SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – I – BUILDING STRUCTURES II– SCIA1306

UNIT I

DESIGN OF BRICK MASONRY

[IS1905-1987] – Reaffirmed 1998

Brick

(Comp. strength → 3.5 to 40 N/mm^2)

Clay — Calcium Silicate (Cement, FAL-G)

Used for,

1. External and internal bearing walls
2. Load bearing piers and columns
3. Paritition walls
4. Brick masonry foundations
5. Floorings and Pavings

Advantages of brick masonry:

Attractive appearance, economical light weight, durable, strength, fire resistance, sound insulation, low thermal conductivity, minimum maintenance.

Classification of bricks: [Based on shapes]

1. Solid bricks – Perforations or holes not greater than 25% of volume
2. Perforated bricks – Perforation is greater than 25% of volume. Advantages:of perforated bricks are high thermal insulation and light weight. Water absorption should not be greater than 15% after 24 hours of insertion and compressive strength not less than 7N/mm^2.
3. Hollow blocks – Holes greater than 20% and sizes of holes greater than 20mm.
4. Cellular bricks – Holes greater than 20% and closed at one end
5. Ornamental bricks – Bricks used in corbels, cornices, etc.

Size of bricks: [As per IS1077]

Standard size – 19 x 9 x 9 cm

Modular brick – 20 x 10 x 10 cm

The average compressive strength of brick unit as per IS3495 (Part I) – 1976 is, 3.5 - 40N/mm^2.

Tests on bricks:

1. Water absorption: Brick units immersed in water for 24 hours has,

   i) upto 12.5 N/mm^2 strength and water absorption should not be greater than 20%

   ii) for higher classes, water absorption should not be greater than 15%

2. Efflorescence: Leaching of water soluble salts (white coloured) under efflorescence.

   Test for efflorescence is done as per IS3495 (Part III) – 1976. The brick is kept in a dish with water height as 25mm and the time for water absorption and evaporation is noted. This value is compared with the same dish with 25mm water height kept for evaporation alone. Based on the code, efflorescence in brick is reported as nil, slight, moderate, heavy and serious.

3. Hardness: For the brick to be hard, it should create no impression by finger nail.
4. Soundness: When two bricks are struck, it should not break and produce a clear ringing sound.
5. Compressive test: 3.5 – 40 N/mm^2
6. Flexure test: Rarely done

Classification of brick based on structure and usage:

1. Solid wall
2. Cavity wall
3. Faced wall
4. Veneered wall

Based on loading, walls are classified as,

1. Axially loaded walls [Load applied at centre t/2]
2. Eccentrically loaded walls
3. Laterally loaded walls [Loading applied at sides]

Design procedure:

Slenderness (Least of l_e/t & h_e/t) : ($\lambda_{max} = 27$)

($\lambda = 60$ for RC columns, $\lambda = 45$ & 30 for braced and unbraced RC walls)

1. Actual stress on the brick masonry wall is found based on the load from slab and self weight of wall.
2. The permissible compressive stress for masonry based on the type of mortar and compressive strength of brick unit is taken from Table 8, IS1905-1987. This table is valid for slenderness ratio $\lambda \leq 6$ and eccentricity $e = 0$.

3. Corrections are applied for slenderness ratio, eccentricity (if any), shape and size of brick unit. Shape modification factor and cross sectional area of masonry (area reduction factor).

4. Slenderness ratio is found as the least of l_e/t or h_e/t, where, (l_e = Effective length and h_e = effective height).

Effective length is found from Table 5, IS1905 – 1987 and effective height is found from Table 4, IS1905-1987.

Table 4 – Effective height:

Support condition	Effective height
Fixed – Fixed	0.75H
Fixed – Hinged	0.85H
Hinged – Hinged	H
Fixed – Free	1.5H

The permissible value of λ is 27 (λ_{max}) for cement mortar (OPC & PPC), given in Table 7, IS1905-1987.

5. Eccentricity of loading is determined (for axial loading e = 0). Eccentricities for various other cases are to be checked as per Appendix B of IS1905-1987.

6. For the permissible stress adopted, shape modification factor is found based on height to width ratio of each brick unit given in Table 10, IS1905-1987.

7. Area reduction factor is applied for elements having cross section less than $0.2m^2$. The area modification factor, k = 0.7 + 1.5 A

8. After applying modification factors, the actual stress is verified with a modified permissible stress, $\sigma_{act} < \sigma_{per}$

The permissible stress (strength of the wall) depends upon the following factors: i) Compressive strength of masonry unit

ii) Compressive strength of mortar used iii) Slenderness ratio of the wall

iv) Eccentricity in loading

v) Shape and size of brick unit

vi) Cross sectional area of masonry

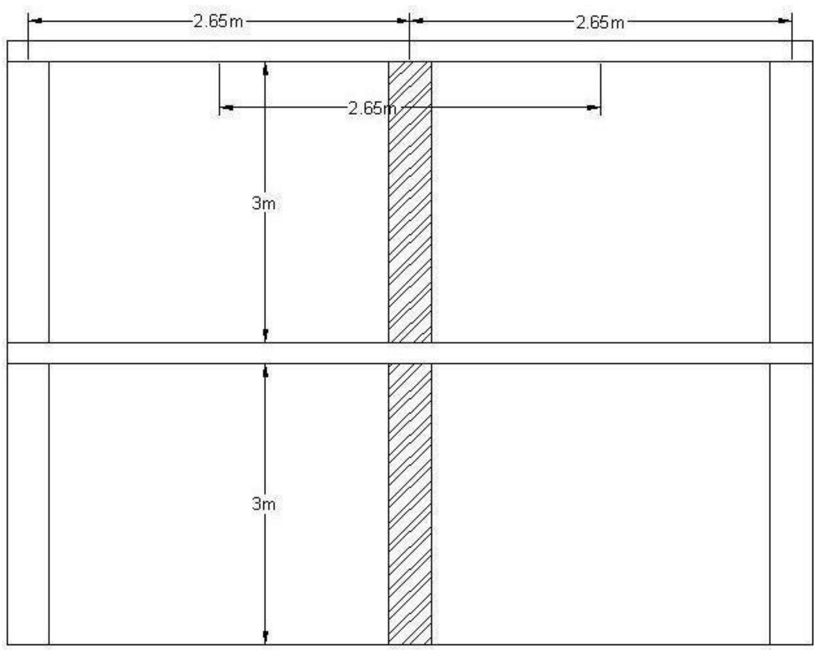
1. Design an interior cross wall for a two storeyed building to carry 100mm thick RC slab with 3m storey height. The wall is unstiffened and supports 2.65m wide slab. Loading on the slab is given as below:

i) Live load on floor slab = 2 kN/m^2

ii) Live load on roof slab = 1.5 kN/m^2

iii) Floor finish = 0.2 kN/m^2

iv) Roof finish = 1.96 kN/m^2



Assume the compressive strength of brick as 10N/mm^2 and mortar type as M1.

The loading on the wall includes the load from slab (LL + DL) and self weight of the wall. Assuming the wall thickness as 100mm and size of each masonry unit as 200 x 100 x 90mm,

Loading on

slab: Live load:

on floor slab = 2 kN/m^2

on roof slab = 1.5 kN/m^2

Dead load:

Floor finish = 0.2 kN/m^2

Roof finish = 1.96 kN/m^2

Self weight of slabs = 2 x 0.1 x 25 = 5

kN/m^2 Load from slab = 10.66 kN/m^2

For 2.65m length of slab,

Load from slab = 10.66 x 2.658 = 28.36 kN/m

Self weight of masonry = 2 x 0.1 x 20 x 3 = 12 kN/m

Total = 40.36 kN/m

Permissible stress of masonry for M1 mortar and masonry unit of compressive strength 10N/mm^2 is taken from Table 8, IS 1905 – 1987.

Permissible stress = 0.96 N/mm^2

Stress reduction factor, Area reduction factor, Shape modification factor are applied as per Cl.5.4.

Stress reduction factor (K_{st})

Slenderness ratio (Least of l_e/t & h_e/t)

From Table 4,

h_e = 0.75 H = 0.75 x 3 = 2.25m [Both ends fixed]

h_e/t = 2.25 / 0.1 = 22.5 < 27

Therefore, the stress reduction factor from Table 10 for λ = 22.5 and no eccentricity condition is,

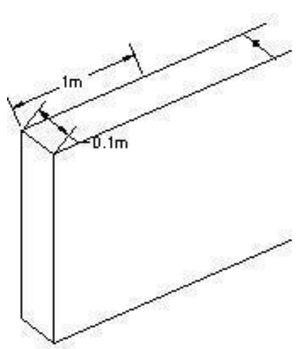
For 22 → 0.56 (e = 0)

For 24 → 0.51

For 22.5 → 0.55

K_{st} = 0.55

Area reduction factor (K_A) [Cl.5.4.1.2, IS1905-1987]



A = 0.1 x 1 = 0.1m^2 < 0.2 m^2

K_A = 0.7 + (1.5 x 0.1) = 0.85

Shape modification factor (K_{sh}) [Cl. 5.4.1.3, IS1905-1987]

Ksh for block of size 200 x 100 x 90 mm laid along 100mm side, from Table 10 for Height to Width ratio of 90 x 100,

Height/ width = 90/100 = 0.9

For $0.75x_o$ → 1

For $1x_o$ → 1.1

For $0.9x_o$ → 1.06

<u>K_{sh} = 1.06</u>

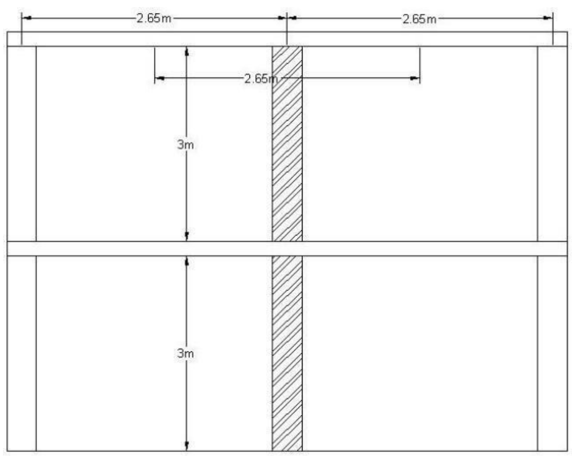
$\sigma_{per\ modified}$ = $K_{st}.K_A.K_{sh}.\ \sigma_{per}$

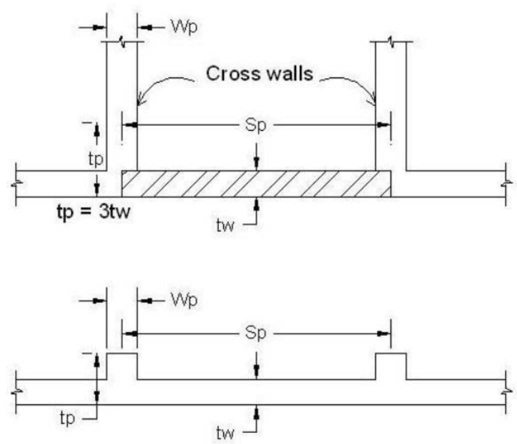
= 0.55 x 0.85 x 1.06 x 0.96 = 0.48 N/mm^2

$\sigma_{act\ /m} = \frac{40.36x10_3}{100x1000}$ = 0.4036 N/mm^2 < σ_{per} [0.48 N/mm^2]

Hence the adopted thickness of 100mm with M1 mortar and masonry unit with compressive strength 10N/mm^2 is safe in carrying the load from slab.

2. In the above problem, design the wall if it is continuous and stiffened by cross wall of 100mm thickness and length of the wall being 3.6m.





Here, S_p = 3.6m

Loading on the masonry wall = 40.36 kN/m

Actual stress = $\frac{40.36x1000}{100x1000}$ = 0.4036 N/mm^2

σ_{per} for M1 mortar and masonry unit of compressive strength 10 N/mm^2 with 100mm thickness,

Permissible stress = 0.96 N/mm^2 [From Table 8, IS1905 – 1987]

Slenderness ratio, λ → Least of H_e/t & L_e/t

H_e = 0.75 H = 0.75 x 3 = 2.25m

L_e = 0.8L = 0.8 x 3.7 = 2.96m [From Table 5, IS1905 – 1987]

For the cross walls provided, stiffening coefficients are found from Table 6, IS1905 – 1987. t_p → Thickness of pier

$t_p = 3t_w$ [for cross walls] cl.4.6.3, IS1905-1987

$S_p = 3.7m$, [$S_p \rightarrow$ c/c spacing of pier]

$t_w = 0.1m$ [$t_p \rightarrow$ thickness of pier]

$t_p = 3t_w = 0.3m$ [$t_w \rightarrow$ thickness of wall]

$w_p = 0.1m$ [$w_p \rightarrow$ width of pier] cl.4.5.3, IS1905-1987

$$\frac{t_p}{t_w} = \frac{0.3}{0.1} = 3, \qquad \frac{S_p}{w_p} = \frac{3.7}{0.1} = 37$$

From Table 6, for $\frac{S_p}{W_p} = 37$, $\frac{t_p}{t_w} = 3$

$S_e = 1$

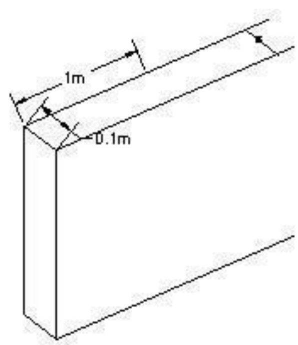
Thickness of wall = 1 x 0.1 =

0.1m [Considering stiffness]

$\lambda = 2.25 / 0.1 = 22.5 < 27$

The stress reduction factor (K_{st}) for Table no.10 for $\lambda = 22.5$ with no eccentricity (e=0) condition,

For $\lambda = 22$	0.56
For $\lambda = 24$	0.51
For $\lambda = 22.5$	0.55



$K_{st} = 0.55$

Area reduction factor for area = 0.1 x 1 = 0.1m2 < 0.2m2,

KA = 0.7 + (1.5 x 0.1) = 0.85

Shape modification factor: [Cl.5.4.1.3]

Ksh for block size of 200 x 100 x 90mm laid along 100mm side from Table 10 for height to width ratio of 90 x 100mm,

$$\frac{Height}{Width} = \frac{90}{100} = 0.9$$

For $H_t/W = 0.75$	1
For $H_t/W = 1$	1.1
For $H_t/W = 0.9$	1.06

$K_{sh} = 1.06$

$\sigma_{per\ modified} = K_{st}.K_A.K_{sh}.\ \sigma_{per}$

$= 0.55 \times 0.85 \times 1.06 \times 0.96 = 0.48\ N/mm^2 > \sigma_{act}$

Provided masonry wall of thickness <u>100mm</u> with <u>M1</u> mortar and compressive strength of each unit <u>10 N/mm^2</u> is safe.

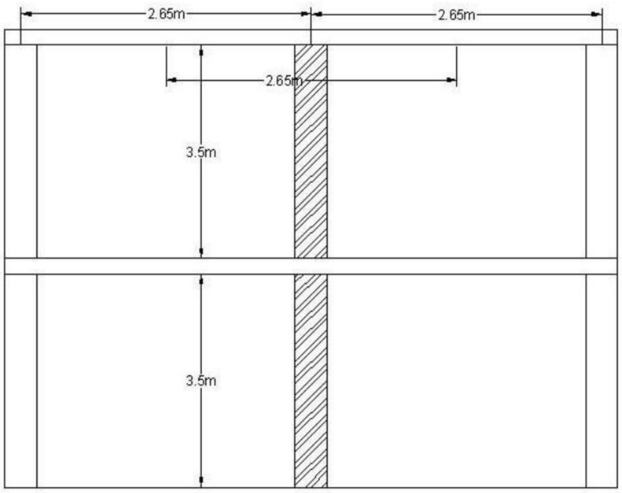
3. Design an interior cross wall for a two storeyed building to carry 100mm thick RC slab. Check the safety of the wall if the wall is continuous and cross wall is available on only one side and the storey height is 3.5m. The wall supports 2.65m wide slabs on both sides. Loading on the slab is given as below:

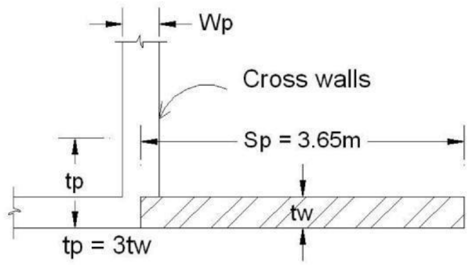
i) Live load on floor slab = 2 kN/m^2

ii) Live load on roof slab = 1.5 kN/m^2

iii) Floor finish = 0.2 kN/m^2

iv) Roof finish = 1.96 kN/m^2





Assume the compressive strength of brick as 10N/mm^2 and mortar type as M1. Loading on slab:

Live load:

on floor slab = 2 kN/m^2

on roof slab = 1.5 kN/m^2

Dead load:

Floor finish = 0.2 kN/m^2

Roof finish = 1.96 kN/m^2

Self weight of slabs = 2 x 0.1 x 25 = 5 kN/m^2 Load from slab = 10.66 kN/m^2

For 2.65m length of slab, Load from slab = 10.66 x 2.658 = 28.36 kN/m

Self weight of masonry = 2 x 0.1 x 20 x 3.5 = 14 kN/m

Total = 42.36 kN/m

$\sigma act/m = \frac{42.36x103}{100x1000} =$ 0.4236 N/mm^2

Permissible stress of masonry for M1 mortar and masonry unit of compressive strength 10N/mm^2 is taken from Table 8, IS 1905 – 1987.

Permissible stress = 0.96 N/mm^2

Stress reduction factor, Area reduction factor, Shape modification factor are applied as per Cl.5.4.

<u>Stress reduction factor (K_{st})</u>

Slenderness ratio (Least of l_e/t & h_e/t)

From Table 4,

Effective height, h_e = 0.75 H = 0.75 x 3.5 = 2.625m [Both ends fixed]

Effective length, l_e = 1.5 L = 1.5 x 3.65 = 5.475m [One end fixed, other end free]

h_e/t = 2.625 / 0.1 = 26.25 < 27

Therefore, the stress reduction factor from Table 10 for λ = 26.25 and no eccentricity condition is,

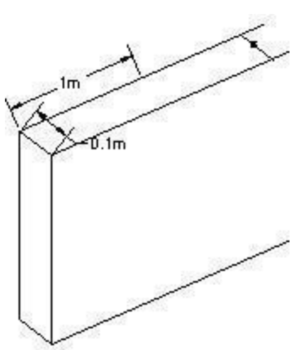
For 26 → 0.45 (e = 0)

For 27 → 0.43

For 26.25 → 0.3375 + 0.1075 = 0.445

<u>K_{st} = 0.445</u>

<u>Area reduction factor (K_A)</u> [Cl.5.4.1.2, IS1905-1987]



A = 0.1 x 1 = 0.1m^2 < 0.2 m^2

<u>K_A</u> = 0.7 + (1.5 x 0.1) = <u>0.85</u>

<u>Shape modification factor (K_{sh})</u> [Cl. 5.4.1.3, IS1905-1987]

K_{sh} for block of size 200 x 100 x 90 mm laid along 100mm side, from Table 10 for Height to Width ratio of 90 x 100,

$$\frac{Height}{Width} = \frac{90}{100} = 0.9$$

For 0.75x_o → 1

For 1x_o → 1.1

For 0.9x_o → 1.06

$\underline{K_{sh} = 1.06}$

$\sigma_{per\ modified} = K_{st}.K_A.K_{sh}.\ \sigma_{per}$

= 0.445 x 0.85 x 1.06 x 0.96 = 0.385

$N/mm^2 < \sigma_{act\ /m}$ [0.4236 N/mm^2]

Hence the adopted thickness of 100mm with M1 mortar and masonry unit with compressive strength $10N/mm^2$ is **not** safe in carrying the load from slab. The thickness of wall is increased to 200mm.

Load from slab = 10.66 kN/m^2

For 2.65m length of slab,

Load from slab = 10.66 x 2.658 = 28.36 kN/m

Self weight of masonry = 2 x 0.2 x 20 x 3.5 = 28 kN/m

Total = <u>56.36 kN/m</u>

Loading on masonry wall = 56.36 kN/m

Actual stress $\sigma_{act} = \frac{56.36x1000}{200x1000} = 0.2818\ N/mm^2$

Permissible stress of masonry for M1 mortar and masonry unit of compressive strength $10N/mm^2$ is taken from Table 8, IS 1905 – 1987.

Permissible stress $\sigma_{per} = 0.96\ N/mm^2$

Stress reduction factor, Area reduction factor, Shape modification factor are applied as per Cl.5.4.

<u>Stress reduction factor (K_{st})</u>

Slenderness ratio (Least of l_e/t & h_e/t)

From Table 4,

Effective height,

Effective length,

h_e = 0.75 H = 0.75 x 3.5 = 2.625m [Both ends fixed]

le = 1.5 L = 1.5 x 3.65 = 5.475m [One end fixed, other end free] h_e/t = 2.625 / 0.2 = 13.125 < 27

Therefore, the stress reduction factor from Table 10 for λ = 22.5 and no eccentricity condition is,

For 12 → 0.84 (e = 0) For
14 → 0.78

For 13.125 → 0.3675 + 0.439 = 0.806
K_{st} = 0.806

Area reduction factor (K_A) [Cl.5.4.1.2, IS1905-1987]

$A = 0.2 \times 1 = 0.2 m^2$

$K_A = 1$

Shape modification factor (K_{sh}) [Cl. 5.4.1.3, IS1905-1987]

K_{sh} for block of size 200 x 100 x 90 mm laid along 100mm side, from Table 10 for Height to Width ratio of 90 x 100,

$$\frac{Height}{Width} = \frac{90}{100} = 0.9$$

For $0.75x_o \rightarrow 1$

For $1x_o \rightarrow 1.1$

For $0.9x_o \rightarrow 1.06$

$K_{sh} = 1.06$

$\sigma_{per\ modified} = K_{st}.K_A.K_{sh}.\ \sigma_{per}$

$= 0.806 \times 1 \times 1.06 \times 0.96 = 0.82$

$N/mm^2 > \sigma_{act\ /m}$ [$0.2818\ N/mm^2$]

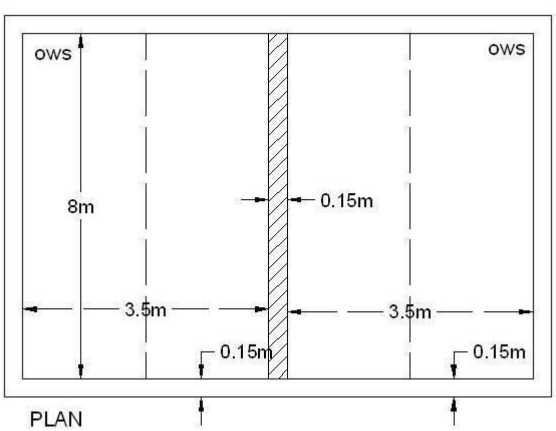
Hence the adopted thickness of 200mm with M1 mortar and masonry unit with compressive strength $10N/mm^2$ is safe in carrying the load from slab.

4. Design the interior wall of a single storey building shown in figure. The height of the ceiling is 3.5m and the load from slab including self weight is $30kN/m^2$.

Load from slab = 30 x 3.65 = 109.5 kN/m

Self weight of wall = 0.15 x 3.5 x 1 x 20 = 10.5 kN/m

Total = 120 kN/m



Actual stress = $\frac{120x1000}{150x1000}$ = 0.8 N/mm^2

Permissible stress of masonry for M1 mortar and masonry unit of compressive strength 10N/mm^2 is taken from Table 8, IS 1905 – 1987.

Permissible stress σ_{per} = 0.96 N/mm^2

Stress reduction factor, Area reduction factor, Shape modification factor are applied as per Cl.5.4.

Stress reduction factor (K_{st})

Slenderness ratio (Least of l_e/t & h_e/t)

From Table 4,

Effective height, h_e = 0.75 H = 0.75 x 3.5 = 2.625m [Both ends fixed]

Effective length, le = 1 L = 1.0 x 8.15 = 8.15m

h_e/t = 2.625 / 0.2 = 13.125 < 27

For the cross walls provided, stiffening coefficients are found from Table 6, IS1905 – 1987.

t_p = 3tw [for cross walls] cl.4.6.3, IS1905-1987

S_p = 8.15m, [S_p → c/c spacing of pier/wall]

t_w = 0.15m [t_p → thickness of pier/wall]

t_p = 3tw = 0.45m [t_w → thickness of wall]

w_p = 0.15m [w_p → width of pier/wall] cl.4.5.3, IS1905-1987

$\frac{t_p}{t_w} = \frac{0.45}{0.15}$ = 3, $\frac{S_p}{w_p} = \frac{8.15}{0.15}$ = 54.3

From Table 6, for $\frac{S_p}{w_p}$ = 54.3, $\frac{t_p}{t_w}$ = 3

S_e = 1

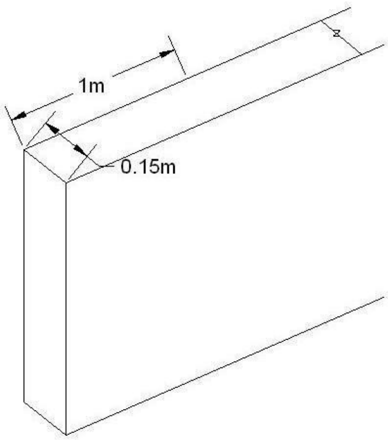
Thickness of wall = 1 x 0.2 = 0.2m [Considering stiffness]

λ = 2.625 / 0.2 = 17.5 < 27

Therefore, the stress reduction factor from Table 10 for λ = 17.5 and no eccentricity condition is,

For 16 → 0.73 (e = 0) For 18 → 0.67

For 13.125 → 0.1825 + 0.5025 = 0.685 K_{st} = 0.685



Area reduction factor (K_A) [Cl.5.4.1.2, IS1905-1987]

$A = 0.15 \times 1 = 0.15m^2$

$K_A = 0.7 + (1.5 \times 0.15) = \underline{0.925}$

Shape modification factor (K_{sh}) [Cl. 5.4.1.3, IS1905-1987]

K_{sh} for block of size 200 x 100 x 90 mm laid along 100mm side, from Table 10 for Height to Width ratio of 90 x 100,

$$\frac{Height}{Width} = \frac{90}{100} = 0.9$$

For $0.75x_o \rightarrow 1$

For $1x_o \rightarrow 1.1$

For $0.9x_o \rightarrow 1.06$

$\underline{K_{sh} = 1.06}$

$\sigma_{per\ modified} = K_{st}.K_A.K_{sh}.\ \sigma_{per}$

$= 0.685 \times 0.925 \times 1.06 \times 0.96 = 0.647$

$N/mm^2 < \sigma_{act\ /m}$ [0.8 N/mm^2]

Hence the adopted M1 mortar and masonry unit with compressive strength $10N/mm^2$ is **not** sufficient in carrying the load.

Increase the strength of brick unit and mortar as,

<u>H1</u> mortar and masonry unit compressive strength $\underline{15N/mm^2}$

$\sigma_{per} = 1.31\ N/mm^2$

$\sigma_{per\ modified} = 0.88\ N/mm^2$

Therefore, the interior wall of 150mm thickness is safe with H1 mortar and brick units of compressive strength 15 N/mm^2.

5. Design a masonry wall of height 4m subjected to a load of 20kN/m. Use M1 mortar. The wall is unstiffened[no need to find effective length] at the ends.

Assume a thickness of wall of 300mm

Actual stress = $\frac{20x1000}{300x1000}$ = 0.066N/mm^2

σ_{per} = 0.96 N/mm^2

H_e = 0.75 H = 0.75 x 4 =

3m $\lambda = \frac{3}{0.3}$ = 10

K_{st} = 0.89

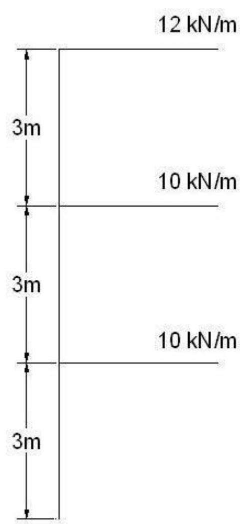
For A = 0.3 x 1 = 0.3 m2, KA =

1 K_{sh} = 1.06

$\sigma_{per\ modified}$ = 0.27 N/mm^2 > σ_{act} [0.066 N/mm^2]

Hence, safe.

6. Design the wall in the GF level for the loading condition as shown in figure.



Loading on brick wall:

Load from slab = 12 + 10 + 10 Weight of = 32 kN/m

wall (self wt.) = 3 x 2 x 0.1 x 3 x 20 Total = 36 kN/m

= 68 kN/m

Actual stress = $\frac{68x1000}{2x100x1000}$ = 0.34 N/mm^2

Use M1 mortar and brick of compressive strength 10 N/mm^2. σ_{per} = 0.96 N/mm^2 [From Table 8, IS1905 – 1987] ($\lambda \le 6$)

h_{eff} = 0.75 x h = 0.75 x 3 = 2.25m

t_e = 2/3(t_w + t_w) = 2/3(0.1 + 0.1) = 0.133m

$\lambda = \frac{2.25}{t_e} = \frac{2.25}{0.133}$ = 16.875

From Table 8, IS1905 – 1987,

λ → 16, 0.75

λ → 18, 0.67

λ → 16.875, (0.421875 + 0.293125 = 0.715) Area of wall (each leaf) = 0.1 x 1 = 0.1m^2 < 0.2 m^2 K_A = 0.85

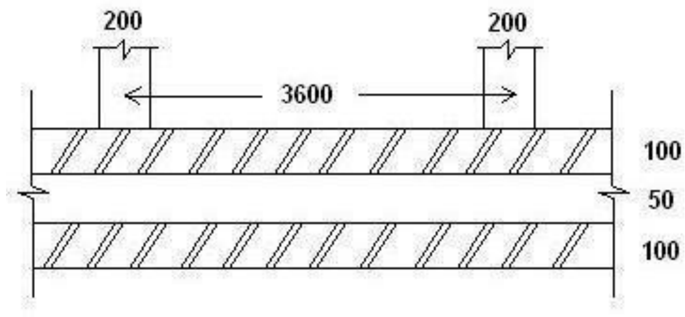
K_{sh} = 1.06

$\sigma_{per\ (modified)} = K_{st} \times K_A \times K_{sh} \times \sigma_{per}$

$= 0.704 \times 0.85 \times 1.06 \times 1.96 = 0.61\ N/mm^2 > \sigma_{act}$

Therefore, the cavity wall is safe with M1 mortar and masonry unit of compressive strength 10N/mm^2.

2. Design a cavity wall of overall thickness 250mm and thickness of each leaf 100mm fora three storeyed building. The wall is stiffened by intersecting walls 200mm thick at 3600mm c/c. The ceiling height is 3m and the loading from roof is 16 kN/m. The loading from each floor is 12.5kN/m.



Load from roof = 16kN/m

Load from floor = 12.5 + 12.5 kN/m

Wall load [3x0.2x20] = 36 kN/m

Total = <u>77 kN/m</u>

Actual stress = $\sigma_{ac} = \dfrac{77x10^3}{200x1000} = 0.39\ N/mm^2 \sigma_{per}$

:

Assume M1 mortar and brick of compressive strength 10 N/mm^2. From Table 7, IS1905 – 1987,

$\sigma_{per} = 0.96\ N/mm^2\ (\lambda \le 6)$

Effective height = $h_{eff} = 0.75 \times 3 = 2.25m$

Effective length = $l_{eff} = 0.8\ l = 0.8 \times 3600 = 2880mm =$ 2.88m Stiffening Coefficient:

Since cross wall is available along one leaf, Sc for

$\dfrac{S_p}{W_p} \quad \dfrac{3.6}{0.2} = 18, \qquad \dfrac{t_p}{t_w} \quad \dfrac{3t_w}{t_w} \quad \dfrac{0.3}{0.1} \quad 3$

From Table 8, IS1905 – 1987,

→ 15, 1.2

→ 20, 1

→ 18, 0.48 + 0.6 = 1.08

Effective thickness of cavity wall, t_e = 2/3 (1.08(0.1) + 0.1) = 0.139m

$\lambda = \dfrac{2.25}{0.139} \quad 16.18$

λ → 16, 0.73

$\lambda \rightarrow 20$, 0.67

$\lambda \rightarrow 16.18$, 0.664 + 0.0603 = 0.7243

$K_{st} = 0.72$

Area = 0.1 x 1 = 0.1m^2 < 0.2

m^2 $K_A = 0.85$

$K_{sh} = 1.06$

$\sigma_{per\ (modified)}$ = 0.62 N/mm^2 > σ_{act} [0.39 N/mm^2]

3. Design a masonry column to carry a load of 150kN. The height of the column is 2000mm. The column is restrained against translation (hinged) only.

Assume a column of size 400 x 400mm.

Use M1 mortar and brick of compressive strength 10 N/mm^2.

Actual stress = $\sigma_{act} = \dfrac{150x10_3}{400x400}$ = 0.94 N/mm^2

σ_{per}:

h_{eff} = h[Table 4, IS1905-

1987] λ = 2000/400 = 5 < 6

There is no need for Stress reduction factor (K_{st} =

1) From Table 7, σ_{per} = 0.96 N/mm^2

A_{st} = 0.4 x 0.4 = 0.16m^2 < 0.2m^2

K_A = 0.7 + (1.5 x 0.4 x 0.4) =

0.94 K_{sh} = 1.06

$\sigma_{per\ (modified)}$ = 0.94 x 0.96 x 1 x 1.06 = 0.98 N/mm^2.

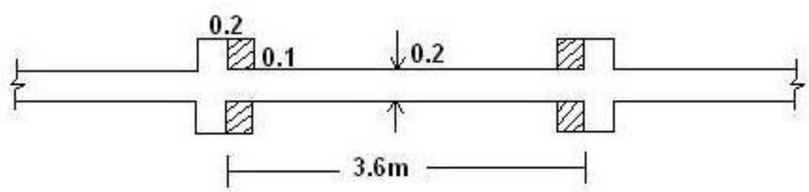
$\sigma_{act} < \sigma_{per}$

Therefore, the masonry column of size 400 x 400mm with M1 mortar and brick unit of compressive strength 10N/mm^2 is safe to carry a load of 150kN.

Note: Boundary condition is assumed if not given.

4. Design an interior wall of a single storeyed workshop building of height 5.4m supporting a RC roof. Assume roof load as 45kN/m. The wall is stiffened by piers at equal intervals shown in figure.

Height = 5.4m, w = 45 kN/m



Since there is an increase in width at the pier, the actual stress is found for the wall length of 3.6m (One bay).

C/s area of one bay = (3.6 x 0.2) + 4(0.1 x 0.1) = 0.76 m^2

Loading per bay (for 3.6m length) = 45 x 3.6 = 162 kN

Load from brick wall = 0.76 x 5.4 x 20 = 82.08 kN

Total = 244.08 kN

Actual stress = $\sigma_{act} = \frac{244.08x10_3}{0.76x10^6}$ = 0.321 N/mm^2

σ_{per}:

h_{eff} = 0.75h = 4.05m [Table 4, IS1905-1987]

demise leff = 0.8l = 0.8 x 3.6 = 2.88m

λ = 2.88 < 6

There is no need for Stress reduction factor (K_{st} =

1) From Table 7, σ_{per} = 0.96 N/mm^2

A_{st} = 0.4 x 0.4 = 0.16m^2 < 0.2m^2

K_A = 0.7 + (1.5 x 0.4 x 0.4) =

0.94 K_{sh} = 1.06

$\sigma_{per\ (modified)}$ = 0.94 x 0.96 x 1 x 1.06 = 0.98 N/mm^2.

$\sigma_{act} < \sigma_{per}$

Therefore, the masonry column of size 400 x 400mm with M1 mortar and brick unit of compressive strength 10N/mm^2 is safe to carry a load of 150kN.

Stiffening coefficient,

$$\frac{S_p}{w_p} \quad \frac{3.6}{0.2} = 18, \qquad \frac{t_p}{t_w} \quad \frac{0.4}{0.2} \quad 2$$

(tp → depth of pier

(0.4)) Sc = 1.04

t_{eff} = 1.04 x 0.2 = 0.208m

λ is the least of H_{eff} and L_{eff}λ

= $\frac{2.88}{0\ 208}$ = 13.85

Kst = 0.785

Area reduction coefficient = $\frac{0.76}{3.6}$ = 0.211 > 0.2

K_A = 1 [Cl.5.4.1.2, IS1905 – 1987]

Ksh = 1.06

$\sigma_{per\ (modified)} = 0.785 \times 0.96 \times 1 \times 1.06 = 0.79\ N/mm^2 > \sigma_{act}$

Inference : Hence the brick wall is safe with M1 mortar and brick of compressive strength $10N/mm^2$.

5. Design a brick masonry column of height 3m, tied effectively, fixed at top and bottom. The load from slab is 100kN, including self weight of the brick pillar.

Load from slab = 100kN

Self weight of brick pillar = 0.4 x 0.4 x 20 x 3 = 9.6kN

Total = 109.6 kN

Assume a column size of 400mm x 400mm.

Actual stress = $\frac{109.6x10_3}{400x400}$ = 0.685 N/mm^2

Assume grade of mortar as M1 and compressive strength of

$0.96N/mm^2$ h_{eff} = 0.75.H = 2.25m

$\lambda = \frac{2.25}{0.4} = 5.625 < 6$

There is no need of stress reduction

factor. K_{st} = 1

$A = 0.4 \times 0.4 = 0.16\ m^2 < 0.2m^2$

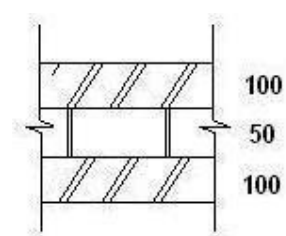
$K_A = 0.7 + (1.5 \times 0.4 \times 0.4) = 0.94$

K_{sh} = 1.06 [Brick unit 200 x 100 x 90]

$\sigma_{per\ (modified)} = 0.9565\ N/mm^2 > \sigma_{act}$

Hence the brick wall is safe with M1 mortar and compressive strength of $10N/mm^2$.

6. Design an interior wall of a 3 storeyed building with ceiling height of each storey as 3m. The wall is unstiffened and 3.6m in length. Load from roof is 12kN/m and from each floor is 10kN/m. Select a cavity wall with overall thickness 250mm and length between each leaf as 50mm.



h = 3m, l = 3.6m

Load from roof = 12 kN/m

Load from each floor = 10 + 10 kN/m

Self weight of wall = 3 x 2 x 0.1 x 3 x 20 = 36 kN/m

Total = 68 kN/m

Actual stress = $\frac{68x10^3}{2x100x1000}$ = 0.34 N/mm^2

Use M1 mortar and brick of compressive strength

10N/mm^2, σ_{per} = 0.96 N/mm^2

h_{eff} = 0.75 x 3 = 2.25m t_e

= 2/3(t_w + t_w) = 0.133m

λ = $\frac{2.25}{}$ =

16.875 t_e

From Table 8, IS1905 –

1987, K_{st} = 0.704

Area of wall = 0.1 x 1 = 0.1 m^2 < 0.2

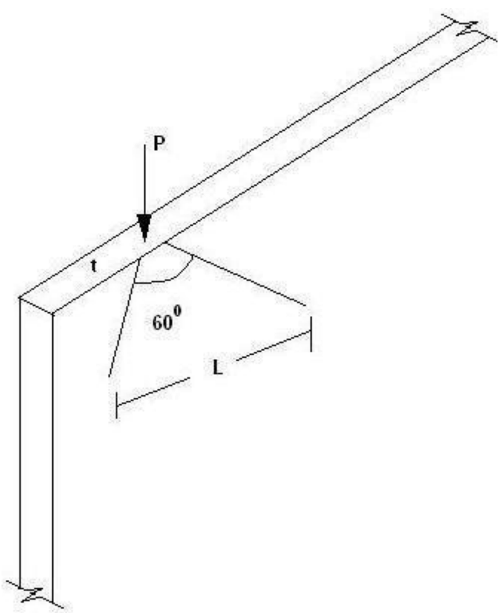
m^2 K_A = 0.85

K_{sh} = 1.06

$\sigma_{per\ (modified)}$ = 0.704 x 0.85 x 1.06 x 0.96 = 0.61 N/mm^2 > σ_{act}

Hence the brickwork is safe with M1 mortar and brick of compressive strength 10N/mm^2.

MASONRY WALL SUBJECTED TO CONCENTRATED LOAD



$\sigma_{br} < \sigma_{br\ per}$

where, σ_{br} = $\frac{Load\_from\_beam}{Bearing\_area}$ $\frac{P}{A_{br}}$,

where, P → reaction from conc. load [Eg: From Beam]

A_{br} → Bearing area

$\sigma_{br\ per}$ = 1.5 (σ_{per}) [σ_{per} given in Table 8, IS1905-1987]

When concentrated load is applied on a masonry wall, the wall is checked for load bearing stress. Permissible stress in bearing is taken as 50% more than the value given in Table 7 of IS1905-1987.

The angle of dispersion below the concentrated load is 30^o on each side. Therefore, the actual stress is,

σ_{act} = P / A,

where, A → Area for 1m run = L x t,

L → Length of load dispersion = $\frac{2H}{\tan 60^0}$, H → Height of wall

For the wall to be safe in carrying the load, $\sigma_{act} < \sigma_{per}$

1) Design a solid wall of a mill building 3m height securely tied with roof and floor units. The wall supports two beams on either side exerting reactions of 30kN and 20kN. Thickness of wall is 230mm and the beam bears on the wall for 115mm (width of beam). Neglect load due to self weight.

A_{br} = 230 x 115 = 26450 mm^2

$$\sigma_{br} = \frac{P}{A_{br}} \quad \frac{(30\ 20)x10^3}{26450} = 1.89\ \text{N/mm}^2 < \sigma_{per} \text{ in bearing}$$

The values given in Table 8 are increased by 50% for σ_{per} in bearing.

Therefore, assume H1 grade of mortar and brick of compressive strength 15 N/mm^2.

σ_{per} = 1.31 N/mm^2

$\sigma_{per\ br}$ = 1.5 x 1.31 = 1.965 N/mm^2

Check for compressive stress:

σ_{act} = P / A, A = L x t, where, L → Length of load dispersion

$$L = \frac{2H}{\tan 60^0} = \frac{2x3}{\tan 60^0} = 3.464 \text{ m}$$

A = 3.464 x 1000 x 230 = 796720 mm^2

$$\sigma_{act} = \frac{50x10_3}{796720} = 0.063\ \text{N/mm}^2$$

σ_{per} = 1.31 N/mm^2

K_{st} = 0.89

K_A = 1

K_{sh} = 1.06

$\sigma_{per\ modified}$ = K_{st} x K_A x K_{sh} x σ_{per}

= 0.89 x 1 x 1 x 1.31 = 1.1659 N/mm^2 > σ_{act} [0.063 N/mm^2]

The wall is safe in carrying a concentrated load with H1 mortar and brick of compressive strength 15N/mm^2.

2) Design the exterior wall of a workshop building 3.6m height carrying steel trusses at 4.5m spacing. The wall is securely tied at roof and floor levels. The wall is of thickness 200mm and the truss bears on the wall for 200mm and load from the truss is 30kN.

[Length is considered only for piers and cross

walls] A_{br} = 200 x 200 = 40000 mm^2

$\sigma_{br} = \frac{30x10_3}{40000} = 0.75$ N/mm^2 < $\sigma_{br\ per}$ → [1.5(σ_{per}) = 1.5 x 0.96 = 1.44 N/mm^2]

Assume M1 mortar and brick of compressive strength 10 N/mm^2.

Check for compressive stress:

σ_{ac} = P/A

$L = \frac{2H}{\tan 60^0} = 4.157m$

A = 4.157 x 10^3 x 200 = 956110mm^2

$\sigma_{act} = \frac{30x10_3}{956110} = 0.036$ N/mm^2

H_{eff} = 0.75 x 3.6 = 2.7m

L_{eff} = 0.8 x 4.157 = 3.3256m

$\lambda = \frac{2.7x10^3}{2300} = 13.5$

A = 4.157 x 0.2 = 0.8314 m^2 > 0.2 m^2

K_A = 1

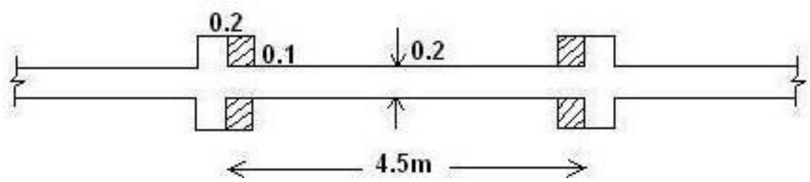
K_{st} = 0.795

K_{sh} = 1.06

$\sigma_{per\ modified}$ = 0.81 N/mm^2

3) In the above problem, design the wall if piers are available below the truss and size of pier is 200 x 400mm.

Here, we need to take the length. If the truss fully rests on pier, bearing area,



A_{br} = 80000 mm^2

P = 30 x 10^3 N

$\sigma_{br} = \frac{30x10^3}{200x400}$

σ_{br} = 0.375N/mm^2 < $\sigma_{br\ per}$ → [1.5(0.96) = 1.44 N/mm^2]

Using M1 mortar and brick of compressive strength 10N/mm^2, Check for compressive stress:

σ_{ac} = P/A

$L = \frac{2H}{\tan 60^0} = 4.16m$

H_{eff} = 0.75 x 3.6 = 2.7m

$L_{eff} = 0.8 \times L = 0.8 \times 4.5 = 3.6m$

$t_e = S_c \times t$

$$\frac{S_p}{W_p} = \frac{4.5}{0.2} = 22.5 \quad , \quad \frac{t_p}{t_w} = \frac{0.4}{0.2} = 2$$

$S_c = 1$

$t_e = 1 \times 0.2 = 0.2m$

$$\lambda = \frac{2.7}{0.2} = 13.5$$

$A = L \times t = 4.5 \times 0.2 = 0.9\ m^2 > 0.2\ m^2$

$K_A = 1$

$K_{st} = 0.795$

$$\sigma_{act} = \frac{30x10^3}{4.5x0.2} = 0.03N/mm^2$$

$\sigma_{per\ modified} = 1 \times 1 \times 0.795 \times 1.06 \times 0.96 = 0.8089\ N/mm^2$

Hence the wall is safe with M1 mortar and brick of compressive strength $10N/mm^2$.

ECCENTRICALLY LOADED BRICK MASONRY

Eccentricity – Offset distance from CG of member to CG of load

Occurs in,

i) Exterior wall –Bearing not sufficient

ii) Flexible slab – Excessive deflection (timber)

iii) When span is very large, code recommends to take some amount of eccentricity

$$\sigma_{act} = \frac{P}{A} \quad \frac{M}{Z} < 1.25(\sigma_{per\ modified}),$$

where, M → P x e, Z → $b.t^2/6$, b = 1m (wall)

<u>Guidelines given in Appendix A</u>

Eccentricity in loading occurs in loading on walls due to various reasons causing reduction in stress. The cases where eccentricity needed to be assumed are given in Appendix A of IS1905-1987.

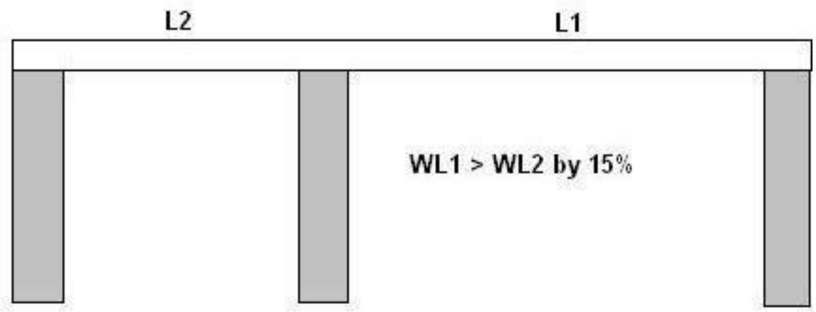
1. For an exterior wall, when span of roof is more than 30 times the thickness of wall, the eccentricity assumed is one sixth of the bearing width.

   $L > 30\ t_w$

   $E = 1/6(t_w)$

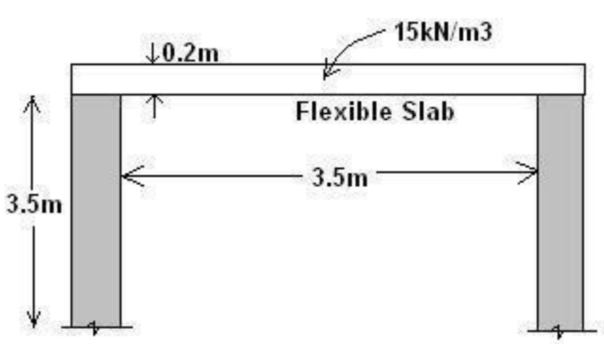
2. When bearing is not sufficient, eccentricity assumed is $t_w/12$

3. When flexible floors are adopted, for full width bearing, eccentricity assumed is $t_w/6$.
4. For interior walls, when there is unequal length of slabs on both sides and then the difference between the loading is greater than 15%, moment is generated for which, e=M/P.



Actual stress (σ_{act}) is the sum of direct compressive stress P/A and bending stress M/Z. The permissible stress given in Table 8 can be increased by 25% and modification factors applied on it.

1) Design an exterior wall of height 3.5m, which is unstiffened. The slab is light weight flexible slab of length 3.5m. Assume the unit weight of slab as 15kN/m^3 with thickness 0.2m.



Half of the load from the slab comes to the wall and since the slab is flexible, eccentricity considered as per Appendix A of IS1905 – 1987.

Assume 200mm thick wall with M1 mortar and brick of compressive strength of 10N/mm^2.

Loading on wall:

Load from slab = 15 x 0.2 x 3.5/2 = 5.25 kN/m

Self weight of wall = 0.2 x 3.5 x 20 = 14 kN/m

Total = <u>19.25 kN/m</u>

e = $t_w/6$ = 33.33mm

M = P x e

Moment due to eccentricity = 19.25 x 10^3 x 33.33 = 641.67 x 10^3

Nmm Z = bt^2/6 = 1000 x 200^2 / 6 = 6.67 x 10^6 mm^3

$M/Z = 0.096\ N/mm^2$

Total stress = $\frac{P}{A}\ \frac{M}{Z} = \frac{19.25x10^3}{1000x200} + 0.096 = 0.192\ N/mm^2$

σ_{per} :

Use M1 mortar and brick of compressive strength $10N/mm^2$,

$\sigma_{per} = 0.96\ N/mm^2$

Stress reduction factor (K_{st})

Slenderness ratio (Least of l_e/t & h_e/t)

From Table 4,

$h_e = 0.75\ H = 0.75 \times 3 = 2.25m$ [Both ends fixed]

$h_e/t = 2.625 / 0.2 = 13.125 < 27$

Therefore, the stress reduction factor from Table 10 for $\lambda = 13.125$ and no eccentricity condition is,

For 12 → 0.78 (e = 0)

For 14 → 0..7

For 13.125 → 0.735

$K_{st} = 0.735$

Area reduction factor (K_A) [Cl.5.4.1.2, IS1905-1987]

For thickness t = 0.2m, A = 0.2 m^2

$K_A = 1$

Shape modification factor (K_{sh}) [Cl. 5.4.1.3, IS1905-1987]

K_{sh} for block of size 200 x 100 x 90 mm laid along 100mm side, from Table 10 for Height to Width ratio of 90 x 100,

$$\frac{Height}{Width} = \frac{90}{100} = 0.9$$

For $0.75x_o$ → 1

For $1x_o$ → 1.1

For $0.9x_o$ → 1.06

$K_{sh} = 1.06$

$\sigma_{per\ modified} = K_{st}.K_A.K_{sh}.\ \sigma_{per}$

$= 0.735 \times 1 \times 1.06 \times 1.25 \times 0.96 = 0.935$

N/mm^2 σ_{act} [$0.192N/mm^2$] < 1.25 σ_{per} [$1.17N/mm^2$]

Note : For brick masonry columns laterally supported by beams,

$H_e = H$

Only when the column is not laterally supported (laterally unsupported), $H_e = 2H$

2) Design a masonry column tied effectively at top and bottom. Load from slab is 100kN. Assume size of column as 400 x 400mm

$\sigma_{act} = \frac{100x10_3}{400x400} = 0.625 \text{ N/mm}^2$

$\sigma_{per} = 0.96 \text{ N/mm}^2$

Heff = H = 2m

$\lambda = 5 < 6$

$K_{st} = 1\ K_{sh}$

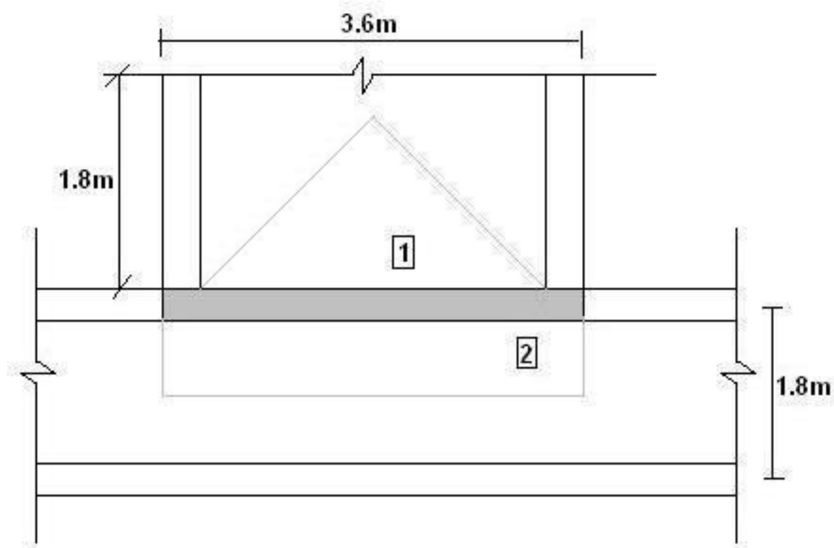
$= 1.06\ K_A$

$= 0.94$

$\sigma_{per\ modified} = 0.957$

$\text{N/mm}^2\ \sigma_{act} < \sigma_{per}$

3) Design an interior wall of a single storeyed building supporting unequal concrete roof slabs. The plan is as shown in figure. Assume triangular pressure distribution and unit weight of slab is 10kN/m^3. The height of the wall is 3.8m and the wall is fixed to the foundation block below.



Height = 3.8m

$e = \frac{t_w}{6} \quad \frac{t_w}{6} = \frac{2t_w}{6} = \frac{t_w}{3}$

$\sigma_{act} = \frac{P}{A} \quad \frac{M}{Z}$

Loading from slab 1:

= Area of triangle x Load intensity on slab = $\frac{(1/2)x3.6x1.8x10}{3.6}$ = 9 kN/m

Loading from slab 2:

= Area of rectangle x Load intensity on slab = $\frac{(1/2)x(1.8/2)x3.6x10}{3.6}$ = 4.5 kN/m

Self weight of brickwork = 0.2 x 3.8 x 20 = 15.2 kN/m

= 28.7 kN/m

Since the difference in loading is 100% [> 15%], there is an eccentricity in the loading. The eccentricity in the loading as per Appendix A of IS1905-1987 is,

$e = \frac{t_w}{6} \quad \frac{t_w}{6} = \frac{2t_w}{6} = \frac{t_w}{3}$

M → (Difference in load from slab x Eccentricity)

= (9 – 4.5) x 10^3 x 200/3 = 300 x 10^3 Nmm

$\sigma_{act} = \frac{P}{A} \quad \frac{M}{Z}$, where $Z = \frac{b.t^2}{6} = \frac{1000x200^2}{6}$ = 6.67 x 10^6 mm^3

$\sigma_{act} = \frac{28.7x10^3}{1000x200} \quad \frac{300x10^3}{6.67x10^3}$ = 0.1885 N/mm^2

$e = t_w/3$ → $e/t_w = 1/3$

λ is least of (H_e/t_e and L_e/t_e)

Assume width of the cross wall as 200mm

$t_e = t \times S_c$

$\frac{S_p}{W_p} \quad \frac{3.8}{0.2}$ = 19, $\frac{t_p}{t_w} \quad \frac{0.6}{0.2}$ 3

S_c = 1.04

t_e = t x Sc = 1.04 x 0.2 = 0.208m

H_{eff} = 0.75 x 3.8 = 2.85m

$\lambda = \frac{2850}{208}$ = 13.7

L_{eff} = 0.8 L = 0.8 x 3.60 =

2.88m λ = 13.8

λ → 12 0.72

λ → 14 0.66

λ → 13.7 0..67

K_{st} = 0.67

A = 0.1 x 1 = 0.1m^2 < 0.2m^2

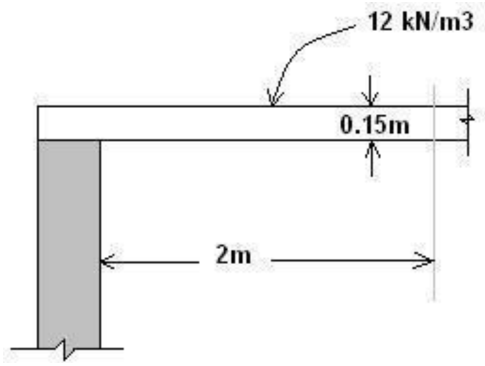
$K_A = 1$ $K_{sh} =$

1.06

$\sigma_{per\ modified} = 0.96 \times 0.67 \times 1 \times 1.06 = 0.68$ N/mm^2

σ_{act} [0.1885 N/mm^2] < 1.25(σ_{per}) → [0.851 N/mm^2]

4) Design an exterior wall of height 4m, unstiffened and supports a flexible slab 150mm thick with unit weight 12 kN/m^3. The length of the slab is 4m.



Load from slab = 2 x 0.15 x 12 = 3.6 kN/m

Self weight of wall = 0.2 x 4 x 20 = 16 kN/m

Total = <u>19.6</u> kN/m

$e = t_w/6 = 200/6 = 25$mm

$M = P \times e = 19.6 \times 10^3 \times 25 = 6.53 \times 10^5$ Nmm

$$Z = \frac{b.t^2}{6} = \frac{1000x200^2}{6} = 6.67 \times 10^6 \text{ mm}^3$$

$$\sigma_{act} = \frac{P}{A} \quad \frac{M}{Z} = 0.098 + 0.097 = 0.195 \text{ N/mm}^2$$

$H_{eff} = 0.75H = 3$m

Assuming M1 mortar and brick of compressive strength 10 N/mm^2,

$$\lambda = \frac{3000}{200} = 15$$

λ → 14	0.7
λ → 16	0.63
λ → 15	0.35 + 0.315 = 0. 665

$K_{st} = 0.665$

A = 0.1 x 1 = 0.1m^2 < 0.2m^2

$K_A = 1$

$K_{sh} = 1.06$

$\sigma_{per\ modified} = 0.96 \times 0.665 \times 1 \times 1.06 = 0.677$ N/mm^2

σ_{act} [0.195 N/mm^2] < 1.25(σ_{per}) → [0.846 N/mm^2]

SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – II – BUILDING STRUCTURES II– SCIA1306

UNIT II

TIMBER TRUSSES

Introduction to sloped roofs: Ridges, hips, valleys and eaves - Slope with respect to roofing materials - Wood rafter span range: roof beams parallel to slope and perpendicular to slope: Trusses - types of trusses, properties and their span ranges - analysis of truss to determine the Forces, deflection of truss - joinery details of different members of timber truss. (No Design Problem)

INTRODUCTION

Roofs are one of a building's primary elements and play a major part in giving a building its character. A roof is part of a building envelope. It is the covering on the uppermost part of a building or shelter which provides protection from animals and weather, notably rain or snow, but also heat, wind and sunlight. The word also denotes the framing or structure which supports that covering.

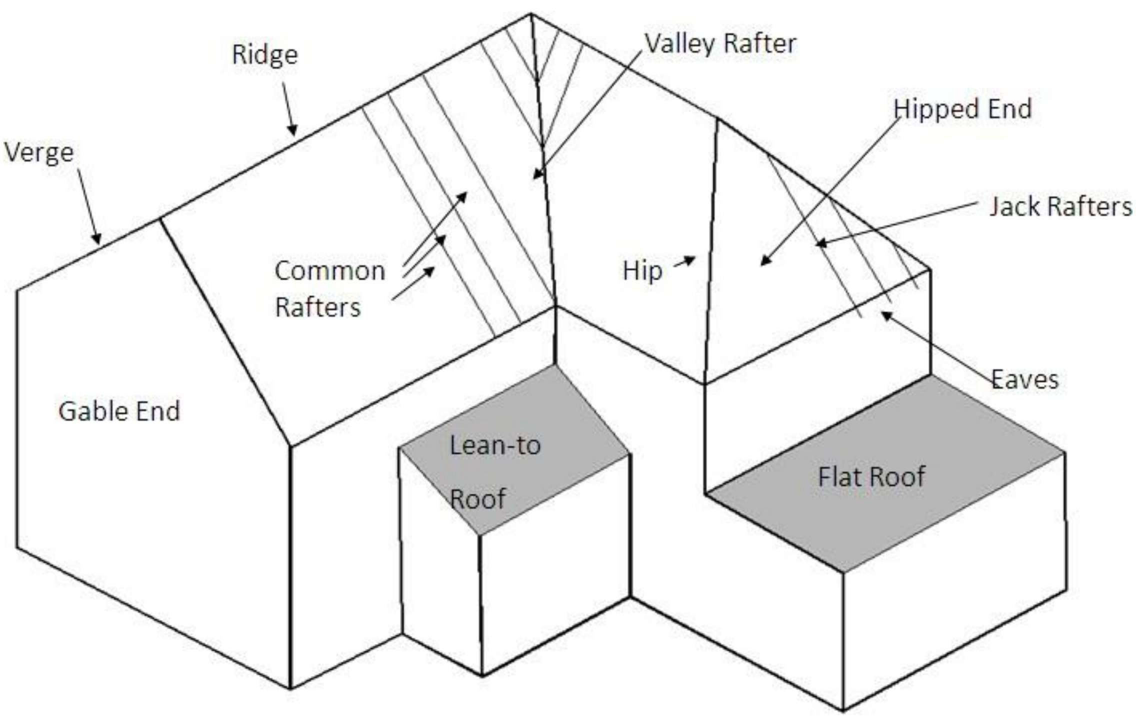
The characteristics of a roof are dependent upon the purpose of the building that it covers, the available roofing materials and the local traditions of construction and wider concepts of architectural design and practice and may also be governed by local or national legislation. In most countries a roof protects primarily against rain. A verandah may be roofed with material that protects against sunlight but admits the other elements. The roof of a garden conservatory protects plants from cold, wind, and rain, but admits light.

The shape of roofs differs greatly from region to region. The main factors which influence the shape of roofs are the climate and the materials available for roof structure and the outer covering.

The basic shapes of roofs are flat, pitched, gabled, hipped, butterfly, arched and domed. There are many variations on these types. Roofs constructed of flat sections that are sloped are referred to as pitched roofs (generally if the angle exceeds 10 degrees) Pitched roofs, including gabled, hipped and skillion roofs, make up the greatest number of domestic roofs. Some roofs follow organic shapes, either by architectural design or because a flexible material such as thatch has been used in the construction.

SLOPED ROOFS

A roof having one or more surfaces with a slope greater than 10° from the horizontal is called sloped proofs.



A sloping roof is known as a pitched roof.the definitions of technical terms used in connection with the pitched roofs are given below.

Pitch: The steeper the pitch the greater the roof area visible. This will result in a larger roof space, you can also use smaller cladding units such as plain tiles and slates. Pitched roofs are the most suitable in countries where there is a high rain fall.

Wall plate: Usually 100 x 50 mm softwood timbers are fixed to the top of load bearing walls to distribute loads and provide fixings for roof timbers.

Ceiling joist: These are timbers which provide a support for fixing ceiling finishes and act as a collar to prevent rafters spreading.

Common rafters: These are inclined timbers fixed between wall plate and ridge which transmit live and dead loads to wall plate.

Ridge : The ridge is a horizontal board set on edge to which the rafters are attached (not required on trussed rafters).

Hip Rafter: A hip rafter is a rafter running from the wall plate to the ridge which forms the external angle of the sloping side of a roof.

Purlin: This is a horizontal roof member supporting the rafters and usually at right angles to these. This enables small section timbers to be used for the rafters.

Hangers: These are timbers hanging from the purlins to the ceiling joist to give additional support to binders.

Fascia : A board fixed vertically to rafter ends, which provide an additional fixing for gutters.

Soffit: A horizontal board fixed to the underside of the rafter outside the building.

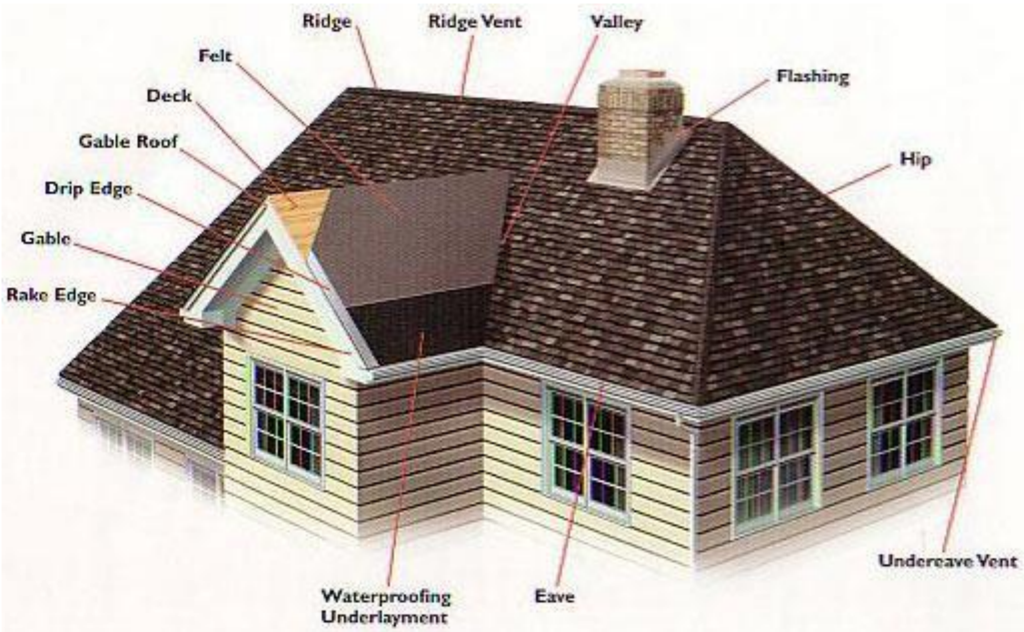
Bargeboard:Verge or gable board.

Dormer : A vertical window coming through a sloping roof.

Valley: This is the name for the intersection between two sloping surfaces, forming an internal angle (the opposite to a hip).

Learn-to-roof: This is the simplest form of sloping roof, provided either for a room of small span, or for the verandah. It has slope only one side.

Gable: A gable is the generally triangular portion of a wall between the edges of a sloping roof. The shape of the gable and how it is detailed depends on the structural system being used (which is often related to climate and availability of materials) and aesthetic concerns. Thus the type of roof enclosing the volume dictates the shape of the gable.



Hip Roof: The hip roof has four sloping sides. It is the strongest type of roof because it is braced by four hip rafters.

Ridge vent: A ridge vent is a type of vent installed at the peak of a sloped roof which allows warm, humid air to escape a building's attic. Ridge vents are most common on shingled residential buildings.

Hip: It is a ridge formed by the intersection of two sloping roofs.

Valley: It is a depression formed at the intersection of two sloping roofs.

Valley rafter: These are the sloping rafters which run diagonally from the ridge to the eaves for supporting valley gutters. They receive the enfs of the purlines and ends of jack rafters on both sides.

Rise: It is the vertical height measured from the lowest to the highest points.

Eaves: The lower edges of a roof which are resting upon or projecting beyond the supporting walls are know as the eaves. Sometimes a thin board of wood is provided at the eaves to cover the ends of the common rafters. Such a board is known as and eaves board pr facia board.

Batterns: These are the thin strips of wood which are fixed on the rafters or ceiling. They support the roof ceiling.

Wall plate: These are long wooden members embedded in the walls to hip to the eaves or from a ridge to a valley.

VALLEY, HIP & RIDGE

Hip & Ridge

A roof "hip" refers to the outwards diagonal joint created by the junction of two roof slopes.

A roof "ridge" refers to the upper most peaks created by the junction of two roof slopes facing opposite directions.

In the age where 3-tab shingles were the standard in roofing, it was common practice to take the same shingle used to cover the roofing slopes, cut it into 3 pieces and use it to protect your hips and ridges. Today, where laminate products are the overwhelming shingle of choice, specific shingles are manufactured for the hips and ridges. It is still OK to use a 3 tab shingle if you can find a colour to match your shingle of choice, but it is recommended by manufacturers to use specialty cap shingles for laminate and designer products to compliment their enhanced warranties.

Hip and Ridge cap shingles are traditionally a single layer shingle, but manufacturers are now producing double layered shingles, and even shingles that are folded to create superior strength and design.

Valley

Valleys are created where two roof slopes meet. Because of the volume of water and lower slope along valley lines, valleys are particularly vulnerable to leakage.

Open Valleys

Open valleys are so defined because the valley line remains uncovered by the primary roofing material. Open valleys are protected by a combination of ice and water shield and metal.

In a proper open valley application:

- 1 row (3’) of ice and water shield is applied in the center of the valley
- A “W” or “V” shaped piece of metal is bent and installed on top of the ice and water
- A second row of ice and water shield is cut in half and used to sandwich the metal on both sides

- Shingles are cut along the line created by the 2nd row of ice and water, and the metal remains "open" to carry the water off of the roof
- The opening should gradually widen to handle the increased capacity at the lower edge

TWO common metal valley designs include:

- "V" - where one bend is made to match the angle of the adjoined roof slopes
- "W" - where a bump is bent in the center to prohibit the water from entering under the shingles on the adjoining roof slope

Many roofers will quote low gauge metal or aluminum as their open valley of choice and exclude a row of ice and water shield. At Dayus, we only use heavy gauge "W" shaped pre-finished galvanized metal valleys (unless otherwise requested). Heavy gauge metal keeps valley lines straight and the "W" shape offers better water control. Be sure to ask what kind of metal is being used and how it is applied when comparing quotations. Not all "metal valleys" are the same. If the price points provided for a metal valley option seem drastically different, there is usually a reason why.

Closed Valleys 

Closed valleys are covered over using the shingles or other primary roofing material.
In a proper closed valley application:

- 1 row (3') of ice and water shield is applied in the center of the valley
- The shingles, or other primary roofing material, are installed though the valley

TWO common closed valley designs include:

- ***Woven style*** - where shingles are overlapped across the valley on alternative slopes
- ***Closed cut*** **-** where shingles are first installed on the lower adjoining roof slope through the valley line, followed by the shingle installation on the higher slope through the valley line. The shingles on the higher slope are then cut in a straight line through the valley. This ensures that the water from the higher slope falls onto the shingles of the lower slope vs. under the shingles to the roof deck

Many roofers overlook the details required for a proper closed style valley application. As with all the detailed areas of your roof, it is important to know the details of each valley style in order to make an educated decision for your roofing system. If you are looking to save money, closed cut valleys are the way to go. However, if you are looking for a valley that will outlast most roofing systems, open style valleys should be installed.

SLOPE WITH RESPECT TO ROOFING MATERIALS

Steep slope:

Steep slope roofing usually refers to roofing materials suitable for roofs that have slopes of 3:12 or greater. This means for every 12 horizontal inches, the roof's rise is 3 inches or greater. Materials suitable for steep slope roofing include asphalt roll roofing as well as asphalt shingles, concrete and clay tiles, wood shakes, slate and metal roofs. Additionally, some of the modified bituminous roofing as well as certain adhered single-ply membranes and SPF foam can be used in certain steep slope roofing applications.

The most common steep slope roofing types are asphalt shingles. Asphalt shingles are composed of fibrous glass mats saturated with asphalt. Because asphalt is easily degraded by UV exposure, varying colored mineral granules are embedded into the exposed side of the asphalt shingle that also imparts its finished color.

Concrete tiles, clay tiles, and wood shakes are most often associated with residential construction but can be used for steep slope commercial roofing as well. Concrete or clay tiles can be installed in overlapping "shingle fashion" on steep slope roofs. These tiles are usually installed with a combination of nails and cementitious mortar.

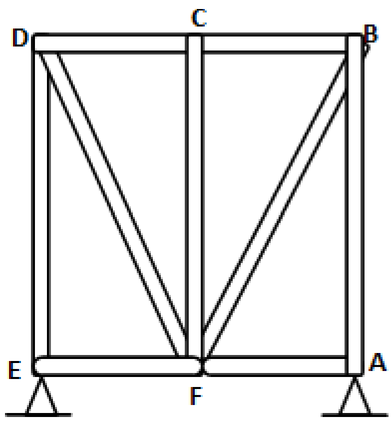
Metal roofing can be used in both steep slope roofing and low slope roofing applications. Metal roofing systems have gained popularity because of their heat resistance, longevity and color customization options.

Low slope:

A "low slope roof" is one that slopes 2" in 12" to 4" in 12" of run.Because low slope roofs also drain water more slowly than moderate or steep sloped roofs, systems such as slate or asphalt shingles that rely on mechanical drainage for successful performance, are not used. Instead we use sealed or membrane type roofing systems similar to those used on "flat" roofs.

TRUSS

Am truss is a network of straight slender members connected at the joints. Members are essentially connected at joints. The every member has force only at extremities. Further for equilibrium the forces in a member reduce to two force member. Thus no moments only two force member. In general trusses are designed to support. Trusses are designed to support weight only in its plane. Therefore trusses in general can be assumed to be 2-dimensional structures. Further in case weight of individual member is to be taken into consideration, half of them are to be distributed at each of the pinned ends. Figure below shows a sample truss. There are nine individual members namely DE, DF, DC, BC, BF, BA, CF, EF, FA. Structure is 2-dimensional structure, supported by pin joints at A and E.



Sometime a member may be given of a shape like:In such a case take the line the line joining their ends as the line of action of force.

TYPES OF TRUSSES

Queen Post Truss

The Queen Post Truss is designed to be a very reliable, simple and versatile type of roof truss that you can use at any given time. It offers a good span, around 10m, and it has a simple design which makes it perfect for a wide range of establishments.

Howe Truss

This type of truss is a combination of steel and wood, which makes it elegant, while also offering a very appealing design. Almost everything is made out of wood, however, the tension members or the vertical members are manufactured out of steel in order to offer extra support and reliability! One thing that makes the Howe Truss extraordinary is the fact that it has a very wide span, as it can cover anything from 6-30m. This makes it versatile and very useful for a wide range of project types.

Fan Truss

The Fan Truss comes with a very simple design and it's made out of steel. In this particular situation, the trusses form a fink roof truss. On top of that, the main characteristic here is that the top chords are split into smaller lengths, as this allows the build to obtain purlin support. Also, you get a medium span with this type, around 10-15m, which is more than enough for most projects.

North Light Roof Truss

The North Light Roof Truss is suitable for the larger spans that go over 20m and get up to 30m. This happens because it’s cheaper to add a truss that has a wide, larger set of lattice girders that include support trusses. This method is one of the oldest, as well as most economical ones that you can find on the market, as it allows you to bring in proper ventilation. Plus, the roof has more resistance too because of that.

If you are looking for types of roof trusses design that bring in durability and versatility, this is a very good one to check out. You can use it for industrial buildings, but this truss also works for drawing rooms and in general those spaces that are very large.

Quadrangular Roof Trusses

These are used for large spans, and this is why you can encounter then in larger spaces, which include auditoriums or even railway sheds.

Parallel Chord Roof Truss

These types of trusses are created specifically for those of us that want to engage in a roof construction without having a large budget to begin with. These are made out of wood and the best part about using them is that they don’t require any beam nor bearing wall. Instead, they opt for full pieces of wood and thus lower the amount of labor necessary for working with them. It does require more space in the attic and the span might not be the best, but the price might justify opting for it if you are on a budget.

Scissor Roof Truss

A Scissor Roof Truss can particularly be found in cathedrals. It doesn’t require beams or bearing walls, however it doesn’t leave that much space for insulation which makes its energy efficiency very poor. On the other hand, the upside here is that the ceiling gets vaulted and you receive more space in the attic.

Raised Heel Roof Truss

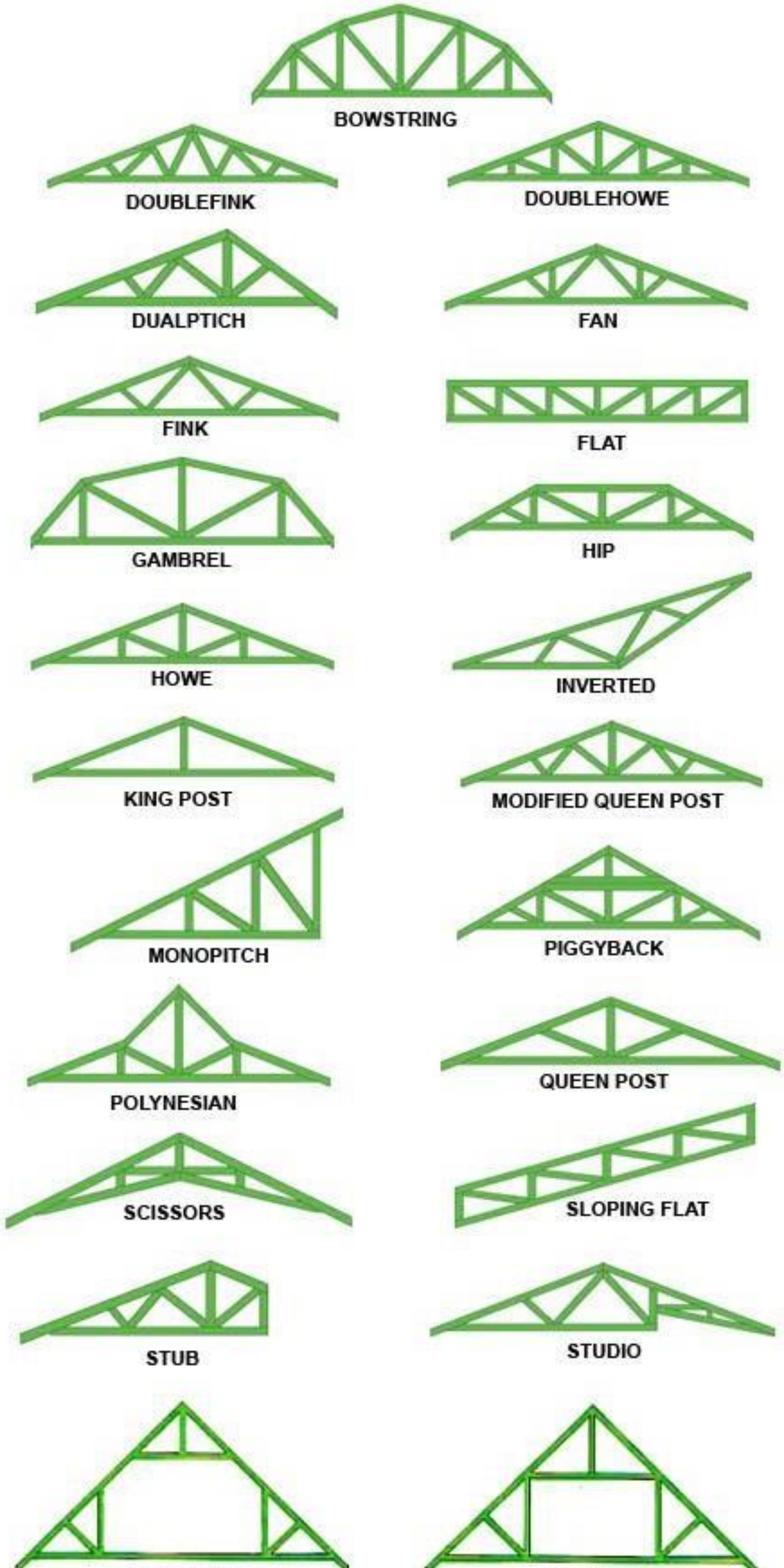
This is one of the most efficient types of timber roof trusses, mostly because it brings a very good room for insulation, but at the same time it also provides you with a very good systemfor structural support. It might require some additional materials in order to make it bring mention that the costs can be a little higher when compared to other truss types, but it does help you lower the energy bill value, so keep that in mind.

Most roof trusses are easy to customize, but even so, there are so many of them that it all comes down to you and the home design, because you are bound to like one that suits your needs. A very good idea is to work with a roof contractor in order to study your home and see which is the most suitable roof truss for your home.

King post truss

One of the simplest truss styles to implement, the **king post** consists of two angled supports leaning into a common vertical point.

TRUSS CHART


BOWSTRING
DOUBLEFINK
DOUBLEHOWE
DUALPTICH
FAN
FINK
FLAT
GAMBREL
HIP
HOWE
INVERTED
KING POST
MODIFIED QUEEN POST
MONOPITCH
PIGGYBACK
POLYNESIAN
QUEEN POST
SCISSORS
SLOPING FLAT
STUB
STUDIO
OPEN PLAN
STORAGE


KINGPOST
DIAGONAL
TOP CHORD

ANALYSIS OF TRUSSES

1.Method of Joints

The most common way to determine forces inside a truss is method of joints. The basic concept of method of joints is that, since the truss is in equilibrium, each joint in truss will also be in equilibrium. The procedure for method of joints is as follows.

Procedure for analysis

The following is a procedure for analyzing a truss using the method of joints:

1. If possible, determine the support reactions

2. Draw the free body diagram for each joint. In general, assume all the force member reactions are tension (this is not a rule, however, it is helpful in keeping track of tension and compressionmembers).

3. Write the equations of equilibrium for each joint, $\sum Fx = 0 \quad \sum Fy = 0$

4. If possible, begin solving the equilibrium equations at a joint where only two unknown reactions exist. Work your way from joint to joint, selecting the new joint using the criterion of two unknown reactions.

5. Solve the joint equations of equilibrium simultaneously,

PROBLEM: 1

Find the force acting in all members of the truss shown in Figure T-01.

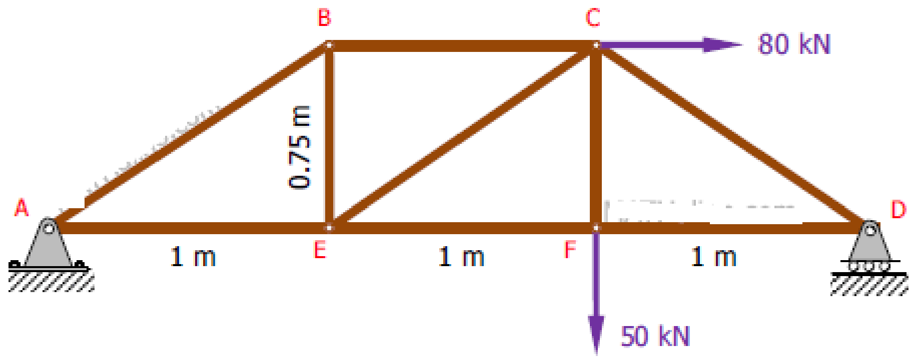


Figure T-01

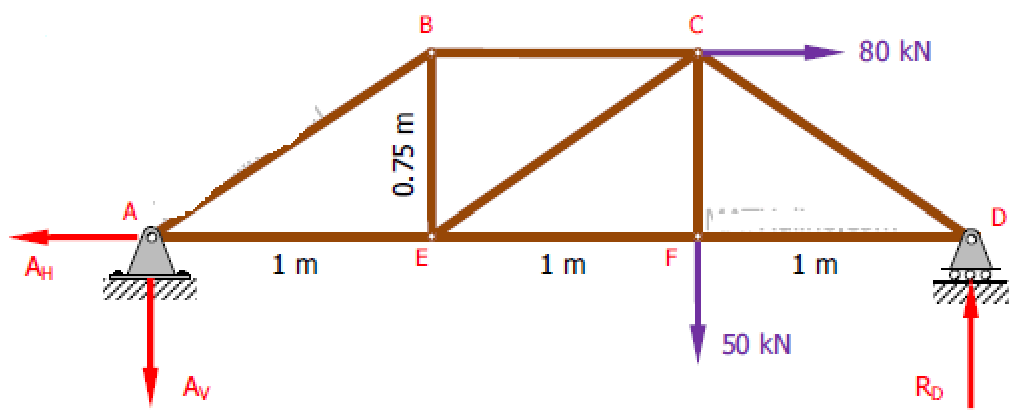
ΣMD=0

$3A_V+50(1) = 80(0.75)$

$A_V = 3.33$ kN

ΣFH=0

$A_H=80$ kN



ΣMA=0

3RD=50(2)+80(0.75)

RD=53.33 kN

At joint A

$\Sigma F_V = 0$

$\frac{3}{5} F_{AB} = 3.33$

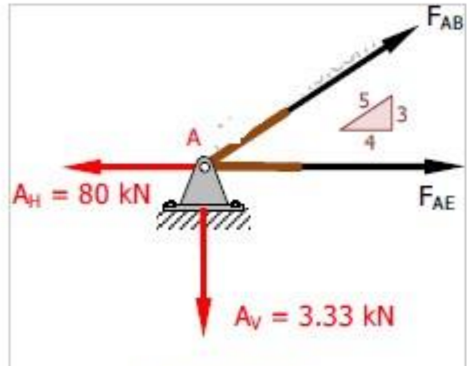
$F_{AB} = 5.56 \text{ kN tension}$

$\Sigma F_H = 0$

$F_{AE} + \frac{4}{5} F_{AB} = 80$

$F_{AE} + \frac{4}{5}(5.56) = 80$

$F_{AE} = 75.56 \text{ kN tension}$



FBD of Joint A

At joint B

$\Sigma F_H = 0$

$F_{BC} = \frac{4}{5} F_{AB}$

$F_{BC} = \frac{4}{5}(5.56)$

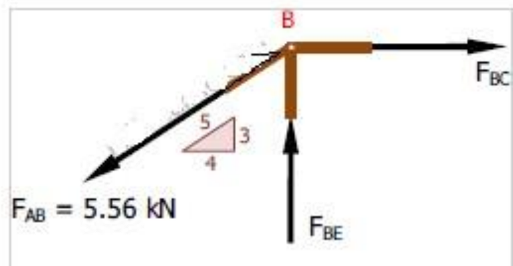
$F_{BC} = 4.45 \text{ kN tension}$

$\Sigma F_V = 0$

$F_{BE} = \frac{3}{5} F_{AB}$

$F_{BE} = \frac{3}{5}(5.56)$

$F_{BE} = 3.34 \text{ kN compression}$



FBD of Joint B

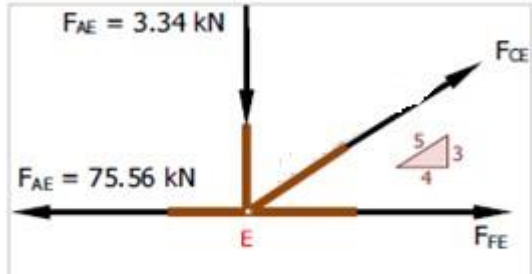
At joint E

$\Sigma F_V = 0$

$\frac{3}{5}F_{CE} = F_{AE}$

$\frac{3}{5}F_{CE} = 3.34$

$F_{CE} = 5.57\ \text{kN tension}$



FBD of Joint E

$\Sigma F_H = 0$

$F_{FE} + \frac{4}{5}F_{CE} = F_{AE}$

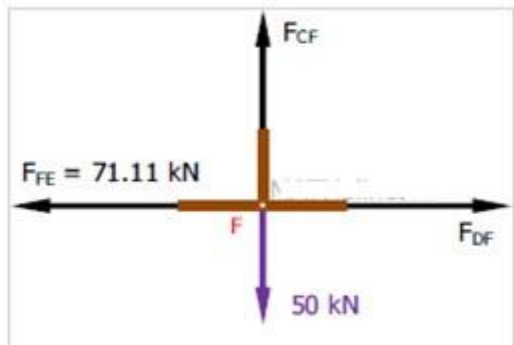
$F_{FE} + \frac{4}{5}(5.57) = 75.56$

$F_{FE} = 71.11\ \text{kN tension}$

At joint F

$\Sigma F_V = 0$

$F_{CF} = 50\ \text{kN tension}$



FBD of Joint F

$\Sigma F_H = 0$

$F_{DF} = F_{FE}$

$F_{DF} = 71.11\ \text{kN tension}$

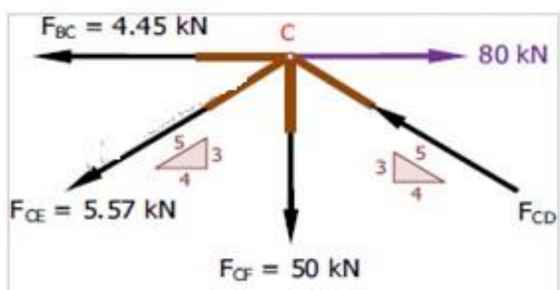
At joint C

$\Sigma F_H = 0$

$\frac{4}{5}F_{CD} + \frac{4}{5}F_{CE} + F_{BC} = 80$

$\frac{4}{5}F_{CD} + \frac{4}{5}(5.57) + 4.45 = 80$

$F_{CD} = 88.87\ \text{kN compression}$



FBD of Joint C

$\Sigma F_V = 0$

$\frac{3}{5}F_{CD} = \frac{3}{5}F_{CE} + F_{CF}$

$\frac{3}{5}(88.87) = \frac{3}{5}(5.57) + 50$

$53.3 = 53.3 \qquad \textit{check}$

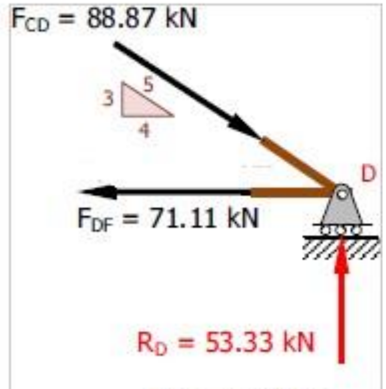
At joint D

$\Sigma F_H = 0$

$\frac{4}{5} F_{CD} = F_{DF}$

$\frac{4}{5}(88.87) = 71.11$

$71.1 = 71.1$ *check*



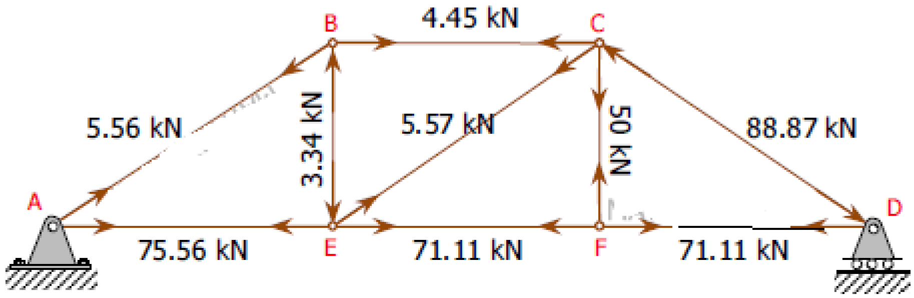
FBD of Joint D

$\Sigma F_V = 0$

$R_D = \frac{3}{5} F_{CD}$

$53.33 = \frac{3}{5}(88.87)$

$53.3 = 53.3$ *check*



$F_{AB} = 5.56$ kN tension

$F_{AE} = 75.56$ kN tension

$F_{BC} = 4.45$ kN tension

$F_{BE} = 3.34$ kN compression

$F_{CD} = 88.87$ kN compression

$F_{CE} = 5.57$ kN tension

$F_{CF} = 50$ kN tension

$F_{DF} = 71.11$ kN tension

$F_{FE} = 71.11$ kN tension

PROBLEM 2

The structure in Fig. T-02 is a truss which is pinned to the floor at point A, and supported by a roller at point D. Determine the force to all members of the truss.

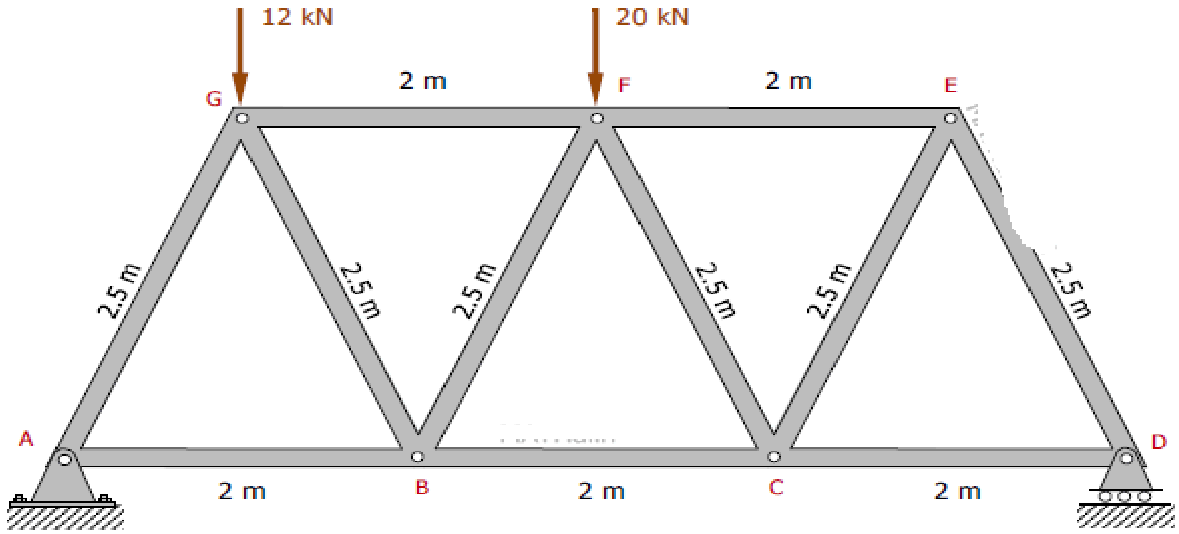


Figure T-02

$\Sigma M_A = 0$

$6R_D = 1(12) + 3(20)$

$R_D = 12 \text{ kN}$

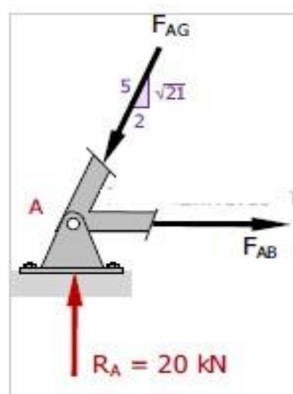
At joint A

$\Sigma F_V = 0$

$\frac{\sqrt{21}}{5} F_{AG} = R_A$

$\frac{\sqrt{21}}{5} F_{AG} = 20$

$F_{AG} = 21.82 \text{ kN compression}$



FBD of Joint A

$\Sigma F_H = 0$

$F_{AB} = \frac{2}{5} F_{AG}$

$F_{AB} = \frac{2}{5}(21.82)$

$F_{AB} = 8.73 \text{ kN tension}$

At joint G

$\Sigma F_V = 0$

$\frac{\sqrt{21}}{5} F_{BG} + 12 = \frac{\sqrt{21}}{5} F_{AG}$

$\frac{\sqrt{21}}{5} F_{BG} + 12 = \frac{\sqrt{21}}{5} (21.82)$

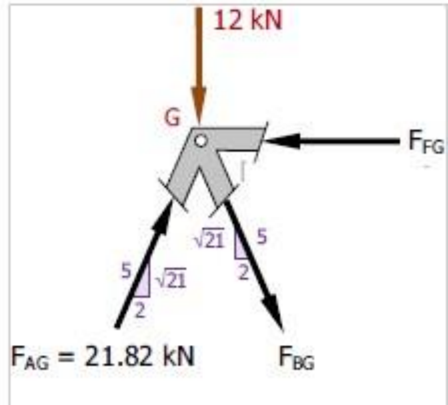
$F_{BG} = 8.73 \text{ kN tension}$

$\Sigma F_H = 0$

$F_{FG} = \frac{2}{5} F_{AG} + \frac{2}{5} F_{BG}$

$F_{FG} = \frac{2}{5} (21.82) + \frac{2}{5} (8.73)$

$F_{FG} = 12.22 \text{ kN compression}$



FBD of Joint G

At joint B

$\Sigma F_V = 0$

$\frac{\sqrt{21}}{5} F_{BF} = \frac{\sqrt{21}}{5} F_{BG}$

$F_{BF} = F_{BG}$

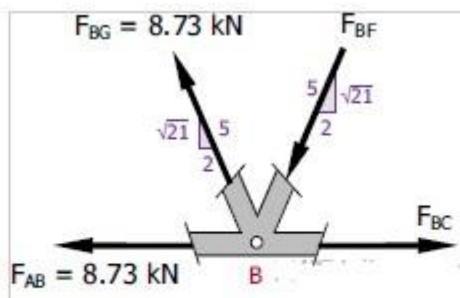
$F_{BF} = 8.73 \text{ kN compression}$

$\Sigma F_H = 0$

$F_{BC} = F_{AB} + \frac{2}{5} F_{BG} + \frac{2}{5} F_{BF}$

$F_{BC} = 8.73 + \frac{2}{5} (8.73) + \frac{2}{5} (8.73)$

$F_{BC} = 15.71 \backslash\text{tension} kNtension$



FBD of Joint B

At joint F

$\Sigma F_V = 0$

$\frac{\sqrt{21}}{5} F_{CF} + \frac{\sqrt{21}}{5} F_{BF} = 20$

$\frac{\sqrt{21}}{5} F_{CF} + \frac{\sqrt{21}}{5} (8.73) = 20$

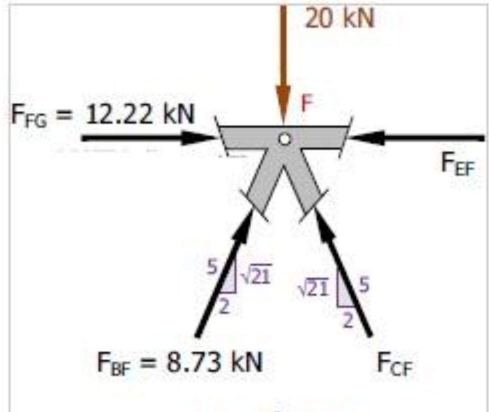
$F_{CF} = 13.09 \text{ kN compression}$

$\Sigma F_H = 0$

$F_{EF} + \frac{2}{5} F_{CF} = \frac{2}{5} F_{BF} + F_{FG}$

$F_{EF} + \frac{2}{5}(13.09) = \frac{2}{5}(8.73) + 12.22$

$F_{EF} = 10.48 \text{ kN compression}$



FBD of Joint F

At joint C

$\Sigma F_V = 0$

$\frac{\sqrt{21}}{5} F_{CE} = \frac{\sqrt{21}}{5} F_{CF}$

$F_{CE} = F_{CF}$

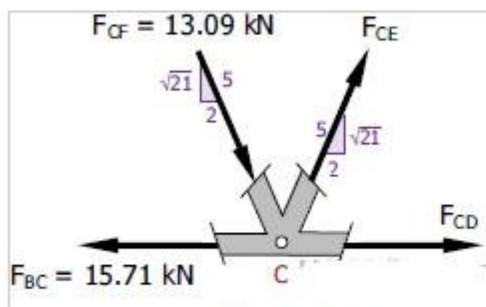
$F_{CE} = 13.09 \text{ kN tension}$

$\Sigma F_H = 0$

$F_{CD} + \frac{2}{5} F_{CE} + \frac{2}{5} F_{CF} = F_{BC}$

$F_{CD} + \frac{2}{5}(13.09) + \frac{2}{5}(13.09) = 15.71$

$F_{CD} = 5.24 \text{ kN tension}$



FBD of Joint C

At joint E

$\Sigma F_V = 0$

$\frac{\sqrt{21}}{5} F_{DE} = \frac{\sqrt{21}}{5} F_{CE}$

$F_{DE} = F_{CE}$

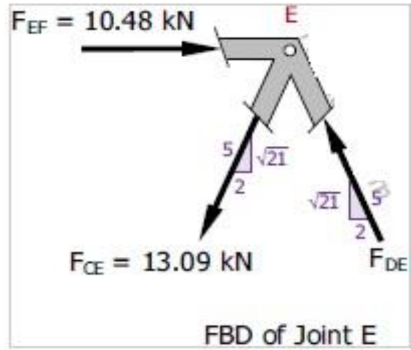
$F_{DE} = 13.09 \text{ kN compression}$

$\Sigma F_H = 0$

$F_{EF} = \frac{2}{5} F_{CE} + \frac{2}{5} F_{DE}$

$10.48 = \frac{2}{5}(13.09) + \frac{2}{5}(13.09)$

$10.5 = 10.5$ *check*



At joint D

$\Sigma F_V = 0$

$R_D = \frac{\sqrt{21}}{5} F_{DE}$

$12 = \frac{\sqrt{21}}{5}(13.09)$

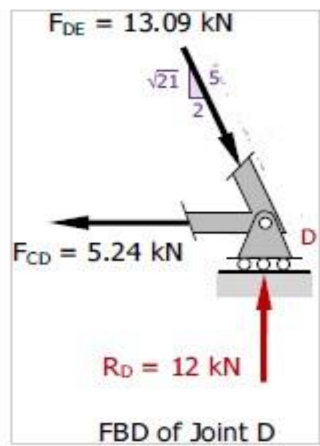
$12 = 12$ *check*

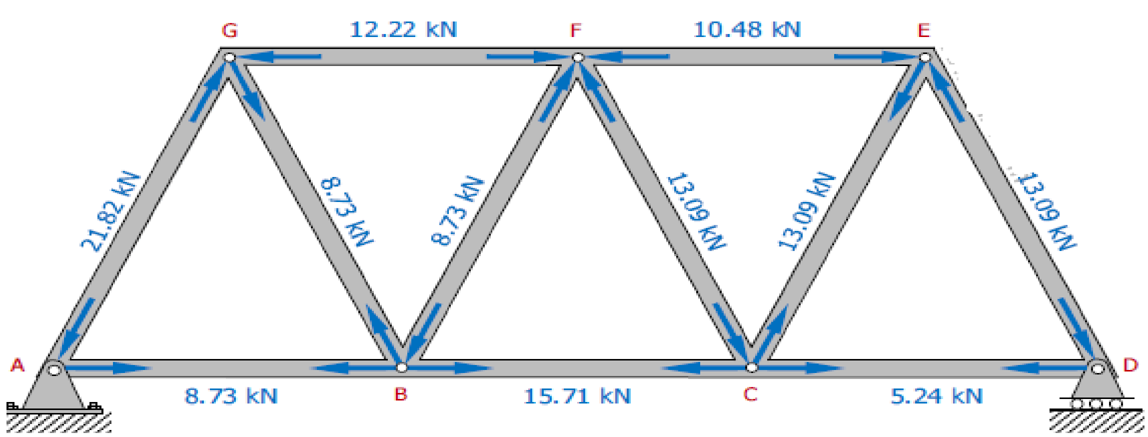
$\Sigma F_H = 0$

$F_{CD} = \frac{2}{5} F_{DE}$

$5.24 = \frac{2}{5}(13.09)$

$5.24 = 5.24$ *check*

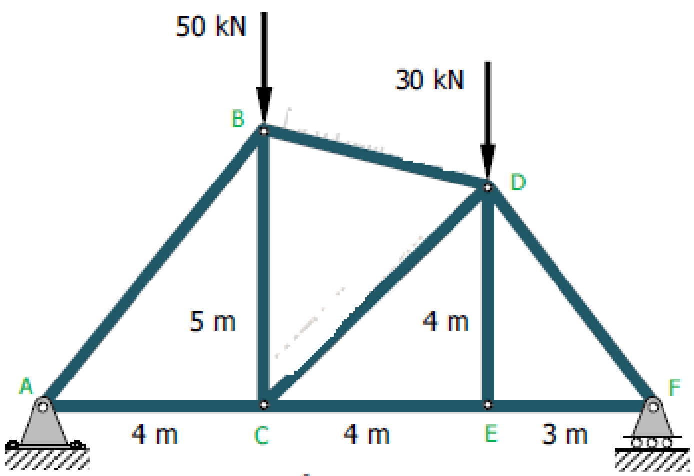




F_{AB} = 8.73 kN tension
F_{AG} = 21.82 kN compression
F_{BC} = 15.71 kN tension
F_{BF} = 8.73 kN compression
F_{BG} = 8.73 kN tension
F_{CD} = 5.24 kN tension
F_{CE} = 13.09 kN tension
F_{CF} = 13.09 kN compression
F_{DE} = 13.09 kN compression
F_{EF} = 10.48 kN compression
F_{FG} = 12.22 kN compression

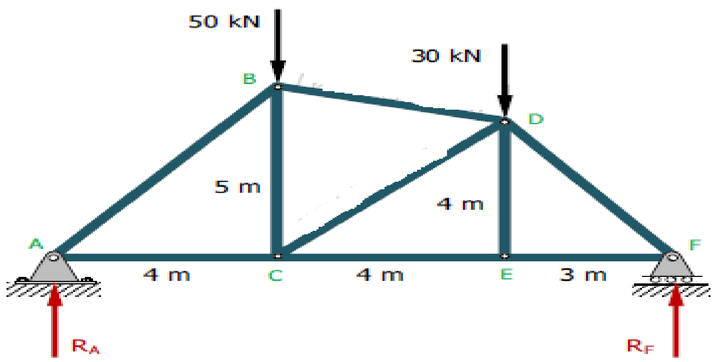
PROBLEM : 3

Compute the force in all members of the truss shown in Fig.



Figure

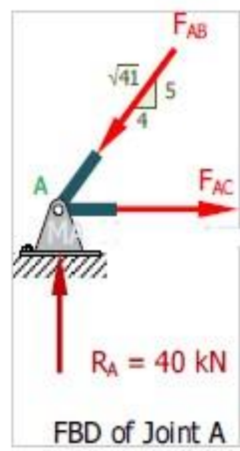
$$\Sigma M_F = 0$$
$$11R_A = 7(50) + 3(30)$$
$$R_A = 40\ \text{kN}$$



$$\Sigma M_A = 0$$
$$11R_F = 4(50) + 8(30)$$
$$R_F = 40\ \text{kN}$$

At joint A

$$\Sigma F_V = 0$$
$$\frac{5}{\sqrt{41}} F_{AB} = R_A$$
$$\frac{5}{\sqrt{41}} F_{AB} = 40$$
$$F_{AB} = 51.22\ \text{kN compression}$$



$$\Sigma F_H = 0$$
$$F_{AC} = \frac{4}{\sqrt{41}} F_{AB}$$
$$F_{AC} = \frac{4}{\sqrt{41}}(51.22)$$
$$F_{AC} = 32\ \text{kN tension}$$

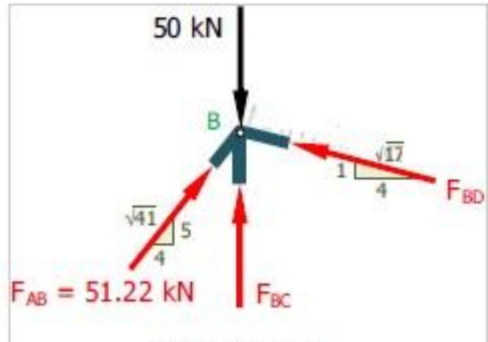
At joint B

$\Sigma F_H = 0$

$\frac{4}{\sqrt{17}} F_{BD} = \frac{4}{\sqrt{41}} F_{AB}$

$\frac{4}{\sqrt{17}} F_{BD} = \frac{4}{\sqrt{41}} (51.22)$

$F_{BD} = 32.98 \text{ kN compression}$



FBD of Joint B

$\Sigma F_V = 0$

$F_{BC} + \frac{5}{\sqrt{41}} F_{AB} + \frac{1}{\sqrt{17}} F_{BD} = 50$

$F_{BC} + \frac{5}{\sqrt{41}} (51.22) + \frac{1}{\sqrt{17}} (32.98) = 50$

$F_{BC} = 2 \text{ kN compression}$

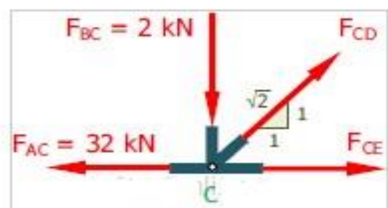
At joint C

$\Sigma F_V = 0$

$\frac{1}{\sqrt{2}} F_{CD} = F_{BC}$

$\frac{1}{\sqrt{2}} F_{CD} = 2$

$F_{CD} = 2.83 \text{ kN tension}$



FBD of Joint C

$\Sigma F_H = 0$

$F_{CE} + \frac{1}{\sqrt{2}} F_{CD} = F_{AC}$

$F_{CE} + \frac{1}{\sqrt{2}} (2.83) = 32$

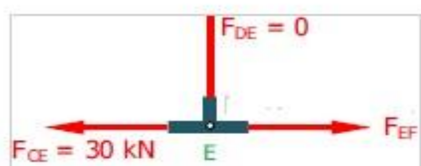
$F_{CE} = 30 \text{ kN tension}$

At joint E

$\Sigma F_H = 0$

$F_{EF} = F_{CE}$

$F_{EF} = 30 \text{ kN tension}$



FBD of Joint E

$\Sigma F_V = 0$

$F_{DE} = 0$

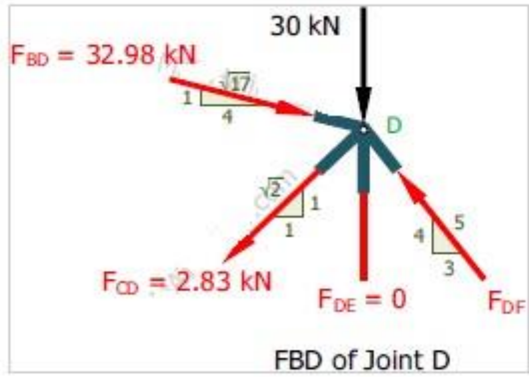
At joint D

$\Sigma F_H = 0$

$\frac{3}{5}F_{DF} + \frac{1}{\sqrt{2}}F_{CD} = \frac{4}{\sqrt{17}}F_{BD}$

$\frac{3}{5}F_{DF} + \frac{1}{\sqrt{2}}(2.83) = \frac{4}{\sqrt{17}}(32.98)$

$F_{DF} = 50 \text{ kN compression}$



$\Sigma F_V = 0$

$\frac{4}{5}F_{DF} = \frac{1}{\sqrt{17}}F_{BD} + \frac{1}{\sqrt{2}}F_{CD} + 30$

$\frac{4}{5}(50) = \frac{1}{\sqrt{17}}(32.98) + \frac{1}{\sqrt{2}}(2.83) + 30$

$40 = 40$ *check*

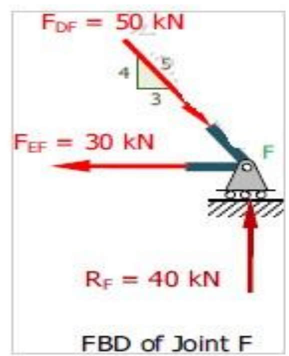
At joint F

$\Sigma F_V = 0$

$\frac{4}{5}F_{DF} = R_A$

$\frac{4}{5}(50) = 40$

$40 = 40$ *check*



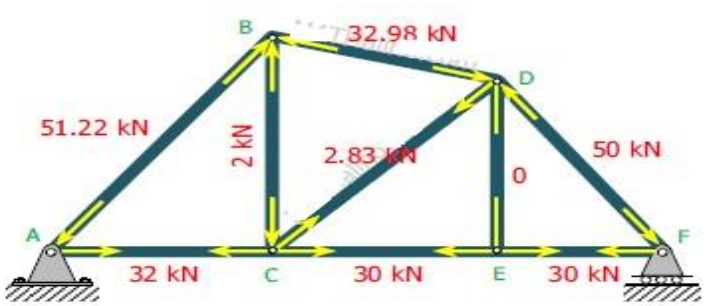
$\Sigma F_H = 0$

$F_{EF} = \frac{3}{5}F_{DF}$

$F_{EF} = \frac{3}{5}(50)$

$30 = 30$ *check*

Summary



Method of Sections

In this method, we will cut the truss into two sections by passing a cutting plane through the members whose internal forces we wish to determine. This method permits us to solve directly any member by analyzing the left or the right section of the cutting plane. To remain each section in equilibrium, the cut members will be replaced by forces equivalent to the internal load transmitted to the members. Each section may constitute of non-concurrent force system from which three equilibrium equations can be written.

$$\Sigma FH = 0,\ \Sigma FV = 0,\quad \text{and}\quad \Sigma MO = 0$$

Because we can only solve up to three unknowns, it is important not to cut more than three members of the truss. Depending on the type of truss and which members to solve, one may have to repeat Method of Sections more than once to determine all the desired forces.

PROBLEM 1

From the truss in Fig. T-01, determine the force in mebers BC, CE, and EF.

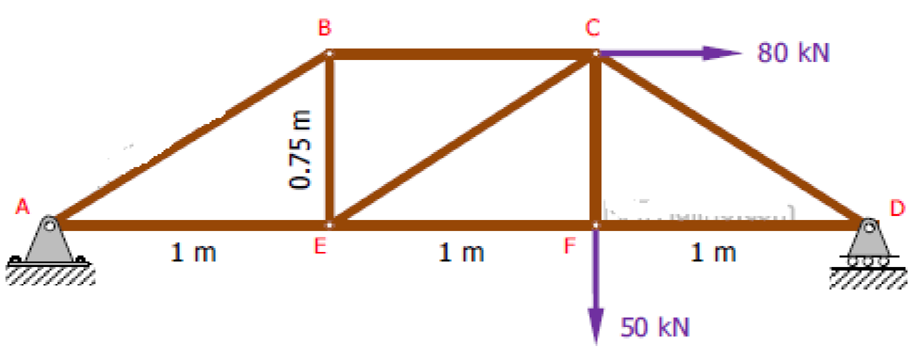
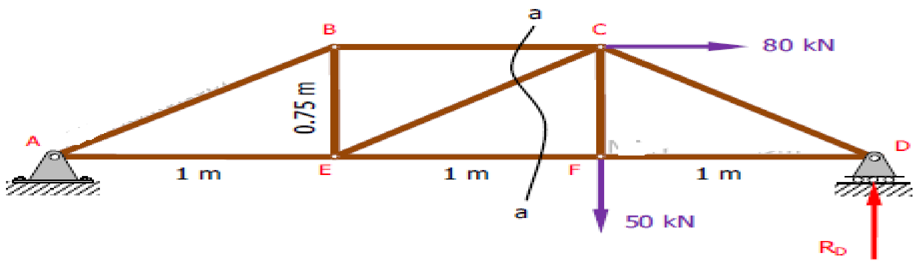


Figure T-01



$\Sigma M_A = 0$

$3R_D = 50(2) + 80(0.75)$

$R_D = 53.33 \text{ kN}$

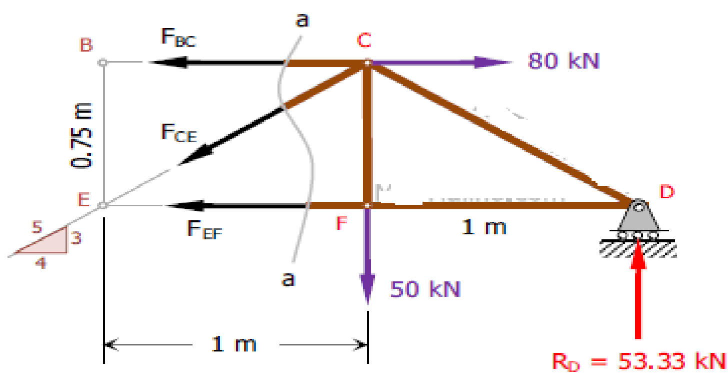
From the FBD of the section through a-a

$\Sigma M_E = 0$

$0.75F_{BC} + 2R_D = 0.75(80) + 1(50)$

$0.75F_{BC} + 2(53.33) = 60 + 50$

$F_{BC} = 4.45 \text{ kN tension}$



Section to the right of a-a

$\Sigma M_C = 0$

$0.75F_{EF} = 1(R_D)$

$0.75F_{EF} = 53.33$

$F_{EF} = 71.11 \text{ kN tension}$

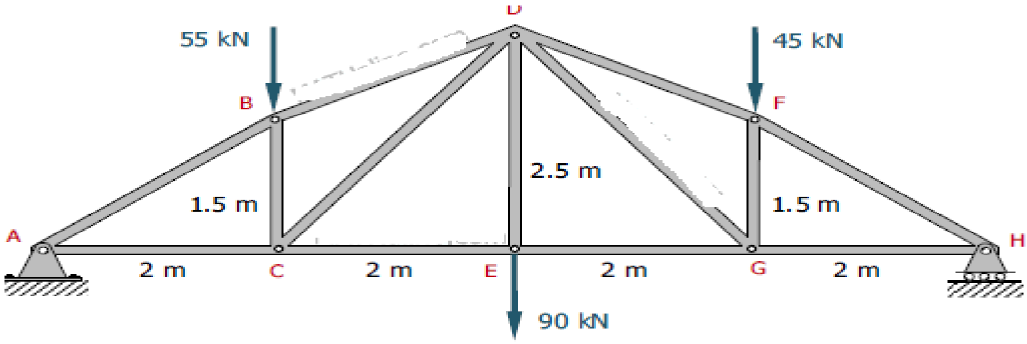
$\Sigma F_V = 0$

$\frac{3}{5}F_{CE} + 50 = R_D$

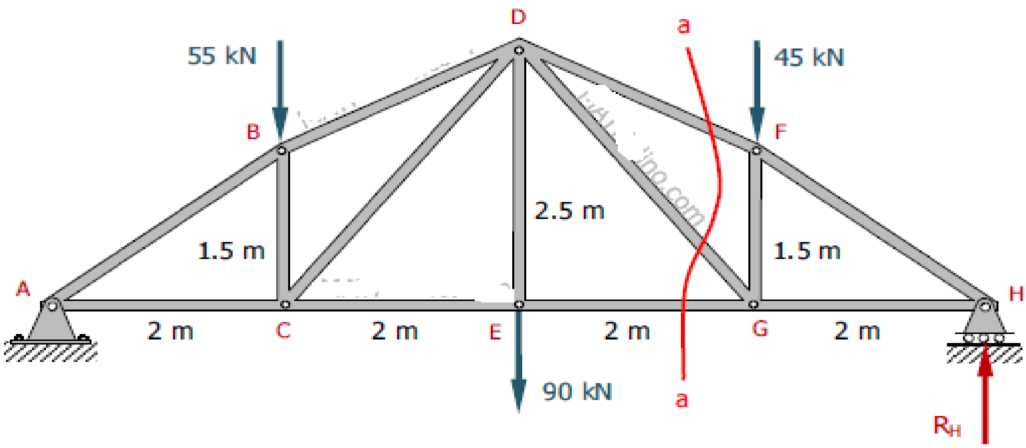
$\frac{3}{5}F_{CE} + 50 = 53.33$

$F_{CE} = 5.55 \text{ kN tension}$

PROBLEM 2



The roof truss shown in Fig is pinned at point A, and supported by a roller at point H. Determine the force in member DG.



$$\Sigma M_A = 0$$

$$8R_H = 2(55) + 4(90) + 6(45)$$

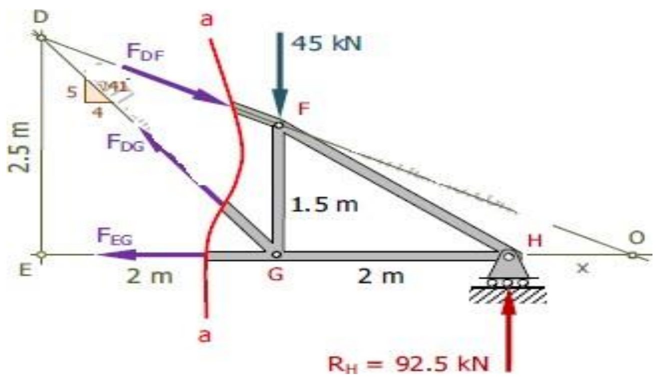
$$R_H = 92.5 \ \text{kN}$$

From section to the right of a-a

$$\frac{x+2}{1.5} = \frac{x+4}{2.5}$$

$$2.5x + 5 = 1.5x + 6$$

$$x = 1 \ \text{m}$$



$$\Sigma M_O = 0$$
$$(x+2)\left(\frac{5}{\sqrt{41}}F_{DG}\right) + xR_H = (x+2)(45)$$
$$(1+2)\left(\frac{5}{\sqrt{41}}F_{DG}\right) + 1(92.5) = (1+2)(45)$$
$$\frac{15}{\sqrt{41}}F_{DG} + 92.5 = 135$$
$$\frac{15}{\sqrt{41}}F_{DG} = 42.5$$
$$F_{DG} = 18.14 \text{ kN tension}$$

DEFLECTION IN TRUSS

Virtual work method

The method of virtual work in trusses is used to determine the deflection of a joint in a truss. In order to determine this deflection, a real and virtual system are developed. In this section we will look at finding vertical or horizontal displacements in determinate trusses caused by external forces.

Principle of virtual work

Virtual work is a method where a "virtual" load (an imaginary load that does not exist as a part of the real loading is applied to a structure at a point where the value of the slope or deflection is desired. This applied virtual load creates a virtual system which, through the principles of work and energy, is related to the real system. The relationship can then be used to solve for the deflection or slope at any point of the real system.

Truss Deflection - Basic Equation

The basic equation for determining the deflection in a truss is as follows:

$$1 * \Delta = \sum Fv * \delta$$

Where: 1 is a virtual, unit force; Δ is an unknown, real displacement; Fv is a set of all virtual internal forces arising from and in equilibrium with 1; δ is a set of internal displacements arising from Fv. The internal displacement δ can be expanded to $\delta = FL/AE$, where F is the real external applied force; L the length of the truss member; A is the cross-sectional area of the member; and E is the modulus of elasticity.

PROCEDURE FOR DETERMINING TRUSS DEFLECTION

1. Determine Whether the Truss is Determinate, Indeterminate, or Unstable

Determine whether the truss is determinate, indeterminate, or unstable using the equations and methods described in the Determinacy, Indeterminacy and Stability section.

2. Find the Support Reactions for the Truss

The support reactions for the truss need to be found first by analyzing the truss with the three equations of equilibrium.

3. Find Real Support and Internal Forces, F

Calculate the real internal forces, F in each truss member caused by the real external loads placed on the truss. These internal forces can be determined by the Method of Joints.

4. Find Virtual Forces, Fv

Remove all real, external loads on the truss. Place the virtual unit load, 1, in the same direction as the displacement to be determined. If the vertical displacement is to be determined at a joint a, apply the virtual load in the vertical direction at joint a. Generally the unit load 1 is in the same units as the external load applied. In the case of a 5 KN external load, the virtual unit load would be 1 KN. Calculate the internal forces in each truss member with the 1 unit force applied. These forces are the virtual forces, Fv.

5. Find the length of each truss member

Calculate the length of each truss member. As the length and height of a truss are generally given the individual member lengths can be solved with basic trigonometry.

6. Find the cross-sectional area, A and the modulus of elasticity, E.

These properties of the truss are generally given. Make sure to pay attention to the units of this given values.

7. Find the real displacement, Δ

Create a table with calculated values for F, Fv, L, and (Fv*F*L)/(A*E). If A and E are uniform throughout the truss they can be removed and multiplied with the final summation. [2]

Virtual Work Table

member	member length, L	real internal force, F	virtual internal force, Fv	FvFL
AB				
BC				
CA				
				∑FvFL

Finally, for uniform A and E, the displacement of the truss can be calculated:

$$\Delta = \frac{1}{1} * \frac{1}{AE} * \sum F_v F L$$

PROCEDURE FOR DETERMINING DEFORMATIONS DUE TO TEMPERATURE AND DEFECTS

Deformations Due to Temperature Change

For temperature change the axial deformation is expressed as

$$\delta = \alpha * \Delta T * L$$

Where α is coefficient of thermal expansion, ΔT is the change in temperature, and L is the length of the truss member.

To get the truss deformation Δ, the axial deformation, δ is subbed into the equation

$$1\Delta = \sum F_v \delta$$ resulting in $$\Delta = \frac{1}{1} * \sum F_v \alpha \Delta T L$$

Deformations Due to Manufacturing Defects

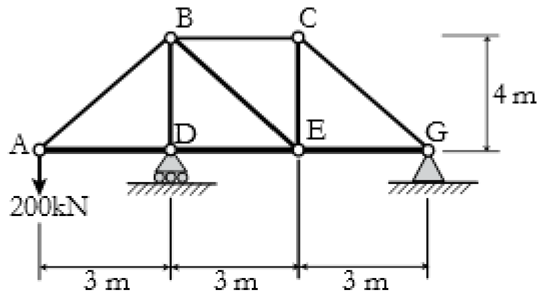
For finding deformations due to manufacturing defects, the axial deformation, δ (negative or positive) is known.

The equation for truss deformation simply becomes

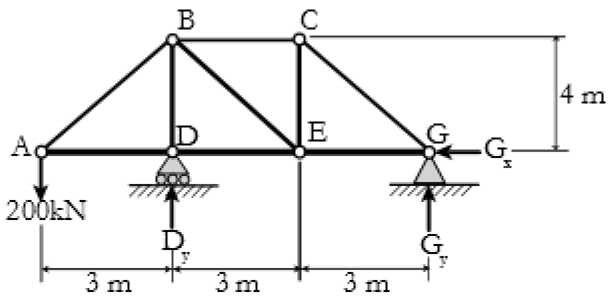
$$\Delta = \frac{1}{1} * \sum F_v * \delta$$

PROBLEM

Determine horizontal displacement at Point B, assuming all members have constant cross-sectional area *A*=300mm2 and modulus of elasticity *E*=250GPa.



We begin our solution by noting that this truss is a statically determinate structure and we proceed by drawing the free body diagram and solving for the support reactions using the equations of static equilibrium.



solve for Supports in Real System

∑Fx=0⇒Gx=0kN

∑MD=0⇒Gy=100kN

∑Fy=0⇒Dy=300kN

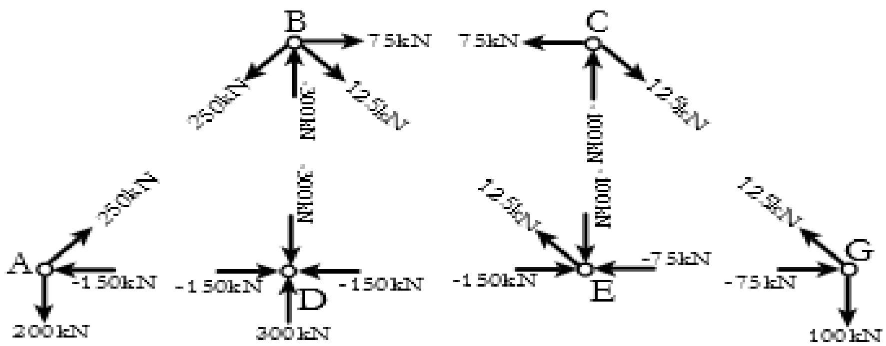
Find Internal Forces in Real System

Having obtained the support reactions, we proceed with a joint analysis of the truss to determine forces acting on each member.

Joint A

∑Fy=0⇒−200kN+45FAB=0⇒FAB=250kN [T]

∑Fx=0⇒FAD=−35FAB⇒FAD=−150kN [C]



Joint D

$\sum Fx=0 \Rightarrow FDE=FAD \Rightarrow FDE=-150\text{kN}$ [C]

$\sum Fy=0 \Rightarrow FBD=-300\text{kN}$ [C]

Joint G

$\sum F_y=0 \Rightarrow -100\text{kN}+45F_{CG}=0 \Rightarrow F_{CG}=125\text{kN}$ [T]

$\sum F_x=0 \Rightarrow F_{EG}=-35F_{CG} \Rightarrow F_{EG}=-75\text{kN}$ [C]

Joint B

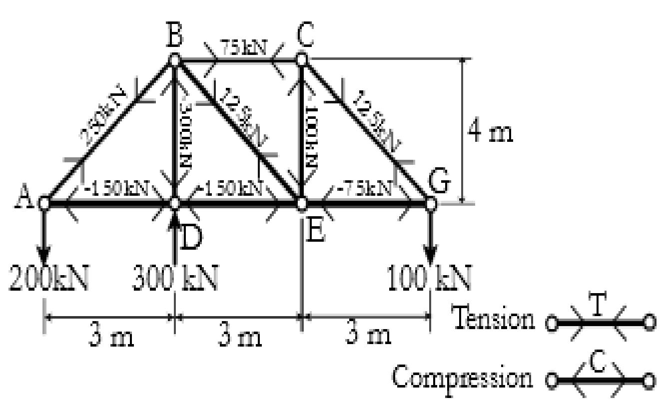
$\sum Fy=0 \Rightarrow -45FBE+FBD+-45FAB=0 \Rightarrow FBE=125\text{kN}$ [T]

$\sum Fx=0 \Rightarrow FBC+-35FAB+35FBE=0 \Rightarrow FBC=75\text{kN}$ [T]

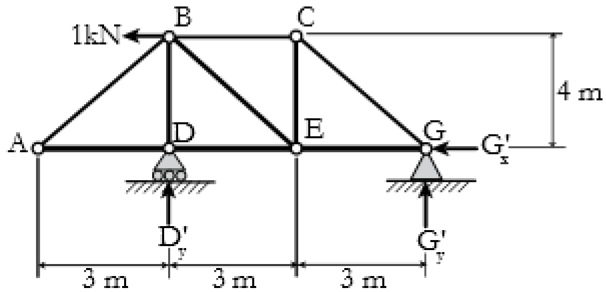
Joint C

$\sum Fy=0 \Rightarrow FCE=-45FCG \Rightarrow FCE=-100\text{kN}$ [C]

After obtaining both the support reactions and all the member forces of the real truss the free body diagram of the real truss can be drawn with the forces included.



Construct Virtual System



Since we are required to find the horizontal displacement at Point B, we apply a leftward, horizontal 1kN virtual load to point B which can be seen in the free body diagram of the virtual system above.

Solve for Supports in Virtual System

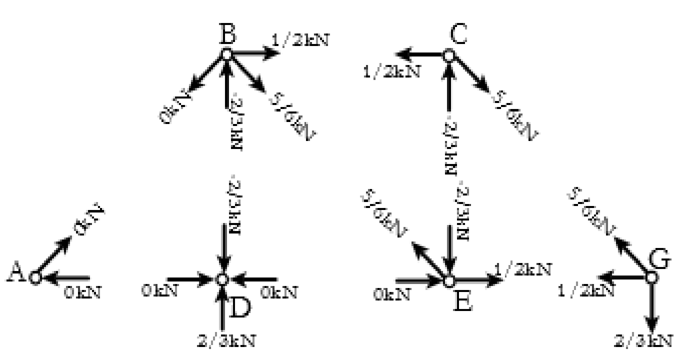
We find the virtual reactions in the same way as we found the real reactions; for brevity we just state the results.

D′y=23kN [up]

G′x=1kN [right]

G′y=23kN [down]

We note the existence of zero-force members *FAB*, *FAD*, and *FDE* and simplify our joint analysis accordingly. Again, the intermediate steps have been omitted.



Joint G

$F'_{CG}=56$kN [T]

$F'_{EG}=12$kN [T]

Joint D

$F'_{BD}=-23$kN [C]

Joint B

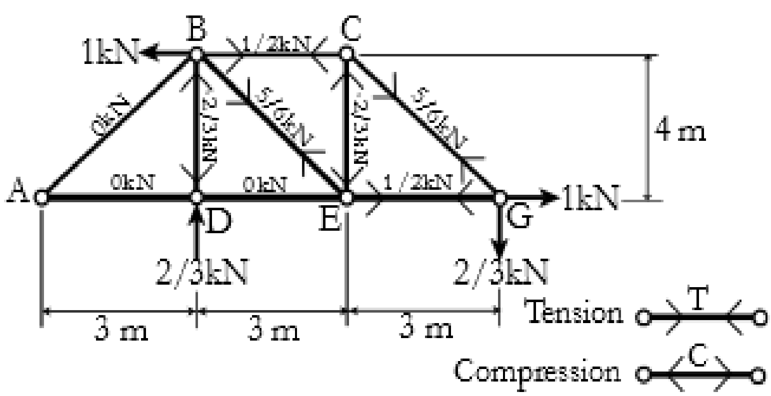
$F'_{BE}=56$kN [T]

$F'_{BC}=12$kN [T]

Joint C

$F'_{CE}=-23$kN [C]

After obtaining both the support reactions and all the member forces of the virtual system the free body diagram of the virtual system can be drawn with the forces included.



Virtual Work Table

member	L	F	F_v	F_vFL
AB	5	250	0	0
BC	3	75	1/2	112.5
CG	5	125	5/6	520.83
BD	4	-300	-2/3	800

CE	4	-100	-2/3	266.67
BE	5	125	5/6	520.83
AD	3	-150	0	0
DE	3	-150	0	0
EG	3	-75	1/2	-112.5
			$\sum F_v F L$	2108.33kN2·m

Δ=28mm [left]

Thus the expected horizontal deflection at B will be 28 mm.

JOINERY DETAILS OF DIFFERENT MEMBERS OF TIMBER TRUSS

The element that most defines a timber framer is the quality, integrity and strength of the joints that hold the beams together. A properly made joint will stay tight for generations. All wooden beams will shrink and check as they dry, as this is part of the unique nature of timber framed buildings, but the joinery should account for these natural ways timber moves and remain in place and stay solid over time.

Joinery options range from traditional, all-wood joints to timber connections with steel plates and bolts. Our traditional joints are pinned together using 1-inch hardwood pegs, with the geometry of the joint itself carrying the structural load and the pegs holding the joint in place. When the spans are greater than 16 feet, steel is often required in order to meet building codes, but can be hidden or exposed depending on the owner’s preferences. When loads are extraordinarily large or when aesthetics call for it, we use heavy steel plates, rods & bolts. All of our joints undergo in-house engineering to confirm their strength and integrity.

SATHYABAMA
INSTITUTE OF SCIENCE AND TECHNOLOGY
(DEEMED TO BE UNIVERSITY)
Accredited "A" Grade by NAAC | 12B Status by UGC | Approved by AICTE
www.sathyabama.ac.in

SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

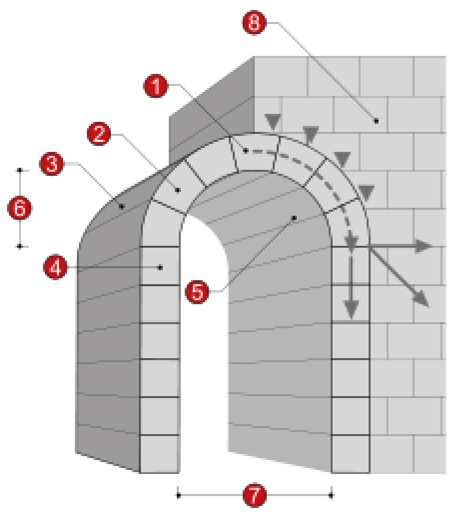
UNIT – III - BUILDING STRUCTURES II– SCIA1306

UNIT III STRUCTURAL ELEMENTS

Arches: Introduction to arches, types of arches –Theory of arches – BM diagram for arches- Design of masonry arches using empirical equations- Drawings:types of arches. Floors and flooring :Principles of flooring and terracing- Types of floors: Bricks, stone, concrete colours with finishes – wooden floors: Study of relevant IS codes.

INTRODUCTION TO ARCHES

An **arch** is a curved structure that spans a space and may or may not support weight above it. Arch may be synonymous with vault, but a vault may be distinguished as a continuous arch forming a roof. Arches appeared as early as the 2nd millennium BC in Mesopotamian brick architecture, and their systematic use started with the Ancient Romans who were the first to apply the technique to a wide range of structures.



A masonry Arch.

1. Keystone
2. Voussoir
3. Extrados
4. Impost
5. Intrados
6. Rise
7. Clear span
8. Abutment

TYPES OF ARCHES:

Arches have many forms, but all fall into three basic categories: circular, pointed, and parabolic. Arches can also be configured to produce vaults and arcades.

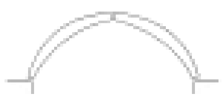
Arches with a circular form, also referred to as rounded arches, were commonly employed by the builders of ancient, heavy masonry arches. Ancient Roman builders relied heavily on the rounded arch to span large, open areas. Several rounded arches placed in-line, end-to-end, form an arcade, such as the Roman aqueduct.

Vaults are essentially "adjacent arches [that] are assembled side by side." If vaults intersect, complex forms are produced with the intersections. The forms, along with the "strongly expressed ribs at the vault intersections, were dominant architectural features of Gothic cathedrals.

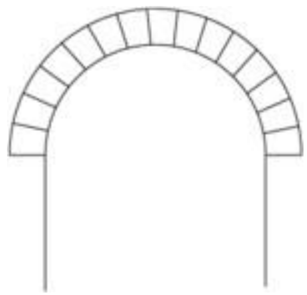
The parabolic arch employs the principle that when weight is uniformly applied to an arch, the internal compression resulting from that weight will follow a parabolic profile. Of all arch types, the parabolic arch produces the most thrust at the base, but can span the largest areas. It is commonly used in bridge design, where long spans are needed.

The catenary arch has a shape different from the parabolic curve. The shape of the curve traced by a loose span of chain or rope, the catenary is the structurally ideal shape for a freestanding arch of constant thickness.

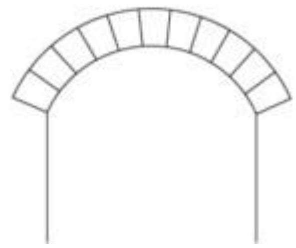
Types of arches displayed chronologically, roughly in the order in which they were developed:



Triangular arch

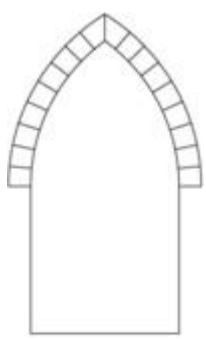


Round arch or Semi-circular arch

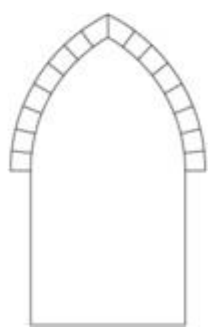


Segmental arch

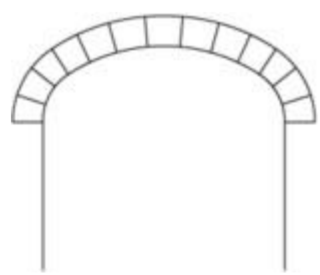
Unequal round arch or Rampant round arch



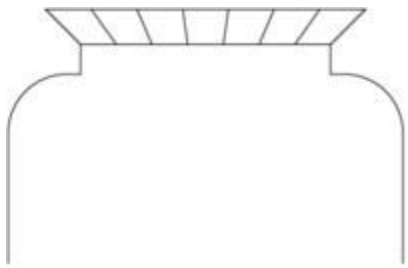
Lancet arch



Equilateral pointed arch

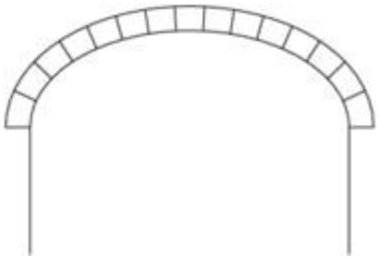


Three centered arch

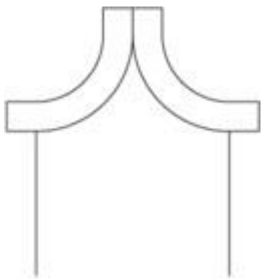


Shouldered flat arch

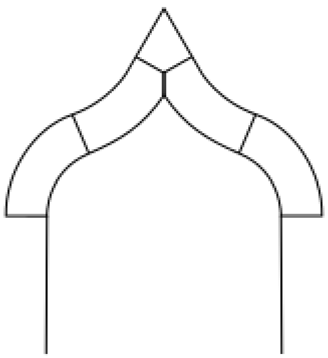
Trefoil-arch or three foiled cusped arch



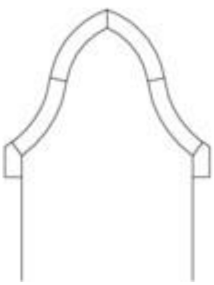
Elliptical arch



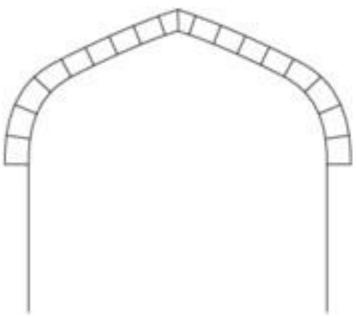
Inflexed arch



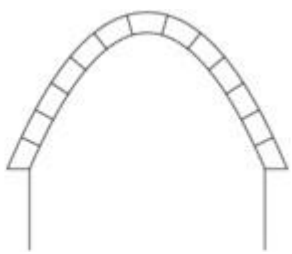
Ogee arch



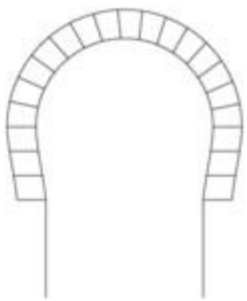
Reverse Ogee arch



Tudor arch



Parabolic arch



Horse shoe arch

THEORY OF ARCHES:

An arch may be looked upon as a curved girder, either a solid rib or braced, supported at its ends and carrying transverse loads which are frequently vertical. An arch is subjected to three restraining forces. They are thrust , shear force and bending moment. Depending upon the number of hinges, arches may be divided in to four classes: 1. Three hinged arch 2. Two hinged arch 3. Single hinged arch 4.Fixed arch(hingeless arch)

A three hinged arch is a statically determinate structure while the rest three arches are statically indeterminate. In bridge construction, especially in railroad bridges, the more frequently used arches are the two-hinged and the fixed end ones.

EDDY'S THEORAM

Eddy's theorem states that "The bending moment at any section of an arch is proportional to the vertical intercept between the linear arch (or theoretical arch) and the centre line of the actual arch."

Principles of flooring:

Floor construction is the most serious activity in the building process serving as the main link between sub-structure and super-structure. Apart from the foundation, the floor of the building serves as the most immediate support for super-structure. The loads of the walls and columns and imposed loads are first transmitted to the floor before reaching the foundation concrete. The inability of a floor to support the above loads and onward transmission will indicate the ineffective nature of the whole building. To ensure that floors are effectively constructed and serve the desired purpose, the knowledge of appropriate principles and practice is very necessary.

The principles and practice of floor construction are as follows: **ENSURING STABILITY OF A FLOOR.** By principle, a good floor is expected to provide a stable link between the sub-structure and the super structure of a building. When the floor is able to sustain the loads and stress subjected to it, then the stability of the super-structure is assured. The failure of any floor to support and protect the walls, columns, beams, and others is indeed a total failure of the construction processes.

To make floors stable, the following are important:

- Using appropriate materials for the floor.
- Acquiring the desired type of aggregates ie sand, stone, binding material and water
- Adopting the correct proportions of the aggregates and binding material.
- Ensure uniform mixture of the concrete and its workability.
- Following the working procedures without any compromise.
- Compaction of the hardcore filling and providing cement grout blinding base.
- Provide key to the surface of the blinding base to ensure good adhesion
- The floor as a principle should support imposed and super- imposed loads.

Types of floor:

The different types of floors are as follows;

- Mud floor
- Brick floor
- Tile floor
- Flagstone floor
- Cement concrete floor
- Terazzo floor
- Screed floors
- Timber floors
- Marble floors etc.

Brick floor:

The floor whose topping is of brick. These are easy to construct and repair. but the surface resulting from these is not smooth and is rough, hence, easily absorbs and retains moisture which may cause dampness in the building.

Method of construction:

For constructing a brick floor, the top surface of earth or muram filling is properly consolidated. Over this compacted earth, a layer of clean sand about 10 cm thick is evenly spread. Then a layer of lime concrete (1:4:8) or lean cement concrete (1:4:16) is laid, compacted and cured. Over this base concrete well soaked bricks are laid in cement mortar (1:4) in any suitable bond. In case pointing is to be done, the minimum thickness of joints should not exceed 2 mm and and the mortar in joints is struck off with a trowel. When the pointing is to be done, the minimum thickness of joints is kept 6mm and the pointin may be done.

Suitability:

The floors are suitable for stores, godowns etc.

Flagstone floor:

The floors whose topping consists of stone slabs is called flagstone floor. The stone slabs used here may not be of the same size but should not be more than 75 cm length and not less than 35 cm in width and 3.8 cm in thickness.

Method of construction:

For constructing a flagstone floor, the same method is applied as in case of tile floor. The slabs are soaked well in water atleast one hour before laying. They should be evenly and firmly bedded in mortar. The thickness of joints should not exceed 4mm and they should be struck off with a trowel while laying.

Suitability:

These type of floorings are suitable in godowns, motor sheds, stores, pavements etc.

5. Cement concrete floor:

The floors whose topping consists of cement concrete is called cement concrete floor or conglomerate floor. These floors consists of 2.5 cm to 5cm thick concrete layer laid over 10 cm thick base concrete and 10 cm thick clean sand over ground whose compaction and consolidation is done. These floors are commonly used these days.

Advantages of cement concrete

1. They are hard & Durable
2. Provide a smooth & non absorbent surface
3. They are more fire resistant
4. They provide more sanitary surface as they can be cleaned & washed easily.
5. They are economical as they require negligible maintenance cost
6. They can be finished with a pleasing appearance.

COLOURED CONCRETE FINISHES

With colored concrete, the creative options and color choices are endless, making it possible to achieve the perfect look. Colored concrete can transform a room or patio from plain to spectacular. Many manufacturers offer a broad palette of colors to choose from, ranging from earth tones to vibrant hues . Also, colored concrete can be used to simulate the look of brick, flagstone, pavers, or tile. Not only is concrete coloring a beautiful design option, but it is also affordable and compatible with both new and existing concrete. With the right products, techniques, and a creative contractor you can produce results that will transform concrete into works of art.

Finishing (Texturing) Techniques for Colored Concrete: Colored concrete can be paired with other concrete finishing techniques to create a striking affect. These techniques include broom finishing, sandblasting, exposing of aggregate and more. Broom finishing can create shadow effects, "swirl" and "fan" patterns on colored concrete. Sandblasting can be used to create designs, or to give the concrete a two-tone appearance by removing a layer of color. Exposing of aggregate gives the concrete a natural textured look by letting the stone or gravel in the concrete show through.

Using Colored Concrete to Break up Large Areas Multiple colors can be used side-by side to break up large areas. Also, color can be paired with other decorative techniques such as stamping , sawcutting, brooming, or sandblasting to make the look even more personal. Another option for breaking up large areas of colored concrete is insetting materials such as granite, marble, tile or personal items.

Colored Concrete to Mimic Nature Color schemes are often chosen to blend with each other and blend with nature. Sometimes concrete is colored and finished to look like a gravel path.

Often it is necessary to have the permanence of concrete (and the durability). But where plain concrete's gray color would draw attention to a drainage ditch or path, colored concrete can be used to avoid drawing attention to these features and help those features blend with the other landscape elements.

Hiding Future Stains by Using Colored Concrete Most driveway, parking lots, and other parking areas are light gray concrete and soon become stained with oil and grease, tire marks, and dirt.

Many property owners will add a dark integral color to parking area concrete and then expose the aggregate in the concrete by sandblasting (medium to heavy) or by using a surface retarder. Roll curbs can be treated the same way. The lighter salt and pepper grays of the aggregate blend with the dark gray matrix of the cement paste.

Grease and oil stains will be much less apparent, lost in the different shades of gray. Tire marks also do not show as bad on exposed aggregate.

Wood flooring is a product manufactured from timber that is designed for use as flooring, either structural or aesthetic. Wood is a common choice as a flooring material due to its environmental profile, durability, and restorability. Bamboo flooring is often considered a form of wood flooring, although it is made from a grass (bamboo) rather than a timber.

Solid Hardwood flooring

Solid hardwood floors are made of planks milled from a single piece of timber. Solid hardwood floors were originally used for structural purposes, being installed perpendicular to the wooden support beams of a building known as joists or bearers. With the increased use of concrete as a subfloor in some parts of the world, engineered wood flooring has gained some popularity. However, solid wood floors are still common and popular. Solid wood floors have a thicker wear surface and can be sanded and finished more times than an engineered wood floor. It is not uncommon for homes in New England, Eastern Canada, and Europe which are several hundred years old to have the original solid wood floor still in use today.

Solid wood manufacturing

Solid wood flooring is milled from a single piece of timber that is kiln or air dried before sawing. Depending on the desired look of the floor, the timber can be cut in three ways: flat-sawn, quarter-sawn, and rift-sawn. The timber is cut to the desired dimensions and either packed unfinished for a site-finished installation or finished at the factory. The moisture content at time of manufacturing is carefully controlled to ensure the product doesn't warp during transport and storage.

There are a number of proprietary features for solid wood floors that are available. Many solid woods come with grooves cut into the back of the wood that run the length of each plank, often called 'absorption strips,' that are intended to reduce cupping. Solid wood floors are mostly manufactured .75 inches (19 mm) thick with a tongue-and-groove for installation.

Other wood manufacturing styles

Rotary-peel

This process involves treating the wood by boiling the log in water at a certain temperature for a certain amount of time. After preparation, the wood is peeled by a blade starting from the outside of the log and working toward the center, thus creating a wood veneer. The veneer is then pressed flat with high pressure. This style of manufacturing tends to have problems with the wood cupping or curling back to its original shape. Rotary-peeled engineered hardwoods tend to have a plywood appearance in the grain.

Sliced-peel

This process begins with the same treatment process that the rotary peel method uses. However, instead of being sliced in a rotary fashion, with this technique the wood is sliced from the log in much the same manner that lumber is sawn from a log – straight through. The veneers do not go through the same manufacturing process as rotary peeled veneers. Engineered hardwood produced this way

tends to have fewer problems with "face checking", and also does not have the same plywood appearance in the grain. However, the planks can't tend to have edge splintering and cracking due to the fact the veneers have been submersed in water and then pressed flat.

Dry solid-sawn

Instead of boiling the hardwood logs, in this process they are kept at a low humidity level and dried slowly to draw moisture from the inside of the wood cells. The logs are then sawed in the same manner as for solid hardwood planks. This style of engineered hardwood has the same look as solid hardwood, and does not have any of the potential problems of "face checking" that rotary-peel and slice-peel products have, because the product is not exposed to added moisture.

Engineered

Wood flooring is a popular feature in many houses.

Engineered wood flooring is composed of two or more layers of wood in the form of a plank. The top layer (lamella) is the wood that is visible when the flooring is installed and is adhered to the core. The increased stability of engineered wood is achieved by running each layer at a 90° angle to the layer above. This stability makes it a universal product that can be installed over all types of subfloors above, below or on grade. Engineered wood is the most common type of wood flooring used globally.

There are several different categories of engineered wood flooring:

All timber wood floors are made from sawn wood and are the most common category of engineered wood flooring. They do not use rotary peeled veneer, composite wood (such as HDF), or plastic in their construction.

Veneer floors use a thin layer of wood over a core that is commonly a composite wood product

.

Acrylic impregnated wood flooring uses a layer of wood that is impregnated with liquid acrylic then hardened using a proprietary process.

Laminate and vinyl floors are often confused with engineered wood floors, but are not—laminate uses an image of wood on its surface, while vinyl flooring is plastic formed to look like wood.

Principles of Terracing:

Flat roof is the one which is either horizontal or horizontal with slope less than 10 degree. Even a perfectly horizontal roof has to have some slope at top so that rain water can be drained off easily and rapidly. Similar to the upper floor the flat roofs can be constructed of flag stones , R.S.J and flag stones, reinforced cement concrete, reinforced brick work, jack arch roof or precast cement concrete units. However, the flat roof differ from the upper floor only from the point view of top finish , commonly called terracing, to protect it from adverse effects of rain, snow, heat etc.

SATHYABAMA
INSTITUTE OF SCIENCE AND TECHNOLOGY
(DEEMED TO BE UNIVERSITY)
Accredited "A" Grade by NAAC | 12B Status by UGC | Approved by AICTE
www.sathyabama.ac.in

SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – IV – BUILDING STRUCTURES II– SCIA1306

UNIT IV COST EFFECTIVE CONSTRUCTION TECHNIQUES

ROOFING TYPES

FERRO CEMENT ROOFING SYSTEM

Ferro cement or **Ferro-cement** (also called **thin-shell concrete** or **Ferro-concrete**) is a system of reinforced mortar or plaster (lime or cement, sand and water) applied over layer of metal mesh, woven expanded-metal or metal-fibers and closely spaced thin steel rods such as rebar, metal commonly used is iron or some type of steel. It is used to construct relatively thin, hard, strong surfaces and structures in many shapes such as hulls for boats, shell roofs, and water tanks. Ferro cement originated in the 1840s in France and is the origin of reinforced concrete. It has a wide range of other uses including sculpture and prefabricated building components

Ferro cement components are extremely thin (15 to 25 mm), but have a higher percentage of reinforcement than reinforced concrete, thus achieving a higher tensile-strength-to-weight ratio. Further strength and rigidity is achieved by curvature or folds.

• Ferro cement roofs can be made in situ or with precast components, the former being useful for free forms, the latter being appropriate for modular and repetitive constructions.

• Depending on the design, Ferro cement roofs can be made to span large areas without supporting structures, thus saving costs and providing unobstructed covered areas. If the Ferro cement surface is properly executed (complete cover of wire mesh, dense and smooth finish, cracks sealed) no surface protection is needed, thus saving further costs. However, it is advantageous to apply a reflective coat on the outer surface to reduce solar heat absorption.

FILLER SLAB ROOFS

Filler slabs are one cost - effective roofing system which is based on the concrete portions and instead placing filler material there.

Concept

The filler slab is based on the principle that for roofs which are simply supported, the upper part of the slab is subjected to compressive forces and the lower part of the slab experience tensile forces. Concrete is very good in withstanding compressive forces and steel bears the load due to tensile forces. Thus the lower tensile region of the slab does not need any concrete except for holding the steel reinforcements together. Therefore in a conventional RCC slab lot of concrete is wasted and it needs extra reinforcement due to added load of the concrete which can otherwise be replaced by low-cost and light weight filler materials, which will reduce the dead weight as well as the cost of the slab to 25% (as 40% less steel is used and 30% less concrete). The filler slabs also result in fewer loads getting transferred to the load-bearing walls and the foundations.

The air gap in between the tiles makes it a good heat insulator and the ceiling looks attractive as well.

- Materials used: Bricks,Tiles,Cellular,Concrete,Blocks,Pots,Waste bottles

Advantages:

PRECAST ROOFS

Precast concrete is a construction product produced by casting **concrete** in a reusable mold or "form" which is then cured in a controlled environment, transported to the construction site and lifted into place. In contrast, standard **concrete** is poured into site-specific forms and cured on site. The primary roof and floors systems are made up of three basic components.

Double Tees

Named for its shape, double-tees are used primarily as floor and roof deck components for any type of structure, including parking structures and all types of buildings.

They are made either:

- Pre-topped :using a flange thickness of 4 in., which creates the wearing surface in parking structures
- Field- topped: with a 2-in. flange, on which a cast-in-place concrete composite topping of 2 to 4 in. is added in the field. For roof construction, there is typically no need to add topping on the 2 in. flange.

Typical widths: 8, 10, 12, and 15 ft.
Typical depths: 24, 26, 28, 30, 32, and 34 in.
Typical span-to-depth ratios: Floors: 25 to 35 / Roofs: 35 to 40
Finishes: Form side will generally be “as cast,” resulting in a smooth, hard finish. This generally remains as is and is not painted, although it can be if desired. The top-of-flange side will be smoothed for roof construction, left rough if it will receive a field topping or broomed (either transversally or longitudinally), or circular swirl-finished if it will be used as the wearing surface in a parking structure.

Hollow-core

Hollow-core slabs are used predominantly for floor and roof deck components for various structures such as residential, hotel, office buildings, schools, and prisons.

Typical widths: 2, 4, and 8 ft; some precasters offer 10 and 12 ft widths
Typical depths: 6, 8, 10, 12, 15, and 16 in.
Typical span-to-depth ratios: Floors: 30 to 40 / Roofs: 40 to 50

Solid Slabs

Solid slabs are used as structural deck components similar to hollow-core slabs. They can be made in a long-line pre-tensioning facility and reinforced with prestressing strand or cast in individual forms with either prestressing strand or conventional reinforcing bars. Sizes can vary to satisfy the structural requirements.

Typical widths: 4 to 12 ft.

Typical spans: 8 to 30 ft.

Typical thicknesses: 4 to 12 in.

Finishes: The form side (bottom) is smooth as cast and typically will remain that way in the finished construction. When it is an exposed surface, it can remain as is or painted without additional treatment. The top side is troweled to the desired degree of smoothness or may be intentionally roughened to receive a cast-in-place concrete topping that will act compositely and provide additional strength.

FUNICULAR SHELLS:

CONCEPT:

A Funicular Shell is a 3 dimensional catenary on a rectilinear base. The roofing system consists of doubly curved shells made with materials of good compressive strength such as waste stone pieces and brick tiles and supported on reinforced concrete edge beams. A series of these shells in variable geometric configurations supported on a grid of concrete beams, identical to a coffer slab, provides an attractive roof for small to medium spans.

COPONENTS:

Edge Beam: This is a reinforced concrete beam which supports and distributes the horizontal thrust of the funicular shell. The beam can be conventionally cast along with the funicular shell. As per the span, the beam is designed for a coffer slab or a grid of beams. Alternatively, the beam can be partially precast, in which case a pre-welded reinforcement cage is placed along the grid and cast half. The cage is fabricated as a truss girder (see overleaf for details) which improves the load bearing capacity of beam considerably, while simultaneously reducing the beam section.

Funicular Shell: The entire area to be roofed is divided into a grid depending on the size of the funicular shell required or the size / shape of moulds available. The rise to span ratio is 1:6, thus the optimal span of the shell is 3 m though it can span up to 15 m. The mould is supported between the edge beams. Timber planks are used to bridge the gap between the edge of the mould and the edge beam. The shell comprises of the materials – bricks, stone waste – laid in the funicular profile topped with cement-sand mortar and concrete screed.

Concrete In-fill: After the shells have been cast, the valley spaces which are formed between the shells can be filled with light-weight material like brick jelly lime concrete and finished flat. The infill will enable the construction of an intermediate floor which can be used to build above.

ADVANTAGES:

- Allows ample flexibility in design- funicular shells can take any shape – square, rectangular, triangular or trapezoidal
- Uses locally available waste stone, normally available from stone cutting and polishing units

- For construction above the intermediate floor, the funicular roof provides greater flexibility for locating walls since the load distribution is uniform because of arch action of the shell
- Design of the funicular roof can be very well adapted to seismic design requirements
- Finishes like plaster and paint for the roof are not needed
- Being a labour intensive technology, leads to employment generation and integrates craftsmanship
- Simple technology which can easily be adapted by semi-skilled labour with minimum supervision
- The funicular roof is aesthetically much better than other roofs – various artistic patterns can be made using brick and stone.

ADVANCED TYPES OF FLOORING:

Steel Fibre Reinforced Flooring:

Steel fibre reinforced concrete is a composite material made of hydraulic cements, water, fine and coarse aggregate, and a dispersion of discon- tinuous, small fibers. It may also contain pozzolans and admixtures commonly used with conventional concrete. This is a type of flooring in which steel fibres will be mixed to the concrete mixture by a specific ratio say 5 % by volume of concrete.

Over the last ten years, ground-supported floor slabs have been designed and built using steel o fibers to enhance concrete properties. They are used in ground-supported slabs for two main reasons.

- To control the formation and development of cracks caused by early age plastic shrinkage and restrained long-term drying shrinkage.
- To provide a degree of post-cracking load-carrying capacity, i.e. an ability of the slab to carry load after the first crack has formed during slab flexure.

Industrial floors are subjected to extensive and heavy use and often wide ranging loading conditions. Well designed and constructed floors impact positively and significantly toward the life cycle costs of a given structure. Over the last 30 years steel fibres have proven their reliability and suitability as a preferred method of reinforcement for floors.

Advantages:

- Time savings in construction as compared to mesh solutions
- Improved durability
- Impact resistance
- High ductility
- Crack control
- Easier concrete placement compared to use of traditional reinforcement
- Higher level of health and safety
- Use of Laserscreed possible

Epoxy Flooring:

Epoxy is a term that's come to represent a class of materials and the act of using those materials. Epoxy "systems" is based on a combination of resins and hardeners. When mixed together, the resin and hardener chemically react to form a rigid plastic material. The final material is strong, durable, resistant, and bonds extremely well to most base layers. Epoxy floors are so strong that they're often used in heavy traffic areas such as industrial environments, hospitals, or sports facilities.

Types of Epoxy Flooring Systems

If you've ever considered an easy-to-maintain, industrial-grade flooring option, we bet there's an epoxy solution for you! At Performance Industrial, we offer over 30 different epoxy and industrial flooring systems. Here are a few of our most popular:

Self-Leveling Epoxy Floors – Self-leveling epoxy is applied over new, old, cracked, or damaged concrete floors to create a smooth, seamless, durable, and low maintenance flooring surface. The self-leveling system is also available in a variety of colors. Colors are used to make decorative designs, denote traffic patterns, or highlight work zones in industrial settings. This system is used when you need aesthetic appeal and/or abrasion, chemical, heat or slip resistance.

Epoxy Mortar Floors– Ultra-tough, this is the strongest epoxy floor system available. This high-build, seamless system is made with 100% solids epoxy and graded sand or quartz sand and then troweled into place. Mortar systems are highly impact and chemical-resistant. This system is also effective in repairing old floors because of its high-build process.

Quartz-Filled Epoxy Floors – Quartz epoxy flooring systems combine high-performance epoxy polymer resin with colored quartz grains. The result is a multi-functional floor that's decorative, sanitary, slip-resistant, and exceptionally durable.

Anti-Static Epoxy Floors (ESD Resistant Floors): Electro-static charge (ESD) can be extremely dangerous in many work environments. Anti-static epoxy flooring helps to reduce static hazards. This flooring system typically contains a conductive material that accumulates static electricity to drain, ground, or dissipate any potential discharge. It's highly recommended in environments that contain flammable materials.

Epoxy Flake Floors: Flake floor systems are made when colored chips or flakes are placed within the epoxy to create vibrant, multi-hued, seamless, resilient surfaces. The chips are not only aesthetically pleasing, but their slightly rough surface reduces slips and falls. The flakes are available in an endless variety of colors, styles, textures, and sizes and can be mixed into the combination of your choice.

Vacuum Flooring:

The vacuum dewatered flooring method is a system for laying high quality concrete floors with superior cost effectiveness. The key to the use of this method is the dewatering of concrete by vacuum process. Surplus water from the concrete is removed immediately after placing and vibration, reducing the water – cement ratio to an optimum level.Through the vacuum treatment it is possible to reduce the water content in concrete by 15% to 25% which greatly increases the compressive strength.

Laying process:

The method involves the laying of concrete in the following sequence,

- Concrete in the conventional way but a higher slump so that the workability is good and concrete pouring and spreading is done fast.
- Poker vibration is always essential for floor thickness of 100m and above.
- Surface vibration using double beams surface vibrator.
- Leveling the vibrated surface with a straight edge.
- Vacuum dewatering using vacuum pump and suction mat top cover and filter pads.
- Floating and troweling of the concrete pavement using skim floaters.

Advantages:

- It gives monolithic and shringage free pavements.
- Single panel without joints upto 100 sq,meters
- Very high spliting strength.
- Controlled and uniform surface finish.
- High quality pavements in terms of strength and flexibility.

Linoleum Flooring:

It is a covering which is available in rolls and is spread directly on concrete or wooden flooring. Linoleum sheet is manufactured by mixing oxidized linseed oil in gum, resins, pigments, wood flour, corkdust, and other filler materials. The sheets are either plain or printed and are avialble in 2 to 6mm thickness and 2 to 4m wide rolls. Linoleum tiles are also available which can be fixed to concrete base or wood floor in different patterns. Linoleum sheet is either spread as such or may be glued to the base by inserting a layer of saturated felt.Linoleum coverings are attractive , resilient , durable and cheap and can be cleaned very easily. However it is subjected to rotting when kept wet or moist for some time. It cannot therefore be used for bathrroms, kitchens etc.

ESD Flooring

Electrostatic discharge (**ESD**) is the sudden flow of electricity between two electrically charged objects caused by contact, an electrical or dielectric breakdown. A buildup of static electricity can be caused by tribo charging or by electrostatic induction. The ESD occurs when differently-charged objects are brought close together or when the dielectric between them breaks down, often creating a visible spark

Static dissipative rubber flooring tiles and sheet goods should never be installed in enviornments where the control of static discharge is required for human safety or the protection of critical environment operations.

Until about 5 years ago, engineers believed that the walking body voltage properties of static dissipative rubber were as effective at inhibiting human body model (HBM) electro static discharge events as permanent static conductive flooring options. Empirical research has emerged recently showing that, in fact, static dissipative flooring is ineffective at reducing charges already stored on the body of a moving person. This problem is particularly acute within spaces older than 5 years; due to chemistry limitations, it is impossible to produce a static dissipative rubber flooring that will remain static dissipative for the life of the floor. Given that static control flooring is rarely tested after installation, the liabilities of a limited shelf life greatly restrict the potential for recommending any type of static dissipative floor when a static conductive option is also available - usually for the same cost.

These findings are the result of long-term longitudinal surveys monitoring the performance of static dissipative rubber installations at several major US based electronic manufacturers. These floors were originally installed between 1995 and 2006.

Easy to Clean and Maintain

Our ESD Rubber Tile features Stain Block technology, which prevents soiling, ensures an easily cleaned surface and makes the flooring look new longer. After a two year study, Lucent Technologies found ESD Rubber to be the easiest ESD surface to care for. Rubber can be washed with neutral cleaners mixed with water. Finishes and buffing are unnecessary.

Low Total Cost of Ownership

Though its installed cost is higher than some flooring options, the total cost of ownership for rubber is low because it is durable and inexpensive to maintain, making it a good choice for companies concerned with the long-term implications of their investment.

The conductive version of our ESD Rubber tile has carbon contact points commonly associated with ESD vinyl, with the attractive patterns of a more decorative floor tile. Ergonomically, rubber is a better anti-fatigue floor than either ESD epoxy or ESD vinyl. Like ESD carpet, rubber dampens noise from rolling carts and automatic equipment.

Low Body Voltage Generation:

Two totally different properties that are key considerations when evaluating and selecting ESD Flooring is the ability of the ESD flooring to inhibit charges generation AND dissipate any accumulated charge with an acceptable decay time. An ESD floor with low body voltage generation properties minimizes the charge created by movement of people and equipment. Our ESD Rubber Tile does just that, and if you generate less charge, that's less charge that you have to dissipate- which puts you ahead of the game.

ESD Rubber is ideal for use in a MRI Imaging suite

Advantages:

- Attractive
- Low Cost of Ownership
- Flexible Modularity
- Extremely Hard-Wearing
- Slip Resistant—meets ADA standards
- Low vibration
- Superior Sound Absorption
- Excellent Wear Abrasion
- Low Maintenance
- Dirt Repellent
- Chemical and Solder Resistant
- Withstands Extra Heavy Traffic
- Capable of Stress load to 850 PSI Rolling load
- Meets ANSI/ESD S.20.20