Eurocodes
Background and Applications

Design of **Steel Buildings**
with worked examples

16-17 October 2014
Brussels, Belgium

Bolts, welds, column base

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Organised and supported by

European Commission
DG Enterprise and Industry
Joint Research Centre
European Convention for Constructional Steelwork
European Committee for Standardization
CEN/TC250/SC3
Motivation

To present
- Content/principles
- Selected particularities
- Questions

To offer
- Worked examples

for design according to **EN1993-1-8: 2005** of

- Bolts
- Welds
- Column bases
Scope of the lecture

**Bolts**
- General
- Design resistance of individual fasteners
  - Non-preloading bolts
  - Slotted holes
- Design for block tearing
- Worked example
- Summary

**Welds**
- General
- Fillet weld
  - Design model
  - Design example
  - Welding to flexible plate
- Summary

**Column bases**
- Basis of design
- Components
  - Base plate and concrete in compression
  - Base plate in bending and bolt in tension
  - Anchor bolt in shear
- Assembly
  - Resistance
  - Stiffness
- Classification
- Worked examples
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Connections made with bolts, rivets or pins in EN1993-1-8: 2005

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   3.1.1 General
   3.1.2 Preloaded bolts
3.2 Rivets
3.3 Anchor bolts
3.4 Categories of bolted connections
   3.4.1 Shear connections
   3.4.2 Tension connections
3.5 Positioning of holes for bolts and rivets
3.6 Design resistance of individual fasteners
   3.6.1 Bolts and rivets
   3.6.2 Injection bolts
3.7 Group of fasteners
3.8 Long joints
3.9 Slip resistant connections using 8.8 or 10.9 bolts
   3.9.1 Design Slip resistance
   3.9.2 Combined tension and shear
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3.10 Deductions for fastener holes
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   3.10.3 Angles connected by one leg and other unsymmetrically connected members in tension
   3.10.4 Lug angles
3.11 Prying forces
3.12 Distribution of forces between fasteners at the ultimate limit state
3.13 Connections made with pins
   3.13.1 General
   3.13.2 Design of pins
Bolt standards

**EN 15048**
Non-preloaded structural bolting assemblies
- Part 1: General requirements
- Part 2: Suitability test

**EN 14399**
High-strength structural bolting for preloading
High-strength structural bolting for preloading

EN 14399
Part 1: General requirements
Part 2: Suitability test for preloading
Part 3: System HR — Hexagon bolt and nut assemblies
Part 4: System HV — Hexagon bolt and nut assemblies
Part 5: Plain washers
Part 6: Plain chamfered washers
Part 7: System HR — Countersunk head bolt and nut assemblies
Part 8: System HV — Hexagon fit bolt and nut assemblies
Part 3: System HR — Hexagon bolt and nut assemblies
Part 4: System HV — Hexagon bolt and nut assemblies
Mechanical properties


Material

Bolts made of carbon steel and alloy steel: 4.6, 4.8, 5.6, 5.8, 6.8, 8.8, 10.9

Nuts made of carbon steel and alloy steel: 4, 5, 6, 8, 10, 12

Bolts made of austenitic stainless steel: 50, 70, 80

Nuts made of austenitic stainless steel: 50, 70, 80

Washers (if appropriate) according to hardness class HV 100 or HV 200
EN 1993-1-8
Material Properties

Nominal values for bolts

- $f_{yb}$ is yield strength
- $f_{ub}$ is ultimate tensile strength

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<td>240</td>
<td>320</td>
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<td>400</td>
<td>480</td>
<td>640</td>
<td>900</td>
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<tr>
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<td>400</td>
<td>400</td>
<td>500</td>
<td>500</td>
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Holes

- **Normal**
  - +1 mm for M 12
  - +2 mm for M 16 up M 24
  - +3 mm for M 27 and bigger

- **Oversized** with 3 mm (M12) up 8 mm (M27)

- **Slotted** (elongated)

- **Close fitting** – flushed bolts

  for bolt M20 must be the clearance $\Delta d < 0.3$ mm

EN 1090-2 Execution of steel structures and aluminium structures – Part 2: Technical requirements for steel structures
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Resistance in shear in one shear plane

\[ F_{v,Rd} = \frac{\alpha_v A f_{ub}}{\gamma_{M2}} \]

where the shear plane passes through the **unthreaded portion of the bolt**

\[ \alpha_v = 0.6 \]

- **A** is the gross cross section of the bolt
- **\( f_{ub} \)** is ultimate tensile strength for bolt
- **\( \gamma_{M2} \)** is partial safety factors for resistance of bolts
Resistance in shear in one shear plane

\[ F_{v,Rd} = \frac{\alpha_v A f_{ub}}{\gamma_{M2}} \]

where the shear plane passes through the **threaded portion** of the bolt

- for classes 4.6, 5.6 and 8.8:
  \( \alpha_v = 0.6 \)

- for classes 4.8, 5.8, 6.8 and 10.9
  \( \alpha_v = 0.5 \)

\( A \) is the tensile stress area of the bolt \( A_s \)
\( f_{ub} \) is ultimate tensile strength for bolt
\( \gamma_{M2} \) is partial safety factors for resistance of bolts
Resistance in bearing

\[ F_{b,Rd} = \frac{k_1 a_b d t f_u}{\gamma_M 2} \]

where \( \alpha_b \) is the smallest of \( \alpha_d \), \( \frac{f_{ub}}{f_u} \) or 1,0

**In the direction of load transfer**

- for end bolts: \( \alpha_d = \frac{e_1}{3 d_0} \), for inner bolts \( \alpha_d = \frac{p_1}{3 d_0} - \frac{1}{4} \)

**Perpendicular to the direction of load transfer**

- for edge bolts \( k_1 \) is the smallest of \( 2,8 \frac{e_2}{d_0} - 1,7 \) or 2,5

- for inner bolts \( k_1 \) is the smallest of \( 1,4 \frac{e_2}{d_0} - 1,7 \) or 2,5
Bearing of Plate and Bolt

Inner bolt

Outer bolt
Resistance in Bearing

Load on a bolt not parallel to the edge
the bearing resistance may be verified separately
for the bolt load components parallel and normal to the edge

in oversized holes reduce bearing by 0.8
Influence of distances to force

\[ F_{b,Rd} = \frac{k_1 \alpha_b d t f_u}{\gamma_{M2}} \]

Parallel to acting force

\[ \alpha_b = \min \left\{ \frac{e_1}{3d_0}, \frac{p_1}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1 \right\} \]
Perpendicular to acting force

\[ F_{b,Rd} = \frac{k_1 \alpha_b d t f_u}{\gamma_{M2}} \]

\[ k_1 = \min \left\{ \begin{array}{l}
2,8 \frac{e_2}{d_0} - 1,7 \\
1,4 \frac{p_2}{d_0} - 1,7 \\
2,5
\end{array} \right\} \]
Influence of distances

Nominal transversal distances:

\[ e_1 = 1,2 \, d_0 \]
\[ p_1 = 2,2 \, d_0 \]
\[ e_2 = 1,2 \, d_0 \]
\[ p_2 = 2,4 \, d_0 \]

\[ k_i = \min \begin{cases} 2,8 \frac{e_2}{d_0} - 1,7 \\ 1,4 \frac{p_2}{d_0} - 1,7 \\ 2,5 \end{cases} = \min \begin{cases} 2,8 \frac{1,2 \, d_0}{d_0} - 1,7 \\ 1,4 \frac{2,4 \, d_0}{d_0} - 1,7 \\ 2,5 \end{cases} = \min \begin{cases} 1,66 \\ 1,66 \\ 2,5 \end{cases} = 1,66 \]

End bolts

\[ \alpha_b = \frac{e_1}{3 \, d_0} = \frac{1,2 \, d_0}{3 \, d_0} = 0,400 \]

Internal bolts

\[ \alpha_b = \frac{p_1}{3 \, d_0} - 0,25 = \frac{2,2 \, d_0}{3 \, d_0} - 0,25 = 0,483 \]
Sum

Sum of resistances

\[ F_{b,\text{Rd}} = 2 \cdot 1,875 \frac{d t f_u}{\gamma_{M2}} + 2 \cdot 1,00 \frac{d t f_u}{\gamma_{M2}} = 5,75 \frac{d t f_u}{\gamma_{M2}} \]

Minimal resistance

\[ F_{b,\text{Rd}} = 4 \cdot 1,0 \frac{d t f_u}{\gamma_{M2}} = 4,0 \frac{d t f_u}{\gamma_{M2}} \]
Pitch distances

Min

\[ p_1 = 2,2 \, d_0 \]
\[ p_2 = 2,4 \, d_0 \]

optimum \( e_1 \) from 1,2 \( d_0 \) to 1,45 \( d_0 \)
\[ e_2 = 1,2 \, d_0 \]

\[
F_{b,Rd} = \frac{k_1 \, a_b \, d_t \, f_u}{\gamma_{M2}} = \frac{1,66 \cdot 0,483 \, d_t \, f_u}{\gamma_{M2}} = 0,802 \, \frac{d_t \, f_u}{\gamma_{M2}}
\]

Large

\[ p_1 = 3,75 \, d_0 \]
\[ p_2 = 3,0 \, d_0 \]
\[ e_1 = 3,0 \, d_0 \]
\[ e_2 = 1,5 \, d_0 \]

\[
F_{b,Rd} = \frac{k_1 \, a_b \, d_t \, f_u}{\gamma_{M2}} = \frac{2,5 \cdot 1,0 \, d_t \, f_u}{\gamma_{M2}} = 2,5 \, \frac{d_t \, f_u}{\gamma_{M2}}
\]
Steel S690

Experiment bolts M12 10.9 minimal and maximal pitch

\[ F_u / F_{EN3} = 1.27 \]
\[ F_u / F_{EN3} \leq 1.02 \]

Comparison to EN

\[ e_1 = 1.0 d_0 < \text{min. } e_1 = 1.2 d_0 \]
\[ e_1 = 2.4 d_0 \text{ and } p_1 = 3.2 d_0 \]
Comparison of simulation VL-100-220-385

Numerical model

- Good agreement
- Difference $D=5.5\%$
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Bearing resistance in slotted holes

Loaded perpendicular to the long direction of the slot

60% of resistance in circular holes
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Block tearing

Failure in shear at the row of bolts along the shear face of the hole group accompanied by Tensile rupture along the line of bolt holes on the tension face of the bolt group
Test

FE model

Topkaya C., 2004, A finite element parametric study on block shear failure of steel tension members, Journal of Constructional Steel Research, 60, 1615 – 1635, ISSN 0143-974X.
Design model

**Symmetric bolt group subject to concentric loading**

\[ V_{\text{eff,1,Rd}} = f_u A_{nt} / \gamma_{M2} + (1/\sqrt{3}) f_y A_{nv} / \gamma_{M0} \]

- \( A_{nt} \) net area subjected to tension
- \( A_{nv} \) net area subjected to shear

**Eccentric loading**

\[ V_{\text{eff,2,Rd}} = 0.5 f_u A_{nt} / \gamma_{M2} + (1/\sqrt{3}) f_y A_{nv} / \gamma_{M0} \]
In plate (staggered rows)

\[ V_{\text{eff,1,Rd}} = \frac{f_u A_{nt}}{Y_{M2}} + \frac{1}{\sqrt{3}} \frac{f_y A_{nv}}{Y_{M0}} \]

\[ = \frac{0.5 \cdot 530 \cdot (35 - 2.9) \cdot 10}{1,25 \cdot 10^3} + \frac{1}{\sqrt{3}} \cdot 220 \cdot \frac{(2 \cdot 240 - 7 \cdot 18 - 1.9) \cdot 10}{1,0 \cdot 10^3} = 36 + 362 = 397 \text{kN} \]

In angle (staggered rows)

\[ V_{\text{eff,2,Rd}} = \frac{0.5 f_{u,p} A_{nt}}{Y_{M2}} + \frac{1}{\sqrt{3}} \frac{f_{y,p} A_{nv}}{Y_{M0}} \]

\[ = \frac{0.5 \cdot 530 \cdot (60 - (18 + 9)) \cdot 10}{1,25 \cdot 10^3} + \frac{1}{\sqrt{3}} \cdot 220 \cdot \frac{(240 - 4 \cdot 18) \cdot 10}{1,0 \cdot 10^3} = 70 + 175 = 245 \text{kN} \]
Worked example –
Fin plate connection

\[ V_{Sd} = 100 \text{ kN} \]
**Worked Example – Fin Plate connection**

**Shear Resistance**

\[
V_{Rd,11} = \frac{0,5 f_{u,b1} A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}} f_{y,b1} \frac{A_{nv}}{\gamma_{M0}} = \frac{0,5 \cdot 360 \cdot (50 - 11) \cdot 7,1}{1,25 \cdot 10^3} + \frac{1}{\sqrt{3}} \cdot \frac{235 \cdot (220 - 2 \times 22 - 11) \cdot 7,1}{1,0 \cdot 10^3}
\]

\[= 39,9 + 159 \text{ kN} = 199 \text{ kN}\]
Worked example – Fin plate, Tying resistance

\[
N_{Rd,u} = \frac{f_{u,b1} A_{nt}}{\gamma_{M,u}} + \frac{1}{\sqrt{3}} f_{y,b1} A_{nv} \frac{A_{nt}}{\gamma_{M0}} = \frac{360 \times 7,1(140 - 2 \times 22)}{1,1} + \frac{1}{\sqrt{3}} \times 235 \times \frac{2 \times 7,1(50 - 11)}{1,0}
\]

= 223 + 75.1 = 298 kN
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Bolted connection of double angle bar

Bolts M20 class 5.6 fully treated
Loading $N_{Ed} = 400$ kN

Angle net section

$$N_{u,Rd} = \frac{0,9 A_{net} f_u}{\gamma_{M2}} = \frac{0,9 \cdot 2 \cdot (935 - 22 \cdot 6) \cdot 360}{1,25} = 416,3 \text{ kN} > N_{Ed} = 400 \text{ kN}$$

Satisfactory
Bolted connection of double angle bar

Bolts in shear

Two shear planes
Shear in bolt thread

Resistance for one bolt

\[ F_{v,Rd} = 2 \frac{\alpha_v A_s f_{ub}}{\gamma_{M2}} = 2 \cdot \frac{0,6 \cdot 245 \cdot 500}{1,25} = 117,6 \text{kN} \]
Bolted connection of double angle bar

Bearing of end bolt

\[ k_1 = \min \left( 2.8 \frac{e_2}{d_0} - 1.7; 2.5 \right) = \min \left( 2.8 \cdot \frac{35}{22} - 1.7; 2.5 \right) = \min(2.75; 2.5) \rightarrow k_1 = 2.5 \]

\[ \alpha_b = \min \left( \frac{e_1}{3d_0}, \frac{40}{3 \cdot 22}, \frac{500}{360}, 1.0 \right) = \min \left( 0.606, 1.0, 1.389 \right) = 0.606 \]

\[ F_{b,Rd} = \frac{k_1 \alpha d t f_u}{\gamma_{M2}} = \frac{2.5 \cdot 0.606 \cdot 20.8 \cdot 360}{1.25} = 87.3 \text{ kN} \]
Bolted connection of double angle bar

Bearing of internal bolt

\[ k_1 = \min \left( 2.8 \frac{e_2}{d_0} - 1.7; 2.5 \right) = \min \left( 2.8 \cdot \frac{35}{22} - 1.7; 2.5 \right) = \min(2.75; 2.5) \quad \rightarrow \quad k_1 = 2.5 \]

\[ \alpha_b = \min \left\{ \frac{p_1}{3 d_0} - \frac{1}{4}, \frac{70}{3 \cdot 22} - \frac{1}{4}, \frac{0.811}{10} \right\} = \min \left\{ \frac{500}{360}, 1.0, 1.0 \right\} = \min(1.389; 0.811) = 0.811 \]

\[ F_{b,Rd} = \frac{2.5 \cdot 0.811 \cdot 20 \cdot 8 \cdot 360}{1.25} = 93.4 \text{ kN} \]
Bolted connection of double angle bar

Check of bolts

- Shear resistance: 117.6 kN
- Bearing resistance – end bolt: 87.3 kN
- Bearing resistance – internal bolt: 93.4 kN

Shear is not guiding the resistance, e.g. bearing as sum
For connection with five bolts

\[87.3 + 3 \cdot 94.3 + 87.3 = 457.5 \text{ kN} > 400 \text{ kN} = N_{Ed}\]  

Satisfactory

Conservatively elastic (minimal) resistance

Lower resistance from bearings

\[5 \cdot 87.3 = 436.5 \text{ kN} > 400 \text{ kN} = N_{Ed}\]  

Unsatisfactory
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Summary for bolted connections

- Connections made with bolts, rivets or pins in Chapter 3 of EN 1993-1-8
- Non-preloaded bolts
- Preloaded bolts
  preload \((0.7 \, f_{ub})\)
- Injection bolts
  replacement of rivets; bolts 8.8 and 10.9
- Pins
  including serviceability
Future?

Advanced models of bolted connections

- **FEM research** models
  - Validated on experiments

- **FEM design** models
  - Verified against analytical and numerical models
  - Bars and springs model
    - In tension – stiffness, resistance
    - In shear – contact

Fan model of bolts
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# Welded connections in EN1993-1-8: 2005

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Electrode classification

EN 499 classification of carbon and low alloy steel electrodes

- Covered electrode / manual metal arc welding
- Strength & elongation
- Impact properties (47J at 0°C)
- Type of electrode covering (rutile-cellulosic)
- Recovery and type of current
- Welding position (all positions)

E 38 0 RC 1 1
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Fillet welds –
Definition of effective throat thickness $a$

The effective throat thickness of a fillet weld should not be less than 3 mm
Design Model of Fillet Welds

- $a$: effective throat thickness of the fillet weld
- $\sigma_\perp$: normal stresses perpendicular to the throat
- $\sigma_\parallel$: normal stresses parallel to the axis of weld (omitted)
- $\tau_\perp$: shear stresses perpendicular to the axis of weld
- $\tau_\parallel$: shear stresses parallel to the axis of weld
**Plane Stresses**

**Huber-Mises-Henckey condition of plasticity (HMH)**
- Triaxial state of stress (needed exceptionally only)
- Plane state of stress (needed very often)

\[
\sigma_x^2 + \sigma_z^2 - \sigma_x \sigma_z^2 + 3\tau^2 \leq \left(\frac{f_y}{\gamma_M}\right)^2
\]

- Uniaxial state of stress (from the material tests)

\[
\sigma \leq \frac{f_y}{\gamma_{M0}} \quad \tau \leq \frac{f_y}{(\gamma_{M0} \sqrt{3})}
\]
Design Resistance of Filet Weld

\[ \sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{II}^2)} \leq \frac{f_u}{(\beta_w \gamma_{Mw})} \]

\[ \sigma_{\perp} \leq \frac{f_u}{\gamma_{Mw}} \]

- \( f_u \): Ultimate tensile strength of connected material
- \( \beta_w \): Correlation factor
- \( \gamma_{Mw} \): Partial safety factor for material of welds
## Correlation factor $\beta_w$ for fillet welds

<table>
<thead>
<tr>
<th>Standard and steel grade</th>
<th>Correlation factor $\beta_w$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>EN 10025</strong></td>
<td></td>
</tr>
<tr>
<td>S 235</td>
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<tr>
<td>S 235 W</td>
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<tr>
<td>S 275</td>
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<td>S 275 N/NL</td>
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<td>S 275 M/ML</td>
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<td>S 355</td>
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<td>S 420 N/NL</td>
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<tr>
<td>S 420 M/ML</td>
<td></td>
</tr>
<tr>
<td>S 460 N/NL</td>
<td></td>
</tr>
<tr>
<td>S 460 M/ML</td>
<td></td>
</tr>
</tbody>
</table>

| **EN 10210**             |                             |
| S 235 H                  |                             |
| S 275 H                  |                             |
| S 275 NH/NLH             |                             |
| S 355 H                  |                             |
| S 355 NH/NLH             |                             |
| S 420 MH/MLH             |                             |

| **EN 10219**             |                             |
| S 235 H                  | 0,80                        |
| S 275 H                  | 0,85                        |
| S 355 H                  | 0,90                        |
| S 420 MH/MLH             | 1,00                        |
| S 460 NH/NLH             | 1,00                        |
Scope of the lecture

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Two fillet welds in parallel shear

\[ \tau_{\parallel} = \frac{F}{2a} \ell_{\parallel} \]

From plane stress analysis is

\[ \frac{F}{2a} \ell_{\parallel} \leq f_u / \left( \beta_w \gamma_{Mw} \sqrt{3} \right) \]

Throat thickness, not leg length
Fillet weld in normal shear

\[ \tau_{II} = 0 \]

\[ \sigma_{\perp} = \tau_{\perp} = \sigma_R / \sqrt{2} \]

Has to be satisfied

\[ \sqrt{\sigma_{\perp}^2 + 3 \tau_{\perp}^2} \leq f_u / (\beta_w \gamma_{Mw}) \]

After substitution

\[ \sqrt{\left(\sigma_R / \sqrt{2}\right)^2 + 3 \left(\sigma_R / \sqrt{2}\right)^2} = \sqrt{2} \sigma_R^2 \leq f_u / (\beta_w \gamma_{Mw}) \]

\[ \sigma_R \leq f_u / (\beta_w \gamma_{Mw} \sqrt{2}) \]
**Connection of cantilever**

Shear force \( V_{Sd} = F_{Sd} \).

Transferred by web \( \tau_{ll} = F_{Sd} / 2 a h \).

Bending moment \( M_{Sd} = F_{Sd} e \).

Transferred by the shape of weld.

Centre of gravity, \( I_{we} \) and cross section modulus \( W_{we} \).

For weld at lower flange cross section modulus \( W_{we,1} \) and stress is

\[ \sigma_{\perp 1} = \tau_{\perp 1} = \left( \frac{M_{Sd}}{\sqrt{2}} \right) / W_{we,1} \]

For upper weld on flange is

\[ \sigma_{\perp 2} = \tau_{\perp 2} = \left( \frac{M_{Sd}}{\sqrt{2}} \right) / W_{we,2} \]
Flange - web weld

Welds are loaded by longitudinal shear force

\[ V_\| = V_{sd} \frac{S}{I} \]

where \( V_{sd} \) shear force
\( S \) static moment of flange to neutral axis
\( I \) moment of inertia

This longitudinal force is carried by two welds effective thickness \( a \)

Shear stress

\[ \tau_\| = \frac{V_\|}{2a} \leq \frac{f_u}{\beta_w \gamma_{Mw}} \sqrt{3} \]

Maximum stress is at the point of maximum shear force
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Welds to flexible plate

Effective width of unstiffened column flanges

EN 1993-1-8 Chapter 4.10

\[ b_{\text{eff}} = t_{wc} + 2s + 7t_{fc} \]

\[ b_{\text{eff}} = t_{wc} + 2s + 7 \left( \frac{t_{fc}^2}{t_{fb}} \right) \left( \frac{f_{yc}}{f_{yb}} \right) \]

- \( t_{wc} \): thickness of column web
- \( t_{fc} \): thickness of column flange
- \( t_{fb} \): thickness of beam flange
- \( s \): equal to fillet radius \( r_c \) for hot rolled column sections
Effective Width

Unstiffened column flanges
In EN1993-1-8 Clause 6.2.4.4

\[
F_{t,fc,Rd} = \left( t_{wc} + 2s + 7k \cdot t_{fc} \right) \frac{t_{fb} \cdot f_{yb}}{\gamma_{M0}}
\]

\[
k = \min \left( \frac{f_{yc} \cdot t_{fc}}{f_{yb} \cdot t_{fb}} ; 1 \right)
\]

- \( t_{wc} \) is thickness of column web
- \( t_{fc} \) thickness of column flange
- \( t_{fb} \) thickness of beam flange
- \( s \) is equal to fillet radius \( r_c \) for hot rolled column sections
Scope of the lecture

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Summary

- Chapter 4 in EN1993-1-8: 2005
  - Rules for connection of open sections

- Chapter 7 in EN1993-1-8: 2005
  - Rules for connection of hollow sections

- New rules for high strength steels

- Size of welds
  - Weld design for full resistance
Weld design for full resistance

Loading by normal by connecting member

Not directly in code

\[ a > 0.7 \frac{\sigma t}{f_u / \gamma_{Mw}} \]

\( \sigma = F_{\text{sd}} / (t h) \)

\( F_{\text{sd}} \) is the acting design force

\( f_u \) is plate design strength

\( t \) is the thinness of connecting plate

\( b \) is width of connecting plate

Full capacity of a plate the thickness for steel S235

\[ a > 0.7 \frac{(f_y / \gamma_{M0}) t}{f_u / \gamma_{Mw}} = 0.7 \frac{(235/1,0) t}{360/1,25} = 0.47 t \approx 0.5 t \]
Weld design for full resistance

Loading by shear force by connecting member

\[ \tau = \frac{V_{sd}}{(t \cdot h)} \]

\( V_{sd} \) is the design shear force in weld

For full capacity of a plate thickness S235

\[ a > 0,85 \frac{\tau \cdot t}{f_w / \gamma_{Mw}} \approx 0,85 \frac{f_y / (\sqrt{3} \cdot \gamma_{M0}) \cdot t}{f_u / \gamma_{Mw}} = 0,85 \frac{235/(1,0 \cdot \sqrt{3}) \cdot t}{360/1,25} = 0,33 \cdot t \equiv 0,3 \cdot t \]
Weld Design
or Full Resistance of Connecting Members

Loading by normal force \( \sim 0.5 \text{ t} \)
Loading by shear force \( \sim 0.3 \text{ t} \)

Compare to AISC is less economical design

<table>
<thead>
<tr>
<th>AISC – LRFD – matching with SMAW</th>
<th>EN1993-1-8: 2005</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_y ) (N/mm(^2))</td>
<td>( f_{EXX} ) (ksi – N/mm(^2))</td>
</tr>
</tbody>
</table>
| 235 | 60 – 414
   70 – 483 | 0.37 t
   0.33 t | S235 | 208 | 0.37 t |
| 355 | 70 – 483 | 0.49 t | S355 | 262 | 0.45 t |
| 420 | 80 – 552 | 0.51 t | S420 M
   S420 N | 230
   240 | 0.61 t
   0.58 t |
| 485 | 90 – 621 | 0.52 t | S460 M
   S460 N | 245
   254 | 0.63 t
   0.61 t |
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Column bases in chap. 6 of EN 1993-1-8: 2005

Unfortunately in 6 clauses

- Resistance in cl. 6.2.8, cl. 6.2.5(7)
- Stiffness in cl. 6.3.1(4)
- Classification in cl. 5.2.2.5

- Component concrete block in compression and base plate in bending in cl. 6.7(2)
- Component anchor bolt in tension and base plate in bending in cl. 6.2.6.11(2)
- Component anchor bolt in shear cl. 6.2.2(6)
Component method
cl. 6.2.8

Components

- Concrete block in compression and base plate in bending
- Anchor bolt in tension and base plate in bending
- Component anchor bolt in shear
- Column web and flange in compression
Background materials

Wald F., Sokol Z., Steenhuis M. and Jaspart, J.P.,
**Component Method for Steel Column Bases**, 
*Heron*. 2008, vol. 53, no. 1/2, 3-20, ISSN 0046-7316

Steenhuis M., Wald F., Sokol Z. and Stark J.W.B.,
**Concrete in Compression and Base Plate in Bending**, 

Wald F., Sokol Z. and Jaspart J.P.,
**Base Plate in Bending and Anchor Bolts in Tension**, 

Gresnight N., Romeijn A., Wald F., and Steenhuis M.,
**Column Bases in Shear and Normal Force**, 

**EN 1992-4 Eurocode 2:**
Design of concrete structures — Part 4: Design of Fastenings for Use in Concrete
Scope of the lecture

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Component
Concrete in compression and base plate in bending

Design principles

- **3D behaviour of concrete** $D$
  - Design bearing strength of the joint $f_{jd}$
- **Flexible base plate on concrete block**
  - Effective rigid area under the flexible plate $A_{eff}$
- **Deformation of concrete block**
  - Stiffness coefficient for concrete deformation under the flexible plate $k_c$

Wald F., Sokol Z., Steenhuis M. and Jaspart, J.P.,
3D behaviour
Concentrated force for at concrete resistance $F_{Rd,u}$

- Design bearing strength of joint cl. 6.2.5(7)
  - Area of crushing of the concrete $A_{c0}$
  - According to cl. 6.7(2) in EN 1992-1-1

$$F_{Rd,u} = A_{c0} f_{cd} \sqrt{\frac{A_{c1}}{A_{c0}}} \leq 3,0 A_{c0} f_{cd}$$

$A_{c0} = b_1 d_1$
$A_{c1} = b_2 d_2$
$h \geq b_2 - b_1; h \geq d_2 - d_1$
$3 \cdot b_1 \geq b_2 \text{ a } 3 \cdot d_1 \geq d_2$
Cursing of the concrete
cl. 6.7(2) in EN 1992-1-1

Homogenous forces on area $A_{c0}$
Concrete design strength in joint

\[ f_{jd} = \frac{\beta_j \cdot F_{Rdu}}{b_{ef} \cdot l_{ef}} = \frac{\beta_j \cdot A_{c0} \cdot f_{cd} \cdot \sqrt{\frac{A_{c1}}{A_{c0}}}}{A_{c0}} = \beta_j \cdot f_{cd} \cdot \sqrt{\frac{A_{c1}}{A_{c0}}} \leq \frac{3,0 \cdot A_{c0} \cdot f_{cd}}{A_{c0}} = 3,0 \cdot f_{cd} \]

\( \beta_j = 2/3 \) is joint coefficient

\( F_{cd} \) is concrete compressive strength
Flexible base plate on concrete block

- Effective rigid plate
- Elastic deformation of plate only

Elastic bending moment of the base plate per unit length is

\[ M' = \frac{1}{6} t^2 \frac{f_y}{\gamma_{M0}} \]

and the bending moment per unit length on the base plate of span \( c \) and loaded by distributed load is

\[ M' = \frac{1}{2} f_j c^2 \]

where \( f_j \) is concrete bearing strength
Effective width of flexible plate $c$

Effective width 

$$c = t \sqrt{\frac{f_y}{3 f_{jd} \gamma_{M0}}}$$

where

$t$ is the plate thickness

$f_y$ is the base plate yield strength

$f_{jd}$ is the design bearing strength of the joint

$\gamma_{M0}$ is the partial safety factor for concrete

Effective area
Stiffness

Deformation of an elastic hemisphere

\[ \delta_r = \frac{F \alpha a_r}{\delta E} = \frac{0.85 F}{E_c \sqrt{a_{rl} L}} \]

Stiffness coefficient

\[ k_c = \frac{F}{\delta E} = \frac{E_c \sqrt{a_{eq,el} L}}{1.5 \cdot 0.85 E} = \frac{E_c \sqrt{a_{eq,el} L}}{1.275 E} \]

Width of effective T stub \( a_r \)
in elastic stage

\[ a_{eq,el} = t_w + 2.5 t \approx a_{eq,\text{st}} = \]

\[ = t_w + 2c = t_w + 2t \frac{f_y}{3 f_{jd} \gamma_{M0}} \]
Validation to experiments

![Graph showing force vs. deformation with data points for concrete and grout, concrete, and calculated strength.](image)

- Prediction based on local and global deformation,
- Prediction based on local deformation only
- Calculated strength

**Legend:**
- Concrete and grout
- Concrete
- Experiment
**Influence of grout**

- Grout higher quality than concrete block
  \[ \beta_1 = \frac{2}{3} \approx 1,0 \]

- Grout lower quality than concrete block
  - Model as plate on liquid

  \[ \beta_j = \frac{2}{3} \]
  
  \[ f_{c,g} \geq 0,2 \ f_c \]
  \[ t_g \leq 0,2 \ \text{min} \ (a ; b) \]
  \[ t_g \geq 0,2 \ \text{min} \ (a ; b) \]
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Component
Anchor bolts in tension and base plate in bending

- Base plate compare to end plate
- Base plate is thicker
- Anchor bolt is longer
- In most cases no prying forces
Contact of edge of T stub on experiment
Question of contact

- End plate – contact or no contact
- Base plate – no contact
Failure Mode 1-2

- $F / \Sigma B_{T,Rd}$

- $B_{t,Rd}$ is bolt tensile resistance
- $M_{pl,Rd}$ is base plate bending resistance of unique length
Free length of anchor bolt embedded in concrete

- Bolt effective free length $L_{be} = 8d$
- No prying force
  - For $Q = 0$
  - Limiting bolt length

$$L_{b,lim} = \frac{8.82m^3 A_s}{L_{eff} t^3} < L_b$$

Failure mode 1-2

Design resistance in collapse 1-2

\[ F_{T,1-2,Rd} = \frac{2 M_{pl,1,Rd}}{m} \]

where \( m \) is the lever arm of the anchor bolt

Plastic moment resistance

\[ M_{pl,1,Rd} = 0.25 \ell_{\text{eff}} t_f^2 / \gamma_{M0} \]

For the effective length \( \ell_{\text{eff}} \) the yield line method may be used.
Length of effective T stub

Bolts inside the flanges

For bolts inside the section

\[ \ell_1 = 2 \alpha m - (4m + 1.25e) \]
\[ \ell_2 = 4 \pi m \]

In case of prying

\[ \ell_1 = 2 \alpha m - (4m + 1.25e) \]
\[ \ell_2 = 2 \pi m \]
## Length of effective T stub

### Bolts outside the flanges

<table>
<thead>
<tr>
<th>Prying case</th>
<th>No prying case</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\ell_1 = 4 \alpha m_x + 1,25 e_x$</td>
<td>$\ell_1 = 4 \alpha m_x + 1,25 e_x$</td>
</tr>
<tr>
<td>$\ell_2 = 2 \pi m_x$</td>
<td>$\ell_2 = 2 \pi m_x$</td>
</tr>
<tr>
<td>$\ell_3 = 0.5 b_p$</td>
<td>$\ell_3 = 0.5 b_p$</td>
</tr>
<tr>
<td>$\ell_4 = 0.5 w + 2 m_x + 0.625 e_x$</td>
<td>$\ell_4 = 0.5 w + 2 m_x + 0.625 e_x$</td>
</tr>
<tr>
<td>$\ell_5 = e + 2 m_x + 0.625 e_x$</td>
<td>$\ell_5 = e + 2 m_x + 0.625 e_x$</td>
</tr>
<tr>
<td>$\ell_6 = \pi m_x + 2 e$</td>
<td>$\ell_6 = 2 \pi m_x + 4 e$</td>
</tr>
<tr>
<td>$\ell_7 = \pi m_x + w$</td>
<td>$\ell_7 = 2 (\pi m_x + w)$</td>
</tr>
<tr>
<td>$\ell_{\text{eff,1}} = \min (\ell_1; \ell_2; \ell_3; \ell_4; \ell_5; \ell_6; \ell_7)$</td>
<td>$\ell_{\text{eff,1}} = \min (\ell_1; \ell_2; \ell_3; \ell_4; \ell_5; \ell_6; \ell_7)$</td>
</tr>
<tr>
<td>$\ell_{\text{eff,2}} = \min (\ell_1; \ell_2; \ell_3; \ell_4; \ell_5)$</td>
<td>$\ell_{\text{eff,2}} = \min (\ell_1; \ell_2; \ell_3; \ell_4; \ell_5)$</td>
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</table>
Length of effective T stub for hollow sections

\[
L_{\text{eff}.1} = \pi m \\
L_{\text{eff}.2} = \frac{b}{2} \\
L_{\text{eff}.3} = \frac{a}{2} \\
L_{\text{eff}.4} = \sqrt{\left(\frac{a-a_c}{2}\right)^2 + \left(\frac{b-b_c}{2}\right)^2} - \sqrt{e_a^2 + e_b^2} \\
L_{\text{eff}.5} = \min\left(L_{\text{eff}.1}, L_{\text{eff}.2}, L_{\text{eff}.3}, L_{\text{eff}.4}, L_{\text{eff}.5}\right)
\]


Stiffness coefficients

of component anchor bolts in tension and base plate in bending

For base plate of thickness $t$

$$k_p = \frac{0.425 \ell_{\text{eff}} t^3}{m^3}$$

For bolt

$$k_b = 2.0 \frac{A_s}{L_b}$$
Comparison to experiments
Stiffens and resistance of anchor bolt with header plate

Model anchor bolts for resistance in CEB documents
Comparison to experiments

Wald F., Sokol Z., Jaspart J.P.,
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Components in Shear
In EN1993-1-8 cl. 6.2.2(6) - Anchor Bolt in Shear

Simplification by limiting of shear

\[ F_{2,\text{vb},\text{Rd}} = \frac{\alpha_b f_{ub} A_s}{\gamma_{Mb}} \]

where \( f_{ub} \) is bolt design strength (in range 640 MPa \( \geq f_{ub} \geq 235 \text{ N/mm}^2 \))

\[ \alpha_b = 0.44 - 0.0003 f_{yb} \]

\[ \gamma_{Mb} \] is safety factor for bolts

Gresnigt N., Romeijn A., Wald F., Steenhuis M.,
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Assembling of components for bending resistance

- Plastic design
- Force equilibrium

\[
\frac{M_{\text{Ed}}}{z} - \frac{N_{\text{Ed}}}{z} z_{c,r} = F_{t,l,Rd}
\]

\[
\frac{M_{\text{Ed}}}{z} + \frac{N_{\text{Ed}}}{z} z_{T,1} = F_{C,R,Rd}
\]

Wald F., Sokol Z., Steenhuis M. and Jaspart J.P.,
M - N interaction diagram
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Assembling of components for bending stiffness

From the component deformation stiffness for two cases

- Bolts activated
- Bolts not activated

Stiffness

Simplified contact area round the axes of compressed flange

\[ M_{Sd} / N_{Sd} = \text{konst.} \]

\[ S_j = \frac{M_{Sd} / N_{Sd}}{M_{Sd} / N_{Sd} - \alpha} \frac{E z^2}{\mu \sum \frac{1}{k_i}} \]

\[ \alpha = \frac{z_c k_c - z_t k_t}{k_c + k_t} \]

\[ \mu = (1.5 \gamma)^{2.7} \]

\[ \gamma = \frac{1 + \frac{r / 2}{M_{Sd} / N_{Sd}}}{M_{Rd} / N_{Sd} + \frac{r / 2}{M_{Sd} / N_{Sd}}} \]
History of loading

- Influence to stiffness
- No influence to resistance
Sensitivity study of base plate thickness

![Graph showing moment vs. rotation for different base plate thicknesses.](image-url)
Validation on experiment

Validation on experiment

Proportional loading, bolt failure

Nonproportional loading, concrete failure
Scope of the lecture

Bolts
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Classification

According to stiffness

Asked accuracy of design
5% in resistance
10% in serviceability

Similar to beam-to-column joints

Wald F., Sokol Z., Steenhuis M. and Jaspart, J.P.,
Non-sway frames by resistance

\[ \beta = \sqrt{\frac{F_{cr,\text{pin}}}{F_{cr,\text{res}}}} \]

\[ S_{j,\text{ini,\text{pin}}} = 7 \, 100 \, \text{kNm} / \text{rad} \]

\[ S_{j,\text{ini,\text{stif}}} = 74 \, 800 \, \text{kNm} / \text{rad} \]
Sway frames for serviceability

\[ \frac{y_s}{y_p} \]

\[ \begin{align*}
S_{j,ini,pin} \\
S_{j,ini,stif}
\end{align*} \]

\[ \log S \]

\[ 115 \text{ kN} \]

\[ 5 \text{ kN} \]

\[ HE 200 B \]

\[ 5 \text{ m} \]

\[ 4 \text{ m} \]
**Classification based on stiffness**

Accuracy - 5% for resistance and 20% for serviceability

\[ S_{j,ini,c,n} = \frac{30EI}{L_c} \]

\[ S_{j,ini,c,s} = \frac{12EI}{L_c} \]

\[ \lambda_0 = 1.36 \]
Non-sway frames by resistance

For \( \bar{\lambda}_o \leq 0,5 \) \( S_{j,ini} \geq 0 \)
For \( 0,5 < \bar{\lambda}_o < 3,93 \) \( S_{j,ini} \geq 7 (2 - 1) E I_c / L_c \)
and for \( \bar{\lambda}_o \geq 3,93 \) \( S_{j,ini} \geq 48 E I_c / L_c \)
where \( \bar{\lambda}_o \) is relative stiffness for simple supported column at both ends
For limited stiffness \( 12 E I_c / L_c \)

For sway frames \( S_{j,ini} \geq 30 E I_c / L_c \)
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Worked example – **Simple base plate**

Base plate resistance?

- Column HE 200 B
- Concrete block 850 x 850 x 900 mm concrete C 12/15
- Base plate 18 mm, steel S 235
- $\gamma_c = 1,50, \gamma_{M0} = 1,00$
Concrete strength in joint

Under the base plate

\[ f_{jd} = \beta_j \cdot f_{cd} \cdot \sqrt{\frac{A_c}{A_{c0}}} = \beta_j \cdot f_{cd} \cdot \sqrt{\frac{b_2 \cdot d_2}{b_1 \cdot d_1}} = \frac{2}{3} \cdot 12/1.5 \cdot \sqrt{\frac{850 \cdot 850}{340 \cdot 340}} = \]

\[ f_{jd} = = 13.3 \text{ MPa} \leq 3.0 \cdot f_{cd} = 3.0 \cdot 12/1.5 = 24 \text{ MPa} \]
Plate effective width

\[ c = t \sqrt{\frac{f_y}{3 f_j \gamma_M^0}} = 18 \cdot \sqrt{\frac{235}{3 \cdot 13.4 \cdot 1.00}} = 43.7 \text{ mm} \]
Base plate compression resistance

Effective area under I cross section

\[ A_{\text{eff}} = \min (b; b_c + 2c) \cdot \min (a; h_f + 2c) - \max [\min (b; b_c + 2c) - t_f - 2c; 0] \cdot \max (h_c - 2t_f - 2c; 0) \]

\[ A_{\text{eff}} = (200 + 2 \cdot 43.7) \cdot (200 + 2 \cdot 43.7) - \\
- (200 + 2 \cdot 43.7 - 9 - 2 \cdot 43.7) \cdot (200 - 2 \cdot 15 - 2 \cdot 43.7) = \\
= 82599 - 15777 = 66722 \text{ mm}^2 \]

The base plate resistance

\[ N_{Rd} = A_{\text{eff}} f_{jd} = 66722 \cdot 13.3 = 887 \cdot 10^3 \text{ N} \]
Work example – rigid base plate

Bending resistance of base plate?

- Column HE 200 B loaded by $F_{Ed} = 500$ kN
- Concrete block C16/20 of $1600 \times 1600 \times 1000$ mm
- Base plate $30$ mm from steel S235
- $\gamma_c = 1.50; \gamma_{M0} = 1.00$ and $\gamma_{Mb} = 1.25$
Component
Anchor bolt and plate in bending

Bolt resistance
for M 24; \( A_s = 253 \text{ mm}^2 \)

\[
F_{T,3Rd} = 2 \cdot B_{t,Rd} = 2 \cdot \frac{0,9 \cdot f_{ub} \cdot A_s}{\gamma_{mb}} =
\]
\[
= 2 \cdot \frac{0,9 \cdot 360 \cdot 353}{1,25} = 183,0 \cdot 10^3 \text{ N}
\]
Component anchor bolt and plate in bending

Bolt distance for weld 6 mm

\[ m = 60 - 0,8 \cdot a_{wf} \cdot \sqrt{2} = 60 - 0,8 \cdot 6 \cdot \sqrt{2} = 53,2 \text{ mm} \]

Length of effective T stub

\[
\begin{align*}
4 \pi m &= 4 \pi 53,2 = 668,6 \\
0,5 \ b &= 0,5 \cdot 420 = 210 \\
4 \ m + 1,25 \ e_a &= 4 \cdot 53,2 + 1,25 \cdot 50 = 275,3 \\
4 \ \pi \ m &= 4 \ \pi 53,2 = 668,6 \\
0,5 \ b &= 0,5 \cdot 420 = 210 \\
2 \ m + 0,625 \ e_b + 0,5 \rho &= 2 \cdot 53,2 + 0,625 \cdot 90 + 0,5 \cdot 240 = 282,7 \\
2 \ m + 0,625 \ e_b + e_a &= 2 \cdot 53,2 + 0,625 \cdot 90 + 50 = 212,7 \\
2 \ \pi \ m + 4 \ e_b &= 2 \ \pi 53,2 + 4 \cdot 90 = 694,2 \\
2 \ \pi \ m + 2 \ \rho &= 2 \ \pi 53,2 + 2 \cdot 240 = 814,2
\end{align*}
\]

\[ \ell_{eff,1} = 210 \text{ mm} \]

T stub resistance

\[
F_{T,1-2,Rd} = \frac{2 \ L_{eff,1} \ t^2 f_y}{4 \ m \ \gamma_{M0}} = \frac{2 \cdot 210 \cdot 30^2 \cdot 235}{4 \cdot 60 \cdot 1,00} = 370,0 \cdot 10^3 \text{ N}
\]
Component
Concrete block and plate in bending

Concrete crushing resistance

\[ f_{jd} = \beta_j \cdot f_{cd} \sqrt{\frac{A_{c1}}{A_{c0}}} = \beta_j \cdot f_{cd} \sqrt{\frac{b_2 \cdot d_2}{b_1 \cdot d_1}} = \frac{2}{3} \cdot 16/1,5 \cdot \sqrt{\frac{1420 \cdot 1420}{420 \cdot 420}} = \]

\[ = 24,0 \text{ MPa} \leq 3,0 \cdot f_{cd} = 3,0 \cdot 16/1,5 = 32 \text{ MPa} \]

Force equilibrium

\[ F_{Ed} = A_{eff} \cdot f_{j} - F_{T,Rd} \]

Contact force for full bolt resistance

\[ A_{eff} = \frac{F_{Ed} + F_{Rd,3}}{f_{jd}} = \frac{500 \cdot 10^3 + 183,0 \cdot 10^3}{24,0} = 28,458 \text{ mm}^2 \]
Effective width of base plate

\[ c = t \sqrt{\frac{f_y}{3 f_{jd} \gamma_{M0}}} = 30 \cdot \sqrt{\frac{235}{3 \cdot 24,0 \cdot 1,00}} = 54,2 \text{ mm} \]

The effective width of the area in contact

\[ b_{\text{eff}} = \frac{A_{\text{eff}}}{b_c + 2c} = \frac{28,458}{200 + 2 \cdot 54,2} = 92,3 \text{ mm} < t_f + 2c = 15 + 2 \cdot 54,2 = 123,4 \text{ mm} \]
Bending moment resistance for acting normal force

Lever arm

\[
 r_c = \frac{h_c}{2} + c - \frac{b_{\text{eff}}}{2} = \frac{200}{2} + 54,2 - \frac{92,3}{2} = 108,1 \text{ mm}
\]

Bending resistance for normal force \( F_{\text{Ed}} = 500 \text{ kN} \)

\[
 M_{\text{Rd}} = F_{T,3,\text{Rd}} \ r_b + A_{\text{eff}} \ f_{\text{jd}} \ r_c
 = 183,0 \cdot 10^3 \cdot 160 + 28458 \cdot 24,0 \cdot 108,1 =
 = 103,1 \cdot 10^6 \text{ Nmm} = 103,1 \text{ kNm}
\]
Bending stiffness

Stiffness coefficient

- Anchor bolt

\[ k_b = 2,0 \frac{A_s}{L_b} = 2,0 \frac{353}{261,5} = 2,7 \text{ mm} \]

- Base plate

\[ k_b = \frac{0,425 \cdot L_{b,\text{eff}} \cdot t^3}{m^3} = \frac{0,425 \cdot 210 \cdot 30^3}{53,2^3} = 16,0 \text{ mm} \]
Bending stiffness

Stiffness coefficient

\[ k_c = \frac{E_c}{1,275 E_s} \sqrt{a_{eq} b_c} = \frac{27500}{1,275 \cdot 210000} \sqrt{90 \cdot 200} = 13,8 \text{ mm} \]

Lever arm in tension \( z_t \) and in compression \( z_c \) to column neutral axes

\[ r_t = \frac{h_c}{2} + e_c = \frac{200}{2} + 60 = 160 \text{ mm} \]

For part in tension

\[ z_c = \frac{h_c}{2} - \frac{t_f}{2} = \frac{200}{2} - \frac{15}{2} = 92,5 \text{ mm} \]

\[ k_t = \frac{1}{k_b} + \frac{1}{k_p} = \frac{1}{2,7} + \frac{1}{16,0} = 2,310 \text{ mm} \]
### Bending stiffness

**Lever arm**

\[ r = r_t + r_c = 160 + 92.5 = 252.5 \text{ mm} \]

\[ a = \frac{k_c r_c - k_t r_t}{k_c + k_t} = \frac{13.8 \cdot 92.5 - 2.3 \cdot 160}{13.8 + 2.3} = 56.4 \text{ mm} \]

**Eccentricity**

\[ e = \frac{M_{Rd}}{F_{Ed}} = \frac{103.1 \cdot 10^6}{500 \cdot 10^3} = 206.2 \text{ mm} \]

**Bending stiffness**

\[ S_{j,ini} = \frac{e}{e + a} \cdot \frac{E_s r^2}{\mu \sum_{i} \frac{1}{k_i}} = \frac{206.2}{206.2 + 56.4} \cdot \frac{210000 \cdot 252.5^2}{1 \cdot \left( \frac{1}{2.31} + \frac{1}{13.78} \right)} = \]

\[ = 20,799 \text{ Nmm/rad} = 20\,799 \text{ kNm/rad} \]
Classification

Bending stiffness for column HE 200 B of length \( L_c = 4,0 \) m

\[
\bar{S}_{j,\text{ini}} = S_{j,\text{ini}} \frac{L_c}{E_s I_c} = 20,799 \cdot 10^9 \frac{4000}{210000 \cdot 56,96 \cdot 10^6} = 6,96
\]

the base plate is **semi-rigid** for braced frames

\[
\bar{S}_{j,\text{ini}} = 6,96 < 12 = \bar{S}_{j,\text{ini,EC3,n}}
\]

and also for unbraced frames

\[
\bar{S}_{j,\text{ini}} = 6,96 < 30 = \bar{S}_{j,\text{ini,EC3,s}}
\]
Base plate M-N interaction diagram

- Normal force, kN
- Moment, kNm

- End column resistance
- HE 200 B
- \( t = 30 \)
- \( h = 1 \, 000 \)
- \( M_{24} \)
- \( 1600 \, 630 \)
- \( 630 \, 340 \)
- \( 1600 \)
Influence of contact in column web

- Normal force, kN
  - One bolt row activated
  - Both bolt rows activated

- Base plate thickness, mm
  - 15
  - 20
  - 25
  - 30

- Moment, kNm

- Column end resistance

- Contact of flanges only

- Model contact column web and flanges

- Contact of flanges only
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Summary

- Component method
  - Good accuracy of prediction

- In EN 1993-1-8 in 6 closures

- Plastic distribution of forces for resistance
  - Concrete 3D strength
  - Effective rigid area under flexible base plate

- Questions for next edition
  - Embedded columns?
  - Column bases with base plate and anchor plate?
  - Advanced models?
Component based model of column base with base plate and **anchor plate**

RFCS project INFASO+
- Design Manual I
- Design Manual II
- Software toll

http://www.steelconstruct.com/site
http://steel.fsv.cvut.cz/infaso
Component based FEM

- Advanced design model based on FEM
- Components integrated FEM
  - Welds
  - Bolts
  - Compressed plates

Allows for column bases
- Generally loaded by $N$ and $M_y$, $M_z$
- Irregular shape of base plate
- Arbitrary positions of anchors
- Opening in column web

Material nonlinear design model of plate

Fan model of anchor bolt

http://www.idea-rs.com
Thank you for your attention

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Design of **Steel Buildings**
with worked examples

16-17 October 2014
Brussels, Belgium

Eurocodes
Background and Applications

Organised and supported by

European Commission
  DG Enterprise and Industry
  Joint Research Centre
European Convention for Constructional Steelwork
European Committee for Standardization
CEN/TC250/SC3