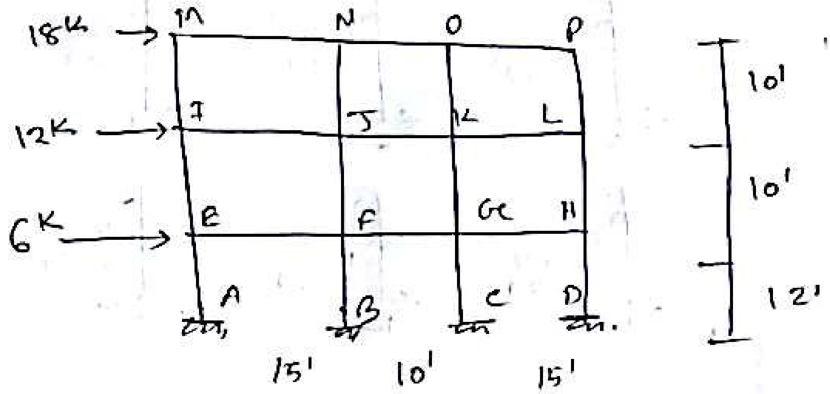
2015-16

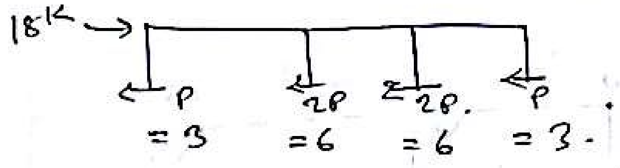
(7)

Portal method



BMD for Beam and column.

column shear force .(3rd level)

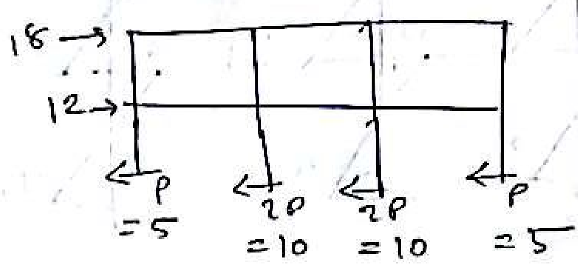


$\Sigma F_x = 0$

$\Rightarrow 18 - 6P = 0$

$\Rightarrow P = 3^K$

column shear (2nd level)

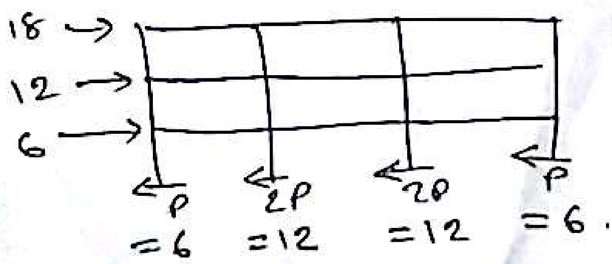


$\Sigma F_x = 0$

$\Rightarrow 18+12 = 6P$

$\Rightarrow P = 5^K$

column shear (1st level)

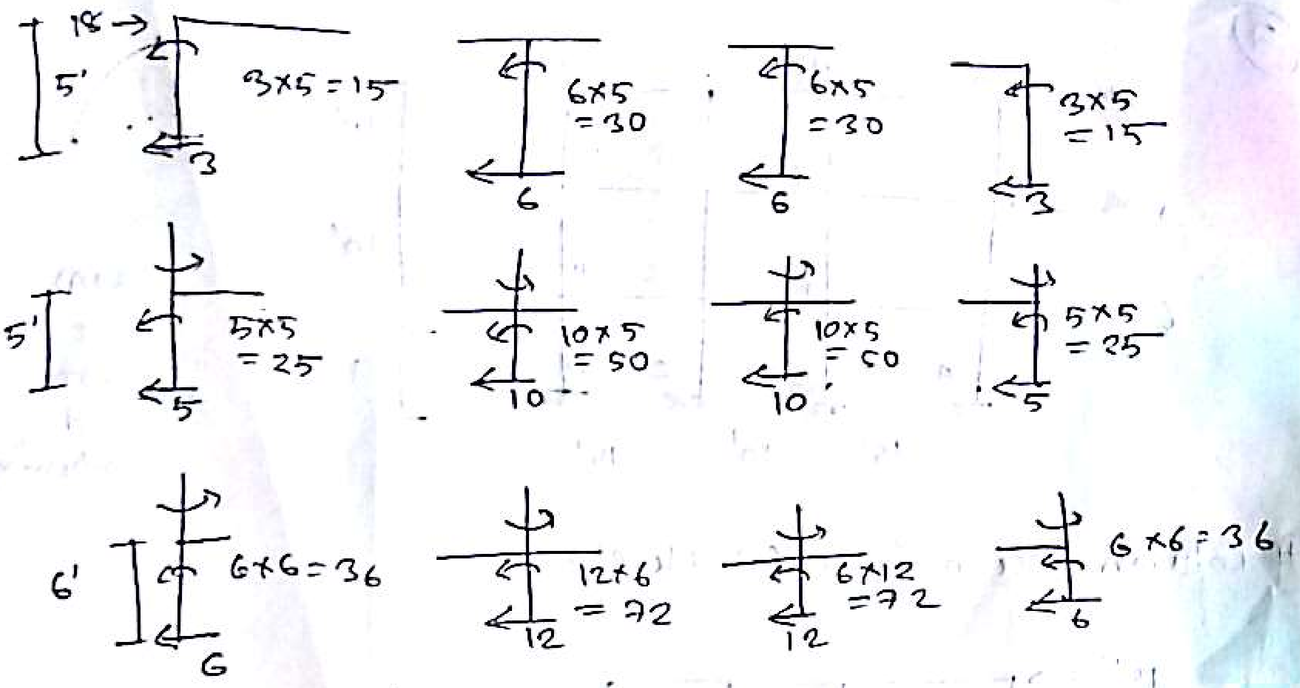


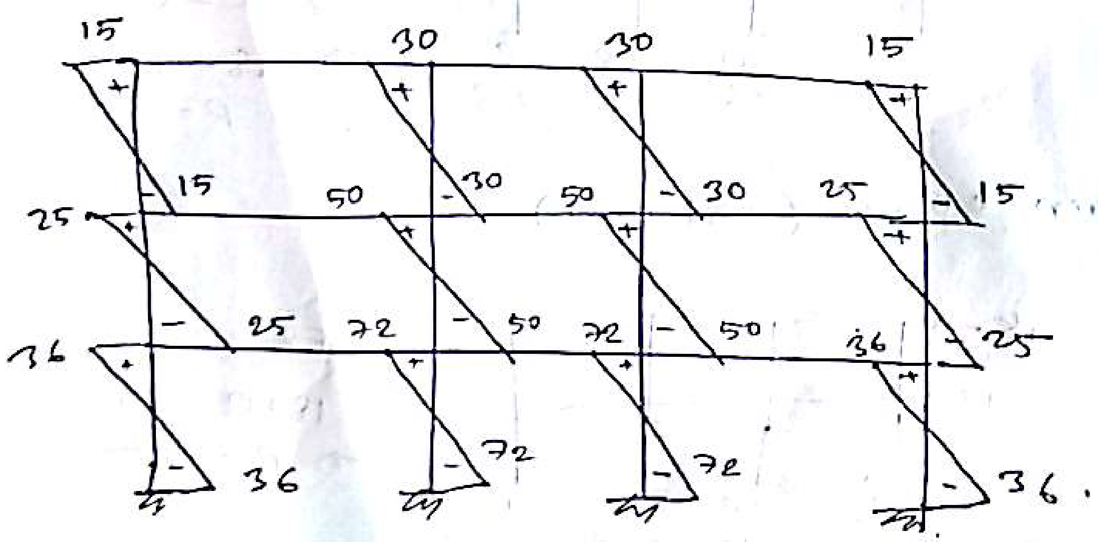
$\Sigma F_x = 0$

$\Rightarrow 18+12+6 = 6P$

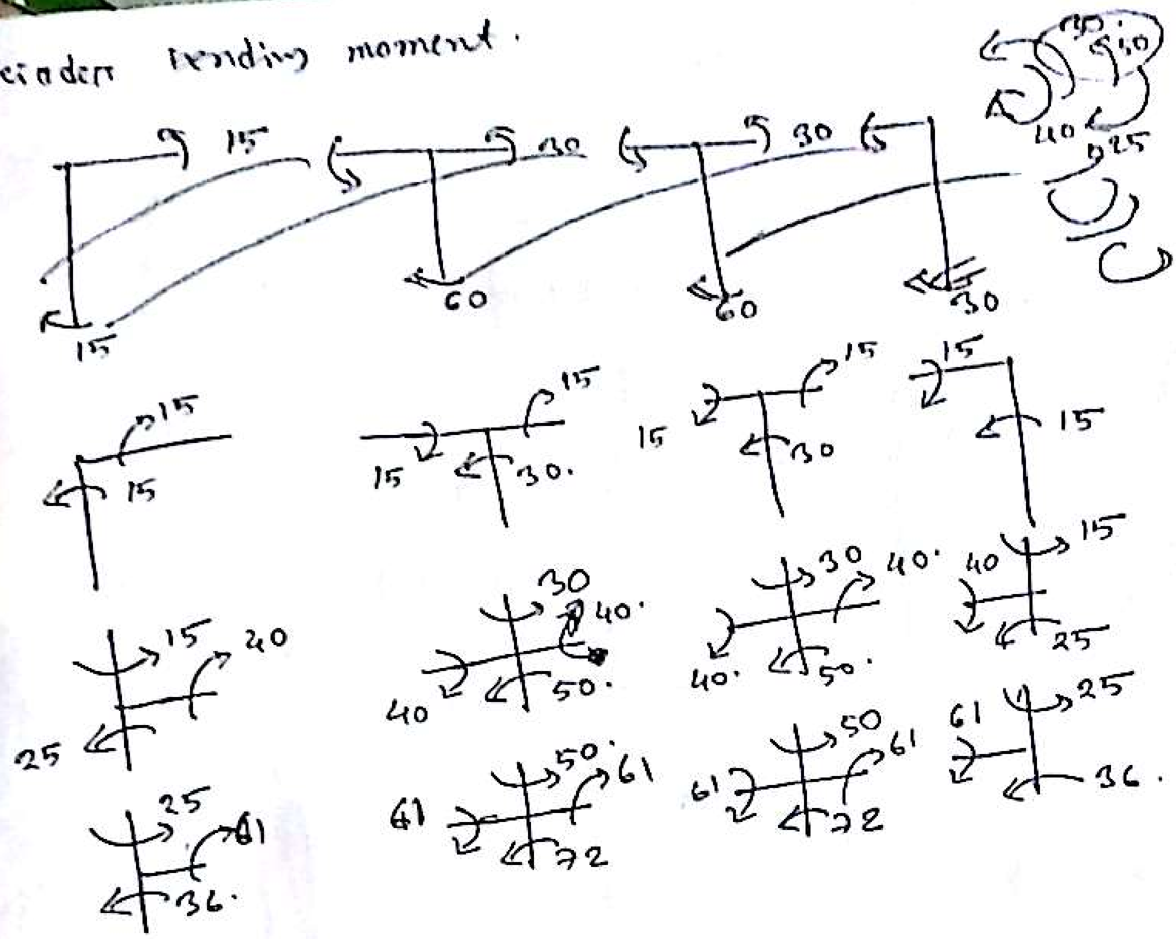
$\Rightarrow P = 6^K$

column bending moment:



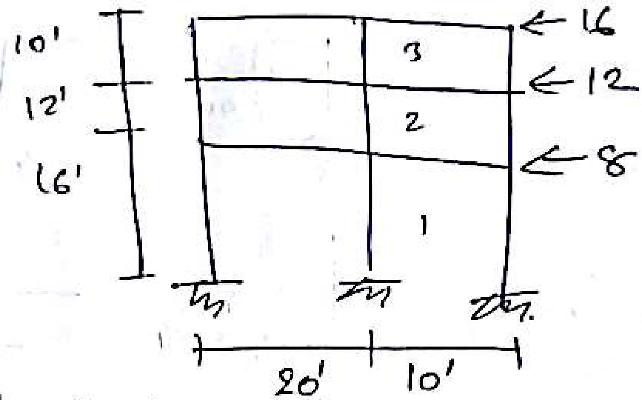


Girder bending moment.



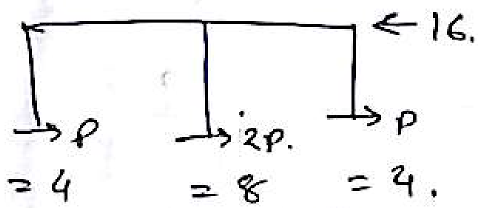
2011-12

(10)



BMD.
SFD → Girder

column Shear: (3rd level)

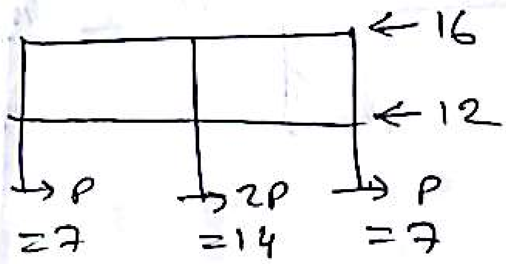


$\Sigma F_x = 0$.

$\Rightarrow 16 = 4P$

$\Rightarrow P = 4$

Column Shear (2nd level)

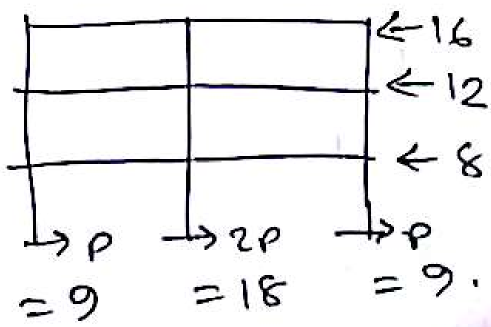


$\Sigma F_x = 0$.

$\Rightarrow 28 = 4P$

$\Rightarrow P = 7$.

Column Shear. (1st level)

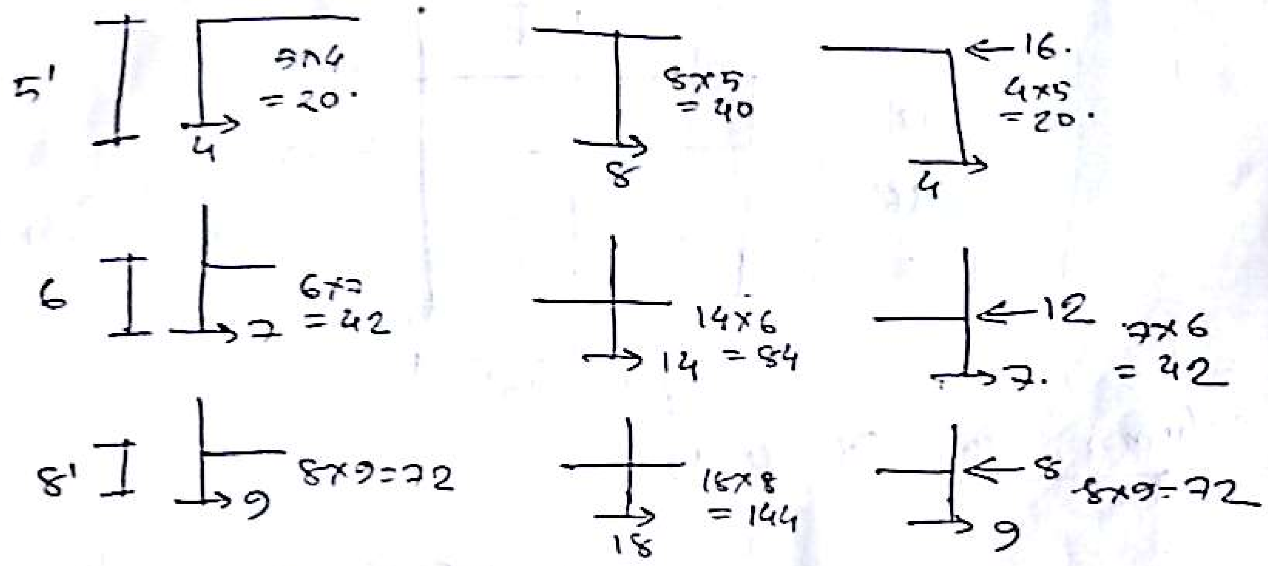


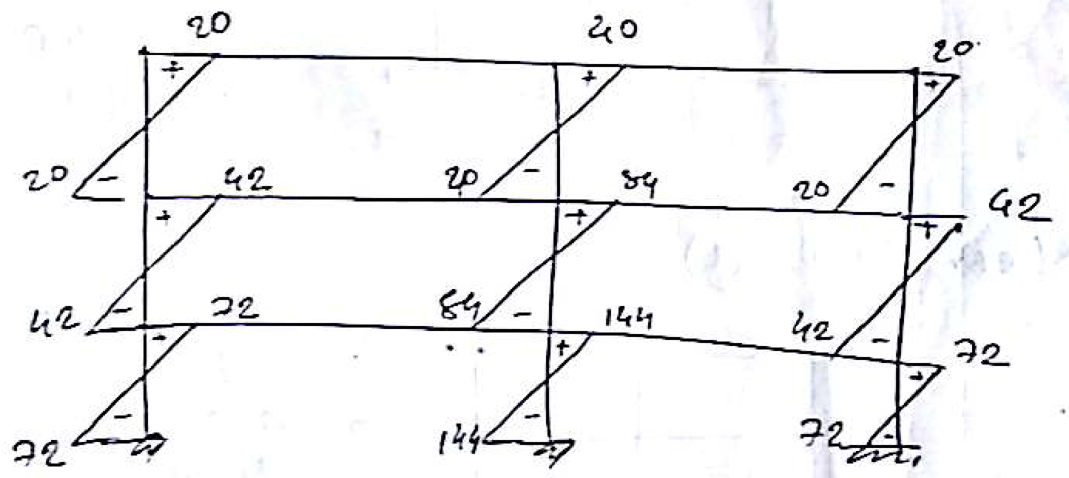
$\Sigma F_x = 0$.

$\Rightarrow 36 = 4P$

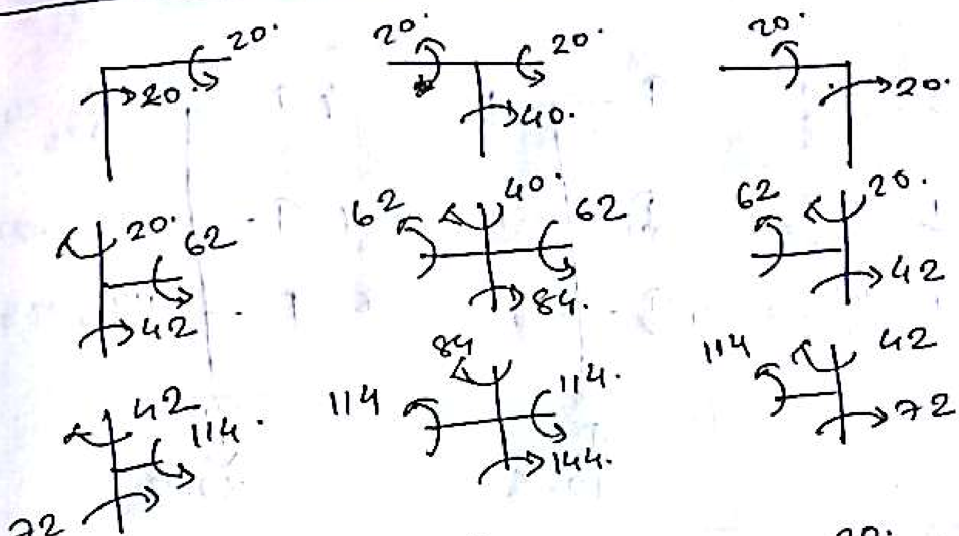
$\Rightarrow P = 9$.

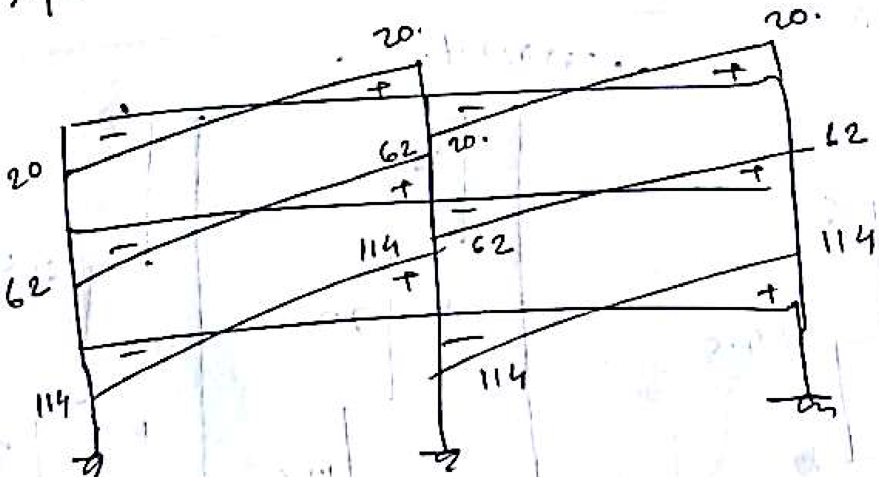
column bending moment:





Girder bending moment:





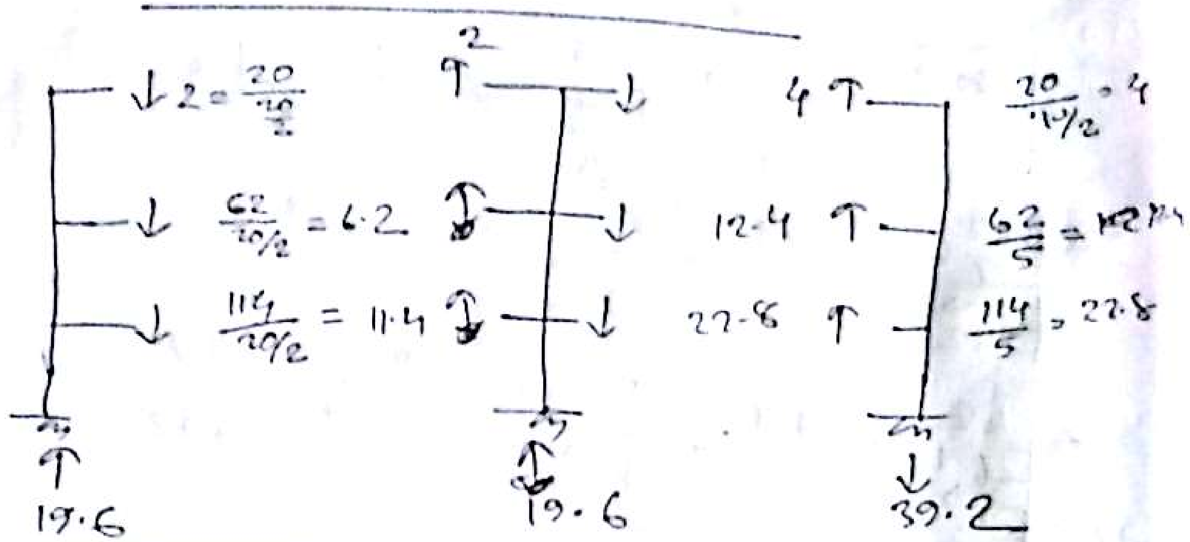
$$SF = \frac{\text{Beam/Girder BM at left or right end}}{\left(\text{Beam/Girder length}/2\right)}$$

e.g SF at left most point

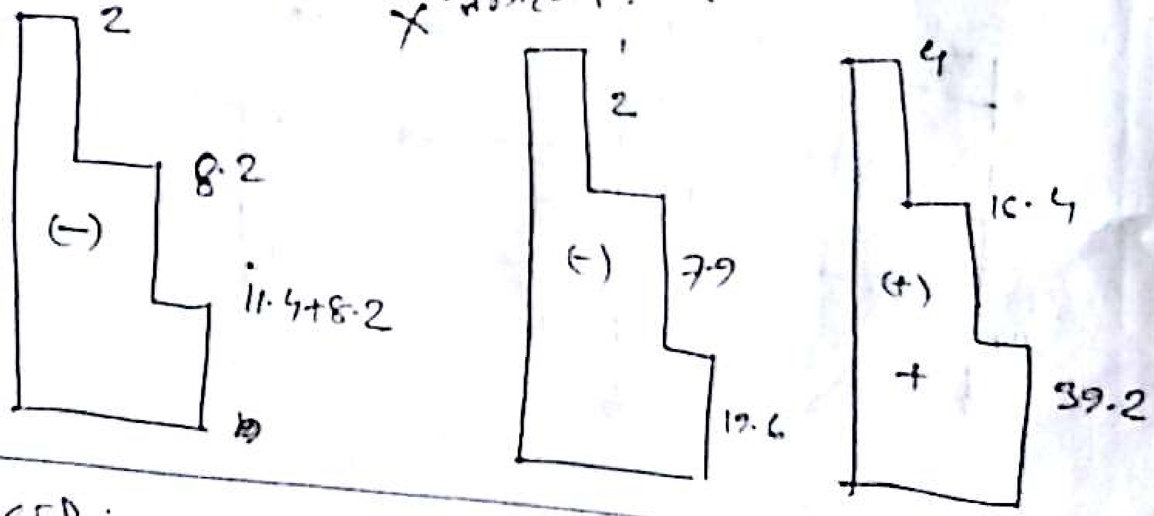
$$SF = \frac{20}{\frac{20}{2}} = 2\,K$$

" " right " " $SF = \frac{20}{10/2} = 4\,K$

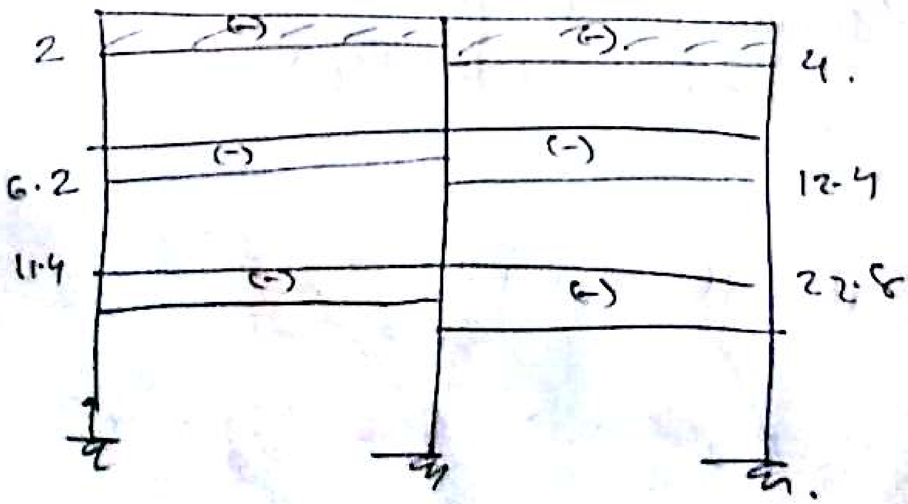
Vul cirta colum AFD



Column AFD



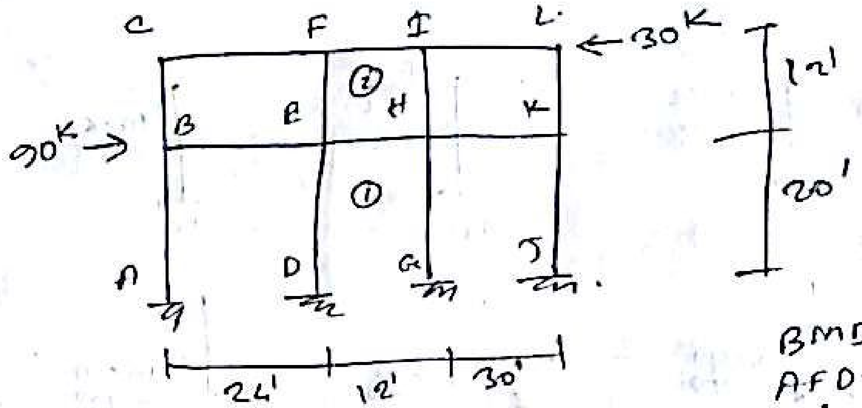
Girder SFD.



2006-7

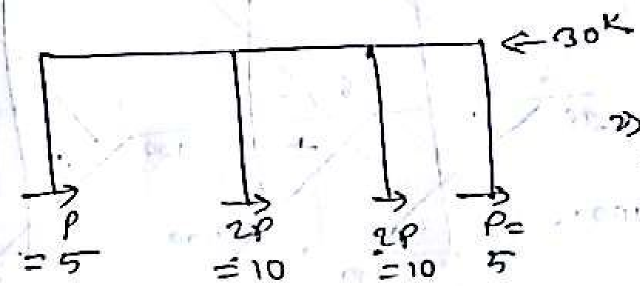
(12)

SAD-2



BMD → Girder
AFD + BMD
↳ column.

Column shear force. (2nd level):

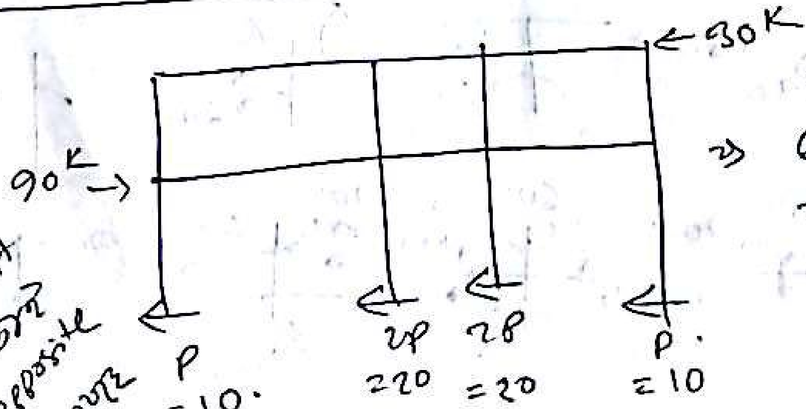


$\Sigma F_x = 0$

$\Rightarrow 30 = 6P$

$\Rightarrow P = 5$

Column shear force (1st level):



$\Sigma F_x = 0$

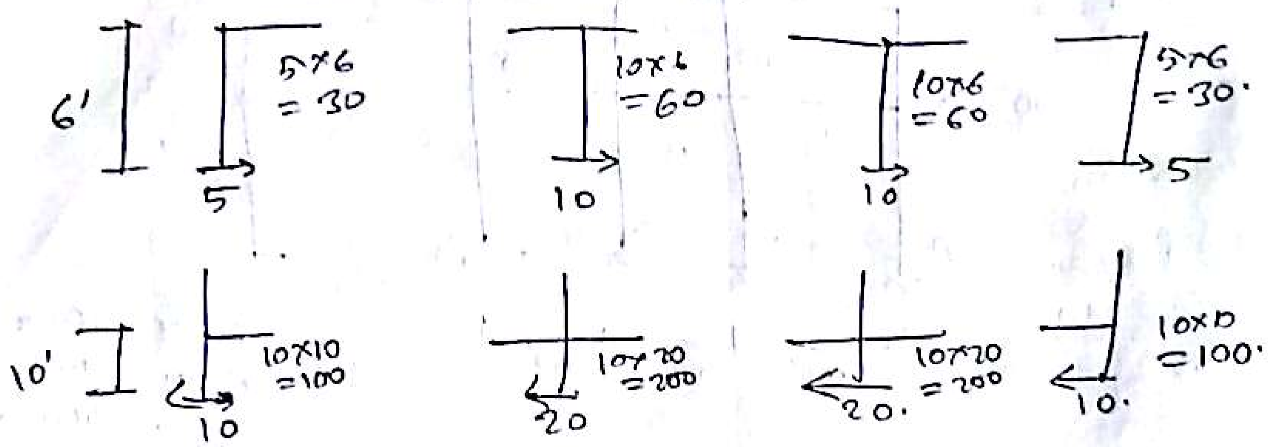
$\Rightarrow 60^K = 6P$

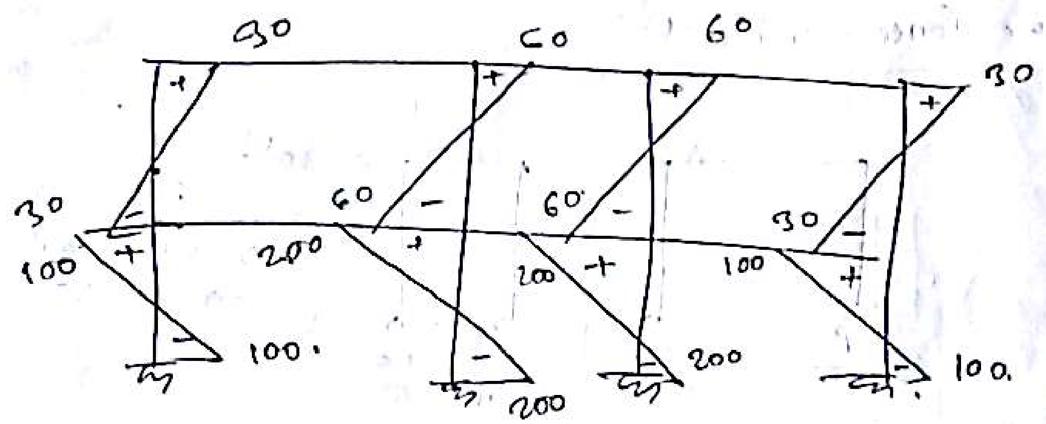
$\Rightarrow P = 10$

resultant [illegible] right side [illegible] shear opposite [illegible]

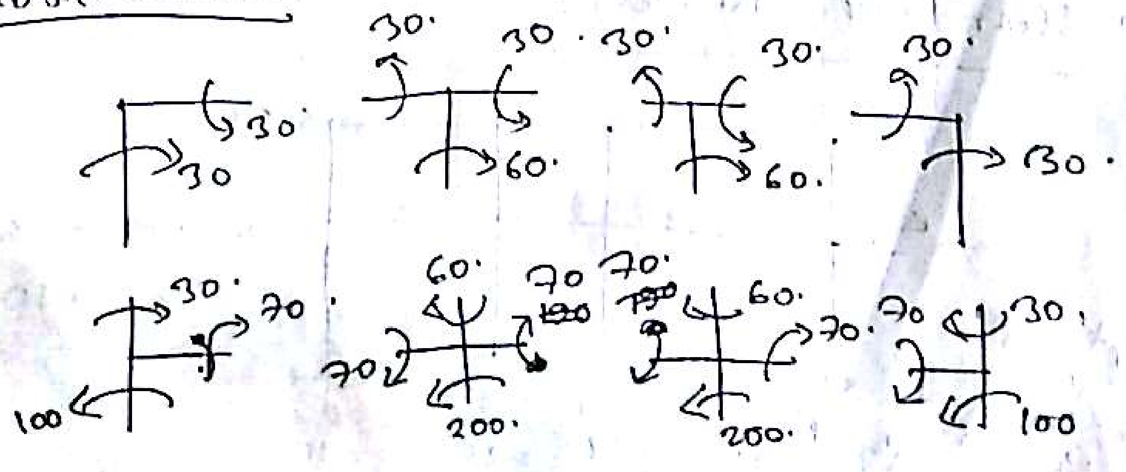
Not Sure ☺

BMD for column:



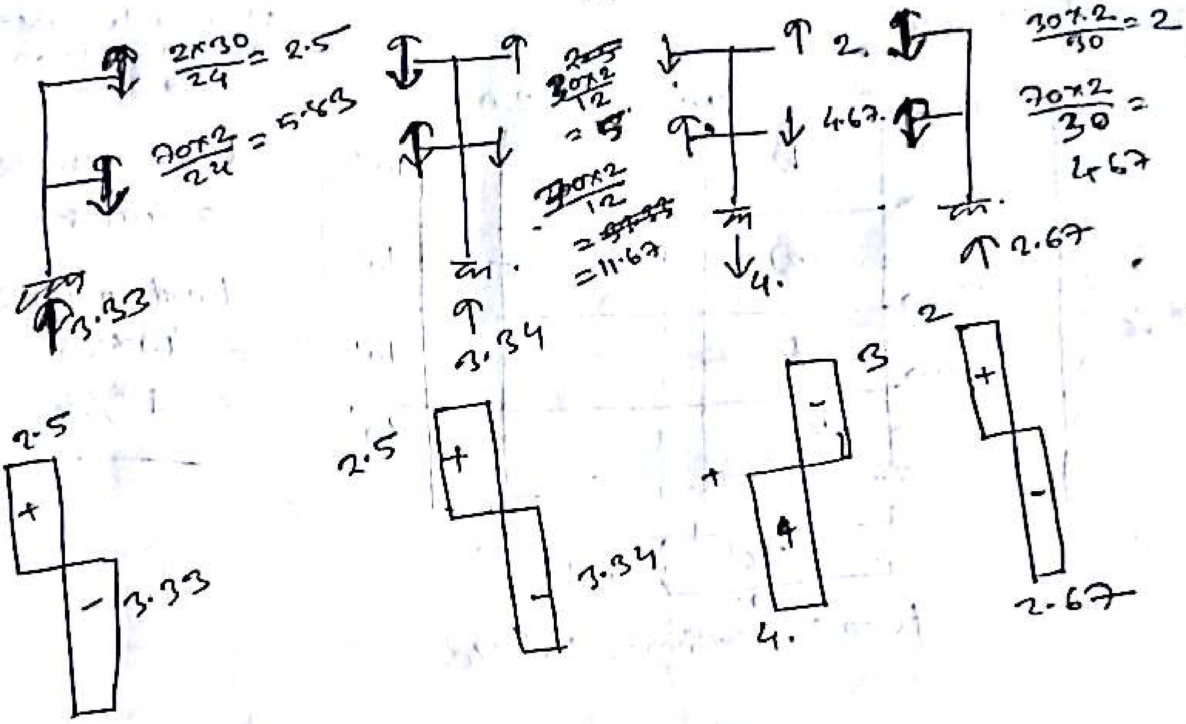


BMD for Girder:



Column Axial force :

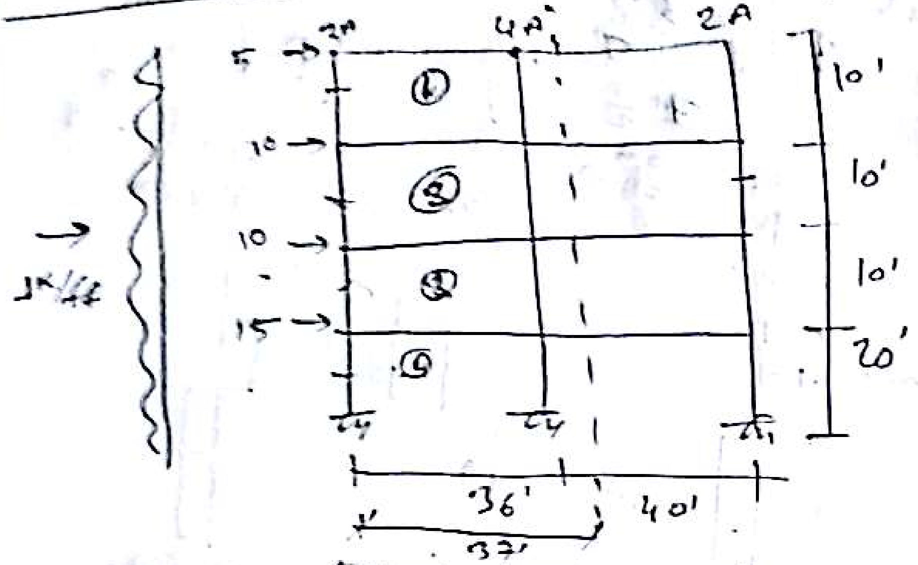
SF = $\frac{2M}{L}$ or $\frac{M}{L/2}$



Cantilever method

Assumption:
↳ There is a hinge at middle point of each column and beam.

Class size sheet



Frame spacing = 25' c/c.

Wind pressure = 40 psf

Load on frame = 40×25 = 1 k/ft.

Stress

Story

⇒ $c.g.(\bar{x}) = \dfrac{4A(36) + 2A(76)}{8A} = 37'$ (from left column)

$I = \Sigma A d^2$

$= 2A\times37^2 + 4A\times1^2 + 2A\times39^2$

$= 5784A.\ (ft^4)$

↳ for all floors

Moment

Story	M.
1	5×5 = 25
2	10×5 + 5×15 = 125
3	10×5 + 10×15 + 5×25 = 325
4	15×10 + 10×20 + 10×30 + 5×40 = 850

$$\sigma = \frac{Mc}{I}.$$

Stress on column.

<u>Story-1</u>

Axial stress = $\frac{Mc}{I} = \frac{4.25 \times C}{5784 A}$

Force = $\frac{25 \times C}{5784 A} \times$ Area of column.

$$C_L = \frac{25 \times 37}{5784 \times A} \times 2A = 0.32$$

$$C_C = \frac{25 \times 1}{5784 \times A} \times 4A = 0.02$$

$$C_R = \frac{25 \times 39}{5784 \times A} \times 2A = (-0.34)$$ ⑦ Compression.

<u>Story-2</u> Similarly.

$$C_2 = \frac{125 \times 37}{5784 A} \times 2A = 1.6$$

$$C_C = \frac{125 \times 1}{5784 A} 4A = 0.09$$

$$C_R = \frac{125 \times 39}{5784 A} . 2A = -1.67.$$

Story-3

$C_L = 4.2$

$C_C = 0.224$

$C_R = -4.4$

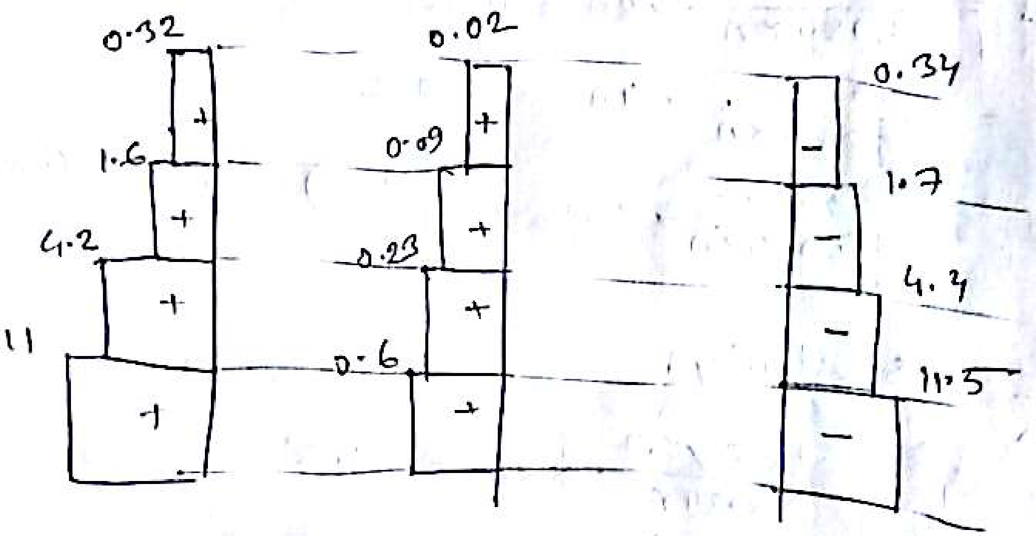
Story-4

$C_L = 11 K.$

$C_C = 0.6$

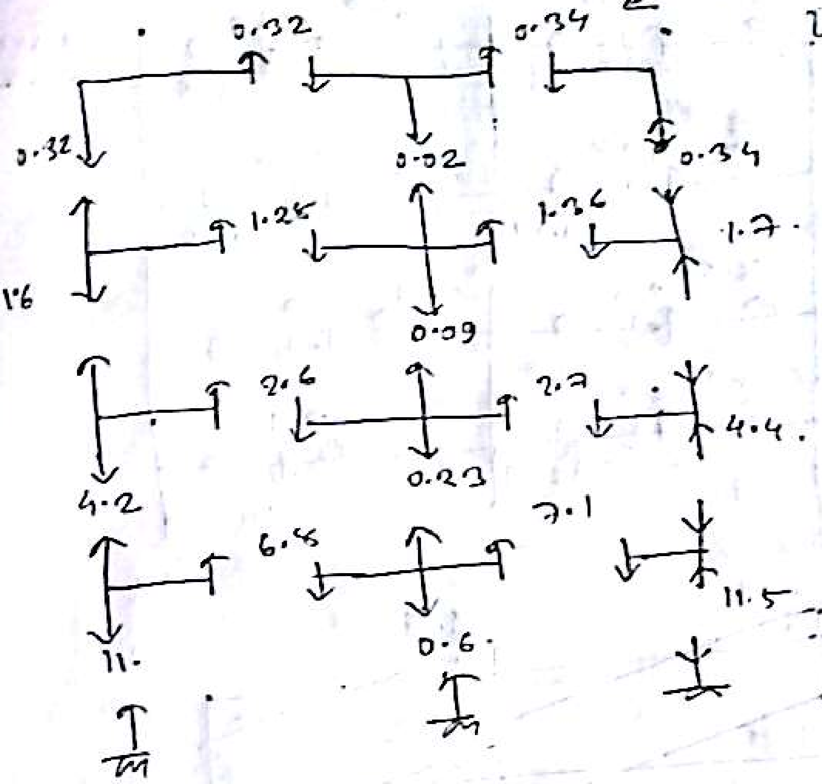
$C_R = -11.5$

Column AF :



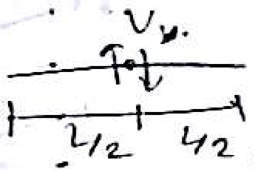
Beam SF. (Freebody)

← value negative chilo. Ls mind the direction.



Beam BM.

$BM = V \times \frac{L}{2}$.



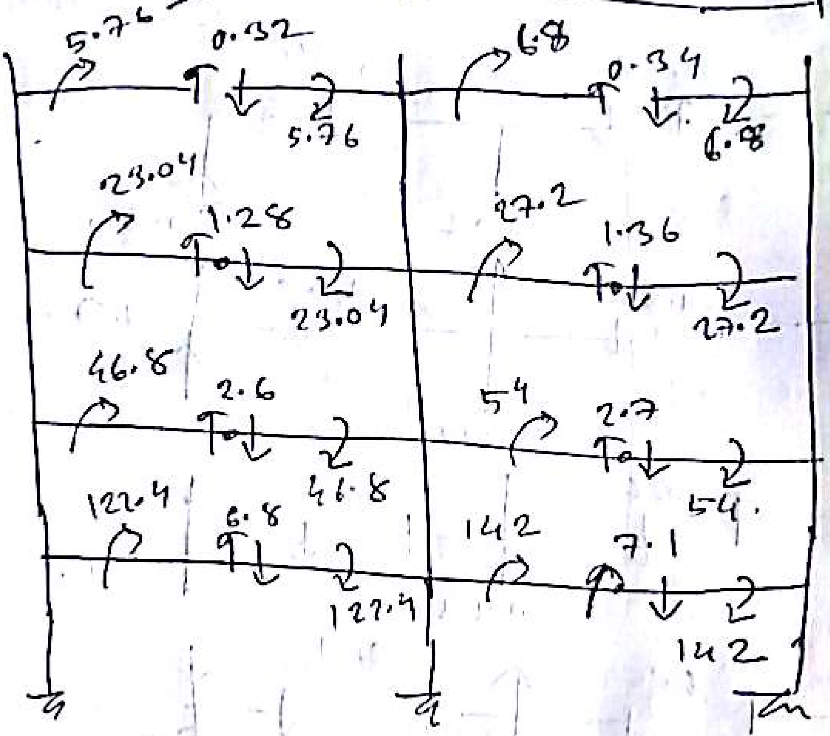
Col BM ⟶ Same type.

Beam BM

36

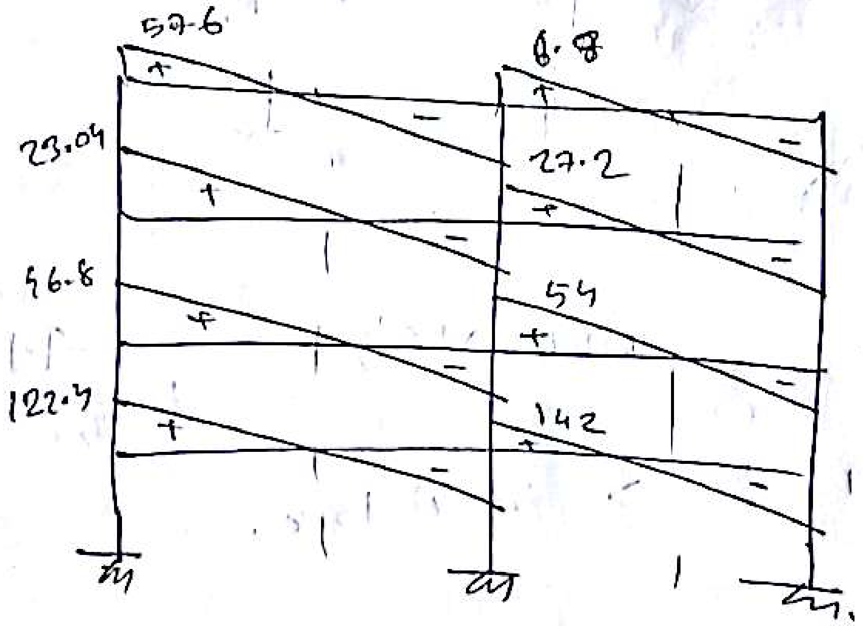
$\frac{36}{2} \times 0.32 = 5.76$

20

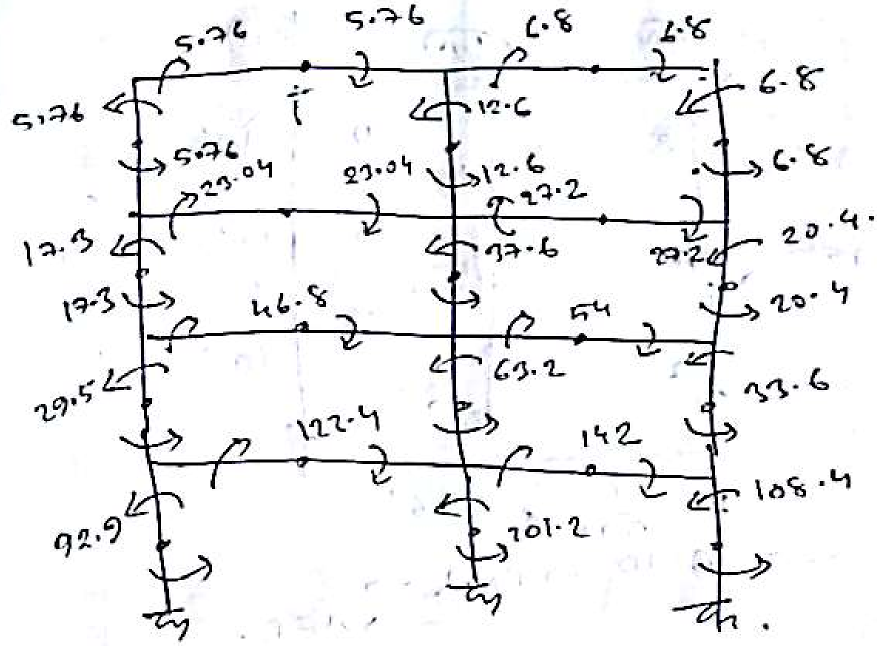


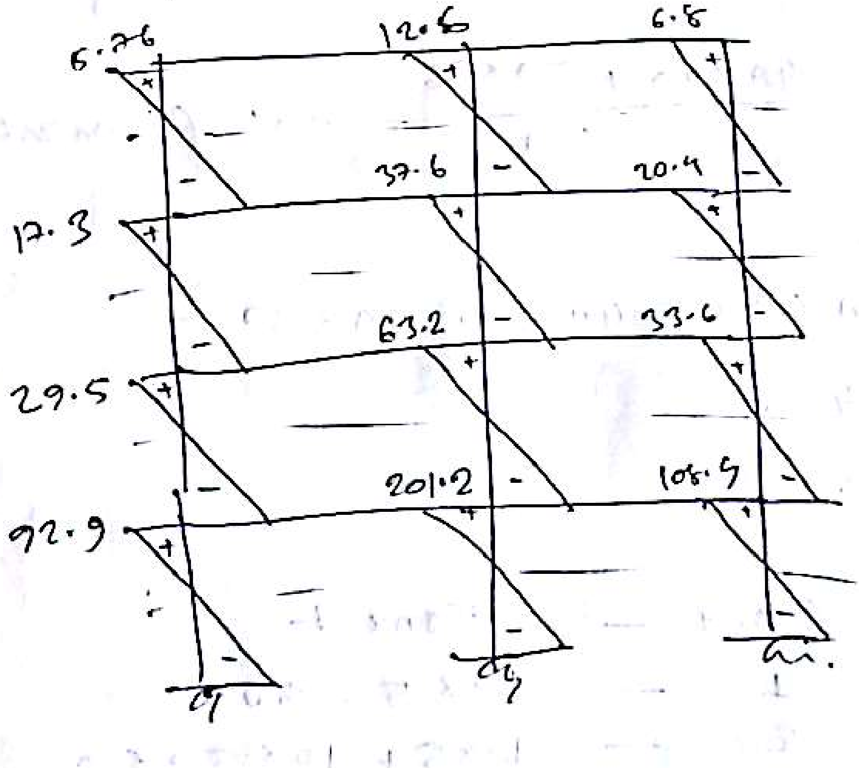
0.34×20 = 1.7

Column



Column BM:

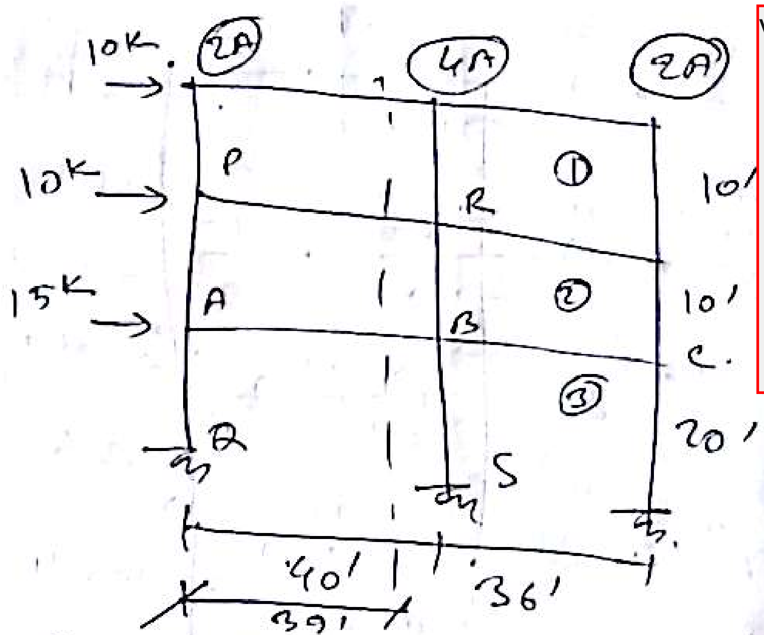

5.76
5.76
6.8
6.8
5.76
12.6
6.8
5.76
23.04
23.04
12.6
27.2
6.8
17.3
37.6
27.2
20.4
17.3
46.8
54
20.4
29.5
63.2
33.6
122.4
142
92.9
201.2
108.4


5.76
12.6
6.8
37.6
20.4
17.3
63.2
33.6
29.5
201.2
108.4
92.9

2015-16 ×

2014-15

(1)



vul eita

ABC → SFD , BMD → PAQ →
→ AFD.

⇒ $\bar{X} = \frac{4A \times 40 + 2A \times 76}{8A} = 39'$ (from left)

$I = Ad^2$

$= 2A \times 39^2 + 4A \times 1^2 + 2A \times 37^2$

$= 5784A$.

Moment :

Story	—	Moment
1	—	10 × 5 = 50
2	—	10×5 + 10×15 = 200
3	—	15 ×10 + 10×20 + 10×30 = 650

Sto

Stress on column:

$\sigma = \frac{mc}{I}$.

Story-1

Axial force $= \frac{50 \times c}{5784A} \times$ (Area of column).

$C_L = \frac{50 \times 39}{5784A} \times 2A = 0.67$.

$C_C = \frac{50 \times 1}{5784A} \times 4A = -0.03$

$C_R = \frac{50 \times 37}{5784A} \times 2A = -0.64$.

} NA to right side.

Story-2

$AF = \frac{200\,c}{5784A} \times$ Area.

$C_L = 2.7$

$C_C = -0.14$

$C_R = -2.56$

Story-3

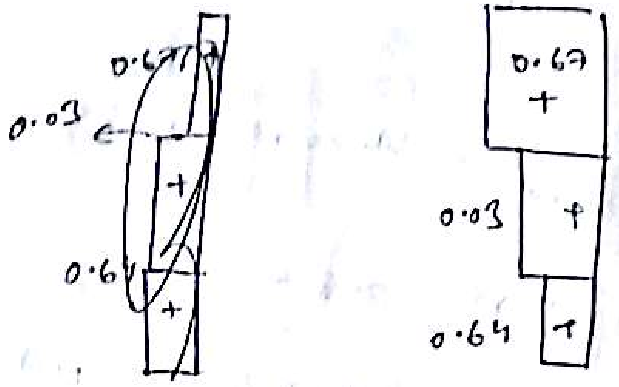
$AF = \frac{650\,c}{5784A} \times$ Area of column.

$C_L = 8.77$

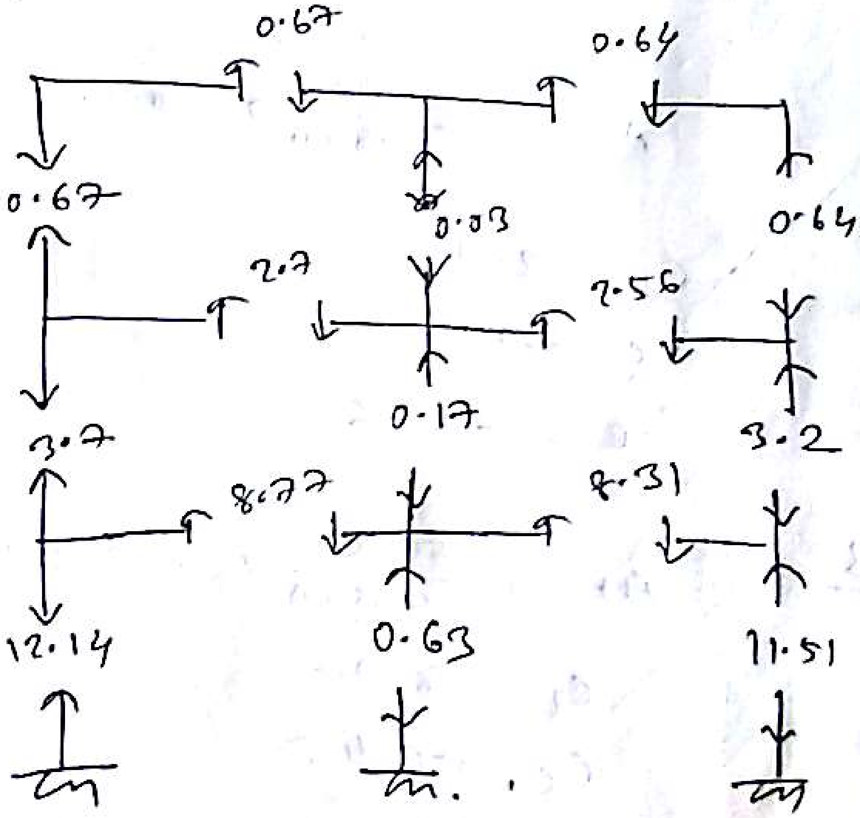
$C_C = -0.45$

$C_R = -8.31$

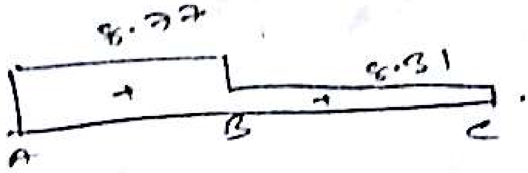
AFD for PAQ



SFD Beams



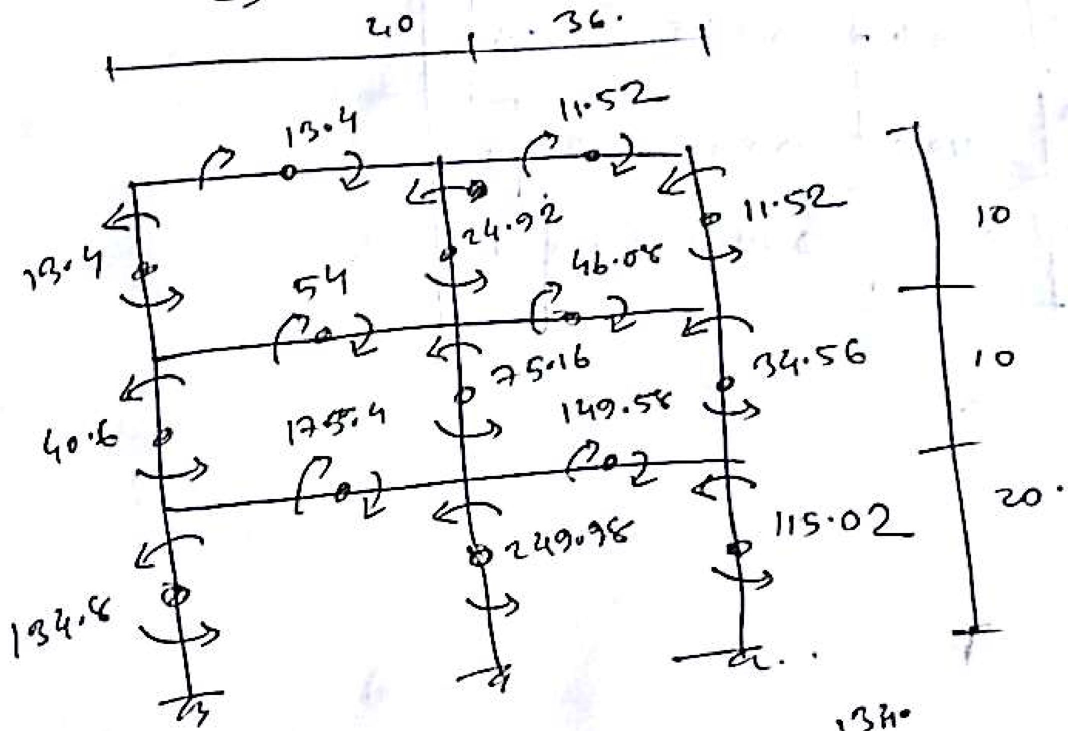
SFD for ABC

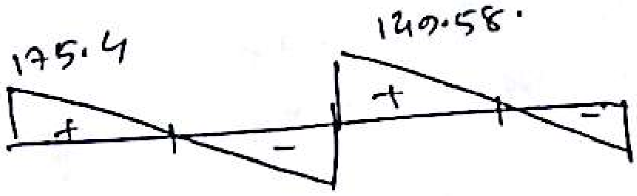


BM - Beam and column.

$\frac{45}{2}$

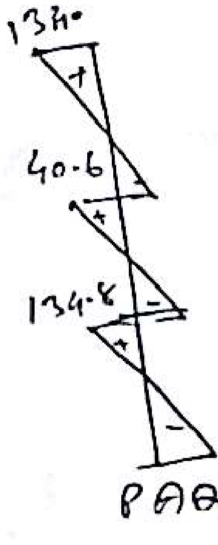
$20 \times 0.67 = 13.4$





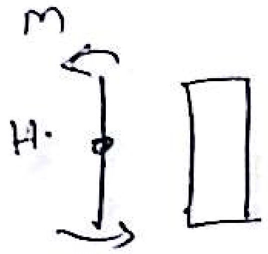
ABC.

Bmd



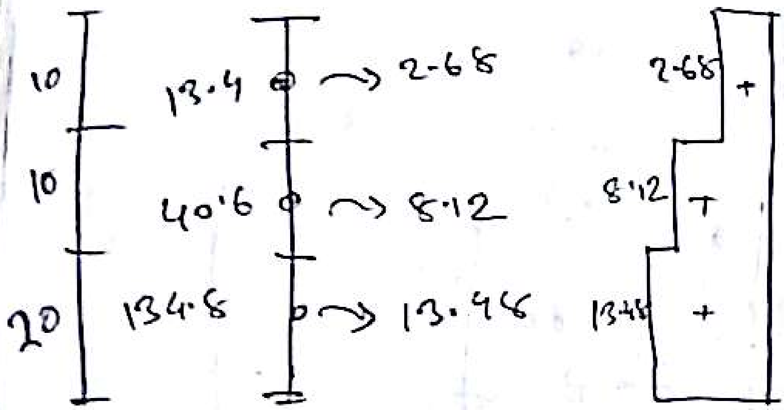
PAQ.

column shear force



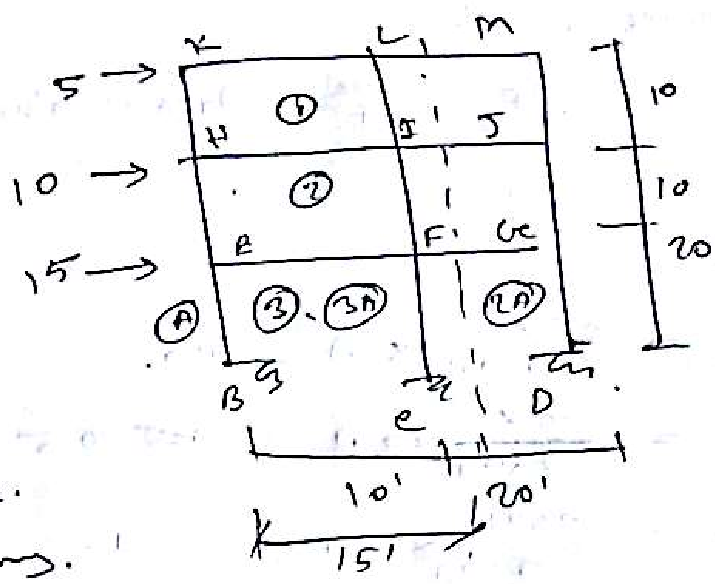
$V_c = \frac{2M}{H}$.

from column bending moment.



2013-14

(9)



SFD, BMD > EFG.
AFD → Column.

$$\Rightarrow \bar{x} = \frac{3A \times 10 + 2A \times 30}{6A} = 15' \text{ (from left)}$$

$$I = \Sigma A d^2$$
$$= A \times 15^2 + 3A \times 5^2 + 2A \times 15^2$$
$$= 950 A.$$

Moment.

Story		Moment.
1.	—	$5 \times 5 = 25$
2	—	$10 \times 5 + 5 \times 15 = 125$
3	—	$15 \times 10 + 10 \times 20 + 5 \times 30 = 500$.

Axial force in colum.

$$AF = \frac{Mc}{I} \times \text{Area of colum}.$$

Story - 1

$$C_L = \frac{25 \times 15}{750A} \times A = 0.5$$

$$C_C = \frac{25 \times 5}{750A} \times 3A = 0.5$$

$$C_R = \frac{25 \times 15}{750A} \times 2A = -1.$$

Story - 2

$$C_L = \frac{125 \times 15}{750A} \times A = 2.5$$

$$C_L = \frac{125 \times 5}{750A} \times 3A = 2.5$$

$$C_R = \frac{125 \times 15}{750A} \times 2A = -5$$

Story - 3

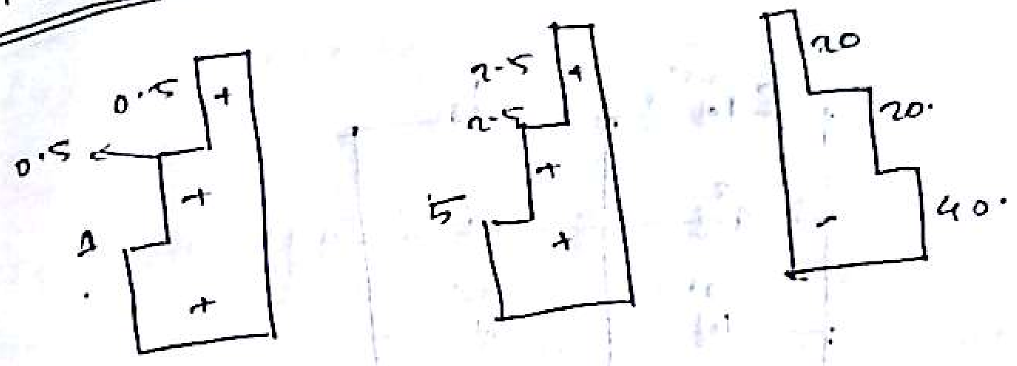
$$C_L = \frac{500 \times 15}{750} \times 2A = 20.$$

$$C_C = 20.$$

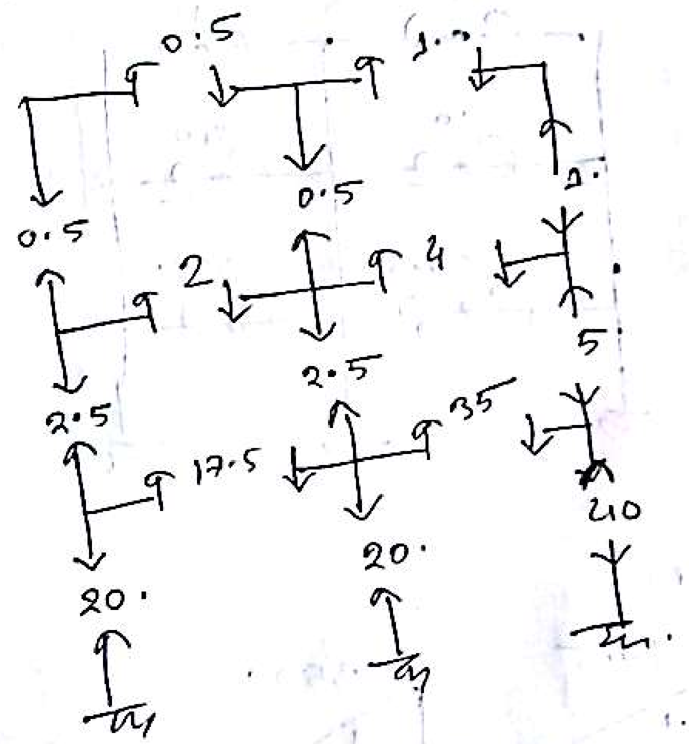
$$C_R = -40.$$

AF

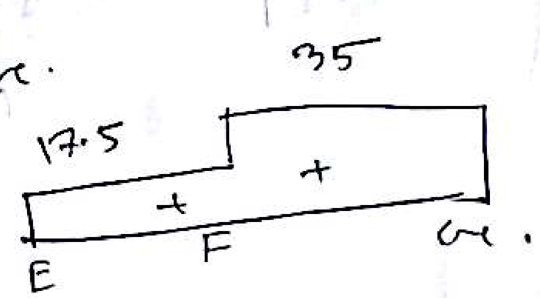
AFD columns



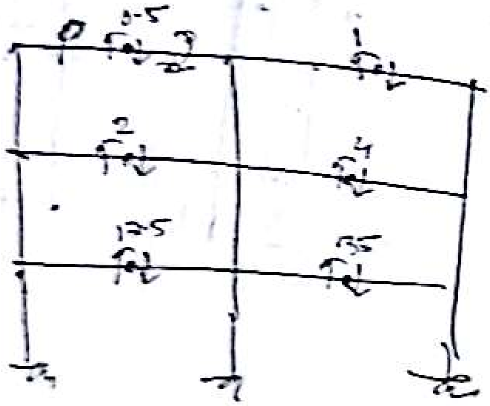
Beam SF.



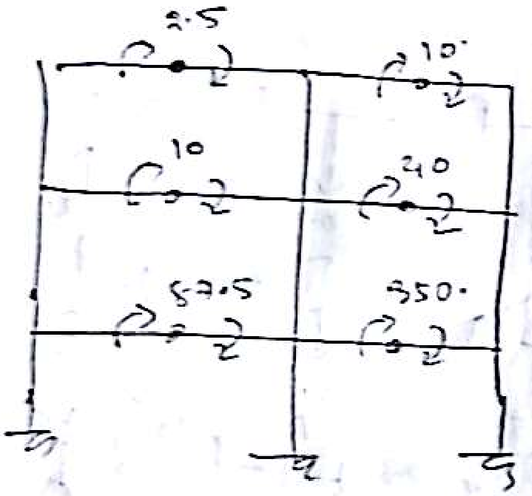
SFD → for EFG.



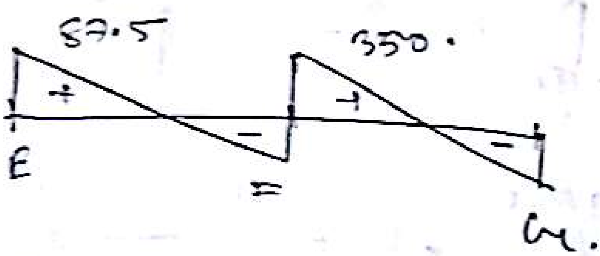
BMD for beam.



M= 2V2/2



BMD for EFG.



12-13
11 → Same type.

11-12

10-11

9-10

8-9

→ SAD-2

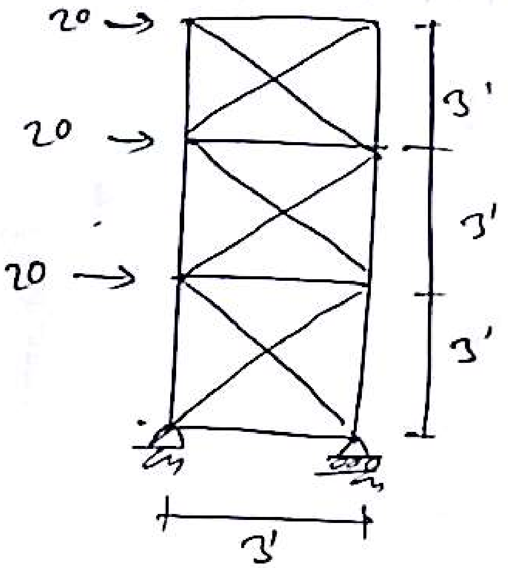
10-11 → (2). Same type.
9-10 → (10)
8-9 → (12)
7-8 → (13)
6-7 → (13) (reverse direction).

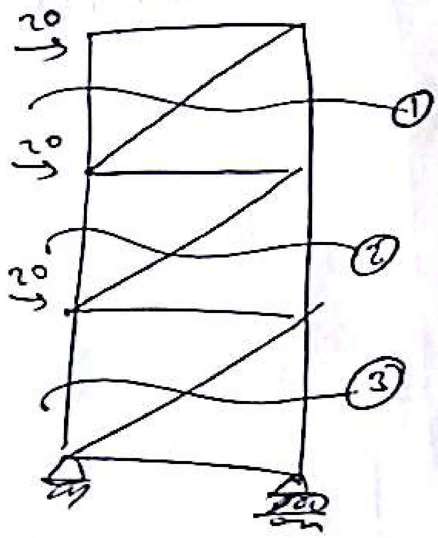
Approximate method:

X braced frames:

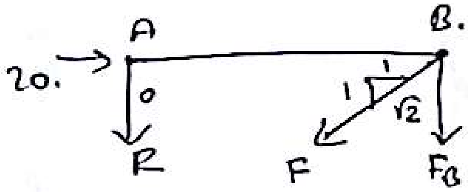
Section

Method-1 Assumption → Diagonal can carry tension only.





Section ①

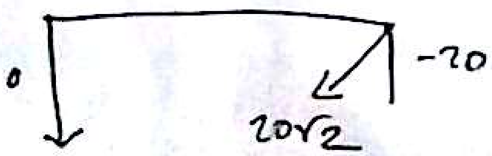


$\Sigma F_x = 0.$

$\Rightarrow 20 - F\,\frac{1}{\sqrt{2}} \Rightarrow F = 20\sqrt{2}$

$\Sigma M_B = 0. \Rightarrow R = 0.$

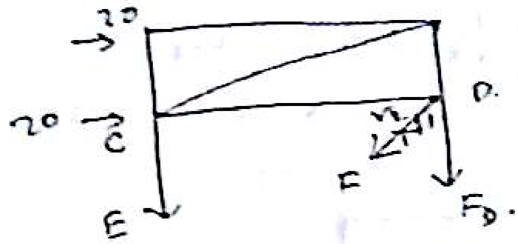
$\Sigma F_y = 0.$



$\Rightarrow R + F_B + \frac{1}{\sqrt{2}} F = 0.$

$\Rightarrow F_B = -20.$

Section 2



$\Sigma F_x = 0 \Rightarrow 20 + 20 - F\frac{1}{\sqrt{2}} \Rightarrow F = 40\sqrt{2}$

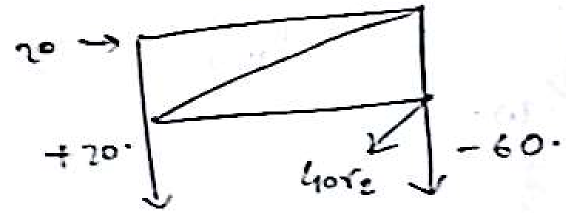
$\Sigma M_D = 0$

$\Rightarrow E \times 3 + 20 \times 3 = 0 \Rightarrow E = +20.$

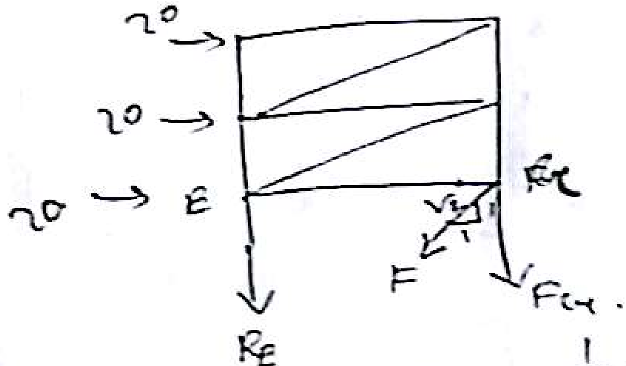
$\Sigma F_y = 0.$

$\Rightarrow 20 + F_D + \frac{40\sqrt{2}}{\sqrt{2}} = 0$

$\Rightarrow F_D = -60.$



Section 3



$\Rightarrow \Sigma F_x = 0 \Rightarrow 60 - \frac{1}{\sqrt{2}} \cdot F \Rightarrow F = 60\sqrt{2}$

$\Sigma M_G = 0.$

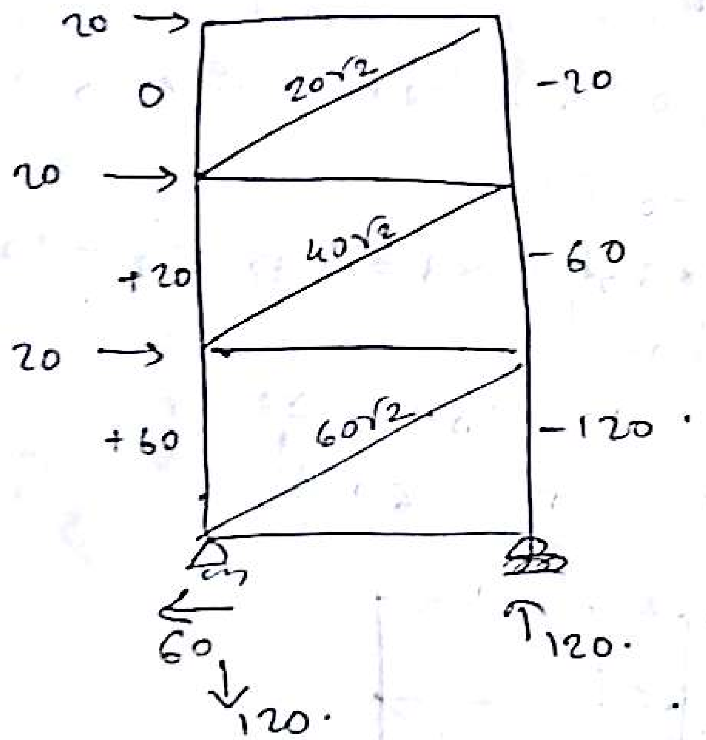
$\Rightarrow R_E \times 3 - 20 \times 0 + 20 \times 3 - 20 \times 6 = 0.$

$\Rightarrow R_E = +60$

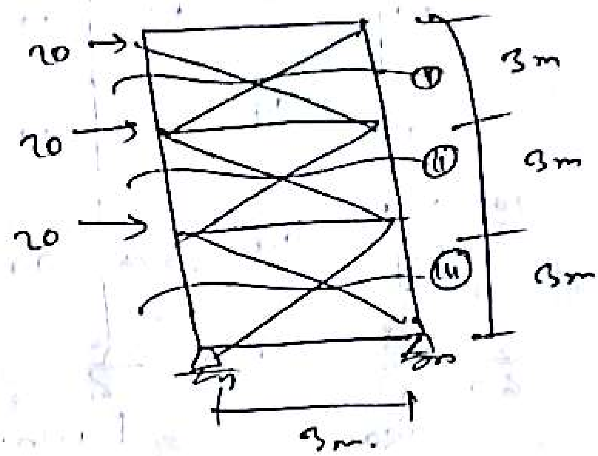
$\Sigma F_y = 0$

$\Rightarrow \quad 60 + \frac{1}{\sqrt{2}} \times 60\sqrt{2} + F_{ue} = 0$

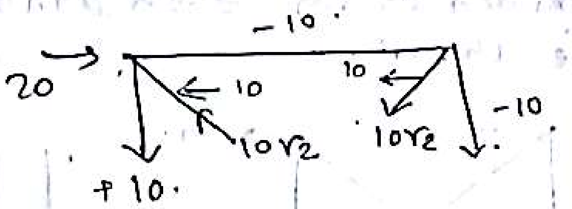
$\Rightarrow F_{ue} = -120.$



Method - 2 For both tension and compression

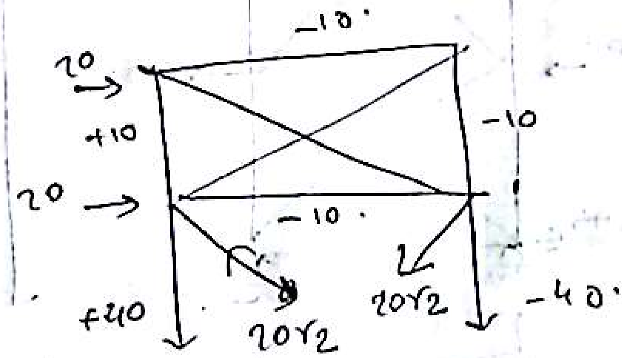


Section ①

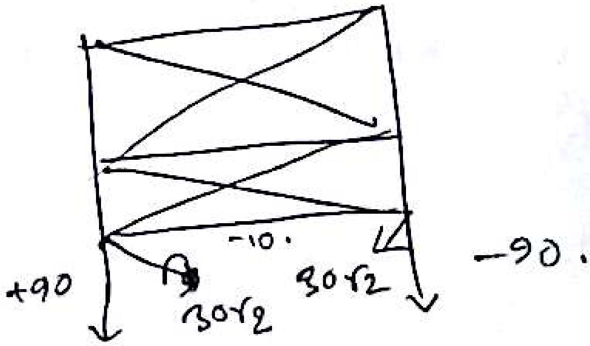


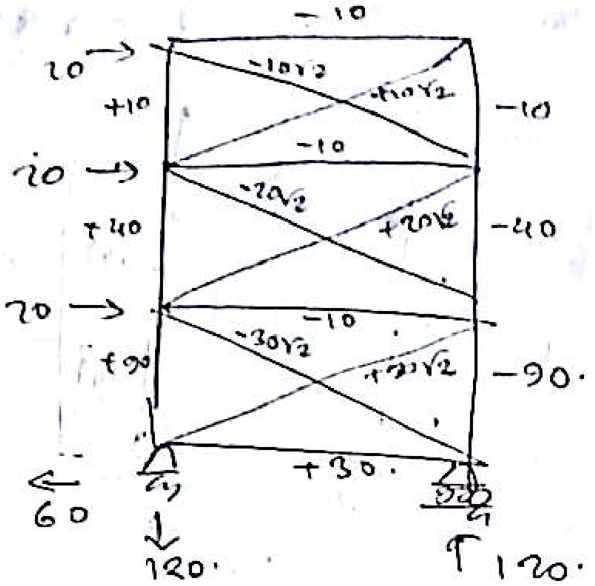
Shear divided equally.

Section ②

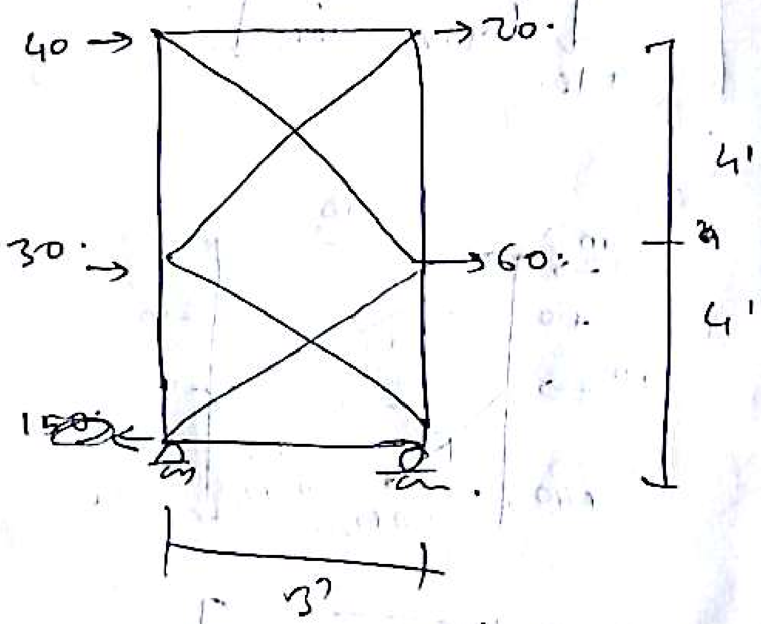


Section - ③

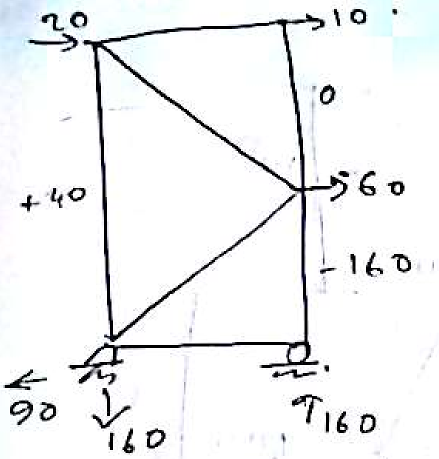


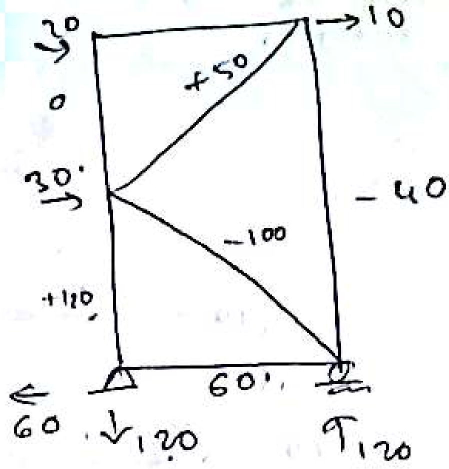


Method-3 if member dividing each panel are absent.



⇒

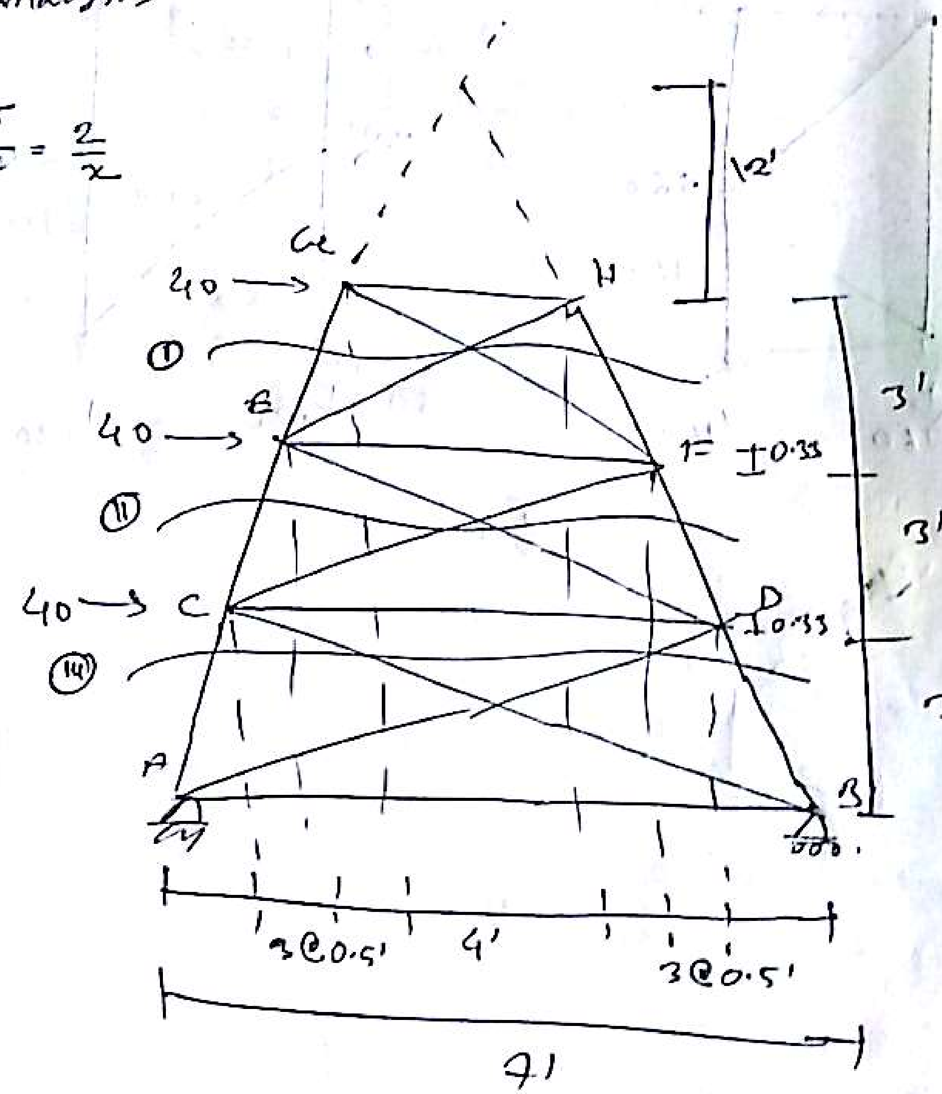




? how.

find the member force using approximate analysis

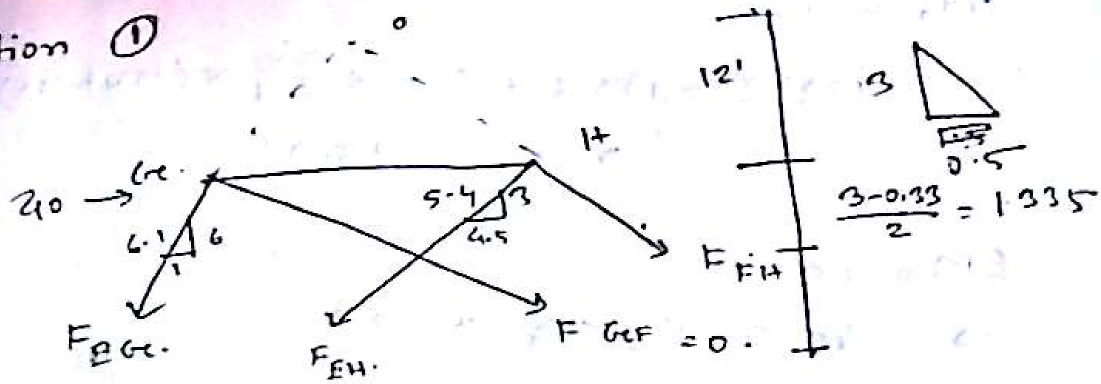
$$\frac{3.5}{9+x} = \frac{2}{x}$$



$$\frac{9+x}{7} = \frac{x}{4}$$

$$\Rightarrow 36 + 4x = 7x \Rightarrow x = 12'$$

assuming diagonals can carry only tension for Section (I)



$\Sigma M_o = 0 \Rightarrow -40\times12 + F_{EH}\times\frac{4.5}{5.4}\times(12+1.33)$

$\Rightarrow F_{EH} = 43.2^K$

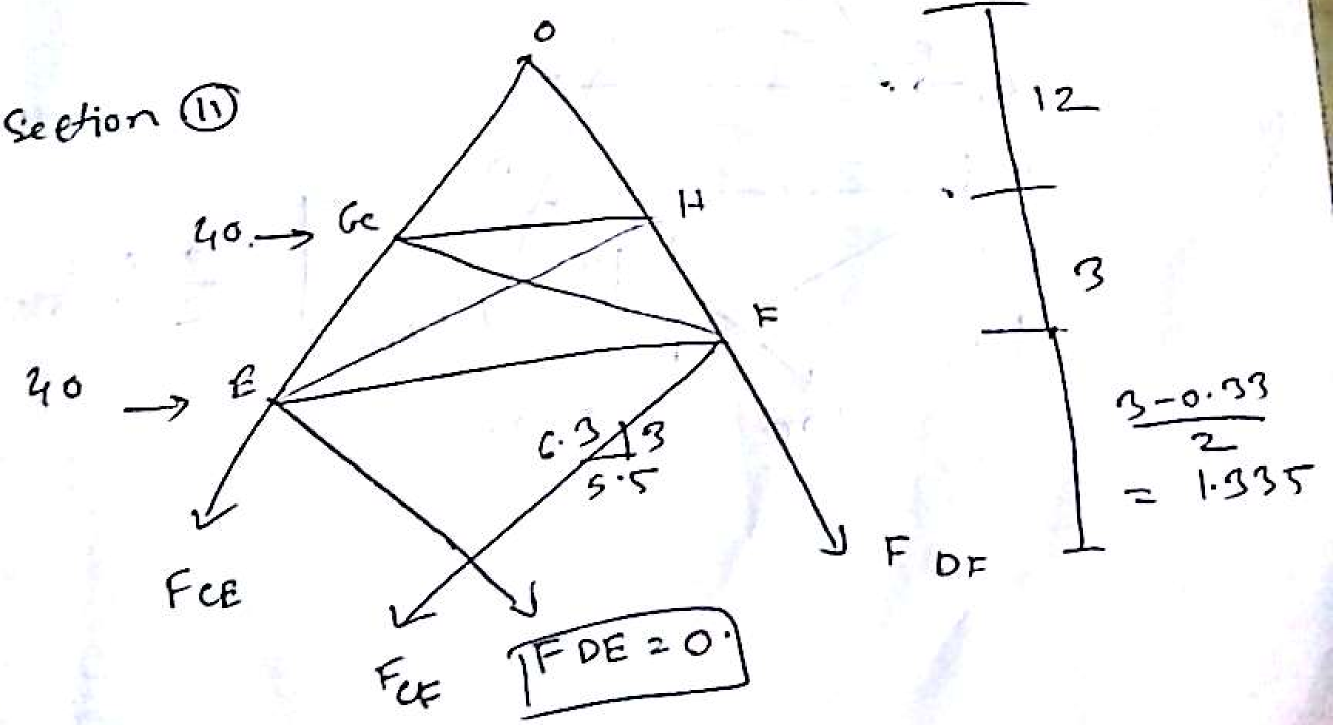
$F_{GF} = 0$ (only tension)

$\Sigma M_H = 0 \Rightarrow F_{EG}\times\frac{6}{6.1}\times4 = 0 \Rightarrow F_{EG} = 0$

$\Sigma F_y = 0 \Rightarrow 43.2\times\frac{3}{5.4} + F_{FH}\times\frac{6}{6.1} = 0$

$\Rightarrow F_{FH} = -24.4^K$

for Section (II)



$\Sigma M_o = 0$

$\Rightarrow \quad -40\times12 - 40\times12 + F_{CF}\times\frac{5.5}{6.3}\times(15+1.334)$

$\Rightarrow F_{CF} = 75.7\text{K}$

$\Sigma M_F = 0$

$\Rightarrow \quad 40\times3 - F_{CE}\times\frac{6}{61}\times5 = 0$

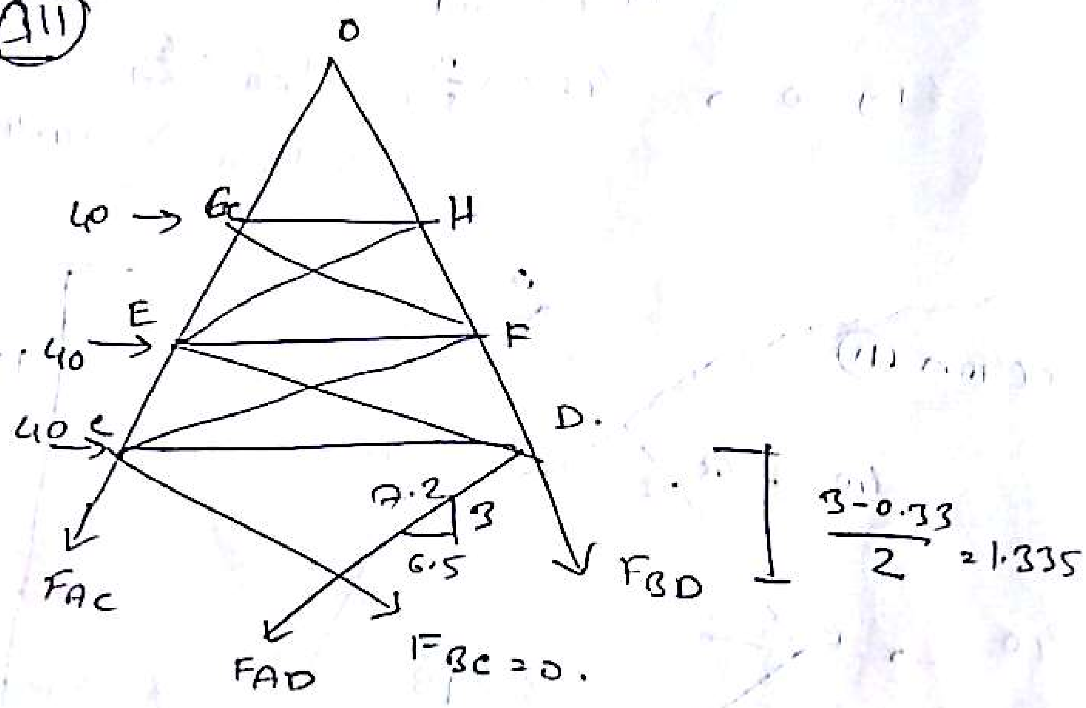
$\Rightarrow F_{CE} = 24.4\text{K}$

$\Sigma F_y = 0$

$24.4\times\frac{6}{6.1} + 75.7\times\frac{3}{6.3} + F_{DF}\times\frac{6}{6.1} = 0$

$\Rightarrow F_{DF} = -61\text{ K}$

Section-(III)



$\Sigma M_o = 0$

$\Rightarrow -40\times12 - 40\times15 - 40\times18 + F_{BC}\times(18+1.335)\times\frac{6.5}{7.2} = 0$

$\Rightarrow F_{BC} = +103.12k.$

$\Sigma M_D = 0$

$\Rightarrow 40\times3 + 40\times6 - F_{AC}\times\frac{6}{6.1}\times6 = 0$

$\Rightarrow F_{AC} = 61K$

$\Sigma F_y = 0$ $61\times\frac{6}{61} + 103.12\times\frac{3}{7.2} + F_{BD}\times\frac{6}{6.1} = 0$

$\Rightarrow F_{BD} = -104.7K$

Joint E.

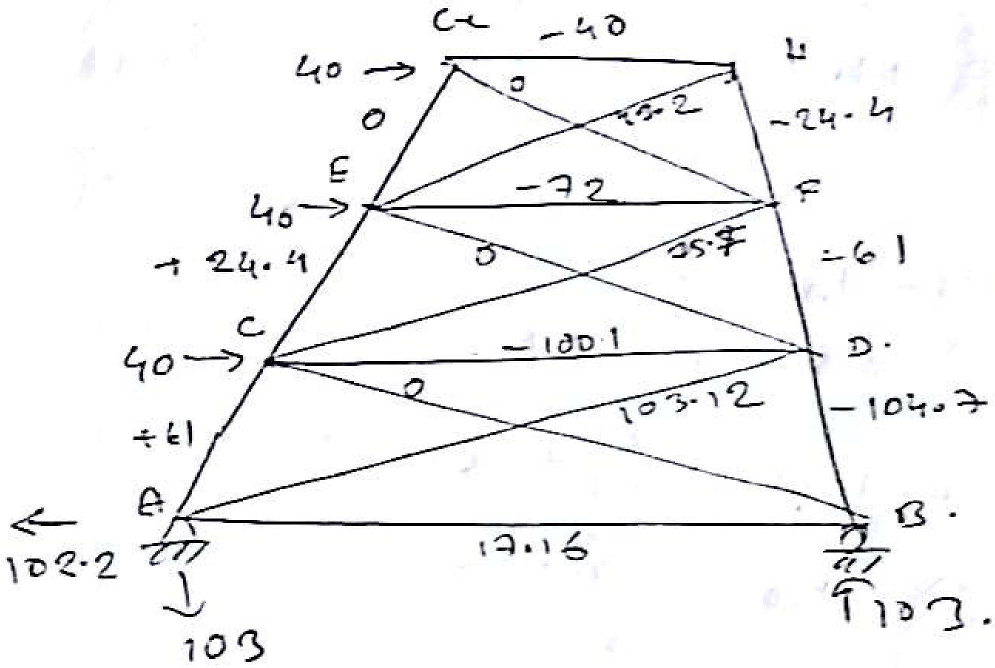
$40 + 43.2\times\frac{4.5}{5.4} - 24.4\times\frac{1}{6.1} + F_{EF} = 0$

$\Rightarrow F_{EF} = -72$

Joint C

$40 + 24.4\times\frac{1}{6.1} + 75.7\times\frac{5.5}{6.3} - 61\times\frac{1}{6.1} + F_{CD} = 0$

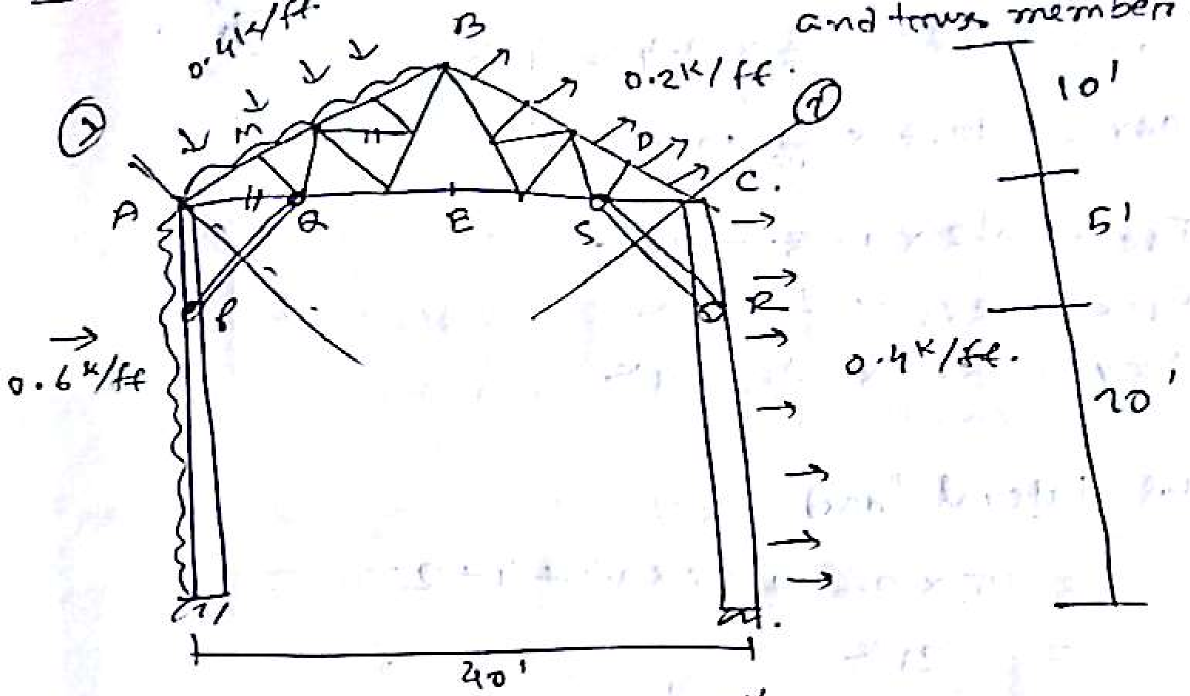
$\Rightarrow F_{CD} = -100K$

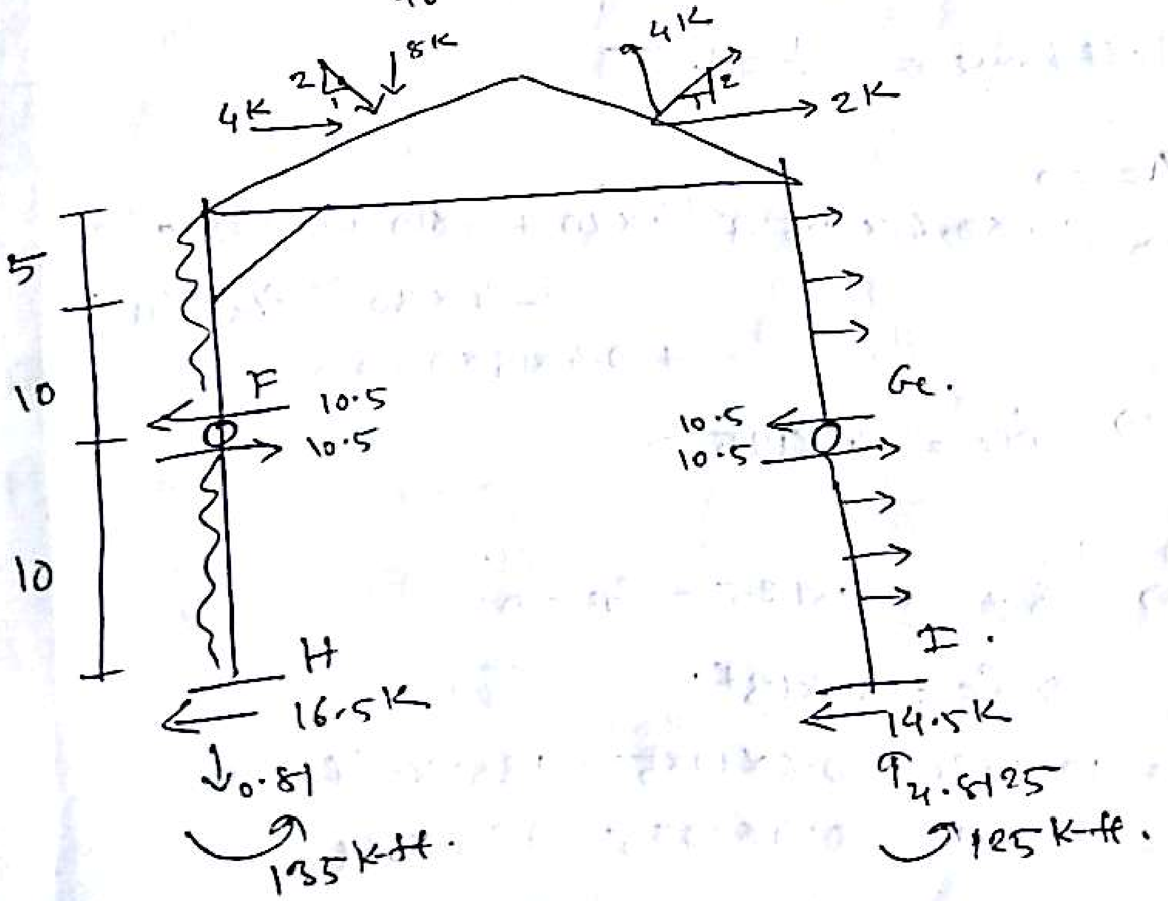


G
-40
H
40
0
0
-24.4
E
40
-72
F
+24.4
0
-61
C
40
-100.1
D
0
103.12
+61
-104.7
A
B
102.2
17.16
103
103

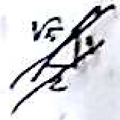
Mill Bent:

SF, BM for column and Bar force in Knee Brace and truss member.





Length of AB = $\sqrt{10^2 + 20^2} = 10\sqrt{5}$



$F_{AB} = 10\sqrt{5} \times \frac{1}{\sqrt{5}} \times 0.4 = 4K$

$F_{ABY} = 10\sqrt{5} \times \frac{2}{\sqrt{5}} \times 0.4 = 8K$

$F_{BC} = 0.2 \times 10\sqrt{5} = 2\sqrt{5}$

$F_{BCX} = 2\sqrt{5} \times \frac{1}{\sqrt{5}} = 2K$

$F_{BCY} = 2\sqrt{5} \times \frac{2}{\sqrt{5}} = 4K$

Total lateral load

$= 15 \times 0.6 + 15 \times 0.4 + 4 + 2$

$= 21^K$

Left hinge $= \frac{21}{2} = 10.5$

$\Sigma M_F = 0.$

$\Rightarrow 15 \times 0.6 \times 7.5 + 4 \times 20 + 8 \times 10 + 2 \times 20 - 4 \times 30 - R_{Ce} \times 41 + 0.4 \times 15 \times 7.5 = 0$

$\Rightarrow R_{Ce} = 4.8125^K$

$\Sigma F_y = 0.$

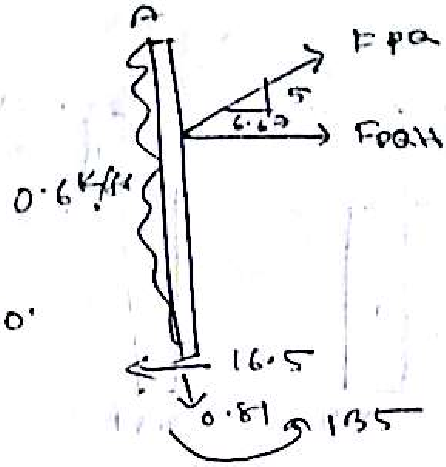
$\Rightarrow -8 + 4 + 4.8125 - R_F = 0.$

$\Rightarrow R_F = 0.813^K.$

$M_H = 10.5 \times 10 + 0.6 \times 10 \times 5 = 135$ K-ft

$M_I = 10.5 \times 10 + 0.4 \times 10 \times 5 = 125$ K-ft

Section - 1



$\Sigma M_A = 0$.

$\Rightarrow -F_{PQH} \times 5 - 0.6 \times 25 \times \frac{25}{2}$

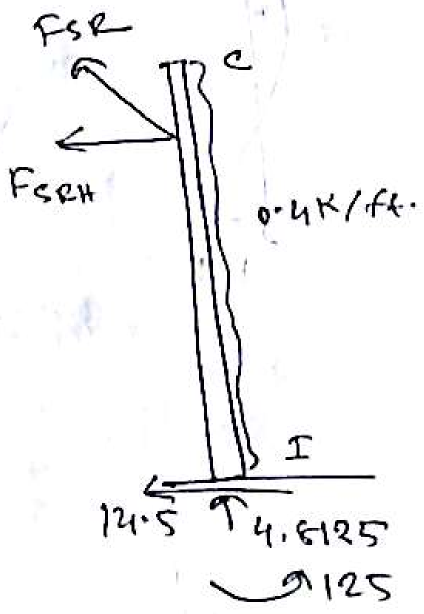
$- 135 + 16.5 \times 25 = 0$.

$\Rightarrow F_{PQH} = 18^K \ (\rightarrow)$

$F_{PQV} = 18 \times \frac{5}{6.67} = 13.5^K (\uparrow)$

$F_{PQ} = \sqrt{13.5^2 + 18^2} = 22.5^K \ (\nearrow)$

Section - 2



$\Sigma M_C = 0$.

$\Rightarrow F_{SRH} \times 5 + 14.5 \times 25 - 125$

$- 0.4 \times 25 \times \frac{25}{2} = 0$.

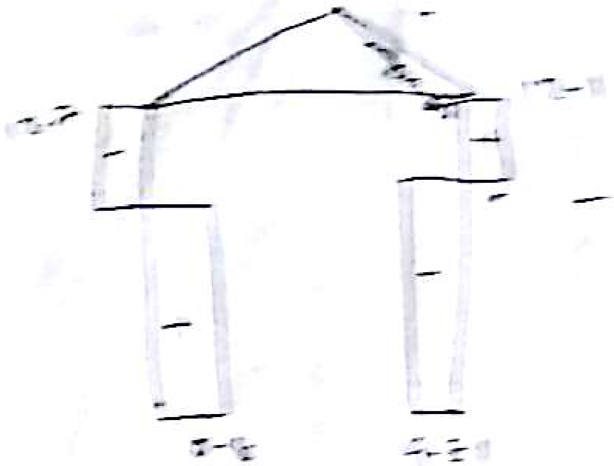
$\Rightarrow F_{SRH} = 22.5^K (\rightarrow)$

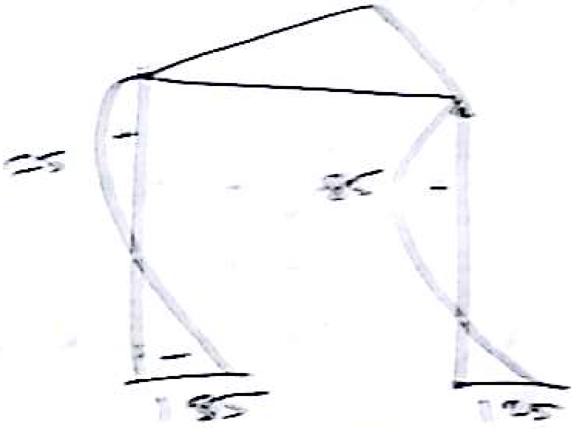
& $F_{SRV} = 22.5 \times \frac{5}{6.67}$

$= 16.87^K (\downarrow)$

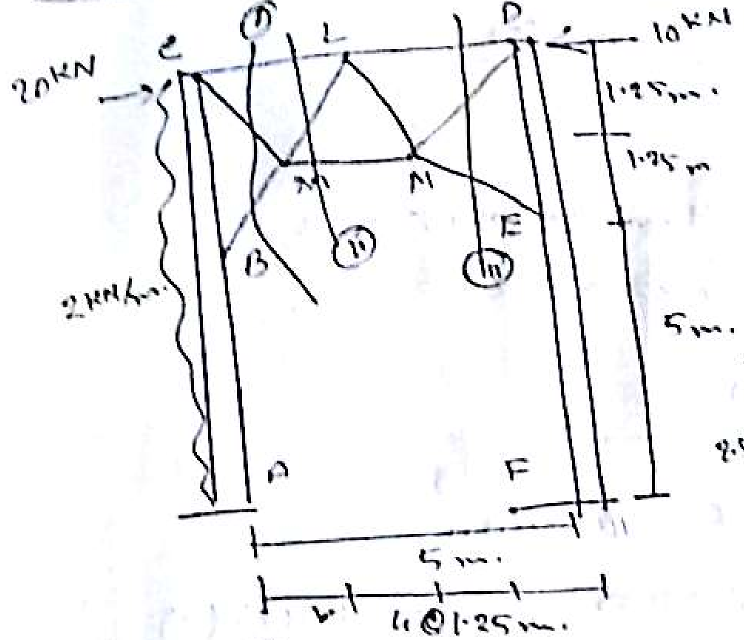
So $F_{SR} = \sqrt{22.5^2 + 16.87^2}$

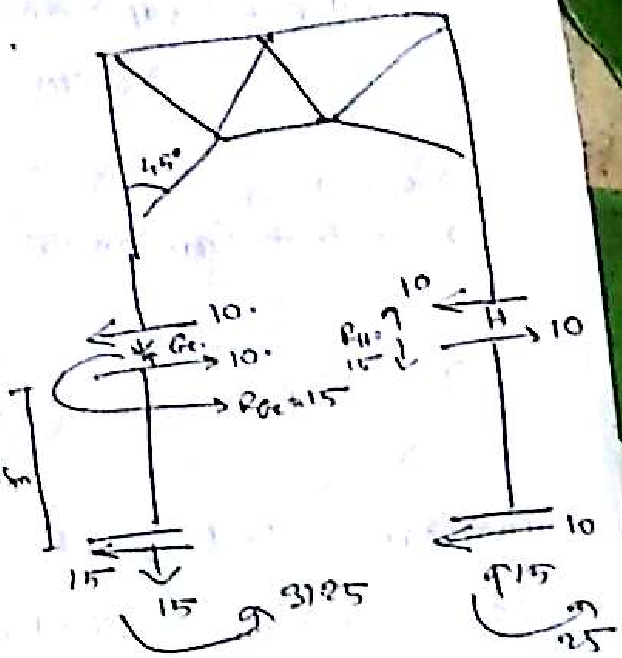
$= 28.12^K$ (C)





Bridge portal:





$\Sigma M_G = 0$

$\Rightarrow 20\times5 - 10\times5 + 2\times5\times2.5 - R_H\times5 = 0$

$\Rightarrow R_H = 15\,KN$

Total lateral force $= 20-10+2\times5 = 20$

$\Sigma M_A = 0 \Rightarrow 10\times2.5 + 2\times2.5\times\frac{2.5}{2} - M_A = 0$

$\Rightarrow M_A = 31.25\ K\text{-}ft$

$\Sigma M_F = 0 \Rightarrow 10\times2.5 - M_F = 0$

$\Rightarrow M_F = 25\ K\text{-}ft$

Section ①

$\Sigma M_e = 0 \Rightarrow -F_{BM}\sin45\times2.5 - 31.25 + 15\times2.5 - 2\times2.5\times\frac{2.5}{2} = 0$

$\Rightarrow F_{BM} = 14.14\ KN\ (T)$

$\Sigma F_y = 0$

$\Rightarrow -15 + F_{BM} \times \sin 45 - F_{CM} \times \sin 45 = 0$

$\Rightarrow F_{CM} = -5\sqrt{2}$ (C)

$\Sigma F_x = 0$.

$\Rightarrow -15 + F_{BM} \sin 45 + F_{CM} \cos 45 + F_{CL} +$

$2 \times 7.5 + 20 = 0$

$\Rightarrow F_{CL} = -25^k$ (C)

Considering left part of (II) Section.

$\Sigma F_y = 0 \Rightarrow F_{ML} \cdot \sin 45 - 15 = 0 \Rightarrow F_{ML} = 21.21 K$ (T)

Considering Right part of (II) section.

$\Sigma M_L = 0 \Rightarrow F_{MN} \times 1.25 - 25 - 15 \times 25 + 10 \times 7.5 = 0$

$\Rightarrow F_{MN} = -10 KN$ (C)

Considering right part of section (III)

$\Sigma M_D = 0 \Rightarrow F_{EN} \times \cos 45 \times 2.5 - 25 + 10 \times 7.5 = 0$

$\Rightarrow F_{EN} = -28.3 K$ (C)

$\Sigma F_y = 0 \Rightarrow 15 - 28.3 \cos 45 - F_{DN} \times \cos 45 = 0$.

$\Rightarrow F_{DN} = -7.07 K$ (C)

$\Sigma F_x = 0 \Rightarrow -10 - 10 + 28.3 \times \cos 45 + 7.07 \times \cos 45$

$+ F_{LD} = 0$.

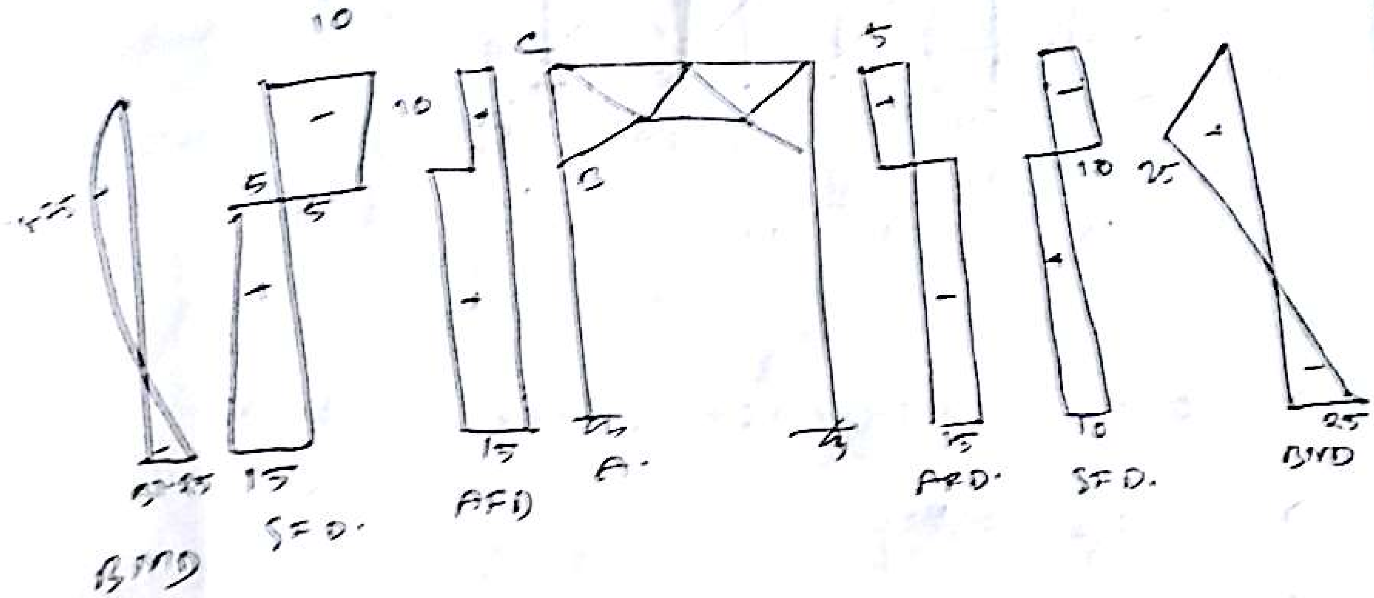
$\Rightarrow F_{LD} = 5 KN$ (T)

at joint L.

$\Sigma F_y = 0 \Rightarrow F_{ML} \cos 45 + F_{LN} \cos 45 = 0$

$\Rightarrow F_{ML} = - F_{LN}$

$\Rightarrow F_{LN} = -21.21$ kN. (c)

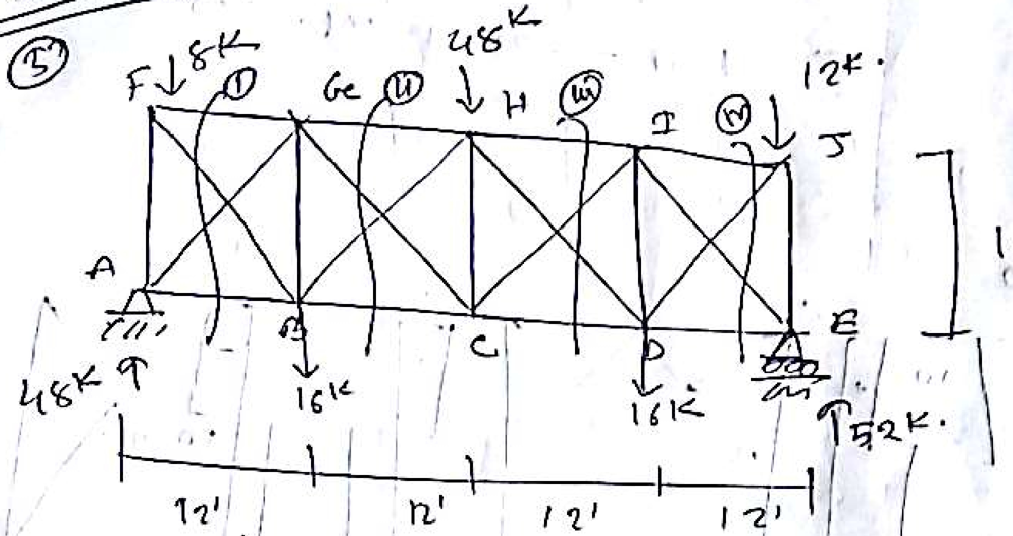


Q Solve

~~2014-15~~

2015-16

(5)



$\sum M_A = 0$

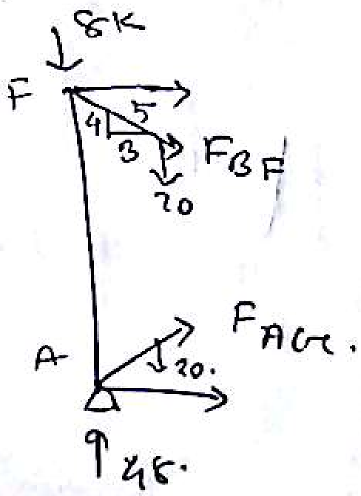
$\Rightarrow 16 \times 12 + 48 \times 24 + 16 \times 36 + 12 \times 48 - R_E \times 48 = 0$

$\Rightarrow R_E = 52^K$

$\sum F_y = 0 \Rightarrow R_A = 48\,K$

$\sum F_x = 0 \Rightarrow R_{Ax} = 0$

From section ①



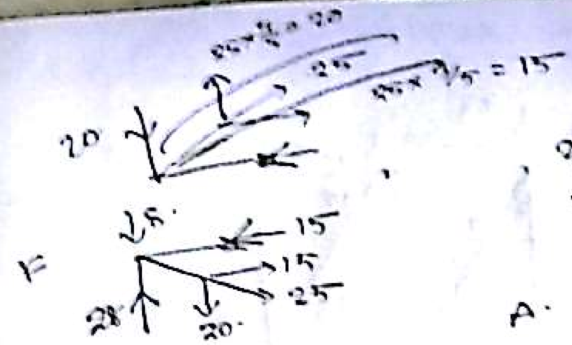
~~$\sum F_y$~~

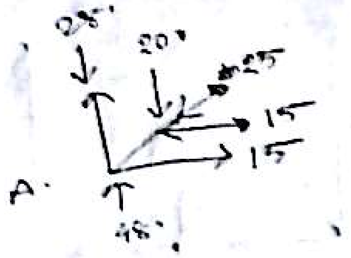
$20 = F_{BF} \times \frac{4}{5}$

$\Rightarrow F_{BF} = 25\,K\ (T)$

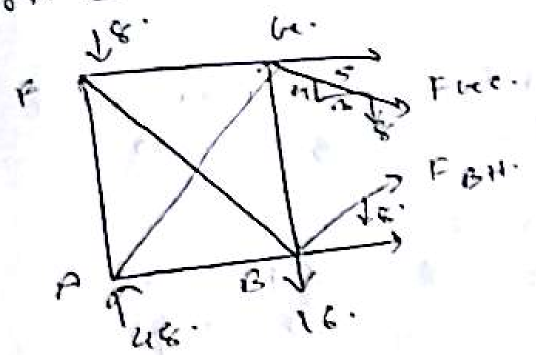
$20 = F_{AG} \times \frac{4}{5}$

$\Rightarrow F_{AG} = 25^K\ (C)$





Section (II)

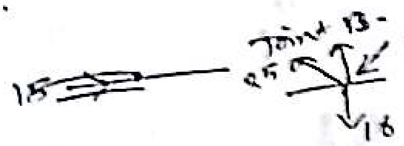


$8 = F_{GC} \times \frac{4}{5}$

$\Rightarrow F_{GC} = 10\ (T)$

$8 = F_{BH} \times \frac{4}{5}$

$\Rightarrow F_{BH} = 10\ (C)$

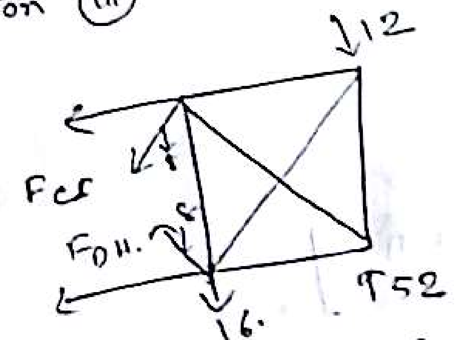


$\Sigma F_y = 0$

$\Rightarrow -16 - 10 + 25 + F = 0$

$\Rightarrow F = 1.$

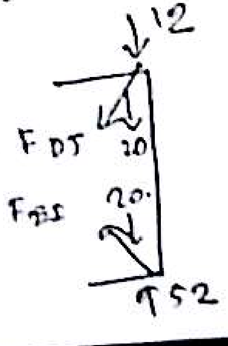
Section (III)



$\frac{4}{5} F_{CI} = 8 \Rightarrow F_{CI} = 10\ (T)$

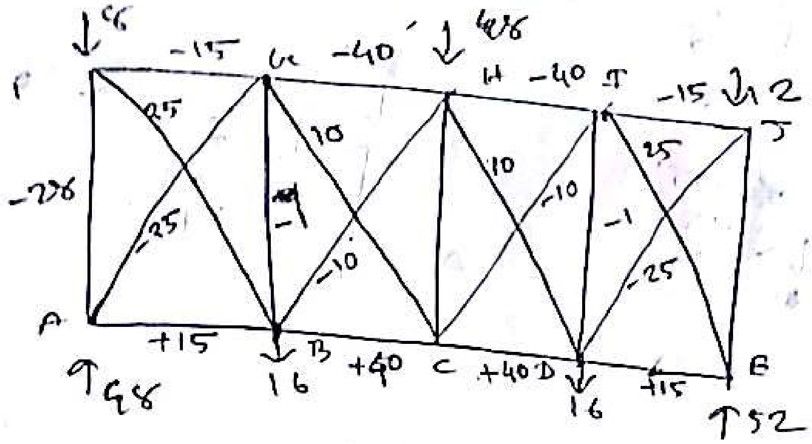
$\frac{4}{5} \times F_{DH} = 8 \Rightarrow F_{DH} = 10\ (C)$

Section. (IV)

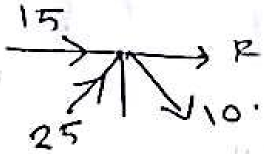


$F_{DJ} = 25\text{ kN}\ (T)$

$F_{EJ} = 25\text{ kN}\ (C).$



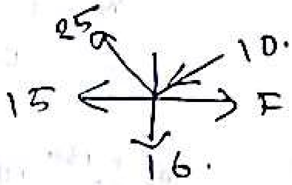
at G.



$15 + 25 \times \frac{3}{5} + F + 10 = 0$

$\Rightarrow F = -40$

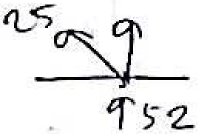
at B.



$-25 \times \frac{3}{5} - 15 - 10 + F = 0$

$\Rightarrow F = 40.$

at E

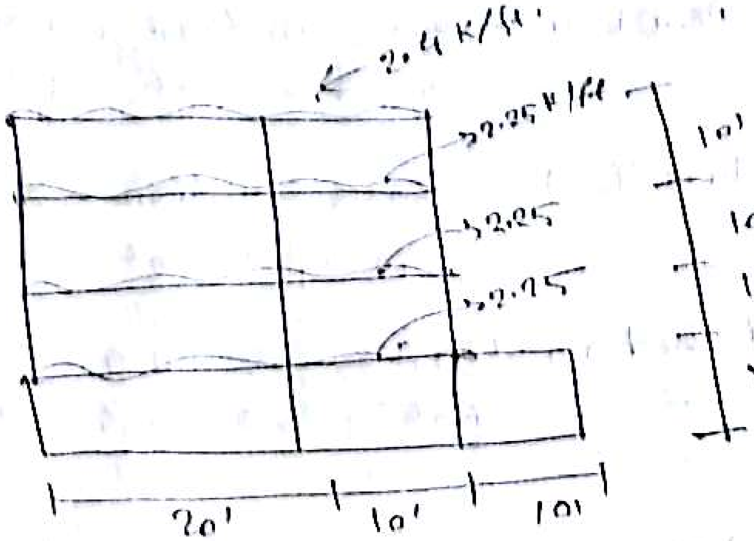


~~$52 + 25 \times \frac{4}{5}$~~

Truss শেষ হলো।

Approximate method for load.

Self Sheet



Spacing at 15' c/c.

Dead load

Floor 6" slab = 75 psf.
FF = 15 psf.
= 90 psf

Roof 7.2" slab = 90 psf.
LC = 30 psf.
120 psf

Live load

Floor = 60 psf

Roof = 40 psf.

Hence load on a Frame:

Dead load = 90×15 on floor = 1.35 k/ft
= 120×15 " Roof = 1.8 k/ft.

Live load = 60×15 " floor = 0.9 k/ft.
= 40×15 " Roof = 0.6 k/ft

total load on floor = 1.35 + 0.9 = 2.25 k/ft
" " " Roof = 1.8 + 0.6 = 2.4 k/ft

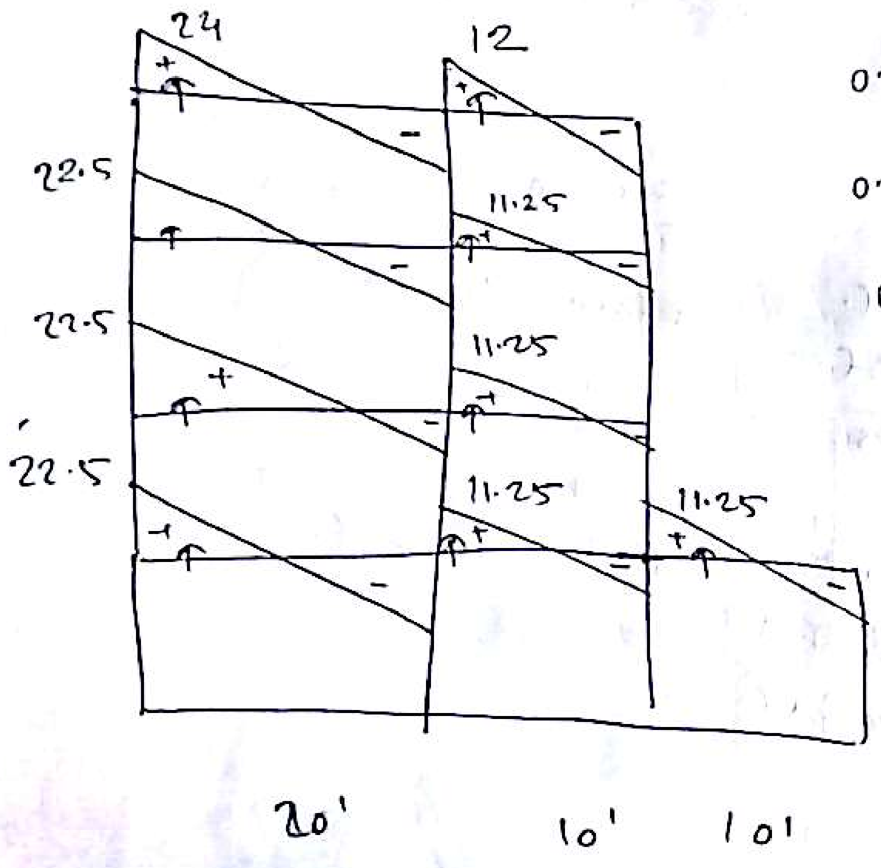
SF in beams = 0.5 wL.

0.5×2.4×20
= 24.

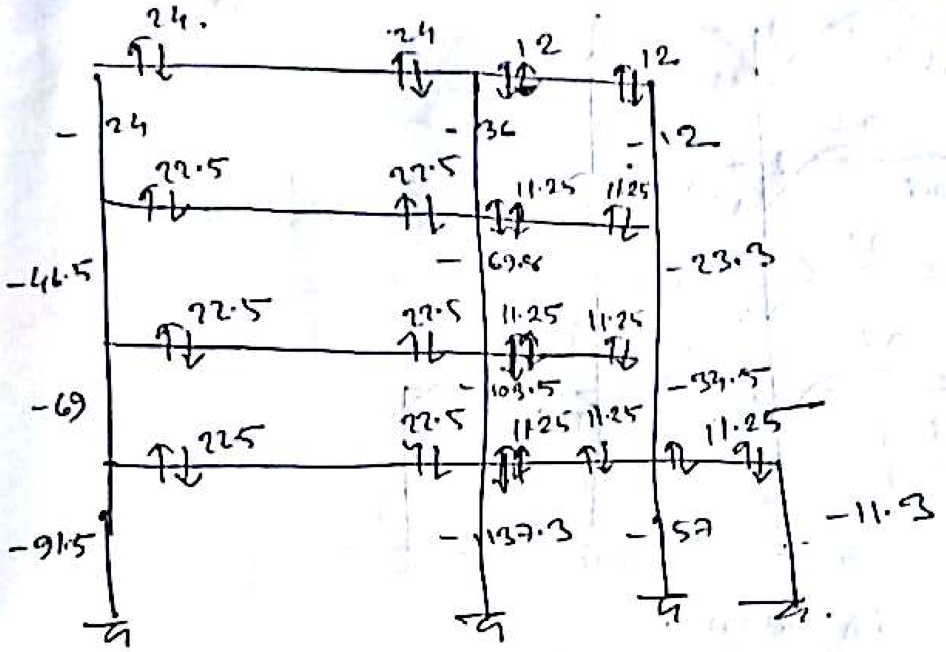
0.5×2.25×20
= 22.5

0.5×2.4×10
= 12

0.5×2.25×10
= 11.25



AF in cols.



BM in Beams

20' span

+ve BM = $0.08 wL^2$ = $0.08 \times 2.4 \times 20^2$ = 76.8 on roof.

= $0.08 \times 2.25 \times 20^2$ = 72 on floor.

-ve BM = $0.045 wL^2$

= $0.045 \times 2.4 \times 20^2$ = 43.2 on roof

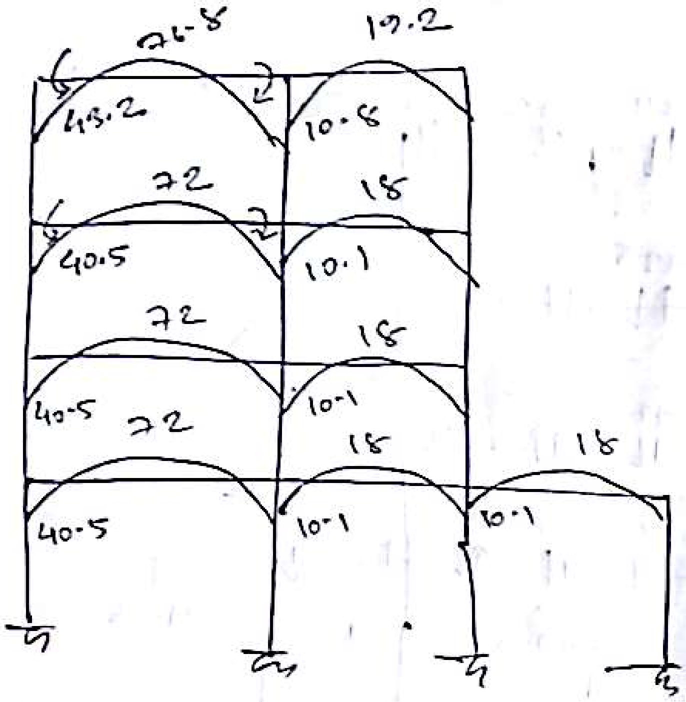
= $0.045 \times 2.25 \times 20^2$ = 40.5 on floor

16' span

+ve BM = 19.2 on roof
= 18 on floor

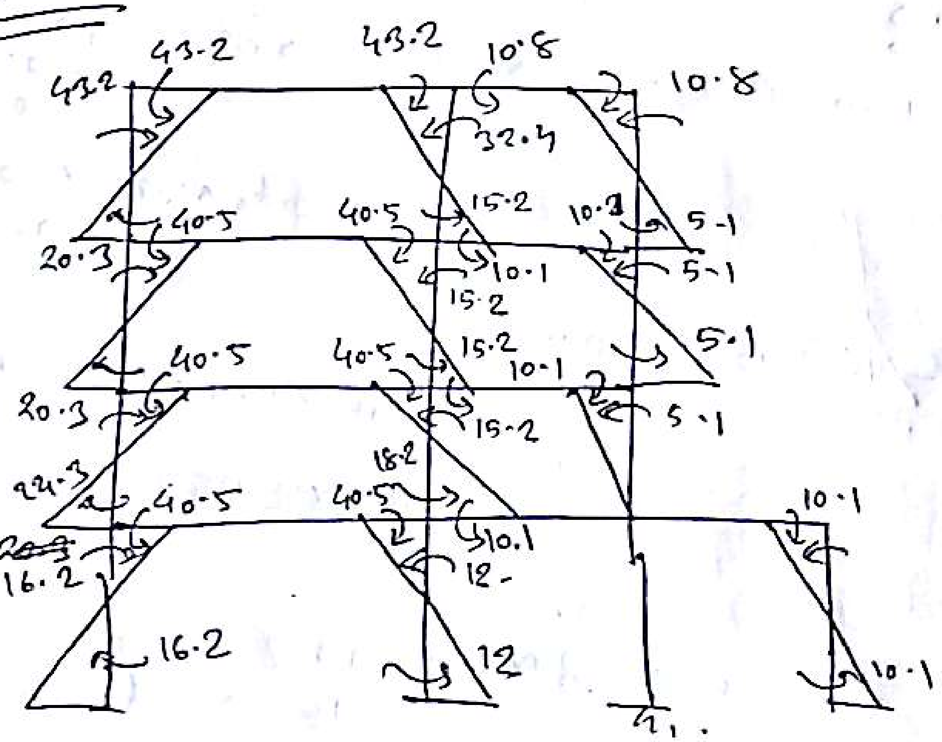
-ve " = 10.8 on roof
= 10.1 on floor

10 → 15.2
12 →



BM in beam.

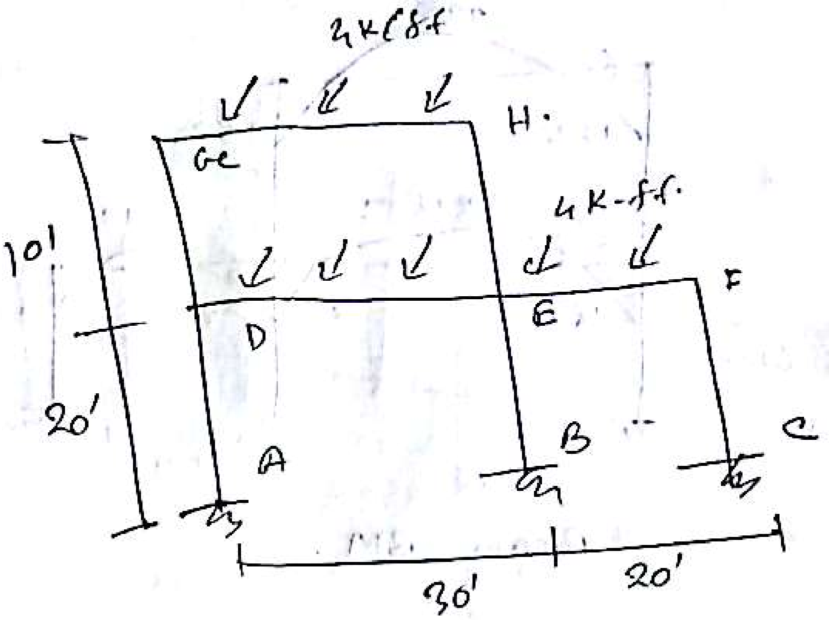
BM in cols



10 ft → 15.2 K-ft.
12' → 18.2 K-ft.

2015-16

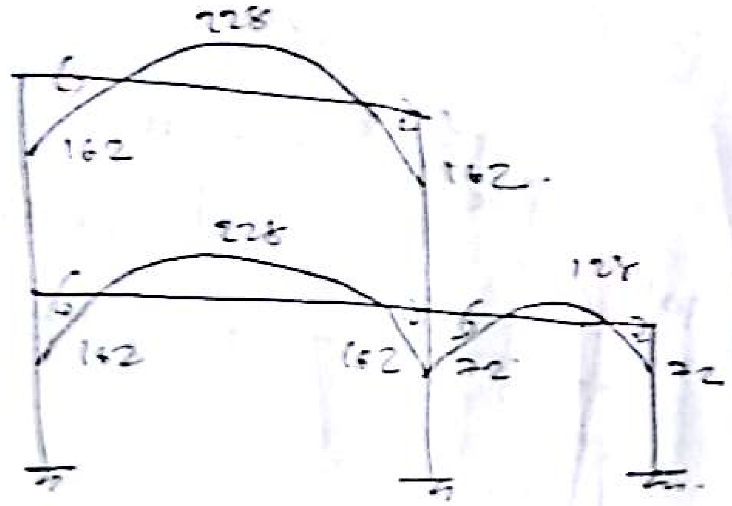
(1)



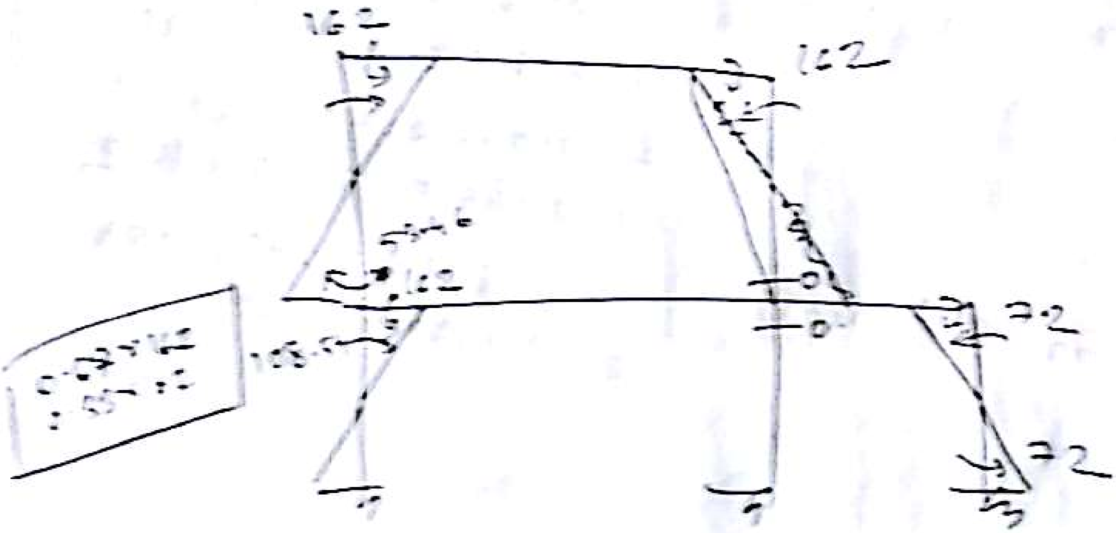
+ve for 30' section = 0.08 wL²
= 0.08 × 4 × 30² = 288.

-ve " " " = 0.045 wL²
= 0.045 × 4 × 30² = 162

+ve for 20' section = 0.08 × 4 × 20² = 128.
-ve " " = 0.045 × 4 × 20² = 72

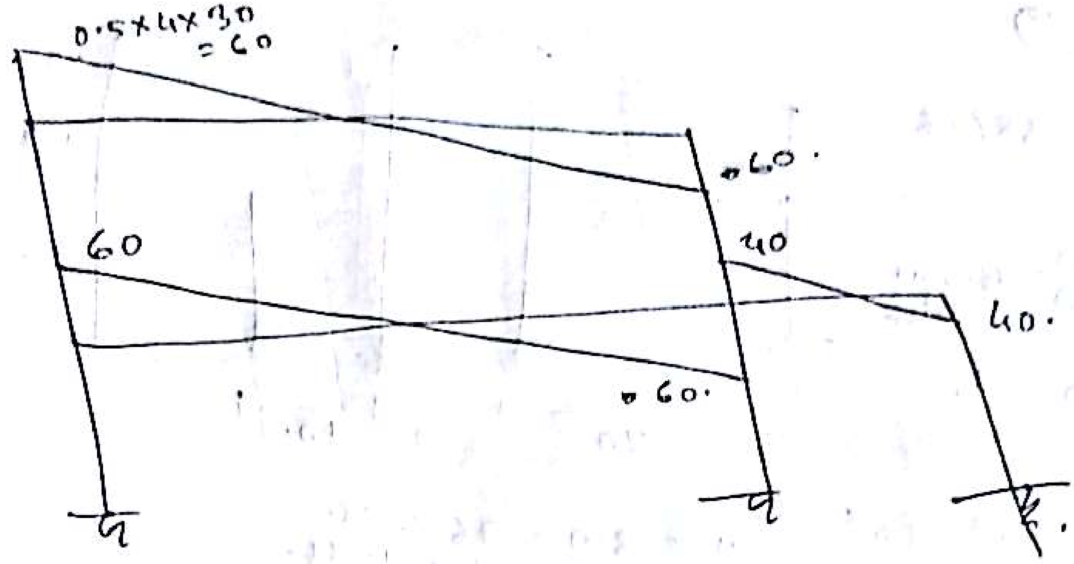


Beam BM.

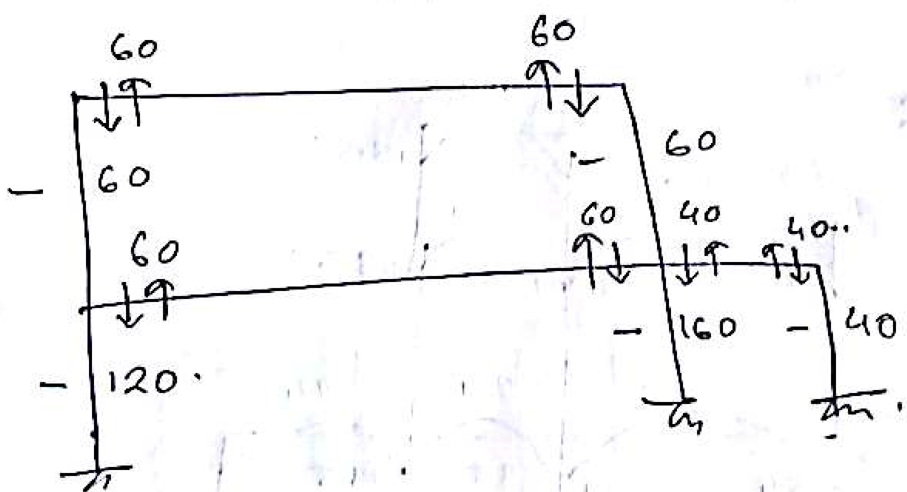


Col BM.

SF in beam = 0.5WL



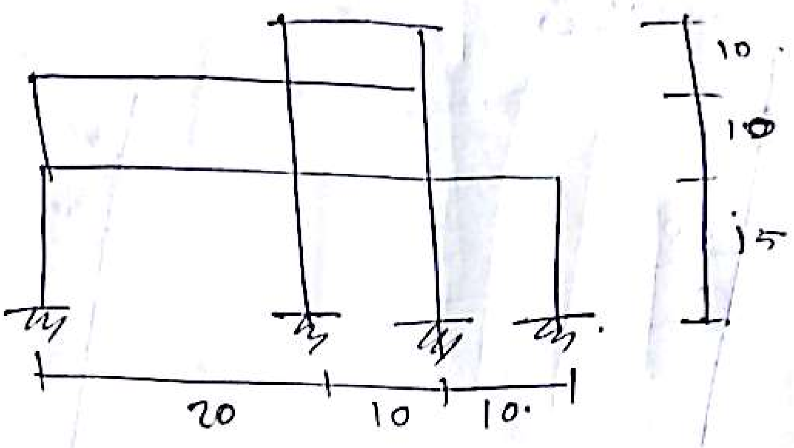
AF in cols.



2014-15

(2)

3K/ft



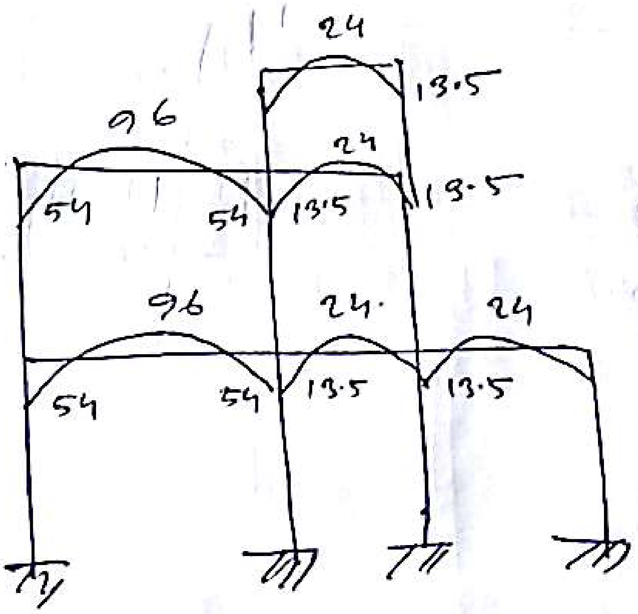
BMD > Beam+
SFD > colm.

AF → col.

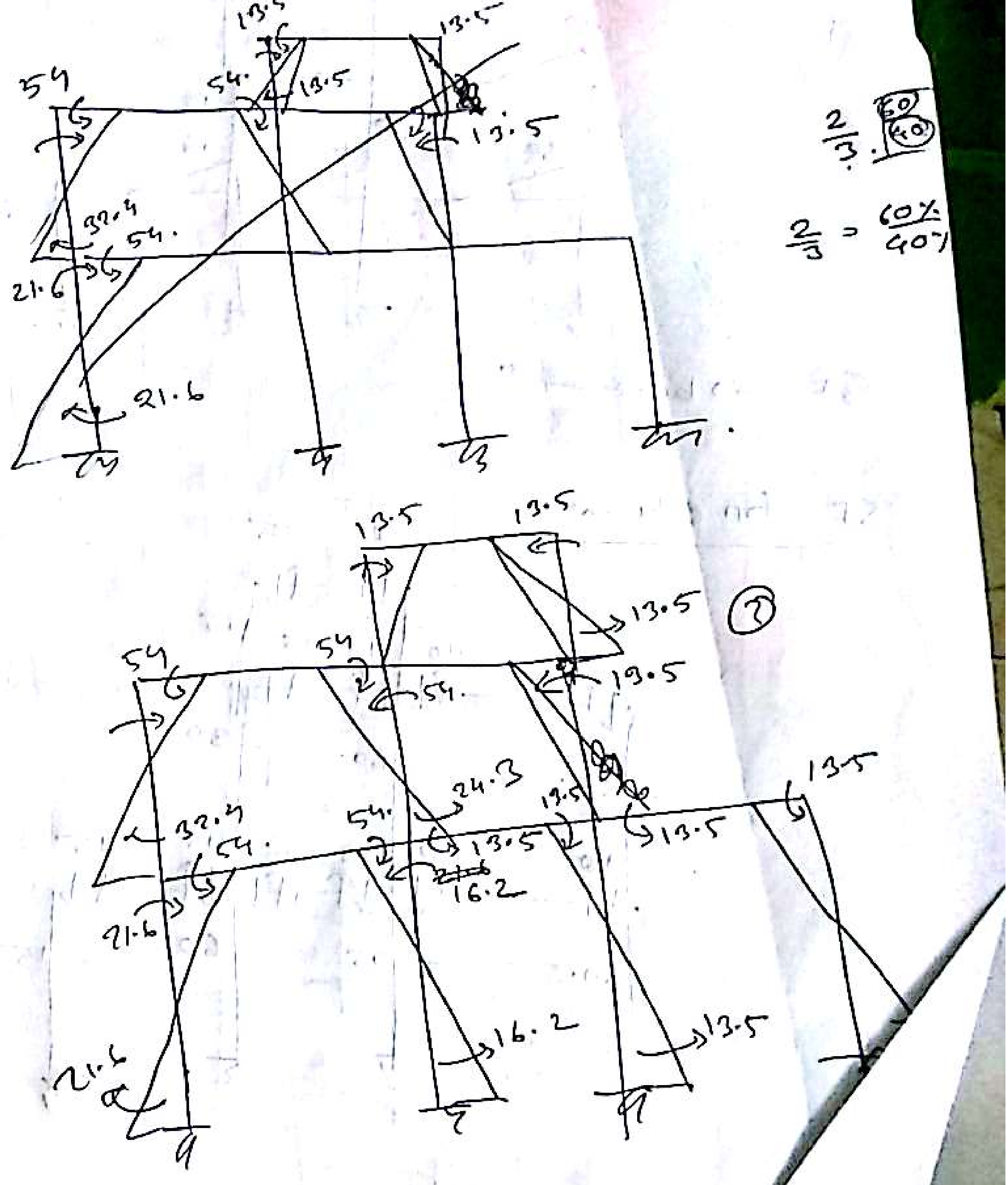
at 20'   +ve BM = 96 K-ft.
         −ve BM = 54 K-ft.

at 10'   +ve BM = 24.
                = 13.5

BM for beam



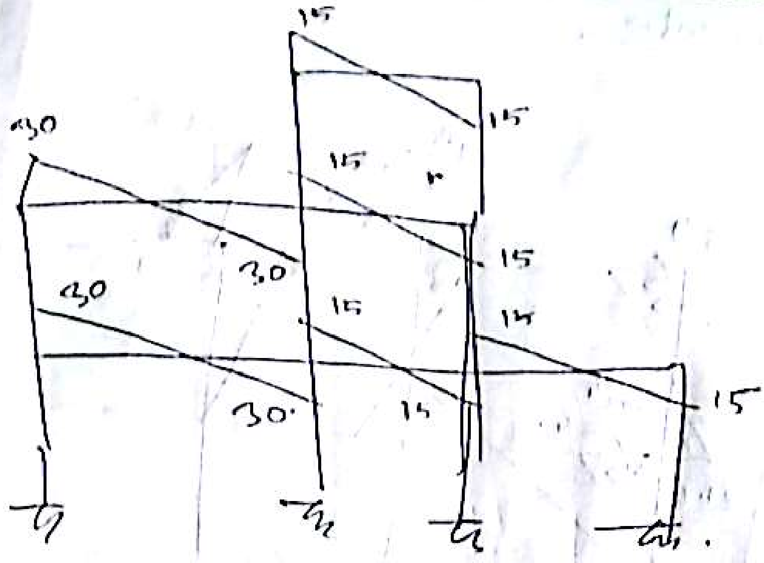
BM for · colm



$\frac{2}{3}$. 60 / 40

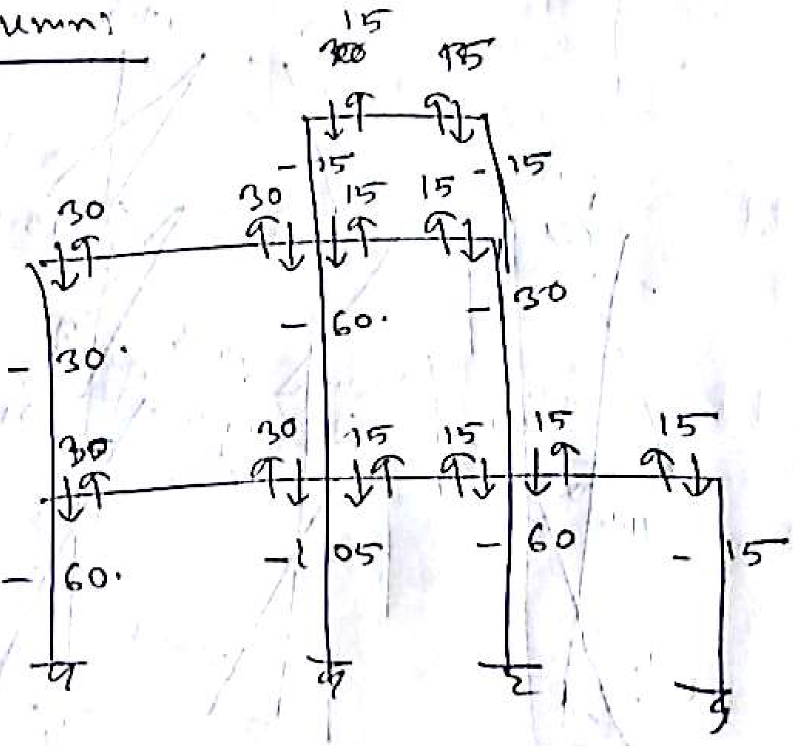
$\frac{2}{3}$ = $\frac{60\%}{40\%}$

SF = 0.5wL



SF → beam →

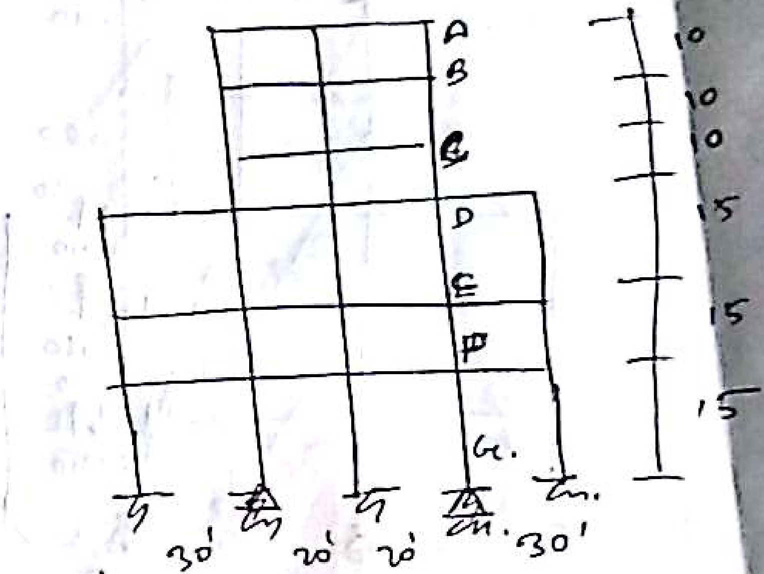
SF for column



2013-14

(10)

2K/ft | AFD | BMD



for AF

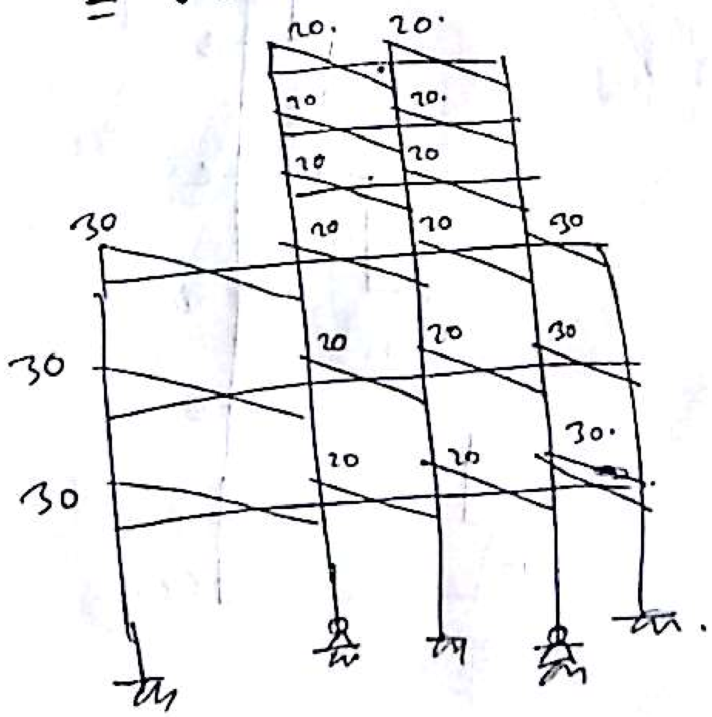
$SF = 0.5wL$.

$= 0.5 \times 2 \times 30 = 30$.

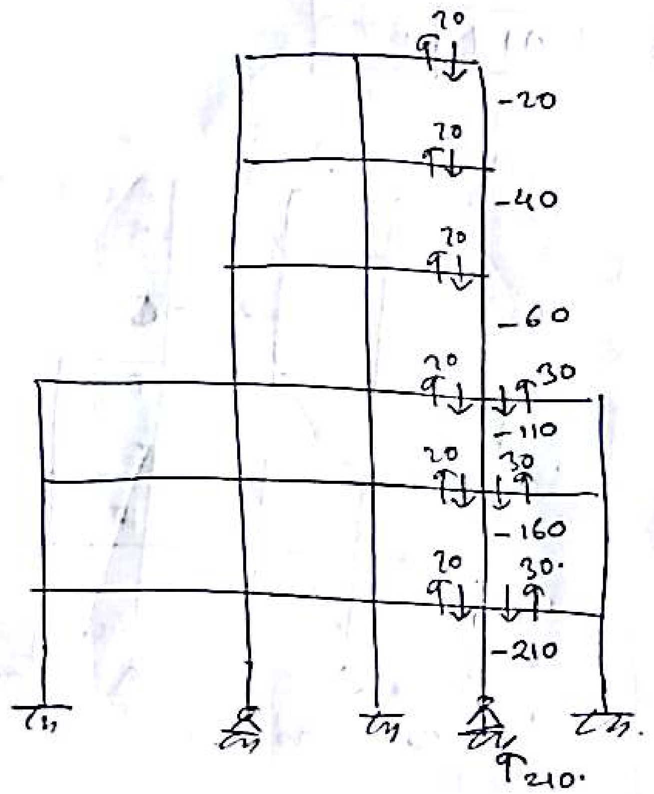
$SF = 0.5 \times wL$

$= 0.5 \times 2 \times 20$

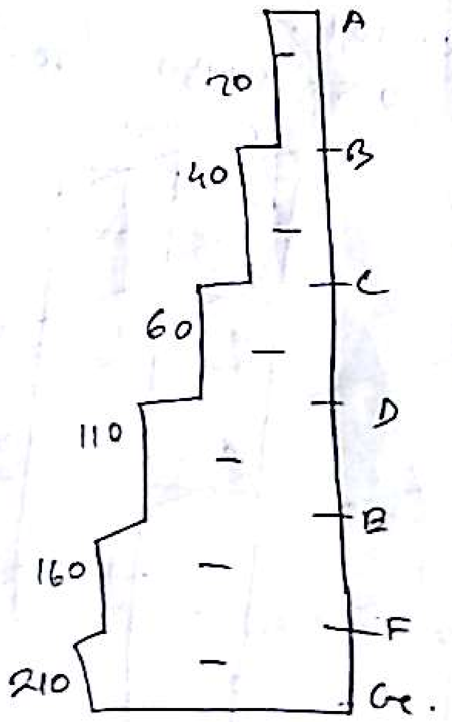
$= 20$.



AF in col.



70
-20
70
-40
70
-60
70
30
-110
70
30
-160
70
30
-210
210

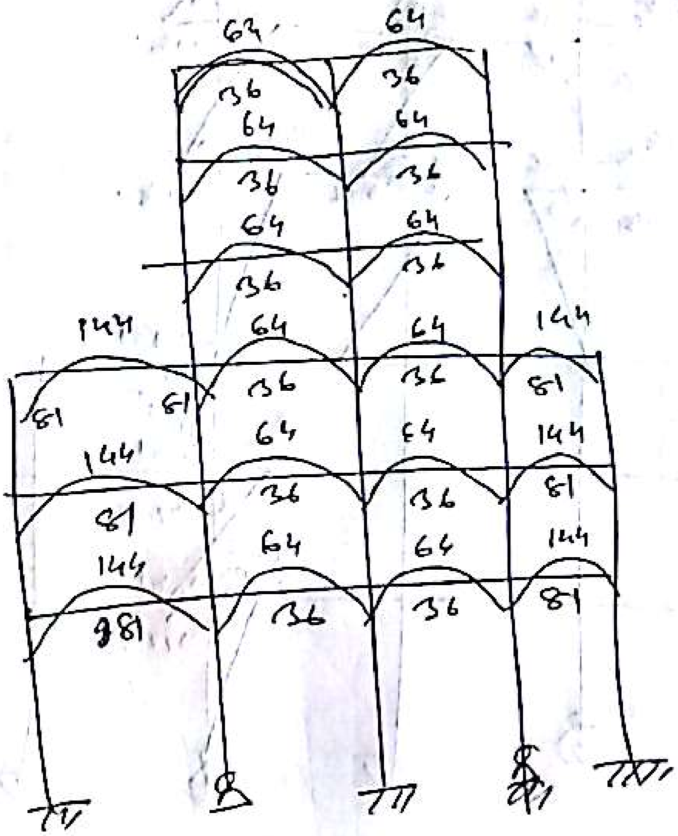


A
70
B
40
C
60
D
110
E
160
F
210
G

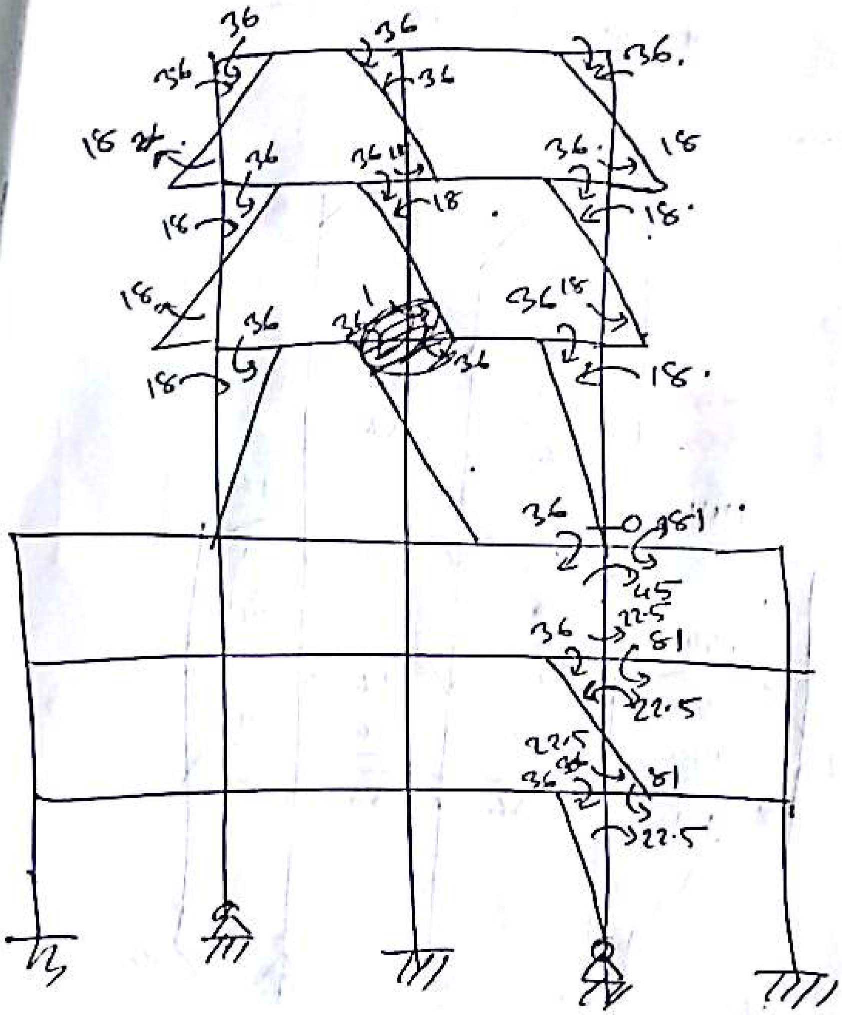
BM +ve in 30' = 144
−ve " 30' = 81

BM +ve in 20' = 64
−ve " 20' = 36

BM for Beam.



BM for column.



Just method ta
bujhte hobe.