1ST LECTURE

Date: 25.02.2015

Structural Analysis and Design I (CE 311)

Course Teacher: Dr. A.M.M Taufiqul Anwar (Room 419 / Head's Room)

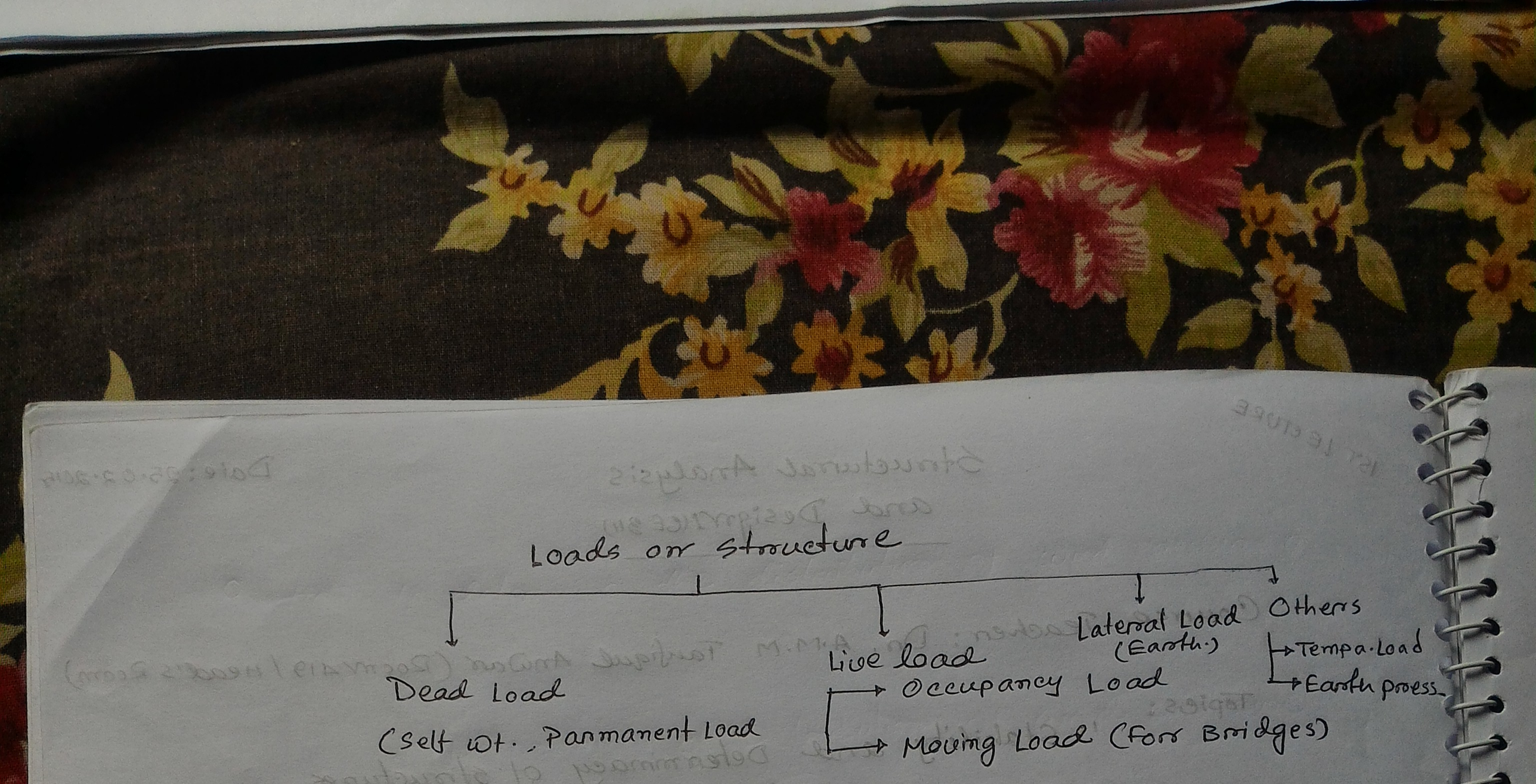
Topics:

1. Stability and Determinacy of structures
2. Analysis of statically determinate trusses and arches
3. Influence lines
4. Moving Loads on beams
5. Frames and trusses
6. Analysis of suspension bridge.
7. Wind and earthquake Loads
8. Approximate analysis of statically indeterminate structures.
9. Braced trusses
10. Portal method
11. Cantilever method and vertical Load analysis of multistoried building frames.
12. Deflection of Beam, truss and frames by virtual Work method.

References:-

1. Elementary Structural Analysis by Charlses Head Norris, John Benson Wilber & Senol utku, 3rd Edition
2. Theory of simple structure by - T.C. Shedd R J. Vawter, second Edition
3. Bangladesh National Building Code, BNBC, 1993.

1ST CLS TEST: 5th / 6th Class TEST



o Lateral Loads are important for Large structure.
Such as TALL BUILDINGS, LONG SPAN BRIDGES, TOWER, AIRCRAFT HANGAR, MILL BUILDINGS ETC.

Wind Loads

2ND LECTURE

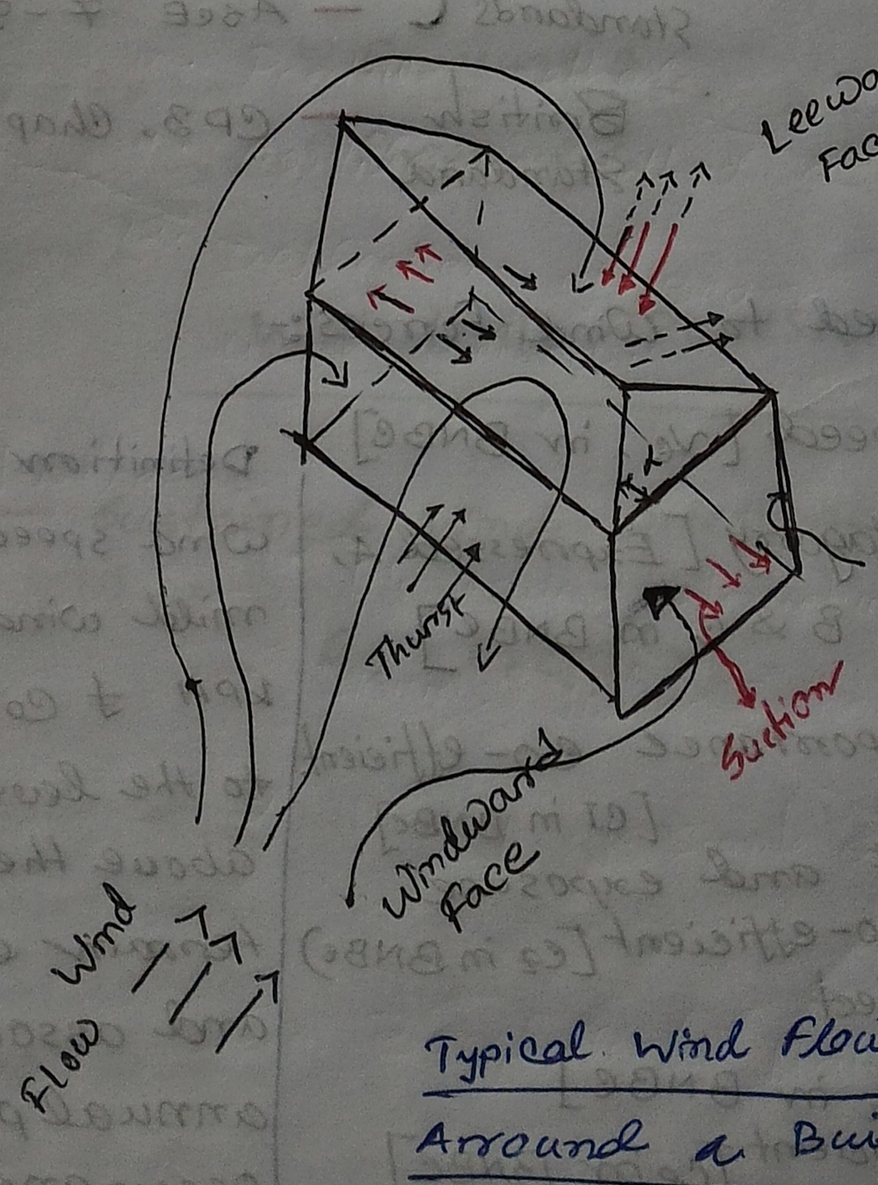
Structural Analysis and Design I

Date : 04-03-2015

Wind Forces

- Lateral Force are two types:-
  1. Wind Load
  2. Earthquake

- **Aerodynamics of Buildings:** Flow of wind around the buildings is a very complex process and depends upon many variables:-
  - Building size and shape
  - Type of exposure
  - Local topography
  - Random nature of wind.



Typical Wind Flow Around a Building

- How to determine the distribution of wind pressure?

Determination of Distribution of Wind Pressure

- By properly conducted Wind Tunnel Tests (expensive & done only for complex structures)
- By Code Guidelines
  - Major Building code of Standards :-
    - BNBC, 1993 (2006)
  - Us Codes/Standards
    - ANSI A58.1, 1982 & updates.
    - UBC 1997 & updates.
    - ASCE 7-95 & updates
  - British Standard
    - CP3, Chap V, Part-2

- Parameters Related to Wind Forces :-
  - Basic wind speed [V_c, in BNBC]
  - Exposure catagory [Expressed A, B & C in BNBC]
  - Structure Importance co-efficient. [CI in BNBC]
  - Combined Height and exposure co-efficient [C_z in BNBC)
  - Wind Gust Effect [C_G in BNBC]
  - Pressure Co-efficient [C_p in BNBC]

Definition of Basic wind speed :- Fastest mild wind speed in kph & corresponding to the level of 10 meters above the ground of terrain exposure B and associated with annual probability of occurance of 0.02

○ Determination of Ve :- Table 6.2.8 & Figure 6.2.1

- Table 6.2.8
- If Location is not found in Table 6.2.8, we will move to figure 6.2.1. we should find it in figure sequentially.
  - Whether it is in Wind Region or not (hatched)
  - ~~Isolates~~ Isotacks
  - By Linear Interpolation.

○ Terrain Exposure :- Definition of Terrain Exposure

Exposure catagory :-

a) Exposure A

b) Exposure B

c) Exposure C

WIND LOAD PROVISION IN BNBC :-

- Sustained wind pressure at height Z (in m) → in kN/m²

$$q_z = C_e C_t C_z V_b^2 \text{ ----- } (2.4.1) \text{ in kN/m}^2$$

3RD LECTURE

Structural Analysis and Design

Date: 11.03.2015

Wind Load provision of BNBC, 1993/2006:

- Sustained Wind pressure at height, z(m)

$$q_z = C_e C_1 C_2 V_e^2 \text{ ------ } (2.4.1) \quad [kN/m^2]$$

Here,

C_e = Conversion co-efficient = 47.2×10^{-6}

C_1 = Structural Importance co-efficient (0.8 to 1.25, Table 6.2.9)

C_2 = Combined Height and Exposure co-efficient. (Table 6.2.10)

V_e = Basic Wind speed, KPH (Fig. 6.2.1a & Table 6.2.8)

- Design wind pressure,

$$P_z = C_G C_p q_z \text{ ---------- } (2.4.2) \quad [kN/m^2]$$

Here, C_G = Gust co-efficient (section 2.4.6.6) — For nonslender

C_p = Pressure co-efficient (see 2.4.6.7) Building. $C_G = C_{GN}$ (Table 6.2.11)

- Wind Gust Effects: For non-slander Building, $C_{Gz} = G_h$.

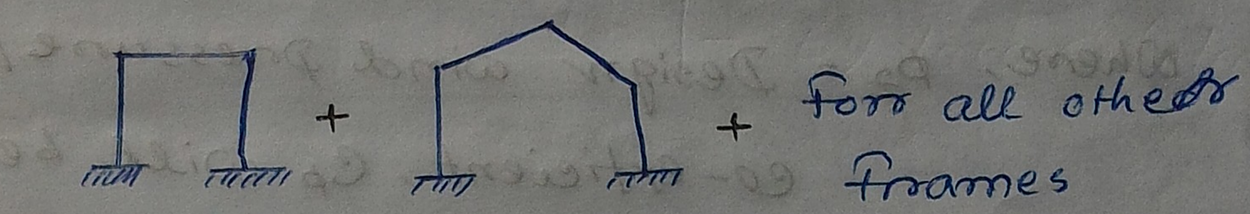
Table -> 6.2.11

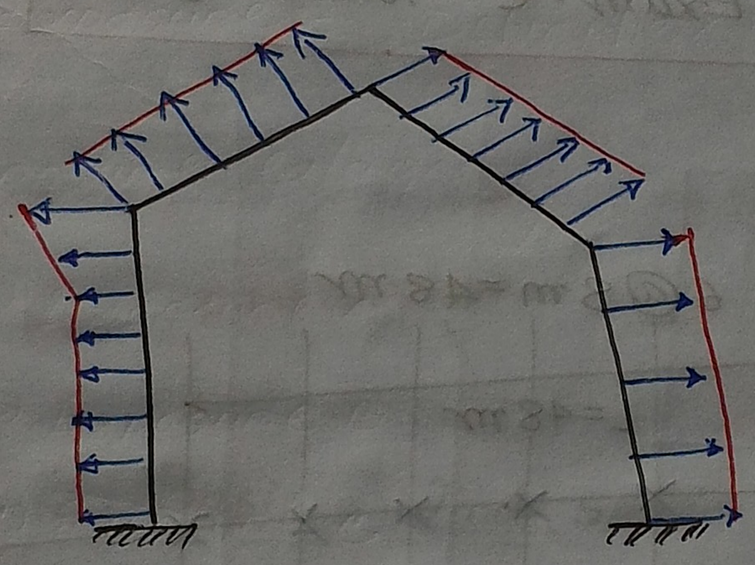
- Pressure co-effient: For Rectangular Buildings :-
  - With surface area method — Fig. 6.2.5 may be used. (Method 1)
  - With projected area (Method-2) — Table 6.2.15 may be used.

2.4.6.4 Design Wind Load for Building and Structures:-

- Method 1 (Surface area Method)

(i) For all framing structures, $F_1 = \sum pA_z$ ------ (2.4.3)





Here, F_1 = Wind Load normal to surface

$p = p_z$ – for windward surface (kN/m²)

p_h – for non-windward surface (kN/m²)

A_z = Area of Building surface (m²)

(ii) Additionally, for gabled frames and single-storey rigid frames :- (2.6.3) (2.6.4)

$$f_1 = \sum (p - p_i) A_z \text{ ------ } (2.4.4)$$

Where, p_i = Initial pressure = $c'_{pi} q_h$

c'_{pi} = Initial peak pressure co-efficient (sec. 2.4.6.7)

q_h = Sustained wind pressure evaluated at mean roof height (Eq. 2.4.1)

- Method-2 (Projected Area Method):

For all structures except for gabled frame and Single storey Rigid frame.

$$F_Z = \sum P_Z \bar{A}_Z \quad \ldots\ldots (2.4.15)$$

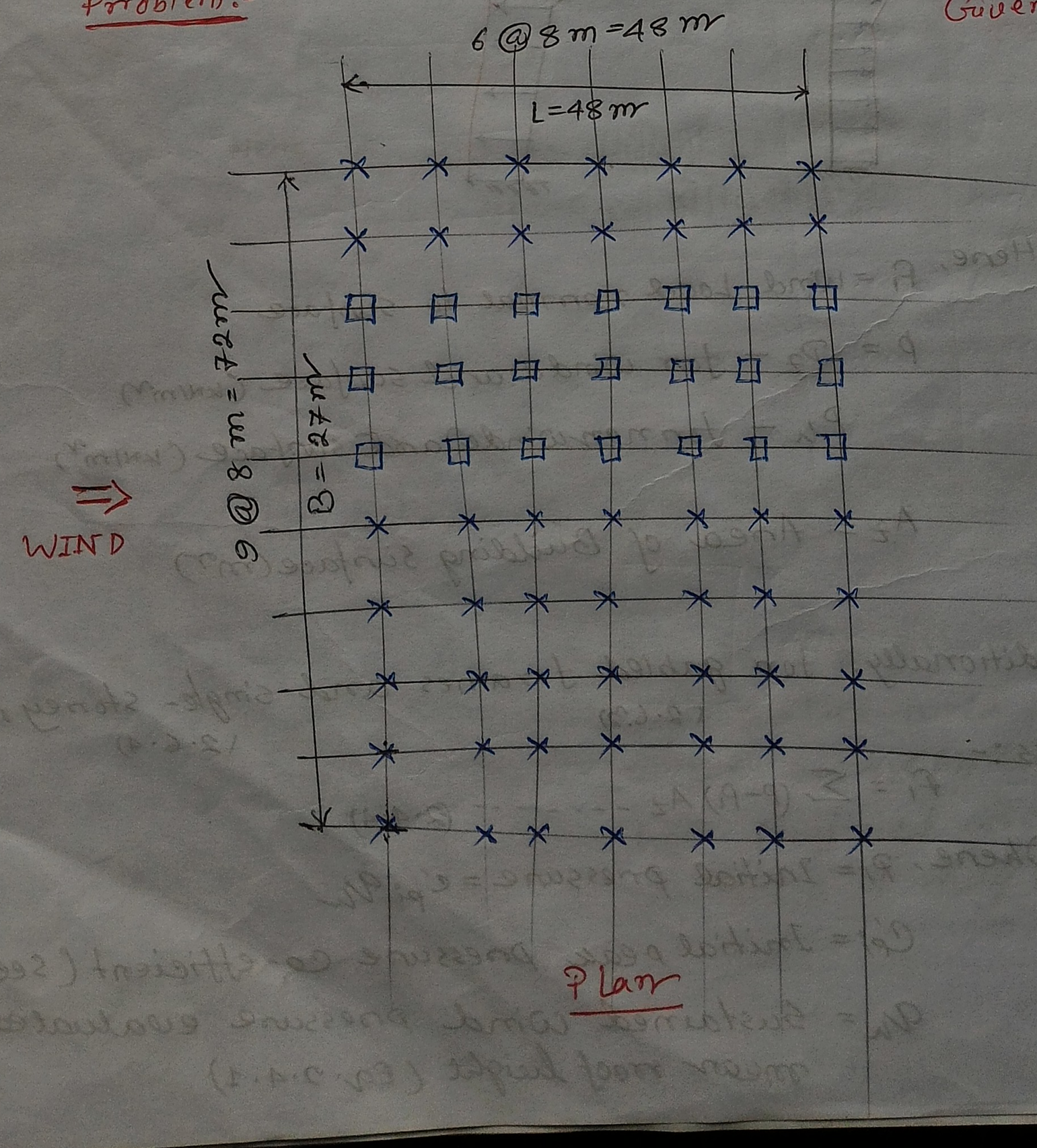
Where, P_Z = Design wind pressure / overall pressure co-efficient, C_p will be used (Table 6.2.15 - 6.2.21)

$\bar{A}_Z$ = Projected frontal Area.

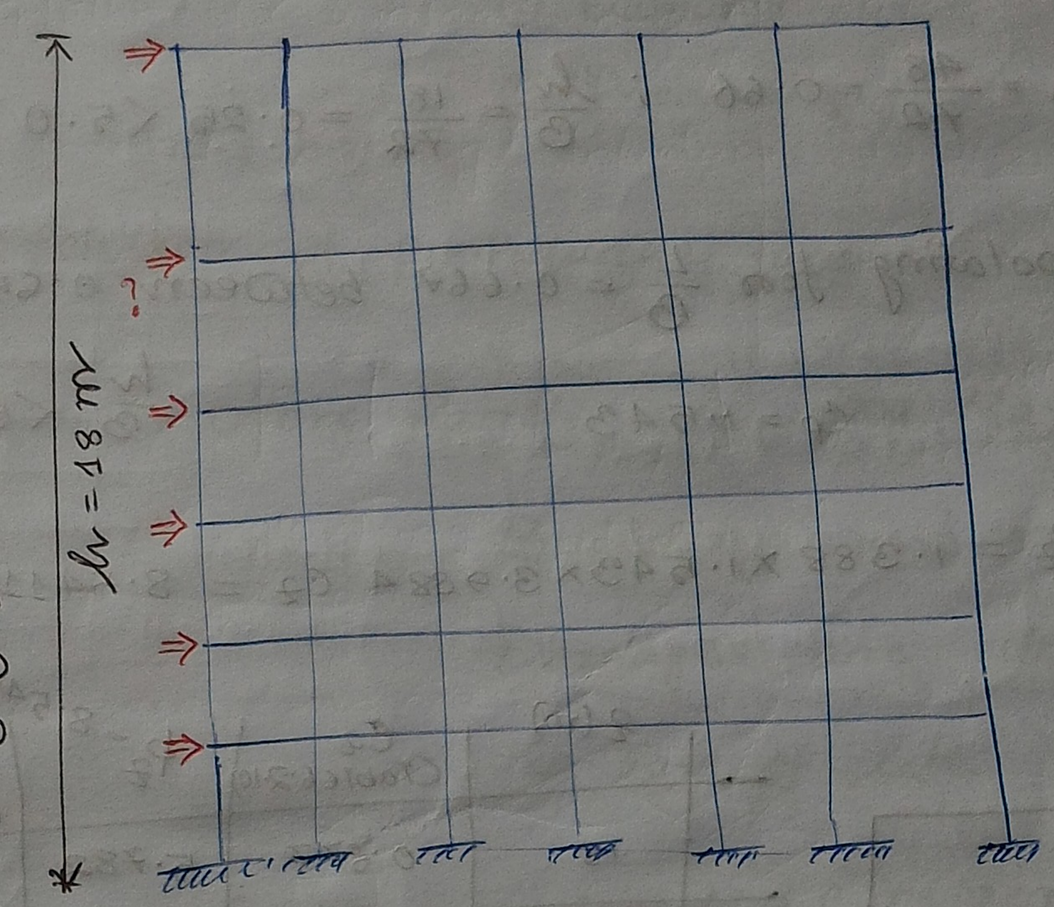
Usuall Exam-e Table 6.2.15

Problem:-

Given:-
- HOSPITAL BUILDING
- Chittagong City
- Forced at Every Storry Level = ?



Plan



Solution:

Basic Wind speed (Table 6·2·8), $V_b = 260$ KPH

Exposeure category : A

Sustained wind pressure, $q_z = C_c C_I C_z V_b^2$

$C_c = 47.2 \times 10^{-6}$

$C_I = 1.25$ (Table 6·2·9)

$C_z = f(z, \text{Exp. A})$ (Table 6·2·10)

$\therefore q_z = 47.2 \times 10^{-6} \times 1.25 \times C_z \times (260)^2 = 3.9884\, C_z$ KN/m²

Design Wind Pressure, $P_z = C_{Gz} C_p q_z$

for ~~a~~ non-slender building, $C_{Gz} = C_{Zw}$ (Table 6·2·11)
(z = 18m)
$= 1.388$

$C_p = \bar{C}_p$ (Art. 2.6.6.7d) =

For Method 2, Projected Area Method

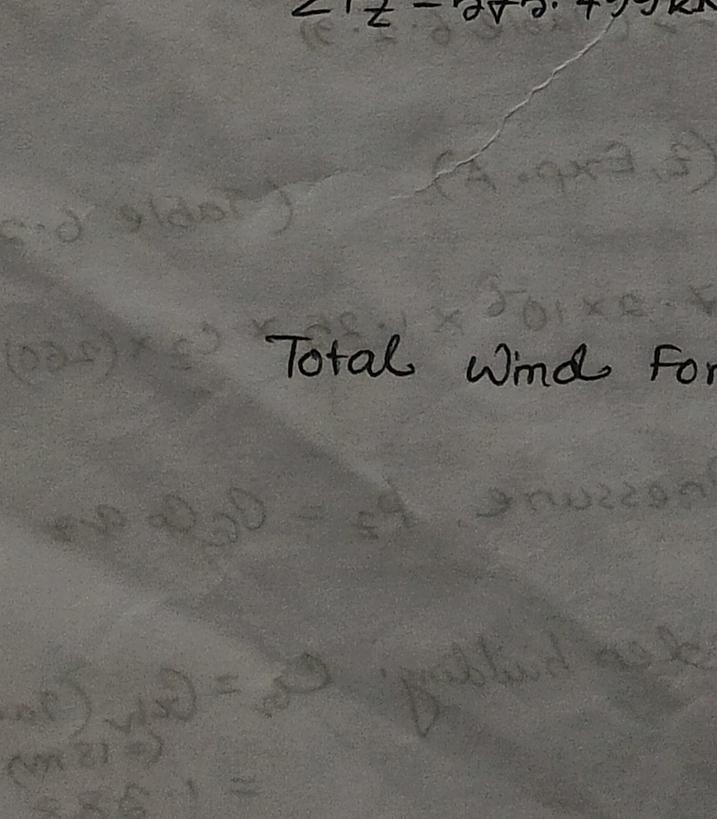
$$L/B = \frac{48}{72} = 0.66 \; ; \; \frac{h}{B} = \frac{18}{72} = 0.25 < 5.0$$

Interpolating for $\frac{L}{B} = 0.667$ between 0.65 and 1 and $\frac{h}{B} < 5.0$,

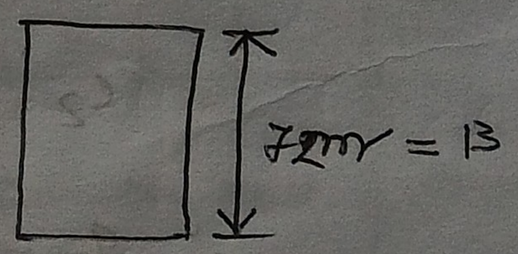
$$C_p = 1.543$$

$$P_z = 1.388 \times 1.543 \times 3.9884 \, C_z = 8.5411 \, C_z \text{ kN/m}^2$$

	Z (m)	C_z (Table 6.2.10)	$P_z = 8.5411 C_z$ (kN/m²)	$\bar{A}_z$ (m²)	Force, $F_2 = \Sigma P_z \bar{A}_z$
	18	0.677	5.782	12	69.388
	15	0.624	5.330	24	127.912
	12	0.565	4.826	24	115.817
	9	0.497	4.245	24	101.878
	6	0.415	3.545	24	85.069
	3	0.368	3.143	24	75.435



$\Sigma F_z = 575.499$ kN L = 48 m



Total Wind Force on the building = $\frac{575.499 \times 72}{8}$

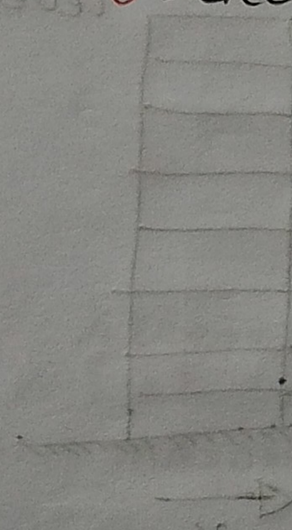
4Th Lecture

Date:- 18-03-2015

Earthquake Provision in BNBC

- Main Considerations:- ① strength
  ② Economy
  ③ Probability

- selection of Earthquake Forces :-



- For Rarely occurring Major EQ

  Building must not collapse
  Damages are allowed
  No loss of life
  → safety consideration

- For moderate consideration:

  – No. Structural Damage allowed
  (serviceability consideration)

Selection of EQ forces :-

2 Methods → Equivalent static Force Method
→ Dynamic Response method

Equivalent static force Method may be used in this course.

5 Th Leeture

Structural Analysis And Design

Date:- 25.03.2015

Equivalent static force Method:- (ESFM)

Design Base shear, $V = \frac{ZIC}{R} W$ [Table 6.2.22 Fig. 6.2.10]

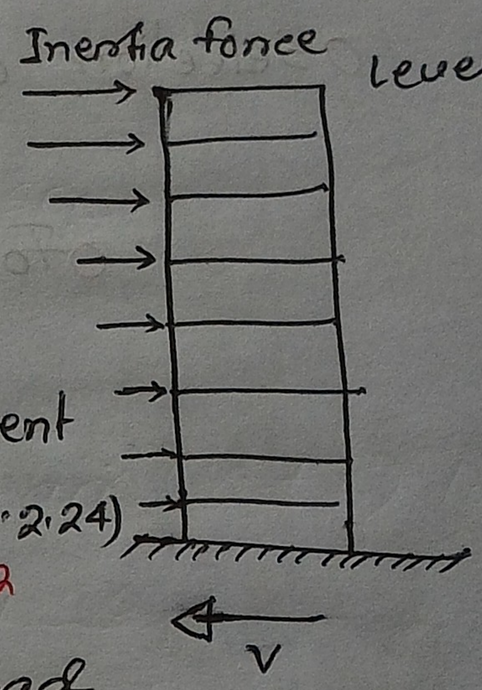
Z = Seismic Zone co-efficient

I = Structural Importance Co-efficient (Table 6.2.23)

R = Response Modification coefficient for structural System (Table 6.2.24)

4~12

W = The total seismic dead load defined in 2.5.5.2



W: Including parmanent portion and appreciable of other loads as described below:

1. In storage and wearehouse occupies a minimum of 25%.
2. All loads of partions, not less than @ 0.6kN/m²
3. Total weight of parmanent equipment shall be included.

S = site co-efficient soil Characteristies. (Table 6.2.25 ~~2.5.2~~)

C = Numerical co-efficient given by relation: $C = \frac{1.25}{T^{2/3}}$

T = Fundamental period of vibration in seconds, Of the structure (2.5.6.2)

Structure period:-

Method A → Easier Method

$$T = C_t (h_n)^{3/4}$$

where, $C_t = 0.083$ for steel moment resisting frame.

$= 0.073$ for R.C moment resisting frame, and eccentric braced steel frames.

$= 0.049$ for all other structural system.

h_n = Height in meters above the base to level n.

Alternatively, the value of C_t building with concrete or masonary shead walls may be taken as $0.031/\sqrt{A_c}$ shall be obtained from relation:

$$A_c = \Sigma A_e [0.2 + (D_e/h_n)^2]$$

A_c = The combined effective area in square meters

A_e = The effective cross sectional area, in square meter of a shear wall.

D_e = The length, in meter of shear wall element in the first storey.

Method B

$$T = 2\pi \sqrt{\sum_{i=1}^{n} \omega_i \delta_i^2 / g \sum_{i=1}^{n} f_i \delta_i}$$

Here, f_i = any static lateral force distributed approximately as EQ forces.

δ_i = deflection at ith level

ω_i = seismic DL of level i

g = Acceleration due to gravity

T(method B) $\leq 1.4T$ (Method A)

Vertical Distribution of Lateral Forces:-

V distributed along height of structure as $V = F_t + \sum_{i=1}^{n} F_i$

F_i = Lateral Force at storey level i

F_t = Concentrated lateral Force at top of building in addition to F_n

$F_t = 0.07TV \leq 0.25V$ when $T > 0.7$ sec

$= 0$ when $T \leq 0.7$ sec.

Remaining portion of load shear $(V - F_t)$ to be distributed over height of building as.

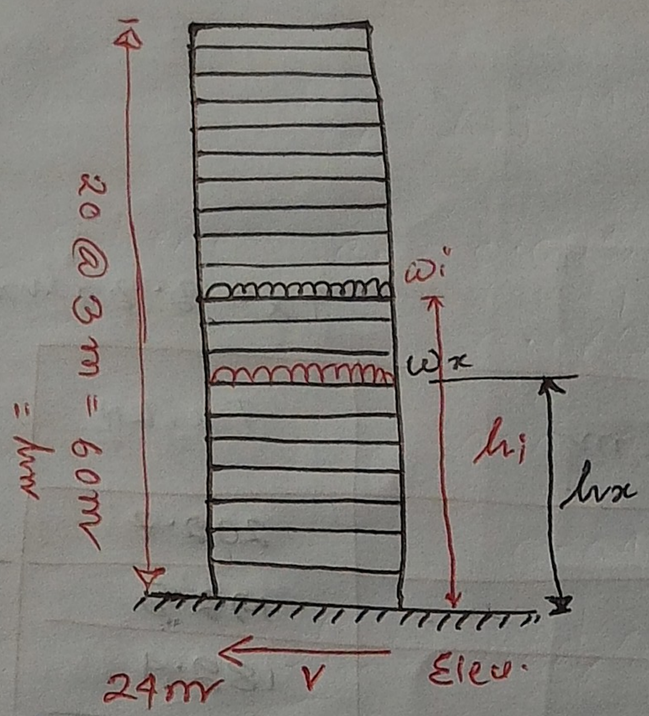
$$F_x = \frac{(V - F_t)\, W_x h_x}{\sum_{i=1}^{n} W_i h_i}$$

6th Lecture

Structural Analysis and Design

Date: 01-04-2015

#



Equiv. static force for EQ = ?

Given: 20 storried office Building at Dhaka ; soil Type : soft to Medium stiff clay.

DL including partitions = 12 kN/m²

Structure type = SMRF in concrete.

Solution:- Base shear,

$$V = \frac{ZIC}{R} W$$

$$\Rightarrow V = \frac{0.15 \times 10 \times 1.386}{12} \times 138240$$

$$\therefore V = 2392 \text{ kN}$$

Z = 0.15 (Zone 2, Table 6.2.22, Fig. 6.2.10)

I = 10

R = 12 (Table 6.2.24)

$$W = 12 \times (24 \times 24) \times 20 = 138240 \text{ kN}$$

$$C = \frac{1.25 S}{T^{2/3}} \quad ; \quad S = 1.5 \text{ (Table 6.2.25)}$$

$$= \frac{1.25 \times 1.5}{1.57^{2/3}} \qquad T = C_t (h_n)^{3/4}$$

$$= 1.386 < 2.75 \text{ Ok.} \qquad = 0.073 \times (60)^{3/4}$$

$$= 1.57 \text{ sec}$$

$$\frac{C}{R} = \frac{1.386}{12} = 0.116 > 0.075$$

Vertical Distribution for EQ Forces

$$F_t = 0.07 TV$$

$$= 0.07 \times 1.57 \times 2392$$

$$= 263 \text{ kN} < 0.25 V \cong 600 \text{ kN (Ok)}$$

$$f_x = \frac{(V - F_t) w_t h_x}{\sum_{i=1}^{n} w_i h_i}$$

$$= \frac{(V - F_t) h_x}{\sum_{i=1}^{n} h_i}$$

$$= \frac{(2392 - 263) \times h_x}{\sum_{i=1}^{n} h_i}$$

$$= \frac{(2392 - 263) \times h_x}{(3 + 6 + \cdots\cdots + 60)}$$

$$= \frac{2128 h_x}{3(1 + 2 + \cdots\cdots + 20)}$$

$$= \frac{2128 \, lhz}{3 \times \frac{20 \times 21}{2}}$$

$$= 3.38 \, lhz$$



$Fx = 3.3 \times lhz$

EQ FORCES

Floor level	lhz, m	Fx, kN
20	60	202.7
19	57	192.5
18	54	182.4
17	51	172.3
16	48	162.1
15	45	152
14	42	141.9
13	39	131.7
12	36	121.6
11	33	111.5
10	30	101.3
9	27	91.2
8	24	81.1
7	21	70.3
6	18	60.8
5	15	50.7
4	12	40.5
3	9	30.4
2	6	20.3
1	3	10.1
		V = 2392 kN

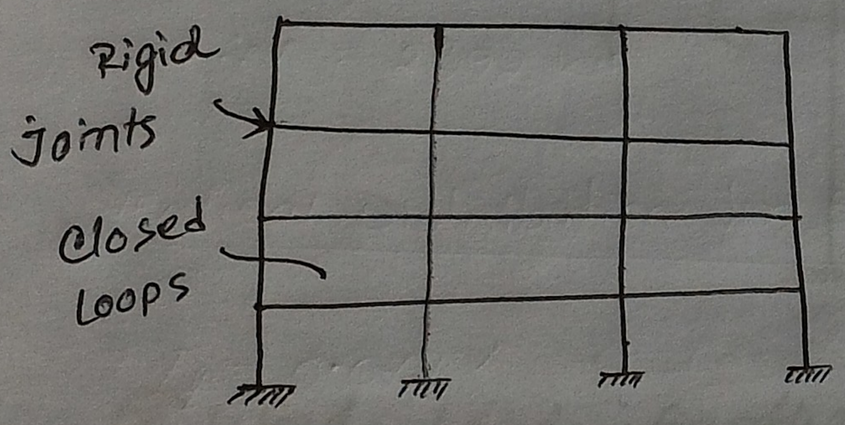
→ 202.7 + 264 = 466.7 kN

Statically Indeterminate Structures; Approximate Analysis

In broad sense, all analysis are approximate. Some are exact in relative sense. Exact analysis are based on usual assumptions such as hinge or fixity, homogeneity, isotropic behaviours etc. etc.

Approximate Analysis: For manual analysis of statically indeterminate structures, these are based on additional assumptions.

Approximate Analysis of Building Frames for vertical Loads.



d.o.i = 3×12 = 36

Degree of indeterminacy = 3 × no. of closed loops − no. of releases

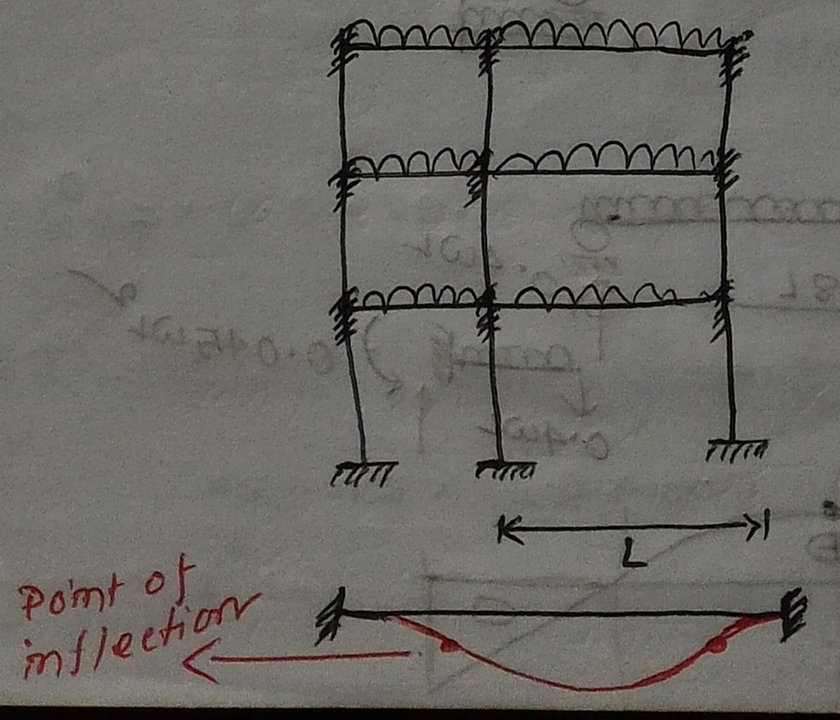
For all fixed joints at bases of col.

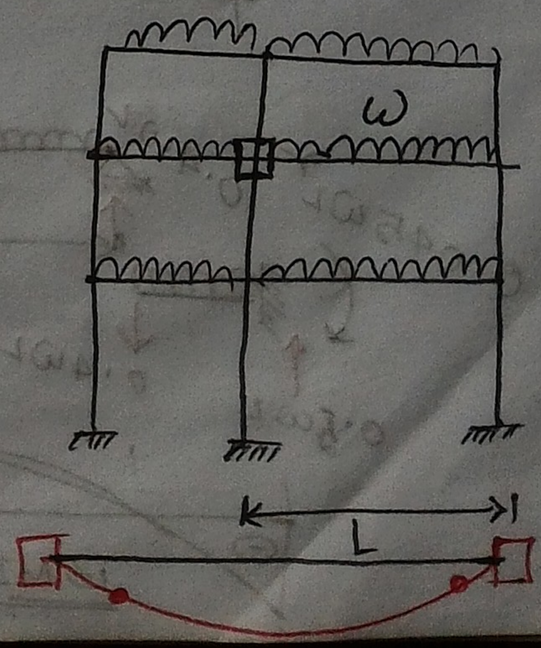
d.o.i = 3 × no. of girders

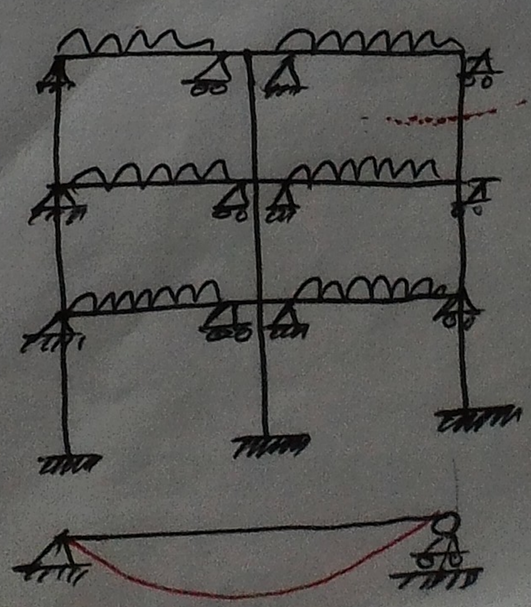
∴ For such frames assumptions read

= 3 for each girder

= 3 × no. of girderrs





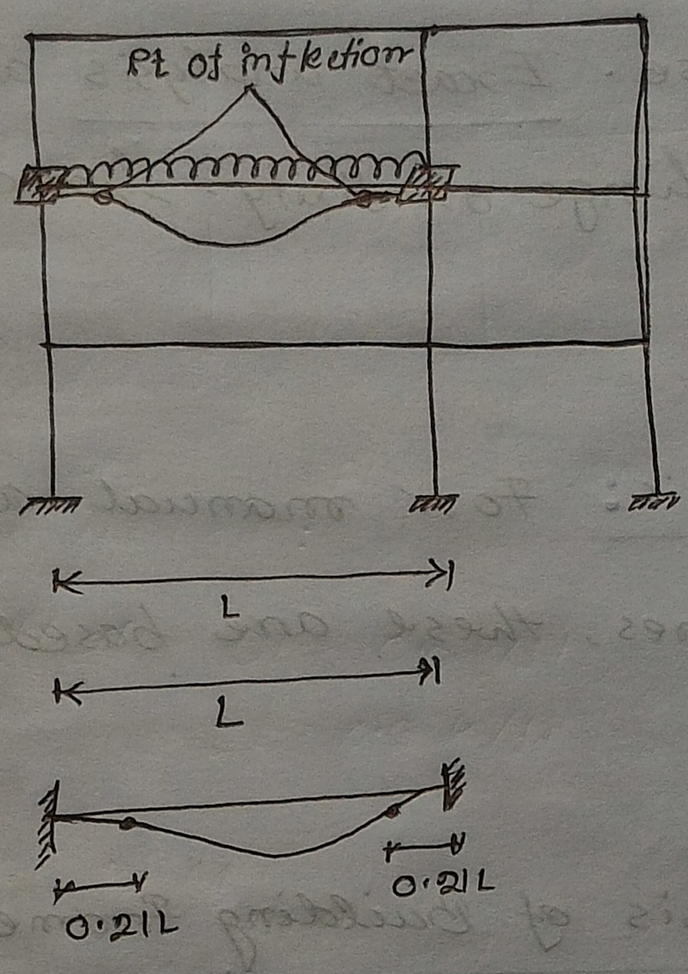


7TH Lecture

Structural Analysis and Design

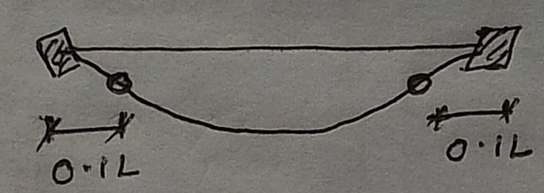
Date: 26.04.2015

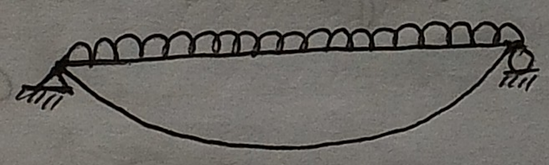
Approximate Analysis of Building Frames: Vertical Loads:-

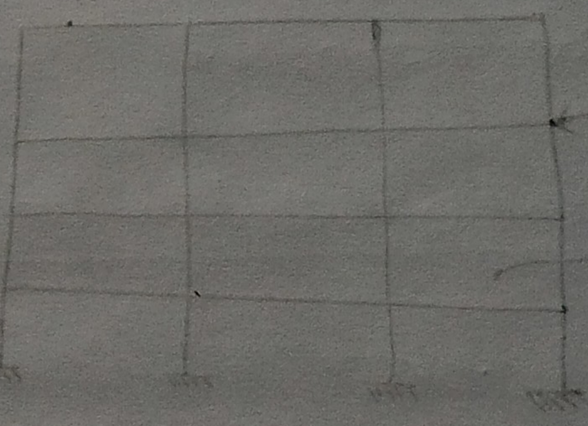


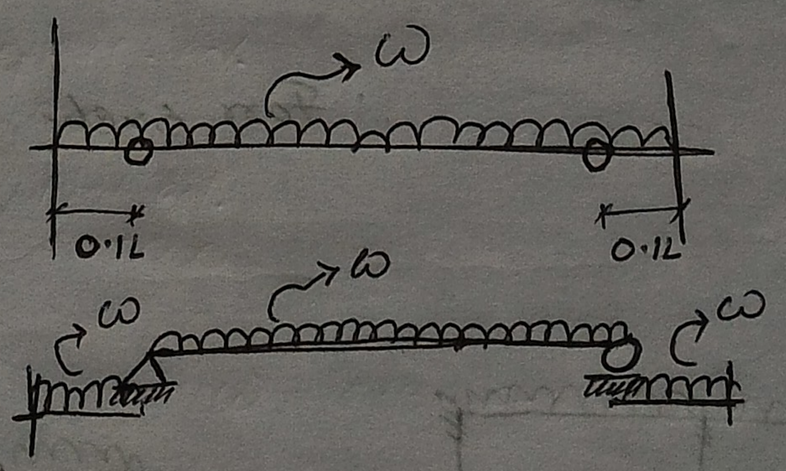
Assumptions:

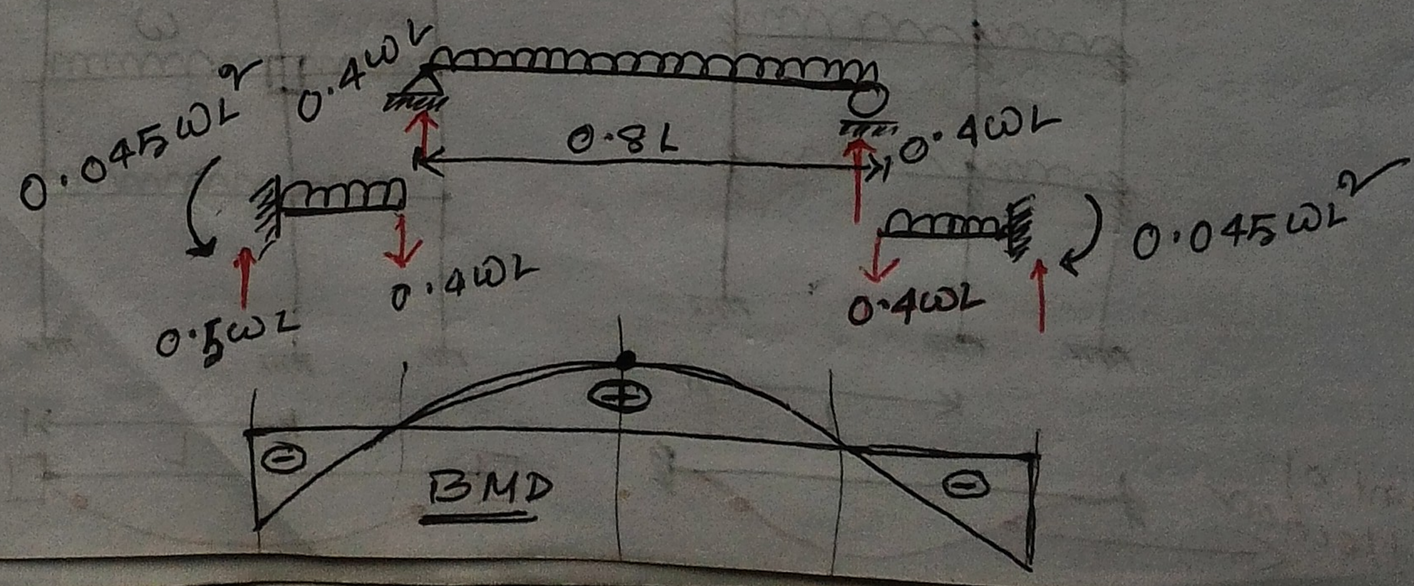
1. Axial force in Girder is zero
2. A point of inflection occurs at the one-tenth point ~~major~~ measured along the span from left support
3. A point of inflection occurs at one-tenth along the span from the right support



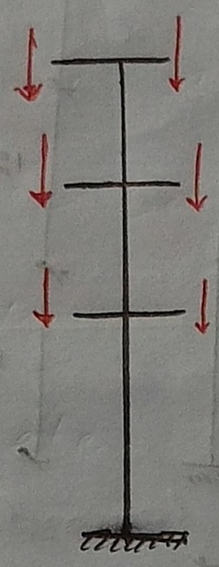






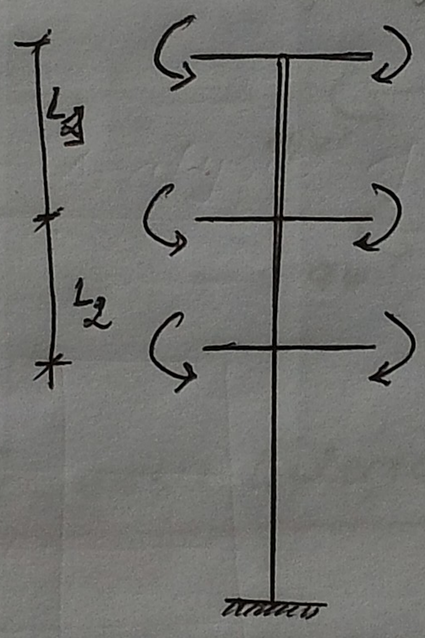


Axial Forces on Columns:



Σ Axial loads in columns
$= \Sigma$ Shear above Girder.

Bending Moment in Columns:

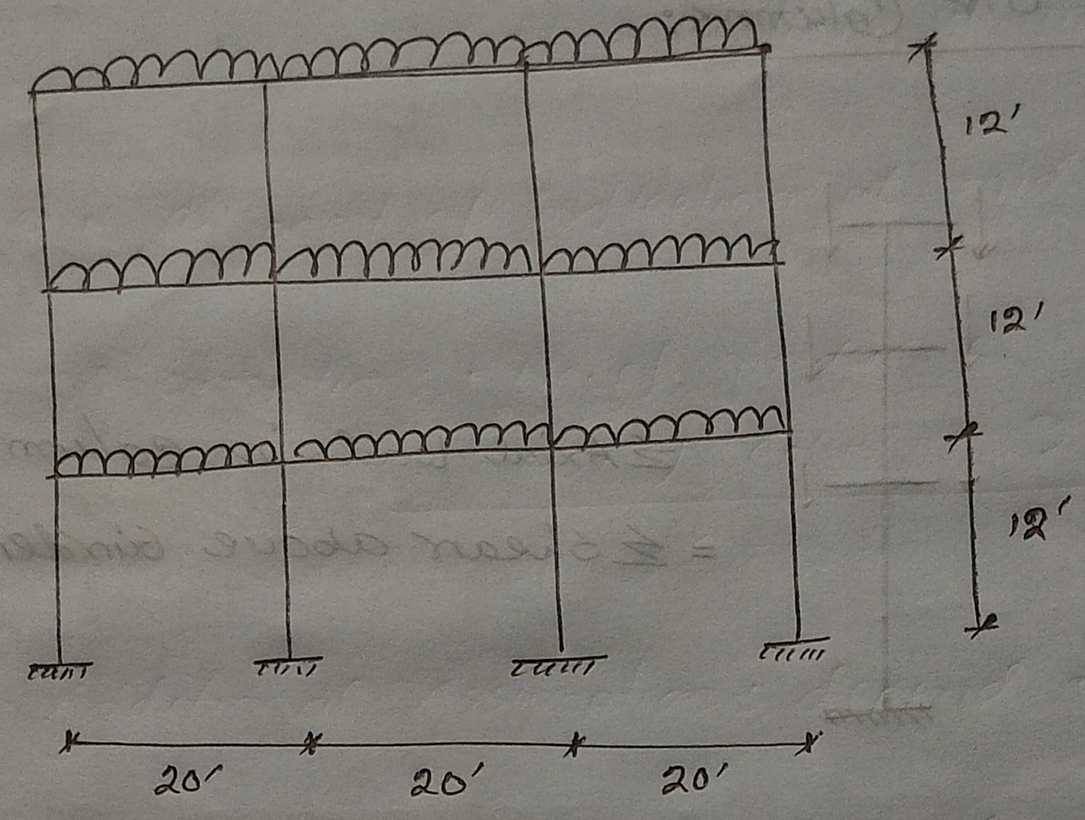


Gurder Moment

(*) Wind load, Earthquake load → sheet আনতে হবে।

Next week wednesday.

Problem 7.6



Given that, DL = 0.5 k/ft

LL = 0.3 k/ft

Find: a) M^{Gr}_{max+}

b) M^{Gr}_{max-}

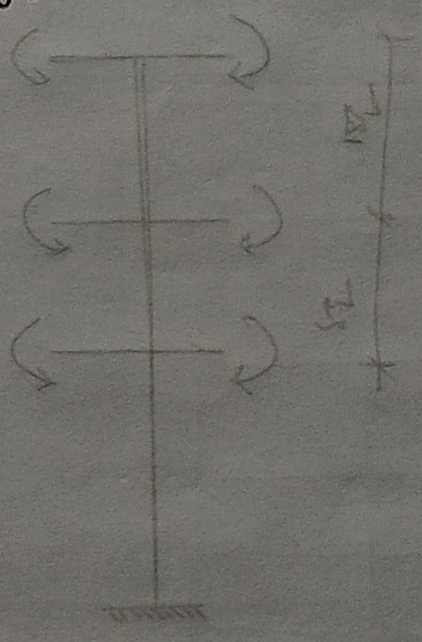
c) V^{Gr}_{max}

d) Pext col. max com.

e) Pint col. max com.

f) M ext. col max

g) M int col. max



Solution:

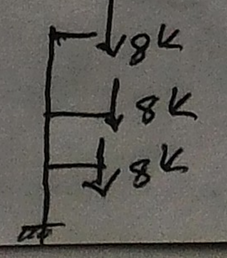
Total $w = 0.5 + 0.3 = 0.8$ k/ft

(a) $M^{Gr}_{max+} = 0.08 wL^2 = 0.08 \times 0.8 \times 20^2 = 25.6$ k'

(b) $M^{Gr}_{Max} = 0.045 wL^2 = 0.45 \times 0.80 \times 20^2 = 14.4$ k-ft

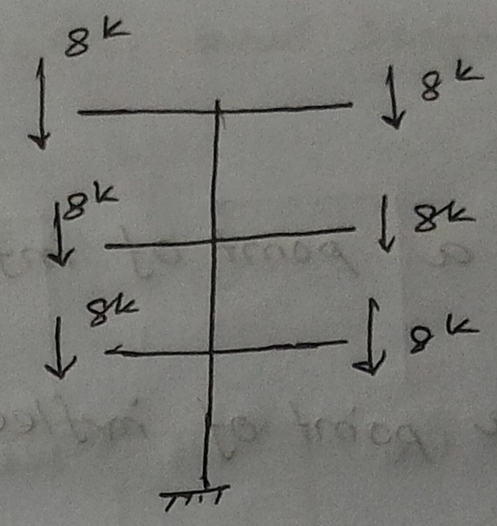
(c) $V_{Gr} = 0.5 wL = 0.5 \times 0.80 \times 20 = 8$k

(d) pext. col. max com. $= 8 + 8 + 8 = 24$k (at Bottom-most col.)



e) pint col. max com. = 8+8+8+8
+8+8

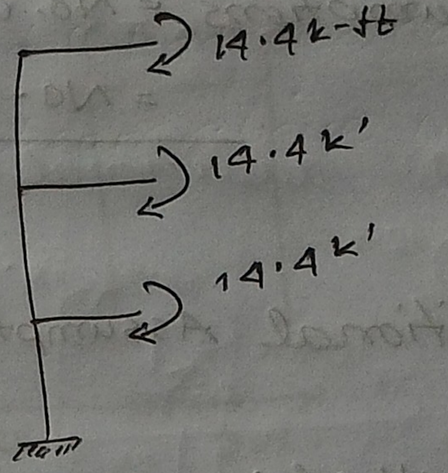
= 48k

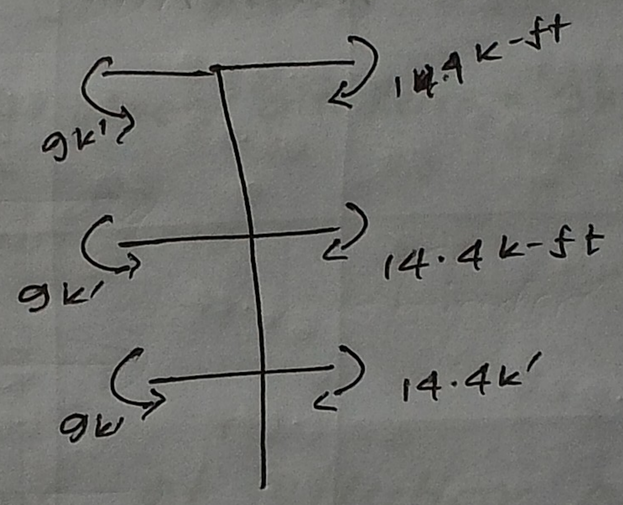


f) M ext col. max = 14.4 k-ft at top of top column

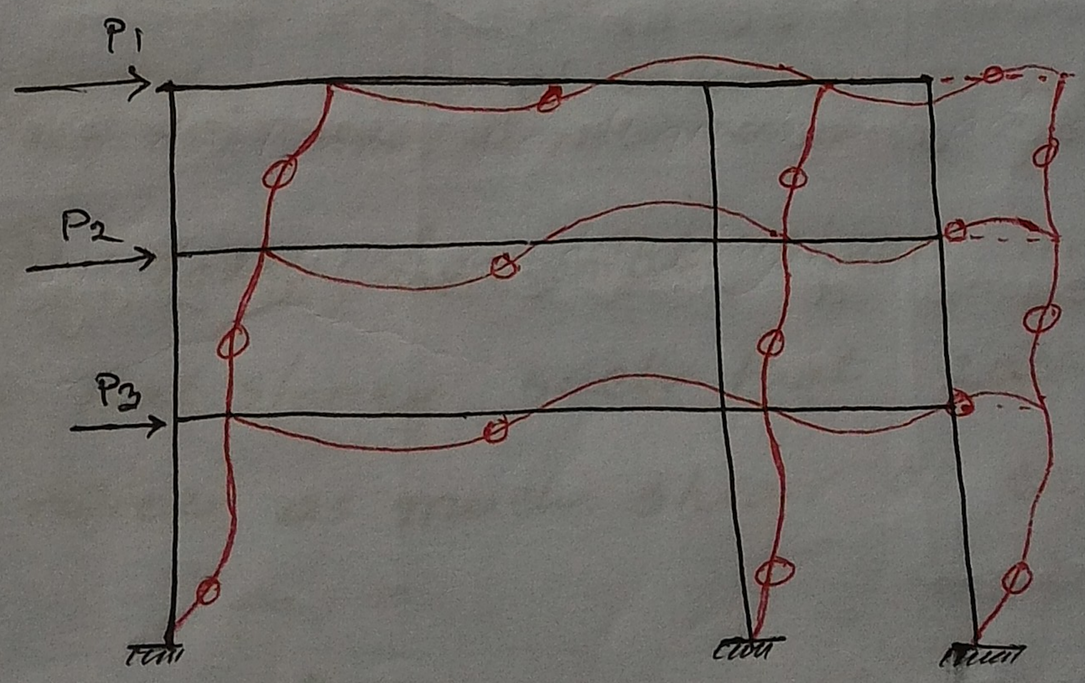
g) M int. col. max = 14.4k' - 9k'

= 5.4k'





Building frames : Lateral Loads (Approximate Analysis)



d.o.i = 3 × no. of closed loops - No. of releases

= 3 × 6 - 0 = 18

(a) Common Assumptions for both portal Method & cantilever Method :-

- There is a point of inflection at mid span of each girder.

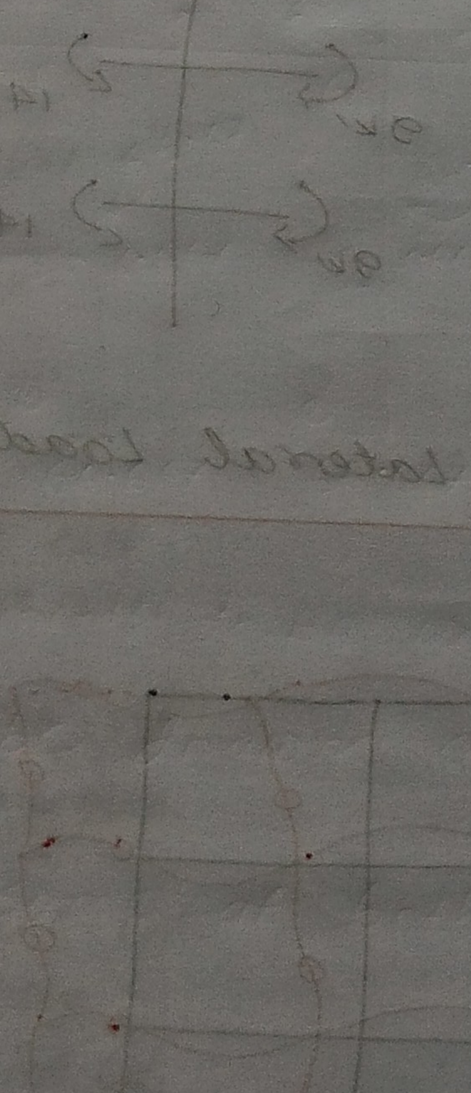
- There is a point of inflection at mid height of each column.

Assumptions = No. of girder = 6
= No. of columns = 9

$\Sigma = 15$

Additional Assumptions depend on Method.

Portal Method: Assumption based on distrib.

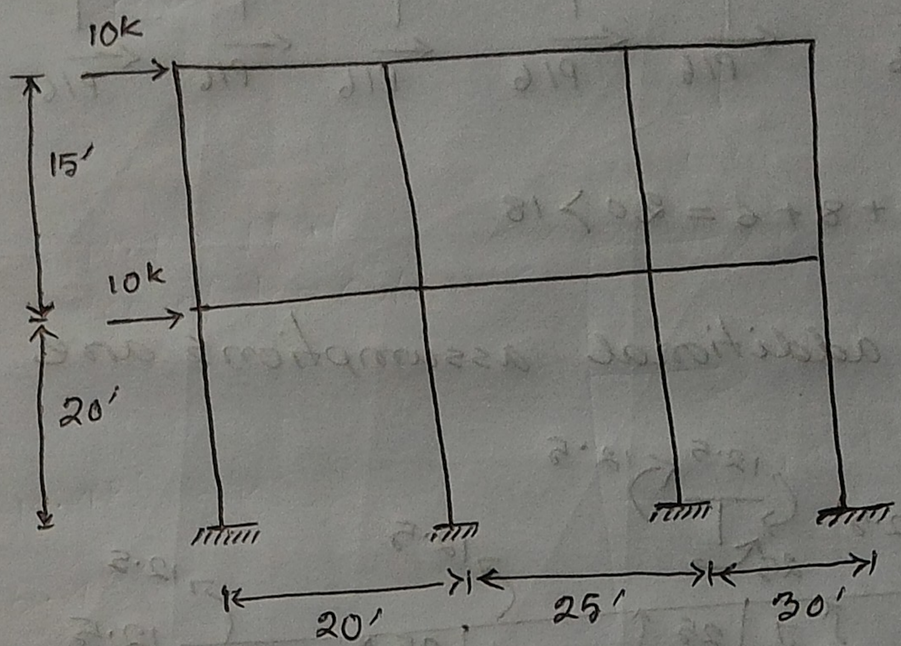


Building Frames: Lateral Loads (Approximate Analysis)

Structural Analysis and Design

Date: 06.05.15

Portal Method:



d.o.i = 3 × no. of given girders = 3×6 = 18

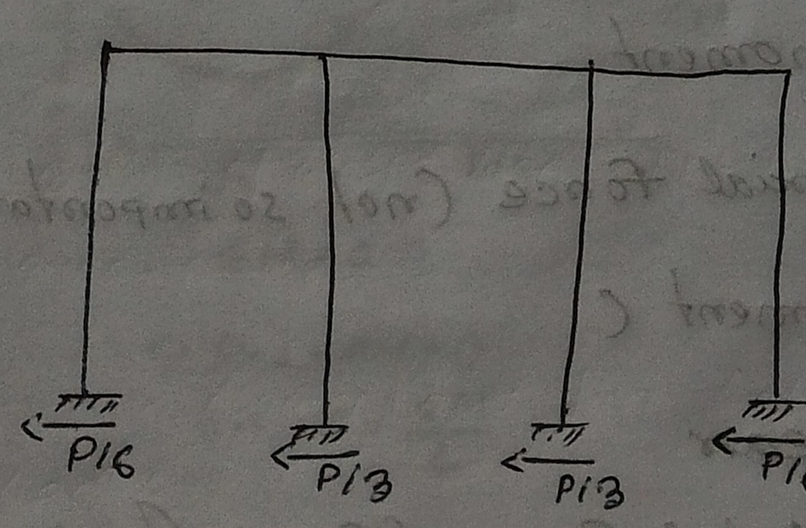
Assumptions Required = 18

Assumptions:

① There is a point of inflection at centre of each girder. No. of girder = 6 assumption

② There is a point of inflection at mid height of each columns. No. of columns = 8 assumptions

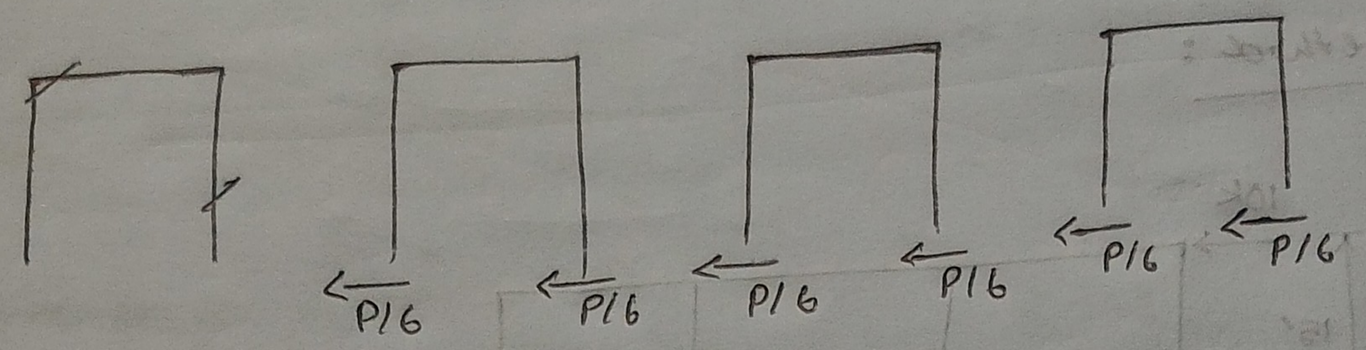
③ Total horizontal shear in each storey is divided in that storey such that each exterior column there is twice as much shear as each exterior column.



= No. of storey × (m−1)
↓ No. of column in a storey

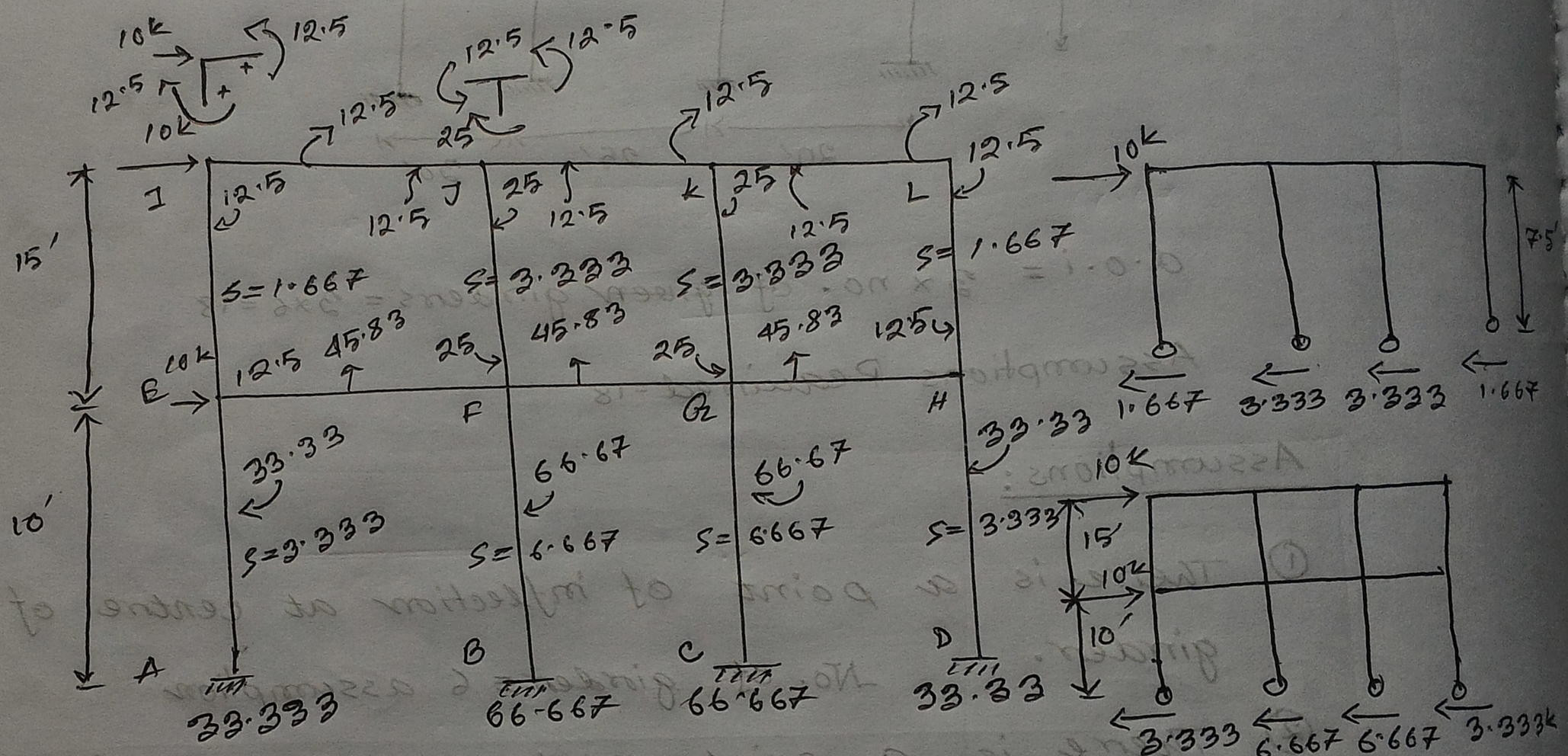
= 2 × (4−1)

= 6



$\Sigma = 6+8+6 = 20 > 18$

However, additional assumptions are consistent.



Values of Moments to be written on tension side

B. Moment to be drawn on compression side

Col. moment same at both ends (as hinge is at its width

Steps in solution:-

① Column shear

② Column moment

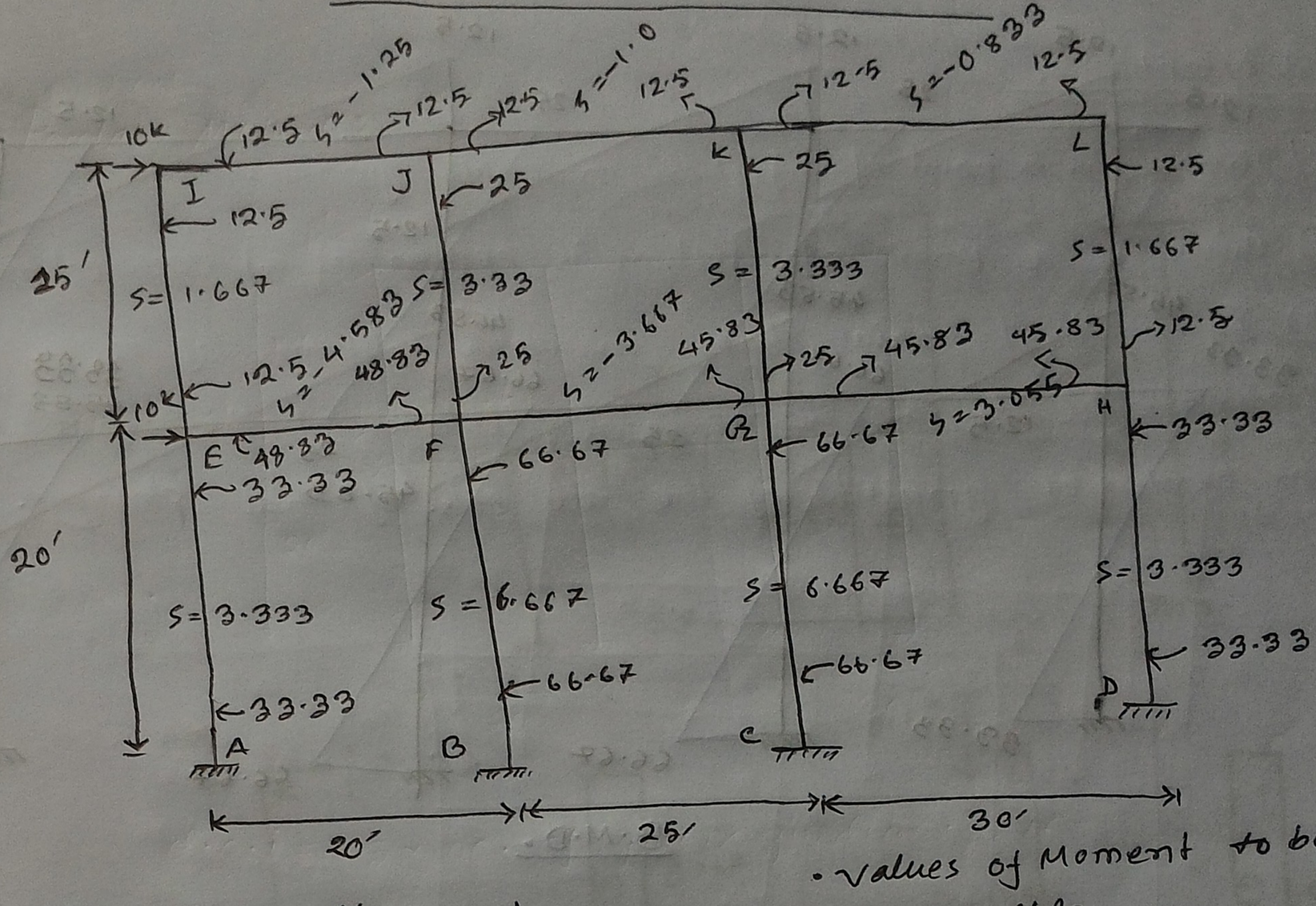
③ Girder Axial force (not so important) from col. shea

④ Girder moment (

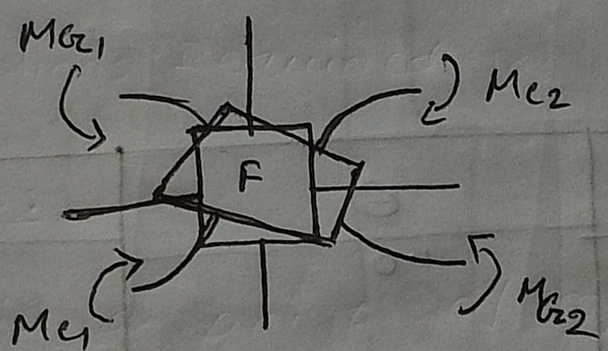
⑤ Girder shear

⑥ Column Axial force (from Girder shear).

Structural Design and Analysis



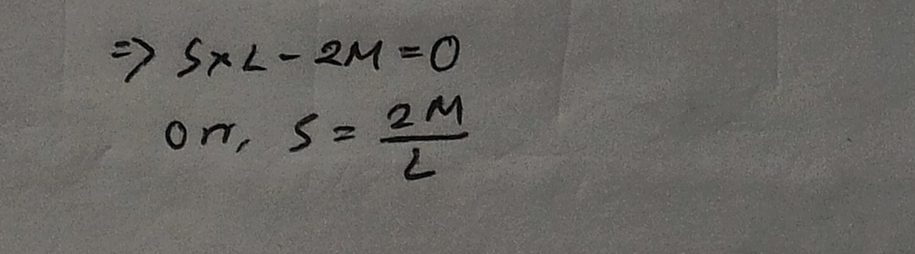
- Values of Moment to be written on tension side
- B.M. diagram to draw on compression side



$\Sigma M = 0$

MC1 Σ Col Moment = Σ Girder Moment

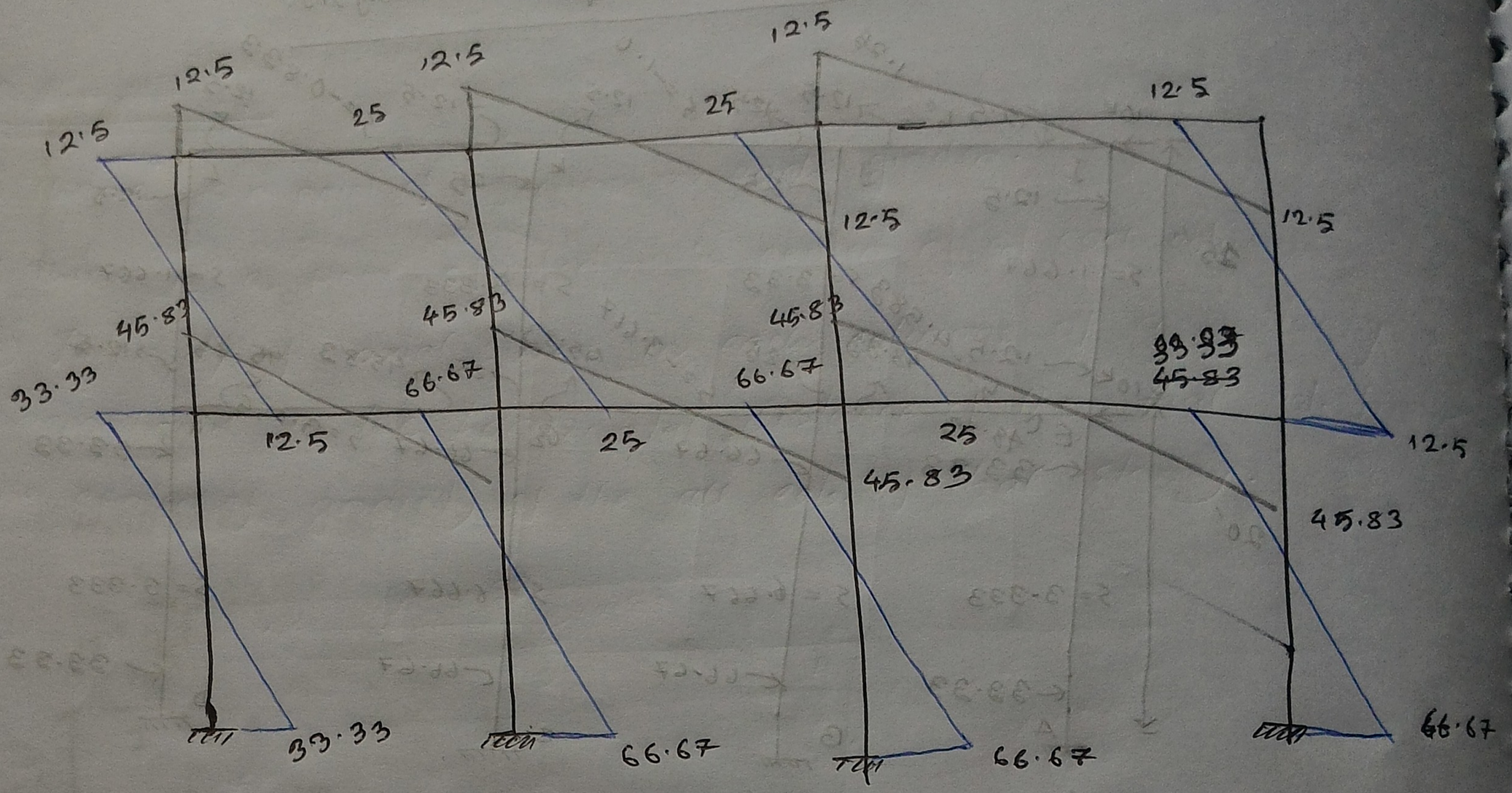
$$M_{C1} + M_{C2} = M_{G1} + M_{G2}$$



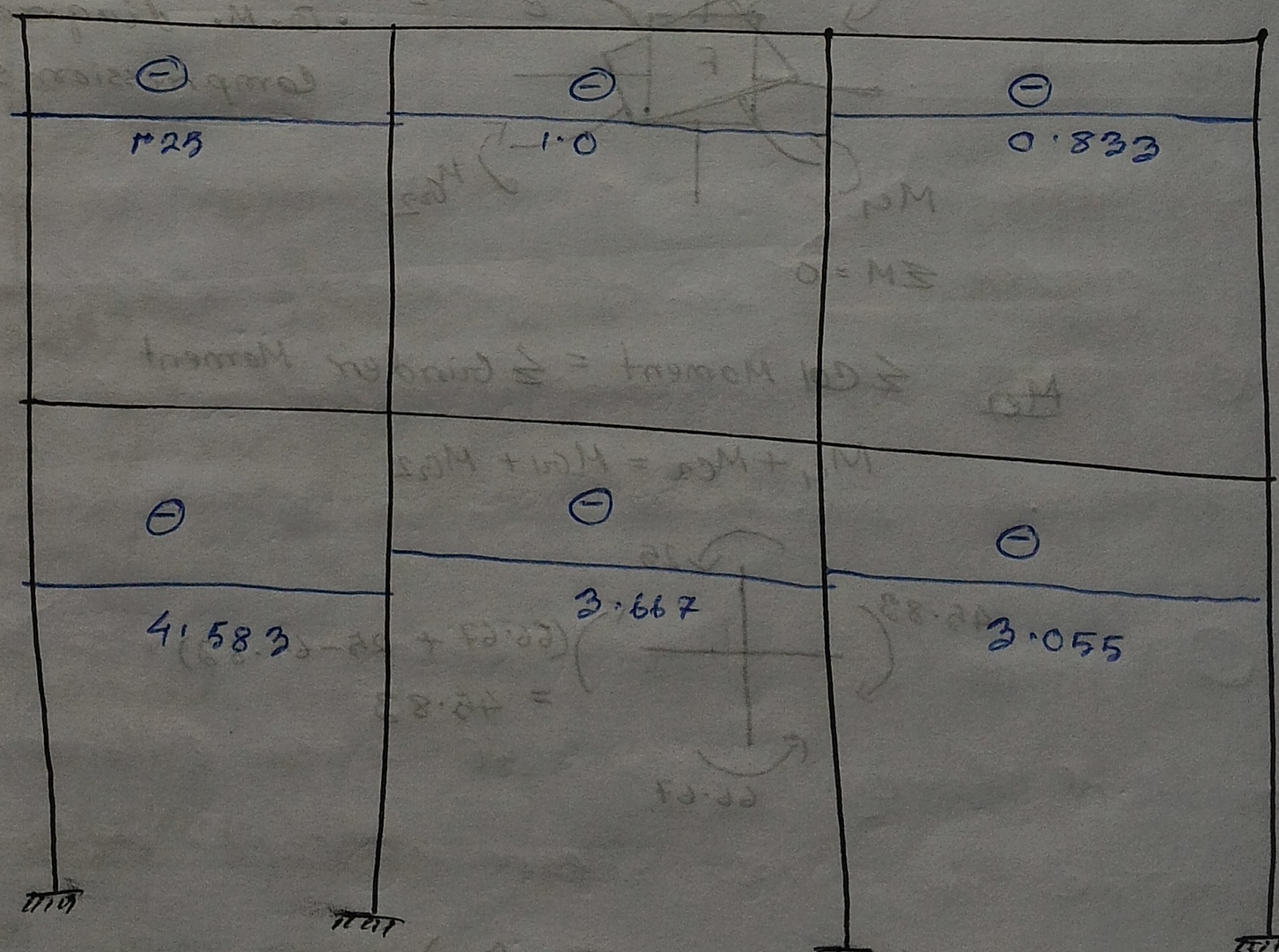
$\Sigma M = 0$

$\Rightarrow S \times L - 2M = 0$

or, $S = \frac{2M}{L}$

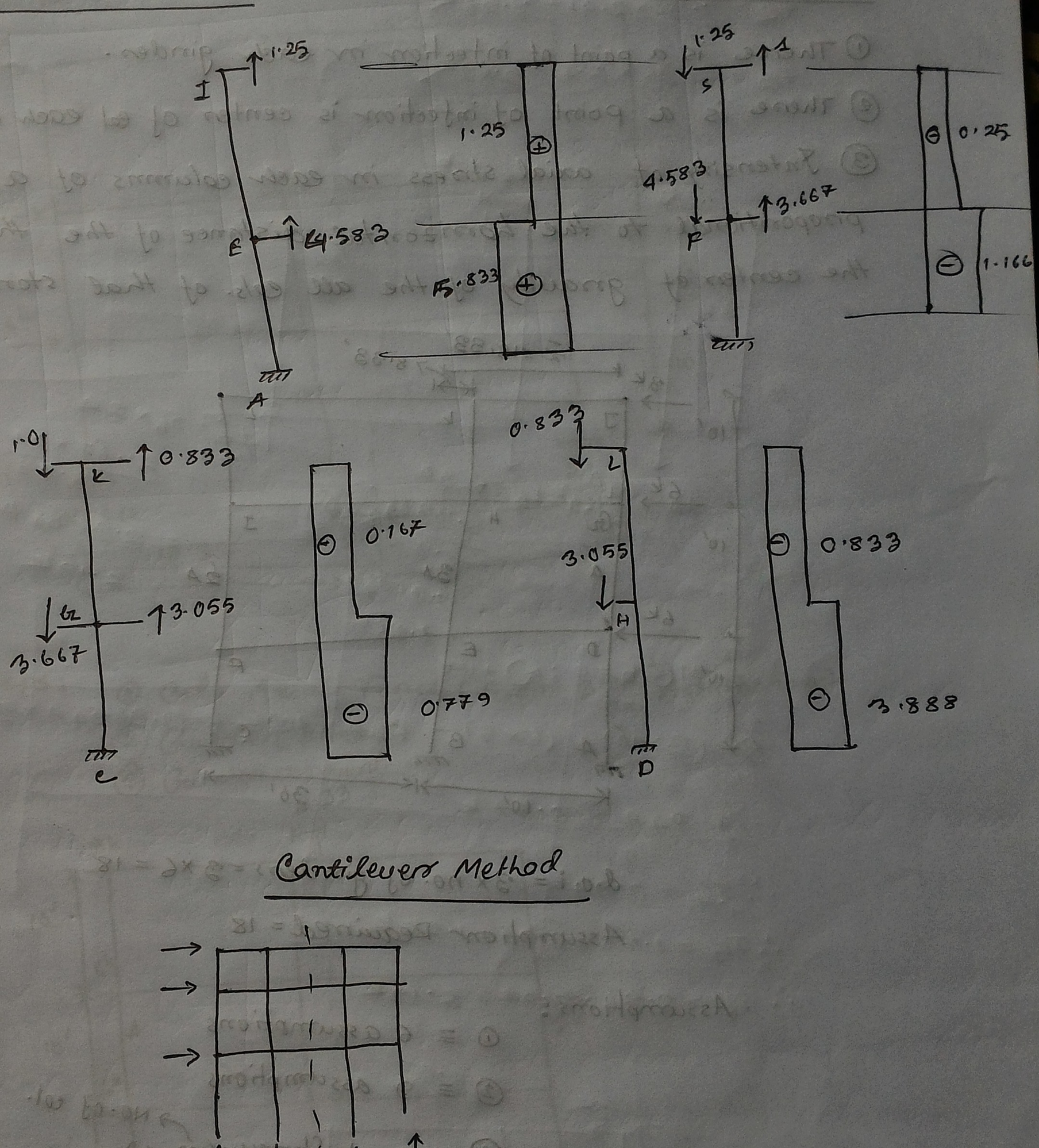


B.M.D:-

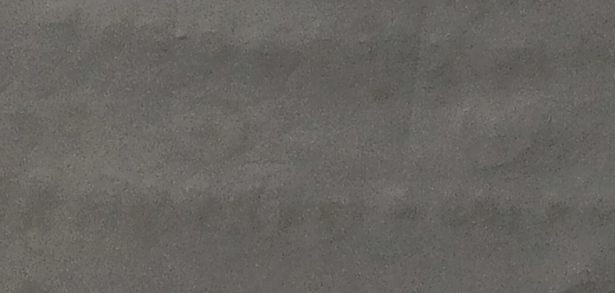


Girder S.F.D

Col. Axial force Dia.



Cantilever Method

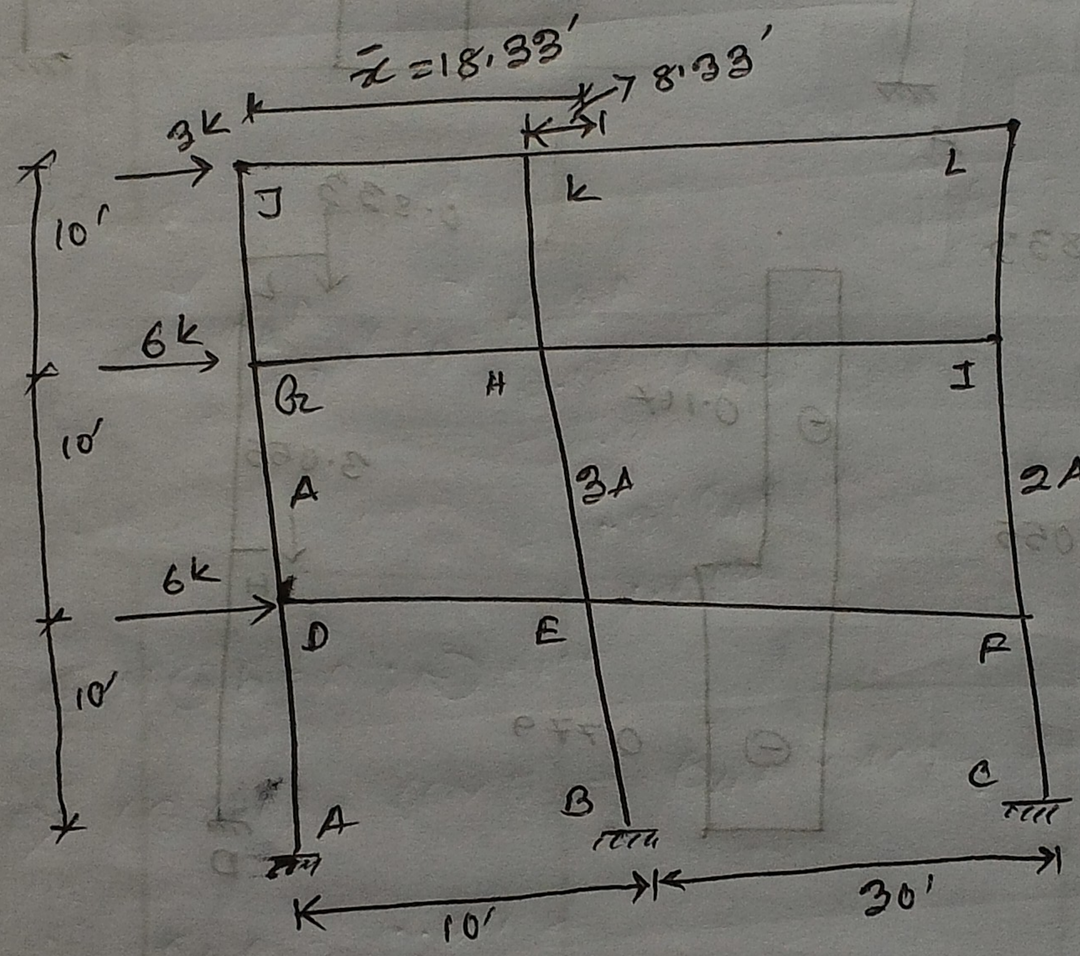


Assumptions :-

① There is a point of infection in each girder.

② There is a point of infection is center of each columns

③ Intensity of axial stress in each columns of a storey is proportional to the horizontal distance of the that col. from the center of gravity of the all cols. of that storey.



d.o.i = 3 × no. of girders = 3 × 6 = 18

Assumption Required = 18

Assumptions :

① ≡ 6 assumptions

② ≡ 9 assumptions

③ ≡ no. of storey × (m − 1) ∅ (m → no. of col.)

≡ 3 × (3 − 1) = 6

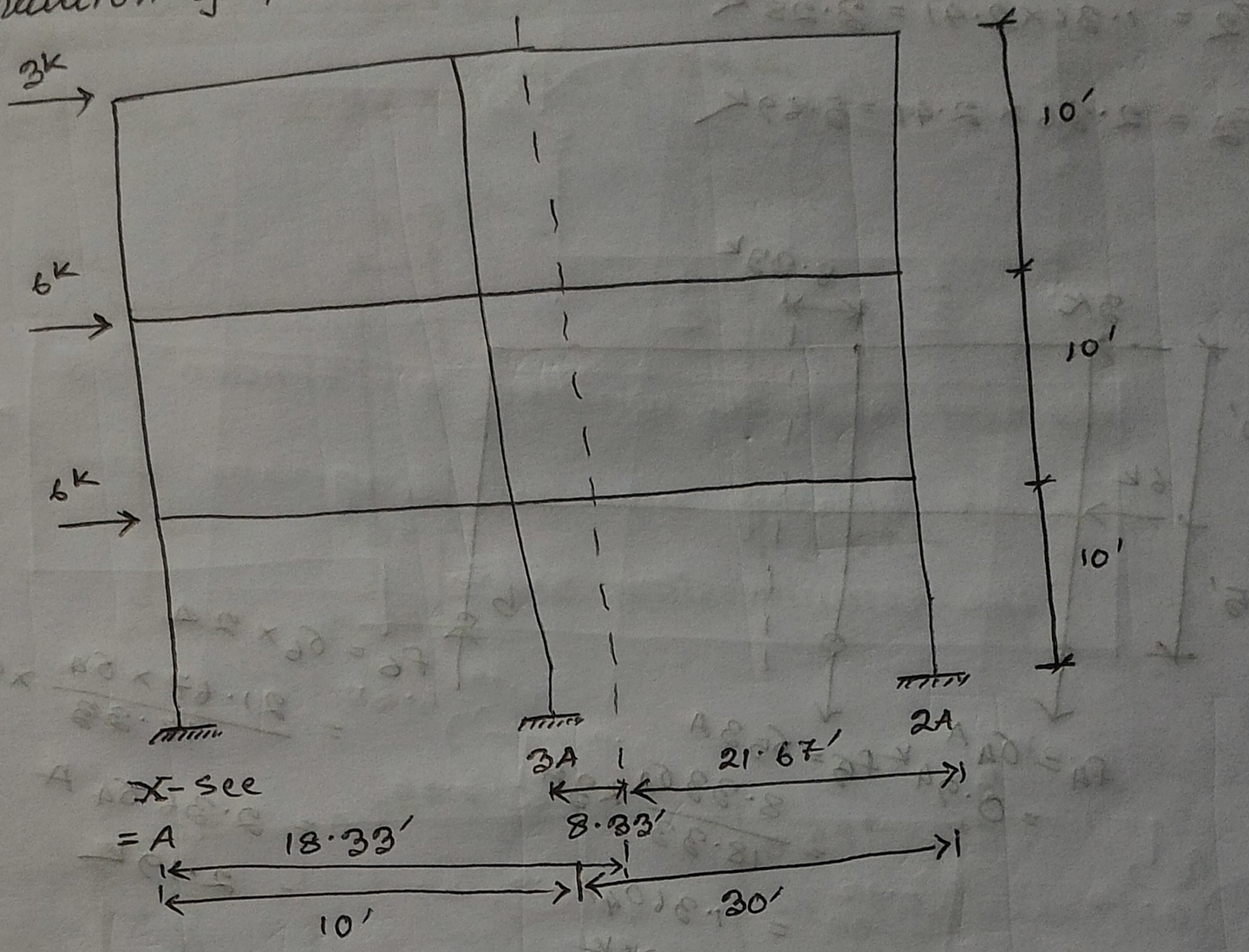
$$\bar{x} = \frac{(A \times 0) + (3A \times 10) + (2A \times 40)}{A + 3A + 2A}$$

$$= \frac{110A}{6A}$$

$$= 18.33'$$

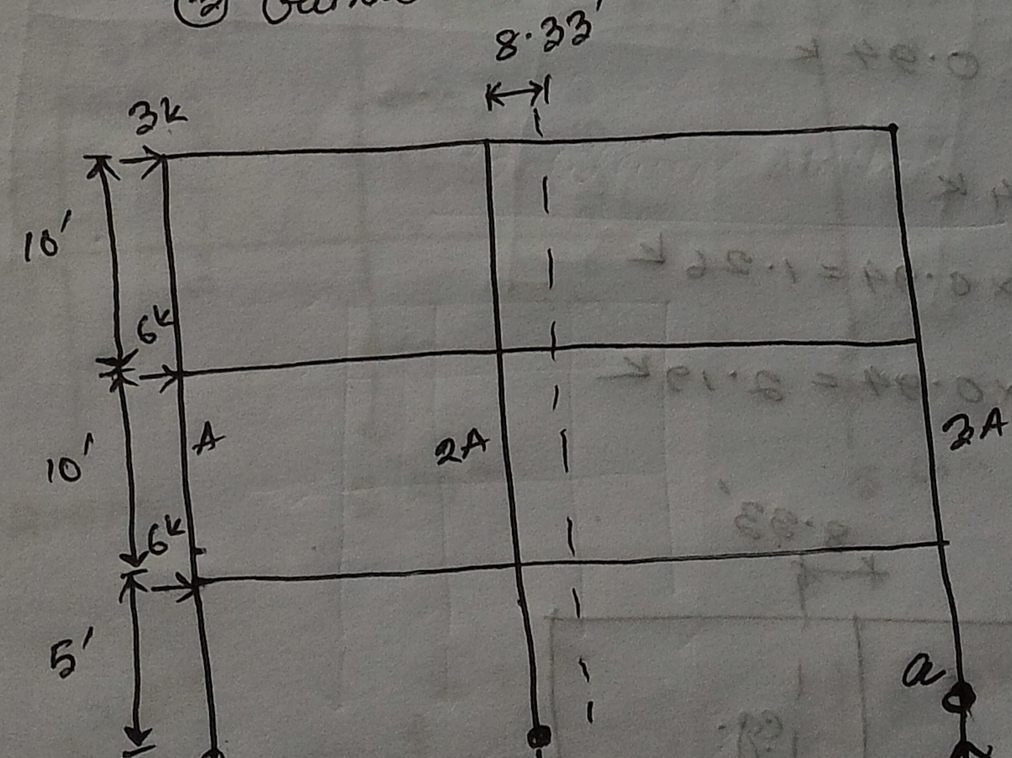
Structural Analysis and Design

Continuation of Previous Class :-



Steps: ① Col. Axial Force (From 3rd Assumption)
② Girder shear
③ Girder moment

Step 1:



$f_1 = \sigma_1 A = 2.41^K$

$f_2 = \sigma_2 \times 3A$
$= \frac{8.33\sigma_1}{18.33} \times A$
$= 1.36\sigma_1 A$
$= 3.28^K$

$f_3 = \sigma_3 \times 2A$
$= \frac{\sigma_1 \times 21.67}{18.33} \times 2A$
$= 2.36\sigma_1 A$
$= 5.69^K$

$\Sigma M @ a = 0$ (+)

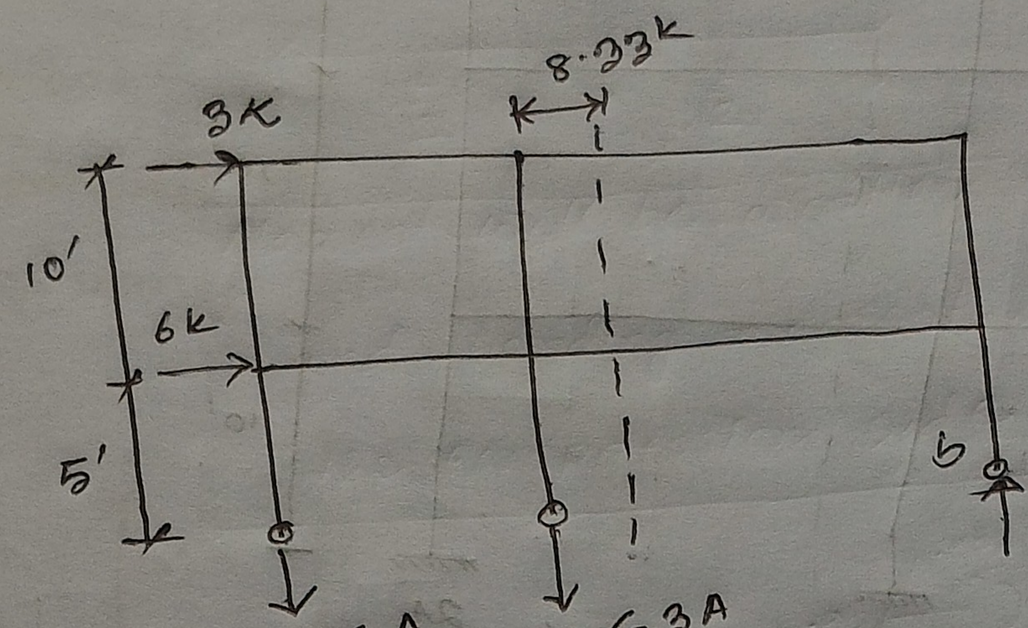
$\Rightarrow (3\times25) + (6\times15) + (6\times5) - 6A_1 \times 40 - 1.36\sigma_1 A \times 30 = 0$

$\therefore \sigma_1 A = 2.41^K$

$\therefore F_1 = 2.41^K$

$F_2 = 1.36 \times 2.41 = 3.28^K$

$F_3 = 2.36 \times 2.41 = 5.69^K$



$f_4 = 0.4\Delta$
$= 0.94^K$

$f_5 = 0.5\Delta$
$= \frac{8.33 \delta_4}{18.33} \times 3\Delta$
$= 1.36 \delta_4 \Delta$
$= 1.21^K$

$f_6 = 0.6 \times 2\Delta$
$= \frac{21.67 \times \delta_4}{18.33} \times 2\Delta$
$= 2.36 \delta_4 \Delta$
$= 2.19^K$

$\sum M @ 6 = 0 \; ; \circlearrowleft +ve$

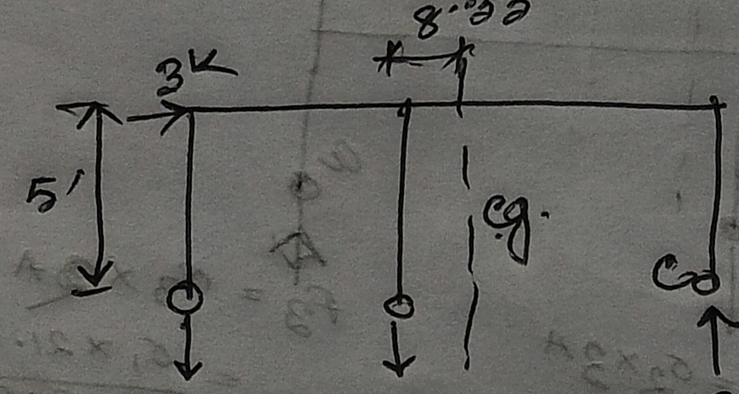
$\Rightarrow (3 \times 10) + (6 \times 5) - (\delta_4 \Delta \times 40) - \leftarrow (1.36 \delta_4 \Delta \times 30) = 0$

$\therefore \quad \delta_4 \Delta = 0.94^K$

$\therefore F_4 = 0.94^K$

$\therefore F_5 = 1.36 \times 0.94 = 1.26^K$

$\therefore F_6 = 2.36 \times 0.94 = 2.19^K$



$F_7 = \delta_7 \Delta$
$= 0.19^K$

$F_8 = \delta_8 \Delta 3$
$= \frac{8.33 \delta_7}{18.33} \times 3\Delta$
$= 1.36 \Delta \delta_7 = 0.25^K$

$F_9 = \delta_9 \Delta \times 2 = \frac{21.67 \delta_7}{18.33} \times 2\Delta$
$= 2.36 \delta_7 \Delta$
$= 0.44^K$

$\sum M @ C = 0 \; : \circlearrowleft (+)$

$\Rightarrow (3 \times 5) - (\delta_7 \Delta \times 40) \leftarrow - (1.36 \delta_7 \Delta \times 30) = 0$

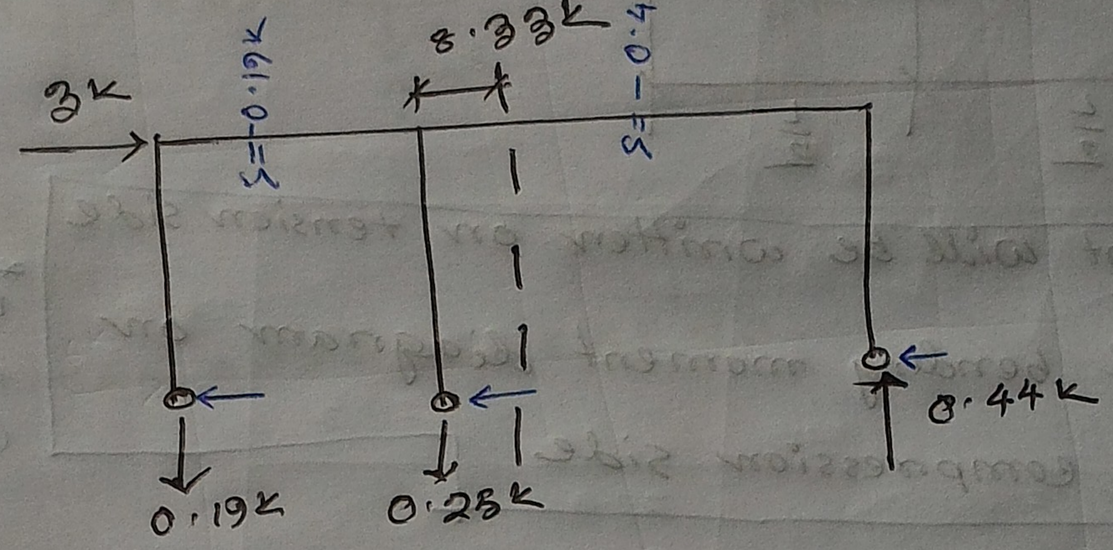
$\therefore \quad \delta_7 \Delta = 0.19^K$

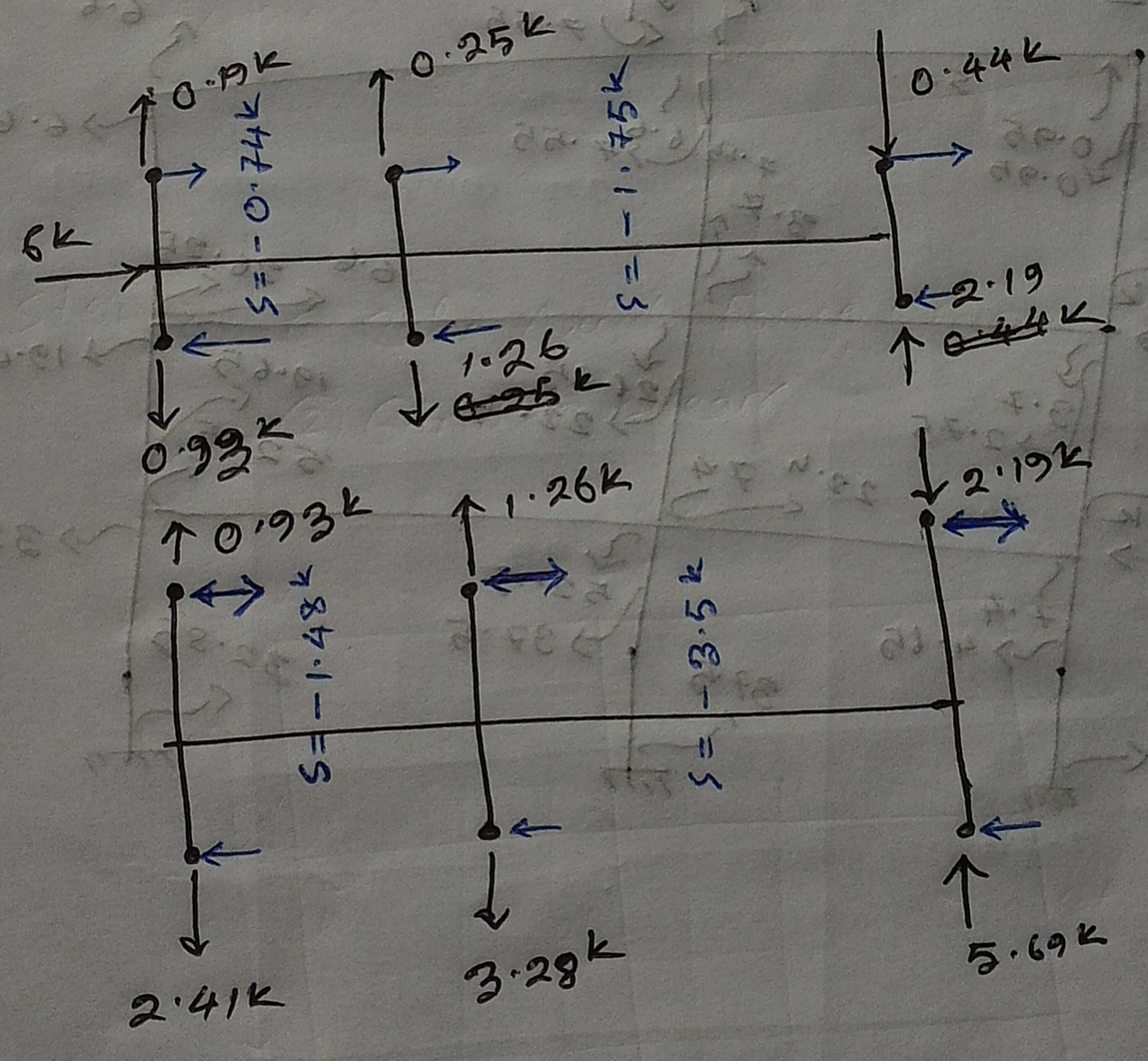
$\therefore F_7 = 0.19^K$

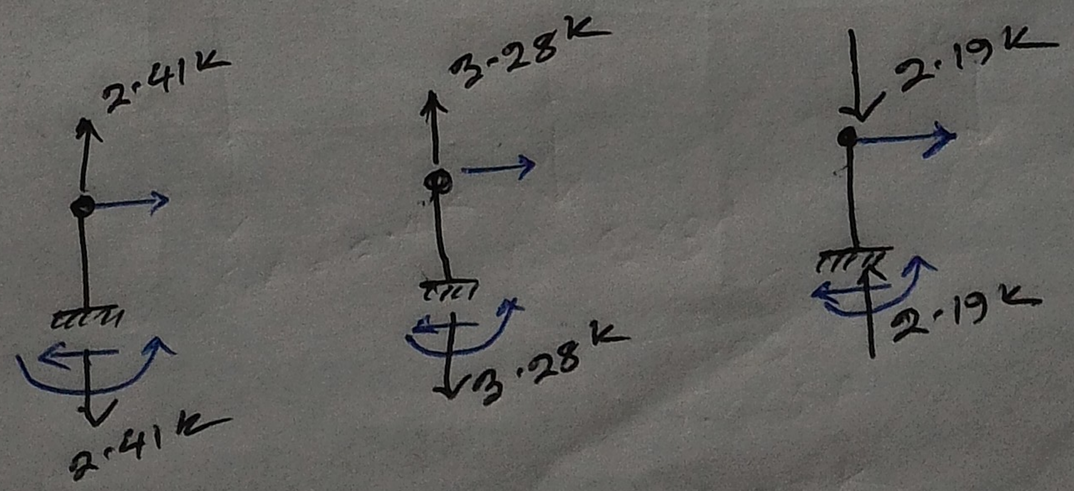
$F_8 = 1.36 \times \sigma_7 A = 1.36 \times 0.19 = 0.25^K$

$F_9 = 2.36 \sigma_7 A = 2.36 \times 0.19 = 0.44^K$

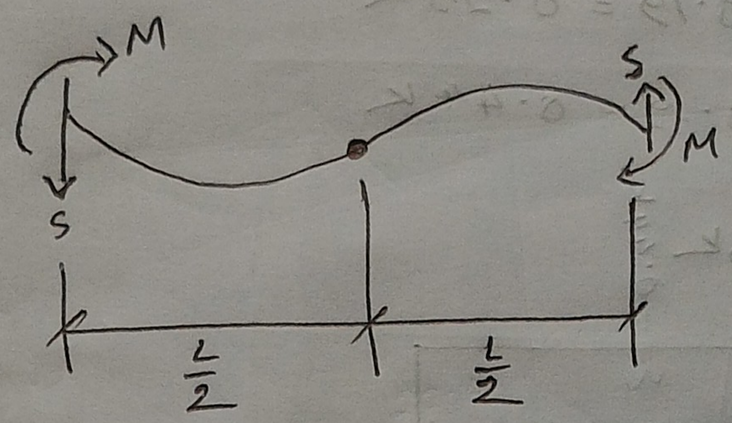
Step 2:-





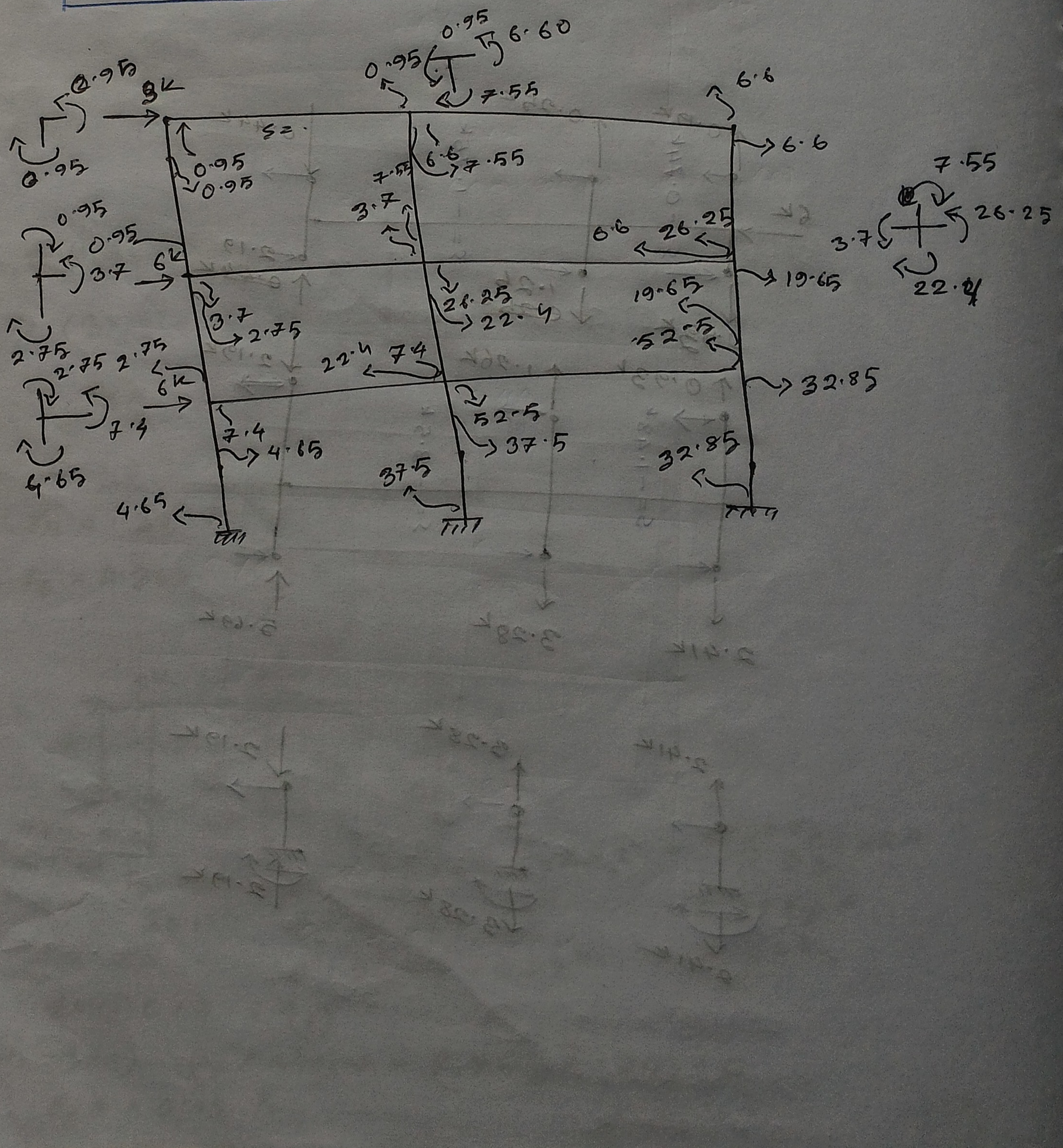


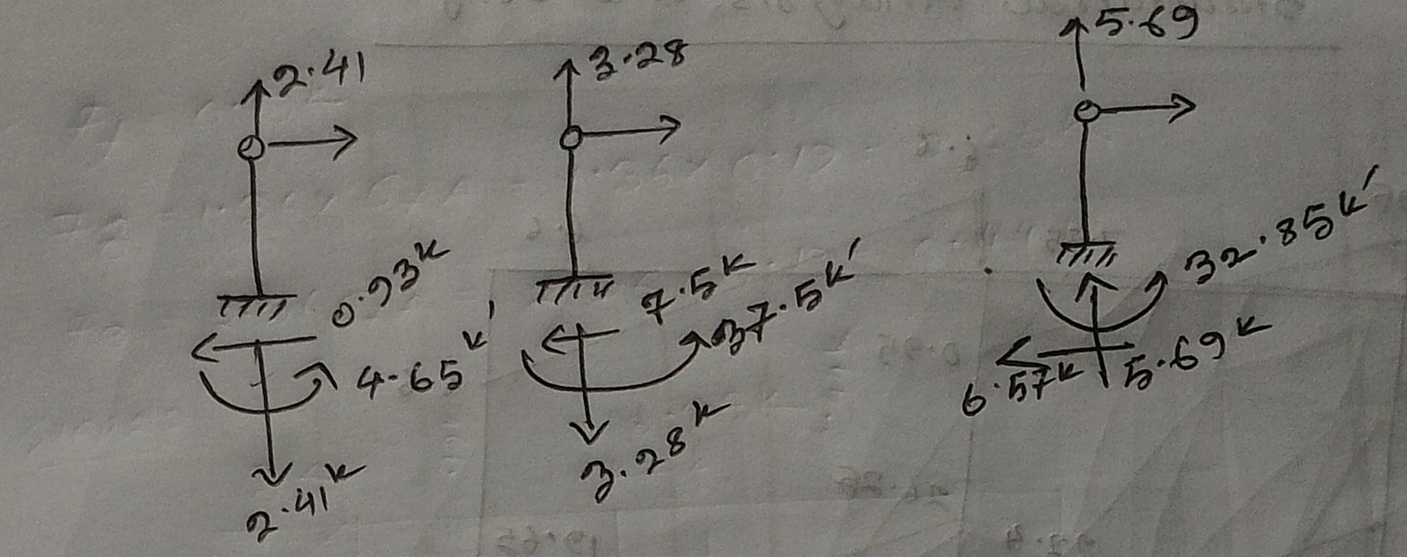
Step 3 (Girder Moment):-



Moment will be written on tension side and bending moment diagram on the compression side

*** Must be maintained (otherwise, No marks)



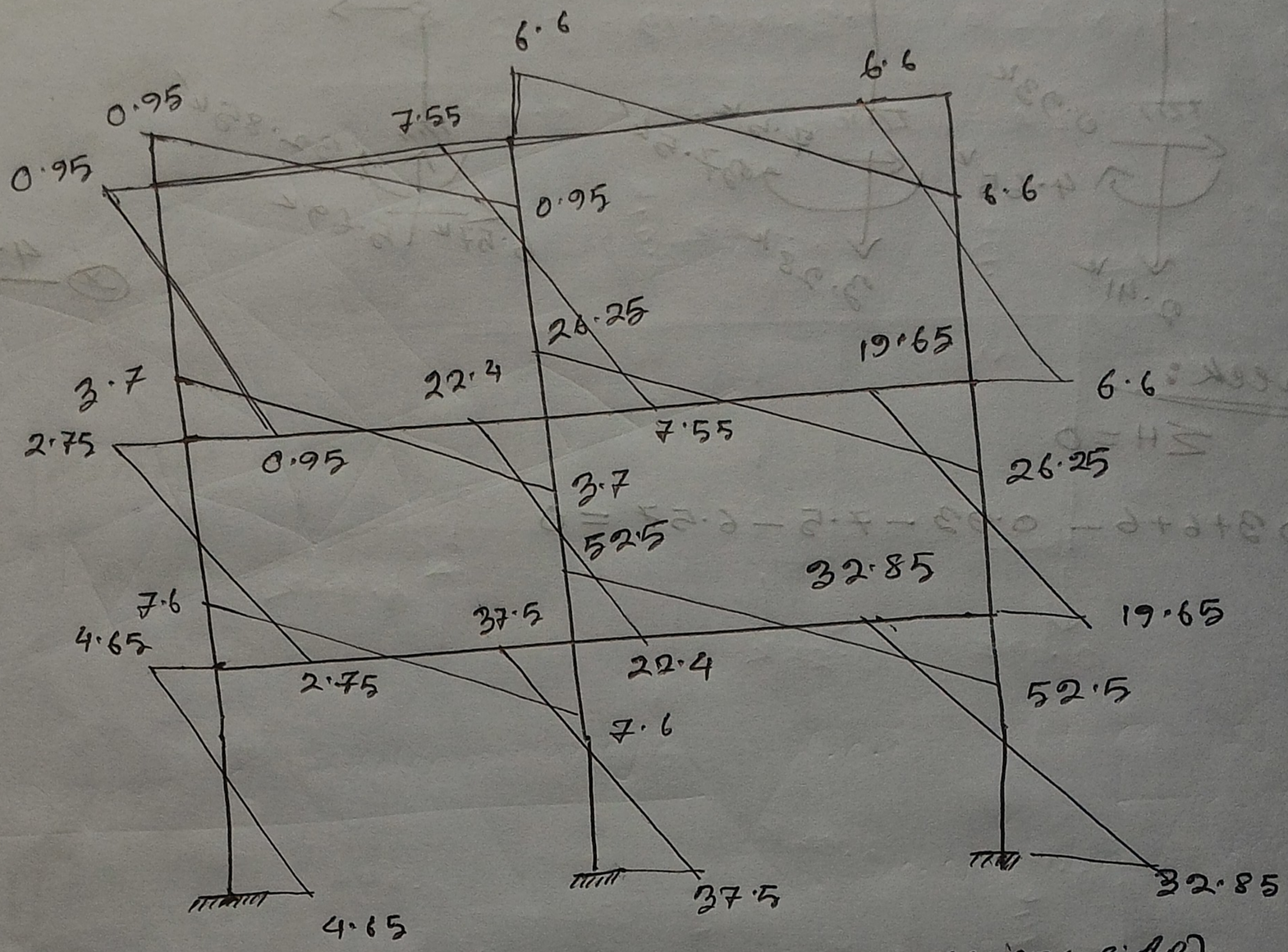


$\frac{4.65^k}{5} = 0.93^k$

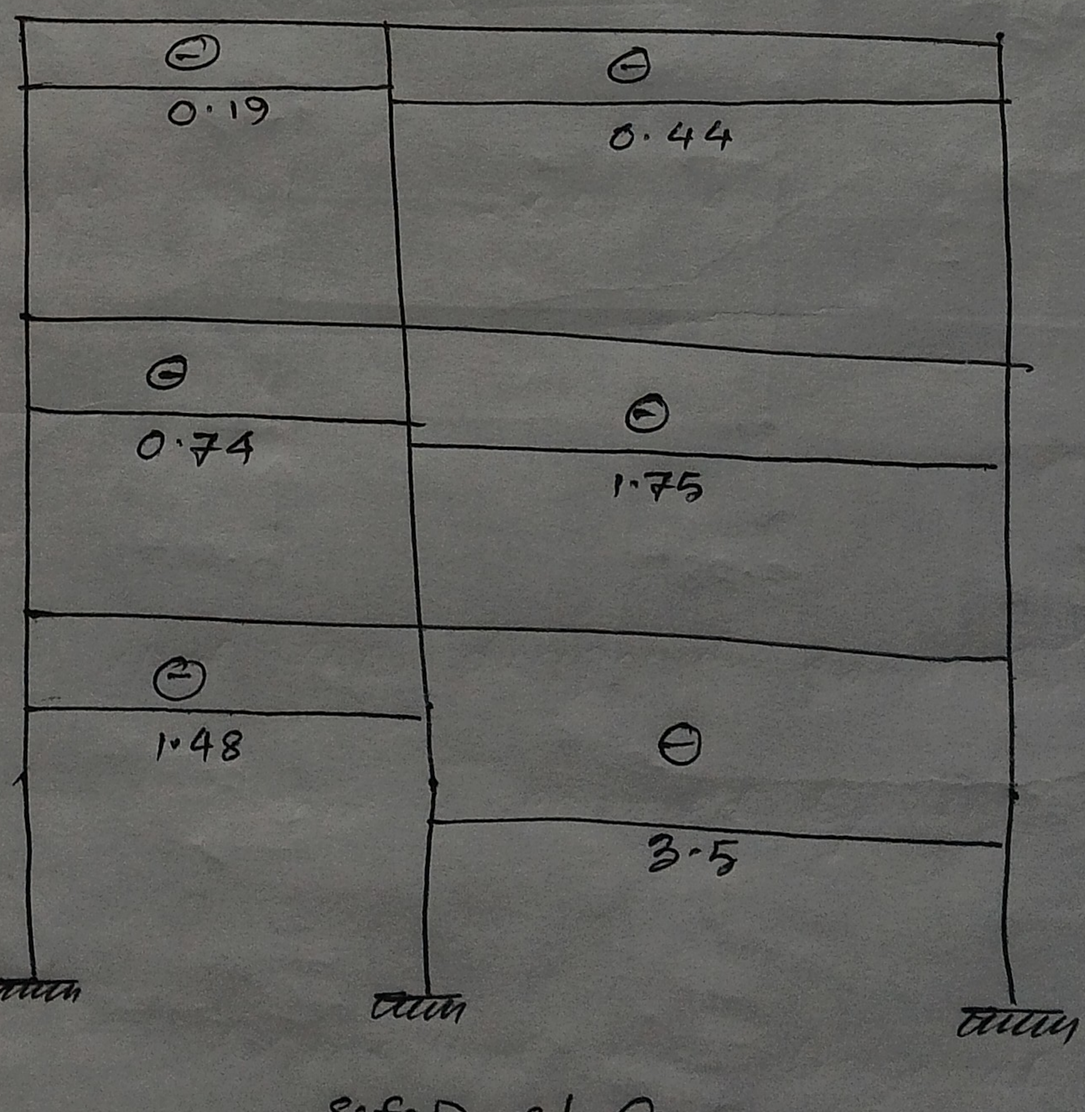
Check:

$\Sigma H = 0$

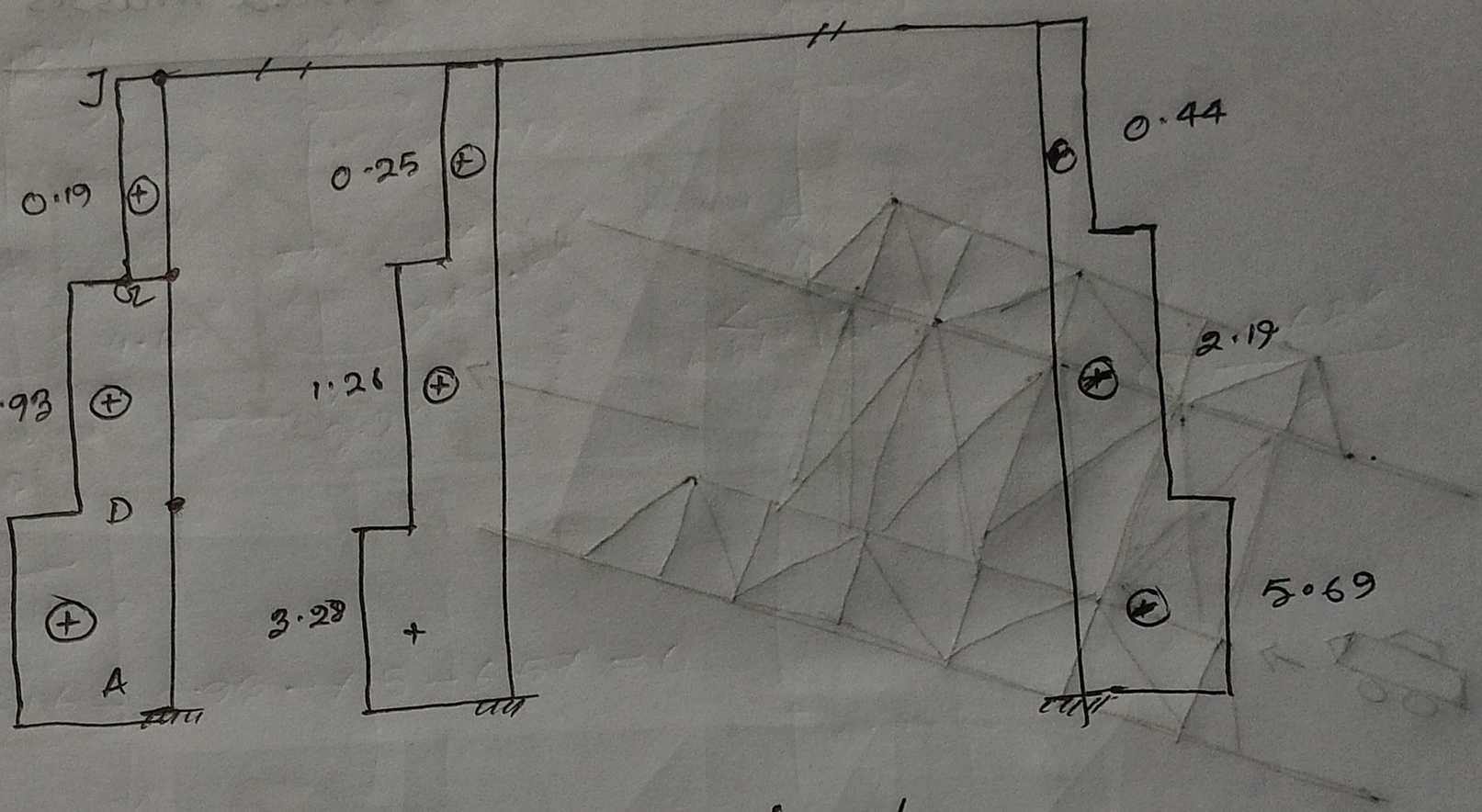
$\Rightarrow 3+6+6-0.93-7.5-6.57 = 0$



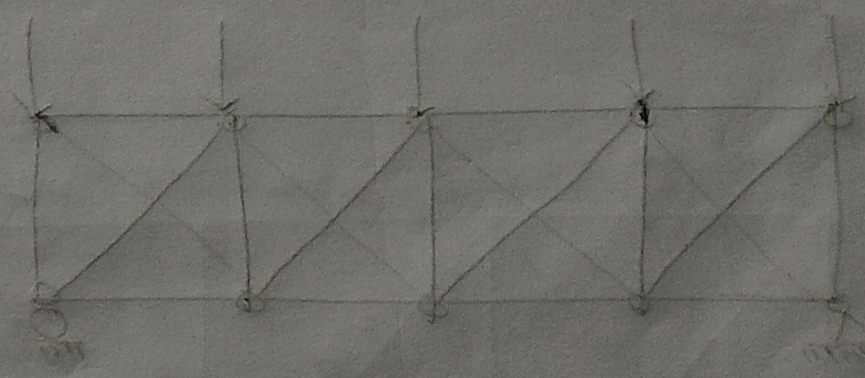
B.M.D. (on compression side)



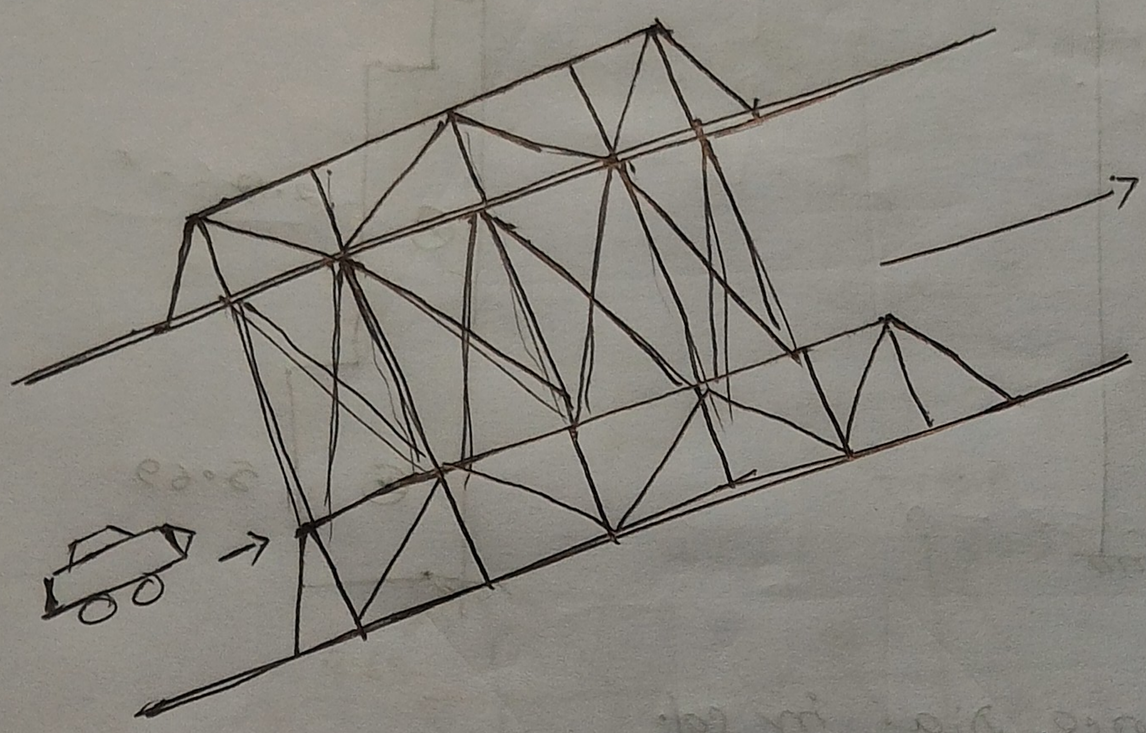
S.f.D. of Girders



Axial force Dia. in Col.



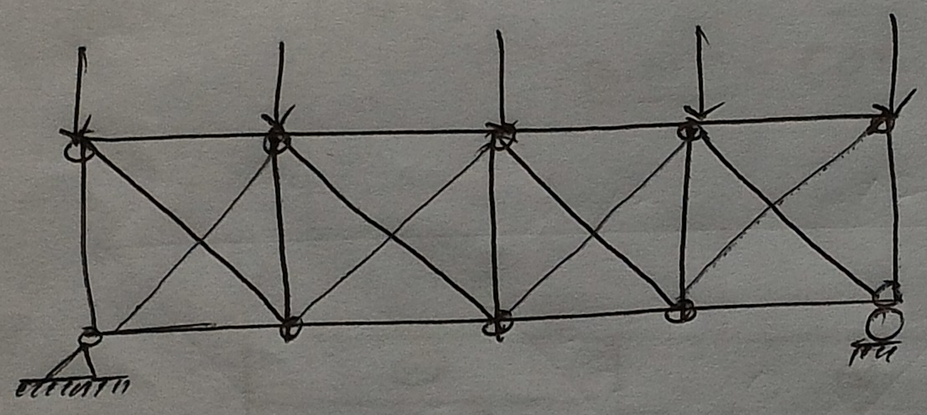
Approximate Methods - X braced Trusses



Degree of indeterminancy:

$$d.o.i = m + r - 2j$$

- m → No. of members
- r → Independent No. of Reactions.
- j → No. of joints



$$d.o.i = 21 + 3 - (2 \times 10) = 4$$

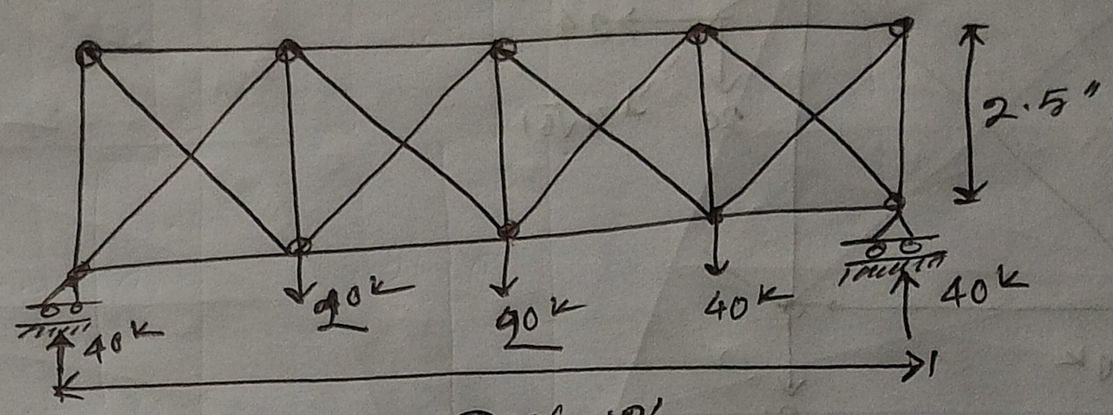
2 Methods

① Methods 1:

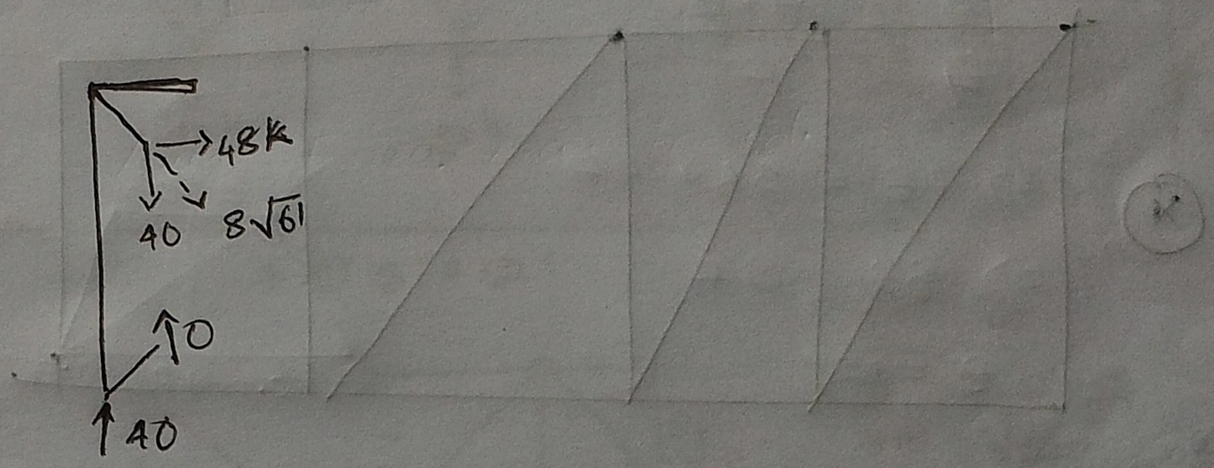
- Assume that the diagonal can carry tension only.
- Full pannel shear is carried by the diagonal tension.

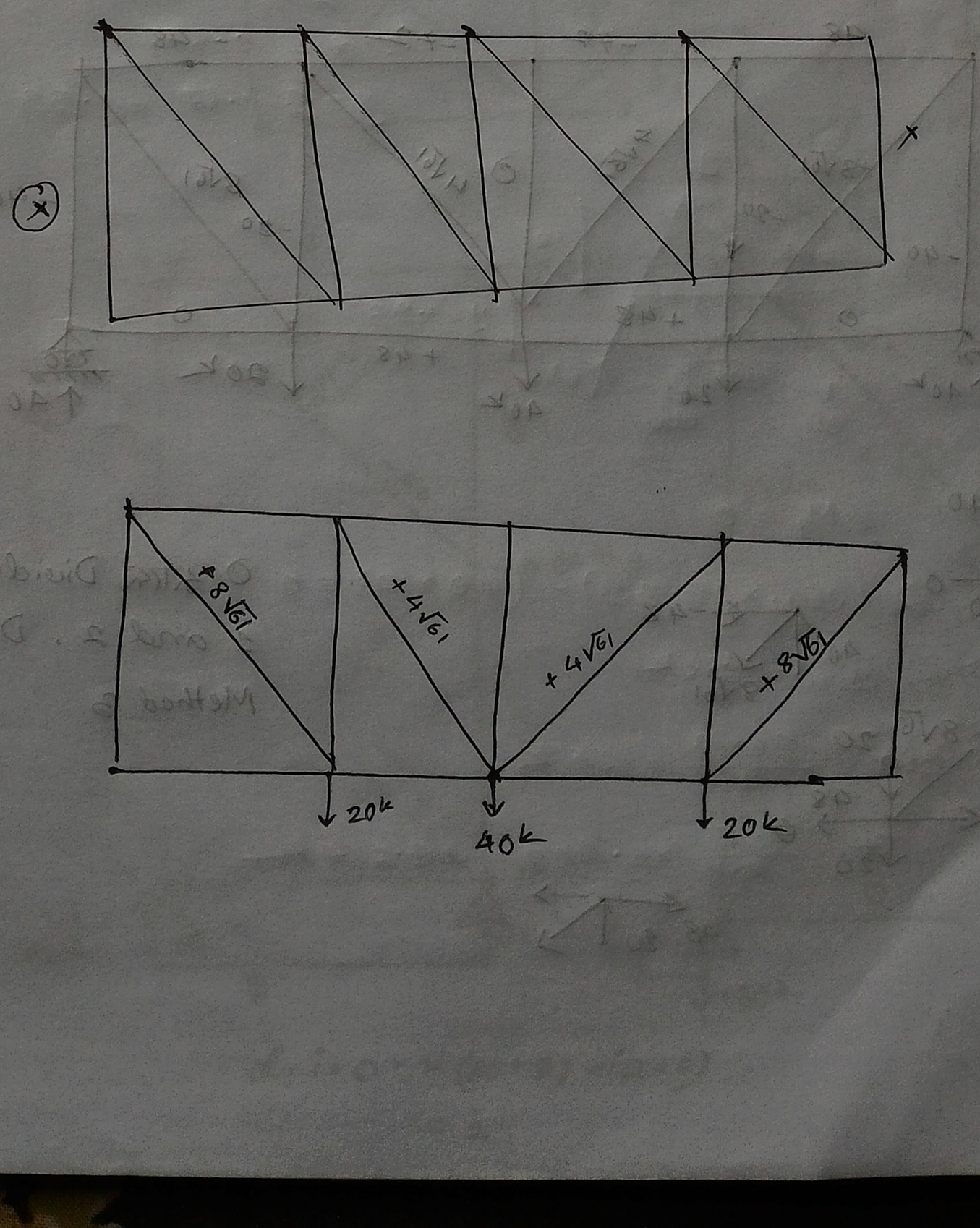
② Method 2:-

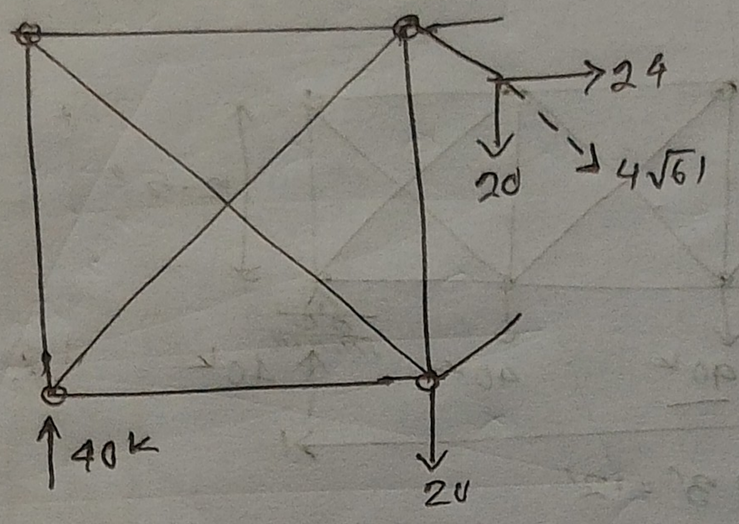
- Assume that the diagonal can carry both tension and compression.
- In each pannel, the shear is equally ~~die~~ divided betn the two diagonal.

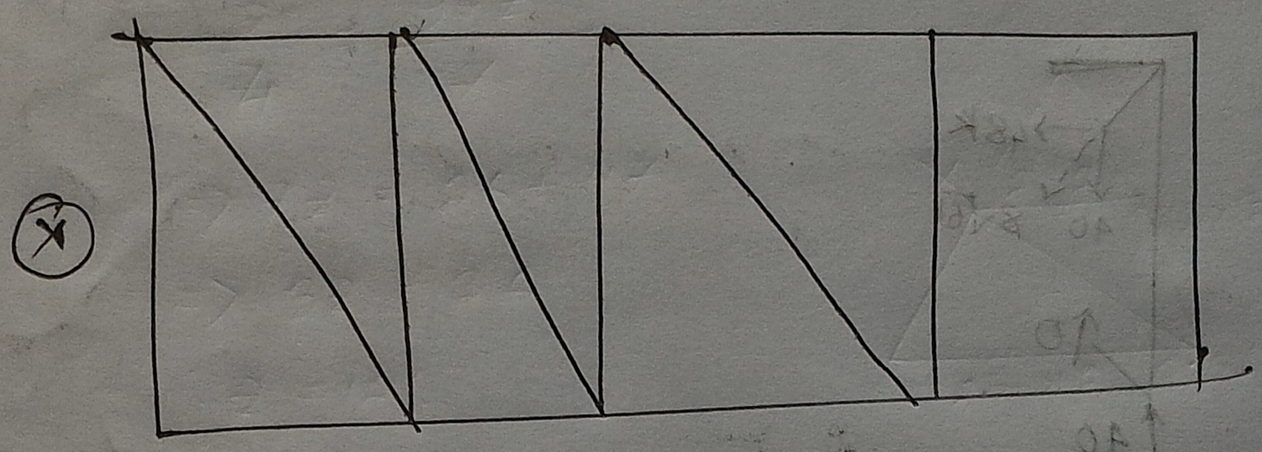

2.5"
40K
90K
90K
40K
40K
40K
4 @ 3' = 12'

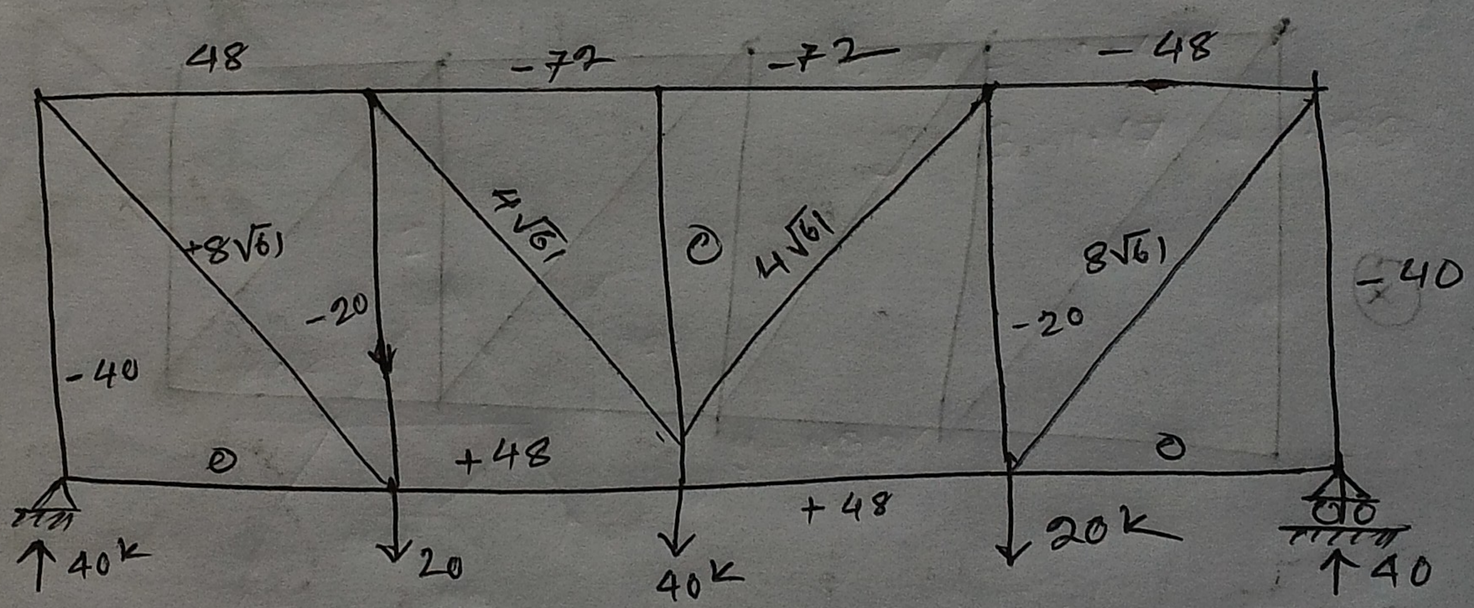
Method 1:

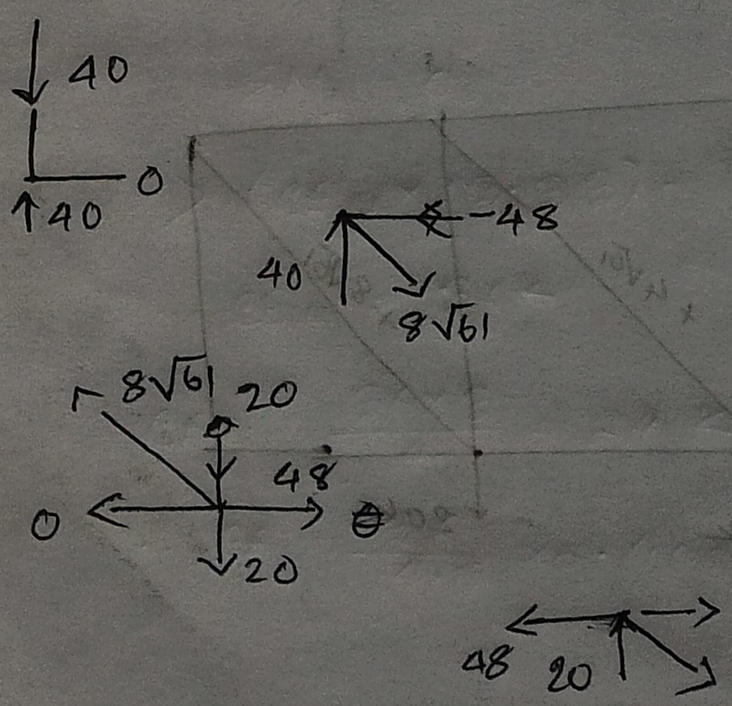

48K
40
8√61
0
40


+
+8√61
+4√61
+4√61
+8√61
20K
40K
20K







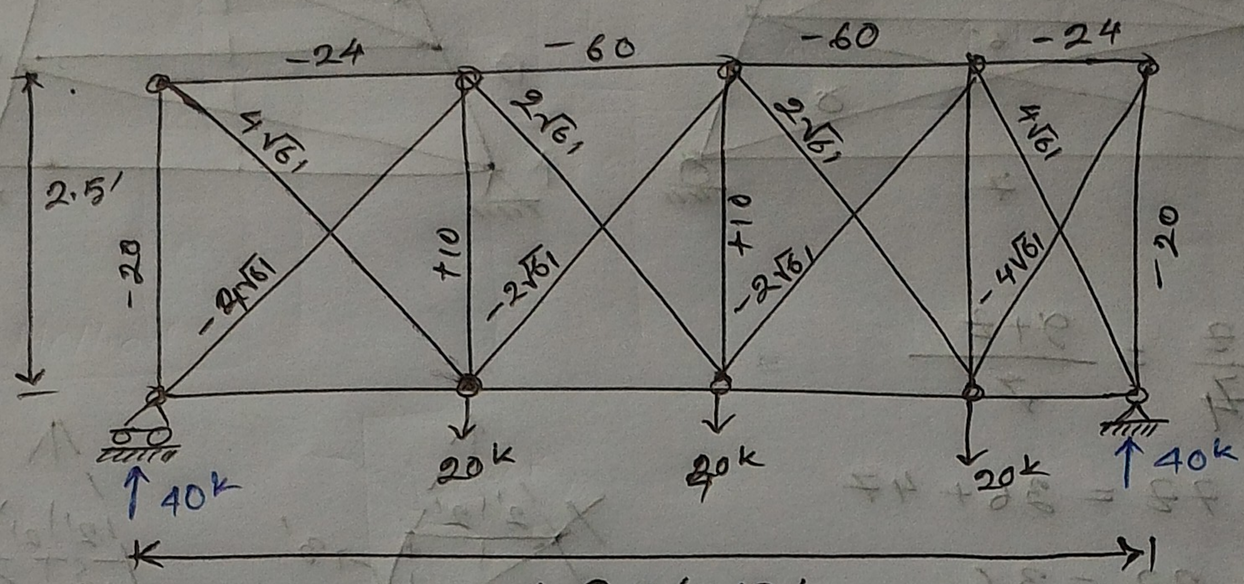


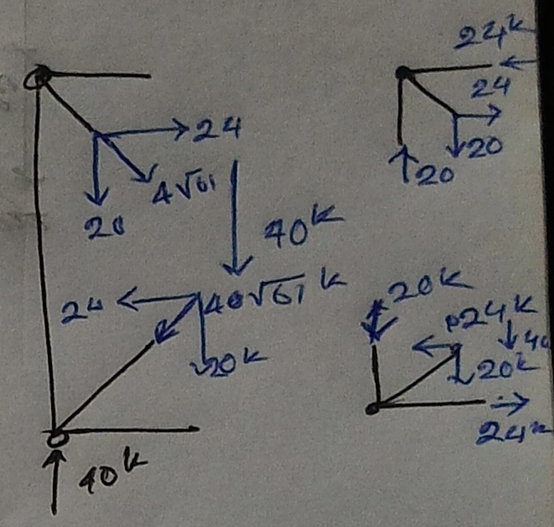
O আকারে Divider থাকলে Method 1 and 2. Divider না থাকলে Method 3

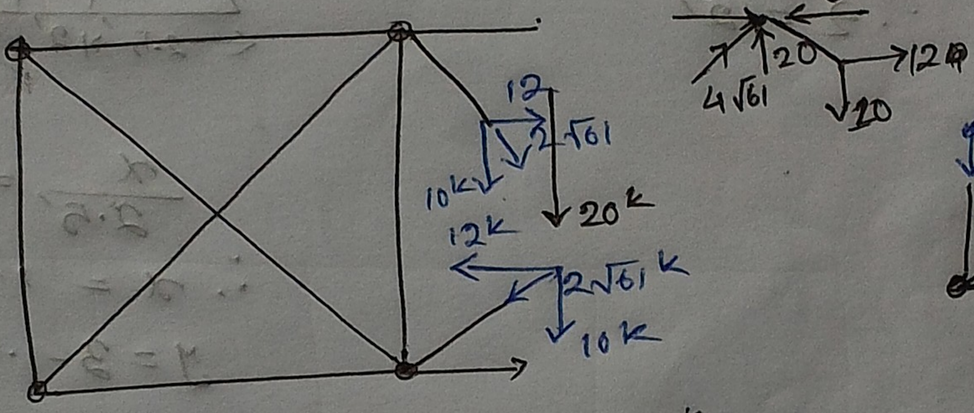
Structural Analysis and Design

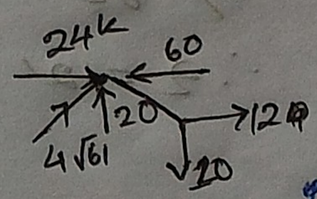
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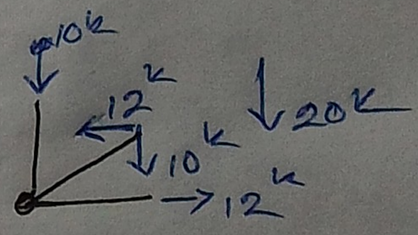
Solution By Method 2:-



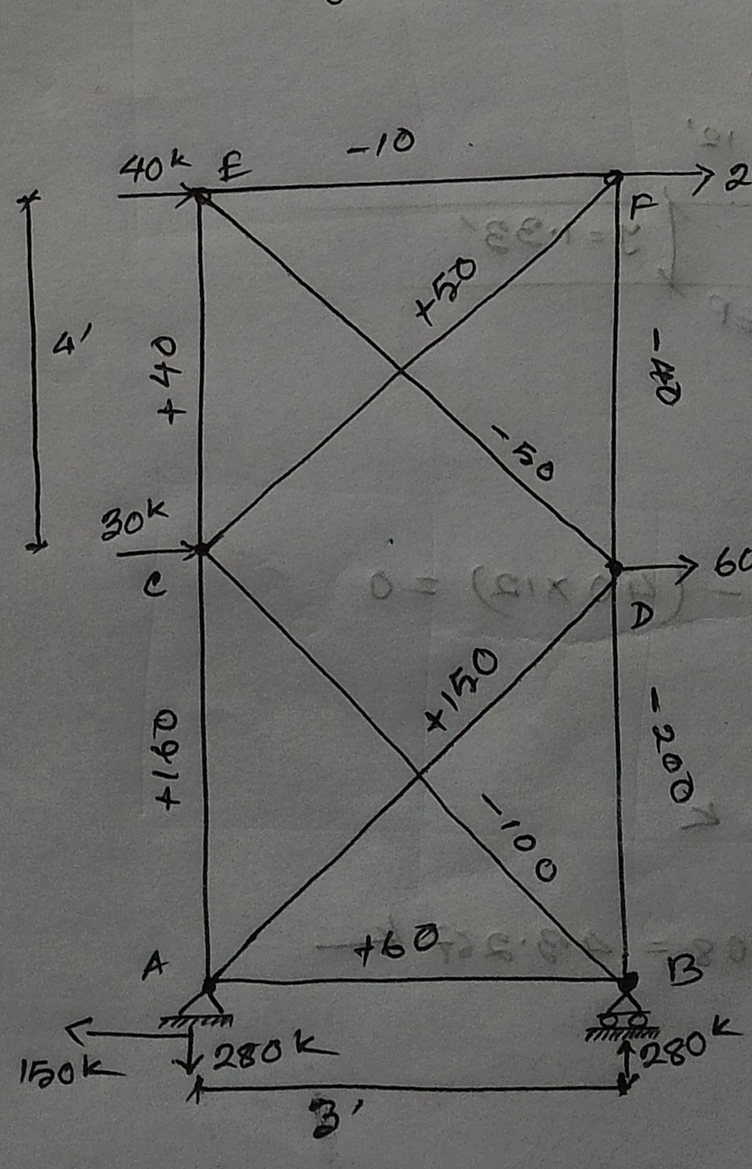


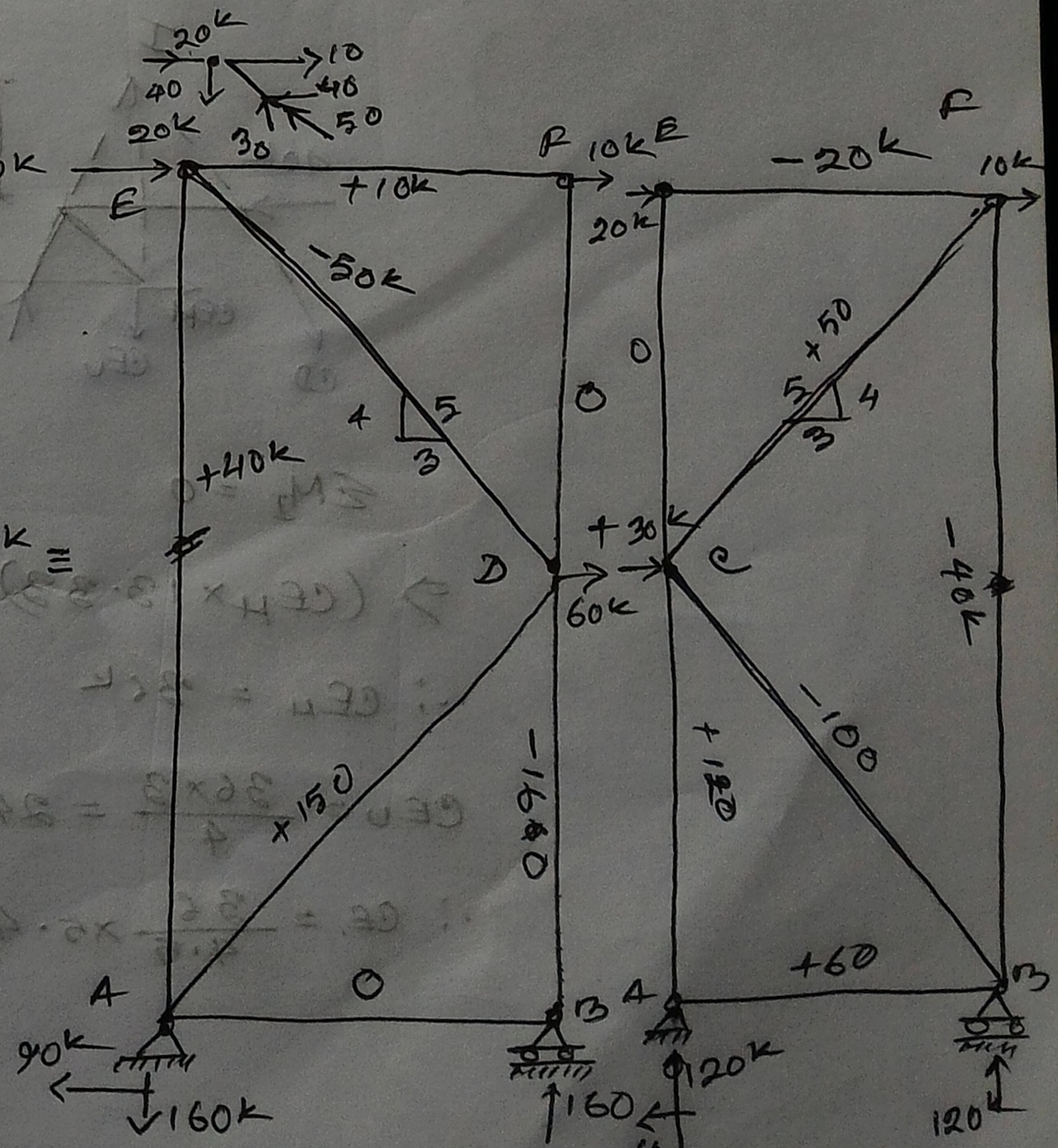






Horizontal Member - Method 3

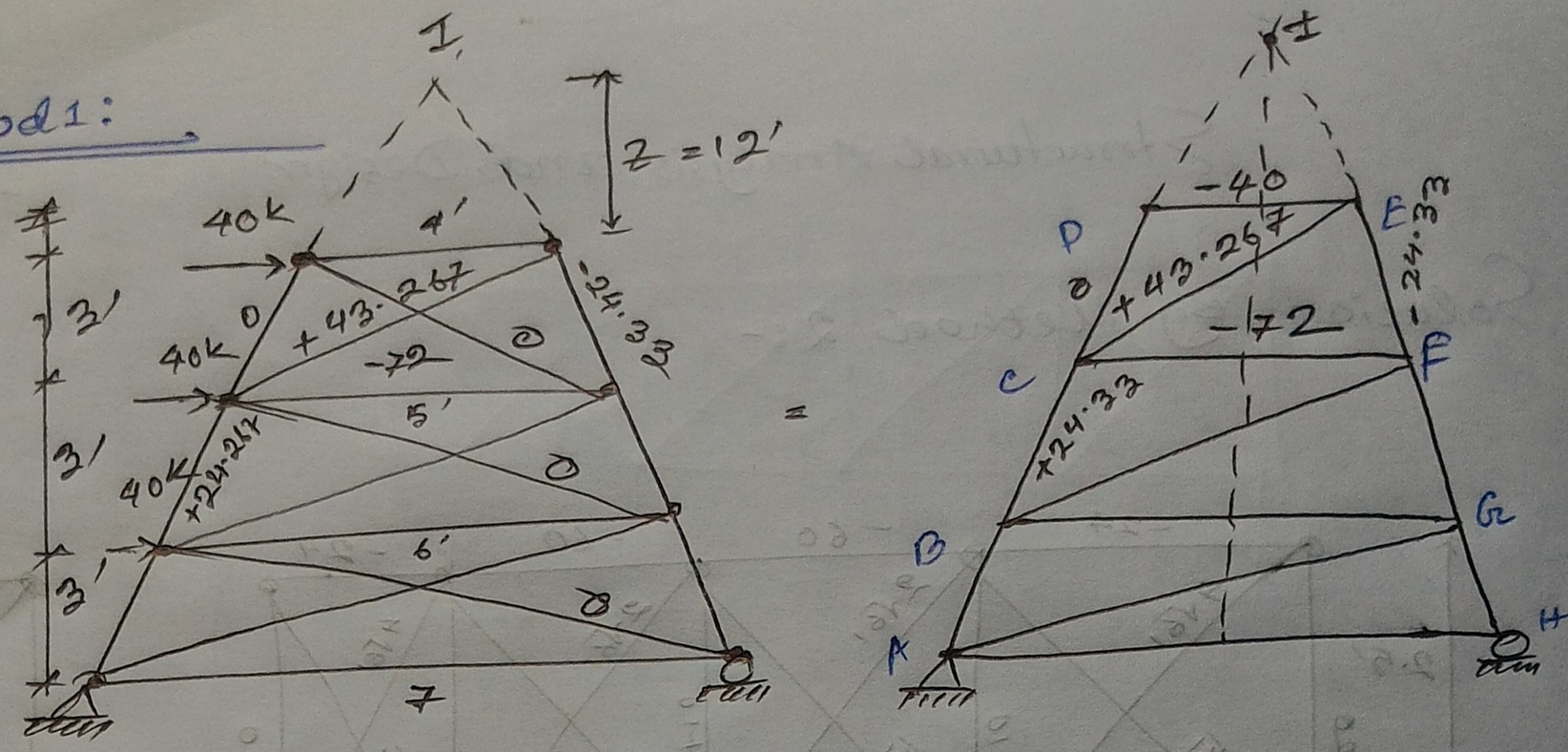




d.i.o. = (10+3) - (2×6)

= 1

* Method 1:

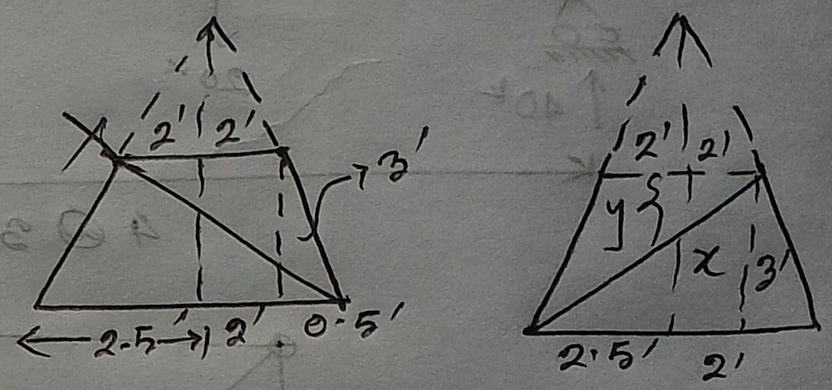


$$\frac{z}{4} = \frac{9+z}{7}$$

$$\Rightarrow 7z = 36 + 4z$$

$$\Rightarrow 3z = 36$$

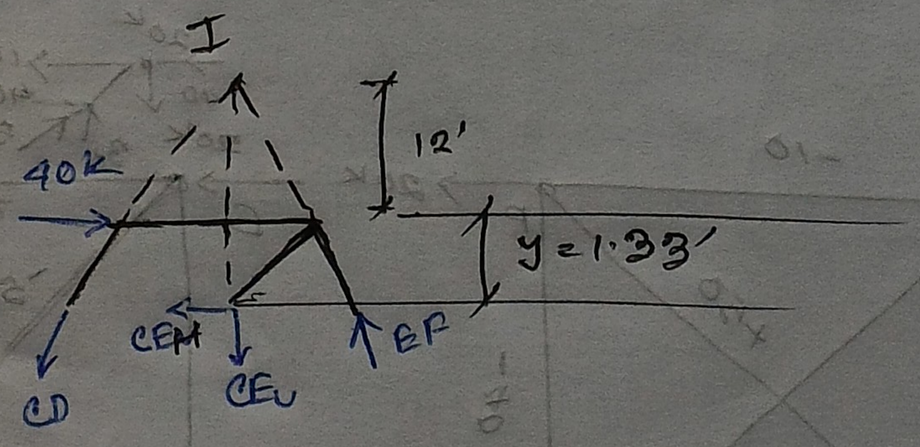
$$\therefore z = 12'$$



$$\frac{x}{2.5'} = \frac{3'}{4.5'}$$

$$\therefore x = 1.67'$$

$$y = 3 - 1.67' = 1.33'$$



$$\Sigma M_I = 0$$

$$\Rightarrow (CE_H \times 13.33) - (40 \times 12) = 0$$

$$\therefore CE_H = 36K$$

$$CE_V = \frac{36 \times 3}{4} = 24K$$

$$\therefore CE = \frac{36}{4.5} \times 5.408 = 43.267K$$