Specific Rules for Design and Detailing of Steel Buildings

Illustrations of Design

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General

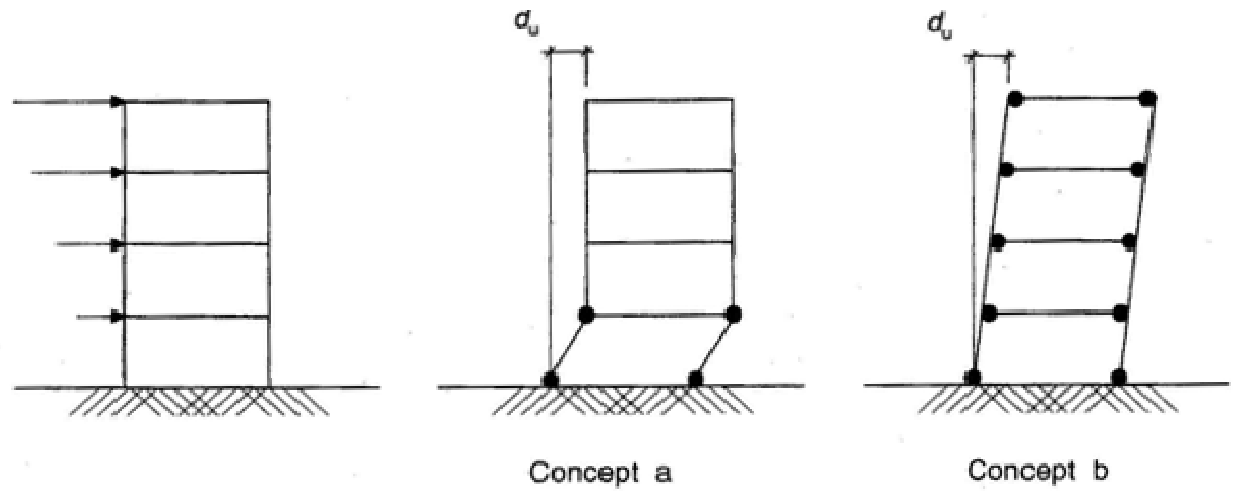
Design objective for dissipative structure:

a global plastic mechanism in a decided scheme

Why global?

To have numerous dissipative zones to dissipate more energy

To avoid excessive local plastic deformation as a result of concentration of deformations in few places



concept a $\Leftrightarrow$ concept b

$\theta_{concept\ a} = d_u / h_{1\ \text{étage}}$

$\theta_{concept\ b} = d_u / h_{4\ \text{étages}}$

$\theta_{concept\ b} = \theta_{\ concept\ a} /4$

Why in a decided scheme ?

Because it is not thinkable to have all zones of the structure with ideal characterisics for plastic deformations

General

=> Design of dissipative structure

1. Define the objective: a global mechanism

2. Pay a price for the mechanism to be global:
   - criteria for numerous dissipative zones
   - capacity design of resistances of all elements other than the plastic zones

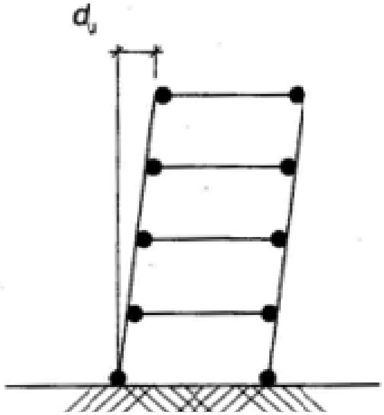
3. Pay a price at local zones: criteria aiming at local ductility

For instance
- In steel: rules for connections
  classes of sections
  plastic rotation capacity
- In composite steel concrete: position of neutral axis

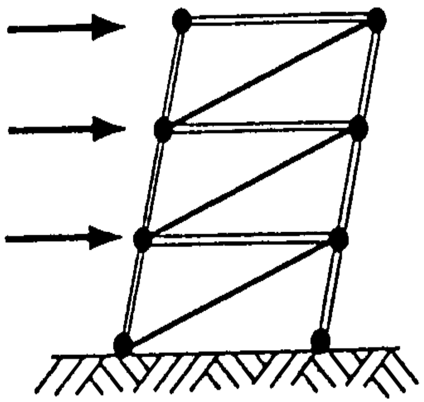
General

Definition of the objective « global plastic mechanism»

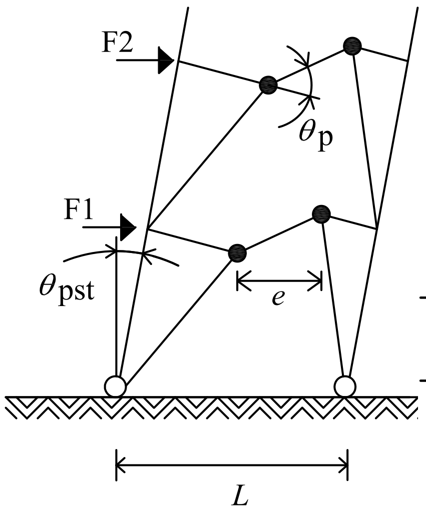
Moment resisting frames: plastic hinges in bending at beam ends

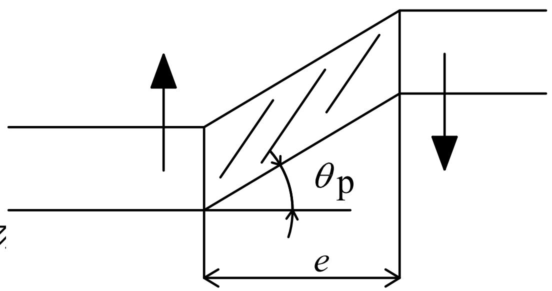


not plastic shear in beams
not plastic in connections
not plastic hinges in columns



Frames with concentric bracings:
diagonals in plastic tension





Frames with eccentric bracings:
dedicated « links »
in plastic shear or bending

General

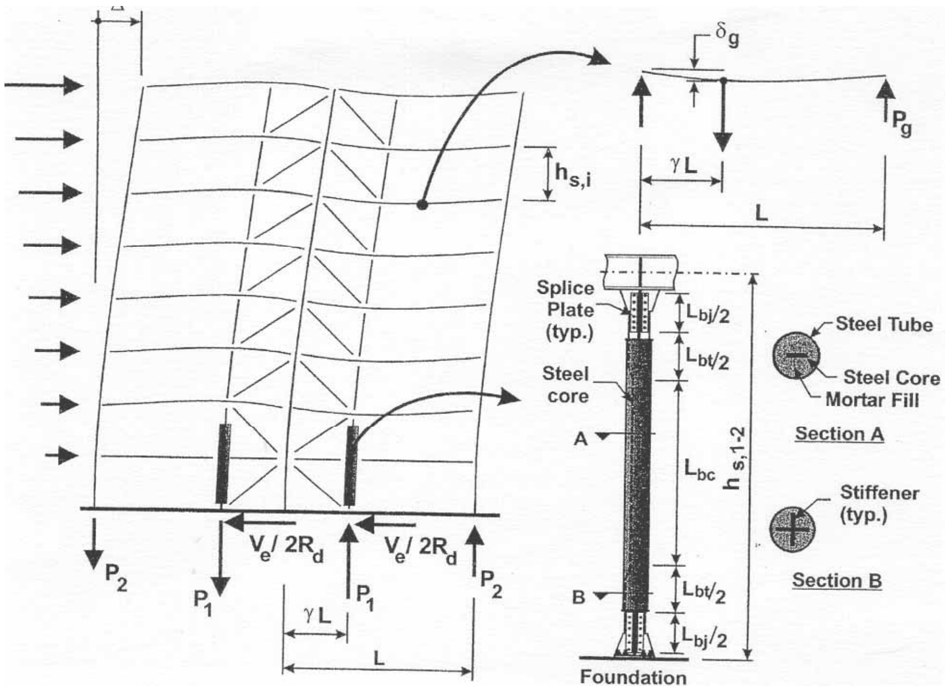
Definition of the objective « global plastic mechanism»

There may be typologies other than the usual ones…

Example

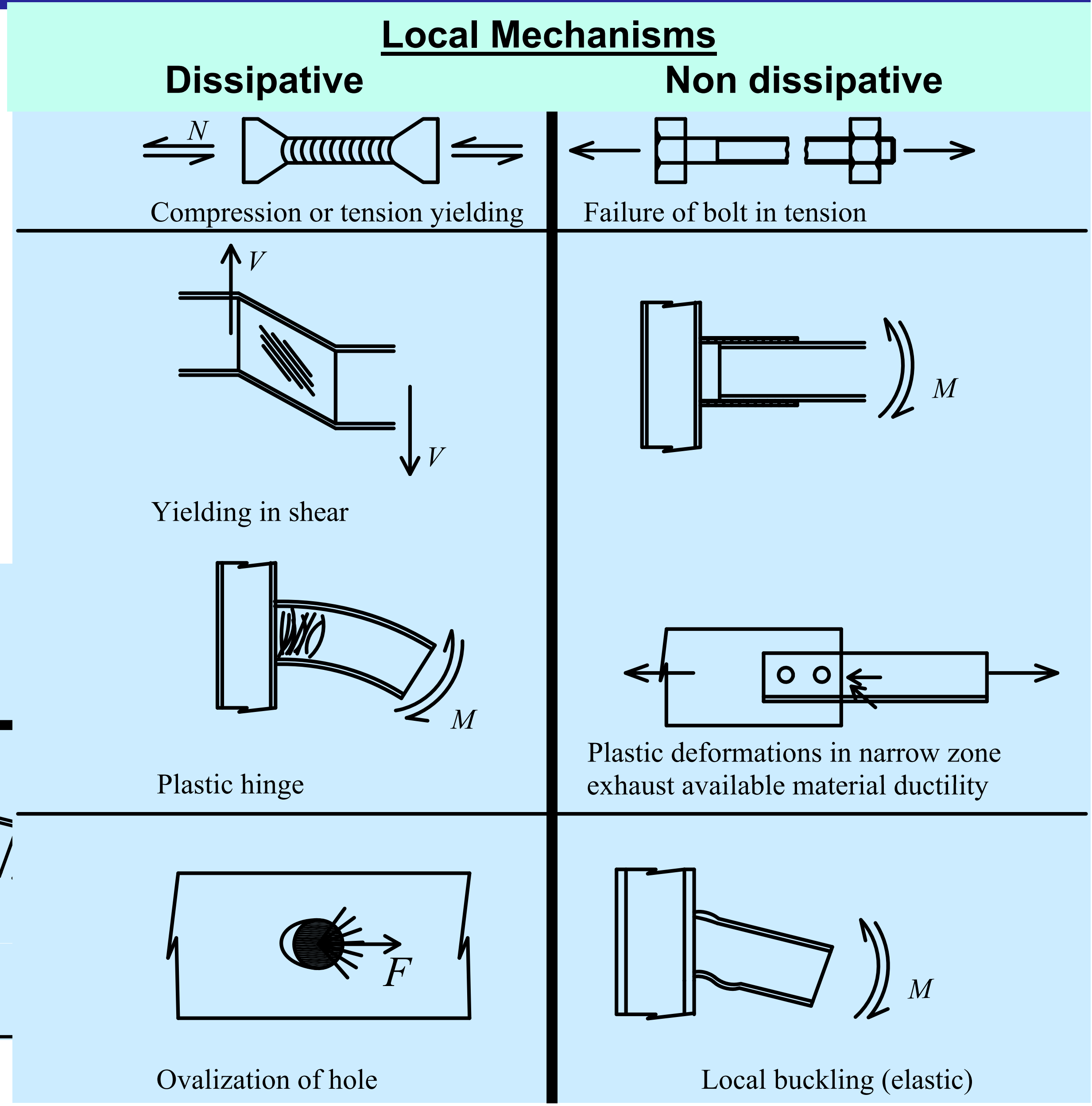
Using « Buckling Restrained Bars » at bottom of frame

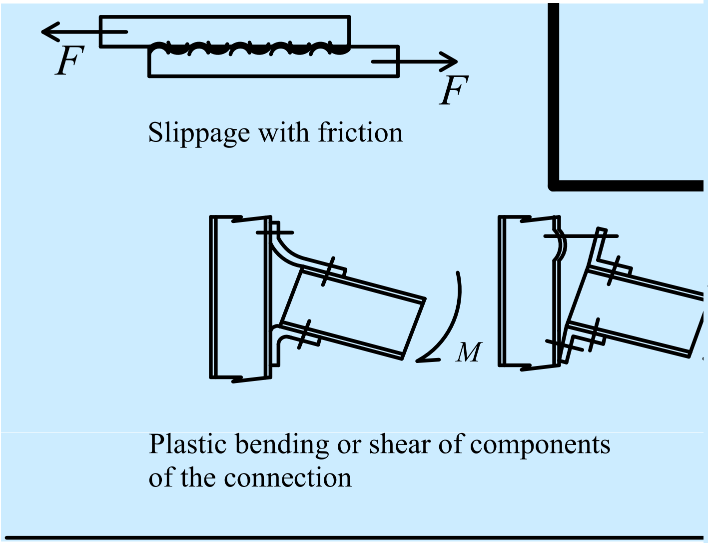
=> similar to reinforced concrete wall: 1 big plastic hinge



General

Local Dissipative & Non dissipative Mechanisms





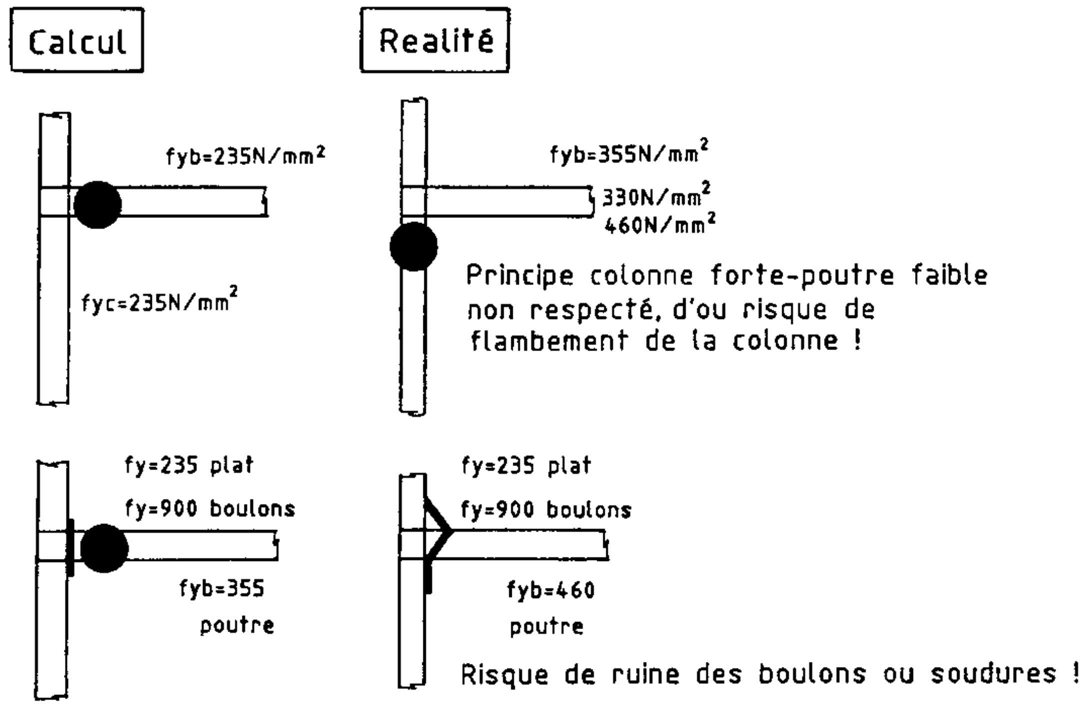
General

Required steel characteristics

- Classical constructional steel
- Charpy toughness: absorbed energy min 27J (at $t°_{usage}$)
- Distribution yield stresses and toughness such that :

  dissipatives zones at intended places

  yielding at those places before the other zones leave the elastic range $f_{ymax} \leq f_{ydesign}$



Correspondance between reality & hypothesis is required

General

Required steel characteristics **Conditions on f_y of dissipative zones**

to achieve $f_{ymax,\ real} \leq f_{ydesign}$

to have a correct reference in capacity design

3 possibilities

a) Compute considering that in dissipative zones: $f_{y,max} = 1{,}1\ \gamma_{ov}\ f_y$

γ_{ov} material overstrength factor f_y : nominal

$\gamma_{ov} = f_{y,real} / f_y$

European rolled sections: $\gamma_{ov} = 1{,}25$

Ex: S235, $\gamma_{ov} = 1{,}25$ => $f_{y,max} = 323\ N/mm^2$

an upper value $f_{y,max}$ is specified for dissipative zones

b) Do design, based on a single nominal yield strength f_y

for dissipative & non dissipative zones

Use nominal f_y for dissipative zones, with specified $f_{y,max}$

Use higher nominal f_y for non dissipative zones and connections

Ex: S235 dissipative zones, with $f_{y,max} = 355\ N/mm^2$

S355 non dissipative zones

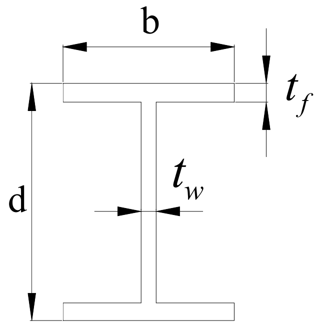
c) $f_{y,max}$ of dissipative zones is measured

is the value used in design => $\gamma_{0v} = 1$

General

TYPE of STRUCTURE	Ductility Class	
	DCM	DCH
Moment resisting frame	4	$5\ \alpha_u / \alpha_1$
Frame with concentric bracings		
diagonal type	4	4
V type	2	2,5
Frame with eccentric bracings	4	$5\ \alpha_u / \alpha_1$
Inverted pendulum	2	$2\ \alpha_u / \alpha_1$
Structures with reinforced concrete core / walls		
Moment resisting frame + concentric bracings	4	$4\ \alpha_u / \alpha_1$
Concrete infills not connected in contact with frame	2	2
Concrete infills connected => composite		
Concrete infills isolated from the frame	4	$5\ \alpha_u / \alpha_1$

General



- Criteria applicable to the primary structure
- Criteria for local ductility:

  Free choice: local dissipative zones can be
  - => in structural elements
  - => in connections But effectiveness to demonstrate

    Semi-rigid or partial strength connections OK if:
    - adequate rotation capacity <=>global deformations
    - members framing into connections are stable
    - effect of connections deformations on drift analysed
- Plastic deformation capacity of elements

  (compression, bending) => limitation of b/t_f or c/t_f

=> classes of sections of Eurocode 3

Ductility Class	Behaviour factor q	Cross Sectional Class
DCH	$q > 4$	class 1
DCM	$2 \leq q \leq 4$	class 2
DCM	$1,5 \leq q \leq 2$	class 3
DCL	$q \leq 1,5$	class 1, 2, 3, 4

Illustration of Design 1

Steel Moment Resisting Frame

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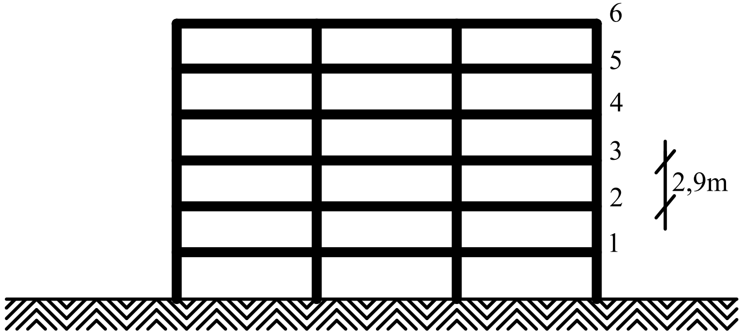
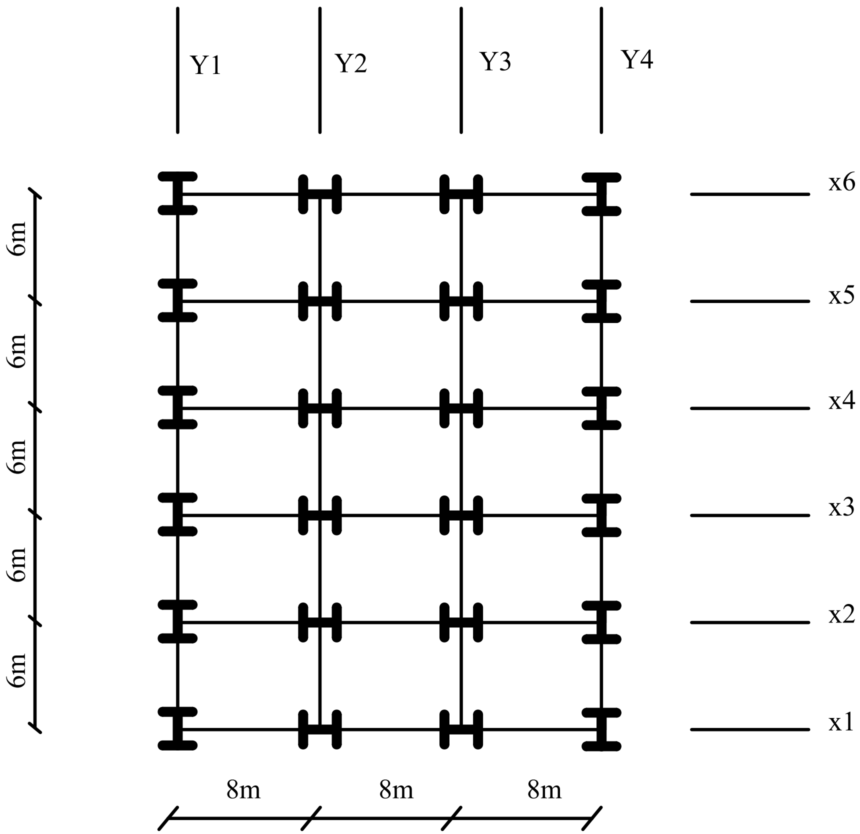
Steel Moment Resisting frame





Design objectives

- Plastic hinges in beams or their connections, not in the columns ‘Weak Beam-Strong Column’ WBSC
  Global ductility

- Plastic rotation capacity at beam ends: 25 mrad DCM 35 mrad DCH
  Local Ductility
  => classes of sections

Seismic resistance

Peripheral and interior moment frames in 2 directions

Max q: 5 α_u/α_1=5x1,3= 6,5

q = 4 chosen

=> DCM Sections class 1 or 2

Steel Moment Resisting frame

Design steps

Preliminary design

Define minimum beam sections deflection & resistance criteria under gravity loading

Iterations until all design criteria fulfilled

- column sections checking ‘Weak Beam Strong Column’
- seismic mass m = ($G + \psi_{Ei}\ Q$)
- period T by code formula
- resultant base shear F_b => storey forces
- static analysis one plane frame
  ‘lateral loads’ magnified by torsion factor δ => E
- static analysis gravity loading ($G + \psi_{2i}\ Q$)
- stability check P-Δ effects parameter θ
  in seismic loading situation: gravity loading= $G + \psi_{2i}\ Q$
- displacement checks under ‘service’ earthquake = 0,5 x design EQ
- combination action effects E + G + $\psi_{2i}\ Q$
- design checks: resistance of sections instability of elements
- Design of connections
- Design with RBS Reduced Beam Sections

Steel Moment Resisting frame

Site and building data

Seismic zone a_{gR}= 2,0 m/s^2

Importance of the building; office building, γ_I=1,0 => a_g= 2,0 m/s^2

Service load Q = 3 kN/m^2

Design spectrum; type 1

Soil B => from code: S = 1,2 T_B = 0,15s T_C = 0,5s T_D = 2s

Behaviour factor: q = 4

Beams

Assumed fixed at both ends. Span l = 8m

Deflection limit: f = l /300 under G+Q

f = pl^4 / 384EI= l/300

min beam sections:

- direction x : IPE400 Wpl = 1307.10^3 mm^3 I =23130.10^4 mm^4
- direction y : IPE360 Wpl = 1019.10^3 mm^3 I =16270. 10^4 mm^4

=> iterations

Steel Moment Resisting frame

After iterations

Beams direction x: IPE 500 I= 48200.10^4 mm^4 Wpl = 2194.10^3 mm^3

direction y: IPEA 450 I= 29760.10^4 mm^4 Wpl= 1494.10^3 mm^3

Columns: HE340M: $I_{\text{strong axis}} = I_y = 76370.10^4$ mm^4

$I_{\text{weak axis}} = I_z = 19710.10^4$ mm^4

$W_{\text{pl,strong axis}} = 4718.10^3$ mm^3 $W_{\text{pl,weakaxis}} = 1953.10^3$ mm^3

Weak Beam-Strong Column (WBSC) check: $\sum M_{\text{Rc}} \geq 1{,}3 \sum M_{\text{Rb}}$

All S355 => criteria is: $\sum W_{\text{pl, columns}} \geq 1{,}3 \sum W_{\text{pl, beams}}$

Seismic mass above ground considered as fixity level

$m = G + \psi_{Ei}\, Q = 3060.10^3$ kg $\psi_{2i} = 0{,}3$ $\varphi = 0{,}5$ $\psi_{Ei} = 0{,}15$

Note: steel frame = 7,5 % seismic mass

could be taken constant in iterations

$G + \psi_{Ei}\, Q$ floors = 70% seismic mass

Steel Moment Resisting frame

Evaluation of seismic design shear using the 'lateral forces' method

- **Estimated fundamental period T of the structure:**
  $T = C_t\ H^{3/4}$ $C_t = 0{,}085$ $H = 6\text{x}\ 2{,}9\ \text{m} = 17{,}4\ \text{m}$
  $\Rightarrow T = 0{,}085 \times 17{,}4^{3/4} = 0{,}72\ \text{s}$
- **Design pseudo acceleration $S_d\ (T)$:** $T_C < T < T_D$
  $S_d\ (T) = (2{,}5\ a_g \times S \times T_C)/(q \times T) = (2{,}5\text{x}2\ \text{x}1{,}2\text{x}\ 0{,}5)/(4\text{x}0{,}72) = 1{,}04\ \text{m/s}^2$
- **Seismic design shear F_{bR}**
  $F_{bR} = m\ S_d\ (T)\ \lambda = 3060.10^3 \times 1{,}04 \times 0{,}85 = 2705.10^3\ \text{N} = 2705\ \text{kN}$
- **6 same frames** **floor diaphragm effective**
  $\Rightarrow$ seismic design shear F_{bX} in one frame: $F_{bX} = F_{bR}/6 = 451\ \text{kN}$
- **Torsion by amplifying F_{bX} by $\delta = 1 + 0{,}6x/L$ $\delta = 1 + 0{,}6 \times 0{,}5 = 1{,}3$**
  F_{bX} including torsion: $F_{bX} = 586\ \text{kN}$

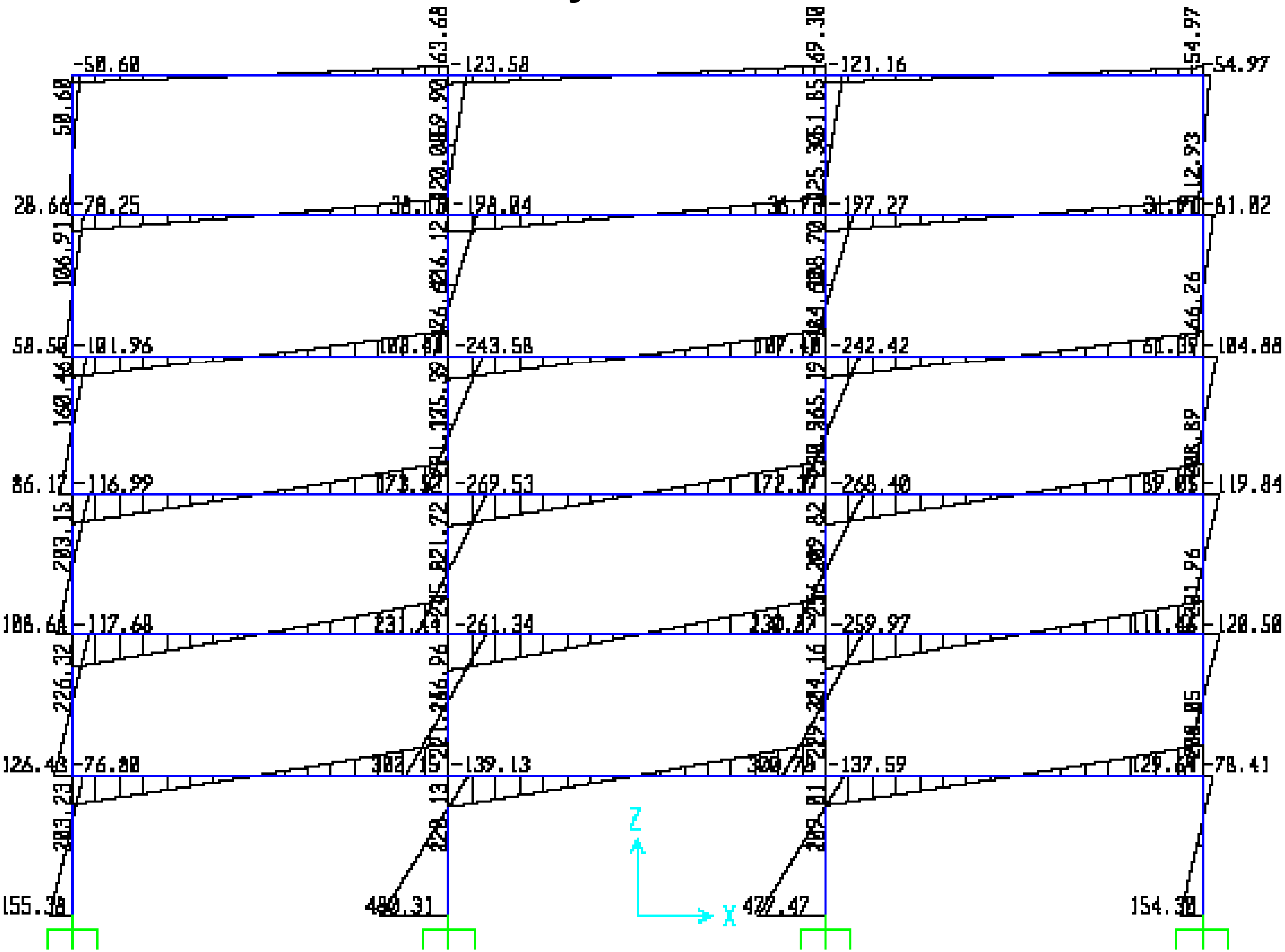
Storey forces **Triangular Distribution in kN**

$F1 = 27{,}9$ $F2 = 55{,}8$ $F3 = 83{,}7$ $F4 = 111{,}6$ $F5 = 139{,}5$ $F6 = 167{,}5$

Steel Moment Resisting frame

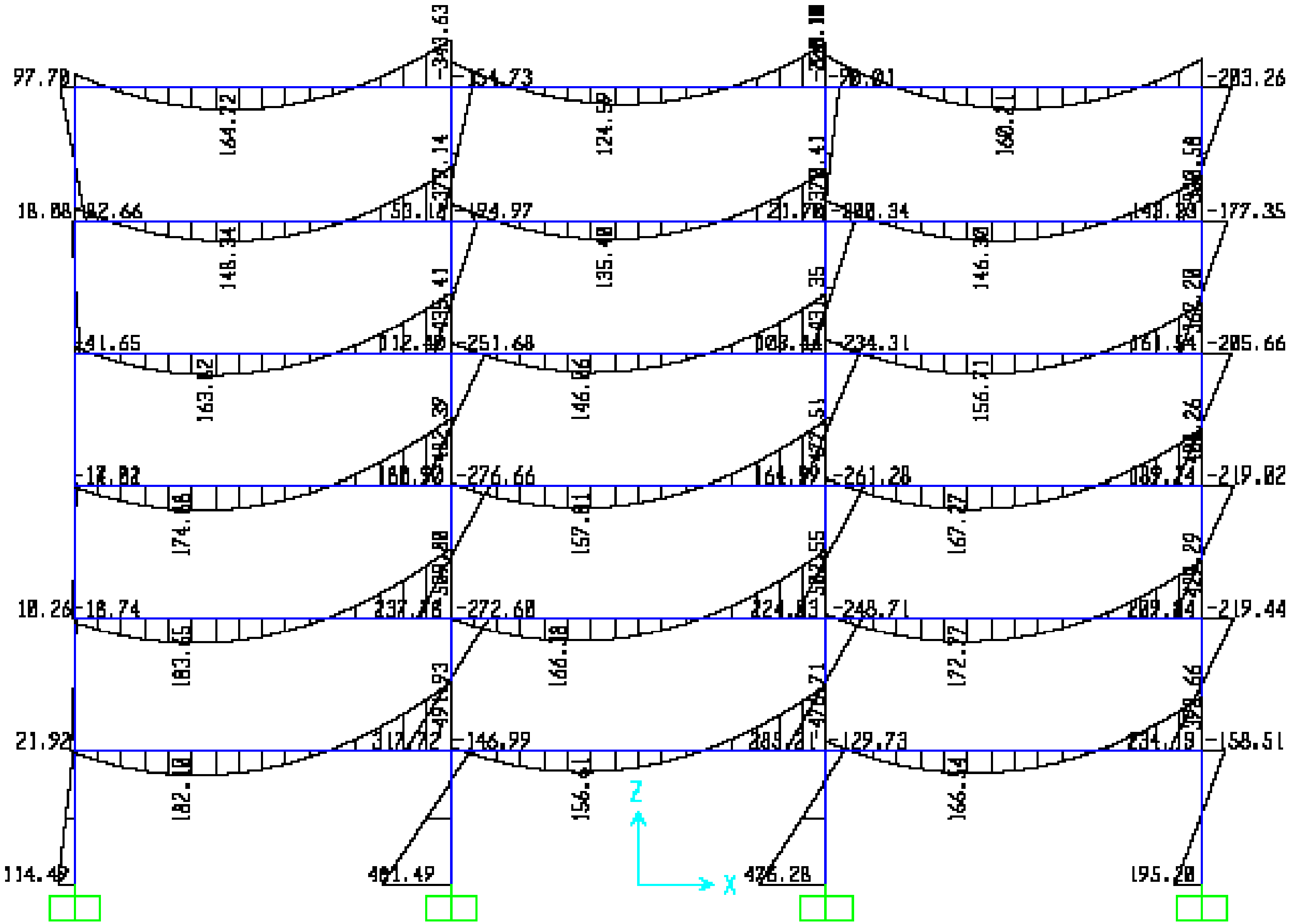
Results of the lateral force method analysis

Diagram Of Bending Moments Under E



Steel Moment Resisting frame

Bending moment diagram: E + G + ψ_{2i} Q
Units: kNm



Steel Moment Resisting frame

Ultimate limit state. No-collapse requirement

Resistance condition $R_d \geq E_d$

R_d design resistance

E_d design value of action effect in seismic design situation:

$E_d = \Sigma\, G_{k,j}$ « + » P « + » $\Sigma \psi_{2i}.Q_{ki}$ « + » $\gamma_1\, A_{Ed}$

In MRF: Check plastic hinges at beam ends $M_{pl,Rd} \geq M_{Ed}$

Limitation of 2^nd^ order effects

If necessary, 2nd order effects are taken into account in the value of E_d

2nd order moments $P_{tot}\, d_r$ ⇔ 1st order moments $V_{tot}\, h$ at every storey

V_{tot} total seismic shear at considered storey

H storey height

P_{tot} total G at and above the storey

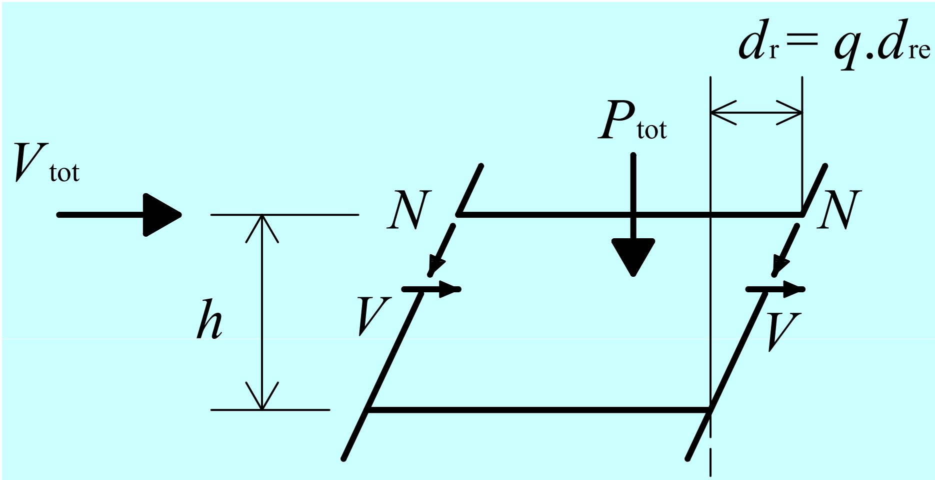
d_r drift based on $d_s = q\, d_e$

Rules

- $\theta \leq 0{,}1$ => P- Δ effects negligible
- $0{,}1 < \theta \leq 0{,}2$

=> multiply action effects by $1/(1-\theta)$

- Always: $\theta \leq 0{,}3$



Steel Moment Resisting frame

Damage limitation

- **Non-structural elements of brittle materials attached to the structure:**

$$d_r \nu \leq 0{,}005\, h$$

- **Ductile non-structural elements:** $d_r \nu \leq 0{,}0075\, h$
- **Non-structural elements not interfering with structural deformations (or no non-structural elements):** $d_r \nu \leq 0{,}010\, h$

d_r design interstorey drift

h storey height;

v reduction factor for lower return period of the seismic action associated with the damage limitation requirement.

Recommended :

v = 0,4 for importance classes III and IV

v = 0,5 for importance classes I and II

Steel Moment Resisting frame

Results of the lateral force method analysis

Lateral force method		= $E_s + G + \psi_{Ei}.Q$								$G + \psi_{Ei}.Q =$ 35,42 kN/m						
Storey	Absolute displacement of the storey : d_i [m]		Design interstorey drift $(d_i - d_{i-1})$: d_r[m]		Storey lateral forces E_i : V_i [kN]		Shear at storey E_i : V_{tot} [kN]		Total cumulative gravity load at storey E_i : P_{tot} [kN]		Storey height E_i : h_i [m]		Interstorey drift sensitivity coefficient $\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \le 0{,}10$	
E_0	d_0	0	d_{r0}											
E_1	d_1	0,033	d_{r1}	0,033	V_1	27,9	$V_{tot\,1}$	586,0	$P_{tot\,1}$	5100	h_1	2,9	θ_1	0,100
E_2	d_2	0,087	d_{r2}	0,054	V_2	55,8	$V_{tot\,2}$	558,1	$P_{tot\,2}$	4250	h_2	2,9	θ_2	0,141
E_3	d_3	0,139	d_{r3}	0,052	V_3	83,7	$V_{tot\,3}$	502,3	$P_{tot\,3}$	3400	h_3	2,9	θ_3	0,122
E_4	d_4	0,184	d_{r4}	0,044	V_4	111,6	$V_{tot\,4}$	418,6	$P_{tot\,4}$	2550	h_4	2,9	θ_4	0,093
E_5	d_5	0,216	d_{r5}	0,033	V_5	139,5	$V_{tot\,5}$	307,0	$P_{tot\,5}$	1700	h_5	2,9	θ_5	0,062
E_6	d_6	0,238	d_{r6}	0,021	V_6	167,5	$V_{tot\,6}$	167,5	$P_{tot\,6}$	850	h_6	2,9	θ_6	0,037

Steel Moment Resisting frame

<u>2^{nd} order effects</u>

$\theta_2 = 0{,}141$

$\theta_3 = 0{,}122$

=> increase M, V, N, d_r in elements at storey 2 and 3

=> make resistance & deformation checks with increased values

<u>Checks under service earthquake</u>

Interstorey drifts D_s max: $D_s = 0{,}5 \times 0{,}054 \times 1/(1- \theta) = 0{,}031$m

Limit: $0{,}10\ h = 0{,}1 \times 2{,}9$ m $= 0{,}029$m $\approx 0{,}31$ m

Steel Moment Resisting frame

Dynamic analysis Modal superposition method

A single plane frame in each direction X or Y is analysed

Torsion effects by $\delta = 1{,}3$

=>a_g for the analysis: $a_g = 2 \times 1{,}3 = 2{,}6\ m/s^2$

Output:

$T_1 = 1{,}17\ s > 0{,}72s$	$F_{bX} = 586\ kN$	lateral force method	one frame
	$F_{bX} = 396\ kN$	dynamic response	one frame

More refined analysis => economy

θ does not differ much

Interstorey drift reduced D_s max:

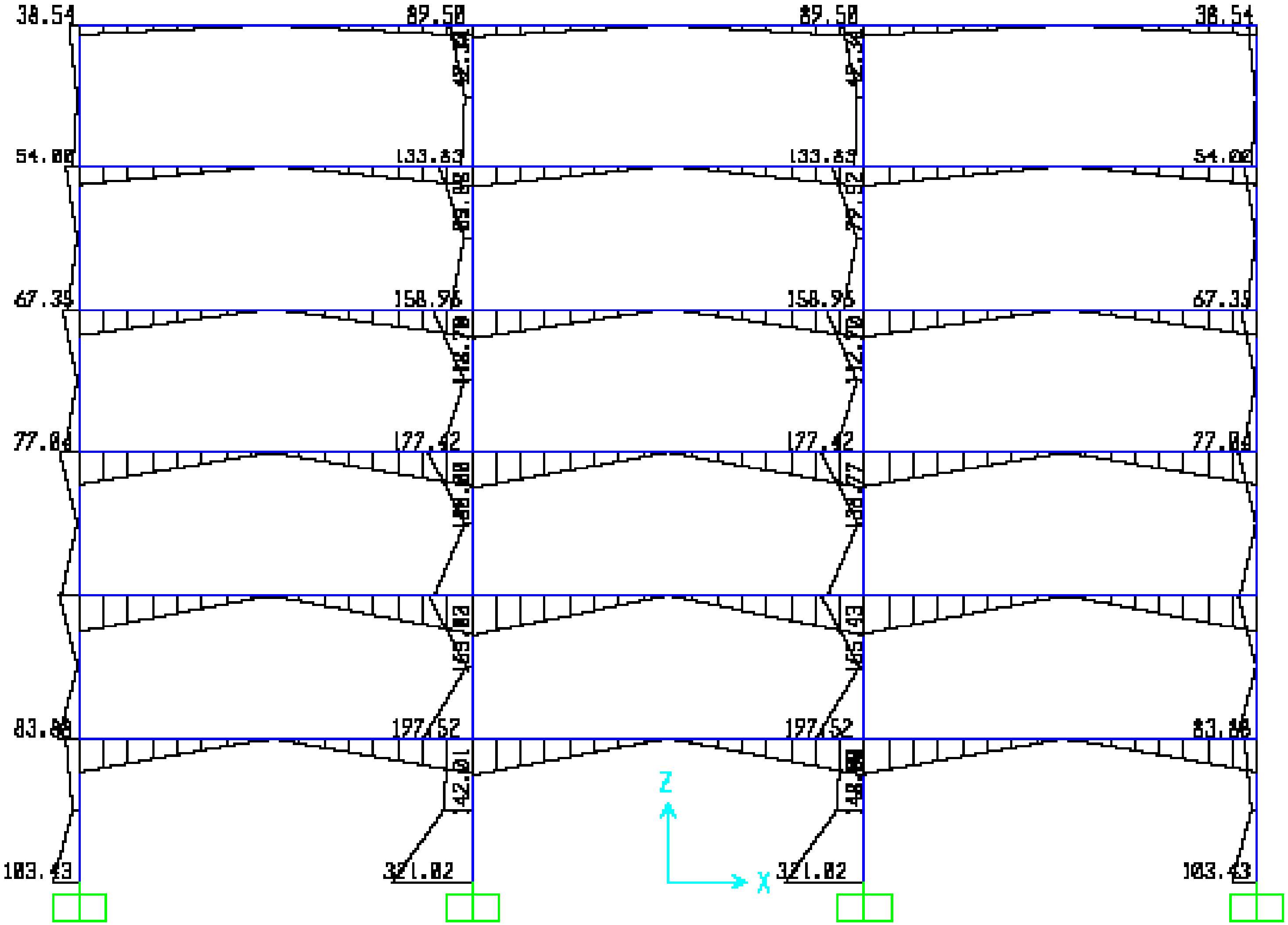
$D_s = 0{,}5 \times 0{,}035 \times 1/\ (1- 0{,}137) = 0{,}020m$

Limit: $0{,}10\ h = 0{,}1 \times 2{,}9\ m = 0{,}029m > 0{,}02\ m$ => OK

Steel Moment Resisting frame

Results of the modal superposition method

Diagram Of Bending Moments Under E



Steel Moment Resisting frame

Results of the modal superposition method

Modal superposition			$= E_s + G + \psi_{Ei}.Q$							$G + \psi_{Ei}.Q =$ 35,42		kN/m		
Storey	Absolute displacement of the storey : d_i [m]		Design interstorey drift $(d_i - d_{i-1})$: d_r[m]		Storey lateral forces E_i : V_i [kN]		Shear at storey E_i : V_{tot} [kN]		Total cumulative gravity load at storey E_i : P_{tot} [kN]		Storey height E_i : h_i [m]		Interstorey drift sensitivity coefficient $\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0{,}10$	
E_0	d_0	0	d_{r0}											
E_1	d_1	0,022	d_{r1}	0,022	V_1	26,6	$V_{tot\,1}$	396,2	$P_{tot\,1}$	5100	h_1	2,9	θ_1	0,099
E_2	d_2	0,057	d_{r2}	0,035	V_2	42,9	$V_{tot\,2}$	369,7	$P_{tot\,2}$	4250	h_2	2,9	θ_2	0,137
E_3	d_3	0,090	d_{r3}	0,033	V_3	50,0	$V_{tot\,3}$	326,8	$P_{tot\,3}$	3400	h_3	2,9	θ_3	0,118
E_4	d_4	0,117	d_{r4}	0,027	V_4	61,1	$V_{tot\,4}$	276,7	$P_{tot\,4}$	2550	h_4	2,9	θ_4	0,086
E_5	d_5	0,137	d_{r5}	0,020	V_5	85,0	$V_{tot\,5}$	215,6	$P_{tot\,5}$	1700	h_5	2,9	θ_5	0,054
E_6	d_6	0,148	d_{r6}	0,012	V_6	130,6	$V_{tot\,6}$	130,6	$P_{tot\,6}$	850	h_6	2,9	θ_6	0,027

Steel Moment Resisting frame

$$N_{\mathrm{Ed}} = N_{\mathrm{Ed,G}} + 1{,}1\gamma_{\mathrm{ov}}\,\Omega N_{\mathrm{Ed,E}}$$

$$M_{\mathrm{Ed}} = M_{\mathrm{Ed,G}} + 1{,}1\gamma_{\mathrm{ov}}\,\Omega M_{\mathrm{Ed,E}}$$

$$V_{\mathrm{Ed}} = V_{\mathrm{Ed,G}} + 1{,}1\gamma_{\mathrm{ov}}\,\Omega V_{\mathrm{Ed,E}}$$

Elements checks

Action effects to consider are:

They take into account:

- **Section overstrength Ω = $M_{pl,Rd}$ / M_{Ed}**
- **Material overstrength $f_{y,real}$ / $f_{y,nominal}$ = γ_{ov}**

Column buckling

Buckling length = 2,9 m = storey height

$N_{b,Rd}$ = 9529 kN > 3732 kN at ground level OK

Plastic hinges at column basis

Interaction M – N Eurocode 3 (EN1993-1-1 cl 6.2.9.1)

$N_{Ed} = G + \psi_{2i}\, Q$ $n = N_{Ed} / N_{pl,Rd}$ = 0,184

a = (A-2bt_f)/A = (31580 – 2 x 309 x 40)/31580 = 0,22 > 0,17 (= n)

$M_{pl,y,Rd} = f_{yd}$ x $W_{pl,y,Rd}$=1674,89 kNm

$M_{N,y,Rd} = M_{pl,y,Rd}$ (1-n)/(1-0,5 a) = 1540 kNm > M_{Ed} = 426 kNm

As $n < a$ => $M_{N,z,Rd} = M_{pl,z,Rd}$ = 693 kNm > M_{Ed} = 114 kNm

=> resisting moments > design action effects M_{Ed} = M (E + G + ψ_{2i} Q)

Steel Moment Resisting frame

Other checks

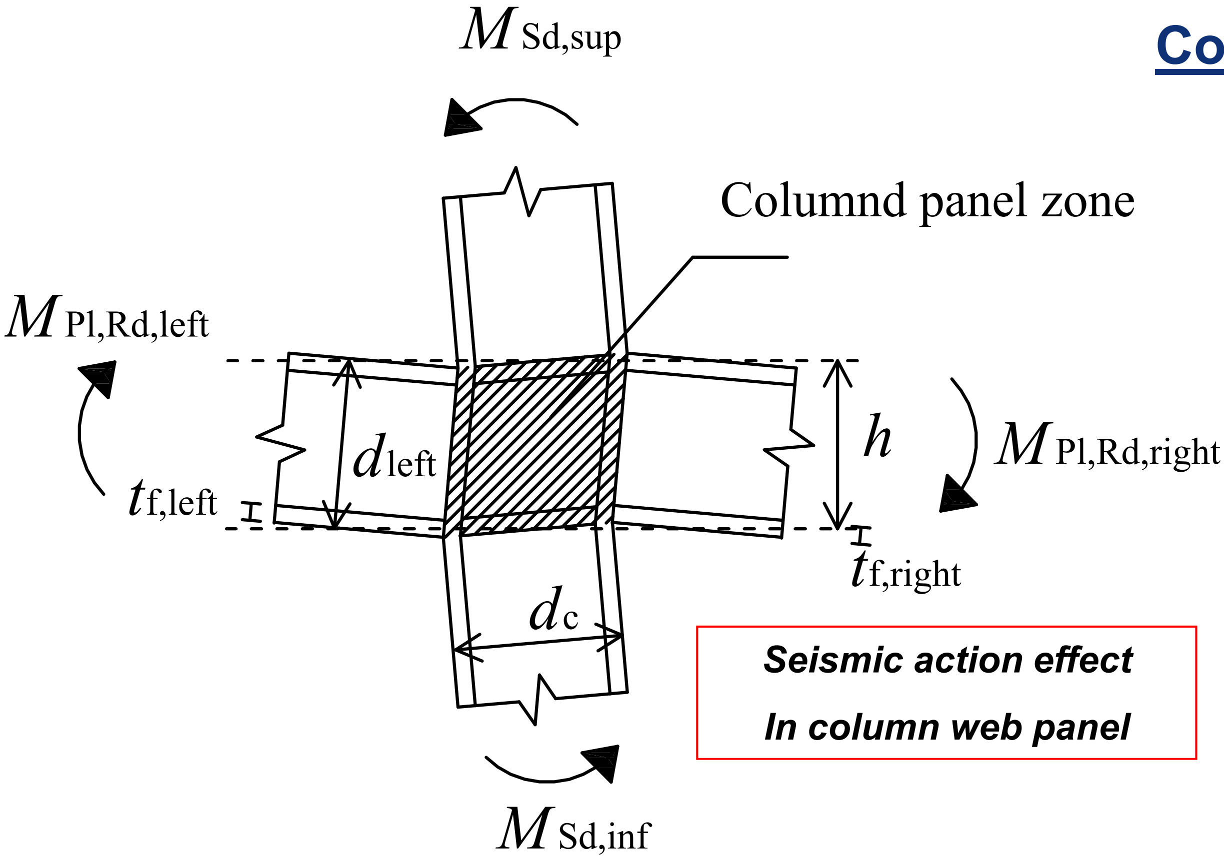
Beam lateral torsional buckling

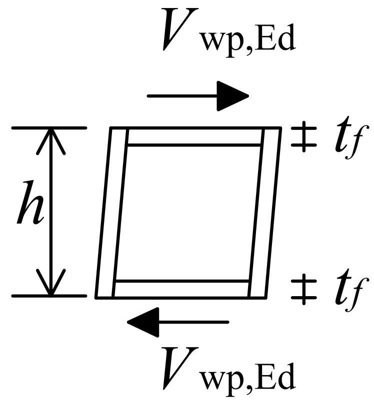
M at beam column connection = M_{pl}

$\Rightarrow$ Lateral supports may be required

Steel Moment Resisting frame

Column web panels





Seismic action effect

In column web panel

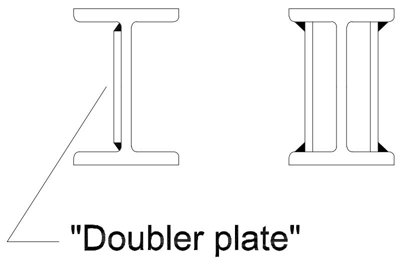
$$V_{wp,Ed} = M_{pl,Rd,\,left} / (d_{left} - 2t_{f,left}) + M_{pl,Rd,\,right} / (d_{right} - 2t_{f,right}) + V_{Ed,\,column}$$

Often: $V_{wp,Ed} > V_{wp,Rd}$

$\Rightarrow$ **« doubler » plates**

welded on web or placed // to web

welds ≥ plate shear resistance



Steel Moment Resisting frame

- **Dissipatives zones can be in beams or in connections**

  Same local ductility requirement:

  $\theta_p = \delta / 0{,}5L > 35$ *mrad* **DCH**

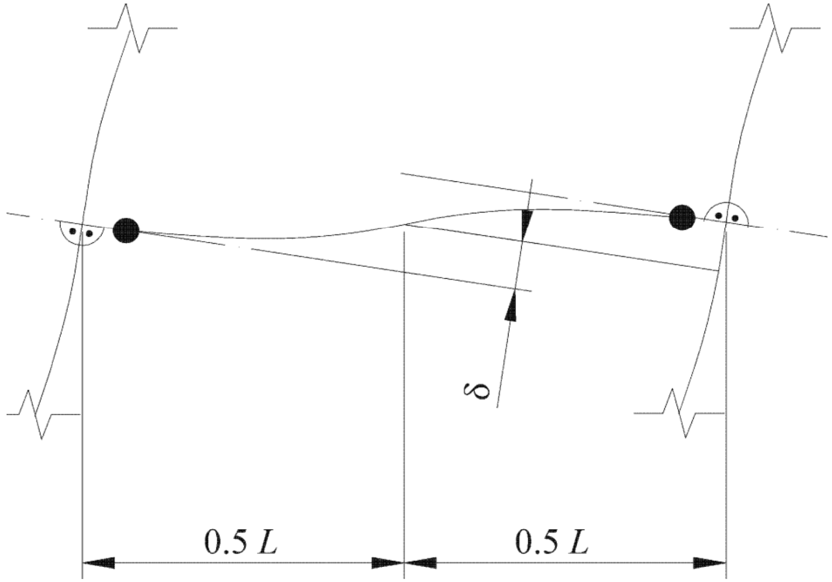
  > 25 *mrad* **DCM (*q* >**

θ_p : plastic rotation capacity

under cyclic loading up to θ_p

strength degradation < *20%*

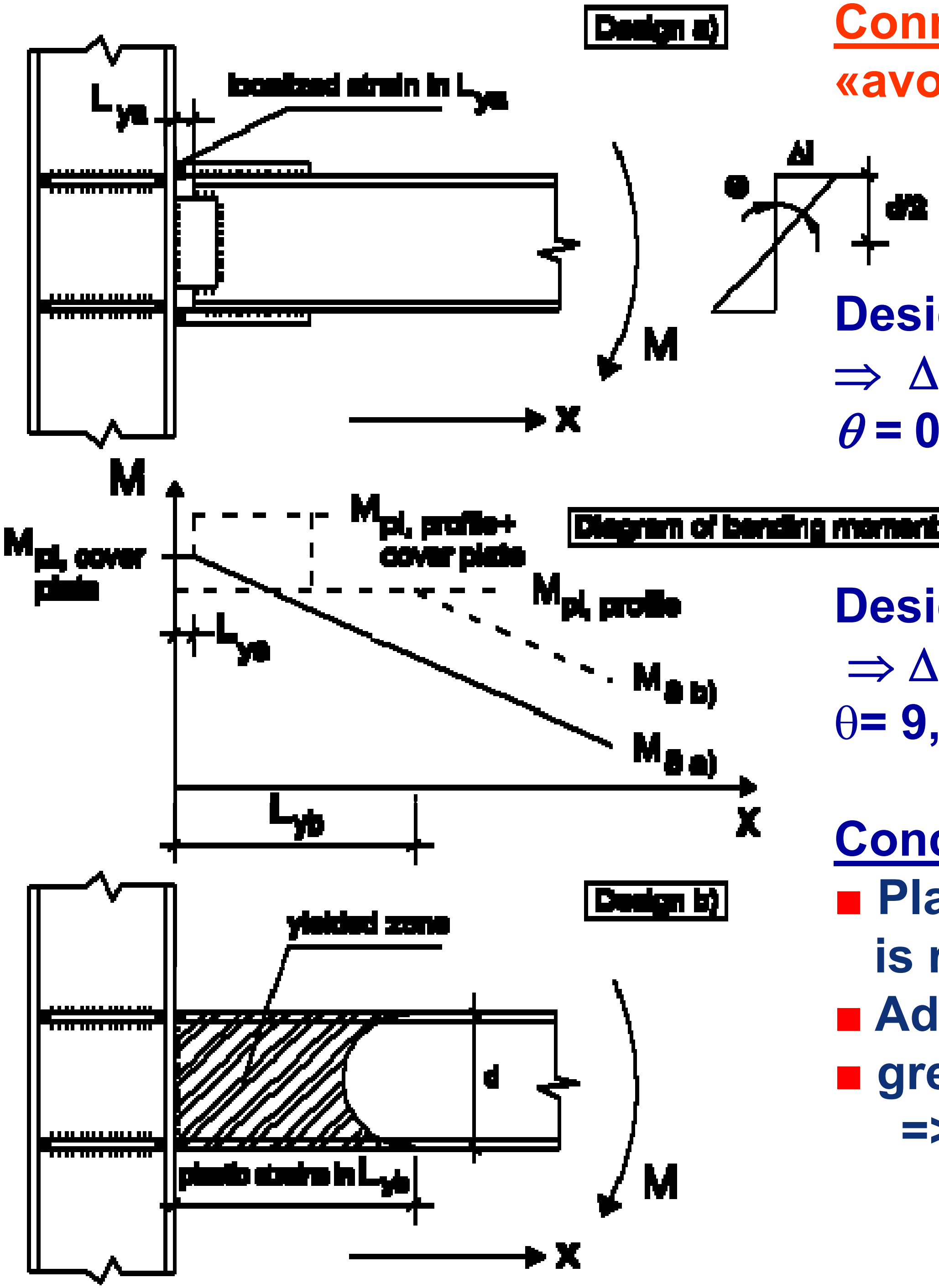
stiffness degradation < *20%*



- **Connection design condition**
  - **if dissipative zones are in beams =>** $M_{Rd,connection} \geq \pm 1{,}1\ \gamma_{ov}\ M_{pl,Rd,beam}$
  - **if dissipative connections**

    => capacity design refers to connection plastic resistance

Connections: EC8
«avoid localisation of plastic strains»



Example

Design a) L_{ya}=10 mm $\varepsilon_{y,\,max}$= 2,38 %

$\Rightarrow \Delta l$ = 0,0238.10 = 0, 238 mm

θ = 0,238/(400/2)=1,2mrad<<<25 mrad

Design b) L_{yb}=400 mm $\varepsilon_{y,\,max}$ = 2,38 %

$\Rightarrow \Delta l$ = 9,52 mm

θ= 9,52/(400/2)= 47,6mrad>>35 mrad

Conclusions

- Plastic zone length ≈ $h_{section}$ is required for effective hinge
- Adequate $\varepsilon_{y,max}$ and f_u / f_y needed
- greater beam depth
  => less rotation capacity

Steel Moment Resisting frame

Design of beam column connections

Detailing: not in EC8 in National Annexes, in AISC2000, AFPS2005,…

However 1 common feature:

1 bad woolf

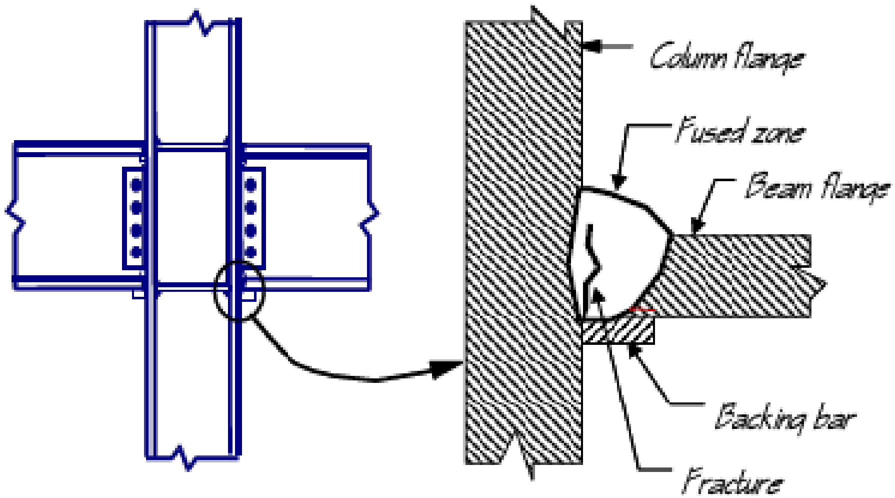
Connection Types and corresponding ductility classes

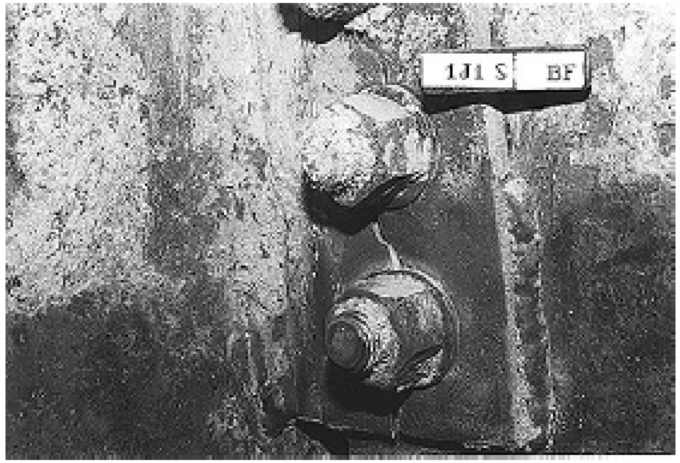
Connection Type	**Maximum Ductility Class allowed**	
	Europe	**US**
Beam flanges welded, beam web bolted to a shear tab welded to column flange. Fig. 34	DCL *	OMF*
Beam flanges welded, beam web welded to a shear tab welded to column flange. Fig. 31	DCH	SMF
Beam flanges bolted, beam web bolted to a shear tab welded to column flange. Fig. 35	DCH	SMF
Unstiffened end plate welded to beam and bolted to column flange by 4 rows of bolts. Fig.36	DCH	SMF
Stiffened end plate welded to beam and bolted to column flange by 8 rows of bolts. Fig. 37	DCH	SMF
Reduced beam section. Beam flanges welded, beam web welded to shear tab welded to column flange. Fig.38	DCH	SMF
Reduced beam section. Unstiffened end plate welded to beam and bolted to column flange by 4 rows of bolts. Same as Fig.36, but with reduced flange sections.	DCH	SMF

*May be considered for DCM (equivalent to IMF) in some countries

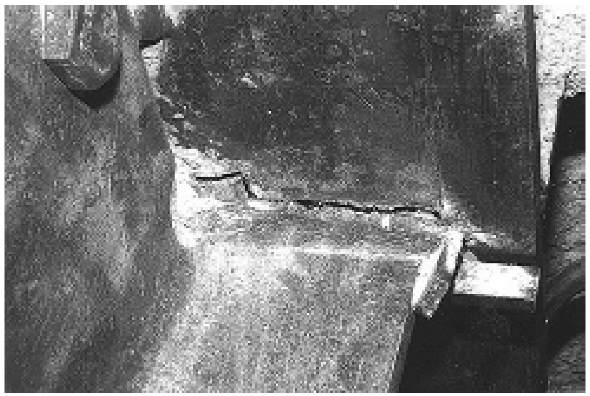
Steel Moment Resisting frame

Steel was ductile…





Vertical Fracture through Beam Shear Plate Connection

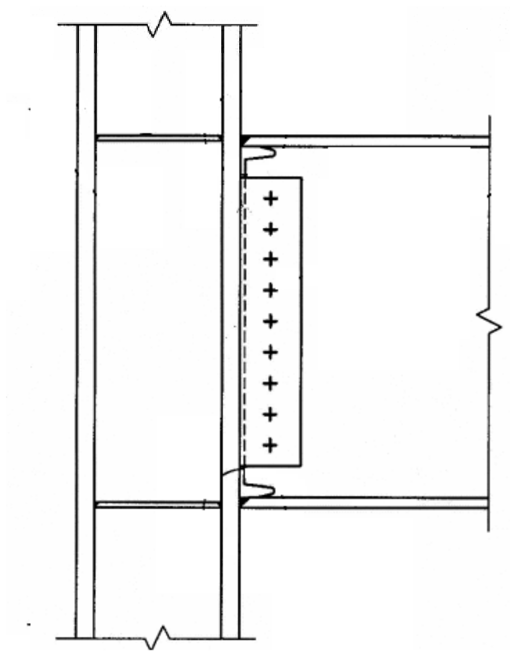


a. Fracture at Fused Zone

b. Column Flange "Divot" Fracture

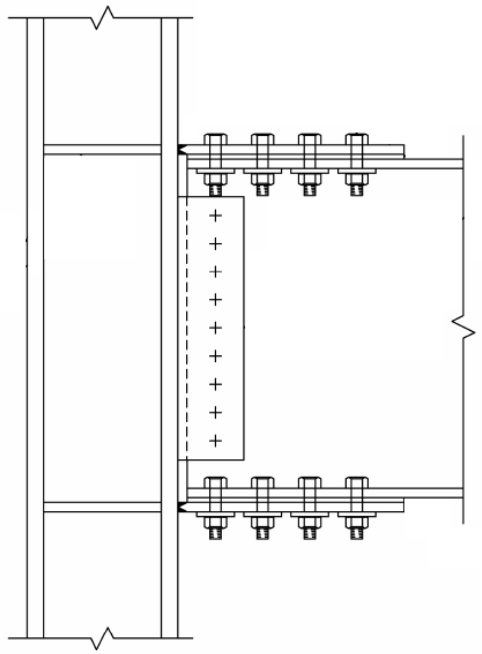
Northridge 1994

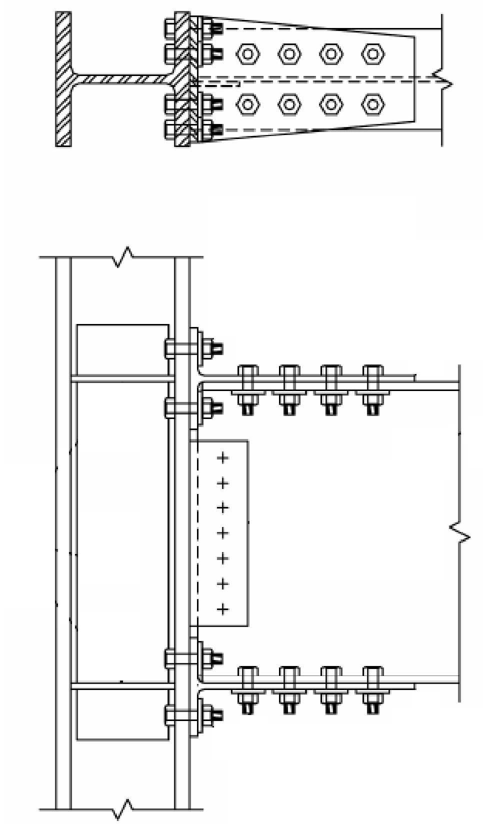
Steel Moment Resisting frame



Beam flanges welded, beam web bolted to shear tab welded to column flange

DCL   low ductility

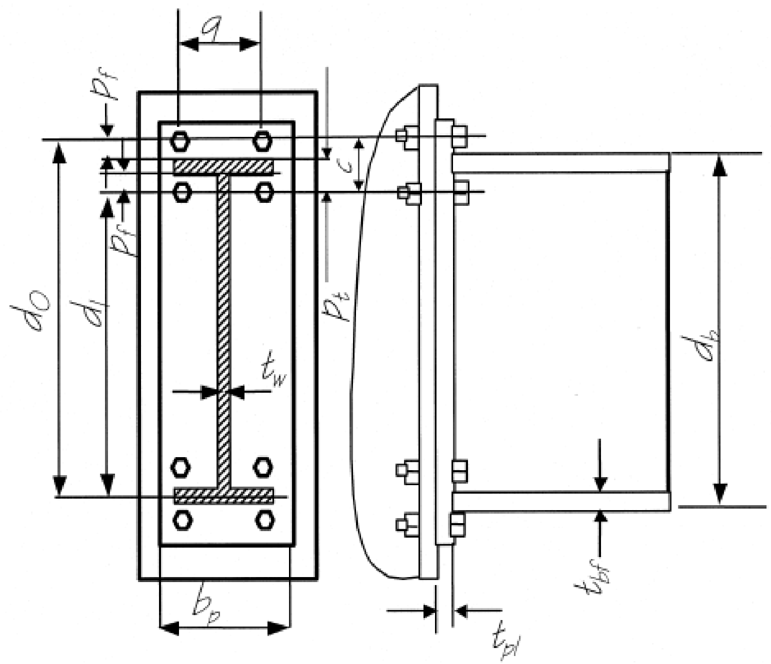




Beam flanges bolted; beam web bolted to shear tab welded to column flange.

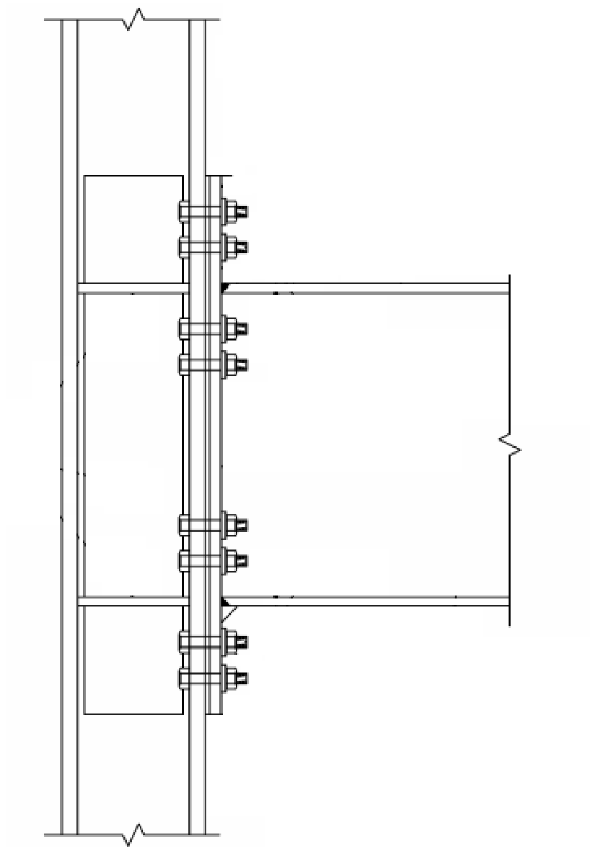
DCM -DCH

Steel Moment Resisting frame



Unstiffened end plate welded to beam and bolted to column flange by 4 rows of bolts

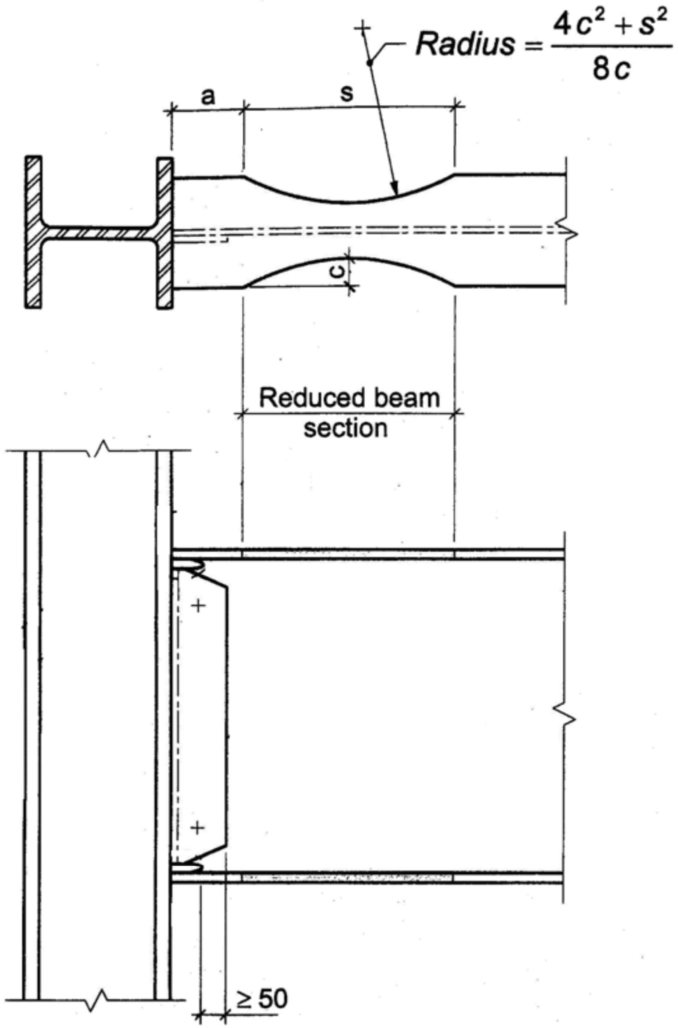
DCM -DCH

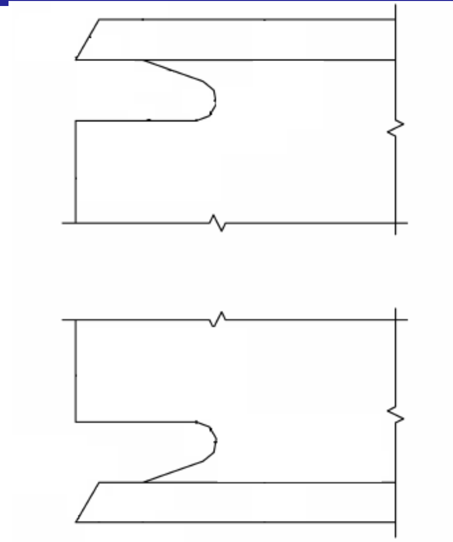


Stiffened end plate welded to beam and bolted to column flange by 8 rows of bolts

DCM -DCH

Steel Moment Resisting frame





Weld access hole details in FEMA 350

<u>Design criteria</u>
0,5b ≤ a ≤ 0,75b
0,65h ≤ s ≤ 0,85h
b: flange width
h: beam depth
0,2b ≤ c ≤ 0,25b
b_e = b – 2c

« Dogbone » or RBS Reduced beam section. Beam flanges welded, beam web welded to shear tab welded to column flange DCM -DCH

Steel Moment Resisting frame

USA. Los Angeles area. 2000.

Grenoble.
Rossignol Ski Factory. 2008.

Steel Moment Resisting frame

A remark

If beam flanges are welded to the column flanges and beam web is welded to a shear tab welded to the column flange

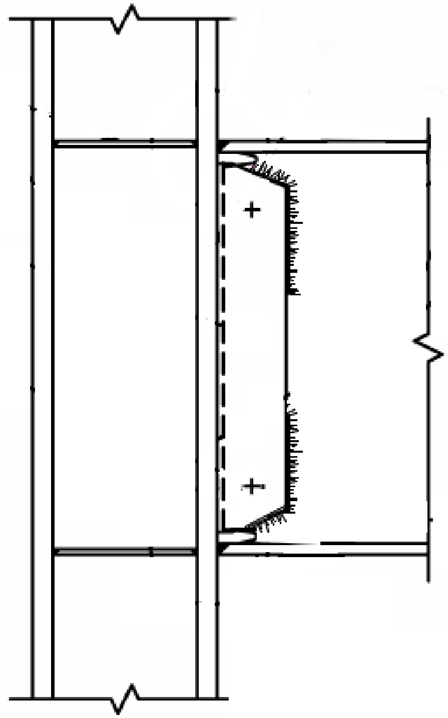
- **the flange butt welds transmit $M_{pl,flanges}$**
- **the web welds transmit $M_{pl,web}$ + shear V_{Ed}**

$$M_{Rd,connection} \geq 1{,}1\ \gamma_{ov}\ M_{pl,Rd,beam}$$

$$M_{pl,flanges} = b_f\ t_f\ f_y\ (d + t_f)$$
$$M_{pl,web} = t_w\ d^2\ f_y / 4$$

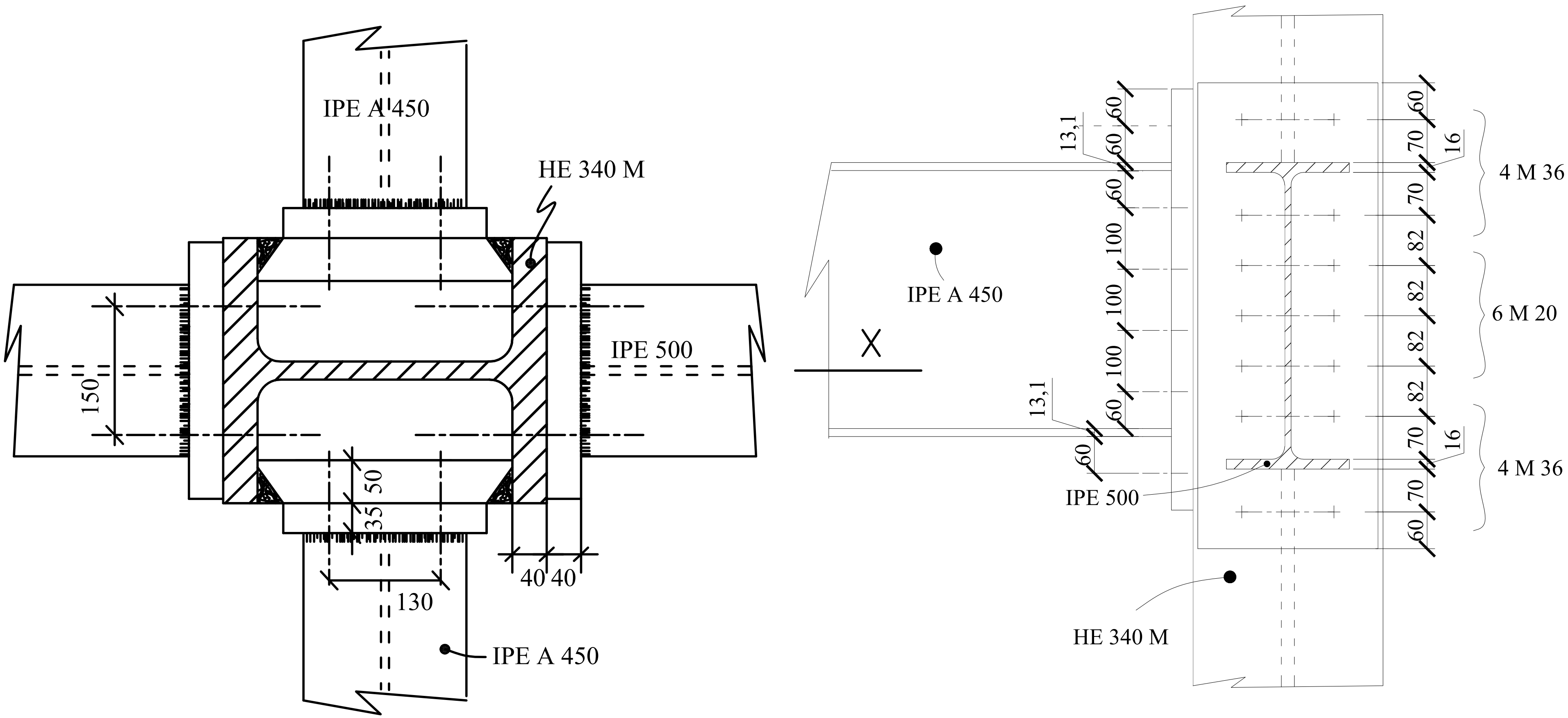
$$M_{Rd,web,connection} \geq 1{,}1\ \gamma_{ov}\ M_{pl,web} = 1{,}1\ \gamma_{ov}\ t_w\ d^2\ f_y / 4$$

=> shear tab stronger than the web
=> Top and bottom welds on shear tab required in addition to web fillet welds for shear



Steel Moment Resisting frame

Design of connection
IPE500 beamX – IPEA450beamY - HE340M column

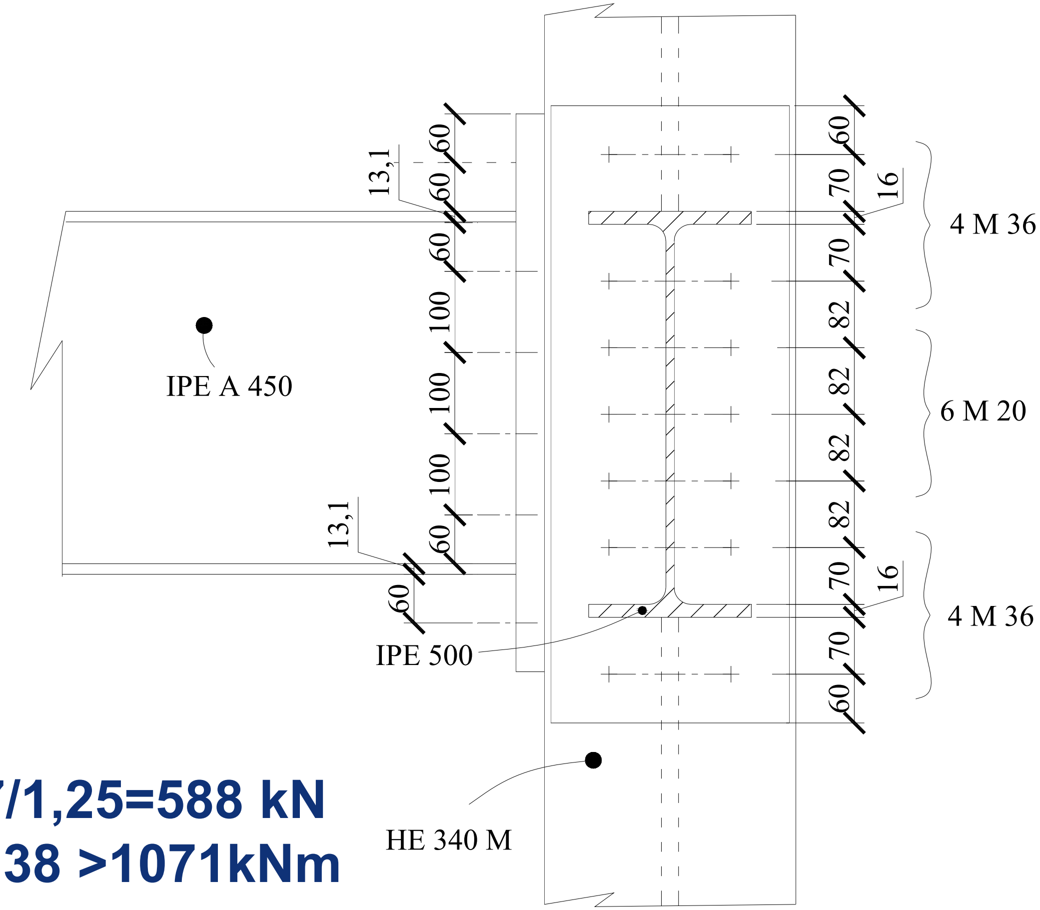


Steel Moment Resisting frame

Design of bolted connection

Capacity design

$M_{Rd,connection} \geq 1{,}1\ \gamma_{ov}\ M_{pl,Rd,beam}$ = 1,1 x 1,25 x 778,9 = 1071 kNm



Bending moment $M_{Rd,connection}$

=> 4 rows x 2 M36 10.9 bolts

row 1: h_r =500–16+70= 554 mm

row 2: h_r = 500–16-70= 414 mm

Resistance $F_{tr,Rd}$ M36 tension:

$F_{tr,Rd}$=0,9$f_u A_s/\gamma_{M2}$=0,9x1000x817/1,25=588 kN

$M_{Rd,connect}$=(554+414)x2x588=1138 >1071kNm

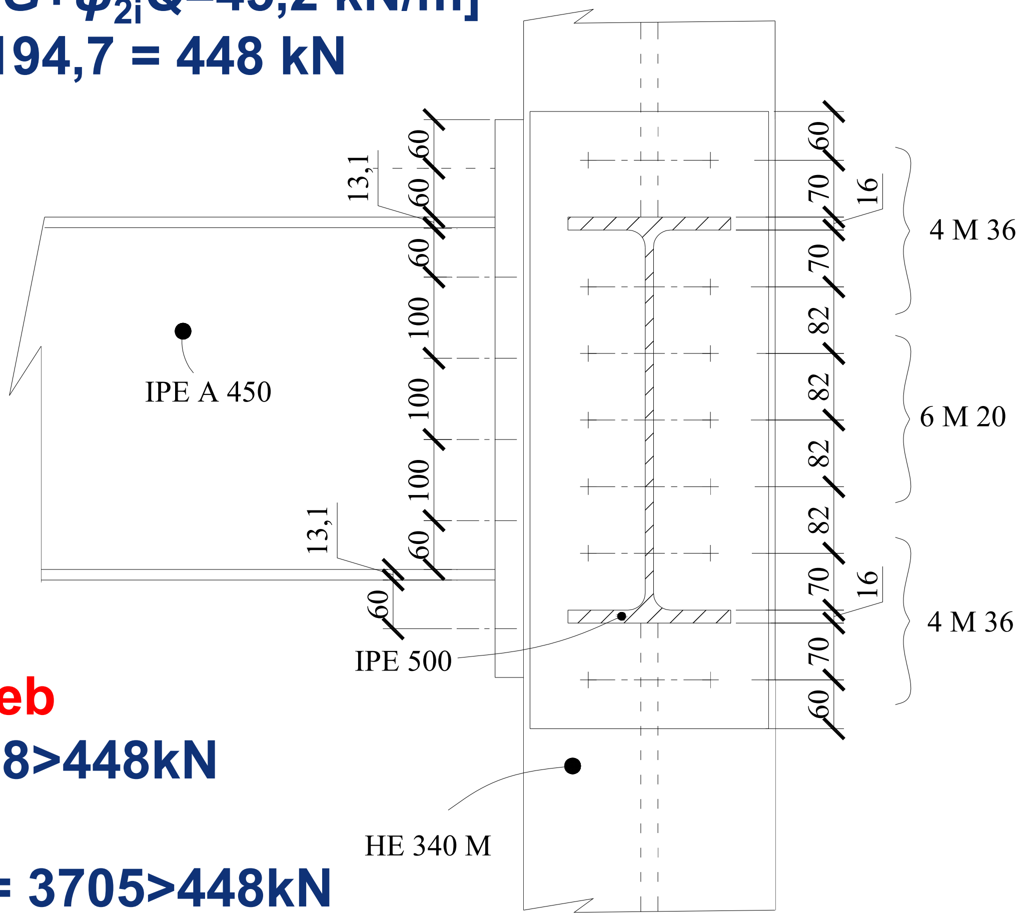
Steel Moment Resisting frame

$V_{Rd,connection} \geq V_{Ed,G} + 1{,}1\ \gamma_{ov}\ V_{Ed,E}$ ***Capacity design***

$V_{Ed,E} = 2\ M_{pl,Rd,beam} / l = 2 \times 778{,}9 / 8 = 194{,}7$ kN

$V_{Ed,G} = 0{,}5 \times 8 \times 45{,}2 = 180{,}8$ kN $[G+\psi_{2i}Q = 45{,}2$ kN/m$]$

$V_{Rd,connection} \geq 180{,}8 + 1{,}1 \times 1{,}25 \times 194{,}7 = 448$ kN



Shear $V_{Rd,connection}$

=> 6 M20 10.9 bolts on sides of web

Bolts resistance: 6x122,5/1,25=588>448kN

Plate bearing resistance:

$V_{Rd,plate} = (6 \times 193 \times 40)/(10 \times 1{,}25) = 3705 > 448$ kN

Steel Moment Resisting frame

Design of end plate

Tension force $F_{tr,Rd}$ applied by one flange to end plate:

$F_{tr,Rd} = M_{Rd}$ / (500- 16) = 2213 kN

Virtual work 4 yield lines

$4\ M_{pl,1,Rd} \times \theta = F_{tr,Rd} \times \theta \times$ m

***M:* distance bolt axis flange surface (70 mm)**

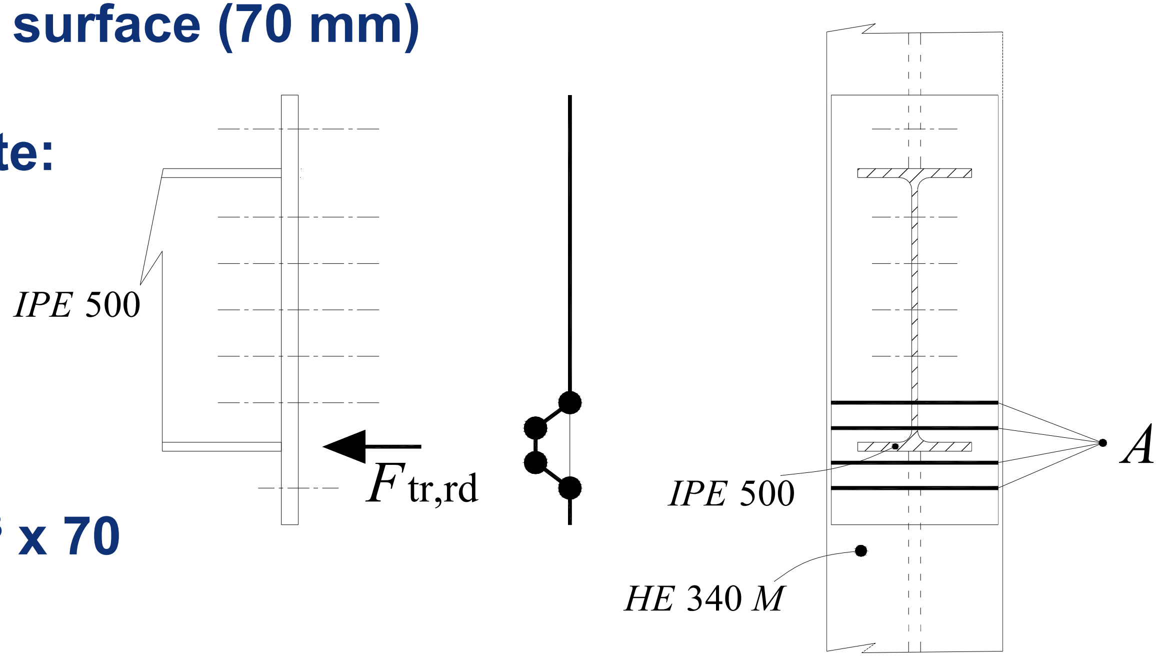
Yielding in beam, not in plate:

$4\ M_{pl,1,Rd} \times \theta > F_{tr,Rd} \times \theta \times m$

$M_{pl,1,Rd} = (l_{eff} \times t^2 \times f_y)/\ 4\gamma_{M0}$

l_{eff} = 300 mm γ_{M0} = 1,0

f_y = 355 N/mm²



(4x300xt^2 x355)/4 = 2213.10³ x 70

$\Rightarrow t$ = 38,1 mm min

$\Rightarrow t$ = 40 mm

Steel Moment Resisting frame

Check of resistance of end plate and column flange to punching.

$B_{p,Rd} > F_{tr,Rd}$?

Check identical for end plate and column flange:

same thickness 40 mm and f_y =355 N/mm²

$F_{tr,Rd}$ = 553 kN

$B_{p,Rd}$ shear resistance punching out a cylinder

diameter d_m head of the bolt =58 mm for M36 bolt

t_p of plate = 40 mm

$B_{p,Rd} = 0{,}6\ \pi\ d_m\ t_p\ f_u = 0{,}6 \times 3{,}14 \times 58 \times 40 \times 500 / 1{,}25 = 2185.10^3$ N

= 2185 kN > 553 kN

Welds between end plates and beams

Butt welds

adequate preparation/execution (V grooves, welding from both side)

⇒ satisfy overstrength criterion => no calculation needed

Steel Moment Resisting frame

Check of column web panel in shear

Plastic hinges in beam sections adjacent to the column

Design shear $V_{wp,Ed}$ in panel zone:

$V_{wp,Ed} = M_{pl,Rd,\,left} / (d_{left} - 2t_{f,left}) + M_{pl,Rd,\,right} / (d_{right} - 2t_{f,right}) + V_{Sd,\,c}$

Neglecting $V_{Sd,c}$: $V_{wp,Ed} = 2 \times 1071.\ 10^3 /(377\text{-}2\text{x}40) = 7212$ kN

$V_{wb,Rd} = (0{,}9\ f_y\ A_{wc})/\ (\sqrt{3} \times \gamma_{M0}) = (0{,}9\text{x}355\text{x}9893)/\sqrt{3}$

= 1824 kN << 7212 kN

Column web increased for shear resistance:

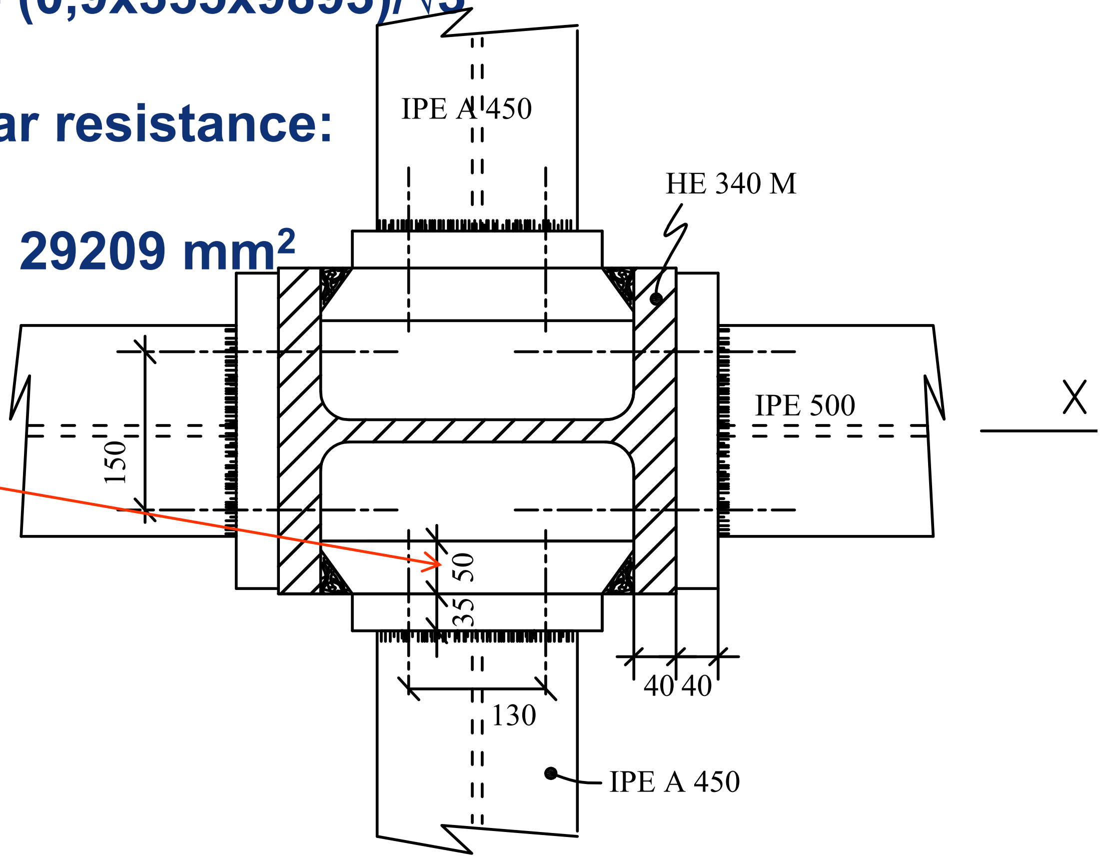
7212–1824 = 5388 kN

Area = $(5388.10^3\sqrt{3})/(355\text{x}0{,}9) =$ 29209 mm^2

2 plates 297 mm length

Thickness: 29209/(2 x 297)

= 49,2 mm => 50 mm



Steel Moment Resisting frame

Check of column web panel in transverse compression

$F_{c,wc,Rd} = \omega\, k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc} \;/\; \gamma_{M0}$

setting ω and k_{wc} at 1,0

$b_{eff,c,wc} = t_{fb} + 5(t_{fc} + s) = 16 + 5\,(40 + 27) = 351$ mm

ignoring the connecting plates of beams in the *y* direction

$F_{c,wc,Rd} = 351 \times 21 \times 355 = 2616.\ 10^3$ N $= 2616$ kN $> F_{tr,Rd} = 2213$ kN

A more comprehensive check

include connecting plates of beams in the *y* direction

$b_{eff,c,wc} = t_{fb} + 5(t_{fc} + s) = 16 + 5\,(40 + 27 + 40 + 40) = 751$ mm

Check of column web panel in transverse tension

$F_{c,wc,Rd} = \omega\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc} \;/\; \gamma_{M0}$

identical to above, satisfied

Steel Moment Resisting frame

Comments on design options

Design governed by limitation of deflections:

- P-Δ — design earthquake
- inter-storey drift — service earthquake

Beam sections possess a safety margin for resistance to design EQ

$M_{pl,Rd,beam}$ = 778 kNm > M_{Ed} =591 kNm (worst case moment)

Reducing the beam sections locally by 'dogbones' or RBS

- change the structure stiffness by few %
- provide a reduction in the design moments and shear applied to the connections

$M_{pl,Rd,beam}$ could be reduced by 778/591 = **1,32**

$\Rightarrow$ Reduce connection design moment $M_{Ed,connection} = 1,1\ \gamma_{ov}\ M_{pl,Rd,beam}$

$\Rightarrow$ reduce bolt diameters, end plate thickness...

At perimeter columns, reduction ratio $M_{pl,Rd,beam} / M_{Ed}$ = **1,61**

Steel Moment Resisting frame

Influence of increase in flexibility due to RBS

Frame flexibility and θ increased:

- by estimated 7% (canadian code)
- can be computed

Revised amplification factors 1/ (1- θ)

Storey	Interstorey drift sensitivity coefficient θ		amplification factor 1/ (1- θ)
	Without RBS	With RBS	With RBS
1	0,099	0,105	1,11
2	0,137	0,147	1,17
3	0,118	0,126	1,14
4	0,086	0,092	1
5	0,054	0,057	1
6	0,027	0,028	1

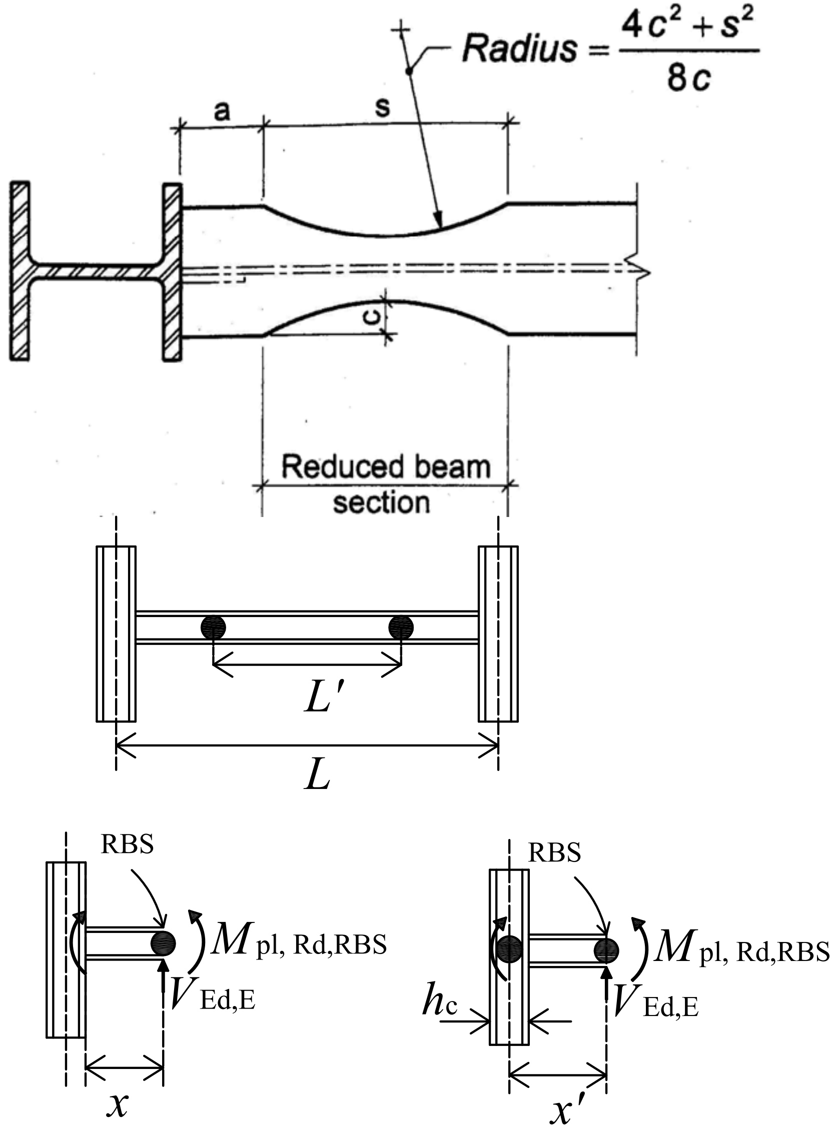
Influence of RBS distance to connection on design moment

a = 0,5 x b = 0,5 x 200 = 100 mm
s = 0,65 x d = 0,65 x 500 = 325 mm

Distance RBS to column face
$a + s/2$ = 162,5 + 100 = 262 mm

Bending moment
≈ linear between beam end -1/3 span
1/3 span = 8000 / 3 = 2666 mm

=> Design bending moment in RBS
$M_{d,RBS}$=596x(2666–262)/2666= 537 kNm



Steel Moment Resisting frame

Definition of section cuts at RBS.

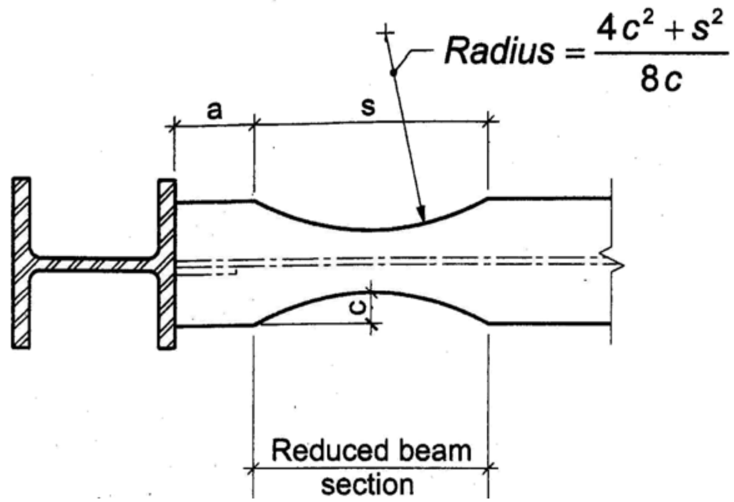
c in the range 0,20b-0,25b c=0,22b= 44 mm

IPE500 $W_{pl,y}\, f_y$ = 2194.10^3 x 355 = **778. 10^6 Nmm**

Flange moment: $b\, t_f\, f_y\, (d - t_f)$ = 16x200x355(500–16) = 549. 10^6 Nmm

Web moment: $t_w\, f_y\, (d - 2t_f)^2/4$=10,2x355 (500 – 32)^2 = 198. 10^6 Nmm

Due to root radii web-flange junctions:(778–549–198) = 31. 10^6 Nmm



Plastic moment of reduced IPE500

$b_e = b - 2c$ = 200 - 88 = 120 mm.

Flange moment: $b_e t_f f_y (\mathrm{d}-t_f)$=16x112x355(500–16)= 308. 10^6 Nmm

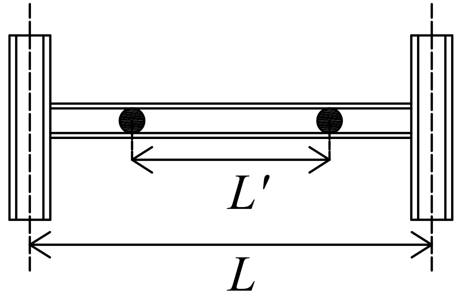
RBS plastic moment: $M_{pl,Rd,RBS}$=(308+198+31)10^6= **537.10^6 Nmm**

For fabrication purposes: radius R of the cut

$R = (4c^2 + s^2) / 8c = (4 \times 32^2 + 325^2)/(8 \times 32)$ = 857 mm

Steel Moment Resisting frame

Design moment and shear at the connection



$V_{Ed,E} = 2\ M_{pl,Rd,,RBS} / L'$

$L' = 8000–377-(2x262,5)=7098mm$

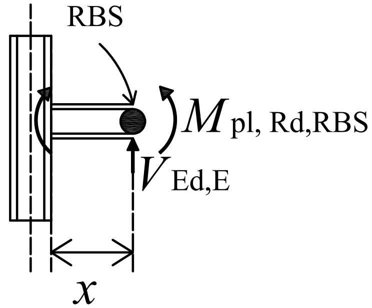
$V_{Ed,E} = 2 \times 537 / 7,098 = 151$ kN

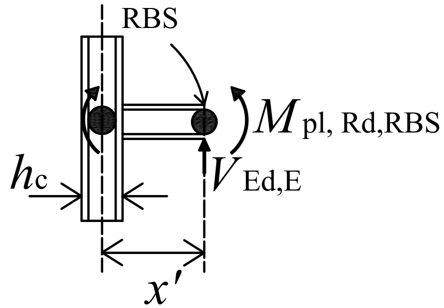
$V_{Ed,G}$ in RBS due to gravity $G+\psi_{2i}Q$: $V_{Ed,G}$ $=0,5x7,098x45,2 = 160,4$ kN

Design shear in RBS:

$V_{Ed,E} = V_{Ed,G} + 1,1\gamma_{ov} V_{Ed,E}$

$V_{Ed,E} = 160,4+1,1x1,25x151 = 368$ kN





$M_{Ed,connection} = 1,1\gamma_{ov} M_{pl,Rd,,RBS} + V_{Ed,E}$ x dist x

$x=a+s/2=262,5$ mm

$M_{Ed,connection} = 1,1x1,25x537+368x0,2625 = 834$ kNm

Due to RBS, $M_{Ed,connection}$ reduced from 1071 kNm to 834 kNm = -28%

$V_{Rd,connection} \geq 448$ kN without RBS $\qquad V_{Rd,connection} \geq 368$ kN with RBS

Reduction in design shear at connection = - 21%

Illustration of Design 2

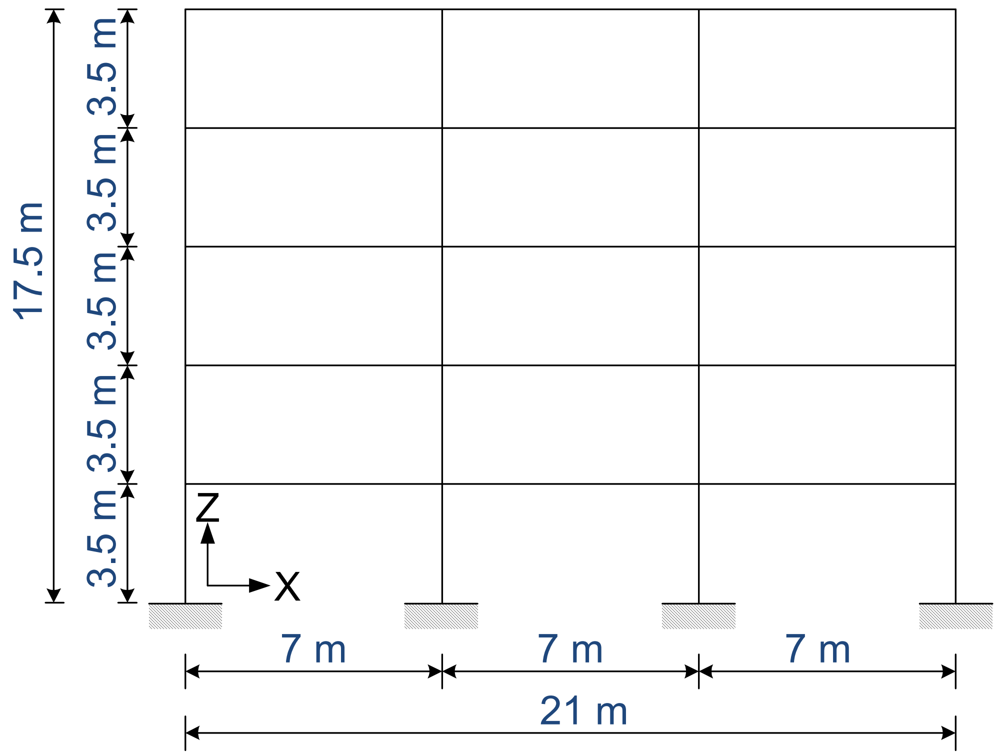
Composite Steel Concrete Moment Resisting Frame

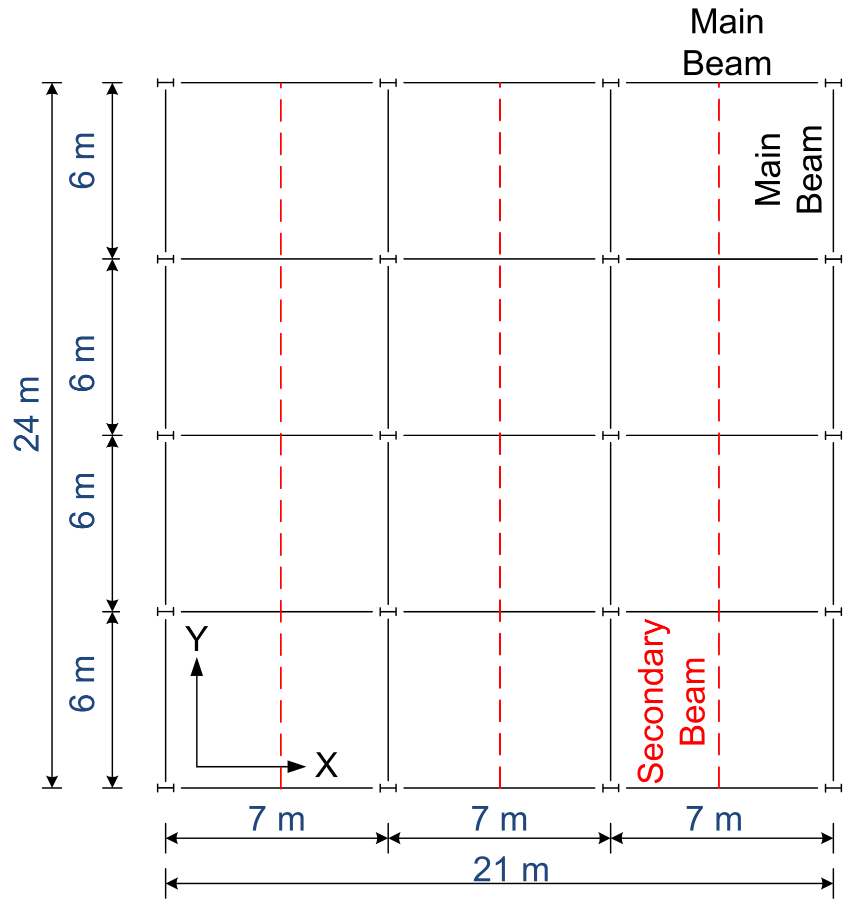
Hughes SOMJA
INSA Rennes

Hervé DEGEE
University of Liege

André PLUMIER
University of Liege

Composite Steel Concrete Moment Resisting Frame





5 storey building
Height 17,5 m
Slab thickness 120 mm

Design from RFCS project "OPUS"

Composite Steel Concrete Moment Resisting Frame

4 design cases Seismicity	Beams	Columns	Steel
High 0,25g	Comp.	steel	S355
High 0,25g	Comp.	Comp.	S355
Low 0,10g	Comp.	steel	S235
Low 0,10g	Comp.	Comp.	S235

- **Permanent Actions**
  Slab: 5 kN/m2 Partitions: 3 kN/m
- **Variable Actions**
  Uniformly distributed loads: q_k = 3 kN/m^2
  Concentrated loads: Q_k = 4 kN
  Snow load altitude A = 1200 m q = 1.1 kN/m^2
  Wind Load : $q_p(Z)$ = 1.4 kN/m2
- **Seismic Action** $\gamma_I = 1{,}00$ a_{gR} =0,25g 0,10g
  type 1 design spectrum soil B DCM q=4
  Values of ψ factors

$$\psi_0 = 0.7$$
$$\psi_1 = 0.5$$
$$\psi_2 = 0.3$$

Composite Steel Concrete Moment Resisting Frame

- **Seismic Mass of the Building $G_k + \psi_{Ei} Q_k$**

  $\psi_{Ei} = \varphi \psi_{2i}$ $\psi_{2i} = 0.3$

  $\varphi = 1$ Clause 4.2.4 and table 4.2 of French NF

  $G = G_{slab} + G_{walls} + G_{steel} + G_{concrete}$ $Q = Q_{imposed} + Q_{snow}$

	Case1	Case2	Case3	Case4
Seismic mass (t)	1900	1963	1916	1994

- **Seismic Base Shear by Lateral Force Method**

$$F_b = m * S_d(T_1) * \lambda$$

$$F_b = 1963 * 0.535 * 0.85$$

$$F_b = 892 \text{ kN}$$

Base shear F_{bx} on each MR frame

$$F_{bX} = \frac{F_b}{5} = \frac{892}{5} = 178.4 \text{ kN}$$

Torsion effect

$$\delta = 1 + 0.6 * \frac{x}{L}$$

$$\delta = 1.3$$

$$F_{bXt} = \delta * F_{bX}$$

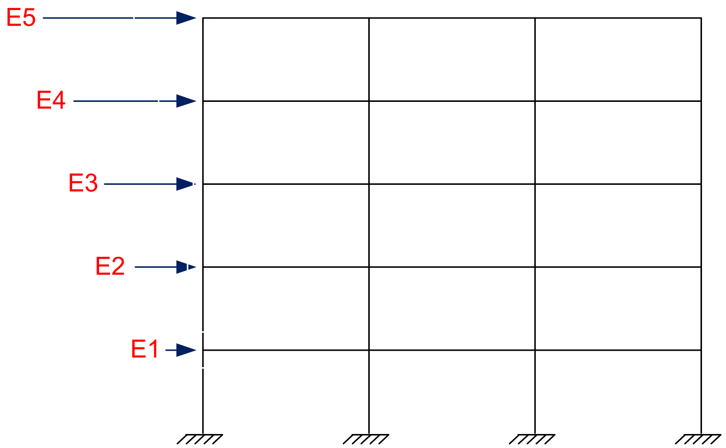
=>

$$F_{bXt} = 1.3 * 178.4$$

$$F_{bXt} = 232 \text{ kN}$$

Composite Steel Concrete Moment Resisting Frame

● Distribution of seismic loads



Seismic static equivalent forces	Case 1	Case 2	Case 3	Case 4
E1 (kN)	15.7	15.5	7.7	7.7
E2 (kN)	31.4	30.9	15.4	15.3
E3 (kN)	47.1	46.4	23.1	23.0
E4 (kN)	62.8	61.9	30.8	30.7
E5 (kN)	78.5	77.3	38.5	38.3

Composite Steel Concrete Moment Resisting Frame

- **Combinations at ULS considered in the analysis for an office building**

$$1.35G + 1.5W + 1.05Q + 0.75S$$

$$1.35G + 1.5W + 1.05S + 0.75Q$$

$$1.35G + 1.5Q + 1.05W + 0.75S$$

$$1.35G + 1.5Q + 1.05S + 0.75W$$

$$1.35G + 1.5W + 1.05\left(S + Q\right)$$

$$1.35G + 1.5\left(S + Q\right) + 1.05W$$

G : Dead load
Q : Imposed load
S : Snow load
W: Wind load

- **Seismic Design Situation**

$G_k + \psi_2 Q_k + E$ **with** $\psi_2 = 0.3$

Composite Steel Concrete Moment Resisting Frame

1. **Structural Analysis & Design**

   Action effects Internal stresses

   Second-Order Effects

   Global and Local Ductility Condition

2. **Damage Limitation checks**

3. **Section and Stability Checks of**

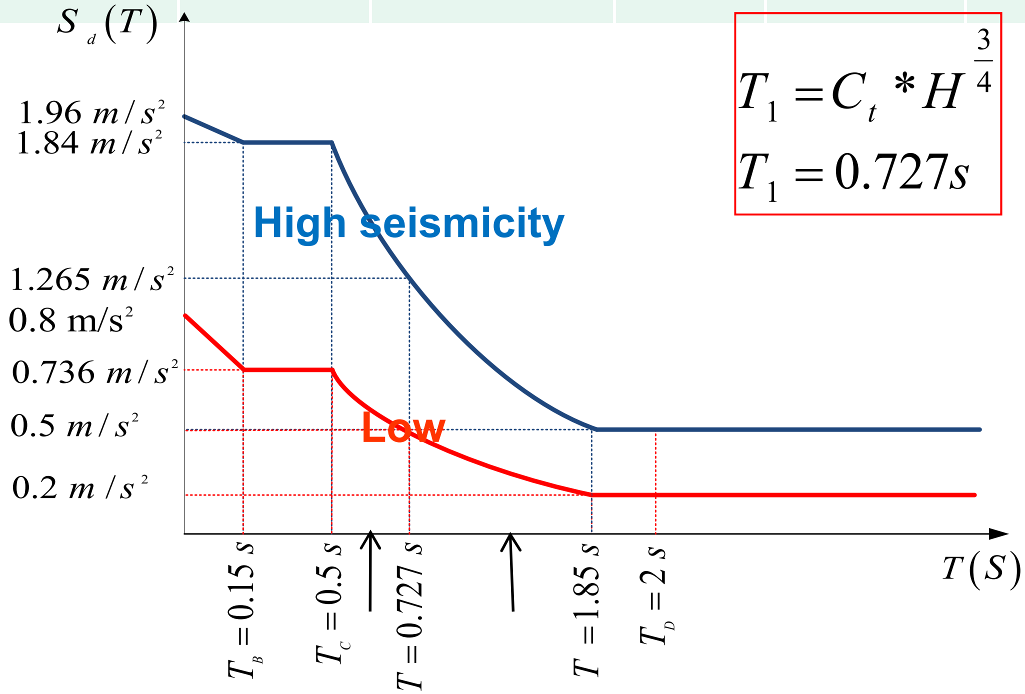
   Composite Beams

   Steel Columns

   Composite Columns

Composite Steel Concrete Moment Resisting Frame

4 design Seismicity		Beams	Columns	Steel	T simpl EC8(s)	Sd(T) EC8 m/s^2	T Exact (s)	Sd(T) Exact m/s^2	Seismic mass t
High	0,25g	Comp.	steel	S355	0,727	1,26	1,64	0,56	1900
High	0,25g	Comp.	Comp.	S355	0,727	1,26	1,72	0,56	1963
Low	0,10g	Comp.	steel	S235	0,727	0,51	1,35	0,27	1916
Low	0,10g	Comp.	Comp.	S235	0,727	0,51	1,41	0,27	1994



$$T_1 = C_t * H^{\frac{3}{4}}$$

$$T_1 = 0.727s$$

Composite Steel Concrete Moment Resisting Frame

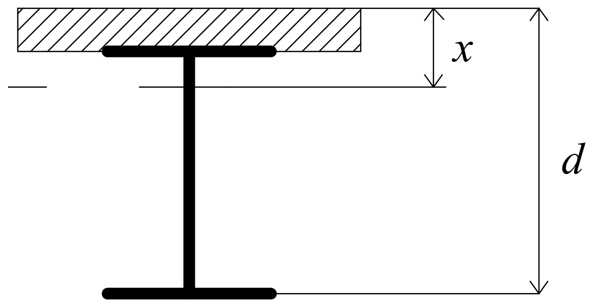
Analysis

Beams: EC8 limited to steel profile + slab like EC4

- **2 flexural stiffness:**

EI_1 for zones under M+ uncracked sections

EI_2 for zones under M- cracked sections



- **An equivalent I_{eq} constant over span may be used:**

$$I_{eq} = 0{,}6\ I_1 + 0{,}4\ I_2$$

- **For composite columns:** $(EI)_c = 0{,}9(\ EI_a + r\ E_{cm}\ I_c + E\ I_s\ )$

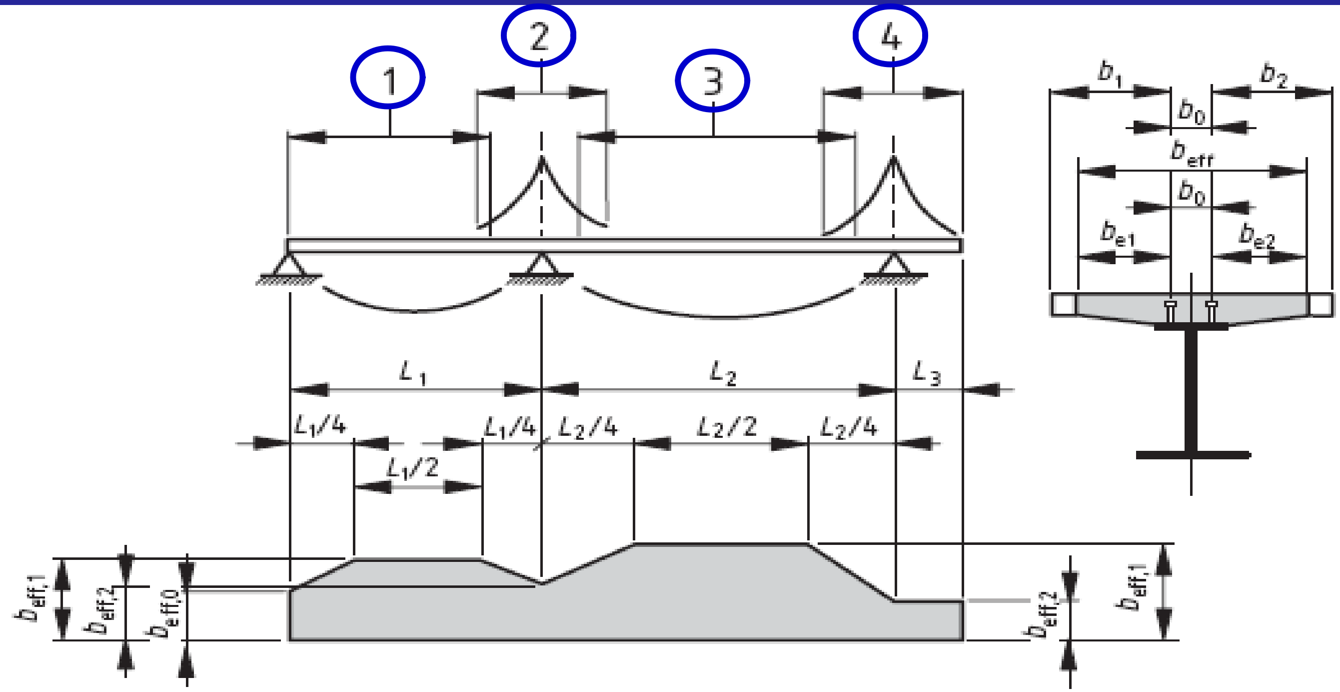
E : steel E_{cm} : concrete

r a reduction factor r = 0,5.

I_a, I_c and I_s : I of steel section, concrete and re-bars respectively

Composite Steel Concrete Moment Resisting Frame

Effective Width
Static
Eurocode 4-1



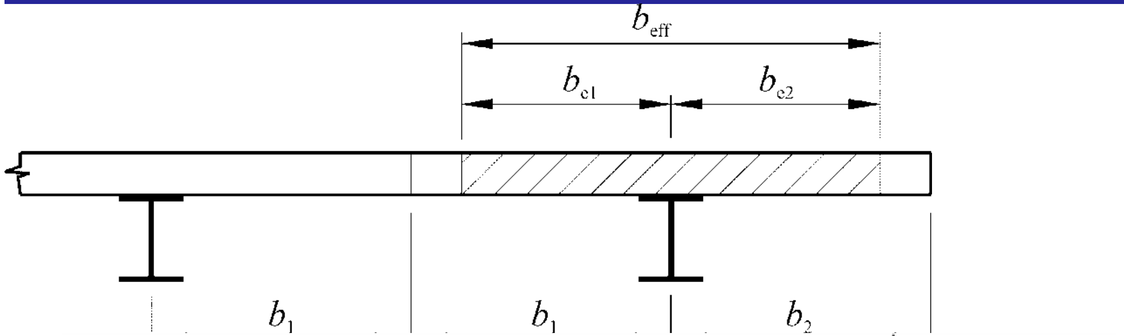
b_0 distance between centres of the outstand shear connectors and it is assumed to be Zero in our example.

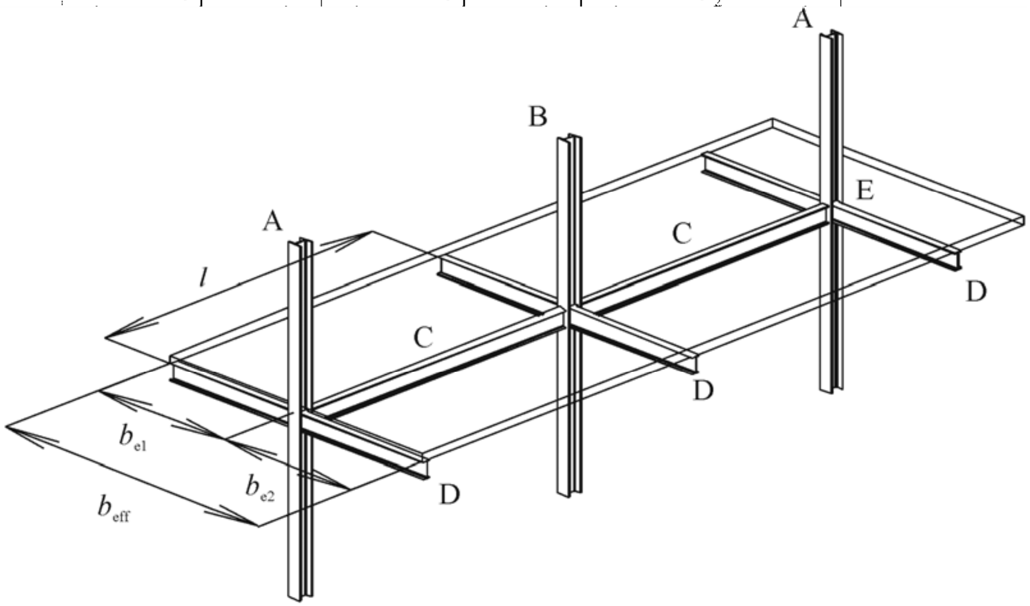
b_{ei} effective width of concrete flange on each side of the web
$= L_e / 8$ not greater than width b_i

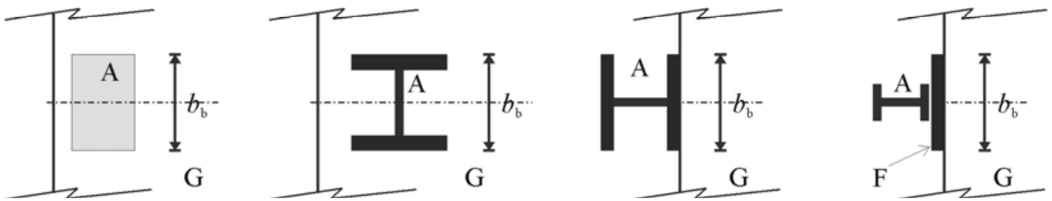
$$b_{eff} = b_0 + \sum b_{ei}$$

$$b_{eff} = \begin{cases} 1225\ mm \text{ (at mid-span)} \\ 875\ \ mm \text{ (at an end support)} \end{cases}$$

Composite Steel Concrete Moment Resisting Frame







<u>Effective Width</u>
<u>Seismic</u>
<u>Eurocode 8-1</u>

Effective width b_{eff} concrete flange: $b_{e1} + b_{e2}$

Partial effective widths b_e in Tables, not ≥ b_1 & b_2

2 Tables. Determination of

- **Elastic stiffness: I**
- **Plastic resistance M_{pl}**

M inducing compression in slab: +

tension -

Composite Steel Concrete Moment Resisting Frame

EC8 Table

Partial effective width b_e of slab for computation of *I* used in elastic analysis

b_e	Transverse element	b_e for I (Elastic Analysis)
At interior column	Present or not present	For negative M : 0,05 l
At exterior column	Present	For positive M : 0,0375 l
At exterior column	Not present, or re-bars not anchored	For negative M : 0 For positive M : 0,025 l

EC8 Table

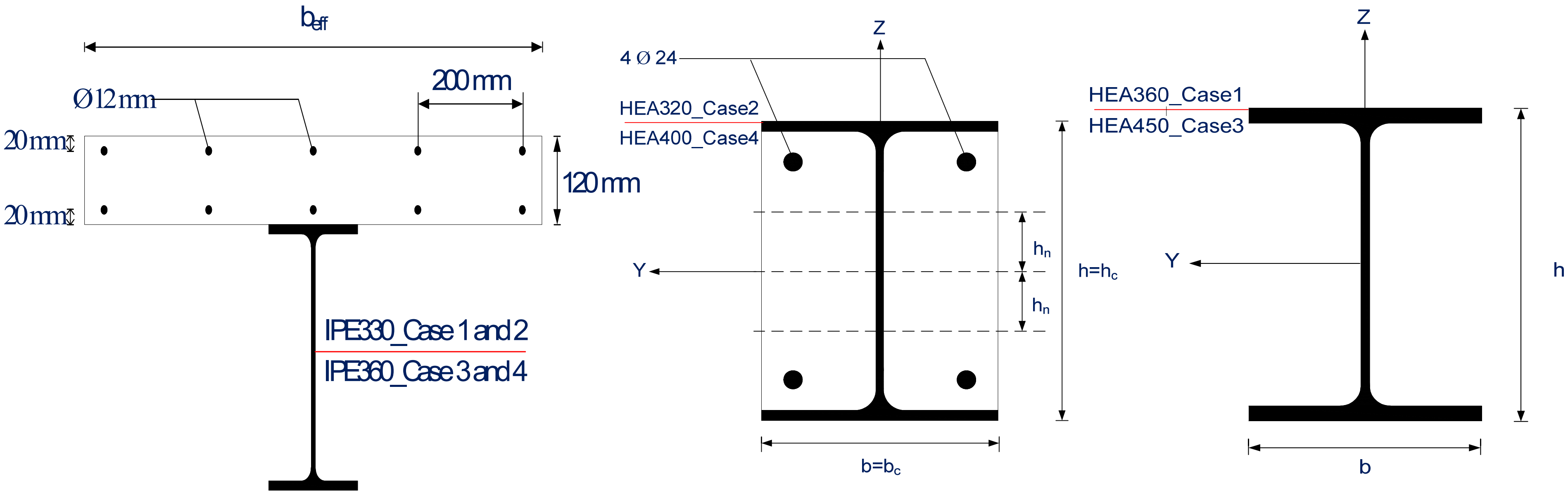
Partial effective width b_e of slab for evaluation of plastic moment M_{pl}

Sign of bending moment M	Location	Transverse element	b_e for M_{Rd} (Plastic resistance)
Negative M	Interior column	Seismic re-bars	0,1 l
Negative M	Exterior column	All layouts with re-bars anchored to façade beam or to concrete cantilever edge strip	0,1 l
Negative M	Exterior column	All layouts with re-bars not anchored to façade beam or to concrete cantilever edge strip	0,0
Positive M	Interior column	Seismic re-bars	0,075 l
Positive M	Exterior column	Steel transverse beam with connectors. Concrete slab up to exterior face of column of H section with strong axis oriented as in Figure 63 or beyond (concrete edge strip). Seismic re-bars	0,075 l
Positive M	Exterior column	No steel transverse beam or steel transverse beam without connectors. Concrete slab up to exterior face of column of H section with strong axis oriented as in Figure 63, or beyond (edge strip). Seismic re-bars	$b_b/2 + 0{,}7\ h_c/2$
Positive M	Exterior column	All other layouts. Seismic re-bars	$b_b/2 \leq b_{e,max}$ $b_{e,max} = 0{,}05l$

Composite Steel Concrete Moment Resisting Frame

Effective slab width b_{eff} (mm) at column	For Positive Moment $M_{pl,Rd}^{+}$	For Negative Moment $M_{pl,Rd}^{-}$
EC4	Not defined	875 mm
EC8 Elastic analysis	525 mm	700 mm
EC8 Plastic Moments	1050 mm	1400 mm

Composite Steel Concrete Moment Resisting Frame



Composite beams **Composite columns** **Steel columns**

Check of c/t ⇔ classes of sections

= <u>condition 1 for local ductility in plastic hinges</u>

Composite beams with IPE330 & IPE360 => class 2
Steel columns with HEA360 & HEA450 => class 1
Composite columns HEA320 & HEA400 => class 1

Composite Steel Concrete Moment Resisting Frame

Remark

Favourable influence of concrete encasement on local ductility.

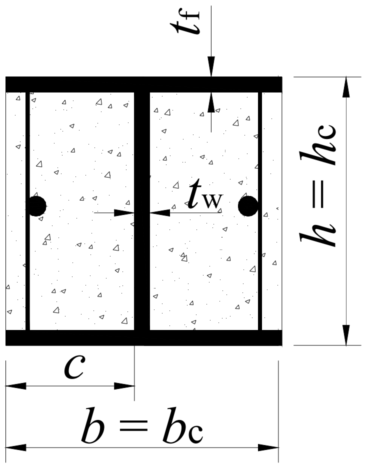
Concrete: - prevents inward local buckling of the steel walls

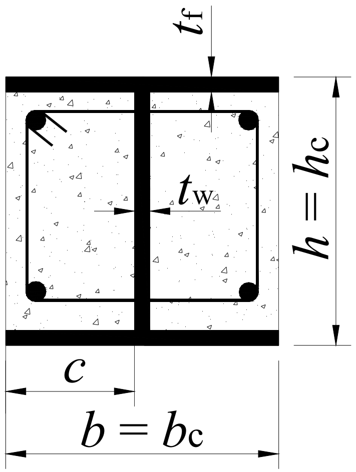
- reduces strength degradation

=> Limits c/t of wall slenderness of composite sections > those for pure steel sections

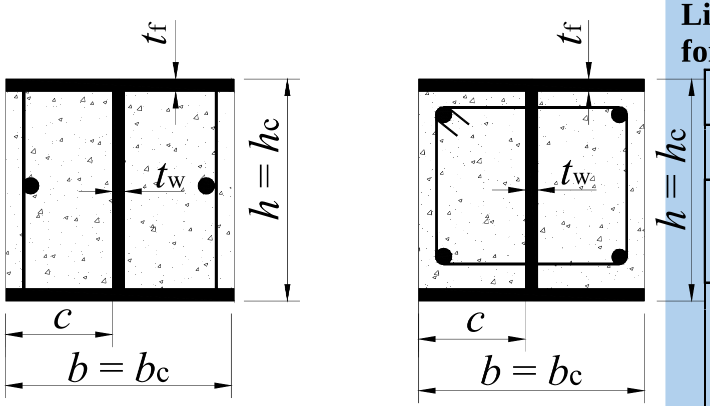
Increase up to 50% if:

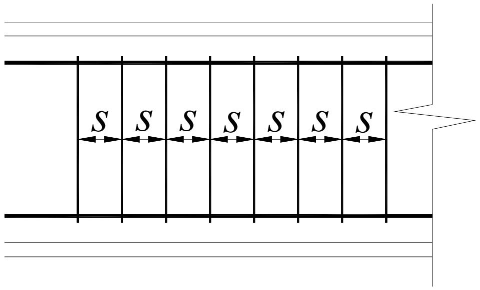
- **confining hoops fully encased sections**
- **additional straight bars welded to inside of flanges for partially encased sections**





Composite Steel Concrete Moment Resisting Frame





Limits of wall slenderness for steel and encased H and I sections for different design details and behaviour factors *q*.

Ductility Class of Structure	**DCM**		**DCH**
Reference value of behaviour factor q	$1,5 < q \leq 2$	$2 < q \leq 4$	$q > 4$
FLANGE outstand limits c/t_f Reference: H or I Section in steel only EN1993-1-1:2004 Table 5.2	14 ε	10 ε	9 ε
FLANGE outstand limits c/t_f H or I Section, partially encased, with connection of concrete to web as in Figure 57 b) or by welded studs. EN1994-1-1:2004 Table 5.2	20 ε	14 ε	9 ε
FLANGE outstand limits c/t_f H or I Section, partially encased + straight links as in Figure 57 a) placed with $s/c \leq 0,5$ EN1998-1-1:2004	30 ε	21 ε	13,5 ε
FLANGE outstand limits c/t_f H or I Section, fully encased + hoops placed with $s/c \leq 0,5$ EN1998-1-1:2004	30 ε	21 ε	13,5 ε
WEB depth to thickness limit c_w / t_w $c_w / t_w = h - 2t_f$ Reference: H or I Section, in steel only, web completely in compression EN1993-1-1:2004 Table 5.2	42ε	38 ε	33 ε
WEB depth to thickness limit c_w / t_w H or I Section, web completely in compression, section partially encased with connection of concrete to web or fully encased with hoops. EN1993-1-1:2004 Table 5.2, EN1994-1-1, cl.5.5.3(3)	38ε	38 ε	33 ε

note: $\varepsilon = (fy/235)^{0.5}$ with f_y in MPa

Composite Steel Concrete Moment Resisting Frame

Condition 2 for local ductility in plastic hinges H steel profile + slab

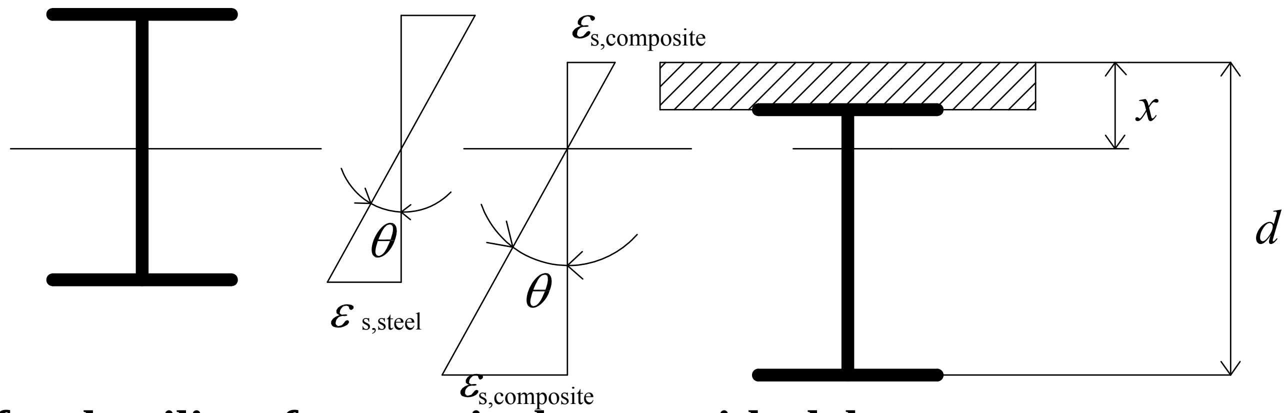
Steel yields: $\varepsilon > \varepsilon_y$ Concrete remain elastic: $\varepsilon < \varepsilon_{cu2}$

$\Rightarrow$ a condition on the position of the plastic neutral axis:

$$x / d < \varepsilon_{cu2} / (\varepsilon_{cu2} + \varepsilon_a)$$

x distance from top concrete compression fibre to plastic neutral axis

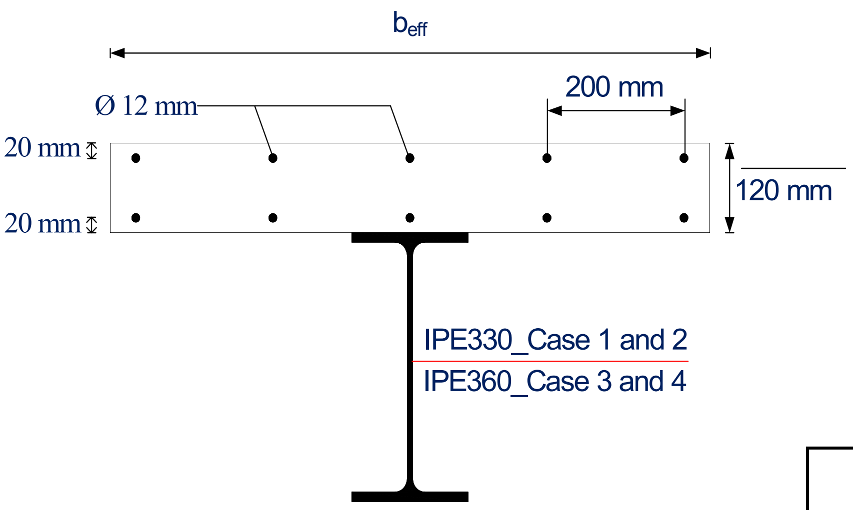
d depth of composite section ε_a total strain in steel at ULS

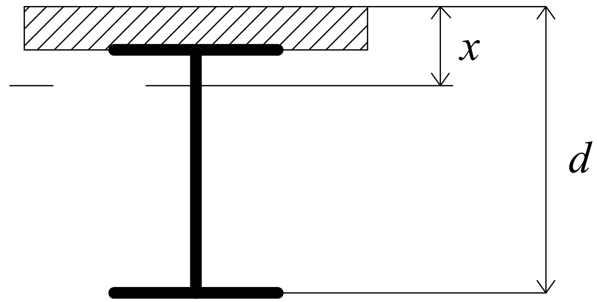


Limiting values of *x*/*d* for ductility of composite beams with slab

Ductility class	q	f_y (N/mm^2)	x/d upper limit
DCM	$1,5 < q \leq 4$	355	0,27
	$1,5 < q \leq 4$	235	0,36
DCH	$q > 4$	355	0,20
	$q > 4$	235	0,27

Composite Steel Concrete Moment Resisting Frame

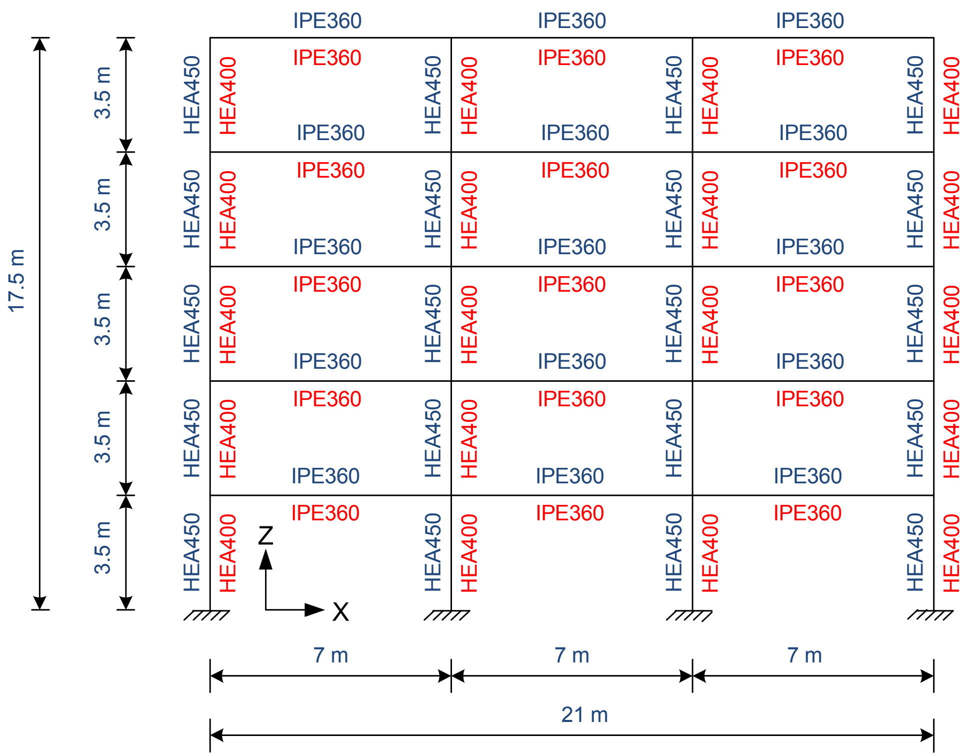




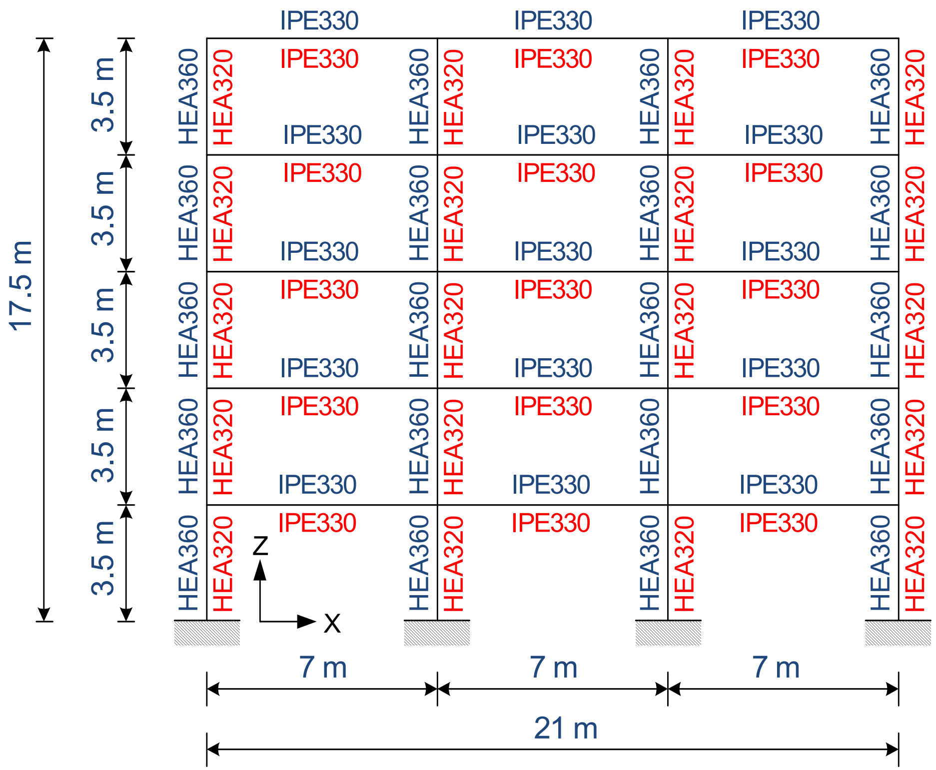
	Case1 IPE330	Case2 IPE330	Case3 IPE360	Case4 IPE360
(x/d) Limit values EC8	0.27	0.27	0.36	0.36
$(x/d)_{max}$ Design values	0.268	0.268	0.239	0.239

Composite Steel Concrete Moment Resisting Frame

Analysis



High seismicity



Low seismicity

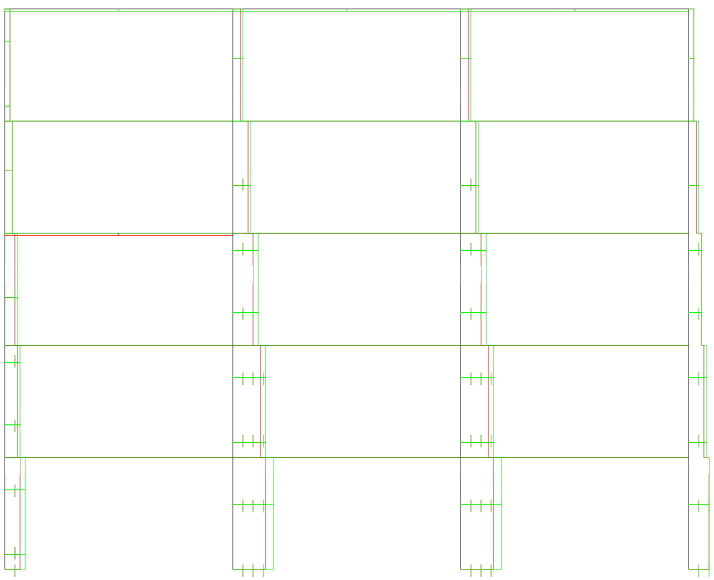
Blue: with steel column

Red: with composite column

All beams are composite

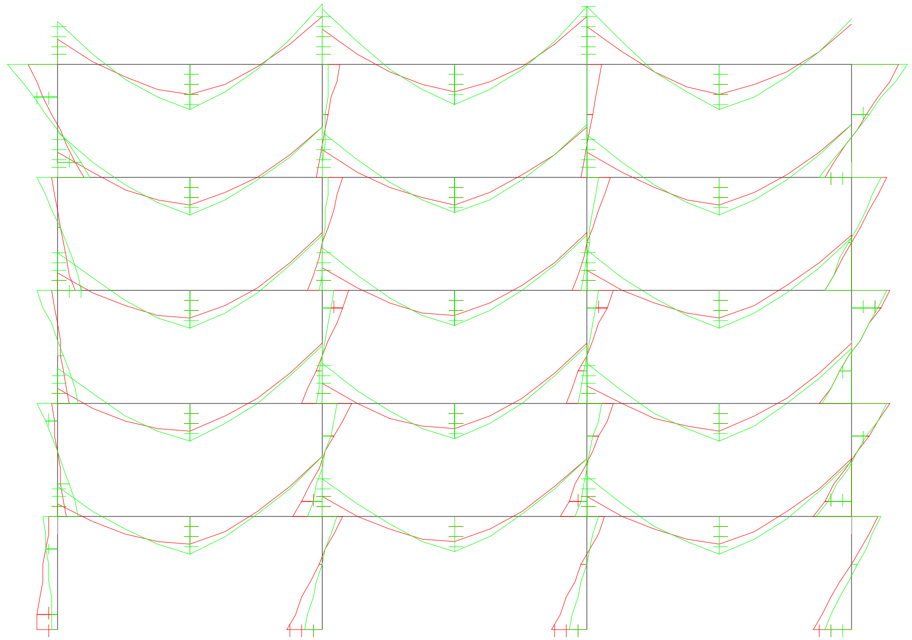
Composite Steel Concrete Moment Resisting Frame

Results of analysis Example



Axial force diagram

N_{max} = 1980 kN



Bending moment diagram

$M_{z,max}$ = 319 kNm

High seismicity – steel columns

Composite Steel Concrete Moment Resisting Frame

EC8 check

Resistance of dissipative zones

Check: plastic hinges at beam ends

$M_{pl,Rd}^{+} \geq M_{Ed}^{+}$

$M_{pl,Rd}^{-} \geq M_{Ed}^{-}$

$M_{pl,Rd}^{-}$

$$M^{-} = W_{plb} * f_y$$

$$M^{-} = \begin{cases} 342 \text{ kN.m (IPE330)} \\ 317 \text{ kN.m (IPE360)} \end{cases}$$

$M_{pl,Rd}^{+}$

$$M^{+} = W_{plb} * f_y$$

$$M^{+} = \begin{cases} 495 \text{ kN.m (IPE330)} \\ 415 \text{ kN.m (IPE360)} \end{cases}$$

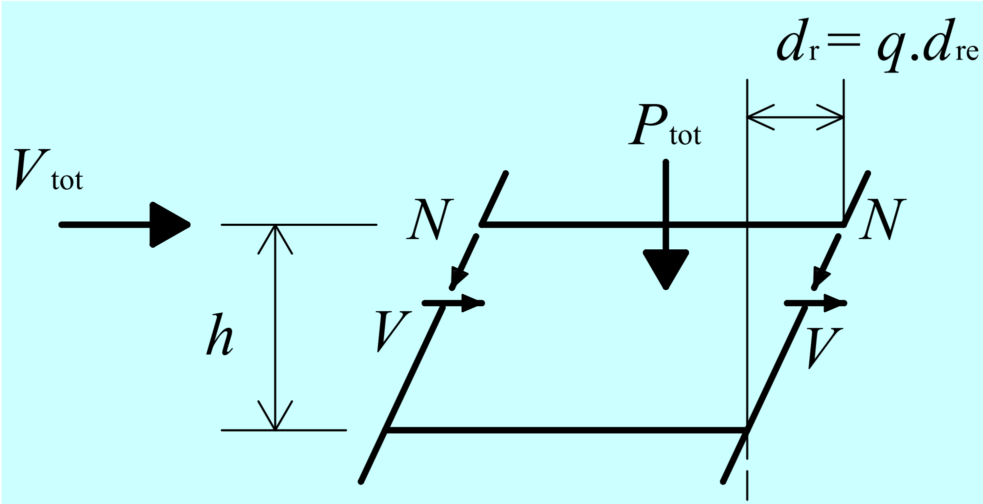
	Maximum "work rate" in beams: M_{Ed} /$M_{pl,Rd}$		Ω_{min}
	Static Actions (EC4)	Seismic Actions (EC8)	Ω_{min} = $M_{pl,Rd}$ / M_{Ed}
Case 1 : high seismicity (steel columns)	0.933	0.826	1,21
Case 2 : high seismicity (composite columns)	0.953	0.840	1,19
Case 3 : low seismicity (steel columns)	0.979	0.764	1,31
Case 4 : low seismicity composite columns)	1.000	0.779	1,28

=> Limited overstrength Ω_{min}

Composite Steel Concrete Moment Resisting Frame

EC8 check
Second order effects

$$\theta = \frac{P_{tot} * d_r}{V_{tot} * h} \leq 0.1$$



Example
High seismicity – steel columns

Storey N°.	d_e [m]	[m]	V [kN]	V_{tot} [kN]	P_{tot} [kN]	θ
1	0.007	0.007	15.70	235.48	3799.96	0.032
2	0.019	0.012	31.40	219.78	3046.62	0.048
3	0.030	0.011	47.10	188.38	2293.28	0.038
4	0.038	0.008	62.79	141.28	1539.94	0.025
5	0.044	0.006	78.49	78.49	786.60	0.017

=> All θ < 0,10

Composite Steel Concrete Moment Resisting Frame

EC8 check
Damage limitations in non-structural elements

$$d_r * v \leq 0.010h \quad \text{with} \quad dr = q * d_r^e$$

***q*=4** **v=0,5**

	d_r * v (mm)				
Storey	Case 1	Case 2	Case 3	Case 4	0,010 h (mm)
1	14	16	4	4	35
2	24	26	8	10	35
3	22	22	8	6	35
4	16	18	6	6	35
5	12	10	4	6	35

All d_r < 0,10h => OK

Composite Steel Concrete Moment Resisting Frame

Elements checks

Action effects to consider are:

$$N_{\mathrm{Ed}} = N_{\mathrm{Ed,G}} + 1{,}1\gamma_{\mathrm{ov}}\,\Omega N_{\mathrm{Ed,E}}$$

$$M_{\mathrm{Ed}} = M_{\mathrm{Ed,G}} + 1{,}1\gamma_{\mathrm{ov}}\,\Omega M_{\mathrm{Ed,E}}$$

$$V_{\mathrm{Ed}} = V_{\mathrm{Ed,G}} + 1{,}1\gamma_{\mathrm{ov}}\,\Omega V_{\mathrm{Ed,E}}$$

They take into account:

- Section overstrength $\Omega = M_{pl,Rd} / M_{Ed}$
- Material overstrength $f_{y,real} / f_{y,nominal} = \gamma_{ov}$

$$\Omega = \min_i \left\{ \Omega_i = M_{pl,Rd,i} / \left| M_{Ed} \right|_{\max,i} \right\}$$

$$\Omega = \frac{393}{324.20} = 1.212 \text{ (Case1)}$$

$$\Omega = \frac{337}{257.00} = 1.311 \text{ (Case3)}$$

CHECKS

► **Beam deflections**

$$f = \frac{W_u L^4}{384EI} + \frac{W_p L^3}{192EI} = \frac{L}{300}$$

Composite Steel Concrete Moment Resisting Frame

► Resistance of beams to Lateral-Torsional Buckling

$$M_{cr} = \frac{k_c C_4}{L}\left[\left(GI_{at} + \frac{k_s L^2}{\pi^2}\right)E_a I_{afz}\right]^{0.5}$$

Real risk: $\left|M_{Ed}\right|_{\max} > M_{b,Rd}$

$\Rightarrow$ **Bracings required**

$\Rightarrow$ **Calculation indicate 1 m interdistance OK**

► Limitation of compression in beams

$$N_{Pl,Rd} = A_a * f_y + \frac{f_{sk} * A_s}{\gamma_s} + 0.85 * \frac{f_{ck} * A_c}{\gamma_c}$$

check: $\frac{N_{Ed}}{N_{pl,Rd}} \leq 0.15$

$$N_{Pl,Rd} = \begin{cases} 5767 \text{ kN} & (\text{IPE330}) \\ 4708 \text{ kN} & (\text{IPE360}) \end{cases}$$

$$\left|N_{Ed}\right|_{\max} = \begin{cases} 149 \text{ kN} < 0.15\, N_{Pl,Rd} = 865 \text{ kN} & (\text{Case1}) \\ 142 \text{ kN} < 0.15\, N_{Pl,Rd} = 865 \text{ kN} & (\text{Case2}) \\ 127 \text{ kN} < 0.15\, N_{Pl,Rd} = 706 \text{ kN} & (\text{Case3}) \\ 121 \text{ kN} < 0.15\, N_{Pl,Rd} = 706 \text{ kN} & (\text{Case4}) \end{cases}$$

Composite Steel Concrete Moment Resisting Frame

► Limitation of shear in beams

$$\left|V_{Ed}\right|_{\max} = \begin{cases} 234\text{ kN } < 0.5V_{Pl,a,Rd} = 315.5\text{ kN (Case1)} \\ 237\text{ kN } < 0.5V_{Pl,a,Rd} = 315.5\text{ kN (Case2)} \\ 231\text{ kN } < 0.5V_{Pl,a,Rd} = 238.5\text{ kN (Case3)} \\ 234\text{ kN } < 0.5V_{Pl,a,Rd} = 238.5\text{ kN (Case4)} \end{cases}$$

► Resistance of columns under combined compression and bending in seismic design situation

$$M_{Ed} \le M_{N,Rd}$$

Example: High seismicity, steel columns

case 1		$N_{Ed,G}$	$M_{Ed,G}$	$N_{Ed,E}$	$M_{Ed,E}$	N^{*}_{Ed}	M^{*}_{Ed}	$M_{N,y,Rd}$
	End	kN	kNm	kN	kNm	kN	kNm	kNm
column 1	lower	-814	-41	119	140	-616	192	751
	upper	-810	79	119	-39	-612	14	751
column 2	lower	-1652	1	-9	158	-1666	264	574
	upper	-1648	-3	-9	-76	-1663	-130	574
column 3	lower	-1652	-1	8	158	-1638	262	578
	upper	-1648	3	8	-76	-1634	-124	579
column 4	lower	-814	41	-118	138	-1011	272	684
	upper	-810	-79	-118	-39	-1007	-143	685

Composite Steel Concrete Moment Resisting Frame

► Shear Resistance of Steel Columns

Example: case 1, high seismicity, steel columns

$$\left|V_{Ed,G}\right|_{\max} = 57.54 \text{ kN}$$

$$\left|V_{Ed,E}\right|_{\max} = \frac{1}{1-\theta}*39.96 = \frac{1}{1-0.048}*39.96$$

$$= 1.05*39.96 = 41.80 \text{ kN}$$

$$\left|V^{*}_{Ed}\right|_{\max} = \left|V_{Ed,G} + 1{,}1\gamma_{ov}\Omega\, V_{Ed,E}\right|_{\max}$$

$$\left|V^{*}_{Ed}\right|_{\max} = 127.47 \text{ kN}$$

$$V_{Pl,a,Rd} = \frac{A_v * f_y}{\sqrt{3}} = \begin{cases} 1003.48 \text{ kN (Case1)} \\ 892.490 \text{ kN (Case3)} \end{cases}$$

Composite Steel Concrete Moment Resisting Frame

► <u>Column buckling</u> Buckling length = storey height

Reduction factors χ for Flexural Buckling

	$\bar{\lambda}_y$	χ_y	$\bar{\lambda}_z$	χ_z
Case 1	0.308	0.961	0.632	0.766
Case 3	0.202	1.000	0.524	0.873

Interaction factors k_{yy} and k_{zz} for uneven moments at column ends

$$k_{yy} = C_{my}\left[1 + \left(\bar{\lambda}_y - 0{,}2\right)\frac{\left|N^*_{Ed}\right|}{\chi_y N_{plRd}}\right]$$

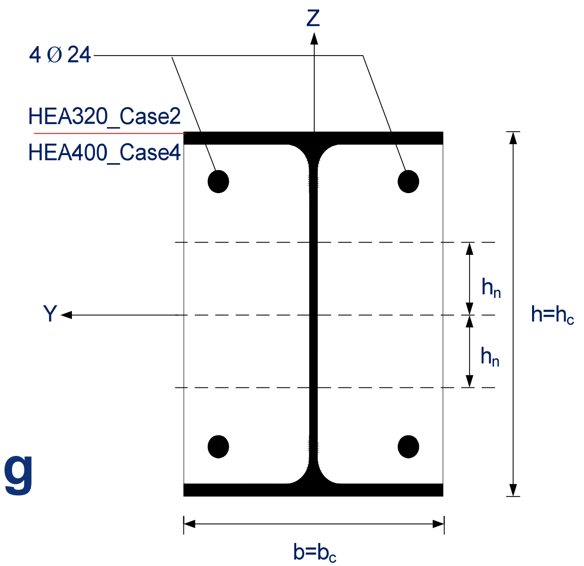
Reduction Factor for Lateral Torsional-Buckling

Stability checks

$$\frac{\left|N^*_{Ed}\right|}{\chi_y N_{plRd}} + k_{yy}\frac{\left|M^*_{y,Ed}\right|_{max}}{\chi_{LT} M_{plRd}} \leq 1$$

$$\frac{\left|N^*_{Ed}\right|}{\chi_z N_{plRd}} + k_{zy}\frac{\left|M^*_{y,Ed}\right|_{max}}{\chi_{LT} M_{plRd}} \leq 1$$

Composite Steel Concrete Moment Resisting Frame



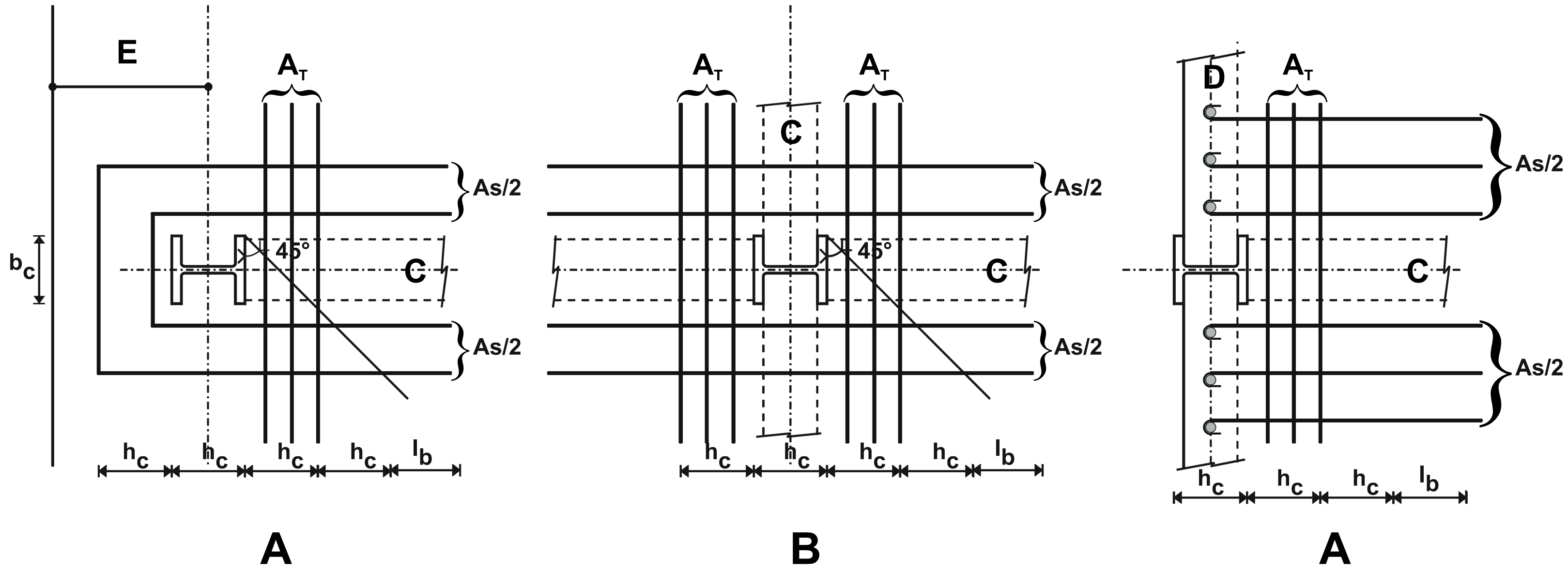
Additional aspects for composite columns

- **Spacing of reinforcing steel bars**
- **Local buckling => section class**
- **Resistance of composite columns in bending**
  can consider concrete and rebars
  Longitudinal shear to check at steel concrete interface
- **Resistance of composite sections in compression**
  can consider concrete and rebars
- **Shear resistance of composite sections**
  In dissipative zones: only the shear resistance of the steel profile
- **Second order effects in composite columns (static combination)**

Composite Steel Concrete Moment Resisting Frame

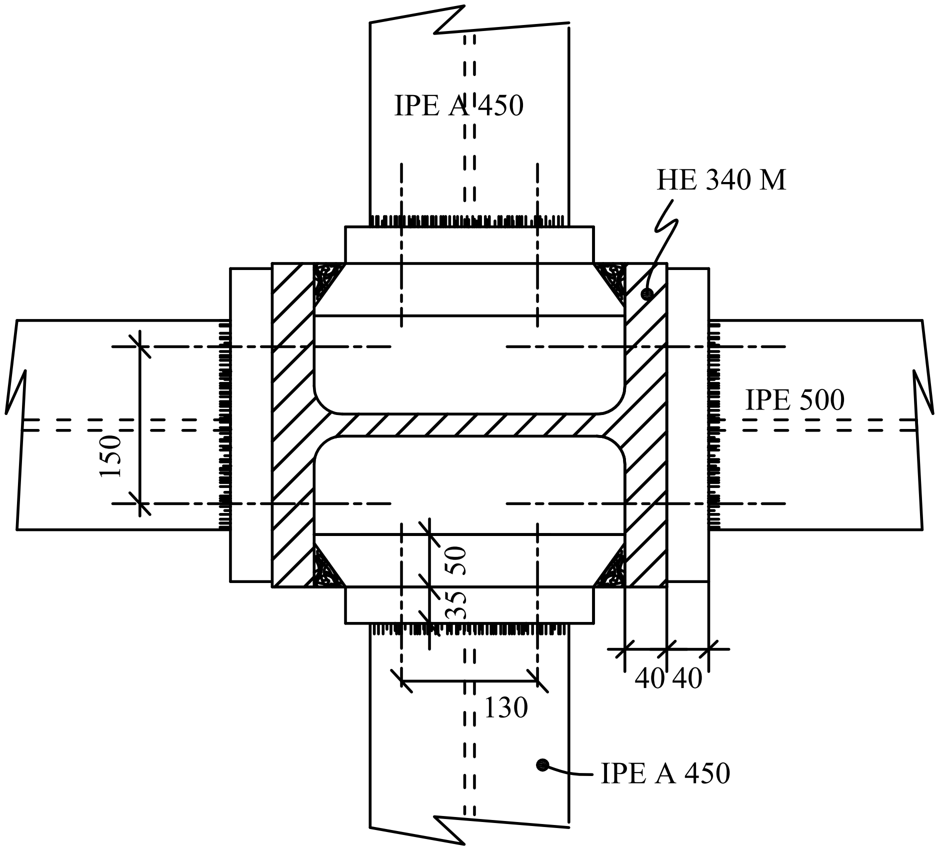
Beam to column connection

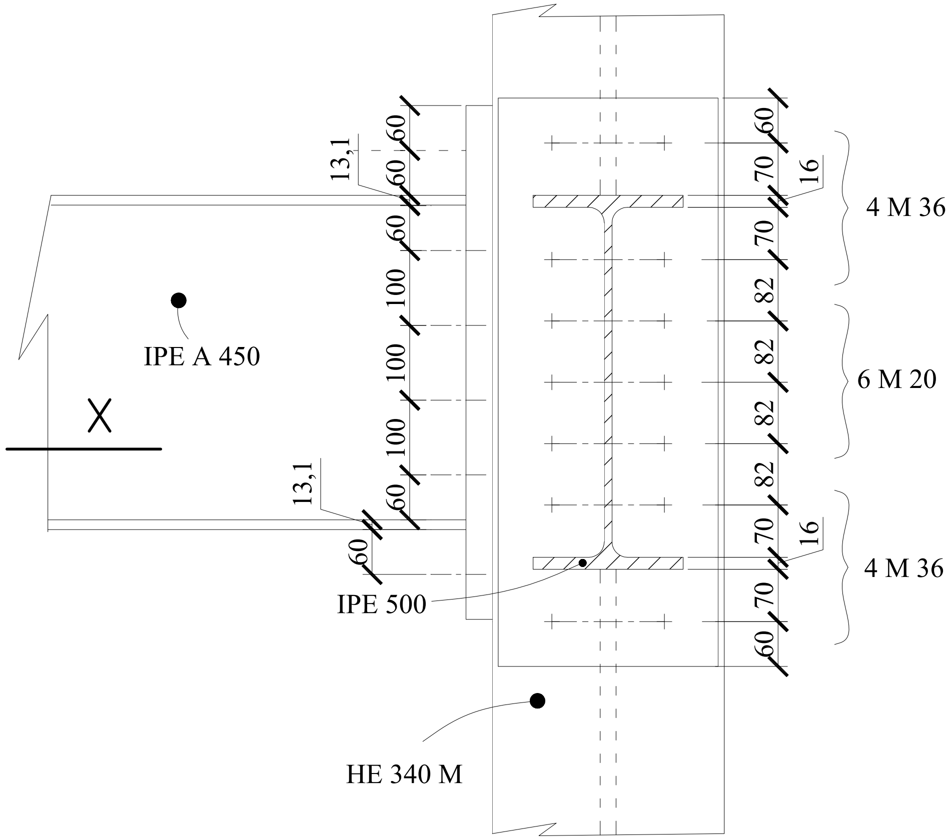
► **In the beam column connection zone of beams (=dissipative zones) specific reinforcement of the slab: “Seismic Re-bars” (EC8 Annex C)**



Composite Steel Concrete Moment Resisting Frame

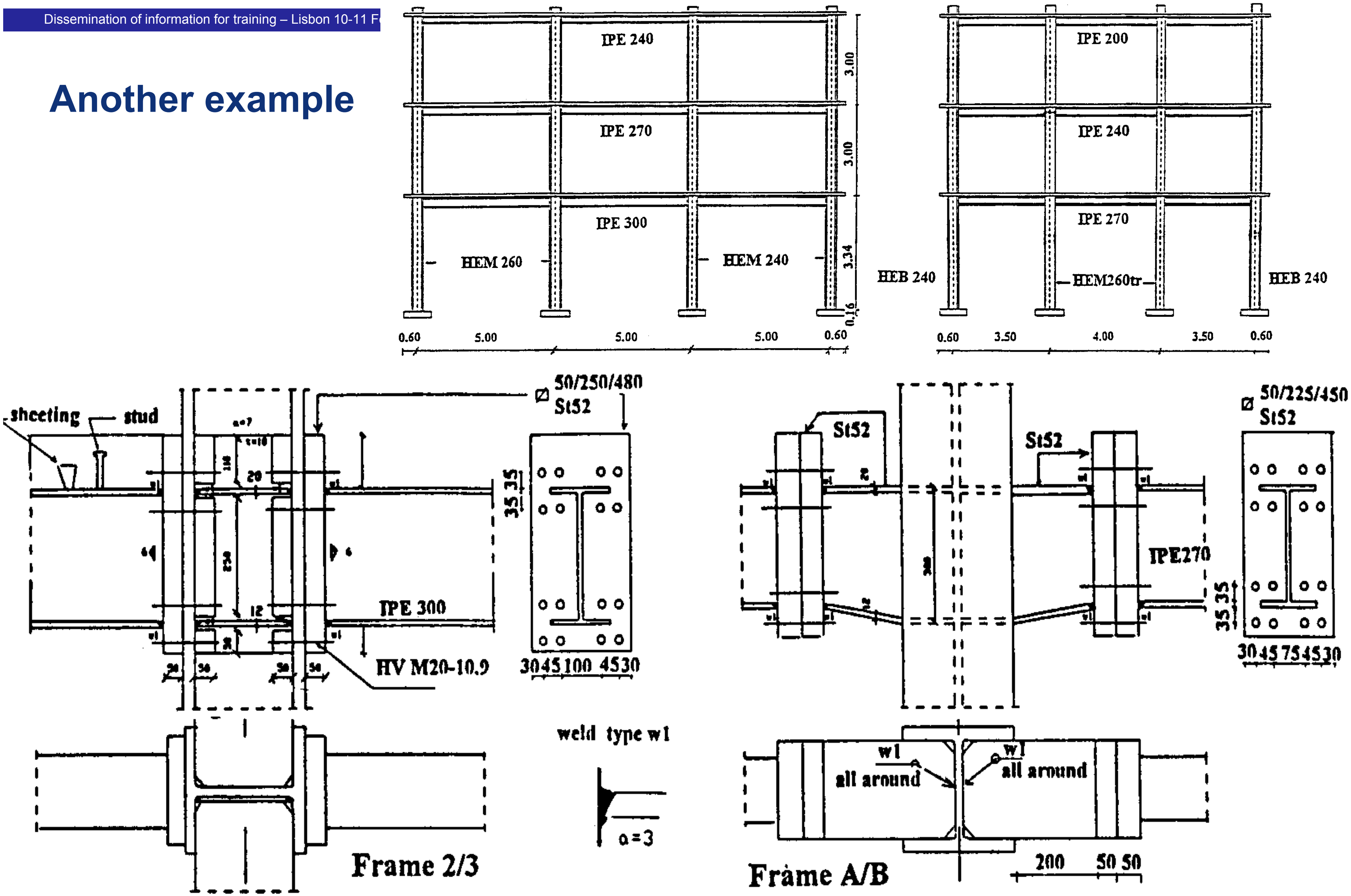
► **The connection of the steel beam to the column: a full strength steel connection: can be that of steel MRF example**





Composite Steel Concrete Moment Resisting Frame

Another example



Composite Steel Concrete Moment Resisting Frame

Ispra test 1999

Composite Steel Concrete Moment Resisting Frame

Design to transmit slab compression/tension force

IPE330 beam HEA360 column t_{slab}=120mm

B_{eff}^{+}= 1050mm B_{eff}^{+}= 1400mm

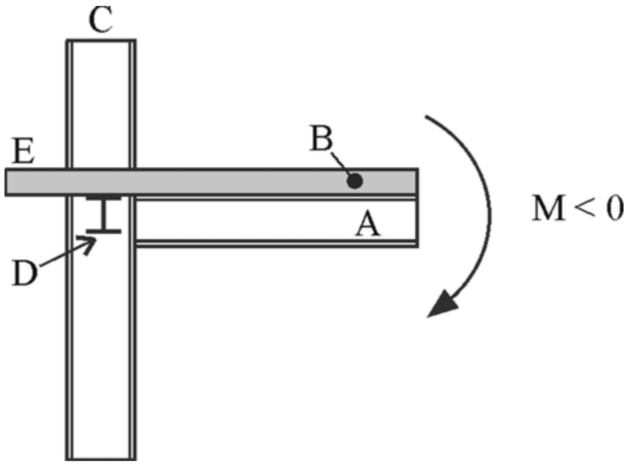
Rebars: S500 T12@200 – 2 layers A_{sl}=14x113=1582 mm² F_{Rds}=791 kN

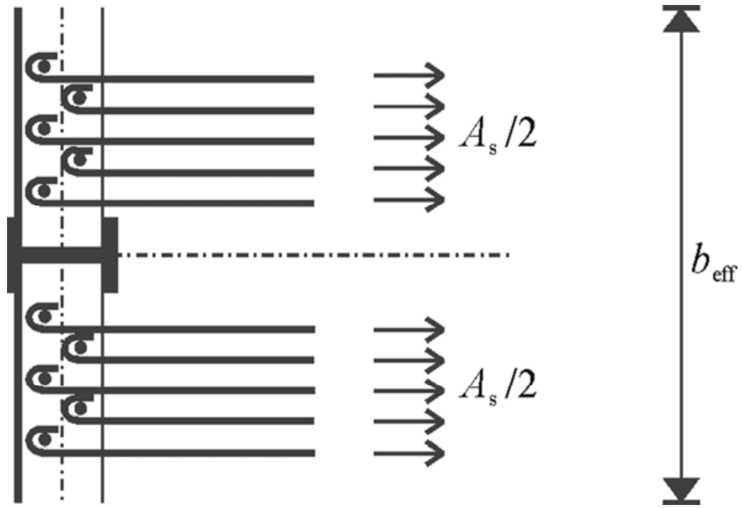
Concrete: C30/37 F_{cd}=30/1,5=20 MPa F_{Rdc}=120x1050x20=2520 kN

- F_{Rds} and F_{Rdc} are the slab force in tension and compression
- They are transmitted to the column to transmit the beam composite plastic moments M_{pl}+ & M_{pl}-

Facade beam-column connection

M-





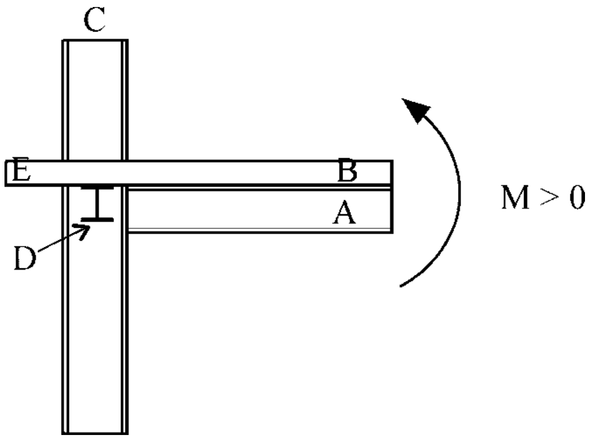
Each rebar: 113 mm² x500 = 56,5kN

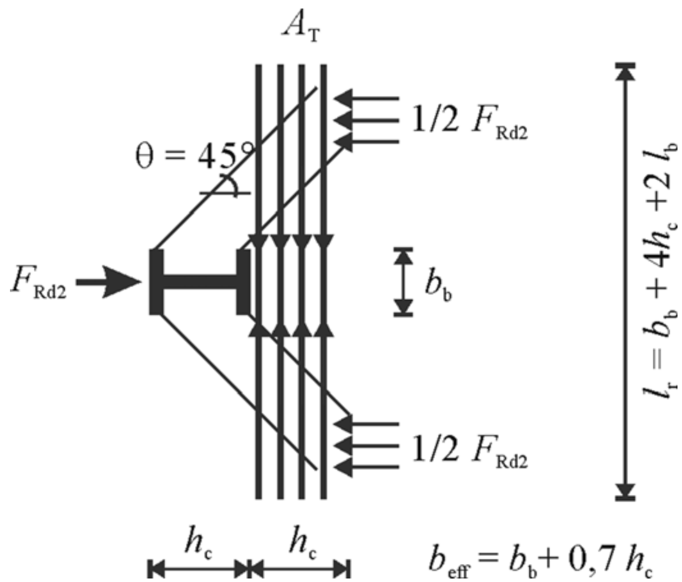
1 stud/rebar 1 stud Φ19=81,6kN>56,5

Composite Steel Concrete Moment Resisting Frame

Facade beam-column connection

M+



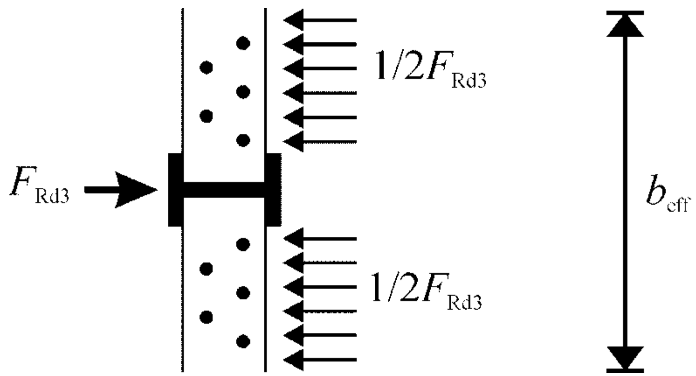


$F_{Rd1} = b_{column} x t_{slab} x f_{cd} = 300x120x20 = 720\ kN$

$F_{Rd2} = h_{column} x t_{slab} x 0{,}7 f_{cd} = 360x120x0{,}7x20 = 604\ kN$

$F_{Rd3} = n_{stud} x\ F_{R,stud} = 14x81{,}6 = 1142\ kN$

Total: $2466 kN \approx 2520\ kN = F_{Rdc}$

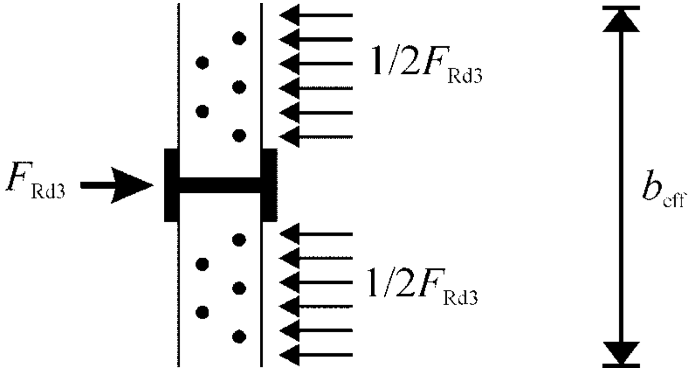


« Seismic rebars » for F_{Rd2} /2

$\Rightarrow A_T = 302000/500 = 604 mm^2$ => 4T16

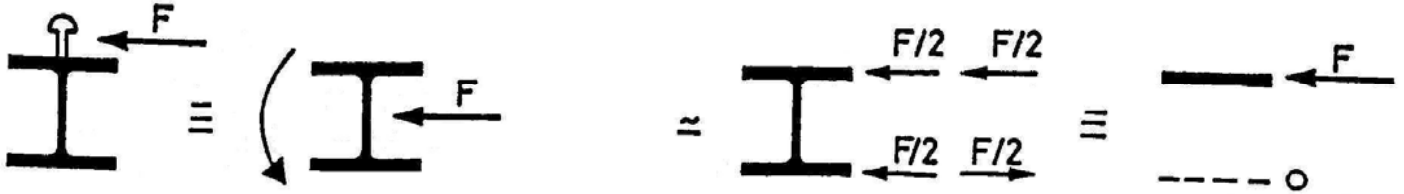
Composite Steel Concrete Moment Resisting Frame

Facade beam-column connection



Check of upper flange in bending+shear due to F_{Rd3} /2 V_E = 571 kN

M_E = 571 x 0,55/2 = 108 kNm



With cover plate t=16mm welded on top of IPE330 beam

M_{plRd} = 16x315²x355/4=140 kNm >108

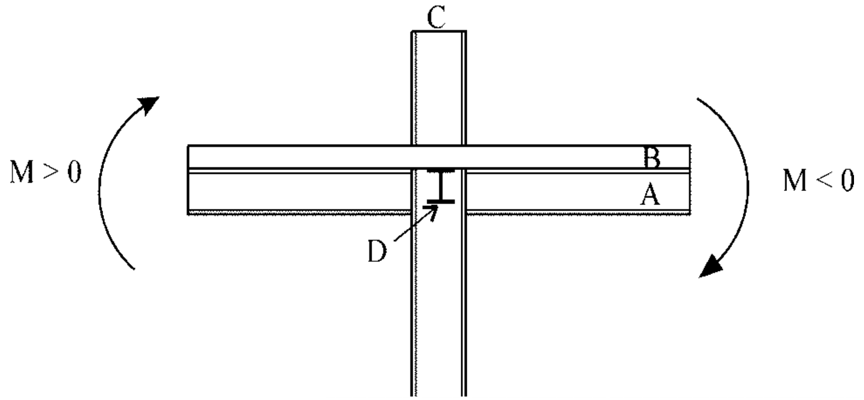
V_{plRd} = 16x315x205=1033kN > 571

Interaction M-N $\rho=(2x571/1033- 1)^2=0{,}01$

=> M_{plRd} unchanged OK

Composite Steel Concrete Moment Resisting Frame

Interior beam-column connection

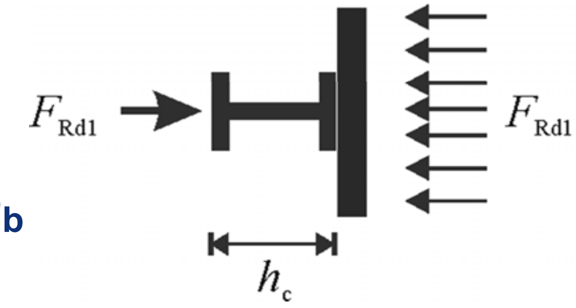


As M+ on 1 side & M- on other side, slab force to transmit:

$F_{Rdc} + F_{Rds}$ = 791+2950 = 3311 kN

791 kN more than in facade connection

Various possible design:



► increase F_{Rd1} = increase column bearing width b_b

but F_{Rd2} =604 kN is lost

With column HEA 360 flange: F_{Rd1} = 720 kN

Width b_b to provide F_{Rd1} = 791+604= 1395 kN

b_b=1395000/(120x20)=581mm

=> (581-300)/2 =140 mm extension both side (+ stiffeners)

Composite Steel Concrete Moment Resisting Frame

Interior beam-column connection

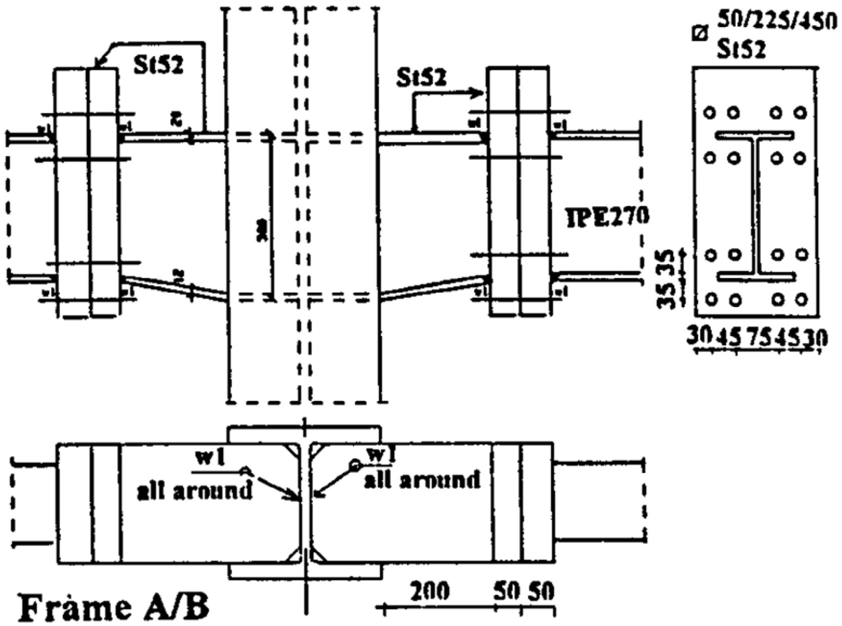
- ► increase F_{Rd2} not possible
- ► increase F_{Rd3} => more studs

  For 791 kN => 791/81,6 =10 studs 5 each side

  + cover plate with increased M_{plRd}&V_{plRd}
- ► design should consider beams present in 2 directions
- ► some other constraints may bring part of the solution

Example:

- increased flange width is anyway part of the design for connection to column weak axis
- connecting plates bring frontal surface within slab thickness allowing to reduce the number of connectors



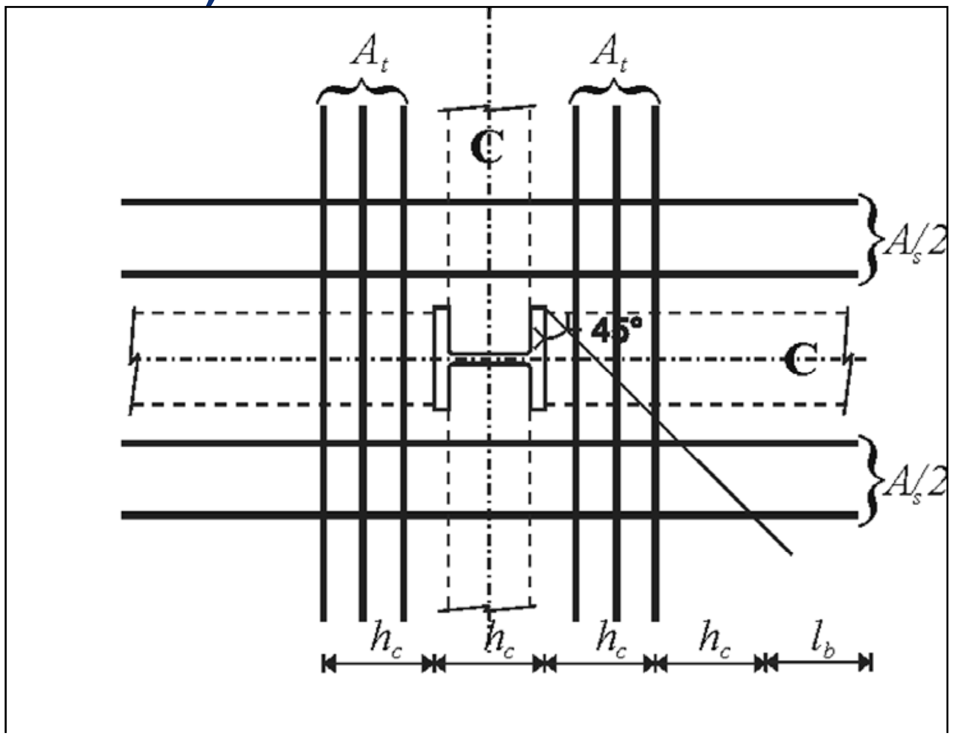
Composite Steel Concrete Moment Resisting Frame

Interior beam-column connection

« Seismic rebars »

► F_{Rd2} and A_T= 4T16 unchanged

► placed on both sides (moment reversal)



Some other aspects

of

Seismic Design

of

Composite Steel Concrete Structures

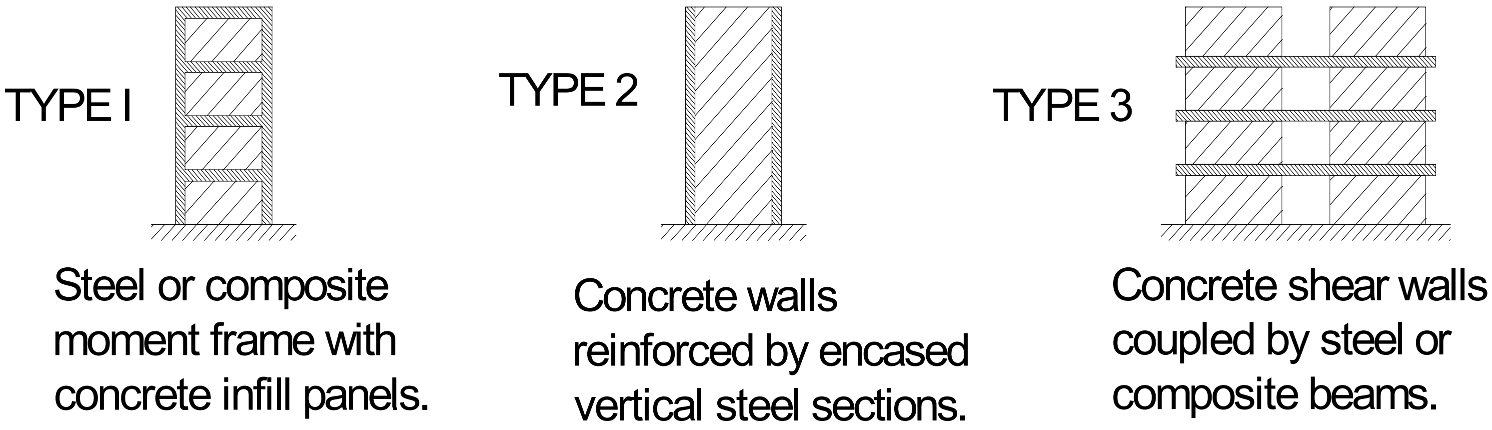
Composite Steel Concrete Structure

Structural Types

- Moment resisting frames
- Frames with concentric bracing
- Frames with eccentric bracings

Specific

- Composite wall structures Type 1 and 2
- Mixed systems Type 3 = Concrete walls/columns.
  Steel or composite beams



- Composite steel plate shear walls

Composite Steel Concrete Structure

A choice in the design: the degree of composite 'character'

► 1. Ductile composite elements/connections

► 2. Ductile steel sections, no input of concrete to resistance of dissipative zones

Option 2 ● ease analysis & execution

● but requires effective disconnection of concrete from steel in potential dissipative zones

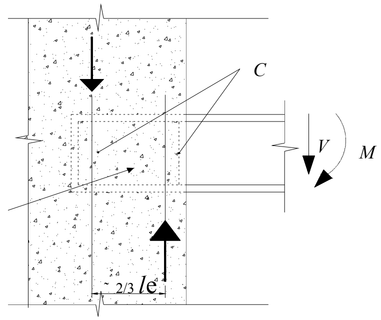
=> correspondence between model and reality

Underestimating stiffness: T ↑ => smaller action effects

Underestimating resistance: capacity designed may be incorrect

=> Risk of failure in the wrong places

Composite Steel Concrete Structure



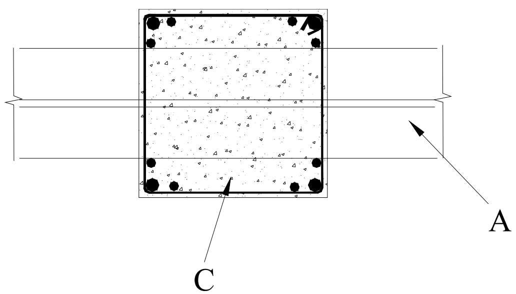
Composite connections in dissipative zones

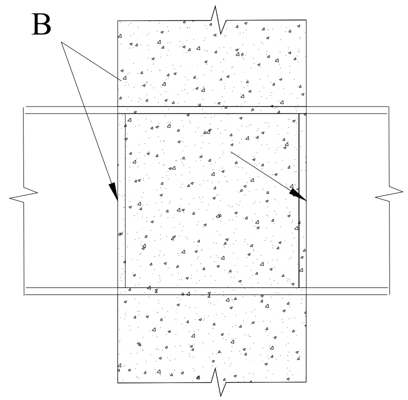
Transfer of bending moment
and shear from beam to RC column
Not treated in EC4

Realised by couple of vertical reactions in concrete
Should be checked:

- ►Capacity of column to bear locally those forces without crushing
  => confining (transverse) reinforcement + "face bearing plates"
- ►Capacity of column to resist locally tension mobilised by vertical forces

=> vertical reinforcements with strength equal to shear in beam
confinement by transverse reinforcement design like RC
+ face bearing plates B





A steel beam
B face bearing plates
C reinforced concrete column

Composite frames with eccentric bracings

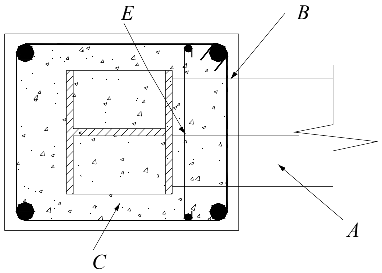
► **Uncertainties with composite components in EBF’s:**
- capacity at large deformations (rotations up to 80 mrad)
- ‘disconnection’ of the slab
- contribution of slab in bending at rotations up to 80 mrad

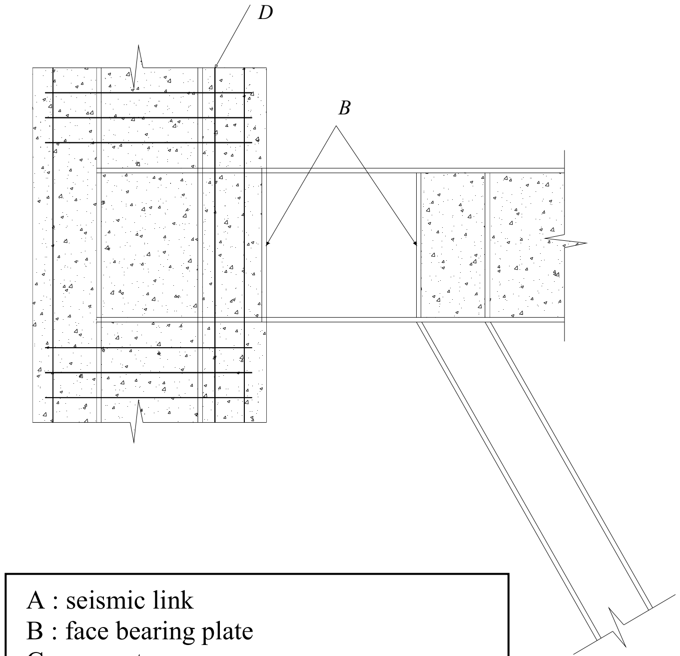
► **Design: dissipative behaviour through yielding in shear of the links**
contribution of slab to shear resistance negligible
=> Links should be short or intermediate length

► **Links may not be encased steel sections**
uncertainties about concrete contribution to shear resistance

► **Vertical steel links: OK**

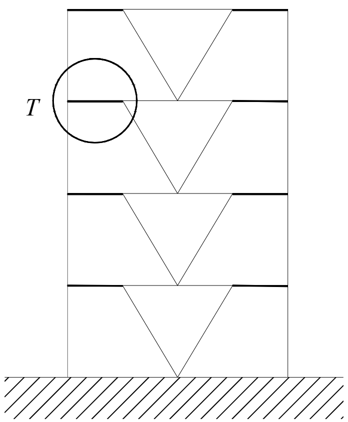
Composite Steel Concrete Structure





Composite frames with eccentric bracings

A : seismic link
B : face bearing plate
C : concrete
D : additional longitudinal rebars
E : confining ties



Specific construction details

- ► **B** face bearing plates for links framing into reinforced concrete columns
- ► **E** transverse reinforcement in 'critical regions' of fully encased composite columns adjacent to links

Composite Frame with Eccentric and Concentric Steel Bracings

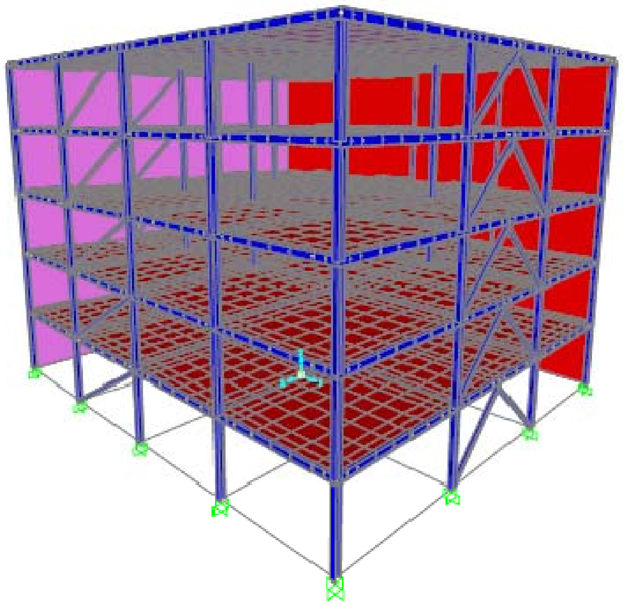
Hervé DEGEE
University of Liege

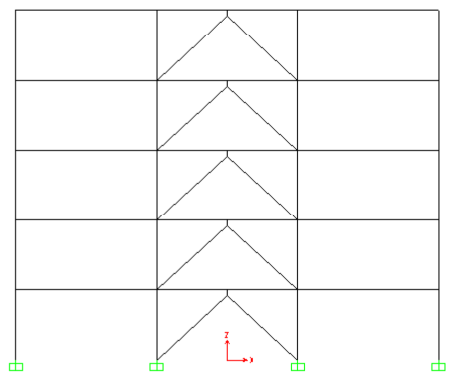
André PLUMIER
University of Liege

Composite frame with Eccentric and Concentric Bracings

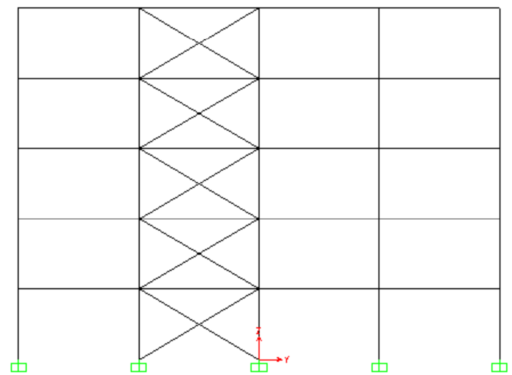
Definition of the structure

Dimensions	Symbol	Value
Storey height	h	3.5 m
Total height of the building	H	17.5 m
Beam length in X-direction EBF	l_X	7 m
Beam length in Y-direction CBF	l_Y	6 m
Building width in X-direction	L_X	21 m
Building width in Y-direction	L_Y	24 m





X-direction – Eccentric bracings



Y-direction – Concentric bracings

Composite frame with Eccentric and Concentric Bracings

Details of values			
Dimensions	Symbol	Value	Units
Characteristic yield strength of reinforcing steel	f_y	500	N/mm²
Partial safety factor for steel rebars	γ_s	1.15	
Design yield strength of reinforcement steel	f_{yd}	434.78	N/mm²
Characteristic compressive strength of concrete	f_c	30	N/mm²
Partial safety factor for concrete	γ_c	1.5	
Design compressive strength of concrete	f_{cd}	20	N/mm²
Secant modulus of elasticity of concrete for the design under gravity loads combinations	E_c	33000	N/mm²
Secant modulus of elasticity of concrete for the design under seismic loads combination	$E_{c,sc}$	16500	N/mm²
Characteristic yield strength of steel profile	f_y	355	N/mm²
Partial factor for steel profile		1	
Modulus of elasticity of steel profile	E_a	210000	N/mm²

Composite frame with Eccentric and Concentric Bracings

Earthquake action

Design ground acceleration 0.25g

soil type B

type 1 response spectrum

DCM design with a behaviour factor q = 4

Type 1 response spectrum - soil type B			
Dimensions	Symbol	Value	Units
Soil factor	S	1.2	
Lower limit of period of constant spectral acceleration branch	T_B	0.15	s
Upper limit of period of constant spectral acceleration branch	T_C	0.5	s
Beginning of the constant displacement response range	T_D	2	s

Composite frame with Eccentric and Concentric Bracings

Loads

Permanent actions + self-weight of the slab $G = 5.858\ kN/m^2$

Variable actions $Q = 3kN/m^2$

Snow $S = 1.11\ kN/m^2$

Wind $W = 1.4\ kN/m^2$

Static loading combinations:

1. $1.35G + 1.5\ W + 1.5\ (0.7Q + 0.5S)$
2. $1.35G + 1.5\ Q + 1.5\ (0.7W + 0.5S)$
3. $1.35G + 1.5\ Q + 1.5\ (0.7S + 0.5W)$
4. $1.35G + 1.5\ S + 1.5\ (0.7Q + 0.5W)$
5. $1.35G + 1.5\ S + 1.5\ (0.7W + 0.5Q)$
6. $1.35G + 1.5\ W + 0.7*1.5\ (Q + S)$
7. $1.35G + 1.5\ (Q + S) + 0.7*1.5\ (W)$

Seismic combination: $G + Q + \psi_{2i} E$ $\psi_{2i} = 0.3$

Seismic mass m = $\sum G_{kj} + \sum \psi_{Ei} \cdot Q_{ki}$ $\varphi = 0.8$ $\psi_{E,i} = \psi_{2,i}\ \varphi = 0{,}24$

Composite frame with Eccentric and Concentric Bracings

Steps

General. Design of slab under gravity loads (no support of EBF) I
Design of columns under gravity loads (no support of EBF) I
Design of beams under gravity loads (no support of EBF) I
Not presented – available in text I

Torsion effects

EBF 2nd order effects P-Δ
Design of eccentric bracings under seismic combination of loads including torsion and P- Δ
Check of beams and of eccentric bracings under gravity loads with EBF as support to the beam
Design of one link connection

CBF Design of concentric bracings under seismic combination of loads including torsion and P- Δ
Check of beams and columns
Design of one diagonal connection

Check of diaphragm

Check of secondary elements

Composite frame with Eccentric and Concentric Bracings

Final design

Composite aspect	Reinforced concrete slab thickness = 18 cm	
	Composite beam steel profiles: IPE 270	
Columns	HE 260 B	HE 280 B
Concentric bracings:	2 UPE	
Eccentric bracings:	HE	

Seismic mass: 1744 tons

Fundamental periods $T_X = 0.83$ s $T_Y = 1.45$ s

Beams considered composite in main span

Slab not connected to columns=> no composite moment frame

=> Primary resisting system = bracings

Secondary: moment frames

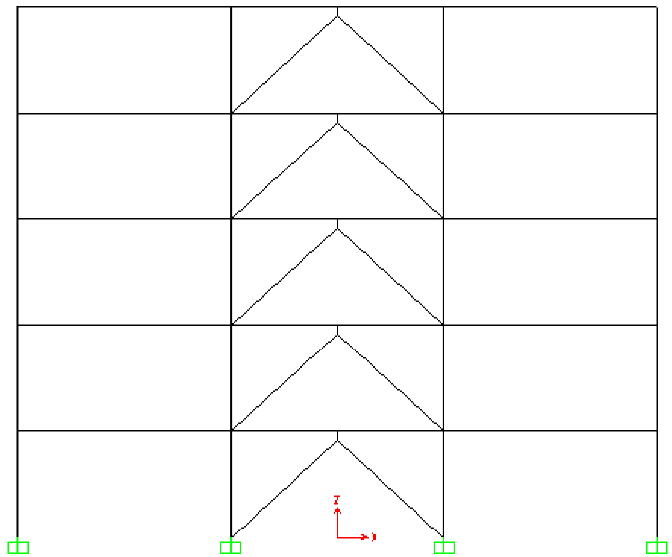
Composite frame with Eccentric and Concentric Bracings

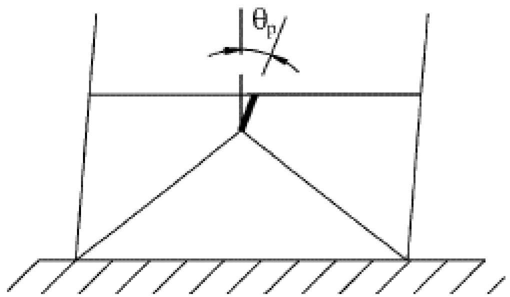
Slab slab thickness = 180 mm cover = 20 mm

Characteristics of slabs					
X-direction					
	Applied moment $M_{Ed,slab,X,GC}$	Resistant moment $M_{Rd,slab,X}$	Rebars for 1m of slab	Steel Section $A_{s,X}$	Spacing of rebars
Unit	[kNm/m]	[kNm/m]	[mm]	[mm²/m]	[mm]
SPAN (lower layer of rebars)	66	73	10 T10 + 2 T16	1187	100 – 50
SUPPORT (upper layer of rebars)	92	95	10 T10 + 4 T16	1585	100 – 50
Y-direction					
SPAN (lower layer of rebars)	35	49	10 T10	785	100
SUPPORT (upper layer of rebars)	41	49	10 T10	785	100

Composite frame with Eccentric and Concentric Bracings

Eccentric bracings EBF in X direction





Seismic link type

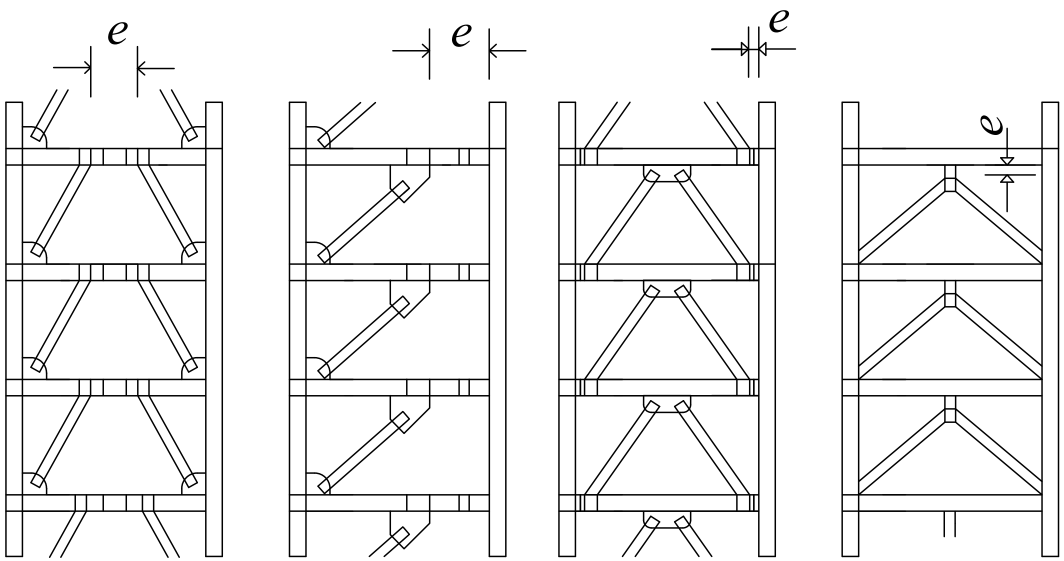
- vertical
- short
- hinged at connection to beam

short links	$e < e_{short} = 0{,}8\ M_{p,link}/V_{p,link}$	yield in shear
long links	$e > e_{long} = 1{,}5\ M_{p,link}/V_{p,link}$	yield in bending
intermediate links	$e_{short} < e < e_{long}$	yield in shear & bending

Composite frame with Eccentric and Concentric Bracings

- **Short links** Stiffer structure
  Plastic deformation are in shear of the web:
  - high ductility, no welds,
  - lateral buckling minor problem

- **Long links** More flexible structure
  Plastic hinges in bending
  → flange buckling & lateral buckling



Examples of frames

with eccentric bracing

e = length of seismic link

Composite frame with Eccentric and Concentric Bracings

- $V_{p,link}$ include V-N interaction

  If N_{ed} / $N_{pl,Rd}$ < 0,15 => $V_{p,link,r} = V_{p,link}\left[1-(N_{Ed}/N_{pl,Rd})^2\right]^{0,5}$

- Homogeneity of links overstrength

  $\Omega_i = 1{,}5\ V_{p,link,i}\ /\ V_{Ed,i}$

  Section overstrength Ω refers to shear because the link is dissipative in shear

  1,5: for high deformations => high strain hardening

  $\Omega_{max} \leq 1{,}25\ \Omega_{min}$

<u>Results of analysis + profiles selected for the links</u>

Level	Link section	N_{Ed} kN	N_{Ed}/N_{pl}	M_{Ed} kNm	M_{pl} kNm	M_{Ed}/M_{pl}	V_{Ed} kN	V_{pl} kN	Ω= 1,5 V_{pl}/V_{Ed}
1	HE450B	75	0,010	285	1141	0,25	950	1182	1,867
2	HE450B	75	0,010	296	1141	0,25	987	1182	1,797
3	HE400B	72	0,011	247	933	0,26	824	1011	1,840
4	HE340B	72	0,011	195	708	0,27	651	761	1,752
5	HE280B	70	0,015	123	455	0,27	405	547	2,028

Ω_{max}=2,03≤ 1,25Ω_{min}=1,25x1,752=2,19 => OK

$N_{ed}/\ N_{pl,Rd} < 0{,}15$

Composite frame with Eccentric and Concentric Bracings

- **Beams, columns, diagonals and connections**

Capacity designed relative to the real strengths of the seismic links

$$N_{Rd}\,(M_{Ed}\,,V_{Ed}\,) \geq N_{Ed,G} + 1{,}1\,\gamma_{ov}\,\Omega\,N_{Ed,E}$$

$$E_d \geq E_{d,G} + 1{,}1\,\gamma_{ov}\,\Omega_i\,E_{d,E}$$

Including torsion effect in $N_{Ed,E}$ by factor $\delta = 1 + 0{,}6\ x/L = 1{,}3$

$$N_{Rd}\,(M_{Ed}\,,V_{Ed}\,) \geq N_{Ed,G} + 1{,}1\,\gamma_{ov}\,\Omega\,\delta\,N_{Ed,E}$$

- **Diagonals**

Max axial loads $N_{Ed,G}$ = 47.4 kN $N_{Ed,E}$ = 495.2 kN

N_{Rd} ≥ 47.4 + 1,1 x 1,25 x 1,75 x 495,2 = 1612 kN

Resistance of diagonal to buckling (weak axis): 1963 kN =>OK

Composite frame with Eccentric and Concentric Bracings

• Action effects and plastic resistance of link

Action effects From analysis	Plastic resistance With f_y=355 MPa	Section overstrength Ω* **	
V_{Ed}=950 kN	$V_{pl,Rd}$ = 1182 kN	1182/952 =1,24	
M_{Ed}=285 kNm	$M_{pl,Rd}$ = 1141 kNm		$M_{Ed}/M_{pl,Rd}$ = 0,25
N_{Ed}=75 kN	$N_{pl,Rd}$ = 7739 kN		$N_{Ed}/N_{pl,Rd}$ = 0,01

* Section overstrength Ω refers to shear => link dissipative in shear

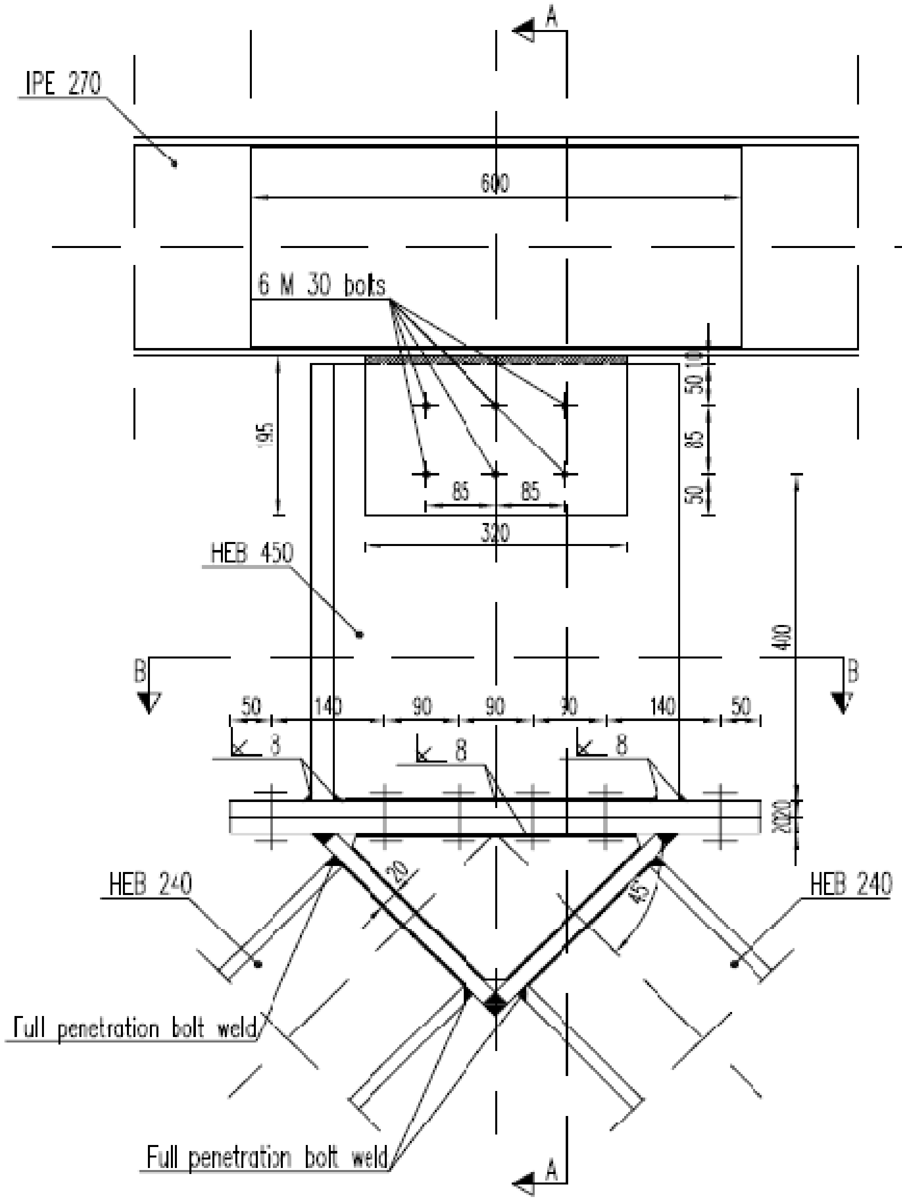
** Connection design made with Ω = 1,24

Note: to revise!

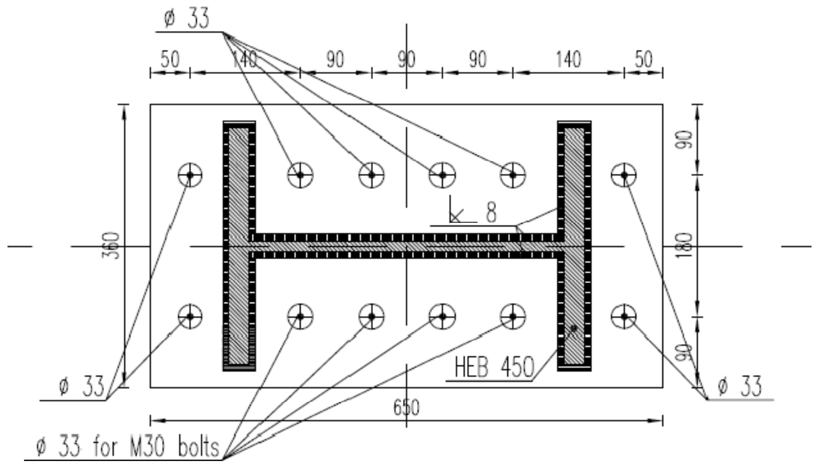
Should be Ω = 1,5 x1,24 = 1,86

Composite frame with Eccentric and Concentric Bracings

Link in elevation

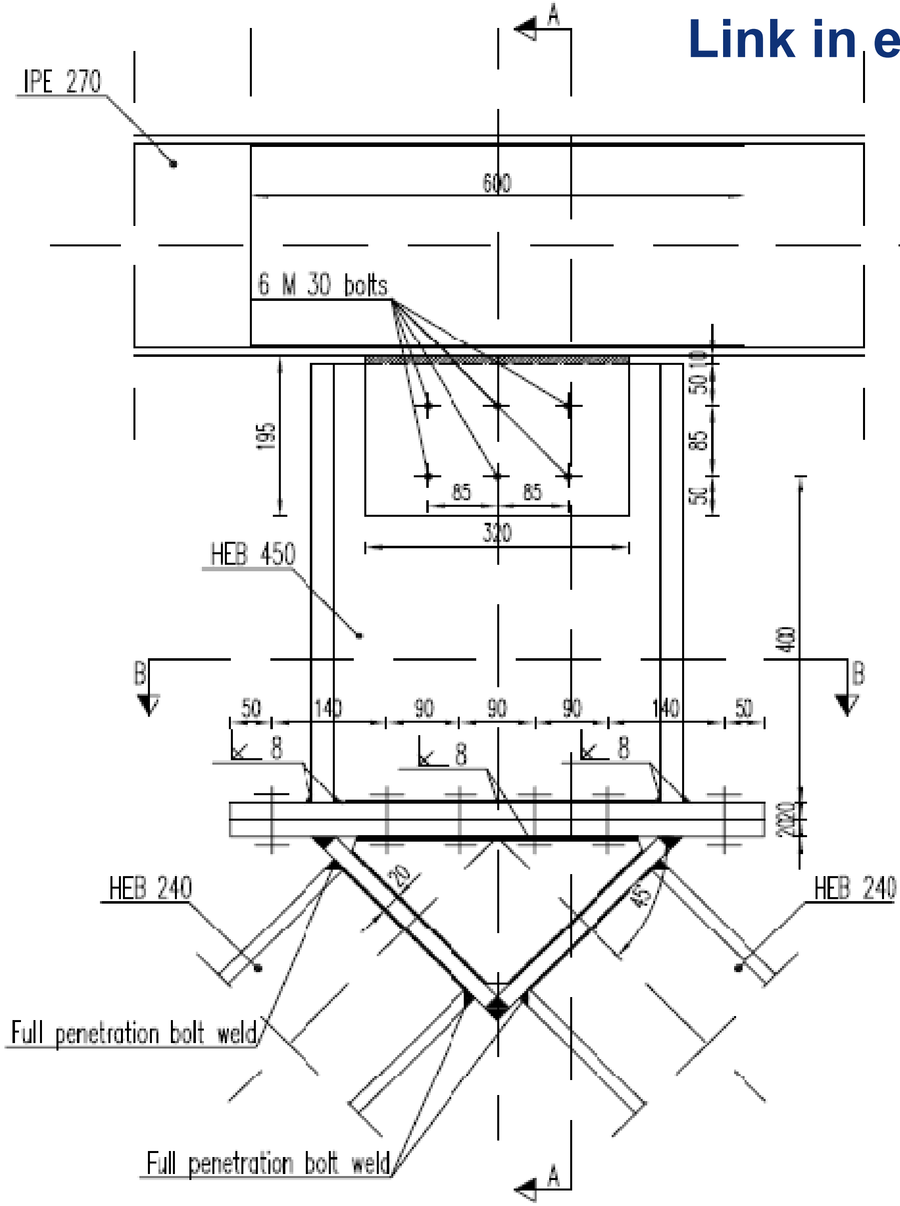


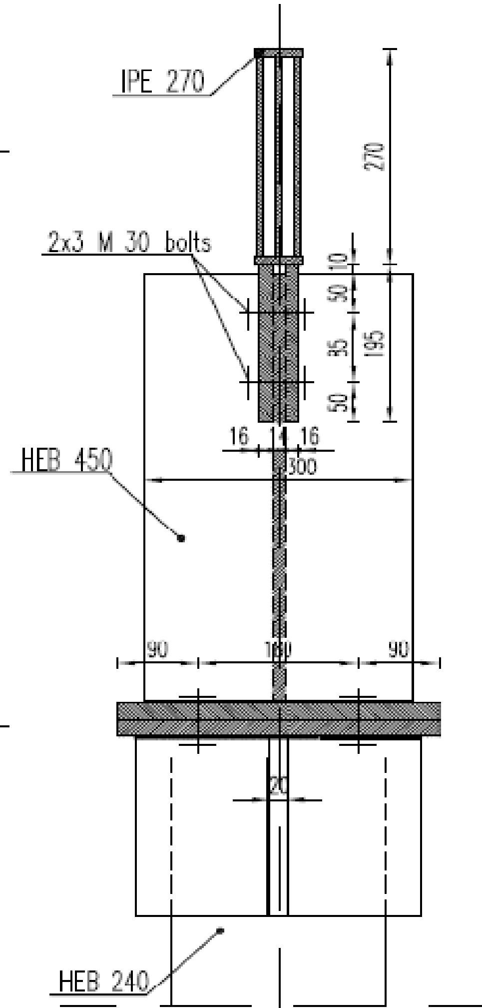
Section BB
Plan view of link base plate



Composite frame with Eccentric and Concentric Bracings

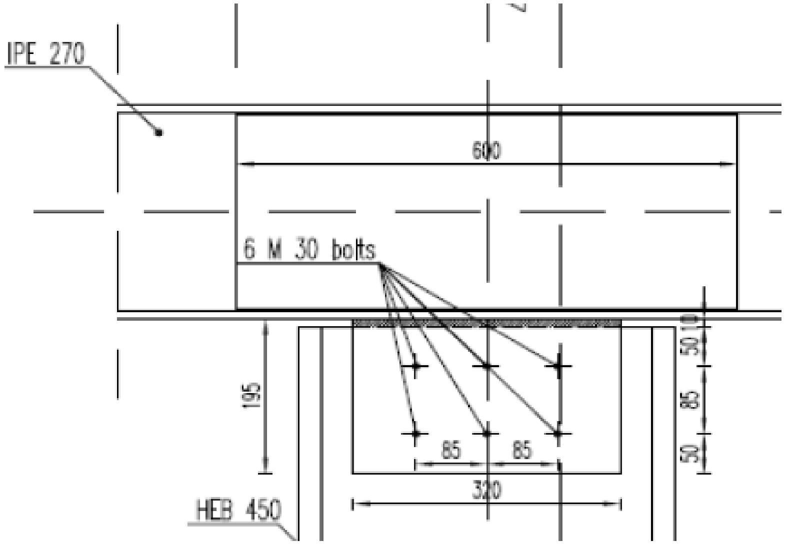
Link in elevation





Section AA
Elevation view of connection

Composite frame with Eccentric and Concentric Bracings



Connection IPE270 beam – HEB450 link

$V_{Ed,\ connection} = 1{,}1\ \gamma_{ov} V_{pl,Rd}$
$= 1{,}1 \times 1{,}25 \times 1182 = 1625$ kN

- **Bolts**
  6 M30 bolts, 2 shear planes:
  $V_{Rd} = 2 \times 6 \times 280/1{,}25 = 2688$ kN > 1625
- **HEB450 web Thickness $t_w = 14$ mm**
  Bearing resistance with $e_1 = 60$ mm, $e_2 = 50$ mm, $p_1 = p_2 = 85$ mm
  $V_{Rd} = 2028$ kN > 1625 kN
- **Bearing resistance < bolt shear resistance**
  2688 kN > 1,2 x 2028 kN = 2433 kN
- **Gussets welded on IPE270 lower flange**
  2 plates t=16 mm $\tau = 1625.\ 10^3 / (2 \times 16 \times 320) = 180 < 355/\sqrt{3} = 204$ MPa

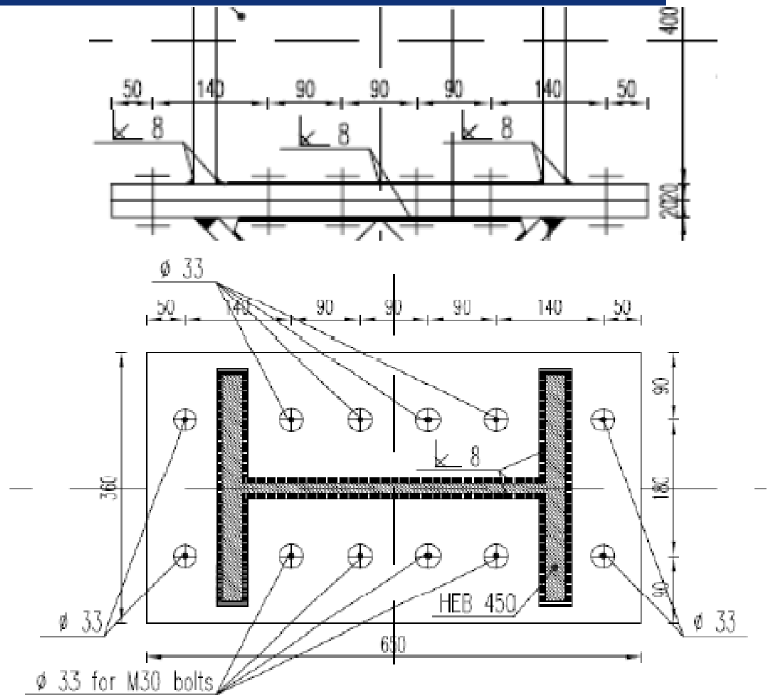
Total thickness provided = 32 mm > $t_{w,HEB450}$ =14 mm => all checks

- **IPE270 web stiffeners**

$t_w = 6{,}6$ mm is not enough => 2 plates t=6mm welded on IPE270 flanges
Provide total thickness 6,6 +6+6=18,6mm > $t_{w,\ HEB450}$=14 mm => all checks

Composite frame with Eccentric and Concentric Bracings

Connection HEB240 diagonals – HEB450 link
Bolted connection of HEB450 link end plate to welded built up triangle



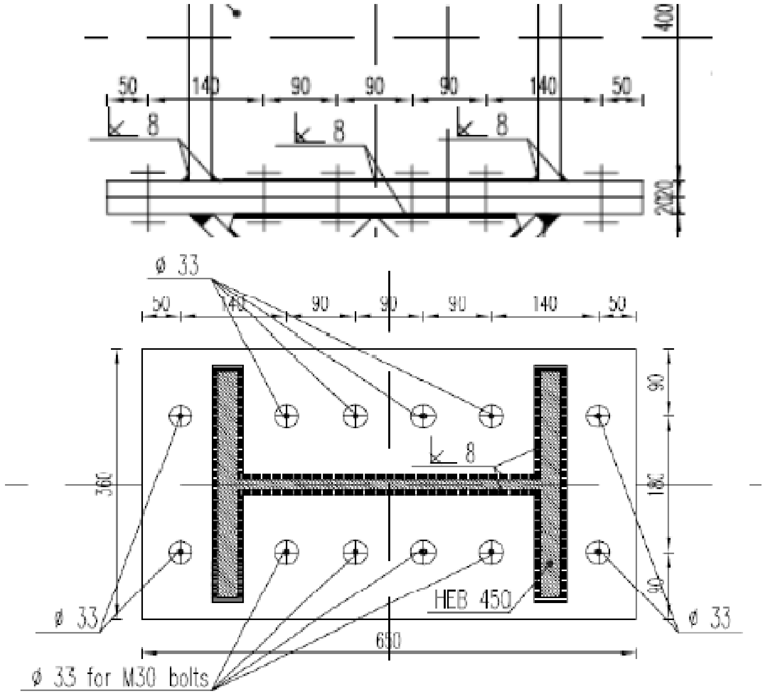
$V_{Ed,\ connection} = 1{,}1\ \gamma_{ov} V_{pl,Rd}$
$= 1{,}1 \times 1{,}25 \times 1182 = 1625$ kN
$M_{Ed,\ connection} = 1{,}1\ \gamma_{ov}\ \Omega\ M_{Ed}$
$= 1{,}1 \times 1{,}25 \times 1{,}24 \times 285 = 485$ kNm

$M_{Ed,\ connection}$ taken by bolts
with lever arm ≈ 450 + 100 = 550 mm
$F_{bolts,total}$ =485/0,55 = 881 kN
=> 2 M30 in tension, each side:
2 x 504,9 /1,25 = 808 kNm
OK for 881 kNm taking into account excess of resistance of web bolts

$V_{Ed,\ connection}$ taken by M30 bolts, single shear plane
8 M30 bolts provide shear resistance 8x280,5/1,25 =1795 kN > 1625 kN
Bearing resistance: 8 x 289,8 x 1,4 = 3245 kN > 1625 kN

Composite frame with Eccentric and Concentric Bracings

Welded connection between HEB450 and end plate



As above:

$V_{Ed,\ connection}$= 1625 kN

$M_{Ed,\ connection}$= 485 kN

$V_{Ed,\ connection}$ taken by the web.

Weld length = 2 x 400 = 800 mm

a=8mm fillet weld provides a resistance:

(8 x 261,7)/1,25=1674 kN > 1625 kN

$M_{Ed,\ connection}$= 485 kN taken by the flanges.

Weld length = 2 x 300 = 600 mm/flange

Tension force in flange = 485/ (2 x 0,2m)=1214 kN => 202 kN/100 mm

An a=8 mm fillet weld provides a resistance:

6 x261,7 /1,25= 1256 kN > 1214 kN

Composite frame with Eccentric and Concentric Bracings

Connection of HEB240 diagonals to welded built up triangle

$N_{Ed,\ 1\ diagonal} = N_{Ed,\ gravity} + 1{,}1\ \gamma_{ov} N_{Ed,E}$ 1612 kN

$N_{pl,Rd}$ 10600x355= 3763 kN

$N_{Ed} / N_{pl,Rd} = 0{,}43$

$M_{Ed,\ 1\ diagonal}$ = 0,5 x link moment due to equilibrium of node

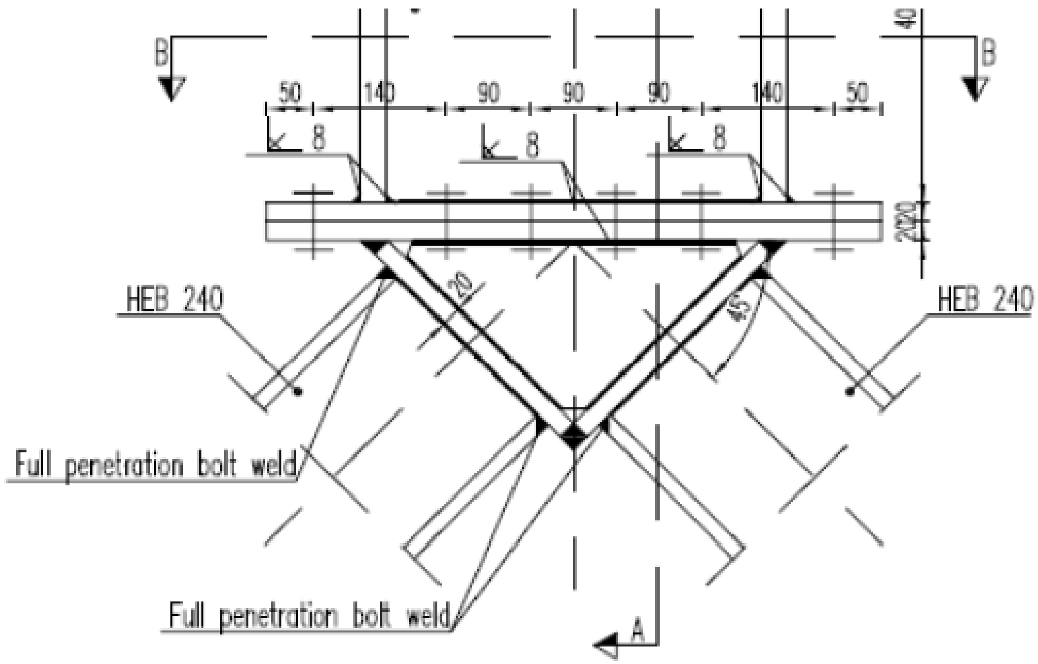
=> $M_{Ed,\ 1\ diagonal}$ = 285/2 = 143 kNm

$M_{pl,Rd} = 1053.\ 10^3 \times 355 = 373$ kN

$M_{Ed} / M_{pl,Rd} = 0{,}38$

Stresses in tension & bending relatively high

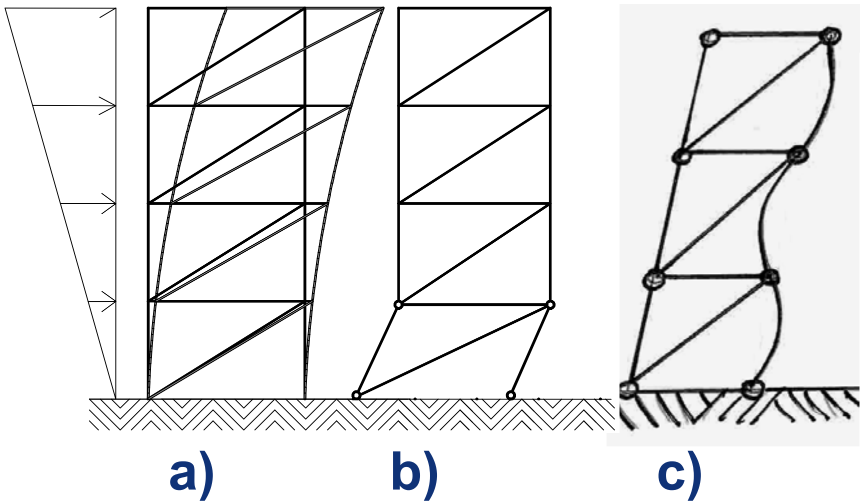
$\Rightarrow$ connection with full penetration butt welds



Composite frame with Eccentric and Concentric Bracings

Concentric Bracing CBF

- **Global plastic mechanism with diagonals or their connection as dissipative zones.**
  No buckling or yielding of beams and columns.



a) Global plastic mechanism the design objective for frames with X bracings.
b) Storey mechanism prevented by the resistance homogenisation condition for the diagonals.
c)Buckling of columns Prevented by capacity design

Diagonals should have

- **similar force-displacement characteristics in both directions**
- **homogeneity of diagonal sections overstrength $\Omega_i = N_{pl,Rdi}/N_{Edi}$**
- **Symetry of bracings at each level:**
  A+ et A- , area of projections of sections comply with $\frac{|A^+ - A^-|}{A^+ - A^-} \leq 0,05$

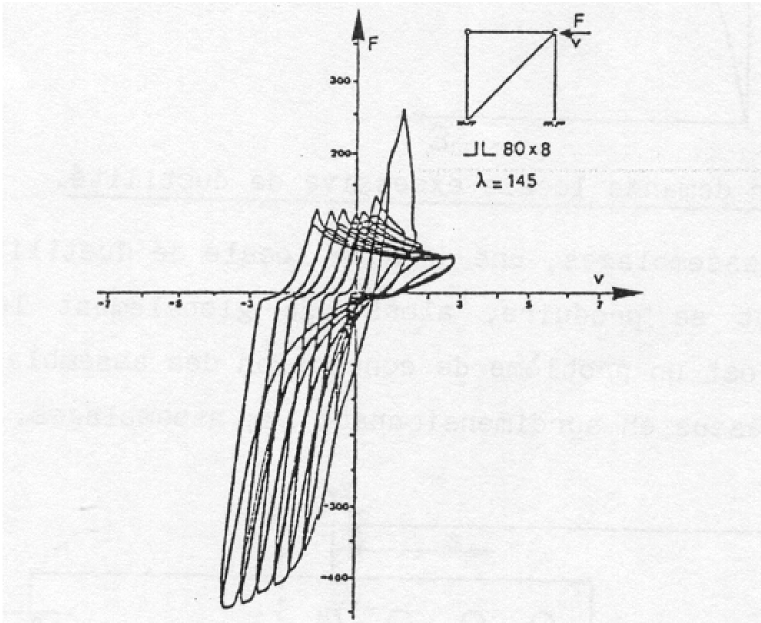
Composite frame with Eccentric and Concentric Bracings

■ **Elastic range:**

compression and tension diagonals contribute equally to stiffness and resistance

■ **1st buckling:**

degradation in behaviour of compression diagonal



Behaviour evolution with cycles

EC8: 2 different design approach

- **X bracings:** tension diagonals only
- **V or Λ bracings:** compression and tension diagonals

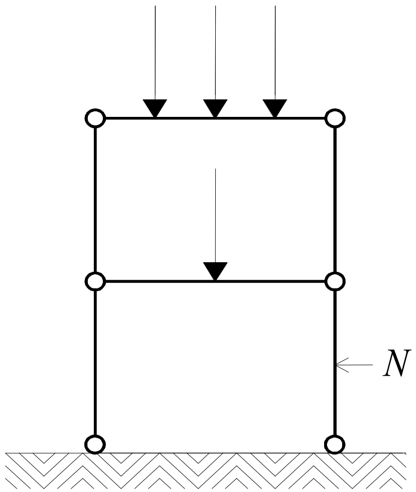
New solutions to avoid problems with analysis

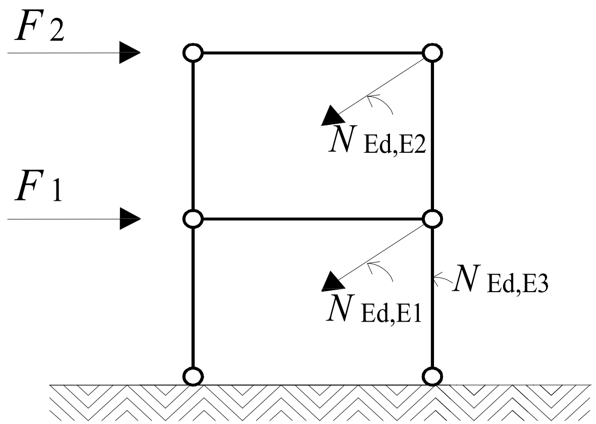
- **dissipative connections with *R*fy < Rbuckling,diagonals**
- **special design of diagonals (Buckling Restrained Bracings -BRB)**

Composite frame with Eccentric and Concentric Bracings

- **Standard analysis: only tension diagonals participate in resistance**

Gravity loading **Beams and columns in the model** **No diagonal**
Seismic action **Beams and columns + tension diagonals in the model**





Design of diagonals

- $N_{pl,Rd} \geq N_{Ed,E}$
- $1{,}3 < \bar{\lambda} \leq 2{,}0$ (not for structures up to 2 levels)
- $\Omega_i = N_{Rd}/N_{ed}$ $\Omega_{max} \leq 1{,}25\ \Omega_{min}$

Composite frame with Eccentric and Concentric Bracings

- $1{,}3 < \bar{\lambda} \leq 2{,}0$ **Why?**

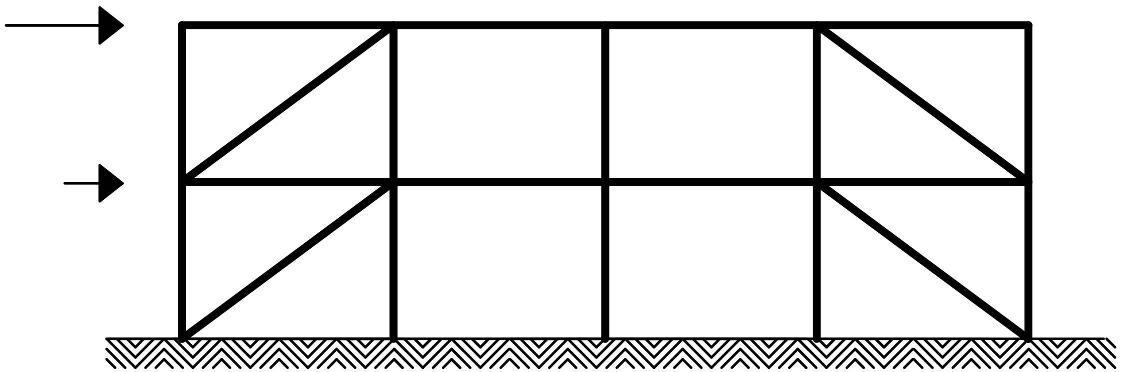
  Design does not include the compression diagonals. Reality does.

Max initial resistance of X brace V_{ini} up to 1st buckling of diagonals should be: $V_{ini} \leq V_{pl,Rd}$ $V_{pl,Rd}$ from analysis with tension diagonal only

If $N_{Rd,buckling} > 0{,}5\ N_{pl,Rd}$ => $V_{ini} \geq V_{pl,Rd}$
=> possible failure of beams and columns capacity designed to $V_{pl,Rd}$
Condition $\bar{\lambda} \geq 1{,}3$ correspond to $\chi = 0{,}47$ at most
avoid too high action effects in beams/columns during 1st buckling of diagonals
Condition $\bar{\lambda} \leq 2{,}0$ to avoid shocks at retensionning

- **If diagonals decoupled**

→ 1 condition only $\bar{\lambda} \leq 2{,}0$
→ $V_{ini} > V_{pl,Rd}$ cannot be
$\bar{\lambda} \geq 1{,}3$ not necessary



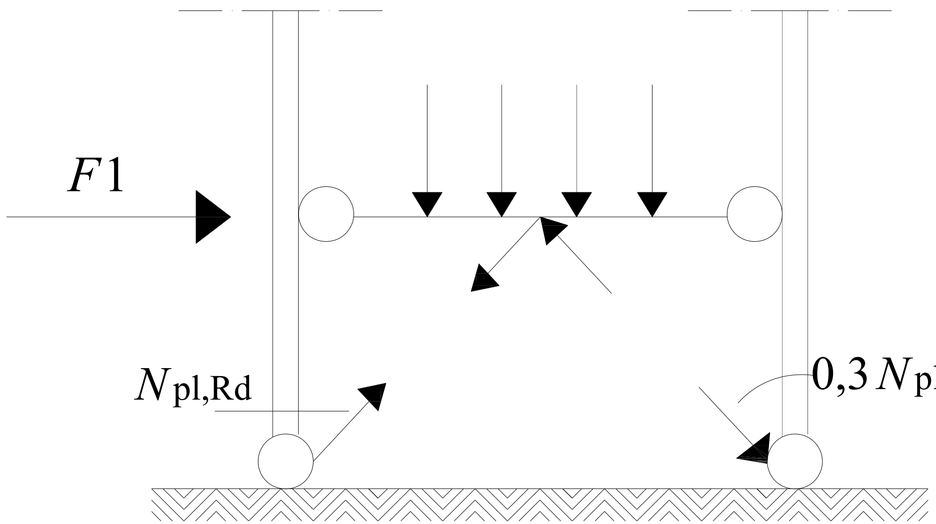
Composite frame with Eccentric and Concentric Bracings

- **Considering compression diagonals in the analysis of X braces?**

Allowed, but require model for diagonals + non linear analysis
static ("pushover") or dynamic
Considering pre and post buckling resistances of diagonals
under cyclic elasto-plastic action effects

1 diagonal in plastic tension
1 diagonal in compression with post buckling strength

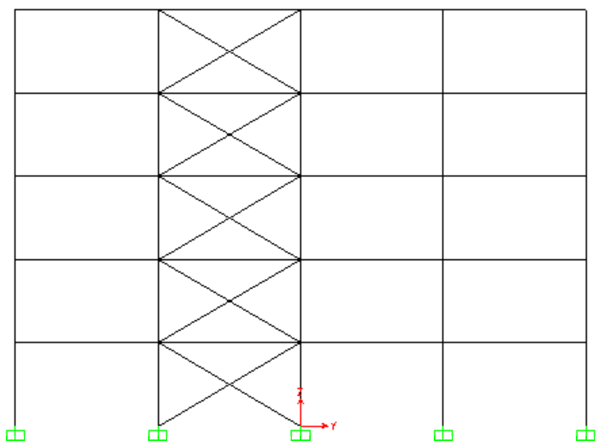
Is done with V bracings



Composite frame with Eccentric and Concentric Bracings

Concentric bracings Y direction Results of analysis

Storey	Steel profile	A mm^2	$N_{Ed,CBi}$ kN	$N_{Rd,CB1}$ kN	Ω_i N_{Rd}/N_{Ed}	$\overline{\lambda}$	θ
1st (ground level)	UPE 160	2170	492	770	1,56	1,80	0,17
2nd	UPE 160	2170	531	770	1,45	1,80	0,17
3rd	UPE 180	2510	657	891	1,35	1,70	0,15
4th	UPE 160	2170	531	770	1,45	1,80	0,14
5th	UPE 120	1540	373	546	1,46	2,15	0,11



- $1{,}3 < \overline{\lambda} \leq 2{,}0$
  except at storey 5
  allowance for 2 upper storeys
- $\Omega_{max} = 1{,}56 \leq 1{,}25\ \Omega_{min} = 1{,}25 \times 1{,}35 = 1{,}69$
- $\theta > 0{,}1 \Rightarrow$ amplification of N_{Ed} by $1/(1-\theta)$

Composite frame with Eccentric and Concentric Bracings

Beams and columns: $N_{Rd}(M_{Ed}, V_{Ed}) \geq N_{Ed.G} + 1.1\gamma_{0v}\Omega N_{Ed.E}$

Capacity design

$\Omega_{Y,min}$ = 1,35 = min section overstrength factor of concentric bracings

γ_{ov}=1,25

Check for columns

N_{Rd} buckling resistance strong&weak axis ≥ $N_{ed,G} + 1{,}1\gamma_{ov}\ \Omega_{Y,min}\ N_{ed,E}$

Check for beams

N_{Rd} resistance under combined M,N,V ≥ $N_{ed,G} + 1{,}1\gamma_{ov}\ \Omega_{Y,min}\ N_{ed,E}$

Composite frame with Eccentric and Concentric Bracings

Connection of a CBF diagonal

At level 1 $N_{Ed,BC1}$= 492 kN

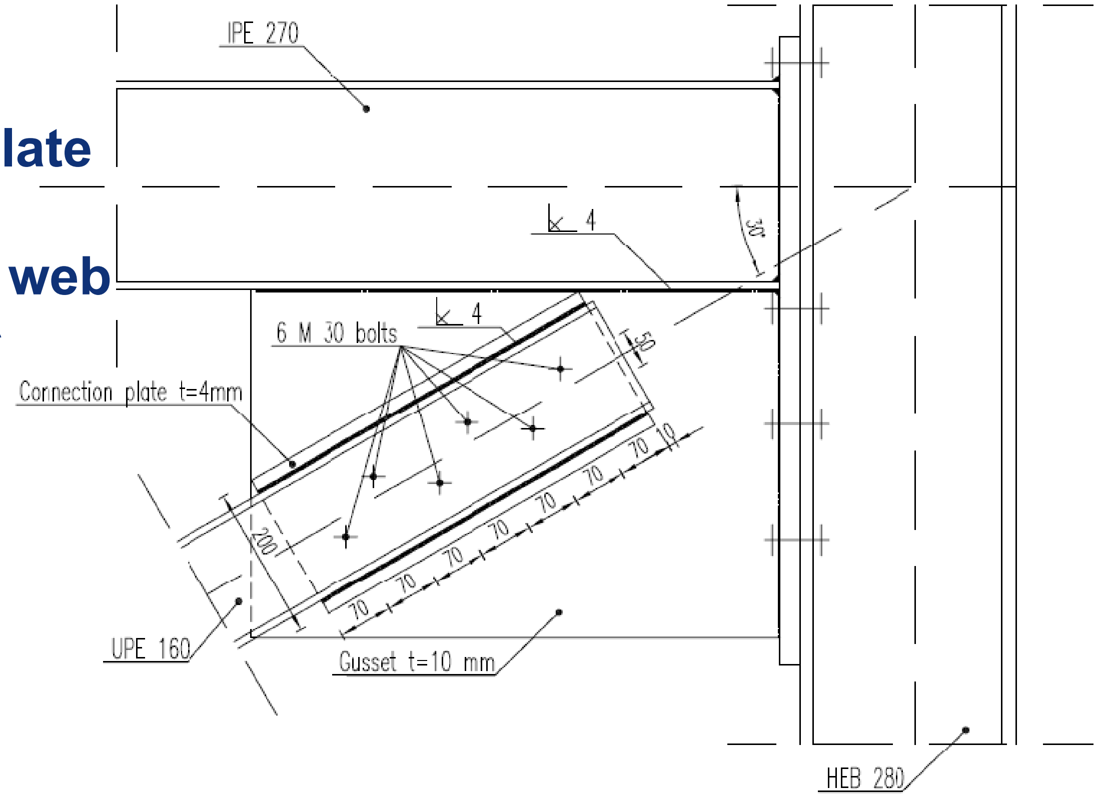
Element design=> UPE160: $N_{pl,Rd}$=A x $f_{y,d}$= 2170 x 355 = 770kN

Connection capacity designed to $N_{pl,Rd}$ UPE160:

$N_{Rd,connect} \geq 1{,}1\ \gamma_{ov}\ N_{pl,Rd}$ = 1,1 x 1,25 x 770 = 1058 kN

Components of the connection

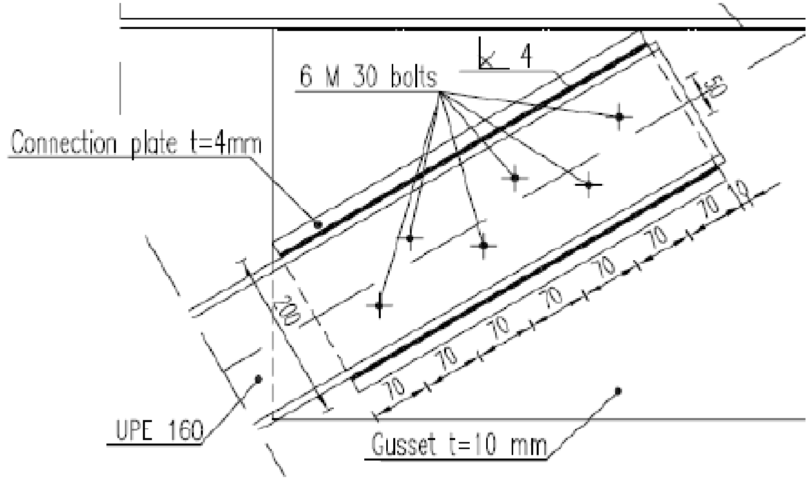
- **gusset welded to beam+end plate**
- **end plate bolted to column**
- **connection plate welded on U web**
  substituting area of U flanges for connection purpose
- **bolts M30 grade 10.9**
- **holes in web+plate & gusset**



Composite frame with Eccentric and Concentric Bracings

6 bolts, resistance in shear, one shear plane, for M30 bolts:

$F_{V,Rd}$= 6 x 280,5 / 1,25 = 1344 kN > 1058 kN



UPE web t= 5,5 mm

additional plate t= 4 mm => total = 9,5 mm

Bearing resistance: $F_{b,Rd} = k_1 \alpha_b f_u dt/\gamma_{M2}$

Here: $\alpha_b \leq 1$ or $\alpha_b = \alpha_d$ as f_{ub} (1000) >f_u (510 for S355)

Values of parameters: e_1 =70 mm e_2=65 mm p_2 = 50mm

α_d=70/(3 x 33)=0,71 end bolt

α_d=70/(3 x 33)-0,25=0,71-0,25=0,45 inner bolt

k_1=(2,8 x 65)/33 – 1,7=3,8 => 2,5 edge bolt k_1: no inner bolts

Bearing resistance:

4x2,5x0,71x30x 51x 9,5/1,25 + 2x2,5 0,45x510x30x9,5

= 1087 kN > 1058 kN

1344 kN >1,2 x 1087 =1304kN bearing resist < bolt shear resistance

Composite frame with Eccentric and Concentric Bracings

Welds of plate placed flat on UPE web:
weld throat cannot be more than t_{plate} x √2/2=4 x 0,707=3mm
Resistance of a 3 mm weld: (98,1kN:1,25)/100mm=78,5kN/100mm
Force to transmit: proportional to plate thickness:
(4 x1058) /(4+5,5)=445 kN

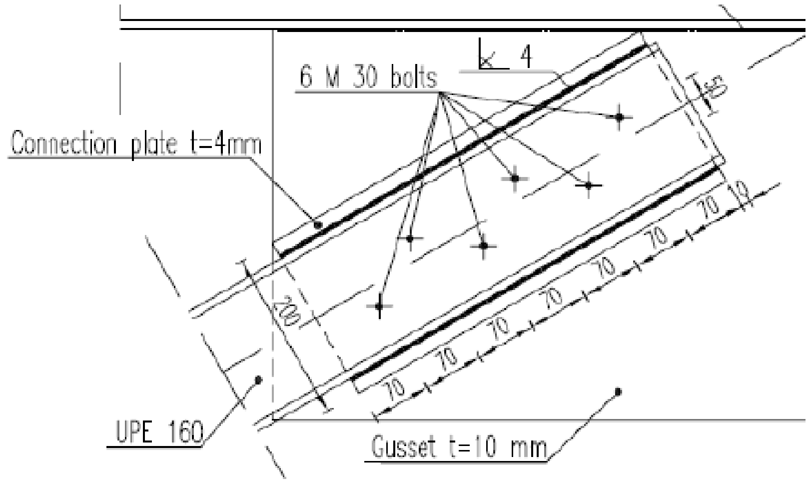


Plate perimeter as from bolted connection: 2 x (7x70+160) = 1300mm
=> resistance = 13 x 78,5 = 1020 kN > 445 kN
Gusset: 10 mm thick plate (as UPE web + 4 mm plate = 9,5 mm)
Welds: length= 2 x (7 x 70 + 160 x 0,707) = 1206 mm x 2 (2 sides)
= 2412 mm = 24 x 100 mm
With a = 4mm fillet welds:(24 x 130,9)/1,25= 2513 kN > 1058 kN

Some words on other ways to make Concentric Bracings

Composite frame with Eccentric and Concentric Bracings

Dissipative connections in frames with concentric bracing

Interest

- designed to have connection resistance < diagonal buckling strength

=> Analytical difficulties avoided

all members in the model for simple analysis.

all the results of the analysis may be used directly

no distinct rules for X, V or decoupled braces

- additional stiffness in comparison to 'tension diagonal only' model compensates for the additional flexibility of semi-rigid connections
- Can be 'standardised' components with calibrated strength, obviating problems of diagonal overstrength in the design of beams and columns => γ_{ov} = 1,0
- After an earthquake, easy replacement of deformed components of connections
- Higher *q* = *6*

Composite frame with Eccentric and Concentric Bracings

Dissipative connections in frames with concentric bracings

- **Design condition:**

  Deformation capacity of connections allows global deformation of the structure

  Dissipative diagonals: low ε in all length *l* provide high $dl = \varepsilon \times l$

  Dissipative connections: *dl* to be realised in the connection

- $dl = d_r / \cos\alpha$ $\cos\alpha = l / (l^2 + h^2)1/2$

  d_r interstorey drift $d_r = q \times d_{re}$

- **Example**

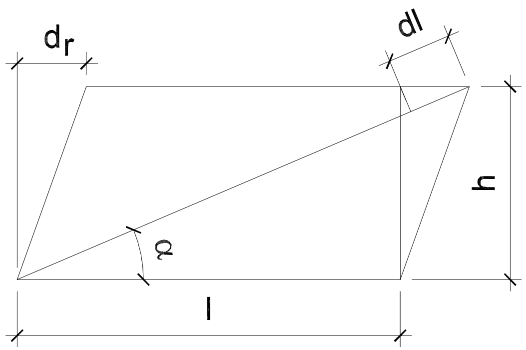
  $d_r / h = 3{,}5\%$; $l = 6$ m ; $h = 3$ m

  $\cos\alpha = 0{,}894$; $d_r = 0{,}105$; $dl = 117$ mm

  Dissipative diagonals: $\varepsilon = 1{,}7\%$

  Dissipative connections:

  required deformation capacity: *117 /2 = 58,5 mm*



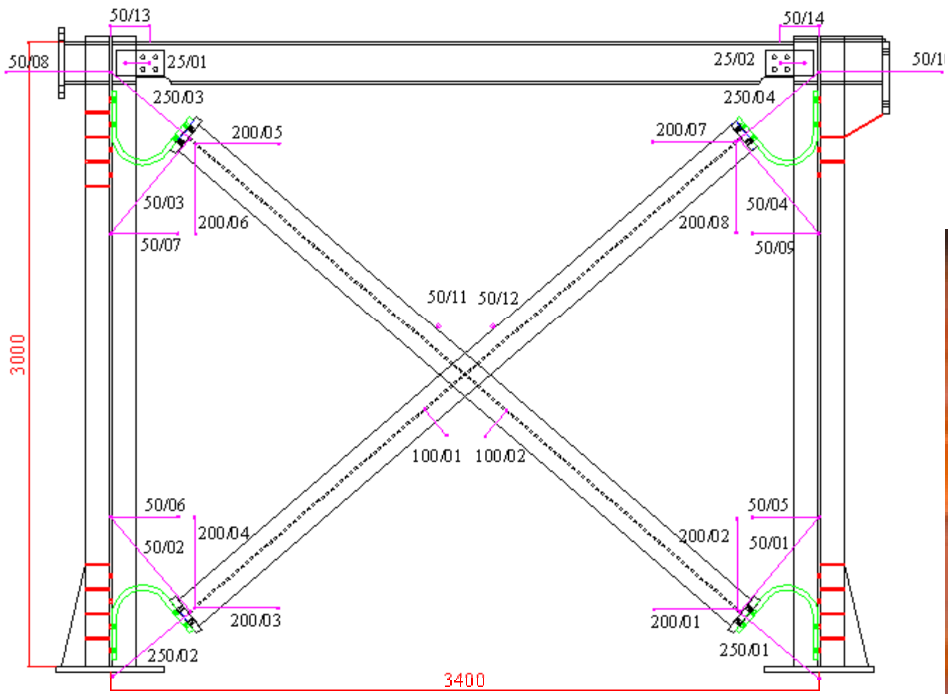
$dl=d_r/\cos\alpha$ $\cos\alpha=l/(l^2+h^2)^{1/2}$

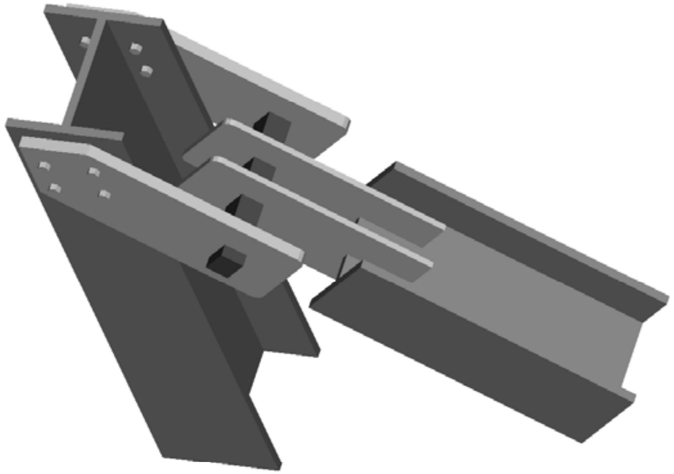
=> Special design 2001 ULg, INERD Project, 2 design:

« pin connection » « U connection »

Composite frame with Eccentric and Concentric Bracings

Frames
with concentric bracings
and dissipative connections
INERD connections





Composite frame with Eccentric and Concentric Bracings

Design Criteria for frames with X, V or Λ concentric bracings and dissipative connections for the diagonals

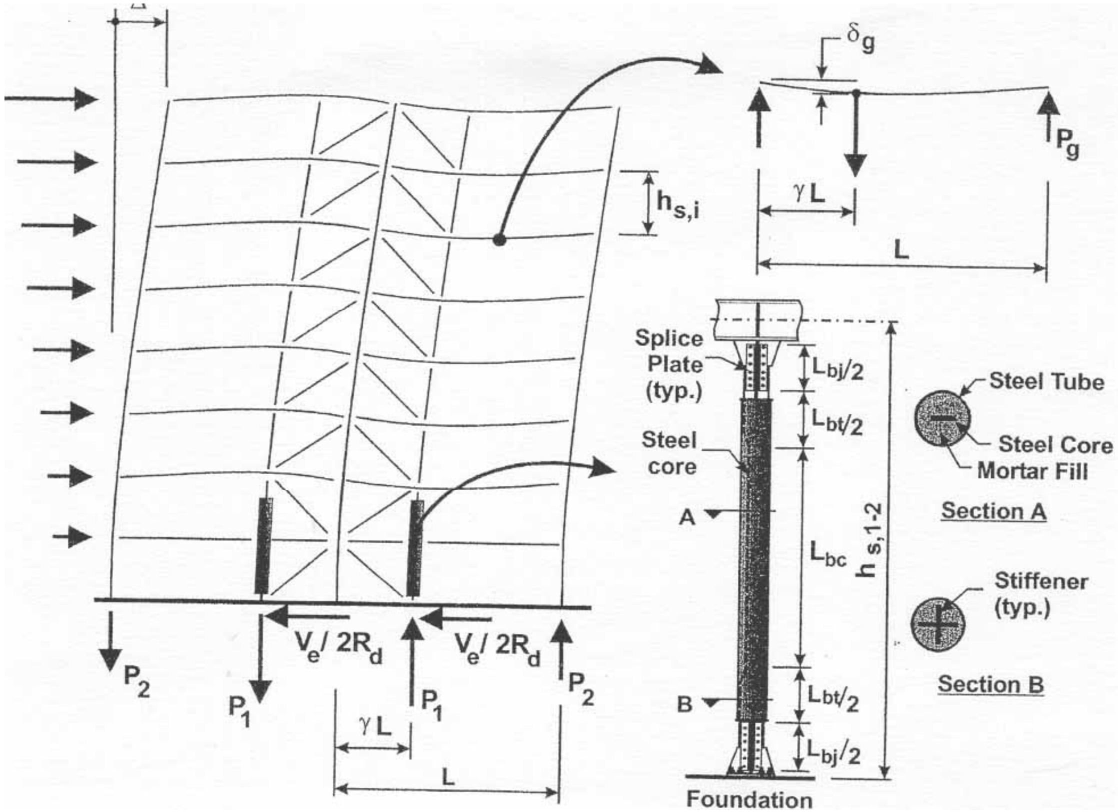
- Resistance $R_{pl,Rd}$ of the dissipative connections: $R_{pl,Rd} \geq N_{Ed}$
- Resistance $N_{b,Rd}$ of the diagonals
  capacity design to the dissipative connections resistance:
  $$N_{b,Rd} > R_{pl,Rd} \geq N_{Ed}$$
- Homogenisation of the dissipative connections overstrengths over the height of the building: $\Omega_i = R_{pl,Rd,i} / N_{Ed,i}$
  $\Omega_{max} \leq 1{,}25\ \Omega_{min}$ $\Omega = \Omega_{min}$
- With a controlled production of standard connections, $R_{pl,Rd}$ is known
  $\gamma_{ov} = 1.0$
- Resistance in tension $N_{pl,Rd}$ or in compression $N_{b,Rd}$ of the non dissipative elements (beams and columns):
  $$N_{pl,Rd} \text{ or } N_{b,Rd}(M_{Ed}) \geq N_{Ed,G} + 1{,}1\gamma_{ov}\ \Omega . N_{Ed,E}$$
- No specific requirements for frames with X, V or Λ bracing.

Composite frame with Eccentric and Concentric Bracings

Buckling restrained braces or BRB

Principle

- active section of diagonal placed in a tube which prevents buckling
- mortal fill to link tube and active section
- tube not submitted to action effects else than buckling prevention



Steel & Composite Frames

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