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

$$\theta_{\text{concept } a} = \frac{d_u}{h_1 \text{ étage}}$$  
$$\theta_{\text{concept } b} = \frac{d_u}{h_4 \text{ étages}}$$  
$$\theta_{\text{concept } b} = \frac{\theta_{\text{concept } a}}{4}$$

Why in a **decided scheme**?  
Because it is not thinkable to have all zones of the structure with ideal characteristics for plastic deformations
=> **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
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
Local
Dissipative &
Non dissipative
Mechanisms

<table>
<thead>
<tr>
<th>Dissipative</th>
<th>Non dissipative</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression or tension yielding</td>
<td>Failure of bolt in tension</td>
</tr>
<tr>
<td>Yielding in shear</td>
<td>Plastic deformations in narrow zone exhaust available material ductility</td>
</tr>
<tr>
<td>Plastic bending or shear of components of the connection</td>
<td>Plastic bending or shear of components of the connection</td>
</tr>
<tr>
<td>Ovalization of hole</td>
<td>Local buckling (elastic)</td>
</tr>
</tbody>
</table>

Slippage with friction

Plastic hinge
**General**

**Required steel characteristics**

- Classical constructional steel
- Charpy toughness: absorbed energy min 27J (at $t^\circ_{usage}$)
- 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**

- Calcul
  - $f_{yb}=235N/mm^2$
  - $f_{yc}=235N/mm^2$
- Réalité
  - $f_{yb}=355N/mm^2$
  - $f_{yb}=355N/mm^2$
  - $330N/mm^2$
  - $460N/mm^2$

- Principe colonne forte-poutre faible non respecté, d’ou risque de flambement de la colonne !

- Calcul
  - $f_{y}=235$ plat
  - $f_{y}=900$ boulons
  - $f_{y}=355$ poutre
- Réalité
  - $f_{y}=235$ plat
  - $f_{y}=900$ boulons
  - $f_{yb}=460$ poutre

Risque de ruine des boulons ou soudures !
**General**

**Required steel characteristics**

- to achieve $f_{y,\text{max},\text{real}} \leq f_{y,\text{design}}$
- to have a correct reference in capacity design

**Conditions on $f_y$ of dissipative zones**

3 possibilities

a) Compute considering that in dissipative zones: $f_{y,\text{max}} = 1.1 \gamma_{ov} f_y$

\[ \gamma_{ov} \text{ material overstrength factor} \quad f_y : \text{nominal} \]

\[ \gamma_{ov} = f_{y,\text{real}} / f_y \]

European rolled sections: $\gamma_{ov} = 1.25$

Ex: S235, $\gamma_{ov} = 1.25$ => $f_{y,\text{max}} = 323 \text{ N/mm}^2$

- an upper value $f_{y,\text{max}}$ 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 $f_{y,\text{max}}$

Use higher nominal $f_y$ for non dissipative zones and connections

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

S355 non dissipative zones

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

- is the value used in design => $\gamma_{ov} = 1$
<table>
<thead>
<tr>
<th>TYPE of STRUCTURE</th>
<th>Ductility Class</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DCM</td>
</tr>
<tr>
<td>Moment resisting frame</td>
<td>4</td>
</tr>
<tr>
<td>Frame with concentric bracings</td>
<td></td>
</tr>
<tr>
<td>diagonal type</td>
<td>4</td>
</tr>
<tr>
<td>V type</td>
<td>2</td>
</tr>
<tr>
<td>Frame with eccentric bracings</td>
<td>4</td>
</tr>
<tr>
<td>Inverted pendulum</td>
<td>2</td>
</tr>
<tr>
<td>Structures with reinforced concrete core / walls</td>
<td></td>
</tr>
<tr>
<td>Moment resisting frame + concentric bracings</td>
<td>4</td>
</tr>
<tr>
<td>Concrete infills not connected in contact with frame</td>
<td>2</td>
</tr>
<tr>
<td>Concrete infills connected =&gt; composite</td>
<td></td>
</tr>
<tr>
<td>Concrete infills isolated from the frame</td>
<td>4</td>
</tr>
</tbody>
</table>
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

<table>
<thead>
<tr>
<th>Ductility Class</th>
<th>Behaviour factor q</th>
<th>Cross Sectional Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>DCH</td>
<td>q &gt; 4</td>
<td>class 1</td>
</tr>
<tr>
<td>DCM</td>
<td>2 ≤ q ≤ 4</td>
<td>class 2</td>
</tr>
<tr>
<td>DCM</td>
<td>1,5 ≤ q ≤ 2</td>
<td>class 3</td>
</tr>
<tr>
<td>DCL</td>
<td>q ≤ 1,5</td>
<td>class 1, 2, 3, 4</td>
</tr>
</tbody>
</table>
Illustration of Design 1

Steel Moment Resisting Frame

André PLUMIER
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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
- Plastic rotation capacity at beam ends: 25 mrad DCM 35 mrad DCH
- Global ductility
- Local Ductility

=> classes of sections

Seismic resistance
- Peripheral and interior moment frames in 2 directions
- Max $q$: $\alpha_u/\alpha_1 = 5 \times 1.3 = 6.5$
- $q = 4$ 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 \( \delta \) => \( E \)
- static analysis gravity loading \( (G + \psi_{2i} Q) \)
- stability check P-\( \Delta \) effects parameter \( \theta \)
  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 \( a_{gR} = 2,0 \text{ m/s}^2 \)
Importance of the building; office building, \( \gamma_I=1,0 \)  \( \Rightarrow \) \( a_g = 2,0 \text{ m/s}^2 \)
Service load \( Q = 3 \text{ kN/m}^2 \)
Design spectrum; type 1
Soil B \( \Rightarrow \) 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 \( G+Q \)
\[ f = p l^4 / 384 E l = l /300 \]
min beam sections:
- direction x : IPE400 \( W_{pl} = 1307.10^3 \text{ mm}^3 \) \( l =23130.10^4 \text{ mm}^4 \)
- direction y : IPE360 \( W_{pl} = 1019.10^3 \text{ mm}^3 \) \( l =16270.10^4 \text{ mm}^4 \)
\( \Rightarrow \) iterations
Steel Moment Resisting frame

After iterations

Beams direction x: IPE 500   \( I = 48200.10^4 \text{ mm}^4 \)   \( W_{pl} = 2194.10^3 \text{ mm}^3 \)
direction y: IPEA 450   \( I = 29760.10^4 \text{ mm}^4 \)   \( W_{pl}= 1494.10^3 \text{ mm}^3 \)

Columns: HE340M:   \( I_{\text{strong axis}} = I_y = 76370.10^4 \text{ mm}^4 \)
\( I_{\text{weak axis}} = I_z = 19710.10^4 \text{ mm}^4 \)
\( W_{pl,\text{strong axis}} = 4718.10^3 \text{ mm}^3 \)   \( W_{pl,\text{weak axis}} = 1953.10^3 \text{ mm}^3 \)

Weak Beam-Strong Column (WBSC) check:
\[ \sum M_{Rc} \geq 1,3 \sum M_{Rb} \]
All S355 => criteria is:
\[ \sum W_{pl, \text{columns}} \geq 1,3 \sum W_{pl, \text{beams}} \]

Seismic mass above ground considered as fixity level
\( m = G + \psi E_i Q = 3060.10^3 \text{ kg} \)   \( \psi_2i = 0,3 \)   \( \varphi=0,5 \)   \( \psi_{Ei} =0,15 \)

Note: steel frame = 7,5 % seismic mass could be taken constant in iterations
\( G + \psi_{Ei} Q \text{ floors} = 70\% \) seismic mass
Steel Moment Resisting frame

Evaluation of seismic design shear using the ‘lateral forces’ method

- Estimated fundamental period $T$ of the structure:
  \[ T = C_t \cdot H^{3/4} \]
  \[ C_t = 0.085 \]
  \[ H = 6 \times 2.9 \text{ m} = 17.4 \text{ m} \]
  \[ \Rightarrow T = 0.085 \times 17.4^{3/4} = 0.72 \text{ s} \]

- Design pseudo acceleration $S_d (T)$:
  \[ T_C < T < T_D \]
  \[ S_d (T) = (2.5 \times a_g \times S \times T_C) / (q \times T) = (2.5 \times 2 \times 1.2 \times 0.5) / (4 \times 0.72) = 1.04 \text{ m/s}^2 \]

- Seismic design shear $F_{bR}$
  \[ F_{bR} = m \cdot S_d (T) \cdot \lambda = 3060.10^3 \times 1.04 \times 0.85 = 2705.10^3 \text{ N} = 2705 \text{ kN} \]

- 6 same frames floor diaphragm effective
  \[ \Rightarrow \text{seismic design shear} \ F_{bX} \text{ in one frame:} \]
  \[ F_{bX} = F_{bR} / 6 = 451 \text{ kN} \]

- Torsion by amplifying $F_{bX}$ by $\delta = 1 + 0.6 \times L \delta = 1 + 0.6 \times 0.5 = 1.3$
  \[ F_{bX} \text{ including torsion:} \]
  \[ F_{bX} = 586 \text{ kN} \]

Storey forces Triangular Distribution in kN

$F_1 = 27.9 \quad F_2 = 55.8 \quad F_3 = 83.7 \quad F_4 = 111.6 \quad F_5 = 139.5 \quad F_6 = 167.5$
Steel Moment Resisting frame

Results of the lateral force method analysis

Diagram Of Bending Moments Under E
Steel Moment Resisting frame

Bending moment diagram: $E + G + \psi_{2i} Q$
Units: kNm
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:
  \[
  E_d = \sum G_{k,j} \; \pm \; P \; \pm \; \sum \psi_{2i} Q_{ki} \; \pm \; \gamma_1 A_{Ed}
  \]

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

Limitation of 2\(^{nd}\) order effects

- If necessary, 2\(^{nd}\) order effects are taken into account in the value of \( E_d \)
- 2\(^{nd}\) order moments \( P_{tot} \; d_r \leftrightarrow 1^{st} \) 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- \( \Delta \) effects negligible
- \( 0,1 < \theta \leq 0,2 \)
- \( \Rightarrow \) multiply action effects by \( 1/(1-\theta) \)
- Always: \( \theta \leq 0,3 \)
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;
- \( \nu \): reduction factor for lower return period of the seismic action associated with the damage limitation requirement.

Recommended:
- \( \nu = 0,4 \) for importance classes III and IV
- \( \nu = 0,5 \) for importance classes I and II
Steel Moment Resisting frame

Results of the lateral force method analysis

<table>
<thead>
<tr>
<th>Stor</th>
<th>Absolute displacement of the storey:</th>
<th>Design interstorey drift:</th>
<th>Storey lateral forces:</th>
<th>Shear at storey</th>
<th>Total cumulative gravity load at storey</th>
<th>Storey drift sensitivity coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>E0</td>
<td>d0 [m]</td>
<td>d0 [m]</td>
<td>E0</td>
<td>V0 = 27,9</td>
<td>P0 = 586,0</td>
<td>θ0 = 0,10</td>
</tr>
<tr>
<td>E1</td>
<td>d1 [m]</td>
<td>d1 [m]</td>
<td>E1</td>
<td>V1 = 55,8</td>
<td>P1 = 558,1</td>
<td>θ1 = 0,141</td>
</tr>
<tr>
<td>E2</td>
<td>d2 [m]</td>
<td>d2 [m]</td>
<td>E2</td>
<td>V2 = 83,7</td>
<td>P2 = 502,3</td>
<td>θ2 = 0,122</td>
</tr>
<tr>
<td>E3</td>
<td>d3 [m]</td>
<td>d3 [m]</td>
<td>E3</td>
<td>V3 = 111,6</td>
<td>P3 = 418,6</td>
<td>θ3 = 0,093</td>
</tr>
<tr>
<td>E4</td>
<td>d4 [m]</td>
<td>d4 [m]</td>
<td>E4</td>
<td>V4 = 139,5</td>
<td>P4 = 307,0</td>
<td>θ4 = 0,062</td>
</tr>
<tr>
<td>E5</td>
<td>d5 [m]</td>
<td>d5 [m]</td>
<td>E5</td>
<td>V5 = 167,5</td>
<td>P5 = 167,5</td>
<td>θ5 = 0,037</td>
</tr>
<tr>
<td>E6</td>
<td>d6 [m]</td>
<td>d6 [m]</td>
<td>E6</td>
<td>V6 = 167,5</td>
<td>P6 = 167,5</td>
<td></td>
</tr>
</tbody>
</table>
Steel Moment Resisting frame

**2\textsuperscript{nd} order effects**
\[ \theta_2 = 0,141 \]
\[ \theta_3 = 0,122 \]

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

=> make resistance & deformation checks with increased values

**Checks under service earthquake**

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

Limit:
\[ 0,10 \times h = 0,1 \times 2,9 \text{m} = 0,029 \text{m} \approx 0,31 \text{m} \]
Dynamic analysis       Modal superposition method

A single plane frame in each direction X or Y is analysed
Torsion effects by \( \delta = 1,3 \)
\( \Rightarrow a_g \) for the analysis: \( a_g = 2 \times 1,3 = 2,6 \text{ m/s}^2 \)

Output:

\[
T_1 = 1,17 \text{ s} > 0,72\text{s} \quad F_{b_X} = 586 \text{ kN} \quad \text{lateral force method} \quad \text{one frame}
\]
\[
F_{b_X} = 396 \text{ kN} \quad \text{dynamic response} \quad \text{one frame}
\]

More refined analysis \( \Rightarrow \) economy

\( \theta \) does not differ much

Interstorey drift reduced \( D_s \) max:
\[
D_s = 0,5 \times 0,035 \times 1/ (1- 0,137) = 0,020\text{m}
\]
Limit: \( 0,10 \ h = 0,1 \times 2,9 \text{ m} = 0,029\text{m} > 0,02 \text{ m} \quad \Rightarrow \text{OK} \)
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

<table>
<thead>
<tr>
<th>Storey</th>
<th>Absolute displacement of the storey: (d_i) [m]</th>
<th>Design interstorey drift ((d_i-d_{i-1})): (d_{ri}) [m]</th>
<th>Storey lateral forces (E_i): (V_i) [kN]</th>
<th>Shear at storey (E_i): (V_{tot}) [kN]</th>
<th>Total cumulative gravity load at storey (E_i): (P_{tot}) [kN]</th>
<th>Storey height (h_i) [m]</th>
<th>Interstorey drift sensitivity coefficient (\theta_i)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(E_0)</td>
<td>(d_0) 0</td>
<td>(d_{r0})</td>
<td>(V_1) 26,6</td>
<td>(V_{tot1}) 396,2</td>
<td>(P_{tot1}) 5100</td>
<td>(h_1) 2,9</td>
<td>(\theta_1) 0,099</td>
</tr>
<tr>
<td>(E_1)</td>
<td>(d_1) 0,022</td>
<td>(d_{r1}) 0,022</td>
<td>(V_2) 42,9</td>
<td>(V_{tot2}) 369,7</td>
<td>(P_{tot2}) 4250</td>
<td>(h_2) 2,9</td>
<td>(\theta_2) 0,137</td>
</tr>
<tr>
<td>(E_2)</td>
<td>(d_2) 0,057</td>
<td>(d_{r2}) 0,035</td>
<td>(V_3) 50,0</td>
<td>(V_{tot3}) 326,8</td>
<td>(P_{tot3}) 3400</td>
<td>(h_3) 2,9</td>
<td>(\theta_3) 0,118</td>
</tr>
<tr>
<td>(E_3)</td>
<td>(d_3) 0,090</td>
<td>(d_{r3}) 0,033</td>
<td>(V_4) 61,1</td>
<td>(V_{tot4}) 276,7</td>
<td>(P_{tot4}) 2550</td>
<td>(h_4) 2,9</td>
<td>(\theta_4) 0,086</td>
</tr>
<tr>
<td>(E_4)</td>
<td>(d_4) 0,117</td>
<td>(d_{r4}) 0,027</td>
<td>(V_5) 85,0</td>
<td>(V_{tot5}) 215,6</td>
<td>(P_{tot5}) 1700</td>
<td>(h_5) 2,9</td>
<td>(\theta_5) 0,054</td>
</tr>
<tr>
<td>(E_5)</td>
<td>(d_5) 0,137</td>
<td>(d_{r5}) 0,020</td>
<td>(V_6) 130,6</td>
<td>(V_{tot6}) 130,6</td>
<td>(P_{tot6}) 850</td>
<td>(h_6) 2,9</td>
<td>(\theta_6) 0,027</td>
</tr>
</tbody>
</table>

Where: 
- \(E_i\) represents the storey index.
- \(d_i\) is the absolute displacement of the storey.
- \(d_{ri}\) is the interstorey drift.
- \(V_i\) is the shear force at the storey.
- \(V_{tot}\) is the total cumulative shear force.
- \(P_{tot}\) is the total cumulative gravity load.
- \(h_i\) is the storey height.

The total cumulative gravity load is given by: 
\[
G + \psi E_i \cdot Q = 35,42 \text{ kN/m}
\]
Elements checks
Action effects to consider are:

\[ N_{Ed} = N_{Ed,G} + 1,1\gamma_{ov} \Omega N_{Ed,E} \]
\[ M_{Ed} = M_{Ed,G} + 1,1\gamma_{ov} \Omega M_{Ed,E} \]
\[ V_{Ed} = V_{Ed,G} + 1,1\gamma_{ov} \Omega V_{Ed,E} \]

They take into account:
- Section overstrength \( \Omega = \frac{M_{pl,Rd}}{M_{Ed}} \)
- Material overstrength \( \frac{f_{y,real}}{f_{y,nominal}} = \gamma_{ov} \)

Column buckling
Buckling length = 2.9 m = storey height
\( N_{b,Rd} = 9529 \text{ kN} > 3732 \text{ kN} \) at ground level \( \text{OK} \)

Plastic hinges at column basis
Interaction \( M - N \) Eurocode 3 (EN1993-1-1 cl 6.2.9.1)
\[ N_{Ed} = G + \psi_{2i} Q \]
\[ n = \frac{N_{Ed}}{N_{pl,Rd}} = 0,184 \]
\[ a = \frac{(A-2bt_f)}{A} = \frac{(31580 - 2 \times 309 \times 40)}{31580} = 0,22 > 0,17 (= n) \]
\[ M_{pl,y,Rd} = f_{yd} \times W_{pl,y,Rd} = 1674,89 \text{ kNm} \]
\[ M_{N,y,Rd} = M_{pl,y,Rd} (1-n)/(1-0,5 a) = 1540 \text{ kNm} \]
\( \Rightarrow M_{Ed} = 426 \text{ kNm} \)
As \( n < a \) => \( M_{N,z,Rd} = M_{pl,z,Rd} = 693 \text{ kNm} \)
\( \Rightarrow M_{Ed} = 114 \text{ kNm} \)
\( \Rightarrow \text{resisting moments > design action effects} \)
\( M_{Ed} = M \) (E + G + \psi_{2i} Q)
Steel Moment Resisting frame

Other checks

Beam lateral torsional buckling

\[ M \text{ at beam column connection} = M_{pl} \]

⇒ Lateral supports may be required
Steel Moment Resisting frame

Column web panels

Seismic action effect
In column web panel

\[ V_{wp,Ed} = \frac{M_{pl,Rd, left}}{d_{left} - 2t_{f,left}} + \frac{M_{pl,Rd, right}}{d_{right} - 2t_{f,right}} + V_{Ed, column} \]

Often: \( V_{wp,Ed} > V_{wp,Rd} \)

⇒ « doubler » plates
welded on web or placed // to web welds \( \geq \) plate shear resistance

"Doubler plate"
Steel Moment Resisting frame

- Dissipative zones can be in beams or in connections
  
  Same local ductility requirement:
  
  \[ \theta_p = \frac{\delta}{0.5L} > 35 \text{ mrad} \quad \text{DCH} \]
  
  \[ > 25 \text{ mrad} \quad \text{DCM (}q > \] 

  \( \theta_p \): plastic rotation capacity under cyclic loading up to \( \theta_p \)

  strength degradation < 20%

  stiffness degradation < 20%

- Connection design condition
  
  - if dissipative zones are in beams => \( M_{\text{Rd,connection}} \geq \pm 1.1 \gamma_{\text{OV}} M_{\text{pl,Rd,beam}} \)

  - if dissipative connections
    
    => capacity design refers to connection plastic resistance
Connections: EC8
«avoid localisation of plastic strains»

Example

Design a) \( L_{ya} = 10 \text{ mm} \quad \varepsilon_{y,\text{max}} = 2.38 \% \)

\[ \Rightarrow \Delta l = 0.0238 \times 10 = 0.238 \text{ mm} \]
\[ \theta = 0.238 / (400 / 2) = 1.2 \text{ mrad} << 25 \text{ mrad} \]

Design b) \( L_{yb} = 400 \text{ mm} \quad \varepsilon_{y,\text{max}} = 2.38 \% \)

\[ \Rightarrow \Delta l = 9.52 \text{ mm} \]
\[ \theta = 9.52 / (400 / 2) = 47.6 \text{ mrad} >> 35 \text{ mrad} \]

Conclusions

- Plastic zone length \( \approx h_{\text{section}} \) is required for effective hinge
- Adequate \( \varepsilon_{y,\text{max}} \) and \( f_u / f_y \) needed
- Greater beam depth \( \Rightarrow \) less rotation capacity
Design of beam column connections
Detailing: not in EC8 in National Annexes, in AISC2000, AFPS2005,…

However 1 common feature:

<table>
<thead>
<tr>
<th>Connection Type</th>
<th>Maximum Ductility Class allowed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Europe</td>
</tr>
<tr>
<td>Beam flanges welded, beam web bolted to a shear tab welded to column flange. Fig. 34</td>
<td>DCL *</td>
</tr>
<tr>
<td>Beam flanges welded, beam web welded to a shear tab welded to column flange. Fig. 31</td>
<td>DCH</td>
</tr>
<tr>
<td>Beam flanges bolted, beam web bolted to a shear tab welded to column flange. Fig. 35</td>
<td>DCH</td>
</tr>
<tr>
<td>Unstiffened end plate welded to beam and bolted to column flange by 4 rows of bolts. Fig.36</td>
<td>DCH</td>
</tr>
<tr>
<td>Stiffened end plate welded to beam and bolted to column flange by 8 rows of bolts. Fig. 37</td>
<td>DCH</td>
</tr>
<tr>
<td>Reduced beam section. Beam flanges welded, beam web welded to shear tab welded to column flange. Fig.38</td>
<td>DCH</td>
</tr>
<tr>
<td>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.</td>
<td>DCH</td>
</tr>
</tbody>
</table>

*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

Northridge 1994
Steel Moment Resisting frame

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

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

DCM - DCH

DCL  low ductility
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

Design criteria
$0.5b \leq a \leq 0.75b$
$0.65h \leq s \leq 0.85h$

$b$: flange width
$h$: beam depth

$0.2b \leq c \leq 0.25b$

$b_e = b - 2c$

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

$\text{DCM - DCH}$

$\text{Radius} = \frac{4c^2 + s^2}{8c}$
Steel Moment Resisting frame

USA. Los Angeles area. 2000.

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
Design of bolted connection

**Capacity design**

\[ M_{Rd,connection} \geq 1,1 \gamma_{Ov} M_{pl,Rd,beam} = 1,1 \times 1,25 \times 778,9 = 1071 \text{ kNm} \]

**Bending moment** \( M_{Rd,connection} \)

=> 4 rows x 2 M36 10.9 bolts

row 1: \( h_r = 500–16+70 = 554 \text{ mm} \)
row 2: \( h_r = 500–16-70 = 414 \text{ mm} \)

Resistance \( F_{tr,Rd} \) M36 tension:

\[ F_{tr,Rd} = 0,9f_u A_s / \gamma_{M2} = 0,9 \times 1000 \times 817 / 1,25 = 588 \text{ kN} \]

\[ M_{Rd,connect} = (554+414) \times 2 \times 588 = 1138 > 1071 \text{ kNm} \]
Steel Moment Resisting frame

\[ V_{Rd,connection} \geq V_{Ed,G} + 1,1 \gamma_{ov} V_{Ed,E} \quad \text{Capacity design} \]

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

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

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

**Shear** \( V_{Rd,connection} \)

=> 6 M20 10.9 bolts on sides of web

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

Plate bearing resistance:

\[ V_{Rd,plate} = (6 \times 193 \times 40) / (10 \times 1,25) = 3705 > 448 \text{ kN} \]
Steel Moment Resisting frame

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

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

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:

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

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

$i_{eff} = 300 \text{ mm} \quad \gamma_{M0} = 1.0$

$f_y = 355 \text{ N/mm}^2$

$$\frac{(4 \times 300 \times t^2 \times 355)}{4} = 2213.10^3 \times 70$$

$$\Rightarrow t = 38.1 \text{ mm} \quad \text{min}$$

$$\Rightarrow t = 40 \text{ mm}$$
Steel Moment Resisting frame

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

\[ B_{p,Rd} > F_{tr,Rd} \text{?} \]

Check identical for end plate and column flange:
same thickness 40 mm and \( f_y = 355 \text{ N/mm}^2 \)

\[ F_{tr,Rd} = 553 \text{ kN} \]

\( B_{p,Rd} \) shear resistance punching out a cylinder
diameter \( d_m \) head of the bolt =58 mm for M36 bolt

\( t_p \) of plate = 40 mm

\[ B_{p,Rd} = 0.6 \pi d_m t_p f_u = 0.6 \times 3.14 \times 58 \times 40 \times 500 /1.25 = 2185 \times 10^3 \text{ N} \]

= 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:
$V_{wp,Ed} = \frac{M_{pl,Rd, left}}{(d_{left} - 2t_{f,\text{left}})} + \frac{M_{pl,Rd, right}}{(d_{right} - 2t_{f,\text{right}})} + V_{Sd,c}$
Neglecting $V_{Sd,c}$: $V_{wp,Ed} = 2 \times 1071.\ 10^3 / (377-2\times40) = 7212 \text{ kN}$
$V_{wb,Rd} = (0,9 \ f_y \ A_{wc}) / (\sqrt{3} \times \gamma_{M0}) = (0,9\times355\times9893)/\sqrt{3}$
= 1824 kN $<<$ 7212 kN
Column web increased for shear resistance:
7212–1824 = 5388 kN
Area = $(5388.10^3\sqrt{3})/(355\times0,9) = 29209 \text{ mm}^2$
2 plates 297 mm length
Thickness: $29209/(2 \times 297)$
= 49,2 mm $=>$ 50 mm
Check of column web panel in transverse compression

\[ F_{c,wc,Rd} = \omega k_{wc} b_{\text{eff,}c,wc} t_{wc} f_{y,wc} / \gamma_{M0} \]

setting \( \omega \) and \( k_{wc} \) at 1.0

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

ignoring the connecting plates of beams in the y direction

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

A more comprehensive check

include connecting plates of beams in the y direction

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

Check of column web panel in transverse tension

\[ F_{c,wc,Rd} = \omega b_{\text{eff,}c,wc} t_{wc} f_{y,wc} / \gamma_{M0} \]

identical to above, satisfied
Comments on design options

Design governed by limitation of deflections:
- \( P-\Delta \) design earthquake
- inter-storey drift service earthquake

Beam sections possess a safety margin for resistance to design EQ
\[
M_{pl,Rd,beam} = 778 \text{ kNm} > M_{Ed} = 591 \text{ kNm} \text{ (worst case moment)}
\]

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

\[
M_{pl,Rd,beam} \text{ could be reduced by } \frac{778}{591} = 1.32
\]

\( \Rightarrow \) 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 $\theta$ increased:
- by estimated 7% (canadian code)
- can be computed

Revised amplification factors $1/ (1- \theta)$

<table>
<thead>
<tr>
<th>Storey</th>
<th>Interstorey drift sensitivity coefficient $\theta$</th>
<th>amplification factor $1/ (1- \theta)$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Without RBS</td>
<td>With RBS</td>
</tr>
<tr>
<td></td>
<td>Without RBS</td>
<td>With RBS</td>
</tr>
<tr>
<td>1</td>
<td>0.099</td>
<td>0.105</td>
</tr>
<tr>
<td>2</td>
<td>0.137</td>
<td>0.147</td>
</tr>
<tr>
<td>3</td>
<td>0.118</td>
<td>0.126</td>
</tr>
<tr>
<td>4</td>
<td>0.086</td>
<td>0.092</td>
</tr>
<tr>
<td>5</td>
<td>0.054</td>
<td>0.057</td>
</tr>
<tr>
<td>6</td>
<td>0.027</td>
<td>0.028</td>
</tr>
</tbody>
</table>
Steel Moment Resisting frame

Influence of RBS distance to connection on design moment

\[ a = 0,5 \times b = 0,5 \times 200 = 100 \text{ mm} \]
\[ s = 0,65 \times d = 0,65 \times 500 = 325 \text{ mm} \]

Distance RBS to column face
\[ a + s/2 = 162,5 + 100 = 262 \text{ mm} \]

Bending moment
\[ \approx \text{linear between beam end -1/3 span} \]
\[ 1/3 \text{ span} = 8000 / 3 = 2666 \text{ mm} \]

\[ \Rightarrow \text{Design bending moment in RBS} \]
\[ M_{d,RBS} = 596 \times (2666 - 262)/2666 = 537 \text{ kNm} \]
Steel Moment Resisting frame

Definition of section cuts at RBS.
c in the range 0,20b-0,25b  c=0,22b= 44 mm
IPE500 $W_{pl,y} \ f_y = 2194.10^3 \times 355 = 778. \ 10^6 \text{ Nmm}$
Flange moment: $b \ t_f \ f_y (d - t_f) = 16\times200\times355(500-16) = 549. \ 10^6 \text{ Nmm}$
Web moment: $t_w \ f_y (d - 2t_f)^2/4=10,2\times355 \ (500 - 32)^2 = 198. \ 10^6 \text{ Nmm}$
Due to root radii web-flange junctions:$(778-549-198) = 31. \ 10^6 \text{ Nmm}$

Plastic moment of reduced IPE500
$b_e = b - 2c = 200 - 88 = 120 \text{ mm}$.
Flange moment: $b_e t_f f_y (d-t_f)=16\times112\times355(500-16)= 308. \ 10^6 \text{ Nmm}$
RBS plastic moment: $M_{pl,Rd,RBS}=(308+198+31)10^6= 537.10^6 \text{ Nmm}$
For fabrication purposes: radius $R$ of the cut
$R = (4c^2 + s^2) / 8c = (4 \times 32^2 + 325^2)/(8 \times 32) = 857 \text{ mm}$
Steel Moment Resisting frame

Design moment and shear at the connection

\[ V_{Ed,E} = 2 \frac{M_{pl,Rd,,RBS}}{L'} \]

\[ L' = 8000 - 377 - (2 \times 262.5) = 7098 \text{ mm} \]

\[ V_{Ed,E} = 2 \times 537 / 7098 = 151 \text{ kN} \]

\[ V_{Ed,G} \text{ in RBS due to gravity } G + \psi_2 l Q: V_{Ed,G} = 0.5 \times 7098 \times 45.2 = 160.4 \text{ kN} \]

Design shear in RBS:

\[ V_{Ed,E} = V_{Ed,G} + 1.1 \gamma_{ov} V_{Ed,E} \]

\[ V_{Ed,E} = 160.4 + 1.1 \times 1.25 \times 151 = 368 \text{ kN} \]

\[ M_{Ed,connection} = 1.1 \gamma_{ov} M_{pl,Rd,,RBS} + V_{Ed,E} \times \text{dist} \times \]

\[ x = a + s/2 = 262.5 \text{ mm} \]

\[ M_{Ed,connection} = 1.1 \times 1.25 \times 537 + 368 \times 0.2625 = 834 \text{ kNm} \]

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

\[ V_{Rd,connection} \geq 448 \text{ kN without RBS} \quad V_{Rd,connection} \geq 368 \text{ kN with RBS} \]

Reduction in design shear at connection = -21%
Composite Steel Concrete Moment Resisting Frame

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

<table>
<thead>
<tr>
<th>Seismicity</th>
<th>Beams</th>
<th>Columns</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>High</td>
<td>0.25g</td>
<td>Comp.</td>
<td>steel</td>
</tr>
<tr>
<td>High</td>
<td>0.25g</td>
<td>Comp.</td>
<td>Comp.</td>
</tr>
<tr>
<td>Low</td>
<td>0.10g</td>
<td>Comp.</td>
<td>steel</td>
</tr>
<tr>
<td>Low</td>
<td>0.10g</td>
<td>Comp.</td>
<td>Comp.</td>
</tr>
</tbody>
</table>

- **Permanent Actions**
  - Slab: 5 kN/m²
  - Partitions: 3 kN/m

- **Variable Actions**
  - Uniformly distributed loads: \( q_k = 3 \text{ kN/m}^2 \)
  - Concentrated loads: \( Q_k = 4 \text{ kN} \)
  - Snow load altitude \( A = 1200 \text{ m} \) \( q = 1.1 \text{ kN/m}^2 \)
  - Wind Load: \( q_p(Z) = 1.4 \text{ kN/m}^2 \)

- **Seismic Action**
  - \( \gamma_I = 1.00 \)
  - \( a_{gR} = 0.25g \) \( 0.10g \)

  \( q = 4 \)

  \( \psi = 0.7 \)

  \( \psi_1 = 0.5 \)

  \( \psi_2 = 0.3 \)

  Type 1 design spectrum soil B DCM
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

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

<table>
<thead>
<tr>
<th>Case1</th>
<th>Case2</th>
<th>Case3</th>
<th>Case4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic mass (t)</td>
<td>1900</td>
<td>1963</td>
<td>1916</td>
</tr>
</tbody>
</table>

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

  $F_b = m \ast S_d (T_1) \ast \lambda$
  $F_b = 1963 \ast 0.535 \ast 0.85$
  $F_b = 892 \text{ kN}$

  Base shear $F_{bx}$ on each MR frame
  
  $F_{bx} = \frac{F_b}{5} = \frac{892}{5} = 178.4 \text{ kN}$

  Torsion effect
  
  $\delta = 1 + 0.6 \ast \frac{x}{L}$
  $\delta = 1.3$

  $\Rightarrow$
  $F_{bx_t} = 1.3 \ast 178.4$
  $F_{bx_t} = 232 \text{ kN}$
Distribution of seismic loads

<table>
<thead>
<tr>
<th>Seismic static equivalent forces</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>E1 (kN)</td>
<td>15.7</td>
<td>15.5</td>
<td>7.7</td>
<td>7.7</td>
</tr>
<tr>
<td>E2 (kN)</td>
<td>31.4</td>
<td>30.9</td>
<td>15.4</td>
<td>15.3</td>
</tr>
<tr>
<td>E3 (kN)</td>
<td>47.1</td>
<td>46.4</td>
<td>23.1</td>
<td>23.0</td>
</tr>
<tr>
<td>E4 (kN)</td>
<td>62.8</td>
<td>61.9</td>
<td>30.8</td>
<td>30.7</td>
</tr>
<tr>
<td>E5 (kN)</td>
<td>78.5</td>
<td>77.3</td>
<td>38.5</td>
<td>38.3</td>
</tr>
</tbody>
</table>
● **Combinations at ULS considered in the analysis for an office building**

\[1.35G + 1.5W + 1.05Q + 0.75S\]

\[1.35G + 1.5W + 1.05S + 0.75Q\]

\[1.35G + 1.5Q + 1.05W + 0.75S\]

\[1.35G + 1.5Q + 1.05S + 0.75W\]

\[1.35G + 1.5W + 1.05(S + Q)\]

\[1.35G + 1.5(S + Q) + 1.05W\]

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

● **Seismic Design Situation**

\[G_k + \psi_2Q_k + E\] 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

<table>
<thead>
<tr>
<th>Seismicity</th>
<th>Beams</th>
<th>Columns</th>
<th>Steel</th>
<th>T simpl EC8(s)</th>
<th>Sd(T) EC8 m/s²</th>
<th>T Exact (s)</th>
<th>Sd(T) Exact m/s²</th>
<th>Seismic mass t</th>
</tr>
</thead>
<tbody>
<tr>
<td>High 0,25g</td>
<td>Comp. steel</td>
<td></td>
<td>S355</td>
<td>0,727</td>
<td>1,26</td>
<td>1,64</td>
<td>0,56</td>
<td>1900</td>
</tr>
<tr>
<td>High 0,25g</td>
<td>Comp. Comp.</td>
<td></td>
<td>S355</td>
<td>0,727</td>
<td>1,26</td>
<td>1,72</td>
<td>0,56</td>
<td>1963</td>
</tr>
<tr>
<td>Low 0,10g</td>
<td>Comp. steel</td>
<td></td>
<td>S235</td>
<td>0,727</td>
<td>0,51</td>
<td>1,35</td>
<td>0,27</td>
<td>1916</td>
</tr>
<tr>
<td>Low 0,10g</td>
<td>Comp. Comp.</td>
<td></td>
<td>S235</td>
<td>0,727</td>
<td>0,51</td>
<td>1,41</td>
<td>0,27</td>
<td>1994</td>
</tr>
</tbody>
</table>

### Diagram

For high seismicity:

- \( T_1 = C \times H^{\frac{3}{4}} \)
- \( T_1 = 0.727 \) s

For low seismicity:

- \( T_{0.15} = 0.15 \) s
- \( T_{0.5} = 0.5 \) s
- \( T_{0.727} = 0.727 \) s
- \( T_{1.85} = 1.85 \) s
- \( T_{2} = 2 \) s
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
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

\[ b_{ei} = \frac{L_e}{8} \text{ not greater than width } b_i \]

\[ b_{eff} = b_0 + \sum b_{ei} \]

\[ b_{eff} = \begin{cases} 1225 \text{ mm (at mid-span)} \\ 875 \text{ mm (at an end support)} \end{cases} \]
Effective Width
Seismic
Eurocode 8-1
Effective width $b_{\text{eff}}$ concrete flange: $b_{e1} + b_{e2}$
Partial effective widths $b_e$ in Tables, not $\geq b_1$ & $b_2$

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**

<table>
<thead>
<tr>
<th>$b_e$</th>
<th>Transverse element</th>
<th>$b_e$ for $I$ (Elastic Analysis)</th>
</tr>
</thead>
<tbody>
<tr>
<td>At interior column</td>
<td>Present or not present</td>
<td>For negative $M : 0,05 I$</td>
</tr>
<tr>
<td>At exterior column</td>
<td>Present</td>
<td>For positive $M : 0,0375 I$</td>
</tr>
</tbody>
</table>
| At exterior column | Not present, or re-bars not anchored | For negative $M : 0$  
For positive $M : 0,025 I$ |

#### EC8 Table

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

<table>
<thead>
<tr>
<th>Sign of bending moment $M$</th>
<th>Location</th>
<th>Transverse element</th>
<th>$b_e$ for $M_{Rd}$ (Plastic resistance)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negative $M$</td>
<td>Interior column</td>
<td>Seismic re-bars</td>
<td>0,1 $I$</td>
</tr>
<tr>
<td>Negative $M$</td>
<td>Exterior column</td>
<td>All layouts with re-bars anchored to façade beam or to concrete cantilever edge strip</td>
<td>0,1 $I$</td>
</tr>
<tr>
<td>Negative $M$</td>
<td>Exterior column</td>
<td>All layouts with re-bars not anchored to façade beam or to concrete cantilever edge strip</td>
<td>0,0</td>
</tr>
<tr>
<td>Positive $M$</td>
<td>Interior column</td>
<td>Seismic re-bars</td>
<td>0,075 $I$</td>
</tr>
<tr>
<td>Positive $M$</td>
<td>Exterior column</td>
<td>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</td>
<td>0,075 $I$</td>
</tr>
<tr>
<td>Positive $M$</td>
<td>Exterior column</td>
<td>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</td>
<td>$b_v/2 + 0,7 h_c/2$</td>
</tr>
</tbody>
</table>
| Positive $M$ | Exterior column | All other layouts. Seismic re-bars | $b_v/2 \leq b_{c,max}$  
$b_{c,max} = 0,05I$ |
Effective slab width $b_{\text{eff}}$ (mm) at column

<table>
<thead>
<tr>
<th></th>
<th>For Positive Moment $M_{\text{pl,Rd}}^+$</th>
<th>For Negative Moment $M_{\text{pl,Rd}}^-$</th>
</tr>
</thead>
<tbody>
<tr>
<td>EC4</td>
<td>Not defined</td>
<td>875 mm</td>
</tr>
<tr>
<td>EC8 Elastic analysis</td>
<td>525 mm</td>
<td>700 mm</td>
</tr>
<tr>
<td>EC8 Plastic Moments</td>
<td>1050 mm</td>
<td>1400 mm</td>
</tr>
</tbody>
</table>
Composite Steel Concrete Moment Resisting Frame

Check of $c/t \Leftrightarrow$ classes of sections
$= \text{condition 1 for local ductility in plastic hinges}$

Composite beams with IPE330 & IPE360 $\Rightarrow$ class 2
Steel columns with HEA360 & HEA450 $\Rightarrow$ class 1
Composite columns HEA320 & HEA400 $\Rightarrow$ class 1
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
## Limits of wall slenderness for steel and encased H and I sections for different design details and behaviour factors \( q \).

<table>
<thead>
<tr>
<th>Ductility Class of Structure</th>
<th>DCM</th>
<th>DCH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference value of behaviour factor ( q )</td>
<td>( 1.5 &lt; q \leq 2 )</td>
<td>( 2 &lt; q \leq 4 )</td>
</tr>
<tr>
<td>FLANGE outstand limits ( c/t_f )</td>
<td>14 ( \varepsilon )</td>
<td>10 ( \varepsilon )</td>
</tr>
<tr>
<td>Reference: H or I Section in steel only EN1993-1-1:2004 Table 5.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>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</td>
<td>20 ( \varepsilon )</td>
<td>14 ( \varepsilon )</td>
</tr>
<tr>
<td>FLANGE outstand limits ( c/t_f )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H or I Section, partially encased + straight links as in Figure 57 a) placed with ( s/c \leq 0.5 ) EN1998-1-1:2004</td>
<td>30 ( \varepsilon )</td>
<td>21 ( \varepsilon )</td>
</tr>
<tr>
<td>FLANGE outstand limits ( c/t_f )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H or I Section, fully encased + hoops placed with ( s/c \leq 0.5 ) EN1998-1-1:2004</td>
<td>30 ( \varepsilon )</td>
<td>21 ( \varepsilon )</td>
</tr>
<tr>
<td>WEB depth to thickness limit ( c_w/t_w )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( 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</td>
<td>42( \varepsilon )</td>
<td>38 ( \varepsilon )</td>
</tr>
<tr>
<td>WEB depth to thickness limit ( c_w/t_w )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>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)</td>
<td>38( \varepsilon )</td>
<td>38 ( \varepsilon )</td>
</tr>
</tbody>
</table>

Note: \( \varepsilon = (f_y/235)^{0.5} \) with \( f_y \) in MPa
Condition 2 for local ductility in plastic hinges H steel profile + slab
Steel yields: $\varepsilon > \varepsilon_y$  
Concrete remain elastic: $\varepsilon < \varepsilon_{cu2}$
⇒ a condition on the position of the plastic neutral axis:
$$\frac{x}{d} < \frac{\varepsilon_{cu2}}{\varepsilon_{cu2} + \varepsilon_a}$$

$x$  distance from top concrete compression fibre to plastic neutral axis
$d$  depth of composite section  
$\varepsilon_a$  total strain in steel at ULS

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

<table>
<thead>
<tr>
<th>Ductility class</th>
<th>$q$</th>
<th>$f_y$ (N/mm²)</th>
<th>$x/d$ upper limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>DCM</td>
<td>$1.5 &lt; q \leq 4$</td>
<td>355</td>
<td>0.27</td>
</tr>
<tr>
<td></td>
<td>$1.5 &lt; q \leq 4$</td>
<td>235</td>
<td>0.36</td>
</tr>
<tr>
<td>DCH</td>
<td>$q &gt; 4$</td>
<td>355</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>$q &gt; 4$</td>
<td>235</td>
<td>0.27</td>
</tr>
</tbody>
</table>
Composite Steel Concrete Moment Resisting Frame

IPE330_Case 1 and 2
IPE360_Case 3 and 4

<table>
<thead>
<tr>
<th></th>
<th>Case1</th>
<th>Case2</th>
<th>Case3</th>
<th>Case4</th>
</tr>
</thead>
<tbody>
<tr>
<td>(x/d) Limit values EC8</td>
<td>0.27</td>
<td>0.27</td>
<td>0.36</td>
<td>0.36</td>
</tr>
<tr>
<td>(x/d)_max Design values</td>
<td>0.268</td>
<td>0.268</td>
<td>0.239</td>
<td>0.239</td>
</tr>
</tbody>
</table>
Composite Steel Concrete Moment Resisting Frame

Analysis

Blue: with steel column
Red: with composite column

High seismicity

Low seismicity

All beams are composite

Dissemination of information for training – Lisbon 10-11 February 2011
Composite Steel Concrete Moment Resisting Frame

Results of analysis  Example

Axial force diagram
\[ N_{\text{max}} = 1980 \text{ kN} \]

Bending moment diagram
\[ M_{z,\text{max}} = 319 \text{ kNm} \]

High seismicity – steel columns
Composite Steel Concrete Moment Resisting Frame

**EC8 check**

**Resistance of dissipative zones**

Check: plastic hinges at beam ends

\[
M_{pl,Rd}^- = W_{plb} \times f_y
\]

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

\[
M^+ = W_{plb} \times f_y
\]

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

Maximum “work rate” in beams: \( M_{Ed} / M_{pl,Rd} \)

<table>
<thead>
<tr>
<th>Case 1: high seismicity (steel columns)</th>
<th>0.933</th>
<th>0.826</th>
<th>1.21</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 2: high seismicity (composite columns)</td>
<td>0.953</td>
<td>0.840</td>
<td>1.19</td>
</tr>
<tr>
<td>Case 3: low seismicity (steel columns)</td>
<td>0.979</td>
<td>0.764</td>
<td>1.31</td>
</tr>
<tr>
<td>Case 4: low seismicity (composite columns)</td>
<td>1.000</td>
<td>0.779</td>
<td>1.28</td>
</tr>
</tbody>
</table>

\[ \Omega_{\min} \]

=> Limited overstrength \( \Omega_{\min} \)
**Composite Steel Concrete Moment Resisting Frame**

**EC8 check**

**Second order effects**

\[
\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0.1
\]

![Diagram showing forces and displacements in a structural frame.

**Example**

**High seismicity – steel columns**

<table>
<thead>
<tr>
<th>Storey No.</th>
<th>(d_e) [m]</th>
<th>(d_r) [m]</th>
<th>(V) [kN]</th>
<th>(V_{tot}) [kN]</th>
<th>(P_{tot}) [kN]</th>
<th>(\theta)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.007</td>
<td>0.007</td>
<td>15.70</td>
<td>235.48</td>
<td>3799.96</td>
<td>0.032</td>
</tr>
<tr>
<td>2</td>
<td>0.019</td>
<td>0.012</td>
<td>31.40</td>
<td>219.78</td>
<td>3046.62</td>
<td>0.048</td>
</tr>
<tr>
<td>3</td>
<td>0.030</td>
<td>0.011</td>
<td>47.10</td>
<td>188.38</td>
<td>2293.28</td>
<td>0.038</td>
</tr>
<tr>
<td>4</td>
<td>0.038</td>
<td>0.008</td>
<td>62.79</td>
<td>141.28</td>
<td>1539.94</td>
<td>0.025</td>
</tr>
<tr>
<td>5</td>
<td>0.044</td>
<td>0.006</td>
<td>78.49</td>
<td>78.49</td>
<td>786.60</td>
<td>0.017</td>
</tr>
</tbody>
</table>

\[d_r = q \cdot d_{re}\]

\[V_{tot}\]

\[P_{tot}\]

\[\theta \leq 0.10\]

\[\theta \leq 0.10\]

\[\theta \leq 0.10\]
EC8 check
Damage limitations in non-structural elements

\[ d_r \cdot v \leq 0.010h \quad \text{with} \quad dr = q \cdot d_r^e \]

\[ q=4 \quad v=0.5 \]

<table>
<thead>
<tr>
<th>Storey</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
<th>0.010 ( h ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>14</td>
<td>16</td>
<td>4</td>
<td>4</td>
<td>35</td>
</tr>
<tr>
<td>2</td>
<td>24</td>
<td>26</td>
<td>8</td>
<td>10</td>
<td>35</td>
</tr>
<tr>
<td>3</td>
<td>22</td>
<td>22</td>
<td>8</td>
<td>6</td>
<td>35</td>
</tr>
<tr>
<td>4</td>
<td>16</td>
<td>18</td>
<td>6</td>
<td>6</td>
<td>35</td>
</tr>
<tr>
<td>5</td>
<td>12</td>
<td>10</td>
<td>4</td>
<td>6</td>
<td>35</td>
</tr>
</tbody>
</table>

All \( d_r < 0.10h \Rightarrow \text{OK} \)
Elements checks
Action effects to consider are:

They take into account:
- Section overstrength $\Omega = \frac{M_{pl,Rd}}{M_{Ed}}$
- Material overstrength $\frac{f_{y,real}}{f_{y,nominal}} = \gamma_{ov}$

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

$$\Omega = \frac{393}{324.20} = 1.212 \text{ (Case1)}$$
$$\Omega = \frac{337}{257.00} = 1.311 \text{ (Case3)}$$

CHECKS

Beam deflections

$$f = \frac{W_u L^4}{384 EI} + \frac{W_p L^3}{192 EI} = \frac{L}{300}$$
Resistance of beams to Lateral-Torsional Buckling

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

Real risk: \[ |M_{Ed}|_{max} > M_{b,Rd} \]

⇒ Bracings required
⇒ Calculation indicate 1 m interdistance OK

Limitation of compression in beams

\[ N_{Pl,Rd} = A_f \gamma_s f_y + f_{sk} \gamma_s A_s + 0.85 f_{ck} \gamma_c A_c \]

Check: \[ \frac{N_{Ed}}{N_{Pl,Rd}} \leq 0.15 \]

\[
\begin{align*}
N_{Pl,Rd} &= \begin{cases} 5767 \text{ kN} & \text{(IPE330)} \\ 4708 \text{ kN} & \text{(IPE360)} \end{cases} \\
N_{Ed} &= \begin{cases} 149 \text{ kN} & < 0.15 \text{ N}_{Pl,Rd} = 865 \text{ kN} \text{ (Case1)} \\ 142 \text{ kN} & < 0.15 \text{ N}_{Pl,Rd} = 865 \text{ kN} \text{ (Case2)} \\ 127 \text{ kN} & < 0.15 \text{ N}_{Pl,Rd} = 706 \text{ kN} \text{ (Case3)} \\ 121 \text{ kN} & < 0.15 \text{ N}_{Pl,Rd} = 706 \text{ kN} \text{ (Case4)} \end{cases}
\end{align*}
\]
## Limitation of shear in beams

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

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

\[
M_{Ed} \leq M_{N,Rd}
\]

### Example: High seismicity, steel columns

<table>
<thead>
<tr>
<th>Case 1</th>
<th>( N_{\text{Ed,G}} )</th>
<th>( M_{\text{Ed,G}} )</th>
<th>( N_{\text{Ed,E}} )</th>
<th>( M_{\text{Ed,E}} )</th>
<th>( N^*_{\text{Ed}} )</th>
<th>( M^*_{\text{Ed}} )</th>
<th>( M_{N,y,Rd} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>End</td>
<td>kN</td>
<td>kNm</td>
<td>kN</td>
<td>kNm</td>
<td>kN</td>
<td>kNm</td>
<td>kNm</td>
</tr>
<tr>
<td>column 1</td>
<td>lower</td>
<td>-814</td>
<td>-41</td>
<td>119</td>
<td>140</td>
<td>-616</td>
<td>192</td>
</tr>
<tr>
<td></td>
<td>upper</td>
<td>-810</td>
<td>79</td>
<td>119</td>
<td>-39</td>
<td>-612</td>
<td>14</td>
</tr>
<tr>
<td>column 2</td>
<td>lower</td>
<td>-1652</td>
<td>1</td>
<td>-9</td>
<td>158</td>
<td>-1666</td>
<td>264</td>
</tr>
<tr>
<td></td>
<td>upper</td>
<td>-1648</td>
<td>-3</td>
<td>-9</td>
<td>-76</td>
<td>-1663</td>
<td>-130</td>
</tr>
<tr>
<td>column 3</td>
<td>lower</td>
<td>-1652</td>
<td>-1</td>
<td>8</td>
<td>158</td>
<td>-1638</td>
<td>262</td>
</tr>
<tr>
<td></td>
<td>upper</td>
<td>-1648</td>
<td>3</td>
<td>8</td>
<td>-76</td>
<td>-1634</td>
<td>-124</td>
</tr>
<tr>
<td>column 4</td>
<td>lower</td>
<td>-814</td>
<td>41</td>
<td>-118</td>
<td>138</td>
<td>-1011</td>
<td>272</td>
</tr>
<tr>
<td></td>
<td>upper</td>
<td>-810</td>
<td>-79</td>
<td>-118</td>
<td>-39</td>
<td>-1007</td>
<td>-143</td>
</tr>
</tbody>
</table>
Shear Resistance of Steel Columns

Example: case 1, high seismicity, steel columns

\[
\begin{align*}
|V_{Ed,G}|_{max} &= 57.54 \text{ kN} \\
|V_{Ed,E}|_{max} &= \frac{1}{1-\theta} \times 39.96 = \frac{1}{1-0.048} \times 39.96 \\
&= 1.05 \times 39.96 = 41.80 \text{ kN} \\
|V_{Ed}^*|_{max} &= |V_{Ed,G} + 1.11\gamma_{ov}\Omega V_{Ed,E}|_{max} \\
|V_{Ed}^*|_{max} &= 127.47 \text{ kN} \\
V_{Pl,a,Rd} &= \frac{A_v \times f_y}{\sqrt{3}} = \begin{cases} 1003.48 \text{ kN (Case1)} \\ 892.490 \text{ kN (Case3)} \end{cases}
\end{align*}
\]
Column buckling

Buckling length = storey height

Reduction factors $\chi$ for Flexural Buckling

<table>
<thead>
<tr>
<th></th>
<th>$\lambda_y$</th>
<th>$\chi_y$</th>
<th>$\lambda_z$</th>
<th>$\chi_z$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>0.308</td>
<td>0.961</td>
<td>0.632</td>
<td>0.766</td>
</tr>
<tr>
<td>Case 3</td>
<td>0.202</td>
<td>1.000</td>
<td>0.524</td>
<td>0.873</td>
</tr>
</tbody>
</table>

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

$$k_{yy} = C_{my} \left[ 1 + \left( \frac{\lambda_y - 0.2}{\lambda_y N_{Ed}} \right) \right]$$

Reduction Factor for Lateral Torsional-Buckling

Stability checks

$$\frac{|N_{Ed}^*|}{\chi_y N_{plRd}} + k_{yy} \frac{|M_{y,Ed}^*|}{\chi_{LT} M_{plRd}} \leq 1$$

$$\frac{|N_{Ed}^*|}{\chi_z N_{plRd}} + k_{zy} \frac{|M_{y,Ed}^*|}{\chi_{LT} M_{plRd}} \leq 1$$
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)
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)
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

Ispra test
1999
Composite Steel Concrete Moment Resisting Frame

Design to transmit slab compression/tension force
IPE330 beam  HEA360 column  $t_{slab}=120\text{mm}$
$B_{eff}^+=1050\text{mm}$  $B_{eff}^- = 1400\text{mm}$
Rebars: S500 T12@200 – 2 layers  $A_{sl}=14\times113=1582\ \text{mm}^2$  $F_{Rds}=791\ \text{kN}$
Concrete: C30/37  $F_{cd} = 30/1.5 = 20\ \text{MPa}$  $F_{Rdc} = 120\times1050\times20 = 2520\ \text{kN}$
► $F_{Rds}$ and $F_{Rdc}$ are the slab force in tension and compression
► They are transmitted to the column to transmit the beam

Facade beam-column connection

$M^-$

Each rebar: $113\ \text{mm}^2 \times 500 = 56,5\text{kN}$
1 stud/rebar  1 stud $\Phi19=81,6\text{kN}>56,5$
Facade beam-column connection

\[ M = 0 \]

- **\( F_{\text{Rd1}} = b_{\text{column}} \times t_{\text{slab}} \times f_{\text{cd}} = 300 \times 120 \times 20 = 720 \text{ kN} \)**
- **\( F_{\text{Rd2}} = h_{\text{column}} \times t_{\text{slab}} \times 0.7f_{\text{cd}} = 360 \times 120 \times 0.7 \times 20 = 604 \text{ kN} \)**
- **\( F_{\text{Rd3}} = n_{\text{stud}} \times F_{R,\text{stud}} = 14 \times 81.6 = 1142 \text{ kN} \)**

Total: \( 2466 \text{ kN} \approx 2520 \text{ kN} = F_{\text{Rdc}} \)

« Seismic rebars » for \( F_{\text{Rd2}} / 2 \)

\[ A_T = \frac{302000}{500} = 604 \text{mm}^2 \Rightarrow 4T16 \]
Facade beam-column connection

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

$M_E = 571 \times 0.55/2 = 108$ kNm

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

$M_{plRd} = 16 \times 315^2 \times 355/4 = 140$ kNm $> 108$
$V_{plRd} = 16 \times 315 \times 205 = 1033$ kN $> 571$

Interaction $M-N$ $\rho = (2 \times 571/1033 - 1)^2 = 0.01$

$\Rightarrow M_{plRd}$ unchanged OK
Interior beam-column connection

As M+ on 1 side & M- on other side, slab force to transmit:
\[ F_{Rdc} + F_{Rds} = 791 + 2950 = 3311 \text{ kN} \]
791 kN more than in facade connection

Various possible design:

- increase \( F_{Rd1} = \) increase column bearing width \( b_b \)
  but \( F_{Rd2} = 604 \text{ kN} \) is lost
With column HEA 360 flange: \( F_{Rd1} = 720 \text{ kN} \)
Width \( b_b \) to provide \( F_{Rd1} = 791 + 604 = 1395 \text{ kN} \)
\( b_b = \frac{1395000}{120 \times 20} = 581 \text{ mm} \)
=> (581-300)/2 = 140 mm extension both side (+ stiffeners)
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
**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
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 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

Dissemination of information for training – Lisbon 10-11 February 2011

Composite frames with eccentric bracings

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
## Definition of the structure

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Storey height</td>
<td>h</td>
<td>3.5 m</td>
</tr>
<tr>
<td>Total height of the building</td>
<td>H</td>
<td>17.5 m</td>
</tr>
<tr>
<td>Beam length in X-direction EBF</td>
<td>l_X</td>
<td>7 m</td>
</tr>
<tr>
<td>Beam length in Y-direction CBF</td>
<td>l_Y</td>
<td>6 m</td>
</tr>
<tr>
<td>Building width in X-direction</td>
<td>L_X</td>
<td>21 m</td>
</tr>
<tr>
<td>Building width in Y-direction</td>
<td>L_Y</td>
<td>24 m</td>
</tr>
</tbody>
</table>

**X-direction – Eccentric bracings**

**Y-direction – Concentric bracings**
## Composite frame with Eccentric and Concentric Bracings

### Details of values

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Symbol</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic yield strength of reinforcing steel</td>
<td>(f_y)</td>
<td>500</td>
<td>N/mm(^2)</td>
</tr>
<tr>
<td>Partial safety factor for steel rebar</td>
<td>(\gamma_s)</td>
<td>1.15</td>
<td></td>
</tr>
<tr>
<td>Design yield strength of reinforcement steel</td>
<td>(f_{yd})</td>
<td>434.78</td>
<td>N/mm(^2)</td>
</tr>
<tr>
<td>Characteristic compressive strength of concrete</td>
<td>(f_c)</td>
<td>30</td>
<td>N/mm(^2)</td>
</tr>
<tr>
<td>Partial safety factor for concrete</td>
<td>(\gamma_c)</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>Design compressive strength of concrete</td>
<td>(f_{cd})</td>
<td>20</td>
<td>N/mm(^2)</td>
</tr>
<tr>
<td>Secant modulus of elasticity of concrete for the design under gravity loads combinations</td>
<td>(E_c)</td>
<td>33000</td>
<td>N/mm(^2)</td>
</tr>
<tr>
<td>Secant modulus of elasticity of concrete for the design under seismic loads combination</td>
<td>(E_{c,sc})</td>
<td>16500</td>
<td>N/mm(^2)</td>
</tr>
<tr>
<td>Characteristic yield strength of steel profile</td>
<td>(f_y)</td>
<td>355</td>
<td>N/mm(^2)</td>
</tr>
<tr>
<td>Partial factor for steel profile</td>
<td></td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Modulus of elasticity of steel profile</td>
<td>(E_a)</td>
<td>210000</td>
<td>N/mm(^2)</td>
</tr>
</tbody>
</table>
Earthquake action
Design ground acceleration 0.25g
soil type B
type 1 response spectrum
DCM design with a behaviour factor q = 4

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Symbol</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil factor</td>
<td>S</td>
<td>1.2</td>
<td></td>
</tr>
<tr>
<td>Lower limit of period of constant spectral acceleration branch</td>
<td>$T_B$</td>
<td>0.15</td>
<td>s</td>
</tr>
<tr>
<td>Upper limit of period of constant spectral acceleration branch</td>
<td>$T_C$</td>
<td>0.5</td>
<td>s</td>
</tr>
<tr>
<td>Beginning of the constant displacement response range</td>
<td>$T_D$</td>
<td>2</td>
<td>s</td>
</tr>
</tbody>
</table>
Composite frame with Eccentric and Concentric Bracings

**Loads**

Permanent actions + self-weight of the slab \( G = 5.858 \text{ kN/m}^2 \)

Variable actions \( Q = 3 \text{ kN/m}^2 \)

Snow \( S = 1.11 \text{ kN/m}^2 \)

Wind \( W = 1.4 \text{ kN/m}^2 \)

**Static loading combinations:**

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

**Seismic combination:** \( G + Q + \psi_{2i} E \)

Seismic mass \( m = \sum G_{kj} + \sum \psi_{Ei} \cdot Q_{ki} \)

\( \psi_{2i} = 0.3 \)

\( \varphi = 0.8 \)

\( \psi_{E,i} = \psi_{2,i} \varphi = 0.24 \)
Composite frame with Eccentric and Concentric Bracings

Steps
General. Design of slab under gravity loads (no support of EBF)
Design of columns under gravity loads (no support of EBF)
Design of beams under gravity loads (no support of EBF)

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**
  - $T_X = 0.83 \text{ s}$
  - $T_Y = 1.45 \text{ s}$

- **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

<table>
<thead>
<tr>
<th>Characteristics of slabs</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>X-direction</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Y-direction</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th><strong>Applied moment</strong></th>
<th><strong>Resistant moment</strong></th>
<th><strong>Rebars for 1m of slab</strong></th>
<th><strong>Steel Section</strong></th>
<th><strong>Spacing of rebars</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$M_{Ed,slab,X,GC}$</td>
<td>$M_{Rd,slab,X}$</td>
<td></td>
<td>$A_{s,X}$</td>
<td></td>
</tr>
<tr>
<td><strong>Unit</strong></td>
<td>[kNm/m]</td>
<td>[kNm/m]</td>
<td>[mm]</td>
<td>[mm²/m]</td>
<td>[mm]</td>
</tr>
<tr>
<td><strong>SPAN (lower layer of rebars)</strong></td>
<td>66</td>
<td>73</td>
<td>10 T10 + 2 T16</td>
<td>1187</td>
<td>100 – 50</td>
</tr>
<tr>
<td><strong>SUPPORT (upper layer of rebars)</strong></td>
<td>92</td>
<td>95</td>
<td>10 T10 + 4 T16</td>
<td>1585</td>
<td>100 – 50</td>
</tr>
<tr>
<td><strong>SPAN (lower layer of rebars)</strong></td>
<td>35</td>
<td>49</td>
<td>10 T10</td>
<td>785</td>
<td>100</td>
</tr>
<tr>
<td><strong>SUPPORT (upper layer of rebars)</strong></td>
<td>41</td>
<td>49</td>
<td>10 T10</td>
<td>785</td>
<td>100</td>
</tr>
</tbody>
</table>
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 $e < e_{\text{short}} = 0.8 \frac{M_p,\text{link}}{V_p,\text{link}}$ yield in shear

long links $e > e_{\text{long}} = 1.5 \frac{M_p,\text{link}}{V_p,\text{link}}$ yield in bending

intermediate links $e_{\text{short}} < e < e_{\text{long}}$ 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

- $V_{p,\text{link}}$ include $V$-$N$ interaction
  
  If $N_{\text{ed}} / N_{pl,Rd} < 0.15$ =>

- Homogeneity of links overstrength
  
  $\Omega_i = 1.5 \frac{V_{p,\text{link},i}}{V_{Ed,i}}$

Section overstrength $\Omega$ refers to shear because the link is dissipative in shear

1.5: for high deformations => high strain hardening

$\Omega_{\text{max}} \leq 1.25 \Omega_{\text{min}}$

Results of analysis + profiles selected for the links

<table>
<thead>
<tr>
<th>Level</th>
<th>Link section</th>
<th>$N_{Ed}$ kN</th>
<th>$N_{Ed}/N_{pl}$</th>
<th>$M_{Ed}$ kNm</th>
<th>$M_{pl}$ kNm</th>
<th>$M_{Ed}/M_{pl}$</th>
<th>$V_{Ed}$ kN</th>
<th>$V_{pl}$ kN</th>
<th>$\Omega = 1.5 \frac{V_{pl}}{V_{Ed}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>HE450B</td>
<td>75</td>
<td>0.010</td>
<td>285</td>
<td>1141</td>
<td>0.25</td>
<td>950</td>
<td>1182</td>
<td>1.867</td>
</tr>
<tr>
<td>2</td>
<td>HE450B</td>
<td>75</td>
<td>0.010</td>
<td>296</td>
<td>1141</td>
<td>0.25</td>
<td>987</td>
<td>1182</td>
<td>1.797</td>
</tr>
<tr>
<td>3</td>
<td>HE400B</td>
<td>72</td>
<td>0.011</td>
<td>247</td>
<td>933</td>
<td>0.26</td>
<td>824</td>
<td>1011</td>
<td>1.840</td>
</tr>
<tr>
<td>4</td>
<td>HE340B</td>
<td>72</td>
<td>0.011</td>
<td>195</td>
<td>708</td>
<td>0.27</td>
<td>651</td>
<td>761</td>
<td>1.752</td>
</tr>
<tr>
<td>5</td>
<td>HE280B</td>
<td>70</td>
<td>0.015</td>
<td>123</td>
<td>455</td>
<td>0.27</td>
<td>405</td>
<td>547</td>
<td>2.028</td>
</tr>
</tbody>
</table>

$\Omega_{\text{max}} = 2.03 \leq 1.25 \Omega_{\text{min}} = 1.25 \times 1.752 = 2.19$ => 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 \(\delta = 1 + 0,6 \times \frac{x}{L} = 1,3\)

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

- **Diagonals**

Max axial loads

\[
N_{Ed,G} = 47.4 \, \text{kN} \quad N_{Ed,E} = 495.2 \, \text{kN}
\]

\[
N_{Rd} \geq 47.4 + 1,1 \times 1,25 \times 1,75 \times 495,2 = 1612 \, \text{kN}
\]

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

=> OK
Composite frame with Eccentric and Concentric Bracings

- Action effects and plastic resistance of link

<table>
<thead>
<tr>
<th>Action effects</th>
<th>Plastic resistance</th>
<th>Section overstrength $\Omega$</th>
</tr>
</thead>
<tbody>
<tr>
<td>From analysis</td>
<td>With $f_y=355$ MPa</td>
<td>$\Omega$***</td>
</tr>
<tr>
<td>$V_{Ed}=950$ kN</td>
<td>$V_{pl,Rd} = 1182$ kN</td>
<td>1182/952 =1,24</td>
</tr>
<tr>
<td>$M_{Ed}=285$ kNm</td>
<td>$M_{pl,Rd} = 1141$ kNm</td>
<td>$M_{Ed}/M_{pl,Rd} = 0,25$</td>
</tr>
<tr>
<td>$N_{Ed}=75$ kN</td>
<td>$N_{pl,Rd} = 7739$ kN</td>
<td>$N_{Ed}/N_{pl,Rd} = 0,01$</td>
</tr>
</tbody>
</table>

* Section overstrength $\Omega$ refers to shear => link dissipative in shear

** Connection design made with $\Omega = 1,24$

Note: to revise!
Should be $\Omega = 1,5 \times 1,24 = 1,86$
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

\[ V_{Ed,\text{connection}} = 1,1 \cdot \gamma_{ov} \cdot V_{pl,Rd} \]
\[ = 1,1 \times 1,25 \times 1182 = 1625 \text{ kN} \]

- **Bolts**
  
  6 M30 bolts, 2 shear planes:
  \[ V_{Rd} = 2 \times 6 \times 280 / 1,25 = 2688 \text{ kN} > 1625 \]

- **HEB450 web Thickness** \( t_{w} = 14 \text{ mm} \)
  
  Bearing resistance with \( e_1 = 60 \text{ mm}, e_2 = 50 \text{ mm}, p_1 = p_2 = 85 \text{ mm} \)
  \[ V_{Rd} = 2028 \text{ kN} > 1625 \text{ kN} \]

- Bearing resistance < bolt shear resistance
  
  \[ 2688 \text{ kN} > 1,2 \times 2028 \text{ kN} = 2433 \text{ kN} \]

- **Gussets welded on IPE270 lower flange**
  
  2 plates \( t=16 \text{ mm} \)
  \[ \tau = 1625. 10^3 / (2 \times 16 \times 320) = 180 < 355/\sqrt{3} = 204 \text{ MPa} \]
  
  Total thickness provided = 32 mm > \( t_{w,\text{HEB450}} = 14 \text{ mm} \) => all checks

- **IPE270 web stiffeners**
  
  \( t_{w} = 6,6 \text{ mm} \) is not enough => 2 plates \( t=6 \text{ mm} \) welded on IPE270 flanges
  
  Provide total thickness 6,6 +6+6=18,6mm > \( t_{w,\text{HEB450}} = 14 \text{ mm} \) => all checks
Connection HEB240 diagonals – HEB450 link
Bolted connection of HEB450 link end plate to welded built up triangle

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

\[ M_{Ed, connection} = 1,1 \gamma_{ov} \Omega M_{Ed} = 1,1 \times 1,25 \times 1,24 \times 285 = 485 \text{ kNm} \]

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

\[ V_{Ed, connection} \text{ taken by M30 bolts, single shear plane} \]
\[ 8 \text{ M30 bolts provide shear resistance } 8 \times 280,5 /1,25 = 1795 \text{ kN} > 1625 \text{ kN} \]
\[ \text{Bearing resistance: } 8 \times 289,8 \times 1,4 = 3245 \text{ kN} > 1625 \text{ kN} \]
Composite frame with Eccentric and Concentric Bracings

Welded connection between HEB450 and end plate

As above:

\[ V_{Ed, \text{connection}} = 1625 \text{ kN} \]

\[ M_{Ed, \text{connection}} = 485 \text{ kN} \]

\( V_{Ed, \text{connection}} \) taken by the web.

Weld length = 2 x 400 = 800 mm

a=8mm fillet weld provides a resistance:

\( (8 \times 261.7)/1.25 = 1674 \text{ kN} > 1625 \text{ kN} \)

\[ M_{Ed, \text{connection}} = 485 \text{ kN} \] taken by the flanges.

Weld length = 2 x 300 = 600 mm/flange

Tension force in flange = \( 485/(2 \times 0.2m) = 1214 \text{ kN} \) => 202 kN/100 mm

An a=8 mm fillet weld provides a resistance:

\( 6 \times 261.7/1.25 = 1256 \text{ kN} > 1214 \text{ kN} \)
Connection of HEB240 diagonals to welded built up triangle

\[ N_{Ed, 1\ \text{diagonal}} = N_{Ed, \text{gravity}} + 1.1 \gamma_{ov} N_{Ed,E} = 1612\ \text{kN} \]
\[ N_{pl,Rd} = 10600 \times 355 = 3763\ \text{kN} \]
\[ N_{Ed} / N_{pl,Rd} = 0.43 \]
\[ M_{Ed, 1\ \text{diagonal}} = 0.5 \times \text{link moment due to equilibrium of node} \]
\[ \Rightarrow M_{Ed, 1\ \text{diagonal}} = \frac{285}{2} = 143\ \text{kNm} \]
\[ M_{pl,Rd} = 1053 \times 10^3 \times 355 = 373\ \text{kN} \]
\[ M_{Ed} / M_{pl,Rd} = 0.38 \]

Stresses in tension & bending relatively high
\[ \Rightarrow \text{connection with} \]
full penetration butt welds
Concentric Bracing CBF

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

Diagonals should have
- similar force-displacement characteristics in both directions
- homogeneity of diagonal sections overstrength $\Omega_i = \frac{N_{pl,Rdi}}{N_{Edi}}$
- Symmetry of bracings at each level:
  - $A^+ \text{ et } A^-$, 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 $R_{fy} < R_{buckling, diagonals}$
- 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**

- \( N_{pl,Rd} \geq N_{Ed,E} \)
- \( 1,3 < \lambda \leq 2,0 \) (not for structures up to 2 levels)

- \( \Omega_i = \frac{N_{Rd}}{N_{ed}} \) \( \Omega_{max} \leq 1,25 \Omega_{min} \)
1,3 < $\bar{\lambda}$ ≤ 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
• 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*
### Concentric Bracings

#### Y direction

<table>
<thead>
<tr>
<th>Storey</th>
<th>Steel profile</th>
<th>$A$ (mm²)</th>
<th>$N_{Ed, CBI}$ (kN)</th>
<th>$N_{Rd, CB1}$ (kN)</th>
<th>$\Omega_i$</th>
<th>$\frac{N_{Rd}}{N_{Ed}}$</th>
<th>$\lambda$</th>
<th>$\theta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt; (ground level)</td>
<td>UPE 160</td>
<td>2170</td>
<td>492</td>
<td>770</td>
<td>1,56</td>
<td>1,80</td>
<td>0,17</td>
<td></td>
</tr>
<tr>
<td>2&lt;sup&gt;nd&lt;/sup&gt;</td>
<td>UPE 160</td>
<td>2170</td>
<td>531</td>
<td>770</td>
<td>1,45</td>
<td>1,80</td>
<td>0,17</td>
<td></td>
</tr>
<tr>
<td>3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>UPE 180</td>
<td>2510</td>
<td>657</td>
<td>891</td>
<td>1,35</td>
<td>1,70</td>
<td>0,15</td>
<td></td>
</tr>
<tr>
<td>4&lt;sup&gt;th&lt;/sup&gt;</td>
<td>UPE 160</td>
<td>2170</td>
<td>531</td>
<td>770</td>
<td>1,45</td>
<td>1,80</td>
<td>0,14</td>
<td></td>
</tr>
<tr>
<td>5&lt;sup&gt;th&lt;/sup&gt;</td>
<td>UPE 120</td>
<td>1540</td>
<td>373</td>
<td>546</td>
<td>1,46</td>
<td>2,15</td>
<td>0,11</td>
<td></td>
</tr>
</tbody>
</table>

- $1,3 < \lambda \leq 2,0$
  - 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:  \[ N_{Rd} (M_{Ed}, V_{Ed}) \geq N_{Ed,G} + 1.1\gamma_{ov} \Omega N_{Ed,E} \]

*Capacity design*

\[\Omega_{Y,\text{min}} = 1.35 = \text{min section overstrength factor of concentric bracings}\]

\[\gamma_{ov} = 1.25\]

Check for columns

\[N_{Rd}\ 	ext{buckling resistance strong&weak axis} \geq N_{ed,G} + 1,1\gamma_{ov} \Omega_{Y,\text{min}} N_{ed,E}\]

Check for beams

\[N_{Rd}\ 	ext{resistance under combined } M,N,V \geq N_{ed,G} + 1,1\gamma_{ov} \Omega_{Y,\text{min}} N_{ed,E}\]
Connection of a CBF diagonal
At level 1  $N_{Ed,BC1} = 492 \text{ kN}$
Element design=> UPE160: $N_{pl,Rd} = A \times f_{y,d} = 2170 \times 355 = 770\text{kN}$
Connection capacity designed to $N_{pl,Rd}$ UPE160:
$N_{Rd,connect} \geq 1.1 \gamma_{ov} N_{pl,Rd} = 1.1 \times 1.25 \times 770 = 1058 \text{kN}$

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
6 bolts, resistance in shear, one shear plane, for M30 bolts:
\[ F_{V,Rd} = 6 \times 280.5 / 1.25 = 1344 \text{ kN} > 1058 \text{ kN} \]

UPE web \( t = 5.5 \text{ mm} \)
additional plate \( t = 4 \text{ mm} \) => 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 \( f_{ub} (1000) > f_u (510 \text{ for S355}) \)
Values of parameters: \( e_1 = 70 \text{ mm} \) \( e_2 = 65 \text{ mm} \) \( p_2 = 50 \text{ mm} \)
\( \alpha_d = 70 / (3 \times 33) = 0.71 \) end bolt
\( \alpha_d = 70 / (3 \times 33) - 0.25 = 0.71 - 0.25 = 0.45 \) inner bolt
\( k_1 = (2.8 \times 65) / 33 - 1.7 = 3.8 \Rightarrow 2.5 \) edge bolt \( k_1: \) no inner bolts
Bearing resistance:
\[ 4 \times 2.5 \times 0.71 \times 30 \times 51 \times 9.5 / 1.25 + 2 \times 2.5 \times 0.45 \times 510 \times 30 \times 9.5 \]
\[ = 1087 \text{ kN} > 1058 \text{ kN} \]
1344 kN > 1.2 x 1087 = 1304 kN 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 $t_{\text{plate}} \times \sqrt{2/2} = 4 \times 0.707 = 3\text{mm}$

Resistance of a 3 mm weld: $(98,1\text{kN}:1.25)/100\text{mm} = 78.5\text{kN}/100\text{mm}$

Force to transmit: proportional to plate thickness:

$$(4 \times 1058)/(4+5.5) = 445\text{ kN}$$

Plate perimeter as from bolted connection:

$$2 \times (7 \times 70 + 160) = 1300\text{mm}$$

=> resistance = $13 \times 78.5 = 1020\text{ kN} > 445\text{ kN}$

Gusset: 10 mm thick plate (as UPE web + 4 mm plate = 9.5 mm)

Welds: length= $2 \times (7 \times 70 + 160 \times 0.707) = 1206\text{ mm} \times 2$ (2 sides)

$= 2412\text{ mm} = 24 \times 100\text{ mm}$

With $a = 4\text{mm fillet welds}$:

$$(24 \times 130.9)/1.25 = 2513\text{ kN} > 1058\text{ kN}$$
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
  - $\gamma_{ov} = 1,0$
- 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 $\varepsilon$ in all length $l$ provide high $dl = \varepsilon \times l$
  - Dissipative connections: $dl$ to be realised in the connection

- **$dl = \frac{d_r}{\cos \alpha}$**
  \[ \cos \alpha = \frac{l}{(l^2 + h^2)^{1/2}} \]
  \[ d_r \text{ interstorey drift} \quad d_r = q \times d_{re} \]

- **Example**
  \[ \frac{d_r}{h} = 3.5\% \quad l = 6 \text{ m} \quad h = 3 \text{ m} \]
  \[ \cos \alpha = 0.894 \quad d_r = 0.105 \quad dl = 117 \text{ mm} \]
  - Dissipative diagonals: $\varepsilon = 1.7\%$
  - Dissipative connections: required deformation capacity: $117 / 2 = 58.5 \text{ mm}$

=> 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:
  $$\Omega_i = \frac{R_{pl,Rd,i}}{N_{Ed,i}}$$
  $$\Omega_{\text{max}} \leq 1,25 \Omega_{\text{min}}$$
  $$\Omega = \Omega_{\text{min}}$$
- 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.
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

The end

Thank you for your attention!
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