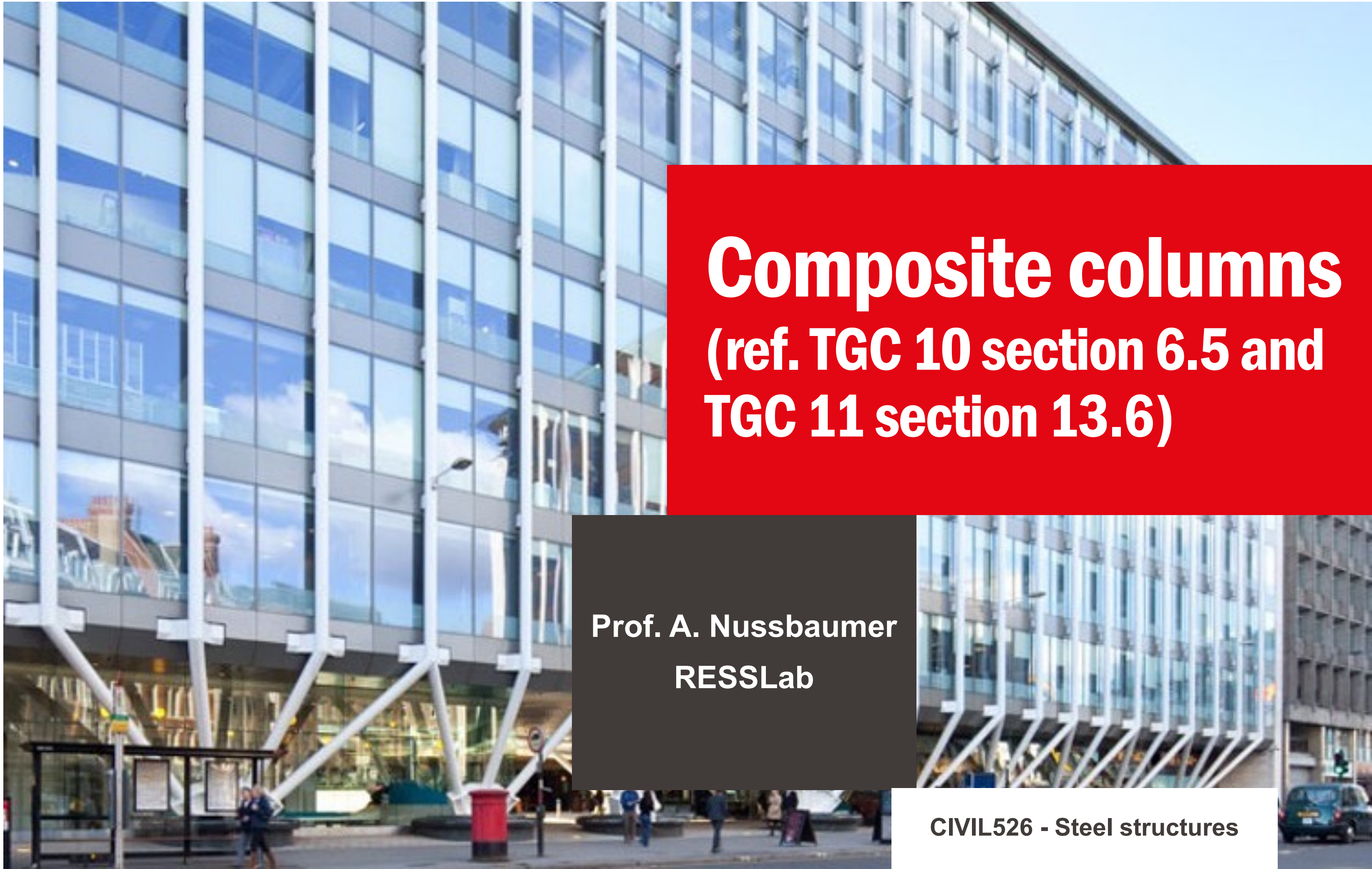


Composite columns
(ref. TGC 10 section 6.5 and TGC 11 section 13.6)

Prof. A. Nussbaumer
RESSLab

CIVIL526 - Steel structures

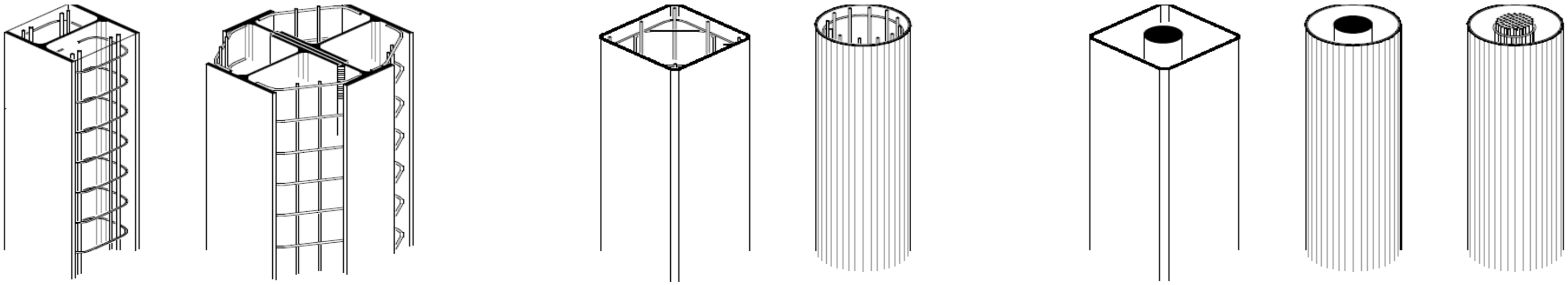
Content

- Design examples
- Column strength
- ELU checks
- Load introduction
- Column footings

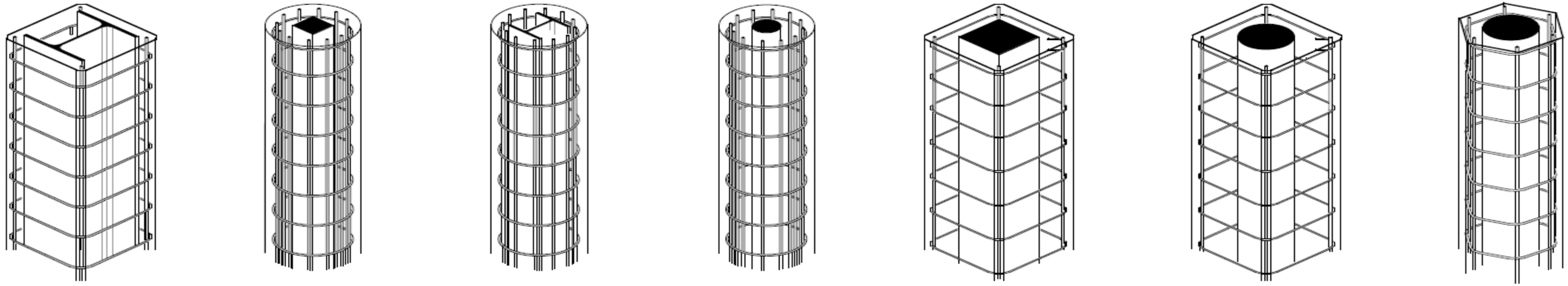
EPFL

Design

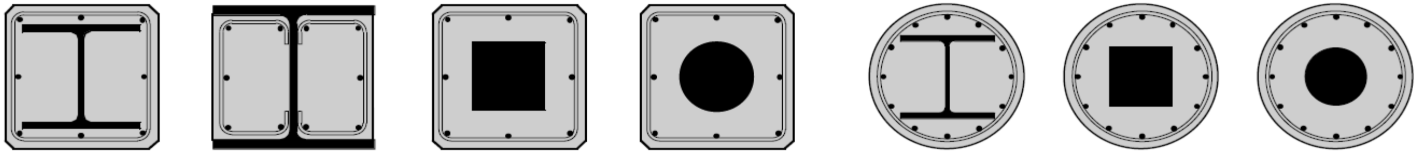
Different solutions



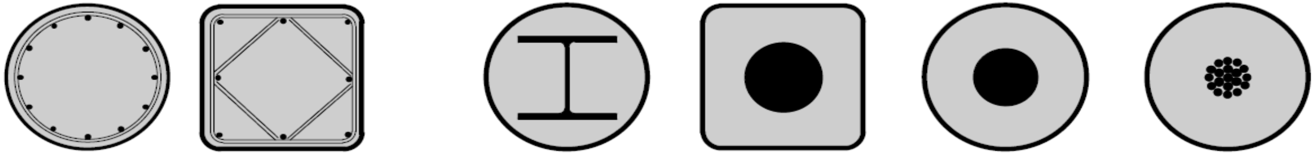
Coated columns (totally or partially) Concrete-filled tubes with steel core



Design

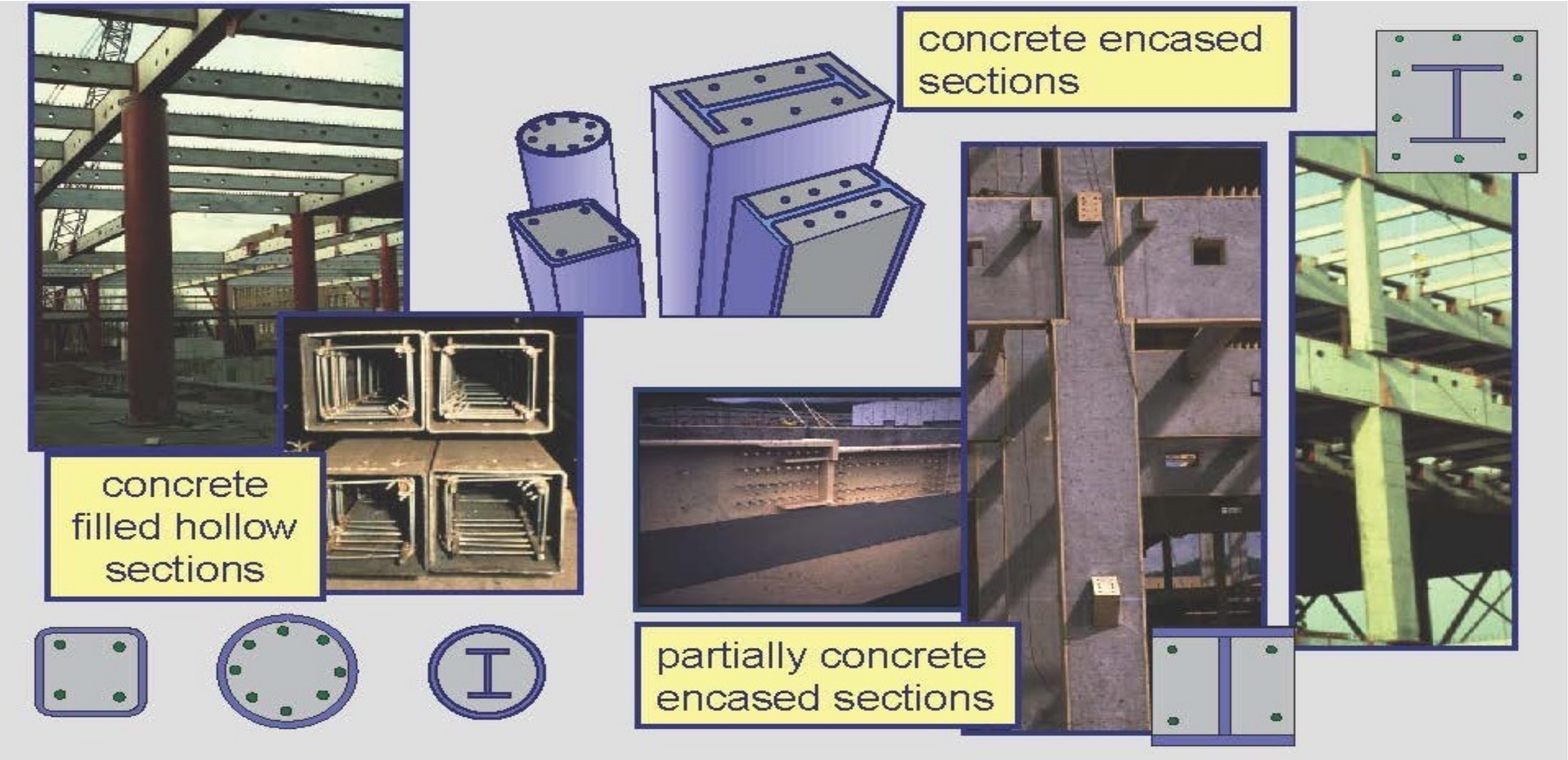


Steel sections encased in concrete



Concrete-filled hollow sections

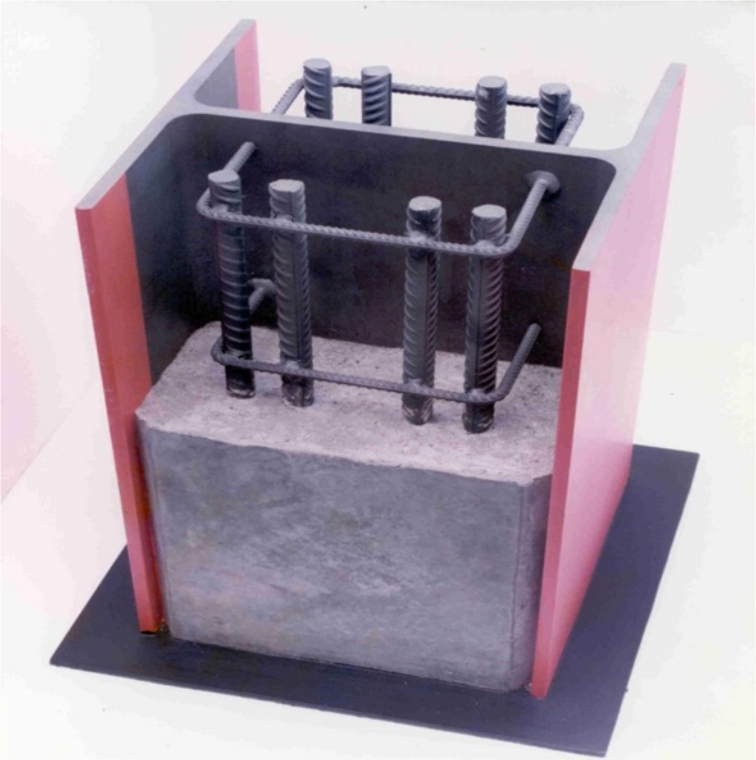
Design

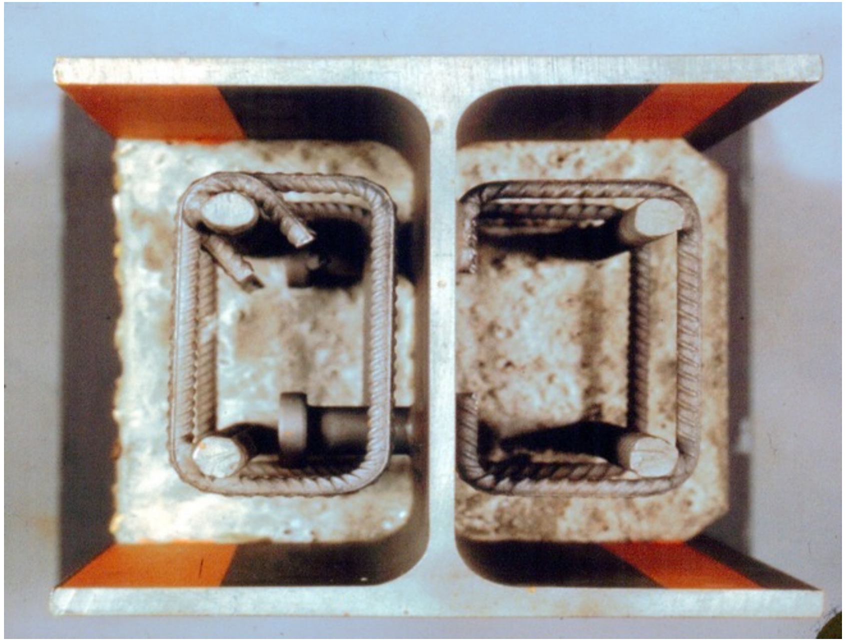


Source: G. Hanswille, Univ. Wuppertal

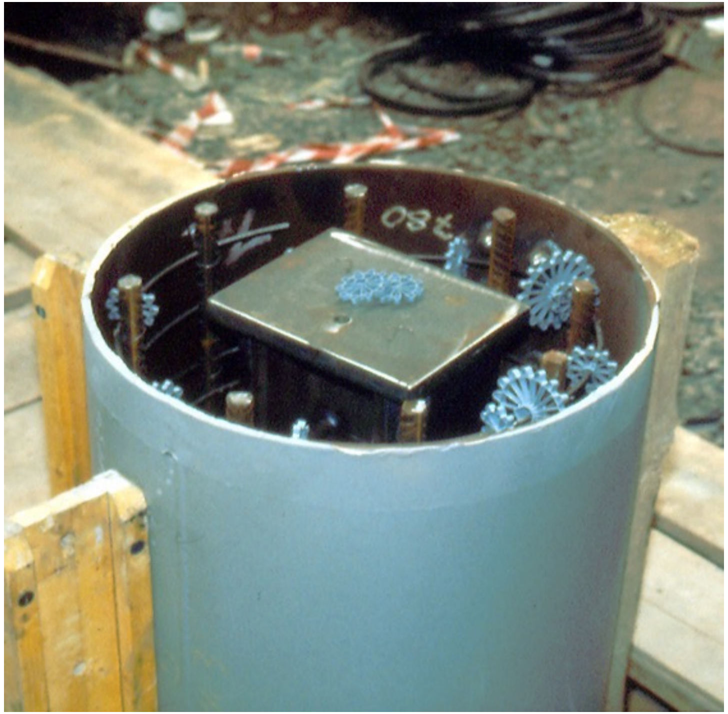
Design

Steel sections partially encased in concrete





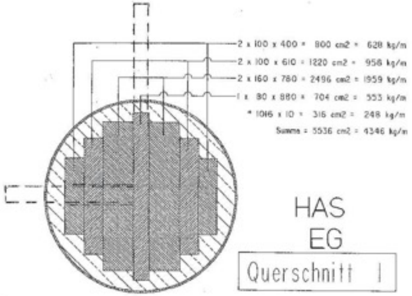
Design



Concrete-filled tubes

Design

High-riser ensemble Munich Business Towers (Germany)



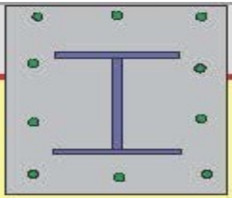
Setting profile for composite columns in plate thickness up to 160 mm from S355J2

Dillinger Hütte GTS

Composite columns, general advantages

- Smaller cross-sections than concrete sections
- Concrete protects steel from fire and from a rapid rise in temperature
- Concrete protects the steel section from impact (in parkings …)
- Prefabrication (better dimensional accuracy)

Sections completely encased in concrete



advantages:

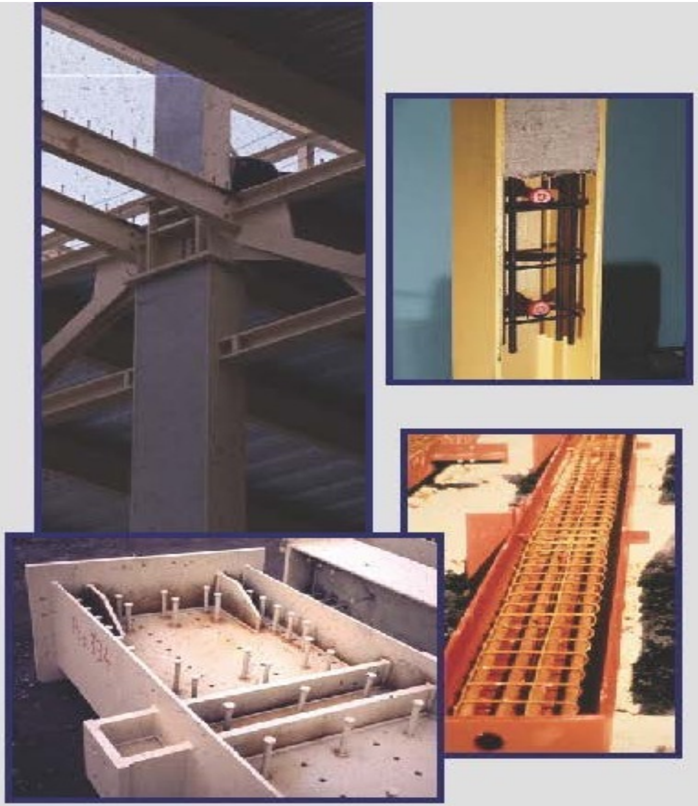
- high bearing resistance
- high fire resistance
- economical solution with regard to material costs

disadvantages:

- high costs for formwork
- difficult solutions for connections with beams
- difficulties in case of later strengthening of the column
- in special case edge protection is necessary

Source: G. Hanswille, Univ. Wuppertal

Sections partially encased in concrete



advantages:

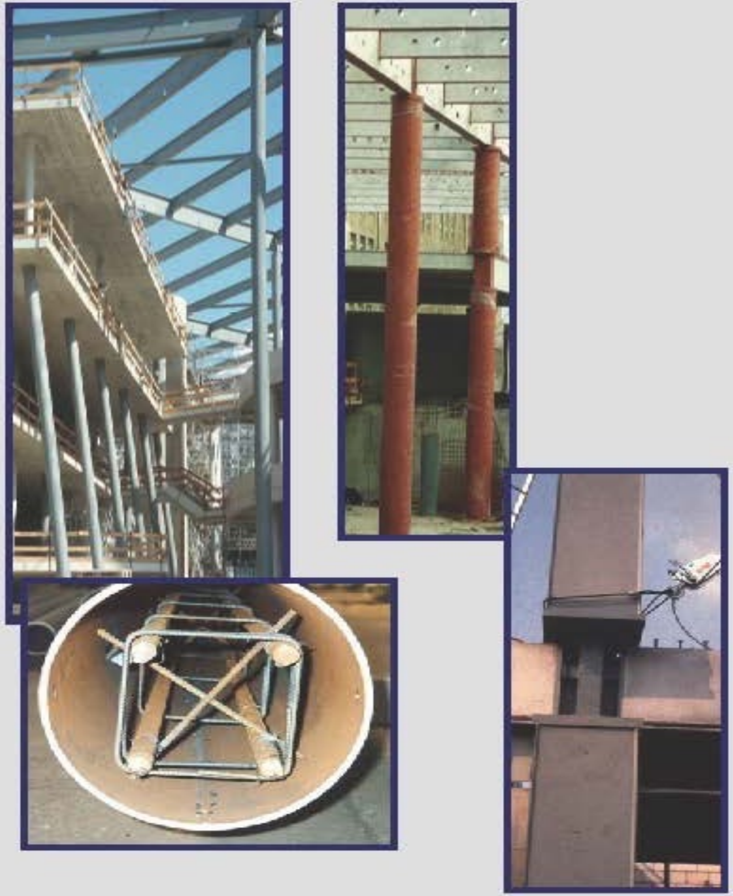
- high bearing resistance, especially in case of welded steel sections
- no formwork
- simple solution for joints and load introduction
- easy solution for later strengthening and additional later joints
- no edge protection

disadvantages:

- lower fire resistance in comparison with concrete encased sections.

Source: G. Hanswille, Univ. Wuppertal

Concrete-filled hollow sections



advantages:

- high resistance and slender columns
- advantages in case of biaxial bending
- no edge protection

disadvantages :

- high material costs for profiles
- difficult casting
- additional reinforcement is needed for fire resistance

Source: G. Hanswille, Univ. Wuppertal

Concrete-filled tubes with steel core

advantages:

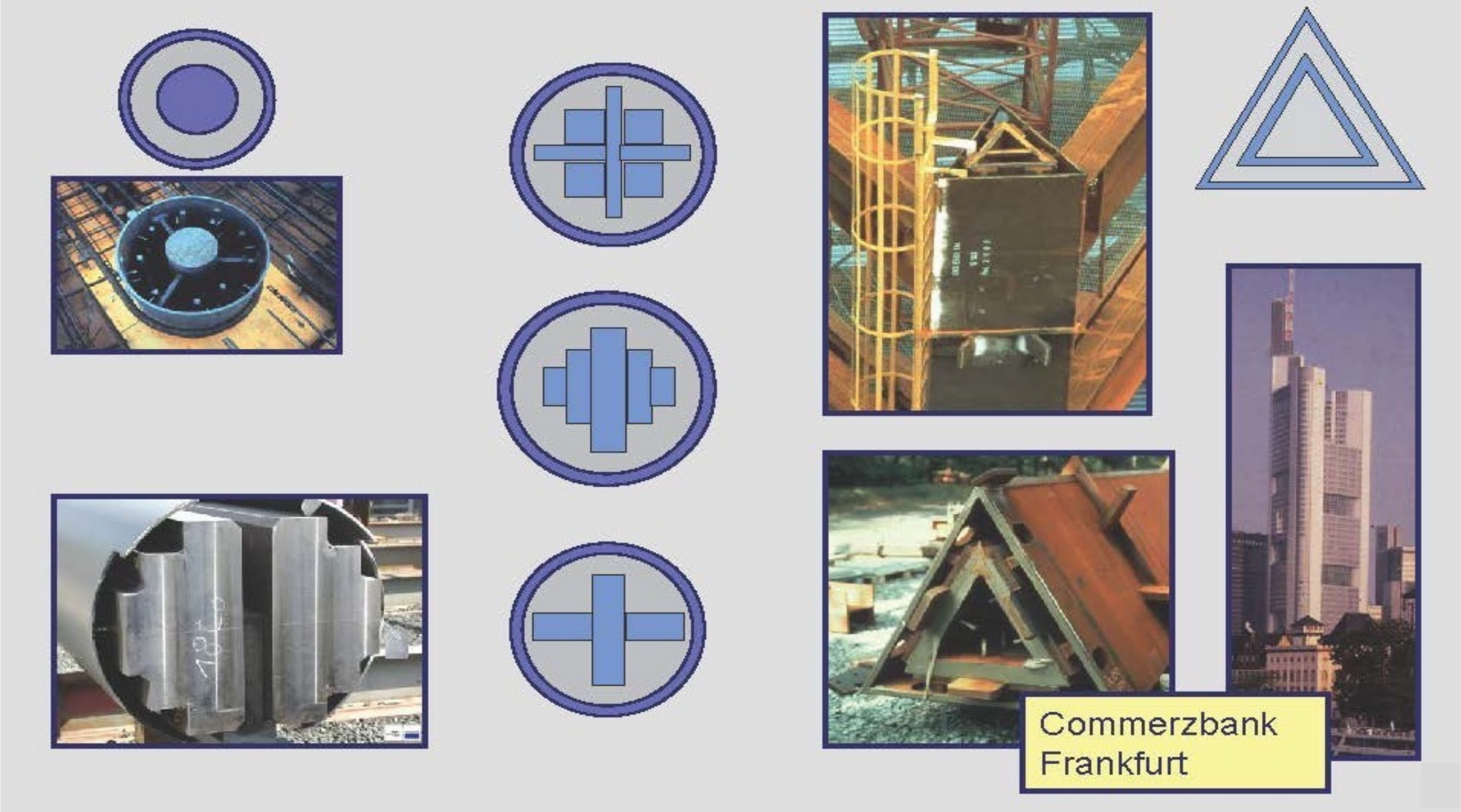
- extreme high bearing resistance in combination with slender columns
- constant cross section for all stories is possible in high rise buildings
- high fire resistance and no additional reinforcement
- no edge protection

disadvantages:

- high material costs
- difficult casting

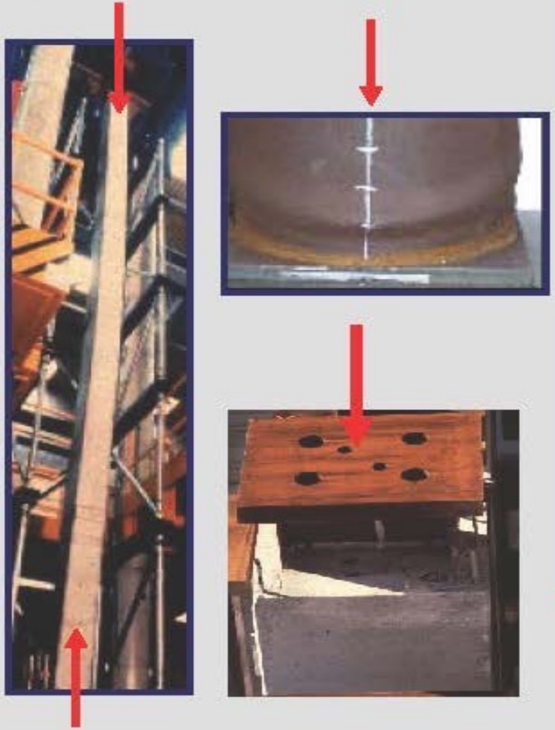
Source: G. Hanswille, Univ. Wuppertal

Concrete-filled tubes with steel core



Source: G. Hanswille, Univ. Wuppertal

Sizing of composite columns to EN 1994-1-1



Verifications for composite columns

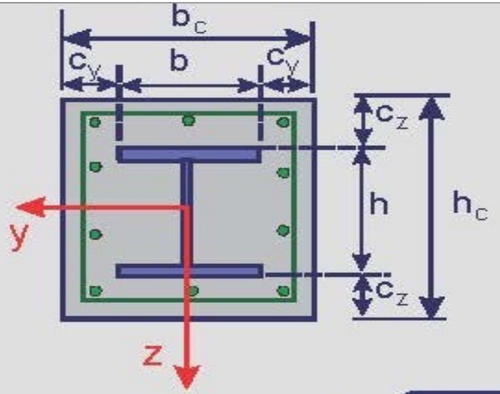
- Resistance of the member for structural stability
  - General method
  - Simplified method ⇐ SIA 264
- Resistance to local Buckling
- Introduction of loads
- Longitudinal shear outside the areas of load introduction

Source: G. Hanswille, Univ. Wuppertal

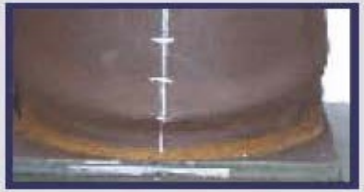
Local buckling resistance to EN1994-1-1

concrete encased cross-sections

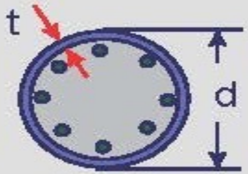
Verification is not necessary where $c_z \geq \begin{cases} 40\ \text{mm} \\ b/6 \end{cases}$



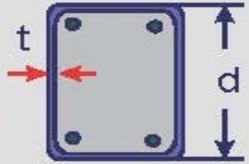
concrete filled hollow section



$$\max\left(\frac{d}{t}\right) = 90\,\varepsilon^2$$



$$\max\left(\frac{d}{t}\right) = 52\,\varepsilon$$

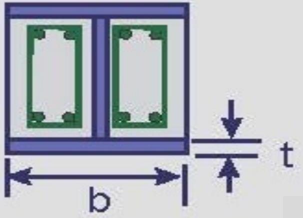


partially encased I sections

$$\varepsilon = \sqrt{\frac{f_{yk,o}}{f_{yk}}}$$

$$f_{yk,o} = 235\ \text{N/mm}^2$$

$$\max\left(\frac{b}{t}\right) = 44\,\varepsilon$$



12

Source: G. Hanswille, Univ. Wuppertal

Resistance to local buckling, SIA 264

Tableau 3: Critères d'élancement d'a
$\varepsilon = \sqrt{235/f_y}$

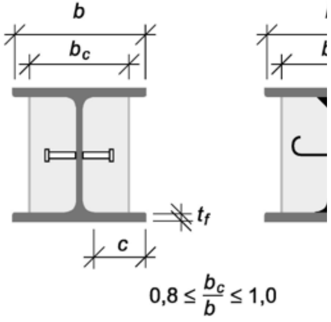
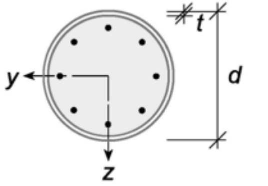
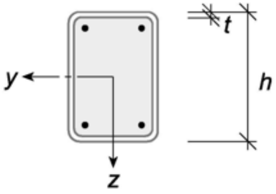
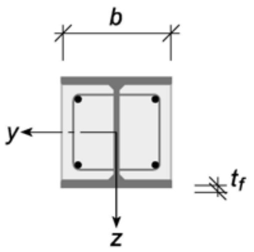
Profilé laminé	Poutres
	
Classe de section	
1	
2	
3	

Tableau 6: Valeurs limites (d/t),(h/t) et (b/t_f)

Section transversale		Max (d/t), max (h/t) et max (b/t)
Profilé creux circulaire		max $(d/t) = 90\,\varepsilon^2$
Profilé creux rectangulaire		max $(h/t) = 52\,\varepsilon$
Profilé à double té à âme enrobée		max $(b/t_f) = 44\,\varepsilon$

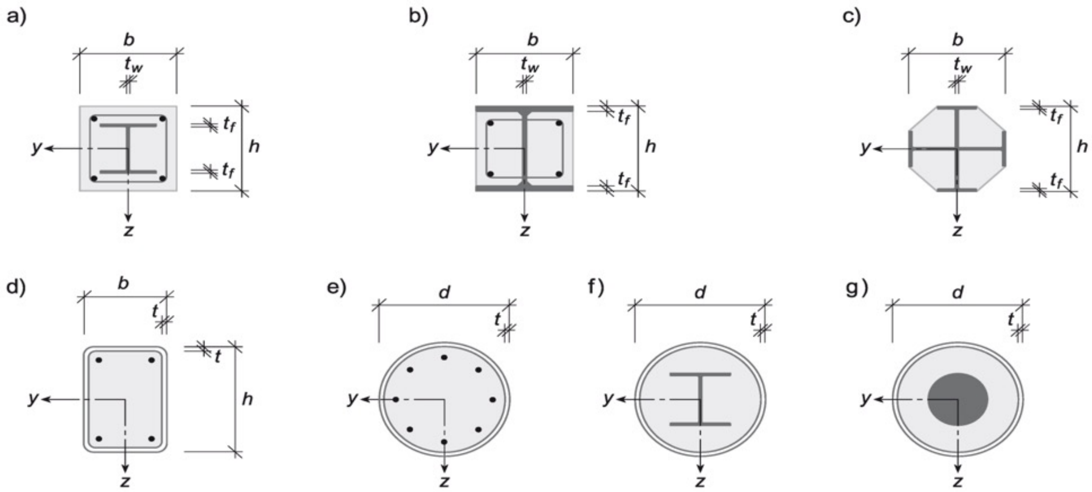
Resistance of composite columns

Definitions and limits

- Concrete columns $\delta < 0.2$
- Mixed columns $0.2 \leq \delta \leq 0.9$
- Steel columns $\delta \geq 0.9$

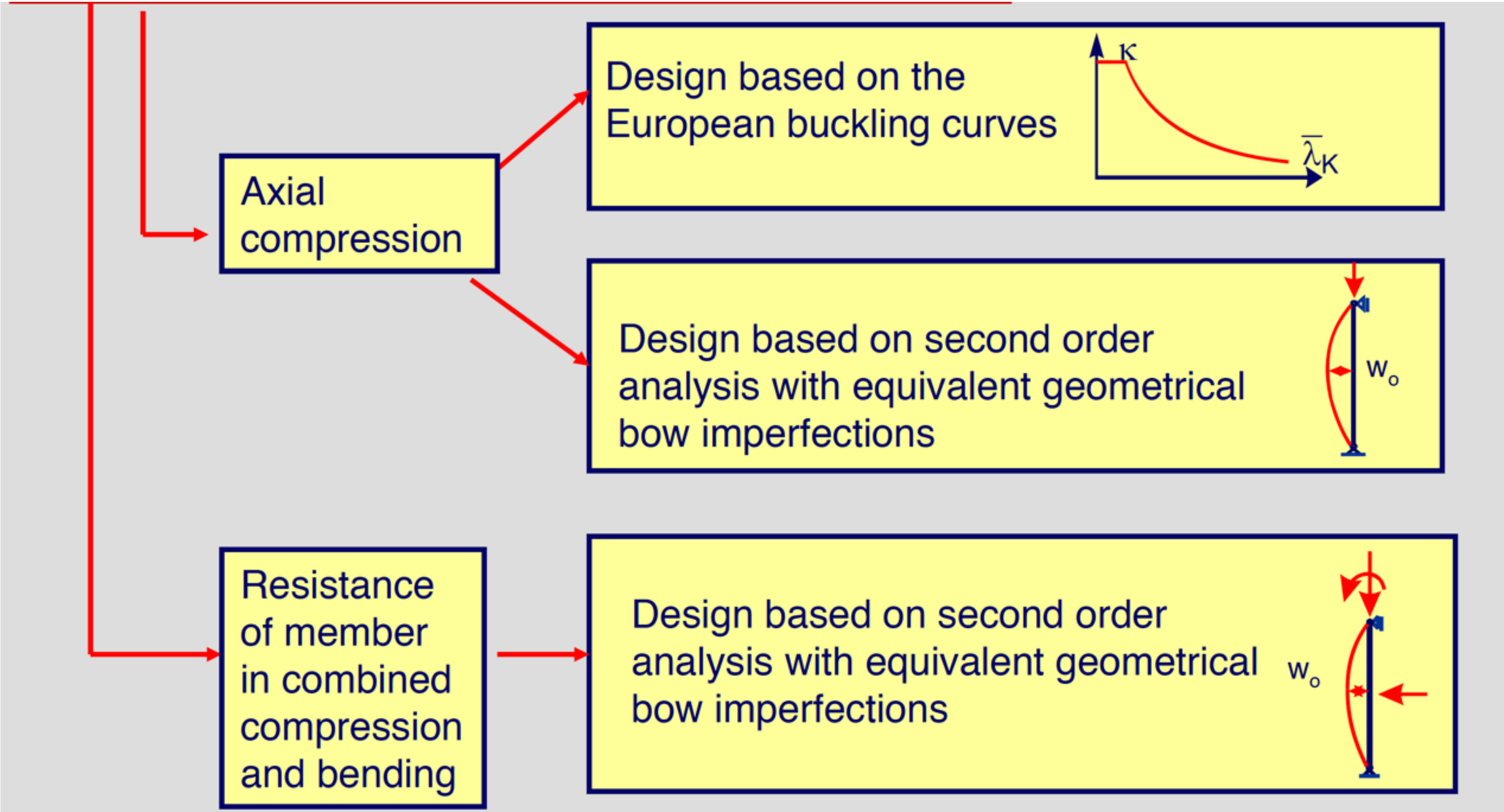
$$\delta = \frac{A_a f_y / \gamma_a}{N_{pl,Rd,b}}$$

For calculation purposes, the longitudinal reinforcement must not exceed **6%** of the concrete cross-section.



EN 1994-1-1, fig. 6.17)

Simplified verification method

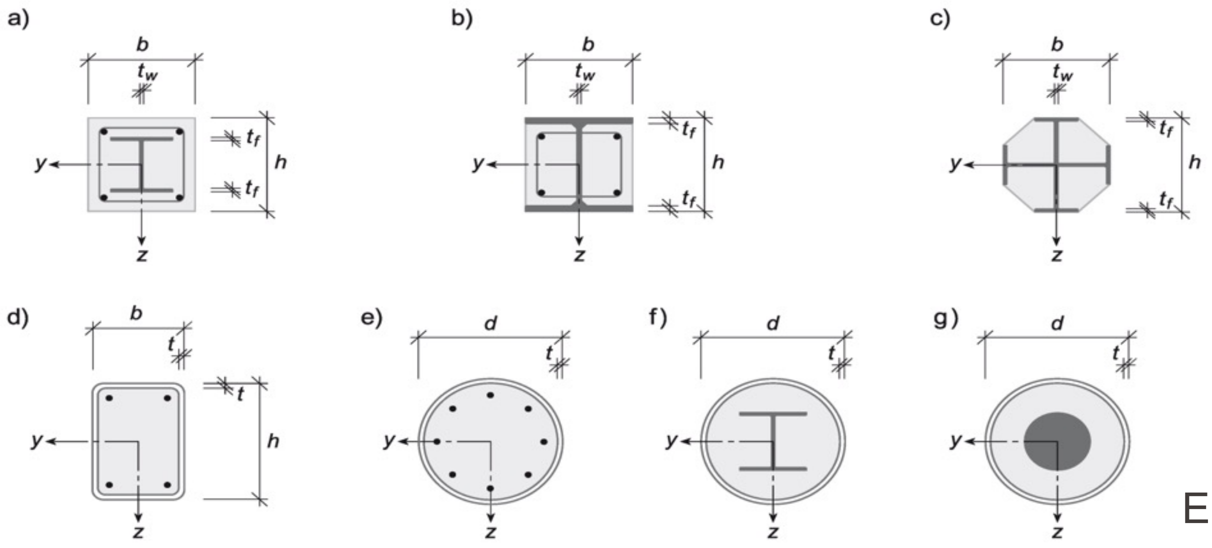


Source: G. Hanswille, Univ. Wuppertal

Resistance of composite columns

Definitions and limits

- Compression check if $e \leq 0.1h$ or $e \leq 0.1d$
  - ➢ In section, and buckling curves
- Otherwise, check for compression <u>and bending</u> along an axis
  - ➢ N-M interaction diagrams



EN 1994-1-1, fig. 6.17)

Resistance of composite columns in section under N

Simplified method for bi-symmetrical sections (including tubes) according to SIA 264 § 5.3.2

Terms and conditions:

$$\bar{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \leq 2{,}0 \qquad 0.2 \leq h/b \leq 5.0$$

The design plastic compressive strength $N_{pl,Rd}$ is equal to:

$$N_{pl,Rd} = A_a \cdot \frac{f_y}{\gamma_a} + A_c \cdot \frac{0{,}85 \cdot f_{ck}}{\gamma_c} + A_s \cdot \frac{f_{sk}}{\gamma_s}$$

$\gamma_a \leftarrow 1.05$ $\quad \gamma_c \leftarrow 1.50$ $\quad \gamma_s \leftarrow 1.15$

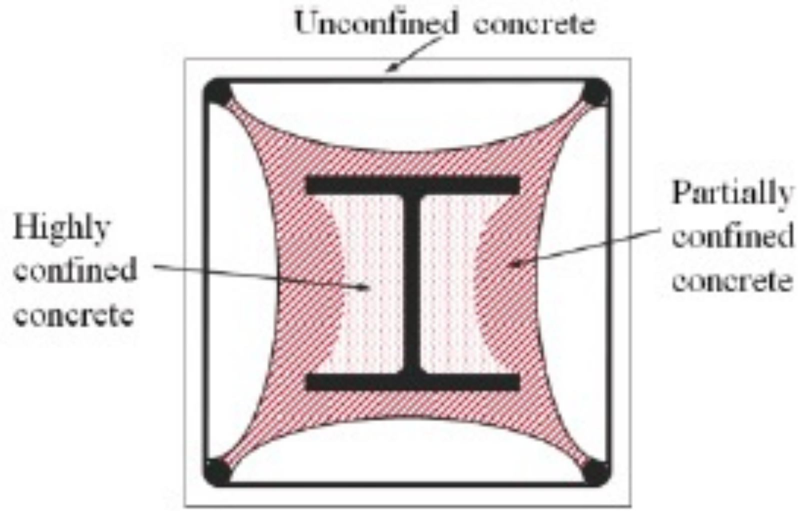
For tubes filled with concrete 0.85 can be replaced by 1.0 in the above expression

The characteristic value of compressive strength is:

$$N_{pl,Rk} = A_a \cdot f_y + A_c \cdot 0{,}85 \cdot f_{ck} + A_s \cdot f_{sk}$$

Resistance of composite columns, confinement

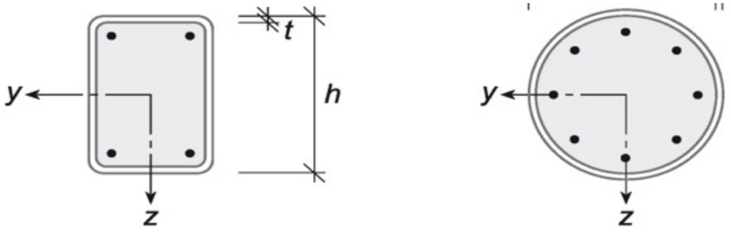
- If fully encased, confinement not considered (≠ concrete only columns)
- But total cross-section is used in calculations
- More realistic model confirms validity of this simplification



Ref: Chiew and Cai - 2018 - Design of high-strength steel reinforced concrete columns

Hollow sections filled with concrete:

- “Medium” confinement ⇒ +15%
- Difference between Poisson’s coeff.
- Shrinkage effects



Resistance of composite columns under N, SIA 264 § 5.3.2

Buckling ($\overline{\lambda_k} - \chi_k$)

$$N_{cr,b} = \frac{\pi^2 \cdot (E \cdot I)_{eff,\lambda}}{L_k^2}$$

$$(E \cdot I)_{eff,\lambda} = E_a \cdot I_a + 0.6 \cdot E_c \cdot I_c + E_s \cdot I_s$$

Influence of long-term effects on concrete stiffness

$$E_{c,eff} = E_{cm} \frac{1}{1 + (N_{G,Ed} / N_{Ed})\varphi_t}$$ (= E_{cm} /2.5 first approximation)

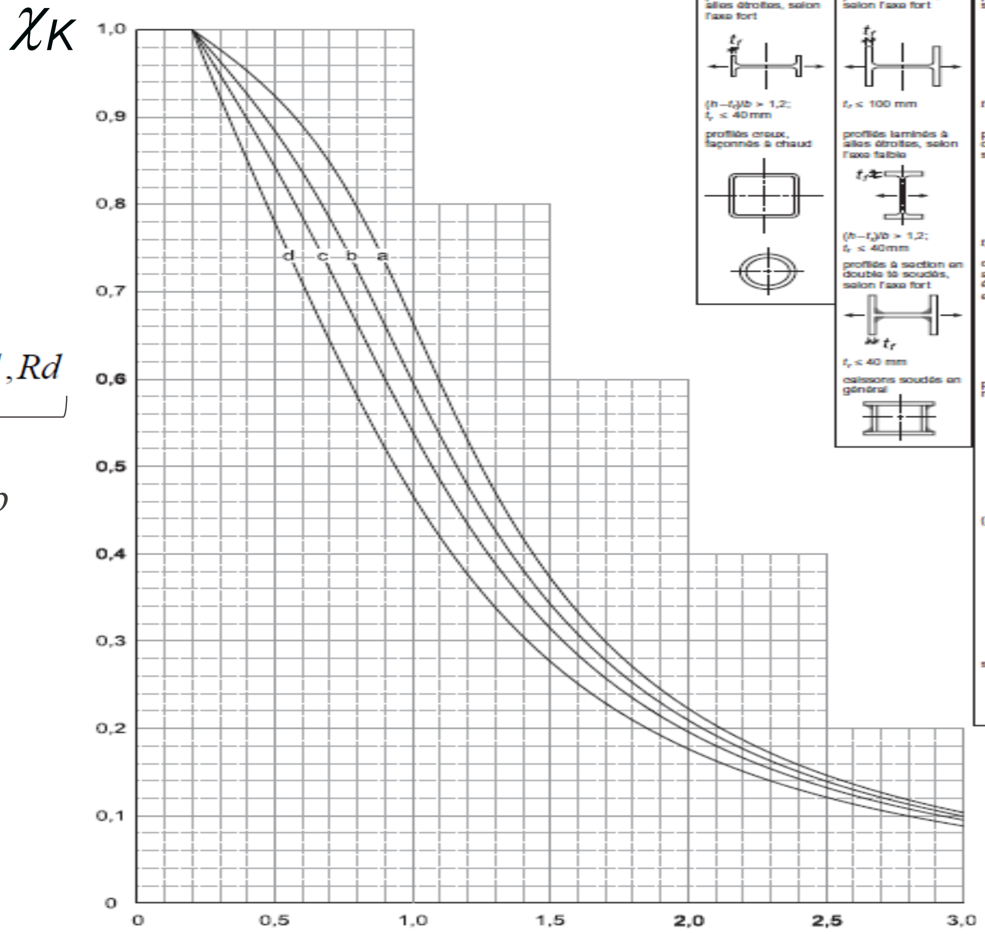
N_{Ed} calculation value of the normal force

$N_{G,Ed}$ part of this normal force acting permanently it is permanent.

φ_t is the creep coefficient

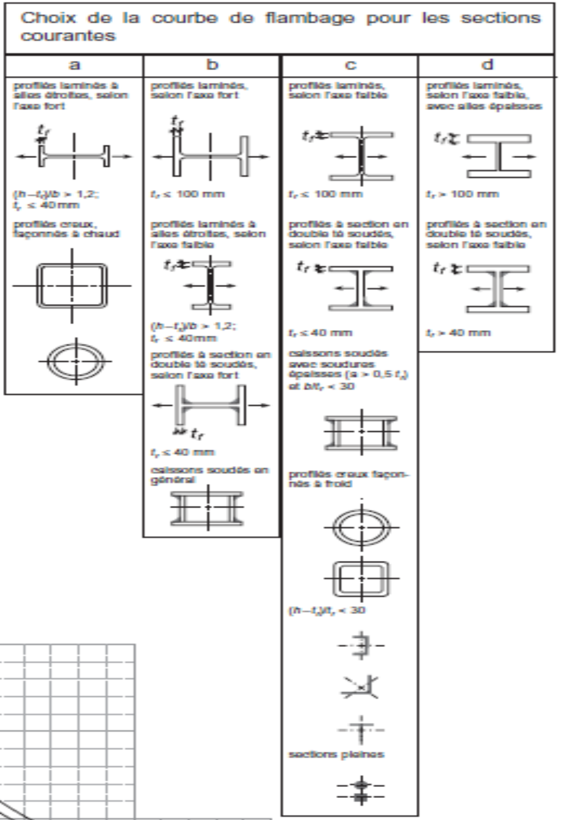
$$\overline{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \leq 2,0 \xrightarrow{\text{Buckling curves}} \chi_K$$

Buckling curves



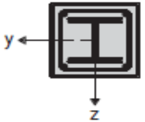
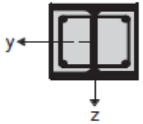
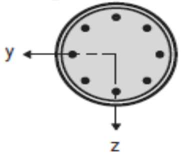
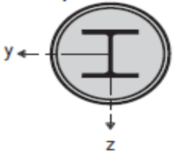
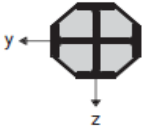
Check:

$$N_{Ed} \leq \underbrace{\chi_K \cdot N_{pl,Rd}}_{Nk_{,Rd,b}}$$



$$\bar{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \leq 2{,}0$$

Tableau 6.5 : Courbes de flambement et imperfections de barres pour les poteaux mixtes

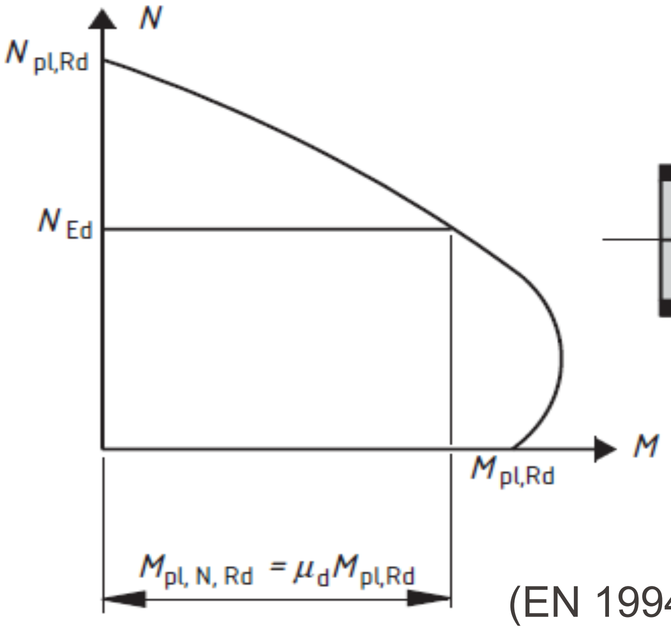
Section	Limites	Axe de flambement	Courbe de flambement	Imperfection d'élément
Profilé enrobé de béton 		y-y	b	$e_1 = L/200$
		z-z	c	$L/150$
Profilé partiellement enrobé de béton 		y-y	b	$L/200$
		z-z	c	$L/150$
Profil creux circulaire et rectangulaire en acier 	$\rho_s \leq 3\%$	quelconque	a	$L/300$
	$3\% < \rho_s \leq 6\%$	quelconque	b	$L/200$
Profil creux circulaire en acier avec adjonction d'un profilé en I 		y-y	b	$L/200$
		z-z	b	$L/200$
Profilé partiellement enrobé de béton avec profilés en I disposés en croix 		quelconque	b	$L/200$

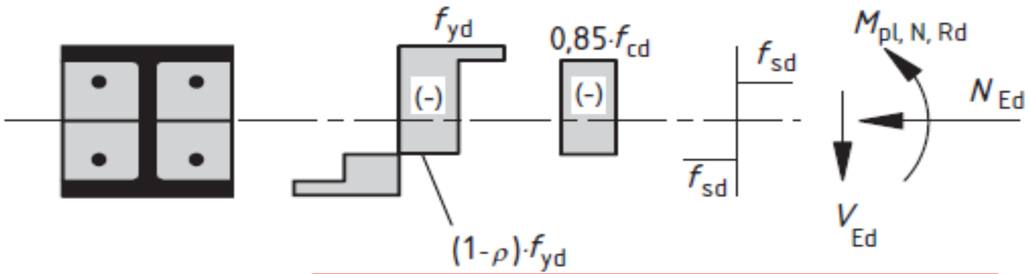
Detailed table for all composite columns

L: length of column

Resistance of a composite column under N+M, SIA 264 § 5.3.4

- Resistance of a section under compression and bending can be calculated using an interaction curve
- Curve calculated assuming plastic stress distribution.

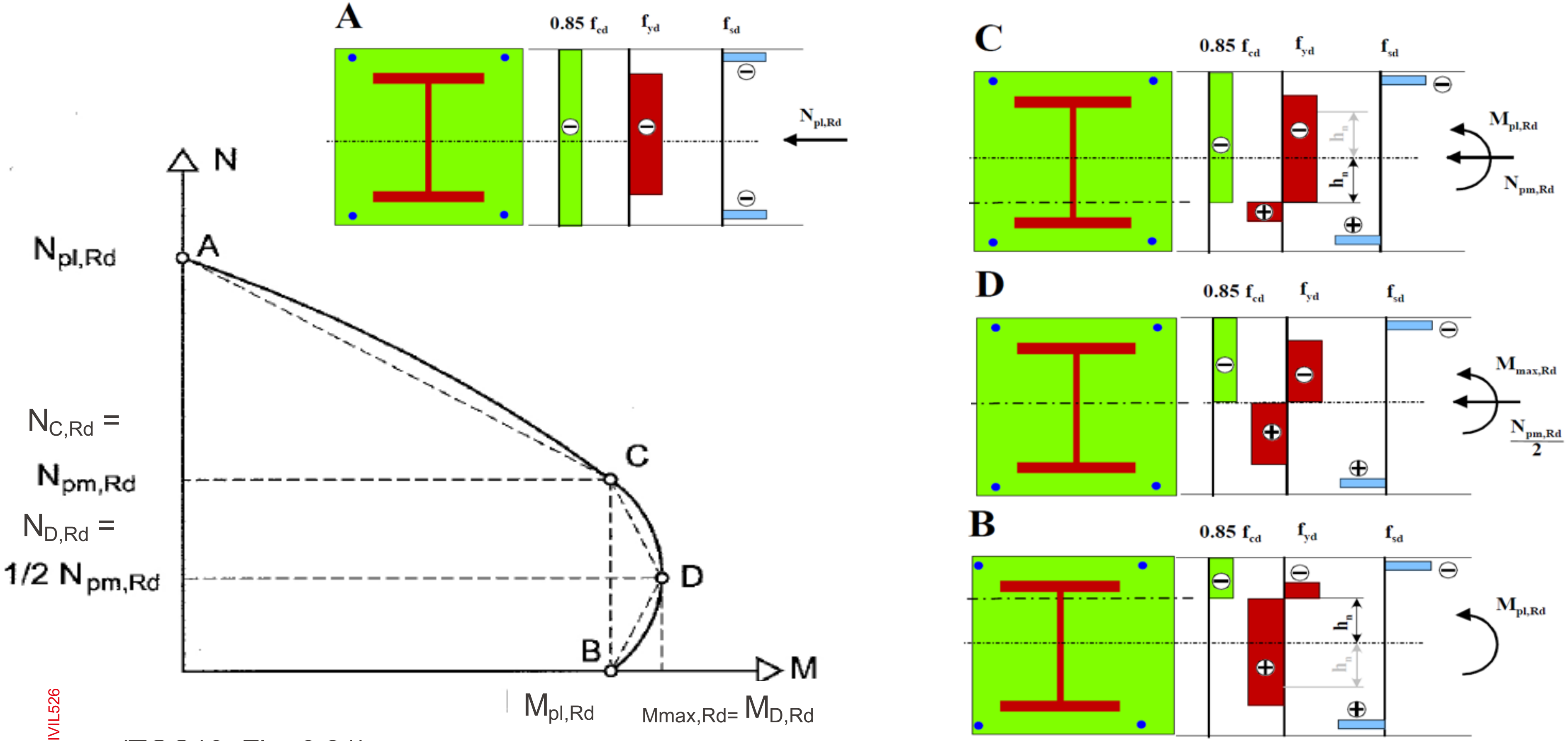




$$M_{pl,N,Rd} = \mu_d \cdot M_{pl,Rd}$$

(EN 1994-1-1, fig. 6.18)

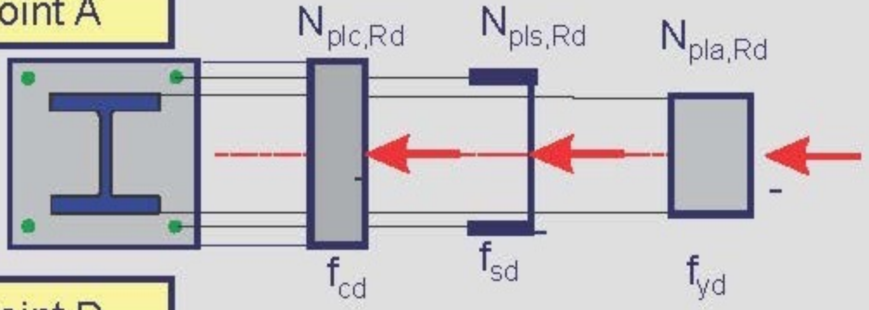
N - M interaction curve and simplification



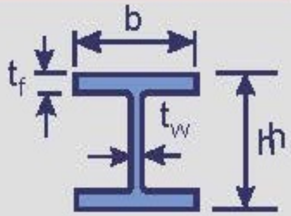
(TGC10, Fig. 6.21)

Complement: Resistance to N + M

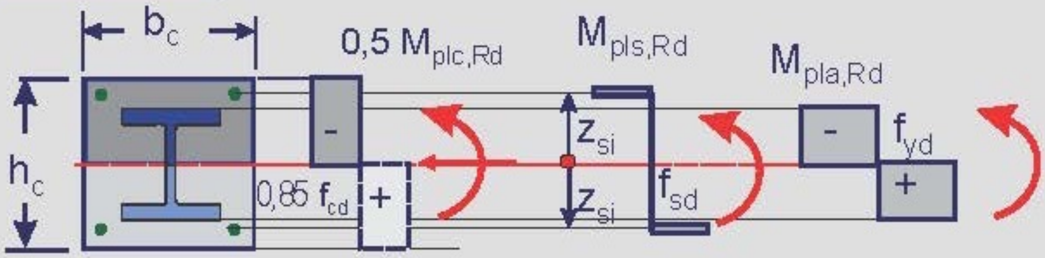
Point A



$$N_{pl,Rd} = N_{pla,Rd} + N_{plc,Rd} + N_{pls,Rd}$$
$$M_{A,Rd} = 0$$



Point D



$$N_{D,Rd} = 0,5\ N_{plc,Rd}$$
$$M_{D,Rd} = M_{max,Rd}$$

$$M_{max,Rd} = M_{pla,Rd} + M_{pls,Rd} + 1/2\ M_{plc,Rd}$$

$$M_{pla,Rd} = W_{pl,a}\ f_{yd} = \left[\underbrace{\frac{(h-2t_f)^2\ t_w}{4}}_{\text{soul}} + \underbrace{b t_f\ (h - t_f)}_{\text{wings}} \right] f_{yd}$$

$$M_{pls,Rd} = W_{pl,s}\ f_{sd} = \left[\sum A_{si}\ z_{si}\right] f_{ys}$$

$$M_{plc,Rd} = W_{pl,c}\ 0,85\ f_{cd} = \left[\frac{b_c\ h_c^2}{4} - W_{pl,a} - W_{pl,s} \right] 0,85\ f_{cd}$$

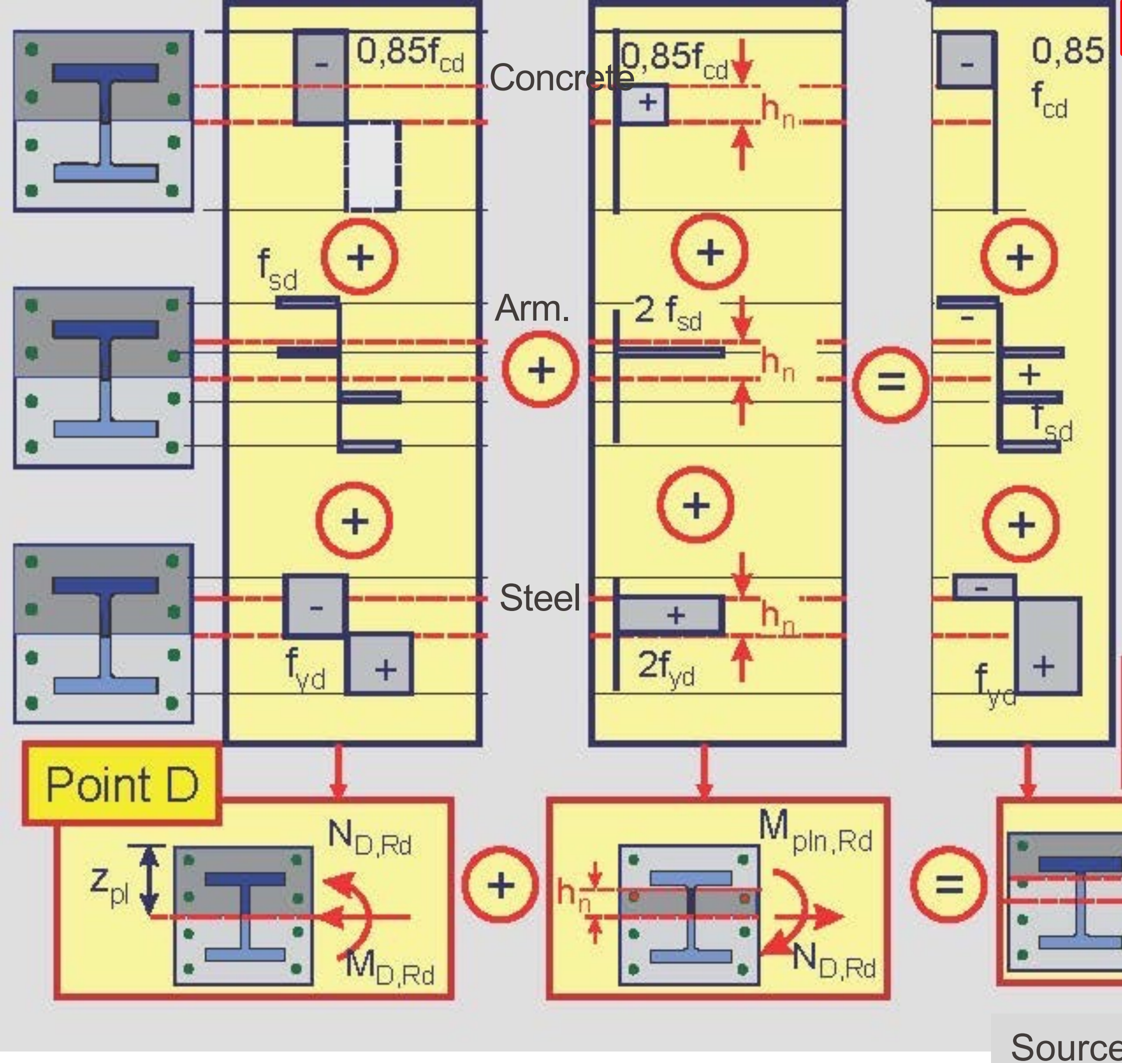
$W_{pl,a}$ plastic section modulus of the structural steel section

$W_{pl,s}$ plastic section modulus of the cross-section of reinforcement

$W_{pl,c}$ plastic section modulus of the concrete section

Source: G. Hanswille, Univ. Wuppertal

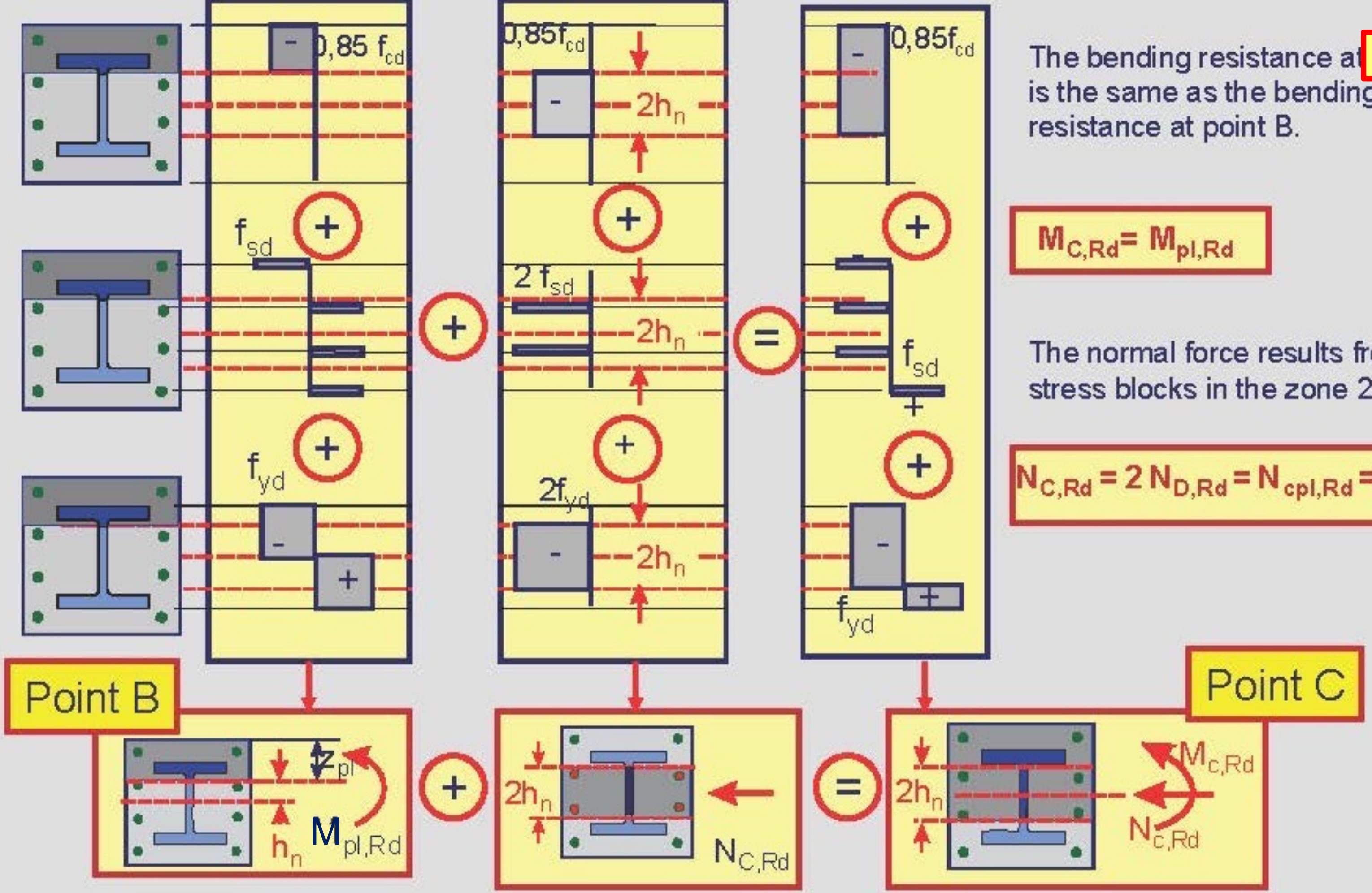
EPFL



Point B is no resistance to compression forces. Therefore the resistance to compression forces at point D results from the additional cross-section zones in compression. With $N_{D,Rd}$ the depth h_n and the position of the plastic neutral axis at point B can be determined. With the plastic bending moment $M_{n,Rd}$ resulting from the stress blocks within the depth h_n the plastic resistance moment $M_{pl,Rd}$ at point B can be calculated by:

$$M_{pl,Rd} = M_{D,Rd} - M_{pln,Rd}$$

Source: G. Hanswille, Univ. Wuppertal



The bending resistance at **Point C** is the same as the bending resistance at point B.

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

The normal force results from the stress blocks in the zone $2h_n$.

$$N_{C,Rd} = 2\,N_{D,Rd} = N_{cpl,Rd} = N_{pm,Rd}$$

Source: G. Hanswille, Univ. Wuppertal

Composite column resistance under N+M

Influence of shear force:

When $V_{a,Ed} > 0.5\ V_{pl,a,Rd}$ → influence of shear on bending strength + normal force. This is taken into account by reducing the yield strength of the web steel, $(1-\rho)f_{yd}$:

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

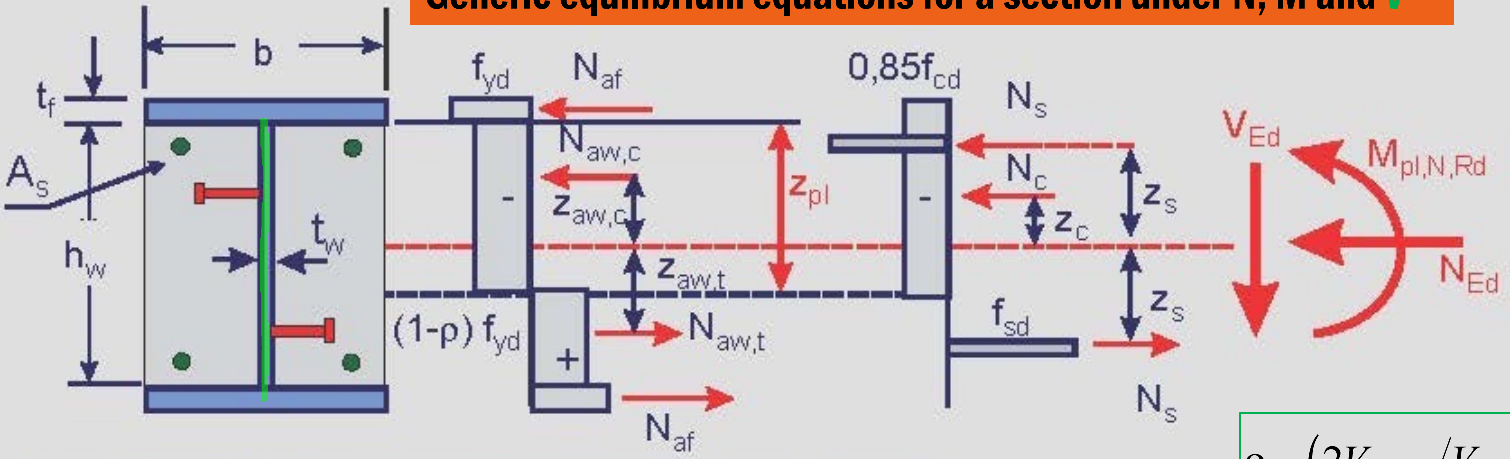
The shear force V_{Ed} can be absorbed by steel alone, or divided into $V_{a,Ed}$ (steel) and $V_{c,Ed}$ (concrete section) according to :

$$V_{a,Ed} = V_{Ed} \frac{M_{pl,a,Rd}}{M_{pl,Rd}}$$

$M_{pl,a,Rd}$ moment of plastic resistance of the steel section

$M_{pl,Rd}$ moment of plastic resistance of the mixed section

Generic equilibrium equations for a section under N, M and V



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

Position of the plastic neutral axis: $\sum N_i = N_{Ed}$

$$N_c + N_{aw,c} - N_{aw,t} = N_{Ed}$$

$$(b - t_w)z_{pl}\,0,85 f_{cd} + t_w z_{pl}(1-\rho)f_{yd} - t_w(h_w - z_{pl})(1-\rho)f_{yd} = N_{Ed}$$

$$z_{pl} = \frac{N_{Ed} + h_w t_w (1-\rho) f_{yd}}{(b - t_w)0,85\ f_{cd} + 2t_w(1-\rho)f_{yd}}$$

$$N_{aw,c} = z_{pl}\, t_w\ (1-\rho)\ f_{yd}$$

$$N_{aw,t} = (h_w - z_{pl})\, t_w\ (1-\rho)\ f_{yd}$$

$$N_{af} = b\, t_f\, f_{yd}$$

$$N_c = (b - t_w)\, z_{pl}\, 0,85\ f_{cd}$$

$$N_s = 2A_s\, f_{sd}$$

Plastic resistance to bending $M_{pl,N,Rd}$ in case of the simultaneously acting compression force N_{Ed} and the vertical shear V_{Ed}:

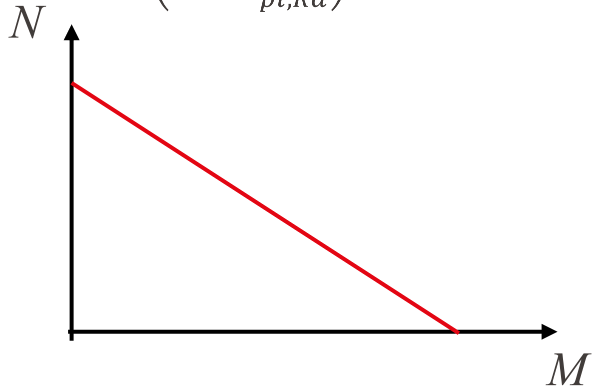
$$M_{pl,N,Rd} = N_c z_c + N_{aw,c} z_{aw,c} + N_{aw,t} z_{aw,t} + N_{af}(h_w + t_f) + 2N_s z_s$$

Source: G. Hanswille, Univ. Wuppertal

Column under N+M. Reminder of the Ayrton-Perry formula

$$\left|\frac{N_{Ed}}{N_{pl,Rd}} + \frac{M_{Ed}}{M_{pl,Rd}}\right| \leq 1.0$$

$$M_{Ed} \leq \left(1 - \frac{N_{Ed}}{N_{pl,Rd}}\right) M_{pl,Rd} \leq M_{pl,Rd}$$



- 2 limits to be respected:

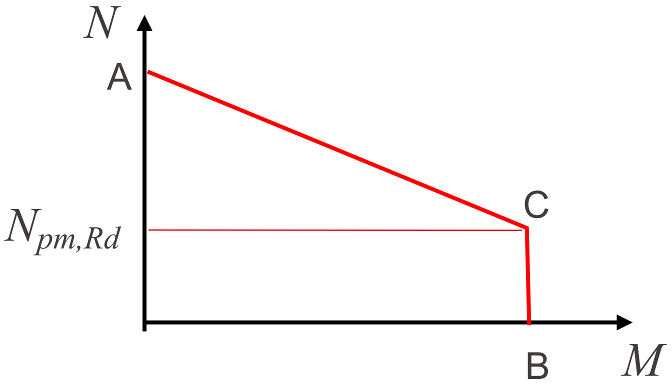
1) $M_{Ed} \leq M_{pl,Rd}$

2) $M_{Ed} \leq \left(1 - \frac{N_{Ed}}{N_{pl,Rd}}\right) M_{pl,Rd}$

- In the case of mixed columns:

1) $M_{Ed} \leq M_{pl,Rd}$

2) $M_{Ed} \leq \underbrace{\left(\frac{N_{pl,Rd} - N_{Ed}}{N_{pl,Rd} - N_{pm,Rd}}\right)}_{\mu_d} M_{pl,Rd}$



Column under N+M. Reminder: interaction verification formulae N_{comp} + M

- Interaction formula:

  SIA 263, equ. (49)

$$\frac{N_{Ed}}{N_{k,Rd}} + \frac{1}{1 - N_{Ed}/N_{cr}} \frac{\omega \cdot M_{Ed,max}}{M_{Rd}} \leq 1.0$$

- Or 2ème order and check in section:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{1}{1 - N_{Ed}/N_{cr}} \frac{M_{e1,Ed}}{M_{Rd}} \leq 1.0$$

$\mathcal{M}_{Ed} = M_{Ed,II}$

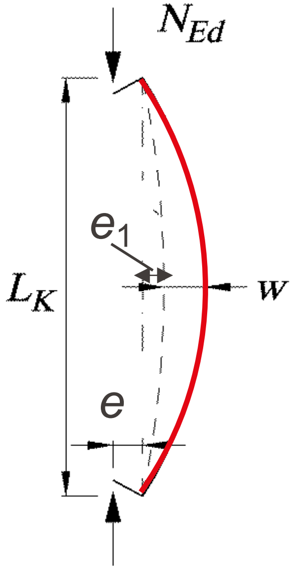
- Calculation M_{Ed} including e_1 :

$$M_{e1,Ed,I} = e_1 \cdot N_{Ed} + M_{Ed,I}$$

- For mixed columns, with 2nd order effect expressed as:

$$M_{Ed,II} = \boldsymbol{k} \cdot M_{Ed} \leq 0.9 \cdot \mu_d \cdot M_{pl,Rd}$$

Column under N+M, buckling: second-order effects



Second-order effects must be taken into account in analysis of the structure if: $\frac{N_{Ed}}{N_{cr,eff}} > 0{,}1$ (with or without bending)

Second-order elastic analysis

Must be made with reduced EI_{eff} :

$$(E \cdot I)_{eff,d} = 0{,}9 \cdot \left(E_a \cdot I_a + 0{,}5 \cdot E_{cm} \cdot I_c + E_s \cdot I_s\right)$$

$$N_{cr,eff} = \frac{\pi^2 \cdot (E \cdot I)_{eff,d}}{L_k^2}$$

Initial geometric imperfections & σ_{res} taken as an equivalent imperfection e_1 of the element (or w_0) arch shape

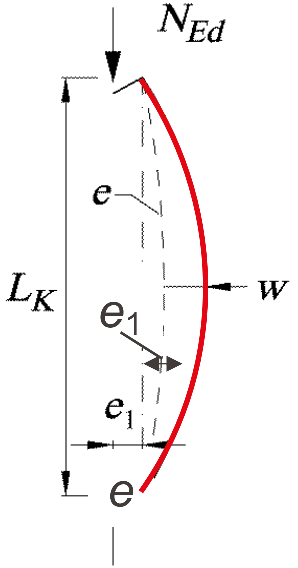
Column under N+M, buckling: second-order effects

Second-order elastic analysis

Second-order effects are considered by amplifying first-order effects using the coefficient k:

$$k = \frac{\beta}{1 - \frac{N_{Ed}}{N_{cr,eff}}} \geq 1.0$$

Influence the moment diagram shape



Resulting second-order moment:

$$M_{Ed,II} = k\left(M_{Ed,I} + N_{Ed} \cdot e_1\right)$$

Depending on the moment diagram, either $M_{Ed,II}$ or $M_{Ed,\,ext}$ can be determined ⇒ Check with $M_{Ed,max}$

Buckling: Summary of $\overline{\lambda_k} - \chi_k$ curves and equivalent imperfections

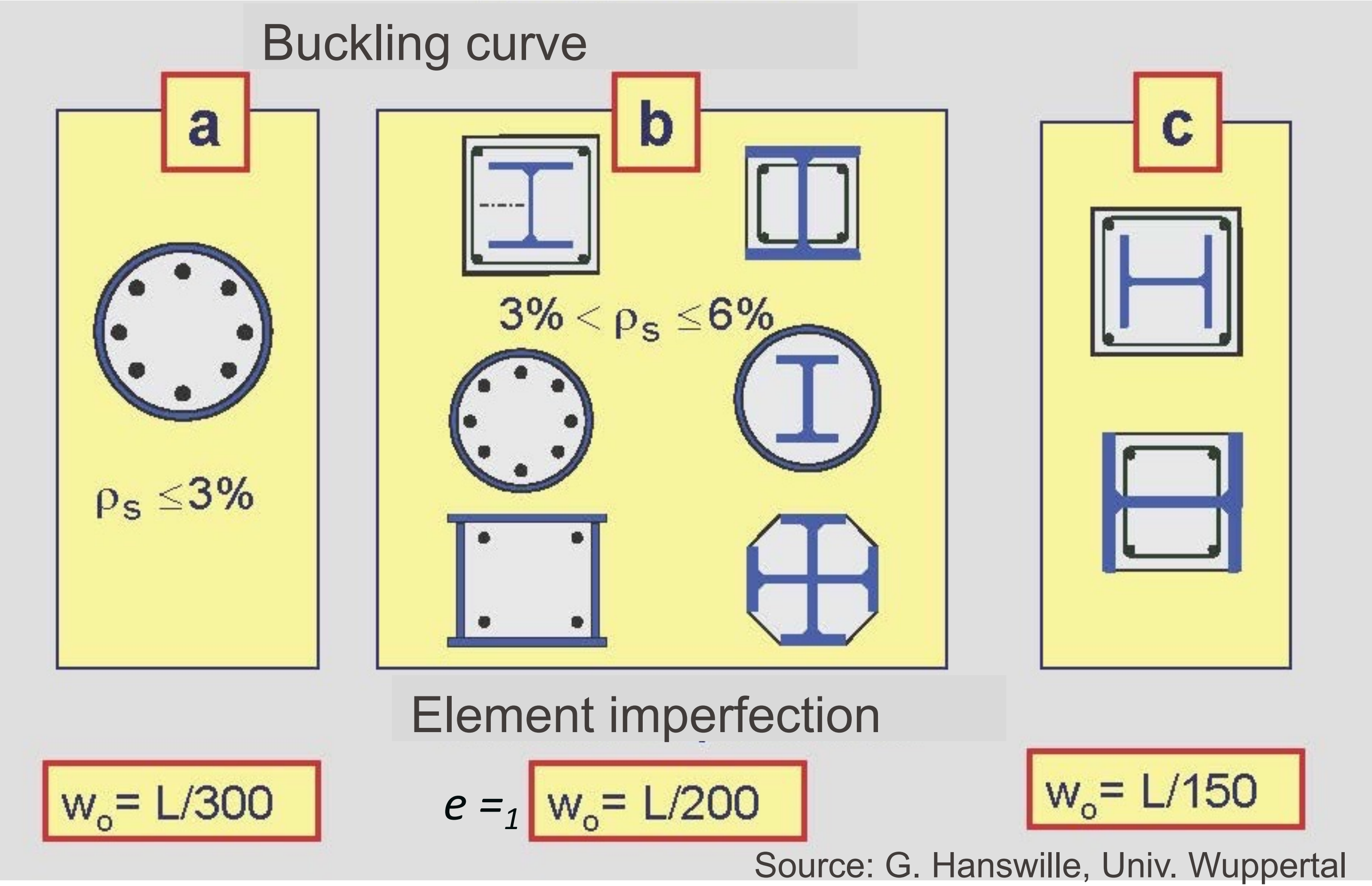
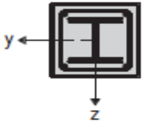
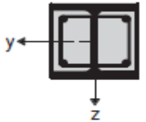
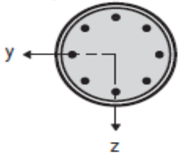
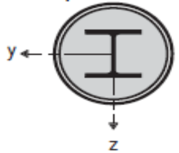
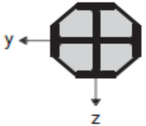


Tableau 6.5 : Courbes de flambement et imperfections de barres pour les poteaux mixtes

Section	Limites	Axe de flambement	Courbe de flambement	Imperfection d'élément
Profilé enrobé de béton 		y-y	b	$e_1 =$ L/200
		z-z	c	L/150
Profilé partiellement enrobé de béton 		y-y	b	L/200
		z-z	c	L/150
Profil creux circulaire et rectangulaire en acier 	$\rho_s \leq 3\%$	quelconque	a	L/300
	$3\% < \rho_s \leq 6\%$	quelconque	b	L/200
Profil creux circulaire en acier avec adjonction d'un profilé en I 		y-y	b	L/200
		z-z	b	L/200
Profilé partiellement enrobé de béton avec profilés en I disposés en croix 		quelconque	b	L/200

Detailed table for all composite columns

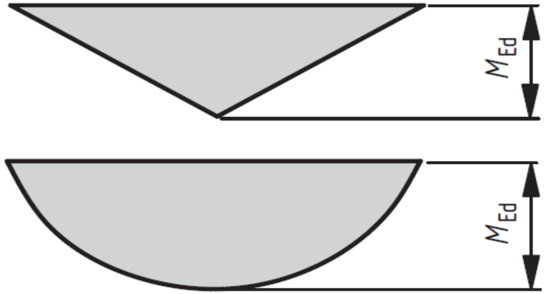
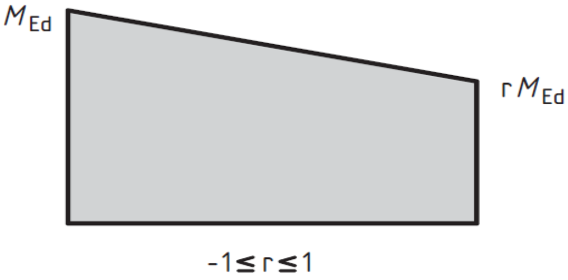
L: length of column

Please note:
Checks with no effect 2ème order only if :

- Biarticulated column held laterally
- No bending

Second-order effects, β factor (EN1994-1-1, tab. 6.4)

Tableau 6.4 : Facteurs β pour la détermination des moments selon la théorie du second ordre

Distribution des moments	Facteurs de moment β	Commentaire
	Moments fléchissants de premier ordre résultant de l'imperfection de l'élément ou de la charge transversale : $\beta = 1,0$	M_{Ed} est le moment fléchissant maximal le long du poteau en ignorant les effets du second ordre
	Moments d'extrémité : $\beta = 0,66 + 0,44r$ mais $\beta \geq 0,44$	M_{Ed} et $r\,M_{Ed}$ sont les moments d'extrémité résultant de l'analyse globale au premier ordre ou au second ordre

Check

Under normal force and bending moment

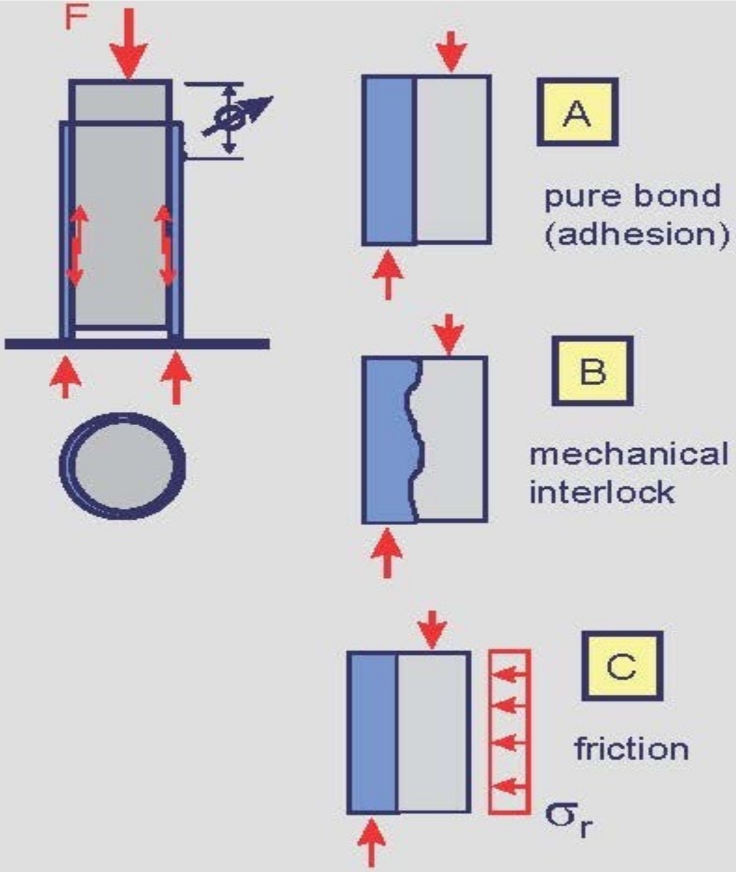
S235, S355 | S420, S460

$$\frac{M_{Ed,II}}{M_{pl,N,Rd}} = \frac{M_{Ed,II}}{\mu_d \cdot M_{pl,Rd}} \leq 0{,}9 \qquad \leq 0.8$$

$M_{pl,Rd}$ design value of the plastic bending resistance (point B on the M - N diagram)

$$M_{Ed,II} = k \cdot M_{e_1,Ed,I} = k\left(M_{Ed,I} + e_1 N_{Ed}\right)$$

Longitudinal shear: 2 Checks



Outside the area of load introduction, longitudinal shear at the interface between concrete and steel should be verified where it is caused by transverse loads and / or end moments. Shear connectors should be provided, based on the distribution of the design value of longitudinal shear, where this exceeds the design shear strength τ_{Rd}.

In absence of a more accurate method, elastic analysis, considering long term effects and cracking of concrete may be used to determine the longitudinal shear at the interface.

Source: G. Hanswille, Univ. Wuppertal

Longitudinal shearing and force introduction

(EN 1994 § 6.7.4, SIA 264 § 5.3.6)

No mechanical connection is required if $\tau_{Ed} < \tau_{Rd}$

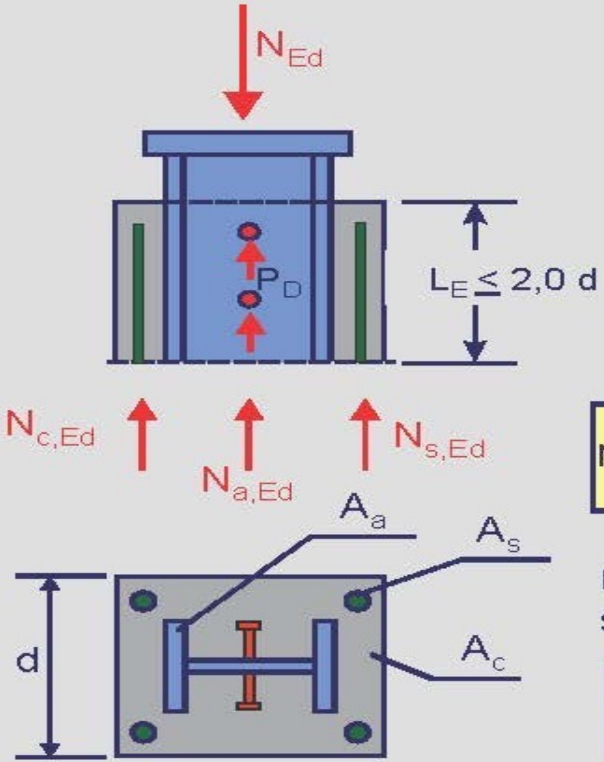
Tableau 6.6 : Résistance au cisaillement de calcul τ_{Rd}

Type de section	τ_{Rd} (N/mm^2)
Profilés en acier totalement enrobés de béton	0,30
Profils creux circulaires remplis de béton	0,55
Profils creux rectangulaires remplis de béton	0,40
Semelles de profilés partiellement enrobés	0,20
Âmes de profilés partiellement enrobés	0,00

2) Checking introduction of forces (plastic model):

- If front plate in contact with concrete, not necessary
- Otherwise, studs, etc. to be sized (in proportion to the $N_{pl\ i}$) and placed within $L_{intro} \leq 2d$

Introduction of forces in steel section only



load introduction by headed studs within the load introduction length L_E

$$\boxed{L_E \leq \begin{cases} 2d \\ L/3 \end{cases}}$$

d minimum transverse dimension of the cross-section

L member length of the column

sectional forces of the cross-section :

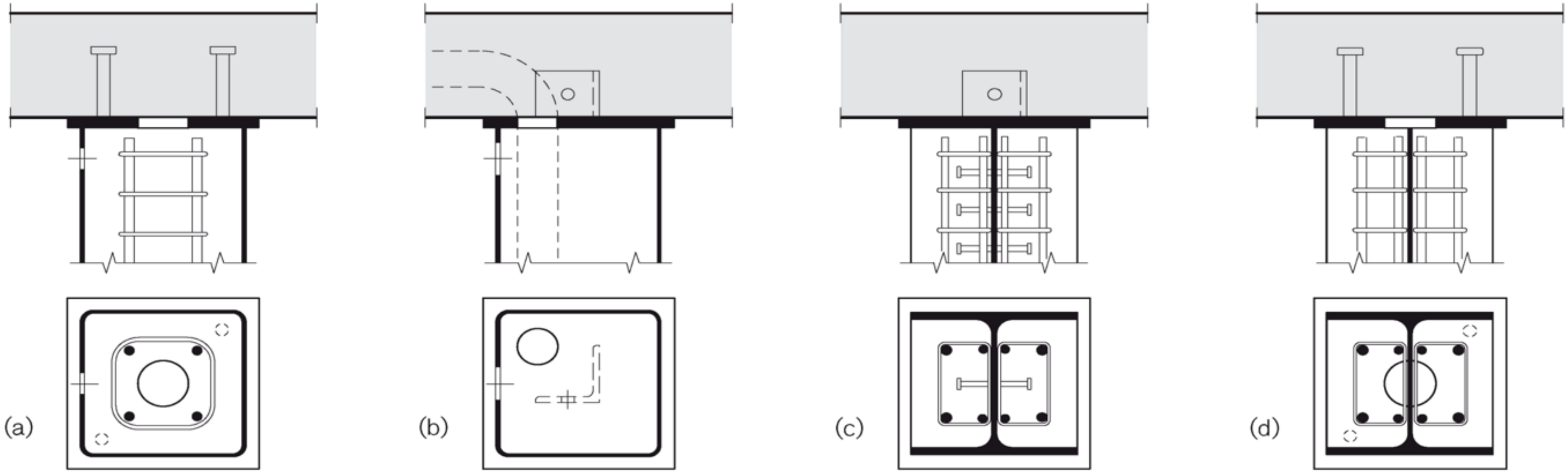
$$N_{a,Ed} = N_{Ed}\frac{N_{pl,a}}{N_{pl,Rd}} \qquad N_{s,Ed} = N_{Ed}\frac{N_{pl,s}}{N_{pl,Rd}} \qquad N_{c,Ed} = N_{Ed}\frac{N_{pl,c}}{N_{pl,Rd}}$$

required number of studs n resulting from the sectional forces $N_{Ed,c} + N_{Ed,s}$:

$$V_{L,Ed} = N_{c,Ed} + N_{s,Ed} = N_{Ed}\left[1 - \frac{N_{pl,a}}{N_{pl,Rd}}\right] \qquad \boxed{V_{L,Rd} = n\,P_{Rd}}$$

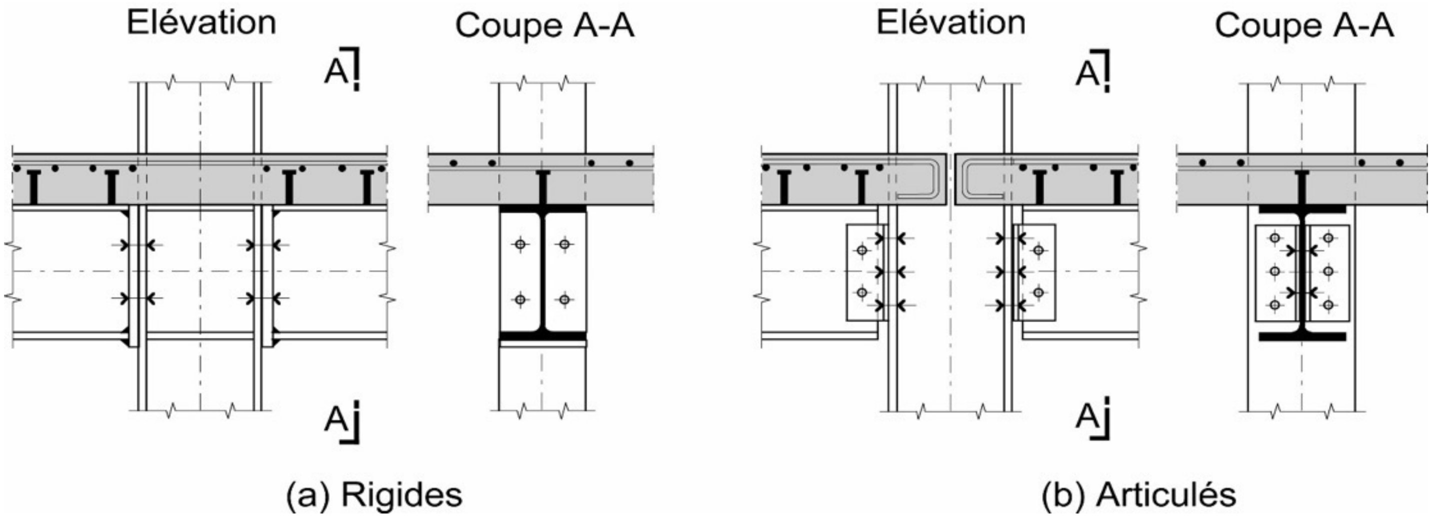
Source: G. Hanswille, Univ. Wuppertal

Detail of connections, interrupted column



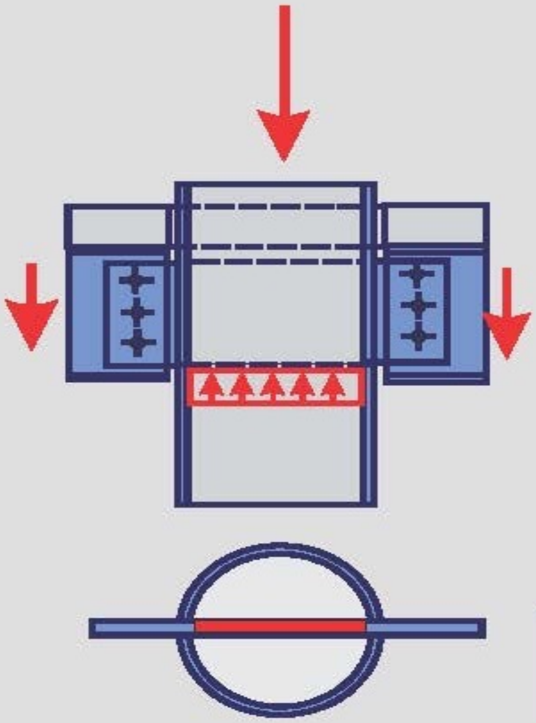
source: SZS

Detail of connections, continuous column



source: TGC 11, fig. 13.17

Detail of connections, continuous column



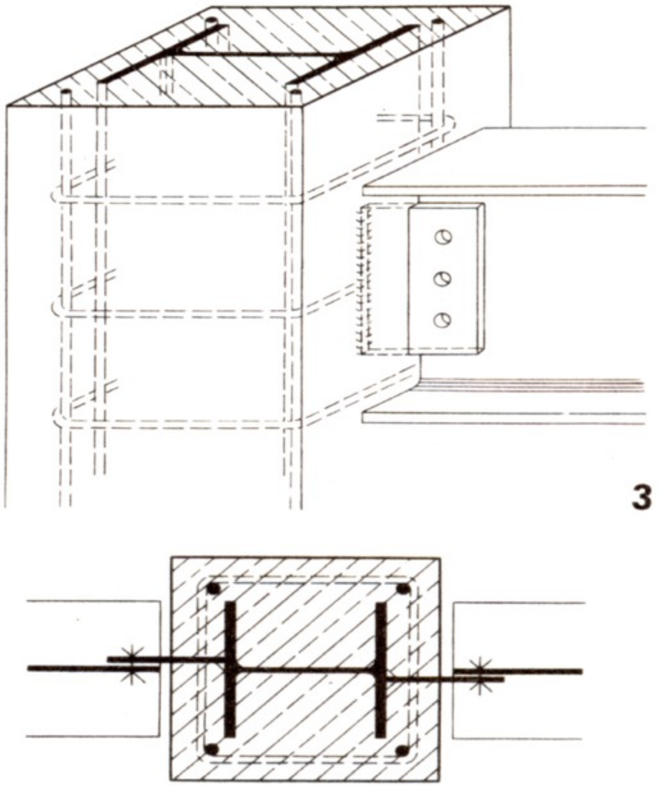
Load introduction with gusset plates

Increases the area to transfer τ and also transfer by additional pressure

Source: G. Hanswille, Univ. Wuppertal

Detail of connections, continuous column

- Find the error (constructive detail)



source: G. Hanswille, Univ. Wuppertal

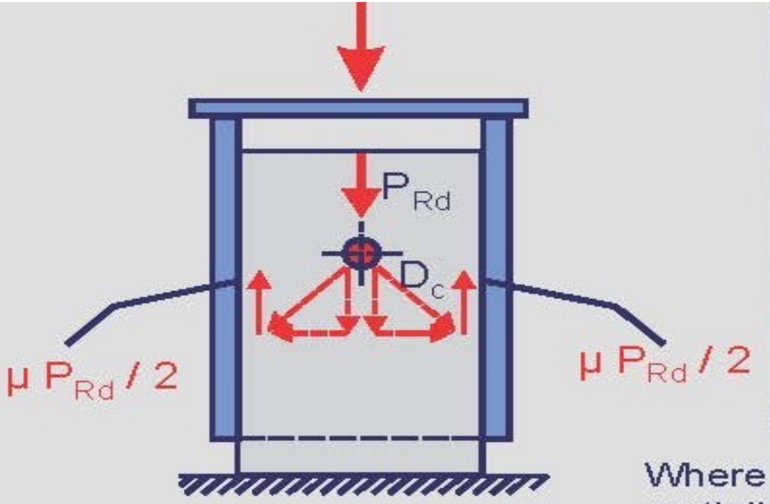
Thank you

Prof. A. Nussbaumer

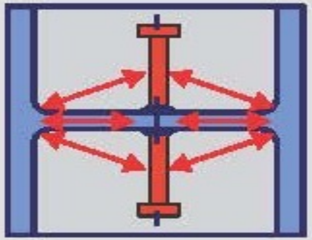
RESSLab

The Mecca Clock Tower
Source: www.mayr-ludescher.de

Appendix: additional shear strength on core in coated section



$$V_{L,Rd} = P_{Rd} + V_{LR,Rd}$$

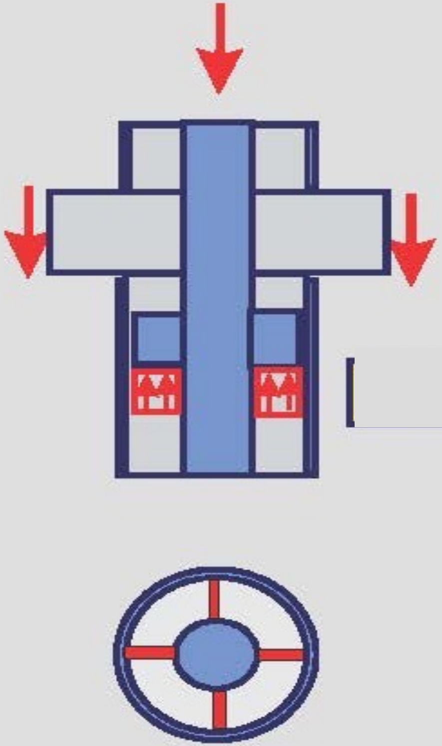


$$V_{LR,Rd} = \mu P_{Rd}$$

Where stud connectors are attached to the web of a fully or partially concrete encased steel I-section or a similar section, account may be taken of the frictional forces that develop from the prevention of lateral expansion of the concrete by the adjacent steel flanges. This resistance may be added to the calculated resistance of the shear connectors. The additional resistance may be assumed to be on each flange and each horizontal row of studs, where μ is the relevant coefficient of friction that may be assumed. For steel sections without painting, μ may be taken as 0,5. P_{Rd} is the resistance of a single stud.

Source: G. Hanswille, Univ. Wuppertal

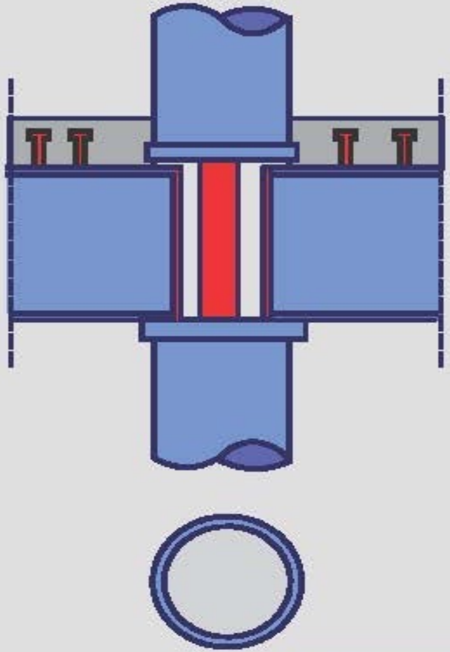
Appendix: Assembly details



Post Tower Bonn

Source: G. Hanswille, Univ. Wuppertal

Appendix: Assembly details



Load introduction with partially loaded end plates

Source: G. Hanswille, Univ. Wuppertal