Design of composite beams according to Eurocode 4-1-1

Lecture:

Ultimate Limit States

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A short introduction

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Design of composite beams according to Eurocode 4-1-1

Contents


Car park,
Messe Stuttgart

Part 1:

SCOPE

Definitions according EN 1994-1-1 [§1.5.2]

COMPOSITE MEMBER

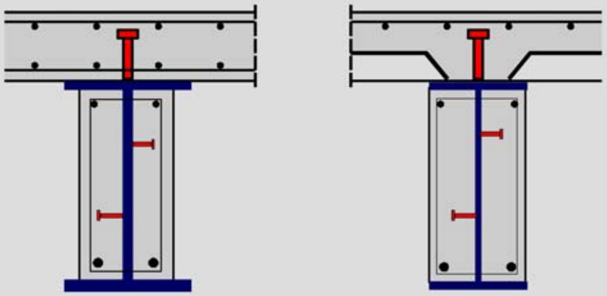
a structural member with components of concrete and of structural or cold-formed steel, interconnected by shear connection so as to limit the longitudinal slip between concrete and steel and the separation of one component from the other

SHEAR CONNECTION

an interconnection between the concrete and steel components of a composite member that has sufficient strength and stiffness to enable the two components to be designed as parts of a single structural member

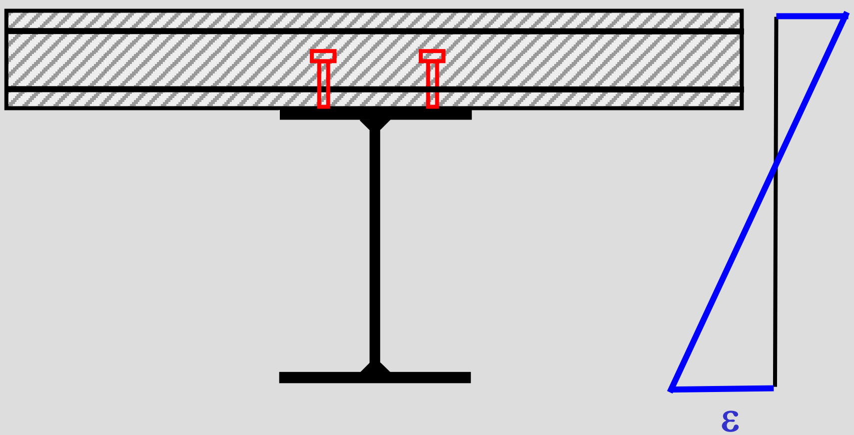
COMPOSITE BEAM

a composite member subjected mainly to bending



COMPOSITE BEHAVIOUR

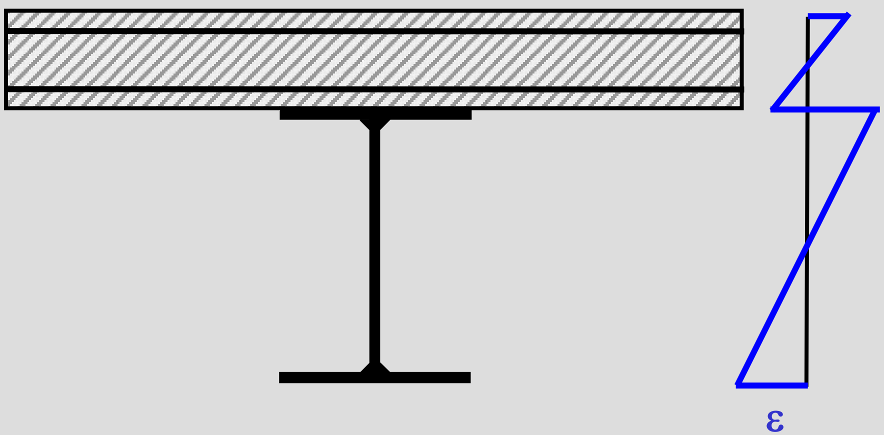
composite beam



composite behaviour

acting as one section

steel beam with concrete slab



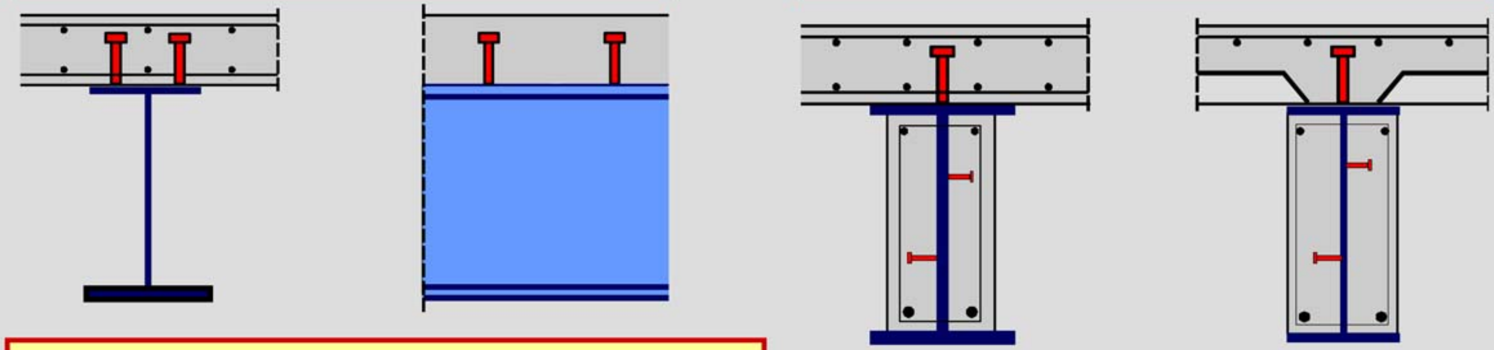
no composite behaviour

acting as two individual sections

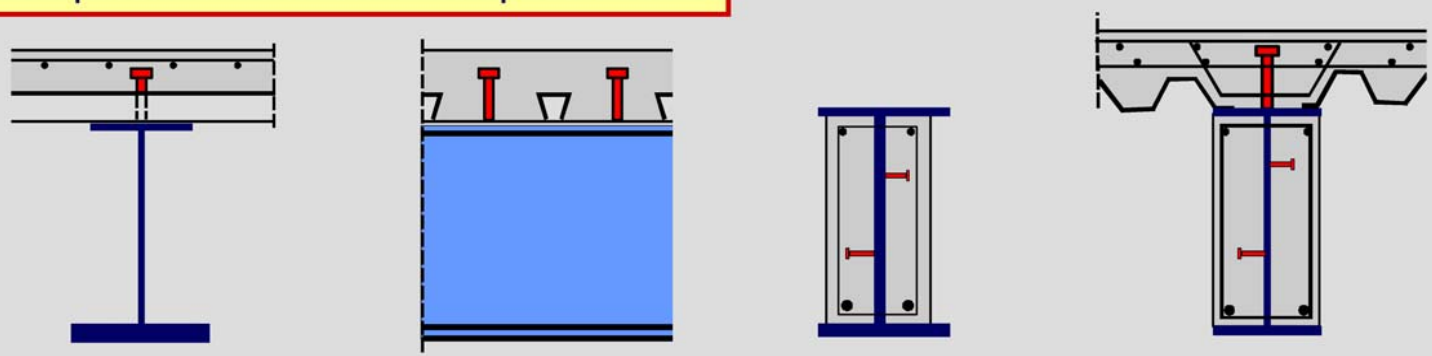
TYPICAL COMPOSITE BEAMS

composite beam with solid slab

partially concrete encased beams



composite beam with composite slab



[Source: Hanswille]

structural steel sections are rolled or welded

Materials according EN 1994-1-1 [§ 3]

CONCRETE

> C 20/25; LC 20/25

< C 60/75; LC 60/75

REINFORCEMENT

Acc. EN 1992-1-1 § 3.2

strength: 400 N/mm² ≤ $f_{y,k}$ ≤ 600 N/mm²

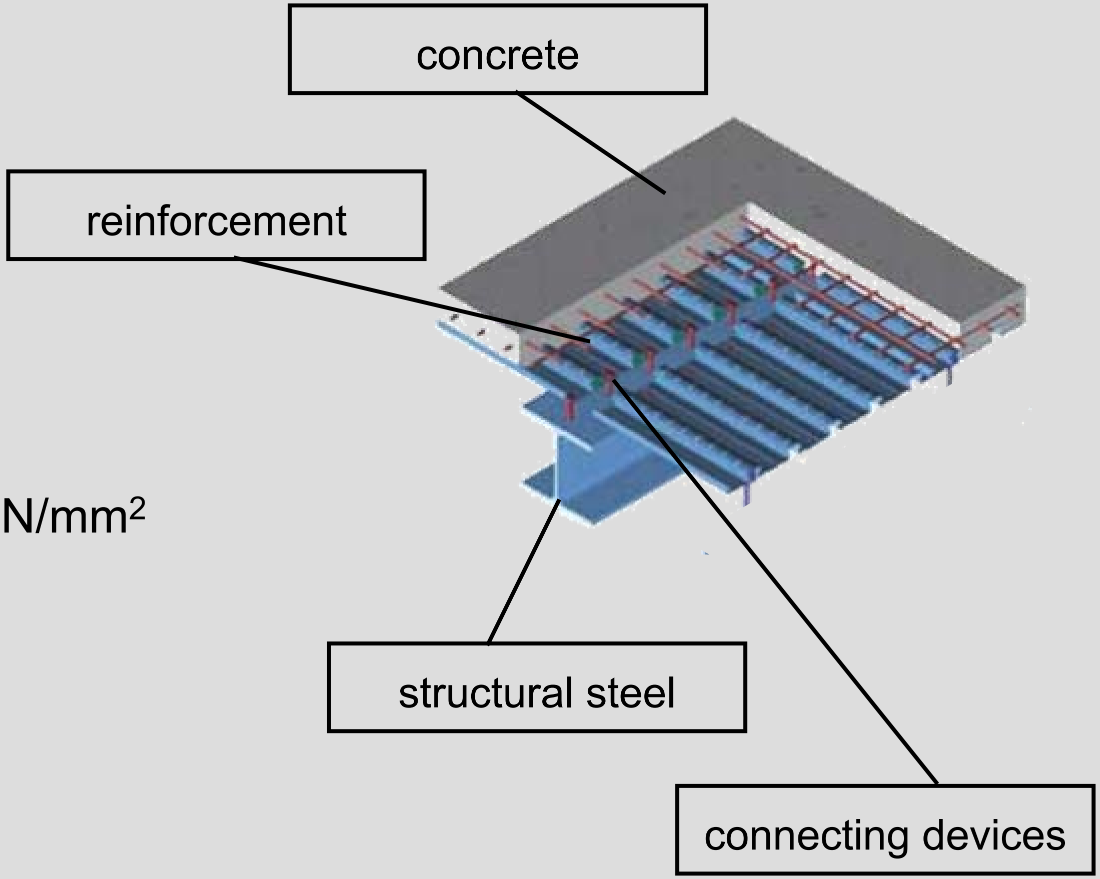
ductility: 1,05 ≤ $(f_t/f_y)_k$ ≤ 1,35

STRUCTURAL STEEL

f_y ≤ 460 N/mm²

CONNECTING DEVICES

Headed stud shear connector acc. EN 13918




source:[ESDEP]

Part 2:

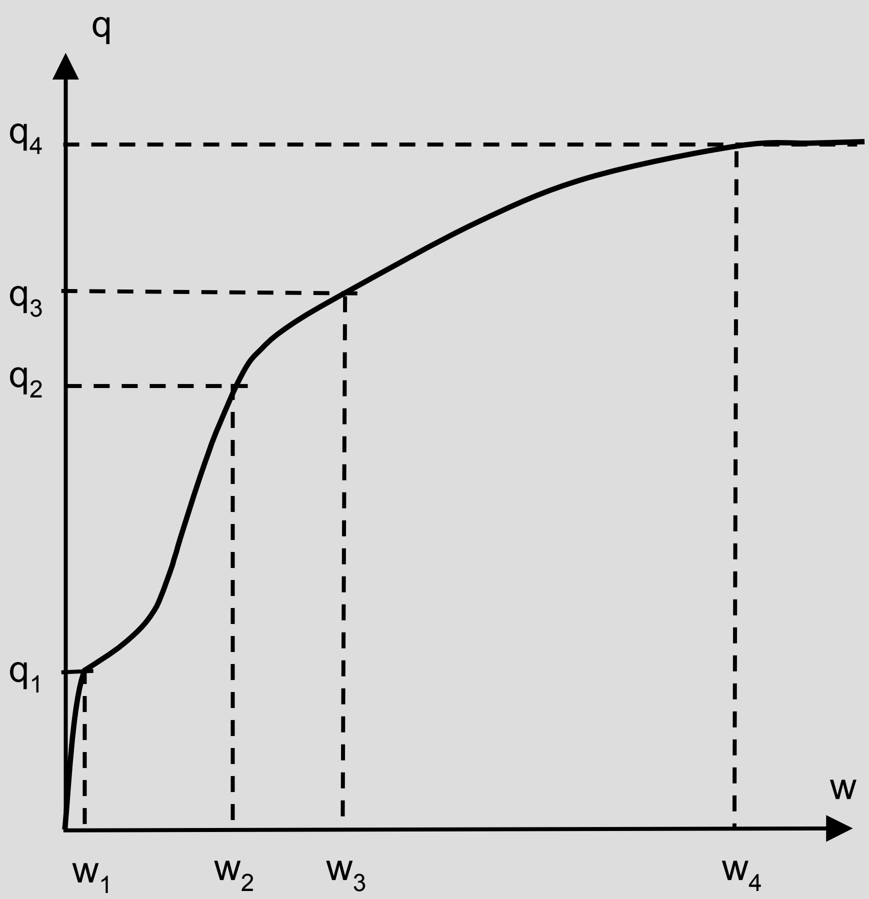
SPECIFIC CHARACTERISTICS

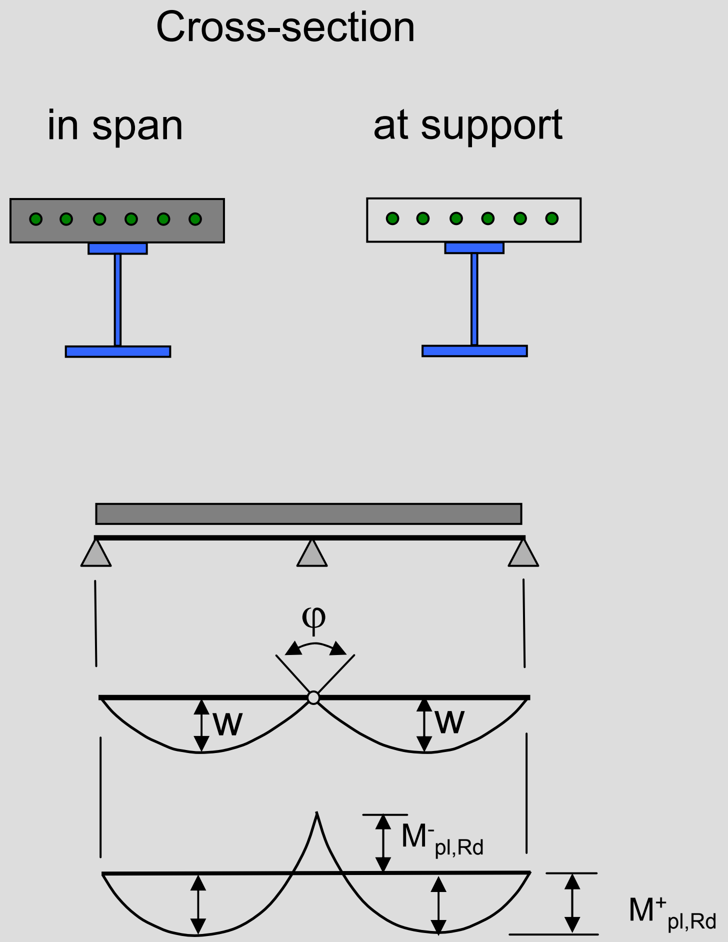
OF STRUCTURAL ANALYSIS

Characteristics

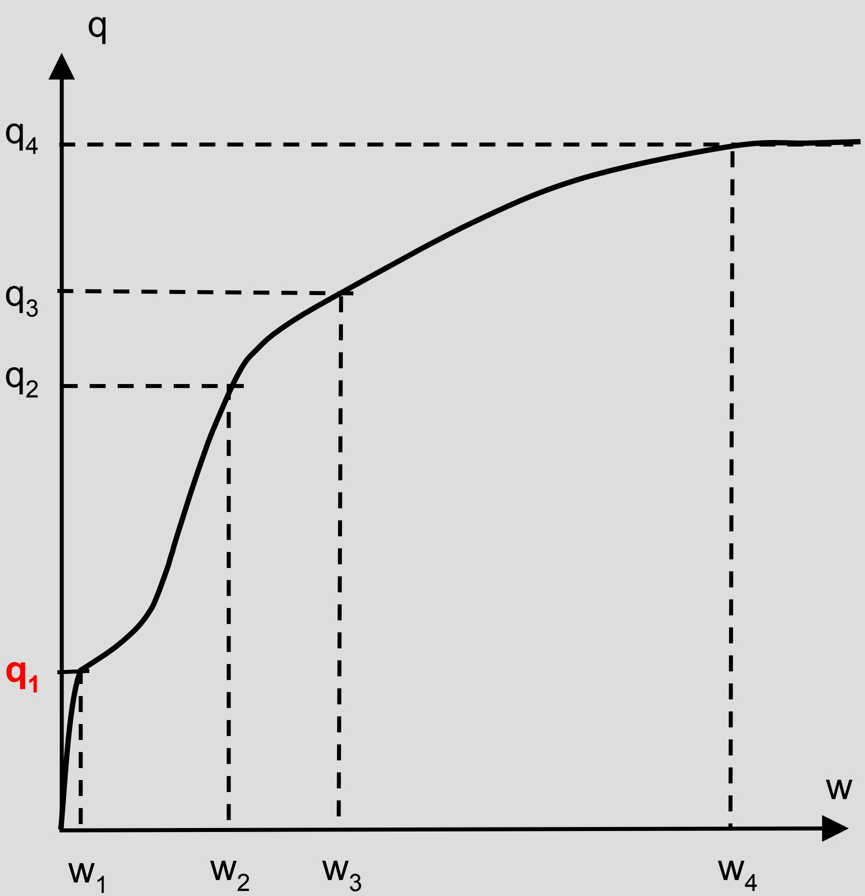
- Non-linear material behaviour
- Influence of erection and load history
- Influence of creep and shrinkage
- Influence of composite interaction

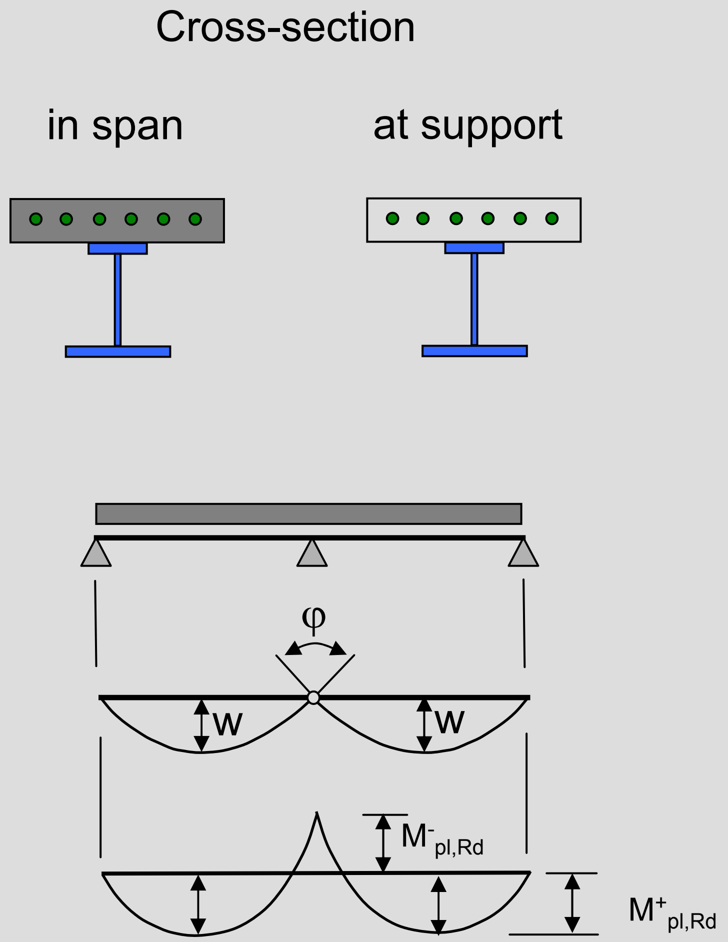
Non-linear material behaviour





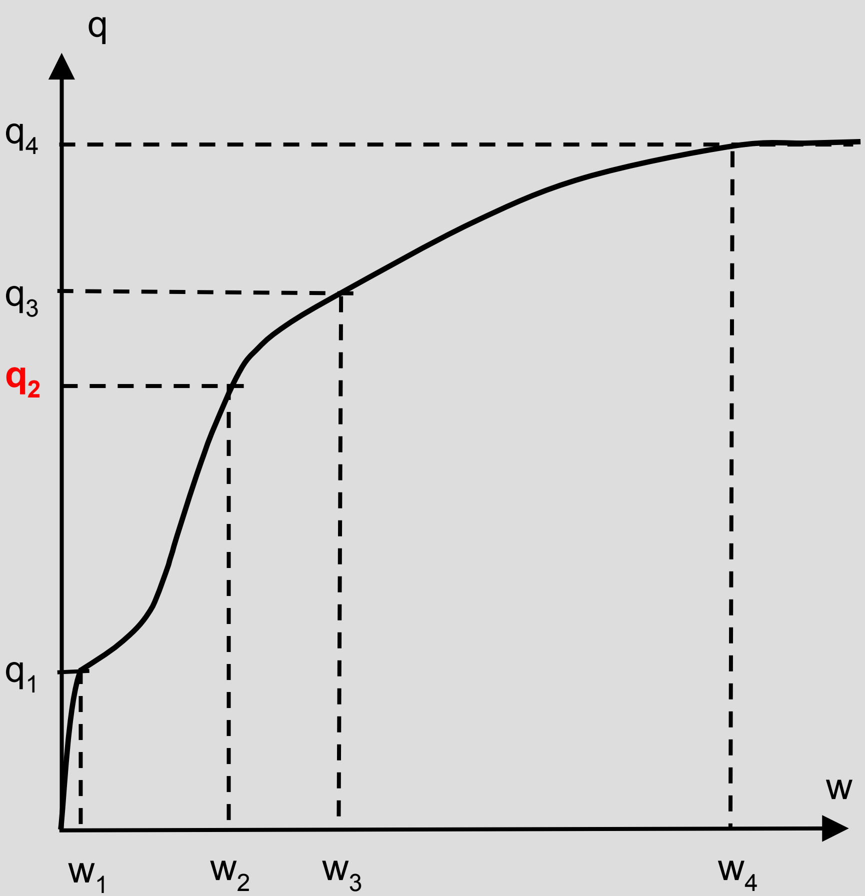
Non-linear material behaviour

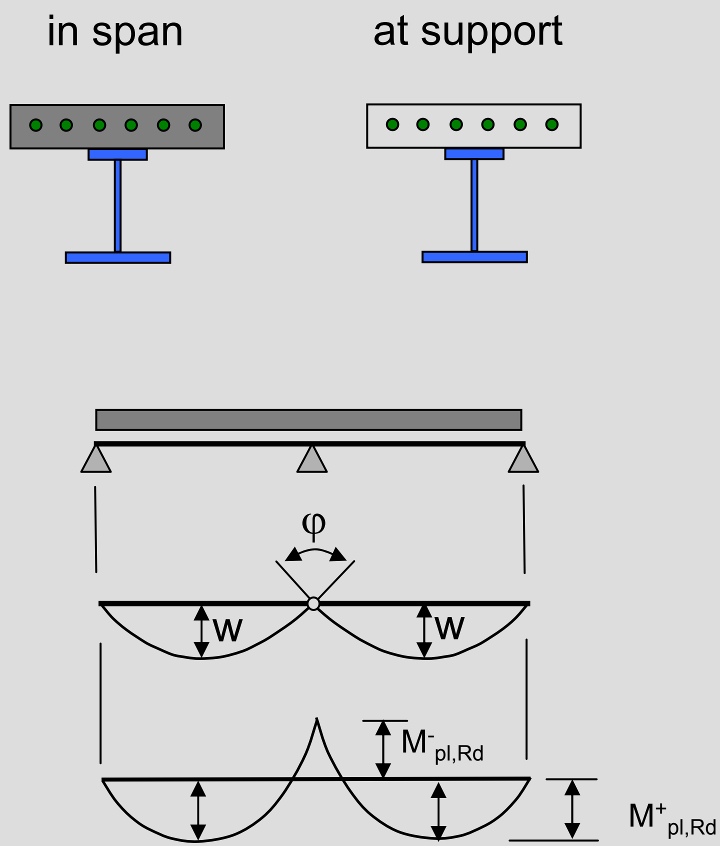




q_1 – first cracking (concrete slab) at support

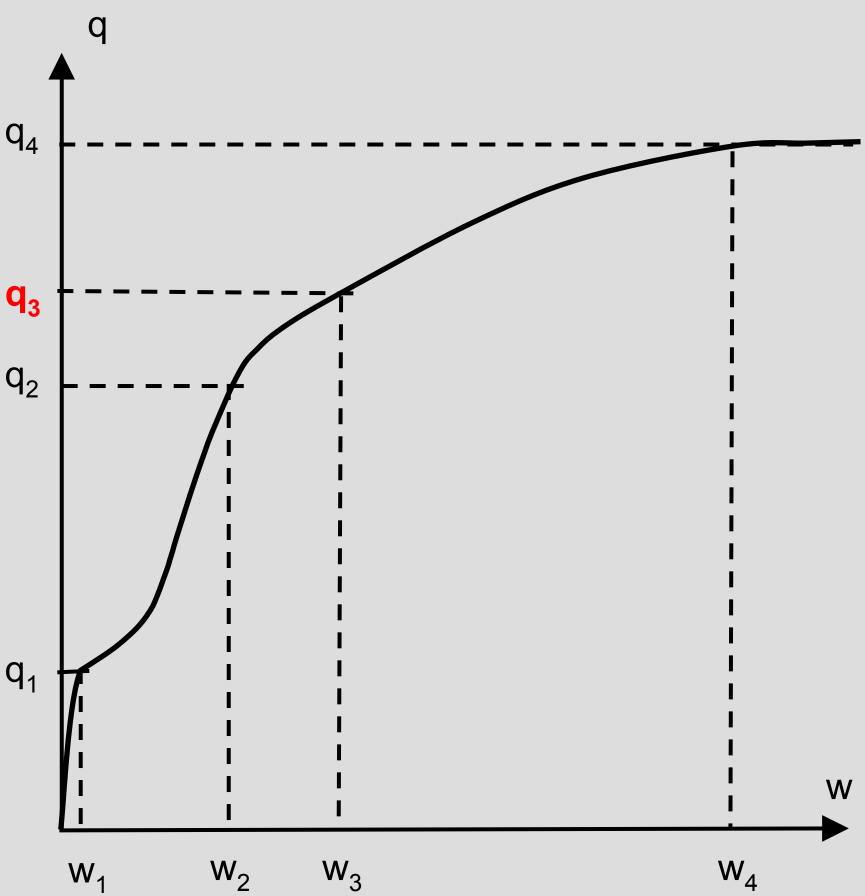
Non-linear material behaviour

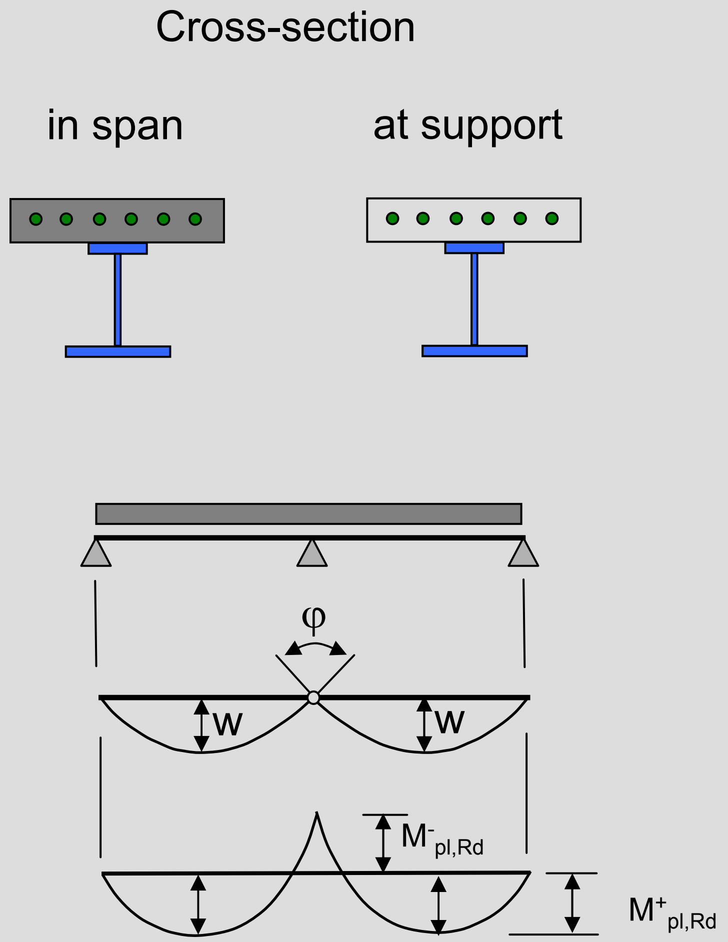




q_2 – first yielding (steel section) at support

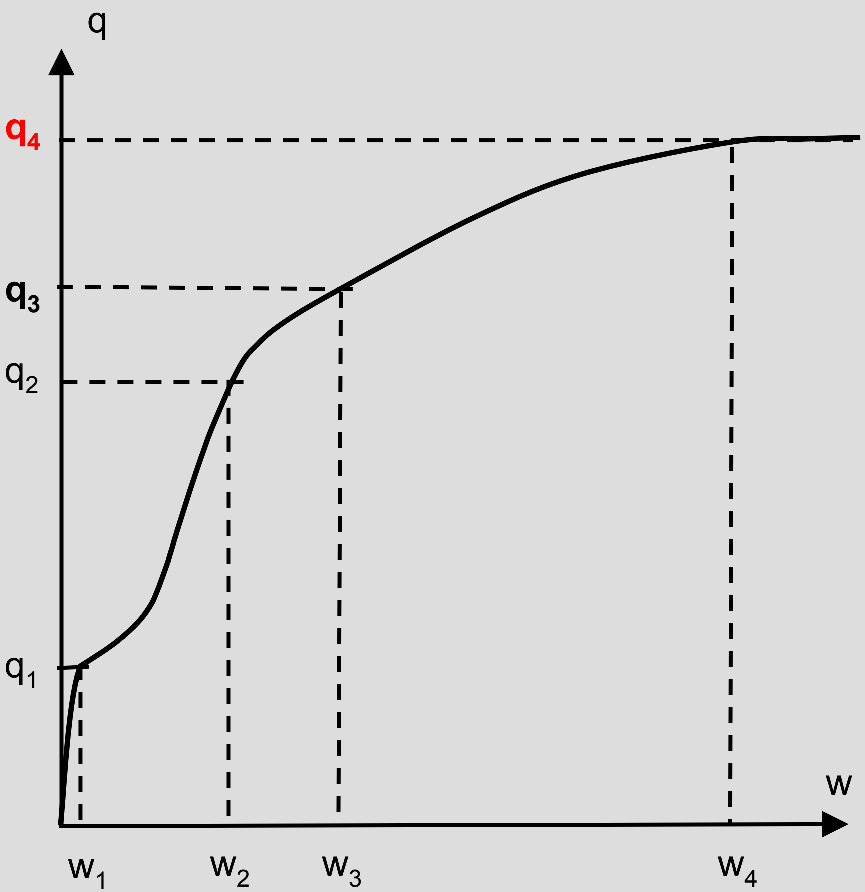
Non-linear material behaviour

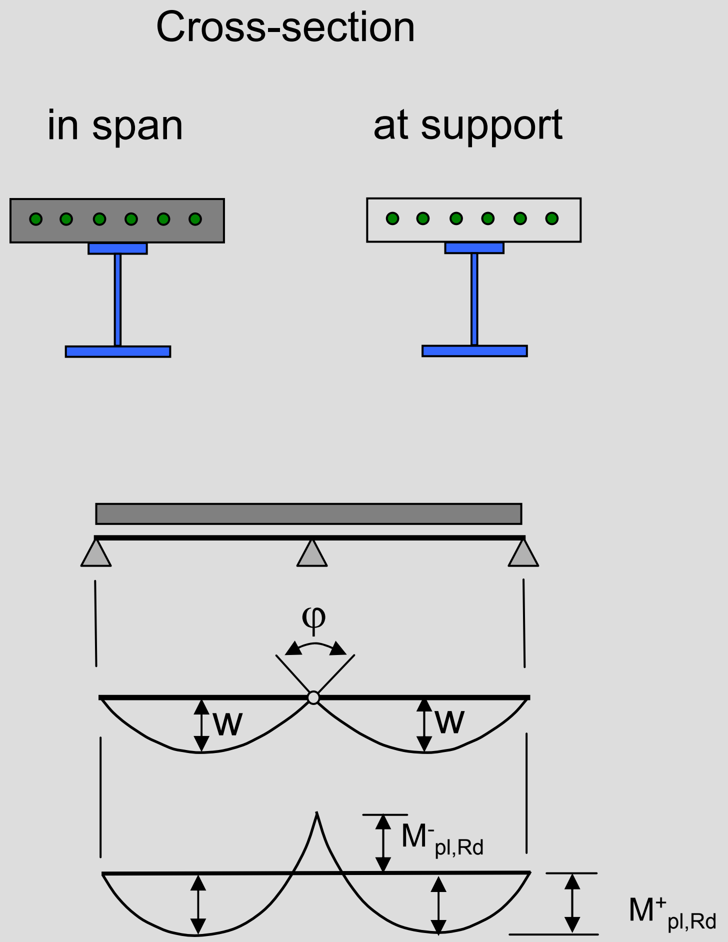




q_3 – first plastic hinge $M^-_{pl.Rd}$ at support

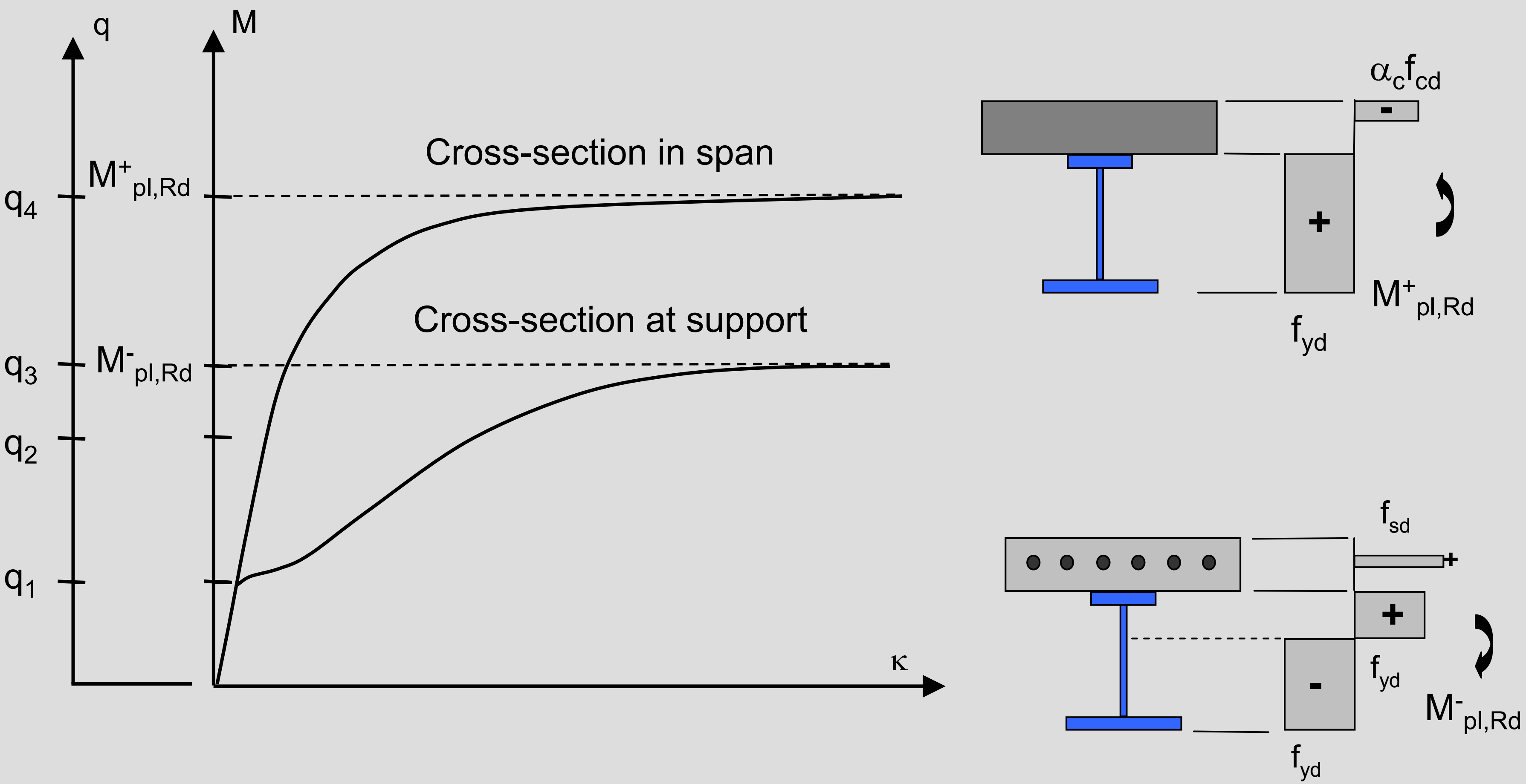
Non-linear material behaviour



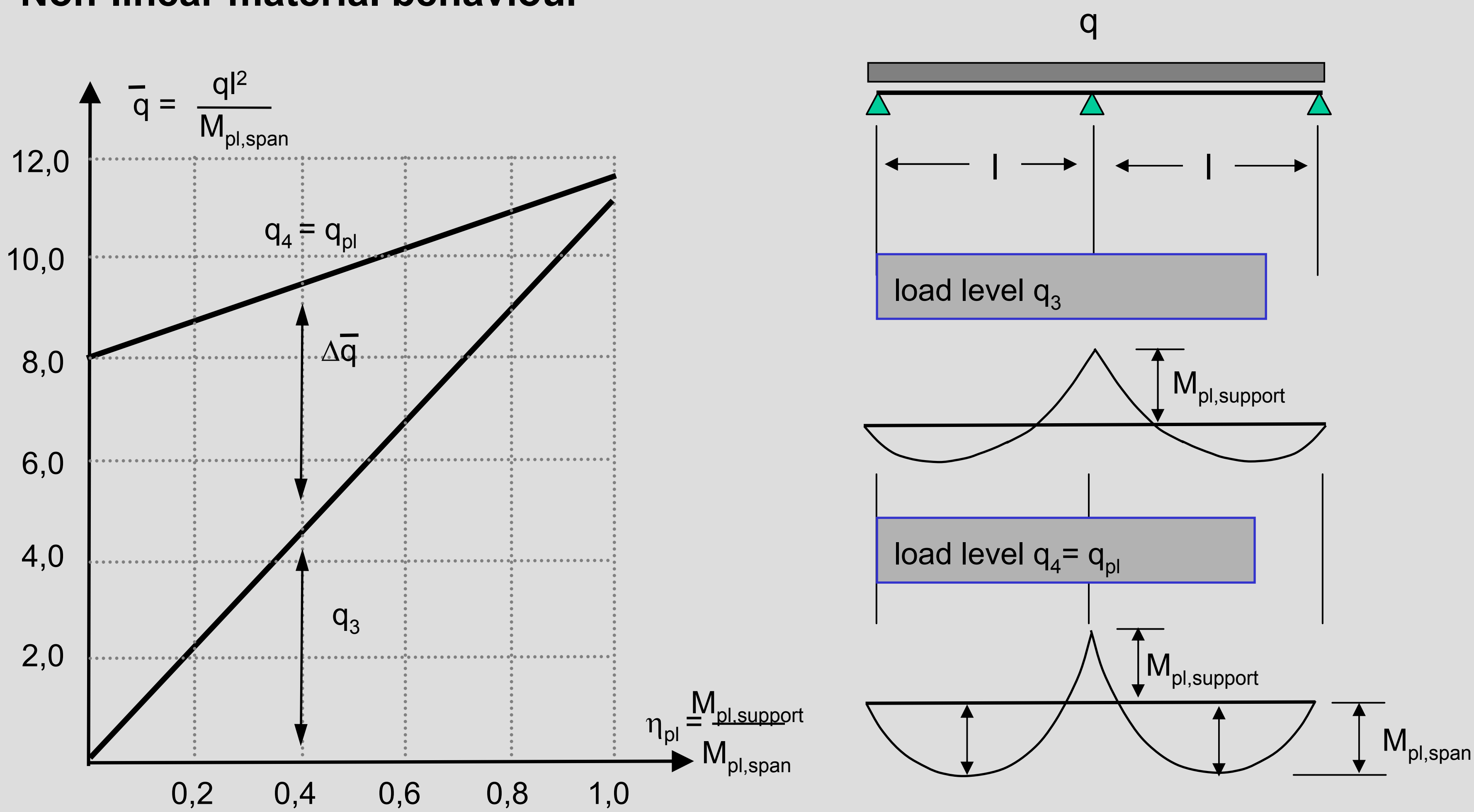


q_4 – last plastic hinge $M^{+}_{pl.Rd}$ in span

Non-linear material behaviour

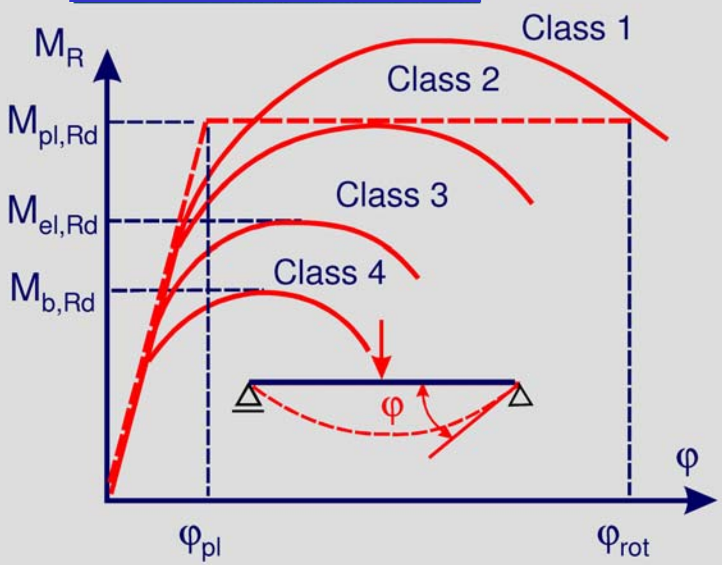


Non-linear material behaviour

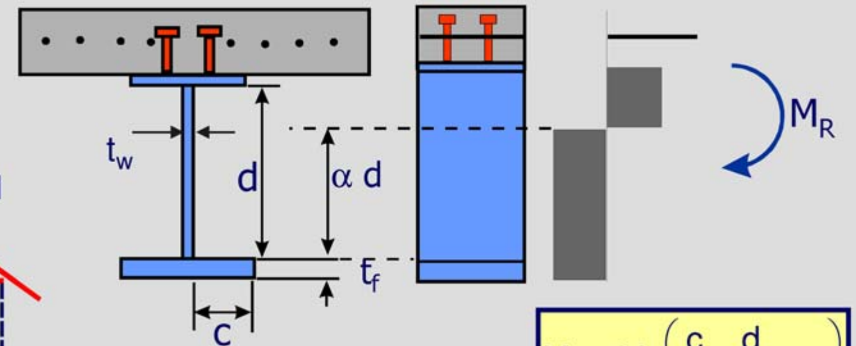


High efficiency of plastic hinge theory due to difference of plastic bending moment in span and at support - requires **rotation capacity** of section with first plastic hinge (at support)

Non-linear material behaviour



Rotation capacity: $R = \frac{\varphi_{rot} - \varphi_{pl}}{\varphi_{pl}} = \frac{\varphi_{rot}}{\varphi_{pl}} - 1$



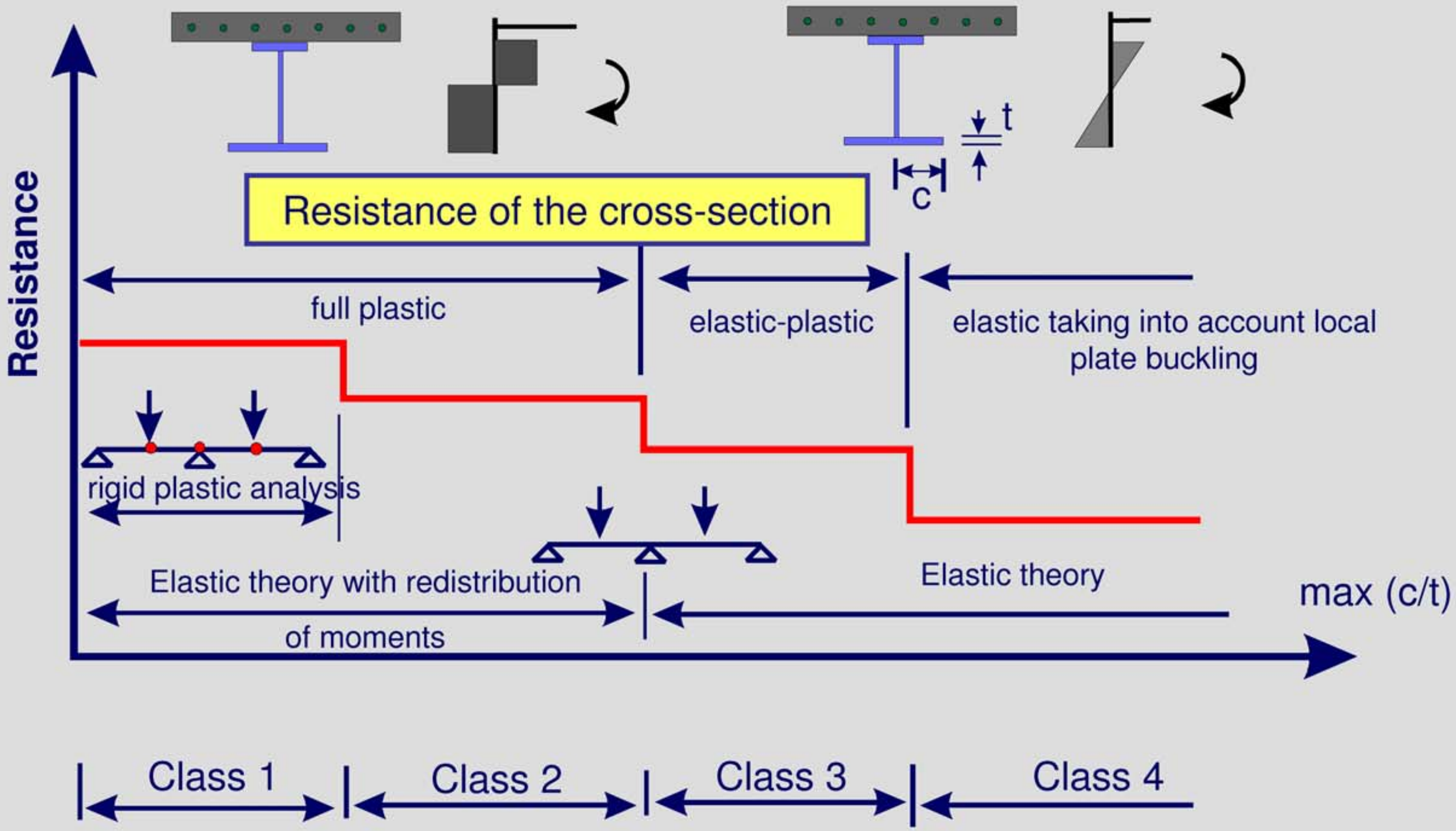
ρ_s – reinforcement ratio

$$M_R = M\left(\frac{c}{t_f}; \frac{d}{t_w}; \rho_s\right)$$

Class	Bending resistance
1 and 2	plastic
3	elastic
4	elastic taking into account local buckling

[Source: Hanswille]

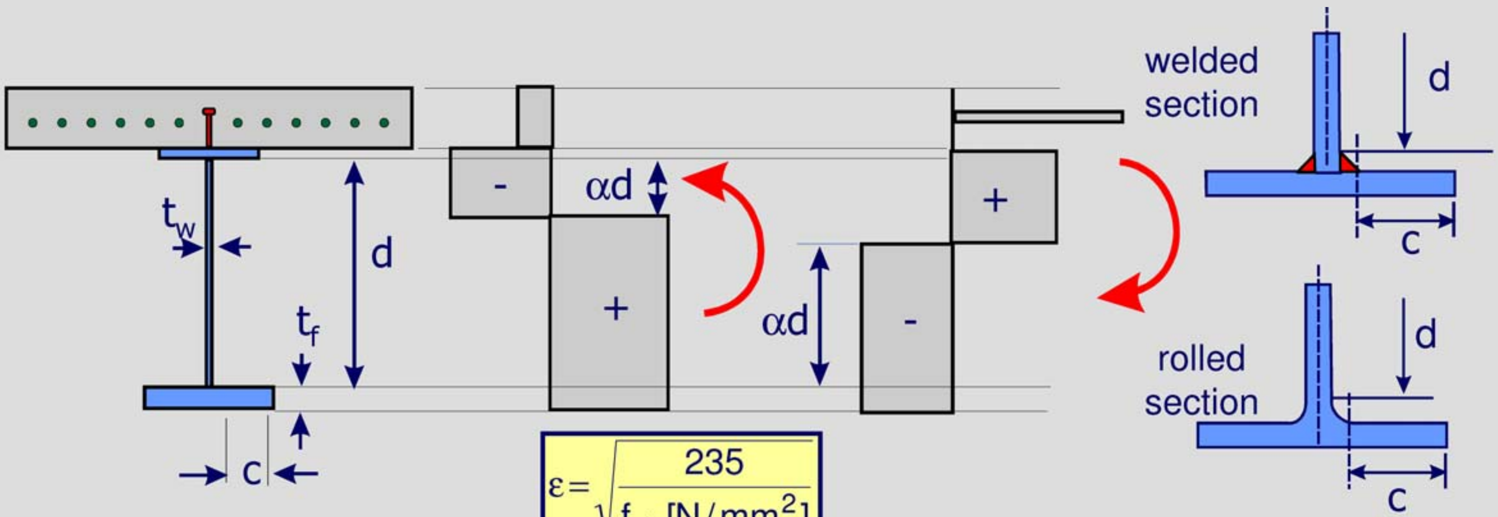
Non-linear material behaviour



[Source: Hanswille]

Non-linear material behaviour

Classes 1 and 2



$$\varepsilon = \sqrt{\frac{235}{f_{yk}\,[\mathrm{N/mm^2}]}}$$

Class 1

web: $\frac{d}{t_w} \leq \frac{36\,\varepsilon}{\alpha}$ for $\alpha \leq 0{,}5$

$\frac{d}{t_w} \leq \frac{396\,\varepsilon}{13\,\alpha - 1}$ for $\alpha > 0{,}5$

flange: $c/t_f \leq 9\varepsilon$

Class 2

web: $\frac{d}{t_w} \leq \frac{41{,}5\,\varepsilon}{\alpha}$ for $\alpha \leq 0{,}5$

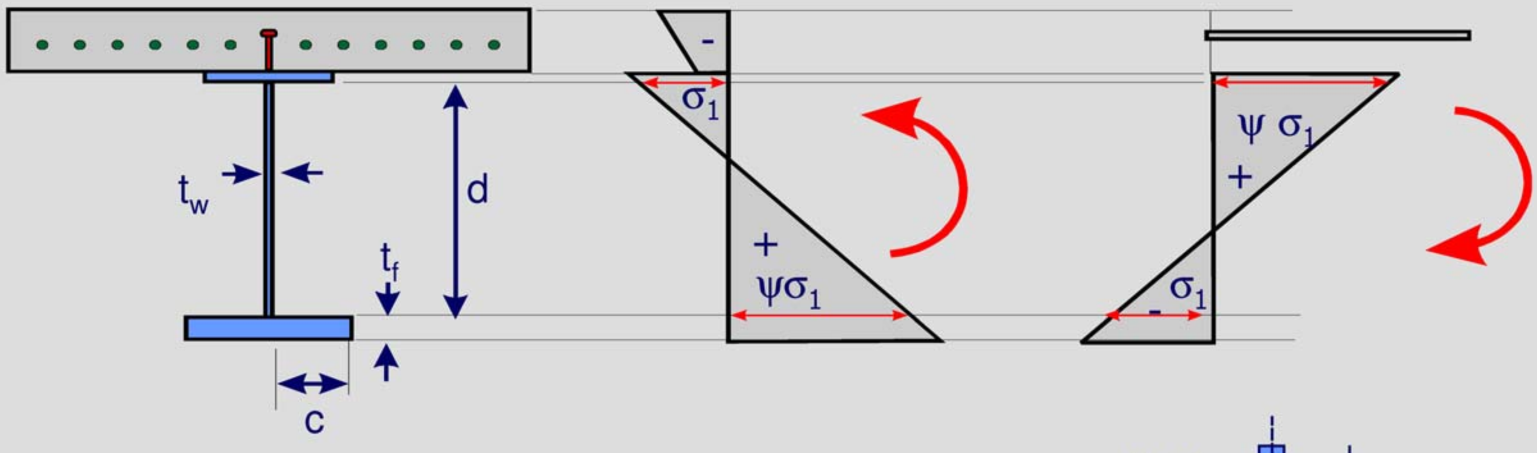
$\frac{d}{t_w} \leq \frac{456\,\varepsilon}{13\,\alpha - 1}$ for $\alpha > 0{,}5$

flange: $c/t_f \leq 10\varepsilon$

[Source: Hanswille]

Non-linear material behaviour

Class 3



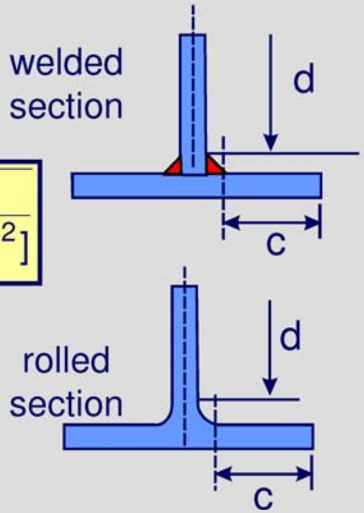
web:

for $\psi > -1{,}0$: $\frac{d}{t_w} \le \frac{42\,\varepsilon}{0{,}67 + 0{,}33\,\psi}$

for $\psi \le -1{,}0$: $\frac{d}{t_w} \le 0{,}62\,\varepsilon\,(1-\psi)\sqrt{(-\psi)}$

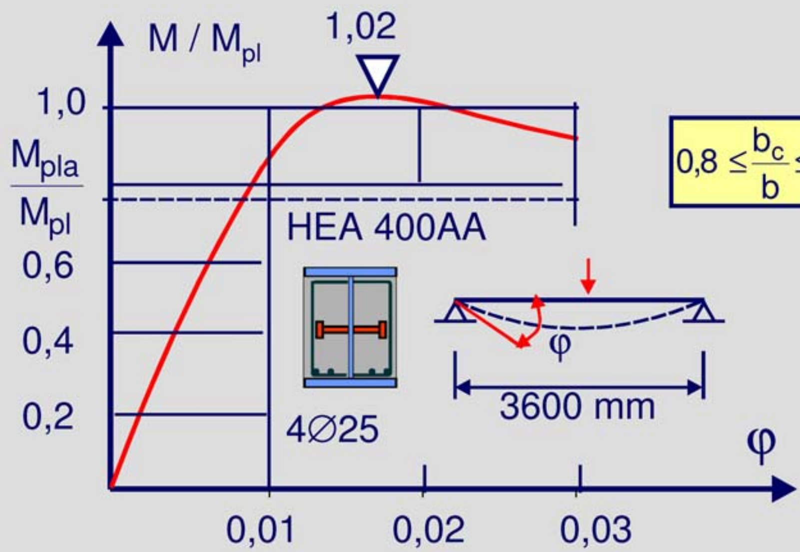
flange: $c/t_f \le 14\varepsilon$

$$\varepsilon = \sqrt{\frac{235}{f_{yk}\,[N/mm^2]}}$$



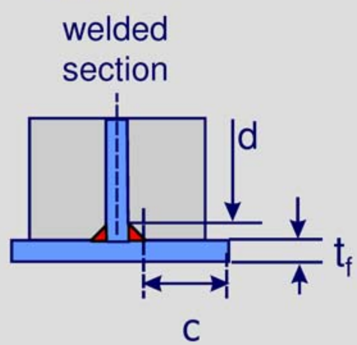
[Source: Hanswille]

Non-linear material behaviour

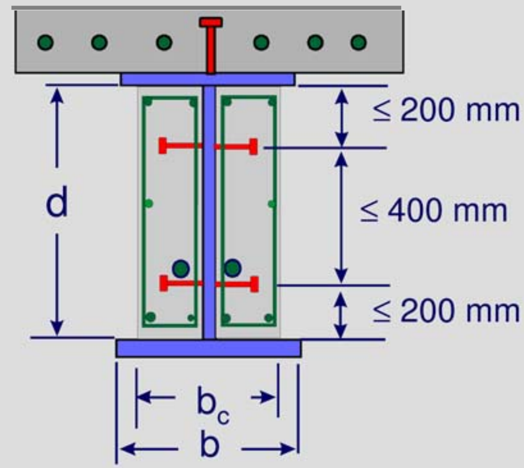


$0{,}8 \le \frac{b_c}{b} \le 1{,}0$

A steel web in class 3 may be represented by an effective web of the same cross-section in Class 2



Classification with partial concrete encasement

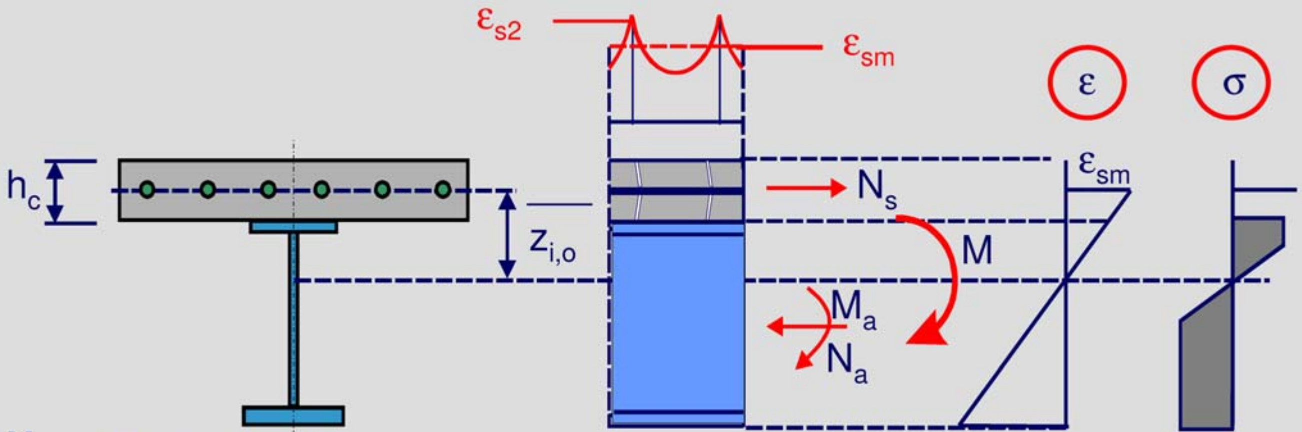


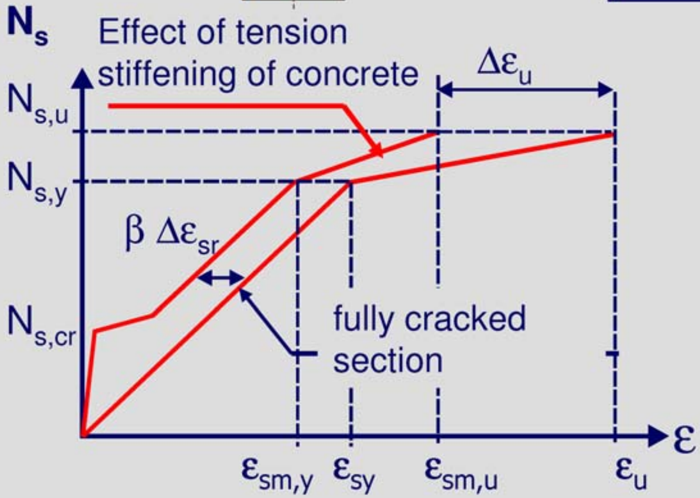
compression flanges	
Class	max c/t_f
1	9ε
2	14 ε
3	20 ε

[Source: Hanswille]

Non-linear material behaviour

Reinforcement in tension flanges





minimum area of reinforcement:

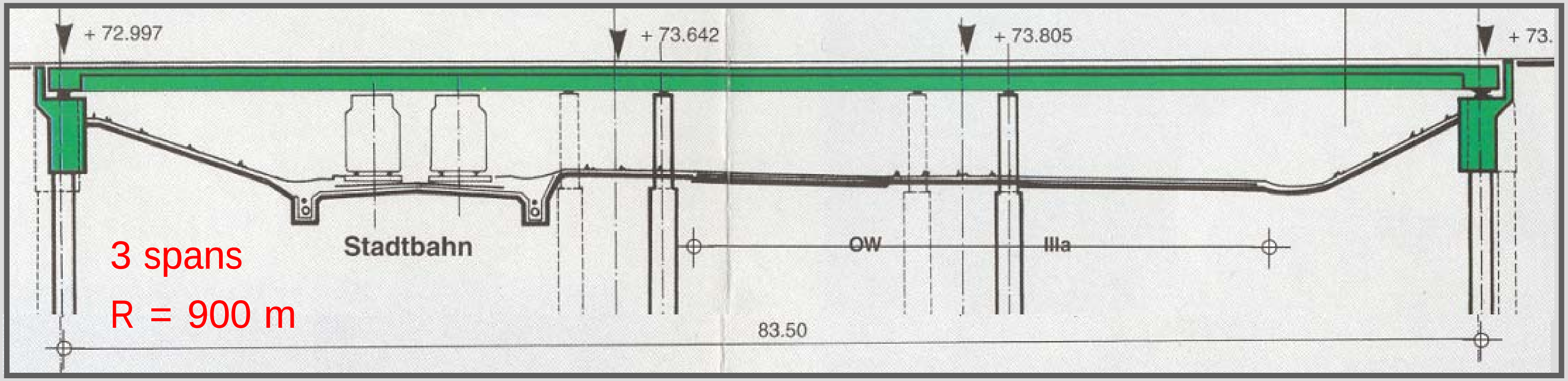
$$\rho_{s,min} = \delta \, \frac{f_{yk}}{240} \, \frac{f_{ctm}}{f_{sk}} \sqrt{\frac{1}{1 + h_c/(2\,z_{io})}}$$

Class 1: $\delta = 1{,}1$

Class 2: $\delta = 1{,}0$

[Source: Hanswille]

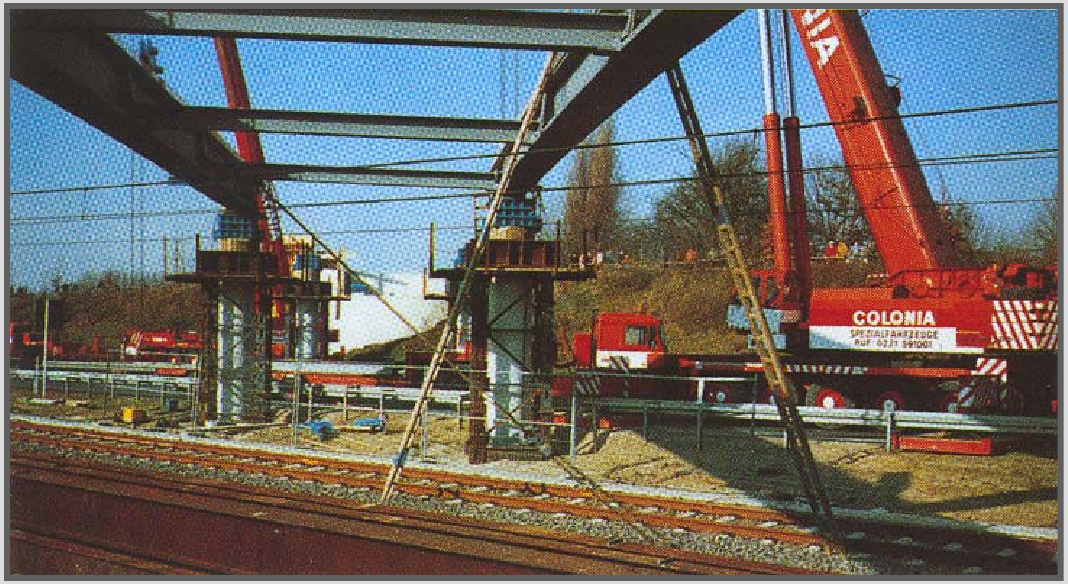
Influence of erection and load history



Example:

Bridge Arminiusstraße in Dortmund

- erection steel structure

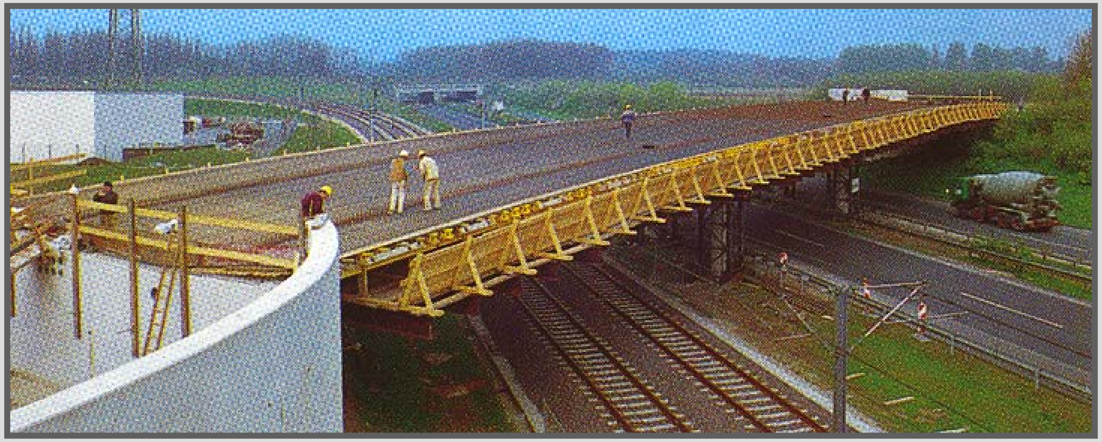


Influence of erection and load history

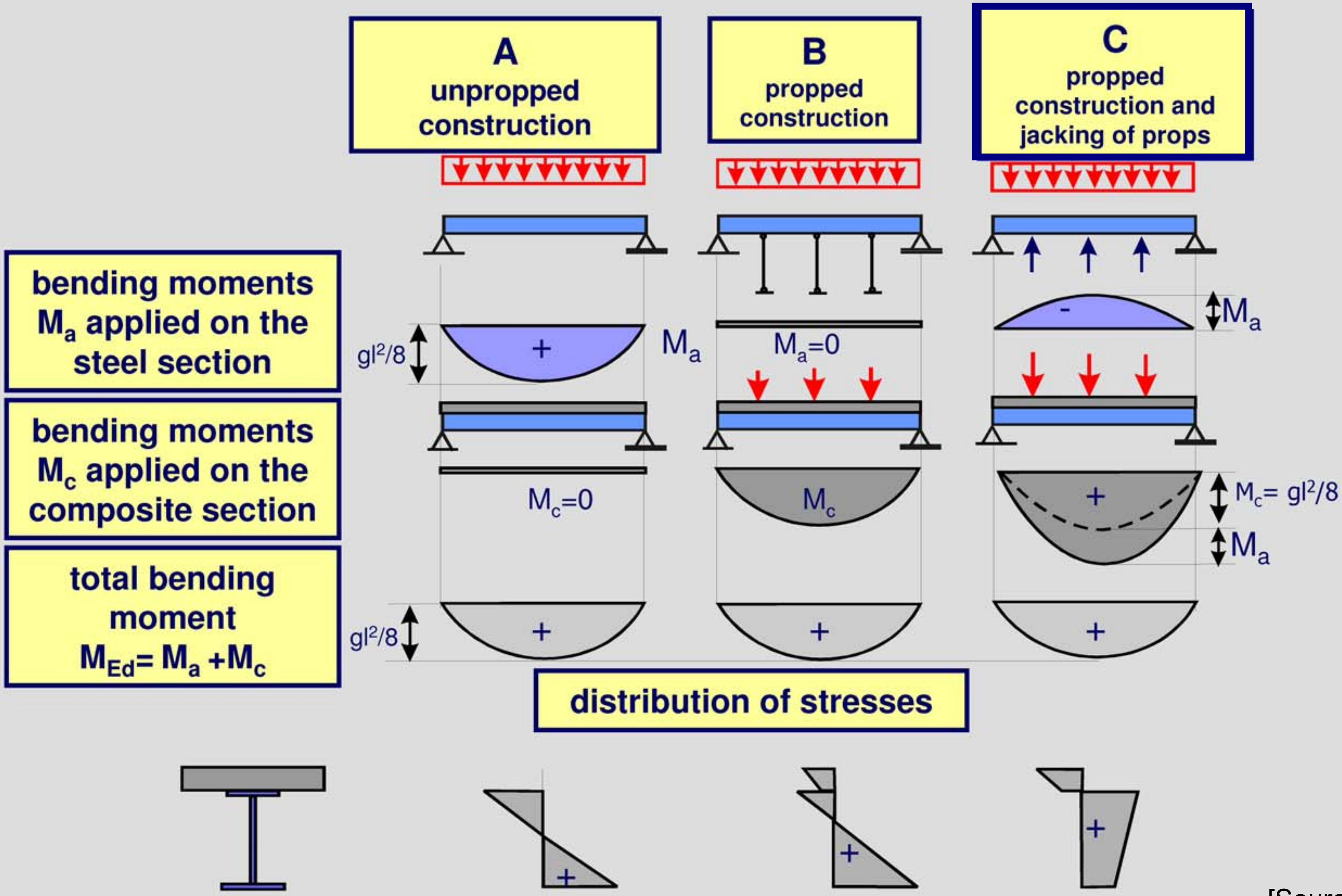
<u>Example:</u>

Bridge Arminiusstraße in Dortmund

- raising at inner supports
- scaffolding hanging at steel structure
- concreting and hardening of concrete
- lowering at inner supports
- finalizing (pavement etc.)
- traffic opening

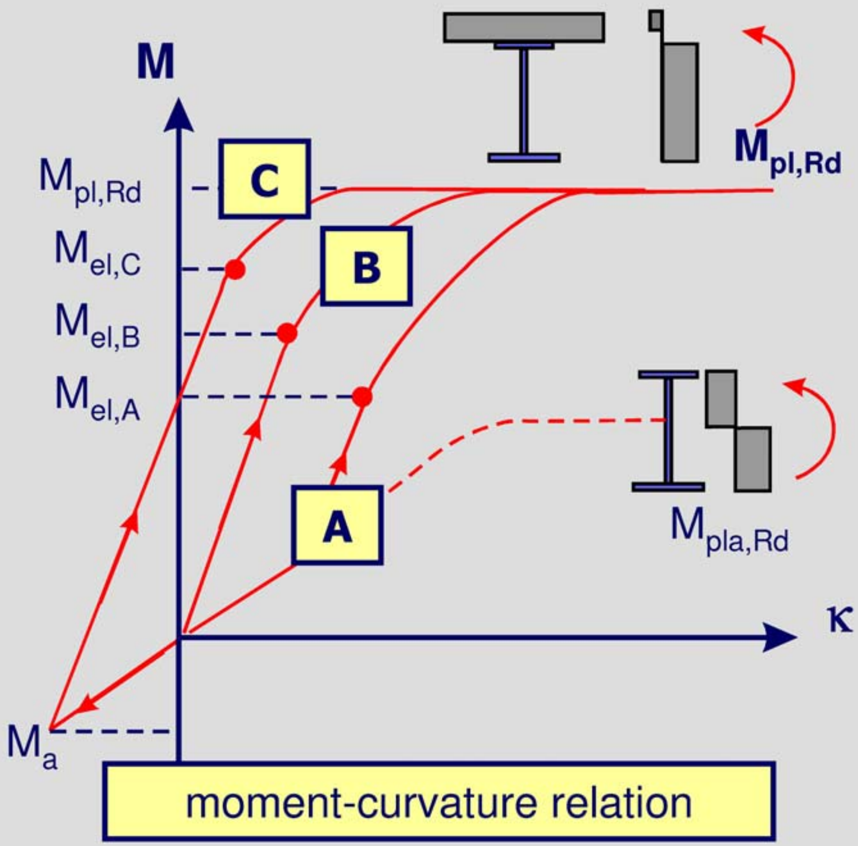


Influence of erection and load history



[Source: Hanswille]

Influence of erection and load history



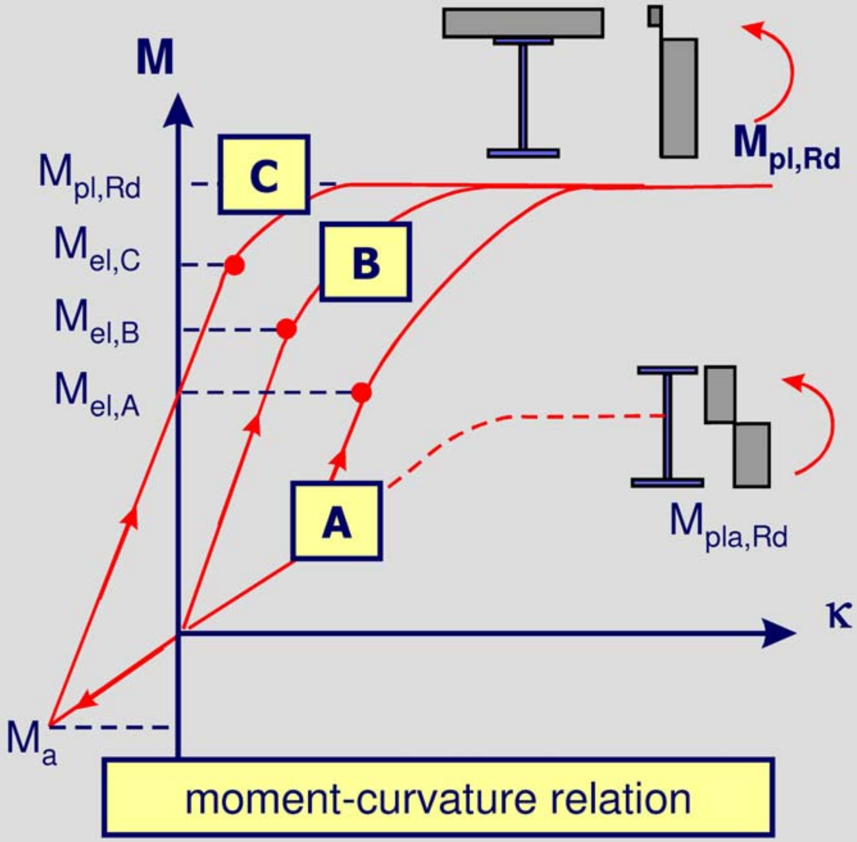
A unpropped construction

B propped construction

C propped construction + jacking of props

[Source: Hanswille]

Influence of erection and load history

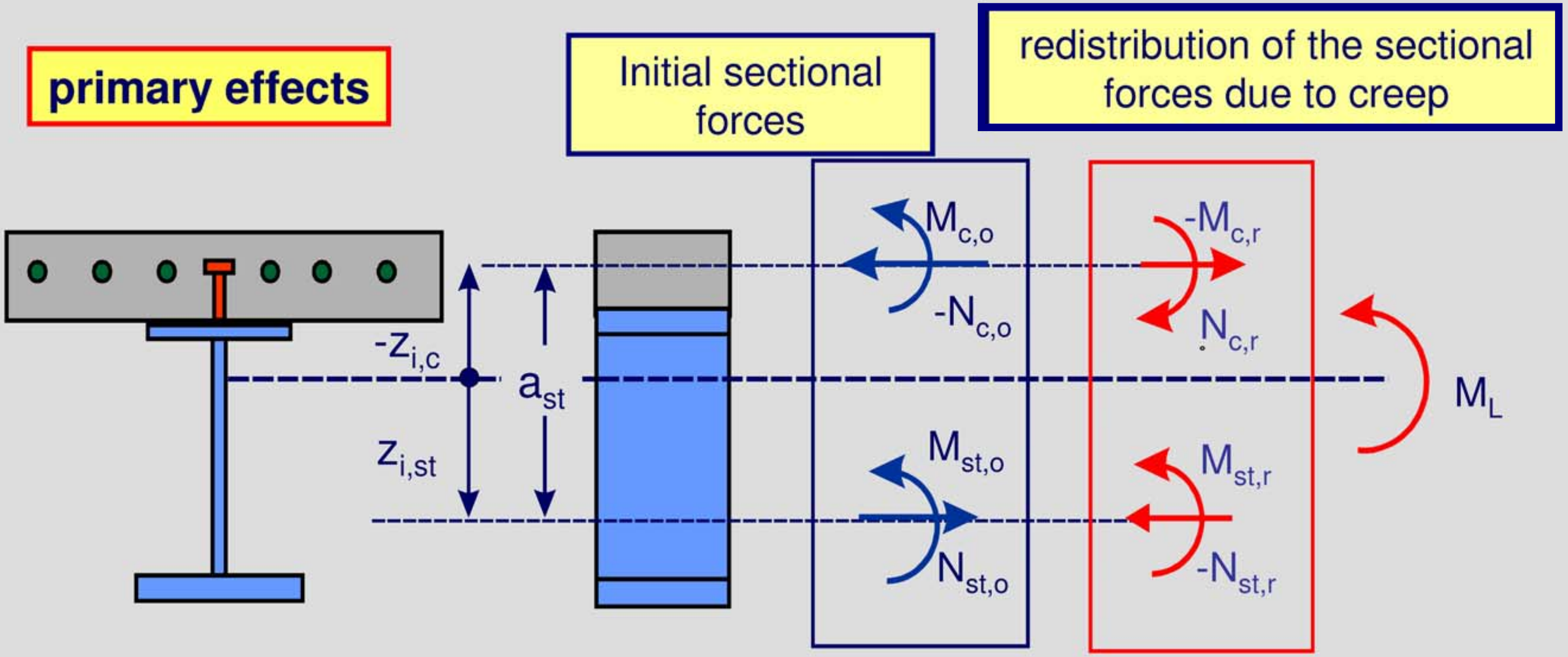


The bending capacity $M_{pl,Rd}$ is independent of the loading history in case of Class 1 or Class 2 cross sections

Using Class 3 or Class 4 cross sections the elastic behaviour of the loading history has to be taken into account in ULS

[Source: Hanswille]

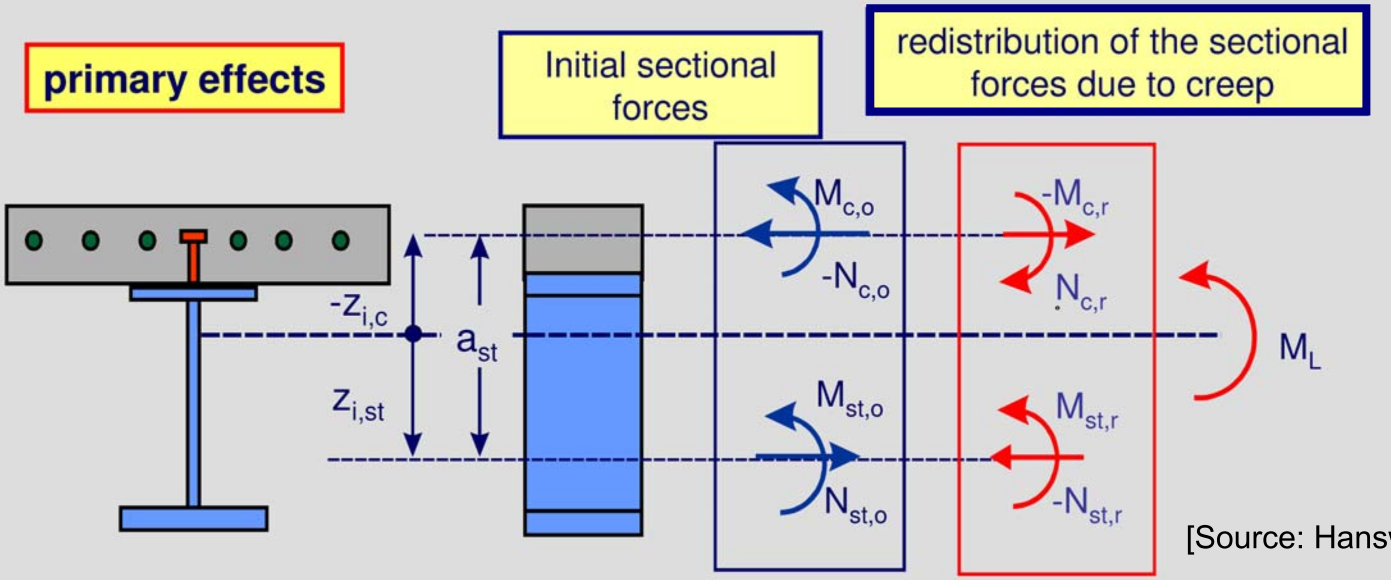
Influence of creep and shrinkage



The effects of shrinkage and creep of concrete result in internal forces in cross sections, and curvatures and longitudinal strains in members

[Source: Hanswille]

Influence of creep and shrinkage



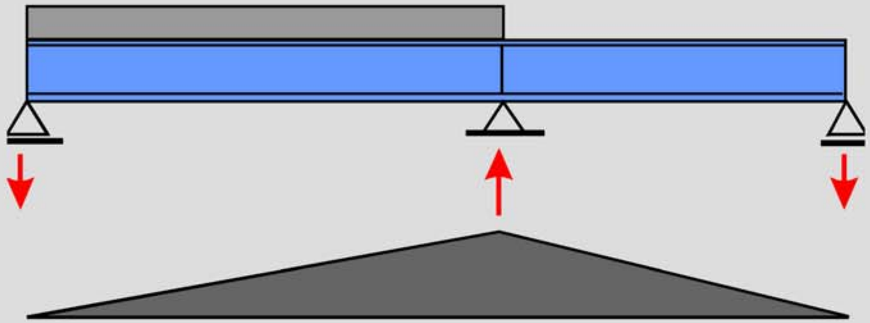
[Source: Hanswille]

Due to creep and shrinkage:	
For statically determinate structures:	only external deformations
For Class 1 and 2 sections	bending capacity independent of creep and shrinkage
For Class 3 and 4 sections	creep and shrinkage has to be considered

Influence of creep and shrinkage

secondary effects

time - dependent statically indeterminate effects

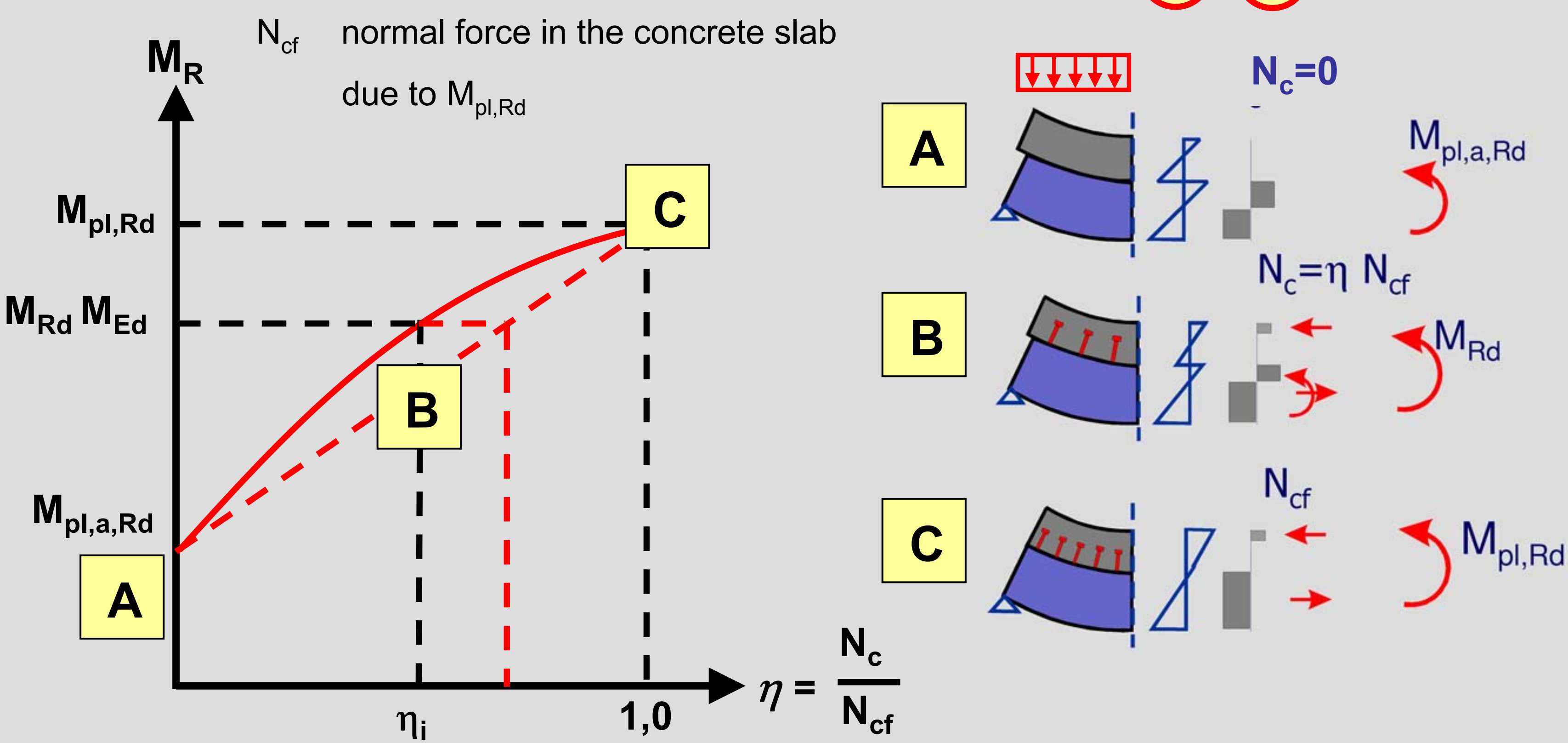


[Source: Hanswille]

In statically indeterminate structures the primary effects of shrinkage and creep are associated with additional action effects, such that the total effects are compatible;

These shall be classified as secondary effects and shall be considered as indirect actions in any case

Influence of composite interaction

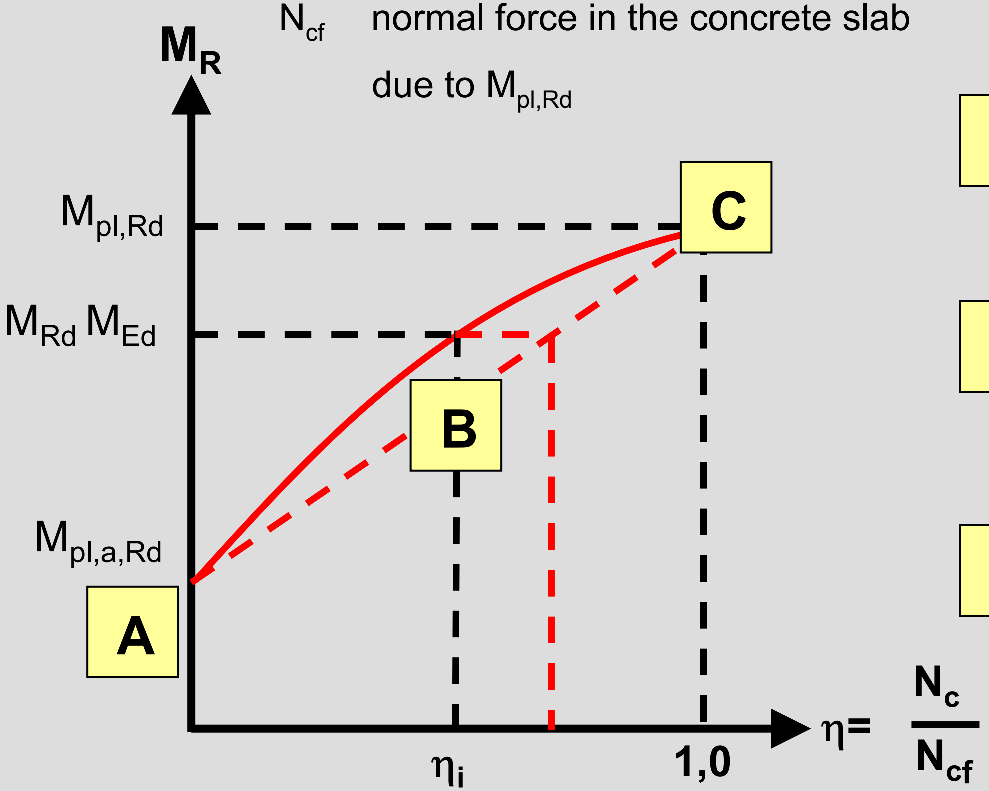


η...degree of shear connections

[Source: Hanswille]

Influence of composite interaction

N_{cf} normal force in the concrete slab due to $M_{pl,Rd}$

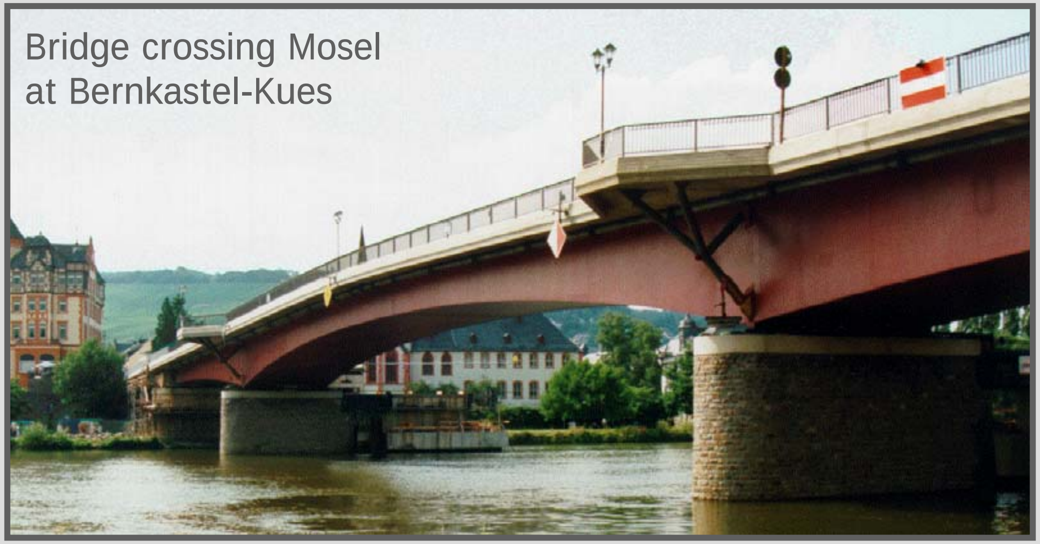


A $\eta = 0$ no shear connection acting as 2 independent sections

B $0 < \eta < 1$ partial shear connection acting as one section with slip at interface bending resistance depending on shear connection

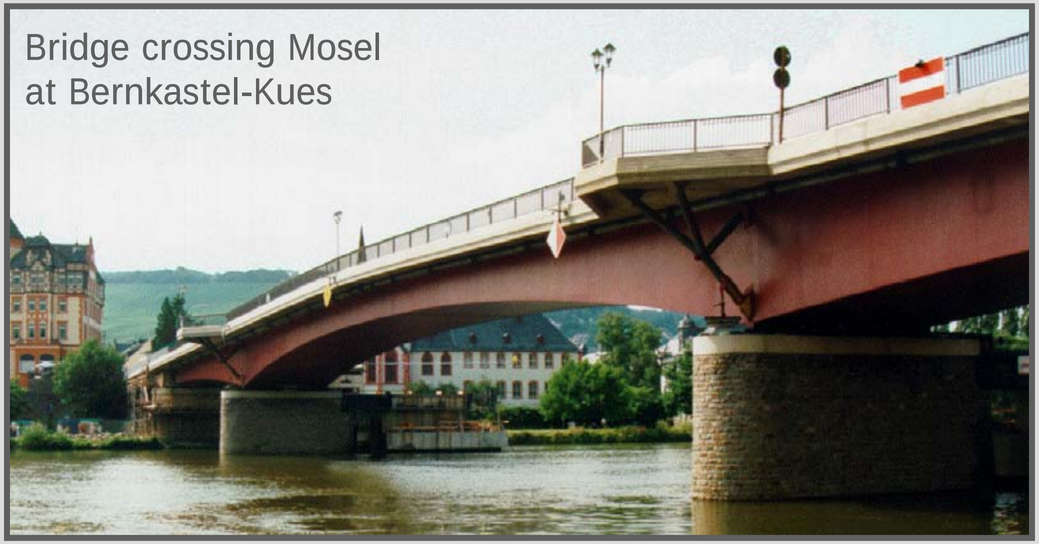
C $\eta = 1$ full shear connection acting as one section without slip full plastic resistance $M_{pl,Rd}$

Bridge crossing Mosel
at Bernkastel-Kues



Part 3:

METHODS OF GLOBAL ANALYSIS



Bridge crossing Mosel at Bernkastel-Kues

- Structural stability
- Calculation of action effects based on elastic theory
- Rigid plastic analysis
- Stresses based on elastic theory

Structural stability

General case

- Portal frames w/ shallow roof slopes
- Beam-and-column type plane frames

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 10$$

5.2.1(3)

y → undeformed geometry

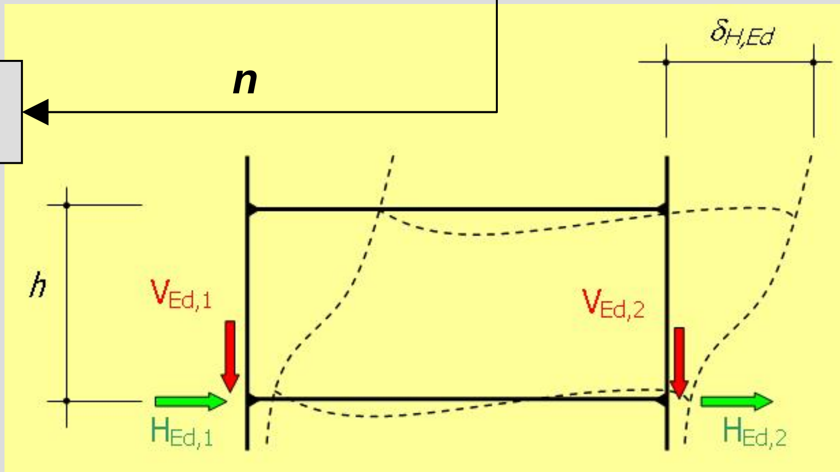
$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) \geq 10$$

5.2.1(4)B

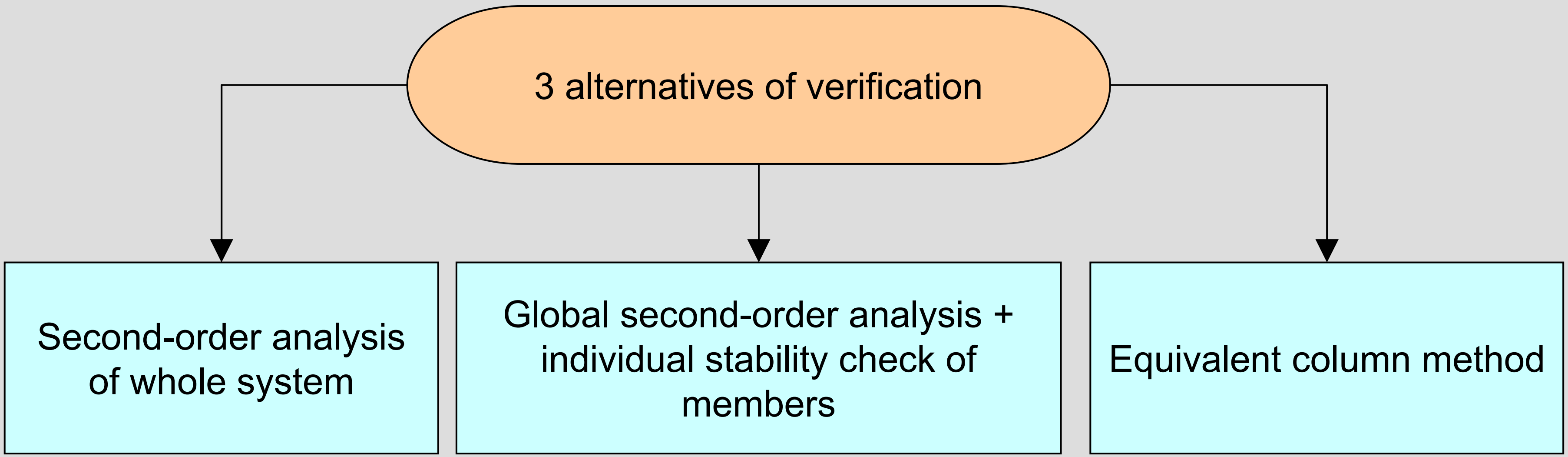
y → undeformed geometry

n → deformed geometry

deformed geometry → 3 alternatives of verification



Structural stability



EN 1994-1-1 5.2.2 (3)
EN 1994-1-1 6.7.3.6 / 7

EN 1994-1-1 5.2.2 (6) b) and 5.2.2 (6) c)

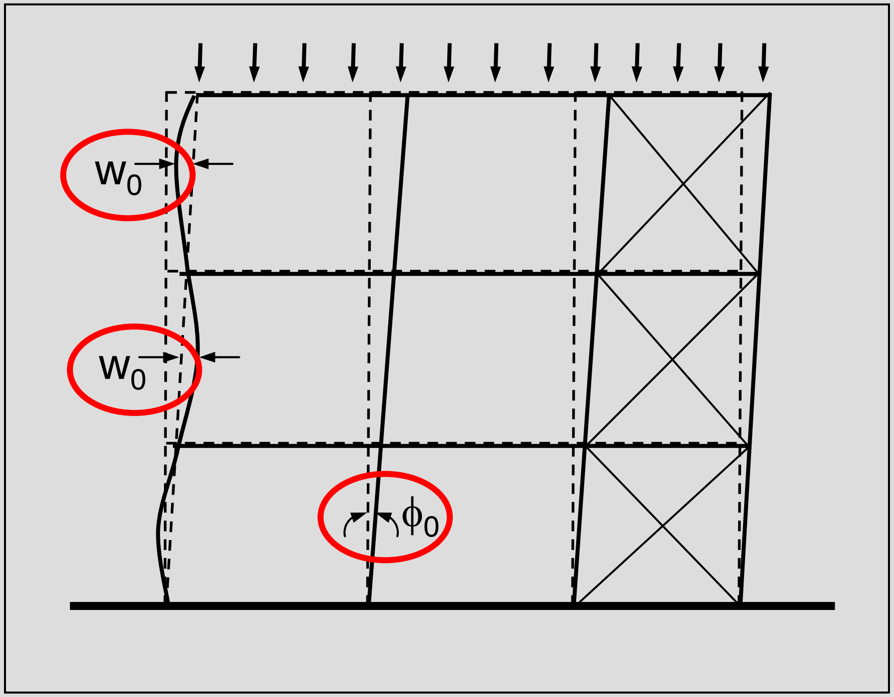
Only for steel columns:
EN 1993-1-1 5.2.2 (3) c)
5.2.2 (8)

Structural stability

3 alternatives of verification

Second-order analysis of whole system

accounting for global and local imperfections



Structural stability

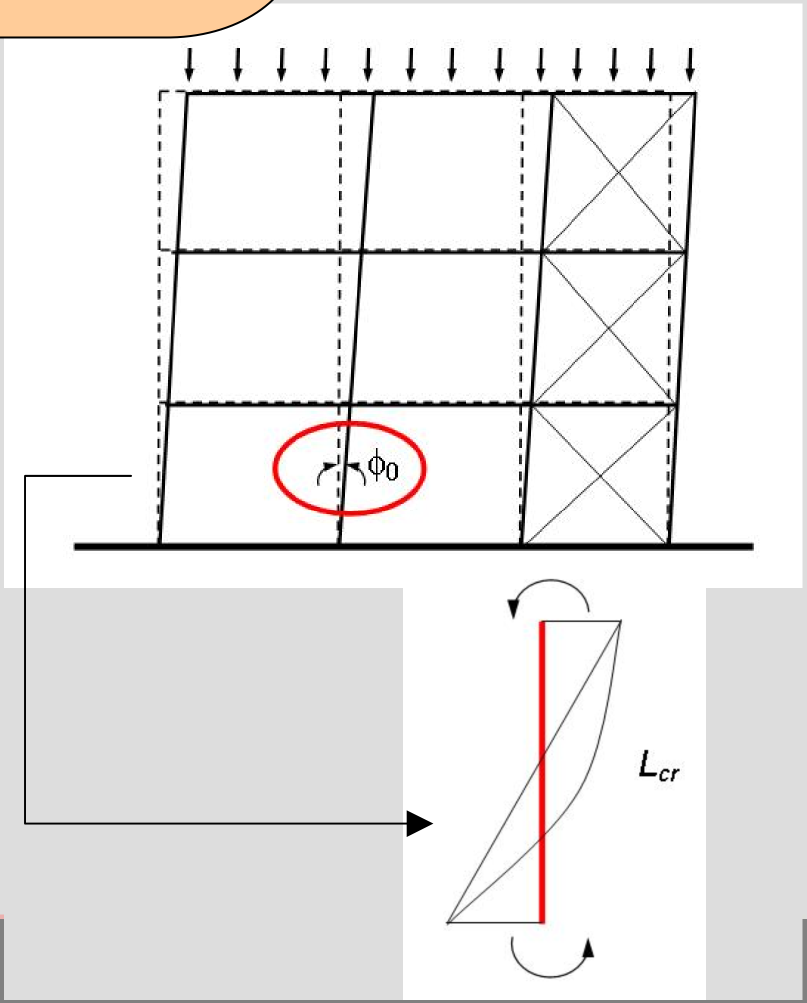
3 alternatives of verification

Global second-order analysis + individual stability check of members

including global imperfections

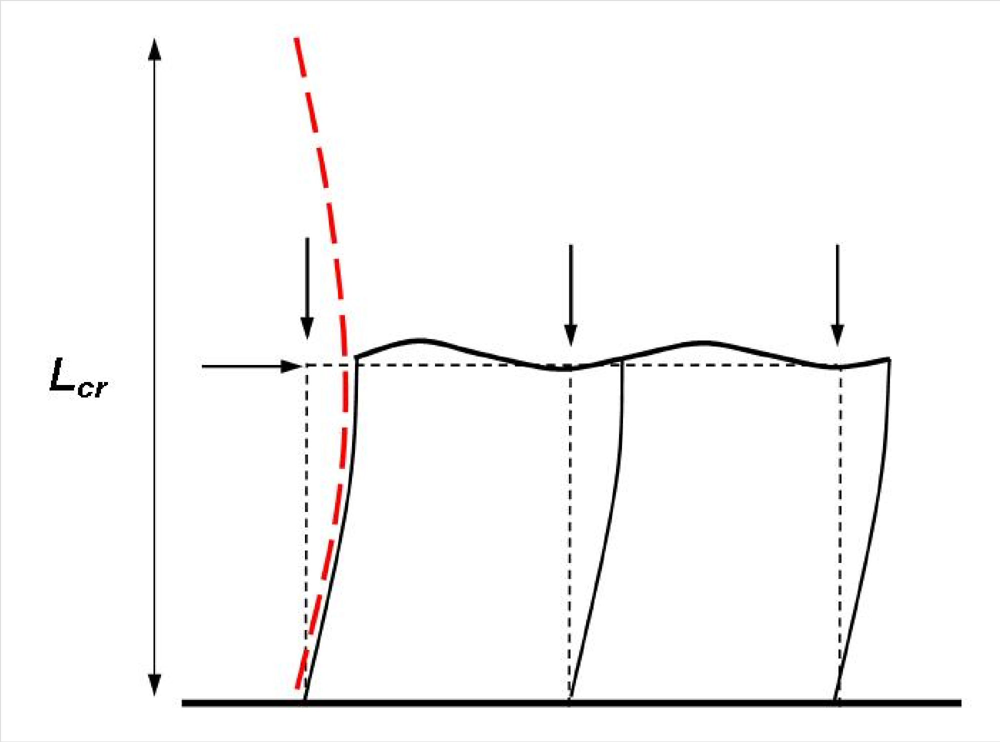
Individual stability check of members acc. to EN 1994-1-1 6.7.3.4 or 6.7.3.5

Buckling length = system length



Structural stability

3 alternatives of verification



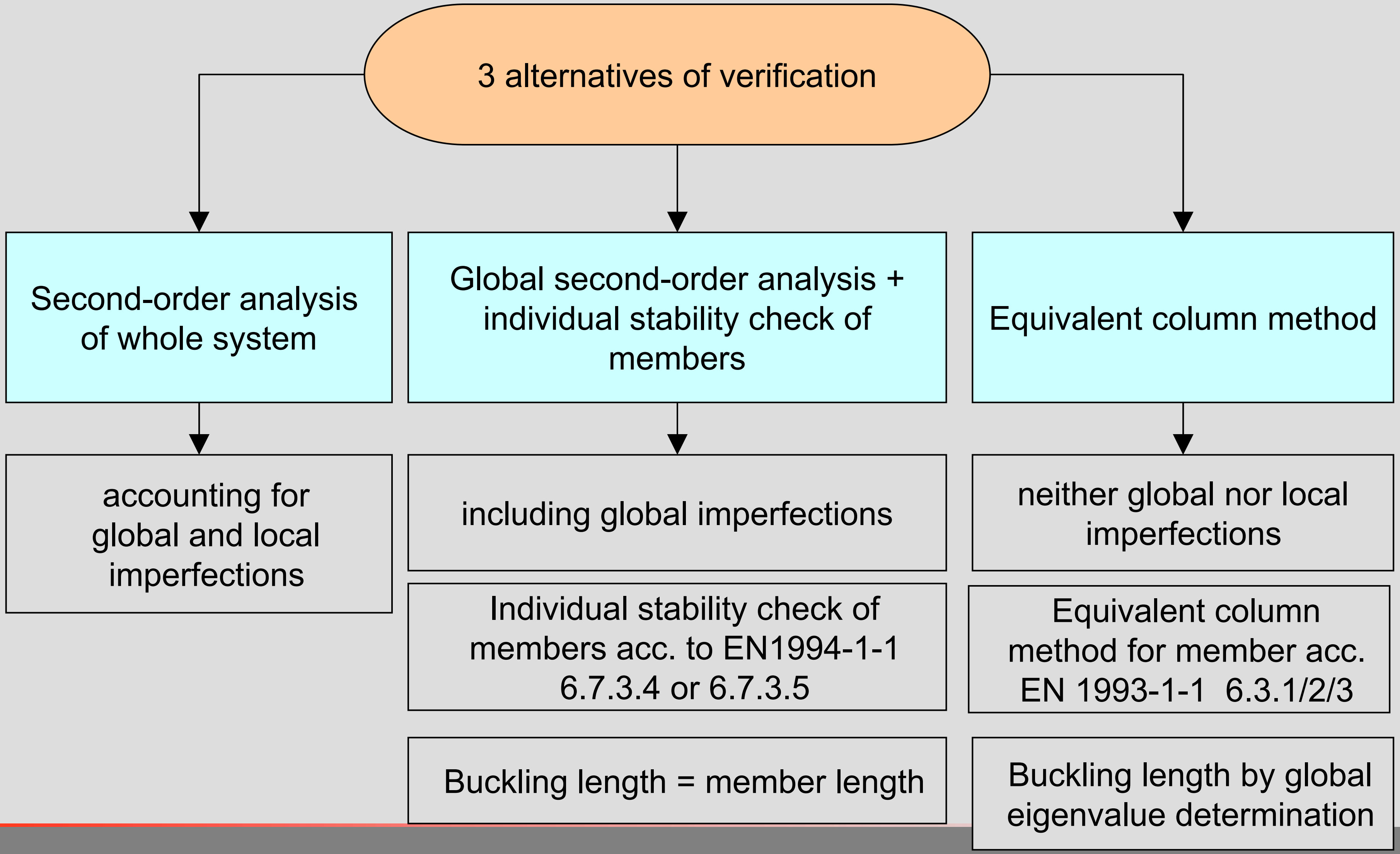
Equivalent column method

neither global nor local imperfections

Equivalent column method for member acc. EN 1993-1-1 6.3.1/2/3

Buckling length by global eigenvalue determination

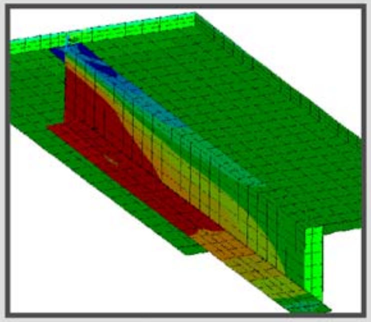
Structural stability



Calculation of action effects based on elastic theory

Methods

non-linear calculation taking into account the non-linear behaviour of materials, sequence of construction, flexibility of shear connection, creep and shrinkage and effects of tension stiffening of concrete between cracks

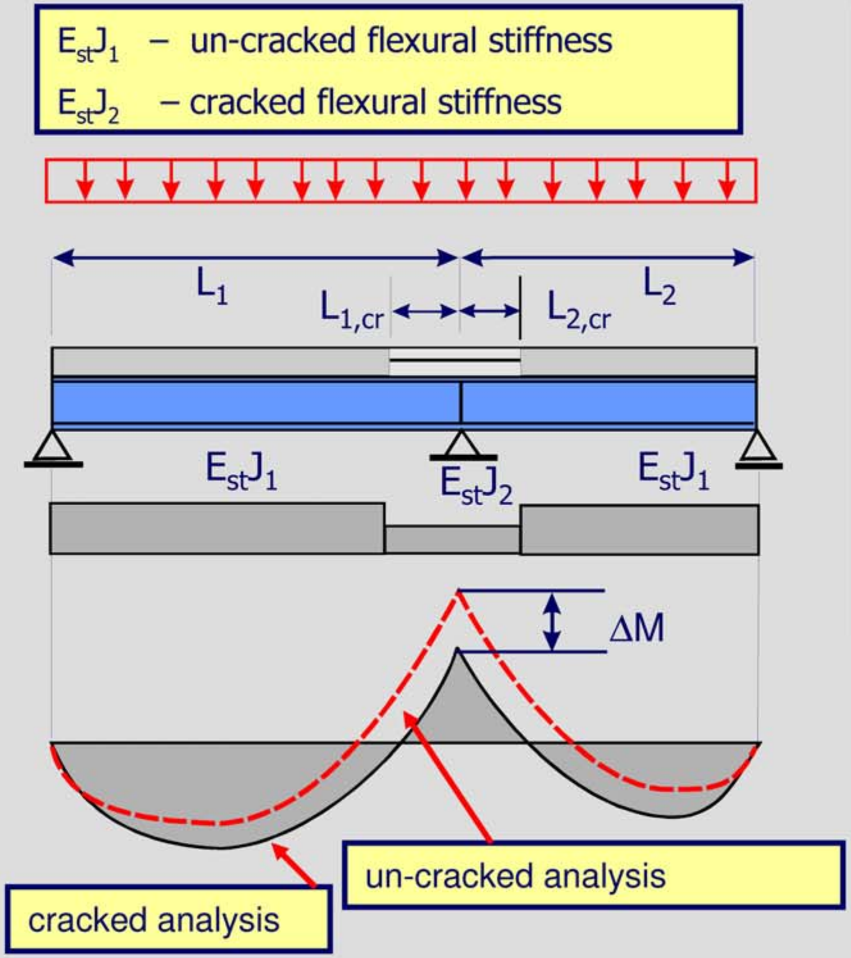


simplified methods based on elastic theory

- direct determination of the cracked regions **(general method)**
- calculation with defined regions of cracking of concrete **(Method I)**
- calculation based on un-cracked analysis and redistribution of bending moments **(Method II)**

[Source: Hanswille]

Calculation of action effects based on elastic theory - General method



- Determination of internal forces by un-cracked analysis for the characteristic combination.
- Determination of the cracked regions with the extreme fibre concrete tensile stress $\sigma_{c,max} = 2{,}0\ f_{ct,m}$.
- Reduction of flexural stiffness to EJ_2 in the cracked regions.
- New structural analysis for the new distribution of flexural stiffness.

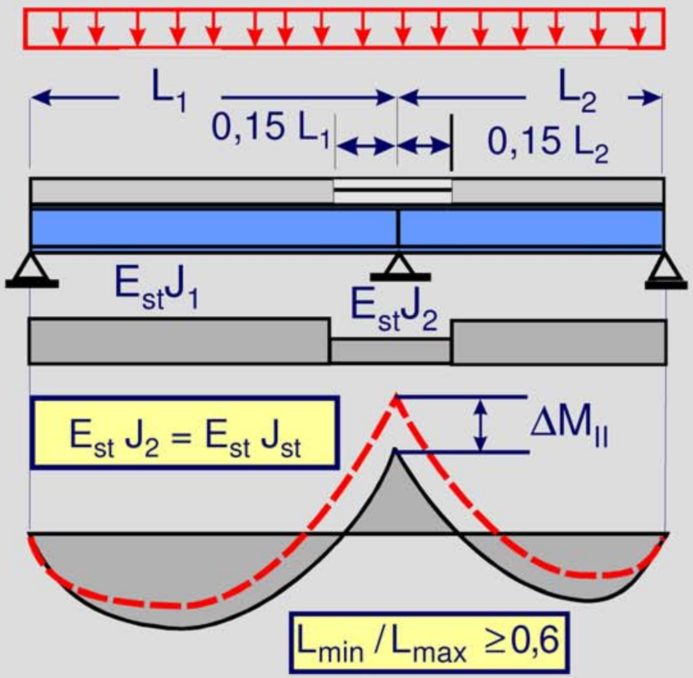
ΔM Redistribution of bending moments due to cracking

[Source: Hanswille]

Calculation of action effects based on elastic theory

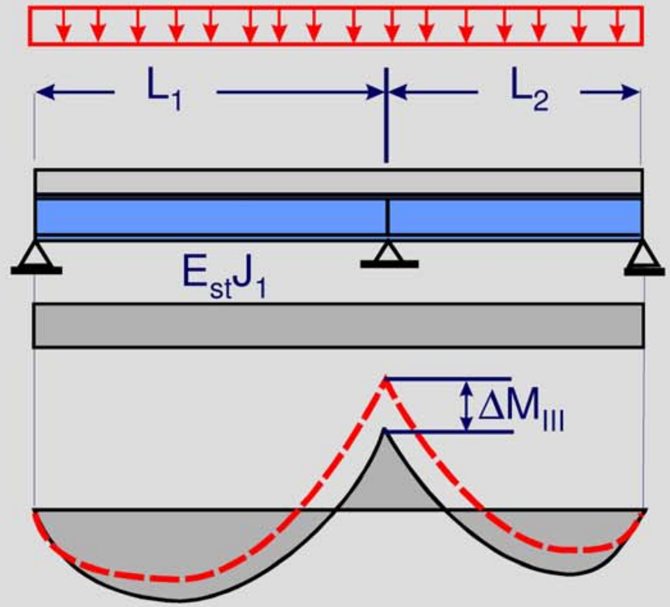
Method I:

Cracked analysis



Method II:

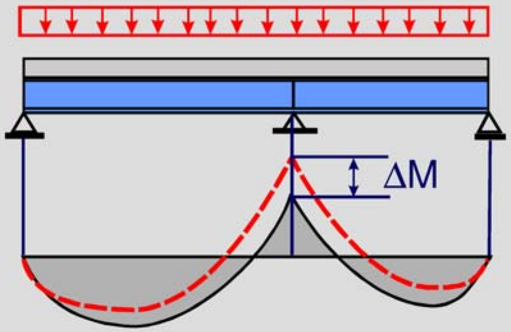
Un-cracked analysis with limited redistribution



Redistribution of bending moments by 10%

[Source: Hanswille]

Calculation of action effects based on elastic theory



- - - bending moments acc. to Method I or II

—— redistribution of bending moments ΔM

percentage of redistribution ΔM					
Class of the cross-section		**1**	**2**	**3**	**4**
Method II (un-cracked analysis)	S235 S355	40	30	20	10
	S420 S460	30		10	10
Method I (cracked analysis) or general method	S235 S355	25	15	10	0
	S420 S460	15		0	0

For composite cross-sections in Class 3 or 4, the maximum percentages of redistribution relate to bending moments assumed in design to be applied to the composite member. Moments applied to the steel member should not be redistributed.

– In beams with all cross-sections in Classes 1 or 2 only, maximum hogging moments may be increased by amounts not exceeding 10%, for un-cracked elastic analysis or 20% for cracked elastic analysis

[Source: Hanswille]

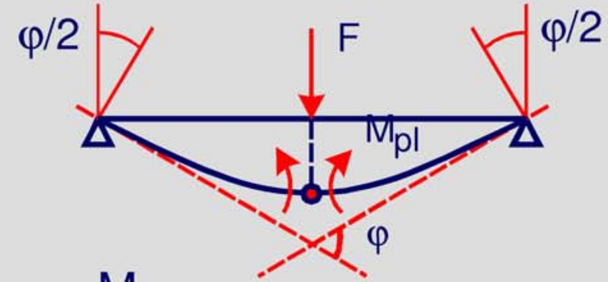
Relation

Classification - method of global analysis - resistance

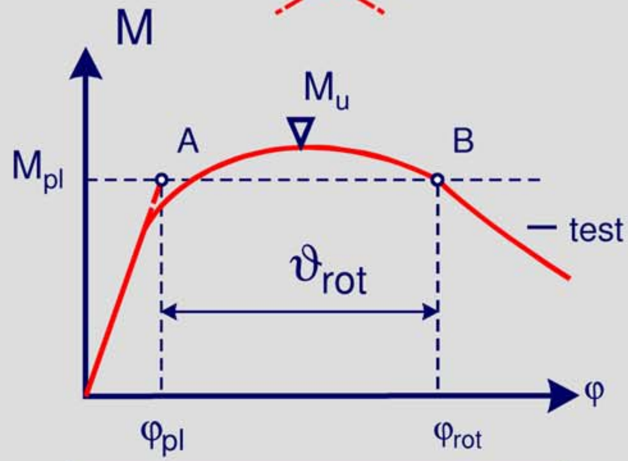
Class of the cross-section	global analysis (action effects E_d)	Consideration of creep, shrinkage, cracking of concrete and sequence of construction	Resistance R_d
1	rigid plastic or elastic analysis with redistribution of bending moments	no	plastic
2	elastic analysis with redistribution of bending moments	no	plastic
3	elastic analysis	yes	elastic
4	elastic analysis	yes	elastic acc. to EN 1993-1-5

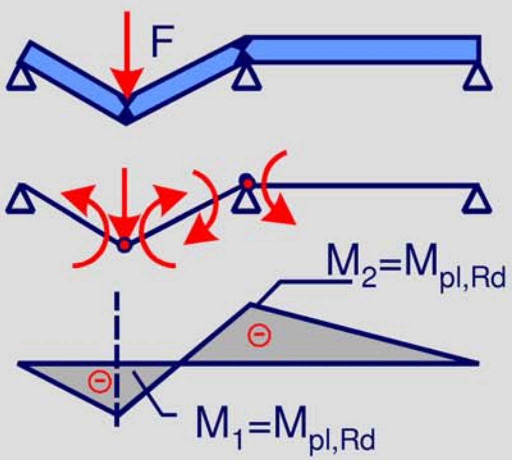
[Source: Hanswille]

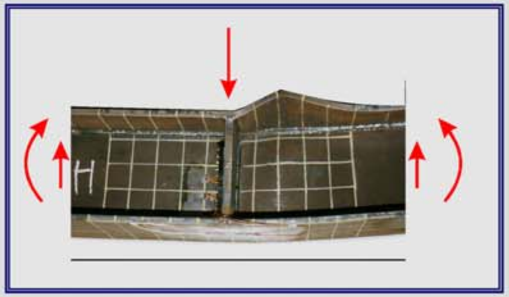
Rigid plastic analysis



Rigid plastic global analysis may be used for ultimate limit state verifications other than fatigue, where second-order effects do not have to be considered and provided that all the members and joints of the frame are steel or composite, the steel **material** satisfies **ductility requirements** EN 1993-1-1, the **cross-sections** of steel members have **sufficient rotation capacity** and the **joints** are able to sustain their plastic resistance moments for a sufficient rotation capacity.



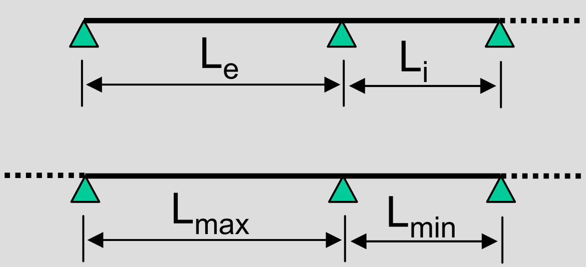




[Source: Hanswille]

Rigid plastic analysis

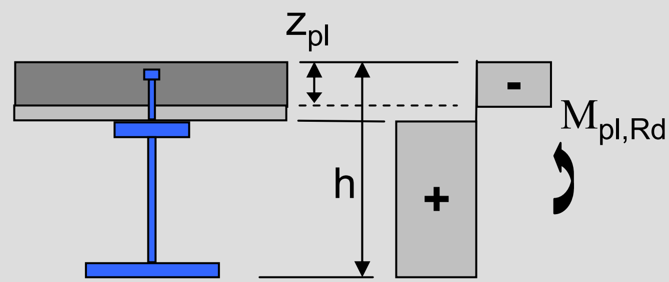
Limitation of span ratio:

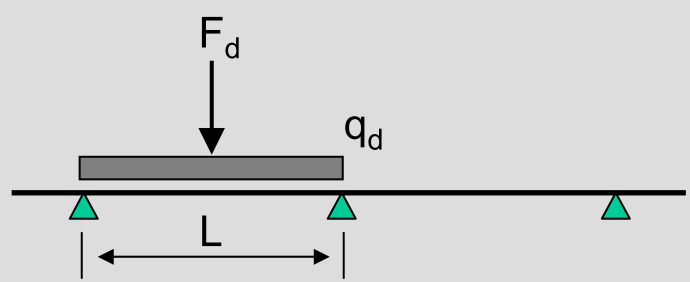


exterior span: $L_e < 1{,}15\ L_i$

interior span: $L_{max}/L_{min} \leq 1{,}50$

Beam with single load and rotation requirements at span:





$$\frac{z_{pl}}{h} \leq 0{,}15 \quad \text{if} \quad \frac{F_d}{F_d + q_d\,L} > 0{,}5$$

Rigid plastic analysis

Where rigid-plastic global analysis is used, at each plastic hinge location:

a) the cross-section of the structural steel section shall be symmetrical about a plane parallel to the plane of the web or webs,

b) the proportions and restraints of steel components shall be such that lateral-torsional buckling does not occur,

c) lateral restraint to the compression flange shall be provided a tall hinge locations at which plastic rotation may occur under any load case,

d) the rotation capacity shall be sufficient, when account is taken of any axial compression in the member or joint, to enable the required hinge rotation to develop and

e) where rotation requirements are not calculated, all members containing plastic hinges shall have effective cross-sections of Class 1 at plastic hinge locations.

Rigid plastic analysis

For composite beams in buildings, the rotation capacity may be assumed to be sufficient where:

a) the grade of structural steel does not exceed S355,

b) the contribution of any reinforced concrete encasement in compression is neglected when calculating the design resistance moment,

c) all effective cross-sections at plastic hinge locations are in Class1; and all other effective cross-sections are in Class1 or Class2,

d) each beam-to-column joint has been shown to have sufficient design rotation capacity, or to have a design resistance moment at least 1,2 times the design plastic resistance moment of the connected beam,

e) adjacent spans do not differ in length by more than 50% of the shorter span,

f) end spans do not exceed 115% of the length of the adjacent span,

g) in any span in which more than half of the total design load for that span is concentrated within a length of one-fifth of the span, then at any hinge location where the concrete slab is in compression, not more than 15% of the overall depth of the member should be in compression; this does not apply where it can be shown that the hinge will be the last to form in that span,

h) the steel compression flange at a plastic hinge location is laterally restrained.

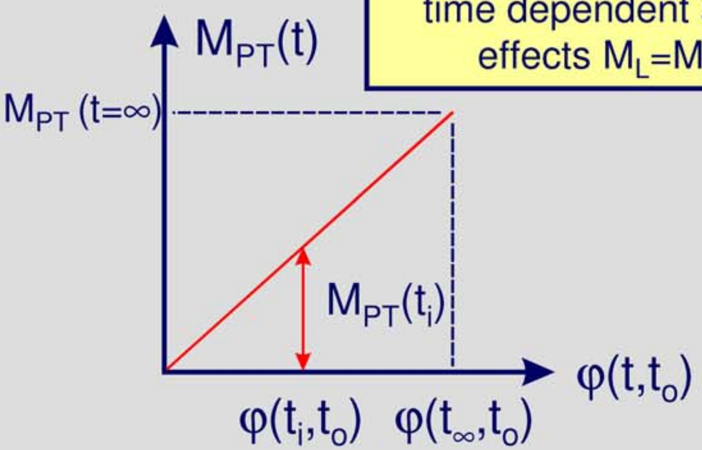
Stresses based on elastic theory

Types of loading and action effects:

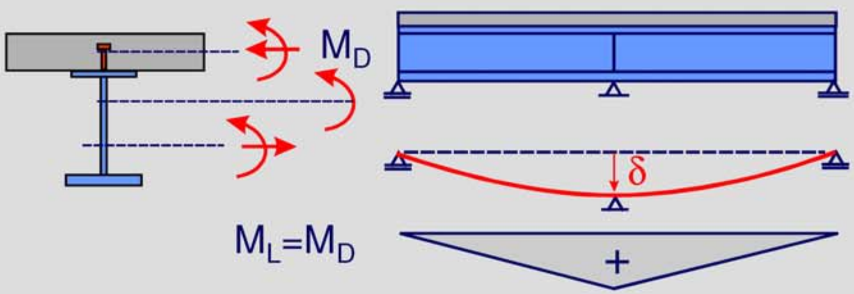
In the following the different types of loading and action effects are distinguished by a subscript L :

- **L=P** for permanent action effects not changing with time
- **L=PT** time-dependent action effects developing affine to the creep coefficient
- **L=S** action effects caused by shrinkage of concrete
- **L=D** action effects due to prestressing by imposed deformations (e.g. jacking of supports)

time dependent action effects $M_L=M_{PT}$:



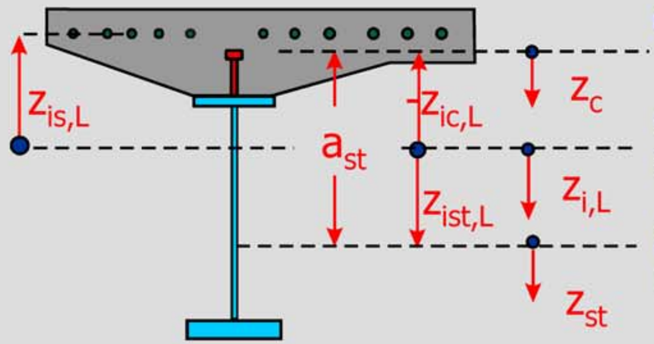
action effects caused by prestressing due to imposed deformation $M_L=M_D$:



[Source: Hanswille]

Stresses based on elastic theory

Modular ratios taking into account effects of creep



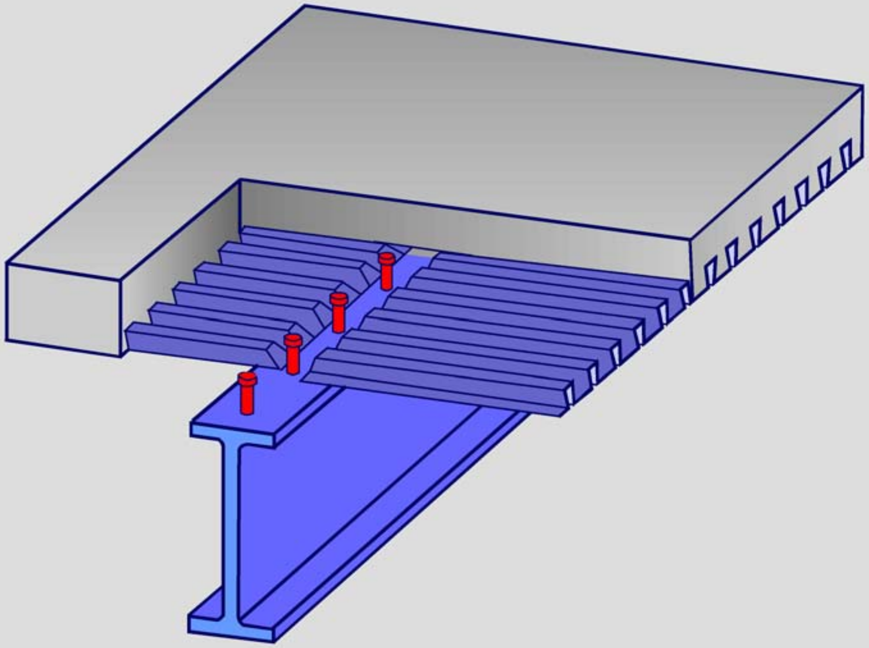
Modular ratios:

$$n_L = n_o \left[1 + \psi_L \, \varphi(t, t_o) \right] \qquad n_o = \frac{E_a}{E_{cm}}$$

Creep multipliers ψ_L:

short term loading	$\Psi=0$
permant action constant in time	$\Psi_P=1,10$
shrinkage	$\Psi_S=0,55$
Prestressing by imposed deformations	$\Psi_D=1,50$
time-dependent action effects	$\Psi_{PT}=0,55$

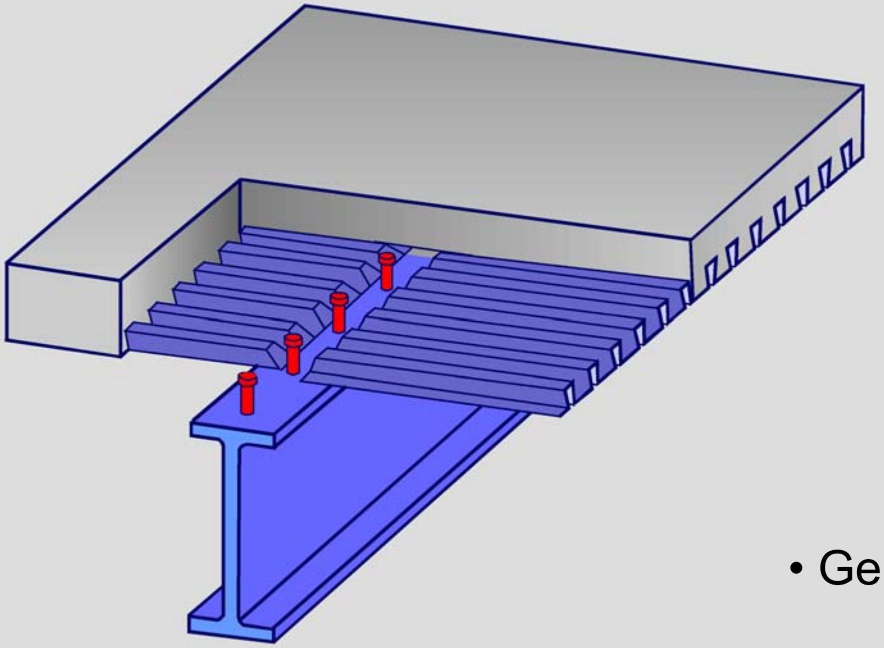
[Source: Hanswille]



Part 4:

VERIFICATION FOR BENDING AND SHEAR

FOR ULTIMATE LIMITE STATE



- General
- Resistance of class 1 and 2 sections
- Resistance of class 3 and 4 sections
- Lateral torsional buckling

General - Basis of design

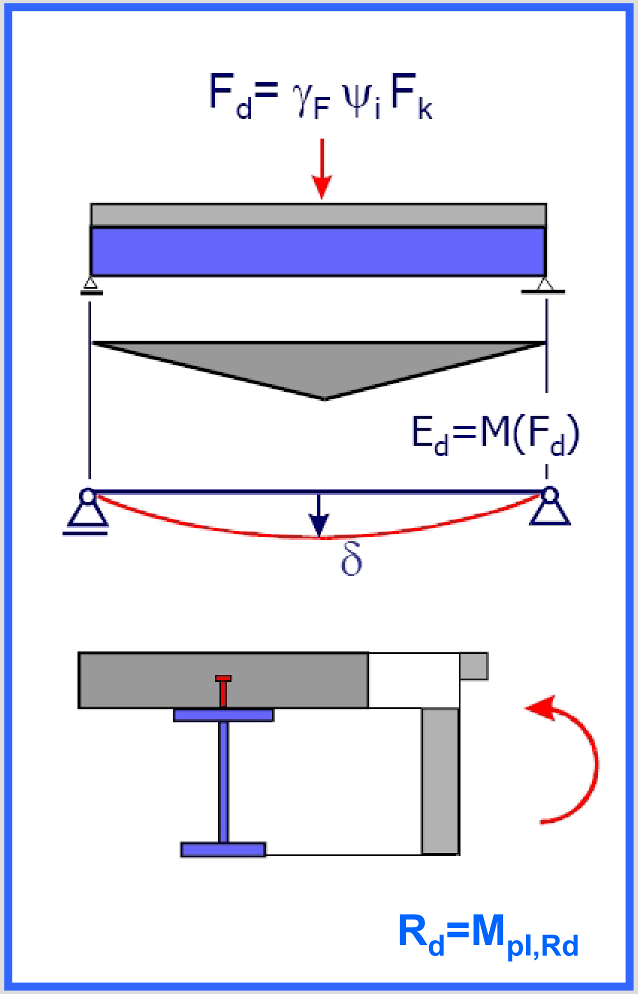
Ultimate limit state: $E_d \leq R_d$

$$R_d = R\left[\frac{f_{yk}}{\gamma_a}, \frac{\alpha_c f_{ck}}{\gamma_c}, \frac{f_{sk}}{\gamma_s}, \frac{P_{Rk}}{\gamma_v}\right]$$

$$E_d = E\left(\gamma_F \psi F_k\right)$$

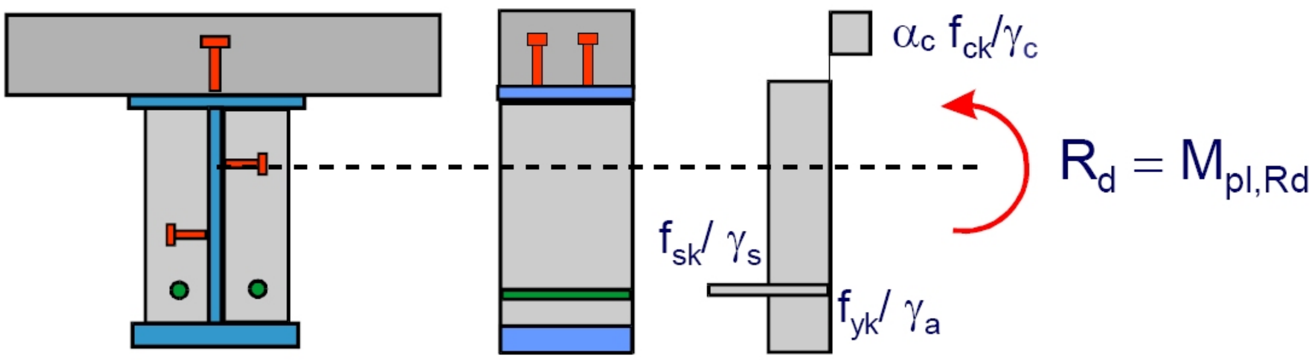
Serviceabilitliy limit state: $E_d \leq C_d$

$$E_d = E\left(\psi F_k\right)$$



[Source: Hanswille]

General - Basis of design



$$R_d = R(f_{cd}, f_{yd}, f_{sd}, P_{Rd}) = R\left(\frac{\alpha\, f_{ck}}{\gamma_c}, \frac{f_{yk}}{\gamma_{Ma}}, \frac{f_{sk}}{\gamma_s}, \frac{P_{Rk}}{\gamma_v}\right)$$

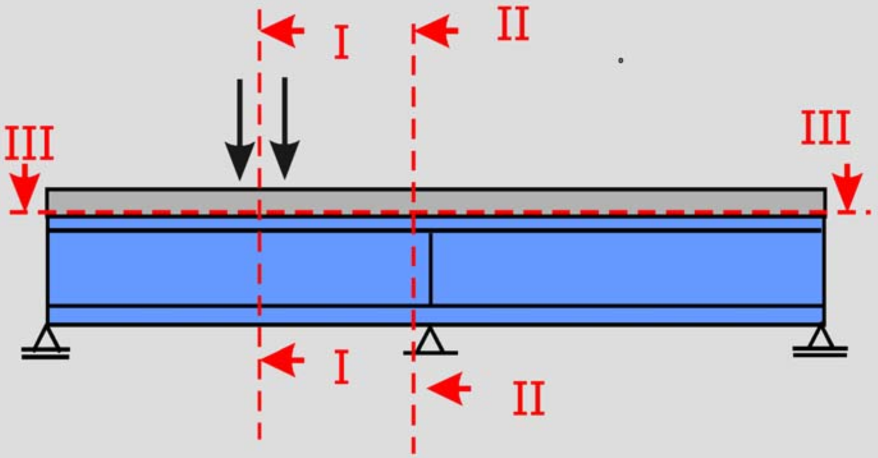
Partial safety factor for concrete
γ_C according to EN 1992-1-1 e.g. γ_C = 1.5

Partial safety factor for reinforcement steel
γ_S according to EN 1992-1-1 e.g. γ_S= 1.15

Partial safety factor for structural steel
γ_{Ma} according to EN 1993-1-1 e.g. γ_{M0} = 1.0

[Source: Hanswille]

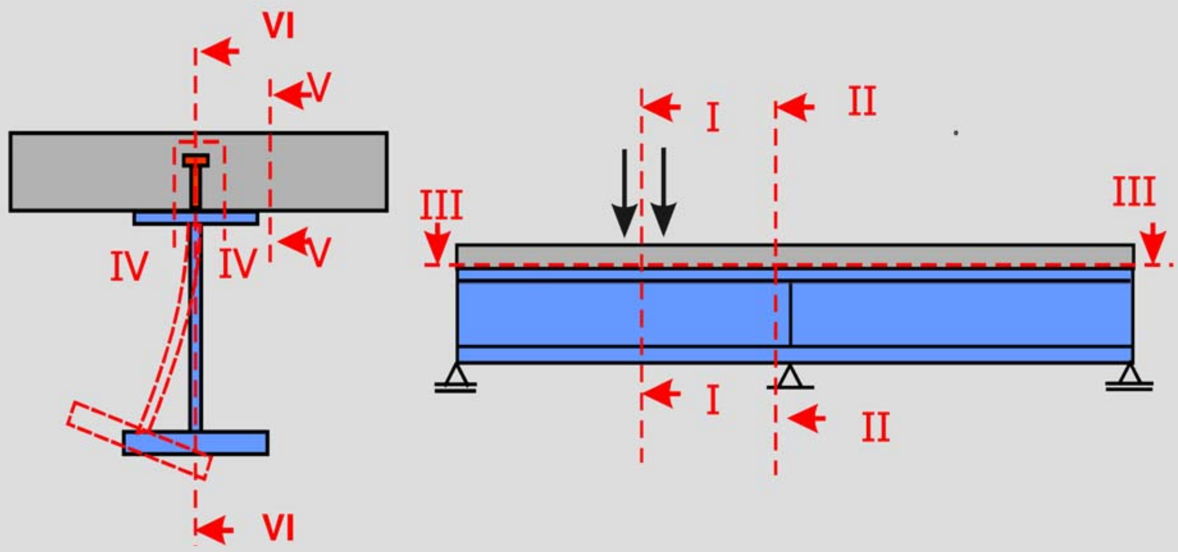
General - Required verifications for composite beams



I-I	resistance to bending and shear
II-II	resistance to bending and shear and M-V interaction
III-III	shear connection – longitudinal shear

[Source: Hanswille]

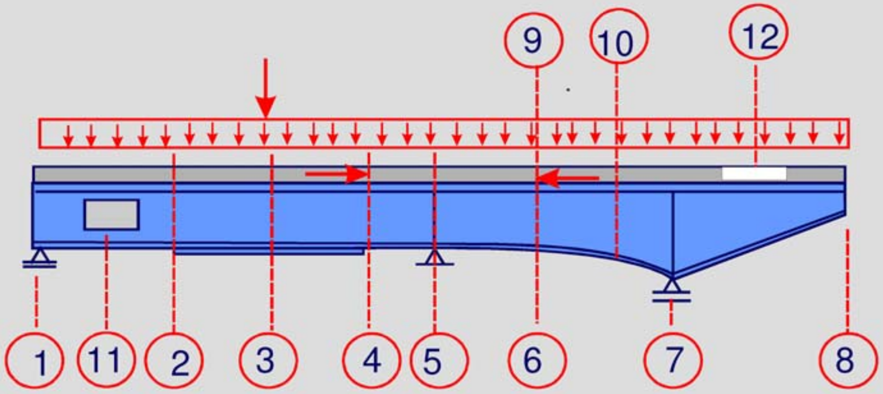
General - Required verifications for composite beams

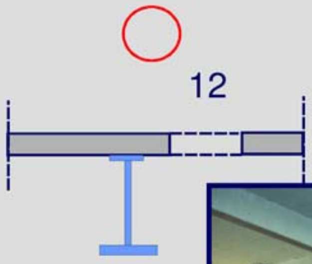


I-I	resistance to bending and shear
II-II	resistance to bending and shear and M-V interaction
III-III	shear connection – longitudinal shear
IV-IV	local introduction of longitudinal shear forces in the concrete slab
V-V	Longitudinal shear of the concrete flange
VI-VI	lateral torsional buckling

[Source: Hanswille]

General – Critical cross section





critical cross-sections

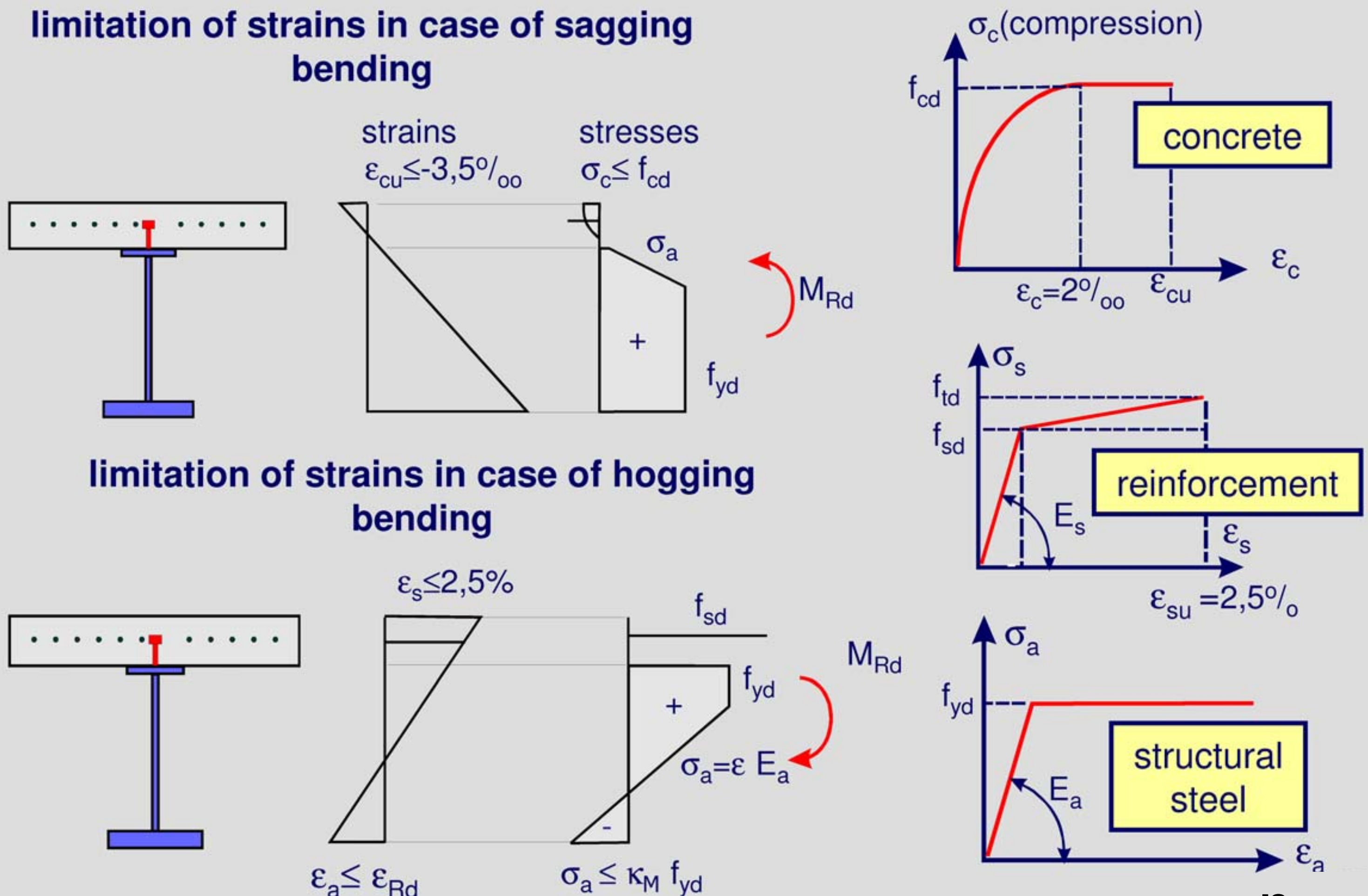
- sections of maximum bending (3,5,7)
- supports (1,5,7)
- sections subjected to concentrated loads (3)
- sudden cange of cross-section (2,4)
- introduction of longitudinal forces (4,6)
- cross-sections with web openeings (11)
- cross-sections with openings in the concrete flange (12)

For longitudinal shear verification, a critical length consists of a length between two critical cross-sections. for this case critical cross-sections also include:

- free ends of cantilevers (8)
- in tapering members, sections so chosen that the ratio of the greater to the lesser plastic resistance moment for any pair of adjacent cross-sections does not exceed 1,5 (9,10).

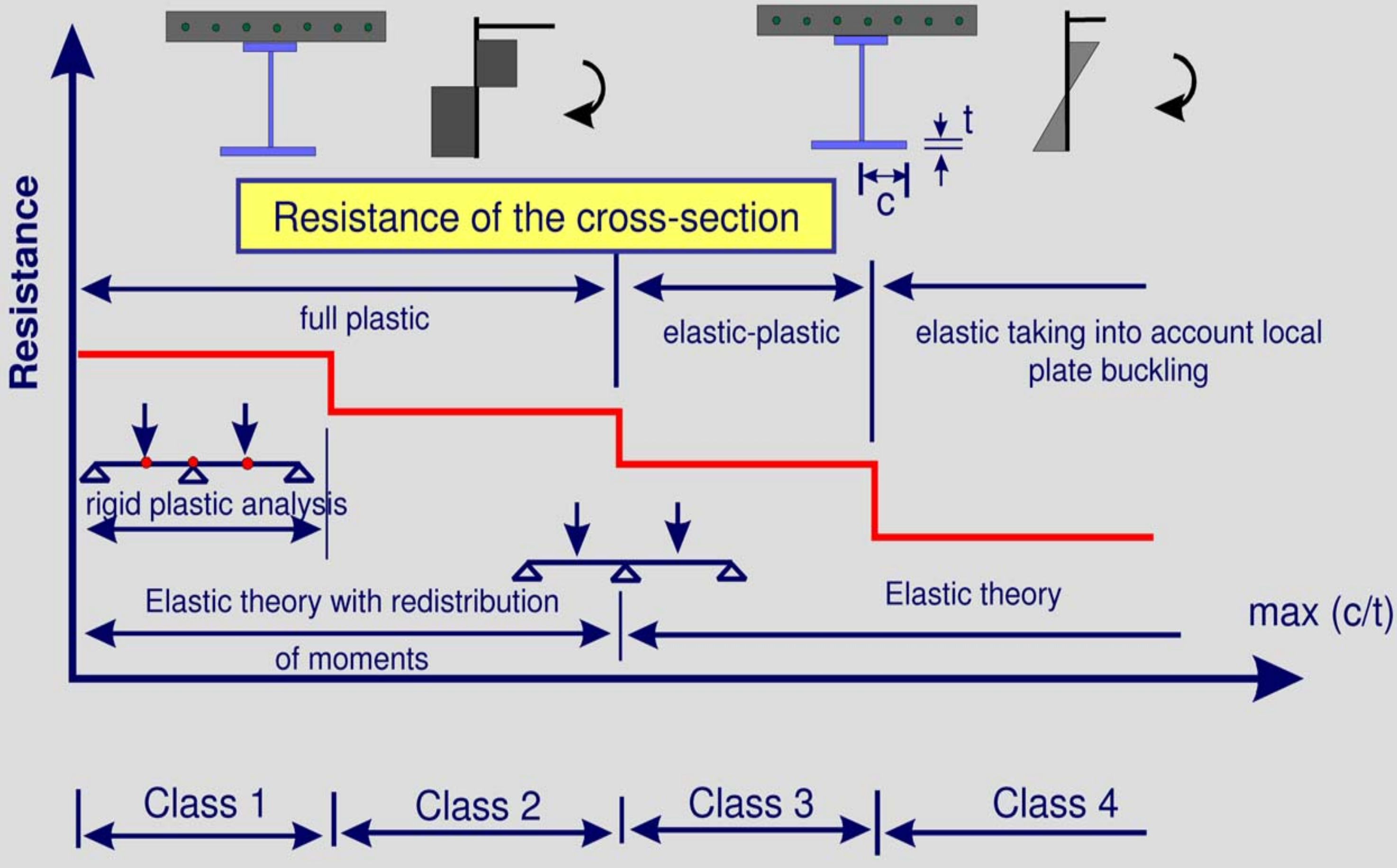
[Source: Hanswille]

General – Non-linear bending resistance



[Source: Hanswille]

Classification girders



[Source: Hanswille]

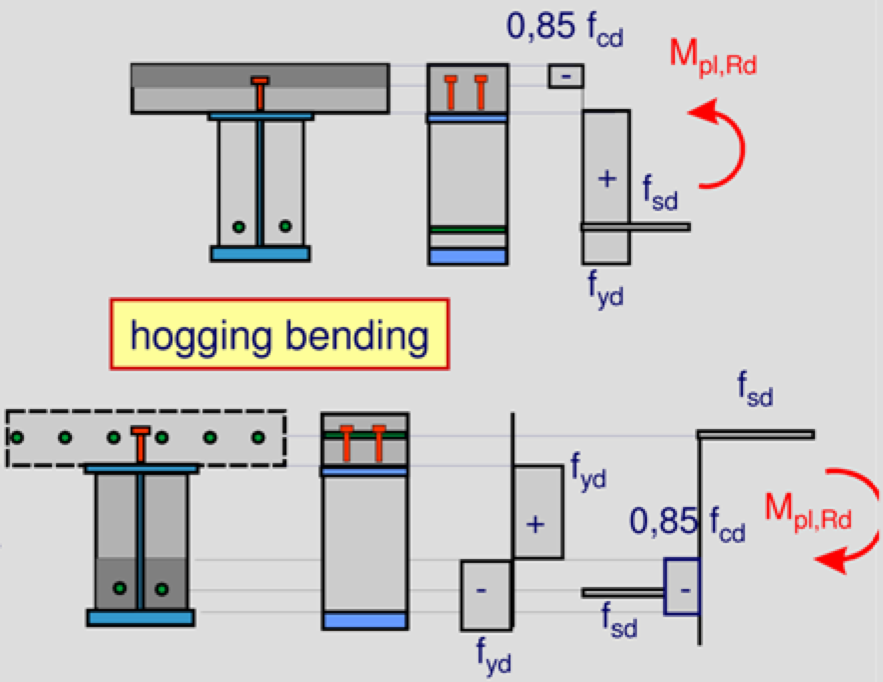
Resistance of class 1 and 2 sections - classification

assumptions for the calculation of the plastic bending resistance

- full interaction between structural steel, reinforcement and concrete,
- the effective area of the structural steel member is stressed to its design yield strength f_{yd} in tension or compression,
- the effective areas of longitudinal reinforcement in tension and in compression are stressed to their design yield strength f_{sd} in tension or compression
- the effective area of concrete in compression resists a stress constant over the whole depth between the plastic neutral axis and the most compressed fibre of the concrete, where f_{cd} is the design cylinder compressive strength of concrete.

sagging bending

hogging bending

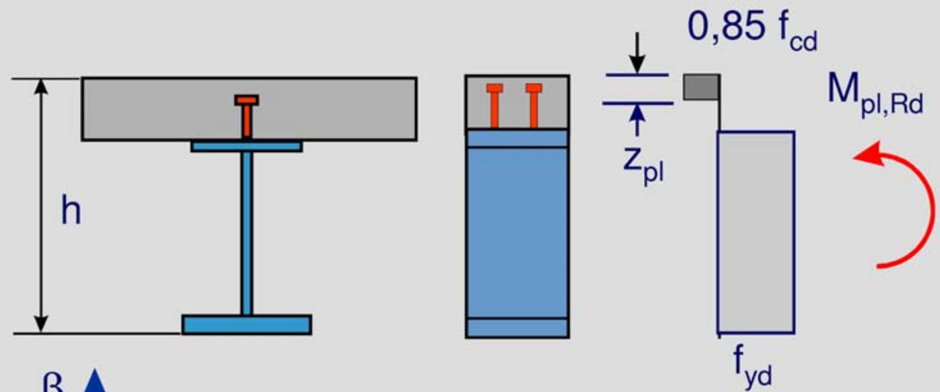


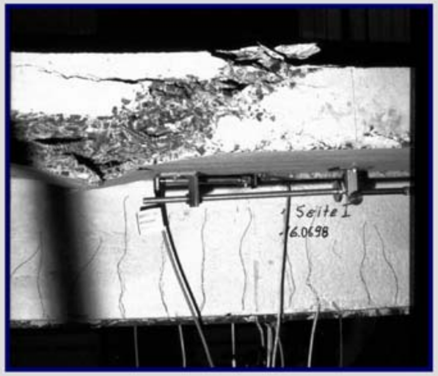
$$f_{yd} = \frac{f_{yk}}{\gamma_a} \qquad f_{cd} = \frac{f_{ck}}{\gamma_c} \qquad f_{sd} = \frac{f_{sk}}{\gamma_s}$$

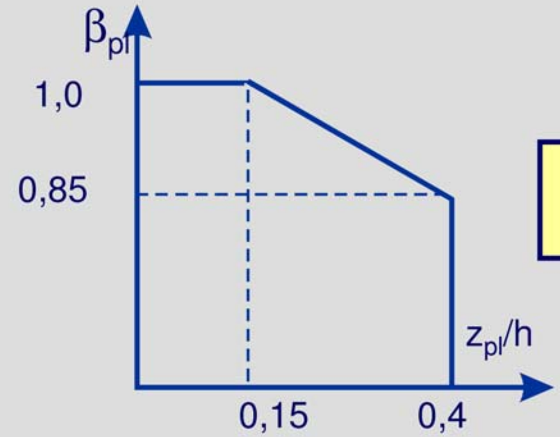
[Source: Hanswille]

Reduction of plastic bending resistance

In case of a deep position of the plastic neutral axis in sagging bending the plastic bending resistance is limited by crushing of concrete in the extreme fibre of the concrete slab.



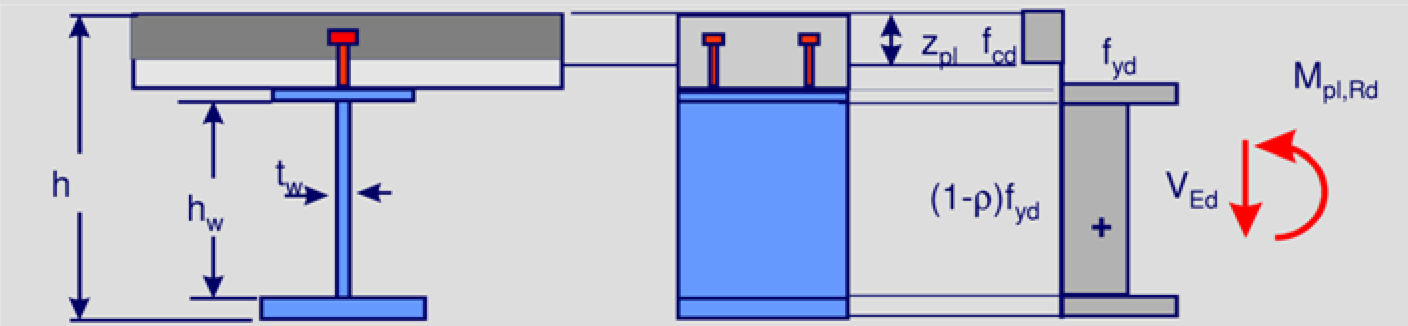




$$M_{Rd} = \beta_{pl} \cdot M_{pl,Rd}$$

[Source: Hanswille]

Resistance of class 1 and 2 sections



plastic resistance to vertical shear $V_{pl,Rd}$:

$$V_{Rd} = V_{pl,Rd} = \frac{f_{yd}\ A_w}{\sqrt{3}}$$

related slenderness of the web:

$$\bar{\lambda}_w = \sqrt{\frac{f_{yk}}{\sqrt{3}\ \tau_{cr}}}$$

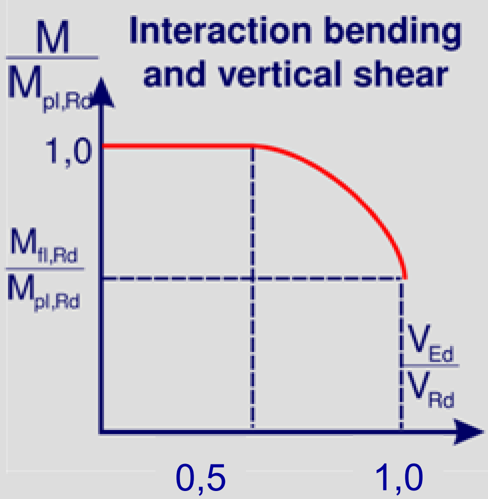
shear buckling resistance V_{Rd} for $\bar{\lambda}_w \geq 0{,}83$

$$V_{Rd} = \chi_w \frac{A_w\ t_w\ f_{yd}}{\sqrt{3}}$$

reduction factor $(1-\rho)$ for the yield strength of the web

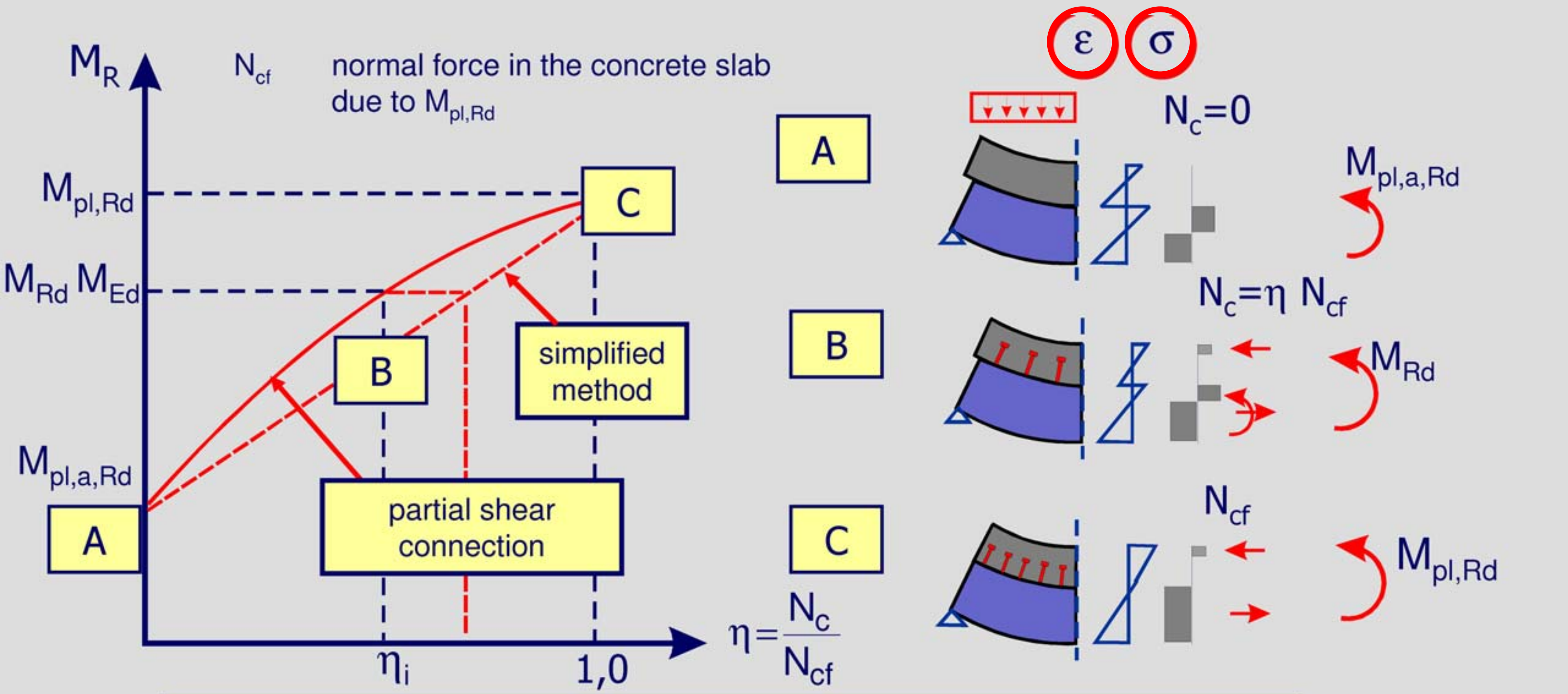
$$V_{Ed} \leq 0{,}5\ V_{Rd} \Rightarrow \rho = 0$$

$$V_{Ed} \geq 0{,}5\ V_{Rd} \Rightarrow \rho = \left(\frac{2V_{Ed}}{V_{Rd}} - 1\right)^2$$



[Source: Hanswille]

Resistance of class 1 and 2 sections - Full and partial shear connection



Simplified method:

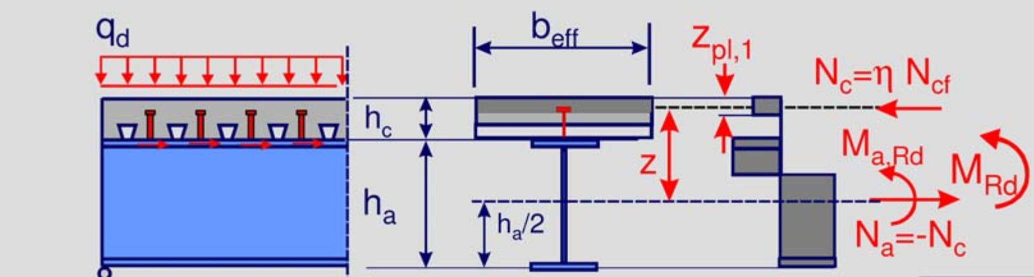
(Linear interpolation between points A and C)

$$M_{Rd} = M_{pla,Rd} + \left[M_{pl,Rd} - M_{pla,Rd}\right] \frac{N_c}{N_{cf}}$$

$$N_c = \frac{M_{Ed} - M_{pl,Rd}}{M_{pl,Rd} - M_{pla,Rd}}\ N_{cf}$$

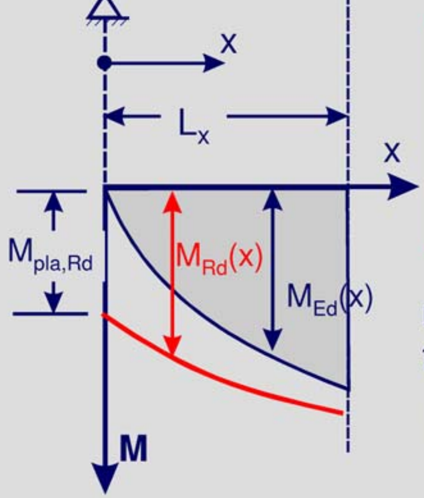
[Source: Hanswille]

Resistance of class 1 and 2 sections - Partial shear connection - general



degree of shear connection:

$$\eta = \frac{\bar{n}\, P_{Rd}}{N_{cf}}\, x$$



verification

$$\frac{M_{Ed}(x)}{M_{Rd}(x)} \le 1{,}0$$

bending resistance at the point x:

$$M_{Rd}(x) = M_{a,Rd} + \eta\, N_{cf}\, z$$

$$M_{Rd}(x) = M_{a,Rd} + \bar{n}\, P_{Rd}\, x\, z$$

normal force in the concrete slab at the point $x=L_x$:

$$N_c = \bar{n}\, L_x\, P_{Rd} = \eta\, N_{cf}$$

$$M_{a,Rd} = M_{pla,Rd}\, \frac{1}{\delta_w} \left[1 - \frac{N_a}{N_{pla,Rd}} \right] \le M_{pla,Rd}$$

$$\delta_w = 1 - 0{,}5\, A_w / A_a \le 0{,}5$$

$$z = \frac{h_a}{2} + h_c - \frac{z_{pl,1}}{2}$$

$$z_{pl,1} = \frac{N_c}{b_{eff}\;\; f_{cd}} \le h_c - h_p$$

$x=0$:
$M_{a,Rd} = M_{pla,Rd}$

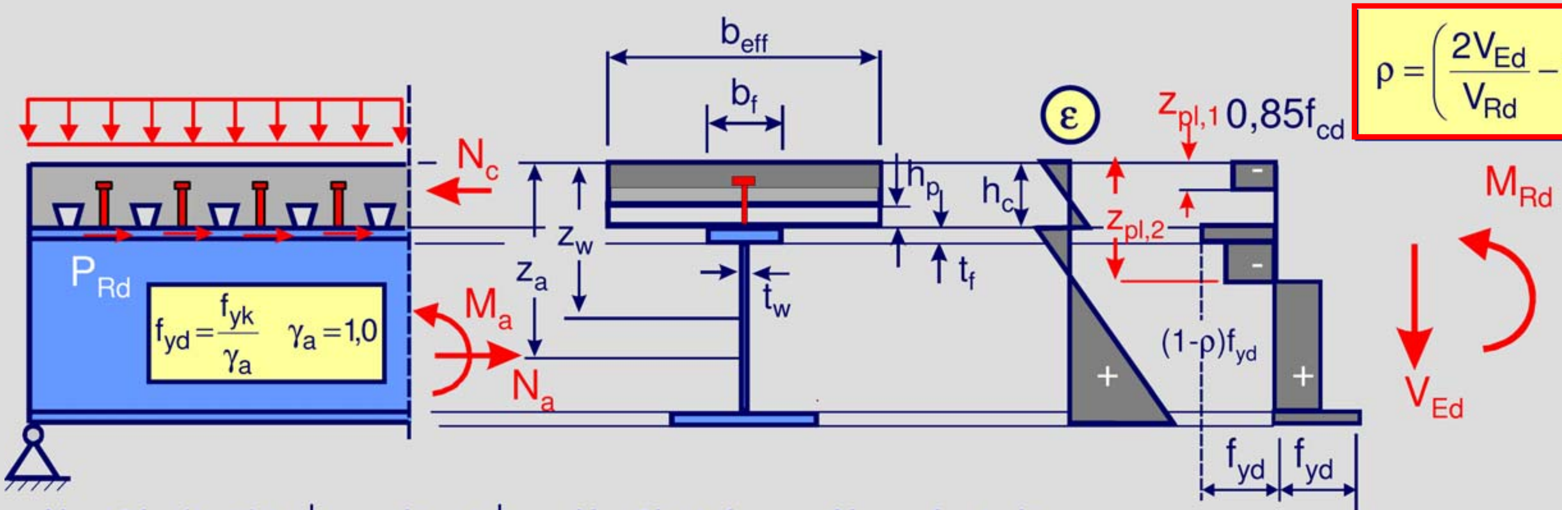
$\bar{n}$	number of connectors per unit length
P_{Rd}	**design resistance of studs**

[Source: Hanswille]

Resistance of class 1 and 2 sections

Partial shear connection – determination of moment resistance

$$\rho = \left(\frac{2V_{Ed}}{V_{Rd}} - 1 \right)^2$$



$N_w = 2 f_{yd} (1-\rho)\, t_w \left[z_{pl,2} - h_c - t_f \right]$ $\quad N_f = 2 b_f\, t_f\, f_{yd}$ $\quad N_{pl,w} = f_{yd}\, t_w\, h_w$

position of the plastic neutral axis in the concrete slab:

$$z_{pl,1} = \frac{\eta\, N_{cf}}{b_{eff}\; 0{,}85\, f_{cd}} = \frac{\sum P_{Rd}}{b_{eff}\; 0{,}85\, f_{cd}} \leq h_c - h_p$$

plastic neutral axis in the web of steel section:

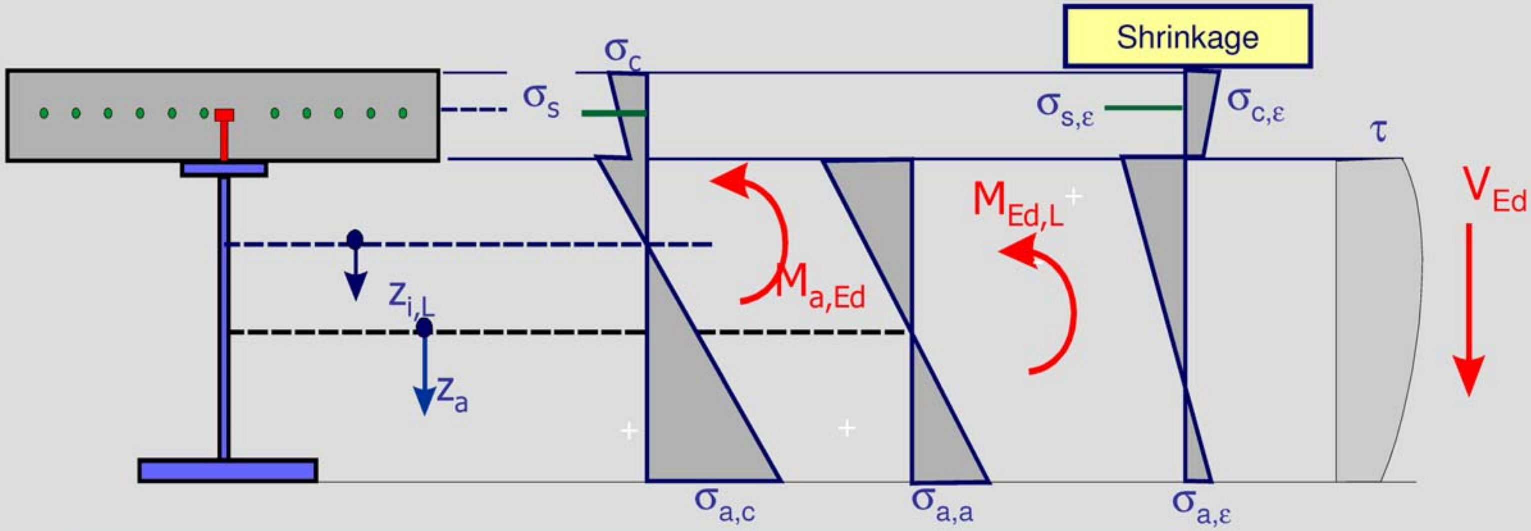
$$z_{pl,2} = h_c + t_f + \frac{N_{pla,Rd} - N_{pl,w}\,\rho - \eta\, N_{cf} - N_f}{2\, f_{yd}\,(1-\rho)\, t_w}$$

Bending resistance:

$$M_{Rd} = N_{pla,Rd}\left(z_a - \frac{z_{pl,1}}{2} \right) - N_{pl,w}\,\rho \left(z_w - \frac{z_{pl,1}}{2} \right) - N_f \left(h_c + \frac{t_f - z_{pl,1}}{2} \right) - N_w \left(\frac{z_{pl,2} + h_c + t_f - z_{pl,1}}{2} \right)$$

[Source: Hanswille]

Resistance of class 3 and 4 sections - class 3



Stress limits:

concrete: $\sigma_c \leq f_{cd}$ reinforcement: $\sigma_s \leq f_{sd}$

structural steel: $\sigma_a \leq f_{yd}$ prestressing steel: $\sigma_p \leq f_{p,0,1,k}/\gamma_p$

For related shear slenderness : $\bar{\lambda}_w \leq 0{,}83$

$$\left(\frac{\sigma_{ax}}{f_{yd}}\right)^2 + \left(\frac{\sqrt{3}\,\tau}{f_{yd}}\right)^2 \leq 1{,}0$$

$$\bar{\lambda}_w = \sqrt{\frac{f_{yk}/\sqrt{3}}{\tau_{cr}}}$$

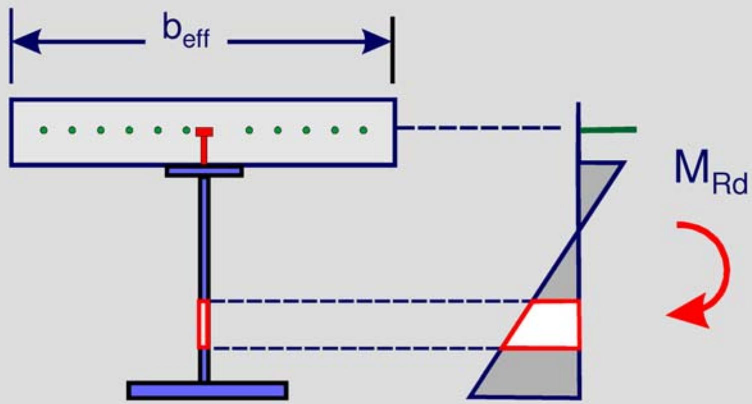
[Source: Hanswille]

Resistance of class 3 and 4 sections - class 4

Methods of verification

Method I:

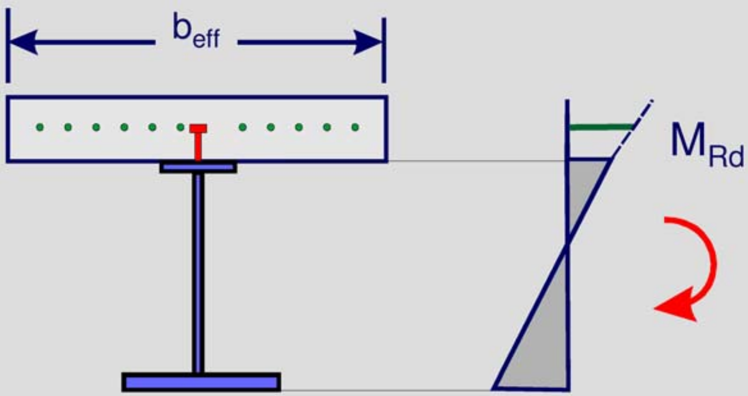
Effective cross-section



$$\sigma_{x,Ed} \leq \frac{M_{Ed}}{J_{st,eff}} z_{st,eff} \leq f_{yd}$$

Method II:

Reduced stresses

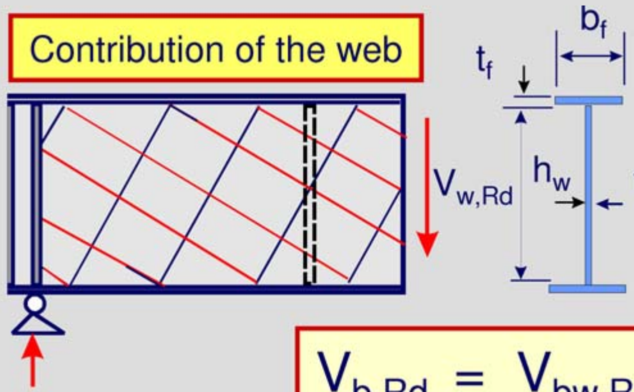


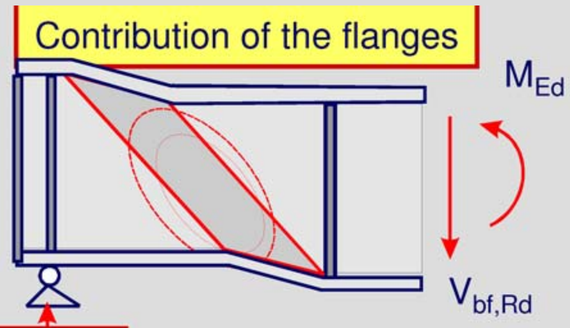
$$\sigma_{x,Ed} \leq \frac{M_{Ed}}{J_{st}} z_{st} \leq \sigma_{xP,Rd}$$

[Source: Hanswille]

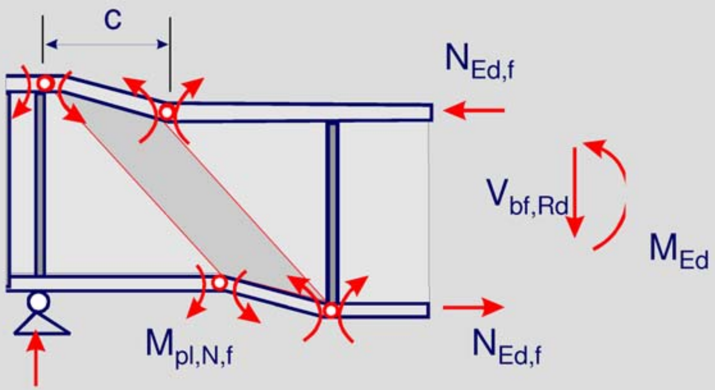
Resistance of class 3 and 4 sections

Resistance to vertical shear





$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd}$$



$$M_{f,Rd} = b_f\, t_f\, f_{yd}\left[(t_f + h_w\right]$$

Contribution of the flanges

$$V_{bf,Rd} = \frac{4 M_{pl,N,f}}{c} \qquad c = a\left[0{,}25 + \frac{1{,}6\, b_f\, t_f^2\, f_{yd}}{t_w\, h_w^2\, f_{yd}}\right]$$

$$M_{pl,N,f} = \frac{b_f\, t_f^2}{4} f_{yd,f}\left[1 - \left(\frac{N_{Ed,f}}{N_{pl,d,f}}\right)^2\right]$$

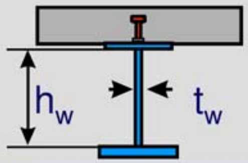
$$V_{bf,Rd} = \frac{b_f\, t_f^2\, f_{yd,f}}{c}\left[1 - \left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right]$$

[Source: Hanswille]

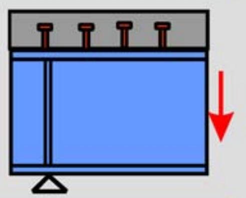
Resistance of class 3 and 4 sections

Resistance to vertical shear

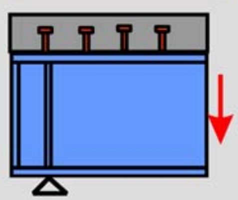
$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \le \frac{\eta f_{yw} h_w t}{\sqrt{3}\,\gamma_{Rd}}$$



non-rigid end post

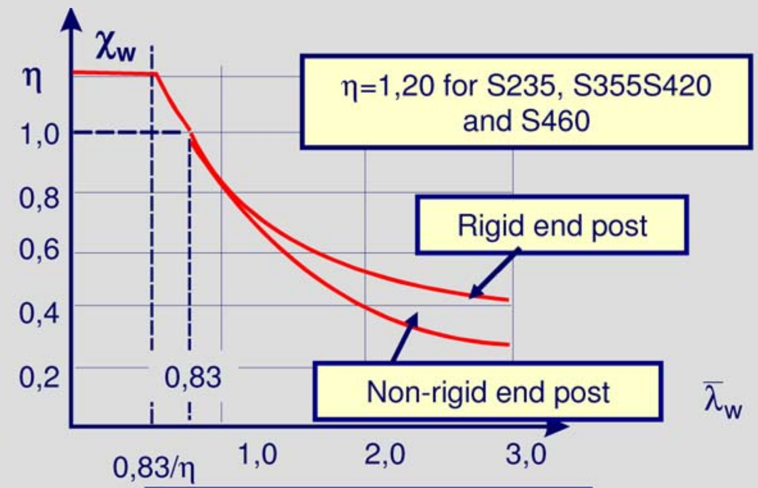


rigid end post



$$\bar{\lambda}_w = \sqrt{\frac{f_{yw}/\sqrt{3}}{\tau_{cr}}}$$

τ_{cr} – elastic critical shear buckling stress

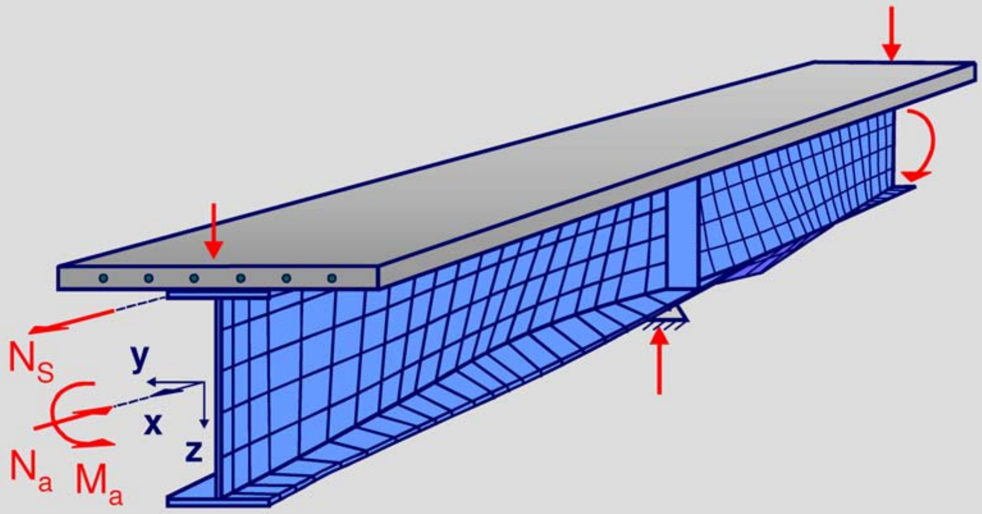


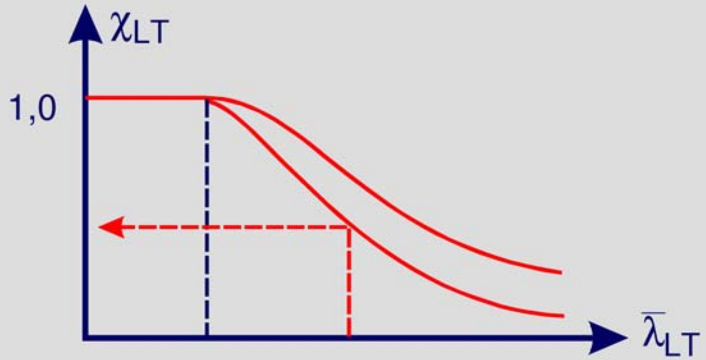
$$V_{bw,Rd} = \chi_w \cdot h_w t_w \frac{f_{yw}}{\sqrt{3}\,\gamma_{Rd}}$$

Reduction factor χ_w		
	rigid end post	Non-rigid end post
$\bar{\lambda}_w < 0{,}83/\eta$	η	η
$0{,}83/\eta \le \bar{\lambda}_w < 1{,}08$	$0{,}83/\bar{\lambda}_w$	$0{,}83/\bar{\lambda}_w$
$\bar{\lambda}_w \ge 1{,}08$	$1{,}37/(0{,}7+\bar{\lambda}_w)$	$0{,}83/\bar{\lambda}_w$

[Source: Hanswille]

Lateral torsional buckling





Cross-sections in class 1 and 2:

$$M_{Rd} = M_{pl,Rd}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{M_{pl,Rk}}{M_{cr}}}$$

$$M_{Ed} \leq \chi_{LT} M_{Rd}$$

Cross-sections in class 3 and 4:

$$M_{Rd} = M_{el,Rd}$$

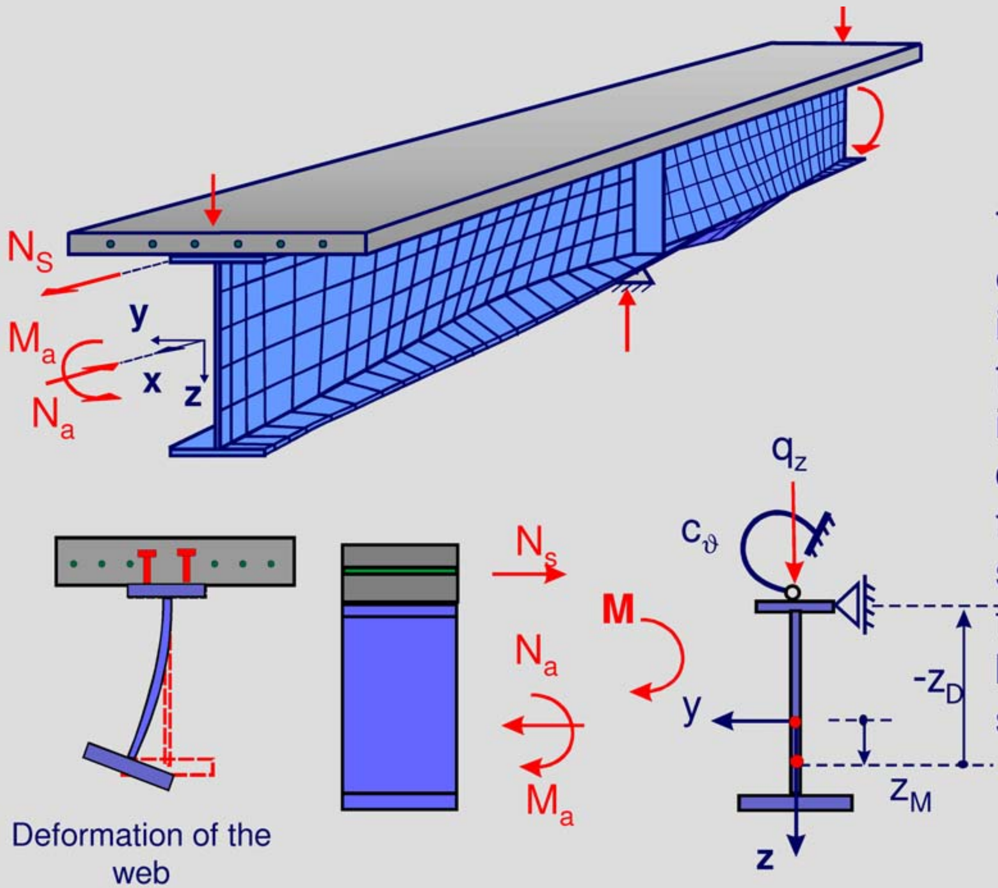
$$\bar{\lambda}_{LT} = \sqrt{\frac{M_{el,Rk}}{M_{cr}}} = \sqrt{\frac{f_{yk}}{\sigma_{cr}}}$$

$$M_{Ed} \leq \chi_{LT} M_{el,Rd}$$

or $\sigma_{Ed} \leq \chi_{LT} f_{yd}$

[Source: Hanswille]

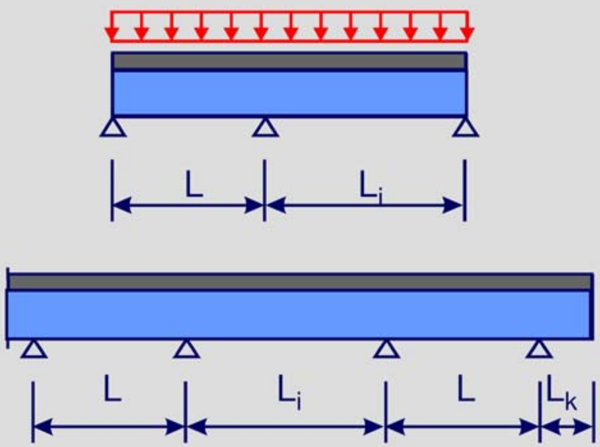
Lateral torsional buckling – elastic critical bending moment



The determination of the elastic critical bending moment is based on the inverted U-frame model. The model takes into account the lateral displacement of the bottom flange causing bending of the steel web and the rotation of the top flange that is resisted by bending of the concrete slab.

[Source: Hanswille]

Lateral torsional buckling – without direct calculation



A direct verification for lateral torsional buckling is not required where special limitations regarding the ratios of span length's and regarding the percentage of the variable actions are fulfilled. The background of simplified method is based on the fact, that the related slenderness of these beams is smaller than 0,4.

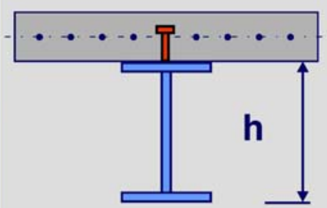
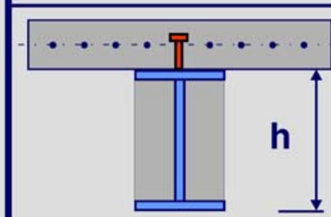
<u>limitation of span ratios:</u>

$0{,}8 \leq l / l_i \leq 1{,}25$

$L_k / L \leq 0{,}15$

<u>limitation regarding variable actions:</u>

$$\frac{\gamma_G G_K}{\gamma_G G_k + \gamma_Q Q_k} \geq 0{,}4$$

maximum depth of the cross-section				
	section	S 235	S 355	S460
 h	IPE	600	400	270
	HE-Profiles	800	650	500
 h	IPE	600	600	400
	HE-Profiles	1000	800	650

[Source: Hanswille]

Part 5:

SHEAR CONNECTION

- **Longitudinal shear forces**
  - Determination of longitudinal shear forces
  - Full and partial shear connection
  - Requirements for shear connectors

- **Headed studs**
  - Head studs as shear connector
  - Horizontally lying studs
  - Headed studs used with profiled steel sheeting

- **Longitudinal shear forces in concrete slab**

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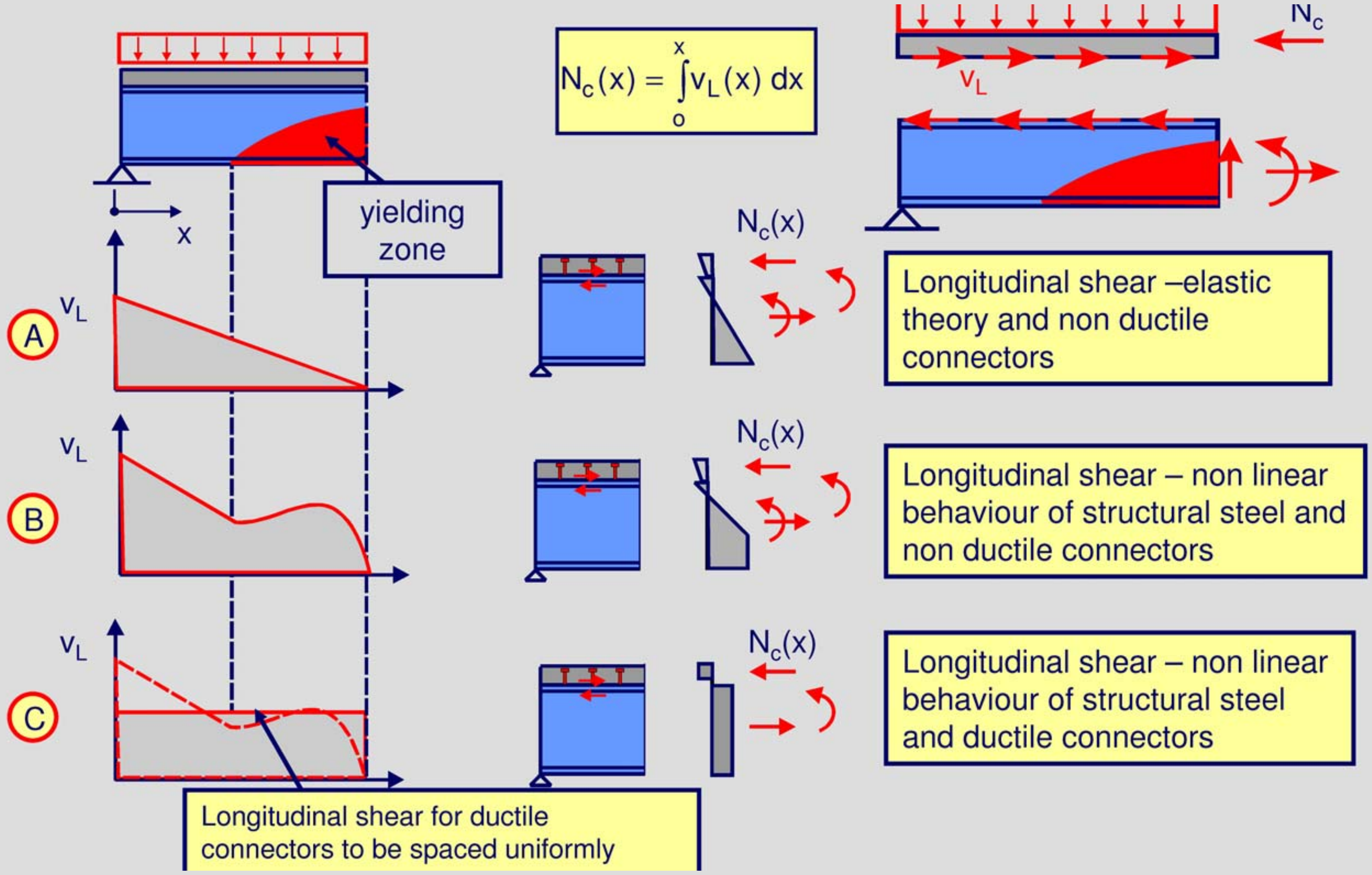
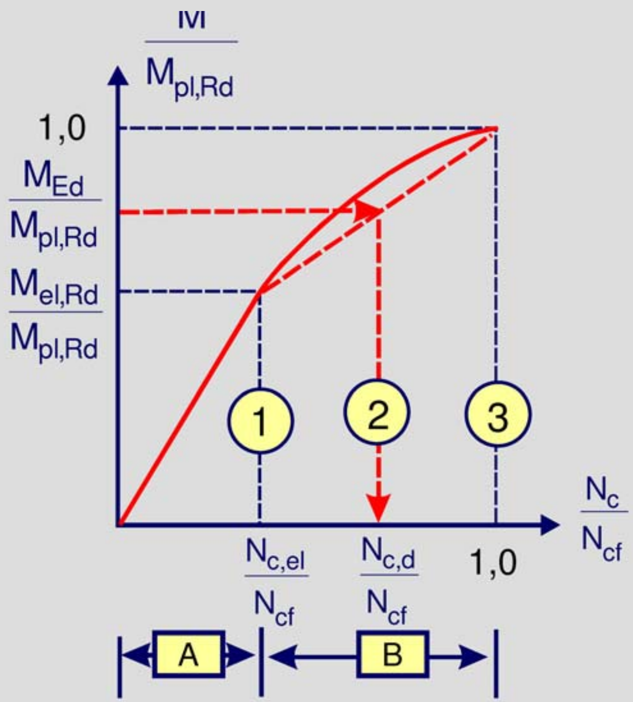
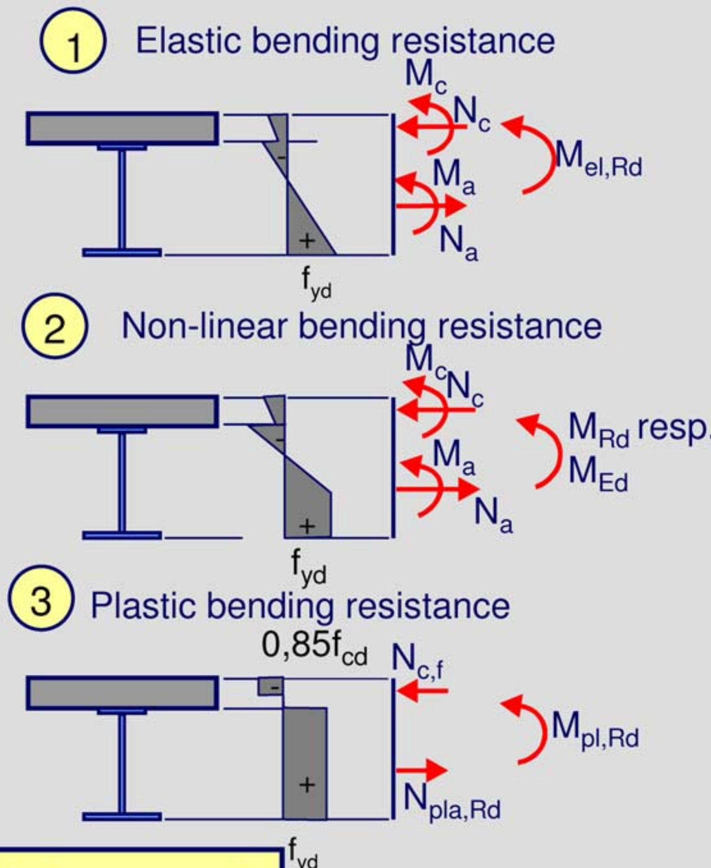
Longitudinal shear forces



[Source: Hanswille]

Determination of longitudinal shear forces - by simplified method for N_c





Region A:
elastic behaviour

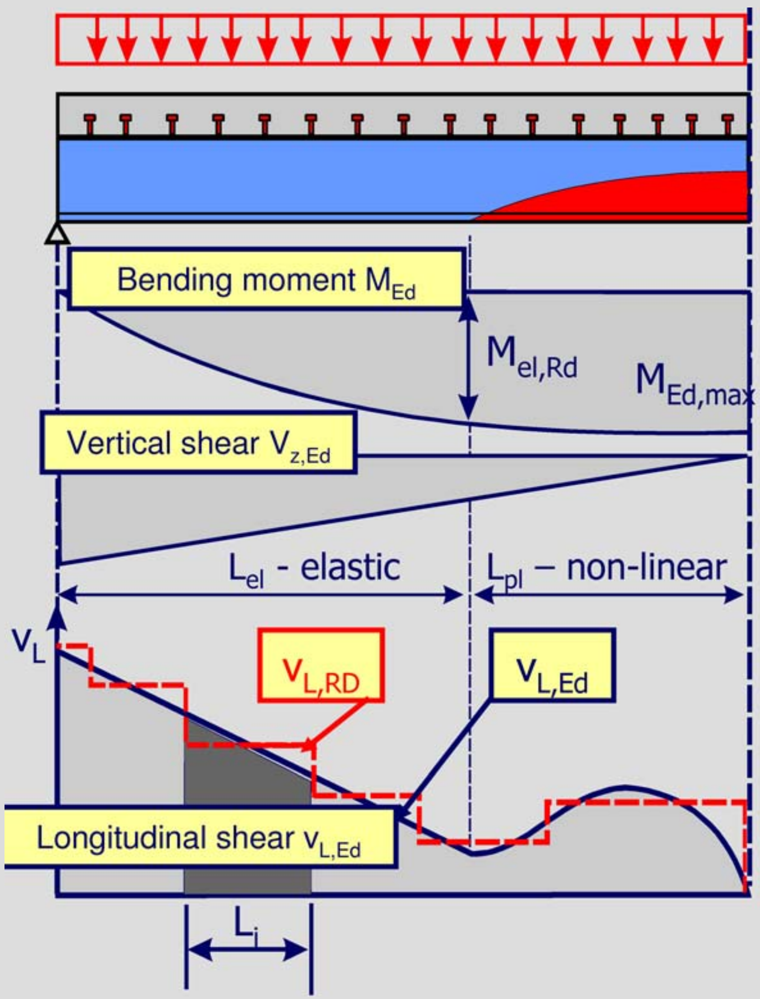
$$N_c = \sum_L M_L \frac{A_{c,L}\, z_{ic,L}}{J_{i,L}}$$

Region B:
inelastic behaviour

$$N_{c,d} = N_{c,el} + \frac{M_{Ed} - M_{el,Rd}}{M_{pl,Rd} - M_{el,Rd}} \left[N_{c,f} - N_{c,el}\right]$$

[Source: Hanswille]

Determination of longitudinal shear forces - general



Elastic region:

$$\frac{V_{L,Ed}}{V_{L,Rd}} \leq 1{,}0 \qquad \frac{v_{L,Ed}}{v_{L,Rd}} \leq 1{,}1 \qquad V_{L,Ed} = \int_{L_i} v_{L,Ed}\, dx$$

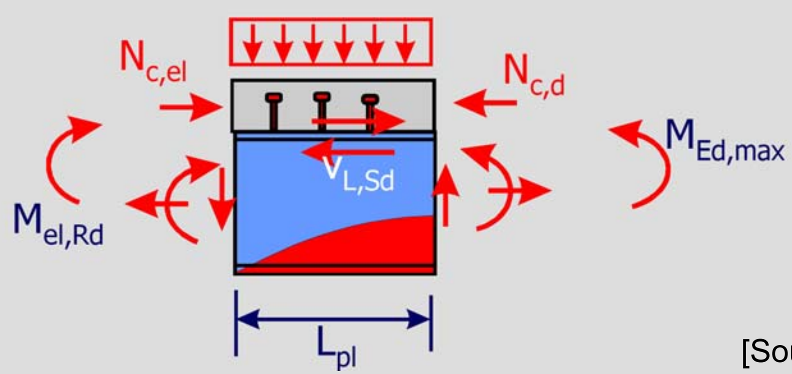
$$v_{L,Ed} = \sum_{L} V_{z,Ed,L} \frac{(A_{c,L} \cdot z_{ic,L}) + (A_s \cdot z_{is,L})}{J_{i,L}}$$

$v_{L,Ed}$ – longitudinal shear per unit length

$V_{L,Ed}$ – total longitudinal shear within the length L_i

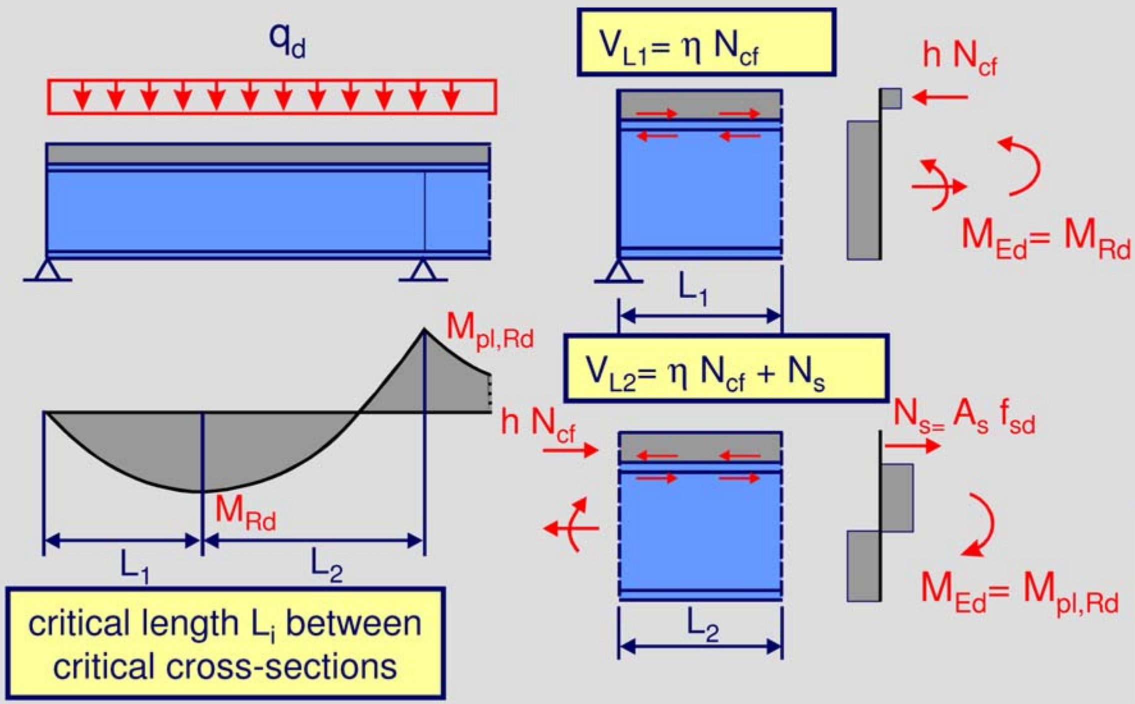
Non-linear region:

$$v_{L,Ed} = \frac{N_{cd} - N_{c,el}}{L_{pl}} \leq v_{L,Rd}$$



[Source: Hanswille]

Partial shear connection – determination of longitudinal shear forces

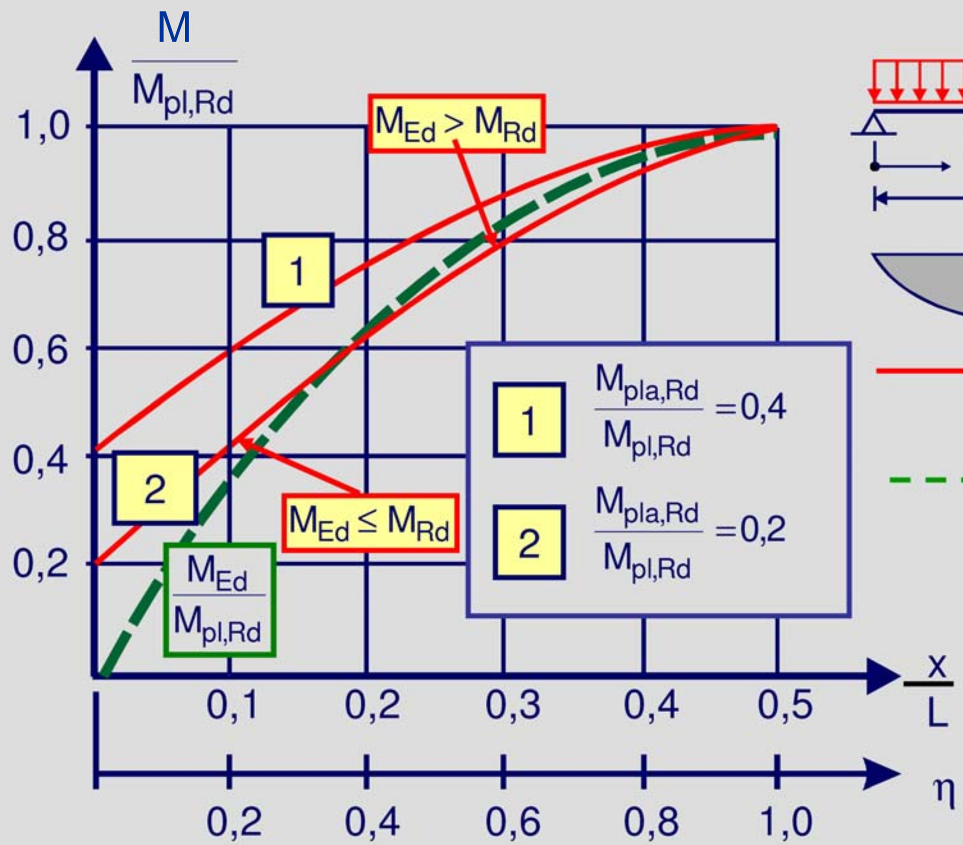


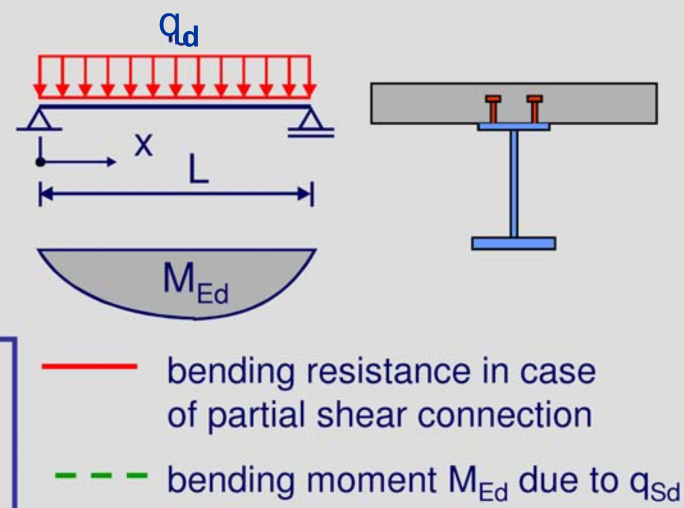
hogging bending region: partial shear connection is not allowed in this regions

regions with partial shear connection: The resistance to bending is limited by the resistance to longitudinal shear in the interface between steel and concrete. Where the condition $M_{pla,Rd} / M_{pl,Rd} > 0,4$ is not fulfilled additional checks at intermediate critical cross-sections should be made.

[Source: Hanswille]

Requirements for shear connection – uniformly distribution





where $M_{pla,Rd}/ M_{pl,Rd}$ is smaller than 0,4, the resistance moment can be smaller the design bending moment in some regions of the beam.

In case of partial shear connection all critical cross-sections must be in Class1 or 2, the degree of shear connection η must fulfill the limits in EC4 and the following condition must be satisfied:

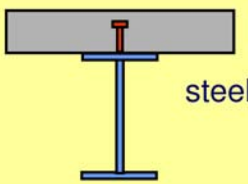
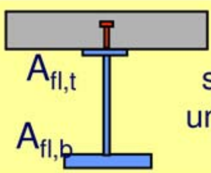
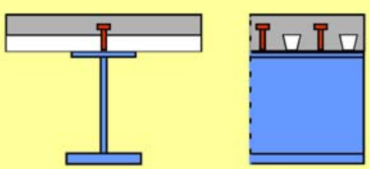
$$\frac{M_{pla,Rd}}{M_{pl,Rd}} \geq 0,4$$

[Source: Hanswille]

Requirements for shear connection – minimum degree

L_e – effective length [m]

The effective length is given by the distance between the points of zero bending moment.

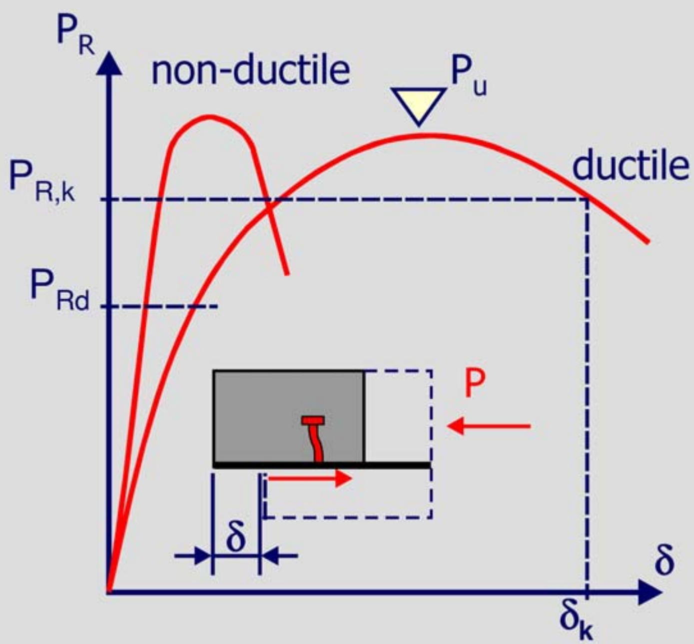
1	 steel sections with equal flanges	$L \leq 25m$	$\eta_{min} \geq 1 - \frac{355}{f_{yk}}(0{,}75 - 0{,}03 \times L_e) \geq 0{,}4$
		$L > 25m$	$\eta_{min} = 1$
2	 steel sections with unequal flanges with $A_{fl,b} \leq 3\ A_{fl,t}$	$L \leq 20m$	$\eta_{min} \geq 1 - \frac{355}{f_{yk}}(0{,}30 - 0{,}015 \times L_e) \geq 0{,}4$
		$L > 20m$	$\eta_{min} = 1$
3		$L \leq 25m$	$\eta_{min} \geq 1 - \frac{355}{f_{yk}}(1{,}00 - 0{,}04 \times L_e) \geq 0{,}4$
		$L > 25m$	$\eta_{min} = 1$

The minimum shear degree according to line 3 apply for steel sections with equal flanges, studs with a diameter of 19 mm and a length not smaller tan 76 mm and one stud per rib of the sheeting and for sheeting with $h_p \leq$ 60mm and $b_o/h_p \geq 2$.

[Source: Hanswille]

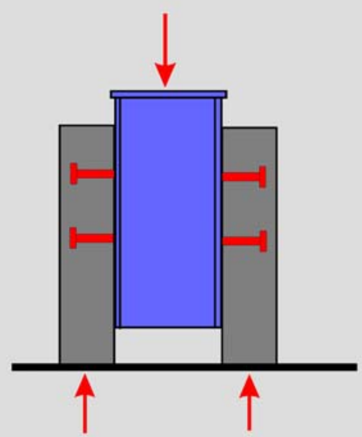
Requirements for shear connection – ductility

Shear connectors may be classified as ductile if the characteristic slip δ_k is at least 6 mm.



P_{Rd} – design value of shear resistance

P_{Rk} - characteristic value of shear resistance



[Source: Hanswille]

- **Longitudinal shear forces**
  - Determination of longitudinal shear forces
  - Full and partial shear connection
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  - Head studs as shear connector
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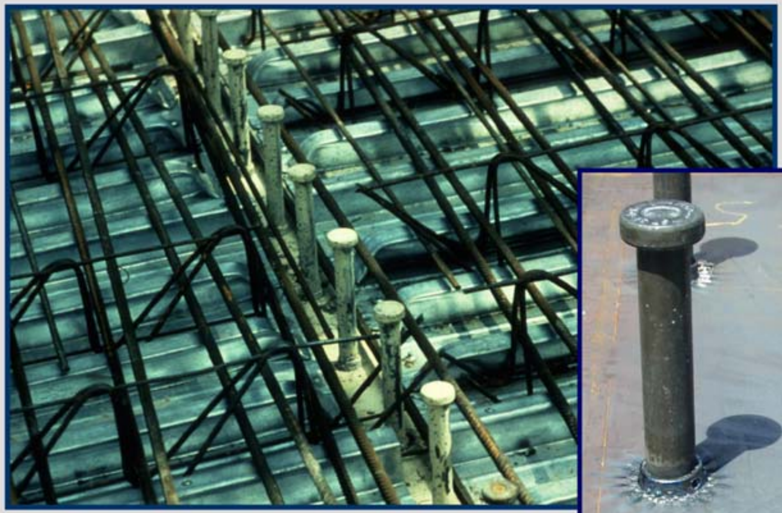
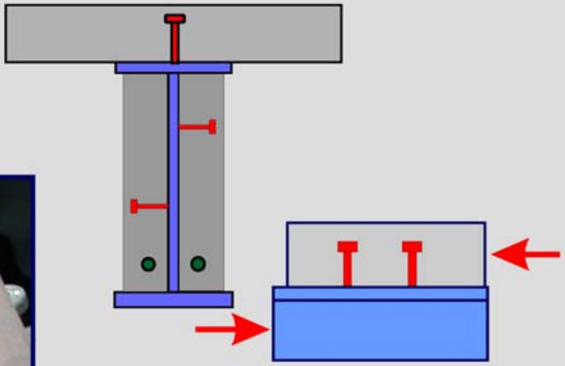
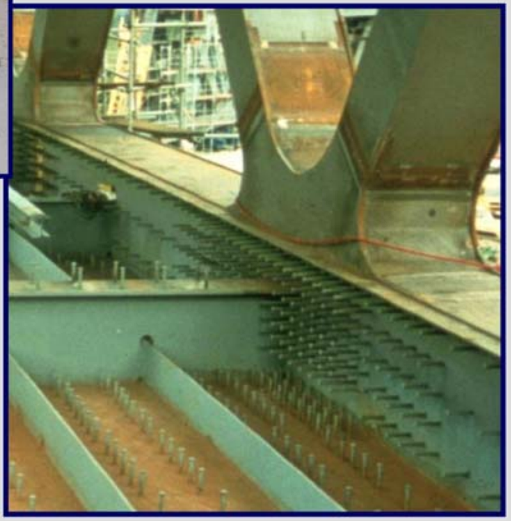
- **Longitudinal shear forces in concrete slab**

Headed studs





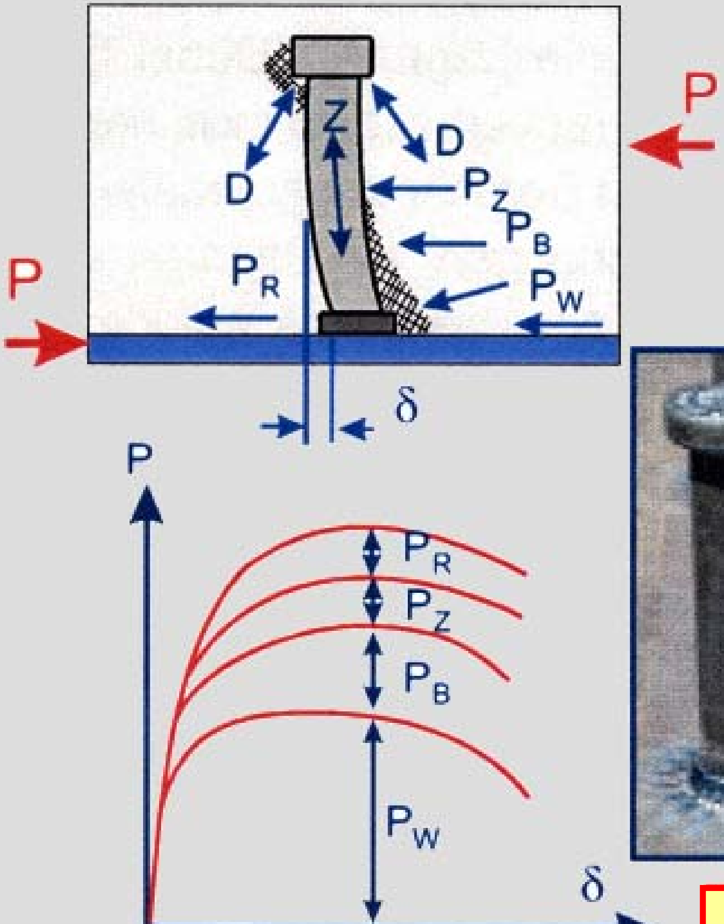


EN ISO 13918: Welding –Studs and ceramic ferrules for arc stud welding (1998)

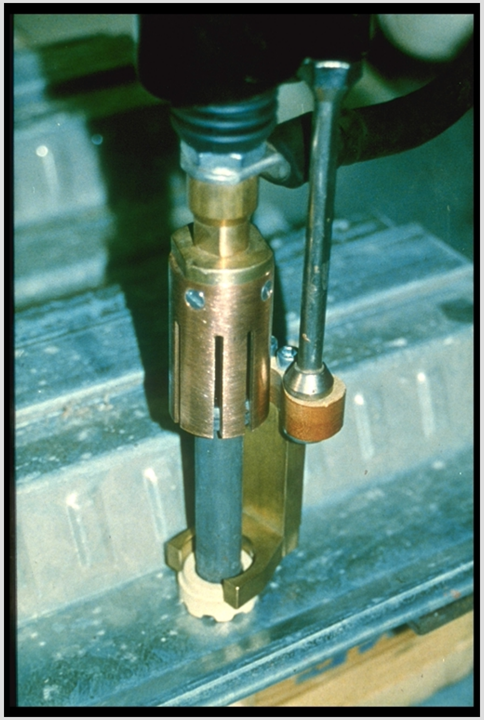
EN ISO 14555: Welding –Arc stud welding of metallic materials(1998)

[Source: Hanswille]

Headed studs – typical load-slip behaviour

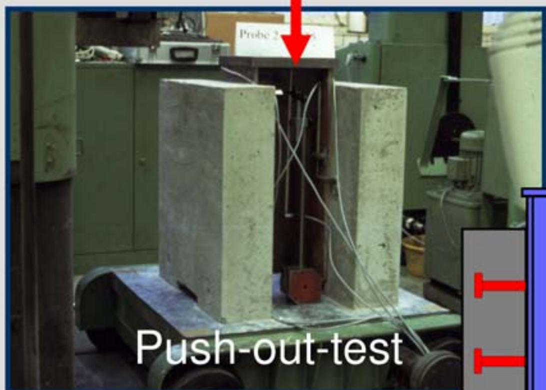


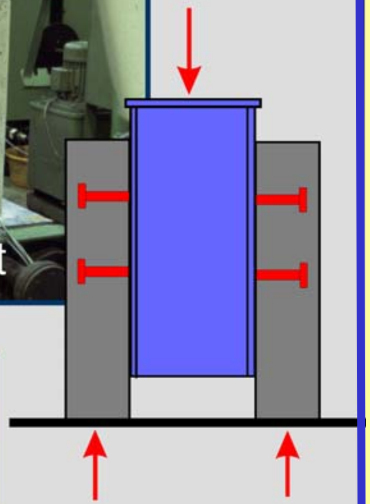
flash



P_w …	flash
P_Z …	stud inclination
P_B …	stud bending
P_R …	friction

Headed studs – design shear resistance



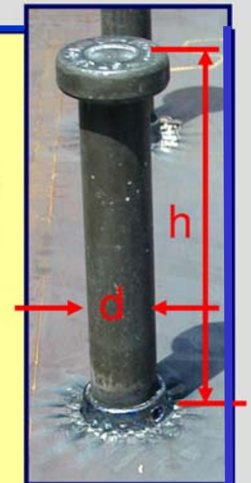


Concrete failure:

$$P_{Rd,1} = 0.29\ \alpha\ d^2 \sqrt{f_{ck}\ E_{cm}}\ \frac{1}{\gamma_V}$$

Shank failure of the stud:

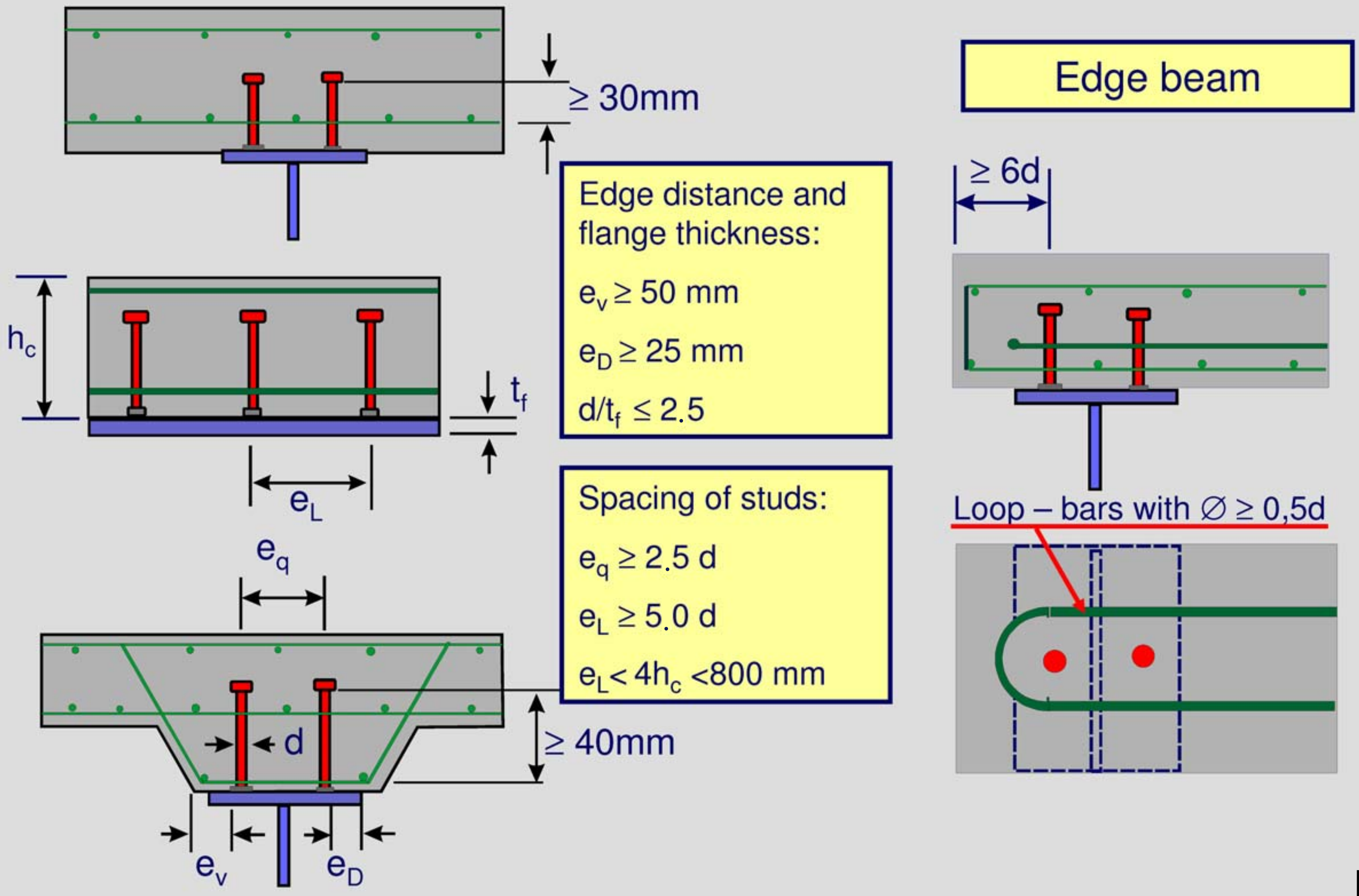
$$P_{Rd,2} = 0.8 \cdot f_u \left(\frac{\pi\, d^2}{4}\right) \frac{1}{\gamma_V}$$



d	diameter of stud shank $16 \le d \le 25$mm	
f_u	specified ultimate tensile strength of the stud material $f_u \le 500$ N/mm²	
f_{ck}	cylinder strength of concrete	
E_{cm}	secant modulus of elasticity of concrete	
a	=0.2 [(h/d)+1]	for $3 \le h/d \le 4$
	=1.0	for $h/d > 4$
γ_V	=**1.5**	partial safety factor concrete failure
	=1.25	partial safety factor steel failure

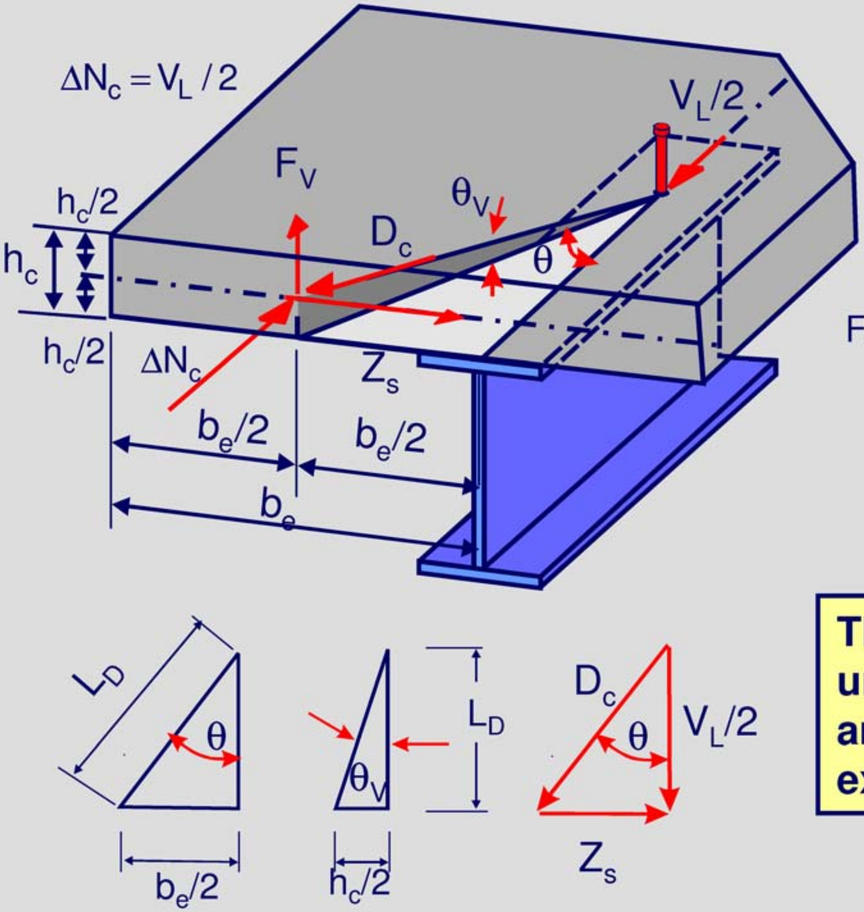
[Source: EC4-1 & Hanswille]

Headed studs – detailing

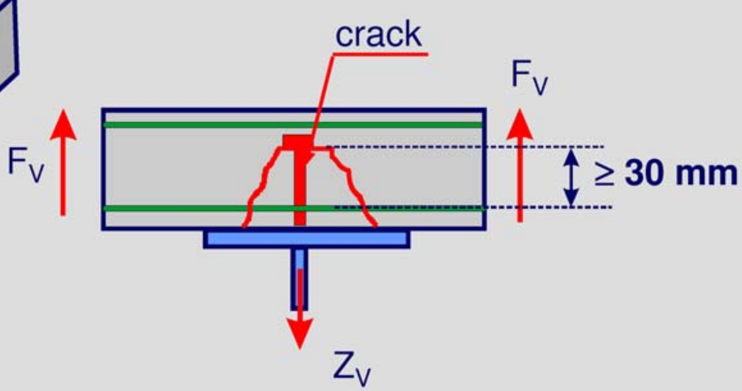


[Source: Hanswille]

Headed studs – uplift forces



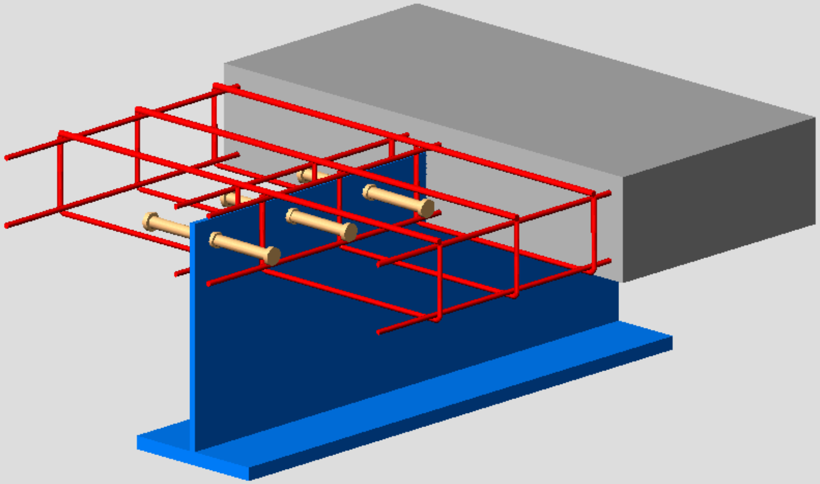
$$Z_V = V_L \frac{h_c}{b_e} \tan\theta$$

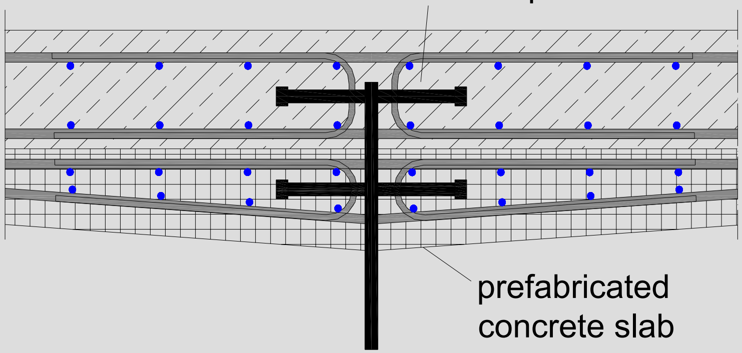


The clear distance between the underside of the head of the stud and the bottom reinforcement should extend not less than 30 mm.

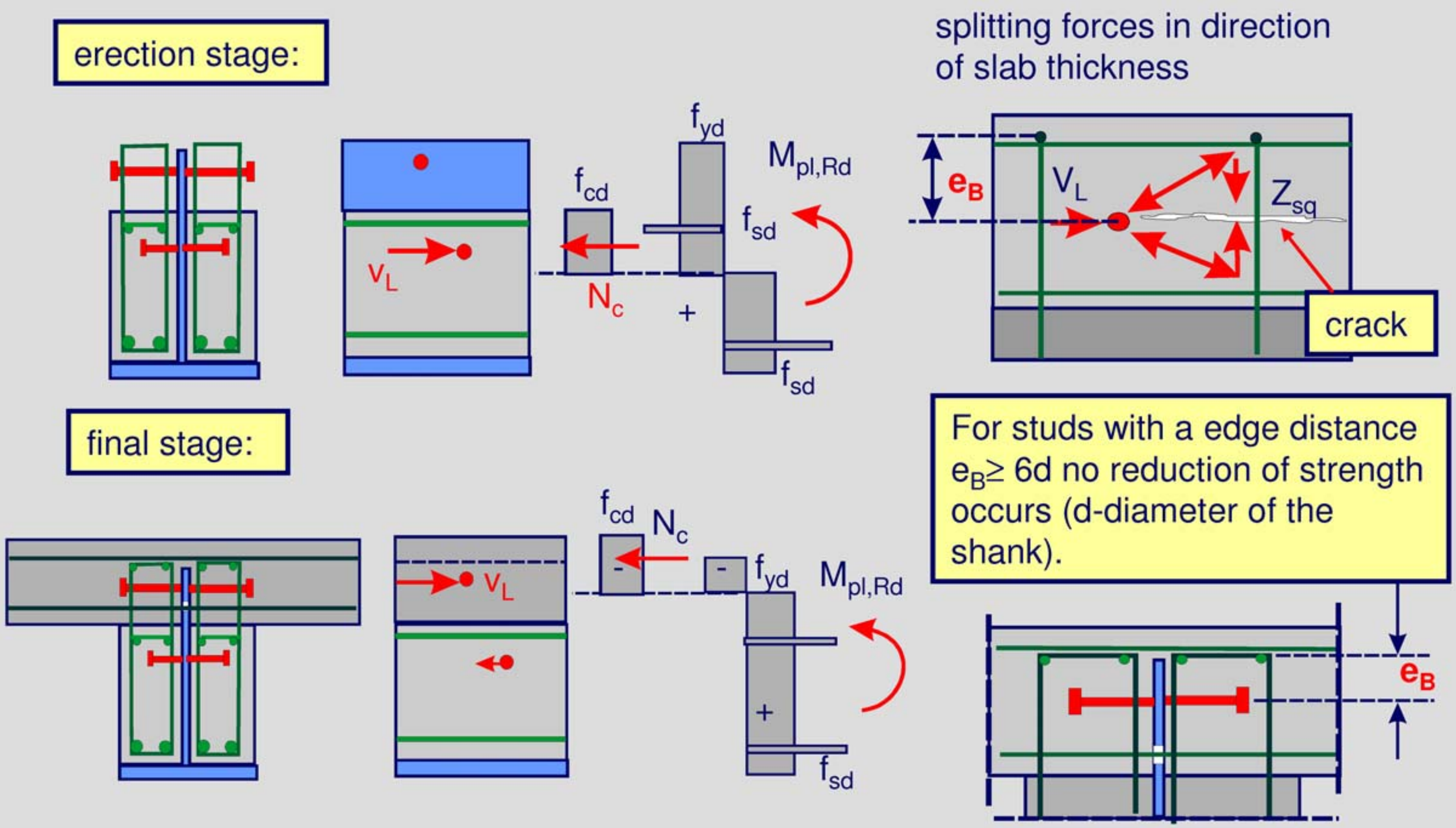
[Source: Hanswille]

Horizontally lying studs – examples



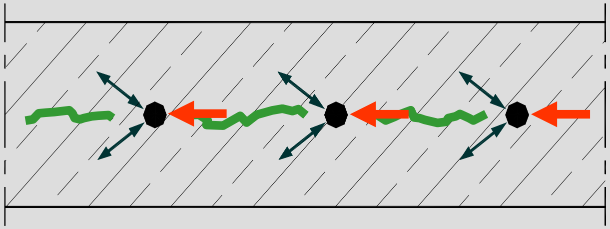


Horizontally lying studs – examples

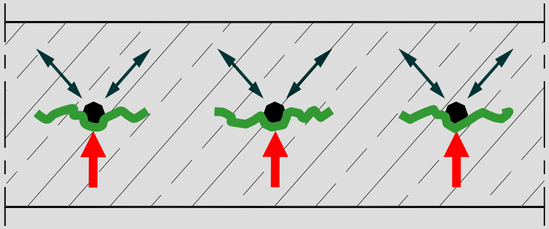


[Source: Hanswille]

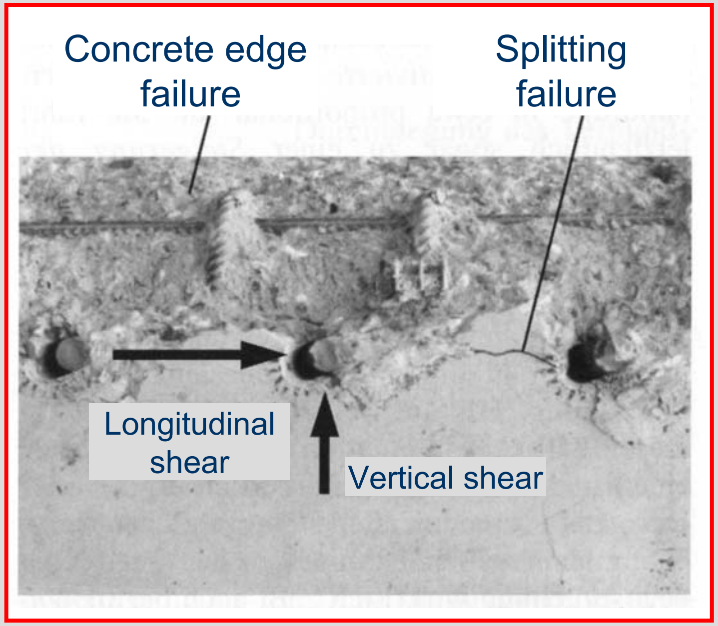
Horizontally lying studs – failure modes and position

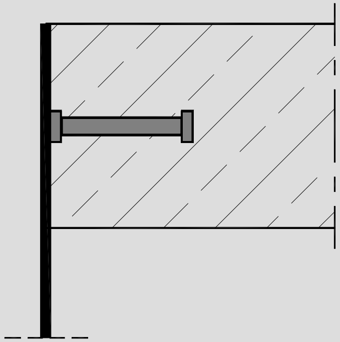


Longitudinal shear
due to beam bending

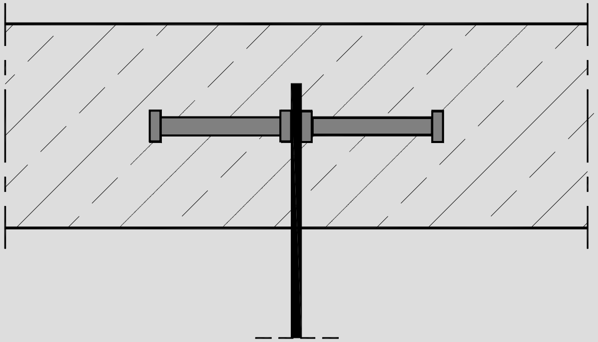


Vertical shear
due to vertical beam support



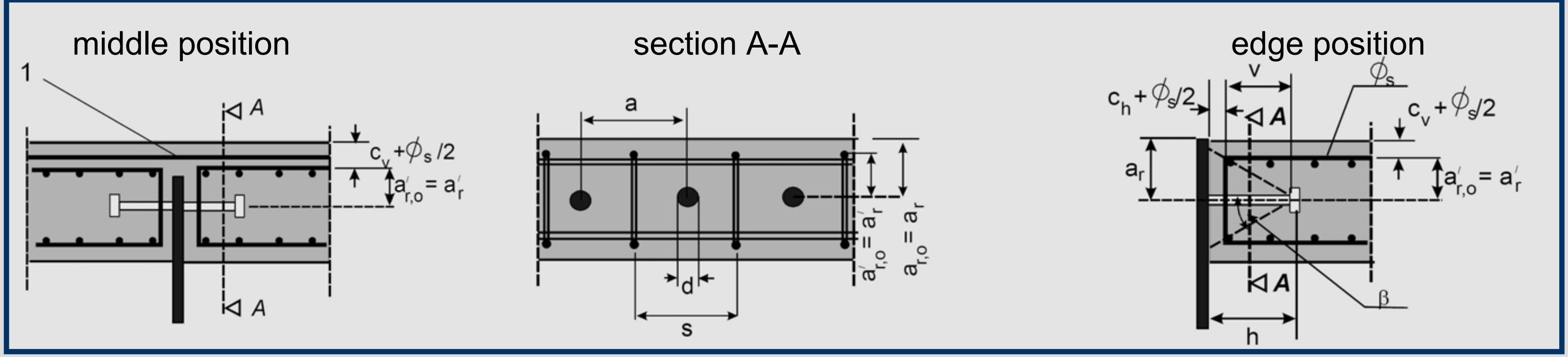


Edge position



Middle position

Horizontally lying studs – load resistance for longitudinal shear



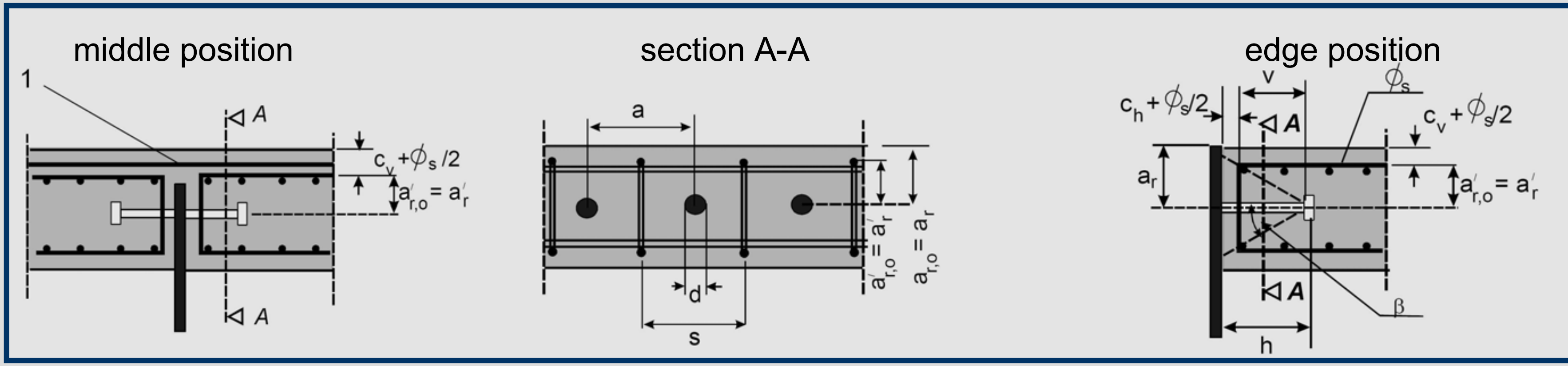
$$P_{Rd,L} = \frac{1.4\, k_v \left(f_{ck}\, d\, a_r'\right)^{0.4} (a/s)^{0.3}}{\gamma_v}$$

stirrups $T_d = 0.3\, P_{Rd,L}$

a'_r	effective edge distance	d … diameter of the stud shank	$19 \le d \le 25$ mm
	$a'_r = a_r - c_v - \varnothing_s/2 \ge 50$ mm	h … overall height of the stud	$h/d \ge 4$
k_v	factor for position of shear connection	s … spacing of stirrups	$a/2 \le s \le a$
	$k_v = 1$ edge position		$s/a'_r \le 3$
	$k_v = 1.4$ middle position	$\varnothing_s$… diameter of stirrups	$\varnothing_s \ge 8$ mm
γ_v	partial factor 1.25	$\varnothing_l$… diameter of longitudinal reinforment	$\varnothing_l \ge 10$ mm

[Source: EN1994-2]

Horizontally lying studs – load resistance for vertical shear



$$P_{Rd,V} = \frac{0.012\,(f_{ck}\,\varnothing_l)^{0.5}\,(d\,a/s)^{0.4}\,(\varnothing_s)^{0.3}\,(a'_{r,o})^{0.7}\,k_v}{\gamma_v}$$

a … spacing of studs
$110 \le a \le 440$ mm

h … overall height of the stud
$h \ge 100$ mm

$\varnothing_s$… diameter of stirrups
$\varnothing_s \ge 12$ mm

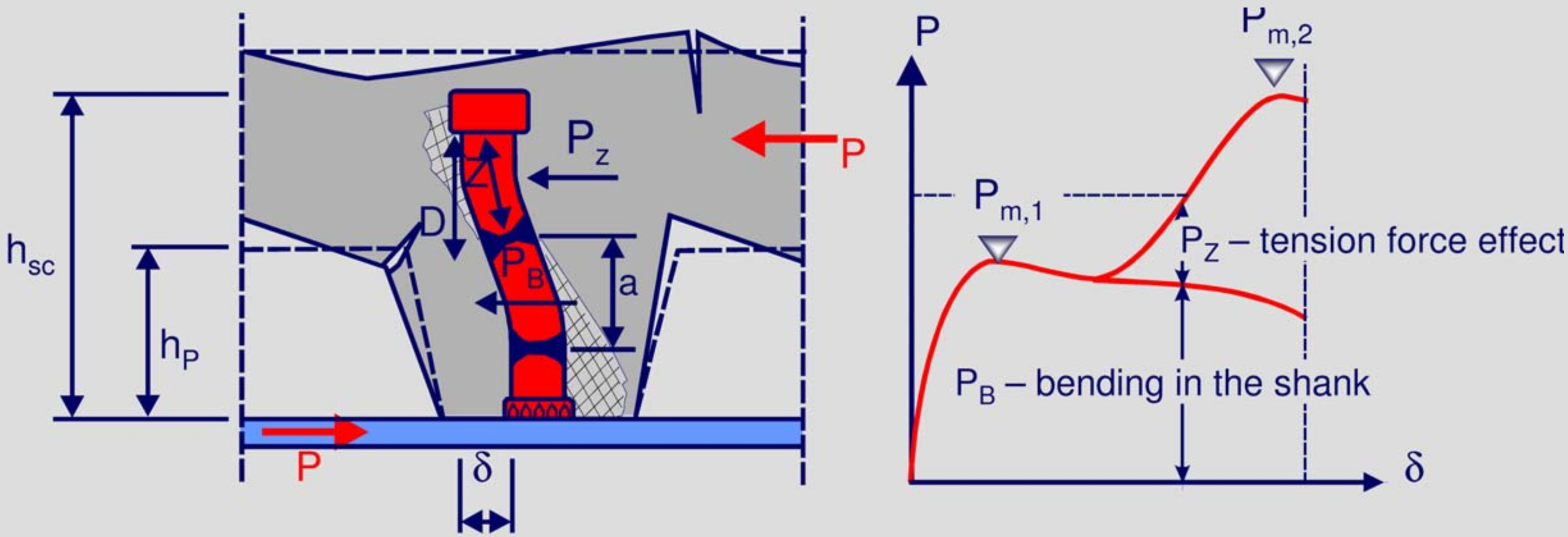
$\varnothing_l$… diameter of longitudinal reinforment
$\varnothing_l \ge 16$ mm

Interaction:

$$\left(\frac{F_{d,L}}{P_{Rd,L}}\right)^{1.2} + \left(\frac{F_{d,V}}{P_{Rd,V}}\right)^{1.2} \le 1$$

[Source: EN1994-2]

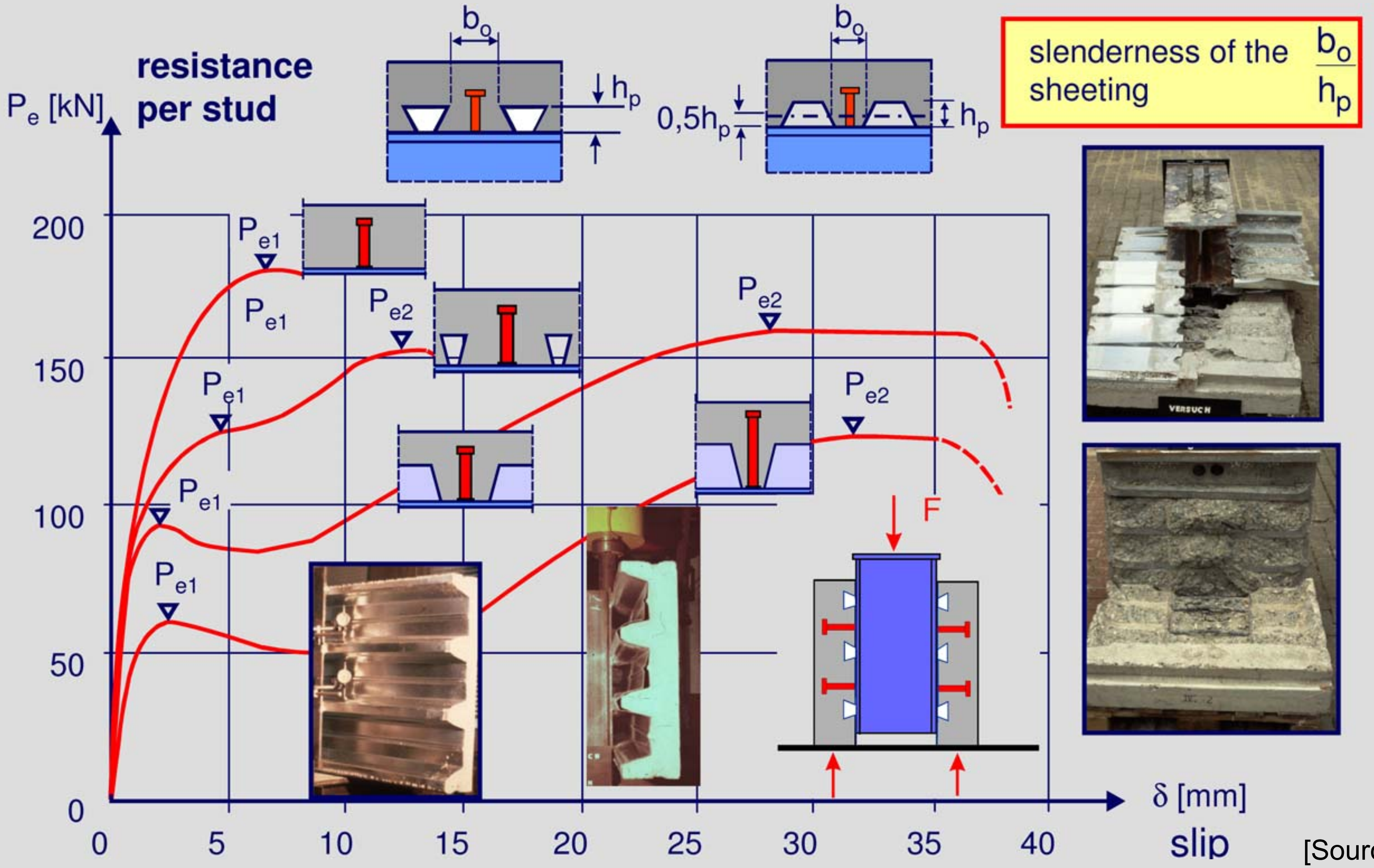
Headed studs used with profiled steel sheeting



The resistance is significantly influenced by the geometry of the sheeting. In the stud the shear force causes bending moments. The first ultimate Load $P_{m,1}$ is reached when in the shank of studs two plastic hinges occur. This is only possible, if the nominal height of the connector extends more than 2 d above the top of steel sheeting, where d is the shank diameter of the stud. In case of slender ribs a second load maximum $P_{m,2}$ is reached. In this stage in the stud significant tension forces occur and the inclination of the studs causes an additional shear resistance. This load level is not taken into account, because significant horizontal reinforcement must be provided to prevent uplift. Otherwise rib shear failure can occur.

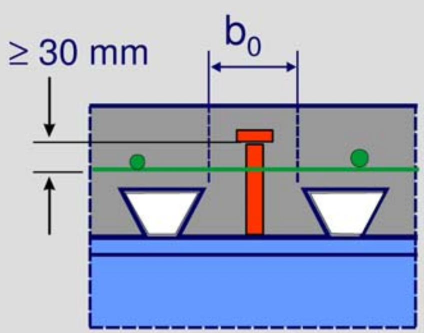
[Source: Hanswille]

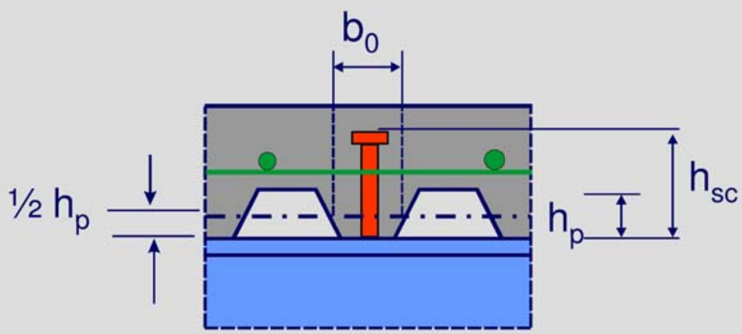
Headed studs used with profiled steel sheeting – load-slip behaviour

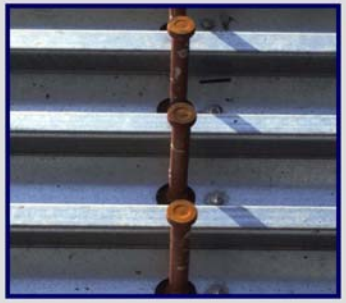


[Source: Hanswille]

Headed studs used with profiled steel sheeting – load resistance







reduction factor $k_{t,max}$			
number of studs per rib	thickness of the sheeting [mm]	studs welded through profiled steel d= 19mm	profiled steel sheeting with holes d=19 und d=22mm
n_r=1	≤1,0 mm	0.85	0.75
	>1,0 mm	1.0	0.75
n_r=2	≤1,0 mm	0.70	0.60
	>1,0 mm	0.80	0.60

$P_{Rd,o}$- resistance of the stud in a solid slab with f_u= 450 N/mm²

Design resistance :

$$P_{Rd} = P_{Rd,o}\, k_t$$

$$k_t = \frac{0.7}{\sqrt{n_r}} \frac{b_o}{h_p} \left[\frac{h_{sc}}{h_p} - 1 \right] \leq k_{t,max}$$

$h_p \leq 85$ mm $h - h_p \geq 2d$

$b_o \geq h_p$ $b_o \geq 50$ mm

[Source: Hanswille]

- **Longitudinal shear forces**
  - Determination of longitudinal shear forces
  - Full and partial shear connection
  - Requirements for shear connectors
- **Headed studs**
  - Head studs as shear connector
  - Horizontally lying studs
  - Headed studs used with profiled steel sheeting
- **Longitudinal shear forces in concrete slab**

Longitudinal shear forces in concrete slab - determination

Slab in compression

$$V_{L,Ed} = \Delta N_c$$

Longitudinal shear per unit length:

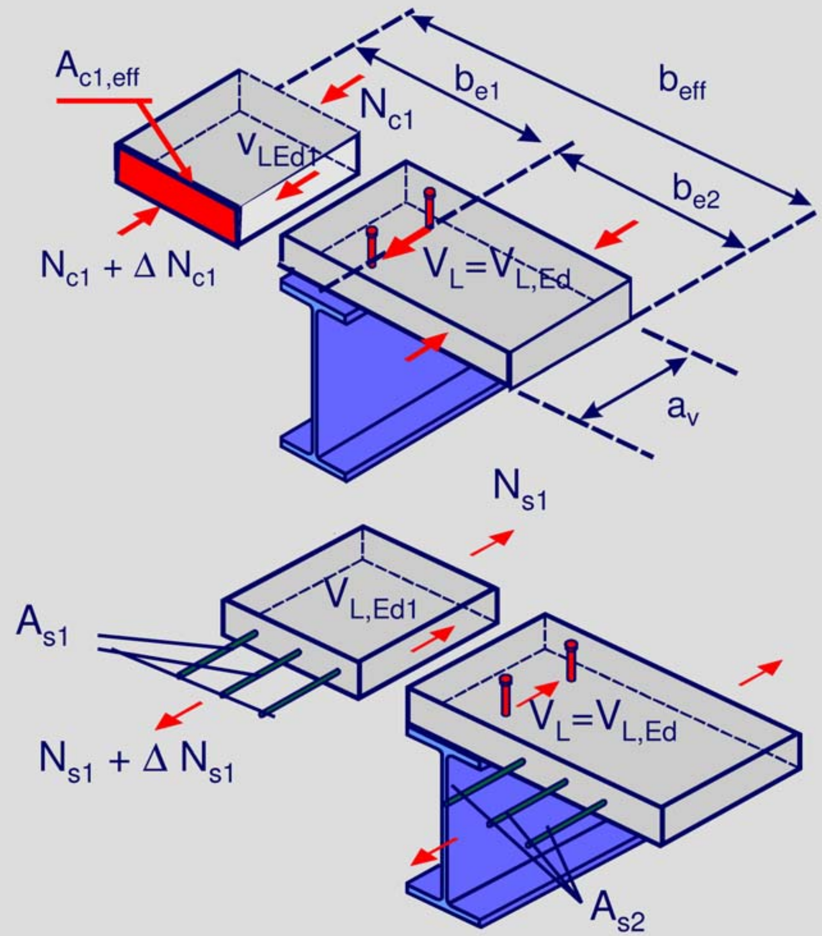
$$v_{L,Ed,1} = \frac{\Delta N_{c1}}{a_v} = \frac{V_{L,Ed}}{a_v} \frac{A_{c1,eff}}{A_{c,eff}}$$

Slab in tension

$$V_{L,Ed} = \Delta N_s$$

Longitudinal shear per unit length:

$$v_{L,Ed,1} = \frac{\Delta N_{s1}}{a_v} = \frac{V_{L,Ed}}{a_v} \frac{A_{s1}}{A_{s1} + A_{s2}}$$



[Source: Hanswille]

Longitudinal shear forces in concrete slab – strut-and-tie model

verification: $v_{L,Ed} \le v_{Rd,max}$

$v_{L,Ed} \le v_{Rd,s}$

Resistance of concrete struts:

$$v_{Rd,max} = A_{cv}\, \nu\, f_{cd}\, \sin\theta \cos\theta$$

$$A_{cv} = h_c\, a_v \qquad f_{cd} = \alpha_{cc}\, f_{ck} / \gamma_c$$

ν strength reduction factor

$$\nu = 0{,}6\,(1-(f_{ck}/250)) \text{ with } f_{ck} \text{ in N/mm}^2$$

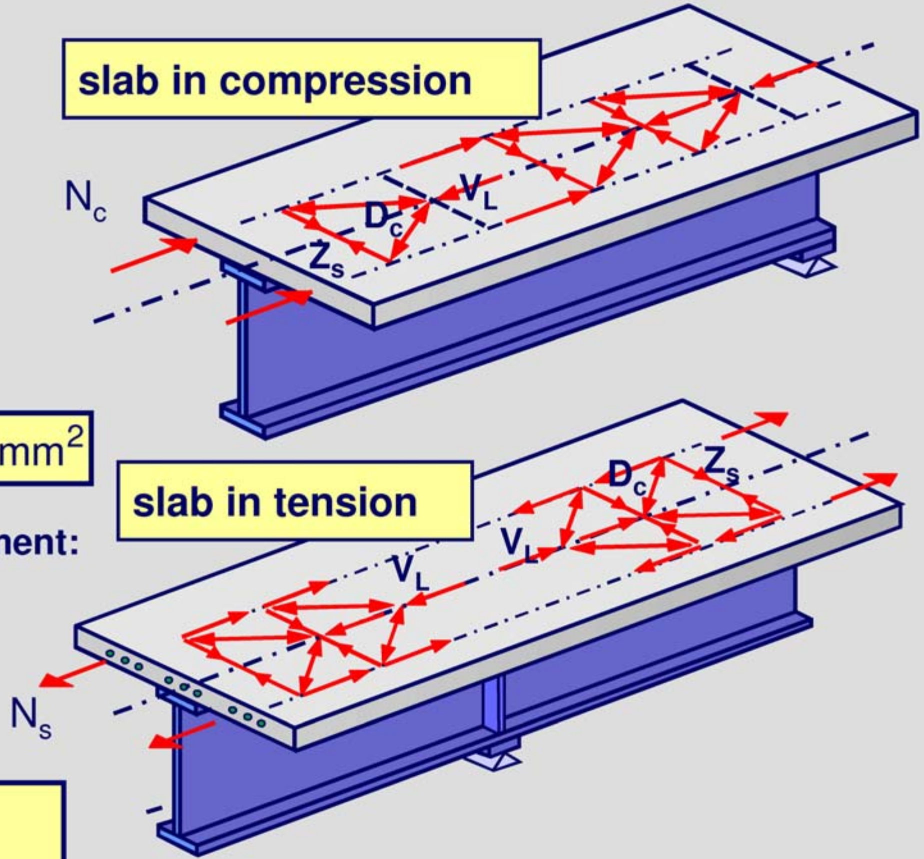
Resistance of transverse reinforcement:

$$v_{Rd,s} = \frac{A_{sf}}{s}\, a_v\, f_{sd} \cot\theta$$

Inclination of concrete compressive struts:

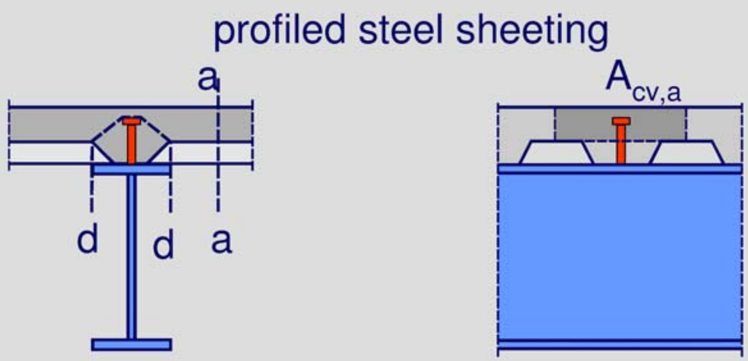
slab in compression: $1{,}0 \le \cot\theta \le 2{,}0$

slab in tension: $1{,}0 \le \cot\theta \le 1{,}25$



[Source: Hanswille]

Longitudinal shear forces in concrete slab – shear plane

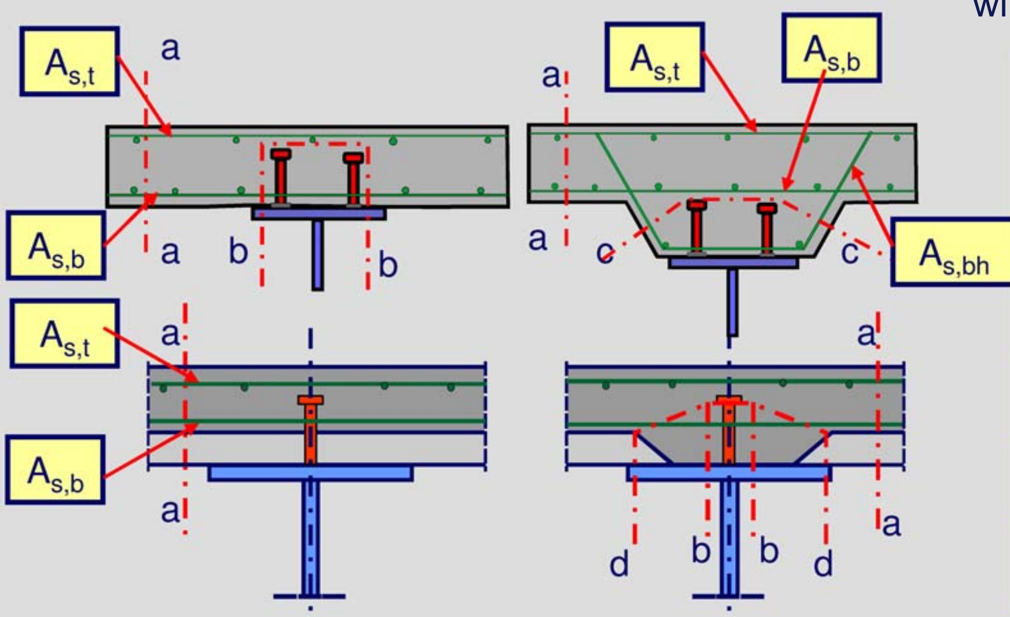


in case of profiled steel only the concrete area above the sheeting should be taken into account

section a-a: $A_{cv} = h_c\, a_v$

section b-b, c-c, d-d: $A_{cv} = L_v\, a_v$

with $L_v = L_{b-b}, L_{c-c}, L_{d-d}$

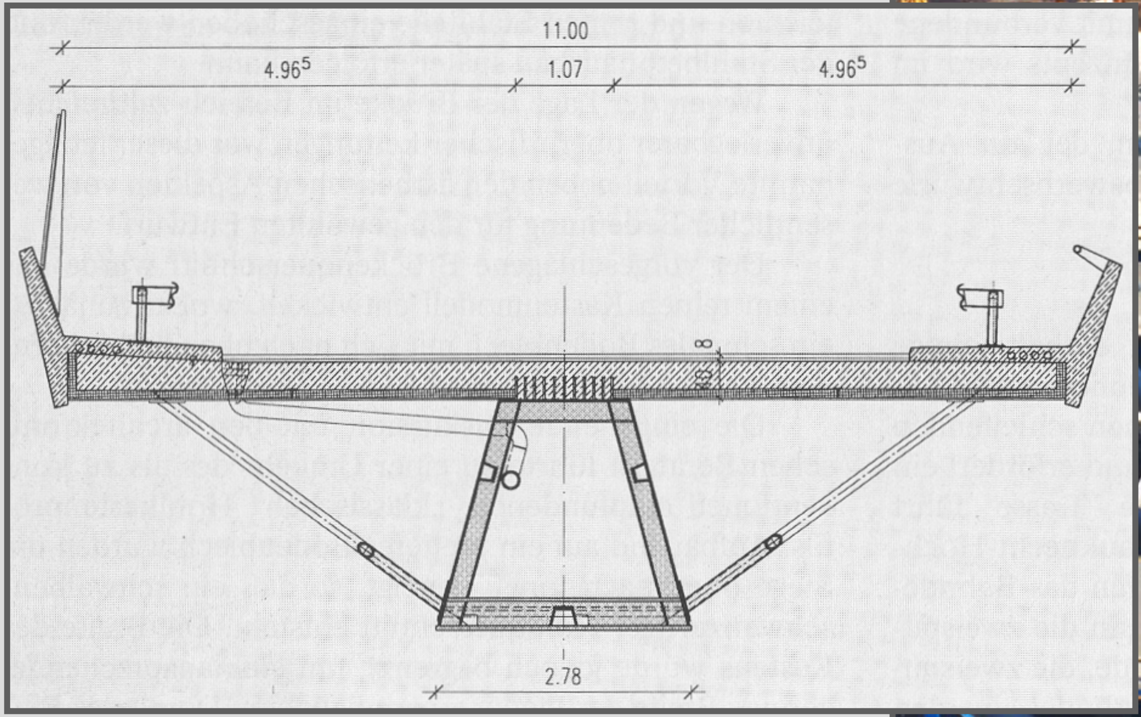


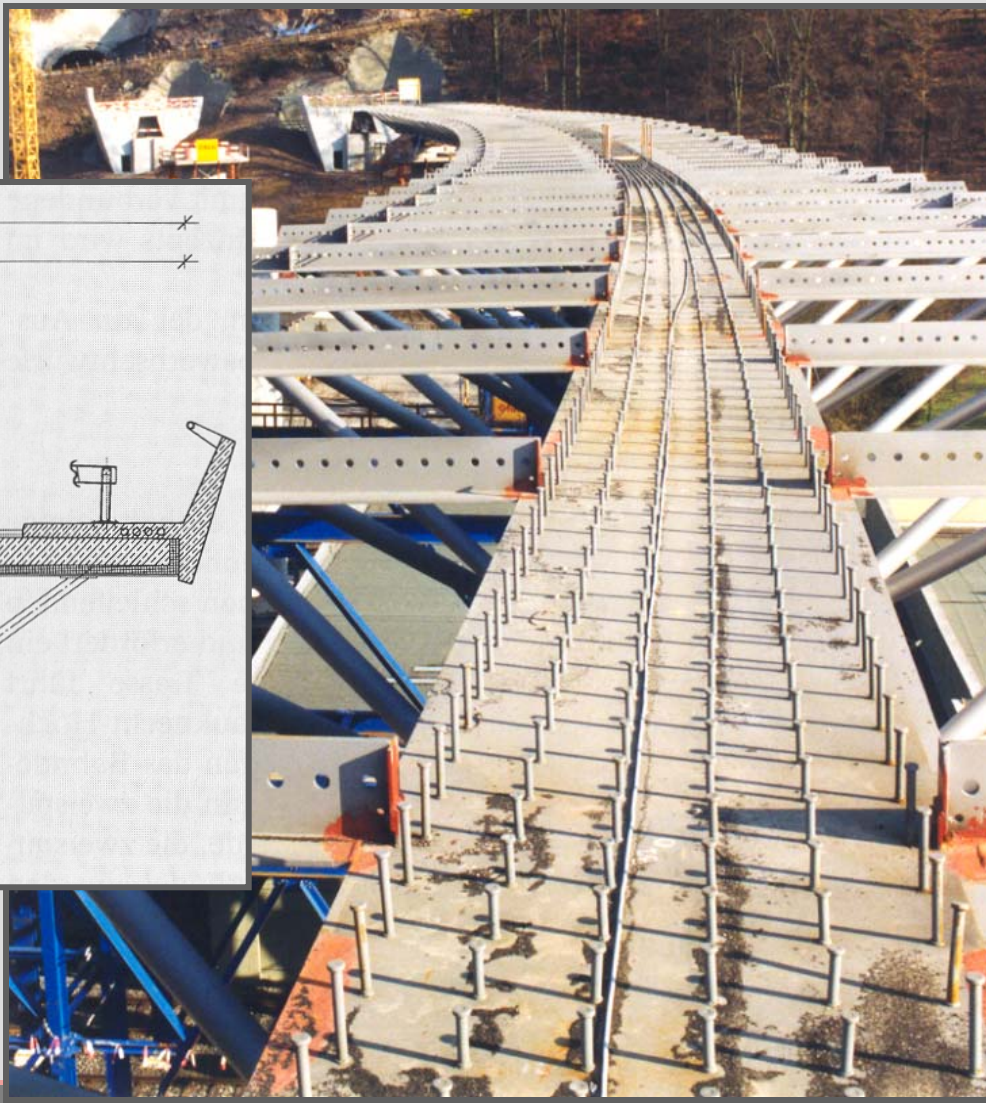
section	effective transverse reinforcement A_{sf}
a-a	$A_{s,t} + A_{s,b}$
b-b	$2\ A_{s,b}$
c-c	$2\ A_{s,bh}$
d-d	$2\ A_{s,b}$

[Source: Hanswille]

Innovative composite bridge structures

Bridge Nesenbachtal, Stuttgart





Nesenbachtal – Result of a design competition

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