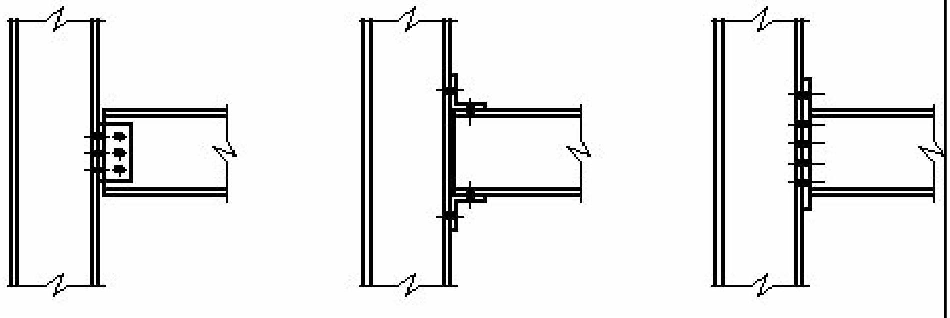
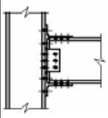


# Ligações Viga x Coluna

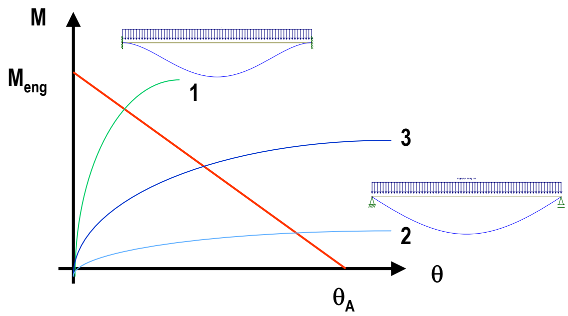

**Programa de Pós-Graduação em Engenharia Civil**
**PGECIV - Mestrado Acadêmico**
**Faculdade de Engenharia – FEN/UERJ**
**Disciplina: Tópicos Especiais em Projeto**
**Professor: Luciano Rodrigues Ornelas de Lima**



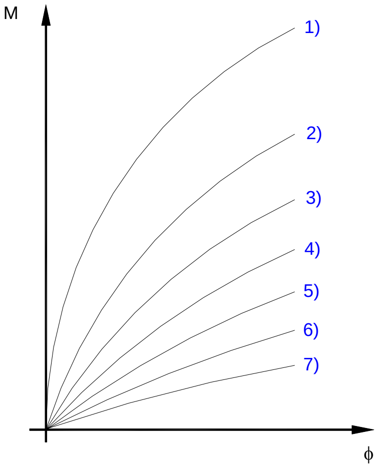
## 1. Introdução

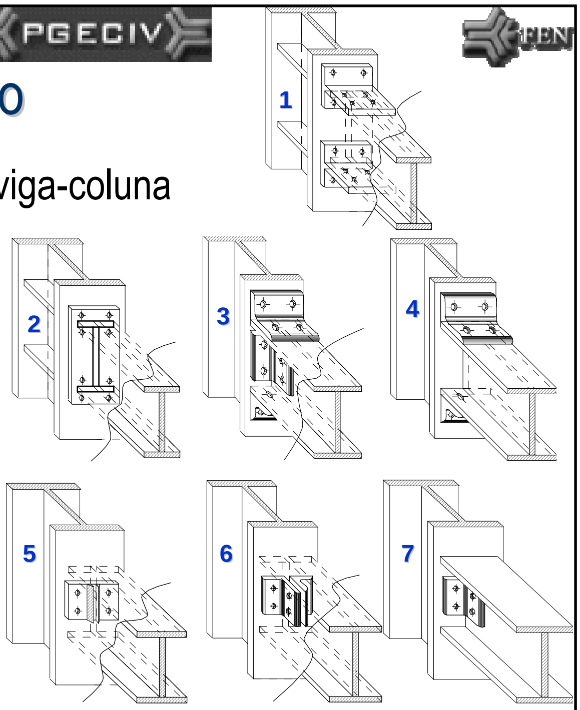
- capacidade de transmitir momentos
  - ✓ rígida (tipo 1) ⟶ $M_t = M_i$
  - ✓ flexível (tipo 2) ⟶ $M_t = 0$
  - ✓ semi-rígida (tipo 3) ⟶ $0 < M_t < M_i$



# 1. Introdução

- Tipos de ligações viga-coluna

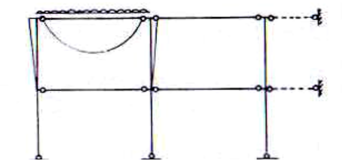


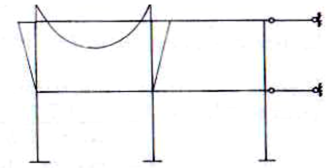



# 1. Introdução

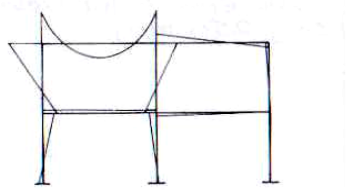
- Deslocável x Indeslocável
  - ✓ tipo II ⟶ Austrália e USA
    - flexível ⟶ deslocável
    - flexível ⟶ cargas gravitacionais
    - rígida ⟶ cargas laterais
  - ✓semi-rígidas ⟶ M x ϕ
    - não possui solução fechada
    - tema de pesquisa
  - ✓flexíveis ⟶ resist. L / h ≤ 20
  - ✓rígidas ⟶ esbeltas ⟶ rigidez



(a) flexível ⟶ + baratas (restrição de deslocamentos com contraventamentos e apoios



(b) semi-rígida



(c) rígida

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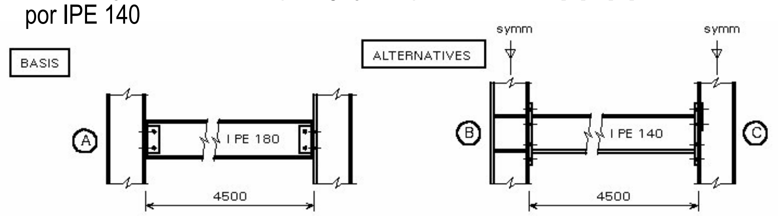
# 1. Introdução

- Vantagem → capacidade de transmitir momentos
  - ✓ viga em pórtico contraventado
  - ✓ projeto 1 → ligações para resistir ao cisalhamento [A]
  - ✓ substituição das “rótulas” por ligações que resistem M [B]& [C] → IPE 180 trocado por IPE 140




| | | |
|---|---|---|
| • Less steel | -16 kg | -22 kg |
| • More welding: | | |
| 4,5m 5 ≈ | +79 kg | |
| 1,1m 5 ≈ | | +20 kg |
| • Fabrication of plates etc | +79 kg | +20 kg |
| • Extra holes | | |
| +2 holes ≈ | + 4 kg | |
| +6 holes ≈ | | +12 kg |
| Difference | +146 kg | +30 kg |



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# 1. Introdução

- Some other aspects which facilitate economy in design are:
  - ✓ Limit the number of bolt diameters, bolt lengths and bolt grades as far as possible. Use for instance standard M20 bolts in grade 8.8
  - ✓ Ensure good access so that welds can be made easily
  - ✓ Minimize situations where precise fitting is required
  - ✓ Achieve repetition of standard details
  - ✓ Provide ease of access for site bolting
  - ✓ Provide means for supporting the self weight of the piece quickly, so that the crane can be released
  - ✓ Achieve ease of adjustment for alignment
  - ✓ Consider maintenance where necessary

7

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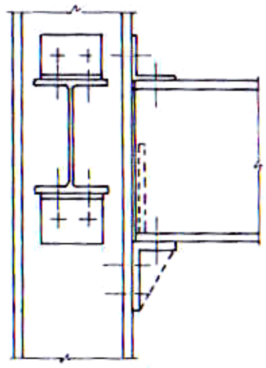
# 2. Ligações Flexíveis

- Transmissão do cortante ⟶ Q
  - ✓ “sitting” (cantoneira de apoio) ⟶ mais econômica para fabricantes automatizados
  - ✓ overlap ⟶ viga x cantoneira
  - ✓ enrijecedor ⟶ compressão
  - ✓ parafusos ⟶ M + Q (excentricidade e)
  - ✓ L de topo não contribui p/ resistência ao cortante (flexíveis)



(a)

8

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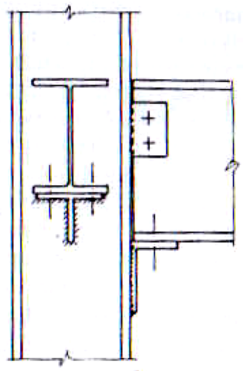
# 2. Ligações Flexíveis

- Transmissão do cortante ⟶ Q
  - ✓ “welded cleats” (cantoneira soldada) ⟶ “T” x “L”
  - ✓ aumentar área de solda na menor inércia
  - ✓ cuidado no transporte / montagem ⟶ danos
  - ✓ pequena L na alma ⟶ estabilidade à torção

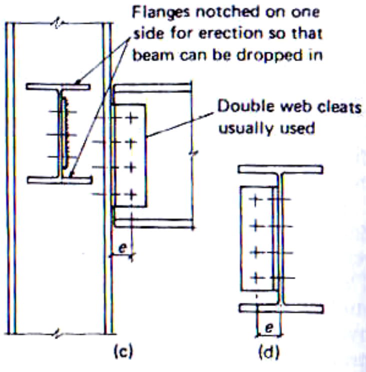


(b)

9

## 2. Ligações Flexíveis

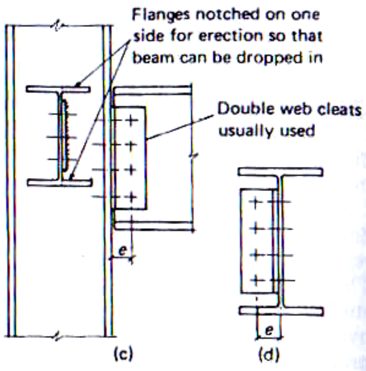
- Transmissão do cortante → Q
  - ✓“web cleats” (cantoneira de alma) → “L” ou placas (“fin plates”) → aparafusadas ou soldadas
  - ✓mesas → pequena solda → montagem
  - ✓excentricidade e → duplo (l > 0,6 h) → não há problema
  - ✓e pequenos → dupla excentricidade




10

## 2. Ligações Flexíveis

- Transmissão do cortante → Q
  - ✓fin plates → capacidade de rotação → parafusos 8.8 → rasgamento → deformação controla (t < 0,5 D)
  - ✓distância a borda ≤ 2 D
  - ✓resistência da solda > momento transmitido pelos parafusos
  - ✓Ductilidade
    - $l_w$ = 1,2 t (643)
    - $l_w$ = 1,4 t (650)



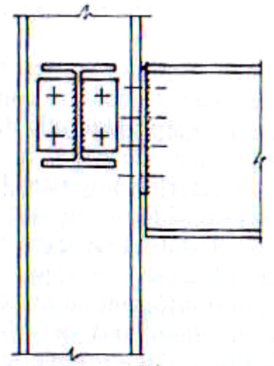
11

# 2. Ligações Flexíveis

- Transmissão do cortante → Q

  ✓placas de extremidade → dimensões precisas / ajustes → $h_{placa} = h_{viga}$ (maior momento de inércia) → caso contrário → cuidados no transporte → evitar dano

  ✓ t ≤ D / 2 (8.8)



(e)

12

# 2. Ligações Flexíveis

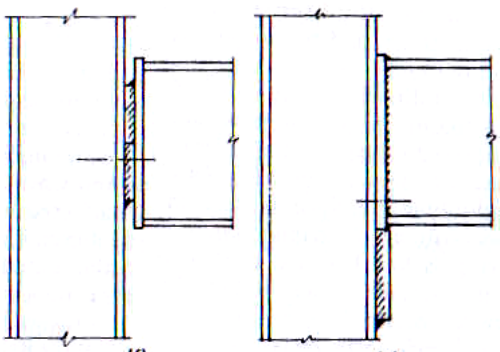
- Transmissão do cortante → Q

  ✓"shear plate"

  ✓f → mais trabalhosa → não existe solda fora da viga

  ✓g → endplate + shear plate → menos solda

  ✓parafusos → centro de rotação → cuidados com escorregamento



(f) (g)

13

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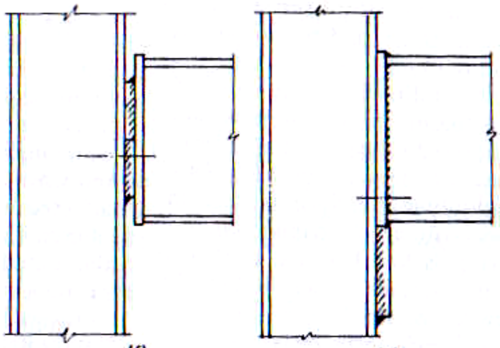
## 2. Ligações Flexíveis

- Transmissão do cortante → Q

✓parafusos suplementares → evitar distorção

✓semi-rígidas

- altura da edificação controla o dimensionamento
- rigidez e não resistência



(f) (g)

14

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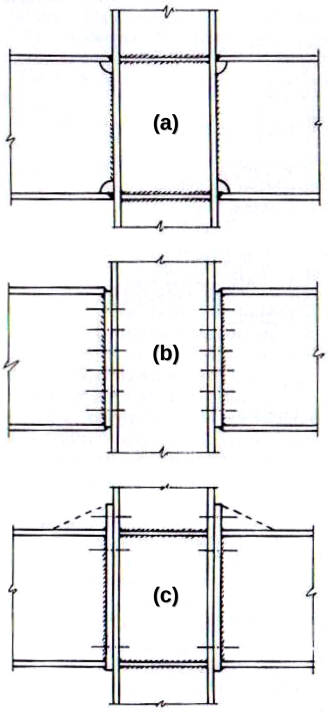
## 3. Ligações Rígidas

- Edifícios Comerciais / Residenciais

✓a → soldas de campos e furos

✓b e c → placas de extremidade

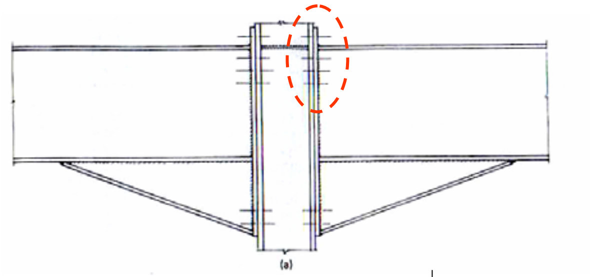
- Estendidas → mais eficientes → $M_{pl}$ e $M_y$ da viga
- h < 500 mm → 6 parafusos 8.8
- $M_{requerido} \leq 0{,}7$ a $0{,}8\ M_y$ → "T" (tração) e "C" (compressão)
- parafusos mesa comprimida → efeito mola → transmitir Q
- $M > 0{,}8M_y$ → alma deve contribuir → enrijecedores triangulares

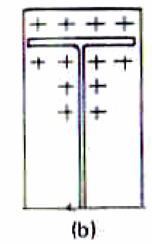


15

# 3. Ligações Rígidas

- Edifícios Industriais ⟶ momentos elevados




✓parafusos adicionais ⟶ aumentar o braço de alavanca

✓mísula ⟶ limitação de altura

✓mesa misulada ⟶ compressão

✓alma da mísula ⟶ Q

✓solda de resistência total

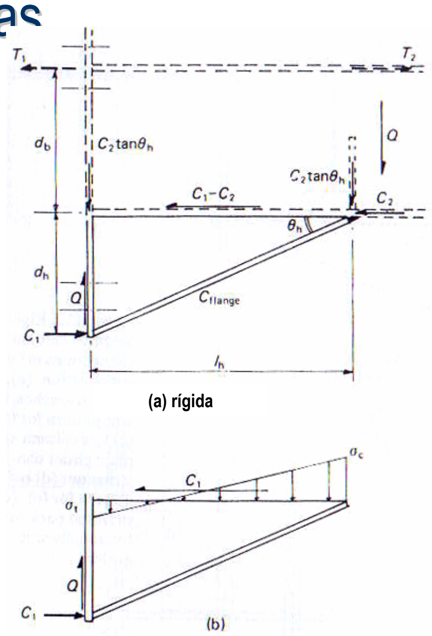
16

# 3. Ligações Rígidas

- Edifícios Industriais

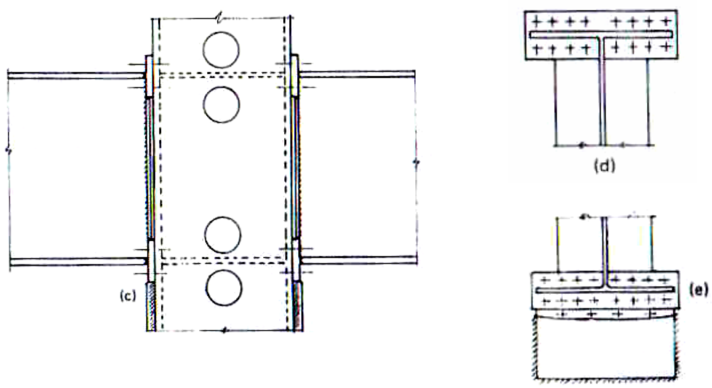
✓comportamento da mísula

  - alma ⟶ Q
  - $C_1$ ⟶ cisalhamento + flexão
  - cisalhamento vertical através da alma da viga ⟶ $C_1$, $C_2$, $T_1$ e $T_2$ ⟶ $\Sigma$M em [1] e [2]



# 3. Ligações Rígidas

- Edifícios Industriais → momentos elevados
  - ✓coluna com seção tubular ou "plate girder"





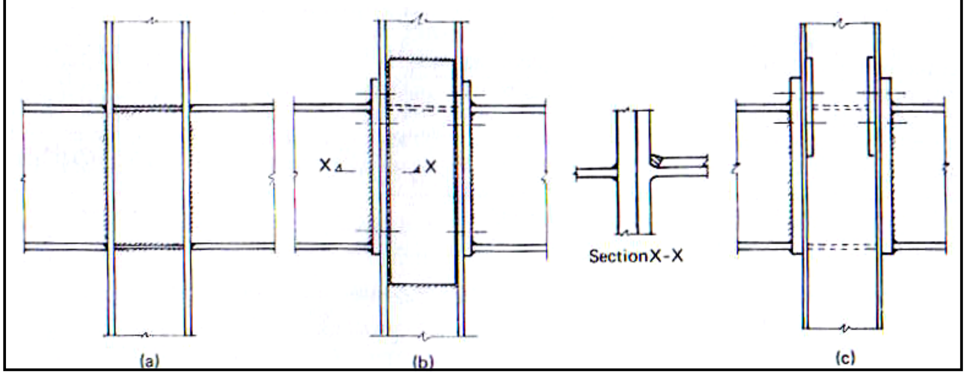

# 3. Ligações Rígidas

- Enrijecedores

✓colunas internas → + comum (a)

✓placa na alma da coluna → preparação da solda (bujão)

✓mesa tracionada → *"backing plates"*



19

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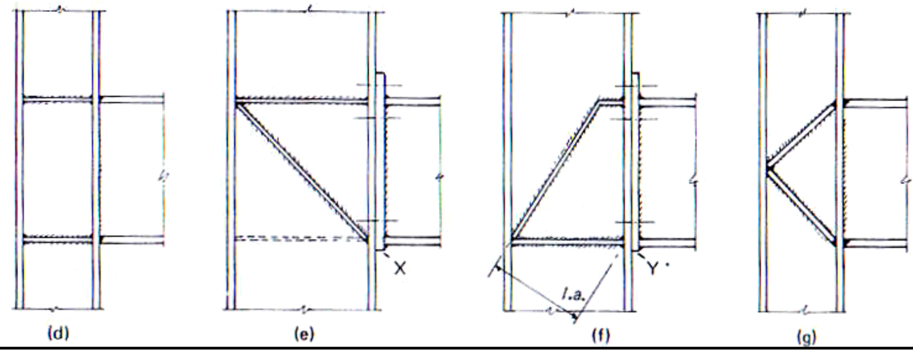
# 3. Ligações Rígidas

- Enrijecedores

✓colunas externas → + comum (d)
✓cisalhamento não transmitido
✓enrijecedor de compressão (e)
✓projeto do enrijecedor diagonal

✓"Morris stiffener" (f) → redução do braço de alavanca → escoamento por compressão → ruína
✓$H_c << H_{viga}$ → 30º<$\alpha$<60º (melhor 45º)




20

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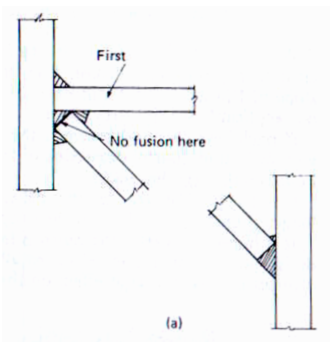
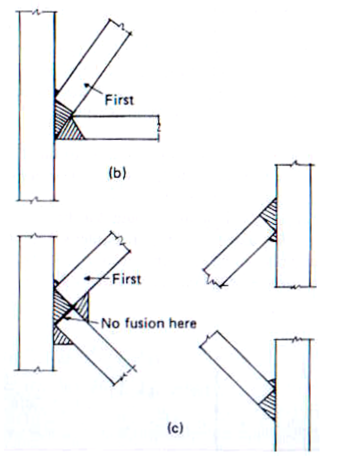
# 3. Ligações Rígidas

- Detalhes das Soldas

✓ 1º enrijecedor tracionado
✓ 2º enrijecedor comprimido
✓ realizadas em fábrica

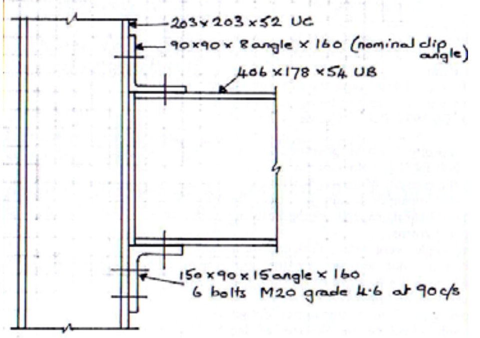


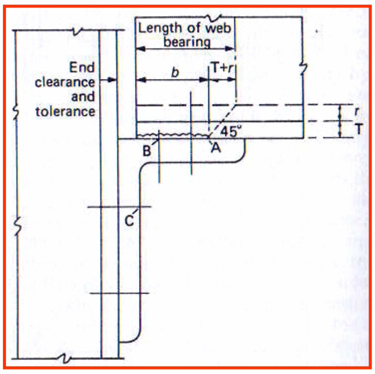



21

# 4. Exemplos (1)

Design a seating cleat connection for a 406 × 178 × 54 UB in Grade 43 steel, to carry a reaction of 145 kN (due to factored loads.) The connection is to the flange of a 203 × 203 × 52 UC in Grade 43 steel.




**Seating cleat**

Length of bearing required at root line of beam $= \frac{R}{t\,p_{yw}} = \frac{145 \times 10^3}{7{\cdot}6 \times 275} = 69{\cdot}4$ mm

Length of bearing on cleat $= b = 69{\cdot}4 - (T + r)$
$= 69{\cdot}4 - (10{\cdot}9 + 10{\cdot}2) = 48{\cdot}3$ mm

End clearance of beam from face of column = 5 mm
Allow 5 mm tolerance

Try 150 × 90 × 15 angle × 160 mm long

Distance from end of bearing on cleat to root of angle (A to B) $= b + 5 + 5 - (t + r_1)$ of angle
$= 48{\cdot}3 + 5 + 5 - (15 + 12)$
$= 31{\cdot}3$ mm

22

# 4. Exemplos (1)

Assume uniformly distributed load over bearing length 'b'.

Moment at root of angle (point B) due to load to right of 'B' $= \frac{145 \times 31{\cdot}3}{48{\cdot}3} \times \frac{31{\cdot}3}{2} = 1471$ Nm

Moment capacity $= 1{\cdot}2\, p_y Z$
$= 1{\cdot}2 \times 275 \times \frac{160 \times 15^2}{6} \times 10^{-3}$
$= 1980$ Nm $> 1471$ Nm  o.k.

Shear capacity of outstand leg of cleat
$= 0{\cdot}6\, p_y \times 0{\cdot}9\, wt = 0{\cdot}6 \times 275 \times 0{\cdot}9 \times 160 \times 15 \times 10^{-3}$
$= 356$ kN $> 145$ kN  o.k.

150 × 90 × 15 seating angle × 160 mm long

Try 4/M20 Grade 4·6 bolts for connection to column flange.

Shear capacity $= 4 \times 160 \times 245 \times 10^{-3}$
$= 156{\cdot}8$ kN
$> 145$ kN  o.k

Bearing capacity (of bolts) on column flange
$= 4 \times 435 \times 20 \times 12{\cdot}5 \times 10^{-3} = 435$ kN
$> 145$ kN  o.k

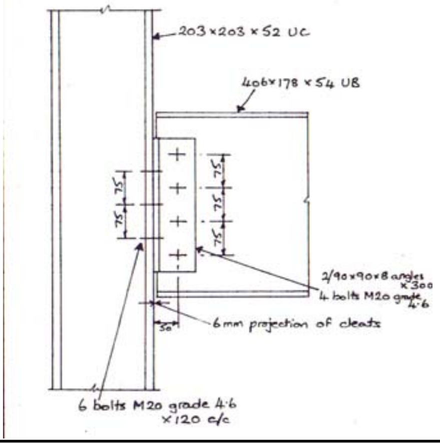
M20 bolts grade 4·6

23

# 4. Exemplos (2)

Beam to column connection – Bolted web cleats

Design a bolted web cleat connection for a 406 × 178 × 54 UB in Grade 43 steel to carry a reaction of 145 kN (due to factored loads). The connection is to the flange of a 203 × 203 × 52 UC in Grade 43 steel.




Connection to web of beam.
For M20 Grade 4.6 bolts

Shear capacity of bolt in double shear
$= 2 \times 160 \times 245 \times 10^{-3} = 78.4$ kN

Bearing capacity (of bolt) on web of beam
$= 435 \times 20 \times 7.6 \times 10^{-3} = 66.1$ kN

Try 4 bolts at 75 mm vertical pitch, 50 mm from heel of cleats

Horizontal shear force on bolt due to moment due to eccentricity
$$= \frac{145 \times 50 \times 112.5}{2(37.5^2 + 112.5^2)} = 29.0 \text{ kN}$$

Vertical shear force per bolt $= \frac{145}{4} = 36.3$ kN

Resultant shear force $= \sqrt{29.0^2 + 36.3^2}$
$= 46.4$ kN
$<$ bolt capacity O.K

$\boxed{\text{4/M20 bolts Grade 4.6}}$

Connection to column flange.
For M20 Grade 4.6 bolts

Shear capacity of bolt in single shear
$= 160 \times 245 \times 10^{-3} = 39.2$ kN

24

# 4. Exemplos (2)

Bearing capacity (of bolt) on 8mm cleat
$= 435 \times 20 \times 8 \times 10^{-3} = 69.6$ kN

Try 6 bolts, 2 rows at 75 mm vertical pitch and 120 mm c/c.

Assuming centre of pressure 25mm below top of cleat, horizontal shear force on bottom bolt due to moment due to eccentricity
$$= \frac{145 \times 0.5 (120 - 7.6) \times 200}{2(50^2 + 125^2 + 200^2)} = 14.0 \text{ kN}$$

Vertical shear force per bolt
$= \frac{145}{6} = 24.2$ kN

Resultant shear force $= \sqrt{14.0^2 + 24.2^2}$
$= 28.0$ kN
$<$ bolt capacity o.k.

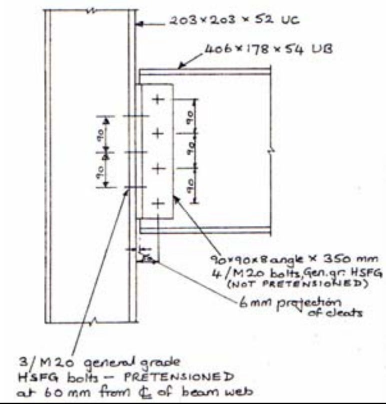
Use 2/90×90×8 angle cleats × 300mm

$\boxed{\text{6/M20 bolts Grade 4.6; 2/90×90×8 angle cleats × 300 mm}}$

25

# 4. Exemplos (3)

Beam to column connection – single bolted web cleat.
Design a bolted connection for a 406 × 178 × 54 UB in Grade 43 steel, using a single web cleat, to carry a reaction of 145 kN (due to factored loads). The connection is to the flange of a 203 × 203 × 52 UC in Grade 43 steel.




Connection to web of beam.
For M20 General Grade HSFG bolts (BS 4395 part 1) used as ordinary bolts (i.e. not pretensioned).

Shear capacity of bolt in single shear
$= 0{\cdot}48 \times 827 \times 245 \times 10^{-3} = 97{\cdot}3$ kN

Bearing capacity on web of beam
$= 460 \times 20 \times 7{\cdot}6 \times 10^{-3} = 69{\cdot}9$ kN

Try 4 bolts at 90 mm vertical pitch, 50 mm from heel of cleat.

Horizontal shear force on bolt due to moment due to eccentricity
$= \dfrac{145 \times 50 \times 135}{2(45^2 + 135^2)} = 24{\cdot}2$ kN

Vertical shear force per bolt $= \dfrac{145}{4} = 36{\cdot}3$ kN

Resultant shear force $= \sqrt{24{\cdot}2^2 + 36{\cdot}3^2}$
$= 43{\cdot}6$ kN
< bolt capacity O.K

4/M20 General Grade HSFG bolts (not pretensioned)

Connection to column flange
Try M20 General Grade HSFG bolts (BS 4395, part 1) used as HSFG bolts.

Slip resistance (per bolt)
$= 1{\cdot}1 \times 0{\cdot}45 \times 144 = 71{\cdot}3$ kN

Bearing resistance of 8 mm cleat (per bolt)
$= 20 \times 8 \times 825 \times 10^{-3} = 132{\cdot}0$ kN

Try 3 bolts at 90 mm vertical pitch, 60 mm from centre line of beam web

Horizontal shear force on bolt due to moment due to eccentricity $= \dfrac{145 \times 60 \times 90}{2 \times 90^2} = 48{\cdot}3$ kN

26

# 4. Exemplos (3)

Connection to column flange
Try M20 General Grade HSFG bolts (BS 4395, part 1) used as HSFG bolts.

Slip resistance (per bolt)
$= 1{\cdot}1 \times 0{\cdot}45 \times 144 = 71{\cdot}3$ kN

Bearing resistance of 8 mm cleat (per bolt)
$= 20 \times 8 \times 825 \times 10^{-3} = 132{\cdot}0$ kN

Try 3 bolts at 90 mm vertical pitch, 60 mm from centre line of beam web

Horizontal shear force on bolt due to moment due to eccentricity $= \dfrac{145 \times 60 \times 90}{2 \times 90^2} = 48{\cdot}3$ kN

Vertical shear force per bolt
$= \dfrac{145}{3} = 48{\cdot}3$ kN

Resultant shear force $= \sqrt{48{\cdot}3^2 + 48{\cdot}3^2}$
$= 68{\cdot}3$ kN
< bolt capacity O.k

3/M20 General Grade HSFG bolts

Cleat angle.
Try 90×90×8 angle × 350 mm
Check bending at bolt line of connection to column flange

Bending moment $= 145 \times 60 = 8700$ Nm

Moment capacity $= p_y Z$
$= 275 \times \dfrac{8 \times 350^2}{6} \times 10^{-3}$
$= 44917$ Nm
> bending moment
O.k. Use

90×90×8 angle cleat × 350

27

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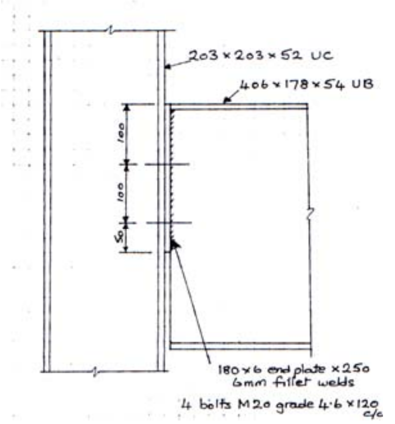
# 4. Exemplos (4)

Beam to column connection — welded end plate.

Design a welded end plate connection for a 406 × 178 × 54 UB in Grade 43 steel to carry a reaction of 145 kN (due to factored loads). The connection is to the flange of a 203 × 203 × 52 UC in Grade 43 steel.




Bolted connection to column.

For M20 Grade 4·6 bolts and a 6 mm thick end plate.

Shear capacity of bolt in single shear
$= 160 \times 245 \times 10^{-3} = 39{\cdot}2$ kN

Bearing capacity (of bolt) on 6 mm end plate $= 435 \times 20 \times 6 \times 10^{-3} = 52{\cdot}2$ kN

Minimum number of bolts $= \frac{145}{39{\cdot}2} = 3{\cdot}7$

Use 4 bolts.

4/M20 bolts Grade 4·6

End plate

Use 180 mm × 6 mm plate × 250 mm deep

Length of fillet welds connecting end plate to beam web $= 250 - 10{\cdot}9 - 8$
$= 231$ mm each side

180×6 end plate × 250

Shear on weld $= \frac{145}{2 \times 231} = 0{\cdot}31$ kN/mm

Use 6mm fillet welds

Shear stress on web of beam
$= \frac{145 \times 10^3}{231 \times 7{\cdot}6} = 83$ N/mm²
$\leq 0{\cdot}6 p_y$ O.K

6 mm Fillet welds.

28

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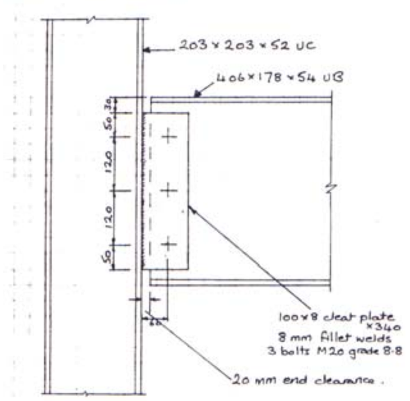
# 4. Exemplos (5)

Beam to column connection — welded web cleat.

Design a web cleat connection (welded to the column and site bolted to the beam) for a 406 × 178 × 54 UB in Grade 43 steel to carry a reaction of 145 kN (due to factored loads). The connection is to the flange of a 203 × 203 × 52 UC in Grade 43 steel.




Connection to web of beam.

Try M20 Grade 8·8 bolts and a 8 mm thick cleat plate.

Shear capacity of bolt in single shear
$= 375 \times 245 \times 10^{-3} = 91{\cdot}9$ kN

Bearing capacity of connected ply
$= 460 \times 20 \times 7{\cdot}6 \times 10^{-3} = 69{\cdot}9$ kN

Minimum number of bolts $= \frac{145}{69{\cdot}9} = 2{\cdot}1$

Use 3 bolts

Cleat plate

Try 100 × 8 plate × 340 mm, with bolt line 60 mm from face of column

Check that proposed cleat is acceptable for eccentricity effects due to offset of beam web.

Minimum thickness $= \frac{5 \times \text{Reaction}}{p_y \times \text{depth of cleat}}$

$= \frac{5 \times 145 \times 10^3}{275 \times 350} = 7{\cdot}5$ mm
$< 8$ mm. Thickness O.K

3/M20 bolts Grade 8·8

29

# 4. Exemplos (5)

Moment at face of column due to eccentricity of load = 145×60 = 8700 Nm

Moment that could be developed by bolts (at their bearing capacity) = 2×69.9×120 = 16776 Nm

Design for moment of 16776 Nm.

Moment capacity of plate = $275 \times \frac{8\times 340^2}{6} \times 10^{-3}$ = 42387 Nm > 16776 Nm. O.K.

100 × 8 plate × 340 mm

Effective length of welds = 340−2×6 = 328 mm

Horizontal shear at end of weld due to moment = $\frac{16776\times 6}{2\times 328^2}$ = 0.468 kN/mm

Vertical shear = $\frac{145}{2\times 328}$ = 0.221 kN/mm

Resultant shear = $\sqrt{0.468^2 + 0.221^2}$ = 0.52 kN/mm

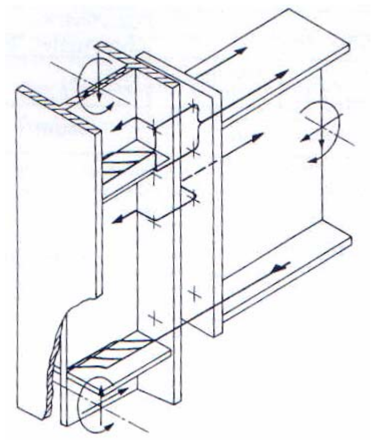
Minimum weld size for "ductility" = $\frac{0.6t}{0.7} = \frac{0.6\times 8}{0.7}$ = 6.9 mm

Use 8 mm fillet welds

8 mm fillet welds

30

# 4. Exemplos (6)



Beam to column connection — Bolted end plate.

Design a connection between a 610×229×101 UB beam in Grade 43 steel and a 305×305×158 UC column in Grade 43 steel.

The connection is to carry the following factored loads:

| | | |
|---|---|---|
| Bending moment | 423 | kNm |
| Axial load in beam | 22 | kN (tension) |
| Shear in beam | 254 | kN |
| Shear in column (above connection) | 26 | kN |