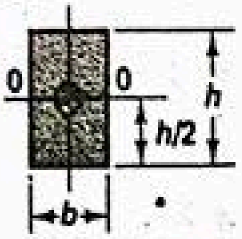
$\boxed{I_x = I_0 + Ad^2}$

TABLE 2. USEFUL PROPERTIES OF AREAS

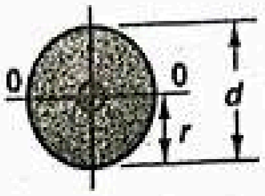
Areas and moments of inertia of areas around centroidal axes

RECTANGLE



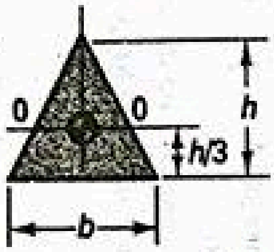
$A = bh$
$I_o = bh^3/12$

CIRCLE



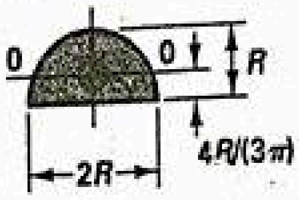
$A = \pi R^2$
$I_o = I_p/2 = \pi R^4/4$

TRIANGLE



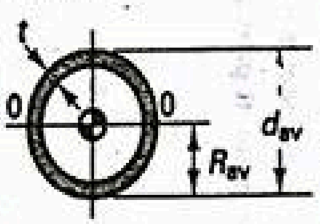
$A = bh/2$
$I_o = bh^3/36$

SEMICIRCLE



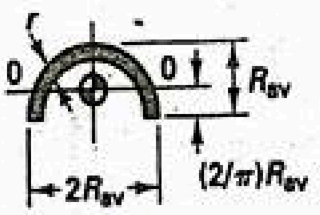
$A = \pi R^2/2$
$I_o = 0.110R^4$

THIN TUBE



$A = 2\pi R_{av} t$
$I_o = I_p/2 \approx \pi R_{av} t$

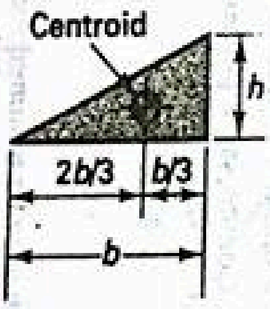
HALF OF THIN TUBE



$A = \pi R_{av} t$
$I_o \approx 0.095\pi R_{av}^3 t$

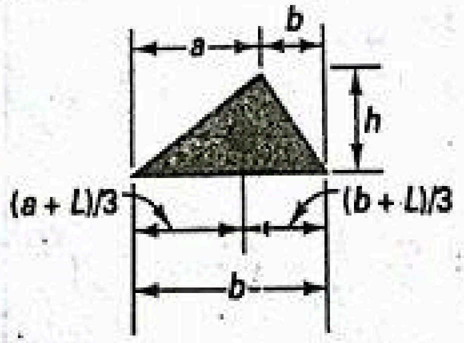
Areas and Centroids of areas

TRIANGLE



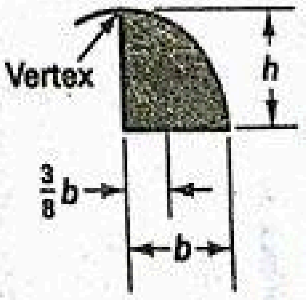
$A = bh/2$

TRIANGLE



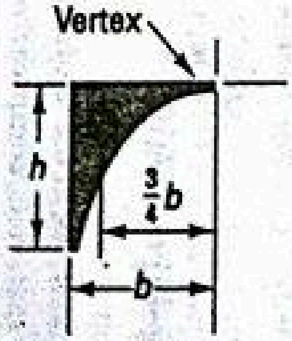
$A = hL/2$

PARABOLA



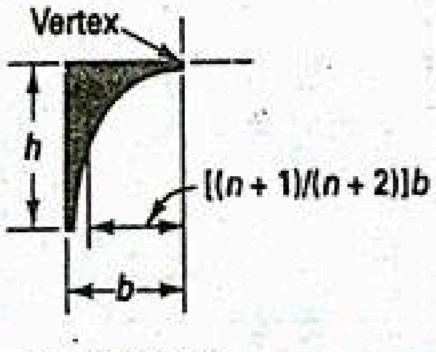
$A = \frac{2}{3} bh$

PARABOLA: $y = -ax^2$



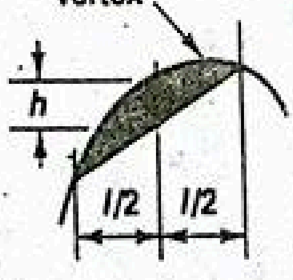
$A = bh/3$

$y = -ax^n$



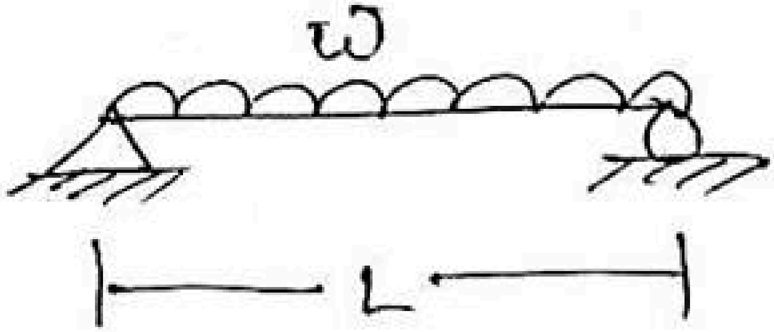
$A = bh/(n + 1)$

PARABOLA

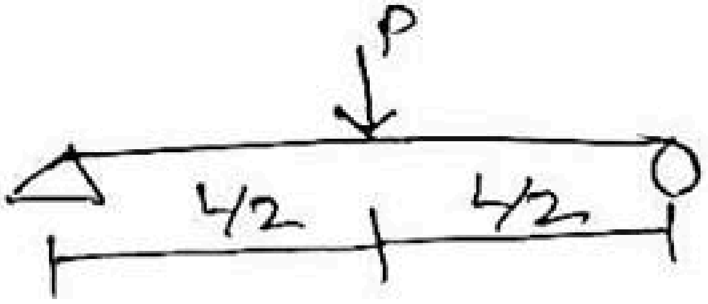


The area for any segment of a parabola is $A = \frac{2}{3} hl$

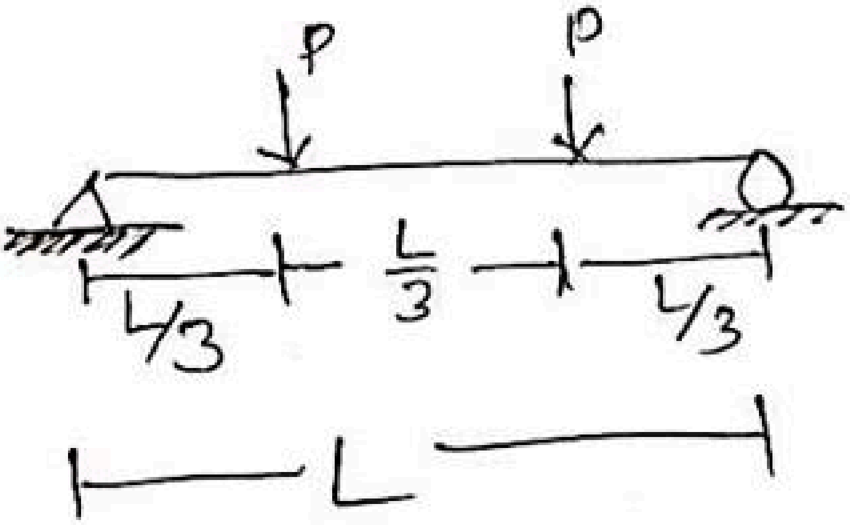
Δ_{max}



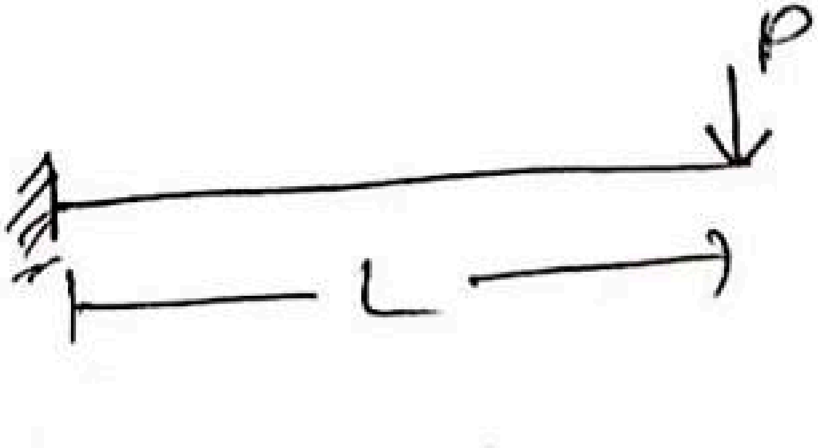
$$= \frac{5wL^4}{384EI}$$



$$= \frac{PL^3}{48EI}$$



$$= \frac{23PL^3}{648EI}$$



$$\frac{PL^3}{3EI}$$

$$= \frac{wL^4}{8EI}$$

2.5.6 Equivalent Static Force Method

This method may be used for calculation of seismic lateral forces for all structures specified in Sec 2.5.5.1(a)

2.5.6.1 Design Base Shear : The total design base shear in a given direction shall be determined from the following relation :

$$V = \frac{ZIC}{R} W \tag{2.5.1}$$

where, Z = Seismic zone coefficient given in Table 6.2.22
I = Structure importance coefficient given in Table 6.2.23
R = Response modification coefficient for structural systems given in Table 6.2.24
W = The total seismic dead load defined in Sec 2.5.5.2
C = Numerical coefficient given by the relation :

$$C = \frac{1.25S}{T^{2/3}} \tag{2.5.2}$$

S = Site coefficient for soil characteristics as provided in Table 6.2.25
T = Fundamental period of vibration in seconds, of the structure for the direction under consideration as determined by the provisions of Sec 2.5.6.2.

The value of C need not exceed 2.75 and this value may be used for any structure without regard to soil type or structure period. Except for those requirements where Code prescribed forces are scaled up by $0.375R$, the minimum value of the ratio C/R shall be 0.075.

Table 6.2.22
Seismic Zone Coefficients, Z

Seismic Zone (see Fig 6.2.10)	Zone Coefficient
1	0.075
2	0.15
3	0.25

Table 6.2.23
Structure Importance Coefficients *I*, *I'*

Structure Importance Category (see Table 6.1.1 for occupancy)	Structure Importance Coefficient	
	I	I'
I Essential facilities	1.25	1.50
II Hazardous facilities	1.25	1.50
III Special occupancy structures	1.00	1.00
IV Standard occupancy structures	1.00	1.00
V Low-risk Structures	1.00	1.00

2.5.6.2 Structure Period : The value of the fundamental period, T of the structure shall be determined from one of the following methods :

a) **Method A** : For all buildings the value of T may be approximated by the following formula :

$$T = C_t (h_n)^{3/4} \tag{2.5.3}$$

where, C_t = 0.083 for steel moment resisting frames
= 0.073 for reinforced concrete moment resisting frames, and eccentric braced steel frames
= 0.049 for all other structural systems
h_n = Height in metres above the base to level n.

Alternatively, the value of C_t for buildings with concrete or masonry shear walls may be taken as $0.031/\sqrt{A_c}$. The value of A_c shall be obtained from the relation :

$$A_c = \sum A_e\left[0.2 + (D_e/h_n)^2\right] \tag{2.5.4}$$

where, A_c = The combined effective area, in square metres, of the shear walls in the first storey of the structure.
A_e = The effective horizontal cross-sectional area, in square metres of a shear wall in the first storey of the structure.
D_e = The length, in metre of a shear wall element in the first storey in the direction parallel to the applied forces.

The value of D_e/h_n for use in Eq (2.5.4) shall not exceed 0.9.

Table 6.2.25

Site Coefficient, S for Seismic Lateral Forces (1)

Type	Site Soil Characteristics: Description	Coefficient, S
S_1	A soil profile with either : a) A rock-like material characterized by a shear-wave velocity greater than 762 m/s or by other suitable means of classification, or b) Stiff or dense soil condition where the soil depth is less than 61 metres	1.0
S_2	A soil profile with dense or stiff soil conditions, where the soil depth exceeds 61 metres	1.2
✓ S_3	A soil profile 21 metres or more in depth and containing more than 6 metres of soft to medium stiff clay but not more than 12 metres of soft clay	1.5
S_4	A soil profile containing more than 12 metres of soft clay characterized by a shear wave velocity less than 152 m/s	2.0

Note : (1) The site coefficient shall be established from properly substantiated geotechnical data. In locations where the soil properties are not known in sufficient detail to determine the soil profile type, soil profile S_3 shall be used. Soil profile S_4 need not be assumed unless the building official determines that soil profile S_4 may be present at the site, or in the event that soil profile S_4 is established by geotechnical data.

b) **Method B** : The fundamental period T may be calculated using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. This requirement may be satisfied by using the following formula :

$$T = 2\pi\sqrt{\sum_{i=1}^{n} w_i\delta_i^2 \Big/ g\sum_{i=1}^{n} f_i\delta_i} \qquad (2.5.5)$$

The values of f_i represent any lateral force distributed approximately in accordance with the principles of Eq (2.5.6), (2.5.7) and (2.5.8) or any other rational distribution. The elastic deflections, δ_i shall be calculated using the applied lateral forces, f_i. The value of T determined from Eq (2.5.5) shall not exceed that calculated using Eq (2.5.3) by more than 40%.

2.5.6.3 Vertical Distribution of Lateral Forces : In the absence of a more rigorous procedure, the total lateral force, which is the base shear V, shall be distributed along the height of the structure in accordance with Eq (2.5.6), (2.5.7) and (2.5.8):

$$✓\ \boxed{V = F_t + \sum_{i=1}^{n} F_i} \qquad (2.5.6)$$

where, F_i = Lateral force applied at storey level -i and
F_t = Concentrated lateral force considered at the top of the building in addition to the force F_n.

The concentrated force, F_t acting at the top of the building shall be determined as follows:

$$\left\{\begin{array}{ll} F_t = 0.07\,TV \le 0.25\,V & \text{when } T > 0.7 \text{ second} \qquad (2.5.7a) \\ F_t = 0.0 & \text{when } T \le 0.7 \text{ second} \qquad (2.5.7b) \end{array}\right.$$

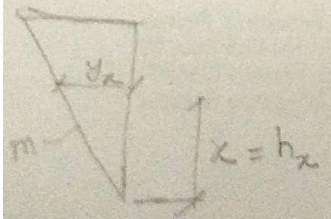
The remaining portion of the base shear $(V\text{-}F_t)$, shall be distributed over the height of the building, including level-n, according to the relation :

$$\left\{ F_x = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^{n} w_i h_i} \right. \qquad (2.5.8)$$

$\int mx = $ Force $= (V-F_t)$

$\frac{y}{h_x}\int x = \frac{(V-F_t)}{}$

$y = \frac{(V-F_t) * h_x}{\int x}$



CE 313

PROBLEM ON EARTHQUAKE

Rouf Sir
Sheet-3
(CE-08(A))

0804056
R-004

Example : Eq. Static EQ Force

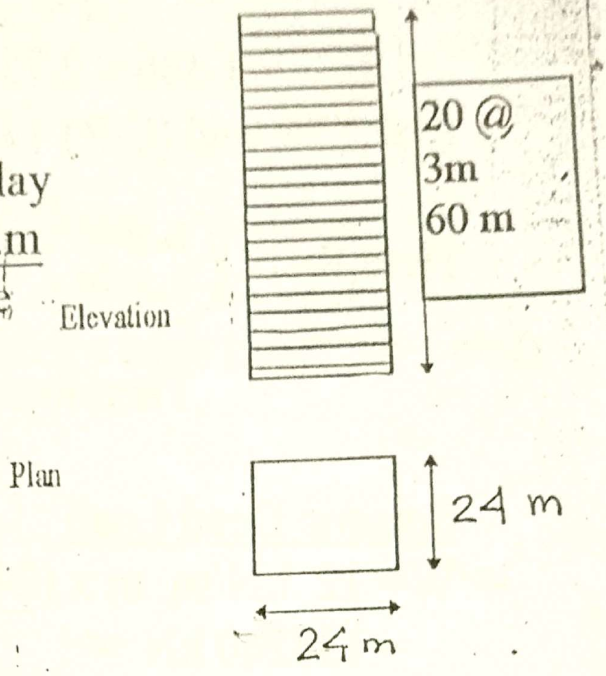
Calculate the distribution of EQ forces on the 20 storied office building shown in figure.

Location : Dhaka

Soil Type : Soft to medium stiff clay

DL including partition=12 kN/sq.m

Structure type : SMRF in concrete



6-2-24
BNBC

Solution:

Zone 2: $Z=0.15$ (Dhaka) $I=1.0$ (Table 6.2.23)

$$C=\frac{1.25S}{T^{2/3}}$$ (Eqn. 2.5.2)

Soil Type $S_3 = 1.5$ (Table 6.2.25)

$T=C_t(h_n)^{3/4}$ (Eqn 2.5.3)

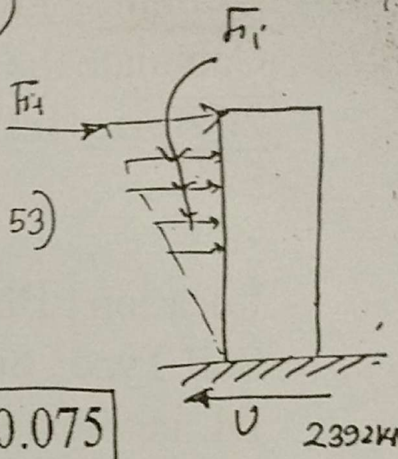
$h_n = 60$ m

$C_t=0.073$

$T=0.073X(60)^{3/4}$

$= 1.57$ Sec

$$C = \frac{1.25 \times 1.5}{(1.57)^{2/3}}$$ (Eqn 2.5.2)



$C = 1.386 < 2.75$ (page 6-53)

$C \not> 2.75$ and $(C/R) \not< 0.075$

Here $\boxed{R=12}$ (Table 6.2.24) hence: $\boxed{C/R=0.115 > 0.075}$

given

Therefore, C=1.386 O.K

Seismic Dead Load (Art. 2.5.5.2)

W= 12 kN/sq. m x (24x24 sq. m)x 20 floors

= 138240 kN

Base Shear, V= (ZIC)x W/R (Eqn. 2.5.1)

= (0.15 x 1 x 1.386 /12)x W

= 0.0173 W

= $\boxed{2392 \text{ kN}}$ 2395

Short ton, 1 ton=2000 lb
long ton, 1 ton= 2240 lb

$\frac{KN}{10}$ = ton

1 metric ton = 1000 kg

Vertical Distribution of Forces: (Eqn. 2.5.6)

The concentrated lateral force F_t at the top of the building:

$F_t = 0.07TV \leq 0.25\,V$ when $T > 0.7$ second
$F_t = 0.0$ when $T \leq 0.7$ second
Eqn. 2.5.7a,b

$F_t = 0.07 \times 1.57 \times 2392 = \boxed{264 \text{ kN}}$ (Eqn 2.5.7a)

in each floor

Scanned by CamScanner

here : $F_t \leq 0.25\ V$ i.e. $F_t = 264$ kN O.K.

*** $$F_x = \frac{(V - F_t)\, w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$ (Eqn. 2.5.8)

$\boxed{W_x = 12 \times 576}$ kN for each floor.
In this example w is same for all the floors.

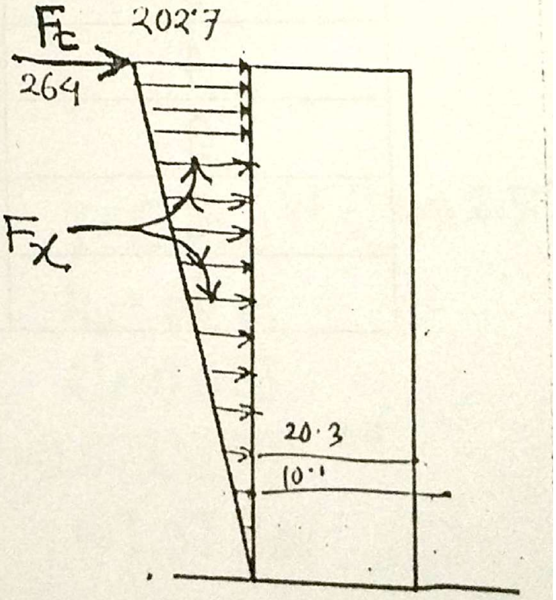
For any building for which w is same for each floor:

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

$$F_x = \frac{(2392 - 264)\, h_x}{(3 + 6 + 9 + \ldots\ldots\ldots + 60)}$$

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

$$\boxed{F_x = 3.38\, h_x}$$

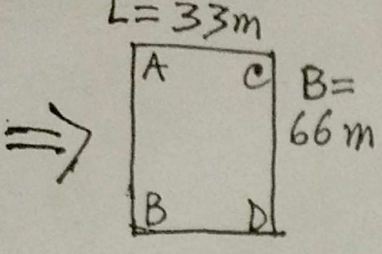


EQ Forces at various levels of the Building.

Floor Level	h_x , m	F_x , kN
20	60	202.7+264
19	57	192.5
18	54	182.4
17		
16		
15	45	152·1
14		
13		
12		
11		
10	30	101·4
9		
8		
7		
6	18	60·8
5		
4		
3		
2	6	20·3
1	3	10·1

PROBLEM OF on WIND LOAD (CE 311)

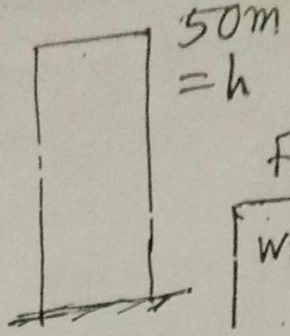
P1/2



$\frac{L}{B} = .5$

$\frac{h}{B} = .76$

Plan



Site: Dhaka ELV =

from Fig. 6.2.5

windward side $c_{pe} = +.8$

leeward side $c_{pe} = -.57$

side walls $c_{pe} = -.7$

$V_b = 210$ km/h

Exposure A

$q_z = C_c C_I C_z V_b^2$

$= (47.2 \times 10^{-6})(1.0) \times c_z (210)^2 = 2.082\, c_z$ KN/m²

(C_z in Table 6.2.10)

$p_z = c_G c_p q_z = c_G c_p\, 2.082\, C_z$ (KN/m²)

$c_G = G_h \longrightarrow$ in Table 6.2.11 (= 1.238 at 50m)

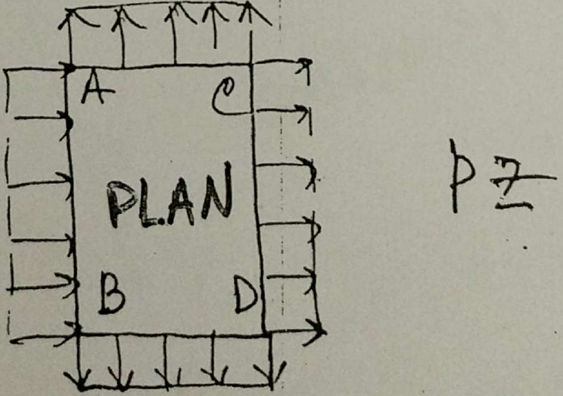
$c_p = c_{pe} \longrightarrow$ Fig. 6.2.5

$= 2.082 (1.238)\, \boxed{c_p}\, c_z$ (KN/m²)

$= \begin{cases} 2.578 (.8)\, c_z & \text{for Windward wall (AB)} \\ 2.578 (-.57)\, c_z & \text{for Leeward wall (CD)} \\ 2.578 (-.7)\, c_z & \text{for Side walls (AC, BD)} \end{cases}$

Method 1

$F_1 = \Sigma p A_z \Rightarrow$



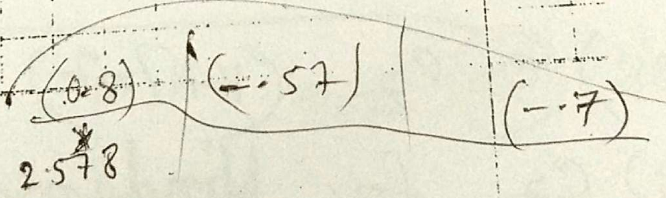
Pressure Diagram

P2/2

By Method 1

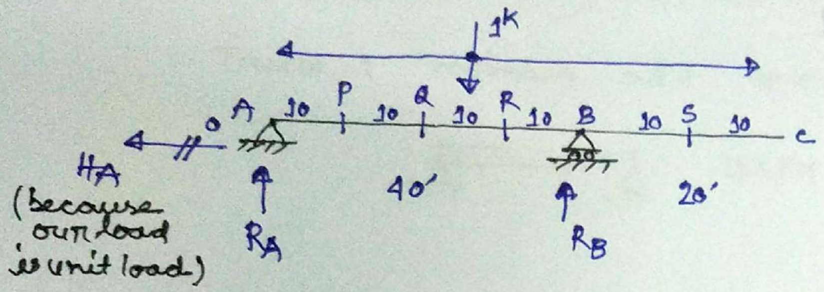
WIND PRESSURE ON (KN/m²)

Ht (m)	Cz	Windward (W)	Leeward (L)	~~Total~~ Side wall
0-4·5	·368	·76 225.72	-·54	-·66
6	·415	·86 255	-·6	-·75
9	·497			
12	·565	1·17 347	-·83	-1·02
15	·624			
18	·677			
21	·725			
24	·769	1·59	-1·13	-1·39
27	·810			
30	·849			
35	·909			
40	·965	1·99	-1·42	-1·74
45	1·017			
50	1·065	2·20 653	-1·56	-1·92



⊟ Influence line for reactions in Beams :

Beam 1 :



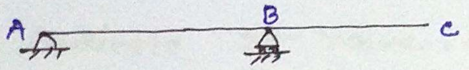
* Live load (vehicular load) vertically downward . so unit load vertically downward as most of the times vehicle ~~move~~ horizontally .

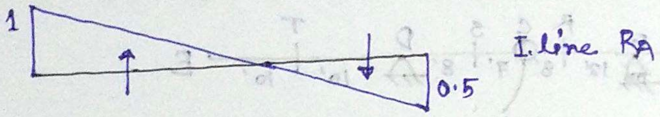
* Influence line is actually variation of reaction due to different position of loads.

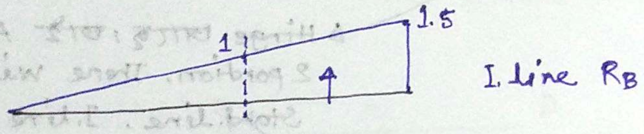
* unit load বিভিন্ন জায়গায় বসিয়ে Reaction বের করব,

unit load at	RA	RB
A	1↑	0
P	0.75↑	0.25↑
Q	0.5↑	0.5↑
R	0.25↑	0.75↑
B	0	1↑
S	0.25↓	1.25↑
C	0.5↓	1.5↑

⇒ এগুলো plot করলে influence line পাওয়া যাবে।







** I.line is a curve which shows the variation of a function under the action of moving unit load.

Drawing of I.line for R's in Beams:-

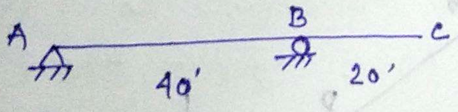
Steps: ① Push the support up by 1.

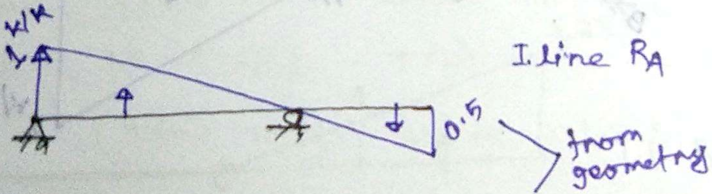
② Draw the δ-shape (deflected) ⇒ is the I.line

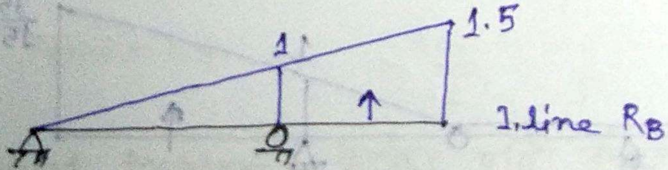
③ Complete the coordinates & direction (sign)

[যে support এর i.line দরকার সেই support কে push করতে হবে by 1.]

Beam 1:

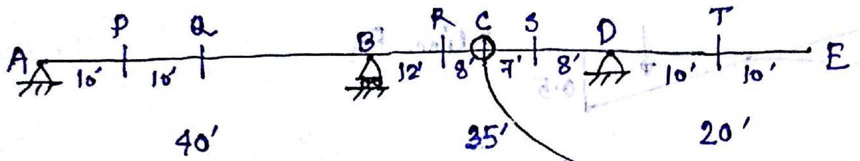






* 1 এর direction always up because load is down.

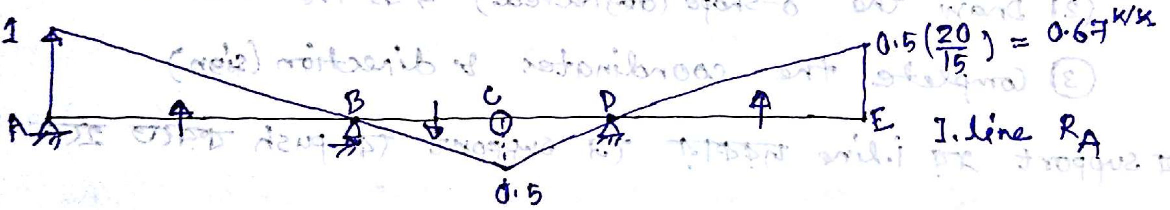
Beam 2:

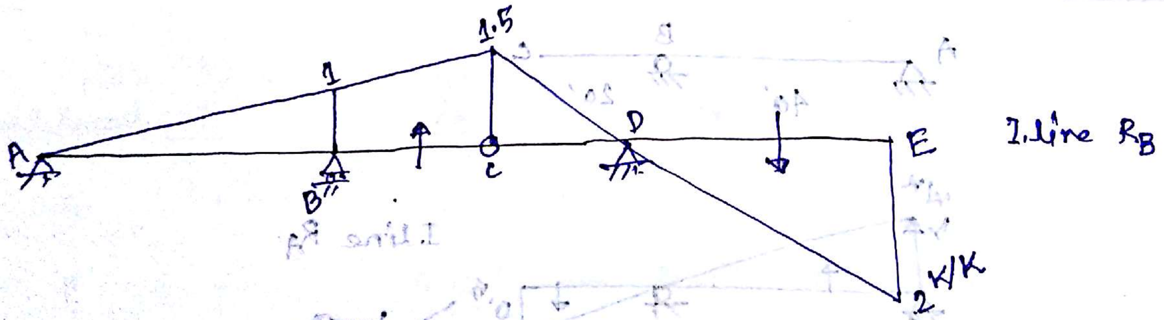


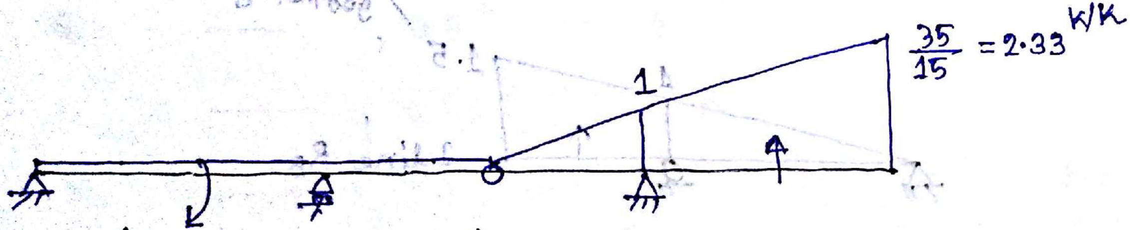
Hinge আছে, তাই ABC, CDE 2 portion. There will be 2 Stght. line. I.line will be combination of 2 Stght. lines.

[** Original beam এ যতগুলো Stght. line, I.line এ ততগুলো Stght. line.
C hinge এ কাটলে, CDE পড়ে যাবে।
ABC → self stable portion. CDE → stable না।
সবসময় ABC portion আগে draw করতে হবে for deflection shape.]

Unit:
K/K
or #/#
or kg/kg
(Reaction যে unit এ থাকবে)





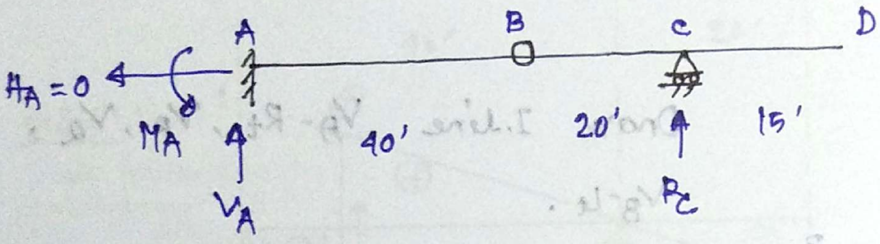


(কোন movement নাই, zero line)

[Design sheet এ Draw করলে ভাল বুঝা যায়]

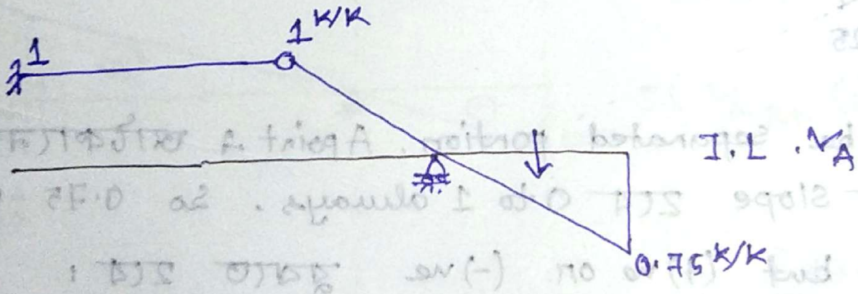
** Shape, Ordinate এর value, direction & unit থাকবে influence line এ.

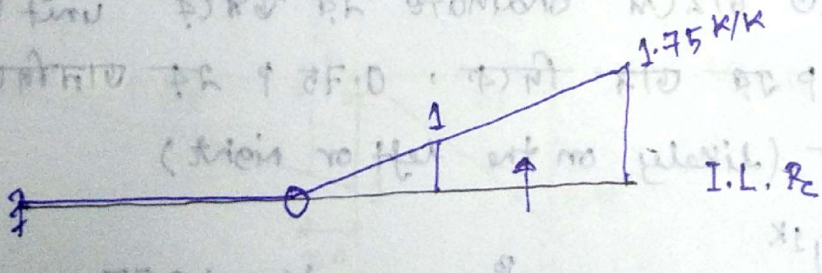
Beam 3:



* M_A হচ্ছে moment reaction. Stable part AB. তাই AB আগে draw করতে হবে।

* Hinge push করলে 90° হয় না। fixed joint এ 90° must be maintained. Line straight উপরে, উঠে থাকবে।





* Practice Solved Problem (উল্টো ছবি দিতে পারে)

I.line for SF in Beams

***Steps:**

① Cut at the section x.

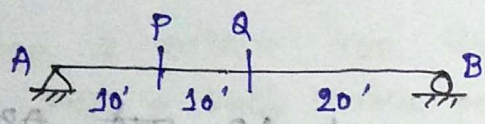
② Push the left part down & right part up (by 2 sides of x)

③ Draw δ-Shape → is the I.line

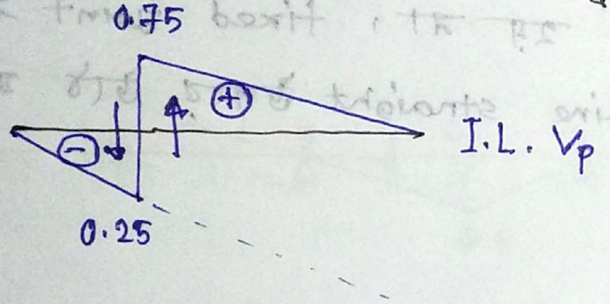
④ Put the ordinates & sign.

Beam 1:

Draw I.line V_{A-Rt}, V_P, V_Q, V_{B-Lt}.

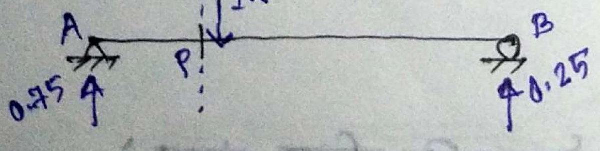


[* support এর উপরে shear force (V) calculate করা যায় না, তাই সাধারনত left or right হিসাব করা হয়।]



* AP & BP will be separated portion. A point এ আটকানো, P তে free. ২টা line এর slope হবে 0 to 1 always. So 0.75 ও 0.25 will be get but (+)ve or (−)ve বুঝাতে হবে।

* sign বের করতে চাইলে ordinate এর উপরে unit load বসাতে হবে exactly P এর ডান দিকে, 0.75 P এর বামদিকে, 0.25 P এর ডানদিকে (likely on the left or right)



⇒ $V_P = +0.75$

Steps:

① প্রথমে reaction বের করতে হবে,

② Section নিতে হবে,

③ $\Sigma F_y = 0$

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☐ I. line for SF in beams:

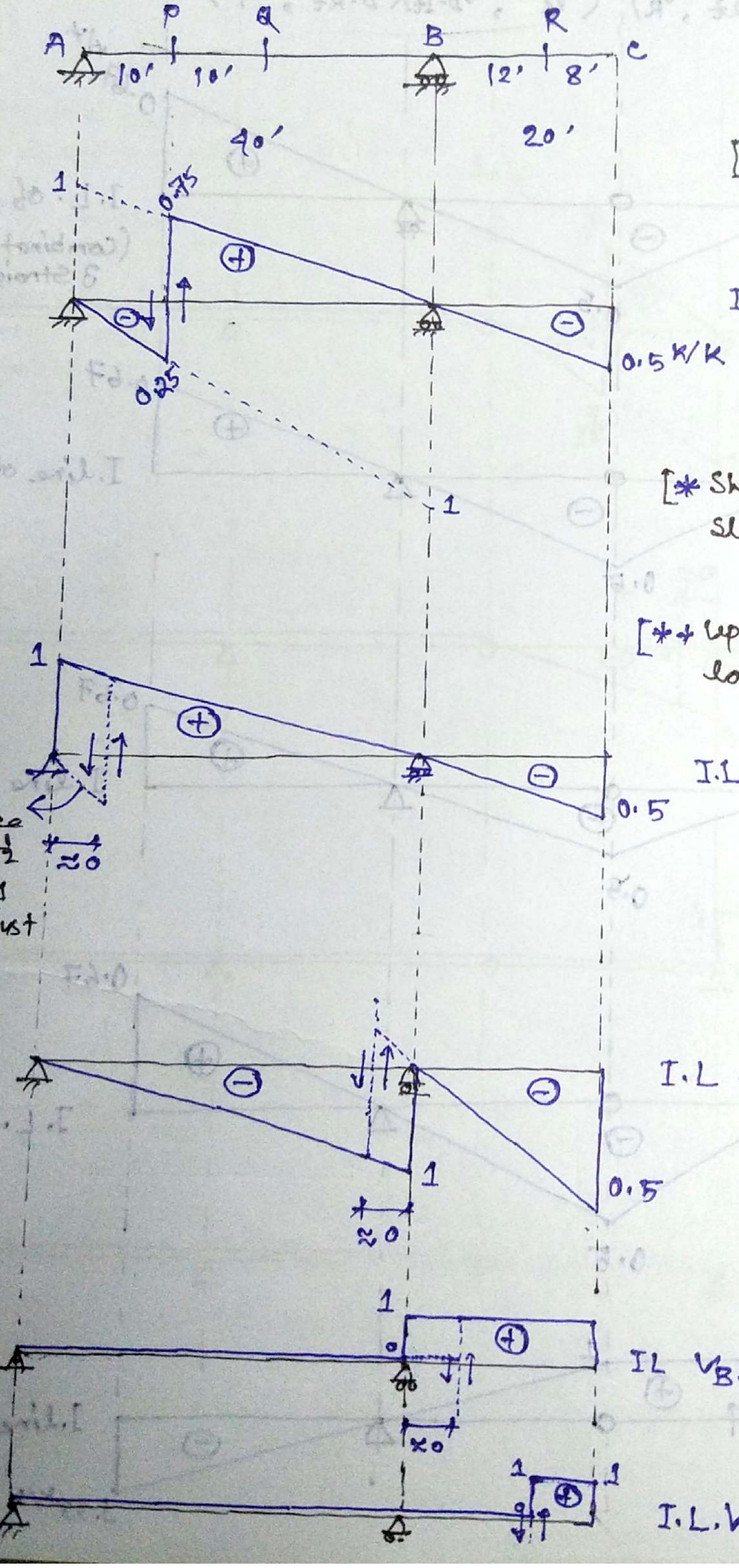
Beam 2 :

Draw I. line for

V_{A-Rt}, V_P, V_Q,
V_{B-Lt}, V_{B-Rt}, V_R

[Total section এর সংখ্যা infinity]

*Solⁿ:



IL V_P → SF at P

[P তে কাটলে 2টা straight line AP & PBC]

[* Shear force এর I. line, 0 & 1 slope এর সবটাই parallel থাকবে]

[** upper portion is (+)ve
lower portion is (-)ve]

I.L. V_{A-Rt} (just right of A)

[যেহেতু distance 0, সেহেতু এই অংশ থাকবে না, তাই adjust করতে হবে]

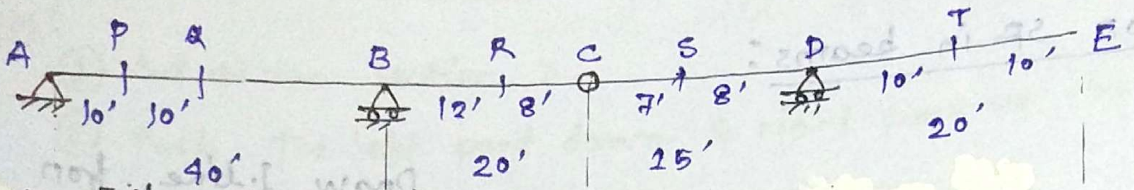
[*** যে section এ কাটলাম ঐ section এর
① দুই পাশের slope will be same
② দুইটা ordinate এর absolute value এর sum will be 1.]

I.L. V_{B-Lt}

IL V_{B-Rt}

I.L. V_R

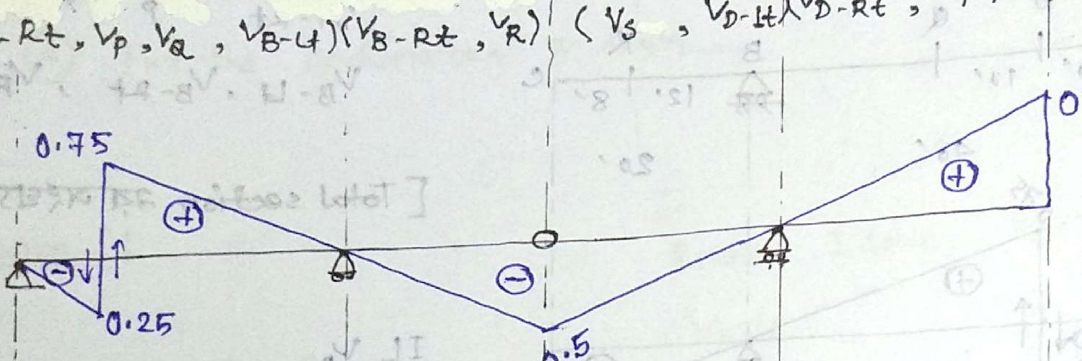
Beam 3 (Beam 1 & 2)



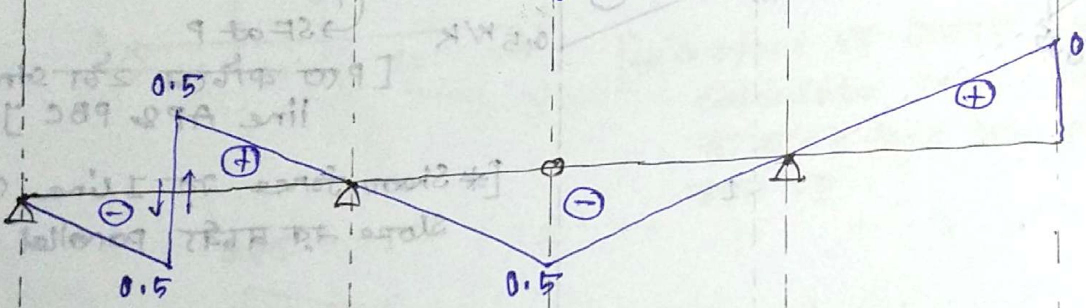
Draw I.line for

$(V_{A-Rt}, V_P, V_Q, V_{B-Lt})(V_{B-Rt}, V_R)$ $(V_S, V_{D-Lt})(V_{D-Rt}, V_T)$

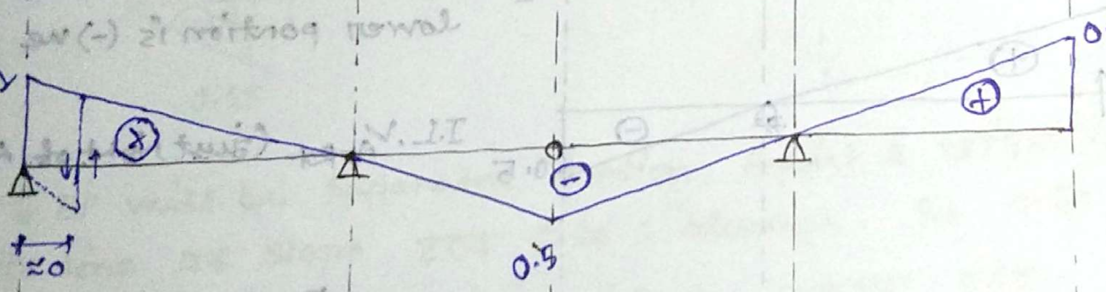
*Soln:



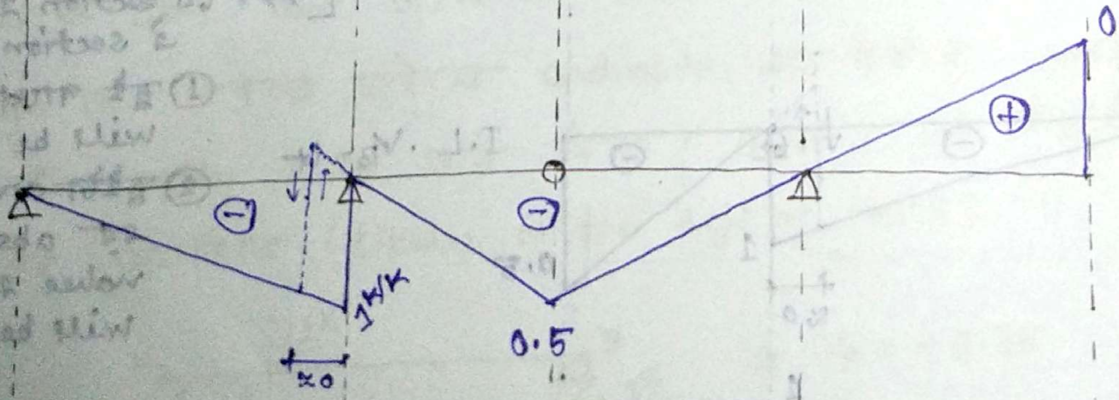
I.L. of V_P
(combination of 3 straight lines)



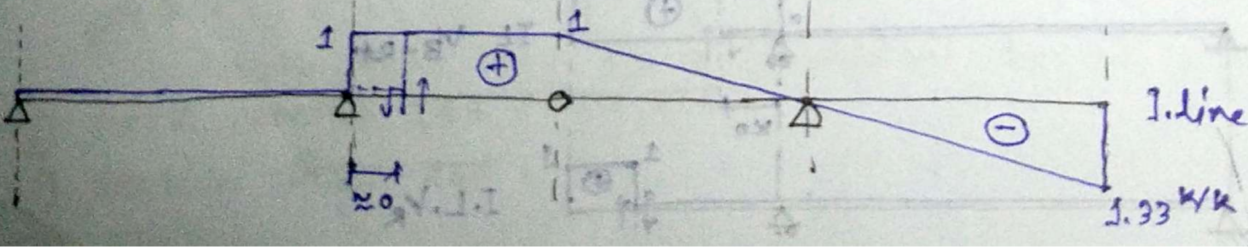
I. line of V_Q



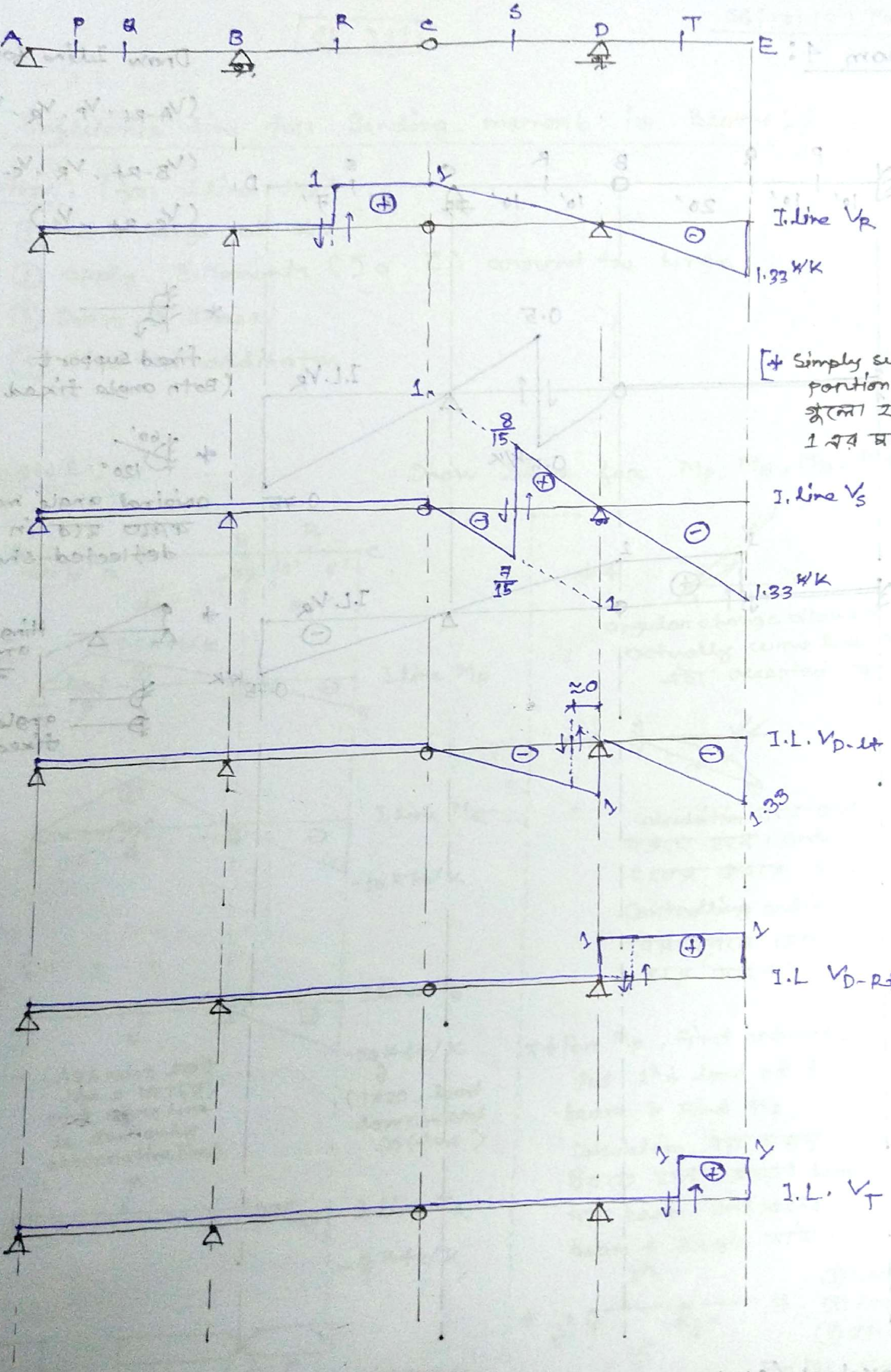
I. line V_{A-Rt}



I. L. V_{B-Lt}.



I.line V_{B-Rt}



[* Simply supported portion এর line গুলো হবে 0 and 1 এর মধ্যে]

Beam 4:

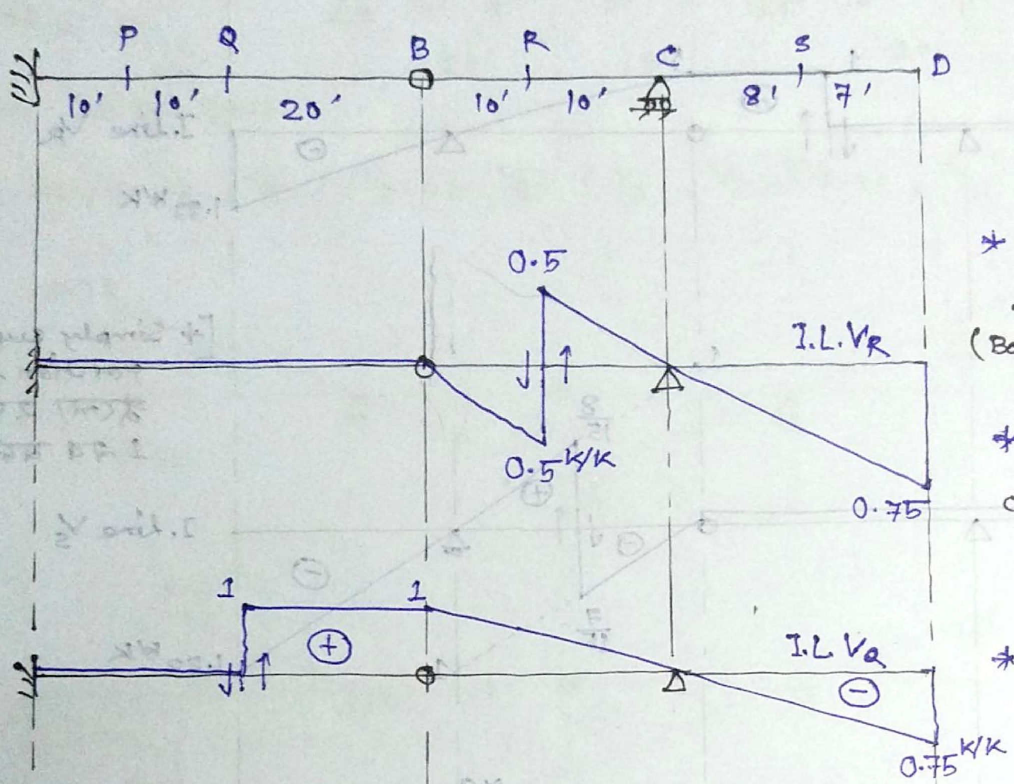
Draw I.line for

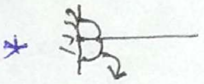
(V_{A-Rt} , V_P , V_Q , V_{B-Lt})

(V_{B-Rt} , V_R , V_{C-Lt})

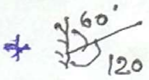
(V_{C-Rt} , V_S)

* Solⁿ :



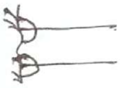


fixed support
(Both angle fixed)



original angle maintain
করতে হবে in
deflected shape

Hinge এ
angle fixed
না।



angle fixed
fixed support এ

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⊟. Influence line for Bending moment in Beams:

 # Steps: (for I.line M_x)

① Put a hinge at x.

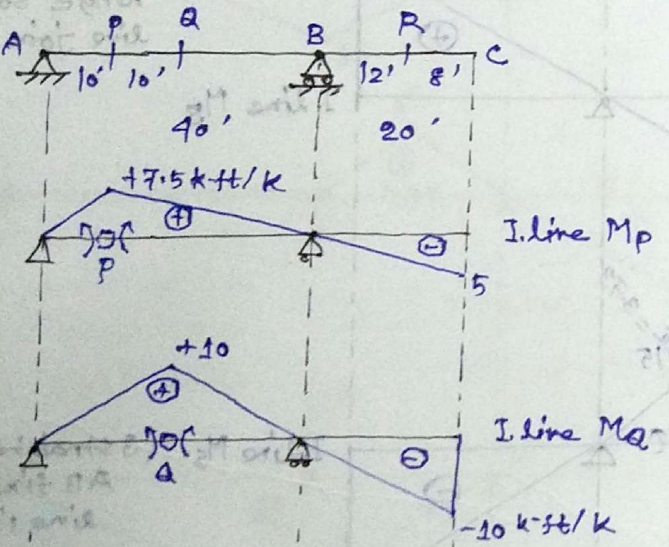
② apply B.Moments (Ↄ o Ꮯ) around the hinge

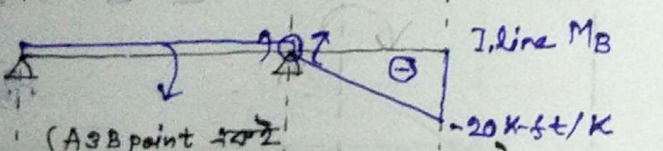
③ Draw δ shape

④ Find the ordinates

 # Beam 2:

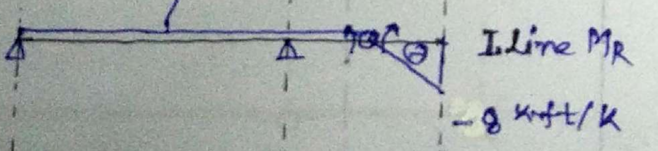
Draw I.line for M_P, M_Q, M_B, M_R

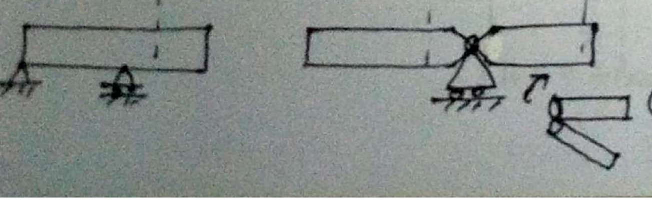




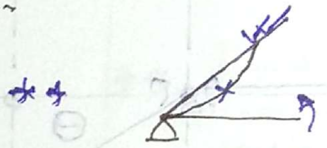
(A ও B point একই line এ আছে, তাই zero line is the only alternative line)

(1×20, load downward so (-)ve)

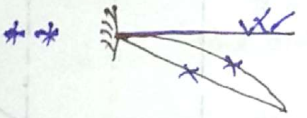




(দরকার মত)



angular change allowed at hinge actually curve line হবে। এটা accepted না।



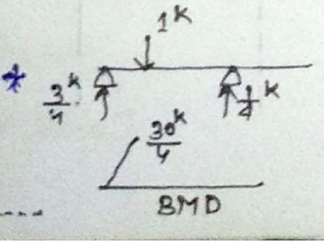
Calculation করে ordinate বের করতে হবে। ordinate at P or C বের করতে হবে।

Controlling ordinates (একটা বের করলে আরেকটা বের হয়ে যাবে)

** For M_P, Find ordinate at P.

Put 1^k↓ load at P in the original beam & Find M_p.

Calculation এর সময় original beam ধরতে হবে, কারণ hinge বসানোর পর beam unstable কিন্তু real beam এ hinge নাই।



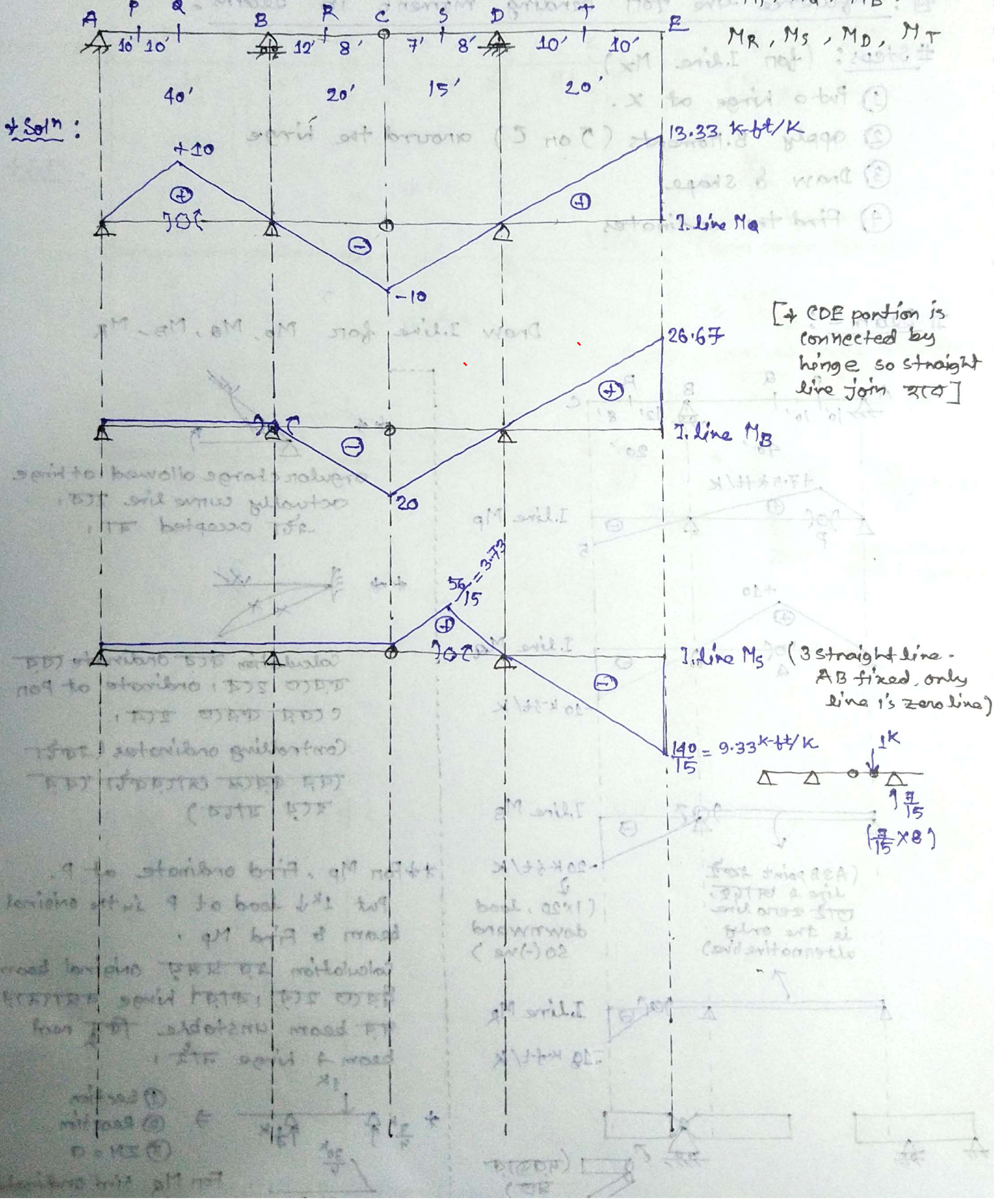
⇒ ① Section
② Reaction
③ ΣM = 0

For M_Q find ordinate at c.

Beam 3 :

Draw I.line for M_P, M_Q, M_B ; M_R, M_S, M_D, M_T

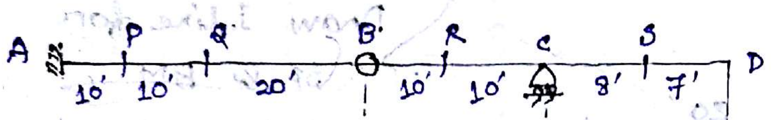
+Soln :



[+ CDE portion is connected by hinge so straight line join হবে]

(3 straight line - AB fixed, only line is zero line)

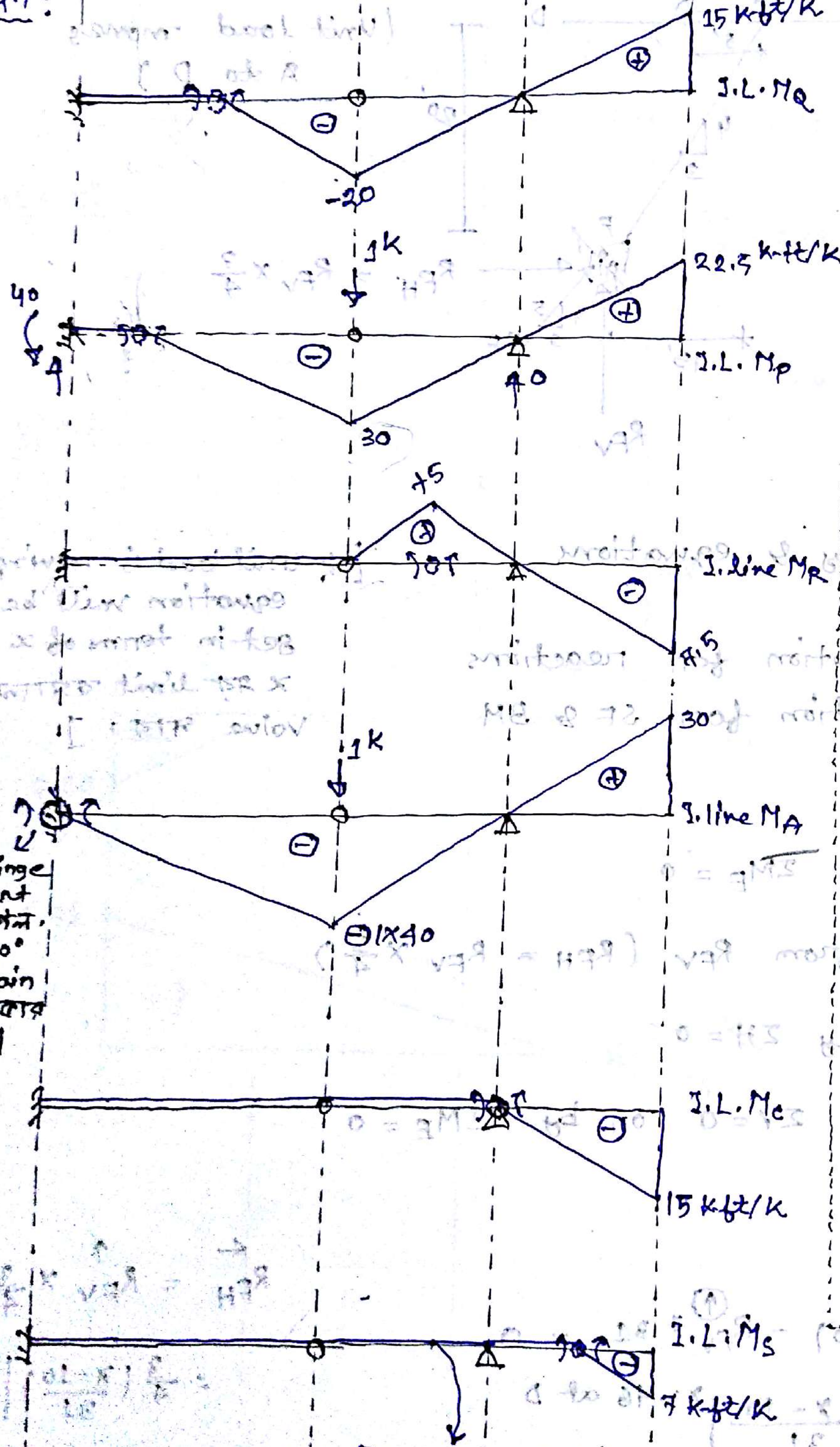
Beam 4 :



Draw I.line for

$M_A, M_P, M_Q, M_R, M_C, M_S$

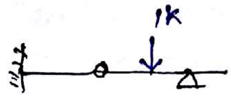
* Soln :



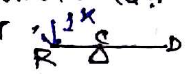
[এখন hinge support হয়ে গেল, তাই 90° maintain এর দরকার নাই]

[B ও C fixed, তাই zero line]

**



right section নিব কারণ left section নিলে vertical reaction ও moment বের করা লাগবে

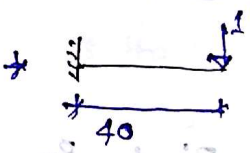


$\Sigma M_B = 0$

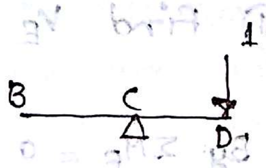
$\Rightarrow R_c = 0.5$ (Pab/L)

(find ordinates 7.5)

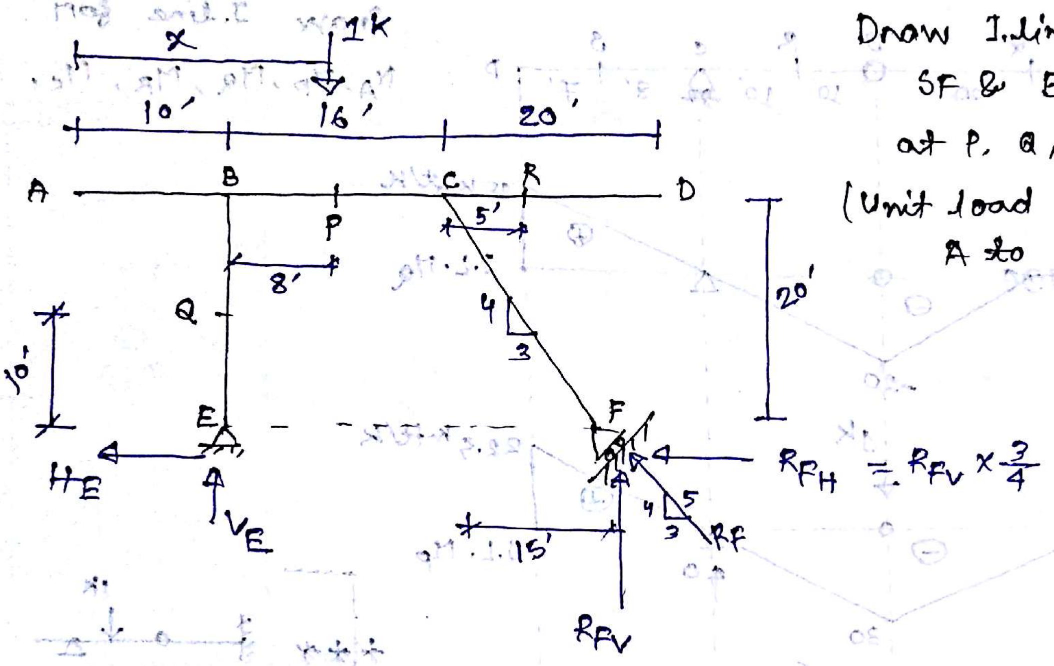
*



*



⊞ I.line for frames :



Draw I.line for SF & BM at P, Q, R. (Unit load moves A to D)

Soln : By freebody & equations

[∗ unit load is moving. equation will be get in terms of x. x এর limit বসালে value পাব।]

Steps: ① Find equation for reactions

② Find equation for SF & BM

R's :

① Find R_{Fv} by $\Sigma M_E = 0$

② Find R_{FH} from R_{FV} $(R_{FH} = R_{FV} \times \frac{3}{4})$

③ Find H_E by $\Sigma H = 0$

④ Find V_E by $\Sigma V = 0$ or by $\Sigma M_F = 0$

R_{Fv} : By $\Sigma M_E = 0$

$$\Rightarrow \overset{\downarrow}{1} * (x-10) - R_{Fv} * 31 = 0$$

$$\therefore R_{FV}^{\uparrow} = \frac{x-10}{31} \Big|_{x=0 \text{ at } A}^{x=46 \text{ at } D}$$

$$= -\overset{\downarrow}{\frac{10}{31}} \text{ at } A \quad \& \quad \overset{\uparrow}{\frac{36}{31}} \text{ at } D$$

$$\overset{\leftarrow}{R_{FH}} = \overset{\uparrow}{R_{FV}} \times \frac{3}{4}$$

$$= \frac{3}{4}\left(\frac{x-10}{31}\right)\Big|_{x=0}^{46}$$

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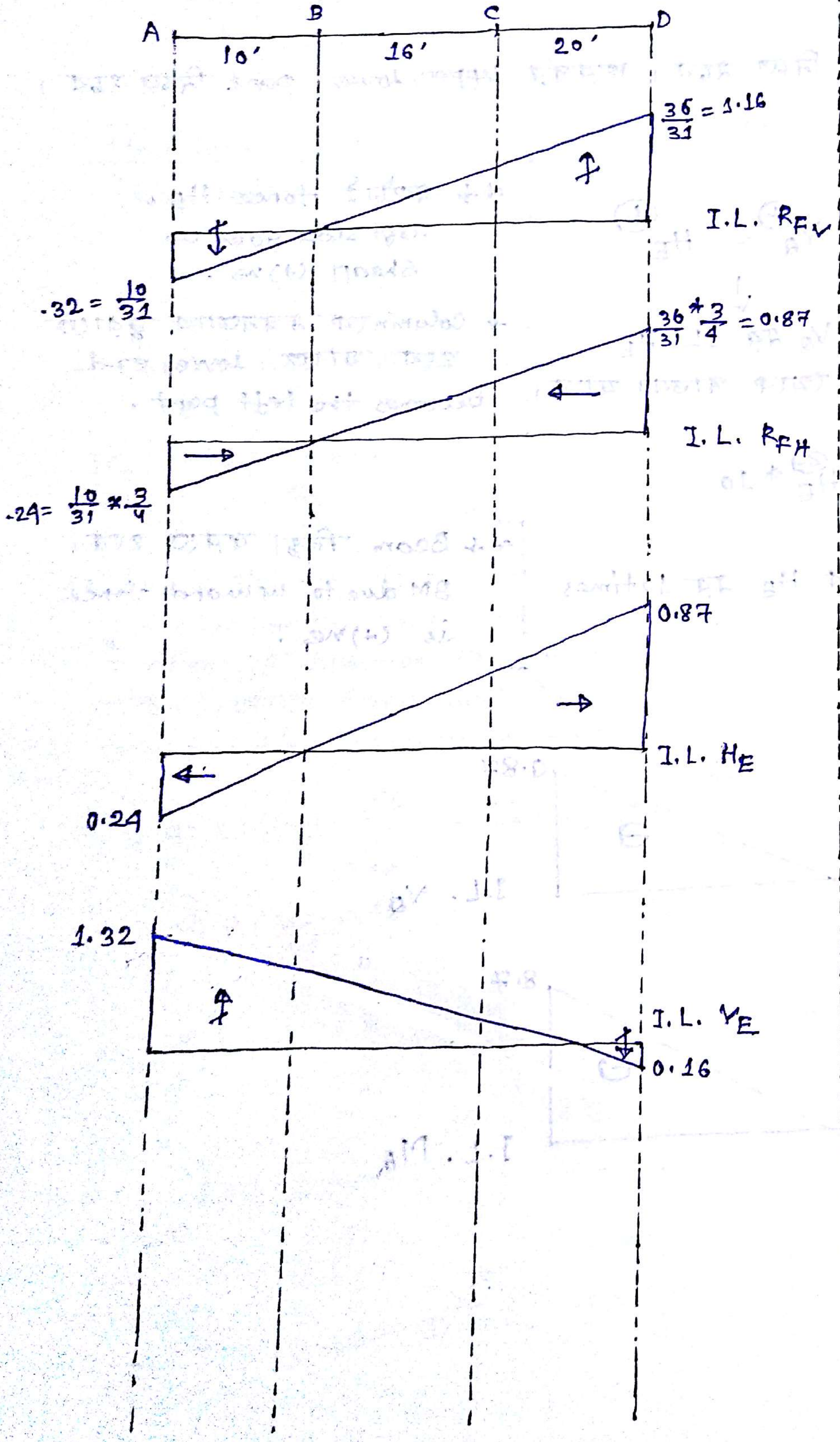


** vertical up হলে horizontal left. So, vertical down হলে horz. right হবে।

R_{FH} is a factor of R_{FV}. So, I. line will be same, ordinate will be different.

* Reaction এর জন্য sign না, direction important

* coordinate always decimal এ লিখতে হবে।

<u>H_E</u>: By $\Sigma H = 0$

$$\Rightarrow \overleftarrow{H_E} + \overleftarrow{R_{FH}} = 0$$

$$\Rightarrow \overleftarrow{H_E} = -\overleftarrow{R_{FH}}$$

$$= -\overset{\uparrow}{R_{FV}} \times \frac{3}{4}$$

$$= -\frac{3}{4}\left(\frac{x-10}{31}\right)\Big]_{x=0}^{46}$$

* একই IL হবে, শুধু direction change হবে। R_{FH} left হলে H_E right হবে as আর কোন horz. reaction নাই

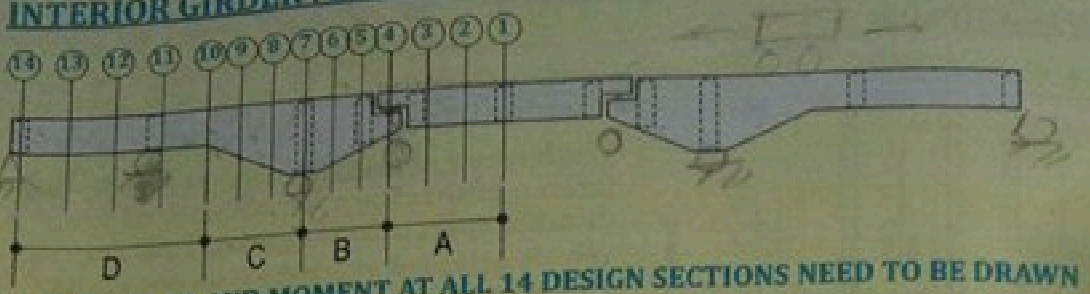
* $\Sigma Y = 0 = 1^{\downarrow} - V_E^{\uparrow} - R_{FV}^{\uparrow}$

$$\therefore V_E^{\uparrow} = 1 - \overset{\uparrow}{R_{FV}}$$

$$= 1 - \left(\frac{x-10}{31}\right)_{x=0}^{46}$$

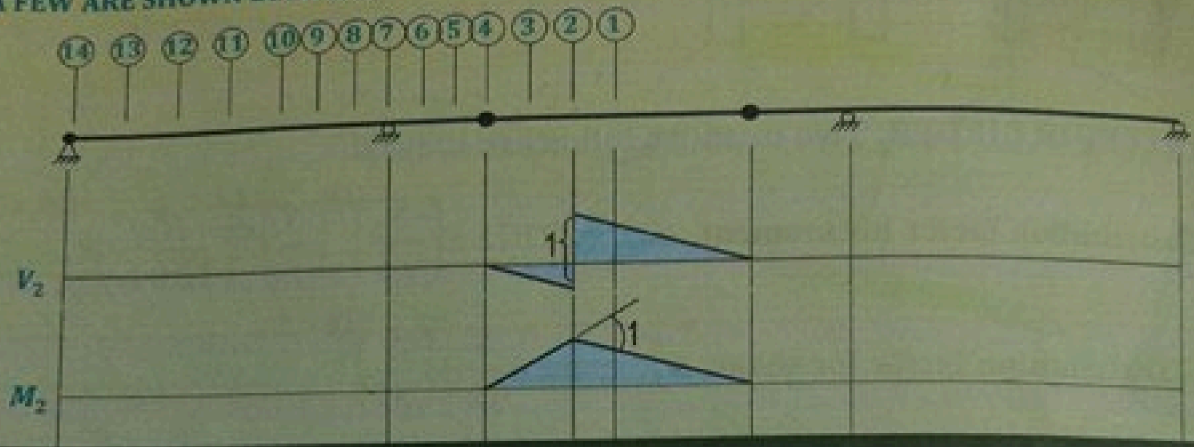
(+)ve হলে $V_E^{\uparrow}$

INTERIOR GIRDER : LIVE LOAD ANALYSIS



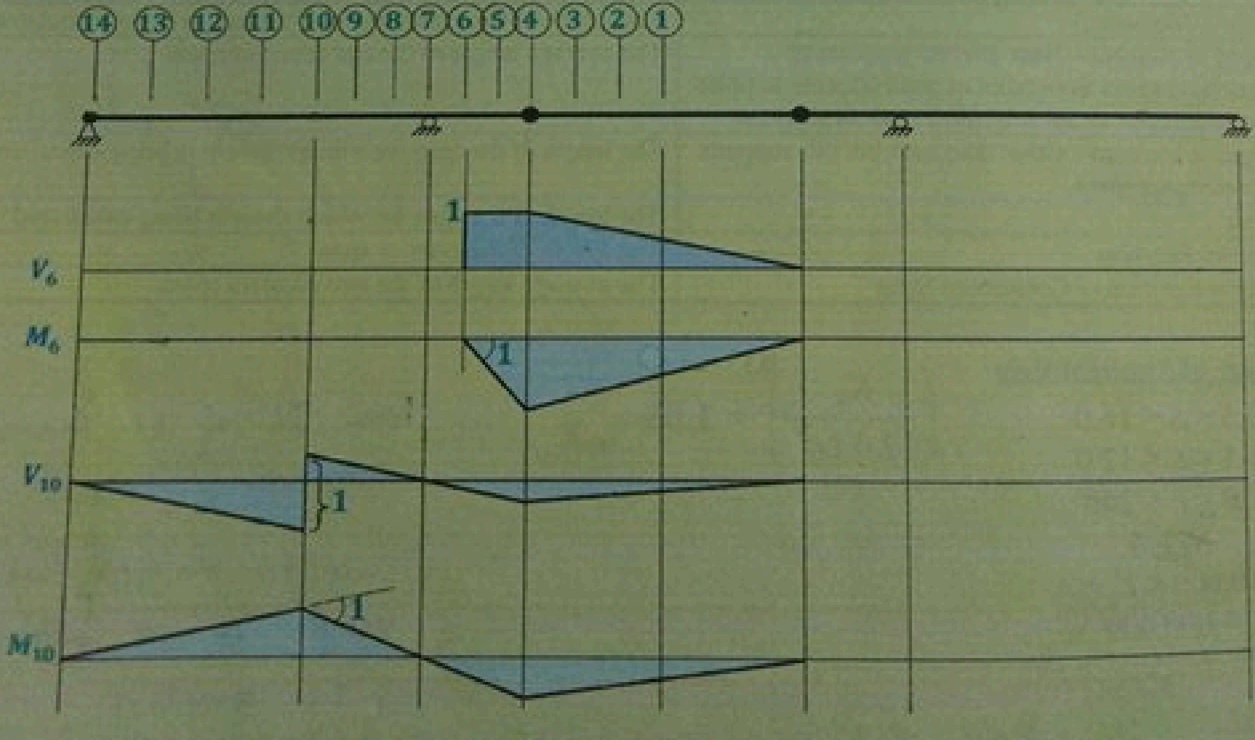
I.L. FOR SHEAR AND MOMENT AT ALL 14 DESIGN SECTIONS NEED TO BE DRAWN

A FEW ARE SHOWN BELOW.....



INTERIOR GIRDER : LIVE LOAD ANALYSIS

INFLUENCE LINES CONTD......



IL of M = 12 diagrams [M = 0 at 4 and 14], IL of V=15 diagrams [two at 7]

Qualitative Influence Lines for Statically Indeterminate Structures

Continuous Beam

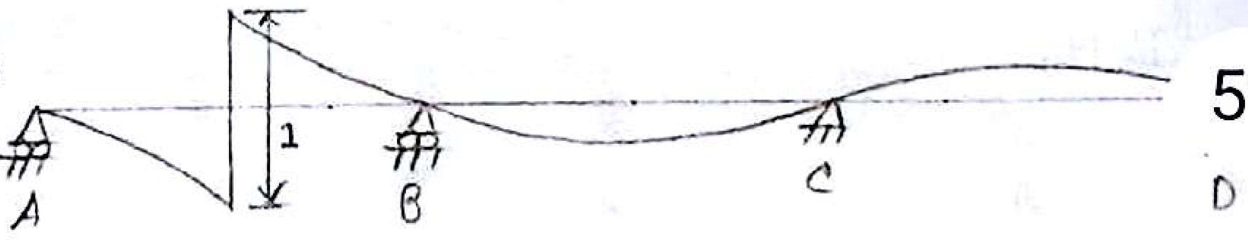
IL-R_A

IL-R_B

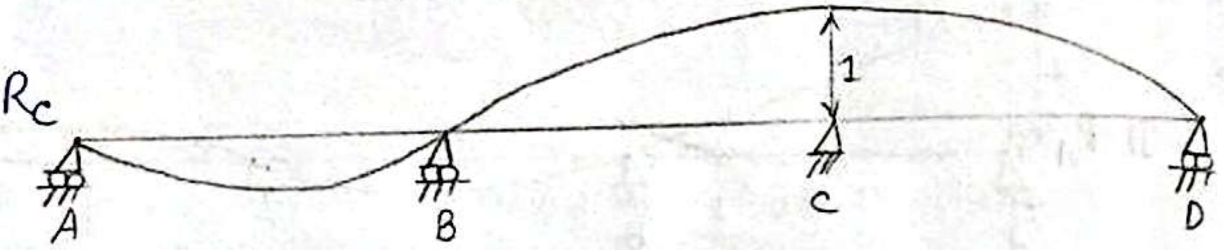
IL-M_x

IL-M_B

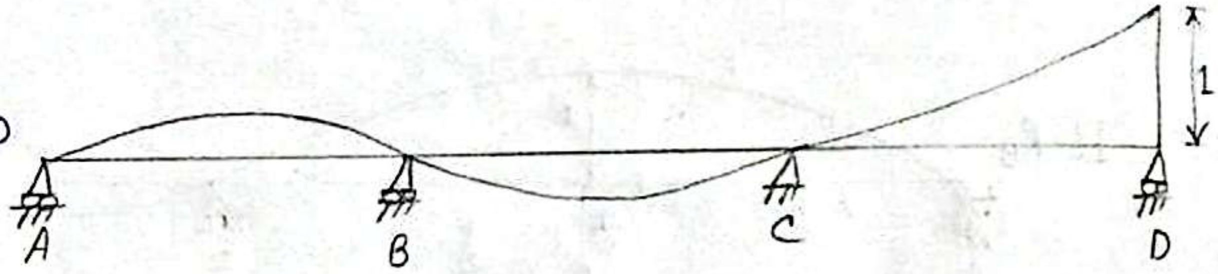
IL-V_x



IL-R_C



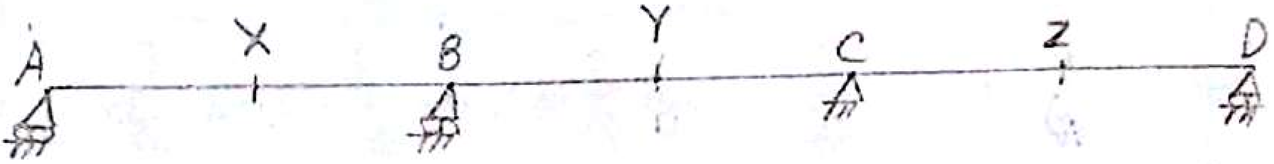
IL-R_D



IL-M_A

IL-M_C

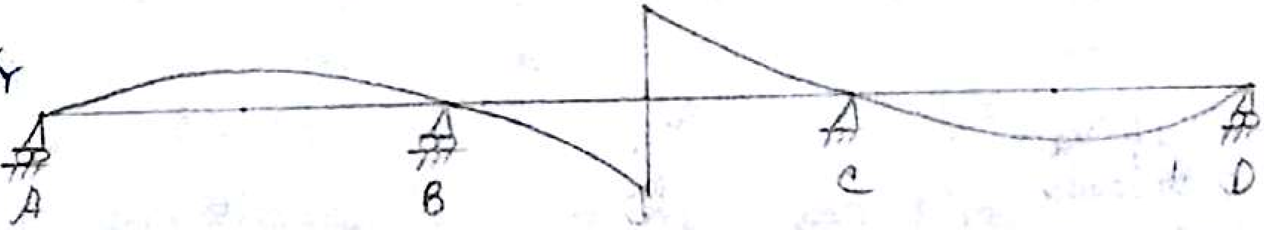
IL-M_D



IL-M_Y

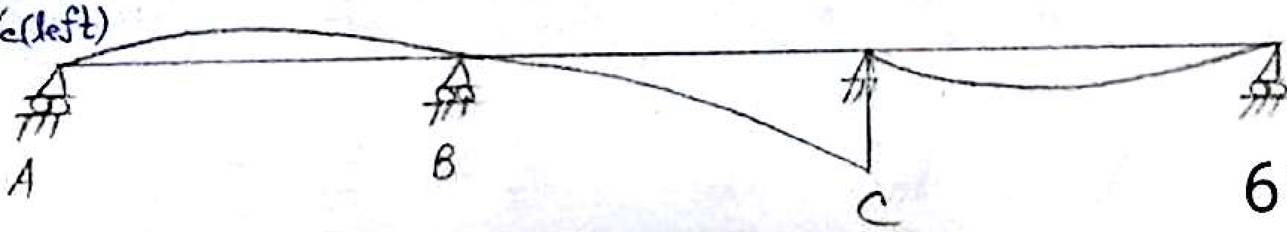
IL-M_Z

IL-V_Y



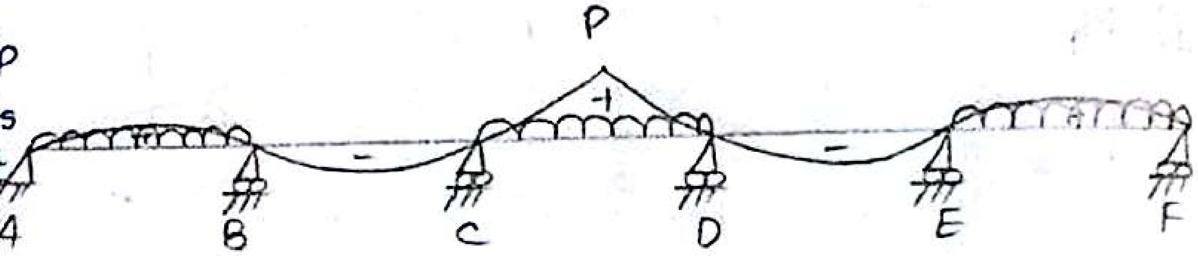
IL-V_Z

IL-$V_{c(left)}$

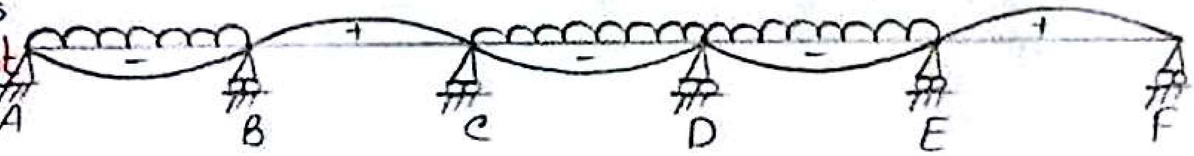


Qualitative Influence Lines & Load Patterns

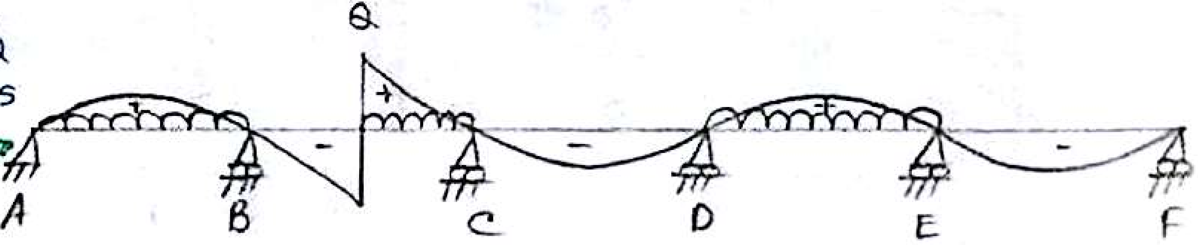
IL-M_P
with loads
for +ve Moment



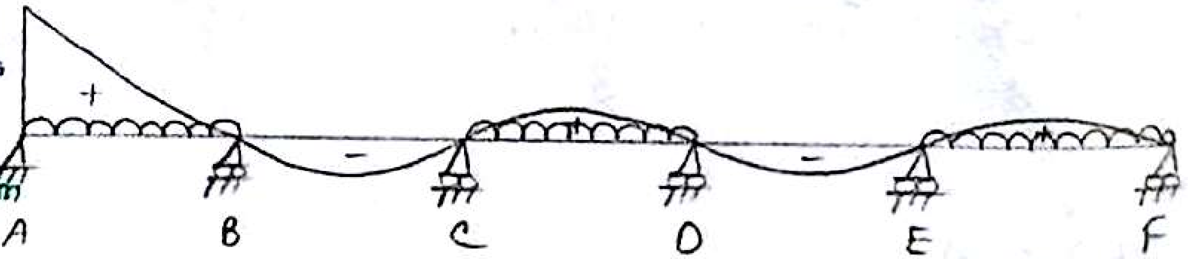
IL-M_D
with loads
for +ve Moment



IL-V_Q
with loads
for Max^m Shear



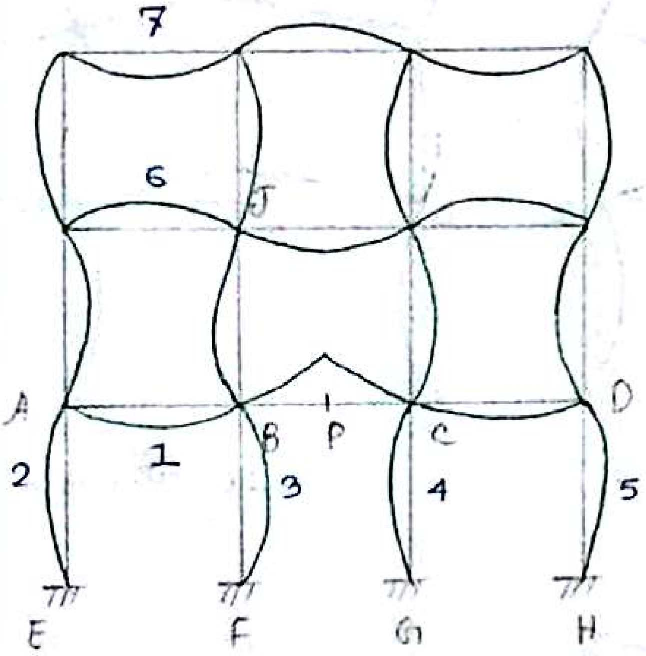
IL-R_A
with loads
for Max^m
value of reaction



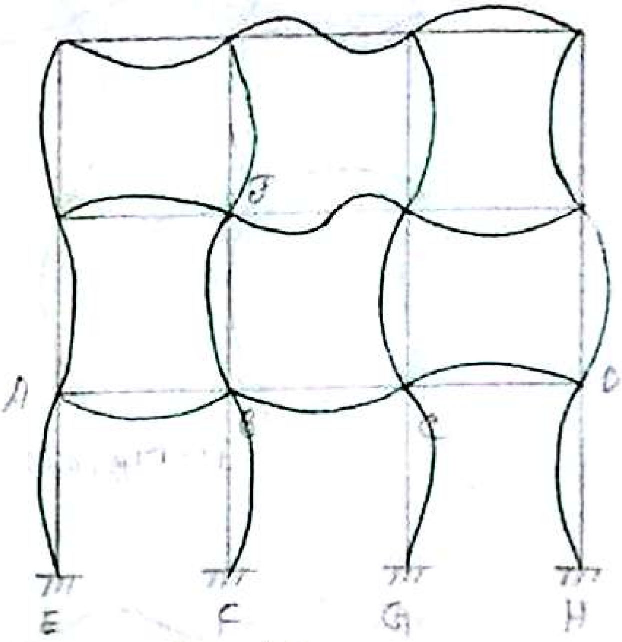
FRAMES

PROBLEM: Draw influence lines for (i) M_P, (ii) M_B of Beam BC (iii) M_B of Beam BA (iv) M_B of Column BF (v) M_B of column BJ (vi) Axial force in column BF (vii) V_P

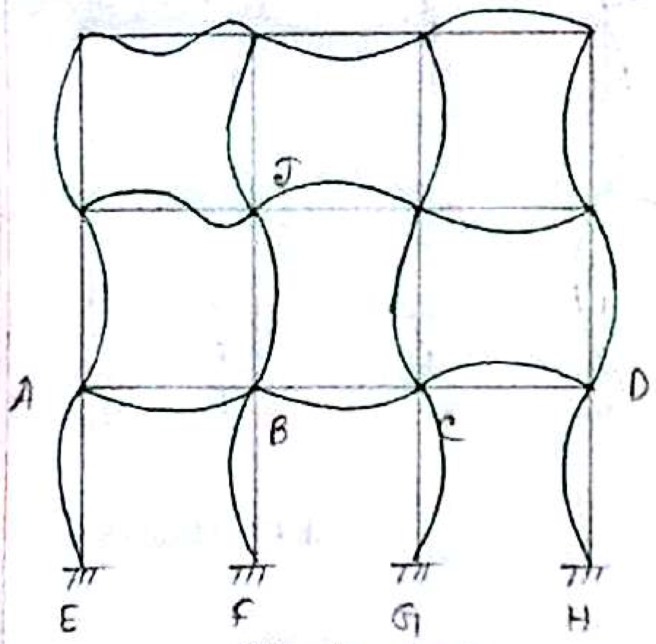
Note: Sequence for drawing IL-1-2 ~3~4~5 ~6~7



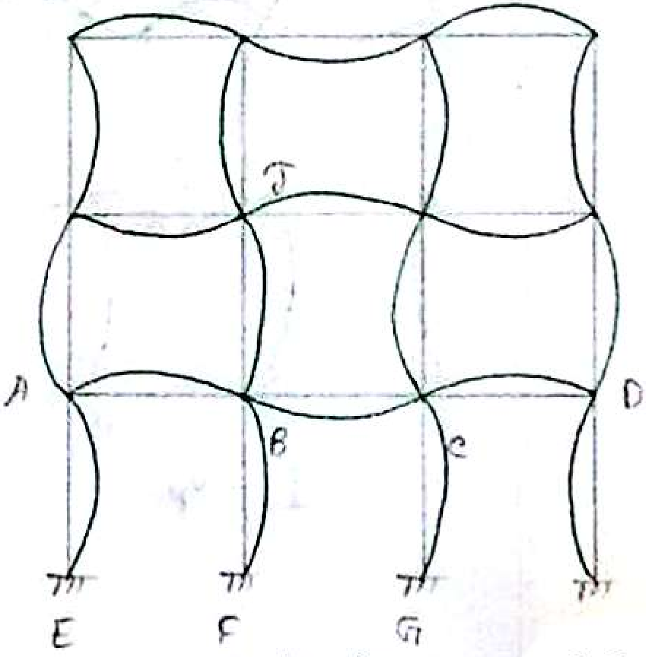
IL- M_P



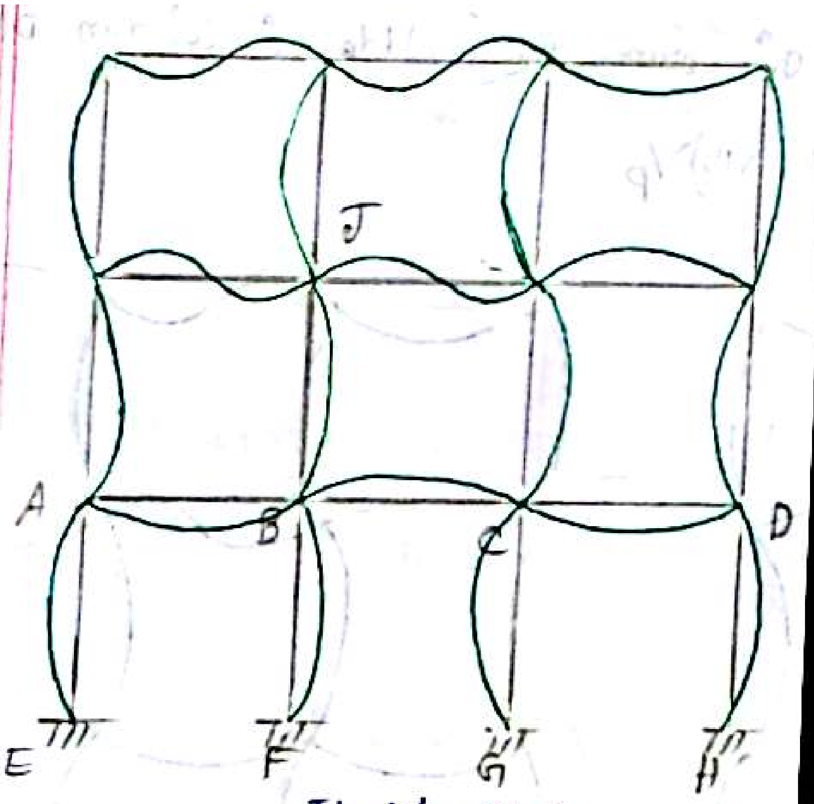
IL- M_B (BC)



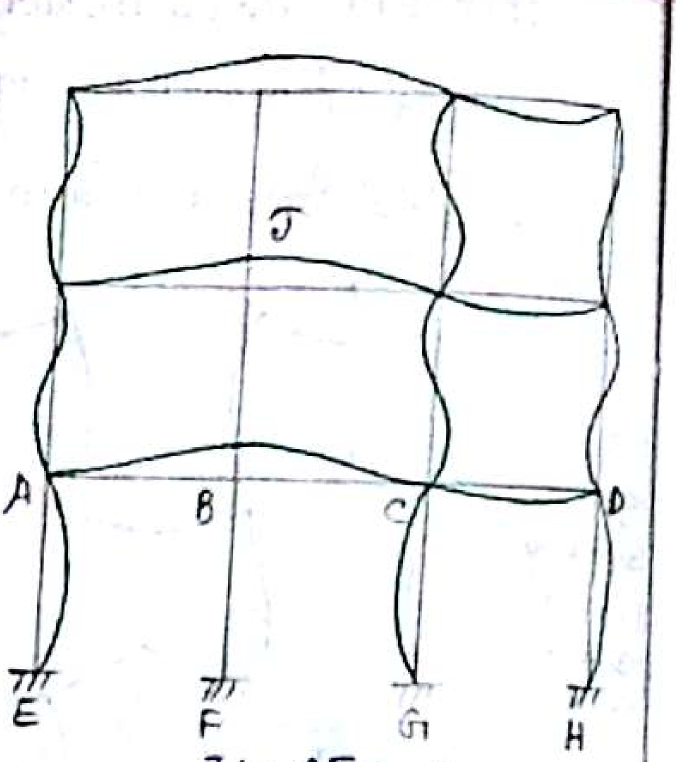
IL- M_B (BA)



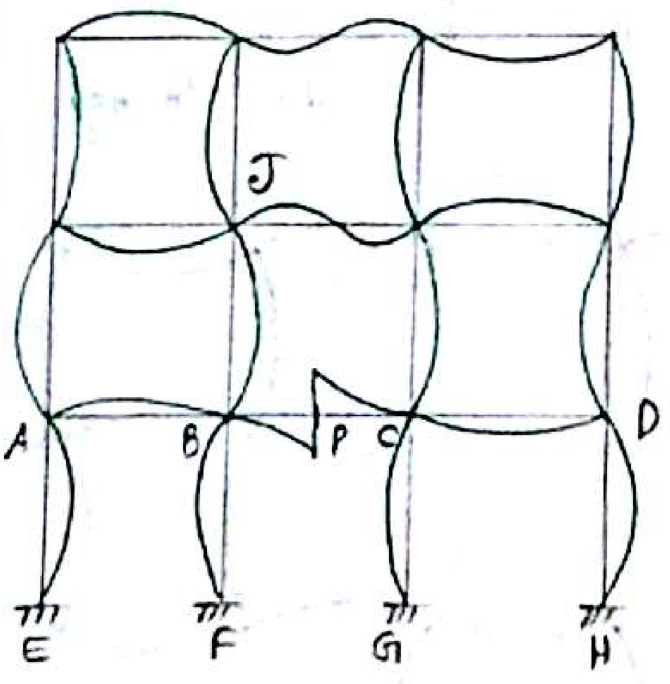
IL- M_B (BF)



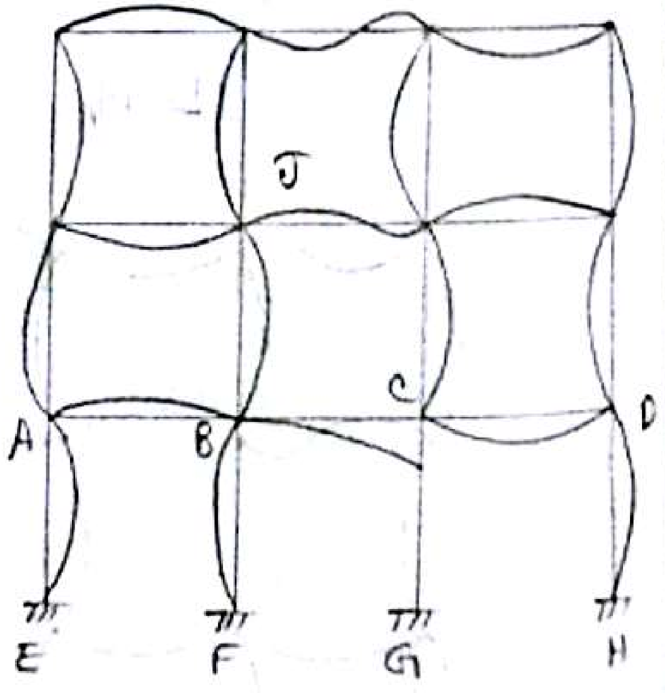
IL-M_B (BJ)



IL- AF. (Bf)



IL-V_P

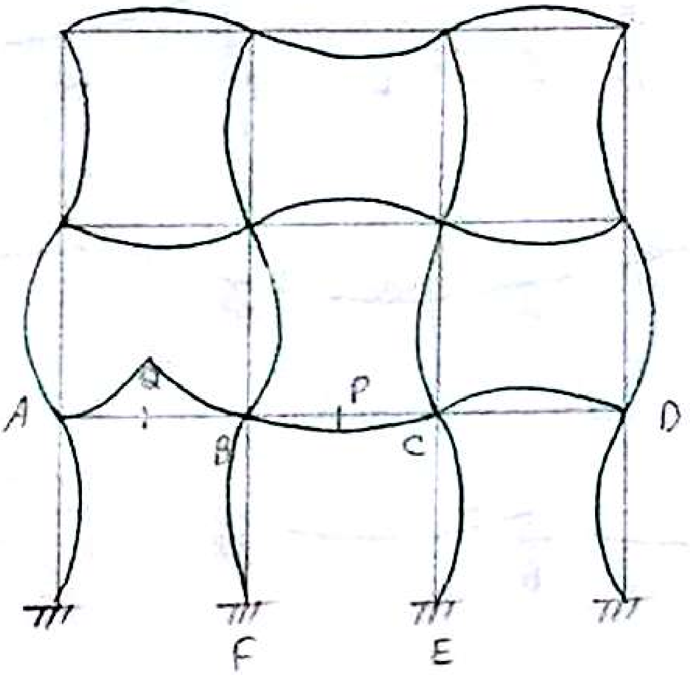


IL-V_c (left)

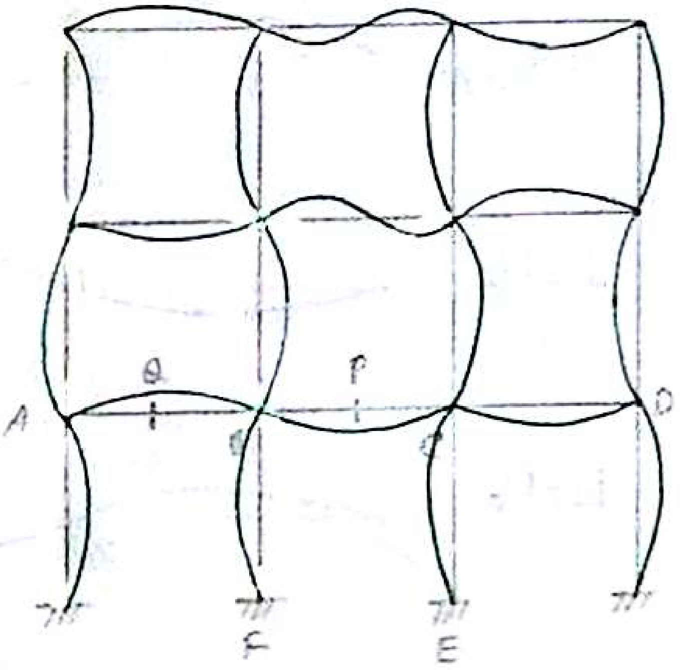
Question Paper Solutions

"2014-15"

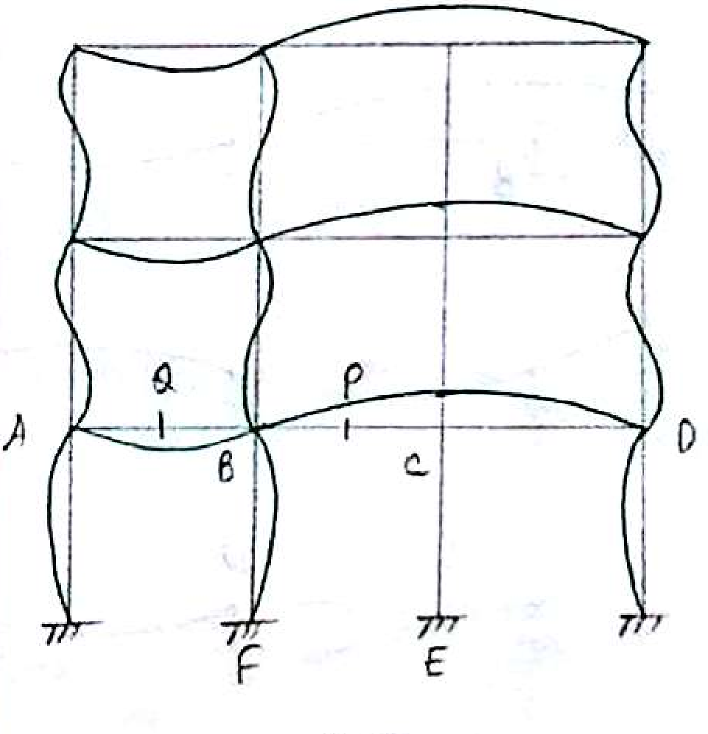
2.(b)



IL-M_Q



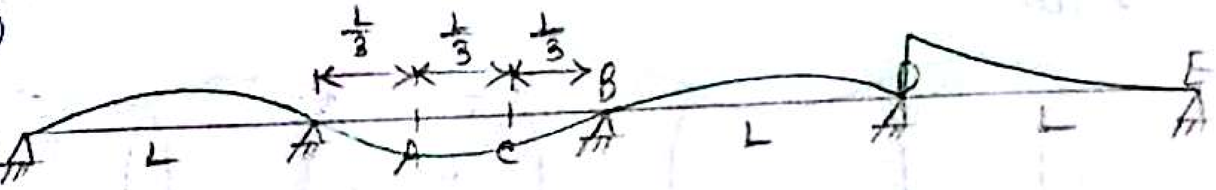
IL-$M_{C(BC)}$



IL-A.F.(CE)

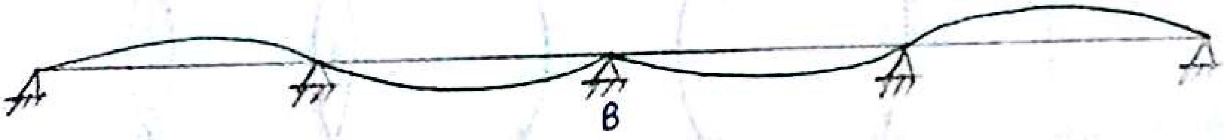
7.(b)

IL-$V_{D(right)}$



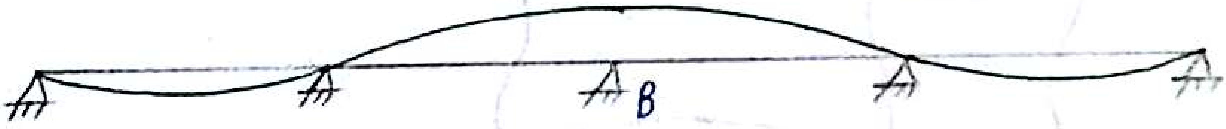
IL-M_A

IL-M_B

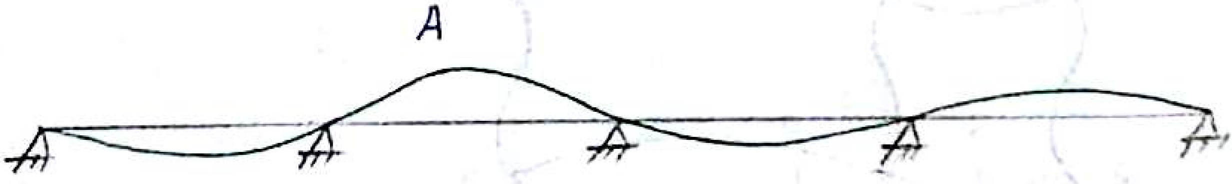


IL-V_A

IL-R_B



IL-Δ_A



IL-$V_{D(left)}$

Example 7-4

Consider earlier Example 7-2 and determine the internal system of forces at sections *a–a* and *b–b*; see Fig. 7-15(a).

SOLUTION.

A free body for the member, including reactions, is shown in Fig. 7-15(a). A free body to the left of section *a–a* in Fig. 7-15(b) shows the maximum ordinate for the isolated part of the applied load. From equilibrium conditions,

$$V_a = -9 + \frac{1}{2} \times 2 \times \frac{2}{3} \times 10 = -2.33 \text{ kN}$$

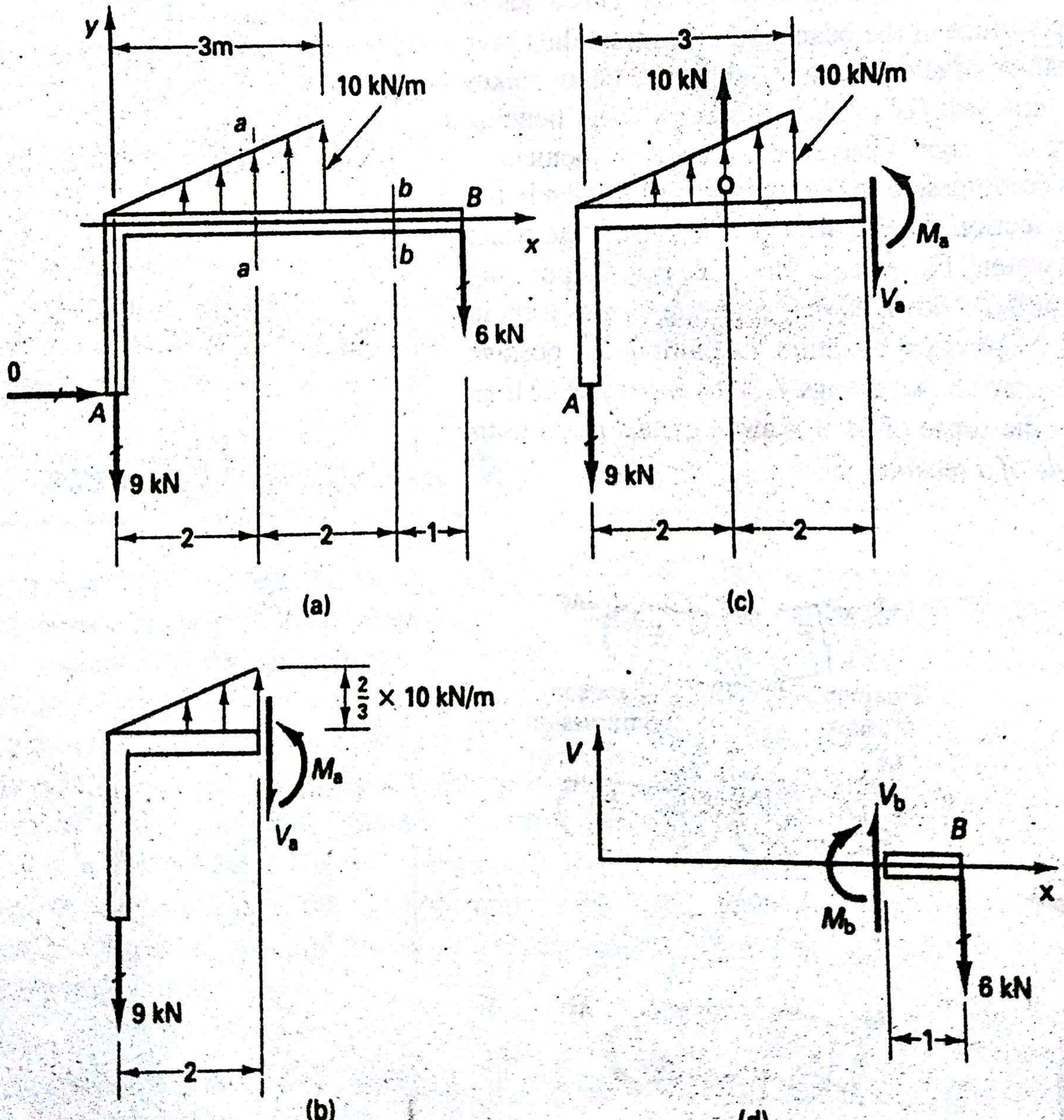


Fig. 7-15

and

$$M_a = -9 \times 2 + \frac{1}{2} \times 2 \times \frac{2}{3} \times 10 \times \frac{1}{3} \times 2 = -13.6\ \text{kN} \cdot \text{m}$$

These forces have opposite senses from those shown in the figure.

A free body to the left of section *b–b* is shown in Fig. 7-15(c), and to the right in Fig. 7-15(d). It is evident that the second free body is simpler for calculations, giving directly

$$V_b = +6\ \text{kN}$$

and

$$M_b = -6 \times 1 = -6\ \text{kN} \cdot \text{m}$$

The same procedure can be used for frames consisting of several members rigidly joined together as well as for curved bars. In all such cases, the sections must be perpendicular to the axis of a member.

ples, algebraic expressions for these functions along a beam will be given.

A systematic method for rapidly constructing shear and moment diagrams will be discussed in the next part of this chapter.

Example 7-5

Construct axial-force, shear, and bending-moment diagrams for the beam shown in Fig. 7-16(a) due to the inclined force $P = 5$ k.

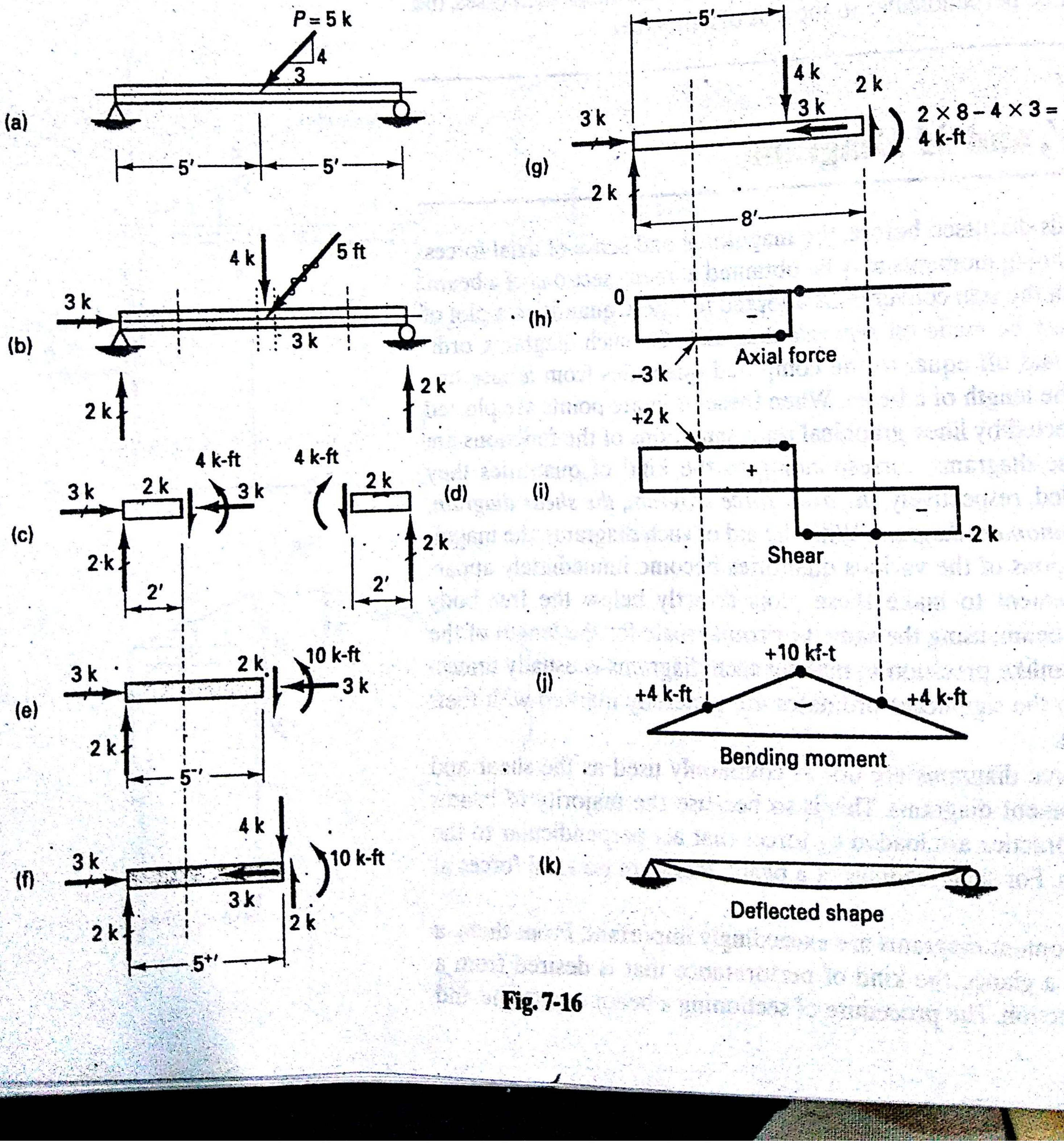


Fig. 7-16

Example 7-6

Determine axial-force, shear, and bending-moment diagrams for the cantilever loaded with an inclined force at the end; see Fig. 7-17(a).

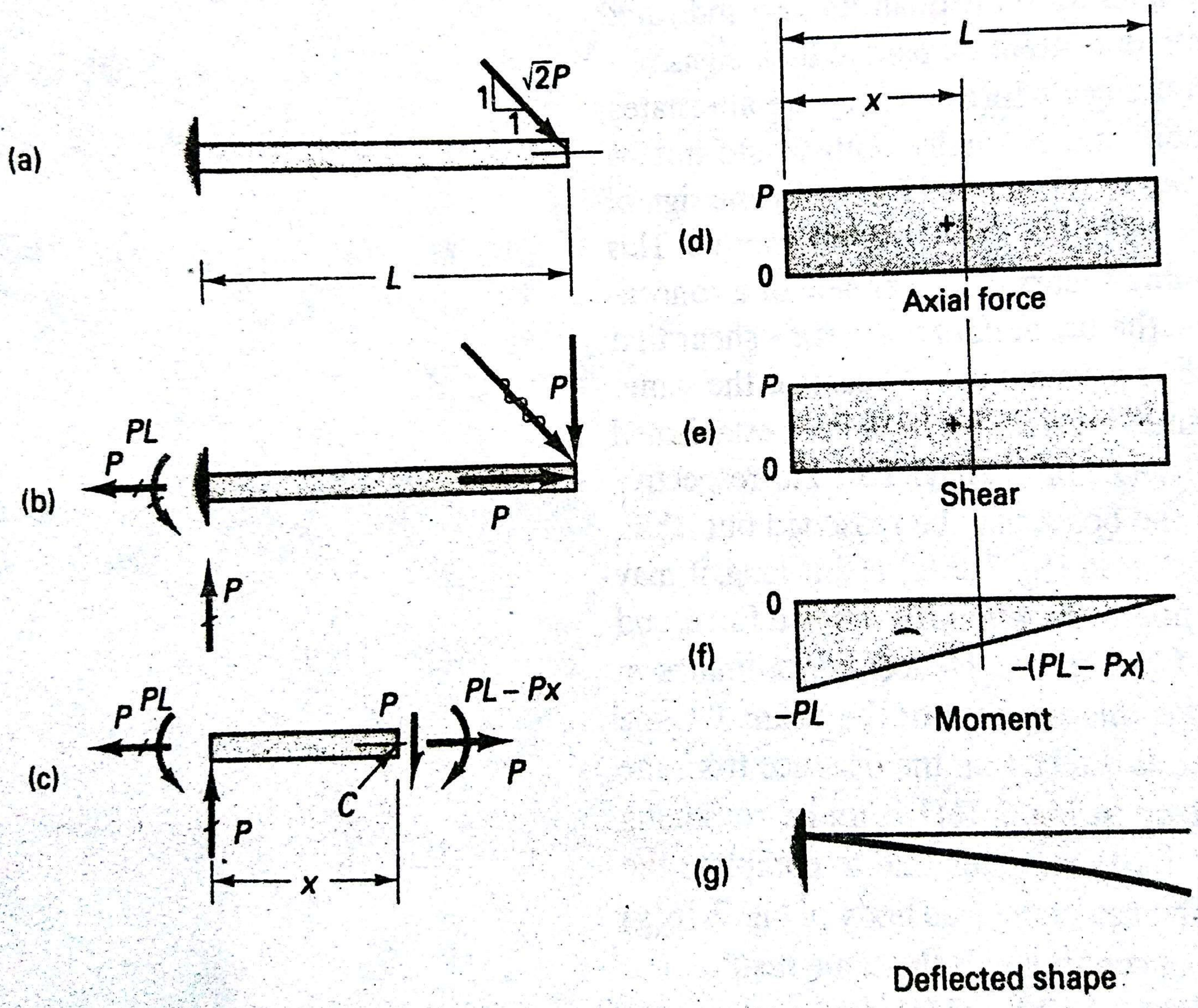
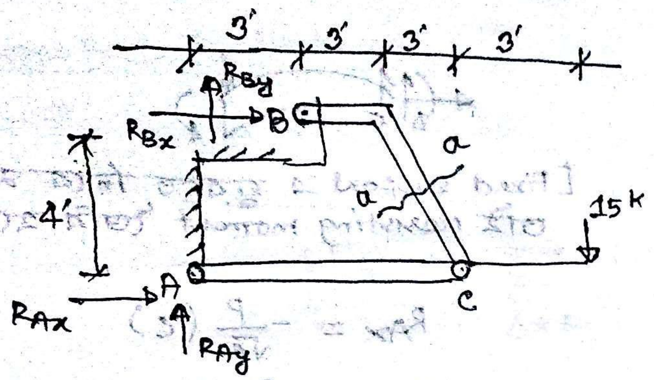


Fig. 7-17

SOLUTION

First, the inclined force is replaced by the two components shown in

Problem (HW)



Find

Internal forces

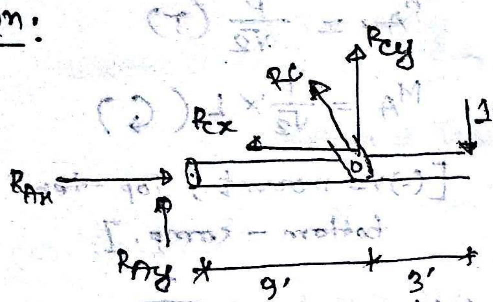
Shear force

Axial force

Bending moment

at sec a-a

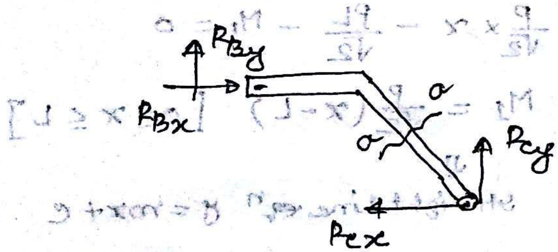
Solⁿ:



$\overset{+}{\circlearrowright}\Sigma M_A = 0$

$\Rightarrow 15\times12 - R_{cy}\times 9 = 0$

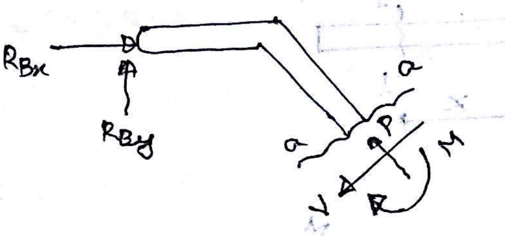
$\therefore R_{cy} = 20^k (\uparrow)$



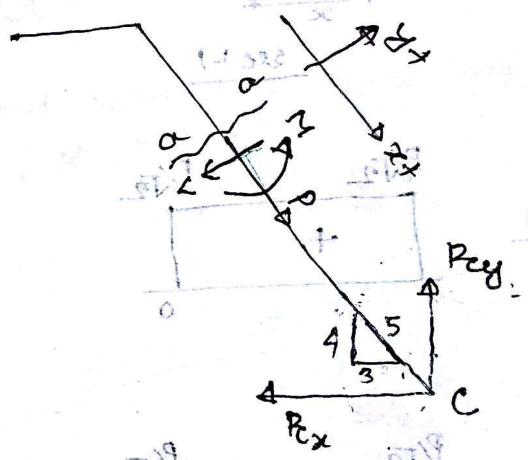
$\overset{+}{\circlearrowright}\Sigma M_B = 0$

$\Rightarrow R_{cx}\times 4 - 20\times 6 =$

$\therefore R_{cx} = 30^k (\leftarrow)$



[+ Internal section কাটলে তবে force আসবে]



$\overset{+}{\rightarrow}\Sigma F_x = 0$

$\Rightarrow P - R_{cx}\times\frac{3}{5} - R_{cy}\times\frac{4}{5} = 0$

$\boxed{\therefore P = 34^k (\rightarrow)}$

$\overset{+}{\uparrow}\Sigma F_y = 0$

$\Rightarrow -V - R_{cx}\times\frac{4}{5} + R_{cy}\times\frac{3}{5} = 0$

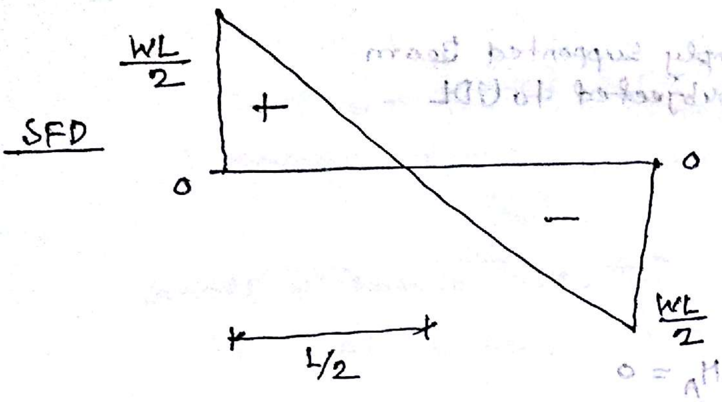
$\boxed{\therefore V = -12^k (\uparrow)}$

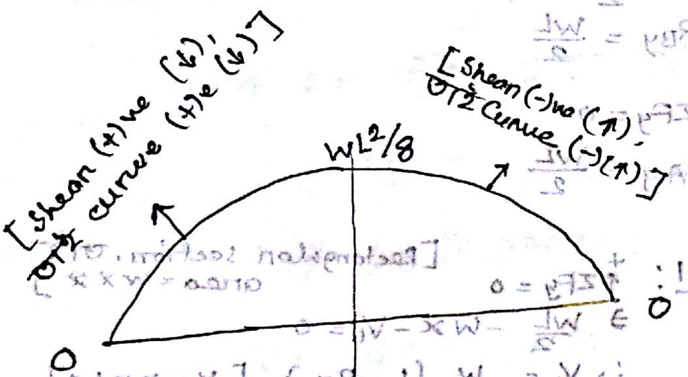
$\overset{+}{\circlearrowright}\Sigma M_{a-a} = 0$

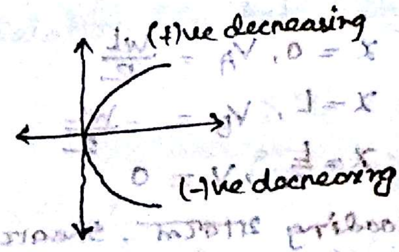
$\Rightarrow R_{cx}\times 2 - R_{cy}\times 1.5 - M = 0$

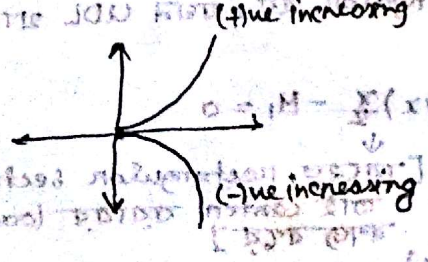
$\boxed{\therefore M = 30\text{ k-ft} (\circlearrowleft)}$

* Summation approach









* upward force (+)ve shear

* $+\frac{WL}{2}$

$-\frac{WL}{2}$

0

$-\frac{WL}{2}$ (downward force. তাই (−)ve]

$-\frac{WL}{2}$

$+\frac{WL}{2}$

0

Or

$+\frac{WL}{2}$

$-WL$

$-WL/2$

$+WL/2$

0

* Total load W

অর্ধেক $\frac{WL}{2}$

* Loading diagram এর যোগফলই হচ্ছে shear এর value

* UDL হলে shear diagram linear

* SFD থেকে BMD আঁকব

SFD এর area এবং moment

(+)ve shear হলে (+)ve moment

* $\frac{1}{2}\times\frac{WL}{2}\times\frac{L}{2}$

$\Downarrow$

$\frac{WL^3}{8}$

$-\frac{WL^2}{8}$ $(-\frac{1}{2}\cdot\frac{WL}{2}\cdot\frac{L}{2})$

0

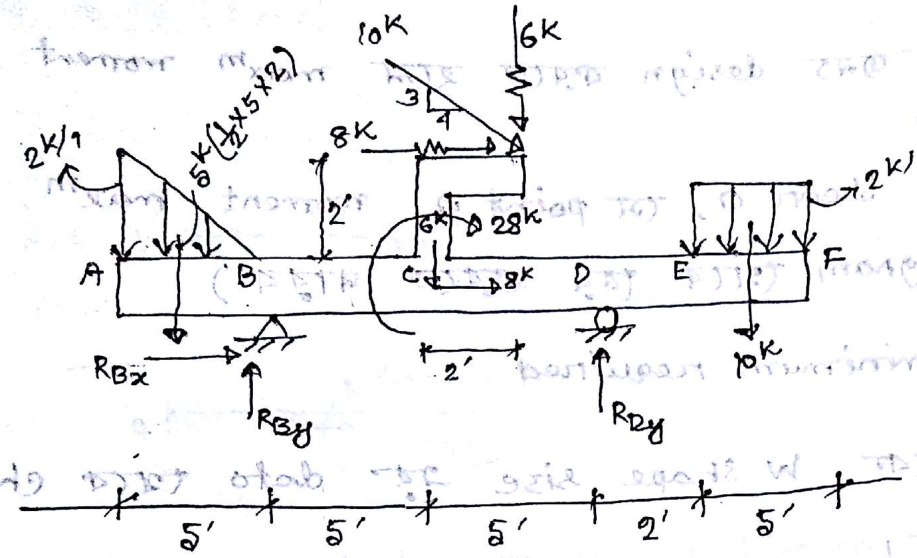
* Area যোগ করতে হবে. তাহলে moment 0 হবে.

* SFD linear হলে MD হবে 1st order parabola

$y = mx \rightarrow y = mx^2$

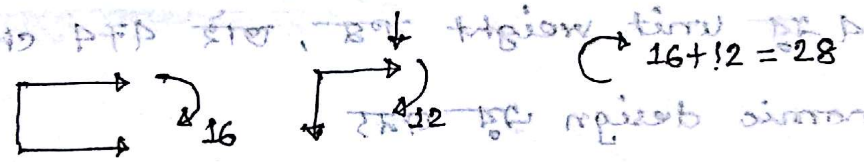
যদি $y = mx^2$ হয় $\rightarrow$ $y = mx^3$ হবে

⊞ Problem



Solⁿ:

parallelly force c point এ নামাই আনলে Moment দিবে।



$\Sigma F_x = 0$

$\Rightarrow R_{Bx} = -8^k$ (←)

$\circlearrowleft^{+} \Sigma M_B = 0$

$\Rightarrow -3.33\times5 + 6\times5 + 28 - R_{Dy}\times10 + 10\times14.5 = 0$

$(\frac{2}{3}5)$

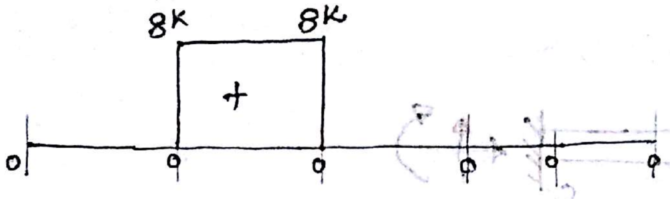
$\therefore R_D = 18.635^k$

$\Sigma F_y = 0$

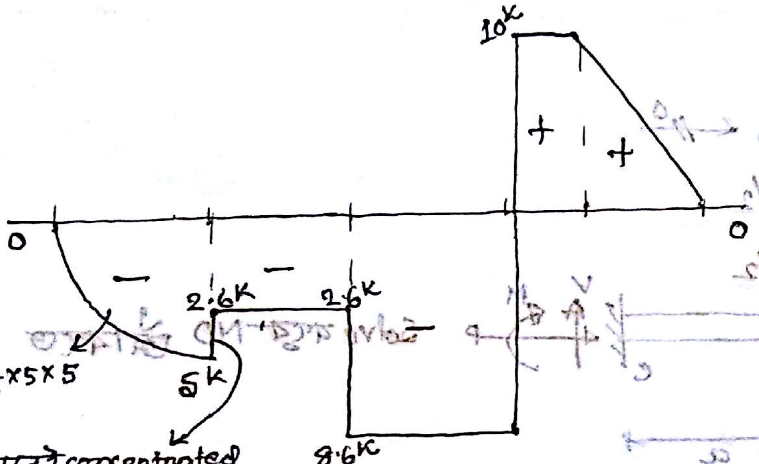
$\Rightarrow -5 + R_{By} - 6 + 18.635 - 10 = 0$

$\therefore R_{By} = 2.4^K$ (↑)

AFD (K)

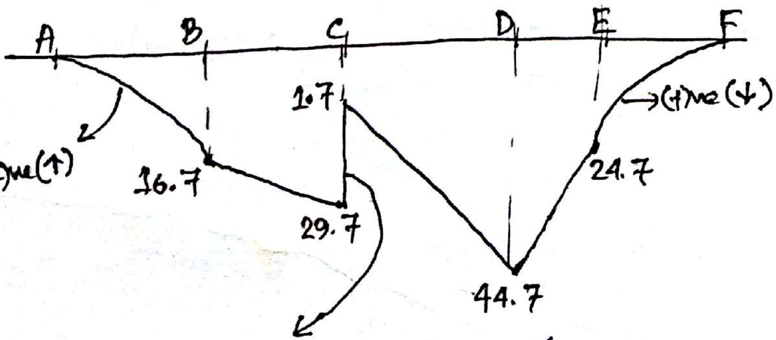


SFD (K)



[যেখানেই concentrated force সেখানেই jump of shear]

* BMD (k-ft)



[যেখানেই concentrated moment সেখানেই jump]

AFD (K)

at B → +8K
at C → −8K
0

SFD (K)

−5K (B point এর just to the left)
+2.4K (B " " " " " right)
−2.6K (C এর just left)
−6K (C " " right)
−8.6K
+18.6K
+10K
−10K
0

[upward force (+)ve shear
downward force (−)ve shear]

(ultimately 0 আসতে হবে)

[যদি concentrated force ও moment থাকে তাহলে just to the left ও right এ হিসাব করতে হবে]

* B ও C এর মাঝে কোন vertical force নাই। তাই shear same থাকে।

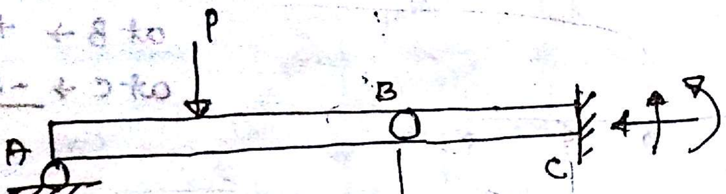
* UDL হলে linear
UVL হলে parabolic

** BMD আঁকার সময় concentrated moment (28K) কিনা খেয়াল রাখতে হবে। C point এ concentrated moment আছে।

BMD

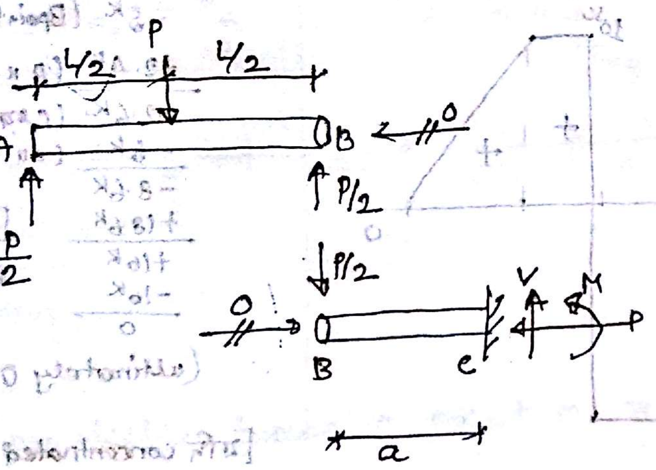
−16.7
−13.0
−29.7
+28
−1.7K
−43
−44.7
+20
−24.7
+25
+0.3 ≈ 0

[Rounding এর জন্য 0.3 আসে, তাই 0 ধরতে হবে]



↳Internal hinge (~~ভিতরে~~ ball bearing দেয়া আছে)

↳ moment 0



Solve করে MD আঁকতে হবে।

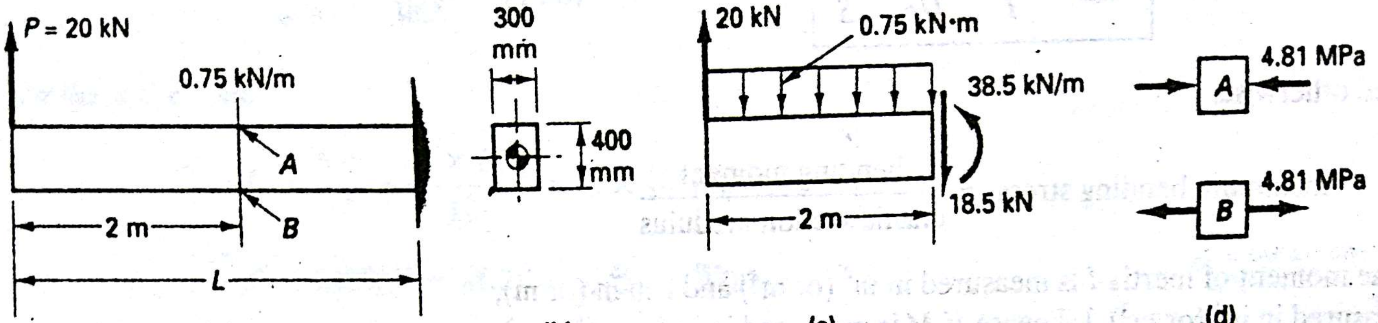
$V = \frac{P}{2}$ (↑)

$P = 0$

$M = -\frac{Pa}{2}$ (2)

Example 8-4

A 300-by-400-mm wooden cantilever beam weighing 0.75 kN/m carries an upward concentrated force of 20 kN at the end, as shown in Fig. 8-12(a). Determine the maximum bending stresses at a section 2 m from the free end.



(a) (b) (c) (d)

Figure 8-12

SOLUTION

A free-body diagram for a 2-m segment of the beam is shown in Fig. 8-12(c). To keep this segment in equilibrium requires a shear of $20 - (0.75 \times 2) = 18.5$ kN and a bending moment of $(20 \times 2) - (0.75 \times 2 \times 1) = 38.5$ kN · m at the section. Both of these quantities are shown with their proper sense in Fig. 8-12(c). The distance from the neutral axis to the extreme fibers $c = 200$ mm. This is applicable to both the tension and the compression fibers.

From Eq. 8-19,

$$I_z = \frac{bh^3}{12} = \frac{300 \times 400^3}{12} = 16 \times 10^8 \text{ mm}$$

From Eq. 8-13,

$$\sigma_{\max} = \frac{Mc}{I} = \frac{38.5 \times 10^6 \times 200}{16 \times 10^8} = \pm 4.81 \text{ MPa}$$

From the sense of the bending moment shown in Fig. 8-12(c), the top fibers of the beam are seen to be in compression and the bottom ones in tension. In the answer given, the positive sign applies to the tensile stress and the negative sign applies to the compressive stress. Both of these stresses decrease at a linear rate toward the neutral axis, where the bending stress is zero. The normal stresses acting on infinitesimal elements at *A* and *B* are shown in Fig. 8-12(d). It is important to learn to make such a representation of an element as it will be frequently used in Chapters 11 and 13.

ALTERNATIVE SOLUTION

If only the maximum stress is desired, the equation involving the section modulus may be used. The section modulus for a rectangular section in algebraic form is

$$S = \frac{I}{c} = \frac{bh^3}{12}\frac{2}{h} = \frac{bh^2}{6} \qquad (8\text{-}22)$$

In this problem, $S = 300 \times 400^2/6 = 8 \times 10^6$ mm^3, and by Eq. 6-21,

$$\sigma_{max} = \frac{M}{S} = \frac{38.5 \times 10^6}{8 \times 10^6} = 4.81 \text{ MPa}$$

Both solutions lead to identical results.

Example 9-5

Find the stress distribution at section $ABCD$ for the block shown in mm in Fig. 9-11(a) if $P = 64$ kN. At the same section, locate the line of zero stress. Neglect the weight of the block.

SOLUTION

In this problem, it is somewhat simpler to recast Eq. 9-5 with the aid of Eq. 8-22, defining the elastic section modulus $S = I/c$ as $bh^2/6$. The normal stress at any Ith corner of the block can be found directly from such a transformed equation. This equation reads

$$\sigma_I = \frac{P}{A} - \frac{M_z}{S_z} + \frac{M_y}{S_y} \tag{9-8}$$

where $S_z = bh^2/6$ and $S_y = hb^2/6$.

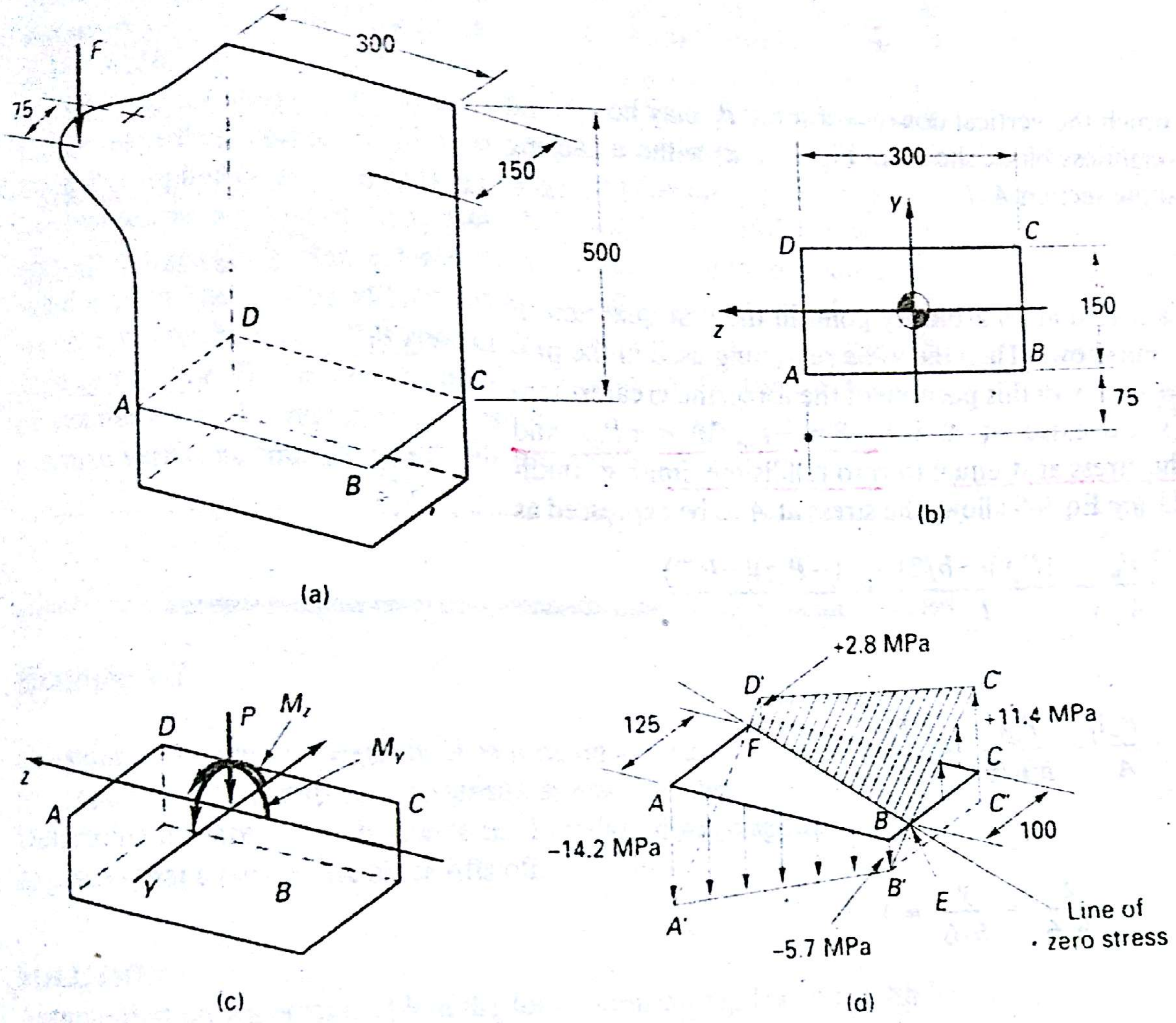
The forces acting on section $ABCD$, Fig. 9-11(c), are $P = -64 \times 10^3$ N, $M_y = -64 \times 10^3 \times 150 = -9.6 \times 10^6$ N · mm, and $M_z = -64 \times 10^3 \times (75 + 75) = -9.6 \times 10^6$ N · mm. The cross-sectional area has the following properties: $A = 150 \times 300 = 45 \times 10^3$ mm^2, $S_z = 300 \times 150^2/6 = 1.125 \times 10^6$ mm^3, and $S_y = 150 \times 300^2/6 = 2.25 \times 10^6$ mm^3.

The normal stresses at the corners are found using Eq. 9-8, assigning signs for the stresses caused by moments by inspection. For example, from Fig. 9-11(c), it can be seen that due to M_y, the stresses at corners A and D are compressive. Other cases are treated similarly. Using this approach,

$$\sigma_A = -\frac{64 \times 10^3}{45 \times 10^3} - \frac{9.6 \times 10^6}{1.125 \times 10^6} - \frac{9.6 \times 10^6}{2.25 \times 10^6}$$

$$= -1.42 - 8.53 - 4.27 = -14.2 \text{ MPa}$$

$$\sigma_B = -1.42 - 8.53 + 4.27 = -5.7 \text{ MPa}$$



(a) (b) (c) (d)

Fig. 9-11

$$\sigma_C = -1.42 + 8.53 + 4.27 = +11.4 \text{ MPa}$$

$$\sigma_D = -1.42 + 8.53 - 4.27 = +2.8 \text{ MPa}$$

These stresses are shown in Fig. 9-11(d). The ends of these four stress vectors at A', B', C', and D' lie in the plane $A'B'C'D'$. The vertical distance between planes $ABCD$ and $A'B'C'D'$ defines the total stress on any point on the cross section. The intersection of plane $A'B'C'D'$ with plane $ABCD$ locates the line of zero stress FE.

By drawing a line $B'C''$ parallel to BC, similar triangles $C'B'C''$ and $C'EC$ are obtained; thus, the distance $CE = [11.4/(11.4 + 5.7)]150 = 100$ mm. Similarly, distance AF is found to be 125 mm. Points E and F locate the line of zero stress. If the weight of the block is neglected, the stress distribution on any other section parallel to $ABCD$ is the same.

where A_o is the cross-sectional area of the elastic part of the cross section. The shear stress distribution for the elastic-plastic case is shown in Fig. 10-15(c). This can be contrasted with that for the elastic case, shown in Fig. 10-15(d). Since equal and opposite normal stresses occur in the plastic zones, no unbalance in longitudinal forces occurs and no shear stresses are developed.

This elementary solution has been refined by using a more carefully formulated criterion of yielding caused by the simultaneous action of normal and shear stresses.[8]

Example 10-5

An I beam is loaded as shown in Fig. 10-16(a). If it has the cross section shown in Fig. 10-16(c), determine the shear stresses at the levels indicated. Neglect the weight of the beam.

SOLUTION

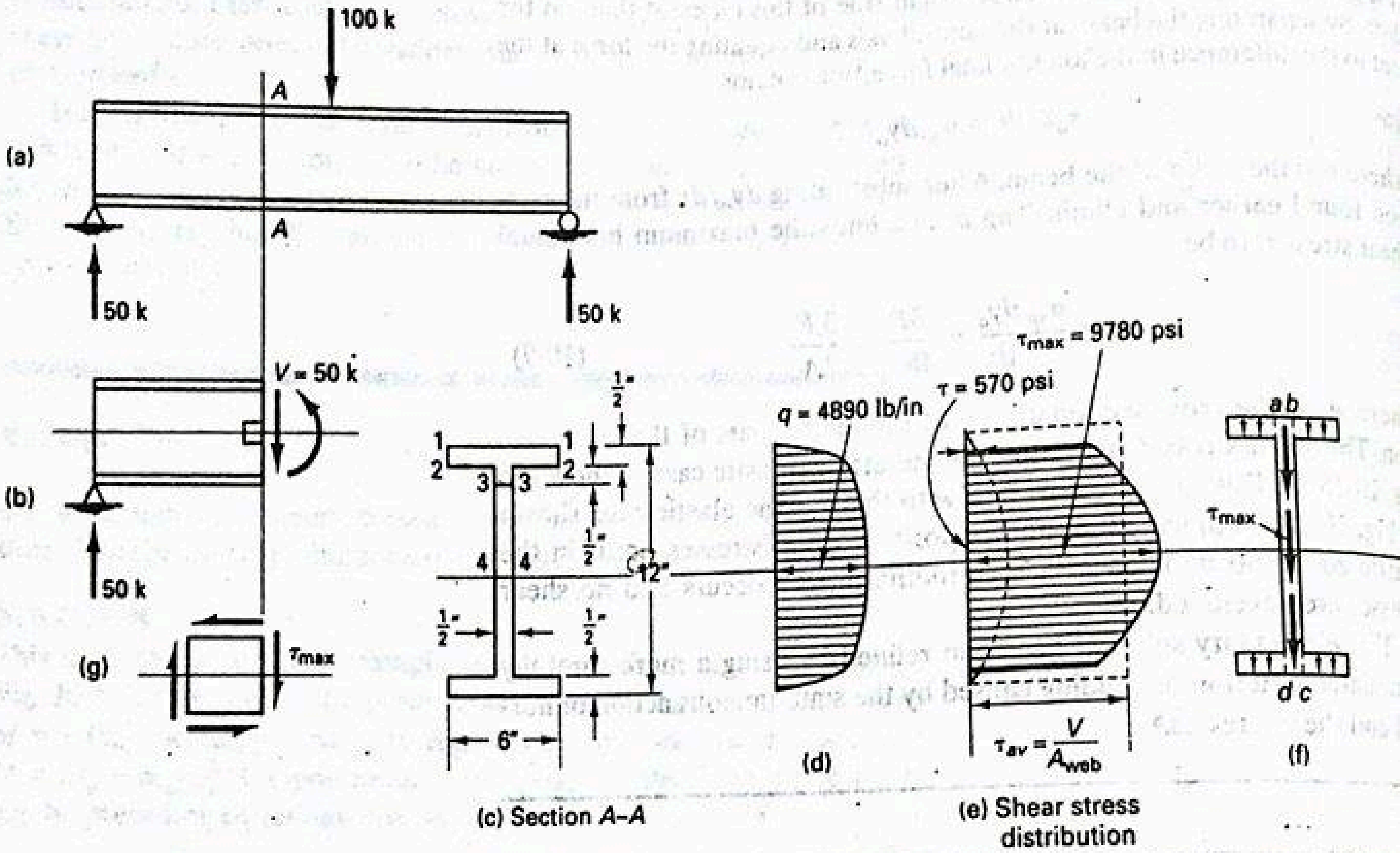
From the free-body diagram of the beam segment in Fig. 10-16(b), it is seen that the vertical shear at all sections is 50 kips. Bending moments do not enter directly into the present problem. The shear flow at the various levels of the beam is computed in the following table using Eq. 10-5. Since $\tau = q/t$, from Eq. 10-6, the shear stresses are obtained by dividing the shear flows by the respective widths of the beam:

$$I = 6 \times 12^3/12 - 5.5 \times 11^3/12 = 254 \text{ in}^4$$

For use in Eq. 10-5, the ratio $V/I = 50{,}000/254 = 197 \text{ lb/in}^4$. Some results at selected horizontal sections are given in the following table:

---

[8] D. C. Drucker, "The effect of shear on the plastic bending of beams," *J. Appl. Mech.*, **23** (1956), 509–514.



(c) Section A–A

(e) Shear stress distribution

Fig. 10-16

Level	A_{fghj}[a]	y[b]	$Q = A_{fghj}y$		$q = VQ/I$	t	τ (psi)
1–1	0	6		0	0	6.0 6.0	0 570
2–2	0.5 × 6 = 3.00	5.75		17.25	3400	0.5	6800
3–3	0.5 × 6 = 3.00 0.5 × 0.5 = 0.25	5.75 5.25	17.25 1.31	18.56	3650	0.5	7300
4–4	0.5 × 6 = 3.00 0.5 × 5.5 = 2.75	5.75 2.75	17.25 7.56	24.81	4890	0.5	9780

[a] A_{fghj} is the partial area of the cross section above a given level in in^2.

[b] y is distance in mm from the neutral axis to the centroid of the partial area.

The positive signs of τ show that, for the section considered, the stresses act downward on the right face of the elements. This sense of the shear stresses coincides with the sense of shear force V. For this reason, a strict adherence to the sign convention is often unnecessary. It is always true that $\int_A \tau\, dA$ is equal to V and has the same sense.

Note that at level 2–2, two widths are used to determine the shear stress—one just above the line 2–2, and one just below. A width of 6 in corresponds to the first case, and 0.5 in to the second. This transition point will be discussed in the next section. The results obtained, which by virtue of symmetry are also applicable to the lower half of the section, are plotted in Figs. 10-16(d) and (e). By a method similar to the one used in the preceding example, it may be shown that the curves in Fig. 10-16(e) are parts of a second-degree parabola.

The variation of the shear stress indicated by Fig. 10-16(e) may be interpreted as shown in Fig. 10-16(f). The maximum shear stress occurs at the neutral axis, and the vertical shear stresses throughout the web of the beam are nearly of the same magnitude. The vertical shear stresses occurring in the flanges are very small. For this reason, the maximum shear stress in an I beam is often approximated by dividing the total shear *V* by the cross-sectional area of the web, with the web height assumed equal to the beam's overall height, area *abcd* in Fig. 10-16(f). Hence,

$$(\tau_{\max})_{\text{approx}} = \frac{V}{A_{\text{web}}} \tag{10-10}$$

In the example considered, this gives

$$(\tau_{\max})_{\text{approx}} = \frac{50{,}000}{0.5 \times 12} = 8330 \text{ psi}$$

This stress differs by about 15% from the one found by the accurate formula. For most cross sections, a much closer approximation to the true maximum shear stress may be obtained by dividing the shear by the web area between the flanges only. For this example, this procedure gives a stress of 9091 psi, which is an error of only about 8%. It should be clear from the foregoing that division of *V* by the whole cross-sectional area of the beam to obtain the shear stress is not permissible.

An element of the beam at the neutral axis is shown in Fig. 10-16(g). At levels 3–3 and 2–2, bending stresses, in addition to the shear stresses, act on the vertical faces of the elements. No shear stresses and only bending stresses act on the elements at level 1–1.

CE 311 12/09/15/Sat

Structural Stability and Determinancy - Indeterminancy

(a) A structure is statically :

(i) unstable if (total unknown forces) < (total equation)

(ii) Stable & Determinant if (") = (")

(iii) Stable & Indeterminant if (") > (")

and D° of Indeterminancy = (") ⊖ (")

* Geometrical এ determinancy নাই, শুধু unstability.

(b) Total unknown forces = Member forces + R's

$= 1b + r$ for a (2D) truss

(AF)

$= 3m + r$ for a (2D) frame

(AF, SF, BM)

$= r$ for a beam/col.

(c) Total eq^n's = Eqb^m eq^n's for joints + Eq^n of conditions

$\Sigma F_x, \Sigma F_y$

$= 2j + 0$ for a (2D) truss

$= 3j + c$ for a (2D) frame

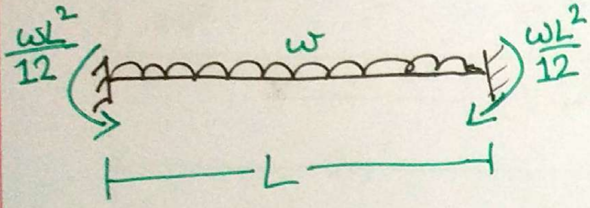
$\Sigma F_x, \Sigma F_y, \Sigma M$

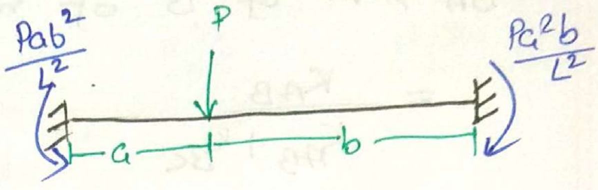
$= 3 + c$ for a beam/ col.

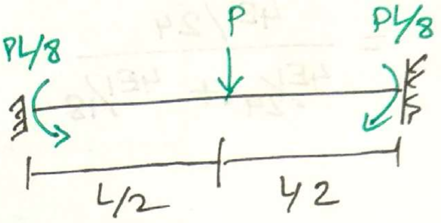
Wed

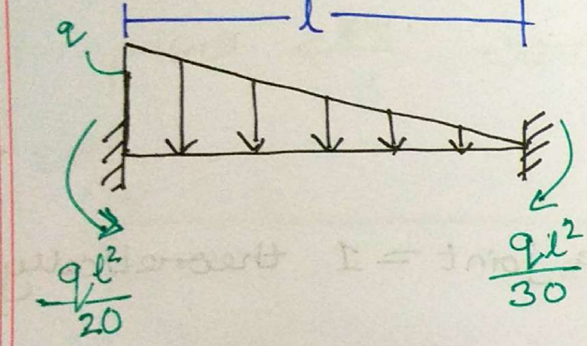
L-4

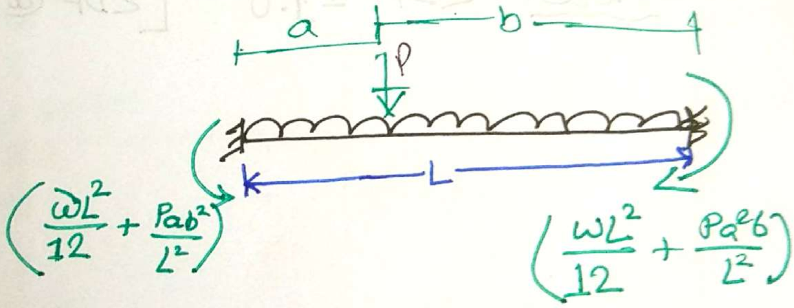
21/09/16



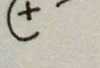


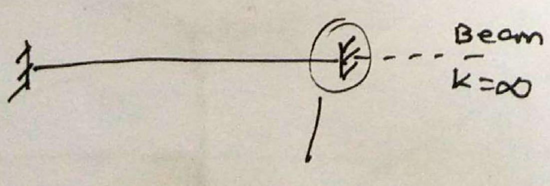




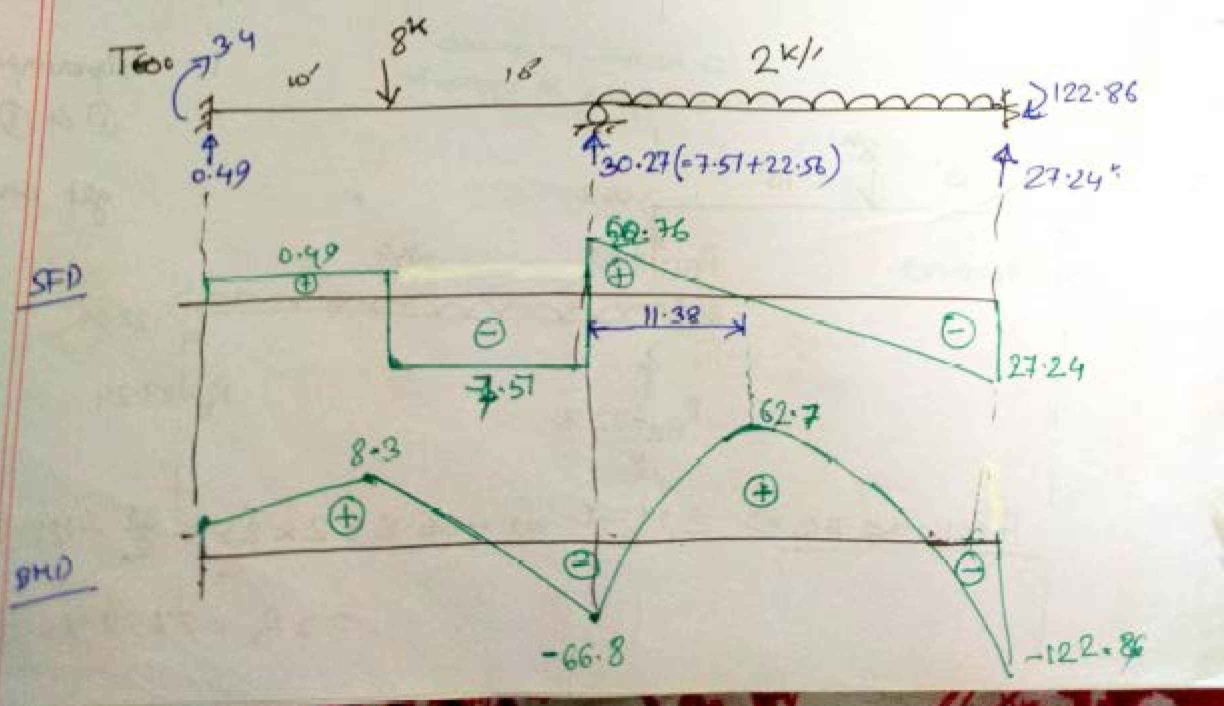


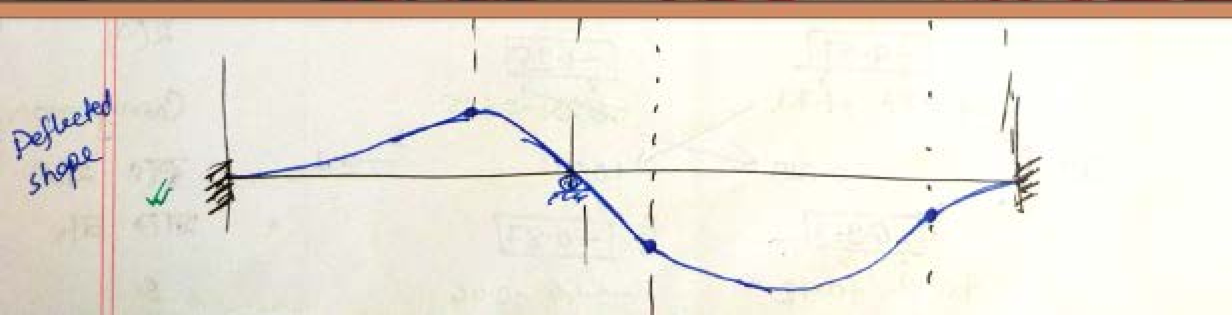
Sign

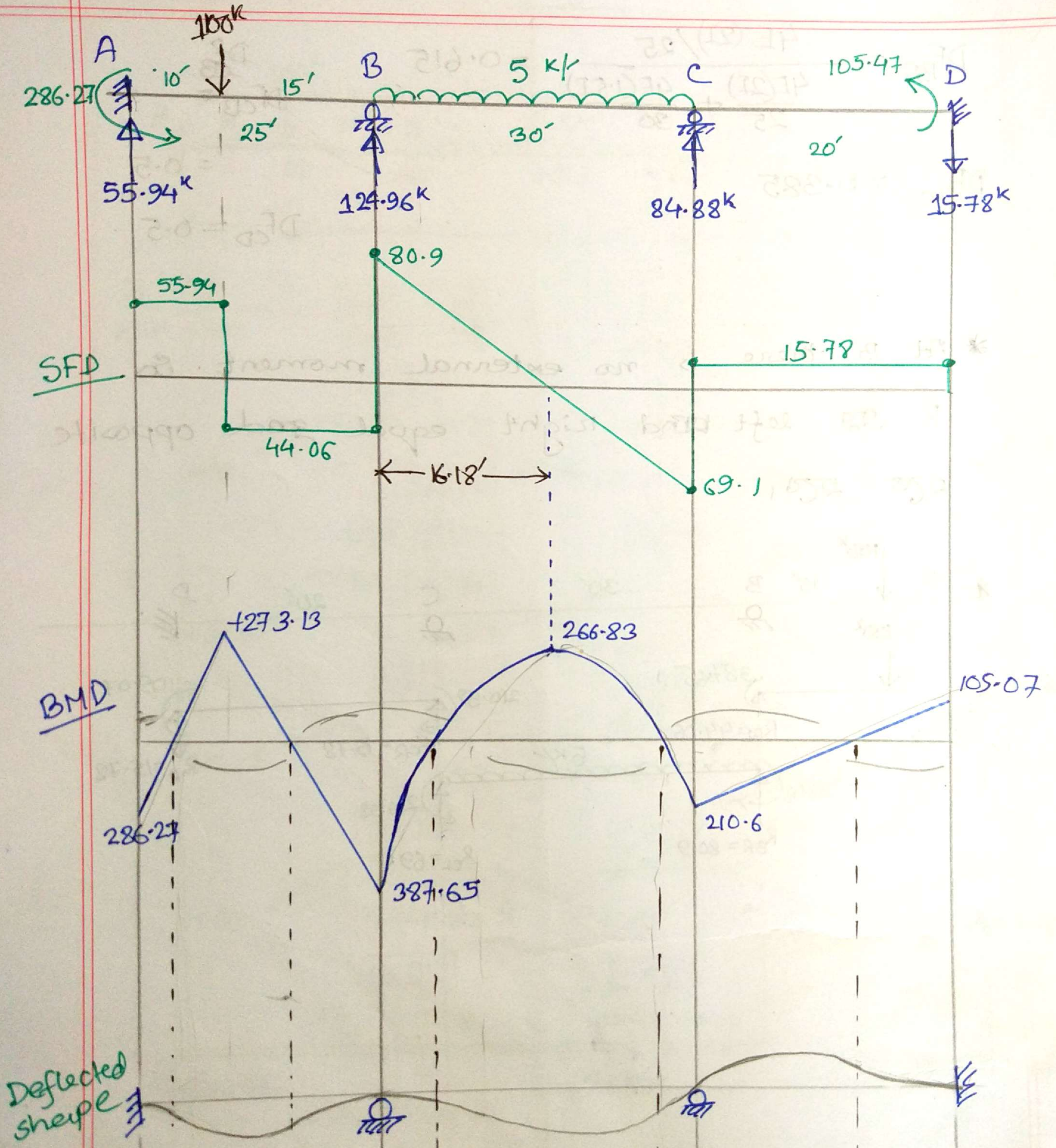




$$k = \frac{\cdot}{\cdot + \infty} = 0 .$$

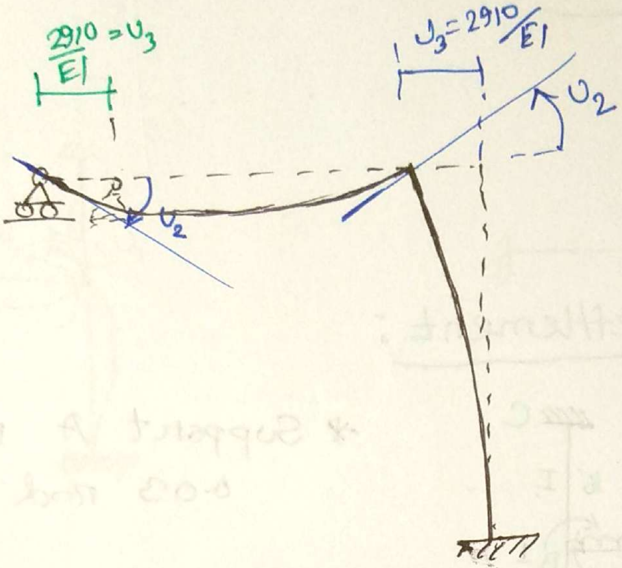

T60
34
8k
10'
18'
2k/'
122.86
0.49
30.27(=7.51+22.56)
27.24k
SFD
0.49
+
22.76
+
11.38
−
−
-7.51
27.24
BMD
8.3
+
62.7
+
−
−
-66.8
-122.86

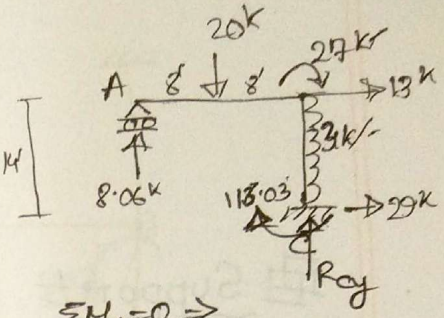

Deflected shape
✓


A
100k
10′
15′
286·27
25′
B
5 k/′
30′
C
105·47
D
20′
55·94k
124·96k
84·88k
15·78k
SFD
55·94
80·9
44·06
16·18′
69·1
15·78
BMD
+273·13
266·83
105·07
286·27
387·65
210·6
Deflected shape

Lec-15 Prob

Deflected shape :

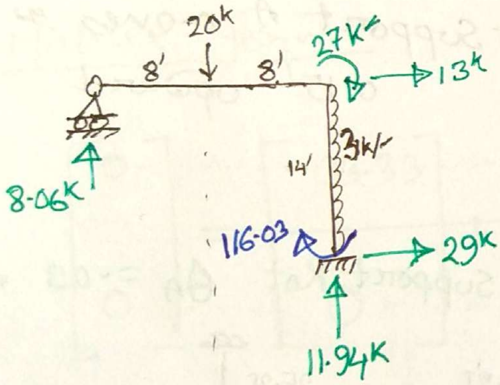




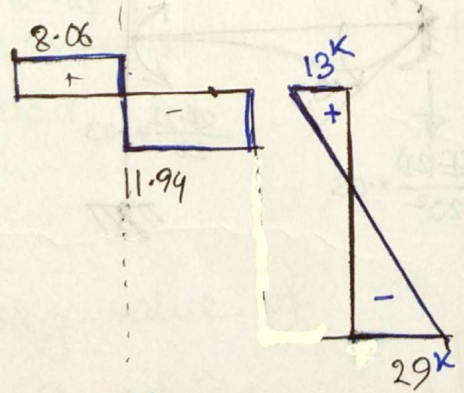
$\Sigma M_A = 0 \Rightarrow$

$$+27 + 116.03 + (20\times 8) - (29\times 14) + \left(3\times 14\times \frac{14}{2}\right) - 16R_{cy} = 0$$

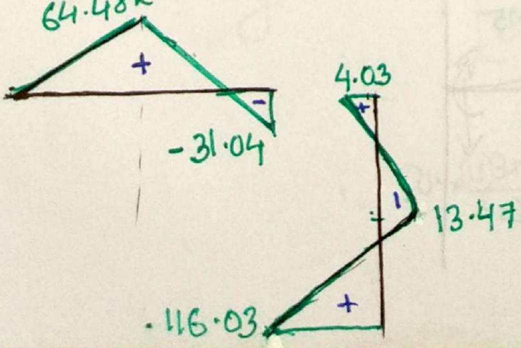
$$\Rightarrow R_{cy} = 11.94^k\ (\uparrow)$$



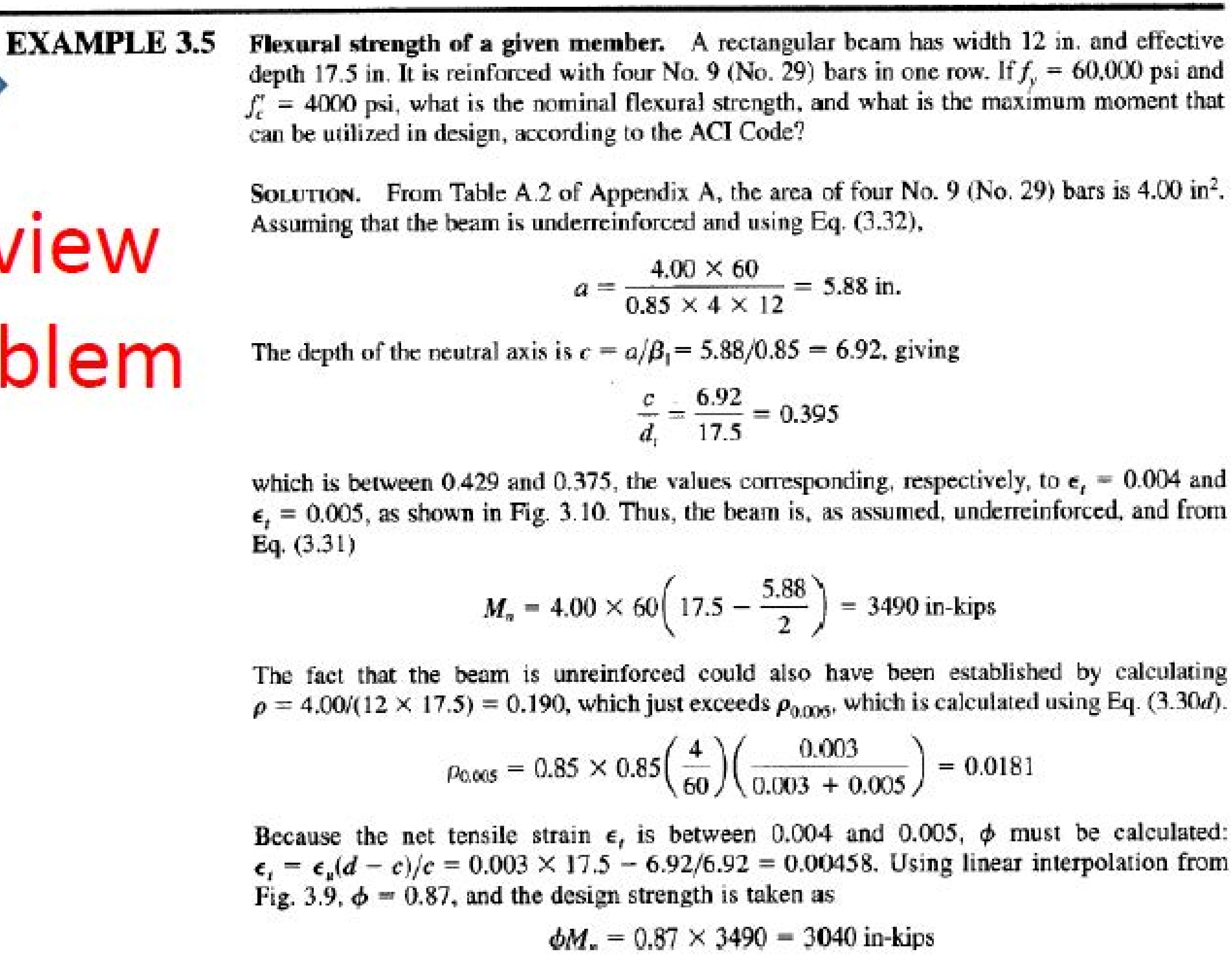
SFD



BMD



Review problem



EXAMPLE 3.5 **Flexural strength of a given member.** A rectangular beam has width 12 in. and effective depth 17.5 in. It is reinforced with four No. 9 (No. 29) bars in one row. If $f_y = 60{,}000$ psi and $f'_c = 4000$ psi, what is the nominal flexural strength, and what is the maximum moment that can be utilized in design, according to the ACI Code?

SOLUTION. From Table A.2 of Appendix A, the area of four No. 9 (No. 29) bars is 4.00 in^2. Assuming that the beam is underreinforced and using Eq. (3.32),

$$a = \frac{4.00 \times 60}{0.85 \times 4 \times 12} = 5.88 \text{ in.}$$

The depth of the neutral axis is $c = a/\beta_1 = 5.88/0.85 = 6.92$, giving

$$\frac{c}{d_t} = \frac{6.92}{17.5} = 0.395$$

which is between 0.429 and 0.375, the values corresponding, respectively, to $\epsilon_t = 0.004$ and $\epsilon_t = 0.005$, as shown in Fig. 3.10. Thus, the beam is, as assumed, underreinforced, and from Eq. (3.31)

$$M_n = 4.00 \times 60\left(17.5 - \frac{5.88}{2}\right) = 3490 \text{ in-kips}$$

The fact that the beam is unreinforced could also have been established by calculating $\rho = 4.00/(12 \times 17.5) = 0.190$, which just exceeds $\rho_{0.005}$, which is calculated using Eq. (3.30*d*).

$$\rho_{0.005} = 0.85 \times 0.85\left(\frac{4}{60}\right)\left(\frac{0.003}{0.003 + 0.005}\right) = 0.0181$$

Because the net tensile strain ϵ_t is between 0.004 and 0.005, ϕ must be calculated: $\epsilon_t = \epsilon_u(d - c)/c = 0.003 \times 17.5 - 6.92/6.92 = 0.00458$. Using linear interpolation from Fig. 3.9, $\phi = 0.87$, and the design strength is taken as

$$\phi M_n = 0.87 \times 3490 = 3040 \text{ in-kips}$$

The ACI Code limits on the reinforcement ratio

$$\rho_{max} = 0.0206$$

$$\rho_{min} = \frac{3\sqrt{4000}}{60{,}000} \geq \frac{200}{60{,}000} = 0.0033$$

are satisfied for this beam.

Design Problem

EXAMPLE 3.6 **Concrete dimensions and steel area to resist a given moment.** Find the concrete cross section and the steel area required for a simply supported rectangular beam with a span of 15 ft that is to carry a computed dead load of 1.27 kips/ft and a service live load of 2.15 kips/ft, as shown in Fig. 3.12. Material strengths are $f'_c = 4000$ psi and $f_y = 60{,}000$ psi.

SOLUTION. Load factors are first applied to the given service loads to obtain the factored load for which the beam is to be designed, and the corresponding moment:

$$w_u = 1.2 \times 1.27 + 1.6 \times 2.15 = 4.96 \text{ kips/ft}$$

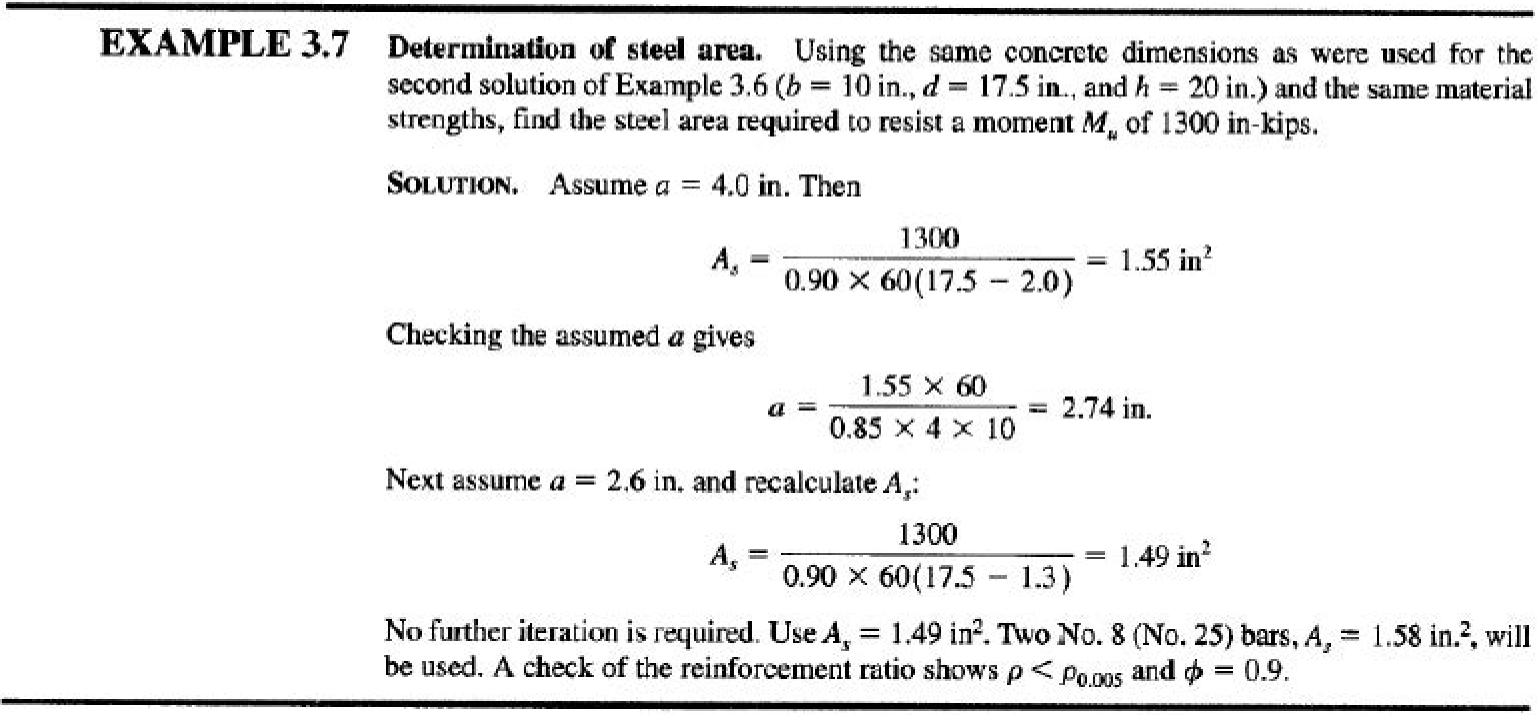
$$M_u = \frac{1}{8} \times 4.96 \times 15^2 \times 12 = 1670 \text{ in-kips}$$

The concrete dimensions will depend on the designer's choice of reinforcement ratio. To minimize the concrete section, it is desirable to select the maximum permissible reinforcement ratio. To maintain $\phi = 0.9$, the maximum reinforcement ratio corresponding to a net tensile strain of 0.005 will be selected (see Fig. 3.9). Then, from Eq. (3.30*d*)

$$\rho_{0.005} = 0.85\beta_1 \frac{f'_c}{f_y} \frac{\epsilon_u}{\epsilon_u + 0.005} = 0.85 \times 0.85 \left(\frac{4}{60}\right)\left(\frac{0.003}{0.003 + 0.005}\right) = 0.0181$$

Using Eq. (3.30*c*) gives $\rho_{\max} = 0.0206$, but would require a lower strength reduction factor. Setting the required flexural strength equal to the design strength from Eq. (3.38), and substituting the selected values for ρ and material strengths,

Determination of steel area



EXAMPLE 3.7 **Determination of steel area.** Using the same concrete dimensions as were used for the second solution of Example 3.6 ($b = 10$ in., $d = 17.5$ in., and $h = 20$ in.) and the same material strengths, find the steel area required to resist a moment M_u of 1300 in-kips.

SOLUTION. Assume $a = 4.0$ in. Then

$$A_s = \frac{1300}{0.90 \times 60(17.5 - 2.0)} = 1.55 \text{ in}^2$$

Checking the assumed a gives

$$a = \frac{1.55 \times 60}{0.85 \times 4 \times 10} = 2.74 \text{ in.}$$

Next assume $a = 2.6$ in. and recalculate A_s:

$$A_s = \frac{1300}{0.90 \times 60(17.5 - 1.3)} = 1.49 \text{ in}^2$$

No further iteration is required. Use $A_s = 1.49 \text{ in}^2$. Two No. 8 (No. 25) bars, $A_s = 1.58 \text{ in.}^2$, will be used. A check of the reinforcement ratio shows $\rho < \rho_{0.005}$ and $\phi = 0.9$.

Doubly Reinforced beam

Nadim Hassoun

1. Calculate ρ, ρ' and $(\rho-\rho')$

   ρ_{max}, ρ_{min}

2. Calculate $$\bar{\rho}_{cy} = 0.85\beta_1 \frac{f_c'}{f_y}\frac{d'}{d}\frac{\varepsilon_u}{\varepsilon_u-\varepsilon_y} + \rho'$$

3. If $\rho \geq \bar{\rho}_{cy}$ compression steel yields, $f_s' = f_y$

   $\rho < \bar{\rho}_{cy}$ " " does not yield $f_s' < f_y$

4. If comp steel yields, then

   a. Check that $\rho_{max} \geq (\rho-\rho') \geq \rho_{min}$ $\phi = 0.9$

   or check $\varepsilon_t \geq 0.005$

   b. Calculate $$a = \frac{(A_s-A_s')f_y}{0.85f_c'b}$$

   c. Calculate $$\phi M_n = \phi\left[(A_s-A_s')f_y\left(d-\frac{a}{2}\right) + A_s'f_y(d-d')\right]$$

5. If comp. steel does not yield, then

a.
$$T = C$$
$$\Rightarrow A_s f_y = 0.85 f_c' * b * \beta_1 c + A_s' f_s'$$
$$A_s f_y = 0.85 \beta_1 b c f_c' + A_s' . E_s \varepsilon_u \frac{c-d'}{c}$$

$\Rightarrow$ solve this quadratic equation to find c.

b. Find $f_s' = E_s \varepsilon_u \frac{c-d'}{c}$

$\phi = 0.9$

$\varepsilon_t =$

c. Check $\rho_{max} \geq \left(\rho - \rho' . \frac{f_s'}{f_y}\right) \geq \rho_{min}$

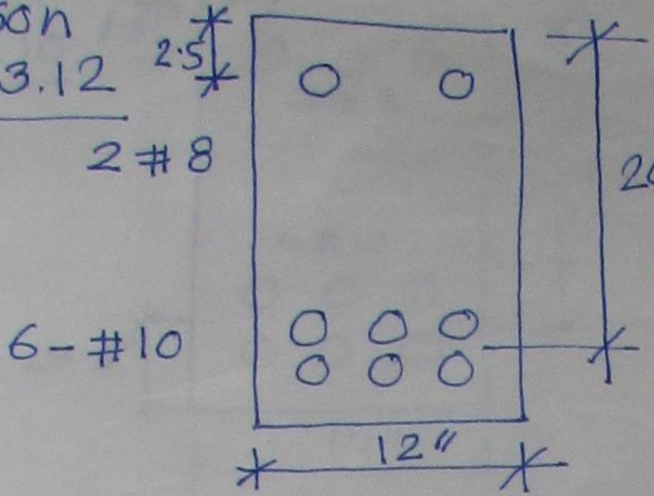
d. Calculate $a = \frac{A_s f_y - A_s' f_s'}{0.85 f_c' b}$ or $a = \beta_1 c$ (check)

e. Calculate

$$\phi M_n = \phi\left[(A_s f_y - A_s' f_s')\left(d - \frac{a}{2}\right) + A_s' f_s' (d - d')\right]$$

Find φM_n

Nilson
Ex 3.12



$f_c' = 5000\,psi$

$f_y = 60{,}000\,psi$

Design moment capacity = ?

$\phi M_n = ?$

Sol

$A_s = 6 \times 1.27 = 7.62\,in^2$ $\rho = \frac{A_s}{bd} = 0.0265$

$A_s' = 2 * 0.79 = 1.58\,in^2$ $\rho' = \frac{A_s'}{bd} = 0.0055$

$$\rho_{max} = 0.85 \beta_1 \frac{f_c'}{f_y} \frac{\varepsilon_u}{\varepsilon_u + .004}$$ $\rho_{max} = 0.0243$

$$\rho_{min} = \frac{3\sqrt{f_c'}}{f_y} \geq \frac{200}{f_y}$$ $\rho_{min} = 0.00354$

$$\bar{\rho}_{cy} = 0.85\beta_1 \frac{f_c'}{f_y}\,\frac{d'}{d}\,\frac{\varepsilon_u}{\varepsilon_u-\varepsilon_y} + \rho'$$

$$= 0.85 * .8\,\frac{5}{60}\,\frac{2.5}{24}\,\frac{.003}{.003-.00207} + .0055 = 0.0245$$

$\rho > \bar{\rho}_{cy} \Longrightarrow$ Comp bars yield when beam fails.

$\rho_{max} = .0243$ ok.

$\rho-\rho' = 0.021$ $\quad\rho_{min} = 0.00354$

$$a = \frac{A_s-A_s'}{0.85 f_c' b} f_y = 7.11 \qquad c = \frac{a}{\beta_1} = 8.88$$

$$\varepsilon_t = 0.003\,\frac{\overset{d_t}{24}-8.88}{8.88} = 0.0051 \Longrightarrow \phi = 0.9$$

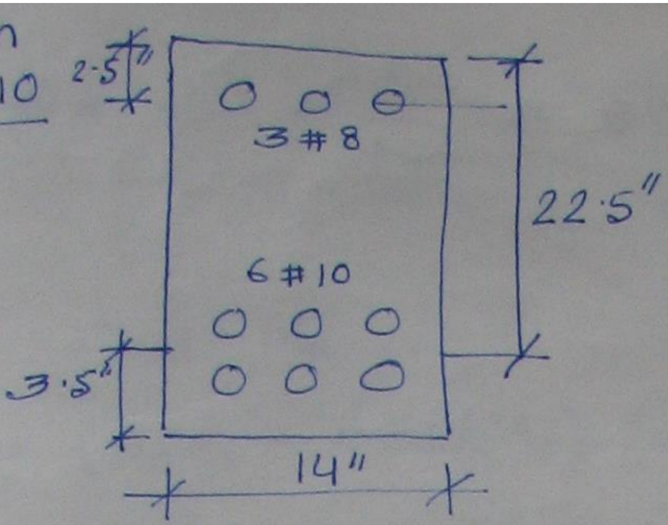
$$M_n = A_s' f_y (d-d') + (A_s-A_s') f_y (d - a/2) = 9447 \text{ kin}$$

$$= 1.58 * 60 (24-2.5) + 6.04 * 60 \left(24 - \frac{7.11}{2}\right)$$

$$\phi M_n = 8503 \text{ k-in}$$

Nadim

Ex 3.10



Find ϕM_n

$f_c' = 5$ ksi $f_y = 60$ ksi.

$A_s' = 3*0.79 = 2.37 in^2$ $\rho' = 0.007524$

$A_s = 6*1.27 = 7.62 in^2$ $\rho = 0.0242$

$\rho - \rho' = 0.01667$.

$\rho_{max} = 0.0243$ ← $(\rho-\rho')$ is ok.

$\rho_{min} = 0.00354$

2. $\bar{\rho}_{cy} = 0.85*0.8*\frac{5}{60}\frac{2.5}{22.5}\frac{.003}{.003-.00207} + 0.007524 = 0.02783$

$\rho < \bar{\rho}_{cy} \Rightarrow$ Comp bar does not yield

to be more correct

$\rho < \rho_{cy} \Rightarrow$ Comp

to be
more
correct

3. $T = C = C_s + C_c$

$\Rightarrow \quad A_s f_y = 0.85 f_c' \beta_1 c . b + A_s' E_s \varepsilon_u \frac{c-d'}{c} - 0.85 f_c' A_s'$

$\Rightarrow \quad 7.62 * 60 = 0.85 * 5 * .8 * c * 14 + 2.37 * 29 * 10^3 * .003 \frac{c-2.5}{c} - .85 * 5 * 2.37$

$\Rightarrow \quad 457.2 = 47.6c + 206.19 \frac{c-2.5}{c} - 10.07$

$\Rightarrow \quad 47.6c^2 - 261.08c - 515.475 = 0$

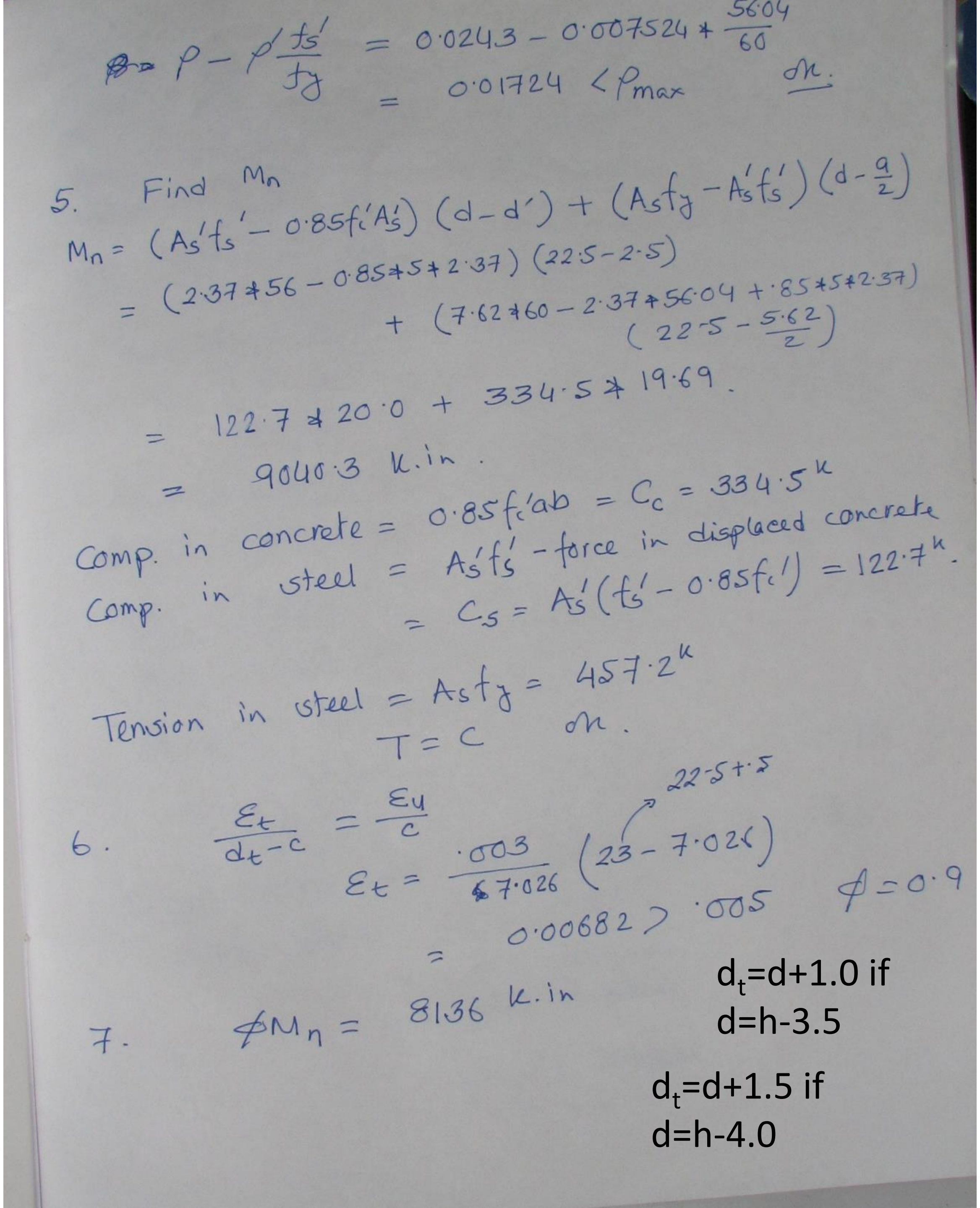
$$c = \frac{261.08 \pm \sqrt{261.08^2 + 4 * 47.6 * 515.475}}{2 * 47.6}$$

$= 7.026$ in

$a = \beta_1 c = 5.62$

$f_s' = E_s \varepsilon_u \frac{c-d'}{c}$

$= 56.04$ ksi.



$$\rho - \rho' \frac{f_s'}{f_y} = 0.0243 - 0.007524 * \frac{56.04}{60}$$
$$= 0.01724 < \rho_{max} \quad ok.$$

5. Find M_n

$$M_n = (A_s' f_s' - 0.85 f_c' A_s')(d - d') + (A_s f_y - A_s' f_s')(d - \frac{a}{2})$$
$$= (2.37 * 56 - 0.85 * 5 * 2.37)(22.5 - 2.5)$$
$$+ (7.62 * 60 - 2.37 * 56.04 + .85 * 5 * 2.37)(22.5 - \frac{5.62}{2})$$

$$= 122.7 * 20.0 + 334.5 * 19.69.$$
$$= 9040.3 \text{ k.in}.$$

Comp. in concrete = $0.85 f_c' ab = C_c = 334.5^k$

Comp. in steel = $A_s' f_s'$ − force in displaced concrete

$= C_s = A_s'(f_s' - 0.85 f_c') = 122.7^k$.

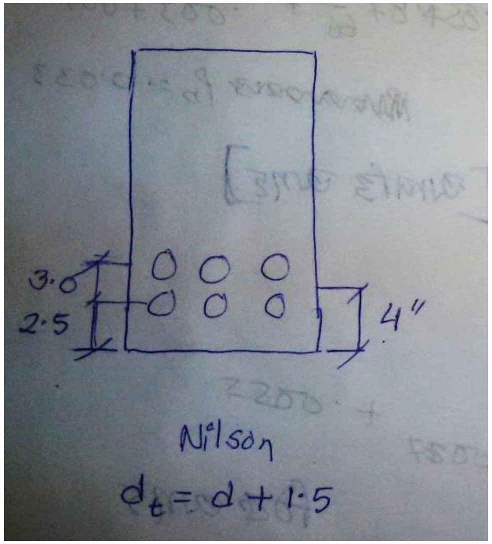
Tension in steel = $A_s f_y = 457.2^k$

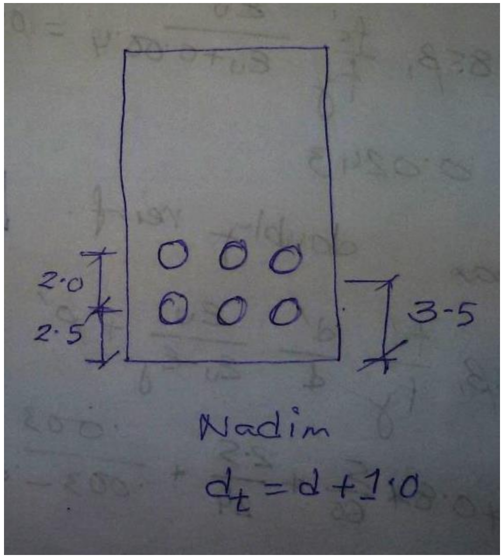
$T = C$ ok.

6. $$\frac{\varepsilon_t}{d_t - c} = \frac{\varepsilon_u}{c}$$

$$\varepsilon_t = \frac{.003}{7.026}(\overset{22.5+.5}{23} - 7.026)$$
$$= 0.00682 > .005 \quad \phi = 0.9$$

7. $\phi M_n = 8136$ k.in





d_t=d+1.0 if
d=h-3.5

d_t=d+1.5 if
d=h-4.0

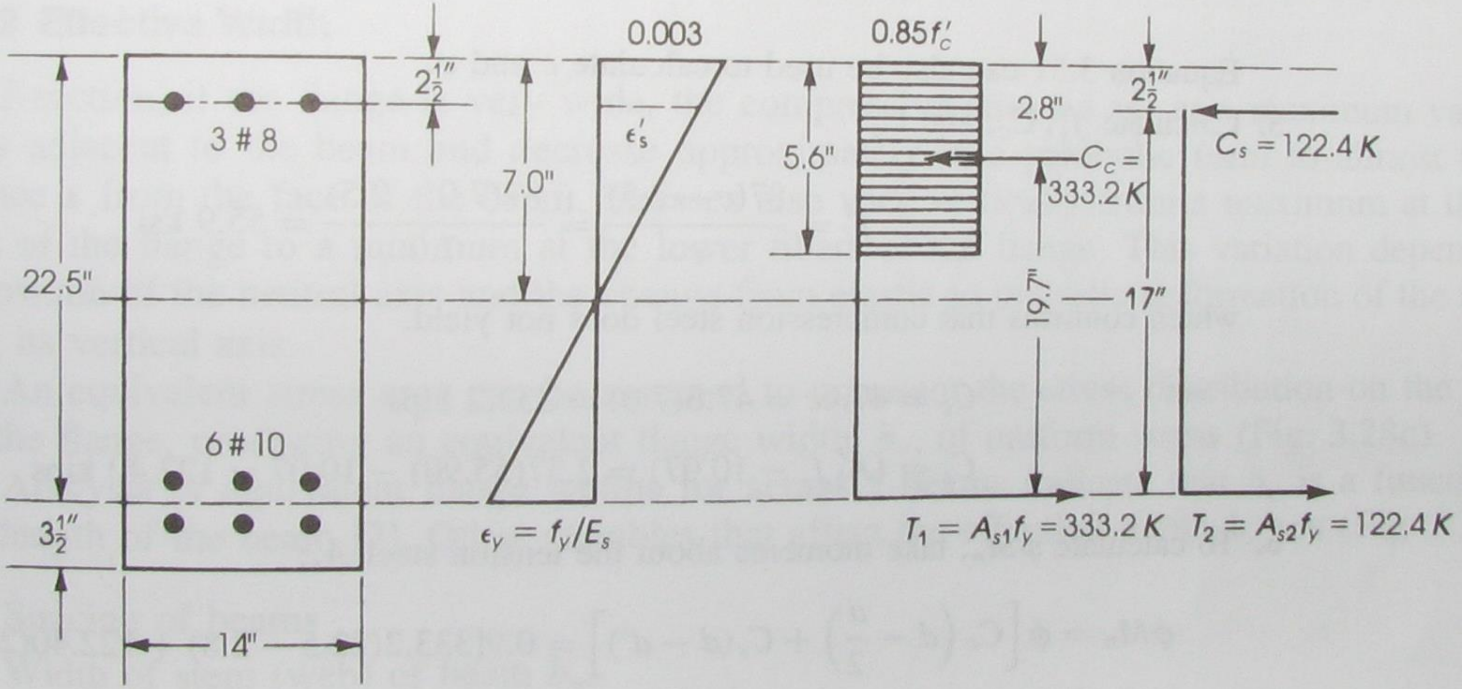


Figure 3.27 Example 3.10 analysis solution.

Design of
Doubly Reinforced Beam

Design problem from Nadim

Ex 4.5 A beam section is limited to a width $b = 10in$ and total depth of $h = 22in$ and has to resist a factored moment of 226.5 k-ft. Calculate the required reinforcement. Given $f_c' = 3ksi$ and $f_y = 50 ksi$.

Sol

$$\rho_{0.005} = 0.85\beta_1 \frac{f_c'}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.005} = 0.01625$$

$d = 22 - 3.5$
$= 18.5$ two layer

$A_s = 0.01625 * 10 * 18.5$
$= 3 in^2$

$$a = \frac{A_s f_y}{0.85 f_c' b} = 5.88 in.$$

$M_u = 226.5 \times 12$
$= 2718$ k-"

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) = 2333.8 \text{ kin.}$$

$$\phi M_n = 0.9 * 2333.8 = 2100.4 \text{ k-in.}$$

Doubly Reinforced beam reqd.

$\phi M_{n1} = 2100.4$

$\phi M_{n2} = 2718 - 2100.4 = 617.6 \text{ k}''$

Assuming comp steel yield

$$A_{s2} = \frac{617.6}{\phi f_y (d-d')} = \frac{617.6}{0.9 * 50 (18.5-2.5)} = 0.86 \text{ in}^2$$

Total tension steel = $A_s = 3.0 + 0.86 = 3.86 \text{ in}^2$

Comp. steel $A_s' = 0.86 \text{ in}^2$.

$\rho' = 0.00465$

$\rho = 0.02086$

$\rho - \rho' = 0.01621$

Use actual area provided

$$\bar{\rho}_{cy} = 0.85\beta_1 \frac{f_c'}{f_y} \frac{d'}{d} \frac{\varepsilon_u}{\varepsilon_u - \varepsilon_y} + \rho'$$

$$= 0.85 * 0.85 \frac{3}{50} \frac{2.5}{18.5} \frac{.003}{.003 - 0.001724} + 0.00465$$

$$= 0.01842$$

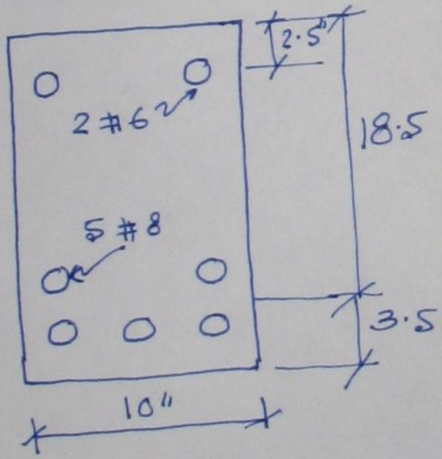
$\rho > \bar{\rho}_{cy}$ comp steel yields.

$$a = \frac{A_s - A_s'}{0.85 f_c' b} f_y = \frac{3.86 - 0.86}{0.85 * 3 * 10} \times 50 = 5.88 \text{ in.}$$

$$c = \frac{a}{\beta_1} = 6.92 \text{ in.}$$

$$\varepsilon_t = \varepsilon_u \frac{d_t - c}{c} = 0.003 \frac{\overset{\text{outer layer}}{19.5} - 6.92}{6.92} = 0.00545$$

$\phi = 0.9$ ok.



Nilson

Ex 3.13

Design

$LL = 2.47\ ^{k/'}$

$DL = 1.05\ ^{k/'}$

simply supported span = 18'

Beam section => $10'' \times 20''$

$f_c' = 4000$ psi $f_y = 60{,}000$ psi

find reinforcement.

Sol

$w_u = 1.2 * 1.05 + 1.6 * 2.47 = 5.212\ ^{k/'}$

$M_u = \frac{1}{8} w_u l^2 = \frac{1}{8} \times 5.212 * 18^2 = 211.09\ ^{k'} = 2533\ ^{k''}$

$d' = 2.5''$ (if needed)

$d = 20 - 4 = 16''$ (two layer)

First check if possible to design singly reinforced.

$\varepsilon_t = 0.005$ $\rho_{0.005} = 0.0181$

$A_s = \rho bd = 0.0181 * 10 * 16 = 2.89 in^2$

$\frac{a}{} = 6''$

$A_s = \rho bd = 0.0181 * 10 * 16 =$

$a = \frac{A_s f_y}{.85 f_c' b} = 5.1 \text{ in}$ $\quad c = \frac{a}{\beta_1} = 6''$

$M_n = A_s f_y (d - \frac{a}{2}) = 2.89 * 60 \left(16 - \frac{5.1}{2}\right) = 2332^{k''}$

$\phi M_n = 0.9 * 2332 = 2099^{k''} < M_u$

Doubly Reinf reqd.

Remaining moment $\phi M_n = 2533 - 2099 = 434^{k''}$

$M_n = 482.4^{k''}$

Assuming comp bars yield

$A_{s2} = \frac{434}{0.9 * 60 * (16 - 2.5)} = 0.6 \text{ in}^2.$

Total tension steel $= 2.89 + 0.6 = 3.49 \text{ in}^2$

Comp steel $= 0.6 \text{ in}^2$

4 # 9
$4 \text{ in}^2.$

2 # 6
$0.88 \text{ in}^2.$

$\rho = \frac{4}{10\times16} = 0.025$ $\rho' = \frac{0.88}{10\times16} = 0.0055$

$$\bar{\rho}_{cy} = 0.85\beta_1 \frac{f_c'}{f_y}\frac{d'}{d}\frac{\varepsilon_u}{\varepsilon_u-\varepsilon_y} + \rho'$$

$$= 0.85 * 0.85 \frac{4}{60}\frac{2.5}{16}\frac{.003}{.003-.00207} + 0.0055$$

$$= 0.0298$$

$\rho < \bar{\rho}_{cy} \Rightarrow$ Comp steel does not yield

$$A_sf_y = 0.85f_c'b\beta_1c + A_s'f_s' - A_s' * 0.85f_c'$$

$$\Rightarrow 4*60 = 0.85*4*10*.85c + A_s' * \left[E_s\varepsilon_u\frac{c-d'}{c} - 0.85f_c'\right]$$

$$\Rightarrow 240c = 28.9c^2 - 2.992c + (76.56c - 191.4)$$

$$\Rightarrow 28.9c^2 - 166.43c - 191.4 = 0$$

$c = 6.74$ in. $a = 5.73$ in.

$f_s' = E_s \varepsilon_u \frac{c-d'}{c} = 54.7^{ksi}$.

$C_c = 0.85 f_c' . b . \beta_1 c = 194.8^{k}$

$C_s = A_s' \left[f_s' - 0.85 f_c' \right] = 45.14^{k}$ checked.

$T = A_s f_y = 4 * 60 = 240^{k}$

$$M_n = C_c \left(d - \frac{a}{2} \right) + C_s \left(d - d' \right)$$

$$= 194.8 \left(16 - \frac{5.73}{2} \right) + 45.14 \left(16 - 2.5 \right) *$$

$$= 2558.5 + 609.44 = 3167.9^{k''}$$

$$\varepsilon_t = \varepsilon_u \frac{d_t - c}{c} = 0.003 \, \frac{\overset{\swarrow\, !}{17.5} - 6.74}{6.74}$$

$$= 0.00479$$

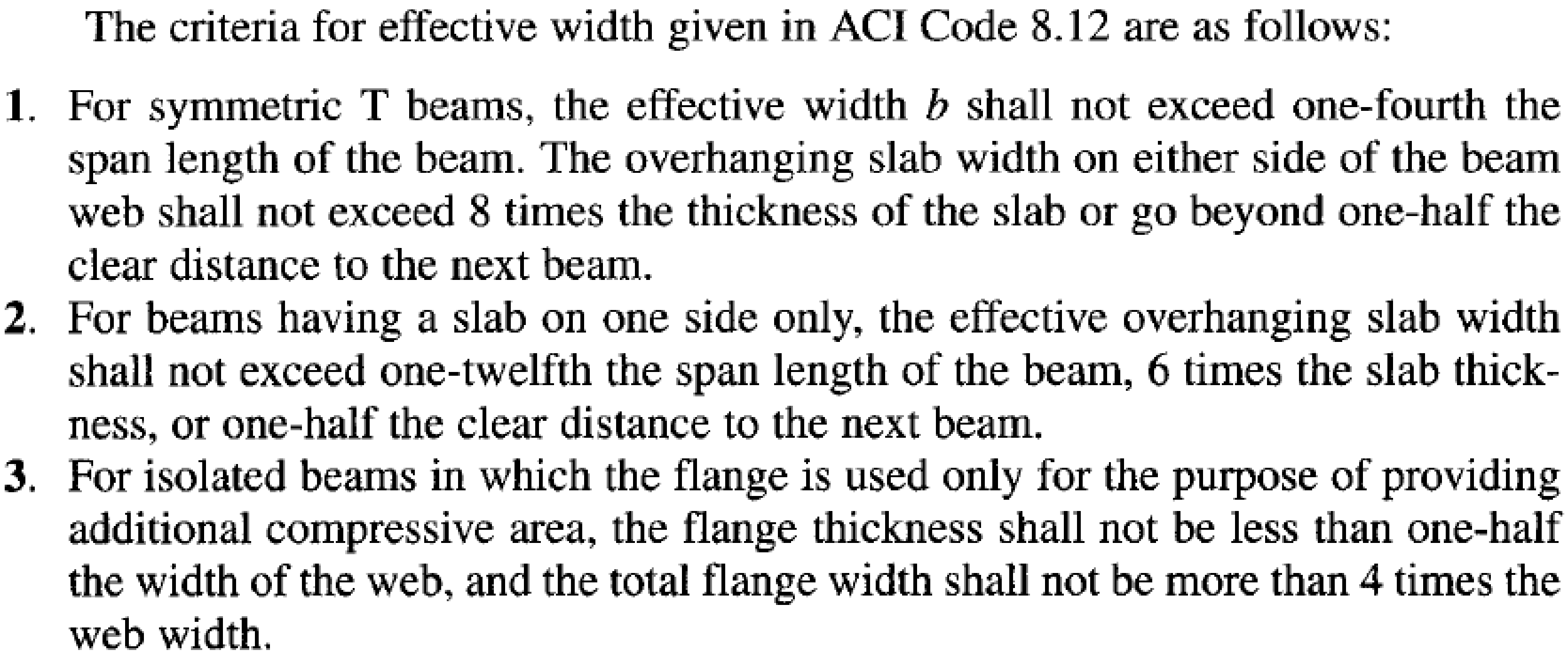
$$\phi = 0.483 + 83.3\,\varepsilon_t = 0.882$$

$$\phi M_n = 2793.9^{k} > M_u = 2533^{k''} \quad \underline{ok}.$$

T-beam

- RC beam and slab are monolithically cast
- Beam stirrups and bent bars extend into the slab
- A part of slab act along with beam top to take longitudinal compression
- Slab forms the beam flange
- Part of beam below slab is called web/stem

Effective flange width



The criteria for effective width given in ACI Code 8.12 are as follows:

1. For symmetric T beams, the effective width *b* shall not exceed one-fourth the span length of the beam. The overhanging slab width on either side of the beam web shall not exceed 8 times the thickness of the slab or go beyond one-half the clear distance to the next beam.
2. For beams having a slab on one side only, the effective overhanging slab width shall not exceed one-twelfth the span length of the beam, 6 times the slab thickness, or one-half the clear distance to the next beam.
3. For isolated beams in which the flange is used only for the purpose of providing additional compressive area, the flange thickness shall not be less than one-half the width of the web, and the total flange width shall not be more than 4 times the web width.

1. Symmetrical T beams

$b < 16h_f+b_w$

$b < \text{Span}/4$

$b < \text{c/c beam spacing}$

2. Beam having slab on one side

$b < \text{span}/12+b_w$

$b < 6h_f+b_w$

$b < \text{Half the clear span } +b_w$

3. Isolated T beam

$h_f > b_w/2$

$b< 4b_w$

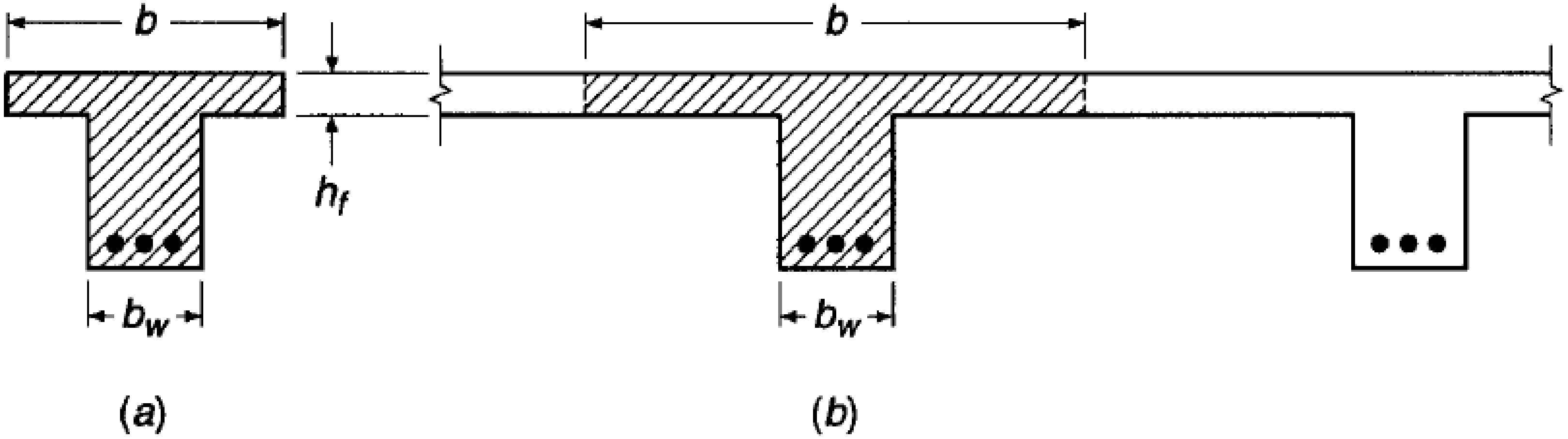
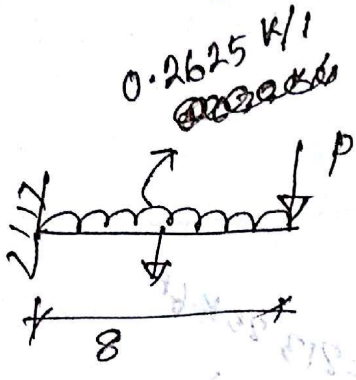


FIGURE 3.17
Effective flange width of T beams.

13-14

$\boxed{7}$ (b)



$$M = PL + \frac{WL^2}{2}$$

$$\Rightarrow 88.24 = 8P + \frac{0.2625 \times 8^2}{2}$$

$$\therefore \boxed{P = 9.98 \text{ Kip}}$$

$0.45\sqrt{f_c'} > f_r$

$\therefore$ Cracked

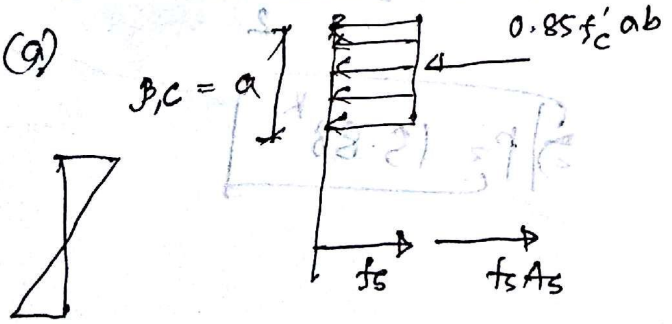
$$M = \boxed{\frac{1}{2} f_c k j b d^2}$$
$$= 88.24 \text{ K-ft}$$

$$M = \boxed{A_s f_s j d}$$
$$= 96.12 \text{ K-ft}$$

$$k = \sqrt{(\rho n)^2 + 2\rho n} - \rho n$$
$$= 0.34 \qquad j = 0.89$$

$$\rho = \frac{A_s}{bd} = 0.011$$

$\boxed{8}$ (a)



$$C = T$$

$$\Rightarrow 0.85 f_c' \underset{\beta_1 c}{\overset{\Downarrow}{a}} b = f_s A_s = \underset{f_y}{\overset{\Downarrow}{f_s}} \rho_b bd$$

$$\therefore \rho_b = 0.85 \beta_1 \frac{f_c'}{f_y} \frac{\epsilon_u}{\epsilon_u + \epsilon_y}$$

$$c = \frac{\epsilon_u}{\epsilon_u + \epsilon_y} d$$

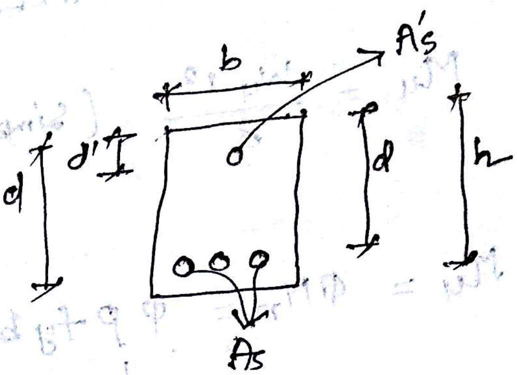
Scanned by CamScanner

Overreinforced / DRB

(Limited in construction for architechtural reasons)

Analysis

(Flexural Strength)



* $\rho > \rho_{max} \Rightarrow$ DRB

$\swarrow \frac{A_s}{bd}$ $\quad \hookrightarrow 0.85\beta_1 \frac{f_c'}{f_y} \frac{3}{7}$

[** $\rho < \rho_{max} \Rightarrow$ SRB যদিও compression steel আছে]

* $a = \frac{(A_s - A_s')f_y}{0.85 f_c' b}$

* $M_n = M_{n_1} + M_{n_2}$

* $M_{n_1} = A_s' f_y (d - d')$

$\hookrightarrow f_s' \; (f_s' < f_y)$

$\epsilon_s' \times E_s \swarrow$ $\quad f_y \; (f_s' > f_y)$ [compression steel yields]

$\downarrow$

$\epsilon_u \times \frac{c-d'}{c} \quad c \hookrightarrow \frac{a}{\beta_1} \xrightarrow{*} \frac{(A_s - A_s')f_y}{0.85 f_c' b}$

* $M_{n_2} = (A_s - A_s') f_y \left(d - \frac{a}{2}\right)$ [$f_s > f_y$]

$\hookrightarrow f_s = \epsilon_s \times E_s$

$\hookrightarrow \epsilon_u \times \frac{d-c}{c}$

* $S_n = \phi M_n$

$\hookrightarrow 0.9$ or $0.65 + (\epsilon_t - 0.002)\frac{250}{3}$

$\epsilon_s \geq 0.005 \qquad \epsilon_s < 0.005$

Shear & Diagonal Tension in Beams

(Shear force at factored loads)

* $V_u \leq \phi V_n$

(0.75)

* $V_u = \phi(V_c + V_s) = \phi V_c + \phi V_s$ (web steel contn to shear strength)

(beam/web width)

* $V_c = 2\lambda\sqrt{f'_c}\, b_w d$

(3000–5000 psi)

(Concrete contn to shear strength)

$\lambda = 1$ (normal weight concrete)

* $A_{v,min} = 50\dfrac{b_w s}{f_y}$ (yield strength of Steel)

* $s = \dfrac{\phi A_v f_y d}{V_u - \phi V_c}$

Vertical stirrups area

*(#3 or #4 rod area × 2 legs)

* $S_{min} = 4''$

* $S_{max} = 24'' = \dfrac{d}{2} = \dfrac{A_v f_y}{50 b_w}$ $\left[V_s \leq 4\sqrt{f'_c}\, b_w d\right]$

$= 12'' = \dfrac{d}{4} = \dfrac{A_v f_y}{100 b_w}$ $\left[V_s > 4\sqrt{f'_c}\, b_w d\right]$

* $V_u < \dfrac{\phi V_c}{2} \Rightarrow$ No web reinf.

* $V_u > \phi V_c \Rightarrow$ web reinf.

* $\dfrac{\phi V_c}{2} < V_u < \phi V_c \Rightarrow$ minm web reinf.

Beam without web reinforcement

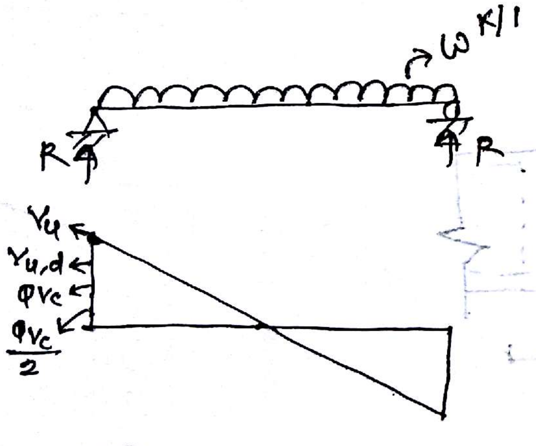
* $V_u = \frac{\phi V_c}{2} = \frac{\phi}{2}(2\lambda\sqrt{f'_c}\, b_w d) \Rightarrow b_w d$

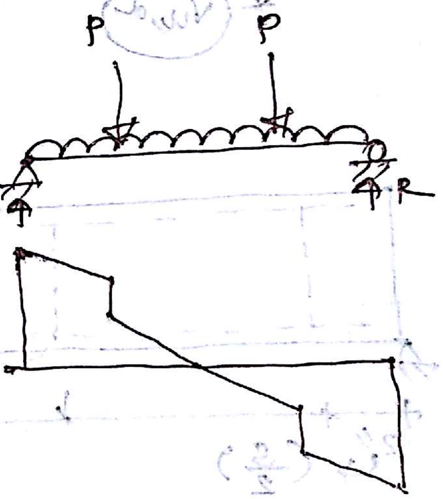
$\Downarrow$

$d = 2b \; (b = 18'')$

* $V_u = \phi V_c = \phi(2\lambda\sqrt{f'_c}\, b_w d) \Rightarrow$ if minimum steel is given.

Design of web reinforcement





* SFD
* V_u (from reaction)
* $V_{u,d}$ d is effective depth
* ϕV_c * (↳ 0.75) $= \phi 2\lambda\sqrt{f'_c}\, b_w d$
* $\frac{\phi V_c}{2}$

** $V = (R - P) - (w \times x)$

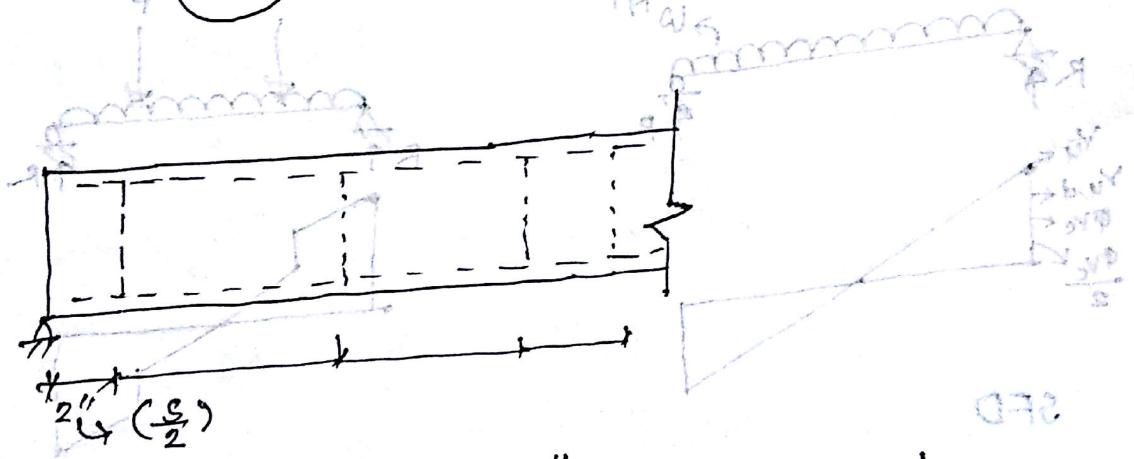
* $S_{min} = 4''$

* $\varphi V_s = \underset{\boxed{* V_{u,d}}}{V_u} - \varphi V_c$

* $S_{max} = 24'' = \frac{d}{2} = \frac{A_v f_y}{50 b_w}$ $[\varphi V_s \le \varphi 4\sqrt{f'_c}\, b_w d]$

$= 12'' = \frac{d}{4} = \frac{A_v f_y}{100 b_w}$ $[\varphi V_s > \varphi 4\sqrt{f'_c}\, b_w d]$

* $S = \frac{\overset{\boxed{* 0.75}}{\varphi} A_v f_y d}{\underset{\boxed{* V_{u,d}}}{V_u} - \varphi V_c}$ ⇒ [lower rounding]



* $\frac{\text{distance for } \frac{\varphi V_c}{2} - 2''}{3} = L_1 \Rightarrow \frac{L_1}{s}$

* $\frac{L - (x_1 + 2)}{2} = L_2 \Rightarrow \frac{L_2}{s}$

↳ $\frac{\varphi A_v f_y d}{V_u - \varphi V_c}$

↳ $(R-P) - (\omega \times x)$

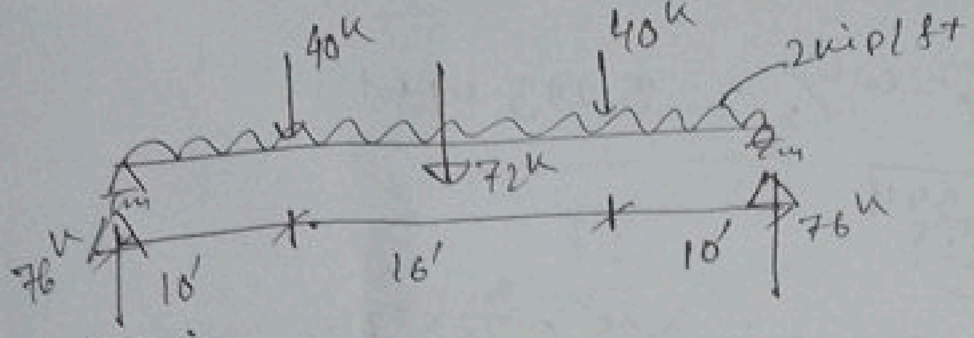
↳ $(x_1 + 2)$

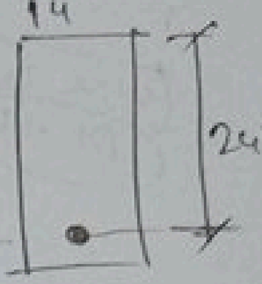
* $\frac{L - (x_1 + x_2 + 2)}{s}$

↳ $\frac{\varphi A_v f_y d}{V_u - \varphi V_c}$

↳ $(R-P) - (\omega \times x)$ ↳ $(x_1 + x_2 + 2)$

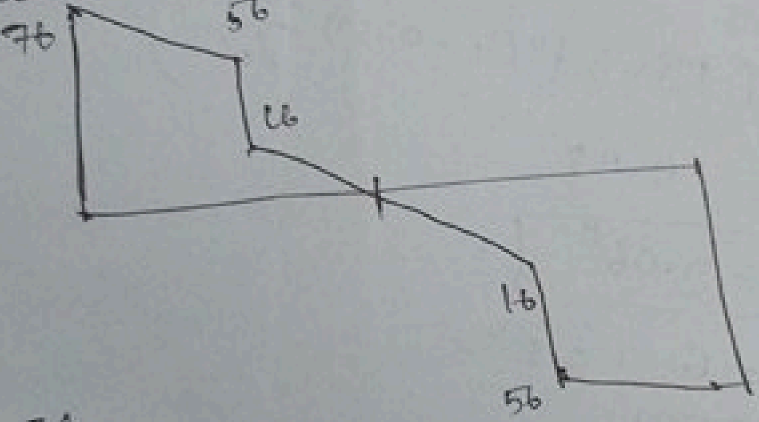
② (a)

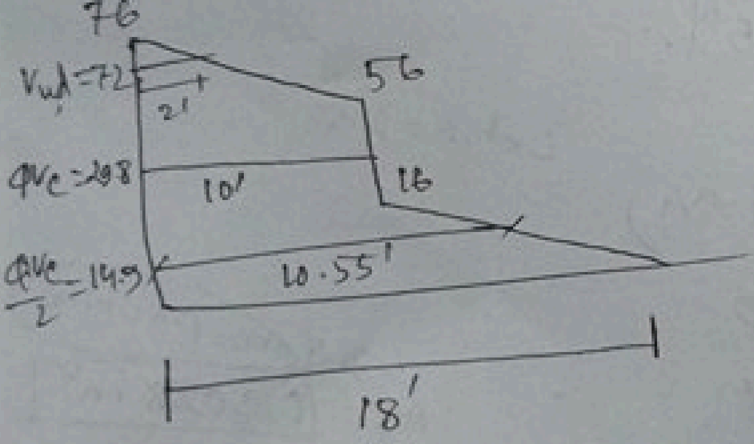




$f_c' = 3.5$ ksi

$f_y = 60$ ksi





$V_u = 76^k$

$d = 24" = 2'$

$V_{u,d} = 76-(2\times 2) = 72^k$

$\phi V_c = 0.75 \times 2\sqrt{f_c}\, b_w d =$ ~~39.8k~~ 39.8 kips × 0.75

must be Psi

$= \boxed{29.8 \text{ kips}}$ from figure distance = 10'

$\frac{\phi V_c}{2} = 14.9$, distance → $14.9 = 76-40-(2\times x)$

$\Rightarrow x = 10.55'$

$S_{min} = 4"$

$\phi V_s = V_{ud} - \phi V_c = 72 - 29.8 = 42.2$ kips.

$4\phi\underline{\sqrt{f_c'}}\,b_w d = 59.6\text{ kip} > \phi V_s$.

in psi

$\therefore S_{max} = \frac{24}{2} = 12''$

$= 24''$

$= \frac{A_v f_{yt}}{0.75\sqrt{f_c'}\,b_w} \leq \boxed{\frac{A_v f_{yt}}{50 b_w}} = \frac{0.22 \times 60000}{50 \times 14}$ psi.

$= 18.86''$.

$S_{max} = 12''$

$\therefore$

$S = \frac{\phi A_v f_{yt} d}{V_u - \phi V_c} = 5.63'' \approx 6''$.

$\therefore \boxed{S = 6''}$

The following spacing patern is satisfactory:

1 space at 2 in = 2"
7 space at 6 in = 42"
7 space at 6 in = 42"
6 space at 7 in = 42"

$= 128'' \approx \boxed{10.66'}$.

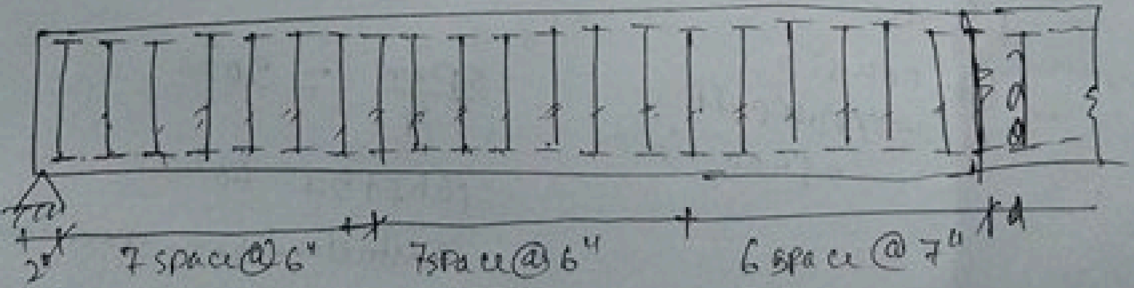


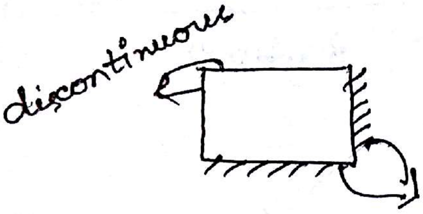
TABLE 13.1
Minimum thickness *h* of nonprestressed one-way slabs

Simply supported	$l/20$
One end continuous	$l/24$
Both ends continuous	$l/28$
Cantilever	$l/10$

Two-Way Slab

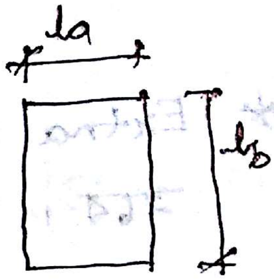
(Design)

* $\frac{\text{Long direction}}{\text{Short direction}} < 2 \Rightarrow$ two way slab



* $M_a = C_a W l_a^2$ → Short direction clear span
tabulated moment coefficients

* $M_b = C_b w l_b^2$ → long direction clear span
uniform load, psf



** Continuous edge has negative moment

** (-)ve M at discontinuous edge = $\frac{1}{3} \times$ positive moment

** Short direction M > long direction M

** $h = \frac{\text{perimeter}}{180}$

min^m slab thickness

** $S < 2h$

Reinforcements –usual sizes

- Slab- No 3, 4, 5 (10mm, 12mm, 16mm)
- Beam- No 5,6, 7, 8 (16 20 ~~22~~ 25mm)
- Stirrup/tie- No, 3 4 (10 12mm)
- Column –No 5, 6 7 8 9 10 11 14 18 (16 20 ~~22~~ 25 28 32 ….)
- Mat- No 4,5,6,8 (12 16 20 25 mm)
- Smaller sizes preferred as long as there is no congestion

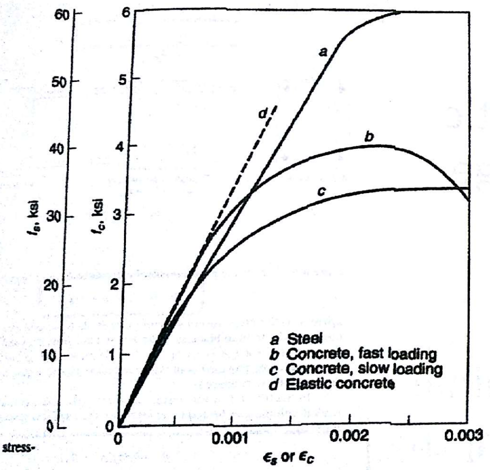


FIGURE 1.16
Concrete and steel stress-strain curves.

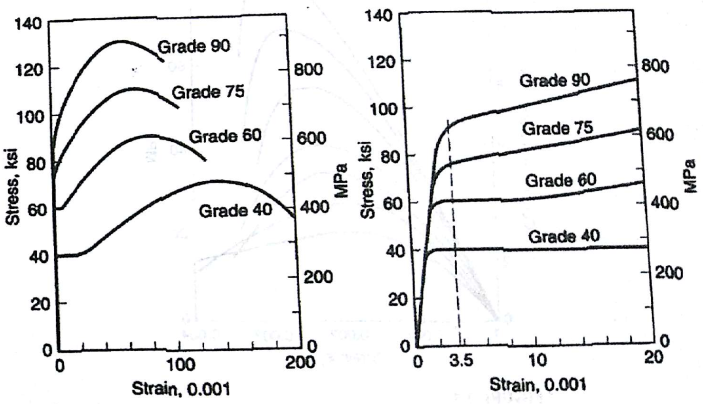
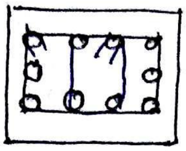


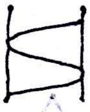
FIGURE 2.15
Typical stress-strain curves for reinforcing bars.

3 types of column :

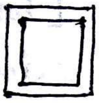
① Tied column



② Spiral column

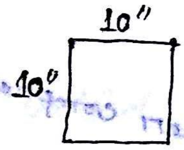


③ Composite "



Main reinforcement:

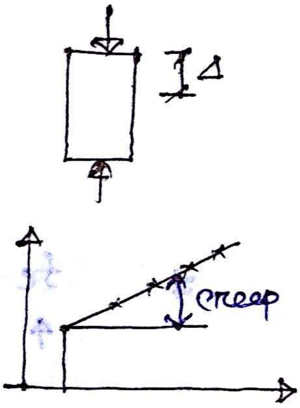
$\boxed{1 \text{ to } 8\%}$



$\Rightarrow$ $A = 100 \text{ in}^2$

Min^m 1 in^2

Max 8 in^2

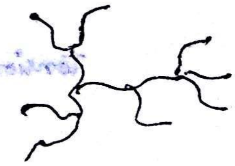


* To reduce bending, creep & shrinkage

we give min^m reinf.

Normally 2-3%

Preferably not exceed 4-5%



Shrinkage

* #5 and nine rod are used.

Min^m 4 longitudinal

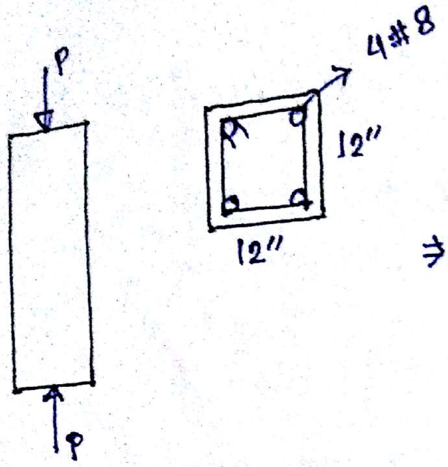
CE 317

②

0|03|16| Sun

#

⊡ Elastic Range :



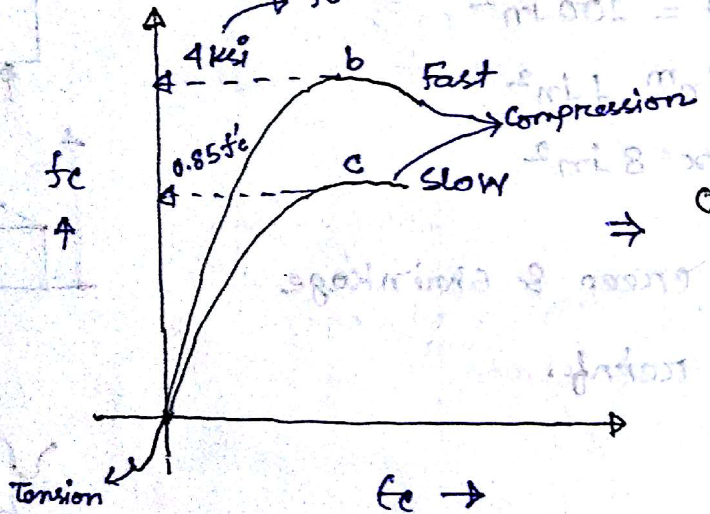
$$\Rightarrow \varepsilon = \frac{\Delta}{L} = \varepsilon_c = \varepsilon_s$$

[because perfect bonding in concrete & steel]

$$P = P_c + P_s$$

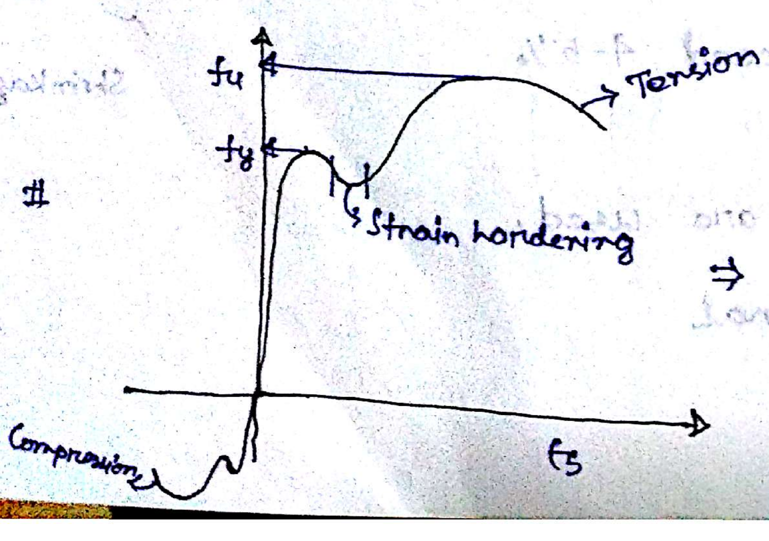
→ f'c = cylinder compressive strength

#



⇒ concrete tension = $\frac{1}{10}$ of compression value

#



⇒ Tension & compression এ Steel এর behaviour একই

compression এ same shape graph পাব

✓

$P_n = 0.85 f'_c A_c + A_{st} f_y$ ⇒ slow loading

$\boxed{** \; P_n = 0.85 f'_c (A_g - A_{st}) + A_{st} f_y}$ $[\epsilon = 0.003]$

Nominal Strength

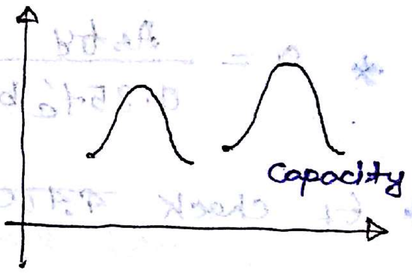
Slow loading এ convert করলাম

Net concrete section

* Nominal → Calculated strength

** Capacity ≥ Requirement (In design)

because of uncertainty



* ϕP_n → Design Strength

⇒ ϕ এর মান column এ কম হবে because column fail করলে building ভেঙ্গে যাবে।

$\boxed{** \; \text{Design strength} = \alpha \phi P_n}$

* ϕ = 0.65 for tied column

= 0.75 for spirally reinf. column

* α = for accidental eccentricity

= 0.8 for tied

= 0.85 for spiral

✓

CE 317

③

13/03/16| Sun

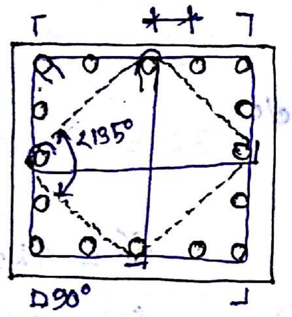
Purpose of ties :

* $P_{cr} = \frac{\pi^2 EI}{L^2}$  $I = 4I + Ad^2$

* Tie দিলে Buckling কম হয়।

ACI provisions for ties :

<6" (6" clean spacing এর বেশি হলে সেসব rod এ tie দিতে হবে)
otherwise buckling হবে



⇒ rod direct bending এ আছে
তাই alternate tie দিতে
হবে।

① All bars

#3 tie for ≤ #10

4 tie for > # 10 & bundled bar

② spacing < 16 $d_{long\ bars}$

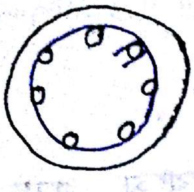
48 $d_{tie\ bars}$

least dimension of column (10×15)
↓
least

* corner always 90°. but max^m limit 135°

* circular tie arrangement এ alternate tie

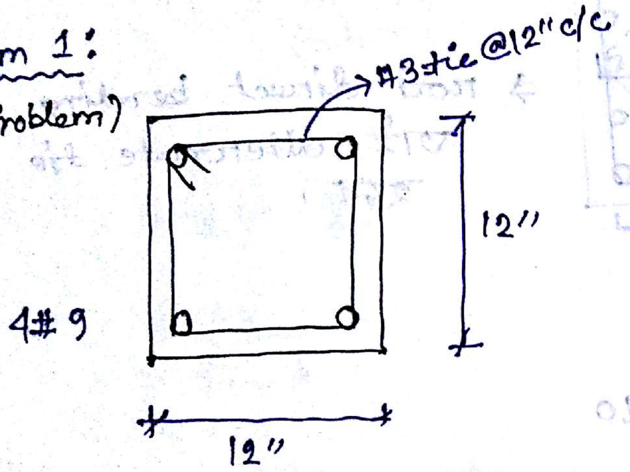
দিতে হয় না।



⇒ rod direct tension এ আছে তাই alternate tie দিতে হবে না

ACI provision for spirals

☐ Problem 1:
(Review Problem)



$f'_c = 4ksi$

$fy = 6ksi$

Nominal Axial Comp. capacity (P_n) = ?

Design 〃 〃 〃 $(\alpha \phi P_n)$ = ?

Sol^n:

$P_n = 0.85 f'_c (A_g - A_{st}) + A_{st} f_y$

$= 0.85 * 4 * (12 * 12 - 4) + 4 * 60$

$= 476 + 240$

$= \boxed{716^K}$

[* 0.002 তে আগেই steel reach করবে, then 0.003 তে concrete reach করবে, then fail করবে]

$\alpha \phi P_n = 0.8 * 0.65 * 716$

$= \boxed{372.3^K}$

*** Tie**

\# 3 tie is OK because # 9 bar

spacing ⇒ $16 * \frac{9}{8} = 18''$

$48 * \frac{3}{8} = 18''$

$12''$

$\boxed{\therefore \# 3 \text{ tie @ } 12'' \text{ c/c is OK.}}$

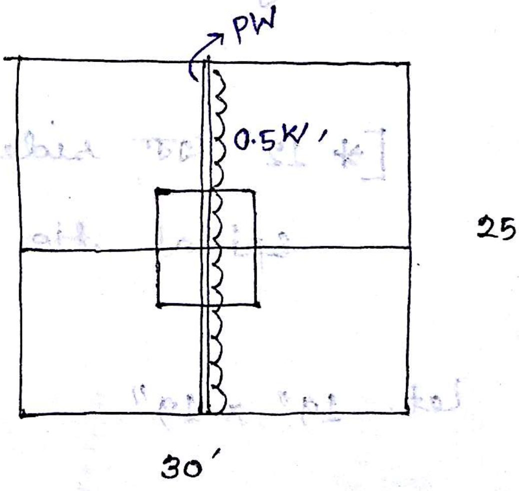
⊞ Problem 2 : (Design Problem)

$P_{DL} = 300^K$

$P_{LL} = 200^K$

$f'_c = 3 \text{ ksi}$

$f_y = 60 \text{ ksi}$



$P_u = 1.2 P_{DL} + 1.6 P_{LL}$

$= 1.2 * 300 + 1.6 * 200$

$= 680^K$

Slab thickness = 6"

$\boxed{\text{Floor finish} = \frac{2}{12} \times 150}$

$PW = \frac{5}{12} * 10' * 1' * 0.12$

* Assignment : DL & LL calculation

✓

Design capacity $\geq$ Demand

$\Rightarrow \alpha \phi P_n \geq P_u$

$\Rightarrow P_u = \alpha \phi P_n = \alpha \phi * [0.85 f'_c (A_g - A_{st}) + A_{st} f_y]$

$\Rightarrow 680 = \alpha \phi A_g [0.85 f'_c (1 - \rho_g) + \rho_g f_y]$

$\downarrow$

$\rho_g = \frac{A_{st}}{A_g} \approx 1-8\%$

Let $\rho_g = 0.02$

$680 = 0.8 * 0.65 \ A_g \ \boxed{[0.85 * 3 (1 - 0.02) + 0.02 * 60]}$

$A_g = 353.5 \ in^2 \Rightarrow 18.8'' \times 18.8''$

[* 12" এর side ধরবে design assignment

spiral tie ধরবে "]

Let, 19" x 19" [*(19×19) × 2% করলে হবে না]

$680 = 0.8 * 0.65 * [0.85 * 3 * (19 \times 19 - A_{st}) + A_{st} * 60]$

$\boxed{A_{st} = 6.73 \ in^2}$

4 # 9 4 * 1

4 # 8 $\underline{4 * 0.79}$

7.16 in²

* Tie

\# 3 size as # 9 bar is used ⇒*[size এর ক্ষেত্রে বড় bar নিব]

spacing:

$$16 * \frac{8}{8} = 16''$$

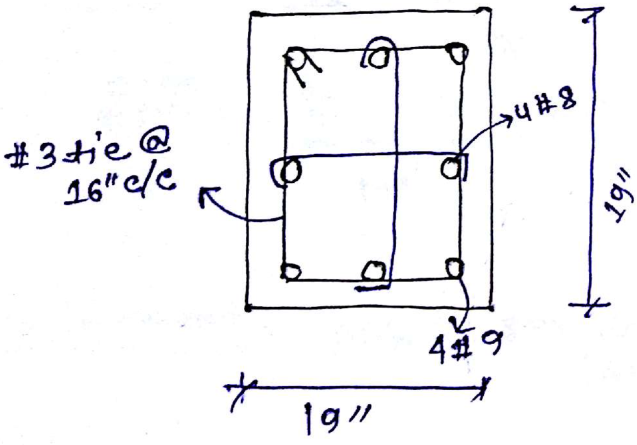
$$48 * \frac{3}{8} = 18''$$

$$19 = 19''$$

⇒*[Spacing এর সময় ছোট bar নিব কারণ ছোট bar এ spacing কম আসে]

$$\boxed{\therefore \#3 \text{ tie } @ \ 16'' \ c/c}$$

⇒ [বড় bar corner এ দিব।]



$$** \ 19'' - \underbrace{2 * 1.5''}_{\text{clear cover}} - \underbrace{2 * \frac{3}{8}}_{\text{tie}} - \underbrace{2 * \frac{9}{8}}_{\#9} - \underbrace{1 * \frac{8}{8}}_{\#8} = 12'' \div \underbrace{2}_{\text{spacing}} = 6''$$

[* 6.1" হলে must tie দিতে হবে।]

[** spacing criteria must check করতে হবে।]

✓

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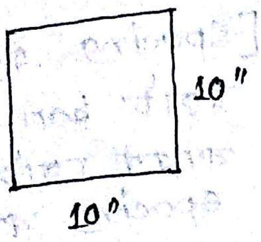
④

20|03|16|Sun

Problem 3 (Design)



$P_{DL} = 60^k$

$P_{LL} = 30^k$

$f'_c = 3 ksi$

$f_y = 60 ksi$

$P_u = 1.2 P_{DL} + 1.6 P_{LL}$

$= 1.2 * 60 + 1.6 * 30$

$= 120^K$

$\alpha \phi P_n \geq P_u$

$P_u = \alpha \phi P_n = \alpha \phi [0.85 f'_c (A_g - A_{st}) + A_{st} f_y]$

$\Rightarrow 120 = 0.8 * 0.65 * [0.85 * 3 (10*10 - A_{st}) + A_{st} * 60]$

$\Rightarrow \underline{A_{st} = -0.42 \; in^2}$

* (-)ve আসা মানে steel লাগবে না।

* Min^m 1% rod দিতে হবে।

✓

Min $A_s \Rightarrow 1\%$

$= 0.01 * 100$

$= \boxed{1\ in^2}$

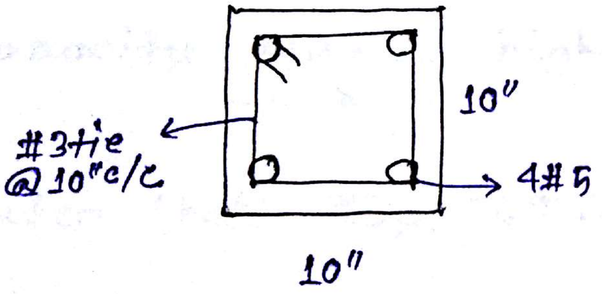
$\boxed{\text{min}\quad 4\#5}$

Tie #3 spacing $16 * \frac{5}{8} = 10''$

$48 * \frac{3}{8} = 18''$

$10'' = 10''$

$\boxed{\therefore\ \#3\text{tie} @ 10''\ c/c}$



Assignment: Minᵐ এর load নিচ্ছে (section) $\alpha \phi P_n = ?$

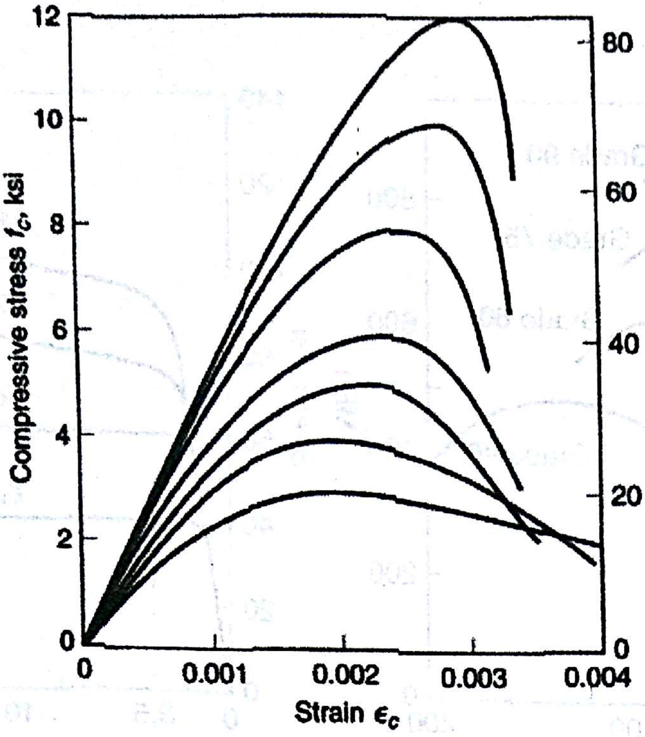
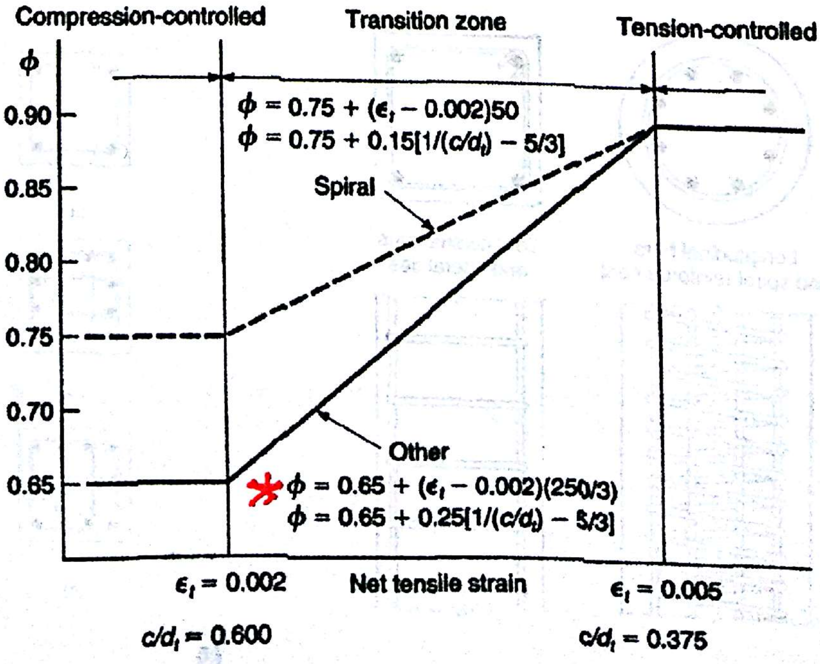


FIGURE 2.3
Typical compressive stress-strain curves for normal-density concrete with w_c = 145 pcf. *(Adapted from Refs. 2.23 and 2.24.)*

FIGURE 3.9
Variation of strength reduction factor with net tensile strain in the steel.

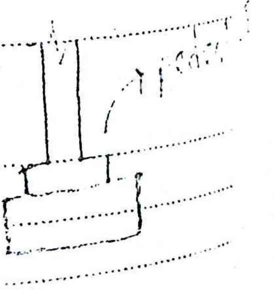


- when load eccentricities [illegible] reinforced column show greater toughness i.e. greater ductility than tied column although this difference fads out as the eccentricity is increased

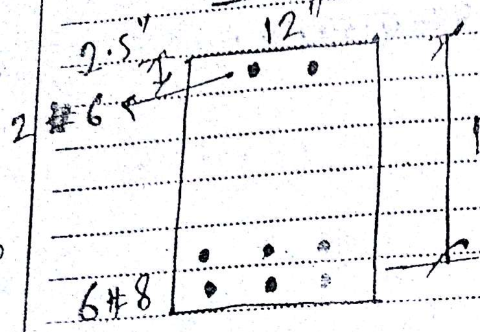
# Tied Column □	Spiral Column ○
① It takes small amount of load compared to the Spiral Column	① It takes generally more loads compared to the tied column
(ii) failure of tied column is abrupt & complete.	(ii) failure of spiral column is gradually & ductile manner.
(iii) $\phi = 0.\cancel{70}\ 0.65$ (ACI CODE)	(iii) $\phi = 0.\cancel{75}\ ,0.70.$ (ACI CODE)

Function of a pedestal →

- A pedestal distributes the column load over a large area of footing & thus contributing to a more economical footing design.



Moment Calculation:



$A_s = 6 \times .79 = 4.74 \text{ in}^2$,

$A_s' = 2 \times 0.44 = 0.88 \text{ in}^2$

$f_c' = 4 \text{ ksi}$, $f_y = 60 \text{ ksi}$

$$* \rho_{max} = 0.85 \beta_1 f_c'/f_y \frac{\epsilon_u}{\epsilon_u + 0.004}$$

$$= 0.85 \times 0.85 \times 4/60 \times \frac{0.003}{0.003 + 0.004}$$

$$= 0.0206$$

$$\rho = A_s/bd = \frac{4.74}{12 \times 16} = 0.0247$$

$$* \rho' = \frac{A_s'}{bd} = \frac{0.88}{12 \times 16} = 0.00458 = 0.0046$$

$$\bar{\rho}_{max} = \rho_{max} + \rho' = 0.0206 + .0046$$

$$= 0.0252 > \rho$$

So compression steel yield.

$$M_{n1} = A_s' f_y (d - d') = 0.88 \times 60 \times (16 - 2.5)$$

$$= 713 \text{ k-in}$$

$$M_{n2} = (A_s - A_s') f_y (d - a/2)$$

$$a = \frac{(A_s - A_s')f_y}{0.85 f_c' b} = \frac{(4.74 - 0.88) \times 60}{0.85 \times 4 \times 12} = 5.68''.$$

$$M_{n2} = (A_s - A_s') f_y (d - a/2)$$

$$= (4.74 - 0.88) \times 60 \times (16 - 5.68/2)$$

$$= 3054 \text{ k-in}$$

$$\therefore M_n = M_{n1} + M_{n2} = 713 + 3054 = 3767 \text{ k-in.}$$

$$\therefore M_u = \phi M_n$$

$$\epsilon_t = 0.003 \times \frac{d-c}{c} \qquad c = a/\beta_1 = \frac{5.68}{0.85} = 6.68$$

$$= 0.003 \times \frac{16 - 6.68}{6.68}$$

$$= 0.00419 < 0.005$$

$$\therefore \phi = 0.483 + 83.3 \times 0.00419$$

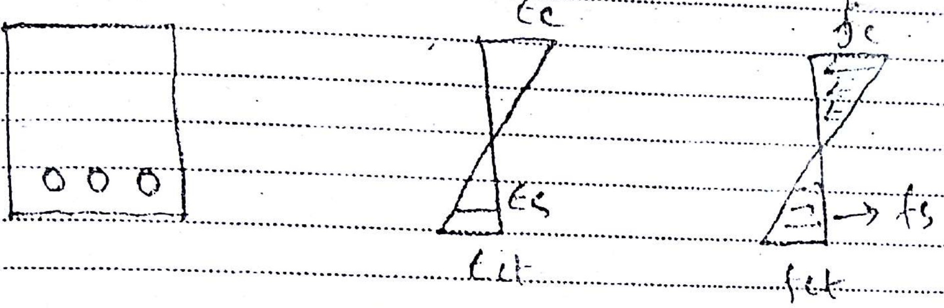
$$= 0.83$$

$$\therefore M_u = \phi M_n = 0.83 \times 3767$$

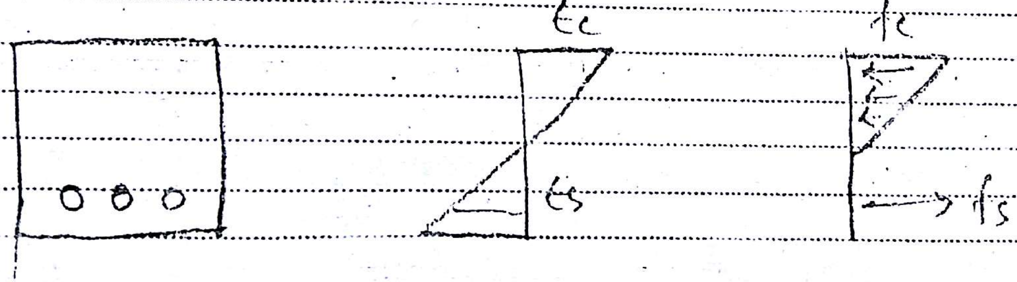
$$= 3128 \text{ k-in.}$$

#. Draw sketches for strain & stress distribution diagrams of a reinforced concrete beam section when subjected to bending for uncracked, cracked & ultimate section.

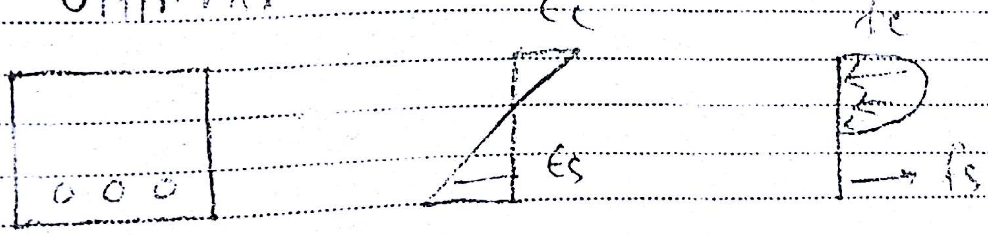
(a) Uncraked



(b) Cracked



(c) Ultimate



Requirements for Intermediate Moment Frames: **Beams**

a) The positive moment strength at the face of the joint shall not be less than one-third the negative moment strength provided at that face (Fig.8.3.12). Neither the negative nor positive moment strength at any section along the length of the member shall be less than one-fifth of the maximum moment strength provided at the face of either joint.

b) At both ends of the member, stirrups shall be provided over lengths equal to twice the member depth measured from the face of the supporting member toward midspan (Fig.8.3.13). The first stirrup shall be located not more than 50 mm from the face of the supporting member. Maximum stirrup spacing shall not exceed (a)*d/4* , (b) 8 times the diameter of the smallest longitudinal bar enclosed, (c) 24 times the diameter of the stirrup bar, and (d) 300 mm.

c) Stirrups shall be placed at not more than d/2 throughout the length of the member.

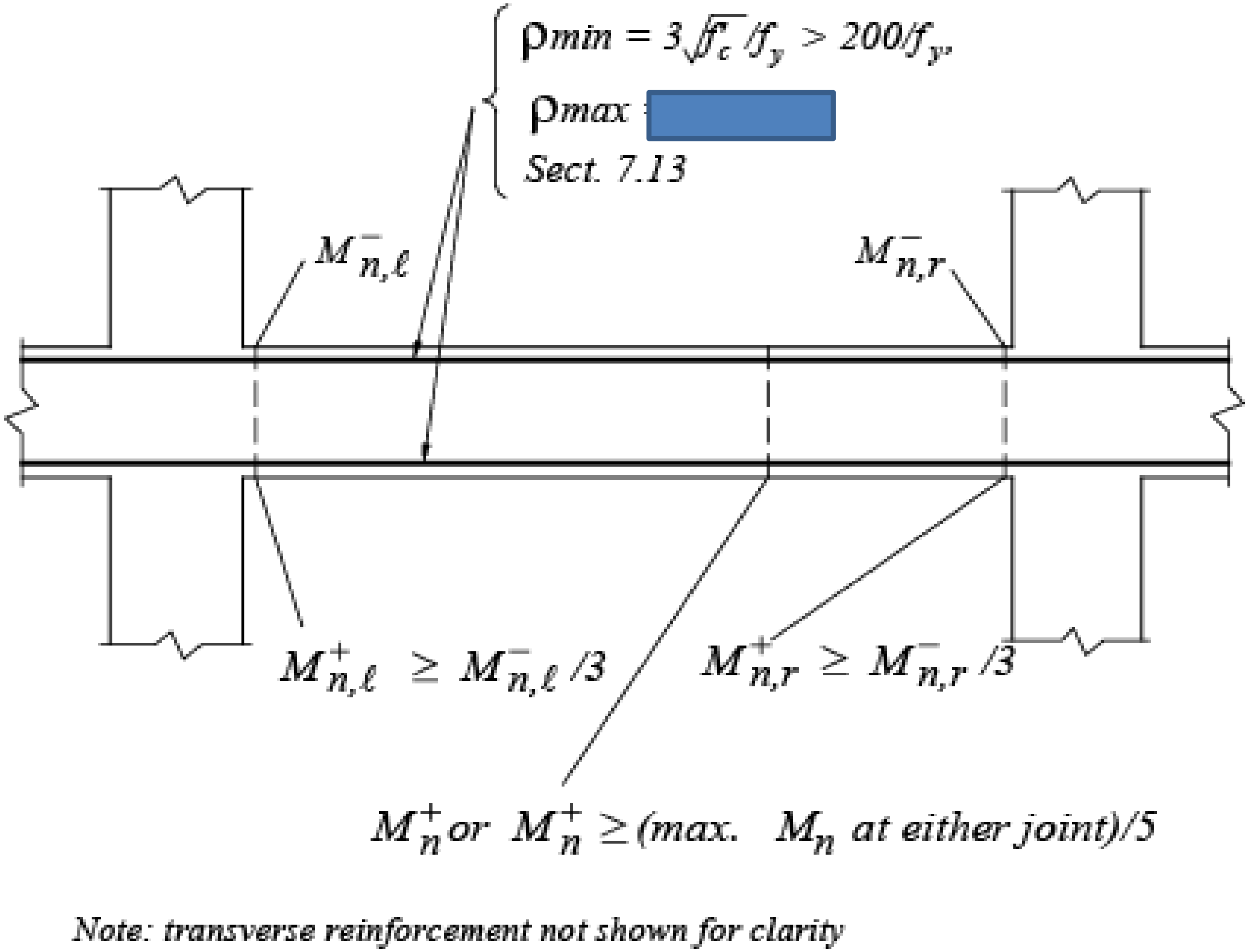


Fig.8.3.12 Flexural Requirements for Beams

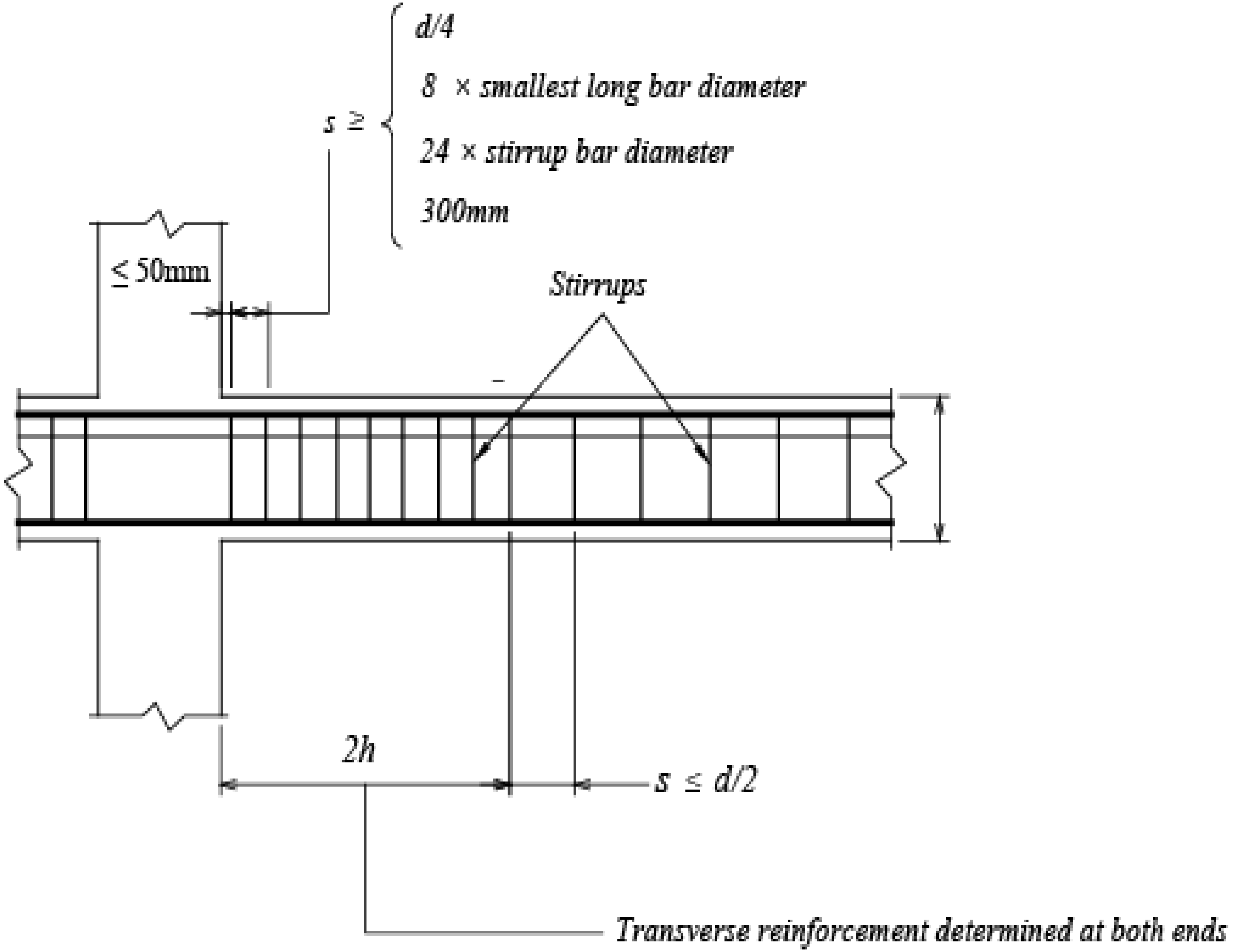


Fig.8.3.13 Transverse reinforcement Requirements for Beams

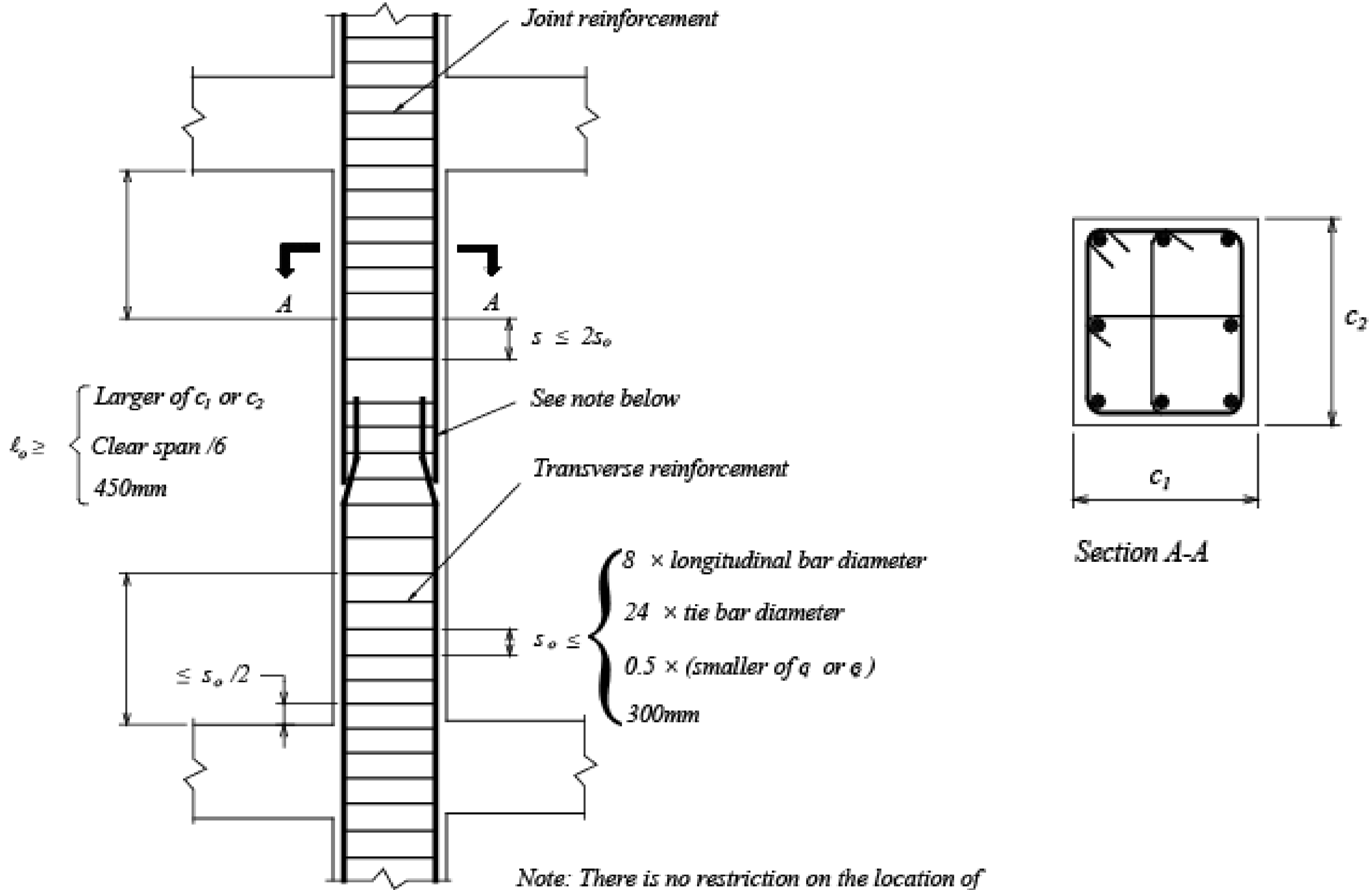
Requirements for Intermediate Moment Frames: **Columns**

a) Maximum tie spacing shall not exceed s_o over a length l_0 measured from the joint face. The spacing s_o shall not exceed (i) 8 times the diameter of the smallest longitudinal bar enclosed, (ii) 24 times the diameter of the tie bar, (iii) one-half of the smallest cross-sectional dimension of the frame member, and (iv) 300 mm. The length l_0 shall not be less than (i) one-sixth of the clear span of the member, (ii) maximum cross-sectional dimension of the member, and (iii) 450 mm.

b) The first tie shall be located not more than $s_0/2$ from the joint face.

c) Joint reinforcement shall conform to 6.4.9.

d) Tie spacing shall not exceed $2s_0$ throughout the length of the member.



Note: There is no restriction on the location of longitudinal bar splices for intermediate moment frames. The splices may be located away from the potential hinge regions as shown above

Fig.8.3.14 Transverse Reinforcement Requirements for Columns