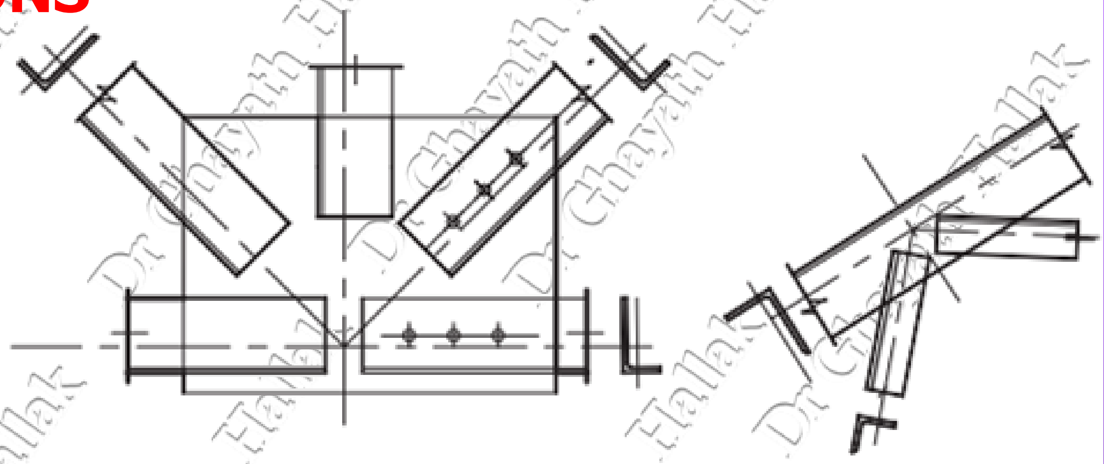
SIMPLE CONNECTIONS

Simple connections

TYPES OF CONNECTIONS

Connections are needed to join:

1. Members together in trusses and lattice girders



Simple connections

TYPES OF CONNECTIONS

2. Plates together to form built-up members

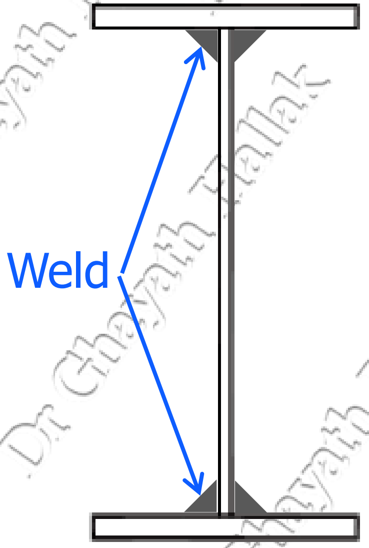
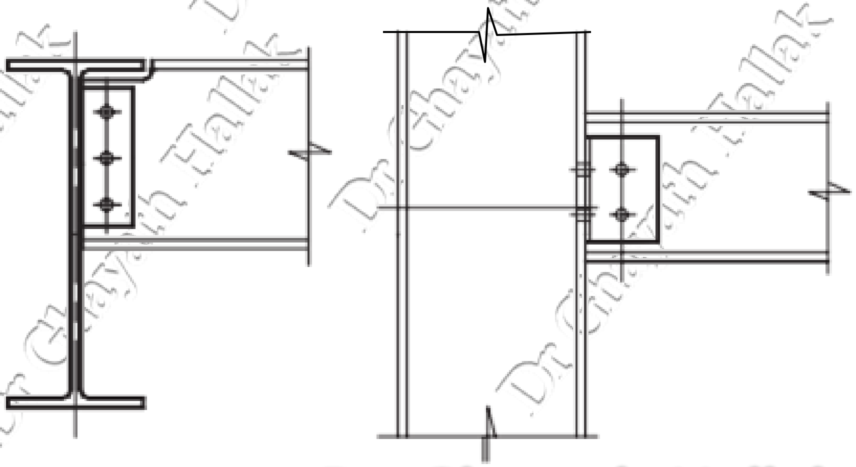


Plate girder

Simple connections

TYPES OF CONNECTIONS

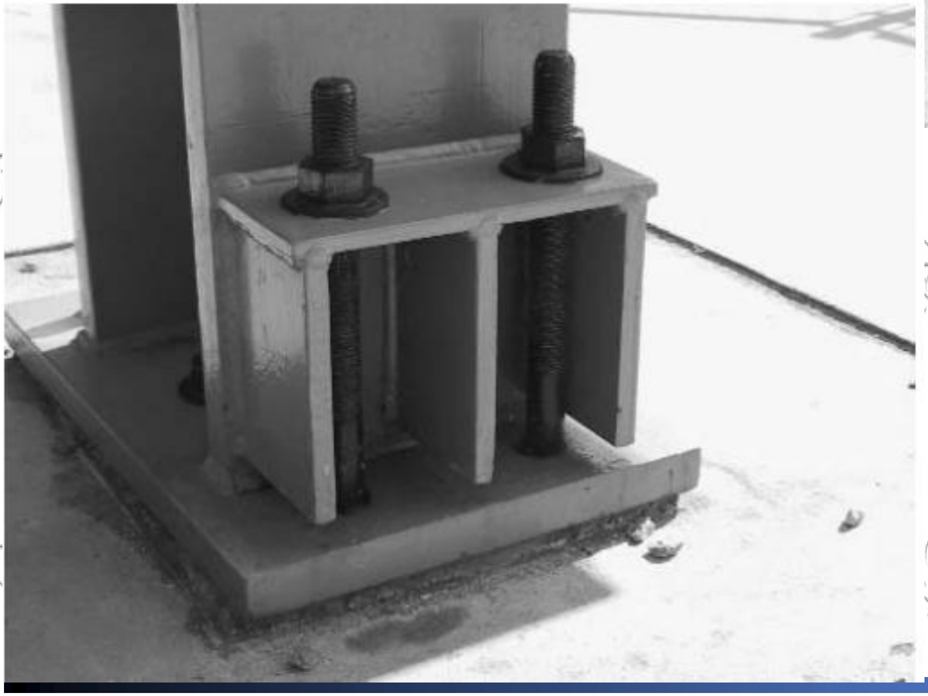
3. Beams to beams, trusses, bracing, etc. to columns in structural frames

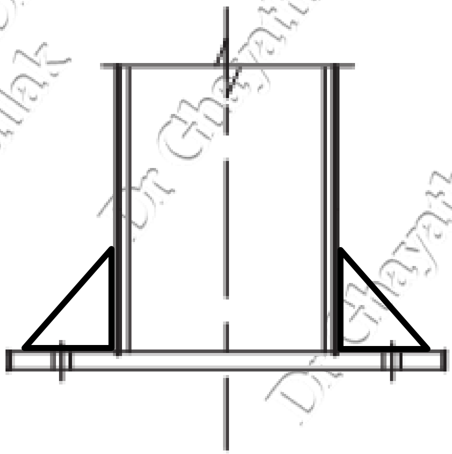


Simple connections

TYPES OF CONNECTIONS

4. Columns to foundations





Simple connections

TYPES OF CONNECTIONS

Connections may be made by:

1- Bolting:

- ❑-ordinary or non-preloaded bolts in standard clearance or oversize holes
- ❑- Preloaded bolt

2- Welding:

- ❑-fillet weld
- ❑- butt weld

Categories of bolted connections

EN 1993-1-8 : 2005, Table 3.2: Categories of bolted connections

Category	Criteria	Remarks
	Shear connections	
A bearing type	$F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used.
B slip-resistant at serviceability	$F_{v,Ed.ser} \leq F_{s,Rd,ser}$ $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at serviceability see 3.9.
C slip-resistant at ultimate	$F_{v,Ed} \leq F_{s,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$ $F_{v,Ed} \leq N_{net,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at ultimate see 3.9. $N_{net,Rd}$ see 3.4.1(1) c).
	Tension connections	
D non-preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used. $B_{p,Rd}$ see Table 3.4.
E preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. $B_{p,Rd}$ see Table 3.4.

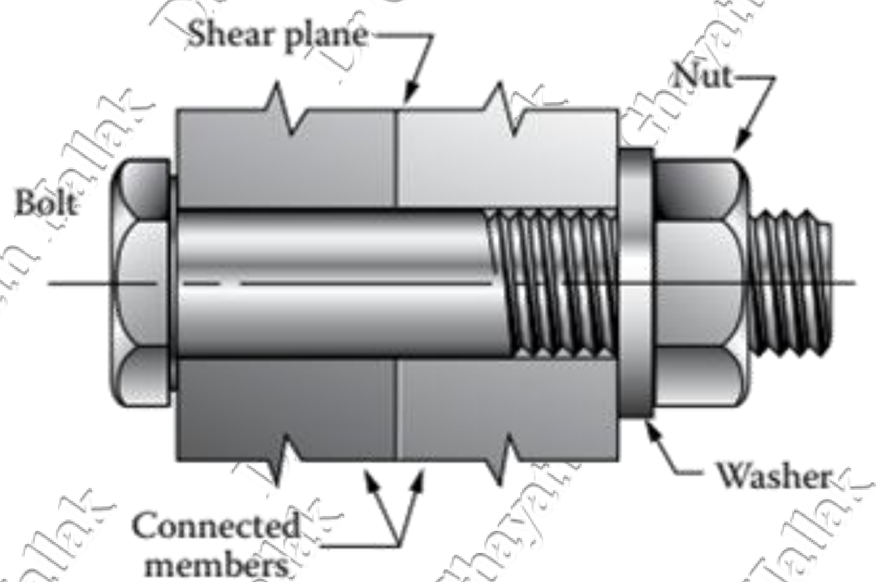
The design tensile force $F_{t,Ed}$ should include any force due to prying action, see 3.11. Bolts subjected to both shear force and tensile force should also satisfy the criteria given in Table 3.4.

Simple connections

1- Connections with Non-Preloaded Bolts

The resistance of a bolted connection is normally determined on the basis of the resistance of the **individual fasteners** and the **connected parts**.

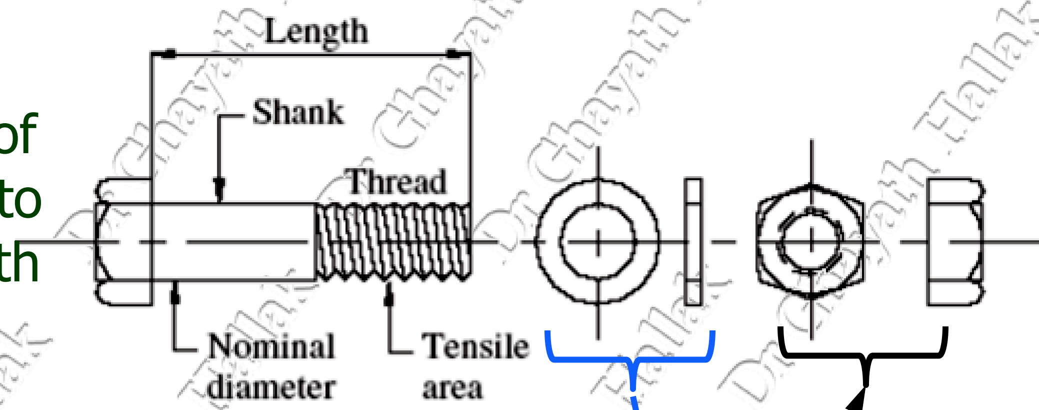
Linear-elastic analysis is most frequently used in the design of the connection. Alternatively non-linear analysis of the connection may be employed, provided that it takes account of the load-deformation characteristics of all the components of the connection.

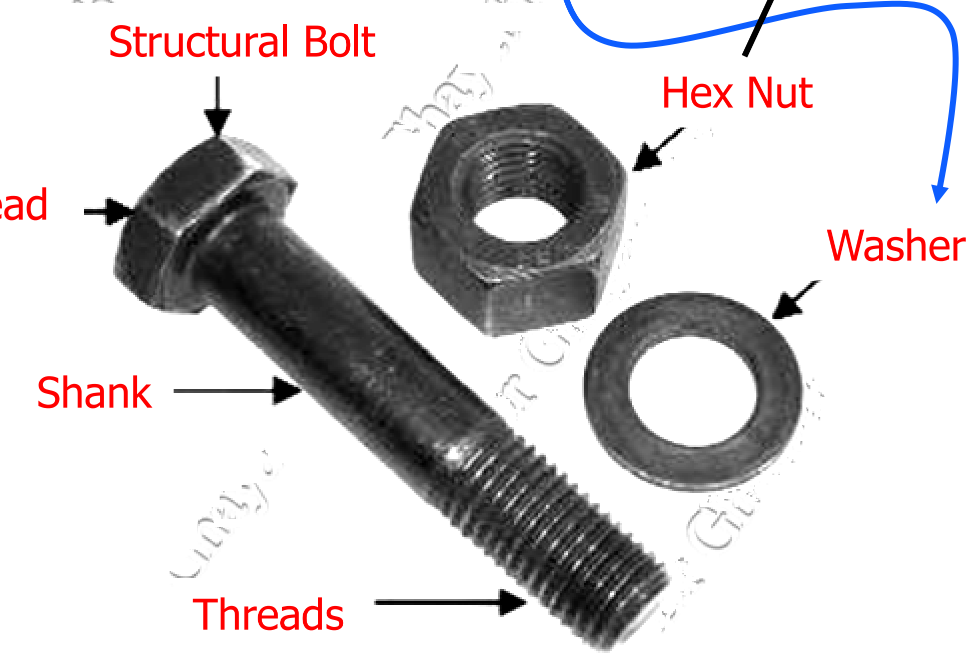


Simple connections

Connections with Non-Preloaded Bolts

the **non- preloaded** bolt, often called an "**ordinary bolt**". It is popular because of its low cost both to buy and to install. Connections made with this type of bolt are often referred to as "bearing-type" so as to distinguish them from the slip resistant connections that employ preloaded bolts.





Where a joint loaded in shear is subject to impact or significant vibration, welding or bolts with locking devices, preloaded bolts or other types of bolt which effectively prevent movement should be used.

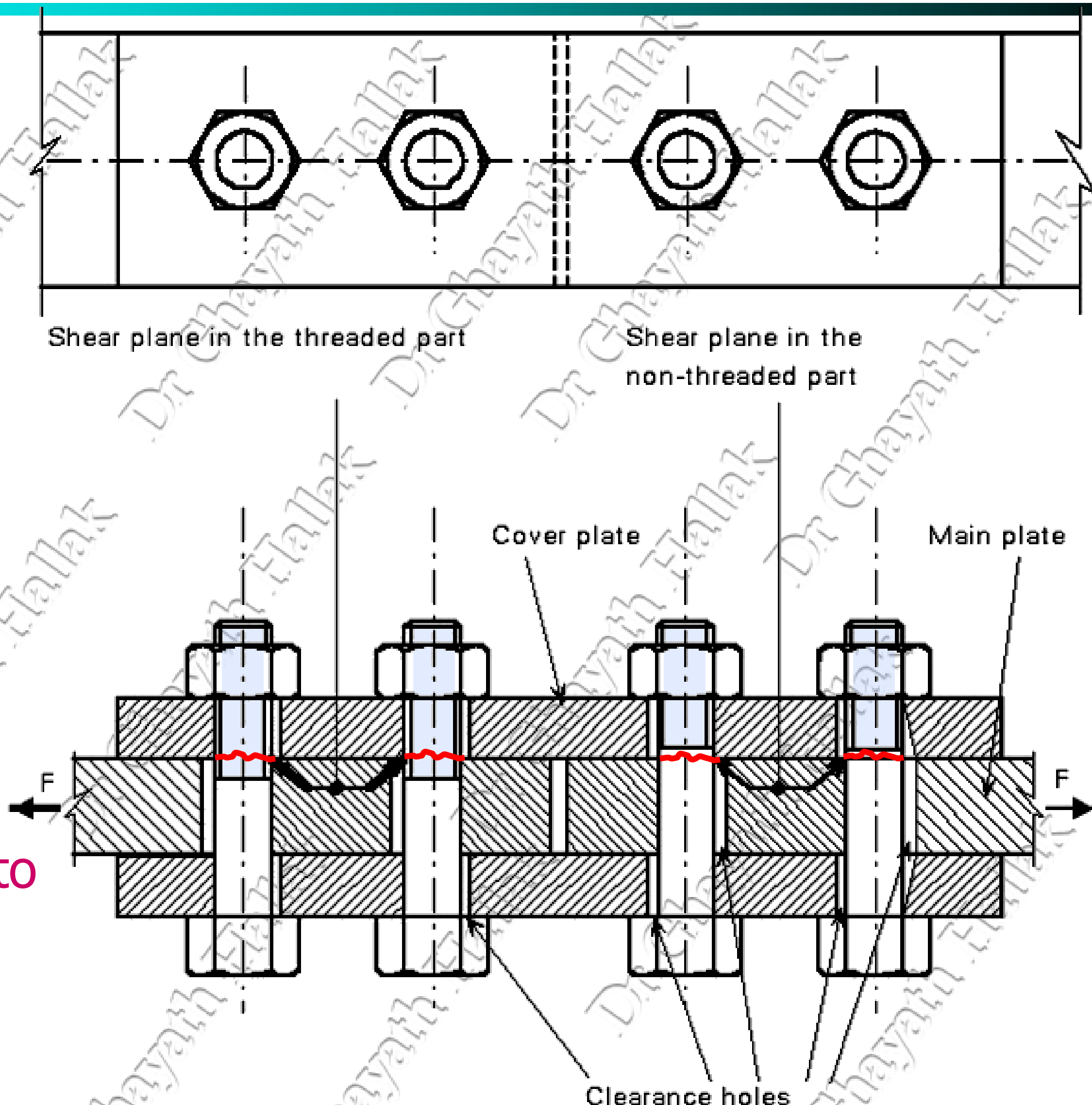
Connections with Non-Preloaded Bolts

For wind and/or stability bracing, bolts in bearing-type connections may normally be used.

LOAD TRANSMISSION

In structural connections, bolts are used to transfer loads from one plate to another.

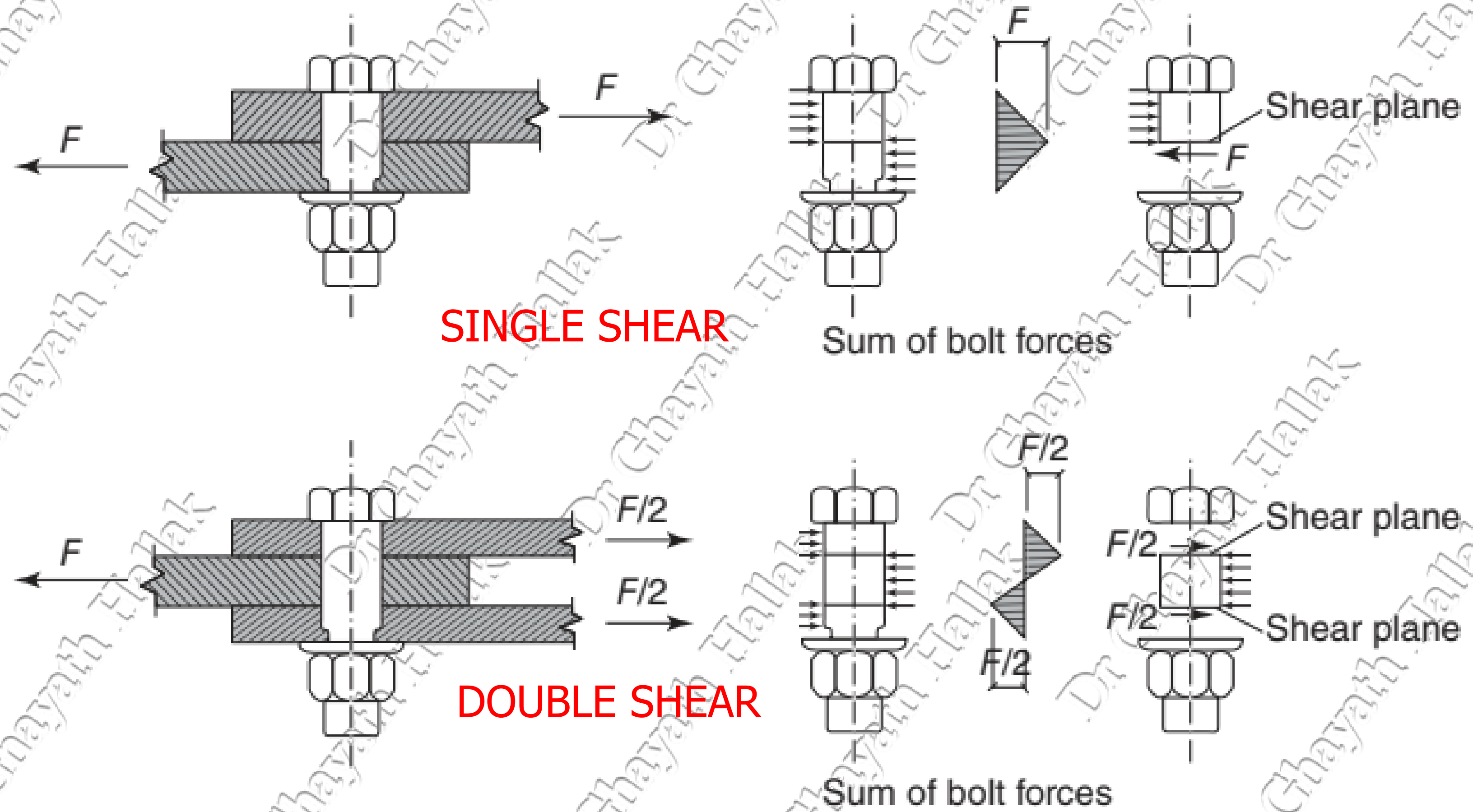
The load is transmitted into and out of the bolts by **bearing on the connected plates.** The forces in the bolts are transmitted by transverse **shear.**



Typical bolted connection with cover plates SUBJECTED TO **SHEAR FORCE**

Connections with Non-Preloaded Bolts

LOAD TRANSMISSION

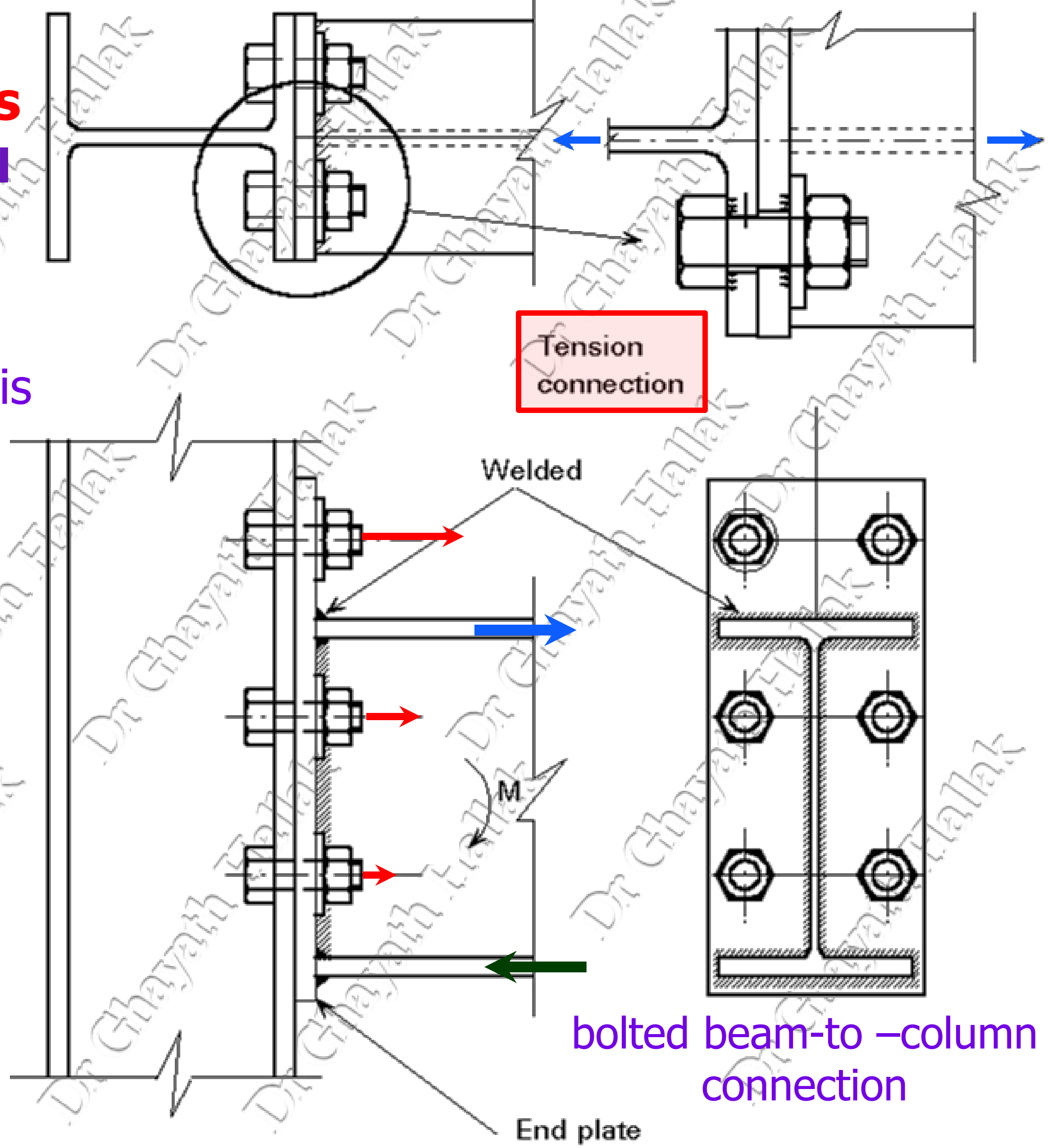


Shear planes and effects on bolts.

Connections with Non-Preloaded Bolts

LOAD TRANSMISSION

In the case of moment loading (M) only, the tension part of the load is transmitted by axial tension in the bolt.

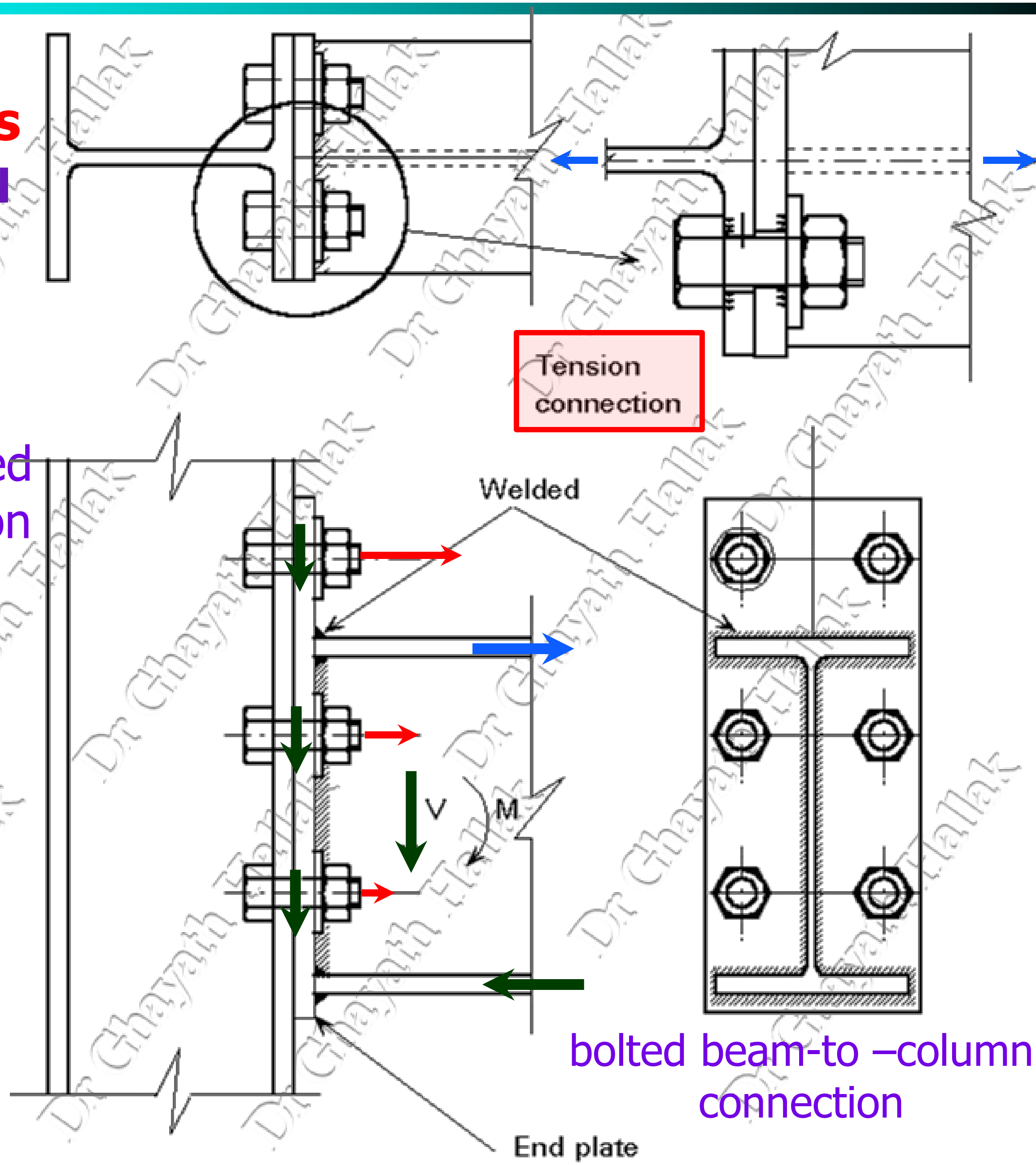


bolted beam-to –column connection

Connections with Non-Preloaded Bolts

LOAD TRANSMISSION

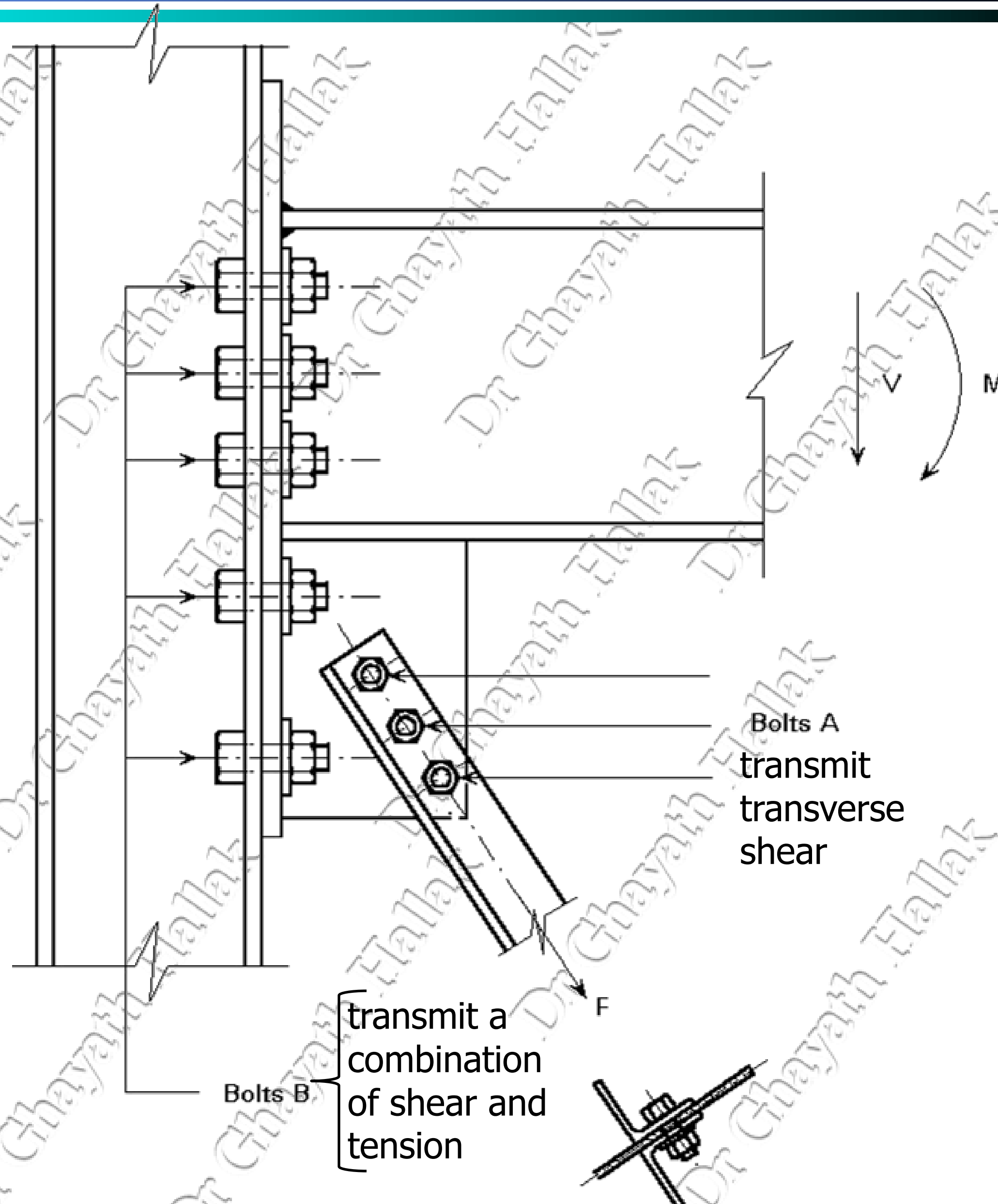
In the case of combined moment (M) and transverse loading (V), the bolts may be required to transmit a combination of transverse shear and axial tension.



bolted beam-to –column connection

Connections with Non-Preloaded Bolts

LOAD TRANSMISSION



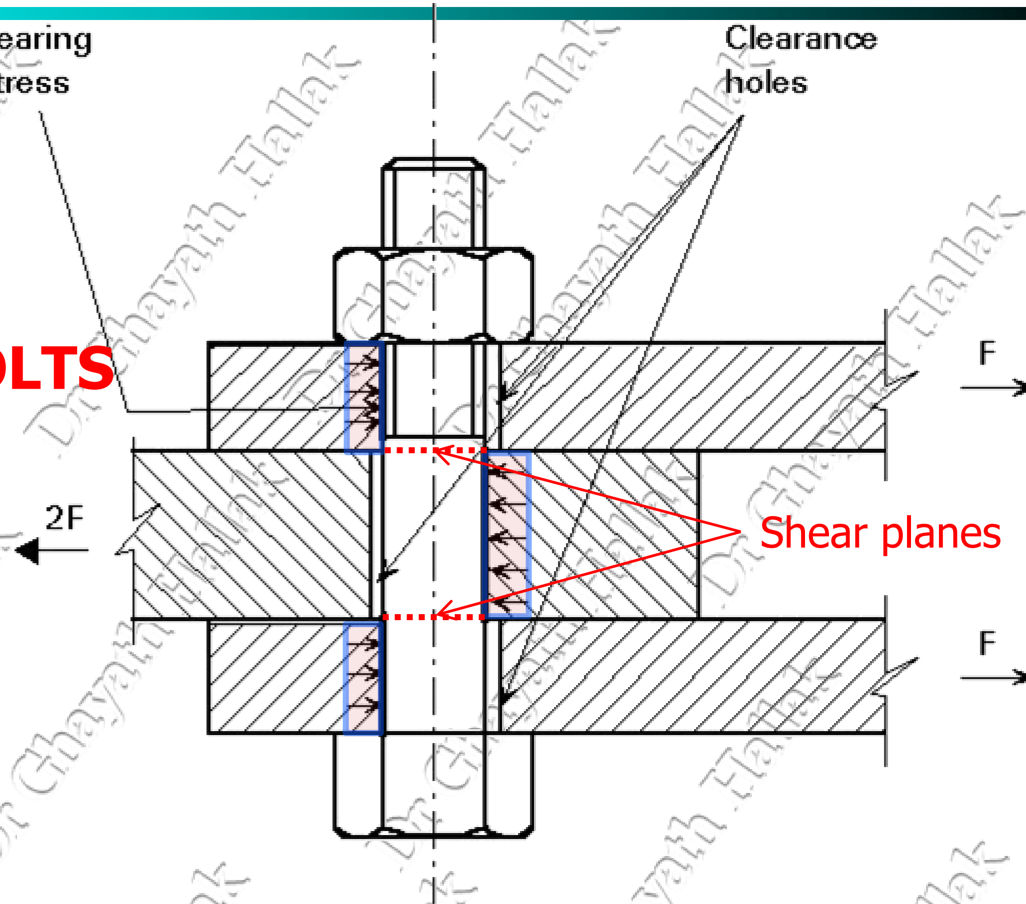
bolted beam-to –column connection with brace

Connections with Non-Preloaded Bolts

LOAD TRANSMISSION IN A SPLICE JOINT

DIMENSIONS OF THE BOLTS

Hexagon headed bolts and nuts are available in a range of sizes up to about 68 mm shank diameter.

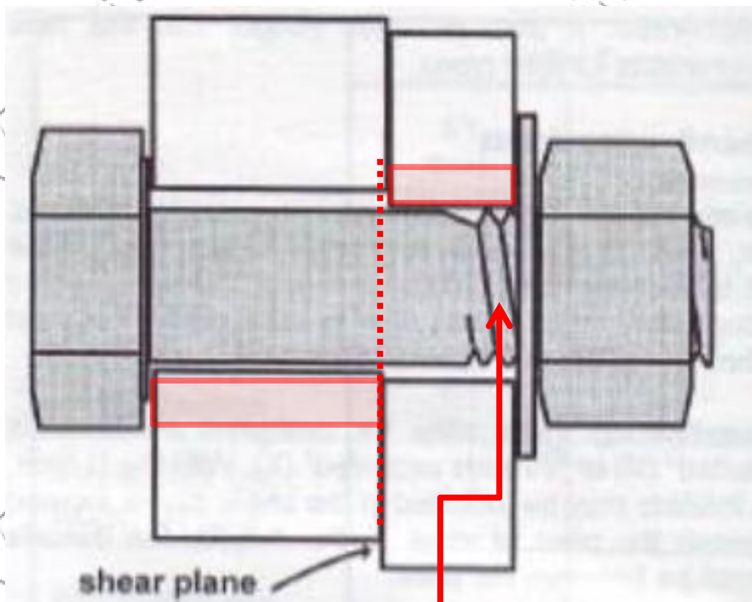


The bolt sizes are indicated by the designation M followed by a number multiplied by another number.

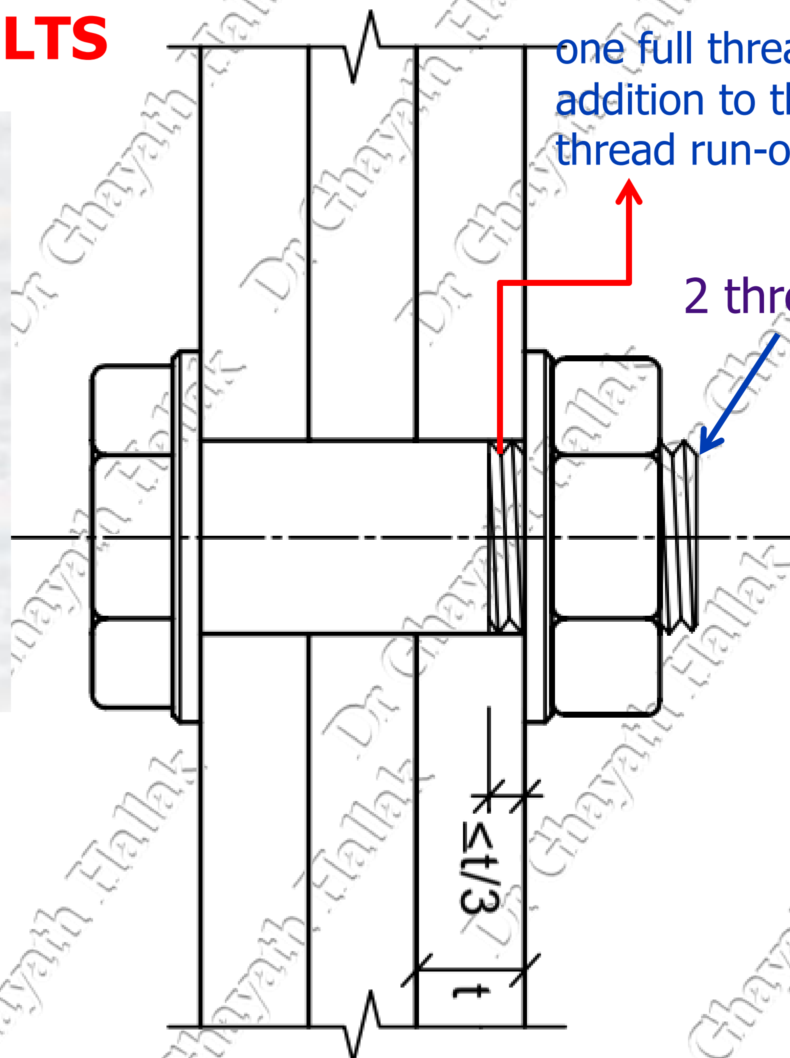
M 20 x 60 where: the diameter of the shank is 20 mm
the length of the shank + the threaded part is 60 mm.
The M stands for metric.

Connections with Non-Preloaded Bolts

DIMENSIONS OF THE BOLTS



one full thread (in addition to the thread run-out)



For bolted connections:

$1.5\, t_{min} \le d \le 2.25\, t_{min}$ (mm) *or* $d = (50\, t_{min})^{0.5} - 4$ (mm)

d nominal diameter of the bolt, t_{min} is the thickness of thinner connected plates

Connections with Non-Preloaded Bolts

BOLT GRADES

Bolts and nuts are available in steels of minimum tensile strengths up to about 1370 MPa.

The grade of the bolts is indicated by two numbers. The most common grades are 4.6, 5.6, 6.5, 6.8, 8.8 and 10.9.

EN 1993-1-8 : 2005- Table 3.1: Nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts

Bolt class	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1000

The design yield stress f_{yb} = first number x second number x 10.

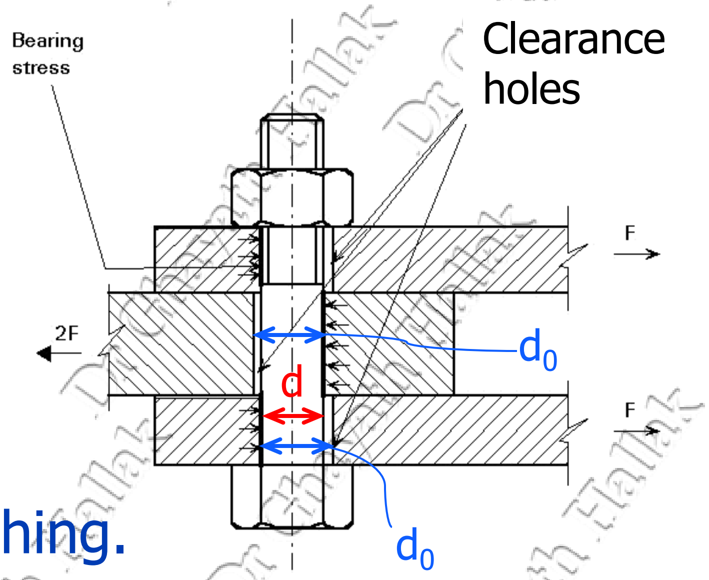
The design ultimate stress f_{ub} = first number x 100 (MPa).

Connections with Non-Preloaded Bolts

DIAMETER OF THE HOLES

Because of the tolerances in the positioning of holes and the tolerances of the bolt diameter (d) and the hole diameter (d_0), a clearance is necessary.

For bearing-type connections, this clearance may cause slip of the plates when they are loaded.



- 1mm for M12 and M14 bolts
- 2mm for M16 to M24 bolts
- 3mm for M27 and larger bolts.

Holes with smaller clearances than standard holes may be specified.

Holes will be formed by drilling or punching.

Connections with Non-Preloaded Bolts

NOMINAL AND STRESS SECTIONS OF A BOLT

Nominal diameter d (mm)	**8**	**10**	**12**	14	**16**	18	**20**	22	**24**	**27**	**30**
Nominal area A (mm^2)	50,3	78,5	113	154	201	254	314	380	452	573	707
Stress area As (mm^2)	36,6	58,0	84,3	115	157	192	245	303	353	459	561

Stress area of bolts

Connections with Non-Preloaded Bolts

SHEAR RESISTANCE

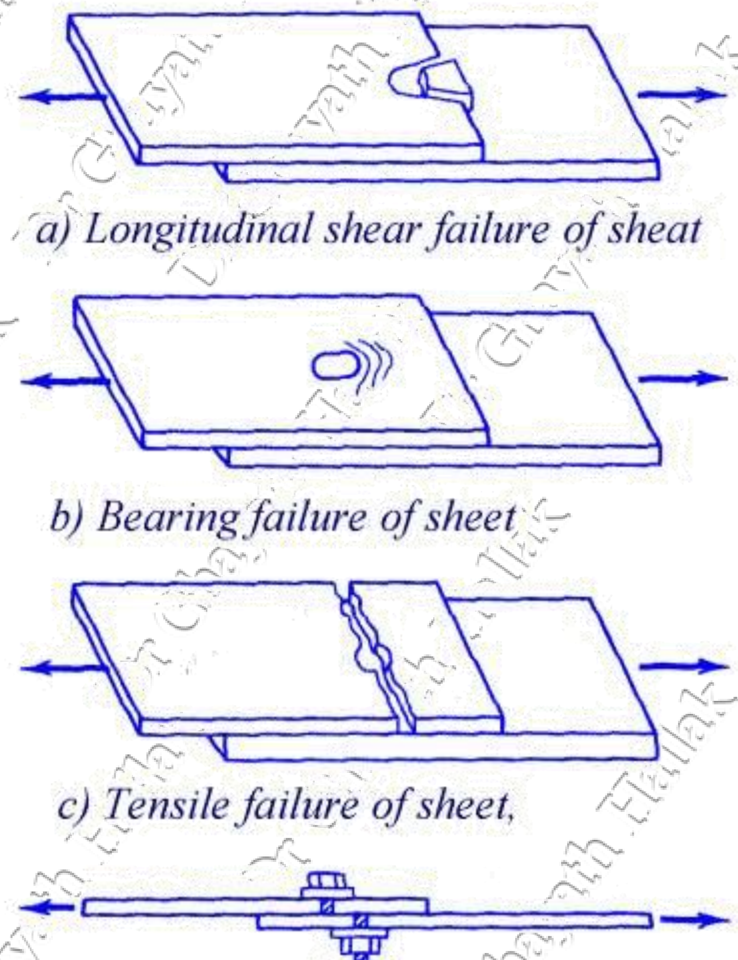
A shear joint can fail in the following four modes:

(a) by shear at the end of the member (plate failure-Block Tear)

(b) by bearing on the member or bolt (plate failure)

(c) by tension in the member (plate failure)

(d) by shear on the bolt shank (bolt failure)


a) Longitudinal shear failure of sheat

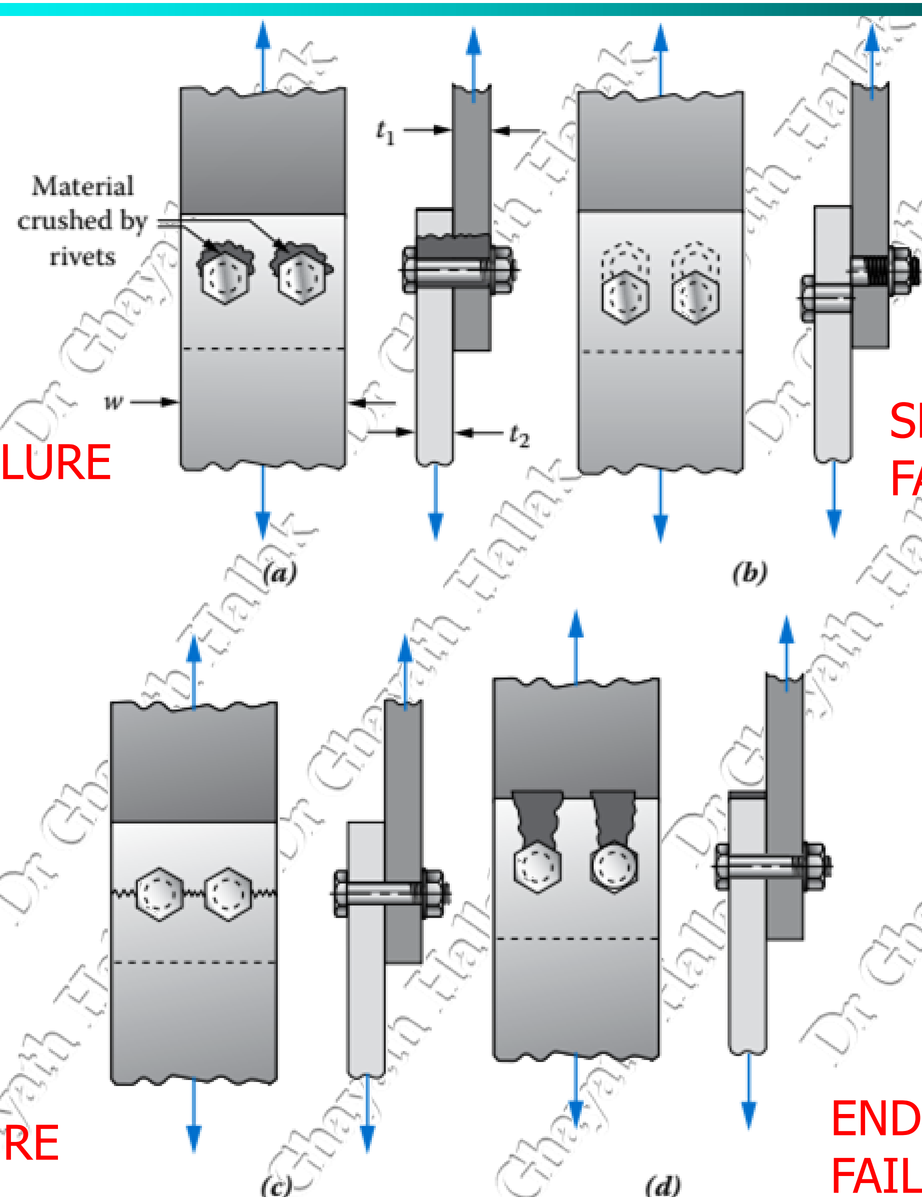
b) Bearing failure of sheet

c) Tensile failure of sheet,

d) Shear failure of bolt

BEARING FAILURE

SHEAR OF BOLTS FAILURE



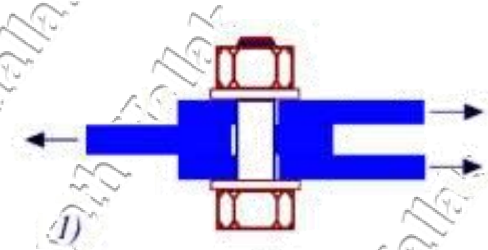
TENSILE FAILURE

END TEAROUT FAILURE

TYPES OF FAILURE OF BOLTED CONNECTIONS FAILURE

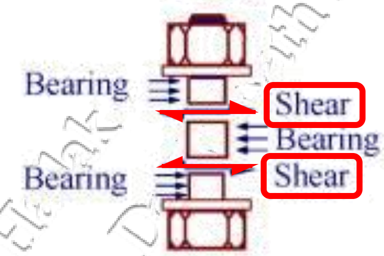
Connections with Non-Preloaded Bolts

SHEAR RESISTANCE



The design shear resistance of a bolt ($F_{v.Rd}$) in normal conditions, per shear plane, is:

(a) For the shear plane passing through the threaded portion of the bolt:



$$\boxed{F_{v,Rd} = \alpha_v f_{ub} A_s/\gamma_{M2}}$$

$\alpha_v = 0.6$ for strength grades 4.6, 5.6 and 8.8

$\alpha_v = 0.5$ for strength grades 4.8, 5.8, 6.8 and 10.9

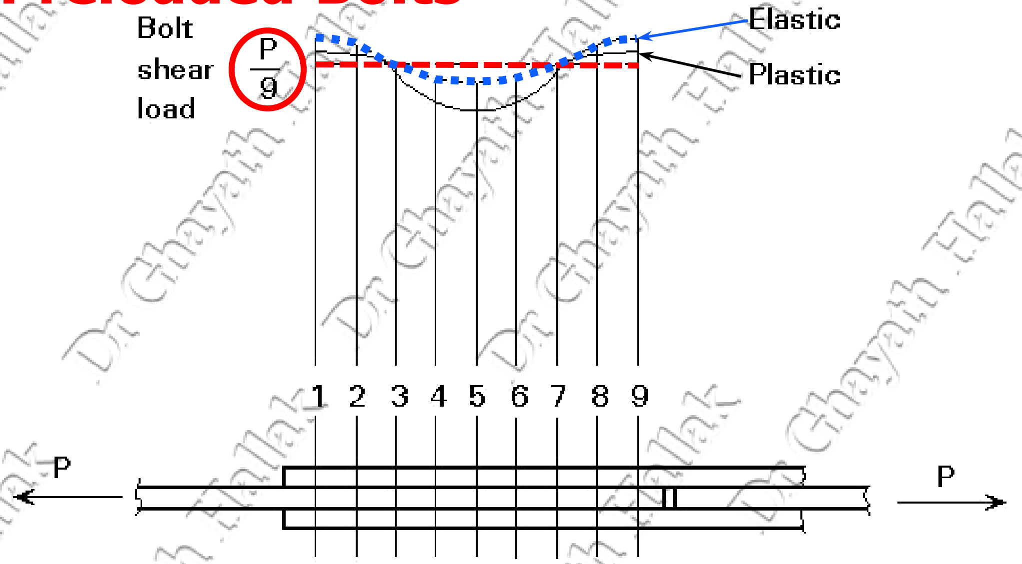
(b) For the shear plane passing through the unthreaded portion of the bolt:

$$\boxed{F_{v,Rd} = 0.6 f_{ub} A/ \gamma_{M2}}$$

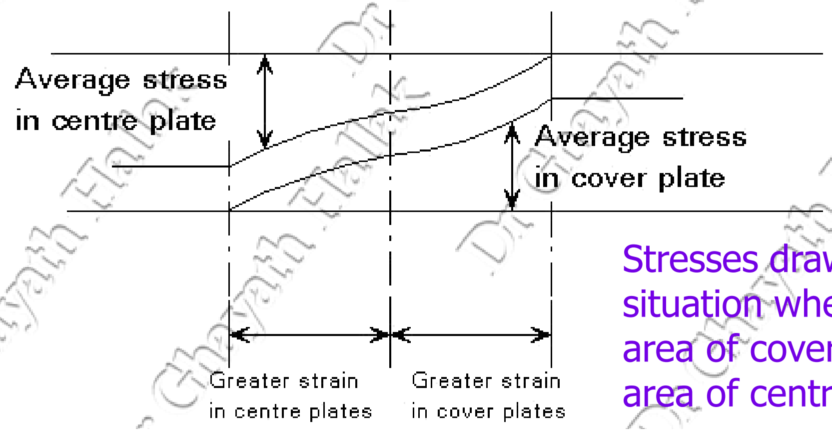
Connections with Non-Preloaded Bolts

LONG JOINTS

The loads transferred through the outer bolts (1 and 9 in the Figure) are greater than those through bolts towards the centre of the joint. If the total area of the cover plates exceeds that of the centre plate the distribution will not be symmetrical, and bolt 1 will transfer more load than any other.



(a) Distribution of bolt shear loads arising from P



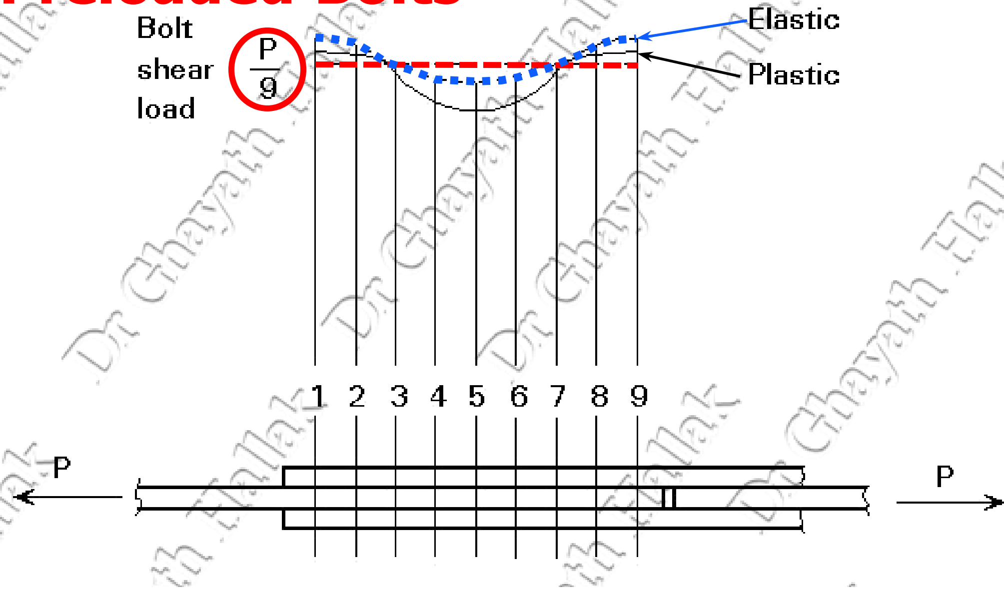
Stresses drawn for situation where total area of cover plates= area of central plate

(b) Incompatibility of tensile strains in connected elements

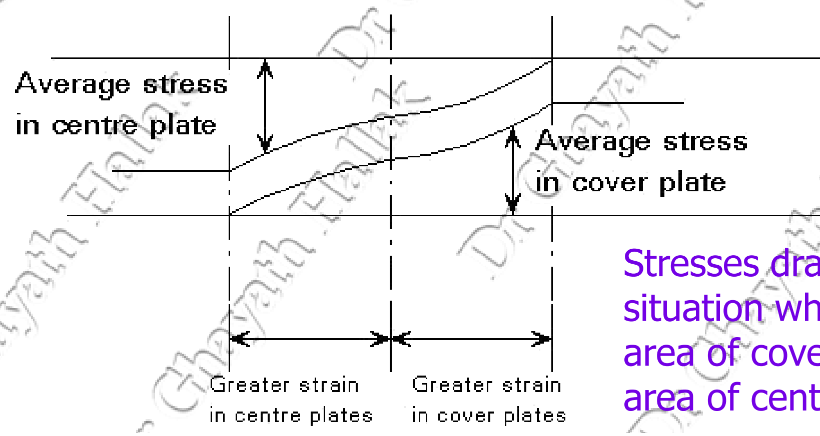
Connections with Non-Preloaded Bolts

LONG JOINTS

When the fasteners yield, their flexibility increases causing a more uniform sharing of the load However, for long steelwork joints of normal proportions this behaviour will be insufficient to produce an equal load distribution. The end-bolts will reach their deformation limit and so fail before the remaining ones have been fully loaded. This will result in progressive failure at an average shear value per bolt below the single-bolt shear resistance.



(a) Distribution of bolt shear loads arising from P



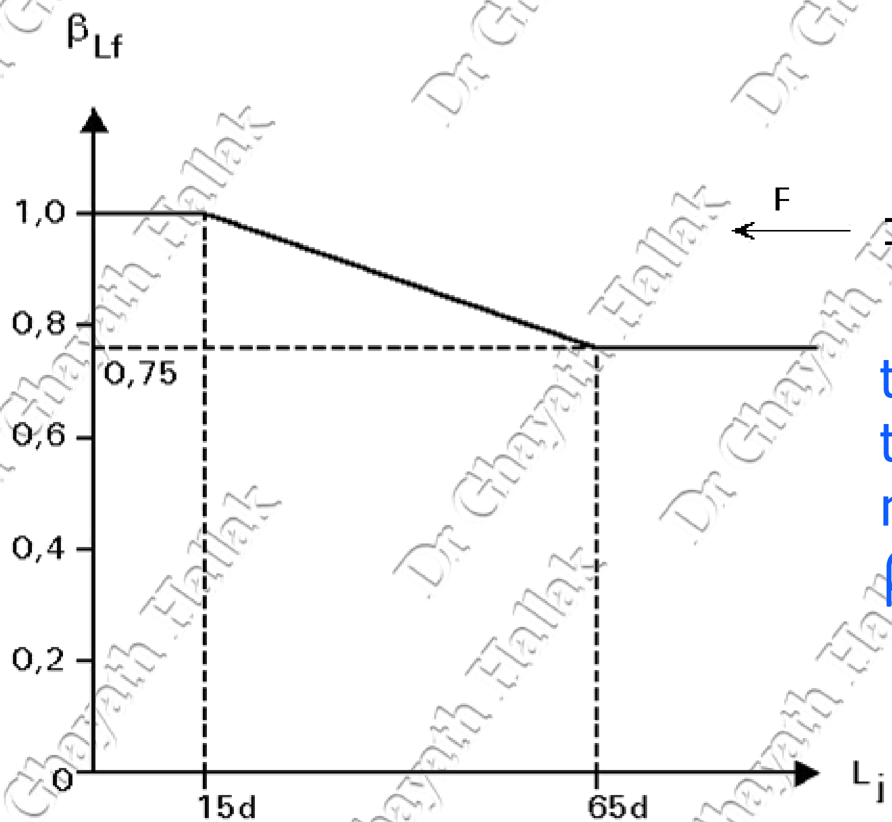
Stresses drawn for situation where total area of cover plates= area of central plate

(b) Incompatibility of tensile strains in connected elements

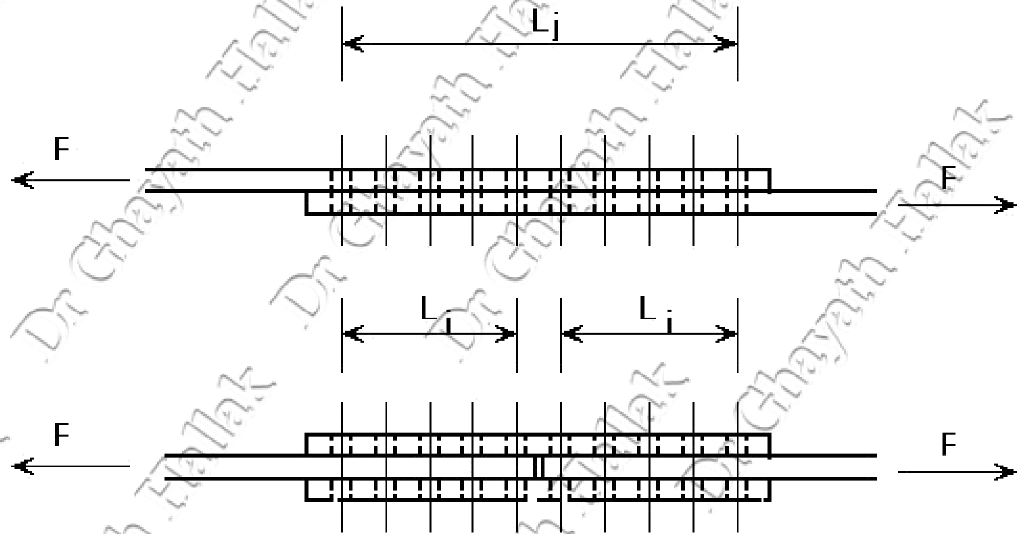
Connections with Non-Preloaded Bolts

LONG JOINTS

$L_j > 15d$



Reduction factor for long joints



the design shear resistance of all the fasteners shall be reduced by multiplying it by a reduction factor β_{Lf}, given by:

$$\beta_{Lf} = 1 - \frac{L_j - 15d}{200d}$$

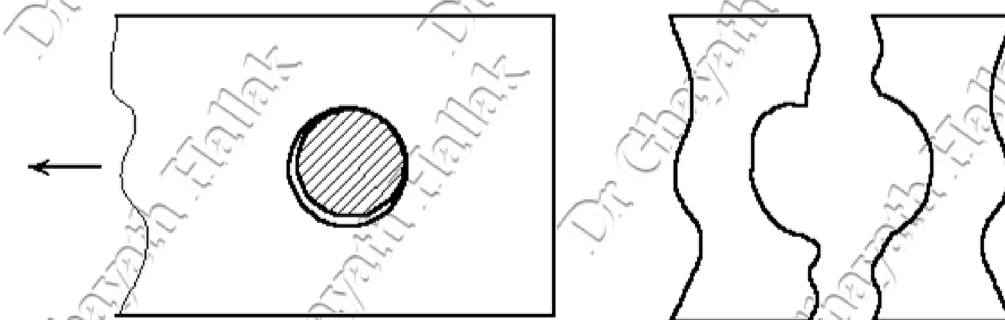
but $\beta_{Lf} \leq 1{,}0$ and $\beta_{Lf} \geq 0{,}75$

Connections with Non-Preloaded Bolts

BEARING RESISTANCE

Yielding due to pressure between the bolt shank and plate material may result in excessive deformation of the plate around the bolt hole and possibly some distortion of the bolt.

The area resisting the bearing pressure is assumed to be the product of the plate thickness and the nominal bolt diameter.

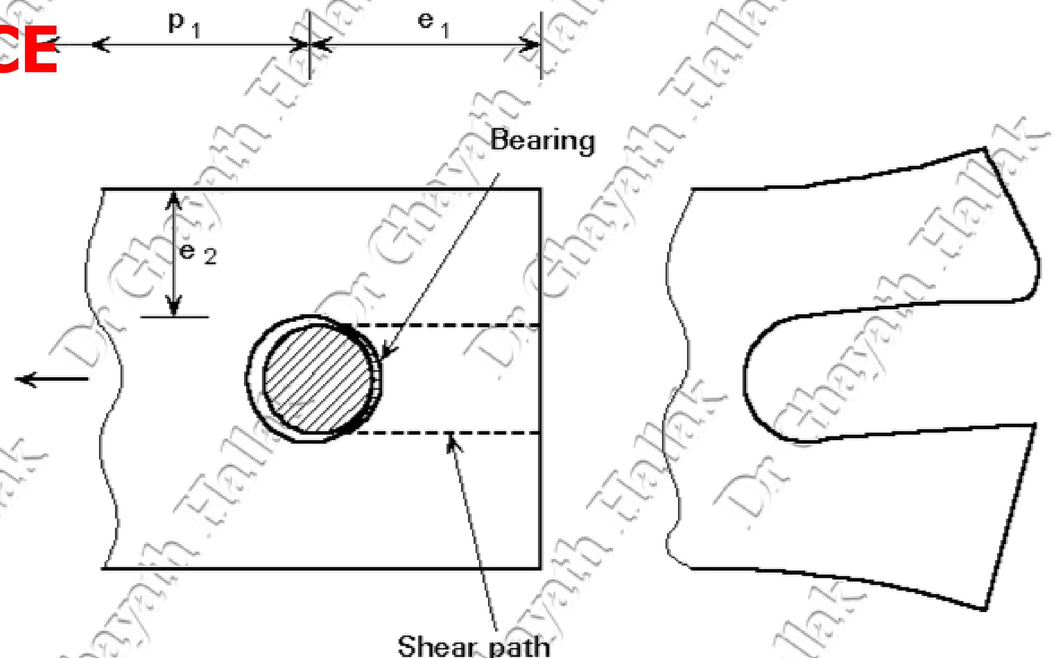


Failure modes of flat member

Connections with Non-Preloaded Bolts

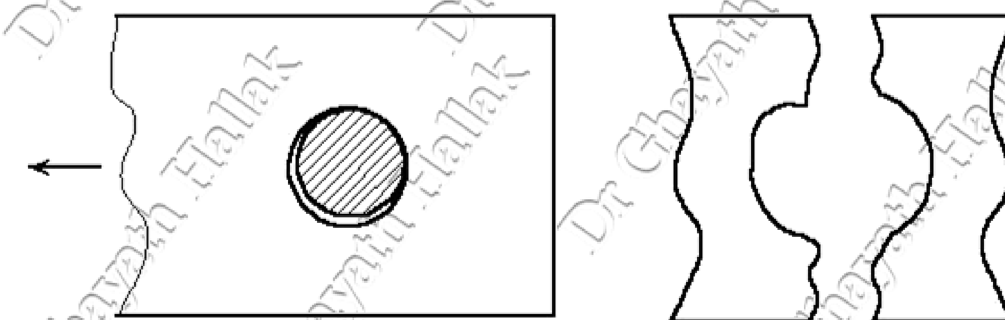
BEARING RESISTANCE

The distance (e_1) of the bolt from the end of the plate must be sufficient to provide adequate resistance to the shearing-out mode of failure shown in the Figure, which is governed by the area of the shear path.



Shearing-out-failure

The presence of threads in the grip does not significantly affect the bearing resistance but will cause some increase of the deformation.



Net section failure

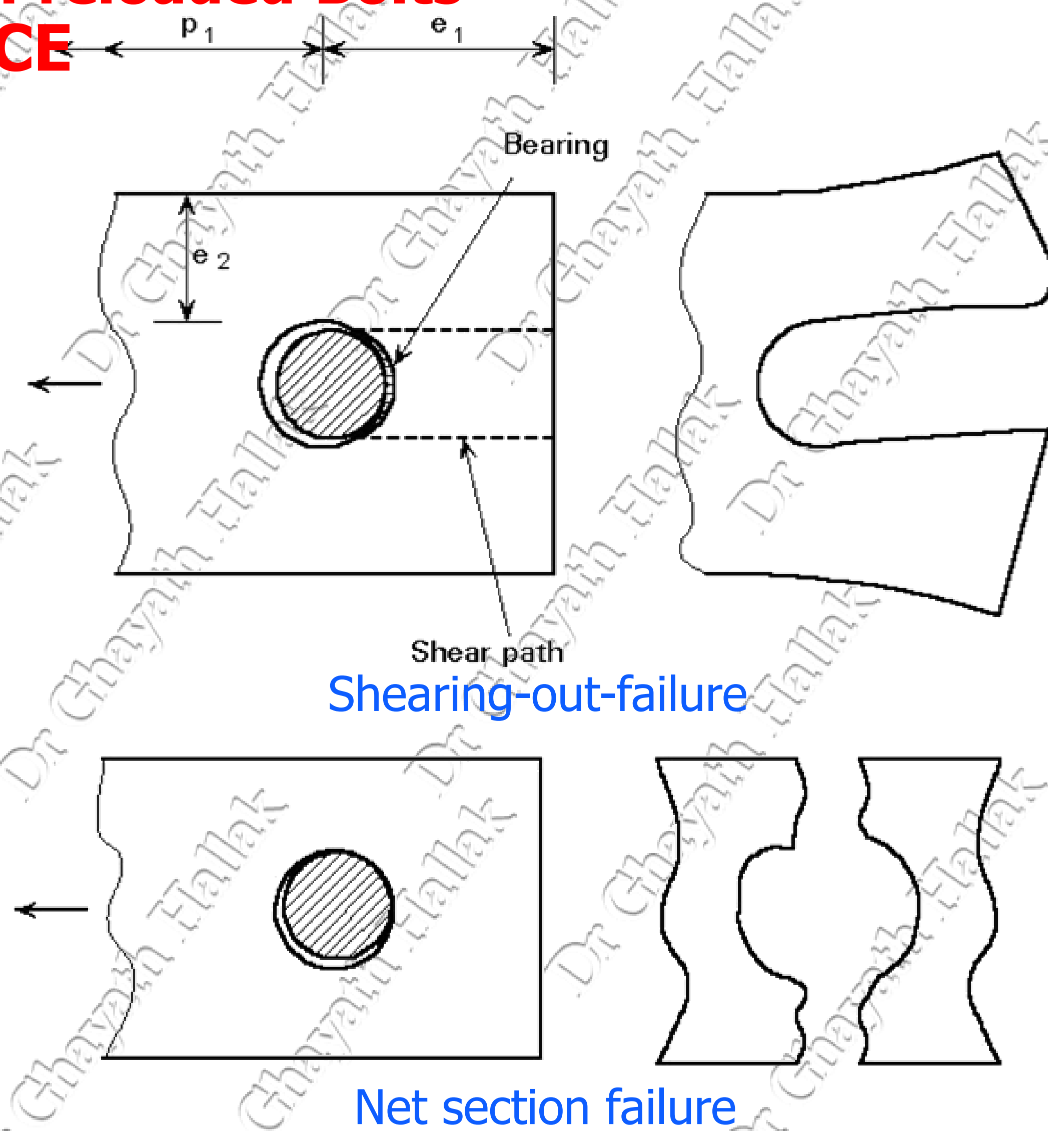
Failure modes of flat member

Connections with Non-Preloaded Bolts

BEARING RESISTANCE

If the shear resistance (connection-bolts) is greater than the bearing resistance of the plates, one of the failure modes shown in the Figure will occur. In this case, the **deformation capacity** of the connection **is very large**. The joint has a **"ductile"** behaviour.

In the other case, when the failure is due to the **shearing** of the bolts, the **deformation capacity** of the connection is **very small** and the joint has a **"brittle"** behaviour.

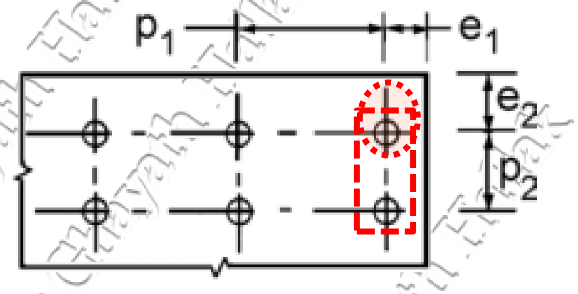


Failure modes of flat member

Connections with Non-Preloaded Bolts

BEARING RESISTANCE

The design bearing resistance of a bolt is given by:



$$F_{b,Rd} = k_1 \;\alpha_b\; f_u\, d\, t \,/\, \gamma_{M2}$$

$$\alpha_b = \text{Min}\{\, \boxed{e_1/3d_0},\; \boxed{[p_1/3d_0 - 1/4]},\; [f_{ub}/f_u],\; 1.0 \}$$

α_d for end bolt

α_d for inner bolt

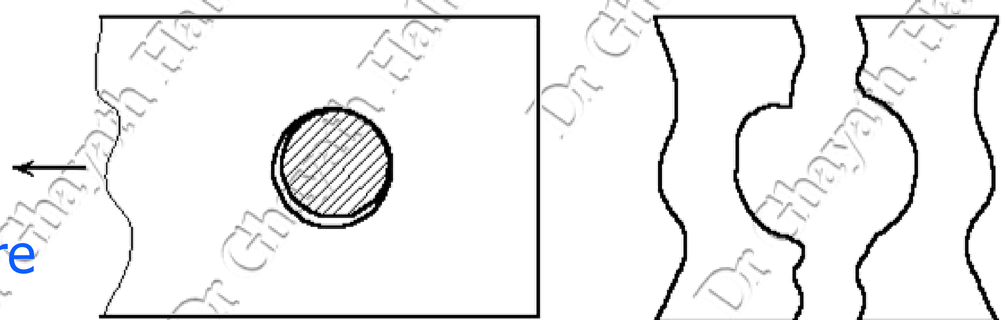
$$k_1 = \text{Min}\{\, \boxed{[(2.8e_2/d_0)-1.7]},\; \boxed{[(1.4p_2/d_0)-1.7]},\; 2.5 \}$$

for edge bolt

for inner bolt

This reduction coefficient α_b is necessary, because when the end distance is short, the capacity of deformation is small.

If the net section of the member is small, net section rupture may govern the failure load of the connection.



Net section failure

Connections with Non-Preloaded Bolts
TENSION RESISTANCE

Tension resistance of a bolt	Punching shear resistance of the plate
 $$F_{t,Rd} = \frac{k_2 \cdot f_{ub} \cdot A_s}{\gamma_{M2}}$$	 $$B_{p,Rd} = \frac{0.6 \cdot \pi \cdot f_u \cdot d_m \cdot t_p}{\gamma_{M2}}$$

Symbols

$k_2 = 0.9$ in general

0.63 for countersunk bolts

f_u = ultimate strength of connected plates

f_{ub} = ultimate strength of bolts

d_m = the mean value between in- and circumscribed diameters of bolt head and bolt nut, whichever is smaller

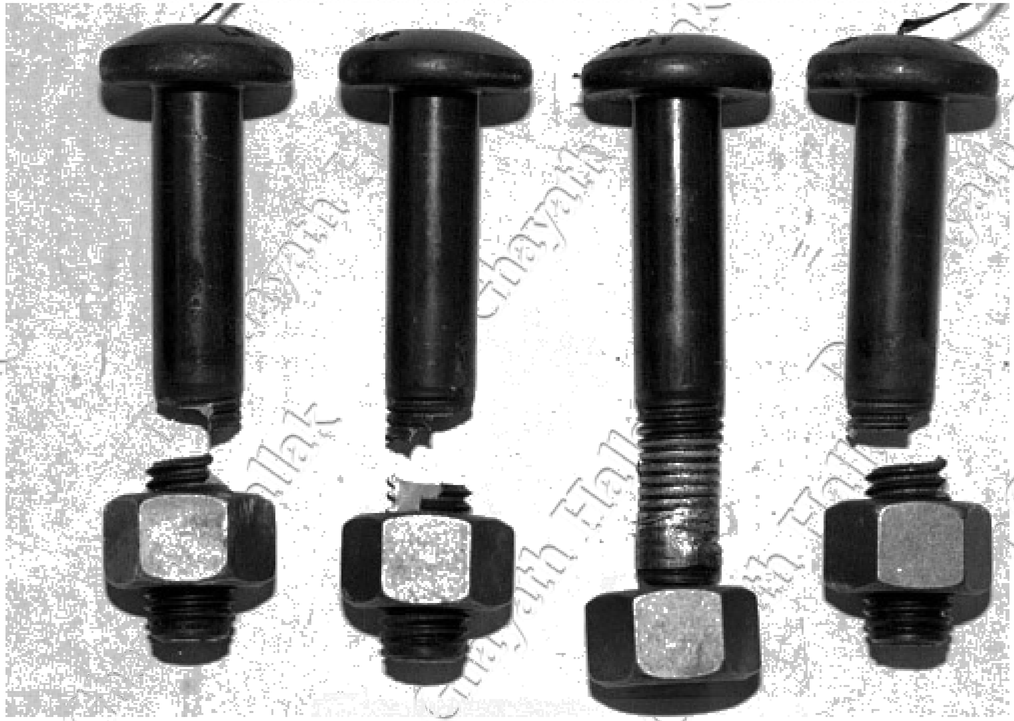
t_p = is the thickness of the plate under the bolt or the nut.

$\gamma_{M2} = 1.25$

Connections with Non-Preloaded Bolts
TENSION RESISTANCE

$$F_{t.Rd} = k_2 \, f_{ub} \, A_s / \gamma_{M2}$$

k_2 = 0,63 for countersunk bolt,
otherwise k_2 = 0,9



Connections with Non-Preloaded Bolts

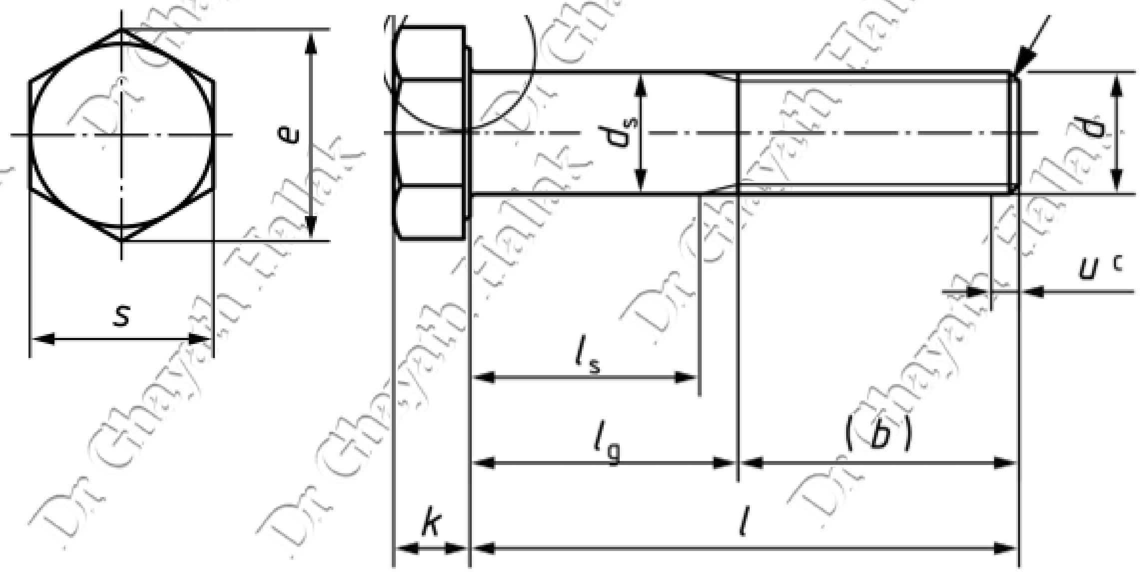
PUNCHING RESISTANCE

$$B_{p,Rd} = \frac{0.6 \cdot \pi \cdot f_u \cdot d_m \cdot t_p}{\gamma_{M2}}$$

$$d_m = \min\left(\left[\frac{e+s}{2}\right]_{head} ; \left[\frac{e+s}{2}\right]_{nut}\right)$$

e is the width across points of the bolt head or the nut
s is the width across flats of the bolt head or the nut
f_u is the ultimate tensile strength of the ply under the bolt head or nut

Non preloaded hexagon head bolts
Classes 4.6,8.8 and 10.9,as specified in BS EN IS0 4014, BS EN IS0 4016, BS EN IS0 4017 and BS EN IS0 4018



Connections with Non-Preloaded Bolts
PUNCHING RESISTANCE

Non preloaded hexagon head bolts

Dimensions in millimetres

Thread, d		M5	M6	M8	M10	M12	M16	M20
P^a		0,8	1	1,25	1,5	1,75	2	2,5
b ref.	b	16	18	22	26	30	38	46
	c	22	24	28	32	36	44	52
	d	35	37	41	45	49	57	65
c	max.	0,5	0,5	0,6	0,6	0,6	0,8	0,8
d_a	max.	6	7,2	10,2	12,2	14,7	18,7	24,4
d_s	max.	5,48	6,48	8,58	10,58	12,7	16,7	20,84
	min.	4,52	5,52	7,42	9,42	11,3	15,3	19,16
d_w	min.	6,74	8,74	11,47	14,47	16,47	22	27,7
e	min.	8,63	10,89	14,2	17,59	19,85	26,17	32,95
k	nom.	3,5	4	5,3	6,4	7,5	10	12,5
	max.	3,875	4,375	5,675	6,85	7,95	10,75	13,4
	min.	3,125	3,625	4,925	5,95	7,05	9,25	11,6
k_w^e	min.	2,19	2,54	3,45	4,17	4,94	6,48	8,12
r	min.	0,2	0,25	0,4	0,4	0,6	0,6	0,8
s	= max.	8,00	10,00	13,00	16,00	18,00	24,00	30,00
	min.	7,64	9,64	12,57	15,57	17,57	23,16	29,16

Preferred threads

Connections with Non-Preloaded Bolts
PUNCHING RESISTANCE

Non preloaded hexagon head bolts

Dimensions in millimetres

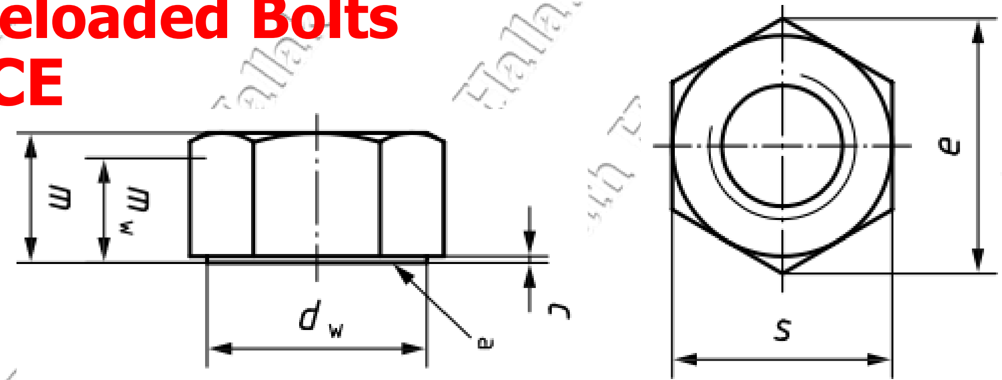
Thread, d		M24	M30	M36	M42	M48	M56	M64
P[a]		3	3,5	4	4,5	5	5,5	6
b ref.	b	54	66	—	—	—	—	—
	c	60	72	84	96	108	—	—
	d	73	85	97	109	121	137	153
c	max.	0,8	0,8	0,8	1	1	1	1
d_a	max.	28,4	35,4	42,4	48,6	56,6	67	75
d_s	max.	24,84	30,84	37	43	49	57,2	65,2
	min.	23,16	29,16	35	41	47	54,8	62,8
d_w	min.	33,25	42,75	51,11	59,95	69,45	78,66	88,16
e	min.	39,55	50,85	60,79	71,3	82,6	93,56	104,86
k	nom.	15	18,7	22,5	26	30	35	40
	max.	15,9	19,75	23,55	27,05	31,05	36,25	41,25
	min.	14,1	17,65	21,45	24,95	28,95	33,75	38,75
k_w[e]	min.	9,87	12,36	15,02	17,47	20,27	23,63	27,13
r	min.	0,8	1	1	1,2	1,6	2	2
s	= max.	36	46	55,0	65,0	75,0	85,0	95,0
	min.	35	45	53,8	63,1	73,1	82,8	92,8

Preferred threads

Connections with Non-Preloaded Bolts

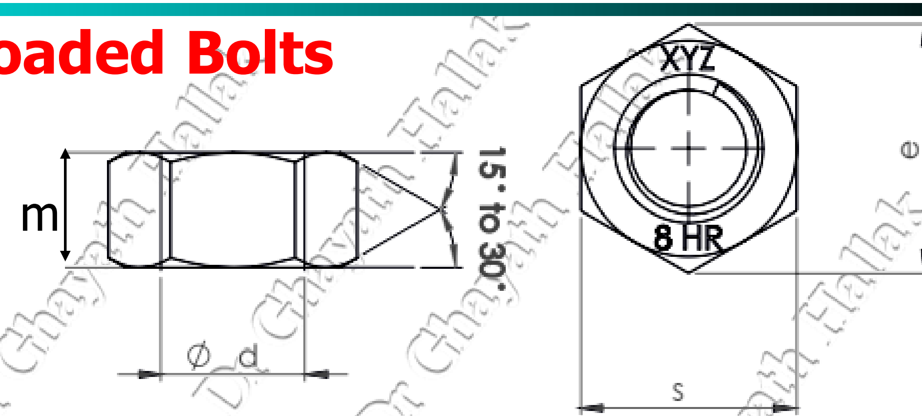
PUNCHING RESISTANCE

BS EN IS0 4032 or
BS EN IS0 4034
Nut Dimensions



Thread *D*		M1,6	M2	M2,5	M3	M4	M5	M6	M8	M10	M12
P_a		0,35	0,4	0,45	0,5	0,7	0,8	1	1,25	1,5	1,75
c	max.	0,20	0,20	0,30	0,40	0,40	0,50	0,50	0,60	0,60	0,60
	min.	0,10	0,10	0,10	0,15	0,15	0,15	0,15	0,15	0,15	0,15
d_a	max.	1,84	2,30	2,90	3,45	4,60	5,75	6,75	8,75	10,80	13,00
	min.	1,60	2,00	2,50	3,00	4,00	5,00	6,00	8,00	10,00	12,00
d_w	min.	2,40	3,10	4,10	4,60	5,90	6,90	8,90	11,60	14,60	16,60
e	min.	3,41	4,32	5,45	6,01	7,66	8,79	11,05	14,38	17,77	20,03
m	max.	1,30	1,60	2,00	2,40	3,20	4,70	5,20	6,80	8,40	10,80
	min.	1,05	1,35	1,75	2,15	2,90	4,40	4,90	6,44	8,04	10,37
m_w	min.	0,80	1,10	1,40	1,70	2,30	3,50	3,90	5,20	6,40	8,30
s	nom. = max.	3,20	4,00	5,00	5,50	7,00	8,00	10,0	13,00	16,00	18,00
	min.	3,02	3,82	4,82	5,32	6,78	7,78	9,78	12,73	15,73	17,73

Connections with Non-Preloaded Bolts
PUNCHING RESISTANCE
BS EN IS0 4032 or
BS EN IS0 4034
Nut Dimensions

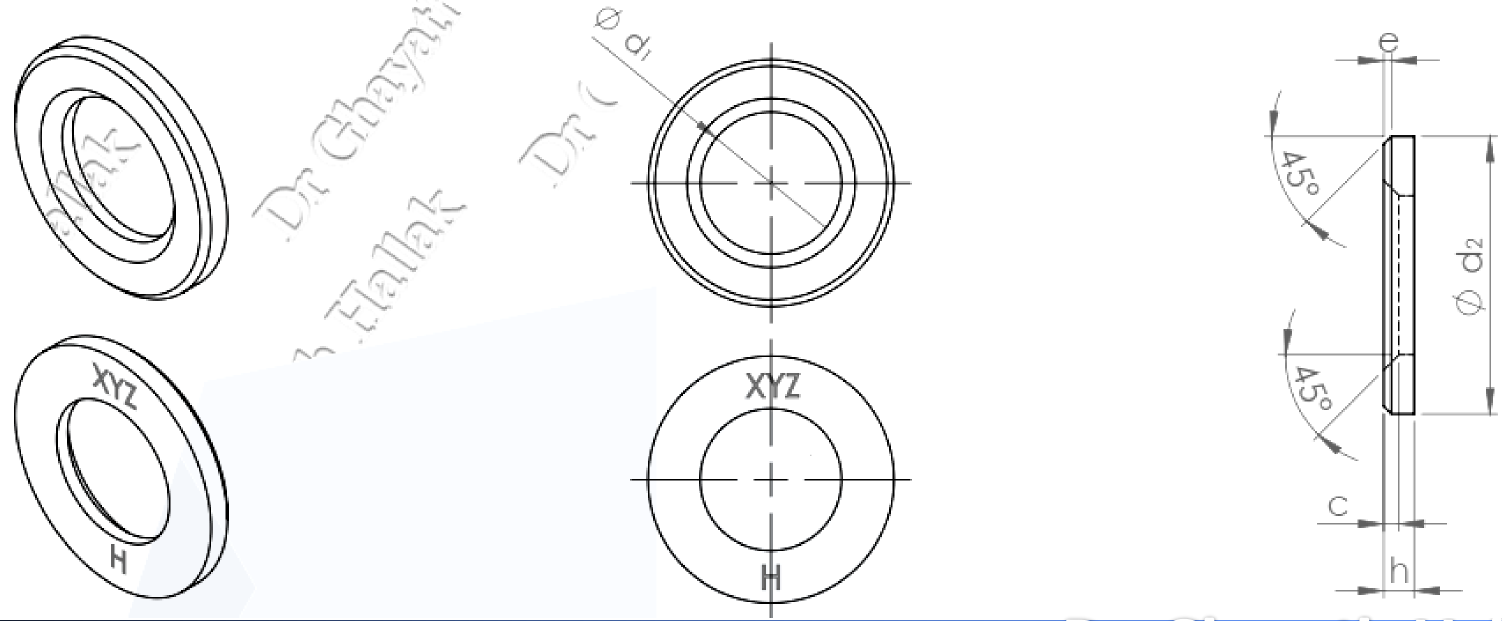


Thread *D*		M16	M20	M24	M30	M36	M42	M48	M56	M64
P_a		2	2,5	3	3,5	4	4,5	5	5,5	6
c	max.	0,80	0,80	0,80	0,80	0,80	1,00	1,00	1,00	1,00
	min.	0,20	0,20	0,20	0,20	0,20	0,30	0,30	0,30	0,30
d_a	max.	17,30	21,60	25,90	32,40	38,90	45,40	51,80	60,50	69,10
	min.	16,00	20,00	24,00	30,00	36,00	42,00	48,00	56,00	64,00
d_w	min.	22,50	27,70	33,30	42,80	51,10	60,00	69,50	78,70	88,20
e	min.	26,75	32,95	39,55	50,85	60,79	71,30	82,60	93,56	104,86
m	max.	14,80	18,00	21,50	25,60	31,00	34,00	38,00	45,00	51,00
	min.	14,10	16,90	20,20	24,30	29,40	32,40	36,40	43,40	49,10
m_w	min.	11,30	13,50	16,20	19,40	23,50	25,90	29,10	34,70	39,30
s	nom. = max.	24,00	30,00	36,00	46,00	55,00	65,00	75,00	85,00	95,00
	min.	23,67	29,16	35,00	45,00	53,80	63,10	73,10	82,80	92,80

Connections with Non-Preloaded Bolts
PUNCHING RESISTANCE

Washer Dimensions EN 14399-6 Plain Chamfered Washers

Thread	EN 14399 Part 6									
	Inside Diameter, d_1		Outside Diameter, d_2		Thickness, h		External Chamfer, e		Internal Chamfer, c	
	min.	max.	min.	max.	min.	max.	min.	max.	min.	max.
M12	13.00	13.27	23.48	24.00	2.7	3.3	0.50	1.00	1.6	1.9
M16	17.00	17.27	29.48	30.00	3.7	4.3	0.75	1.50	1.6	1.9
M20	21.00	21.33	36.38	37.00	3.7	4.3	0.75	1.50	2.0	2.5
M22	23.00	23.33	38.38	39.00	3.7	4.3	0.75	1.50	2.0	2.5
M24	25.00	25.33	43.38	44.00	3.7	4.3	0.75	1.50	2.0	2.5
M27	28.00	28.52	49.00	50.00	4.4	5.6	1.00	2.00	2.5	3.0
M30	31.00	31.62	54.80	56.00	4.4	5.6	1.00	2.00	2.5	3.0
M36	37.00	37.62	64.80	66.00	5.4	6.6	1.25	2.50	2.5	3.0

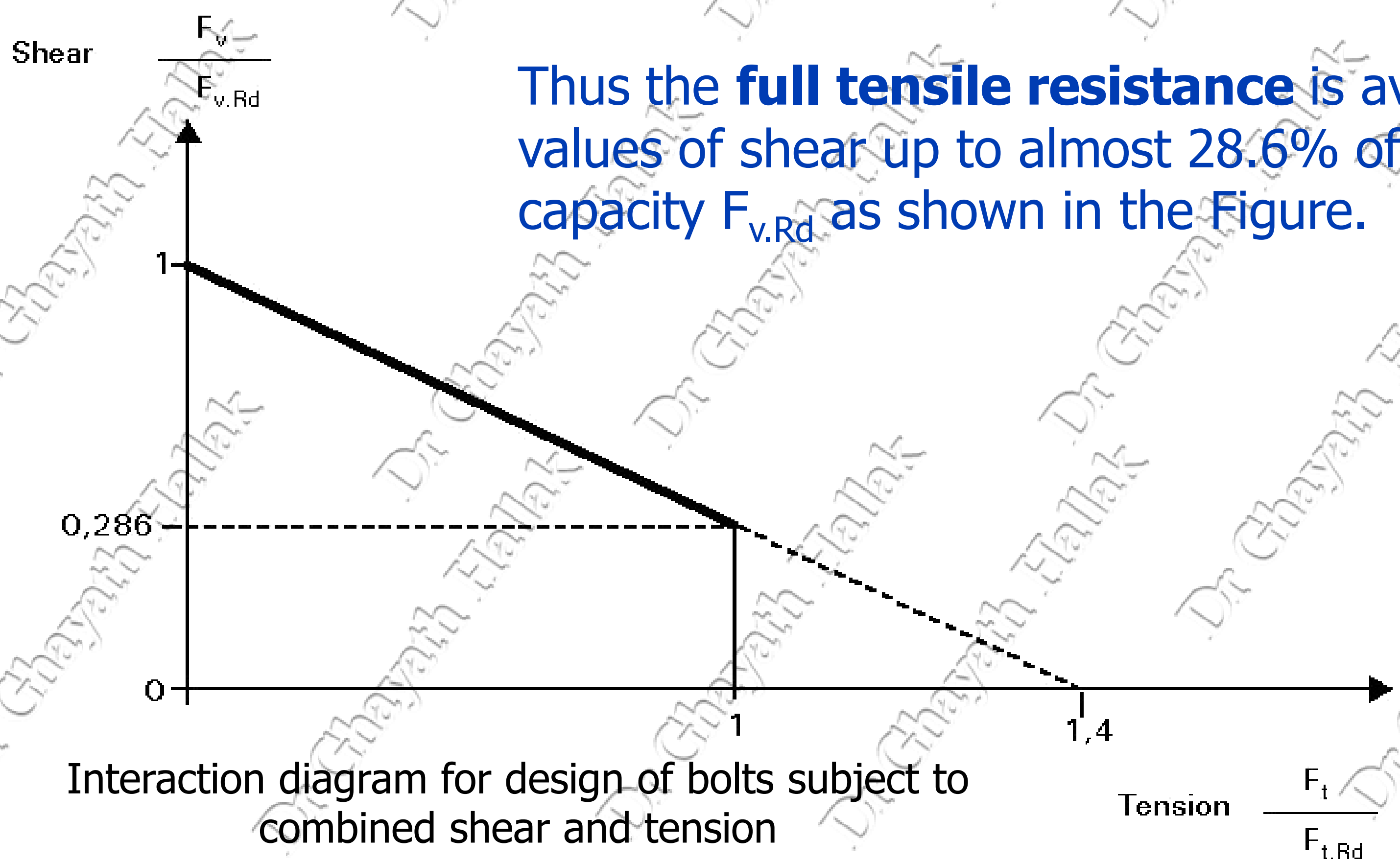


Connections with Non-Preloaded Bolts

BOLTS SUBJECT TO SHEAR AND TENSION

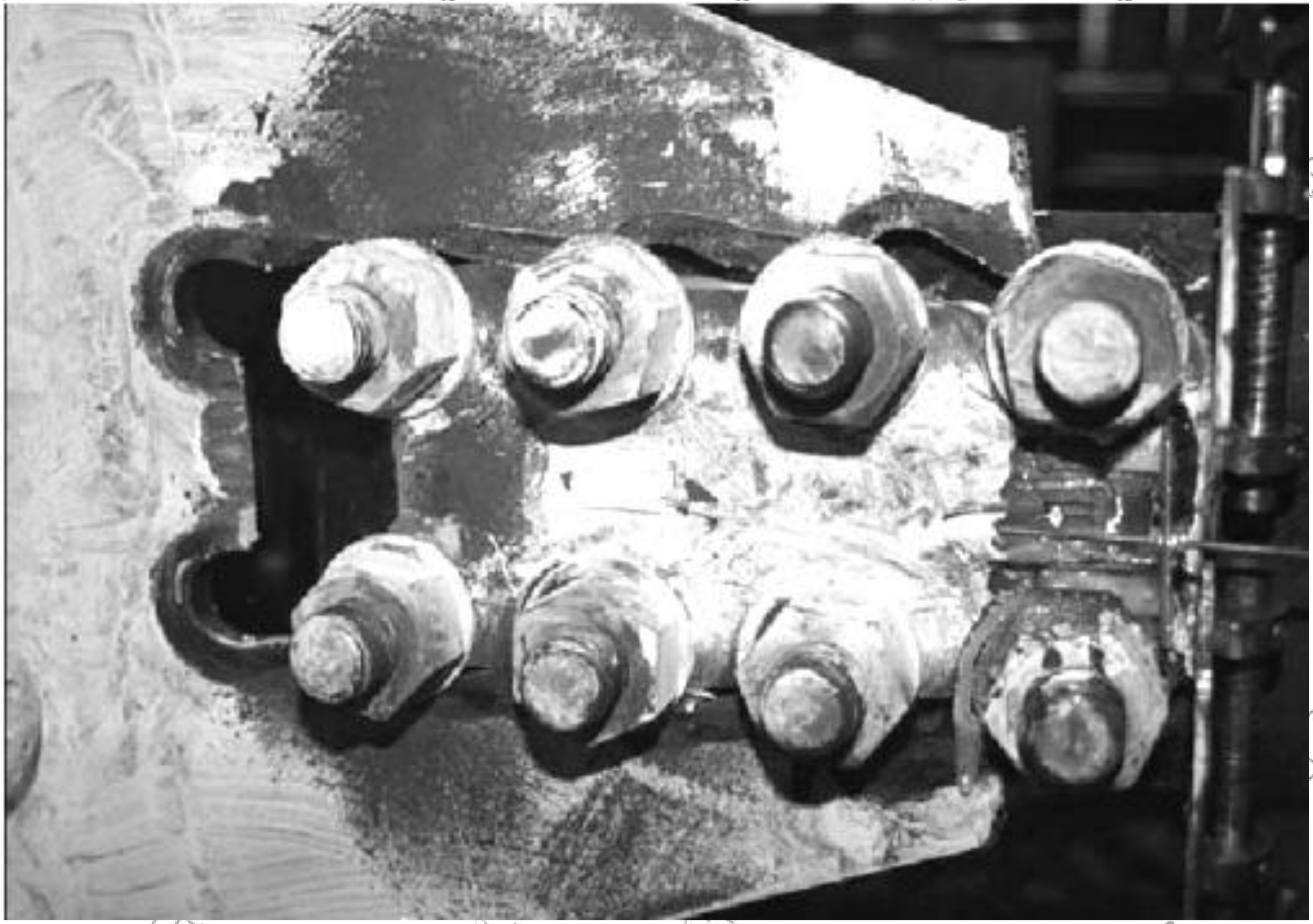
$$\frac{F_v}{F_{v.Rd}} + \frac{F_t}{1{,}4\,F_{t,Rd}} \leqslant 1{,}0$$

Thus the **full tensile resistance** is available for values of shear up to almost 28.6% of the shear capacity $F_{v.Rd}$ as shown in the Figure.



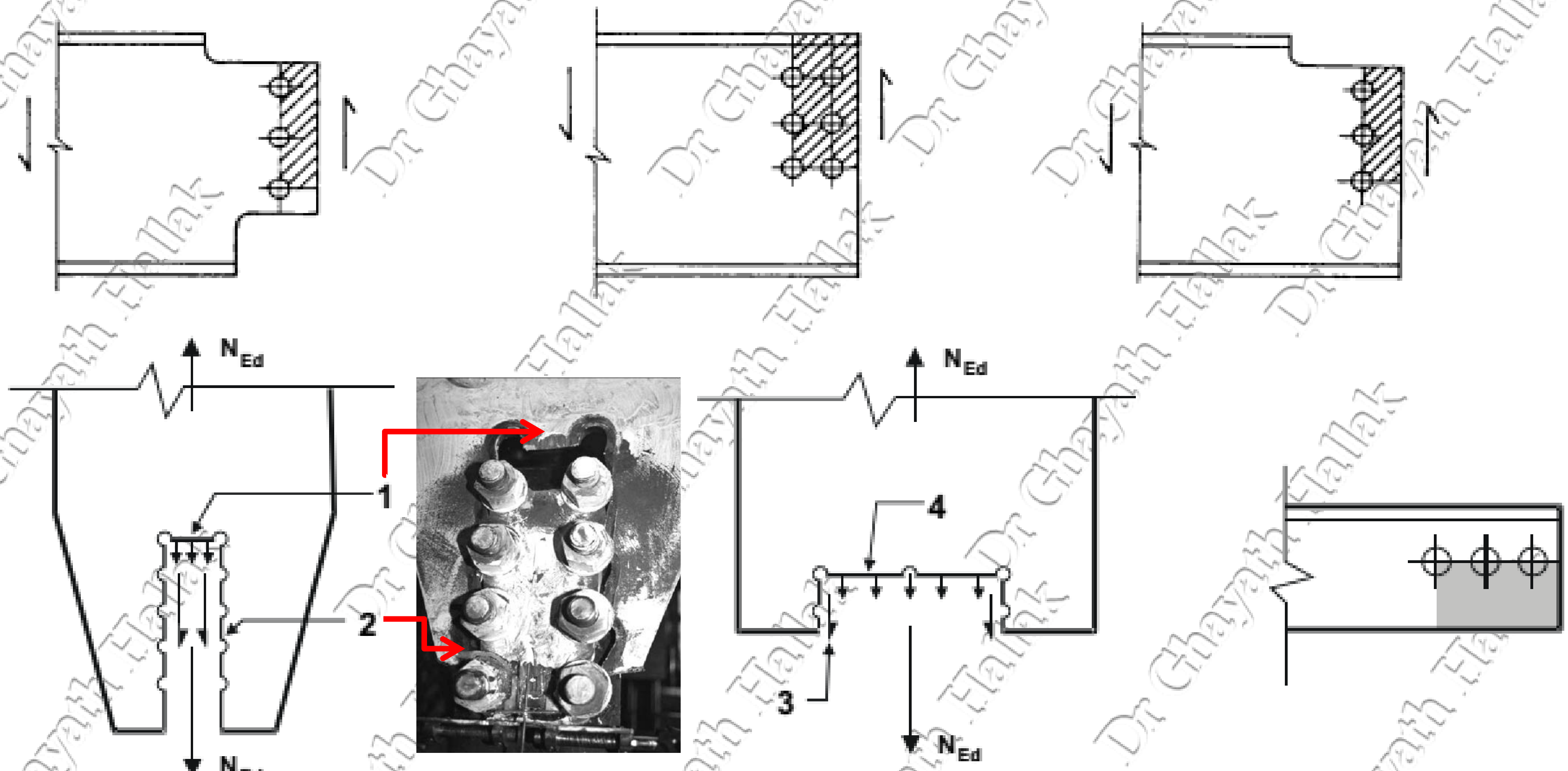
Interaction diagram for design of bolts subject to combined shear and tension

Connections with Non-Preloaded Bolts
Block tearing



Block Shear Rupture Limit State

Connections with Non-Preloaded Bolts
Block tearing

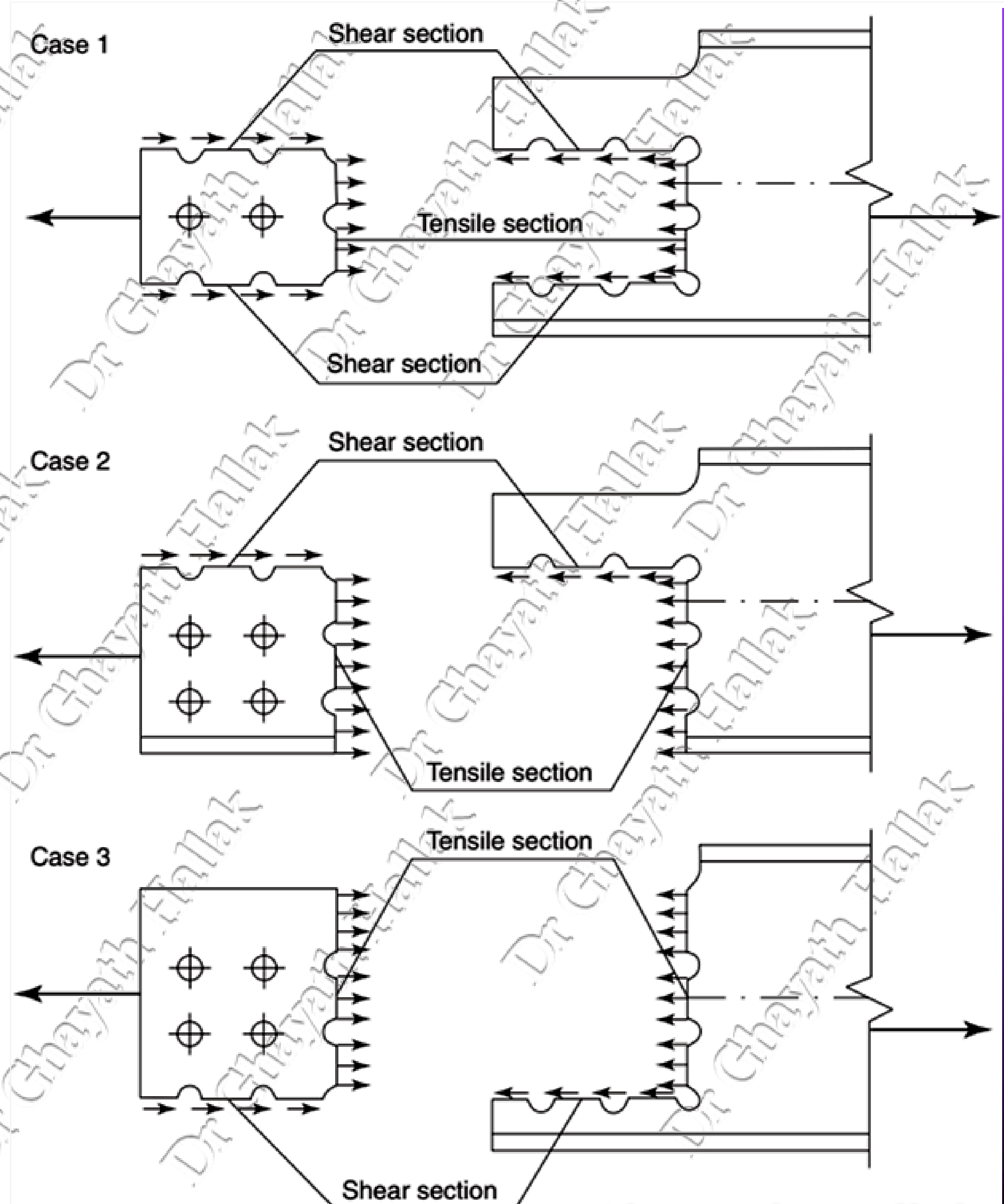


1 small tension force *2 large shear force* *3 small shear force* *4 large tension force*

Connections with Non -Preloaded Bolts

Block tearing

Block tearing consists of failure in shear at the row of bolts along the shear face of the hole group accompanied by tensile rupture along the line of bolt holes on the tension face of the bolt group.



Connections with Non-Preloaded Bolts

Block tearing **EN 1993-1-8 : 2005 close 3.10.2**

For a symmetric bolt group subject to **concentric loading** the design block tearing resistance, $V_{eff,1,Rd}$ is given by:

$$V_{eff,1,Rd} = f_u A_{nt} / \gamma_{M2} + (1 / \sqrt{3}) f_y A_{nv} / \gamma_{M0}$$

A_{nt} is net area subjected to tension;

A_{nv} is net area subjected to shear

For a bolt group subject to **eccentric loading** the design block shear tearing resistance $V_{eff,2,Rd}$ is given by:

$$V_{eff,2,Rd} = 0.5 f_u A_{nt} / \gamma_{M2} + (1 / \sqrt{3}) f_y A_{nv} / \gamma_{M0}$$

Connections with Non-Preloaded Bolts

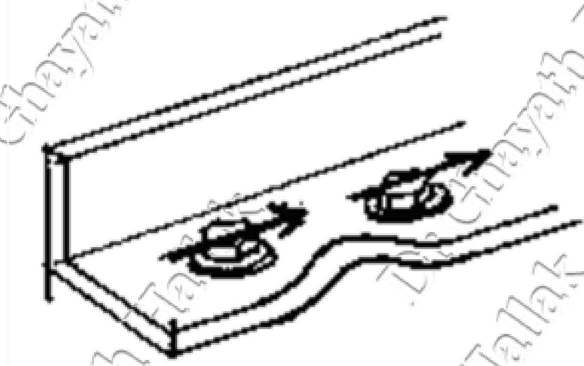
SPACING REQUIREMENTS

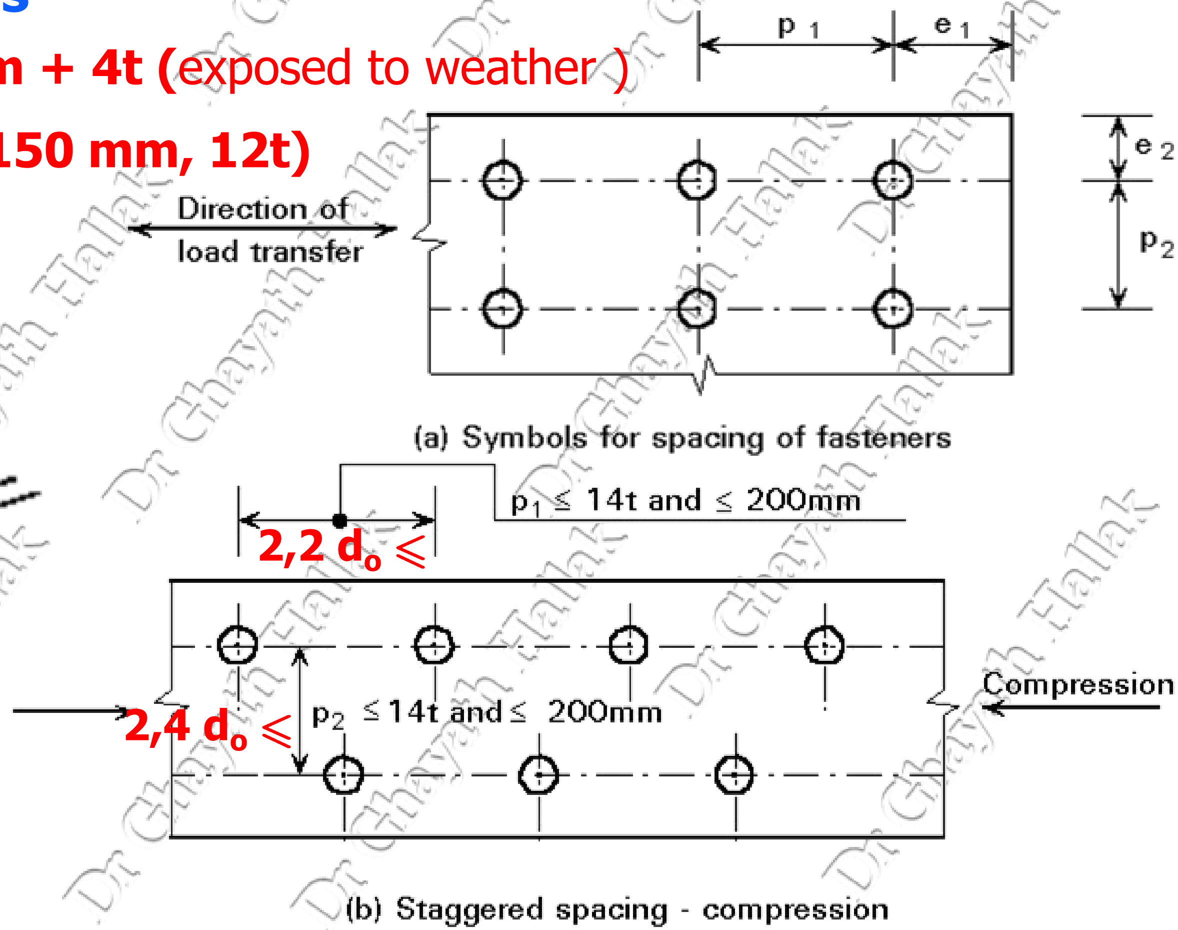
The positioning of holes for bolts should be such as to prevent corrosion and local buckling and to facilitate the installation of the bolts.

Connections of plates

$1{,}2\ d_o \leqslant e_2$ or $e_1 \leqslant 40$ mm + 4t (exposed to weather **)**

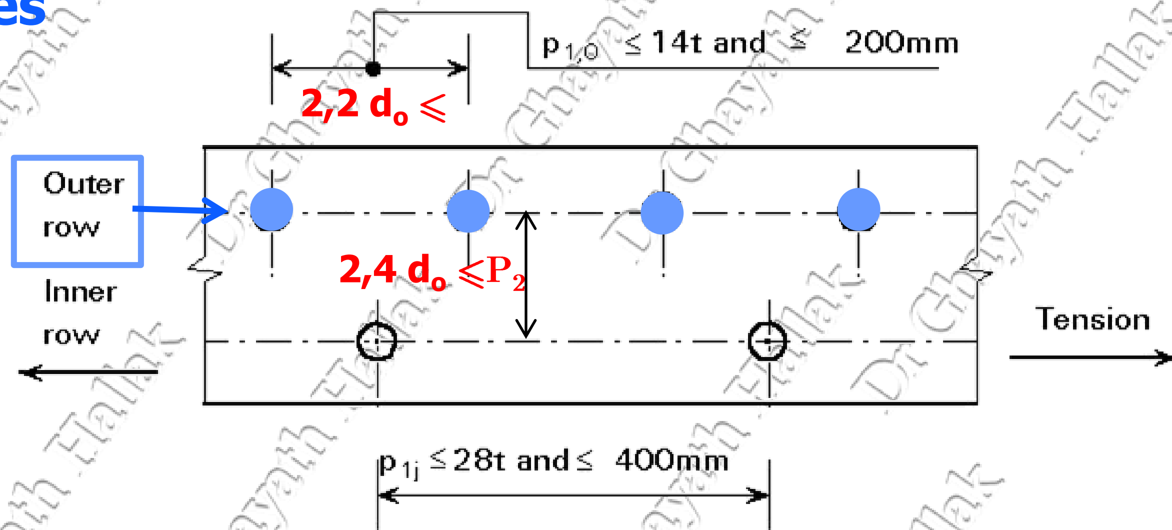
$1{,}2\ d_o \leqslant e_2$ or $e_1 \leqslant$ Max(150 mm, 12t)



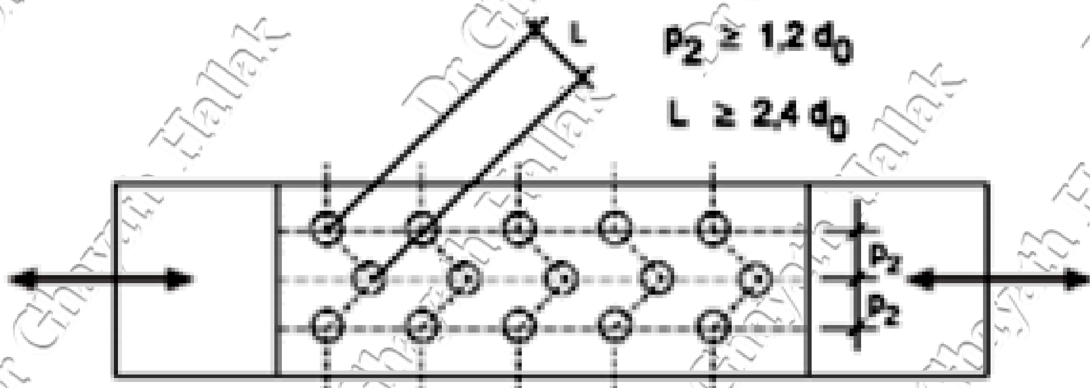


Connections with Non-Preloaded Bolts
SPACING REQUIREMENTS

Connections of plates



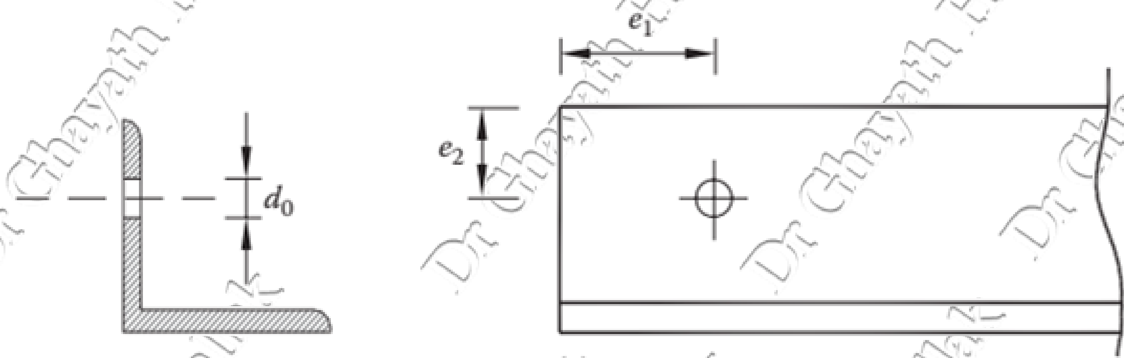
(c) Spacing in tension members



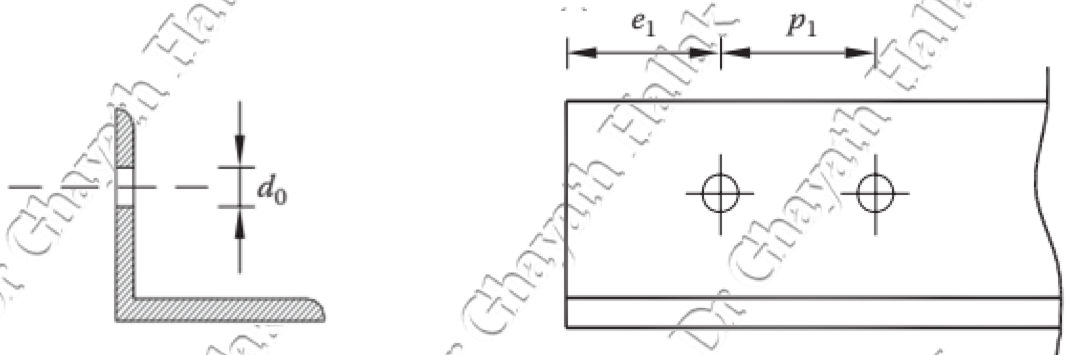
Staggered Rows of fasteners

Connections with Non-Preloaded Bolts

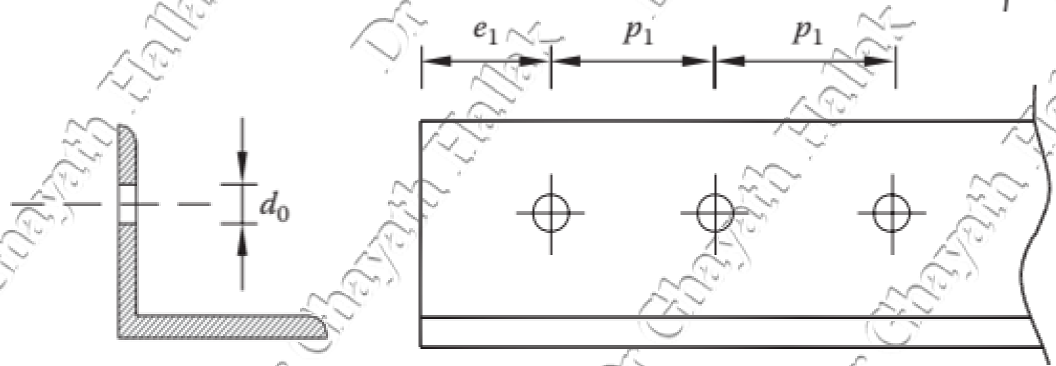
SPACING REQUIREMENTS **Angles Connected by One Leg**



$$A_{net,eff} = 2(e_2 - 0.5d_0)t$$



$$A_{net,eff} = \beta 2\ A_{net}$$



$$A_{net,eff} = \beta 3\ A_{net}$$

A_{net} is the net area of the angle.

Connections with Non-Preloaded Bolts

SPACING REQUIREMENTS

Angles Connected by One Leg

Table 3.8: Reduction factors β 2 and β 3

Pitch p_1	$\leq 2.5\ d_0$	$\geq 5.0\ d_0$
β_2	0.4	0.7
β_3	0.5	0.7

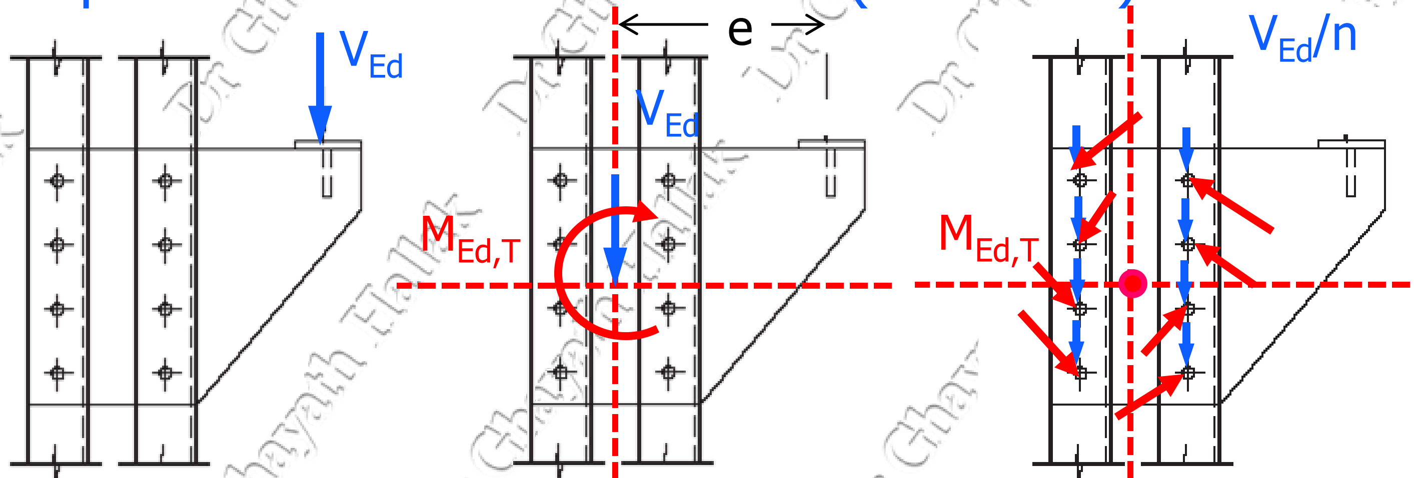
Note: For intermediate values of pitch p_1, values of β may be determined by linear interpolation.

Connections with Non-Preloaded Bolts

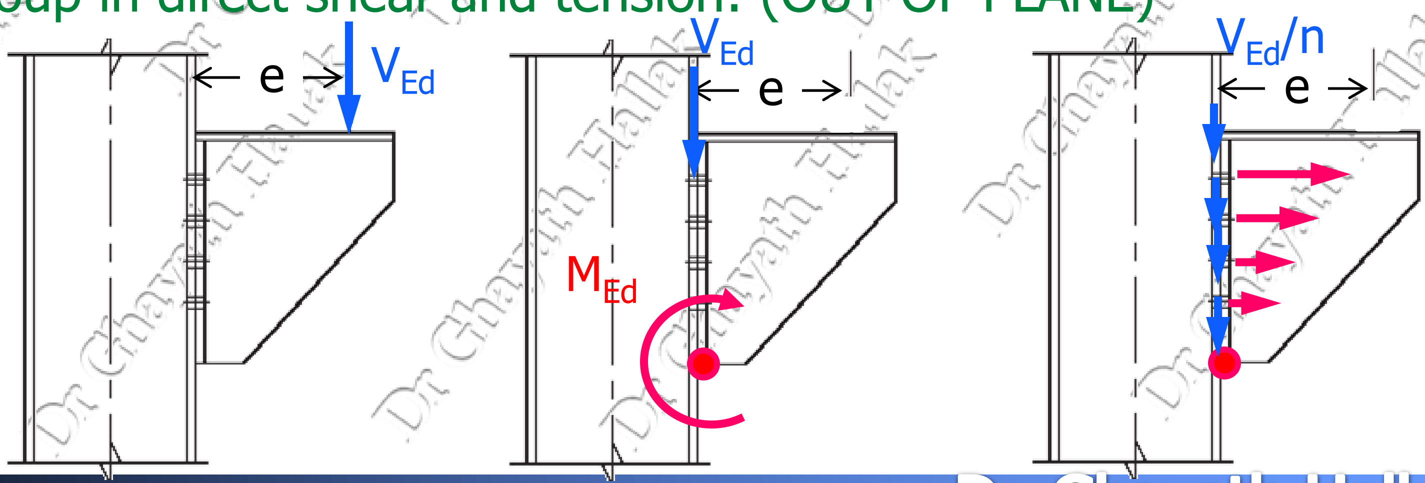
Eccentric connections

There are two principal types of eccentrically loaded connections:

1. Bolt group in direct shear and torsion: (IN PLANE)



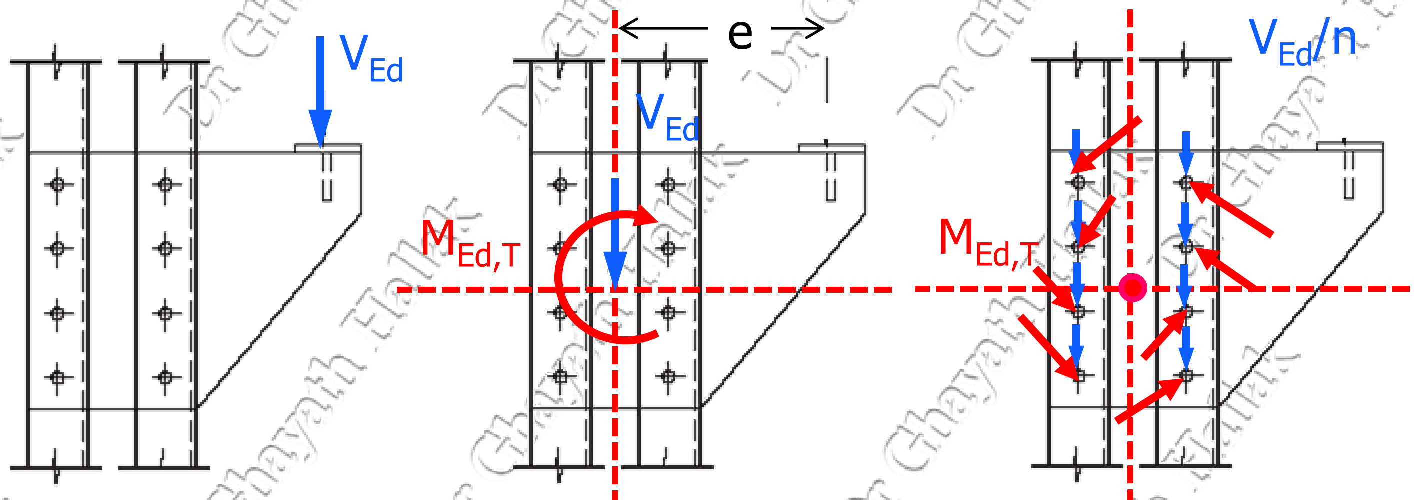
2. Bolt group in direct shear and tension: (OUT OF PLANE)



Connections with Non-Preloaded Bolts

Eccentric connections

1. Bolt group in direct shear and torsion:



❑- the moment is applied in the plane of the connection and the bolt group rotates about its centre of gravity.

❑- A linear variation of loading due to moment is assumed, with the bolt furthest from the centre of gravity of the group carrying the greatest load.

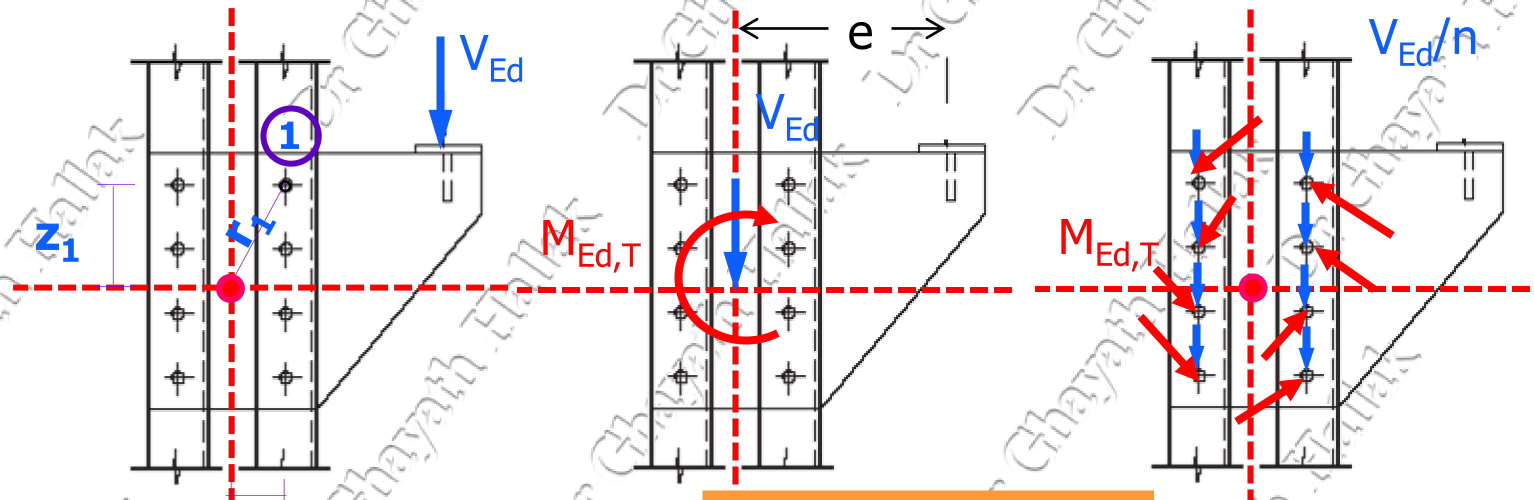
❑-The direct shear is divided equally between the bolts and the side plates are assumed to be rigid.

Connections with Non-Preloaded Bolts

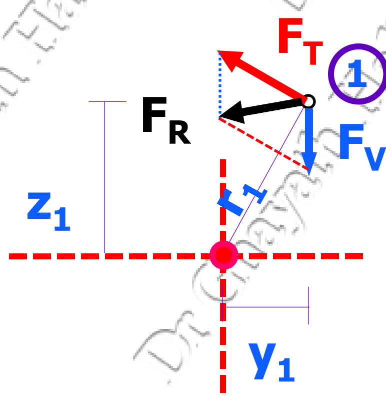
Eccentric connections

1. Bolt group in direct shear and torsion:

❑ -FORCE DUE TO TORSION



$$\tau_{M,T} = M_T \times r/I_p$$



$$I_P = (I_Y + I_Z)$$

$$I_P = (A_s \sum Z^2_i + A_s \sum Y^2_i)$$

$$I_P = A_s \times (\sum Z^2_i + \sum Y^2_i)$$

$$I_P = A_s \times \sum r^2_i$$

I_P :Polar moment of Inertia for the bolt group

n : number of bolts

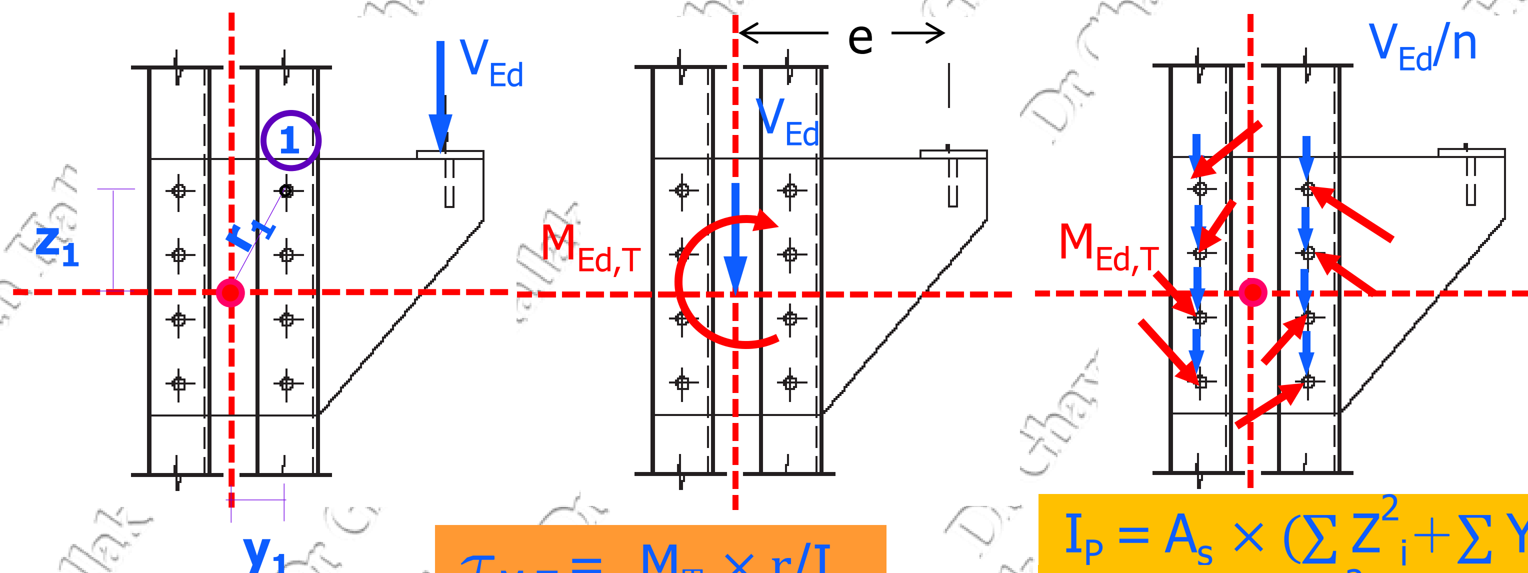
A_s: single bolt Area

Connections with Non-Preloaded Bolts

Eccentric connections

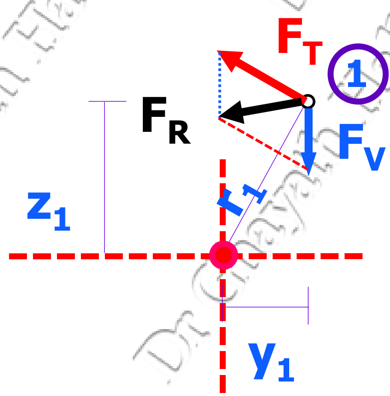
1. Bolt group in direct shear and torsion:

- -FORCE DUE TO TORSION



$$I_P = A_s \times (\sum Z^2_i + \sum Y^2_i)$$
$$I_P = A_s \times \sum r^2_i$$

$$\tau_{M,T} = M_T \times r/I_p$$



For the furthest bolt 1

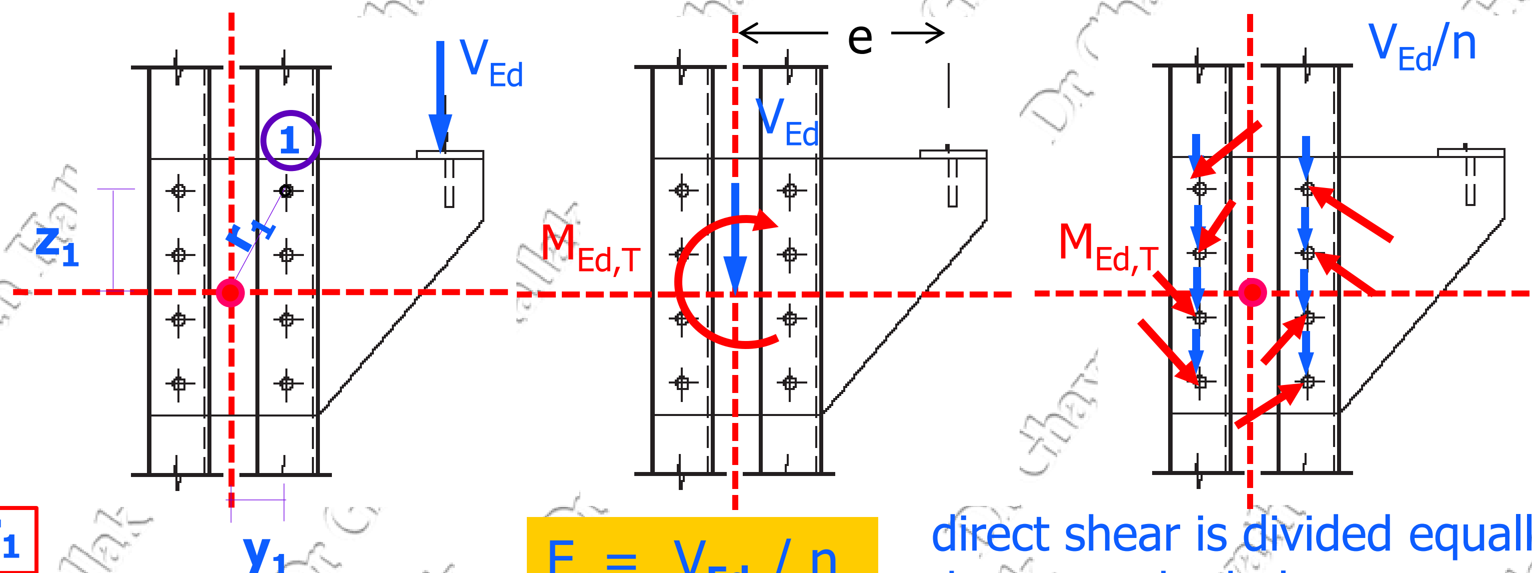
$$A_s \times \tau_{M,T} = A_s \times (M_T \times r_1/I_p)$$
$$F_T = A_s \times (M_T \times r_1/I_p)$$
$$F_T = A_s \times (M_T \times r_1/(A_s \times \sum r^2_i))$$
$$F_T = (M_T \times r_1/(\sum r^2_i))$$

Connections with Non-Preloaded Bolts

Eccentric connections

1. Bolt group in direct shear and torsion:

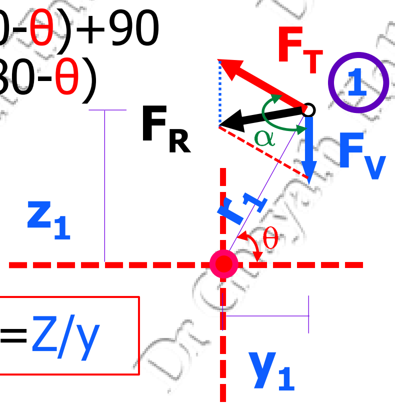
- -FORCE DUE TO VERTICAL SHEAR



$F_T \perp r_1$

$\alpha=(90-\theta)+90$

$\alpha=(180-\theta)$



$\tan\theta = Z/y$

$$F_v = V_{Ed} / n$$

direct shear is divided equally between the bolts

For the furthest bolt 1

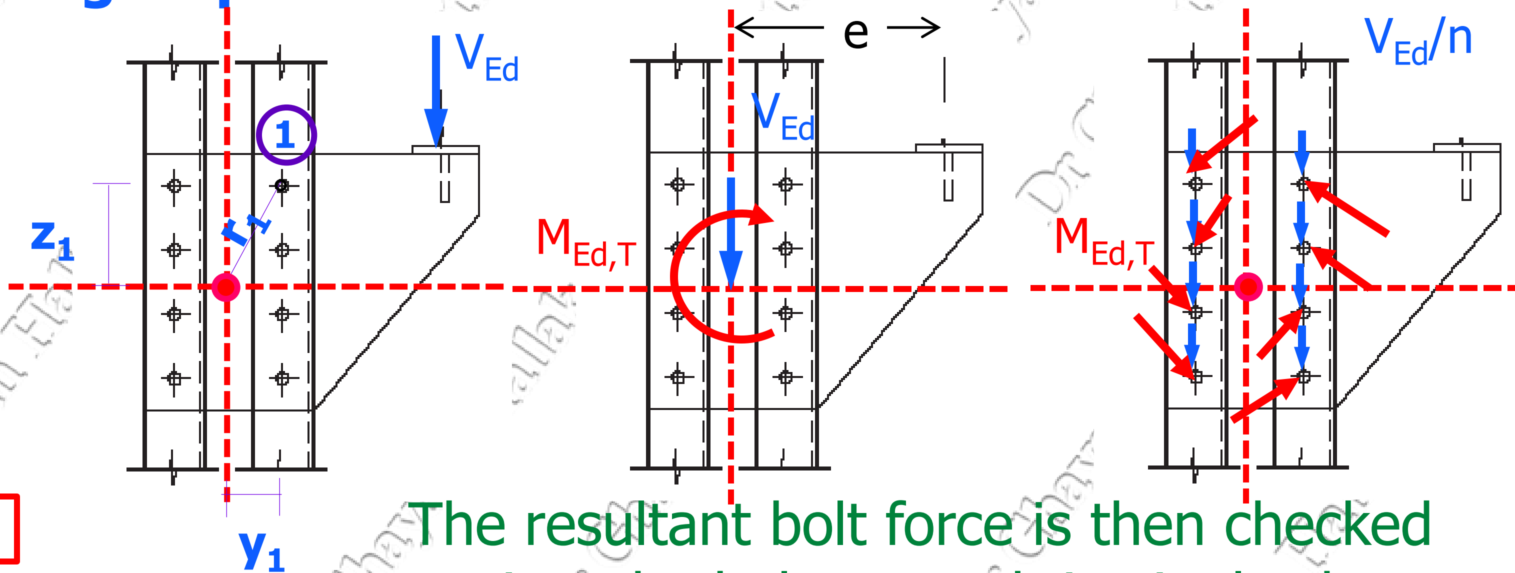
The combined force per bolt is the resultant of these two

$$F_{v,Ed}=F_R=\sqrt{(F_T^2+F_V^2+2\,F_T\,F_V\cos\alpha)}\leq F_{v,Rd}\ \&\ F_{b,Rd}$$

Connections with Non-Preloaded Bolts

Eccentric connections

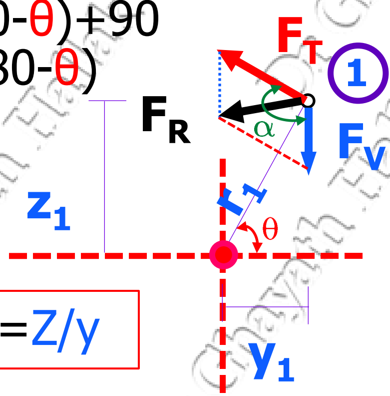
1. Bolt group in direct shear and torsion:



$F_T \perp r_1$

$\alpha=(90-\theta)+90$

$\alpha=(180-\theta)$



$\tan\theta = Z/y$

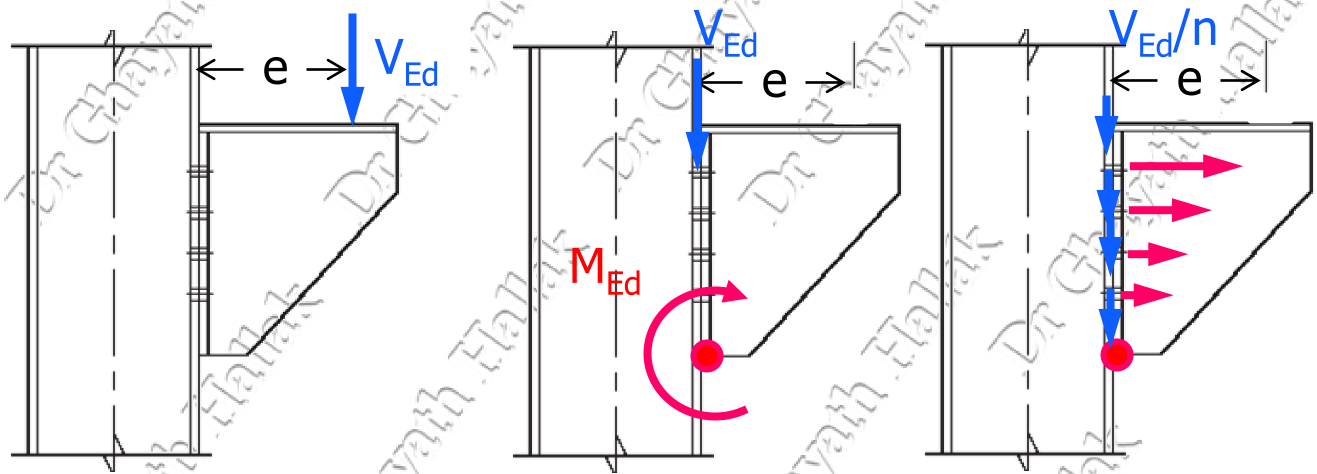
The resultant bolt force is then checked against the bolt strength in single shear, double shear or bearing as is appropriate. In the case of bearing, however, it should be remembered that the full strength cannot be achieved if the **end distance** measured along the line of the resultant **is less than twice the diameter of the bolt**.

Connections with Non-Preloaded Bolts

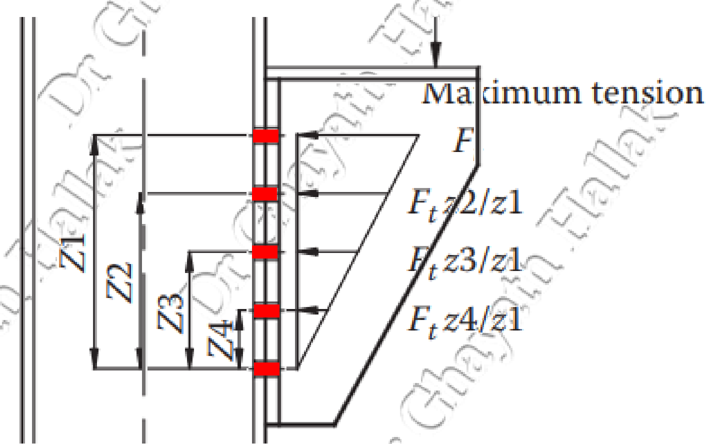
Eccentric connections

2. Bolt group in direct shear and tension. (out-of-plane)

$$F_{v,Ed} \leq F_{v,Rd}$$
$$F_{v,Ed} \leq F_{b,Rd}$$
$$F_{t,Ed} \leq F_{t,Rd}$$



$$\boxed{\frac{F_v}{F_{v.Rd}} + \frac{F_t}{1{,}4\,F_{t,Rd}} \leq 1{,}0}$$



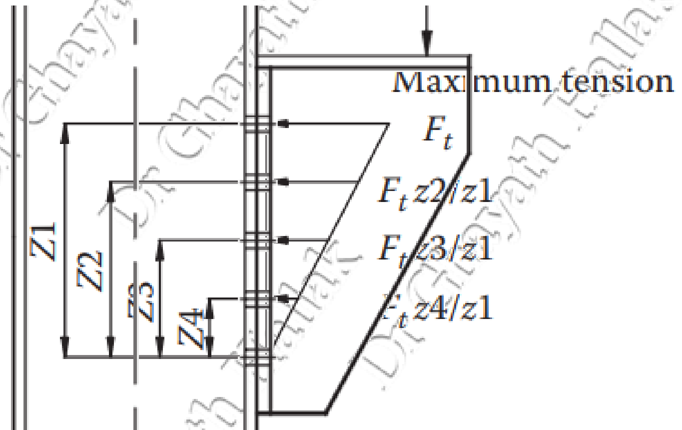
Approximate Method

The centre of rotation is assumed to be at the bottom bolt in the group

Connections with Non-Preloaded Bolts

Eccentric connections

2. Bolt group in direct shear and tension.



$$M_R = 2[F_t \times Z_1 + F_t \times Z_2 \times Z_2 / Z_1 + \cdots]$$

$$= 2F_t / Z_1 \cdot [Z^2{}_1 + Z^2{}_2 + \ldots]$$

$$M_R = (2F_t / Z_1).\sum Z^2$$

$$M_R = P \, . \, e$$

The maximum bolt tension is

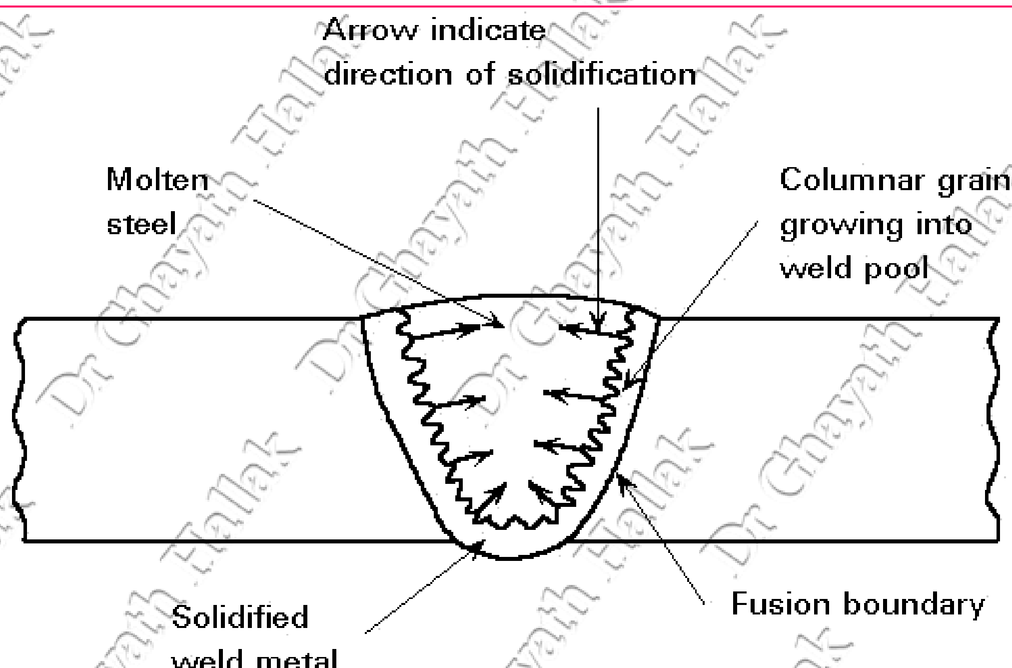
$$F_t = (P \, . \, e \, . \, Z_1) / (2 \sum Z^2)$$

The vertical shear per bolt is

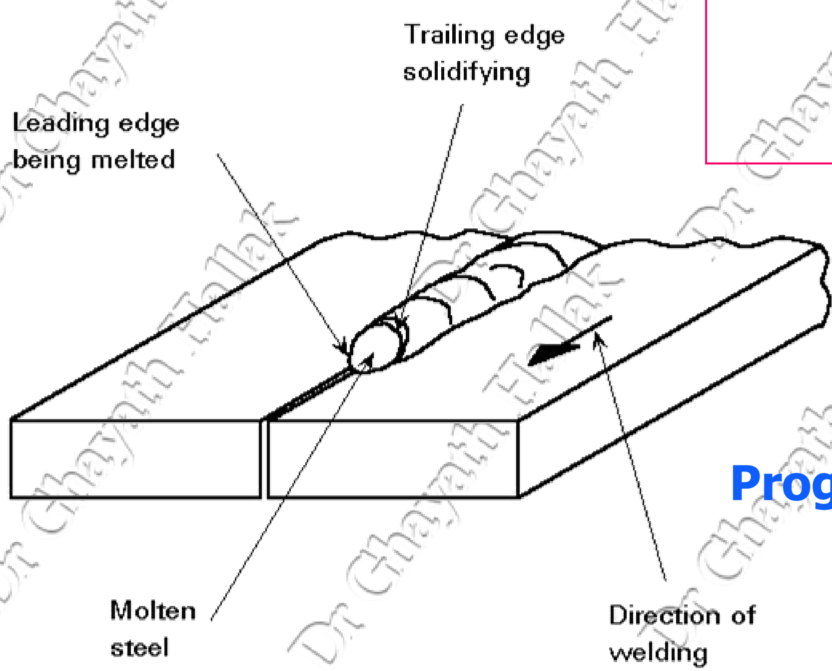
$$F_s = P / \text{no. of bolts}$$

Welded Connections

Welding is the process of joining metal parts by fusing them and filling in with molten metal from the electrode



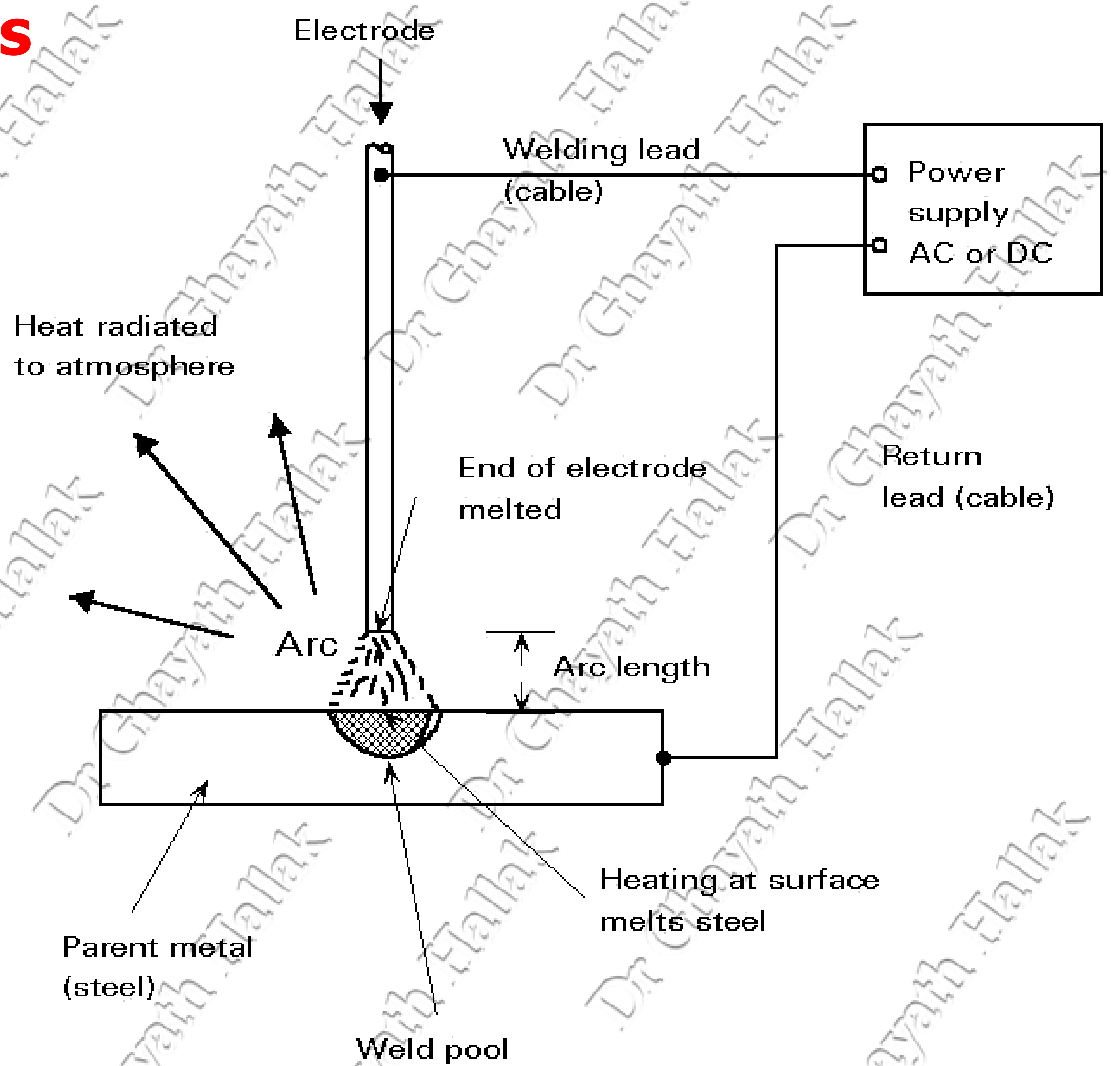
Solidification of weld metal



Progressive fusion and solidification

Welded Connections

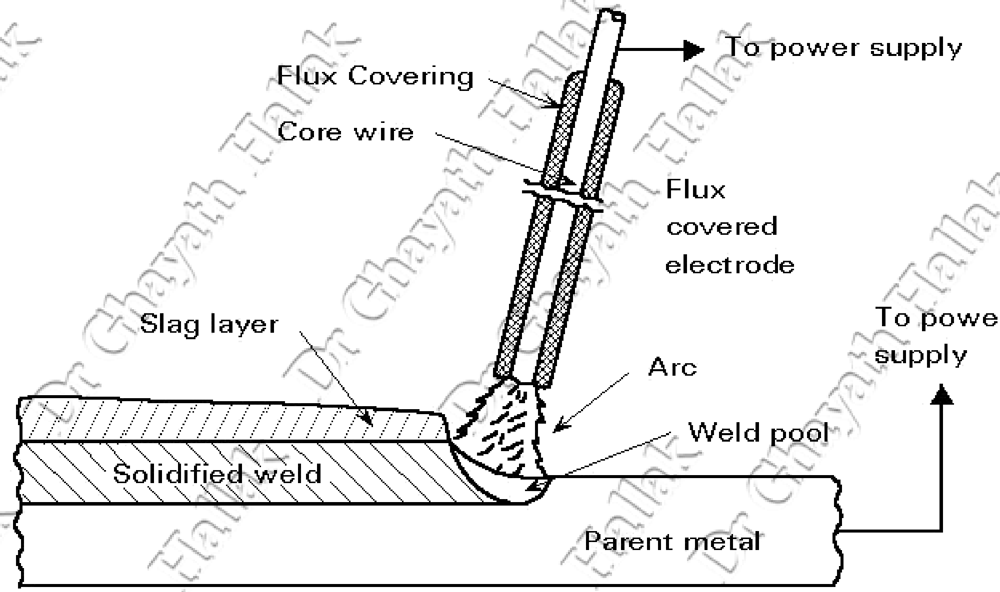
The most commonly used heat source, in structural work, is a low voltage (15 to 35 volt), high current (50 to 1000 amp) arc.



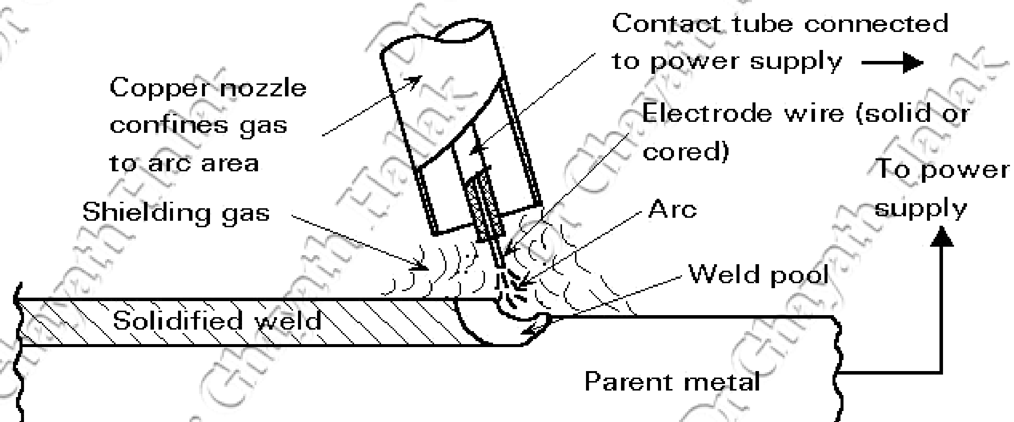
Welding Arc

Welded Connections

The molten steel in the pool will readily absorb oxygen and nitrogen from the air, which could lead to porosity in the solidified weld and possibly to metallurgical problems. The Figure shows how this is avoided by covering the pool with a molten flux, as in Manual Metal Arc (MMA) and Submerged Arc Welding (SAW), or by replacing the air around the arc by a non-reactive gas, as in Metal Active Gas (MAG) Welding or cored wire welding.



Flux shielding in MMA



Gas shielding in MAG

Shielding methods

Welded Connections

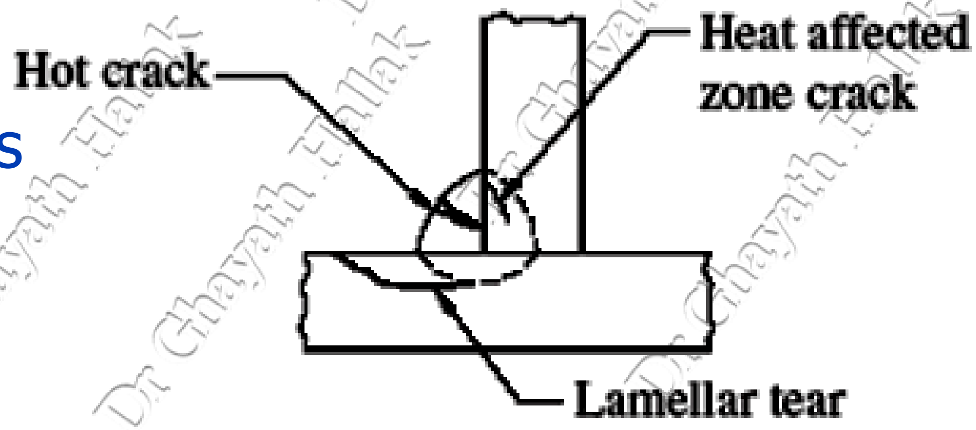
Advantages of welding

Welding offers many advantages over other joining methods:

- Freedom of design, and the opportunity to develop innovative structures;
- Easy introduction of stiffening elements;
- Less weight than in bolted joints because fewer plates are required;
- Welded joints allow increased usable space in a structure;
- Protection against the effects of fire and corrosion are easier and more effective.

Defects of welding

- Cracks can occur in welds and adjacent parts of the members being joined

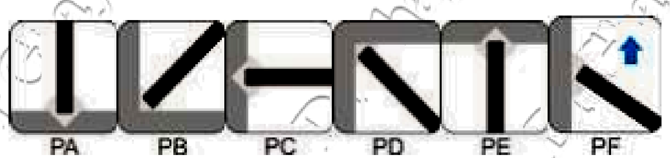


Welded Connections

European Electrode Classification System

Based on EN ISO 2560 (EN 499)

E	42	0	RR	1	2

E	covered electrode
42	The Yield strength. For electrodes suitable for multi-run welding, symbol" 35, 38, 42, 46, 50" is used to indicated a minium yield strength of 355 N/mm², 380 N/mm², 420 N/mm², 460 N/mm², or 500 N/mm², respectively.
0	Symbol for impact properties of allweld metal (Z, A, 0, 2, 3, 4, 5, 6)
RR	Symbol for type of electrode covering A- Acid covering, C – cellulosic cobering, R- rutile covering, RR- rutile thick covering, RC – rutile-cellulosic covering, RA-rutile-acid covering, RB- rutile-basic covering, B- basic covering
1	Symbol for nominal electrode efficiency and type of current
2	Symbol for welding position 

Welded Connections

European Electrode Classification System

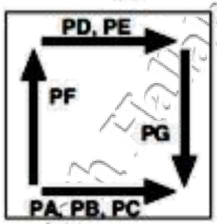
Based on EN ISO 2560 (EN 499)

E	42	0	RR	1	2

Symbol	Min Yield Strength N/mm	Tensile Strength N/mm	Minimum Elongation %
35	355	440 - 570	22
38	380	470 - 600	20
42	420	500 - 640	20
46	460	530 - 680	20
50	500	560 - 720	18

Strength and elongation symbols

Welding Positions



PA	downhand/flat position
PB	horizontal position
PC	horizontal vertical position
PD	horizontal overhead position
PE	overhead position
PF	vertical position up
PG	vertical position down

Welding Positions

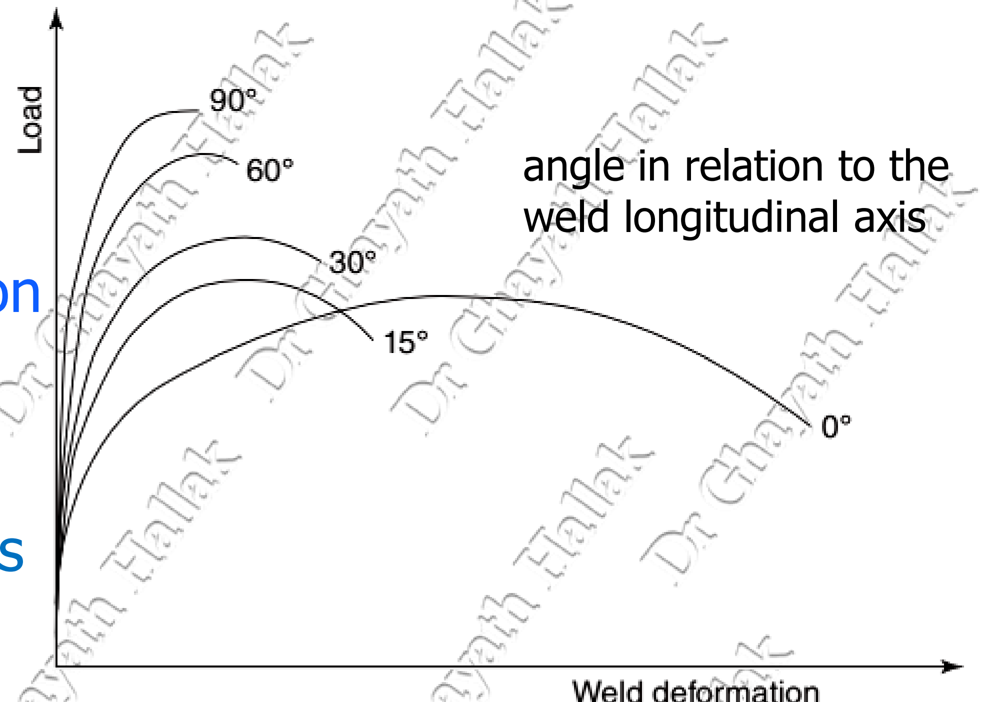
Symbol	Positions
1	PA, PB, PC, PD, PE, PF, PG
2	PA, PB, PC, PD, PE, PF
3	PA, PB
4	PA
5	PA, PB, PG

Symbol for welding position

Welded Connections

Failure of Welds

- Welds have an extremely limited capacity of deformation



Weld deformation depending on the load angle.

It is therefore common practice that, for small-to-medium carpentry jobs, plates are welded to completely restore the full resistance (**two fillets with a throat of 5 mm restore a 10-mm plate with good approximation**). This kind of design (full strength) also means that the checks are omitted in the calculation reports.
For jobs of medium- to large-sized structures and for moment connections, the verification is required for both safety reasons and to avoid unnecessary over sizing.

Welded Connections

Failure of Welds

The exact result that is obtained in calculating the **thickness of double fillet welds** which guarantees the **full strength** of connected plates is, depending on the quality of the material, **a throat thickness greater than**

0.46 times the thickness for S235,
0.48 for S275, and
0.55 for S355.

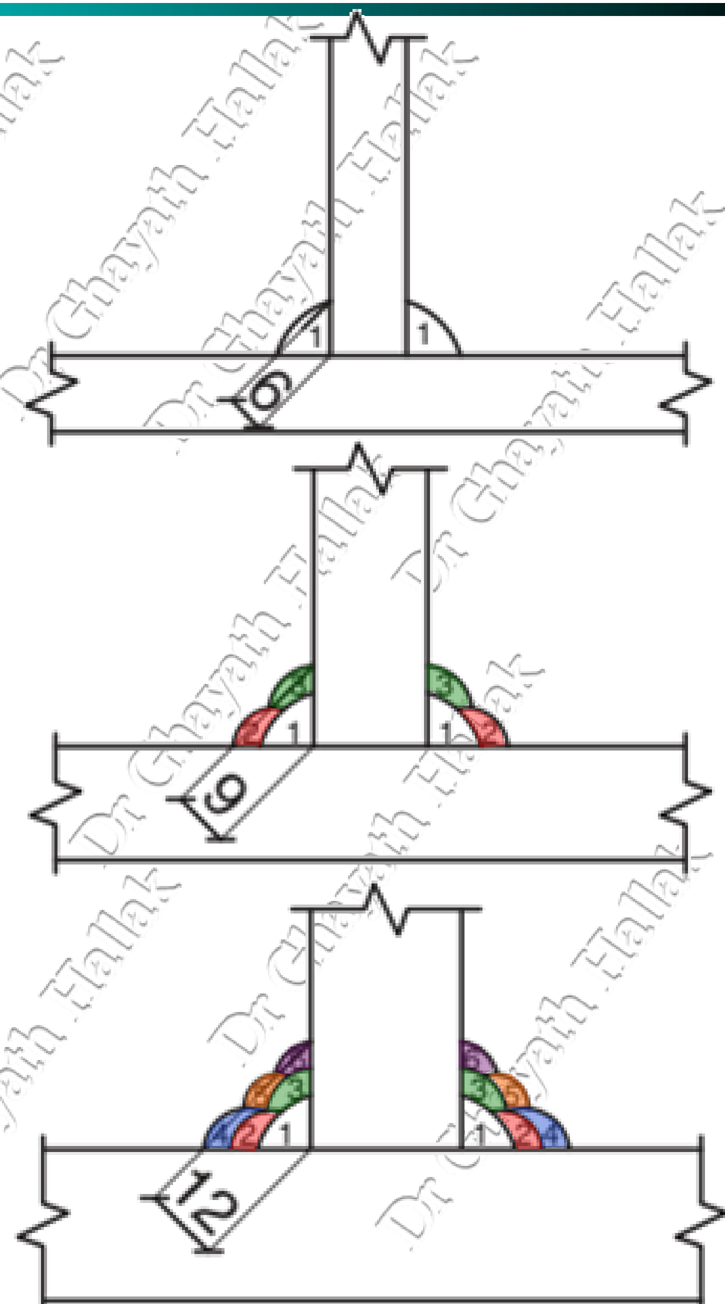
For **S420 and S460** materials, each fillet must be greater than values between **0.68 and 0.74** times the thickness.

The economy of the welds is the fact that large welds require multiple runs (also known as "passes"). In fact, up to a throat of about **6 mm** (1⁄4 in.) a **single pass** may suffice, but for greater thicknesses it is advisable to have multiple passes to achieve good welding quality. As Figure below illustrates, many passes are required to reach a slightly higher thickness.

Welded Connections

Failure of Welds

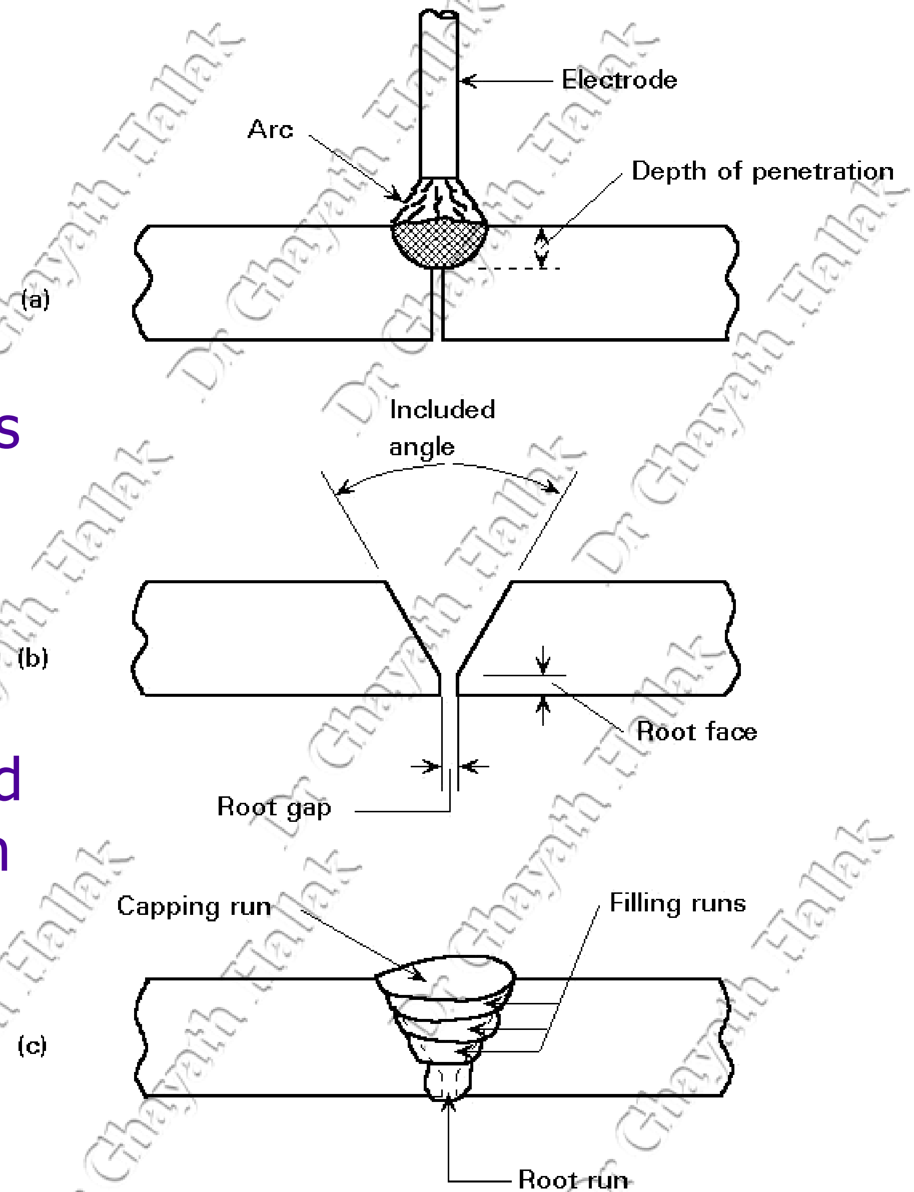
For example, a throat thickness of 9 mm requires about three passes, while one of 12 mm requires about five or six runs. This means that, to achieve a resistance equal to about 50% more than a 6 mm fillet, three times more labor is necessary (without considering that it is necessary to "clean" the various welds) and even five or six times the work for double strength. We conclude, then, wherever possible, that it is preferable to "stretch" the welded area with fillets that are not thick, rather than having very thick fillets of limited length.



Welded Connections

EDGE PREPARATION FOR BUTT WELDS

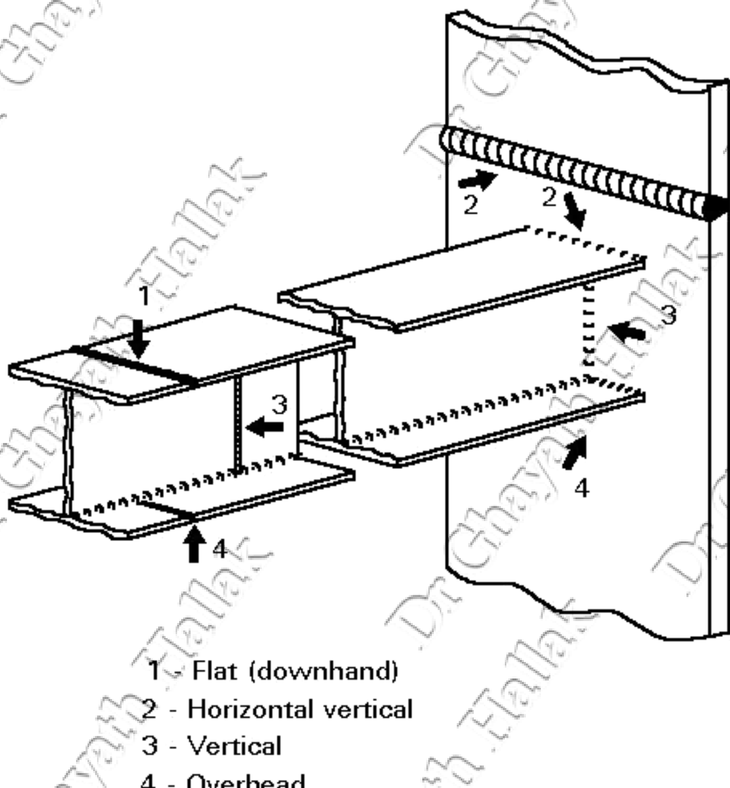
"full-penetration" weld, which will restore the strength of the connected elements but requires more preparation and control and therefore increasing costs for the fabrication shop (in contrast this solution is inexpensive for the engineer and would avoid any calculation with the simple full-penetration instruction).



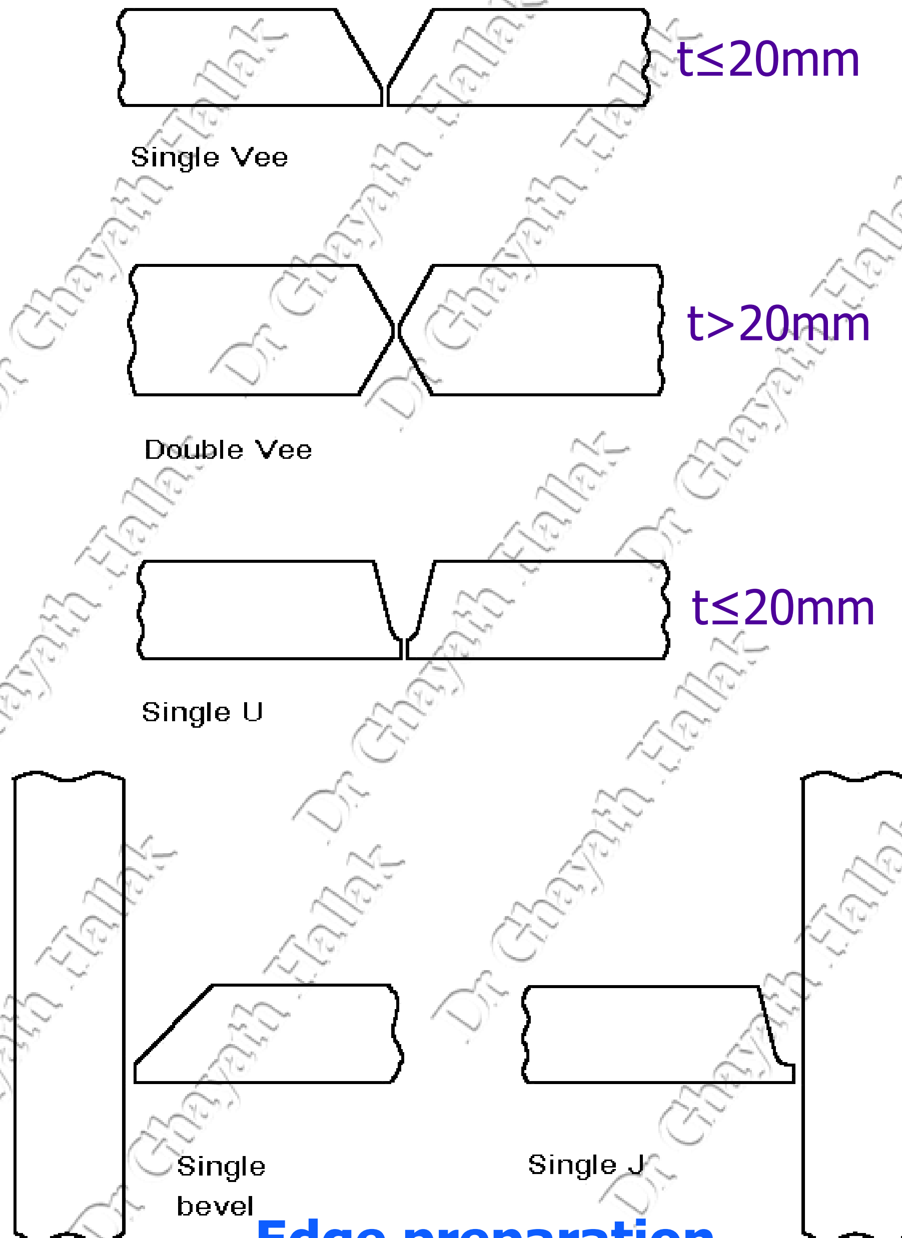
Penetration in arc welding

Welded Connections

EDGE PREPARATION and welding positions



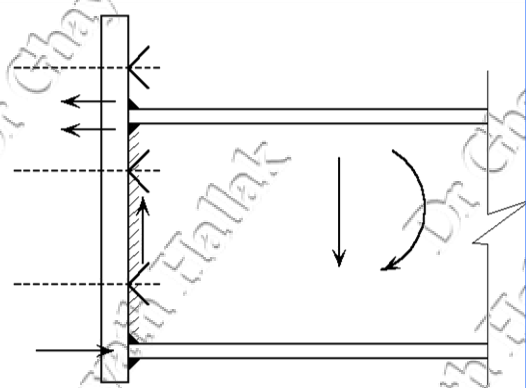
Welding positions



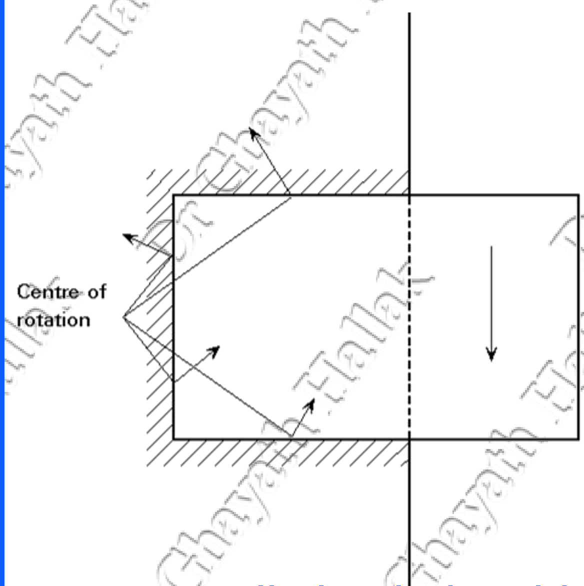
Edge preparation

Welded Connections

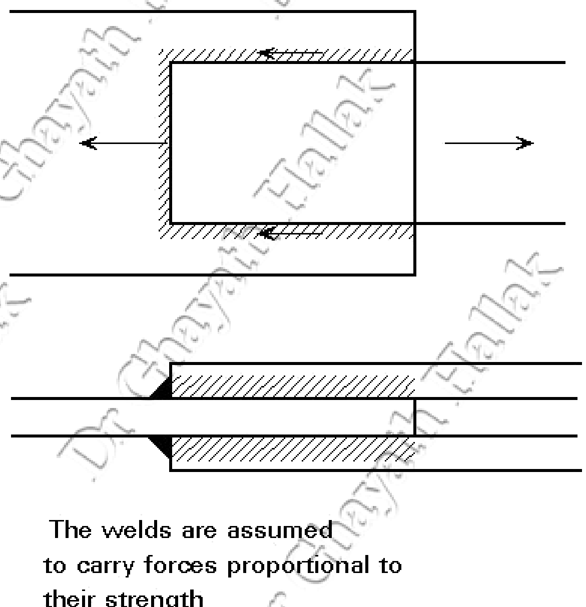
- --Where there are favourable working conditions, welding is the most economical way to make strong connections.
- --Therefore, workshop connections are usually welded.
- --Where site connections are necessary (erection) they are usually bolted, but the connections are often prepared in the workshop with welded plates, etc. necessary for the bolted joint.



Welded connection between beam and end plate



Eccentrically loaded weld group



Lap joint

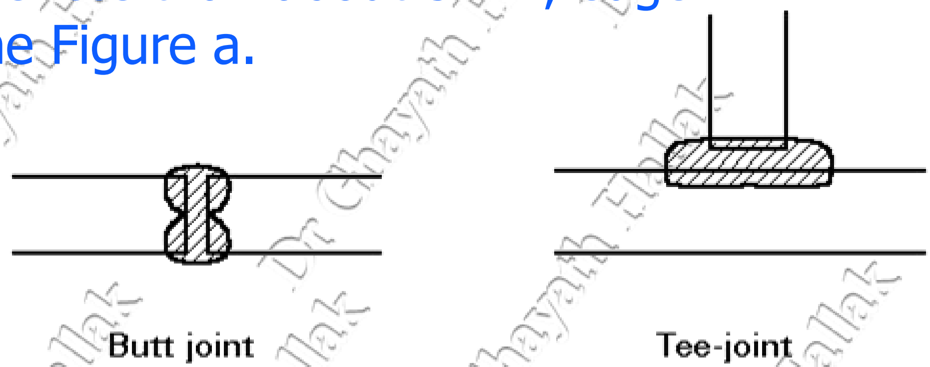
Welded Connections

TYPES OF WELDS

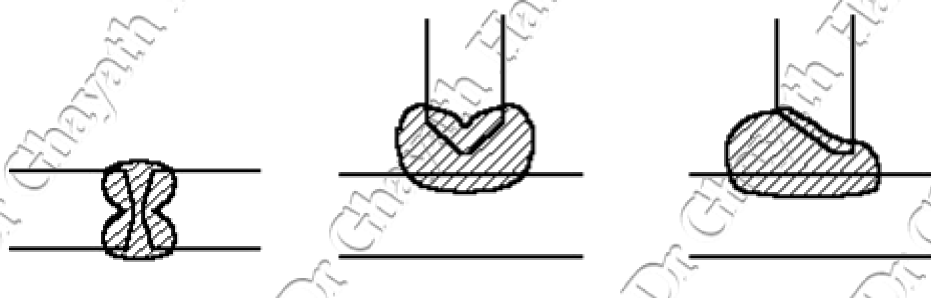
In welded construction for buildings approximately 80% of the welds are fillet welds and 15% are butt welds. The remaining 5% are plug, slot and spot welds.

1 Butt Welds

the plate edges have to be prepared before welding, see the Figure. In some cases, if the plate thickness is less than about 5mm, edge preparation can be avoided, see the Figure a.



(a) No edge preparation



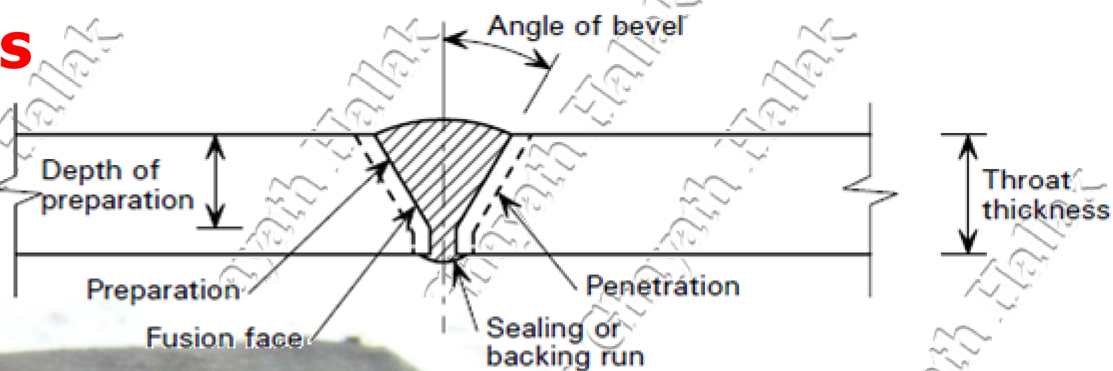
(b) Edge preparation

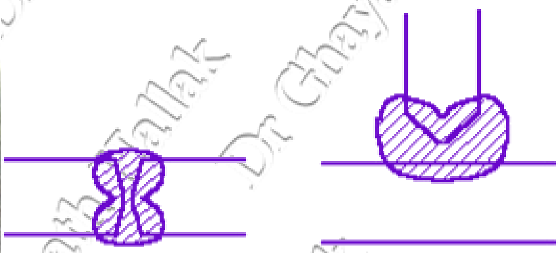
Butt Welds with full penetration

Welded Connections

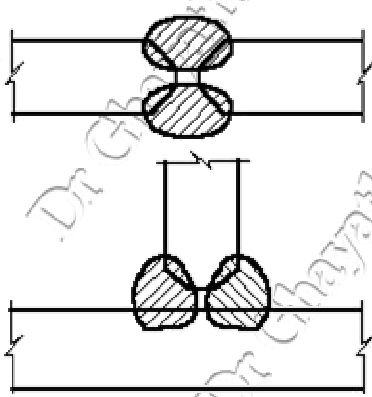
TYPES OF WELDS

1 Butt Welds





Butt welds with full penetration



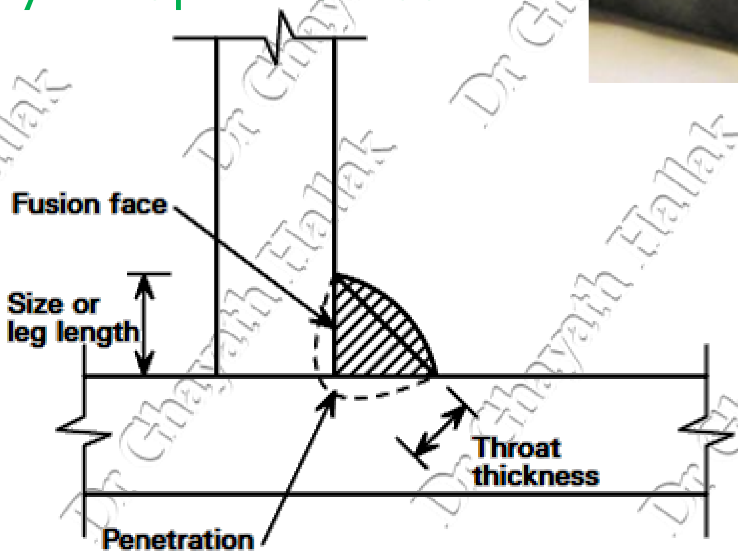
Butt welds with partial penetration

Welded Connections

TYPES OF WELDS

2 Fillet Welds

- == A fillet weld is a weld of approximately triangular cross-section applied to the surface profile of the plates.
- == No edge preparation is needed. Therefore, fillet welds are usually cheaper than butt welds.



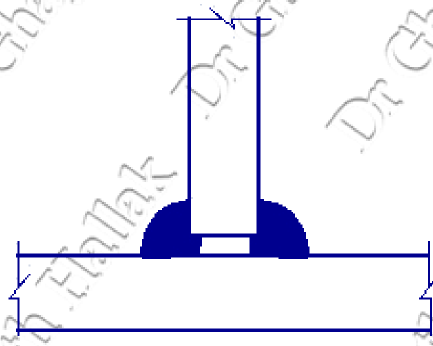
Welded Connections

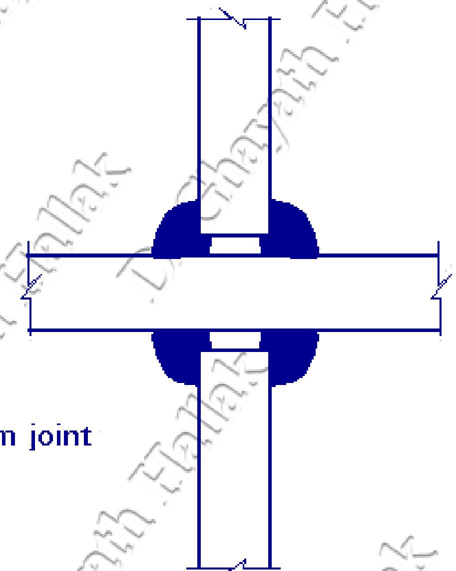
TYPES OF WELDS

2 Fillet Welds

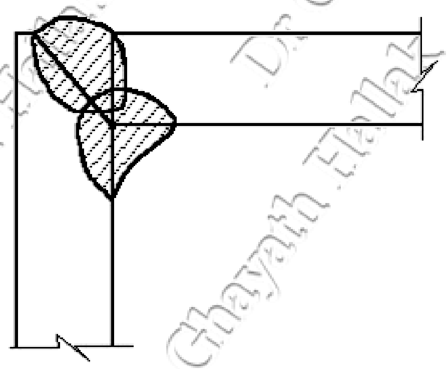
Fillet welds that can be laid in a single run are particularly economic; in the workshop 8mm welds are often possible but if site welding is to be used this figure may be reduced e.g. to 6mm.

(a) Lap joint

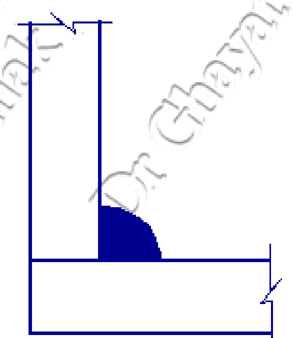




(b) Tee and cruciform joint



Corner joint with butt and fillet welds



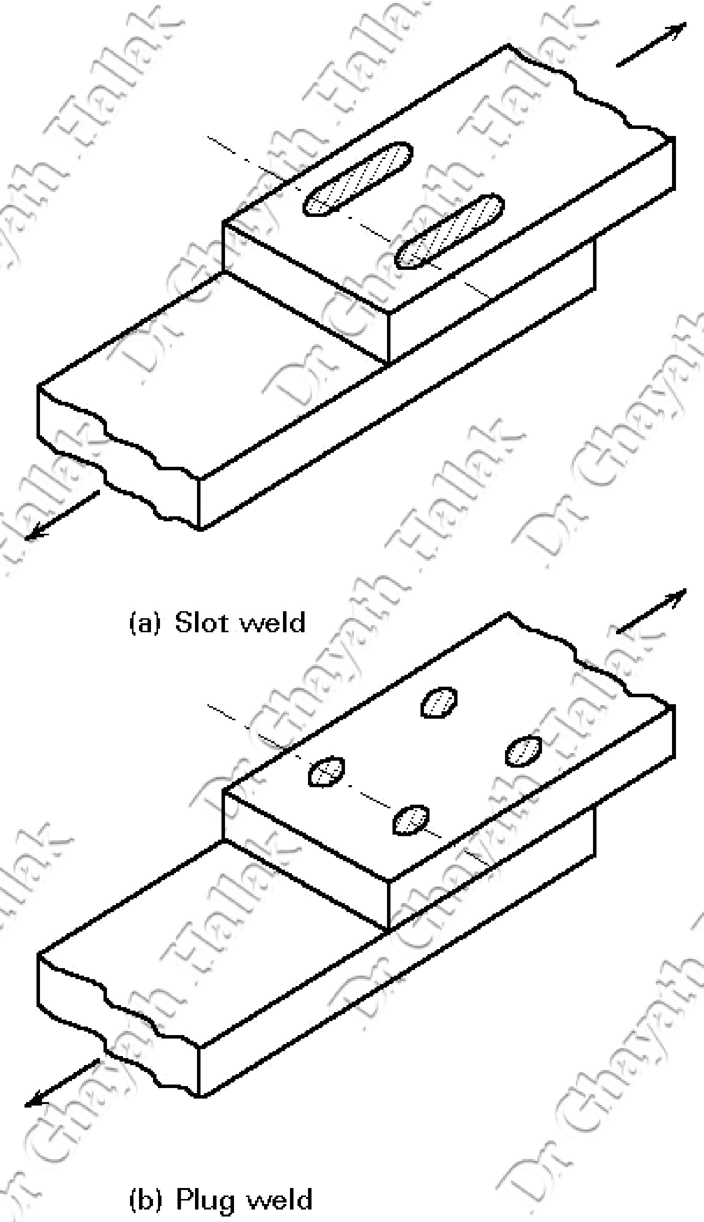
(c) Corner joint

Welded Connections

TYPES OF WELDS

3 Plug and Slot Welds

Slot and plug welds, are seldom used in building structures. They principally prevent buckling or separation of lapped plates

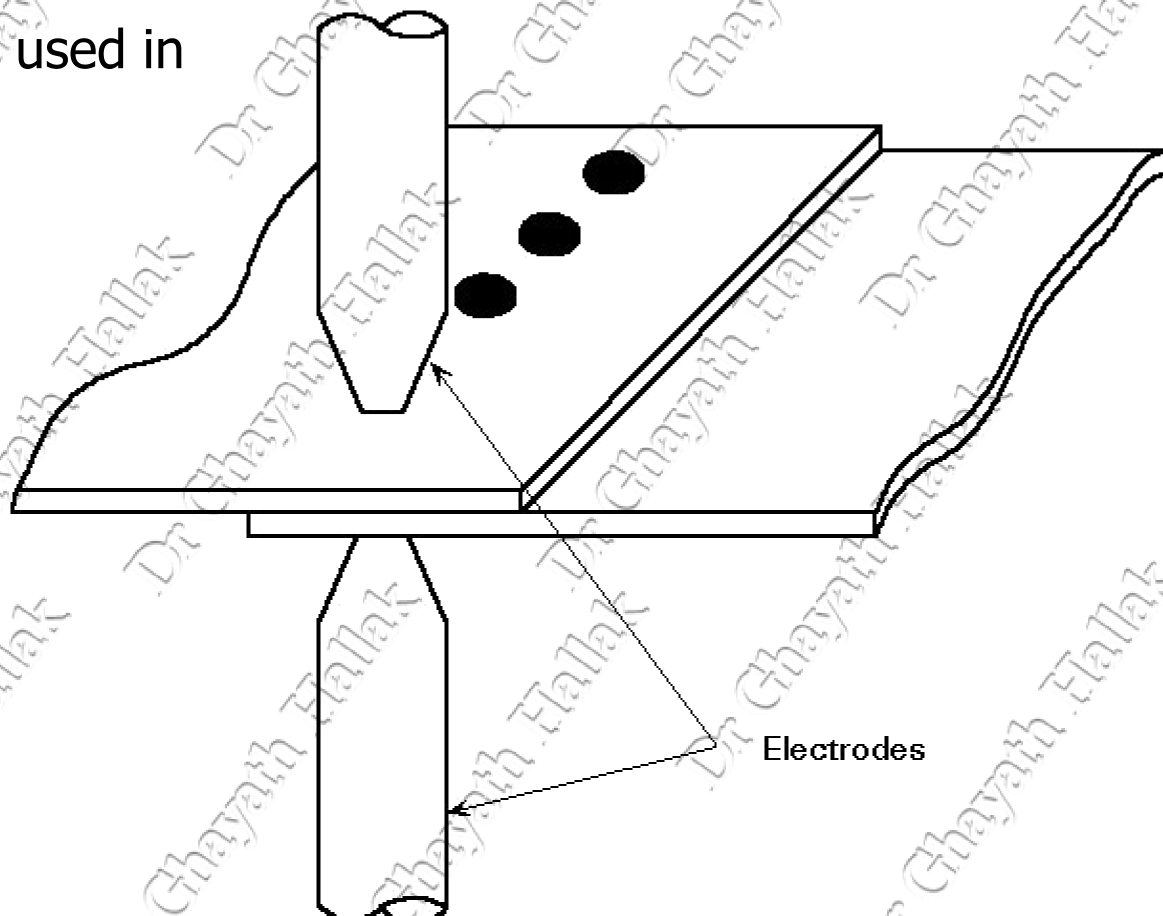


Welded Connections

TYPES OF WELDS

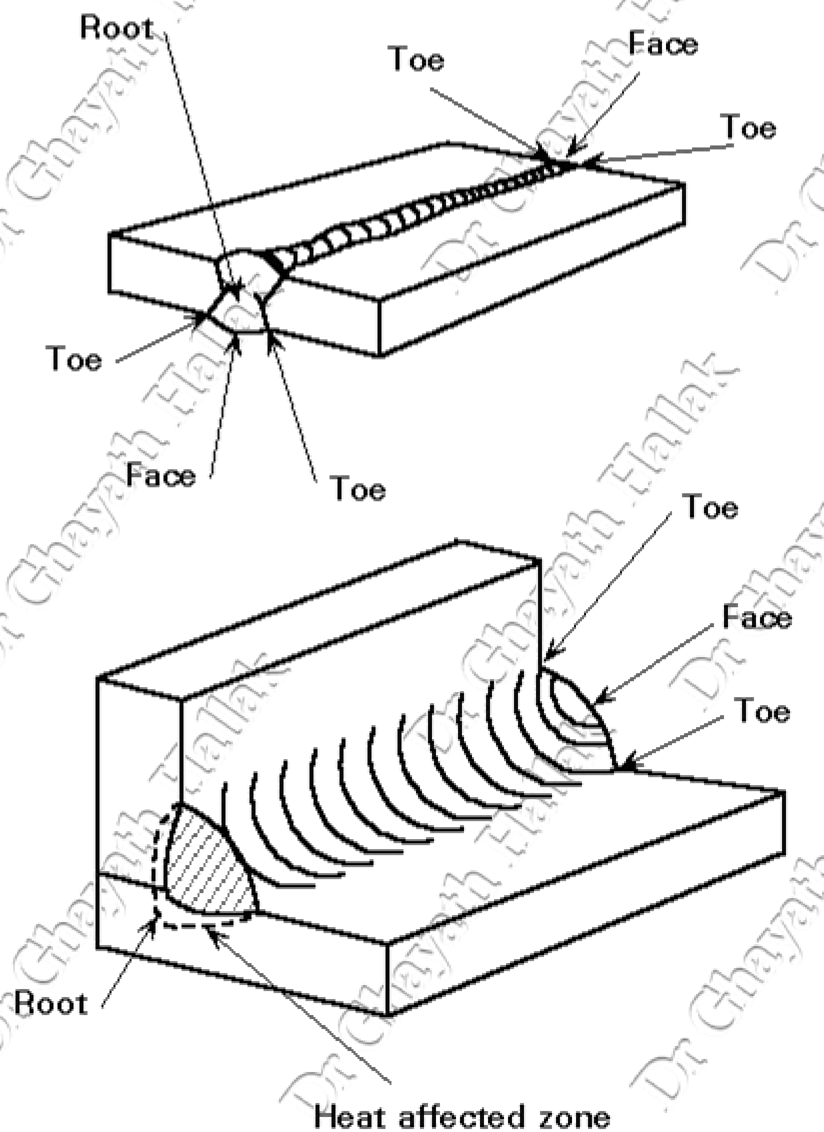
4 Spot Welds

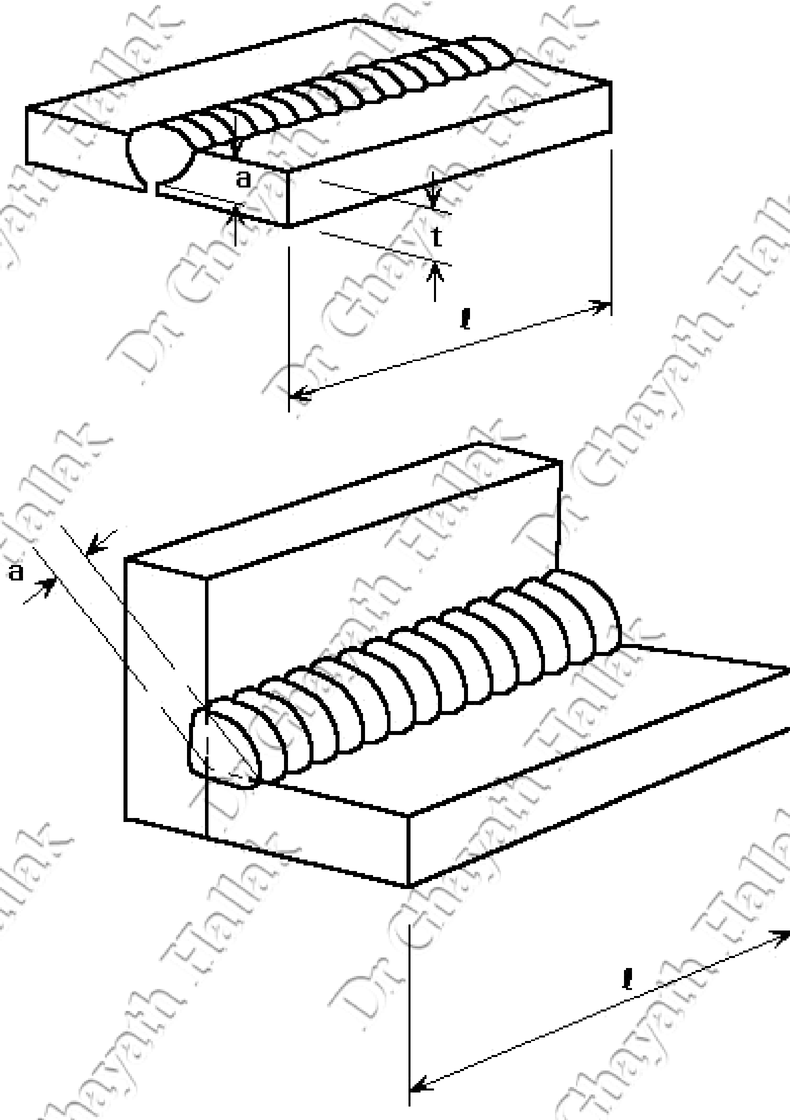
Spot welds are seldom used in building structures.



Welded Connections

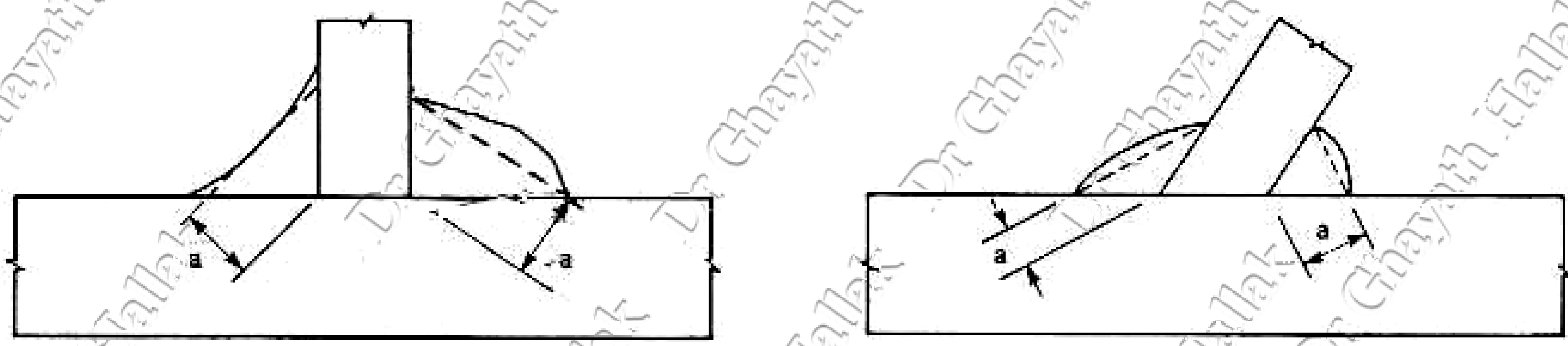
DESCRIPTION OF WELDS



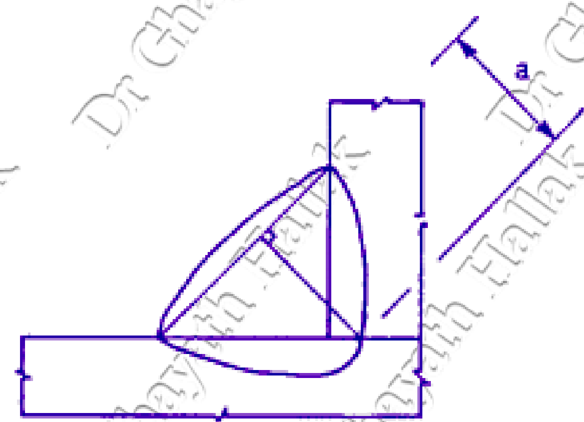


Throat thickness of welds

Welded Connections
DESCRIPTION OF WELDS

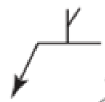
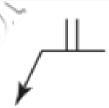


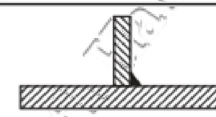
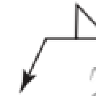
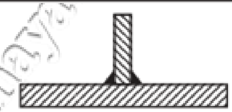
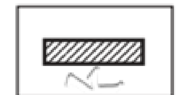
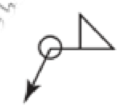
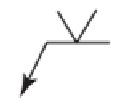
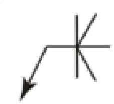
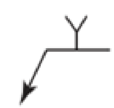
Throat thickness of a fillet weld



Throat thickness of a deep penetration fillet weld

Welded Connections
SYMBOLES OF WELDS

Type	Example	Symbol
Partial penetration half Y weld		
Partial penetration K weld		
Full penetration square butt weld		

Type	Example	Symbol
Fillet weld		
Double fillet weld		
All-around weld		
Site weld		
Full penetration single V butt weld		
Full penetration single-bevel butt weld		
Full penetration double V butt weld		
Full penetration double-bevel butt weld		
Partial penetration Y weld		

Welded Connections

MECHANICAL PROPERTIES OF MATERIALS

parent metal

The parent metal must have the weldability properties In accordance with EN 1993-1-1 and EN 10025. Hot-rolled steel grades S235, S275 and S355 are suitable for all welding processes.

Filler metal

According to Eurocode 3 the filler metal must have mechanical properties (yield strength, ultimate tensile strength, elongation at failure and minimum Charpy V-notch energy value) equal to or better than the values specified for the steel grade being welded.

Basis for Weld Calculation

For weld design, three fundamental assumptions are made:

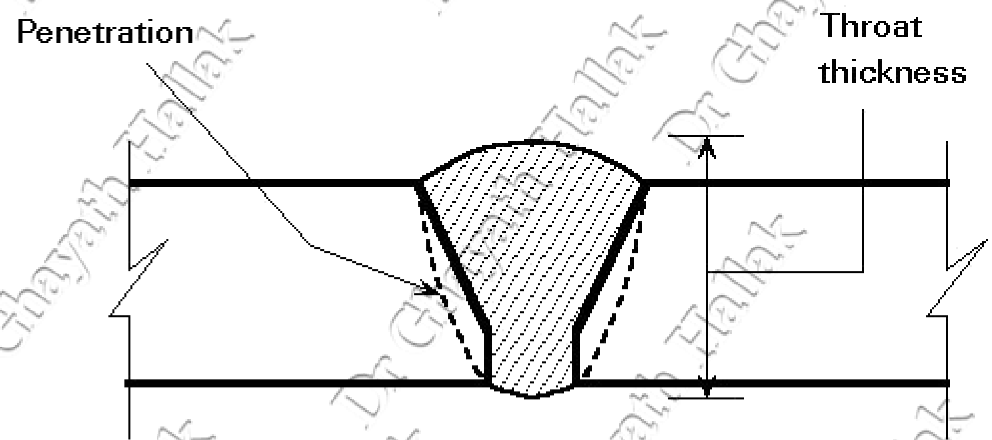
- The welds are homogeneous and isotropic elements.
- The parts connected by the welds are rigid and their deformations are negligible.
- Only nominal stresses due to external loads are considered. Effects of residual stresses, stress concentrations and shape of the welds are neglected in static design.

Welded Connections

BUTT WELD CALCULATION

1- Full Penetration Butt Welds

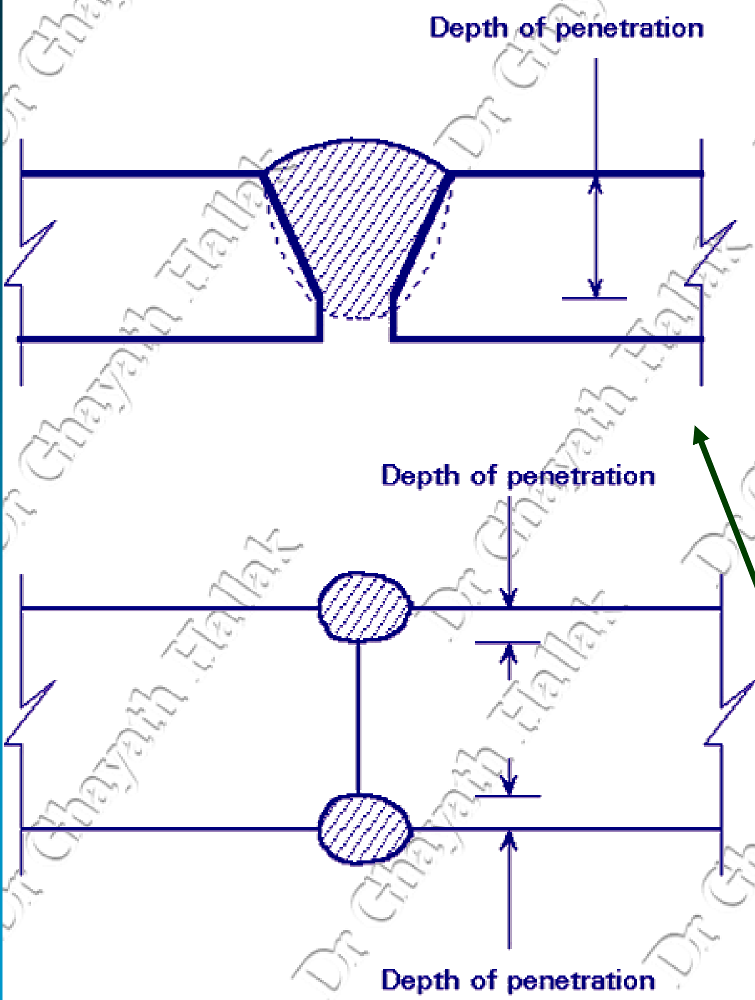
For a full penetration butt weld, calculation is not necessary because the filler metal strength is at least as high as the parent metal strength of the weaker part joined and the throat thickness of the weld is equal to the thickness of the plate, see Figure. Thus the butt weld may effectively be regarded simply as replacing the parent material

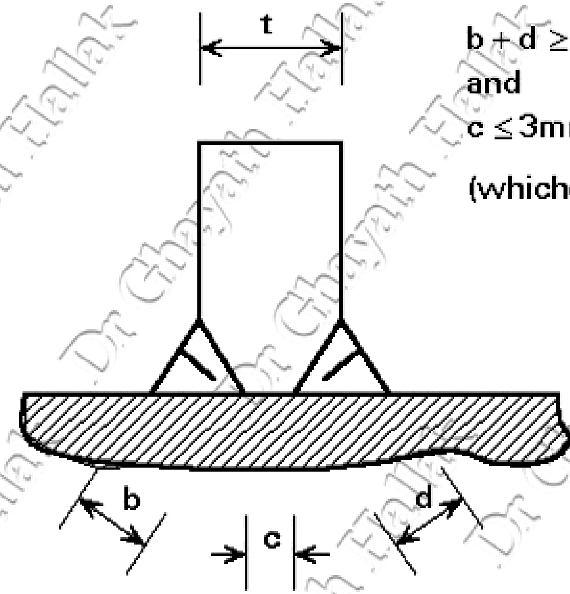


Welded Connections

BUTT WELD CALCULATION

2 Partial Penetration Butt Welds





$b + d \geq t$
and
$c \leq 3\text{mm}$ or $c \leq \frac{t}{5}$
(whichever is less)

Partial Penetration butt weld considered as a full penetration butt weld (**T-butt welds)**

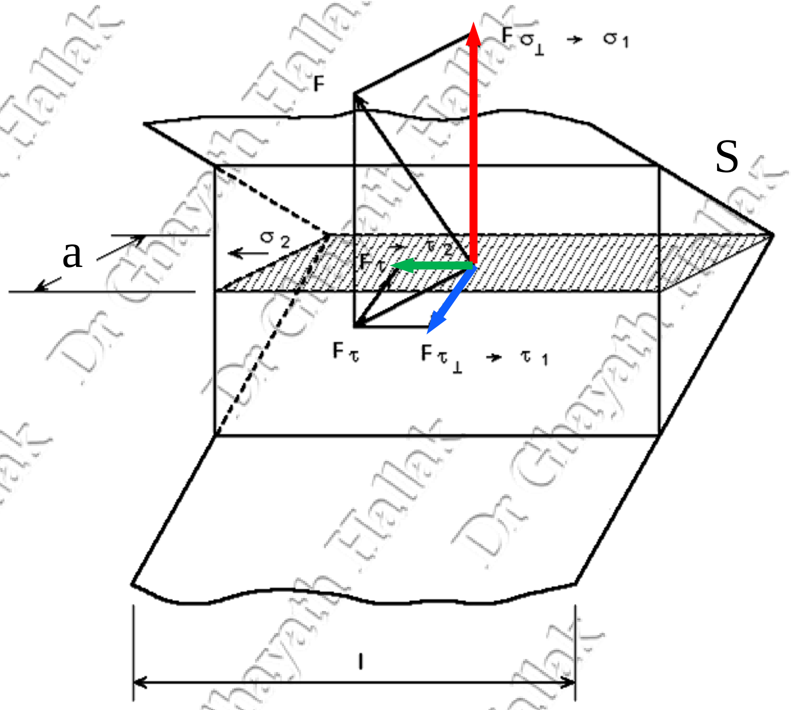
According to Eurocode 3, Throat thickness taken as minimum depth of penetration, reduced by 3 mm for most partial penetration butt welds, see Figure

Welded Connections

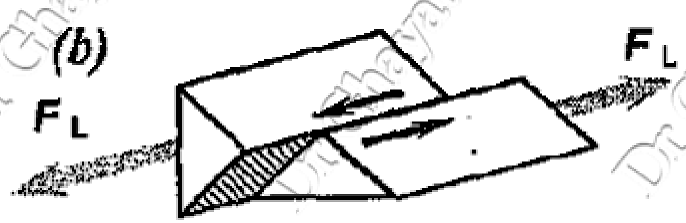
FILLET WELD CALCULATION

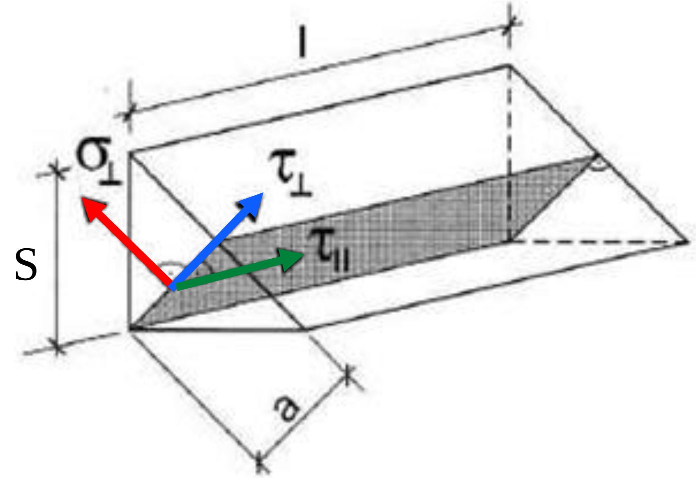
Two methods are permitted for the design of fillet welds:

1- the directional method, in which , the forces Transmitted by a unit length of weld are resolved into components parallel and transverse to the Longitudinal axis of the weld and normal and transverse to the plane of its throat.



2- the simplified method, in which only longitudinal shear is considered.





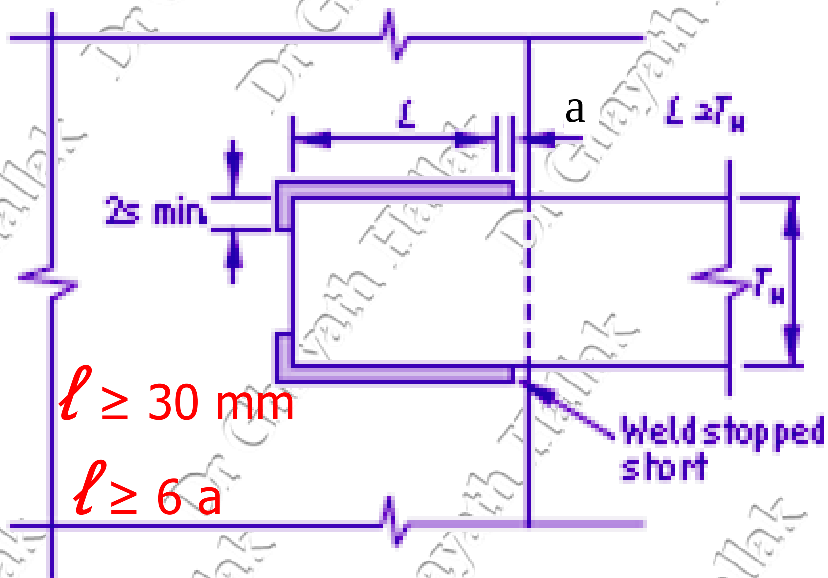
Welded Connections

FILLET WELD CALCULATION

1- the directional method:

the mechanical properties of the filler metal shall be compatible with the parent material properties.

The effective length of a fillet weld is the actual length less twice the throat thickness to allow for the starting and stopping of the weld. It should not be less than 30mm or less than six times the throat thickness. When a fillet weld terminates at the end or edge of a plate it should be returned continuously round the corner for a distance of twice the leg length.



$\ell \geq 30$ mm

$\ell \geq 6\,a$

$A_W = \sum a\,\ell_{eff}$, $\ell_{eff} = \ell - 2a$ (For poor welding at the stop and start positions of the weld)

$\ell_{eff} = \ell$ (provided that the weld is full size throughout its length including starts and terminations- good welding at the stop and start positions)

Welded Connections-FILLET WELD CALCULATION-

1- the directional method:

$\sigma_1 = F_{\sigma\perp} / a\, \ell_{eff}$ is the normal stress perpendicular to the plane of the throat area.

$\sigma_\perp$

$\tau_1 = F_{\tau\perp} / a\, \ell_{eff}$ is the shear stress in the plane of the throat area, transverse to the weld axis.

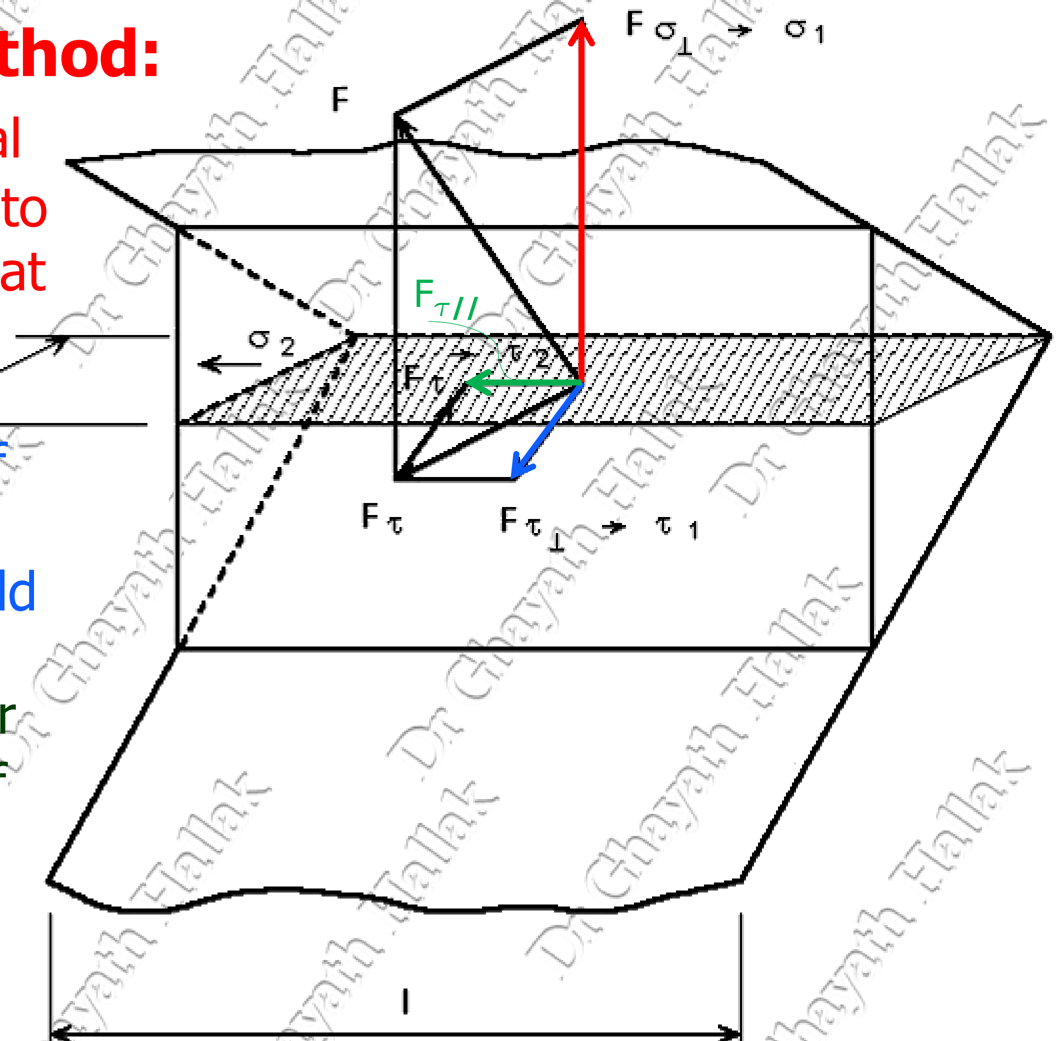
$\tau_\perp$

$\tau_2 = F_{\tau//} / a\, \ell_{eff}$ is the shear stress in the plane of the throat area, parallel to the weld axis.

$\tau_{//}$

σ_2 is the normal stress parallel to the weld axis.

$\sigma_{//}$

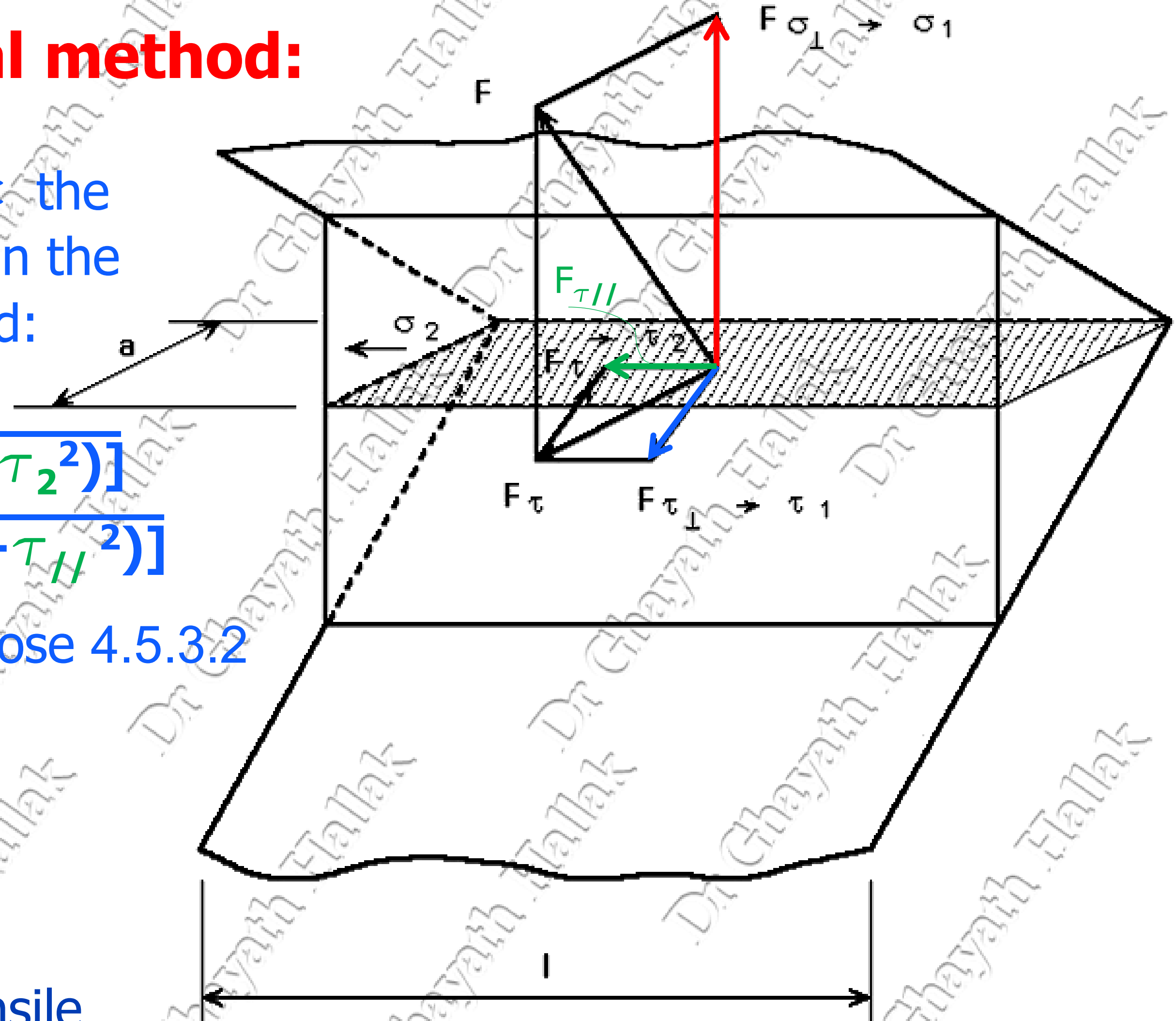


$\sigma_2 = 0$ the cross-section of the weld is very small and has negligible strength in comparison with the strength of the throat area

Welded Connections-FILLET WELD CALCULATION-

1- the directional method:

Von Mises criterion $\Rightarrow$ the equivalent stress σ_{eq} in the throat area of the weld:



$$\sigma_{eq}=\sqrt{[\sigma_1^2+3(\tau_1^2+\tau_2^2)]}$$

$$\sigma_{eq}=\sqrt{[\sigma_\perp^2+3(\tau_\perp^2+\tau_{//}^2)]}$$

EN 1993-1-8 : 2005 close 4.5.3.2

$$\sigma_{eq} \leqslant f_u/(\beta_w \gamma_{M2})$$

and

$$\sigma_1 \leqslant 0.9\, f_u / \gamma_{M2}$$

Where:

f_u is the ultimate tensile strength of the weaker part joined

Welded Connections-FILLET WELD CALCULATION-
1- the directional method:

EN 1993-1-8 : 2005 close 4.5.3.2

$\sigma_{eq} \leqslant f_u / (\beta_w \gamma_{M2})$

and

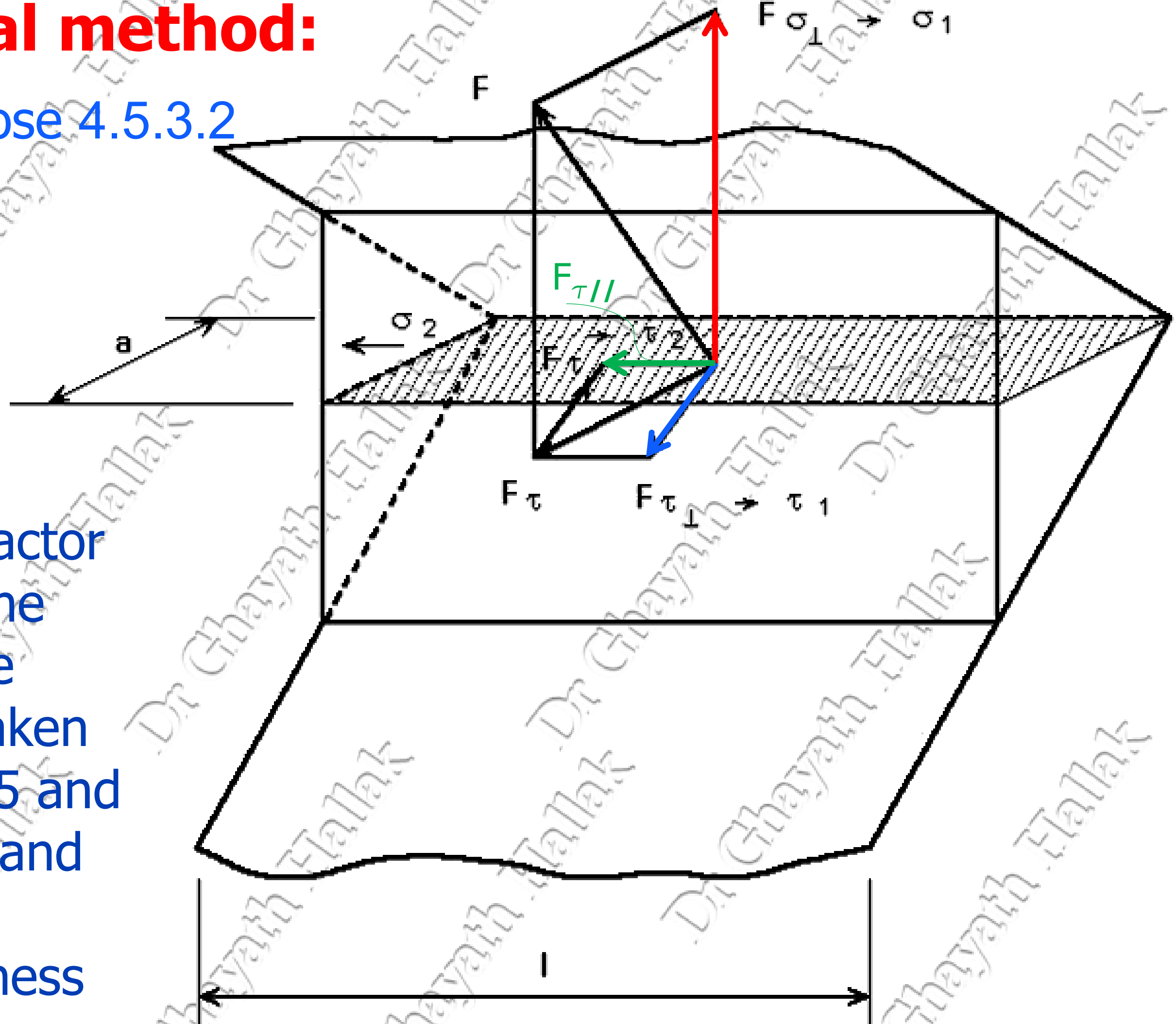
$\sigma_1 \leqslant 0.9\, f_u / \gamma_{M2}$

where:

β_w is the correlation factor according to the strength of the weaker part taken as 0.8 for S235 and 0.85 for S275 and 0.90 for S355

a is the throat thickness of the weld

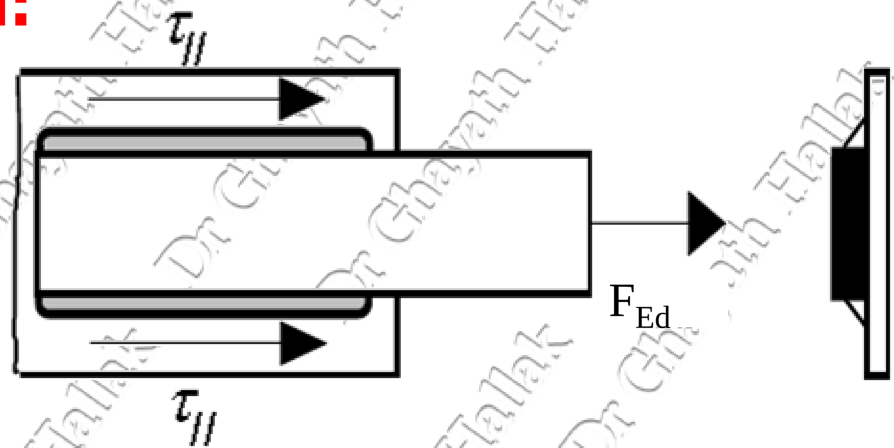
$\gamma_{M2} = 1.25$



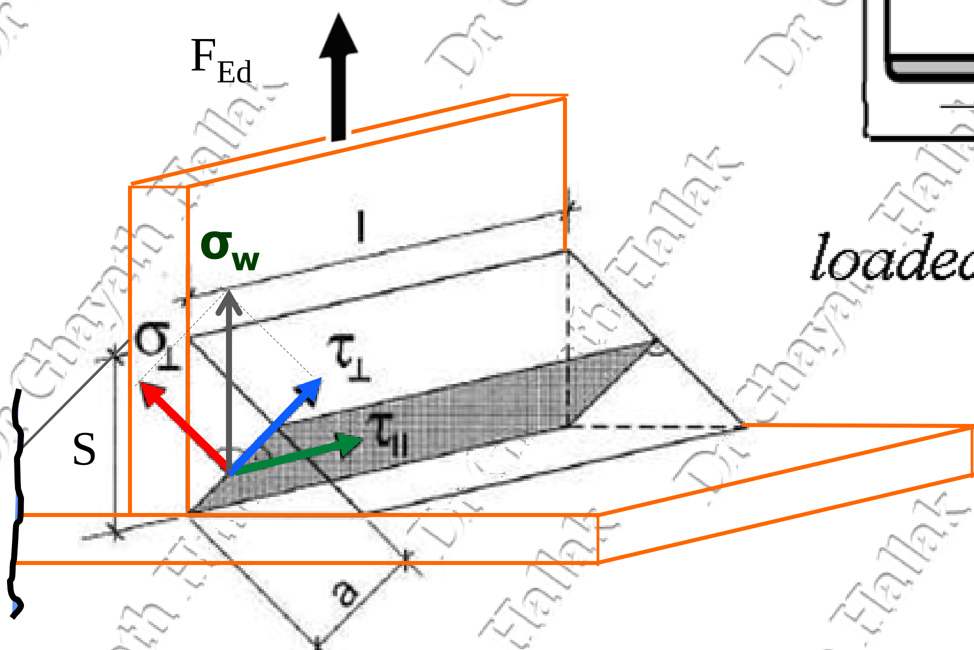
Welded Connections -FILLET WELD CALCULATION-

1- the directional method:

EN 1993-1-8 : 2005 close 4.5.3.2



loaded by force parallel to weld



$$\sigma_\perp = \tau_\perp = \frac{\sigma_w}{\sqrt{2}} \quad \text{and} \quad \tau_{//} = 0 \,.$$

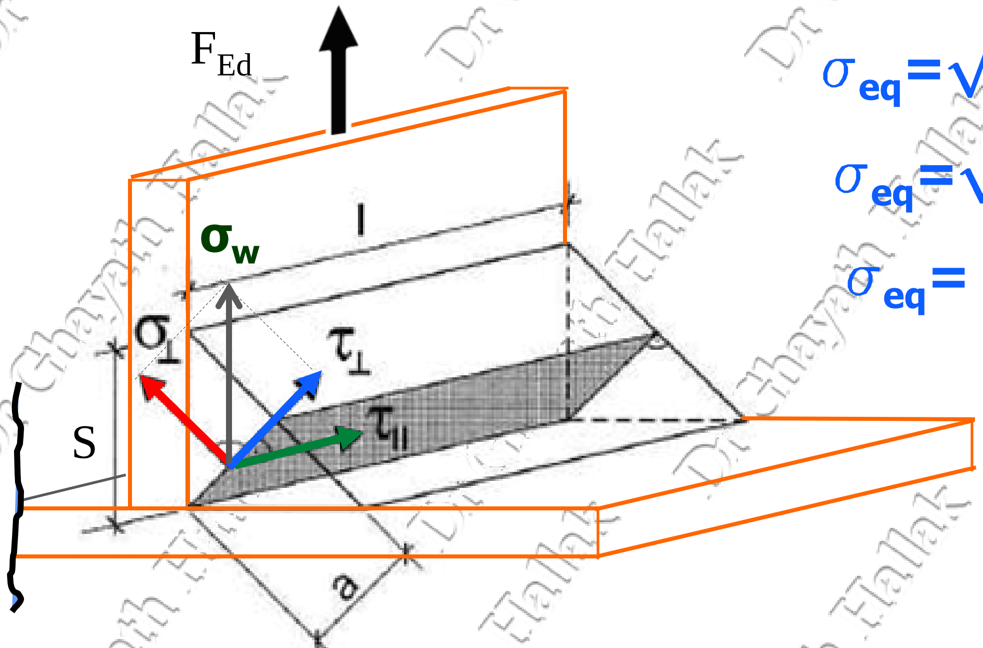
loaded by force perpendicular to weld

Welded Connections -FILLET WELD CALCULATION-

1- the directional method:

"full strength" throat thicknesses

If reference is made to the directional method, the design resistance of a fillet weld is checked as follows:



$$\sigma_{eq}=\sqrt{[\sigma_\perp{}^2+3(\tau_\perp{}^2+\tau_{//}{}^2)]}$$

$$\sigma_{eq}=\sqrt{[\sigma w^2/2+3(\sigma w^2/2+0)]}$$

$$\sigma_{eq}=\sqrt{2}\ \sigma_w$$

$\sigma_{eq} \leqslant f_u/(\beta_w \gamma_{M2}) \rightarrow$

$\sigma_w \leqslant f_u/(\sqrt{2}\ \beta_w \gamma_{M2})$

and

$\sigma_w \leqslant 0.9\sqrt{2}\ f_u/\ \gamma_{M2}$

$$\sigma_\perp = \tau_\perp = \frac{\sigma_w}{\sqrt{2}} \quad \text{and} \quad \tau_{//} = 0\ .$$

loaded by force perpendicular to weld

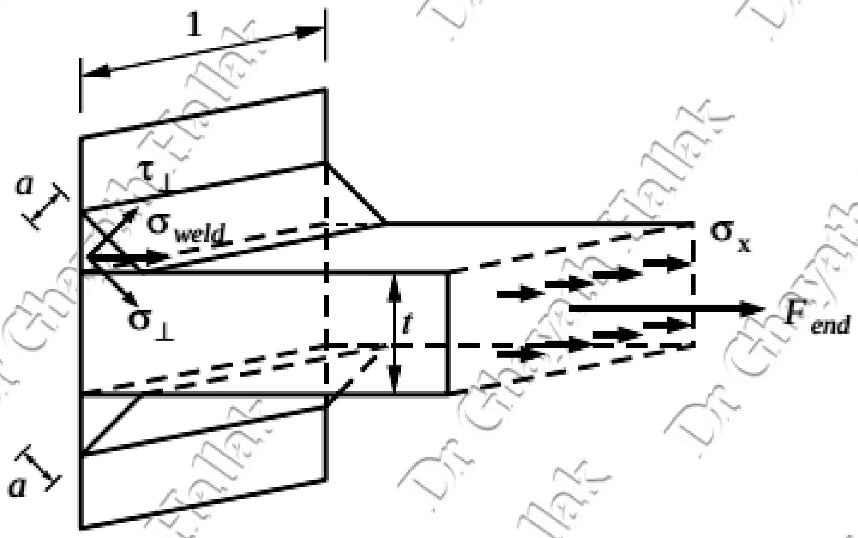
Welded Connections -FILLET WELD CALCULATION-

1- the directional method:

"full strength" throat thicknesses

For double end fillet welds, the above-expressed "full strength" criterion writes: $\mathbf{F_{weld} \geq F_{end}}$ $\rightarrow$ $2\,a\,l\,\sigma_{weld} \geq \sigma_x\,t\,l$

$2\,a\,l\,\sigma_{weld} \geq f_y\,t\,l\,/\,\gamma_{M0}$



The minimum weld size to satisfy the full strength design requirement is therefore expressed as

$$a \geq f_y\,t\,/\,[2\sigma_{weld}\,\gamma_{M0}]$$

Since $\sigma_{weld} = f_{w,u,end} = f_u/(\sqrt{2}\;\beta_w\gamma_{M2}) \rightarrow$

$a \geq f_y\,t\,/\,[2\,.\,f_u/(\sqrt{2}\;\beta_w\gamma_{M2})\,\gamma_{M0}] \rightarrow a \geq (f_y\,/\,f_u)\,(\beta_w/\sqrt{2})\,(\gamma_{M2}/\gamma_{M0})\,t$

Welded Connections -FILLET WELD CALCULATION-

1- the directional method:

"full strength" throat thicknesses

Steel grade	f_y	f_u	β_w	$f_{w,u,end}$	Full strength double fillet welds
	N/mm^2	N/mm^2		N/mm^2	
S235	235	360	0.80	255	$a \geq 0.46\ t$
S275	275	430	0.85	286	$a \geq 0.48\ t$
S355	355	510	0.90	321	$a \geq 0.55\ t$
S420 M	420	520	1.00	294	$a \geq 0.71\ t$
S420 N	420	550	1.00	311	$a \geq 0.68\ t$
S460 M	460	550	1.00	311	$a \geq 0.74\ t$
S460 N	460	580	1.00	328	$a \geq 0.70\ t$

Values of β_w and $f_{w,u,end}$ for steels according to EN 10025 and minimum "full strength" required weld thickness in case of double fillet end welds (plate thickness smaller than 40 mm; γ_{M0} = 1.0 and γ_{M2} = 1.25)

Welded Connections

FILLET WELD CALCULATION

2- Simplified method for design resistance of fillet weld EN 1993-1-8 : 2005 close 4.5.3.3

$$\tau_N = N_{Ed} / A_w$$

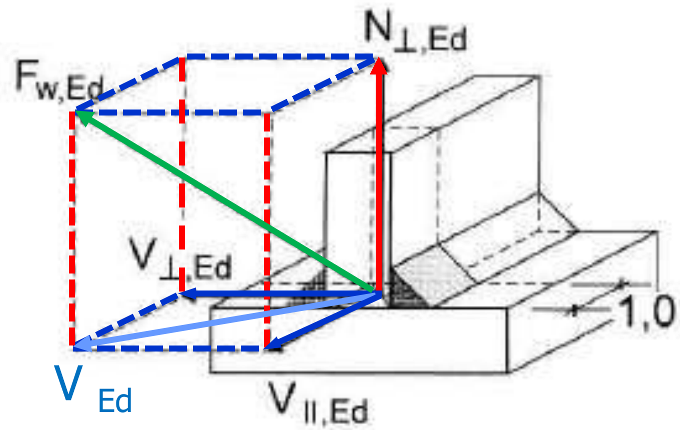
$$\tau_V = V_{Ed} / A_w$$

$F_{w,Ed} \leq F_{w,Rd}$

$$\tau_w = \sqrt{\left(\sum \tau_N\right)^2 + \left(\tau_{V\perp} + \tau_{V//}\right)^2} < \frac{fu}{\sqrt{3}\beta_w \gamma_{M2}}$$

Independent of the orientation of the weld throat plane to the applied force, the design resistance per unit length $F_{w,Rd}$ should be determined from:

$$F_{w,Rd} = f_{vw.d}\ a$$



where:

$f_{vw.d}$ is the design shear strength of the weld$= f_{vw.d} = f_u / (\sqrt{3}\beta_w \gamma_{M2})$

f_u is the ultimate tensile strength of the weaker part joined

β_w is the correlation factor according to the strength of the weaker part taken as 0.8 for S235 and 0.85 for S275 and 0.90 for S355

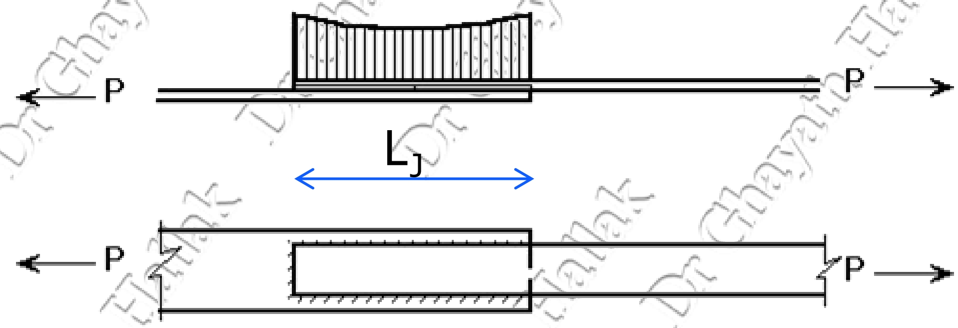
a is the throat thickness of the weld, $\gamma_{M2} = 1.25$

Welded Connections

FILLET WELD CALCULATION

Long joints EN 1993-1-8 : 2005 close 4.11

In lap joints the design resistance of a fillet weld should be reduced by multiplying it by a reduction factor β_{Lw} to allow for the effects of non-uniform distribution of stress along its length.



Stress distribution in long welds

In lap joints longer than 150a the reduction factor β_{Lw} :

$$\beta_{Lw.1} = 1,2-0,2L_j/(150a) \leqslant 1,0$$

For fillet welds longer than 1,7 metres connecting transverse stiffeners in plated members:

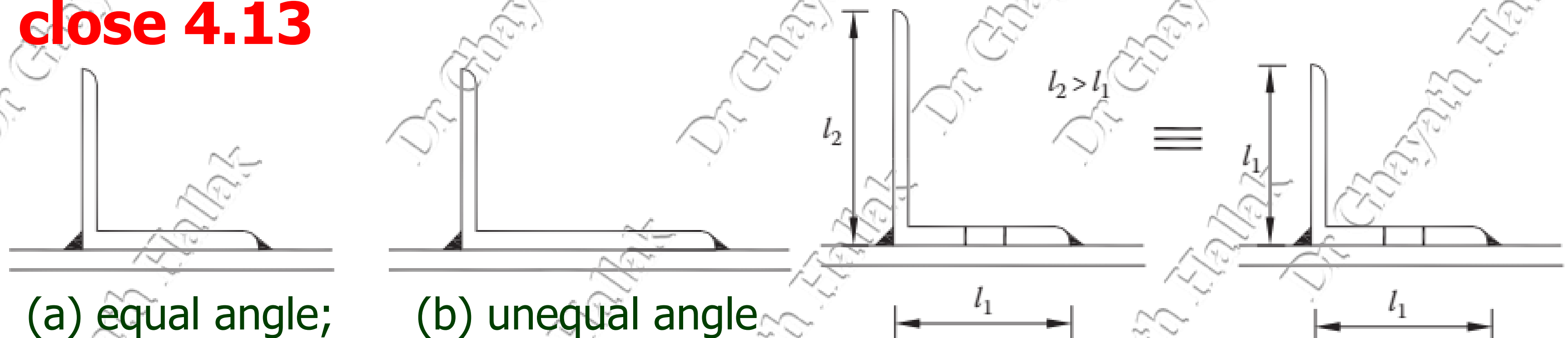
$$\beta_{Lw.2} = 1,1-L_W/17 \leqslant 1,0$$

L_w is the length of the weld (in metres).

Welded Connections

FILLET WELD CALCULATION

Angles connected by one leg EN 1993-1-8 : 2005 close 4.13



(a) equal angle; (b) unequal angle

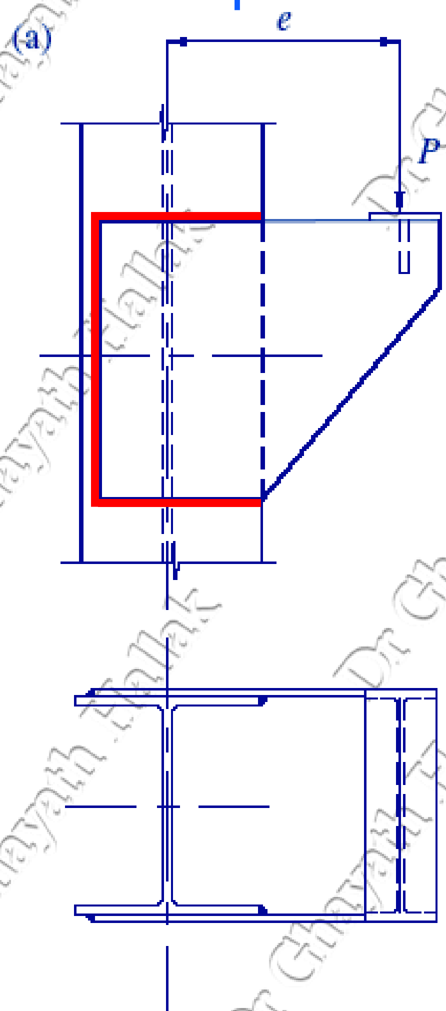
Effective area = gross area

(c) effective area taken as gross area of a smaller angle.

Effective area of welding connected angle

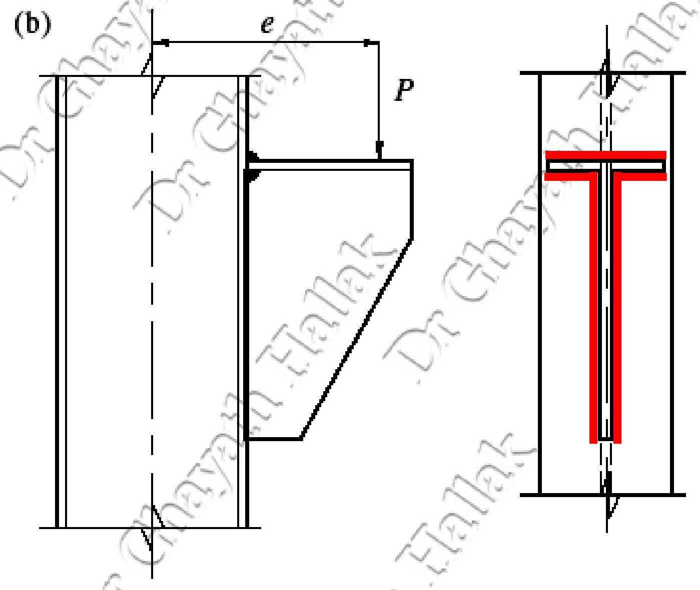
Eccentric connections

1. The torsion joint with the load in the plane of the weld



Load in plane of weld

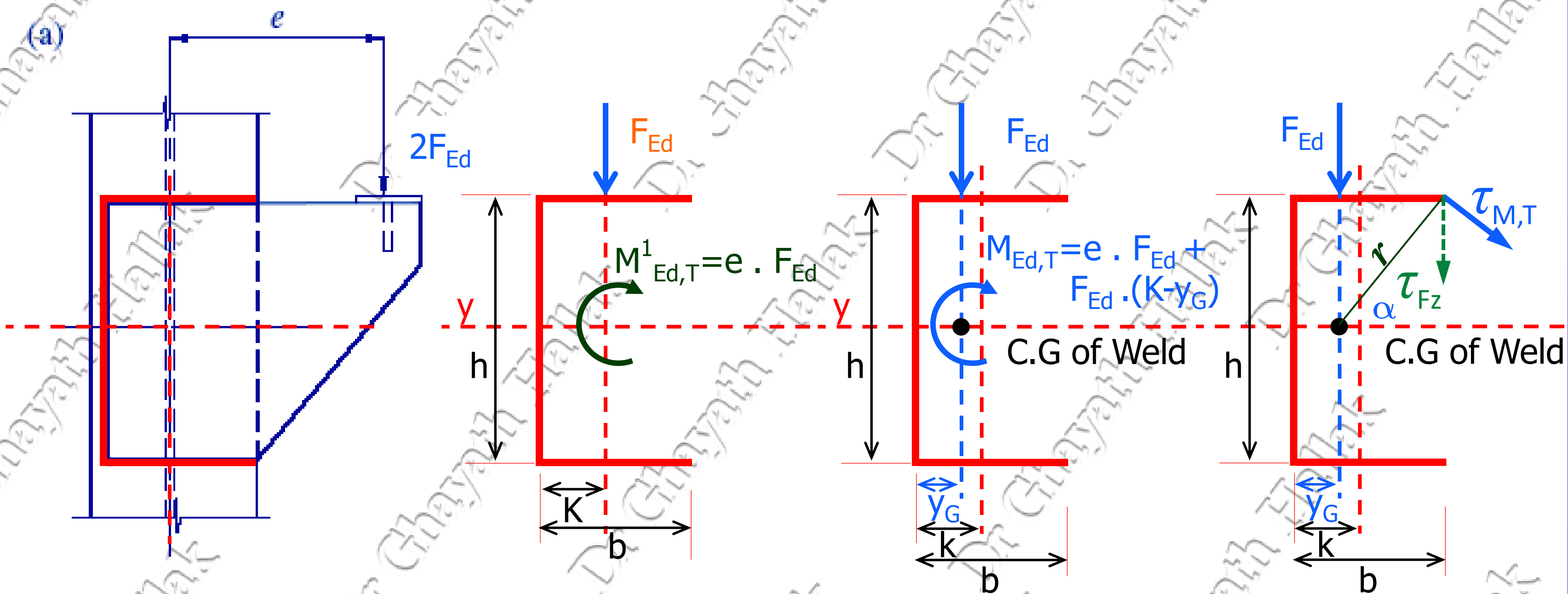
2. The bracket connection



Bracket connection

Eccentric connections

1. The torsion joint with the load in the plane of the weld



Stresses Due to Torsion $M_{Ed,T} = e \cdot F_{Ed} + F_{Ed} \cdot (K-y_G)$:

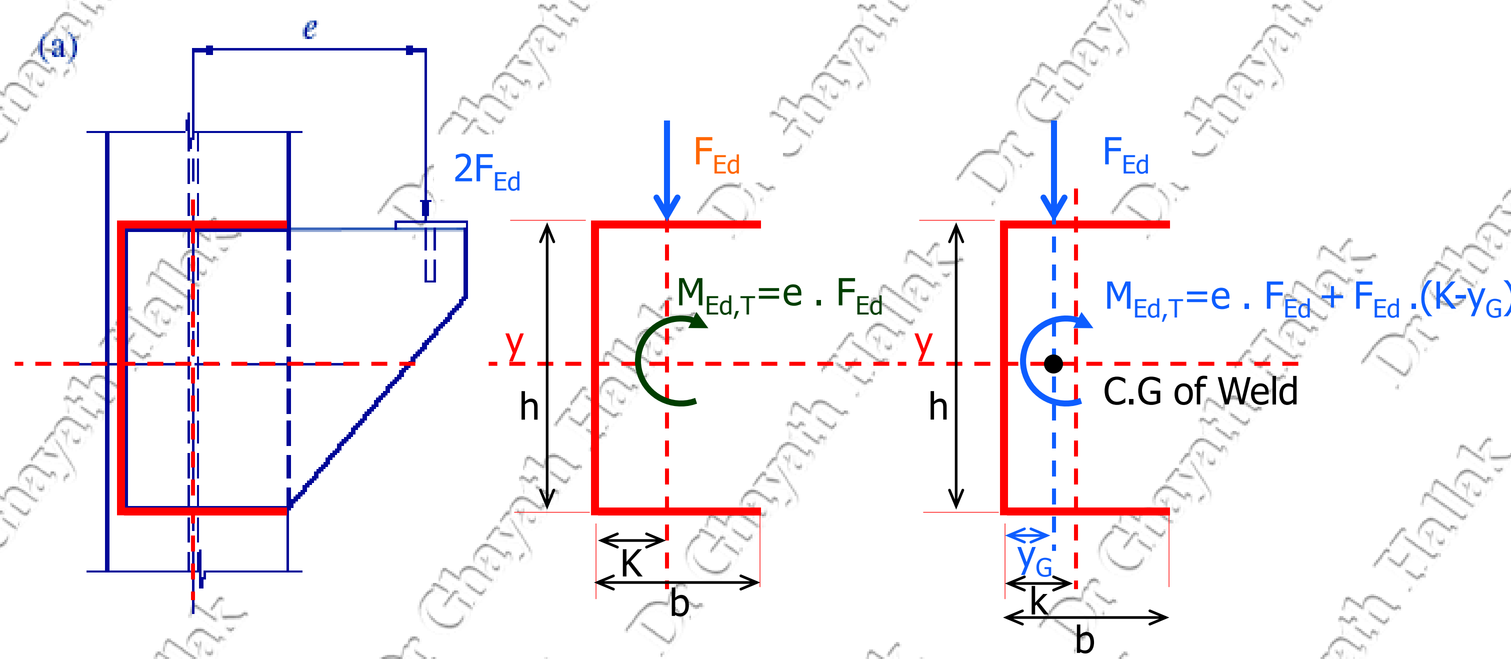
$$\tau_{M,T} = M_{Ed,T} \cdot r / (I_p) = M_{Ed,T} \cdot r / (I_{WY} + I_{WZ})$$

Stresses Due to Shear Force F_{Ed} applied at the center of gravity of welds:

$$\tau_{FZ} = F_{Ed} / A_W$$

Eccentric connections

1. The torsion joint with the load in the plane of the weld



$A_W = \sum a \cdot \ell_{eff}$; $\ell_{eff} = (2b+h)-2\,a$; $y_G = b^2/(2b+h)$ (POOR WELD)

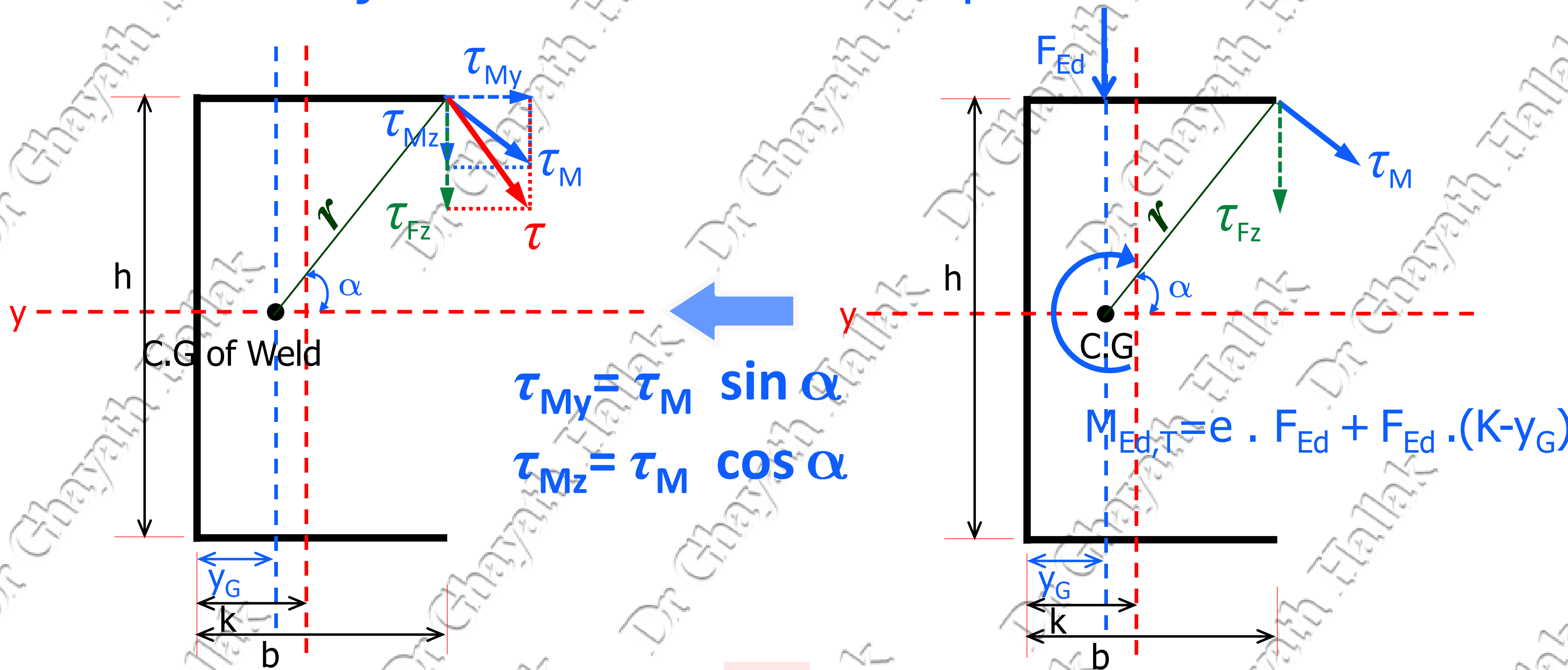
$I_{WY} = 2\,b\,a\,(h/2)^2 + a\,h^3/12$;

$I_{WZ} = 2\,[a\,b^3/12 + b\,a\,(k - y_G)^2\,] + a\,h\,y_G^2$

$\tau_{FZ} = F_{Ed}/A_W$; $\tau_{M,T} = M_{Ed,T} \cdot r\,/\,(I_{WY} + I_{WZ})$

Eccentric connections

1. The torsion joint with the load in the plane of the weld



$$\tau_{My} = \tau_M \ \sin\alpha$$

$$\tau_{Mz} = \tau_M \ \cos\alpha$$

$$M_{Ed,T} = e \cdot F_{Ed} + F_{Ed} \cdot (K - y_G)$$

$A_W = \sum a \cdot \ell_{eff}$; $\ell_{eff} = (2b+h) - 2\,a$; $y_G = b^2/(2b+h)$ (POOR WELD)

$I_{WY} = 2\,b\,a\,(h/2)^2 + a\,h^3/12$;

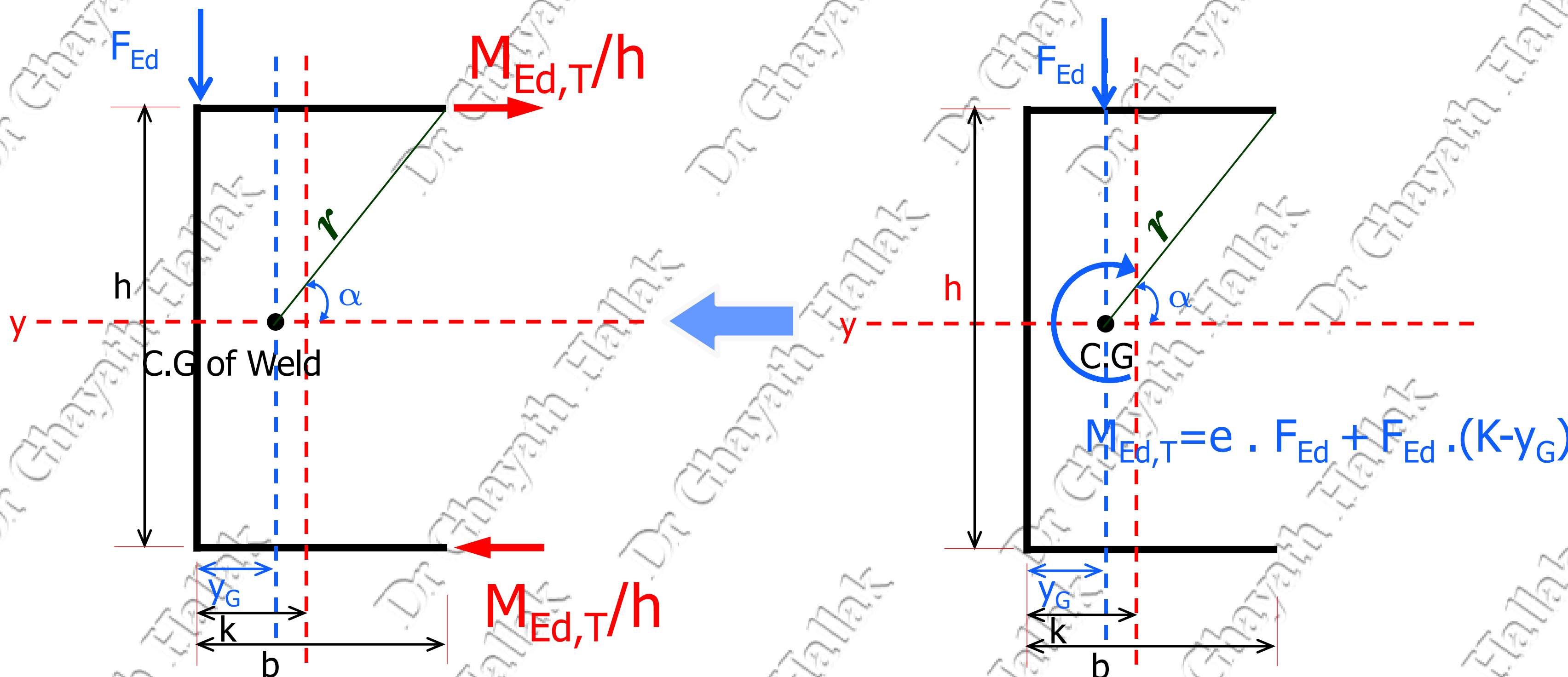
$I_{WZ} = 2\,[a\,b^3/12 + b\,a\,(k - y_G)^2\,] + a\,h\,y_G^2$

$$\tau = [\tau_{My}^{\ 2} + (\tau_{Mz} + \tau_{Fz})^2]^{0.5} < f_{vw,d}$$

Eccentric connections

1. The torsion joint with the load in the plane of the weld

Alternative Method



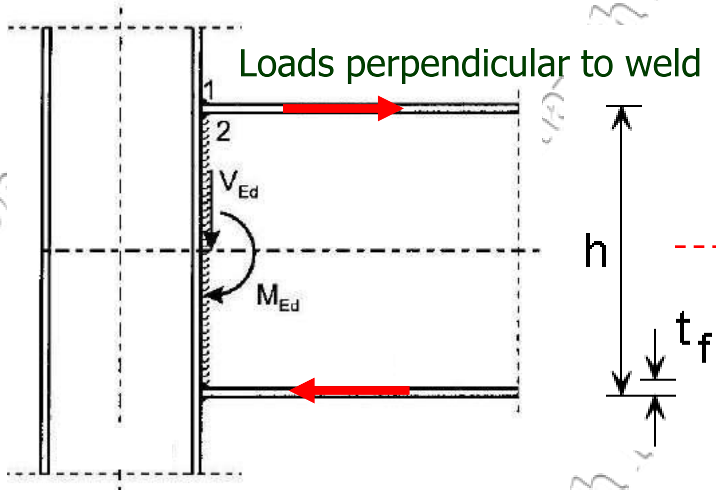
$$\tau_{Fz} = F_{Ed}/A_{Wz} < f_{vw,d}\ ,\ A_{Wz} = (h-2a)\ a$$

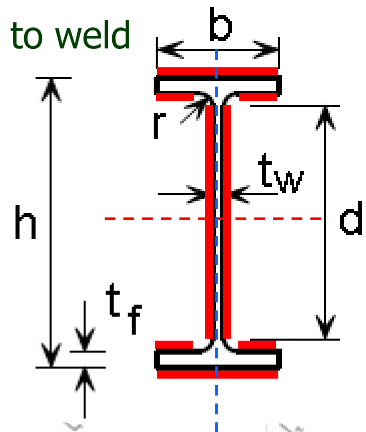
$$\tau_{Fy} = M_{Ed,T}/h\ A_{Wy} < f_{vw,d}\ ,\ A_{Wy} = (b-2a)\ a$$

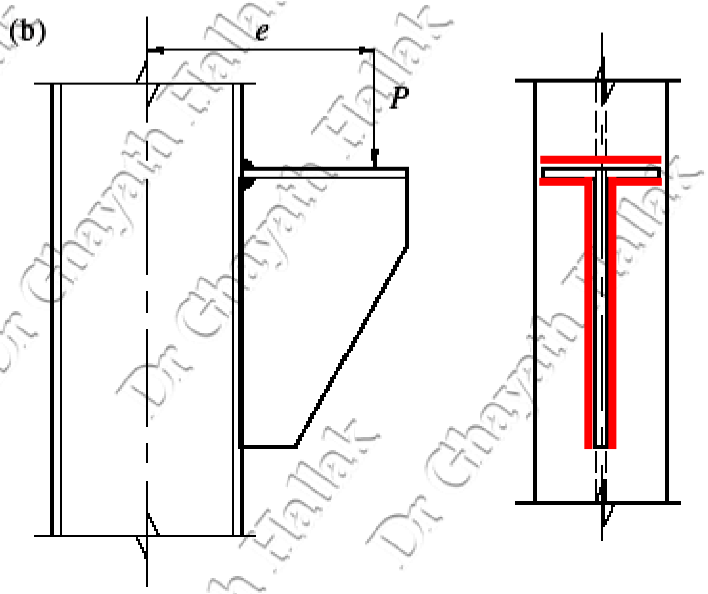
(POOR WELD)

Eccentric connections

2. The bracket connection







Bracket connection

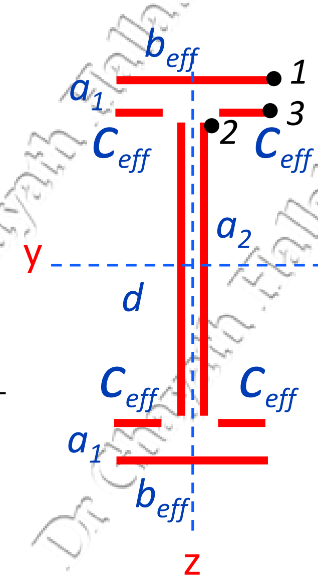
<u>Stress in point 1:</u> $\sigma_1 = M_{Ed} \text{ x } z_1/I_y$

$$I_y = 2\cdot[b_{eff}\cdot a_1\cdot Z_1^{\,2} + 2\cdot c_{eff}\cdot a_1\cdot Z_3^{\,2} + a_2\cdot d^{\,3}/12]$$

Loads perpendicular to weld $\quad \therefore \sigma_\perp = \tau_\perp = \sigma_1/\sqrt{2}$

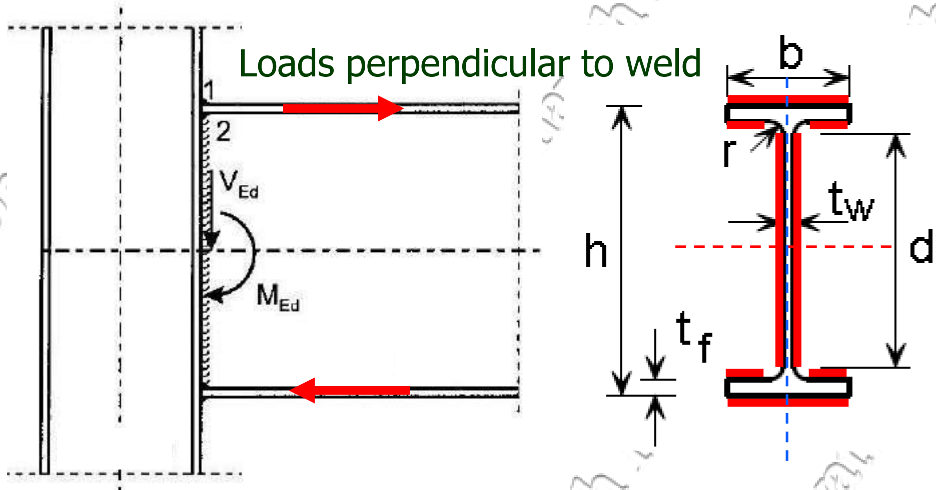
$$\sigma_{eq} = \sqrt{\sigma_\perp^2 + 3\left(\tau_\perp^2 + \tau_{//}^2\right)} = \sqrt{\frac{\sigma_1^2}{2} + 3\left(\frac{\sigma_1^2}{2} + 0\right)} = \sqrt{2}\,\sigma_1 \le \frac{f_u}{\beta_w \gamma_{M2}}$$

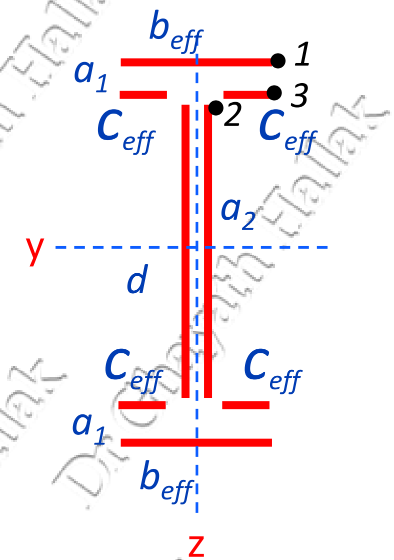
$$\sigma_\perp = \sigma_1/\sqrt{2} \le \frac{0.9\,f_u}{\gamma_{M2}}$$



Eccentric connections

2. The bracket connection





Stress in point 2: $\sigma_2 = M_{Ed} \times z_2 / I_y$

$$I_y = 2\cdot[b_{eff}\cdot a_1\cdot Z_1^2 + 2\cdot c_{eff}\cdot a_1\cdot Z_3^2 + a_2\cdot d^3/12]$$

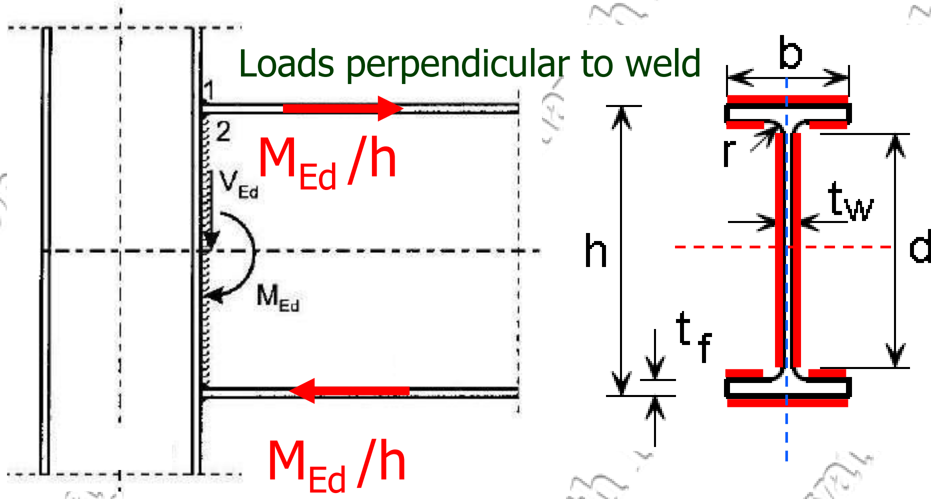
Loads perpendicular to weld $\therefore \sigma_\perp = \tau_\perp = \sigma_2/\sqrt{2}$

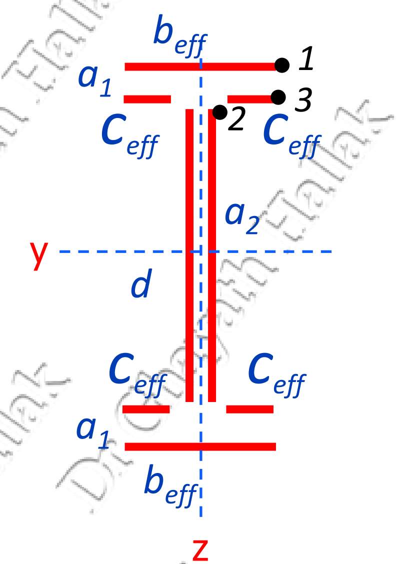
Loads parallel to weld $\tau_{//} = V_{Ed}/A_{W,w}\,,\ A_{W,w} = 2d\,a_2$

$$\sigma_{eq} = \sqrt{\sigma_\perp^2 + 3\left(\tau_\perp^2 + \tau_{//}^2\right)} = \sqrt{\frac{\sigma_2^2}{2} + 3\left(\frac{\sigma_2^2}{2} + \frac{V_{Ed}}{A_{W,w}}\right)} \le \frac{f_u}{\beta_w \gamma_{M2}} \quad ,\sigma_\perp = \sigma_2/\sqrt{2} \le \frac{0.9 f_u}{\gamma_{M2}}$$

Eccentric connections

2. The bracket connection (Alternative method)





$$\tau_{Fy} = M_{Ed,T}/h\ A_{Wy} < f_{vw,d}\ ,\ A_{Wy} = 2\,[(b_{eff} - 2a_1) + 2(C_{eff} - 2a_1)]\ a_1$$

$$\tau_{Fz} = V_{Ed}/A_{Wz} < f_{vw,d}\ ,\ A_{Wz} = 2\,(d - 2a_2)\ a_2$$

(POOR WELD)