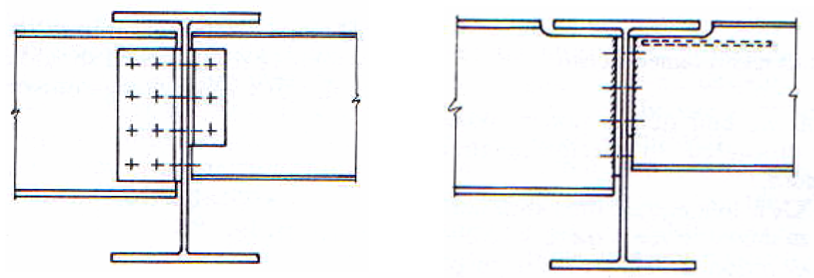
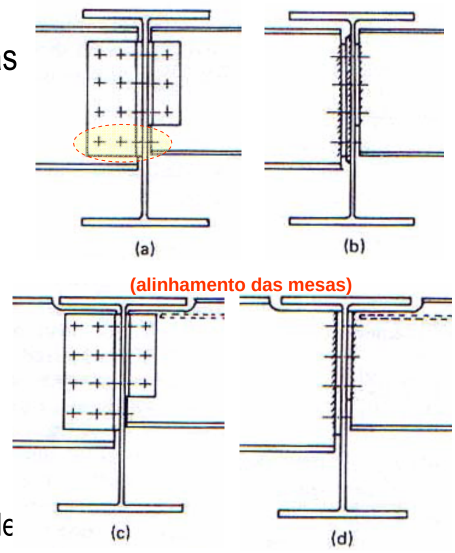


Ligações Viga x Viga

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

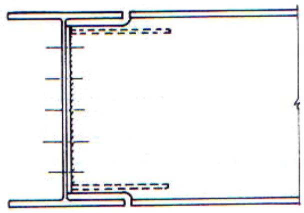
1. Introdução

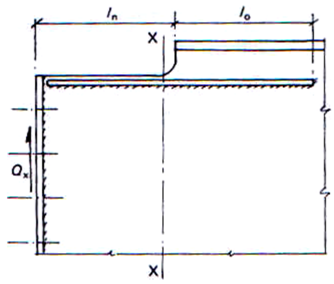
- Simples → cisalhamento das vigas secundárias para as vigas principais
  - ✓ alma fina
  - ✓ alinhamento dos parafusos / duplo corte
  - ✓ dificuldades de montagem - parafuso adicional do lado esquerdo
  - ✓ placas de extremidade
    - montagem → "welding packs"
    - solda (a) → senão, flexão nos parafusos
  - ✓ recortes x "shear block" → simples ou duplo → necessidade de enrijecedores



1. Introdução

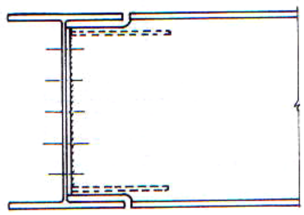
- Simples ⟶ cisalhamento das vigas secundárias para as vigas principais
  - ✓ vigas com mesma altura (secundária e principal) ⟶ efetuar recorte
  - ✓ momento $M = Q \times l_n$
    - Recorte simples ⟶ M resistido pelo "T" formado pela alma e mesa inferior da viga
    - Duplo ⟶ M resistido pela alma da viga apenas
  - ✓ $\sigma_{\text{fibras superiores}} \leq \sigma_{y\_comp}$
    - $\lambda_{\text{flamb. local}} = 1{,}5 l_n / (t_w / \sqrt{12})$

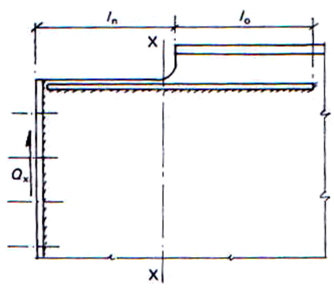




1. Introdução

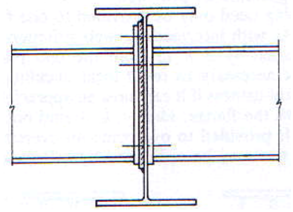
- ✓ comprimento efetivo ⟶ 1,5 l_n (deslocamentos para fora do plano no final do recorte)
- ✓ comprimento efetivo ⟶ 0,7 l_n (deslocamentos restringido)
- ✓ seção XX ⟶ enrijecedores
  - cargas laterais de montagem;
  - efeito de temperatura ⟶ viga comprimida
  - torsor ⟶ "overload" ($l_0 \geq l_n$)
  - apenas em uma face da alma
  - distância $3t_w$ da placa de extremidade

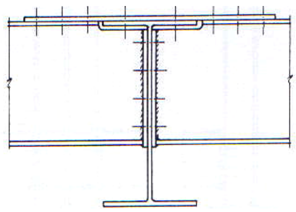




1. Introdução

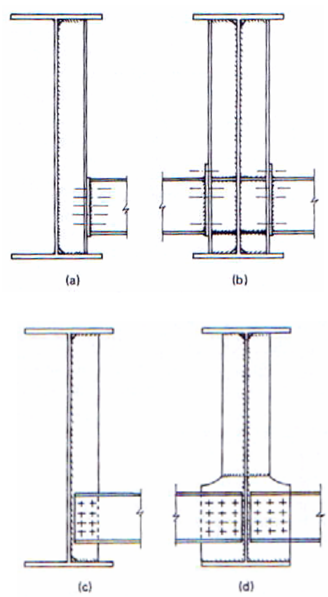
- Rígidas → cisalhamento das vigas secundárias para as vigas principais + continuidade de M
  - ✓ vigas secundárias em alturas diferentes das principais → + econômica com placas de extremdiade
  - ✓ "packing plates" → montagem (ligações simples)
  - ✓ mesas superiores coplanares → combinação de emenda + placa de extremidade





1. Introdução

- Vigas Principais x Vigas "Cross-Girder"
  - ✓ transferência de M e Q
  - ✓ restrição lateral contra flambagem
  - ✓ parte inferior → região de momento negativo
  - ✓ parte superior → região de momento positivo
  - ✓ placas de extremidades
  - ✓ outros tipos

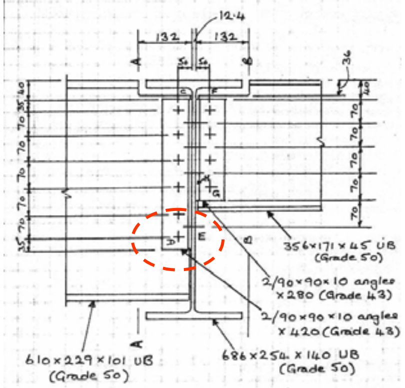


7

2. Exemplos (1)

Beam to beam connection with bolted web cleats

Design bolted web cleat connections for a 610×229×101 UB (540 kN reaction due to factored loads) and a 356×171×45 UB (228 kN reaction due to factored loads) connected to the web of a 686×254×140 UB. All the beams are in Grade 50 steel



18 bolts M20 grade 8.8 (bolts in outstand legs at 120 mm cross centres)

For M20 Grade 8.8 bolts

Shear capacity of bolt in single shear
$= 375 \times 245 \times 10^{-3} = 91.9$ kN

Shear capacity in double shear
$= 2 \times 91.9 = 183.8$ kN

Left hand beam (610×229×101 UB)

Connection to web of left hand beam.

Bearing capacity of web of beam
$= 550 \times 20 \times 10.6 \times 10^{-3}$
$= 116.6$ kN

Try 6 bolts at 70mm vertical pitch, 50mm from heel of cleats.

Horizontal shear force on bolt due to moment due to eccentricity
$$= \frac{540 \times (50 + 0.5 \times 12.4) \times 175}{2(35^2 + 105^2 + 175^2)}$$
$= 61.9$ kN

Vertical shear force per bolt $= \frac{540}{6} = 90$ kN

Resultant shear force $= \sqrt{61.9^2 + 90^2}$
$= 109.2$ kN
< bolt capacity O.K

6/M20 bolts Grade 8.8

Connection to web of supporting beam

Bearing capacity on 10mm cleat (grade 43)
$= 460 \times 20 \times 10 \times 10^{-3} = 92$ kN

Single shear capacity of 91.9 kN governs

Try 8 bolts, 2 columns at 120mm c/c (vertical pitch as in sketch)

Assume centre of pressure 25mm below top of cleat.

8

2. Exemplos (1)

Horizontal shear force on bottom bolt due to moment due to eccentricity
$$= \frac{540 \times 0.5(120 - 10.6) \times 325}{2(45^2 + 115^2 + 185^2 + 325^2)}$$
$= 30.9$ kN

Vertical shear force per bolt $= \frac{540}{8} = 67.5$ kN

Resultant shear force $= \sqrt{30.9^2 + 67.5^2}$
$= 74.2$ kN
< bolt capacity O.K

Use 2/90×90×10 angle cleats ×420

8/M20 bolts Grade 8.8
2/90×90×10 angle cleats ×420 mm

Right hand beam (356×171×45 UB)

Connection to web of right hand beam

Bearing capacity of web of beam
$= 550 \times 20 \times 6.9 \times 10^{-3} = 75.9$ kN

Try 4 bolts at 70mm vertical pitch, 50 mm from heel of cleats

Horizontal shear force on bolt due to moment due to eccentricity
$$= \frac{228 \times 50 \times 105}{2 \times (35^2 + 105^2)} = 48.9 \text{ kN}$$

Vertical shear force per bolt $= \frac{228}{4} = 57.0$ kN

Resultant shear force $= \sqrt{48.9^2 + 57.0^2}$
$= 75.1$ kN
< bolt capacity O.K

4/M20 bolts Grade 8.8

Connection to web of supporting beam.
Try 6 bolts, 2 columns of 3 bolts at 120 mm c/c, 70 mm pitch.

Assuming centre of pressure 25 mm below top of cleat, horizontal shear force on bottom bolt due to moment due to eccentricity
$$= \frac{228 \times 0.5(120 - 6.9) \times 185}{2(45^2 + 115^2 + 185^2)}$$
$= 24.1$ kN

Vertical shear force per bolt $= \frac{228}{6} = 38$ kN

Resultant shear force $= \sqrt{24.1^2 + 38^2}$
$= 45.0$ kN
< bolt capacity O.K

Use 2/90×90×10 angle cleats ×280

6/M20 bolts Grade 8.8
2/90×90×10 angle cleats ×280 mm

Web of supporting beam. Check for combined load from left and right hand beams

Bearing capacity on 12.4 mm web
$= 550 \times 20 \times 12.4 \times 10^{-3}$
$= 136.4$ kN

Total horizontal shear force due to moment due to eccentricity (on third row)
$$= \frac{30.9 \times 185}{325} + 24.1 = 41.7 \text{ kN}$$

Total vertical shear force $= 67.5 + 38.0$
$= 105.5$ kN

Resultant shear force $= \sqrt{41.7^2 + 105.5^2}$
$= 113.4$ kN
< bearing capacity O.K

Left hand beam.
At end of notch (A–A)
Depth of section $= 602.2 - 36 = 566.2$ mm

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2. Exemplos (1)

Shear capacity
$= 0{\cdot}6 \times 355 \times 0{\cdot}9 \times 566{\cdot}2 \times 10{\cdot}6 \times 10^{-3}$
$= 1151$ kN

Shear force = 540 kN < shear capacity O.K.

Slenderness of compression zone of web
$= \dfrac{1{\cdot}5\,(132-50)}{10{\cdot}6/\sqrt{12}} = 40{\cdot}2$

Compressive strength = 300 N/mm²

Moment capacity $> 300 \times \dfrac{10{\cdot}6 \times 566{\cdot}2^2}{6} \times 10^{-6}$
$= 170$ kNm

Moment $= 540 \times \left(132 + \dfrac{12{\cdot}4}{2}\right) \times 10^{-3}$
$= 74{\cdot}6$ kNm
< moment capacity O.K.

Check shear capacity through bolt holes (C-D-E)

Block shear capacity
= shear capacity of C-D + ½ (tensile capacity of D-E)
$= 0{\cdot}6 \times 355 \times 0{\cdot}9 \times 1{\cdot}1\,(5 \times 70 + 39 - 5{\cdot}5 \times 22) \times 10{\cdot}6 \times 10^{-3}$
$+ 0{\cdot}5 \times 355 \times 1{\cdot}1\,(48 - 0{\cdot}5 \times 22) \times 10{\cdot}6 \times 10^{-3}$
$= 599{\cdot}0 + 76{\cdot}6$
$= 675{\cdot}6$ kN

Shear force = 540 kN < Block shear capacity O.K.

Right hand beam.
At end of notch (B-B)

Depth of section = 352 − 36 = 316 mm

Shear capacity $= 0{\cdot}6 \times 355 \times 0{\cdot}9 \times 316 \times 6{\cdot}9 \times 10^{-3}$
$= 418$ kN

Shear force = 228 kN < shear capacity O.K

Slenderness of compression zone of web
$= \dfrac{1{\cdot}5\,(132-50)}{6{\cdot}9/\sqrt{12}} = 61{\cdot}8$

Compression strength = 242 N/mm²

Moment capacity $> 242 \times \dfrac{6{\cdot}9 \times 316^2}{6} \times 10^{-6}$
$= 27{\cdot}8$ kNm

Moment $= 228 \times 132 \times 10^{-3} = 30{\cdot}1$ kNm

this is greater than moment capacity based on modulus of web only, but would be less than the moment capacity based on the full modulus of the Tee section.

Check shear capacity through bolt holes (F-G-H)

Block shear capacity
= shear capacity of F-G + ½ (tensile capacity of G-H)
$= 0{\cdot}6 \times 355 \times 0{\cdot}9 \times 1{\cdot}1\,(3 \times 70 + 39 - 3{\cdot}5 \times 22) \times 6{\cdot}9 \times 10^{-3}$
$+ 0{\cdot}5 \times 355 \times 1{\cdot}1\,(48 - 0{\cdot}5 \times 22) \times 6{\cdot}9 \times 10^{-3}$
$= 250{\cdot}3 + 49{\cdot}8$
$= 300{\cdot}1$ kN

Shear force = 228 kN < Block shear capacity O.K