Structural Steel Design Project	Job No:	Sheet ***1 of 1***	Rev
	Job Title: *Eccentrically Loaded Bolt Group*		
	Worked Example – 1		
		Made by *SRSK*	Date *01-10-00*
Calculation Sheet		*Checked by VK*	Date

Design Example 1: *Design a bolted connection between a bracket 8 mm thick and the flange of an ISHB 400 column using HSFG bolts, so as to carry a vertical load of 100 kN at a distance of 200 mm from the face of the column as shown in Fig. E1.*

Remarks
Ref:
Section 2.1

Solution:

1) Bolt force:

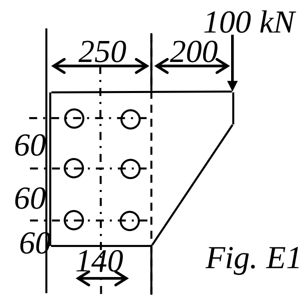

Fig. E1

$P_x = 0$; $P_y = 100$ kN;

Total eccentricity $x' = 200 + 250/2 = 325$ mm

$M = P_y x' = 100 \times 325 = 32500$ kN-mm

Try the arrangement shown in Fig. E1
Note: minimum pitch = 60 mm and
minimum edge dist. = 60 mm

$n = 6$

$\sum r_i^2 = \sum x_i^2 + \sum y_i^2 = 6(70)^2 + 4(60)^2 = 43800\ mm^2$

Equation (8)

Shear force on the farthest bolts (corner bolts)

$$R_i = \sqrt{\left\{\left[\frac{32500 \times 60}{43800}\right]^2 + \left[\frac{100}{6} + \frac{32500 \times 70}{43800}\right]^2\right\}} = 81.79\ kN$$

2) Bolt capacity
Try M20 HSFG bolts

Bolt capacity in single shear $= 1.1\ K\ \mu\ P_o = 1.1 \times 0.45 \times 177 = 87.6$ kN

ISHB 400 flange is thicker than the bracket plate and so bearing on the bracket plate will govern.
Bolt capacity in bearing $= d\ t\ p_{bg} = 20 \times 8 \times 650 \times 10^{-3} = 104$ kN

$\therefore$ *Bolt value = 87.6 kN > 81.79 safe.*

Use 6 M20 HSFG bolts as shown.

Structural Steel Design Project	Job No:	Sheet ***1 of 2***	Rev
	Job Title: *Beam Splice*		
	Worked Example – 2		
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Calculation Sheet		Checked by *VK*	Date

Design Example 2: *Design a bolted splice for an ISMB 450 section to transfer a factored bending moment of 150 kN-m and a factored shear of 100 kN. Assume that the flange splices carry all of the moment and that the web splice carries only the shear.*

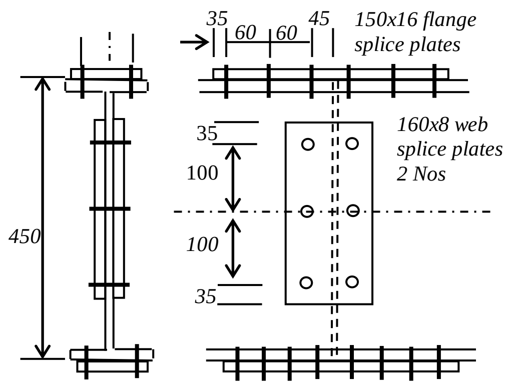

Fig. E2

Solution:

1) Flange Splices :

Flange force $= BM/(D-t_f) = 150 \times 10^3/(450-17.4) = 346.7$ *kN*

For M20 Gr.8.8 HSFG bolts in single shear

Slip resistance per bolt $= 1.1 \times 0.45 \times 144 = 71.3$ *kN* — $1.1 K_s \mu P_o$

Bearing resistance on flange per bolt $= 20 \times 17.4 \times 650 \times 10^{-3} = 226.2$ *kN* — $d t p_{bg}$

Bolt value = 71.3 kN

Use 3 rows of 2 bolts at a pitch of 60 mm

Net area of flange $= (150-2 \times 22)\,17.4 = 1844.4$ mm^2

Flange capacity $= (250/1.15) \times 1844 \times 10^{-3} = 400.9$ *kN > flange force OK* — $\gamma_m = 1.15$

Try 150 mm wide splice plate

Thickness of splice plate required

$= 346.7 \times 10^3/1.0 \times 250(150-2 \times 22)/1.15 = 15.1$ *mm Use 16 mm*

Use flange splice plate of size $400 \times 150 \times 16$

Flange splice plate of size $400 \times 150 \times 16$

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	Job Title: *Beam Splice*		
	Worked Example – 2		
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Calculation Sheet		Checked by *VK*	Date

2) Web Splice

For M20 HSFG bolts of Gr.8.8 in double shear
Slip resistance per bolt $= 2 \times 1.1 \times 0.45 \times 144 = 142.6$ *kN*

Try 8 mm thick web splice plates on both sides of the web.

Therefore bearing on web will govern
Bearing Resistance per bolt $= 20 \times 9.4 \times 650 \times 10^{-3} == 122.2$ *kN*
Bolt value = 122.2 kN

Try 3 bolts at 100 mm vertical pitch and 45 mm from the center of joint.

Horizontal shear force on bolt due to moment due to eccentricity
$= 100 \times 45 \times 100/(2 \times 100^2) = 22.5$ *kN*

Vertical Shear force per bolt = 100/3 = 33.3 kN

Resultant shear force $= \sqrt{(22.5^2+33.3^2)} = 40.2$ *kN < 122.2(bolt cap) OK*

Use web splice plate of size 270×160×8 - 2 nos.

Web splice plate of size 270×160×8 with 3 M20 bolts on each side of the splice.

Structural Steel Design Project		Sheet ***1 of 2***	Rev
	Job Title: *Column Splice*		
	Worked Example – 3		
		Made by *SRSK*	Date *01-10-00*
Calculation Sheet		Checked by *VK*	Date

Design Example 3:*Design a bolted cover plate splice for an ISHB 200 @ 50.94 kg/m column supported by an ISHB 200 @ 47.54 kg/m column so as to tramsfer a factored axial load of 440 kN. The splice is near a point of lateral restraint. The ends are not prepared for full contact in bearing.*

Remarks

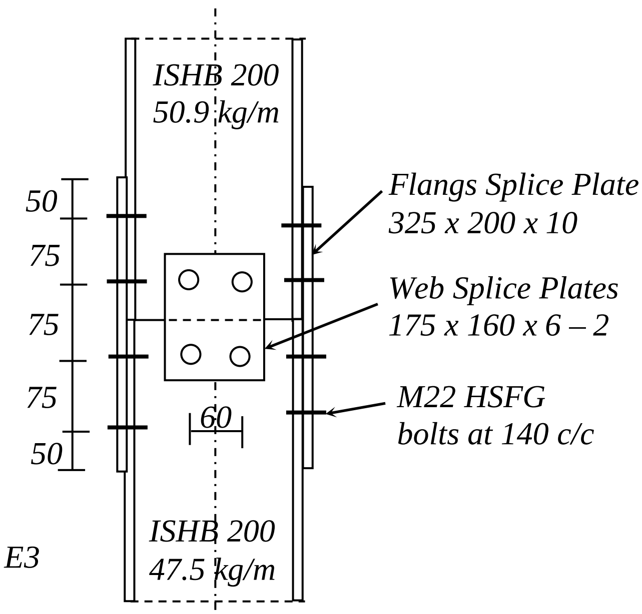

Fig. E3

Solution:

1) Area of ISHB 200 @ 47.54 kg/m section = 4754 mm^2

Area of web = (200-2 × 9) × 6.1 = 1110.2 mm^2

2) Web Splice

Portion of load carried by web = 440 × 1110.2/4754 = 102.8 kN

For M22 HSFG bolts, 2 Nos in double shear

Shear force /bolt = 102.8/2 = 51.4 kN

Slip resistance/bolt = 2 × 1.1 × 0.45 × 177 = 175.2 kN

Bearing resistance/bolt = $22 \times 6.1 \times 650 \times 10^{-3} = 87.62$ *kN*

Bolt value = 87.62 kN > bolt force of 51.4 kN ∴ OK

End distance > $51.4 \times 10^3/(1/3 \times 6.1 \times 650) = 39$ *mm*

Also end distance > 1.4(22+1.5) = 35 mm Use 50 mm

Use 175 × 160 × 6 mm web splice plates – 2 Nos.

Web splice
175 × 160 × 6

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3) Flange Splice

Portion of load carried by each flange = 0.5(440-102.8) = 168.6 kN
For M22 HSFG bolts, 4 Nos in single shear
Shear force /bolt = 168.6/4 = 42.15 kN

Slip resistance/bolt $= 1.1 \times 0.45 \times 177 = 87.62$ *kN*
Bearing resistance/bolt = $22 \times 9 \times 650 \times 10^{-3} = 128.7$ *kN*
Bolt value = 87.62 kN > bolt force of 42.15 kN $\therefore$ *OK*

End distance > $42.15 \times 10^{3} / (1/3 \times 9 \times 650) = 21.62$ *mm*
Also end distance > 1.4(22+1.5) = 35 mm Use 50 mm

Use 325×200×10 mm flange splice with bolts at 140 mm gauge, 75 mm pitch

flange splice 325×200×10

Structural Steel Design Project	Job No:	Sheet **1 of 2**	Rev
	Job Title: *Bolted Seating Angle Connection*		
	Worked Example – 4		
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Calculation Sheet		Checked by *VK*	Date

Design Example 4: *Design a Seating angle connection for an ISMB 400 beam to an ISHB 200 column so as to transfer a shear of 150 kN.*

Remarks

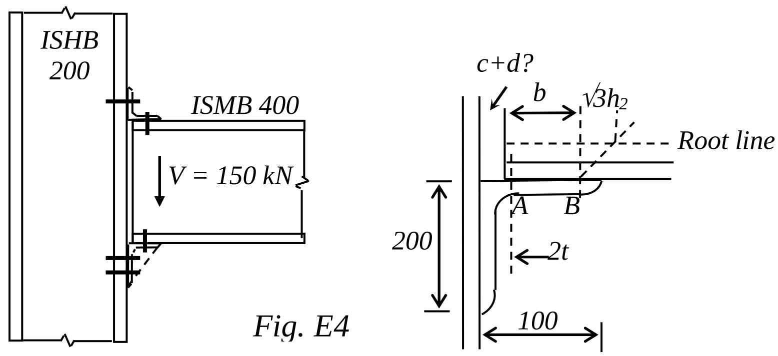

Fig. E4

1) Seating Angle

The support reaction acts as a UDL over length $(b + \sqrt{3}h_2)$ on the web

Length of bearing required at root line of beam $(b + \sqrt{3}\,h_2)$

$= V/(t_w p_{yw}) = 150 \times 10^3/(8.9 \times 250/1.15) = 77.53$ *mm*

Length of bearing on cleat $= b = 77.53 - \sqrt{3}h_2 = 77.53 - (\sqrt{3})32.8 = 20.7$ *mm*

end clearance of beam from the face of the column $c = 5$ *mm*

allow tolerance $d = 5$ *mm*

minimum length of angle leg required for seating $= b + c + d = 30.7$ *mm*

Try ISA $200 \times 100 \times 12$ *angle of length* $w = b_f = 140$ *mm*

Distance from end of bearing on cleat to root of angle (A to B)

$= b + c + d - (t + r)$ *of angle; assuming* $r = t$ *for angle*

$= b + 5 + 5 - (2t) = 20.7 + 5 + 5 - 24 = 6.7$ *mm*

assuming the load to be uniformly distributed over the bearing length b

moment at the root of angle $= (150/20.7) \times 6.7^2/2 = 162.6$ *N-m*

Moment capacity $= 1.2 p_y Z = 1.2 \times (250/1.15) \times (140 \times 12^2/6) \times 10^{-3}$

$= 876.5$ *N-m OK*

Note :[The maximum moment occurs to the left of point A. To account for it the section modulus is taken as $1.2wT^2/6$ *instead of* $wT^2/4$*].*

Shear Capacity of outstanding leg of cleat

$= 0.6 p_y \times 0.9 wt = 0.6 \times (250/1.15) \times 0.9 \times 140 \times 12 \times 10^{-3}$

$= 197.2$ *kN* > 150 *kN OK*

Use ISA $200 \times 100 \times 12$

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2) Connection of seating angle to column flange *Bolts required to resist only shear* *Try 4 bolts of 20mm dia and grade 4.6 at angle back marks* *Total shear capacity* $= 4 \times 160 \times 245 \times 10^{-3} = 156.8$ *kN > 150 kN OK* *Column flange critical for bearing of bolts* *Total bearing capacity* $= 4 \times 418 \times 20 \times 9.0 \times 10^{-3} = 301$ *kN > 150 kN OK* *3) Provide nominal clip angle of ISA* $50 \times 50 \times 8$ *at the top*	 $Np_{bs}dt_f$

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Design Example 5: *Design a bolted web cleat beam-to-column connection between an ISMB 400 beam and an ISHB 200 @ 40 kg/m column. The connection has to transfer a factored shear of 150 kN. Use bolts of diameter 20 mm and grade 4.6.*

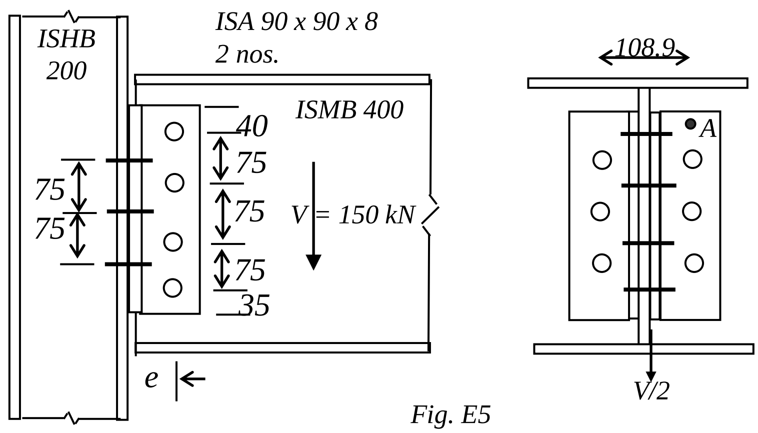

Fig. E5

1) The recommended gauge distance for column flange is 100 mm.
Therefore required angle back mark is 50 mm.
Use web cleats of ISA 90x90x8 giving gauge g = 50+50+8.9=108.9 mm — *g for ISHB200 is 100 mm OK*

2) Connection to web of beam- Bolt capacity
shear capacity of bolt in double shear = $2\times160\times245\times10^{-3} = 78.4$ kN
bearing capacity of bolt on the beam web = $418\times20\times9.0\times10^{-3} = 75.24$ kN
bolt value = 75.24 kN

Try 4 bolts as shown in the Figure with vertical pitch of 75 mm

Assuming the shear to be acting on the face of the column, its eccentricity with the centre of the bolt group will produce horizontal shear forces in the bolts in addition to the vertical shear.

horizontal shear force on top bolt due to moment due to eccentricity e
$= 150\times50\times112.5/2(37.5^2+112.5^2) = 30.0$ kN — $P_x\, e\, r_i/\Sigma r_i^2$

vertical shear force per bolt = 150/4 = 37.5 kN

resultant shear = $\sqrt{(30.0^2+37.5^2)} = 48.0$ kN < *bolt value Safe !*

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3) Connection to column flange: Bolt capacity

shear capacity of bolt in single shear $= 160 \times 245 \times 10^{-3} = 39.2$ *kN*
bearing capacity of bolt on column flange $= 418 \times 20 \times 9.0 \times 10^{-3} = 75.24$ *kN*
bolt value = 39.2 kN

Try 6 bolts as shown in the Fig.E5 with vertical pitch of 75 mm

4) Check bolt force
Similar to the previous case, the shear transfer between the beam web and the angle cleats can be assumed to take place on the face of the beam web. However, unlike the previous case, no relative rotation is possible between the angle and the beam web.

Assuming centre of pressure 25 mm below top of cleat (point A), horizontal shear force on bolt due to moment due to eccentricity e
$= (150 \times 50/2) \times 200/(50^2+125^2+200^2) = 12.9$ *kN* — $(V/2)e_x r_i/\Sigma r_i^2$

vertical shear force per bolt = 150/6 = 25.0 kN

resultant shear $= \sqrt{(12.9^2+25.0^2)} = 28.13$ *kN < bolt value OK*

Use 2 Nos ISA 90x90x8 of length 375 mm as angle cleats — *ISA 90x90x8*
Length 375mm

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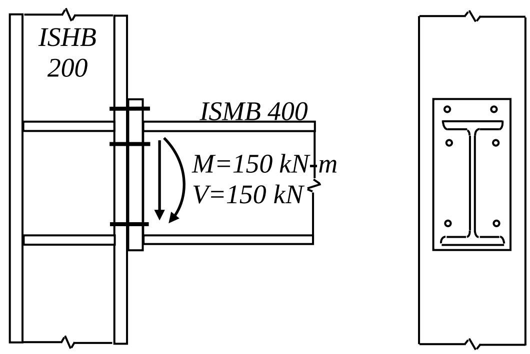
Design Example 6: *Design a bolted end plate connection between an ISMB 400 beam and an ISHB 200 @ 40 kg/m column so as to transfer a hogging factored bending moment of 150 kN-m and a vartical factored shear of 150 kN. Use HSFG bolts of diameter 22 mm.*



1) bolt forces

taking moment about the centre of the bottom flange and neglecting the contribution of bottom bolts and denoting the force in the top bolts by F

$4F \times 384 = 150 \times 10^3$

$F = 97.6$ kN

tension capacity of M22 bolt $= 0.9P_o = 159.3$ kN

allowable prying force $Q = 159.3 - 97.6 = 61.7$ kN

2) design for prying action

try 30 mm thick end plate of width $b_e = 180$ mm

distance from the centre line of bolt to prying force n is the minimum of edge distance or $1.1T\sqrt{\beta Po/Py} = 1.1 \times 30\ \sqrt{(2 \times 512/250)} = 55.66$ mm

$n = 40$ mm

assuming 10 mm fillet weld,

distance from center line of bolt to toe of fillet weld $b = 60-10 = 50$ mm;

moment at the toe of the weld $= Fb - Qn = 97.6 \times 50 - 61.7 \times 40 = 2412$ N-m

effective width of end plate per bolt $w = be/2 = 180/2 = 90$ mm

moment capacity $= (250/1.15)(90 \times 30^2/4) = 4402$ N-m > 2412 N-m *Safe !* $(py/1.15) \times (wT^2/4)$

Structural Steel Design Project	Job No:	Sheet ***2 of 2***	Rev
	Job Title: *Bolted End Plate Connection*		
	Worked Example - 6		
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$$min\ Q = \frac{50}{2\times 40}\left[97.6 - \frac{2\times 1.5\times 0.587\times 90\times 30^4}{27\times 40\times 50^2}\right]$$	$Q = \frac{b}{2n}\left[F - \frac{\beta\gamma P_o w T^4}{27nb^2}\right]$
Q = 31.8 kN < 61.7 kN OK	*β=2 (non-preloaded)* *γ =1.5 (for factored load)*
3) Check for combined shear and tension	
Shear capacity of M20 HSFG Ps l= 87.6 kN	
Shear per bolt Fs = 150/6 = 25 kN	
= (25.0/87.6) + (97.6+31.8)/159.3 = 0.936 < 1.0 Safe !	$F_s/P_{sl} + 0.8f_t/P_t$

Structural Steel Design Project	Job No:	Sheet ***1 of 2***	Rev
	Job Title: *Beam to Beam Connection*		
	Worked Example - 7		
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Calculation Sheet		Checked by *VK*	Date

Design Example 7*: Design a double web cleat connection for an ISMB 400 coped beam to an ISMB 600 main beam so as to transfer a factored load of 300 kN using HSFG bolts of 20mm diameter and grade 8.8.*

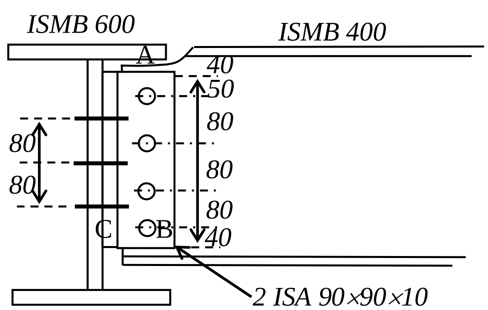

Fig. E7

Solution:

1) Connection to web of ISMB 400

For M20 Gr.8.8 HSFG bolts in double shear
Slip resistance per bolt = $2\times1.1\times0.45\times144$ = 142.6 kN
Bearing capacity of web per bolt = $20\times8.9\times650\times10^{-3}$ = 115.7 kN
Bolt value = 115.7 kN

Try 4 bolts as shown in the Figure with vertical pitch of 80 mm

Assuming the shear to be acting on the face of the ISMB 600 web, its eccentricity with the centre of the bolt group will produce horizontal shear forces in the bolts in addition to the vertical shear.

horizontal shear force on top bolt due to moment due to eccentricity e
$= (300/2)\times50\times112.5/(37.5^2+112.5^2) = 60.0$ kN

vertical shear force per bolt = 300/4 = 75.0 kN

resultant shear = $\sqrt{(60^2+75^2)}$ = 96.0 kN < bolt value Safe !

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2) *Connection to web of ISMB 600*

Try 6 bolts as shown in the Figure with vertical pitch of 80 mm

For M20 Gr.8.8 HSFG bolts in single shear
Slip resistance per bolt $= 1.1 \times 0.45 \times 144 = 71.28$ *kN*
Bearing capacity of web per bolt $= 20 \times 12 \times 650 \times 10^{-3} = 156$ *kN*
Bolt value $= 71.28$ *kN*

Assuming center of pressure 27.5 mm below the top of the angle

horizontal shear force on bottom bolt due to moment due to eccentricity e
$= (300/2)\times 50\times 200/(50^2+125^2+200^2) = 25.82$ *kN*

vertical shear force per bolt $= 300/6 = 50.0$ *kN*

resultant shear $= \sqrt{(25.82^2+50^2)} = 56.27$ *kN < bolt value Safe !*

3) *Check web of ISMB 400 for block shear*

Block shear capacity = shear capacity of AB + $0.5\times$*tensile capacity of BC*
$= 0.6\times 250\times 0.9\times 1.1(3\times 80+50-3.5\times 22)\times 8.9\times 10^{-3}$
$+ 0.5\times 250\times 1.1(45-0.5\times 22)\times 8.9\times 10^{-3} = 323.12 > 300$ *kN* *Safe!*