ADVANCED STEEL DESIGN
M.TECH- STRUCTURAL ENGINEERING

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UNIT 1

SIMPLE CONNECTION – RIVETED, BOLTED PINNED & WELDED CONNECTION

Connections and Tension Member Design

Connections

Connections must be able to transfer any axial force, shear, or moment from member to member or from beam to column.

Steel construction accomplishes this with bolt and welds. Wood construction uses nails, bolts, shear plates, and split-ring connectors.

Bolted and Welded Connections

The limit state for connections depends on the loads:

1. tension yielding
2. shear yielding
3. bearing yielding
4. bending yielding due to eccentric loads
5. rupture

Welds must resist tension AND shear stress. The design strengths depend on the weld materials.

Bolted Connection Design

Bolt designations signify material and type of connection where SC: slip critical
N: bearing-type connection with bolt threads *included* in shear plane
X: bearing-type connection with bolt threads *excluded* from shear plane

Bolts rarely fail in **bearing**. The material with the hole will more likely yield first. Standard bolt holes are

1/16" larger than the bolt diameter.

BOLTS AND THREADED PARTS
Bearing
Allowable loads in kips

TABLE I-E. BEARING
Slip-critical and Bearing-type Connections

Material Thickness	F_u = 58 ksi Bolt dia.			F_u = 65 ksi Bolt dia.			F_u = 70 ksi Bolt dia.			F_u = 100 ksi Bolt dia.		
	3/4	7/8	1	3/4	7/8	1	3/4	7/8	1	3/4	7/8	1
1/8	6.5	7.6	8.7	7.3	8.5	9.8	7.9	9.2	10.5	11.3	13.1	15.0
3/16	9.8	11.4	13.1	11.0	12.8	14.6	11.8	13.8	15.8	16.9	19.7	22.5
1/4	13.1	15.2	17.4	14.6	17.1	19.5	15.8	18.4	21.0	22.5	26.3	30.0
5/16	16.3	19.0	21.8	18.3	21.3	24.4	19.7	23.0	26.3	28.1	32.8	37.5
3/8	19.6	22.8	26.1	21.9	25.6	29.3	23.6	27.6	31.5	33.8	39.4	45.0
7/16	22.8	26.6	30.5	25.6	29.9	34.1	27.6	32.2	36.8		45.9	52.5
1/2	26.1	30.5	34.8	29.3	34.1	39.0	31.5	36.8	42.0			60.0
9/16	29.4	34.3	39.2	32.9	38.4	43.9		41.3	47.3			
5/8	32.6	38.1	43.5		42.7	48.8		45.9	52.5			
11/16		41.9	47.9		46.9	53.6			57.8			
3/4		45.7	52.2			58.5						
13/16			56.6									
7/8			60.9									
15/16												
1	52.2	60.9	69.6	58.5	68.3	78.0	63.0	73.5	84.0	90.0	105.0	120.0

Notes:
This table is applicable to all mechanical fasteners in both slip-critical and bearing-type connections utilizing standard holes. Standard holes shall have a diameter nominally 1/16-in. larger than the nominal bolt diameter (d + 1/16 in.).
Tabulated bearing values are based on $F_p = 1.2\ F_u$.
F_u = specified minimum tensile strength of the connected part.
In connections transmitting axial force whose length between extreme fasteners measured parallel to the line of force exceeds 50 in., tabulated values shall be reduced 20%.
Connections using high-strength bolts in slotted holes with the load applied in a direction other than approximately normal (between 80 and 100 degrees) to the axis of the hole and connections with bolts in oversize holes shall be designed for resistance against slip at working load in accordance with AISC ASD Specification Sect. J3.8.
Tabulated values apply when the distance l parallel to the line of force from the center of the bolt to the edge of the connected part is not less than 1½ d and the distance from the center of a bolt to the center of an adjacent bolt is not less than $3d$. See AISC ASD Commentary J3.8.
Under certain conditions, values greater than the tabulated values may be justified under Specification Sect. J3.7.
Values are limited to the double-shear bearing capacity of A490-X bolts.
Values for decimal thicknesses may be obtained by multiplying the decimal value of the unlisted thickness by the value given for a 1-in. thickness.

BOLTS, THREADED PARTS AND RIVETS
Shear
Allowable load in kips

TABLE I-D. SHEAR

	ASTM Designation	Connection Type[a]	Hole Type[b]	F_v ksi	Loading[c]	Nominal Diameter d, in.							
						5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
						Area (Based on Nominal Diameter) in.^{2}							
						.3068	.4418	.6013	.7854	.9940	1.227	1.485	1.767
Bolts	A307	—	STD NSL	10.0	S	3.1	4.4	6.0	7.9	9.9	12.3	14.8	17.7
					D	6.1	8.8	12.0	15.7	19.9	24.5	29.7	35.3
	A325	SC[a] Class A	STD	17.0	S	5.22	7.51	10.2	13.4	16.9	20.9	25.2	30.0
					D	10.4	15.0	20.4	26.7	33.8	41.7	50.5	60.1
			OVS, SSL	15.0	S	4.60	6.63	9.02	11.8	14.9	18.4	22.3	26.5
					D	9.20	13.3	18.0	23.6	29.8	36.8	44.6	53.0
			LSL	12.0	S	3.68	5.30	7.22	9.42	11.9	14.7	17.8	21.2
					D	7.36	10.6	14.4	18.8	23.9	29.4	35.6	42.4
		N	STD, NSL	21.0	S	6.4	9.3	12.6	16.5	20.9	25.8	31.2	37.1
					D	12.9	18.6	25.3	33.0	41.7	51.5	62.4	74.2
		X	STD, NSL	30.0	S	9.2	13.3	18.0	23.6	29.8	36.8	44.5	53.0
					D	18.4	26.5	36.1	47.1	59.6	73.6	89.1	106.0
	A490	SC[a] Class A	STD	21.0	S	6.44	9.28	12.6	16.5	20.9	25.8	31.2	37.1
					D	12.9	18.6	25.3	33.0	41.7	51.5	62.4	74.2
			OVS, SSL	18.0	S	5.52	7.95	10.8	14.1	17.9	22.1	26.7	31.8
					D	11.0	15.9	21.6	28.3	35.8	44.2	53.5	63.6
			LSL	15.0	S	4.60	6.63	9.02	11.8	14.9	18.4	22.3	26.5
					D	9.20	13.3	18.0	23.6	29.8	36.8	44.6	53.0
		N	STD, NSL	28.0	S	8.6	12.4	16.8	22.0	27.8	34.4	41.6	49.5
					D	17.2	24.7	33.7	44.0	55.7	68.7	83.2	99.0
		X	STD, NSL	40.0	S	12.3	17.7	24.1	31.4	39.8	49.1	59.4	70.7
					D	24.5	35.3	48.1	62.8	79.5	98.2	119.0	141.0
Rivets	A502-1	—	STD	17.5	S	5.4	7.7	10.5	13.7	17.4	21.5	26.0	30.9
					D	10.7	15.5	21.0	27.5	34.8	42.9	52.0	61.8
	A502-2 A502-3	—	STD	22.0	S	6.7	9.7	13.2	17.3	21.9	27.0	32.7	38.9
					D	13.5	19.4	26.5	34.6	43.7	54.0	65.3	77.7
Threaded Parts	A36 (F_u=58 ksi)	N	STD	9.9	S	3.0	4.4	6.0	7.8	9.8	12.1	14.7	17.5
					D	6.1	8.7	11.9	15.6	19.7	24.3	29.4	35.0
		X	STD	12.8	S	3.9	5.7	7.7	10.1	12.7	15.7	19.0	22.6
					D	7.9	11.3	15.4	20.1	25.4	31.4	38.0	45.2
	A572, Gr. 50 (F_u=65 ksi)	N	STD	11.1	S	3.4	4.9	6.7	8.7	11.0	13.6	16.5	19.6
					D	6.8	9.8	13.3	17.4	22.1	27.2	33.0	39.2
		X	STD	14.3	S	4.4	6.3	8.6	11.2	14.2	17.5	21.2	25.3
					D	8.8	12.6	17.2	22.5	28.4	35.1	42.5	50.5
	A588 (F_u=70 ksi)	N	STD	11.9	S	3.7	5.3	7.2	9.3	11.8	14.6	17.7	21.0
					D	7.3	10.5	14.3	18.7	23.7	29.2	35.3	42.1
		X	STD	15.4	S	4.7	6.8	9.3	12.1	15.3	18.9	22.9	27.2
					D	9.4	13.6	18.5	24.2	30.6	37.8	45.7	54.4

[a]SC = Slip critical connection.
N: Bearing-type connection with threads *included* in shear plane.
X: Bearing-type connection with threads *excluded* from shear plane.
[b]STD: Standard round holes (d + 1/16 in.)
OVS: Oversize round holes
LSL: Long-slotted holes normal to load direction
SSL: Short-slotted holes
NSL: Long-or short-slotted hole normal to load direction (required in bearing-type connection).
[c]S: Single shear D: Double shear.
For threaded parts of materials not listed, use $F_v = 0.17F_u$ when threads are included in a shear plane, and $F_v = 0.22F_u$ when threads are excluded from a shear plane.
To fully pretension bolts 1⅛-in. dia. and greater, special impact wrenches may be required.
When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 in., tabulated values shall be reduced by 20%. See AISC ASD Commentary Sect. J3.4.

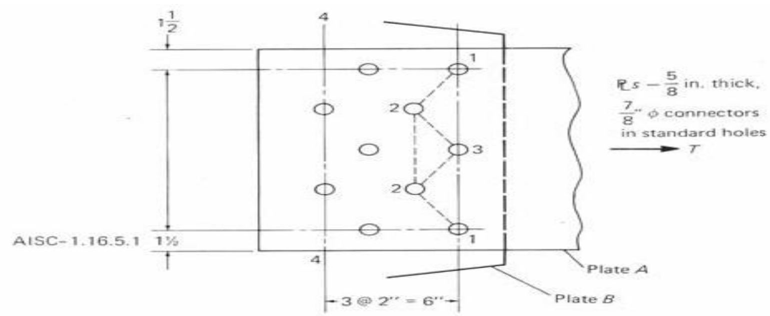
Tension Member Design

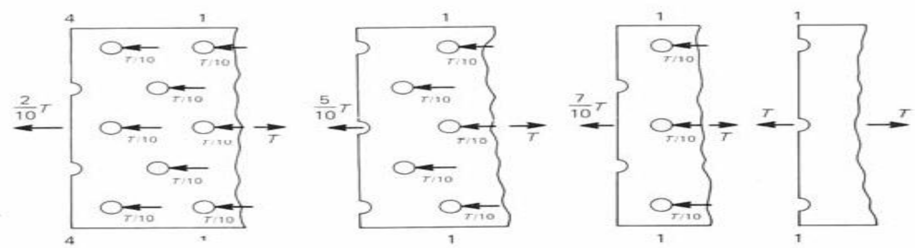
In steel tension members, there may be bolt holes that reduce the size of the cross section.

Effective Net Area:

The smallest effective are must be determined by subtracting the bolt hole areas. With staggered holes, the shortest length must be evaluated.

A series of bolts can also transfer a portion of the tensile force, and some of the effective net areas see reduced stress.





ASD

For other than pin connected members: $F_t = 0.60F_y$ *on gross area* $F_t = 0.50F_u$ *on net area*

For pin connected members: $F_t = 0.45F_y$ *on net area*

For threaded rods of approved steel: $F_t = 0.33F_u$ *on major diameter (static loading only)*

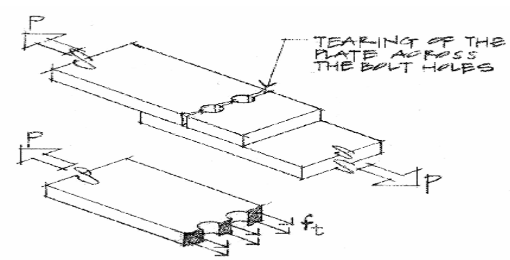
LRFD

The limit state for tension members are: $P_u \leq \phi_t P_n$

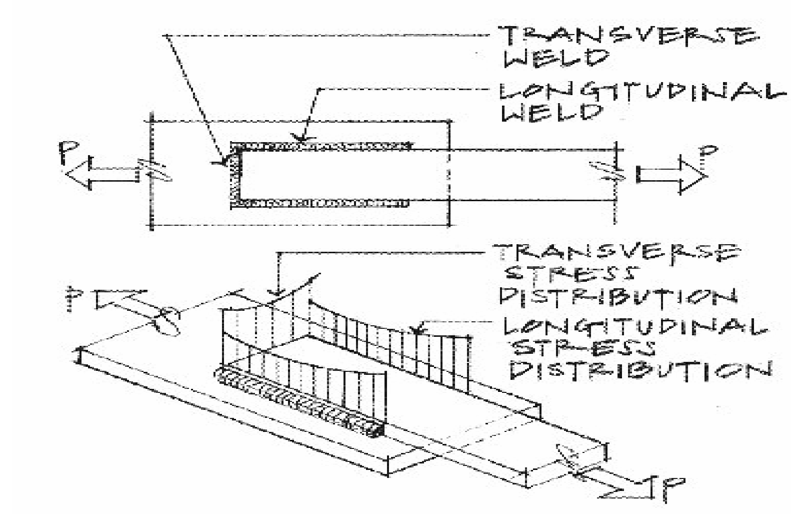
1. yielding $\phi_t = 0.9$ $P_n = F_y A_g$
2. rupture $\phi_t = 0.75$ $P_n = F_u A_e$

where

A_g = the gross area of the member (excluding holes)
A_e = the effective net area (with holes, etc.) F_u = the tensile strength of the steel (ultimate)



Welded Connections



Weld designations include the strength in the name, i.e. E70XX has F_y = 70 ksi.

The throat size, T, of a fillet weld is determined trigonometry by:
T = 0.707 ×weld size

ASD

Allowable shear stress of a weld is limited to 30% of the nominal strength.

F_v = 18 ksi for E60XX

F_v = 21 ksi for E70XX

Weld sizes are limited by the size of the parts being put together and are given in AISC manual table J2.4 along with the allowable strength per length of fillet weld, referred to as *S*.

The *maximum* size of a fillet weld:

a) can't be greater than the material thickness if it

Intermittent fillet welds can not be less that four times the weld size, not to be less than 1 ½".

Allowable Strength of Fillet Welds per inch of weld (*S*)

Weld Size (in.)	E60XX (k/in.)	E70XX (k/in.)
3/16	2.39	2.78
¼	3.18	3.71
5/16	3.98	4.64
3/8	4.77	5.57
7/16	5.57	6.94
½	6.36	7.42
5/8	[illegible]	9.27
¾		11.13

TABLE J2.4
Minimum Size of Fillet Welds

Material Thickness of Thicker Part Joined (in.)	Minimum Size of Fillet Weld[a] (in.)
To ¼ inclusive	⅛
Over ¼ to ½	3/16
Over ½ to ¾	¼
Over ¾	5/16

[a]Leg dimension of fillet welds. Single-pass welds must be used.

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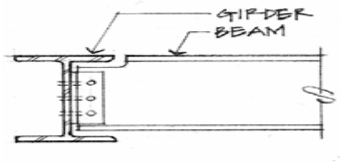
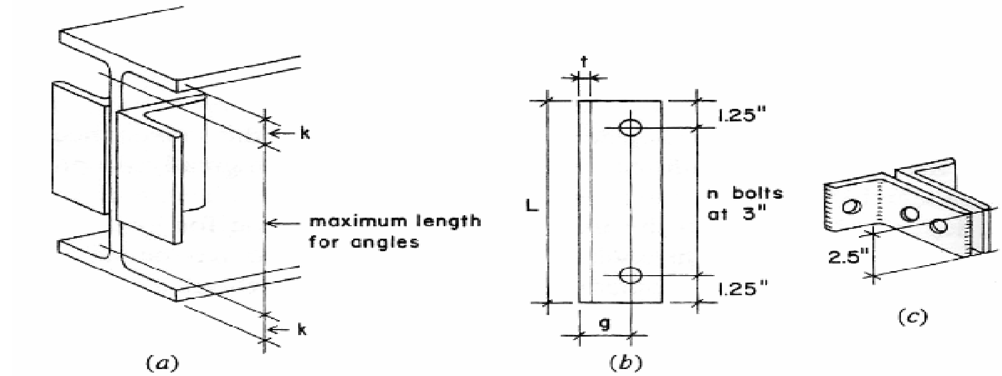
Framed Beam Connections



ing is the term for cutting away part of the flange to connect a beam to another beam using led or bolted angles.

AISC provides tables that give angle sizes knowing bolt type, bolt diameter, angle leg thickness, and number of bolts (determined by *shear* capacity).

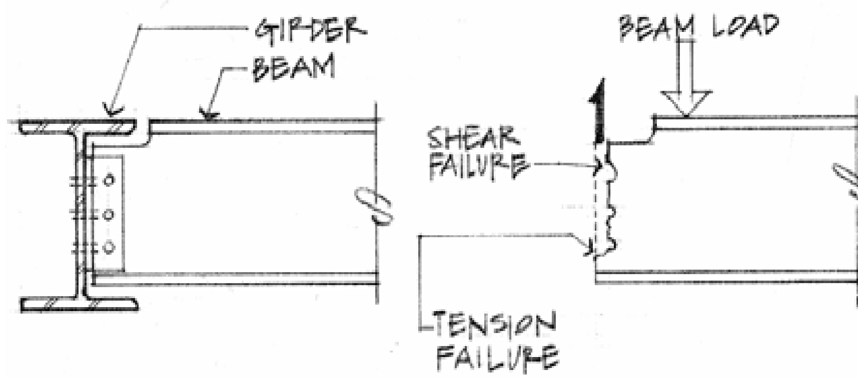


Load and Factor Resistance Design

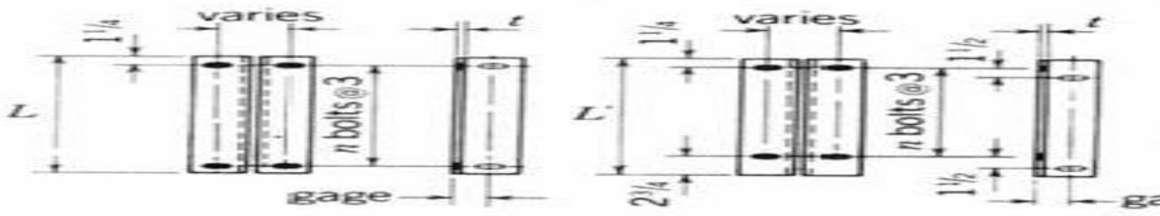
In addition to resisting shear and tension in bolts and shear in welds, the connected materials may be subjected to shear, bearing, tension, flexure and even prying action. Coping can significantly reduce design strengths and may require web reinforcement. All the following must be considered:

- shear yielding
- shear rupture
- block shear rupture -

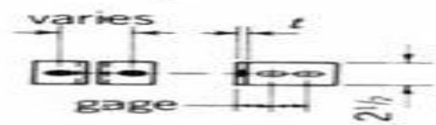
  failure of a block at a beam as a result of shear and tension
- tension yielding
- tension rupture
- local web buckling
- lateral torsional buckling



FRAMED BEAM CONNECTIONS
Bolted
TABLE II Allowable loads in kips



STAGGERED BOLT
ALTERNATE



Note: For $L = 2\frac{1}{2}$ use one half the tabular load value shown for $L = 5\frac{1}{2}$, for the same bolt type, diameter, and thickness.

TABLE II-A Bolt Shear

For bolts in **bearing-type** connections with standard or slotted holes.

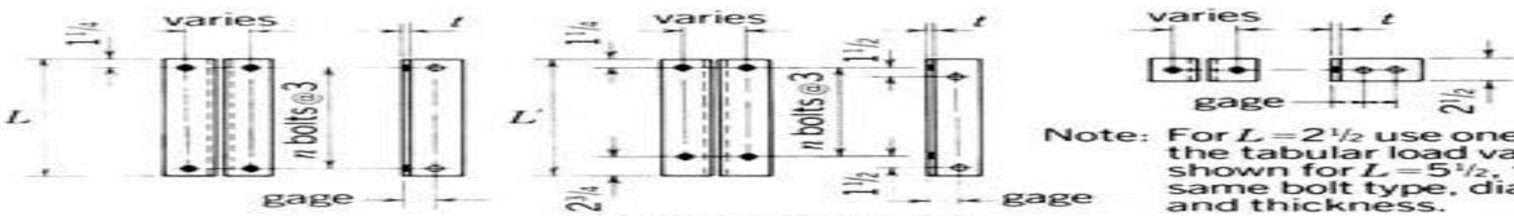
Bolt Type			A325-N			A490-N			A325-X			A490-X		
F_v, Ksi			21.0			28.0			30.0			40.0		
Bolt Dia., d In.			¾	⅞	1	¾	⅞	1	¾	⅞	1	¾	⅞	1
Angle Thickness t, In.			5/16	⅜	⅝	⅜	½	⅝	⅜	⅝	⅝	½	⅝	⅝
L In.	L' In.	n												
29½	31	10	186	253	330	247	337	**440**	265	361		353		
26½	28	9	167	227	297	223	303	**396**	239	325		318		
23½	25	8	148	202	264	198	269	**352**	212	289		283		
20½	22	7	130	**177**	231	173	**236**	**308**	186	253		247		
17½	19	6	111	**152**	198	148	**202**	**264**	159	216		212	**289**	
14½	16	5	92.8	**126**	165	124	**168**	**220**	**133**	180		**177**	**242**	
11½	13	4	74.2	**101**	132	99.0	**135**	**176**	**106**	144	**188**	**141**	**192**	
8½	10	3	55.7	**75.8**	99.0	74.2	**101**	**132**	**79.5**	108	**141**	**106**	**144**	
5½	7	2	37.1	**50.5**	66.0	49.5	**67.3**	**88.0**	**53.0**	72.2	**94**	**70.7**	**96**	

Tabulated load values are based on double shear of bolts.

Shaded values are based on double shear of the bolts; however, for length L, net shear on the angle thickness specified is critical. See Table II-C.

For shaded cells without values, shear rupture is critical for lengths L and L' on angle thickness specified. See Table II-C.

FRAMED BEAM CONNECTIONS
Bolted
TABLE II Allowable loads in kips



STAGGERED BOLT ALTERNATE

Note: For $L = 2\frac{1}{2}$ use one half the tabular load value shown for $L = 5\frac{1}{2}$, for the same bolt type, diameter, and thickness.

TABLE II-A Bolt Shear[a]

For A307 bolts in standard or slotted holes and for A325 and A490 bolts in **slip-critical** connections with standard holes and Class A, clean mill scale surface condition.

Bolt Type			A307			A325-SC			A490-SC		
F_v, Ksi			10.0			17.0			21.0		
Bolt Dia., d In.			¾	⅞	1	¾	⅞	1	¾	⅞	1
Angle Thickness t, In.			¼	¼	¼	¼	5/16	½	5/16	½	⅝
L In.	L' In.	n									
29½	31	10	88.4	120	157	150	204	267	186	253	330
26½	28	9	79.5	108	141	135	184	240	167	227	297
23½	25	8	70.7	96.2	126	120	164	214	148	202	264
20½	22	7	61.9	84.2	110	105	143	187	130	177	231
17½	19	6	53.0	72.2	94.2	90.1	123	160	111	152	198
14½	16	5	44.2	60.1	78.5	75.1	102	134	92.8	126	165
11½	13	4	35.3	48.1	62.8	60.1	81.8	107	74.2	101	132
8½	10	3	26.5	36.1	47.1[b]	45.1	61.3	80.1	55.7	75.8	99.0
5½	7	2	17.7	24.1	31.4[b]	30.0	40.9	53.4	37.1	50.5	66.0

Note: For slip-critical connections with oversize or slotted holes, see Table II-B.

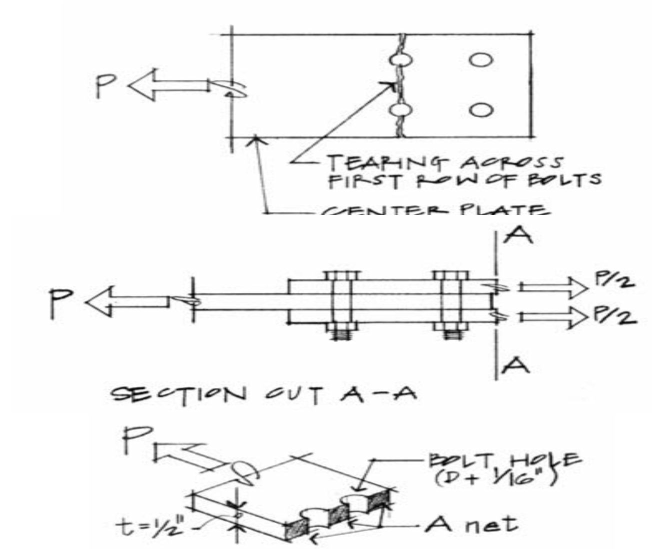
Notes:

[a]Tabulated load values are based on double shear of bolts unless noted. See RCSC Specification for other surface conditions.

[b]Capacity shown is based on double shear of the bolts; however, for length L, net shear on the angle thickness specified is critical. See Table II-C.

Example 1

10.2 The butt splice shown in Figure 10.22 uses two 8 × ⅜″ plates to "sandwich" in the 8 × ½″ plates being joined. Four ⅞″ϕ A325-SC bolts are used on both sides of the splice. Assuming A36 steel and standard round holes, determine the allowable capacity of the connection.



Solution:

Shear, bearing, and net tension will be checked to determine the critical condition that governs the capacity of the connection.

Shear: Using the AISC allowable shear in Table 10.1: (Table I-D)

$$P_v = 20.4\,\text{k/bolt} \times 4\ \text{bolts} = 81.6\,\text{k}\ \ \text{(double shear)}$$

Bearing: Using the AISC bearing in Table 10.2: (Table I-E)

The thinner material with the largest proportional load governs, therefore, the ½″ center plate governs. Assume the bolts are at a *3d* spacing, center to center.

$$P_b = 30.5\,\text{k/bolt} \times 4\ \text{bolts} = 122\,\text{k}$$

Tension: The center plate is critical since its thickness is less than the combined thickness of the two outer plates.

Hole diameter = (bolt diameter) + 1/16″ = 7/8″ + 1/16″ = 15/16″.

$$A_{net} = (8'' - 2 \times 15/16'') \times (1/2'') = 3.06\ \text{in.}^2$$

$$P_t = F_t \times A_{\text{net}}$$

where:

$$F_t = 0.5F_u = 0.5(58\ \text{ksi}) = 29\ \text{ksi}$$

$$P_t = 29\,\text{k/in.}^2 \times 3.06\ \text{in.}^2 = 88.7\,\text{k}$$

For yielding in the cross section without holes:

$$A_{gross} = (8'') \times (1/2'') = 4.0\ \text{in.}^2$$

$$P_t = F_t \times A_{gross}$$

where:

$$F_t = 0.6F_y = 0.6(36\ \text{ksi}) = 21.6\ \text{ksi}$$

$$P_t = 21.6\ \text{k/in.}^2 \times 4.0\ \text{in.}^2 = 86.4\,\text{k}$$

The maximum connection capacity is governed by shear.

$$P_{\text{allow}} = 81.6\,\text{k}$$

Example 2

10.7 Determine the capacity of the connection in Figure 10.44 assuming A36 steel with E70XX electrodes.

Solution:

Capacity of weld:

For a 5/16″ fillet weld, S = 4.64 k/in

Weld length = 22″

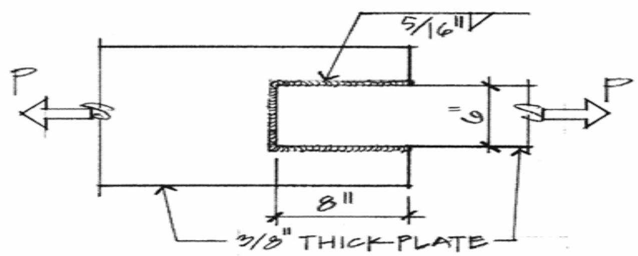
Weld capacity = 22″ × 4.64 k/in = 102.1 k

Capacity of plate:

$$\underset{\text{allow}}{F_t} = 0.6F_y = 22\,\text{ksi}$$

Plate capacity = 3/8″ × 6″ × 22 k/in.^{2} = 49.5 k

∴ Plate capacity governs, $P_{\text{allow}} = 49.5$ k



The weld size used is obviously too strong. What size, then, can the weld be reduced to so that the weld strength is more compatible to the plate capacity? To make the weld capacity ≈ plate capacity:

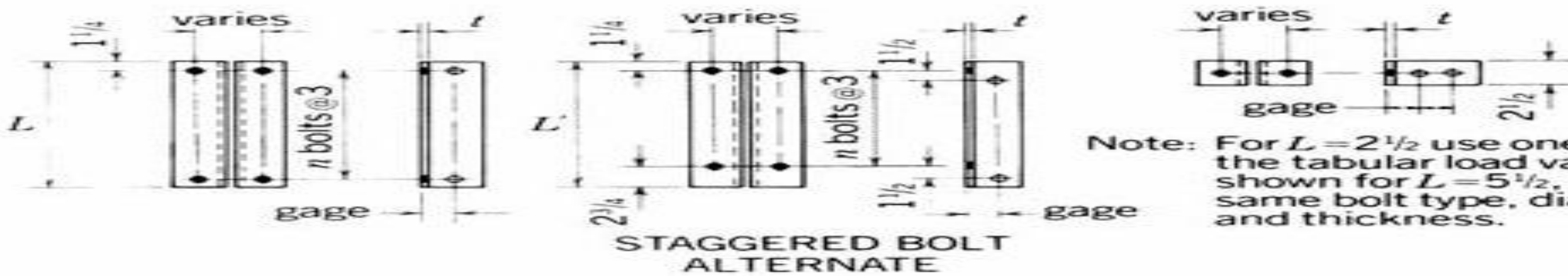
22″ × (weld capacity per in.) = 49.5 k

$$\text{Weld capacity per inch} = \frac{49.5\,\text{k}}{22\,\text{in.}} = 2.25\,\text{k/in.}$$

From Table 10.5, use 3/16″ weld ($S = 2.78$ k/in.).

Minimum size fillet = 3/16″ based on a 3/8″ thick plate.

FRAMED BEAM CONNECTIONS
Bolted
TABLE II Allowable loads in kips



Note: For $L = 2\frac{1}{2}$ use one half the tabular load value shown for $L = 5\frac{1}{2}$, for the same bolt type, diameter, and thickness.

TABLE II-A Bolt Shear[a]

For A307 bolts in standard or slotted holes and for A325 and A490 bolts in **slip-critical** connections with standard holes and Class A, clean mill scale surface condition.

Bolt Type			A307			A325-SC			A490-SC			
F_v, Ksi			10.0			17.0			21.0			Note: For slip-critical connections with oversize or slotted holes, see Table II-B.
Bolt Dia., d In.			¾	⅞	1	¾	⅞	1	¾	⅞	1	
Angle Thickness t, In.			¼	¼	¼	¼	5/16	½	5/16	½	⅝	
L In.	L' In.	n										
29½	31	10	88.4	120	157	150	204	267	186	253	330	
26½	28	9	79.5	108	141	135	184	240	167	227	297	
23½	25	8	70.7	96.2	126	120	164	214	148	202	264	
20½	22	7	61.9	84.2	110	105	143	187	130	177	231	
17½	19	6	53.0	72.2	94.2	90.1	123	160	111	152	198	
14½	16	5	44.2	60.1	78.5	75.1	102	134	92.8	126	165	
11½	13	4	35.3	48.1	62.8	60.1	81.8	107	74.2	101	132	
8½	10	3	26.5	36.1	47.1[b]	45.1	61.3	80.1	55.7	75.8	99.0	
5½	7	2	17.7	24.1	31.4[b]	30.0	40.9	53.4	37.1	50.5	66.0	

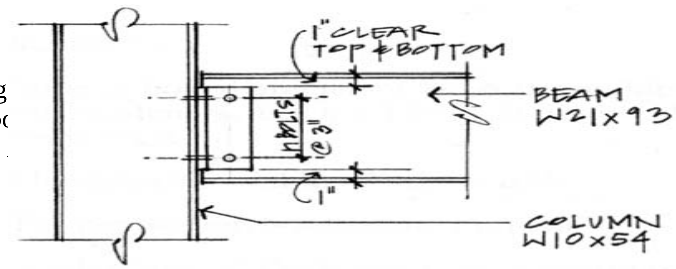
Notes:

[a]Tabulated load values are based on double shear of bolts unless noted. See RCSC Specification for other surface conditions.

[b]Capacity shown is based on double shear of the bolts; however, for length L, net shear on the angle thickness specified is critical. See Table II-C.

Example 3

The steel used in the connection and beams is A992 with $F_y = 50$ ksi, and $F_u = 65$ ksi. Using N bolt material, determine the maximum capacity of the connection based on shear in the bc bearing in all materials and pick the number of bolts and angle length (not staggered). Use steel for the angles.



W21x93: d = 21.62 in, t_w = 0.58 in, t_f = 0.93 in W10x54: t_f = 0.615 in

SOLUTION:

The maximum length the angles can be depends on how it fits between the top and bottom flange with some clearance allowed for the fillet to the flange, and getting an air wrench in to tighten the bolts. This example uses 1" of clearance:

Available length = beam depth – both flange thicknesses – 1" clearance at top & 1" at bottom

= 21.62 in – 2(0.93 in) – 2(1 in) = 17.76 in.

The standard lengths for non-staggered holes (L) and staggered holes (L') are shown in Table II-A. The closest size within the available length is 17 ½ in. This will fit 6 bolts (n) with a standard spacing.

We have a choice of bolt diameters of ¾", 7/8" and 1" in Table II-A. These have allowable loads for **shear** (double) of 148 kips, 202 kips, and 264 kips. But the last two values are shaded and the note says that "net shear on the angle thickness specified is critical" and to see Table II-C. The angle thickness (t) is listed below the bolt diameter.

Table II-C gives a value of 207 kips for a 7/8" bolt diameter, ½" angle thickness, and 17.5" length. It gives a value of 242 kips for a 1" bolt diameter, 5/8" angle thickness, and 17.5" length. Therefore, 242 kips is the maximum value limited by shear in the *angle*.

P_p = 264 kips for double shear of 1" bolts (Table I-D: 6 bolts·(44 k/bolt) = 264 kips)

P_v = 242 kips for net shear in angle

We also need to evaluate **bearing** of bolts on the angles, beam web, and column flange where there are bolt holes. Table I-E provides allowable bearing load for the material type, bolt diameter and some material thicknesses. The last note states that "Values for decimal thicknesses may be obtained by multiplying the decimal value of the unlisted thickness by the value given for a 1-in. thickness". This comes from the definition for bearing stress:

$$f_p = \frac{P}{td} \le F_p$$ where $P_p = t \cdot d \cdot F_p$ at the allowable bearing stress.

For a constant diameter and allowable stress, the allowable load depends only on the thickness.

a)Bearing for 5/8" thick angle: There are 12 bolt holes through two angle legs to the column, and 12 bolt holes through two angle legs either side of the beam. The material is A36 (F_u = 58 ksi), with 1" bolt diameters.

P_p = 12 bolts·(43.5 k/bolt) = 522 kips

b)Bearing for column flange: There are 12 bolt holes through two angle legs to the column. The material is A992 (F_u = 65 ksi), 0.615" thick, with 1" bolt diameters.

P_p = 12 bolts·(78 k/bolt/1")·(0.615 in) = 576 kips.

c)Bearing for beam web: There are *6* bolt holes through two angle legs either side of the beam. The material is A992 (F_u = 65 ksi), 0.58" thick, with 1" bolt diameters

P_p = 6 bolts·(78 k/bolt/1")·(0.58 in) = 271 kips.

Although, the bearing in the beam web is the smallest at 271 kips, with the shear on the bolts even smaller at 264 kips, *the maximum capacity for the simple-shear connector is 242 kips* limited by net shear in the angles.

UNIT II

Eccentric and moment Connections

Instructional Objectives:

At the end of this lesson, the students should be able to understand:

- Meaning of eccentricity in loading.
- Procedure for designing a screw/bolted joint in eccentric loading.
- Procedure for designing riveted joint under eccentric loading.

In many applications, a machine member is subjected to load such that a bending moment is developed in addition to direct normal or shear loading. Such type of loading is commonly known as eccentric loading.

(i) Screw joint

(ii) Riveted joint

(iii) Welded joint

1. Eccentrically loaded screwed joint:

Consider a bracket fixed to the wall by means of three rows of screws having two in each row as shown in figure 11.1.1. An eccentric load *F* is applied to the extreme end of the bracket. The horizontal component, F_h, causes direct tension

in the screws but the vertical component, F_v, is responsible for turning the bracket about the lowermost point in left (say point O), which in an indirect way introduces tension in the screws.

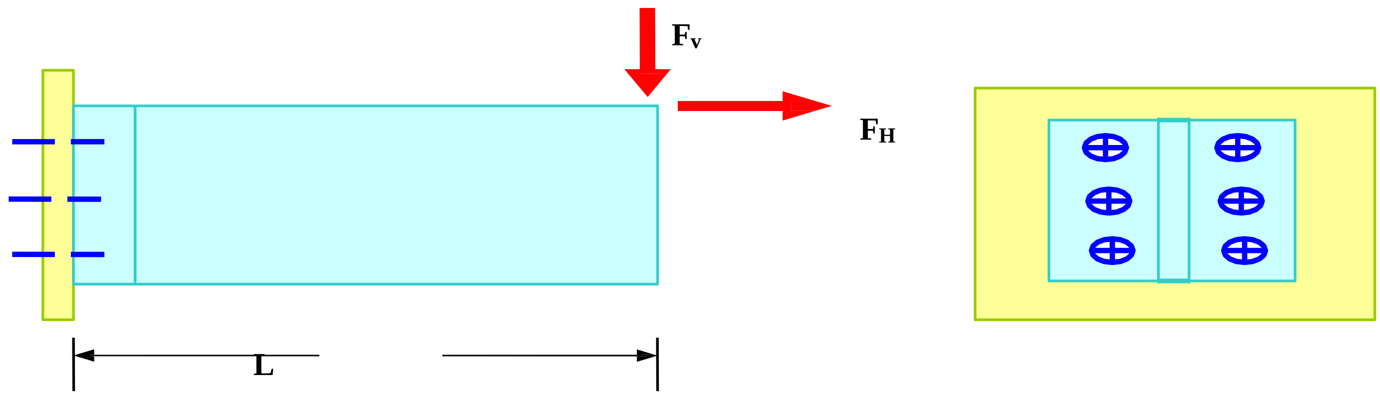


Figure 11.1.1: Eccentrically loaded bolted joint

It is easy to note that the tension in the screws cannot be obtained by equations of statics alone. Hence, additional equations must be formed to solve for the unknowns for this statically indeterminate problem. Since there is a tendency for the bracket to rotate about point O then, assuming the bracket to be rigid, the following equations are easily obtained.

$$\theta \approx \tan\theta = \frac{y_1}{l_1} = \frac{y_2}{l_2} = \frac{y_3}{l_3}$$

where y_i =elongation of the i-th bolt

l_i =distance of the axis of the i-th bolt from point O.

If the bolts are made of same material and have same dimension, then

$$f_i = k y_i$$

where f_i =force in the i-th bolt

k =stiffness of the bolts

Thus $f_i \infty l_i$ or $f_i = \alpha l_i$ (α =proportionality constant)

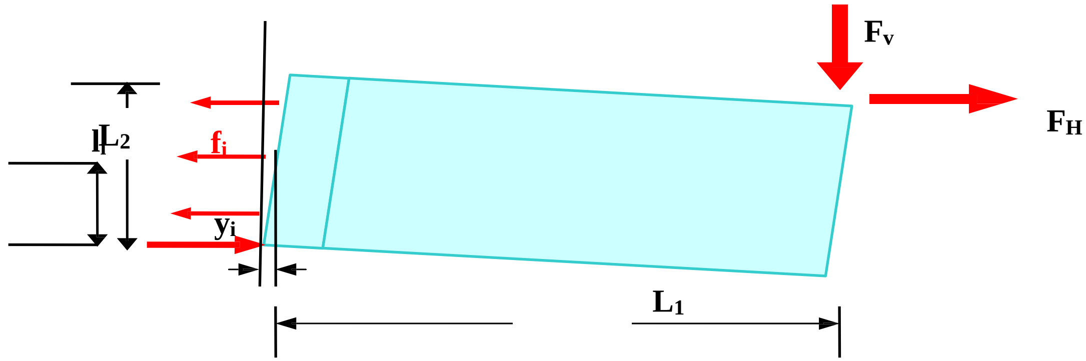


Figure 11.1.2: Determination of forces in bolts

Using the moment balance equations about O, the lowermost point in the left side, the following equation is obtained.

$$2\sum f_i l_i = F_h L_1 + F_v L_2$$

i.e., $\alpha = \dfrac{F_h L_1 + F_v L_2}{2\sum l_i^2}$. The factor 2 appears because there are two bolts in a row.

Thus the force in the i-th screw is

$$f_i = \left(\frac{F_h L_1 + F_v L_2}{2\sum l_i^2}\right) l_i + \frac{F_h}{n}$$

, where n = total number of bolts.

where s_t=allowable tensile stress of the bolt.

Note that F_v causes also direct shear in the bolt. Its effect may be ignored for a preliminary design calculation.

2. Eccentrically loaded riveted joint:

Consider, now, a bracket, which carries a vertical load F . The bracket, in this case, is connected to the wall by four rivets as shown in figure 11.1.2. The force,

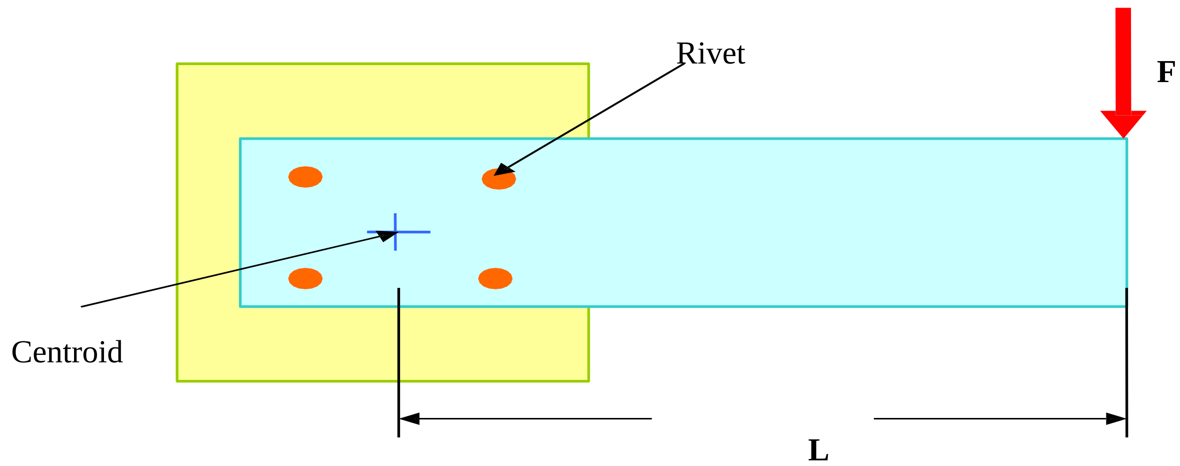


Figure 11.1.3: Eccentrically loaded rivet joint

in addition to inducing direct shear of magnitude $\frac{F}{4}$ in each rivet, causes the whole assembly to rotate. Hence additional shear forces appear in the rivets.

Once again, the problem is a statically indeterminate one and additional assumptions are required. These are as following:

(i) magnitude of additional shear force is proportional to the distance between the rivet center and the centroid of the rivet assembly, whose co- ordinates are defined as

$$\bar{x} = \frac{\sum A_i x_i}{\sum A_i}, \quad \bar{y} = \frac{\sum A_i y_i}{\sum A_i}$$

(A_i=area of the cross-section of the i-th rivet)

(ii) directions of the force is perpendicular to the line joining centroid of the rivet group and the rivet center and the sense is governed by the rotation of the bracket.

Noting that for identical rivets the centroid is the geometric center of the rectangle, the force in the i-th rivet is

$$f_i = \alpha l_i$$

where α =proportional constant

l_i=distance of the i-th rivet from centroid.

Taking moment about the centroid

$$\sum_i f_i l_i = FL \quad \text{or} \quad \alpha = \frac{FL}{\sum_i l_i^2}$$

Thus, the additional force is $f_i = \dfrac{FL}{\sum_i l_i^2} l_i$.

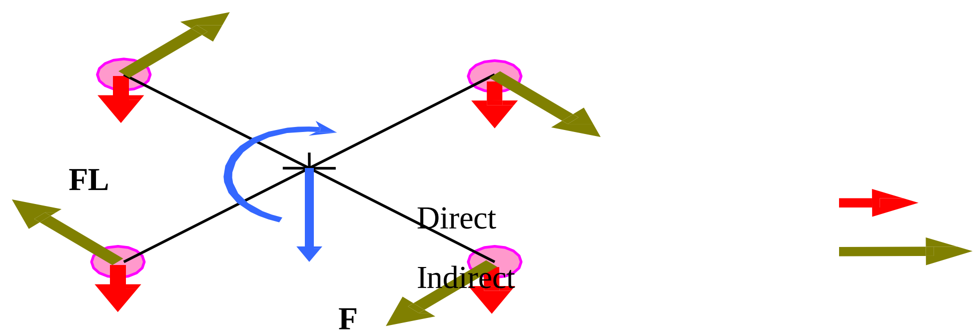


Figure 11.1.4: Forces on rivets due to

The net force in the i-th rivet is obtained by parallelogram law of vector addition as

$$f_i' = \sqrt{f_i^2 + \left(\frac{F}{4}\right)^2 + 2\cdot\frac{F}{4} f_i \cos\theta_i}$$

where θ_i =angle between the lines of action of the forces shown in the figure.

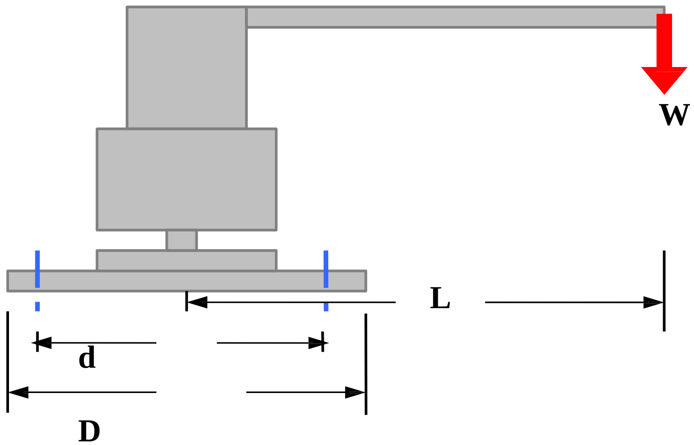
For safe designing we must have

$$\tau = \max\left(\frac{f_i'}{A}\right) \le s_s$$

where s_s =allowable shear stress of the rivet.

Model questions and answers:

Q. 1. The base of a pillar crane is fastened to the foundation by *n* bolts equally placed on a bolt circle of diameter *d*. The diameter of the pillar is *D*. Determine the maximum load carried by any bolt when the crane carries a load *W* at a distance *L* from the center of the base.



Ans. In this case the pillar have a tendency to topple about the point on the outer diameter lying closest to the point of application of the load.

Choose the line joining the center of the base and the point of application of the load as the reference line.

Q. 2. A bracket is supported by means of 4 rivets of same size as shown in figure 6. Determine the diameter of the rivet if the maximum shear stress is 140 MPa.

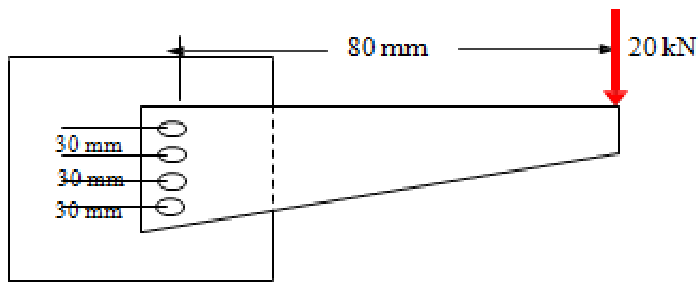
Ans. F_1 = The direct shear force =5 kN per rivet. The maximum indirect shear force occurs in the topmost or bottommost rivet and its magnitude is

$$F_2 = \frac{20 \times 80}{2 \times 15^2 + 2 \times 45^2} \times 45 \text{ kN}$$ and the direction is horizontal.

Therefore the maximum shear force on the rivet assembly is $F = \sqrt{F_1^2 + F_2^2}$.

Hence

$$\frac{\pi d}{4} \times s = F$$ which yields $d \approx 16$ mm.



SIMPLE CONNECTIONS

Applied load passes through C.G of connections

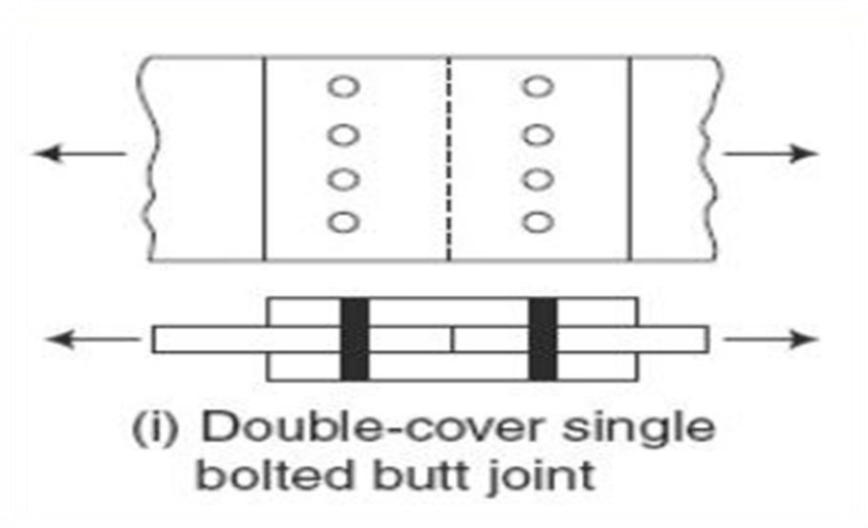


Fig -1a

Fig -1b

ECCENTRIC CONNECTIONS

Applied load does not pass through C.G of connections

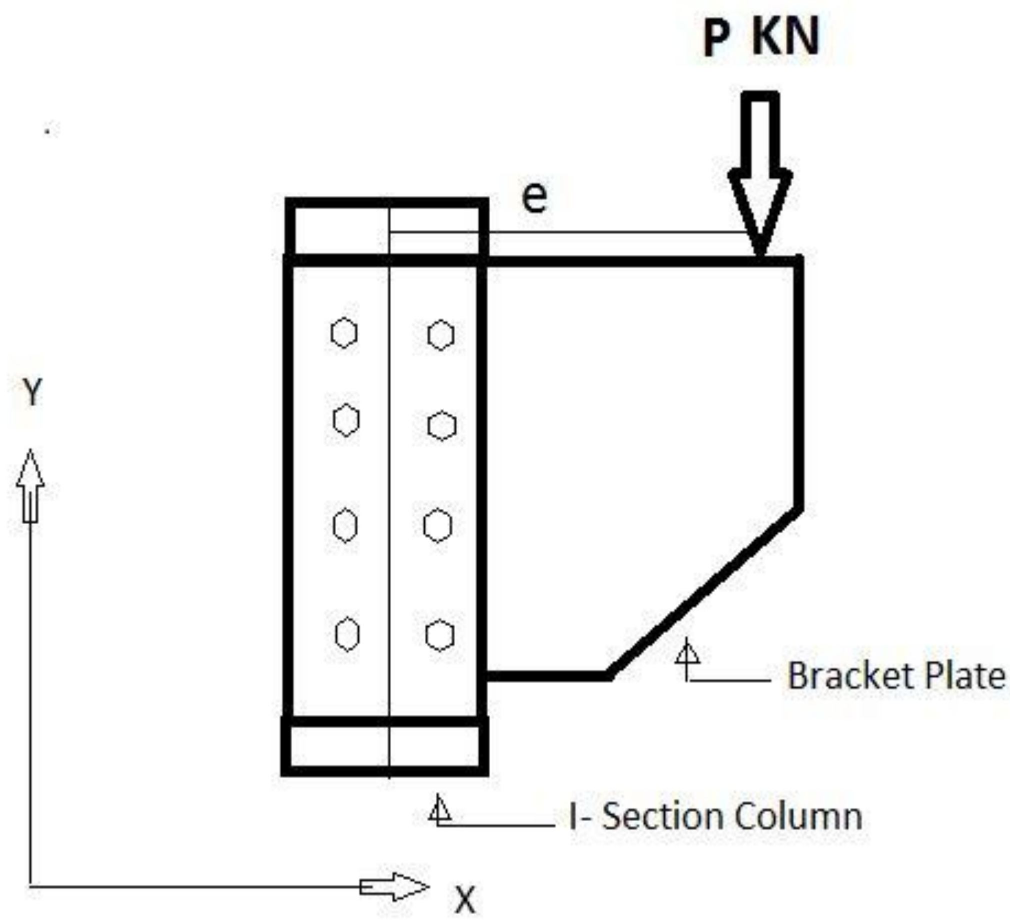


Fig 2

CONNECTIONS SUBJECTED TO ECCENTRIC SHEAR

Seat Connections
Framed Connections
Bracket Connections

SEAT CONNECTION

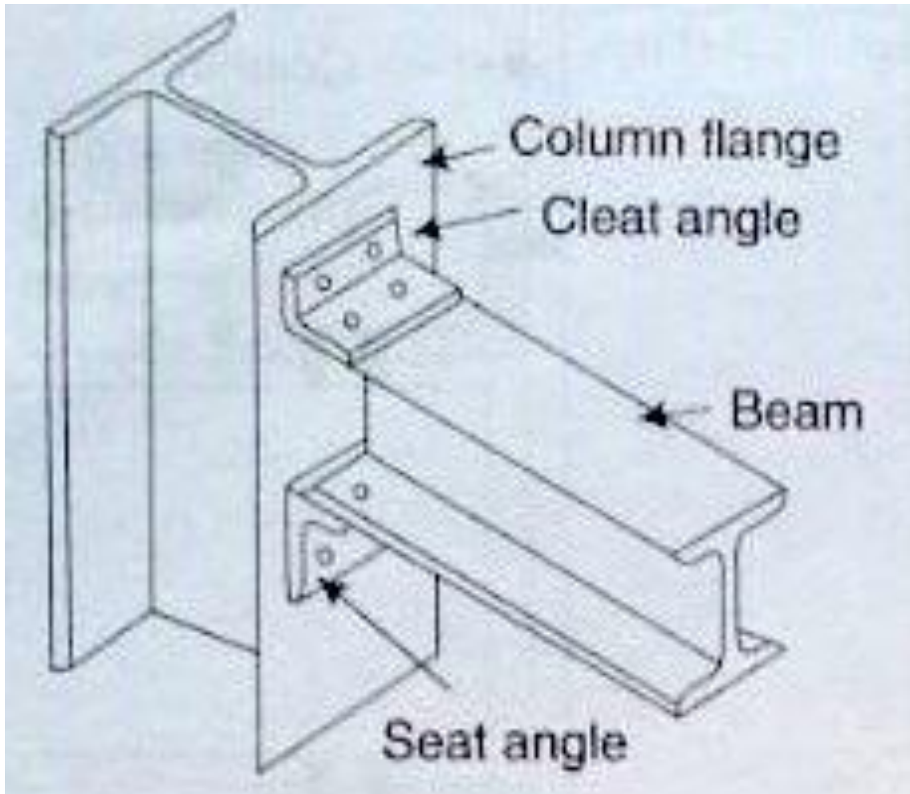


Fig 3a – Unstiffened Seat Connections

SEAT CONNECTION

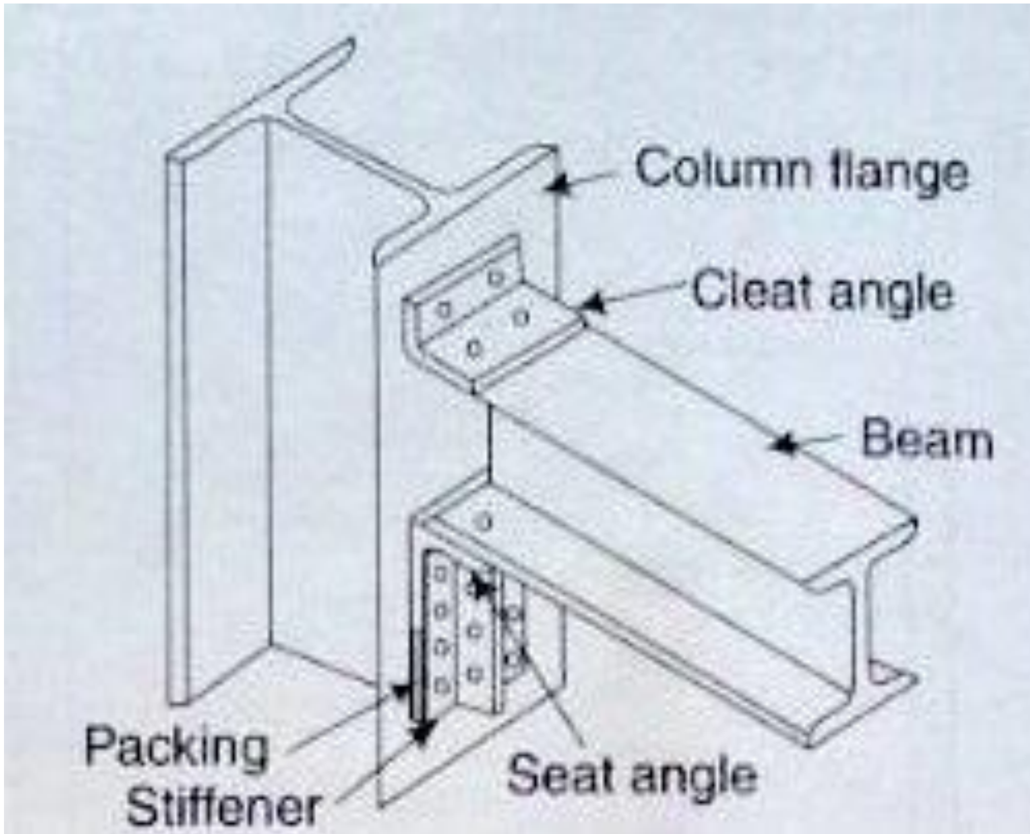


Fig 3b – Stiffened Seat Connections

FRAMED CONNECTIONS

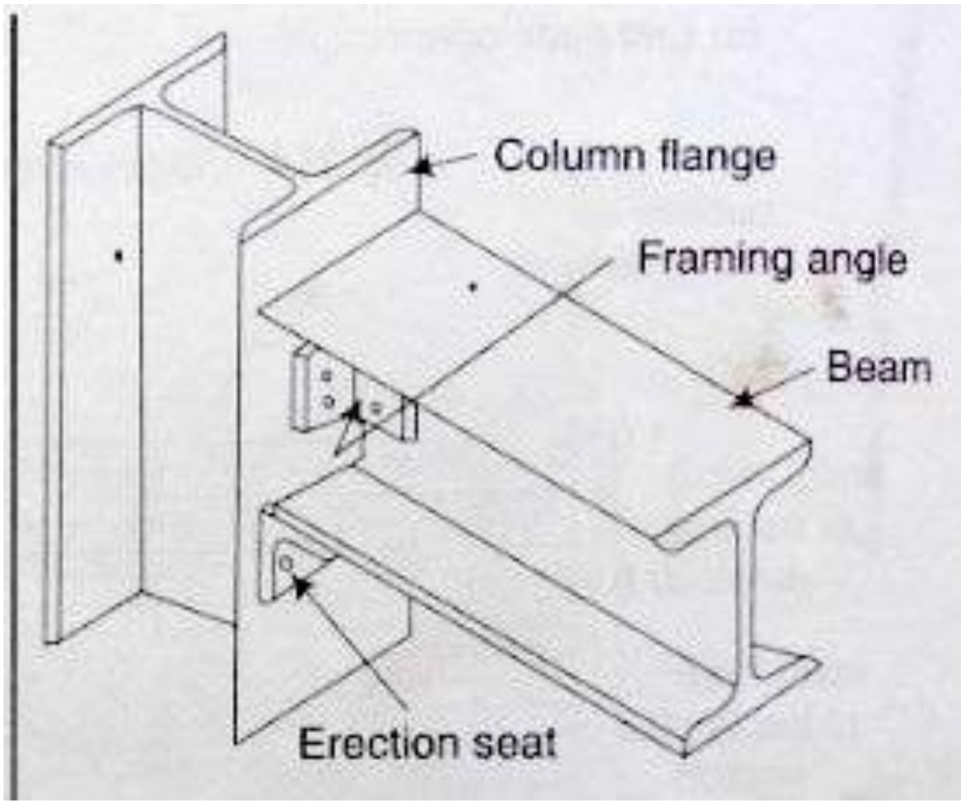


Fig - 4

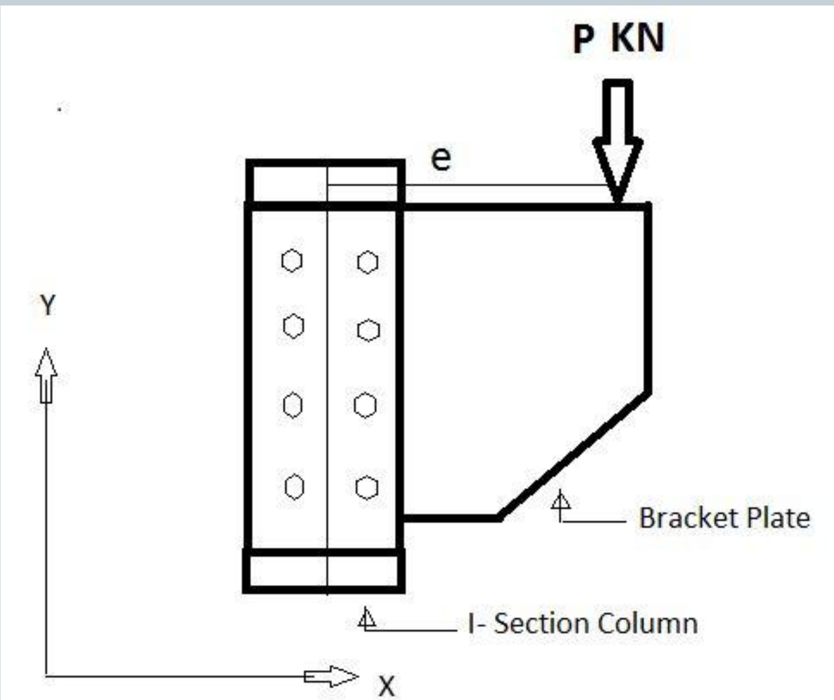
BRACKET CONNECTIONS

1) Bolted bracket – type I connections
2) Bolted bracket – type II connections
3) Welded bracket – type I connections
4) Welded bracket – type II connections

Theoretical Background

BRACKET CONNECTIONS

BOLTED BRACKET CONNECTIONS TYPE 1



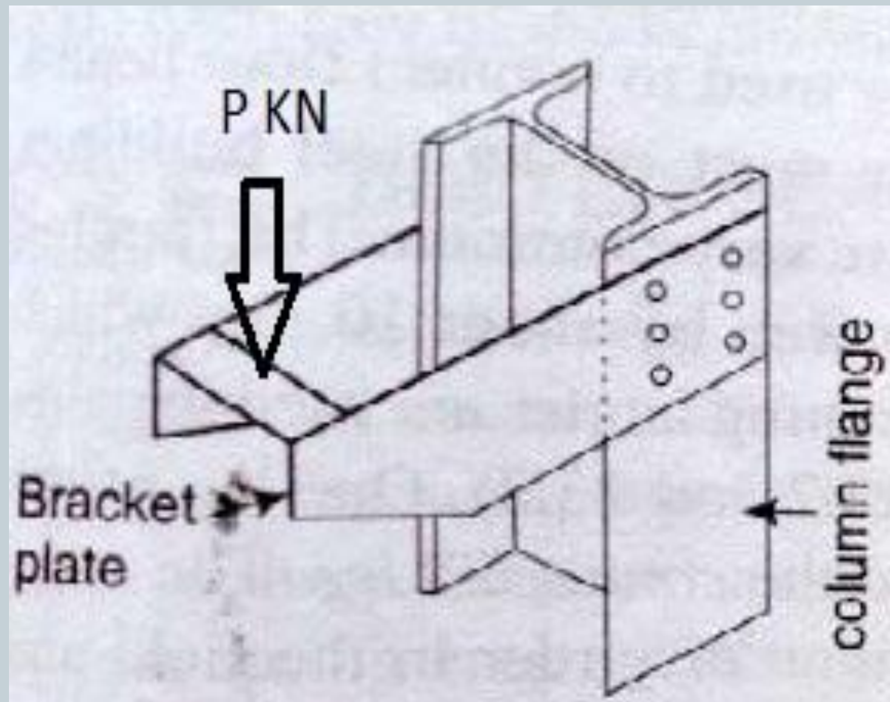


Fig 5

FORCES ON BRACKET TYPE 1

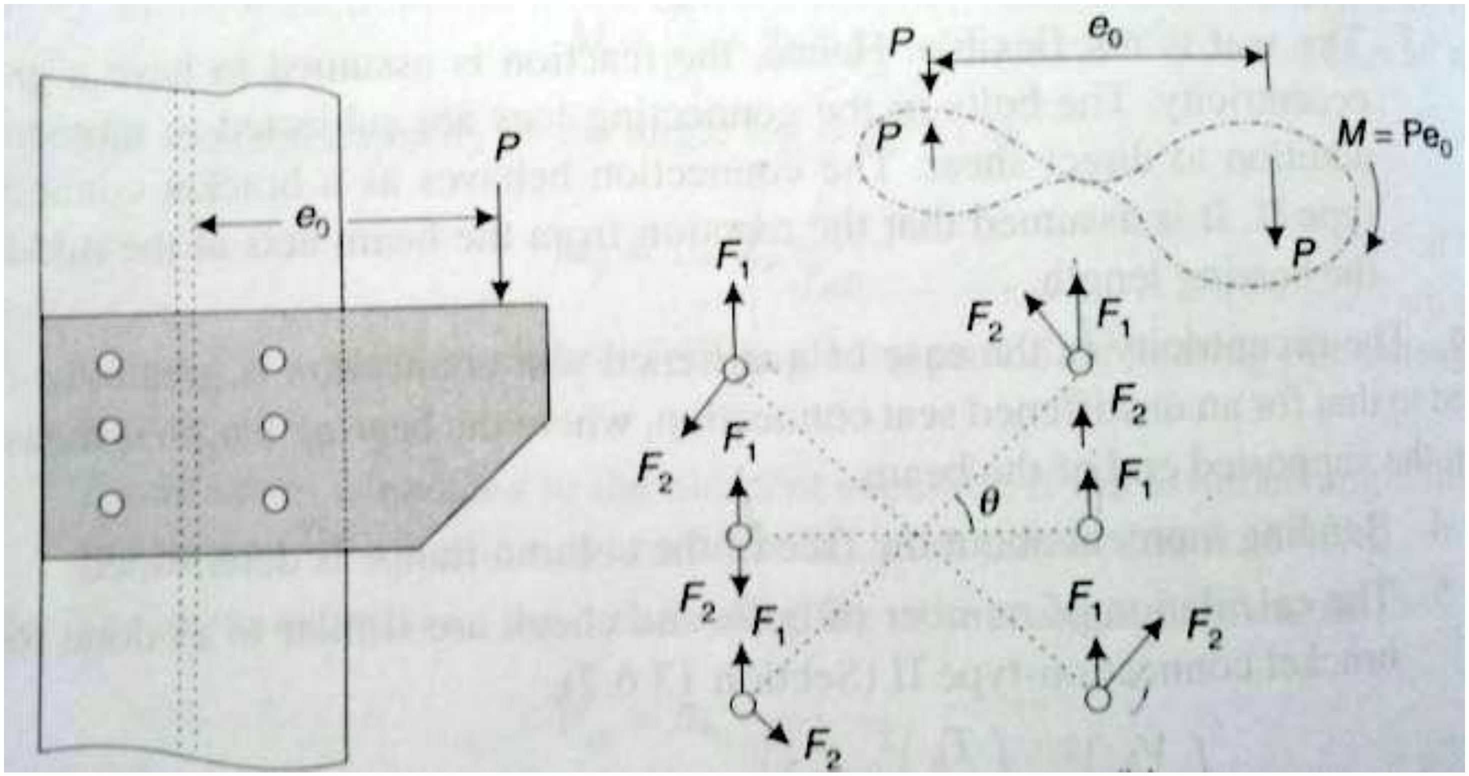


Fig 6

From the assumption made in the concentric bolted joints, ‘ the load over the joint is shared equally by all the bolts’ ,force in any bolt due to direct load is,

$$F_1 = \frac{P}{n} \qquad (1)$$

We know that,

$$\frac{T}{J} = \frac{f}{r}$$

And also

Force = Stress * Area

Therefore,

$$F_2 \propto r$$

$$F_2 = k * r$$

$$k = \frac{F_2}{r}$$

Therefore, the Torque about the center of rotation of the bolt group

$$= F_2 r = krr = kr^2$$

Total resisting toq $= \sum kr^2$

$$= k\sum r^2$$

$$= \frac{F_2}{r}\sum r^2$$

The resisting torque should be equal to torque over the connection. Hence,

$$M = \frac{F_2}{r}\sum r^2$$

$$Pe_0 = \frac{F_2}{r}\sum r^2$$

$$\frac{Pe_0 r}{\sum r^2} = F_2$$

Force F_2 is maximum when distance r is maximum. Let the distance of the extreme bolt be r_n . then,

$$F_2 = \frac{Pe_0 r_n}{\sum r^2} \tag{2}$$

The two forces F_1 and F_2 act at some angle on various bolts in the connection. Let θ be the angle between these forces on the critical bolt. Then the resultant force F on the critical bolt will be

$$F = \sqrt{({F_1}^2 + {F_2}^2 + 2F_1F_2\cos\theta)} \tag{3}$$

For the connection to be safe, this force must be less than the strength of the bolt.

BOLTED BRACKET CONNECTIONS TYPE 2

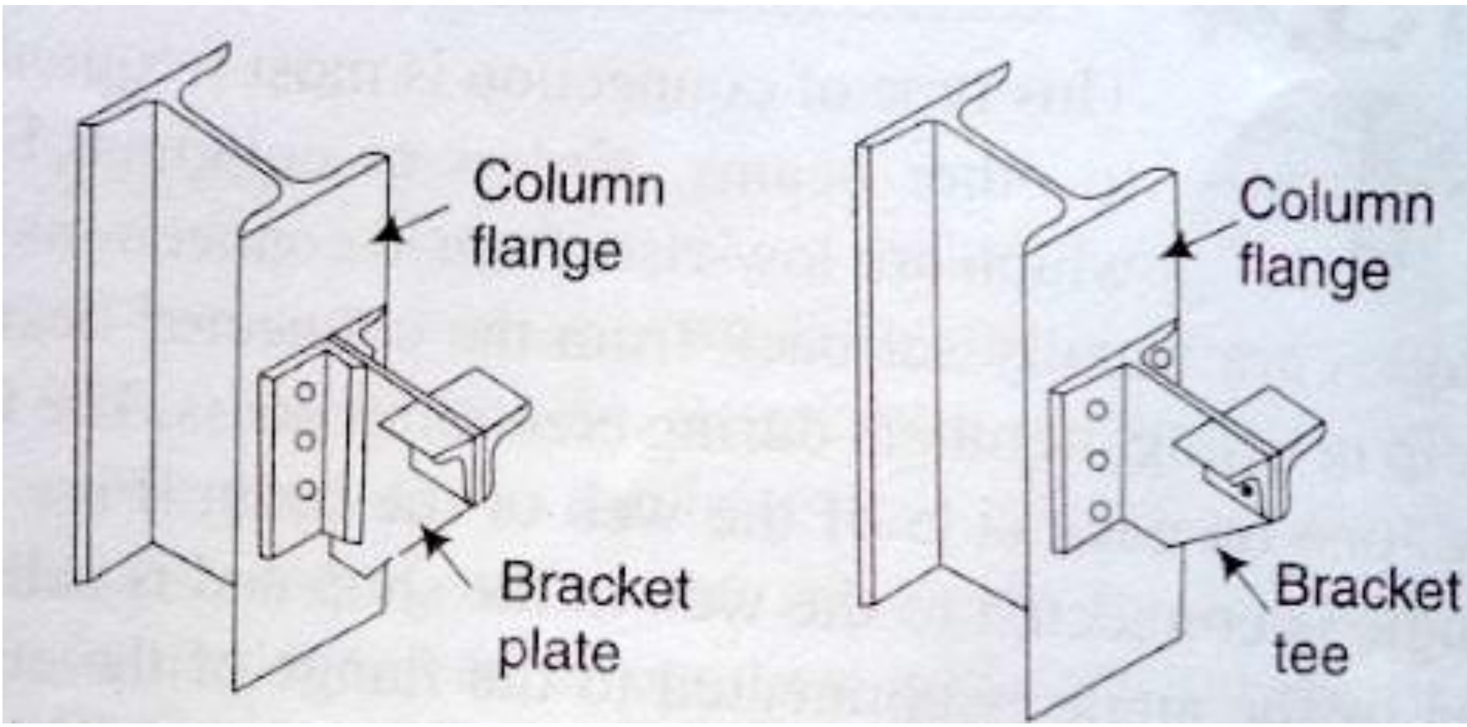
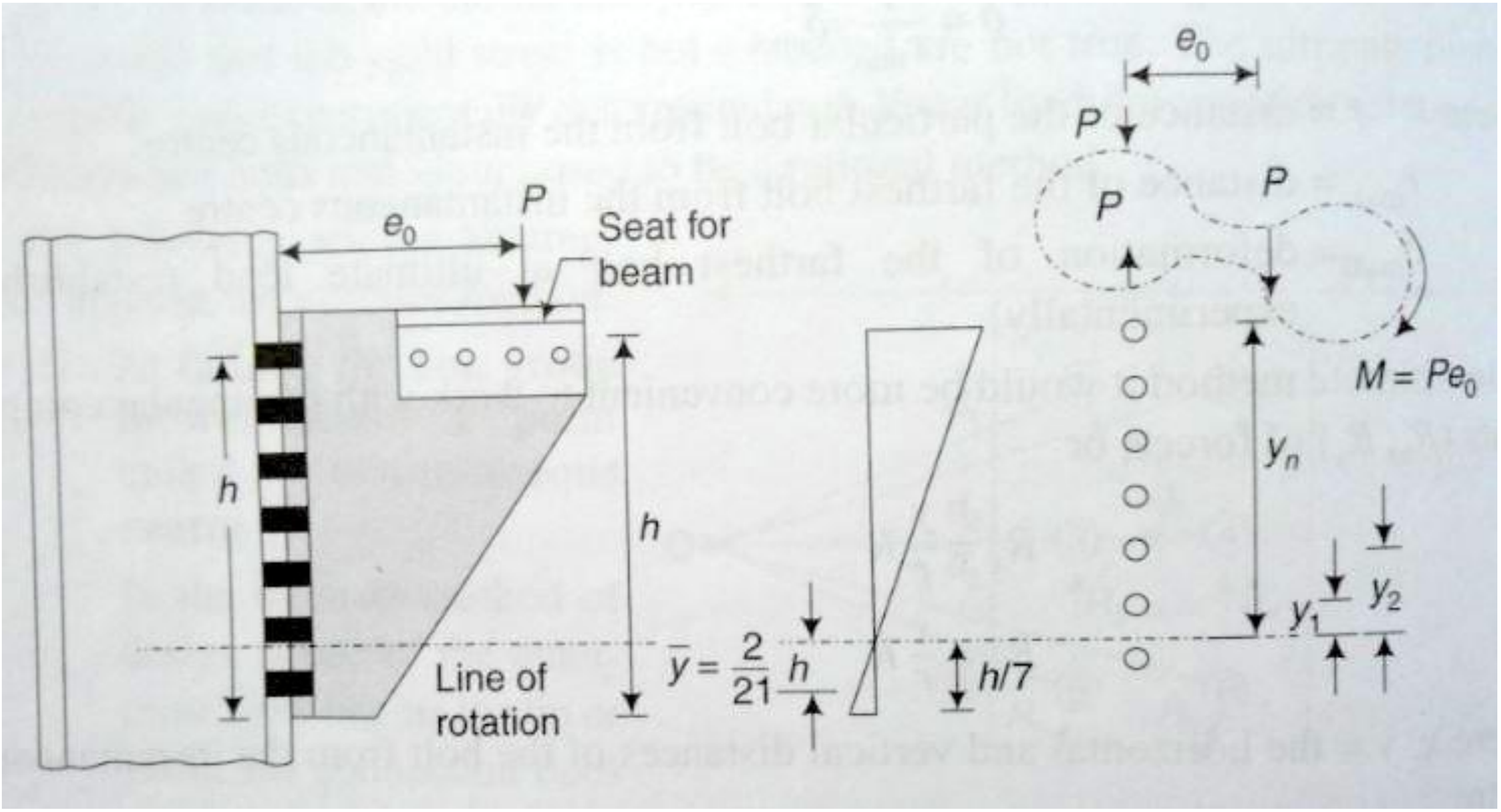


Fig 7



e_0
P
P
P
P
Seat for beam
h
h
M = Pe_0
y_n
y_2
y_1
Line of rotation
$\bar{y} = \frac{2}{21} h$
h/7

Where,

$M = Pe_o$ moment in Nmm, caused by the eccentric load

e_o = the eccentricity of the load P from the bolt plane to the line of action of load in mm

P = the load acting over the joint in N

M' = moment of resistance provided by bolts in tension

V_b = force in a bolt due to direct shear P

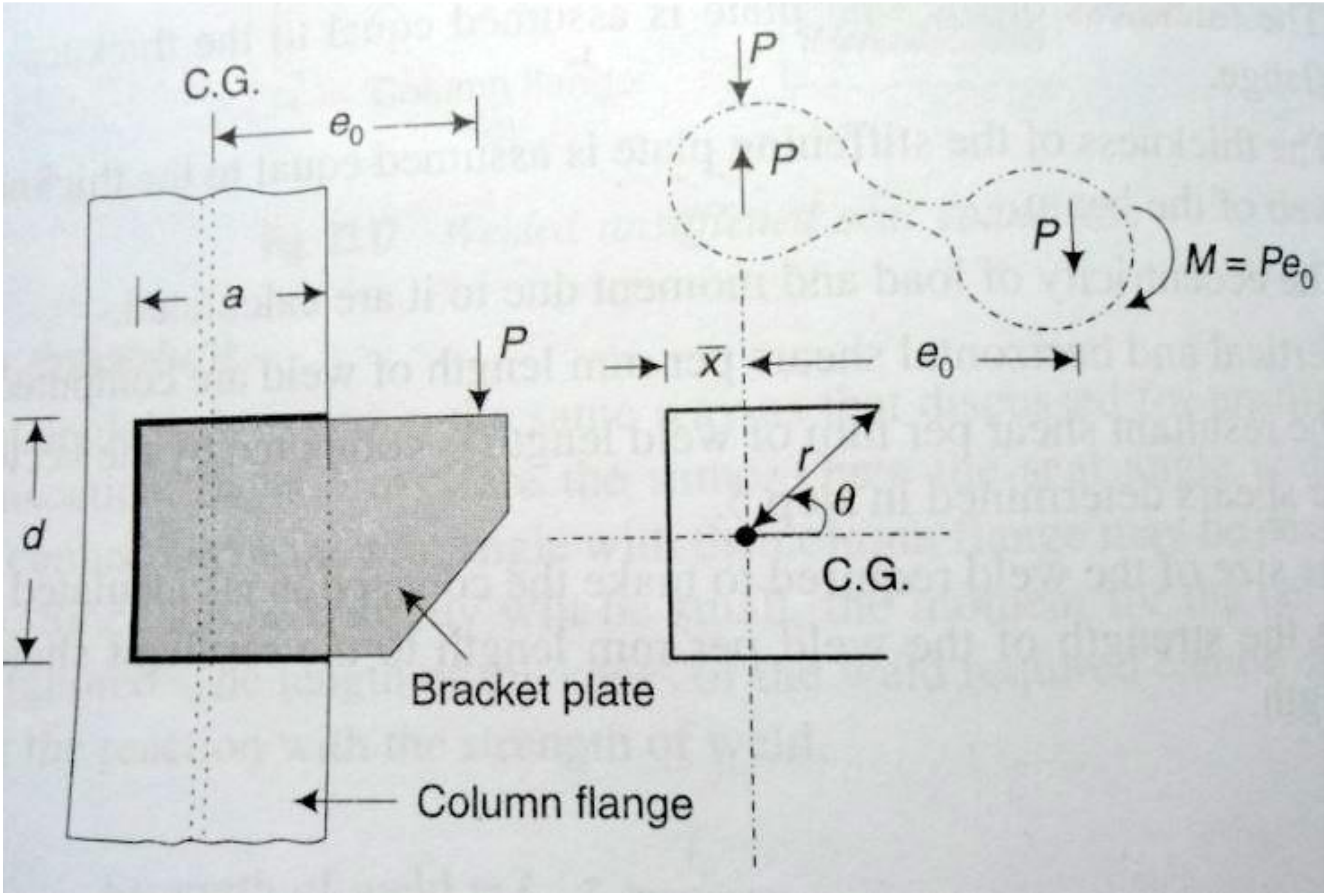
T_b = tensile force in the bolt due to bending moment (Pe_o)

n = number of bolts in the bolt group

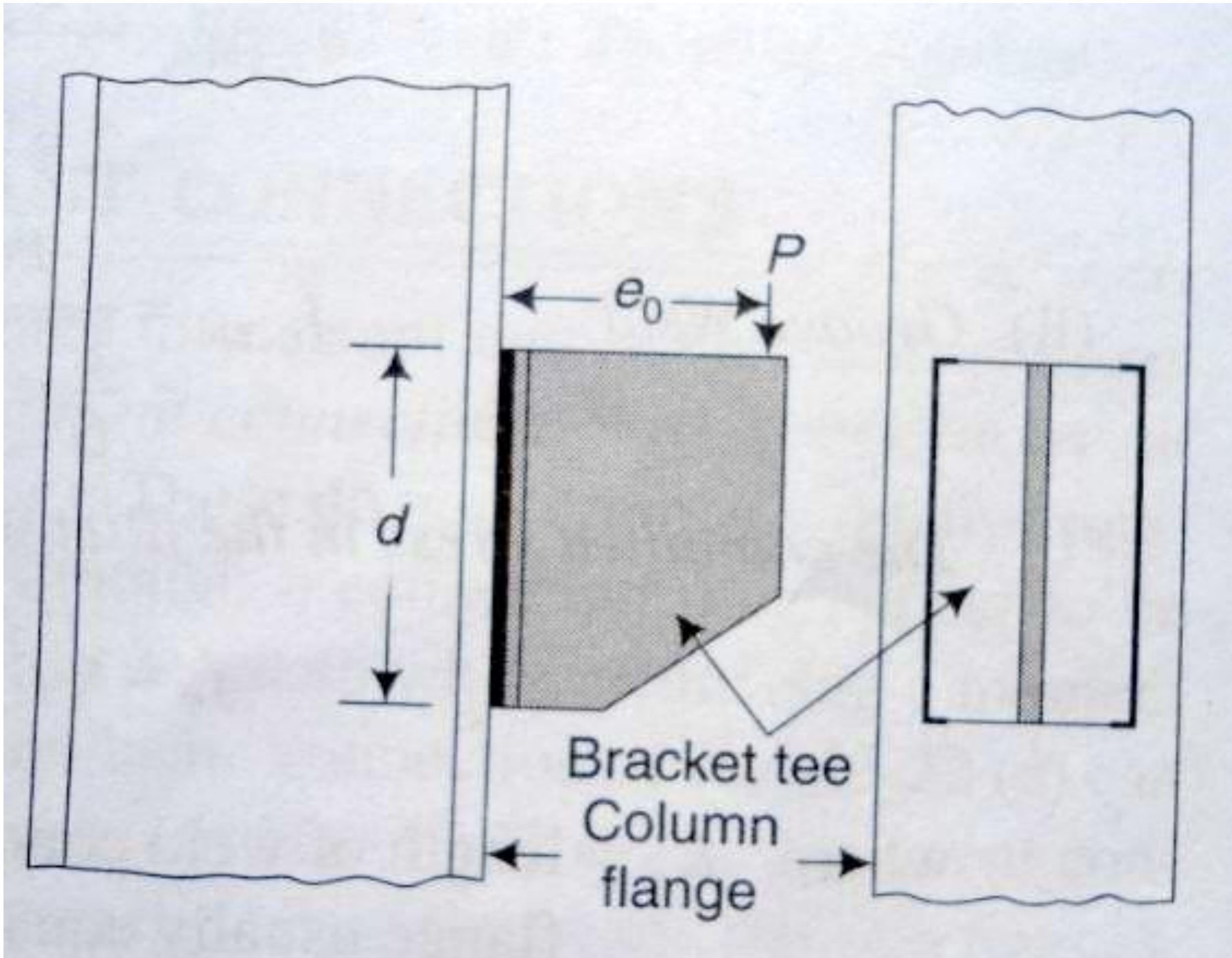
$y_1, y_2 \ldots y_n$ = distance of the bolts in tension from the axis of rotation

y = distance as shown in figure.

WELDED BRACKET CONNECTIONS TYPE 1



WELDED BRACKET CONNECTIONS TYPE 2



Experimental Analysis

Reference paper -The Strength of Eccentrically Loaded Shear Connections
(Ultimate Method)

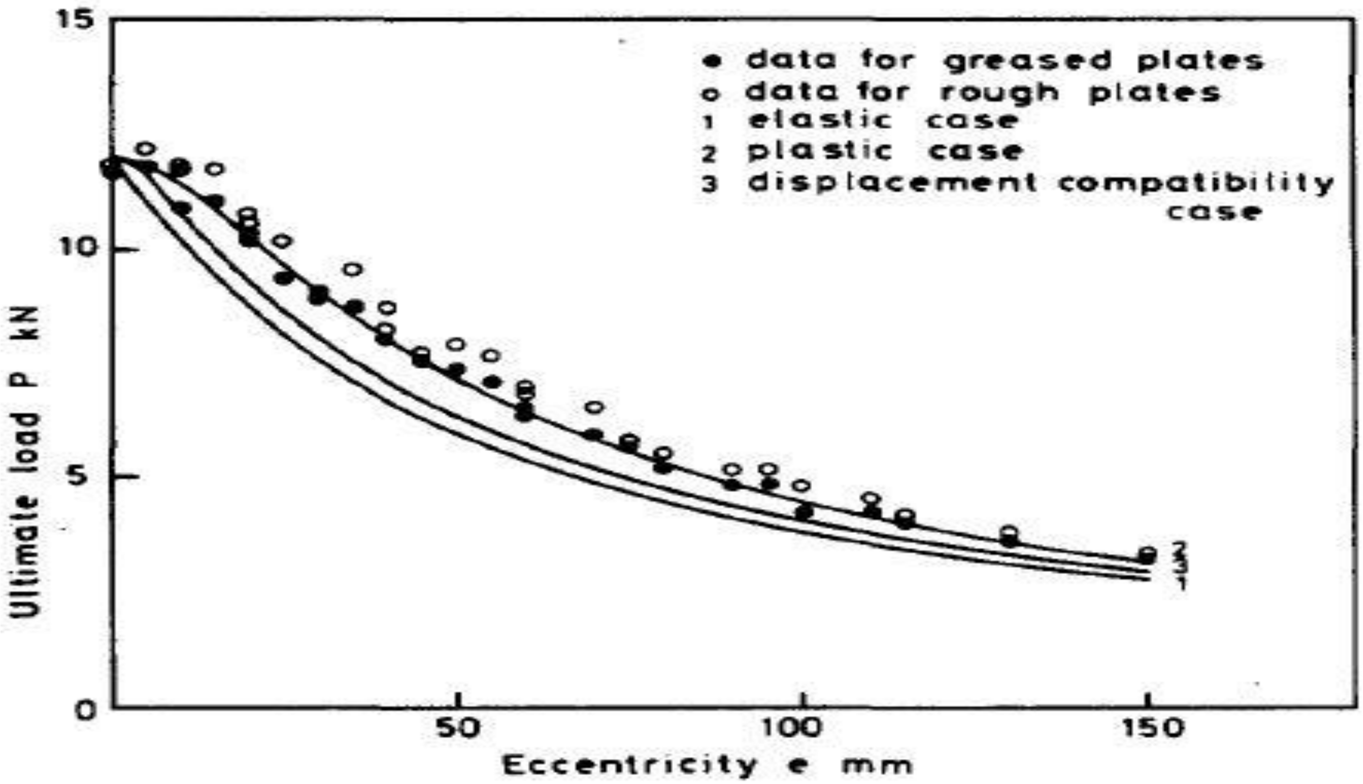


Figure 12 Ultimate load versus eccentricity for 2 by 2 groups of hollow steel rivets, pitch and gauge 60 mm

Reference paper - Eccentric Connection design- geometric approach
(Geometric Method)

TABLE 1. Comparison of Methods and Experimental Results

Mark (1)	e in. (2)	e cm (3)	$n_x \times n_y$ (4)	S_y in. (5)	S_y cm (6)	S_x in. (7)	S_x cm (8)	Test kips (9)	Test (kN) (10)	C_{test} (11)	C_g (12)	C_{sm} (13)	C_u (14)
(a) Normalized Bolt Capacities													
B1	8	20	1 × 5	2.5	6	—[a]	—[a]	225	1,001	1.52	1.62	1.69	1.70
B2	10	25	1 × 5	3	8	—[a]	—[a]	240	1,023	1.55	1.57	1.63	1.66
B3	12	30	1 × 5	3	8	—[a]	—[a]	190	485	1.28	1.35	1.39	1.39
B4	13	33	1 × 6	3	8	—[a]	—[a]	251	1,116	1.70	1.84	1.89	1.88
B5	15	38	1 × 6	3	8	—[a]	—[a]	221	983	1.49	1.63	1.66	1.65
B6	12	30	2 × 4	3	8	2.5	6	264	1,174	1.78	1.86	1.89	2.02
B7	15	38	2 × 4	3	8	2.5	6	212	943	1.43	1.52	1.53	1.61
B8	15	38	2 × 5	2	6	2.5	6	266	1,183	1.80	1.88	1.91	2.09
(b) Normalized Rivet Capacities													
TP-1	2.5	6	1 × 3	3	8	—[a]	—[a]	216	961	2.02	1.51	1.78	2.00
TP-2	3.5	9	1 × 3	3	8	—[a]	—[a]	161	716	1.51	1.28	1.43	1.64
TP-3	6.5	17	1 × 3	3	8	—[a]	—[a]	100	445	0.94	0.82	0.86	0.92
TP-4	2.5	6	1 × 6	3	8	—[a]	—[a]	550	2,446	5.15	4.36	4.55	5.22
TP-5	4.5	11	1 × 6	3	8	—[a]	—[a]	440	1,957	4.12	3.61	3.82	4.25
TP-6	6.5	17	1 × 6	3	8	—[a]	—[a]	362	1,610	3.39	3.00	3.17	3.38
TP-7	3.5	9	2 × 2	3	8	2.5	6	222	987	2.08	1.60	1.53	1.85
TP-8	6.5	17	2 × 2	3	8	2.5	6	120	534	1.12	0.89	0.88	1.15
TP-9	3.5	9	2 × 4	3	8	2.5	6	568	2,526	5.32	4.59	4.87	5.28
TP-10	6.5	17	2 × 4	3	8	2.5	6	354	1,575	3.31	3.11	3.22	3.48

[a]Not applicable.

UNIT III

ANALYSIS AND DESIGN OF INDUSTRIAL BUILDINGS

Example Problem

An Industrial building of plan 15m×30m is to be constructed as shown in Fig.E1. Using plastic analysis, analyse and design the single span portal frame with gabled roof. The frame has a span of 15 m, the column height is 6m and the rafter rise is 3 m and the frames are spaced at 5 m centre-to-centre. Purlins are provided over the frames at

2.7 m c/c and support AC sheets. The dead load of the roof system including sheets, purlins and fixtures is 0.4 kN/m^2 and the live load is 0.52 kN/m^2. The portal frames support a gantry girder at 3.25 m height, over which an electric overhead travelling (EOT) crane is to be operated. The crane capacity is to be 300 kN and the crane girder weighs 300 kN while the crab (trolley) weight is 60 kN.

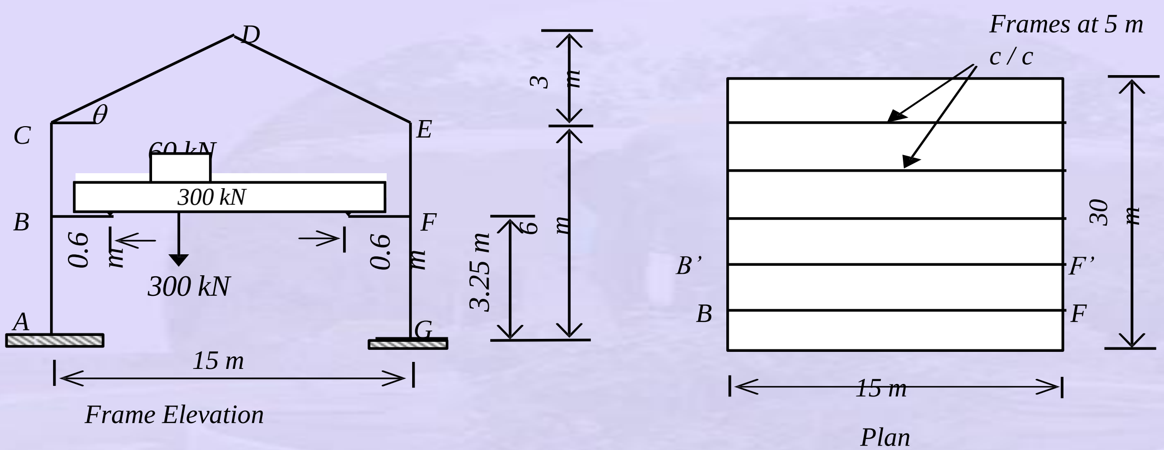


Fig. E1 Details of an Industrial Building

1.0 Load Calculations

1.1 Dead Load of roof given as 0.4 kN/m^2

1.2 Dead load/m run on rafter = 0.4 * 5 ≈ 2.0 kN/m

Live load/m run on rafter = 0.52 * 5 ≈ 2.6 kN/m

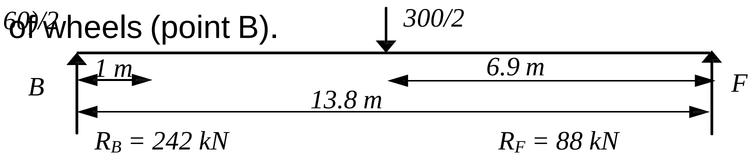
1.3 Crane Load

The extreme position of crane hook is assumed as 1 m from the centre line of rail. The span of crane is approximately taken as 13.8 m. And the wheel base along the gantry girder has been taken as 3.8 m

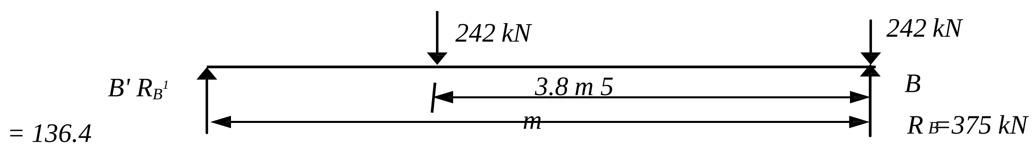
1.3.1 Vertical load on gantry

The weight of the crane is shared by two portal frames At the extreme position of

crab, the reaction on wheel due to the lifted weight and the crab can be obtained by taking moments about the centreline of wheels (point B).



To get maximum wheel load on a frame from gantry girder BB', taking the gantry girder as simply supported.



Centre to centre distance between frames is 5 m c/c. Assuming impact

factor of 25%

Maximum wheel Load @ B = 1.25 (242 (1 + (5-3.8)/5)

= 375 kN.

Minimum wheel Load @ B = (88 /242)*375

=136.4 kN

1.3.2 Transverse Load (Surge):

Lateral load per wheel = 5% (300 + 60)/2 = 9 kN

(i.e. Lateral load is assumed as 5% of the lifted load and the weight of the crab acting on each rail).

Lateral load on each column = $\dfrac{9}{242} * 375$ = 13.9 kN

(By proportion)

1.4 Wind Load

Design wind speed, $V_z = k_1\ k_2\ k_3\ V_b$ From Table 1; IS: 875 (part 3) – 1987

k_1 = 1.0 (risk coefficient assuming 50 years of design life)

From Table 2; IS: 875 (part 3) – 1987 k_2 = 0.8 (assuming terrain category 4) k_3 = 1.0 (topography factor)

Assuming the building is situated in Chennai, the basic wind speed is 50 m

/sec

Design wind speed,

$V_z = k_1\ k_2\ k_3\ V_b$ V_z = 1

* 0.8 *1 * 50

V_z = 40 m/sec

Design wind pressure, $P_d = 0.6 * V_z^2$

$= 0.6 * (40)^2$

$= 0.96$ kN/m^2

1.4.1. Wind Load on individual surfaces

The wind load, W_L acting normal to the individual surfaces is given by $W_L = (C_{pe} - C_{pi}) A*P_d$

(a)Internal pressure coefficient

Assuming buildings with low degree of permeability

$C_{pi} = \pm 0.2$

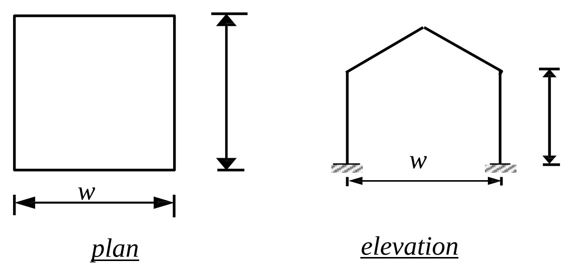
(b)External pressure coefficient

External pressure coefficient for walls and roofs are tabulated in Table 1 (a) and Table 1(b)

1.4.2 Calculation of total wind load

(a) For walls

$h/w = 6/15 = 0.4$ $L/w = 30/15 = 2.0$



h

Exposed area of wall per frame @ 5 m c/c is A = 5 * 6 = 30 m^2

Wind load on wall / frame, $A\,p_d = 30 * 0.96 = 28.8$ kN

Table 1 (a): Total wind load for wall

Wind Angle θ	C_{pe}		C_{pi}	$C_{pe} - C_{pi}$		Total wind(kN) $(C_{pe}-C_{pi})Ap_d$	
	Wind- ward	Lee- ward		Wind ward	Lee ward	Wind ward	Lee ward
0^0	0.7	-0.25	0.2	0.5	-0.45	14.4	-12.9
			-0.2	0.9	-0.05	25.9	-1.4
90^0	-0.5	-0.5	0.2	-0.7	-0.7	-20.2	-20.2
			-0.2	-0.3	-0.3	-8.6	-8.6

(b) For roofs

Exposed area of each slope of roof, per frame (5m length) is

$$A = 5 * \sqrt{(3.0)^2 + (7.5)^2} = 40.4\, m^2$$

For roof, Ap_d = 38.7 kN

Wind angle	Pressure Coefficient			$C_{pe} - C_{pi}$		Total Wind Load(kN) $(C_{pe} - C_{pi})\,Ap_d$	
	C_{pe}	C_{pe}	C_{pi}	Wind ward	Lee ward	Wind ward	Lee ward
	Wind	Lee				Int.	Int.
0^0	-0.328	-0.4	0.2	-0.528	-0.6	-20.4	-23.2
	-0.328	-0.4	-0.2	-0.128	-0.2	-4.8	-7.8
90^0	-0.7	-0.7	0.2	-0.9	-0.9	-34.8	-34.8
	-0.7	-0.7	-0.2	-0.5	-0.5	-19.4	-19.4

2.1 Dead Load

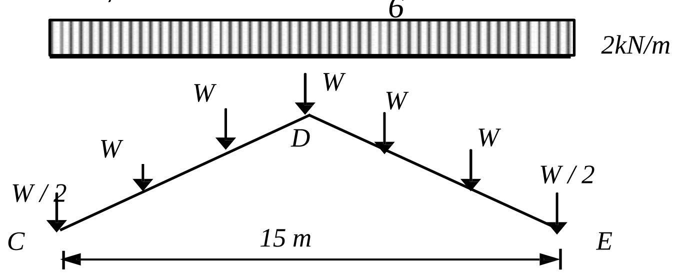
Replacing the distributed dead load of 2kN/m on rafter by equivalent concentrated loads at two intermediate points corresponding to purlin locations on each rafter,

$$W_D = \frac{2.0 * 15}{6} = 5kN$$

2.2 Superimposed Load

Superimposed Load = 2.57 kN/m

Concentrated load , $W_L = \frac{2.57 * 15}{6} = 6.4\, kN$



2.3 Crane Load

Maximum Vertical Load on columns = 375 kN (acting at an eccentricity of 600 mm from column centreline)

Moment on column = 375 *0.6 = 225 kNm.

Minimum Vertical Load on Column = 136.4 kN (acting at an eccentricity of 600 mm) Maximum moment = 136.4 * 0.6 = 82 kNm

1. Partial Safety Factors

2. Load Factors

For dead load, γ_f = 1.5

For leading live load, γ_f = 1.5

For accompanying live load, γ_f = 1.05

3. Material Safety factor

$$\gamma_m = 1.10$$

4.0 Analysis

In this example, the following load combinations is considered, as it is found to be critical. Similar steps can be followed for plastic analysis under other load combinations.

(i) 1.5D.L + 1.5 C .L + 1.05 W.L

4.1. 1.5 D.L + 1.5 C.L+ 1.05 W.L

4.1.1Dead Load and Wind Load along the ridge (wind angle = 0 ^{o})

(a) Vertical Load

w @ intermediate points on windward side w = 1.5 * 5.0 – 1.05 *(4.8/3) cos21.8

= 6 kN.

$$w\ @\ eaves = \frac{6}{2} = 3.0\ kN$$

$\frac{w}{2}$ @ eaves = $\frac{5.0}{2}$ = 2.5 kN

Total vertical load @ the ridge = 3.0 + 2.5 = 5.5 kN

b) Horizontal Load

H @ intermediate points on windward side H = 1.05 * 4.8/3 sin

21.8

= 0.62 kN

H/2 @ eaves points = 0.62/2

= 0.31 kN

H @ intermediate purlin points on leeward side

= 1.05 * 7.8 /3 sin 21.8

= 1 kN

H/2 @ eaves = 0.5 kN

Total horizontal load @ the ridge = 0.5 - 0.31 = 0.19 kN

Table 3: Loads acting on rafter points

Intermediate Points	Vertical Load (kN)		Horizontal Load (kN)	
	Windward	Leeward	Windward	Leeward
	5.2	4.2	0.62	1.0
Eaves	2.6	2.1	0.31	0.5
Ridge	4.7		0.19	

4.1.2 Crane Loading

Moment @ B Moment @ F = 1.5 * 225 = 337.5 kNm

= 1.5 * 82 = 123 kNm

Horizontal load @ B & @ F = 1.5 * 13.9 = 20.8 kN

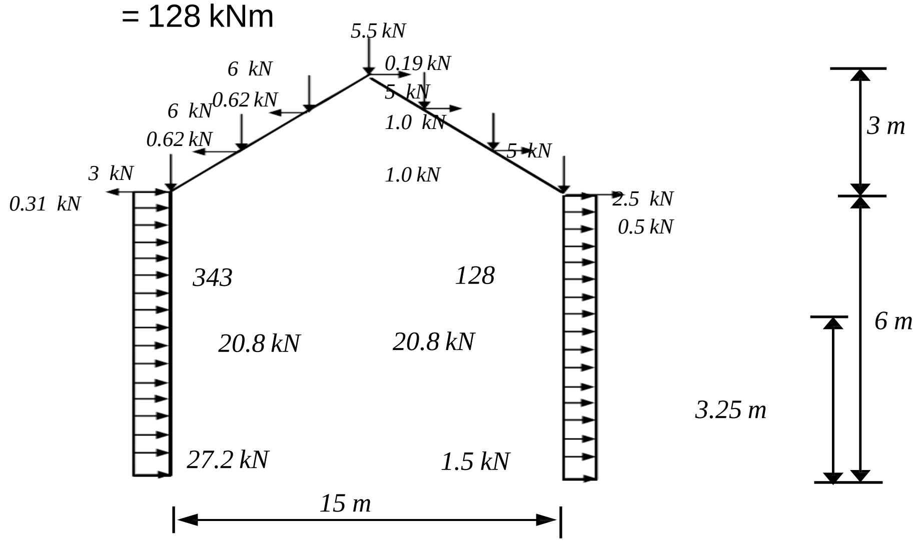
Note: To find the total moment @ B and F we have to consider the moment due to the dead load from the weight of the rail and the gantry girder. Let us assume the weight of rail as 0.3 kN/m and weight of gantry girder as 2.0 kN/m

Dead load on the column = $\left(\frac{2+0.3}{2}\right) * 5 = 5.75\ kN$ acting at e=0.6m

Factored moment @ B & F = 1.5 * 5.75 * 0.6 = 5.2 kNm Total moment

@B = 337.5 + 5.2 = 342 kNm

@ F = 123 + 5.2 = 128 kNm



Factored Load (1. 5D.L+1.5 C.L +1.05 W.L)

4.2 1.5 D.L + 1.5 C.L + 1.05 L.L

4.2.1 Dead Load and Live Load

@ intermediate points on windward side = 1.5 * 5.0 + 1.05 * 6.4
= 14.2 kN

@ ridge = 14.2 kN

@ eaves = 14.2 / 2 ≈ 7.1 kN.

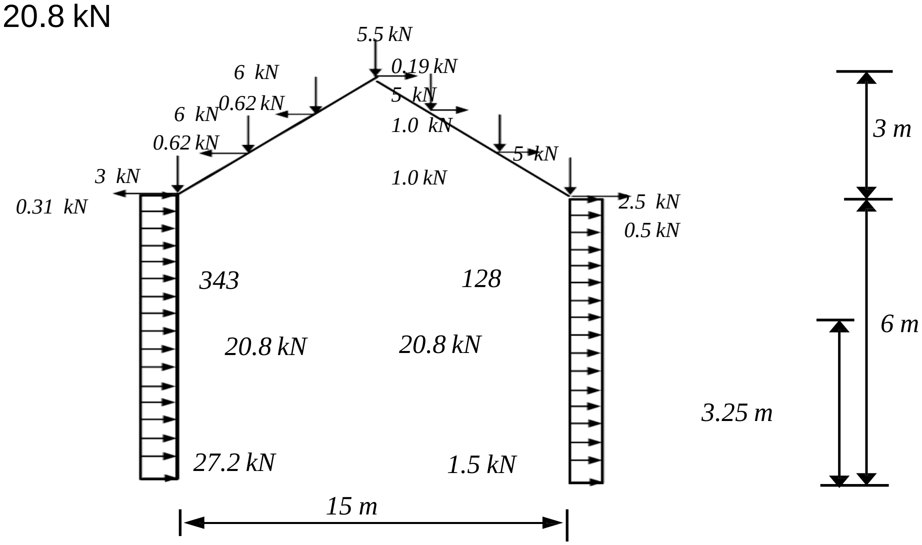
4.2.2 Crane Load

Moment @ B = 342 kNm Horizontal load @

B = 20.8 kN

Moment @ F = 128 kNm

Horizontal load @ F = 20.8 kN



Factored Load (1. 5D.L+1.5 C.L +1.05 W.L)

4.3 Mechanisms

We will consider the following mechanisms, namely

(i) Beam mechanism

(ii) Sway mechanism

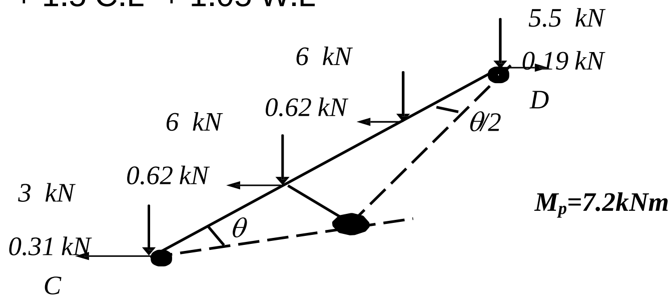
(iii) Gable mechanism and

(iv) Combined mechanism

(v) Beam Mechanism

(1) Member CD

Case 1: 1.5 D.L + 1.5 C.L + 1.05 W.L



Internal Work done, Wi $= M_p\theta + M_p(\theta/2) + M_p(\theta + \theta/2)$

$= M_p(3\theta)$

External Work done, $W_e = 6 * 2.5\theta - 0.62 * 1 * \theta + 6 * 2.5 * \theta/2 – 0.62 * 1 * \theta/2$

$= 21.6\theta$

Equating internal work done to external work done

$W_i = W_e$

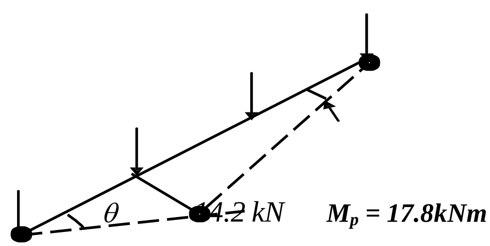
$M_p(3\theta) = 21.6\theta$

$M_p = 7.2$ kNm

Case 2: 1.5 D.L + 1.5 C.L + 1.05 L.L

Internal Work done,

$W_i = M_p 3\theta$ (as in case 1)



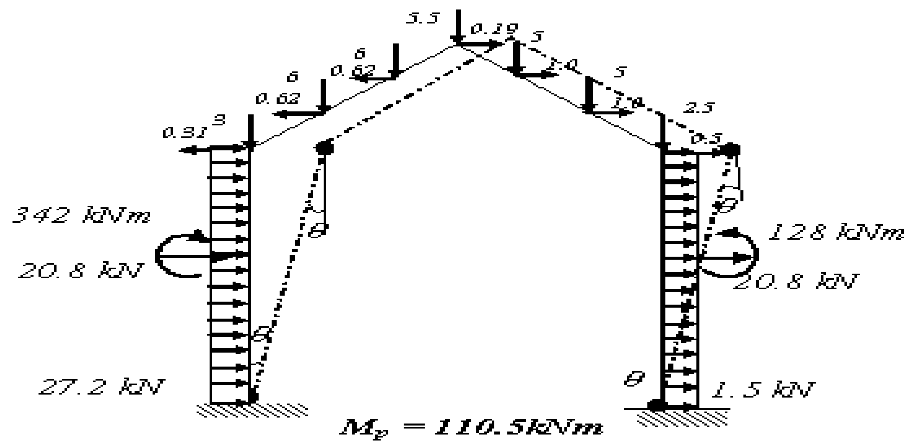
External work done, $W_e = 14.2 * 2.5\,\theta + 14.2 * 2.5\theta / 2$

$= 53.3\theta$

.

4.3.1 Panel Mechanism

Case 1: 1.5 D.L + 1.5 C.L + 1.05 W.L



Internal Work done, $W_i = M_p\ (\theta) + M_p\ (\theta) + M_p\ (\theta) + M_p\ (\theta)$

$= 4M_p\theta$

External Work done, W_e

$W_e = 1/2\ (27.2) * 6\theta + 20.8 * 3.25\theta + 342\theta - 0.31 * 6\theta - 0.62 * 6\theta - 0.62\ (6\theta) + 0.19 * 6\theta + 1.0 * 6\theta + 1.0 * 6\theta + 0.5 * 6\theta + 1/2\ (1.5) * 6\theta + 20.8 * 3.25\theta - 128 * \theta$

$= 442.14\theta$

Equating $W_i = W_c$, we get $4M_p\theta = 442.14\theta$

$M_p = 110.5$ kNm

The second load combination will not govern.

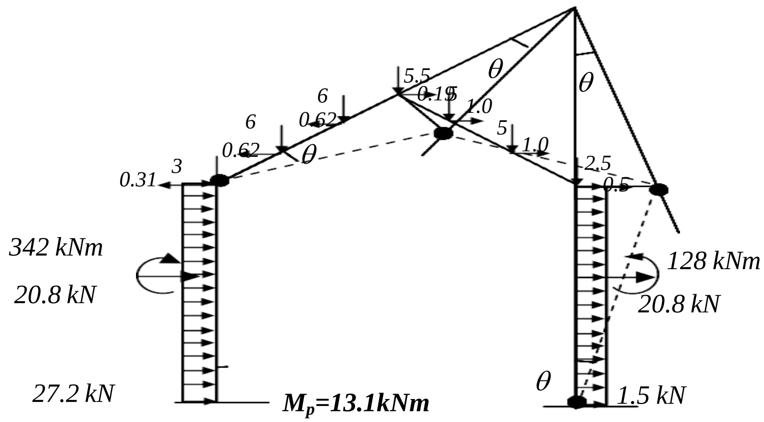
4.3.3 Gable Mechanism

Case 1: 1.5 D.L + 1.05 W.L + 1.5 C.L

Internal Work done = $M_p\theta + M_p 2\theta + M_p (2\theta) + M_p\theta = 6M_p\theta$ External Work done, W_e =

$-0.62 * 1 * \theta - 0.62 * 2 * \theta + 0.19 * 3 * \theta + 1.0 * 4 * \theta + 1.0 * 5 * \theta + 0.5 * 6 * \theta + 6 * 2.5 * \theta + 6 * 5 * \theta + 5.5 * 7.5 * \theta + 5 * 5 * \theta + 5 * 2.5 * \theta + ½ * 1.5 * 6\theta + 20.8 * 3.25 * \theta - 128*\theta$

$W_e = 78.56\theta$

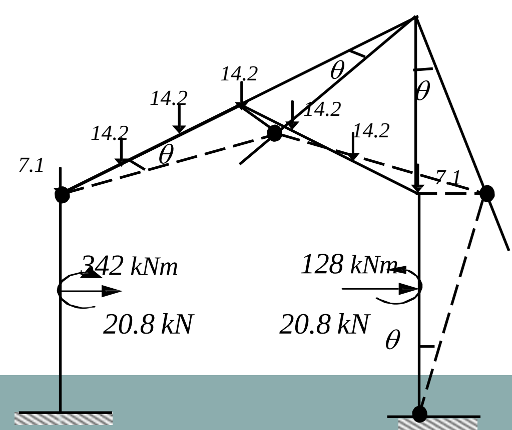


Equating $W_i = W_e$, we get $6M_p = 78.56\theta$

M_p = 13.1 kNm.

Case 2: 1.5 D.L + 1.05L.L + 1.5 C.L

M_p=37.3kNm



Internal Work done, W_i = $M_p\theta + M_p (2\theta) + M_p (2\theta) + M_p\theta = 6M_p\theta$ External Work done, W_e

$= 14.2 * 2.5*\theta + 14.2 * 5 * \theta + 14.2 * 7.5\theta + 14.2 * 5 * \theta + 14.2 * 2.5\theta - 128 * \theta + 20.8 * 3.25\theta$
$= 223.6\theta$

Equating $W_i = W_e$, we get

$6M_p\theta = 223.6\theta$

$M_p = 37.3$ kNm

4.3.4 Combined Mechanism

Case1: 1.5 D.L + 1.05 W.L + 1.5 C.L

(i)

Internal Work done, $W_i = M_p (\theta) + M_p (\theta + \theta/2) + M_p (\theta/2 + \theta/2) + M_p (\theta/2)$

$= M_p (\theta + \theta + \theta/2 + \theta/2 + \theta/2 + \theta/2 + \theta/2)$

$= 4 M_p\theta$

$$M_p = 100.7$$

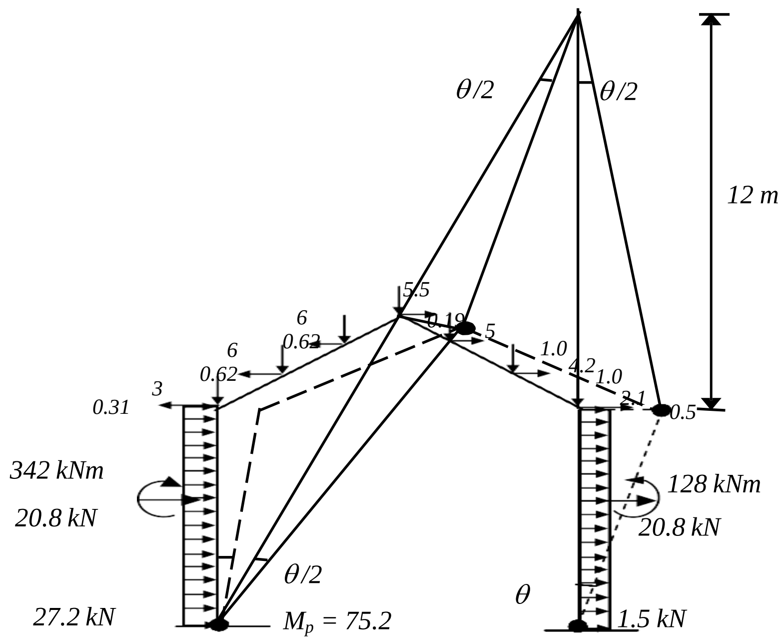
External Work done, W_e=
$1/2 * 27.2 * 6\theta + 20.8 * 3.25* \theta + 342\theta - 0.31 * 12 * \theta/2 - 0.62 * 11 * \theta/2$
$- 0.62 * 10 *\theta/2 + 0.19 * 9 * \theta/2 + 1.0 * 8 * \theta /2 + 1.0 * 7 * \theta /2 + 0.5 * 6* \theta/2 + 1/2 (1.5) * 6\theta/2 + 20.8 * 3.25 * \theta/2 - 128 *$
$\theta/2 - 6 * 2.5 * \theta/2 - 6 * 5.0 * \theta/2 - 5.5 * 7.5 * \theta/2 - 5 * 5 *$
$\theta/2 - 5 * 2.5 * \theta/2$
$= 402.86\theta$

Equating $W_i = W_e$

$4M_p\theta = 402.86\theta$ $M_p = 100.7$ kNm

(ii) Internal work done, $W_i = M_p\,\theta/2 + M_p(\theta/2 + \theta/2) + M_p(\theta/2 + \theta) + M_p\theta$

$W_i = 4M_p\theta$



External Work done,

$$W_e = 20.8*3.25*\frac{\theta}{2} + 342*\frac{\theta}{2} + \frac{1}{2}*27.2*6\left(\frac{\theta}{2}\right) - 0.31*6*\frac{\theta}{2} - 0.62*7*\frac{\theta}{2}$$

$$- 0.62*8*\frac{\theta}{2} + 0.19*9*\frac{\theta}{2} + 6*2.5*\frac{\theta}{2} + 6*5.0*\frac{\theta}{2} + 5.5*7.5*\frac{\theta}{2} + 1.0*10*\frac{\theta}{2}$$

$$+ 1.0*11*\frac{\theta}{2} + 0.5*12*\frac{\theta}{2} + 5*5.0*\frac{\theta}{2} + 5*2.5*\frac{\theta}{2} + 20.8*3.25\,\theta - 128*\theta$$

$$+ \frac{1}{2}*1.5*6\theta$$

$= 300.85\theta$

Equating $W_i = W_e$, we get $4M_p\theta = 300.85\theta$

$M_p = 75.2$ kNm

Similarly analysis can be performed for hinges occurring at purlin locations also but they have been found to be not critical in this example case

From all the above analysis, the largest value of M_p required was for member EG

under

1.5 DL + 1.5 CL + 1.05 WL

Therefore the Design Plastic Moment = 116.1 kNm.

1. DESIGN

For the design it is assumed that the frame is adequately laterally braced so

that it fails by forming mechanism. Both the column and rafter are analysed assuming equal plastic moment capacity. Other ratios may be adopted to arrive at an optimum design solution.

2. Selection of section

Plastic Moment capacity required= 116 kNm Required section modulus, Z_p = M_p/ f_{yd}

$$= \frac{\left(116*10^6\right)}{250/1.10}$$

$$= 510.4*10^3 \; mm^3$$

ISMB 300 @ 0.46 kN/ m
provides

Z_p = 683 * 10^{-3} mm^3

b = 140 mm T_i =

13.1 mm

A = 5.87 * 10^{3} mm^2

t_w =7.7 mm

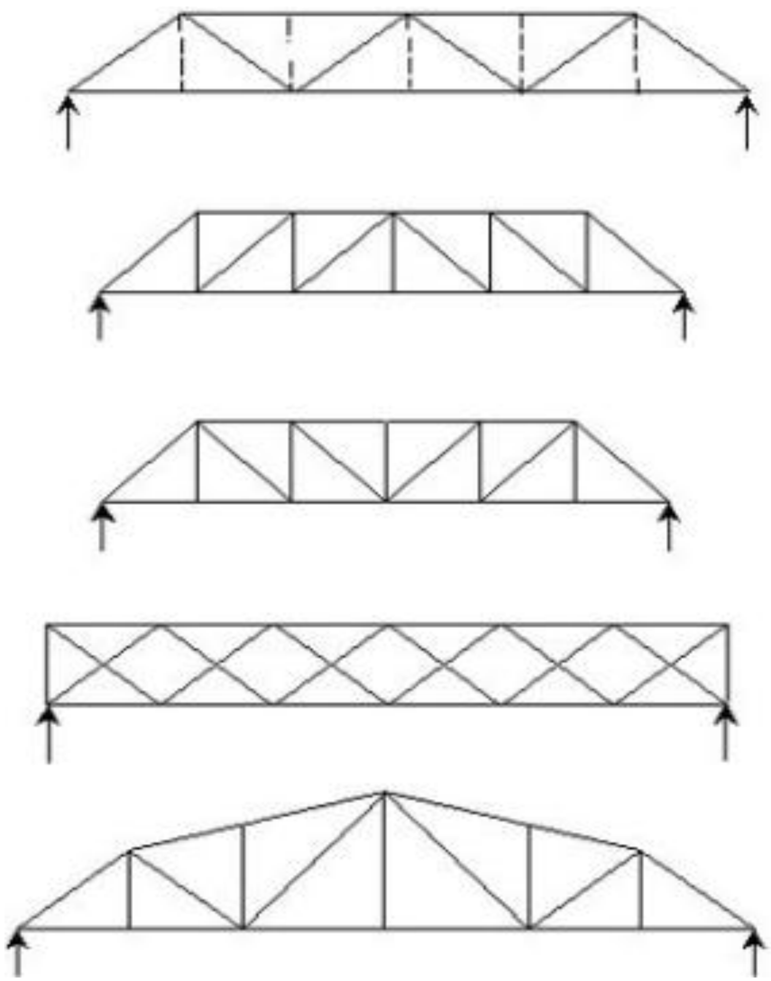
r_{xx} =124 mm

r_{yy} =28.6 mm

UNIT IV

DESIGN OF STEEL TRUSS GIRDER BRIDGES

Truss bridges



(a) Warren truss

(b) Howe truss

(c) Pratt truss

(d) Double Warren truss

(e) Varying depth Warren truss

some of the trusses that are used in steel bridges

- Truss Girders, lattice girders or open web girders are efficient and economical structural systems, since the members experience essentially axial forces and hence the material is fully utilised.

- Members of the truss girder bridges can be classified as chord members and web members.

- Generally, the chord members resist overall bending moment in the form of direct tension and compression and web members carry the shear force in the form of direct tension or compression.

- Due to their efficiency, truss bridges are built over wide range of spans.

- Truss bridges compete against plate girders for shorter spans, against box girders for medium spans and cable-stayed bridges for long spans.

General design principles

Optimum depth of truss girder

- The optimum value for span to depth ratio depends on the magnitude of the live load that has to be carried.

- The span to depth ratio of a truss girder bridge producing the greatest economy of material is that which makes the weight of chord members nearly equal to the weight of web members of truss.

- It will be in the region of 10, being greater for road traffic than for rail traffic.

Design of compression chord members

❖Generally, the effective length for the buckling of compression chord member in the plane of truss is not same as that for buckling out-of-plane of the truss i.e. the member is weak in one plane compared to the other.

❖The ideal compression chord will be one that has a section with radii of gyration such that the slenderness value is same in both planes.

❖ In other words, the member is just likely to buckle in plane or out of plane.

❖ These members should be kept as short as possible and consideration is given to additional bracing, if economical.

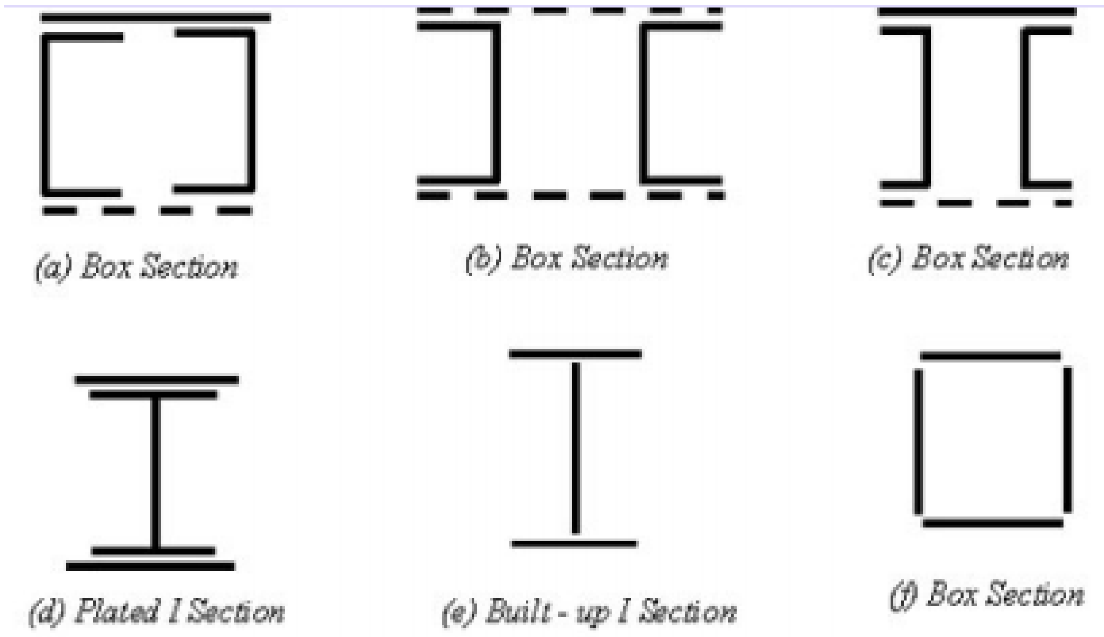
Design of tension chord members

•Tension members should be as compact as possible, but depths have to be large enough to provide adequate space for bolts at the gusset positions and easily attach cross beam.

• The width out-of-plane of the truss should be the same as that of the verticals and diagonals so that simple lapping gussets can be provided without the need for packing.

•It should be possible to achieve a net section about 85% of the gross section by careful arrangement of the bolts in the splices.

• This means that fracture at the net section will not govern for common steel grades.



Typical cross-section for truss members

Design of vertical and diagonal members

✓ Diagonal and vertical members are often rolled sections, particularly for the lightly loaded members, but packing may be required for making up the rolling margins

✓. This fact can make welded members more economical, particularly on the longer trusses where the packing operation might add significantly to the erection cost.

✓Aesthetically, it is desirable to keep all diagonals at the same angle, even if the chords are not parallel.

✓ This arrangement prevents the truss looking overcomplex when viewed from an angle.

✓ In practice, however, this is usually overruled by the economies of the deck structure where a constant panel length is to be preferred.

Lateral bracing for truss bridges

- Lateral bracing in truss bridges is provided for transmitting the longitudinal live loads and lateral loads to the bearings and also to prevent the compression chords from buckling.

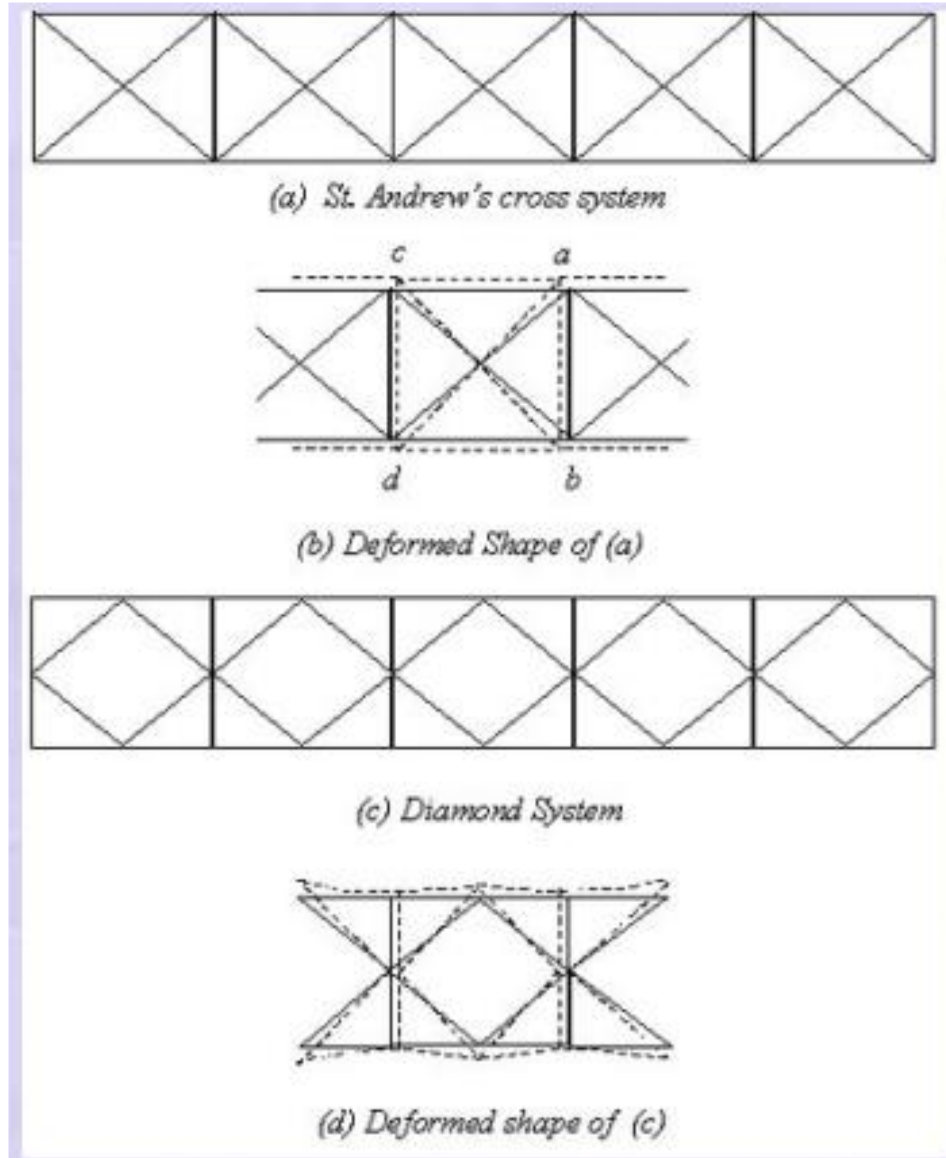
- This is done by providing stringer bracing, braking girders and chord lateral bracing.

- In case of highway truss bridges, concrete deck, if provided, also acts as lateral bracing support system.

▪The nodes of the lateral system coincide with the nodes of the main trusses.

▪Due to interaction between them the lateral system may cause as much as 6% of the total axial load in the chords.

▪ This should be taken into account.

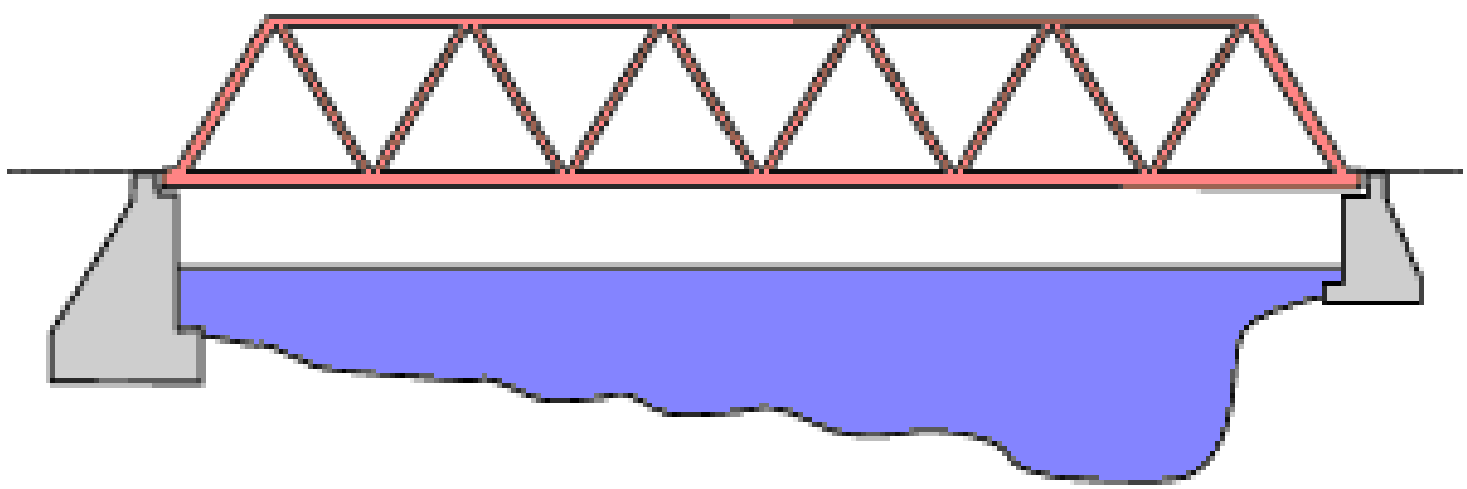


Lateral bracing systems

- It is assumed that wind loading on diagonals and verticals of the trusses is equally shared between top and bottom lateral bracing systems.

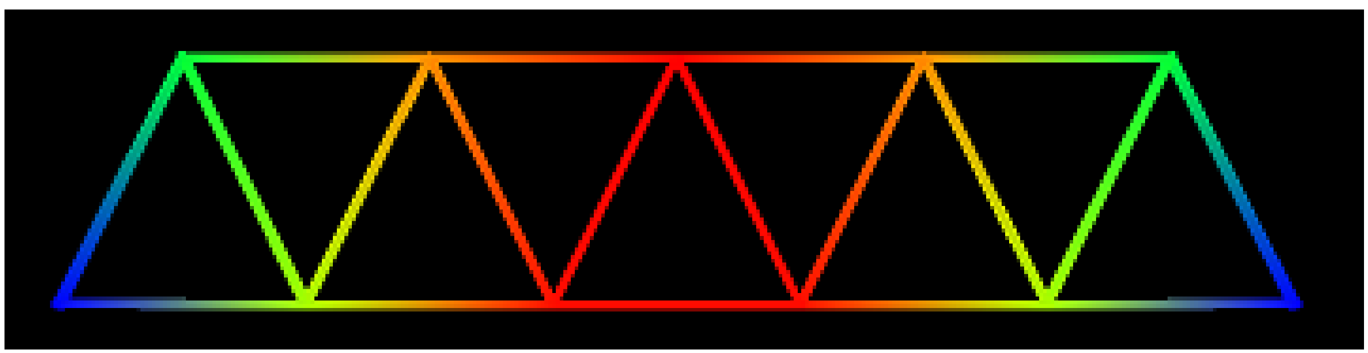
- The end portals (either diagonals or verticals) will carry the load applied to the top chord down to the bottom chord.

- In cases, where only one lateral system exists (as in Semithrough trusses), then the single bracing system must carry the entire wind load.

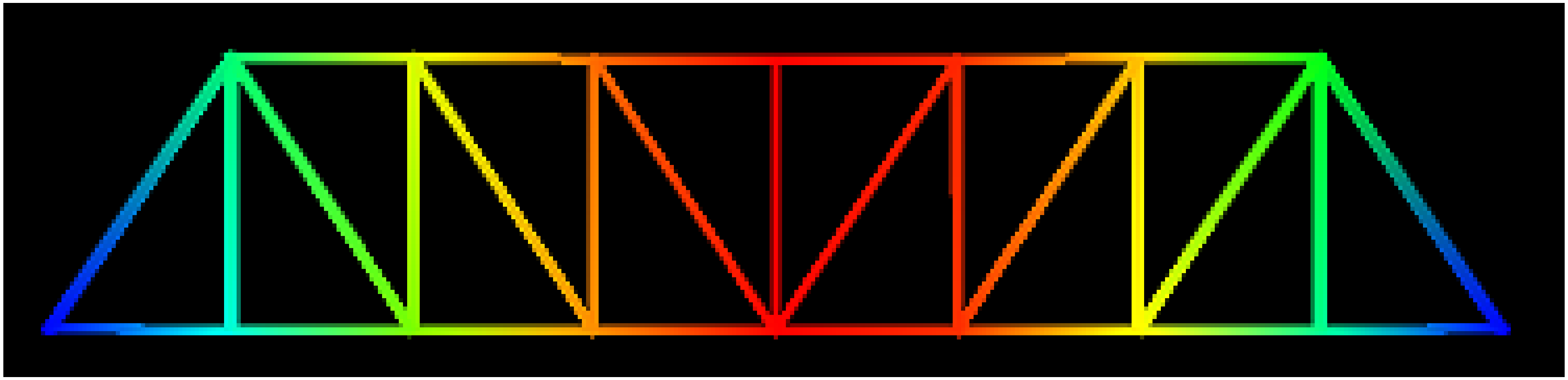


Truss bridges

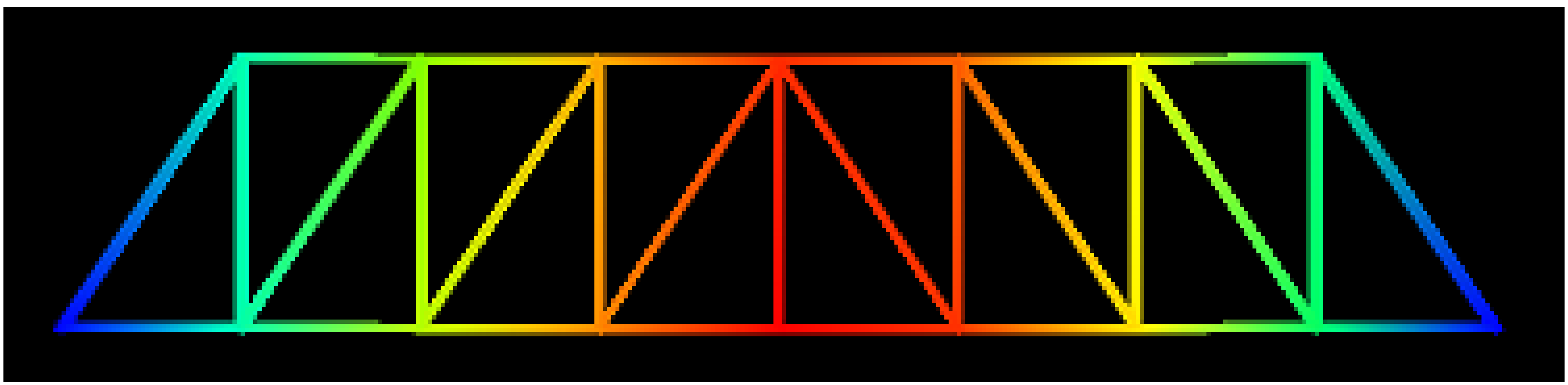
- Warren Truss
  - The Warren truss pattern features a series of isoceles or equilateral triangles. In contrast to the Pratt and Howe patterns, the diagonals alternate in direction.
  - Warren trusses are typically used in spans of between **150-300 feet**
  - The **most common truss.** For smaller spans, no vertical members are used lending the structure a simple look. For longer spans vertical members are added providing extra strength



- Pratt Truss
  - The Pratt truss design contains a downward pointing V in the center with parallel diagonals on each side.
  - Except for those diagonal members near the center, all the diagonal members are subject to tension forces only while the shorter vertical members handle the compressive forces. This allows for thinner diagonal members resulting in a **more economic** design.



- Howe Truss
  - The Howe truss pattern features an upward pointing V formed by the central diagonals with parallel diagonals on either side. Unlike the Pratt pattern the diagonals will be in compression when loaded
  - It is the opposite of the Pratt truss. The diagonal members face in the opposite direction and handle compressive forces. This makes it very **uneconomic** design for steel bridges and is rarely used.

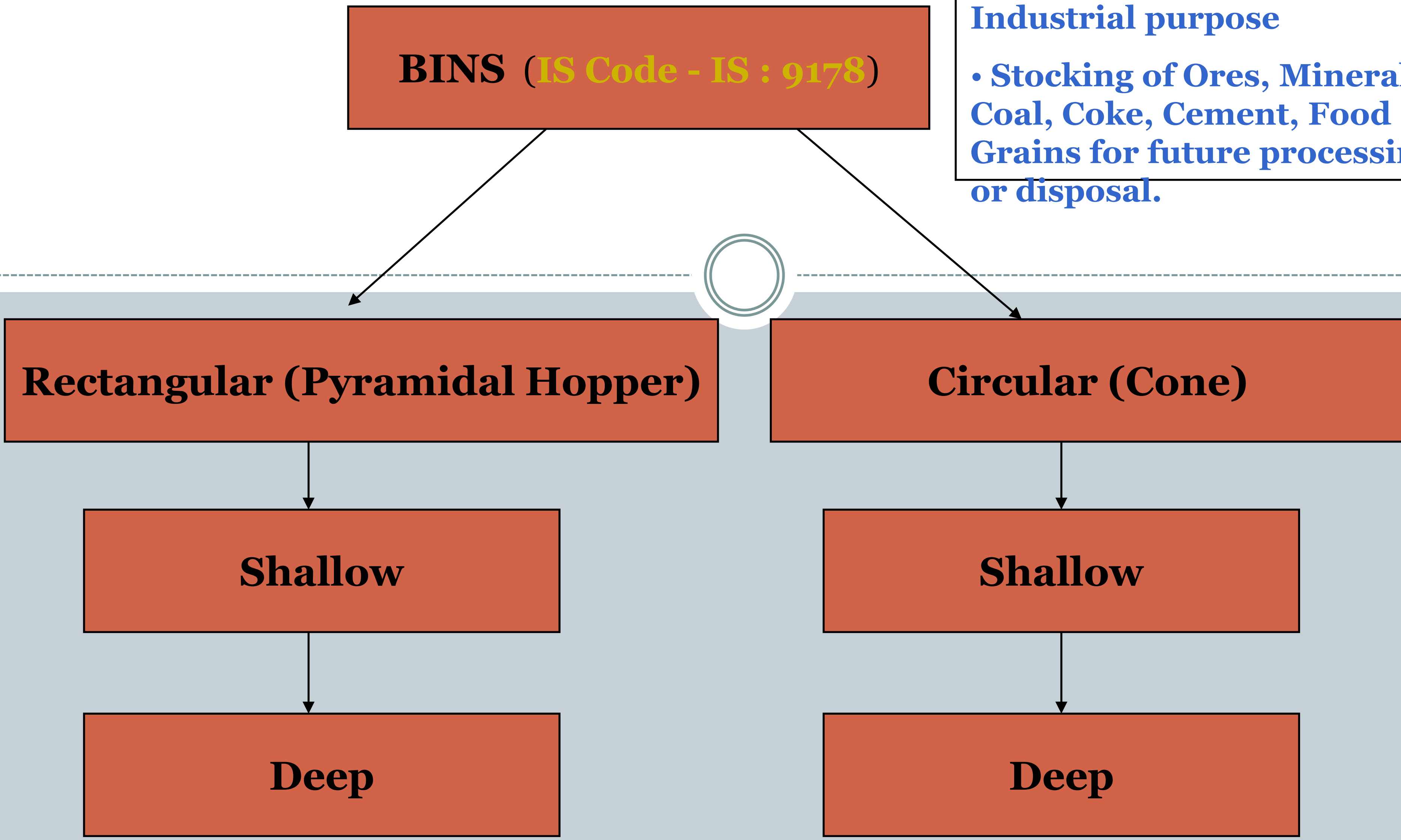


UNIT V

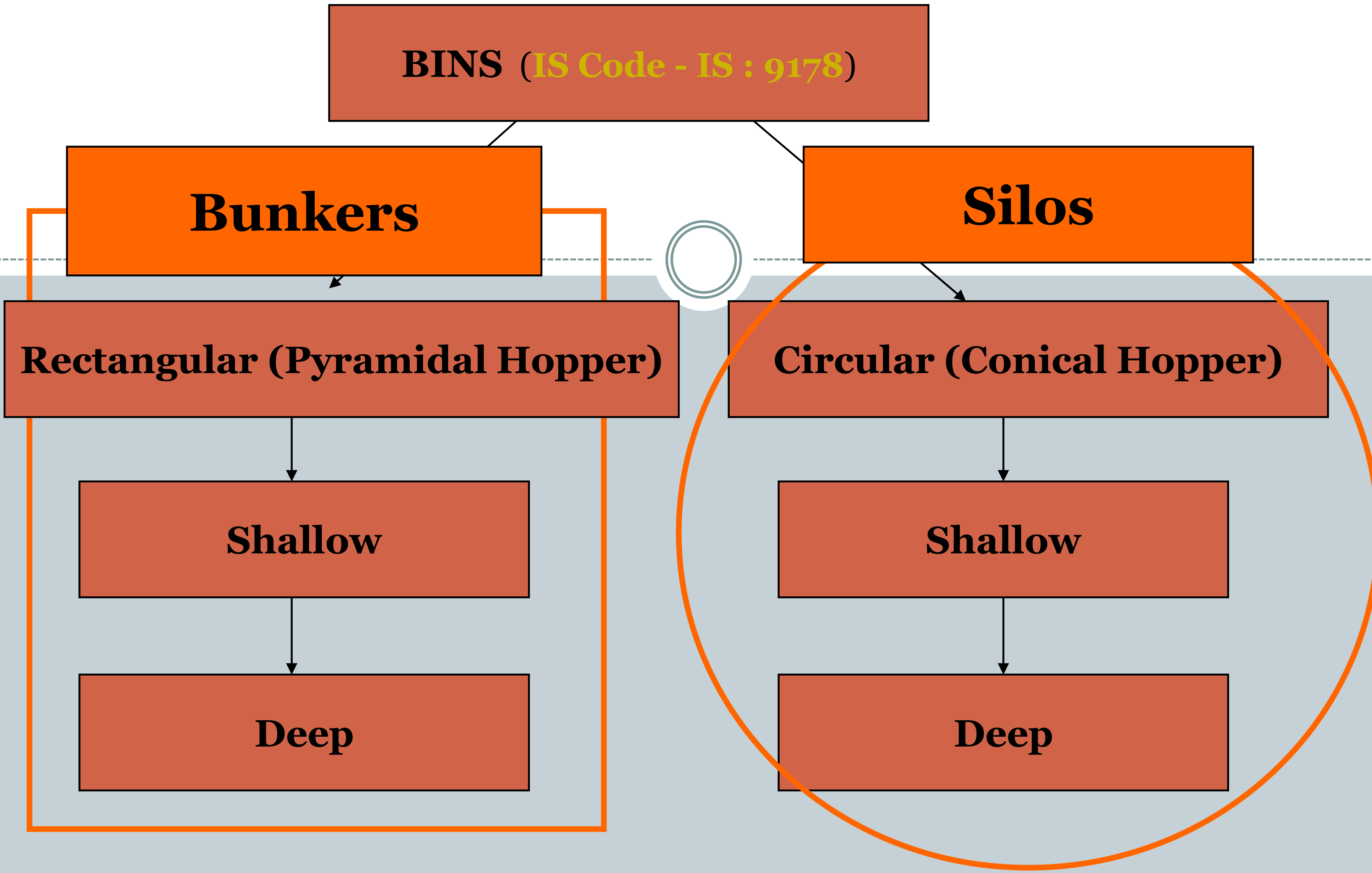
DESIGN OF STEEL BUNKERS & SILOS

Design of Bins - Bunkers and Silos :: Introduction

- **Storage Containers for Industrial purpose**
- **Stocking of Ores, Minerals, Coal, Coke, Cement, Food Grains for future processing or disposal.**



Design of Bins - Bunkers and Silos :: Introduction

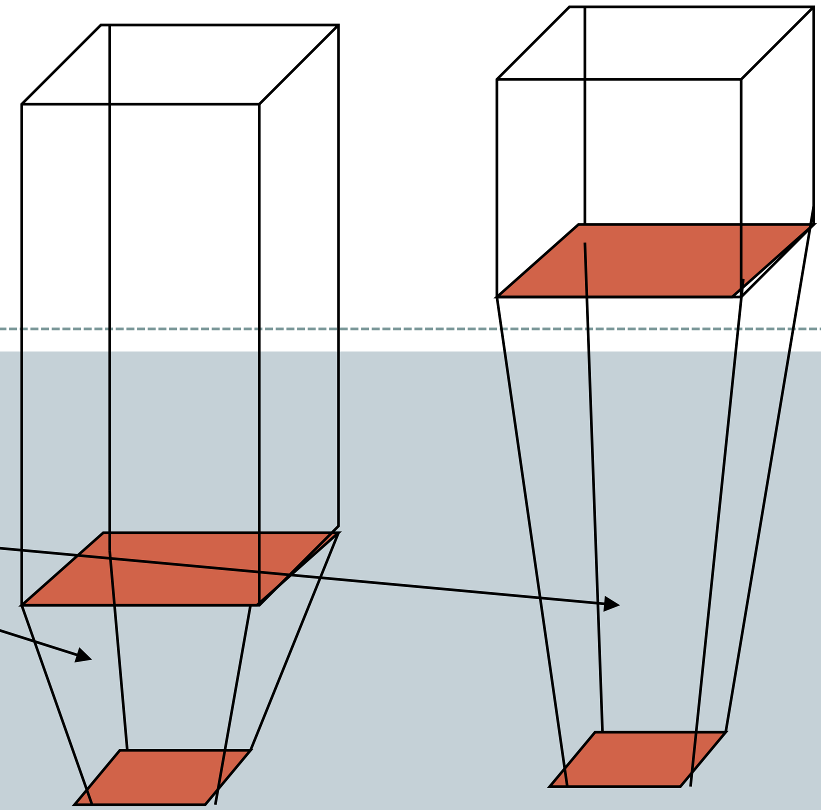


Bunkers

Rectangular (Pyramidal Hopper)

Shallow

Deep



Bunkers

Rectangular (Pyramidal Hopper)

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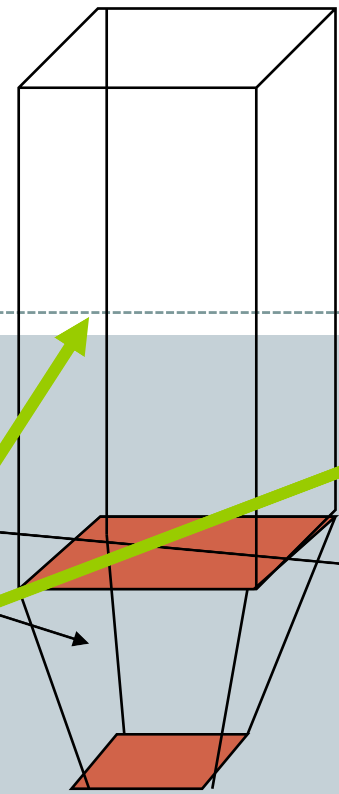
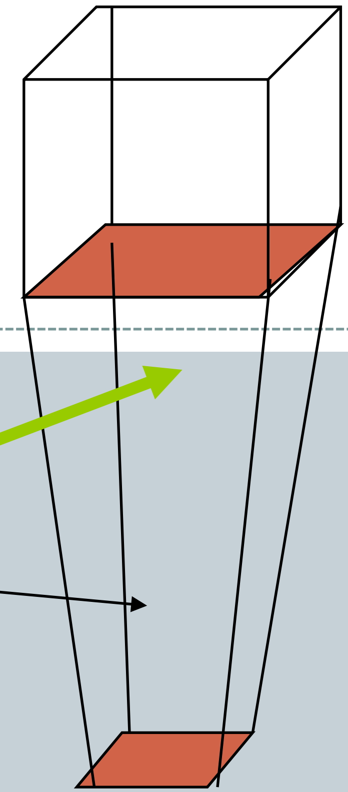
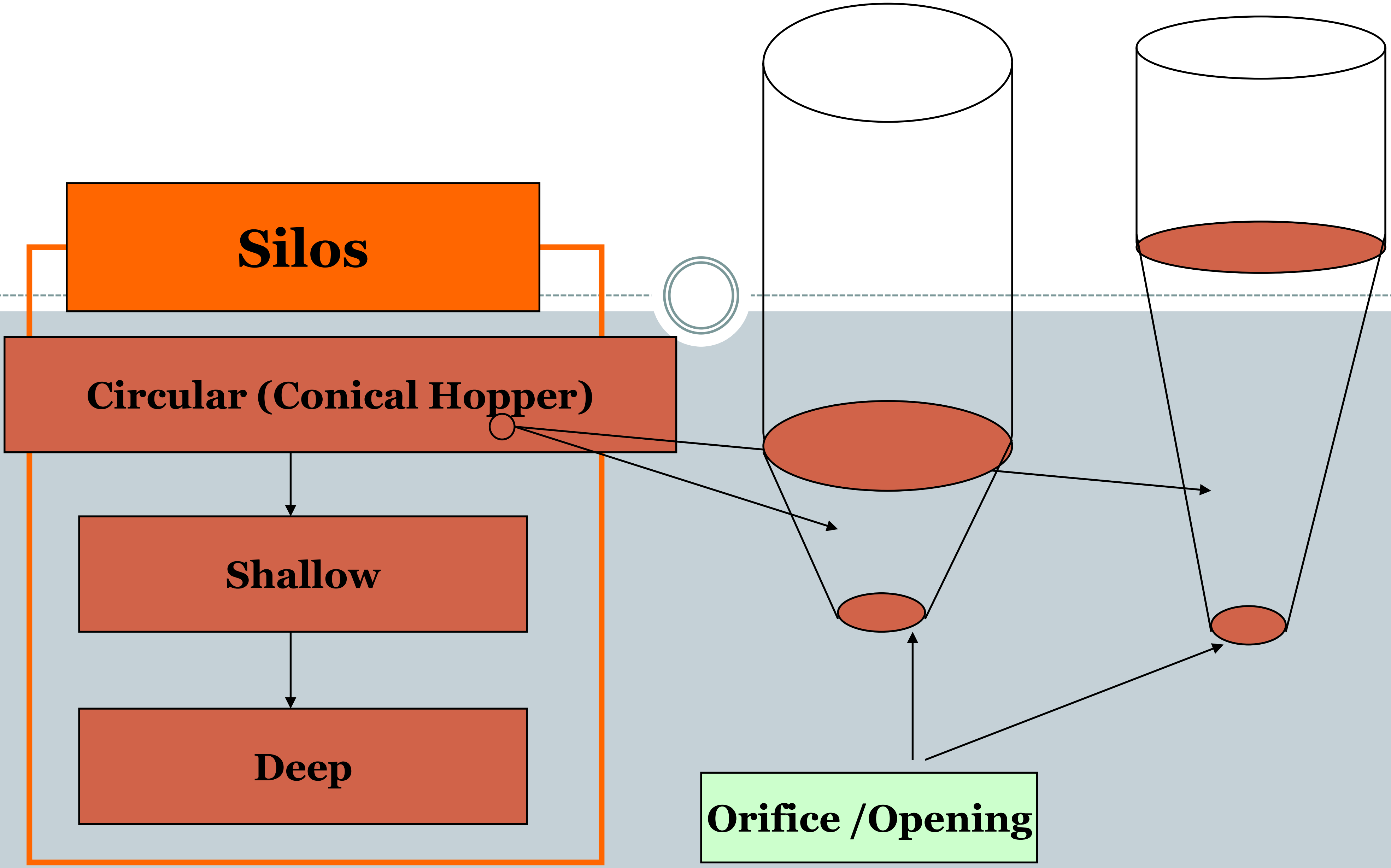





Silos
Circular (Conical Hopper)
Shallow
Deep
Orifice /Opening

1. **Filling the Bunker – Feed and Loading arrangement at the top.**
    - Conveyor / Tripper Conveyor Feed
    - Bucket Elevator Feed
    - Other Mechanical Considerations.
2. **Emptying the bunker ***
    - free flow from Bottom opening or orifice
    - avoid Material Arching, Make use mechanical vibrator.
    - Plan orifice locations to overcome discharge problem.
3. **Stocking of Material**
    - Bunker Hopper + Wall system to be designed strong enough to stock the material for the required duration.
    - Proper application of Design Theories based on geometry of bunker + Nature of material to be stocked + type of filling and emptying.

4. **Provision of Wearing Surface – * Use of liner plates and ease of Maintenance**
   - Rubber Liner, SS/MS Plates, Grating
   - Bricks or Tiles
5. **Minimum Slope of Trough**
   - 50 to 60 degree Wall slope.
   - Consider Corner Angle for Pyramidal Bottom.
6. **Guarding Against Over Loading**
   - Application of Load Cells at support point.
7. **Method of Support**
   - Bunker supporting Beam Arrangement.
   - Bunker Supporting Beam Connections with Portal Frames of Building.
   - Battery of Bunkers – Common Beams, Continuous, Multi Span Beams.

8. Materials of Construction & Method of Construction

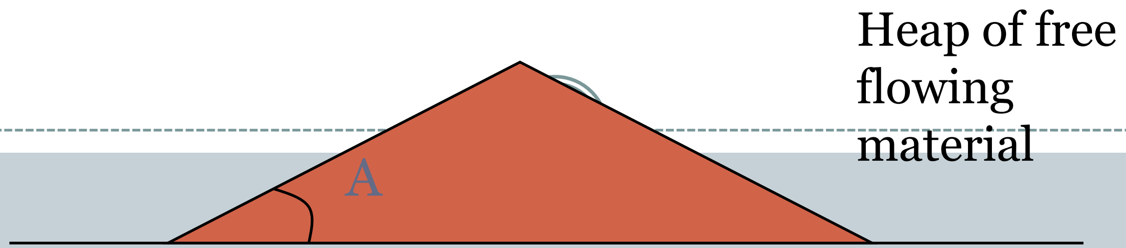
- MS Welded Construction as per IS:800-1984, IS-814,
- MS Plates/Sections IS:2062 Gr-A.
- SAIL-MA or High Yield Stress Wieldable Structural Steel may turn out to be economical for large span bunker beams spanning more than 9 m.

9. Factors of Safety and Working Stresses

- Building Frame Loading as per IS-875 (Dead + Live + Wind), IS-1893 (Eq. Load) and IS:9178 (Material Density and Angle of repose).
- Working stresses as per IS:800-1984.

Design of Bins - Bunkers and Silos :: Theory relating to Granular Mass

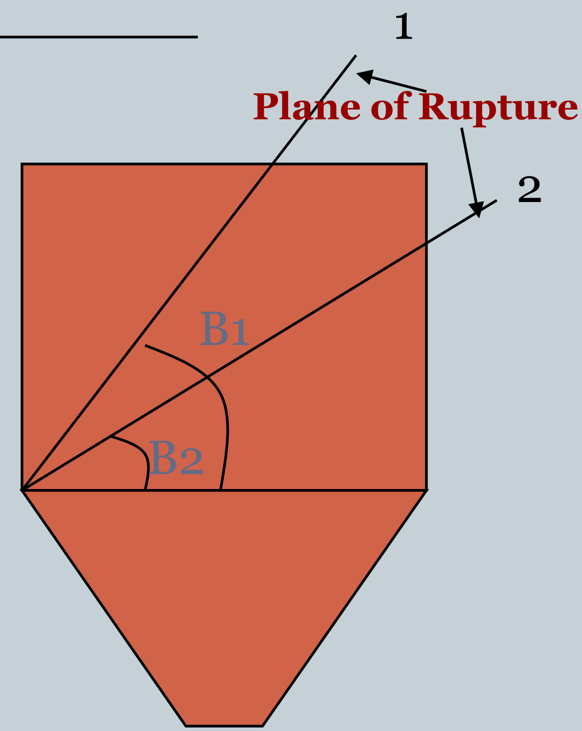
Angle of Repose, A and Angle of Internal friction



Angle of Rupture, B and Plane of Rupture

$B = 45 + A/2$

B and A are in degrees



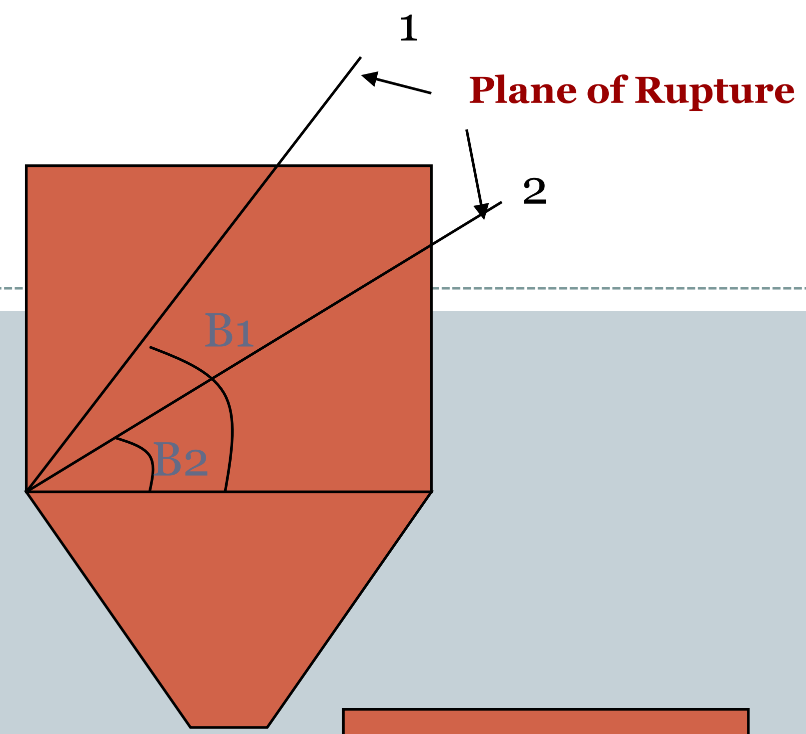
Plane of Rupture

Design of Bins - Bunkers and Silos :: Theory relating to Granular Mass – Pressure Calculation

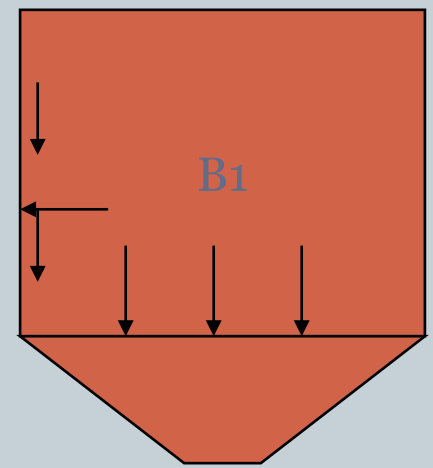
- **Rankines Theory – Case 1**
  - **Incompressible, homogeneous, granular, cohesion less, particle of mass hold together by friction on each other, indefinite extent of mass.**
  - **pv = Y. h**
  - **ph = K.Y.h**
  - **Pn = m .Y. h**

  Where,
  Y = Volumetric or bulk Density.
  K = Rankine's factor = (1- sin ø) / (1+sin ø)

  $m = \cos^2 \alpha + K . \sin^2 \alpha$

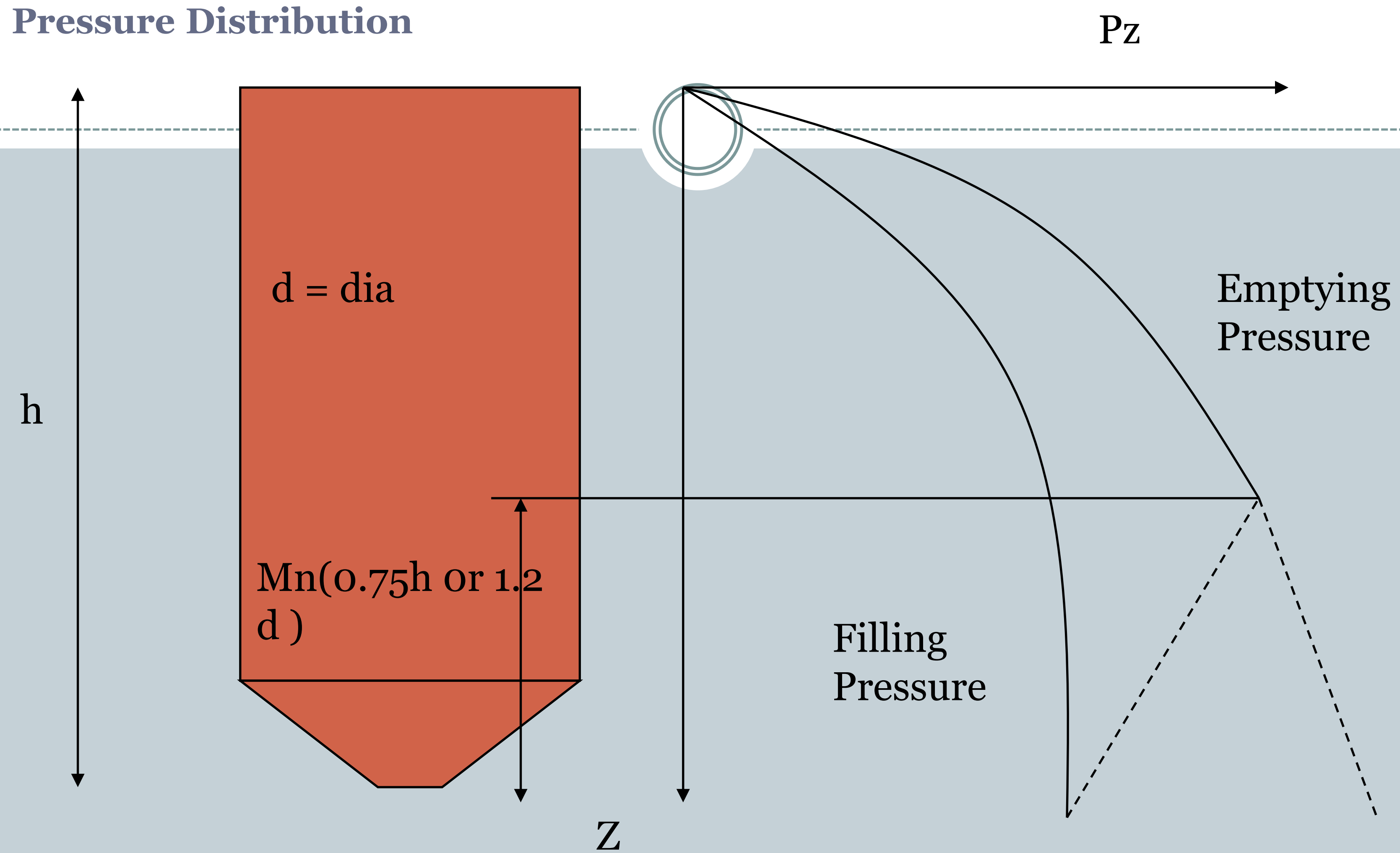


- **Jansen Theory (Recommended by IS:9178) – Case 2**
  - **Friction on the wall predominant and certain quantity of the contents will be carried on the walls due to wall friction.**
  - **Refer IS:1893 part 1 page 14 for formulae.**

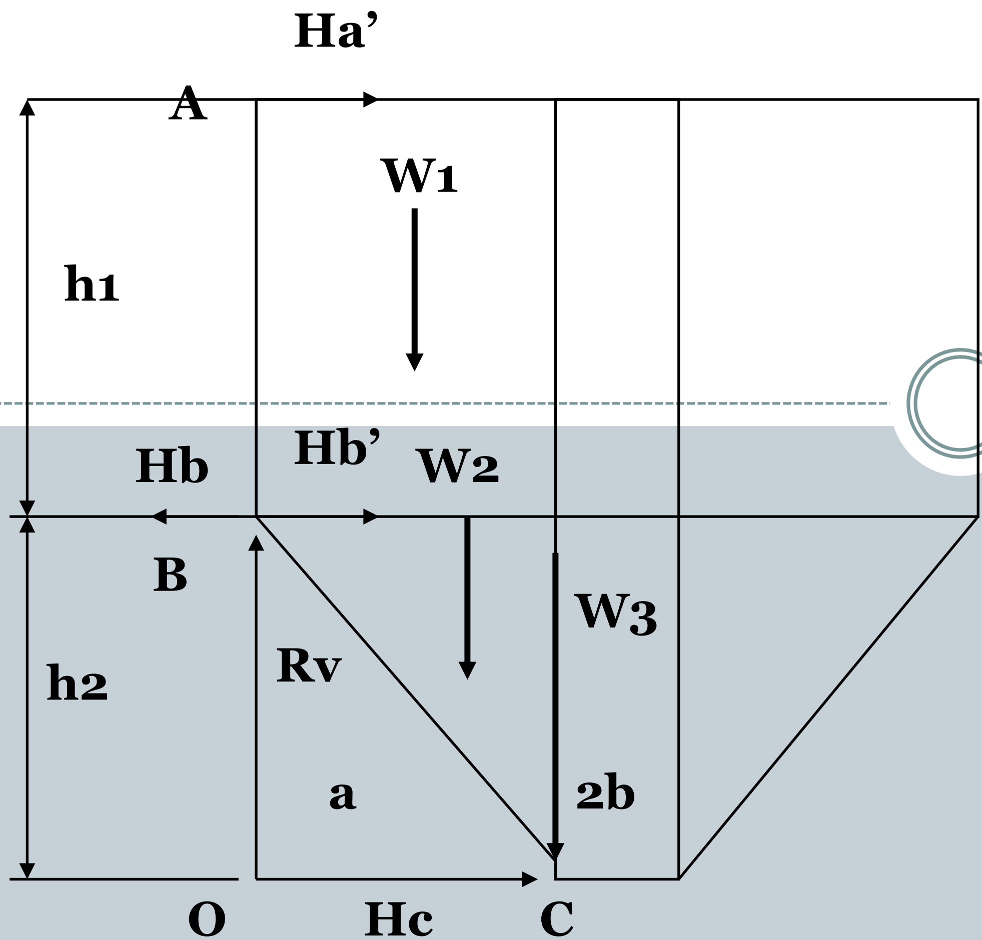


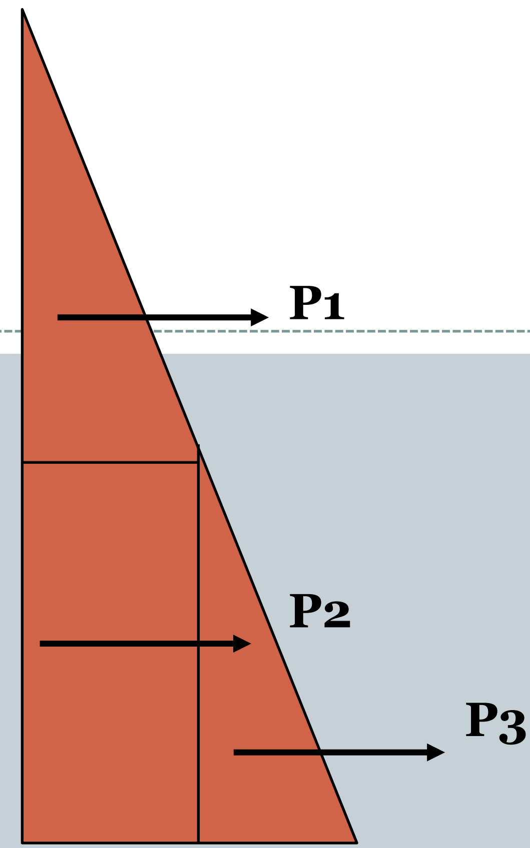
Design of Bins - Bunkers and Silos :: Theory relating to Granular Mass – Pressure Calculation

- Jansen Theory (Recommended by IS:9178) – Case 2
  - Pressure Distribution



Design of Bins - Bunkers and Silos :: Analysis of Bunker Forces : Shallow Rectangular





Refer : IS 9178 (Part II) fig 1. Page 6

Find Hb by taking moment about O and Hc abt B.

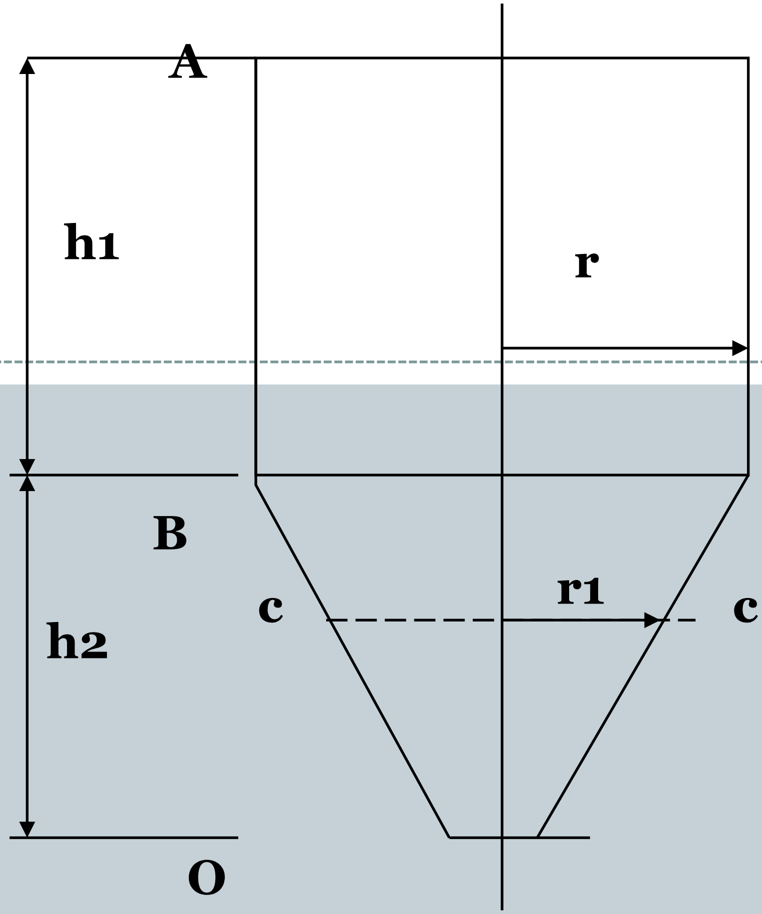
$Ha' = P1/3 : Hb' = 2P1/3$

$W1 = Y.a.h1 : P1 = K.Y.h1.h1/2$

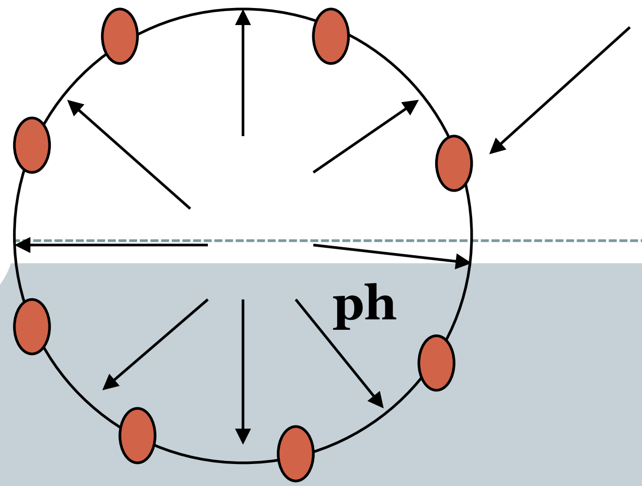
$W2 = Y.a.h2/2 : P2 = K.Y.h1.h2$

$W3 = Y.b.(h1+h2) : P3 = K.Y.h2.h2/2$

Design of Bins - Bunkers and Silos :: Analysis of Bunker Forces : Shallow Circular



Refer : IS 9178 (Part II) fig 1. Page 14



Hoop tension Wall = Ph. r

Hoop tension Hopper = Pn. R1. cosec α

TL Longitudinal tension = Wt / (2. pi . r1. cosec α)

Wt = Total weight at c-c

Design of Bins - Bunkers and Silos :: Design Methodology

- Bunker Layout – JVSL Stock House
- Bunker Structural Components
  - Bunker Beam Wall Component
    - **Wall or Web plate**
    - **Vertical Stiffeners**
    - **Horizontal Stiffeners**
    - **Bottom Flange or Trough Beams**
    - **Top Flange + Floor Beams**
  - Hopper Component
    - **Skin Plate or Sheathing Plate**
    - **Ribs Beams/Angles**