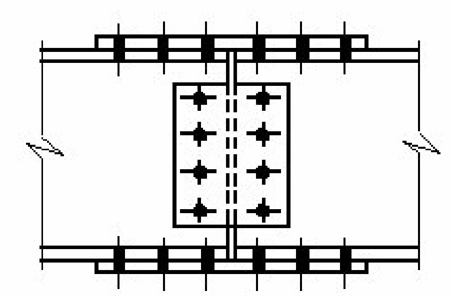
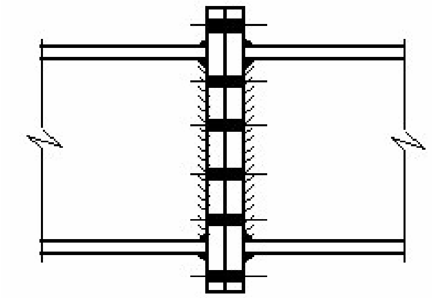
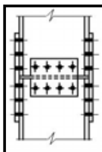




Emendas de Vigas, Colunas

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

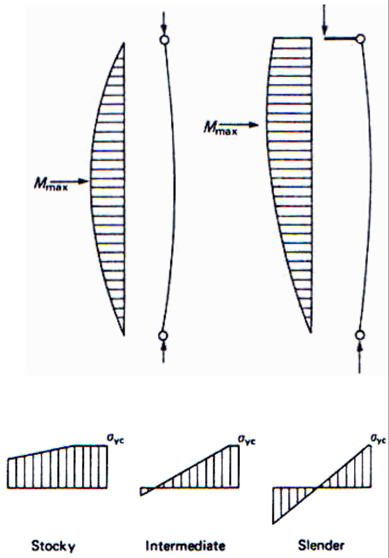


1. Introdução

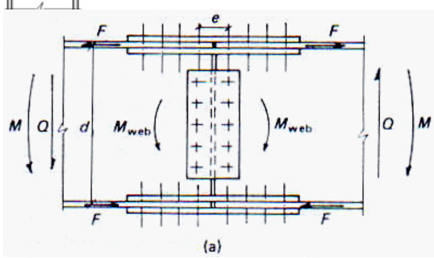
- Localização: posições afastadas de seções críticas
  - ✓ regiões com momento inferior ao momento máximo;
  - ✓ nas vigas, ponto onde o esforço cortante seja inferior a resistência da seção
    - casos onde isto não é possível → custo elevado da ligação
  - ✓ ligações de emendas em elementos sujeitos a algum tipo de instabilidade, por exemplo, pilares → localizadas próximas de um ponto travado (contraventado)

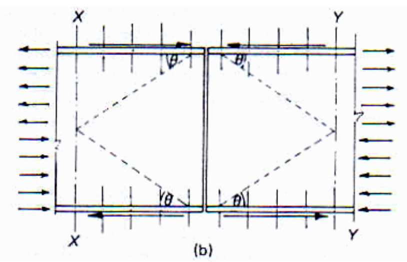
1. Introdução

- Emendas fora dos pontos de contraventamento → garantia de continuidade da peça em termos de rigidez
  - ✓peças esbeltas → resistência a flexão no eixo de menor inércia → 100% transmitida
  - ✓peças curtas → 30% com EI mínimo → 2 linhas de parafusos em cada metade (placas da mesma largura da mesa)
  - ✓peças intermediárias → entre 30% e 100%



2. Análise





- Modelo a ser adotado:
  - ✓ cortante (Q) pela ligação na alma
  - ✓ todo o momento (M) é resistido pela ligação das mesas

$$M_{alma} = 0 \qquad F = \frac{M}{d}$$

  - ✓ se

$$M_{alma} \neq 0 \qquad F = \frac{M - M_{alma}}{d}$$

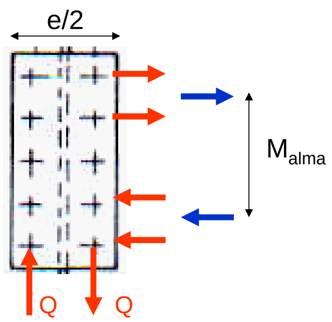
M = momento atuante

M_{alma} = momento a ser usado p/ dimensionar a ligação

2. Análise

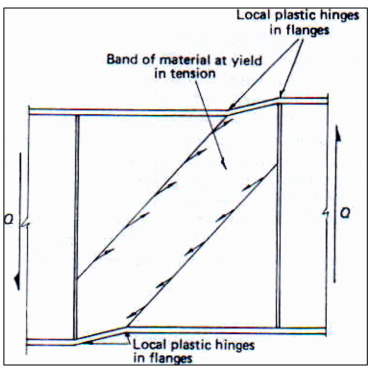
- ✓ supondo a emenda completamente carregada, ou seja, submetida a $M_p \rightarrow$ mesma simplificação de Q e M
- ✓ seção crítica na 1ª linha de parafusos (XX) → área efetiva das mesas deve ser utilizada
- ✓ a excentricidade de Q gera um momento adicional Q.e

$$M_{alma\ (total)} = M_{alma} + \frac{Q.e}{2}$$



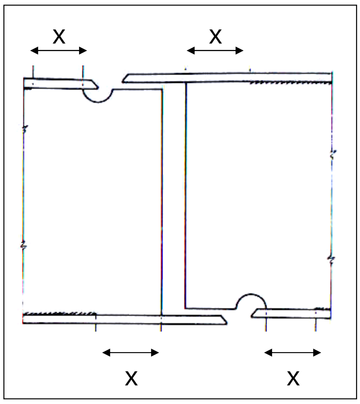
2. Análise

- ✓ M resistido pelas mesas → pode não ser usado para "plate girders" com almas esbeltas
- ✓ a alma resiste parcialmente ao cortante através da formação do campo de tração → fila dupla de parafusos em cada lado da emenda



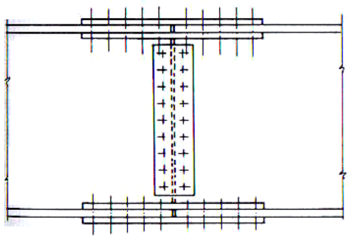
3. Tipos de Emendas - Vigas

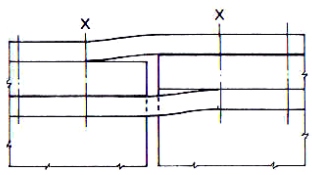
- **Soldas de entalhe com penetração total** → mais caras
  - ✓ suporte temporário p/ fase de soldagem → transpasse dos flanges (x)
  - ✓ furos não preenchidos pela solda → podem gerar tensões residuais
  - ✓ retração / distorção das soldas → excessiva quantidade de calor → 1º mesas e 2º alma (passes sucessivos)



3. Tipos de Emendas - Vigas

- **Aparafusadas com placas**
  - ✓ parafusos de alta resistência
  - ✓ chapa simples ou dupla nas mesas
  - ✓ $M_{máx}$ → dupla nas mesas → mais econômica pois utiliza menos parafusos – 2 planos de corte
  - ✓ redução da 1ª linha de parafusos → desalinhamento vertical
  - ✓ contato a compressão → *"bearing"* (não recomendável)
  - ✓ mudança de espessura ou altura → uso de chapas adicionais

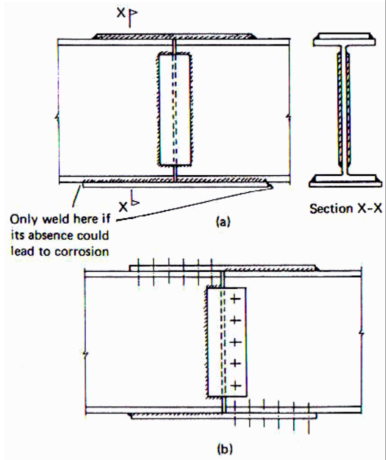

(a)



(b)

3. Tipos de Emendas - Vigas

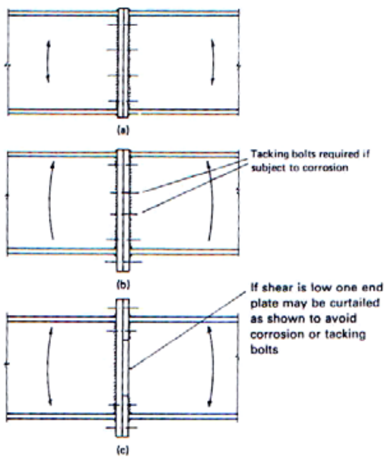
- **Com placas soldadas**
  - ✓não se usa chapa dupla nas mesas → vigas com alturas menores
- **Com placas soldadas e aparafusada**
  - ✓chapas simples nas mesas
  - ✓tipo de ligação não muito comum



3. Tipos de Emendas - Vigas

- **Placas de extremidade (M↓)**
  - ✓fratura lamelar → $t_p \geq 25$ mm
  1. mesas → "T"
  2. centro de rotação

  } efeito de alavanca → prying action
  - ✓parafusos → lado comprimido, partes em contato
  - ✓distorção da placa → solda
  - ✓+ ou – 5mm de tolerâncias → ajustes dos parafusos



End-plate beam splices. (a) Short end plates; (b) singly extended end plates; (c) doubly extended end plates

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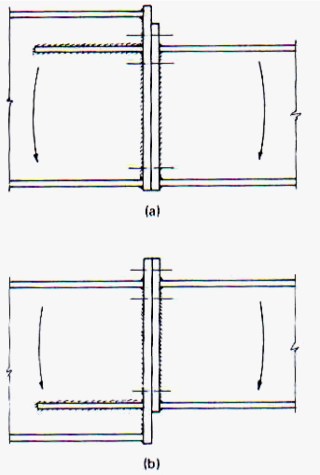
3. Tipos de Emendas - Vigas

- **Placas de extremidade (alturas diferentes)**
  - ✓mudança de altura
  - ✓utilização de enrijecedores
  - ✓consideração do ângulo de dispersão
  - ✓tamanho do cordão de solda



End-plate beam connections between elements of different serial size. (a) Coplanarity of compression flange; (b) coplanarity of tension flange

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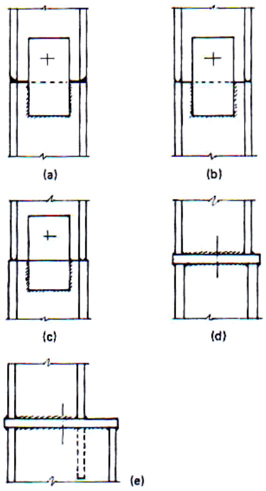
3. Tipos de Emendas - Colunas

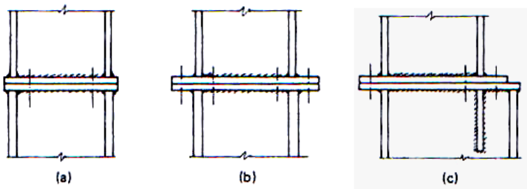
  - ✓ mais econômicas → compressão
- **Soldadas**
  - ✓ compressão é transmitida diretamente pelas mesas
  - ✓com ou sem solda
  - ✓com placa de extremidade
  - ✓enrijecedor
  - ✓mudanças de espessuras das mesas
  - ✓realizadas fora dos nós → um pouco acima do pavimento



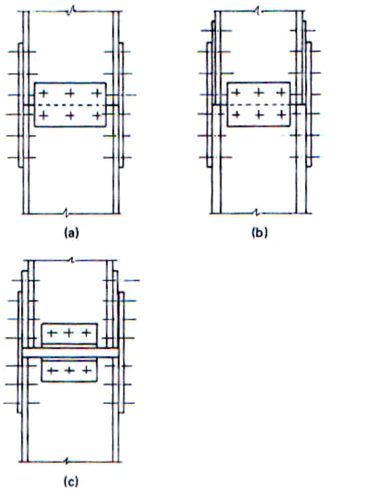
Butt-welded column splices. (a) and (b) Same serial size; (c) and (d) different rolling weight, same serial size; (e) different serial size

3. Tipos de Emendas - Colunas

- **Aparafusadas**
  - ✓ parafusos de alta resistência
  - ✓ indicadas para vigas-colunas (presença de momento)
  - ✓ placa de extremidade (M↓)



Column splices using end plates. (a) Similar sections, low moment; (b) similar sections, high moment; (c) different serial sizes, high moment



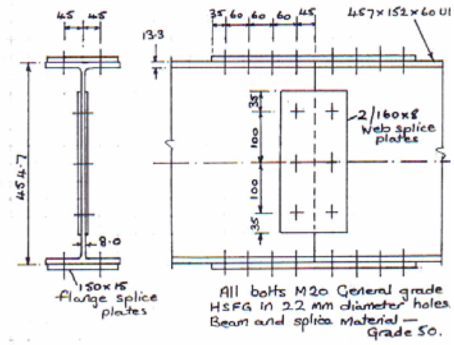
Bolted, splice plate column splices. (a) Similar columns; (b) different rolling weights, same serial size; (c) different serial size

4. Exemplos (1)

Bolted cover plate splice for Universal Beam

Design a bolted cover plate splice for a 457 × 152 × 60 UB in grade 50 steel.
The splice is to carry the following loading (due to factored loads):

Bending moment 227 kNm
Shear 113 kN



8/M20 General grade HSFG bolts in each flange (each side of joint)

Assume that all of the moment is carried by the flange splices and that the web splice only carries the shear.

Flange splices

$$\text{Flange force} = \frac{227 \times 10^3}{454{\cdot}7 - 13{\cdot}3} = 514{\cdot}3 \text{ kN}$$

For M20 General grade HSFG bolts in single shear

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

Bearing resistance of flange (per bolt) $= 20 \times 13{\cdot}3 \times 1065 \times 10^{-3} = 283{\cdot}3$ kN

$$\text{No of bolts required} = \frac{514{\cdot}3}{71{\cdot}3} = 7{\cdot}2$$

Use 4 rows of 2 bolts.

Effective area of flange $= 1{\cdot}0 \times (152{\cdot}9 - 2 \times 22) \times 13{\cdot}3 = 1448 \text{ mm}^2$

Flange capacity $= 355 \times 1448 \times 10^{-3}$
$= 514{\cdot}2$ kN
$=$ Flange force O.K

4. Exemplos (1)

Try 150 mm wide splice plate

Thickness of splice plate required

$$= \frac{514.3 \times 10^3}{1.0 \times 355(150 - 2\times22)} = 13.7 \text{ mm}$$

Use 15 mm

150×15 flange splice plates (Grade 50)

Web splice

For M20 General grade HSFG bolts in double shear

Slip resistance (per bolt)

$$= 2\times1.1\times0.45\times144 = 142.6 \text{ kN}$$

Bearing resistance of web (per bolt)

$$= 20\times8\times1065\times10^{-3} = 170.4 \text{ kN}$$

Try 3 bolts at 100 mm vertical pitch 45 mm from the centre of the joint

Horizontal shear force on bolt due to moment due to eccentricity $= \frac{113\times45\times100}{2\times100^2} = 25.4$ kN

Vertical shear force per bolt $= \frac{113}{3} = 37.7$ kN

Resultant shear force $= \sqrt{25.4^2 + 37.7^2}$

$= 45.5$ kN

< bolt capacity o.k.

Use 2 No. 8 mm thick splice plates

3/M20 General grade HSFG bolts at 100 mm pitch in web (each side of joint)

2 No. 8 mm web splice plates.

4. Exemplos (2)

Bolted cover plate splice for Plate Girder

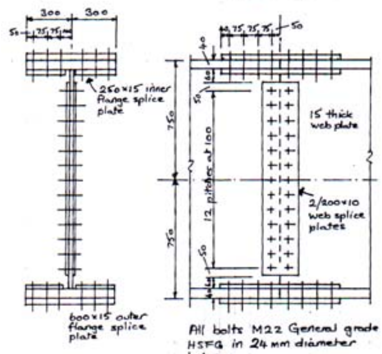
Design a bolted cover plate splice for a 1500 mm × 600 mm plate girder, with 40 mm thick flanges and a 15 mm thick web in grade 43 steel.

The splice is to carry the following loading (due to factored loads):

Bending moment	4456 kNm
Shear	620 kN
Axial tensile load	106 kN

Properties of plate girder $I = 2.916\times10^6$ cm^4

Area = 693 cm^2



All bolts M22 General grade HSFG in 24 mm diameter holes.
Plate girder and splice material grade 43.

13/M22 General grade HSFG bolts at 100 mm pitch in web (each side of joint)

2 No 10 mm web splice plates

Web splice

Try a single row of 13 bolts at 100mm vertical pitch, 50 mm from the centre of the joint.

Bending stress in web of girder at level of top and bottom bolts due to moment

$$= \frac{4456\times10^6\times600}{2.916\times10^6\times10^4} = 91.7 \text{ N/mm}^2$$

Horizontal shear force on bolt due to moment

$$= 91.7\times100\times15\times10^{-3} = 137.5 \text{ kN}$$

Average stress on girder due to axial load

$$= \frac{106\times10^3}{693\times10^2} = 1.53 \text{ N/mm}^2$$

Horizontal shear force on bolt due to axial load

$$= 1.53\times100\times15\times10^{-3} = 2.3 \text{ kN}$$

Vertical shear force on bolt due to shear

$$= \frac{620}{13} = 47.7 \text{ kN}$$

Horizontal shear force on bolt due to moment due to eccentricity

$$= \frac{620\times50\times600}{2(100^2+200^2+300^2+400^2+500^2+600^2)} = 10.2 \text{ kN}$$

Resultant shear force $= \sqrt{(47.7)^2 + (137.5+2.3+10.2)^2} = 157.4$ kN

For M22 General grade HSFG bolts in double shear

Slip resistance (per bolt) $= 2\times1.1\times0.45\times177 = 175.2$ kN

Bearing resistance of web (per bolt) $= 22\times15\times825\times10^{-3} = 272.2$ kN

O.k. Use

Use 2 No 10 mm. thick splice plates.

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4. Exemplos (2)

Flange splices. Second moment of area of portion of web which has its share of the applied moment and axial load carried by the web splice $= I_w' = \dfrac{15 \times 1300^3}{12 \times 10^4} = 0{\cdot}275 \times 10^6\ cm^4$

Area of the same portion of web
$= A_w' = 15 \times 1300 \times 10^{-2} = 195\ cm^2$

Proportion of applied moment carried by web splice $= \dfrac{I_w'}{I_{girder}} = \dfrac{0{\cdot}275 \times 10^6}{2{\cdot}916 \times 10^6} = 0{\cdot}094$

Proportion of axial load carried by web splice
$= \dfrac{A_w'}{A_{girder}} = \dfrac{195}{693} = 0{\cdot}281$

Force to be carried by flange splice
$= \dfrac{4456 \times 10^3}{1500 - 40} \times (1 - 0{\cdot}094) + \dfrac{106}{2}(1 - 0{\cdot}281)$
$= 2765{\cdot}2 + 38{\cdot}1 = 2803{\cdot}3\ kN$

No of M22 General grade HSFG bolts required in double shear $= \dfrac{2803{\cdot}3}{175{\cdot}2} = 16{\cdot}0$

Assuming that slip is critical use 4 rows of 6 bolts (which includes an extra row to cover any lack of fit)

Effective area of flange
$= 1{\cdot}2 \times (600 - 24 \times 6) \times 40 = 21{,}888\ mm^2$
(Gross area $= 600 \times 40 = 24{,}000\ mm^2$)

Flange capacity $= 265 \times 21888 \times 10^{-3}$
$= 5800\ kN$
$>$ flange force o.k

Effective area of splice plates required
$= \dfrac{2803{\cdot}3 \times 10^3}{275} = 10194\ mm^2$

Outer splice plate, try 600 mm × 15 mm flat
Effective area $= 1{\cdot}2 \times (600 - 6 \times 24) \times 15$
$= 8208\ mm^2$
$> \frac{1}{2} \times 10{,}194\ mm^2$ o.k.

Inside splice plates, try 2 No 250 mm × 15 mm flats.
Effective area $= 1{\cdot}2 \times 2 \times (250 - 3 \times 24) \times 15$
$= 6408\ mm^2$
$> \frac{1}{2} \times 10{,}194\ mm^2$ o.k

24/M22 General grade HSFG bolts in each flange (4 rows of 6 bolts each side of joint)

600×15 outer flange splice plates

250×15 inner flange splice plates

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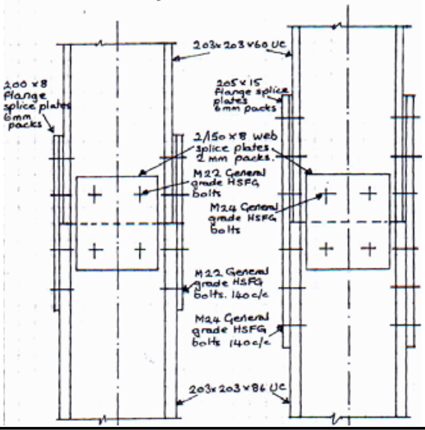
4. Exemplos (3)

Bolted cover plate splice for Universal Column

Design a bolted cover plate splice for a 203 × 203 × 60 UC connected to a 203 × 203 × 86 UC. Both columns are grade 43 steel. The splice is to carry an axial load of 700 kN (due to factored loads.)



Case (a) The splice is near a point of lateral restraint. The ends are not prepared for full contact in bearing
Area of 203 × 203 × 60 UC = $75{\cdot}8\ cm^2$
Area of web $= (209{\cdot}6 - 2 \times 14{\cdot}2) \times 9{\cdot}3 \times 10^{-2}$
$= 16{\cdot}9\ cm^2$

Web splice
Portion of load carried by web $= 700 \times \dfrac{16{\cdot}9}{75{\cdot}8}$
$= 156{\cdot}1\ kN$

For 2 No. M22 General grade HSFG bolts in double shear
Total slip resistance $= 2 \times 2 \times 1{\cdot}1 \times 0{\cdot}45 \times 177$
$= 350{\cdot}5\ kN$ o.k
Total bearing resistance
$= 2 \times 22 \times 9{\cdot}3 \times 825 \times 10^{-3}$
$= 337{\cdot}6\ kN$ o.k
End distance $\not< \dfrac{156{\cdot}1 \times 10^3}{2 \times \frac{1}{3} \times 9{\cdot}3 \times 825} = 30{\cdot}5\ mm$

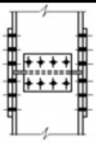
Also end distance $\not< 1{\cdot}4(22+2) = 33{\cdot}6\ mm$

Use 2 No 8 mm thick splice plates

Flange splices
Portion of load carried by each flange
$= \frac{1}{2}(700 - 156{\cdot}1) = 272{\cdot}0\ kN$

For 4 No M22 General grade HSFG bolts in single shear
Total slip resistance $= 4 \times 1{\cdot}1 \times 0{\cdot}45 \times 177$
$= 350{\cdot}5\ kN$ o.k.
Total bearing resistance $=$
$= 4 \times 22 \times 14{\cdot}2 \times 825 \times 10^{-3}$
$= 1031\ kN$ o.k.
End distance $\not< \dfrac{272{\cdot}0 \times 10^3}{4 \times \frac{1}{3} \times 14{\cdot}2 \times 825} = 17{\cdot}4\ mm$

Also $\not<$ 33·6 mm



4. Exemplos (3)

200 mm wide splice plate
Bolts at 140 mm cross centres 75 mm pitch
Thickness of splice plate required

$$= \frac{272 \times 10^3}{275(200-140+2\times 75 \tan 30^\circ)}$$

$= 6{\cdot}7$ mm

Use 8 mm plates

Bearing resistance on bolt

$= 4 \times 22 \times 8 \times 825 \times 10^{-3} = 581$ kN O.K.

Case (b)

The splice is at the midpoint of the effective length. The ends are not prepared for full contact in bearing.

As the splice is not near to the end of the member, account is taken of the moment induced by strut action.

For an effective length of 6m, the moment is equal to 38·7 kNm about the y-y axis

Web splice. As in (a) above, but use M24 bolts to match flange splice.

Flange splices

Portion of axial load carried by each flange $= \frac{1}{2}(700 - 156{\cdot}1) = 272$ kN

Assume 2 lines of bolts at 140 mm centres in each flange

Axial load per line of bolts $= \frac{272}{2} = 136$ kN

Load per line of bolts due to moment

$$= \frac{38{\cdot}7 \times 10^3}{2 \times 140} = 138{\cdot}2 \text{ kN}$$

Total load per line $= 136 + 138{\cdot}2 = 274{\cdot}2$ kN

For M 24 General grade HSFG bolts in single shear

Slip resistance per bolt $= 1{\cdot}1 \times 0{\cdot}45 \times 207 = 102{\cdot}5$ kN

Bearing resistance ok by inspection

Number of bolts required in each line

$$= \frac{274{\cdot}2}{102{\cdot}5} = 2{\cdot}68$$

Use 3×2/M 24 bolts

The size of the flange splice plates should be not less than that of the associated flanges (of the lighter section when there is a change of section.)

Use 205×15 flange splice plates

205×15 flange splice plates.

6/M 24 General grade HSFG bolts in each flange