 \cdot 10 - 2 \cdot 22 \cdot 5{,}6 = 12{,}5 \cdot 10^2\ mm^2 \geq A_v \cdot f_y/f_u = 15{,}0 \cdot 10^2 \cdot 355/510 = 10{,}4 \cdot 10^2\ mm^2$

$\longrightarrow$ fastener holes need not be allowed for in shear verification

Reference: 5.3.3; [5.4.6]

$$V_{pl.Rd} = A_v \frac{f_y}{\sqrt{3}\gamma_{Mo}} = 15{,}0 \cdot 10^2 \cdot \frac{355}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 279{,}5\ kN > 90{,}0\ kN$$

$V_{Sd} < 0{,}5 V_{pl.Rd} \longrightarrow$ no interaction bending - shear is required

$W_{el} = 150^2 \cdot 10/6 = 37{,}5 \cdot 10^3\ mm^3$

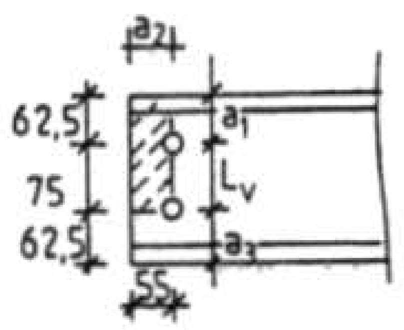
Reference: 5.3.2; [5.4.5]

$M_{Rd} = W_{el} f_y/\gamma_{Mo} = 37{,}5 \cdot 10^3 \cdot 355 \cdot 10^{-6}/1{,}1 = 12{,}1\ kNm > 5{,}4\ kNm$

3.4 Block shear resistance

Reference: 5.5.2; [6.5.2.2]

Commentary: The formulae given in table 5.34 of E-EC3 are not used.



$L_1 = a_1 = 62{,}5\ mm < 5d = 5 \cdot 20 = 100 \longrightarrow L_1 = 62{,}5\ mm$

$L_2 = (a_2 - kd_{o.t})(f_u/f_y) = (50 - 0{,}5 \cdot 22)510/355 = 56\ mm$

$L_3 = L_v + a_1 + a_3 = 75 + 62{,}5 + 62{,}5 = 200\ mm$

$< (L_v + a_1 + a_3 - nd_{o.v})(f_u/f_y) = (200 - 2 \cdot 22)510/355 = 224{,}1\ mm$

$L_{v.eff} = L_v + L_1 + L_2 = 75 + 62{,}5 + 56 = 193{,}5\ mm < L_3 = 200\ mm \longrightarrow L_{v.eff} = 193{,}5\ mm$

$$V_{eff.Rd} = L_{v.eff}\, t \frac{f_y}{\sqrt{3}\ \gamma_{Mo}} = 193{,}5 \cdot 5{,}6 \cdot \frac{355}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 201{,}9\ kN > 90\ kN$$

3.5 Resistance of the fillet weld

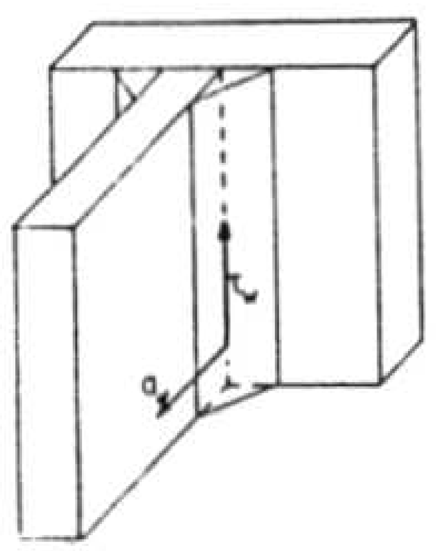
Commentary: elastic stress distribution is assumed

Action effects at fillet weld:

Shear force: $V_{Sd} = 90\ kN$

Bending moment: $M_{Sd} = 90 \cdot 60 \cdot 10^{-3} = 5{,}4\ kNm$

Commentary: The alternative method given in Annex M of Eurocode 3 may lead to a smaller weld size, however the calculation is more complex.



Geometrical data of the weld:

$a = 4\ mm$

$A_w = 2 \cdot 150 \cdot 4 = 12 \cdot 10^2\ mm^2$

$W_w = A_w \cdot h/6 = 12 \cdot 10^2 \cdot 150/6 = 30 \cdot 10^3\ mm^3$

Reference: 6.3.4.3; [6.6.5.3]

Weld stresses:

$\tau_w = V_{Sd}/A_w = 90 \cdot 10^3/12 \cdot 10^2 = 75\ N/mm^2$

$\sigma_w = M_{Sd}/W_w = 5{,}4 \cdot 10^6/30 \cdot 10^3 = 180\ N/mm^2$

$\sqrt{(\tau_w^2 + \sigma_w^2)} = \sqrt{(75^2 + 180^2)} = 195\ N/mm^2$

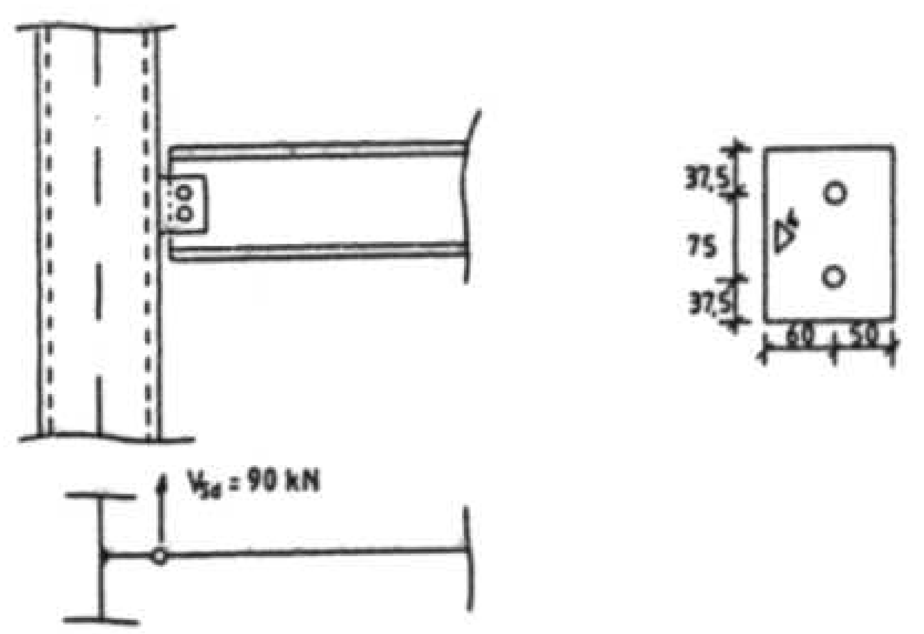
$$f_{vw.d} = \frac{f_u}{\sqrt{3}\ \beta_w\ \gamma_{Mw}} = \frac{430}{\sqrt{3} \cdot 0{,}85 \cdot 1{,}25} = 234\ N/mm^2 > 195\ N/mm^2$$

Problem, Sketch, Calculation	Reference	Commentary

Example 3.1.5: Fin plate connection with a supporting member as a RHS

Commentary: Fin plate connections are assumed as hinged connections. The location of the hinge is controlled by the stiffness of the supporting member.
Here: The column flange is a flexible support of the fin plate. Therefore it could be assumed that the hinge occurs at the welds.

However; in practice, most supports are neither fully rigid nor entirely flexible. Therefore it is safer to design both the bolts and the welds for shear force and moment. This principle may also be taken where bolted connections are used instead of weldings in fin plate connections.



1. System:

Column member:	RHS 200 x 8,0	Steel grade:	Fe 430
Beam member:	IPE 200	Steel grade:	Fe 510
Fin plate:	150 x 110 x 10	Steel grade:	Fe 510
Bolts:	M 20, 8.8	$\longrightarrow d_o = 22$ mm	

2. Loading

Shear force: $V_{Sd} = 90$ kN

3. Determination of the resistance of the connection:

Commentary: **Assumption:** Shear in thread

3.1 Design resistance of the connection (fin plate - beam)

Reference: [6.5.5; Table 6.5.3]

Bending moment: $M_{Sd} = 90 \cdot 60 \cdot 10^{-3} = 5{,}4$ kNm

Horizontal shear force per bolt: $M_{Sd}/75 \cdot 10^{-3} = 5{,}4/75 \cdot 10^{-3} = 72$ kN

Vertical shear force per bolt: $V_{Sd} = 90/2 = 45$ kN

Resultant shear force per bolt: $F_{Sd} = \sqrt{(45^2 + 72^2)} = 84{,}9$ kN

Reference: 6.2.3.2; Table 6.7

Shear resistance of bolts:
$$F_{v.Rd} = \frac{0{,}6\, f_{ub}\, A_s}{\gamma_{Mb}} = \frac{0{,}6 \cdot 800 \cdot 2{,}45 \cdot 10^2}{1{,}25} \cdot 10^{-3} = 94{,}1 \text{ kN} > 84{,}9 \text{ kN}$$

or
$$F_{v.Rd} = \frac{117{,}6}{1{,}25} = 94{,}1 \text{ kN} > 84{,}9 \text{ kN}$$

Reference: 6.2.3.1; Table 6.6 [6.5.5; Table 6.5.3]

Commentary: The edge distance e_1 should be taken as the minimum value of both sides when the acting force effects not parallel to one of both edge distances

Bearing resistance:

Fin plate: $e_1/3d_o = 37{,}5/3 \cdot 22 = 0{,}57$; $p_1/3d_o - 1/4 = 75/3 \cdot 22 - 1/4 = 0{,}89$; $\longrightarrow \alpha = 0{,}57$

$$F_{b.Rd} = \frac{2{,}5\,\alpha\, f_u d t}{\gamma_{Mb}} = \frac{2{,}5 \cdot 0{,}57 \cdot 510 \cdot 20 \cdot 10}{1{,}25} \cdot 10^{-3} = 116{,}3 \text{ kN} > 84{,}9 \text{ kN}$$

Beam web: $e_1/3d_o = 50/3 \cdot 22 = 0{,}76$; $p_1/3d_o - 1/4 = 75/3 \cdot 22 - 1/4 = 0{,}89$; $\longrightarrow \alpha = 0{,}76$

$$F_{b.Rd} = \frac{2{,}5\,\alpha\, f_u d t}{\gamma_{Mb}} = \frac{2{,}5 \cdot 0{,}76 \cdot 510 \cdot 20 \cdot 5{,}6}{1{,}25} \cdot 10^{-3} = 86{,}8 \text{ kN} > 84{,}9 \text{ kN}$$

3.2 Shear resistance of the beam

$A_{v.net} = 11{,}2 \cdot 10^2 - 2 \cdot 22 \cdot 5{,}6 = 8{,}7 \cdot 10^2 \text{ mm}^2 \geq A_v \cdot f_y/f_u = 11{,}2 \cdot 10^2 \cdot 355/510 = 7{,}8 \cdot 10^2 \text{ mm}^2$

$\longrightarrow$ fastener holes need not be allowed for in shear verification

Reference: 5.3.3; [5.4.6]

$$V_{pl.Rd} = A_v \frac{f_y}{\sqrt{3}\,\gamma_{Mo}} = 11{,}2 \cdot 10^2 \cdot \frac{355}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 208{,}7 \text{ kN} > 90{,}0 \text{ kN}$$

Problem, Sketch, Calculation	Reference	Commentary

3.3 Design resistance of fin plate

$A_{v.net} = 150 \cdot 10 - 2 \cdot 22 \cdot 5{,}6 = 12{,}5 \cdot 10^2\ mm^2 \geq A_v \cdot f_y/f_u = 15{,}0 \cdot 10^2 \cdot 355/510 = 10{,}4 \cdot 10^2\ mm^2$

$\longrightarrow$ fastener holes need not be allowed for in shear verification

Reference: 5.3.3; [5.4.6]

$$V_{pl.Rd} = A_v \frac{f_y}{\sqrt{3}\gamma_{Mo}} = 15{,}0 \cdot 10^2 \cdot \frac{355}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 279{,}5\ kN > 90{,}0\ kN$$

$V_{Sd} < 0{,}5V_{pl.Rd} \longrightarrow$ no interaction bending - shear is required

$W_{el} = 150^2 \cdot 10/6 = 37{,}5 \cdot 10^3\ mm^3$

Reference: 5.3.2; [5.4.5]

$M_{Rd} = W_{el} f_y/\gamma_{Mo} = 37{,}5 \cdot 10^3 \cdot 355 \cdot 10^{-6}/1{,}1 = 12{,}1\ kNm > 5{,}4\ kNm$

3.4 Block shear resistance

Reference: 5.5.2; [6.5.2.2]

$L_1 = a_1 = 62{,}5\ mm < 5d = 5 \cdot 20 = 100 \longrightarrow L_1 = 62{,}5\ mm$

$L_2 = (a_2 - kd_{o.t})(f_u/f_y) = (55 - 0{,}5 \cdot 22)510/355 = 63{,}2\ mm$

$L_3 = L_v + a_1 + a_3 = 75 + 62{,}5 + 62{,}5 = 200\ mm$

$< (L_v + a_1 + a_3 - nd_{o.v})(f_u/f_y) = (200 - 2 \cdot 22)510/355 = 224{,}1\ mm$

$L_{v.eff} = L_v + L_1 + L_2 = 75 + 62{,}5 + 63{,}2 = 200{,}7\ mm > L_3 = 200\ mm \longrightarrow L_{v.eff} = 200\ mm$

$$V_{eff.Rd} = L_{v.eff}\, t \frac{f_y}{\sqrt{3}\ \gamma_{Mo}} = 200 \cdot 5{,}6 \cdot \frac{355}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 208{,}7\ kN > 90\ kN$$

3.5 Resistance of the fillet weld

Commentary: Elastic stress distribution is assumed for the welds.
$a = 4\ mm$
$A_w = 2 \cdot 150 \cdot 4 = 12 \cdot 10^2\ mm^2$
$W_w = A_w \cdot h/6 = 12 \cdot 10^2 \cdot 150/6 = 30 \cdot 10^3\ mm^3$

Action effects at fillet weld:

Shear force: $V_{Sd} = 90\ kN$

Bending moment: $M_{Sd} = 90 \cdot 60 \cdot 10^{-3} = 5{,}4\ kNm$

Weld stresses:

$\tau_{\parallel} = V_{Sd}/A_w = 90 \cdot 10^3/12 \cdot 10^2 = 75\ N/mm^2$

$\sigma_{\perp} = M_{Sd}/W_w = 5{,}4 \cdot 10^6/30 \cdot 10^3 = 180\ N/mm^2$

$\sqrt{(\tau_{\parallel}^2 + \sigma_{\perp}^2)} = \sqrt{(75^2 + 180^2)} = 195\ N/mm^2$

Reference: 6.3.4.3; [6.6.5.3]

$$f_{vw.d} = \frac{f_u}{\sqrt{3}\ \beta_w\ \gamma_{Mw}} = \frac{430}{\sqrt{3} \cdot 0{,}85 \cdot 1{,}25} = 234\ N/mm^2 > 195\ N/mm^2$$

3.6 Joint resistance of fin plate welded to RHS - chord

Reference: [Annex K, revised version]

Commentary:
For compression: $k_n = 1{,}3 + 0{,}4/(t_i/b_o) \cdot n$
but $k_n \leq 1{,}0$
where $n = \sigma_o/f_{yo}$
Here: Assumption: $k_n = 1{,}0$
For tension: $k_n = 1{,}0$

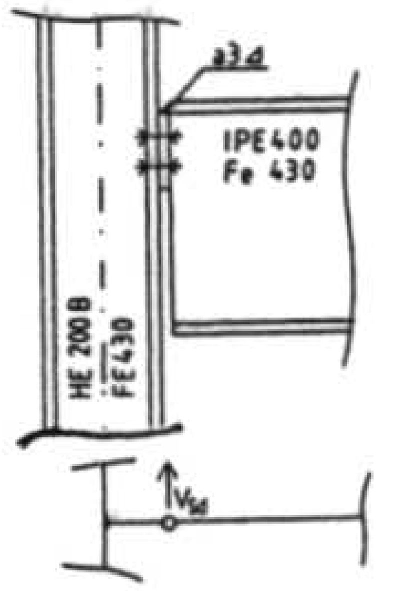
$$M_{ip.Rd} = \frac{f_{yo} t_o^2}{1 - \frac{t_i}{b_o}} \left(2\frac{h_i}{b_o} + 4 \cdot \sqrt{1 - \frac{t_i}{b_o}}\right) \frac{h_i}{2} k_n$$

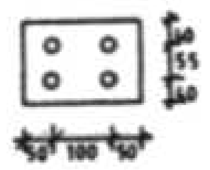
$$= \frac{275 \cdot 8{,}0^2}{1 - \frac{10}{200}} \left(2\frac{150}{200} + 4 \cdot \sqrt{1 - \frac{10}{200}}\right) \frac{150}{2} = 7{,}5\ kNm > 5{,}4\ kNm$$

Problem, Sketch, Calculation	Reference	Commentary

Example 3.1.6: Flexible end plate connection

From practical evidence flexible end plate connections are assumed as hinged connections.





t = 8 mm
Fe 430

1. System

Column member:	HE 200 B	Steel grade:	Fe 430
Beam member:	IPE 400	Steel grade:	Fe 430
End plate:	135x115x8	Steel grade:	Fe 430
Bolts:	M 16, 8.8		
	$\longrightarrow d_o = 18$ mm		

Reference: Table 6.7 [7.5.2]

2. Loading

Shear force: $V_{Sd} = 150$ kN
per bolt: $V_{Sd}/4 = 37{,}5$ kN

3. Determination of the resistance of the connection:

3.1 Weld resistance

throat thickness chosen to be a = 3 mm

Reference: 6.3.4.3; Table 6.15 [6.6.5.3]

$$F_{w.Rd} = \frac{f_u}{\sqrt{3}\,\beta_w\,\gamma_{Mw}}\,aL = 2\cdot\frac{430}{\sqrt{3}\cdot 0{,}85\cdot 1{,}25}\,3\cdot 135\cdot 10^{-3} = 189{,}3 \text{ kN} > 150{,}0 \text{ kN}$$

3.2 Shear resistance of bolts

Reference: 6.2.3.2; Table 6.7 [6.5.5; Table 6.5.3]

Assumption: Shear in thread

$$F_{v.Rd} = \frac{0{,}6\,A_s\,f_{ub}}{\gamma_{Mb}} = \frac{0{,}6\cdot 157\cdot 800}{1{,}25}\cdot 10^{-3} = 60{,}3 \text{ kN} > 37{,}5 \text{ kN}$$

or

$$F_{v.Rd} = \frac{0{,}6\,A_s\,f_{ub}}{\gamma_{Mb}} = \frac{75{,}4}{1{,}25} = 60{,}3 \text{ kN} > 37{,}5 \text{ kN}$$

3.3 Bearing resistance of the endplate

$e_1/3d_o = 40/3\cdot 18 = 0{,}74$
$p_1/3d_o - 1/4 = 55/3\cdot 18 - 1/4 = 0{,}77 \longrightarrow \alpha = 0{,}74$

Reference: 6.2.3.1; Table 6.6 [6.5.5; Table 6.5.3]

The choice of $e_1 = 40$ mm is conservative in view of the presence of the transverse weld between flange and flexible endplate.
f_{ub}/f_u is normally greater than 1,0; however α is equal or lesser than 1,0.

$$F_{b.Rd} = \frac{2{,}5\,\alpha\,f_u\,d\,t}{\gamma_{Mb}} = \frac{2{,}5\cdot 0{,}74\cdot 430\cdot 16\cdot 8}{1{,}25}\cdot 10^{-3} = 81{,}5 \text{ kN} > 37{,}5 \text{ kN}$$

or

$$F_{b.Rd} = \frac{2{,}5\,\alpha\,f_u\,d\,t}{\gamma_{Mb}} = \frac{127{,}4}{1{,}25}\cdot\frac{8}{10} = 81{,}5 \text{ kN} > 37{,}5 \text{ kN}$$

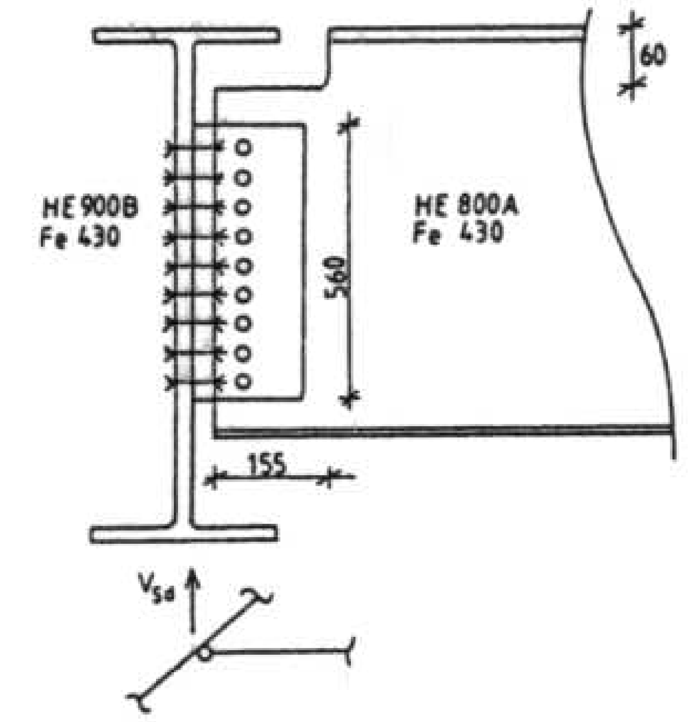
3.4 Bearing resistance of the column flange

not relevant because of $t_f > t_p$

Problem, Sketch, Calculation	Reference	Commentary
3.5 Shear resistance of the endplate $A_{v.net} = 2\bullet(135 - 2\bullet18)\bullet8 = 15{,}8\bullet10^2\ mm^2 > A_v f_y/f_u = (135\bullet8\bullet2)\bullet275/430 = 13{,}8\bullet10^2\ mm^2$ $V_{pl.Rd} = A_v \dfrac{f_y}{\sqrt{3}\gamma_{Mo}} = 21{,}6\cdot10^2\cdot\dfrac{275}{\sqrt{3}\cdot1{,}1}\cdot10^{-3} = 311{,}8\ kN > 150{,}0\ kN$		
3.6 Local shear resistance of the beam web $A_v = 8{,}6\bullet135 = 11{,}61\bullet10^2\ mm^2$ $V_{pl.Rd} = A_v \dfrac{f_y}{\sqrt{3}\gamma_{Mo}} = 11{,}61\cdot10^2\cdot\dfrac{275}{\sqrt{3}\cdot1{,}1}\cdot10^{-3} = 167{,}6\ kN > 150{,}0\ kN$	5.3.3 [5.4.6]	

Problem, Sketch, Calculation	Reference	Commentary

Example 3.1.7: Beam to beam connection with cleats



1. System

Main girder:	HE 900 B	Steel grade:	Fe 430
Side girder:	HE 800 A	Steel grade:	Fe 430
Angles:	2 L 90 x 9	Steel grade:	Fe 430
Bolts:	M 22, 8.8		
	$\longrightarrow d_o$ = 24 mm		

Reference: Table 6.7 [7.5.2]

Commentary: Cleated connections on web only are assumed as hinged connections. Torsional effects may be neglected because of the large length of the cleats.

2. Loading

Shear force: V_{Sd} = 890 kN

3. Determination of the resistance of the connection

3.1 Design resistance of the connection (angle - main girder)

Commentary: **Assumption:** Shear in thread

Reference: [6.5.5; Table 6.5.3]

Shear resistance of bolts:

$$F_{v.Rd} = \frac{0{,}6\ f_{ub}\ A_s}{\gamma_{Mb}} = \frac{0{,}6 \cdot 800 \cdot 303}{1{,}25} \cdot 10^{-3} = 116{,}4\ \text{kN} > \frac{890}{18} = 49{,}4\ \text{kN}$$

Reference: 6.2.3.2; Table 6.7

or

$$F_{v.Rd} = \frac{145{,}4}{1{,}25} = 116{,}3\ \text{kN} > \frac{890}{18} = 49{,}4\ \text{kN}$$

Bearing resistance:

Commentary: The bearing resistance is calculated per member.

Angle:

$e_1/3d_o$ = 40/(3•24) = 0,56 ; $p_1/3d_o$ - 1/4 = 60/(3•24) - 1/4 = 0,58 ; $\longrightarrow \alpha$ = 0,56

Reference: 6.2.3.1; Table 6.6 [6.5.5; Table 6.5.3]

$$F_{b.Rd} = \frac{2{,}5\,\alpha f_u dt}{\gamma_{Mb}} = \frac{2{,}5 \cdot 0{,}56 \cdot 430 \cdot 22 \cdot 9}{1{,}25} \cdot 10^{-3} = 95{,}4\ \text{kN} > \frac{890}{2 \cdot 9} = 49{,}4\ \text{kN}$$

Beam web:

$p_1/3d_o$ - 1/4 = 60/(3•24) - 1/4 = 0,58 ; $\longrightarrow \alpha$ = 0,58

$$F_{b.Rd} = \frac{2{,}5\,\alpha f_u dt}{\gamma_{Mb}} = \frac{2{,}5 \cdot 0{,}58 \cdot 430 \cdot 22 \cdot 18{,}5}{1{,}25} \cdot 10^{-3} = 203{,}0\ \text{kN} > \frac{890}{9} = 98{,}9\ \text{kN}$$

3.2 Determination of the moment due to eccentricity of the bolt group (angle - side girder)

M_{Sd} = 59,25•10^{-3}•890 = 52,7 kNm

Commentary: The reaction force V_{Sd} is applied to the connection at the notched main girder - angle cleat position

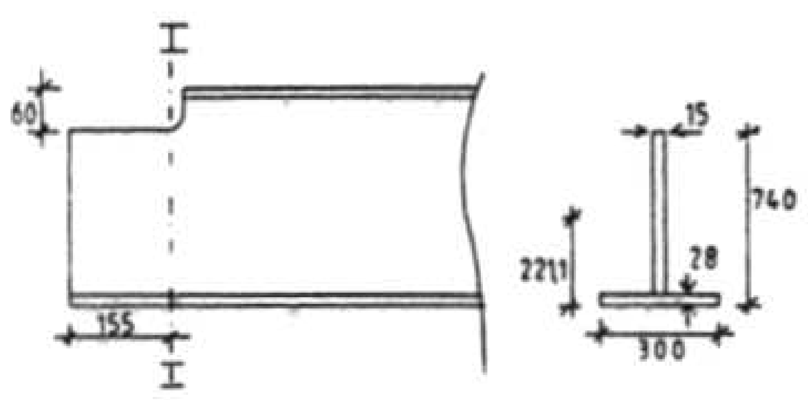
3.3 Design resistance of the connection (angle - side girder)

I_p = 2160•10^4 mm^2

Horizontal shear force:	M_{Sd}•z/I_p = 52,7•10^6•240/2160•10^4 = 58,6 kN
Vertical shear force per bolt:	V_{Sd} = 890/9 = 98,9 kN
Resultant shear force:	$F_{Sd} = \sqrt{(98{,}9^2 + 58{,}6^2)}$ = 115,0 kN

Commentary: The distribution of internal forces between fasteners needs to be proportional to the distance from the centre of rotation in slip-resistant connections $\longrightarrow$ polar moment of inertia of the bolt configuration:

$I_p = \Sigma y^2 + \Sigma z^2 = 2 \cdot (60^2 + 120^2 + 180^2 + 240^2)$
$= 2160 \cdot 10^4$ mm^2

Problem, Sketch, Calculation	Reference	Commentary
Shear resistance of bolts: $F_{v.Rd} = m_s \frac{0{,}6\, f_{ub}\, A_s}{\gamma_{Mb}} = 2\, \frac{0{,}6 \cdot 800 \cdot 303}{1{,}25} \cdot 10^{-3} = 232{,}7 \text{ kN} > 115{,}0 \text{ kN}$ $F_{v.Rd} = 2\, \frac{145{,}4}{1{,}25} = 232{,}6 \text{ kN} > 115{,}0 \text{ kN}$	6.2.3.2; Table 6.7 [6.5.5; Table 6.5.3]	**Assumptions:** m_s = number of members; here: two shear planes - Shear in thread - elastic distribution of internal forces
Bearing resistance: Angle: $e_1/3d_o = 40/(3 \cdot 24) = 0{,}56$; $p_1/3d_o - 1/4 = 60/(3 \cdot 24) - 1/4 = 0{,}58$; $\longrightarrow \alpha = 0{,}56$ $F_{b.Rd} = \frac{2{,}5\, \alpha\, f_u\, d\, t}{\gamma_{Mb}} = \frac{2{,}5 \cdot 0{,}56 \cdot 430 \cdot 22 \cdot 9}{1{,}25} \cdot 10^{-3} = 95{,}4 \text{ kN} > \frac{115{,}0}{2} = 57{,}5 \text{ kN}$	6.2.3.1; Table 6.6 [6.5.5; Table 6.5.3]	The bearing resistance is calculated per member.
Beam web: $e_1/3d_o = 36/(3 \cdot 24) = 0{,}5$; $p_1/3d_o - 1/4 = 60/3 \cdot 24 - 1/4 = 0{,}58$; $\longrightarrow \alpha = 0{,}50$ $F_{b.Rd} = \frac{2{,}5\, \alpha\, f_u\, d\, t}{\gamma_{Mb}} = \frac{2{,}5 \cdot 0{,}50 \cdot 430 \cdot 22 \cdot 15}{1{,}25} \cdot 10^{-3} = 141{,}9 \text{ kN} > 115{,}0 \text{ kN}$	6.2.2; [6.5.4]	The edge distance e_1 should be taken as the minimum value of both sides when the acting force effects not parallel to one of both edge distances.
3.4 Shear resistance of the angle $A_{v.net} = 560 \cdot 9 - 9 \cdot 24 \cdot 9 = 31{,}0 \cdot 10^2 \text{ mm}^2 \leq A_v \cdot f_y/f_u = 560 \cdot 9 \cdot 275/430 = 32{,}2 \cdot 10^2 \text{ mm}^2$ $\longrightarrow$ fastener holes have to be allowed for in shear verification $A_{v.eff} = A_{v.net}\, f_u/f_y = 31{,}0 \cdot 10^2 \cdot 430/275 = 48{,}5 \cdot 10^2 \text{ mm}^2 < 50{,}4 \cdot 10^2 \text{ mm}^2 = A_v$ $V_{pl.Rd} = A_v \frac{f_y}{\sqrt{3}\, \gamma_{Mo}} = 48{,}5 \cdot 10^2 \cdot \frac{275}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 700{,}0 \text{ kN} > \frac{890{,}0}{2} = 445{,}0 \text{ kN}$	5.3.3; [5.4.6]	Two angle cleats connect the side girder to the main girder. The shear resistance is calculated per member.
3.5 Design resistance of the notched beam at cut I - I $V_{Sd} = 890 \text{ kN} \longrightarrow M_{I.Sd} = 890 \cdot 155 \cdot 10^{-3} = 138 \text{ kNm}$ **Cross-sectional dimensions at cut I - I** $A_I = 300 \cdot 28 + 712 \cdot 15 = 190{,}8 \cdot 10^2 \text{ mm}^2$  $z_I = 712 \cdot 15\, \frac{712/2 + 14}{190{,}8 \cdot 10^2} + 14 = 221{,}1 \text{ mm}$ $I_I = 712^3 \cdot 15/12 + 300 \cdot 28 \cdot 207{,}1^2 + 712 \cdot 15 \cdot 162{,}9^2 = 109486{,}8 \cdot 10^4 \text{ mm}^4$ $W_{I.el} = 109486{,}8 \cdot 10^4/518{,}9 = 2110 \cdot 10^3 \text{ mm}^3$ $A_{I.v} = 712 \cdot 15 = 106{,}8 \cdot 10^2 \text{ mm}^2$		

Problem, Sketch, Calculation	Reference	Commentary

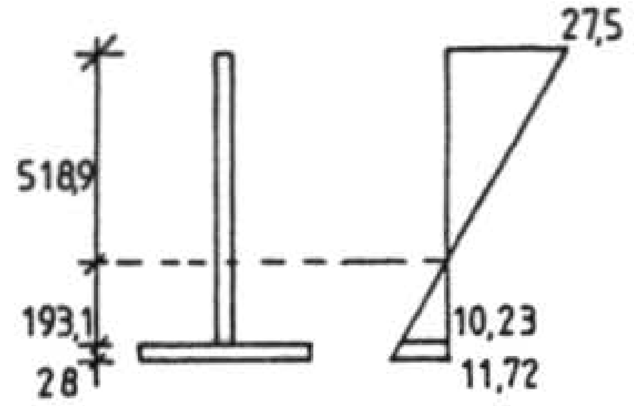
$$M_{el.Rd} = \frac{W_{el}\, f_y}{\gamma_{Mo}} = \frac{2110 \cdot 10^3 \cdot 275}{1{,}1} \cdot 10^{-6} = 527{,}5 \text{ kNm} > 138{,}0 \text{ kNm}$$

Reference: 5.3.2; [5.4.5]

$$0{,}5\, V_{Rd} = 0{,}5\, \frac{A_v\, f_y}{\sqrt{3}\, \gamma_{Mo}} = 0{,}5 \cdot \frac{106{,}8 \cdot 10^2 \cdot 275}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 770{,}8 \text{ kN} < 890{,}0 \text{ kNm}$$

Reference: 5.3.3; [5.4.6]

$\longrightarrow$ Interaction Bending - shear is required



$$M_{el.V.Rd} = M_{el.Rd}\,(1 - \rho)$$

$$\rho = \left(2\, \frac{V_{Sd}}{V_{Rd}} - 1\right)^2 = \left(\frac{2 \cdot 890}{2 \cdot 770{,}8} - 1\right)^2 = 0{,}024$$

Reference: Table 5.16

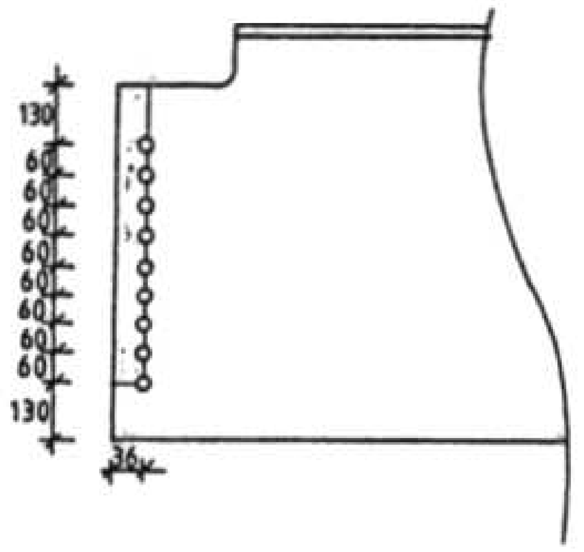
$$M_{el.V.Rd} = 527{,}5\,(1 - 0{,}024) = 514{,}8 \text{ kNm} > 138 \text{ kNm}$$

More accurate method

$M_{el.v.Rd}$ =
$[(51{,}89^2 \cdot 1{,}5 \cdot 27{,}5/3$ +
+ $19{,}31^2 \cdot 1{,}5 \cdot 10{,}23/3) \cdot (1 - 0{,}024)$ +
+ $2{,}8 \cdot 30 \cdot (19{,}31 + 2{,}8/2) \cdot$
$\cdot (11{,}72 + 10{,}23)/2]/(100 \cdot 1{,}1) = 519{,}0$ kNm

3.6 Block shear resistance

Reference: 5.5.2; [6.5.2.2]



$L_1 = a_1 = 130 \text{ mm} > 5d = 5 \cdot 22 = 110 \longrightarrow L_1 = 110 \text{ mm}$
$L_2 = (a_2 - kd_{o.t})(f_u/f_y) = (36 - 0{,}5 \cdot 24)430/275 = 37{,}5 \text{ mm}$
$L_3 = L_v + a_1 + a_3 = 480 + 130 + 130 = 740 \text{ mm}$
$< (L_v + a_1 + a_3 - nd_{o.v})(f_u/f_y) = (740 - 9 \cdot 24)430/275 = 819{,}3 \text{ mm}$
$L_{v.eff} = L_v + L_1 + L_2 = 480 + 110 + 37{,}5 = 627{,}5 \text{ mm} < L_3 = 740 \text{ mm}$

$$V_{eff.Rd} = L_{v.eff}\, t\, \frac{f_y}{\sqrt{3}\, \gamma_{Mo}} = 627{,}5 \cdot 15 \cdot \frac{275}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 1358{,}6 \text{kN} > 890 \text{ kN}$$

3.7 Design shear resistance of side girder

Reference: 5.3.3; [5.4.6]

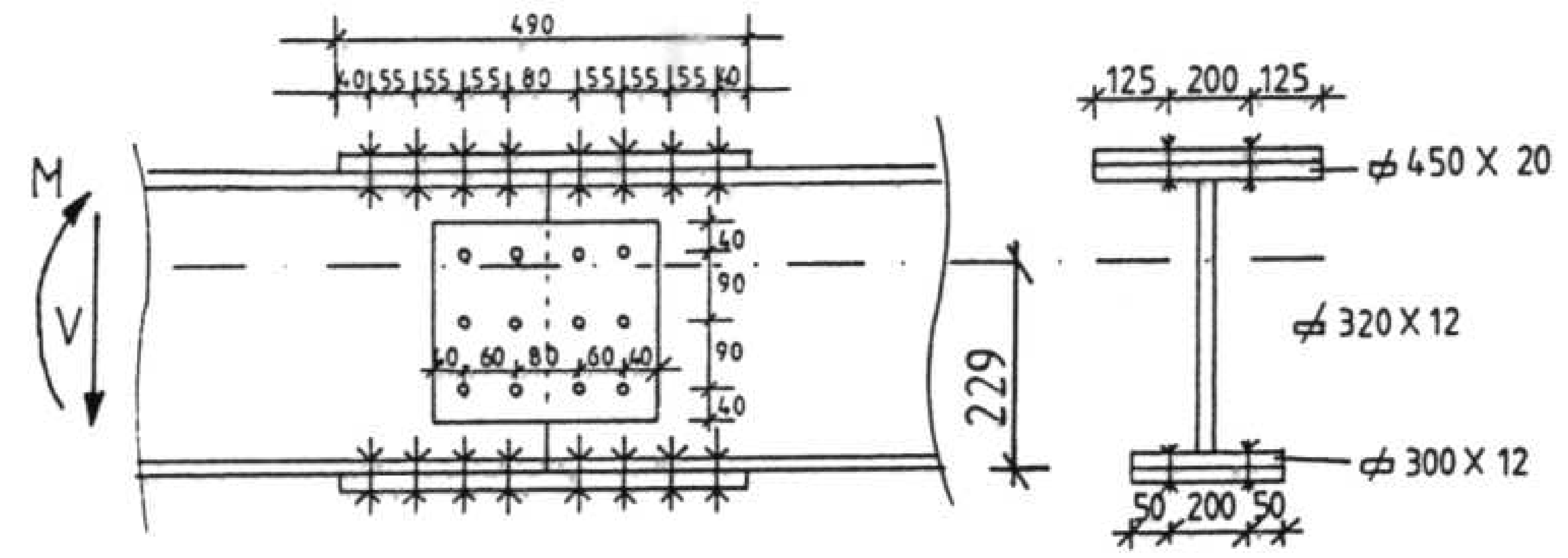
$A_{v.net} = (740 - 28) \cdot 15 - 9 \cdot 24 \cdot 15 = 74{,}4 \cdot 10^2 \text{ mm}^2 \geq A_v \cdot f_y/f_u = (740 - 28) \cdot 15 \cdot 275/430 = 68{,}3 \cdot 10^2 \text{ mm}^2$
$\longrightarrow$ fastener holes need not be allowed for in shear verification

$$V_{pl.Rd} = A_v\, \frac{f_y}{\sqrt{3}\gamma_{Mo}} = 106{,}8 \cdot 10^2 \cdot \frac{275}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 1541{,}5 \text{ kN} > 890{,}0 \text{ kN}$$

Though it looks like a "long joint", it is not because the rules for long joints refer to load introduction problems only. The progressive introduction of the load rather than simultaneous introduction as is the case here.

Problem, Sketch, Calculation	Reference	Commentary

Example 3.1.8: Splice of an unsymmetrical I-section



1. System

Member: see sketch
Steel grade: Fe 430
All plates: Steel grade: Fe 430
Bolts: M 16, 8.8 preloaded
⟶ d_o = 18 mm
Surface: class A

Reference: Table 6.7 [7.5.2]

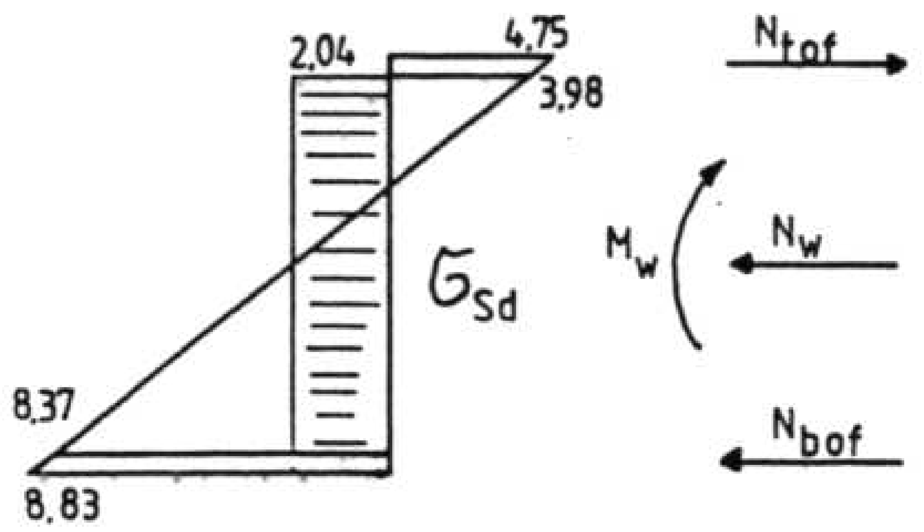
Commentary:
$A = 164{,}4 \cdot 10^2\ mm^2$
$I = 33953 \cdot 10^4\ mm^4$
$W_{tof} = 2760 \cdot 10^3\ mm^3$; tof - top flange
$W_{bof} = 1483 \cdot 10^3\ mm^3$; bof - bottom flange

flange cover plate (tof): 490 x 450 x 15
flange cover plate (bof): 490 x 300 x 10
web cover plates: 280 x 260 x 10

2. Loading

Ultimate limit state:
V_{Sd} = 237 kN
M_{Sd} = 131 kNm
Serviceability limit state:
V_{Sd} = 158 kN
M_{Sd} = 87,3 kNm

3. Determination of the resistance of connection



Distribution of action effects on web and flanges

Ultimate limit state:
$N_{tof.Sd}$ = 392,9 kN $N_{w.Sd}$ = 78,3 kN
$N_{bof.Sd}$ = 309,6 kN $M_{w.Sd}$ = 12,3 kNm
$V_{w.Sd}$ = 237 kN

Serviceability limit state:
$N_{tof.Sd}$ = 261,9 kN $N_{w.Sd}$ = 52,2 kN
$N_{bof.Sd}$ = 206,4 kN $M_{w.Sd}$ = 8,2 kNm
$V_{w.Sd}$ = 158 kN

Commentary:
Requirement:
No slip at serviceability limit state
Assumptions:
(1) Shear force is transmitted by the web alone
(2) Due to unsymetrical section bending moment and normal force exists in web

3.1 Flange cover plate

3.1.1 Member resistance of the flange cover plate

Tension resistance

Reference: 5.5.1 [5.4.3]

gross section:
$$N_{pl.Rd} = bt\frac{f_y}{\gamma_{Mo}} = 300 \cdot 10 \cdot \frac{275}{1{,}1} \cdot 10^{-3} = 750{,}0\ kN > 309{,}6\ kN$$

net section:
$$A_{net} = 300 \cdot 10 - 2 \cdot 18 \cdot 10 = 26{,}4 \cdot 10^2\ mm^2$$

$$N_{u.Rd} = 0{,}9 A_{net}\frac{f_u}{\gamma_{M2}} = 0{,}9 \cdot 26{,}4 \cdot 10^2 \cdot \frac{430}{1{,}25} \cdot 10^{-3} = 817{,}3\ kN > 309{,}6\ kN$$

Compression resistance

Reference: 5.4.2 [5.5.1]

gross section:
$$N_{pl.Rd} = bt\frac{f_y}{\gamma_{Mo}} = 450 \cdot 15 \cdot \frac{275}{1{,}1} \cdot 10^{-3} = 1687{,}5\ kN > 392{,}9\ kN$$

Problem, Sketch, Calculation	Reference	Commentary

3.1.2 Resistance of the connection

Ultimate limit state:

Shear resistance of bolts: $F_{v.Rd} = \dfrac{0{,}6\, f_{ub}\, A}{\gamma_{Mb}} = \dfrac{0{,}6 \cdot 800 \cdot 201}{1{,}25} \cdot 10^{-3} = 77{,}2 \text{ kN} > \dfrac{309{,}6}{8} = 38{,}7 \text{ kN}$

Reference: 6.2.3.2; Table 6.6; [6.5.5; Table 6.5.3]

Commentary: **Assumption:** Shear in shank $A = \pi \cdot 16^2/4 = 2{,}01 \cdot 10^2 \text{ mm}^2$

Bearing resistance: $e_1/3d_o = 40/(3 \cdot 18) = 0{,}74$; $p_1/3d_o - 1/4 = 55/(3 \cdot 18) - 1/4 = 0{,}77$; $\longrightarrow \alpha = 0{,}74$

Flange cover plate: $F_{b.Rd} = \dfrac{2{,}5\, \alpha\, f_u d t}{\gamma_{Mb}} = \dfrac{2{,}5 \cdot 0{,}74 \cdot 430 \cdot 16 \cdot 10}{1{,}25} \cdot 10^{-3} = 101{,}8 \text{ kN} > 38{,}7 \text{ kN}$

Beam flange: Verification not needed because of $t_f > t_p$

Reference: 6.2.3.1 Table 6.7 [6.5.5; Table 6.5.3]

Commentary: The edge distance e_1 should be taken as the minimum value of both sides when the acting force effects are not parallel to one of the edge distances

Serviceability limit state:

Slip resistance: $F_{s.Rd} = \dfrac{0{,}7 \cdot \mu \cdot f_{ub} \cdot A_s}{\gamma_{Ms.ser}} = \dfrac{0{,}7 \cdot 0{,}5 \cdot 800 \cdot 157}{1{,}10} \cdot 10^{-3} = 40{,}0 \text{ kN} \geq \dfrac{206{,}4}{8} = 25{,}8 \text{ kN}$

$$F_{s.Rd} = \frac{17{,}6 \cdot 2{,}5}{1{,}10} = 40{,}0 \text{ kN} \geq \frac{206{,}4}{8} = 25{,}8 \text{ kN}$$

Reference: 4.3; 6.2.4; Table 4.5 [6.5.8]

Commentary: No slip shall occur at the serviceability limit state

3.2 Web cover plate

Ultimate limit state:

Vertical shear force:	$V_{Sd} = 237 \text{ kN}$
Vertical shear force per bolt:	$V_{Sd}/n = 237{,}0/6 = 39{,}5 \text{ kN}$
Moment due to vertical shear:	$M_{Sd} = 237{,}0 \cdot 70 \cdot 10^{-3} + 12{,}3 = 28{,}9 \text{ kNm}$
Horizontal shear force per bolt:	$H_{Sd} = 28{,}9 \cdot 90 \cdot 10^3/378 \cdot 10^2 + 78{,}3/6 = 81{,}9 \text{ kN}$
Resultant shear force at the outer bolt:	$F_{Sd} = \sqrt{(39{,}5^2 + 81{,}9^2)} = 90{,}9 \text{ kN}$

Serviceability limit state:

Vertical shear force	$V_{Sd} = 158 \text{ kN}$
Vertical shear force per bolt:	$V_{b.Sd}/n = 158/6 = 26{,}3 \text{ kN}$
Torsional moment	$M_{Sd} = 158 \cdot 70 \cdot 10^{-3} + 8{,}2 = 19{,}3 \text{ kNm}$
Horizontal shear force per bolt:	$H_{Sd} = 19{,}3 \cdot 90 \cdot 10^3/378 \cdot 10^2 + 52{,}2/6 = 54{,}7 \text{ kN}$
Resultant shear force at the outer bolt:	$F_{Sd} = \sqrt{(26{,}3^2 + 54{,}7^2)} = 60{,}7 \text{ kN}$

Reference: 6.2.2 [6.5.4]

Commentary: Forces acting on the bolt group on each side of the splice.
The distribution of internal forces between fasteners need to be proportional to the distance from the centre of rotation in slip-resistant connections $\longrightarrow$ polar moment of inertia of the bolt configuration:
$I_p = \Sigma y^2 + \Sigma z^2 = 6 \cdot 30^2 + 4 \cdot 90^2 = 378 \cdot 10^2 \text{ mm}^2$

3.2.1 Member resistance of the web cover plate

Shear resistance: $A_{v.net} = 2 \cdot (260 - 3 \cdot 18) \cdot 10 = 41{,}2 \cdot 10^2 \text{ mm}^2 > 2 \cdot 260 \cdot 10 \cdot 275/430 = 33{,}3 \cdot 10^2 \text{ mm}^2$

$\longrightarrow$ no reduction of A_v is required

$$V_{pl.Rd} = A_v \frac{f_y}{\sqrt{3}\,\gamma_{Mo}} = 2 \cdot 260 \cdot 10 \cdot \frac{275}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 750{,}6 \text{ kN} > 237{,}0 \text{ kN}$$

$237{,}0 \text{ kN} < 0{,}5 \cdot 750{,}6 = 375{,}3 \text{ kN}$

$\longrightarrow$ no interaction bending - shear is required

Reference: 5.3.3 [5.4.6]

Commentary: Two web cover plates are used.

Problem, Sketch, Calculation	Reference	Commentary

3.2.2 Bending resistance of the web cover plate

$W_{pl} = 2 \cdot [260^2 \cdot 10/4 - 18 \cdot 10 \cdot 90] = 305{,}6 \cdot 10^3\ mm^3$

$$M_{pl.Rd} = W_{pl} \frac{f_y}{\gamma_{Mo}} = 305{,}6 \cdot 10^3 \cdot \frac{275}{1{,}1} \cdot 10^{-6} = 76{,}4\ kNm > 28{,}9\ kNm$$

Reference: 5.3.2 [5.4.5]

3.2.3 Resistance of the connection
Ultimate limit state:

Shear resistance of bolts:

$$F_{v.Rd} = m_s \frac{0{,}6\ f_{ub}\ A}{\gamma_{Mb}} = 2\ \frac{0{,}6 \cdot 800 \cdot 201}{1{,}25} \cdot 10^{-3} = 154{,}4\ kN > 90{,}9\ kN$$

Bearing resistance:

Web cover plate: $e_1/3d_o = 40/(3 \cdot 18) = 0{,}74$; $p_1/3d_o - 1/4 = 60/(3 \cdot 18) - 1/4 = 0{,}86$; $\longrightarrow \alpha = 0{,}74$

$$F_{b.Rd} = \frac{2{,}5\,\alpha\, f_u d t}{\gamma_{Mb}} = \frac{2{,}5 \cdot 0{,}74 \cdot 430 \cdot 16 \cdot 10}{1{,}25} \cdot 10^{-3} = 101{,}8\ kN > \frac{90{,}9}{2} = 45{,}5\ kN$$

Beam web: $e_1/3d_o = 40/(3 \cdot 18) = 0{,}74$; $p_1/3d_o - 1/4 = 60/(3 \cdot 18) - 1/4 = 0{,}86$; $\longrightarrow \alpha = 0{,}74$

$$F_{b.Rd} = \frac{2{,}5\,\alpha\, f_u d t}{\gamma_{Mb}} = \frac{2{,}5 \cdot 0{,}74 \cdot 430 \cdot 16 \cdot 12}{1{,}25} \cdot 10^{-3} = 122{,}2\ kN > 90{,}9\ kN$$

Reference: 6.2.3.2; Table 6.6; 6.2.3.1 Table 6.7 [6.5.5; Table 6.5.3]

Commentary:
Assumption: Shear in shank.
m_s - number of shear planes
Here: two shear planes)
The edge distance e_1 should be taken as the minimum value of both sides when the acting force effects are not parallel to one of the edge distances
The bearing resistance is calculated per member.

Serviceability limit state

Slip resistance:

$$F_{s.Rd} = m_f \cdot \frac{0{,}7 \cdot \mu \cdot f_{ub} \cdot A_s}{\gamma_{Ms.ser}} = 2 \cdot \frac{0{,}7 \cdot 0{,}5 \cdot 800 \cdot 157}{1{,}10} \cdot 10^{-3} = 80{,}0\ kN > 60{,}7\ kN$$

or

$$F_{s.Rd} = 2 \cdot \frac{17{,}6 \cdot 2{,}5}{1{,}10} = 80{,}0\ kN > 60{,}7\ kN$$

Reference: 4.3; 6.2.4 [6.5.8]

Commentary:
No slip shall occur at the serviceability limit state
m_f = number of friction interfaces
Here: Two friction interfaces are available

3.3 Block shear resistance of the beam member

$L_1 = a_1 = 90\ mm > 5d = 5 \cdot 16 = 80 \longrightarrow L_1 = 80\ mm$

$L_2 = (a_2 - kd_{o,t})(f_u/f_y) = (99 - 2{,}5 \cdot 18)430/275 = 84\ mm$

$L_3 = L_v + a_1 + a_3 = 180 + 85 + 85 = 350\ mm$

$< L_v + a_1 + a_3 - nd_{o,v})(f_u/f_y) = (350 - 4 \cdot 18)430/275 = 434{,}7\ mm$

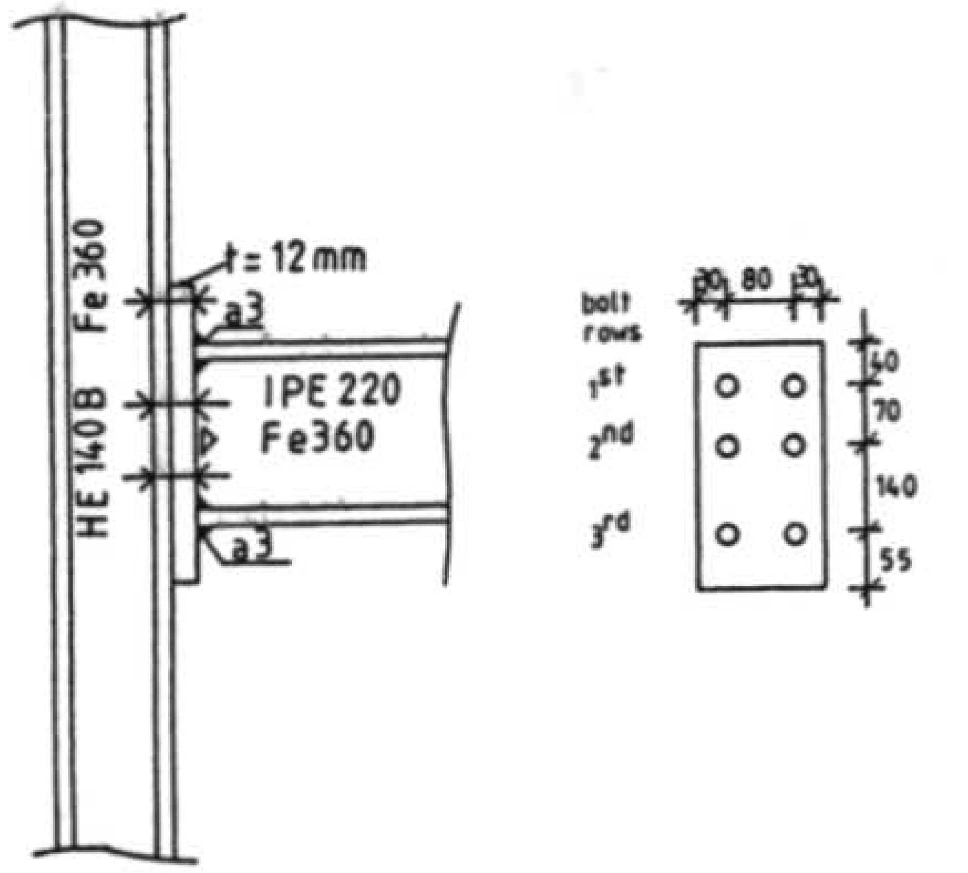
$L_{v,eff} = L_v + L_1 + L_2 = 180 + 80 + 84{,}4 = 344\ mm < L_3 = 350\ mm$

$$V_{eff.Rd} = L_{v.eff}\ t\ \frac{f_y}{\sqrt{3}\ \gamma_{Mo}} = 344 \cdot 12 \cdot \frac{275}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 595{,}8kN > 237\ kN$$

Reference: 5.5.2 [6.5.2.2]

Problem, Sketch, Calculation	Reference	Commentary

Example 3.1.9: Bolted end plate connection



1. System

Beam: IPE 220, Fe 360
Column: HE 140 B, Fe 360
End plate: 305 x 140 x 12, Fe 360
Bolts: M 16, 8.8

Commentary: This example illustrates, how a bolted beam-to-column connection is designed by a simplified hand calculation using the T-stub model. When the moment resistance of complex bolted beam-to-column connections should be calculated based on the full procedure given in Annex J it is recommended to develop a software programme.

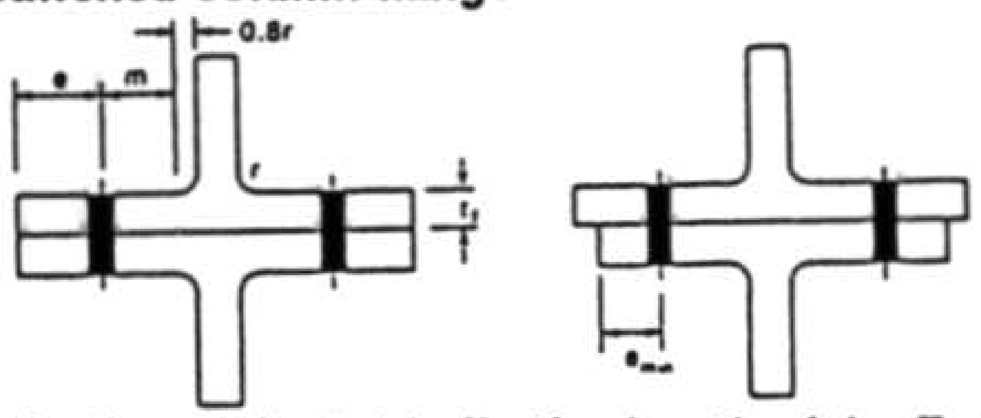
2. Loading

Bending moment: M_{Sd} = 20,0 kNm
Shear force: V_{Sd} = 120 kN

3. Determination of the moment resistance
3.1 Tension zone
3.1.1 Unstiffened column flange

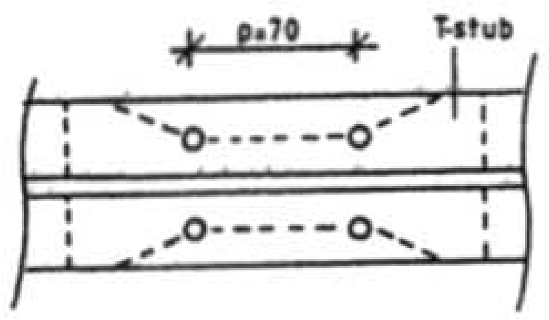
Reference: -
[Figure J.3.1]

Commentary: **Simplification:**
Calculation of an equivalent T-stub for bolt rows 1 and 2.



Dimensions of a T-stub
$n = e_{min} = e = 30$ mm
$m = (80 - 7 - 2 \cdot 0{,}8 \cdot 12)/2 = 26{,}9$ mm

(1) Determination of the total effective length of the T-stub with 2 bolt rows
Relevant bolt pattern



1st bolt row (end bolt)
$\ell_{eff} = 0{,}5p + 2m + 0{,}625e$
$\ell_{eff} = 0{,}5 \cdot 70 + 2 \cdot 26{,}9 + 0{,}625 \cdot 30 = 107{,}55$ mm

2th bolt row (end bolt)
$\ell_{eff} = 0{,}5p + 2m + 0{,}625e$
$\ell_{eff} = 0{,}5 \cdot 70 + 2 \cdot 26{,}9 + 0{,}625 \cdot 30 = 107{,}55$ mm

$\ell_{eff} = 215{,}1$ mm

Reference: -
[J.3.4.1;
Figure J.3.4]

Commentary: The effective length ℓ_{eff} for each row of bolts is based on the yield line patterns and should be taken as the smallest value of the different failure modes. **Here:** The relevant failure mode is the "combined bolt group pattern"

Calculation of the plate bending moment of the T-stub

$$M_{pl.Rd} = 0{,}25\ \ell_{eff}\ t_f^2\ \frac{f_y}{\gamma_{Mo}}\ k_r \quad ; \text{ assumption: } k_r = 1{,}0$$

$$= 0{,}25 \cdot 215{,}1 \cdot 12^2 \cdot \frac{235}{1{,}1} \cdot 10^{-6} = 1{,}65 \text{ kNm}$$

$t_p = 12 \text{ mm} > 8 \text{ mm} = 0{,}5d$
$\longrightarrow B_{t.Rd} = F_{t.Rk} / \gamma_{Mb} = 113{,}0/1{,}25 = 90{,}4$ kN

Reference: -
[J.3.3]

6.2.3.3(2)
Table 6.8
[6.5.5.(4)]

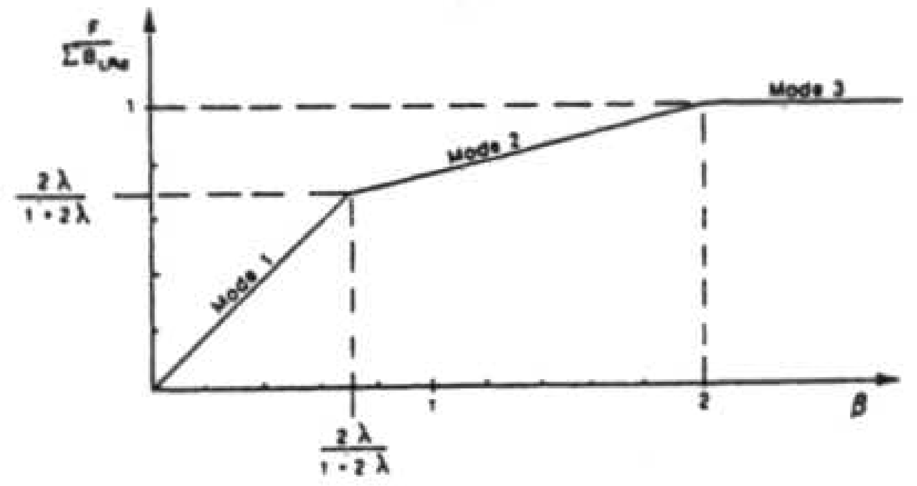
Commentary: **Assumption:**
The compressive normal stress $\sigma_{n.Ed}$ in the column flange due to axial force and bending moment does not exceed 180 N/mm² at the location of the tension zone.
$\longrightarrow$ The reduction factor $k_r = 1{,}0$.

$B_{t.Rd}$ is the design tension resistance of a single bolt plate assembly.

Problem, Sketch, Calculation	Reference	Commentary

Determination of the failure mode of the T-stub

Reference: - [J.3.3; Figure J.3.3]



$$\lambda = \frac{n}{m} = \frac{30}{26,9} = 1,115$$

where n = 50 mm

$$\frac{2\lambda}{1+2\lambda} = \frac{2 \cdot 1,115}{1+2 \cdot 1,115} = 0,690$$

$$\beta = \frac{4 \cdot M_{pl.Rd}}{m \cdot \sum B_{t.Rd}}$$

$$= \frac{4 \cdot 1,65}{26,9 \cdot 10^{-3} \cdot 4 \cdot 90,4} = 0,679$$

$\beta = 0,679 < 0,690 \longrightarrow$ failure mode 1

Design tension resistance of the T-stub

Reference: - [J.3.3; Figure J.3.2]

Commentary: The design tension resistance of the T-stub is dependent on the failure mode. The effects of prying forces are taken into account.

$$F_{t.Rd} = \frac{4 M_{pl.Rd}}{m}$$

$$= \frac{4 \cdot 1,65}{26,9 \cdot 10^{-3}} = 245,4 \text{ kN}$$

3.1.2 End plate

(1) Bolt outside tension flange

Reference: - [J.3.4.4; Figure J.3.8]

Commentary: The extended end plate is considered as a simple T-stub, see sketch, neglecting the stiffening effect of the beam web.

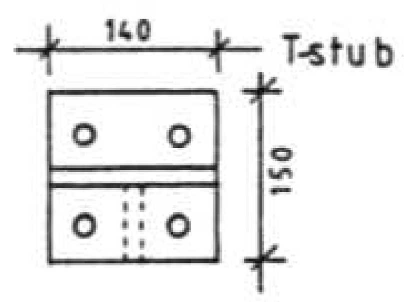
Dimensions of a T-stub

$n = e_{min} = e = 40$ mm

$m = (70 - 9,2 - 2 \cdot 0,8 \cdot 8 \cdot \sqrt{2})/2 = 21,3$ mm

(2) Determination of the total effective length of the T-stub with 2 bolt rows

Relevant bolt pattern



$\ell_{eff} = b_p$

$\ell_{eff} = 140$ mm

Problem, Sketch, Calculation	Reference	Commentary

Calculation of the plate bending moment

Reference: - [J.3.3]

$$M_{pl.Rd} = 0{,}25\ \ell_{eff}\ t_f^2\ \frac{f_y}{\gamma_{Mo}}$$

$$= 0{,}25 \cdot 140 \cdot 12^2 \cdot \frac{235}{1{,}1} \cdot 10^{-6} = 1{,}08\ \text{kNm}$$

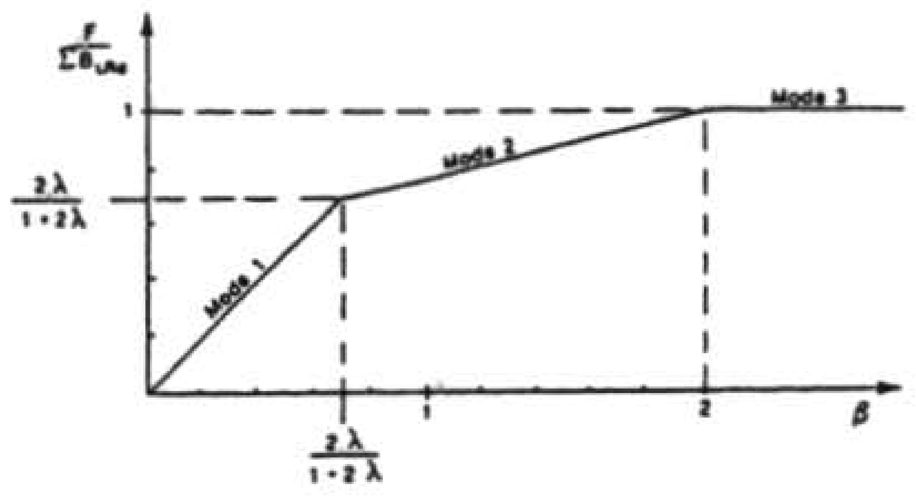
$t_p = 12\ \text{mm} > 8\ \text{mm} = 0{,}5d$

$\longrightarrow B_{t.Rd} = F_{t.Rk} / \gamma_{Mb} = 113{,}0/1{,}25 = 90{,}4\ \text{kN}$

Reference: 6.2.3.3.(2); Table 6.8 [6.5.5.(4)]

Determination of the failure mode of the T-stub

Reference: - [J.3.3; Figure J.3.3]



$$\lambda = \frac{n}{m} = \frac{40}{21{,}3} = 1{,}878$$

$$\frac{2\lambda}{1+2\lambda} = \frac{2 \cdot 1{,}878}{1 + 2 \cdot 1{,}878} = 0{,}79$$

$$\beta = \frac{4 \cdot M_{pl.Rd}}{m_x \cdot \sum B_{t.Rd}}$$

$$= \frac{4 \cdot 1{,}08}{21{,}3 \cdot 10^{-3} \cdot 4 \cdot 90{,}4} = 0{,}561$$

$\beta = 0{,}561 < 0{,}79 \longrightarrow$ failure mode 1

Design tension resistance of the T-stub

Reference: - [J.3.3; Figure J.3.2]

Commentary: The design tension resistance of the T-stub is dependent on the failure mode. The effects of prying forces are taken into account.

$$F_{t.Rd} = \frac{4 M_{pl.Rd}}{m}$$

$$= \frac{4 \cdot 1{,}08}{21{,}3 \cdot 10^{-3}} = 202{,}8\ \text{kN}$$

3.1.3 Resistance of column web

Reference: - [J.3.4.6]

Commentary: The effective width b_{eff} of the column web in tension should be taken as equal to the total effective length of the bolt pattern in the tension zone connection.

$b_{eff} = 220 \cdot 1{,}5 = 330\ \text{mm}$

$$F_{t.Rd} = t_{wc}\ b_{eff}\ \frac{f_{yc}}{\gamma_{Mo}}$$

$$= 7 \cdot 330 \cdot \frac{235}{1{,}1} \cdot 10^{-3} = 493{,}5\ \text{kN}$$

Problem, Sketch, Calculation	Reference	Commentary

3.2 Resistance of shear zone

Reference: 5.3.3 [J.3.6]

$d/t = 92/7 = 13{,}1 < 69 = 69\epsilon$ ⟶ no shear buckling verification

$A_v = 9{,}8 \cdot 10^2\ mm^2$

$$V_{pl.Rd} = A_v \frac{f_{yc}}{\sqrt{3}\ \gamma_{Mo}}$$

$$= 9{,}8 \cdot 10^2 \cdot \frac{235}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 120{,}9\ kN$$

3.3 Resistance of compression zone

3.3.1 Crushing resistance

Reference: - [J.3.5.1.(1)]

Commentary: For this example it is assumed that $\sigma_{n.Ed} < f_y$. The compression load is introduced by contact ⟶ $a_p = a_{min} = 3$ mm

$a_p = 3$ mm

$b_{eff} = t_{fb} + 2\sqrt{2}a_p + 2t_p + 5(t_{fc} + r_c)$

$= 9{,}2 + 2\sqrt{2} \cdot 3 + 2 \cdot 12 + 5 \cdot (12 + 12) = 161{,}7$ mm

$$F_{t.Rd} = t_{wc}\ b_{eff} \frac{f_{yc}}{\gamma_{Mo}}$$

$$= 7 \cdot 161{,}7 \cdot \frac{235}{1{,}1} \cdot 10^{-3} = 241{,}8\ kN$$

3.3.2 Buckling resistance

Reference: 5.6.5 [J.3.5.1.(2); 5.7.5]

Commentary: The buckling length of the compression member (column web) is determined from the conditions of lateral and rotational restraint at the flanges at the point of load introduction. Non sway mode is assumed ⟶ $\ell \approx 0{,}6d$

$b_{eff} = h_c = 140$ mm

$\ell \approx 0{,}6d = 0{,}6 \cdot 92 = 55{,}2$ mm

$A = b_{eff} \cdot t_{wc} = 140 \cdot 7 = 9{,}8 \cdot 10^2\ mm^2$ $\quad I = b_{eff} t_{wc}^3/12 = 140 \cdot 7^3/12 = 0{,}4 \cdot 10^4\ mm^4$

$i = \sqrt{(I/A)} = \sqrt{(0{,}4 \cdot 10^4 / 9{,}8 \cdot 10^2)} = 2{,}0$ mm

$$\bar{\lambda} = \frac{\ell}{i} \frac{1}{\lambda_1}$$

$$= \frac{55{,}2}{2{,}0} \frac{1}{93{,}9} = 0{,}29$$

⟶ using buckling curve c ⟶ $\chi_c = 0{,}9542$

Reference: 5.4.2 [5.5.1]

$$F_{t.Rd} = \chi_c\ A \frac{f_{yc}}{\gamma_{M1}}$$

$$= 0{,}9542 \cdot 9{,}8 \cdot 10^2 \cdot \frac{235}{1{,}1} \cdot 10^{-3} = 199{,}8\ kN$$

Problem, Sketch, Calculation	Reference	Commentary
3.4 Moment resistance of the connection	- [J.3.4.5]	
Maximum potential resistance of the tension zone, shear zone and compression zone: $\longrightarrow$ shear zone is relevant		
$\max F_{Rd} = V_{pl.Rd} = 120{,}9$ kN		
Distribution of the bolt forces according to the tension resistance of the unstiffened column flange and the end plate The moment resistance of the connection is determined to: $M_{Rd} = \max F_{Rd} h_i = 120{,}9 \cdot (220 - 9{,}2) \cdot 10^{-3} = 25{,}5$ kNm $M_{pl.Rd} = W_{pl} f_y / \gamma_{M0} = 285 \cdot 10^{3} \cdot 235 \cdot 10^{-6} / 1{,}1 = 60{,}9$ kNm $\longrightarrow$ The beam-to-column connection is classified as partial strength [6.4.3.3; 6.9.6.3]	- [J.3.4.5]	The maximum potential design resistance of the column flange is generally not the same as the maximum potential design resistance of the beam end plate. In order to determine the actual design resistance of the tension zone a compatible distribution of bolt-row forces should be obtained.
3.5 Shear resistance of the bolted connection		**Assumption:** Shear in thread Only the lower two bolts are used to transfer the shear force from the beam into the column.
Shear resistance of bolts: $F_{v.Rd} = \frac{0{,}6\, f_{ub}\, A_s}{\gamma_{Mb}} = \frac{0{,}6 \cdot 800 \cdot 157}{1{,}25} \cdot 10^{-3} = 60{,}3 \text{ kN} > \frac{120}{2} = 60{,}0$ kN	[6.5.5; Table 6.5.3]	
or $F_{v.Rd} = \frac{75{,}4}{1{,}25} = 60{,}3 \text{ kN} > \frac{120}{2} = 60{,}0$ kN	6.2.3.2; Table 6.7	
Bearing resistance: $e_1/3d_o = 55/(3 \cdot 18) = 1{,}01$; $p_1/3d_o - 1/4 = 140/(3 \cdot 18) - 1/4 = 2{,}34$; $\longrightarrow \alpha = 1{,}0$ $F_{b.Rd} = \frac{2{,}5\, \alpha\, f_u d t}{\gamma_{Mb}} = \frac{2{,}5 \cdot 1{,}0 \cdot 360 \cdot 16 \cdot 12}{1{,}25} \cdot 10^{-3} = 138{,}2 \text{ kN} > \frac{120}{2} = 60{,}0$ kN	[6.5.5; Table 6.5.3] 6.2.3.1; Table 6.6	The eccentricity Δe is covered by the design resistances for bolts and need not to be taken into account
3.6 Determination of the "web weld" resistance	- [6.6.5; 6.6.9]	
$F_{w.Rd} = \frac{f_u}{\sqrt{3}\, \beta_w\, \gamma_{Mw}} a \cdot L = \frac{360}{\sqrt{3} \cdot 0{,}8 \cdot 1{,}25} \cdot 3 \cdot 177 \cdot 2 \cdot 10^{-3} = 220{,}7 \text{ kN} > 120$ kN	6.3.4.3 Table 6.15	
or $F_{w.Rd} = \frac{77{,}9}{1{,}25} \cdot \frac{177 \cdot 2}{100} = 220{,}6 \text{ kN} > 120$ kN		

3.2 Welded connections

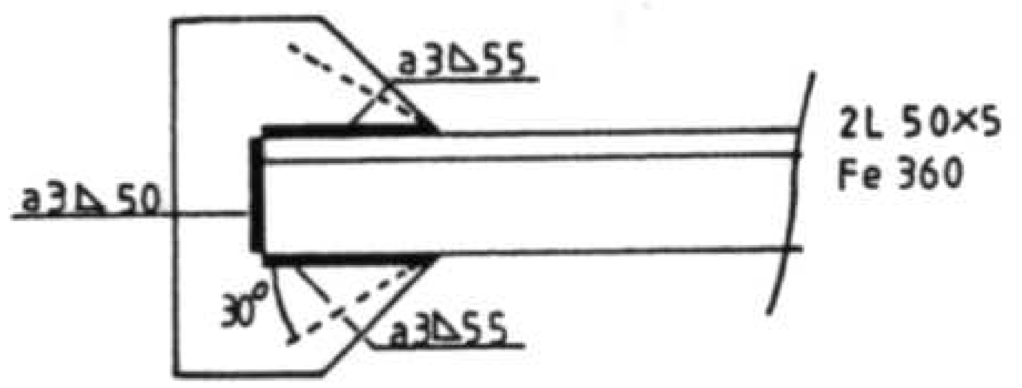
These examples demonstrate the verification of welded connections assuming design values of action effects (N_{Sd}, V_{Sd}, M_{Sd}, etc) which have been calculated by an analysis of the structure and these values already include ψ, γ_F etc.
The results presented in the examples are rounded values. For the purpose of easy re-calculation each formula and each check is calculated with the rounded values.

The following examples are included in this chapter:

Example 3.2.1:	Double angle welded to a gusset plate
Example 3.2.2:	Bracket welded on a column
Example 3.2.3:	Welded beam to column connection without stiffeners
Example 3.2.4:	Welded beam to column connection with stiffeners
Example 3.2.5:	Hollow section lattice girder joint

Problem, Sketch, Calculation	Reference	Commentary

Example 3.2.1: 2 angles welded on a gusset plate



1. System

Gusset plate:	t = 10 mm	Steel grade:	Fe 360
Angle:	2 L 50 x 5	Steel grade:	Fe 360
throat thickness:	a = 3 mm		
weld length:	$\ell = 4 \times 55 + 2 \cdot 50 = 320$ mm		

2. Loading

Tension force: $N_{Sd} = 188$ kN

3. Determination of the resistance of the connection

3.1 Weld resistance

$$\max \left\{ \begin{matrix} 40 \text{ mm} \\ 6 \cdot a = 6 \cdot 3 = 18 \text{ mm} \end{matrix} \right\} < 55 \text{ mm} = \ell < 450 \text{ mm} = 150 \cdot a$$

$$F_{w.Rd} = \frac{f_u}{\sqrt{3}\, \beta_w\, \gamma_{Mw}} a \cdot L = \frac{360}{\sqrt{3} \cdot 0{,}8 \cdot 1{,}25} \cdot 3 \cdot 320 \cdot 10^{-3} = 199{,}5 \text{ kN} > 188 \text{ kN}$$

or $$F_{w.Rd} = \frac{77{,}9}{1{,}25} \cdot \frac{320}{100} = 199{,}4 \text{ kN} > 188 \text{ kN}$$

3.2 Check of Tear-out

$$N_{Rd} = 2\, A_\tau \frac{f_y}{\sqrt{3}\, \gamma_{M0}} + A_\sigma \frac{f_y}{\gamma_{M0}} = \left(2 \cdot 55 \cdot 10 \cdot \frac{235}{\sqrt{3} \cdot 1{,}1} + 50 \cdot 10 \cdot \frac{235}{1{,}1} \right) \cdot 10^{-3} = 242{,}5 \text{ kN} > 188 \text{ kN}$$

As an alternative the check of **Tension resistance of the gusset plate** may be done by assuming a load dispersion with an angle of 30°, see sketch.

$$N_{t.Rd} = A \frac{f_y}{\gamma_{M0}} = (2 \cdot 55 \cdot \tan 30° + 50) \cdot 10 \cdot \frac{235}{1{,}1} \cdot 10^{-3} = 242{,}5 \text{ kN} > 188 \text{ kN}$$

Reference

-
[6.6.10]

-
[6.6.5.2.(2)]

-
[6.6.5; 6.6.9]

6.3.4.3
Table 6.15

Commentary

In Angles connected by one leg, the welded lap joint end connection may be allowed for by adopting an effective cross-sectional area and then treating as concentrically loaded, as presented in this example. A more accurate method can also be used by considering the eccentricity.
For an equal-leg angle the effective area is taken as equal to the gross section.
The throat thickness of a fillet weld should not be less than 3 mm

The design force for checking fillet welds should be taken as a resultant of the forces to be transmitted by the weld.
By using the alternative method [Annex M] the resistance of the weld is:
$F_{w.Rd} = 213{,}6$ kN

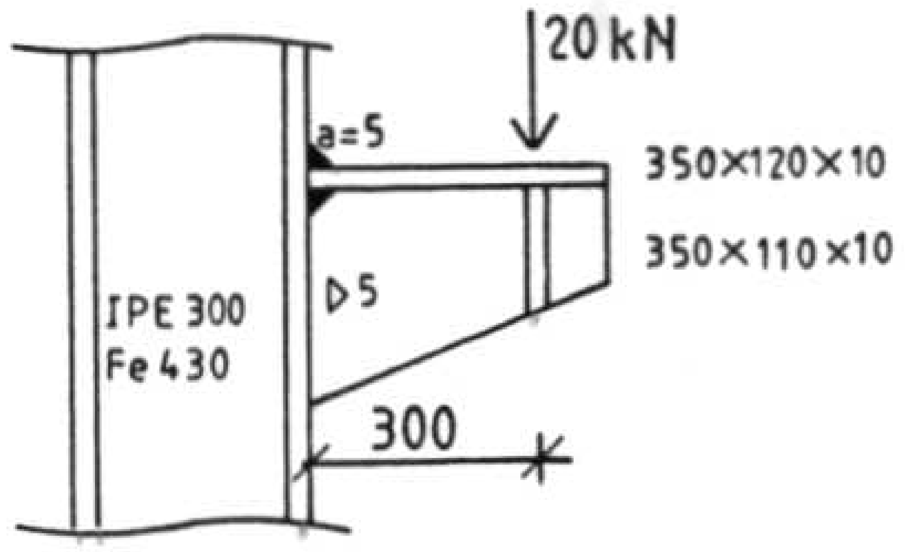
Model used is based on yield strength.

Problem, Sketch, Calculation	Reference	Commentary

Example 3.2.2: Bracket welded on a column

Commentary: Elastic stress distribution in cross-section and in the weld is assumed.

1. System



Column:	IPE 300	Steel grade:	Fe 430
Bracket:	plate 350x120x10	Steel grade:	Fe 430
Bracket:	plate 350x110x10	Steel grade:	Fe 430
Throat thicknesses:	Connection:		
	Flange and web: a = 5 mm		
	Section:		
	a = 3 mm		

2. Loading

Single load: $P = 15{,}0 \text{ kN} \longrightarrow M_{Sd} = 0{,}3 \cdot 15 = 4{,}5 \text{ kNm}$

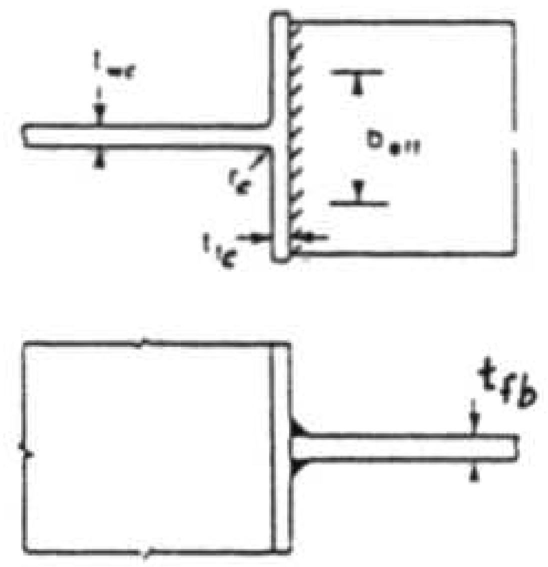
Commentary: Load factors are included

3. Determination of the weld resistance:

3.1 Effective breadth in tension zone:

Reference: 6.3.6 [6.6.8]

Commentary: A reduced effective breadth shall be taken into account both for the parent material and for the welds.



$$b_{eff} = \min \begin{cases} t_{wc} + 2r_c + 7t_{fc} = 7{,}1 + 2 \cdot 15 + 7 \cdot 10{,}7 = 112 \text{ mm} \\ t_{wc} + 2r_c + 7\dfrac{t_{fc}^2}{t_{fb}}\dfrac{f_{yc}}{f_{yb}} = 7{,}1 + 2 \cdot 15 + 7 \cdot \dfrac{10{,}7^2}{10} = 117 \text{ mm} \end{cases}$$

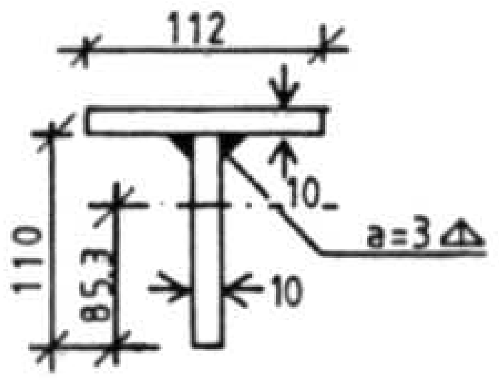
$b_{eff} = 112 \text{ cm} < 120 \text{ cm} = b_c$

$> 0{,}7 \cdot 120 = 84 \text{ mm}$

$\longrightarrow$ the joint need not be stiffened

3.2 Cross-section verification

Commentary: Elastic stress distribution in the cross-section is assumed.



$$e_w = \frac{112 \cdot 10 \cdot 115 + 110 \cdot 10 \cdot 55}{112 \cdot 10 + 110 \cdot 10} = 85{,}3 \text{ mm}$$

$I = 110^3 \cdot 10/12 + 29{,}7^2 \cdot 112 \cdot 10 + 30{,}3^2 \cdot 110 \cdot 10 = 310{,}7 \cdot 10^4 \text{ mm}^4$

$W_u = 310{,}7 \cdot 10^4/85{,}3 = 36{,}4 \cdot 10^3 \text{ mm}^3$

Reference: 5.3.2 [5.4.5]

$$M_{el.Rd} = W_{el} \cdot \frac{f_y}{\gamma_{M0}} = 36{,}4 \cdot 10^3 \cdot \frac{275}{1{,}1} \cdot 10^{-6} = 9{,}1 \text{ kNm} > 4{,}5 \text{ kNm}$$

Reference: 5.3.3 [5.4.6]

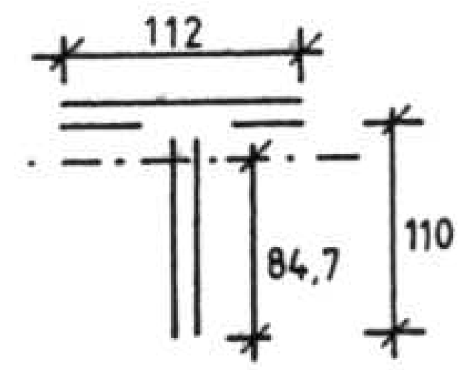
Commentary: No interation bending - shear is required because of $V_{Sd} = 15 \text{ kN} < 0{,}5V_{pl.Rd} = 79{,}4 \text{ kN}$ (5.3.4 [5.4.7])

$$V_{pl.Rd} = \frac{A_v \cdot f_y}{\sqrt{3}\,\gamma_{M0}} = \frac{110 \cdot 10 \cdot 275}{\sqrt{3}\; 1{,}1} \cdot 10^{-3} = 158{,}8 \text{ kN} > 15 \text{ kN}$$

Problem, Sketch, Calculation | **Reference** | **Commentary**

3.3 Area and section modulus of the weld

As elastic stress distribution is assumed in the cross-section, it is also assumed for the weld.



$$A_{wf} = 5 \cdot 112 + 5 \cdot (112 - 10) = 10{,}7 \cdot 10^2\ \text{mm}^2$$

$$A_{ww} = 2 \cdot 110 \cdot 5 = 11{,}0 \cdot 10^2\ \text{mm}^2$$

$$e_w = \frac{112 \cdot 5 \cdot 120 + (112 - 10) \cdot 5 \cdot 110 + 110 \cdot 5 \cdot 2 \cdot 55}{10{,}7 \cdot 10^2 + 11{,}0 \cdot 10^2} = 84{,}7\ \text{mm}$$

$$I_w = 2 \cdot 110^3 \cdot 5/12 + 2 \cdot 29{,}7^2 \cdot 110 \cdot 5 + 25{,}3^2 \cdot (112 - 10) \cdot 5 + 35{,}3^2 \cdot 112 \cdot 5 = 221{,}6 \cdot 10^4\ \text{mm}^4$$

$$W_{w.el} = 221{,}6 \cdot 10^4/84{,}7 = 26{,}2 \cdot 10^3\ \text{mm}^3$$

3.4 Determination of the stresses in the weld

Assumption: Elastic stress distribution in weld

$$\tau_{Sd} = 15 \cdot 10^3/11{,}0 \cdot 10^2 = 13{,}6\ \text{N/mm}^2$$

$$\sigma_{Sd} = 4{,}5 \cdot 10^6/26{,}2 \cdot 10^3 = 171{,}8\ \text{N/mm}^2$$

$$\sigma_{w.Sd} = \sqrt{(13{,}6^2 + 171{,}8^2)} = 172{,}3\ \text{N/mm}^2$$

3.5 Verification

Reference: 6.3.4.3 [6.6.5.3]

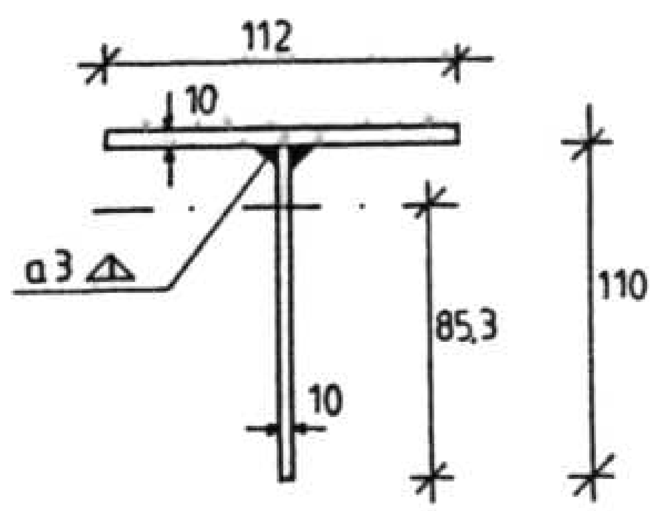
No deformation capacity is necessary for a bracket.
⟶ flange weld need not to develop the full design resistance of the beam flange.

$$f_{w.Rd} = \frac{f_u}{\sqrt{3}\ \gamma_{Mw}\ \beta_w}$$

$$= \frac{430}{\sqrt{3} \cdot 0{,}85 \cdot 1{,}25} = 233{,}7\ \text{N/mm}^2 > 172{,}3\ \text{N/mm}^2$$

3.6 Weld of the section

Elastic stress distribution is assumed.

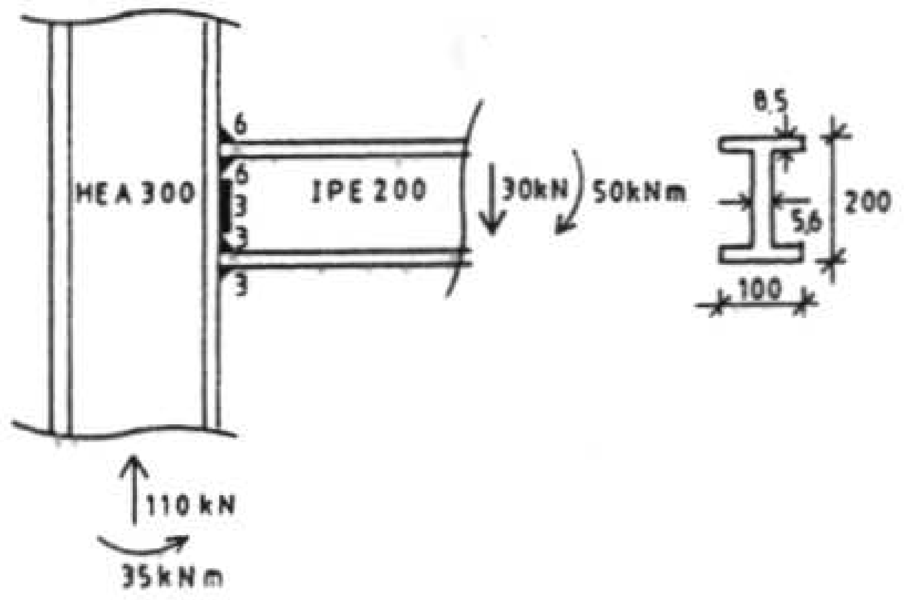


$$S = 112 \cdot 10 \cdot 28{,}7 = 32{,}1 \cdot 10^3\ \text{mm}^3$$

$$\tau_{Sd} = \frac{V_{Sd} \cdot S}{I \cdot 2a} = \frac{15 \cdot 32{,}1 \cdot 10^3}{310{,}7 \cdot 10^4 \cdot 2 \cdot 3} \cdot 10^3 = 25{,}8\ \text{N/mm}^2 < 233{,}7\ \text{N/mm}^2$$

Problem, Sketch, Calculation	Reference	Commentary

Example 3.2.3: Welded beam-to-column connection without stiffeners



1. System

column:	HE 300 A	Steel grade: Fe 360
beam:	IPE 200	Steel grade: Fe 360
throat thickness:		
tension flange:		a = 6 mm
web:		a = 3 mm
compression flange:		a = 3 mm

2. Loading

Bending moment beam:	$M_{b.Sd}$ = 45 kNm
Bending moment column:	$M_{b.Sd}$ = 35 kNm
Shear force:	$V_{b.Sd}$ = 30 kN
Axial force:	$N_{c.Rd}$ = 110 kN

3.Determination of the moment resistance
3.1 Tension zone:
3.1.1 Unstiffened column flange:

6.3.6
[J.2.3.1.(1)]

$$F_{t.Rd} = (f_{yb}\, t_{fb}\, (t_{wc} + 2_{rc}) + 7\,(f_{yc}\, t^2_{fc}))/\gamma_{Mo}$$
$$= (235 \cdot 8{,}5 \cdot (8{,}5 + 2 \cdot 27) + 7 \cdot (235 \cdot 14^2)) \cdot 10^{-3}/1{,}1$$
$$= 406{,}6 \text{ kN}$$

[J.2.3.2.(2)]

or $$F_{t.Rd} \leq f_{yb}\, t_{fb}\, (t_{wc} + 2_{rc} + 7\, t_{fc})/\gamma_{Mo}$$
$$= 235 \cdot 8{,}5(8{,}5 + 2 \cdot 27 + 7 \cdot 14) \cdot 10^{-3}/1{,}1$$
$$= 291{,}5 \text{ kN}$$

$\longrightarrow F_{t.Rd} = 291{,}5$ kN $\quad > 127{,}1 \text{ kN} = 0{,}7 \cdot 235 \cdot 8{,}5 \cdot 100 \cdot 10^{-3}/1{,}1 = 0{,}7 f_{yb} t_{fb} b_{fb}/\gamma_{Mo}$

$\longrightarrow$ no stiffening required

Verification of the weld in tension zone:

6.3.6.(5)
[J.2.3.1.(3)]

The welds connecting the beam flange to the column should be designed to develop the full design resistance of the beam flange equal to $f_{yb} \cdot t_{fb} \cdot b_{fb}/\gamma_{Mo}$

L = 100 + 100 - 5,6 - 2•12 = 170,4 mm

$$N_{pl.Rd} = b_{fb}\, t_{fb}\, \frac{f_{yb}}{\gamma_{Mo}} = \frac{100 \cdot 8{,}5 \cdot 235}{1{,}1} \cdot 10^{-3} = 181{,}6 \text{ kN}$$

$$F_{w.Rd} = \frac{a\,L}{\sqrt{3}\,\beta_w}\,\frac{f_u}{\gamma_{Mw}} = \frac{6 \cdot 170{,}4 \cdot 360}{\sqrt{3} \cdot 0{,}8 \cdot 1{,}25} \cdot 10^{-3} = 212{,}5 \text{ kN}$$

3.1.2 Unstiffened column web:

-
[J.2.3.2]

$$F_{t.Rd} = f_{yc} \cdot t_{wc} \cdot b_{eff}/\gamma_{Mo}$$
$$b_{eff} = t_{fb} + 2\sqrt{2}\, a_b + 5\,(t_{fc} + r_c)$$
$$b_{eff} = 8{,}5 + 2\sqrt{2} \cdot 6 + 5\,(14 + 27) = 230{,}5 \text{ mm}$$
$$F_{t.Rd} = 235 \cdot 8{,}5 \cdot 230{,}5 \cdot 10^{-3}/1{,}1 = 418{,}6 \text{ kN}$$

$\longrightarrow$ Resistance of the tension zone: $F_{t.Rd}$ = 291,5 kN

Problem, Sketch, Calculation	Reference	Commentary

3.2 Compression zone

3.2.1 Design crushing resistance of unstiffened column web

Reference: - [J.2.4.1]

$$F_{c.Rd} = f_{yc} t_{wc} \left[1{,}25 - 0{,}5\frac{\sigma_n}{f_{yc}}\right] b_{eff} \frac{1}{\gamma_{M0}}$$

$z = ½(h - 2t_f) = ½(290 - 2•14) = 131$ mm

$\sigma_{n.Ed} = N_{c.Sd}/A_c + M_{c.Sd}/I \cdot z = 110•10^3/113•10^2 + 35•10^6•131/18260•10^4 = 35\ N/mm^2$

$1{,}25 - 0{,}5\gamma_{M0}\sigma_{n.Ed}/f_{yc} = 1{,}25 - 0{,}5•1{,}1•35/235 = 1{,}17 > 1{,}0$

$b_{eff} = t_{fb} + 2\sqrt{2}\, a_b + 5\,(t_{fc} + r_c)$

$= 8{,}5 + 2\{2 \cdot 3 + 5\,(14 + 27) = 222$ cm

$F_{c.Rd} = f_{yb}\, t_{wc}\, b_{eff}/\gamma_{M0}$

$= 235 \cdot 8{,}5 \cdot 222•10^{-3}/1{,}1 = 403{,}1$ kN

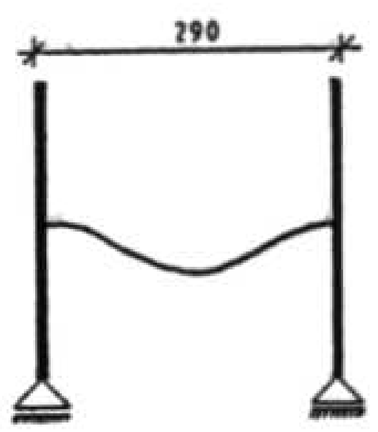
Commentary: σ_n - the maximum compressive normal stress in the web of the column due to axial force and bending.

The compression force is introduced by contact.Therefore in the compression zone the minimum throat thickness may be chosen as a = 3 mm.

3.2.2 Design resistance of the column web to buckling:

Non-sway mode:

Reference: 5.6.5 [J.2.4.1.(3); 5.7.5]



$b_{eff} = h_c = 290$ mm

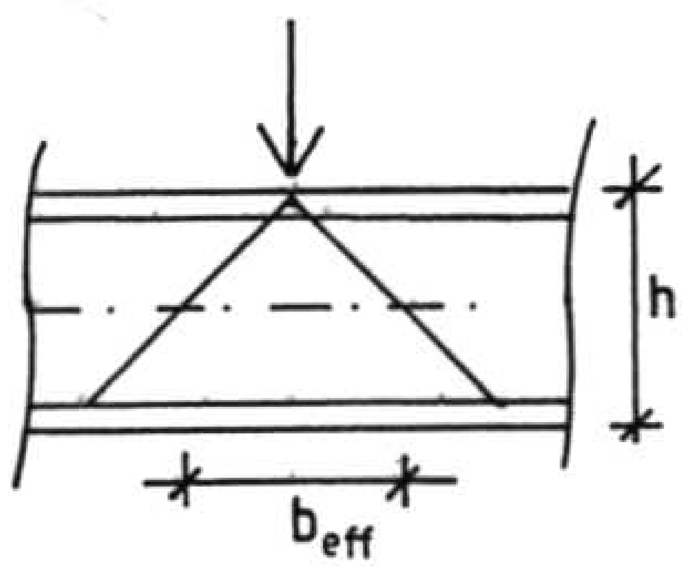
The buckling length of the virtual compression member should be determined from the conditions of at lateral and rotational restraint at the flanges at the point of load application:

here: $\ell = 0{,}6•h = 0{,}6•290 = 174$ mm

$$I = \frac{290 \cdot 8{,}5^3}{12} = 1{,}484 \cdot 10^4\ mm^4$$

$A = 290•8{,}5 = 24{,}65•10^2\ mm^2$

$$i = \sqrt{\frac{I}{A}} = \sqrt{\frac{1{,}484 \cdot 10^4}{24{,}65 \cdot 10^2}} = 2{,}454\ mm$$



$$\bar{\lambda} = \frac{174}{2{,}454}\,\frac{1}{93{,}9} = 0{,}76$$

Using buckling curve c ⟶ $\chi = 0{,}6873$

$R_{b.Rd} = 0{,}6873•8{,}5•29•235•10^{-3}/1{,}1 = 361{,}9$ kN

⟶ Resistance of the compression zone: $F_{c.Rd} = 361{,}9$ kN

Commentary: The sway mode should normally be prevented by constructional restraint.

The buckling resistance $R_{b.Rd}$ should be obtained by considering the web as a compression member with an effective breadth b_{eff} obtained from Table 5.39 [Figure 5.7.3].

Problem, Sketch, Calculation	Reference	Commentary

3.3 Resistance of the shear zone:

Reference: 5.3.3 [J.2.5]

$d/t = 208/8{,}5 = 24{,}5 < 69 = 69\varepsilon \longrightarrow$ no shear buckling verification required

$$V_{Rd} = \frac{f_{yc}\, A_v}{\sqrt{3}\ \gamma_{Mo}} = \frac{235 \cdot 24{,}6 \cdot 10^2}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 303{,}4 \text{ kN}$$

$\longrightarrow$ Resistance of the shear zone: $F_{c.Rd} = 303{,}4$ kN

3.4 Bending moment of the connection:

$$\longrightarrow M_{Rd} = F_{min}\,(h_b - t_{fb})$$

Potential failure modes:

Tension zone $F_{t.Rd} = 291{,}5$ kN
Compression zone $F_{c.Rd} = 361{,}9$ kN
Shear zone $F_{v.Rd} = 303{,}4$ kN

$$\longrightarrow F_{min} = F_{t.Rd} = 291{,}5 \text{ kN}$$

$$M_{Rd} = 291{,}5 \cdot (200 - 8{,}5)\ 10^{-3} = 55{,}8 \text{ kNm} > 45{,}0 \text{ kNm}$$

3.5 Cross-section resistance of the beam

Reference: 5.3.3 [5.4.6]

Shear resistance: $$V_{Rd} = \frac{A_v\, f_y}{\sqrt{3}\ \gamma_{Mo}} = \frac{11{,}2 \cdot 10^2 \cdot 235}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 138{,}1 \text{ kN} > 30 \text{ kN}$$

Reference: 5.3.4 [5.4.7]

$0{,}5V_{Rd} = 69{,}1$ kN > 30 kN $= V_{Sd} \longrightarrow$ no interaction bending - shear is needed

Reference: 5.3.2 [5.4.5]

Moment resistance: $$M_{Rd} = W_{pl}\,\frac{f_y}{\gamma_{Mo}} = 221 \cdot 10^3 \cdot \frac{235}{1{,}1} \cdot 10^{-6} = 47{,}2 \text{ kNm} > 45 \text{ kNm}$$

3.6 Verification of the "web weld"

Reference: [6.6.5.3]

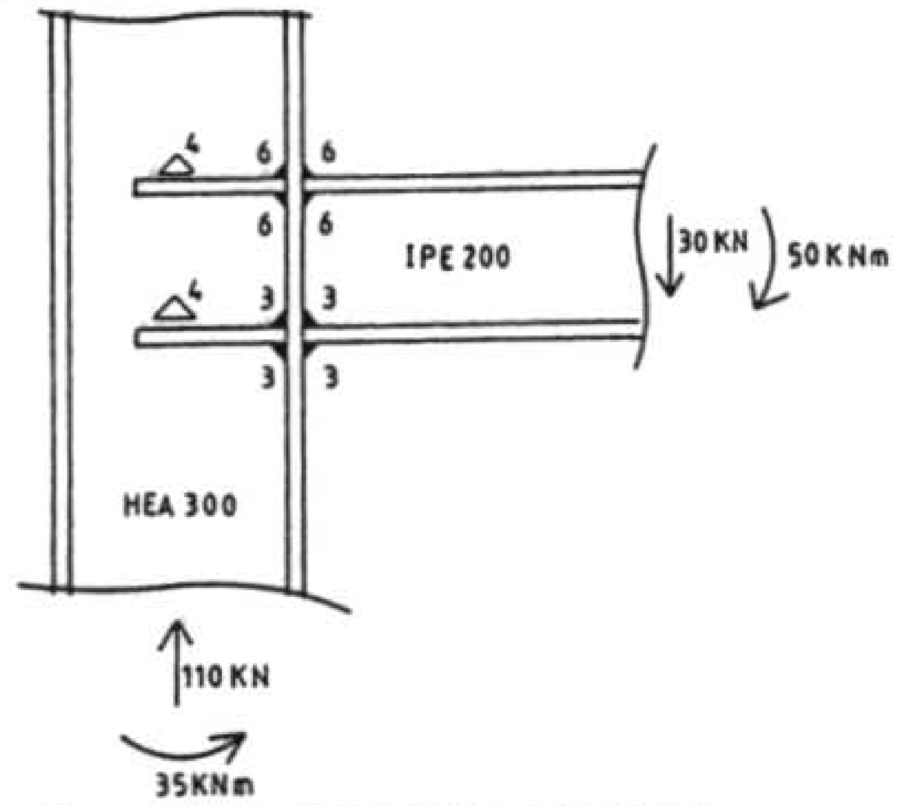
$$F_{w.Rd} = \frac{f_u}{\sqrt{3}\ \beta_w\ \gamma_{Mw}}\, a \cdot L = \frac{360}{\sqrt{3} \cdot 0{,}8 \cdot 1{,}25} \cdot 3 \cdot 159 \cdot 2 \cdot 10^{-3} = 198{,}3 \text{ kN} > 110 \text{ kN}$$

Reference: 6.3.4.3 Table 6.15

or $$F_{w.Rd} = \frac{77{,}9}{1{,}25} \cdot \frac{2 \cdot 159}{100} = 198{,}2 \text{ kN} > 110 \text{ kN}$$

Problem, Sketch, Calculation	Reference	Commentary

Example 3.2.4: Welded beam-to-column connection with stiffeners



1. System

column:	HE 300 A	Steel grade: Fe 360
beam:	IPE 200	Steel grade: Fe 360
throat thickness:		
tension flange:		a = 6 mm
web:		a = 3 mm
compression flange:		a = 3 mm

2. Loading

Bending moment beam:	$M_{b.Sd}$ = 45 kNm
Bending moment column:	$M_{b.Sd}$ = 35 kNm
Shear force:	$V_{b.Sd}$ = 30 kN
Axial force:	$N_{c.Rd}$ = 110 kN

Reference: 6.3.6 [J.2.3.1.(1)]

3.Determination of the moment resistance

3.1 Tension and compression zone:

The design resistance of a stiffened column subject to a transverse tensile or compressive force is at least equal to the design resistance of the beam flange, provided the thickness of the stiffener is not smaller than the flange thickness of the beam. The welds need sufficient deformation capacity.

Reference: [J.2.3.2.(2)]

Reference: 6.3.6.(5) [J.2.3.1.(3)]

Verification of the weld between the stiffeners and the column flanges in tension zone:

L = 100 + 100 - 5,6 - 2•12 = 170,4 mm

$$N_{pl.Rd} = b_{fb}\, t_{fb}\, \frac{f_{yb}}{\gamma_{Mo}} = \frac{100 \cdot 8{,}5 \cdot 235}{1{,}1} \cdot 10^{-3} = 181{,}6 \text{ kN}$$

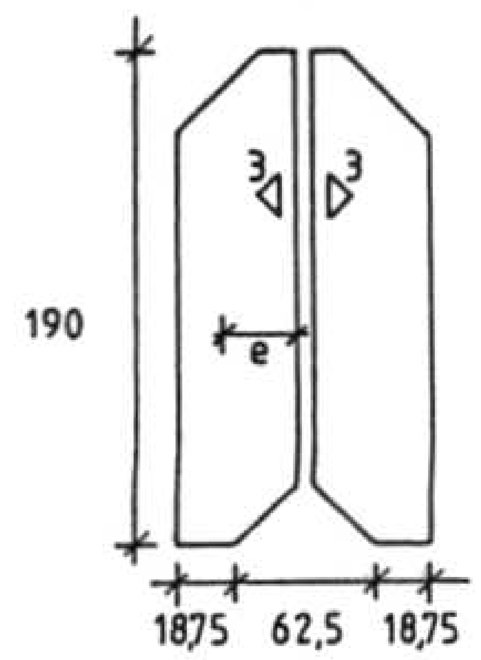
$$F_{w.Rd} = \frac{a\, L}{\sqrt{3}\, \beta_w} \frac{f_u}{\gamma_{Mw}} = \frac{6 \cdot 170{,}4 \cdot 360}{\sqrt{3} \cdot 0{,}8 \cdot 1{,}25} \cdot 10^{-3} = 212{,}5 \text{ kN}$$

Commentary: The welds connecting the beam flange to the column should be designed to develop the full design resistance of the beam flange equal to $f_{yb} \cdot t_{fb} \cdot b_{fb} / \gamma_{Mo}$

Verification of the weld between the stiffeners and the column web

Reference: - [J.2.3.3]

Commentary: st - stiffener



$$f_{w.Rd} = \frac{f_u}{\sqrt{3}\, \beta_w\, \gamma_{Mw}} = \frac{360}{\sqrt{3} \cdot 0{,}8 \cdot 1{,}25} = 207{,}8 \text{ N/mm}^2$$

max $F_{st.Sd} = f_{yb}\, t_{fb}\, b_{fb}/\gamma_{Mo}$ = 235•8,5•18,75•10^{-3}/1,1 = 34,0 kN ; A_w = 2•160•3 = 9,6•10^2 mm^2

M_{Sd} = max $F_{st.Sd}$•e = 34,0•36,375•10^{-3} = 1,24 kNm ; W_w = 2•160^2•3/6 = 25,6•10^3 mm^3

τ_{Sd} = 34,0•10^3/9,6•10^2 = 35,4 N/mm^2

σ_{Sd} = 1,24•10^6/25,6•10^3 = 48,4 N/mm^2

$\sigma_{w.Sd} = \sqrt{(35{,}4^2 + 48{,}4^2)}$ = 60,0 N/mm^2 < 207,8 N/mm^2 = $f_{vw.d}$

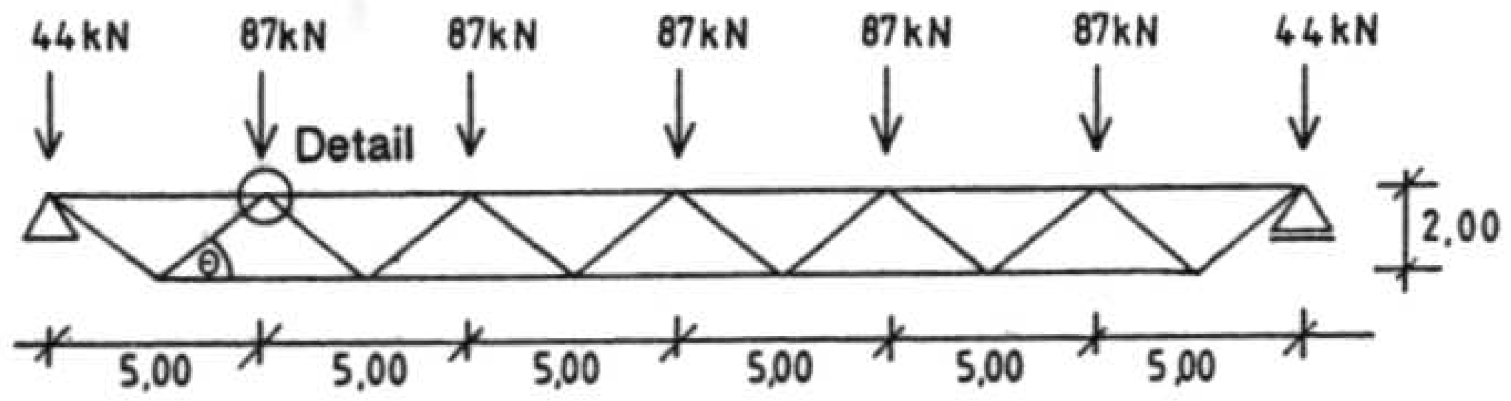
Problem, Sketch, Calculation	Reference	Commentary
3.3 Resistance of the shear zone:	5.3.3	
$d/t = 208/8{,}5 = 24{,}5 < 69 = 69\varepsilon \longrightarrow$ no shear buckling verification required	[J.2.5]	
$V_{Rd} = \dfrac{f_{yc}\,A_v}{\sqrt{3}\,\gamma_{Mo}} = \dfrac{235 \cdot 24{,}6 \cdot 10^2}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 303{,}4\ kN$		
$\longrightarrow$ Resistance of the shear zone: $F_{c.Rd} = 303{,}4\ kN$		
3.4 Bending moment of the connection:		
$\longrightarrow M_{Rd} = F_{min}\,(h_b - t_{fb})$		
Potential failure modes: Shear zone $F_{v.Rd} = 303{,}4\ kN$		
$\longrightarrow F_{min} = F_{t.Rd} = 303{,}4\ kN$		
$M_{Rd} = 303{,}4 \cdot (200 - 8{,}5)\ 10^{-3} = 58{,}1\ kNm > 45{,}0\ kNm$		
3.5 Cross-section resistance of the beam		
Shear resistance: $V_{Rd} = \dfrac{A_v\,f_y}{\sqrt{3}\,\gamma_{Mo}} = \dfrac{11{,}2 \cdot 10^2 \cdot 235}{\sqrt{3} \cdot 1{,}1} \cdot 10^{-3} = 138{,}1\ kN > 30\ kN$	5.3.3 [5.4.6]	
$0{,}5V_{Rd} = 69{,}1\ kN > 30\ kN = V_{Sd} \longrightarrow$ no interaction bending - shear is needed	5.3.4 [5.4.7]	
Moment resistance: $M_{Rd} = W_{pl}\,\dfrac{f_y}{\gamma_{Mo}} = 221 \cdot 10^3 \cdot \dfrac{235}{1{,}1} \cdot 10^{-6} = 47{,}2\ kNm > 45\ kNm$	5.3.2 [5.4.5]	
$\longrightarrow$ the beam-to-column connection is classified as full strength [6.4.3.2; 6.9.6.3]		

Problem, Sketch, Calculation	Reference	Commentary

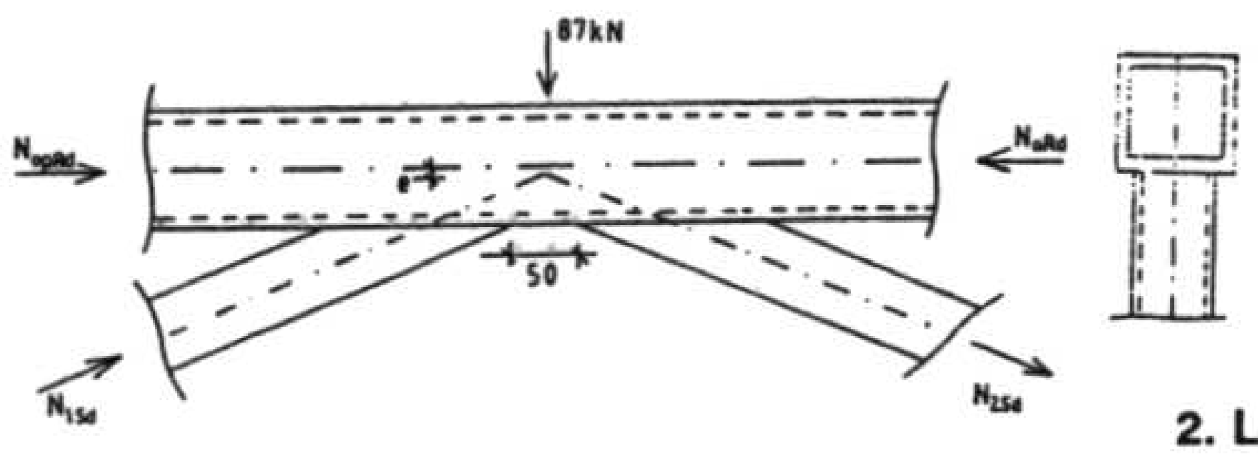
Example 3.2.5: Hollow section lattice girder joint

Here: No member design is included; however the joint eccentricity has to be taken into account in the compression chord member verification. Buckling verification of the bracing member is also required.

1. System



Detail:



Chord member: RHS 160 x 7,1; hot finished Steel grade Fe 510
Bracing member: RHS 90 x 5,6; hot finished Steel grade Fe 430
chosen: gap g = 50 mm

2. Loading

$N_{o.Sd}$ = 880 kN (Compression force)
$N_{op.Sd}$ = 365 kN (Compression force)
$N_{1.Sd}$ = 400 kN (Compression force)
$N_{2.Sd}$ = 260 kN (Tension force)

3. Determination of the resistance

[Table K.7.1]

For joints outside the range of validity a more detailed analysis should be made as specified in [6.10.1].

3.1 Range of validity

$\beta = b_i/b_o = 90/160 = 0{,}56 \quad > 0{,}35$

$\qquad\qquad\qquad\qquad\quad > 0{,}1 + 0{,}01 \cdot 160/7.1 = 0{,}33$

$b_i/t_i = 90/5{,}6 = 16 < 1{,}25\sqrt{(E/f_{yi})} = 1{,}25\sqrt{(210000/275)} = 34{,}5$

$15 \leq b_o/t_o = 160/7{,}1 = 22{,}5 < 35$

$$0{,}6 \leq \frac{b_1 + b_2}{2\, b_1} = 1{,}0 \leq 1{,}3$$

$0{,}5(1 - \beta) = 0{,}5(1 - 0{,}56) = 0{,}22 \leq g/b_o = 50/160 = 0{,}3125 \leq 1{,}5(1 - \beta) = 1{,}5(1 - 0{,}56) = 0{,}66$
$g = 50 \text{ mm} > t_1 + t_2 = 2 \cdot 5{,}6 = 11{,}2 \text{ mm}$

$\Theta_i = 38{,}7^\circ > 30^\circ$

Problem, Sketch, Calculation	Reference	Commentary

3.2 Joint resistance

Verification of the eccentricity:

-
[K.3.(1)]

$$e = \left(\frac{h_1}{2\,\sin\theta_1} + \frac{h_2}{2\,\sin\theta_2} + g\right)\frac{\sin\theta_1\,\sin\theta_2}{\sin(\theta_1 + \theta_2)} - \frac{h_o}{2}$$

$$= \left(\frac{90}{\sin 38{,}7} + 50\right)\frac{\sin^2 38{,}7}{\sin(38{,}7 + 38{,}7)} - \frac{160}{2} = -2{,}3 \text{ mm}$$

-2,3/160 = - 0,01 > - 0,55 ---> satisfied

Determination of the efficiency:

-
[K.7.2; Table K.7.2]

$$n = \frac{N_{o.Sd}\,\gamma_{Mo}}{A f_{yo}} = \frac{880 \cdot 1{,}1 \cdot 10^3}{41{,}7 \cdot 10^2 \cdot 355} = 0{,}65$$

$$k_n = 1{,}3 - \frac{0{,}4n}{\beta} = 1{,}3 - \frac{0{,}4 \cdot 0{,}65}{0{,}56} = 0{,}84 < 1{,}0$$

$$N_{iRd} = 8{,}9\,\frac{f_{yo}\,t_o^2}{\sin\theta_i}\,\beta\gamma\,k_n\,\frac{1{,}1}{\gamma_{Mj}}$$

$$= 8{,}9\,\frac{35{,}5 \cdot 0{,}71^2}{\sin 38{,}66}\,\frac{90}{160}\sqrt{\frac{16}{2 \cdot 0{,}71}}\,0{,}84 = 404{,}4\ kN$$

$N_{i.Sd} < 465{,}8$ kN $= N_{pl.Rd} = A f_{yi}/\gamma_{Mo}$

3.3 Weld design

[K.5.(4)]

If the criterion in [K.5.(4)] is fulfilled the resistance of the weld is equal or higher than the member resistance.

$$a \geq 0{,}87\,\frac{1{,}1}{\gamma_{Mj}}\,\frac{\gamma_{Mw}}{1{,}25}\,t_i$$

$$= 0{,}87 \cdot \frac{1{,}1}{1{,}1}\,\frac{1{,}25}{1{,}25} \cdot 5{,}6 = 4{,}9 \text{ mm}$$

chosen: a = 5 mm

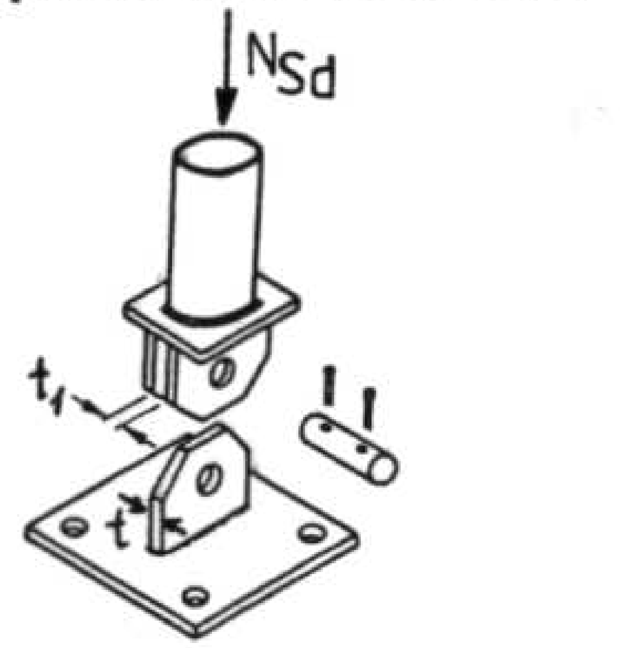
3.3 Pin connections

This example demonstrates the verification of pin connections assuming design values of action effects (N_{Sd}, V_{Sd}, M_{Sd}, etc) which have been calculated by an analysis of the structure and these values already include ψ, γ_F etc.
The results presented in the examples are rounded values. For the purpose of easy re-calculation each formula and each check is calculated with the rounded values.

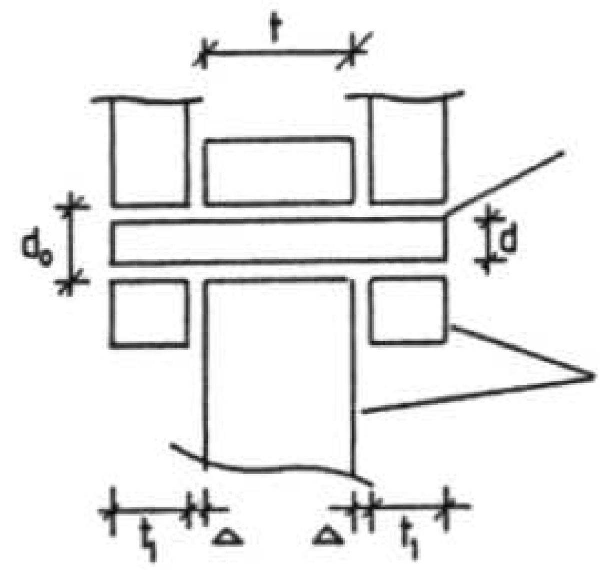
Example 3.3.1: Pin connection

Problem, Sketch, Calculation	Reference	Commentary

Example 3.3.1: Pin connection



1. **System**
Steel grade: Fe 430
pin: M 36, 8.8
$\longrightarrow d_o = 39$ mm
$d = 36$ mm, $\Delta = 1$ mm
$t_1 = 12$ mm, $t = 20$ mm



Commentary: Such a connection is used if free rotation is required.

2. **Loading**
$N_{Sd} = 230{,}0$ kN (compression force)
$M_{Sd} = N_{Sd} \cdot (20 + 4 \cdot 1 + 2 \cdot 12) \cdot 10^{-3}/8 = 1{,}38$ kNm (bending moment in the bolt)

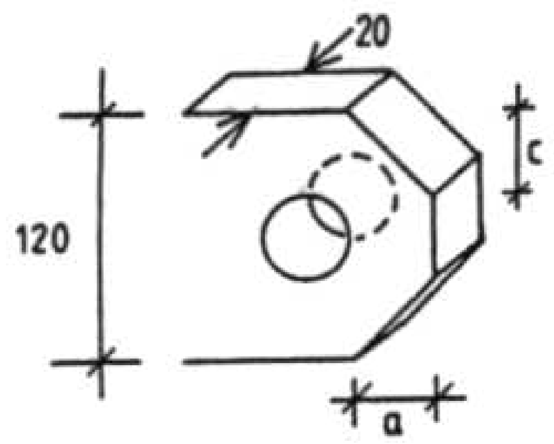
Reference: - [Figure 6.5.12]

Commentary: The bending moment in the bolt is calculated by $M_{Sd} = N_{Sd} \cdot (t + 4\Delta + 2t_1)/8$

3. **Determination of the pin resistance**
3.1 **Geometrical conditions for plate by plate thickness t = 20 mm:**

Reference: - [Table 6.5.6]

Commentary: Pin connections should be arranged to avoid eccentricity



$$a > \frac{N_{Sd}\,\gamma_{Mp}}{2\,t\,f_y} + \frac{2\,d_o}{3} = \frac{230 \cdot 1{,}25 \cdot 10^3}{2 \cdot 20 \cdot 275} + \frac{2 \cdot 39}{3} = 52 \text{ mm}$$

chosen: a = 60 mm

$$c > \frac{N_{Sd}\,\gamma_{Mp}}{2\,t\,f_y} + \frac{d_o}{3} = \frac{230 \cdot 1{,}25 \cdot 10^3}{2 \cdot 20 \cdot 275} + \frac{39}{3} = 39 \text{ mm}$$

chosen: c = 40 mm

3.2 **Shear resistance of the pin**

Reference: - [Table 6.5.7]

$$F_{v.Rd} = 0{,}6\,A\,\frac{f_{up}}{\gamma_{Mp}} = 0{,}6\,\frac{\pi \cdot 36^2}{4}\,\frac{800}{1{,}25} \cdot 10^{-3} = 390{,}9 \text{ kN} > 230{,}0 \text{ kN}$$

3.3 **Bearing resistance of plate and pin**

Reference: - [Table 6.5.7]

$$F_{b.Rd} = 1{,}5\,t\,d\,\frac{f_y}{\gamma_{Mp}} = 1{,}5 \cdot 20 \cdot 36\,\frac{275}{1{,}25} \cdot 10^{-3} = 237{,}6 \text{ kN} > 230{,}0 \text{ kN}$$

3.4 **Bending resistance of pin**

Reference: - [Table 6.5.7]

Commentary: $W_{el} = \pi d^3/32$

$$M_{Rd} = 0{,}8\,W_{el}\,\frac{f_{yp}}{\gamma_{Mp}} = 0{,}8 \cdot \frac{\pi \cdot 36^3}{32}\,\frac{640}{1{,}25} \cdot 10^{-6} = 1{,}88 \text{ kNm} > 1{,}38 \text{ kNm}$$

3.5 **Interaction shear and bending**

Reference: - [Table 6.5.7]

$$\left(\frac{M_{Sd}}{M_{Rd}}\right)^2 + \left(\frac{F_{v.Sd}}{F_{v.Rd}}\right)^2 = \left(\frac{1{,}38}{1{,}88}\right)^2 + \left(\frac{230{,}0}{390{,}9}\right)^2 = 0{,}89 < 1{,}0$$

Annex A

Values of the reduction factor χ for
the appropriate non-dimensional slenderness $\bar{\lambda}$
of buckling curves a_o, a, b, c, d
are given in the following tables

Reduction factors χ for buckling curve a_0 (α = 0,13)											
$\bar{\lambda}$	0,00	0,01	0,02	0,03	0,04	0,05	0,06	0,07	0,08	0,09	
0,00	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	0,00
0,10	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	0,10
0,20	1,0000	0,9986	0,9973	0,9959	0,9945	0,9931	0,9917	0,9903	0,9889	0,9874	0,20
0,30	0,9859	0,9845	0,9829	0,9814	0,9799	0,9783	0,9767	0,9751	0,9735	0,9718	0,30
0,40	0,9701	0,9684	0,9667	0,9649	0,9631	0,9612	0,9593	0,9574	0,9554	0,9534	0,40
0,50	0,9513	0,9492	0,9470	0,9448	0,9425	0,9402	0,9378	0,9354	0,9328	0,9302	0,50
0,60	0,9276	0,9248	0,9220	0,9191	0,9161	0,9130	0,9099	0,9066	0,9032	0,8997	0,60
0,70	0,8961	0,8924	0,8886	0,8847	0,8806	0,8764	0,8721	0,8676	0,8630	0,8582	0,70
0,80	0,8533	0,8483	0,8431	0,8377	0,8322	0,8266	0,8208	0,8148	0,8087	0,8025	0,80
0,90	0,7961	0,7895	0,7828	0,7760	0,7691	0,7620	0,7549	0,7476	0,7403	0,7329	0,90
1,00	0,7253	0,7178	0,7101	0,7025	0,6948	0,6870	0,6793	0,6715	0,6637	0,6560	1,00
1,10	0,6482	0,6405	0,6329	0,6252	0,6176	0,6101	0,6026	0,5951	0,5877	0,5804	1,10
1,20	0,5732	0,5660	0,5590	0,5520	0,5450	0,5382	0,5314	0,5248	0,5182	0,5117	1,20
1,30	0,5053	0,4990	0,4927	0,4866	0,4806	0,4746	0,4687	0,4629	0,4572	0,4516	1,30
1,40	0,4461	0,4407	0,4353	0,4300	0,4248	0,4197	0,4147	0,4097	0,4049	0,4001	1,40
1,50	0,3953	0,3907	0,3861	0,3816	0,3772	0,3728	0,3685	0,3643	0,3601	0,3560	1,50
1,60	0,3520	0,3480	0,3441	0,3403	0,3365	0,3328	0,3291	0,3255	0,3219	0,3184	1,60
1,70	0,3150	0,3116	0,3083	0,3050	0,3017	0,2985	0,2954	0,2923	0,2892	0,2862	1,70
1,80	0,2833	0,2804	0,2775	0,2746	0,2719	0,2691	0,2664	0,2637	0,2611	0,2585	1,80
1,90	0,2559	0,2534	0,2509	0,2485	0,2461	0,2437	0,2414	0,2390	0,2368	0,2345	1,90
2,00	0,2323	0,2301	0,2280	0,2258	0,2237	0,2217	0,2196	0,2176	0,2156	0,2136	2,00
2,10	0,2117	0,2098	0,2079	0,2061	0,2042	0,2024	0,2006	0,1989	0,1971	0,1954	2,10
2,20	0,1937	0,1920	0,1904	0,1887	0,1871	0,1855	0,1840	0,1824	0,1809	0,1794	2,20
2,30	0,1779	0,1764	0,1749	0,1735	0,1721	0,1707	0,1693	0,1679	0,1665	0,1652	2,30
2,40	0,1639	0,1626	0,1613	0,1600	0,1587	0,1575	0,1563	0,1550	0,1538	0,1526	2,40
2,50	0,1515	0,1503	0,1491	0,1480	0,1469	0,1458	0,1447	0,1436	0,1425	0,1414	2,50
2,60	0,1404	0,1394	0,1383	0,1373	0,1363	0,1353	0,1343	0,1333	0,1324	0,1314	2,60
2,70	0,1305	0,1296	0,1286	0,1277	0,1268	0,1259	0,1250	0,1242	0,1233	0,1224	2,70
2,80	0,1216	0,1207	0,1199	0,1191	0,1183	0,1175	0,1167	0,1159	0,1151	0,1143	2,80
2,90	0,1136	0,1128	0,1120	0,1113	0,1106	0,1098	0,1091	0,1084	0,1077	0,1070	2,90
3,00	0,1063	0,1056	0,1049	0,1043	0,1036	0,1029	0,1023	0,1016	0,1010	0,1003	3,00
3,10	0,0997	0,0991	0,0985	0,0979	0,0972	0,0966	0,0960	0,0955	0,0949	0,0943	3,10
3,20	0,0937	0,0931	0,0926	0,0920	0,0915	0,0909	0,0904	0,0898	0,0893	0,0888	3,20
3,30	0,0882	0,0877	0,0872	0,0867	0,0862	0,0857	0,0852	0,0847	0,0842	0,0837	3,30
3,40	0,0832	0,0828	0,0823	0,0818	0,0814	0,0809	0,0804	0,0800	0,0795	0,0791	3,40
3,50	0,0786	0,0782	0,0778	0,0773	0,0769	0,0765	0,0761	0,0756	0,0752	0,0748	3,50
3,60	0,0744	0,0740	0,0736	0,0732	0,0728	0,0724	0,0720	0,0717	0,0713	0,0709	3,60

Reduction factors χ for buckling curve a $(\alpha = 0,21)$

$\bar{\lambda}$	0,00	0,01	0,02	0,03	0,04	0,05	0,06	0,07	0,08	0,09	
0,00	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	0,00
0,10	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	0,10
0,20	1,0000	0,9978	0,9956	0,9934	0,9912	0,9889	0,9867	0,9844	0,9821	0,9798	0,20
0,30	0,9775	0,9751	0,9728	0,9704	0,9680	0,9655	0,9630	0,9605	0,9580	0,9554	0,30
0,40	0,9528	0,9501	0,9474	0,9447	0,9419	0,9391	0,9363	0,9333	0,9304	0,9273	0,40
0,50	0,9243	0,9211	0,9179	0,9147	0,9114	0,9080	0,9045	0,9010	0,8974	0,8937	0,50
0,60	0,8900	0,8862	0,8823	0,8783	0,8742	0,8700	0,8657	0,8614	0,8569	0,8524	0,60
0,70	0,8477	0,8430	0,8382	0,8332	0,8282	0,8230	0,8178	0,8124	0,8069	0,8014	0,70
0,80	0,7957	0,7899	0,7841	0,7781	0,7721	0,7659	0,7597	0,7534	0,7470	0,7405	0,80
0,90	0,7339	0,7273	0,7206	0,7139	0,7071	0,7003	0,6934	0,6865	0,6796	0,6726	0,90
1,00	0,6656	0,6586	0,6516	0,6446	0,6376	0,6306	0,6236	0,6167	0,6098	0,6029	1,00
1,10	0,5960	0,5892	0,5824	0,5757	0,5690	0,5623	0,5557	0,5492	0,5427	0,5363	1,10
1,20	0,5300	0,5237	0,5175	0,5114	0,5053	0,4993	0,4934	0,4875	0,4817	0,4760	1,20
1,30	0,4703	0,4648	0,4593	0,4538	0,4485	0,4432	0,4380	0,4329	0,4278	0,4228	1,30
1,40	0,4179	0,4130	0,4083	0,4036	0,3989	0,3943	0,3898	0,3854	0,3810	0,3767	1,40
1,50	0,3724	0,3682	0,3641	0,3601	0,3561	0,3521	0,3482	0,3444	0,3406	0,3369	1,50
1,60	0,3332	0,3296	0,3261	0,3226	0,3191	0,3157	0,3124	0,3091	0,3058	0,3026	1,60
1,70	0,2994	0,2963	0,2933	0,2902	0,2872	0,2843	0,2814	0,2786	0,2757	0,2730	1,70
1,80	0,2702	0,2675	0,2649	0,2623	0,2597	0,2571	0,2546	0,2522	0,2497	0,2473	1,80
1,90	0,2449	0,2426	0,2403	0,2380	0,2358	0,2335	0,2314	0,2292	0,2271	0,2250	1,90
2,00	0,2229	0,2209	0,2188	0,2168	0,2149	0,2129	0,2110	0,2091	0,2073	0,2054	2,00
2,10	0,2036	0,2018	0,2001	0,1983	0,1966	0,1949	0,1932	0,1915	0,1899	0,1883	2,10
2,20	0,1867	0,1851	0,1836	0,1820	0,1805	0,1790	0,1775	0,1760	0,1746	0,1732	2,20
2,30	0,1717	0,1704	0,1690	0,1676	0,1663	0,1649	0,1636	0,1623	0,1610	0,1598	2,30
2,40	0,1585	0,1573	0,1560	0,1548	0,1536	0,1524	0,1513	0,1501	0,1490	0,1478	2,40
2,50	0,1467	0,1456	0,1445	0,1434	0,1424	0,1413	0,1403	0,1392	0,1382	0,1372	2,50
2,60	0,1362	0,1352	0,1342	0,1332	0,1323	0,1313	0,1304	0,1295	0,1285	0,1276	2,60
2,70	0,1267	0,1258	0,1250	0,1241	0,1232	0,1224	0,1215	0,1207	0,1198	0,1190	2,70
2,80	0,1182	0,1174	0,1166	0,1158	0,1150	0,1143	0,1135	0,1128	0,1120	0,1113	2,80
2,90	0,1105	0,1098	0,1091	0,1084	0,1077	0,1070	0,1063	0,1056	0,1049	0,1042	2,90
3,00	0,1036	0,1029	0,1022	0,1016	0,1010	0,1003	0,0997	0,0991	0,0985	0,0978	3,00
3,10	0,0972	0,0966	0,0960	0,0954	0,0949	0,0943	0,0937	0,0931	0,0926	0,0920	3,10
3,20	0,0915	0,0909	0,0904	0,0898	0,0893	0,0888	0,0882	0,0877	0,0872	0,0867	3,20
3,30	0,0862	0,0857	0,0852	0,0847	0,0842	0,0837	0,0832	0,0828	0,0823	0,0818	3,30
3,40	0,0814	0,0809	0,0804	0,0800	0,0795	0,0791	0,0786	0,0782	0,0778	0,0773	3,40
3,50	0,0769	0,0765	0,0761	0,0757	0,0752	0,0748	0,0744	0,0740	0,0736	0,0732	3,50
3,60	0,0728	0,0724	0,0721	0,0717	0,0713	0,0709	0,0705	0,0702	0,0698	0,0694	3,60

Reduction factors χ for buckling curve b (α = 0,34)

$\bar{\lambda}$	0,00	0,01	0,02	0,03	0,04	0,05	0,06	0,07	0,08	0,09	
0,00	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	0,00
0,10	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	0,10
0,20	1,0000	0,9965	0,9929	0,9894	0,9858	0,9822	0,9786	0,9750	0,9714	0,9678	0,20
0,30	0,9641	0,9604	0,9567	0,9530	0,9492	0,9455	0,9417	0,9378	0,9339	0,9300	0,30
0,40	0,9261	0,9221	0,9181	0,9140	0,9099	0,9057	0,9015	0,8973	0,8930	0,8886	0,40
0,50	0,8842	0,8798	0,8752	0,8707	0,8661	0,8614	0,8566	0,8518	0,8470	0,8420	0,50
0,60	0,8371	0,8320	0,8269	0,8217	0,8165	0,8112	0,8058	0,8004	0,7949	0,7893	0,60
0,70	0,7837	0,7780	0,7723	0,7665	0,7606	0,7547	0,7488	0,7428	0,7367	0,7306	0,70
0,80	0,7245	0,7183	0,7120	0,7058	0,6995	0,6931	0,6868	0,6804	0,6740	0,6676	0,80
0,90	0,6612	0,6547	0,6483	0,6419	0,6354	0,6290	0,6226	0,6162	0,6098	0,6034	0,90
1,00	0,5970	0,5907	0,5844	0,5781	0,5719	0,5657	0,5595	0,5534	0,5473	0,5412	1,00
1,10	0,5352	0,5293	0,5234	0,5175	0,5117	0,5060	0,5003	0,4947	0,4891	0,4836	1,10
1,20	0,4781	0,4727	0,4674	0,4621	0,4569	0,4517	0,4466	0,4416	0,4366	0,4317	1,20
1,30	0,4269	0,4221	0,4174	0,4127	0,4081	0,4035	0,3991	0,3946	0,3903	0,3860	1,30
1,40	0,3817	0,3775	0,3734	0,3693	0,3653	0,3613	0,3574	0,3535	0,3497	0,3459	1,40
1,50	0,3422	0,3386	0,3350	0,3314	0,3279	0,3245	0,3211	0,3177	0,3144	0,3111	1,50
1,60	0,3079	0,3047	0,3016	0,2985	0,2955	0,2925	0,2895	0,2866	0,2837	0,2809	1,60
1,70	0,2781	0,2753	0,2726	0,2699	0,2672	0,2646	0,2620	0,2595	0,2570	0,2545	1,70
1,80	0,2521	0,2496	0,2473	0,2449	0,2426	0,2403	0,2381	0,2359	0,2337	0,2315	1,80
1,90	0,2294	0,2272	0,2252	0,2231	0,2211	0,2191	0,2171	0,2152	0,2132	0,2113	1,90
2,00	0,2095	0,2076	0,2058	0,2040	0,2022	0,2004	0,1987	0,1970	0,1953	0,1936	2,00
2,10	0,1920	0,1903	0,1887	0,1871	0,1855	0,1840	0,1825	0,1809	0,1794	0,1780	2,10
2,20	0,1765	0,1751	0,1736	0,1722	0,1708	0,1694	0,1681	0,1667	0,1654	0,1641	2,20
2,30	0,1628	0,1615	0,1602	0,1590	0,1577	0,1565	0,1553	0,1541	0,1529	0,1517	2,30
2,40	0,1506	0,1494	0,1483	0,1472	0,1461	0,1450	0,1439	0,1428	0,1418	0,1407	2,40
2,50	0,1397	0,1387	0,1376	0,1366	0,1356	0,1347	0,1337	0,1327	0,1318	0,1308	2,50
2,60	0,1299	0,1290	0,1281	0,1272	0,1263	0,1254	0,1245	0,1237	0,1228	0,1219	2,60
2,70	0,1211	0,1203	0,1195	0,1186	0,1178	0,1170	0,1162	0,1155	0,1147	0,1139	2,70
2,80	0,1132	0,1124	0,1117	0,1109	0,1102	0,1095	0,1088	0,1081	0,1074	0,1067	2,80
2,90	0,1060	0,1053	0,1046	0,1039	0,1033	0,1026	0,1020	0,1013	0,1007	0,1001	2,90
3,00	0,0994	0,0988	0,0982	0,0976	0,0970	0,0964	0,0958	0,0952	0,0946	0,0940	3,00
3,10	0,0935	0,0929	0,0924	0,0918	0,0912	0,0907	0,0902	0,0896	0,0891	0,0886	3,10
3,20	0,0880	0,0875	0,0870	0,0865	0,0860	0,0855	0,0850	0,0845	0,0840	0,0835	3,20
3,30	0,0831	0,0826	0,0821	0,0816	0,0812	0,0807	0,0803	0,0798	0,0794	0,0789	3,30
3,40	0,0785	0,0781	0,0776	0,0772	0,0768	0,0763	0,0759	0,0755	0,0751	0,0747	3,40
3,50	0,0743	0,0739	0,0735	0,0731	0,0727	0,0723	0,0719	0,0715	0,0712	0,0708	3,50
3,60	0,0704	0,0700	0,0697	0,0693	0,0689	0,0686	0,0682	0,0679	0,0675	0,0672	3,60

Reduction factors χ for buckling curve c (α = 0,49)											
$\bar{\lambda}$	0,00	0,01	0,02	0,03	0,04	0,05	0,06	0,07	0,08	0,09	
0,00	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	0,00
0,10	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	0,10
0,20	1,0000	0,9949	0,9898	0,9847	0,9797	0,9746	0,9695	0,9644	0,9593	0,9542	0,20
0,30	0,9491	0,9440	0,9389	0,9338	0,9286	0,9235	0,9183	0,9131	0,9078	0,9026	0,30
0,40	0,8973	0,8920	0,8867	0,8813	0,8760	0,8705	0,8651	0,8596	0,8541	0,8486	0,40
0,50	0,8430	0,8374	0,8317	0,8261	0,8204	0,8146	0,8088	0,8030	0,7972	0,7913	0,50
0,60	0,7854	0,7794	0,7735	0,7675	0,7614	0,7554	0,7493	0,7432	0,7370	0,7309	0,60
0,70	0,7247	0,7185	0,7123	0,7060	0,6998	0,6935	0,6873	0,6810	0,6747	0,6684	0,70
0,80	0,6622	0,6559	0,6496	0,6433	0,6371	0,6308	0,6246	0,6184	0,6122	0,6060	0,80
0,90	0,5998	0,5937	0,5876	0,5815	0,5755	0,5695	0,5635	0,5575	0,5516	0,5458	0,90
1,00	0,5399	0,5342	0,5284	0,5227	0,5171	0,5115	0,5059	0,5004	0,4950	0,4896	1,00
1,10	0,4842	0,4790	0,4737	0,4685	0,4634	0,4583	0,4533	0,4483	0,4434	0,4386	1,10
1,20	0,4338	0,4290	0,4243	0,4197	0,4151	0,4106	0,4061	0,4017	0,3974	0,3931	1,20
1,30	0,3888	0,3846	0,3805	0,3764	0,3724	0,3684	0,3644	0,3606	0,3567	0,3529	1,30
1,40	0,3492	0,3455	0,3419	0,3383	0,3348	0,3313	0,3279	0,3245	0,3211	0,3178	1,40
1,50	0,3145	0,3113	0,3081	0,3050	0,3019	0,2989	0,2959	0,2929	0,2900	0,2871	1,50
1,60	0,2842	0,2814	0,2786	0,2759	0,2732	0,2705	0,2679	0,2653	0,2627	0,2602	1,60
1,70	0,2577	0,2553	0,2528	0,2504	0,2481	0,2457	0,2434	0,2412	0,2389	0,2367	1,70
1,80	0,2345	0,2324	0,2302	0,2281	0,2260	0,2240	0,2220	0,2200	0,2180	0,2161	1,80
1,90	0,2141	0,2122	0,2104	0,2085	0,2067	0,2049	0,2031	0,2013	0,1996	0,1979	1,90
2,00	0,1962	0,1945	0,1929	0,1912	0,1896	0,1880	0,1864	0,1849	0,1833	0,1818	2,00
2,10	0,1803	0,1788	0,1774	0,1759	0,1745	0,1731	0,1717	0,1703	0,1689	0,1676	2,10
2,20	0,1662	0,1649	0,1636	0,1623	0,1611	0,1598	0,1585	0,1573	0,1561	0,1549	2,20
2,30	0,1537	0,1525	0,1514	0,1502	0,1491	0,1480	0,1468	0,1457	0,1446	0,1436	2,30
2,40	0,1425	0,1415	0,1404	0,1394	0,1384	0,1374	0,1364	0,1354	0,1344	0,1334	2,40
2,50	0,1325	0,1315	0,1306	0,1297	0,1287	0,1278	0,1269	0,1260	0,1252	0,1243	2,50
2,60	0,1234	0,1226	0,1217	0,1209	0,1201	0,1193	0,1184	0,1176	0,1168	0,1161	2,60
2,70	0,1153	0,1145	0,1137	0,1130	0,1122	0,1115	0,1108	0,1100	0,1093	0,1086	2,70
2,80	0,1079	0,1072	0,1065	0,1058	0,1051	0,1045	0,1038	0,1031	0,1025	0,1018	2,80
2,90	0,1012	0,1006	0,0999	0,0993	0,0987	0,0981	0,0975	0,0969	0,0963	0,0957	2,90
3,00	0,0951	0,0945	0,0939	0,0934	0,0928	0,0922	0,0917	0,0911	0,0906	0,0901	3,00
3,10	0,0895	0,0890	0,0885	0,0879	0,0874	0,0869	0,0864	0,0859	0,0854	0,0849	3,10
3,20	0,0844	0,0839	0,0835	0,0830	0,0825	0,0820	0,0816	0,0811	0,0806	0,0802	3,20
3,30	0,0797	0,0793	0,0789	0,0784	0,0780	0,0775	0,0771	0,0767	0,0763	0,0759	3,30
3,40	0,0754	0,0750	0,0746	0,0742	0,0738	0,0734	0,0730	0,0726	0,0722	0,0719	3,40
3,50	0,0715	0,0711	0,0707	0,0703	0,0700	0,0696	0,0692	0,0689	0,0685	0,0682	3,50
3,60	0,0678	0,0675	0,0671	0,0668	0,0664	0,0661	0,0657	0,0654	0,0651	0,0647	3,60

Reduction factors χ for buckling curve d (α = 0,76)

$\bar{\lambda}$	0,00	0,01	0,02	0,03	0,04	0,05	0,06	0,07	0,08	0,09	
0,00	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	0,00
0,10	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	1,0000	0,10
0,20	1,0000	0,9921	0,9843	0,9765	0,9688	0,9611	0,9535	0,9459	0,9384	0,9309	0,20
0,30	0,9235	0,9160	0,9086	0,9013	0,8939	0,8866	0,8793	0,8721	0,8648	0,8576	0,30
0,40	0,8504	0,8432	0,8360	0,8289	0,8218	0,8146	0,8075	0,8005	0,7934	0,7864	0,40
0,50	0,7793	0,7723	0,7653	0,7583	0,7514	0,7444	0,7375	0,7306	0,7237	0,7169	0,50
0,60	0,7100	0,7032	0,6964	0,6897	0,6829	0,6762	0,6695	0,6629	0,6563	0,6497	0,60
0,70	0,6431	0,6366	0,6301	0,6237	0,6173	0,6109	0,6046	0,5983	0,5921	0,5859	0,70
0,80	0,5797	0,5736	0,5675	0,5615	0,5556	0,5496	0,5438	0,5379	0,5322	0,5265	0,80
0,90	0,5208	0,5152	0,5096	0,5041	0,4987	0,4933	0,4879	0,4826	0,4774	0,4722	0,90
1,00	0,4671	0,4620	0,4570	0,4521	0,4472	0,4423	0,4375	0,4328	0,4281	0,4235	1,00
1,10	0,4189	0,4144	0,4099	0,4055	0,4012	0,3969	0,3926	0,3884	0,3843	0,3802	1,10
1,20	0,3762	0,3722	0,3683	0,3644	0,3605	0,3568	0,3530	0,3493	0,3457	0,3421	1,20
1,30	0,3385	0,3350	0,3316	0,3282	0,3248	0,3215	0,3182	0,3150	0,3118	0,3086	1,30
1,40	0,3055	0,3024	0,2994	0,2964	0,2935	0,2906	0,2877	0,2849	0,2821	0,2793	1,40
1,50	0,2766	0,2739	0,2712	0,2686	0,2660	0,2635	0,2609	0,2585	0,2560	0,2536	1,50
1,60	0,2512	0,2488	0,2465	0,2442	0,2419	0,2397	0,2375	0,2353	0,2331	0,2310	1,60
1,70	0,2289	0,2268	0,2248	0,2228	0,2208	0,2188	0,2168	0,2149	0,2130	0,2112	1,70
1,80	0,2093	0,2075	0,2057	0,2039	0,2021	0,2004	0,1987	0,1970	0,1953	0,1936	1,80
1,90	0,1920	0,1904	0,1888	0,1872	0,1856	0,1841	0,1826	0,1810	0,1796	0,1781	1,90
2,00	0,1766	0,1752	0,1738	0,1724	0,1710	0,1696	0,1683	0,1669	0,1656	0,1643	2,00
2,10	0,1630	0,1617	0,1604	0,1592	0,1580	0,1567	0,1555	0,1543	0,1532	0,1520	2,10
2,20	0,1508	0,1497	0,1486	0,1474	0,1463	0,1452	0,1442	0,1431	0,1420	0,1410	2,20
2,30	0,1399	0,1389	0,1379	0,1369	0,1359	0,1349	0,1340	0,1330	0,1320	0,1311	2,30
2,40	0,1302	0,1292	0,1283	0,1274	0,1265	0,1257	0,1248	0,1239	0,1231	0,1222	2,40
2,50	0,1214	0,1205	0,1197	0,1189	0,1181	0,1173	0,1165	0,1157	0,1149	0,1142	2,50
2,60	0,1134	0,1127	0,1119	0,1112	0,1104	0,1097	0,1090	0,1083	0,1076	0,1069	2,60
2,70	0,1062	0,1055	0,1048	0,1042	0,1035	0,1029	0,1022	0,1016	0,1009	0,1003	2,70
2,80	0,0997	0,0990	0,0984	0,0978	0,0972	0,0966	0,0960	0,0954	0,0948	0,0943	2,80
2,90	0,0937	0,0931	0,0926	0,0920	0,0914	0,0909	0,0904	0,0898	0,0893	0,0888	2,90
3,00	0,0882	0,0877	0,0872	0,0867	0,0862	0,0857	0,0852	0,0847	0,0842	0,0837	3,00
3,10	0,0832	0,0828	0,0823	0,0818	0,0814	0,0809	0,0804	0,0800	0,0795	0,0791	3,10
3,20	0,0786	0,0782	0,0778	0,0773	0,0769	0,0765	0,0761	0,0757	0,0752	0,0748	3,20
3,30	0,0744	0,0740	0,0736	0,0732	0,0728	0,0724	0,0721	0,0717	0,0713	0,0709	3,30
3,40	0,0705	0,0702	0,0698	0,0694	0,0691	0,0687	0,0683	0,0680	0,0676	0,0673	3,40
3,50	0,0669	0,0666	0,0663	0,0659	0,0656	0,0652	0,0649	0,0646	0,0643	0,0639	3,50
3,60	0,0636	0,0633	0,0630	0,0627	0,0624	0,0620	0,0617	0,0614	0,0611	0,0608	3,60