# Sections 6 and 7.

# Steel and Composite Steel Concrete Buildings.

*Prof. André PLUMIER*

Université de Liège

1. Context and background of Sections on Steel and Composite Steel – Concrete.
2. Design Rules for Steel Structures
3. Design Rules for Composite Steel Concrete Structures
4. Dissemination

# Eurocode 8 rules on steel & composite structures

**1986. ECCS Design Recommandations**

ECCS: European Convention for Constructional Steelwork

Aribert, Ballio, Mazzolani, Plumier, Sedlacek

**1994. Eurocode 8 = ENV**

Steel structures ≈ ECCS Recommendations

Composite steel concrete: poor information

**1994: Northridge earthquake** **1995: Kobe earthquake**

**Many cracked steels connections**

**=>1994 – 2004: EU research projects**

**STEELQUAKE RECOS 3D Ispra test ICONS**

**Large testing installation funding: ECOEST ECOLEADER**

**+ US and Japan research** **=> Improved design rules.**

# Steel. EU Background. STEELQUAKE Project



## 1994-1998.
## Connection design

**Ductility**

**Low cycle fatigue**

**Welding procedure**

**POLIMI**
**ISMES**
**NTUA**
**ULIEGE**
**JRC Ispra**

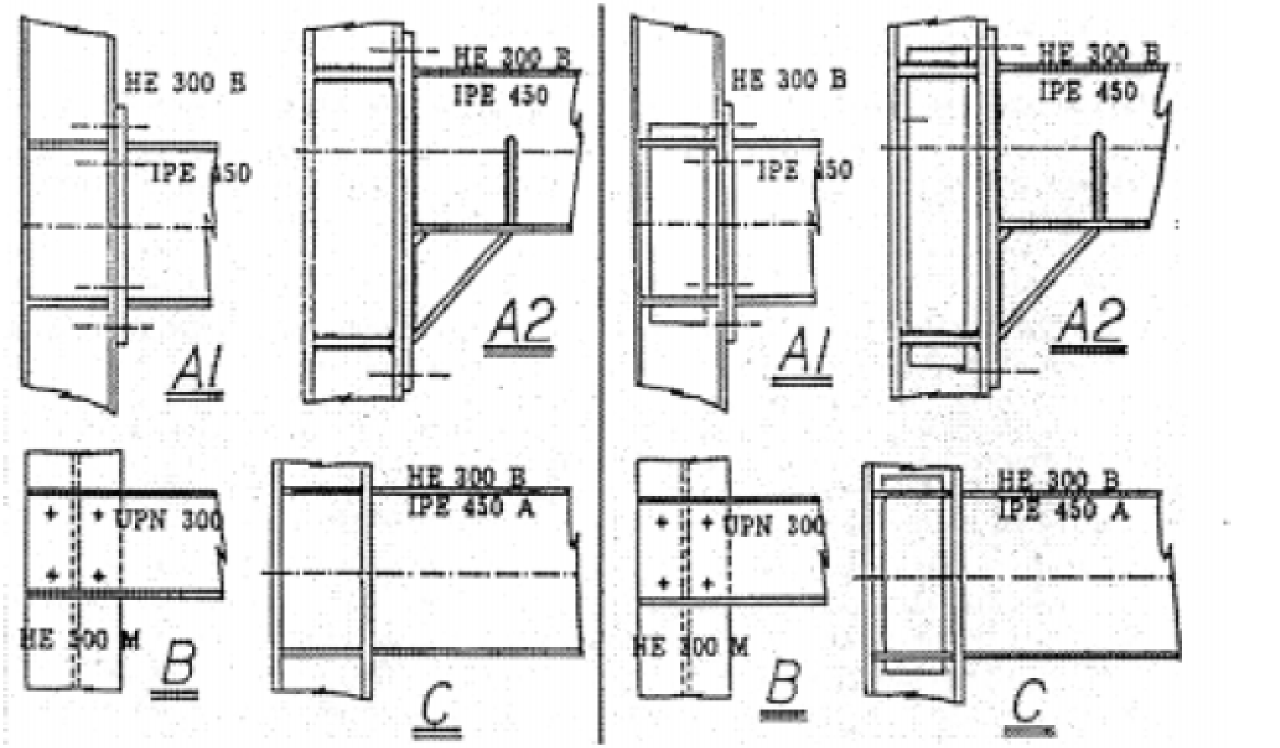



Figure 1. Beam to column connection typologies without (left) and with (right) node reinforcing plate.

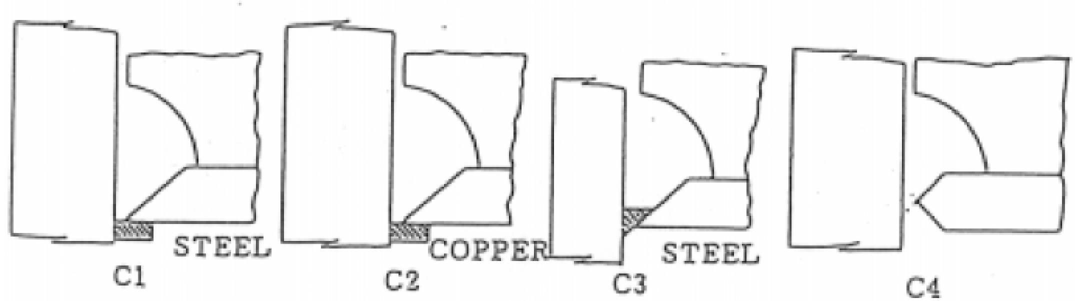
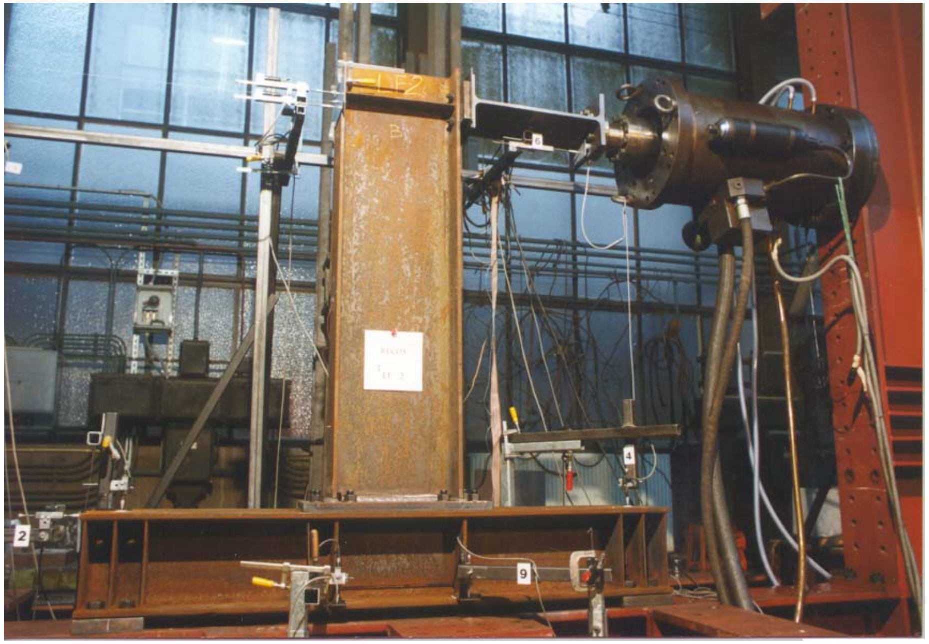



Figure 2. Different welding procedures C1 and C3: with steel backing bar; C2: with polyester backing bar.

## Moment Resisting Connections of Steel Frames in Seismic Area

**1997 – 1999. DG XII European Commission-Copernicus Programme.**
**U.NAPOLI, U.SOFIA, INSA.RENNES, NTU.ATHENS, IST.LISBON,TIMISOARA, U.LJUBLJANA, INCERC.TIMISOARA, ULIEGE.**

**Material behaviour, Ductility of members and connections.**
**Models for the cyclic behaviour of connections. Global seismic performance.**
**Failure modes. Ductility demand. *q*-factors**
**Tests on subassemblages - full and partial strength connections,**
**Strain rate effects.**



# Steel. US Background.



**Following Northridge 1994: a strong push 200 million $ 10 years**

**Output**

**Material** — Now equivalent to EN 1090

**Design of connections** — for some time, 4 prescribed types or demonstration by tests
Now: open, requirements on plastic rotation capacity

**Execution of connections**
=> details
Cope hole
Reduced Beam sections
etc

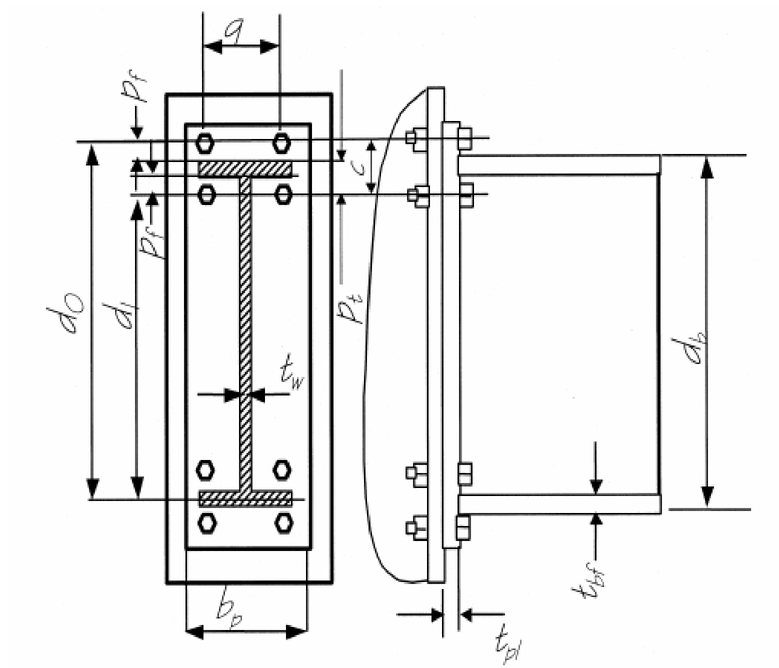
**Reduced beam sections RBS or "dogbones"**
- invented in Europe (1989)
- Improved in US

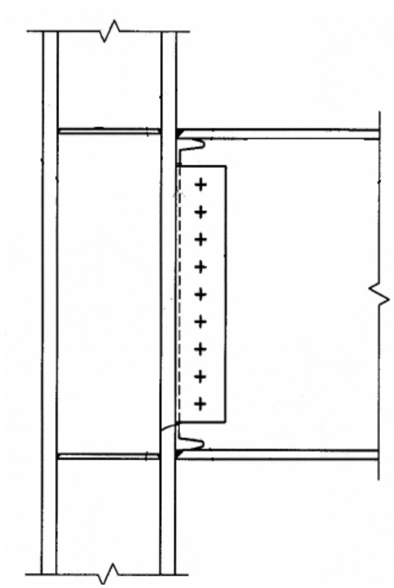
# Comments



**Steel connections damaged by hundreds as in 1994, Northridge earthquake: unlikely with Eurocodes 3 and 8 and European practice**

| <u>Europe</u> | <u>US pre-Northridge</u> |
|---|---|
| Weldability of base material | Weld&base material: low toughness |
| Required steel properties: toughness | welds “not for dynamic applications” |
| Welding process Europe: shop welds | site welding |
| Connection design: | |
| welded end plate at shop-bolts on site | mix of bolts & welds in 1 section |



# Composite. EU Background



## EUROPEAN RESEARCH ACTIVITY ON SEISMIC BEHAVIOUR OF COMPOSITE MOMENT RESISTING FRAMES

| | |
|---|---|
| 1. Definition of problems & Data bank of references | 1996-97 |
| 3. Definition of research needed | 1997 |
| 4. Testing & data processing activity<br>Numerical modelling activity. | 1997-2001 |
| 6. Code drafting activity. | 1998-2001 |

## Analysis of structures:

(1) neglecting concrete — How to disconnect and how far? Unsafe capacity design?

(2) considering concrete. Problems:

- ► evaluate effective width of slab in the elastic & plastic field?
- ► define conditions of ductility of sections?
- ► behaviour factors?
- ► layout of shear connectors?
- ► contribution of transverse beam to M transfer?
- ► partial strength connections?

# Composite. EU Background

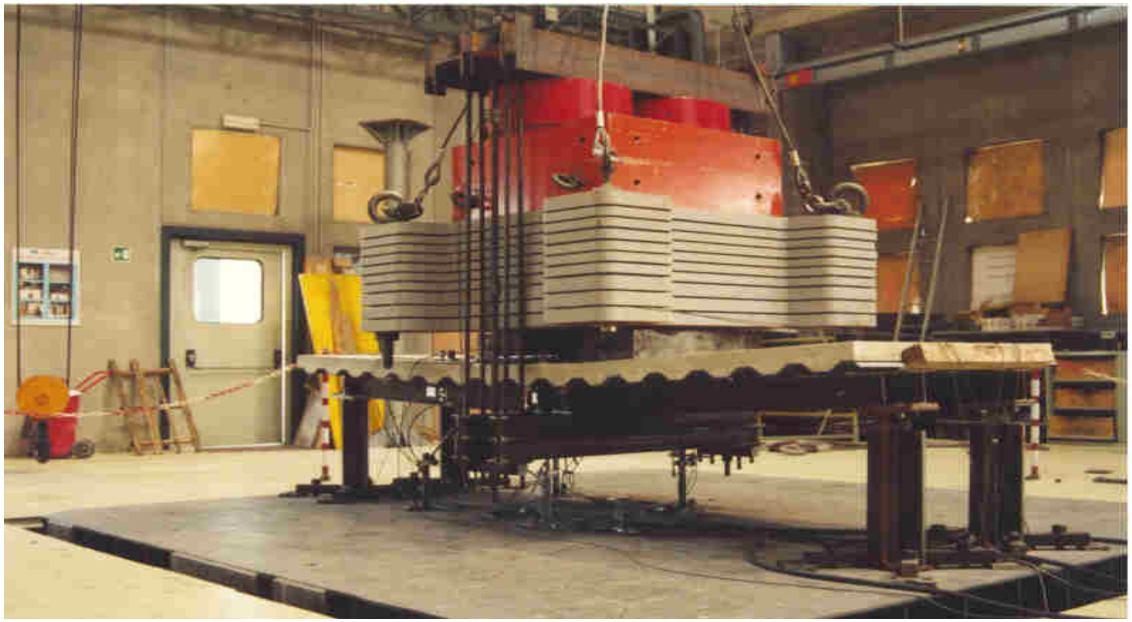


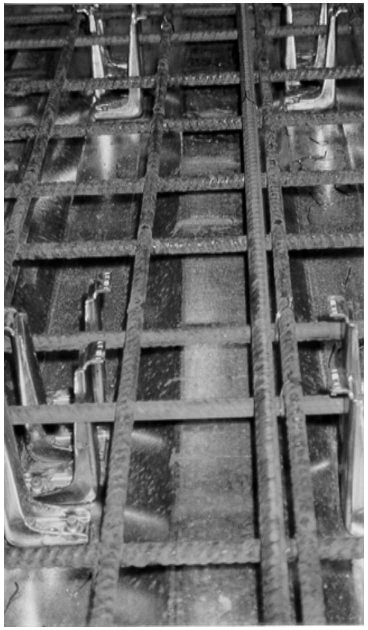
**Ductility in beam ends.**
**Where to yield ?**

- **steel members:** *M+ OK M- buckling*
- **concrete:** *Little, because degradation.*
- **connectors:** *No, low cycle fatigue.*
- **rebars:** *M- OK*
- **connections** *Component method needed*

**Shaking Table tests on simply supported beams (ISMES Bergamo)**

- **ductility of Composite sections**
- **ultimate Concrete strain in dynamic conditions.**





# SLAB DESIGN IN CONNECTION ZONE OF MOMENT FRAMES

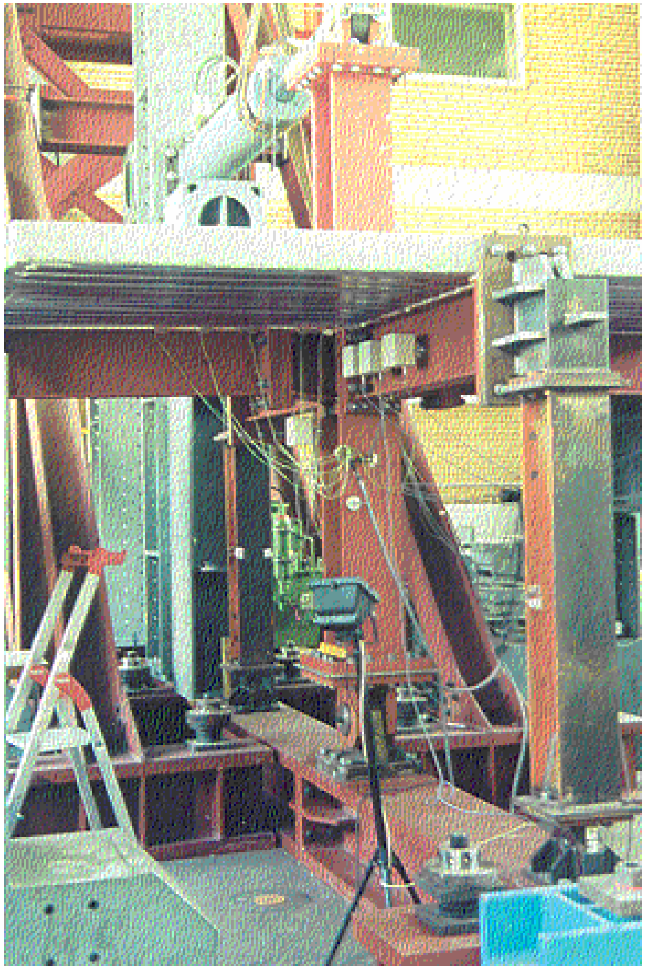
**Development of design approach for:**

► dimensions of T section (steel profile + slab)
► specific "seismic" re-bars
► shear connectors on beams
► effective width and cyclic behaviour

**Limited scope:**

► rigid connections
► ductility by yielding of steel profile

**Darmstadt tests on sub-assemblages**



**Problems studied.**

► Density of slab reinforcements
► Contribution of transverse beam
► Disconnection of concrete
► Steel deck waves directions

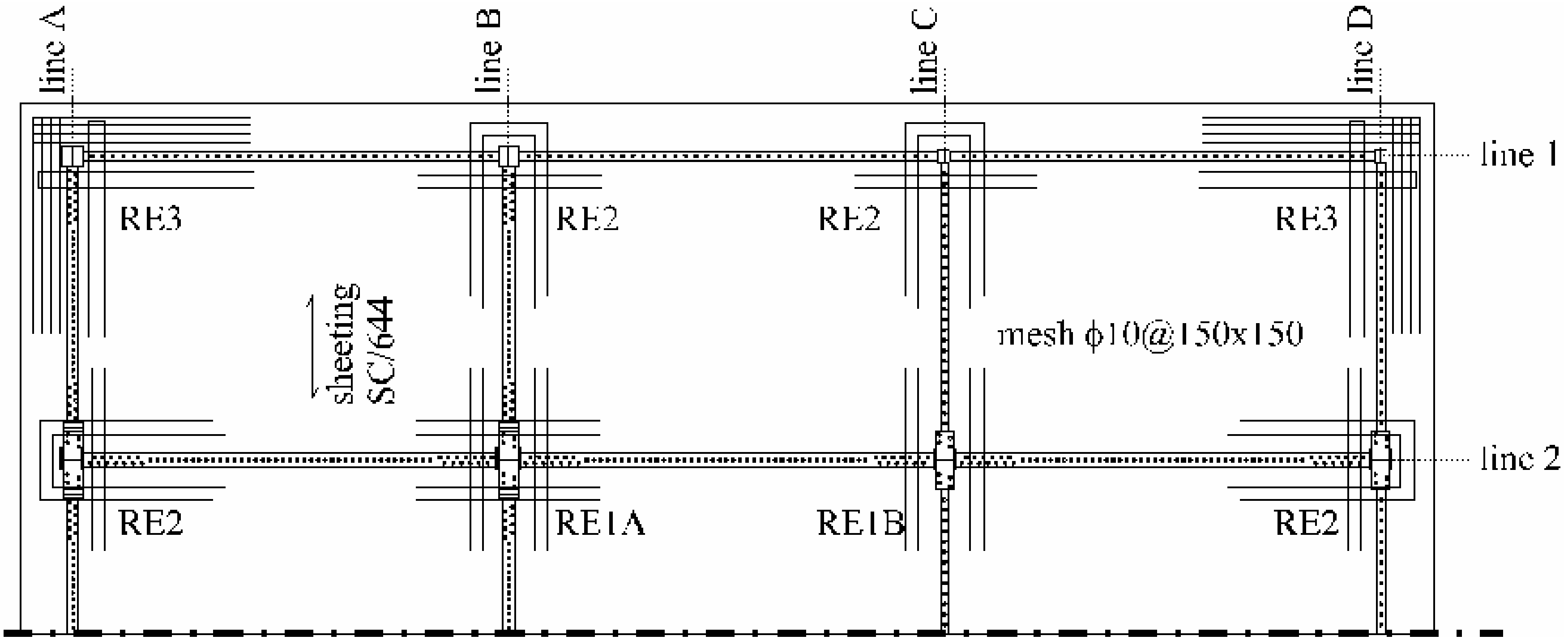
## Bi-directional cyclic response study of 3-D composite frame
## => European JRC ISPRA test

# Composite. EU Background



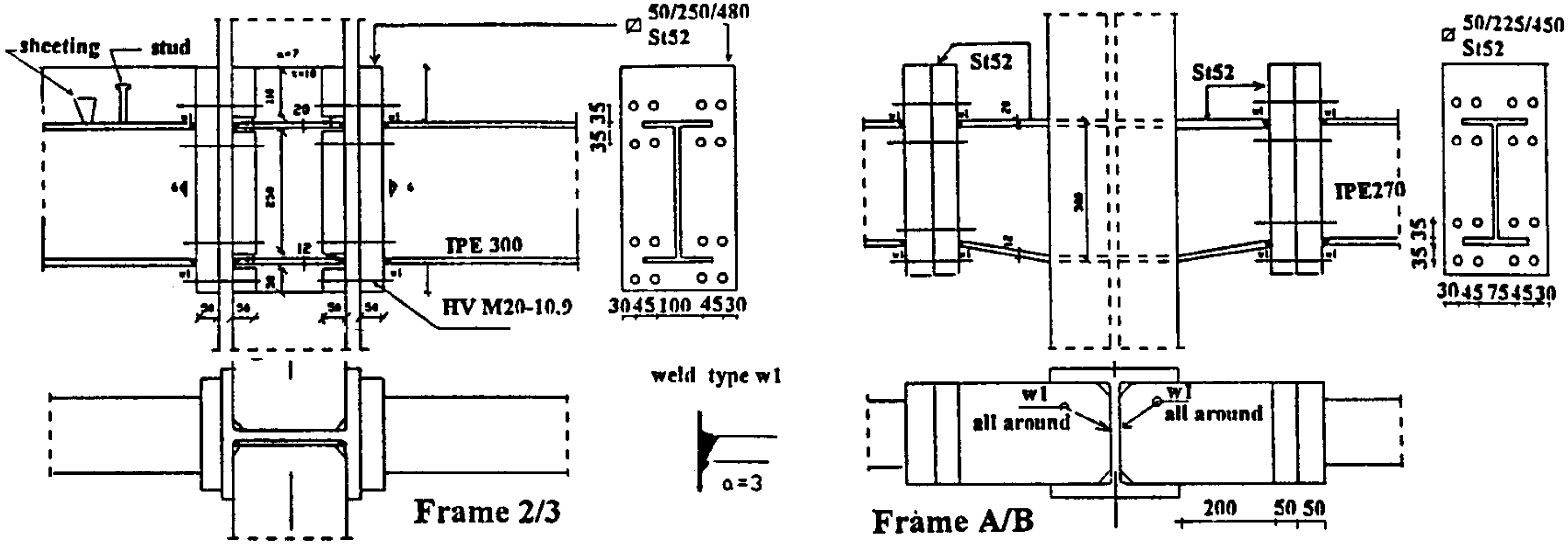
## A variety of design situations

### In the concrete slab



# Composite. EU Background






**Ispra test**
**Yielding, buckling and fracture of bottom flange**

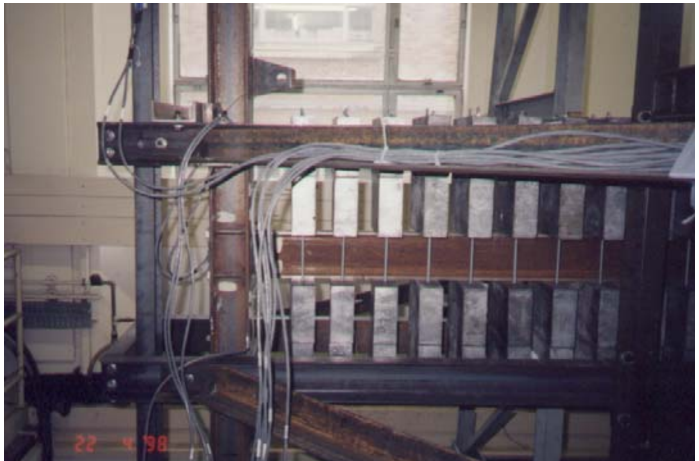
# Composite. EU Background



**Shaking table & cyclic static tests**
**on connections at NTUA.**
**Partial & full shear connection connection**

**Tests on shear connectors.** **Aribert-Lachal**
**Various loading histories**
**=>design resistance of connectors**

**Compared Experimental Assessment**
**of Steel and Composite Frames.**
**El Nashai& M.Tsuji**

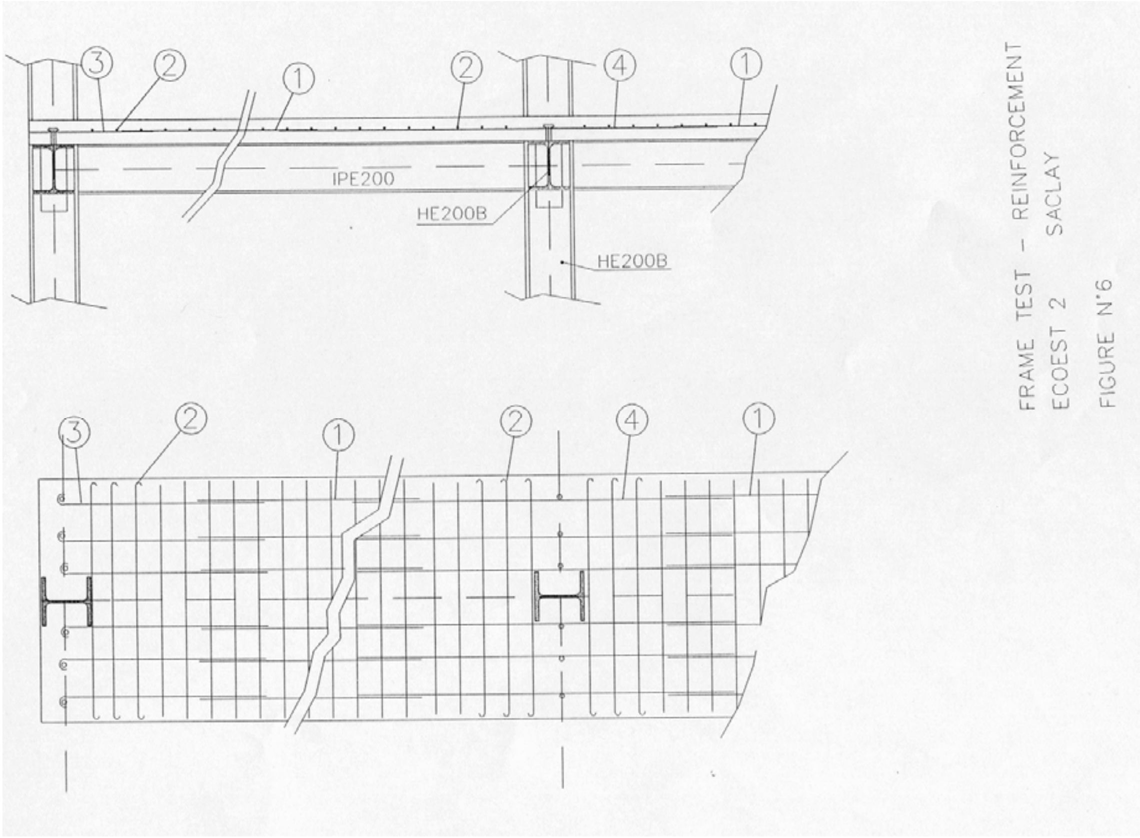


# Composite. EU Background



## Cyclic test on Composite moment frame ULg – CEA

- ► Redistribution of moments in beams
- ► Density of shear connectors (2 density )
- ► Slab design: reinforcements section and lay out
- ► Effective width for I and M p
- ► Low cycle fatigue of composite sections



# Composite. EU Background.



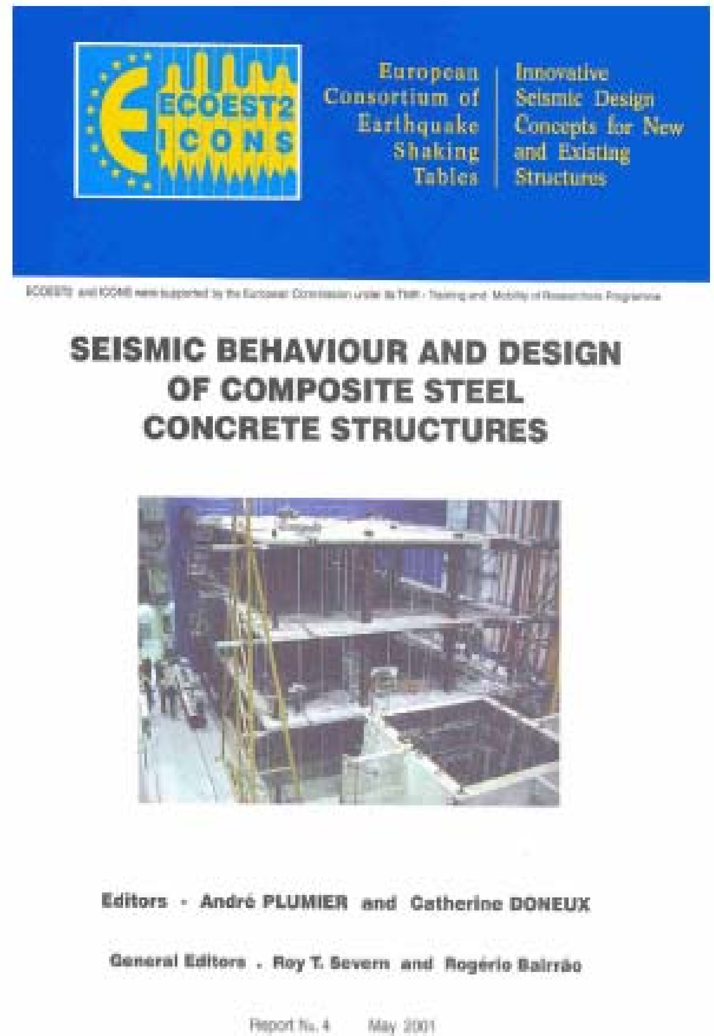
## Innovative Concepts in Seismic Design (ICONS) 1997 – 2000

**DG XII - European Commission.**
**JRC Ispra, U.ROMA , U.PAVIA ,**
**U.PATRAS, LNEC, POLI.MILANO,**
**GEO, INSA.LYON, ENS.CACHAN.**
**U.LIEGE, TH.DARMSTADT,**
**Imperial College, UP.MADRID.**

+ invited contribution using mobility funds:

**Trinity College Dublin**
**INSA de Rennes**
**University of Trento**
**University Federico II of Napoli**
**Politecnico di Milano.**

**ICONS Topic 4 Report =**
**Background document to Eurocode 8**
**on composite steel concrete structures**


ECOEST2 ICONS
European Consortium of Earthquake Shaking Tables
Innovative Seismic Design Concepts for New and Existing Structures

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

## 6.1 General

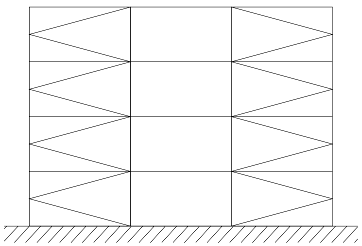
| Design Concepts | q | Ductility classes | |
|---|---|---|---|
| Non Dissipative Structures | $1 \leq q \leq 1{,}5$ | DCL | L for Low |
| Dissipative Structures | $1{,}5 < q < 4$ | DCM | M for Medium |
| Dissipative structures | $q \geq 4$ | DCH | H for High |

**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

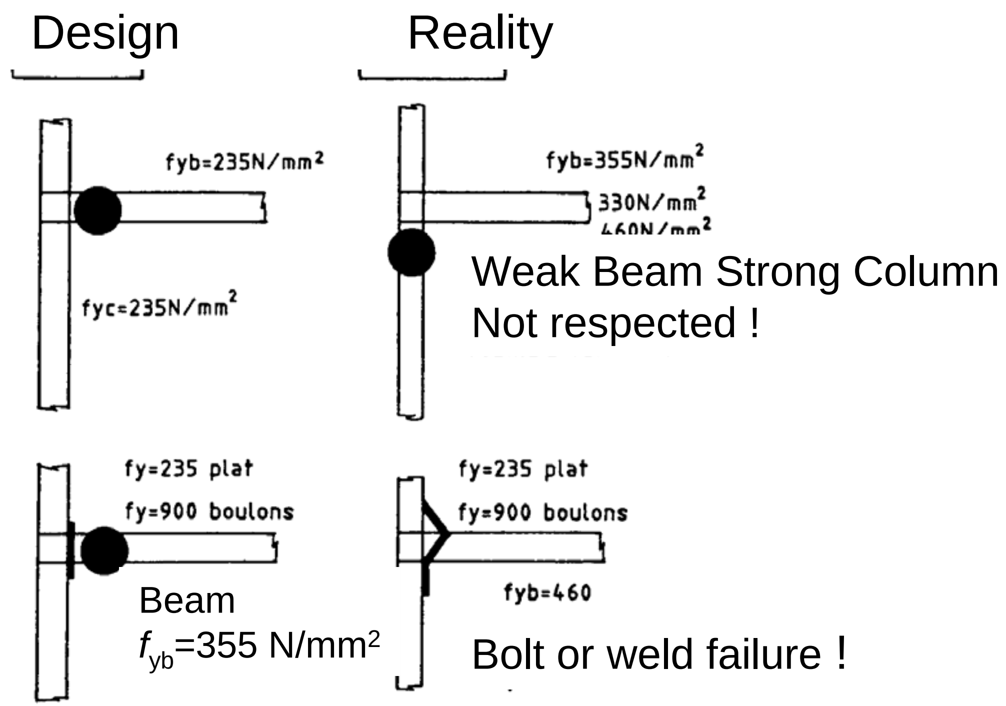


# Required steel characteristics



- **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***

# 6.2 Material



**Conditions on *fy* 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: $f_{y,max} < 1,1\ \gamma_{ov}\ f_y$

$\gamma_{ov}$ material overstrength factor $f_y$ : nominal

Ex: S235, $\gamma_{ov}$ = [1,25 ] => $f_{y,max}$ = 323 N/mm2

an upper yield strength is specified for dissipative zones

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

for both dissipative and non dissipative zones

- use nominal $f_y$ for dissipative zones

with higher value $f_{y,max}$ specified for dissipative zones;

- use higher 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

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

# 6.12 Control of design and construction



**Bolts** 8.8 ou 10.9 preloaded EN 1090

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

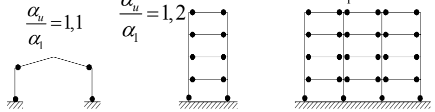
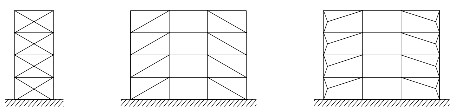
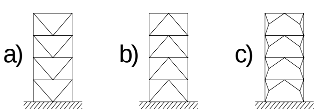
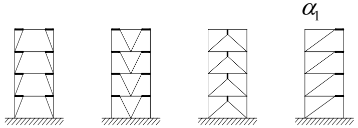
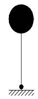
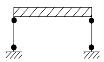
▲ Tightening of bolts to EN 1090

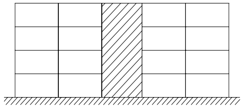
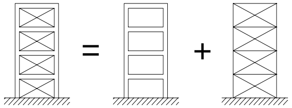
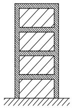
▲ No structural changes involving a variation in stiffness or strength of more than 10 % of the values assumed in design

▲ If not, appropriate corrections or justifications

# Structural types & maximum behaviour factors $q$



| | 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. a) b) c)  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 |

# 6.5.2 General Criteria for Dissipative Structural Behaviour



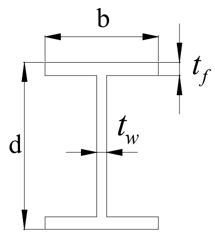
▲Dissipative zones: adequate ductility and resistance

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

<u>Elements in Compression or Bending</u>

| 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, $R_{fy}$ computed with "appropriate estimation $f_{yd}$ of the actual value of the yield strength". "appropriate" was a problem***

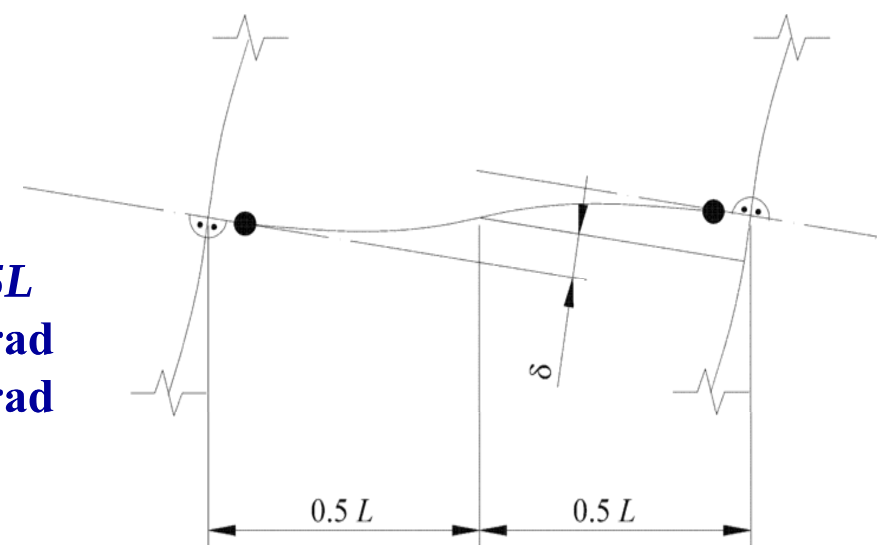
**(6) The adequacy of design should be supported by experimental evidence …to conform with requirements defined… for each structural type and ductility class.**

**Example: moment resisting frames**

**plastic rotation capacity** $\theta_p = \delta / 0{,}5L$

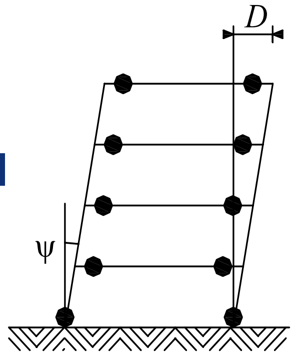
**ductility class DCH :** $\theta_p \geq 35$ **mrad**

**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**

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

*$V_{Ed}$ :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**

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

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

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

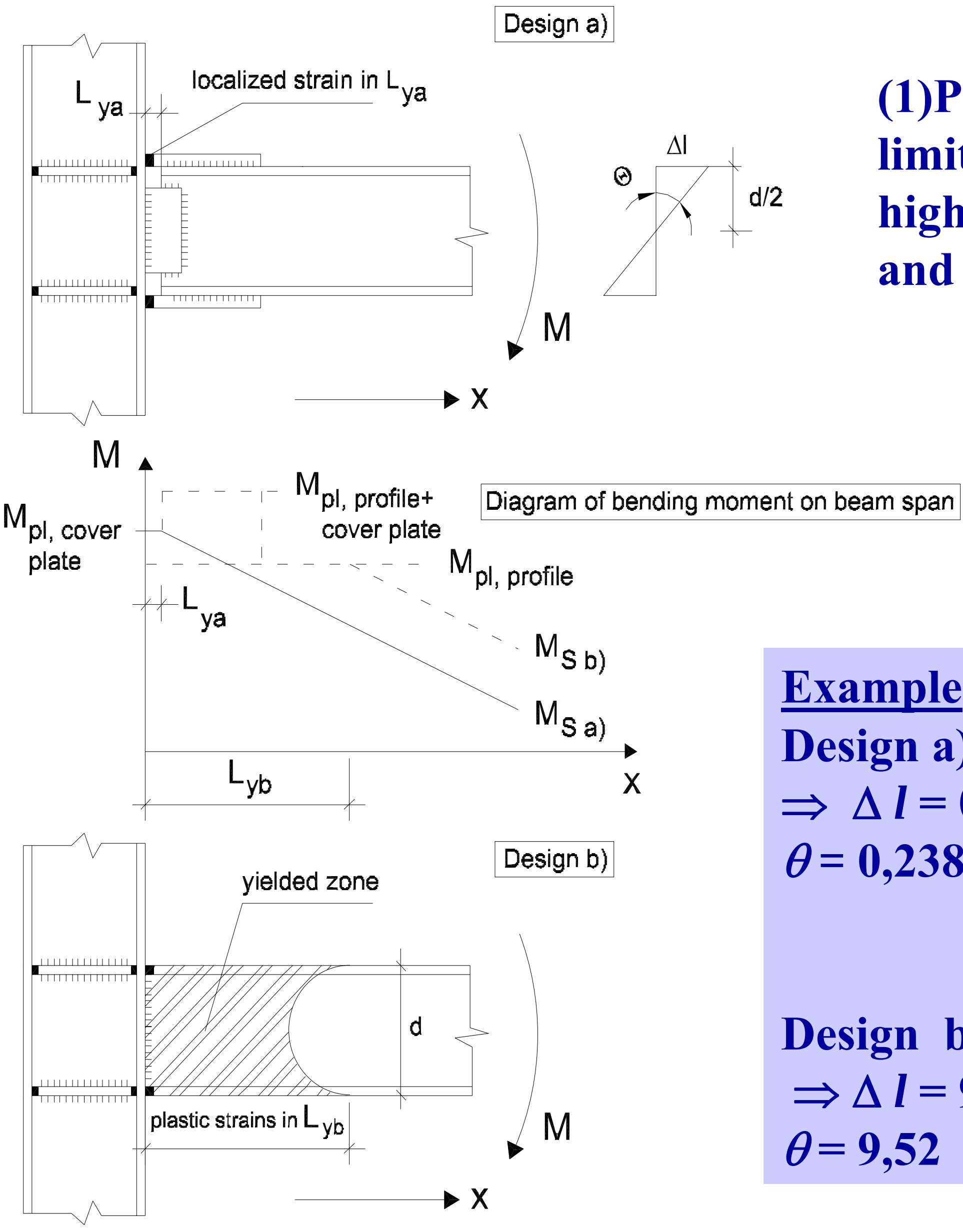
*$\Omega$ minimum section overstrength $\Omega_i = M_{pl,Rd,i}/M_{Ed,i}$ of all beams dissipative zones*

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

M*pl,Rd,i. plastic moment*

# 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.**

**<u>Example</u>**

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

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

$\theta = 0{,}238 / (400/2) = 1{,}2$ mrad <<< 25 mrad

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

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

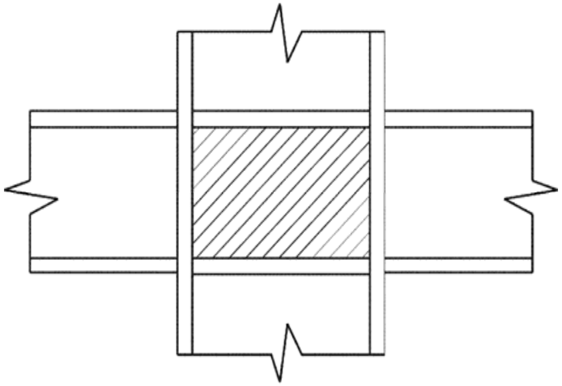
$\theta = 9{,}52 / (400/2) = 47{,}6$ mrad >> 35 mrad

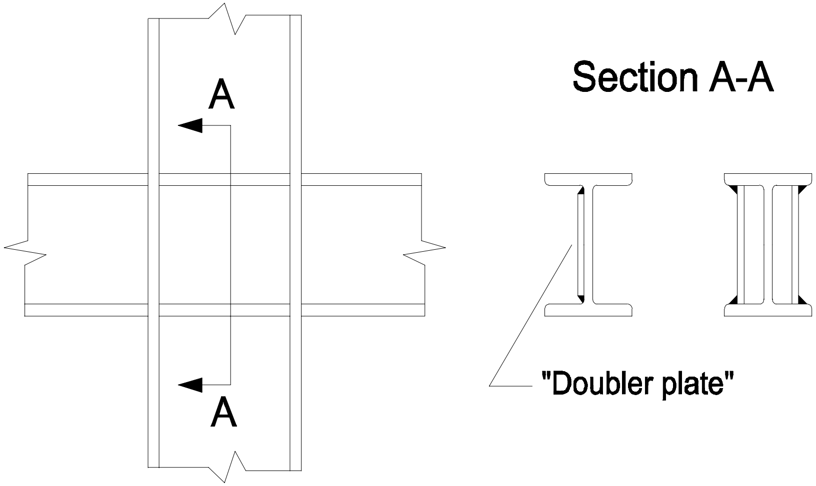
# Rules for connections in dissipative zones



## Connection design detail ⇔ Ductility classes: National Annexes

## Shear resistance of framed web panels





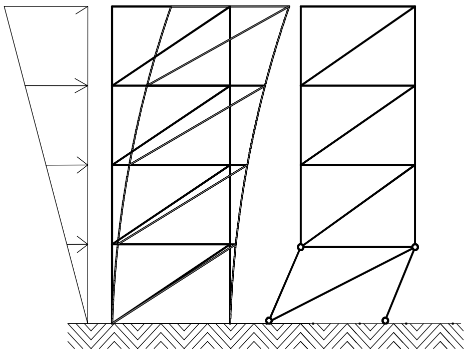

$$\frac{V_{wp,Ed}}{V_{wp,Rd}} \leq 1,0$$

$V_{wp,Rd}$ 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**

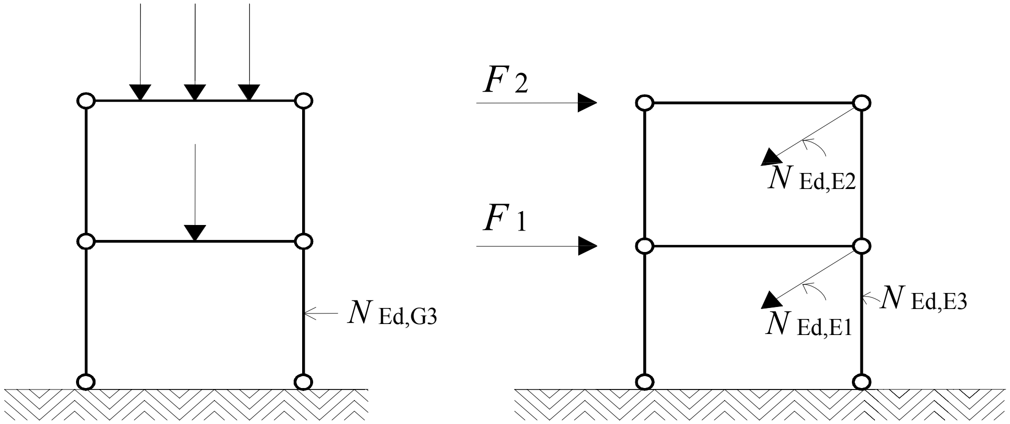
**+non linear analysis**

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

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

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

**Beams &columns**




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

**section overstrength of diagonal**

**Homogeneous dissipative behaviour:**

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

# 6.7 Frames with concentric bracings. (Continued)



## ▲Frames with V or Λ bracings

**Dissipative elements: diagonals in tension & compression**

**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 $\overline{\lambda} \leq 2{,}0$

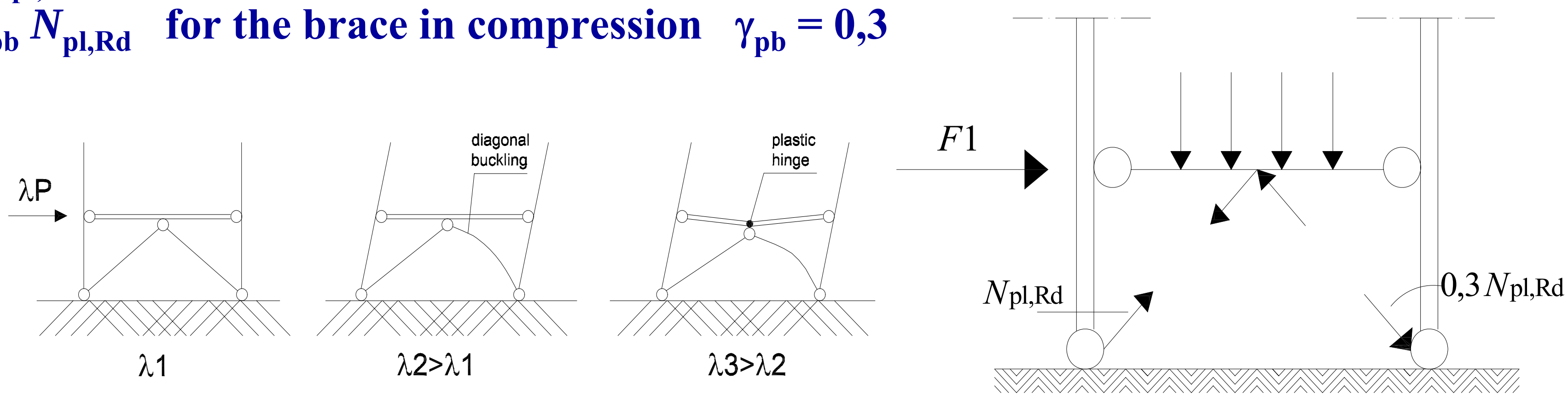
**Beams and columns** $N_{pl,Rd}(M_{Ed}) \geq N_{Ed,G} + 1{,}1\gamma_{ov}\,\Omega . N_{Ed,E}$

**Capacity design to diagonals** $\Omega$ **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, calculated using:**

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

$\gamma_{pb}\,N_{pl,Rd}$ **for the brace in compression** $\gamma_{pb} = 0{,}3$



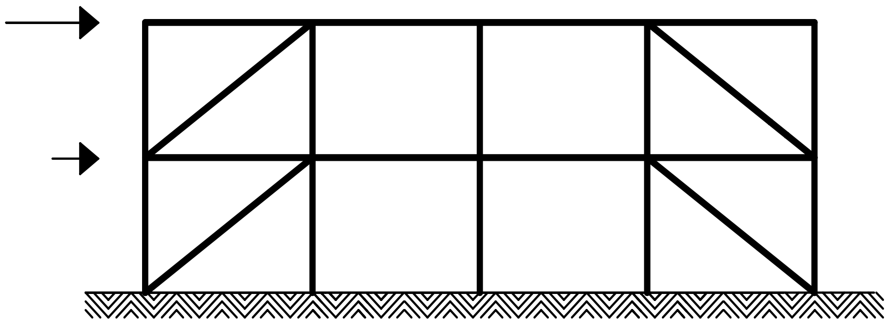
# 6.7 Frames with concentric bracings. Continued.



▲ **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**

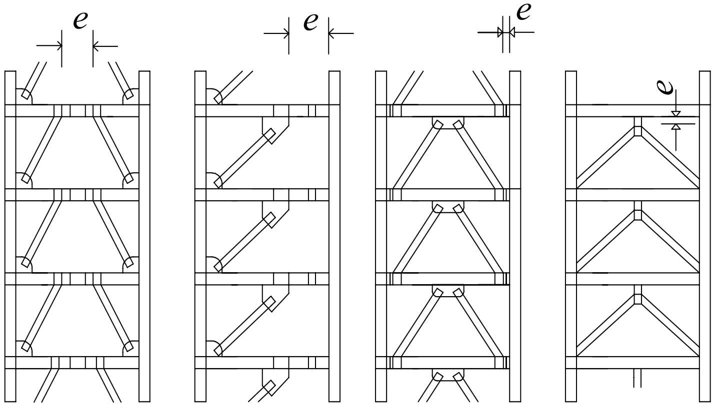
**Limitation:** $\bar{\lambda} \leq 2$



EUROCODES
Background and Applications

# 6.8 Frames with eccentric bracings

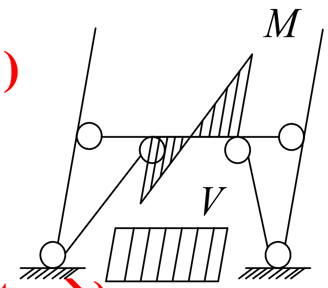





**Elements called “seismic links” designed to dissipate energy**

**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-→)**

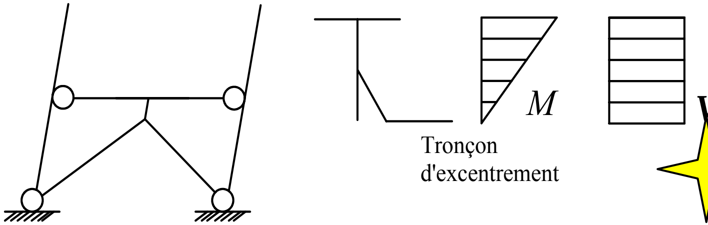
short links $e < 1{,}6\ M_{p,link}/V_{p,link}$

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

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

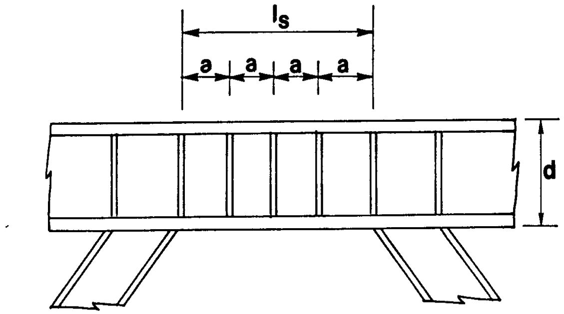
short links $e_s < 0{,}8\ M_{p,link} / V_{p,link}$

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



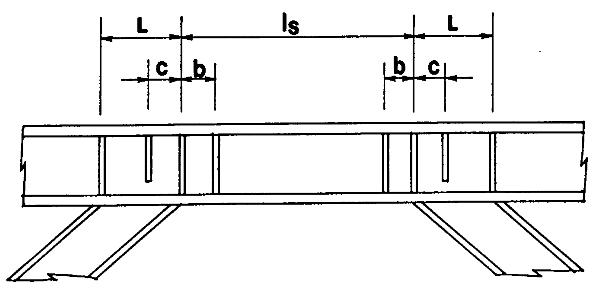
# 6.8 Frames with eccentric bracings. Continued.






**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**

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

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

# Other types of steel structures



## 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.**

# 3. EC 8 Section 7: 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

# Steel Concrete Structures. Materials & Structural Types



## 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: C20/25 < $f_c$ < C40/50 => C30/35**

**Rebars: 2 classes (ductile-non ductile)**

**$f_u / f_y$ A%**

## 7.3 Structural types

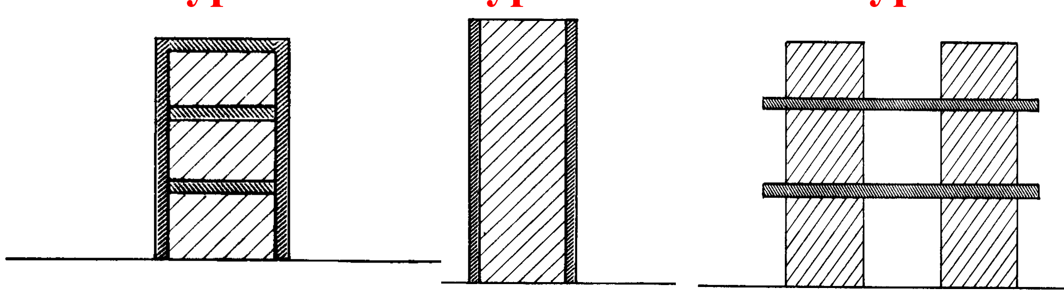
**Moment resisting frames.** Beams & columns: steel or composite

**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 1 Type 2 Type 3



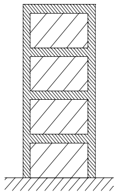
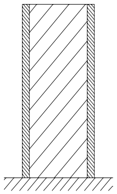
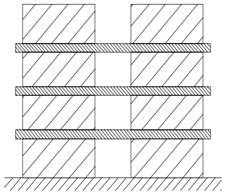
# Behaviour factors *q*



**- Composite MRF's**

**Braced frames: *q* steel = *q* composite**

**- Wall systems: Table**

| | DCH | DCM |
|---|---|---|
| e) Reinforced concrete shear wall elements. $\frac{\alpha_u}{\alpha_1} \approx 1.1$<br>TYPE I  Steel or composite moment frame with concrete infill panels.<br>TYPE 2  Concrete walls reinforced by encased vertical steel sections.<br>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. $\frac{\alpha_u}{\alpha_1} \approx 1.2$ | $4\frac{\alpha_u}{\alpha_1}$ | $2.5\frac{\alpha_u}{\alpha_1}$ |

# Analysis & Design. General



## 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).

# Composite connections in dissipative zones



**Design objective: integrity of concrete, yielding in steel**

**-Dissipative connections allowed**

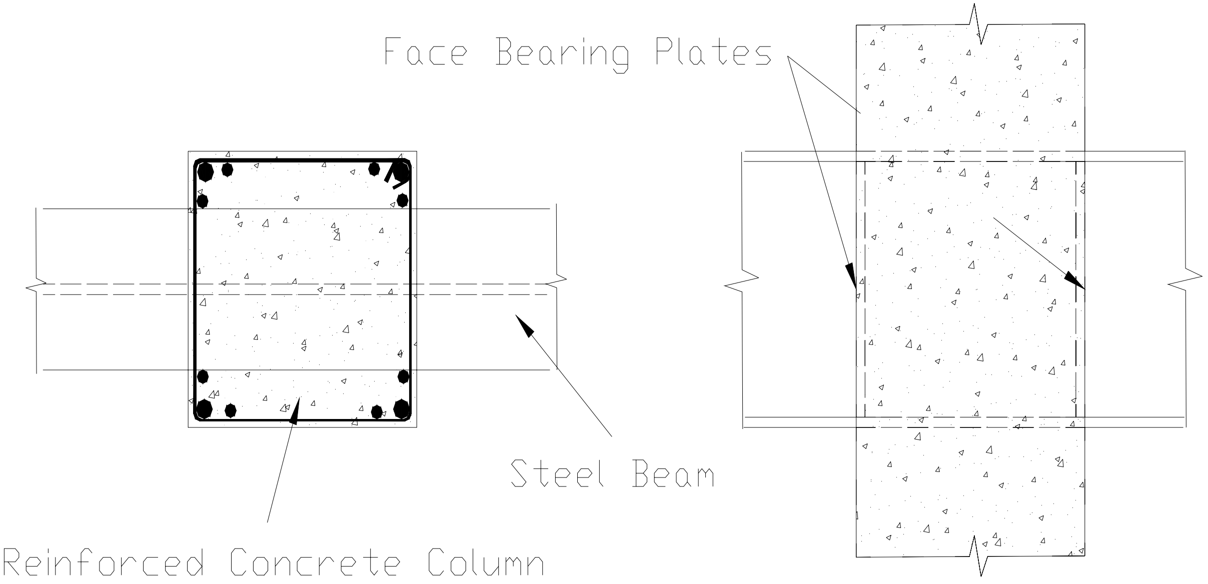
**-Rebars sections in joint region: models satisfying equilibrium**

**-Yielding of rebars allowed**

**-In fully encased framed web panels of beam/column connections**

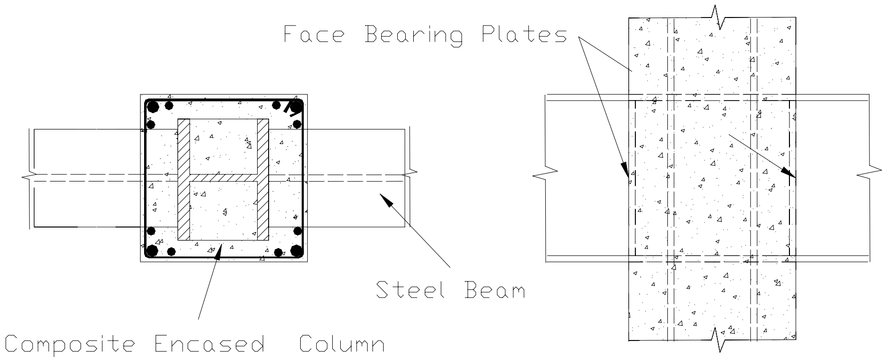
**-Panel zone resistance = Σ concrete & steel shear panel resistance**

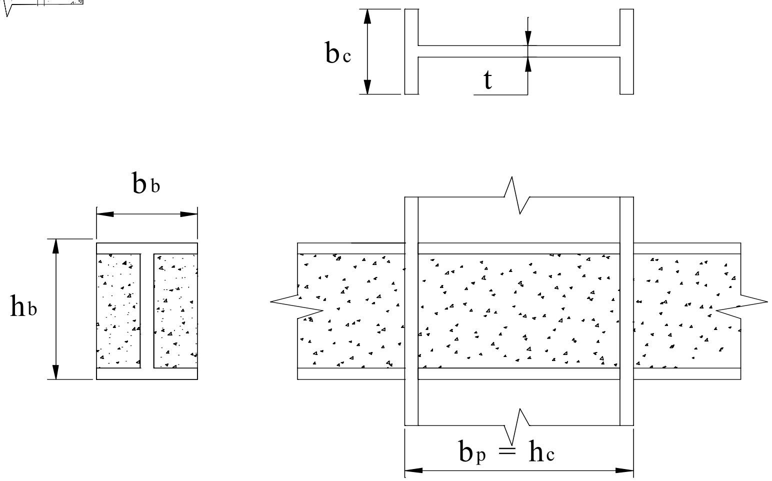
**aspect ratio $h_b/b_p$ of the panel satisfies conditions**



# Composite connections in dissipative zones





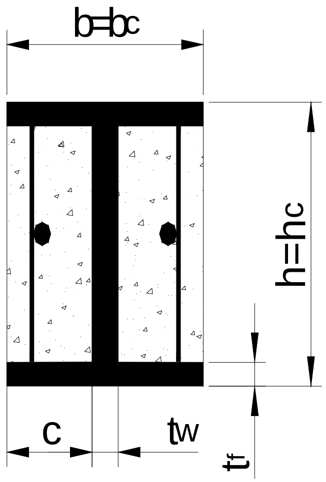




**In partially encased stiffened web panels: similar Σ, additional conditions**

**Vertical rebars to take beam shear force**
**If composite column, distribute beam shear between steel and concrete**

# Members & classes of Ductility






## 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 $\varepsilon = (f_y/235)0.5$ | 80 $\varepsilon^2$ | 85 $\varepsilon^2$ | 90 $\varepsilon^2$ |

# 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**

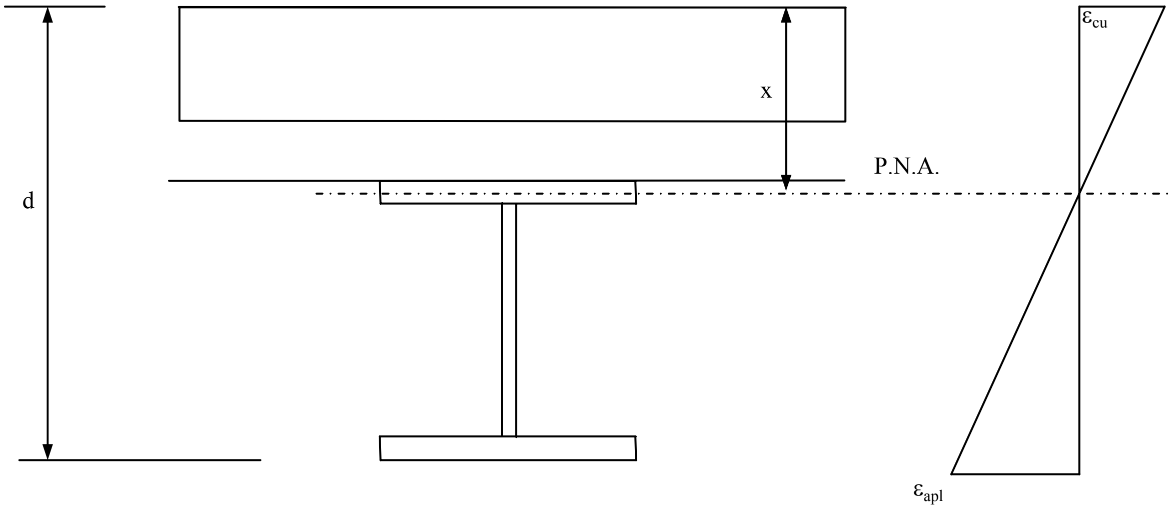
# Steel beams with slab



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

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

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




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

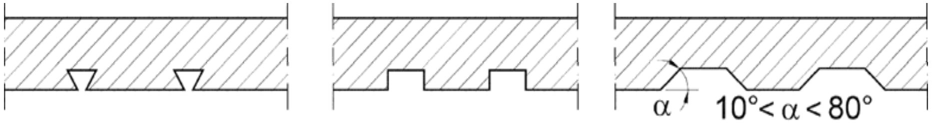
| Ductility class | $q$ | $f_y$ | $x/d$ upper limit |
|---|---|---|---|
| DCH | $q \geq 4$ | 355 | 0,19 |
| DCH | $q \geq 4$ | 235 | 0,26 |
| DCM | $1{,}5 < q < 4$ | 355 | 0,26 |
| DCM | $1{,}5 < q < 4$ | 235 | 0,35 |

# 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**

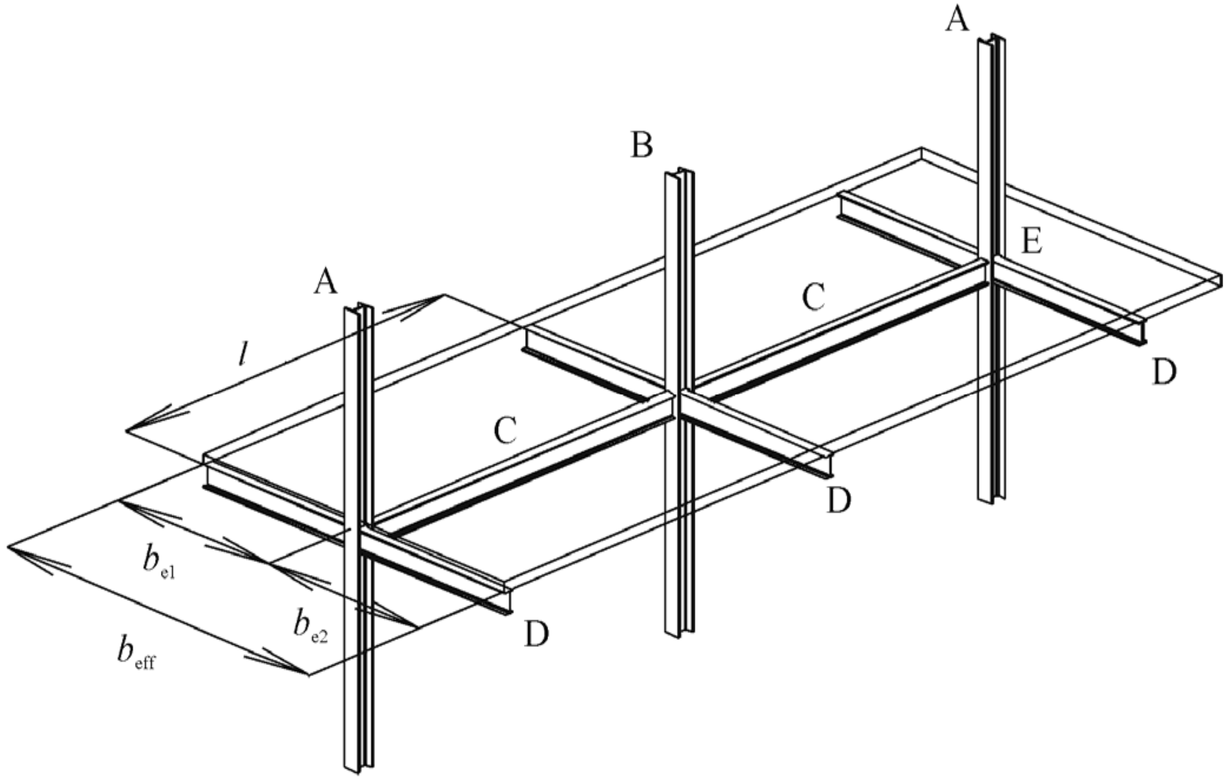



***kr* = 1** ***kr* = 1** ***kr* = 0,8**

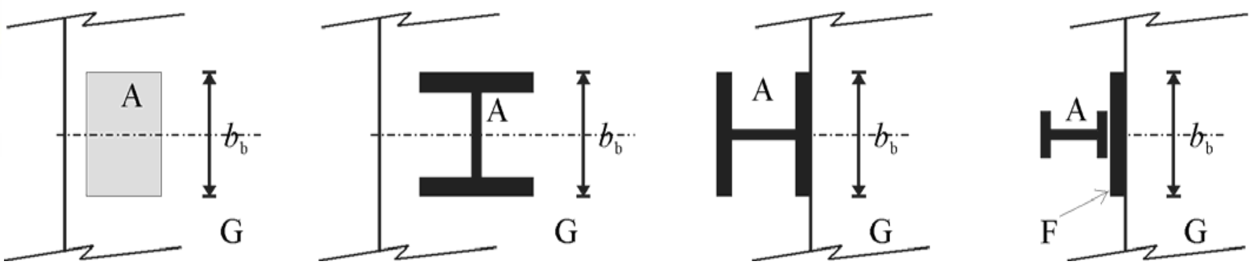
**-Full shear connection required with non ductile connectors**

# Steel beams with slab






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



# Effective width $b_{eff}$



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

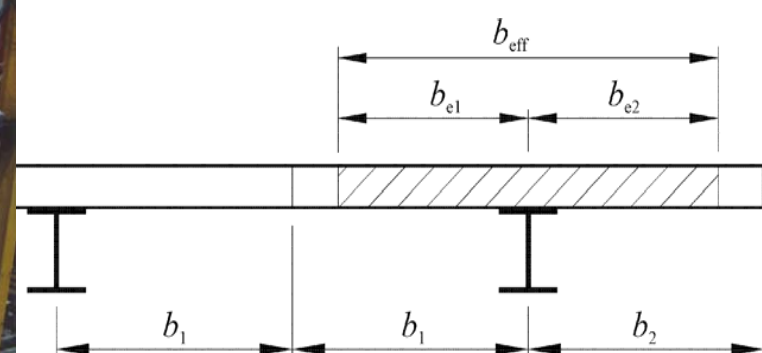
**Effective width $b_{eff}$**

$b_{eff} = 2\ b_e$

$b_{eff} \neq$

for $I$ elastic analysis

$M_{pl}$ plastic resistance



# Slab « seismic » rebars

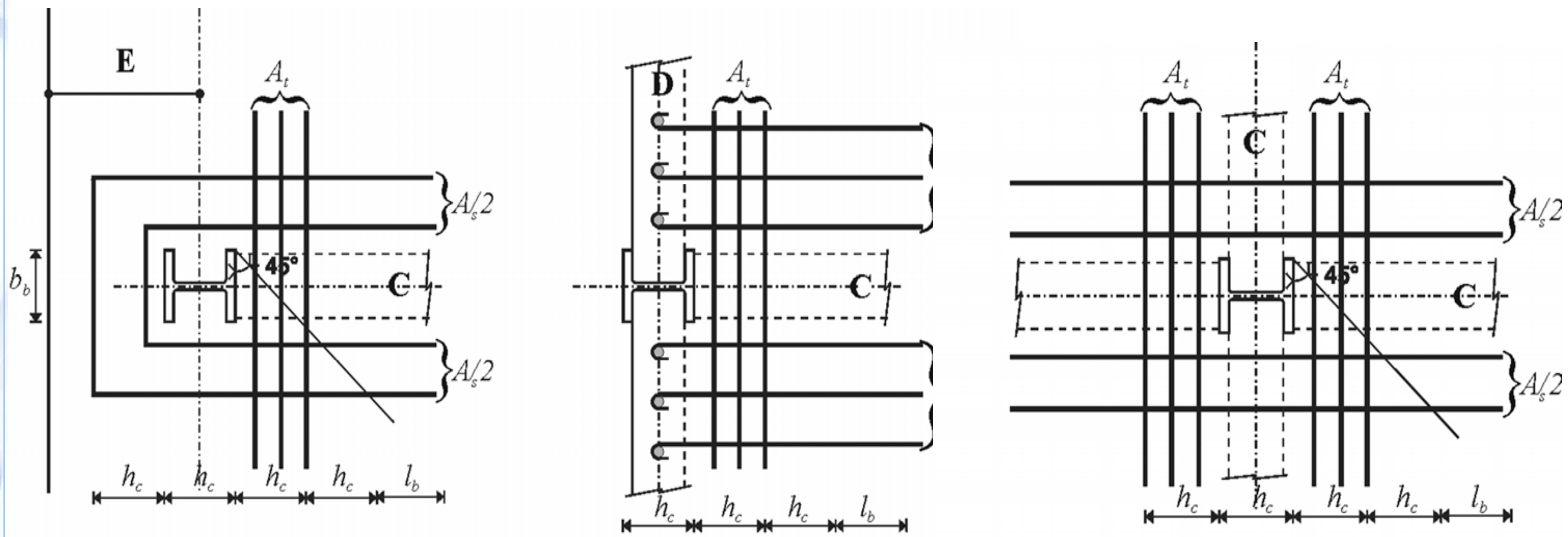


**Moment Resisting Frames**

**Dissipative zones in beam with slab: vicinity of columns**

**“Seismic rebars” needed**

**Section and layout to achieve ductility => Annex C**



# Moment Resisting Frames. Analysis.



**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



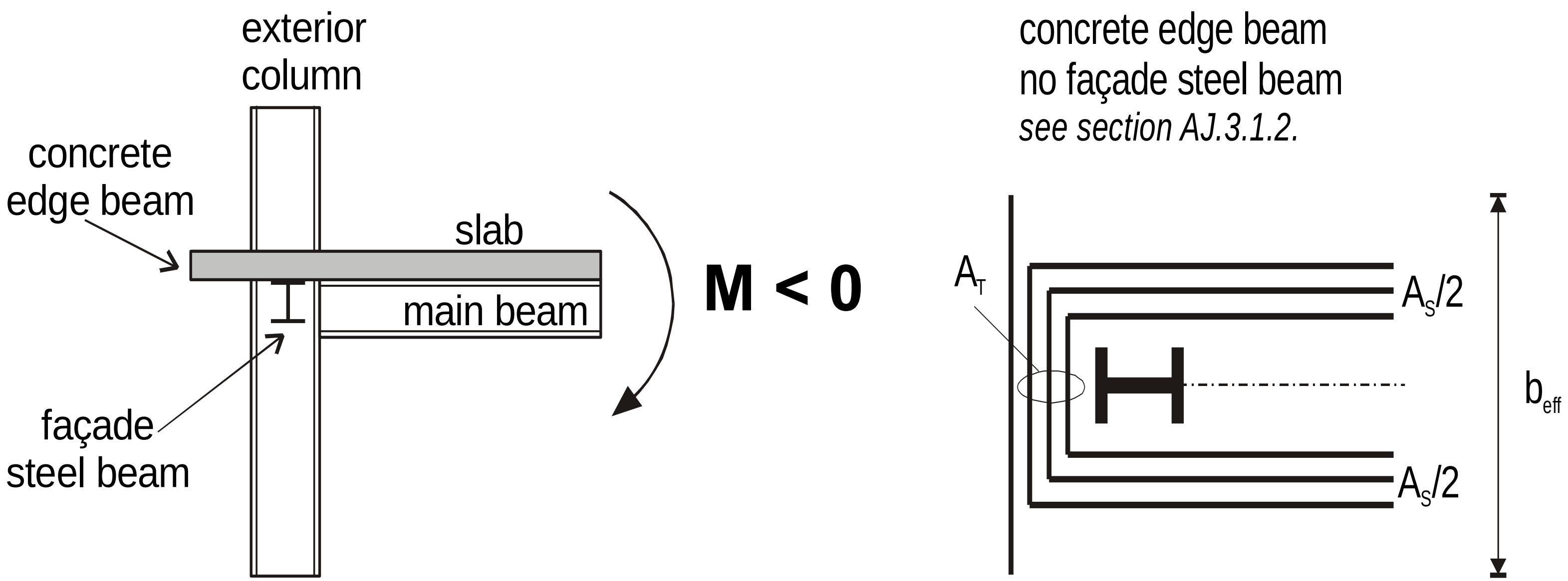
## SEISMIC DESIGN OF THE SLAB REINFORCEMENTS OF COMPOSITE T BEAMS WITH SLAB IN MOMENT FRAMES

**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)

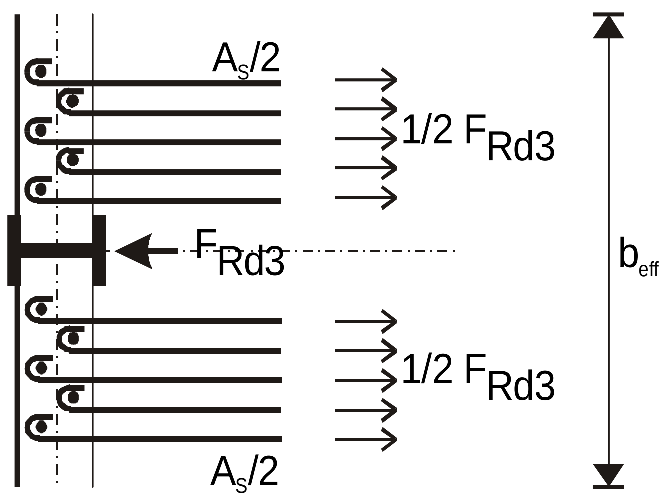
=> 2 limits of section $A_S$ of reinforcement in the slab

**EC4: negative moment & no transverse steel beam**

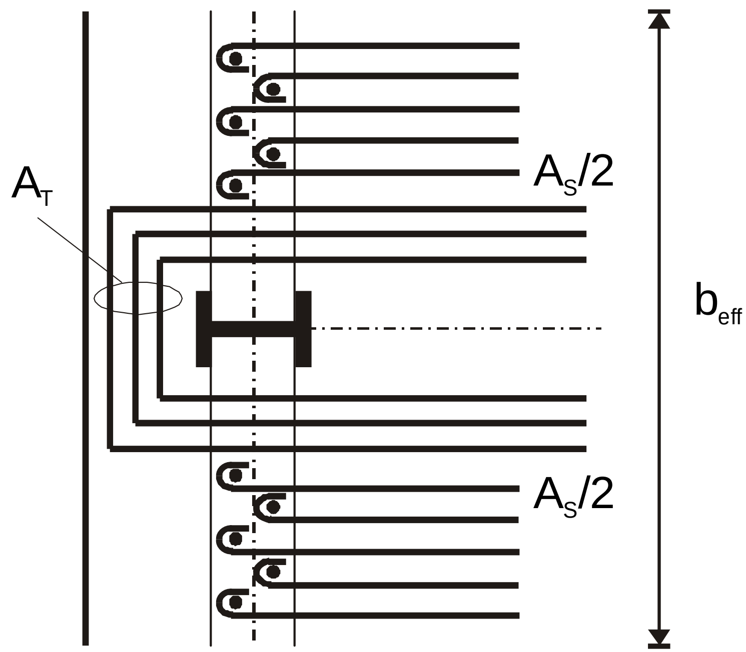


## M<0 Exterior Column Case

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.*




$A_S = F_{Rd3}/f_{yd}$ $F_{Rd3} = n \times P_{Rd}$

$n$ = number of connectors in effective width

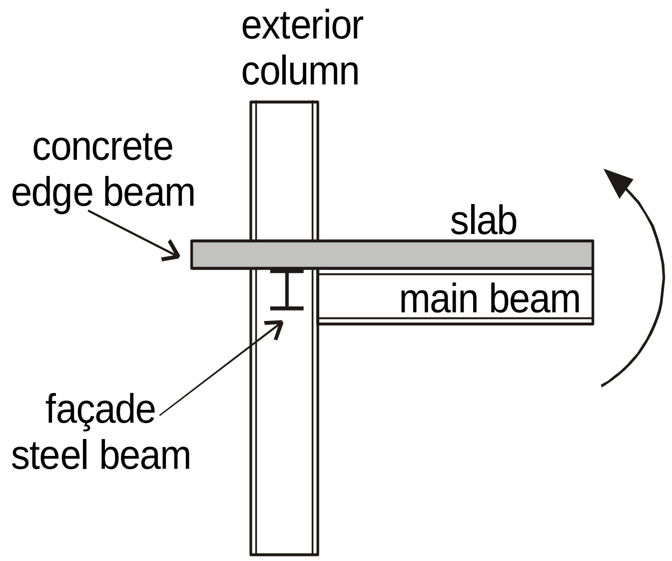
$P_{Rd}$ = design resistance of 1 connector

**Façade beam to check in bending**
**shear**
**torsion**

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

# ANNEX C






**M > 0**

## Exterior Column Case

**3 Force Transfer Mechanisms of Slab Compression**

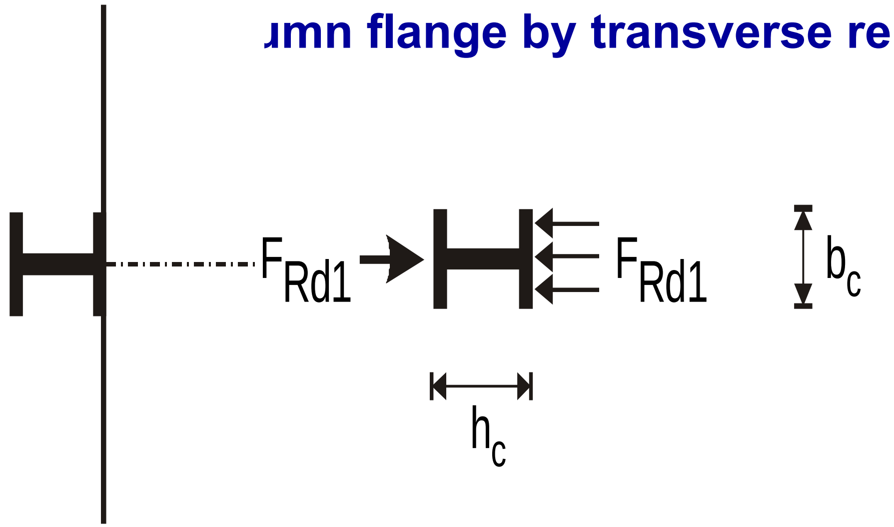
### Mechanism 1

**Direct compression on column**

$F_{Rd1} = b_c \, d_{eff} \, f_{cd}$

**=>Confinement of concrete close to**
**umn flange by transverse re-bars**

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

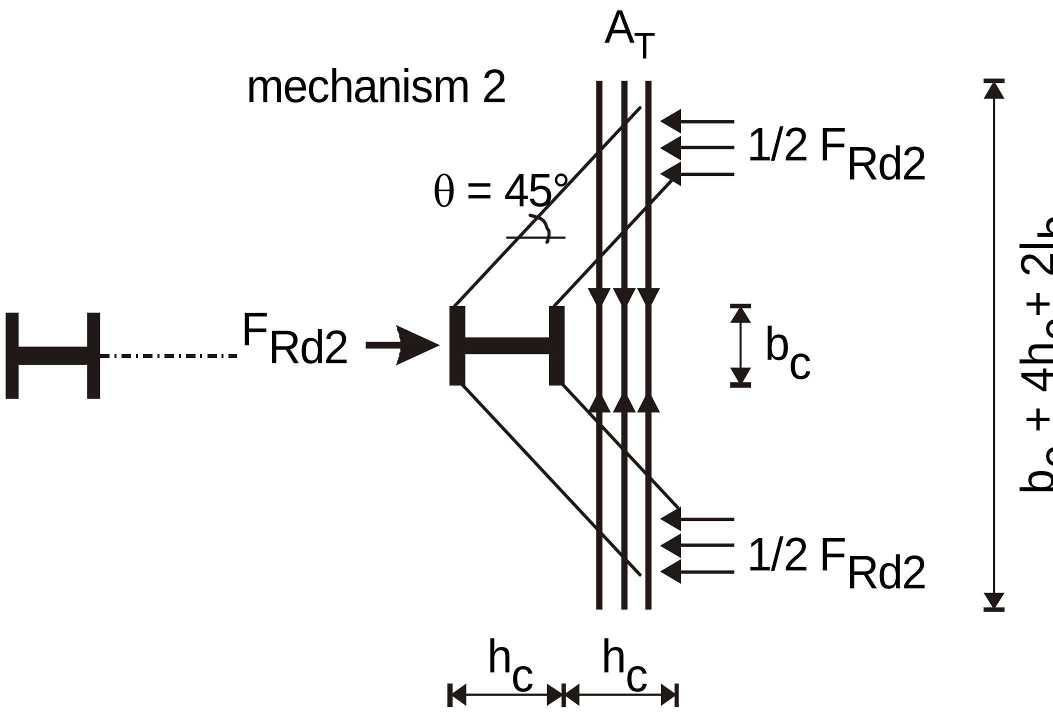


# ANNEX C



## 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.*




$$A_T \geq \frac{F_{Rd2}}{f_{yd,T}}$$

$F_{Rd2} = 0.7\, h_c\, d_{eff}\, f_{cd}$

$d_{eff}$ : effective slab thickness

# ANNEX C

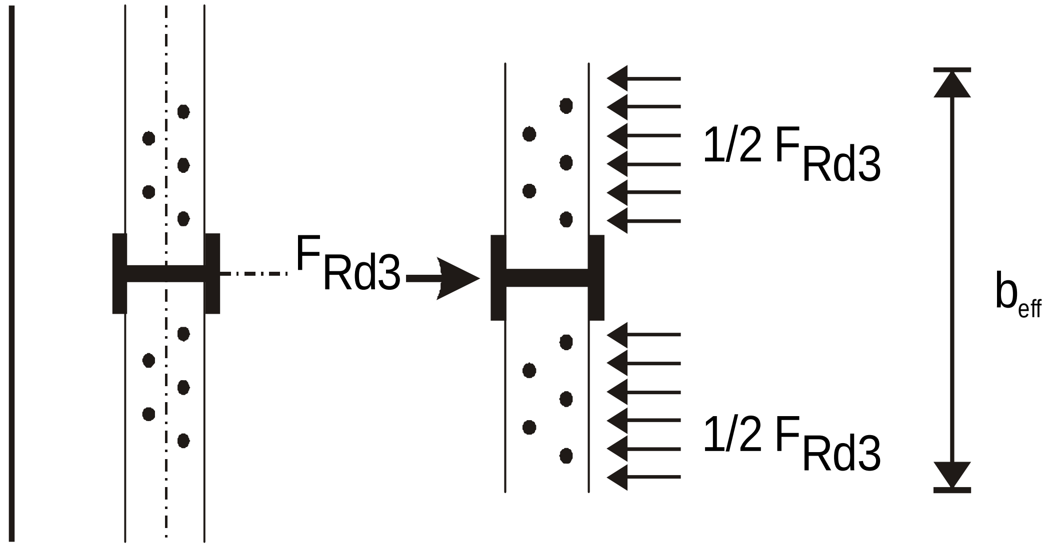


## Mechanism 3

**Compression on connectors of facade steel beam**

$F_{Rd3} = n \times P_{Rd}$ — n = number of connectors in effective width; $P_{Rd}$ = design resistance of one connector

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



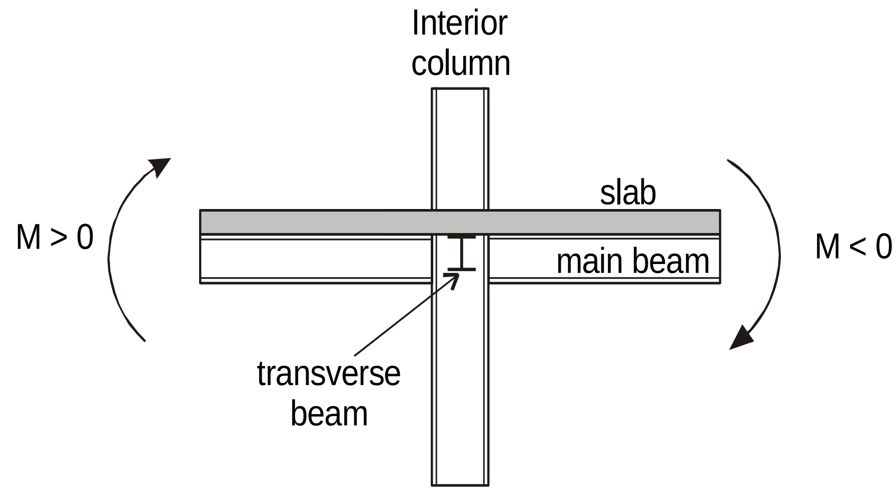

**Maximum compression force** $b_{eff}\, d_{eff}\, f_{cd}$

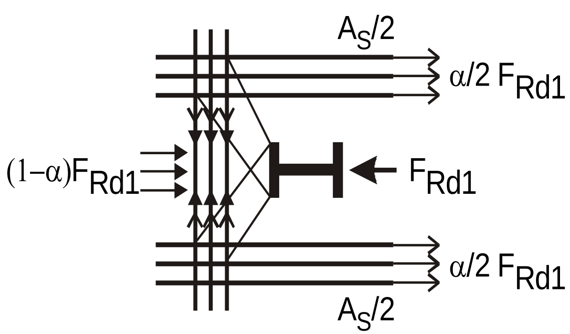
**Transmitted if:** $F_{Rd1} + F_{Rd2} + F_{Rd3} > b_{eff}\, d_{eff}\, f_{cd}$

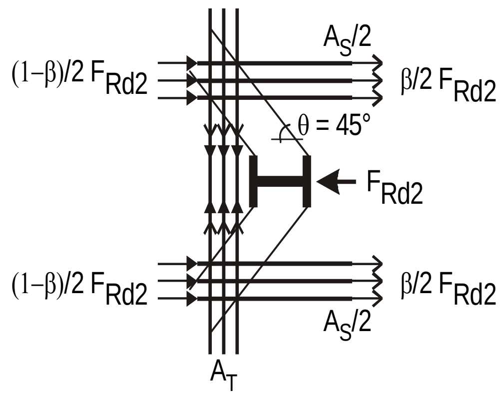
**=> choose n to achieve adequate $F_{Rd3}$**

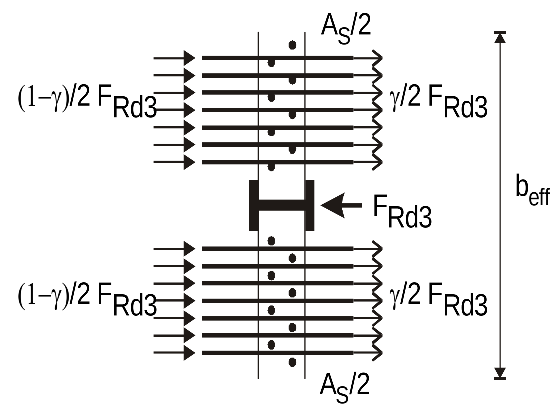
# ANNEX C

## Interior Column Case




**Mechanism 1**

**direct compression on the column hypothesis: $A_T = A_S / 2$**

**Mechanism 2**

**Compressed concrete struts**

**Mechanism 3**

**Connectors on transverse beams**

# Composite Braced Frames



## 7.8 Composite concentrically braced frames

Concepts

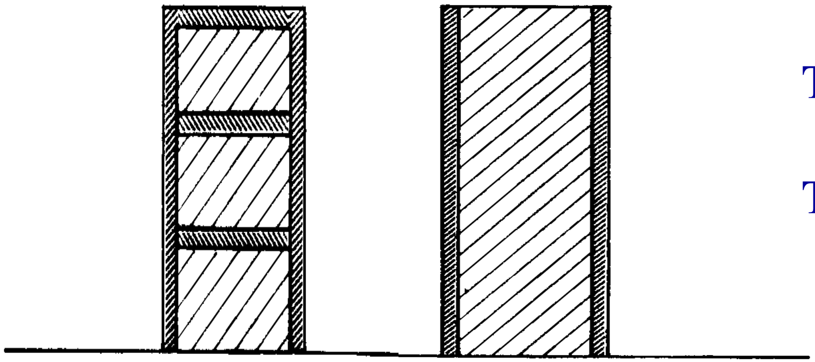
- Yielding of diagonals in tension
- Tension only design & no composite braces

## 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** **Type 2**

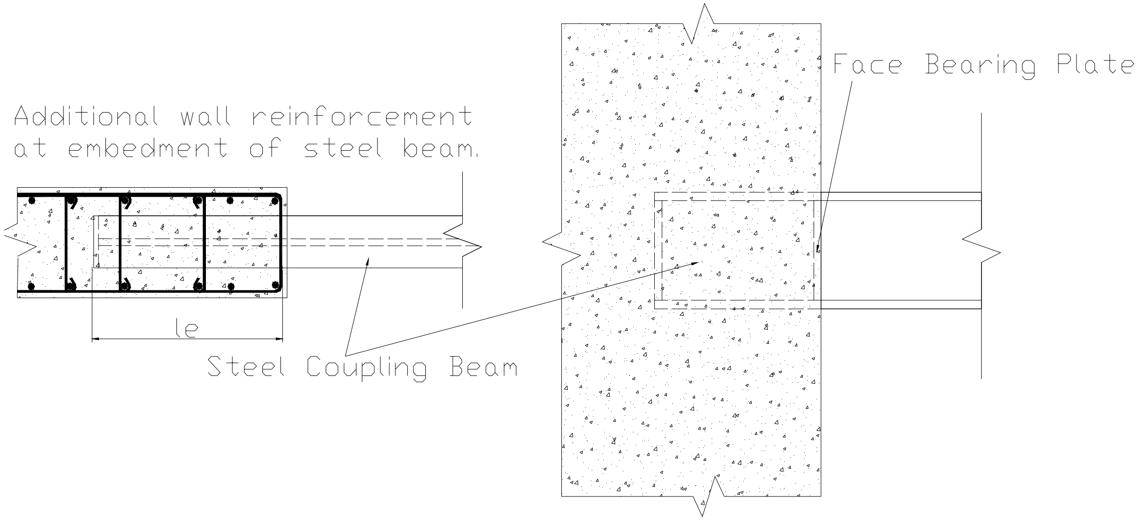
**walls with plastic hinge at base**
**vertical encased steel = reinforcements for bending**

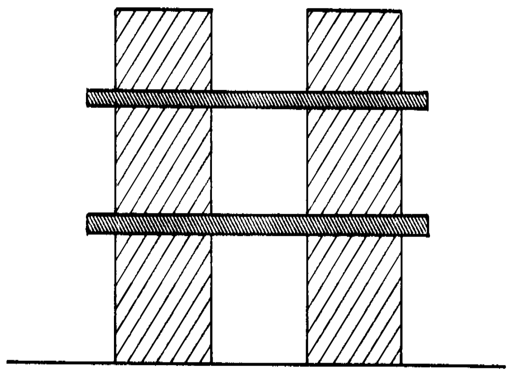
**Shear carried by the reinforced concrete wall**
**Gravity and overturning moment carried by the wall acting composedly with the vertical boundary members**

# Composite Systems



**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.**

**BOOKS on Seismic Design closely related to Eurocode 8.**

► « Designers Guide to EN 1998-1 and 1998-5 »
Thomas Telford Publisher, 2005
Explanations on Eurocode 8

► « Guide AFPS des Dispositions constructives parasismiques des ouvrages en acier, béton, bois et maçonnerie » 2007
info on website: www. afps-seisme.org

► « Earthquake Resistant Steel Structures » 2008
ArcelorMittal Technical Brochure free in French & English

► « Constructions en zone sismique » 2007
Textbook for students University of Liege
info: website www.argenco.ulg.ac.be (in french, free Download)

## SEMINARS on Eurocode 8. Some possibilities.

**In France, organised by:**

► **AFPS** Association Française de Génie Parasismique
info on website: www. afps-seisme.org

► **« Le Moniteur »**
info on website: www.groupemoniteur.fr

**In Norway:**

► **Tekna** - The Norwegian Society of Chartered Scientific and Academic Professionals
info on website: www.tekna.no

**In Belgium:**

► University of Liege, Department ARGENCO
info on website: www.argenco.ulg.ac.be

**Thank you for your attention !**