| CHAPTER 2 |

Connections

❍ **Learning Objectives**

- Analysis of different types of bolts and their connections.
- Analysis of different types of welds and their connections.
- Production of valid designs for bolted and welded tension splice connections.
- Production of valid designs for beam-column bolted double angle web cleat connections.
- Production valid designs for beam-column welded fin plate connections.

1. Bolts and Welds

Connections can be made by
(a) Bolts - ordinary bolts in clearance holes
(b) Friction grip bolts
(c) Welding - fillet and butt weld.

1.1 Bolted Connections

Ordinary bolts are in three strength grades in BS4190:2001 as specified below:

Table 1 - Strength of Bolts

Grade	Yield Stress (N/mm^2)	Ultimate Tensile Stress (N/mm^2)
4.6	240	400
8.8	640	800
10.9	900	1000

* Grade 8.8 bolts will be used in our examples

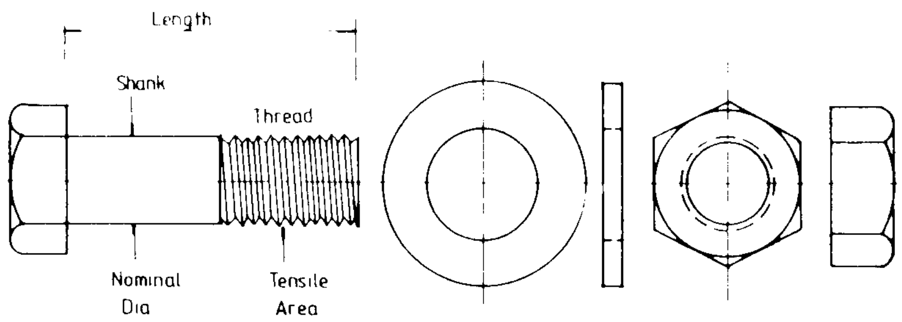


Figure 1 - Hexagon head bolt, nut and washer (extracted from ref. 6)

The main diameters of bolts used are 12, 16, 20, (22), 24, (27) and 30 mm. Diameters in brackets are non-preferred sizes. The nominal diameters of holes for ordinary bolts are greater than the bolt diameter by:

2 mm for bolts up to and including 24 mm diameter

3 mm for larger diameter bolts.

Bolts may be arranged to act in single or double shear, as shown below.

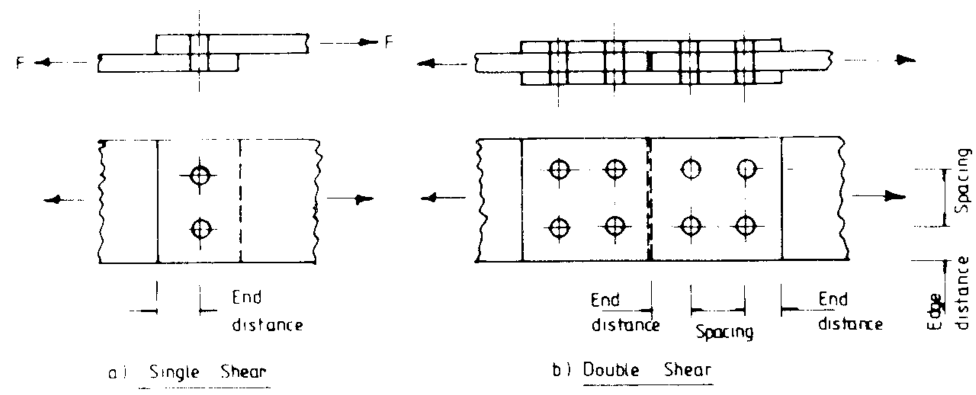


Figure 2 – Bolts in single and double shear (extracted from ref. 6)

As you can see from a detailed look of a typical hexagon bolt, the shank and the thread have different cross sectional areas. The thread area is usually called the tensile stress area as this area is used to calculate the tensile capacity of a bolt. The shank area and thread area of typical bolts are shown below :

Table 2 - Areas of Bolts

Nominal Diameter (mm)	Shank Area (mm^2)	Tensile Stress Area (mm^2)
12	113	84.3
16	201	157
20	314	245
22	380	303
24	452	353
30	707	561

1.1.1 Bolt spacing (clause 9.3.1)

(1) The minimum spacing between centres of bolts in the direction of load transfer is 2.5 times the nominal bolt diameter ***d***.

(2) The spacing between centres of bolts measured perpendicular to the direction of load transfer should normally be 3 times the nominal bolt diameter ***d***.

(3) The maximum spacing between centres of bolts measured either parallel or perpendicular to the direction of load transfer should be the lesser of 12 ***t*** or 150 mm where ***t*** is the thickness of the thinner plate connected.

1.1.2 End and edge distances (clause 9.3.2)

The end distance shall be sufficient to provide adequate bearing capacity. The minimum end and edge distances are listed in Table 9.3 below.

Table 9.3 - Minimum end and edge distances of holes (for standard holes)

Bolt Size	At sheared and hand flame cut edge(mm)	At rolled edges of plates, shapes, bars or gas cut edges (mm)
M12	22	18
M16	28	22
M18	32	24
M20	34	26
M22	38	28
M24	42	30
M27 and over	1.75*d*	1.25*d*

Table 3 – Extract of Table 9.3 of the HK Code[1]

The maximum edge distance is 11 $t\,\varepsilon$, where $\varepsilon = \sqrt{\dfrac{275}{p_y}}$ and ***t*** is the thickness of the connected thinner plate.

Table 4 - Load Capacity for Grade 4.6 Bolts

Nominal diameter (mm)	Shank area (mm^2)	Tensile stress area (mm^2)	Tension Capacity @ 240 N/mm^2 (kN)	Shear Capacity @ 160 N/mm^2 (kN) at shank	Shear Capacity @ 160 N/mm^2 (kN) at thread
12	113	84.3	20.2	18.1	13.5
16	201	157	37.7	32.1	25.1
20	314	245	58.8	50.2	39.2
22	380	303	72.7	60.8	48.4
24	452	353	84.7	72.3	56.4
30	707	561	134.6	113.1	89.8

Table 5 - Load Capacity for Grade 8.8 Bolts

Nominal diameter (mm)	Shank area (mm^2)	Tensile stress area (mm^2)	Tension Capacity @ 560 N/mm^2 (kN)	Shear Capacity @ 375 N/mm^2 (kN) at shank	Shear Capacity @ 375 N/mm^2 (kN) at thread
12	113	84.3	47.2	42.4	31.6
16	201	157	87.9	75.4	58.9
20	314	245	137.2	117.8	91.9
22	380	303	169.7	142.5	113.6
24	452	353	197.7	169.6	132.4
30	707	561	314.2	265.1	210.4

Table 6- Load Capacity - Grade 8.8 Bolts and Grade S355 Steel						
Nominal diameter (mm)	Bearing Capacity for Grade 4.6 bolts @ 460 N/mm^2 (kN)		Bearing Capacity for Grade 8.8 bolts @ 1000 N/mm^2 (kN)		Bearing Capacity for connecting parts Grade S355 @ 550 N/mm^2 (kN)	
	plate thickness		plate thickness		plate thickness	
	6 mm	8 mm	6 mm	8 mm	6 mm	8 mm
12	33.1	44.2	72.0	96.0	39.6	52.8
16	44.1	58.9	96.0	128.0	52.8	70.4
20	55.2	73.6	120.0	160.0	66.0	88.0
22	60.7	80.9	132.0	176.0	72.6	96.8
24	66.2	88.3	144.0	192.0	79.2	105.6
30	82.8	110.4	180.0	240.0	99.0	132.0

1.2 Welded Connections

Welding is the process of joining metal parts by fusing them and filling in with molten metal from the electrode. Electric arc welding is the main system used. Welding produces neat, strong and more efficient joints than bolting. However, it should be carried out in close supervision.

1.2.1 Types of welds

The two main types of welds are ***butt*** weld and ***fillet*** weld. Butt welds are named after the edge preparation used. Single and double U and V welds are shown in Figure 3. Some other types of welds - the partial butt and deep penetration fillet weld are also shown. A 90° fillet weld is shown but other angles can be used. The weld size is specified by the leg length.

All welded fabrication must be checked, tested and approved before being accepted. Tests included visual inspection for uniformity, surface tests for cracks using dyes or magnetic particles, and X-ray and ultrasonic tests to check for defects inside the weld.

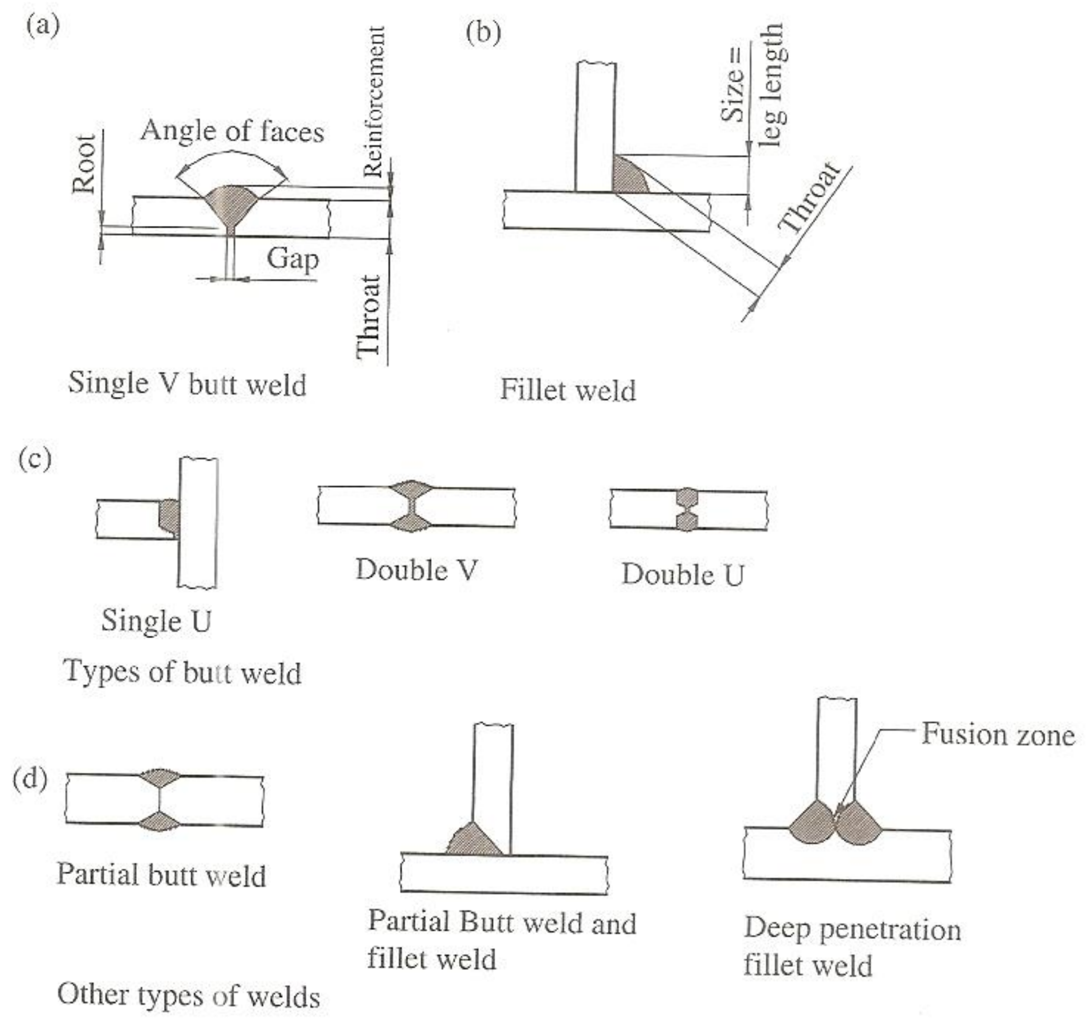


Figure 3 – Weld Types: Fillet and Butt Welds (extracted from ref. 2)

1.3 Design of bolts

A shear joint can fail in the following five ways:
(1) By shear on the bolt shank;
(2) By bearing on the member or bolt;
(3) By tension in the member;
(4) By shear at the end of the member
(5) Block shear.

The failure modes are shown in the following figures.

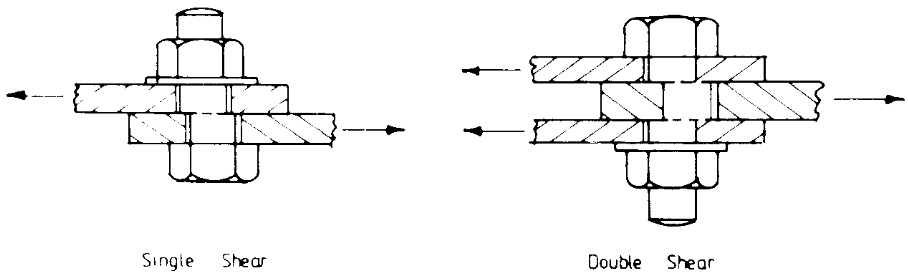


Figure 4 – Shearing of Bolt Shank (extracted from ref.6)

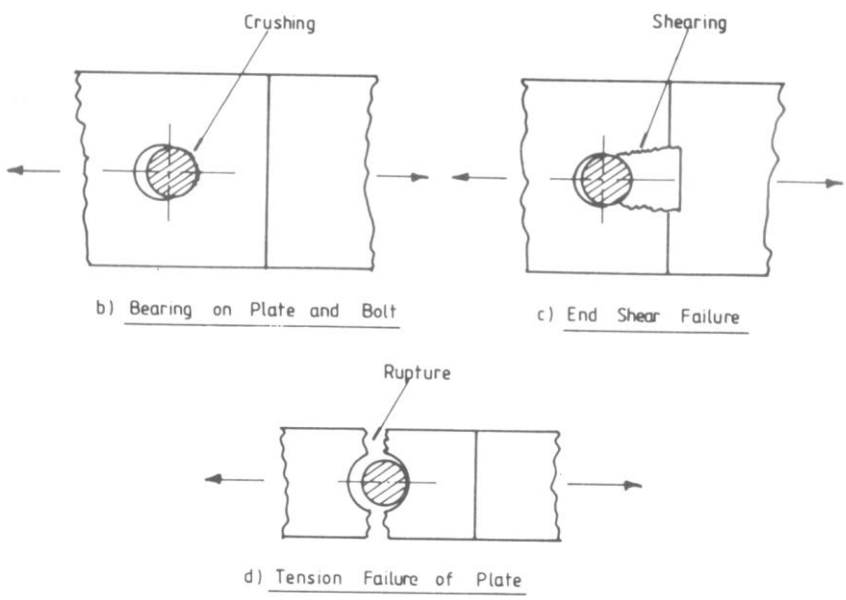


Figure 5 – Failure Modes of a Bolted Joint (extracted from ref. 2)

1.3.1 Ordinary Bolts - Shear

Shear capacity = $P_s = p_s A_s$

where shear area A_s = tensile stress area (or shank area where it is certain that threads cannot occur in the shear plane)

The shear strength (p_s)is given in Table 9.5 of the Code.

Table 9.5 - Design shear strength of bolts

Bolt grade			Design shear strength p_s (N/mm²)
ISO	4.6		160
	8.8		375
	10.9		400
BS	General grade HSFG	≤ M24	400
		≥ M27	350
	Higher grade HSFG		400
ASTM	A307		124
	A325		248
	A490		311
GB50017	8.8		250
	10.9		310
Other grades ($U_b \leq 1000$ N/mm²)			$0.4U_b$
Note: U_b is the specified minimum tensile strength of the bolt.			

Table 7 – Extract of Table 9.5 of the HK Code[1]

1.3.2 Ordinary Bolts - Bearing

Bearing capacity of bolt = $P_{bb} = d\, t_p\, p_{bb}$

Where d is the nominal diameter of the bolt

t_p is the thickness of the thinner connecting part.

The bearing strengths p_{bb} are given in Tables 9.6.

Table 9.6 - Design bearing strength of bolts

Bolt grade			Design bearing strength p_{bb} (N/mm²)
ISO	4.6		460
	8.8		1000
	10.9		1300
BS	General grade HSFG	≤ M24	1000
		≥ M27	900
	Higher grade HSFG		1300
ASTM	A307		400
	A325		450
	A490		485
GB50017	8.8		720
	10.9		930
Other grades ($U_b \leq 1000$ N/mm²)			$0.7(U_b + Y_b)$
Note: U_b is the specified minimum tensile strength of the bolt. Y_b is the specified minimum yield strength of the bolt.			

Table 8 – Extract of Table 9.6 of the HK Code[1]

Bearing capacity of connecting parts should be the least of the followings

$P_{bs} = k_{bs}\, d\, t_p\, p_{bs}$

$P_{bs} = 0.5\, k_{bs}\, e\, t_p\, p_{bs}$

and $P_{bs} = 1.5\, I_c\, t_p\, U_s \quad \leq \quad 2.0\, d\, t_p\, U_b$

where e = end distance measured in the direction of load transfer

k_{bs} = 1.0 for standard holes, 0.7 for oversize and short slotted holes

I_c = net distance between the bearing edge of the holes and the near edge of adjacent holes in the direction of load transfer

p_{bs} = bearing strength of connected parts

for steel grade S275, p_{bs} = 460 MPa

for steel grade S355, p_{bs} = 550 MPa

for steel grade S460, p_{bs} = 670 MPa

U_s is the specified minimum tensile strength of parent metal

for steel grade S235, U_s = 360 MPa

for steel grade S275, U_s = 430 MPa

for steel grade S355, U_s = 510 MPa

U_b is the specified minimum tensile strength of bolts

1.3.3 Ordinary Bolts - tension

Tension capacity = $P_t = A_s\, p_t$

The tension strength (p_t) is given in Table 9.8 of the code.

Table 9.8 - Design tension strength of bolts

Bolt grade			Design tension strength p_t (N/mm²)
ISO	4.6		240
	8.8		560
	10.9		700
BS	General grade HSFG	≤ M24	590
		≥ M27	515
	Higher grade HSFG		700
ASTM	A307		310
	A325		620
	A490		780
GB50017	8.8		400
	10.9		500
Other grades ($U_b \leq 1000$ N/mm²)			$0.7\, U_b$ but $\leq Y_b$

Note: U_b is the specified minimum tensile strength of the bolt.
Y_b is the specified minimum yield strength of the bolt.

Table 9 – Extract of Table 9.8 of the HK Code[1]

Clause 9.3.7.2 states that design against prying force is not required provided that the following conditions are satisfied.
(i) Bolt tension capacity P_t is reduced to $P_{nom} = 0.8\ A_s\ p_t$ where P_{nom} is the nominal tension capacity of the bolt.
(ii) The bolt gauge G on the flange of UB, UC and T sections does not exceed 0.55 B as shown below.

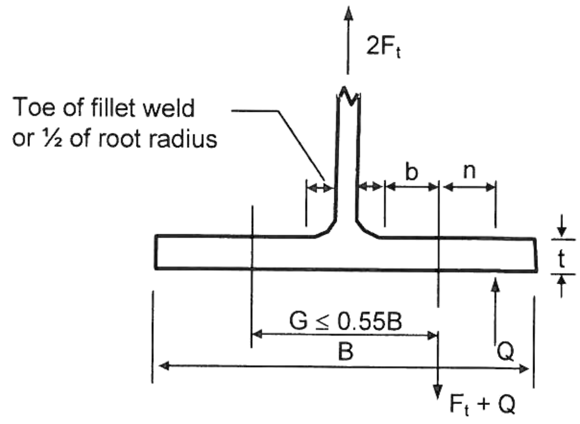


Figure 9.11 - Prying force

Figure 6 – Extract of Figure 9.11 of the HK Code[1]

1.4 Sectional area of connected parts (clause 9.3.4)

Gross area
The gross area a_g should be computed as the products of the thickness and the gross width of the element, measured normal to its axis.

Net area
The net area a_n should be the gross area less the deductions for bolt holes.

Deduction of bolt holes
(a) Holes not staggered – the deduction should be the sum of sectional areas of the bolt holes.
(b) Staggered Holes – the deduction should be the greater of
  (i) The deduction of non-staggered holes, see line 1 of Figure 7
  (ii) The sum of the sectional area of all holes lying on diagonal or zig-zag line less a justification factor of $0.25S^2t/g$ for each gauge that it traverses diagonally, see line 2 and 3 of Figure 7
  (iii) For angles with bolts on both legs, the gauge length **g** should be the sum of the gauge lengths on each leg g_1 and g_2 measured from the heel minus the thickness of the angle, see Figure 7

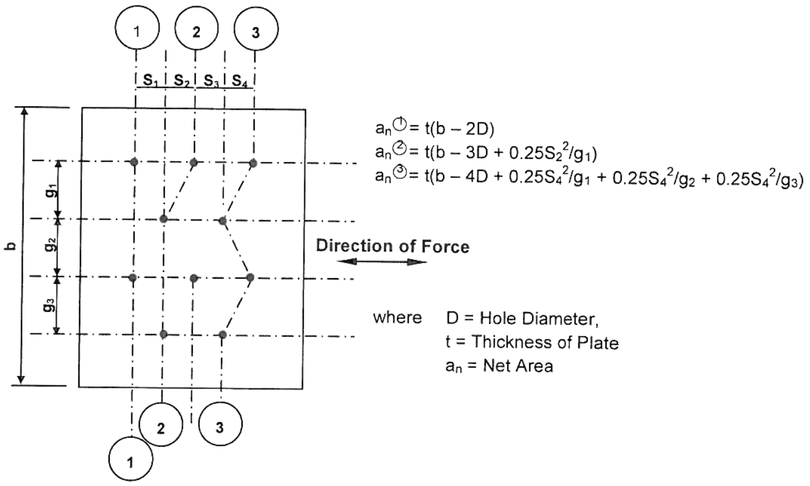


Figure 9.7 - Staggered holes

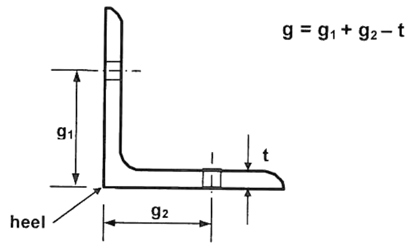


Figure 9.8 - Angles with hole in legs

Figure 7 – Extract of Figure 9.7 and Figure 9.8 of the HK Code[1]

Effective area for tension

The effective area a_e perpendicular to the force direction should be determined from

$$a_e = K_e\, a_n \quad \leq a_g$$

where K_e is the effective net area coefficient and

K_e = 1.2 for steel grade S275

= 1.1 for steel grade S355

= 1.0 for steel grade S460

a_n = net cross sectional area of leg deducted for hole openings

a_g = gross sectional area without deduction for openings

Effective area for shear

Bolt holes need not be allowed for in the shear area provided that

$$A_{v.\,net} \geq 0.85\, A_v / K_e$$

where A_v = gross shear area before hole reduction

$A_{v.\,net}$ = net shear area after deducting bolt holes

Otherwise the net shear capacity should be taken as $0.7\, p_y\, K_e\, A_{v.\,net}$.

Block Shear (clause 9.3.5)

Block shear failure at a group of bolt holes near the end of web of a beam or bracket should be prevented by proper arrangement of a bolt pattern as shown in Figure 8.

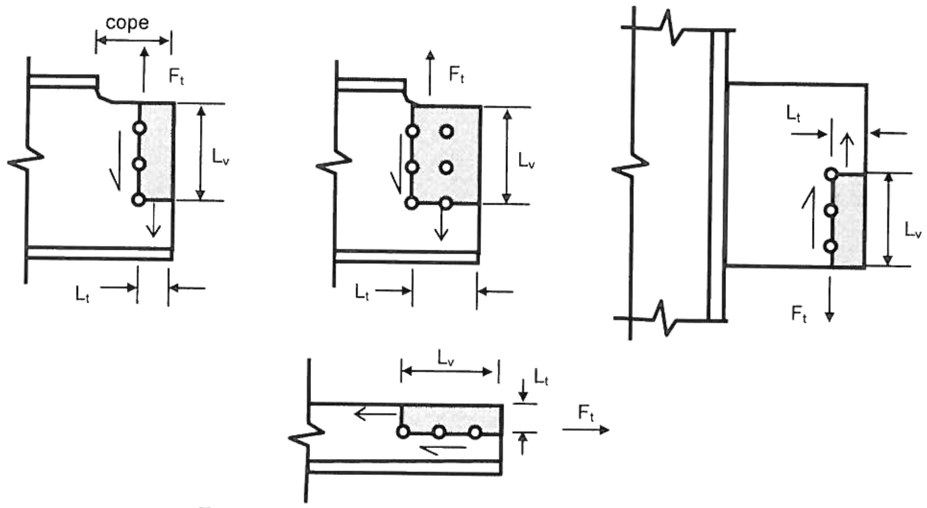


Figure 9.9 - Block shear and effective shear area

Figure 8 – Extract of Figure 9.9 of the HK Code[1]

The mode of failure generally consists of tensile rupture on the tension face along the bolt line accompanied by gross section yielding in shear at the row of bolt holes along the shear face of the bolt group. On the tension side, the block shear capacity should be taken as,

$$P_r = 0.6\, p_y\, A_{v.\,eff}$$

where $A_{v.\,eff}$ is the effective shear area defined as,

$$A_{v.\,eff} = t\,[L_v + K_e\,(L_t - k\,D_t)]$$

t = thickness of connected part

L_v = length of shear face, as shown in Figure 8

L_t = length of tension face

k = 0.5 for single row of bolts or 2.5 for two rows of bolts

D_t = hole diameter for the tension face, but for slotted holes the dimension perpendicular to load direction should be used.

K_e = the effective net area coefficient defined on previous page

1.5 Tension Member (clause 8.6)

1.5.1 Tension capacity

The tension capacity P_t of a member is generally taken as,

$$P_t = p_y A_e$$

in which A_e is the sum of effective areas a_e of all elements in the cross section.

For single angle connected through one leg, single channel through the web or a single T section through the flange, the tension capacity should be obtained as

for bolted connections $P_t = p_y (A_e - 0.5\, a_2)$

for welded connections $P_t = p_y (A_e - 0.3\, a_2)$

where $a_2 = A_g - a_1$

A_g is the sum of gross cross sectional area

a_1 is the gross area of the connected leg, taken as the product of thickness and the leg length of an angle, the depth of a channel or the flange width of a T section

For double angle, channels and T-sections connected on both sides of a gusset plate and interconnected by bolts or welds, the tension capacity should be obtained as

for bolted connections $P_t = p_y (A_e - 0.25\, a_2)$

for welded connections $P_t = p_y (A_e - 0.15\, a_2)$

1.5.2 Tension members with moments (clause 8.8)

When members are connected eccentric to the axis of the member the resulting moment has to be allowed for as below, except for angles, channels and T-sections to be designed as axially loaded members as described above.

The capacity of a tension member under biaxial moment should be checked using the following equation:

$$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

where $\boldsymbol{F_t}$ is the design axial tension at critical section

$\boldsymbol{M_x}$ is the design moment about the major axis at critical section

$\boldsymbol{M_y}$ is the design moment about the minor axis at critical section

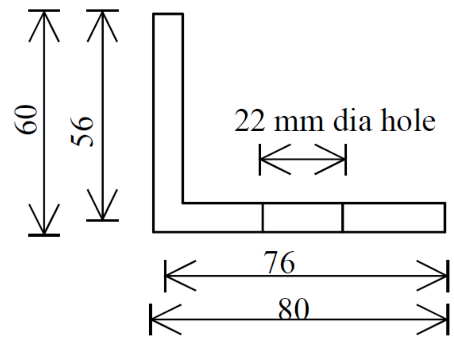
$\boldsymbol{P_t}$ is the tension capacity (see clause 8.6.1)

$\boldsymbol{M_{cx}}$ is the moment capacity about the major axis (see clause 8.2.2)

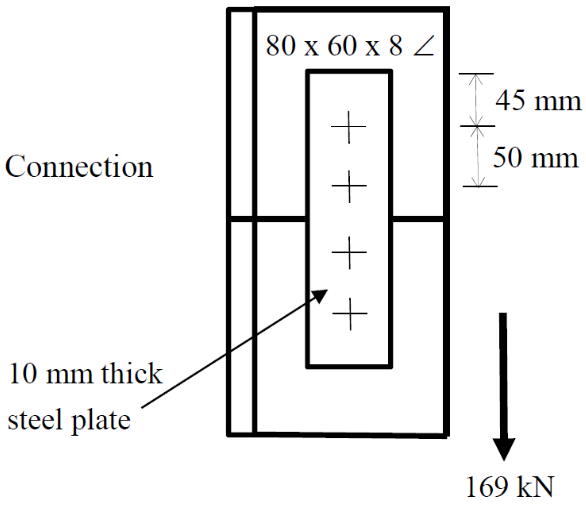
$\boldsymbol{M_{cy}}$ is the moment capacity about the minor axis (see clause 8.2.2)

Example 1 - Angle Connected through One Leg

Check if a Grade S355 unequal single angle (80 x 60 x 8) connected through the long leg as shown below is adequate to carry a dead load of 75 kN and an imposed load of 40 kN using 2 nos. of Grade 8.8 M20 bolted connection. Assume single shear connection with 10 mm plate, one row of bolts, holes are standard and not staggered.   Bolt spacing = 50mm and end distance **e** = 45mm.



Bolted Connection



Solution

Factored load = 1.4 x 75 + 1.6 x 40 = 169 kN

<u>Check capacity of bolts</u>

From Table 9.5 & 9.6, p_s = 375 N/mm^2, p_{bb} = 1000 N/mm^2

Shear capacity per bolt = $P_s = p_s A_s$ = 375 x 245 x 10^{-3} = 91.8 kN

Total shear capacity = 2 x 91.8 = 183.6 kN > 169 kN

$\Rightarrow$ bolt shear is satisfactory.

Bearing capacity of bolt = $P_{bb} = d\, t_p\, p_{bb}$

Comparing angle thickness 8 mm and plate thickness 10 mm, t_p = 8 mm

$P_{bb} = d\, t_p\, p_{bb}$ = 20 x 8 x 1000 x 10^{-3} = 160 kN

Total bolt bearing capacity = 2 x 160 = 320 kN > 169 kN

$\Rightarrow$ bolt bearing is satisfactory.

Bearing capacity of connecting part should be the least of the followings

$P_{bs} = k_{bs}\, d\, t_p\, p_{bs}$

$P_{bs} = 0.5\, k_{bs}\, e\, t_p\, p_{bs}$

and $P_{bs} = 1.5\, l_c\, t_p\, U_s \;\leq\; 2.0\, d\, t_p\, U_b$

Now, for Grade S355, p_{bs} = 550 N/mm^2 and U_s = 510 N/mm^2,
and for standard hole, k_{bs} = 1.0, d = 20 mm, t_p = 8 mm, e = 45 mm
Net distance between the bearing edge and the near edge of adjacent holes in the direction of load transfer I_c = 50 – 22 = 28 mm

For grade 8.8, U_b = 800 N/mm^2, therefore,

$P_{bs} = k_{bs}\, d\, t_p\, p_{bs}$ = 1.0 x 20 x 8 x 550 x 10^{-3} = 88 kN

$P_{bs} = 0.5\, k_{bs}\, e\, t_p\, p_{bs}$ = 0.5 x 1.0 x 45 x 8 x 550 x 10^{-3} = 99 kN

and $P_{bs} = 1.5\, I_c\, t_p\, U_s \leq 2.0\, d\, t_p\, U_b$ = 1.5 x 28 x 8 x 510 x 10^{-3} = 171.4 kN

$\leq$ 2.0 x 20 x 8 x 800 x 10^{-3} = 256 kN

Hence bearing capacity of connecting part = 88 kN
Total bolt bearing capacity = 2 x 88 = 176 kN > 169 kN

$\Rightarrow$ bearing of connecting part is satisfactory.

Check tension capacity of angle

The bolt hole is 22 mm diameter for 20 mm diameter bolts. Design strength from Table 3.2 of HKC p_y = 355 N/mm^2 for thickness less than 16 mm.
a_{g1} = gross area of connected leg = 76 x 8 = 608 mm^2
a_{g2} = gross area of un-connected leg = 56 x 8 = 448 mm^2
a_g = gross area of the section = $a_{g1} + a_{g2}$ = 608 + 448 = 1056 mm^2

a_{n1} = net area of connected leg = (76 – 22) x 8 = 432 mm^2
a_{n2} = net area of un-connected leg = 448 mm^2

a_{e1} = effective area of connected leg = $K_e\, a_{n1}$ = 1.1 x 432 = 475.2 mm^2
< gross area = 608 mm^2
a_{e2} = effective area of un-connected leg = $K_e\, a_{n1}$ = 1.1 x 448 = 492.8 mm^2
> gross area = 448 mm^2
a_{e2} = 448 mm^2

A_e = effective area of the angle = $a_{e1} + a_{e2}$ = 475.2 + 448 = 923.2 mm^2
< gross area a_g = 1056 mm^2

For single angle connected through one leg using bolted connection,
Tension capacity $P_t = p_y\,(A_e - 0.5\, a_2)$
where $a_2 = A_g - a_1$ = gross area of unconnected leg = 448 mm^2

Hence, P_t = 355 x (923.2 – 0.5 x 448) x 10^{-3} = 248.2 kN > 169 kN

$\Rightarrow$ The angle is satisfactory.

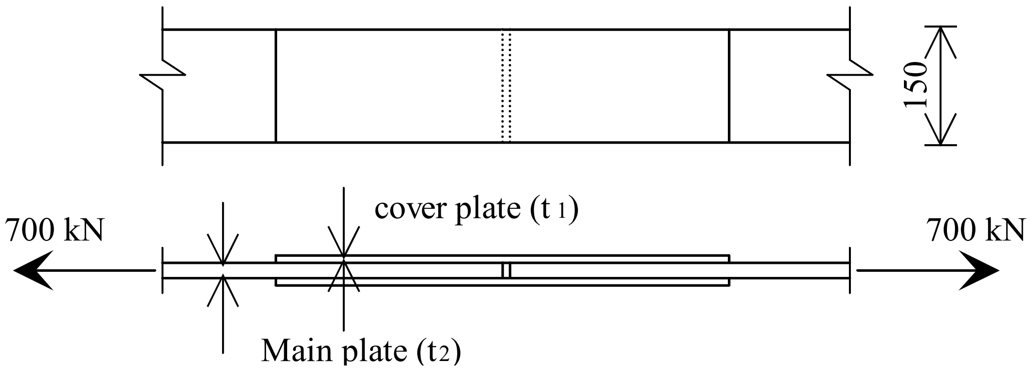
Example 2 - Tension Splice with Bolts

Design a splice to connect two main plates together as shown below, subjected to an ultimate tensile force of 700 kN, with cover plates at both sides of the main plates. Use bolted connection with M20 grade 8.8 bolts in 22 mm holes, with bolt spacing = 50 mm and end distance = 40 mm

Data: Grade S355 steel

Cover plate thickness t_1 = 12 mm

Main plate thickness t_2 = 20 mm



Solution

Adopting bolted connection with M20 grade 8.8 bolts in 22 mm holes

From Table 9.5 & 9.6, p_s = 375 N/mm^2, p_{bb} = 1000 N/mm^2, and p_{bs} = 550 N/mm^2

Bolt capacity:

Shear capacity of bolts:

Double shear capacity on threads

$P_s = 2p_sA_s$ = 2*375*245*10^{-3}= 2*91.8 = 183.8 kN per bolt

Comparing main plate thickness 20 mm and TWO cover plate thickness 2 x 12 = 24 mm, t_p = 20 mm

Bolt: Bearing capacity of bolts

$P_{bb} = p_{bb}*d*t_p$ = 1000*20*20*10^{-3} = 400 kN per bolt

Plate: Bearing capacity

$P_{bs} = k_{bs}\, d\, t_p\, p_{bs}$

$P_{bs} = 0.5\, k_{bs}\, e\, t_p\, p_{bs}$

and $P_{bs} = 1.5\, I_c\, t_p\, U_s \leq 2.0\, d\, t_p\, U_b$

Now, for Grade S355, p_{bs} = 550 N/mm^2 and U_s = 510 N/mm^2,
for standard hole, k_{bs} = 1.0, d = 20 mm, t_p = 20 mm, e = 40 mm
Net distance between the bearing edge and the near edge of adjacent holes in the direction of load transfer I_c = 50 – 22 = 28 mm
For grade 8.8, U_b = 800 N/mm^2, therefore,

$$P_{bs} = k_{bs}\, d\, t_p\, p_{bs} = 1.0 \times 20 \times 20 \times 550 \times 10^{-3} = 220 \text{ kN}$$

$$P_{bs} = 0.5\, k_{bs}\, e\, t_p\, p_{bs} = 0.5 \times 1.0 \times 40 \times 20 \times 550 \times 10^{-3} = 220 \text{ kN}$$

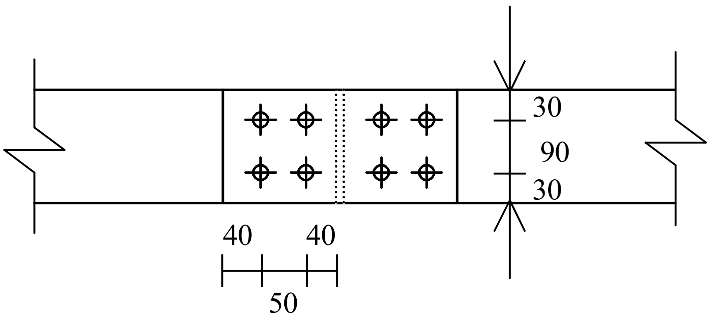
& $$P_{bs} = 1.5\, I_c\, t_p\, U_s \le 2.0\, d\, t_p\, U_b = 1.5 \times 28 \times 20 \times 510 \times 10^{-3} = 428.4 \text{ kN}$$
$$\le 2.0 \times 20 \times 20 \times 800 \times 10^{-3} = 640 \text{ kN}$$

Hence bearing capacity of connecting part = 220 kN per bolt

Overall bolt capacity

Overall bolt capacity = 183.8 per bolt (double shear capacity governs)
No. of bolt required = 700 / 183.8 = 3.8
$\Rightarrow$ Use 4 nos. 20 mm dia. bolts

Bolt layout is shown below.



Check bolt layout:
Min. spacing = 2.5d = 2.5*20 = 50 mm.
Max spacing = lesser of 12 t (= 12 x 12 = 144 mm) or 150 mm
Spacing provided = 50 mm $\Rightarrow$ O.K.
Spacing perpendicular to direction of load = 90 mm > 3 x bolt diameter

Min. end and edge distance, rolled edge = 26 mm (Table 9.3)
Edge distance provided = 30 mm $\Rightarrow$ O.K.
End distance provided = 40 mm, $\Rightarrow$ O.K.

Max. edge distance $= 11t_1\varepsilon = 11*12*\sqrt{\frac{275}{355}} = 116 \text{ mm} > 30 \text{ mm}$

$\Rightarrow$ O.K.

$= 11t_2\varepsilon = 11*20*\sqrt{\frac{275}{345}} = 196 \text{ mm} > 30 \text{ mm}$

$\Rightarrow$ O.K.

<u>Check Strength of Plates</u>

Main plates:
Gross area = 150*20 = 3000 mm^2
Design strength $\boldsymbol{p_y}$ = 345 N/mm^2 for 16 mm < $\boldsymbol{t}$ ≤ 40 mm
Effective area $\boldsymbol{A_e}$ = $\boldsymbol{K_e}$*net area = ***1.1****20*(150 – 2*22) = 2332 mm^2
≤ gross area = 3000 mm^2

Capacity $\boldsymbol{P_t}$ = $\boldsymbol{A_e}$*$\boldsymbol{p_y}$ = 2332*345*10^{-3} = 804.5 kN > 700 kN ⇒ O.K.

Each cover plate:
Gross area = 150*12 = 1800 mm^2
Design strength $\boldsymbol{p_y}$ = 355 N/mm^2 for $\boldsymbol{t}$ < 16 mm
Effective area $\boldsymbol{A_e}$ = $\boldsymbol{K_e}$*net area = ***1.1****12*(150 – 2*22) = 1399 mm^2
≤ gross area = 1800 mm^2

Capacity $\boldsymbol{P_t}$ = $\boldsymbol{A_e}$*$\boldsymbol{p_y}$ = 1399*355*10^{-3} = 496.6 kN
Therefore the capacity of TWO cover plates = 2*496.6 = 993.2 kN
> 700 kN ⇒ O.K.

1.6 Eccentric connections

There are two principal types of eccentrically loaded connections:
(1) Bolt group in direct shear and torsion; and
(2) Bolt group in direct shear and tension.
These connections are shown in Figure 9.

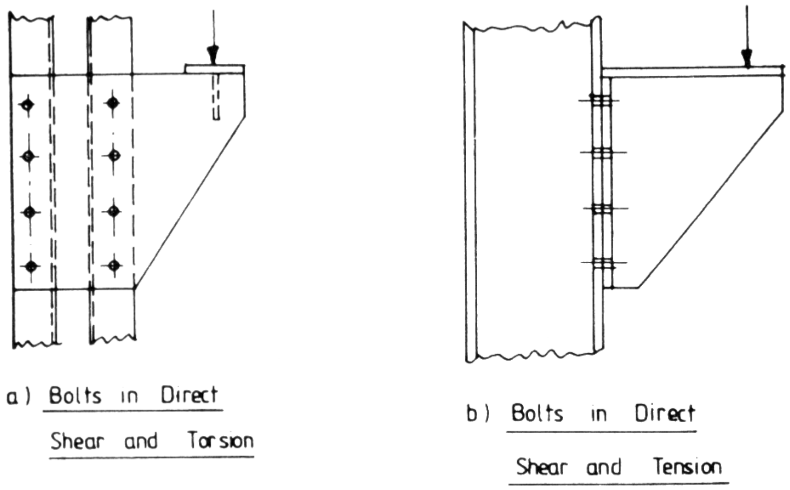


Figure 9 – Eccentrically Loaded Connections (extracted from ref. 6)

1.6.1 Bolts in direct shear and torsion

In the connection shown in Figure 9a the moment is applied in the plane of the connection and the bolt group rotates about its center of gravity. A linear variation of loading due to moment is assumed, with the bolt farthest from the center of gravity of the group carrying the greatest load. The direct shear is divided equally between the bolts and the side plates are assumed to be rigid.

Consider the group of bolts shown in Figure 10a, where the load P is applied at an eccentricity e. The bolts A, B, etc. are at distances r_1, r_2, etc. from the centroid of the group. The coordinates of each bolt are (x_1, y_1), (x_2, y_2), etc. Let the force due to the moment on bolt A be F_T. This is the force on the bolt farthest from the center of rotation. Then the force on a bolt r_2 from the center of rotation is $F_T\, r_2/r_1$ and so on for all the other bolts in the group. The moment of resistance of the bolt group is given by Figure 10:

$$M_R = F_T.r_1 + F_T.r_2.r_2 / r_1 + \ldots\ldots\ldots$$
$$= (F_T/r_1)(r_1^2 + r_2^2 + \ldots\ldots.)$$
$$= (F_T/r_1)\sum r^2$$
$$=(F_T/r_1)(\sum x^2 + \sum y^2)$$
$$= \text{applied moment} = P.\, e$$

The load F_T due to moment on the maximum loaded bolt A is given by

$$F_T = P\,e\,r_1 / (\sum x^2 + \sum y^2)$$

The load F_s due to direct shear is given by

$$F_s = P / (\text{No. of bolts})$$

The resultant load F_R on bolt A can be found graphically, as shown in Figure 10b. The algebraic formula can be derived by referring to Figure 10c.

Resolve the load F_T vertically and horizontally to give

Vertical load on bolt $A = F_s + F_T \cos\varphi$

Horizontal load on bolt $A = F_T \sin\varphi$

Resultant load on bolt A

$$F_R = [(F_T \sin\varphi)^2 + (F_s + F_T \cos\varphi)^2]^{0.5}$$
$$= [F_s^2 + F_T^2 + 2F_s F_T \cos\varphi]^{0.5}$$

The size of bolt required can then be determined from the maximum load on the bolt.

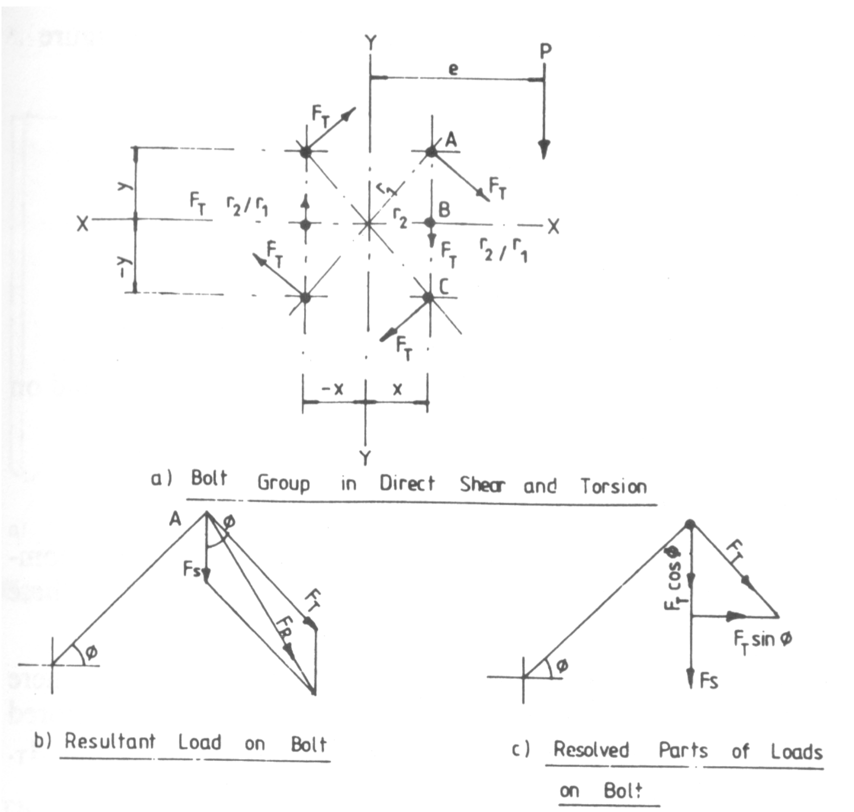


Figure 10 – Bolt Group in Direct Shear and Torsion
(extracted from ref. 6)

1.6.2 Bolts in direct shear and tension (clause 9.3.8)

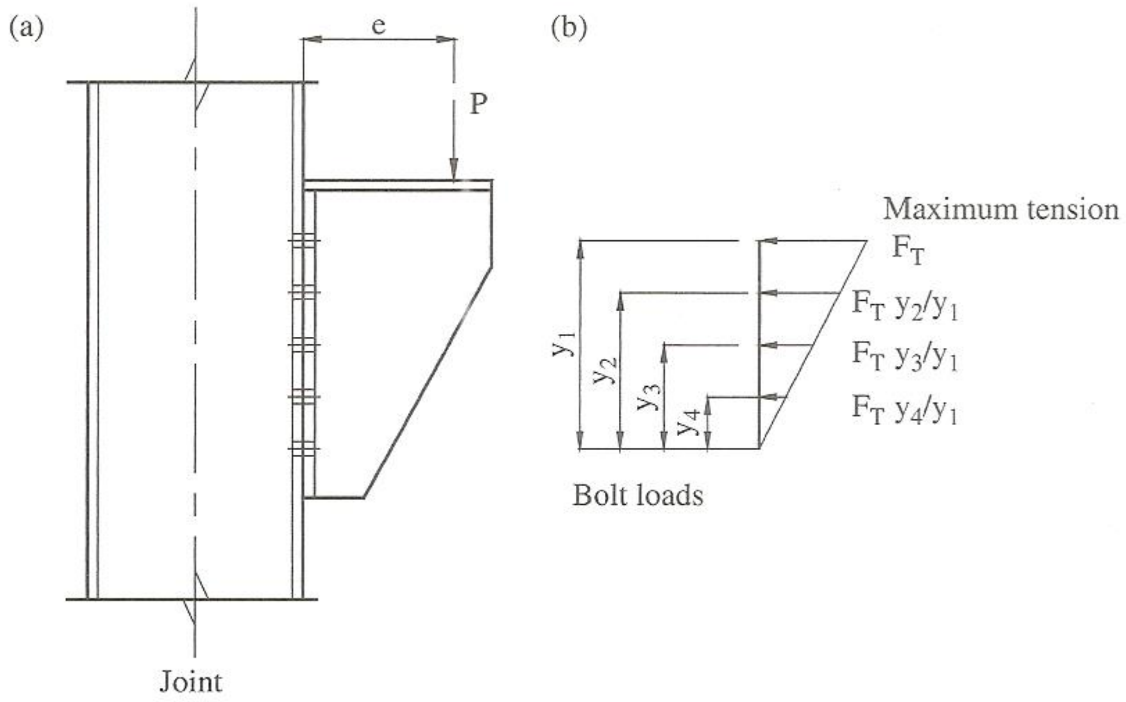


Figure 11 – Bolts in Direct Shear and Tension (Approx. Method) (extracted from ref. 2)

In the bracket type connection shown above, the bolts are in combined shear and tension. The factored applied shear F_s must not exceed the shear capacity P_s, where $P_s = p_s A_s$. The bearing capacity checks must also be satisfactory. The factored applied tension F_T must not exceed the tension capacity P_{nom}, where $P_{nom} = 0.8\, p_t A_s$. In addition, the following relationship must be satisfied.

<u>Ordinary bolts without prying forces</u>

$$\frac{F_s}{P_s} + \frac{F_T}{P_{nom}} \le 1.4$$

An approximate method of analysis that gives conservative results is described below. As shown above, a bracket is subjected to a factored load P at an eccentricity e. The center of rotation is assumed to be at the bottom bolt in the group. The loads vary linearly as shown in Figure 11, with the maximum load F_T in the top bolt.

The moment of resistance of the bolt group is

$$\boldsymbol{M_R = 2\,[\,F_T\,y_1 + F_T\,y_2^2\,/y_1 + \ldots\quad]}$$
$$\boldsymbol{= 2\,F_T\,/y_1\,[y_1^2 + y_2^2 + \ldots\quad]}$$
$$= \frac{2\,F_T}{y_1}\sum y^2$$
$$= \boldsymbol{P\,e}$$

The maximum bolt tension is: $\boldsymbol{F_T} = \dfrac{P\,e\,y_1}{2\sum y^2}$

The vertical shear per bolt: $\boldsymbol{F_S = P/\ \text{No. of bolts}}$

A bolt size is assumed and checked for combined shear and tension as described above.

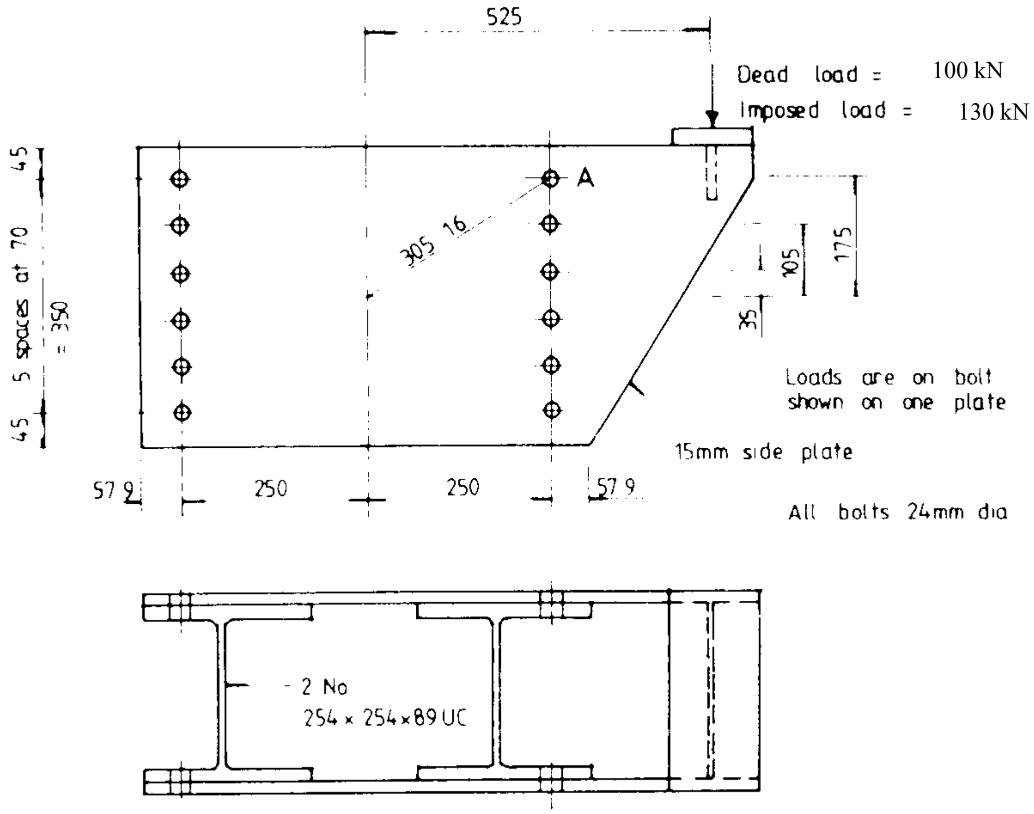
1.6.3 Long joints and large grip lengths

When the joint length of a splice or end connection in a compression or tension element containing more than two bolts exceeds 500 mm, the shear capacity should be reduced for ordinary and HSFG bolts. (See clause 9.3.6.1.4)

A reduction factor is given for large grip lengths (i.e. the total thickness of the connected plies exceeds five times the nominal diameter of the bolts) for ordinary bolts in clause 9.3.6.1.5.

Example 3 - Bolted bracket subjected to direct shear and torsion[6]

Check that the joint shown below is adequate. All data required are given in the figure. All bolts: grade 8.8, all steel: grade S355.



Solution

Factored Load $\boldsymbol{P}$ = (1.4*100 + 1.6*130) = 348 kN
Moment $\boldsymbol{M}$ = 348 * 0.525 = 182.7 kNm

Bolt group

$$\sum x^2 = 12*250^2 = 750*10^3$$
$$\sum y^2 = 4*\left(35^2 + 105^2 + 175^2\right) = 171.5*10^3$$
$$\sum x^2 + \sum y^2 = (750+171.5)*10^3 = 921.5*10^3$$

cos φ = 250 / 305.16 = 0.819

Bolt $\boldsymbol{A}$ is the bolt with the maximum load:
Load due to moment

$$F_T = \frac{Pe \cdot r_1}{(\sum x^2 + \sum y^2)} = \frac{348*0.525*10^6*305.16}{921.5*10^3} = 60.5\text{ kN}$$

Load due to shear

$$F_s = \frac{P}{\text{No. of bolts}} = \frac{348}{12} = 29\text{ kN}$$

Resultant load on bolt

$$F_R = \sqrt{F_s^{\ 2} + F_T^{\ 2} + 2F_sF_T\cos\varphi}$$
$$= \sqrt{29^2 + 60.5^2 + 2*29*60.5*0.819} = 85.9\text{ kN}$$

Single shear of 24 mm dia. Bolt on threads,

$\boldsymbol{P_s = p_s*A_s}$ = 375*353*10^{-3} = 132.4 kN > 85.9 kN ⇒ O.K.

Universal column flange thickness = 17.3 mm
Side plate thickness = 15 mm (side plate controls), $\boldsymbol{t_p}$ = 15 mm
Min. end distance = 45 mm

Bearing capacity of the bolt,

$\boldsymbol{P_{bb} = p_{bb}*d*t_p}$ = 1000*24*15*10^{-3} = 360 kN > 85.9 kN ⇒ O.K.

Bearing capacity of the plate,

$\boldsymbol{P_{bs} = k_{bs}\, d\, t_p\, p_{bs}}$
$\boldsymbol{P_{bs} = 0.5\, k_{bs}\, e\, t_p\, p_{bs}}$
and $\boldsymbol{P_{bs} = 1.5\, I_c\, t_p\, U_s \le 2.0\, d\, t_p\, U_b}$

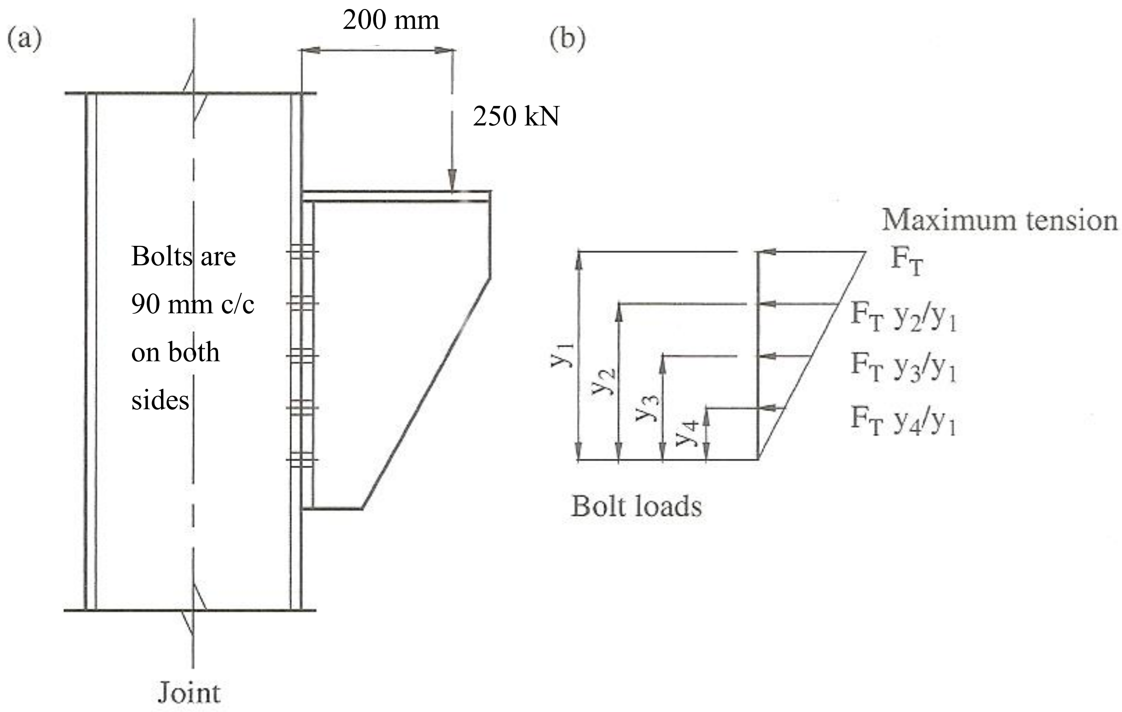
Now, for Grade S355, $\boldsymbol{p_{bs}}$ = 550 N/mm^2 and $\boldsymbol{U_s}$ = 510 N/mm^2,
for standard hole, $\boldsymbol{k_{bs}}$ = 1.0, $\boldsymbol{d}$ = 24 mm, $\boldsymbol{t_p}$ = 15 mm, $\boldsymbol{e}$ = 45 mm
Net distance between the bearing edge and the near edge of adjacent holes in the direction of load transfer $\boldsymbol{I_c}$ = 70 – 26 = 44 mm
For grade 8.8, $\boldsymbol{U_b}$ = 800 N/mm^2, therefore,

$\boldsymbol{P_{bs} = k_{bs}\, d\, t_p\ \ p_{bs}}$ = 1.0 x 24 x 15 x 550 x 10^{-3} = 198 kN
$\boldsymbol{P_{bs} = 0.5\, k_{bs}\, e\, t_p\, p_{bs}}$ = 0.5 x 1.0 x 45 x 15 x 550 x 10^{-3} = 186 kN
& $\boldsymbol{P_{bs} = 1.5\, I_c\, t_p\, U_s \le 2.0\, d\, t_p\, U_b}$ = 1.5 x 44 x 15 x 510 x 10^{-3} = 504.9 kN
≤ 2.0 x 24 x 15 x 800 x 10^{-3} = 576 kN
Hence bearing capacity of connecting part = 186 kN > 85.9 kN

The strength of the joint is controlled by the single shear of the bolt.
Overall joint capacity = single shear capacity of the bolt
= 132.4 kN > 85.9 kN
The joint is satisfactory.

Example 4 - Bolted bracket subjected to direct shear and tension[2]

A bolted bracket connection subjected to an ultimate load of 250 kN is shown below. Check the adequacy of the bolts. All bolts: M16, grade 8.8, all steel: grade S355.



Solution

Conditions to be satisfied:

$$F_s \le P_s, \qquad F_T \le P_{nom}, \qquad \frac{F_s}{P_s} + \frac{F_T}{P_{nom}} \le 1.4$$

Single shear capacity on threads,
$\boldsymbol{P_s = p_s A_s}$ = 375*157*10^{-3} = 58.9 kN

Tension capacity,
$\boldsymbol{P_{nom} = 0.8\, p_t A_s}$ = 0.8*560*157*10^{-3} = 70.3 kN
Use approximate method of analysis for conservative results.

$$\sum y^2 = 2*\left(90^2 + 180^2 + 270^2 + 360^2\right) = 2*243*10^3 = 486*10^3 \text{ mm}^2$$

The maximum bolt tension is at bolts $\boldsymbol{y_1}$ from the lowest bolts:

$$F_T = \frac{Pe \cdot y_1}{\sum y^2} = \frac{250*200*360}{486*10^3} = 37.0\ \text{kN} < P_{nom} = 70.3\ \text{kN}$$

The vertical shear per bolt:

$$F_s = \frac{P}{\text{No. of bolts}} = \frac{250}{10} = 25\ \text{kN} < \text{P}_\text{s} = 58.9\ \text{kN}$$

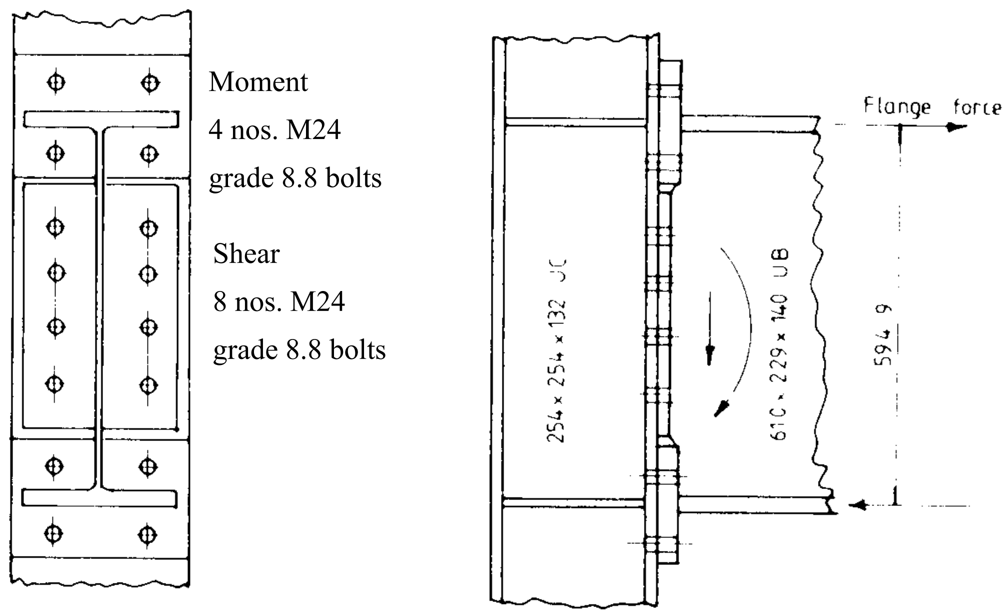
$$\frac{F_s}{P_s} + \frac{F_T}{P_{nom}} = \frac{25}{58.9} + \frac{37}{70.3} = 0.424 + 0.526 = 0.95 \leq 1.4$$

The bolts are satisfactory.

Example 5 - Moment Connection[6]

Design the moment and shear connections between the floor beam and column in a steel frame building as shown below using M24 grade 8.8 bolts. The following data are given:

Floor beam	610*229*140 UB (Grade S355)			
Column	254*254*132 UC (Grade S355)			
Moment due to:		Dead Load	=	160 kNm
		Imposed Load	=	90 kNm
Shear due to:		Dead Load	=	250 kN
		Imposed Load	=	160 kN



Solution

<u>Moment Connection:</u>
The moment is assumed to be taken by the flange bolts in tension.

Factored moment = 1.4*160 + 1.6*90 = 368 kNm
Flange force = 368/0.595 = 618.5 kN

Provide 24 mm dia. Grade 8.8 bolts
Tensile stress area = 353 mm^2 and $\boldsymbol{p_t}$ = 560 N/mm^2

Tension capacity of four bolts $P_{nom} = 4*0.8*p_t*A_s = 4*0.8*560*353*10^{-3}$

$= 632.6$ kN

> 618.5 kN $\Rightarrow$ O.K.

The joint is satisfactory for moment. Four bolts are also provided at the bottom of the joint but these are not loaded by the moment in the direction shown.

Shear Connection:

The shear is resisted by the web bolts.

Factored shear = 1.4*250 + 1.6*160 = 606 kN

Shear capacity of eight no. 24 mm diameter bolts,

$\boldsymbol{P_s = 8*p_s*A_s} = 8*375*353*10^{-3} = 8*132.4 = 1059.2$ kN

Bearing resistance:

Provide end plate = 12 mm thick and end distance of the two top bolts = 55mm and bolt spacing = 70 mm

$\boldsymbol{P_{bb} = p_{bb}*d*t} = 1000*24*12 = 288$ kN

$\boldsymbol{P_{bs} = k_{bs}\, d\, t_p \quad p_{bs}} = 1.0 \times 24 \times 12 \times 550 \times 10^{-3} = 158.4$ kN

$\boldsymbol{P_{bs} = 0.5\, k_{bs}\, e\, t_p\, p_{bs}} \quad = 0.5 \times 1.0 \times 55 \times 12 \times 550 \times 10^{-3} = 181.5$ kN

& $\boldsymbol{P_{bs} = 1.5\, l_c\, t_p\, U_s} \quad \leq \quad \boldsymbol{2.0\, d\, t_p\, U_b} = 1.5 \times 44 \times 12 \times 510 \times 10^{-3} = 403.9$ kN

$\leq \quad 2.0 \times 24 \times 12 \times 800 \times 10^{-3} = 460.8$ kN

Hence bearing capacity of connecting part = 158.4 kN > 132.4 kN

$\Rightarrow$ (Shear capacity controls)

$\therefore$ Shear capacity of the web bolts = 1059.2 > 606 kN

Therefore 8 nos. 24 mm diameter bolts are satisfactory.

Note that only the bolts have been designed. The welds, end plates and stiffeners must be designed and the column flange and web checked.

1.7 Design of fillet Welds

The strength of a fillet weld is calculated using the throat size **a**. For a 90° fillet weld, the throat size is taken as 0.7 x leg length **s**. The design strength for fillet welds p_w is given in Table 9.2a of the Code and is shown below:

Table 9.2a - Design strength of fillet welds p_w for BS-EN Standards

Steel grade	Electrode classification 35 N/mm²	42 N/mm²	50 N/mm²	For other types of electrode and/or steel grades:
				$p_w = 0.5U_e$ but $p_w \leq 0.55\ U_s$ where
S 275	220	(220)[a]	(220)[a]	U_e is the minimum tensile strength of the electrode specified in the relevant product standard;
S 355	(220)[b]	250	(250)[a]	U_s is the specified minimum tensile strength of the parent metal.
S 460	(220)[b]	(250)[b]	280	

a) Over-matching electrodes.
b) Under-matching electrodes.

Table 10 – Extract of Table 9.2a of the HK Code[1]

Hence the strength of weld = 0.7 leg length x p_w x 10^{-3} kN/mm

Example

The strength of 6 mm fillet weld using Class 42 electrode on Grade S355 steel
= 0.7 x 6 x 250 x 10^{-3} = 1.05 kN/mm

Values of the strength of fillet weld are often tabulated for easy reference.

Table 11 – Strength of Fillet Welds (kN/mm)

Weld size or leg length (mm)	Steel Grade and Electrodes Strength BS EN499		
	Gr. S275 Class 35	**Gr. S355, Class 42, 50**	Gr. S460 Class 50
4	0.62	***0.70***	0.78
5	0.77	***0.88***	0.98
6	0.92	***1.05***	1.18
8	1.23	***1.40***	1.57
10	1.54	***1.75***	1.96
12	1.85	***2.10***	2.35

1.7.1 Capacity of fillet weld

The capacity of a fillet weld should be calculated using the throat size ***a*** and the following methods:

(1) Simplified method

Stresses should be calculated from the vector sum of forces from all directions divided by the weld throat area to ensure that it does not exceed the design strength of weld p_w.

(2) Directional method

For more accurate behaviour, the force per unit length should be resolved into a longitudinal shear F_L parallel to the axis of the weld and a resultant transverse force F_T perpendicular to this axis. The corresponding capacities per unit length are

$P_L = p_w\, a$ (longitudinal direction)

$P_T = K\, P_L$ (transverse direction)

where $K = 1.25\sqrt{\dfrac{1.5}{1+\cos^2\theta}}$ and θ is the angle between the force and the throat of weld. (see Fig. 9.4)

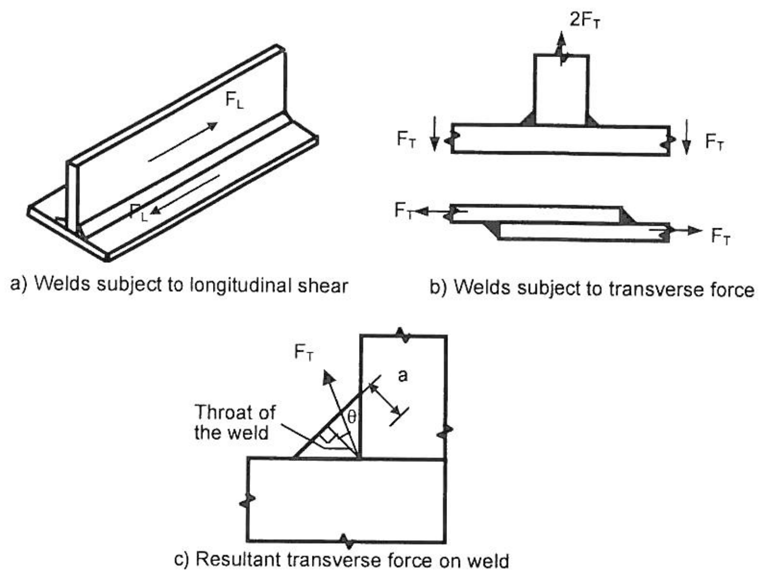


Figure 9.4 - Fillet welds – directional method

Figure 12 – Extract of Figure 9.4 of the HK Code[1]

The stress resultant should satisfy the following relationship

$$\left(\frac{F_L}{P_L}\right)^2 + \left(\frac{F_T}{P_T}\right)^2 \le 1$$

Important provisions regarding fillet welds are set out below:

(1) The minimum leg length of a fillet weld should not be less than that specified in Table 9.1.

Table 9.1 - Minimum leg length of a fillet weld

Thickness of the thicker part (mm)	Minimum leg length (for unequal leg weld, the smaller leg length should be considered) (mm)
up to and including 6	3
7 to 13	5
14 to 19	6
over 19	8

Table 12 – Extract of Table 9.1 of the HK Code[1]

(2) For a weld along the edge of a plate of thickness less than 6 mm, the maximum leg length should be the thickness of the plate. For plate equal or thicker than 6 mm, the maximum leg length should be the thickness of the plate minus 2 mm.

(3) Effective length of a fillet weld should be its full length of weld less 2**s** with its end return excluded, but should not be less than 40mm. A fillet weld with effective length less than 4 **s** or 40mm should not be used to carry load.

(4) Fillet welds should be returned around corners for a length of less than 2 **s,** if a return is not practical, terminated not less than **s** from edges. Typical end returns details are shown in Figure 9.3 of the Code.

(5) In lap joints the minimum lap length should not be less than 5 t or 25 mm whichever is the greater, where t is the thickness of the thinner plate. For lap joints longer than 100 s, refer to clause 9.2.5.1.10 for details.

(6) In end connections the length of weld should not be less than the transverse spacing between the welds.

(7) Intermittent welds should not be used under fatigue conditions. The spacing between intermittent welds should not exceed 300 mm nor 16 t for parts in compression nor 24 t for parts in tension, where t is the thickness of the thinner parts.

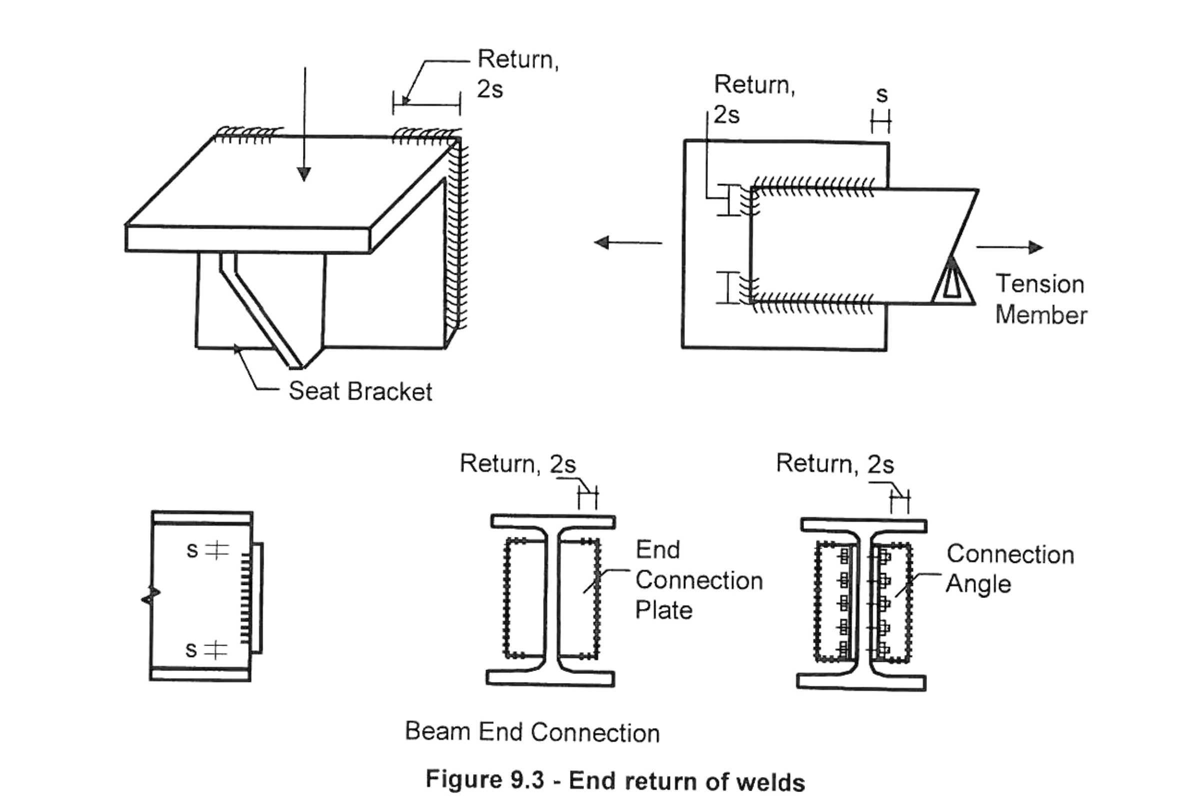


Figure 13 – Extract of Figure 9.3 of the HK Code[1]

1.8 Design of butt Welds

The design of butt welds is covered in clause 9.2.5.2 of the code. The design strength should be taken as equal to that of the parent metal provided that the strength of the weld metal is not less than that of the parent metal. Full penetration depth is ensured if the weld is made from both sides or if a backing run is made on a butt weld made from one side. Full penetration is also achieved by using a backing plate.

1.9 Eccentrically Loaded Welded Connections

In Figure 14a, the welded connection is subjected to direct shear and torsion. While the connection in Figure 14b, it is subjected to direct shear and tension.

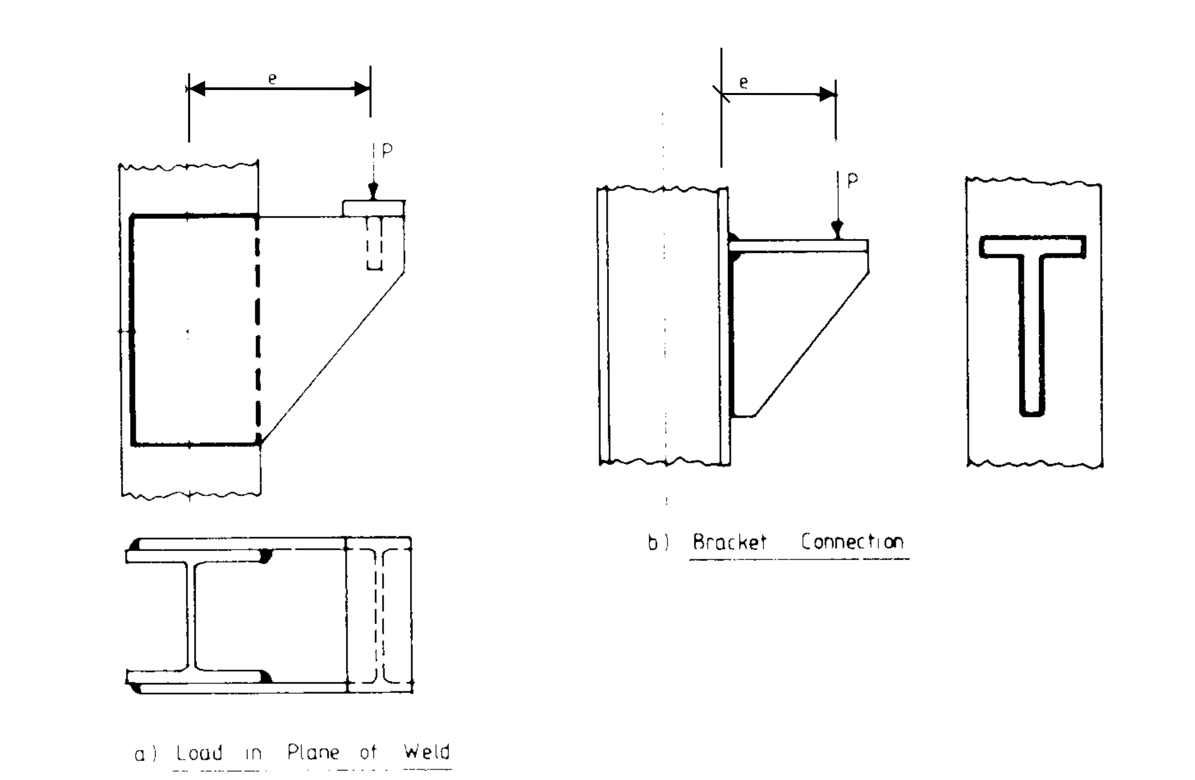


Figure 14 – Eccentrically Loaded Welded Connections (extracted from ref 6)

1.9.1 Torsion joint with load in plane of weld

The weld is in direct shear and torsion. The eccentric load causes rotation about the center of gravity of the weld group. The force in the weld due to torsion is taken to be directly proportional to the distance from the center of gravity and is found by a torsion formula.

The direct shear is assumed to be uniform throughout the weld.
The resultant shear is found by combining the shear due to moment and the direct shear. The side plate is assumed to be rigid.

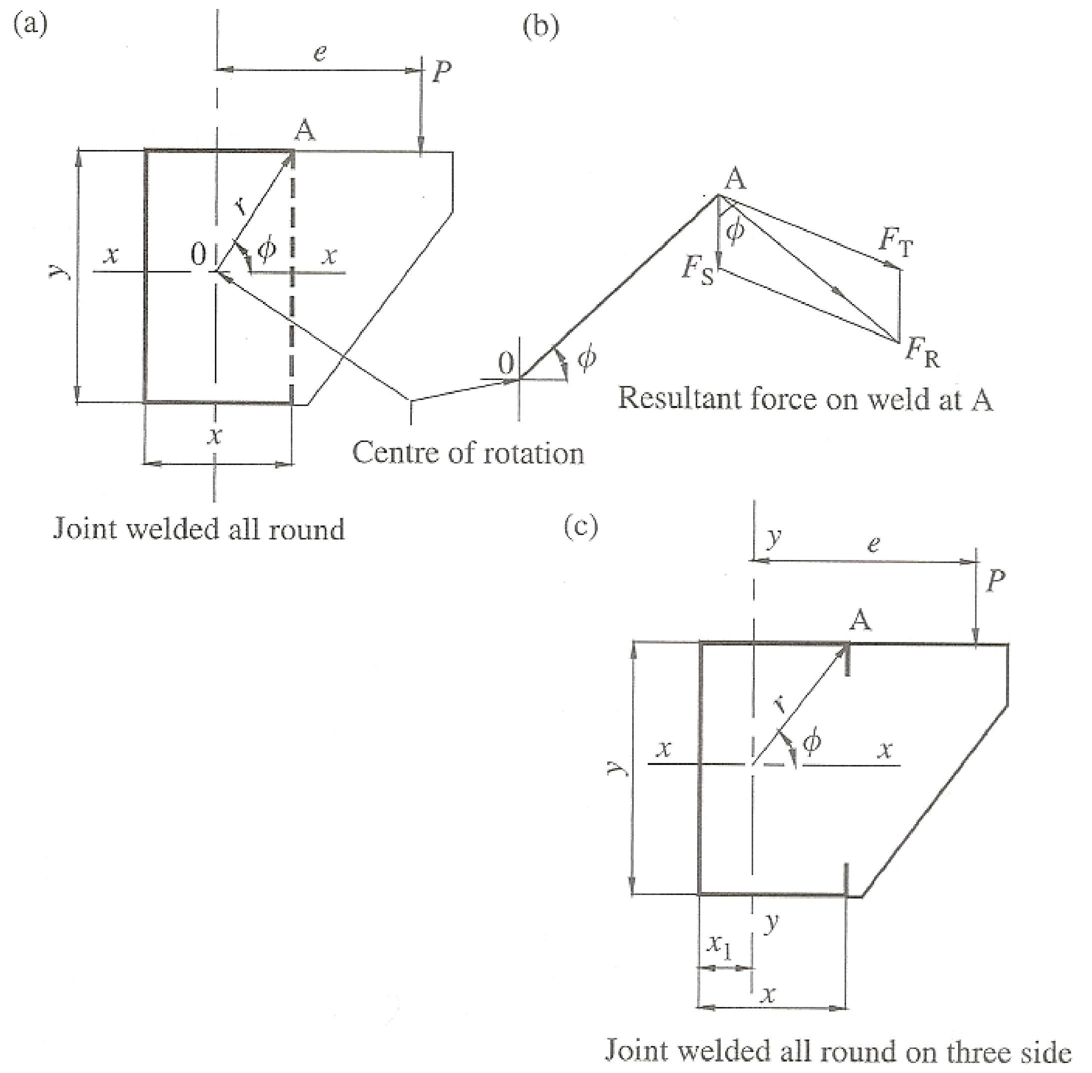


Figure 15 – Torsion Joints Load in Plane of Weld (extracted from ref. 2)

Weld is on four sides (see Figure 15a)

Refer to Figure 15a, the weld is of unit leg length throughout:

Direct shear $F_s = \dfrac{P}{\text{Length of weld}} = \dfrac{P}{2(x+y)}$

Consider the I_x and I_y of the weld group,

$$I_x = \frac{1*y^3}{12}\cdot 2 + (1*x)\left(\frac{y}{2}\right)^2 * 2 + \frac{x*1^3}{12}*2$$
$$= \frac{y^3}{6} + \frac{xy^2}{2} + \frac{x}{6} \approx \frac{y^3}{6} + \frac{xy^2}{2}$$

Similarly $I_y = \frac{x^3}{6} + \frac{x^2 y}{2} + \frac{y}{6} \approx \frac{x^3}{6} + \frac{x^2 y}{2}$

Shear due to torsion, $F_T = \frac{Per}{I_p} = \frac{Per}{(I_x + I_y)}$ and $r = \sqrt{\left(\frac{x}{2}\right)^2 + \left(\frac{y}{2}\right)^2}$

The heaviest loaded length of weld is that at ***A***, farthest from the center of rotation ***O***.

The resultant shear on a unit length of weld at ***A*** is:

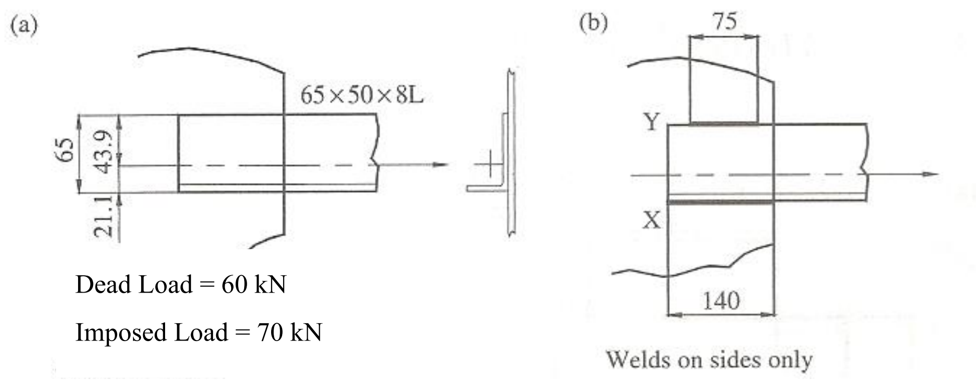
$$F_R = \sqrt{F_s^{\,2} + F_T^{\,2} + 2F_s F_T \cos\varphi}$$

The weld size can be calculated from:

Required leg length or weld size = $\dfrac{\text{Required strength of weld} * 10^3}{0.7 p_w}$

Example 6 - Welded Connection[2]

Design the fillet weld for the direct shear connection for the angle loaded as shown below. The load is assumed to act through the centroidal axis of the angle (Steel grade ***S355*** and class ***42*** electrode).



Solution

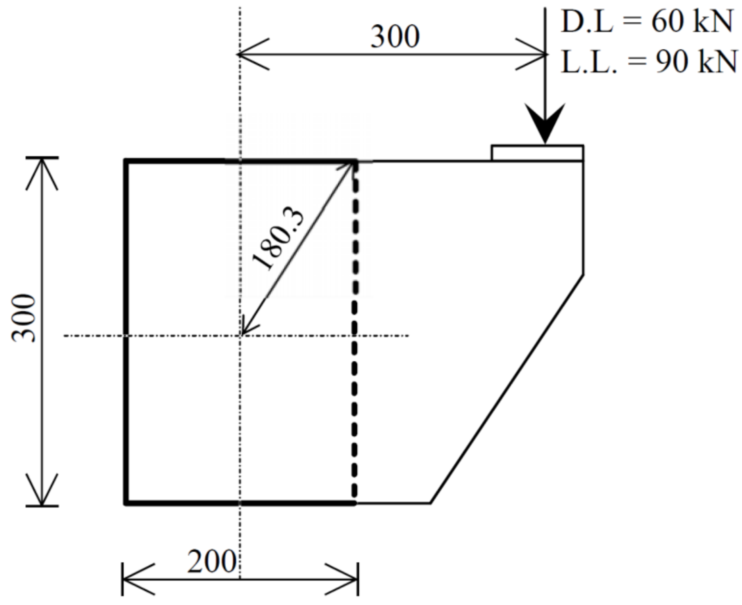
Factored load = (1.4 x 60) + (1.6 x 70) = 196 kN
Try using 6 mm fillet weld, strength = 1.05 kN/mm
Length required = 196/1.05 = 186.7 mm

The layout of the fillet weld can be arranged as shown in (b), the weld on each side should be balanced according to the centroidal position.
Side X, length = 186.7x 43.9/65 = 126 mm
Add 12 mm, final length = 138 mm, say 140 mm (> 40 mm)

Side Y, length = 186.7 – 126 mm = 60.7 mm
Add 12 mm, final length = 60.7 + 12 = 72.7 mm, say 75 mm (> 40 mm)
Greater than the transverse spacing of the welds = 65 mm

Example 7 - Torsion Connection with load in plane of weld

One side plate of an eccentrically loaded connection is shown below. The plate is welded on four sides. Find the maximum shear force in the weld and select a suitable fillet weld by calculation from first principle. The steel is Grade S355 and Class 42 electrode.



Solution

Length L = 2*(200 +300) = 1000 mm

$r = \sqrt{100^2 + 150^2} = 180.3\text{ mm}$

Eccentricity of load e = 300 mm

Moment of inertia:

$$I_x = \frac{300^3}{6} + \frac{200 * 300^2}{2} = 1.35*10^7\text{ mm}^3$$

$$I_y = \frac{200^3}{6} + \frac{200^2 * 300}{2} = 7.333*10^6\text{ mm}^3$$

$I_p = (13.5 + 7.333)*10^6 = 20.833*10^6\text{ mm}^3$

$$\cos\varphi = \frac{100}{180.3} = 0.555$$

Factored load = (1.4*60 + 1.6*90) = 228 kN

Direct shear F_s = 228/1000 = 0.228 kN/mm

Shear due to torsion on weld at corner:

$$F_T = \frac{228 * 300 * 180.3}{20.833 * 10^6} = 0.592 \text{ kN/mm}$$

Resultant shear:

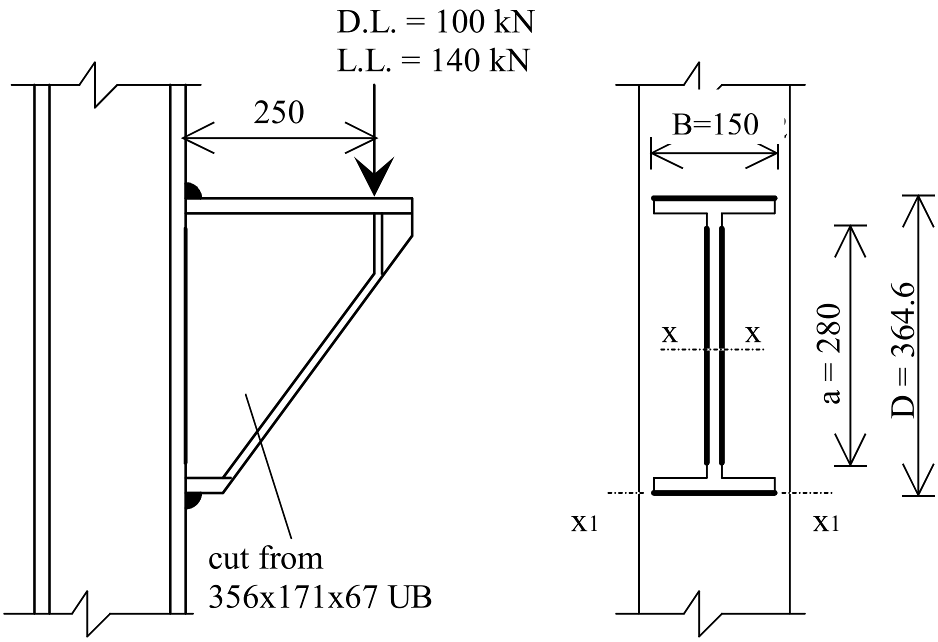
$$F_R = \sqrt{0.228^2 + 0.592^2 + 2 * 0.228 * 0.592 * 0.555} = 0.743 \text{ kN/mm}$$

Required leg length or weld size = required strength of weld*10^3/(0.7$\boldsymbol{p_w}$)
= 0.743*1000/(0.7*250) = 4.2 mm

Use 6 mm fillet weld

Example 8 - Bracket Connection

Determine the size of fillet weld required for the bracket connection shown below. The web welds are to be taken as one half the leg length of the flange welds. All dimensions and loads are shown in the figure. The steel is grade S355 and the class 42 electrode.



Solution

Design assuming rotation about X_1-X_1 axis.

The flange weld resists the moment

$$F_T = \frac{Pe}{BD} = \frac{(1.4*100+1.6*140)*250}{364.6*(150-2*12)} = 1.98 \text{ kN/mm}$$

Required leg length or weld size = (Required strength of weld)*10^3/(0.7p_w)

= 1.98*1000/(0.7*250) = 11.3 mm

Use 12 mm fillet weld for flanges

The web welds resist the shear:

$$F_s = \frac{P}{2a} = \frac{364}{2*(280-2*6)} = 0.68 \text{ kN/mm}$$

Required leg length or weld size = (Required strength of weld)*10^3/(0.7p_w)

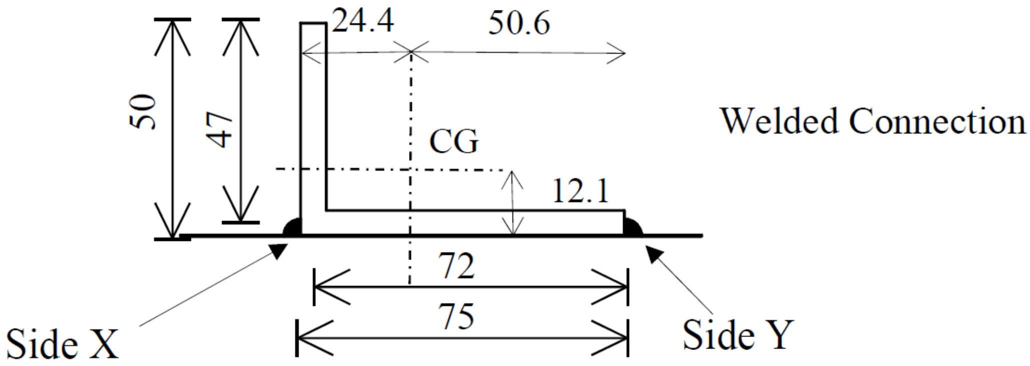
= 0.68*1000/(0.7*250) = 3.9 mm

Use 6 mm fillet weld for web

(the minimum size recommended)

Example 9 - Angle Connected through One Leg

Check if a Grade S355 unequal single angle (75 x 50 x 6) connection through the long leg as shown below is adequate to carry a dead load of 95 kN and an imposed load of 40 kN using welded connection. Design the weld size.



Solution

Factored load = 1.4 x 95 + 1.6 x 40 = 197 kN

a_1 = area of connected leg = 72 x 6 = 432 mm^2
a_2 = area of unconnected leg = 47 x 6 = 282 mm^2
A_e = 432 + 282 = 714 mm^2

For single angle connected through one leg using welded connection,

Tension capacity $P_t = p_y (A_e - 0.3 a_2)$
$= 355 (714 - 0.3 \times 282) \times 10^{-3} = 223.4$ kN.

$\Rightarrow$ The angle is satisfactory.

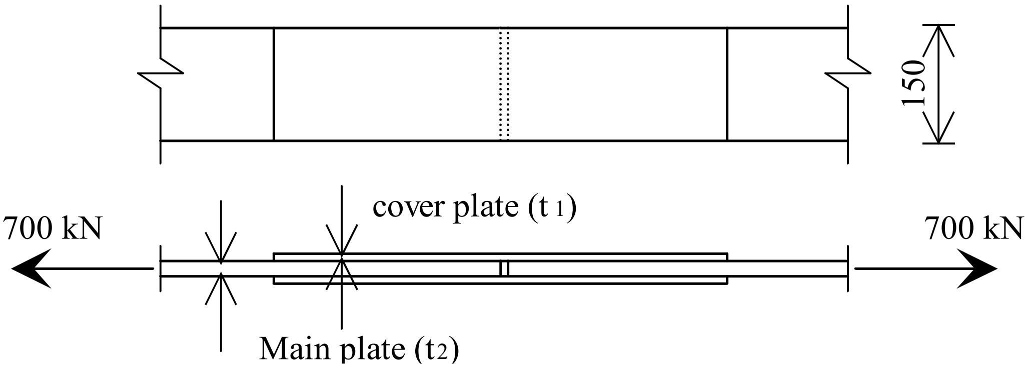
Try using 4 mm fillet weld, strength = 0.70 kN/mm
Length required = 197/0.70 = <u>282</u> mm
From section table, centroid is 24.4 mm from side X, the weld on each side should be balanced according to the centroidal position.
Side X, length = 282 x 50.6/75 = 190 mm
Add 8 mm, final length = 190 + 8 = 198 mm, say 200 mm

Side Y, length = 282 – 190 mm = 92 mm
Add 12 mm, final length = 92 + 8 = 100 mm, say 100 mm O.K.

Example 10 - Tension Splice with Welding

Design a splice to connect two main plates together as shown below, subjected to an ultimate tensile force of 700 kN, with cover plates at both sides of the main plates using welded connection. (class 42 Electrode)

Data: Grade S355 steel
Cover plate thickness t_1 = 12 mm
Main plate thickness t_2 = 20 mm



Solution

In order to accommodate the welds on the flat surface of the main plate, it is necessary to use a cover plate of less than 150 mm. Since its full cross section will be effective, a 100*12 mm steel plate on each side as cover plates are used.

Welding strength:
Use 6 mm fillet weld,
Strength of weld = 0.7*leg length*p_w / 1000 kN/mm
= 0.7*6*250/1000 = 1.05 kN/mm

Length required = 700 / 1.05 = 667 mm
Balance the weld on 4 sides as shown.

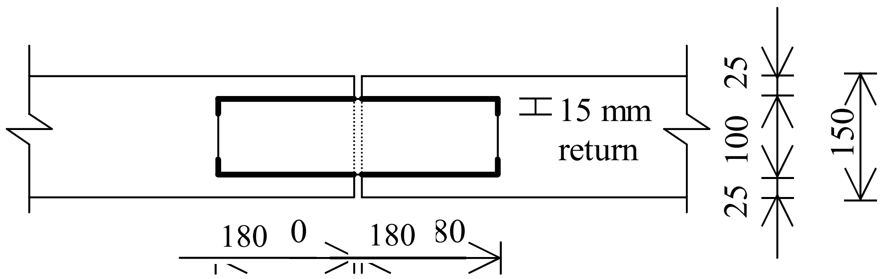


Figure 6.21

Length at each side = 667/4 = 167 mm

Use length = 167 + 2*6 = 179 mm (say 180mm)

Min. lap length = 5*12 = 60 mm < 180 mm $\Rightarrow$ O.K.
Min. length of return = 2*6 = 12 mm
Provide 15 mm return $\Rightarrow$ O.K.

Weld length on each side ≥ transverse spacing between welds = 100 mm
Provide 180 mm $\Rightarrow$ O.K.

Check Strength of Plates

Main plates:
Gross area = 150*20 = 3000 mm^2
Design strength p_y = 345 N/mm^2 for 16 mm < t ≤ 40 mm

Capacity $P_t = A_e * p_y$ = 3000*345 = 1035 kN > 700 kN $\Rightarrow$ O.K.

Each cover plate:
Gross area = 100*12 = 1200 mm^2
Design strength p_y = 355 N/mm^2 for t < 16 mm

Capacity $P_t = A_e * p_y$ = 1200*355 = 426 kN
Therefore the capacity of TWO cover plates = 2*426 = 852 kN
> 700 kN $\Rightarrow$ O.K.

| TUTORIAL 2 |

Q1. Figure Q1 shows a steel tension splice with an applied ultimate tensile force of 750 kN. Check the structural capacities against the applied load with respect to shear, bearing capacities, plate tensile capacity and bolt layout, for the bolted connection. There are 6 nos. of 24 mm diameter grade 8.8 bolts in 26 mm diameter holes on each end with threads in shear plane. Steel plates are grade S355.

Q2. Check the welded connection as shown in Figure Q2 subjected to an applied ultimate tensile force 850 kN. Fillet weld: class 42 electrode, steel: grade S355.

Q3. A bolted eccentric connection as shown in Figure Q3 is subjected to a vertical ultimate load of 200 kN. Determine the size of grade 8.8 bolts required if the load is applied at an eccentricity of 300 mm.

Q4. The bolted bracket connection shown in Figure Q4 carries a characteristic dead load of 200 kN and a characteristic imposed load of 150 kN placed at an eccentricity of 250 mm. Use approximate method to check that 12 nos. 20 mm diameter grade 8.8 bolts are adequate.

Q5. The welded connection for a tension member in a roof truss is shown in Figure Q5. Using class 42 electrode on grade S355 steel, design for the welding if the ultimate tension in the members is 350 kN. Check the adequacy of the angles.

Q6. Determine the leg length of fillet weld required for the eccentric joint shown in Figure Q6. The characteristic dead load and the characteristic imposed load are 350 kN and 300 kN respectively. The loads are placed at 200 mm from the face of column. (Steel: grade S355, class 42 electrode)

Q7. A bracket cut from a 533 x 210 UB 82 of grade S355 steel is welded to a column, as shown in Figure Q7. The ultimate vertical load on the bracket is 600 kN applied at an eccentricity of 250 mm. Design the welds between the bracket and column. Assuming that the bending moment is resisted by the welds on the flanges, and the shear is resisted by the welds on the web (class 42 electrode).

| TUTORIAL 2 |

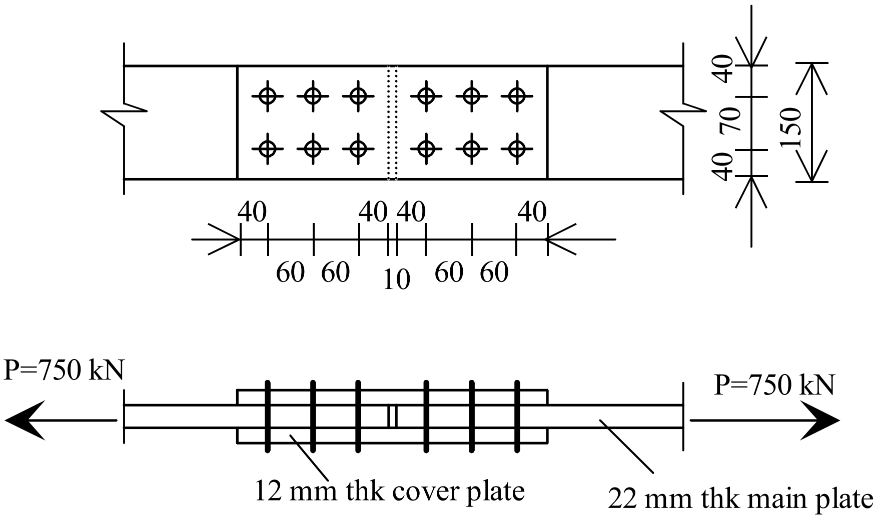


Figure Q1

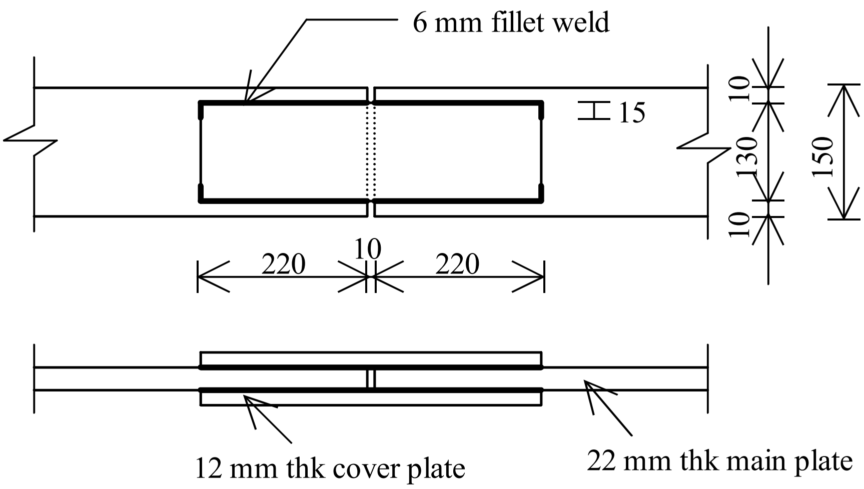


Figure Q2

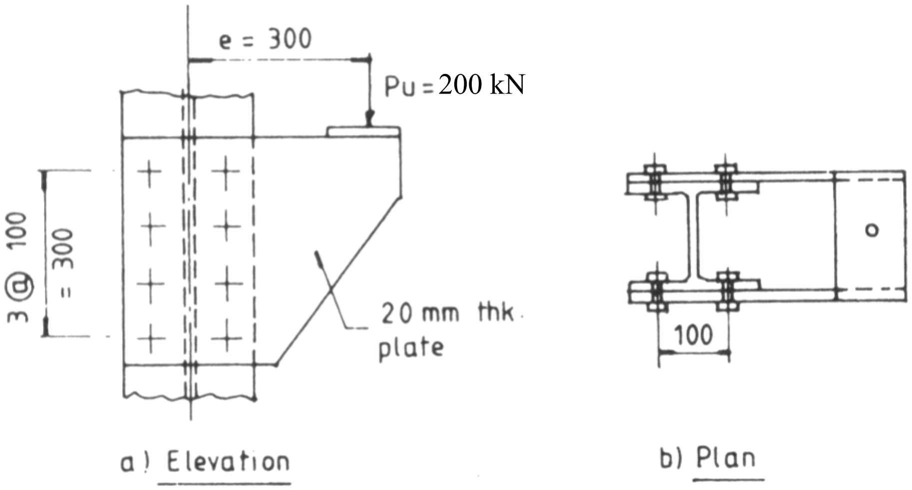


Figure Q3[6]

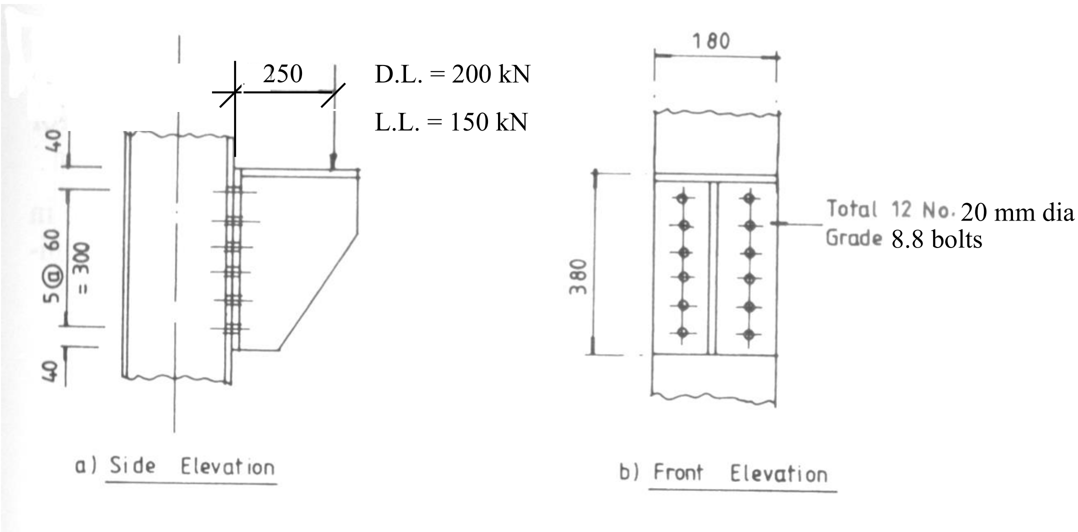


Figure Q4[6]

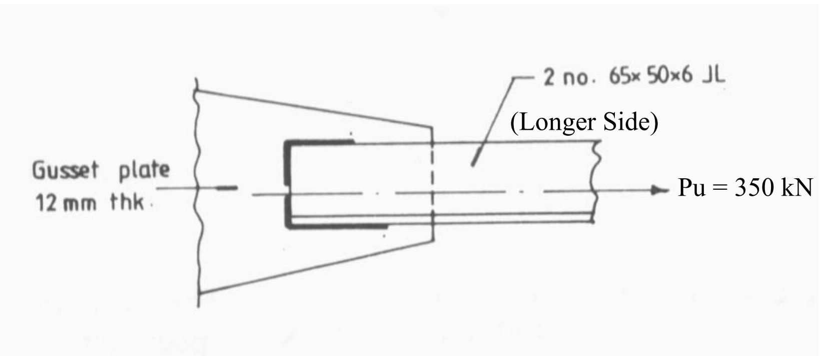


Figure Q5[6]

| TUTORIAL 2 |

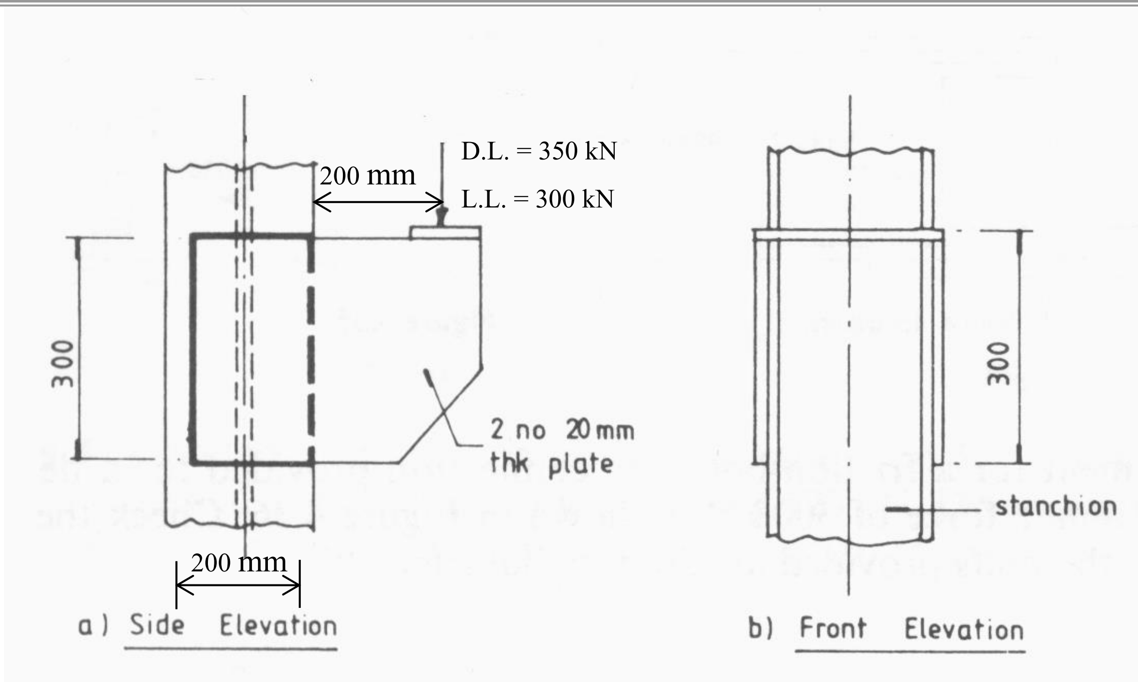


Figure Q6[6]

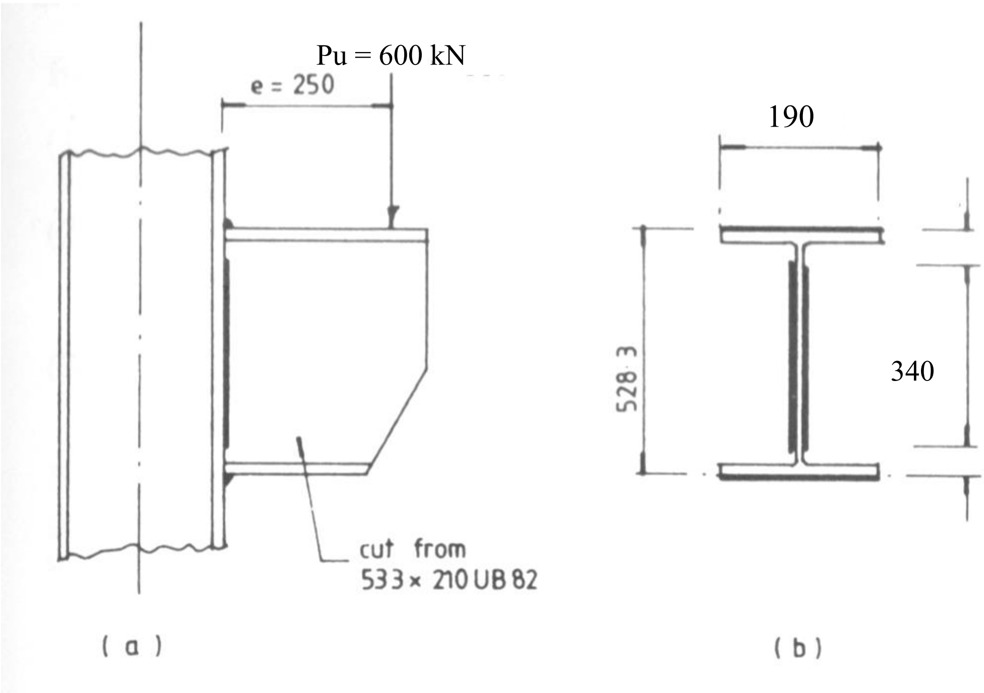


Figure Q7[6]

🕮 Revision

Read reference 2 on P.131 – 143, and on page 284 to 321.

🕮 Main Reference

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