Eurocode 8 – Buildings.

Steel and Composite.

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Foreword…

Origin of Eurocode 8 rules on steel and composite steel – concrete structures

1986. ECCS Design Recommandations

ECCS: European Convention for Constructional Steelwork

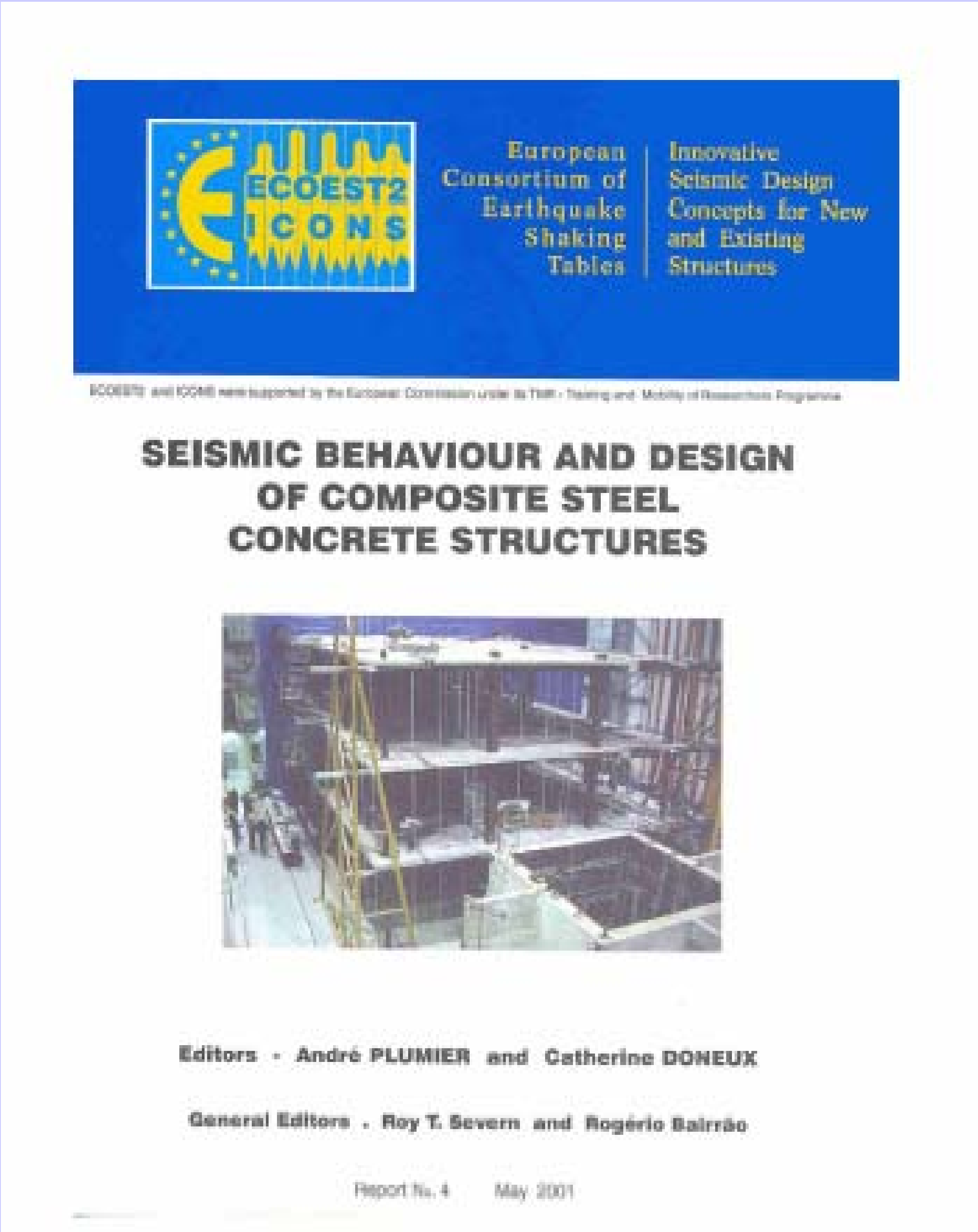
Authors: Aribert, Ballio, Mazzolani, Plumier, Sedlacek

1994. 1st Eurocode 8 = ENV

For steel structures	**≈ ECCS Recommendations**
For composite:	**really weak.**

1994 – 2000 Research: ICONS Project

ECOEST, ECOLEADER


ECOEST2 ICONS

European Consortium of Earthquake Shaking Tables

Innovative Seismic Design Concepts for New and Existing Structures

ECOEST2 and ICONS were supported by the European Commission under its TMR - Training and Mobility of Researchers Programme

SEISMIC BEHAVIOUR AND DESIGN OF COMPOSITE STEEL CONCRETE STRUCTURES

Editors - André PLUMIER and Catherine DONEUX

General Editors . Roy T. Severn and Rogério Bairrão

Report No. 4 May 2001

ICONS Reports

<= Topic 4

= Background document to Eurocode 8 on composite steel concrete structures.

The world most developed code for those structures

Eurocode 8. Section 6.
Steel Buildings

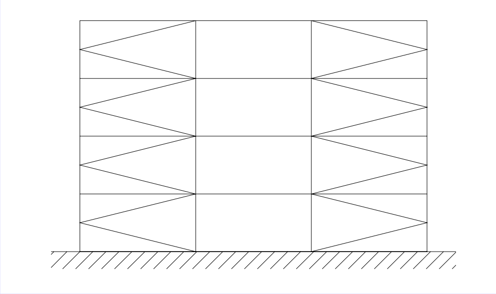
6.1 General

Design Concepts	q	Ductility classes	
Non Dissipative Structures	$1 \leq q \leq 1{,}5$	DCL	L for Low
Dissipative Structures	$1{,}5 < q < 4$	DCM	M for Medium
Dissipative structures	$q \geq 4$	DCH	H for High

Ductility classes:
plastic deformation capacity without degradation of resistance

Design of non dissipative structures. (Eurocode 3)

- requirements on steel material + bolts 8.8 -10.9
- preferably in low seismicity regions
- K bracings may not be used



6.2 Material

f_y and toughness of steel components and the welds at service temperature
=> dissipative zones at expected places

Conditions on fy

a) dissipative zones $f_{y,max} < 1{,}1\ \gamma_{ov} f_y$

γ_{ov} material overstrength factor $\qquad$ *fy* : nominal

Ex: S235, γ_{ov} = [1,25] $\qquad$ => *fy*,max = 323 N/mm2

b) - design based on a single nominal yield strength f_y for both dissipative and non dissipative zones
- a higher value $f_{y,max}$ specified for dissipative zones;
- nominal f_y for non dissipative zones and connections

Ex: S355 non dissipative zones
S235 dissipative zones, with $f_{y,max}$ = 355 N/mm2

c) $f_{y,max}$ of dissipative zones is measured=> $\gamma_{0v} = 1$

Bolts 8.8 ou 10.9 preloaded EN 1090

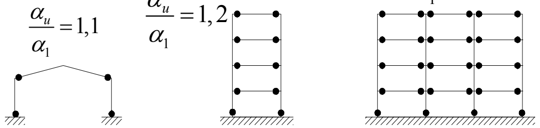
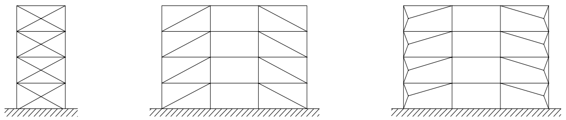
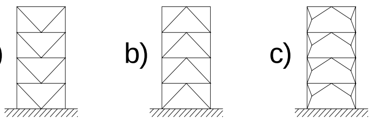
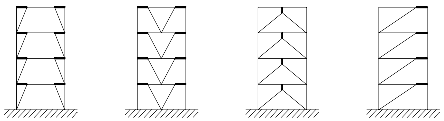
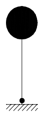
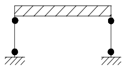
6.12 Control of design and construction

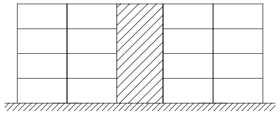
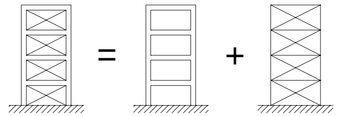
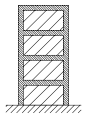
- ▲ Drawings indicate details, steel grades…
  noting the maximum permissible yield stress f_{ymax} of the steel
  to be used in the dissipative zones

- ▲ Tightening of bolts to EN 1090

- ▲ No structural changes involving a variation in stiffness or strength of more than 10 % of the values assumed in design

- ▲ If not, appropriate corrections or justifications

	DCH	DCM
a) Moment resisting frame. $\frac{\alpha_u}{\alpha_1}=1,1$ $\frac{\alpha_u}{\alpha_1}=1,2$ $\frac{\alpha_u}{\alpha_1}=1,3$  • Dissipative zones in the beams and bottom of columns	$5\frac{\alpha_u}{\alpha_1}$	4
b) Frame with concentric bracings.		
Diagonal bracings.  Dissipative zones -tension diagonals only-.	4	4
V - bracings.  Dissipative zones (tension & compression diagonals).	2,5	2
c) Frame with eccentric bracings. $\frac{\alpha_u}{\alpha_1}=1,2$  – Dissipative zones (bending or shear links).	$5\frac{\alpha_u}{\alpha_1}$	4
d) Inverted pendulum. $\frac{\alpha_u}{\alpha_1}=1$  - Dissipative zones at the column base.  $\frac{\alpha_u}{\alpha_1}=1,1$ - Dissipative zones in columns $N_{Sd}/N_{Pl.Rd}>0,3$	$2\frac{\alpha_u}{\alpha_1}$	2

	DCH	DCM
e) Structures with concrete cores or concrete walls. 	See section 5.	
f) Dual structures. Moment frame with concentric bracing.  $\frac{\alpha_u}{\alpha_1}=1,2$ Dissipative zones: in moment frame and in tension diagonals.	$4\frac{\alpha_u}{\alpha_1}$	4
g) Mixed structures (steel moment resisting frames with infills). 		
Unconnected concrete or masonry infills, in contact with the frame.	2	2
Connected reinforced concrete infills.	See section 7.	
Infills isolated from moment frame: see moment frames.	$5\frac{\alpha_u}{\alpha_1}$	4

Structural types and maximum behaviour factors *q*

6.5.2 General Criteria for Dissipative Structural Behaviour

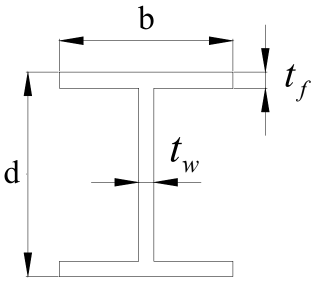
▲ Dissipative zones: adequate ductility and resistance

▲ Yielding, buckling, hysteretic behaviour do not affect stability.

Elements in Compression or Bending

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

=> limits of b/t_f



▲ Semi-rigid - partial strength connections:

OK if:
- adequate rotation capacity (<=>global deformations)
- members framing into connections are stable
- effect of connections deformations on drift analysed

▲ Non-dissipative parts and the elements connecting them to dissipative parts have overstrength (development of cyclic yielding of dissipative parts)

6.5.5 Connections in dissipative zones

(3)For fillet weld or bolted non dissipative connections

$$R_d \geq 1{,}1\ \gamma_{ov}\ R_{fy}$$

R_d **resistance of the connection according to Eurocode 3,**

R_{fy} **plastic resistance of the connected dissipative member**

In ENV, Rfy computed with "appropriate estimation fyd of the actual value of the yield strength". "appropriate" was a problem

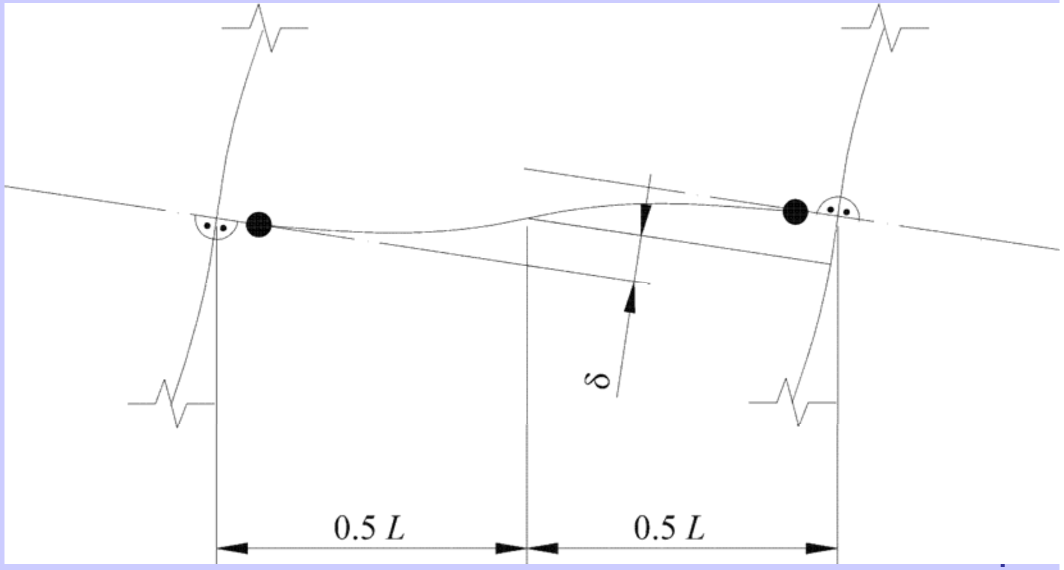
(6) The adequacy of design should be supported by experimental evidence …to conform with requirements defined… for each structural type and ductility class.

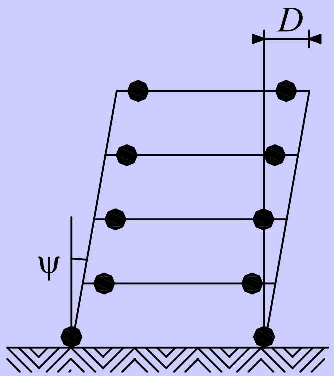
Example: moment resisting frames

plastic rotation capacity $\theta p = \delta / 0{,}5L$

ductility class DCH : $\theta p \geq 35$ mrad

DCM with $q > 2$ $\theta p \geq 25$ mrad





6.6 Moment frames

Design Criteria

Target global mechanism:

plastic hinges in beams, not in columns

(waived at base, at top level, in 1 storey buildings if in columns: $N_{Sd} / N_{Rd} < 0{,}3$

General criterion: $\sum M_{Rc} \geq 1{,}3 \sum M_{Rb}$

Beams $\frac{M_{Ed}}{M_{pl,Rd}} \leq 1{,}0$ $\frac{N_{Ed}}{N_{pl,Rd}} \leq 0{,}15$ $\frac{V_{Ed}}{V_{pl,Rd}} \leq 0{,}5$

V_{Ed} :capacity design to beam plasic moments $M_{pl,RD}$

$$V_{Ed} = V_{Ed,G} + V_{Ed,M}$$

$$V_{Ed,M} = (M_{pl,Rd,A} + M_{pl,Rd,B})/L$$

Columns

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

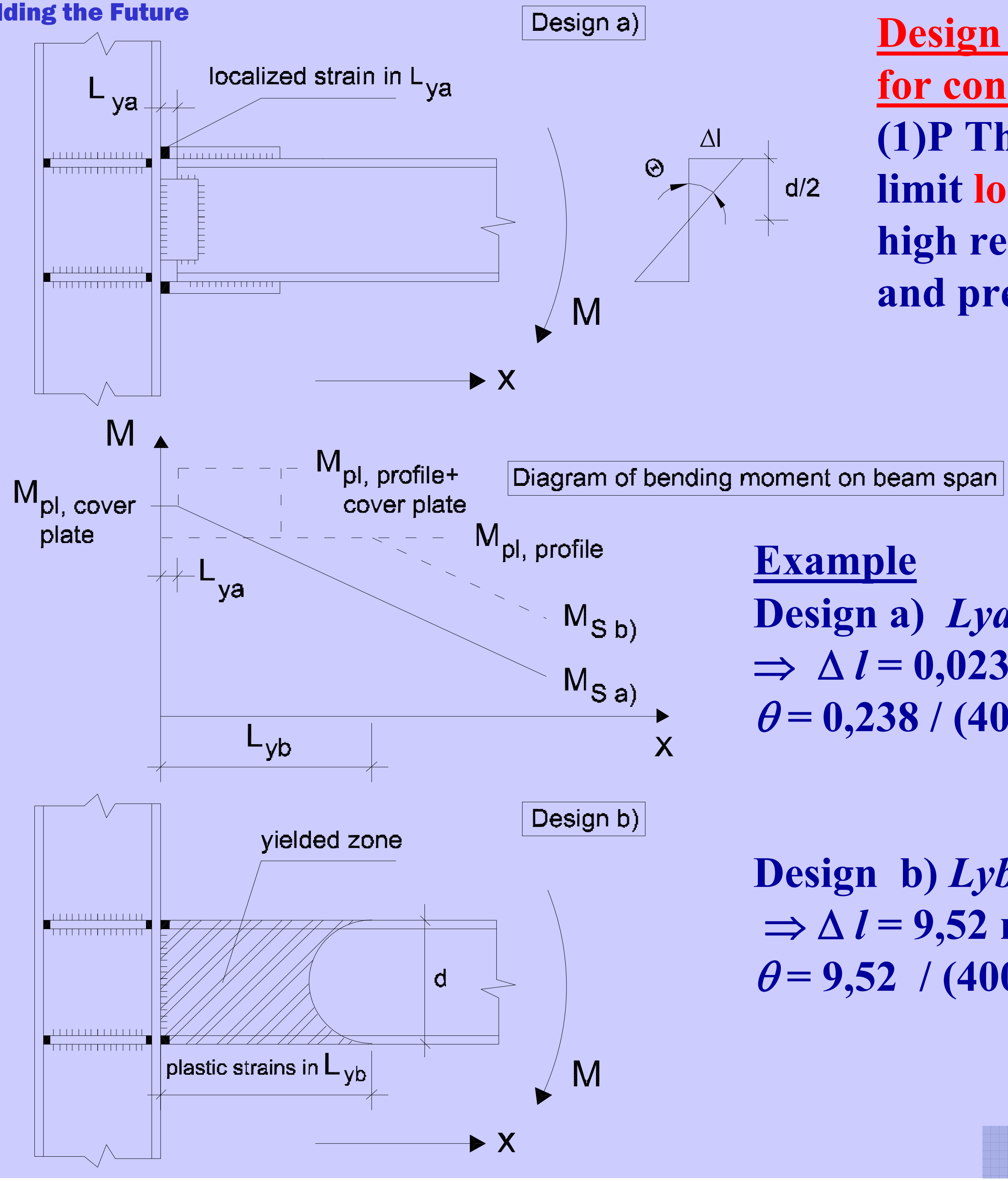
$$M_{Ed} = M_{Ed,G} + 1{,}1\gamma_{ov}\,\Omega M_{Ed,E}$$

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

Ω minimum section overstrength $\Omega_i = M_{pl,Rd,i}/M_{Ed,i}$ of all beams dissipative zones

M*Ed,i design bending moment in beam i (seismic situation)*

M*pl,Rd,i. plastic moment*



Design rules
for connections in dissipative zones

(1)P The design of connections shall… limit localization of plastic strains, high residual stresses and prevent fabrication defects.

Example

Design a) Lya = 10 mm εy, max = 2,38 %

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

θ = 0,238 / (400/2) = 1,2 mrad <<< 25 mrad

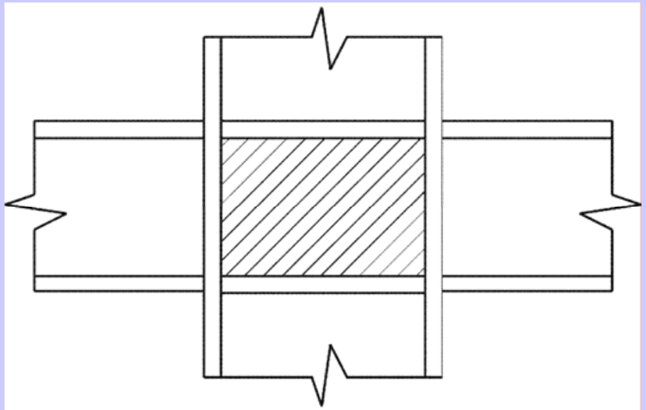
Design b) Lyb = 400 mm εy, max = 2,38 %

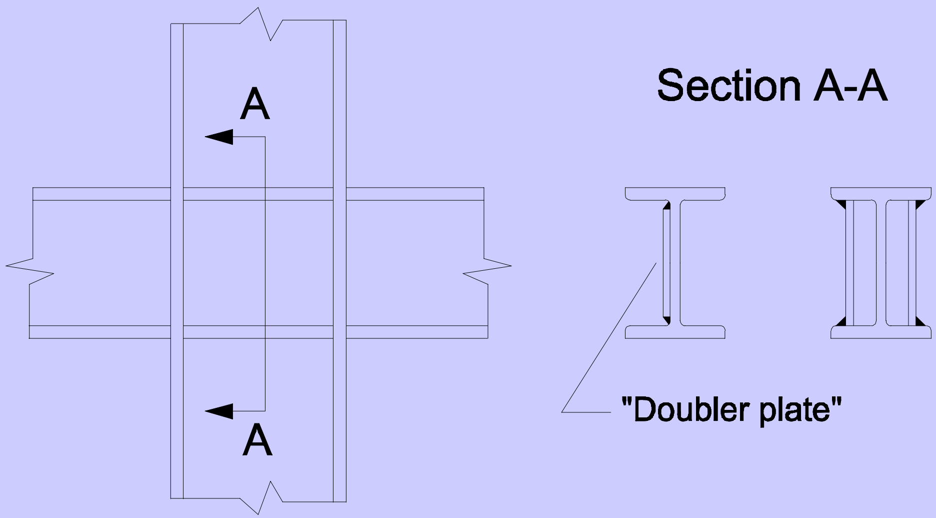
$\Rightarrow \Delta l$ = 9,52 mm

θ = 9,52 / (400/2) = 47,6 mrad >> 35 mrad

Connection design detail ⇔ Ductility classes: National Annexes

Shear resistance of framed web panels



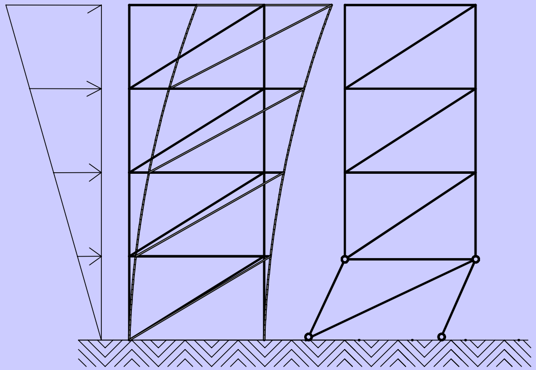


$$\frac{V_{wp,Ed}}{V_{wp,Rd}} \leq 1,0$$

$V_{wp,Rd}$ **shear resistance of the web panel**

$V_{wp,Ed} < V_{wb,Rd}$ $V_{wb,Rd}$ **shear buckling resistance of the web panel**

6.7 Frames with concentric bracings



Dissipative elements: diagonals in tension

Beams and columns resist gravity loads

Diagonals considered in the analysis under seismic action

▲ Frames with diagonal bracings

Standard model: only tension diagonals participate in structural resistance

allowed to consider compression diagonal, if model OK

+non linear analysis

Diagonals $N_{pl,Rd} \geq N_{ed}$

diagonal slenderness: $1,3 < \bar{\lambda} \leq 2,0$

Beams &columns

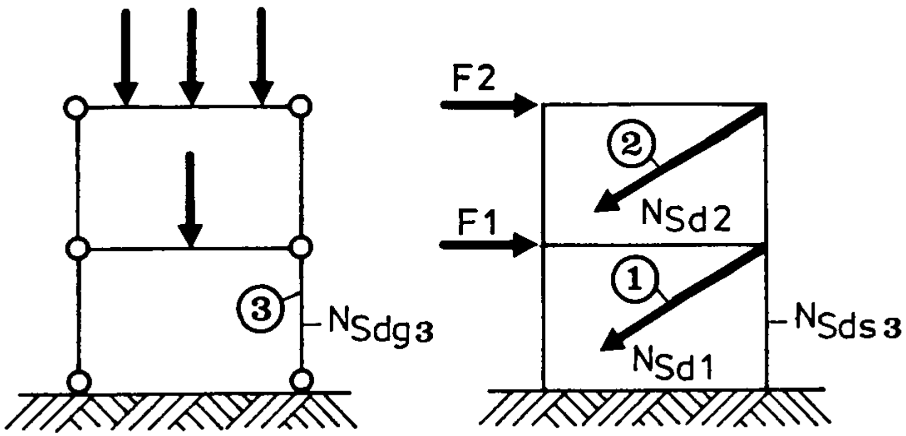
$$N_{Rd}(M_{Ed}) \geq N_{Ed,G} + 1,1\gamma_{ov}\Omega.N_{Ed,E}$$

$\Omega_i = N_{pl,Rdi}/N_{Edi}$

section overstrength of diagonal

For homogeneous dissipative behaviour

$(\max \Omega_i - \min \Omega_i)/\Omega_i = 0,25$



▲ Frames with V or Λ bracings

Dissipative elements: diagonals in tension

Standard model: only beams and columns are in the model for gravity loads

Compression and tension diagonals participate in structural resistance to seismic action

+ and - diagonals considered in standard analysis

Diagonals $N_{pl,Rd} \geq N_{Ed}$ $N_{pl,Rd}$ design bukling resistance

$\bar{\lambda} \leq 2{,}0$

Beams and columns $N_{pl,Rd}(M_{Ed}) \geq N_{Ed,G} + 1{,}1\gamma_{ov}\ \Omega . N_{Ed,E}$

Capacity design to diagonals Ω minimum value of $\Omega_i = N_{pl,Rd,i}/N_{Ed,i}$

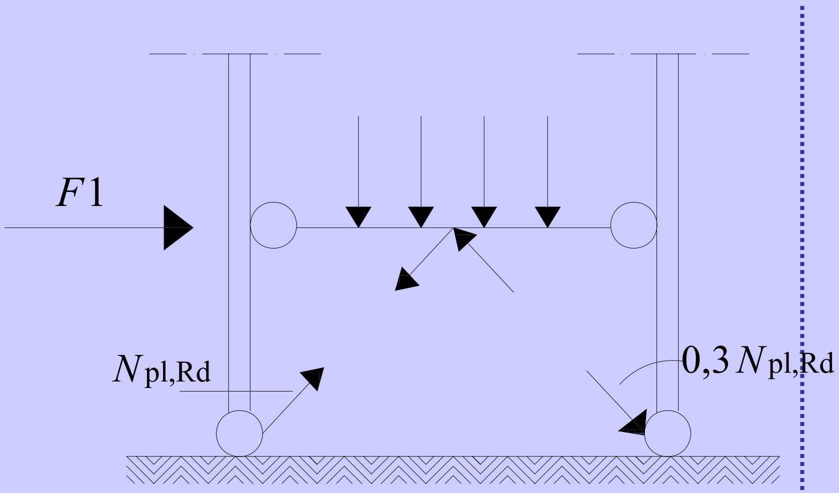
Beams resist all non-seismic actions without considering the intermediate support given by the diagonals+ the unbalanced vertical seismic action effect applied to the beam by the braces after buckling of the compression diagonal.

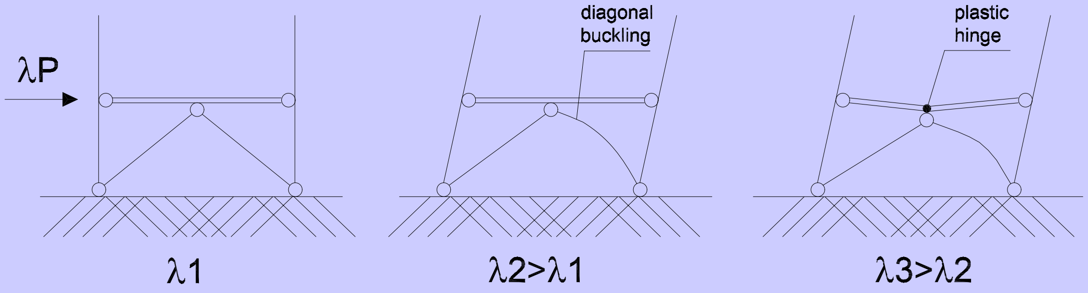
This force is calculated using:

$N_{pl,Rd}$ for the brace in tension

$\gamma_{pb} N_{pl,Rd}$ for the brace in compression

$\gamma_{pb} = 0{,}3$



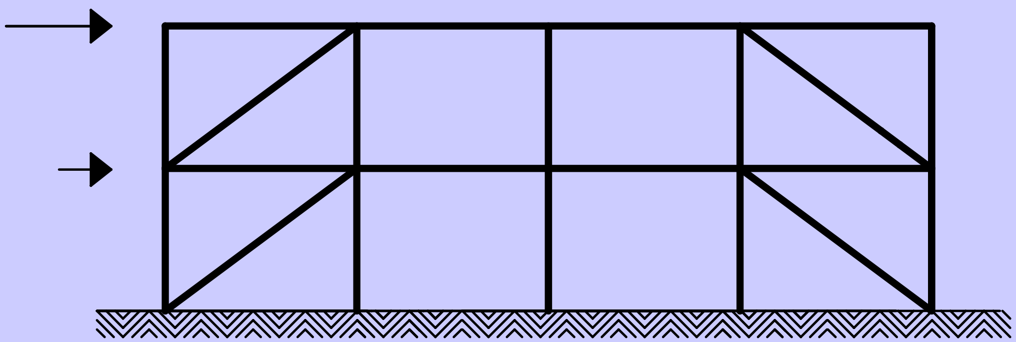


▲ Diagonal bracings - Tension and compression diagonals not intersecting

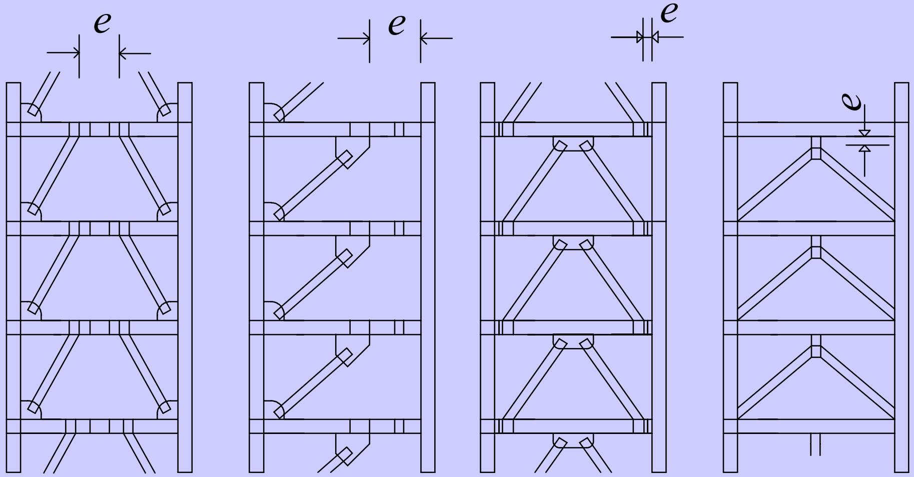
Design should consider tensile and compression forces in columns

- adjacent to diagonals in compression

- corresponding to buckling load of diagonals



6.8 Eccentric bracings



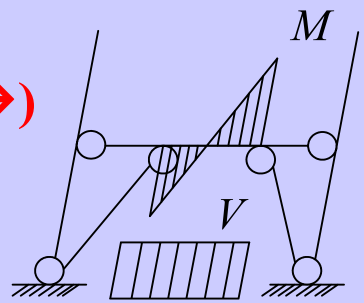
Elements called "seismic links" are designed to dissipate energy

3 categories: short links dissipate energy by yielding in shear
long links dissipate energy by yielding in bending
intermediate links... bending and shear

Length e of links defining categories (symetrical action effects-→)

short links $e < 1{,}6\ M_{p,link}/V_{p,link}$

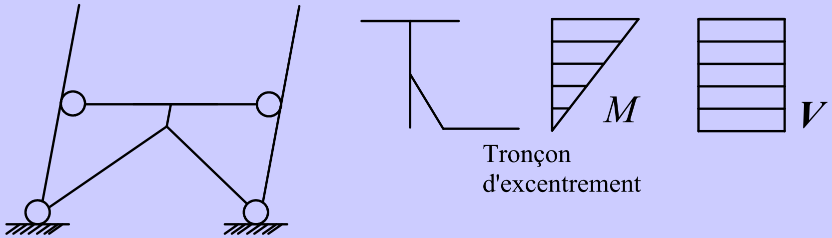
long links $e > 3{,}0\ M_{p,link}/V_{p,link}$



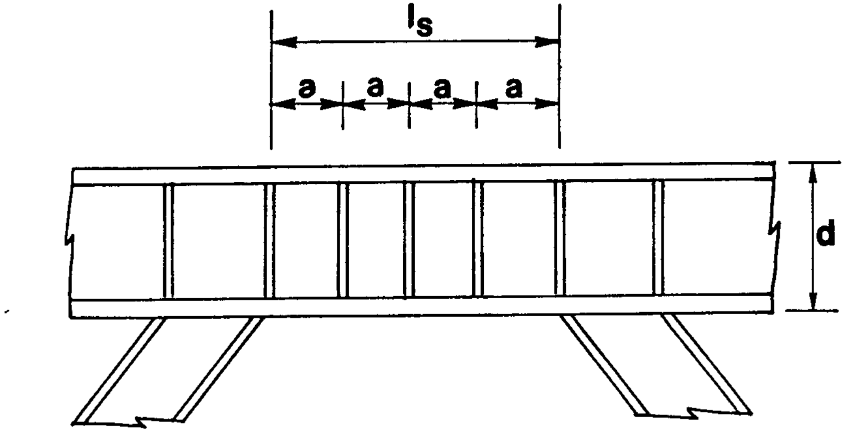
Length e of links defining categories (non symetrical action effects-→)

short links $e_s < 0{,}8\ M_{p,link} / V_{p,link}$

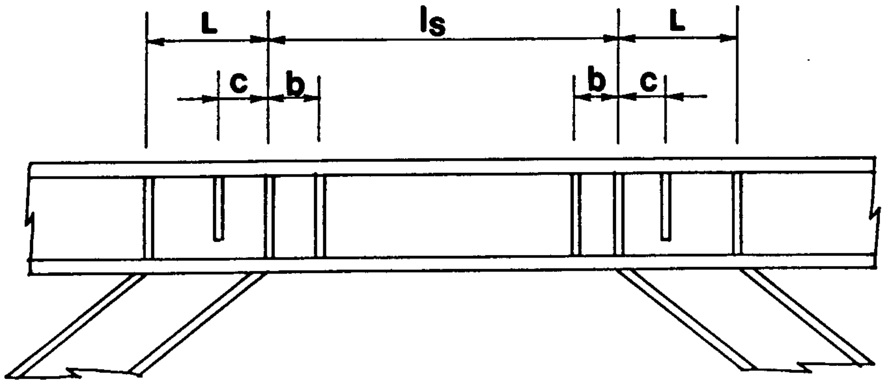
long links $e_L > 1{,}5\ M_{p,link} / V_{p,link}$



Stiffeners in links.



Short links
(shear on complete length)



Long links
(plastic hinges at both ends)

Members not containing seismic links:

Capacity design to the links. Checks: like for concentric bracings

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

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

6.9 Inverted pendulum structure

$\bar{\lambda} \leq 1{,}5$

$\theta \leq 0{,}20$

6.10

Structures with concrete cores or concrete walls

Concrete structure is primary structure

Dual structures

Moment resisting frames and braced frames acting in the same direction: designed using a single *q* factor.

Horizontal forces: distributed between frames according to their elastic stiffness

Mixed structures

Reinforced concrete infills positively connected to steel structure=> composite

Moment resisting frame with infills structurally disconnected from frame on lateral and top sides: design as steel structures.

Infills in contact: frame-infill interaction to take into account.

2 comments.

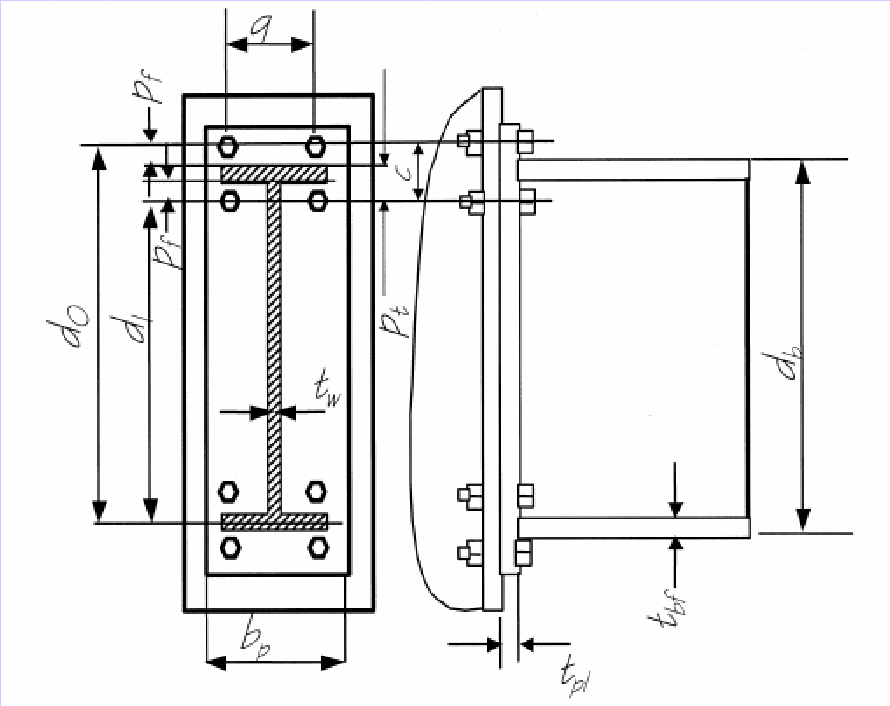
1. In 1994, Northridge earthquake:

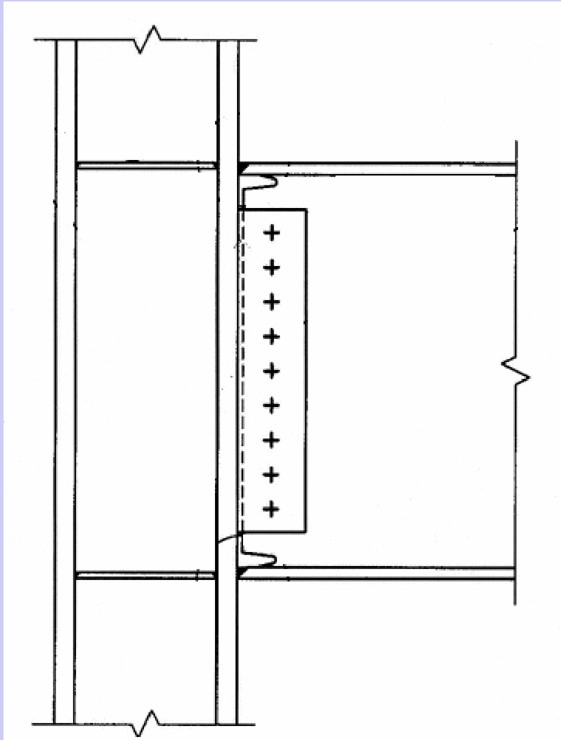
steel connections damaged by hundreds

Unlikely with Eurocodes 3 and 8 and European practice

Europe	US
-Required steel properties toughness	very low
-Weldability of base material	"not for dynamic applications"
-Welding process Europe: shop welds	site welding
-Connection design:	mix of bolts&welds in 1 section

welded end plate at shop-bolts on site





2. Reduced beam sections RBS or "dogbones" were invented in Europe.

Eurocode 8 Section 7.
Composite Steel Concrete Structures.

7.1 General

Design Options

- Steel only => Disconnection (defined)
- Composite=> Rules EC4 + EC8

Design Concepts	q	Ductility classes
Non Dissipative	$1 \leq q \leq 1{,}5$	DCL
Dissipative	$1{,}5 < q < 4$	DCM
	$q \geq 4$	DCH

Ductility classes: plastic deformation capacity without buckling

Non dissipative structures. Eurocode 3 & 4

Requirements on steel material + bolts 8.8 -10.9

only in low seismicity regions

K bracings may not be used

7.2 Materials

Steel: like for seismic design of steel structures

f_y max (not more than 35% higher the steel grade e.g. 235 for S 235)

toughness

Concrete: f_c > C20/25 f_c < C40/50 => C30/35

Rebars: 2 classes (ductile-non ductile)

f_u / f_y A%

7.3 Structural types

Moment resisting frames. **Beams & columns: steel or composite**

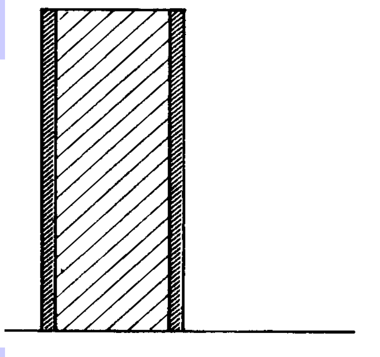
Concentric braced frames. **Columns & beams: steel or composite. Braces: steel**

Eccentrically braced frames. **Columns & beams: steel or composite**
Links: steel, working in shear

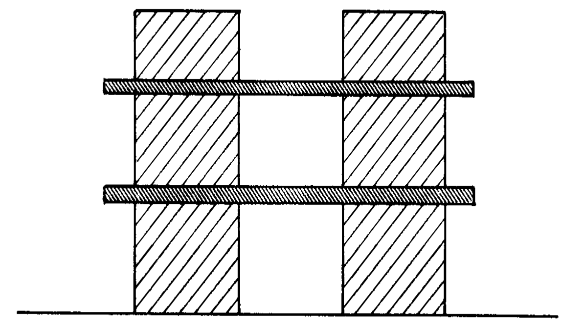
Structural systems. R.C.walls behaviour =>

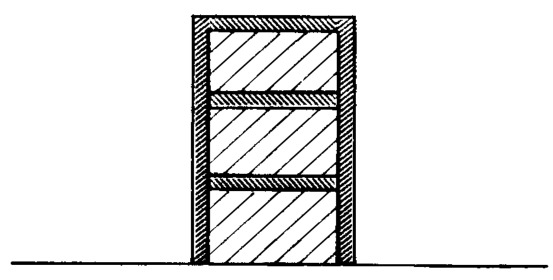
Composite steel plate shear walls

Type 2



Type 3



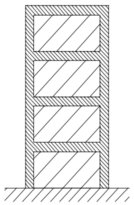
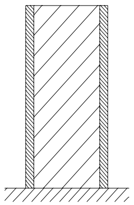
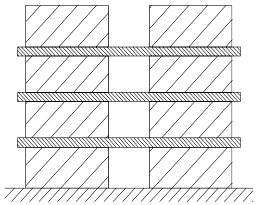


Type 1

Behaviour factors *q*

- ***q*** **for composite moment and braced frames: like steel structures**

- **wall systems. Table**

	DCH	DCM
e) Reinforced concrete shear wall elements. $\frac{\alpha_u}{\alpha_1} \approx 1.1$ TYPE I  Steel or composite moment frame with concrete infill panels. TYPE 2  Concrete walls reinforced by encased vertical steel sections. TYPE 3  Concrete shear walls coupled by steel or composite beams.	$4\frac{\alpha_u}{\alpha_1}$	$2.5\frac{\alpha_u}{\alpha_1}$
f) Composite steel plate shear walls with RC elements. $\frac{\alpha_u}{\alpha_1} \approx 1.2$	$4\frac{\alpha_u}{\alpha_1}$	$2.5\frac{\alpha_u}{\alpha_1}$

7.4. Structural Analysis

Scope: dynamic elastic

$E_a / E_c = 7$

2 Stiffness of sections => effective concrete (*M*+)

=> only rebars (*M*-)

7.5.2 General Criteria for Dissipative Structural Behaviour

Like steel 6.5.2

7.5.3 Plastic resistance of dissipative zones

Two plastic resistances considered:

- a lower bound in checks of sections of dissipative elements
  of global seismic resistance
  computed considering concrete and ductile steel components

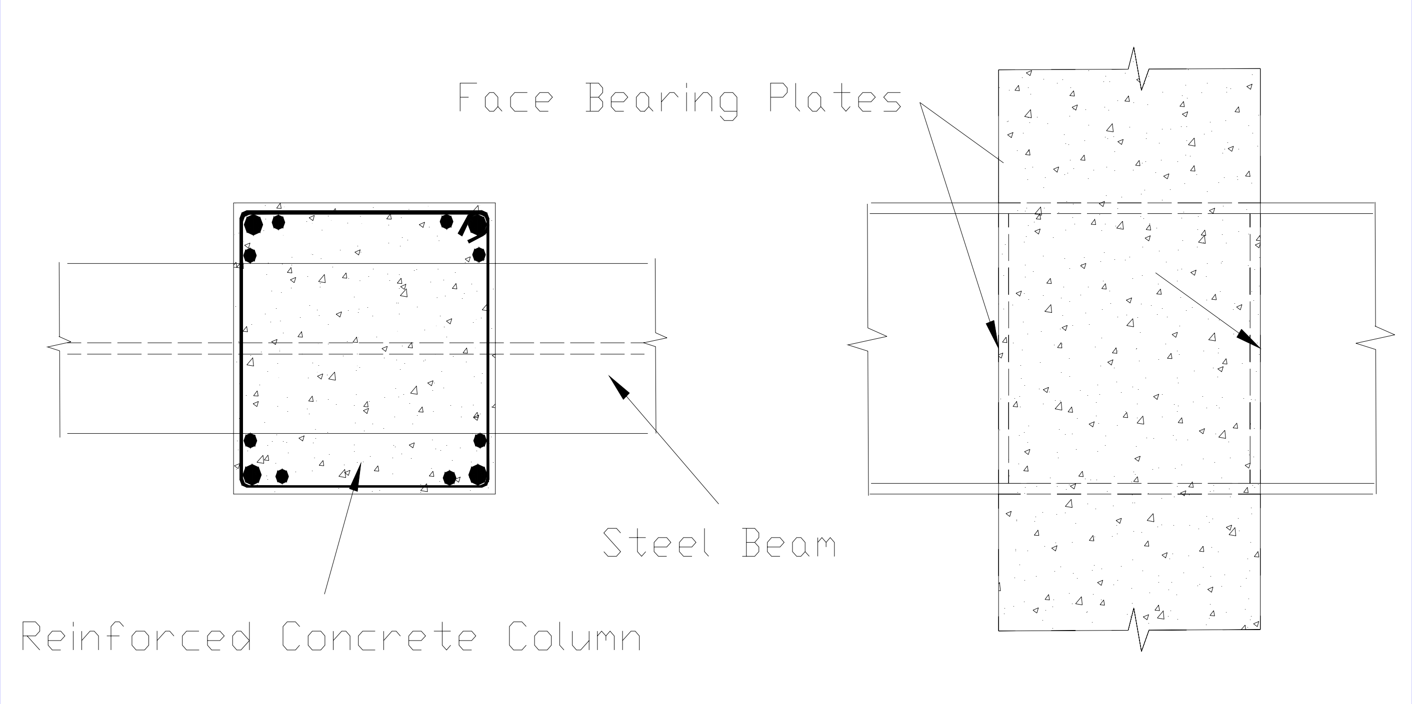
- an upper bound for capacity design of elements&connections
  adjacent to the dissipative zone
  computed considering all components in the section
  including non ductile ones (e.g. welded meshes).

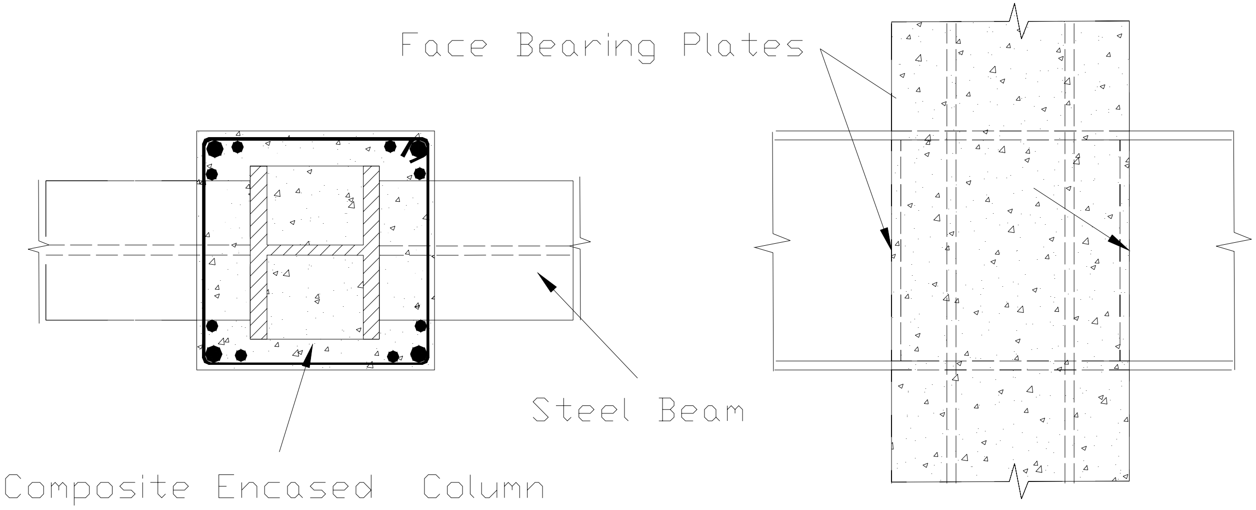
7.5.4 Detailing rules for composite connections in dissipative zones

Design objective: integrity of concrete, yielding in steel

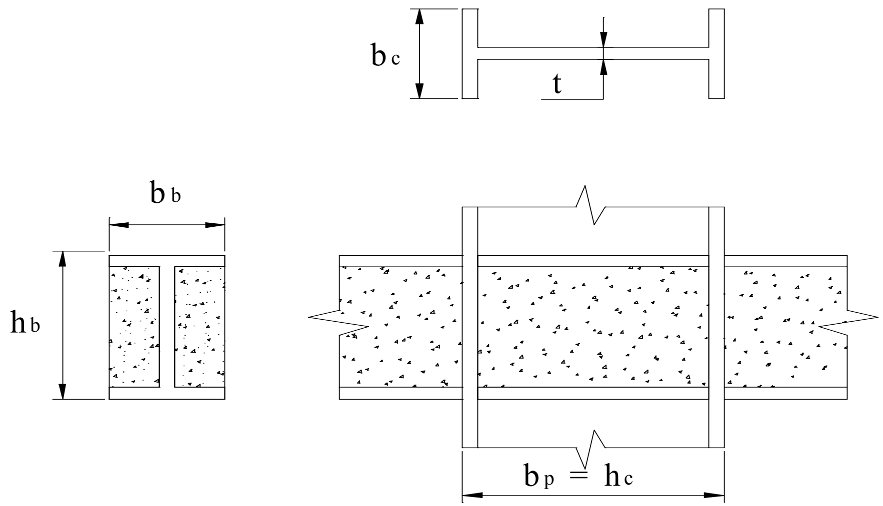
- **Dissipative connections allowed**
- **Rebars sections in joint region: models satisfying equilibrium**
- **Yielding of rebars allowed**
- **In fully encased framed web panels of beam/column connections**
- **Panel zone resistance = Σ concrete & steel shear panel resistance**

aspect ratio h_b/b_p of the panel satisfies conditions



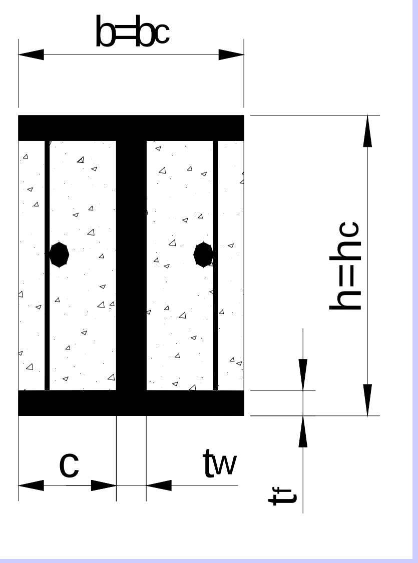


In partially encased stiffened web panels: similar Σ, additional conditions



Vertical rebars to take beam shear force

If composite column, distribute beam shear between steel and concrete



7.6 Rules for members. General

Local ductility of members in compression and/or bending
=> walls slenderness DCH: 35 mrad DCM: 25 mrad
Steel and unencased steel parts of composite sections: EC3-EC4

Limits for partially encased relaxed if straight bars provided
Section classes for partially encased: DCH, DCM, DCL
=> Class 1, 2, 3 of EC4

Ductility Class of Structure	DCH	DCM	DCL
Behaviour Factor q	$4 < q$	$1.5 < q < 4$	$1 < q < 1.5$
Partially Encased			
flange outstand limits c/t	9 ε	14 ε	20 ε
with straight bars welded to flanges	13,5 ε	21 ε	30 ε
Filled Rectangular			
h/t limits	24 ε	38 ε	52 ε
Filled Circular			
d/t limits	80 ε²	85 ε²	90 ε²

$\varepsilon = (f_y/235)0.5$

Columns

Columns generally not dissipative => EC 4 design

Columns may be dissipative : - at ground level in moment frames
- top&bottom of fully encased columns at any storey
(= "critical zones" of RC)

Bond and friction shear resistance not reliable in cyclic conditions

In non-dissipative columns design bond stress = 1/3 static

If bond stress insufficient => shear connectors

For all columns, in bending, steel alone or combined resistances of steel and concrete may be considered

For shear resistance: strong restrictions (research needed)

fully encased => concrete section resistance

partially encased => steel section resistance

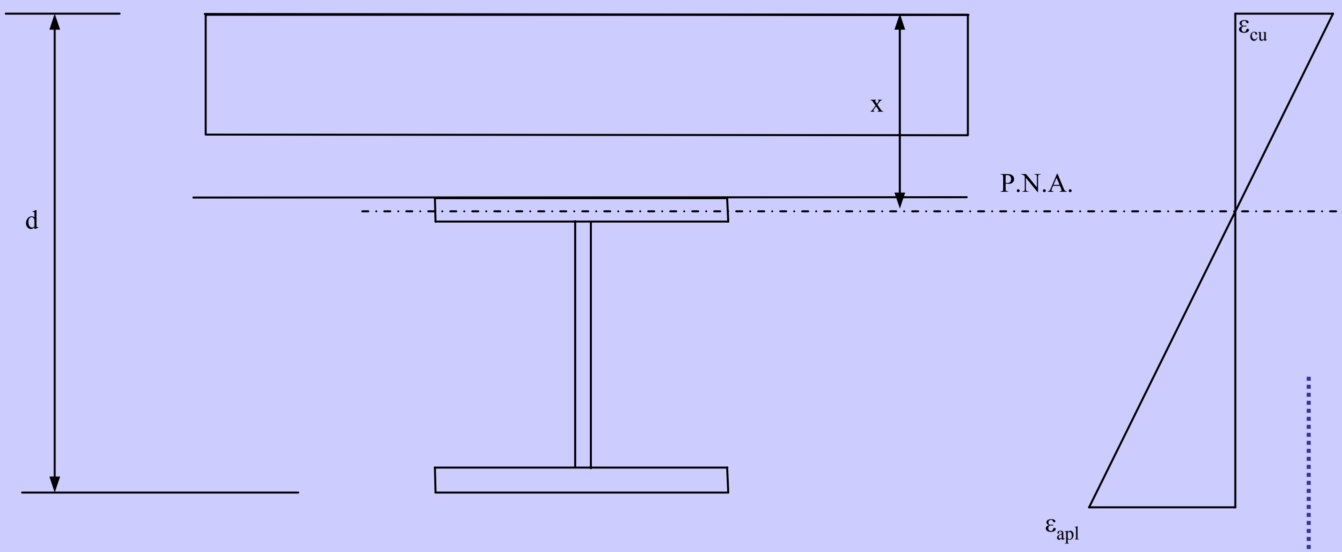
filled => either steel or concrete considered resistance

Steel beams with slab

Design objective: - maintain integrity of slab
- yielding in steel section and/or rebars

Ductility in plastic hinges
P.N.A= Plastic Neutral Axis
ε_{cu} = concrete crushing strain
ε_{apl} = plastic strain of steel

$$x/d \leq \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{apl}}$$



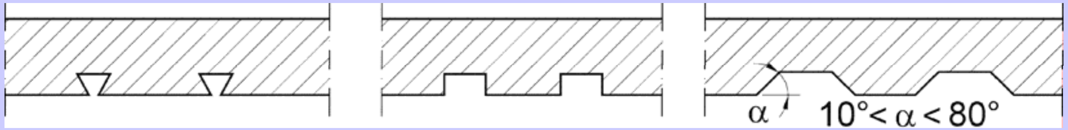
ε_{cu}= 2,5 10-3
$\varepsilon_s = q\ \varepsilon_y = q\ f_y\ / E$
=>x/d upper limits

Ductility class	q	f_y	x/d upper limit
DCH	$q \geq 4$	355	0,19
DCH	$q \geq 4$	235	0,26
DCM	$1,5 < q < 4$	355	0,26
DCM	$1,5 < q < 4$	235	0,35

Steel beams with slab

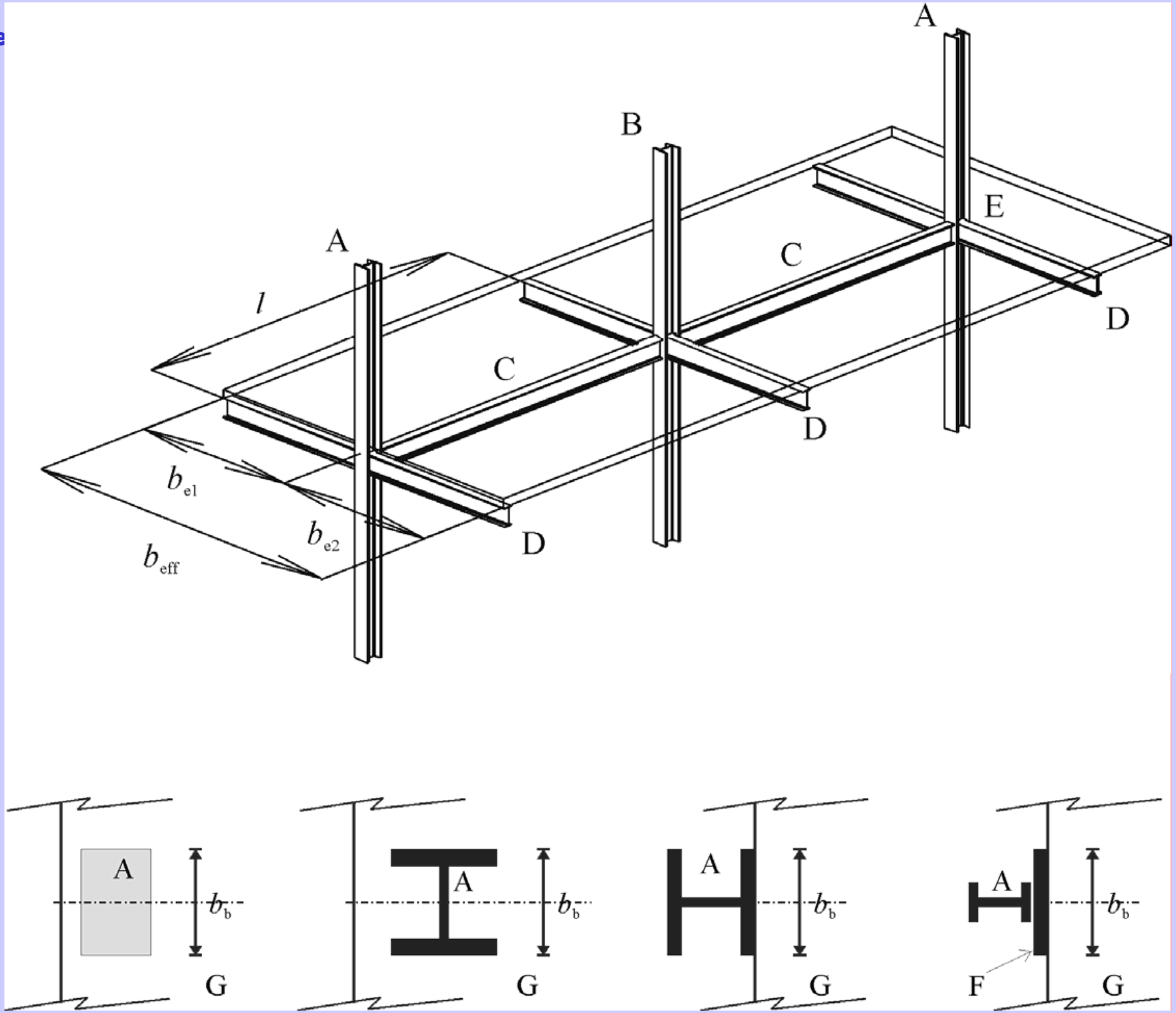
- Partial shear connection in dissipative zones of beams OK if
  - # in *M*>0 region, connection degree > 0,8
  - # total resistance of connectors in *M*<0 region > plastic resistance of rebars.

-Reduction of shear resistance by a rib shape efficiency factor k_r if steel sheeting with ribs transverse to beams



***k*r = 1** **   *k*r = 1** **   *k*r = 0,8**

-Full shear connection required with non ductile connectors



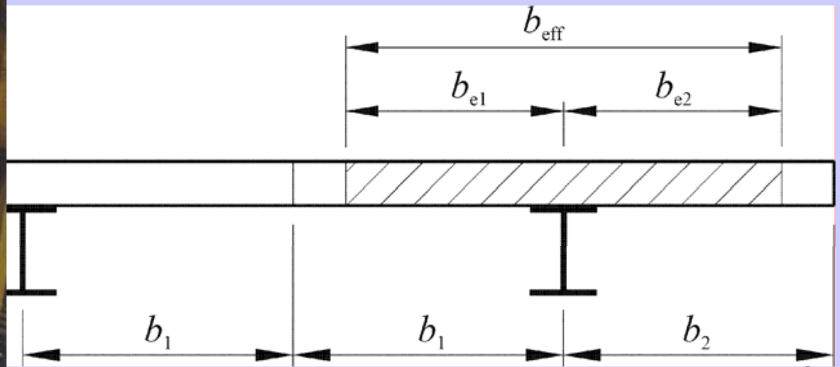
Definition of longitudinal & transverse elements + details in Moment Frame Structure

Effective width b_{eff}

$b_{eff} = 2\ b_e$ $b_{eff} \neq$ for I elastic analysis

M_{pl} plastic resistance

b_e	Trans.beam	b_e for M_{Rd}	b_e for I
-Interior	Present	M^-: 0,1L	0,05 L
column or not		M^+: 0,075 L	0,025
-Exterior	Fixed to	M^-: 0,1 L	0,05L
column column		M^+: 0,075 L	0,025L
-Exterior	Not active.	M^-: 0	0
column		M^+:$b_c/2$ or $h_c/2$	0,025 L

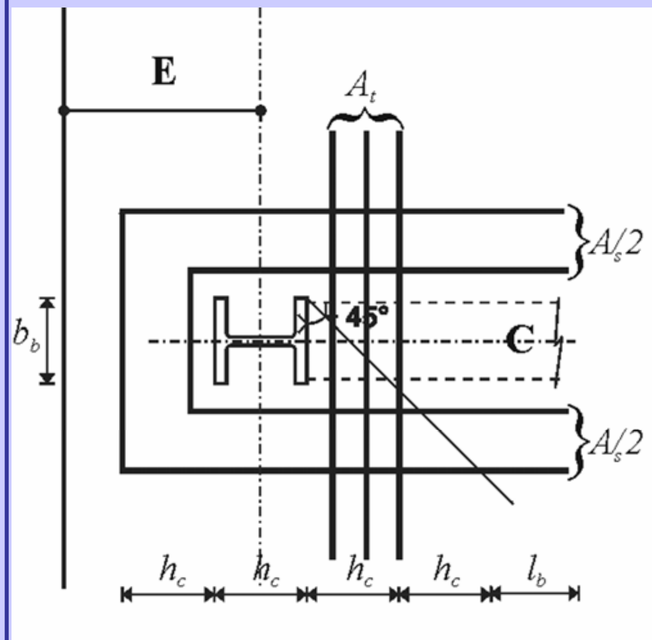


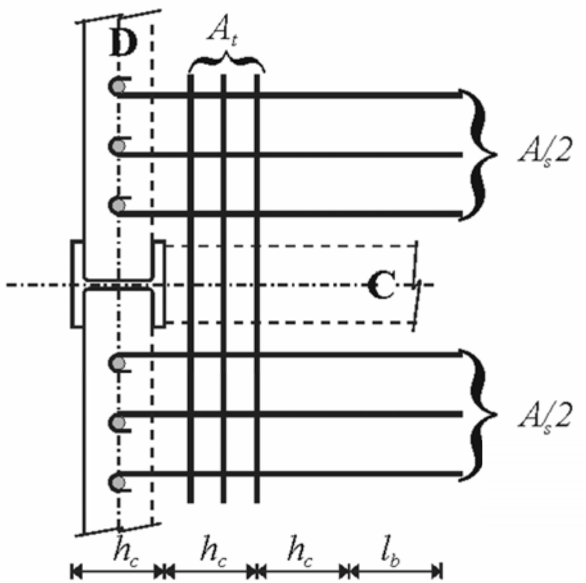
Moment Resisting Frames

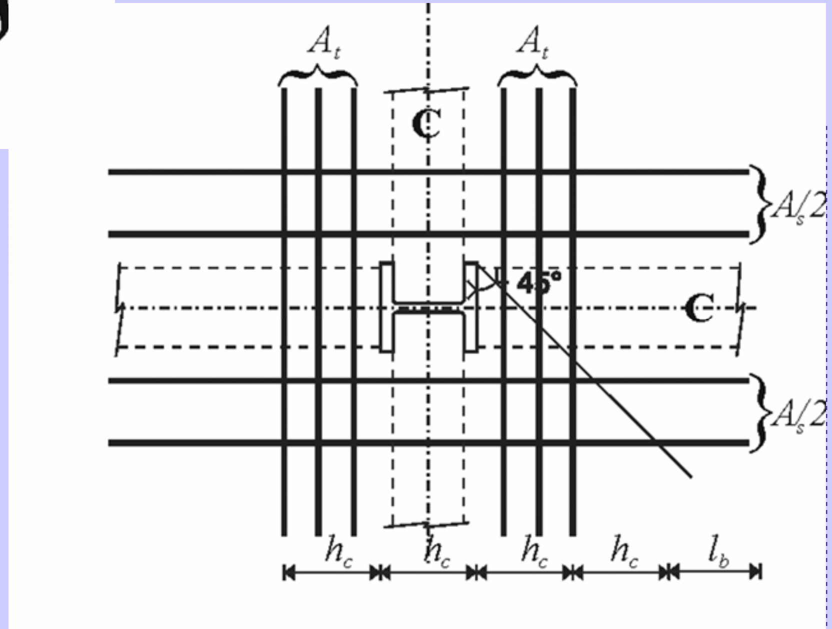
Dissipative zones in beam with slab: vicinity of columns

"Seismic rebars" needed

Section and layout to achieve ductility => Annex C







7.7 Moment frames

In beams, two different stiffness :

EI_1 part of spans submitted to $M > 0$ (slab uncracked)

EI_2 $M < 0$ (slab cracked)

Or an equivalent inertia I_{eq} : $I_{eq} = 0.6\ I_1 + 0.4\ I_2$

Columns: $(EI)c = EI_a + 0.5\ E_{cm}\ I_c + E\ I_s$

E_s and E_{cm} : modulus of elasticity for steel and concrete

I_a , I_c and I_s : moment of inertia of steel section, concrete and rebars

Composite trusses may not be used as dissipative beams.

Concrete disconnection rule

Beam plastic resistance: only steel if slab totally disconnected from steel frame in a diameter $2b_{eff}$ zone around a column

ANNEX C:

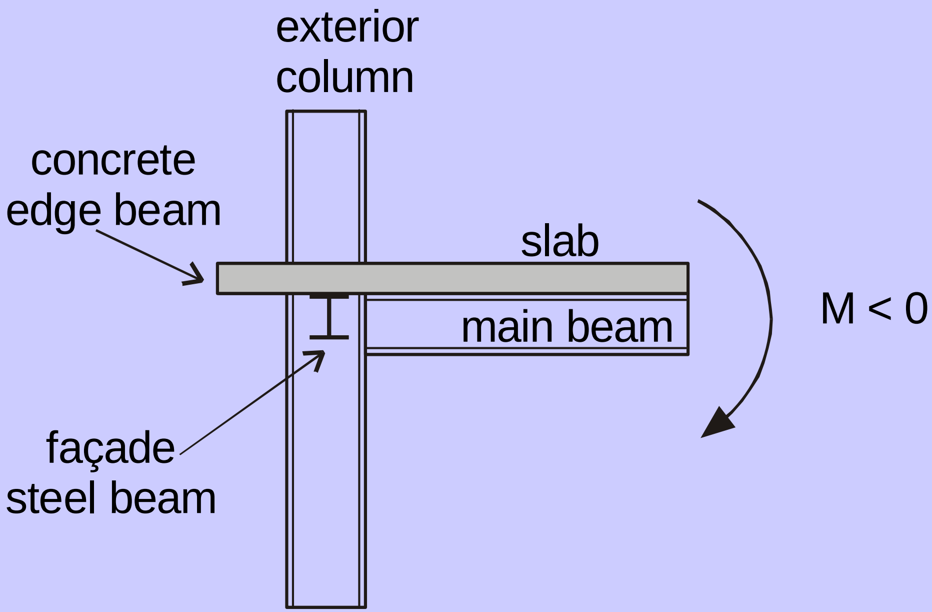
SEISMIC DESIGN OF THE SLAB REINFORCEMENTS OF COMPOSITE T BEAMS WITH SLAB IN MOMENT FRAMES

General: 2 conditions to ensure ductility in bending

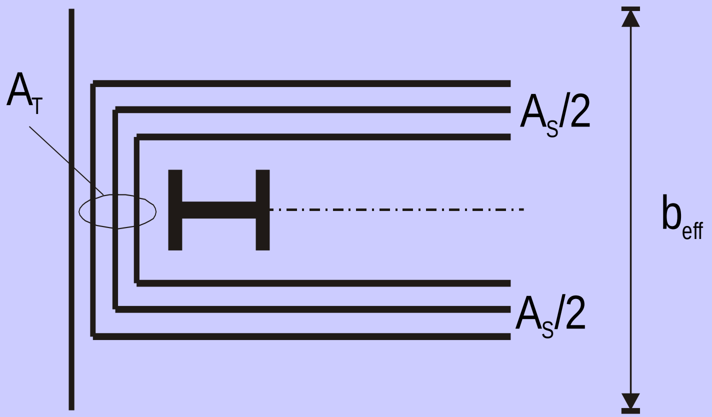
- avoid early buckling of steel section (classes of sections + *x/d*)
- avoid early crushing of slab concrete (*x/d* limit + rebars required)

=> 2 limits of section A_S of reinforcement in the slab

EC4: negative moment & no transverse steel beam

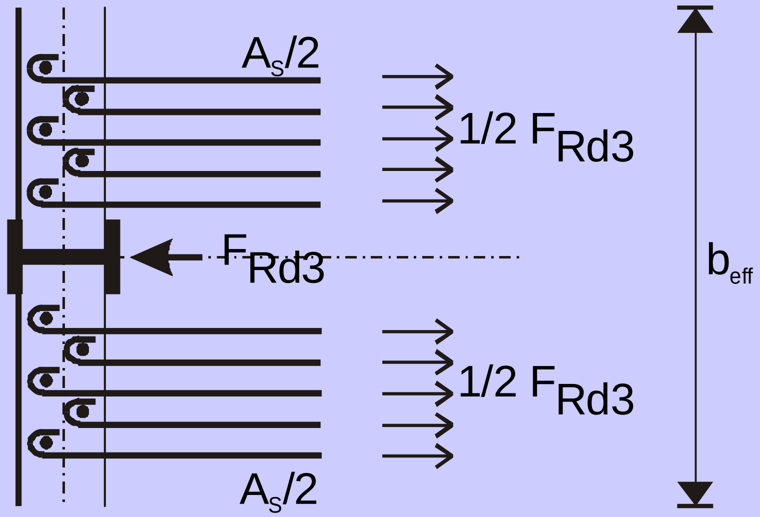


concrete edge beam
no façade steel beam
see section AJ.3.1.2.

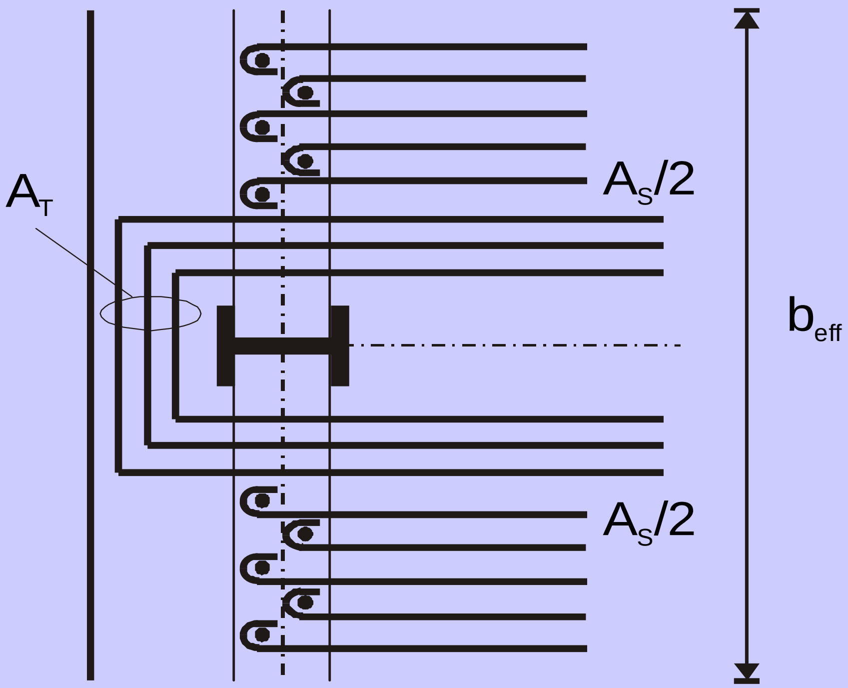


M<0 Exterior Column Case

no concrete edge beam
façade steel beam
see section AJ.3.1.3.



concrete edge beam
façade steel beam
see section AJ.3.1.4.

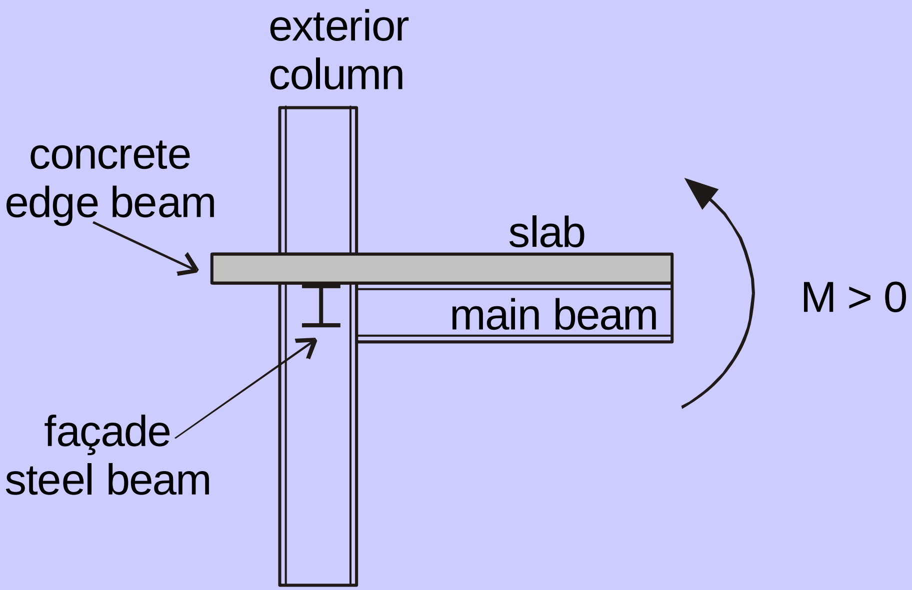


$A_S \leq F_{Rd3}/(f_{sk}/\gamma_s)$
$F_{Rd3} = n \times F_{stud}$
n = number of connectors in the effective width
$F_{stud} = P_{Rd}$ = design resistance of one connector
façade beam checked in bending, shear and torsion

Rebars: Hairpin (EC4)
+
bars anchored in facade beam

Exterior Column Case

3 Force Transfer Mechanisms of Slab Compression



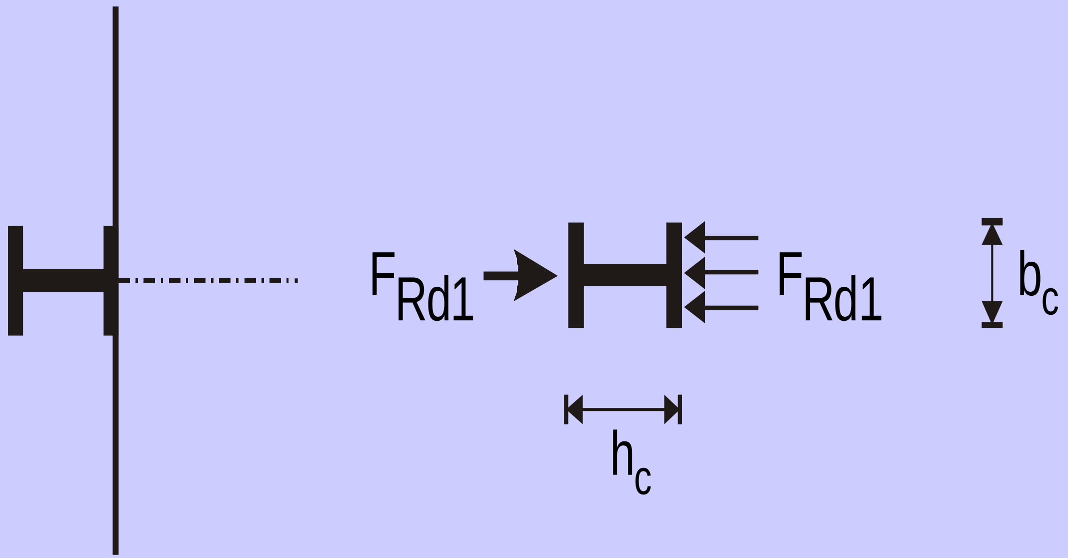
Mechanism 1

Direct compression on column

$F_{Rd1} = b_c\, d_{eff}\,(0.85\, f_{ck}/\gamma_c)$

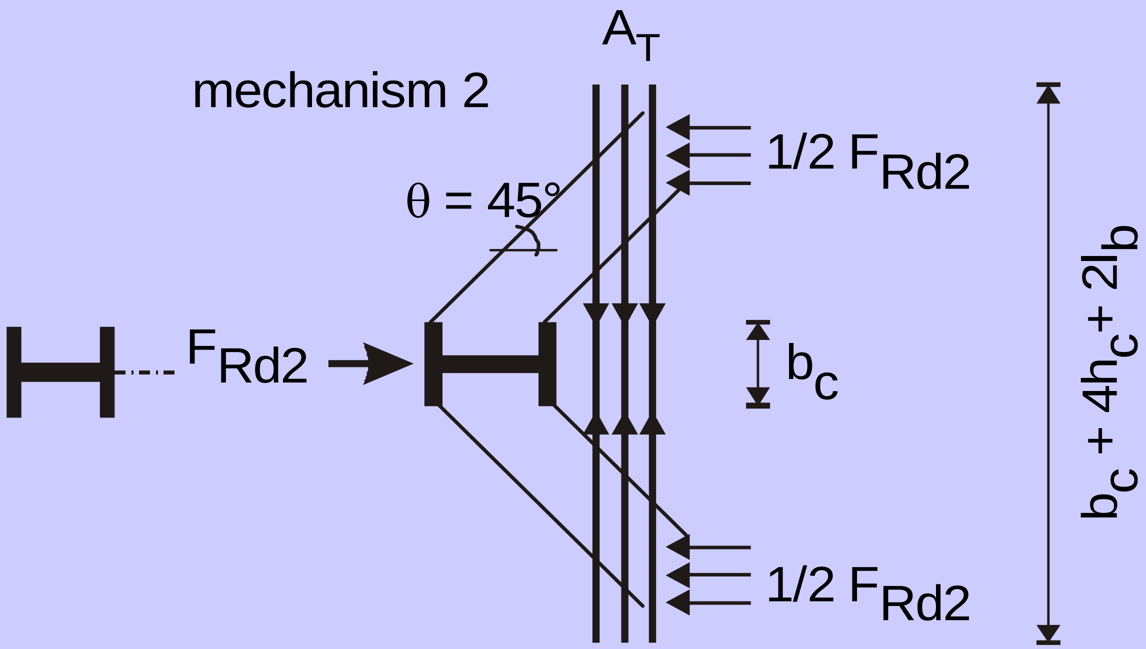
Confinement of concrete close to column flange by transverse re-bars

no concrete edge beam
no façade steel beam
see section AJ.3.2.1.



Mechanism 2
Compression on column sides by concrete struts

concrete edge beam or concrete into the column flanges
no façade steel beam
see section AJ.3.2.2.



$$F_{Rd2} = 0.7\, h_c\, d_{eff}\, (0.85\, f_{ck}/\gamma_c)$$

$$A_T \geq \frac{F_{Rd2}}{f_{sk,T}/\gamma_s} = 0.3\, h_c\, d_{eff}\, \frac{f_{ck}/\gamma_c}{f_{sk,T}/\gamma_s}$$

d_{eff} : depth of the slab

max compr. force : $F_{Rd1} + F_{Rd2} = b_{eff}\, d_{eff}\, (0.85\, f_{ck}/\gamma_c)$

$b^{+}_{eff\ connec} = 0.7\, h_c + b_c \cong 1.7\, b_c \cong 0.085\, L$

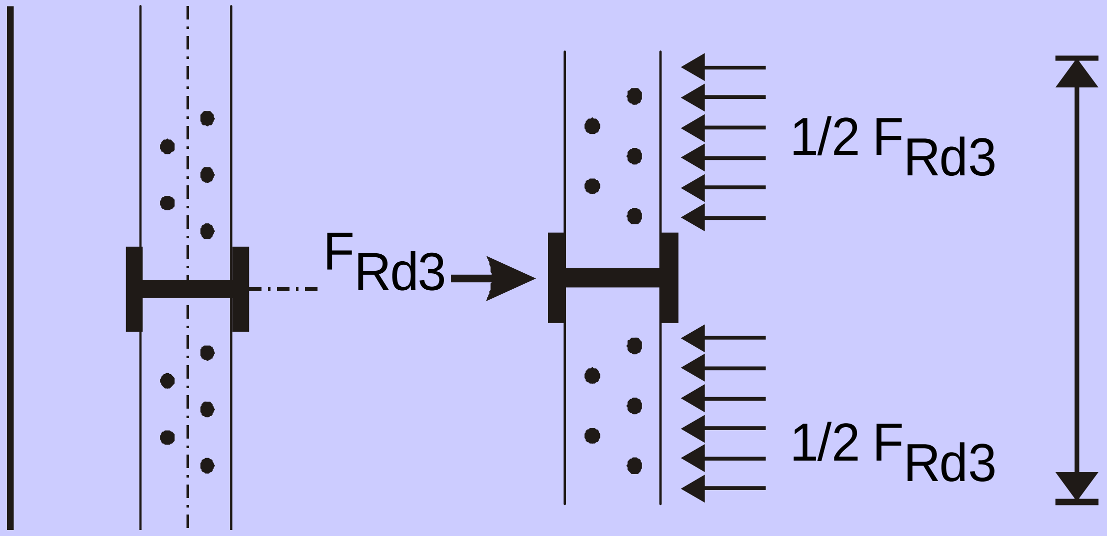
$<< b^{+}_{eff} = 0.15\, L \cong 0.5\, b^{+}_{eff}$ *(EC4)*

Mechanism 3 Compression on connectors of facade steel beam

$F_{Rd3} = n \times F_{stud}$ n = number of connectors in effective width

$F_{stud} = P_{Rd}$ = design resistance of one connector

concrete edge beam
present or not
façade steel beam
see section AJ.3.2.3.



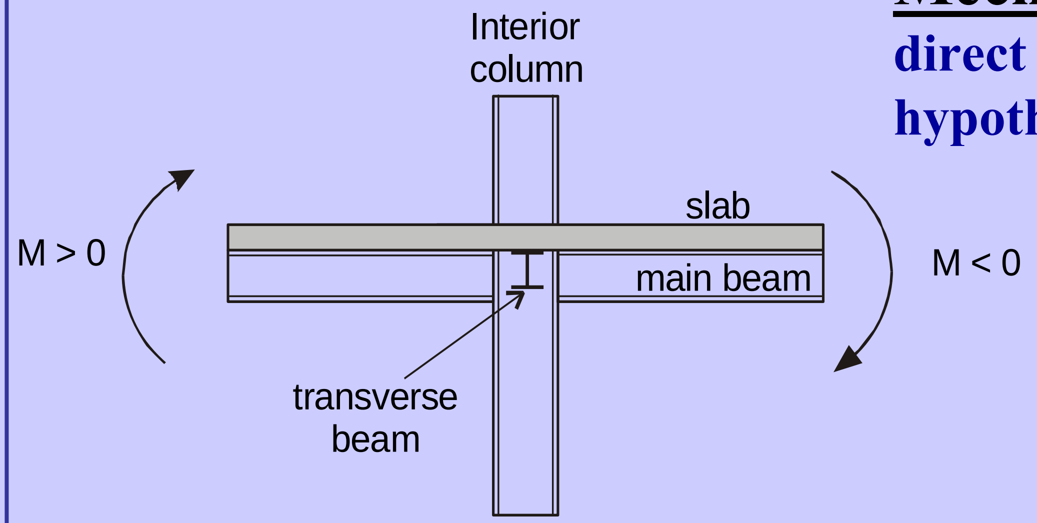
maximum compression force $b_{eff}\, d_{eff}\, (0.85\, f_{ck}/\gamma_c)$

transmitted if:

$$F_{Rd1} + F_{Rd2} + F_{Rd3} > b_{eff}\, d_{eff}\, (0.85\, f_{ck}/\gamma_c)$$

=> choose n to achieve adequate F_{Rd3}

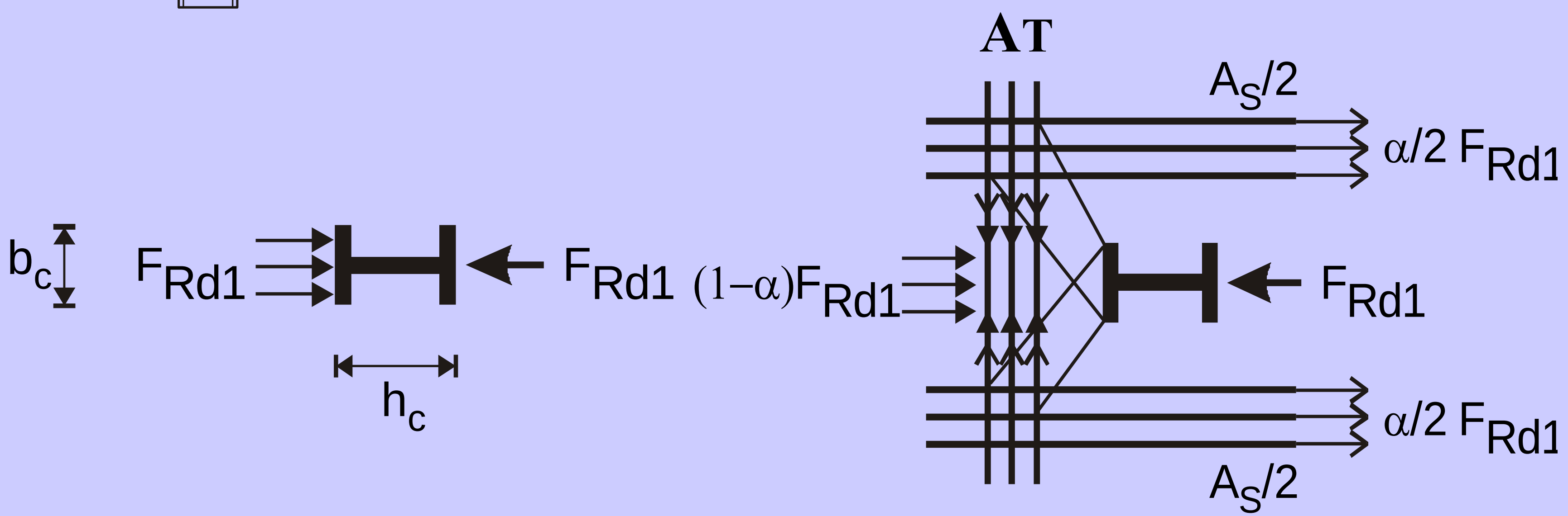
Interior Column Case



Mechanism 1

direct compression on the column

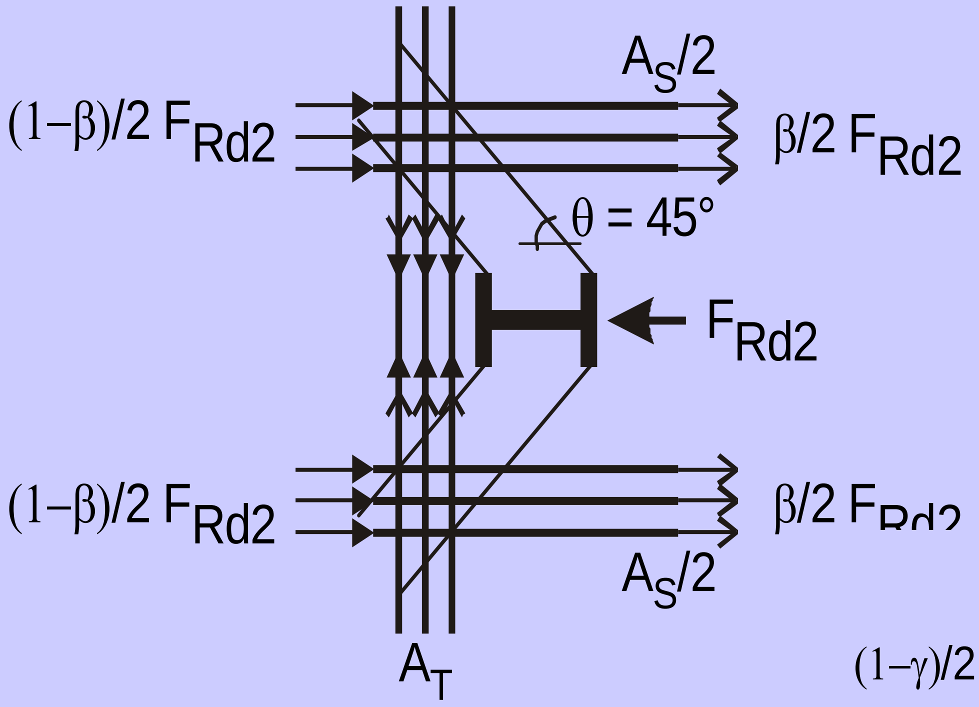
hypothesis: $A_T = A_S / 2$



Mechanism 2

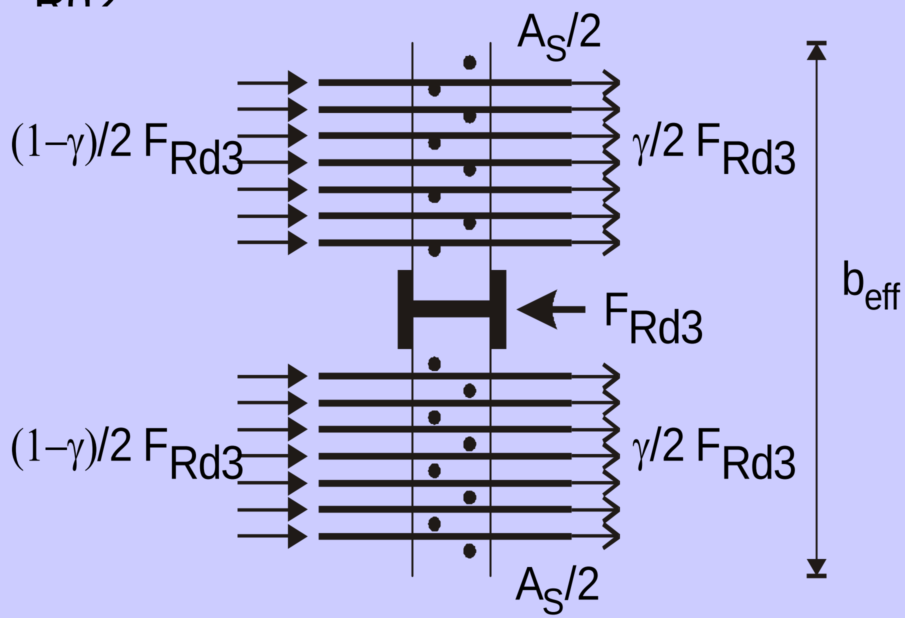
Compressed concrete struts

Interior Column Case



Mechanism 3

Connectors on transverse beams



Interior Column Case

Without Transverse Beam:

$$A_T \geq \frac{F_{Rd2}}{f_{sk,T}/\gamma_s} = 0.3 h_c d_{eff} \frac{f_{ck}/\gamma_c}{f_{sk,T}/\gamma_s}$$

$F_{Rd1} = b_c\, d_{eff}\, (0.85\, f_{ck}/\gamma_c)$
$F_{Rd2} = 0.7\, h_c\, d_{eff}\, (0.85\, f_{ck}/\gamma_c)$
same section A_T on each side of column

Resistance: $F_{Rd1} + F_{Rd2} = (0.7\, h_c + b_c)\, d_{eff}\, (0.85\, f_{ck}/\gamma_c)$
Applied force : tension of re-bars (M- side) + compression of concrete (M+ side)
$F_{St} + F_{Sc} = A_S\, (f_{sk}/\gamma_s) + b^{+}_{eff}\, d_{eff}\, (0.85\, f_{ck}/\gamma_c)$

Impossible to transfer force corresponding to effective width under M > 0 & M < 0
=>situation is not controlled = no ductility

With Transverse Beam

F_{Rd3} activated $F_{Rd3} = n \text{ x } F_{stud}$
Resistance: $F_{Rd1} + F_{Rd2} + F_{Rd3} = (0.7\, h_c + b_c)\, d_{eff}\, (0.85\, f_{ck}/\gamma_c) + n\, F_{stud}$
Check $1.2\, (F_{Sc} + F_{St}) \leq F_{Rd1} + F_{Rd2} + F_{Rd3}$

The situation is controlled and the transferred forces correspond to the EC8 effective widths b-eff = 0.2 L and b+eff = 0.15L

7.8 Composite concentrically braced frames

Concepts

- Yielding of diagonals in tension
- Tension only design & no composite braces

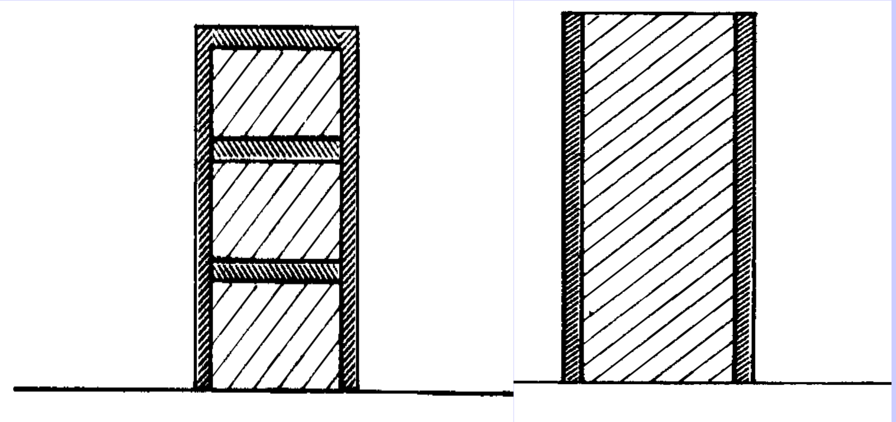
7.9 Composite eccentrically braced frames

- Dissipative action occur through yielding in shear of links
- All other members remain elastic
- Links may be short or intermediate with a maximum length *e*

  $e < 2M_{p,\,link} / V_{p,\,link}$
- Links are made of steel sections, possibly composite with slabs, not encased
- In a composite brace under tension, only the steel section is considered in the resistance of the brace
- Failure of connections is prevented

7.10 Systems made of reinforced concrete shear walls composite with structural steel elements

Type 1 and 2 designed to behave as shear walls and dissipate energy in vertical steel sections and rebars



Type 1 Steel or composite frame with concrete infills

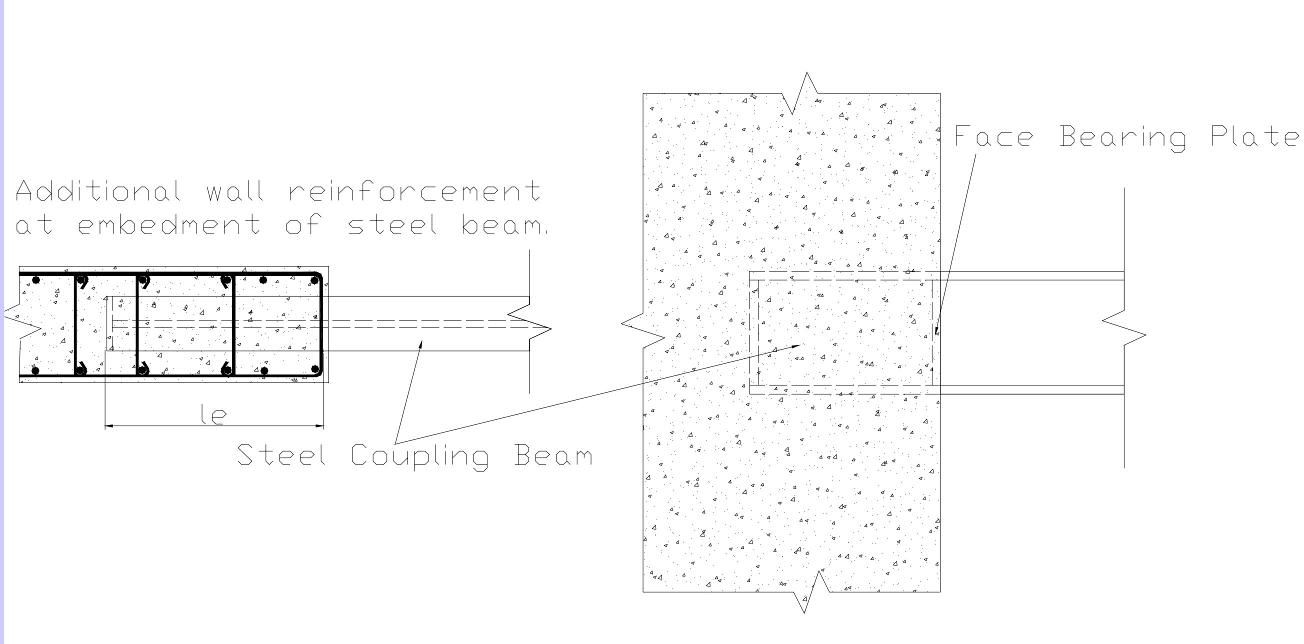
Type 2 Concrete walls reinforced by vertical steel sections

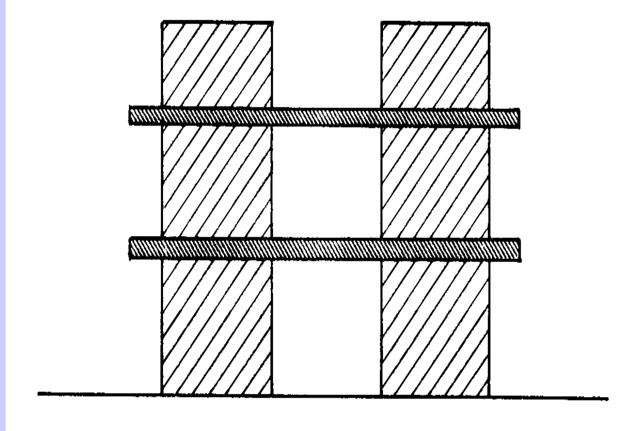
Type 1 and 2 = walls with plastic hinge at base
vertical encased steel = reinforcements for bending

Shear carried by the reinforced concrete wall
Gravity and overturning moment carried by the wall acting composedly with the vertical boundary members

Type 3

designed to dissipate energy in the walls
in the coupling beams



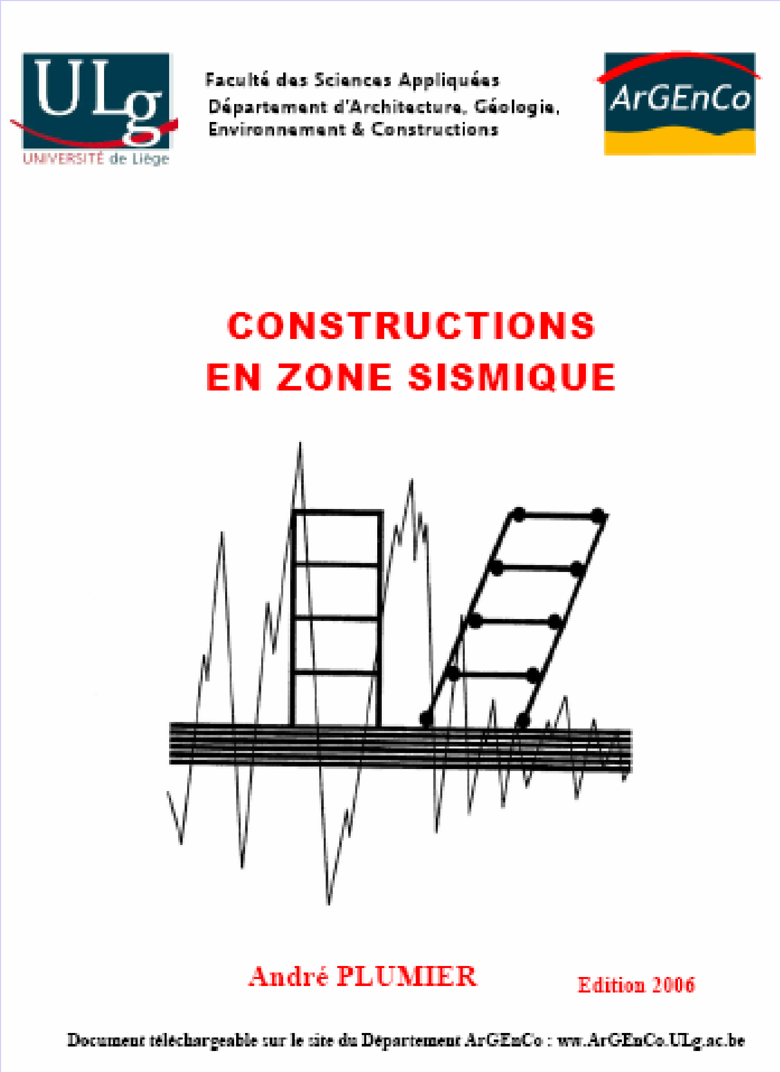


Embedment length le required l_e **= 1,5 x steel beam depth**
Rules on connections apply: face bearing plates, vertical rebars sections, etc

7.11 Composite steel plate shear walls

Designed to yield through shear of the steel plate

Stiffened by encasement and attachment to reinforced concrete to prevent buckling of steel.



And finally… education is the key

2 personal involvments:
- writing 1 book for students
- Organising seminars in Algeria for a total of 15 days
On seismic design of bridges, soils and foundations, buildings and retrofitting.

With the financial help
of the European Investment Bank
With the friendly contribution of a number of specialists.
With constant reference to Eurocode 8

Thank you for your attention !