1 Eaves Joint

Beam to Column Flange End-Plated Connection to EC 3 (UK NAD)

Loading Case 001

Basic Data

User Defined Applied Forces at Column/Right Rafter Interface

Right Rafter Forces, M, Fv, Fr

Resultant Forces, M, Fv, F

Load directions

Design to

Weld Grades

Basic Dimensions

Column-610x178 UB 92 [S275]

Rafter-457x191 UB 67 [S275]

Haunch-457x191 UB 67 [S275]

Bolts 20 mm Ø in 22 mm holes

Plates S 275

Rafter Capacities, Mc, Fvc, Fc

Summary of Results (Unity Ratios)

Step 1: Tension Zone

BOLT ROW 1

Column Flange row 1 only
<table>
<thead>
<tr>
<th>Job Ref</th>
<th>1472</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheet</td>
<td>C/663</td>
</tr>
<tr>
<td>Made by</td>
<td>Tommy White</td>
</tr>
<tr>
<td>Date</td>
<td>04 August 2017 / Ver. 2017.08</td>
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<tr>
<td>Checked</td>
<td>663</td>
</tr>
<tr>
<td>Approved</td>
<td>663</td>
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</table>

### Column Flange rows 1 to 2 combined

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>l_{eff,cp}</td>
<td>294.4, 44.4, 80.0</td>
</tr>
<tr>
<td>l_{eff,acp}</td>
<td>min(Circle, Indiv End)</td>
</tr>
<tr>
<td>M_{pl,1}</td>
<td>166.5, 15.0, 15.0, 275.0, 4/1.0</td>
</tr>
<tr>
<td>M_{pl,2}</td>
<td>257.0, 6.0, 15.0</td>
</tr>
<tr>
<td>(2\alpha_{\text{cp}})</td>
<td>1547.8</td>
</tr>
<tr>
<td>F_{T,1,Rd}</td>
<td>(8\times36.49+24.25\times2576.0/1547.8)</td>
</tr>
<tr>
<td>F_{T,2,Rd}</td>
<td>(2\times218.8+44.40\times214.11/2529.5)</td>
</tr>
<tr>
<td>F_{T,2,Rd}</td>
<td>(2\times235.7+44.40\times214.11/2529.5)</td>
</tr>
</tbody>
</table>

### Beam Web Tension row 1 only

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\omega_{\text{ac}})</td>
<td>fn(165.6, 10.9, 6929, 1.00)</td>
</tr>
<tr>
<td>F_{ac,Rd}</td>
<td>0.96 \times 165.6 \times 10.9 \times 275/1.00</td>
</tr>
</tbody>
</table>

### End Plate row 1 only

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>m, c, m_n</td>
<td>36.0, 60.0, 40.8, 6.6</td>
</tr>
<tr>
<td>l_{cp}</td>
<td>225.9, 235.7</td>
</tr>
<tr>
<td>M_{pl,1}</td>
<td>259.20 \times 20.0 \times 265.0/1.0</td>
</tr>
<tr>
<td>M_{pl,2}</td>
<td>259.20 \times 20.0 \times 265.0/1.0</td>
</tr>
<tr>
<td>(2\alpha_{\text{cp}})</td>
<td>5985.8 \times 10.9</td>
</tr>
<tr>
<td>F_{T,1,Rd}</td>
<td>(2\times218.8+44.40\times214.11/2529.5)</td>
</tr>
<tr>
<td>F_{T,2,Rd}</td>
<td>(2\times235.7+44.40\times214.11/2529.5)</td>
</tr>
</tbody>
</table>

### Beam Web Tension row 2 only

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\omega_{\text{pl}})</td>
<td>fn(173.1, 10.9, 6929, 1.00)</td>
</tr>
<tr>
<td>F_{ac,Rd}</td>
<td>0.96 \times 173.1 \times 10.9 \times 275/1.00</td>
</tr>
</tbody>
</table>

### BOLT R2

#### Column Flange row 2 only

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>l_{cp}</td>
<td>294.4, 44.4</td>
</tr>
<tr>
<td>l_{eff,cp}</td>
<td>225.9, 235.7</td>
</tr>
<tr>
<td>M_{pl,1}</td>
<td>2677.0, 6.0, 15.0</td>
</tr>
<tr>
<td>M_{pl,2}</td>
<td>2677.0, 6.0, 15.0</td>
</tr>
<tr>
<td>(2\alpha_{\text{cp}})</td>
<td>1547.8</td>
</tr>
<tr>
<td>F_{T,1,Rd}</td>
<td>(2\times235.7+44.40\times214.11/2529.5)</td>
</tr>
<tr>
<td>F_{T,2,Rd}</td>
<td>(2\times218.8+44.40\times214.11/2529.5)</td>
</tr>
</tbody>
</table>

#### Beam Web Tension row 2 only

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\omega_{\text{pl}})</td>
<td>fn(173.1, 10.9, 6929, 1.00)</td>
</tr>
<tr>
<td>F_{ac,Rd}</td>
<td>0.96 \times 173.1 \times 10.9 \times 275/1.00</td>
</tr>
</tbody>
</table>

**Master Series**

3 Castle Street
Carrickfergus
Co. Antrim BT38 7BE
Tel: 028 9036 5950

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**BOLT ROW 3**

### Column Flange row 3 only

- **m, c**: 29.4, 44.4
- **l_{eff,cp}**: 184.7 mm, **l_{eff,ncp}**: 173.1 mm

#### Mode 1
- **l_{eff,cp}**: 184.7 mm, **l_{eff,ncp}**: 173.1 mm

#### Mode 2
- **l_{eff,cp}**: 173.1 mm, **l_{eff,ncp}**: 173.1 mm

- **M_{pl,1}**: 173.1\(\times 15.0\times 15.0\times 275.0/4.0\times 1.0 = 2677.0\) kN.mm
- **M_{pl,2}**: 173.1\(\times 15.0\times 15.0\times 275.0/4.0\times 1.0 = 2677.0\) kN.mm

#### Column Web Tension row 3 only
- **M_{pl,1}**: 2677.0 kN.mm
- **M_{pl,2}**: 2677.0 kN.mm

### End Plate row 3 only
- **M_{pl,1}**: 325.7 mm, **M_{pl,2}**: 325.7 mm, **M_{pl,3}**: 325.7 mm

- **M_{pl,1}**: 218.8 mm, **M_{pl,2}**: 218.8 mm

### Potential resistance of Bolt Row 2

- **F_{Rb,1}**: 698.5 kN
to 2 combined

---

**Beam Web Tension rows 1 to 2 combined**

- **ω_{f,ncp}**: 0.91

#### Column Flange row 1 to 2 combined

- **M_{pl,1}**: 325.7 mm, **M_{pl,2}**: 325.7 mm, **M_{pl,3}**: 325.7 mm

#### Column Web Tension row 1 to 2 combined

- **M_{pl,1}**: 325.7 mm, **M_{pl,2}**: 325.7 mm, **M_{pl,3}**: 325.7 mm

### BOLT ROW 3

- **F_{Rb,1}**: 526.6 kN

---

<table>
<thead>
<tr>
<th>Joint Type</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>l_{eff,cp}</td>
<td>184.7 mm</td>
<td></td>
</tr>
<tr>
<td>l_{eff,ncp}</td>
<td>173.1 mm</td>
<td></td>
</tr>
<tr>
<td>M_{pl,1}</td>
<td>2677.0 kN.mm</td>
<td></td>
</tr>
<tr>
<td>M_{pl,2}</td>
<td>2677.0 kN.mm</td>
<td></td>
</tr>
<tr>
<td>M_{pl,3}</td>
<td>2677.0 kN.mm</td>
<td></td>
</tr>
<tr>
<td>M_{pl,1}</td>
<td>218.8 mm</td>
<td></td>
</tr>
<tr>
<td>M_{pl,2}</td>
<td>218.8 mm</td>
<td></td>
</tr>
<tr>
<td>M_{pl,3}</td>
<td>218.8 mm</td>
<td></td>
</tr>
</tbody>
</table>

---

**Column Flange rows 2 to 3 combined**

- **l_{eff,cp}**: 225.9 mm, **l_{eff,ncp}**: 218.8 mm

#### Mode 1
- **l_{eff,cp}**: 225.9 mm, **l_{eff,ncp}**: 218.8 mm

#### Mode 2
- **l_{eff,cp}**: 218.8 mm, **l_{eff,ncp}**: 218.8 mm

- **M_{pl,1}**: 218.8 mm, **M_{pl,2}**: 218.8 mm, **M_{pl,3}**: 218.8 mm

### Potential resistance of Bolt Row 2

- **F_{Rb,2}**: 511.4 kN

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<table>
<thead>
<tr>
<th>Joint Type</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>l_{eff,cp}</td>
<td>225.9 mm</td>
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<tr>
<td>l_{eff,ncp}</td>
<td>218.8 mm</td>
<td></td>
</tr>
<tr>
<td>M_{pl,1}</td>
<td>218.8 mm</td>
<td></td>
</tr>
<tr>
<td>M_{pl,2}</td>
<td>218.8 mm</td>
<td></td>
</tr>
<tr>
<td>M_{pl,3}</td>
<td>218.8 mm</td>
<td></td>
</tr>
</tbody>
</table>
Column Web Tension rows 2 to 3 combined

\[ \omega = \min(263.1, 10.9, 6929.1, 1.00) \]

\[ F_{W,3,Rd} = \omega \times \{ \begin{array}{ll}
\text{Row 1} & 2.631 \\
\text{Row 2} & 10.9 \\
\text{Row 3} & 6929.1 \\
\text{Row 4} & 1.00
\end{array} \} \]

Column Flange rows 1 to 3 combined

\[ \omega = \min(29.39, 80.00, 29.39) \]

\[ F_{B,1,Rd} = \omega \times \{ \begin{array}{ll}
\text{Row 1} & 29.39 \\
\text{Row 2} & 80.00 \\
\text{Row 3} & 29.39
\end{array} \} \]

End-Plate rows 2 to 3 combined

\[ \omega = \min(35.95, 35.95, 35.95) \]

\[ F_{B,5,Rd} = \omega \times \{ \begin{array}{ll}
\text{Row 2} & 35.95 \\
\text{Row 3} & 35.95 \\
\text{Row 4} & 35.95
\end{array} \} \]

Beam Web Tension rows 2 to 3 combined

\[ F_{W,5},Rd = \{ \begin{array}{ll}
\text{Row 1} & 308.8 \\
\text{Row 2} & 308.8 \\
\text{Row 3} & 308.8
\end{array} \} \]

Beam Flange rows 1 to 3 combined

\[ \omega = \min(35.95, 35.95, 35.95) \]

\[ F_{B,6},Rd = \omega \times \{ \begin{array}{ll}
\text{Row 1} & 35.95 \\
\text{Row 2} & 35.95 \\
\text{Row 3} & 35.95
\end{array} \} \]

End-Plate rows 1 to 3 combined

\[ \omega = \min(35.95, 35.95, 35.95) \]

\[ F_{B,7},Rd = \omega \times \{ \begin{array}{ll}
\text{Row 1} & 35.95 \\
\text{Row 2} & 35.95 \\
\text{Row 3} & 35.95
\end{array} \} \]
Potential Tension Capacity

\[ \text{Tension} = (N_t - 1) \cdot F \]

Bearing Force

\[ F_{b,Rd} = \min(N_{b,Rd}, F_{b,Ed,Rd}) \]

Potential Tension Capacity

\[ F_{t,Ed} = \min(F_{c,fc,Rd}, F_{t,rd}, F_{t,Ed}) \]

Web Buckling

\[ N_{w,Rd} = \min(2b,t+2b_{t}+c_{b}) \cdot f_{w,Rd} \cdot \gamma_{M1} \]

Total Area Flange

\[ A_{t,Rd} = (2b_{t}+2b+c_{b}) \cdot f_{w,Rd} \cdot \gamma_{M1} \]

Web Compression

Beam Compression Zone

\[ F_{c,Rd} = \min(189.9, 214.9) \cdot 12.0 \div (2 \cdot 15.0) \div 2.0 = 195.7 \text{ kN.m} \]

\[ \text{Total Flange} = A_{t,Rd} \cdot f_{w,Rd} = 24.1 \cdot 275 \cdot 1.40 = 632.5 \text{ kN} \]

\[ \text{Step 3: Column Web Shear} \]

\[ V_{s,Rd} = 0.9 \cdot f_{s} \cdot A_{w} / (k_{0} \cdot s / 3) = 0.9 \cdot 275 \cdot 6928.5 / (1.10 \cdot s / 3) = 990.0 \text{ kN} \]

Potential Compression Capacity

\[ F_{c,Rd} = \min(F_{c,Ed,Rd}, F_{c,Rd,Ed}) \]

Shear limit \( F_{s,Rd} = \min(F_{c,Ed,Rd}, F_{s,Rd}) \)

Bolt Forces and Moment Capacities

\[ M_{b,Rd} = \min(M_{b,Ed}, M_{b,Rd}) \]

\[ F_{s,Rd} = \min(F_{s,Ed,Rd}, F_{s,Ed,Rd}) \]

\[ F_{t,Ed} = \min(F_{t,Ed,Rd}, F_{t,Ed,Rd}) \]

\[ \text{Step 5: Shear Bolts} \]

\[ F_{c,Rd,Ed} = \min(F_{c,Ed,Rd}, F_{c,Ed,Rd}) \]

\[ F_{s,Ed,Rd,Ed} = \min(F_{s,Ed,Rd,Ed}, F_{s,Ed,Rd,Ed}) \]

\[ F_{t,Ed,Ed} = \min(F_{t,Ed,Rd,Ed}, F_{t,Ed,Rd,Ed}) \]

\[ \text{Shear} = N_{s} \cdot F_{s,Ed} \]

\[ \text{Tensile} = (N_{t} - 1) \cdot F_{t,Ed} + F_{t,Ed,Tensile} \]

\[ \text{Total} = \text{Shear} + \text{Tensile} \]

**Steps 7&8: Welds**

Beam force $f_{web}=f_d/(\sqrt{3}\times b_w)/\gamma_M$  
410.0 / $\sqrt{3}/0.85 / 1.25$  
222.8 N/mm²

**Flange Tension Weld**

$Ft_{max}=min(B\times T\times P_\gamma, F_{fI}, F_{f2})$  
Min(189.9\times12.7\times275, 203.2 + 172.2)  
375.5 kN

**Flange Compression Weld**

$F_{ccap}=2\times k\times 0.7 \times t \times L \times t_{w,d}$  
2\times 1.225\times 0.7 \times 8 \times (189.9 - 2\times 0.7\times 8)\times 223  
551.3 kN OK

**Web Welds in Tension Zone**

Web Weld OK if $>= 0.69\times T$  
6 $>= 0.69\times 8.5$  
$> = 5.8$ mm OK

**Web Welds in Shear Zone**

$L_{w,d}=D+(T+T_h) - t_{w}\times L\times L_{w,d}$  
900.0 - 26.1 - 10.2 - 10.2 - 295  
558.6 mm

**Haunch Welds**

$F_{h,Ed}=min(M_0/(ht_0-t_0), b_r\times t_0\times t_1)$  
Min(350.0/(453.4 - 12.7), 189.9\times12.7\times275)  
663.2 kN

**Haunch End Weld**

$t_{min}(S_n-\gamma I) = Cos((90-ThetaH1)/2)$  
350.0/(453.4 - 12.7)\times Cos((90-11.0)/2)  
9.3 mm

**Haunch Web Weld**

$F_{web}=2\times 0.7\times t_{w,d}\times L\times t_{w,d}$  
2\times 0.7\times 558.6\times 223  
1057.5 kN OK

**Haunch Web Weld**

Web force $F_{web}=F_{hcap} - F_{h,Ed}$  
613.0 - 331.6  
281.3 kN

**Compression Stiffener Weld Web**

Column force $f_{web}=f_d/(\sqrt{3}\times b_w)/\gamma_M$  
410.0 / $\sqrt{3}/0.85 / 1.25$  
222.8 N/mm²

**Web Buckling**

$A_{eff}=2b_r\times t+L_{w,d}$  
2\times 90.0 + 8.0 + 243.7 + 8.5  
3512 mm²

**Step 8: End of Haunch Compression Zone**

**Force Applied - Transverse to Beam Web**

$F_{T,Ed}=F_{web} \times Tan(ThetaH1)$  
331.6\times Tan(11.0)  
64.5 kN

**Web Bearing**

$n=min(5, 2 + 0.6\times b_r/(tb_0+t_0))$  
Min(5, 2 + 0.6\times 453.4/22.9)  
5.000

$\beta_{stiff}=min(2\times \gamma I, 12(7.0 + 10.2) + 0.00, 1.25)$  
12.00 + 2\times 0.0 + 5.000(12.70 + 10.20) + 0.00  
126.5

$\phi_{stiff}=\phi_{stiff}(tb_0, t_{w}, A_{eff}, \beta)$  
$t_{w}=189.9/2 - 8.5 - 10.0$  
76.0 mm

$N_{b,Rd}/\gamma_M=(2b_r\times t+ b_{off}\times t_{un})f_{y,web}/\gamma_M$  
(2\times 126.5 + 126.5 + 8.5)\times 275 + 100  
630.1 kN OK

$N_{b,Rd}=A_{off}\times f_d/\gamma_M$  
2\times 90.0 + 8.0 + 243.7 + 8.5  
3512 mm²

$I_{stiff}(2b_r\times t+L_{w,d})/12$  
8.0\times 90.0 + 8.5)/12  
4465220 mm⁴

$\lambda_{stiff}=(h_b-2t_0)/t_0$  
((4465220/3512)\times 2\times 126.5, 35.66

$K=min(1.0, 1/(\delta + \sqrt{(2\times \lambda^2)})$  
min(1.0, 1/(0.49 + \sqrt{(0.99^2 - 0.14^2)})  
1.000

$N_{b,Rd}=K\times A_{eff}\times f_d/\gamma_M$  
1.00\times 3512\times 275\times 100  
965.7 kN OK
Apex Joint
Beam to Beam End-Plated Connection to EC 3 (UK NAD)
Loading Case 001

Basic Data

User Defined Applied Forces at End-plate Interface
Right Rafter Forces M, Fvr, Fr
-150.0 kNm, 400.0 kN, 300.0 kN
Resultant Forces M, Fv, F
-150.0 kNm, 328.9 kN, 376.6 kN
Load directions
Bottom of Joint in Tension, Rafter moving Down and in Compression.
Design to
SCI Green Book
P398: Joints in steel construction: Moment-Resisting Joints to Eurocode 3

Weld Grades
All weld grades provided to suit minimum connected steel grade

Basic Dimensions
Rafter-457x191 UB67 [28]
D=453.4, B=189.9, T=12.7, t=8.5, r=10.2, py=275
Haunch-457x191 UB67 [28]
D=453.4, B=189.9, T=12.7, t=8.5, r=10.2, py=275
Bolts 20 mm Ø in 22 mm holes
Grade 8.8 Bolts
Plates S 275
All weld grades provided to suit minimum connected steel grade
Rafter Capacities Mc, Fvc, Fc
1046.2 kN.m, 1252.6 kN, 3395.2 kN
Fvc = 1252.6 kN

Summary of Results (Unity Ratios)
Moment Capacity 524.4 kNm (for 4 rows of bolts) (Modified Applied Mom. M_{mod}=66.6 kNm) 0.13 OK
Moment Capacity 203.7 kNm (for the 1 rows of bolts required in the tension zone) 0.33 OK
Shear Capacity 0.41 OK
Flange Welds 0.15 OK
Web Welds 0.49, 0.24 OK
Haunch Welds 0.58, 0.23 OK
### Step 1: Tension Zone

**BOLT ROW 1**

**End Plate row 1 only**

<table>
<thead>
<tr>
<th>m, c, mx, αl</th>
<th>36.0, 57.5, 40.8, 6.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lpl modes</td>
<td>232.0</td>
</tr>
</tbody>
</table>

- **Mode 1**
  - \( l_{eff,cp} = 225.9, l_{left,aeq} = 232.0 \)
- **Mode 2**
  - \( l_{eff,cp} = 232.0 \)

- \( M_{pl,1} = (2\times35.95\times44.94-8.25\times(35.95+44.94)) \)
- \( M_{pl,2} = (2\times35.95\times44.94-8.25\times(35.95+44.94)) \)

- \( F_{tl,Rd} = \min(F_{tl,Rd, mode 1.2,3}) \)
  - \( 436.7 \times 245.5 \)

**Beam Web Tension row 1 only**

- \( F_{tl,Rd,1} = \min(l_{eff,aeq}, l_{left,aeq}) \)
  - \( 225.9 \times 8.5 \times 275/1.00 \)
- **Potential resistance of Bolt Row 1**
  - \( 424.5 \times 0.8 \)

**BOLT ROW 2**

**End-Plate rows 1 to 2 combined**

- \( l_{eff,cp} = 232.0 \)
- \( l_{left,aeq} = \min(l_{left,aeq}, l_{left,aeq}) \)

- **Mode 1**
  - \( l_{left,aeq} = \min(l_{left,aeq}, l_{left,aeq}) \)
  - \( 225.9 \times 232.0 \)
- **Mode 2**
  - \( 225.9 \times 232.0 \)

- \( M_{pl,1} = (2\times7485.0 \times 44.94 \times 141.1) / (35.95 + 44.94) \)
- \( M_{pl,2} = (2\times7485.0 \times 44.94 \times 141.1) / (35.95 + 44.94) \)

- \( F_{tl,Rd} = \min(F_{tl,Rd, mode 1.2,3}) \)
  - \( 436.7 \times 245.5 \)

**BOLT ROW 3**

**End-Plate rows 1 to 3 combined**

- \( l_{eff,cp} = 232.0 \)
- \( l_{left,aeq} = \min(l_{left,aeq}, l_{left,aeq}) \)

- **Mode 1**
  - \( l_{left,aeq} = \min(l_{left,aeq}, l_{left,aeq}) \)
  - \( 225.9 \times 232.0 \)
- **Mode 2**
  - \( 225.9 \times 232.0 \)

- \( M_{pl,1} = (2\times35.95\times44.94-8.25\times(35.95+44.94)) \)
- \( M_{pl,2} = (2\times35.95\times44.94-8.25\times(35.95+44.94)) \)

- \( F_{tl,Rd} = \min(F_{tl,Rd, mode 1.2,3}) \)
  - \( 436.7 \times 245.5 \)

**BOLT ROW 4**

**End-Plate row 4 only**

- \( M_{pl,1} = (2\times7485.0 \times 44.94 \times 141.1) / (35.95 + 44.94) \)
- \( M_{pl,2} = (2\times7485.0 \times 44.94 \times 141.1) / (35.95 + 44.94) \)

- \( F_{tl,Rd} = \min(F_{tl,Rd, mode 1.2,3}) \)
  - \( 436.7 \times 245.5 \)
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MasterSeries Sales Team
3 Castle Street
Carrickfergus
Co. Antrim BT38 7BE
Tel: 028 9036 5950

F_{t,rd}=\min(\frac{F_{t,rd}}{mode 1,2,3})
min(467.5, 266.8, 282.2) 266.8 kN

Step 1C Plastic distribution Limit

Step 2: Compression Zone

Potential Tension Capacity
Sigma F_{t,rd}
245.5 + 191.2 + 218.7 + 266.8 kN 922.3 kN

Beam Compression
Beam Compression Zone
Flange and Web in Compression Utilising 20% OverStressing
Total Area Flange and Web
189.9\times12.7 + 8.5\times179.0 39.3 cm²

Potential Compression Capacity
F_{c,rd,min}
1298.0 1298.0 kN OK

Final Bolt Forces and Moment Capacities
Bolt row 4: M_{rd}=F_{c,rd} \times \alpha \times \beta
266.8 - 0.9 = 265.9\times139.6 37.1 kN.m
Bolt row 3 M_{rd,3}=F_{c,rd} \times \alpha \times \beta
218.7\times49.6 142.1 kN.m
Bolt row 2 M_{rd,2}=F_{c,rd} \times \alpha \times \beta
191.2\times79.6 141.4 kN.m
Bolt row 1 M_{rd,1}=F_{c,rd} \times \alpha \times \beta
245.5\times82.6 203.7 kN.m

Tension Bolts
Only the first 1 rows are required to resist the applied moment
The remaining rows shall be considered to be part of the shear zone.
M_{c,rd} for 1 rows
203.7 203.7 kN.m
F_{c,rd} for 1 rows
245.5
F_{c,rd} design=M_{c,rd} \times \alpha \times \beta
245.5\times66.6\times203.7 80.3 kN

Final Web Compression Zone Height
F_{c,rd}=F_{c,rd} \times \alpha \times \beta \times \gamma_{M,2}
0.6 \times 800 \times 245.0 / 1.25 94.1 kN
Bearing F_{c,rd}=End Plate, End
p=1\times80, e=58, k_{e}=2.5, \gamma_{M,2}=1.00, \gamma_{M,2}=1.00, d=20, t=15, f_{y}=410
246.0 kN
Bearing F_{c,rd}=End Plate, Inner
p=1\times90, e=58, k_{e}=2.5, \gamma_{M,2}=1.00, \gamma_{M,2}=1.00, d=20, t=15, f_{y}=410
246.0 kN

Flange Tension Weld
F_{t,rd}=\min(B+T\times P, F_{t,1})
\gamma \times (3\times8) / 1.25 222.8 N/mm²

Web Welds in Tension Zone
Lwt=\frac{L_{proj}-T\times root+1.73\times g}{2}
80 - 20 - 12.9 - 10.2 + 1.73 90/2 114.8 mm
Load per row
Row_{S}=k_{s} \times F_{t,1}
(41/36 + 41)\times246 130.5 kN
Total Load F_{t}
130.5
F_{w,cap}=2\times k_{s} \times t \times \frac{L_{wt} \times t_{w,d}}{2}
2\times1225\times0.7 \times 189.9 \times 0.7 \times 223 551.3 kN OK

Steps 7&8: Welds

Steps 7&8: Welds

Web Welds in Tension Zone
Lwt=\frac{L_{proj}-T\times T_{w} - T_{w} - L_{wt}}{2}
900.0 - 25.9 - 10.2 - 10.2 - 115 739.0 mm
F_{w,cap}=2\times0.7 \times 739.0 \times 223 1396.7 kN OK

Haunch Welds

\[ M_a = \min(M_{\text{app}}, M_{\text{total}}) \]
\[ F_{b,\text{Ed}} = \min(M_a/(b - t_b), b_t \cdot f_{t_b}) \]
\[ \text{Haunch/Beam Flange area ratio, } \alpha \]
\[ F_{\text{hub}} = F_t \cdot \min(0.5, \frac{t_b}{h_c}) \]
\[ F_{\text{hub}} = (1/F_t \cdot 1.27 + 1.27) / \cos(-12.0) \]
\[ 0.50 \]
\[ 170.2 \text{ kN} \]

Haunch End Weld

\[ t = \min(S_{\text{hub}}, t_b) \cdot \cos((90 - \Theta_{H1})/2) \]
\[ K = \sqrt{3/(1 + 2 \cdot \cos((90 - \Theta_{H1})/2)^2)} \]
\[ \text{EndCap} = K \cdot (B - 2 \cdot w) \cdot f_{\text{vw,d}} \]
\[ 295.7 \geq 170.2 \]
\[ 170.2 \text{ kN} \]
\[ OK \]

Haunch Web Weld

\[ \text{Web force } F_{\text{sh}} = F_{\text{hub}} \cdot F_{\text{shweb}} \]
\[ 670.1 - 170.2 \]
\[ 499.9 \text{ kN} \]

Other calculations:

\[ L_w = (H/3 - D/2 - T)/\cos(\Theta) \]
\[ 1211.5 \text{ mm} \]
\[ 1152.9 \text{ mm} \]
\[ 2141.8 \geq 499.9 \]
\[ 499.9 \text{ kN} \]
\[ OK \]
End-Plated Splice
Beam to Beam End-Plated Connection to EC 3 (UK NAD)

Loading Case 001
Basic Data

User Defined Applied Forces at End-plate Interface
Resultant Forces M, Fv, F
Load directions
Design to
SCI Green Book
P398: Joints in steel construction: Moment-Resisting Joints to Eurocode 3

Weld Grades
All weld grades provided to suit minimum connected steel grade

Basic Dimensions
Beam 457x191 UB 67 [28]
D=453.4, B=189.9, t=8.5, r=10.2, py=275
Bolts 20 mm Ø in 22 mm holes
Grade 8.8 Bolts
Plates S 275
All weld grades provided to suit minimum connected steel grade
Rafter Capacities Mc, Fvc, Fc
404.5 kN.m, 649.9 kN, 2351.3 kN
Fvc = 649.9 kN OK

Summary of Results (Unity Ratios)
Moment Capacity 191.5 kNm (for 4 rows of bolts) (Modified Applied Mom. Mmod=168.9 kNm) 0.88 OK
Moment Capacity 191.5 kNm (for the 3 rows of bolts required in the tension zone) 0.88 OK
Shear Capacity 0.75 OK
Flange Welds 0.46 0.46 OK
Web Welds 0.77, 0.77, 0.68 0.77 OK

Step 1: Tension Zone
BOLT ROW 1
End Plate row 1 only
### MasterSeries Sales Team

3 Castle Street
Carrickfergus
Co. Antrim BT38 7BE
Tel: 028 9036 5950

---

### BOLT ROW 2

#### End Plate row 2 only

- **m, c:** 36.0, 60.0
- **l**
  - Mode 1 \( l_{eff,cp} = \min(l_{eff,cp}, l_{eff,ncp}) \): 225.9 mm
  - Mode 2 \( l_{eff,ncp} = \frac{1}{4}(m+n) \): 218.8 mm
- **M**
  - Mode 1 \( M_{1,1} = \frac{1}{4}(m+n) \): 218.8 mm
  - Mode 2 \( M_{2,1} = \frac{1}{4}(m+n) \): 218.8 mm
- **T**
  - Mode 1 \( T_{1,1} = \frac{1}{4}(m+n) \): 218.8 mm
  - Mode 2 \( T_{2,1} = \frac{1}{4}(m+n) \): 218.8 mm
- **F**
  - Mode 1 \( F_{1,1} = \frac{1}{4}(m+n) \): 218.8 mm
  - Mode 2 \( F_{2,1} = \frac{1}{4}(m+n) \): 218.8 mm
- **Potential resistance of Bolt Row 1**
  - Mode 2

---

### BOLT ROW 3

#### End Plate row 3 only

- **m, c:** 36.0, 60.0
- **l**
  - Mode 1 \( l_{eff,cp} = \min(l_{eff,cp}, l_{eff,ncp}) \): 225.9 mm
  - Mode 2 \( l_{eff,ncp} = \frac{1}{4}(m+n) \): 218.8 mm
- **M**
  - Mode 1 \( M_{1,1} = \frac{1}{4}(m+n) \): 218.8 mm
  - Mode 2 \( M_{2,1} = \frac{1}{4}(m+n) \): 218.8 mm
- **T**
  - Mode 1 \( T_{1,1} = \frac{1}{4}(m+n) \): 218.8 mm
  - Mode 2 \( T_{2,1} = \frac{1}{4}(m+n) \): 218.8 mm
- **F**
  - Mode 1 \( F_{1,1} = \frac{1}{4}(m+n) \): 218.8 mm
  - Mode 2 \( F_{2,1} = \frac{1}{4}(m+n) \): 218.8 mm
- **Potential resistance of Bolt Row 2**
  - Mode 2

---

### Potential resistance of Bolt Row 1

- **Mode 2**

---

### Potential resistance of Bolt Row 2

- **Mode 2**
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---

**Potential Tension Capacity**

**End-Plate rows 1 to 4 combined**

\[ F_{\text{eff,ncp}} = 2m + 0.625e \]

\[ F_{\text{eff,cp}} = \pi \cdot m \]

\[ M_{\text{pl,1}} = I_{\text{eff,1}} \cdot \gamma_{\text{w}} \cdot \gamma_{\text{y},\text{wb}} \]

\[ T_{\text{Rd}} = \frac{T_{\text{Rd}}}{T_{\text{Rd}}} \]

\[ F_{\text{R,B2},2} = \text{min}(F_{\text{R,B2},1}, 395.7 \text{ mm}) \]

\[ F_{\text{R,B1}} = \text{min}(F_{\text{R,B1},1}, 126.3 \text{ mm}) \]

\[ F_{\text{R,B2}} = \text{min}(F_{\text{R,B2},1}, 545.9 \text{ mm}) \]

**Beam Web Tension rows 2 to 3 combined**

\[ F_{\text{R,B1}} = \text{min}(F_{\text{R,B1},1}, 395.7 \text{ mm}) \]

**End-Plate rows 1 to 3 combined**

\[ F_{\text{R,B1},1} = \text{min}(F_{\text{R,B1},1}, 395.7 \text{ mm}) \]

\[ F_{\text{R,B2},1} = \text{min}(F_{\text{R,B2},1}, 545.9 \text{ mm}) \]

**Beam Web Tension rows 1 to 3 combined**

\[ F_{\text{R,B1}} = \text{min}(F_{\text{R,B1},1}, 395.7 \text{ mm}) \]

**BOLT ROW 4**

**End-Plate rows 1 to 4 combined**

\[ F_{\text{R,B1}} = \text{min}(F_{\text{R,B1},1}, 395.7 \text{ mm}) \]

\[ F_{\text{R,B2}} = \text{min}(F_{\text{R,B2},1}, 545.9 \text{ mm}) \]

**Step 1C Plastic distribution Limit**

\[ T_{\text{pl,2}} < 0.1 \text{ of } \gamma_{\text{f,pl}} \]

\[ F_{\text{R,B1}} < 1.9 \cdot F_{\text{Rd}} \]

**Potential Tension Capacity**

\[ \sigma_{F_{\text{R,B}}} = 246.9 \text{ kN} \]

---

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Step 2: Compression Zone

Potential Compression Capacity

\[ F_{c,\text{Rd}} = \min \left( \frac{M_{c,\text{Rd}}}{(b_d-t_h)} \right) \]

903.4 kN

Step 4: Moment Capacity

Final Bolt Forces and Moment Capacities

| Bolt row 4: | M_{c,\text{Rd}} = (F_{c,\text{Rd}} - F_{\text{v,Rd}}) \cdot h_t | 187.4 - 205.8 = 0.0 \cdot 147.1 | 0.0 kN.m |
| Bolt row 3: | M_{c,\text{Rd}} = (F_{c,\text{Rd}} - F_{\text{v,Rd}}) \cdot h_t | 187.4 - 18.4 = 169.0 \cdot 227.1 | 38.4 kN.m |
| Bolt row 2: | M_{c,\text{Rd}} = (F_{c,\text{Rd}} - F_{\text{v,Rd}}) \cdot h_t | 187.4 - 307.1 | 57.5 kN.m |
| Bolt row 1: | M_{c,\text{Rd}} = (F_{c,\text{Rd}} - F_{\text{v,Rd}}) \cdot h_t | 246.9 - 387.1 | 95.6 kN.m |

\[ M_{\text{c,Rd}} = \frac{M_{\text{Ed}} \cdot N_{\text{Ed}}}{h_t} \]

235.0 - 300.0 \cdot 220.4

168.9 kN.m

Tension Bolts

Only the first 3 rows are required to resist the applied moment.

The remaining rows shall be considered to be part of the shear zone.

95.6, 57.5, 38.4

191.5 kN.m

603.4 kN

532.2 kN

Step 5: Shear Bolts

| Bearing \( F_{\text{R,Ed}} \): End Plate, End | \( e_1=60, e_2=60, k_i=2.5, \gamma_0=0.91, d=20, t=15, f_y=410 \) | 223.6 kN |
| Bearing \( F_{\text{R,Ed}} \): End Plate, Inner | \( p_1=80, e_2=60, k_i=2.5, \gamma_0=0.96, d=20, t=15, f_y=410 \) | 236.7 kN |

Min(236.7, 94.1)

94.1 kN

Min(236.7, 26.3)

26.3 kN

Min(223.6, 26.3)

236.3 kN

494.1

376.3 kN

+26.3 + 26.3

158.1 kN

376.3 + 158.1

534 kN

222.8 N/mm²

Step 7: Welds

Beam \( f_{w,\text{Ed}}=f_d/\sqrt{3} \cdot b_h / (2b_h) / \gamma_2 \)

410.0 / \sqrt{3} \cdot 0.85 / 1.25

827.0 kN

OK

Flange Tension Weld

\( F_{\text{w,Ed}} = \min(1.2 \cdot T_{\text{Ed}}, F_{\text{w,Ed}} + F_{\text{v,Ed}}) \)

Min(189.9\cdot12.7\cdot275, 217.8 + 165.3)

383.1 kN

2\cdot1.225\cdot0.7\cdot187.9 \cdot 2.07\cdot223

827.0 kN

OK

Flange Compression Weld

Web Welds in Tension Zone

\( L\text{w}=L_{\text{proj}}-T_{\text{root}}+1.73\cdot g/2 \)

240 - 20 - 12.7 - 10.2 + 1.73 \cdot 90/2

275.0 mm

Load per row

Row 1: \( F_{\text{w,Ed}} = 10002 \cdot (141/36 + 1.41) \cdot 247 \)

131.4 kN

Row 2: \( F_{\text{w,Ed}} = 10002 \cdot 187 \)

187.4 kN

Row 3: \( F_{\text{w,Ed}} = 10002 \cdot 187 \)

169.0 kN

Total Load \( F_{\text{c,Ed}} \)

131.4, 187.4, 169.0

487.9 kN

2\cdot1.225\cdot0.7\cdot275 + 2\cdot1.225\cdot0.7\cdot275 + 2\cdot1.225\cdot0.7\cdot275 + 2\cdot1.225\cdot0.7\cdot275 + 2\cdot1.225\cdot0.7\cdot275 + 2\cdot1.225\cdot0.7\cdot275 + 2\cdot1.225\cdot0.7\cdot275

636.6 kN

OK

Web Welds in Shear Zone

\( L\text{w}=D(T_\text{Ed}+T_\text{w})-T_{\text{root}} \cdot L\text{w} + 2.7 \cdot 132.7 \cdot 223 \)

453.4 - 25.4 - 10.2 - 10.2 - 275

132.7 mm

2\cdot0.7\cdot132.7\cdot223

250.7 kN

Require More Shear Capacity

Determining Residual Shear Capacity of Tension Zone Weld.

Load per row

Row 1: \( F_{\text{w,Ed}} = 10002 \cdot (141/36 + 1.41) \cdot 247 \)

131.4 kN

Row 2: \( F_{\text{w,Ed}} = 10002 \cdot 187 \)

187.4 kN

Row 3: \( F_{\text{w,Ed}} = 10002 \cdot 187 \)

169.0 kN

Total Load \( F_{\text{c,Ed}} \)

131.4, 187.4, 169.0

487.9 kN

2\cdot1.225\cdot0.7\cdot275 + 2\cdot1.225\cdot0.7\cdot275 + 2\cdot1.225\cdot0.7\cdot275 + 2\cdot1.225\cdot0.7\cdot275 + 2\cdot1.225\cdot0.7\cdot275 + 2\cdot1.225\cdot0.7\cdot275 + 2\cdot1.225\cdot0.7\cdot275

636.6 kN

OK

Tension Utilisation

487.9/636.6

0.77

Shear Utilisation Available \( V_s \)

\( \sqrt{(1 - 0.77)} \)

0.64

Residual \( V_s = f_{w,Ed}(D\cdot2\cdot0.7\cdot6)\cdot275 \)

233.0(6.42+2\cdot0.7\cdot6)\cdot275

333.88 kN

Total Shear Capacity \( V_s + V_t \)

333.88 + 250.73

584.6 kN

OK

Asymmetrical Base-Plate
Base-Plate Connection to EC 3 (UK NAD)
Loading Case 001

Basic Data

Resultant Forces M, Fv, F
- Moment +125.0 kNm, Shear +500.0 kN, Axial +1300.0 kN
  (Left side in tension, Axial Compression)

Concrete Pressure
- 1.00
  OK

Base-Plate thickness in Compression
- 0.96
  OK

Horizontal Shear
- 0.86
  OK

Flange & Web Welds
- 0.72
  OK

Step 2 - Compression T-Stub
- Using SCI default Value
- 1.50

Step 1-2: Base-Plate Pressure
- 96.2 mm

Summary of Results

User Defined Applied Forces at Interface

D=402.6, B=177.7, T=10.9, t=7.7, r=10.2, py=275
Grade 8.8 Bolts
All weld grades provided to suit minimum connected steel grade
20 N/mm², 32 N/mm², 0.35 N/mm², 265 N/mm², 30 deg to vertical
SCI Green Book
P398: Joints in steel construction: Moment-Resisting Joints to Eurocode 3

Basic Dimensions

Column: 406x178UB54 [28]
Bolts 20 mm Ø in 24 mm holes
Plates S 275
Groat Fck, Conc Fck, fy, slope
Design to

Base-Plate 600 x 390 x 30 mm (55 mm holes with 7 No. 24 mm holes)
For 20 mm Grade 8.8 Bolts.
All Plates S 275
Reinforcement 12 @ 200 mm cc each way. Cover 0 mm

Bearing Stress Fjd = βJ • α • Fcd
Fcd = αcc • Fck / γc

Contact

Pressure Configuration
- Compression Only with Optimised Compression Area

Pressure = P=1000/4

Ac = x<wi> • x<wi> • x<wi> • x<wi>

Nc = M/cb/Fc

h = h <tb>

\[
F_{c,cr} = \frac{M_{cr}}{h_t} = \frac{221.6 \times 1000}{(391.70)} = 565.8 \text{ kN}
\]

**Step 2a: Plate Compression Bending**

\[
e = \frac{C}{63.4} = 63.4 \text{ mm}
\]

**Warning**

\[
M_{app} = \frac{pe^2}{2} = 18.1 \times 63.4^2 = 36403 \text{ Nmm/mm}
\]

**Step 4: Shear**

**Base Friction**

Friction Fr=0.30*Fc = 0.30 + 1300.0 = 390.0 kN

**Check 4a: Bolt Shear**

\[
\varphi = 0.44 - 0.0003 \times \frac{640}{1.25} = 0.248
\]

\[
F_{v,Rd} = \frac{\varphi \times f_{ub} \times A}{\gamma_{M2}} = \frac{0.248 \times 800 \times 245.0}{1.25} = 38.9 \text{ kN}
\]

**Check 4b: Base-Plate Bearing**

\[
\begin{align*}
P_1 & = 160.0, \ P_2 = 50.0, \ P_3 = 300.0, \ P_4 = 45.0, \ P_5 = 0.694, \ P_6 = 2.500 \\
F_{b,Rd} & = k_1 \times \gamma_{M2} = 2.5 \times 0.694 \times 410 \times 20 \times 30.0 / 1.25 = 341.7 \text{ kN}
\end{align*}
\]

**Total Shear Capacity**

\[
\begin{align*}
\text{Total Cap} & = F_r + P_{s,s} + P_{t,s} = 390.0 + 54.4 + 136.0 = 580.4 \text{ kN}
\end{align*}
\]

**Step 5: Flange & Web Welds**

\[
F_{app} = M_{la} - F_{A} \times A / A = 125.0 / (402.6 - 10.9) - 1300.0 \times 19.4 / 69.0 = -46.1 \text{ kN}
\]

No Resultant Tensile Force

**Web Welds**

\[
\begin{align*}
\text{Web weld load} & = F_v / (D - 2(f_w + T)) = 500.0 / (402.6 - 2(6 + 10.9)) = 1.36 \text{ kN/mm} \\
\text{Weld } f_{w,d} & = f_{y} / 1.73 = 410 / 1.73 / 0.85 / 1.25 = 223 \text{ N/mm}^2 \\
\text{Fcap} & = 2 \times 0.7 \times f_{w,d} = 2 \times 0.7 \times 223 = 1.87 \text{ kN/mm}
\end{align*}
\]

OK
Base-Plate CHS
Base-Plate Connection to EC 3 (UK NAD)
Loading Case 001

Basic Data

User Defined Applied Forces at Interface
Resultant Forces M, Fv, F
Moment +225.0 kNm, Shear +470.0 kN, Axial +1605.0 kN
(Left side in tension, Axial Compression)

Basic Dimensions
Column: 508x12CHS [28]
Ø=508.0, T=12.0, py=275
Bolts 30 mm Ø in 35 mm holes
Grade 8.8 Bolts
Plates S 275
All weld grades provided to suit minimum connected steel grade
Grout Fck, Conc Fck, fy, slope
12 N/mm², 12 N/mm², 0.35 N/mm², 265 N/mm², 30 deg to vertical
Design to
SCI Green Book
P398: Joints in steel construction: Moment-Resisting Joints to Eurocode 3

Column Capacities Mc, Fvc, Fc
812.0 kN.m, 1781.3 kN, 5142.2 kN
Fc = 5142.2 kN OK

Summary of Results (Unity Ratios)
Concrete Pressure 1.00 OK
Base-Plate thickness in Compression 0.97 OK
Horizontal Shear 0.61 OK
Weld 0.39 OK

Step 2 - Compression T-Stub
Using SCI default Value
Pad Diffusion Ratio \( \alpha \)
0.85 • 12.00 / 1.50
6.80 N/mm²
Grout Ratio \( \beta_g \)
0.67
Bearing Stress \( F_{bd} = \beta_g \cdot \alpha \cdot F_{cd} \)
0.67 • 1.50 • 6.80
6.80 N/mm²
Max \( c = T_{cp} \cdot (f_{cp} / (3\cdot f_{cp}^2 \cdot \gamma_{M0})) \)
35 • (265 / 3 • (6.80 • 1.0))
126.1 mm

Step 1: Base-Plate Pressure
Using SCI default Value
Pad Diffusion Ratio \( \alpha \)
0.85 • 12.00 / 1.50
6.80 N/mm²
Grout Ratio \( \beta_g \)
0.67
Bearing Stress \( F_{bd} = \beta_g \cdot \alpha \cdot F_{cd} \)
0.67 • 1.50 • 6.80
6.80 N/mm²
Base ecc=M/F
225.0/1605.0
140.2 mm
Pressure Configuration
Compression Only with Optimised Compression Area
Step 2a: Plate Compression Bending

\[ e = C \]
\[ M_{\text{app}} = p e^2 / 2 \]
\[ t_p = \sqrt{6 \cdot M_{\text{app}} \cdot \gamma_{M0} / p_y} \]

Step 4: Shear

Base Friction

Friction \( F_r = 0.30 \cdot F_c \)

Check 4a: Bolt Shear

\[ \phi_{ds} = 0.44 - 0.0003 \cdot f_{ub} \]
\[ F_{v,Rd} = \phi_{ds} \cdot f_{ub} \cdot A / \gamma_{M2} \]

Check 4b: Base-Plate Bearing

\[ F_{b,Rd} = k_1 \cdot \phi_b \cdot f_{up} \cdot d \cdot t_p / \gamma_{M2} \]

Total Shear Capacity

Total Cap = \( F_r + P_{ss} \)

Step 5: Weld

Axial Load \( F_{ax} = F / (\pi \cdot D) \)

Moment load \( F_m = M / (D \cdot D) \)

Shear load \( F_v = V / (\pi \cdot D \cdot 0.6) \)

Shear (Longitudinal)

\[ F_{cap} = 1 \cdot 0.7 \cdot f_{w,d} \cdot 1.25 \]

Beam-Splice Outer Plates

Non Bearing - Beam to Beam Moment Splice Connection to EC 3 (UK NAD)

Loading Case 001

Basic Data

User Defined Applied Forces at Interface

| Resultant Forces M, Fv, F | +200.0 kNm, +0.0 kN, +0.0 kN |
| Beam Gap= 5 mm | Therefore No direct bearing. |
| HSFG Bolts | Non-slip at service |
| Design to | SCI Green Book. |
| | P398: Joints in steel construction: Moment-Resisting Joints to Eurocode 3 |

Basic Dimensions

Beam-533x210UB101 [28] | D=536.7, B=210.0, T=17.4, t=10.8, r=12.7, py=265 |
| Bolts 20 mm Ø in 22 mm holes | HSFG - Pt 2 Bolts, Non-slip at Service |
| Plates S 275 | All weld grades provided to suit minimum connected steel grade |
| Beam Capacities Mc, Fvc, Fc | 692.2 kN.m, 946.7 kN, 3409.5 kN |
| | Mc = 692.2 kN.m OK |

Summary of Results (Unity Ratios)

| Top Flange in Compression Bolt Capacity | 0.45 OK |
| Top Flange in Compression Axial Capacity | 0.32 OK |
| Bottom Flange in Tension Bolt Capacity | 0.60 OK |
| Bottom Flange in Tension Axial Capacity | 0.32 OK |
| Web Bolt Capacity | 0.41, 0.34 OK |
| Web Plate Shear Capacity | 0.00, 0.00, 0.00 OK |
| Web Plate Moment Capacity | 0.35 OK |
| Web Axial Capacity | 0.00 OK |

Resultant Forces

| Distribution of Moments | Total 200.0 kNm, Flanges 164.0 kNm, Web 36.0 kNm |
| Flange forces | Min Flange Force=10% • Ar • py |
| | Top Flange Force=Ff / 2-Mf / d |
| | Bot Flange Force=Ff / 2+Mf / d |
| | 10% • 3654 • 265 |
| | 0.0 / 2 - 164.0 / 0.537 |
| | 0.0 / 2 + 164.0 / 0.537 |
| | 96.8 kN |
| | -305.6 kN |
| | 305.6 kN |
Web Forces
M res Fm(Fv, ecc, Mecc, Mweb) 0.0, 95.0, 0.0, 36.0 36.0 kN.m
Lever Arms Lv, Lh, Ld 135.0, 45.0, 142.3
Moment Forces Fmv, Fmh 16.7, 50.0
Shear Component Fv=Fv / bolts 0.0 / 8 0.0 kN
Axial Component Fp=Fp / bolts 0.0 / 8 0.0 kN

Resultant Web Bolt Forces
Fv=Fmv+Fp 50.0 + 0.0 50.0 kN
Fv=Fmv+Fp 16.7 + 0.0 16.7 kN
Fv=Fmv+Fp \sqrt{(Fvx^2+Fvy^2)} 52.7 kN
Bolts In Line

Top Flange in Compression

Bolt Shear Capacity
Outer Plate: P1, e1, P2, e2, c0h, k1 70.0, 35.0, 120.0, 45.0, 0.530, 2.500
Fb,R=K \cdot \sigma_{fb}\cdot d t / \gamma_m
2.5 \cdot 0.530 \cdot 410 \cdot 20 \cdot 20.0 / 1.25 173.9 kN
Flange: P1, e1, P2, e2, c0h, k1 70.0, 52.5, 120.0, 45.0, 0.795, 2.500
Fb,R=K \cdot \sigma_{fb}\cdot d t / \gamma_m
2.5 \cdot 0.795 \cdot 410 \cdot 20 \cdot 17.4 / 1.25 227.0 kN

Plate and Flange Capacity
Outer Plate \max(p_{1p}, p_{1i}) 110
Buckling factor, K 1.00
Nv,fp,rd Outter Plate \kappa T+B/P \cdot \gamma_m
1.00 \cdot 20.0+210.0+265 / 1.0 1113.0 kN
Nh,fp,rd Flange \kappa T+B/P \cdot \gamma_m
17.4 \cdot 210.0+265 / 1.0 968.3 kN
Resultant Axial load
Axial Capacity
Min(1113.0, 968.3) 968.3 kN

Bottom Flange in Tension

Bolt Shear Capacity
Outer Plate: P1, e1, P2, e2, c0h, k1 90.0, 35.0, 120.0, 45.0, 0.530, 2.500
Fb,R=K \cdot \sigma_{fb}\cdot d t / \gamma_m
2.5 \cdot 0.530 \cdot 410 \cdot 20 \cdot 20.0 / 1.25 173.9 kN
Flange: P1, e1, P2, e2, c0h, k1 90.0, 42.5, 120.0, 45.0, 0.644, 2.500
Fb,R=K \cdot \sigma_{fb}\cdot d t / \gamma_m
2.5 \cdot 0.644 \cdot 410 \cdot 20 \cdot 17.4 / 1.25 183.8 kN

Plate and Flange Capacity
Outer Plate
Fb,R=K \cdot \sigma_{fb}\cdot d t / \gamma_m
20.0 \cdot 210.0+265 / 1.0 1113.0 kN
Flange
Fb,R=K \cdot \sigma_{fb}\cdot d t / \gamma_m
17.4 \cdot 210.0+265 / 1.0 968.3 kN
Resultant Axial load
Axial Capacity
Min(1113.0, 968.3) 968.3 kN

Web Zone

Bolt Shear Capacity
Plate: P1, e1, P2, e2, c0h, k1h, c0h, k1h 90.0, 50.0, 90.0, 50.0, 0.758, 2.500, 0.758, 2.500
Fb,R=K \cdot \sigma_{fb}\cdot d t / \gamma_m
2.5 \cdot 0.758 \cdot 410 \cdot 20 \cdot 16.0 / 1.25 198.8 kN
Flange
Fb,R=K \cdot \sigma_{fb}\cdot d t / \gamma_m
2.5 \cdot 0.758 \cdot 410 \cdot 20 \cdot 16.0 / 1.25 198.8 kN

Plate Capacity per plate

App. Shear V/2

\[
V_{Rd,\text{App}} = \left( \frac{h}{p} \cdot \frac{t_p}{\gamma_{M0}} \right) \left( \frac{f_{yp}}{\sqrt{3}} \right)
\]

\[
V_{Rd,\text{App}} = \left( \frac{2296}{856} \right) \left( \frac{275}{\sqrt{3}} \right)
\]

\[
V_{Rd,\text{App}} = 5237 \text{ mm}^2
\]

\[
V_{w,n,Rd} = A_{v,\text{net}} \cdot \frac{t_{w,\text{net}}}{\gamma_{M2}}
\]

\[
V_{w,n,Rd} = 5237 \cdot 1000 \cdot 1.25
\]

\[
V_{w,n,Rd} = 127.5 \text{ kN}
\]
Column Splice with division Plate

Bearing - Column Splice Connection to EC 3 (UK NAD)

Basic Data

User Defined Applied Forces at Interface

<table>
<thead>
<tr>
<th>Force</th>
<th>Description</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>F_C</td>
<td>Maximum Factored Axial Load</td>
<td>150.0 kN</td>
<td>Compression</td>
</tr>
<tr>
<td>F_CD</td>
<td>Minimum Factored Axial Load</td>
<td>140.0 kN</td>
<td>Compression</td>
</tr>
<tr>
<td>M</td>
<td>Factored Moment</td>
<td>25.0 kN.m</td>
<td></td>
</tr>
<tr>
<td>F_v</td>
<td>Factored Shear Force</td>
<td>0.0 kN</td>
<td></td>
</tr>
</tbody>
</table>

Web Division Plate

Direct Bearing assumed

All Moment and Axial loads resisted by Flanges

All Shear loads resisted by Web bolts and Friction

HSFG Bolts

Non-slip at service

Ultimate/Service factor for service force calculation. 1.2

Design to


SCI Green Book

P358: Joints in steel construction: Simple Joints to Eurocode 3

Basic Dimensions

Upper Column-203x203 UC 46 [S275]

Lower Column-254x254 UC 73 [S275]

External Flange Plates

2/546 x 190 x 10 mm (2x8 kg)

External Packers

2/256 x 190 x 25 (approx) mm (2x10 kg)

Welded to Flange with 6 mm FW (top & sides)

Bearing Divider Plate

203 x 205 x 25 mm (10 kg)

Web Angle Cleats

4/90 x 90 x 8 Ang x 150 mm (4x2 kg)

2 bolts per leg, 90 cc with 50 back mark

HSFG - Pt 2 Bolts (non-slip at service)

32 No. 20 mm Ø Bolts

In 22 mm Holes

All Plates S 275

Column Prepared for Direct Bearing

Summary of Results (Unity Ratios)

<table>
<thead>
<tr>
<th>Force</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>F_p</td>
<td>Flange &amp; Flange Capacity Tension</td>
<td>0.12</td>
</tr>
<tr>
<td>F_b</td>
<td>Flange &amp; Flange Capacity Bolt Capacity</td>
<td>0.16 OK</td>
</tr>
<tr>
<td>F_w</td>
<td>Flange &amp; Flange Capacity Packer Weld</td>
<td>0.05 OK</td>
</tr>
</tbody>
</table>

Resultant Forces

<table>
<thead>
<tr>
<th>Force</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>F_1p</td>
<td>+M/D+F_C/2</td>
<td>198.0 kN</td>
</tr>
<tr>
<td>F_1n</td>
<td>-M/D+F_C/2</td>
<td>-53.0 kN</td>
</tr>
<tr>
<td>F_2p</td>
<td>+M/D-F_C/2</td>
<td>193.0 kN</td>
</tr>
<tr>
<td>F_2n</td>
<td>-M/D-F_C/2</td>
<td>-53.0 kN</td>
</tr>
<tr>
<td>F_1</td>
<td>max(0, F_1p, F_1n, F_2p, F_2n)</td>
<td>198.0 kN</td>
</tr>
<tr>
<td>F_2</td>
<td>min(0, F_1p, F_1n, F_2p, F_2n)</td>
<td>-53.0 kN</td>
</tr>
</tbody>
</table>

### Detailing Practice

#### Bearing

Bearing provided

\[ T_{\text{req}} = \frac{(D_1 - 2) \times T_{\text{v}}}{2} \]  

\[ ((254.1 - 2 \times 14.2) - (203.2 - 2 \times 11.0))/2 \]  

\[ 25.0 \text{ mm} \]

\[ 22.3 \text{ mm} \]

OK

#### External Cover Plates

- \[ T_{\text{c}} = T_{\text{v}} / 2 \]  
- \[ L_\text{p} = B_1 \times 225 \]  
- \[ W_p = B_1 \times 6 \]  
- \[ H \text{ c/c} = \text{Table G.24} \]  
- \[ \text{Packer} < 4 \times \Phi \]  

\[ \text{min} = 2 \]  

\[ 25.5 \text{ mm} \]  

welded OK

- \[ \text{OK} \]

#### Plate & Flange Capacity in Compression

- \[ N = 0.9 \times (T \times B) \]  
- \[ \gamma = \gamma_M - 2 \times T \times l \]  

\[ \gamma = 0.9 \times (10.0 \times 146.0 - 2 \times 81) \times 410 / 1.1 \]  

\[ 10.0 \times 190.0 \times 275 / 1.1 \]  

\[ 508.2 \text{ kN} \]

#### Compression Capacity

- \[ C = \min \{ F \text{p}, \text{Fy}, \text{Ft} \} \]  
- \[ \gamma = \gamma_M - 2 \times T \times l \]  

\[ \gamma = 0.9 \times (10.0 \times 146.0 - 2 \times 81) \times 410 / 1.1 \]  

\[ 10.0 \times 190.0 \times 275 / 1.1 \]  

\[ 508.2 \text{ kN} \]

#### Plate & Flange Capacity in Tension

- \[ \gamma = \gamma_M - 2 \times T \times l \]  
- \[ \gamma = \gamma_M - 2 \times T \times l \]  

\[ \gamma = 0.9 \times (10.0 \times 146.0 - 2 \times 81) \times 410 / 1.1 \]  

\[ 10.0 \times 190.0 \times 275 / 1.1 \]  

\[ 508.2 \text{ kN} \]

#### Flange Bolts Shear Capacity

- \[ B = \min \{ F \text{p}, \text{Fy}, \text{Ft} \} \]  
- \[ \gamma = \gamma_M - 2 \times T \times l \]  

\[ \gamma = 0.9 \times (10.0 \times 146.0 - 2 \times 81) \times 410 / 1.1 \]  

\[ 10.0 \times 190.0 \times 275 / 1.1 \]  

\[ 508.2 \text{ kN} \]

#### Tension Capacity

- \[ C = \min \{ F \text{p}, \text{Fy}, \text{Ft} \} \]  
- \[ \gamma = \gamma_M - 2 \times T \times l \]  

\[ \gamma = 0.9 \times (10.0 \times 146.0 - 2 \times 81) \times 410 / 1.1 \]  

\[ 10.0 \times 190.0 \times 275 / 1.1 \]  

\[ 508.2 \text{ kN} \]

#### Flange Bolts Shear Capacity

- \[ B = \min \{ F \text{p}, \text{Fy}, \text{Ft} \} \]  
- \[ \gamma = \gamma_M - 2 \times T \times l \]  

\[ \gamma = 0.9 \times (10.0 \times 146.0 - 2 \times 81) \times 410 / 1.1 \]  

\[ 10.0 \times 190.0 \times 275 / 1.1 \]  

\[ 508.2 \text{ kN} \]

**OK**

### Flange Bolts Shear Capacity

- \[ B = \min \{ F \text{p}, \text{Fy}, \text{Ft} \} \]  
- \[ \gamma = \gamma_M - 2 \times T \times l \]  

\[ \gamma = 0.9 \times (10.0 \times 146.0 - 2 \times 81) \times 410 / 1.1 \]  

\[ 10.0 \times 190.0 \times 275 / 1.1 \]  

\[ 508.2 \text{ kN} \]

**OK**
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**660x20CHS [S275]**

End-Plate 860x15 mm (68 kg)

Plates S 275

12 No. M20 Grade 8.8 Bolts in 22 mm holes

On 760 mm PCD

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**CHS Hollow Section Splice**

**Hollow Section Splice Connection to EC 3 (UK NAD)**

**Loading Case 001**

**Basic Data**

**User Defined Applied Forces at Interface**

- **Applied Forces**: \( F = -300.0 \text{ kN} (T), M_{xx} = +30.0 \text{ kNm}, \)
- **Design to**: EC 3: Part 1-8: 2005 Design of Joints
  - SCI Green Book
  - P358: Joints in steel construction: Simple Joints to Eurocode 3

**Basic Dimensions**

- **Beam-660x20CHS [28]**: \( \phi = 660.0, T = 20.0, py = 265 \)
- **Bolts 20 mm Ø in 22 mm holes**: Grade 8.8 Bolts
- **Plates S 275**: All weld grades provided to suit minimum connected steel grade

**Summary of Results (Unity Ratios)**

- **End-Plate & Bolts**: 0.35, 0.39, 0.25
- **Welds**: 0.04, 0.01, 0.04
- **Section Capacity**: 0.39 OK
- **Compression zone**: \( H_{\text{comp}} = 5.00 \Rightarrow A \cdot Py = 3.82 \text{ cm}^2 \cdot 265 \text{ N/mm}^2 \)
- **F_{\text{max}} = M / \sum L_a**: 30/3036.4
- **Design F = F - F_{\text{moment}} N_{tot}**: -300 - 9.9 • 12

- **Check 1: Detailing Practice**
  - Bolt Pitch: 2.5 • 20 \( \Rightarrow P \leq 10 \cdot 20 \) 50 \( \Rightarrow P \leq 200 \)
  - 199 mm OK

**Additional Bolt force from Moment**

- **P_{10} = P_{net} / A**
  - 265 • -300/402.12
  - 265 N/mm²

- **Compression zone**
  - \( H_{\text{comp}} = 5.00 \Rightarrow A \cdot Py = 3.82 \text{ cm}^2 \cdot 265 \text{ N/mm}^2 \)
  - 101.31 kN

- **F_{\text{max}} = M / \sum L_a**
  - 30/3036.4
  - 9.88 kN

- **Design F = F - F_{\text{moment}} N_{tot}**
  - -300 - 9.9 • 12
  - -418.56 kN
Check 2: Complete end-plate yielding

\[ K_1 = \ln(r_2 / r_3) \]
\[ K_2 = K_1 + 2 \]
\[ K_3 = 1/(2K_1 + 1)(K_1^2 / 4 - K_0) \]
\[ F_{int} = \frac{\pi^2}{2\gamma} \frac{F_3}{2} \]
\[ F_t = \frac{1}{2K_1} \left( K_3 + \sqrt{K_3^2 - 4K_1} \right) \]
\[ \frac{1}{2 \times 0.172} \left( 2.172 + \sqrt{2.172^2 - 4 \times 0.172} \right) \]
\[ 12.2 \text{ kN} \]
\[ K_3 = K_1 + 2 \]
\[ F_t = \frac{1}{2 \times 0.172} \left( 2.172 + \sqrt{2.172^2 - 4 \times 0.172} \right) \]
\[ 1181.8 \text{ kN} \]

Check 3: Bolt failure with end-plate yielding

\[ F_{tb} = \frac{K_3}{\gamma} \frac{F_{t,rb}}{\gamma_{np}} \]
\[ F_t = N \cdot F_{t,rb} / (1 - 1/F_3 + 1/(F_3 \cdot \ln(r_1/r_2))) \]
\[ 12 \times 141.1 / (1 - 1 / 12.16 + 1 / (12.16 \times \ln(430.0/380.0))) \]
\[ 1069.7 \text{ kN} \]

Check 4: Bolt Failure

\[ F_{tb} = F_t \cdot F_{t,rb} \]
\[ 12.0 \times 141.1 \]
\[ 1693.4 \text{ kN} \]

Check 5: Welds

\[ f_{w,sc} = f_{w} / (\sqrt{3} \cdot \theta_{sc} \cdot \gamma_{sc}) \]
\[ F_w = F / \pi D \]
\[ Z_{weld} = \pi (D_1^4 - D_2^4) / (32 \cdot D_1) \]
\[ F_{w,sc} = M / Z_{weld} \]
\[ F_t = 0.145 + 0.088 \]
\[ P_t = 0.7 \times f_{w,sc} \]
\[ 0.145 + 0.088 \]
\[ 0.233 \text{ kN/mm} \]

Weld in Tension (Load per mm run)

\[ f_{w,sc} = f_w / (\sqrt{3} \cdot \theta_{sc} \cdot \gamma_{sc}) \]
\[ 410.0 / (\sqrt{3} \times 0.85 \times 1.25) \]
\[ 222.8 \text{ N/mm}^2 \]

Check 6: Member Capacity

\[ F_{cap} = A \cdot p_y \]
\[ M_{cap} = F_n(2\alpha, S_{xx}, p_y) \]
\[ Combined F_{app}/F_{cap} + M_{app}/M_{cap} \]
\[ 300/10656.3 + 30/2171.6 \]
\[ 0.042 \text{ kN} \]

OK
Beam to Beam Flexible End-Plate - with Asymmetrical Plate Girders
Beam to Beam Flexible End-plate Connection to EC 3 (UK NAD)

Basic Data

User Defined Applied Forces at Interface
Shear Forces
Left = 235.0 kN, Right = 170.0 kN

Tie Force
75.0 kN

Design to

SCI Green Book
P358: Joints in steel construction: Simple joints to Eurocode 3

Basic Dimensions
Left-457x191UB82 [28]
Supporting-250x25T+160x20B+405x15WPg121.87 [28]
Right-356x171UB57 [28]

Bolts 20 mm Ø in 22 mm holes
Plates S 275

All weld grades provided to suit minimum connected steel grade

Summary of Results (Unity Ratios)

Left Hand Beam
Check 4 Supported Beam Web Shear
410.2 >= 235.0kN
0.57  OK

Checks 1 & 2 Detailing Practice
1.00, 0.83, 1.00, 0.64, 0.710.94
1.00  OK

Check 5: Notch Resistance
0.50

Check 6: Notch Stability
0.54

Checks 8 & 9 Bearing & Shear
451.6, 451.6, 451.6, 964.1, 760.8 >= 235.0kN
0.52  OK

Check 10 Shear
596.7, 626.2 >= 181.3kN
0.30  OK

Check 10 Combined Bearing
199.4 >= 60.4kN
0.30  OK

Check 11-14 Tie Forces
296.9, 506.3, 962.2, 1070.1 >= 235.0kN
0.79  OK
Right Hand Beam

Check 4: Supported Beam Web Shear 335.7 >= 170.0 kN 0.51 OK
Checks 1 & 2 Detailing Practice 1.00, 0.83, 1.00, 0.64, 0.710.77 1.00 OK
Check 5: Notch Resistance 0.39 0.39 OK
Check 6: Notch Stability 0.35 0.35 OK
Checks 8 & 9 Bearing & Shear 602.1, 602.1, 602.1, 869.4, 691.0 >= 170.0 kN 0.28 OK
Check 11-14 Tie Forces 371.5, 663.0, 1282.9, 875.5 >= 170.0 kN 0.46 OK

Left Hand Beam

Check 1: Recommended Detailing Practice
Plate Depth >= 0.6D 0.6 • 460 = 276 mm 290 mm OK
10 >= t_q <= 12 10.0 mm OK
90 <= p_3 <= 140 90 mm OK

Check 4: Supported Beam Web Shear
\[ V_{pl,rd} = \frac{0.9 \times t_{q} \times A_{pl}}{\gamma_{md}} \] (\( \sqrt{3} \cdot \gamma_{md} \)) 0.9 • 9.9 • 290 • 275 / (\( \sqrt{3} \cdot 1.0 \)) 410.2 kN OK

Check 2: Supported Beam Welds
a_0 = a_1 = 0.4 • t_{q} = 9.9 0.4 • 9.9 4.0 mm OK
a_1 = a_2 \times V_{pl,rd} = 1.27 4.0 • 235.0 / 410.2 • 1.27 (SCI AD370) 2.9 mm OK
S_{min} = \max(3, \min( a_1, a_2)) / 0.7 4.3 mm OK

Check 5: Supported Beam Notch Resistance
\[ V_{rd} = 0.9 \times A_{pl} \cdot DN \cdot t_{q} / (\sqrt{3} \cdot \gamma_{md}) \] \[ M_{rd} = \min(V_{rd}, M_{rd} / \gamma_{md}) \] \[ V_{rd} = \min(V_{rd}, M_{rd} / \gamma_{md}) \] 0.9 • 3713 • 275 / (\( \sqrt{3} \cdot 1.0 \)) 530.5 kN Low Shear
63.8 kN.m 472.7 kN OK

Check 6: Supported Beam - Notch Stability (restrained)
\[ D_{0} <= h / 5 \] 50.0 <= 460.0 / 5 mm 50.0 <= 92.0 OK
\[ L_{0} <= h_{0} \] 125 <= 460 OK

Check 7: Unrestrained Supported Beam. Overall Stability of Notched Beam
Beam assumed to be restrained ignored

Check 8a: Bolt Shear
\[ F_{t,b} \geq 0.8 \times F_{pl,rd} \times cols \] 0.6 • 800 • 245.0 / 1.25 94.1 kN OK
\[ V_{rd} = 0.8 \times n \times F_{pl,rd} \] 0.8 • 3 • 94.1 • 2 451.6 kN OK

Check 8b: End-Plate Bearing
\[ P_{r}, c_1, P_{s}, c_2, \alpha_{o}, k_{1} \] 70.0, 40.0, 90.0, 30.0, 0.606, 2.118
\[ F_{b,rd} = k_{1} \times \frac{t_{q}}{\gamma_{md}} \] 2.1 • 0.606 • 410 • 20 • 10.0 / 1.25 84.2 kN OK
\[ 0.8 \times F_{b,rd} < F_{b,rd} \] 0.80 • 3 • 94.1 • 2 451.6 kN OK

Check 8c: Supporting Beam Web Bearing
\[ \alpha_{o} = \min( p_{d}/d, 3/4, t_{q} / 1.4 \cdot F_{pl} / \gamma_{md}) \] \[ k_{1,md} = \min(2.5, 1.4 \cdot p_{d} / d, 1.7) \] \[ F_{b,rd} = k_{1} \times t_{q} \] 2.5 • 0.811 • 410 • 20.0 / 1.25 199.4 kN
\[ 0.80 \times F_{b,rd} < F_{b,rd} \] 0.80 • 3 • 94.1 • 2 451.6 kN OK

Check 9a: End-Plate Plane Shear
\[ V_{rd} = 2 \times h_{q} \times F_{pl} / (1.27 \times \gamma_{md}) \] 2 • 290 • 10 • 275 / (1.27 • \( \sqrt{3} \cdot 1.0 \)) 725.1 kN OK
\[ V_{rd} = 2 \times t_{q} \times (h_{q} - h_{0,rd}) \times F_{pl} / (\gamma_{md} / (\sqrt{3} \cdot 1.1)) \] 2 • 100 • (290 - 3) • 22 • 410 / (\( \sqrt{3} \cdot 1.1 \)) 964.1 kN OK

Check 9b: End-Plate Block Shear
\[ A_{w} = A_{w} \times n_{w} / (4) \] (10, 290, 40, 30, 3, 22) 1950, 190 mm²
\[ V_{rd} = 2 \times F_{pl} \times A_{w} / (2 \times (\gamma_{md} / (\sqrt{3} \cdot 1.0))) \] 2 • 410 • 190 / 1.1 + 2751950 / (\( \sqrt{3} \cdot 1.0 \)) 760.8 kN OK

Check 10: Supporting Member - Local Resistance
\[ V_{rd} = V_{rd} + V_{dl} \times n_{sl} / (4) \] (235 + 170 • 3) / 2 (per shear plane) 181.3 kN
\[ A_{w} = A_{w} \times n_{w, sl} / (4) \] (75, 45, 70, 3, 22, 15.0) 3900, 2910 mm²
\[ V_{rd} = A_{w} \times F_{pl} / (\gamma_{md} / (\sqrt{3} \cdot 1.0)) \] 3900 • 265 / (\( \sqrt{3} \cdot 1.0 \)) 596.7 kN OK

Check 10: Combined Load Bearing on supporting web

\[ F_{rd} = V_{rd} + V_{rd}/a \leq F_{v,rd} \]

\[ F_{rd} = 235.0/6 + 170.0/8 \]

(per bolt) 60.4 kN  OK

Check 11: End-Plate in Bending

\[ F_{bd} = \frac{\gamma_{bd} \gamma_{bd} - nS(k_{s} k_{s} / \gamma_{bd})}{(m+n)} \]

\[ F_{bd} = 235.0 \text{kN} \]

Check 12: Beam Web Tension

\[ F_{bd} = 9.9 \times 290 \times 10.0 \times 1.1 \]

1070.1 kN  OK

Check 13: Welds

n/a

Covered by Check 2  OK

Right Hand Beam

Check 1: Recommended Detailing Practice

Plate Depth \( \geq 0.6 \times D \)

10 \( > \) \( t_{b} = 12 \)

90 \( < \) \( p_{b} = 140 \)

Check 4: Supported Beam Web Shear

\[ V_{bd} = \frac{0.9 \times V_{bd}/\gamma_{bd}}{\gamma_{bd}} \]

335.7 kN  OK

Check 2: Supported Beam Welds

\[ V_{d} = \text{max}(0.4 \times t_{b} \times a_{b}, 2) \]

3.2 mm  OK

Check 5: Supported Beam Notch Resistance

\[ V_{bd} = \text{Min}(V_{bd}, M_{bd} / L_{b}) \]

431.6 kN  Low Shear

Check 6: Supported Beam - Single Notched Web Stability (restrained)

\[ D_{bd} = h_{b} / 2 \]

35.0 \( \leq \) 358.0/2 mm  OK

\[ L_{d} = h_{d} / 2 \]

125 \( \leq \) 358  OK

Check 7: Unrestrained Supported Beam, Overall Stability of Notched Beam

Beam assumed to be restrained  ignored

Check 8a: Bolt Shear

\[ F_{bd} = \frac{0.8 \times n \times F_{bd} \times a_{b} / h_{b}}{\gamma_{bd}} \]

602.1 kN  OK

Check 8b: End-Plate Bearing

\[ F_{bd} = k_{1} \times k_{1} \times F_{bd} \times a_{b} / h_{b} \]

84.2 kN  OK

Check 8c: Supporting Beam Web Bearing

\[ F_{bd} = \text{Min}(70/223/4, 800/410, 1) = \text{Min}(0.811, 1.951, 1) \]

0.811

\[ k_{1} = \text{min}(2.5, 1.4 \times 90/22 - 1.7) \]

2.500
### Check 9a: End-Plate Plane Shear

$$V_{Rd} = \frac{2 \times h_p \times t_p \times f_{y} \times (1.27 \times \sqrt[3]{3} \times \gamma_{M2})}{1.27 \times \sqrt{3} \times \gamma_{M0}}$$

<table>
<thead>
<tr>
<th>Condition</th>
<th>Shear Force</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Condition</td>
<td>290 \times 10 \times 275 / (1.27 \times \sqrt[3]{3} \times 1.0)</td>
<td>725.1 kN</td>
</tr>
<tr>
<td>Condition</td>
<td>10 \times 40 \times 22 \times 410 / (\sqrt[3]{3} \times 1.1)</td>
<td>869.4 kN</td>
</tr>
</tbody>
</table>

### Check 9b: End-Plate Block Shear

$$A_{nv}, A_{nt} = F_n(t_p, h_p, e_1, e_2, n_1, d_0)$$

<table>
<thead>
<tr>
<th>Condition</th>
<th>Shear Force</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Condition</td>
<td>(410 \times 190 / 1.1 + 2751730 / (\sqrt[3]{3} \times 1.0))</td>
<td>691.0 kN</td>
</tr>
</tbody>
</table>

### Check 10: Supporting Member - Local Resistance

See other side

### Tie Forces

**Applied Tie Force**

170.0 kN

### Check 11: End-Plate in Bending

$$L_{eff} = e_1 A + (n-1) \times p_1 A + c_1$$

<table>
<thead>
<tr>
<th>Condition</th>
<th>Moment</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Condition</td>
<td>290.0 mm</td>
<td>2.70 kN.m</td>
</tr>
</tbody>
</table>

### Check 12: Beam Web Tension

$$F_{Rd} = t_{bw} \times h_{peff} \times f_{ubw} / \gamma_{Mu}$$

<table>
<thead>
<tr>
<th>Condition</th>
<th>Tension Force</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Condition</td>
<td>8.1 \times 290 \times 410 / 1.1</td>
<td>875.5 kN</td>
</tr>
</tbody>
</table>

### Check 13: Welds

Covered by Check 2
Beam to Beam Fin-Plate
Beam to Beam Fin-plate Connection to EC 3 (UK NAD)

Basic Data

User Defined Applied Forces at Interface

Shear Forces
Left = 250.0 kN, Right = 120.0 kN

Tie Force
75.0 kN

Design to

SCI Green Book
P358: Joints in steel construction: Simple joints to Eurocode 3

Basic Dimensions

Left-457x152 UB 67 [S275]
Top Notch: 35 x 102 mm lg. net
Plate: 10 x 100 x 360 mm dp. (3 kg)
With 5 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Right-457x191 UB 98 [S275]
Top Notch: 35 x 102 mm lg. net
Plate: 10 x 130 x 360 mm dp. (4 kg)
With 8 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Summary of Results (Unity Ratios)

Left Hand Beam
Check 1: Detailing Practice 1.00
Check 2: Bolt Shear & Bearing 0.65, 0.58, 0.69 0.69
Check 3: Sup Beam Connect Elmts. 0.56, 0.46, 0.59, 0.21 0.59
Check 4: Supported Beam Resist. 0.40, 0.40, 0.69 0.69
Check 5: Notch Resistance 0.40 0.40
Check 6: Notch Stability 0.22 0.22
Check 8: Welds 0.890.89
Check 10: Supporting Member 0.34 0.34
Check 11: Structural integrity-Plate 0.28, 0.30, 0.30, 0.5, 0.4 0.50

All weld grades provided to suit minimum connected steel grade

OK

Check 1: Recommended Detailing Practice
Plate Depth ≥ 0.6*D
\( t_{b} \leq 0.5 * F \)
\( t_{w} \leq 0.5 * F \)

Check 2: Supported beam - Bolt Group
\( \beta = (n \sigma / (n \sigma + 1) \rho 1) \)
\( F_{v, net} = F_{v, net} / \gamma_{M} \)
\( V_{n, net} = F_{v, net} \sqrt{(1 + n \sigma)^{2} / (n \sigma)^{2}} \)

Check 3: Sup. Beam Connect Elmts.
Vert P, c1, p2, c2, \( \gamma_{b} \)
\( F_{v, net} = \gamma_{b} \sqrt{(1 + n \sigma)^{2} / (n \sigma)^{2}} \)
\( V_{n, net} = N / ((1 + n \sigma) / F_{v, net}) \)
\( + (N / F_{n, vert. R})^{2} \)

Check 5: Notch Resistance
\( \beta = (6 \times z) / (n \gamma_{b} / 0.6) \)
\( \gamma_{b} \geq 2.73 \times \gamma_{b} \)

Check 6: Notch Stability
\( \beta = (6 \times z) / (n \gamma_{b} / 0.6) \)
\( \gamma_{b} \geq 2.73 \times \gamma_{b} \)

Check 8: Welds
\( V_{n, net} = N / (F_{w, hor. R})^{2} \)
\( + (N / F_{n, vert. R})^{2} \)

Check 11: Structural integrity-Plate
\( \beta = (6 \times z) / (n \gamma_{b} / 0.6) \)
\( \gamma_{b} \geq 2.73 \times \gamma_{b} \)

Check 12: Structural integrity-Beam
\( \beta = (6 \times z) / (n \gamma_{b} / 0.6) \)
\( \gamma_{b} \geq 2.73 \times \gamma_{b} \)

Left Hand Beam

Right Hand Beam
Check 7: Unrestrained Supported Beam. Overall Stability of Notched Beam

Beam assumed to be restrained

Check 8: Supporting Member Welds

\[ a = 0.7 \times \sqrt{h_p} \]

5.6 = 0.7 \times 8 >= 0.5 \times 10.0

OK

Check 10: Supporting Member Local Resistance

i) Combined Shear

\[ V_{Ed,2} = (V_{Ed,1} + V_{Ed,2}) \times h_p \min \]

(250.0 / 360 + 120.0 / 360) \times 360

370.0 kN

OK

ii) Shear

\[ F_{Rd,2} = \frac{2 \times h_p \min \times T}{\sqrt[3]{3} \times \gamma_{fd}} \]

2 \times 360 \times 9.6 \times 275/\sqrt{3} \times 1.0

1097.4 kN

OK

Check 11: Structural integrity - Fin Plate

i) Tension

\[ F_{Rd} = F_{p,d} \times \frac{A}{\gamma_{Mu}} \]

0.6 \times 800 \times 245.0 / 1.1

106.9 kN

OK

\[ F_{Rd} = 5 \times 106.9 \]

534.5 kN

OK

ii) Shear

\[ F_{Rd} = \frac{F_{p,d} \times d}{\gamma_{Mu}} \]

0.0, 50.0, 70.0, 40.0, 0.758, 2.500

141.2 kN

OK

\[ F_{Rd} = 5 \times 141.2 \]

705.9 kN

OK

Check 12: Structural integrity - Supported Beam

i) Tension

\[ F_{Rd} = F_{p,d} \times \frac{A}{\gamma_{Mu}} \]

410 \times 1989 / 1.1

782.8 kN

OK

\[ F_{Rd} = 5 \times 101.7 \]

101.7 kN

OK

ii) Bearing

\[ F_{Rd} = 5 \times 141.2 \]

705.9 kN

OK

Check 1: Recommended Detailing Practice

Plate Depth >= 0.6\( \times \)D

0.6 \times 467.2 = 280.3 mm

360 mm

OK

\[ t_u = 0.5 \times F_{p,d} \]

10.0 mm

OK

\[ t_u = 0.5 \times F_{p,d} \]

10.0 mm

OK

Check 2: Supported beam - Bolt Group

i) Bolts in Shear

\[ l = \frac{p^2}{2} \times \frac{1}{8} \times (n_i - 1) \times p_{1/2} \times \frac{1}{6} \]

8 \times 40/2 + 8 \times (8 - 1) \times 40/6

140800

\[ a = \frac{a_{p,d}}{2} \times l \]

7040/2/2000

0.010

\[ \beta = \frac{a_{p,d}}{2} \times l \times (n_i - 1) \]

7040/2/2000 \times (8 - 1)

0.070

\[ F_{Rd} = \frac{F_{p,d} \times \gamma_{fd}}{\gamma_{Mu}} \]

0.6 \times 800 \times 245.0 / 1.25

94.1 kN

OK

\[ V_{Ed,2} = \frac{F_{Rd} \times \gamma_{fd}}{\gamma_{Mu}} \]

(8 \times 94.08) / \sqrt{(1 + 0.010 \times 8) \times (0.070 \times 8) \times 2}

619.6 kN

OK

Right Hand Beam

ii) Fin plate bearing
Vert P1, c1, P3, c2, c3, k1 40.0, 40.0, 40.0, 40.0, 0.356, 0.845
Fb>Rd=k•σRd•fY•/γM2
0.8 • 0.356 • 410 • 20 • 10.0 / 1.25
19.7 kN
Horiz P1, c1, P2, c2, c3, k1 40.0, 40.0, 40.0, 40.0, 0.356, 0.845
Fb>Rd=k•σRd•fY•/γM2
0.8 • 0.356 • 410 • 20 • 10.0 / 1.25
19.7 kN
Vb=Rd=√(((1+a•N)/ Fb•v•wb)²
8.00 / √(((1+ 0.01 • 8.00) / 19.75)²
+(B/N / Fb•v•wRd)²)
130.1 kN OK

iii) Beam Web bearing
Vert P1, c1, P3, c2, c3, k1 40.0, 40.0, 40.0, 40.0, 0.356, 0.845
Fb>Rd=k•σRd•fY•/γM2
0.8 • 0.356 • 410 • 20 • 11.4 / 1.25
22.5 kN
Horiz P1, c1, P2, c2, c3, k1 40.0, 40.0, 40.0, 40.0, 0.356, 0.845
Fb>Rd=k•σRd•fY•/γM2
0.8 • 0.356 • 410 • 20 • 11.4 / 1.25
22.5 kN
Vb=Rd=√(((1+a•N)/ Fb•v•wb)²
8.00 / √(((1+ 0.01 • 8.00) / 22.51)²
+(B/N / Fb•v•wRd)²)
148.3 kN OK

Check 3: Supported Beam - Connecting Elements
i) Shear
Vb>Rd=(h•sb/ 1.27)•(fY•/√3/γM2)
(360•10/1.27 ) • (275/√3/1.0)
450.1 kN OK

Check 4: Supported Beam - Resistance at the connection
i) Shear
Vb>Rd=Av•w•fY•/γM2
5015.28 • 265.00/√3/1.0
767.3 kN OK
Vb>Rd=Av•w•fY•/γM2
3008.88 • 410.00/√3/1.1
647.5 kN OK
Vb>Rd=0.5•Av•fY•/γM2
((0.5 • 410 • 535.80) / 1.1)
370.2 kN OK

Check 6: Supported Beam - Single Notched Web Stability (restrained)

Check 7: Unrestricted Supported Beam. Overall Stability of Notched Beam
Beam assumed to be unrestrained

Check 8: Supporting Member Welds
a=0.7s >= 0.5 tp
5.6 • 0.7 • 8 >= 0.5 • 10.0
OK

Tie Forces
Applied Tie Force
120.0 kN

Check 11: Structural integrity - Fin Plate
i) Tension
Part Block Tension
Only 1 Shear Plane
Fb>Rd=fY•Av•/γM2
410 • 1550 / 1.1
652.3 kN OK
Fb>Rd=fY•Av•/γM2
275 • 470 / 1.73 / 1.0
617.2 kN OK

ii) Shear
\[ F_{v,Rd} = \frac{\alpha v \cdot f_{ub} \cdot A}{\gamma_{M0}} \]
\[ F_{v,Rd} = 8 \cdot 106.9 \]
\[ 106.9 \text{ kN} \]
\[ 855.3 \text{ kN} \]
\[ \text{OK} \]

iii) Bearing
\[ P_1, c_1, P_2, c_2, \cdots, k_1 \]
\[ F_{b,Rd} = k_1 \cdot \frac{\alpha b \cdot f_{ub} \cdot d \cdot t_p}{\gamma_{M0}} \]
\[ F_{b,Rd} = 8 \cdot 22.4 \]
\[ 22.4 \text{ kN} \]
\[ 179.5 \text{ kN} \]
\[ \text{OK} \]

Check 12: Structural integrity - Supported Beam

i) Tension

Part Block Tension
\[ F_{u.b} = \frac{F_{u,b} \cdot A_{nt}}{\gamma_{M0}} \]
\[ 410 \cdot 1767 / 1.1 \]
\[ 410 \cdot 1436 / 1.1 \]
\[ 410 \cdot 1072 / 1.73 \]
\[ 699.3 \text{ kN} \]
\[ \text{OK} \]

Full Block Tension
\[ F_{b,Rd} = 0.9 \cdot F_{u,b} \cdot A_{nt,p} / \gamma_{M0} \]
\[ 0.9 \cdot 410 \cdot 2921 / 1.1 \]
\[ 979.8 \text{ kN} \]
\[ \text{OK} \]

ii) Bearing

Part Block Tension
\[ F_{b,Rd} = k_1 \cdot \frac{\alpha b \cdot f_{ub} \cdot d \cdot t_p}{\gamma_{M0}} \]
\[ 8 \cdot 25.6 \]
\[ 204.7 \text{ kN} \]
\[ \text{OK} \]
Left: 457x152 UB 67 [S275]
2 No. 90 x 90 x 10 L x 360 mm (2x5 kg)
With 5 No. 22 mm holes in webs
and 5 No. 22 mm holes in toes
For 15 No. 20 mm Ø Grade 8.8 Bolts.

Beam: 533x210 UB 82 [S275]
Beam to web gap 10 mm
Web to cleat gap 2 mm
Cleats grade: S 275

Beam to Beam Angle Cleats
Beam to Beam Angle Cleat Connection to EC 3 (UK NAD)

Basic Data

User Defined Applied Forces at Interface
Shear Forces
Left = 200.0 kN, Right = 0.0 kN

Tie Force
75.0 kN

Design to

SCI Green Book
P358: Joints in steel construction: Simple joints to Eurocode 3

Basic Dimensions
Left-457x152UB/B67 [28]
D=458.0, B=153.8, T=15.0, t=9.0, r=10.2, py=275
Supporting-533x210UB/B82 [28]
D=528.3, B=208.8, T=13.2, t=9.6, r=12.7, py=275

Bolts 20 mm Ø in 22 mm holes
Grade 8.8 Bolts

Plates S 275
All weld grades provided to suit minimum connected steel grade

Cleat Annotation
(Web) - Area of Cleats adjacent to supported beam web (like a fin-plate)
(Toe) - Area of Cleats Perpendicular to supported beam web (like an end-plate)

Summary of Results (Unity Ratios)

Left Hand Beam
Checks 8 & 9 Bearing & Shear (Toes) 752.6, 589.1, 752.6, 1076.0, 914.2 > = 200.0 kN 0.34 OK
Check 10 Shear (Toes) 617.3, 609.4 > = 100.0 kN 0.16 OK
Tie checks 0.58, 0.39, 0.58, 0.52 0.58 OK
Check 2: Bolt Shear & Bearing (web) 0.26, 0.25, 0.46 0.46 OK
Check 3: Sup.Beam Connect Elmts. (web) 0.22, 0.19, 0.25, 0.08 0.25 OK
Check 4: Shear & Bending (web) 0.29, 0.27, 0.21 0.29 OK
Left Hand Beam

Check 8a: Bolt Shear (Toes)
\[ F_{R,b}\text{shear} = \frac{\text{f}_{\text{A}}\cdot A}{\gamma_{M2}} \]
\[ V_{\text{R}} = 0.8\cdot n\cdot F_{\text{A}}\cdot \text{cols} \]
\[ 0.8 \cdot 5 \cdot 94.1 \cdot 2 \]
94.1 kN
752.6 kN
OK

Check 8b: End-Plate Bearing (Toes)
\[ P_1, c_1, c_2, c_3, c_0, k_1 \]
\[ F_{R,b}\text{shear} = \frac{\text{f}_{\text{A}}\cdot d x}{\gamma_{M2}} \]
\[ 1.5 \cdot 0.606 \cdot 410 \cdot 20 \cdot 10 / 1.25 \]
58.9 kN

Check 9a: End-Plate Plane Shear (Toes)
\[ F_{R,b}\text{shear} = \frac{0.80\cdot F_{R,\text{bolts}}}{n_1} \]
\[ 5 \cdot 58.9 \cdot 2 \]
589.1 kN
OK

Check 9b: End-Plate Block Shear (Toes)
\[ A_{\text{net}} = \text{A}_{\text{net}}(t, \beta, 1, 2, 3, 4, 5, 60) \]
\[ \text{V}_{\text{R}} = 2\cdot(F_{\text{A}}\cdot A_{\text{net}}(1/3\gamma_{M2}) + F_{\text{A}}\cdot A_{\text{net}}(3/\gamma_{M2})) \]
\[ 2\cdot(410 \cdot 285 / 1.1 + 2752210 / (\sqrt{3} \cdot 1.0)) \]
914.2 kN
OK

Check 10: Supporting Member - Local Resistance (Toes)
\[ A_{\text{net}} = \text{A}_{\text{net}}(t, \beta, 1, 2, 3, 4, 5, 60) \]
\[ \text{V}_{\text{R}} = 2\cdot(F_{\text{A}}\cdot A_{\text{net}}(1/3\gamma_{M2}) + F_{\text{A}}\cdot A_{\text{net}}(3/\gamma_{M2})) \]
\[ (10, 360, 40, 5, 22) \]
2210, 285 mm²

Tie force Checks

Basic Data

Tie Force

Tie forces are analysed independently of any vertical loads.

EC3 Design

Using conservative SCI P212 method to BS 5950

New Tie force

Magnified by 1.2 for lower EC3 load Factors

240.0 kN

Cleat toe L = 1 + e + 2 + (n-1) - p - nd

40 + 40 + (5 - 1) x 70 - 5 x 22.0
249.0 mm

Cleat Ten cap = 0.60\cdot L\cdot t\cdot p

0.6 x 249.0 x 0 x 275
410.9 kN
OK

Beam Web L = 1 + e + 2 + (n-1) - p - nd

40 + 40 + (5 - 1) x 70 - 5 x 22.0
250.0 mm

Web Ten cap = L\cdot t\cdot p

250.0 x 9 x 275
618.8 kN
OK

Web Bear cap = F_{\text{net}}(t, \beta, 1, 2, \text{ord}, \text{kgs})

9.0, 460.0, 40, 5, 0, 0, 0, 0

414.0 kN
OK

Prying Ratio Pr = (1 + 2 + k) / (2 + k)

(9 + 2 x 10) / (2 x 10)

1.46

Bolt Cap = \text{A}_{\text{net}}(1.25\cdot 2 + 1.5\cdot \text{Pr})\cdot \text{bolts}

785 x 245/(1.25 x 1.46) x 10
1056.0 kN
OK

Check 2: Supported beam - Bolt Group (web)

i) Bolts in Shear

\[ \beta = (0.25) / (n1\cdot n1\cdot 1)\cdot \text{p1} \]

\[ (6\cdot 50) / (5 \cdot 5 + 1) \cdot 70 \]

\[ 0.143 \]

\[ F_{\text{R,b}}\text{shear} = \frac{\text{f}_{\text{A}}\cdot A}{\gamma_{M2}} \]

\[ 0.6 \cdot 800 \cdot 245.0 / 1.25 \]

94.1 kN

ii) Fin plate bearing

\[ V_{\text{R}} = 2\cdot n\cdot F_{\text{A}}\cdot \sqrt{(1/\gamma_{M2}) + (1/\gamma_{M2})^2} \]

\[ (2\cdot 5 + 94.08) / \sqrt{((1+0.00005)^2 + (0.143 + 5)\cdot 1^2)} \]

765.6 kN
OK

iii) Beam Web bearing

\[ V_{\text{R}} = 2\cdot n\cdot F_{\text{A}}\cdot \sqrt{(1/\gamma_{M2}) + (1/\gamma_{M2})^2} \]

\[ + (1/\text{A}_{\text{net}}^{\text{Bolt}}) \]

\[ 0.143 \cdot (5.00 + 0.500) / (99.39) \]

808.8 kN
OK

### Check 3: Supported Beam - Connecting Elements (web)

#### i) Shear

\[ V_{Rd}\text{, }G = 2\left( h_p \times t_p / 1.27 \right) \times \left( f_y / \sqrt{3/\gamma_M} \right) \times (275/\sqrt{3/1.0}) \]

\[ V_{Rd}\text{, }N = 2\left( 0.5 \times f_y \times A_{nt} / \sqrt{3/\gamma_M} \right) + 2\left( F_{yp} \times A_{nv} / \sqrt{3/\gamma_M} \right) \]

\[ V_{Rd}\text{, }B = 2\times(0.5 \times f_y \times A_{nt} / \sqrt{3/\gamma_M}) \]

\[ V_{Rd} = 432.6 \text{kN} \quad \text{OK} \]

#### ii) Bending

\[ V_{Rd} = 900.1 \text{kN} \quad \text{OK} \]

#### iii) Lateral torsional buckling

\[ V_{Rd} = 809.9 \text{kN} \quad \text{OK} \]

### Check 4: Supported Beam - Resistance at the connection (web)

#### i) Shear

\[ V_{Rd,\text{wb}} = A_{vwb} \times f_y \times b_1 / \sqrt{3/\gamma_M} \]

\[ V_{Rd,\text{wb,net}} = A_{vwb,\text{net}} \times f_y \times b_1 / \sqrt{3/\gamma_M} \]

\[ V_{Rd,\text{wb}} = 695.7 \text{kN} \quad \text{OK} \]

#### ii) Shear & Bending Interaction of the Beam Web

<table>
<thead>
<tr>
<th>Moment Terms</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{c,Bc,Rd} )</td>
<td>32.3 kN.m</td>
</tr>
<tr>
<td>( M_{c,Bc,Ed} )</td>
<td>10.0 kN.m</td>
</tr>
<tr>
<td>( M_{c,Rd} )</td>
<td>48.34 kN.m</td>
</tr>
</tbody>
</table>

OK
Beam 1: 457x152 UB 67 [S275]
Plate: 10 x 150 x 455 mm dp. (5 kg)
With 12 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Beam 2: 457x152 UB 67 [S275]
Plate: 10 x 150 x 455 mm dp. (5 kg)
With 12 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.
Beam to Column Flexible End-Plate
Beam to Column Flexible End-plate Connection to EC 3 (UK NAD)

Basic Data

User Defined Applied Forces at Interface
Shear Forces  Flange 1 = 375.0 kN, Web 1 = 275.0 kN
Shear Forces  Flange 2 = 375.0 kN, Web 2 = 375.0 kN
Tie Forces     Flanges = 75.0 kN, Webs = 75.0 kN
SCI Green Book P358: Joints in steel construction: Simple joints to Eurocode 3

Basic Dimensions
Flange 1-457x152UB67 [28]  D=458.0, B=153.8, T=15.0, t=9.0, r=10.2, py=275
Flange 2-457x152UB67 [28]  D=458.0, B=153.8, T=15.0, t=9.0, r=10.2, py=275
Web 1-457x152UB67 [28]     D=458.0, B=153.8, T=15.0, t=9.0, r=10.2, py=275
Web 2-457x152UB67 [28]     D=458.0, B=153.8, T=15.0, t=9.0, r=10.2, py=275
Column -203x203UC46 [28]  D=203.2, B=203.6, T=11.0, t=7.2, r=10.2, py=275

Bolts 20 mm Ø in 22 mm holes
Grade 8.8 Bolts
Plates S 275
All weld grades provided to suit minimum connected steel grade

Summary of Results (Unity Ratios)

Checks for Flange Beams 1 & 2
Check 4 Supported Beam Web Shear 695.7 >= 375.0kN 0.54 OK
Checks 1 & 2 Detailing Practice 1.00, 0.83, 1.00, 0.64, 0.56, 0.710.86 1.00 OK
Check 8 Bearing 903.2, 903.2, 903.2 >= 375.0kN 0.42 OK
Check 10 Shear 864.5, 859.3 >= 375.0kN 0.42 OK
Check 11-14 Tie Forces 751.1, 1041.9, 1924.4 >= 375.0kN 0.62 OK

Checks for Web Beams 1 & 2
Check 4 Supported Beam Web Shear 695.7 >= 375.0kN 0.54 OK
Checks 1 & 2 Detailing Practice 1.00, 0.83, 1.00, 0.64, 0.56, 0.710.86 1.00 OK
Check 8 Bearing 903.2, 903.2, 903.2 >= 375.0kN 0.42 OK
Check 10 Shear 565.9, 562.4 >= 325.0kN 0.62 OK
Check 10 Combined Bearing 95.7 >= 54.2kN 0.57 OK
Check 14 Tie Forces Col Web 236.2 >= 100.0kN 0.42 OK
Check 11-13 Tie Forces 751.1, 1041.9, 1924.4, 1435.7, 602.3 >= 375.0kN 0.62 OK

Checks for Flange Beams 1 & 2 as Full Depth End-Plate

Check 1: Recommended Detailing Practice
Plate Depth >= 0.6•D  0.6 • 458 = 274.8 mm 455 mm OK
10 >= t_s <= 12 10.0 mm OK
90 <= p <= 140 90 mm OK
\[ t_p = \frac{d}{1.9\sqrt{\frac{f_{ub}}{f_{yy}}}} \] \[ 20/1.9 \times \sqrt{800 / 275} = 18 \text{ mm} \] OK

Check 4: Supported Beam Web Shear
\[ A_v = A_t - B_t \times B_t - t_t \times t_t = 8555 - 153.8 \times 15.0 - 153.8 \times 15.0 + (9.0+2\times10.2) \times 15.0 = 4382 \text{ mm}^2 \]
\[ V_{pl,Rd} = A_v \times f_{bw} / (\sqrt{3} \times \gamma\gamma_{M2}) = 4382 \times 275 / (\sqrt{3} \times 1.0) = 695.7 \text{ kN} \] OK

Check 2: Supported Beam Welds
\[ a_1 = a_1 \times V_{Ed} / V_{pl,Rd} = 4.9 \times 375.0 / 695.7 = 2.5 \text{ mm} \]
\[ S_{min} = \max(3, \min( a, a_1 )) / 0.7 = 4.3 \text{ mm} \] OK

Check 8a: Bolt Shear
\[ F_{pl,Rd} = 0.8 \times \frac{V_{Ed}}{n} \times f_{pl,Rd} = 0.8 \times 6 \times 94.1 = 94.1 \text{ kN} \]
\[ F_{pl,Rd} = 0.8 \times \frac{V_{Ed}}{n} \times f_{pl,Rd} = 0.8 \times 6 \times 94.1 = 903.2 \text{ kN} \] OK

Check 8b: End-Plate Bearing
\[ P_1 = 70.0, 40.0, 90.0, 30.0, 0.811, 2.118 \]
\[ F_{Ed,Rd} = F_{pl,Rd} = 112.6 \text{ kN} \]
\[ F_{Ed,Rd} = F_{pl,Rd} = 903.2 \text{ kN} \] OK
### Checks for Web Beams 1 & 2 as Full Depth End-Plate

#### Check 1: Recommended Detailing Practice

<table>
<thead>
<tr>
<th>Condition</th>
<th>Formula</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate Depth</td>
<td>&gt;= 0.6*D</td>
<td>455 mm OK</td>
</tr>
<tr>
<td>10 &gt;= t_p</td>
<td>&lt;= 12</td>
<td>10.0 mm OK</td>
</tr>
<tr>
<td>90 &lt;= t_p</td>
<td>&lt;= 140</td>
<td>90 mm OK</td>
</tr>
<tr>
<td>t_p</td>
<td>&lt;= 1.9*sqrt(f_u/f_y)</td>
<td>18 mm OK</td>
</tr>
</tbody>
</table>

#### Check 2: Supported Beam Web Shear

<table>
<thead>
<tr>
<th>Condition</th>
<th>Formula</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A_n = A - t_p<em>B_y + t_p</em>B_y + t_p*B_y</td>
<td>4382 mm²</td>
</tr>
<tr>
<td></td>
<td>V_{ph,min} = A / sqrt(3)*f_{ph,min}</td>
<td>695.7 kN OK</td>
</tr>
</tbody>
</table>

#### Check 2: Supported Beam Welds

<table>
<thead>
<tr>
<th>Condition</th>
<th>Formula</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>a_1 = a_1</td>
<td>a_1 = 4.3 mm OK</td>
<td></td>
</tr>
<tr>
<td>a_2 = a_2</td>
<td>a_2 = 2.5 mm OK</td>
<td></td>
</tr>
</tbody>
</table>

#### Check 3: Bolt Shear

<table>
<thead>
<tr>
<th>Condition</th>
<th>Formula</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>V_{bolt} = 0.8 * f_{m} * n * e_cols</td>
<td>903.2 kN OK</td>
</tr>
</tbody>
</table>

#### Check 8b: End-Plate Bearing

<table>
<thead>
<tr>
<th>Condition</th>
<th>Formula</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P_1, P_2, c_1, k_1</td>
<td>112.6 kN OK</td>
</tr>
</tbody>
</table>
### Check 8: Supporting Column Web Bearing

- **Check 8:** Supporting Column Web Bearing

  \[ \gamma_{col} = \min\left( \frac{p}{d_0} / 3.4, \frac{f_{yw}}{f_{yw, 1}} \right) \]

  \[ k_{col} = \min(2.5, 1.4 \cdot p / d_0, 1.7) \]

  \[ F_{vw, 1} = k \cdot \gamma_{col} \cdot d_0 / \gamma_{bw} \]

  \[ F_{vw, 2} = 0.8 \cdot F_{vw, 1} \]

  \[ F_{vw, 3} = 0.8 \cdot F_{vw, 1} / \gamma_{bw} \]

  \[ F_{bw} = 0.8 \cdot F_{vw, 3} \]

  \[ 0.8 \cdot F_{bw} / F_{bw, 2} \]

  \[ 0.8 \cdot 6 \cdot 94.1 \cdot 2 \]

  \[ 903.2 \text{ kN} \]

  **OK**

### Check 10: Supporting Member - Local Resistance

- **Check 10:** Supporting Member - Local Resistance

  \[ V_{ed} = \left( V_{ed, 1} + V_{ed, 2} \right) / 2 \]

  \[ 375 + 275 / 2 \] (per shear plane)

  \[ 325.0 \text{ kN} \]

- **Check 10:** Combined Load Bearing on supporting web

  \[ F_{bw} = V_{ed} / n + V_{ed, 2} / 2 \]

  \[ 375.0 / 12 + 275.0 / 12 \] (per bolt)

  \[ 54.2 \text{ kN} \]

  **OK**

### Tie Forces

- **Applied Tie Force**

  \[ 375.0 \text{ kN} \]

### Check 11: End-Plate in Bending

- **Check 11:** End-Plate in Bending

  \[ L_{eff} = L_{eff, 1} \cdot n + L_{eff, 2} / 2 \]

  \[ 129.4 + (6-1) \cdot 70 + 98.5 \]

  \[ 577.9 \text{ mm} \]

  \[ M_{pl} = 0.25 \cdot L_{eff} \cdot (\gamma_{bw} / \gamma_{bw, 1}) \]

  \[ 0.25 \cdot 577.9 \cdot 10^2 \cdot 410 / 1.1 \]

  \[ 5.39 \text{ kN.m} \]

- **Check 11:** Combined Load Bearing on supporting web

  \[ F_{bw} = \min(2 \cdot M_{pl, 1} + n \cdot S( k_2 \cdot f_{ubw} / \gamma_{bw})) / (m + n) \]

  \[ 12 \cdot 0.90 \cdot 800 \cdot 245 / 1.1 \]

  \[ 1924.4 \text{ kN} \]

  **OK**

### Check 12: Beam Web Tension

- **Check 12:** Beam Web Tension

  \[ F_{bw} = t_{bw} \cdot (D - T_{tbw}) + F_{ubw} / \gamma_{bw} \]

  \[ 9.0 \cdot (458.0 - 15.0 - 15.0) \cdot 410 / 1.1 \]

  \[ 1435.7 \text{ kN} \]

  **OK**

### Check 13: Welds

- **Check 13:** Welds

  \[ Web \]

  \[ F_{vw} = f_l / \sqrt[3]{3 / 0.85 / 1.1} \]

  \[ 410.0 \]

  \[ 253.2 \text{ N/mm}^2 \]

  **OK**

- **Flange**

  \[ F_{flange} = 2 \cdot F_{vw} \cdot \sqrt{n / (0.85 / 1.1)} \]

  \[ 2 \cdot 253.2 \cdot 4.2 \cdot (150.0 - 2.4) \]

  \[ 301.1 \text{ kN} \]

  **OK**

### Check 14: Supporting Web

- **Web Tie Force Applied**

  \[ 100.0 \text{ kN} \]

  \[ S \cdot M_{struct} = 8 \cdot (1.5 \cdot (1 - 0.56) - (1 - 0.14)) / (1 - 0.56) \]

  \[ 236.18 \text{ kN} \]

  **OK**
Beam 1: 430x100 PFC [S275]
Beam to Column Gap 10 mm
Plate: 10 x 100 x 360 mm dp. (3 kg)
With 5 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Beam 2: 250x25T+300x25B+400x12WPg145.62 [S275]
Beam to Column Gap 10 mm
Plate: 10 x 200 x 360 mm dp. (6 kg)
With 10 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Beam 1: 533x210 UB 82 [S275]
Beam to Column Gap 10 mm
Plate: 10 x 100 x 430 mm dp. (3 kg)
With 6 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Beam 2: 610x178 UB 82 [S275]
Beam to Column Gap 10 mm
Plate: 10 x 100 x 500 mm dp. (4 kg)
With 7 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.
Beam to Column Fin-Plate

Beam to Column Fin-plate Connection to EC 3 (UK NAD)

Basic Data

User Defined Applied Forces at Interface

Shear Forces

| Flange 1 | 200.0 kN |
| Flange 2 | 0.0 kN  |

Tie Forces

| Flanges | 75.0 kN |
| Webs    | 75.0 kN  |

Design to

| SCI Green Book: Joints in steel construction: Simple joints to Eurocode 3 |

Basic Dimensions

| Flange 1 | 430x100PFC [28] |
| D=430.0, B=100.0, T=19.0, r=11.0, py=265 |
| Flange 2 | 250x25+300x25+400x12WPg145.62 [28] |
| D=450.0, B=250.0, B=300.0, T=25.0, t=12.7, py=265 |
| Web 1   | 533x210UB82 [28] |
| D=528.3, B=208.8, T=13.2, t=9.6, r=11.0, py=275 |
| Web 2   | 610x178UB82 [28] |
| D=598.6, B=177.9, T=12.8, t=10.0, r=11.0, py=275 |
| Column  | 203x203UC46 [28] |
| D=203.2, B=203.6, T=11.0, t=7.2, r=11.0, py=275 |
| Bolts   | 20 mm Ø in 22 mm holes |
| Grade 8.8 Bolts |
| Plates  | S 275 |

All weld grades provided to suit minimum connected steel grade

Summary of Results (Unity Ratios)

Checks for Flange Beam on Side 1

Check 1: Detailing Practice
1.00 OK

Check 2: Bolt Shear & Bearing
0.52, 0.46, 0.38
1.00 OK

Check 3: Sup.Beam Connect Elmts.
0.44, 0.37, 0.47, 0.17
1.00 OK

Check 4: Shear & Bending
0.25, 0.23, 0.18
1.00 OK

Check 8: Welds
1.191.19
1.00 OK

Check 10: Supporting Member
0.16, 0.13
1.00 OK

Check 11: Structural integrity-Plate
0.23, 0.24, 0.24, 0.4, 0.3
1.00 OK

Check 12: Structural integrity-Beam
0.24, 0.23, 0.17, 0.3
1.00 OK

Checks for Flange Beam on Side 2

Checks for Web Beam on Side 1

Check 1: Recommended Detailing Practice
0.6 • 430 = 258 mm
10.0 mm
10.0 mm

Check 2: Supported beam - Bolt Group

i) Bolts in Shear

\[ \frac{\beta}{(n_1 A) / (n_1 A + \beta)} \times \frac{F}{t_{p1}} \times \frac{\gamma}{\gamma_{A1}} \]

\[ F_{s, Rd} = \gamma_{A1} \times \frac{F}{t_{p1}} \times \frac{\gamma}{\gamma_{A1}} = 0.6 \times 800 \times 245.0 / 1.25 \]

\[ V_{s, Rd} = \gamma_{A1} \times \frac{F}{t_{p1}} \times \frac{\gamma}{\gamma_{A1}} = 91.4 \times 94.08 / (4 \times 0.0005) = 382.8 \text{ kN} \]

ii) Fin plate bearing

\[ \frac{\gamma}{(1 + \beta)} \times \frac{F}{t_{p1}} \times \frac{\gamma}{\gamma_{A1}} \]

\[ F_{b, Rd} = k \times \gamma_{A1} \times \frac{F}{t_{p1}} \times \frac{\gamma}{\gamma_{A1}} = 2.5 \times 758 \times 410 \times 20 / 10.0 \times 1.25 \]

99.4 kN

99.4 kN

124.2 kN
Check 3: Supported Beam - Connecting Elements

i) Shear
\[ V_{Rd}=\frac{A_{s}\cdot f_{y}}{\sqrt{3}} \cdot \frac{1.25}{1.27} \cdot \frac{73}{1.0} \cdot \left( \frac{73}{3} \right) \]
\[ V_{Rd}=\frac{A_{s}\cdot f_{y}}{\sqrt{3}} \cdot \frac{1.25}{1.27} \cdot \frac{73}{1.0} \cdot \left( \frac{73}{3} \right) \]

Check 4: Supported Beam - Resistance at the connection

i) Shear
\[ V_{Rd}=\frac{A_{s}\cdot f_{y}}{\sqrt{3}} \cdot \frac{1.25}{1.27} \cdot \frac{73}{1.0} \cdot \left( \frac{73}{3} \right) \]
\[ V_{Rd}=\frac{A_{s}\cdot f_{y}}{\sqrt{3}} \cdot \frac{1.25}{1.27} \cdot \frac{73}{1.0} \cdot \left( \frac{73}{3} \right) \]

Check 8: Supporting Member Welds
\[ a=0.75 \times 0.5 \]

Check 10: Supporting Member Local Resistance

i) Local Shear
\[ F_{Rd}=2 \cdot h_{p} \cdot t_{p} \cdot f_{y} \cdot \frac{1}{\sqrt{3}} \]
\[ F_{Rd}=2 \cdot h_{p} \cdot t_{p} \cdot f_{y} \cdot \frac{1}{\sqrt{3}} \]

ii) Punching Shear
\[ t_{50}(u.2)h_{p}^{2}/(V_{Rd}/(f_{y} \cdot 3/3)) > 1 \]
\[ t_{50}(u.2)h_{p}^{2}/(V_{Rd}/(f_{y} \cdot 3/3)) > 1 \]

Tie Forces

Applied Tie Force
200.0 kN

Check 11: Structural integrity - Fin Plate

i) Tension
Part Block Tension
Only 1 Shear Plane
\[ P_{Rd}=f_{\mu}\cdot A_{u} \cdot f_{y}/g \]
\[ P_{Rd}=f_{\mu}\cdot A_{u} \cdot f_{y}/g \]

Full Block Tension
Only 2 Shear Planes
\[ P_{Rd}=f_{\mu}\cdot A_{u} \cdot f_{y}/g \]
\[ P_{Rd}=f_{\mu}\cdot A_{u} \cdot f_{y}/g \]

Full Depth Tension
No Shear Planes
\[ P_{Rd}=0.9\cdot f_{\mu}\cdot A_{u}\cdot f_{y}/g \]
\[ P_{Rd}=0.9\cdot f_{\mu}\cdot A_{u}\cdot f_{y}/g \]

ii) Shear
\[ F_{Rd}=G_{s}\cdot f_{y}/g \]
\[ F_{Rd}=G_{s}\cdot f_{y}/g \]

iii) Bearing
\[ P_{Rd}=C_{d}\cdot f_{y}/g \]
\[ P_{Rd}=C_{d}\cdot f_{y}/g \]

Check 12: Structural integrity - Supported Beam

i) Tension
### Checks for Flange Beam on Side 2

No Shear Load

### Checks for Web Beam on Side 1

#### Check 1: Recommended Detailing Practice

- **Plate Depth**: \(\geq 0.6+0.1D\)
  - \(t_w = 0.6 \cdot 528.3 = 317\) mm
  - \(t_w = 0.5+0.1\)
  - \(10.0\) mm
  - \(t_w = 0.5+0.1\)
  - \(10.0\) mm

#### Check 2: Supported beam - Bolt Group

**i) Bolts in Shear**

\[
\begin{align*}
\beta = f_s / (f_u (n + 1) p) \\
F_{b,Rd} &= f_{b,Rd} A / \gamma_M \\
V_{b,Rd} &= \sqrt{V^2 + (B+6)^2} \cdot f_{b,Rd} \\
\end{align*}
\]

#### ii) Fin plate bearing

- **Vert**: \(P_v, c_1, P_2, c_2, c_3, k_1\)
  - \(70.0, 40.0, 0.0, 0.0, 0.606, 2.500\)
  - \(F_{b,Rd} = f_{b,Rd} A / \gamma_M\)
  - \(99.4\) kN

- **Horz**: \(P_v, c_1, P_2, c_2, c_3, k_1\)
  - \(0.0, 50.0, 70.0, 0.0, 0.758, 2.500\)
  - \(V_{b,Rd} = N / \sqrt{((1+n)N / F_{b,Rd})^2 + (0.10 \cdot 6.00 / 124.24)^2}\)
  - \(535.6\) kN

#### iii) Beam Web bearing

- **Vert**: \(P_v, c_1, P_2, c_2, c_3, k_1\)
  - \(70.0, 60.0, 0.0, 0.0, 0.811, 2.500\)
  - \(F_{b,Rd} = f_{b,Rd} A / \gamma_M\)
  - \(127.6\) kN

- **Horz**: \(P_v, c_1, P_2, c_2, c_3, k_1\)
  - \(0.0, 40.0, 70.0, 68.0, 0.606, 2.500\)
  - \(V_{b,Rd} = N / \sqrt{((1+n)N / F_{b,Rd})^2 + (0.10 \cdot 6.00 / 95.42)^2}\)
  - \(592.4\) kN

#### Check 3: Supported Beam - Connecting Elements

**i) Shear**

\[
\begin{align*}
V_{b,Rd} &= (f_s / 1.27) / \sqrt{N / (3/5)} \\
V_{b,Rd} &= A_{v,Rd} / \sqrt{N / (3/5)} \\
V_{b,Rd} &= (0.5 \cdot f_u / A_{v,Rd}) / \sqrt{N / (3/5)} \\
\end{align*}
\]

**ii) Bending**

\[H_p = \geq 2.73x\]

**iii) Lateral torsional buckling**

\[W_{pl} = (\sqrt{f_{pl} / z}) / (275 / 1.0)\]

#### Check 4: Supported Beam - Resistance at the connection

**i) Shear**

\[
\begin{align*}
V_{b,Rd} &= A_{v,Rd} f_s / \sqrt{N / (3/5)} \\
V_{b,Rd} &= A_{v,Rd} f_s / \sqrt{N / (3/5)} \\
\end{align*}
\]

**iii) Shear & Bending Interaction of the Beam Web**

\[
\begin{align*}
M_{b,Rd} &= M_{b,Rd} f_u / \gamma_M \\
M_{b,Rd} &= M_{b,Rd} f_u / \gamma_M \\
M_{b,Rd} &= M_{b,Rd} f_u / \gamma_M \\
\end{align*}
\]
Check 8: Supporting Member Welds

\[ a = 0.7s > 0.5 t_p \]

\[ a = 0.7 \times 0.5 > 0.5 \times 10.0 \]

**Warning**

Check 10: Supporting Member Local Resistance

**i) Local Shear**

\[ F_{Rs,l} = 2t_b t_p \sigma_y / \sqrt{3} \gamma_{Ml} \]

\[ F_{Rs,l} = 2 \times 430 \times 7.2 \times 275 / (\sqrt{3} \times 1.0) \]

983.1 kN OK

**ii) Punching Shear**

\[ t_2 (u.2) t_p / (V_{Ed} \times 6 \times t_p \gamma_{Ml}) > 1 \]

\[ 7.2 \times 410 \times 430^2 / (200.0 \times 6 \times 50 \times 1.25) \]

7.278 > 1 OK

Tie Forces

**Applied Tie Force**

200.0 kN

Check 11: Structural integrity - Fin Plate

**i) Tension**

Part Block Tension

\[ F_{Rd,b} = f_u \cdot A / \gamma_{Mu} \]

\[ F_{Rd,b} = 410 \times 2690 / 1.1 \]

1064.6 kN OK

Full Block Tension

\[ F_{Rd,b} = f_u \cdot A / \sqrt{3} / \gamma_{Mu} \]

\[ F_{Rd,b} = 410 \times 2400 / 1.1 \]

1018.4 kN OK

Full Depth Tension

\[ F_{Rd,n} = 0.9 \cdot f_y \cdot A_{net,p} / \gamma_{Mu} \]

\[ F_{Rd,n} = 0.9 \times 410 \times 2980 / 1.1 \]

999.7 kN OK

**ii) Shear**

\[ F_{v,Rd} = \alpha_v \cdot f_u \cdot A / \gamma_{Mu} \]

\[ F_{v,Rd} = 0.6 \times 800 \times 245.0 / 1.1 \]

106.9 kN

\[ F_{v,Rd} = 6 \times 106.9 \]

641.5 kN OK

**iii) Bearing**

\[ F_{b,Rd} = k_1 \cdot \alpha_b \cdot f_u \cdot d \times t_p / \gamma_{Mu} \]

\[ F_{b,Rd} = 2.5 \times 0.758 \times 410 \times 20 / 1.1 \]

141.2 kN

\[ F_{b,Rd} = 6 \times 141.2 \]

847.1 kN OK

Check 12: Structural integrity - Supported Beam

**i) Tension**

Part Block Tension

\[ F_{Rd,b} = f_u \cdot A / \gamma_{Mu} \]

\[ F_{Rd,b} = 410 \times 2760 / 1.1 \]

1072.9 kN OK

Full Block Tension

\[ F_{Rd,b} = f_u \cdot A / \sqrt{3} / \gamma_{Mu} \]

\[ F_{Rd,b} = 410 \times 2304 / 1.1 \]

947.2 kN OK

Full Depth Tension

\[ F_{Rd,n} = 0.9 \cdot f_y \cdot A_{net,p} / \gamma_{Mu} \]

\[ F_{Rd,n} = 0.9 \times 410 \times 3804 / 1.1 \]

1276.2 kN OK

**ii) Bearing**

\[ F_{b,Rd} = k_1 \cdot \alpha_b \cdot f_u \cdot d \times t_p / \gamma_{Mu} \]

\[ F_{b,Rd} = 2.5 \times 0.606 \times 410 \times 20 / 1.1 \]

108.4 kN

\[ F_{b,Rd} = 6 \times 108.4 \]

650.6 kN OK

Checks for Web Beam on Side 2

No Shear Load
Beam 1: 457x191 UB 67 [S275]
2 No. 90 x 90 x 10 L x 360 mm (2x5 kg)
With 5 No. 22 mm holes in webs and 5 No. 22 mm holes in toes
For 15 No. 20 mm Ø Grade 4.8 Bolts.

Beam 2: 457x191 UB 67 [S275]
2 No. 75 x 75 x 8 L x 360 mm (2x3 kg)
With 5 No. 22 mm holes in webs and 5 No. 22 mm holes in toes
For 15 No. 20 mm Ø Grade 4.8 Bolts.

Beam to Column Angle Cleat Connection to EC 3 (UK NAD)

Basic Data

User Defined Applied Forces at Interface
Shear Forces Flange 1 = 200.0 kN, Flange 2 = 200.0 kN
Tie Forces 75.0 kN

Design to
- SCI Green Book P358: Joints in steel construction: Simple joints to Eurocode 3

Basic Dimensions
- Flange 1-457x191UB67 [28]
- Flange 2-457x191UB67 [28]
- Web 1-457x191UB67 [28]
- Web 2-457x191UB67 [28]
- Column -203x203UC46 [28]
- Bolts 20 mm Ø in 22 mm holes
- Plates S 275
- Cleat Annotation

All weld grades provided to suit minimum connected steel grade
(Web) - Area of Cleats adjacent to supported beam web (like a fin-plate)
(Toe) - Area of Cleats Perpendicular to supported beam web (like an end-plate)
Summary of Results (Unity Ratios)

Checks for Flange Beams 1 & 2
Checks 8 & 9 Bearing & Shear (Toes) 313.6, 313.6, 313.6, 1076.0, 912.4 >= 200.0kN 0.64 OK
Check 10 Shear (Toes) 707.3, 698.3 >= 100.0kN 0.14 OK
Tie checks 0.59, 0.41, 0.61, 0.52 0.61 OK
Check 2: Bolt Shear & Bearing (web) 0.63, 0.25, 0.49 0.63 OK
Check 3: Sup.Beam Connect Elmts. (web) 0.22, 0.19, 0.25, 0.08 0.25 OK
Check 4: Shear & Bending (web) 0.31, 0.29, 0.22 0.31 OK

Checks for Web Beams 1 & 2

Checks for Flange Beams 1 & 2

Check 8a: Bolt Shear (Toes)
F_{V+H}=0.8\times F_{V,\text{bullet}} \times \text{cols} 0.5 \times 400 \times 245.0 / 1.25 39.2 kN OK
V_{Rad}=0.8 \times 39.2 / 2 313.6 kN OK

Check 8b: End-Plate Bearing (Toes)
P_1, c_1, P_2, c_2, \gamma_1, k_1 70.0, 40.0, 50.0, 39.3, 0.606, 1.482
F_{V,Rad}=k_1 \times \gamma_1 \times \text{cols} / \gamma_2 1.5 \times 0.606 \times 410 \times 20.0 / 1.10 / 1.25 58.9 kN
0.80 \times F_{V,\text{Rad}} < F_{V,Rad} 0.80 \times 5 \times 39.2 / 2 313.6 kN OK

Check 8c: Supporting Column Flange Bearing (Toes)
V_{\text{col}}=\gamma_1 \times F_{V,\text{bullet}} / \gamma_2 707.3, 698.3 >= 100.0 kN OK
V_{\text{col}}=\gamma_1 \times F_{V,\text{bullet}} / \gamma_2 68.7 kN OK

Check 8d: Supporting Column Flange Bearing (Toes)

Check 9a: End-Plate Plane Shear (Toes)
V_{\text{Rad}}=2 \times h_{n-1} \times F_{V,\text{bullet}} / (\gamma_2 / \gamma_1) 1076.0 kN OK
V_{\text{Rad}}=2 \times h_{n-1} \times F_{V,\text{bullet}} / (\gamma_2 / \gamma_1) 400.1 kN OK

Check 9b: End-Plate Block Shear (Toes)
A_{n-1}, A_{n-1}=F_{n-1} (h_{n-1}, c_{n-1}, \gamma_1, \gamma_2, \gamma_3) (10, 360, 40, 39.5, 22) 2210, 283 mm²
V_{\text{Rad}}=2 \times F_{n-1} / (\gamma_2 / \gamma_1) 912.4 kN OK

Check 10: Supporting Member - Local Resistance (Toes)
A_n, A_{n-1}=F_{n-1} (h_{n-1}, c_{n-1}, \gamma_1, \gamma_2, \gamma_3) 4455, 3245 mm²
V_{\text{Rad}}=A_n \times F_{n-1} / (\gamma_2 / \gamma_1) 707.3 kN OK
V_{\text{Rad}}=A_n \times F_{n-1} / (\gamma_2 / \gamma_1) 698.3 kN OK

Tie force Checks

Basic Data
Tie force = 200.0 kN

Tie Force
Tie forces are analysed independently of any vertical loads.

EC3 Design
Using conservative SCI P212 method to BS 5950

New Tie force
Magnified by 1.2 for lower EC3 load Factors 240.0 kN

Cleat toe Lc=1+e2+(n-1)\times \text{pe} \times \text{nd}\times f 39 + 39 + (5-1) \times 70 - 5 \times 22.0 248.5 mm
Cleat Ten cap \times h_{n-1} \times F_{V,\text{bullet}} 0.60 x 248.5 x 10 x 275 410.0 kN OK
Beam Web Lb=1+e2+(n-1)\times \text{pe} \times \text{nd}\times f 40 + 40 + (5-1) \times 70 - 5 \times 22.0 250.0 mm
Web Ten cap \times h_{n-1} \times F_{V,\text{bullet}} 250.0 x 9 x 275 584.4 kN OK
Web Bear cap = F_{n-1} (h_{n-1}, c_{n-1}, \gamma_1, \gamma_2, \gamma_3) 8.5, 460, 40, 5, 20, 1.00 391.0 kN OK
Cleat Bear cap = F_{n-1} (h_{n-1}, c_{n-1}, \gamma_1, \gamma_2, \gamma_3) 10.0, 460, 40, 5, 20, 1.00 460.0 kN OK
Prying Ratio Pr=(1+2 \times k) / (2 \times k) (9 + 2 x 10) / (2 x 10) 1.46
Bolt Cap\times U\times A e\times (1.25\times P_r) 393 x 2451/1.25 x 1.46) x 10 526.0 kN OK

Check 2: Supported beam - Bolt Group (web)

i) Bolts in Shear
\beta=\left(6^{\gamma_1}\right) / (n_{n-1}+1)^{\gamma_1} (6 \times 50) / (5 \times (5 + 1) \times 70) 0.143
F_{V,\text{bullet}}=F_{V,\text{bullet}} / \gamma_2 0.5 \times 400 \times 245.0 / 1.25 39.2 kN OK
V_{\text{bullet}}=\left(6^{\gamma_1}\right) / (n_{n-1}+1)^{\gamma_1} (2 \times 5 \times 39.20) / (\gamma_1(x+0.0005)^2 + (0.143\times 5)^2) 319.0 kN OK

ii) Fin plate bearing

iii) Beam Web bearing  
Vert P_1, c_1, P_2, c_2; \gamma_n, k_1  
70.0, 65.0, 0.0, 8.11, 2.500  
F_{b,Rd} = k_1 \cdot \alpha_b \cdot f_{ub} \cdot d \cdot t_p / \gamma_M  
= 2.5 \cdot 8.11 \cdot 410 \cdot 8.5 / 1.1  
= 113.0 kN  

Horz P_1, c_1, P_2, c_2; \gamma_n, k_1  
0.0, 40.0, 70.0, 65.0, 0.606, 2.500  
F_{b,Rd} = k_1 \cdot \alpha_b \cdot f_{ub} \cdot d \cdot t_p / \gamma_M  
= 2.5 \cdot 6.5 \cdot 410 \cdot 8.5 / 1.1  
= 84.5 kN  

V_{Rd} = N / \sqrt{((1 + a \cdot N) / F_{b,Rd})^2 + (0.14 \cdot N / F_{b,Rd})^2}  
= 2 \cdot 5.00 / \sqrt{((1 + 0.00 \cdot 5.00) / 113.00)^2 + (0.14 \cdot 5.00) / 84.48)^2}  
= 408.5 kN  

OK

Check 3: Supported Beam - Connecting Elements (web)  
i) Shear  
V_{Rd} = 2 \cdot (b_p \cdot f_{yp} / 1.27) \cdot (f_{yp} / \sqrt{3/\gamma_M})  
= 2 \cdot (360 \cdot 10 / 1.27) \cdot (275 / \sqrt{3/1.0})  
= 900.1 kN  

OK

V_{Rd,n} = 2 \cdot A_{v,Net} \cdot f_{up} / \sqrt{3 / \gamma_M}  
= 2 \cdot 2500 \cdot 410 / \sqrt{3/1.1}  
= 1076.0 kN  

OK

ii) Bending  
V_{Rd,b} = 2 \cdot (0.5 \cdot f_{up} \cdot A_{nt} / \gamma_M)  
+ 2 \cdot (f_{yp} \cdot A_{nv} / \sqrt{3/\gamma_M})  
+ 2 \cdot (275 \cdot 2210 / \sqrt{3/1.0})  
= 809.9 kN  

OK

iii) Lateral torsional buckling  
V_{Rd} = 2 \cdot (W_{el,P} / z) \cdot (f_{yp} / \gamma_M)  
= 2 \cdot (216000 / 50) \cdot (275 / 1.0)  
= 2376.0 kN  

OK

Check 4: Supported Beam - Resistance at the connection (web)  
i) Shear  
V_{Rd} = A_{v,wb} \cdot f_{y,b1} / \sqrt{3 / \gamma_M}  
= 4093.57 \cdot 275.00 / \sqrt{3/1.0}  
= 649.9 kN  

OK

V_{Rd,n} = A_{v,wb,net} \cdot f_{u,b1} / \sqrt{3 / \gamma_M}  
= 3158.57 \cdot 410.00 / \sqrt{3/1.1}  
= 679.7 kN  

OK

iii) Shear & Bending Interaction of the Beam Web  
M_{c,BE} = f_{y,p} \cdot t_w / 6 / \gamma_M  
= (275.00 \cdot 8.50 / 6 / 1.0) \cdot (5-1) \cdot 70  
= 30.5 kN.m  

OK

M_{c,ED} = Z_p  
= 200.00 \cdot 50.00  
= 10.00 kN.m  

OK

M_{c,RD} = M_{c,BE} + V_{AB,Rd} \cdot (n_1-1) \cdot p_1  
= 30.54 + 53.98 \cdot (5-1) \cdot 70  
= 45.66 kN.m  

OK

Checks for Web Beams 1 & 2

No Shear Load
1 Eaves Joint
Beam to Column Flange End-Plated Connection to EC 3 (UK NAD)
Loading Case 001

Basic Data

User Defined Applied Forces at Column/Right Rafter Interface
Right Rafter Forces M, Fvr, Fr
Resultant Forces M, Fv, F
Load directions
Design to
Weld Grades

Basic Dimensions
Column-610x178UB92 [28]
Rafter-457x191UB67 [28]
Haunch-457x191UB67 [28]
Bolts 20 mm Ø in 22 mm holes
Plates S 275
Rafter Capacities Mc, Fvc, Fc

Summary of Results (Unity Ratios)
Moment Capacity 473.6 kNm (for 3 rows of bolts) (Modified Applied Mom. Mmod=410.1 kNm)
Shear Capacity
Flange Welds
Web Welds
Haunch Welds
Column Compression stiff Web Weld
End of Haunch Compression Zone

End-Plate 940 x 210 x 20 mm (31 kg)
8 No. M20 Grade 8.8 Bolts in 22 mm holes
Haunch Stiff 8 mm with 6 FW

Plates S 275
Beam 457x191 UB 67 [S275]
Haunch 457x191 UB 67 [S275]
Column 610x178 UB 92 [S275]
Top 20 above top flange
Bot 400 below bottom flange

2500
880
90

8 fw
6 fw
6 fw
6 fw
6 fw
7.5 deg

12 fw
12 tk 8 fw

100
580
900
20
20

603.0, B=178.8, T=15.0, t=10.9, r=12.7, py=275
453.4, B=189.9, T=12.7, t=8.5, r=10.2, py=275
453.4, B=189.9, T=12.7, t=8.5, r=10.2, py=275
Grade 8.8 Bolts
All weld grades provided to suit minimum connected steel grade

20 mm Ø in 22 mm holes

1252.6 kN, 3395.2 kN
Mc = 1046.2 kN.m

Mc = 1046.2 kN.m

0.87
0.90
0.68
0.97
0.22
0.10
OK
OK
OK
OK
OK
OK
OK
OK
OK
OK

Apex Joint
Beam to Beam End-Plated Connection to EC 3 (UK NAD)
Loading Case 001

Basic Data

User Defined Applied Forces at End-plate Interface
Right Rafter Forces M, Fvr, Fr  -150.0 kNm, 400.0 kN, 300.0 kN
Resultant Forces M, Fv, F      -150.0 kNm, 328.9 kN, 376.6 kN
Load directions               Bottom of Joint in Tension, Rafter moving Down and in Compression.
Weld Grades                   All weld grades provided to suit minimum connected steel grade

Basic Dimensions
Rafter-457x191 UB 67 [28]     D=453.4, B=189.9, T=12.7, t=8.5, r=10.2, py=275
Haunch-457x191 UB 67 [28]     D=453.4, B=189.9, T=12.7, t=8.5, r=10.2, py=275
Bolts 20 mm Ø in 22 mm holes  Grade 8.8 Bolts
Plates S 275                   All weld grades provided to suit minimum connected steel grade
Rafter Capacities Mc, Fvc, Fc  1046.2 kN.m, 1252.6 kN, 3395.2 kN
                                      Fvc = 1252.6 kN               OK

Summary of Results (Unity Ratios)

- Moment Capacity 524.4 kNm (for 4 rows of bolts) (Modified Applied Mom. M_{mod}=66.6 kNm) 0.13 OK
- Moment Capacity 203.7 kNm (for the 1 rows of bolts required in the tension zone) 0.33 OK
- Shear Capacity 0.41 OK
- Flange Welds 0.15 OK
- Web Welds 0.49, 0.24 0.49 OK
- Haunch Welds 0.58, 0.23 0.58 OK
**User Defined Applied Forces at End-plate Interface**

Resultant Forces M, Fv, F
- 235.0 kNm, 400.0 kN, 300.0 kN

Load directions
- Bottom of Joint in Tension, Rafter moving Down and in Compression.

Design to

Weld Grades
- All weld grades provided to suit minimum connected steel grade

**Basic Dimensions**

Beam - 457x191 UB 67 [S275]
- D=453.4, B=189.9, T=12.7, t=8.5, r=10.2, py=275

Bolts 20 mm Ø in 22 mm holes
- Grade 8.8 Bolts

Plates S 275
- All weld grades provided to suit minimum connected steel grade

Rafter Capacities Mc, Fvc, Fc
- 404.5 kN.m, 649.9 kN, 2351.3 kN

Fvc = 649.9 kN

**Summary of Results (Unity Ratios)**

Moment Capacity 191.5 kNm (for 4 rows of bolts) (Modified Applied Mom. Mmod=168.9 kNm)
- 0.88 OK

Moment Capacity 191.5 kNm (for the 3 rows of bolts required in the tension zone)
- 0.88 OK

Shear Capacity
- 0.75 OK

Flange Welds
- 0.46 OK

Web Welds
- 0.77, 0.77, 0.68 OK

---

**Plates S 275**

**Beam 457x191 UB 67 [S275]**

**End-Plate 493 x 210 x 15 mm (12 kg)**

**10 No. M20 Grade 8.8 Bolts in 22 mm holes**

**End-Plated Splice**

**Beam to Beam End-Plated Connection to EC 3 (UK NAD)**

**Loading Case 001**

**Basic Data**
Base-Plate 600 x 390 x 30 mm (55 Washers 100 x 100 x 15 mm thick with 50 mm Void With 7 No. 24 mm holes For 20 mm Grade 8.8 Bolts.

All Plates S 275
Reinforcement 12 @ 200 mm cc each way. Cover 0 mm

Asymmetrical Base-Plate
Base-Plate Connection to EC 3 (UK NAD)
Loading Case 001

Basic Data

Resultant Forces M, Fv, F
Moment +125.0 kNm, Shear +500.0 kN, Axial +1300.0 kN
(Left side in tension, Axial Compression)

Basic Dimensions
Column: 406x178 UB 54 [S275]
D=402.6, B=177.7, T=10.9, t=7.7, r=10.2, py=275
Bolts 20 mm Ø in 24 mm holes Grade 8.8 Bolts
All weld grades provided to suit minimum connected steel grade
Grout Fck, Conc Fck, fy, slope 20 N/mm², 32 N/mm², 0.35 N/mm², 265 N/mm², 30 deg to vertical
Column Capacities Mc, Fvc, Fc 290.0 kN.m, 528.3 kN, 1896.1 kN
Fvc = 528.3 kN OK

Summary of Results (Unity Ratios)

<table>
<thead>
<tr>
<th>Description</th>
<th>Ratio</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Pressure</td>
<td>1.00</td>
<td>OK</td>
</tr>
<tr>
<td>Base-Plate thickness in Compression</td>
<td>0.96</td>
<td>OK</td>
</tr>
<tr>
<td>Horizontal Shear</td>
<td>0.86</td>
<td>OK</td>
</tr>
<tr>
<td>Flange &amp; Web Welds</td>
<td>0.72</td>
<td>OK</td>
</tr>
</tbody>
</table>
Beam-Splice Outer Plates
Non Bearing - Beam to Beam Moment Splice Connection to EC 3 (UK NAD)
Loading Case 001

Basic Data
User Defined Applied Forces at Interface
Resultant Forces M, Fv, F

+200.0 kNm, +40.0 kN, +0.0 kN
(Bottom in tension)

Beam Gap= 5 mm
Therefore No direct bearing.
HSFG Bolts
Non-slip at service
Ultimate/Service factor for service force calculation. 1.2
Design to

Basic Dimensions
Beam-533x210UB101 [28]
D=536.7, B=210.0, T=17.4, t=10.8, r=12.7, py=265
Bolts 20 mm Ø in 22 mm holes
All weld grades provided to suit minimum connected steel grade
Plates S 275
Beam Capacities Mc, Fvc, Fc
692.2 kN.m, 946.7 kN, 3409.5 kN
Mc = 692.2 kN.m OK

Summary of Results (Unity Ratios)
Top Flange in Compression Bolt Capacity 0.45 OK
Top Flange in Compression Axial Capacity 0.32 OK
Bottom Flange in Tension Bolt Capacity 0.60 OK
Bottom Flange in Tension Axial Capacity 0.32 OK
Web Bolt Capacity 0.41, 0.34
Web Plate Shear Capacity 0.00, 0.00, 0.00
Web Plate Moment Capacity 0.35 OK
Web Axial Capacity 0.00 OK
Beam-Splice end-plated splice
Bearing - Beam to Beam Moment Splice Connection to EC 3 (UK NAD)

Loading Case 001

User Defined Applied Forces at Interface
Resultant Forces M, Fv, F
+255.0 kNm, +0.0 kN, +0.0 kN
(Bottom in tension)

Web End plates
Direct Bearing assumed
All Moment resisted by Flanges
All Shear loads resisted by Web bolts and Friction

HSFG Bolts
Non-slip at service
Ultimate/Service factor for service force calculation. 1.2

Design to

Basic Dimensions
Beam-533x210UB101 [28] D=536.7, B=210.0, T=17.4, t=10.8, r=12.7, py=265
Bolts 20 mm Ø in 22 mm holes HSFG - Pt 2 Bolts, Non-slip at Service
Plates S 275 All weld grades provided to suit minimum connected steel grade
Beam Capacities Mc, Fvc, Fc
692.2 kN.m, 946.7 kN, 3409.5 kN

Summary of Results (Unity Ratios)
Top Flange in Compression Axial Capacity 0.49 OK
Bottom Flange in Tension Bolt Capacity 0.94 OK
Bottom Flange in Tension Axial Capacity 0.49 OK
Column Splice with division Plate
Bearing - Column Splice Connection to EC 3 (UK NAD)

Basic Data

User Defined Applied Forces at Interface

<table>
<thead>
<tr>
<th>Force</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>FC</td>
<td>Maximum Factored Axial Load</td>
<td>150.0 kN</td>
</tr>
<tr>
<td>FCD</td>
<td>Minimum Factored Axial Load</td>
<td>140.0 kN</td>
</tr>
<tr>
<td>M</td>
<td>Factored Moment</td>
<td>25.0 kN.m</td>
</tr>
<tr>
<td>Fv</td>
<td>Factored Shear Force</td>
<td>0.0 kN</td>
</tr>
</tbody>
</table>

Web Division Plate
Direct Bearing assumed
All Moment and Axial loads resisted by Flanges
All Shear loads resisted by Web bolts and Friction

HSFG Bolts
Non-slip at service
Ultimate/Service factor for service force calculation. 1.2

Design to

Basic Dimensions

Upper Column - 203x203 UC 46 [S275]
D=203.2, B=203.6, T=11.0, t=7.2, r=10.2, py=275, Fc=1615 kN

Lower Column - 254x254 UC 73 [S275]
D=254.1, B=254.6, T=14.2, t=8.6, r=12.7, py=275, Fc=2560 kN

Bolts 20 mm Ø in 22 mm holes
HSFG - Pt 2 Bolts, Non-slip at Service
All weld grades provided to suit minimum connected steel grade

Summary of Results (Unity Ratios)

<table>
<thead>
<tr>
<th>Category</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate &amp; Flange Capacity in Tension</td>
<td>0.12 OK</td>
</tr>
<tr>
<td>Flange Bolts Shear Capacity Bolt Capacity</td>
<td>0.16 OK</td>
</tr>
<tr>
<td>Flange Bolts Shear Capacity Packer Weld</td>
<td>0.05 OK</td>
</tr>
</tbody>
</table>
CHS Hollow Section Splice
Hollow Section Splice Connection to EC 3 (UK NAD)
Loading Case 001

Basic Data

User Defined Applied Forces at Interface

Applied Forces
F = -300.0 kN (T), Mxx = +30.0 kNm,

Design to

Basic Dimensions

Beam-660x20CHS [28]
Ø=660.0, T=20.0, py=265

Bolts 20 mm Ø in 22 mm holes
Grade 8.8 Bolts

Plates S 275
All weld grades provided to suit minimum connected steel grade

Summary of Results (Unity Ratios)

End-Plate & Bolts 0.35, 0.39, 0.25 0.39 OK
Welds 0.19 OK
Section Capacity 0.04, 0.01, 0.04 0.04 OK
Beam to Beam Flexible End-Plate - with Asymetrical Plate Girders
Beam to Beam Flexible End-plate Connection to EC 3 (UK NAD)

Basic Data

User Defined Applied Forces at Interface
Shear Forces
Left = 235.0 kN, Right = 170.0 kN
Tie Force
75.0 kN
Design to

Basic Dimensions
Left-457x191 UB 82 [S275]  
Supporting-250x25T+160x20B+405x15WPg121.87 [S275]  
Right-356x171 UB 57 [S275]

Bolts 20 mm Ø in 22 mm holes
Grade 8.8 Bolts
All weld grades provided to suit minimum connected steel grade

Summary of Results (Unity Ratios)

Left Hand Beam
Check 4 Supported Beam Web Shear
410.2 >= 235.0kN  0.57  OK
Check 1 & 2 Detailing Practice
1.00, 0.83, 1.00, 0.64, 0.710.94  1.00  OK
Check 5: Notch Resistance
0.50  0.50  OK
Check 6: Notch Stability
0.54  0.54  OK
Check 8 & 9 Bearing & Shear
451.6, 451.6, 451.6, 964.1, 760.8 >= 235.0kN  0.52  OK
Check 10 Bearing
596.7, 626.2 >= 181.3kN  0.30  OK
Check 10 Combined Bearing
199.4 >= 60.4kN  0.30  OK
Check 11-14 Tie Forces
296.9, 506.3, 962.2, 1070.1 >= 235.0kN  0.79  OK
### Right Hand Beam

<table>
<thead>
<tr>
<th>Check Description</th>
<th>Value</th>
<th>Factor</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Check 4 Supported Beam Web Shear</td>
<td>335.7 &gt;= 170.0kN</td>
<td>0.51</td>
<td>OK</td>
</tr>
<tr>
<td>Checks 1 &amp; 2 Detailing Practice</td>
<td>1.00, 0.83, 1.00, 0.64, 0.710.77</td>
<td>1.00</td>
<td>OK</td>
</tr>
<tr>
<td>Check 5: Notch Resistance</td>
<td>0.39</td>
<td>0.39</td>
<td>OK</td>
</tr>
<tr>
<td>Check 6: Notch Stability</td>
<td>0.35</td>
<td>0.35</td>
<td>OK</td>
</tr>
<tr>
<td>Checks 8 &amp; 9 Bearing &amp; Shear</td>
<td>602.1, 602.1, 602.1, 869.4, 691.0 &gt;= 170.0kN</td>
<td>0.28</td>
<td>OK</td>
</tr>
<tr>
<td>Check 11-14 Tie Forces</td>
<td>371.5, 663.0, 1282.9, 875.5 &gt;= 170.0kN</td>
<td>0.46</td>
<td>OK</td>
</tr>
</tbody>
</table>
Beam to Beam Fin-Plate

Beam to Beam Fin-plate Connection to EC 3 (UK NAD)

Basic Data

User Defined Applied Forces at Interface
Shear Forces
Left = 250.0 kN, Right = 120.0 kN
Tie Force
75.0 kN
Design to

Basic Dimensions
Left-457x152 UB 67 [S275]
Supporting-533x210 UB 82 [S275]
Right-457x191 UB 98 [S275]

Bolts 20 mm Ø in 22 mm holes
Plates S 275

All weld grades provided to suit minimum connected steel grade

Summary of Results (Unity Ratios)

Left Hand Beam
Check 1: Detailing Practice
1.00
1.00
OK
Check 2: Bolt Shear & Bearing
0.65, 0.58, 0.69
0.69
OK
Check 3: Sup. Beam Connect Elmts.
0.56, 0.46, 0.59, 0.21
0.59
OK
Check 4: Supported Beam Resist.
0.40, 0.40, 0.69
0.69
OK
Check 5: Notch Resistance
0.40
0.40
OK
Check 6: Notch Stability
0.22
0.22
OK
Check 8: Welds
0.890.89
0.89
OK
Check 10: Supporting Member
0.34
0.34
OK
Check 11: Structural integrity-Plate
0.28, 0.30, 0.30, 0.5, 0.4
0.50
OK
Check 12: Structural integrity-Beam
0.32, 0.34, 0.26, 0.5
0.50
OK
## Right Hand Beam

<table>
<thead>
<tr>
<th>Check</th>
<th>Description</th>
<th>Value</th>
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</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Detailing Practice</td>
<td>1.00</td>
<td>OK</td>
</tr>
<tr>
<td>2</td>
<td>Bolt Shear &amp; Bearing</td>
<td>0.92</td>
<td>OK</td>
</tr>
<tr>
<td>3</td>
<td>Sup. Beam Connect Elmts.</td>
<td>0.36</td>
<td>OK</td>
</tr>
<tr>
<td>4</td>
<td>Supported Beam Resist.</td>
<td>0.32</td>
<td>OK</td>
</tr>
<tr>
<td>5</td>
<td>Notch Resistance</td>
<td>0.16</td>
<td>OK</td>
</tr>
<tr>
<td>6</td>
<td>Notch Stability</td>
<td>0.22</td>
<td>OK</td>
</tr>
<tr>
<td>7</td>
<td>Welds</td>
<td>0.89</td>
<td>OK</td>
</tr>
<tr>
<td>11</td>
<td>Structural integrity - Plate</td>
<td>0.70</td>
<td>OK</td>
</tr>
<tr>
<td>12</td>
<td>Structural integrity - Beam</td>
<td>0.60</td>
<td>OK</td>
</tr>
</tbody>
</table>
Beam to Beam Angle Cleats

Beam to Beam Angle Cleat Connection to EC 3 (UK NAD)

**Basic Data**

**User Defined Applied Forces at Interface**

- Shear Forces: Left = 200.0 kN, Right = 0.0 kN
- Tie Force: 75.0 kN

**Basic Dimensions**

- Left: 457x152 UB 67 [S275]
  2 No. 90 x 90 x 10 L x 360 mm (2x5 kg)
  With 5 No. 22 mm holes in webs
  and 5 No. 22 mm holes in toes
  For 15 No. 20 mm Ø Grade 8.8 Bolts.

- Supporting: 533x210 UB 82 [S275]
  Beam to web gap 10 mm
  Web to cleat gap 2 mm
  Cleats grade: S 275

**Summary of Results (Unity Ratios)**

- Check 1: Bearing & Shear (Toes)
  752.6, 0.34, OK

- Check 2: Bolt Shear & Bearing (web)
  0.26, 0.46, OK

- Check 3: Sup Beam Connect Elmts. (web)
  0.22, 0.25, 0.08, OK

- Check 4: Shear & Bending (web)
  0.29, 0.27, 0.21, OK
Beam 1: 457x152 UB 67 [S275]  
Plate: 10 x 150 x 455 mm dp. (5 kg)  
With 12 No. 22 mm holes  
For 20 mm Ø Grade 8.8 Bolts.

Beam 2: 457x152 UB 67 [S275]  
Plate: 10 x 150 x 455 mm dp. (5 kg)  
With 12 No. 22 mm holes  
For 20 mm Ø Grade 8.8 Bolts.

Beam to Column Flexible End-Plate  
Beam to Column Flexible End-plate Connection to EC 3 (UK NAD)  
Basic Data

User Defined Applied Forces at Interface

Shear Forces
Flange 1 = 375.0 kN, Web 1 = 275.0 kN
Flange 2 = 375.0 kN, Web 2 = 375.0 kN

Tie Forces
Flanges = 75.0 kN, Webs = 75.0 kN

Design to

Basic Dimensions
Flange 1-457x152UB67 [28]  
D=458.0, B=153.8, T=15.0, t=9.0, r=10.2, py=275
Flange 2-457x152UB67 [28]  
D=458.0, B=153.8, T=15.0, t=9.0, r=10.2, py=275
Web 1-457x152UB67 [28]  
D=458.0, B=153.8, T=15.0, t=9.0, r=10.2, py=275
Web 2-457x152UB67 [28]  
D=458.0, B=153.8, T=15.0, t=9.0, r=10.2, py=275
Column -203x203UC46 [28]  
D=203.2, B=203.6, T=11.0, t=7.2, r=10.2, py=275

Bolts 20 mm Ø in 22 mm holes  
Grade 8.8 Bolts
Plates S 275  
All weld grades provided to suit minimum connected steel grade
### Summary of Results (Unity Ratios)

#### Checks for Flange Beams 1 & 2

- **Check 4 Supported Beam Web Shear**: $695.7 \geq 375.0\text{kN}$  
  - $0.54$  
  - **OK**

- **Checks 1 & 2 Detailing Practice**: $1.00, 0.83, 1.00, 0.64, 0.56, 0.710.86$  
  - $1.00$  
  - **OK**

- **Check 8 Bearing**: $903.2, 903.2, 903.2 \geq 375.0\text{kN}$  
  - $0.42$  
  - **OK**

- **Check 10 Shear**: $864.5, 859.3 \geq 187.5\text{kN}$  
  - $0.22$  
  - **OK**

- **Check 11-14 Tie Forces**: $751.1, 1041.9, 1924.4, 919.8, 1236.1, 1435.7, 602.3 \geq 375.0\text{kN}$  
  - $0.62$  
  - **OK**

#### Checks for Web Beams 1 & 2

- **Check 4 Supported Beam Web Shear**: $695.7 \geq 375.0\text{kN}$  
  - $0.54$  
  - **OK**

- **Checks 1 & 2 Detailing Practice**: $1.00, 0.83, 1.00, 0.64, 0.56, 0.710.86$  
  - $1.00$  
  - **OK**

- **Check 8 Bearing**: $903.2, 903.2, 903.2 \geq 375.0\text{kN}$  
  - $0.42$  
  - **OK**

- **Check 10 Shear**: $565.9, 562.4 \geq 325.0\text{kN}$  
  - $0.58$  
  - **OK**

- **Check 10 Combined Bearing**: $95.7 \geq 54.2\text{kN}$  
  - $0.57$  
  - **OK**

- **Check 14 Tie Forces Col Web**: $236.2 \geq 100.0\text{kN}$  
  - $0.42$  
  - **OK**

- **Check 11-13 Tie Forces**: $751.1, 1041.9, 1924.4, 1435.7, 602.3 \geq 375.0\text{kN}$  
  - $0.62$  
  - **OK**
Beam 1: 430x100 PFC [S275]
Beam to Column Gap 10 mm
Plate: 10 x 100 x 360 mm dp.
With 5 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Beam 2: 250x25T+300x25B+400x12WPg145.62 [S275]
Beam to Column Gap 10 mm
Plate: 10 x 200 x 360 mm dp.
With 10 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Beam 1: 533x210 UB 82 [S275]
Beam to Column Gap 10 mm
Plate: 10 x 100 x 430 mm dp.
With 6 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Beam 2: 610x178 UB 82 [S275]
Beam to Column Gap 10 mm
Plate: 10 x 100 x 500 mm dp.
With 7 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.
Beam to Column Fin-Plate
Beam to Column Fin-plate Connection to EC 3 (UK NAD)

Basic Data

User Defined Applied Forces at Interface
- Shear Forces: Flange 1 = 200.0 kN, Web 1 = 200.0 kN, Flange 2 = 0.0 kN, Web 2 = 0.0 kN
- Tie Forces: Flanges = 75.0 kN, Webs = 75.0 kN

Basic Dimensions
- Flange 1-430x100PFC [28]: D=430.0, B=100.0, T=19.0, t=11.0, r=15.0, py=265
- Flange 2-250x25T+300x25B+400x12WPg145.62 [28]: D=450.0, B=250.0, B=300.0, T=25.0, t=12.0, r=12.7, py=265
- Web 1-533x210UB82 [28]: D=528.5, B=208.8, T=13.2, t=9.6, r=12.6, py=275
- Web 2-610x178UB82 [28]: D=598.6, B=177.9, T=12.8, t=10.0, r=12.7, py=275
- Column -203x203UC46 [28]: D=203.2, B=203.6, T=11.0, t=7.2, r=10.2, py=275
- Bolts 20 mm Ø in 22 mm holes: Grade 8.8 Bolts
- Plates S 275: All weld grades provided to suit minimum connected steel grade

Summary of Results (Unity Ratios)

**Checks for Flange Beam on Side 1**
- Check 1: Detailing Practice 1.00
- Check 2: Bolt Shear & Bearing 0.32, 0.46, 0.38
- Check 3: Sup.Beam Connect Elmts. 0.44, 0.37, 0.47, 0.17
- Check 4: Shear & Bending 0.25, 0.23, 0.18
- Check 8: Welds 1.191.19
- Check 10: Supporting Member 0.16, 0.13
- Check 11: Structural integrity-Plate 0.23, 0.24, 0.24, 0.4, 0.3
- Check 12: Structural integrity-Beam 0.24, 0.23, 0.17, 0.3

**Checks for Flange Beam on Side 2**
- Check 1: Detailing Practice 1.00
- Check 2: Bolt Shear & Bearing 0.42, 0.37, 0.34
- Check 3: Sup.Beam Connect Elmts. 0.37, 0.31, 0.40, 0.12
- Check 4: Shear & Bending 0.23, 0.22, 0.13
- Check 8: Welds 1.191.19
- Check 10: Supporting Member 0.20, 0.14
- Check 11: Structural integrity-Plate 0.19, 0.20, 0.20, 0.3, 0.2
- Check 12: Structural integrity-Beam 0.19, 0.21, 0.16, 0.3

**Checks for Web Beam on Side 1**
- Check 1: Detailing Practice 1.00
- Check 2: Bolt Shear & Bearing 0.42, 0.37, 0.34
- Check 3: Sup.Beam Connect Elmts. 0.37, 0.31, 0.40, 0.12
- Check 4: Shear & Bending 0.23, 0.22, 0.13
- Check 8: Welds 1.191.19
- Check 10: Supporting Member 0.20, 0.14
- Check 11: Structural integrity-Plate 0.19, 0.20, 0.20, 0.3, 0.2
- Check 12: Structural integrity-Beam 0.19, 0.21, 0.16, 0.3

**Checks for Web Beam on Side 2**
Beam 1: 457x191 UB 67 [S275]
2 No. 75 x 75 x 8 L x 360 mm (2x3 kg)
With 5 No. 22 mm holes in webs and 5 No. 22 mm holes in toes
For 15 No. 20 mm Ø Grade 4.8 Bolts.

Beam 2: 457x191 UB 67 [S275]
2 No. 90 x 90 x 10 L x 360 mm (2x5 kg)
With 5 No. 22 mm holes in webs and 5 No. 22 mm holes in toes
For 15 No. 20 mm Ø Grade 4.8 Bolts.

Beam to Column Angle Cleat
Beam to Column Angle Cleat Connection to EC 3 (UK NAD)

User Defined Applied Forces at Interface

<table>
<thead>
<tr>
<th>Flange 1</th>
<th>Flange 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>200.0 kN</td>
<td>200.0 kN</td>
</tr>
</tbody>
</table>

Tie Forces
75.0 kN

Design to

Basic Dimensions

<table>
<thead>
<tr>
<th>Flange 1</th>
<th>Flange 2</th>
<th>Web 1</th>
<th>Web 2</th>
<th>Column</th>
<th>Plates</th>
</tr>
</thead>
<tbody>
<tr>
<td>453.4</td>
<td>189.9</td>
<td>12.7</td>
<td>8.5</td>
<td>10.2</td>
<td>S275</td>
</tr>
<tr>
<td>453.4</td>
<td>189.9</td>
<td>12.7</td>
<td>8.5</td>
<td>10.2</td>
<td>S275</td>
</tr>
<tr>
<td>453.4</td>
<td>189.9</td>
<td>12.7</td>
<td>8.5</td>
<td>10.2</td>
<td>S275</td>
</tr>
<tr>
<td>203.2</td>
<td>203.6</td>
<td>11.0</td>
<td>7.2</td>
<td>10.2</td>
<td>S275</td>
</tr>
</tbody>
</table>

Grade 4.8 Bolts

All weld grades provided to suit minimum connected steel grade

(Clearance) - Area of Cleats adjacent to supported beam web (like a fin-plate)

(Toe) - Area of Cleats Perpendicular to supported beam web (like an end-plate)
## Summary of Results (Unity Ratios)

**Checks for Flange Beams 1 & 2**

| Checks 8 & 9 Bearing & Shear (Toes) | 313.6, 313.6, 313.6, 1076.0, 912.4 >= 200.0kN | 0.64 | OK |
| Check 10 Shear (Toes)               | 707.3, 698.3 >= 100.0kN                         | 0.14 | OK |
| Tie checks                          | 0.59, 0.41, 0.61, 0.52                          | 0.61 | OK |
| Check 2: Bolt shear & Bearing (web) | 0.63, 0.25, 0.49                                | 0.63 | OK |
| Check 3: Sup. Beam Connect Elmts. (web) | 0.22, 0.19, 0.25, 0.08                  | 0.25 | OK |
| Check 4: Shear & Bending (web)      | 0.31, 0.29, 0.22                               | 0.31 | OK |

**Checks for Web Beams 1 & 2**
1 Eaves Joint

 Plates S 275
 Beam 457x191 UB 67 [S275]
 Haunch 457x191 UB 67 [S275]
 Column 610x178 UB 92 [S275]
 Top 20 above top flange
 Bot 400 below bottom flange

End-Plate 940 x 210 x 20 mm (31 kg)
8 No. M20 Grade 8.8 Bolts in 22 mm holes
Haunch Stiff 8 mm with 6 FW
Apex Joint

Plates S 275
Beam 457x191 UB 67 [S275]
Haunch 457x191 UB 67 [S275]

Apex Joint

End-Plate 940 x 205 x 15 mm (23 kg)
10 No. M20 Grade 8.8 Bolts in 22 mm holes
End-Plated Splice

Plates S 275
Beam 457x191 UB 67 [S275]

End-Plate 493 x 210 x 15 mm (12 kg)
10 No. M20 Grade 8.8 Bolts in 22 mm holes
Asymmetrical Base-Plate

Base-Plate 600 x 390 x 30 mm (55 kg)
With 7 No. 24 mm holes
For 20 mm Grade 8.8 Bolts.

Washers 100 x 100 x 15 mm thick with 50 mm Void
All Plates S 275
Reinforcement 12 @ 200 mm cc each way. Cover 0 mm
Beam-Splice Outer Plates

Splice Plates

Top Flange
1 No 600 x 210 x 20 mm (20 kg)

Web
2 No 380 x 375 x 8 mm (2x9 kg)

Bottom Flange
1 No 520 x 210 x 20 mm (17 kg)

HSFG - Pt 2 Bolts (non-slip at service)
44 No. 20 mm Ø Bolts
In 22 mm Holes

Beams 533x210 UB 101 [S275]
All Plates S 275
Beam gap 5 mm
Column Splice with division Plate

Upper Column 203x203 UC 46 [S275]
Lower Column 254x254 UC 73 [S275]

External Flange Plates
2/56 x 190 x 10 mm (2x8 kg)
External Packers
2/256 x 190 x 25 (aprox) mm (2x10 kg)
Welded to Flange with 6 mm FW (top & sides)

Bearing Divider Plate
254 x 205 x 25 mm (10 kg)

Web Angle Cleats
4/90 x 90 x 8 Ang x 150 mm (4x2 kg)
2 bolts per leg. 90 cc with 50 back mark

HSFG - Pt 2 Bolts (non-slip at service)
32 No. 20 mm Ø Bolts
In 22 mm Holes

All Plates S 275
Column Prepared for Direct Bearing
CHS Hollow Section Splice

660x20CHS [S275]
End-Plate 860x15 mm (68 kg)
Plates S 275
12 No. M20 Grade 8.8 Bolts in 22 mm holes
On 760 mm PCD
Beam to Beam Flexible End-Plate - with Asymetrical Plate Girders

Left Beam: 457x191 UB 82 [S275]
Top Notch: 35 x 123 mm lg. net
Bot Notch: 50 x 123 mm lg. net
Plate: 10 x 150 x 290 mm dp. (3 kg)
With 6 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Beam: 250x25T+160x20B+405x15WPg121.87 [S275]
End-Plates S 275

Right Beam: 356x171 UB 57 [S275]
Top Notch: 35 x 123 mm lg. net
Plate: 10 x 150 x 290 mm dp. (3 kg)
With 8 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.
**Beam to Beam Fin-Plate**

Left: 457x152 UB 67 [S275]
Top Notch: 35 x 102 mm lg. net
Plate: 10 x 100 x 360 mm dp. (3 kg)
With 5 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Beam: 533x210 UB 82 [S275]
Beam to web gap 10 mm
Fin-Plates S 275

Right: 457x191 UB 98 [S275]
Top Notch: 35 x 102 mm lg. net
Plate: 10 x 130 x 360 mm dp. (4 kg)
With 8 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.
Beam to Beam Angle Cleats

Left: 457x152 UB 67 [S275]
2 No. 90 x 90 x 10 L x 360 mm (2x5 kg)
With 5 No. 22 mm holes in webs
and 5 No. 22 mm holes in toes
For 15 No. 20 mm Ø Grade 8.8 Bolts.

Beam: 533x210 UB 82 [S275]
Beam to web gap 10 mm
Web to cleat gap 2 mm
Cleats grade: S 275
Beam 1: 457x152 UB 67 [S275]
Plate: 10 x 150 x 455 mm dp. (5 kg)
With 12 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Beam 2: 457x152 UB 67 [S275]
Plate: 10 x 150 x 455 mm dp. (5 kg)
With 12 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.
Beam to Column Fin-Plate

Beam 1: 430x100 PFC [S275]
Beam to Column Gap 10 mm
Plate: 10 x 100 x 360 mm dp. (3 kg)
With 5 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Beam 2: 250x25T+300x25B+400x12WPg145.62 [S275]
Beam to Column Gap 10 mm
Plate: 10 x 200 x 360 mm dp. (6 kg)
With 10 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Beam 1: 533x210 UB 82 [S275]
Beam to Column Gap 10 mm
Plate: 10 x 100 x 430 mm dp. (3 kg)
With 6 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.

Beam 2: 610x178 UB 82 [S275]
Beam to Column Gap 10 mm
Plate: 10 x 100 x 500 mm dp. (4 kg)
With 7 No. 22 mm holes
For 20 mm Ø Grade 8.8 Bolts.
Beam to Column Angle Cleat

Beam 1: 457x191 UB 67 [S275]
2 No. 75 x 75 x 8 L x 360 mm (2x3 kg)
With 5 No. 22 mm holes in webs
and 5 No. 22 mm holes in toes
For 15 No. 20 mm Ø Grade 4.8 Bolts.

Beam 2: 457x191 UB 67 [S275]
2 No. 75 x 75 x 8 L x 360 mm (2x3 kg)
With 5 No. 22 mm holes in webs
and 5 No. 22 mm holes in toes
For 15 No. 20 mm Ø Grade 4.8 Bolts.