14.02.15

(1)

Taufiq Sir

CE 311

Structural Analysis & Design I

- Prerequisites → CE 213, CE 211, CE 101
- Teachers:

  2 classes — Zakaria Ahmed Sir

  1 class — Abdur Rouf Sir

  1 class — Dr. A. M. M. Taufiqul Anwar Sir

  Room no. - 419

- References:

① Elementary Structural Analysis by Charles Head Norris, John Benson Wilbur & Senol Utku (3rd edition, McGraw Hill International Edition, 1977)

② Theory of Simple Structures by T. C. Shedd & J. Vawter (2nd edition, 1941, Wiley International Edit

③ Bangladesh National Building Code (BNBC), 1993

·02·15

1 Taufiq Sir

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3. Bangladesh National Building Code (BNBC), 1993

Course Outline :

Stability & determinacy of structures ; analysis of statically determinate trusses and arches ; influence of lines ; moving loads on beams, frames and trusses ; analysis of suspension bridges

y T.A
o
Sir

Wind & Earthquake loads ; approximate analysis of statically indeterminate structures : braced trusses, portal method, cantilever method and vertical load analysis of multistoried building frames ; deflection of beams, trusses and frames by virtual work method

- Must bring Sheets in this class

Class Test : On 5th or 6th week .

Sir will take 1 CT (5 th week) . May take another CT later .

- BNBC → for Sir's part (Wind & Earthquake loads)

BNBC → was enacted as law in 2006 .
→ can get in library
→ handout is BNBC given by sir

Structure → Norris & Wilbur
Structure → Shedd & Vorster
} may collect from library

- Approximate Analysis is for indeterminate structures

Loads on Structures

- **Dead Load** (fixed during structure life)
- **Live Loads** (changes during life time)
  - Occupancy Load (Buildings)
  - Moving Load (Bridges)
- **Lateral Load**
  - Wind Load
  - Earthquake Load
- **Other Loads**
  - snow
  - rain
  - temperature etc.

Lateral loads are very important for design of large structures.

Eg. → tall buildings ; long span bridges ; towers ; Aircraft Hangers , Mill bents .

Wind loads → effect on building is very complex.
→ aerodynamics is involved
(have to design aerodynamically)

28.02.15

(2)

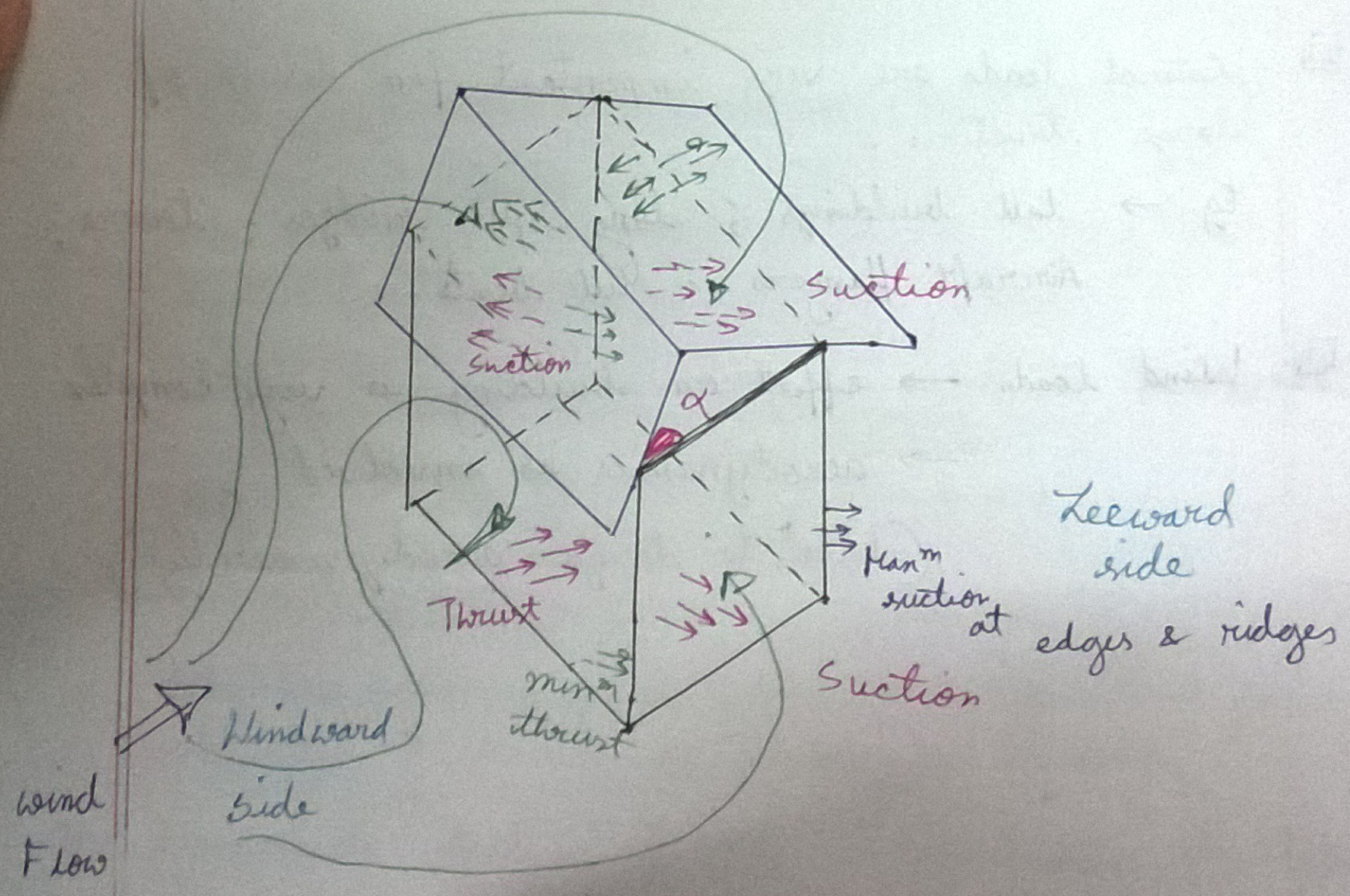
Taufiq Sir

Wind Forces

Aerodynamics of Buildings:

Flow of wind around buildings is an extremely complex process and depends on many ~~so~~ variables:

- building size & shape
- type of exposure
- local topography
- random nature of wind



Thrust or +ve pressure develops at windward side.

At sides & leeward side → Suction → −ve pressure.

On roof, both +ve & −ve pressure can develop.

α = Roof incidence

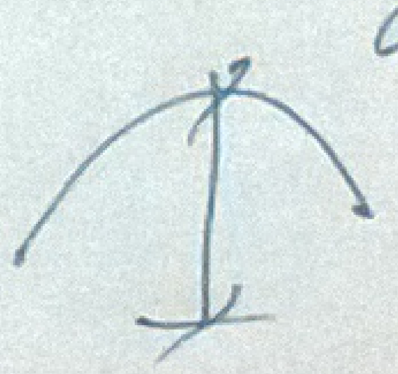
If $\alpha > 30^\circ$ → Thrust

If $\alpha < 30^\circ$ → Suction

If more steeper → then won't blow away by wind

Suction max^n ~~at~~ when $\alpha \approx 0^\circ$

at $\alpha = 10^\circ$



To evaluate complex wind pressure on structures

- by Properly conducted by Wind Tunnel Test (expensive & essential for complex structures)
- by Code Guidelines (approximate)

Major Building Codes & Standard for Evaluation of Wind Load:

- BNBC, 1993 for Bangladesh.

US codes
- ANSI A 58.1, 1982 & updates (American National Standard Institute)
- UBC, 1997 & updates (Uniform Building codes)
- ASCE, 7-95 & updates. (American Society of Civil Engineers)

British Code
- CP 3, Chapter V, part 2

Parameters Related to Wind Forces:

- Basic Wind Speed, [V_b in BNBC]
- Exposure Category [Exposure A, B, C in BNBC]
- Structure Importance Coefficient [C_I in BNBC]

- Combine height & Exposure Coefficient [C_z in BNBC]
- Wind Gust Effects [C_G in BNBC]
- Pressure Coefficients [C_p in BNBC]

Basic Wind Speed:

Def^n!

Fastest mild wind speen in km/hr corresponding to the level of 10 meters above the ground of terrain exposure ~~B~~ B & associated with an annual probability of occurrance of 0.02

(Fig 6.2.1 & Tab 6.2.8. in BNBC)

→ Tab 6.2.8

If not oblained from it, move to fig 6.2.1.

→ Fig 6.2.1 → Wind Regions

→ ~~isotachs~~ on isotachs

If not on isotach → connect 2 isotach by a ⊥ line & then interpolate

→ Interpolation bet^n isotachs.

\# Costal → Exposure B
buildings → " A
In bet^n → " C

(2.4.4) → BNBC

Wind Load Provisions of BNBC, 1993:

Sustained Wind Pressure at height, z (in metre)

$$q_z = C_c C_I C_z V_b^2 \quad \cdots (2.4.1)$$

[V_b in km/hr → kph
q_z in KN/m²

Article 2.4.6.6 → BNBC

C_c → a conversion coefficient → since unit is changes.

C_z → from Table 6.2.10

V_b → Table 6.2.8 or fig 6.2.1.

On

→ ~~isotachs~~ Isotachs

If not on isotach → connect 2 isotach by a ⊥ line & then interpolate

→ Interpolation betn isotachs.

Coastal → Exposure B
buildings → " A
In landn → " C

(2.4.4) → BNBC

Wind Load Provisions of BNBC, 1993:

Sustained Wind Pressure at height, z (in metre)

$$q_z = C_c C_I C_z V_b^2 \quad \cdots (2.4.1)$$

V_b in km/hr → kph
q_z in kN/m²

Article 24.6.6 → BNBC

C_c → a conversion coefficient → since unit is changes.

C_z → from Table 6.2.10
V_b → Table 6.2.8 or fig 6.2.1.

7.03.15

③

Toufiq Sir

④ Wind load Provisions of Bangladesh (BNBC, 1993)

— Sustained Wind Pressure of height, z (in m)

$$q_z = C_c C_I C_z V_b^2 \quad \cdots (2.4.1)$$

[kN /m²]

C_c = Conversion Coefficient = 47.2×10^{-6}

C_I = Structure Importance Coefficient [Table 6.2.9, $\underbrace{0.8 \text{ to } 1.25}_{\text{varies within this range}}$]

high coefficient value for hazardous structure
→ so that design is done for higher load.

[page - 32, 33]

C_z = Combined height & Exposure coefficient [Tab 6.2.10]

V_b = basic wind speed [in km/h]
[Fig 6.2.1 & Tab 6.2.8]

⑤ Sustained wind pressure can be converted to Design Wind pressure.

Wind - Structural Interaction

Design Wind Pressure

$$P_z = C_G C_p q_z \quad \ldots\ 2.4.2 \ [kN/m^2]$$

C_G = Gust coefficient [sec. 2.4.6.6]

For non-standard building,

$C_G = G_h$ [Table 6.2.11]

(page-34)

Table 6.2.11

$G_h \rightarrow$ at height $h \rightarrow$ does not vary.

h = mean roof heigh

$h \rightarrow$ constant

$z \rightarrow$ variable

C_p = pressure coefficient [section 2.4.6.7]

For Rectangular buildings :

- With surface area method (Method 1) → fig 6.2.5 may be used.
- With Projected Area Method (Method 2) → Table 6.2.15 may be used.

we'll use this one more

page - 40, fig 6.2.5

page - 45, Table - 6.2.15

Article 2.4.6.4

Design Wind Loads for Buildings & Structures.

(a) Method 1 (Surface Area Method)

(i) For all framing systems;

$$F_1 = \sum p A_z \quad \cdots \quad (2.4.3)$$

(Must use for cable frame or single storey rigid frame).

F_1 = Wind force;

p = design wind pressure

$[\, p = p_z$ or $p = p_h \,]$

$p = p_z$ ↓ wind ward surface (Eq 2.4.2)

$p = p_h$ ↓ For non wind ward surface (Eq - 2.4.2)

A_z = (Cross-sectional) Area, m^2

(ii) For Gabled Frames and Single-Storey Rigid Frames; additionally

$$F_1 = \sum (p - p_i) A_z \quad \cdots \quad (2.4.4)$$

[p_i = internal pressure calculated using internal peak pressure coefficient]

$$p_i = C'_{pi}\, q_h \;;$$

C'_{pi} = Internal peak pressure coefficient (sec 2.4.6.7)

q_h = sustained wind pressure [Eq. 2.4.1)

Page - 39

Section 2.4.6.7

(b) Method 2 (Projected Area Method)

Applicable for all framing system, all buildings except for Gable frames & Single storey Rigid Frames.

(We'll use this method as it is easier to apply)

(It is very simple → total force is found)

Total Wind Force on Primary Framing System of a building or structure

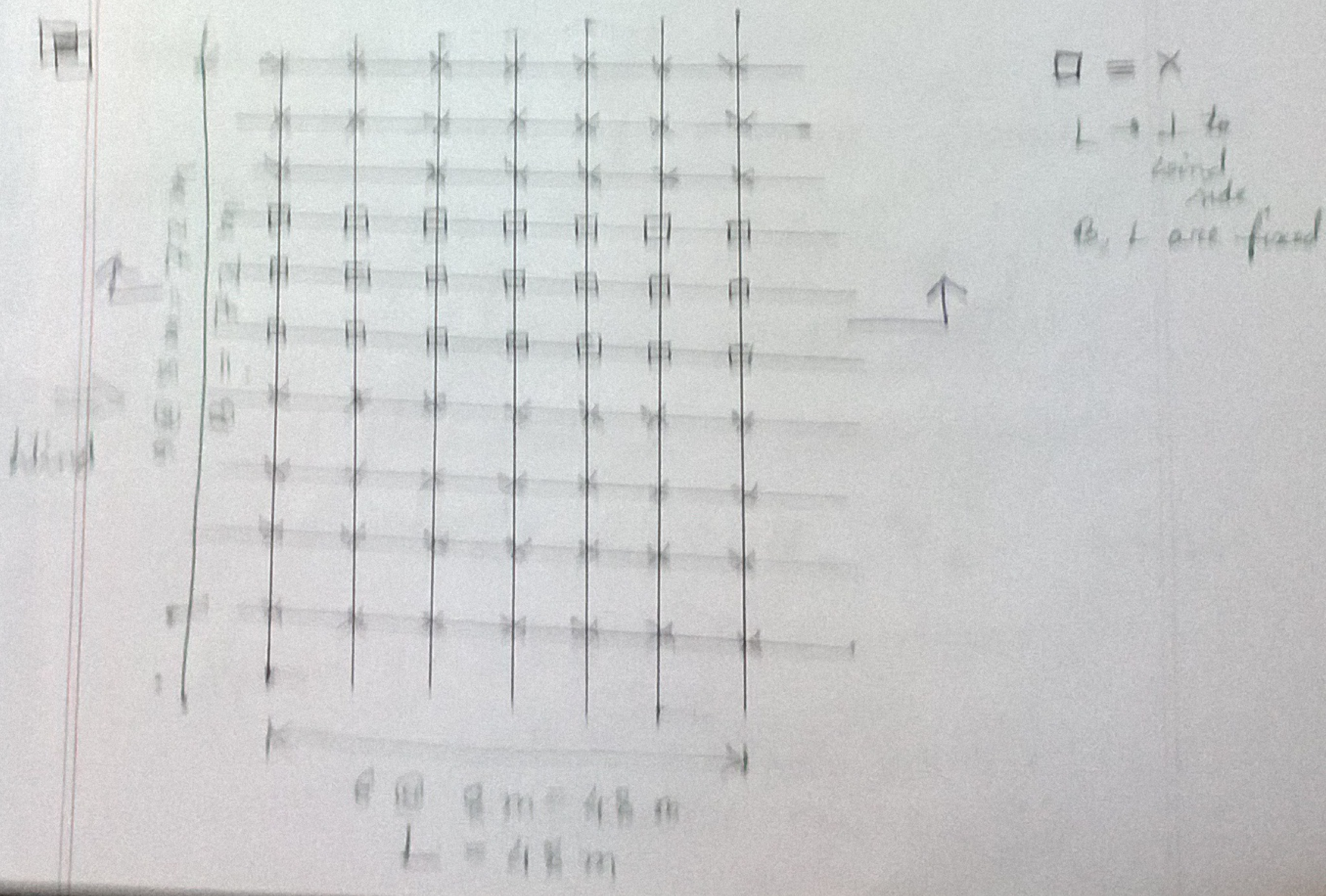
$$F_z = \sum p_z A_z \quad \ldots\ldots (2.4.5)$$

p_z = design wind pressure (in kN/m^2)

C_f = using overall pressure coefficient, C_p ~~for ... ...~~ (Tab 6.2.15 to Tab 6.2.21)

[For Rectangular bldg, use Tab 6.2.15]

A_z = projected frontal area



$$F_z = \sum p_z A_z \quad \cdots\cdots (2.4.5)$$

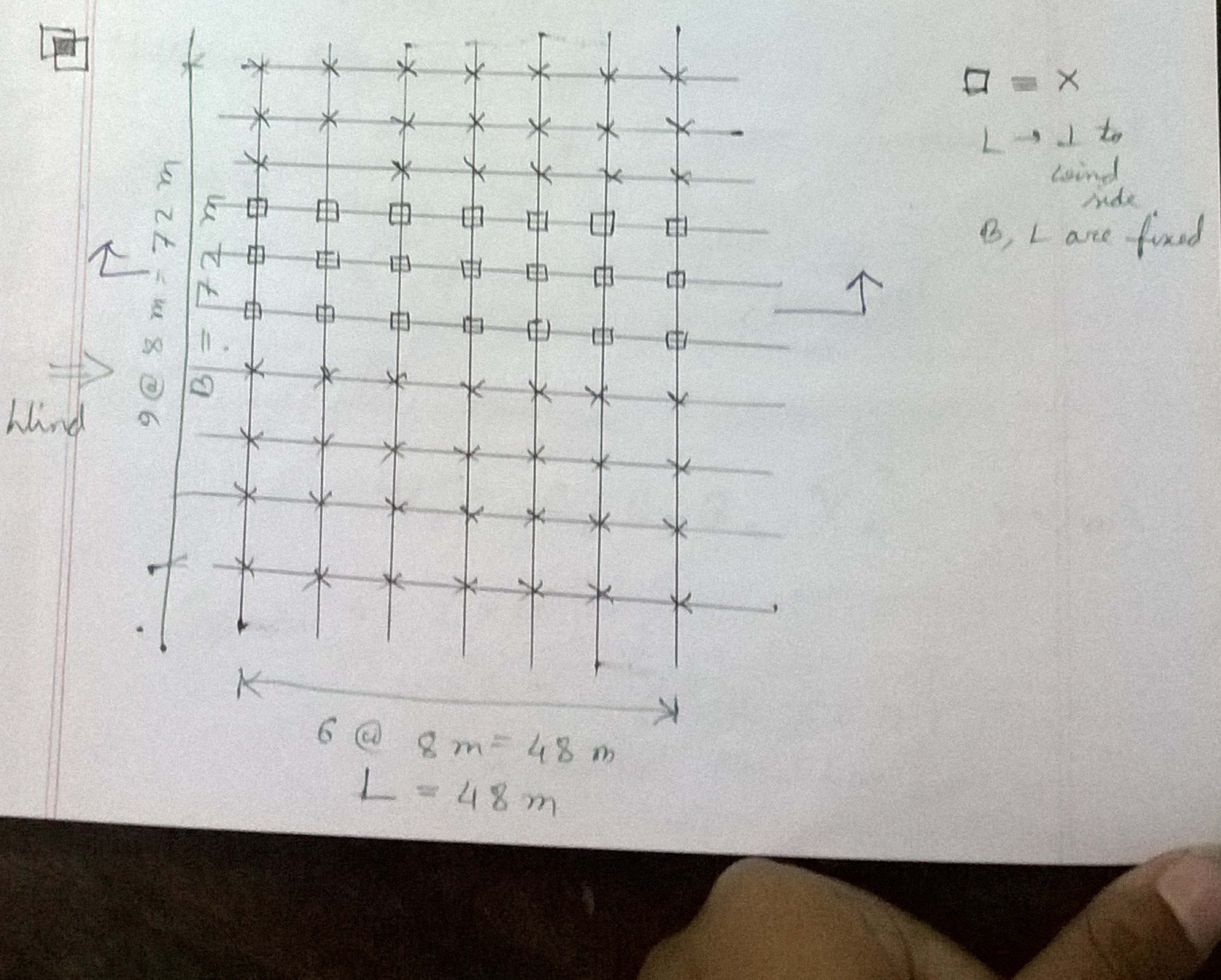
p_z = design wind pressure (in kN/m^2)

p_z = using overall pressure coefficient, C_p

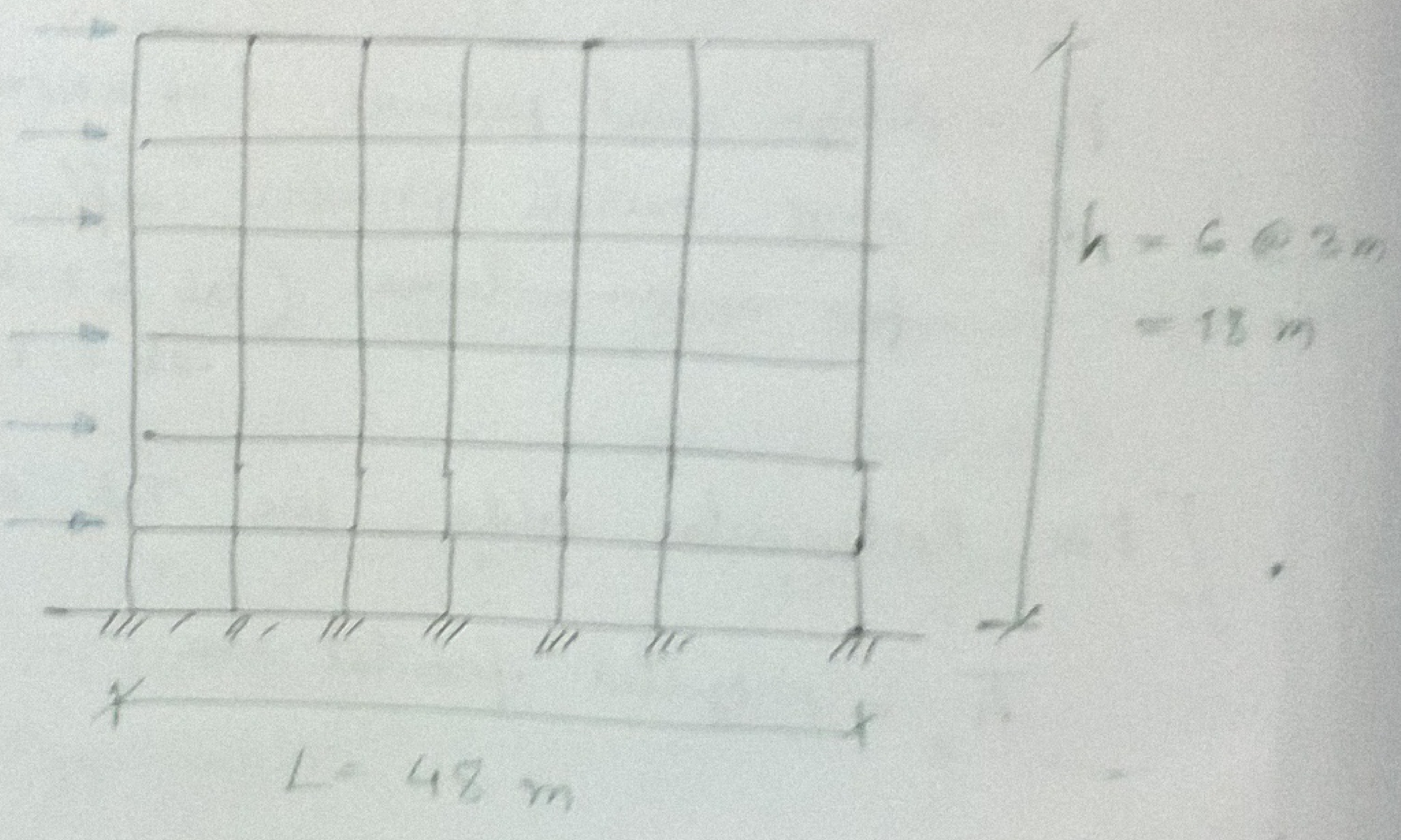
~~for cross sectional~~ (Tab 6.2.15 to Tab 6.2.21)

[For Rectangular bldg, use Tab 6.2.15]

$\bar{A}_z$ = projected frontal area.



Elevation.



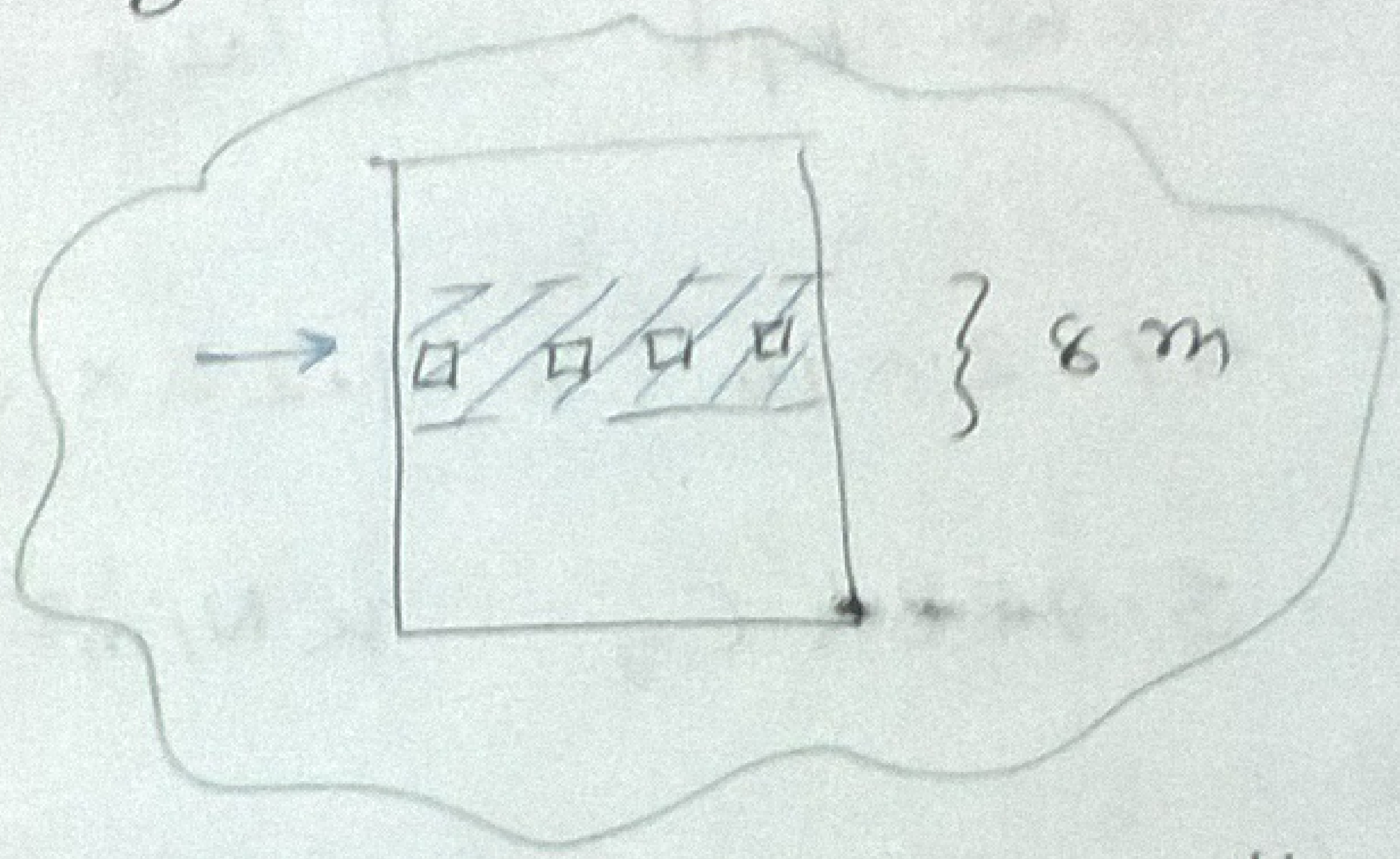
21.03.15

④

Taufiq Sir

Wind Forces on a Typical Intermediate Frame.

Given, Bldg 48 m × 78 m in plan.



6 storied @ 3 m Hospital Bldg at Chittagong city.

10 numbers of 6 span frames spaced @ 8 m along 72 m length.

Show forces at storey level.

Solⁿ:

Sustained Wind Pressure:

$$q_z = C_c C_I C_z V_b^2 \quad kN/m^2$$

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

$C_I = 1.25$ (Table 6.2.9)

[Essential Facilities]

Exposure Category :→ A (urban area)

$C_z = f(z, \text{Exp. } A)$

↳ kept general as different height for each floor.

$V_b = 260$ kph [Table 6.2.8]

$\therefore q_z = 47.2 \times 10^{-6} \times 1.25 \times C_z \times (260)^2$

$= 3.9884\, C_z$ kN/m²

Design Wind Pressure, p_z

$p_z = C_G C_p q_z$ kN/m²

$C_G = G_h$ (For non-slender Bldg.)

(we'll assume all as non-slender)

$= G_{h=18\,m}$ [Table 6.2.11 for exposure A]

$= 1.388$

$C_p = \bar{C}_p$ [Art. 2.4.6.7 d]

(method 2 is easier)

For Method 2: Projected Area Method.

(Our one is rectangular bldg.)

$= 1.543$ [Tab 6.2.15 for Rectangular Plan Bldg]

$h = 18$ m

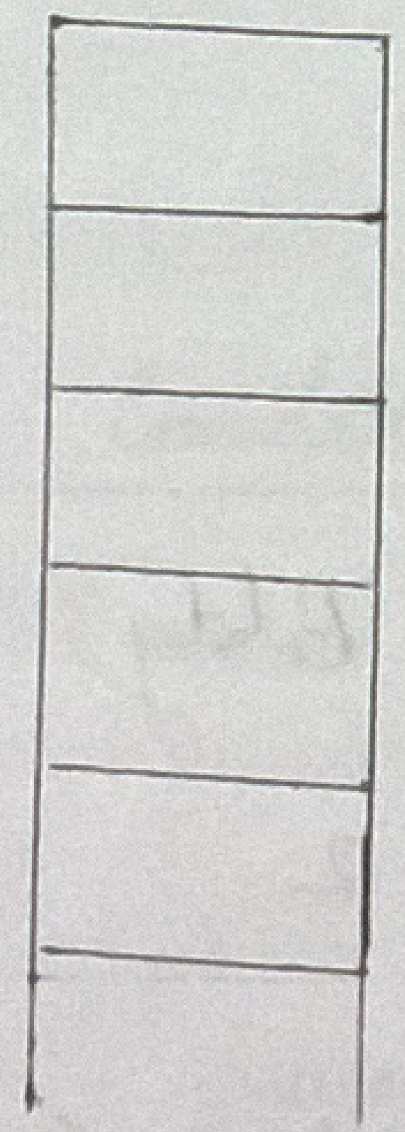
$$\frac{L}{B} = \frac{48}{72} = 0.667$$

$$\frac{h}{B} = \frac{18}{72} = 0.25$$

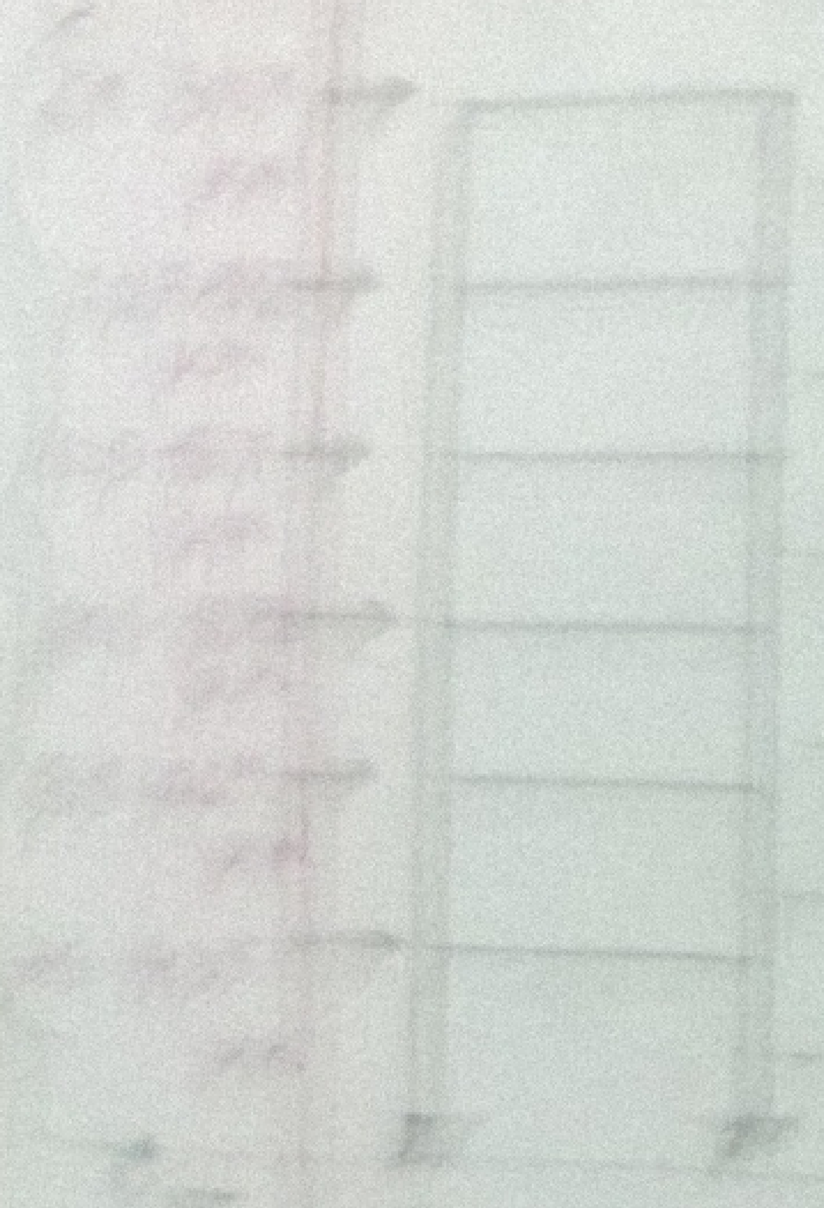
[Interpolated value bet^n 1.55 & 1.40]

$$P_z = 1.388 \times 1.543 \times 3.9884\, C_z$$

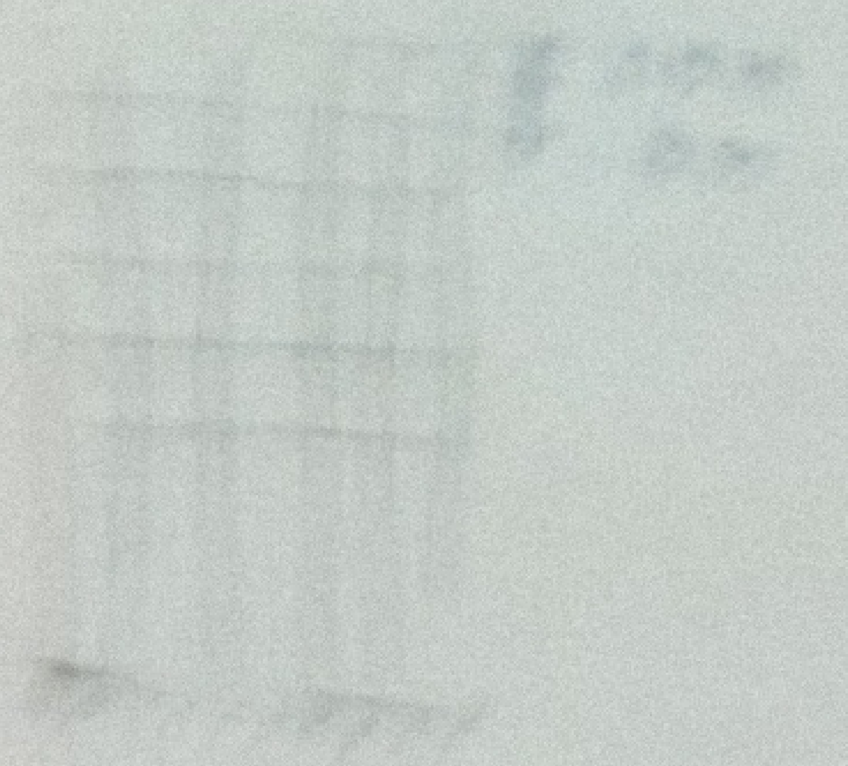
$$= 8.5411\, C_z \quad kN/m^2$$



[illegible] z (m)	k_2 [illegible]	[illegible]	[illegible]
18	6.677	5.792	[illegible]
15	6.424	5.333	[illegible]
12	6.565	4.826	[illegible]
9	6.297	4.265	[illegible]
6	6.415	3.543	[illegible]
3	6.968	3.143	[illegible]



Σ = 575.699 kN → Total wind force (for 1 frame)



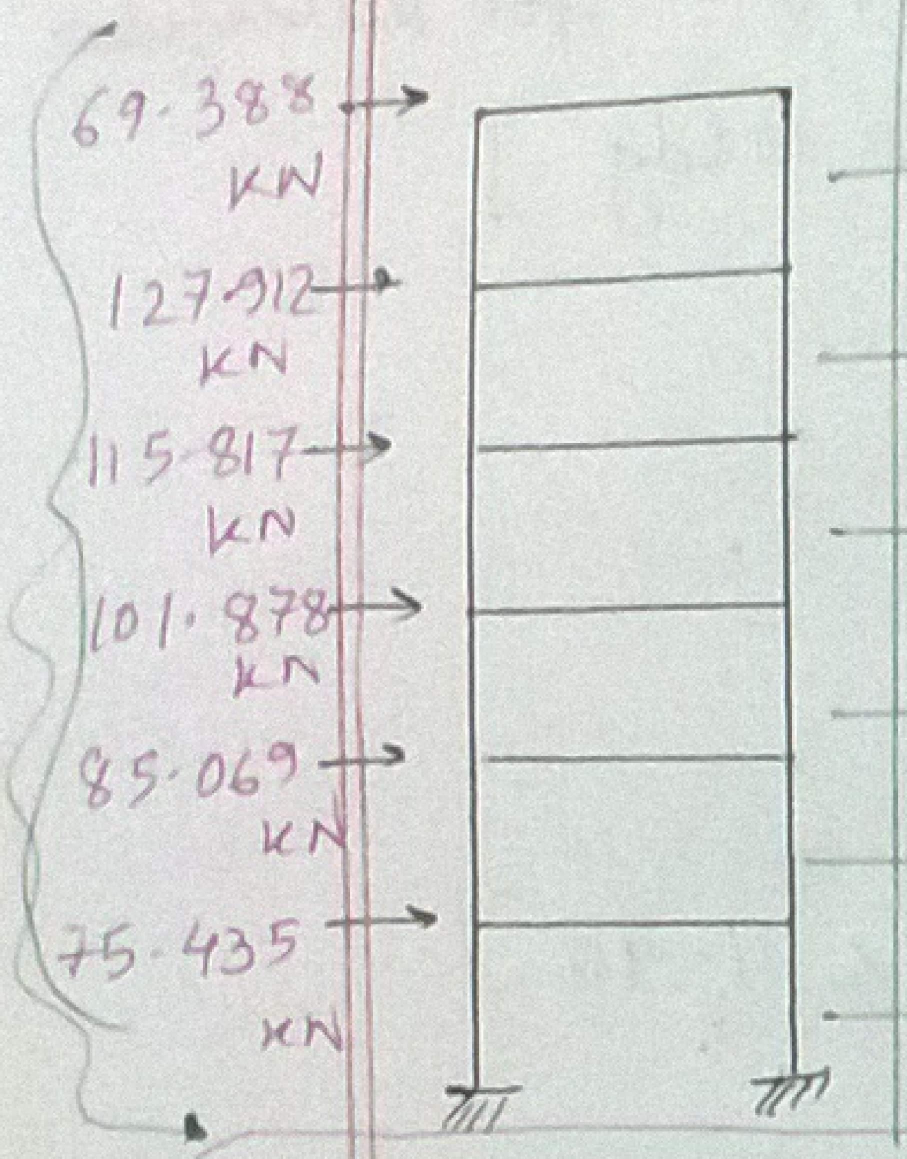
Total wind force on whole bldg

$$= \frac{575.699}{6} \times 72$$

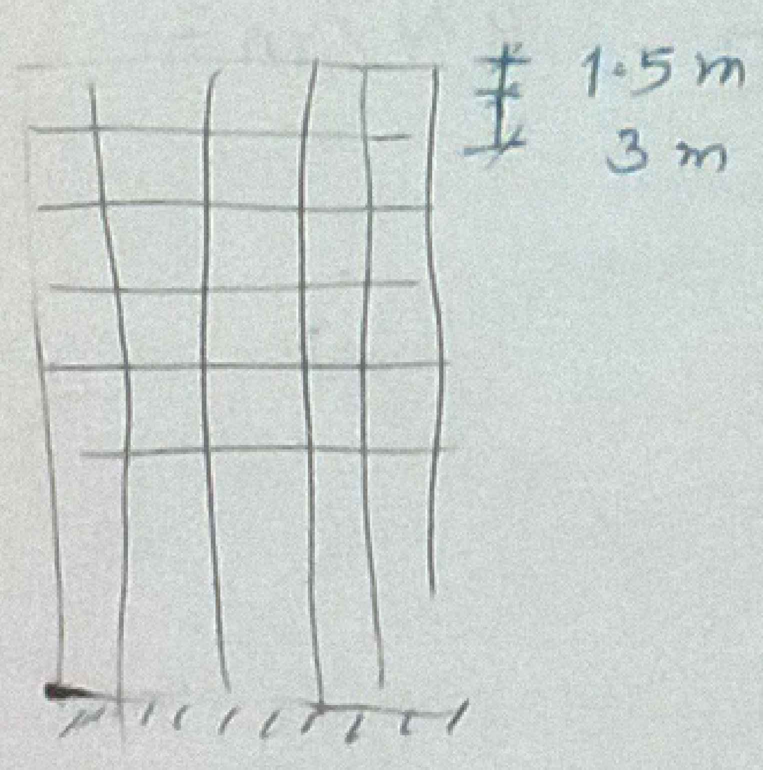
$$= 6909.691 \text{ kN}$$

(For a Typical Frame)

Ht. above Ground, z, m	C_z (Tab 6.2.10)	$P_z = 8.5411\,C_z$ kN/m²	A_z Area, m²
18	0.677	5.782	1.5×8 = 12
15	0.624	5.333	3×8 = 24
12	0.565	4.826	3×8 = 24
9	0.497	4.245	3×8 = 24
6	0.415	3.545	3×8 = 24
3	0.368	3.143	3×8 = 24



$\Sigma = 575.499$ kN → Total Wind force (for 1 frame)



∴ Total Wind Force on Whole Bldg

$$= \frac{575.499}{8} \times 72$$

$$= 5179.491 \text{ kN.}$$

$F_2 = \sum P_z A_z$ [Eqn 2.4.5], kN	
69.388	
127.912	
115.817	
101.878	
85.069	
75.435	

Earthquake Loading Provisions in BNBC

Main Considerations:
- Strength
- Economy
- Probability

EQ forces are selected, such that

- Bldg will not collapse in the event of rarely occuring major EQ, although it may be subjected to damage without loss of life or injury.

(We don't want loss of life but there can be repairable damage)

- Bldg should be able to respond without structural damage, to ~~shocks~~ shocks of moderate intensities (serviceability consideration).

Determination of Seismic Lateral Forces either

by → Equivalent Static Force Method (ESFM)
- For limited structures

or by B. Dynamic Response Method (DRM)
- Applied for all structures

ESFM may be used for:

—

—

—

—

We'll get these

Dynamic Response method shall be used :

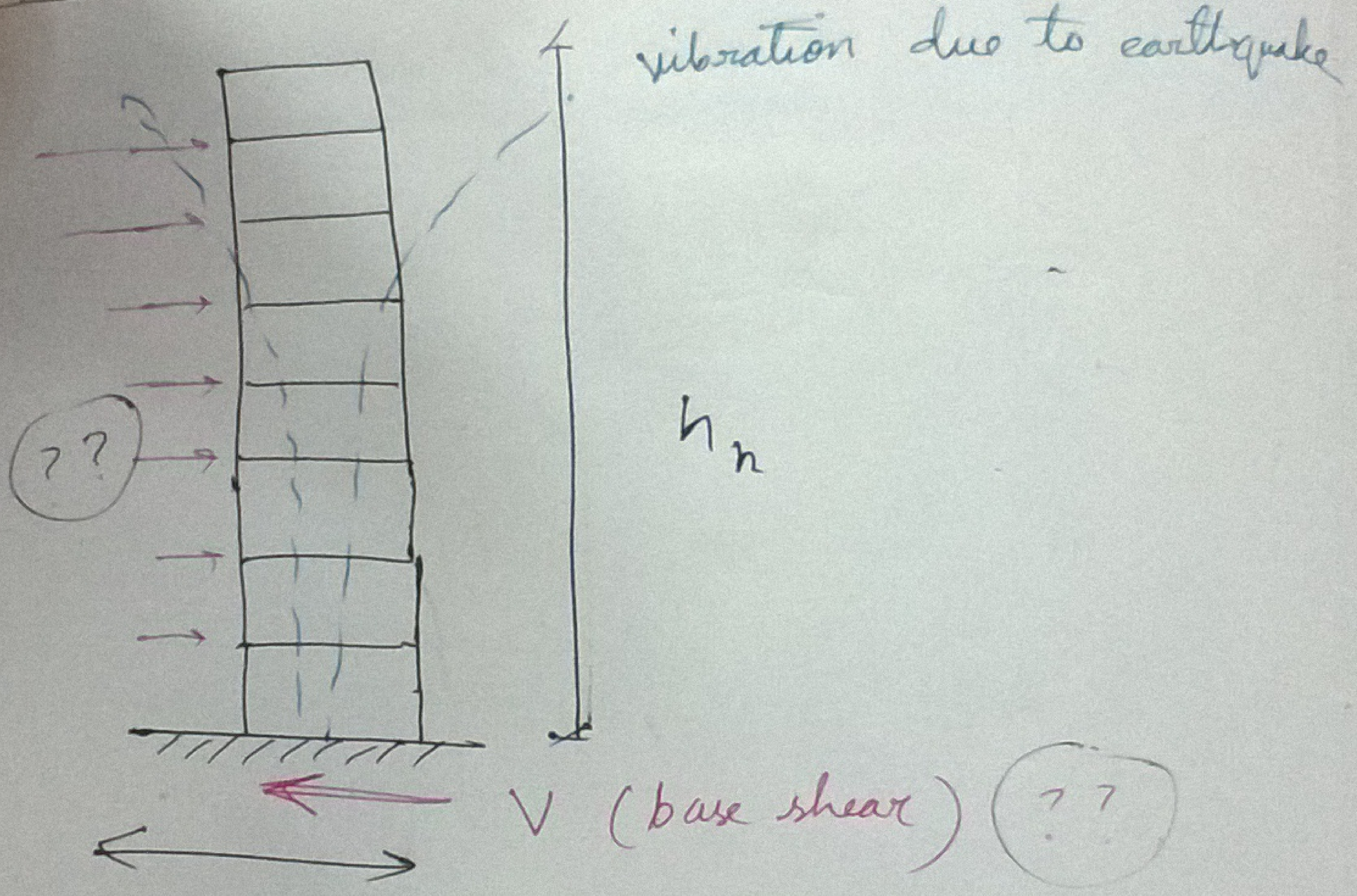
in Art. 2.5.5.1 (page 51)

Taufiq Sir

DRM needs computer assistance

We'll learn ESFM

Equivalent Static Force Method : (ESFM)



Base shear, $V = \frac{ZIC}{R} W$

Z → Seismic Zone Coefficient
(Table 6.2.22 or
fig 6.2.10 ;
Z = 0.075, 0.15 or 0.25)

I → Structure Importance Coefficient
(Tab 6.2.23 ; 1 or 1.25)

R → Response modification coefficient for structural system. (Table 6.2.24 ; 4 to 12)

(b, c → we'll need these more)

[In Q, only SMRF may be mentioned]
IMRF
OMRF

W → Total Seismic Dead Load defined in sec 2.5.5.2

- The total Seismic Dead Load, W is the total dead load of a structure, including permanent partitions & applicable portions of other loads as listed below:

— 25% of floor live load in storage or warehouse occupancies

— all loads of partitions, but not less than 0.6 KN/m²

— Total wt of permanent equipment.

(*) C is a derived value.

$$C = \frac{1.25 S}{T^{2/3}}$$

S = site coefficent for soil characteristics.

(Table 16-J ; 1, 1.2, 1.5 or 2.0)

If geotechnical info is not available, consider $S_3 = 1.5$

T → Fundamental period of vibration in seconds of a structure

$C \ngtr 2.75$; $\frac{C}{R} \nless 0.075$

Determination of T :

Structure Period, T :

Method A

Method B

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

h_n → height in meters above base to level n

C_t → 0.085 for steel moment resisting frames

→ 0.073 for RC ... & eccentric braced steel frames

→ 0.049 for all other structural systems

alternatively for buildings with concrete or masonry shear walls, $C_t = 0.0731/\sqrt{A_c}$

(Tab 6.2.2.5 ; 1, 1.2, $\frac{1.5}{S_3}$ or 2.0)

If geotechnical info is not available, consider $S_3 = 1.5$

T ⇒ Fundamental period of vibration in seconds of a structure.

$C \not> 2.75$; $\frac{C}{R} \not< 0.075$

Determination of T :

Structure Period, T :

Method A

$T = C_t \, h_n^{3/4}$;

h_n → height in meters above base to level, n

C_t → 0.083 for steel moment resisting frames

→ 0.073 for RC " " "
& eccentric ~~br~~ braced steel frames

→ 0.049 for all other structural systems

alternatively for buildings with Concrete or Masonry Shear Walls, $C_t = 0.031/\sqrt{A_c}$

Method B

(Tab 6.2.25 ; 1, 1.2, $\underline{\underline{1.5}}$ or 2.0)
S_3

If geotechnical info is not available, consider $S_3 = 1.5$

$T \Rightarrow$ Fundamental period of vibration in seconds of a structure.

$C \not> 2.75$; $\frac{C}{R} \not< 0.075$

Determination of T:

Structure Period, T:

Method A ~~Method B~~

$T = C_t \, h_n^{3/4}$;

$h_n \rightarrow$ height in meters above base to level, n

$C_t \rightarrow$ 0.083 for steel moment resisting frames
$\rightarrow$ 0.073 for RC ....
& eccentric braced steel frames
$\rightarrow$ 0.049 for all other structural systems

alternatively for buildings with Concrete or Masonry Shear Walls, $C_t = 0.031/\sqrt{A_c}$

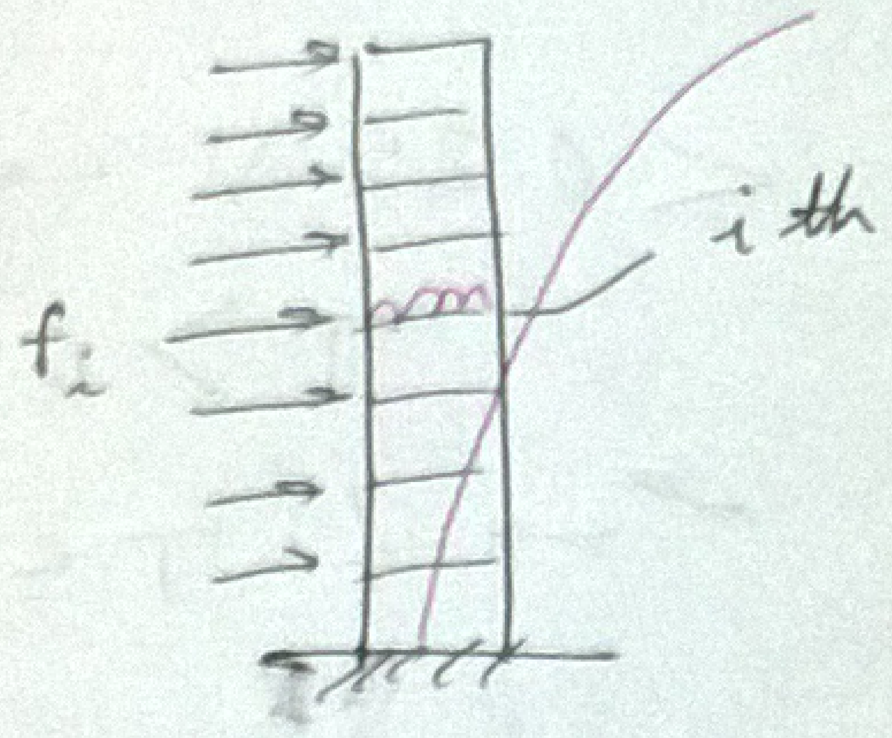
$A_c = \sum A_e$

[A_e = horizontal cross-sectional area].

$D_e / h_n \leq 0.9$

Method B

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



f_i = any ~~distributed~~ static lateral force distributed approximately as earthquake force.

δ_i = Deflection at ith level due to f_i

W_i = seismic dead load at ith level

g = acceleration due to gravity.

$T_{\text{method B}} \leq 1.4\, T_{\text{method A}}$

Now, $V \rightarrow$ known
have to distribute in various floors

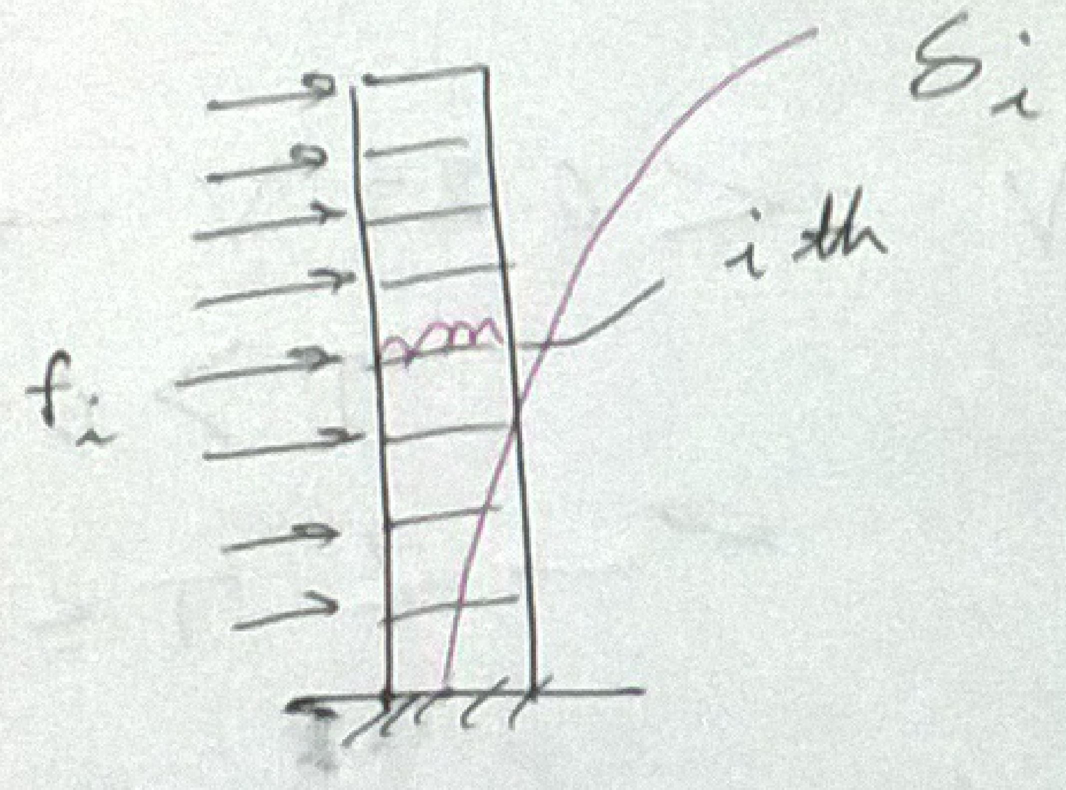
$$A_c = \sum A_e \left[0.2 + (D_e/h_n)^2\right]$$

[A_e = horizontal cross-sectional area].

$$D_e/h_n \le 0.9$$

Method B

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



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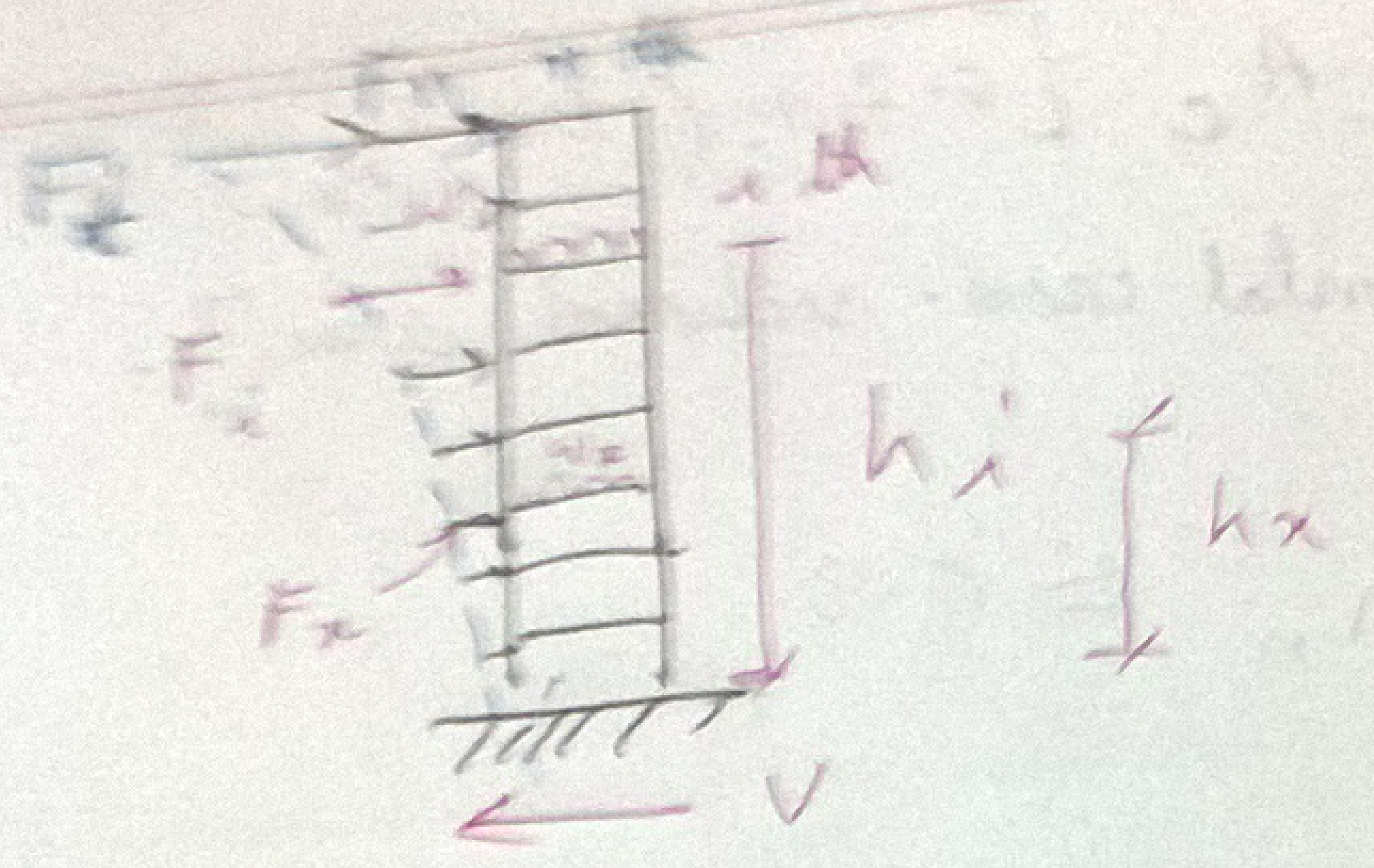
$$T_{\text{method B}} \le 1.4\, T_{\text{method A}}$$

Now, $V \rightarrow$ known

have to distribute in various floors



Vertical Distribution of Lateral Force:

$$V = F_t + \sum_{i=1}^{n} F_i$$

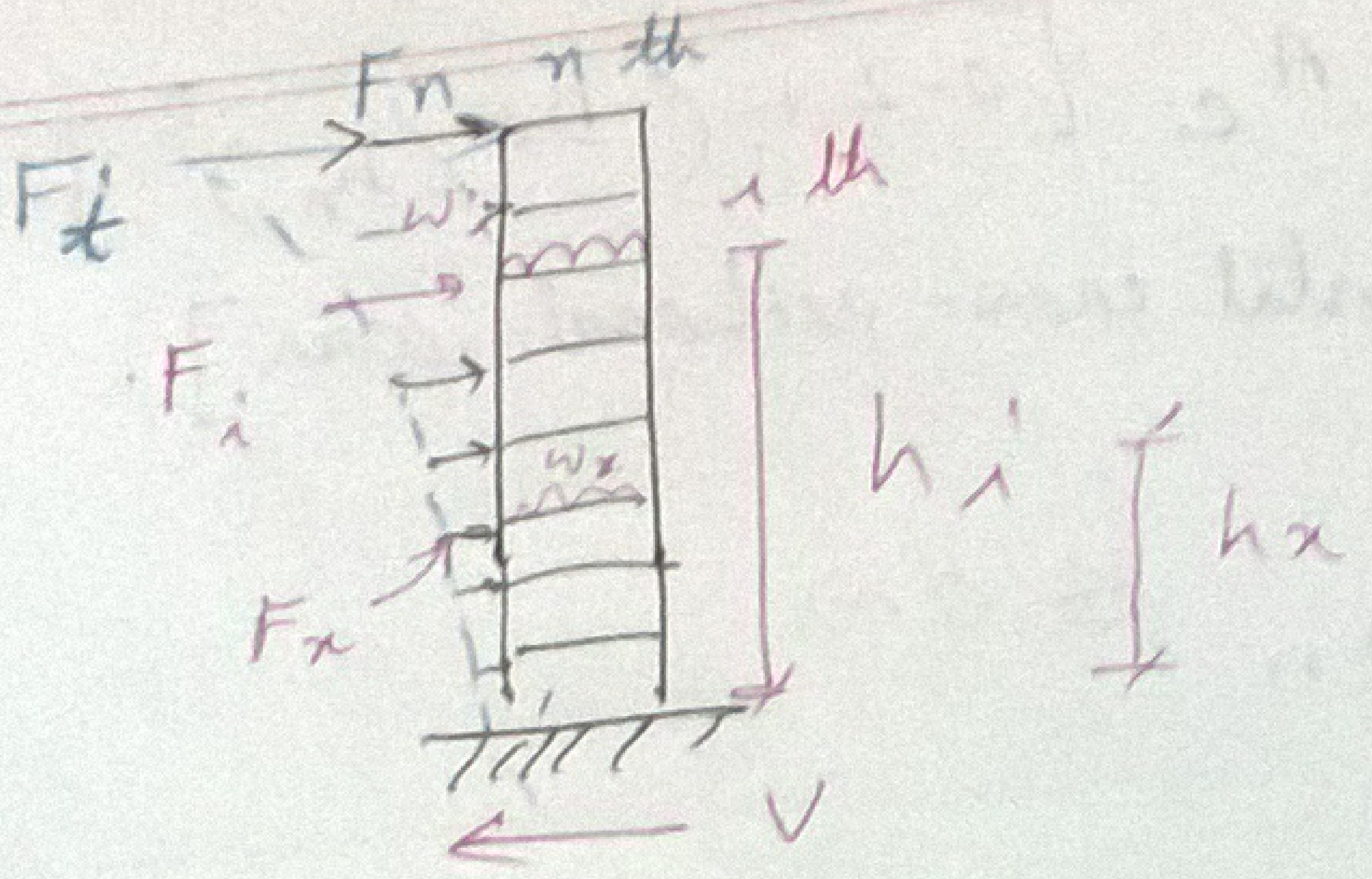
$$F_t = 0.07\,TV \quad < 0.25\,V \rightarrow \text{if } T > 0.7 \text{ sec}$$

$$= 0 \quad \rightarrow \text{if } T \leq 0.7 \text{ sec}$$

The remaining base shear, $(V - F_t)$ is distributed over height as,

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

- CT → after 1 week → on earthquake reinforce.



Vertical Distribution of Lateral Force:

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The remaining base shear, $(V - F_t)$ is distributed over height as,

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

- CT → after 1 week → on earthquake reinforce.

25.04.15

Taufiq Sir

$\boxed{\text{Example}}$: Calculate EQ. Forces?

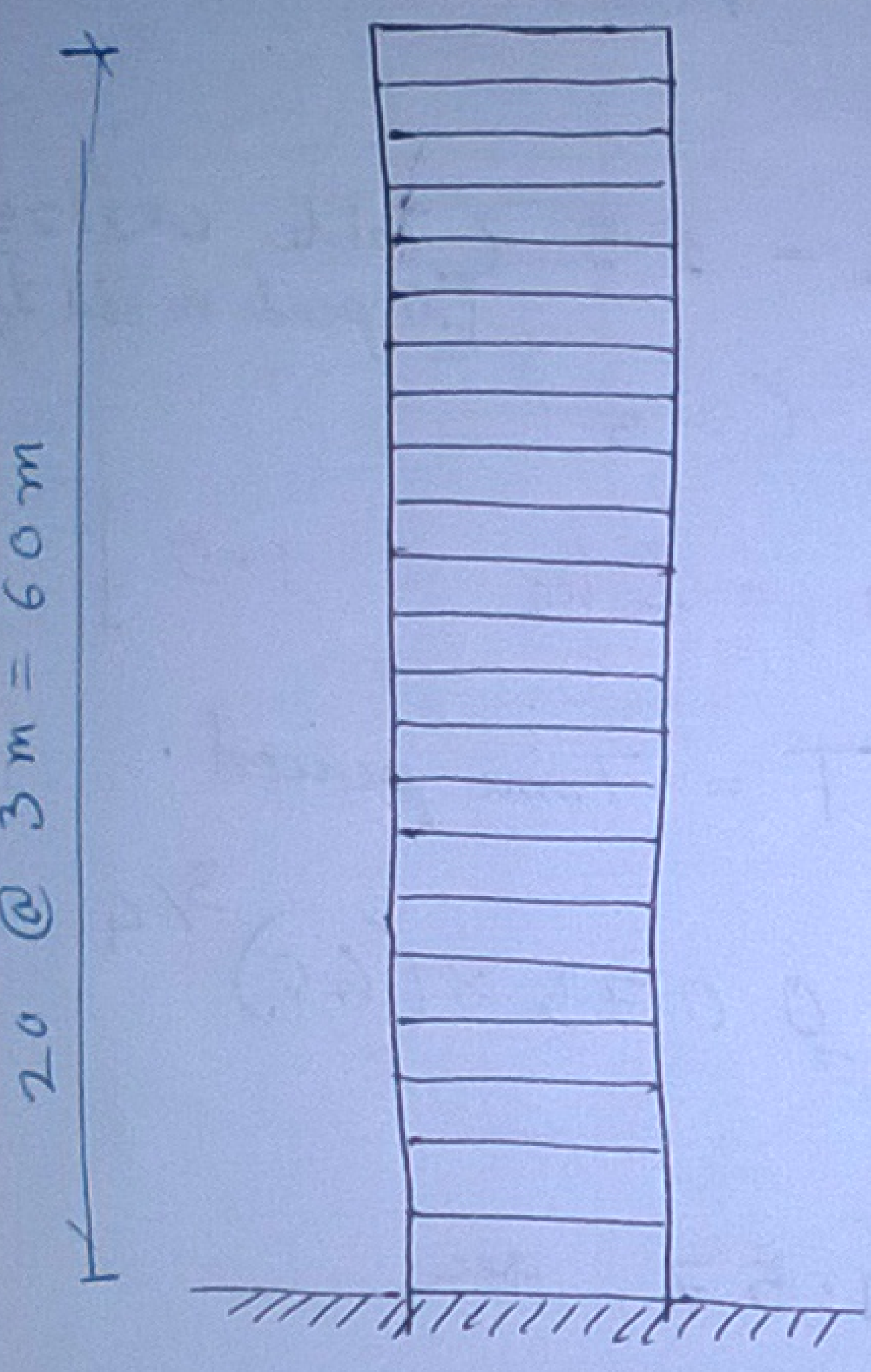
Given,

20 storied office bldg of Dhaka.

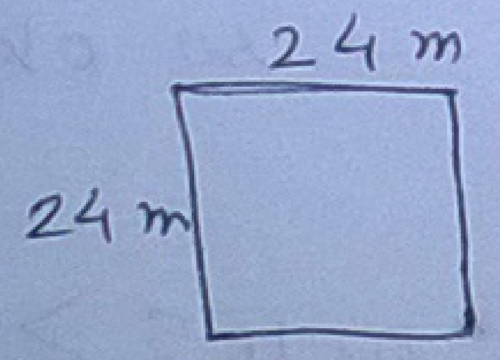
Soil Type: Soft to medium ~~soft~~ Stiff Clay

DL including Partitions $= 12\ kN/m^2$

Structure type : SMRF in Concrete



Elev.



Plan

Sol^n.

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

Z = (Fig. 6.2.10 & Table 6.2.2.2

(Zone 2, coeff – 0.15)

$Z = 0.15$

$I = 1.0$ (Table 6.2.23)

(Standard Occupancy Structures)

$R = 12$ (Table 6.2.24)

(Moment resisting Frame system → SMRF → Concrete → 12)

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Recruitment Test, March 2015
Date : 20 March, 2015
Name of Post:
Assistant Engineer (ME)

ROLL NO.	20199

umpa

$W = $ (seismic dead load)

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

12: load with partition; 20: Floors

$C = \dfrac{1.25S}{T^{2/3}}$; $S = 1.5$ (Table 6.2.25) [Depends on soil type] (S_3)

[If soil type not known, take $S = 1.5$]

T = Time period.

$T = C_t (h_n)^{3/4} = 0.073 \times (60)^{3/4}$

53 page

$= 1.57$ sec

$C = \dfrac{1.25 \times 1.5}{1.57^{2/3}} = 1.386 < 2.75$

→ so ok.

Another Criteria.

$\dfrac{C}{R} = \dfrac{1.386}{12} = 0.115 > 0.075$

→ so ok

$\therefore C = 1.386$ (accepted)

$V = \dfrac{0.15 \times 1 \times 1.386}{12} \times 138240$

$= 392$ kN

POWER DEVELOPMENT BOARD
Recruitment Test, March 2015
Date : 20 March, 2015
Name of Post:
Assistant Engineer (ME)
ROLL NO. 20199

W = ~~12 kN/m²~~ (Seismic dead load)

$W = 12 \times 24 \times 24 \times 20 = 138240\ kN$

12 ↑ load with partition

20 ↑ Floors

$C = \dfrac{1.25S}{T^{2/3}}$; $S = 1.5$ (Table 6.2.25) [Depends on soil type]

(S_3)

[If soil type not known, take S = 1.5]

T = Time period.

$T = C_t (h_n)^{3/4} = 0.073 \times (60)^{3/4}$

0.073 ↗ 53 page

$= 1.57$ sec

$C = \dfrac{1.25 \times 1.5}{1.57^{2/3}} = 1.386 < 2.75$

→ so ok.

Another Criteria,

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→ so ok

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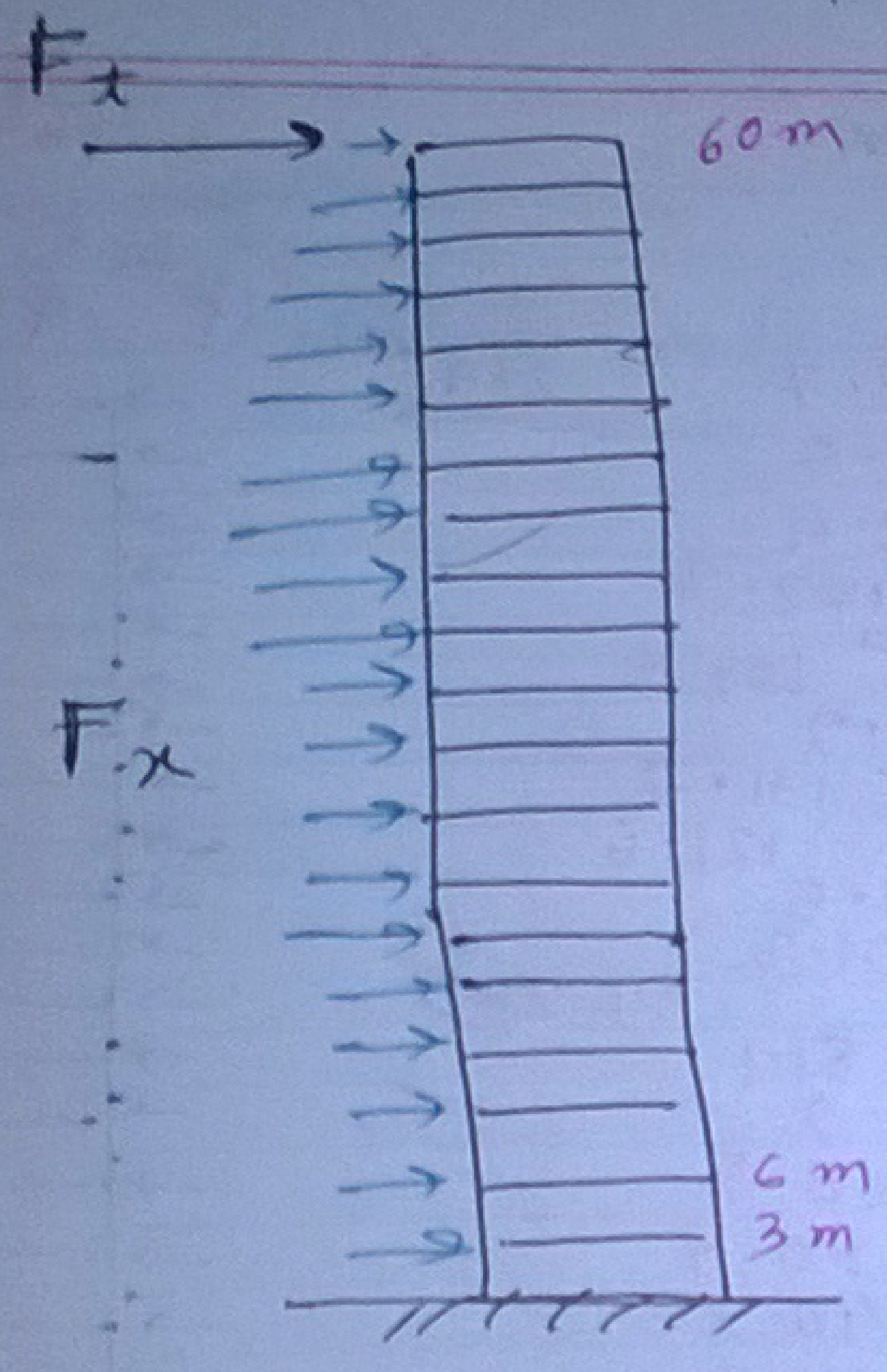
$= 392\ kN$

POWER DEVELOPMENT BOARD
Recruitment Test, March 2015
Date : 20 March, 2015
Name of Post:
Assistant Engineer (ME)

ROLL NO.	20199



$F_t = 0.07\, T\, V$

If $T > 0.7$ sec.

$1.57 > 0.7$ sec.

So,

$F_t = 0.07\, T\, V$

$= 0.07 \times 1.57 \times 2392$

$= 264$ kN $< 0.25V$

$= 0.25 \times 2392$

(ok)

$$F_x = \frac{(V - F_t) \times w_x h_x}{\sum_{i=1}^{n} w_i h_i} = \frac{(2392 - 264)\cancel{w_x} h_x}{\cancel{w_i} \sum_{i=1}^{n} h_i}$$

$w_x = w_i = 120 \times 24 \times 24$

$$= \frac{2128\, h_x}{\sum_{i=1} h_i}$$

$$= \frac{2128\, h_x}{(3+6+9+\cdots+57+60)}$$

$$= \frac{2128\, h_x}{3(1+2+3+\cdots+19+20)}$$

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

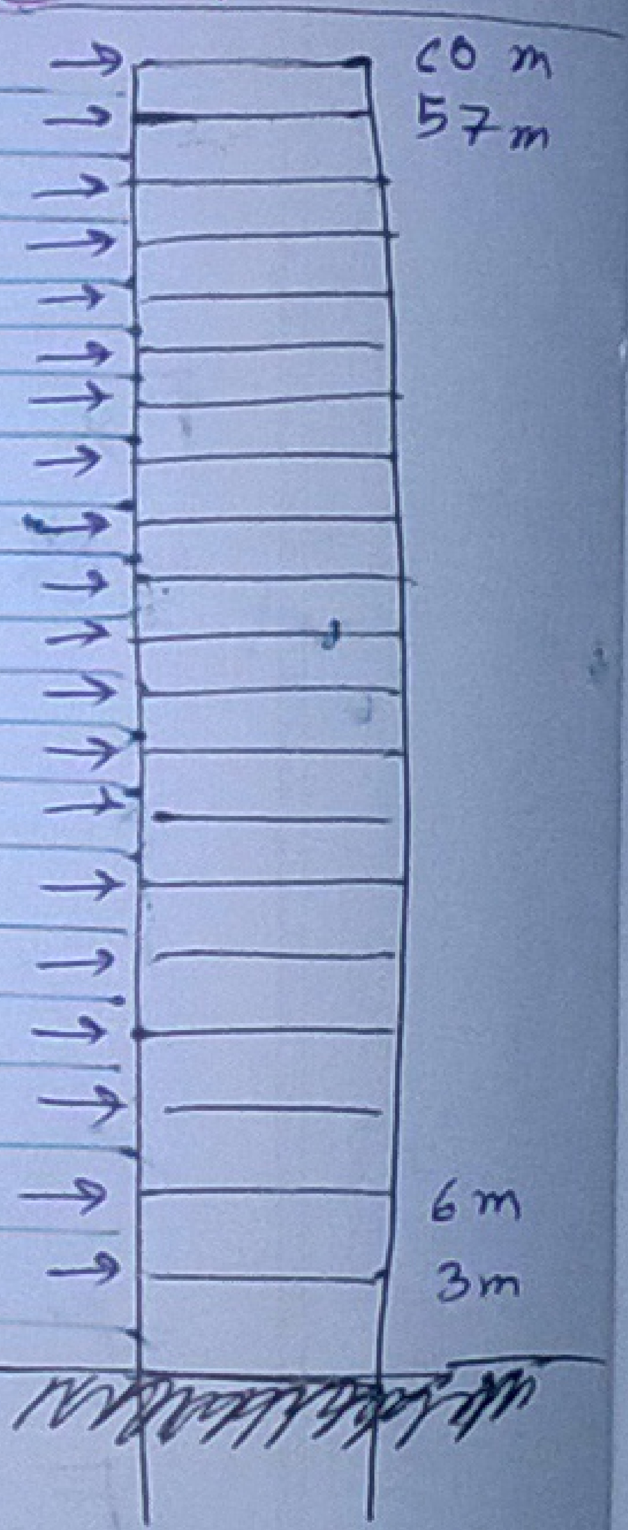
$$= 3.38\, h_x$$

$F_x = 3.38\, h_x$

466.7 kN (force over the height of the building)

Floor level	h_x, m	F_x, kN
20	60	202.7
19	57	192.5
18	54	182.4
17	51	172.3
16	48	162.1
15	45	152.0
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	70.9
6	18	60.8
5	15	50.7
4	12	40.5
3	9 m	30.4
2	6 m	20.3
1	3 m	10.1

264 kN →



Statically Indeterminate Structures : Approximate Analysis

Analysis
- Exact (in Relative Form) — based on usual assumptions
- Approximate — based on additional assumptions ~~used~~ over usual

POWER DEVELOPMENT BOARD
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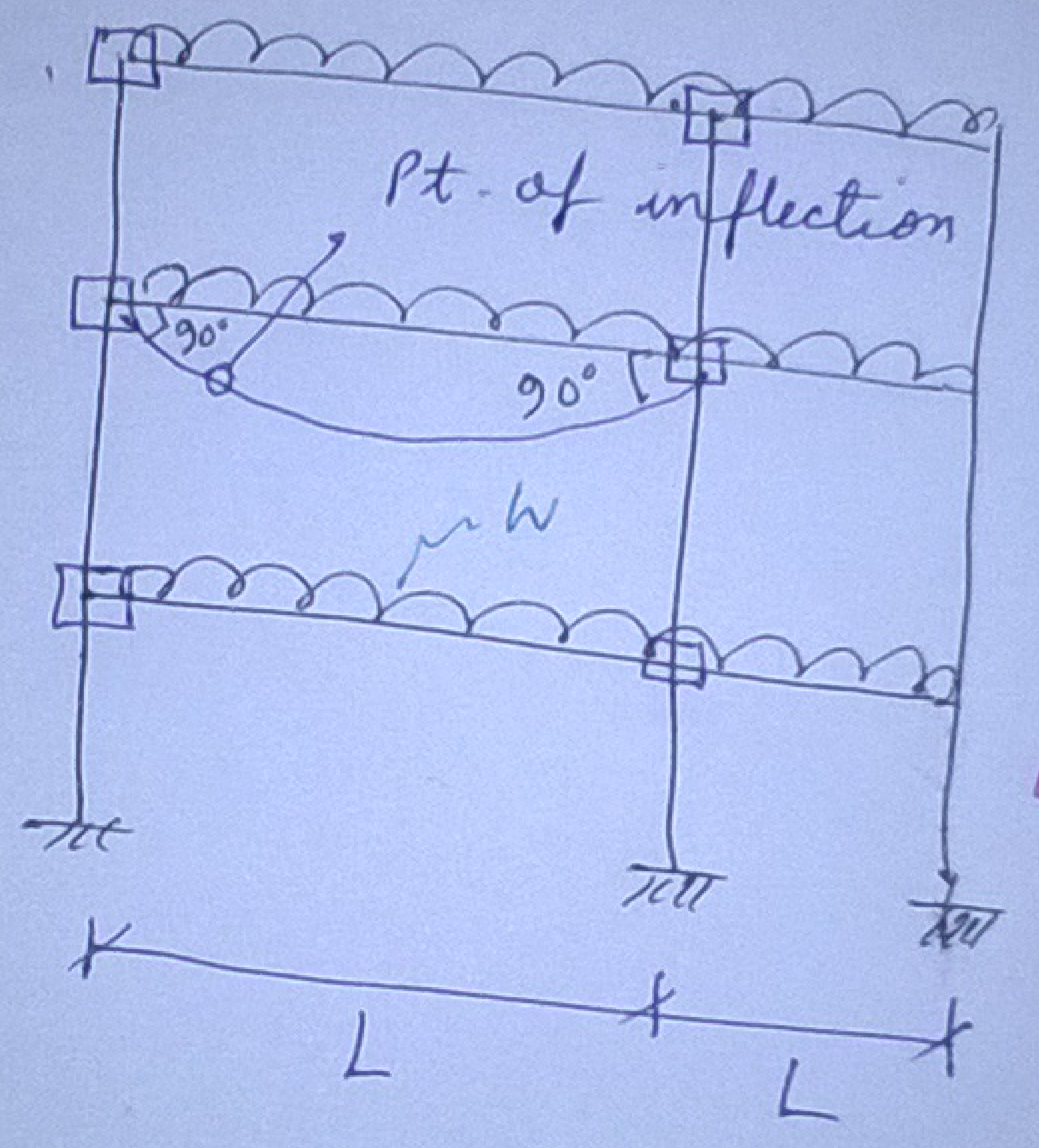
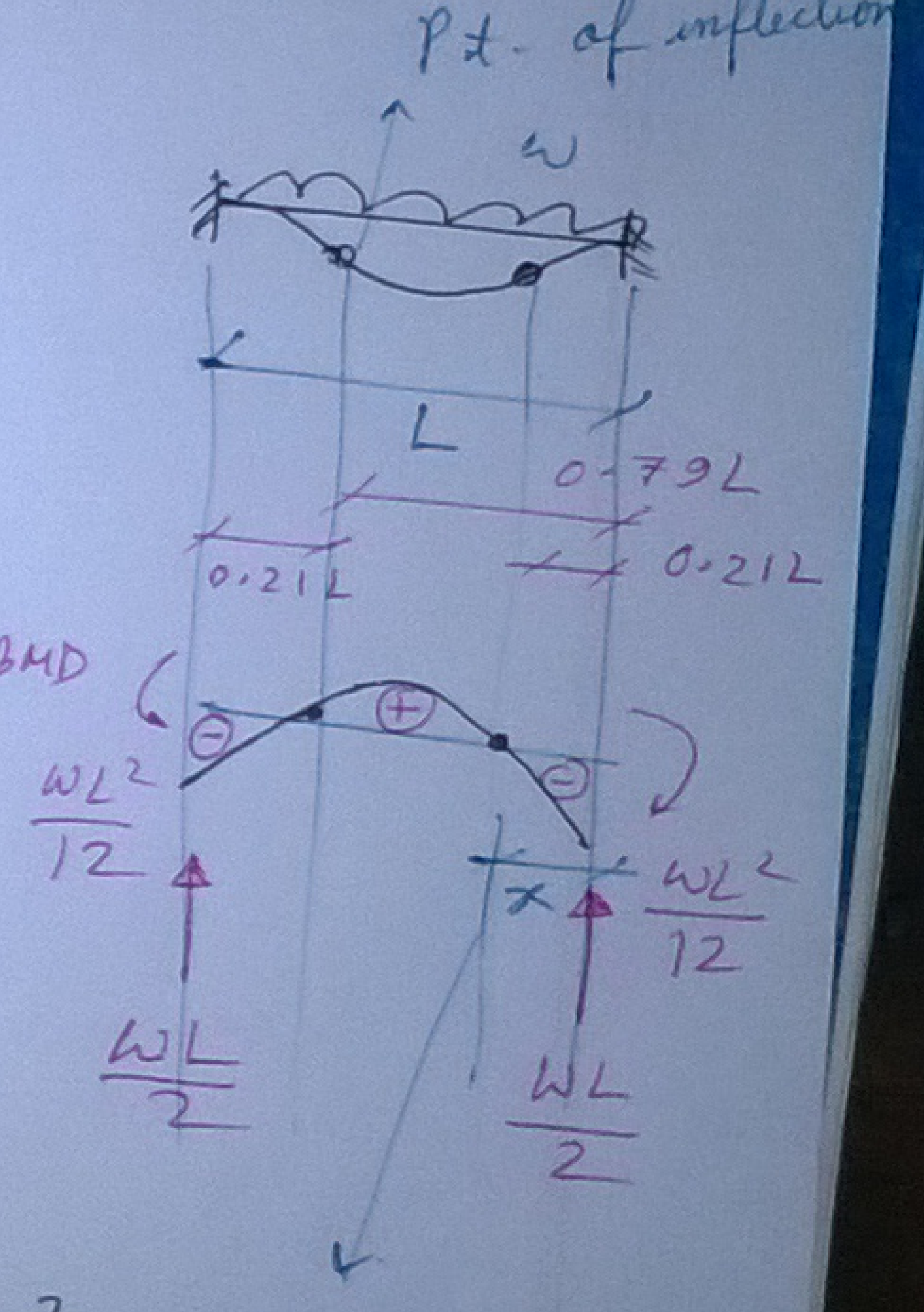
Assistant Engineer (ME)
ROLL NO. 20199

CT → Wind & Eq. → Sat → Must bring Handouts.

Approximate Analysis of Building Frames for Vertical Loads:

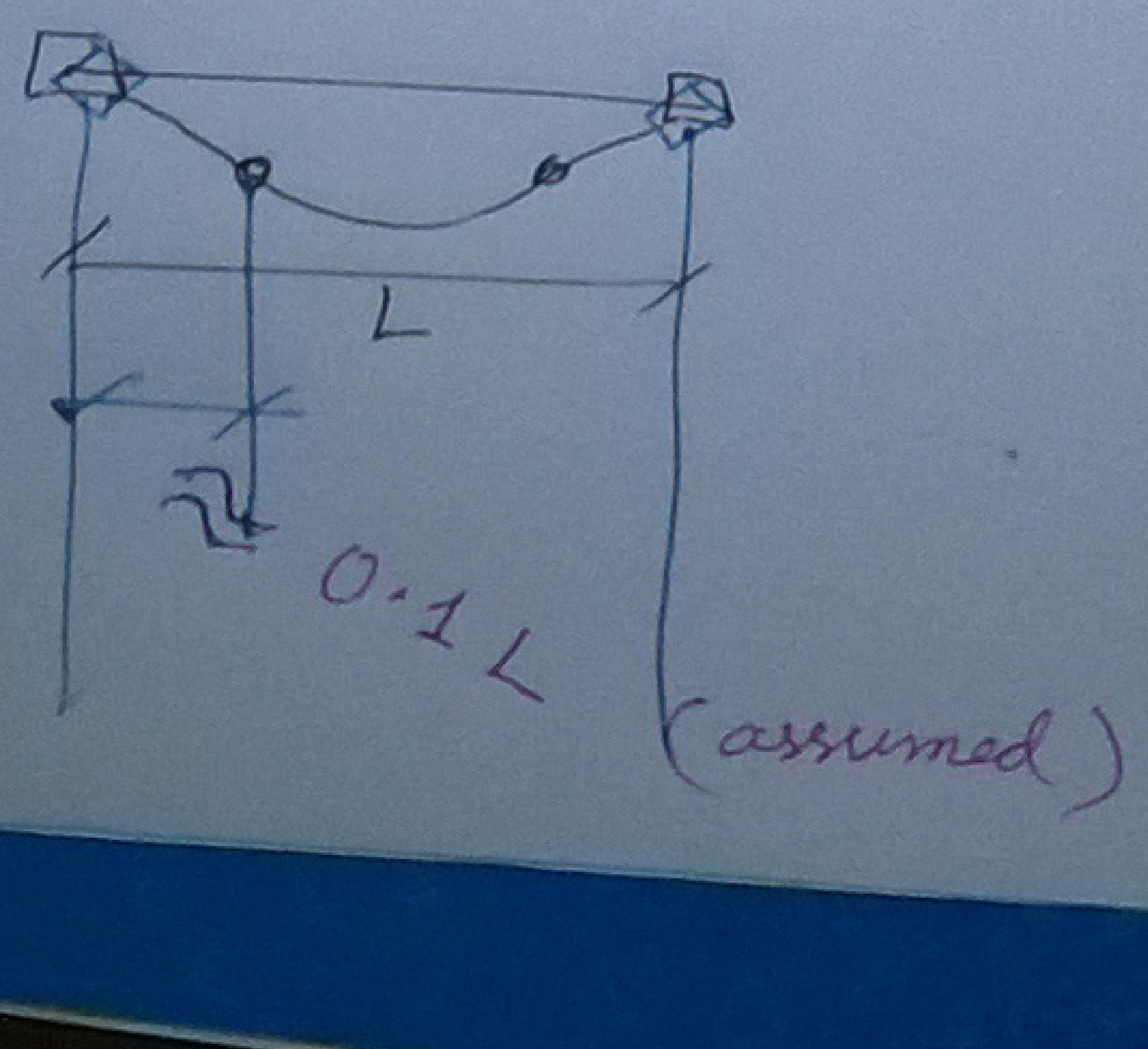


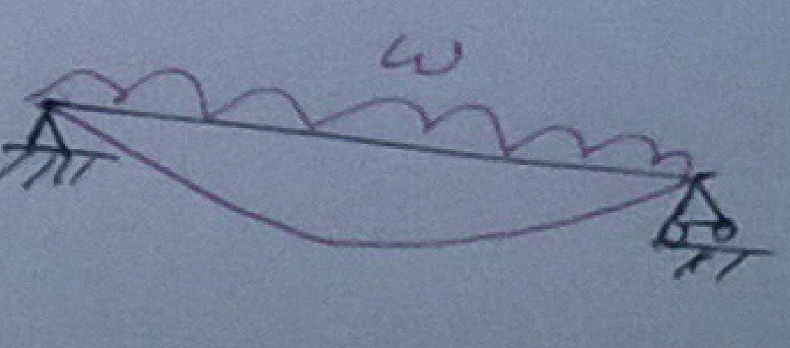


$$-\frac{wL^2}{12} + \frac{wL}{2} \times x - \frac{w x \times x}{2} = 0$$

$$x_1 = 0.21L \text{ or } 0.79L$$

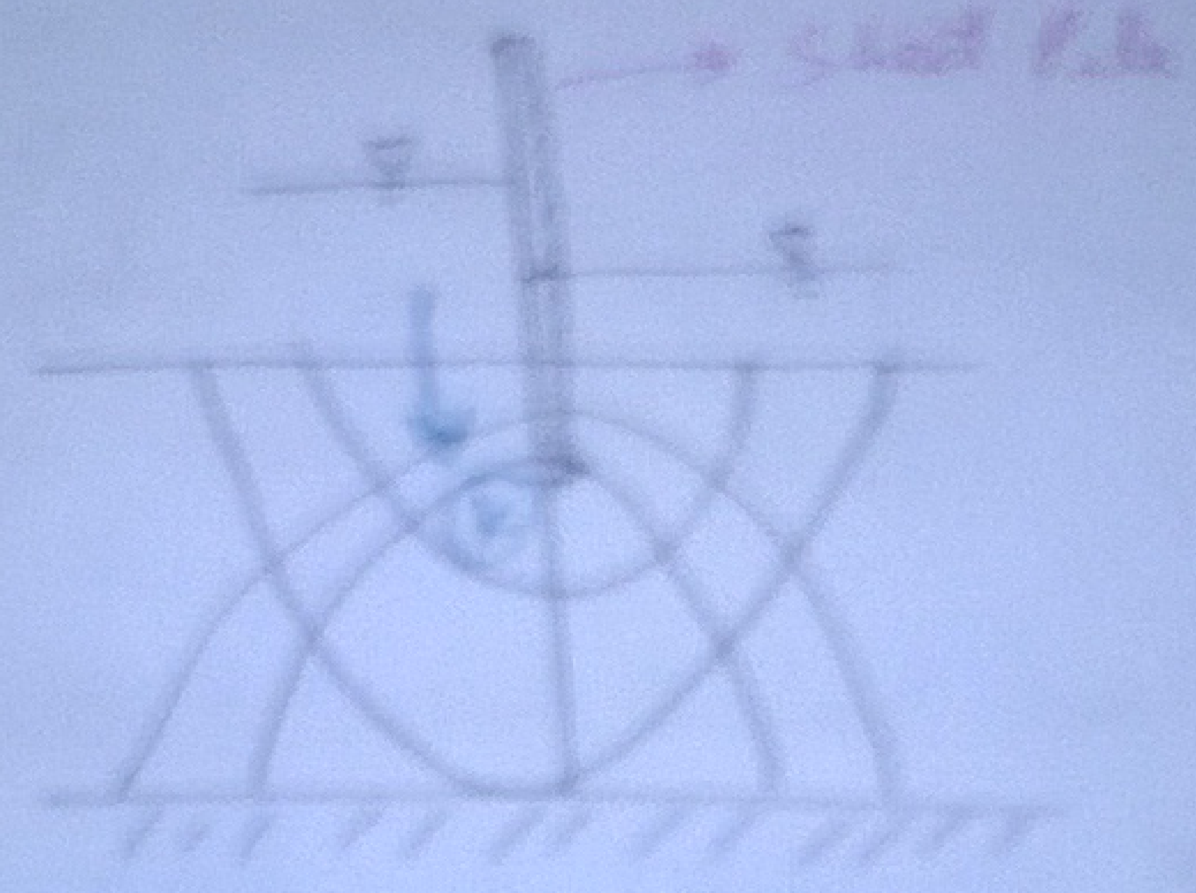
Rotates



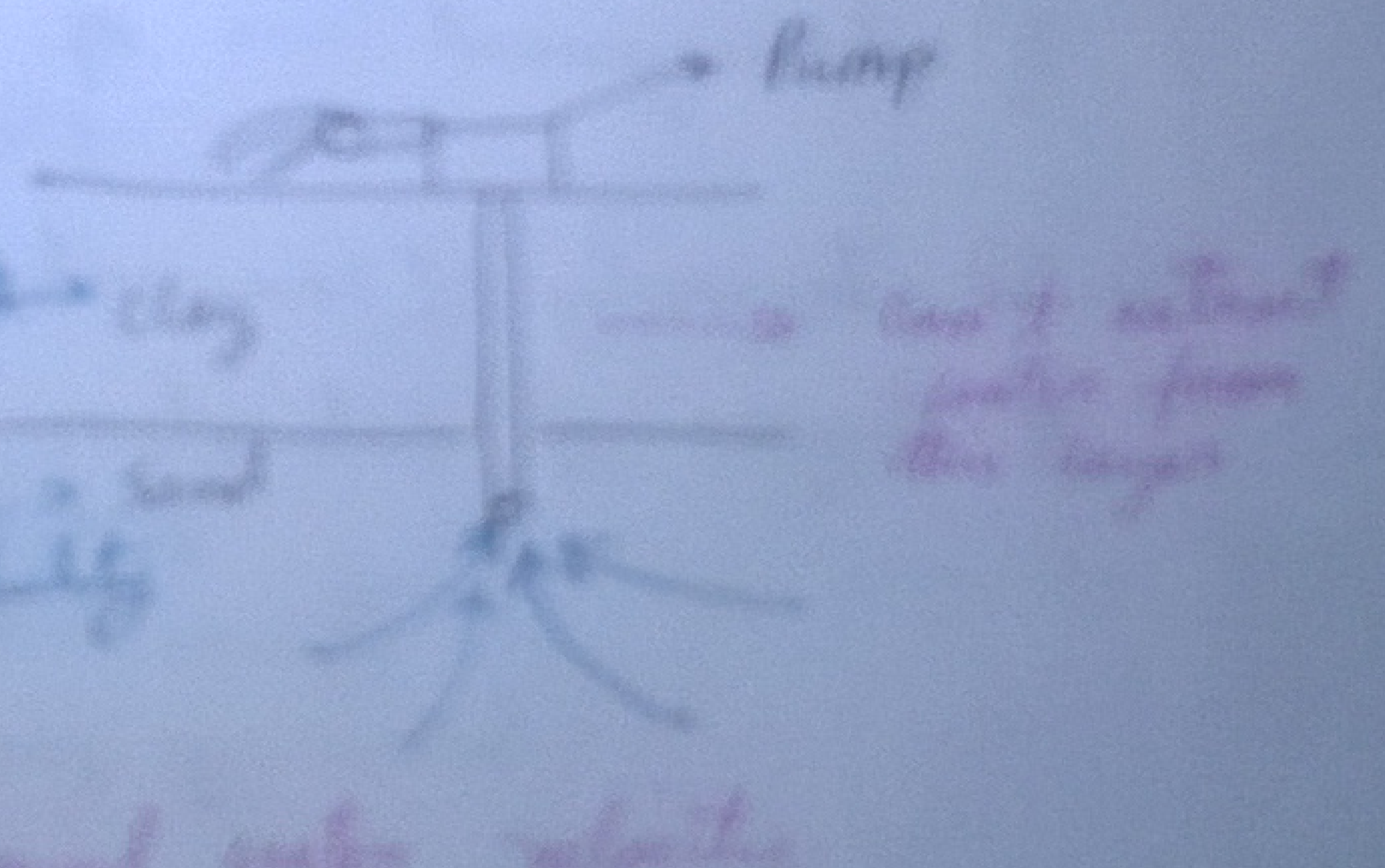


Permeability Test

Def^n of Permeability, Unit, K



flow through soil depends on K



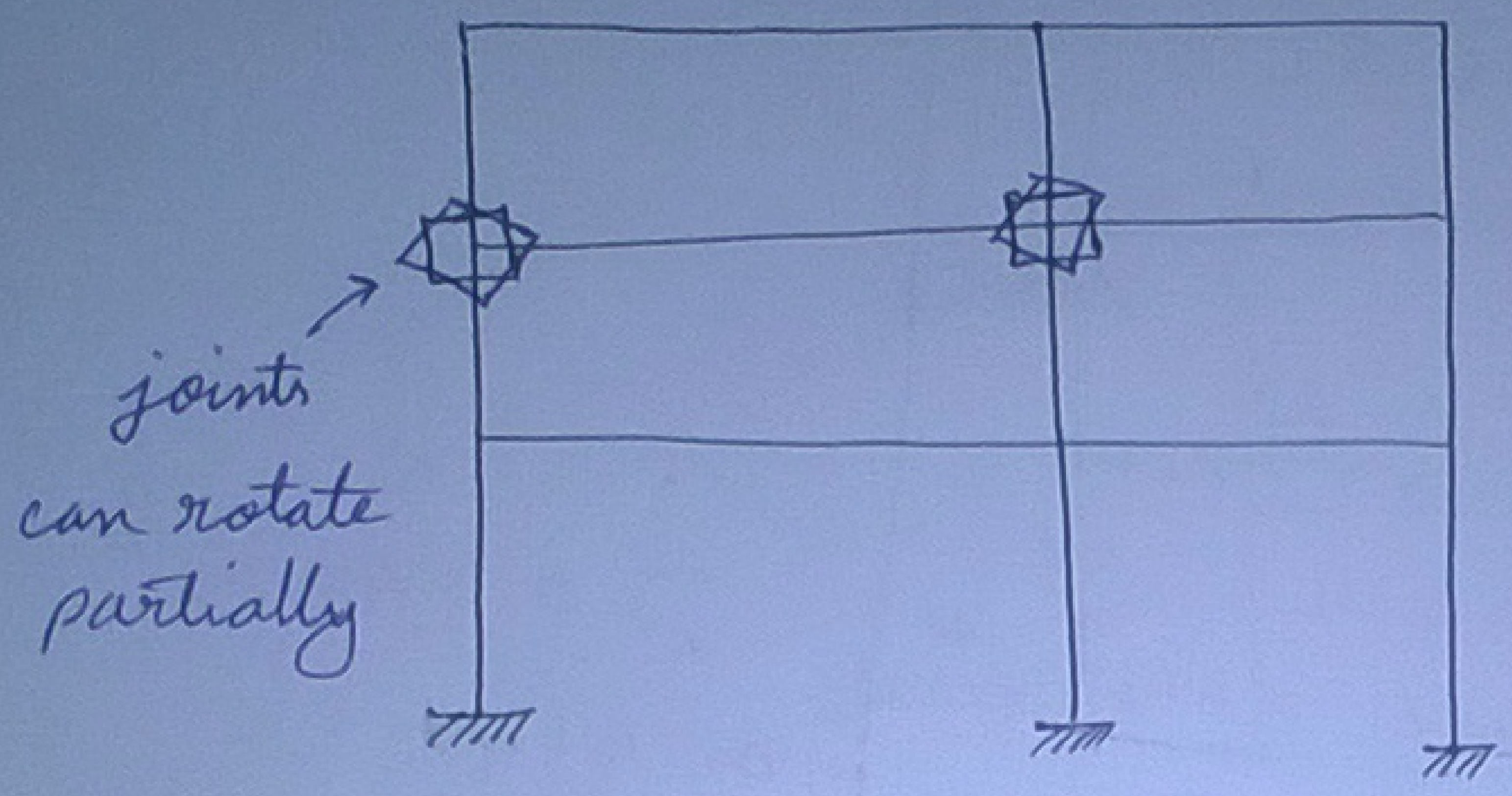
$K = \frac{L}{x}$ → ground water velocity

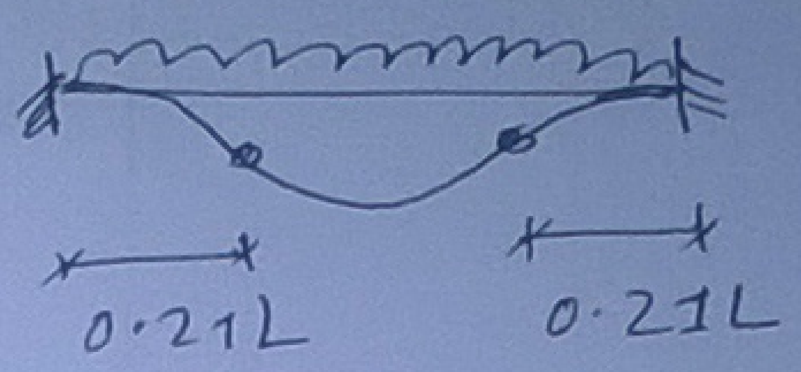
In rainy season, ground water table is closed to ground level.

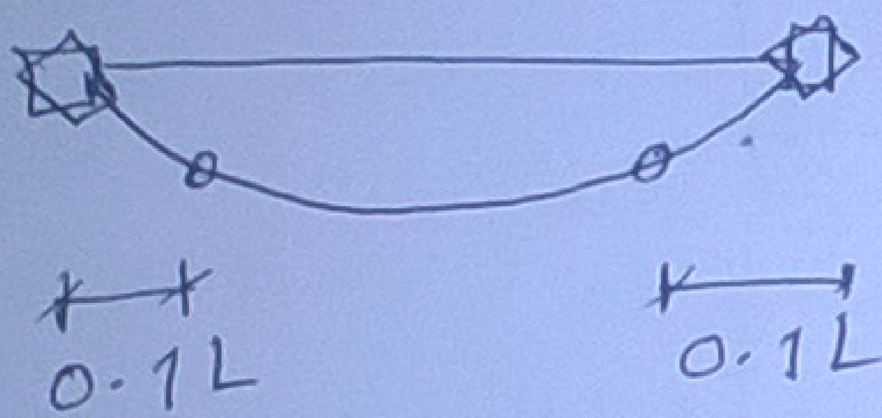
biodegradable

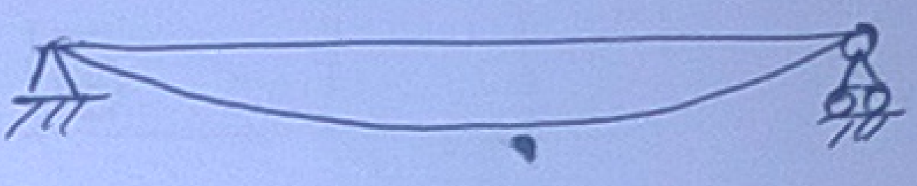
27.04.15

Taufiq Sir









Degree of Indeterminancy, d.o.i = 3× no. of closed loops − No. of releases.

→ d.o.i = 3×6 − 0 = 18

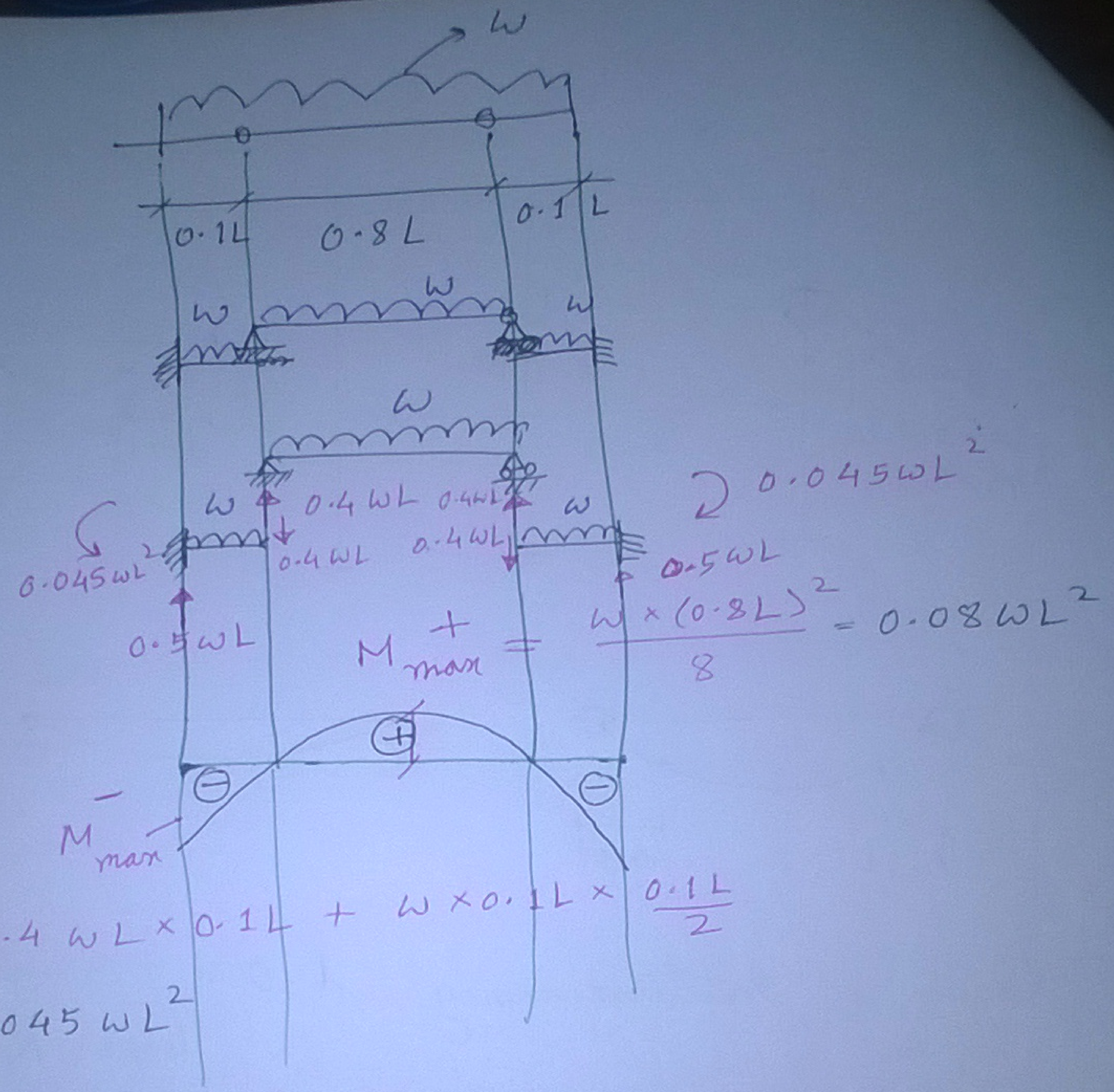
If no releases, d.o.i = 3× no. of closed loops
= 3× no. of girders

Assumptions required = 3× no. of girders

or 3 assumptions per girder.

Assumptions For Each Girder:

1. The axial force in the girder is zero.
2. A point of inflection occurs at the 0.1L (one tenth) point measured ~~all~~ along the span from the left support
3. A point of inflection occurs at the 0.1L (one-tenth) point measured along the span from the right support

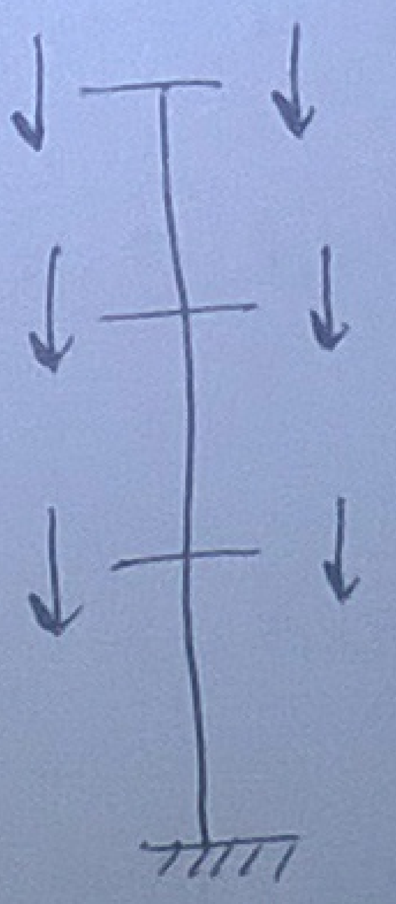


$$M^{+}_{max} = \frac{w \times (0.8L)^2}{8} = 0.08\,wL^2$$

$$M^{-}_{max} = 0.4\,wL \times 0.1L + w \times 0.1L \times \frac{0.1L}{2}$$

$$= 0.045\,wL^2$$

Axial Forces in columns:

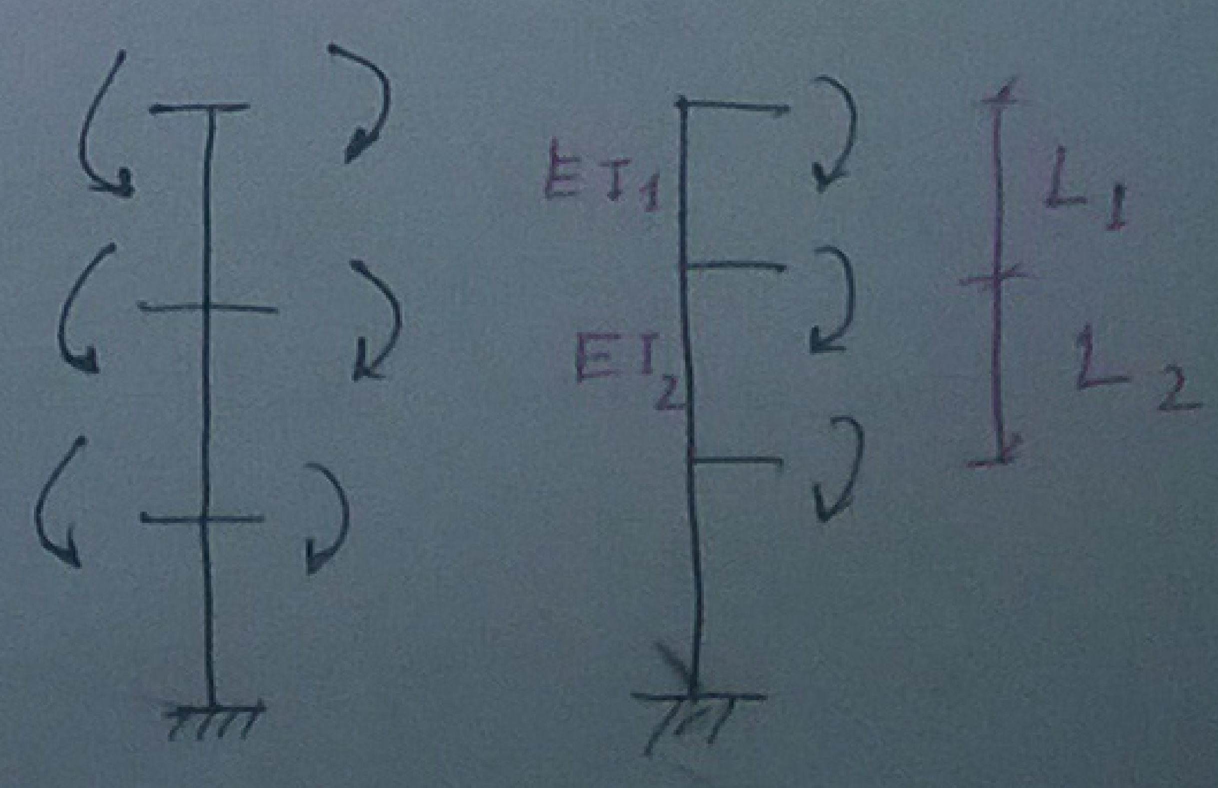


Column
axial Force = Σ of Girder shears from top to section concerned.

VVI

(Bending moment in columns) Moments in Cols^m:

Σ Column moment = algebraic Σ Girder moment



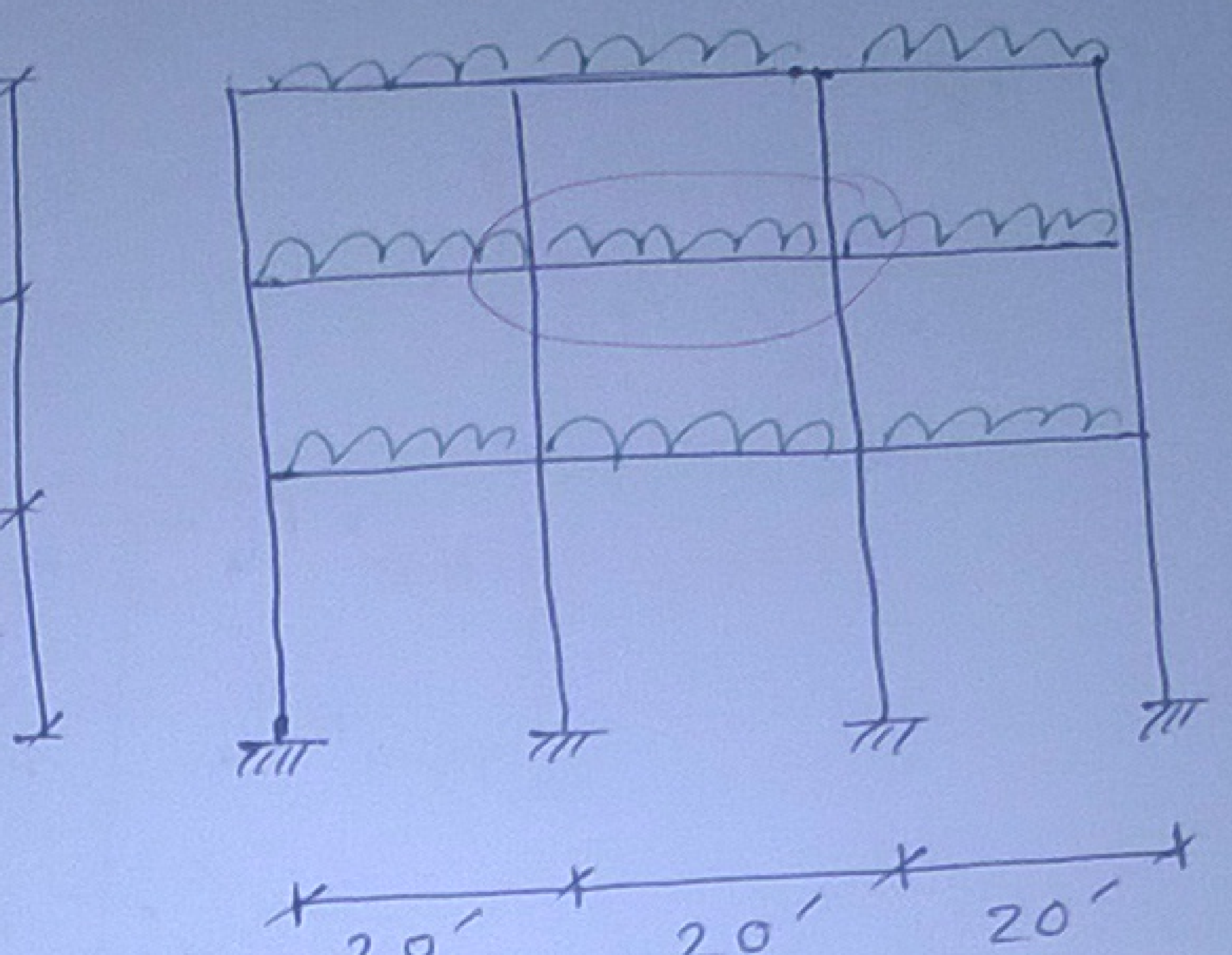
\# Distribute Column moments between the top & bottom column in proportion to their stiffness EI/L

Problem 7.6 (Norris)

Given,

$DL = 0.5\ k/'$

$LL = 0.3\ k/'$



(a) M_{max}^{+}

(b) $M^{G}_{max}{}^{-}$

(c) V^{G}_{max} (max^m shear at girder)

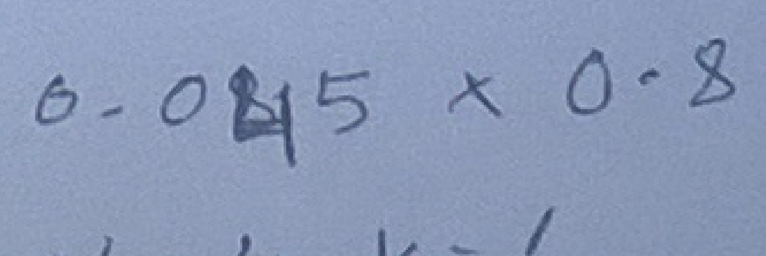
(d) $P^{\text{exterior col.}}_{\text{max}^m\text{ compression}}$

(e) $P^{\text{int. col}}_{\text{max}^m\text{ compression}}$

(f) $M^{\text{ext. col.}}_{max}$

(g) $M^{\text{int. col.}}_{max}$

$W = 0.5 + 0.3 = 0.8\ k/'$

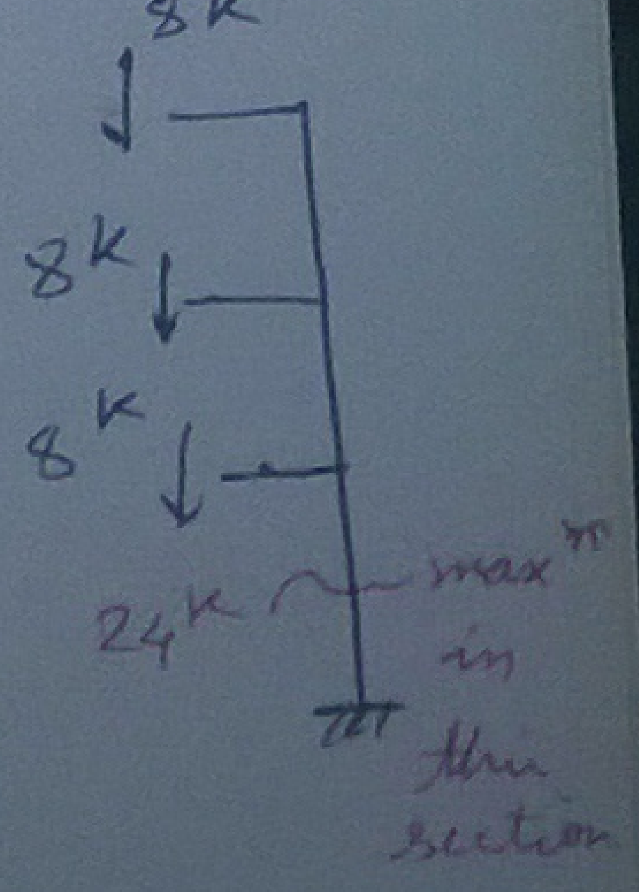


Sol^n. (a) $M_{max}^{+} = 0.08\, wL^2 = 0.08 \times (0.8) \times (20)^2$
$= 25.6\ k\text{-}ft$

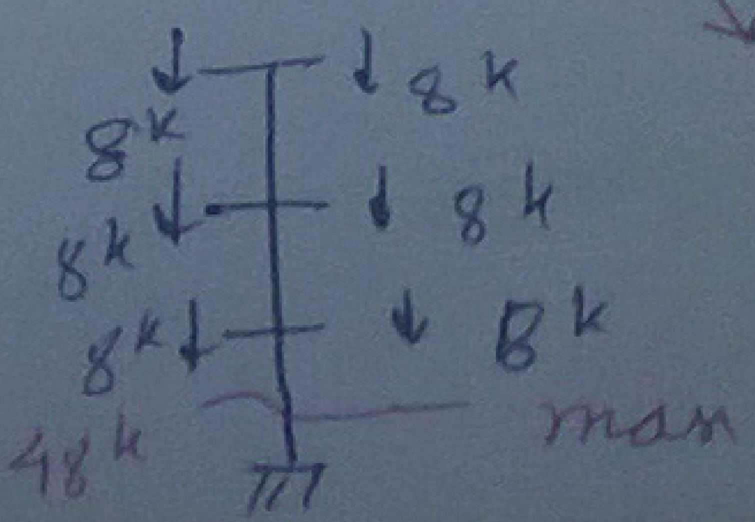
(b) $M^{G}_{max}{}^{+} = 0.045\, wL^2 = 0.045 \times 0.8 \times 20^2$
$= 14.4\ k\text{-}'$

(c) $V^{G}_{max} = 0.5\, wL = 0.5 \times 0.8 \times 20 = 8^k$

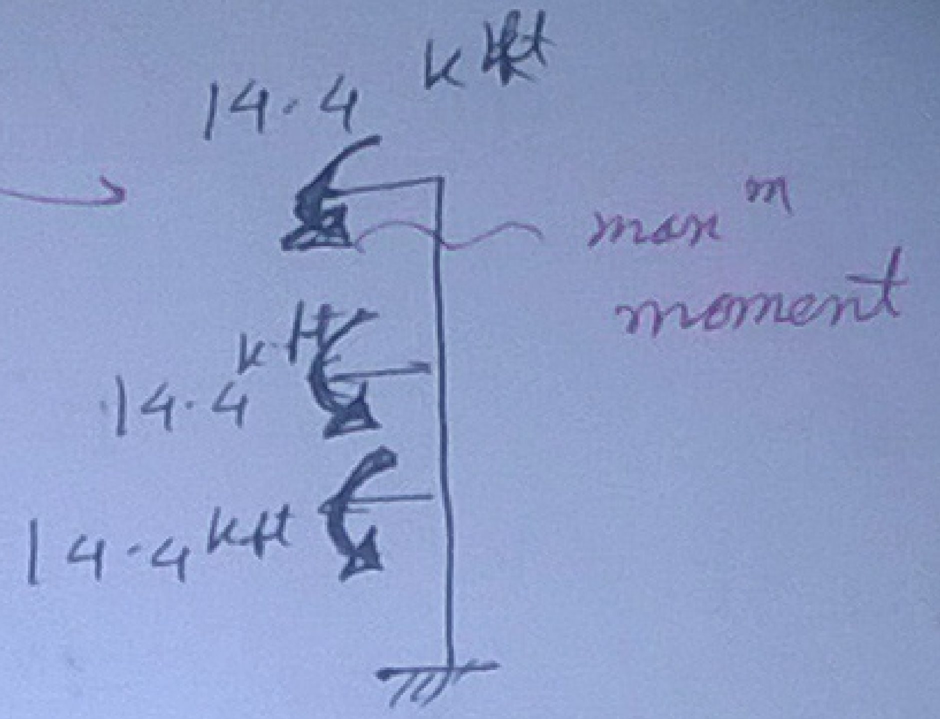
(d) $P^{\text{exterior col}}_{\text{max}^m\text{ compression}} = 8 + 8 + 8 = 24^k$



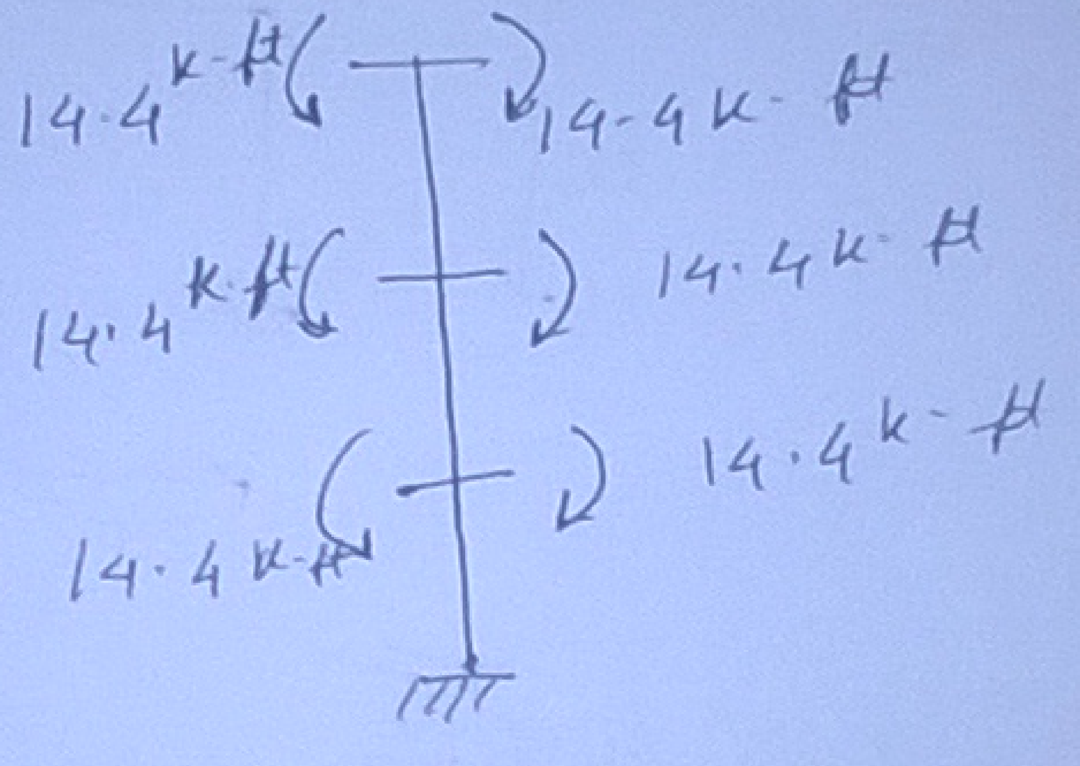
(e) $P^{\text{int col}}_{\text{max}^m\text{ compression}} = 8+8+8+8+8+8 = 48^k$



(f) $M_{max}^{ext.\ col.} = 14.4^{k-ft}$

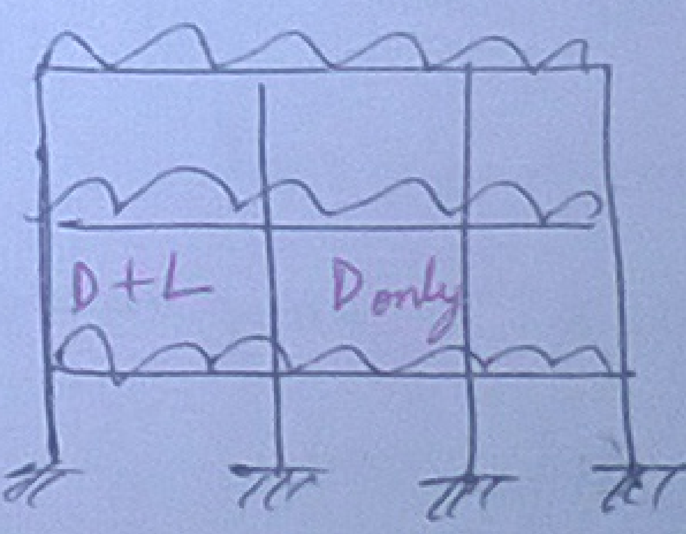


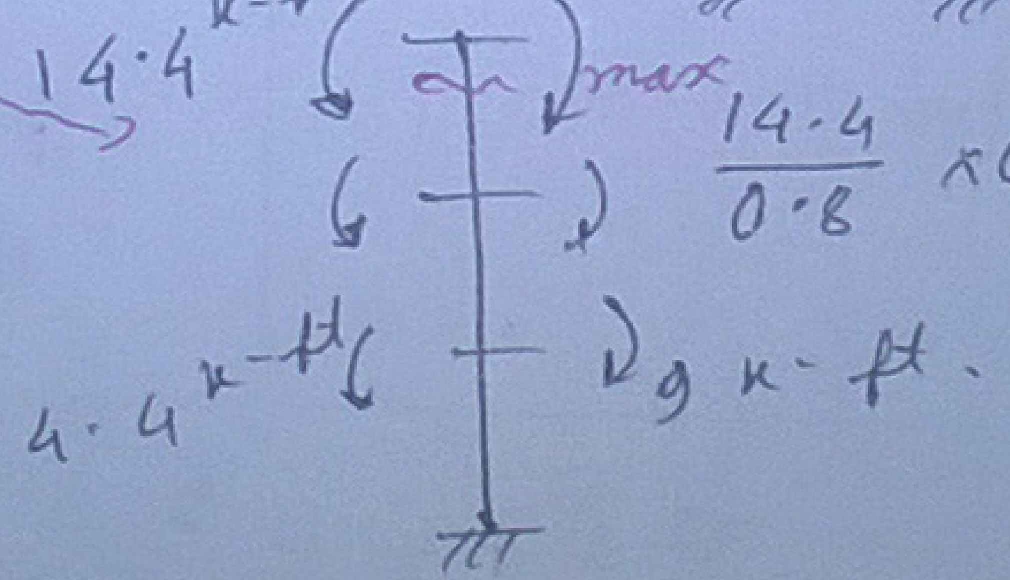
(g) $M_{max}^{int.\ col.} = 14.4 - 9$
$= 5.4^{k-ft}$



Let,

Let,





$\frac{14.4}{0.8} \times 0.5 = 9$ k-ft (for D. Load only)

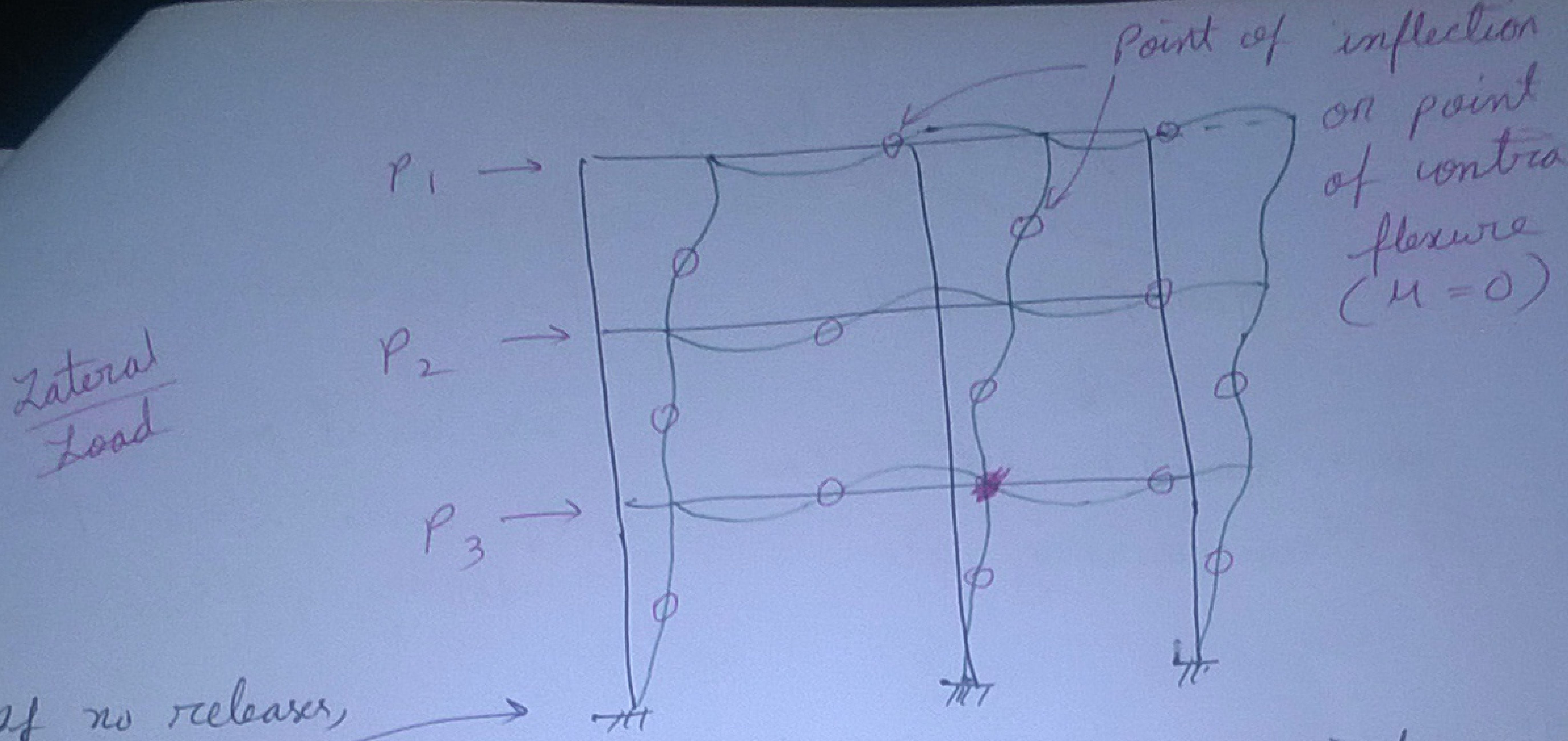
Maths

(+) Span can be different

(*) There can be hinge (col^m moment = 0)

Building Frames ; Lateral Loads

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



If no releases, then

d.o.i. = 3× no. of closed loops − 3× no. of girders

d.o.i = 3 × 6 = 18

∴ Assumptions required = 18

* Bends in double curvature (moment changes the sign)

☐ For Lateral Load Analysis
↓
2 methods

Portal Method — Cantilever Method

For both methods Common Assumption.

- There is a point of inflection at mid height of each column ≡ no. of col's = 9 assumptions
- There is a P.O.I. at mid span of each girder ≡ No. of Girders = 6 assumption

∴ Σ = 15 assumptions

(Additional assumptions (18 − 15 = 3) will depend on methods)

Additional Assumption

For Portal Method

↓

are based on distribution of stress among cols.

For Cantilever method

↓

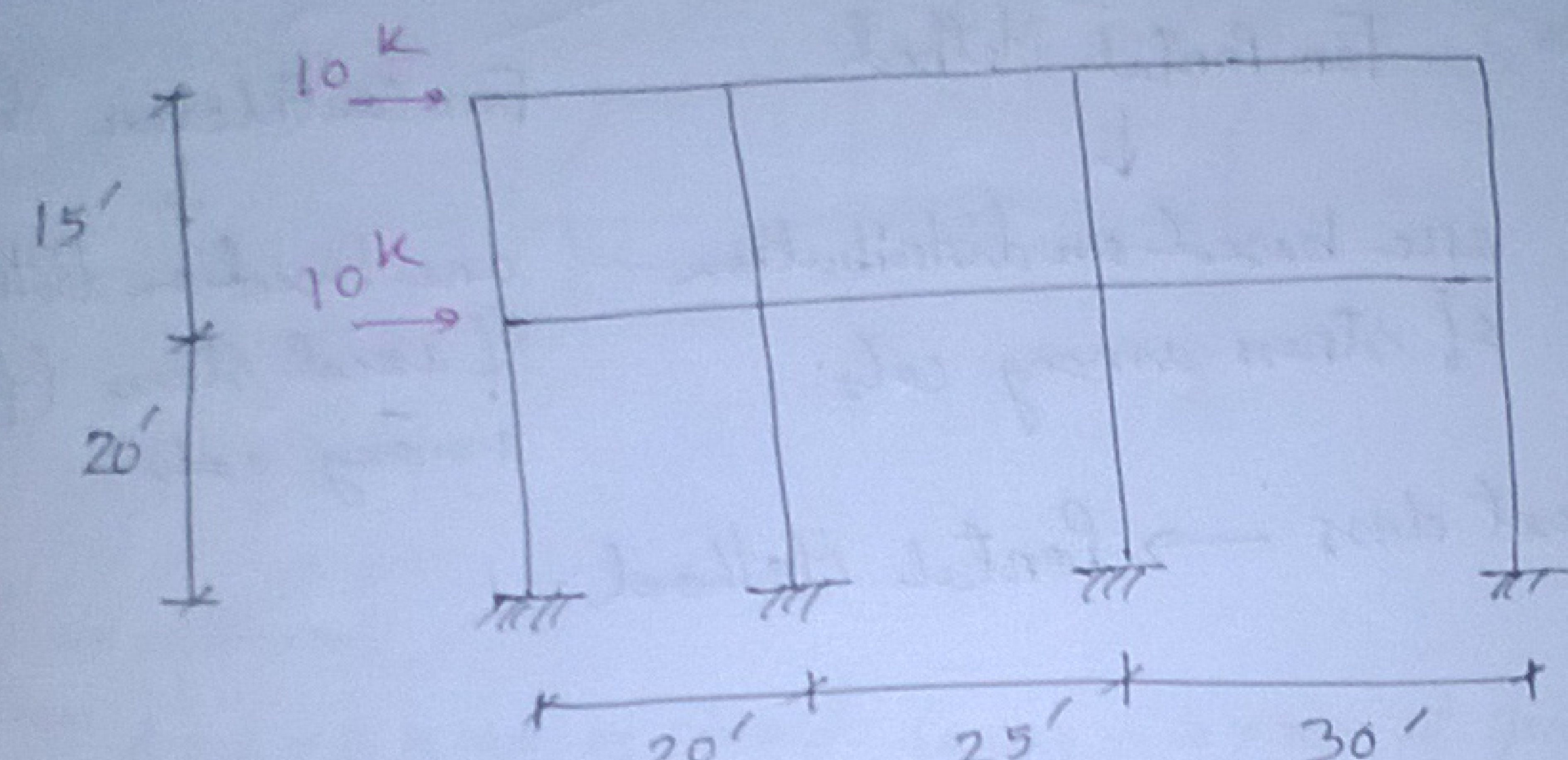
are based on distribution of axial stress (force) among cols.

Next class → Portal Method.

2.08.15

Taufiq Sir

Portal Method :



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

No. of assumptions required = 18

Assumptions :

1. There is a point of inflection at centre of each girder

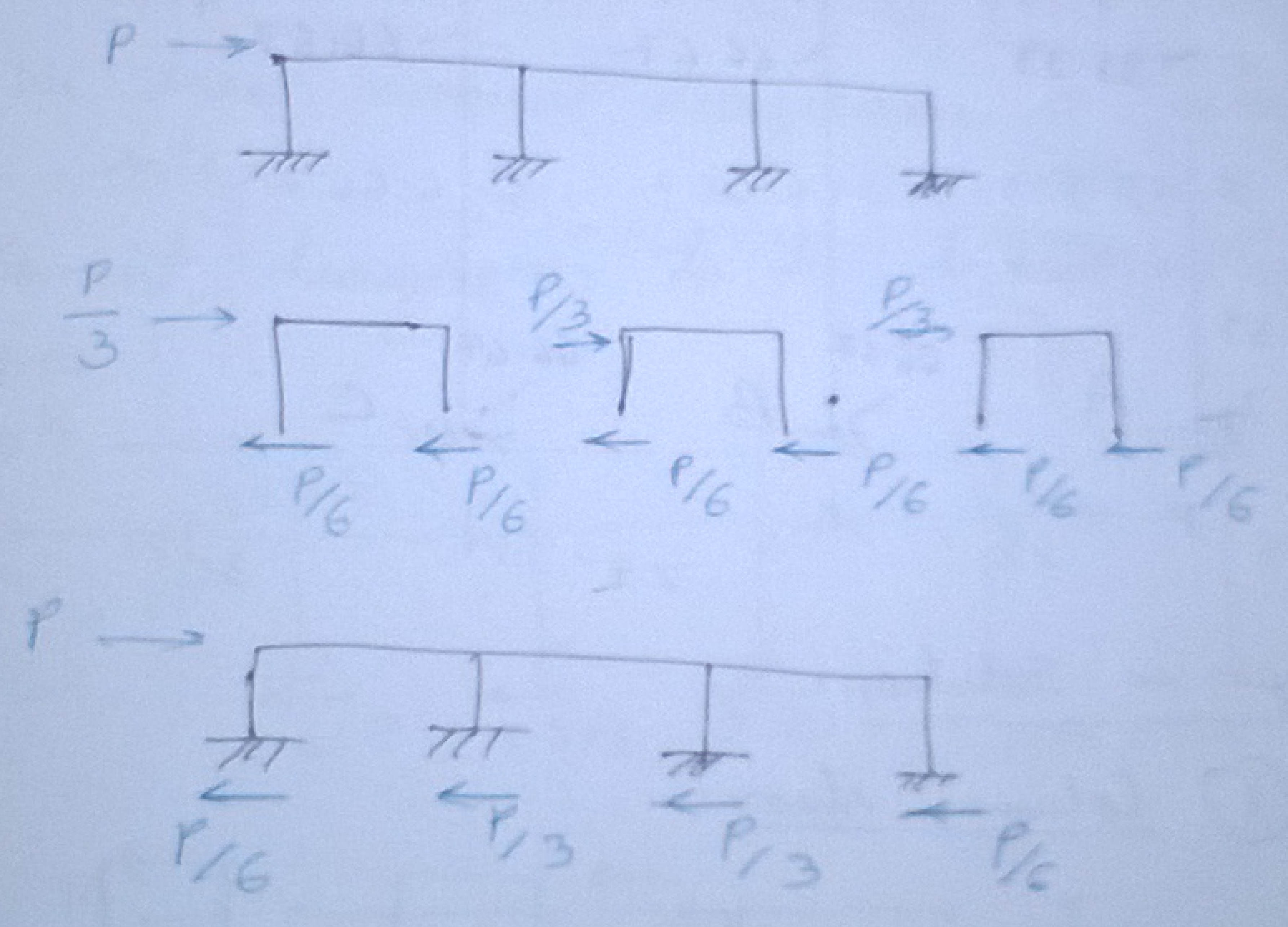
   ① ≡ no. of girder $\boxed{① \equiv 6 \text{ assumptions}}$

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

   ② ≡ no. of columns $\boxed{② \equiv 8 \text{ assumptions}}$

[We need more 18 − 14 = 4 assumptions]

③ Total horizontal shear on each storey is divided between the columns in that storey such that each interior column ~~carries~~ carries twice as much shear as each exterior column.

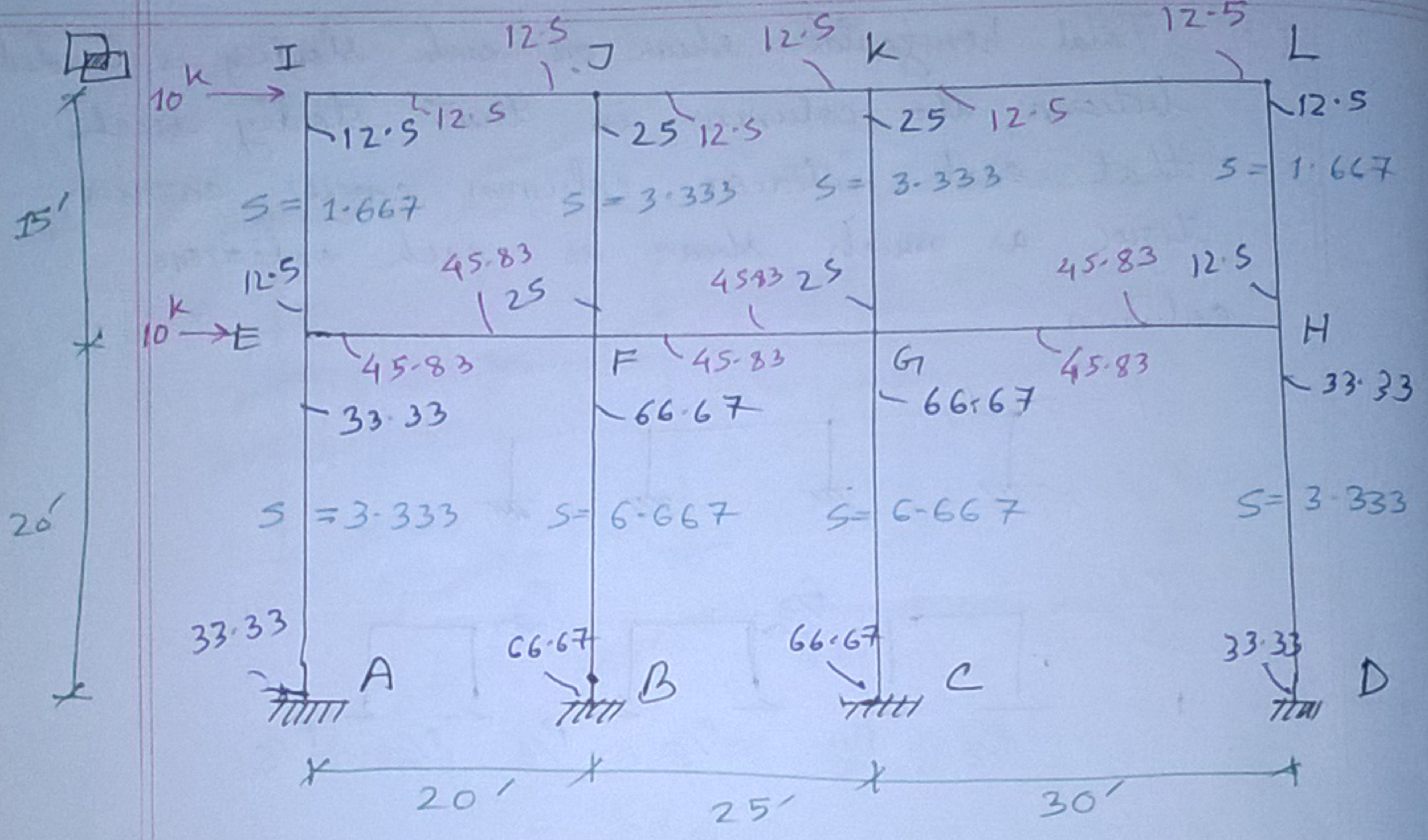


③ = no. of storey × (m − 1)
where, m = no. of columns in a storey

③ = 2 × (4 − 1) assumptions = 6

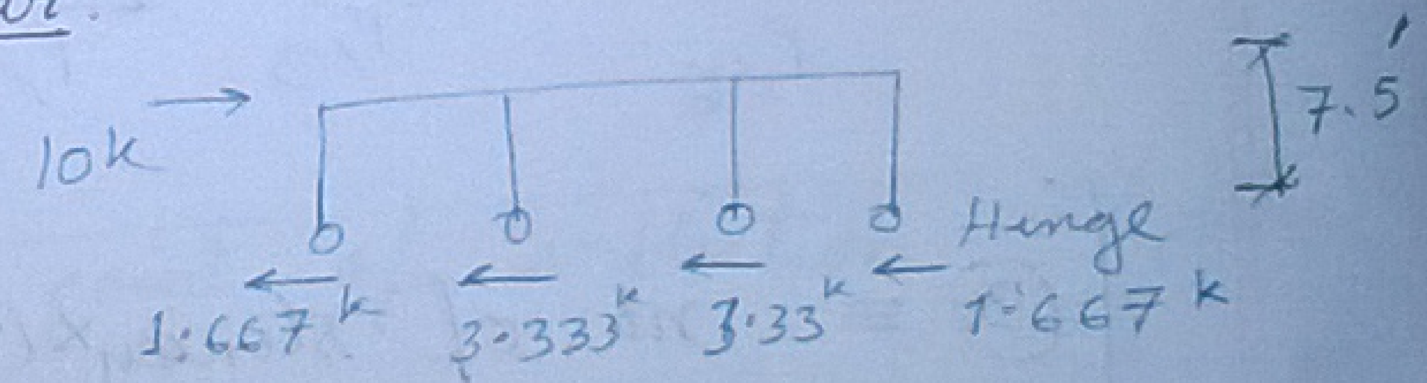
Total $\Sigma 20 < 18$ assumptions

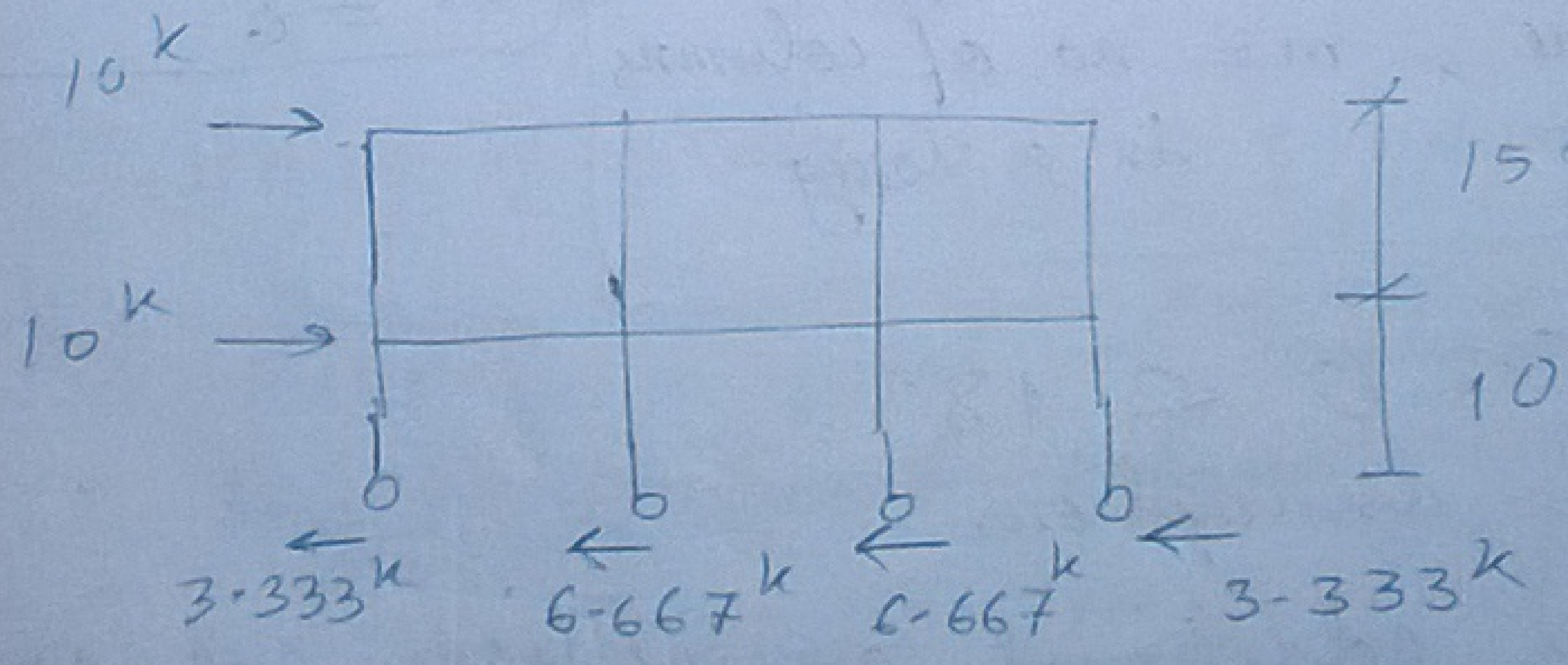
However, additional assumptions are consistant.



Steps in Solution:

① Column shear:





20k is divided into 6 part

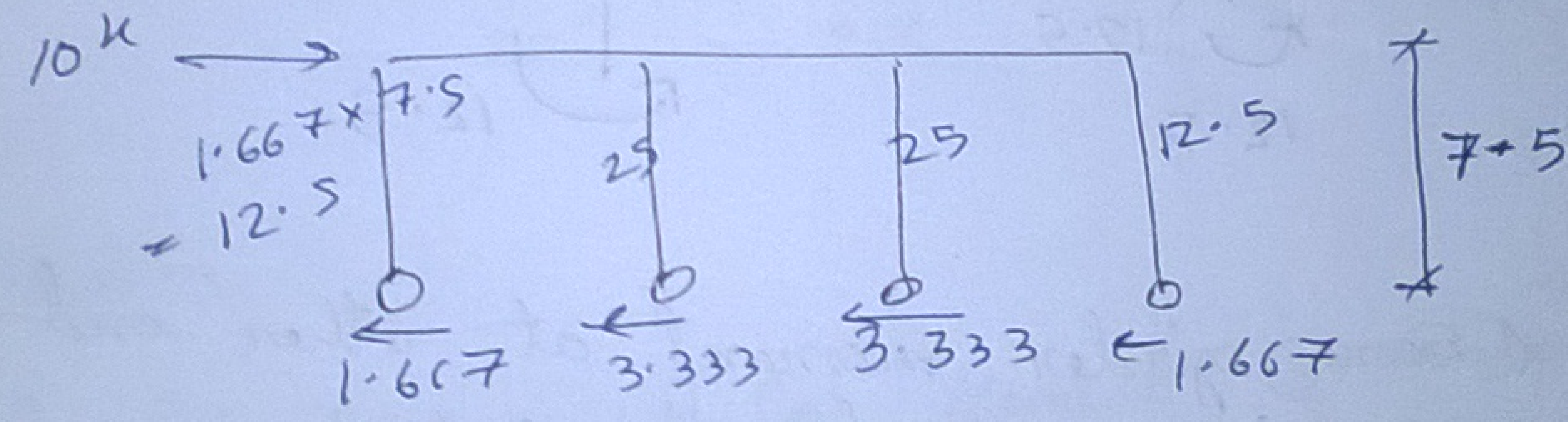
② Column Moment:

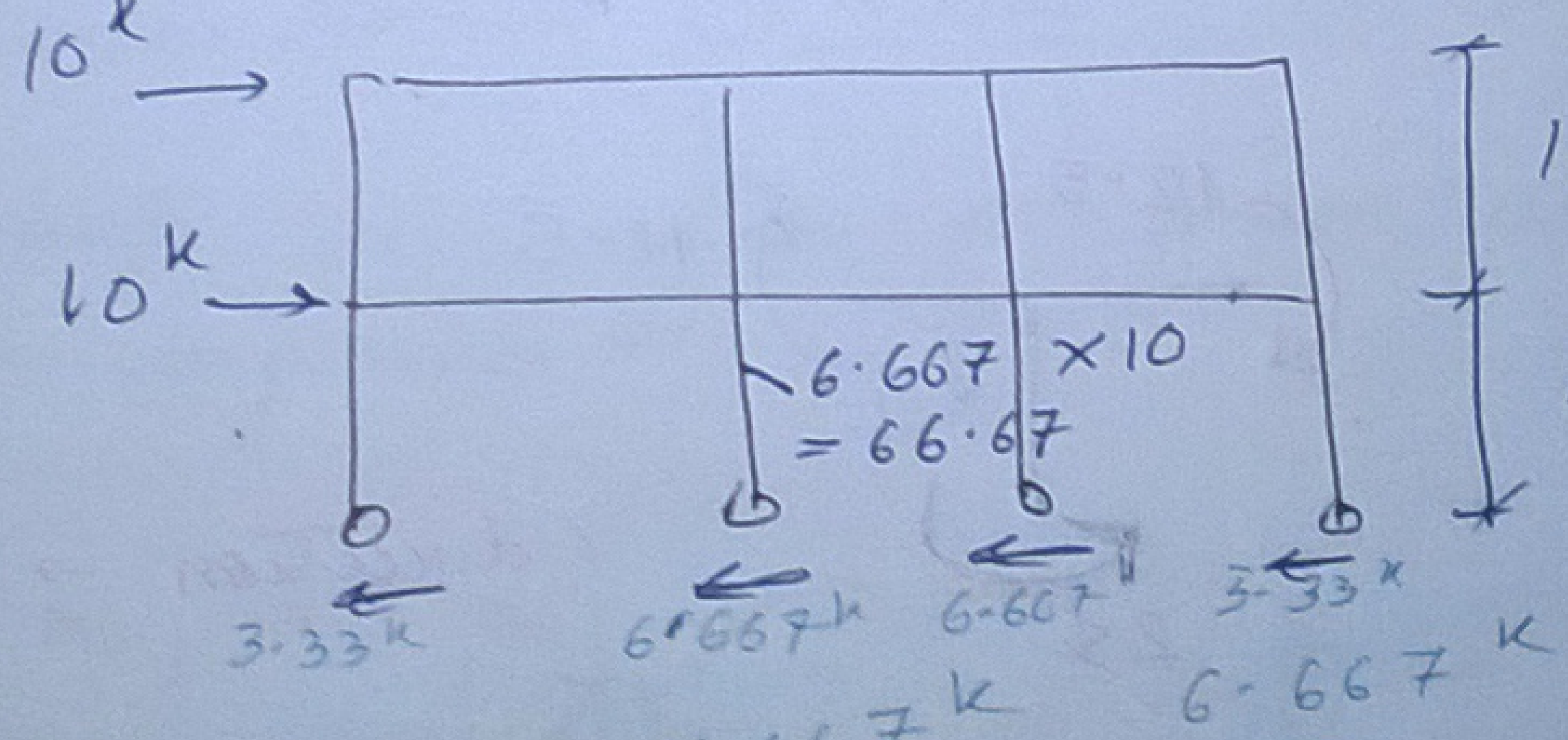
(column shear × half the height of column)

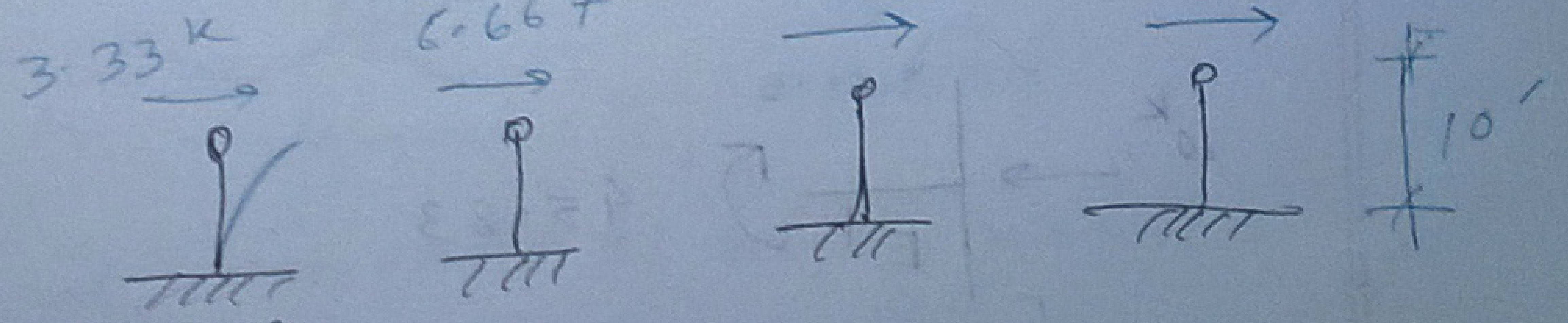
Column moment = col shear × $\frac{ht.\ of\ col}{2}$ (write figures on tension side)

(We draw moment diagram in compression side)

Moment diagrams to be drawn on compression side.





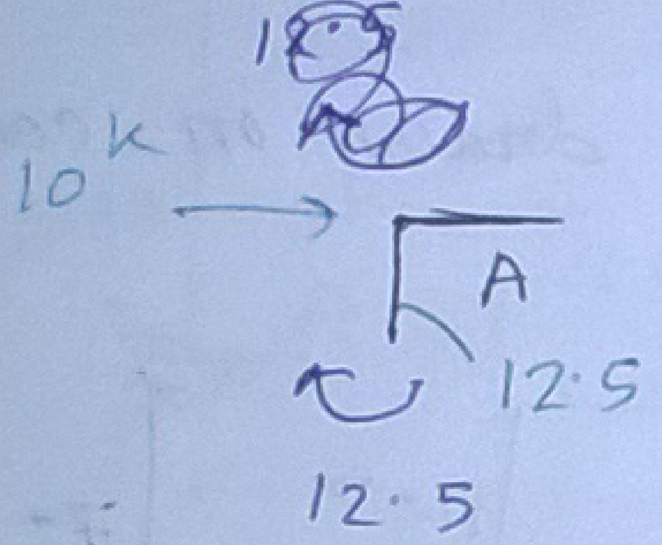


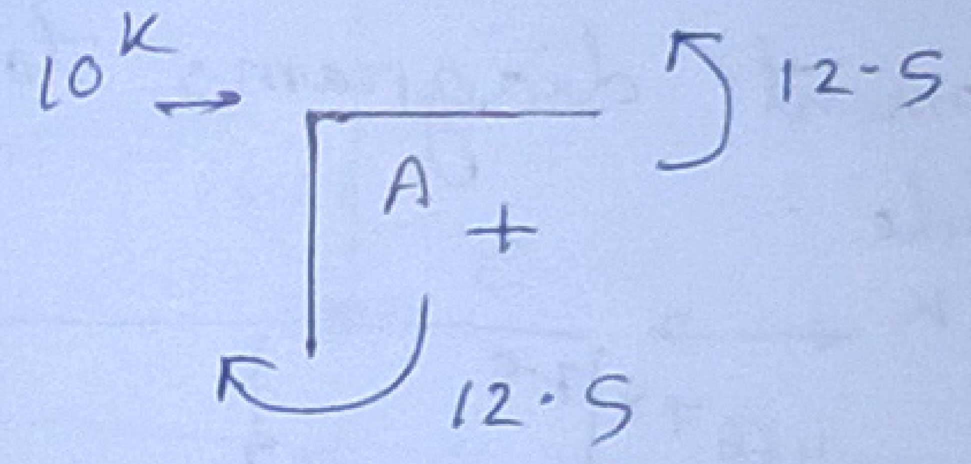
3.333 × 10'
= 33.33

③ Girder Axial Force (Not so important)

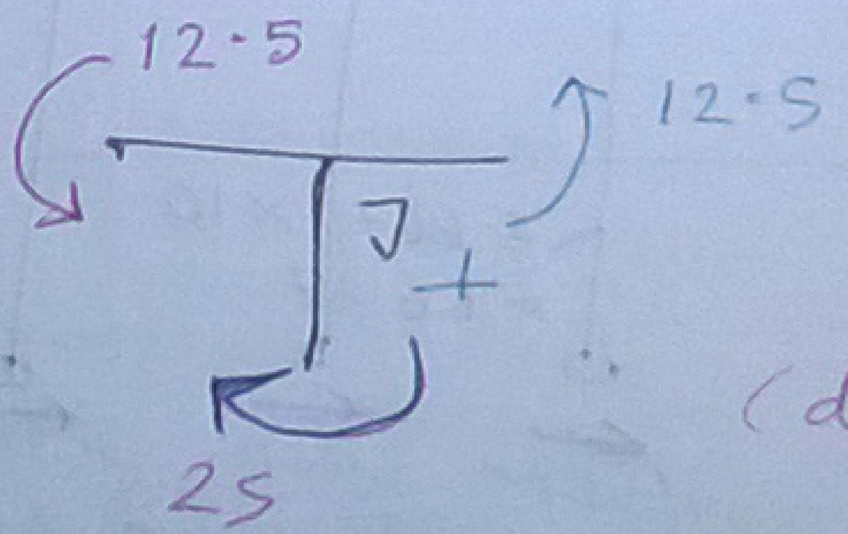
④ Girder Moment :→ From column moment (From joint equilibrium)

(Start from exterior end of a girder & move)

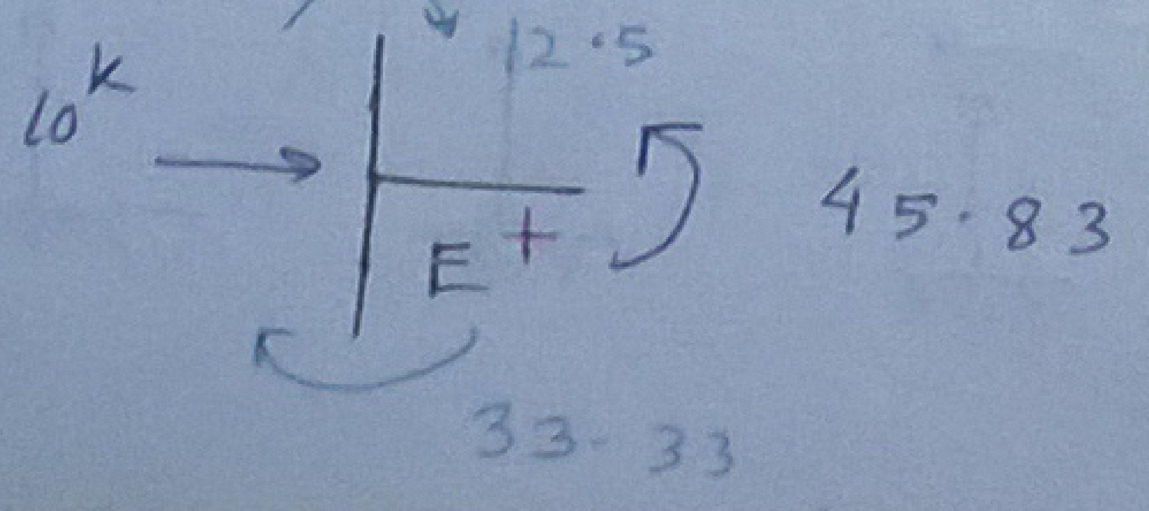




(Same girder moment at other end as no distributed load on girder)



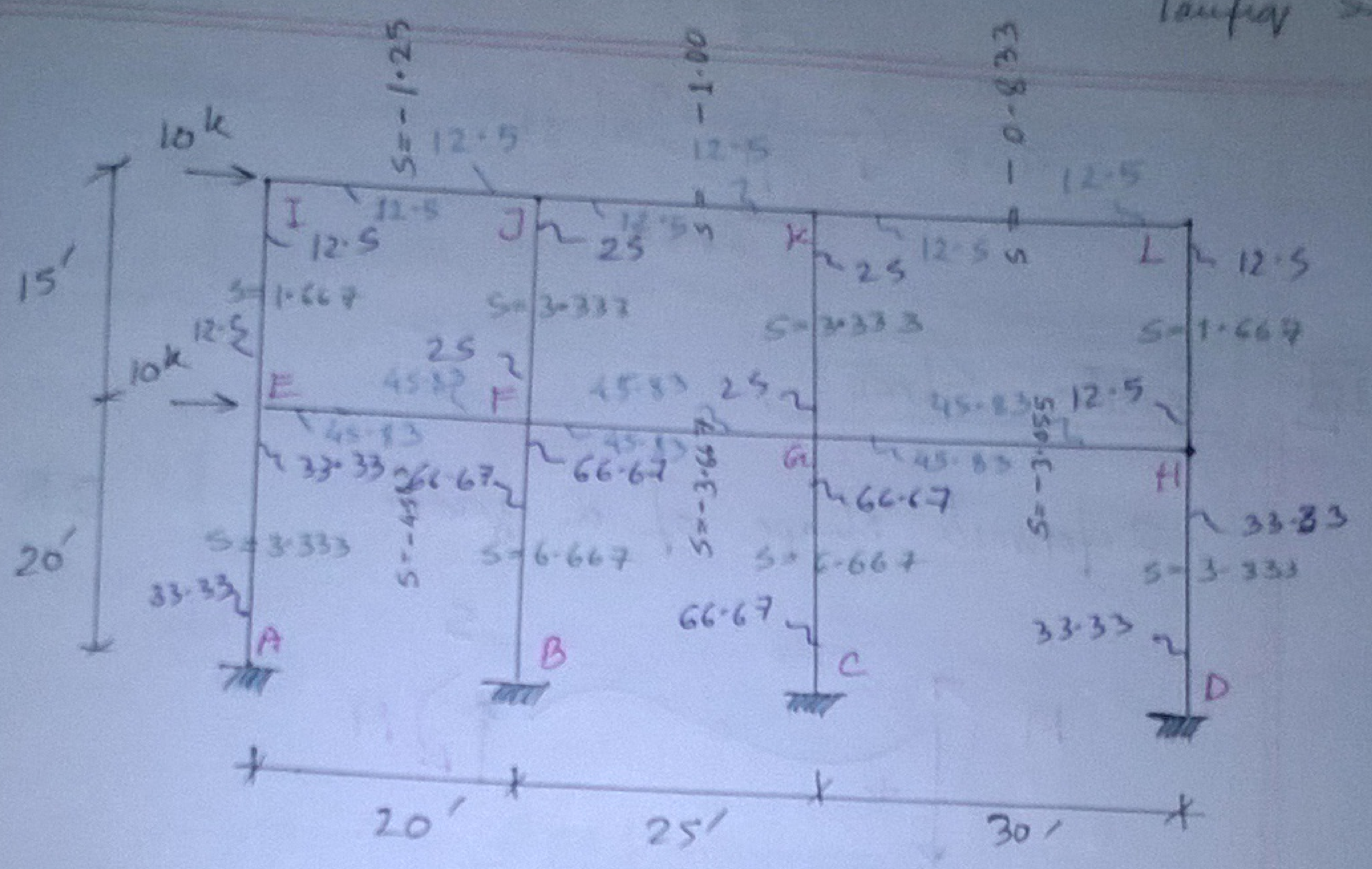
(direction → based on ~~tension~~ tension)



9.06.15

(5)

Taufiq Sir

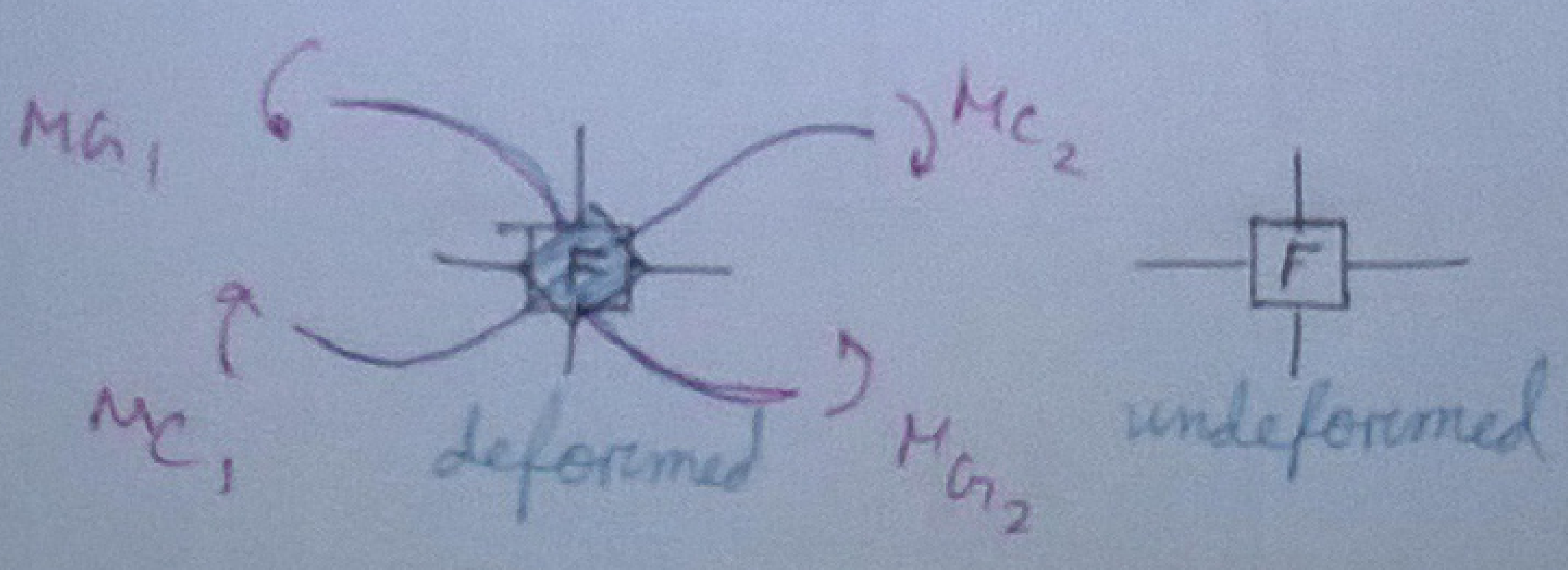


<u>Steps:</u>

(1) Col. Shear (2) Col. Moment (3) Girder Ax. Force

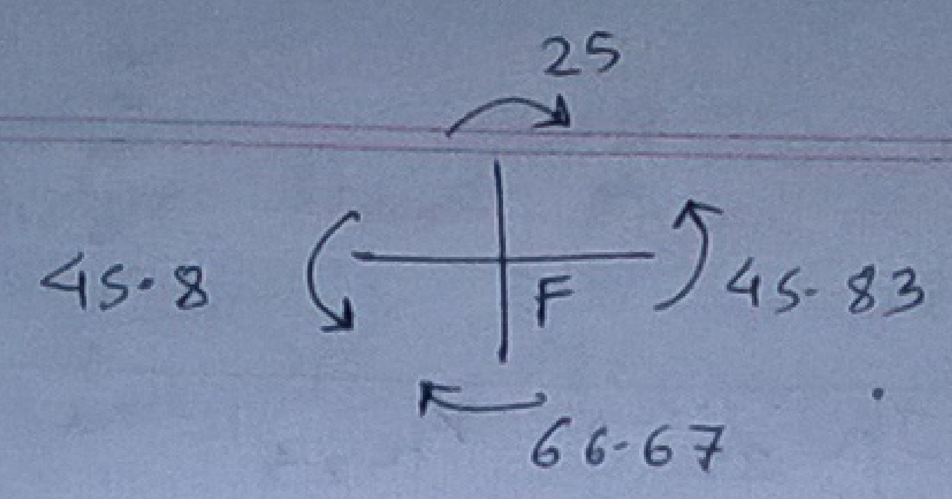
(4) Girder Moment (5) Girder Shear (6) Axial Force in Columns from Girder Shear

(*) Moments were written in tension side as we'll draw in compression side



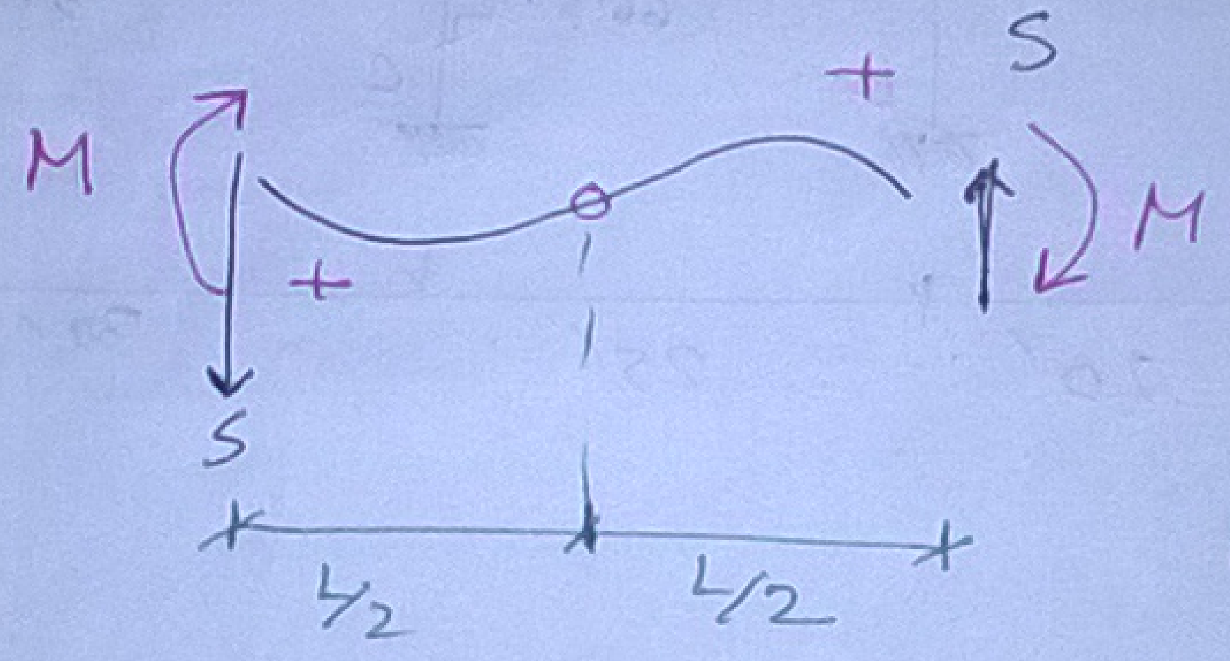
(*) Σ Col. mom. = Σ of Girder moment.

$$M_{C_1} + M_{C_2} = M_{G_1} + M_{G_2}$$



Girder Shear :

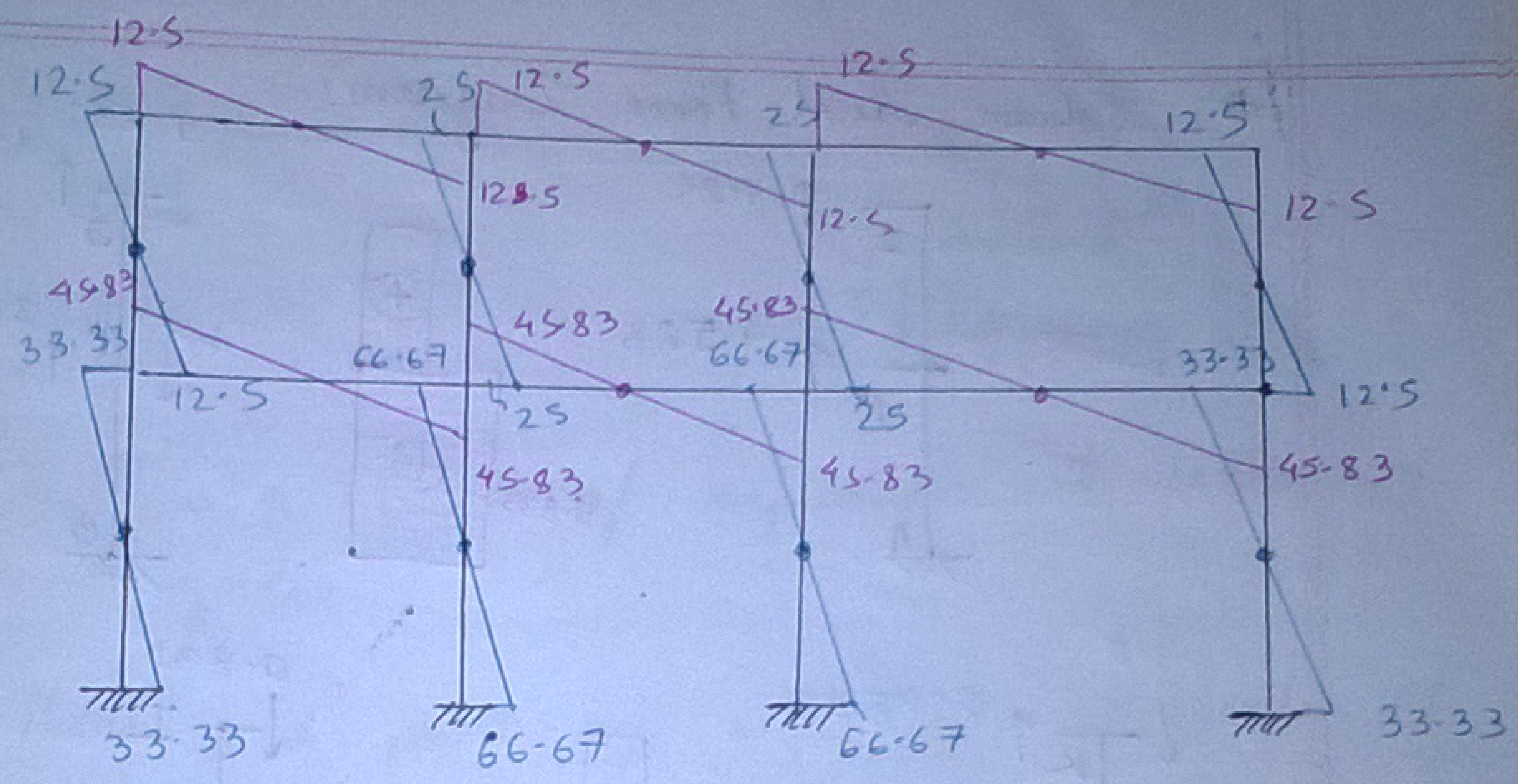
Deformed shape of a girder.



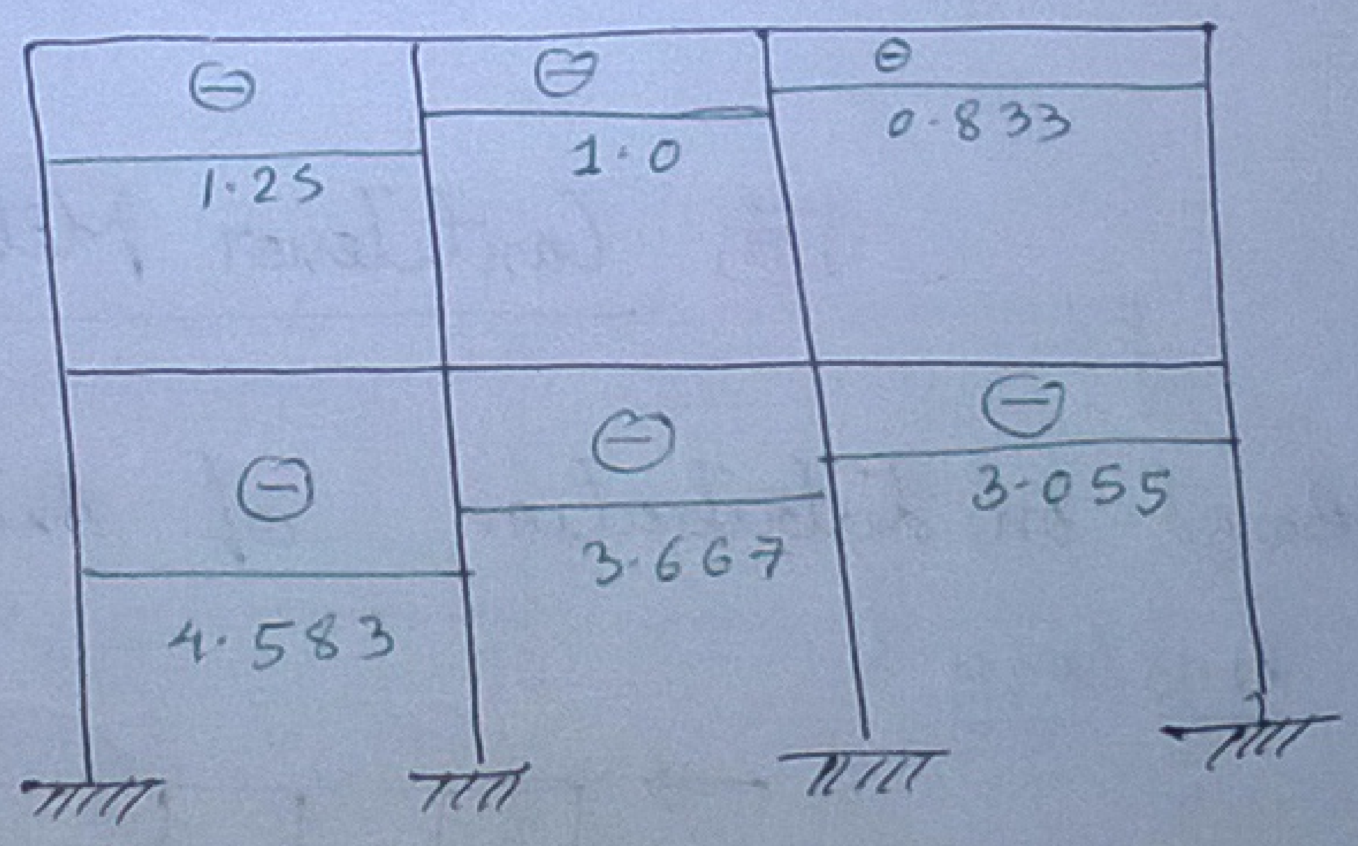
$$M - S \times \frac{L}{2} = 0$$

$$\therefore \; S = \frac{2M}{L}$$

Axial Force in Columns from Girder Shear:

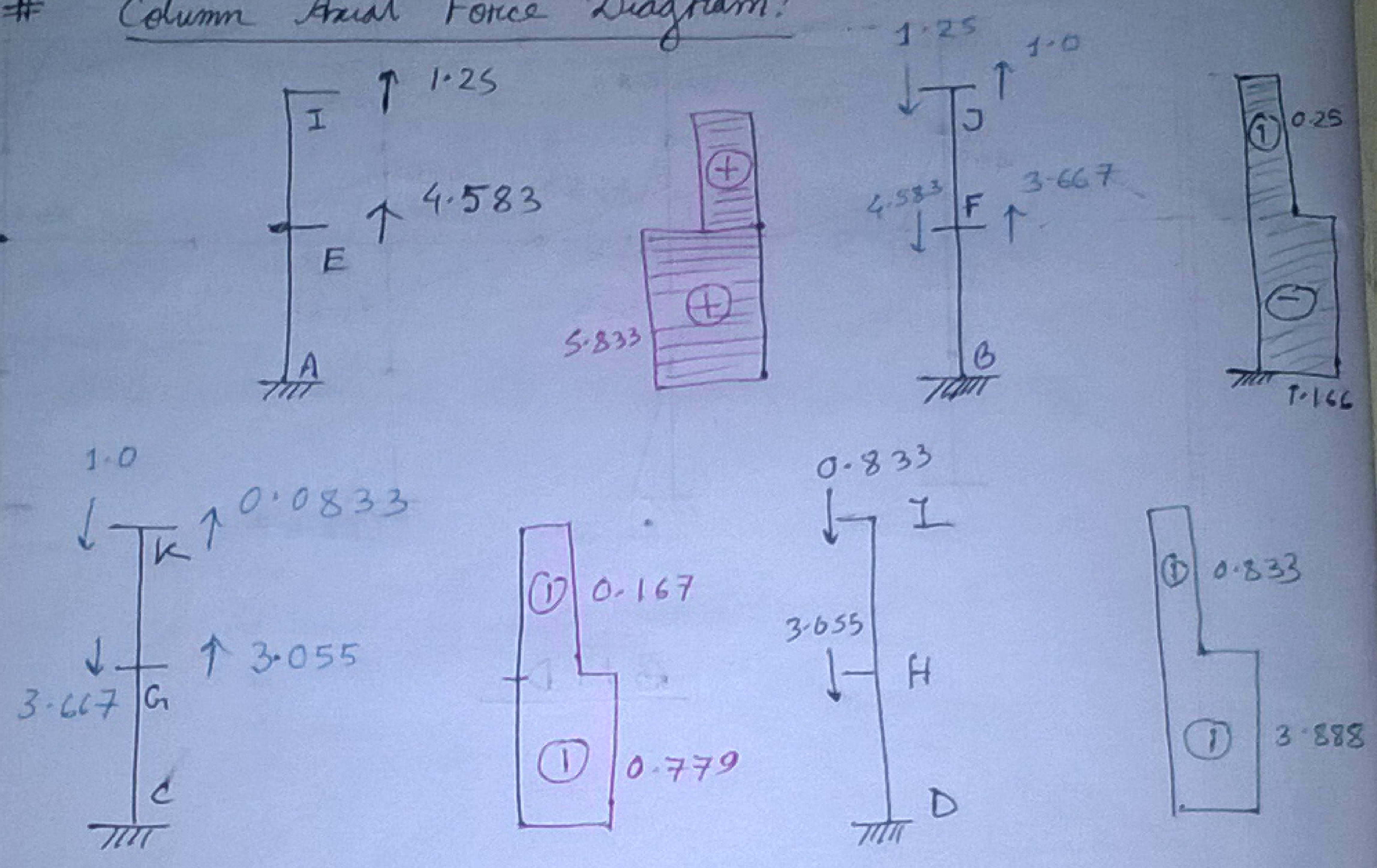

12.5
12.5
25
12.5
25
12.5
12.5
12.5
12.5
12.5
12.5
45.83
33.33
12.5
66.67
25
45.83
45.83
66.67
25
33.33
12.5
45.83
45.83
45.83
33.33
66.67
66.67
33.33

BMD


1.25
1.0
0.833
4.583
3.667
3.055

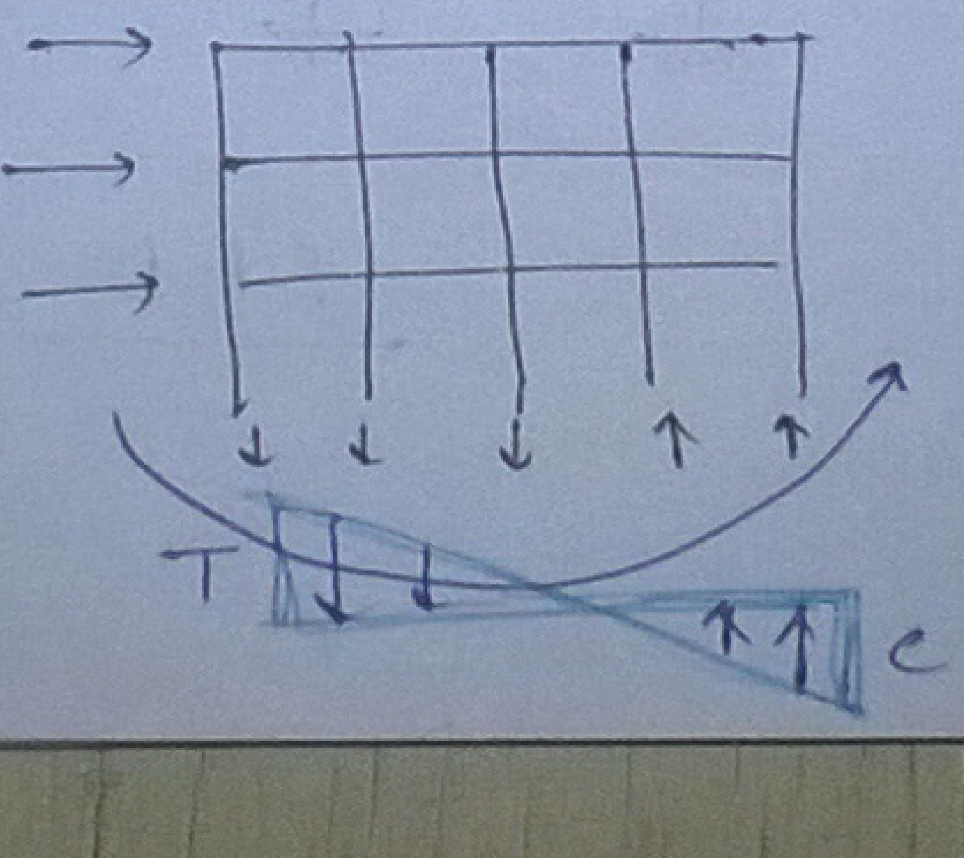
SFD

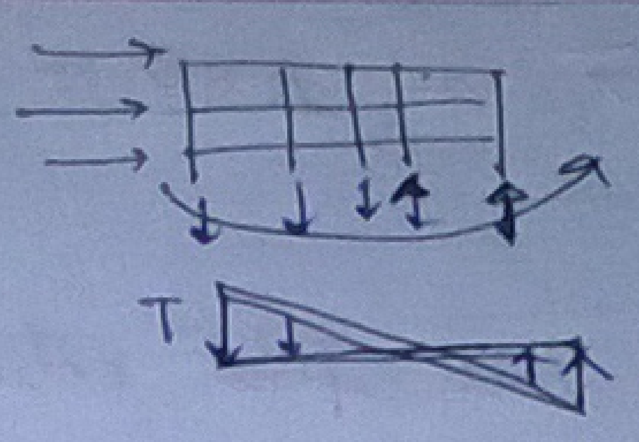
Column Axial Force Diagram:



Cantilever Method

* Based on distribution of Axial stress on column
* Analogy:





Assumptions:

1. There is a point of inflection at centre of each girder

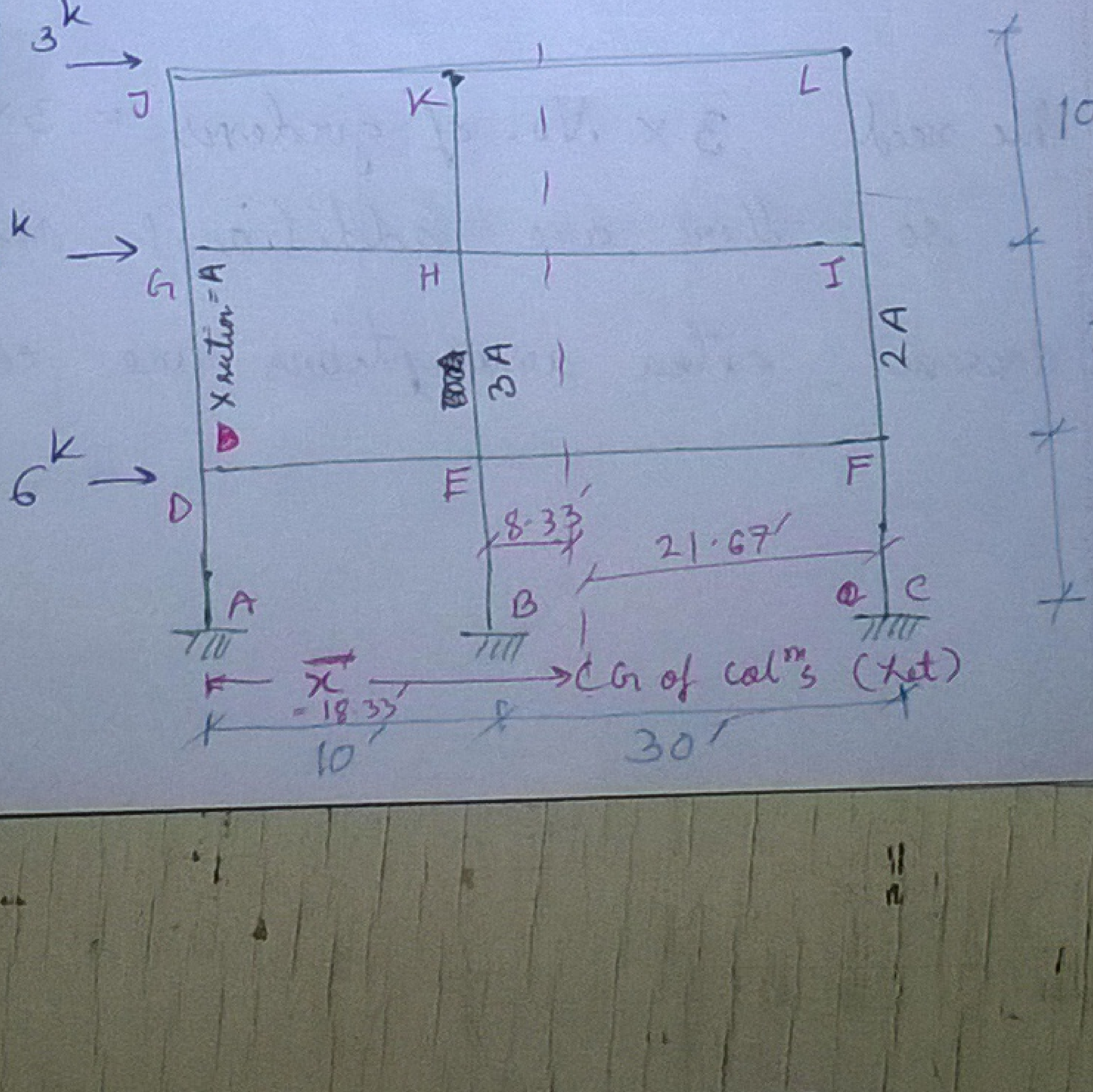
2. There is a point of inflection at centre of each column.

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

difference bet^n portal & cantilever method

Problem

Cross-Sectional Area of Col^m given



$$\bar{x} = \frac{A \times 0 + 3A \times 10 + 2A \times 40}{A + 3A + 2A} = \frac{110}{6A}$$

$$= 18.33'$$

1st will find column axial force.

Assumptions

$$\Sigma = 21 \begin{cases} ① \equiv 6 \\ ② \equiv 9 \\ ③ \equiv \text{No of storey} \times (\text{no. of column} - 1) \\ \quad \equiv \text{No. of storey} \times (m - 1) \end{cases}$$

{ no. of col^{m} per storey

$\equiv 3 \times (3 - 1)$

$\equiv 6$

We need $3 \times$ No. of girders $= 3 \times 6 = 18$

so, there are additional assumption

However, extra assumptions are consistent.

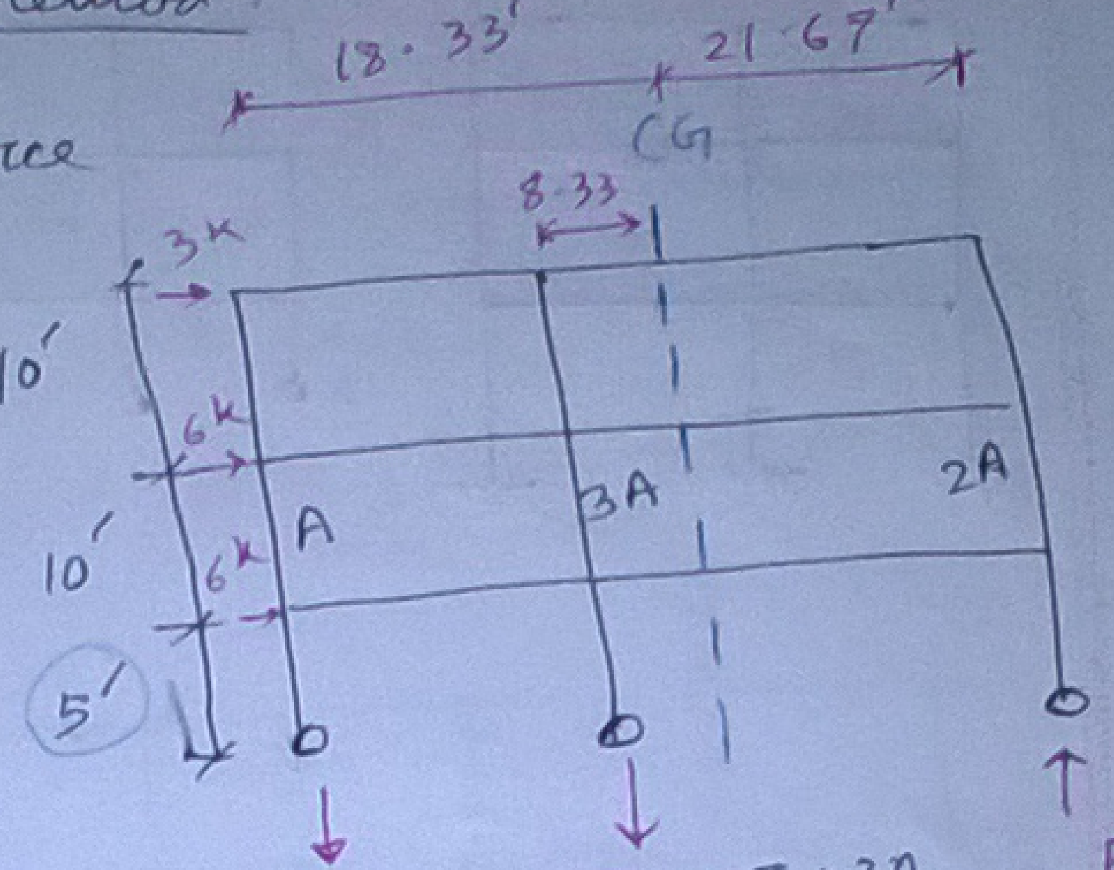
11.05.15

(10)

Taufiq Sir

Steps in Cantilever Method:

1. Column Axial Force
2. Girder Shear
3. Girder Moment
4. Column Moment (Start from top of a column and go down)
5. Column Shear (From Column moment)
6. Girder Axial Force (From Column Shear)



$F_1 = \sigma_1 A$
$= 2.41^k$

$F_2 = \sigma_2 \times 3A$
$= \frac{\sigma_1 \times 8.33 \times 3A}{18.33}$
$= 1.36 \sigma_1 A$
$= 3.28^k$

$F_3 = \sigma_3 \times 2A$
$= \frac{\sigma_1 \times 21.67}{18.33} \times 2A$
$= 2.36 \sigma_1 A$
$= 5.69^k$

$\Sigma M_a = 0 +$ $= 3 \times 25 + 6 \times 15 + 6 \times 5 - (\sigma_1 A) \times 40$
$- 1.36 \sigma_1 A \times 30' = 0$

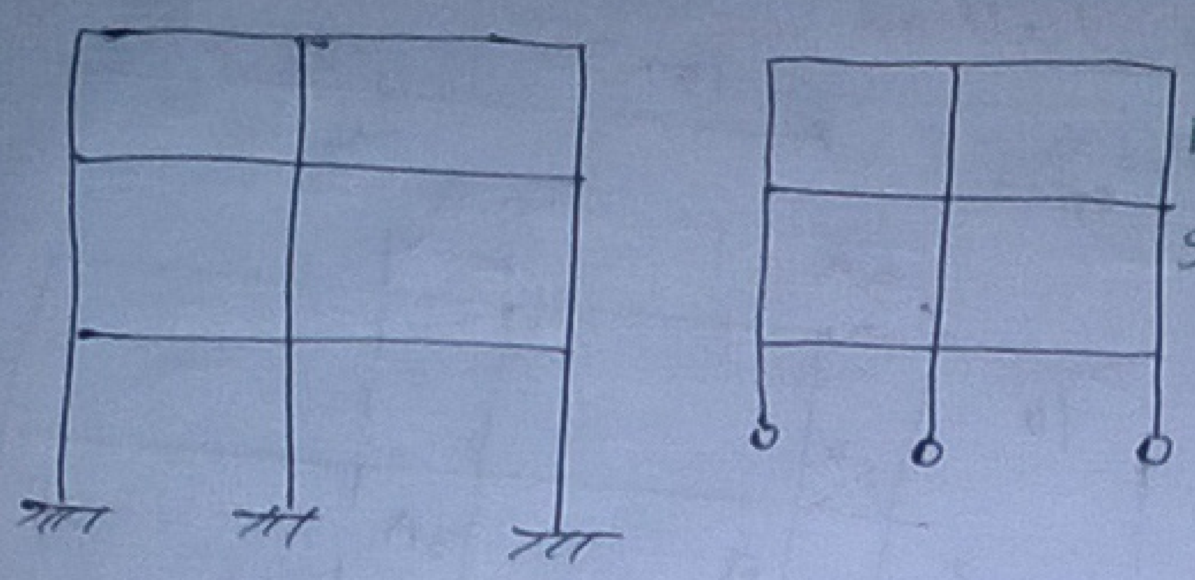
$\Rightarrow \sigma_1 A = 2.41^k$

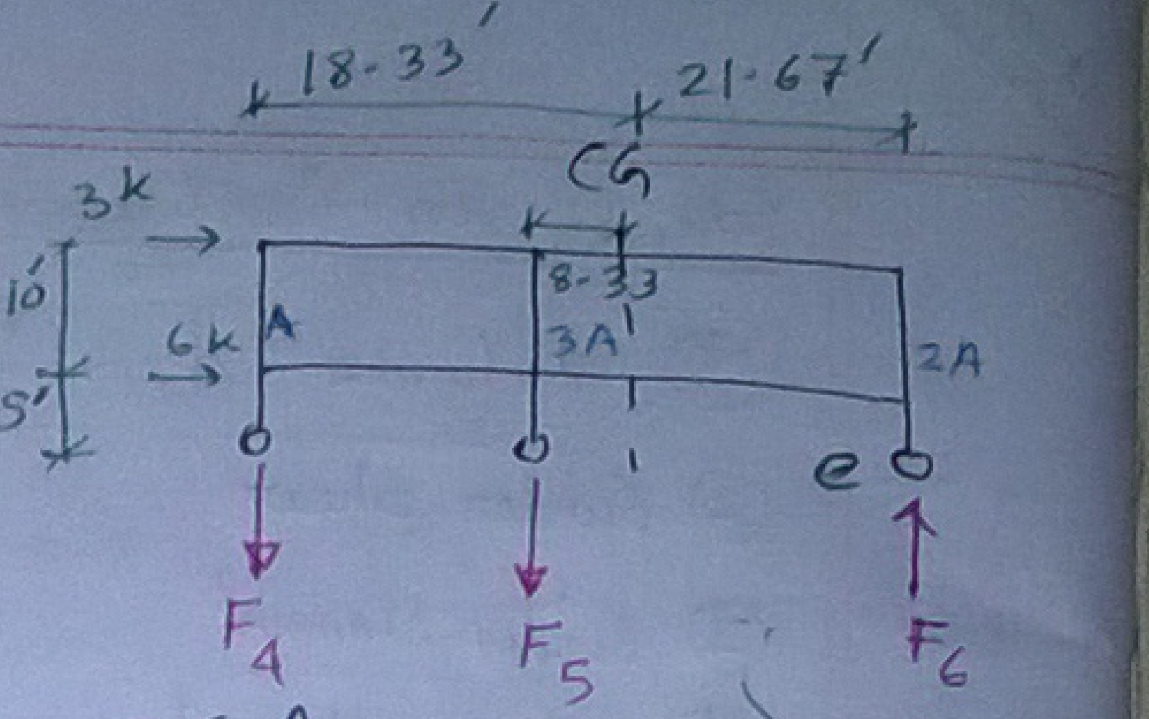
$\therefore F_1 = \sigma_1 A = 2.41^k$

$\& \; F_2 = 1.36 \times \sigma_1 A = 1.36 \times 2.41 = 3.28^k$

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

(axial forces at ground level)





$= \sigma_4 A$

$= 0.93^k$

$= \sigma_5 \times 3A$

$= \dfrac{\sigma_4 \times 8.33}{18.33} \times 3A$

$= 1.36\,\sigma_4 A$

$= 1.26^k$

$= \sigma_6 \times 2A$

$= \dfrac{\sigma_4 \times 21.67}{18.33} \times 2A$

$= 2.36\,\sigma_4 A$

$= 2.19^k$

$\Sigma M_e = 0$ (+) $3 \times 15 + 6 \times 5 - \sigma_4 A \times 40$

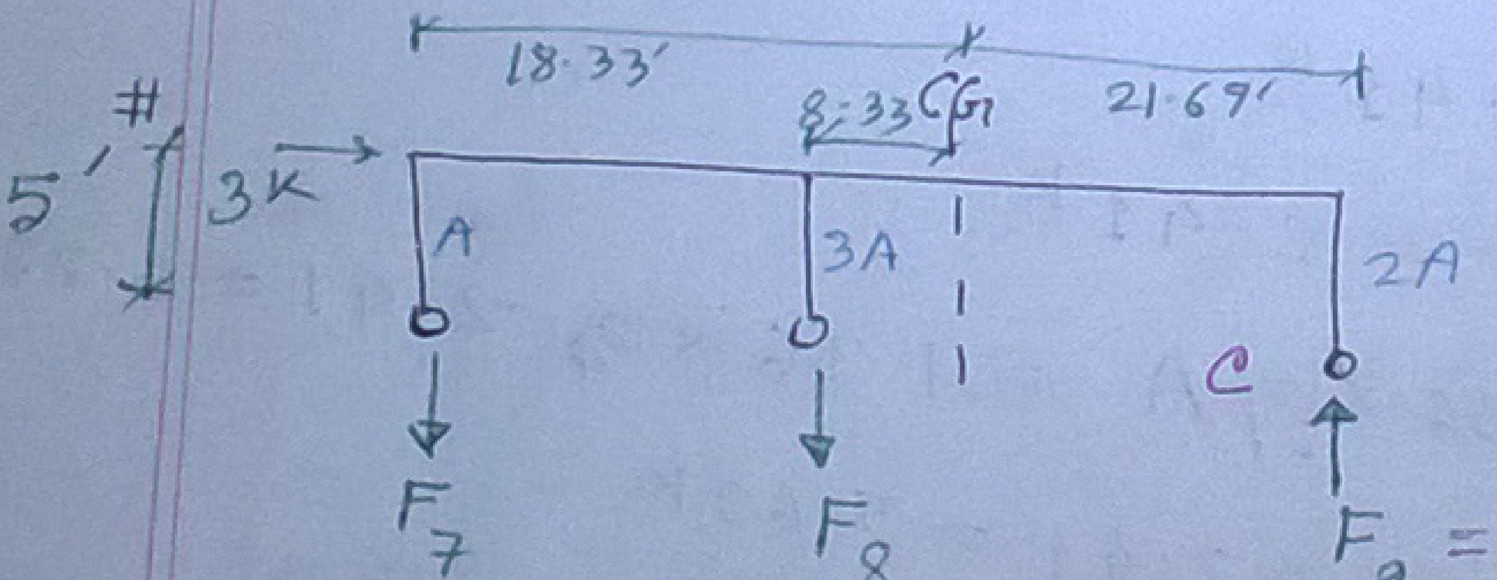
$- 1.36\,\sigma_4 A \times 30' = 0$

or, $\sigma_A A = 0.93^k$

$\therefore$ $F_4 = \sigma_A A = 0.93^k$;

$F_5 = 1.36 \times 0.93 = 1.26^k$

$F_6 = 2.36 \times 0.93 = 2.19^k$



$F_9 = \sigma_9 \times 2A$

$= \dfrac{\sigma_7 \times 21.67}{18.33}\, 2A$

$= 2.36\,\sigma_7 A$

$= 0.44^k$

$F_7 = \sigma_7 \times A$

$= 0.19^k$

$F_8 = \sigma_8 \times 3A$

$= \dfrac{\sigma_7 \times 18.33}{18.33}\, 3A$

$= 1.36\,\sigma_7 A$

$= 0.25^k$

$\Sigma M_c = 0$

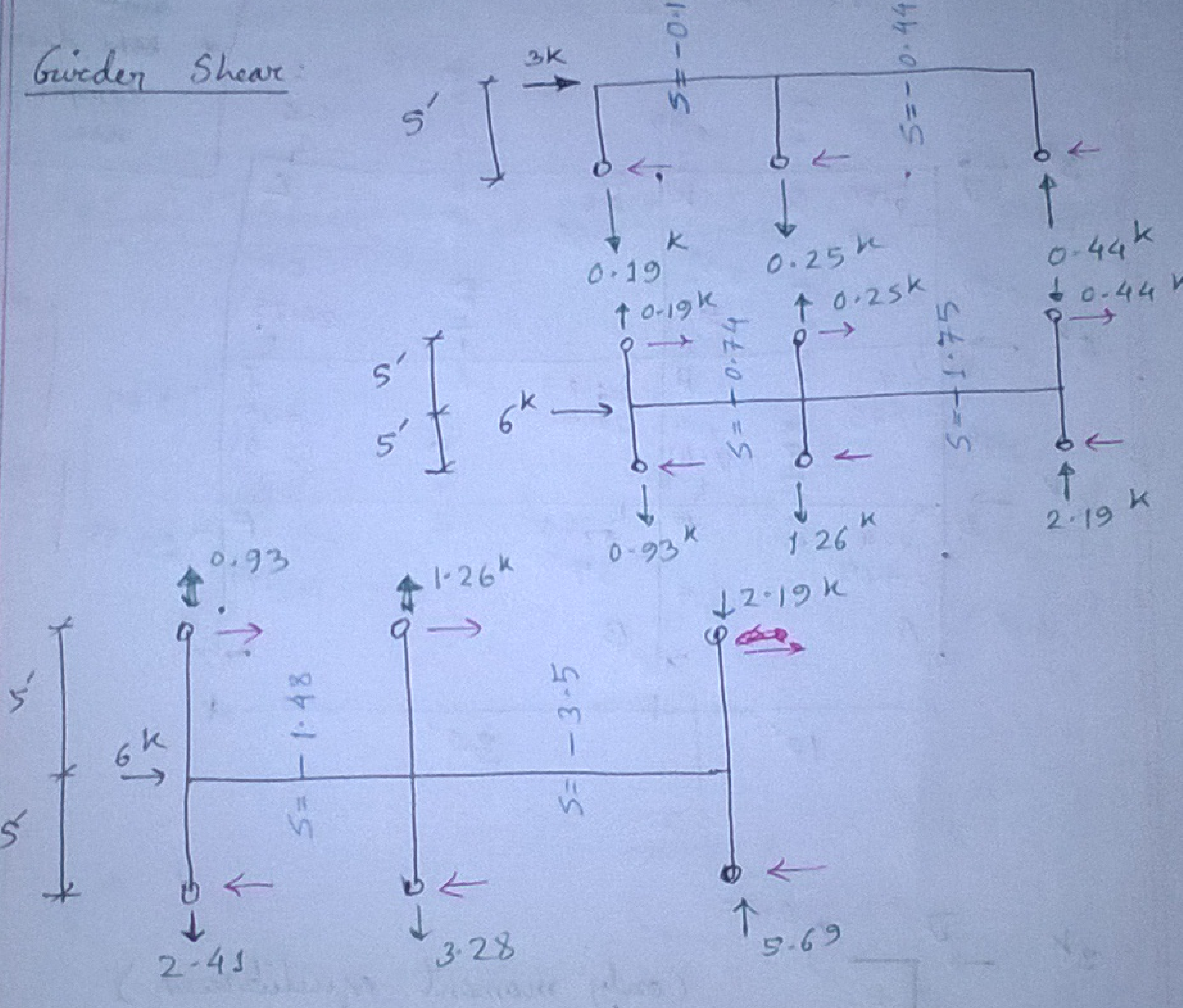
$3 \times 5 - \sigma_7 A \times 40 - 1.36\, \sigma_7 A \times 30 = 0$

$\therefore\ \sigma_7 A = 0.19^k$

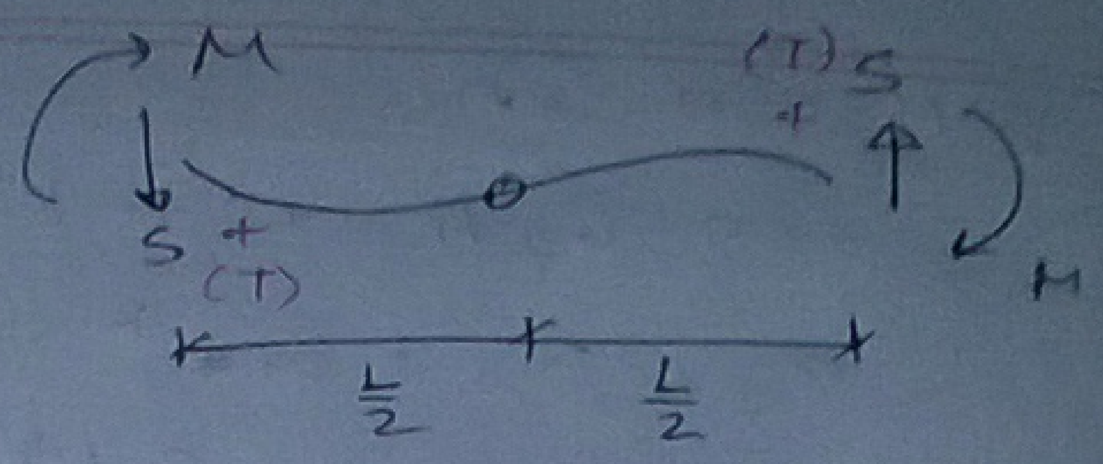
$F_7 = \sigma_7 A = 0.19^k\ ;\quad F_8 = 1.36 \times 0.19 = 0.25^k$

$F_9 = 2.36\, \sigma_7 A = 2.36 \times 0.19 = 0.44^k$

Girder Shear



Girder Moment:

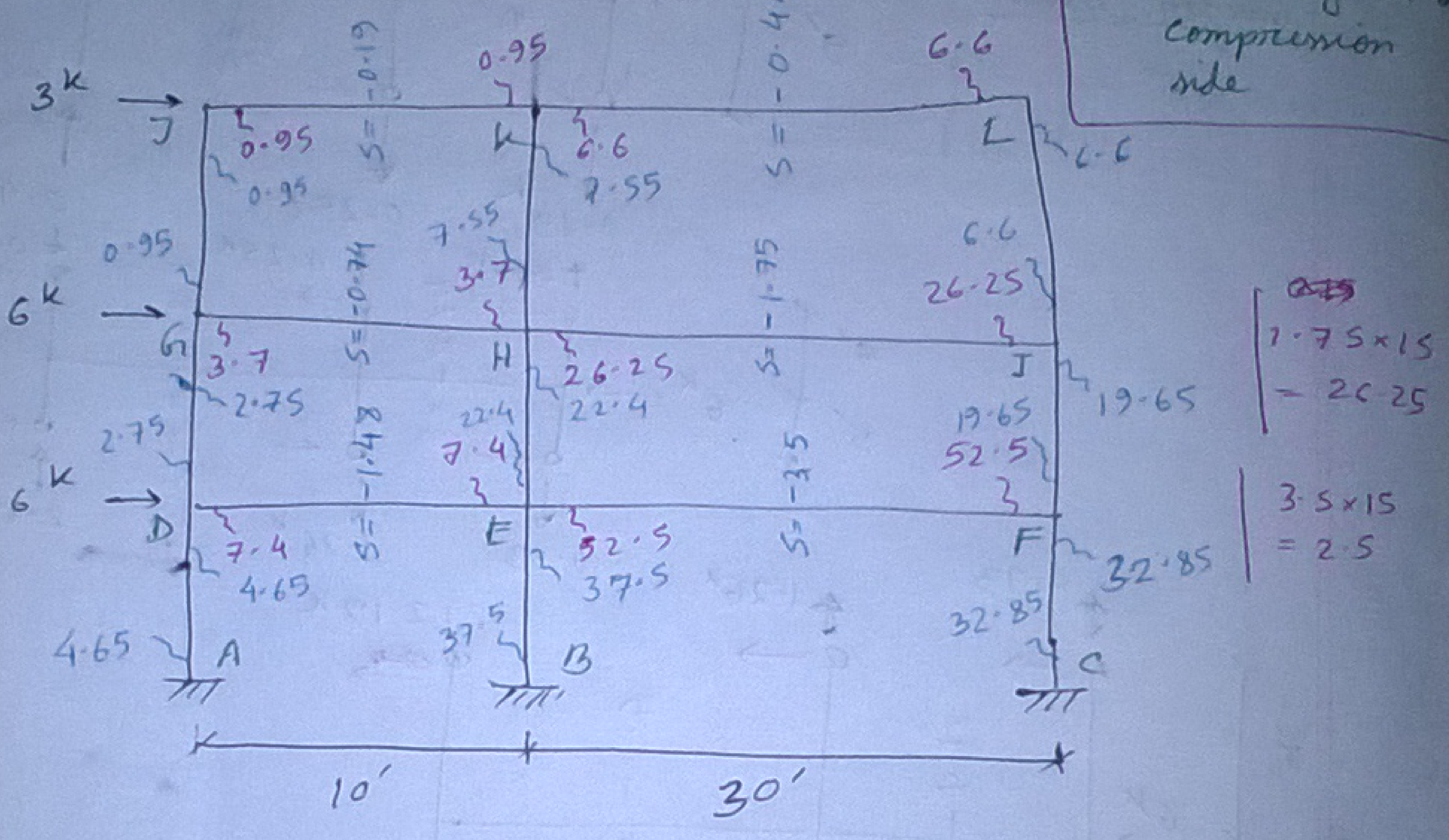


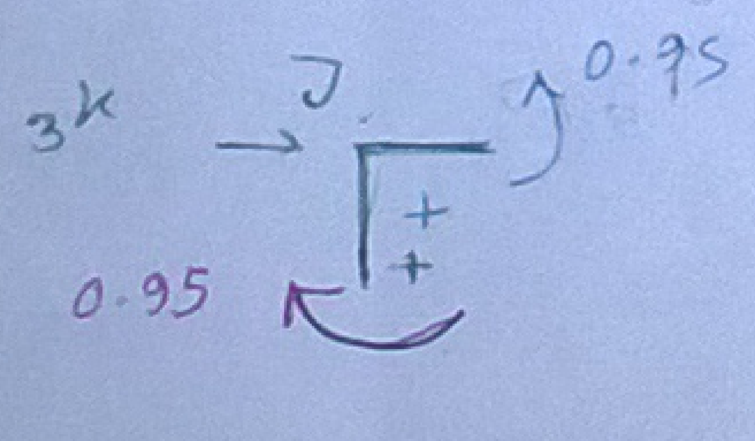
$$2M = S \times L$$

$$\Rightarrow S = \frac{2M}{L}$$

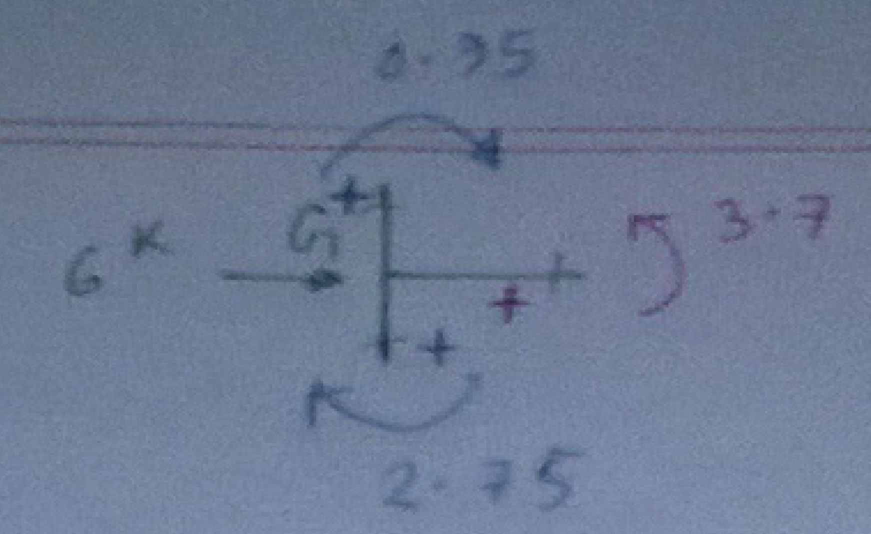
or, $M = S \times \frac{L}{2}$

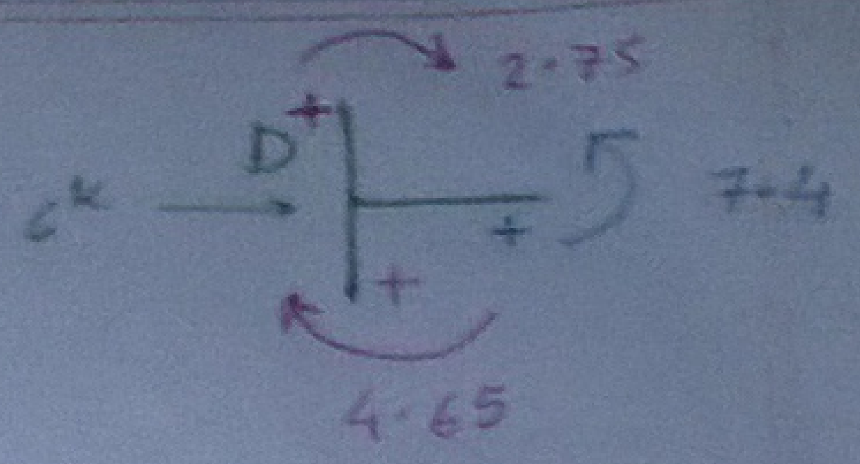
* Figures / Values of BM on Tension side
* BM diagram on compression side

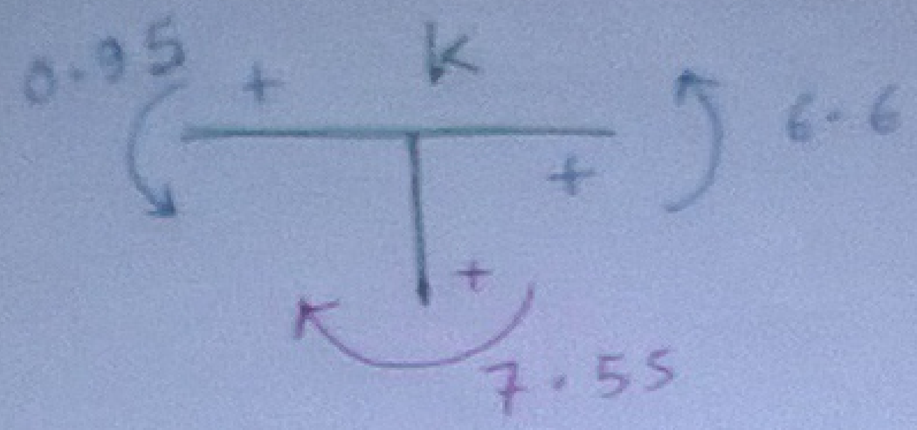




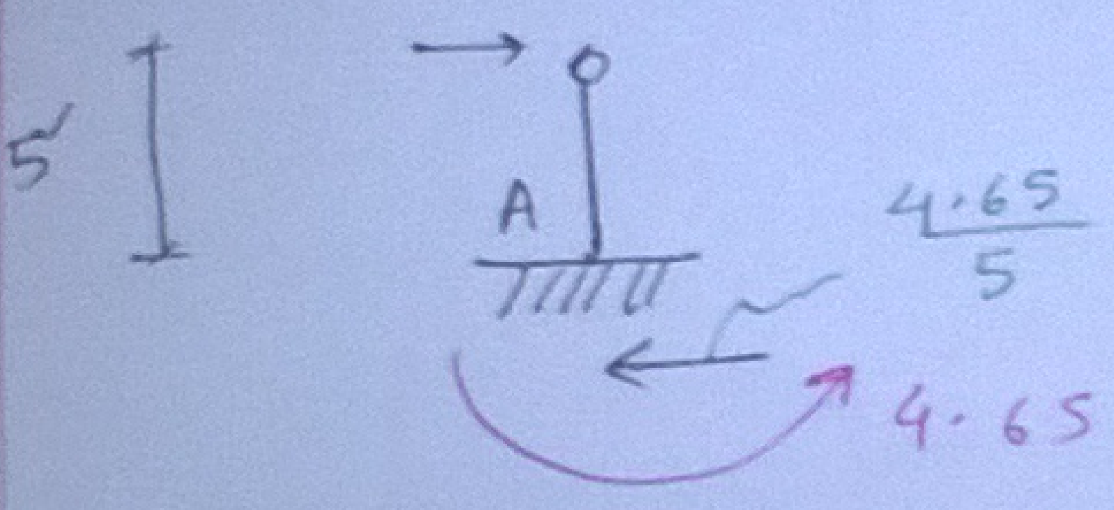
(only moment equilibrium)



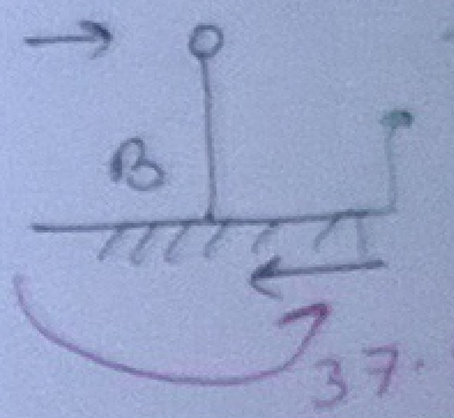




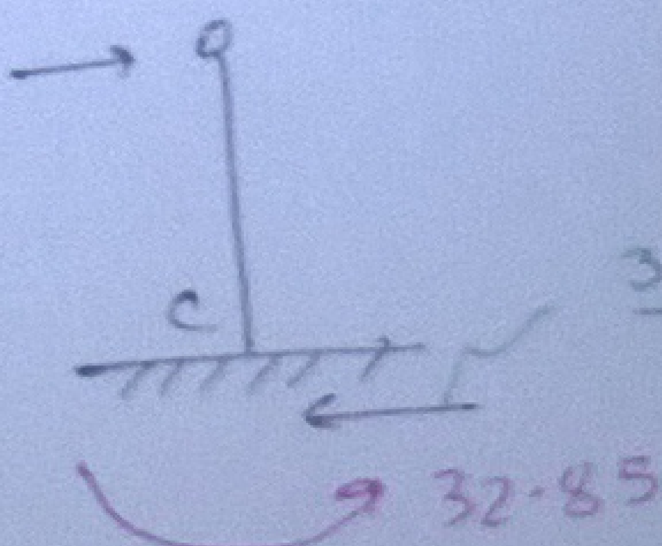
Column Shear: not important ; so skipped.



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



$\frac{37.5}{5} = 7.5^K$



$\frac{32.85}{5} = 6.57^K$

Horizontal Equilibrium :

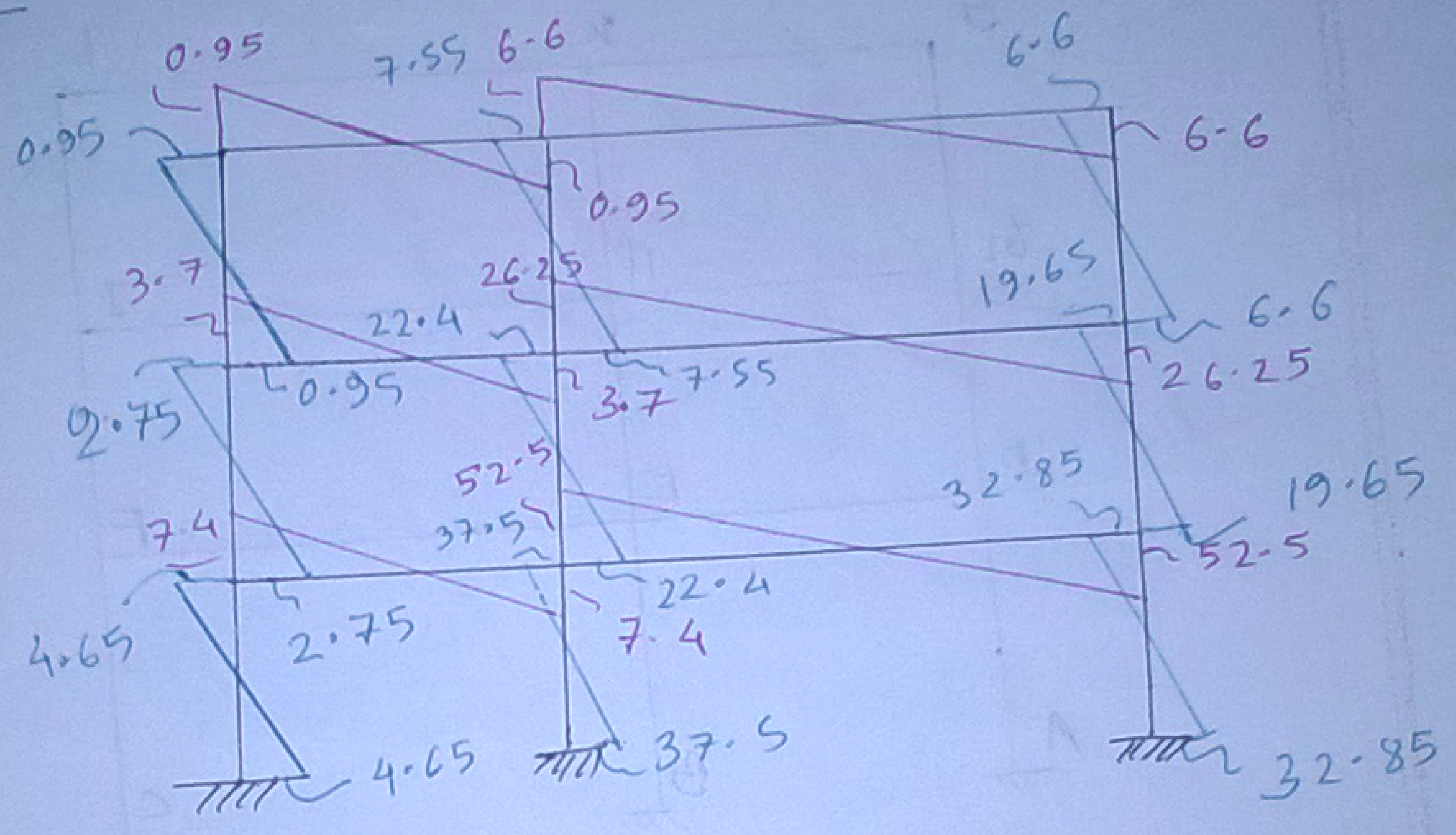
$3 + 6 + 6 - 0.93 - 7.5 - 6.57 = 0$

(This is a check)

23.05.15

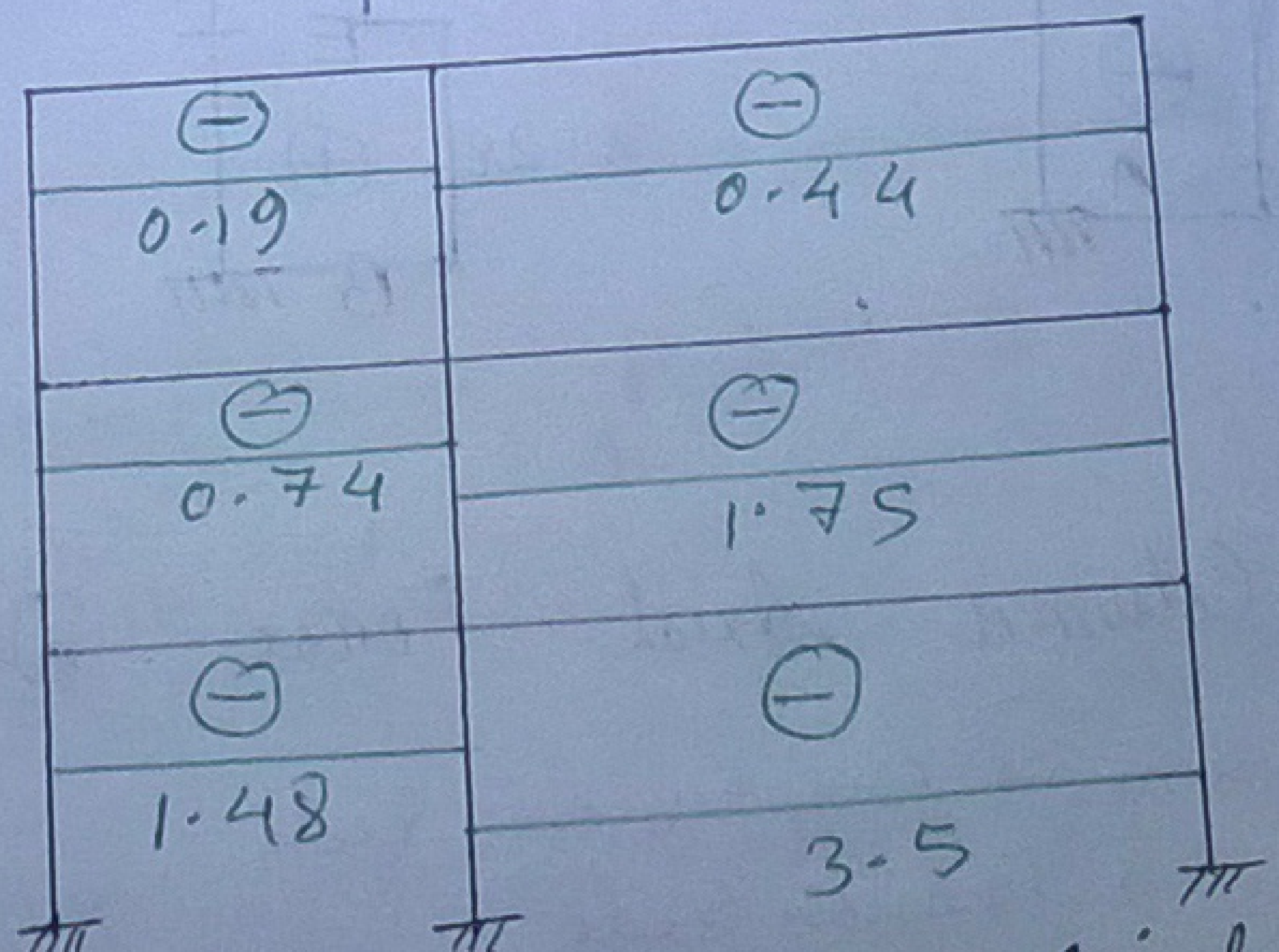
Taufiq Sir

Bending Moment Diagrams for both Girders & Columns:



BMD

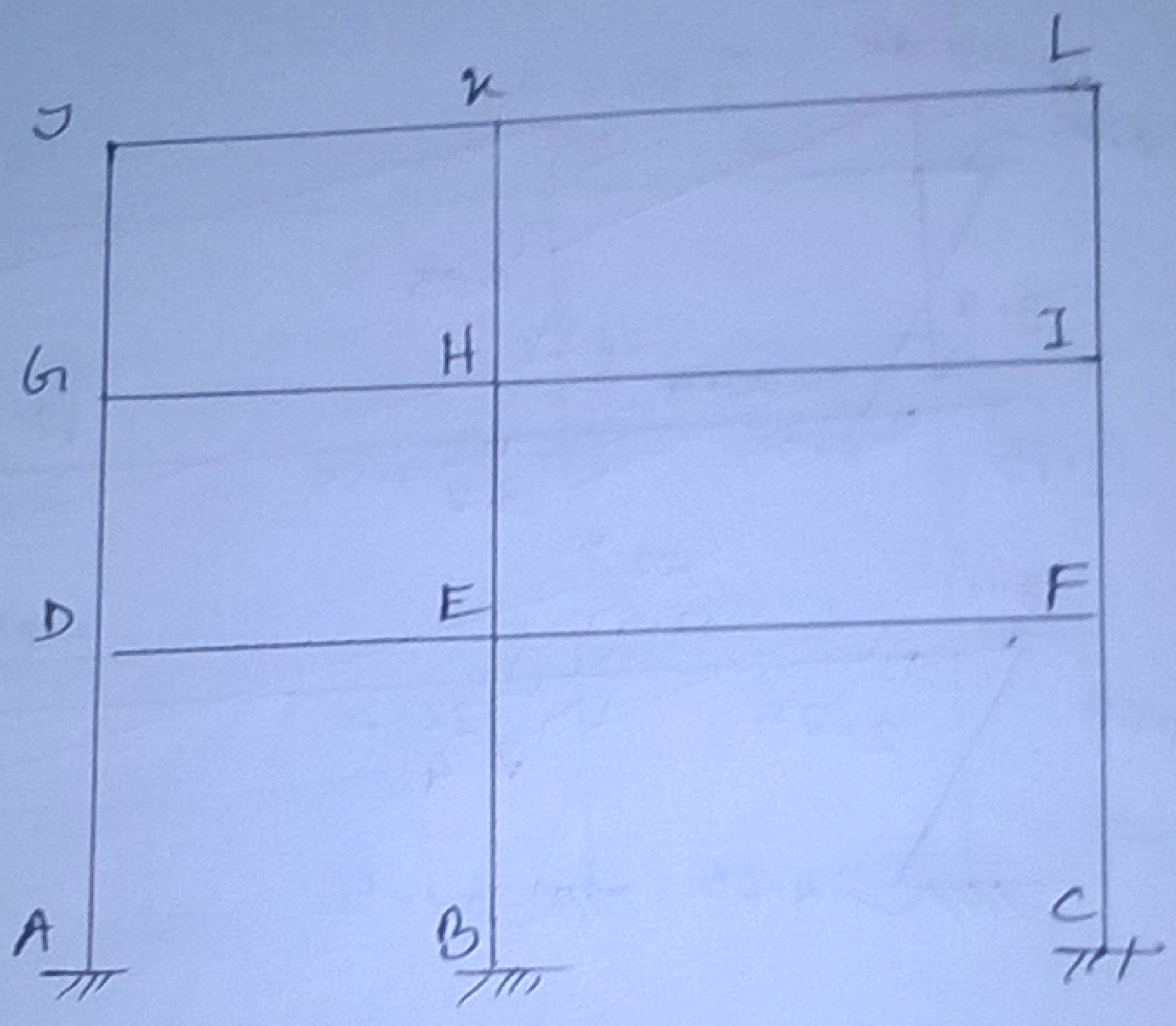
Shear Force in Girders:

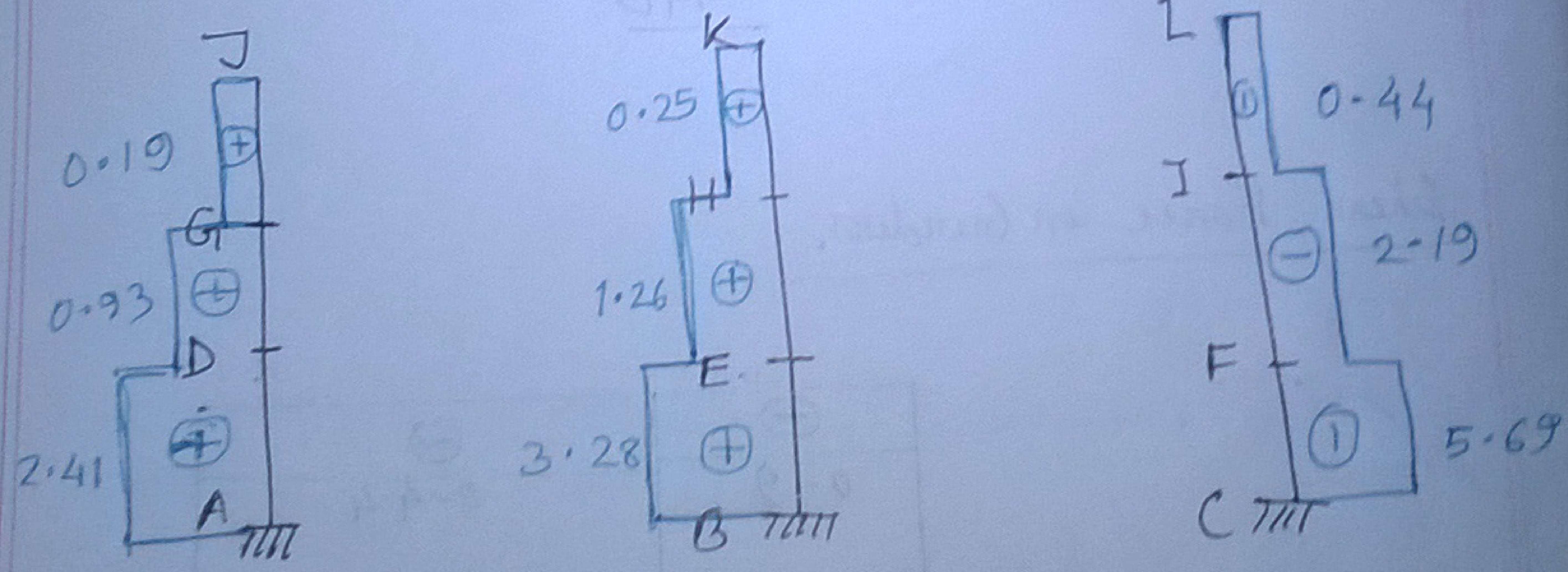


SFD in Girders

VVI

Column Axial Force



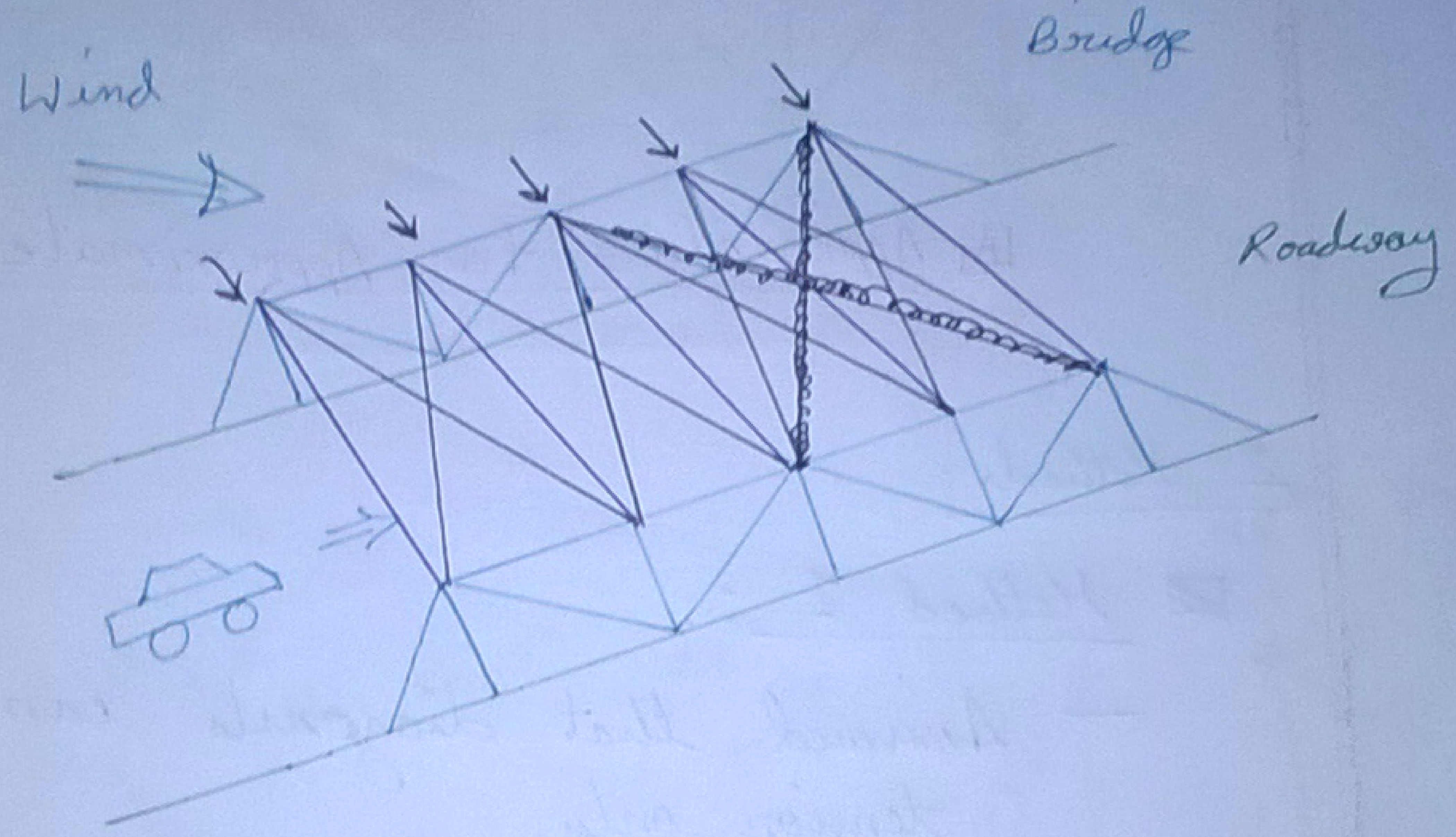


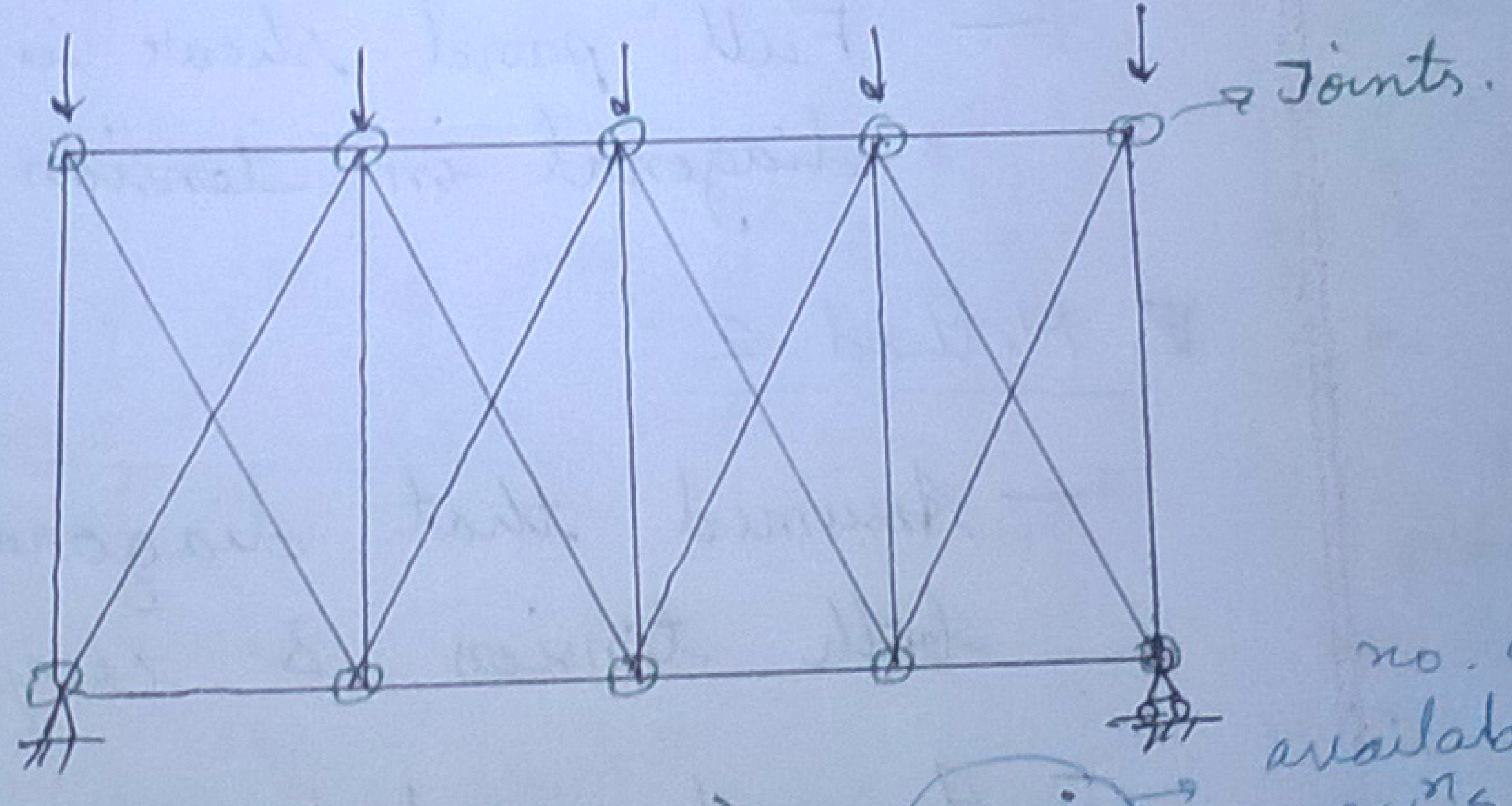
Column Axial Force Diagram

Diagram → Compression Side

Value → Tension Side

Approximate Analysis of Cross (X) Based Trusses





A system of coplanar concurrent forces (2 eqⁿs – $\Sigma F_x = 0$, $\Sigma F_y = 0$)

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

no. of available eqⁿs

m: no. of members

r: no. of independent reaction components

j: no. of joints

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

(*) If truss formed with only $\Delta \rightarrow$ Statically Determinate Structure

Approaches for Approximate Analysis

2 Methods:

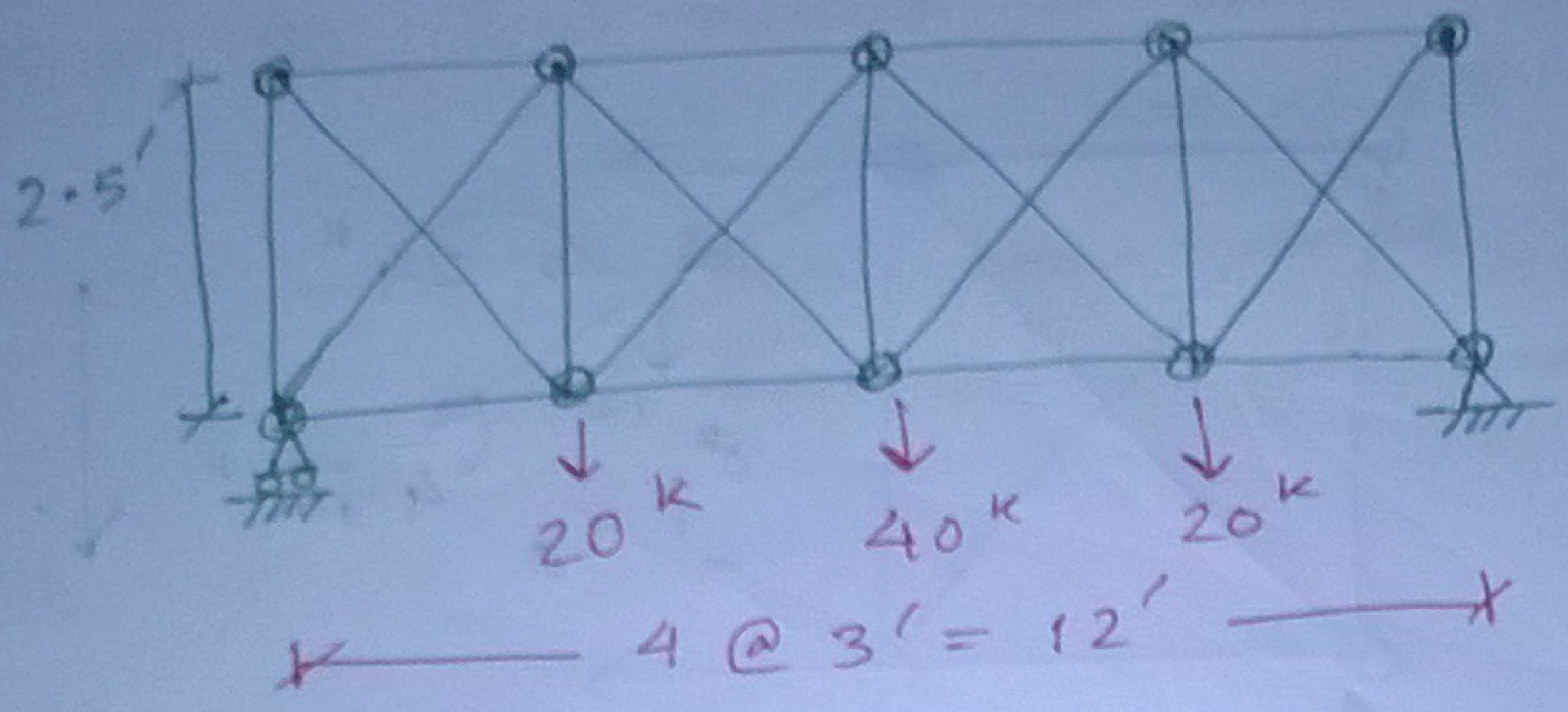
Method 1:

- Assumed that diagonals can carry tension only.
- Full panel shear is carried by diagonal in tension.

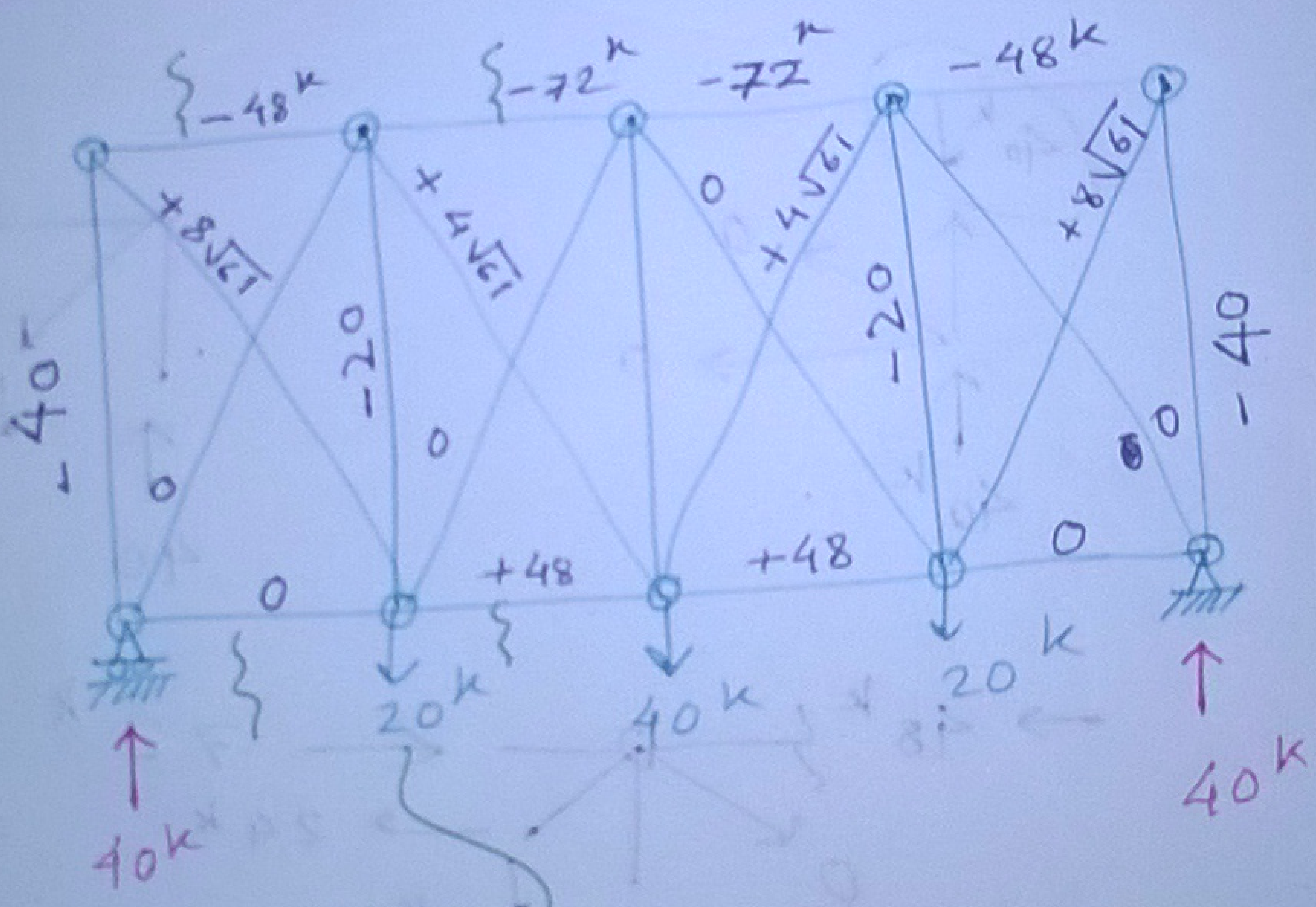
Method 2:

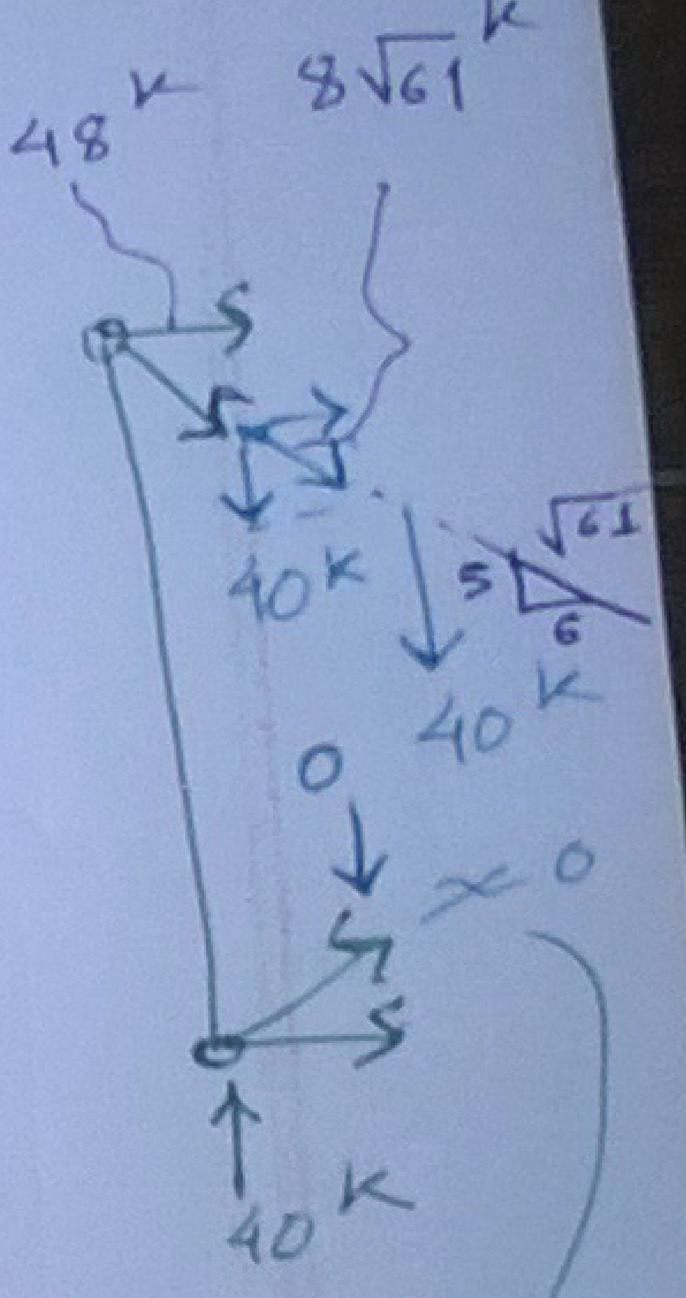
- Assumed that diagonals can carry both tension & compression.
- In each panel, shear is equally divided between the two diagonals.

Problem:



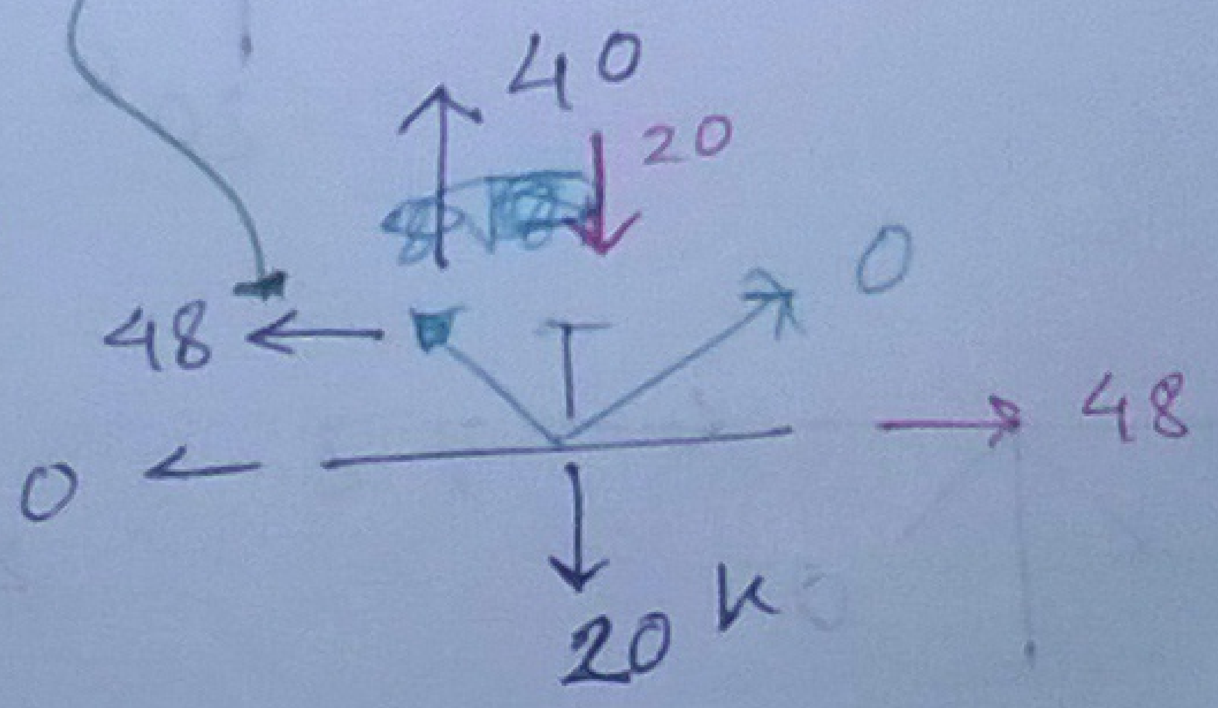
Method 1:

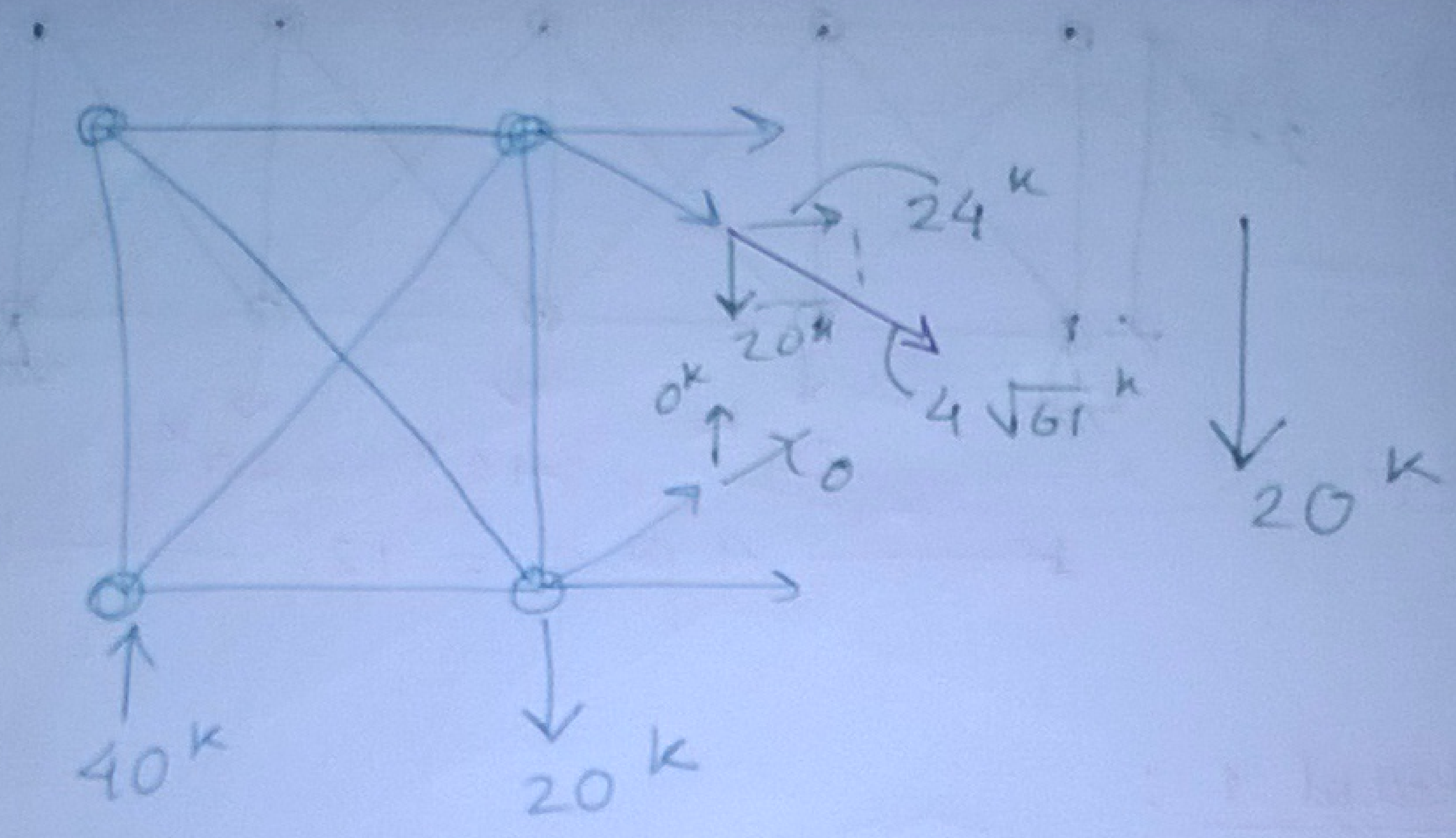




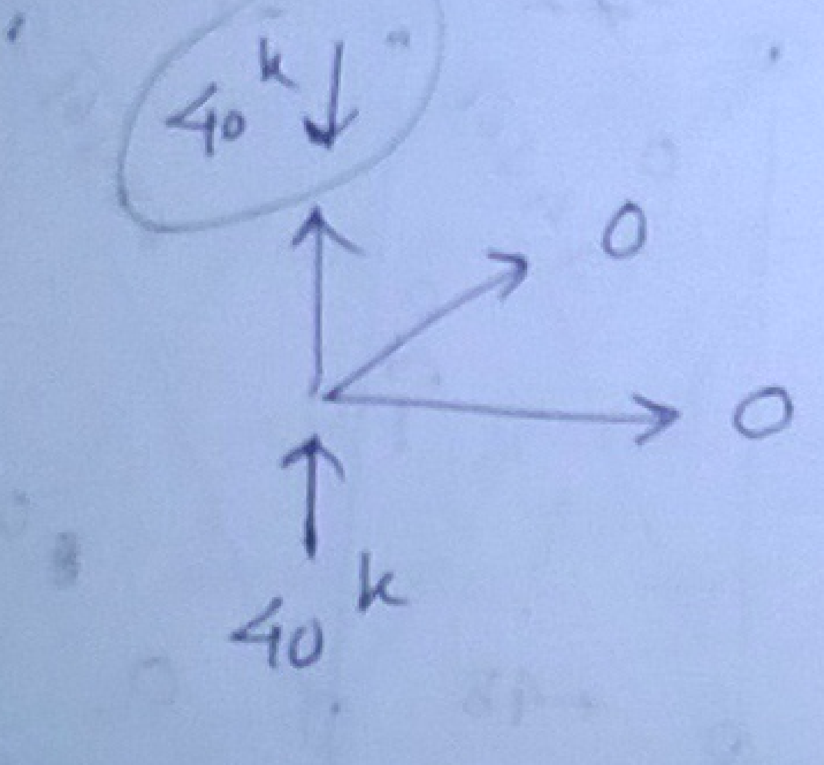
0 because can't carry compression

*) Symmetric Problem

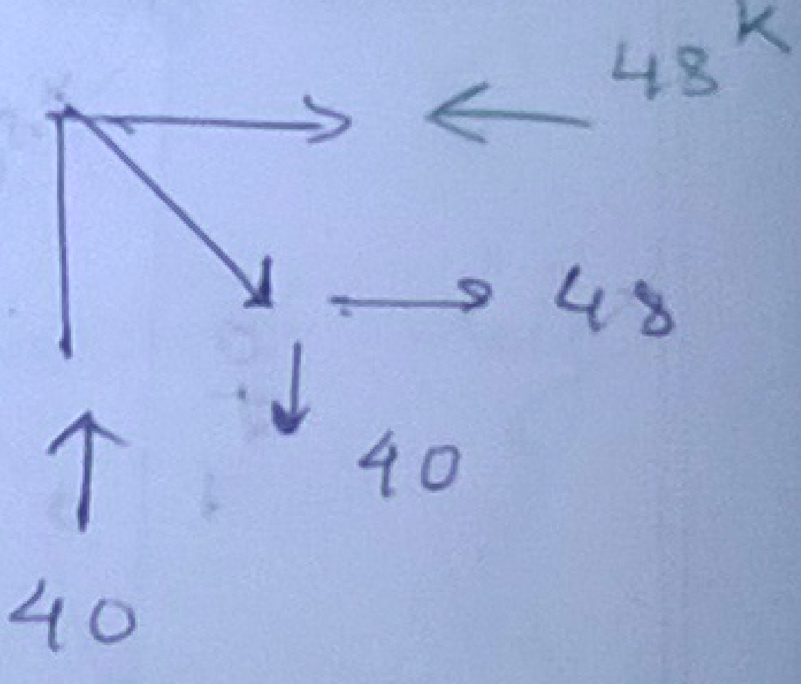




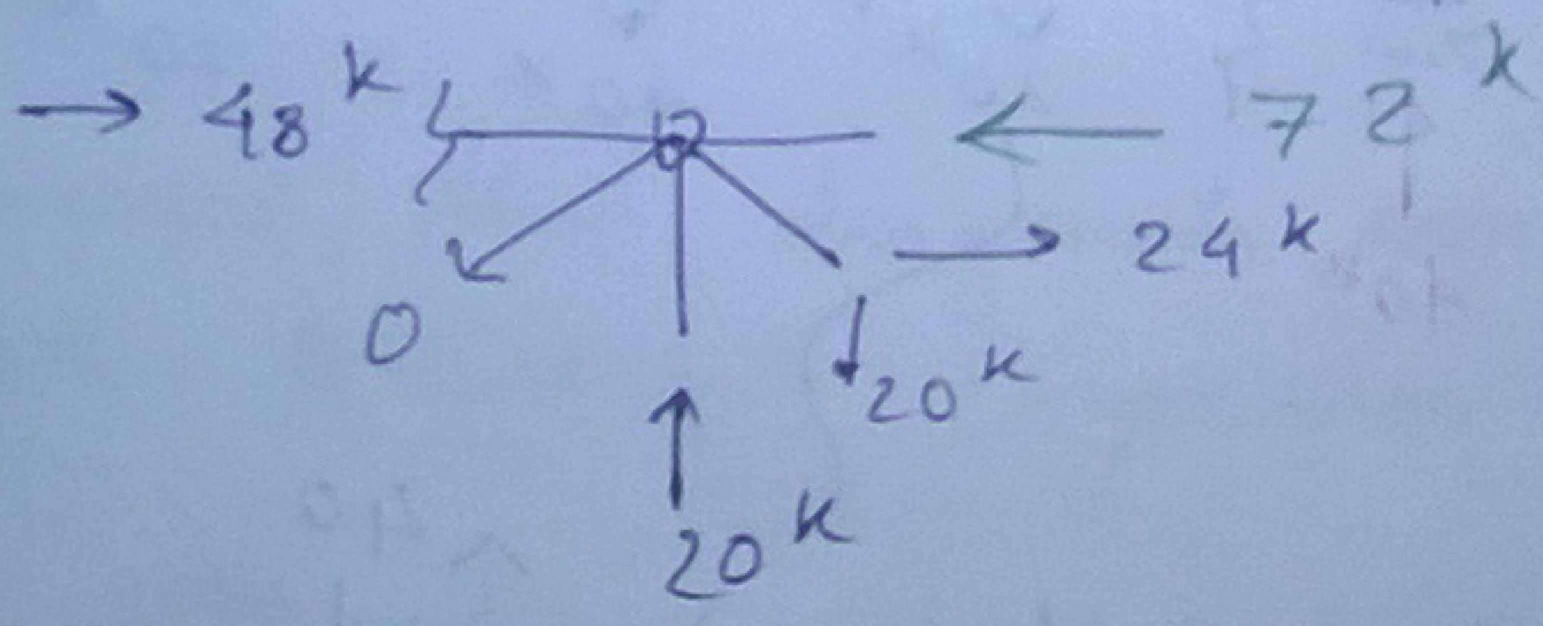
24^k
20^k
4√61 ^k
0^k
x_0
20^k
40^k
20^k



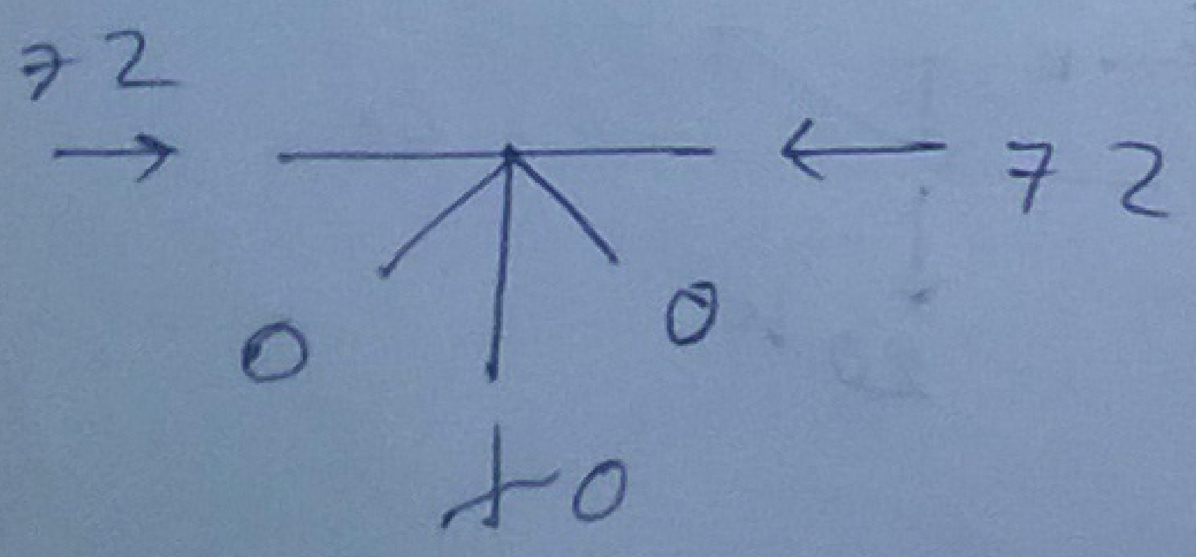
40^k
0
0
40^k



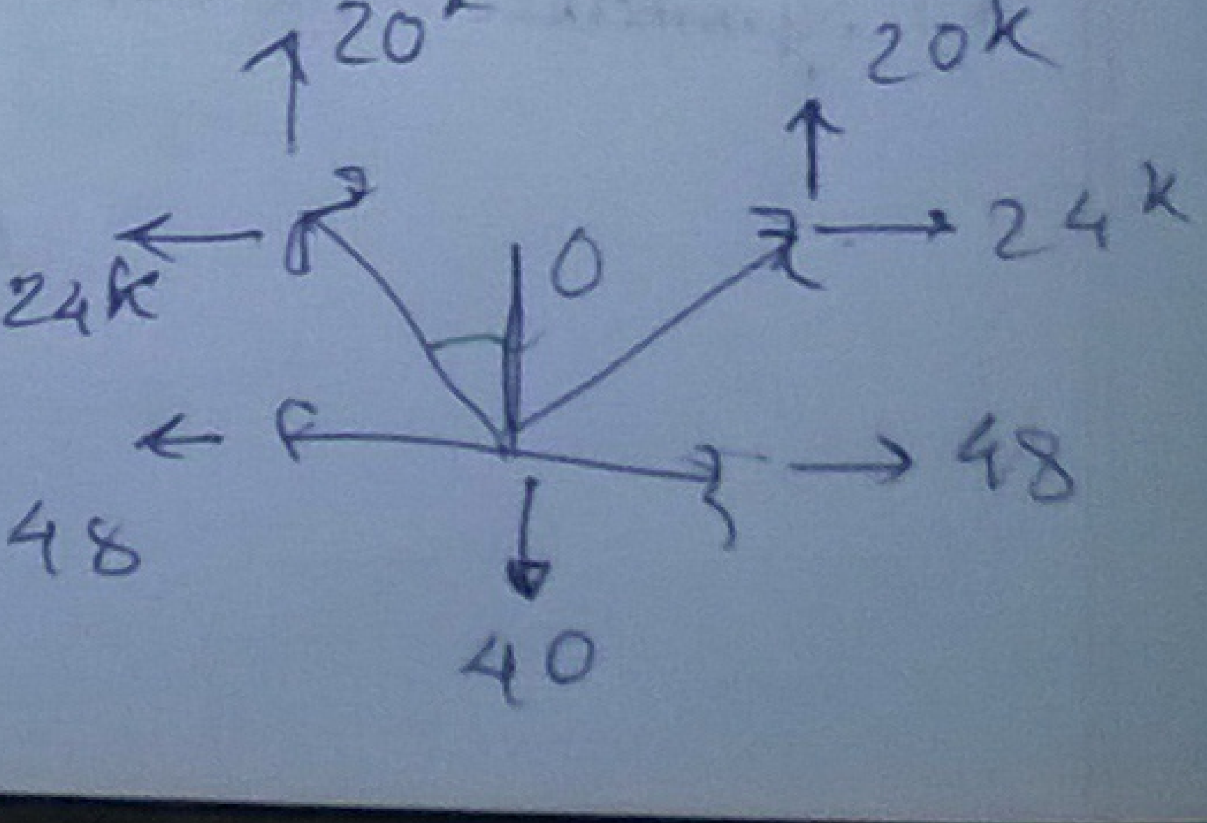
48^k
48
40
40



48^k
72^k
24^k
0
20^k
20^k



72
72
0
0
40



20^k
20k
24k
0
24^k
48
48
40

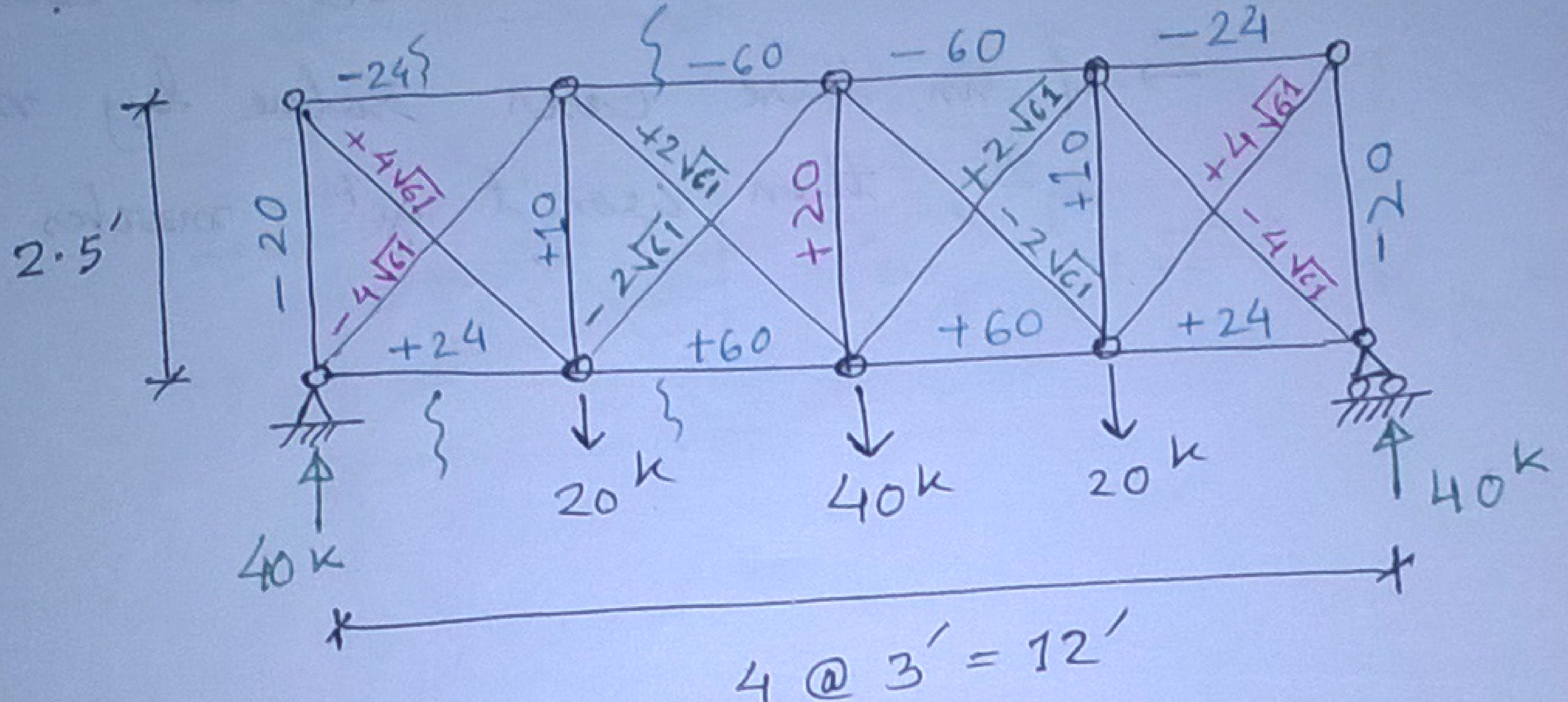
There is also 3rd method → when panels at middle are ~~abstract~~ absent.

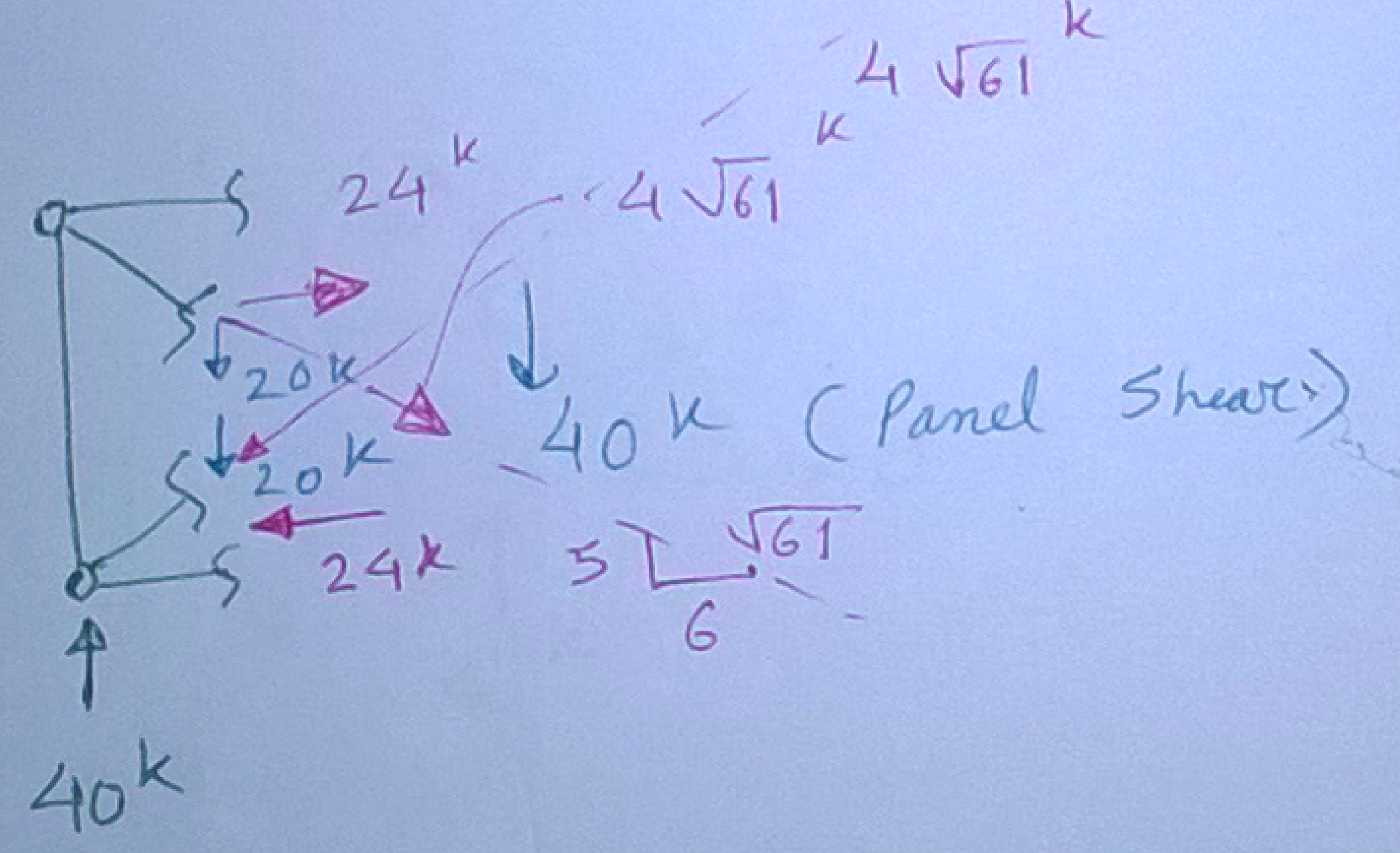
→ if in these cases solve by method 1 & 2, then won't get marks.

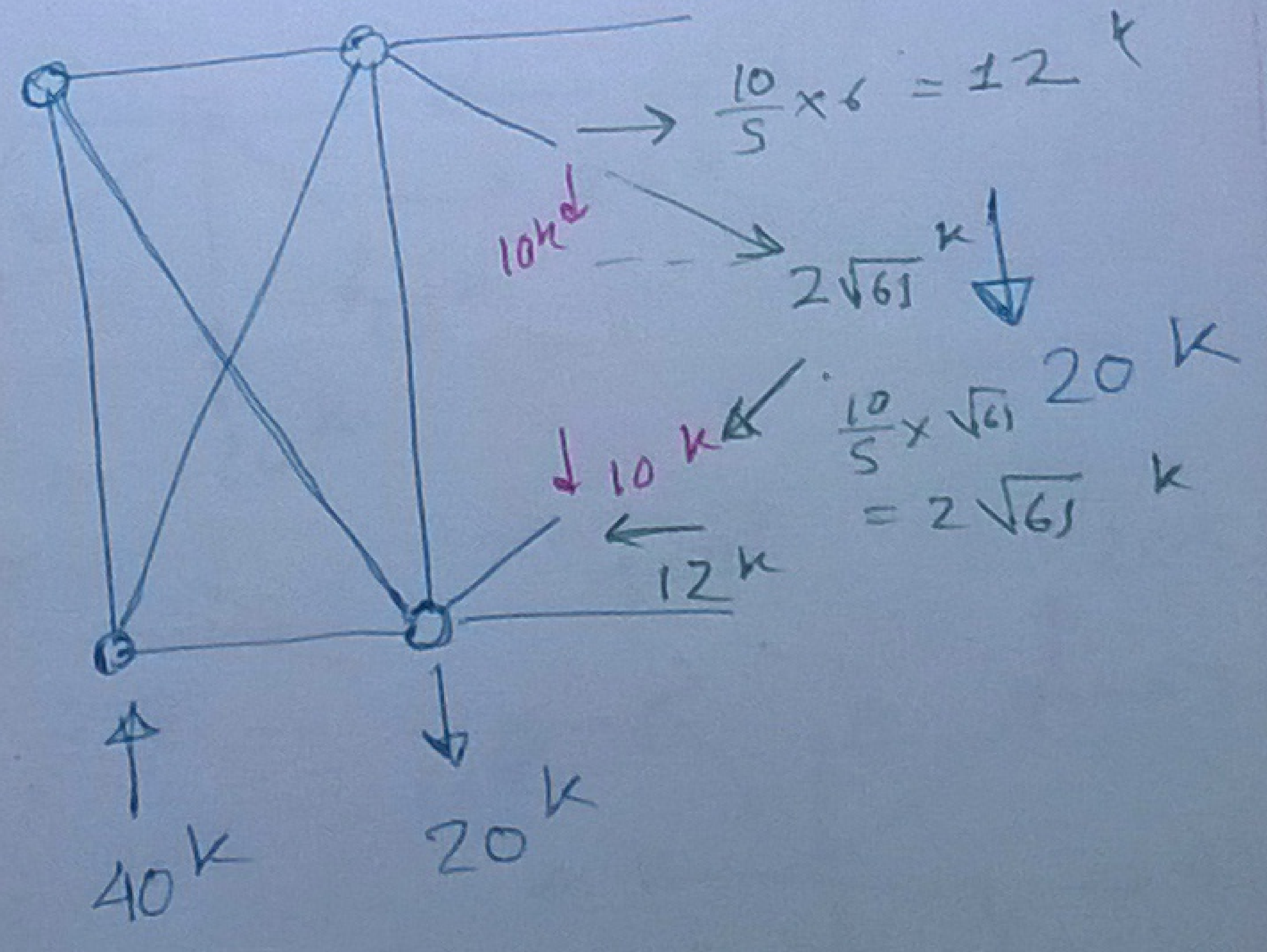
.05.15

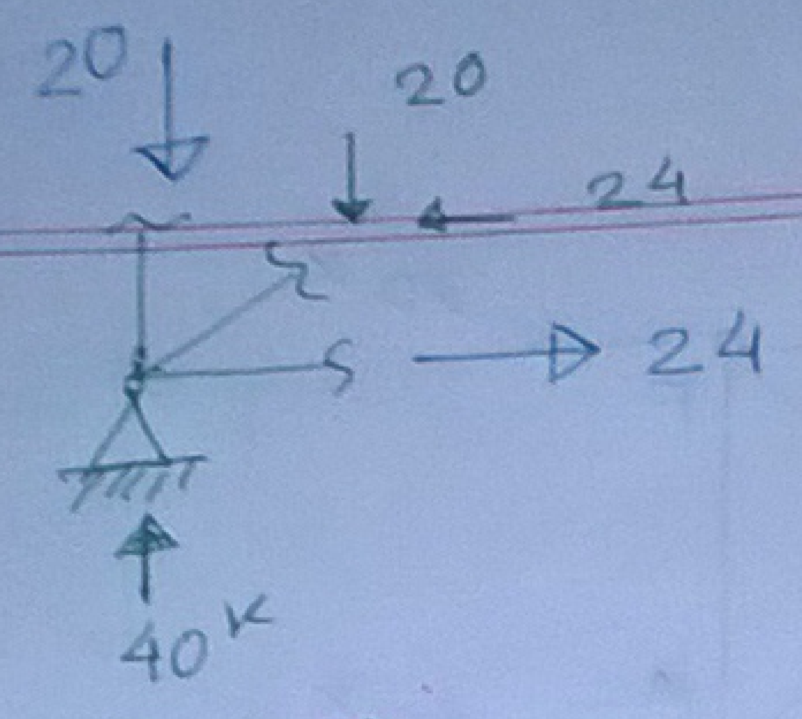
Taufiq Sir

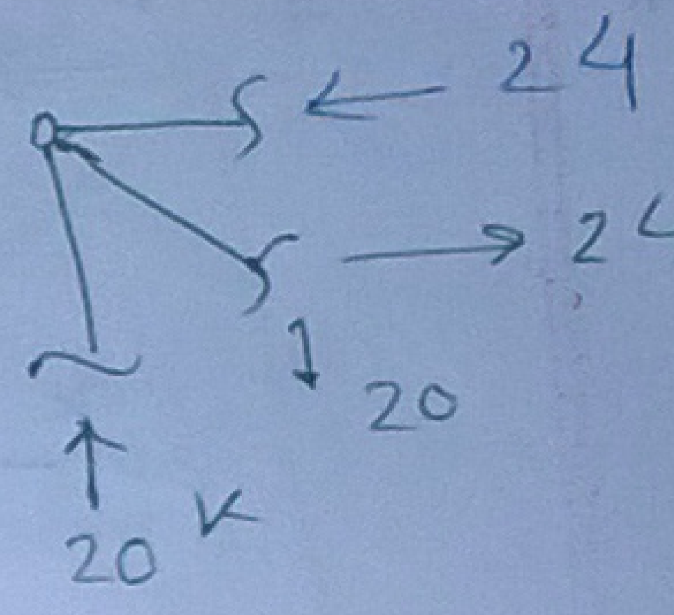
Method - 2

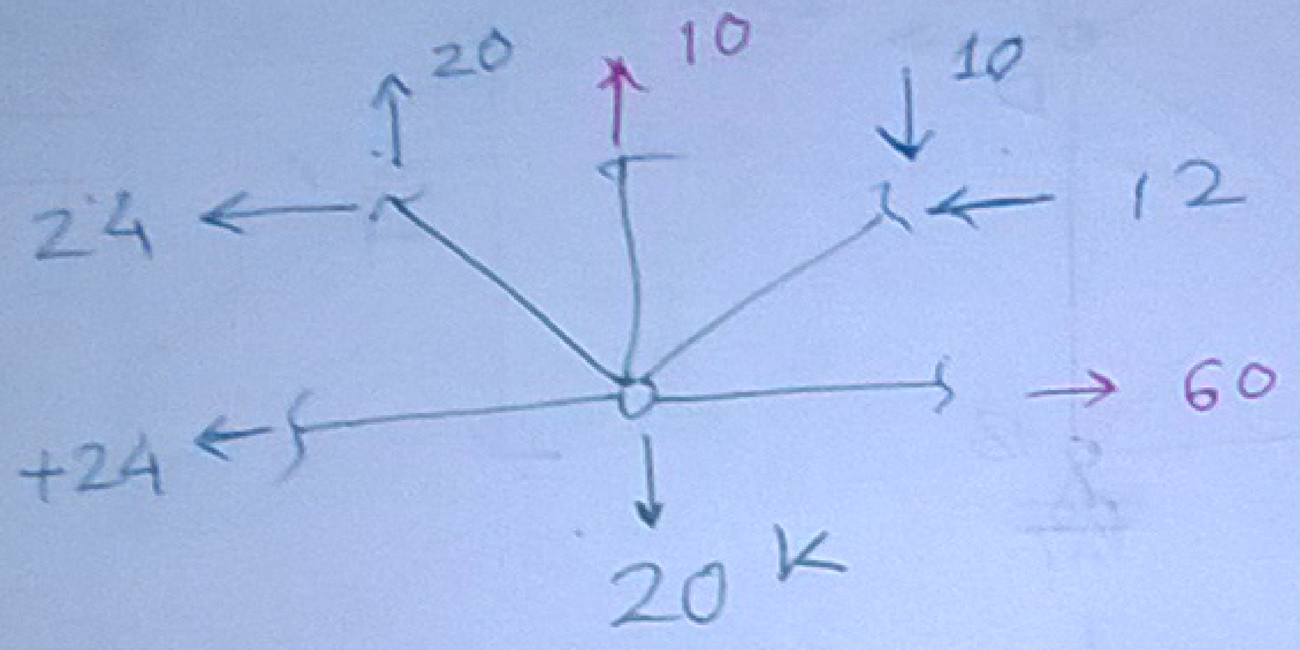


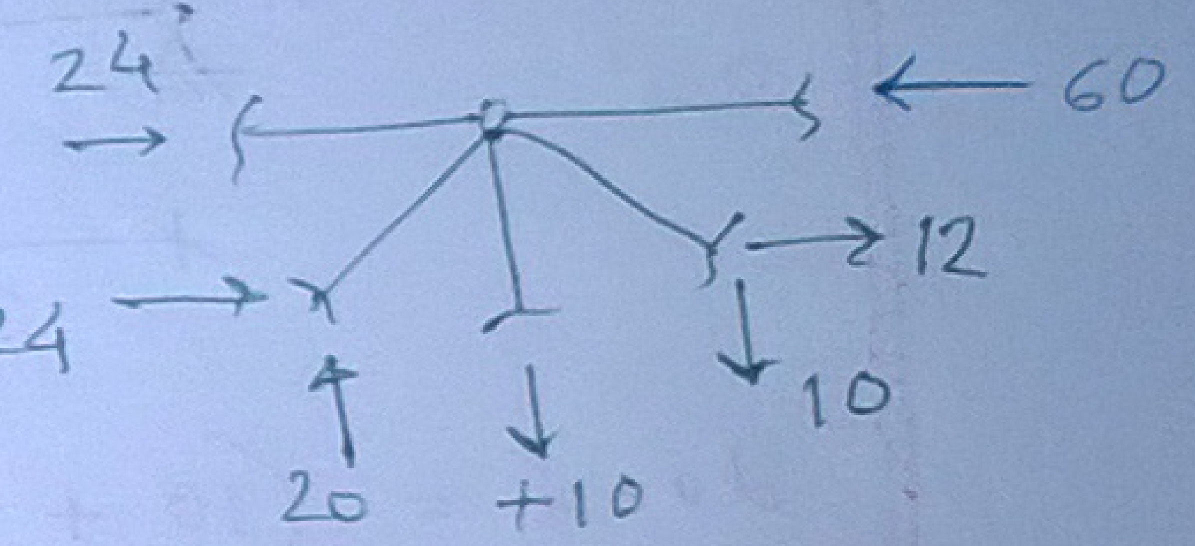


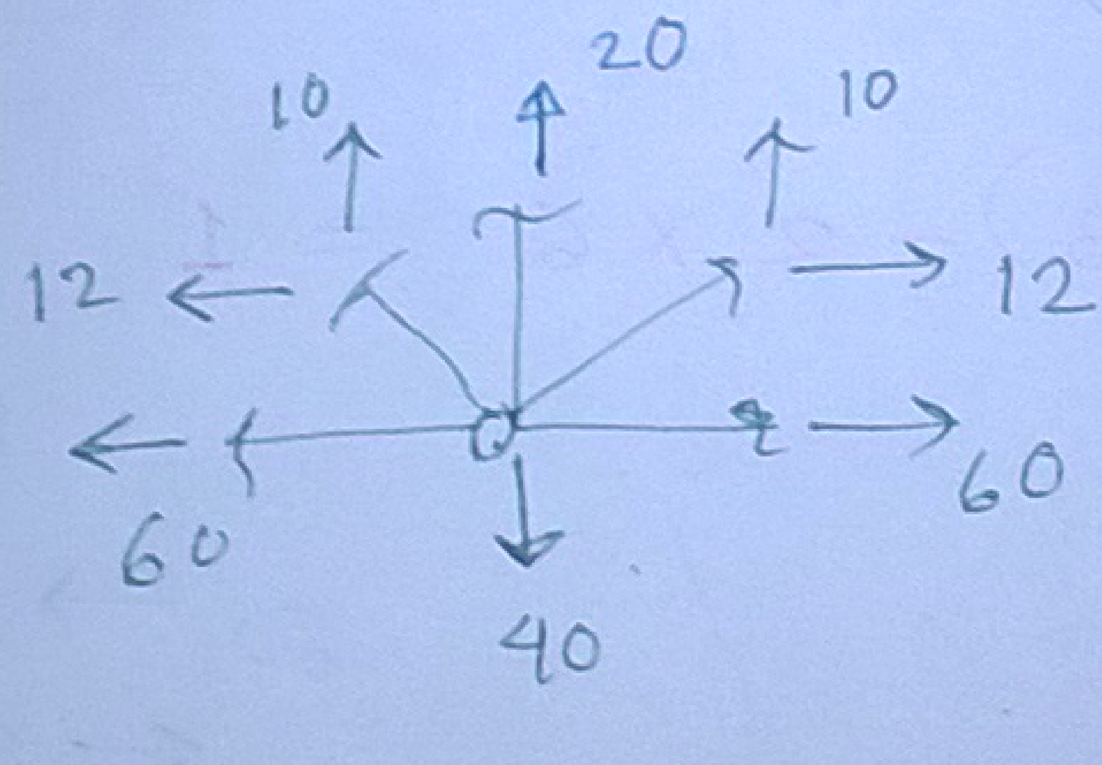




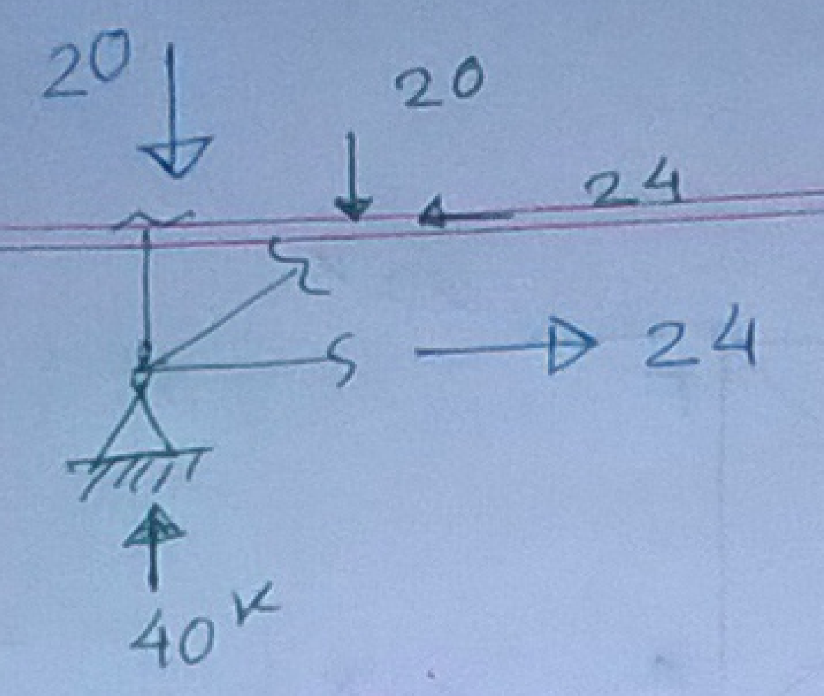


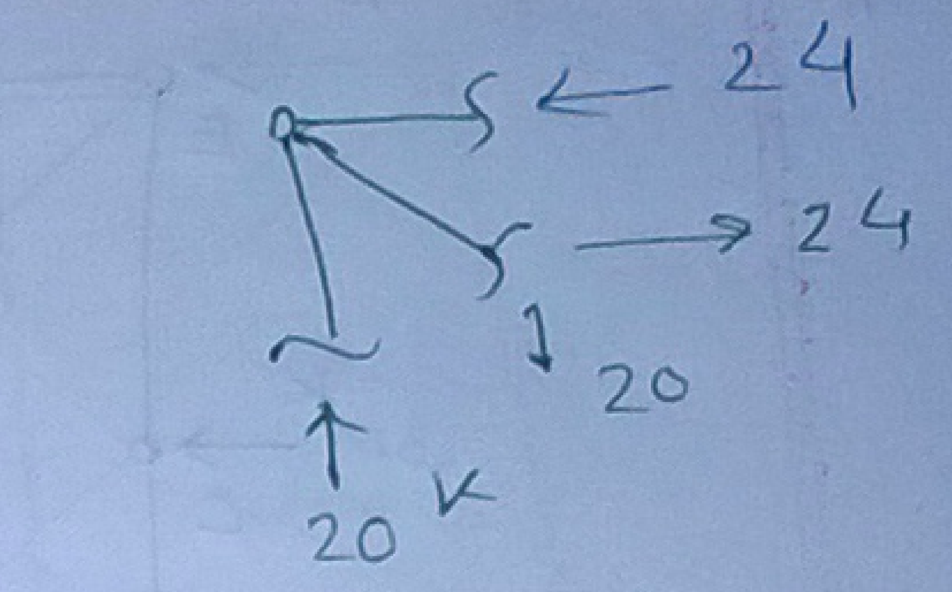


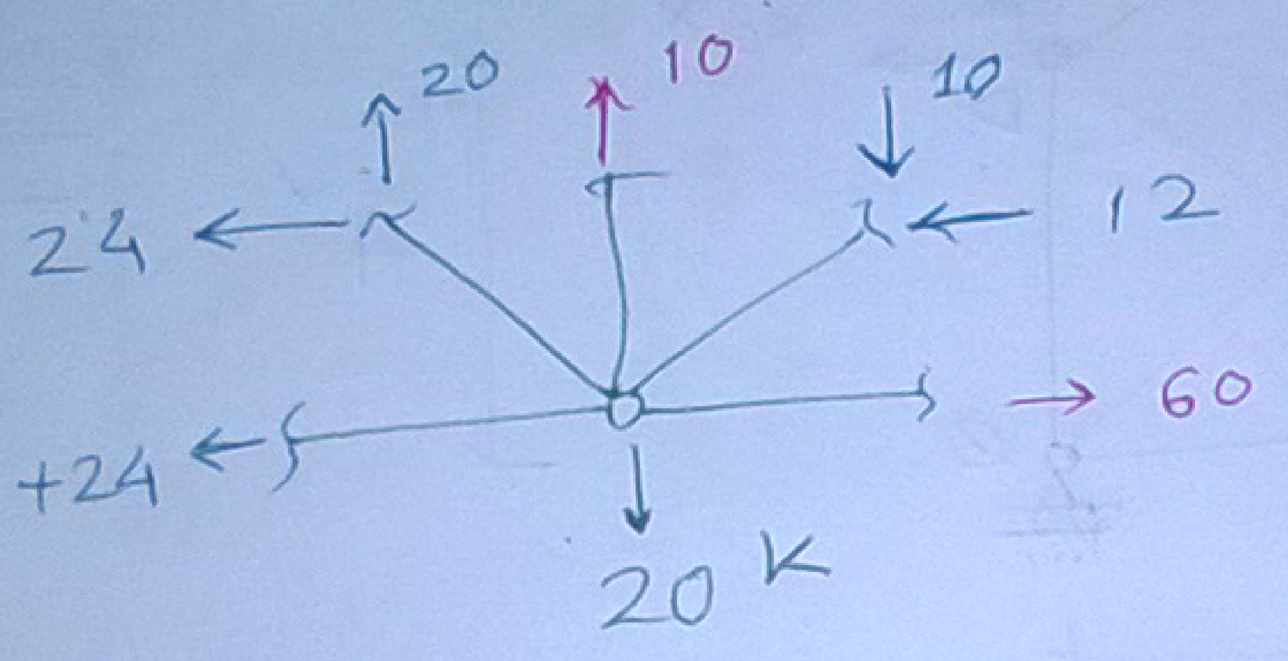


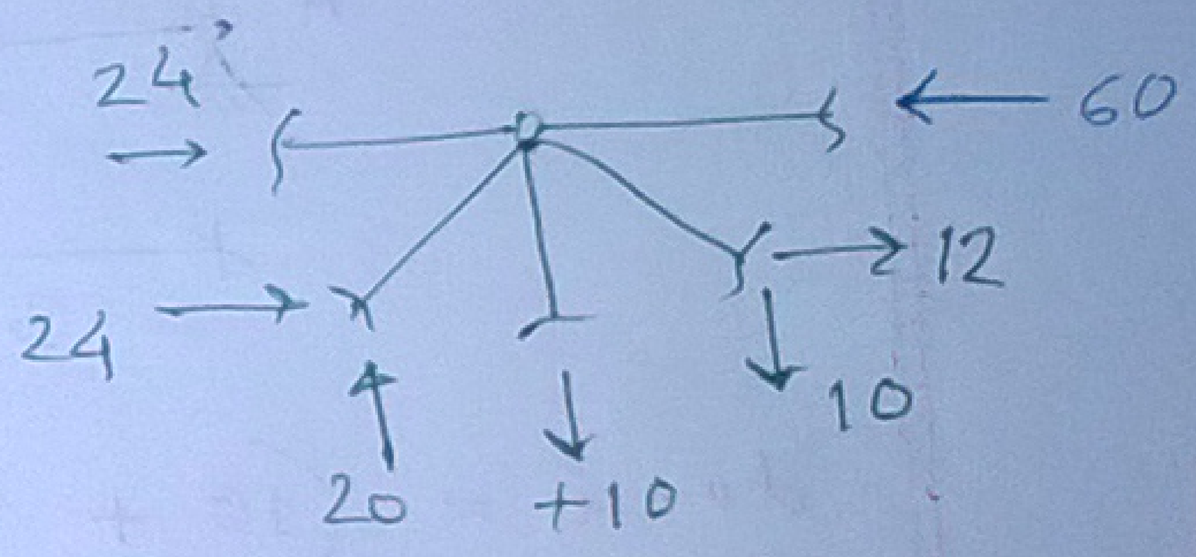


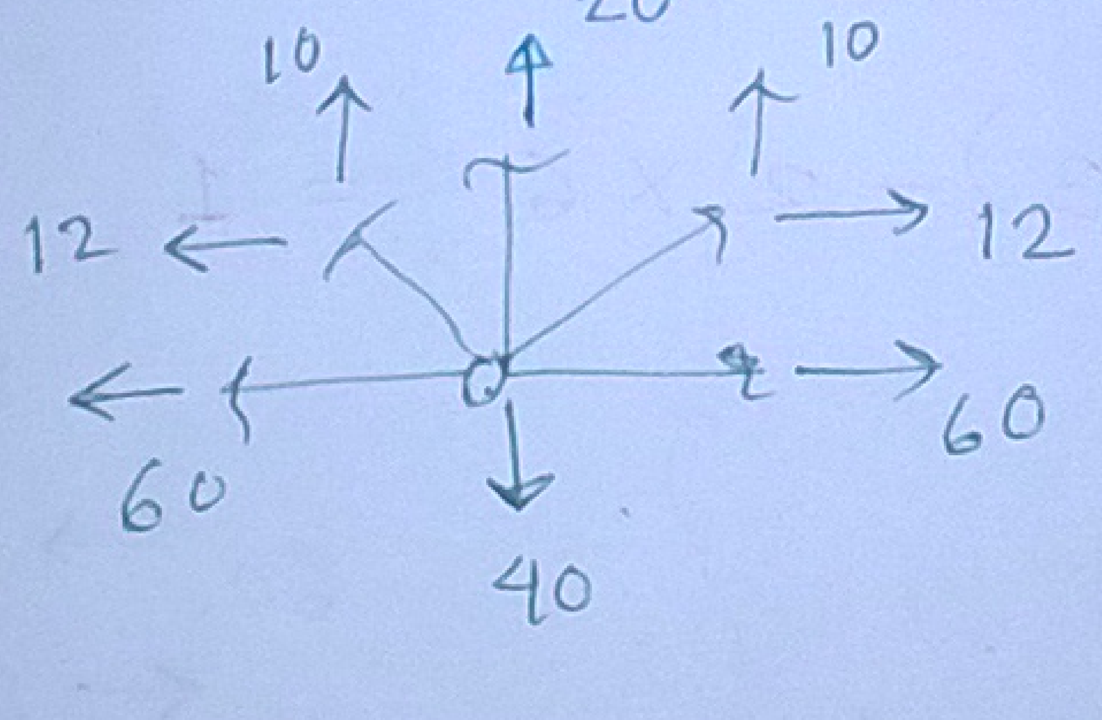
☑ If middle members are absent, Method 1 & Method 2 are not applicable.





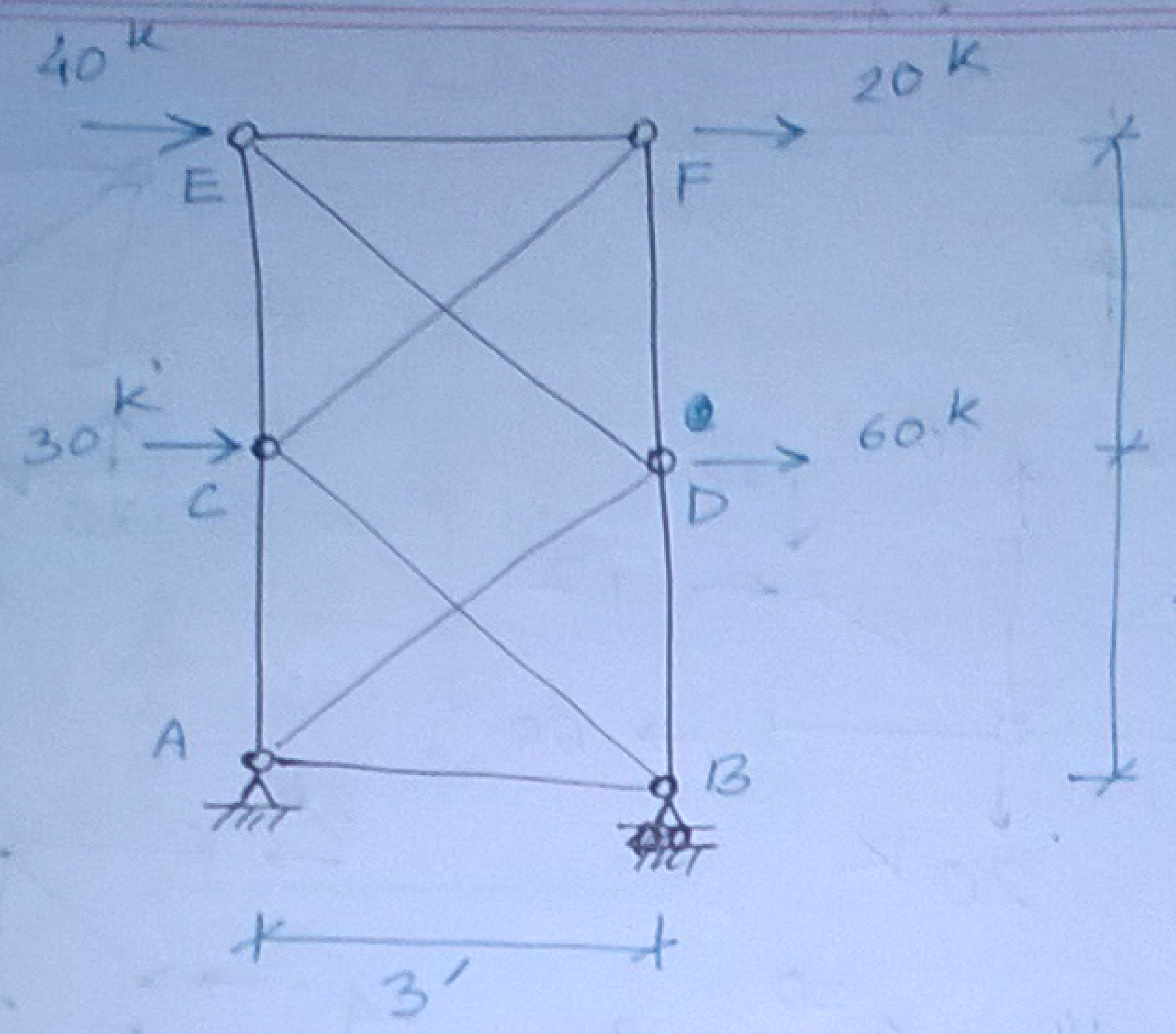






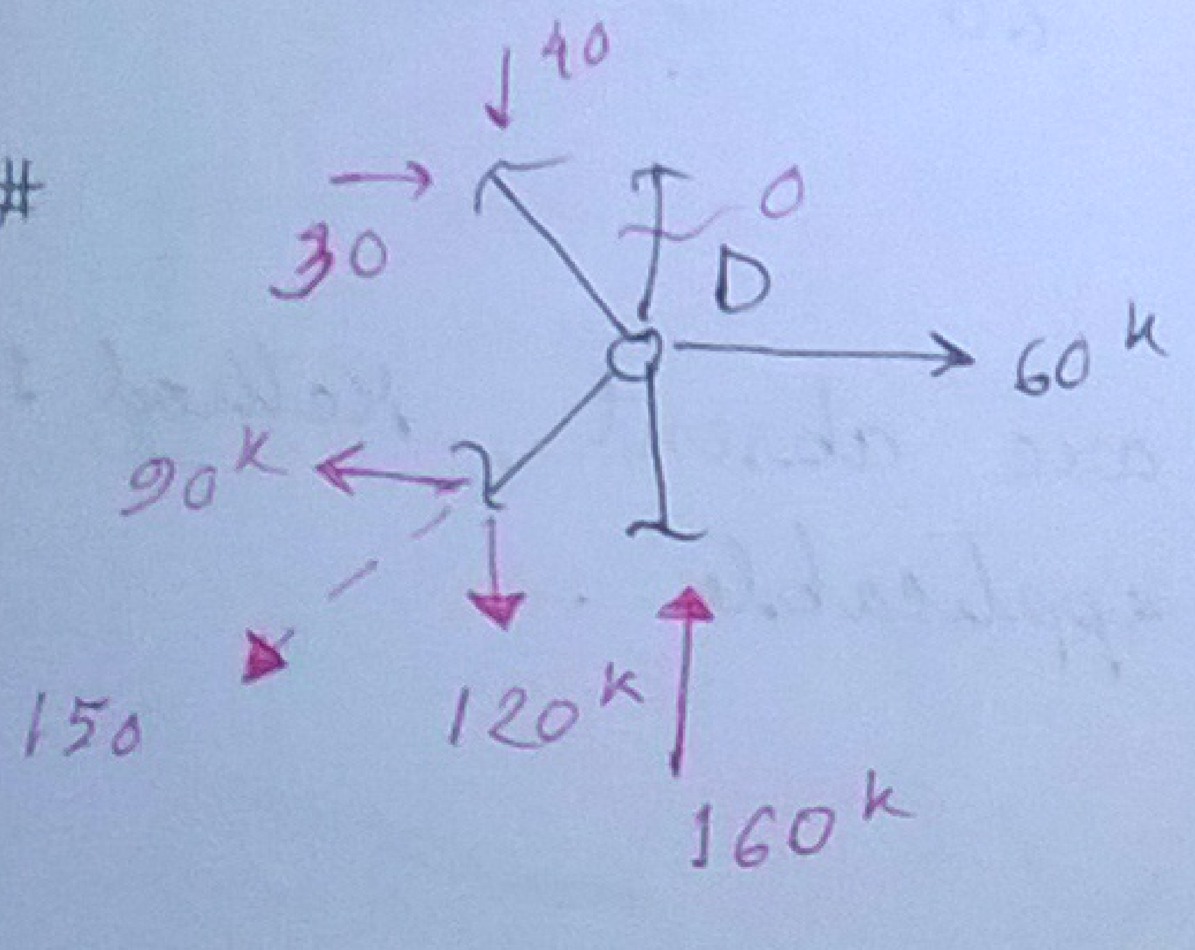
☑ If middle members are absent, Method 1 & Method 2 are not applicable.

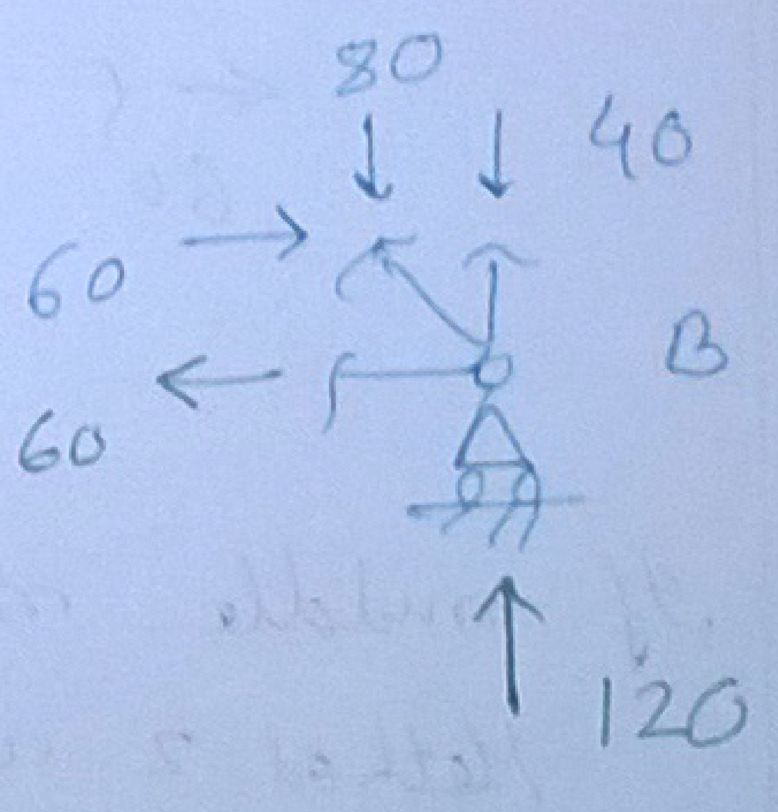
#

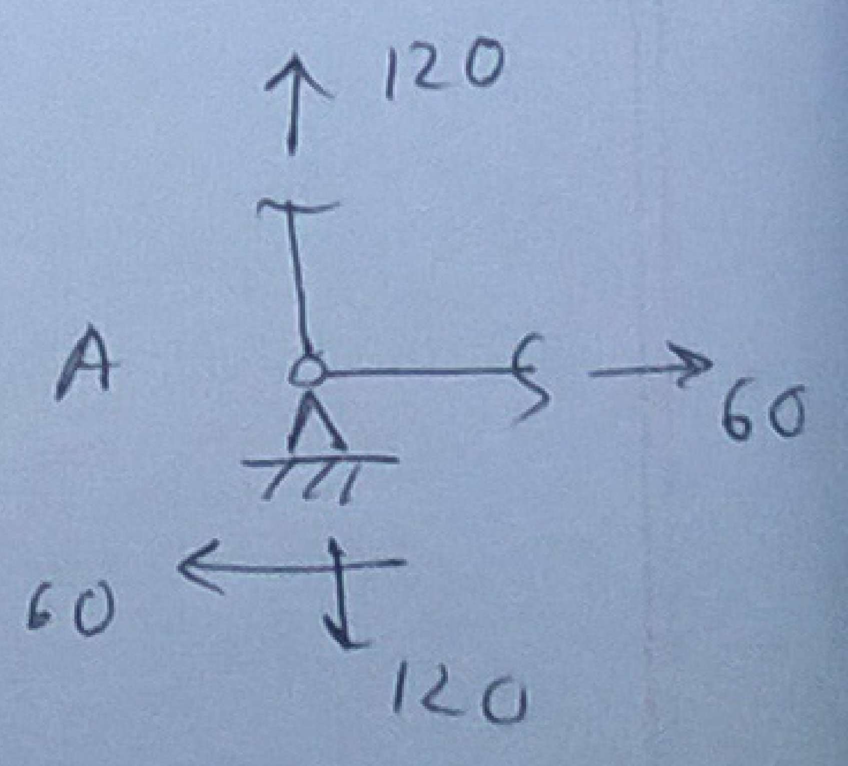


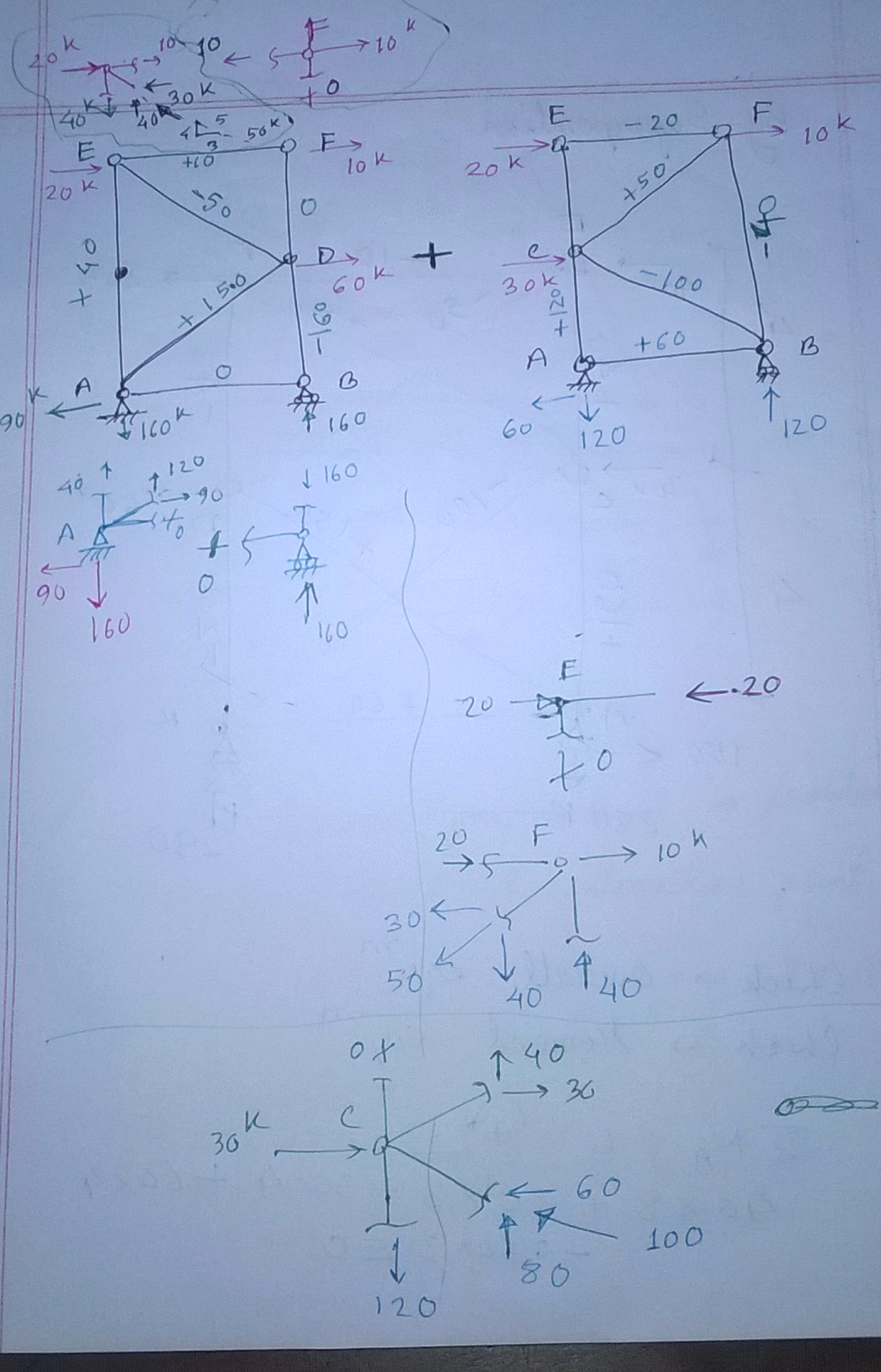
$d.o.i = (10 + 3) - 2 \times 6 = 1$

#



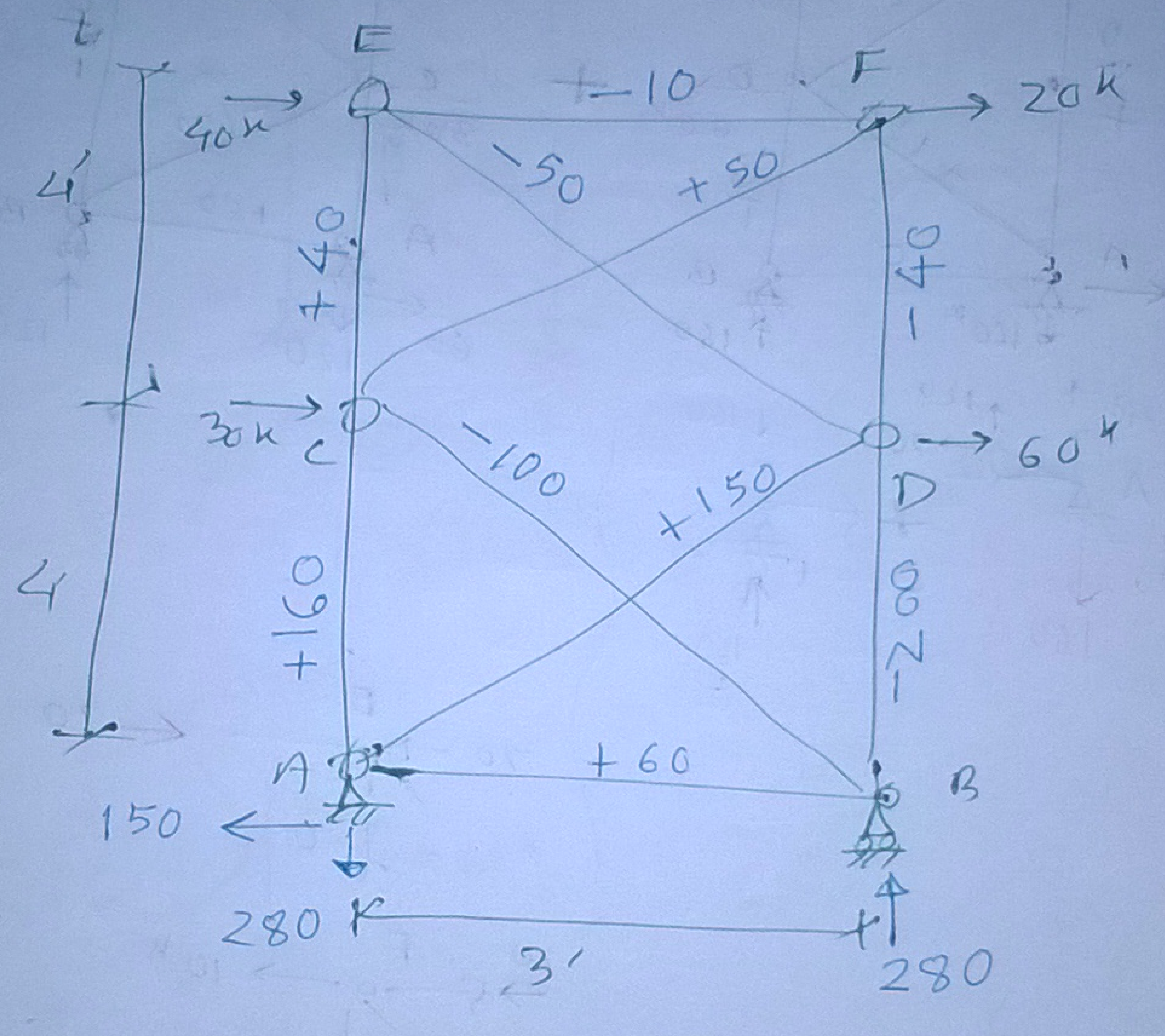






E
F
D
A
B
C
20 k
10 k
60 k
30 k
+

By Superposition,

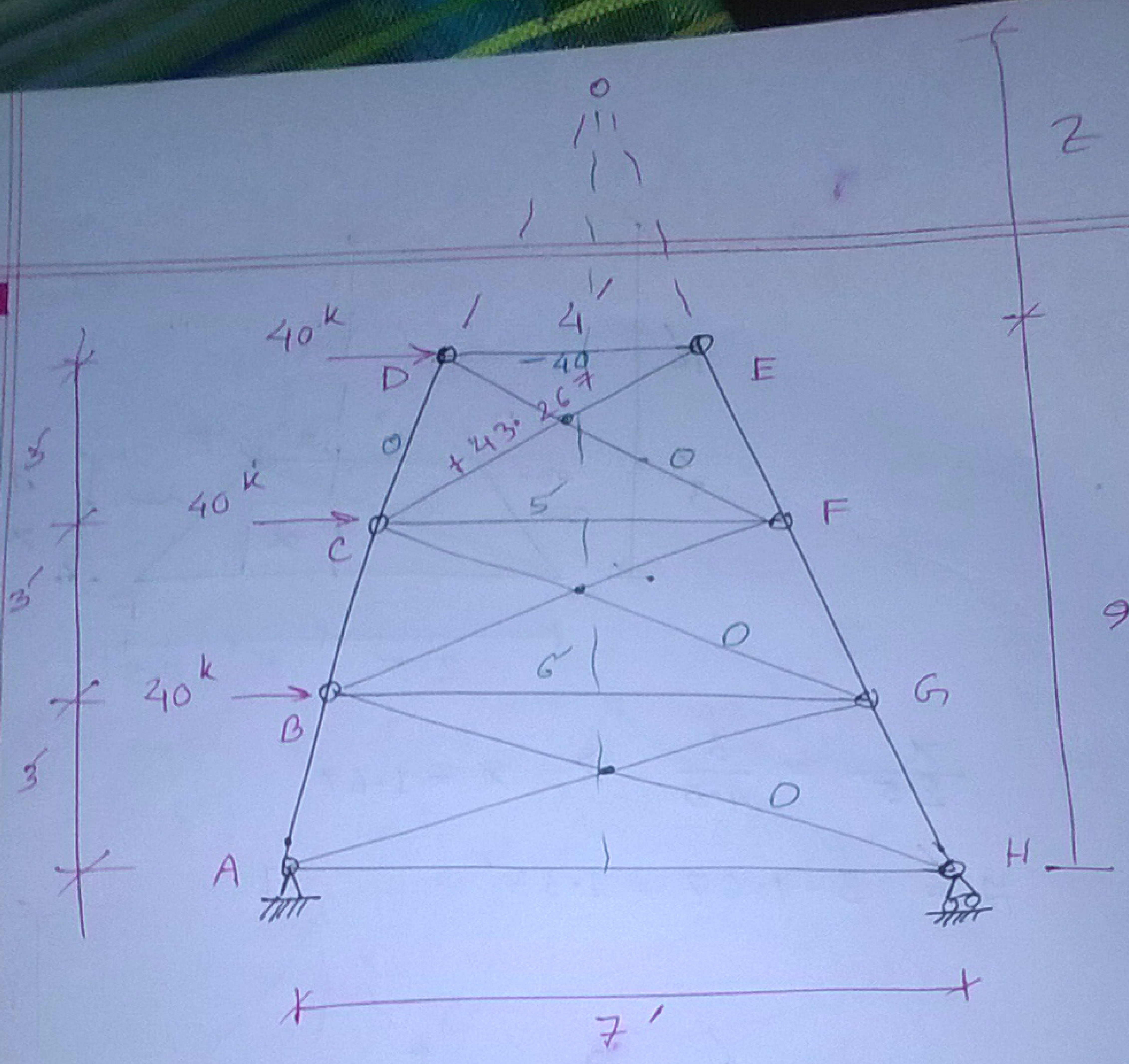


Check → Overall eq^m

Check → Moment eq^m

$\Sigma M_A = 0$ (+)

$40 \times 8 + 20 \times 8 + 30 \times 4 + 60 \times 4 - 280 \times 3 = 0$



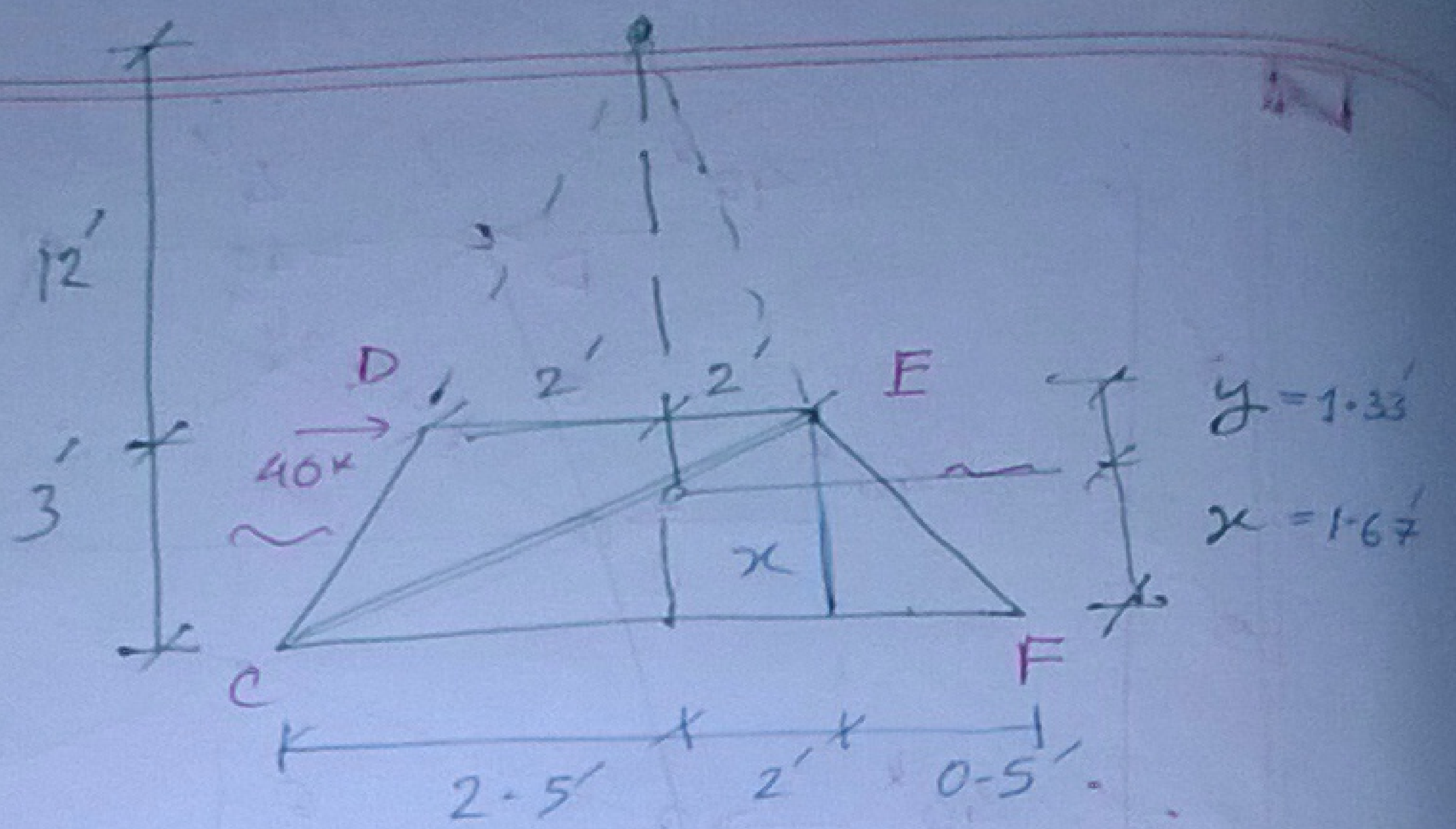
d.o.i = 3

Since main members are inclined → problematic.

Use Technique → Find intersection point, I

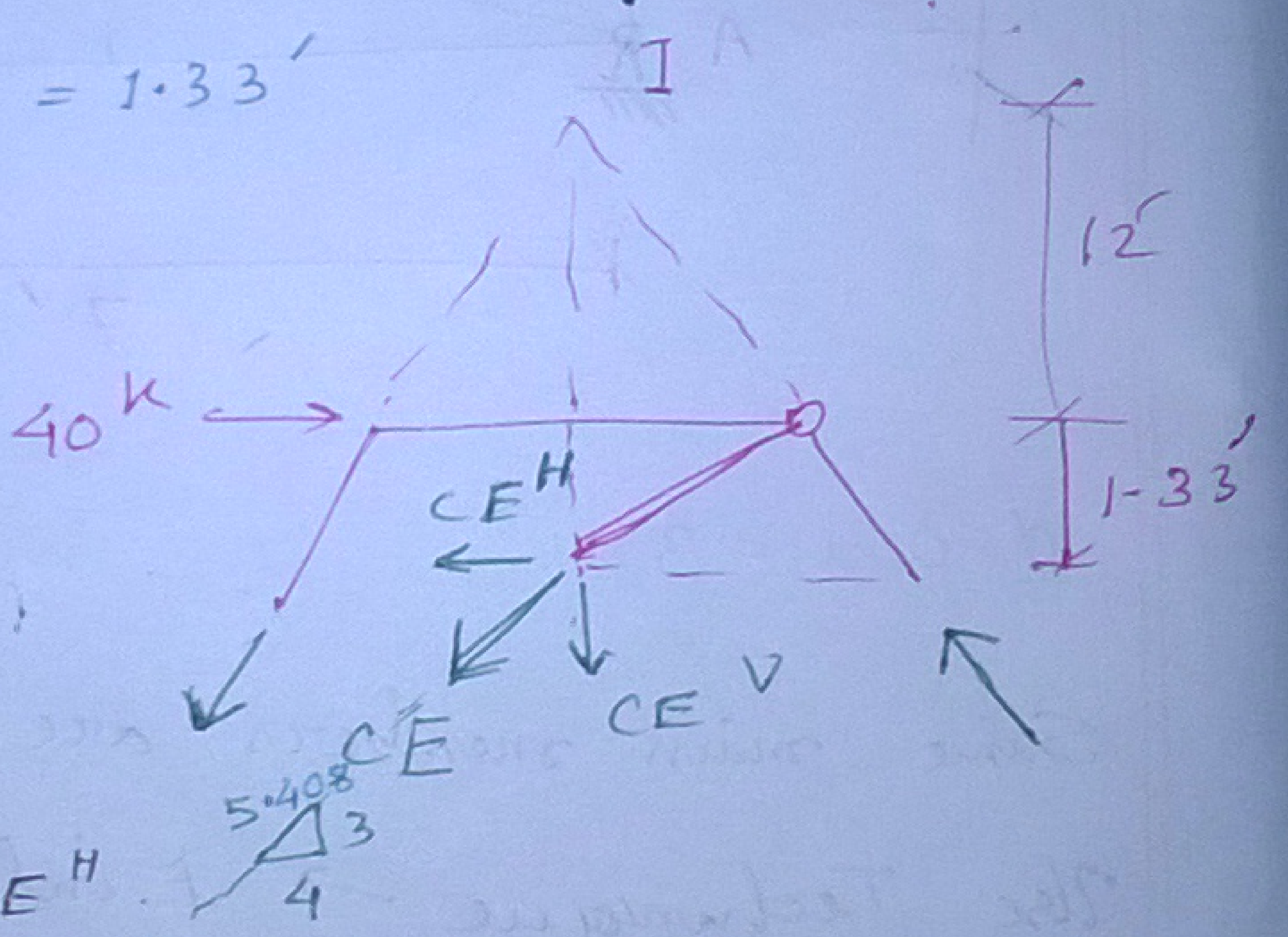
$$\frac{Z}{4} = \frac{Z+9}{7} \Rightarrow Z = 12'$$

Method 1



$$\frac{x}{2.5} = \frac{3}{4.5} \Rightarrow x = 1.67'$$

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



CE member is in tension

Only unknown is CE^H, others intersect at I

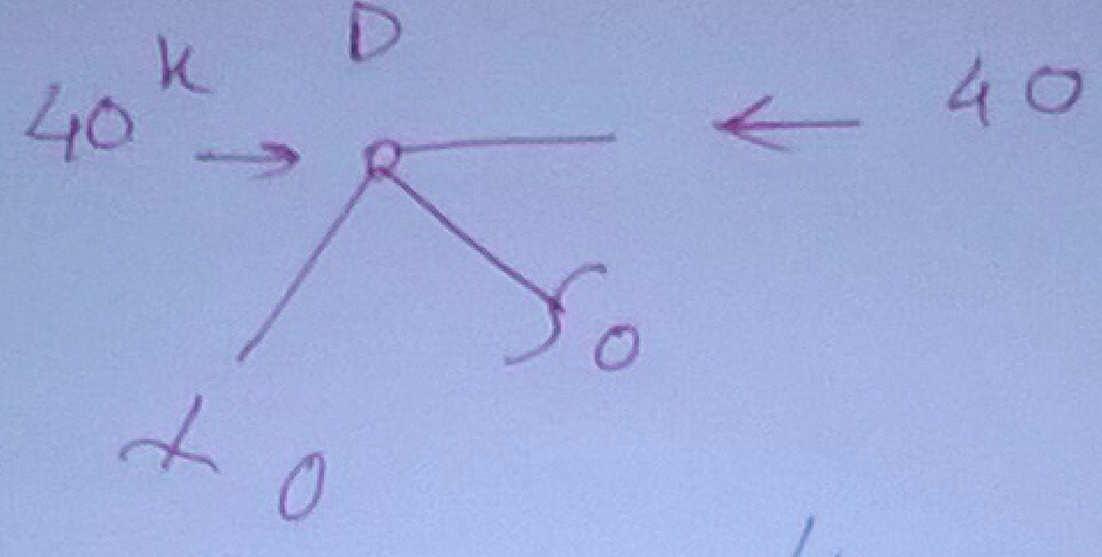
$$\sum M_I = 0 \quad (+)$$

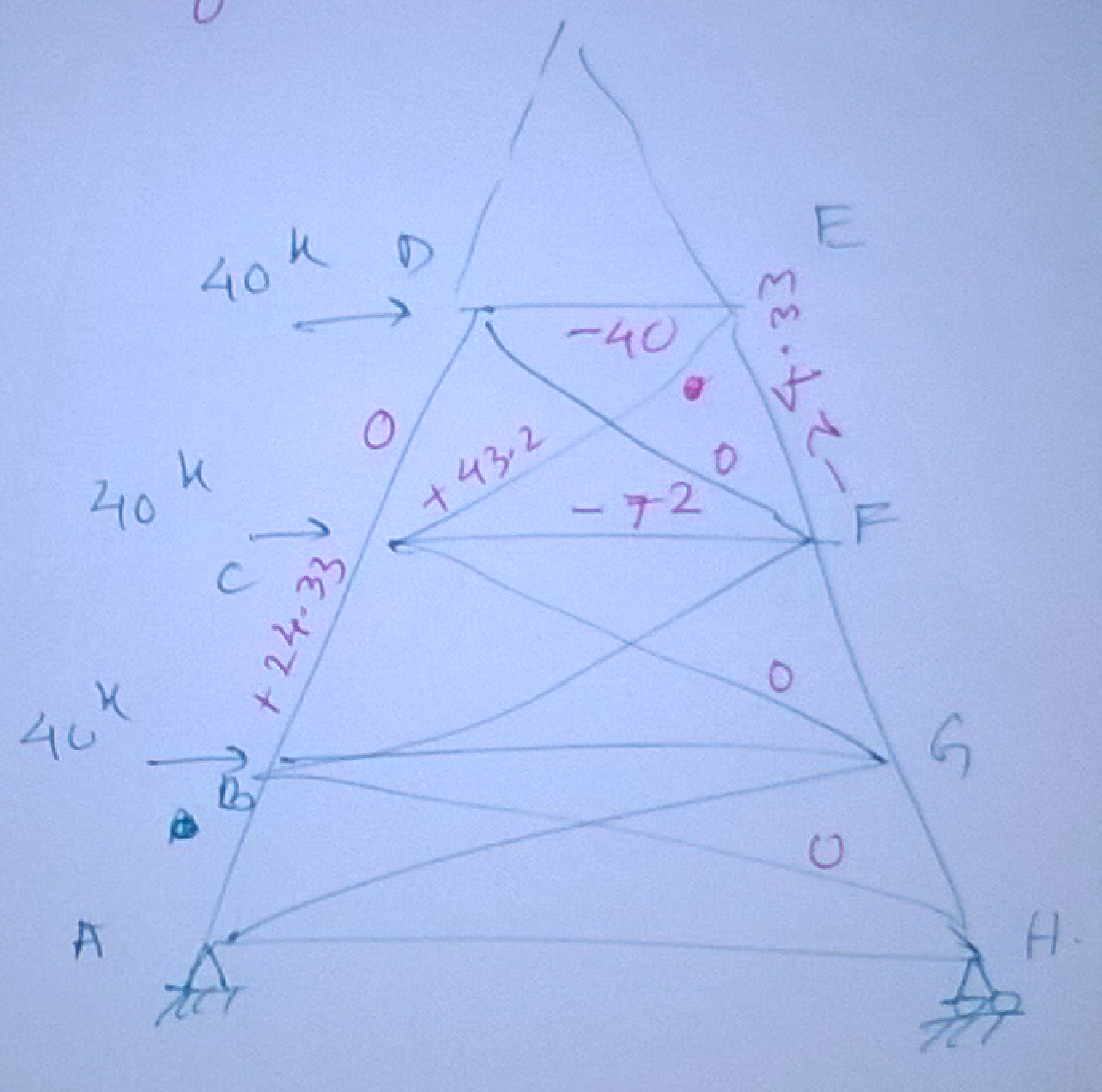
$$CE^H \times 13.33' - 40 \times 12 = 0$$

$$\therefore CE^H = 36^k$$

$$CE = \frac{36}{4.5} \times 5.408 = 43.267^k$$

$$CE_V = \frac{36^k}{4.5} \times 3 = 24^k$$


40 k
D
40
0
0


40 k
D
E
-40
0
0
40 k
+43.2
-72
C
F
+24.33
40 k
0
B
G
0
A
H.